FOUNDATION INVESTIGATION AND DESIGN REPORT

Stormwater Management Dry Pond, QEW Widening from West of Mississauga Road to west of Hurontario Street, City of Mississauga Ministry of Transportation, Ontario, G.W.P. 2002-13-00

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March 4, 2019

GEOCRES No. 30M12-436

Latitude: 43.557930 **Longitude**: -79.610152

Distribution List

1 PDF & 1 Copy - Ministry of Transportation, Ontario (Central Region)

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Table of Contents

PART A – FOUNDATION INVESTIGATION REPORT

PART B – FOUNDATION DESIGN REPORT

REFERENCES

DRAWINGS

FIGURES

APPENDICES

PART A

FOUNDATION INVESTIGATION REPORT
STORMWATER MANAGEMENT DRY POND
QEW WIDENING FROM WEST OF MISSISSAUGA ROAD TO WEST OF HURONTARIO STREET, CITY OF MISSISSAUGA
MTO, G.W.P. 2002-13-00

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by Morrison Hershfield Limited (MH) on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services associated with a number of structures / facilities, including stormwater management ponds, in support of the widening of the Queen Elizabeth Way (QEW) from west of Mississauga Road to west of Hurontario Street in the City of Mississauga, in the Regional Municipality of Peel, Ontario.

The purpose of this investigation is to establish the subsurface soil, bedrock and groundwater conditions at the location of the proposed Stormwater Management (SWM) Dry Pond, and along the alignment of the stormwater Emergency Overflow Sewer Outlet and the Outlet Connection to Culvert 6 extending to the Stavebank Creek Culvert, by borehole drilling and laboratory testing on selected soil and bedrock core samples.

The Terms of Reference (TOR) and the scope of work for the foundation investigation are outlined in MTO's Request for Proposal, dated July 2016, and the approved Change Request letters, which forms part of the Consultant's Assignment Number (Number 2015-E-0033) for this project. The work has been carried out in accordance with Golder's Supplementary Specialty Plan for foundation engineering services for this project, dated February 3, 2017.

2.0 SITE DESCRIPTION

The proposed site for the SWM Dry Pond is located in the upland plateau between the Credit River and Stavebank Road, about 70 m east of the Credit River, and approximately 45 m north of QEW (refer to the Key Plan on Drawing 1). The proposed SWM Dry Pond is oriented generally in a southwest-northeast direction as shown on Drawing 1; for the purposes of this report, the alignment is described as being in an east-west orientation.

The current ground surface within the footprint of the proposed SWM Dry Pond is covered with wood chips resulting from advanced tree clearing operations in preparation for the widening of the QEW and associated structures. North of the footprint for the proposed SWM Dry Pond the ground is vegetated by mixed deciduous and coniferous trees and it is understood that this area is currently protected as bat habitat. Overall, the existing ground surface in the area of the proposed SWM Dry Pond gradually slopes down at an inclination of about 4.5 horizontal to 1 vertical (4.5H:1V), from about Elevation 95.5 m west of Stavebank Road to about Elevation 92.0 m near the west end of the proposed pond, and then steeply downward at an inclination of about 1.4H:1V to the east bank of the Credit River at about Elevation 76.0 m. In the southern portion of the SWM Dry Pond area there is an existing gulley paralleling the QEW that drains southwesterly towards the Credit River. Along the northwestern side of the gulley the ground surface slopes downward to the north at an inclination of about 4.5H:1V, from about Elevation 92.0 m to Elevation 88.0 m, forming a depression along the north/western perimeter of the proposed SWM Dry Pond. The ground surface topography between Premium Way and the north side of the QEW from Stavebank Road easterly to the Stavebank Creek Culvert is up to about 2 m higher than Premium Way; however, along the south side of Premium Way east of Stavebank Road the ground slopes down towards the culvert to a height of about 2 m to 3 m lower than Premium Way.

3.0 INVESTIGATION PROCEDURES

The field work for the foundation investigation was carried out on July 27 and 31, 2018, during which time a total of four sampled boreholes (designated as Boreholes SWME-1 to SWME-4) were advanced within the outline of the proposed SWM Dry Pond. These boreholes are supplemented with eleven boreholes (Borehole AR-1, AR-2, CRB-5, CRB-5A, CRB-6, NW3-1, NW3-2, NW3-3, PED-01, S2 and S3) drilled for other immediately adjacent structures, such as noise barrier wall, Credit River bridge, the east access road, Stavebank Creek Culvert and the North-South Active Transportation Pedestrian bridge.

The locations of the boreholes are shown in plan on Drawing 1 and in profile / cross-section on Drawings 2 and 3. The various boreholes / drillholes were advanced during the following periods:

- PED-01 – September 13 and October 9, 2018
- NW3-2 – August 23, 2017
- NW3-1 – October 16 and 17, 2017
- CRB-6 – October 18 and 20, 2017
- CRB-5 and CRB-5A – February 13 to 16, 2018
- AR-1 and AR-2 – July 30, 2018
- NW3-3 – August 9, 2018
- S2 and S3 – September 13 and October 9, 2018

The borehole investigation was carried out using a track-mounted CME 55 drill rig and CME 75 drill rig supplied and operated by Davis Drilling Ltd. of Milton; a track-mounted CME 850 drill rig and truck-mounted CME 55 drill rig supplied and operated by Aardvark Drilling Inc. of Guelph; and a CME-55 track-mounted drill rig supplied and operated by Geo-environmental Drilling Inc. of Halton Hills, Ontario. The boreholes were advanced using 159 mm, 203 mm or 210 mm outside diameter hollow-stem augers through the overburden, and HW casing and an HQ size core barrel through the bedrock in Boreholes SWME-4, AR-2, CRB-5, CRB-5A, CRB-6, NW3-1 and PED-01. Soil and bedrock core samples were obtained at 0.75 m and 1.5 m intervals of depth, using a 50 mm outer diameter split-spoon sampler driven by an automatic hammer in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586-08)[1].

The boreholes were either advanced to split-spoon refusal (i.e. on bedrock present in sample) or cored into bedrock, to depths ranging from about 3.1 m to 25.4 m below existing ground surface, including coring bedrock for lengths / depths between 3.1 m to 8.2 m in Boreholes SWME-4, AR-2, CRB-5, CRB-5A, CRB-6, NW3-1 and PED-01.

The groundwater conditions and water levels in the open boreholes were observed during and immediately following drilling operations. A standpipe piezometer was installed in Boreholes SWME-3, CRB-6 and CRB-5A (adjacent to Borehole CRB-5) to permit monitoring of the groundwater level at the borehole location. The standpipe piezometers consist of 50 mm diameter PVC pipe, with a slotted screen within a sand filter pack sealed across lower strata bedrock interface. Above the sand filter pack and piezometer screen, the annulus surrounding the piezometer pipe was backfilled to the ground surface with bentonite pellets. All remaining boreholes were backfilled to ground surface with bentonite upon completion, in accordance with Ontario Regulation 903 (as amended).

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

The field work was observed by a member of Golder's engineering and technical staff, who located the boreholes, arranged for the clearance of underground services, observed the drilling, sampling and in-situ testing operations, logged the boreholes and examined the soil and bedrock samples. The samples were identified in the field, placed in appropriate containers, labelled and transported to Golder's Mississauga geotechnical laboratory where the samples underwent further visual examination and laboratory testing. All of the 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. Unconfined compression (UC) test (including core bulk density determination) was carried out on selected specimen of the bedrock core samples by Geomechanica Inc. on behalf of Golder.

The as-drilled borehole locations and the ground surface elevations were obtained using a GPS (Trimble Geo 7X), having an accuracy of 0.1 m in both the vertical and in the horizontal directions. The locations given on the Record of Borehole/Drillhole sheets and shown on Drawings 1 to 3 are positioned relative to MTM NAD 83 (Zone 10) northing and easting coordinates and the ground surface elevations are referenced to Geodetic datum. The borehole locations, ground surface elevations and drilled depths are summarized below.

Borehole No.	Location (MTM NAD 83)		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing (m) (Latitude, °)	Easting (m) (Longitude, °)		
SWME-1	4,824,195.5 (43.557643)	295,896.6 (-79.610206)	89.5	5.5
SWME-2	4,824,204.6 (43.557725)	295,881.3 (-79.610395)	87.4	3.1
SWME-3	4,824,227.3 (43.557930)	295,901.0 (-79.610152)	91.9	5.4
SWME-4	4,824,255.4 (43.558183)	295,917.6 (-79.609946)	95.5	13.3 (including 3.3 m of bedrock core)
AR-1	4,824,236.4 (43.558012)	295,944.3 (-79.609616)	95.7	9.2
AR-2	4,824,172.2 (43.557434	295,921.4 (-79.609899)	88.4	11.6 (including 7.0 m of bedrock core)
CRB-5	4,824,128.9 (43.557044)	295,914.2 (-79.609986)	79.2	15.5 (including 8.3 m of bedrock core)
CRB-5A*	4,824,130.9 (43.557062)	295,910.6 (-79.610032)	79.3	17.2 (including 9.5 m of bedrock core)
CRB-6	4,824,196.7 (43.557650)	295,929.5 (-79.609801)	91.7	13.3 m (including 8.2 m of bedrock core)
NW3-1	4,824,275.8 (43.558358)	295,959.8 (-79.609422)	96.5	15.4 (including 3.6 m of bedrock core)
NW3-2	4,824,342.4	295,994.3	95.3	12.3

Borehole No.	Location (MTM NAD 83)		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing (m) (Latitude, °)	Easting (m) (Longitude, °)		
	(43.558958)	(-79.608996)		
NW3-3	4,824,329.2 (43.558840)	296,002.3 (-79.608895)	90.6	8.1
PED-01	4,824,314.1 (43.558703)	295,977.3 (-79.609205)	96.3	25.4 (including 3.1 m of bedrock core)
S2	4,824,357.2 (43.559092)	296,001.4 (-79.608907)	94.9	17.4
S3	4,824,337.3 (43.558912)	296,021.0 (-79.608665)	90.0	16.6

* Note that Borehole CRB-5A is presented herein only to the extent that it applies to groundwater level monitoring as it was drilled adjacent to Borehole CRB-5 and instrumented with a standpipe piezometer.

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

The project area is located within the Iroquois Plain physiographic region, as delineated in *The Physiography of Southern Ontario* (Chapman and Putman, 1984)[2].

The glacial Iroquois Plain stretches along the northern shoreline of Lake Ontario, extending from the Niagara Escarpment in the west to the Scarborough Bluffs in the east. The Iroquois Plain soils consist of glaciolacustrine sediments deposited in Lake Iroquois, primarily sands, silts and gravels, with a shallow cover of till remaining over the bedrock.

The bedrock of the Georgian Bay Formation that underlies the study area consists mainly of blue-grey shale, containing siltstone, sandstone and limestone interbeds. Outcrops of this formation are commonly found along water courses on the west side of Toronto and in Mississauga, notably in the Humber River, Mimico Creek, Etobicoke Creek and Credit River valleys.

4.2 Subsurface Conditions

4.2.1 General

The detailed subsurface soil, bedrock and groundwater conditions as encountered in the boreholes, the piezometer installation details and water level readings, and the results of the laboratory tests carried out on selected soil and bedrock core samples are presented on the Records of Boreholes and Drillholes provided in Appendix A. Photographs of the recovered bedrock core samples are presented on Figures A-1 to A-6, in Appendix A. The results of the in-situ field tests (i.e. SPT “N” values) as presented on the borehole records and in of Section 4.2 are uncorrected. Lists of abbreviations and symbols and lithological and geotechnical work description terminology are

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[2] Chapman, L.J. and Putman, D.F., 1984, The Physiography of Southern Ontario, Ontario Geological Society, Special Volume 2, Third Edition. Accompanied by Map p. 2715, Scale 1:600,000.)

also included in Appendix A to assist in the interpretation of the borehole and drillhole records. The detail results of the geotechnical laboratory testing on soil and bedrock core samples obtained during the investigation are presented in Appendix B.

The stratigraphic boundaries shown on the borehole records and the stratigraphic profiles and cross-sections on Drawings 2, and 3 are inferred from non-continuous sampling, observations of drilling progress and the results of Standard Penetration Tests. These boundaries, therefore, represent transitions between soil types and soil/bedrock rather than exact planes of geological change. Furthermore, subsurface conditions will vary between and beyond the borehole locations; however, the factual data presented in the borehole and drillhole records govern any interpretation of the site conditions. It should be noted that the interpreted stratigraphy shown on Drawings 2 and 3 is a simplification of the subsurface conditions.

4.2.2 SWM Dry Pond

In general, the subsurface conditions in the area of the proposed SWM Dry Pond, as characterized by Boreholes SWME-1 to SWME-4, AR-1, AR-2 and NW3-1, consist of a layer of topsoil, fill comprising of sandy silt / silty sand with a thin layer of clayey silt. The topsoil or fill in places is underlain by deposits of sand, silt or clayey silt with sand to silty clay, and / or underlain by a till deposit and / or residual soil comprised of clayey silt. The native soil deposits are underlain by shale bedrock, as shown on the stratigraphic profile and cross-sections on Drawings 2. A detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.2.1 Topsoil

A layer of topsoil ranging in thickness from about 150 mm to 700 mm was encountered at the ground surface in Boreholes SWME-2, SWME-4, AR-1, AR-2 and NW3-1. SPT “N”-values measured within the topsoil layer range between 3 blows and 9 blows per 0.3 m of penetration, suggesting a soft to stiff consistency.

4.2.2.2 Fill

A 0.5 m to 1.7 m thick layer of fill consisting of sandy silt / silty sand / sand and gravel was encountered from ground surface in Boreholes SWME-1, SWME-3 and CRB-6, and underlying the topsoil in Boreholes SWME-4, AR-1 and NW3-1. The surface of the fill layer was encountered between Elevation 96.3 m and 89.5 m and extends to depths between about 0.7 m and 1.7 m below ground surface to between Elevation 95.0 m and 88.1 m. Underlying the non-cohesive fill in Borehole SWME-1 a 0.3 m thick layer of clayey silt fill was encountered at a depth of 1.4 m below ground surface (Elevation 88.1 m) and extends to a depth of about 1.7 m below ground surface (Elevation 87.8 m).

SPT “N”-values measured within the non-cohesive fill deposit range between 5 blows and 12 blows per 0.3 m of penetration, suggesting a loose to compact compactness condition.

The results of grain size distribution tests completed on three selected samples of the non-cohesive fill are shown on Figure B-1 in Appendix B.

The natural water content measured on samples of the non-cohesive fill ranges between 4 per cent and 21 per cent.

4.2.2.3 Sandy Silt to Silty Sand

Underlying the fill in Boreholes SWME-1, SWME-3, SWME-4, AR-1 and NW3-1, a non-cohesive deposit ranging in composition from sandy silt to silt and sand to silty sand to sand, trace to some clay, trace gravel, was encountered at depths between about 0.7 m and 1.7 m below ground surface (between Elevations 95.0 m and 87.8 m). The thickness of the non-cohesive ranges from about 0.5 m to 5.7 m and the deposit extends to depths of about 2.2 m and 7.2 m below ground surface (Elevation 91.8 m and 86.5 m).

SPT “N”-values measured within the sandy silt to sand deposit range from 3 blows and 78 blows per 0.3 m of penetration, suggesting a very loose to very dense compactness condition.

The results of grain size distribution tests completed on seven selected samples of the sandy silt to sand deposit are shown on Figure B-2 in Appendix B.

The natural water content measured on samples of the sandy silt to sand deposit ranges between 6 per cent and 28 per cent.

4.2.2.4 Silt

Underlying the sandy silt deposit in Borehole SWME-3, clayey silt in Borehole SWME-4 (discussed in Section 4.2.2.5) and the sand in Borehole AR-1 a deposit consisting of silt, trace to some sand and trace to some clay, was encountered at depths between about 2.2 m and 5.6 m below ground surface (between Elevation 90.4 m and 89.7 m). The thickness of the silt ranges from about 1.0 m to 2.0 m and the deposit extends to depths between about 3.2 m and 7.6 m below ground surface (between Elevation 88.7 m and 87.9 m).

SPT “N”-values measured within the silt deposit are 16 blows, 19 blows and 56 blows per 0.3 m of penetration, suggesting a compact to very dense compactness condition.

The results of the grain size distribution test completed on three selected samples of the silt deposit are shown on Figure B-3 in Appendix B.

The natural water content measured on samples of the silt deposit range from 14 per cent to 17 per cent.

4.2.2.5 Clayey Silt with Sand to Silty Clay

Underlying the topsoil in Borehole SWME-2, the fill in Borehole CRB-6, the granular deposits in Borehole SWME-1, and the silt deposit in Borehole SWME-3, AR-1, SWME-4 and NW3-1 a deposit consisting of clayey silt with sand to silty clay was encountered at depths between about 0.5 m and 7.6 m below ground surface (between Elevation 90.0 m and 86.5 m). The thickness of the cohesive deposit ranges from about 0.5 m to 1.5 m and the deposit extends to depths between about 1.5 m and 8.7 m below ground surface (between Elevation 89.9 m and 85.8 m).

SPT “N”-values measured within the cohesive deposit range between 2 blows and 17 blows per 0.3 m of penetration, suggesting a very soft to very stiff consistency.

The results of the grain size distribution test completed on six selected samples of the cohesive deposit are shown on Figure B-4 in Appendix B.

Atterberg limits tests were carried out on seven samples of the cohesive deposit and measured liquid limits ranging from 22 per cent to 42 per cent, plastic limits ranging from 14 per cent to 21 per cent, and plasticity indices ranging from 8 per cent to 21 per cent. These test results, which are plotted on a plasticity chart on Figure B-5 in Appendix B, indicate that the deposit can be classified as a clayey silt of low plasticity to silty clay of intermediate plasticity.

The natural water content measured on samples of the silty clay to clayey silt with sand deposit ranges between 11 per cent and 33 per cent.

4.2.2.6 Silty Sand to Clayey Silt (Till)

Underlying the topsoil in Borehole AR-2 and underlying the cohesive deposit in Boreholes SWME-3, SWME-4 and NW3-1 a till deposit comprised of silty sand / clayey silt to sandy silty clay was encountered at depths between about 0.6 m and 8.7 m below ground surface (between Elevation 88.2 m and 87.4 m). The thickness of the till layer varies from about 0.6 m to 3.2 m, and extends to depths between about 3.8 m and 10.1 m below ground surface (between Elevation 87.4 m and 84.6 m).

SPT “N”-values measured within the non-cohesive and cohesive till deposit are between 6 blows and 14 blows per 0.3 m of penetration, suggesting a firm to very stiff consistency cohesive till and a loose to compact compactness condition in the non-cohesive till.

Grain size distribution tests were carried out on three selected samples of the non-cohesive and cohesive till deposit are shown on Figure B-6 in Appendix B. The cohesive till deposit contains trace to some sand and trace to some gravel, and in Borehole AR-2 is interlayered by a 1.1 m thick till zone of silty sand, trace to some clay and trace to some gravel.

Atterberg limits tests were carried out on four samples of the cohesive till deposit and measured liquid limits ranging from about of 23 per cent to 38 per cent, plastic limits ranging from about 14 per cent to 20 per cent, and plasticity indices ranging from about 8 per cent to 18 per cent. These test results, which are plotted on a plasticity chart on Figure B-7 in Appendix B, indicate that the deposit is comprised of clayey silt of low plasticity to a silty clay of intermediate plasticity.

The natural water content measured on samples of the cohesive till deposit range from 12 per cent to 18 per cent. The natural water content measured on a sample of the non-cohesive till deposit is 16 per cent.

4.2.2.7 Clayey Silt (Residual Soil)

Underlying the cohesive deposit in Boreholes SWME-1, SWME-2, AR-1 and CRB-6 and the till deposit in Boreholes AR-2 and NW3-1, a 0.2 m to 1.7 m thick deposit of residual soil comprised of clayey silt some sand to sandy, some gravel to gravelly, and containing trace to some shale fragments was encountered at depths ranging between about 1.5 m and 10.1 m below ground surface (between Elevations 87.6 m and 84.6 m). The cohesive residual soil deposit is derived from weathering of the underlying shale bedrock and extends to the bedrock surface to depths of between about 2.2 m and 11.8 m below ground surface (Elevations 86.9 m and 83.8 m).

SPT “N”-values measured within the residual soil deposit are 28 blows and 64 blows per 0.3 m of penetration and three values of 50 blows per 0.13 m of penetration, suggesting a very stiff to hard consistency.

The result of a grain size distribution test completed on one selected sample of the residual soil is shown on Figure B-8 in Appendix B. Atterberg limits tests were carried out on three samples of the residual soil and measured liquid limits of about of 23 per cent, plastic limits ranging from about 15 per cent to 16 per cent, and corresponding plasticity indices ranging from about 7 per cent to 8 per cent. These test results, which are plotted on a plasticity chart on Figure B-9 in Appendix B, indicate that the deposit is comprised of clayey silt of low plasticity.

The natural water content measured on samples of the residual soil ranges between 9 per cent and 14 per cent.

4.2.2.8 Shale Bedrock

Bedrock was encountered and confirmed by resistance to augering operations and split-spoon sampling in Boreholes SWME-1 to SWME-3 and AR-1 and bedrock core samples were obtained in Boreholes SWME-4, AR-2, CRB-6 and NW3-1. The depth to bedrock below ground surface and the corresponding bedrock surface elevations are summarized below.

Borehole	Depth to Bedrock Surface / Refusal (m)	Bedrock Surface / Refusal Elevation (m)	Comments
NW3-1	11.8	84.7	Bedrock cored 3.6 m.
SWME-1	4.7	84.8	0.8 m penetration by augering and split-spoon sampling.
SWME-2	2.1	85.3	1.0 m penetration by augering and split-spoon sampling.
SWME-3	4.5	87.4	0.9 m penetration by augering and split-spoon sampling.
SWME-4	9.0	86.5	1.0 m penetration by augering and split-spoon sampling; Bedrock cored 3.3 m.
AR-1	9.0	86.7	0.2 m penetration by augering and split-spoon sampling.
AR-2	4.6	83.8	0.1 m penetration by augering and split-spoon sampling; Bedrock cored 7.0 m.
CRB-6	4.8	86.9	0.3 m penetration by augering and split-spoon sampling; Bedrock cored 8.2 m.

Based on a review of the bedrock core samples, the bedrock consists of shale of the Georgian Bay Formation. In general, the bedrock samples are described as slightly weathered, thinly bedded, fine grained, faintly to non-porous, weak, grey, with medium strong to strong limestone interbeds at varying intervals, as presented in the drillhole records in Appendix A, and shown on the photographs of the recovered core samples on Figures A-1 to A-4 in Appendix A. The degree of weathering of the bedrock samples (i.e. slightly weathered –W2), and the strength classification of the intact rock mass based on field identification (i.e. weak – R2) 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 ranges between 47 per cent and 100 per cent, with one core run from the bedrock surface of 18 per cent, indicating a rock mass of poor to excellent quality, as per Table 3.10 of CFEM (2006)[4].and very poor quality for the near surface zone The Total Core Recovery (TCR) and Solid Core Recovery (SCR) of samples recovered range between 86 and 100 per cent and between 18 per cent and 97 per cent, respectively.

Unconfined compression (UC) tests (ASTM D7012)[5] were carried out on four selected core samples of the shale bedrock and the uniaxial compressive strength (UCS), bulk density and tangent Young's modulus (in one sample) of the intact sample are summarized below and the details are presented on the Rock Laboratory Test Results report from Geomechanica in Appendix B.

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[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.

[5] ASTM D7012 – Standard Test Method for Compressive Strength and Elastic Moduli of Intact Rock Core Specimens

Borehole Number	Sample Depth Interval (m)	Sample Elevation Interval (m)	Uniaxial Compressive Strength (UCS) (MPa)	Bulk Density (g/cm^3)	Tangent Young's Modulus (GPa)
SWME-4 (Run #1)	10.4 – 10.5	85.1 to 85.0	13.5	2.59	--[1]
AR-2 (Run #2)	5.9 – 6.1	82.5 to 82.3	9.1	2.57	--[1]
AR-2 (Run #4)	8.6 – 8.8	79.8 to 79.6	11.5	2.59	--[1]
CRB-6 (Run #1)	6.1 – 6.2	85.6 – 85.5	14.6	2.17	0.63

Note:

1. Not tested

A total of six diametral and six axial point load tests were also carried out on shale bedrock core specimen and measured axial point load indices ranging from 0.4 MPa to 0.7 MPa and diametral point load indices from 0.25 MPa to 0.5 MPa (average of about 0.6 MPa and 0.3 MPa, respectively). Based on the laboratory UC test and also considering the point load indices, in accordance with Table 3.5 in CFEM (2006)[4], the shale bedrock is classified as weak (R2, 5 MPa < UCS < 25 MPa).

4.2.2.9 Groundwater Conditions

The overburden samples obtained from the boreholes were generally moist. Boreholes SWME-1, SWME-2 and AR-2 were observed to be open and dry upon completion of drilling; however, these observations are not necessarily representative of the stabilized groundwater level at the site. Upon completion of drilling Boreholes SWME-4, AR-1 and NW3-1 the water level in the open boreholes was measured at depths of 7.6 m, 3.7 m and 4.5 m, respectively, below ground surface (corresponding to Elevations 87.9 m, 92.0 m and 92.0 m), upon completion of drilling and prior to rock coring in Boreholes SWME-4 and NW3-1. A standpipe piezometer was installed in each Boreholes SWME-3 and CRB-6 (located in an area adjacent to the SWM Dry Pond), sealed within the clayey silt till / residual soil deposit /shale bedrock in Borehole SWME-3 and sealed within the shale bedrock in Borehole CRB-6, and the recorded water levels are summarized below.

Borehole Number / Foundation Unit	Stratum Well Sealed Into	Ground Surface Elevation (m)	Water Level Depth (m)	Water Elevation (m)	Date of Piezometer Reading
SWME-3: SWM Dry Pond	Clayey Silt Till / Clayey Silt Residual Soil / Shale Bedrock	91.9	4.0	87.9	August 14, 2018
			3.8	88.1	November 6, 2018
CRB-6: Credit River Bridge - East Abutment	Shale Bedrock	91.7	5.6	86.0	November 12, 2017
			5.0	86.7	March 12, 2018
			4.9	86.8	April 30, 2018
			4.9	86.8	November 6, 2018

It should be noted that the groundwater level in the area is subject to seasonal fluctuations and precipitation events, and should be expected to be higher during wet periods of the year. In addition, as discussed in Section 2.0 there is a depression within and immediately adjacent to the north-western section of the footprint of the proposed SWM Dry Pond and it is likely that surfaced water will naturally accumulate within the depression and the water level in this area may be higher than that recorded in the standpipe piezometers installed in boreholes advanced further to the southeast within the area between the east bank of the Credit River and Stavebank Road.

4.2.3 Emergency Overflow Sewer Outlet

In general, the subsurface conditions along the alignment of the proposed Stormwater Emergency Overflow Sewer Outlet, as characterized by Boreholes AR-2, CBR-5 and CBR-6, consists of topsoil in places and fill comprised of silty sand to clayey silt. The topsoil or surficial fill is underlain by deposits of sandy clayey silt or silty sand to silt and sand, and / or underlain by a till deposit. In the vicinity of the Emergency Overflow Outlet the fill is underlain by a silty sand deposit interlayered with an organic clayey silt deposit. The cohesive deposit and till deposit are underlain by residual soil comprised of clayey silt. The native soil deposits are underlain by shale bedrock as shown on the stratigraphic profile on Drawing 3. A detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.3.1 Topsoil

A 600 mm thick layer of topsoil was encountered at the ground surface in Borehole AR-2 and a SPT “N”-value of 9 blows per 0.3 m of penetration was measured within the topsoil layer suggesting a firm consistency.

4.2.3.2 Fill

In Boreholes CRB-5 and CRB-6, a 0.7 m and 1.7 m thick layer of non-cohesive fill consisting of silty sand, was encountered at the ground surface, respectively. The silty sand fill extends to Elevations 78.5 m and 90.0 m at Boreholes CRB-5 and CRB-6, respectively. The non-cohesive fill in Borehole CRB-6 contains trace to some gravel, trace organics, rootlets and brick fragments. Underlying the non-cohesive fill in Borehole CRB-5, a 1.7 m thick layer of fill comprised of clayey silty with sand containing trace organics and shale fragments was encountered at a depth of 0.7 m below ground surface (Elevation 78.5 m) and extends to a depth about 2.4 m below ground surface (Elevation 76.8 m).

SPT “N”-values measured within the non-cohesive fill deposit range between 5 blows and 11 blows per 0.3 m of penetration, suggesting a loose to compact compactness condition. SPT “N”-values measured within the cohesive fill are as 23 blows and 25 blows per 0.3 m of penetration, suggesting a very stiff consistency.

The results of a grain size distribution test completed on one sample of the silty sand fill from Borehole CRB-6 is shown on Figure B-10A in Appendix B.

The natural water content measured on two samples of the non-cohesive fill is 8 per cent and 22 per cent. The natural water content measured on two samples of the cohesive fill is at 9 per cent and 10 per cent.

4.2.3.3 Silty Sand to Sand and Silt

Underlying the fill in Borehole CRB-5, a 4.8 m thick non-cohesive deposit comprised of silty sand to silt and sand was encountered at a depth of 2.4 m below ground surface (Elevation 76.8 m) and the deposit extends to a depth of 7.2 m below ground surface (Elevation 72.0 m). Within the silty sand deposit, a 1 m thick deposit of organic clayey silt was encountered (described in Section 4.2.3.4).

SPT “N”-values measured within the silty sand to silt and sand deposit range from 0 blows (weight of hammer) to 6 blows per 0.3 m of penetration, suggesting a very loose to loose compactness condition.

The results of grain size distribution tests completed on two selected samples of the silty sand to silt and sand deposit are shown on Figure B-12 in Appendix B. The non-cohesive deposit contains trace to some gravel, trace clay and trace organics / wood fragments and rootlets.

The natural water content measured on samples of the sandy silt to silt and sand deposit ranges between 11 per cent and 27 per cent. Organic content testing completed on one sample from this deposit was measured at 1.1 per cent.

4.2.3.4 Sandy Clayey Silt / Organic Clayey Silt with Sand

Underlying the fill in Borehole CRB-6, a 2.7 m thick cohesive deposit comprised of sandy clayey silt, trace to some gravel, was encountered at a depth of 1.7 m below ground surface (Elevation 90.0 m). The deposit extends to a depth of 4.4 m below ground surface (Elevation 87.3 m). In Borehole CRB-5 advanced near the outlet of the Emergency Overflow pipe a 1.0 m thick interlayer of organic clayey silt with sand, trace gravel was encountered within the silty sand to silt and sand deposit, at a depth of 4.7 m below ground surface (Elevation 74.5 m).

SPT “N”-values measured within the sandy clayey silt deposit range from 5 blows to 9 blows per 0.3 m of penetration, suggesting a soft to stiff consistency. SPT “N”-values measured within the organic clayey silt with sand interlayer are 1 blow and 5 blows per 0.3 m of penetration, indicating a soft to firm consistency.

The results of grain size distribution testing completed on one select sample of the sandy clayey silt deposit and one sample of the organic clayey silt with sand layer are shown on Figure B-13 in Appendix B. An Atterberg limits test was carried out on one sample of the cohesive deposit and measured a liquid limit of about 23 percent, a plastic limit of about 14 per cent, and a plasticity index of about 9 per cent. The test result is plotted on a plasticity chart on Figure B-14 in Appendix B and indicates that the deposit is comprised of clayey silt of low plasticity. An Atterberg limits test was carried out on one sample of the organic cohesive deposit (from Borehole CRB-5) and measured a liquid limit of about 38 percent, a plastic limit of about 31 per cent, and a plasticity index of about 9 per cent. The test result which plotted on a plasticity chart on Figure B-15 in Appendix B, indicates that the deposit is comprised of organic clayey silt of intermediate plasticity.

The natural water content measured on samples of the sandy clayey silt deposit ranges between 12 per cent and 32 per cent and the water content measured on a sample of the organic clayey silt is 46 per cent. Organic content testing completed on one sample from this deposit measured 7.1 per cent organics.

4.2.3.5 Silty Sand to Sandy Silty Clay (Till)

Underlying the topsoil in Borehole AR-2 an interlayered till deposit comprised of clayey silt / silt and sand / sandy silty clay was encountered at a depth of 0.6 m below ground surface (Elevation 87.8 m). The till layer is approximately 3.2 m thick and extends to a depth of 3.8 m below ground surface (Elevation 84.6 m). The cohesive portion of the till deposit contains some sand trace to some gravel, trace shale fragments, and the non-cohesive portion of the till deposit contains trace to some clay trace to some gravel and trace shale fragments.

SPT “N”-values measured within the non-cohesive and cohesive interlayers of the till deposit range between 6 blows and 14 blows per 0.3 m of penetration, suggesting a firm consistency of the cohesive till and a loose to compact compactness condition in the non-cohesive till.

Grain size distribution tests carried out on two selected samples of the non-cohesive and cohesive interlayers of the till deposit are shown on Figure B-16A in Appendix B. Atterberg limits tests were carried out on two samples of the cohesive interlayers of the till deposit and measured liquid limits of about 27 per cent and 38 per cent, plastic limits of about 16 per cent and 20 per cent, and plasticity indices of about 11 per cent and 18 per cent. These test results, which are plotted on a plasticity chart on Figure B-17A in Appendix B, indicate that the cohesive interlayers of till deposit are comprised of clayey silt of low plasticity to silty clay of intermediate plasticity.

The natural water content measured on two samples of the cohesive interlayers of the till deposit is 12 per cent and 18 per cent. The natural water content measured on a sample of the non-cohesive interlayers of the till deposit is 15 per cent.

4.2.3.6 Clayey Silt (Residual Soil)

Underlying the till deposit in Borehole AR-2 and the cohesive deposit in Borehole CRB-6, a 0.8 m and 0.4 m thick deposit of residual soil, comprised of sandy clayey silt to clayey silt, some sand, and containing trace to some shale fragments, was encountered at depths of about 3.8 m and 4.4 m below ground surface (Elevations 84.6 m and 87.3 m), respectively. The cohesive residual soil deposit is derived from weathering of the underlying shale bedrock and extends to the bedrock surface to depths of about 4.6 m and 4.8 m below ground surface (Elevations 83.8 m and 86.9 m), respectively.

SPT “N”-values measured within the residual soil deposit are 50 blows 0. 08 m of penetration and 50 blows for 0.13 m of penetration in the respective boreholes, suggesting a hard consistency.

An Atterberg limits test was carried out on one sample of the residual soil and measured a liquid limit of about 23 per cent, a plastic limit of about 15 per cent, and a corresponding plasticity index of about 8 per cent. The test result, which is plotted on a plasticity chart on Figure B-9 in Appendix B, indicates that the deposit is comprised of clayey silt of low plasticity.

The natural water content measured on two samples of the residual soil is 9 per cent and 11 per cent.

4.2.3.7 Shale Bedrock

Bedrock core samples were obtained in Boreholes AR-2, CRB-5 and CRB-6. The depth to bedrock below ground surface and the corresponding bedrock surface elevations are summarized below. Based on the boreholes advanced in the vicinity of the proposed pipes between the inlet and the emergency outfall, the bedrock surface is sloping down towards the Credit River.

Borehole	Depth to Bedrock Surface / Refusal (m)	Bedrock Surface / Refusal Elevation (m)	Comments
CRB-6	4.8	86.9	0.3 m penetration by augering and split-spoon sampling; Bedrock cored 8.2 m.
AR-2	4.6	83.8	0.1 m penetration by augering and split-spoon sampling; Bedrock cored 7.0 m.
CRB-5	7.2	72.2	0.5 m penetration by augering and split-spoon sampling; bedrock cored 8.3 m.

Based on review of the bedrock core samples, the bedrock consists of shale of the Georgian Bay Formation. In general, the bedrock samples are described as highly weathered to fresh, thinly bedded, fine grained, faintly to non-porous, very weak to weak, grey, with medium strong to strong limestone interbeds at varying intervals, as presented in the drillhole records in Appendix A, and shown on the photographs of the recovered core samples on Figures A-2, A-3 and A-5 in Appendix A. The degree of weathering of the bedrock samples (i.e. slightly weathered –W2), and the strength classification of the intact rock mass based on field identification (i.e. weak – R2) 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 generally ranges between 47 per cent and 100 per cent, with three runs from the bedrock surface of 0 per cent and 18 per cent, indicating a rock mass of poor to excellent quality, as per Table 3.10 of CFEM (2006)[4] and very poor quality for the near surface zone. The Total Core Recovery (TCR) and Solid Core Recovery (SCR) of samples recovered range between 51 and 100 per cent and between 18 per cent and 100 per cent, respectively.

Unconfined compression (UC) tests (ASTM D7012)[6] were carried out on four selected core samples of the shale bedrock and the uniaxial compressive strength (UCS), together with the measured bulk density and the interpreted tangent Young’s modulus (in two specimens) of the intact sample are summarized below.

Borehole Number	Sample Depth Interval (m)	Sample Elevation Interval (m)	Uniaxial Compressive Strength (UCS) (MPa)	Bulk Density (g/cm^3)	Tangent Young’s Modulus (GPa)
AR-2 (Run #2)	5.9 – 6.1	82.5 to 82.3	9.1	2.57	--[1]
AR-2 (Run #4)	8.6 – 8.8	79.8 to 79.6	11.5	2.59	--[1]
CRB-5 (Run #5)	13.7 – 13.9	65.5 – 65.3	15.5	2.61	0.61
CRB-6 (Run #1)	6.1 – 6.2	85.6 – 85.5	14.6	2.17	0.63

Note:

1. Not tested

A total of six axial and six diametral point load tests were also carried out on shale bedrock core specimen and measured axial point load indices ranging from 0.27 MPa to 1.45 MPa and diametral point load indices ranging from 0.07 MPa to 0.76 MPa (average of about 0.68 MPa and 0.41 MPa, respectively). Based on the laboratory UC test results and also considering the point load indices, in accordance with Table 3.5 in CFEM (2006)[4], the shale bedrock is classified as weak (R2, 5 MPa < UCS < 25 MPa).

4.2.3.8 Groundwater Conditions

The overburden samples obtained from the boreholes were generally moist. Boreholes CRB-6 and AR-2 were observed to be open and dry upon completion of drilling; however, these observations are not necessarily representative of the stabilized groundwater level at the site. Upon completion of drilling Borehole CRB-5, the water level was measured at a depth of 4.3 m below ground surface (Elevation 74.9 m), prior to rock coring. A standpipe piezometer was installed in Borehole CRB-5A drilled adjacent to Borehole CRB-5 and in Borehole CRB-6 and the recorded water levels are summarized below.

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[6] ASTM D7012 – Standard Test Method for Compressive Strength and Elastic Moduli of Intact Rock Core Specimens

Borehole Number / Foundation Unit	Stratum Well Sealed Into	Ground Surface Elevation (m)	Water Level Depth (m)	Water Elevation (m)	Date of Piezometer Reading
CRB-5A Credit River Bridge -East Pier	Silty Sand / Organic clayey silt	79.3	1.6	77.7	March 12, 2018
			4.0	75.3	April 30, 2018
			4.6	74.7	November 6, 2018
CRB-6 Credit River Bridge - East Abutment	Shale Bedrock	91.7	5.6	86.1	November 12, 2017
			5.0	86.7	March 12, 2018
			4.9	86.8	April 30, 2018
			4.9	86.8	November 6, 2018

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

4.2.4 Outlet Connection to Culvert 6 (Stavebank Creek Culvert)

In general, the subsurface conditions along the alignment of the proposed Stormwater Outlet Connection to Culvert 6 (Stavebank Creek Culvert), as characterized in Boreholes NW3-1 to NW3-3, PED-01, S2 and S3, consists of a layer of topsoil or asphalt in places, underlain by interlayers of non-cohesive fill comprised of silty sand to sand and gravel and cohesive fill comprised of gravelly clayey silt with sand to sandy clayey silt. The fill deposits are underlain by native silt and sand to silty sand deposit and / or underlain by a to silty clay deposit and / or underlain by a cohesive glacial till deposit. The till deposit is in turn underlying by deposits of silty sand or sand or clayey silt residual soil, as shown on the stratigraphic profile on Drawing 3.

A detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.4.1 Topsoil / Asphalt

A 150 mm thick layer of topsoil was encountered at ground surface in Boreholes NW3-1 and S3.

A 150 mm thick layer of asphalt was encountered at ground surface in Boreholes PED-01, NW3-2 and S2.

4.2.4.2 Fill

A 1.3 m to 7.8 m thick layer of fill consisting of gravelly clayey silt with sand / sandy silt to silty sand / gravelly silty sand to sand and gravel was encountered from ground surface in Boreholes NW3-3, and underlying the topsoil in Boreholes NW3-1 and S3, underlying the asphalt in Boreholes PED-01, NW3-2 and S2. The fill is described as containing variable amounts of organics, asphalt fragments, and wood fragments.

The depth and elevation of the top and bottom of this granular deposit and the corresponding thickness and soil type are summarized below.

Borehole No.	Top of Layer		Bottom of Layer		Thickness (m)	Fill Soil Type
	Depth (m)	Elevation (m)	Depth (m)	Elevation (m)		
NW3-1	0.2	96.3	1.5	95.0	1.3	Silty Sand
PED-01	0.2	96.1	2.7	93.6	2.5	Gravelly Silt Sand
	2.7	93.6	5.6	90.7	2.9	Silt and Sand
NW3-3	0.0	90.6	1.5	89.2	1.5	Silty Sand
NW3-2	0.2	95.1	2.6	92.7	2.4	Silty Sand
	2.6	92.7	3.7	91.6	1.1	Sandy Clayey Silt
	3.7	91.6	5.3	90.0	1.6	Sand and Gravel
	5.3	90.0	7.8	87.5	2.5	Silty Sand
S2	0.2	94.7	0.4	94.5	0.2	Sand and Gravel
	0.4	94.5	0.7	94.2	0.3	Sandy Silt
	0.7	94.2	0.9	94.0	0.2	Gravelly Sand
	0.9	94.0	3.7	91.2	2.8	Gravelly Clayey Silt with Sand
	3.7	91.2	4.5	90.4	0.8	Sand
	4.5	90.4	5.6	89.3	1.1	Silty Sand Fill
S3	0.2	89.8	2.7	87.3	2.5	Silt and Sand Fill

SPT “N”-values measured within the non-cohesive fill range between 1 blow and 34 blows per 0.3 m of penetration, suggesting a very loose to dense compactness condition. SPT “N”-values measured in the cohesive fill deposits range between 8 blows and 29 blows per 0.3 m of penetration, suggesting a firm to very stiff consistency.

The results of grain size distribution tests completed on nine selected samples of the non-cohesive fill are shown on Figures B-10A and B-10B in Appendix B. The non-cohesive fill contains varying amounts of silt, sand and gravel, trace to some clay, trace rootlets/wood organics and asphalt fragments. The results of grain size distribution testing completed on one selected sample of the cohesive fill is shown on Figure B-10B in Appendix B.

Atterberg limits tests were carried out on three samples of the cohesive fill deposit and measured liquid limits ranging from about 19 per cent to 24 per cent, plastic limits ranging from about 13 per cent to 15 per cent, and plasticity indices ranging from about 6 per cent to 11 per cent. These test results, which are plotted on a plasticity chart on Figure B-11 in Appendix B, indicate that the cohesive fill may be classified as a clayey silt of low plasticity.

Water content measured on samples of the non-cohesive fill deposit range between 3 per cent and 38 per cent. Water content measured on samples of the cohesive fill deposit range between 12 per cent and 16 per cent.

4.2.4.3 Silt and Sand to Silty Sand

Underlying the fill in Boreholes NW3-1 and S2, a non-cohesive deposit varying in composition from silt and sand to silty sand, trace to some clay and trace gravel, was encountered at depths between about 1.5 and 5.6 m below ground surface (between Elevations 95.0 m and 89.3 m), respectively. The thickness of the non-cohesive deposit in Boreholes NW3-1 and S2 is about 5.7 m and 1.5 m and the deposit extends to depths of about 7.2 m and 7.1 m below ground surface (Elevation 89.3 m and 87.8 m), respectively.

SPT "N"-values measured within the silt and sand to silty sand deposit range from 10 blows and 78 blows per 0.3 m of penetration, suggesting a loose to very dense compactness condition.

The results of grain size distribution tests completed on three selected samples of the silt and sand to silty sand deposit are shown on Figure B-12 in Appendix B.

The natural water content measured on samples of the sandy silt to sand deposit ranges between 6 per cent and 23 per cent.

4.2.4.4 Clayey Silt to Silty Clay

Underlying the fill in Borehole PED-01 and the non-cohesive silt and sand deposit in Borehole NW3-1, a deposit consisting of clayey silt / silty clay was encountered at depths of about 5.6 m and 7.2 m below ground surface (Elevation 90.7 m and 89.3 m), respectively. The thickness of the cohesive deposit in Boreholes PED-01 and NW3-1 is about 1.5 m and 1.6 m and the deposit extends to depths of about 7.2 m and 8.7 m below ground surface (Elevation 89.1 m and 87.8 m), respectively.

SPT "N"-values measured within the cohesive deposit are 4 blows and 9 blows per 0.3 m of penetration, suggesting a soft to stiff consistency.

Atterberg limits tests were carried out on two samples of the cohesive deposit and measured liquid limits of 26 per cent and 36 per cent, plastic limits of 14 per cent and 16 per cent, and plasticity indices of 12 per cent and 20 per cent. These test results, which are plotted on a plasticity chart on Figure B-14 in Appendix B, indicate that the deposit can be classified as a clayey silt of low plasticity and silty clay of intermediate plasticity.

The natural water content in samples of the cohesive deposit were measured at 23 per cent and 31 per cent

4.2.4.5 Silt and Sand / Clayey Silt with Sand to Sandy Clayey Silt (Till)

Underlying the fill at Boreholes NW3-2, NW3-3 and S3, the cohesive clay deposit at Boreholes NW3-1 and PED-01, and the silty sand deposit encountered at Borehole S2, a till deposit comprised of clayey silt with sand to sandy clayey silt, containing trace gravel to gravelly, was encountered at depths between about 1.5 m and 8.7 m below ground surface (between Elevation 89.2 m and 87.3 m). The cohesive till deposit contains layers of non-cohesive till consisting of silt and sand to gravelly sand. In Boreholes NW3-1, PED-01 and S2 the till deposit extends to depths between about 10.1 m and 14.7 m below ground surface (between Elevations 86.4 m and 75.3 m). Boreholes NW3-2, NW3-3 and S3 terminated within this deposit at depths of 12.3 m, 8.1 m and 16.6 m below ground surface (Elevation 83.0 m, 82.6 m and 73.4 m), after penetrating between 4.5 m, 6.6 m and 13.9 m respectively.

SPT "N"-values measured within the cohesive and non-cohesive till deposit range between 4 blows and 50 blows per 0.3 m of penetration and between 100 blows for 0.13 m and 100 blows for 0.08m of penetration, suggesting a soft to hard consistency. Generally the lower SPT "N"-values (4 blows to 28 blows per 0.3 m of penetration) were measured near the top of the till deposit.

Grain size distribution tests were carried out on eight selected samples of the cohesive till deposit and are shown on Figures B-16A and B-16B in Appendix B. A grain size distribution tests was carried out on two selected samples of the non-cohesive till deposit and the results are shown on Figure B-16B in Appendix B. Atterberg limits tests were carried out on eight samples of the cohesive till deposit and measured liquid limits ranging from about 18 per cent to 29 per cent, plastic limits ranging from about 13 per cent to 17 per cent, and plasticity indices ranging from about 5 per cent to 12 per cent. These test results, which are plotted on a plasticity chart on Figures B-17A and B-17B in Appendix B, indicate that the deposit is comprised of clayey silt of low plasticity.

Atterberg limits tests were also carried out on two samples of the non-cohesive interlayers of the till deposit and measured liquid limits of about 17 per cent and 18 per cent, plastic limits of about 14 per cent, and plasticity indices of about 3 per cent and 4 per cent. These test results, which are plotted on a plasticity chart on Figure B-17B in Appendix B, indicate that the non-cohesive interlayers of the till deposit is comprised of sit and sand of slight plasticity.

The natural water content measured on samples of the cohesive till deposit range from 6 per cent to 19 per cent. The natural water content measured on a sample of the non-cohesive till deposit is 12 per cent.

4.2.4.6 Sand to Sand and Gravel

Underlying the cohesive till deposit in Borehole PED-01, a 10.6 m thick non-cohesive deposit of sand to sand and gravel was encountered at a depth of about 11.7 m below ground surface (Elevation 84.6 m) and extends to the bedrock surface at a depth of about 22.3 m below ground surface (Elevation 74.0 m). The sand layer of the deposit contains trace to some clay and trace to some gravel and the sand and gravel layer of the deposit contains some silt.

SPT “N”-values measured within the sand to sand and gravel deposit range from 50 blows to 121 blows per 0.3 m of penetration with two values of 100 blows for 0.05 m penetration and one value of 100 blows for 0.13 m penetration, suggesting a very dense compactness condition.

Grain size distribution tests were carried out on three selected samples of the sand deposit and are shown on Figure B-18 in Appendix B.

The natural water content measured on samples of the sand to sand and gravel deposit range from 3 per cent to 9 per cent.

4.2.4.7 Sandy Gravelly Clayey Silt (Residual Soil)

Underlying the cohesive till deposit in Borehole NW3-1, a 1.7 m thick deposit of residual soil comprised of sandy gravelly clayey silt, containing some shale fragments was encountered at a depth of about 10.1 m below ground surface (Elevation 84.6 m). The cohesive residual soil deposit is derived from weathering of the underlying shale bedrock and extends to the bedrock surface to a depth of about 11.8 m below ground surface (Elevation 84.7 m).

SPT “N”-values measured within the residual soil deposit are 64 blows per 0.3 m of penetration and 100 blows for 0.08 m of penetration, suggesting a hard consistency.

The natural water content measured on a sample of the cohesive residual soil is 11 per cent.

4.2.4.8 Shale Bedrock

Bedrock was encountered and bedrock core samples were obtained in Boreholes NW3-1 and PED-01. The depth to bedrock below ground surface and the corresponding bedrock surface elevations are summarized below.

Borehole	Depth to Bedrock Surface / Refusal (m)	Bedrock Surface / Refusal Elevation (m)	Comments
NW3-1	11.8	84.7	Bedrock cored 3.6 m.
PED-01	22.3	74.0	Bedrock cored 3.1 m.

Based on a review of the bedrock core samples, the bedrock consists of shale of the Georgian Bay Formation. In general, the bedrock samples are described as slightly weathered to fresh, thinly bedded, fine grained, faintly to non-porous, weak, grey, with medium strong to strong limestone interbeds at varying intervals, as presented in the drillhole records in Appendix A, and shown on the photographs of the recovered core samples on Figures A-4 and A-6 in Appendix A. The degree of weathering of the bedrock samples (i.e. slightly weathered –W2), and the strength classification of the intact rock mass based on field identification (i.e. weak – R2) are described in accordance with the International Society for Rock Mechanics (ISRM[7]) standard classification system.

The Rock Quality Designation (RQD) measured on the core samples ranges between 90 per cent and 97 per cent, indicating a rock mass of excellent quality, as per Table 3.10 of CFEM (2006)[8]. The Total Core Recovery (TCR) and Solid Core Recovery (SCR) of samples recovered range between 92 and 100 per cent and between 90 per cent and 100 per cent, respectively.

A total of three diametral and three axial point load tests were carried out on shale bedrock core specimen and measured axial point load indices ranging from 0.4 MPa to 0.6 MPa and diametral point load indices from 0.2 MPa to 0.4 MPa. Based on the laboratory point load indices, in accordance with Table 3.5 in CFEM (2006)[4], the shale bedrock is classified as weak (R2, 5 MPa < UCS < 25 MPa).

4.2.4.9 Groundwater Conditions

The overburden samples obtained from the boreholes were generally moist and the silt and sand to silty sand deposits were moist to wet. Boreholes PED-01, NW3-2, NW3-3 and S2 were observed to be open and dry upon completion of drilling, or prior to rock coring and / or introduction of water during the drilling process; however, these observations are not necessarily representative of the stabilized groundwater level at the site. Upon completion of drilling, but prior to bedrock coring in Borehole NW3-1, the water level in the open borehole was measured at a depth of 4.5 m below ground surface (Elevation 92.0 m). A standpipe piezometer was installed in Borehole S3, sealed within the silt and sand fill and within the clayey silty with sand till deposit and the recorded water level is summarized below.

