



REPORT

Foundation Investigation Report

*MH856-MH845 Storm Sewer Trenchless Crossing West of Dixie Road
Underpass, QEW Improvements from East of Cawthra Road to the East Mall,
Cities of Mississauga and Etobicoke
Ministry of Transportation, Ontario
GWP 2432-13-00*

Submitted to:

AECOM

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Submitted by:

WSP Canada Inc.

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1530382-W-REV1

February 13, 2024

GEOCRES No.: 30M12-527

Latitude: 43.595680°

Longitude: -79.569977°

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1.0 INTRODUCTION

WSP Canada Inc. (WSP, formerly Golder Associates Ltd., amalgamated with WSP in 2023) has been retained by AECOM on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services in support of two trenchless storm sewer crossings, included as part of the Queen Elizabeth Way (QEW) improvements from east of Cawthra Road to The East Mall, in the Cities of Mississauga and Etobicoke, Ontario.

This report addresses the trenchless sewer crossing the QEW between MH856 and MH845, located west of the QEW-Dixie Road Underpass at approximately Station 12+700 as shown on the Key Plan on Drawing 1.

The work has been carried out in accordance with Golder's Supplementary Specialty Plan for foundation engineering services for this project, dated June 6, 2016, under our Consultant Assignment Number (Number 2015-E-0001) for this project.

2.0 SITE DESCRIPTION

The existing QEW-Dixie Road Underpass is located at Station 12+825, approximately 1.9 km east of the QEW-Cawthra Road interchange and 2.5 km west of the QEW-Highway 427 interchange, in the City of Mississauga. The QEW alignment in the project area is oriented generally in a southwest-northeast direction; for the purposes of this report, the QEW alignment is described as being in an east-west orientation.

At the crossing location, the QEW is a divided highway with three travel lanes in each direction separated by a narrow concrete and asphalt median with a steel guide rail. There are paved shoulders and wall mounted noise barrier walls also present along both sides of the highway. Both the North Service and South Service Roads run parallel to the QEW. The installation will cross under the QEW and both the North and South Service Roads.

The natural ground surface in the vicinity of the crossing is at about Elevation 106 m decreasing to about Elevation 105 m and 104 m at the north and south ends of the crossing, respectively. Land use in the surrounding area is primarily residential, and a large commercial development is located southwest of the crossing. The sewer will ultimately terminate in the future stormwater management pond south of the QEW.

3.0 INVESTIGATION PROCEDURES

Borehole BH-2W was drilled on February 1, 2024, near the middle of the proposed sewer crossing between MH856 and MH845. Borehole NW3-6 was drilled in October 2016 and is located just west of MH845 at the north end of the crossing. Boreholes SWM-A-1 to SWM-A-3, were drilled in October 2016 south of MH856. The locations of the boreholes are listed below and shown on Drawing 1. The borehole records, which includes a summary of laboratory testing results and inferred soil strata from this investigation, is presented in Appendix A

The investigations were carried out using a truck-mounted CME75, supplied and operated by specialized drilling contractors. The boreholes were advanced through the overburden using 108 mm and 203 mm outside diameter solid-stem and hollow stem augers respectively. Soil samples were obtained either continuously or at 0.75 m intervals of depth, using a 50 mm outside diameter split-spoon sampler driven by an automatic hammer in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586). The results of the in-situ field tests (i.e., SPT "N"-values) as presented on the borehole records and in Section 4.2 are uncorrected. The split-spoon samplers used in the investigation limit the maximum particle size that can be sampled and tested to about 35 mm, therefore particles or objects that may exist within the soils that are larger than this dimension would not be sampled or represented in the grain size distributions. SPT samples and augering was completed into the top portion of bedrock in some boreholes. Samples of the bedrock were also obtained using an 'NQ'-size rock core

barrel and coring techniques in Borehole SWM-A-2 and an “HQ”-size rock core barrel and coring techniques in Borehole BH-2W.

The groundwater conditions were noted in the boreholes upon completion of drilling and a standpipe piezometer was installed in Borehole SWM-A-2 to permit monitoring of the groundwater level at the stormwater management pond location. The standpipe piezometer consists of 75 mm diameter PVC pipe, sealed at a selected depth interval within the bedrock. The borehole annulus surrounding the piezometer screen was backfilled with filter sand. Above the sand filter pack and piezometer screen, the annulus surrounding the piezometer pipe was backfilled to the ground surface with bentonite pellets.

The field work was observed on a full-time basis by a member of our technical staff who located the boreholes, arranged for the clearance of underground services, observed the drilling, sampling and in situ testing operations, and logged the boreholes. The samples were identified in the field, placed in appropriate containers, labelled, and transported to Golder’s Whitby laboratory where the samples underwent further visual examination. Geotechnical laboratory testing including water contents, grain size distributions, and Atterberg limits was carried out on selected soil samples in accordance with MTO and/or ASTM Standards, as applicable. The results of the laboratory testing on select samples are included in Appendix B.

The Total Core Recovery (TCR), Solid Core Recovery (SCR), Rock Quality Designation (RQD), weathering and strength indices, and discontinuity characterises of the bedrock core samples were recorded in Boreholes SWM-A-2 and BH-2W in the field based on visual observation and measurement. The bedrock was sequentially photographed, packed, and transported to Golder’s Mississauga laboratory for further visual examination. Laboratory testing consisting of Uniaxial Compressive Strength (UCS) testing was carried out on a select specimen of the bedrock core samples, by Geomechanica of Mississauga, Ontario.

The as-drilled borehole location and the ground surface elevation was obtained using a GPS (Trimble XH 3.5G), having an accuracy of 0.1 m in the vertical and 0.1 m in the horizontal directions. The location given in the borehole/drillhole record and shown on Drawing 1 is positioned relative to MTM NAD 83 (Zone 10) northing and easting coordinates and the ground surface elevations is referenced to Geodetic datum. The borehole location, including in geographic coordinates of latitude and longitude, ground surface elevation and drilled depth is summarized below.

Borehole No.	MTM NAD83 Northing (Latitude, °)	MTM NAD83 Easting (Longitude, °)	Ground Surface Elevation (m)	Borehole Depth (m)
NW3-6	4,828,434.5 (43.595823)	299,136.6 (-79.570143)	105.0	4.7
BH-2W	4,828,422.5 (43.595715)	299,161.2 (-79.569839)	105.2	6.1
SWM-A-1	4,828,407.6 (43.595582)	299,195.7 (-79.569412)	104.2	4.7
SWM-A-2	4,828,383.6 (43.595366)	299,214.1 (-79.569182)	104.1	6.0
SWM-A-3	4,828,385.7 (43.595384)	299,179.6 (-79.569609)	103.1	4.6

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)¹. 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

The subsurface soil, bedrock, and groundwater conditions encountered in the boreholes are presented on the Record of Borehole and Record of Drillhole sheets presented in Appendix A. In addition, lists on abbreviations and symbols and lithological, geotechnical rock description terminology, field estimation of rock hardness and rock weathering classification are also included in Appendix A to assist in the interpretation of the borehole and drillhole records. The results of the geotechnical laboratory testing on the soil and bedrock samples are presented in Appendix B.

The strata boundaries on the borehole records and on the interpreted stratigraphic profile on Drawing 1 have been inferred from drilling observations and non-continuous sampling. Therefore, these boundaries represent transitions between soil types rather than exact planes of geological change. Furthermore, subsurface conditions will vary between and beyond the borehole locations.

In general, the subsurface conditions in the vicinity of the proposed trenchless crossing consist of asphalt, concrete, or topsoil underlain by fill and a native silty sand to sandy silt deposit. The silty sand to sandy silt deposit is generally underlain by a deposit of clayey silt to silty clay. The overburden deposits are typically underlain by a deposit of clayey silt residual soil overlying shale bedrock. A more detailed description of the soil deposits at the site is provided in the following sub-sections.

4.2.1 Topsoil

A 0.15 m thick layer of topsoil was encountered at the ground surface in Boreholes SWM-A-1 to SWM-A-3.

4.2.2 Asphalt

Borehole NW3-6 was advanced through the North Service Road surface and encountered an approximately 150 mm thick layer of asphalt at ground surface. Borehole BH-2W was advanced through the QEW road surface and encountered an approximately 70 mm thick layer of asphalt at ground surface.

4.2.3 Concrete

A 460 mm layer of concrete was encountered beneath the asphalt layer at Borehole BH-2W.

¹ 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.)

4.2.4 Fill

An approximate 0.5 m and 0.2 m thick deposit of Granular 'A' or sand and gravel fill was encountered underlying the asphalt in Borehole NW3-6 and underlying the concrete in Borehole BH-2W, respectively. The surface of the sand and gravel fill was encountered at Elevation 104.9 m in Borehole NW3-6 and Elevation 104.7 m in Borehole BH-2W and extends to a depth of about 0.6 m below ground surface in Borehole NW3-6 (Elevation 104.4 m) and 0.7 m in Borehole BH-2W (Elevation 104.5 m).

A 0.6 m and 0.8 m thick deposit of silty sand, trace clay fill was encountered at ground surface in Borehole SWM-A-1 and underlying the sand and gravel fill layer in Borehole BH-2W, respectively. The fill was noted to have oxidation staining in Borehole BH-2W. The surface of the silty sand fill deposit was encountered at Elevation 104.0 m in Borehole SWM-A-1 and Elevation 104.5 m in Borehole BH-2W. A lower 0.7 m thick cohesive layer of clayey silt, some sand was encountered in Borehole SWM-A-1. The fill extends to a depth of about 1.5 m below ground surface in Boreholes SWM-A-1 and BH-2W (Elevations 102.7 m and 103.7, respectively).

The SPT "N"-values measured within the fill deposit are 6 and 7 blows per 0.3 m of penetration in the silty sand and 22 blows per 0.3 m of penetration in the clayey silt layer, indicating that the non-cohesive fill has a loose state of compactness, and the cohesive fill has a very stiff consistency.

The water content measured on one sample of the cohesive fill is about 8%.

4.2.5 Silty Sand to Sandy Silt

In Borehole NW3-6, a 0.8 m thick deposit of silty sand was encountered underlying the non-cohesive fill. The surface of the deposit was encountered at a depth of 0.6 m below ground surface (Elevation 104.4 m) and extends to a depth of 1.4 m below ground surface (Elevation 103.6 m).

In Boreholes SWM-A-2, SWM-A-3, and BH-2W, a 0.6 m thick deposit of sandy silt, trace gravel to silty sand, trace gravel was encountered underlying the topsoil in Boreholes SWM-A-2 and SWM-A-3 and underlying the silty sand fill deposit in Borehole BH-2W. Trace topsoil and trace rootlets were observed in the sandy silt deposit in Borehole SWM-A-2. The surface of the deposit was encountered at depths ranging from about 0.2 m to 1.5 m below ground surface (between Elevations 103.9 m and 102.9 m) and extends to depths ranging from about 0.8 m to 2.1 m below ground surface (between Elevations 103.3 m and 102.3 m).

The SPT "N"-values measured within the granular deposit range from 7 to 23 blows per 0.3 m of penetration, indicating a loose to compact state of compactness.

Grain size distribution testing was carried out on one sample of the deposit and the results are presented on Figure B-1 in Appendix B.

The water content measured on one sample of the granular deposit is about 23%.

4.2.6 Clayey Silt to Silty Clay

A 1.0 m to 0.3 m thick deposit of clayey silt to silt, trace to some sand, trace gravel was encountered underlying the silty sand in Boreholes NW3-6 and BH-2W, respectively. The surface of the clayey silt to silt deposit was encountered at a depth of 1.4 m below ground surface (Elevation 103.6 m) in Borehole NW3-6 and 2.1 m below ground surface in Borehole BH-2W (Elevation 103.1 m) and extends to a depth of 2.4 m below ground surface in Borehole NW3-6 (Elevation 102.6 m) and 2.4 m below ground surface in Borehole BH-2W (Elevation 102.8 m).

A 0.3 m thick deposit of silty clay was encountered underlying the silty sand in Borehole SWM-A-3. The surface of the silty clay deposit was encountered at a depth of 0.8 m below ground surface (Elevation 102.3 m) and extends to a depth of 1.1 m below ground surface (Elevation 102.0 m).

Two SPT “N”-values measured within the cohesive deposit are 10 and 17 blows per 0.3 m of penetration, suggesting a stiff to very stiff consistency. In Borehole BH-2W, one SPT “N”-value of 55 blows per 0.3 m of penetration was measured, suggesting a hard consistency, but is not considered representative of the deposit due to the underlying shale bedrock.

Grain size distribution testing was carried out on two samples of the cohesive deposit and the results are presented on Figure B-2 in Appendix B. Atterberg limits testing was carried out on two samples of the cohesive deposit and the results are presented on Figure B-3 in Appendix B. The Atterberg limits testing measured liquid limits of about 21% and 30%, plastic limits of about 15% and 16%, and plasticity indices of about 6% and 14%, indicating the material is of low plasticity.

The water content measured on two samples of the cohesive deposit are about 17% and 18%.

4.2.7 Clayey Silt (Residual Soil)

A 0.6 m and 1.2 m thick deposit of residual soil was encountered underlying the silty sand deposit in Borehole SWM-A-2 and the silty clay deposit in Borehole SWM-A-3, respectively. The surface of the residual soil deposit was encountered at a depth of about 0.8 m below ground surface in Borehole SWM-A-2 and 1.1 m below ground surface in SWM-A-3 (Elevations 103.3 m and 102.0 m in the respective boreholes). Residual soil is a heterogeneous mix of fully weathered bedrock that is disintegrated into a soil like texture material that no longer retains the structure of parent bedrock. The residual soil deposit consists of clayey silt, trace gravel to gravelly, some sand to with sand. The deposit was noted to contain some shale and limestone fragments in Borehole SWM-A-3.

The SPT “N”-values measured within the residual soil deposit range from 17 to 22 blows per 0.3 m of penetration, suggesting a very stiff consistency.

Grain size distribution testing was carried out on two samples of the residual soil deposit and the results are presented on Figure B-4 in Appendix B. Atterberg limits testing was carried out on one sample of the residual soil deposit and the results are presented on Figure B-5 in Appendix B. The Atterberg limits testing measured a liquid limit of about 27%, a plastic limit of about 17%, and a plasticity index of about 10%, indicating the material is a clayey silt of low plasticity.

The water content measured on two samples of the residual soil deposit range from about 9% to 16%.

4.2.8 Bedrock

Bedrock was inferred underlying the fill deposit in Borehole SWM-A-1, underlying the residual soil deposit in Boreholes SWM-A-2 and SWM-A-3, and underlying the cohesive deposit in Boreholes NW3-6 and BH-2W. The bedrock was sampled by split-spoon sampling in all boreholes and was also proven by coring in Boreholes SWM-A-2 and BH-2W. Photos of the rock core are presented in Figure B-8 and B-9.

The depths and elevations of the bedrock below ground surface, as inferred from split spoon sampling and confirmed by bedrock coring, and the corresponding bedrock surface elevations are summarized in the table below.

Borehole	Ground Surface Elevation (m)	Highly to Moderately Weathered Bedrock		Bedrock Split-Spoon Sampled / Augered Length (m)	Slightly Weathered to Fresh Bedrock		Cored Length (m) / Notes
		Depth (m)	Elevation (m)		Depth (m)	Elevation (m)	
NW3-6	105.0	2.4 – 4.7	102.6 – 100.3	2.3	-	-	Split Spoon Sample Only
BH-2W	105.2	2.4 – 2.7	102.8 – 102.5	0.3	2.7 – 6.1	102.5 – 99.2	3.3
SWM-A-1	104.2	1.5 – 2.2	102.7 – 102.0	3.2	2.2 – 4.7	102.0 – 99.5	Split Spoon Sample Only
SWM-A-2	104.1	1.4 – 5.0	102.7 – 99.1	1.6	5.0 – 6.0	99.1 – 98.1	3.0
SWM-A-3	103.1	2.3 – 2.9	100.8 – 100.2	2.3	2.9 – 4.6	100.2 – 98.5	Split Spoon Sample Only

The inferred depths to the bedrock surface and thicknesses / degree of weathering of the deposits provided in the table above were based on drilling behaviour, observations of drilling cuttings, and split-spoon sampling where no coring was carried out.

The SPT “N”-values measured within the inferred highly to slightly weathered bedrock obtained by split-spoon sampling range from 15 blows to 95 blows per 0.3 m of penetration and up to 100 blows for 0.05 m of penetration, suggesting a very stiff to hard consistency to refusal of penetration within the rock mass.

Grain size distribution testing was carried out on one disturbed sample of the moderately weathered shale bedrock from SPT-sampling and the results are presented on Figure B-6 in Appendix B. Atterberg limits testing was carried out on two disturbed samples of the moderately weathered shale bedrock and the results are presented on Figure B-7 in Appendix B. The Atterberg limits testing measured liquid limits between about 29% and 32%, plastic limits of about 17%, and plasticity indices between about 12% and 15%, indicating the material is of low plasticity.

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 highly weathered to fresh, thinly laminated to medium bedded, very fine to medium grained, faintly porous to non-porous, extremely weak to weak, grey, with weak to very strong limestone/siltstone interbeds at varying intervals, as presented in the drillhole records in Appendix A. The degree of weathering of the bedrock samples (i.e. fresh to highly weathered – W1 to W4), and the strength classification of the intact rock mass based on field identification (i.e. extremely weak to weak – R0 to R2) are described in accordance with the International Society for Rock Mechanics (ISRM)² standard classification system.

The Rock Quality Designation (RQD) measured on the core samples generally ranges from about 0% to 82%, indicating a rock mass of very poor to good quality as per Table 3.10 of CFEM (2006)³. One RQD of 100% was measured in the first core run of Borehole BH-2W, which was only 0.3 m in total length. The Total Core Recovery (TCR) ranges from about 22% to 100% and the Solid Core Recovery (SCR) ranges from about 6% to 91%, respectively.

² 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.