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[7] 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.

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

Borehole Number / Foundation Unit	Stratum Well Sealed Into	Ground Surface Elevation (m)	Water Level Depth (m)	Water Elevation (m)	Date of Piezometer Reading
S3	Silt and Sand (Fill) / Clayey Silt with Sand (Till)	90.0	0.8	89.2	November 6, 2018

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

5.0 CLOSURE

This report was prepared by Ms. Alex MacMillan and Ms. Katelyn Nero, both are Geotechnical Engineer-In-Training with Golder. Ms. Sandra McGaghran, M.Eng., P.Eng., a Geotechnical Engineer and Associate with Golder reviewed the report. Mr. Jorge Costa, P.Eng., MTO Foundations Designated Contact for Golder and Senior Consultant, conducted a quality control review of the report.

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PART B

FOUNDATION DESIGN REPORT
STORMWATER MANAGEMENT DRY POND
QEW WIDENING FROM WEST OF MISSISSAUGA ROAD TO WEST OF HURONTARIO STREET, CITY OF MISSISSAUGA
MTO, G.W.P. 2002-13-00

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides geotechnical recommendations for the design of the proposed Storm Water Management (SWM) Dry Pond, Stormwater Emergency Overflow Sewer Outlet and Stormwater Outlet to Stavebank Creek Culvert, located, northeast of the east abutment of the proposed widened QEW Credit River bridge. These recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the current subsurface investigation at the proposed SWM Dry Pond location and from boreholes advanced at adjacent structures/facilities such as the east abutment of the new westbound Credit River Bridge, East Access Road, N-S AT Pedestrian Bridge, Noise Barrier Wall and Stavebank Creek Culvert. The discussion and recommendations contained in this report are intended to provide the designers with sufficient information to complete the detail design of the proposed SWM Dry Pond and associated West from conveyance pipes. The foundation investigation report, discussion and recommendations are intended for the use of the Ministry of Transportation, Ontario (MTO) and shall not be used or relied upon for any other purpose or by any other parties, including the construction or design-build contractor. The contractor undertaking the work must make their own interpretation based on the factual data in Part A (Foundation Investigation) of the report. Where comments are made on construction, they are provided to highlight those aspects that could affect the design of the project, and for which special provisions may be required in the Contract Documents. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

6.1.1 Design Details – SWM Dry Pond

The following summarizes the proposed SWM Dry Pond design elements based on the General Arrangement drawing provided by Morrison Hershfield Limited (MH) on November 16, 2017 (with an updated design drawing provided on November 22, 2018 and updated drawing provided on January 7, 2019) and the subsurface conditions encountered in Boreholes SWME-1 to SWME-4, AR-1, AR-2, NW3-1 and CBR-6:

Design Pond Bottom Elevation (m)	Design Top of Pond Berm Elevation (m)	Approximate Maximum Excavation / Cut Depth (m)	Approximate Excavation / Cut Depth into Shale Bedrock (m)	Approximate Maximum Perimeter Berm Fill Height (m)
91.8	93.8	5.2	0.0	4.2

The proposed SWM Dry Pond is oriented generally in a southwest-northeast direction as shown on Drawing 1; for the purposes of this report, the alignment is described as being in an east-west orientation parallel to the QEW highway, which in the area is considered oriented east-west. The SWM Dry Pond will be situated at / near the top of the existing east valley slope of the Credit River and will be constructed by a combination of cut and fill operations. Within the proposed eastern portion of the pond the existing grade will be required to be cut, while fill will be required to be placed on the west half of the perimeter containment berm and on the pond base area. Based on the GA, up to about 2.2 m of fill will be required to be placed in the area of the pond base to raise the existing ground to the proposed design grade. A 3 m to 5 m wide maintenance access road will be provided around the perimeter of the pond.

A stormwater management pond designed on the premise that it will perform as a “dry pond” may be constructed without a low-permeability liner where conditions are considered suitable / favorable to allow for surface water infiltration, or it may include a low-permeability liner where provision is made for positive discharge of inflowing stormwater. The subsurface conditions at the proposed location of the SWM Dry Pond at this site consist of existing non-cohesive fill, native silt to silty sand, clayey silt to silty clay, till and residual soils, and the groundwater table is present at about Elevation 88.1 m (as measured in the piezometer in Borehole SWME-3) or about 3.7 m below the proposed bottom of the pond. Assuming that the groundwater regime is not a source of potable water in the area, then not including a low-permeability liner in the pond design allows for the subsurface regime to provide for filtration of sediments from stormwater runoff flowing into the pond, for attenuation of stormwater infiltration rates, and for some attenuation of containments carried by stormwater. While an assessment of surface water / groundwater interaction related to contaminant impacts and groundwater mounding / recharge rates is beyond / outside the geotechnical scope of work for design of this pond, infiltration of storm water through the base of the pond would recharge the groundwater regime and could result in mounding of the groundwater level in the area, potentially negatively impacting the overall / global stability of the adjacent Credit River valley east slope. Therefore, a low-permeability natural clay liner or a geosynthetic clay liner should be included in the design / construction of the SWM Dry Pond to reduce the potential for seepage from the pond mounding the groundwater level and leading to slope instability.

A positive outlet should be provided in the design to allow for discharge of stormwater to the local stormwater management / conveyance system, such as a storm sewer or local surface water receptor. The level at which the positive outlet is provided relative to the pond bottom level and its discharge capacity, will control the level of ponding of inflow water; therefore, the liner should extend from the pond bottom up the side slopes to at least the estimated high water level, which is based on the design storm inflow, the outlet discharge capacity and the required retention period. A natural clay liner is preferred over a geosynthetic clay liner (GCL) because it is less susceptible to damage from plant root penetration and excavation operations for sediment removal, whereas a GCL would require a chemical coating to minimize plant root penetration and, given its very thin composition, is more prone to damage during de-sedimentation maintenance operations. Recommendations for a compacted natural clay liner are provided in Section 6.3 of this report

The stormwater Emergency Overflow Sewer Outlet will extend from MH 770 near the Dry Pond inlet to the Emergency Overflow Outlet structure located on the east bank of the Credit River, adjacent to the new westbound Credit River Bridge east abutment. The proposed stormwater Emergency Overflow Sewer Outlet will consist of an 825 mm diameter sewer pipe extending from MH 770 to an intermediary Maintenance hole MH 772 at about invert Elevation 84.0m, (0.9% grade), and extending from MH 772 to the Emergency Outflow at about invert Elevation 79.0 m (at a 1% grade).

The stormwater Outlet Connection to Culvert 6 (Stavebank Creek Culvert) will consist of a 375 mm diameter sewer pipe extending from the outlet control MH 774 at the east end of the Dry Pond to Culvert 6 at about invert Elevations 91 m to 89 m (on a 1.5% grade).

6.2 Pond Base Stability – Construction and Operation Maintenance Periods

As discussed in Section 4.2.2.9, the groundwater level measured the standpipe piezometer installed in Borehole SWME-3 is at about Elevation 88.1 m and is therefore about 3.7 m below the proposed bottom of the SWM dry pond, such that the SWM pond will be operating as a “dry pond”, active only for the management of stormwater.

SWM Pond	Design Pond Base Elevation (m)	Groundwater Elevation (m)	Groundwater Level Relative to Pond Base (m)	Potential for Base Instability
SMW Dry Pond	91.8	88.1	-3.7	No

As the groundwater level is below the design bottom of the pond, the net hydraulic gradient is downwards through the base of the pond. Therefore, there is no expected risk of base instability due to base heave. It is however noted that there may be localized zones of softening / loosening of the pond bottom and sloughing of the lower portion of the pond banks during construction, and shortly after periods of heavy precipitation which may temporarily raise the groundwater level.

Further, it is anticipated that only limited seepage into the excavation will occur during construction. In the eastern portion of the proposed SWM Dry Pond that will be constructed in cut, localized seepage should be anticipated during construction where "perched" groundwater may be encountered in the cohesionless fill deposits overlying any native cohesive deposits / layers.

6.3 Permanent Pool Design and Pond Liner Considerations

The pond will be dry during normal operating conditions, as the design pond bottom will be several meters above the groundwater level. During precipitation events, the pond will receive stormwater runoff inflow and a negative, or downward seepage gradient, will develop such that any seepage (infiltration) from the pond will result in recharge to the groundwater table. Given the relatively highly permeable nature of the foundation materials (comprised of cohesionless fills and native sands), high exfiltration rates from the pond will likely occur.

The proposed SWM Dry Pond is located at / near the crest of the Credit River valley, in proximity to the Credit River. An elevated groundwater table caused by infiltration from the Dry Pond could cause negative impacts on the global slope stability of the adjacent river valley wall due to an increase in porewater pressures. Additionally, seepage from the Dry Pond may emanate from / exit on the nearby valley wall / slopes potentially resulting in increased erosion of the river bank.

To minimize seepage losses from the pond and recharge of the local groundwater regime, it is recommended that a liner, comprised of compacted natural clay or a manufactured composite geosynthetic / bentonite product (normally called a geosynthetic clay liner – GCL) be constructed on the pond bottom and on the inside side slope.

6.3.1 Compacted Clay / Geosynthetic Clay Liner

A compacted natural clay liner, or a geosynthetic clay liner (GCL) is recommended on the base and side slopes of the pond. The clay liner should extend up the side slopes of the pond to at least the maximum design storm event water level.

The pond side slopes should be formed no steeper than 3 horizontal to 1 vertical (3H:1V) to allow construction equipment to place and compact of the natural clay material or place the GCL and top layer of protective soil. It should be noted that for safety considerations, some municipalities stipulate that the perimeter slopes be benched at 7H:1V over the length of the slope from 1 m below to 1 m above the operating pond level.

The natural clay soil for the pond liner should have a minimum clay content of 15 per cent, and a plasticity index greater than 10 per cent. The natural clay liner should be constructed to a thickness of at 450 mm, placed in three equal thickness loose lifts and each lift compacted to at least 95 per cent of the material’s Standard Proctor maximum dry density. If a geosynthetic clay liner (GCL) is used in lieu of a natural compacted clay liner, it should be placed / overlapped / secured as stipulated by the proprietary manufacturer / supplier, including carrying out any construction quality control / assurance inspection and testing operations. A Non-Standard Special Provision to address the supply and placement of the compacted natural clay liner is provided in Appendix C, for inclusion in the Contract Documents.

The liner, whether comprised of compacted natural clay or a GCL does not require an overlying ballast fill layer as no hydrostatic uplift forces are expected; however, the liner requires a minimum 300 mm thick cover of granular soil, such as OPSS.PROV 1010 (*Aggregates*) Granular B Type I or selected subgrade material (SSM), for protection during maintenance operations. In addition, a Class I non-woven geotextile as specified in OPSS.PROV 1860, having a filtration opening size (FOS) of 600 µm (microns), should be incorporated between the liner and granular cover soil to act as a warning element of the presence of underlying clay liner if maintenance operations require excavating to the bottom of the pond.

6.4 Subgrade Preparation, Pond Base and Berm Construction

Based on the General Arrangement drawing provided by MH, the topography of the site is generally lower than the required top of pond and therefore the perimeter slopes of the SWM Dry Pond will be primarily constructed in fill (i.e. a berm), with only the southeastern portion of the pond constructed in cut. The pond base will require up to 2.2 m of fill placement to meet the design grade (Elevation 91.8 m). Perimeter containment berms will require up to approximately 4.2 m of total fill above the existing ground level, or 2 m of additional fill above the pond base level.

The existing fill materials and native soil strata encountered in the boreholes advanced in the area of the SWM Dry pond are considered to be an appropriate subgrade for construction of the pond base and perimeter berms. Prior to construction of the pond base and perimeter berms, it is recommended that all topsoil, organics, and softened fill be removed from the pond footprint; and the exposed subgrade be proof-rolled with a smooth drum roller to densify loosened zones and identify any softened zones that may require replacement.

Fill for the pond base and perimeter berms should consist of suitable earth fill borrow material from on-site deep cut excavation areas within the project boundaries or imported from off-site meeting the requirements of OPSS.PROV 212 (*Earth Borrow*). Should steeper than 2.25H:1V overall slope inclinations be required for construction of the perimeter berms, the fill must be granular consisting of Select Subgrade Material, Granular ‘A’ or Granular ‘B’ Type I meeting the specifications of OPSS.PROV 1010 (*Aggregates*) as discussed further in Section 6.5. The fills should be placed and compacted in accordance with OPSS.PROV 206 (*Grading*) and OPSS.PROV 501 (*Compacting*).

All fills should be placed in lifts with loose thickness to satisfy OPSS.PROV 206 (*Grading*) for compaction and compacted to at least 95 per cent of the Standard Proctor Maximum Dry Density of the material. Inspection and field density testing should be carried out by qualified personnel during fill placement operations to ensure that appropriate materials are used and that adequate levels of compaction have been achieved.

6.5 Global Stability of Pond Slopes

We understand that the SWM Dry Pond interior perimeter slopes are proposed to be constructed primarily at an inclination of 4H:1V. At the eastern section of the pond, the cut slope above the pond crest (i.e. above Elevation 93.8 m) is proposed to be constructed at a steeper inclination, varying from 2H:1V to 4H:1V. The exterior perimeter fill slopes at the western section to the south side of the pond are also proposed to be constructed at slopes varying from 2H:1V to 4H:1V. Two-dimensional limit equilibrium slope stability analyses were performed using the commercially available program Slide (Version 6.0), developed by Rocscience Inc., employing the Morgenstern-Price method of analysis. Morgenstern-Price is a general method of slices which is based on equilibrium of forces and moments acting on each slice of soil mass above the potential failure surface. The Factor of Safety is defined as the ratio of the forces tending to resist failure to the driving forces tending to cause failure. For the purpose of the stability analysis, the Factor of Safety is equal to the inverse of the product of the consequence factor, Ψ, and the geotechnical resistance factor, ϕ_{gu}. (i.e., $FoS = 1/(\Psi \cdot \phi_{gu})$). Accordingly, minimum Factors of Safety of 1.33 and 1.54 have been used for the design of the interior and exterior perimeter slopes for the short-term/temporary and long-term/permanent conditions, respectively, as per Table 6.2 of *CHBDC* (2014). This minimum factor of safety is considered appropriate for the proposed SWM Dry Pond side slopes on this project, considering the design requirements and the available field and laboratory testing data.

The following parameters have been used in the static global stability analyses, based on field and laboratory test data as well as accepted correlations (CFEM, 2006; Bowles, 1984; and Kulhawy and Mayne, 1990):

Soil Deposit	Bulk Unit Weight (kN/m^3)	Effective Friction Angle	Undrained Shear Strength (kPa)
Existing Non-cohesive Fill	19	32°	-
Pond Bottom/Berm Fill (Earth Fill)	20	32°	-
Pond Bottom/Berm Fill (Granular Fill)	22	36°	-
Very Soft to Stiff Clayey Silt	20	30°	35
Very Loose to Very Dense Silt and Sand/ Sand	19	32°	-
Compact to Very Dense Silt	20	30°	-
Firm to Stiff Silty Clay	18	28°	50
Firm to Very Stiff Clayey Silt Till	21	34°	200

For the normal operating conditions case (i.e. when the Pond is dry), the piezometric level used in the stability analyses is based on a design groundwater level that has been assumed to be at the “stabilized” groundwater conditions (i.e. at about Elevation 88.1 m).

Stability analyses for the maximum operating water level case (i.e. during storm conditions) considered the pond water level when the pond is full of run-off water (assumed to be at the emergency spillway invert Elevation 93.5 m), and the piezometric groundwater level is assumed to be the same as the normal operating conditions under the assumption that a low-permeable pond liner is an element of the pond bottom/slopes which will prevent any steady-state seepage regime from developing through the pond bottom/side slopes.

The results of the static global stability analyses indicate that a Factor of Safety greater than 1.54 is achieved for the SWM Dry Pond permanent interior cut slopes and Earth Fill berm slopes with pond crest Elevation 93.8 m inclined at 4H:1V and cut slopes at the east end of the pond inclined at 2H:1V, both under normal operating condition and during the storm water storage level condition.

For the SWM Dry Pond's permanent exterior berm slopes, the static global stability analyses indicate that berms constructed of Earth Fill inclined at 2H:1V will have a Factor of Safety less than 1.54 (long-term), and the side slopes constructed of Earth Fill are required to be inclined at a minimum of 2.25H:1V to achieve a minimum Factor of Safety of 1.54 (long-term). The static global stability analyses indicate that berm fill slopes inclined at 2H:1V would have to be constructed of granular fill meeting the requirements of OPSS.PROV 1010 (*Aggregates*) Select Subgrade Material, Granular 'A' or 'B' Type I fill to achieve a minimum Factor of Safety of 1.54 (long-term), both under normal operating conditions and during storm conditions.

The results of the global static stability analyses are shown on Figures 1 to 4 for cross-sections at selected critical locations.

A maximum (steepest) interior pond cut slope inclination of 4H:1V below the pond crest Elevation 93.8 m is also recommended to promote surficial stability of the cut slopes under changes in the operating water level and to reduce the potential for surface erosion of the cut slopes. Recommendations for protection and enhancement of the surficial stability of the pond side slopes are provided in Section 6.7.

6.6 Settlement of Pond Base and Berm Fill

Settlement of the subgrade soils beneath the fill portions (berms) of the SWM Dry Pond can be expected as a result of up to 4.2 m of new fills placed on the existing fill material and underlying native deposits of clayey silt, silty clay, sand, clayey silt to silty clay till, silty sand till and residual soil. Settlement of new fill (either earth or granular) that is properly placed and compacted for construction of the pond perimeter berms would occur during construction. It is assumed that all surficial topsoil, organic matter and any other unsuitable materials near surface are removed prior to fill placement.

To estimate the magnitude of the expected settlements of the foundation materials below the pond bottom and berms, settlement analyses were carried out for three critical sections using both hand calculations and the commercially available software *Settle-3D* from Rocscience. The critical sections correspond to the greatest fill height on the foundation soils encountered in Boreholes SWME-1, SWME-2 and AR-2.

Settlement analyses were carried out using the estimated elastic deformation moduli and consolidation parameters as given below, based on correlations with the SPT "N" values, laboratory test results and correlations published in literature (Bowles, 1984; Kulhawy and Mayne, 1990; Peck et al., 1974), and engineering judgement from experience with similar soils in this region of Ontario. The existing cohesive fill, native clayey silt and silty clay deposits encountered in Boreholes SWME-1 and SWME-2 were assumed to be slightly over-consolidated for the analyses. The coefficient of consolidation, c_v (cm^2/s), required in the time-rate analysis was established using correlation with plasticity indices. A bulk unit weight of 21 kN/m^3 was employed for the proposed pond bottom and berm grading fill in calculating the loading of the new fill on the subgrade subsoils. The thickness of the compressible soils and the thickness/height of the new fill will vary along the proposed perimeter berm alignment and pond base footprint, and as such the foundation settlements will similarly vary. Given that the analyses were carried out at critical fill sections (i.e. at the thickest fill and/or softer subgrade soils locations) of the pond, the settlement estimated will generally represent the maximum values.

Foundation Soil	Thickness (m)			Bulk Unit Weight (kN/m^3)	Estimated Deformation Properties
	Borehole SWME-1	Borehole SWME-2	Borehole AR-2		
Very Loose to Loose Existing Non-Cohesive fill	1.4	-	-	20	E'=10 MPa
Firm Existing Cohesive Fill	0.3	-	-	19	e_0=0.52, C_c=0.2
Very Loose to Loose Sand	1.3			20	E'=10 MPa
Firm Clayey Silt	-	1.0	-	19	e_0=0.52, C_c=0.2
Firm Silty Clay	0.7	-	-	18	e_0=0.61, C_c=0.3
Loose to Compact Silty Sand Till	-	-	1.1	21	E'=50 – 75 MPa
Stiff Clayey Silt to Silty Clay Till	-	-	2.1	21	E'=50 – 75 MPa

Based on the results of the settlement analyses, the maximum total settlements are comprised of immediate settlement (i.e. settlement during or shortly after construction) due mainly to compression of the very loose to loose non-cohesive existing fills, loose native sand, stiff clayey silt to silty clay till and loose to compact silty sand till and primary consolidation settlement (i.e. time-dependent settlement) of the existing cohesive fill and native clayey silt and silty clay deposits. The maximum settlements of the subgrade under the berms and pond bottom are estimated to range from 75 mm to 150 mm, depending on the fill height/thickness and foundation conditions.

The initial (immediate) compression settlement of the existing non-cohesive fill and native sand under the loadings from the new pond fill and berm fill is expected to range from 5 mm to 25 mm and is expected to occur during or shortly after construction in response to the placement of the new fill.

The time dependent settlement of the compressible foundation soils (clayey silt and silty clay) under the fill loading is estimated to be between 50 mm and 145 mm. Based on an estimated co-efficient of consolidation (c_v) of 1×10^{-3} cm^2/s for the soft to firm cohesive soils (and the imposed loading conditions, and assuming two-way drainage of the cohesive deposit), the majority (up to 90%) of the primary consolidation settlement within the cohesive subsoils below the new fill would occur within the first 30 days after completion of construction of the pond fill. In this regard, it is recommended that the final grading of the pond be carried out 30 days after completion of bulk fill placement to allow for any additional fill placement that may be required to accommodate for any post construction settlement. If the construction schedule permits, the construction of the clay liner should also be delayed until 30 days after bulk fill placement to mitigate the impacts of differential settlements of the clay liner.

6.7 Surficial Stability and Erosion Protection

The requirements for design of erosion protection measures at the water inlet and outlet works should be assessed by the hydraulic design engineer. As a minimum, rip-rap treatment at the inlet and outlet to/from the SWM Dry Pond pipes should be consistent with the standard presented in OPSD 810.010 (*General Rip-Rap Layout for Sewer and Culvert Outlets*) Rip-Rap Treatment Type A, with the rip-rap placed to above the pipe obvert, in combination with cut-off headwalls, if these are adopted.

The pond berm/cut slopes should be vegetated as soon as practical after construction to minimize the potential for erosion due to surface water run-off, either by placement of topsoil as per OPSS 802 (*Topsoil*) and seeding as per OPSS.PROV 804 (*Seed and Cover*) or pegged sod in accordance with OPSS.PROV 803 (*Sodding*). Alternatively, consideration could also be given to protecting the slopes with a minimum 150 mm thick layer of OPSS.PROV 1004 (*Aggregates*) R-10 rip-rap, constructed in accordance with OPSS.PROV 511 (*Rip-Rap, Rock Protection and Granular Sheeting*).

In addition, a granular drainage blanket may be required to control surficial sloughing of cut slopes through saturated existing non-cohesive fill and native non-cohesive soil zones or layers if perched water tables are encountered. Determination of the frequency, extent and exact locations of such seepage zones from the limited borehole data is not possible. Therefore, an observational approach is required involving examination of the cut slopes during and following construction to identify any areas of water-bearing cohesionless soils; where lenses or layers of water-bearing cohesionless soils are observed, a granular drainage blanket comprised of OPSS.PROV 1004 (*Aggregates)* granular sheeting material minimum 0.3 m thick should be placed to minimize surficial sloughing and/or erosion.

6.8 Hydraulic Conductivity

The hydraulic conductivity of the non-cohesive soils anticipated to be present along the base and cut slopes of the proposed SWM Dry Pond as encountered in Boreholes SWME-1 to SWME-4 and AR-1 has been estimated based on the grain size distribution test results on selected samples from these boreholes using the following empirical correlation developed by Hazen as referenced in Freeze and Cherry (1979):

$$K=A(d_{10})^2$$

Where: K = Hydraulic conductivity (cm/s)

A = constant equal to 1

d_{10} = grain size for which 10 per cent of the particles are finer (mm)

The hydraulic conductivities of the non-cohesive fills are estimated to range between about $2.5\text{x}10^{-3}$ cm/s and $2\text{x}10^{-4}$ cm/s; the native silt between about $1\text{x}10^{-5}$ cm/s to $2\text{x}10^{-5}$; the native sandy silt to sand about $2.5\text{x}10^{-3}$ cm/s to $1.6\text{x}10^{-5}$ cm/s; and native non-cohesive tills about $5\text{x}10^{-5}$ cm/s or less (Freeze and Cherry, 1979).

The hydraulic conductivity of the fine-grained soils encountered in the above noted boreholes including the cohesive fills, silty clay to clayey silt, silty clay to clayey silt till and residual soil was estimated based on published typical ranges to be between $1\text{x}10^{-7}$ cm/s and $1\text{x}10^{-9}$ cm/s.

6.9 Construction Considerations

6.9.1 Excavation for Pond Construction

The proposed SWM Dry Pond will require an excavation at its east end to depths of up to about 5.2 m below the current ground surface. Permanent and temporary excavations for the pond will be made through topsoil, loose to dense non-cohesive fill and loose to very dense silt and sand to sand. Excavation into the underlying native cohesive soils, till or residual soil is not anticipated based on the stratigraphy encountered in Boreholes SWME-3, SWME-4 and AR-1 advanced in the east portion of the pond footprint. The existing fill and loose native deposits are considered to be Type 3 soils according to Ontario Regulation 213 (Ontario Occupational Health and Safety Act (OHSA) for Construction Projects), as amended.

Temporary excavations are required within or adjacent to the proposed SWM Dry Pond for drainage structures (e.g. for Emergency Overflow Outfall structure, intermediate Maintenance hole (MH 772) and associated sewer pipe outlet structures (MH770 and MH774) and the Outflow Connection to Culvert 6. A summary of the proposed Dry Pond drainage structures and temporary excavations details, based on the plan and section drawings provided by MH, are provided below.

Dry Pond Drainage Structure Element	Reference Borehole(s)	Approximate Design Base Elevation (m)	Anticipated Temporary Excavation Depth* (m)	Anticipated Temporary Excavation Depth into Shale Bedrock* (m)	Anticipated Temporary Excavation Depth below Groundwater* (m)
Emergency Overflow Inlet Structure (MH 770)	CRB-6 and SWME-1	89.2 (MH 768) 83.9 (MH 770)	1.0 to 6.3	3.0	0.0 to 2.9
MH 772	AR-2	79.0	8.0	4.8	-
Emergency Overflow Outfall Structure	CRB-5	77.4	1.0	-	-
Outlet Control Structure (MH 774)	AR-1	90.5	6.0	-	1.5
Outlet Connection to Culvert C6	NW3-2 and S2	88.3	3.0	-	3.7

Note:
* Depths/Elevation are estimated relative to the existing ground surface level at the structure location which is different from that at the reference borehole location.

Temporary excavations for the SWM Dry Pond drainage outflow structures will be made through topsoil, very loose to very dense non-cohesive fill, stiff to very stiff cohesive fill, firm to stiff clayey silt to silty clay, very loose to very dense silt to silty sand, stiff clayey silt to silty clay till, very stiff to hard clayey silt residual soil deposits based on the stratigraphy encountered in the reference Boreholes listed above. According to the Ontario Regulation 213 (Ontario Occupational Health and Safety Act for Construction Projects) (OHSACP) (as amended) the very loose and soft to very soft native deposits and soils below the water table are considered to be Type 4 soils; the existing fill and loose to compact and firm to stiff native deposits are considered to be Type 3 soils; the very stiff and dense native cohesive soils are considered to be Type 2 soils; and the hard residual soils are considered to be Type 1 soils. Accordingly, temporary open-cut (unsupported) excavations through/into: Type 1 and Type 2 soils can be made with a 1.2 m vertical face from the base of the excavation and then sloped at 1H:1V; Type 3 soils can be made with walls slope from its bottom with a slope having a minimum gradient of 1H:1V; and Type 4 soils whose walls are sloped from its bottom with a slope having minimum gradient of 3H:1V. Alternatively, in accordance with OHSA, a site-specific analysis can be carried out by a geotechnical engineer to determine the stability of the cut slopes, other than for excavations made in Type 4 soil.

All excavations must be carried out in accordance with the latest edition of the OHSA. However, if water inflow is encountered from perched groundwater, the cut slopes may need to be made flatter to reduce the extent of sloughing of the slopes and the potential for undermining overlying strata. 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 the time of construction.

Conventional excavation equipment is expected to be suitable for construction of the pond; however, grinding of the augers was noted during the advancement of some of the boreholes for the geotechnical investigation and shale and limestone fragments were observed within the till and residual soil strata at the site. Based on these observations, the presence of cobbles and boulders is inferred. The presence of boulders may interfere with or slow the progress of excavation operations. It is recommended that a Notice to Contractor be included in the Contract Documents to alert the Contractor of these obstructions and to ensure that the Contractor is equipped to handle such obstructions. An example Non-Standard Special Provision (NSSP) is included in Appendix C for inclusion in the Contract Documents.

Shale bedrock containing strong limestone interbeds is anticipated to be encountered where excavations extend into bedrock and equipment must be capable of penetrating into such material. Techniques similar to hoe-ramming will likely be required to penetrate through the harder limestone interbeds, as noted present within the bedrock core in Boreholes CRB-6 and AR-2, to reach the elevation of the proposed sewer connecting Maintenance Holes MH 770 and MH 772 to the Emergency Overflow Sewer Outlet. It is recommended that an NSSP be included in the Contract Documents to alert the Contractor of the bedrock characteristics, and that excavation into the bedrock will require appropriate equipment and construction procedures. An NSSP is provided in Appendix C for inclusion in the Contract Documents.

6.9.2 Groundwater Control During and Following Construction

As discussed in Section 6.1.1, the groundwater level measured in the monitoring well installed in Borehole SWME-3 is approximately 3.7 m below the design pond base elevation and therefore no significant groundwater seepage is anticipated to be encountered during pond construction.

Temporary excavations for drainage structures could extend up to about 3.7 m below the groundwater table. Groundwater seepage from temporary cut faces through saturated non-cohesive soils, residual soils above the bedrock and the upper weathered bedrock sections can be expected. Although seepage from the shale bedrock excavation is not anticipated to be a concern and can be managed by pumping from sumps, it may be necessary to carry out advanced dewatering prior to excavating below the groundwater level in some areas to control running / loosening of the saturated non-cohesive soils and control erosion and instability / sloughing of the temporary cut slopes. If advanced dewatering is not performed, some flattening of the temporary cut slopes may be required to allow sufficient time to allow the soil deposits to drain and groundwater levels to stabilize prior to cutting to the final grade. A 1 m wide bench is recommended at the interface of the overburden and bedrock along any excavation perimeter to mitigate for the potential for some localized erosion / loosening of the highly weathered bedrock and sloughing of overlying overburden material.

During wet periods of the year or during periods of precipitation there is the potential that perched water conditions may develop within the non-cohesive portions of the existing fill and native materials that may be underlain by a less permeable stratum and some minor seepage may occur out of the cut slope at the east portion of the pond. Such seepage is anticipated to be relatively limited in duration and quantity and should be able to be managed by pumping from local sumps.

6.9.3 Vibration Monitoring During Construction

A maximum peak particle velocity (PPV) of 100 mm/s is generally considered an applicable vibration level for bridge structures in good condition. Based on vibration monitoring experience at other bridge sites, it is considered unlikely that vibrations induced by conventional construction activities such as hoe-ramming and line drilling of shale bedrock excavations will reach this threshold level and, therefore, vibration monitoring of the existing Credit River Bridge is not expected to be required during construction at this site. It is noted however, based on the grades of the Dry Pond Emergency Overflow Sewer Outlet structures that excavations into the bedrock is expected at the outlet MH 770 (and possibly at the adjacent ditch inlet MH 768, at the intermediate MH 772 and the Overflow Outfall) and given the proximity of these proposed structures to the east abutment, Piers 5 and 6 of the existing Credit River bridge, it is considered prudent that vibration monitoring of the bridge be carried out during temporary excavation for the Dry Pond drainage structures.

Residential homes are located within about 100 m of the proposed Dry Pond. A lower PPV threshold of 25 mm/s is generally considered applicable for vibration impacts on buildings, and the zone of influence could extend to about 250 m. Therefore, vibration monitoring should be carried out at the existing structures located within this zone of influence during bedrock excavation. An NSSP describing the requirements for vibration monitoring is presented in Appendix C for inclusion in the Contract Documents.

7.0 CLOSURE

This report was prepared by Mr. Matt Soderman, P.Eng., a Geotechnical Engineer with Golder. Ms. Sandra McGaghran, M.Eng., P.Eng., a Geotechnical Engineer and Associate with Golder reviewed the technical aspects of the report. Mr. Jorge Costa, P.Eng., MTO Foundations Designated Contact for Golder and Senior Consultant, conducted a quality control audit of the report.

Golder Associates Ltd.

LICENSED PROFESSIONAL ENGINEER
M. A. SODERMAN
100160416
MARCH 4th, 2019
PROVINCE OF ONTARIO

Matt Soderman, P.Eng
Geotechnical Engineer

Sandra McGaghran, M.Eng., P.Eng.
Senior Geotechnical Engineer, Associate

LICENSED PROFESSIONAL ENGINEER
J. M. A. COSTA
Mar 4/2019
PROVINCE OF ONTARIO

Jorge M.A. Costa, P.Eng.
MTO Foundations Designated Contact, Senior Consultant

AM/MAS/SMM/JMAC/rb

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https://golderassociates.sharepoint.com/sites/11176g/shared documents/07-reporting/foundations/8 - dry pond/3. final/1662333 fidr - swme dry pond 2019march4.docx

REFERENCES

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

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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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.

Kulhawy, F.H. and Mayne, P.W., 1990. *Manual on Estimating Soil Properties for Foundation Design*. EL 6800, Research Project 1493 6. Prepared for Electric Power Research Institute, Palo Alto, California.

Peck, R.B., Hanson, W.E., and Thornburn, T.H., 1974. *Foundation Engineering*, Second Edition, John Wiley and Sons, New York.

ASTM International:

ASTM D1586	Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils
ASTM D7012	Standard Test Method for Compressive Strength and Elastic Moduli of Intact Rock Core Specimens under Varying States of Stress and Temperatures

Commercial Software:

Slide (Version 8.0) by Rocscience Inc.

Settle^{3D} (Version 4.0) by Rocscience Inc.

Ontario Provisional Standard Drawing:

OPSD 810.010	General Rip-Rap Treatment Layout for Sewer and Culvert Outlets

Ontario Provincial Standard Specification:

OPSS.PROV 206	Construction Specification for Grading
OPSS.PROV 212	Construction Specification for Earth Borrow
OPSS.PROV 501	Construction Specification for Compacting
OPSS.PROV 511	Construction Specification for Rip Rap, Rock Protection and Granular Sheeting
OPSS 802	Construction Specification for Topsoil
OPSS.PROV 803	Construction Specification for Sodding
OPSS.PROV 804	Construction Specification for Seed and Cover
OPSS.PROV 1004	Material Specification for Aggregates – Miscellaneous
OPSS.PROV 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material

OPSS.PROV 1860	Material Specification for Aggregates - Geotextiles

Ontario Water Resources Act:

Ontario Regulation 903	Wells (as amended)

Ontario Occupational Health and Safety Act:

Ontario Regulation 213/91	Construction Projects (as amended)

MINISTRY OF TRANSPORTATION, ONTARIO

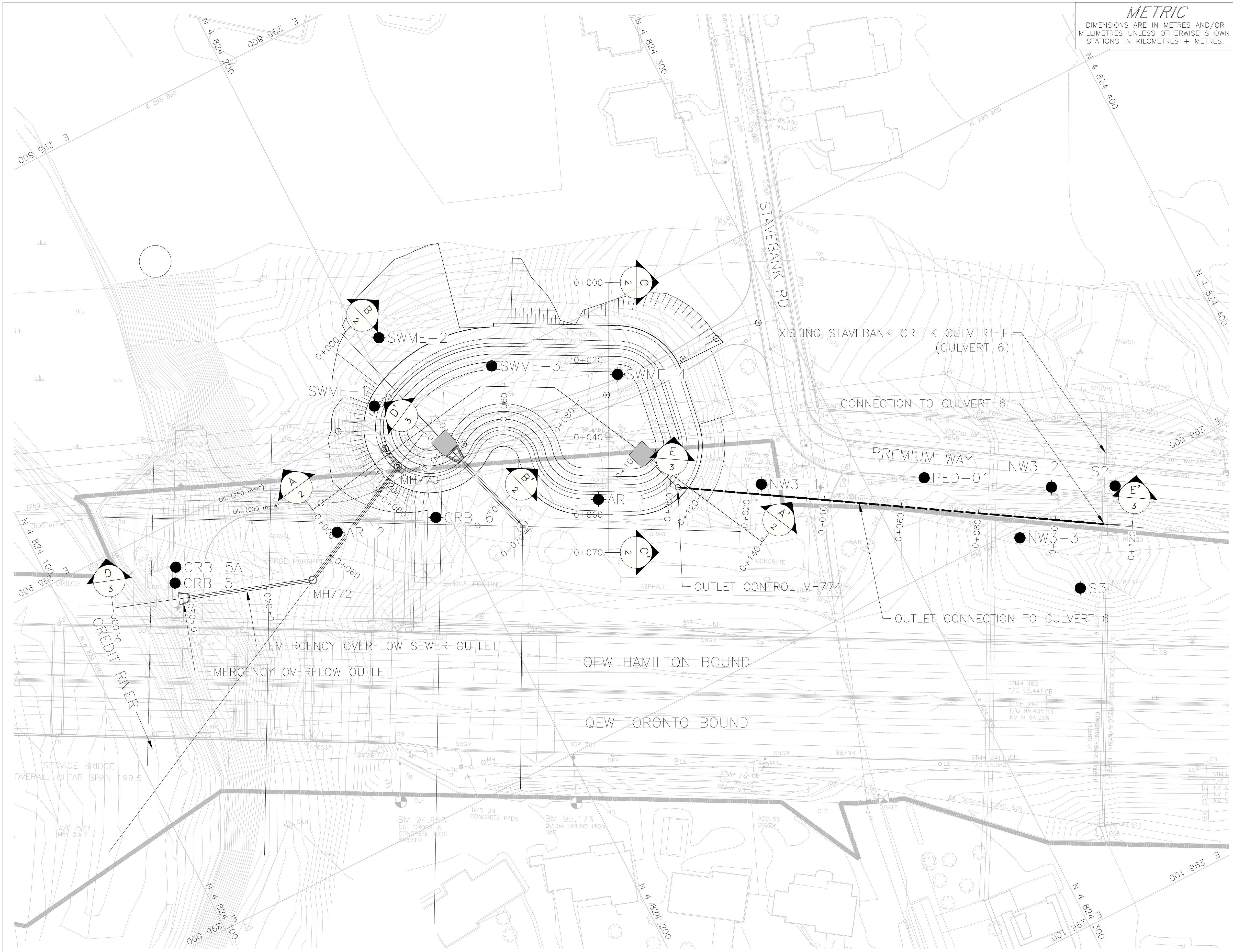
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No. 2019–2016
GWP No. 2002–13–00

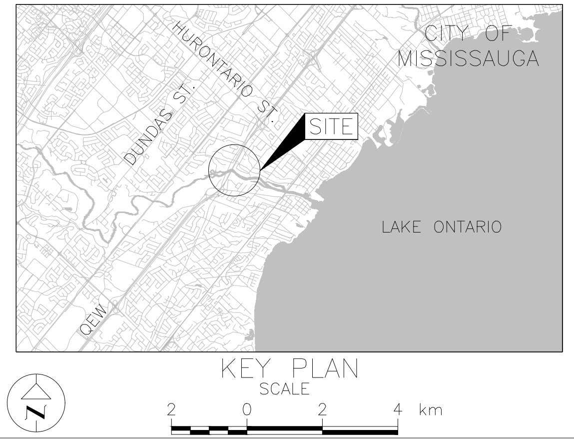
SHEET

QEW WIDENING MISSISSAUGA RD TO HURONTARIO ST
STORMWATER MANAGEMENT DRY POND
AND STORM SEWER
BOREHOLE LOCATIONS

GOLDER



PLAN
SCALE
10 0 10 20 m



KEY PLAN
SCALE
2 0 2 4 km

LEGEND

● ● Borehole – Current Investigation

BOREHOLE CO-ORDINATES (MTM NAD 83 ZONE 10)			
No.	ELEVATION	NORTHING	EASTING
SWME–4	95.5	4824255.4	295917.6
SWME–3	91.9	4824227.3	295901.0
SWME–2	87.4	4824204.6	295881.3
SWME–1	89.5	4824195.5	295896.6
S3	90.0	4824337.3	296021.0
S2	94.9	4824357.2	296001.4
PED–01	96.3	4824314.1	295977.3
NW3–3	90.6	4824329.2	296002.3
NW3–2	95.3	4824342.4	295994.3
NW3–1	96.5	4824275.8	295959.8
CRB–6	91.7	4824196.7	295929.5
CRB–5A	79.3	4824130.9	295910.6
CRB–5	79.2	4824128.9	295914.2
AR–2	88.4	4824172.2	295921.4
AR–1	95.7	4824236.4	295944.2

NOTES

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

REFERENCE

Base plans provided in digital format by Morrison Hershfield, drawing file no. X11609340Base.dwg, received April 12, 2018.
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Access Road plan provided in digital format by Morrison Hershfield, drawing file 1160934 – Construction Access Road.dwg, received July26, 2018.
Dry pond plan provided in digital format by Morrison Hershfield, drawing file no. 11609340–Proposed_PONDS–3D.dwg, received DEC. 14, 2018.

J.M.A.COSTA
Feb. 15, 2019

S. McGaghran
Feb. 15, 2019

NO.	DATE	BY	REVISION

Geocres No. 30M12–436

HWY. QEW		PROJECT NO. 1662333	DIST. CENTRAL
SUBM'D. DM	CHKD. MS	DATE: 02/15/2019	SITE:
DRAWN: DD	CHKD. SMM	APPD. JMAC	DWG. 1

PLOT DATE: March 1, 2019
FILENAME: S:\Clients\MTO\QEW–Credit_River\99_PROJ\1662333_MH_P&F\40_PROD\0015_SWM_Pond_Dry\1662333–0015–BG–0001.dwg

MINISTRY OF TRANSPORTATION, ONTARIO

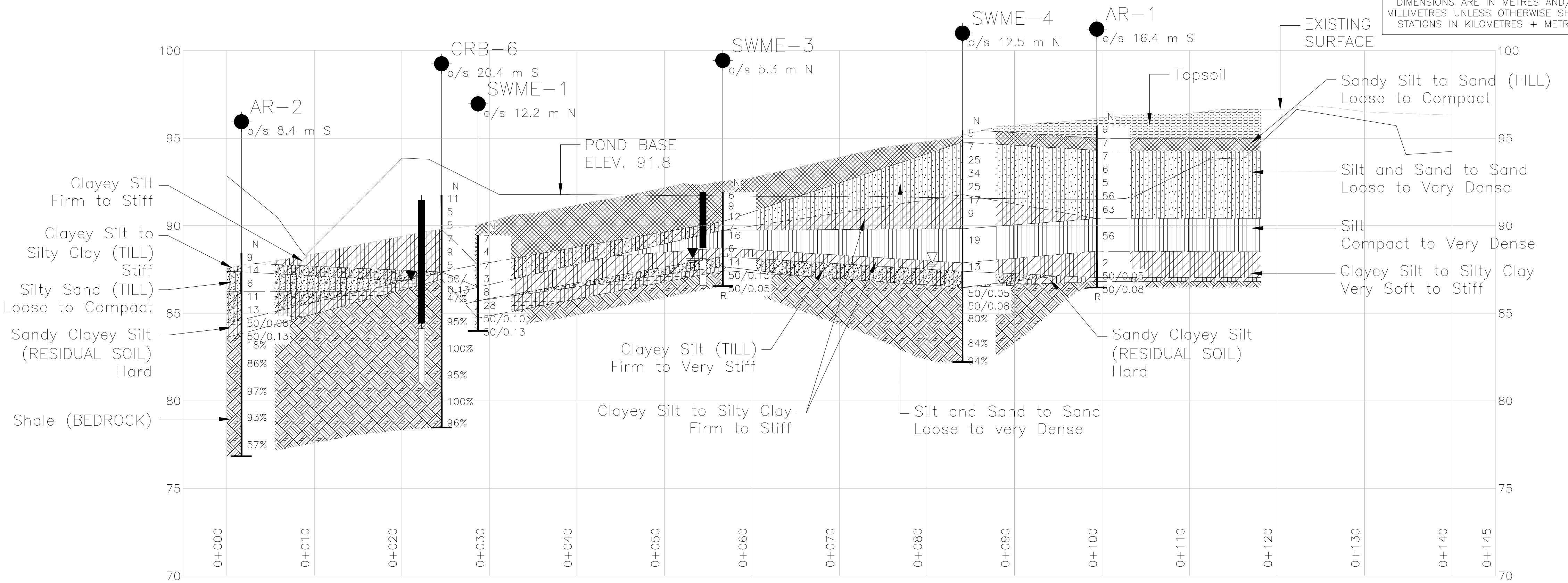
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No. 2019-2016
GWP No. 2002-13-00

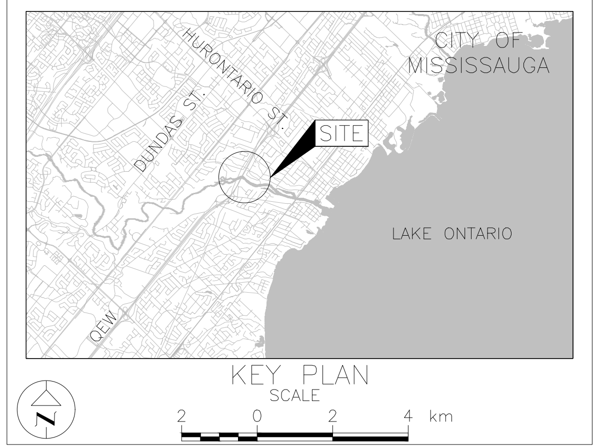
QEW WIDENING MISISSAUGA RD TO HURONTARIO ST
STORMWATER MANAGEMENT DRY POND
SOIL STRATA

SHEET

GOLDER



A-A / 1 PROFILE A-A'



KEY PLAN
SCALE

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)
- R Split-Spoon Refusal
- ▼ WL in piezometer, measured on NOV 6, 2018
- ▽ WL upon completion of drilling

BOREHOLE CO-ORDINATES (MTM NAD 83 ZONE 10)

No.	ELEVATION	NORTHING	EASTING
SWME-4	95.5	4824255.4	295917.6
SWME-3	91.9	4824227.3	295901.0
SWME-2	87.4	4824204.6	295881.3
SWME-1	89.5	4824195.5	295896.6
CRB-6	91.7	4824196.7	295929.5
AR-2	88.4	4824172.2	295921.4
AR-1	95.7	4824236.4	295944.2

NOTES

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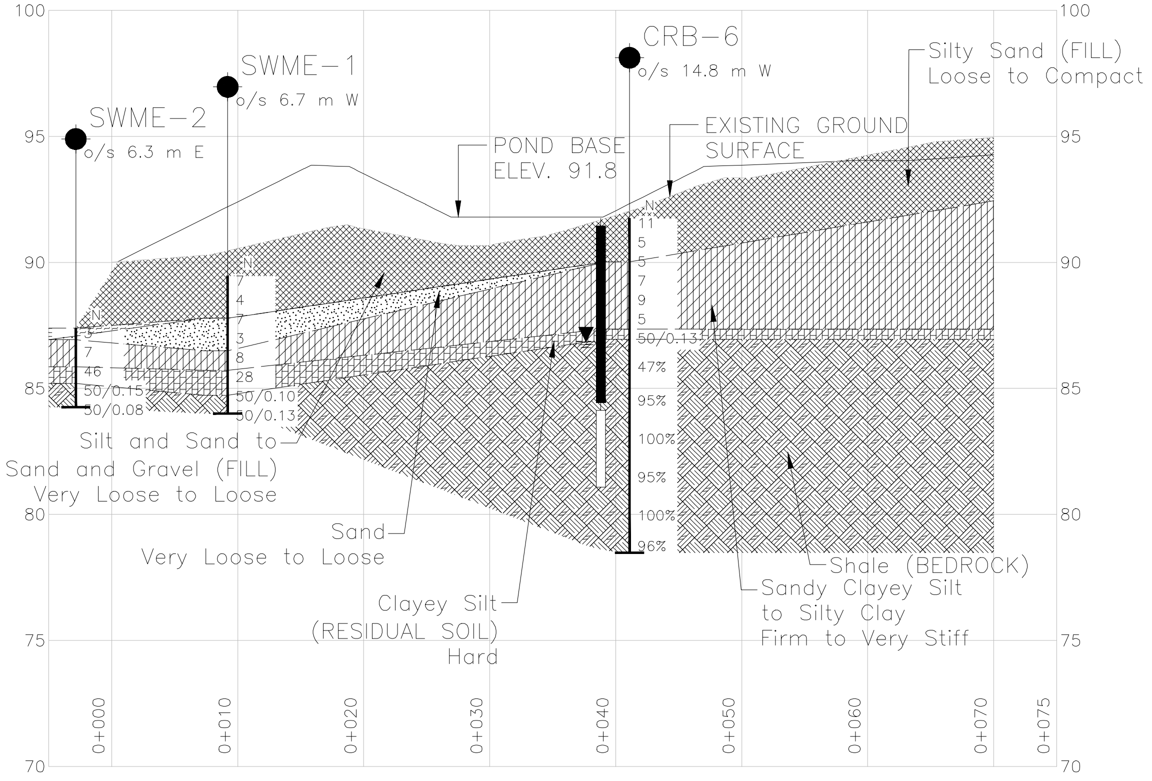
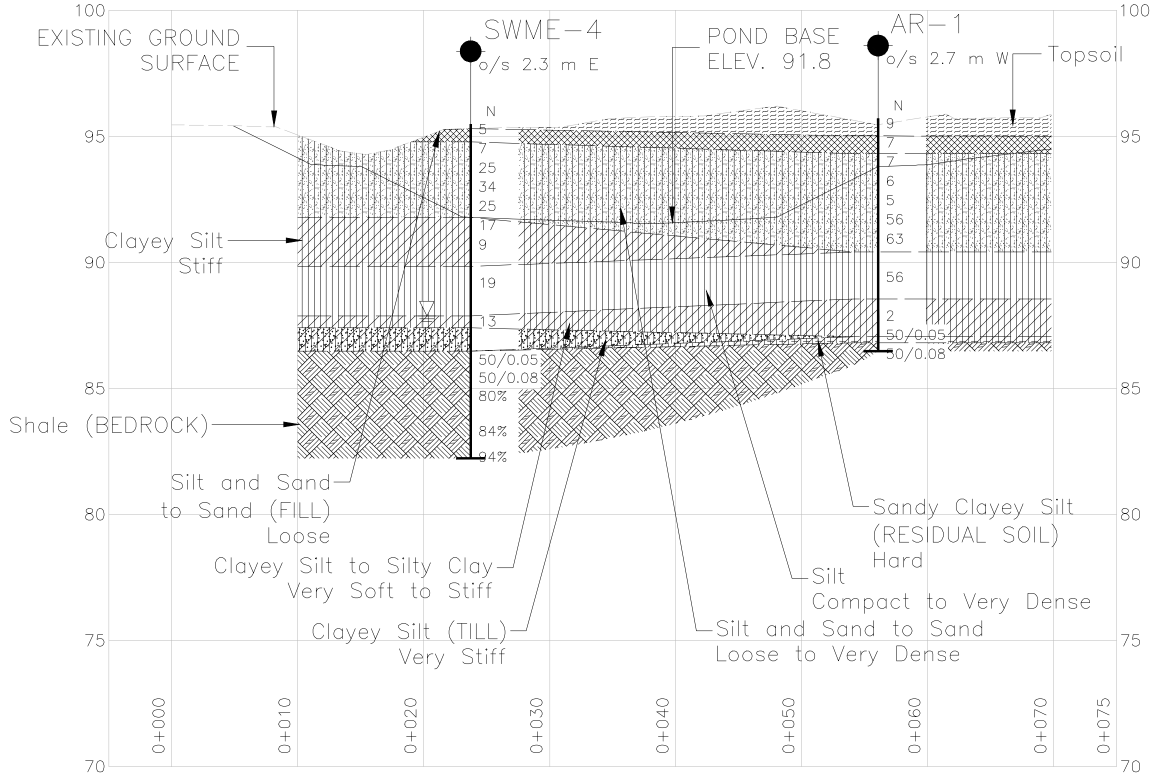