³ Canadian Geotechnical Society, 2006. Canadian Foundation Engineering Manual (CFEM), 4th Edition. The Canadian Geotechnical Society, BiTech Published Ltd., British Columbia.

One Unconfined Compression (UC) test (ASTM D7012) was carried out on a selected core sample of the shale bedrock obtained in Borehole SWM-A-2. The result is summarized in the table below and the details are presented on the Rock Laboratory Test Result Reports from Geomechanica Inc. included in Appendix B.

Borehole	Sample Depth (m)	Elevation (m)	UCS (MPa)	Bulk Density (g/cm ³)
SWM-A-2	5.1 – 5.3	99.0 – 98.8	17.7	2.6

The core recovered from Borehole BH-2W is considered extremely weak to weak as it was not possible to select a sample for testing without the core breaking in Runs 1 and 2. Based on field review of the rock core and the laboratory UCS results, in accordance with Table 3.5 in CFEM (2006)⁴, the shale bedrock is generally classified as very weak to weak (R1 to R2, 1 MPa < UCS < 25 MPa).

4.3 Groundwater Conditions

Details of the water levels observed in the boreholes upon completion of drilling and of the piezometer installation are summarized on the Record of Boreholes in Appendix A. In general, the overburden samples obtained were moist. The open boreholes were observed to be dry upon completion of drilling and the piezometer reading was taken the same day as installation; thus, these observations are not considered representative of the stabilized groundwater level at the site. The recorded water level in the standpipe piezometer installed in Borehole SWM-A-2, sealed within the shale bedrock, is summarized below. It should be noted that the groundwater level is subject to seasonal fluctuations and precipitation events and should be expected to be higher during wet periods of the year.

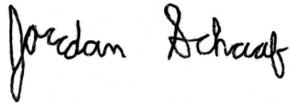
Borehole	Ground Surface Elevation (m)	Stratum Sealed Into	Depth to Groundwater Level (m)	Water Level Elevation (m)	Date of Measurement
SWM-A-2	104.1	Bedrock	2.2	101.9	17-Oct-16

5.0 CLOSURE

This Foundation Investigation Report was prepared by Mr. Jordan Schaaf, Associate Geotechnical Analyst, and was reviewed by Ms. Sarah E. M. Poot, P.Eng., MTO Principal Foundations Contact and RAQS-Approved Tunnelling Specialist – High Complexity. Mr. Kevin Bentley, P.Eng. and MTO Principal Foundations Contact conducted a quality control review of this report.

Signature Page

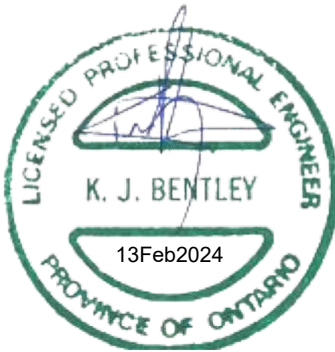
WSP Canada Inc.



Jordan Schaaf
Associate Geotechnical Analyst



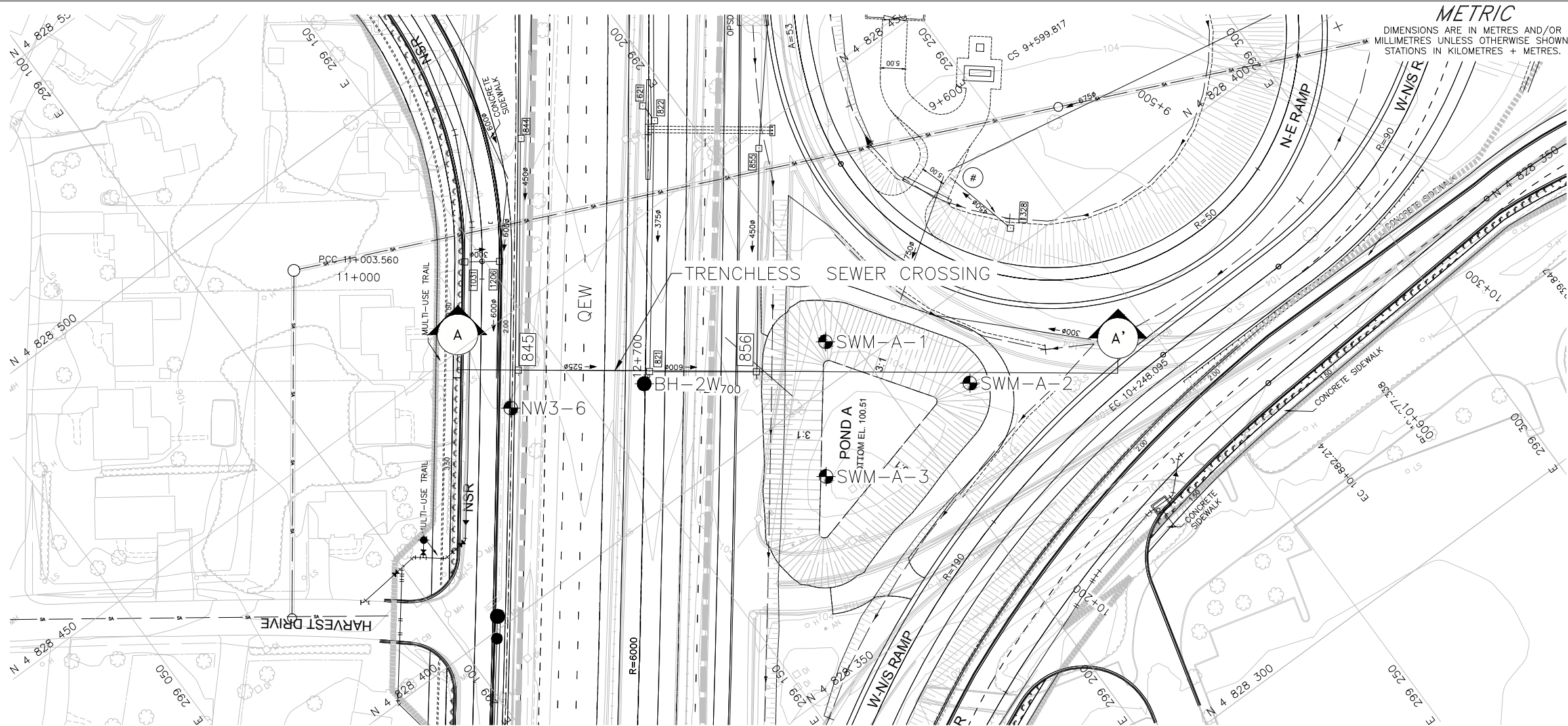
Sarah E. M. Poot, P.Eng.
MTO Principal Foundations Contact



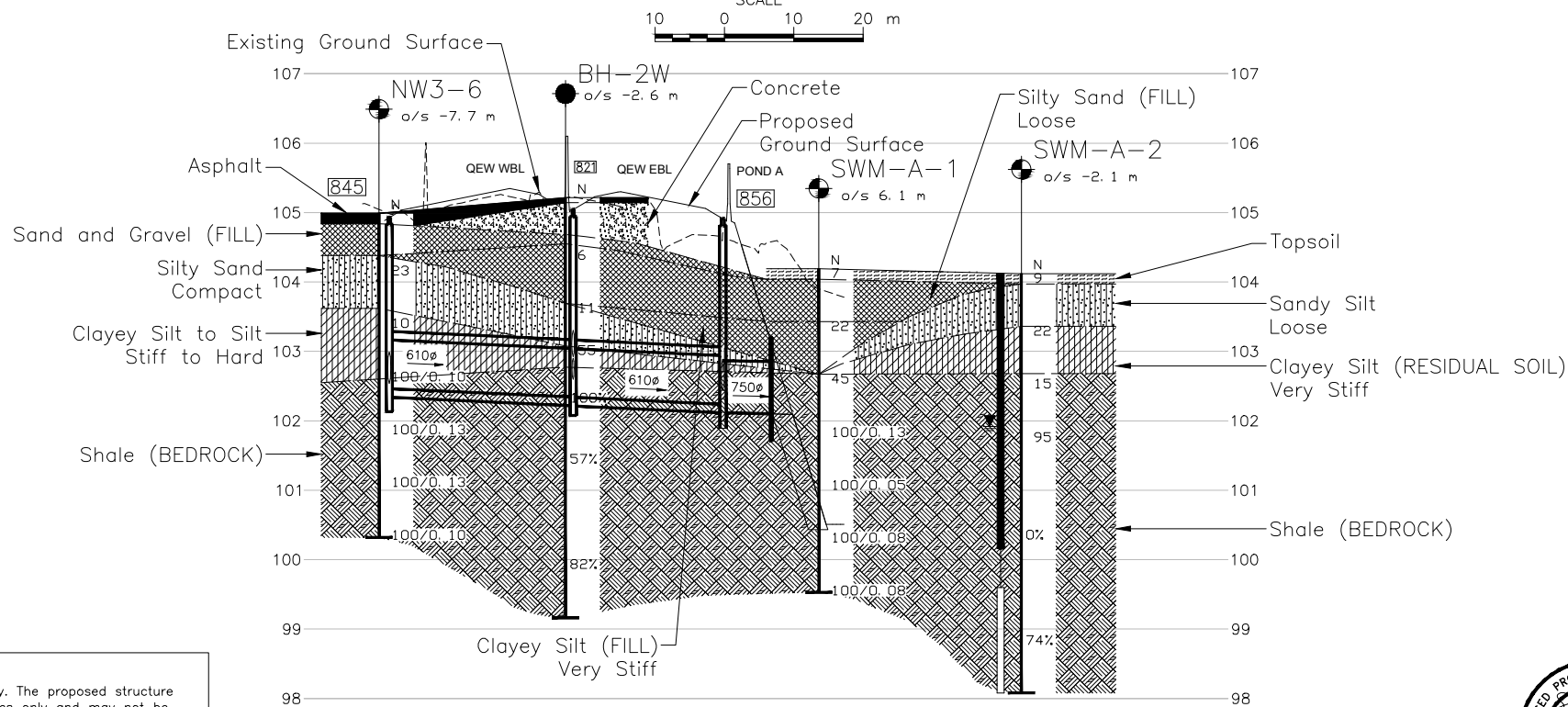
Kevin J. Bentley, P.Eng.
MTO Principal Foundations Contact

JNS/KJB/SEMP/al

[https://golderassociates.sharepoint.com/sites/19542g/1 foundations/09 - reports/26 - trenchless sewer 2024/4 - rev1 final/2.west mh856-mh845/1530382 fir rev1 2024'02'13-mh856-mh845 trenchless.docx](https://golderassociates.sharepoint.com/sites/19542g/1%20foundations/09%20-%20reports/26%20-%20trenchless%20sewer%202024/4%20-%20rev1%20final/2.west%20mh856-mh845/1530382%20fir%20rev1%202024%2002%2013-mh856-mh845%20trenchless.docx)



PLAN SCALE



PROFILE A-A'

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.

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

METRIC

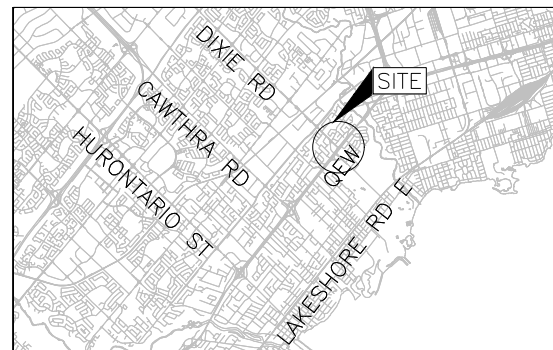
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No. 2021-2127
GWP No. 2432-13-00

QEW - EAST OF CAWTHRA RD. TO THE EAST MALL
MH856-MH845 STORM SEWER TRENCHLESS CROSSING WEST
OF DIXIE RD. OVERPASS
BOREHOLE LOCATIONS AND SOIL
STRATA



SHEET



KEY PLAN

SCALE

2 0 2 4 km

LEGEND

- Borehole - Current Investigation
- ⊗ Borehole - Previous 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 October 17, 2016

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
BH-2W	105.2	4828422.5	299161.2
NW3-6	105.0	4828434.5	299136.6
SWM-A-1	104.2	4828407.6	299195.6
SWM-A-2	104.1	4828383.6	299214.1
SWM-A-3	103.1	4828385.7	299179.6

REFERENCE

Trenchless Crossing plan and profile provided in digital format by AECOM, drawing file no. 228D_nc_12+700.dwg, received February 6, 2024.
Hanlan Watermain plan provided in digital format by AECOM, drawing file no. WS_18_CONT2.dwg, received April 9, 2021.
Retaining walls plans provided in digital format by AECOM, drawing file nos. 04_Retaining Wall_New_24-887W.dwg and 07_Retaining Wall_NewPortion_24-888W.dwg, received January 18, 2018, R.Wall New 24-8XXW .dwg, received April 19, 2018.
Design plans provided in digital format by AECOM, drawing file nos. QEW_Dixie_Cont1_plan.dwg and QEW_Dixie_Cont2_plan.dwg, received July 21, 2017.
Watermain plan provided in digital format by AECOM, drawing file no. QEW_DixieC_UTL_PROP_WATERMAIN.dwg, received March 16, 2021.
Base plans provided in digital format by AECOM, drawing file nos. QEW_DixieC_base.dwg and QEW_DixieC_plan.dwg, dated July 20, 2016, received Dec. 06, 2016.
Existing ground contours provided in digital format by AECOM, drawing file no. QEW_DixieC_Contours3D.dwg, received Nov. 08, 2016, contour interval 0.5 m.
Key plan base data - MNR/LIO, obtained 2015.



NO.	DATE	BY	REVISION
Geocres No. 30M12-527	Latitude 43.595680, Longitude -79.569977		
HWY. QEW	PROJECT NO. 1530382-2010	DIST. Central	
SUBM'D. JS	CHKD. JS	DATE: 02/13/2024	SITE: -
DRAWN: ZS/SA/DD	CHKD. SEMP	APPD. SEMP	DWG. 1

APPENDIX A

Borehole and Drillhole Records



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
ν	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

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

(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 = σ'_p / σ'_{vo}

(d) Shear Strength

τ_p, τ_r	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction = $\tan \delta$
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

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

Notes: 1
2

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

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



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, 60° cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

PH: Sampler advanced by hydraulic pressure

PM: Sampler advanced by manual pressure

WH: Sampler advanced by static weight of hammer

WR: Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)

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

III. SOIL DESCRIPTION

(a) Non-Cohesive (Cohesionless) Soils

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	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



LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERINGS STATE

Fresh: no visible sign of weathering

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

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

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

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

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

BEDDING THICKNESS

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

JOINT OR FOLIATION SPACING

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

GRAIN SIZE

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

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

CORE CONDITION

Total Core Recovery (TCR)

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

Solid Core Recovery (SCR)

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

Rock Quality Designation (RQD)

The percentage of solid drill core, greater than 100 mm length, 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° 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	

FIELD ESTIMATION OF ROCK HARDNESS

Grade	Description	Field Identification	Approx. Range of UCS (MPa)
R0	Extremely Weak Rock	Indented by thumbnail	0.25 - 1
R1	Very Weak Rock	Material can be peeled or shaped with a knife. Crumbles under firm blows from geological hammer.	1 - 5
R2	Weak Rock	Knife cuts material but too hard to shape into triaxial specimens or material can be peeled with a knife with difficulty. Shallow (<5mm) indentations made by firm blows from pick of a geological hammer.	5 - 25
R3	Moderately Strong Rock	Cannot be peeled or scraped with a knife. Hand held specimens can be fractured with single firm blow of geological hammer.	25 - 50
R4	Strong Rock	Hand held specimen requires more than one blow of geological hammer to fracture.	50 - 100
R5	Very Strong Rock	Hand held specimen requires many blows of geological hammer to fracture.	100 - 250
R6	Extremely Strong Rock	Specimen can only be chipped under repeated hammer blows, rings when hit.	> 250

Notes:

1. Hand held specimens should have height approximately 2 times the diameter.
2. Materials having a uniaxial compressive strength of less than approximately 0.5 MPa and cohesionless materials should be classified using soil classification systems.
3. Rocks with a uniaxial compressive strength below 25 MPa (i.e. below R2) are likely to yield highly ambiguous results under point load testing.

Reference:

- Brown, 1981. "Suggested Methods for Rock Characterization Testing and Monitoring", International Society for Rock Mechanics.
- Hoek, E., Kaiser, P.K., Bawden, W.F., 1995. "Support of Underground Excavations in Hard Rock", Balkema, Rotterdam.

ROCK WEATHERING CLASSIFICATION

Term	Symbol	Description	Discoloration Extent	Fracture Condition	Surface Characteristics
Residual soil	W6	All rock material is converted to soil. The mass structure and material fabric are destroyed. There is a large change in volume, but the soil has not been significantly transported.	Throughout	N/A	Resembles soil
Completely weathered	W5	100% of rock material is decomposed and/or disintegrated to soil. The original mass structure is still largely intact.	Throughout	Filled with alteration minerals	Resembles soil
Highly weathered	W4	More than 50% of the rock material is decomposed and/or disintegrated to a soil. Fresh or discoloured rock is present either as a discontinuous framework or as corestones.	Throughout	Filled with alteration minerals	Friable and possibly pitted
Moderately weathered	W3	Less than 50% of the rock material is decomposed and/or disintegrated to a soil. Fresh or discoloured rock is present either as a discontinuous framework or as corestones. Visible texture of the host rock still preserved. Surface planes are weathered (oxidized or carbonate filling) even when breaking the "intact rock".	>20% of fracture spacing on both sides of fracture	Discoloured, may contain thick filling	Partial to complete discoloration, not friable except poorly cemented rocks
Slightly weathered	W2	Discoloration indicates weathering of rock material on discontinuity surfaces (usually oxidized). Less than 5% of rock mass altered.	<20% of fracture spacing on both sides of fracture	Discoloured, may contain thin filling	Partial discoloration
Fresh	W1	No visible sign of rock material weathering.	None	Closed or discoloured	Unchanged

Reference:

Brown, 1981. "Suggested Methods for Rock Characterization Testing and Monitoring", International Society for Rock Mechanics.