B-B / 1 CROSS SECTION B-B'



C-C / 1 CROSS SECTION C-C'

HORIZONTAL SCALE
6 0 6 12 m

VERTICAL SCALE
3 0 3 6 m

LICENSED PROFESSIONAL ENGINEER
J.M.A. COSTA
Feb. 15, 2019

LICENSED PROFESSIONAL ENGINEER
S. McGaghran
Feb. 15, 2019

NO.	DATE	BY	REVISION

Geocres No. 30M12-436

HWY. QEW	PROJECT NO. 1662333		DIST. CENTRAL
SUBM'D. DM	CHKD. MS	DATE: 02/15/2019	SITE:
DRAWN: DD	CHKD. SMM	APPD. JMAC	DWG. 2

PLOT DATE: February 19, 2019
FILENAME: S:\Clients\MTO\QEW-Credit_River\99_PROJ\1662333_MH_P&F\40_PROD\0015_SWM_Pond_Dry\1662333-0015-BG-0002.dwg

MINISTRY OF TRANSPORTATION, ONTARIO

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

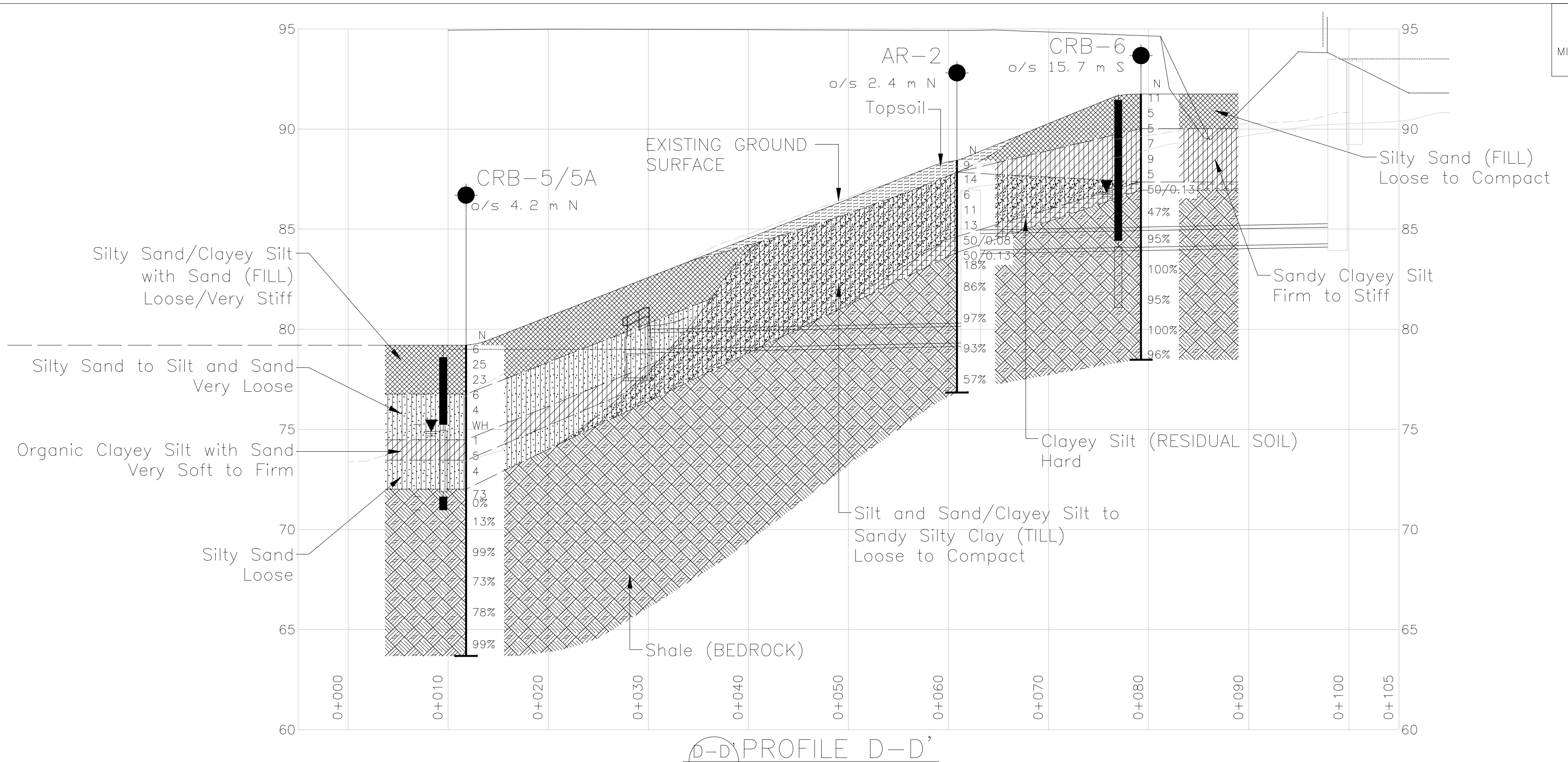
CONT No. 2019-2016
GWP No. 2002-13-00

QEW WIDENING - MISSISSAUGA RD TO HURONTARIO ST
STORMWATER MANAGEMENT POND

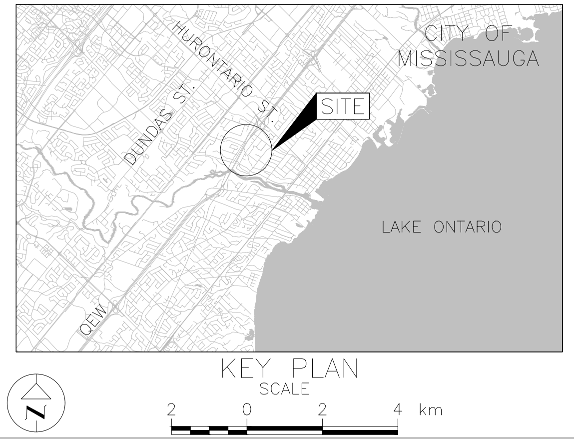
SOIL STRATA

SHEET

GOLDER



D-D' 1 PROFILE D-D'



KEY PLAN

SCALE

2 0 2 4 km

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, measured on NOV 6, 2018
▽	WL upon completion of drilling

BOREHOLE CO-ORDINATES (MTM NAD 83 ZONE 10)

No.	ELEVATION	NORTHING	EASTING
S3	90.0	4824337.3	296021.0
S2	94.9	4824357.2	296001.4
PED-01	96.3	4824314.1	295977.3
NW3-3	90.6	4824329.2	296002.3
NW3-2	95.3	4824342.4	295994.3
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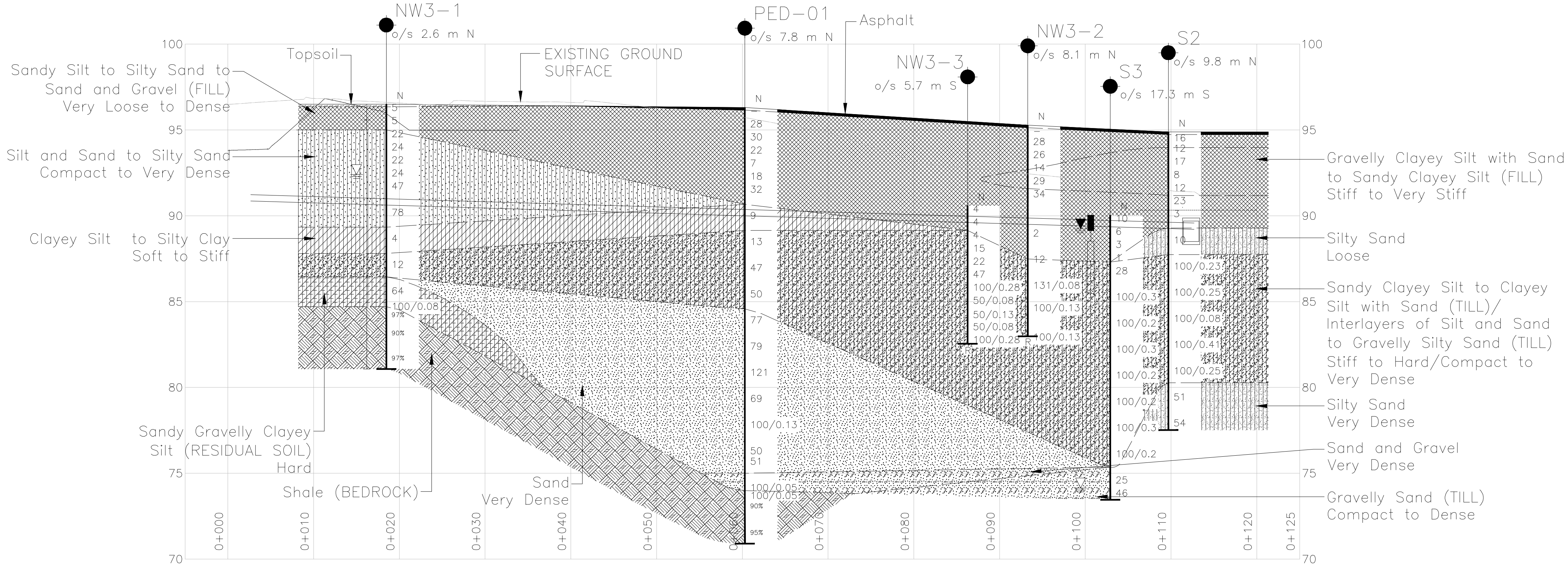
CRB-5	79.2	4824128.9	295914.2
AR-2	88.4	4824172.2	295921.4



E-E' 1 PROFILE E-E'

HORIZONTAL SCALE
6 0 6 12 m

VERTICAL SCALE
3 0 3 6 m

NOTES

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J.M.A.COSTA
Mar. 04, 2019

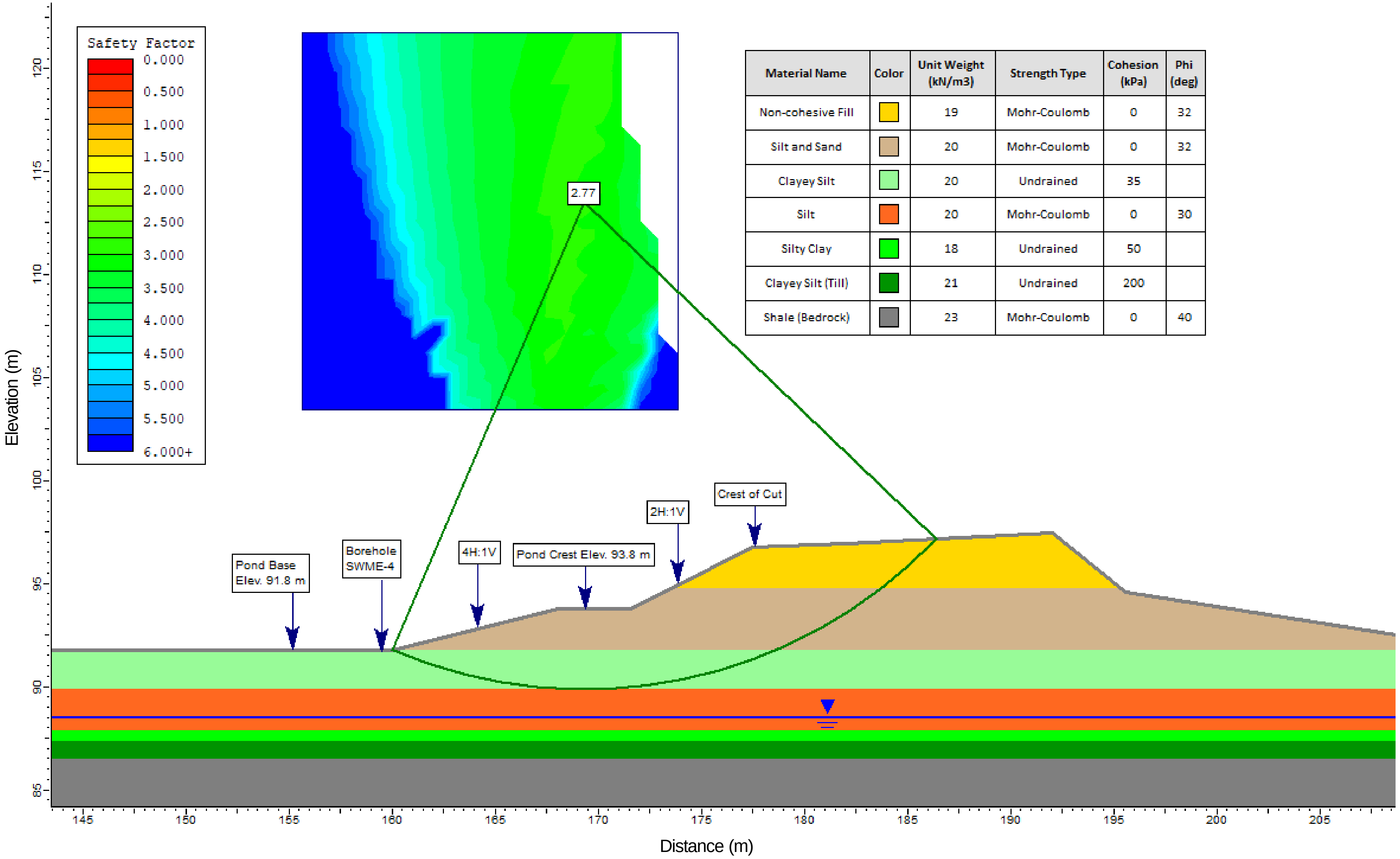
S. McGaghran
Mar. 04, 2019

NO.	DATE	BY	REVISION

Geocres No. 30M12-436

HWY. QEW		PROJECT NO. 1662333	DIST. CENTRAL
SUBM'D. DM	CHKD. KN	DATE: 2019-03-04	SITE:
DRAWN: DD	CHKD. SMM	APPD. JMAC	DWG. 3

PLOT DATE: March 4, 2019
FILENAME: S:\Clients\MTO\QEW-Credit_River\99_PROJ\1662333_MH_P&F\40_PROD\0015_SWM_Pond_Dry\1662333-0015-BG-0003.dwg



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
Non-cohesive Fill	🟨	19	Mohr-Coulomb	0	32
Silt and Sand	🟫	20	Mohr-Coulomb	0	32
Clayey Silt	🟩	20	Undrained	35	
Silt	🟧	20	Mohr-Coulomb	0	30
Silty Clay	🟩	18	Undrained	50	
Clayey Silt (Till)	🟩	21	Undrained	200	
Shale (Bedrock)	⬛	23	Mohr-Coulomb	0	40

CLIENT
MORRISON HERSHFIELD LIMITED

CONSULTANT
GOLDER

YYYY-MM-DD	2019-01-14
PREPARED	JIL
DESIGN	JIL
REVIEW	MAS
APPROVED	SMM

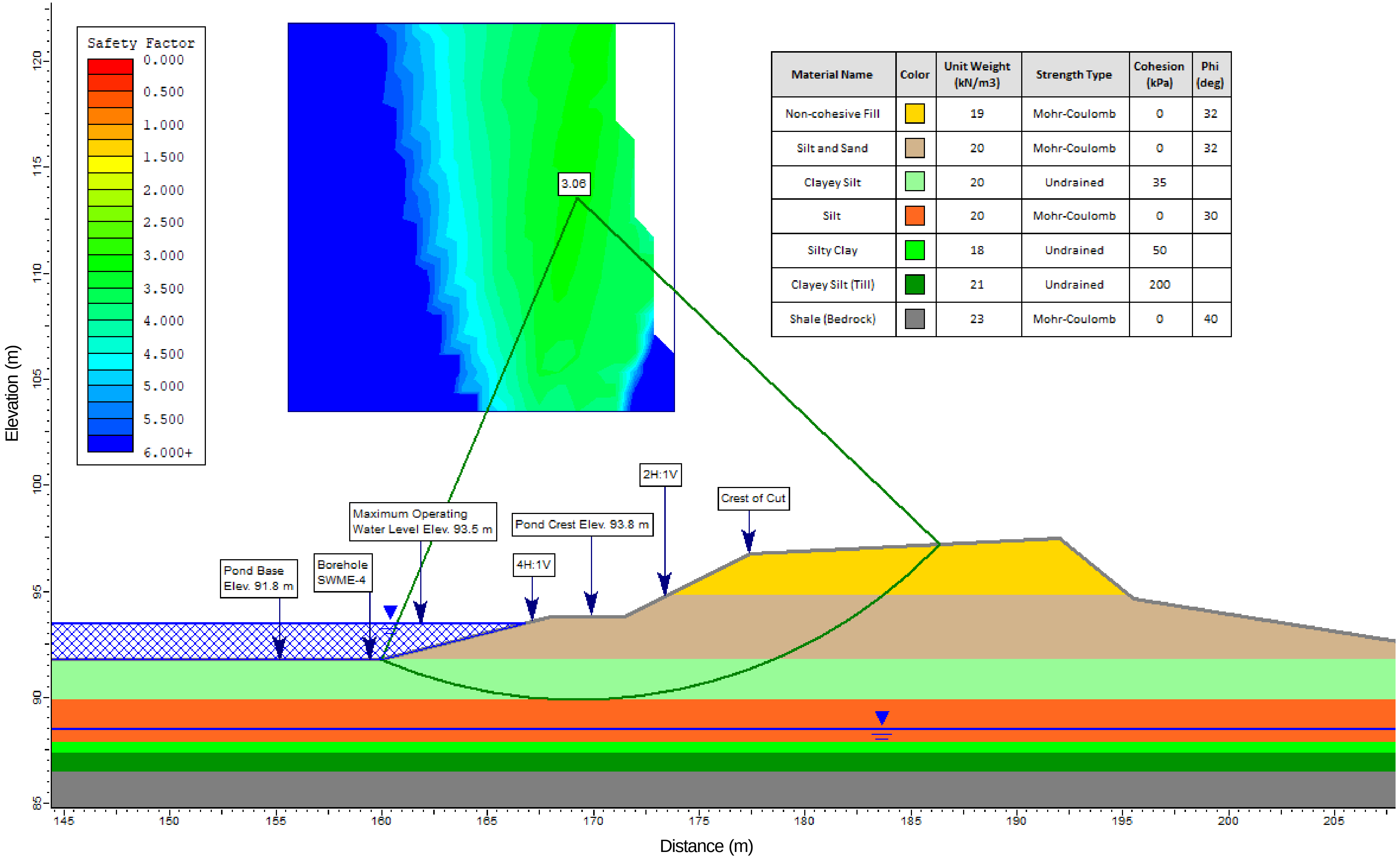
PROJECT
STORMWATER MANAGEMENT POND
QEW WIDENING FROM WEST OF MISSISSAUGA ROAD TO
WEST OF HURONTARIO STREET, CITY OF MISSISSAUGA
MINISTRY OF TRANSPORTATION, ONTARIO, G.W.P. 2002-13-00

TITLE
STATIC SLOPE STABILITY ANALYSIS – SWM DRY POND – CUT SLOPE NORMAL OPERATING WATER LEVEL CONDITIONS (DRY)

PROJECT No.
1662333

FIGURE No.
1

IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: ANSI A
1 in
0



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
Non-cohesive Fill	🟨	19	Mohr-Coulomb	0	32
Silt and Sand	🟫	20	Mohr-Coulomb	0	32
Clayey Silt	🟩	20	Undrained	35	
Silt	🟧	20	Mohr-Coulomb	0	30
Silty Clay	🟩	18	Undrained	50	
Clayey Silt (Till)	🟩	21	Undrained	200	
Shale (Bedrock)	◼	23	Mohr-Coulomb	0	40

CLIENT
MORRISON HERSHFIELD LIMITED

CONSULTANT

GOLDER

YYYY-MM-DD	2019-01-14
PREPARED	JIL
DESIGN	JIL
REVIEW	MAS
APPROVED	SMM

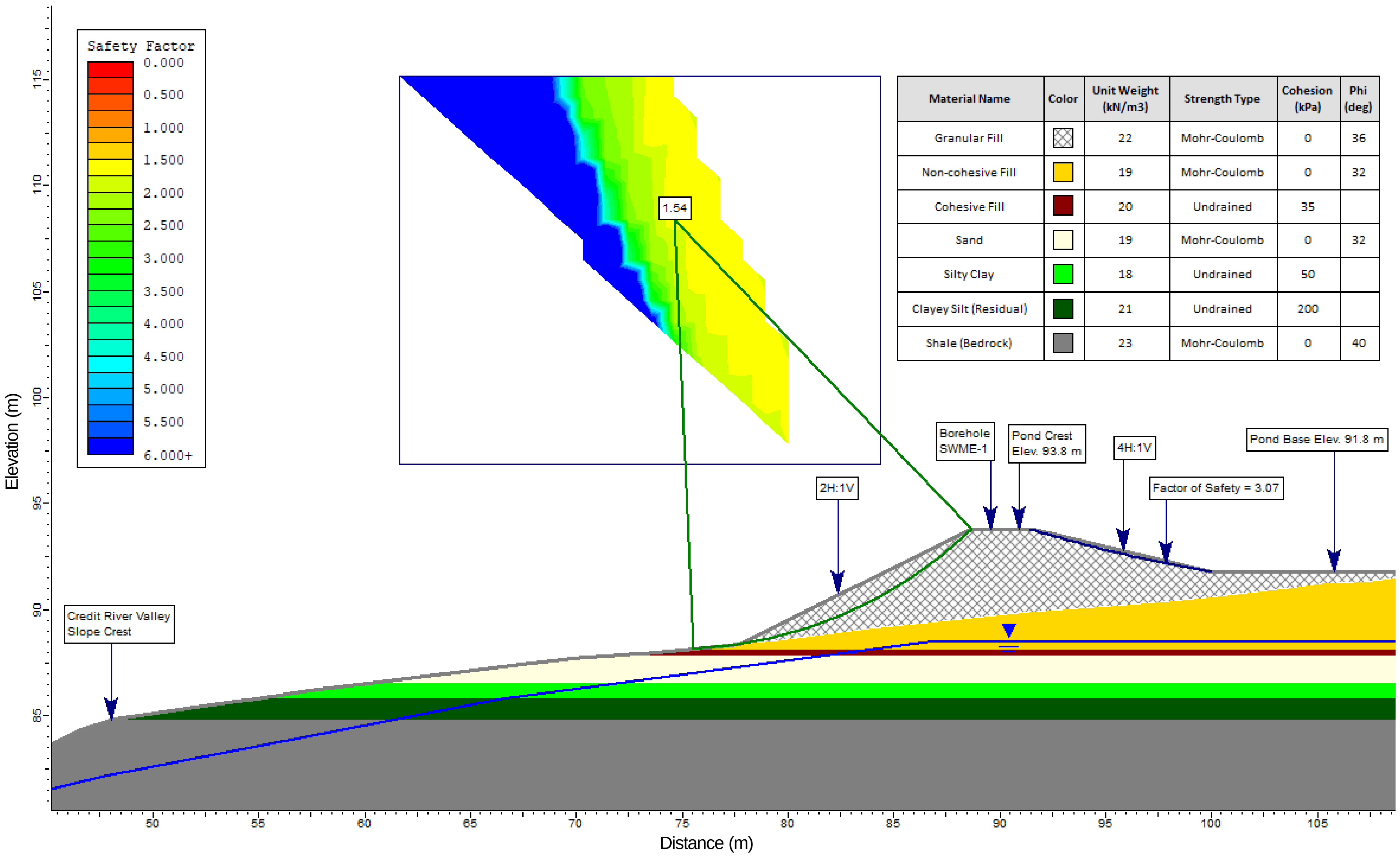
PROJECT
STORMWATER MANAGEMENT POND
QEW WIDENING FROM WEST OF MISSISSAUGA ROAD TO
WEST OF HURONTARIO STREET, CITY OF MISSISSAUGA
MINISTRY OF TRANSPORTATION, ONTARIO, G.W.P. 2002-13-00

TITLE
STATIC SLOPE STABILITY ANALYSIS – SWM DRY POND – CUT SLOPE
MAXIMUM OPERATING WATER LEVEL CONDITIONS (STORM EVENTS)

PROJECT No.
1662333

FIGURE No.
2

IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: ANSI A
1 in
0



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
Granular Fill	▨	22	Mohr-Coulomb	0	36
Non-cohesive Fill	🟨	19	Mohr-Coulomb	0	32
Cohesive Fill	🟥	20	Undrained	35	
Sand	⬜	19	Mohr-Coulomb	0	32
Silty Clay	🟩	18	Undrained	50	
Clayey Silt (Residual)	🟩	21	Undrained	200	
Shale (Bedrock)	⬛	23	Mohr-Coulomb	0	40

CLIENT
MORRISON HERSHFIELD LIMITED

CONSULTANT

GOLDER

YYYY-MM-DD	2019-01-14
PREPARED	JIL
DESIGN	JIL
REVIEW	MAS
APPROVED	SMM

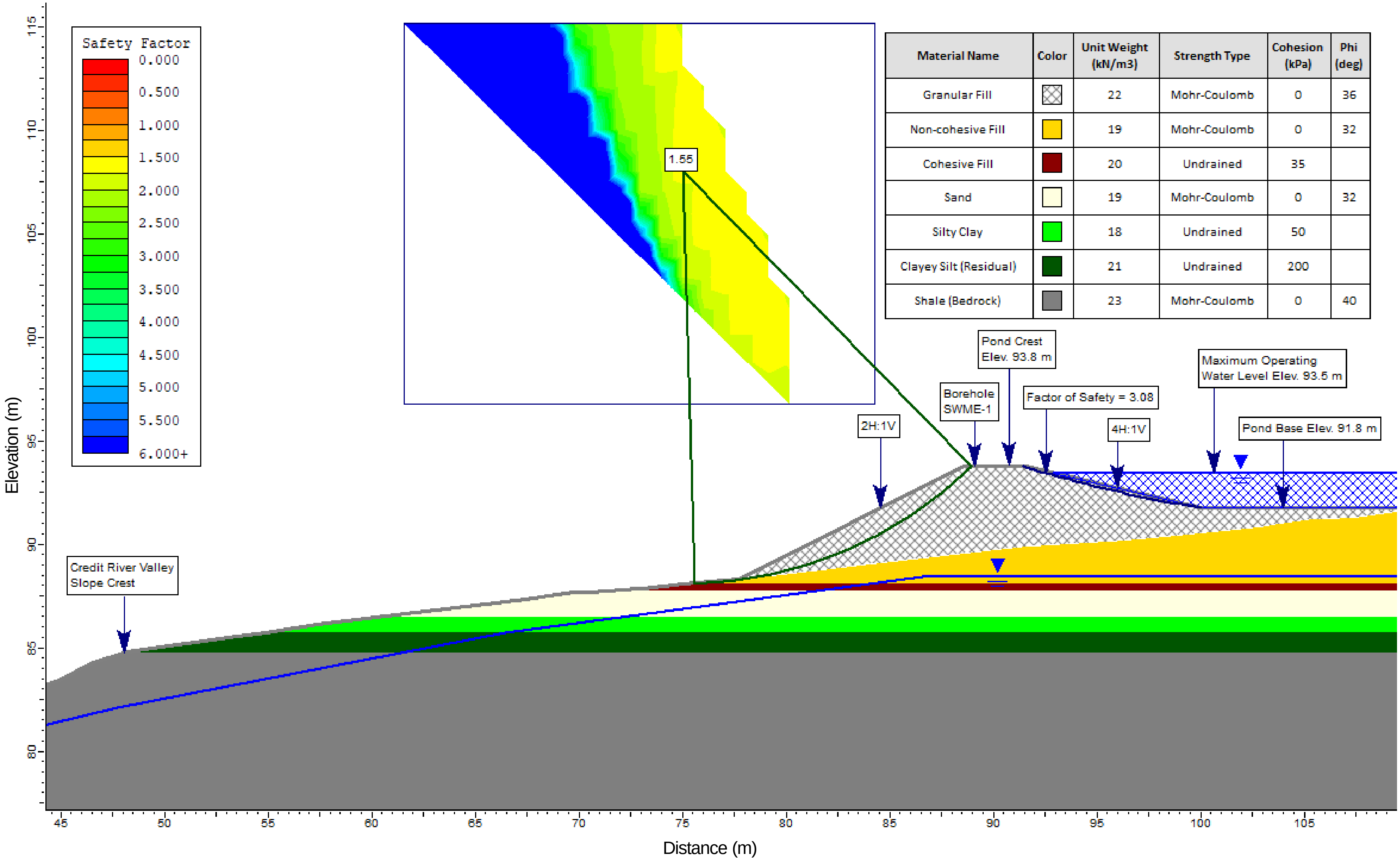
PROJECT
STORMWATER MANAGEMENT POND
QEW WIDENING FROM WEST OF MISSISSAUGA ROAD TO
WEST OF HURONTARIO STREET, CITY OF MISSISSAUGA
MINISTRY OF TRANSPORTATION, ONTARIO, G.W.P. 2002-13-00

TITLE
STATIC SLOPE STABILITY ANALYSIS – SWM DRY POND – GRANULAR FILL SLOPE – NORMAL OPERATING WATER LEVEL CONDITIONS (DRY)

PROJECT No.
1662333

FIGURE No.
3

IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: ANSI A
1 in
0



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
Granular Fill	▨	22	Mohr-Coulomb	0	36
Non-cohesive Fill	🟨	19	Mohr-Coulomb	0	32
Cohesive Fill	🟥	20	Undrained	35	
Sand	⬜	19	Mohr-Coulomb	0	32
Silty Clay	🟩	18	Undrained	50	
Clayey Silt (Residual)	🟩	21	Undrained	200	
Shale (Bedrock)	⬛	23	Mohr-Coulomb	0	40

CLIENT
MORRISON HERSHFIELD LIMITED

CONSULTANT
GOLDER

YYYY-MM-DD	2019-01-14
PREPARED	JIL
DESIGN	JIL
REVIEW	MAS
APPROVED	SMM

PROJECT
STORMWATER MANAGEMENT POND
QEW WIDENING FROM WEST OF MISSISSAUGA ROAD TO
WEST OF HURONTARIO STREET, CITY OF MISSISSAUGA
MINISTRY OF TRANSPORTATION, ONTARIO, G.W.P. 2002-13-00

TITLE
STATIC SLOPE STABILITY ANALYSIS – SWM DRY POND – GRANULAR FILL SLOPE – MAXIMUM OPERATING WATER LEVEL CONDITIONS (STORM EVENTS)

PROJECT No.
1662333

FIGURE No.
4

IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: ANSI A
1 in
0

APPENDIX A

Record of Borehole and Drillhole Sheets and Bedrock Core Photographs

LIST OF SYMBOLS

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 $= (w_l - w_p)$
w_s	shrinkage limit
I_L	liquidity index $= (w - w_p) / I_p$
I_C	consistency index $= (w_l - w) / I_p$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
I_D	density index $= (e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(d) Shear Strength

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

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

LIST OF ABBREVIATIONS

The abbreviations commonly employed on Records of Boreholes, on figures and in the text of the report are as follows:

I. SAMPLE TYPE

AS	Auger sample
BS	Block sample
CS	Chunk sample
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
SS	Split-spoon
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample

II. 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.) drive open sampler for a distance of 300 mm (12 in.)

Dynamic Cone Penetration Resistance; 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, 60º 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

Piezo-Cone Penetration Test (CPT)

A 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 (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

III. SOIL DESCRIPTION

(a) Non-Cohesive (Cohesionless) Soils

Compactness Condition	N Blows/300 mm or Blows/ft
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

(b) Cohesive Soils

Consistency

	c_u, s_u kPa	c_u, s_u psf
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1,000
Stiff	50 to 100	1,000 to 2,000
Very stiff	100 to 200	2,000 to 4,000
Hard	over 200	over 4,000

IV. SOIL TESTS

w	water content
w_p	plastic limit
w_l	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test[1]
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement[1]
D_R	relative density (specific gravity, G_s)
DS	direct shear test
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	field vane (LV-laboratory vane test)
γ	unit weight

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

V. MINOR SOIL CONSTITUENTS

Per cent by Weight	Modifier	Example
0 to 5	Trace	Trace sand
5 to 12	Trace to Some (or Little)	Trace to some sand
12 to 20	Some	Some sand
20 to 30	(ey) or (y)	Sandy
over 30	And (non-cohesive (cohesionless)) or With (cohesive)	Sand and Gravel Silty Clay with sand / Clayey Silt with sand

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, recovered at full diameter, measured relative to the length of the total core run. RQD varied from 0% for completely broken core to 100% for core in solid sticks.

DISCONTINUITY DATA

Fracture Index

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

Dip with Respect to Core Axis

The angle of the discontinuity relative to the axis (length) of the core. In a vertical borehole a discontinuity with a 90^o 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		

RECORD OF BOREHOLE No SWME-1

SHEET 1 OF 1

METRIC

PROJECT 1662333
G.W.P. 2002-13-00
DIST Central HWY QEW
DATUM Geodetic
LOCATION N 4824195.5; E 295896.6 MTM NAD 83 ZONE 10 (LAT. 43.557643; LONG. -79.610206)
BOREHOLE TYPE CME 55, 210 mm O.D., Hollow Stem Augers
DATE July 31, 2018
ORIGINATED BY ACM
COMPILED BY CC
CHECKED BY SMM

SOIL PROFILE

ELEV DEPTH	DESCRIPTION
89.5	GROUND SURFACE
0.0	Sand and gravel (FILL) Loose Brown grey Moist
88.8 0.7	Silt and sand, trace clay, trace gravel (FILL) Very loose Brown Moist
88.1 1.7	Clayey silt, trace to some sand (FILL), oxidation staining Grey Moist
	SAND, some silt, trace clay, trace rootlets Very loose to loose Brown to dark brown Moist - organics between depths of 2.6 m and 2.9 m
86.5 3.0	SILTY CLAY, some sand, trace gravel, trace organics Firm to very stiff Grey brown Moist
85.8 3.7	CLAYEY SILT, some gravel, some shale fragments (RESIDUAL SOIL) Hard Grey Moist
84.8 4.7	SHALE (BEDROCK) Grey
84.0 5.5	END OF BOREHOLE SPLIT-SPOON REFUSAL

NOTE:

1. Open borehole dry upon completion of drilling.

SAMPLES

NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
1	SS	7						
2	SS	4			1	64	30	5
3	SS	7						
4	SS	3			0	77	19	4
5A 5B	SS	8			2	13	49	36
6	SS	28						
7	SS	50/0.10						
8	SS	50/0.13						

REMARKS & GRAIN SIZE DISTRIBUTION (%): GR SA SI CL

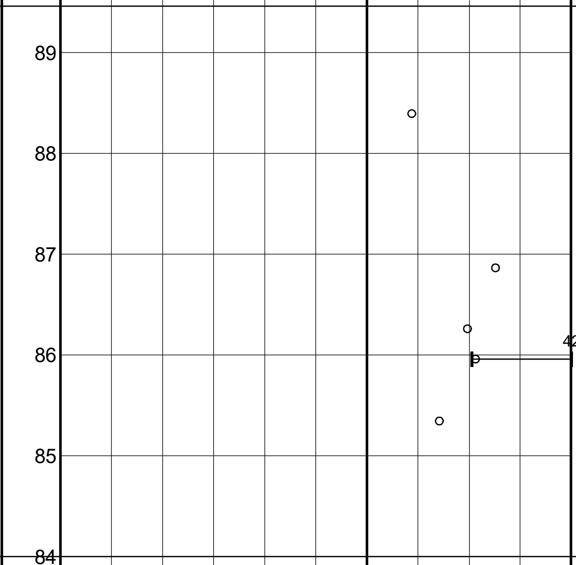
ELEVATION SCALE

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



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

RECORD OF BOREHOLE No SWME-2

SHEET 1 OF 1

METRIC

PROJECT 1662333

G.W.P. 2002-13-00

DIST Central HWY QEW

DATUM Geodetic

LOCATION N 4824204.6; E 295881.3 MTM NAD 83 ZONE 10 (LAT. 43.557725; LONG. -79.610395)

BOREHOLE TYPE CME 55, 210 mm O.D., Hollow Stem Augers

DATE July 27, 2018

ORIGINATED BY ACM

COMPILED BY CC

CHECKED BY SMM

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES			GR	SA	SI	CL
87.4	GROUND SURFACE									
0.0 86.9	TOPSOIL (500 mm) Soft	1	SS	3		87				
0.5	CLAYEY SILT with SAND, some gravel Firm Brown Moist - Trace to some rootlets, organics, wood, tree fragments from 0.8 m to 1.4 m	2	SS	7		86	20	31	34	15
85.9 1.5	CLAYEY SILT, trace to some sand, some shale fragments (RESIDUAL SOIL) Hard Brown Moist	3	SS	46						
85.2 2.2		4	SS	50/0.15		85				
84.3 3.1	SHALE (BEDROCK) Grey	5	SS	50/0.08						
	END OF BOREHOLE SPLIT-SPOON REFUSAL									

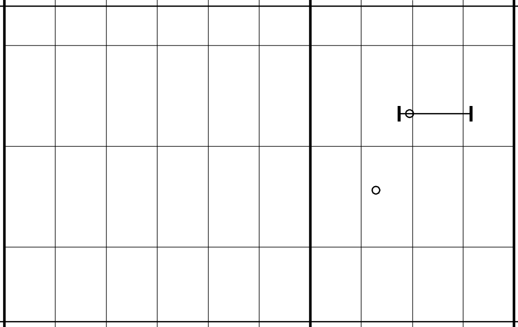
DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

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



NOTE:

1. Open borehole dry upon completion of drilling.

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

$\circ^{3\%}$ STRAIN AT FAILURE

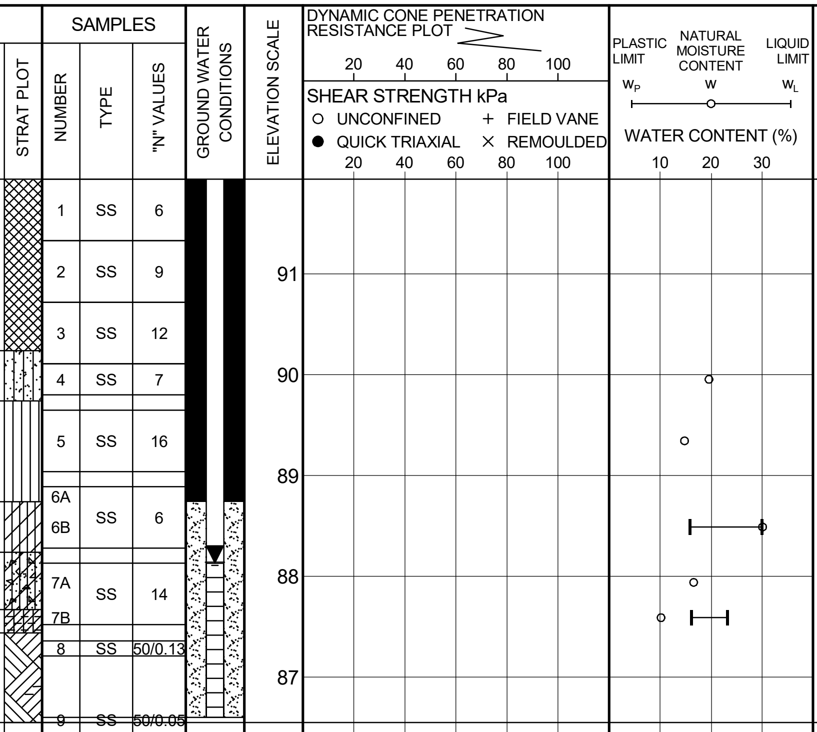
RECORD OF BOREHOLE No SWME-3

SHEET 1 OF 1

METRIC

PROJECT 1662333
G.W.P. 2002-13-00
LOCATION N 4824227.3; E 295901.0 MTM NAD 83 ZONE 10 (LAT. 43.557930; LONG. -79.610152)
ORIGINATED BY ACM
DIST Central HWY QEW
BOREHOLE TYPE CME 55, 210 mm O.D., Hollow Stem Augers
COMPILED BY CC
DATUM Geodetic
DATE July 27, 2018
CHECKED BY SMM

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLES – NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
91.9	GROUND SURFACE							
0.0	Sandy silt to silty sand to, trace to some clay (FILL) Loose to compact Brown Moist - Some oxidation between depths of 0.6 m and 1.2 m	1 2 3	SS SS SS	6 9 12				
90.2 1.7	Sandy SILT, trace to some clay Loose Brown Moist	4	SS	7	0	29	64	7
89.7 2.2	SILT, some sand, trace to some clay Compact Grey Moist	5	SS	16	0	13	79	8
88.7 3.2	CLAYEY SILT, trace sand Firm Grey Moist	6A 6B	SS	6	0	2	59	39
88.2 3.7	CLAYEY SILT, trace gravel, trace sand (TILL) Firm to stiff Grey Moist to wet	7A 7B	SS	14				
87.6 4.5	CLAYEY SILT, some sand, some shale fragments (RESIDUAL SOIL) Grey Moist	8	SS	50/0.13				
86.5 5.4	SHALE (BEDROCK) Grey	9	SS	50/0.05				
	END OF BOREHOLE SPLIT-SPOON REFUSAL							



UNIT WEIGHT γ kN/m^3

NOTE:

1. Water level in open borehole at a depth of 5.3 m (Elev. 86.6 m) below ground surface upon completion of drilling.

2. Groundwater level measurements in piezometer:

Date	Depth (m)	Elev. (m)
14/08/18	4.0	87.9
06/11/18	3.8	88.1

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

$\circ^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No SWME-4

SHEET 1 OF 1

METRIC

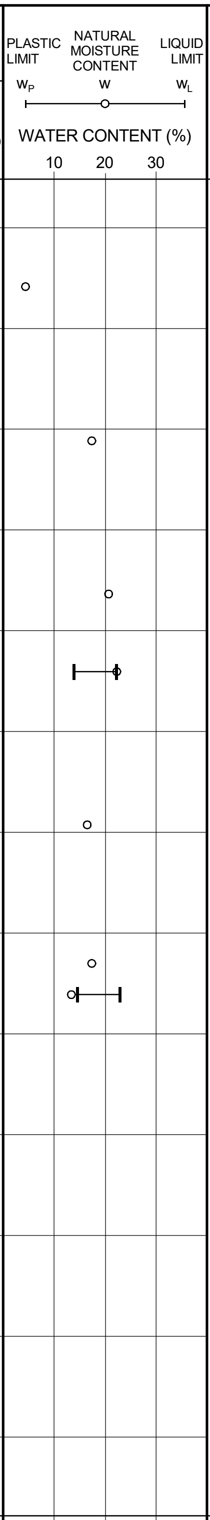
PROJECT 1662333
G.W.P. 2002-13-00
DIST Central HWY QEW
DATUM Geodetic
LOCATION N 4824255.4; E 295917.6 MTM NAD 83 ZONE 10 (LAT. 43.558183; LONG. -79.609946)
BOREHOLE TYPE CME 55, 210 mm O.D., Hollow Stem Augers
DATE July 27, 2018
ORIGINATED BY ACM
COMPILED BY CC
CHECKED BY SMM

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
95.5	GROUND SURFACE					
0.0	TOPSOIL					
0.2	Silt and sand, trace to some clay, trace gravel (FILL) Loose Brown Moist - Trace organics and rootlets to 0.6 m	1	SS	5		
94.8 0.7	SILT and SAND, trace to some clay, trace gravel Compact Brown to grey at 3.2 m Moist - Pockets of sand present between 1.6 m and 2.1 m - Clayey silt interlayer between 3.0 m and 3.2 m	2	SS	7		
		3	SS	25		
		4	SS	34		1 50 42 7
		5	SS	25		
91.8 3.7	CLAYEY SILT, trace sand, trace gravel, some silt and sand interlayers Stiff Grey Moist	6	SS	17		
		7	SS	9		1 13 60 26
89.9 5.6	SILT, trace to some sand, trace to some clay Compact Grey Moist to wet - Pocket of sand from 6.1 m to 6.2 m	8	SS	19		0 11 82 7
87.9 7.6	SILTY CLAY Stiff Grey	9A	SS	13	∇	
87.4 8.1	Moist CLAYEY SILT, some gravel, trace sand (TILL) Very stiff Grey Moist	9B				
86.5 9.0	SHALE (BEDROCK) Grey Bedrock cored from a depth of 10.0 m to 13.3 m. For bedrock coring details, refer to Record of Drillhole SWME-4.	10	SS	50/0.05		
		11	SS	50/0.08		
		1	RC	REC 100%		RQD = 80%
		2	RC	REC 100%		RQD = 84%
		3	RC	REC 100%		RQD = 94%
82.2 13.3	END OF BOREHOLE NOTE: 1. Water level in open borehole at a depth of 7.6 m (Elev. 87.9) below ground surface prior to rock coring.					

ELEVATION SCALE: 95, 94, 93, 92, 91, 90, 89, 88, 87, 86, 85, 84, 83

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED: 20 40 60 80 100

UNIT WEIGHT γ kN/m³



$+^3, \times^3$: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

PROJECT: 1662333

RECORD OF DRILLHOLE: SWME-4

SHEET 1 OF 1

LOCATION: N 4824255.4 ;E 295917.6

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: July 27, 2018

DRILL RIG: CME 55

DRILLING CONTRACTOR: Davis Drilling

DATUM: Geodetic

JN - Joint
FLT - Fault
SHR- Shear
VN - Vein
CJ - Conjugate

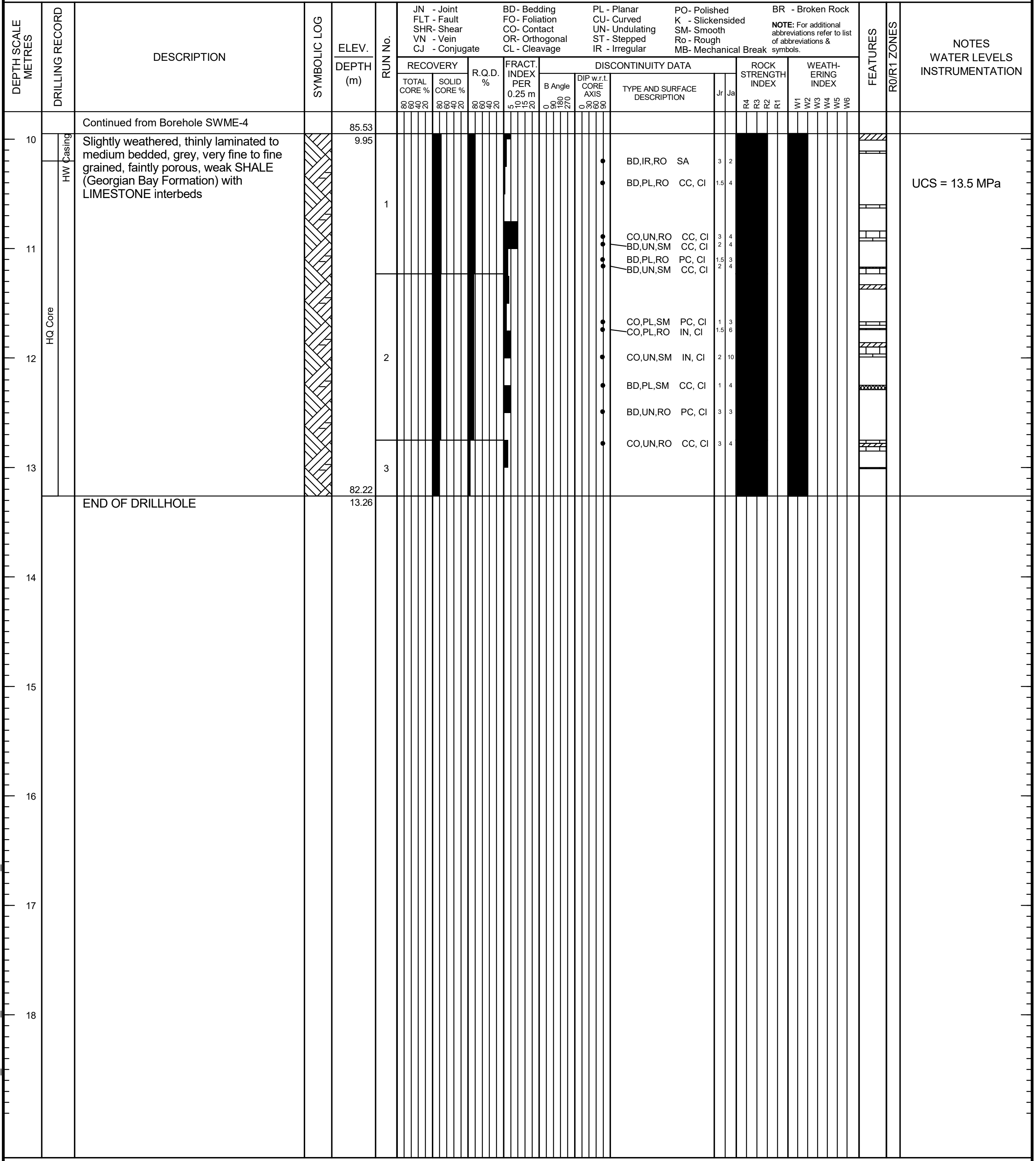
BD- Bedding
FO- Foliation
CO- Contact
OR- Orthogonal
CL - Cleavage

PL - Planar
CU- Curved
UN- Undulating
ST - Stepped
IR - Irregular

PO- Polished
K - Slickensided
SM- Smooth
Ro - Rough
MB- Mechanical Break

BR - Broken Rock

NOTE: For additional abbreviations refer to list of abbreviations & symbols.



DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	NOTES WATER LEVELS INSTRUMENTATION
		Continued from Borehole SWME-4	85.53		
10	HW Casing HQ Core	Slightly weathered, thinly laminated to medium bedded, grey, very fine to fine grained, faintly porous, weak SHALE (Georgian Bay Formation) with LIMESTONE interbeds	9.95	1 2 3	UCS = 13.5 MPa
			82.22 13.26		
		END OF DRILLHOLE			

TYPE AND SURFACE DESCRIPTION	Jr	Ja
BD,IR,RO SA	3	2
BD,PL,RO CC, CI	1.5	4
CO,UN,RO CC, CI	3	4
BD,UN,SM CC, CI	2	4
BD,PL,RO PC, CI	1.5	3
BD,UN,SM CC, CI	2	4
CO,PL,SM PC, CI	1	3
CO,PL,RO IN, CI	1.5	6
CO,UN,SM IN, CI	2	10
BD,PL,SM CC, CI	1	4
BD,UN,RO PC, CI	3	3
CO,UN,RO CC, CI	3	4

FEATURES LEGEND

▨	BROKEN CORE
▩	CLAY SEAM
▦	LIMESTONE
■	LOST CORE

DEPTH SCALE
1 : 50

GOLDER

LOGGED: ACM
CHECKED: AB

RECORD OF BOREHOLE No AR-1

SHEET 1 OF 1

METRIC

PROJECT 1662333

G.W.P. 2002-13-00

DIST Central HWY QEW

DATUM Geodetic

LOCATION N 4824236.4; E 295944.3 MTM NAD 83 ZONE 10 (LAT. 43.558012; LONG. -79.609616)

BOREHOLE TYPE CME 55, 114 mm I.D., Hollow Stem Augers

DATE July 30, 2018

ORIGINATED BY ACM

COMPILED BY CC

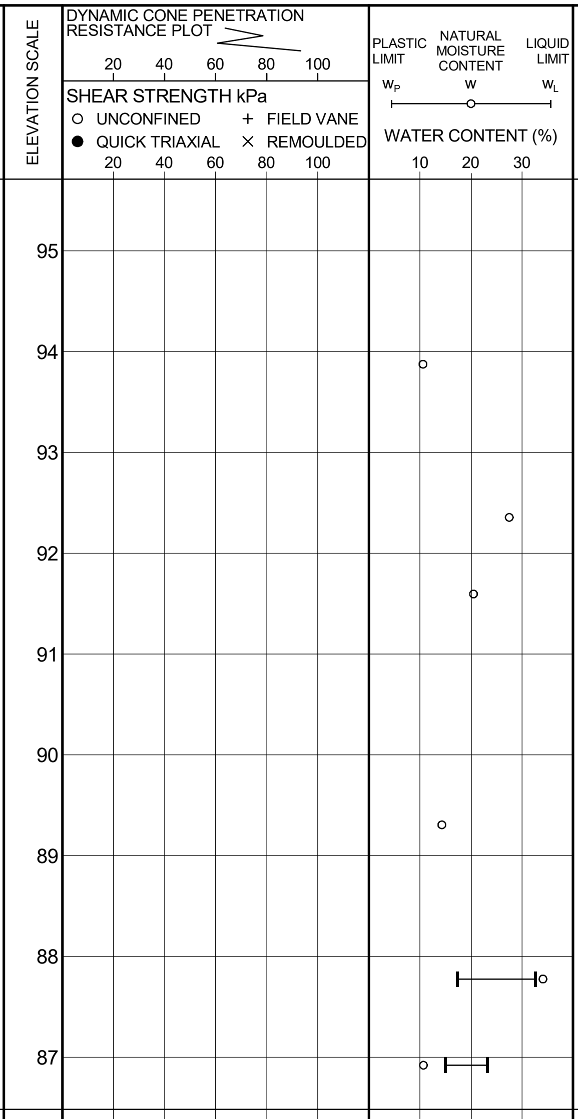
CHECKED BY SMM

ELEV DEPTH	SOIL PROFILE: DESCRIPTION	SAMPLES: NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
95.7	GROUND SURFACE									
0.0	TOPSOIL (700 mm)	1	SS	9						
95.0 0.7	Sand, some silt to silt and sand, trace clay (FILL) Loose Brown Moist	2	SS	7						
94.3 1.4	SAND, some silt SILT and SAND, trace clay Loose to very dense Brown Moist - Oxidation staining from 2.3 m to 3.7 m - Wet from 3.0 m to 3.7 m	3	SS	7			0	81	16	3
		4	SS	6						
		5	SS	5						
		6	SS	56			0	35	61	4
		7	SS	63						
90.4 5.3	SILT, trace to some sand, trace to some clay Very dense Brown to grey Moist	8	SS	56			0	11	82	7
88.5 7.2	CLAYEY SILT, trace sand Very soft Grey Moist to wet	9	SS	2			0	1	55	44
87.0	Sandy CLAYEY SILT, trace to some gravel, some shale fragments (RESIDUAL SOIL) Hard Grey Moist to wet	10A 10B	SS	50/0.05			7	29	46	18
86.5 9.2	SHALE (BEDROCK) Grey	11	SS	50/0.08						
	END OF BOREHOLE SPLIT-SPOON REFUSAL									

REMARKS & GRAIN SIZE DISTRIBUTION (%)

NOTES:

1. Water level at a depth of approximate 3.7 m below ground surface (Elev. 92.0 m) upon completion of drilling.



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

$\circ^{3\%}$ STRAIN AT FAILURE

PROJECT 1662333

RECORD OF BOREHOLE No AR-2

SHEET 1 OF 1

METRIC

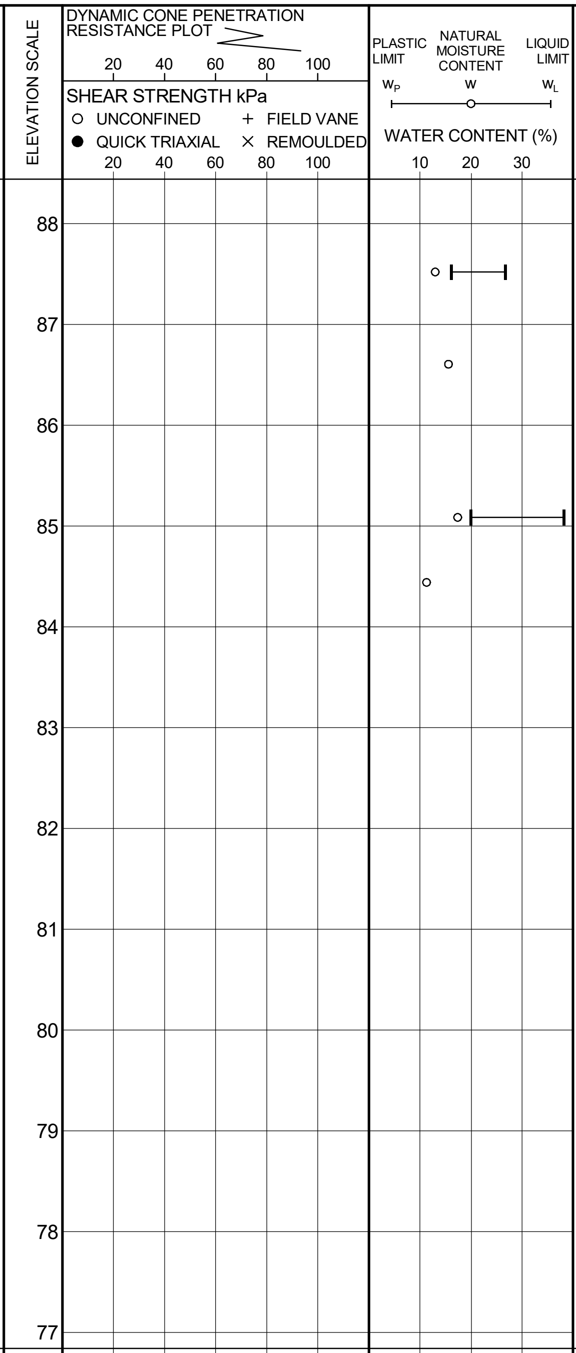
G.W.P. 2002-13-00
LOCATION N 4824172.2; E 295921.4 MTM NAD 83 ZONE 10 (LAT. 43.557434; LONG. -79.609899)
ORIGINATED BY ACM
DIST Central HWY QEW
BOREHOLE TYPE CME 75, 114 mm I.D. Hollow Stem Augers, HQ Casing
COMPILED BY CC
DATUM Geodetic
DATE July 30, 2018
CHECKED BY SMM

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
88.4	GROUND SURFACE
0.0	TOPSOIL (600 mm)
87.8 0.6	CLAYEY SILT, some sand, some gravel, trace rootlets, shale fragments (TILL) Stiff Brown to grey Moist
87.3 1.1	SILT and SAND, trace to some clay, trace to some gravel, clayey silt pockets, shale fragments (TILL) Loose to compact Brown Moist
86.2 2.2	Sandy SILTY CLAY, trace to some gravel, trace shale fragments (TILL) Stiff Brown grey with oxidation staining Moist
84.6 3.8	CLAYEY SILT, some sand, some shale fragments (RESIDUAL SOIL) Hard Brown grey Moist
83.8 4.6	SHALE (BEDROCK) Grey Bedrock cored from a depth of 4.6 m to 11.6 m For bedrock coring details, refer to Record of Drillhole AR-2
76.8 11.6	END OF BOREHOLE NOTES: 1. Borehole dry prior to rock coring.

SAMPLES

NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	9			
2A 2B	SS	14			
3	SS	6			6 56 31 7
4	SS	11			
5	SS	13			7 22 44 27
6	SS	50/0.08			
7	SS	50/0.13			
1	RC	REC 100%			RQD = 18%
2	RC	REC 100%			RQD = 86%
3	RC	REC 100%			RQD = 97%
4	RC	REC 100%			RQD = 93%
5	RC	REC 100%			RQD = 57%



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

$\circ^{3\%}$ STRAIN AT FAILURE

GTA-MTO 001 S:\CLIENTS\MTO\QEW-CREDIT_RIVER\02_DATA\GINT\QEW-CREDIT_RIVER.GPJ GAL-GTA.GDT 1/9/19

PROJECT: 1662333

RECORD OF DRILLHOLE: AR-2

SHEET 1 OF 1

LOCATION: N 4824172.2 ;E 295921.4

DRILLING DATE: July 30, 2018

DATUM: Geodetic

DRILL RIG: CME 75

INCLINATION: -90° AZIMUTH: ---

DRILLING CONTRACTOR: Davis Drilling

JN - Joint
FLT - Fault
SHR- Shear
VN - Vein
CJ - Conjugate

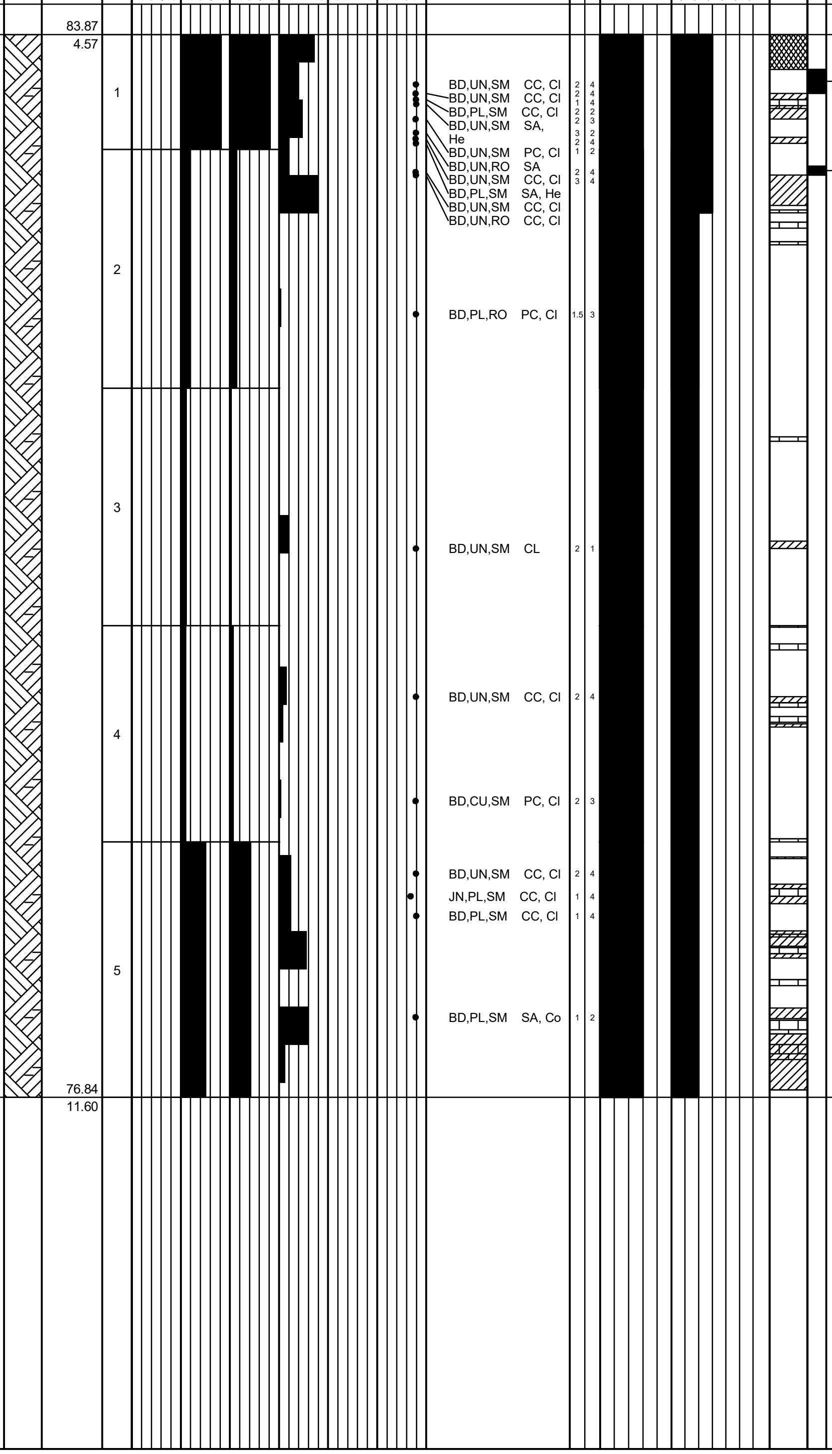
BD- Bedding
FO- Foliation
CO- Contact
OR- Orthogonal
CL - Cleavage

PL - Planar
CU- Curved
UN- Undulating
ST - Stepped
IR - Irregular

PO- Polished
K - Slickensided
SM- Smooth
Ro - Rough
MB- Mechanical Break

BR - Broken Rock
NOTE: For additional abbreviations refer to list of abbreviations & symbols.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. DEPTH (m)	RUN No.
		Continued from Borehole AR-2	83.87	
	HQ Core	Moderately to slightly weathered, thinly laminated to medium bedded, grey, fine grained, faintly porous, weak SHALE (Georgian Bay Formation) with LIMESTONE interbeds	4.57	1 2 3 4 5
			76.84 11.60	
		END OF DRILLHOLE		



TYPE AND SURFACE DESCRIPTION	Jr	Ja
BD,UN,SM CC, Cl	2	4
BD,UN,SM CC, Cl	2	4
BD,PL,SM CC, Cl	1	4
BD,UN,SM SA, He	2	2
BD,UN,SM PC, Cl	2	3
BD,UN,RO SA	3	2
BD,UN,SM CC, Cl	2	4
BD,PL,SM SA, He	1	2
BD,UN,SM CC, Cl	2	4
BD,UN,RO CC, Cl	3	4
BD,PL,RO PC, Cl	1.5	3
BD,UN,SM CL	2	1
BD,UN,SM CC, Cl	2	4
BD,CU,SM PC, Cl	2	3
BD,UN,SM CC, Cl	2	4
JN,PL,SM CC, Cl	1	4
BD,PL,SM CC, Cl	1	4
BD,PL,SM SA, Co	1	2

NOTES WATER LEVELS INSTRUMENTATION

UCS = 9.1 MPa

UCS = 11.5 MPa

FEATURES LEGEND

▨	BROKEN CORE
▩	CLAY SEAM
▦	LIMESTONE
■	LOST CORE

DEPTH SCALE
1 : 50

GOLDER

LOGGED: ACM
CHECKED: AB

GOLDER

RECORD OF BOREHOLE No CRB-5

SHEET 1 OF 2

METRIC

PROJECT 1662333

G.W.P. 2002-13-00

DIST Central HWY QEW

DATUM Geodetic

LOCATION N 4824128.9; E 295914.2 MTM NAD 83 ZONE 10 (LAT. 43.557044; LONG. -79.609986)

BOREHOLE TYPE CME 55, 203 mm O.D., 108 mm I.D. Hollow Stem Augers (Auto Hammer)

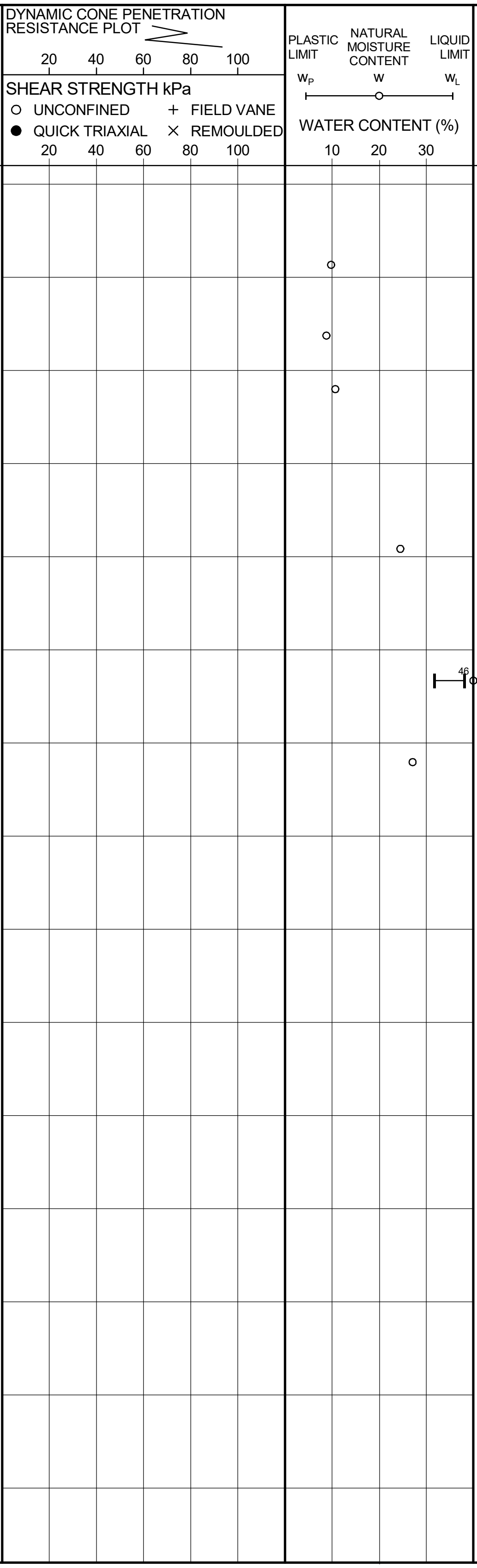
DATE February 13, 2018

ORIGINATED BY JL

COMPILED BY KN

CHECKED BY SMM

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES				GR	SA	SI	CL
79.2	GROUND SURFACE										
0.0	Silty sand, some gravel, contains rootlets (FILL) Loose Brown Wet	1	SS	6		79					
78.5 0.7	Clayey silt with sand, trace to some gravel, contains rootlets / organics. contains clayey silt pockets and shale fragments (FILL) Very stiff Brown and grey Moist/frozen	2	SS	25		78					
		3	SS	23		77					
76.8 2.4	Silty SAND to SILT and SAND, trace gravel, trace clay, trace organics, contains clayey silt pockets and rootlets Very loose to loose Brown to grey Moist to wet	4A 4B	SS	6							
		5	SS	4		76					
		6	SS	WH	∇	75	OC= 1.1%	5	60	29	6
74.5 4.7	ORGANIC CLAYEY SILT with SAND, trace gravel Very soft to firm Brown Moist	7A 7B	SS	1		74					
		8A 8B	SS	5			OC= 7.1%	1	41	50	8
73.5 5.7	Silty SAND, trace to some gravel, trace clay, contains clayey silt pockets, contains wood fragments Loose Grey Wet	9	SS	4		73		8	68	20	4
		10A									
72.0 7.2	SHALE (BEDROCK) Grey Bedrock cored from a depth of 7.2 m to 15.5 m For bedrock coring details, refer to Record of Drillhole CRB-5	10B	SS	73		72					
		1	RC	REC 53%				RQD = 0%			
		2	RC	REC 51%		71		RQD = 13%			
		3	RC	REC 100%		70 69		RQD = 99%			
		4	RC	REC 100%		68		RQD = 73%			
		5	RC	REC 97%		67 66		RQD = 78%			
		6	RC	REC 100%		65		RQD = 99%			



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

$\bigcirc^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No CRB-5

SHEET 2 OF 2

METRIC

PROJECT 1662333			
G.W.P. 2002-13-00	LOCATION N 4824128.9; E 295914.2 MTM NAD 83 ZONE 10 (LAT. 43.557044; LONG. -79.609986)	ORIGINATED BY JL
DIST Central HWY QEW	BOREHOLE TYPE CME 55, 203 mm O.D., 108 mm I.D. Hollow Stem Augers (Auto Hammer)	COMPILED BY KN
DATUM Geodetic	DATE February 13, 2018	CHECKED BY SMM

SOIL PROFILE			SAMPLES					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/m^3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE				
	--- CONTINUED FROM PREVIOUS PAGE ---										
	SHALE (BEDROCK) Grey	▨	6	RC	REC 100%		64				RQD = 99%
63.7 15.5	END OF BOREHOLE										

NOTES:

1. Water level encountered during drilling at a depth of about 3.7 m (Elev. 75.5 m)below ground surface.

2. Water level measured in open borehole at a depth of about 4.3 m (Elev. 74.9 m) below ground surface prior to rock coring.

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

$\bigcirc^{3\%}$ STRAIN AT FAILURE

PROJECT: 1662333

RECORD OF DRILLHOLE: CRB-5

SHEET 1 OF 1

LOCATION: N 4824128.9 ;E 295914.2

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: February 13, 2018

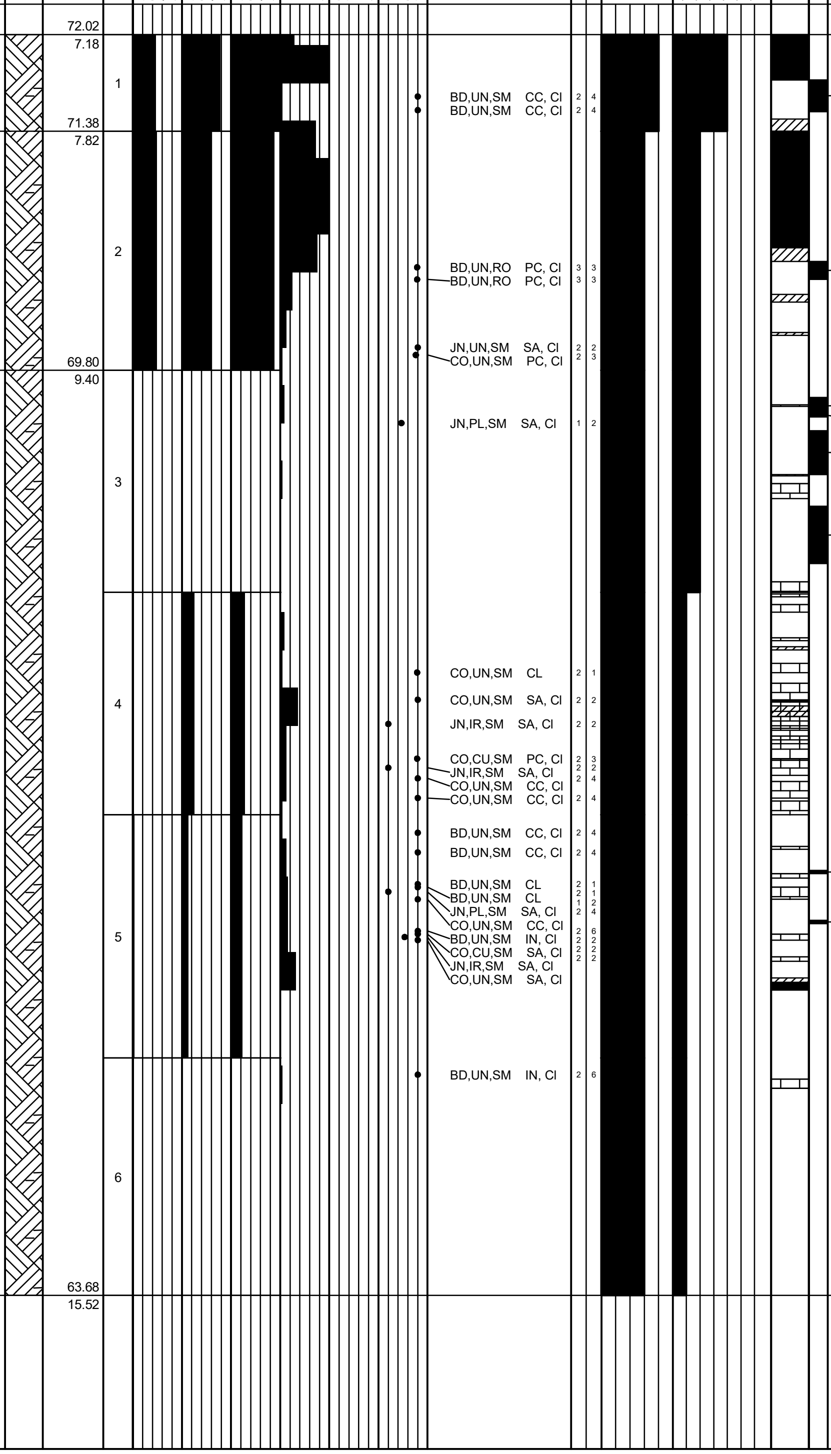
DRILL RIG: CME 55 Track

DRILLING CONTRACTOR: Geo-Environmental Drilling

DATUM: Geodetic

JN - Joint, FLT - Fault, SHR- Shear, VN - Vein, CJ - Conjugate
BD- Bedding, FO- Foliation, CO- Contact, OR- Orthogonal, CL - Cleavage
PL - Planar, CU- Curved, UN- Undulating, ST - Stepped, IR - Irregular
PO- Polished, K - Slickensided, SM- Smooth, Ro - Rough, MB- Mechanical Break
BR - Broken Rock

NOTE: For additional abbreviations refer to list of abbreviations & symbols.



DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. / DEPTH (m)	RUN No.
		Continued from Borehole CRB-5	72.02	
	HQ Core	Highly weathered, thinly laminated to medium bedded, brown to grey, very fine to fine grained, faintly porous, very weak SHALE (Georgian Bay Formation)	7.18	1
8		Slightly weathered to fresh, thinly laminated to medium bedded, grey, very fine to fine grained, faintly porous, weak SHALE (Georgian Bay Formation) with LIMESTONE interbeds	71.38 7.82	2
9, 10, 11, 12, 13, 14, 15		Slightly weathered to fresh, thinly to medium bedded, grey, fine grained, faintly porous, very weak to weak SHALE (Georgian Bay Formation) with LIMESTONE interbeds	69.80 9.40	3, 4, 5, 6
16		END OF DRILLHOLE	63.68 15.52	

TYPE AND SURFACE DESCRIPTION	Jr	Ja
BD,UN,SM CC, CI	2	4
BD,UN,SM CC, CI	2	4
BD,UN,RO PC, CI	3	3
BD,UN,RO PC, CI	3	3
JN,UN,SM SA, CI	2	2
CO,UN,SM PC, CI	2	3
JN,PL,SM SA, CI	1	2
CO,UN,SM CL	2	1
CO,UN,SM SA, CI	2	2
JN,IR,SM SA, CI	2	2
CO,CU,SM PC, CI	2	3
JN,IR,SM SA, CI	2	2
CO,UN,SM CC, CI	2	4
CO,UN,SM CC, CI	2	4
BD,UN,SM CC, CI	2	4
BD,UN,SM CC, CI	2	4
BD,UN,SM CL	2	1
BD,UN,SM CL	2	1
JN,PL,SM SA, CI	1	2
CO,UN,SM CC, CI	2	4
BD,UN,SM IN, CI	2	6
CO,CU,SM SA, CI	2	2
JN,IR,SM SA, CI	2	2
CO,UN,SM SA, CI	2	2
BD,UN,SM IN, CI	2	6

NOTES WATER LEVELS INSTRUMENTATION

PLT(D) = 0.76 MPa
PLT(A) = 0.37 MPa

PLT(A) = 1.45 MPa

PLT(D) = 0.07 MPa

UC = 18.6 MPa

PLT(D) = 0.52 MPa
PLT(A) = 0.27 MPa

FEATURES LEGEND

▨ BROKEN CORE ▩ CLAY SEAM ▦ LIMESTONE ■ LOST CORE

DEPTH SCALE
1 : 50

GOLDER

LOGGED: JL
CHECKED: DM

GTA-RCK 054 S:\CLIENTS\MTO\QEW-CREDIT_RIVER\02_DATA\GINT\QEW-CREDIT_RIVER.GPJ GAL-MISS.GDT 2/12/19

RECORD OF BOREHOLE No CRB-5A

SHEET 1 OF 2

METRIC

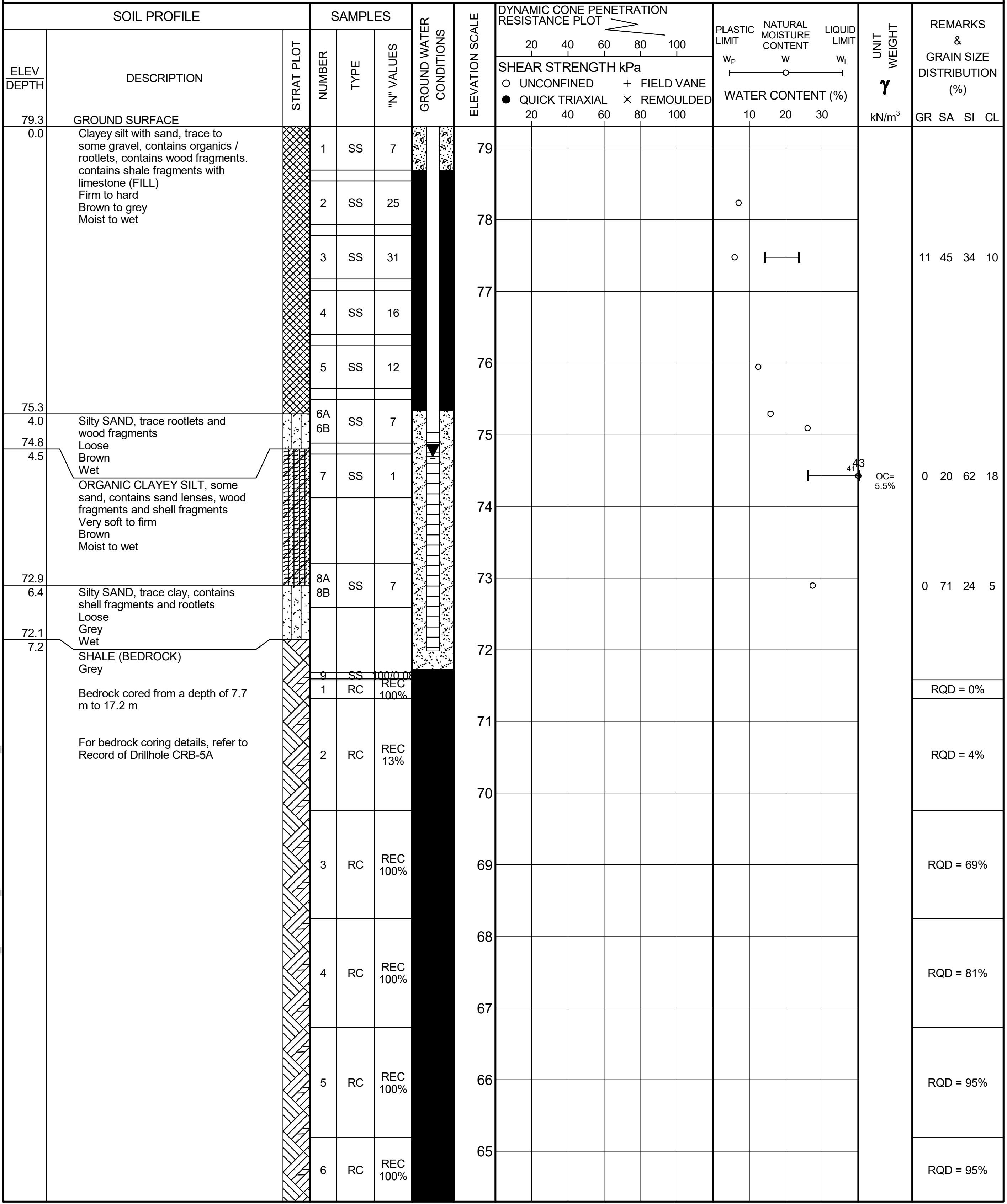
PROJECT 1662333
G.W.P. 2002-13-00
DIST Central HWY QEW
DATUM Geodetic
LOCATION N 4824130.9; E 295910.6 MTM NAD 83 ZONE 10 (LAT. 43.557062; LONG. -79.610032)
BOREHOLE TYPE CME 55, 159 mm O.D., 70 mm I.D. Hollow Stem Augers (Auto Hammer)
DATE February 15 and 16, 2018
ORIGINATED BY JL
COMPILED BY KN
CHECKED BY SMM

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
79.3	GROUND SURFACE
0.0	Clayey silt with sand, trace to some gravel, contains organics / rootlets, contains wood fragments. contains shale fragments with limestone (FILL) Firm to hard Brown to grey Moist to wet
75.3 / 4.0	Silty SAND, trace rootlets and wood fragments Loose Brown Wet
74.8 / 4.5	ORGANIC CLAYEY SILT, some sand, contains sand lenses, wood fragments and shell fragments Very soft to firm Brown Moist to wet
72.9 / 6.4	Silty SAND, trace clay, contains shell fragments and rootlets Loose Grey Wet
72.1 / 7.2	SHALE (BEDROCK) Grey Bedrock cored from a depth of 7.7 m to 17.2 m For bedrock coring details, refer to Record of Drillhole CRB-5A

SAMPLES

NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
1	SS	7					
2	SS	25					
3	SS	31		11	45	34	10
4	SS	16					
5	SS	12					
6A 6B	SS	7					
7	SS	1	OC= 5.5%	0	20	62	18
8A 8B	SS	7		0	71	24	5
9	SS	100/0.08					
1	RC	REC 100%		RQD = 0%			
2	RC	REC 13%		RQD = 4%			
3	RC	REC 100%		RQD = 69%			
4	RC	REC 100%		RQD = 81%			
5	RC	REC 100%		RQD = 95%			
6	RC	REC 100%		RQD = 95%			



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+³, ×³: Numbers refer to Sensitivity

○³% STRAIN AT FAILURE

RECORD OF BOREHOLE No CRB-5A

SHEET 2 OF 2

METRIC

PROJECT	1662333				
G.W.P.	2002-13-00	LOCATION	N 4824130.9; E 295910.6 MTM NAD 83 ZONE 10 (LAT. 43.557062; LONG. -79.610032)	ORIGINATED BY	JL
DIST	Central HWY QEW	BOREHOLE TYPE	CME 55, 159 mm O.D., 70 mm I.D. Hollow Stem Augers (Auto Hammer)	COMPILED BY	KN
DATUM	Geodetic	DATE	February 15 and 16, 2018	CHECKED BY	SMM

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 × 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 ---										
	SHALE (BEDROCK) Grey Bedrock cored from a depth of 7.7 m to 17.2 m	▨	6	RC	REC 100%	■	64				RQD = 95%
	For bedrock coring details, refer to Record of Drillhole CRB-5A	▨	7	RC	REC 100%	■	63				RQD = 100%
62.1 17.2	END OF BOREHOLE										

NOTES:

1. Water level encountered during drilling at a depth of about 4.0 m (Elev. 75.3 m) below ground surface.

2. Water level measured in open borehole at a depth of about 3.6 m (Elev. 75.7 m) below ground surface prior to rock coring.

3. Groundwater level measurements in piezometer:

Date	Depth (m)	Elev. (m)
12/03/18	1.6	77.7
30/04/18	4.0	75.3
06/11/18	4.6	74.7

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

$\circ^{3\%}$ STRAIN AT FAILURE

PROJECT: 1662333

RECORD OF DRILLHOLE: CRB-5A

SHEET 1 OF 2

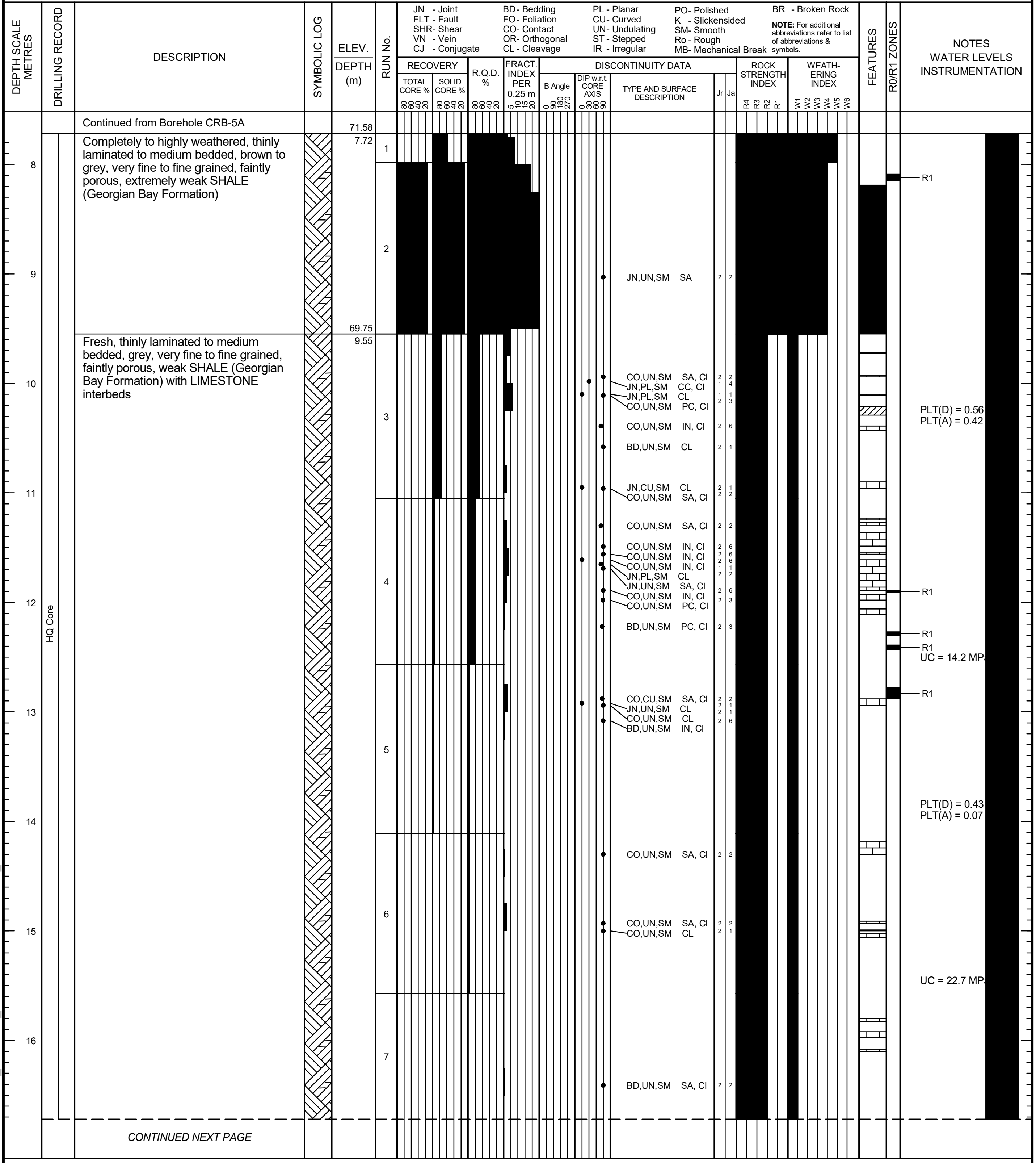
LOCATION: N 4824130.9 ;E 295910.6
INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: February 15 and 16, 2018
DRILL RIG: CME 55 Track
DRILLING CONTRACTOR: Geo-Environmental Drilling

DATUM: Geodetic

JN - Joint, FLT - Fault, SHR- Shear, VN - Vein, CJ - Conjugate
BD- Bedding, FO- Foliation, CO- Contact, OR- Orthogonal, CL - Cleavage
PL - Planar, CU- Curved, UN- Undulating, ST - Stepped, IR - Irregular
PO- Polished, K - Slickensided, SM- Smooth, Ro - Rough, MB- Mechanical Break
BR - Broken Rock

NOTE: For additional abbreviations refer to list of abbreviations & symbols.



Continued from Borehole CRB-5A

DESCRIPTION	ELEV. / DEPTH (m)	RUN No.
Completely to highly weathered, thinly laminated to medium bedded, brown to grey, very fine to fine grained, faintly porous, extremely weak SHALE (Georgian Bay Formation)	71.58 7.72	1 2
Fresh, thinly laminated to medium bedded, grey, very fine to fine grained, faintly porous, weak SHALE (Georgian Bay Formation) with LIMESTONE interbeds	69.75 9.55	3 4 5 6 7

TYPE AND SURFACE DESCRIPTION	Jr	Ja
JN,UN,SM SA	2	2
CO,UN,SM SA, CI	2	2
JN,PL,SM CC, CI	1	4
JN,PL,SM CL	1	1
CO,UN,SM PC, CI	2	3
CO,UN,SM IN, CI	2	6
BD,UN,SM CL	2	1
JN,CU,SM CL	2	1
CO,UN,SM SA, CI	2	2
CO,UN,SM SA, CI	2	2
CO,UN,SM IN, CI	2	6
CO,UN,SM IN, CI	2	6
CO,UN,SM IN, CI	2	6
JN,PL,SM CL	1	1
JN,UN,SM SA, CI	2	2
CO,UN,SM IN, CI	2	6
CO,UN,SM PC, CI	2	3
BD,UN,SM PC, CI	2	3
CO,CU,SM SA, CI	2	2
JN,UN,SM CL	2	1
CO,UN,SM CL	2	1
BD,UN,SM IN, CI	2	6
CO,UN,SM SA, CI	2	2
CO,UN,SM SA, CI	2	2
CO,UN,SM CL	2	1
BD,UN,SM SA, CI	2	2

PLT(D) = 0.56
PLT(A) = 0.42

UC = 14.2 MP

PLT(D) = 0.43
PLT(A) = 0.07

UC = 22.7 MP

CONTINUED NEXT PAGE

FEATURES LEGEND

▨ BROKEN CORE	▩ CLAY SEAM	▦ LIMESTONE	■ LOST CORE

DEPTH SCALE
1 : 50

GOLDER

LOGGED: JL
CHECKED: DM

PROJECT: 1662333

RECORD OF DRILLHOLE: CRB-5A

SHEET 2 OF 2

LOCATION: N 4824130.9 ;E 295910.6

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: February 15 and 16, 2018

DRILL RIG: CME 55 Track

DRILLING CONTRACTOR: Geo-Environmental Drilling

DATUM: Geodetic

JN - Joint
FLT - Fault
SHR- Shear
VN - Vein
CJ - Conjugate
BD- Bedding
FO- Foliation
CO- Contact
OR- Orthogonal
CL - Cleavage
PL - Planar
CU- Curved
UN- Undulating
ST - Stepped
IR - Irregular
PO- Polished
K - Slickensided
SM- Smooth
Ro - Rough
MB- Mechanical Break
BR - Broken Rock

NOTE: For additional abbreviations refer to list of abbreviations & symbols.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	RECOVERY TOTAL CORE % 80 60 40 20	RECOVERY SOLID CORE % 80 60 40 20	R.Q.D. % 80 60 40 20	FRACT. INDEX PER 0.25 m 5 10 15 20	DISCONTINUITY DATA B Angle 0 90 180 270	DIP w.r.t. CORE AXIS 0 30 60 90	TYPE AND SURFACE DESCRIPTION	Jr	Ja	ROCK STRENGTH INDEX R4 R3 R2 R1	WEATHERING INDEX W1 W2 W3 W4 W5 W6	FEATURES	R0/R1 ZONES	NOTES WATER LEVELS INSTRUMENTATION
		--- *CONTINUED FROM PREVIOUS PAGE* ---																	
17	HQ Core		▧	62.14	7										■	■			PLT(D) = 0.82 PLT(A) = 0.24
		END OF DRILLHOLE		17.16															
18																			
19																			
20																			
21																			
22																			
23																			
24																			
25																			

FEATURES LEGEND

▨ BROKEN CORE	▩ CLAY SEAM	▦ LIMESTONE	■ LOST CORE

DEPTH SCALE
1 : 50

GOLDER

LOGGED: JL
CHECKED: DM

RECORD OF BOREHOLE No CRB-6

SHEET 1 OF 2

METRIC

PROJECT 1662333
G.W.P. 2002-13-00
LOCATION N 4824196.7; E 295929.5 MTM NAD 83 ZONE 10 (LAT. 43.557650; LONG. -79.609801)
ORIGINATED BY JL
DIST Central HWY QEW
BOREHOLE TYPE CME 850, 210 mm O.D. Hollow Stem Augers, HQ Casing (Auto Hammer)
COMPILED BY MPL
DATUM Geodetic
DATE October 18-20, 2017
CHECKED BY SMM

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
91.7	GROUND SURFACE				
0.0	Silty sand, trace to some gravel, trace clay, contains brick fragments (FILL) Loose to compact Brown Moist	1	SS	11	
		2	SS	5	7 67 24 2
90.0 1.7	Sandy CLAYEY SILT, trace to some gravel Firm to stiff brown Moist to wet - Mottled brown-grey below a depth of about 2.3 m - Becoming gravelly at a depth of about 3.7 m - Auger grinding at a depth of about 3.7 m	3A 3B	SS	5	
		4	SS	7	
		5	SS	9	
		6	SS	5	10 26 44 20
87.3 4.4	Sandy CLAYEY SILT, some shale fragments (RESIDUAL SOIL) Hard Grey Moist	7	SS	50/0.13	
86.9 4.8	SHALE (BEDROCK) Grey Bedrock cored from a depth of 5.1 m to 13.3 m For bedrock coring details, refer to Record of Drillhole CRB-6	1	RC	REC 86%	RQD = 47%
		2	RC	REC 100%	RQD = 95%
		3	RC	REC 100%	RQD = 100%
		4	RC	REC 100%	RQD = 95%
		5	RC	REC 100%	RQD = 100%
		6	RC	REC 96%	RQD = 96%
78.4 13.3	END OF BOREHOLE				

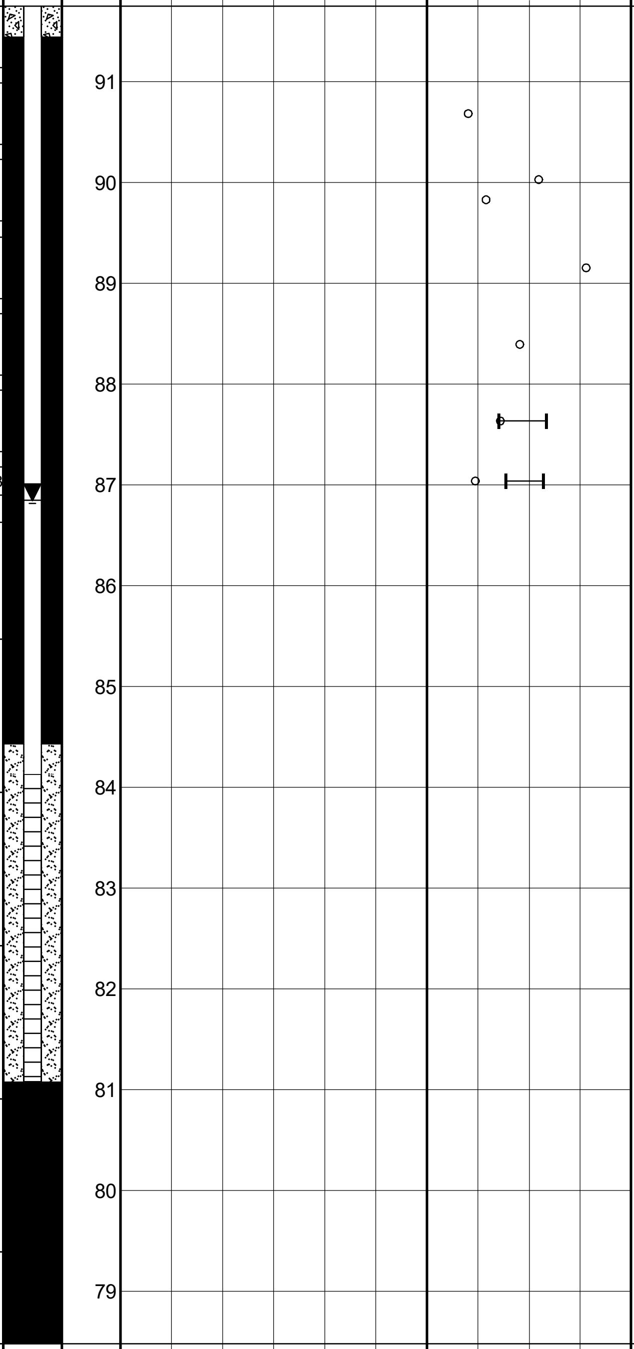
STRAT PLOT | GROUND WATER CONDITIONS | ELEVATION SCALE | UNIT WEIGHT γ kN/m³

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



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

RECORD OF BOREHOLE No CRB-6

SHEET 2 OF 2

METRIC

PROJECT 1662333

G.W.P. 2002-13-00

LOCATION N 4824196.7; E 295929.5 MTM NAD 83 ZONE 10 (LAT. 43.557650; LONG. -79.609801)

ORIGINATED BY JL

DIST Central HWY QEW

BOREHOLE TYPE CME 850, 210 mm O.D. Hollow Stem Augers, HQ Casing (Auto Hammer)

COMPILED BY MPL

DATUM Geodetic

DATE October 18-20, 2017

CHECKED BY SMM

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 × 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 --- *CONTINUED FROM PREVIOUS PAGE* ---	STRAT PLOT	NUMBER	TYPE	"N" VALUES						

NOTES:

1. Borehole dry prior to rock coring.

2. Water level measured in standpipe piezometer:

Date	Depth (m)	Elev. (m)
12/11/17	5.6	86.1
12/03/18	5.0	86.7
30/04/18	4.9	86.8
06/11/18	4.9	86.8

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

$\bigcirc^{3\%}$ STRAIN AT FAILURE

PROJECT: 1662333

RECORD OF DRILLHOLE: CRB-6

SHEET 1 OF 1

LOCATION: N 4824196.7 ;E 295929.5

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: October 18-20, 2017

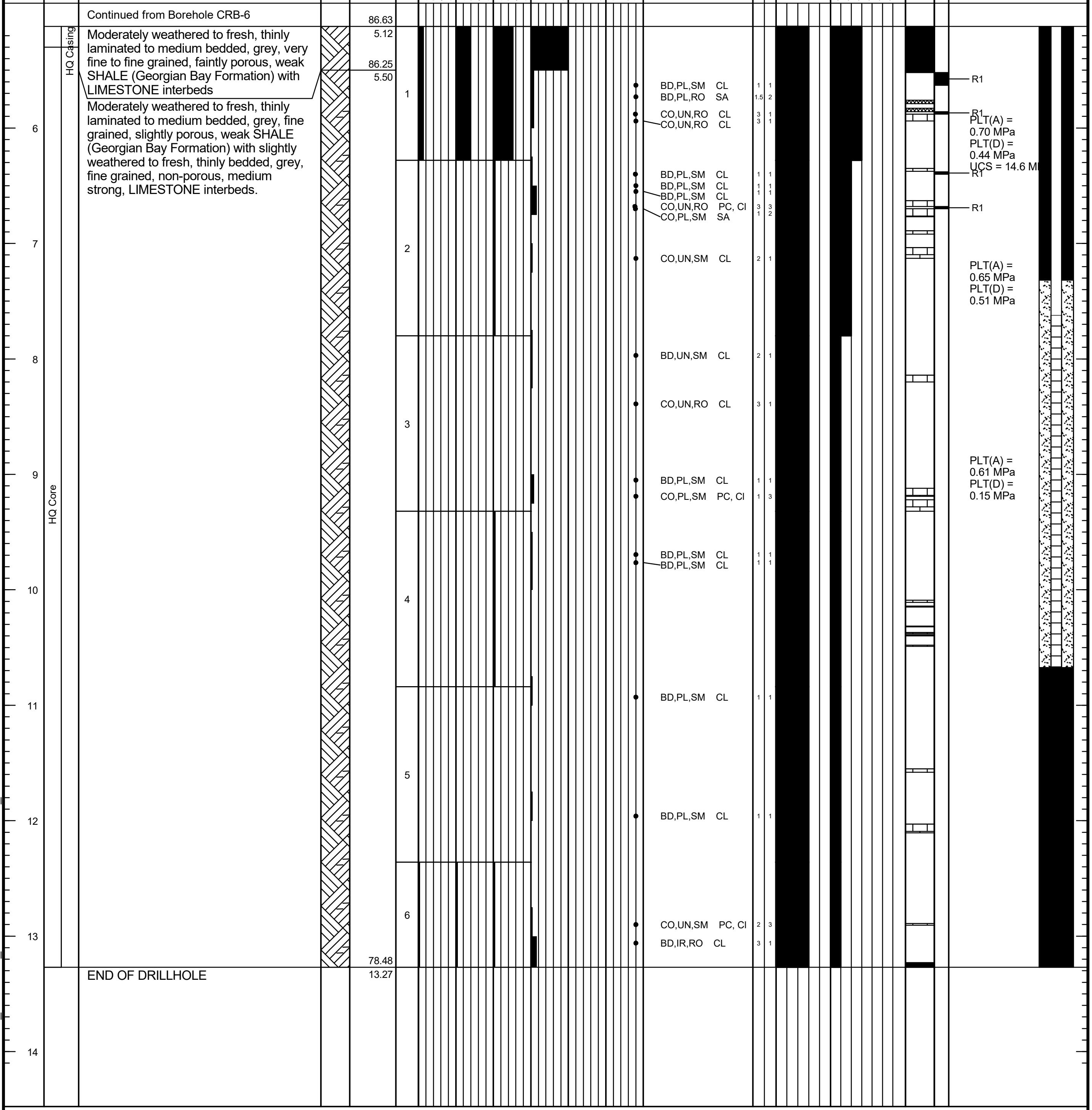
DRILL RIG: CME 850 Track

DRILLING CONTRACTOR: Aardvark Drilling

DATUM: Geodetic

JN - Joint	BD - Bedding	PL - Planar	PO - Polished	BR - Broken Rock
FLT - Fault	FO - Foliation	CU - Curved	K - Slickensided	**NOTE:** For additional abbreviations refer to list of abbreviations & symbols.
SHR - Shear	CO - Contact	UN - Undulating	SM - Smooth	
VN - Vein	OR - Orthogonal	ST - Stepped	Ro - Rough	
CJ - Conjugate	CL - Cleavage	IR - Irregular	MB - Mechanical Break	

DEPTH SCALE METRES | DRILLING RECORD | DESCRIPTION | SYMBOLIC LOG | ELEV. DEPTH (m) | RUN No. | RECOVERY (TOTAL CORE % / SOLID CORE %) | R.Q.D. % | FRACT. INDEX PER 0.25 m | DISCONTINUITY DATA (B Angle; DIP w.r.t. CORE AXIS; TYPE AND SURFACE DESCRIPTION; Jr; Ja) | ROCK STRENGTH INDEX (R4 R3 R2 R1) | WEATHERING INDEX (W1 W2 W3 W4 W5 W6) | FEATURES | R0/R1 ZONES | NOTES WATER LEVELS INSTRUMENTATION



ELEV. DEPTH (m)	DESCRIPTION
86.63	Continued from Borehole CRB-6
86.63 5.12	Moderately weathered to fresh, thinly laminated to medium bedded, grey, very fine to fine grained, faintly porous, weak SHALE (Georgian Bay Formation) with LIMESTONE interbeds
86.25 5.50	Moderately weathered to fresh, thinly laminated to medium bedded, grey, fine grained, slightly porous, weak SHALE (Georgian Bay Formation) with slightly weathered to fresh, thinly bedded, grey, fine grained, non-porous, medium strong, LIMESTONE interbeds.
78.48 13.27	END OF DRILLHOLE

Drilling record: HQ Casing; HQ Core

RUN No.	TYPE AND SURFACE DESCRIPTION	Jr	Ja
1	BD,PL,SM CL	1	1
	BD,PL,RO SA	1.5	2
	CO,UN,RO CL	3	1
	CO,UN,RO CL	3	1
2	BD,PL,SM CL	1	1
	BD,PL,SM CL	1	1
	BD,PL,SM CL	1	1
	CO,UN,RO PC, CI	3	3
	CO,PL,SM SA	1	2
	CO,UN,SM CL	2	1
3	BD,UN,SM CL	2	1
	CO,UN,RO CL	3	1
	BD,PL,SM CL	1	1
	CO,PL,SM PC, CI	1	3
4	BD,PL,SM CL	1	1
	BD,PL,SM CL	1	1
	BD,PL,SM CL	1	1
5	BD,PL,SM CL	1	1
6	CO,UN,SM PC, CI	2	3
	BD,IR,RO CL	3	1

R0/R1 ZONES: R1; R1; R1; R1

NOTES:

PLT(A) = 0.70 MPa
PLT(D) = 0.44 MPa
UCS = 14.6 MPa

PLT(A) = 0.65 MPa
PLT(D) = 0.51 MPa

PLT(A) = 0.61 MPa
PLT(D) = 0.15 MPa

FEATURES LEGEND

▨	BROKEN CORE
▩	CLAY SEAM
▦	LIMESTONE
■	LOST CORE

DEPTH SCALE
1 : 50

GOLDER

LOGGED: JL
CHECKED: MPL

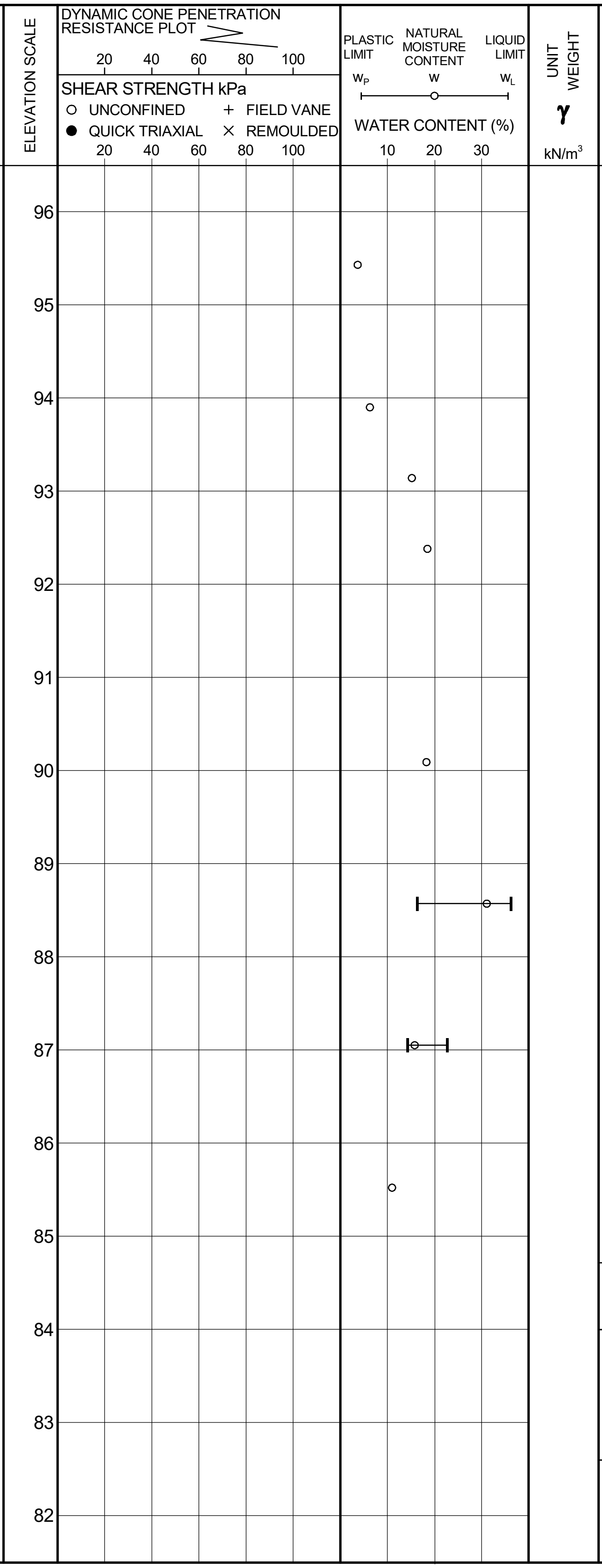
RECORD OF BOREHOLE No NW3-1

SHEET 1 OF 2

METRIC

PROJECT 1662333			
G.W.P. 2002-13-00	LOCATION N 4824275.8; E 295959.8 MTM NAD 83 ZONE 10 (LAT. 43.558358; LONG. -79.609422)	ORIGINATED BY JL
DIST Central HWY QEW	BOREHOLE TYPE CME 850, 210 mm O.D. Hollow Stem Augers	COMPILED BY MPL
DATUM Geodetic	DATE October 16-17, 2017	CHECKED BY SMM

ELEV DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLES – NUMBER	SAMPLES – TYPE	SAMPLES – "N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
96.5	GROUND SURFACE					
0.0	TOPSOIL (150mm)					
0.2	Silty sand, trace clay (FILL) Loose Brown Moist	1	SS	5		
		2	SS	5		0 74 23 3
95.0 1.5	SILT and SAND to Silty SAND, trace clay, trace gravel Compact to very dense Brown Moist to wet	3	SS	22		
	- Silt pocket at a depth of about 2.6 m	4	SS	24		0 68 30 2
		5	SS	22		
	- Becoming wet at a depth of about 3.7 m	6	SS	24	∇	
		7	SS	47		
	- Becoming grey at a depth of about 6.3 m	8	SS	78		5 32 61 2
89.3 7.2	SILTY CLAY, trace to some sand, trace gravel Soft Grey Wet	9	SS	4		
87.8 8.7	Sandy CLAYEY SILT, trace to some gravel (TILL) Stiff Grey Moist to wet	10	SS	12		10 29 45 16
86.4 10.1	Sandy gravelly CLAYEY SILT, some shale fragments (RESIDUAL SOIL) Hard Grey Moist to wet - Tricone grinding at a depth of about 10.1 m	11	SS	64		
	- Tricone grinding at a depth of about 11.6 m	12	SS	100/0.08		
84.7 11.8	Shale (BEDROCK) Grey	1	RC	REC 100%		RQD = 97%
	Bedrock cored from a depth of 11.8 m to 15.4 m	2	RC	REC 96%		RQD = 90%
	For bedrock coring details, refer to Record of Drillhole NW3-01	3	RC	REC 97%		RQD = 97%



Continued Next Page

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

RECORD OF BOREHOLE No NW3-1

SHEET 2 OF 2

METRIC

PROJECT 1662333

G.W.P. 2002-13-00

LOCATION N 4824275.8; E 295959.8 MTM NAD 83 ZONE 10 (LAT. 43.558358; LONG. -79.609422)

ORIGINATED BY JL

DIST Central HWY QEW

BOREHOLE TYPE CME 850, 210 mm O.D. Hollow Stem Augers

COMPILED BY MPL

DATUM Geodetic

DATE October 16-17, 2017

CHECKED BY SMM

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 × 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 *--- CONTINUED FROM PREVIOUS PAGE ---*	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
81.1	Shale (BEDROCK) Grey	▨	3	RC	REC 97%						RQD = 97%
15.4	END OF BOREHOLE NOTES: 1. Water level measured at a depth of about 4.5 m (Elev. 92.0 m) below ground surface prior to start of rock coring. 2. Water level measured at top of casing (Elev. 96.9 m) following completion of bedrock coring.										

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

○$^{3\%}$ STRAIN AT FAILURE

PROJECT: 1662333

RECORD OF DRILLHOLE: NW3-1

SHEET 1 OF 1

LOCATION: N 4824275.8 ;E 295959.8

INCLINATION: -90° AZIMUTH: ---

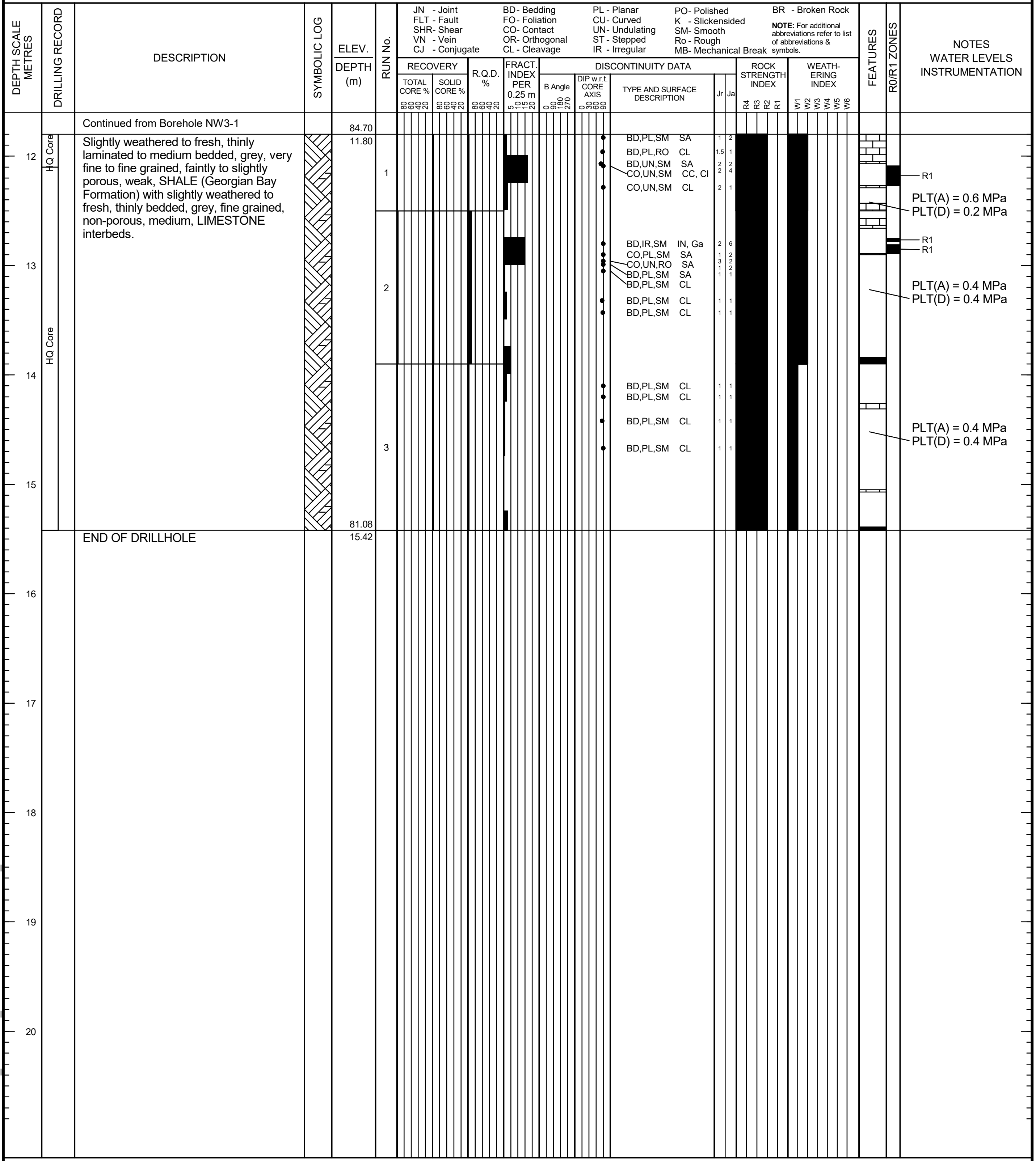
DRILLING DATE: October 16-17, 2017

DRILL RIG: CME 850 Track

DRILLING CONTRACTOR: Aardvark Drilling

DATUM: Geodetic

JN - Joint	BD- Bedding	PL - Planar	PO- Polished	BR - Broken Rock
FLT - Fault	FO- Foliation	CU- Curved	K - Slickensided	**NOTE:** For additional abbreviations refer to list of abbreviations & symbols.
SHR- Shear	CO- Contact	UN- Undulating	SM- Smooth	
VN - Vein	OR- Orthogonal	ST - Stepped	Ro - Rough	
CJ - Conjugate	CL - Cleavage	IR - Irregular	MB- Mechanical Break	



DESCRIPTION	ELEV. DEPTH (m)
Continued from Borehole NW3-1	84.70
Slightly weathered to fresh, thinly laminated to medium bedded, grey, very fine to fine grained, faintly to slightly porous, weak, SHALE (Georgian Bay Formation) with slightly weathered to fresh, thinly bedded, grey, fine grained, non-porous, medium, LIMESTONE interbeds.	11.80
	81.08 15.42
END OF DRILLHOLE	

TYPE AND SURFACE DESCRIPTION		Jr	Ja
BD,PL,SM	SA	1	2
BD,PL,RO	CL	1.5	1
BD,UN,SM	SA	2	2
CO,UN,SM	CC, Cl	2	4
CO,UN,SM	CL	2	1
BD,IR,SM	IN, Ga	2	6
CO,PL,SM	SA	1	2
CO,UN,RO	SA	3	2
BD,PL,SM	SA	1	2
BD,PL,SM	CL	1	1
BD,PL,SM	CL	1	1
BD,PL,SM	CL	1	1
BD,PL,SM	CL	1	1
BD,PL,SM	CL	1	1
BD,PL,SM	CL	1	1
BD,PL,SM	CL	1	1

NOTES WATER LEVELS INSTRUMENTATION

PLT(A) = 0.6 MPa
PLT(D) = 0.2 MPa

PLT(A) = 0.4 MPa
PLT(D) = 0.4 MPa

PLT(A) = 0.4 MPa
PLT(D) = 0.4 MPa

FEATURES LEGEND

▨	BROKEN CORE	▩	CLAY SEAM	▦	LIMESTONE	■	LOST CORE

DEPTH SCALE

1 : 50

GOLDER

LOGGED: JL

CHECKED: JC

GOLDER

RECORD OF BOREHOLE No NW3-2

SHEET 1 OF 1

METRIC

PROJECT 1662333

G.W.P. 2002-13-00

LOCATION N 4824342.4; E 295994.3 MTM NAD 83 ZONE 10 (LAT. 43.558958; LONG. -79.608996)

ORIGINATED BY FC

DIST Central HWY QEW

BOREHOLE TYPE CME 55, 203 mm O.D. Hollow Stem Augers

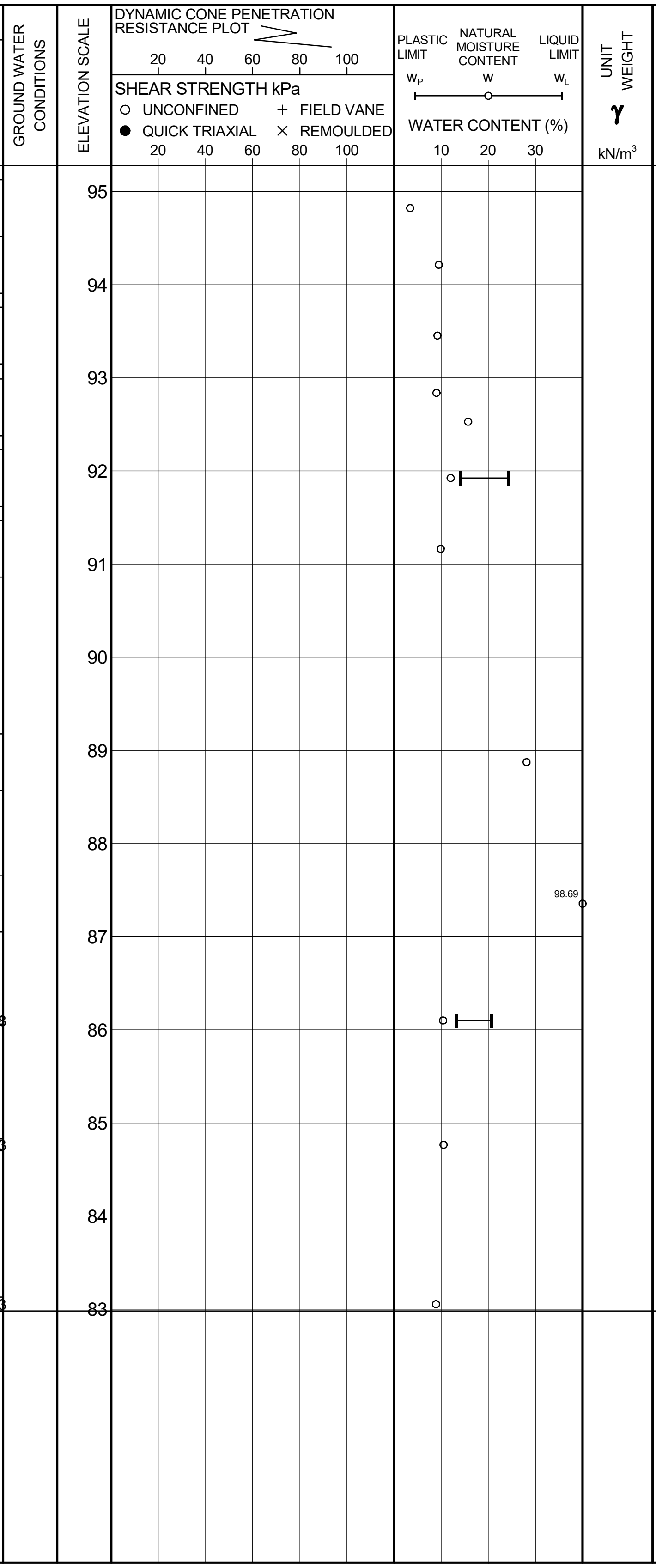
COMPILED BY KN

DATUM Geodetic

DATE August 23, 2017

CHECKED BY MWK

ELEV DEPTH	SOIL PROFILE DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
95.3	GROUND SURFACE							
0.0	ASPHALT (150 mm)							
0.2	Silty sand, some gravel to gravelly, trace to some clay (FILL) Compact Brown Moist	1	AS	-				
		2	SS	28				
	- Asphalt fragments at a depth of about 1.8 m	3	SS	26	16	58	20	6
92.7 2.6	Sandy clayey silt, trace to some gravel (FILL) Stiff to very stiff Brown to grey, mottled Moist	4A 4B	SS	14				
		5	SS	29				
91.6 3.7	Sand and gravel, some silt, trace clay (FILL) Dense Grey to brown Moist to wet - Trace asphalt fragments at a depth of about 4.0 m	6	SS	34	43	41	12	4
90.0 5.3	Silty sand, trace clay, trace gravel, trace organics, trace asphalt fragments (FILL) Very loose Brown Moist to wet	7	SS	2	1	70	26	3
	- 100 mm silty sand, organic layer and pieces of wood at a depth of 7.6 m							
87.5 7.8	CLAYEY SILT with SAND, some gravel (TILL) Stiff to hard Grey Moist to wet - Trace organics from a depth of about 8.5 m	8	SS	12				
		9	SS	131/0.08	12	38	38	12
		10	SS	100/0.13				
		11	SS	100/0.13				
83.0 12.3	END OF BOREHOLE NOTE: 1. Borehole dry prior to tricone drilling below a depth of 3.4 m and introduction of wash water.							



REMARKS & GRAIN SIZE DISTRIBUTION (%)

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

○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No NW3-3

SHEET 1 OF 1

METRIC

PROJECT 1662333

G.W.P. 2002-13-00

LOCATION N 4824329.2; E 296002.3 MTM NAD 83 ZONE 10 (LAT. 43.558840; LONG. -79.608895)

ORIGINATED BY ACM

DIST Central HWY QEW

BOREHOLE TYPE CME 55, 114 mm I.D., Hollow Stem Augers

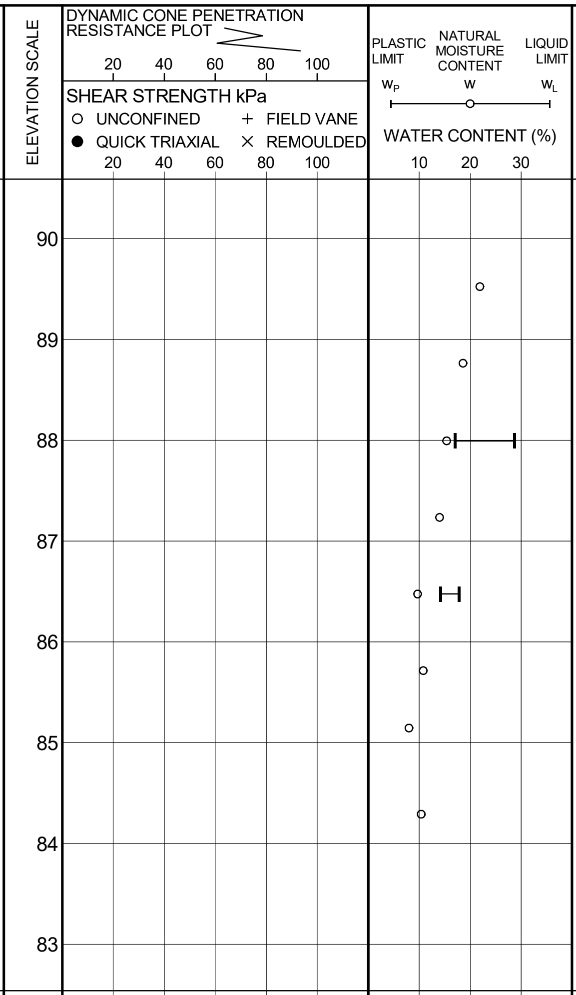
COMPILED BY SK

DATUM Geodetic

DATE August 9, 2018

CHECKED BY SMM

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES			GR	SA	SI	CL
90.6	GROUND SURFACE									
0.0	Silty sand, some gravel, trace to some clay, trace organics (FILL) Very loose Dark brown Moist to wet below 0.7 m - Hydrocarbon odour at 0.8 m	1	SS	4						
		2	SS	4			18	49	27	6
89.2 1.5	CLAYEY SILT with SAND, trace to some gravel (TILL) Soft to stiff Grey-brown to grey at 3.7 m with oxidation staining Moist	3	SS	4						
		4	SS	15			8	31	44	17
		5	SS	22						
87.2 3.4	SILT and SAND, some gravel. trace to some clay (TILL) Compact to very dense Grey Moist - Auger grinding from 3.4 m to 3.8 m	6	SS	47						
		7	SS	100/0.28						
		8	SS	50/0.08						
	- Auger grinding from 6.1 m to 6.9 m - Limestone layer between depths of 6.2 m and 6.3 m	9A 9B	SS	50/0.13						
		10	SS	50/0.08						
		11	SS	100/0.28						
82.6 8.1	END OF BOREHOLE NOTES: 1. Open borehole dry upon completion of drilling.									



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

$\bigcirc^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No PED-01

SHEET 1 OF 2

METRIC

PROJECT 1662333
G.W.P. 2002-13-00
DIST Central HWY QEW
DATUM Geodetic
LOCATION N 4824314.1; E 295977.3 MTM NAD 83 ZONE 10 (LAT. 43.558703; LONG. -79.609205)
BOREHOLE TYPE CME 55, 203 mm O.D. Hollow Stem Augers, HQ Casing
DATE August 17-18, 2017
ORIGINATED BY FC
COMPILED BY KN
CHECKED BY SMM

SOIL PROFILE

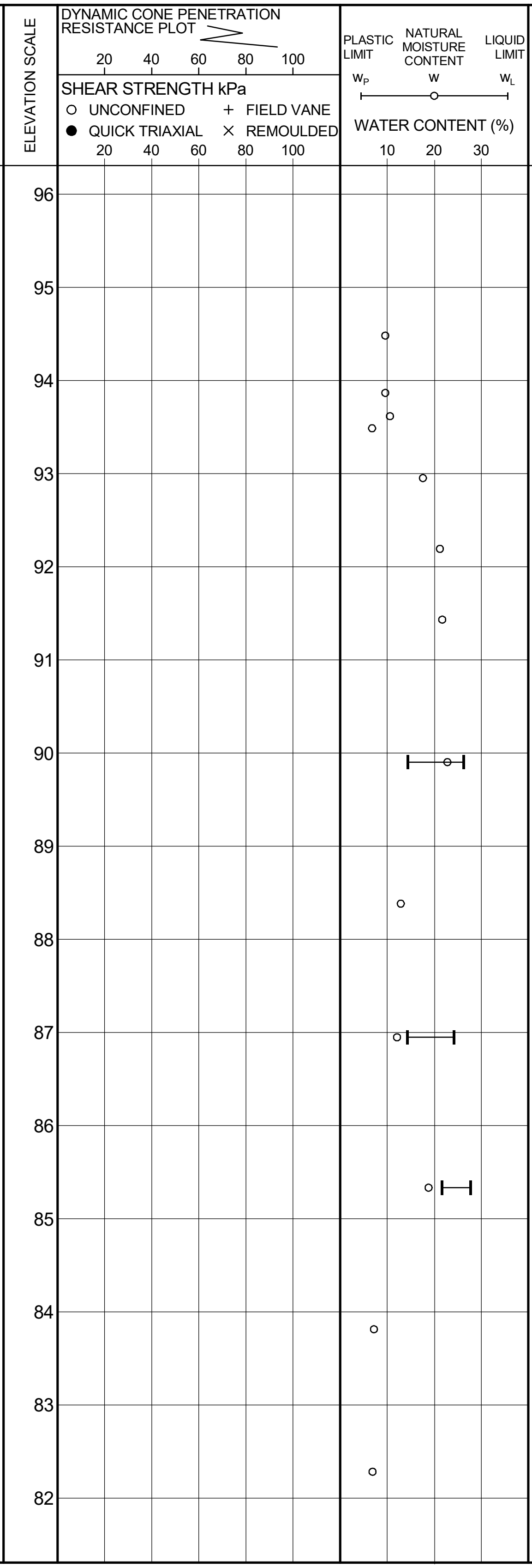
ELEV / DEPTH	DESCRIPTION
96.3	GROUND SURFACE
0.0	ASPHALT (150 mm)
0.2	Gravelly silty sand, trace to some clay, trace rootlets, trace silty clay pockets (FILL) Compact to dense Brown Moist - Asphalt fragments from a depth of about 1.7 m to 1.8 m
93.6 / 2.7	- Clayey silt pocket/zone, trace asphalt at a depth of about 2.5 m to 2.7 m
	Silt and sand, trace clay (FILL) Loose to dense Brown Moist - Wet at a depth of about 3.4 m
90.7 / 5.6	CLAYEY SILT, trace sand, trace gravel Stiff Grey Moist to wet
89.1 / 7.2	CLAYEY SILT with SAND to some sand, trace to some gravel (TILL) Stiff to hard Grey Moist to wet
84.6 / 11.7	SAND, trace to some silt, trace to some clay, trace to some gravel Very dense Grey Moist to wet - Clayey silt lens from a depth of about 14 m to 14.1 m

Continued Next Page

SAMPLES

NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
1	AS	-				
2	SS	28				
3	SS	30	21	52	21	6
4A 4B 4C	SS	22				
5	SS	7				
6	SS	18	0	37	60	3
7	SS	32				
8	SS	9				
9	SS	13				
10A 10B	SS	47	11	32	42	15
11	SS	50				
12	SS	77	3	75	15	7
13	SS	79				

STRAT PLOT · GROUND WATER CONDITIONS · UNIT WEIGHT γ kN/m³ · REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL



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

$\bigcirc^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No PED-01

SHEET 2 OF 2

METRIC

PROJECT 1662333

G.W.P. 2002-13-00

DIST Central HWY QEW

DATUM Geodetic

LOCATION N 4824314.1; E 295977.3 MTM NAD 83 ZONE 10 (LAT. 43.558703; LONG. -79.609205)

BOREHOLE TYPE CME 55, 203 mm O.D. Hollow Stem Augers, HQ Casing

DATE August 17-18, 2017

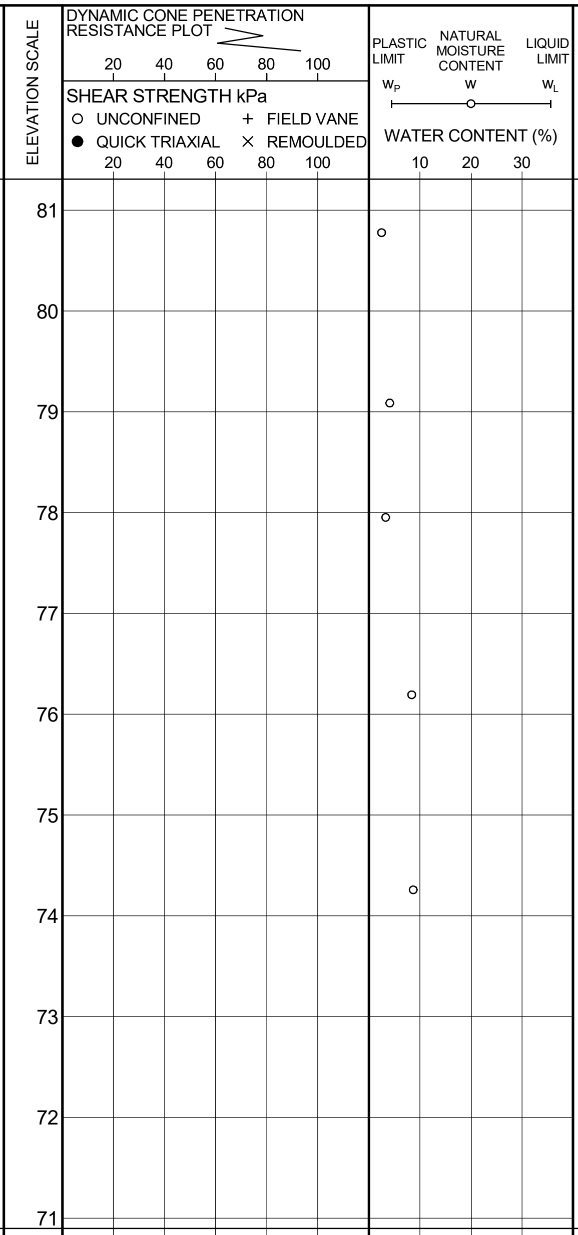
ORIGINATED BY FC

COMPILED BY KN

CHECKED BY SMM

SOIL PROFILE		SAMPLES			REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---				
	SAND, trace to some silt, trace to some clay, trace to some gravel Very dense Grey Moist to wet	14	SS	121	14 73 10 3
		15A 15B	SS	69	
	- Gravelly at a depth of about 17.8 m	16	SS	100/0.13	
		17	SS	50	10 81 7 2
		18	SS	51	
75.0 21.3	SAND and GRAVEL, some silt Very dense Grey Moist to wet	19	SS	100/0.05	
		20	SS	100/0.05	
74.0 22.3	Shale (BEDROCK) Grey Bedrock cored from a depth of 22.3 m to 25.4 m For bedrock coring details, refer to Record of Drillhole PED-01	1	RC	REC 92%	RQD = 90%
		2	RC	REC 100%	RQD = 95%
70.9 25.4	END OF BOREHOLE NOTE: 1. Borehole dry prior to rock coring.				

STRAT PLOT | GROUND WATER CONDITIONS | ELEVATION SCALE | UNIT WEIGHT γ kN/m³



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

PROJECT: 1662333

RECORD OF DRILLHOLE: PED-01

SHEET 1 OF 1

LOCATION: N 4824314.1 ;E 295977.3

DRILLING DATE: August 17-18, 2017

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

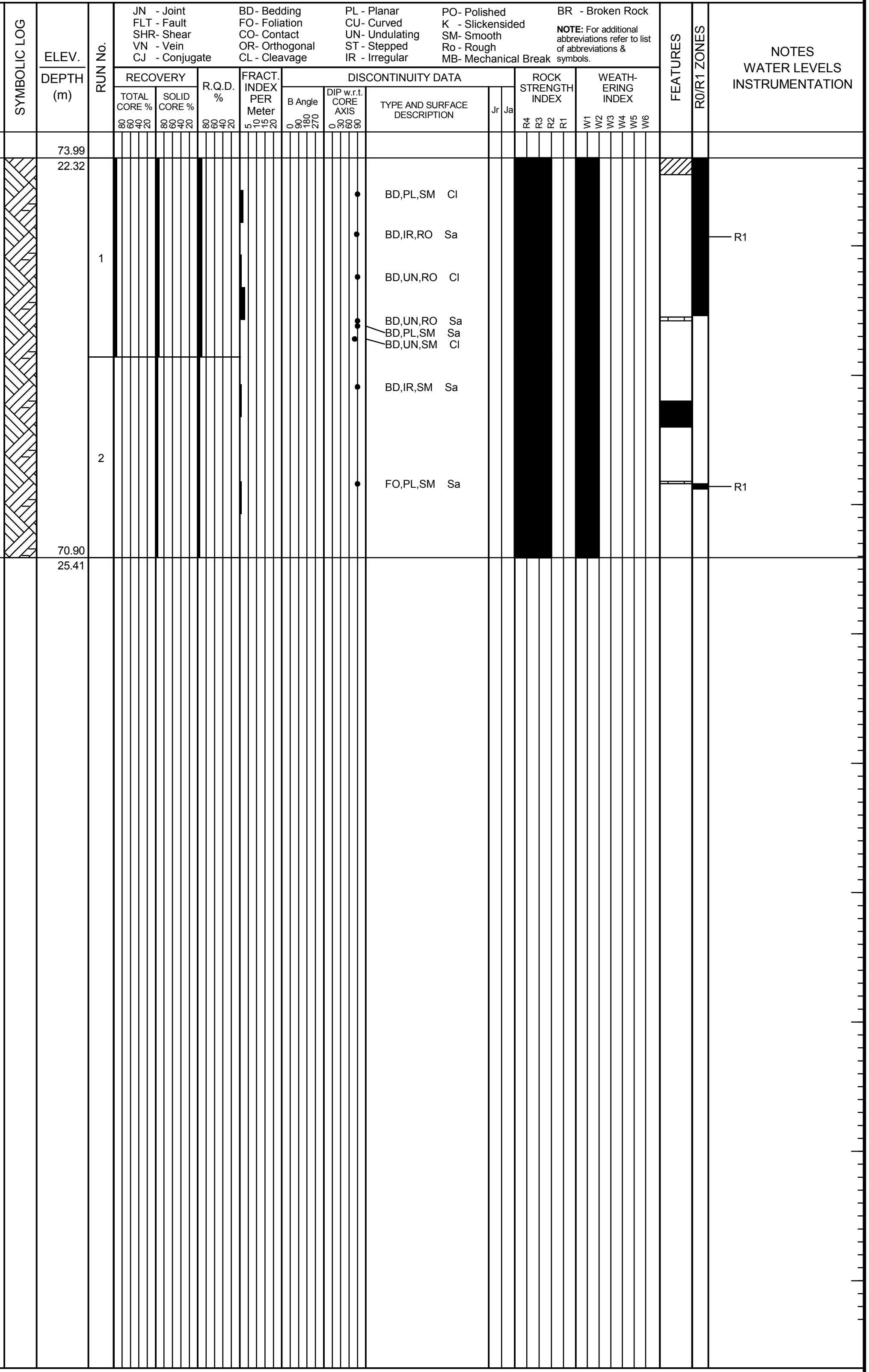
DRILL RIG: CME 55

DRILLING CONTRACTOR: Aardvark Drilling

JN - Joint
FLT - Fault
SHR- Shear
VN - Vein
CJ - Conjugate
BD- Bedding
FO- Foliation
CO- Contact
OR- Orthogonal
CL - Cleavage
PL - Planar
CU- Curved
UN- Undulating
ST - Stepped
IR - Irregular
PO- Polished
K - Slickensided
SM- Smooth
Ro - Rough
MB- Mechanical Break
BR - Broken Rock

NOTE: For additional abbreviations refer to list of abbreviations & symbols.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. DEPTH (m)	RUN No.
		Continued from Borehole PED-01	73.99	
	HQ Casing NQ Rock Core	Slightly weathered, thinly to medium bedded, grey, fine grained, slightly porous, weak, SHALE (Georgian Bay Formation) with slightly weathered to fresh, thinly bedded, grey, fine grained, non-porous, medium strong, LIMESTONE interbeds.	22.32	1 2
			70.90 25.41	
		END OF DRILLHOLE		



BD,PL,SM Cl
BD,IR,RO Sa
BD,UN,RO Cl
BD,UN,RO Sa
BD,PL,SM Sa
BD,UN,SM Cl
BD,IR,SM Sa
FO,PL,SM Sa

FEATURES LEGEND

▨ BROKEN CORE
▩ CLAY SEAM
▦ LIMESTONE
■ LOST CORE

DEPTH SCALE
1 : 50

GOLDER

LOGGED: FC
CHECKED: JC

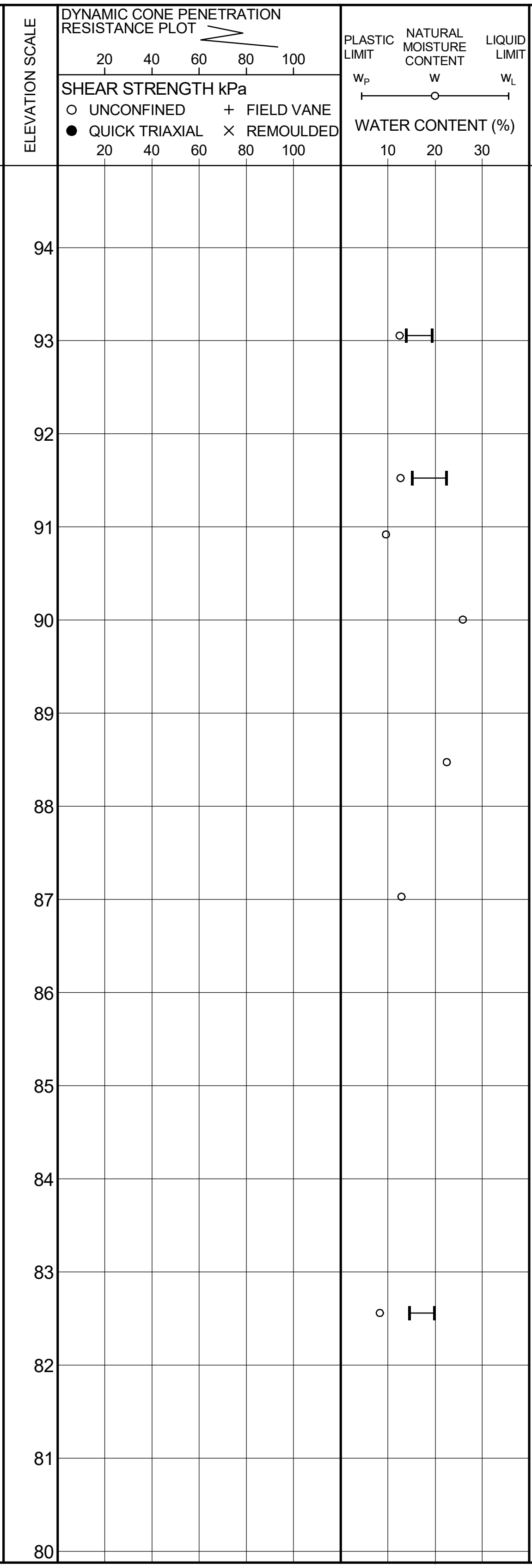
RECORD OF BOREHOLE No S2

SHEET 1 OF 2

METRIC

PROJECT 1662333
G.W.P. 2002-13-00
DIST Central HWY QEW
DATUM Geodetic
LOCATION N 4824357.2; E 296001.4 MTM NAD 83 ZONE 10 (LAT. 43.559092; LONG. -79.608907)
BOREHOLE TYPE CME 75, 210 mm O.D. Hollow Stem Augers
DATE September 13, 2018
ORIGINATED BY ACM
COMPILED BY JMP
CHECKED BY SMM

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
94.9	GROUND SURFACE									
0.0	ASPHALT (150 mm)									
	Sand and gravel, some silt (FILL) Brown Moist	1A 1B	SS	16						
94.2 0.9	Sandy silt, trace organics, some wood fragments (FILL) Compact Brown Moist	2A 2B	SS	12						
	Gravelly sand, trace silt (FILL) Grey Moist	3	SS	17						
	Gravelly clayey silt with sand (FILL) Stiff to very stiff Grey-brown Moist	4	SS	8						
	- Auger grinding from 1.5 m to 2.7 m	5	SS	12			21	49	21	9
91.2 3.7	Sand, some silt, some gravel, trace clay, asphalt pieces (FILL) Compact Brown-grey, contains oxidation staining Moist	6A 6B	SS	23			19	61	15	5
90.4 4.5	Silty sand, trace to some clay (FILL) Loose Brown to grey Wet	7	SS	3						
89.3 5.6	Silty SAND, trace to some clay Loose Brown to grey Wet	8	SS	10			0	72	22	6
87.8 7.1	Sandy CLAYEY SILT, some gravel to gravelly (TILL) Hard Grey Moist	9	SS	100/0.23						
		10	SS	100/0.25						
	- Auger grinding from 9.8 m to 10.1 m	11	SS	100/0.08						
	- Auger grinding from 11.0 m to 11.6 m	12	SS	100/0.41			20	29	42	9
	- Auger grinding at 13.4 m	13	SS	100/0.25						
80.3 14.6	Silty SAND									



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

$\circ^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No S2

SHEET 2 OF 2

METRIC

PROJECT 1662333
G.W.P. 2002-13-00
DIST Central HWY QEW
DATUM Geodetic
LOCATION N 4824357.2; E 296001.4 MTM NAD 83 ZONE 10 (LAT. 43.559092; LONG. -79.608907)
BOREHOLE TYPE CME 75, 210 mm O.D. Hollow Stem Augers
DATE September 13, 2018
ORIGINATED BY ACM
COMPILED BY JMP
CHECKED BY SMM

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 × 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 ---													
	Silty SAND, some gravel, trace to some clay Very dense Grey Moist		14	SS	51									
							79							
							78							
77.5			15	SS	54				○		13	55	25	7
17.4	END OF BOREHOLE NOTE: 1. Borehole dry upon completion of drilling.													

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

○ $^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No S3

SHEET 1 OF 2

METRIC

PROJECT 1662333

G.W.P. 2002-13-00

LOCATION N 4824337.3; E 296021.0 MTM NAD 83 ZONE 10 (LAT. 43.558912; LONG. -79.608665)

ORIGINATED BY ACM

DIST Central HWY QEW

BOREHOLE TYPE CME 55, 108 mm I.D., Hollow Stem Augers

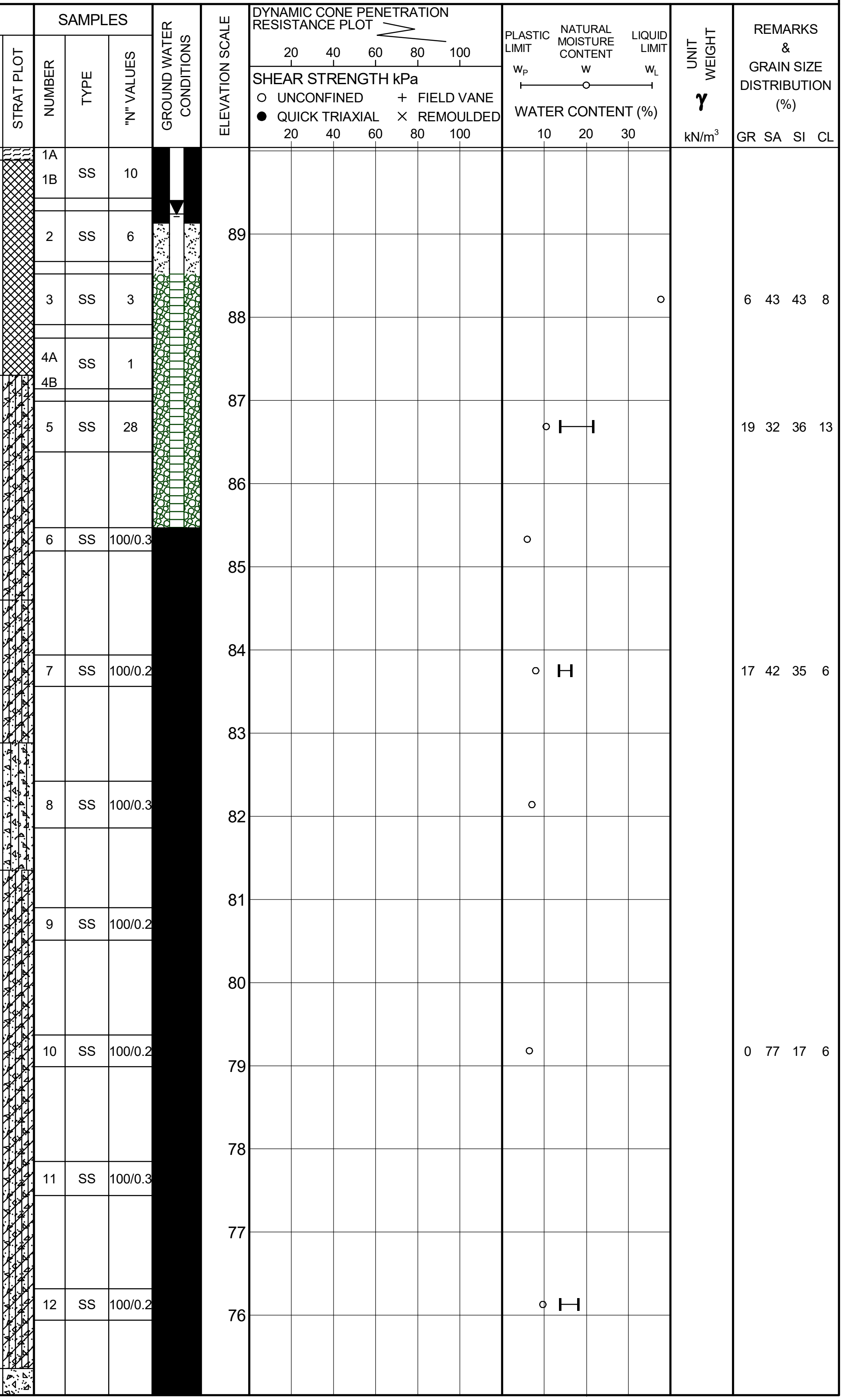
COMPILED BY JMP

DATUM Geodetic

DATE October 9, 2018

CHECKED BY SMM

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
90.0	GROUND SURFACE							
0.0	TOPSOIL (150 mm)	1A						
0.2	Silt and sand, trace to some clay, trace to some gravel, wood pieces, organic odour (FILL) Very loose to compact Brown and black Moist to wet - Some gravel from 0.2 m to 0.6 m	1B	SS	10				
		2	SS	6				
		3	SS	3	6	43	43	8
		4A	SS	1				
87.3 2.7	CLAYEY SILT with SAND, some gravel (TILL) Very stiff to hard Grey Moist - Auger grinding from 3.4 m to 3.7 m	4B						
		5	SS	28	19	32	36	13
		6	SS	100/0.3				
84.6 5.4	SILT and SAND, some gravel (TILL) Very dense Grey Moist	7	SS	100/0.2	17	42	35	6
	- Auger grinding from 7.0 m to 7.3 m							
82.8 7.2	Gravelly silty SAND (TILL) Very dense Grey Moist	8	SS	100/0.3				
81.3 8.7	CLAYEY SILT with SAND (TILL) Hard Grey Moist	9	SS	100/0.2				
		10	SS	100/0.2	0	77	17	6
	- Auger grinding from 11.3 m to 11.4 m							
		11	SS	100/0.3				
	- Oxidation staining at 13.9 m - Auger grinding from 14.0 m to 14.3 m	12	SS	100/0.2				
75.3 14.7	Gravelly SAND (TILL)							



Continued Next Page

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

$\bigcirc^{3\%}$ STRAIN AT FAILURE

PROJECT 1662333

RECORD OF BOREHOLE No S3

SHEET 2 OF 2

METRIC

G.W.P. 2002-13-00

LOCATION N 4824337.3; E 296021.0 MTM NAD 83 ZONE 10 (LAT. 43.558912; LONG. -79.608665)