PROJECT		1530382		RECORD OF BOREHOLE No NW3-6				SHEET 1 OF 1		METRIC							
G.W.P.		2102-13-00; 2432-13-00		LOCATION		N 4828434.5; E 299136.6 MTM NAD 83 ZONE 10 (LAT. 43.595823; LONG. -79.570143)		ORIGINATED BY		PKS							
DIST		Central HWY QEW		BOREHOLE TYPE		108 mm O.D. Continuous Flight Solid Stem Augers		COMPILED BY		ACK							
DATUM		Geodetic		DATE		October 6, 2016		CHECKED BY		SMM							
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
105.0	GROUND SURFACE																
0.0	ASPHALT (150 mm)																
0.2	Sand and gravel (FILL) Brown Moist																
104.4	Silty SAND Compact Brown Moist																
0.6			1	SS	23		104										
103.6	CLAYEY SILT, trace to some sand, trace gravel Stiff to hard Brown Moist - Containing shale pieces below a depth of 1.8 m																
1.4			2	SS	10		103										3 11 51 35
102.6	SHALE (BEDROCK)																
2.4			3	SS	100/0.10												
			4	SS	100/0.13		102										
			5	SS	100/0.13		101										
100.3	END OF BOREHOLE		6	SS	100/0.10												
4.7	NOTE: 1. Open borehole dry upon completion of drilling.																

GTA-MTO 001 S:\CLIENTS\MTQ\QEW-DIXIE02_DATAGINT\QEW-DIXIE_GPJ GAL-GTA.GDT 4-16-18 GPK



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

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

SHEET 1 OF 1

DATUM: Geodetic

DRILLING CONTRACTOR: Davis Drilling Ltd.

[illegible]

FEATURES LEGEND



BROKEN CORE



CLAY SEAM



LIMESTONE



LOST CORE

DEPTH SCALE

1 : 50



GOLDER

LOGGED: PKS

CHECKED: CEC/AB

STA-RCK 054 S:\CLIENTS\MTQ\QEW-DIXIE\02 DATA\GIN\QEW-DIXIE.GPJ GAL-MISS.GDT 19-5-27



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

ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES AND TEST PITS

MINISTRY OF TRANSPORTATION, ONTARIO

PARTICLE SIZES OF CONSTITUENTS

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

MODIFIERS FOR SECONDARY COMPONENTS^{1,2}

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

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

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

PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

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

Cone Penetration Test (CPT)

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

Dynamic Cone Penetration Resistance (DCPT); N_d:

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

PH: Sampler advanced by hydraulic pressure

PM: Sampler advanced by manual pressure

WH: Sampler advanced by static weight of hammer

WR: Sampler advanced by weight of sampler and rod

SAMPLES

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

SOIL TESTS

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

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

COARSE-GRAINED SOILS

Compactness¹

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

- Definition of compactness terms are based on SPT 'N' ranges as provided in Terzaghi, Peck and Mesri (1996). Many factors affect the recorded SPT 'N' value, including hammer efficiency (which may be greater than 60% in automatic trip hammers), overburden pressure, groundwater conditions, and grain size. As such, the recorded SPT 'N' value(s) should be considered only an approximate guide to the soil compactness. These factors need to be considered when evaluating the results, and the stated compactness terms should not be relied upon for design or construction.
- SPT 'N' in accordance with ASTM D1586, uncorrected for the effects of overburden pressure.

FINE-GRAINED SOILS

Consistency

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

- SPT 'N' in accordance with ASTM D1586, uncorrected for overburden pressure effects; approximate only.
- SPT 'N' values should be considered ONLY an approximate guide to consistency; for sensitive clays (e.g., Champlain Sea clays), the N-value approximation for consistency terms does NOT apply. Rely on direct measurement of undrained shear strength or other manual observations.

Field Moisture Condition

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



PROJECT		1530382		RECORD OF BOREHOLE		No BH-2W		SHEET 1 OF 1		METRIC			
G.W.P.		2102-13-00; 2432-13-00		LOCATION		N 4828422.5; E 299161.2 MTM NAD 83 ZONE 10 (LAT. 43.595715; LONG. -79.569839)		ORIGINATED BY		AM			
DIST		Central HWY QEW		BOREHOLE TYPE		CME 75 truck, 203 mm OD Hollow Stem Augers		COMPILED BY		DP			
DATUM		Geodetic		DATE		February 1, 2024		CHECKED BY		JS			
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	W _p W W _L	WATER CONTENT (%)		
105.2	GROUND SURFACE												
0.0	ASPHALT (70 mm)												
0.1	CONCRETE (460 mm)												
104.7	Granular A (130 mm)												
0.7	SILTY SAND (SM), trace gravel, oxidation staining (FILL) Loose Brown Moist		1	SS	6								
103.7	SILTY SAND (SM) Compact Brown Wet		2	SS	11								
103.1	CLAYEY SILT-SILT (CL-ML), trace sand		3A	SS	55								
102.8	Hard Grey Moist		3B										
102.5	Inferred highly weathered grey SHALE (BEDROCK) (Georgian Bay Formation) Fresh		1	RC	REC 100%								RQD = 100%
2.7	Bedrock cored from depths of 2.7 m to 6.1 m (between Elev. 102.5 m and 99.1 m). For bedrock coring details refer to Record of Drillhole BH-2W		2	RC	REC 100%								RQD = 57%
			3	RC	REC 100%								RQD = 82%
99.2	END OF BOREHOLE												
6.1													

GTA-MTO 001 S:\CLIENTS\MTQEQW-DIXIE\02_DATA\INTQEQW-DIXIE.GPJ GAL-GTA.GDT 2/12/24

SHEET 1 OF 1

DATUM: Geodetic

DRILL RIG: CME 75 Truck Mounted

DRILLING CONTRACTOR: Pontil

[illegible]