ORIGINATED BY ACM

DIST Central HWY QEW

BOREHOLE TYPE CME 55, 108 mm I.D., Hollow Stem Augers

COMPILED BY JMP

DATUM Geodetic

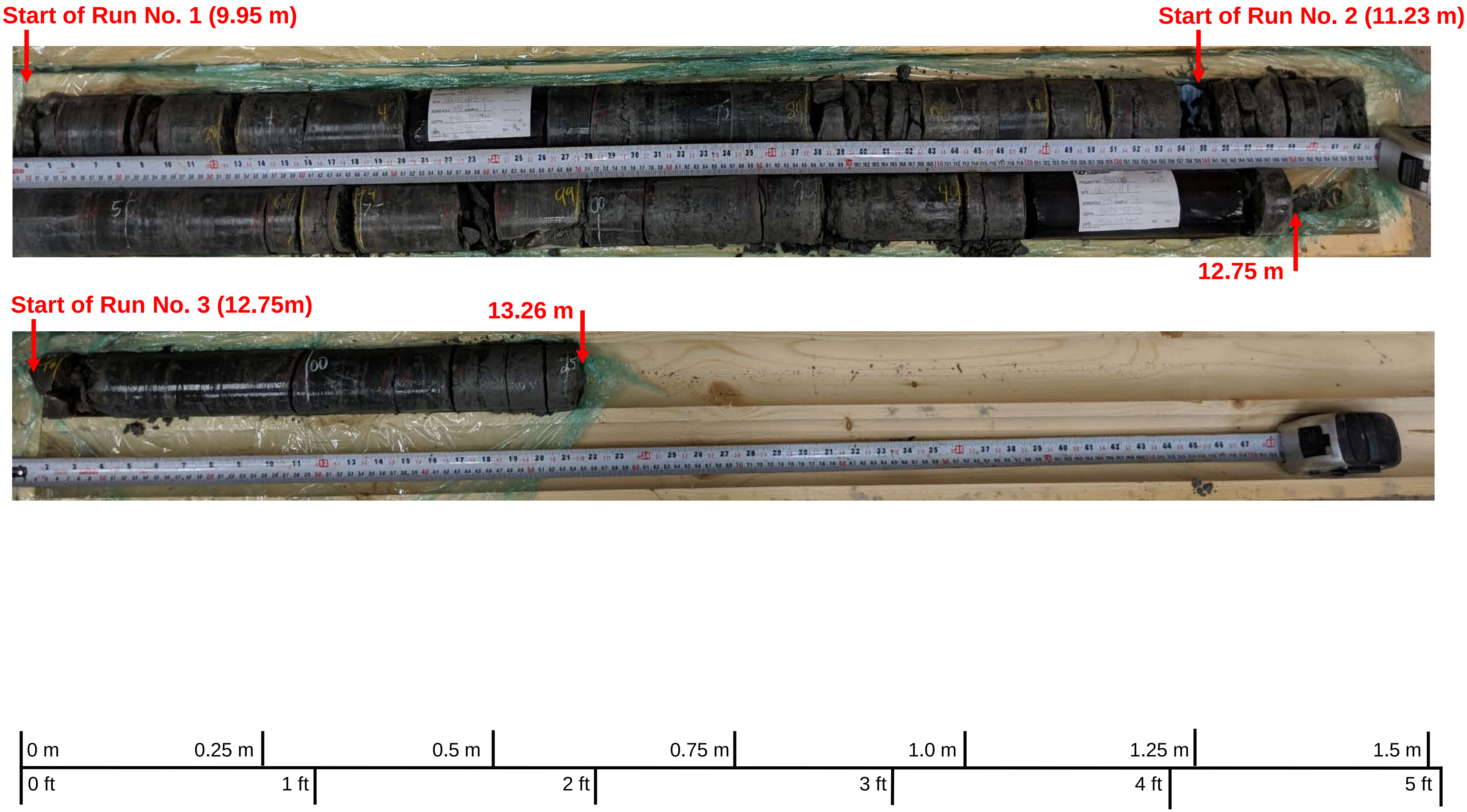
DATE October 9, 2018

CHECKED BY SMM

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 × 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 ---													
	Gravelly SAND, trace to some silt, trace clay (TILL) Compact to dense Grey Wet		13	SS	25				○		20	72	6	2
							74							
73.4			14	SS	46									
16.6	END OF BOREHOLE NOTES: 1. Water level measured at a depth of 15.9 m below ground surface (Elev. 74.1 m) upon completion of drilling. 2. Groundwater level measurements in piezometer: Date Depth (m) Elev. (m) 06/11/18 0.8 89.2													

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

$\circ^{3\%}$ STRAIN AT FAILURE



Scale

REVISION DATE: March 7, 2018 BY: JIL Project: 1662333

PROJECT

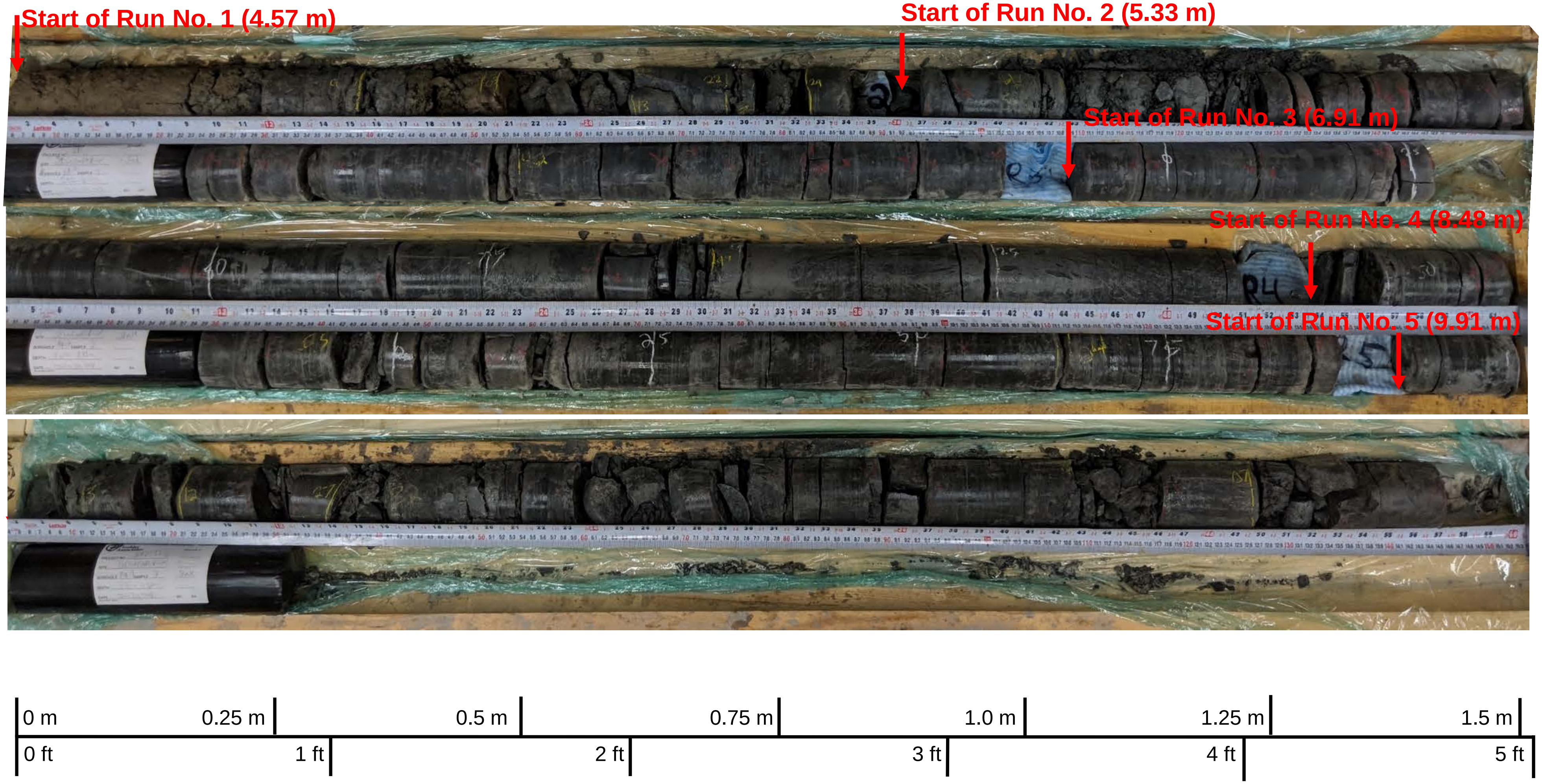
MTO Assignment 2015-E-0033
QEW Widening Between
Mississauga Road and Hurontario Street

TITLE

Bedrock Core Photograph
Borehole SWME-4 (9.95 m to 13.26 m)

GOLDER

PROJECT No. 1662333			FILE No. ----		
DRAFT	JIL	20180307	SCALE	AS SHOWN	VER. 1.
CADD	--		**FIGURE A-1**		
CHECK	SMM	20181116			
REVIEW	JMAC	20181116			



Scale

PROJECT

MTO Assignment 2015-E-0033
QEW Widening Between
Mississauga Road and Hurontario Street

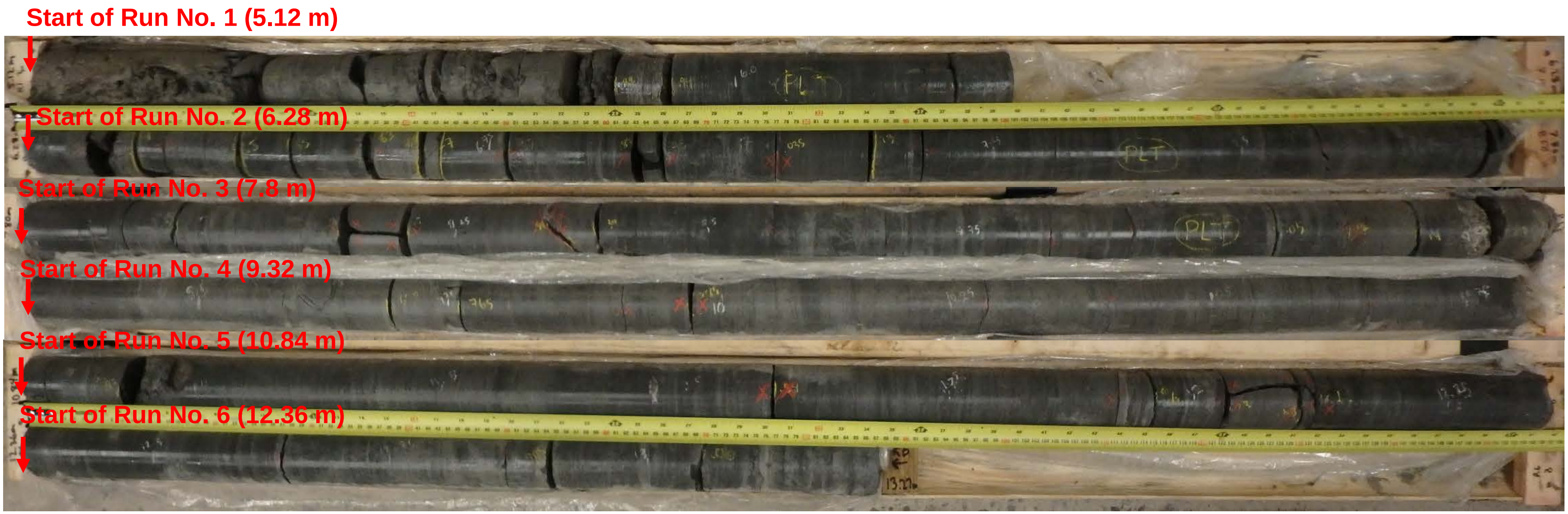
TITLE

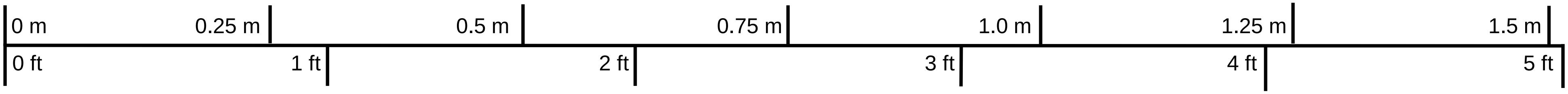
Bedrock Core Photograph
Borehole AR-2 (4.57 m to 11.60 m)

GOLDER

PROJECT No. 1662333			FILE No. ----	
DRAFT	SE	20180821	SCALE AS SHOWN	VER. 1.
CADD	--			
CHECK	SMM	20181116	**FIGURE A-2**	
REVIEW	JMAC	20181116		

REVISION DATE: August 21, 2018 BY: SE Project: 1662333





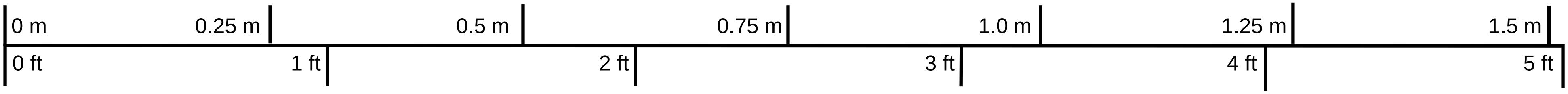
Scale

REVISION DATE: March 7, 2018 BY: JIL Project: 1662333

PROJECT	**MTO Assignment 2015-E-0033 QEW Widening Between Mississauga Road and Hurontario Street**
TITLE	**Bedrock Core Photograph Borehole CRB-6 (5.12 m to 13.27 m)**

GOLDER

PROJECT No. 1662333			FILE No. ----	
DRAFT	JIL	Mar 2018	SCALE AS SHOWN	VER. 1.
CADD	--		**FIGURE A-3**	
CHECK	SMM	20181116		
REVIEW	JMAC	20181116		



Scale

PROJECT

MTO Assignment 2015-E-0033
QEW Widening Between
Mississauga Road and Hurontario Street

TITLE

Bedrock Core Photograph
Borehole CRB-5 (7.18 m to 15.52 m)

GOLDER

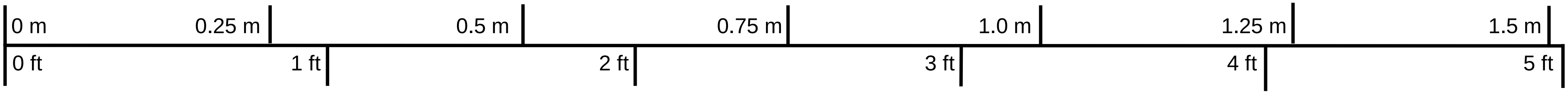
PROJECT No. 1662333			FILE No. ----	
DRAFT	JIL	Mar 2018	SCALE AS SHOWN	VER. 1.
CADD	--		**FIGURE A-4**	
CHECK	DM	June 2018		
REVIEW	SMM	June 2018		

REVISION DATE: March 7, 2018 BY: JIL Project: 1662333

Start of Run No. 1 (11.78 m)

Start of Run No. 2 (12.5 m)

Start of Run No. 3 (13.9 m)



Scale

PROJECT

MTO Assignment 2015-E-0033
QEW Widening Between
Mississauga Road and Hurontario Street

TITLE

Bedrock Core Photograph
Borehole NW3-1 (11.80 m to 15.42 m)

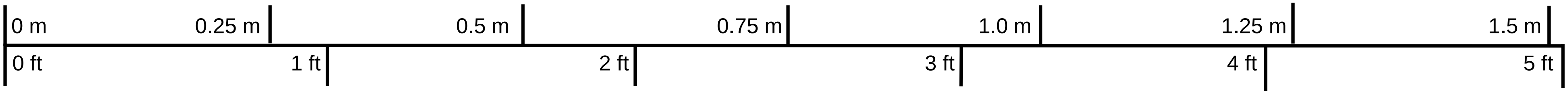
GOLDER

PROJECT No. 1662333			FILE No. ----		
DRAFT	SE	20180821	SCALE	AS SHOWN	VER. 1.
CADD	--		**FIGURE A-5**		
CHECK	JMAC	11/14/2018			
REVIEW	SMM	11/14/2018			

REVISION DATE: August 21, 2018 BY: SE Project: 1662333

Start of Run No. 1 (22.32 m)

Start of Run No. 2 (23.82 m)



Scale

PROJECT

MTO Assignment 2015-E-0033
QEW Widening Between
Mississauga Road and Hurontario Street

TITLE

Bedrock Core Photograph
Borehole PED-01 (22.32 m to 23.82 m)

GOLDER

PROJECT No. 1662333			FILE No. ----		
DRAFT	JIL	Mar 2018	SCALE	AS SHOWN	VER. 1.
CADD	--		**FIGURE A-6**		
CHECK	DM	APR 2018			
REVIEW	SMM	APR 2018			

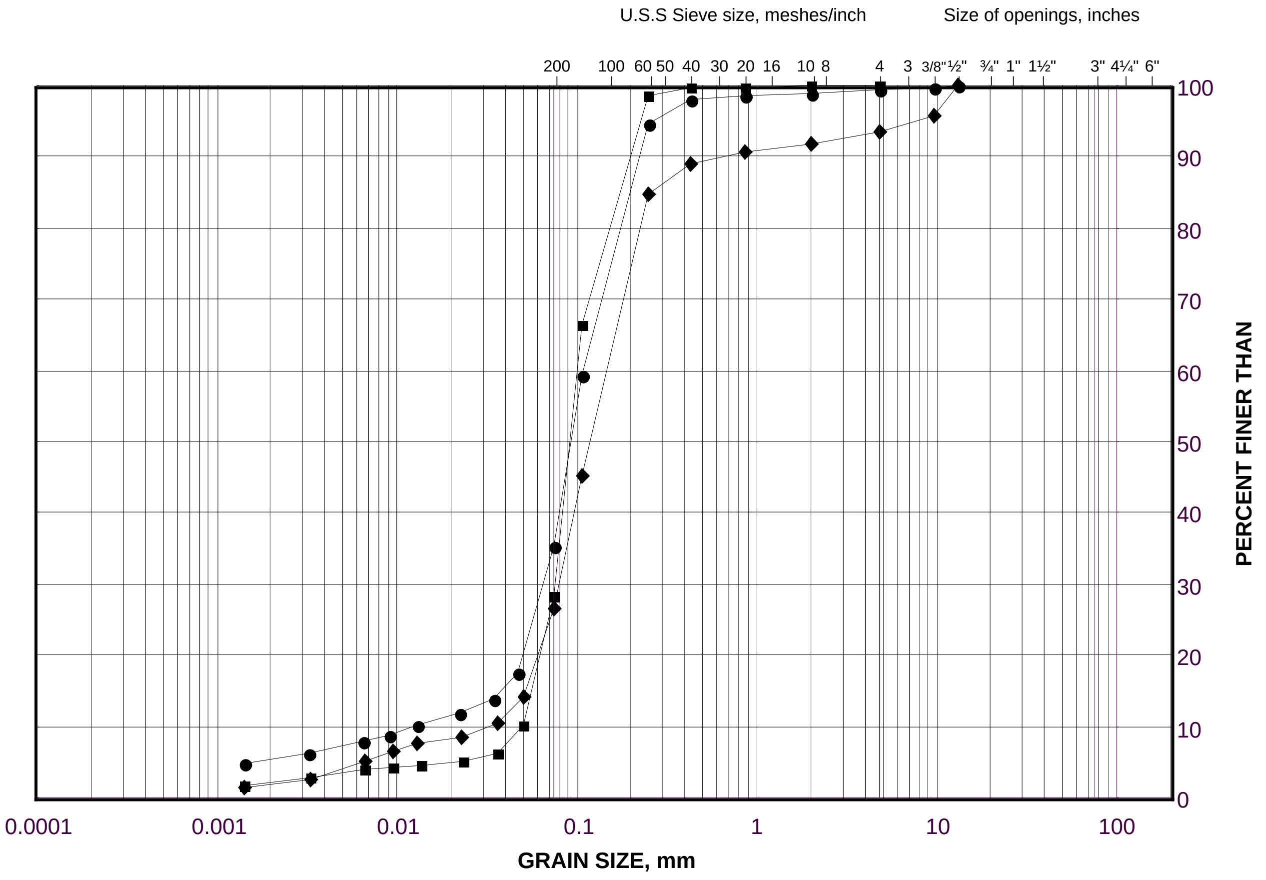
REVISION DATE: March 7, 2018 BY: JIL Project: 1662333

APPENDIX B

Geotechnical Laboratory Test Results (incl. Geomechnica Test Results on Rock)

GRAIN SIZE DISTRIBUTION
Silt to Sand to Silty Sand (Fill)

FIGURE B-1



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	SWME-1	2	88.4
■	NW3-1	2	95.4
◆	CRB-6	2	90.7

Project Number: 1662333

Checked By: SMM

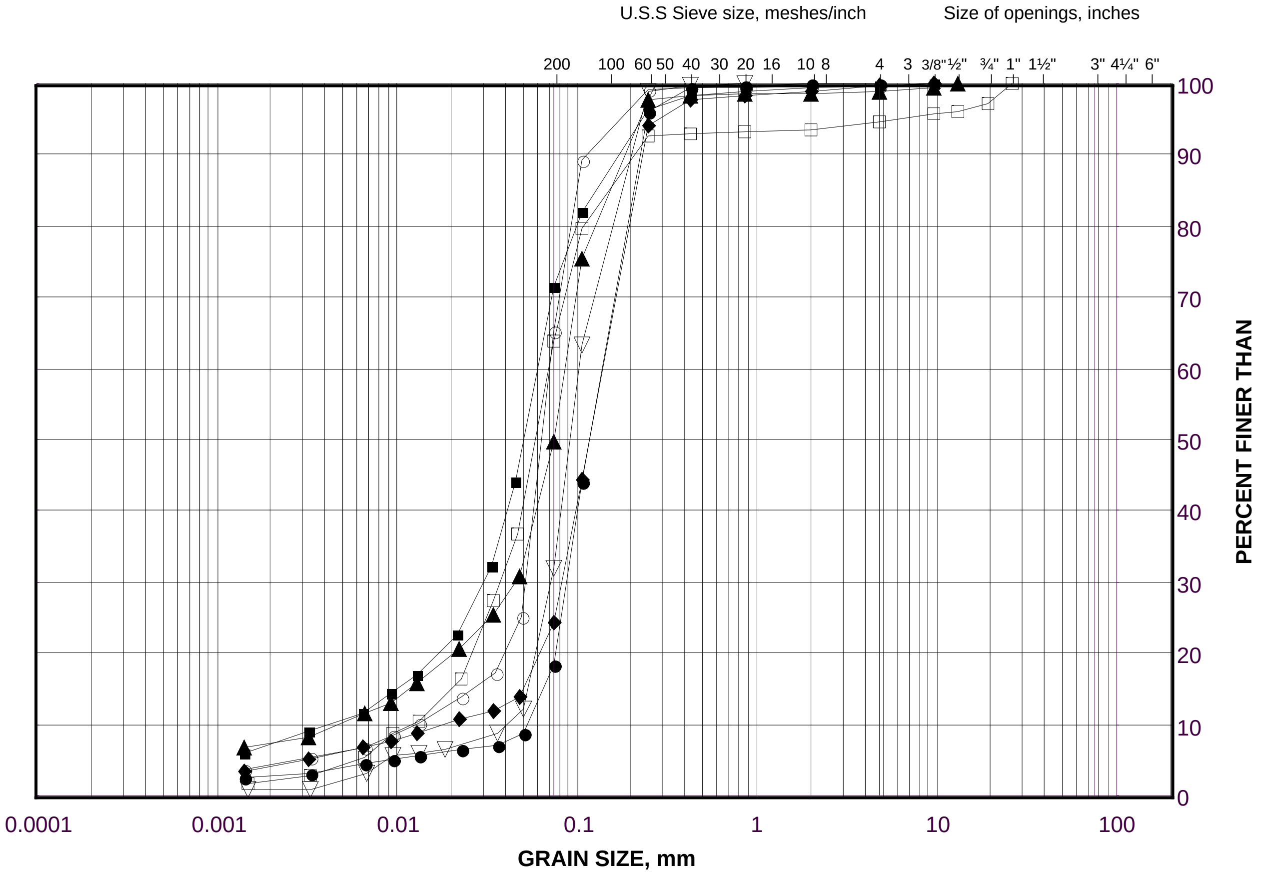
Golder Associates

Date: 06-Dec-18

GRAIN SIZE DISTRIBUTION

Sandy Silt to Sand

FIGURE B-2



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	AR-1	3	93.9
■	SWME-3	4	89.9
◆	SWME-1	4	86.9
▲	SWME-4	4	92.9
▽	NW3-1	4	93.9
❍	AR-1	6	91.6
□	NW3-1	8	90.1

Project Number: 1662333

Checked By: SMM

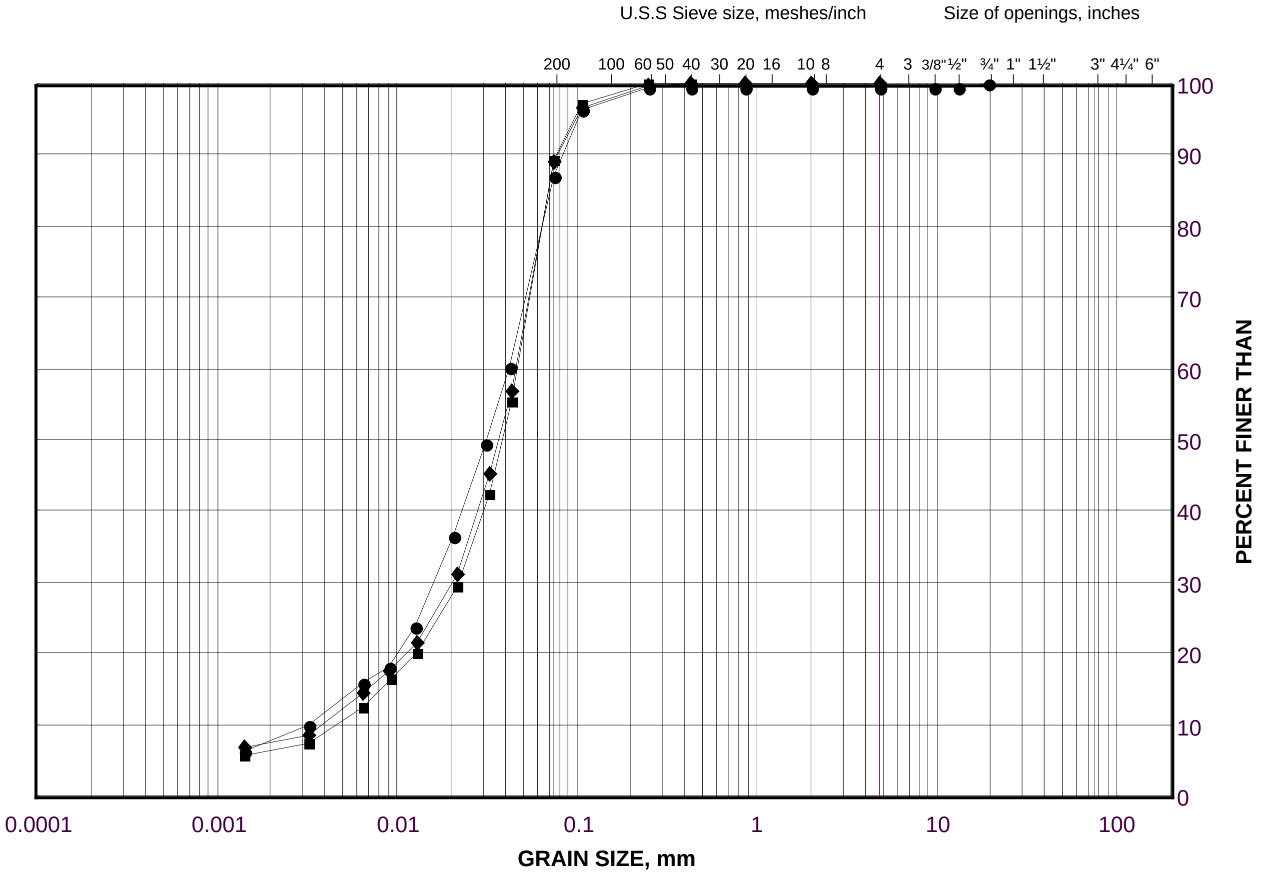
Golder Associates

Date: 06-Dec-18

GRAIN SIZE DISTRIBUTION

Silt

FIGURE B-3



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	SWME-3	5	89.3
■	AR-1	8	89.3
◆	SWME-4	8	89.1

Project Number: 1662333

Checked By: SMM

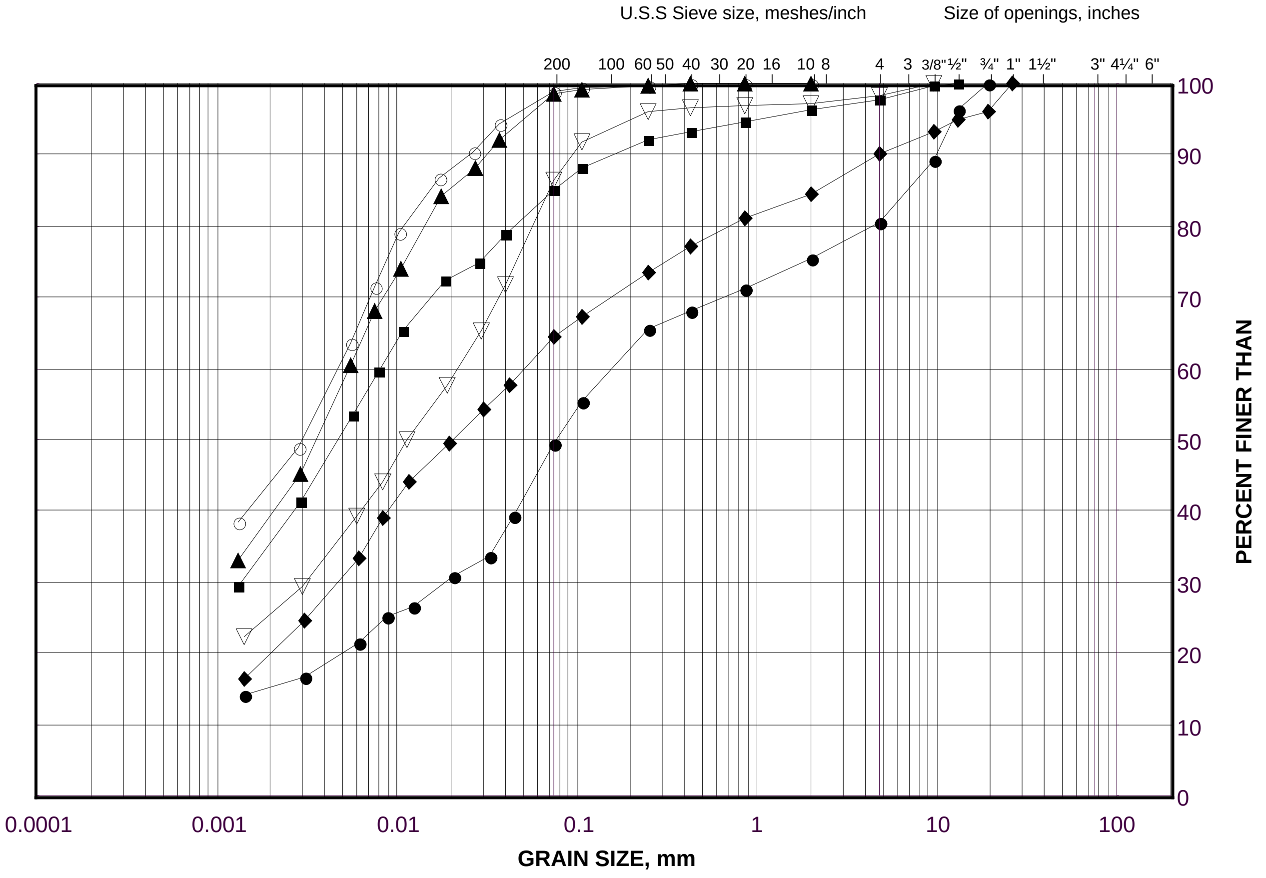
Golder Associates

Date: 06-Dec-18

GRAIN SIZE DISTRIBUTION

Clayey Silt with Sand to Silty Clay

FIGURE B-4



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	SWME-2	2	86.3
■	SWME-1	5B	85.9
◆	CRB-6	6	87.7
▲	SWME-3	6B	88.5
▽	SWME-4	7	90.6
❍	AR-1	9	87.9

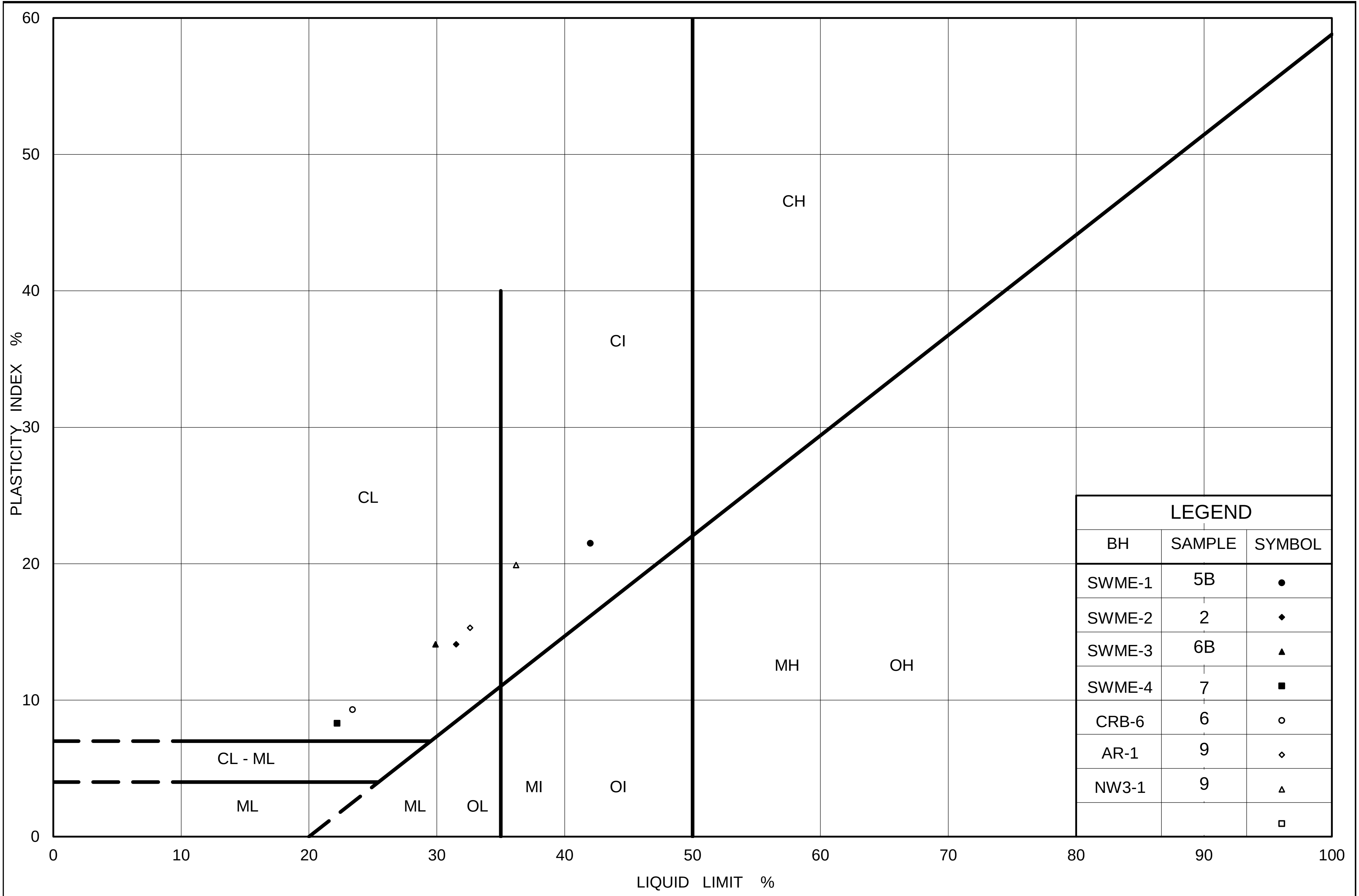
Project Number: 1662333

Checked By: SMM

Golder Associates

Date: 06-Dec-18

Oct 75, FF-S-21



Ministry of Transportation

Ontario

PLASTICITY CHART
Clayey Silt with Sand to Silty Clay

Figure No. B-5

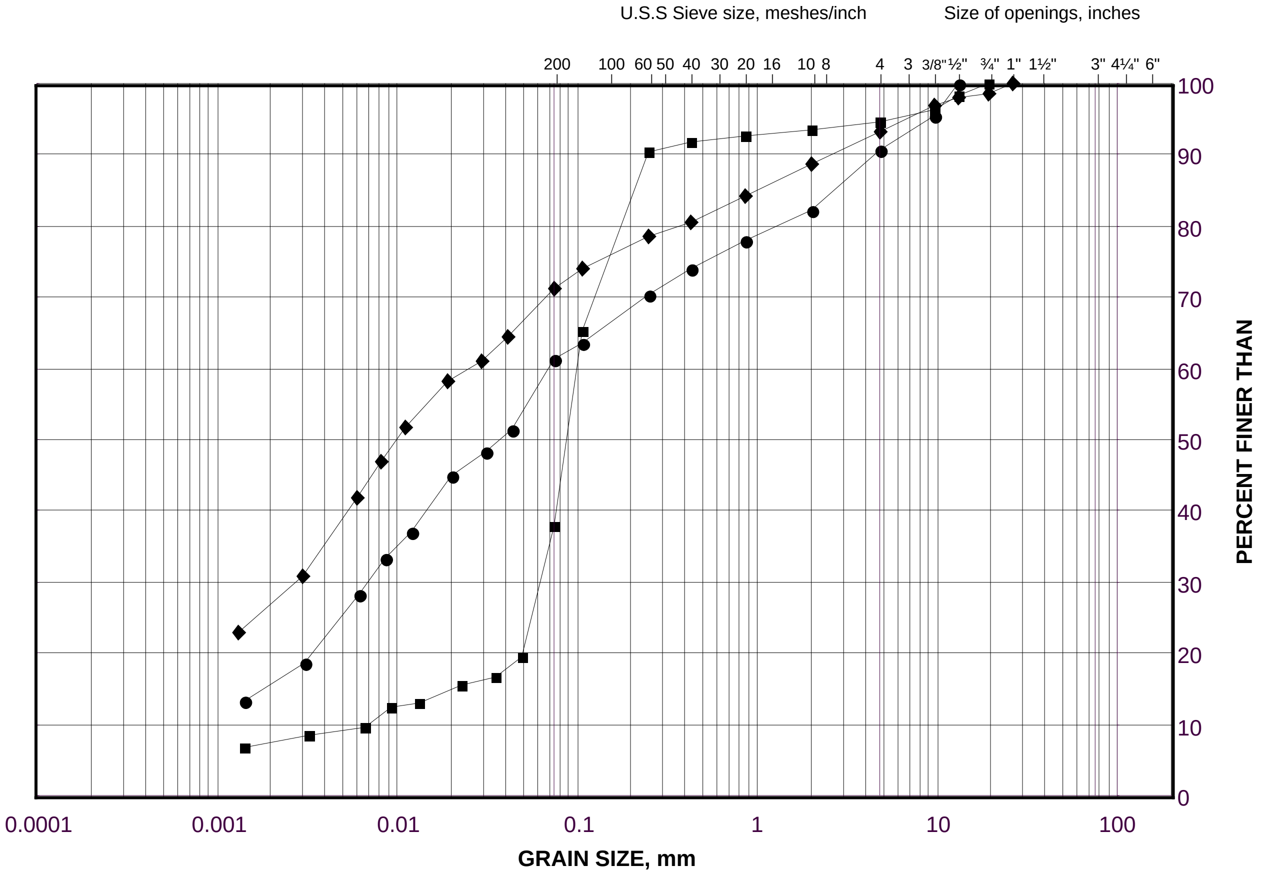
Project No. 1662333

Checked By: SMM

GRAIN SIZE DISTRIBUTION

Silty Sand to Sandy Clayey Silt to Sandy Silty Clay (Till)

FIGURE B-6



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	NW3-1	10	87.1
■	AR-2	3	86.7
◆	AR-2	5	85.1

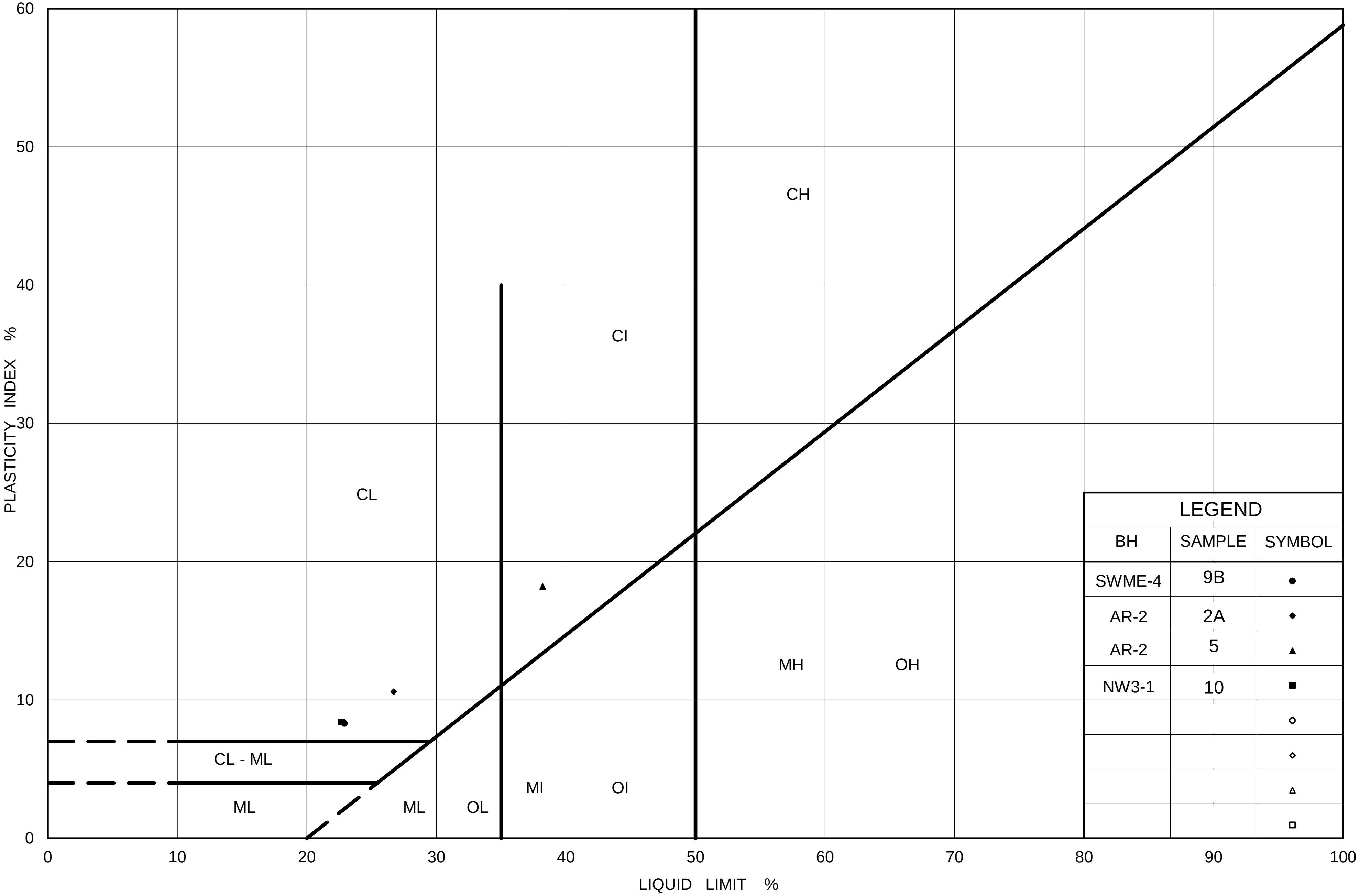
Project Number: 1662333

Checked By: SMM

Golder Associates

Date: 07-Dec-18

Oct 75, FF-S-21



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PLASTICITY CHART
Sandy Clayey Silt to Sandy Silty Clay (Till)

Figure No. B-7

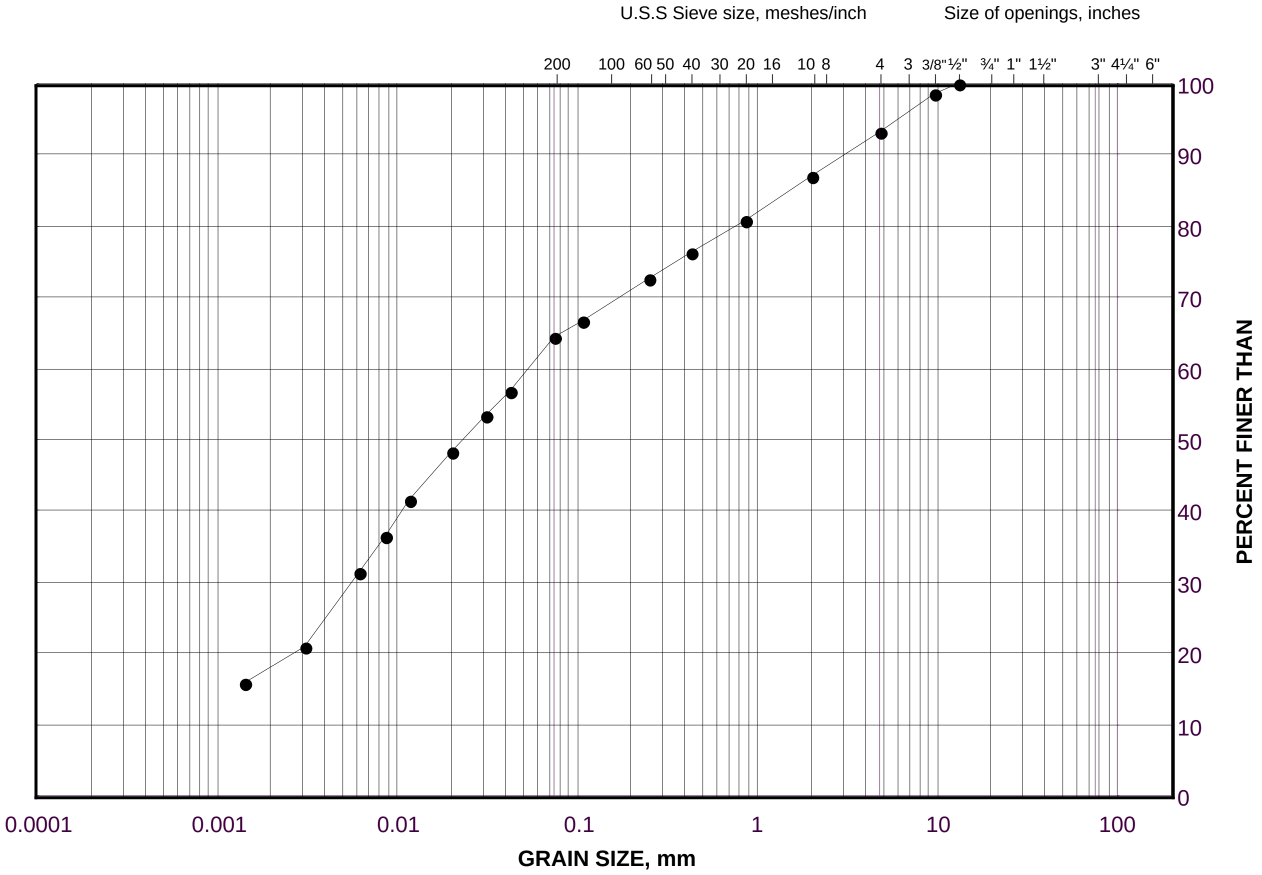
Project No. 1662333

Checked By: SMM

GRAIN SIZE DISTRIBUTION

Sandy Clayey Silt (Residual Soil)

FIGURE B-8



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	AR-1	10B	86.9

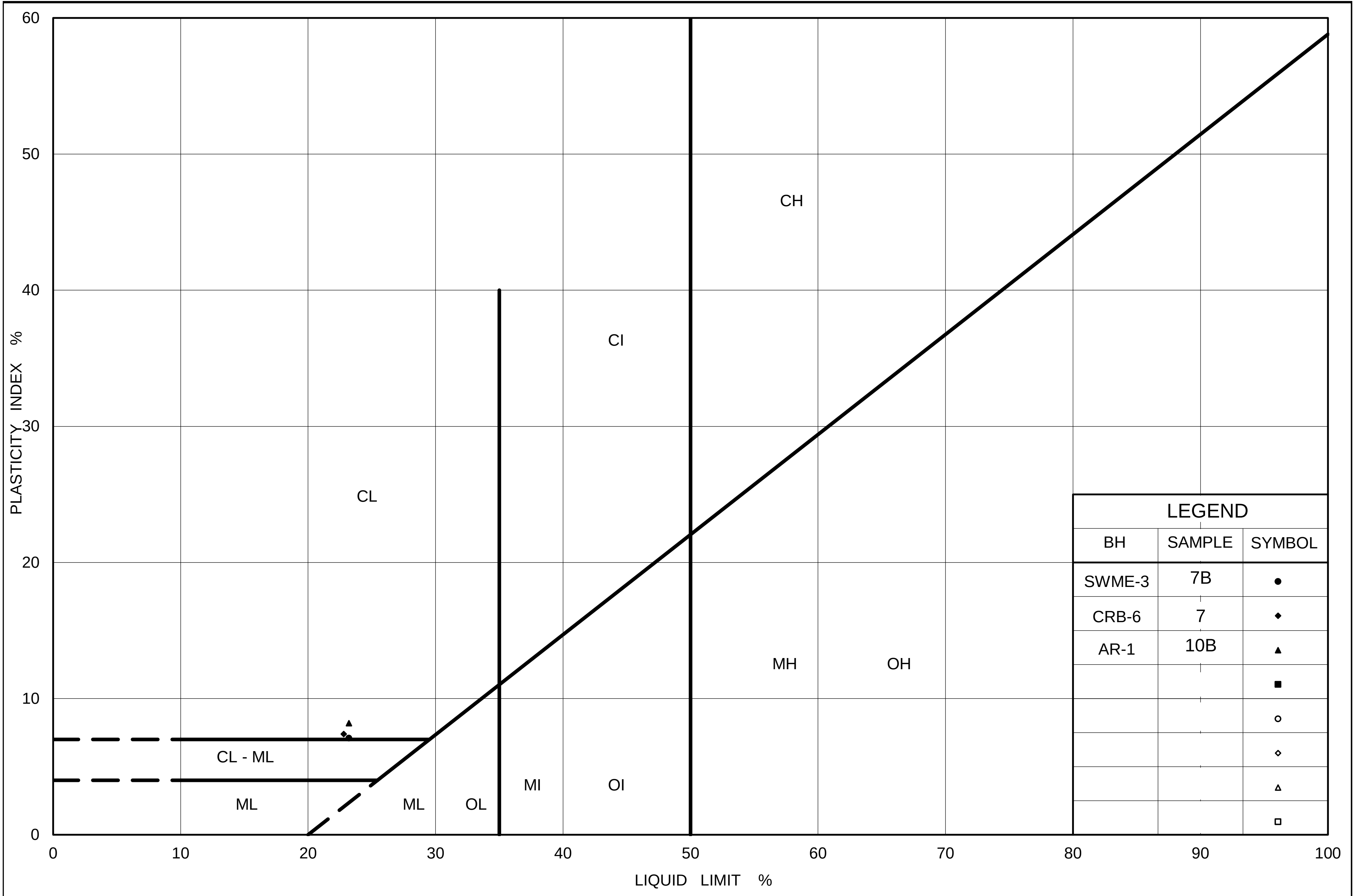
Project Number: 1662333

Checked By: SMM

Golder Associates

Date: 06-Dec-18

Oct 75, FF-S-21



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PLASTICITY CHART
Sandy Clayey Silt to Clayey Silt (Residual Soil)

Figure No. B-9

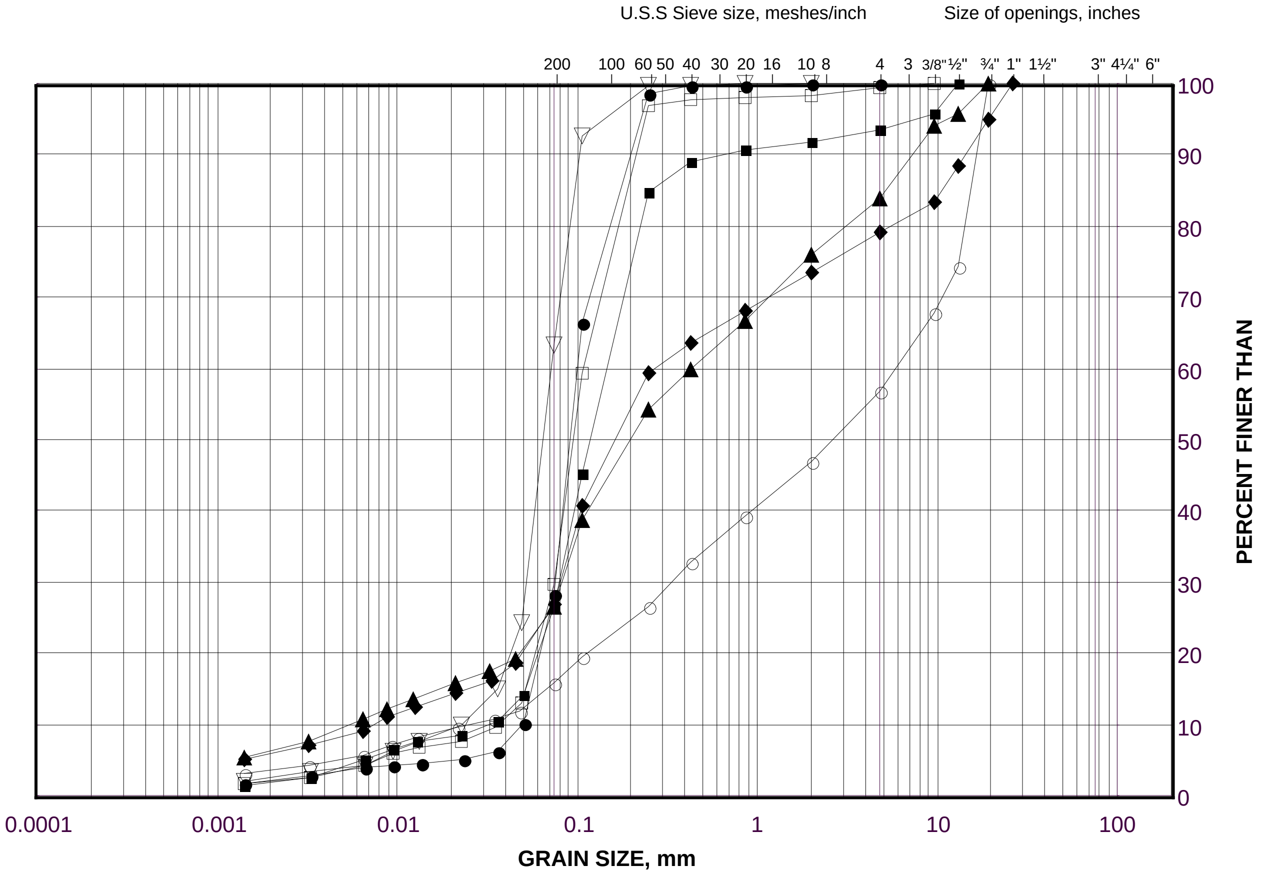
Project No. 1662333

Checked By: SMM

GRAIN SIZE DISTRIBUTION

Silt and Sand to Silty Sand to Gravelly Silty Sand to Sand and Gravel (Fill)

FIGURE B-10A



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	NW3-1	2	95.4
■	CRB-6	2	90.7
◆	PED-01	3	94.5
▲	NW3-2	3	93.5
▽	PED-01	6	92.2
❍	NW3-2	6	91.2
□	NW3-2	7	88.9

Project Number: 1662333

Checked By: SMM

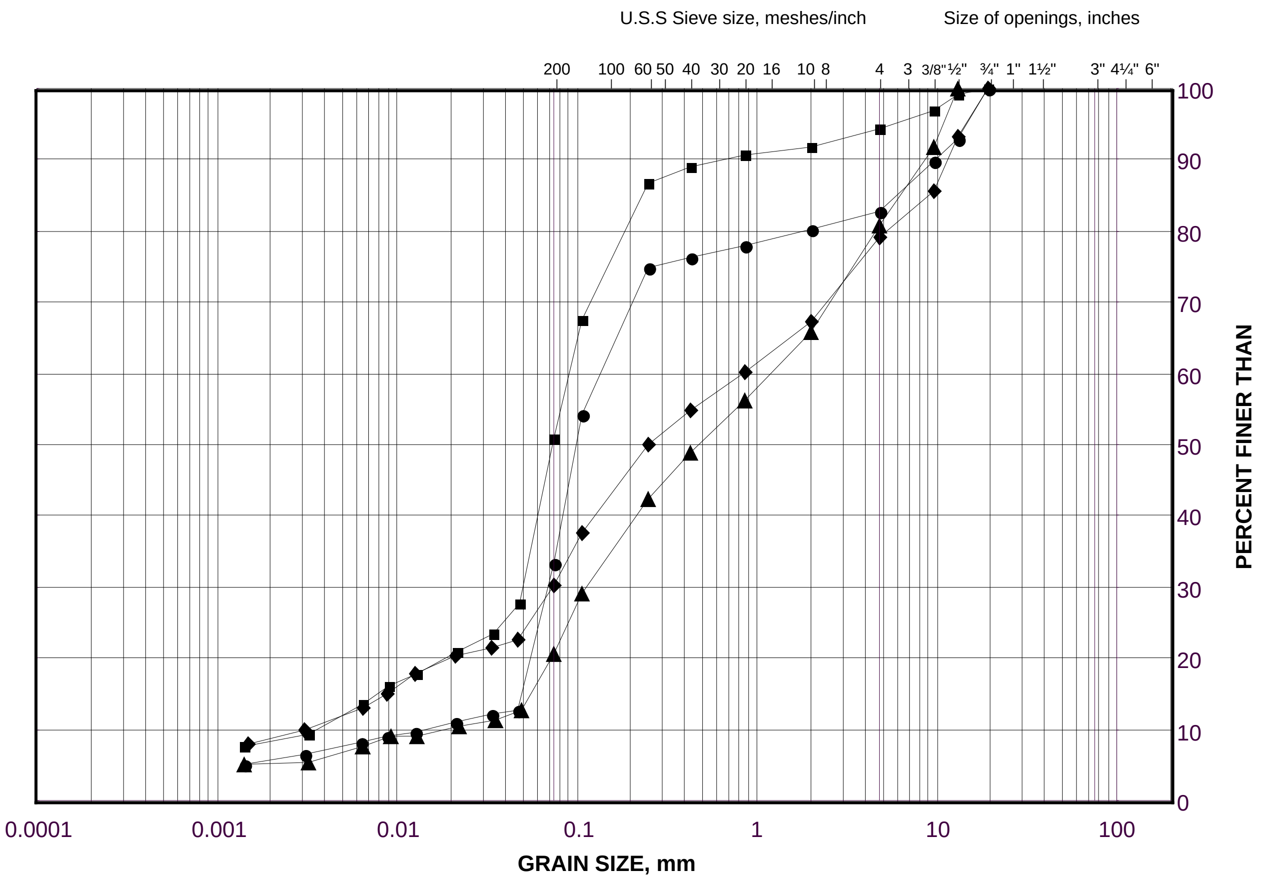
Golder Associates

Date: 12-Feb-19

GRAIN SIZE DISTRIBUTION

Gravelly Clayey Silt with Sand to Silt and Sand to Silty Sand to Sand (Fill)

FIGURE B-10B



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	NW3-3	2	89.5
■	S3	3	88.2
◆	S2	5	91.6
▲	S2	6A	90.9

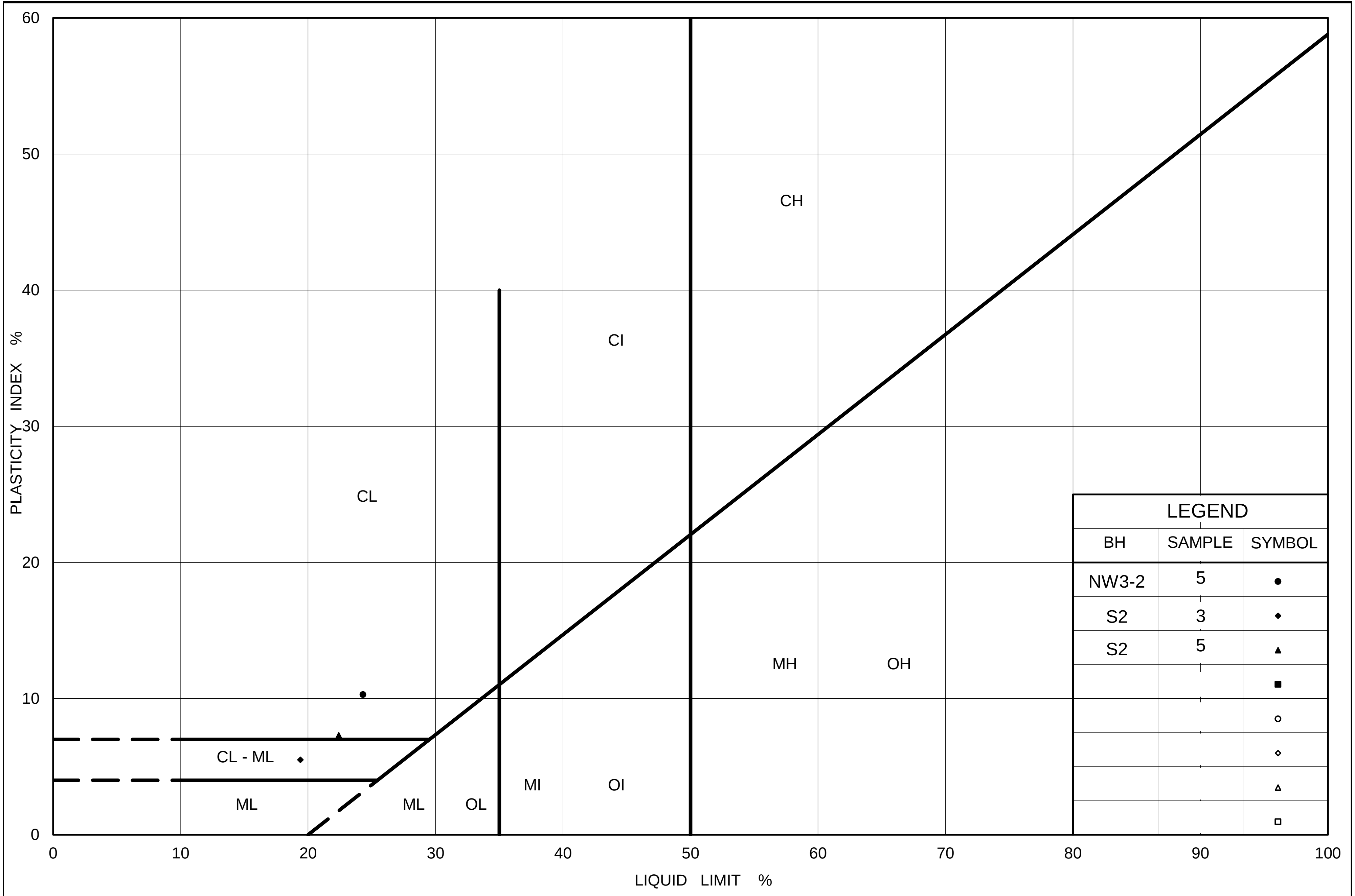
Project Number: 1662333

Checked By: SMM

Golder Associates

Date: 12-Feb-19

Oct 75, FF-S-21



Ministry of Transportation

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PLASTICITY CHART

Sandy Clayey Silt to Gravelly Clayey Silt with Sand (Fill)

Figure No. B-11

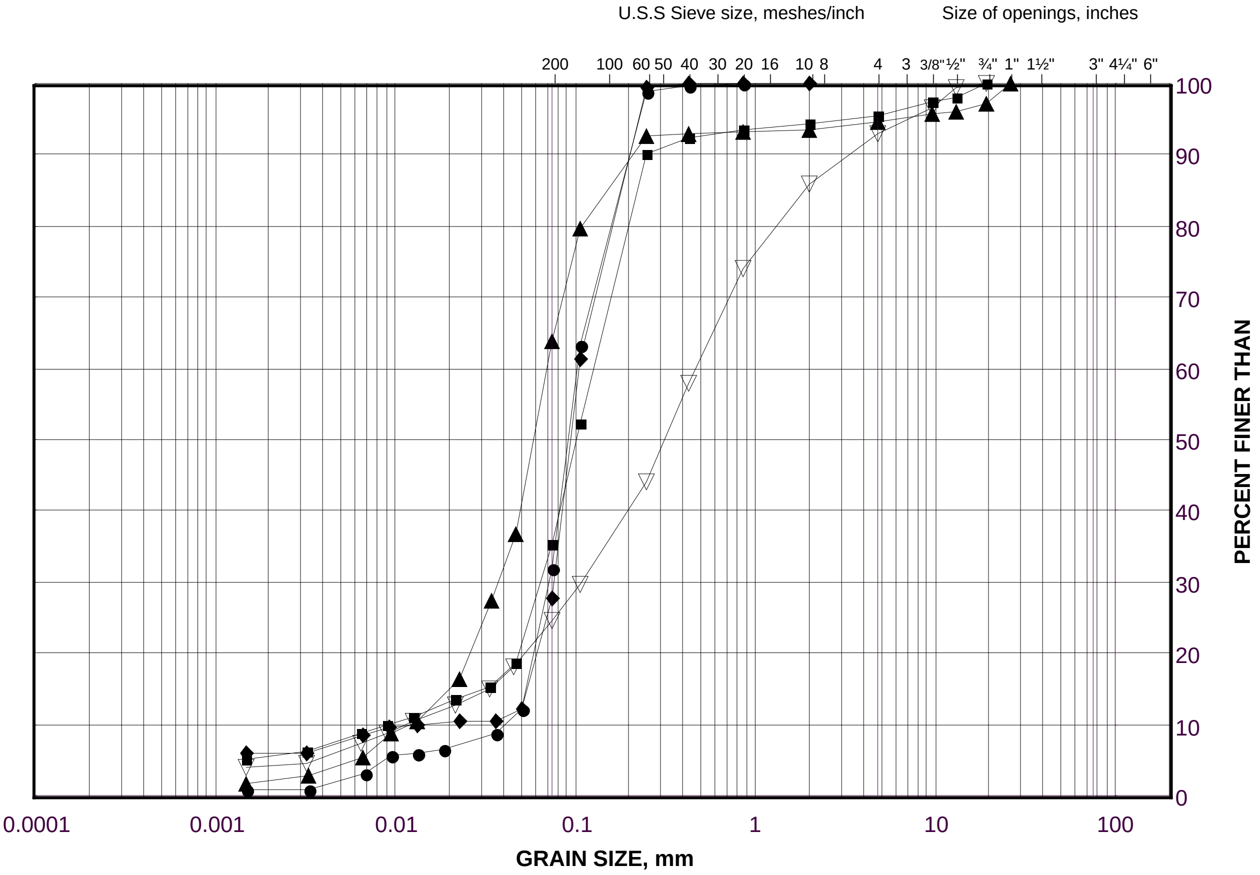
Project No. 1662333

Checked By: SMM

GRAIN SIZE DISTRIBUTION

Silt and Sand to Silty Sand

FIGURE B-12



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	NW3-1	4	93.9
■	CRB-5	6	75.1
◆	S2	8	88.5
▲	NW3-1	8	90.1
▽	CRB-5	9	72.8

Project Number: 1662333

Checked By: SMM

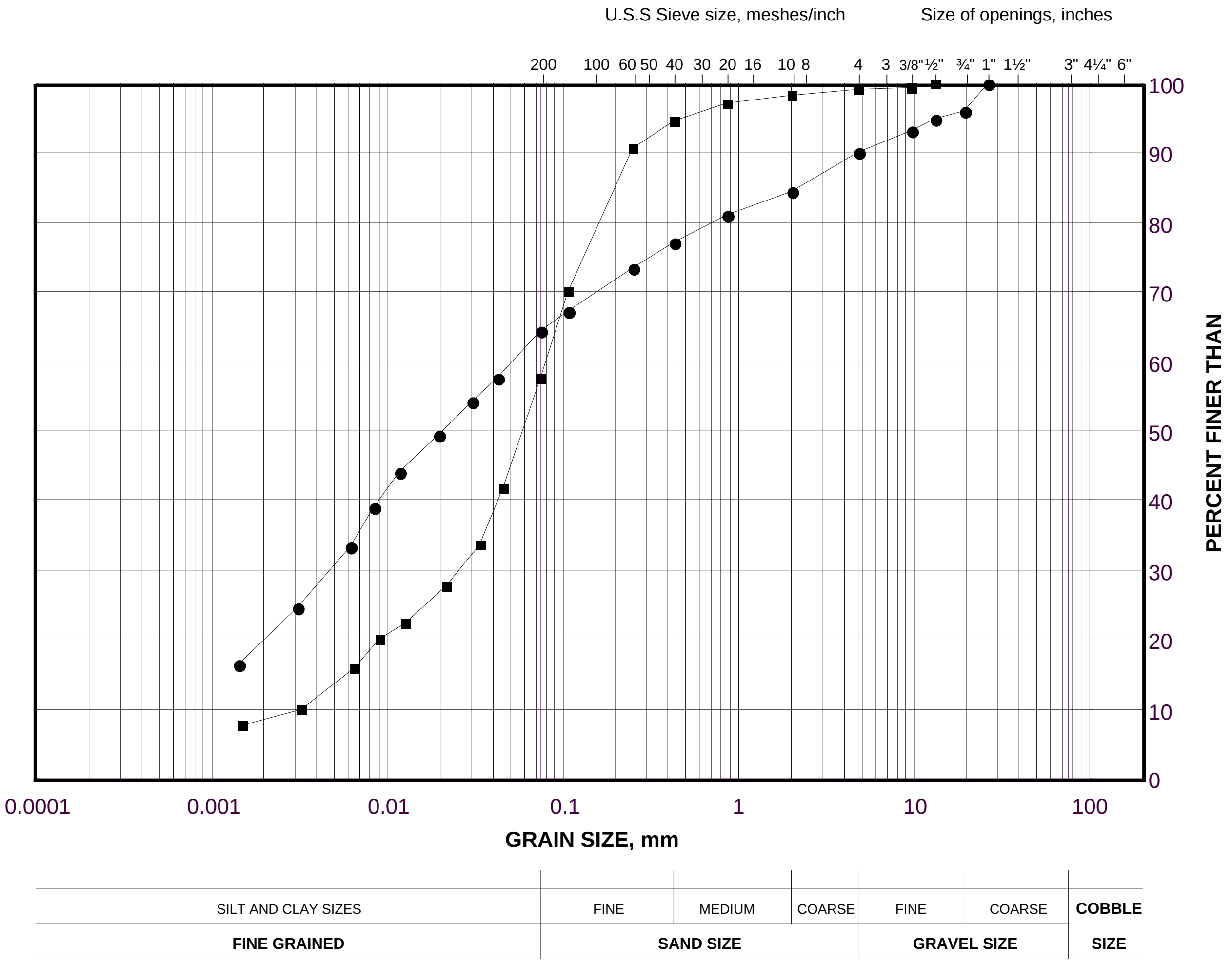
Golder Associates

Date: 08-Feb-19

GRAIN SIZE DISTRIBUTION

Sandy Clayey Silt/Organic Clayey Silt with Sand

FIGURE B-13



LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	CRB-6	6	87.6
■	CRB-5	8A	73.7

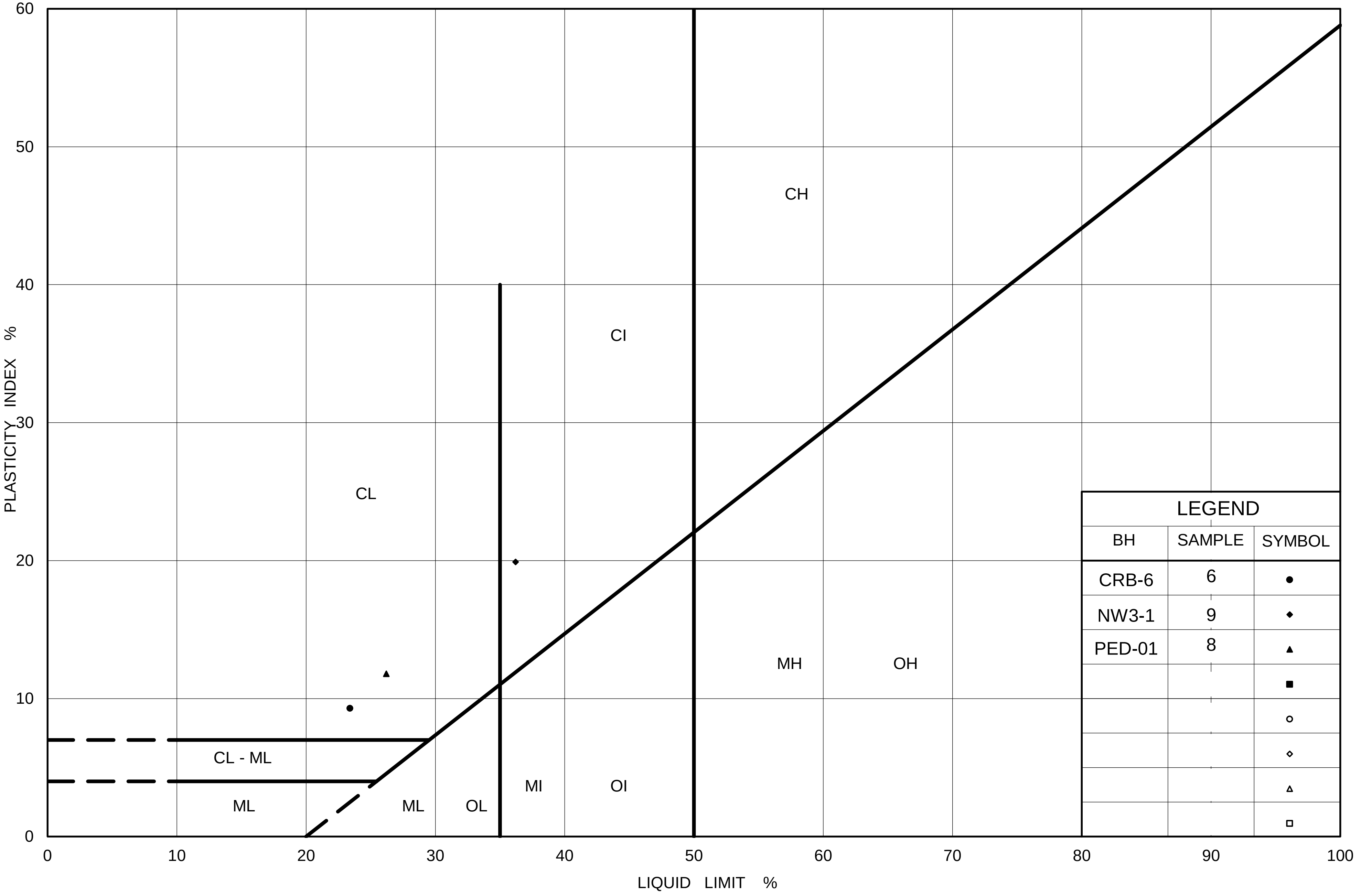
Project Number: 1662333

Checked By: SMM

Golder Associates

Date: 12-Feb-19

Oct 75, FF-S-21



Ministry of Transportation

Ontario

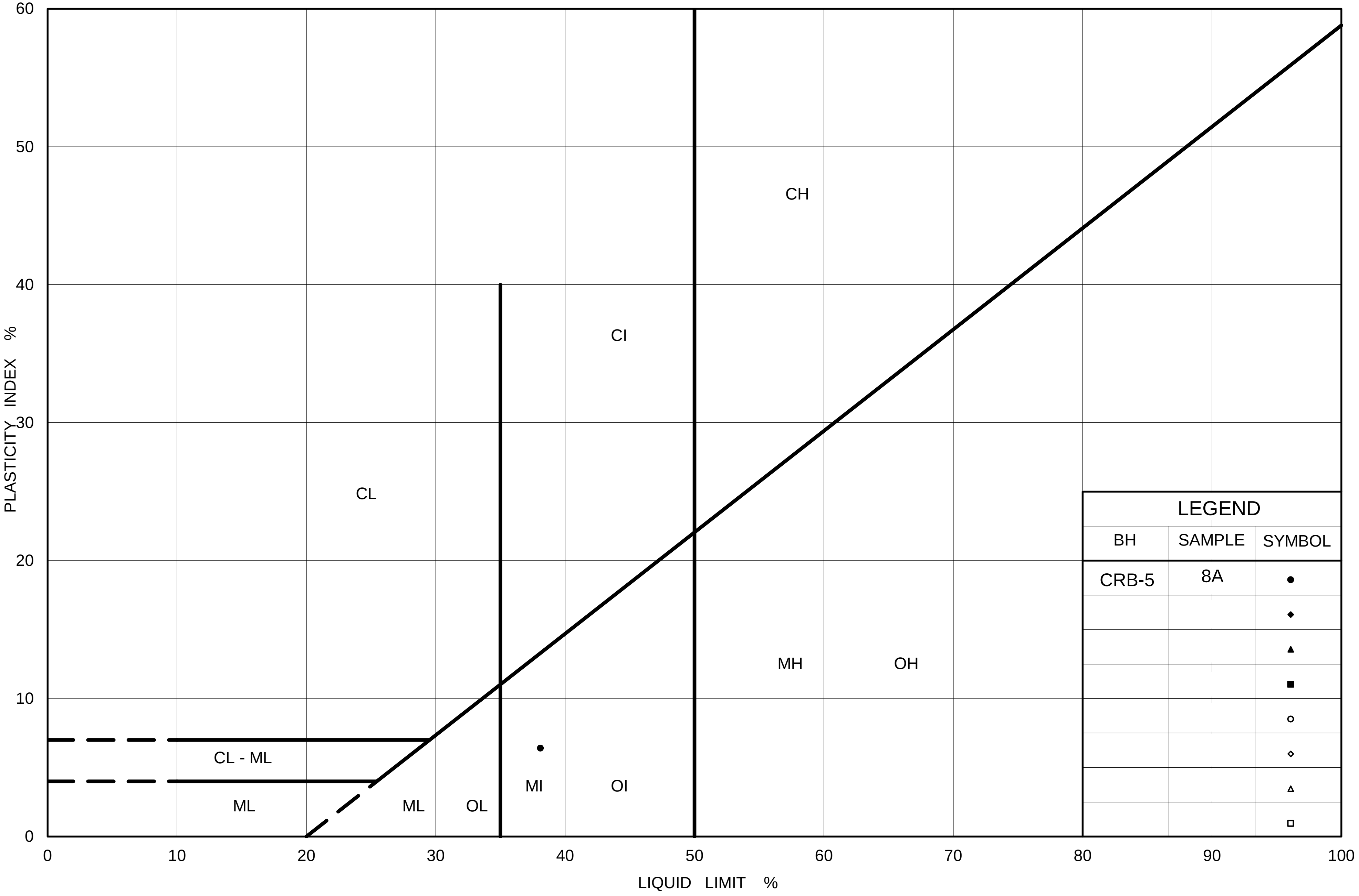
PLASTICITY CHART
Sandy Clayey Silt to Silty Clay

Figure No. B-14

Project No. 1662333

Checked By: SMM

Oct 75, FF-S-21



Ministry of Transportation

Ontario

PLASTICITY CHART
Organic Clayey Silt with Sand

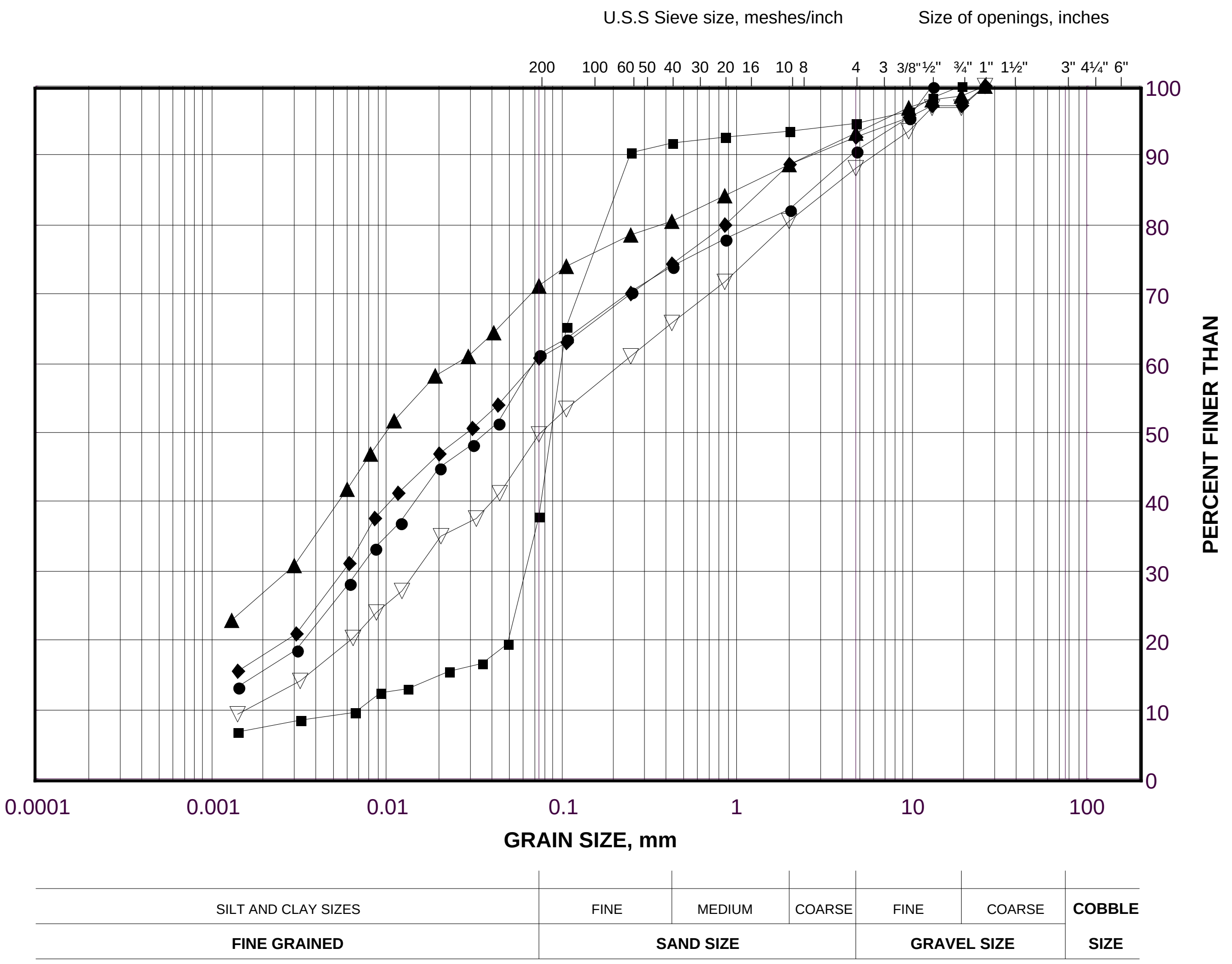
Figure No. B-15

Project No. 1662333

Checked By: SMM

GRAIN SIZE DISTRIBUTION
Silt and Sand to Sandy Silty Clay (Till)

FIGURE B-16A



LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	NW3-1	10	87.1
■	AR-2	3	86.6
◆	NW3-3	4	88.0
▲	AR-2	5	85.1
▽	NW3-2	9	85.9

Project Number: 1662333

Checked By: SMM

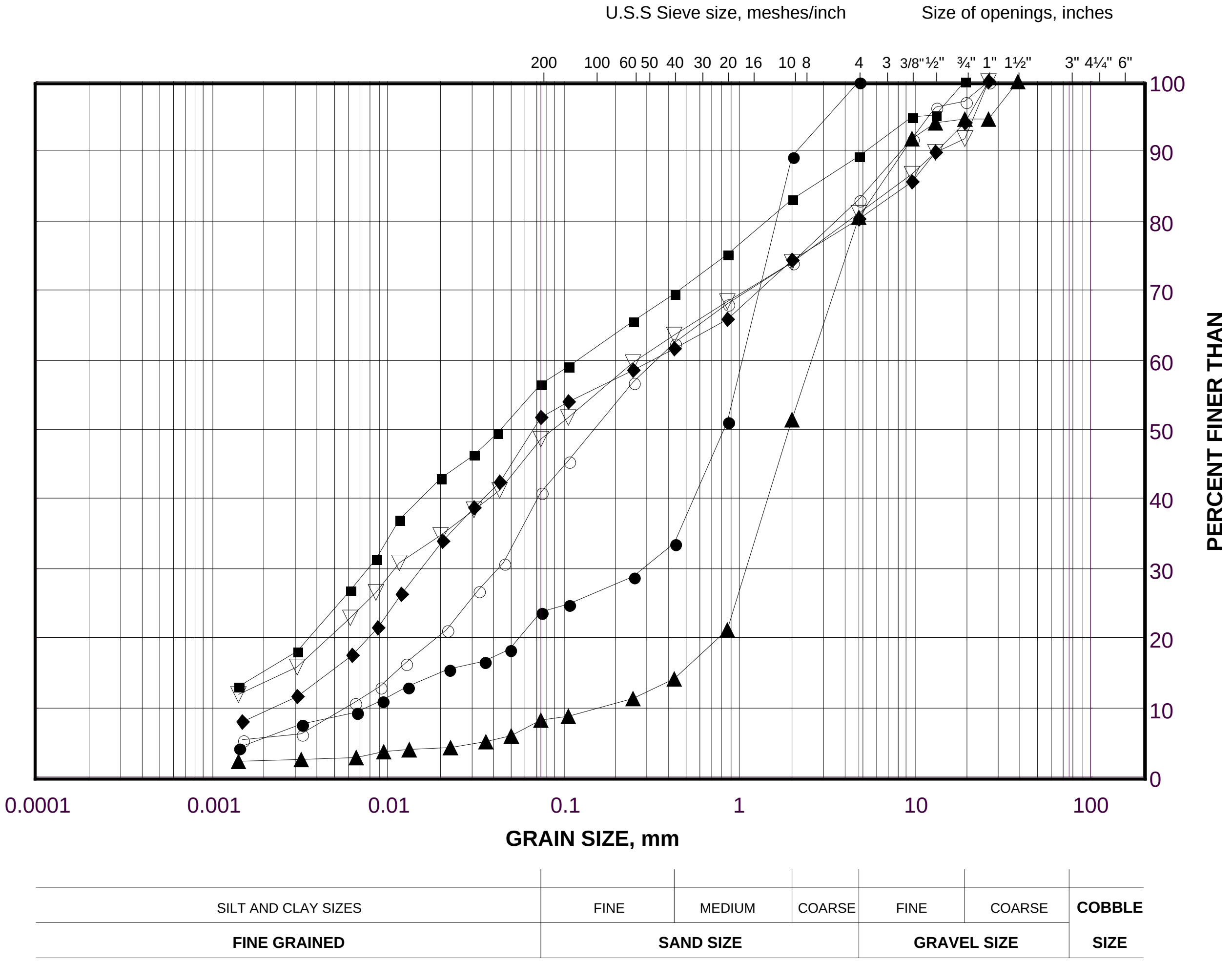
Golder Associates

Date: 12-Feb-19

GRAIN SIZE DISTRIBUTION

Silt and Sand to Clayey Silt with Sand to Sandy Silty Clay (Till)

FIGURE B-16B



LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	S3	10	79.2
■	PED-01	10A	86.9
◆	S2	12	82.6
▲	S3	13	74.4
▽	S3	5	86.7
❍	S3	7	83.8

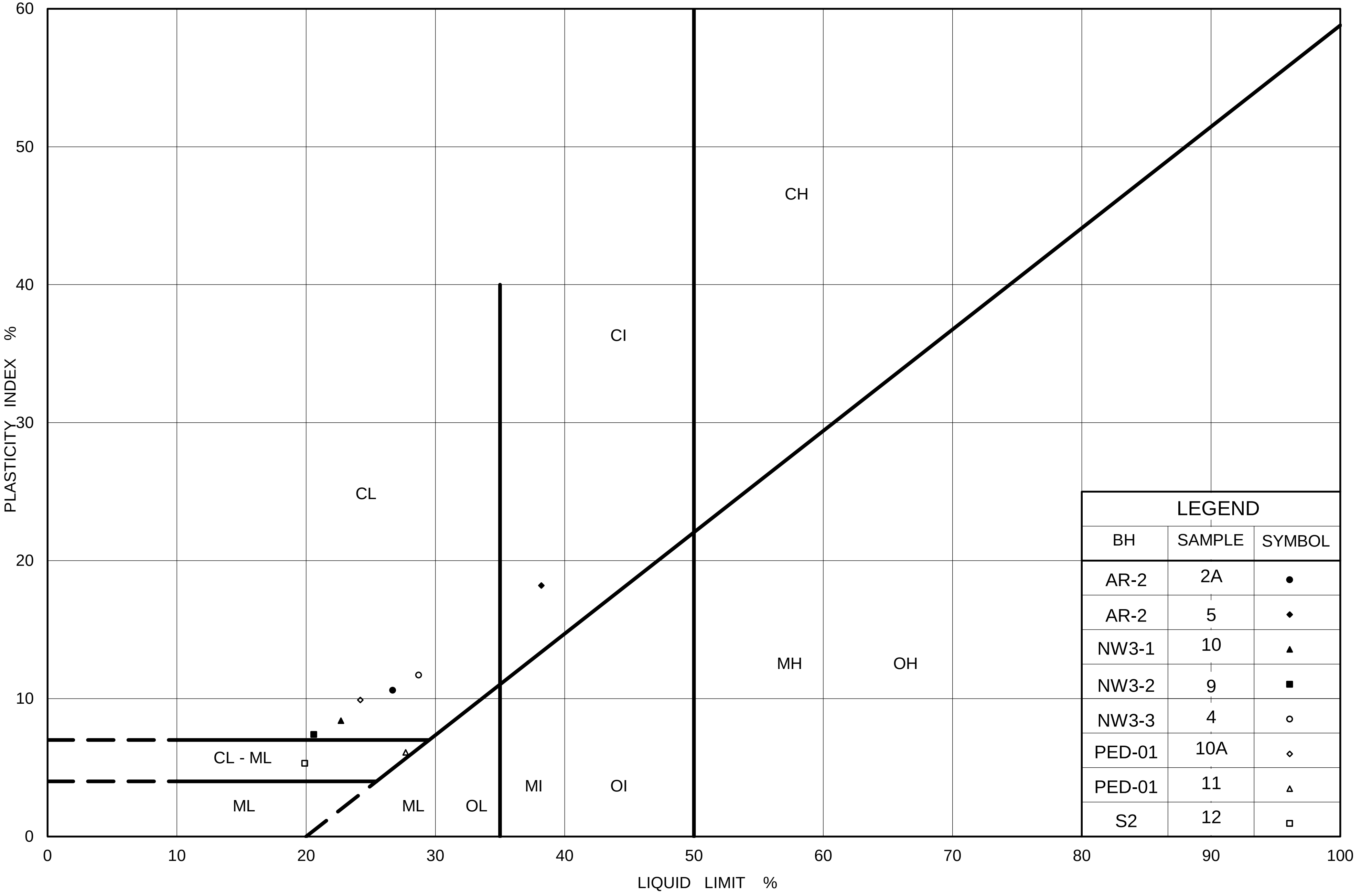
Project Number: 1662333

Checked By: SMM

Golder Associates

Date: 01-Mar-19

Oct 75, FF-S-21



Ministry of Transportation

Ontario

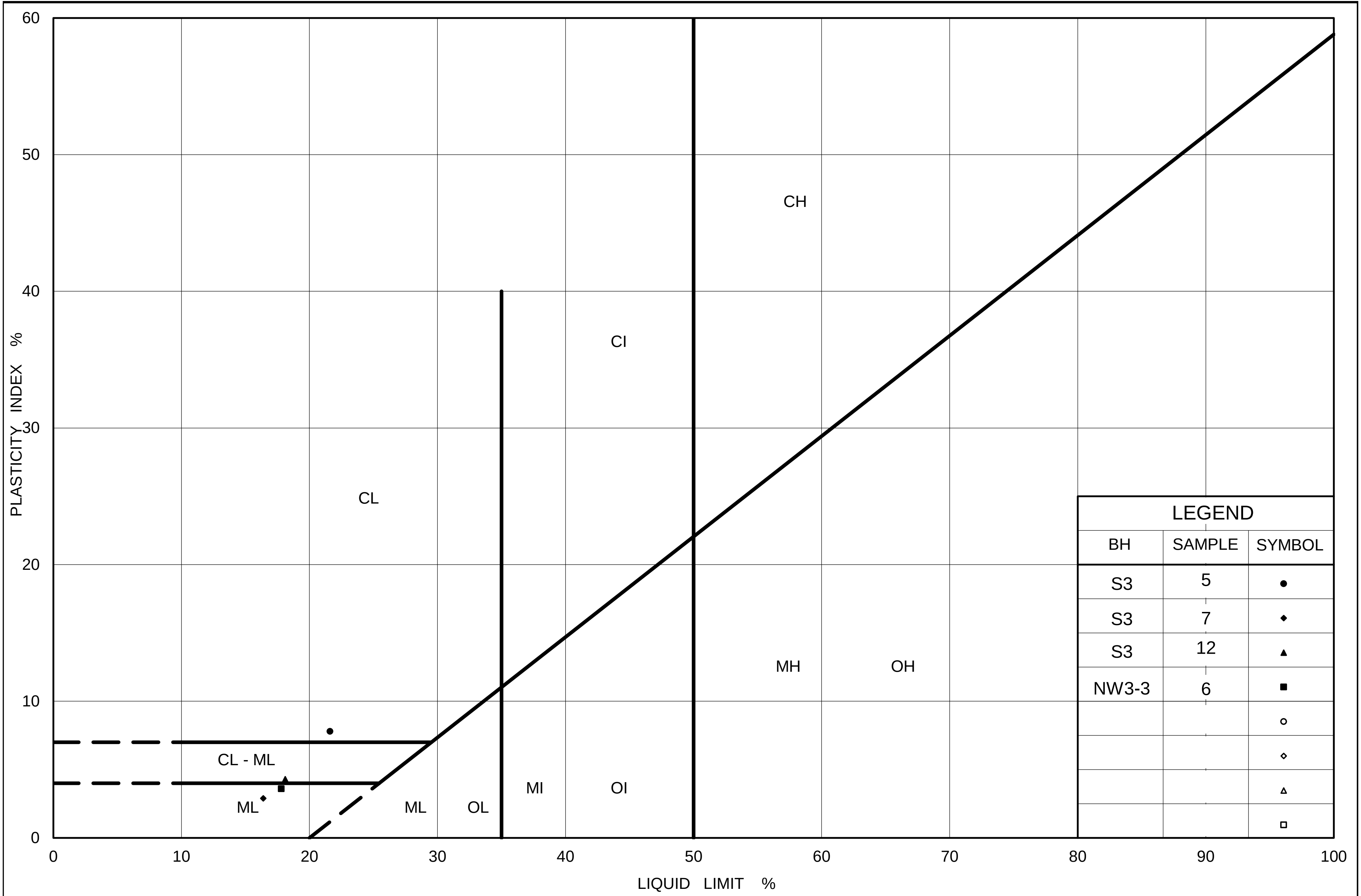
PLASTICITY CHART
Clayey Silt with Sand to Sandy Silty Clay (Till)

Figure No. B-17A

Project No. 1662333

Checked By: SMM

Oct 75, FF-S-21



LEGEND

BH	SAMPLE	SYMBOL
S3	5	●
S3	7	◆
S3	12	▲
NW3-3	6	■
		○
		◇
		△
		□

Ministry of Transportation

Ontario

PLASTICITY CHART
Silt and Sand/Clayey Silt with Sand (Till)

Figure No. B-17B

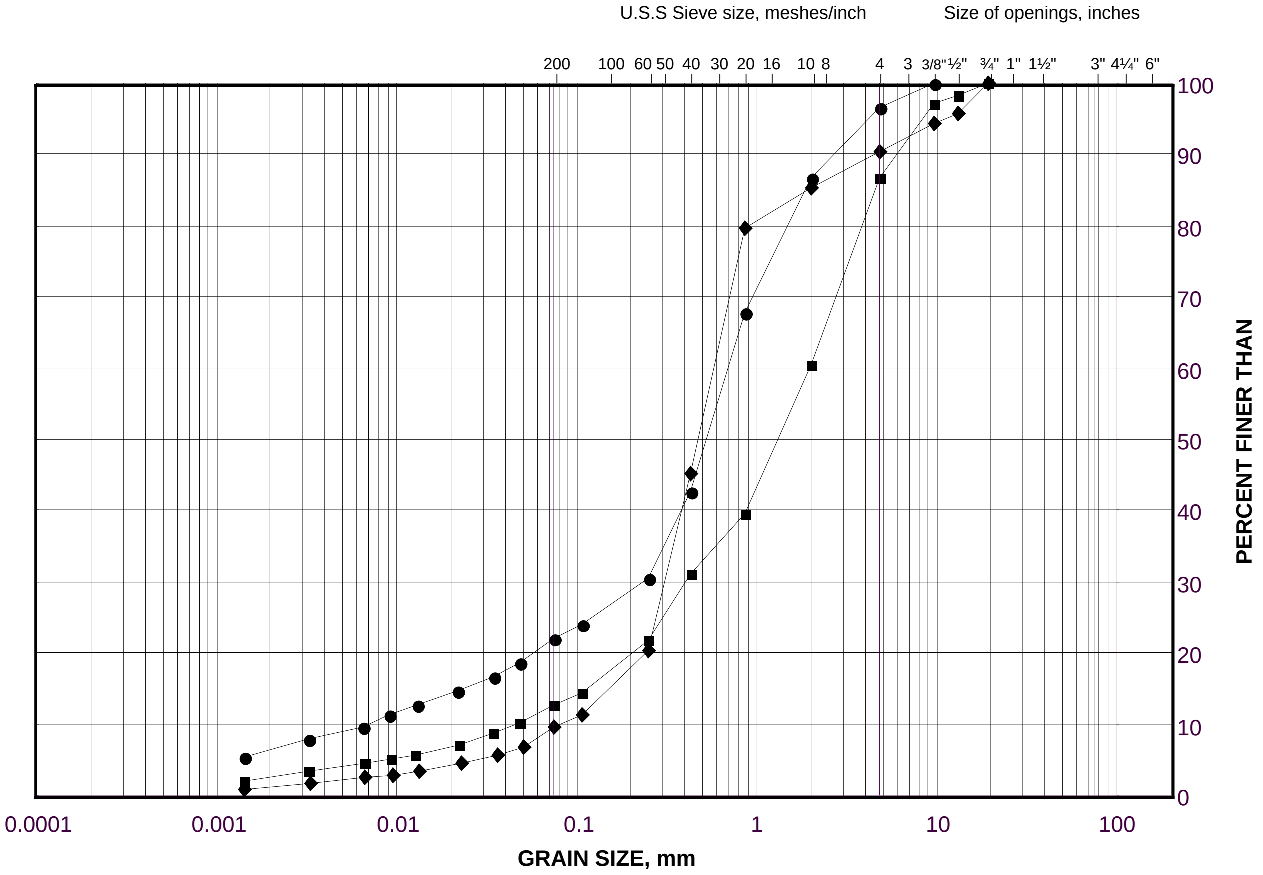
Project No. 1662333

Checked By: SMM

GRAIN SIZE DISTRIBUTION

Sand

FIGURE B-18



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	PED-01	12	83.8
■	PED-01	14	80.8
◆	PED-01	17	76.2

Project Number: 1662333

Checked By: SMM

Golder Associates

Date: 01-Mar-19

GEOMECHANICA

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

November 22, 2017

Mr. David Marmor
Golder Associates Ltd.
6925 Century Avenue, Suite #100
Mississauga, Ontario
Canada L5N 7K2

Re: UCS + E testing
(Golder Project No. 166233)

Dear Mr. Marmor:

On November 3, 2017 four (4) HQ-sized core samples were received by Geomechanica Inc. via courier. These samples were identified as being from boreholes drilled as part of Golder project 166233 (denoted as QEW/Credit River UCS samples). A uniaxial compressive strength (UCS) specimen was prepared and tested from each of these samples (4 tests total).

Details regarding the steps of specimen preparation and testing along with the test results and specimen photographs before and after testing are presented in the accompanying laboratory report.

Sincerely,

Giovanni Grasselli Ph.D., P. Eng.

Geomechanica Inc.
Tel: (647) 478-9767
Email: giovanni.grasselli@geomechanica.com

Rock Laboratory Testing Results

A report submitted to:
David Marmor
Golder Associates Ltd.
6925 Century Avenue, Suite #100
Mississauga, Ontario
Canada L5N 7K2

Prepared by:
Bryan Tatone, PhD
Omid Mahabadi, PhD
Giovanni Grasselli, PhD, PEng

Geomechanica Inc
#300-90 Adelaide St W
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M5H 3V9 Canada
Tel: +1-647-478-9767
info@geomechanica.com

November 22, 2017
Project number: 1662333

Abstract

This document summarizes the results of 4 uniaxial compression tests on HQ-sized core samples for Golder Project 1662333. Results including uniaxial compressive strength (UCS) and Young's modulus along with photographs of samples before and after testing are presented.

In this document:

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 Overview

This report summarizes the results of laboratory testing of 4 uniaxial compression tests on HQ-sized core samples for Golder Project 1662333. The tests were performed in Geomechanica's laboratory in Oakville, Ontario, Canada using a 1.3 MN capacity Forney compression testing machine (Figure 1). The specimens were loaded with a nearly constant axial displacement rate of 0.150 mm/min.The specimen preparation and testing procedure included the following:

1. Unwrapping of the core samples, inspecting them for damage, and re-wrapping them in electrical tape to minimize disturbance during subsequent specimen preparation.
2. Diamond cutting of core samples to obtain cylindrical specimens with an appropriate length (length:diameter = 2:1) and nearly parallel end faces.
3. Surface grinding of specimens to obtain flat (within ±0.025 mm) and parallel end faces (within 0.25°).
4. Placing each specimen into the loading frame, applying a 0.5-1.0 kN axial load, removing the electrical tape, and subsequently increasing the axial load gradually to cause rupture while continuously recording axial force and axial deformation to determine peak strength (UCS) and (tangent) Young's modulus.

Figure 1: UCS Test setup.

2 Results

The results of the tests are summarized in Table 1. The corresponding stress-strain curves for the uniaxial compression 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% of the peak strength.

Table 1: Summary of laboratory test results.

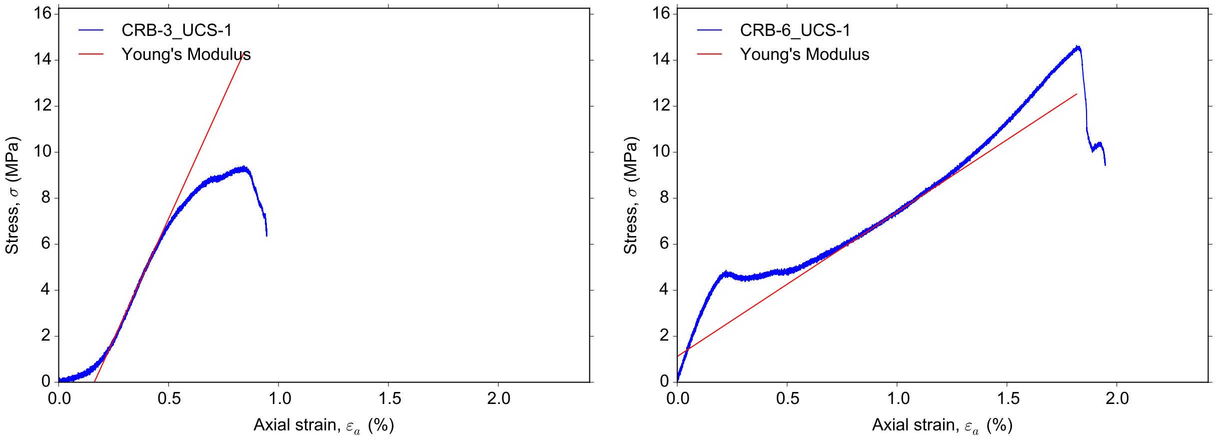
Sample	Depth (m)	Bulk density ρ (g/cm^3)	UCS (MPa)	Young's Modulus E (GPa)	Notes
CRB-3, UCS-1	11.44 - 11.66	2.61	9.4	2.10	[1]
CRB-6, UCS-1	6.06 - 6.17	2.17	14.6	0.63	[1,2]
CRB-7, UCS-1	9.21 - 9.369	2.59	15.5	0.65	[1,2]
CRB-7, UCS-3	12.11 - 12.36	2.59	7.4	1.28	
Mean		2.49	11.7	1.2	
Standard Deviation		0.18	3.4	0.6	

[1] Specimen emitted fresh pore water upon loading

[2] length:diameter ratio $<$ 2:1.

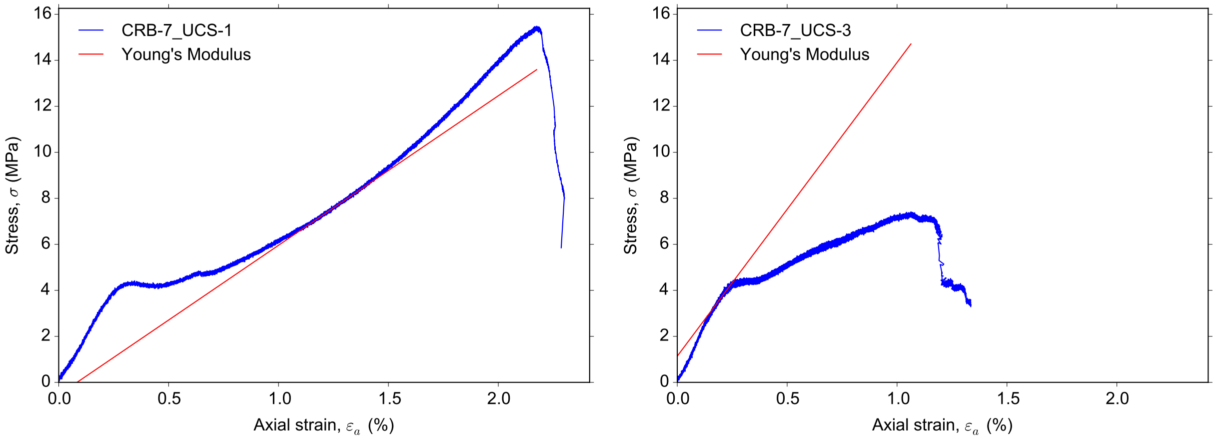
2.1 Specimen photographs

Photographs of the specimens before and after testing are presented in Figure 3.