LOGGED: AM

CHECKED: JS

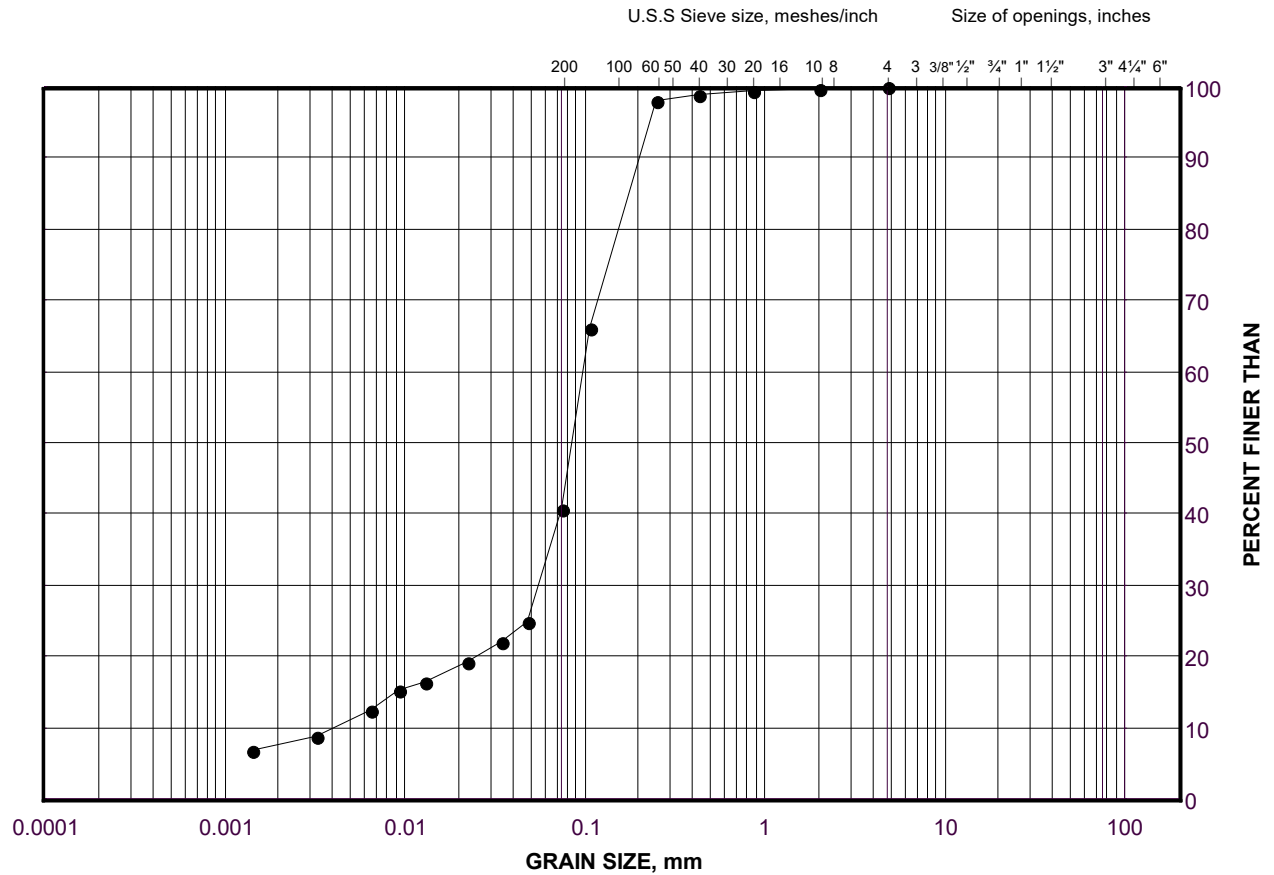
APPENDIX B

Geotechnical Laboratory Test Results

GRAIN SIZE DISTRIBUTION

SILTY SAND

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)
•	BH-2W	2	103.4

Project Number: 1530382

Checked By: SEMP

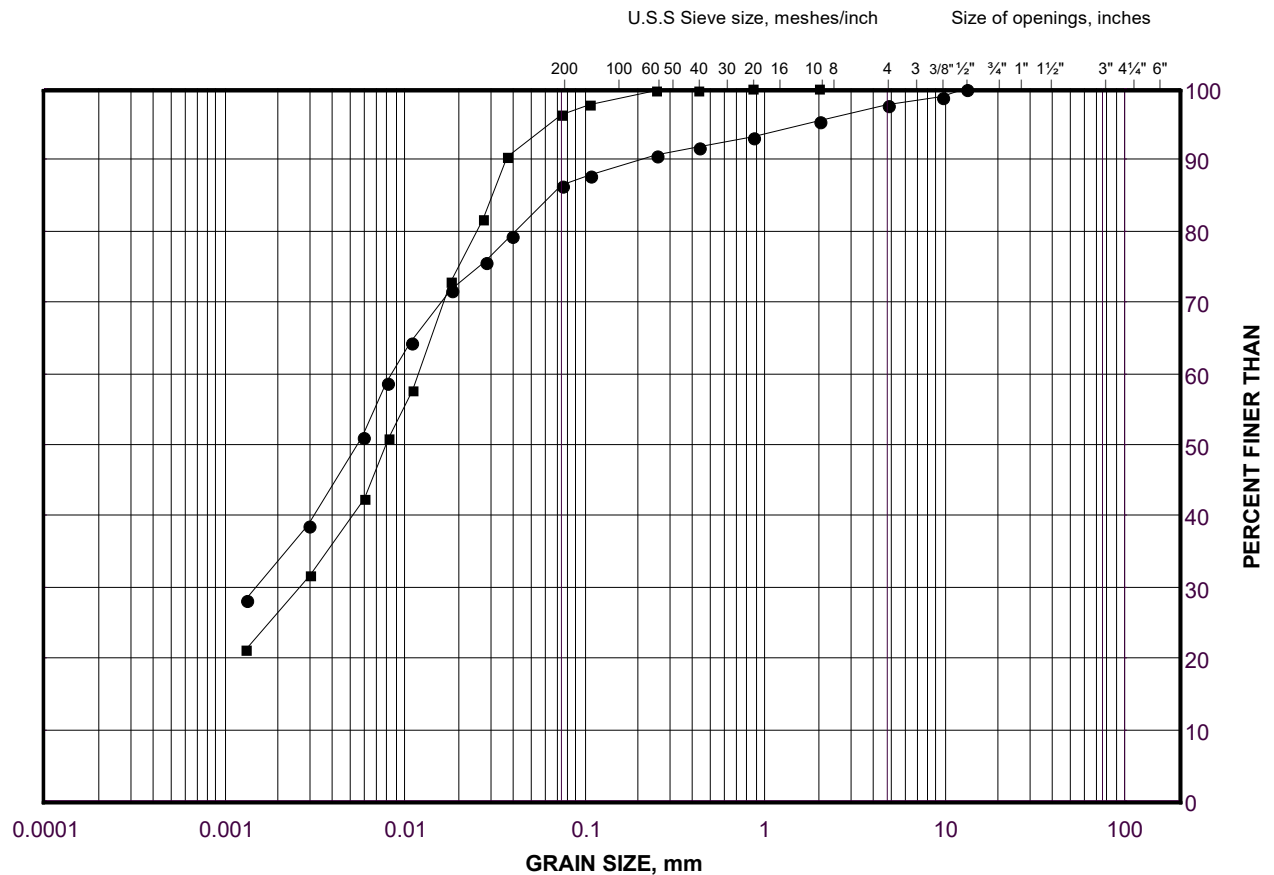
WSP Canada Inc

Date: 12-Feb-24

GRAIN SIZE DISTRIBUTION

CLAYEY SILT to SILT

FIGURE B-2



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

LEGEND

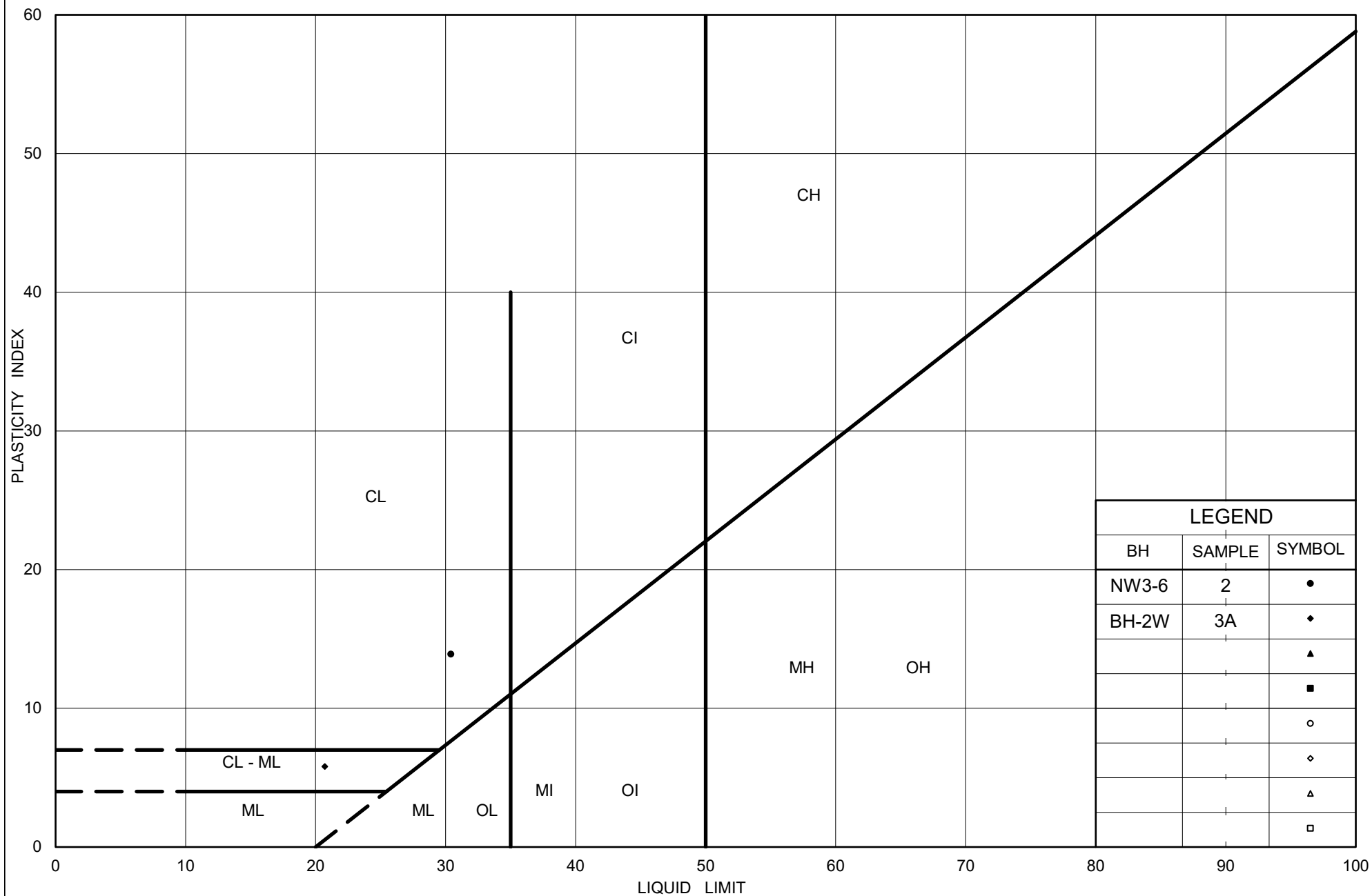
SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	NW3-6	2	103.4
■	BH-2W	3A	102.9