(a) CRB-3, UCS-1

(b) CRB-6, UCS-1



(c) CRB-7, UCS-1

(d) CRB-7, UCS-3

Figure 2: Measured stress-strain curves.

CRB-3, UCS-1
11.44 m – 11.66 m

CRB-6, UCS-1
6.06 m – 6.17 m

CRB-7, UCS-1
9.21 m – 9.36 m

CRB-7, UCS-3
12.11 m – 12.36 m

Figure 3: Photographs of specimens prior to testing.

GEOMECHANICA

Geomechanica Inc.
Suite 900 – 390 Bay St.
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April 09, 2018

Mr. David Marmor
Golder Associates Ltd.
6925 Century Avenue, Suite #100
Mississauga, Ontario
Canada L5N 7K2

Re: UCS + E testing
(Golder Project No. 1662333)

Dear Mr. Marmor:

On March 27, 2018 three (3) NQ-sized and eight (8) HQ-sized core samples were received by Geomechanica Inc. via drop-off by Golder personnel. These samples were identified as being from boreholes drilled as part of Golder project. A uniaxial compressive strength (UCS) specimen was prepared and tested from each of these samples (11 tests total).

Details regarding the steps of specimen preparation and testing along with the test results and specimen photographs before and after testing are presented in the accompanying laboratory report.

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:
David Marmor
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
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M5H 2Y2 Canada
Tel: +1-647-478-9767
info@geomechanica.com

April 9, 2018
Project number: 1662333

Abstract

This document summarizes the results of 11 uniaxial compression tests on a combination of NQ and HQ core samples. Results, including uniaxial compressive strength (UCS) and Young's modulus, along with photographs of test specimens before and after testing are presented.

In this document:

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 Overview

This report summarizes the results of 11 uniaxial compression tests. The specimen preparation and testing procedure included the following:

1. Unwrapping of the core samples, inspecting them for damage, and re-wrapping them in electrical tape to minimize disturbance during subsequent specimen preparation.
2. Diamond cutting of core samples to obtain cylindrical specimens with an appropriate length (length:diameter = 2:1) and nearly parallel end faces.
3. Surface grinding of specimens to obtain flat (within ± 0.025 mm) and parallel end faces (within $0.25°$).
4. Placing each specimen into the loading frame, applying a 0.5-1.0 kN axial load, removing the electrical tape, and axial loading at a constant displacement rate to rupture while continuously recording axial force and axial deformation to determine the peak strength (UCS) and (tangent) Young's modulus (E).

2 Results

The results of the tests are summarized in Table 1. The corresponding stress-strain curves for the uniaxial compression tests are presented in Figure 1 to 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.

Table 1: Summary of laboratory test results.

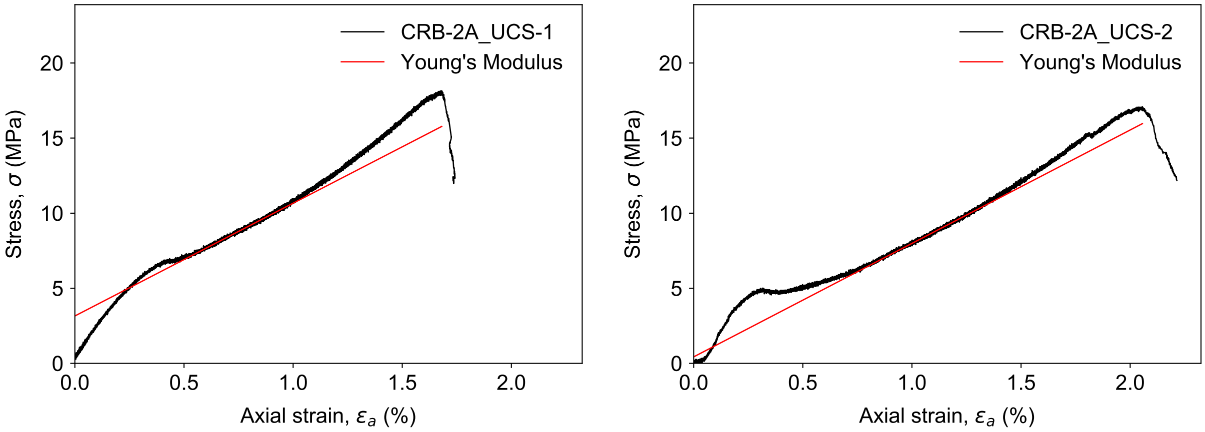
Sample	Rock (m)	Depth type	Bulk density ρ (g/cm^3)	UCS (MPa)	Young's Modulus E (GPa)	Notes
CRB-2A, UCS-1	Shale	4.31 - 4.46	2.59	18.2	0.75	1, 2
CRB-2A, UCS-2	Shale	4.92 - 5.15	2.60	17.1	0.76	1
CRB-3C, UCS-3	Limestone	7.87 - 7.98	2.61	114.1	22.91	2, 3
CRB-2, UCS-2	Shale	7.75 - 7.92	2.58	11.2	0.83	1
CRB-2, UCS-3	Shale	11.37 - 11.52	2.61	13.0	2.19	3
CRB-3A, UCS-3	Shale	10.19 - 10.33	2.60	8.9	0.48	1, 4 - 2 limestone layers $\tilde{5}$-10 mm thick
CRB-3A, UCS-5	Shale	12.99 - 13.28	2.62	16.9	0.67	1
CRB-4, UCS-3	Shale	13.62 - 13.80	2.61	18.6	0.84	1
CRB-5, UCS-2	Shale	13.68 - 13.95	2.61	15.5	0.61	1
CRB-5A, UCS-2	Shale	12.43 - 12.57	2.60	14.2	0.96	1
CRB-5A, UCS-4	Shale	15.34 - 15.57	2.64	22.7	0.93	1

[1] Upon loading specimen emitted pore water

[2] Irregular diameter $>$ 0.5 mm

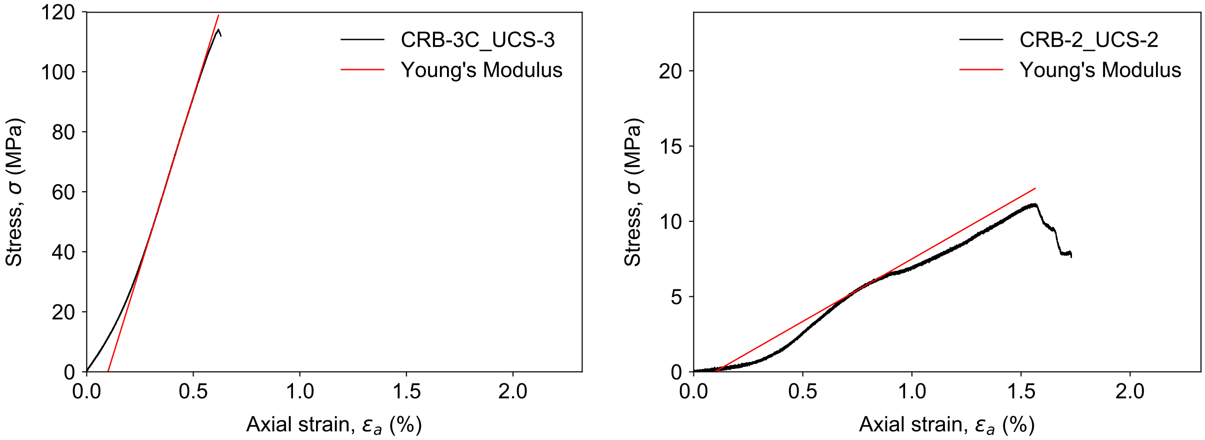
[3] Length:Diameter ratio less than 2

[4] Inter-bedded limestone and shale



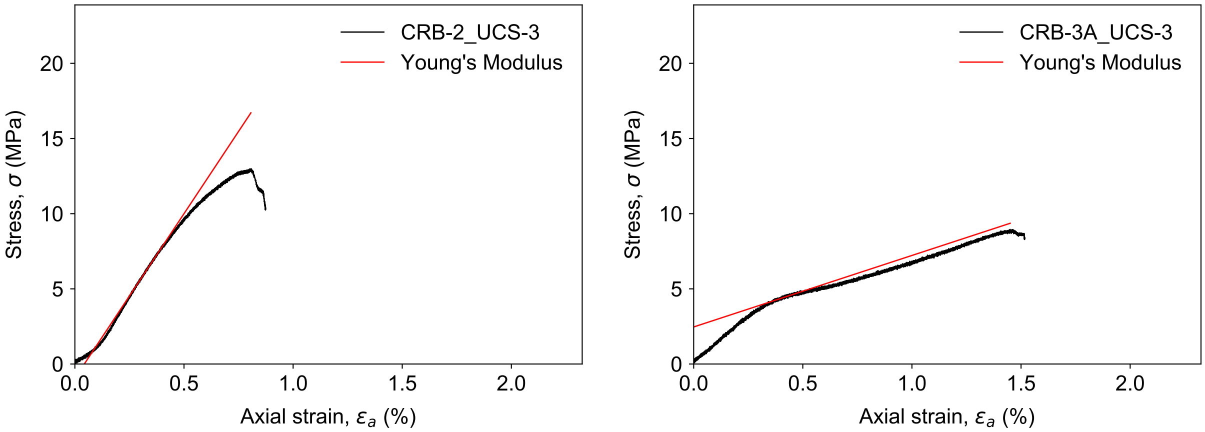
(a) CRB-2A, UCS-1

(b) CRB-2A, UCS-2



(c) CRB-3C, UCS-3

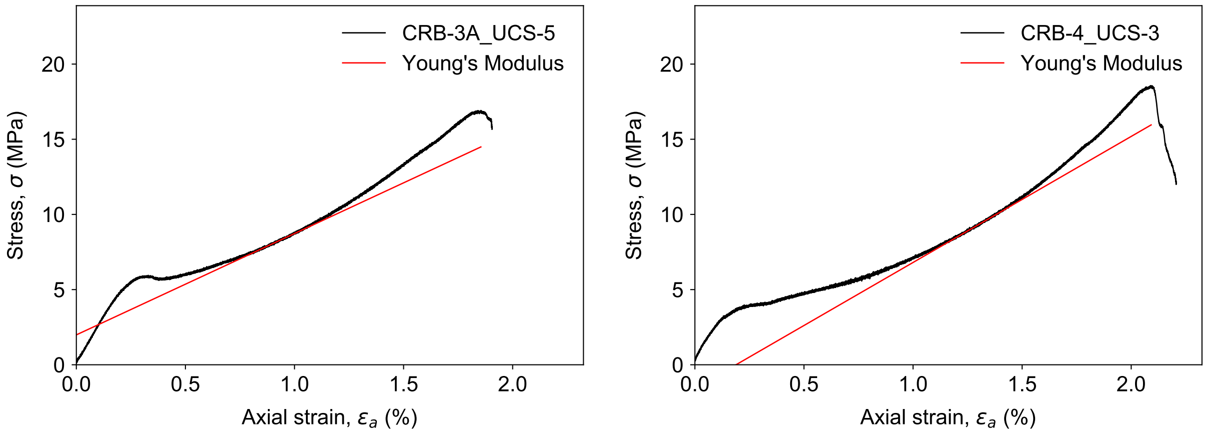
(d) CRB-2, UCS-2



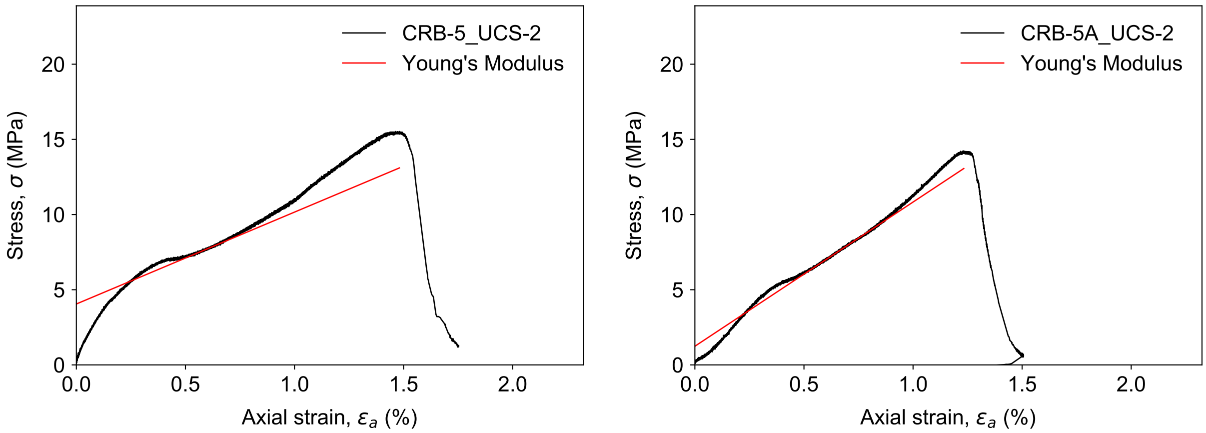
(e) CRB-2, UCS-3

(f) CRB-3A, UCS-3

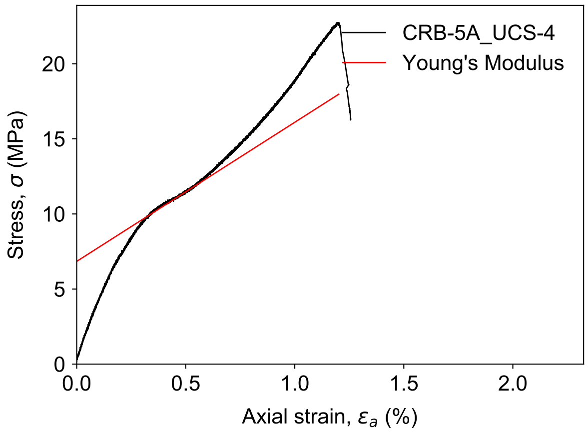
Figure 1: Measured stress-strain curves.



(a) CRB-3A, UCS-5 (b) CRB-4, UCS-3



(c) CRB-5, UCS-2 (d) CRB-5A, UCS-2



(e) CRB-5A, UCS-4

Figure 2: Measured stress-strain curves.

2.1 Specimen photographs

Photographs of the specimens before and after testing are presented in Figure 3 and Figure 4

Figure 3: Photographs of specimens prior to testing.

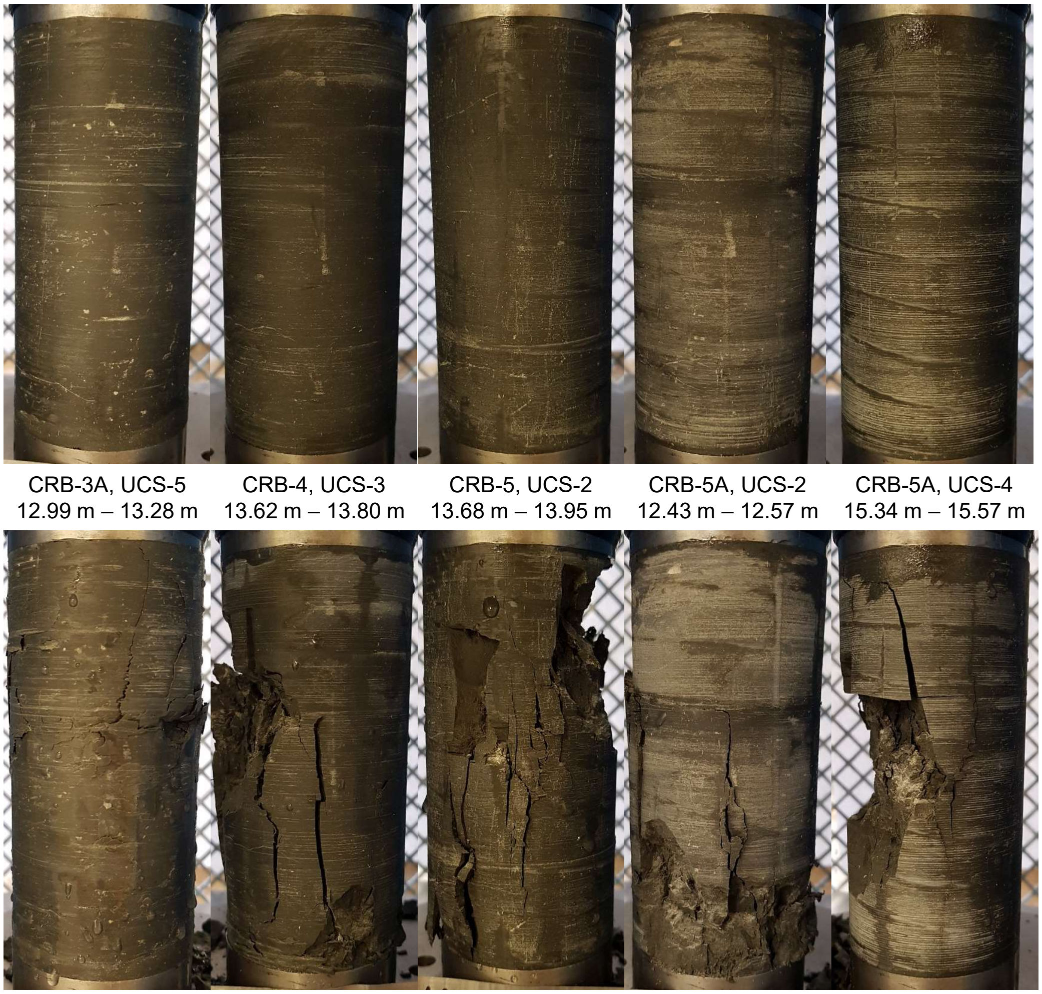


Figure 4: Photographs of failed specimens after testing.

GEOMECHANICA

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

August 27, 2018

Mr. David Marmor
Golder Associates Ltd.
6925 Century Avenue, Suite #100
Mississauga, Ontario
Canada L5N 7K2

Re: UCS only and UCS + E testing
(Golder Project No. 1662333)

Dear Mr. Marmor:

On July 31, 2018 and August 17, 2018 seven (7) and six (6) HQ-sized core samples were received by Geomechanica Inc. via drop-off by Golder personnel, respectively. These samples were identified as being from boreholes drilled as part of Golder project 1662333. A total of 13 uniaxial compressive strength (UCS) specimens were prepared and tested from these samples. The tangent elastic modulus was measured for 5 of these 13 tests.

Details regarding the steps of specimen preparation and testing along with the test results and specimen photographs before and after testing are presented in the accompanying laboratory report.

Sincerely,

Bryan Tatone Ph.D., P. Eng.

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

GEOMECHANICA

Rock Laboratory Testing Results

A report submitted to:

David Marmor
Golder Associates Limited
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
info@geomechanica.com

August 27, 2018
Project number: 1662333

Abstract

This document summarizes the results of rock laboratory testing of 13 uniaxial compressive strength (UCS) tests. Results, including uniaxial compressive strength (UCS) and Young's modulus (for select samples) along with photographs of samples before and after testing are presented. Additional specimen information is included in an accompaning summary spreadsheet.

In this document:

Disclaimer: This report was prepared by Geomechanica Inc. for Golder Associates Limited. 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 (UCS) testing

This report summarizes the results of 13 uniaxial compressive strength (UCS) tests. 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 0.15 mm/min for shale and and 0.075 mm/min for limestone samples (Figure 1). This displacement rate was selected to target specimen failure to occur within 2 - 15 minutes.

The specimen preparation and testing procedure 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 a cylindrical specimen 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. Placement of the specimen into the loading frame, applying a 1 kN axial load, and removing the electrical tape.
5. Axial loading to rupture while continuously recording axial force and axial deformation to determine the peak strength (UCS) and (tangent) Young's modulus (E) mfor select samples.

Figure 1: UCS test setup.

1.1 Results

The results of the tests are summarized in Table 1. The corresponding stress-strain curves for the uniaxial compression tests are presented in Figure 2 and 3. Young's modulus is the tangent modulus, calculated as the slope of the best fit line through ±300 data points on either side of the point representing 50.0% of the peak strength. Additional specimen information is included in the accompaning summary spreadsheet.

Table 1: Summary of laboratory test results.

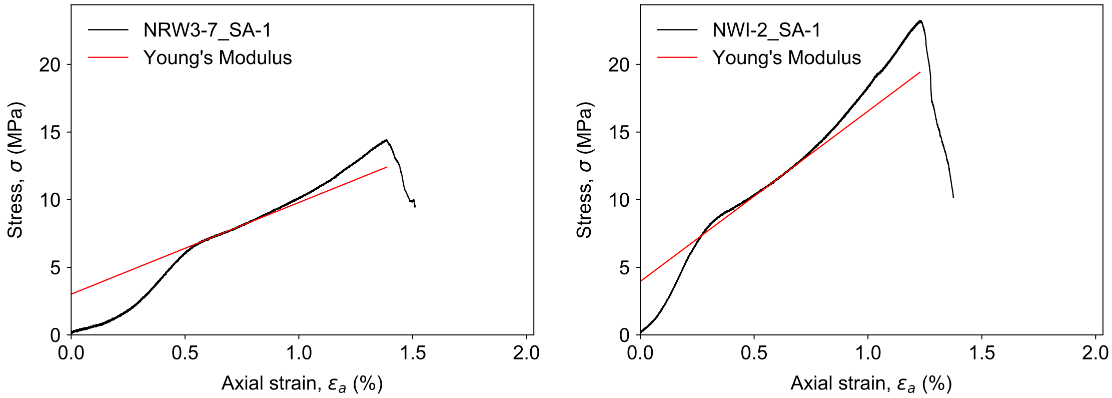
Sample	Depth (m)	Lithology description	Bulk density ρ (g/cm^3)	UCS (MPa)	Young's Modulus E (GPa)	Failure description
NRW3-7, SA-1	9.57 - 9.71	Georgian Bay Formation - Shale	2.596	14.4	0.68	Axial splitting [1, 2]
NWI-2, SA-1	5.06 - 5.31	Georgian Bay Formation - Shale	2.619	23.3	1.26	Inclined shear fracture [2]
NWI-3, SA-1	4.29 - 4.44	Georgian Bay Formation - Shale with several limestone lenses < 5 mm	2.601	16.8	-	Localized crushing [2]
NW5-4, SA-1	5.47 - 5.61	Georgian Bay Formation - Limestone	2.732	196.3	60.84	Inclined shear fracture
OHS-1, SA-1	5.26 - 5.44	Georgian Bay Formation - Shale	2.591	13.0	-	Inclined shear fracture [2]
OHS-2, SA-1	5.38 - 5.49	Georgian Bay Formation - Shale with 2 limestone layers $\approx$5 mm thick	2.449	23.4	-	Hourglass failure [1, 2]
OHS-5, SA-1	6.13 - 6.27	Georgian Bay Formation - Shale	2.603	16.7	-	Axial splitting [2]
AR-2, SA-1	5.92 - 6.12	Georgian Bay Formation - Shale	2.574	9.1	-	Axial splitting [2]
AR-2, SA-2	8.60 - 8.82	Georgian Bay Formation - Shale	2.588	11.5	-	Axial splitting [2]
NW5-1, SA-1	4.29 - 4.45	Georgian Bay Formation - Shale	2.593	13.6	-	Hourglass failure [2]
SWME-4, SA-1	10.40 - 10.54	Georgian Bay Formation - Shale	2.586	13.5	-	Axial splitting [2]
HMPL-1, SA-1	4.81 - 4.96	Georgian Bay Formation - Shale	2.573	11.8	0.50	Localized crushing [2]
HMPL-2, SA-1	3.70 - 3.85	Georgian Bay Formation - Shale	2.594	13.7	0.88	Axial splitting [2]

[1] Specimen Length:Diameter ratio < 2 due to short sample length

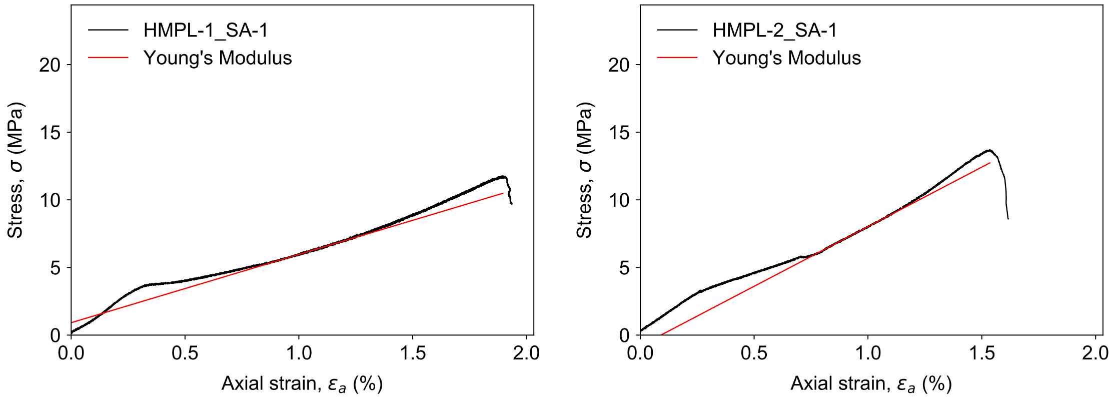
[2] Specimen emitted pore water upon loading

1.2 Specimen photographs

Photographs of the specimens before and after testing are presented in Figures 4 to 6.

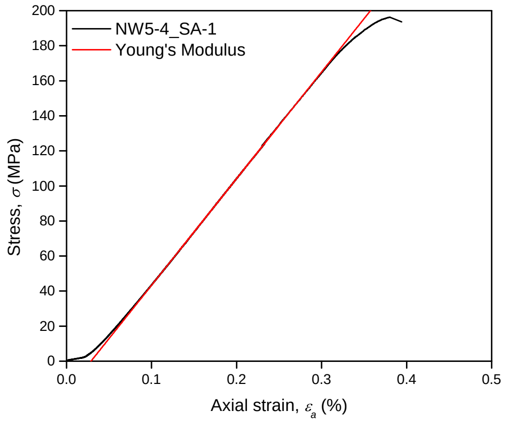


(a) NRW3-7, SA-1 (b) NWI-2, SA-1



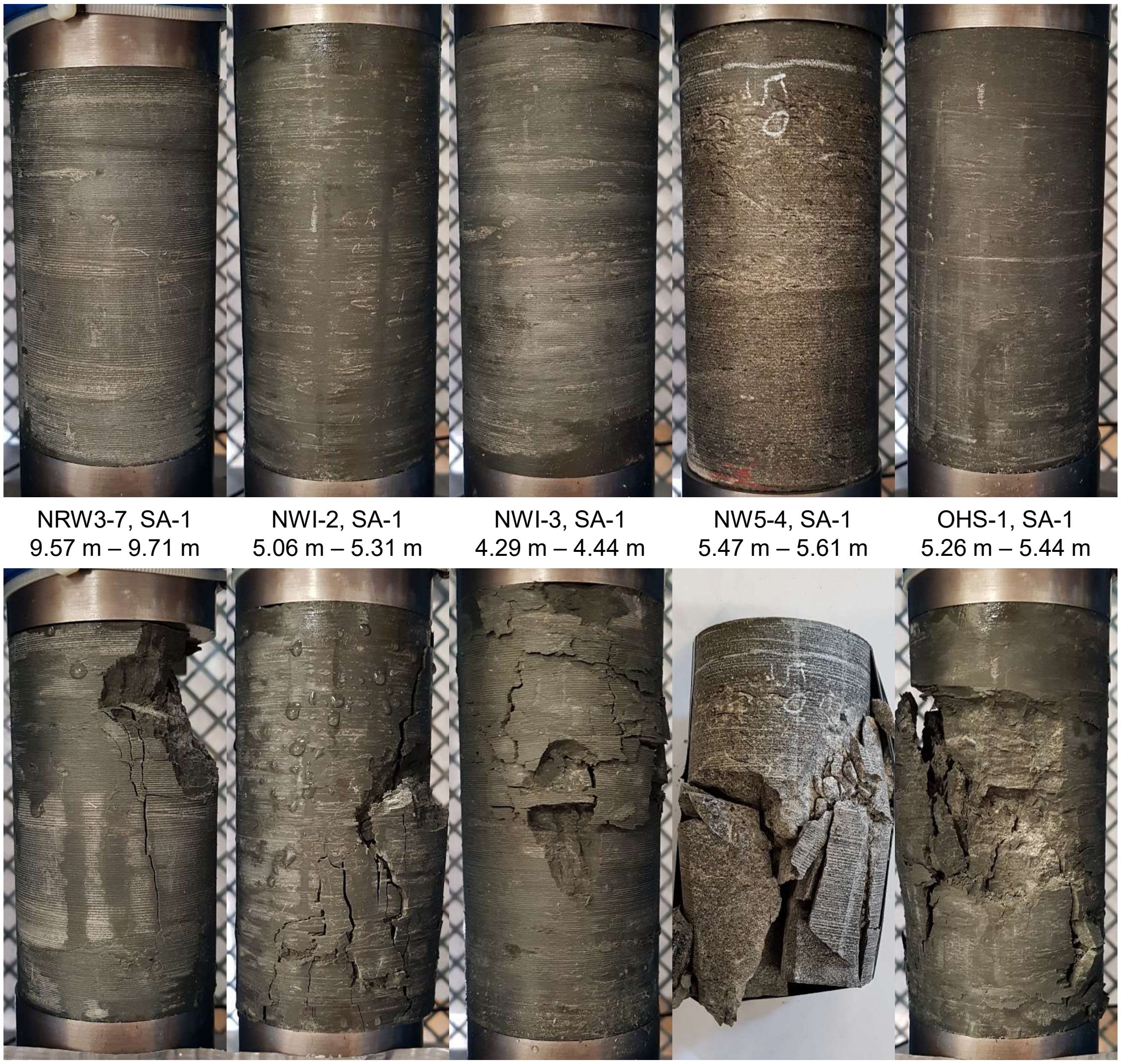
(c) HMPL-1, SA-1 (d) HMPL-2, SA-1

Figure 2: Measured stress-strain curves for shale samples.



(a) NRW5-4, SA-1

Figure 3: Measured stress-strain curves for limestone samples.



NRW3-7, SA-1
9.57 m – 9.71 m

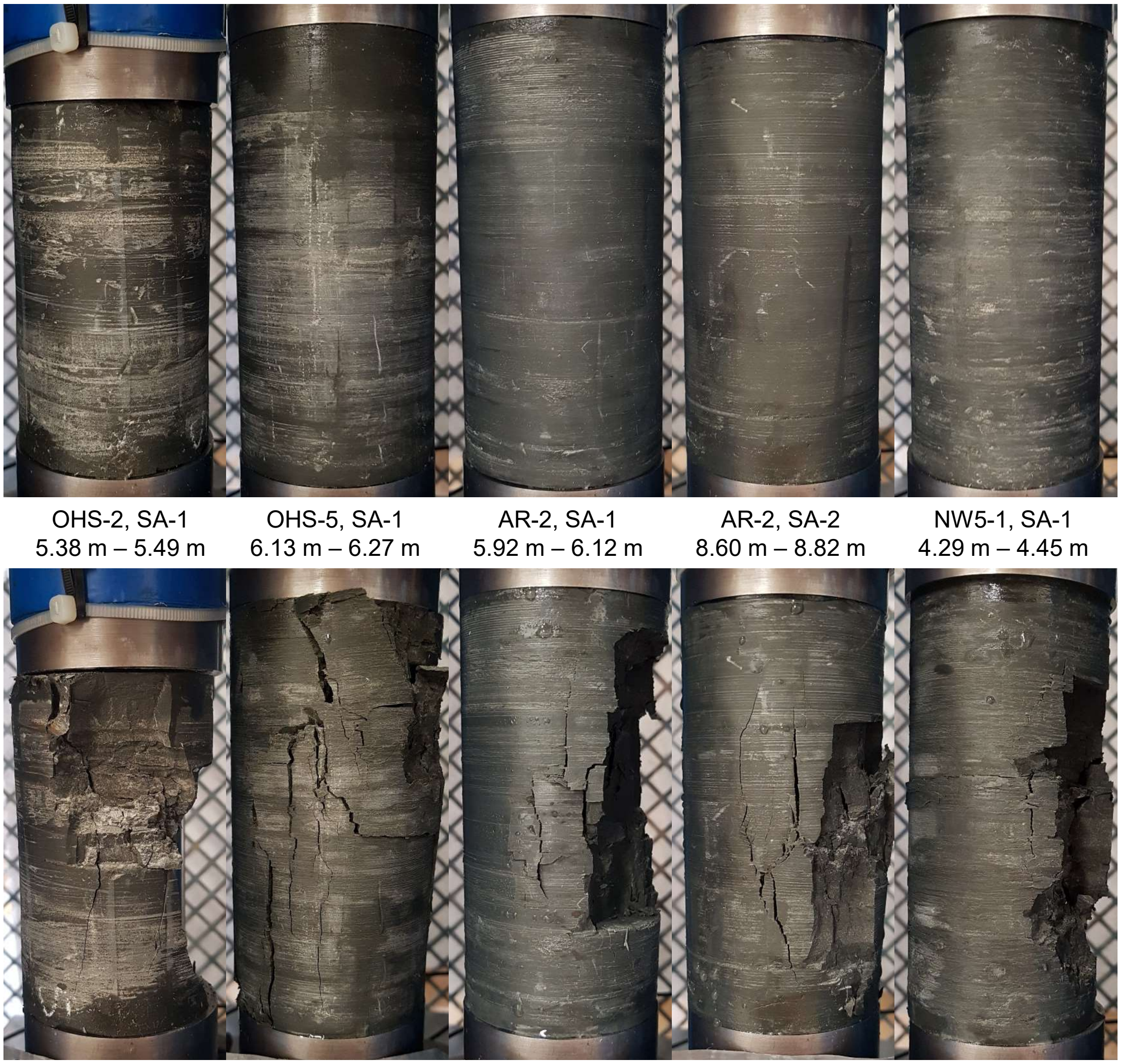
NWI-2, SA-1
5.06 m – 5.31 m

NWI-3, SA-1
4.29 m – 4.44 m

NW5-4, SA-1
5.47 m – 5.61 m

OHS-1, SA-1
5.26 m – 5.44 m

Figure 4: Photographs of specimens before and after testing.



OHS-2, SA-1
5.38 m – 5.49 m

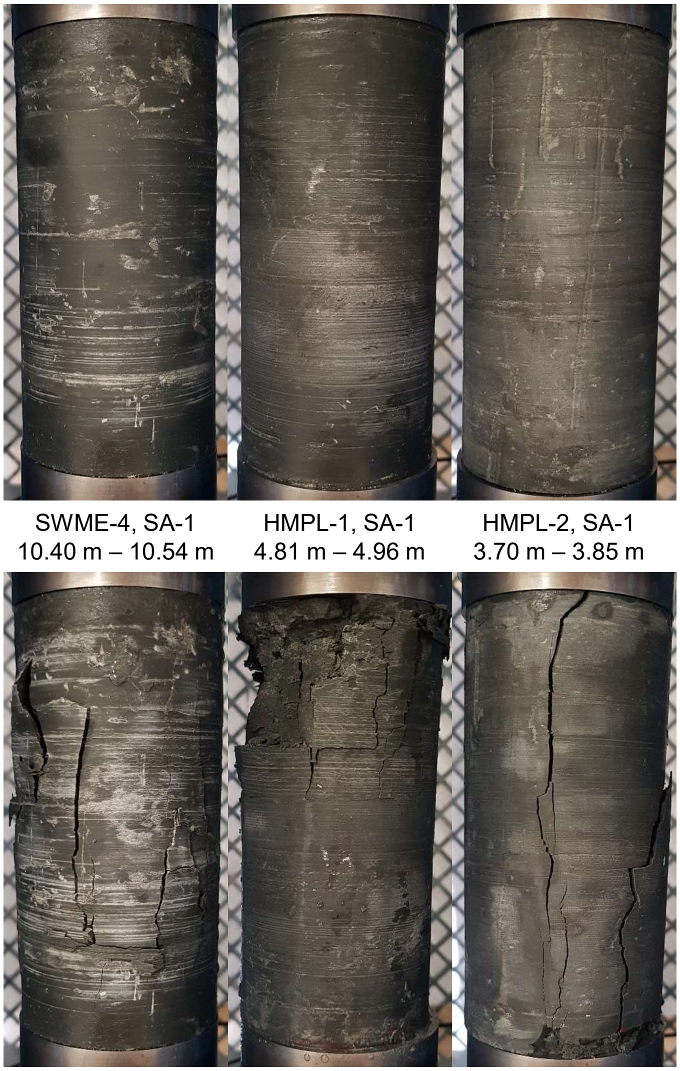
OHS-5, SA-1
6.13 m – 6.27 m

AR-2, SA-1
5.92 m – 6.12 m

AR-2, SA-2
8.60 m – 8.82 m

NW5-1, SA-1
4.29 m – 4.45 m

Figure 5: Photographs of failed specimens before and after testing (continued).



SWME-4, SA-1
10.40 m – 10.54 m

HMPL-1, SA-1
4.81 m – 4.96 m

HMPL-2, SA-1
3.70 m – 3.85 m

Figure 6: Photographs of failed specimens before and after testing (continued).

APPENDIX C

Non-Standard Special Provisions and Notice to Contractor

COMPACTED CLAY LINER FOR SWM POND - Item No.

Special Provision

1.0 SCOPE

This special provision describes the requirements for the construction of the 450 mm (minimum) thick compacted clay liner over the base and side slopes of the SWM Pond.

2.0 REFERENCES

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

Ontario Provincial Standard Specifications, Material
OPSS 1205 Clay Seal

American Society for Testing and Materials (ASTM)

- ASTM D 4643, Determination of Moisture Content of Soil by the Microwave Oven Method
- ASTM D 5261, Test Method for Measuring Mass per Unit Area of Geotextiles
- ASTM D 5887, Measurement of Index Flux through Saturated Geosynthetic Clay Liner Specimens Using a Flexible Wall Permeameter
- ASTM D 5890, Standard Test Method for Swell Index of Clay Mineral Component of Geosynthetic Clay Liners
- ASTM D 5891, Standard Test Method for Fluid Loss of Clay Component of Geosynthetic Clay Liners
- ASTM D 5993, Standard Test Method for Measuring Bentonite Mass per Unit Area of Geosynthetic Clay Liners

3.0 DEFINITIONS – Not Used

4.0 DESIGN AND SUBMISSION REQUIREMENTS – Not Used

5.0 MATERIALS

The clay liner material shall have the following properties:

Per cent clay-size material (i.e. per cent finer than 0.002 mm)	15% or greater
Plasticity index	10% or greater
Maximum particle size	100 mm

As an alternative to a natural source, a clay mixture meeting the requirements of OPSS 1205.05.03 could be used for the clay liner, provided that permeability testing (in accordance with ASTM 5084 – Permeability of Saturated Soils Using a Flexible Wall Permeameter) demonstrates that the clay mixture can attain a hydraulic conductivity of 1x10-7 cm/s or less.

The suitability of the soil or clay mixture for clay liner construction must be confirmed by the Contractor’s Engineer.

6.0 EQUIPMENT – Not Used

7.0 CONSTRUCTION

7.0.01 Clay Liner Construction

The clay liner shall be constructed on the prepared shale bedrock subgrade. No standing water or excessive moisture shall be present on the subgrade surface at the time of clay liner construction. The clay liner is to be constructed over the entire base and the side slopes of the SWM Pond, as shown on the Contract Drawings.

The clay material shall be compacted at a water content that is within the range of 0 to 4 per cent wetter than the Standard Proctor optimum water content, as determined by the Contractor’s Engineer. If water content testing by the Contractor’s Engineer indicates that the water content needs to be adjusted to meet the above-noted range in compaction water content, then the adjustment (either wetting or drying) shall be carried out after placement of each loose lift and the loose lift shall be tilled to promote moisture uniformity through the full thickness of the lift prior to compaction of the lift.

The soil liner shall have a minimum final compacted thickness of 450 mm, as measured perpendicular to the subgrade surface. The soil liner shall be constructed in three lifts of equal loose thickness. Each lift shall be compacted to achieve an in situ density equal to or greater than 95 per cent of the material’s Standard Proctor maximum dry density as determined by the Contractor’s Engineer (by ASTM D698). Each lift shall receive a minimum of six one-way passes of the compactor to ensure kneading/bonding of the material.

The Contractor’s Engineer shall perform in situ density tests and collect samples of the compacted clay liner at the in situ density test locations.

All perforations in the compacted clay liner shall be backfilled using dry bentonite pellets. Perforations that must be filled include, but are not limited to, the following:

- Nuclear density test probe holes;
- Holes made by a small spade near the nuclear density test locations to obtain a sample for laboratory water content testing; and
- Holes resulting from removal of any foreign material present in the liner material.

The size of the bentonite pellets used for backfill of the perforations shall be less than one-half the diameter of the perforation, or 25 mm, whichever is smaller. For the nuclear density test probe holes, the pellets shall be placed in lifts and compacted using a tamping rod.

The final surface of the compacted clay liner shall be shaped to the specified contours and sealed by at least one pass of a smooth drum roller.

8.0 QUALITY ASSURANCE

The Construction Quality Control test data collected by the Contractor’s Engineer shall be provided to the Contract Administrator’s geotechnical/foundations consultant for review and concurrence.

Monitoring of the soil liner construction shall be carried out by the Contractor's Engineer. The Contractor's Engineer's work shall include the following:

- Measurement of water content, grain size distribution, Atterberg limits, and Standard Proctor maximum dry density and optimum water content on representative samples of the clay liner material taken from the borrow area.
- Observation of the lift thickness as placed loose and after compaction.
- Monitoring of the number of passes used to compact each lift.
- Measurement of the in situ density and water content of the clay liner material after compaction.
- Inspection of the condition of the finished surface of the compacted clay liner prior to placement of the overlying ballast fill.

The proposed minimum testing frequencies are presented in Tables 1 and 2. Actual test frequencies may vary. Sampling/testing locations shall be selected by the Contractor's Engineer.

Table 1
Minimum Construction Quality Assurance Testing Frequencies for Borrow Source

Test	Method	Minimum Frequency of Testing
Standard Proctor maximum dry density	ASTM D 698	1 per 1,000 m^3
Atterberg Limits	ASTM D 4318	1 per 500 m^3
Water content (Micro-wave Method)	ASTM D 4643	1 per 500 m^3
Clay size content (i.e. percent finer than 0.002 mm)	ASTM D1140	1 per 500 m^3
Maximum particle size	Visual inspection	Continuous

Table 2
Minimum Construction Quality Assurance Testing Frequencies after Compaction of Clay Liner Material

Test	Method	Minimum Frequency of Testing
In situ density test, per lift (except lowermost lift)	ASTM D 2922 (Nuclear Method)	1 per 500 m^2
In situ water content test, per lift (except lowermost lift)	ASTM D 3017 (Nuclear Method)	1 per 500 m^2
Laboratory water content test, per lift (all lifts)	ASTM D 4643 (Microwave Method)	1 per 500 m^2

9.0 MEASUREMENT FOR PAYMENT

Measurement is by Plan Quantity, as may be revised by Adjusted Plan Quantity, in square metres following the contours of the SWM pond.

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.

VIBRATION MONITORING – Item No.

Special Provision

TABLE OF CONTENTS

1.0 SCOPE

This special provision describes requirements for vibration monitoring during excavations and installation of spread/strip footings, deep foundations, cofferdams and temporary protection systems for the construction of the QEW Credit River bridge, Mississauga Road overpass, East-West Active Transport bridge, North-South Active Transport bridge, stormwater management ponds, east access road, culverts, overhead sign supports, high mast light pole foundations and caissons for noise barrier walls.

2.0 REFERENCES

The subsurface conditions at the site are described in the following Foundation Investigation Report entitled:

> FOUNDATION INVESTIGATION REPORT
> Queen Elizabeth Way (QEW) - Credit River Bridge, Structure Site No. 24-203, QEW Widening from West of Mississauga Road to West of Hurontario Street, Mississauga, Ministry of Transportation, Ontario, GWP 2002-13-00

> FOUNDATION INVESTIGATION REPORT
> Queen Elizabeth Way (QEW) – Mississauga Road Overpass Replacement, Structure Site No. 24-196, QEW Widening from West of Mississauga Road to West of Hurontario Street, Mississauga, Ministry of Transportation, Ontario, GWP 2002-13-00

> FOUNDATION INVESTIGATION REPORT

Queen Elizabeth Way (QEW) – Stormwater Management Pond, QEW Widening from West of Mississauga Road to West of Hurontario Street, Mississauga, Ministry of Transportation, Ontario, GWP 2002-13-00

FOUNDATION INVESTIGATION REPORT
Queen Elizabeth Way (QEW) – North-South Active Transport Crossing Structure Over QEW, QEW Widening from West of Mississauga Road to West of Hurontario Street, Mississauga, Ministry of Transportation, Ontario, GWP 2002-13-00

FOUNDATION INVESTIGATION REPORT
Queen Elizabeth Way (QEW) – East-West Active Transport Bridge Along Credit River Bridge, QEW Widening from West of Mississauga Road to West of Hurontario Street, Mississauga, Ministry of Transportation, Ontario, GWP 2002-13-00

FOUNDATION INVESTIGATION REPORT
Queen Elizabeth Way (QEW) – Overhead Sign Supports and High Mast Light Poles, QEW Widening from West of Mississauga Road to West of Hurontario Street, Mississauga, Ministry of Transportation, Ontario, GWP 2002-13-00

FOUNDATION INVESTIGATION REPORT
Queen Elizabeth Way (QEW) – Stormwater Management Pond (Dry), QEW Widening from West of Mississauga Road to West of Hurontario Street, Mississauga, Ministry of Transportation, Ontario, GWP 2002-13-00

FOUNDATION INVESTIGATION REPORT
Queen Elizabeth Way (QEW) – East Access Road, QEW Widening from West of Mississauga Road to West of Hurontario Street, Mississauga, Ministry of Transportation, Ontario, GWP 2002-13-00

FOUNDATION INVESTIGATION REPORT
Queen Elizabeth Way (QEW) – Noise Barrier Wall, QEW Widening from West of Mississauga Road to West of Hurontario Street, Mississauga, Ministry of Transportation, Ontario, GWP 2002-13-00

FOUNDATION INVESTIGATION REPORT
Queen Elizabeth Way (QEW) – Stavebank Creek and Kenollie Creek Culvert Replacements, QEW Widening from West of Mississauga Road to West of Hurontario Street, Mississauga, Ministry of Transportation, Ontario, GWP 2002-13-00

3.0 DEFINITIONS

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

Contractor’s Engineer means an Engineer with a minimum of five (5) years’ experience in the field of installation of piling and vibration monitoring or, alternatively, with expertise demonstrated by providing satisfactory quality verification services for a minimum of two (2) projects of similar scope to the Contract. The Contractor’s Engineer shall be retained by the Contractor to ensure general conformance with the Contract Documents and issue certificates of conformance.

Peak Particle Velocity (PPV) means the maximum component velocity in millimetres per second 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 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 construction operations.

4.0 DESIGN AND SUBMISSION REQUIREMENTS

4.1 Submission Requirements

The Contractor/Contractor's Engineer shall submit details of the vibration monitoring plan to the Contract Administrator for information purposes at least 2 weeks prior to any work related to strip footing, deep foundation, cofferdam and temporary protection system installation. 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, structures, utilities, wells, or other potentially vibration-sensitive structures within a 250 m radius from the excavation and installation of spread/strip footings, cofferdams, deep foundations and temporary protection systems, as applicable.
e) Proposed frequency of readings.
f) Action plan to be taken to adjust excavation, deep foundation and protection system installation methods 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 250 m of excavation and installation of spread/strip footings, cofferdams, deep foundations and temporary protection systems.

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/well within a 250 m radius of excavation and installation of spread/strip footings, cofferdams, deep foundations and temporary protection systems, shall be completed a minimum of two (2) weeks prior to commencement of excavation and installation of shallow and deep foundations and temporary protection systems. Only one Pre-Construction Condition Survey per structure or facility is required to be carried out in advance of excavation and installation of spread/strip footings, cofferdams, deep foundations or temporary protection system 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 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.

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 at each structure within a 250 m radius of the bridge, is required within two (2) months of completion of the excavation and installation of spread/strip footing, cofferdams, deep foundation and during installation of temporary protection systems.

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 excavation and installation of spread/strip footings, deep foundations and temporary protection systems.

7.2 Monitoring

The vibration monitoring equipment shall be placed on the ground surface in the vicinity of each foundation element or protection system, and on the ground surface at radial distances of 25 m, 50 m, and 100 m from the foundation element or protection system locations within the project. The Contractor shall take readings continuously during excavation and installation of spread/strip footing, cofferdams, deep foundation and during installation of temporary protection systems, and shall immediately notify the Contract Administrator if the vibrations exceed the limits specified herein.

The vibrations measured on private structures, wells, etc. shall not exceed 25 mm/s. Those measured on utilities, if applicable, shall not exceed 10 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.

NOTICE TO CONTRACTOR – Rock Excavation

Special Provision

Excavations for the Stormwater Management Pond west of Mississauga Road, the East Access Road on the east side of the Credit River, overhead sign foundations, high mast light foundations, noise barrier wall foundations, Emergency Overflow Sewer Outlet and the north portion of Kenollie Creek Culvert near the inlet will extend into the shale bedrock, which is very weak to weak, contains clay seams and medium strong to very strong limestone interlayers at varying depths/elevations. The bedrock condition shall be considered by the Contractor in the selection of appropriate equipment and procedures for various activities, including but not limited to excavation, grading, installation of the foundations and installation of temporary protection systems, where required, and potentially for construction of cofferdam at/near the inlet to Kenollie Creek Culvert.

NOTICE TO CONTRACTOR – Subsurface Obstructions

Special Provision

The Contactor shall be alerted to the potential presence of cobbles, boulders and limestone and shale fragments in the fill and native soils, glacially derived soils and residual soils, as encountered in various boreholes advance at the various structure locations associated with the QEW widening from Mississauga Road to Hurontario Street. Consideration of the presence of these obstructions must be made in the selection of appropriate equipment and procedures for advancing caissons, excavations for shallow foundations, stormwater management pond, overhead sign supports, high mast light pole foundations, noise barrier walls, culverts, sewer pipes extending from the inlet and the outlet of the stormwater management dry pond and installation of any temporary protection systems that may be required.

The Contactor is hereby notified that in some areas of the site, and in particular in the general vicinity of the east pier for the QEW Credit River Bridge WB, rip-rap and other cobble and boulder size obstructions are present at and below ground surface. These obstructions may impede or prevent excavation, grading, construction of access roads and/or crane pads and lay-down areas, and the installation of some types of protection systems/cofferdams.

The Contractor is hereby notified that in some areas of the site, and in particular in the general vicinity of the front and side slopes adjacent to the west abutment for the QEW Credit River Bridge WB, soil/rock anchor obstructions are present at and below the ground surface. These obstructions may impede or prevent excavation, grading, and construction of the abutment and/or the Multi-Use Trail and are to be removed where encountered above the elevation of the existing upper access road only. No soil/rock anchors are to be removed below the elevation of the existing upper access road.

The Contractor is hereby notified that between the west abutment of the existing QEW Credit River Bridge and the west abutment of the existing multi-use path (beneath the existing QEW Credit River Bridge) soil/rock anchor obstructions are present at and below the ground surface. These obstructions may impede or prevent the advancement of the drilled shafts for the west abutment of the East-West Active Transportation bridge. If they are encountered the Contract Administrator is to be notified immediately and this may require adjustments to the drilled shaft layout.

The presence of the above-noted near surface conditions shall be considered by the Contractor in the selection of appropriate equipment and procedures for various activities, including but not limited to excavation, grading, installation of the foundations and installation of cofferdams/protection systems.

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