Project Number: 1530382

Checked By: SEMP

WSP Canada Inc

Date: 12-Feb-24



Ministry of Transportation

Ontario

PLASTICITY CHART CLAYEY SILT to SILT

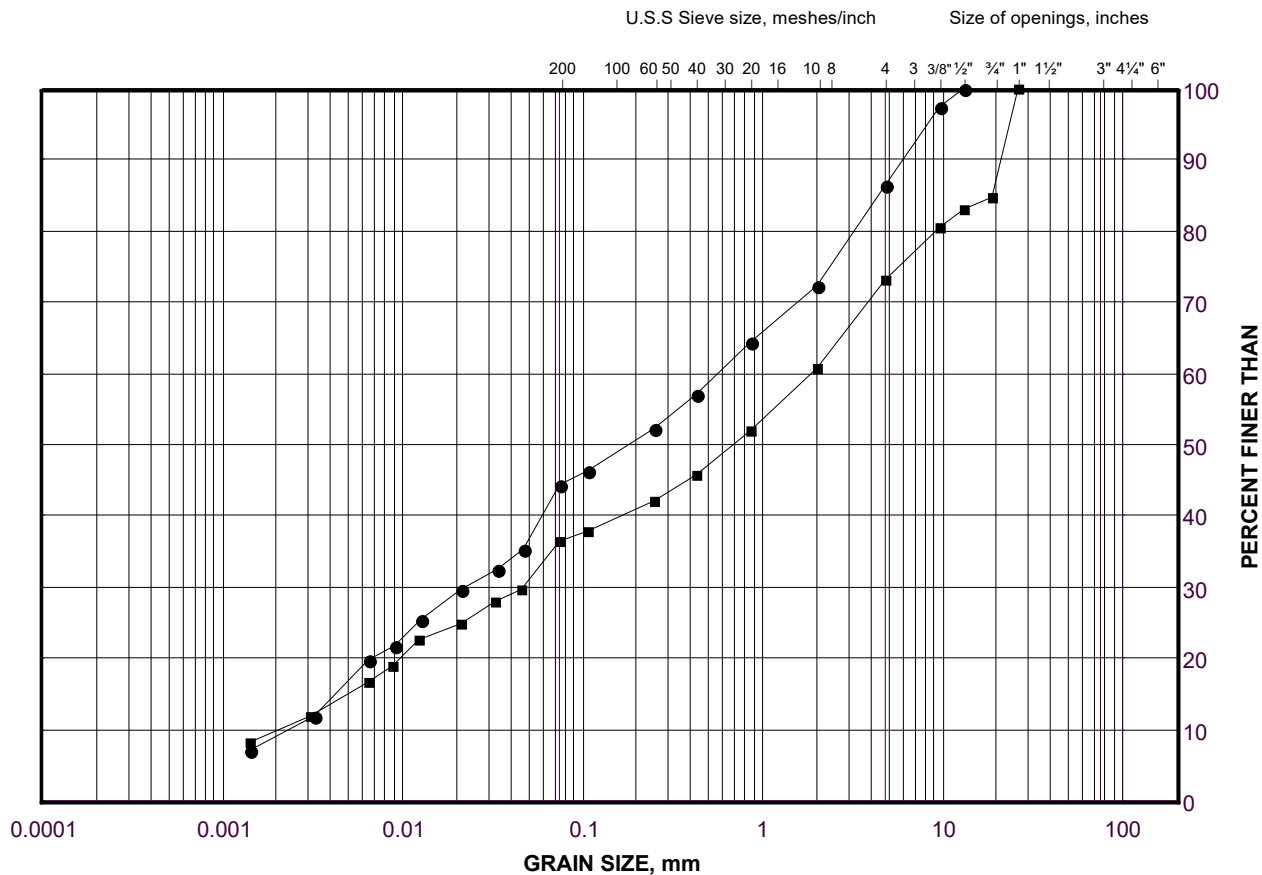
Figure No. B-3

Project No. 1530382

Checked By: SEMP

GRAIN SIZE DISTRIBUTION
GRAVELLY CLAYEY SILT with SAND (RESIDUAL SOIL)

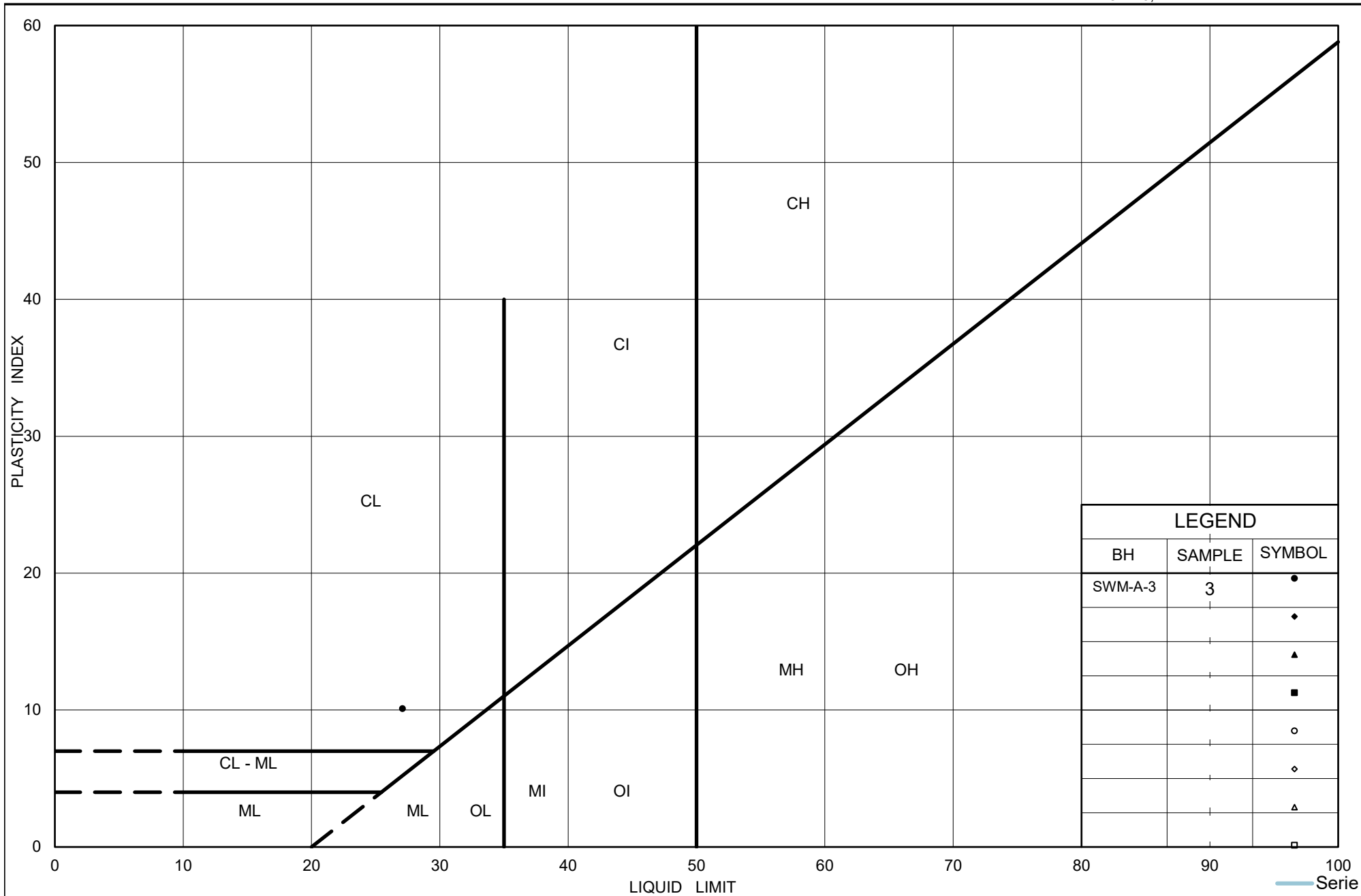
FIGURE B-4



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	SWM-A-3	2B	102.0
■	SWM-A-3	3	101.6



Ministry of Transportation

PLASTICITY CHART GRAVELLY CLAYEY SILT with SAND (RESIDUAL SOIL)

Ontario

Figure No. B-5

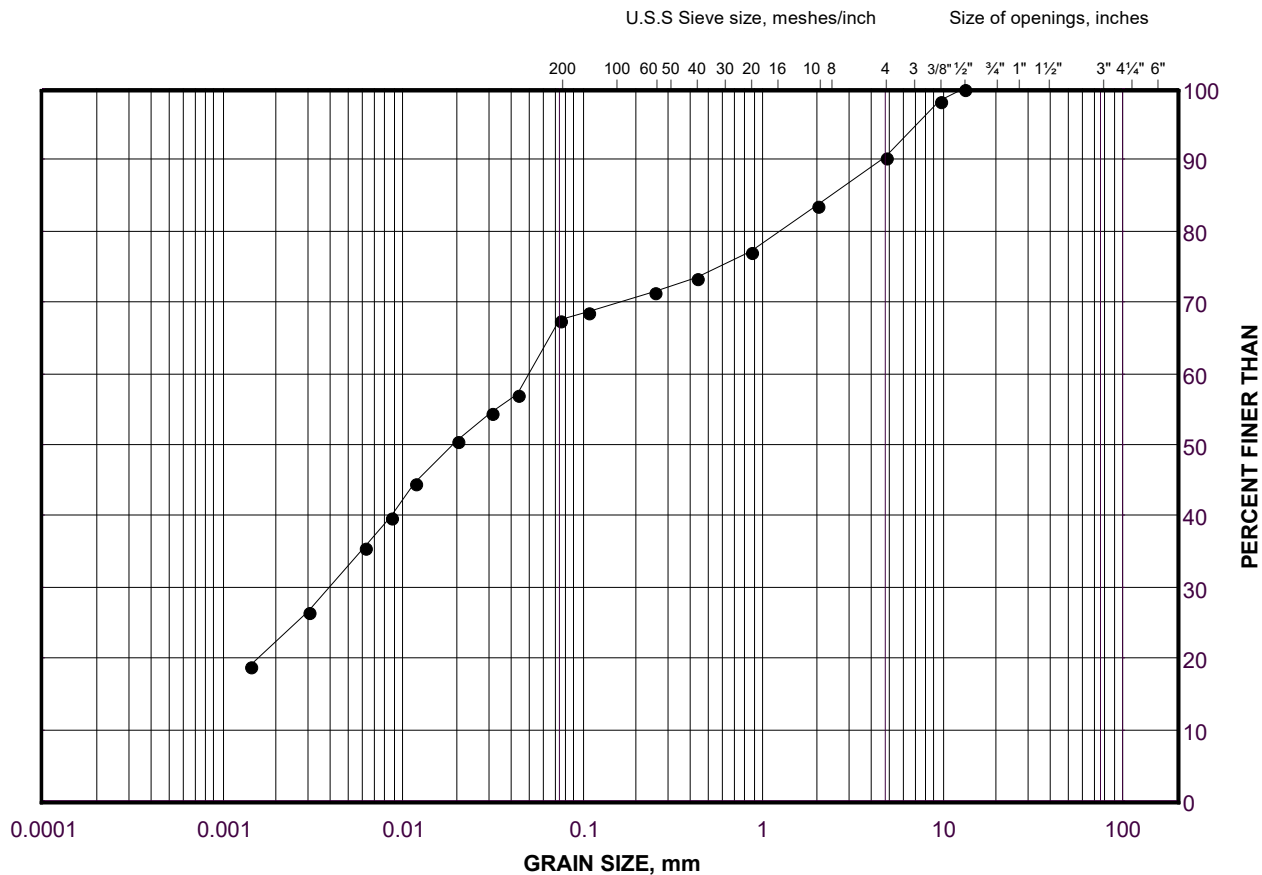
Project No. 1530382

Checked By: SEMP

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s29

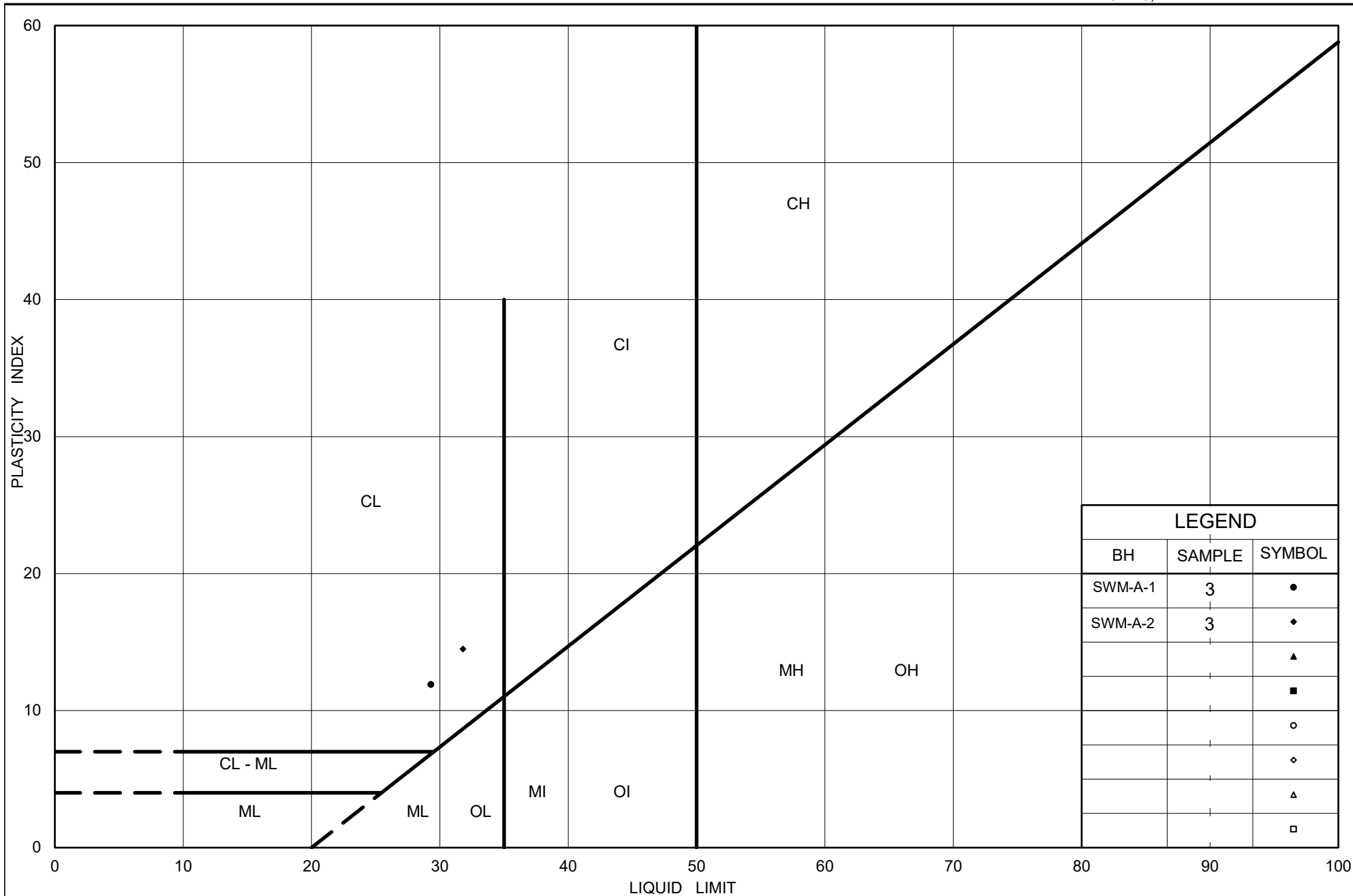
GRAIN SIZE DISTRIBUTION
INFERRED MODERATELY WEATHERED SHALE (BEDROCK)

FIGURE B-6



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
•	SWM-A-1	3	102.7



Ministry of Transportation

Ontario

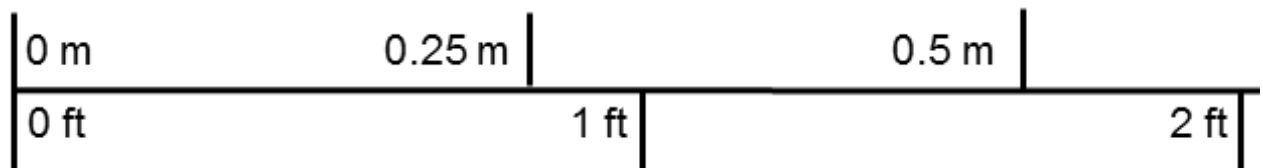
PLASTICITY CHART
INFERRED HIGHLY to MODERATELY
WEATHERED SHALE (BEDROCK)

Figure No. B-7

Project No. 1530382

Checked By: SEMP


Start of Run No. 1 (3.00 m)



Start of Run No. 2 (4.52 m)



End of Borehole (6.04 m)

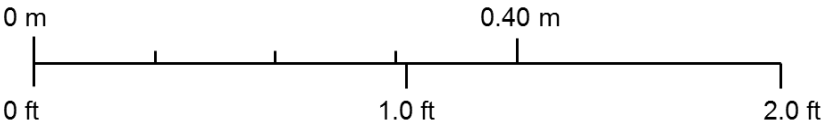
PROJECT				QEW IMPROVEMENT FROM EAST OF CAWTHRA ROAD TO EAST MALL			
TITLE				Bedrock Core Photographs Borehole SWM-A-2			
				PROJECT No. 1530382		FILE No. ----	
				DESIGN	MWK		SCALE NTS
				CADD	--		REV.
				CHECK	PG		FIGURE B-8
				REVIEW	JMAC		




Box 1: 2.44 m to 4.50 m



Box 2: 4.50 m to 6.05 m



Scale

PROJECT						
QEW IMPROVEMENT FROM EAST OF CAWTHRA TO EAST MALL						
TITLE						
Bedrock Core Photographs Borehole BH-2W						
	PROJECT No. 1530382			FILE No. ----		
	DRAFT	AM	20240201	SCALE	NTS	VER. 1.
	CADD	--		FIGURE B-9		
	CHECK	DP	20240206			
	REVIEW	JS	20240207			

December 16, 2016

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

Re: UCS Testing of shale samples - Golder Associates Project No. 1530382

Dear Ms. McGaghran:

On December 2, 2016 three (3) NQ-sized core samples were received by Geomechanica Inc. via drop-off. These samples were identified as shale from a drilling investigation near the QEW and Dixie Road in Mississauga, Ontario. Three (3) uniaxial compressive strength (UCS) test specimens were prepared and tested (one from each sample). The tangent elastic modulus was measured during one (1) of these three tests.

Details regarding the steps of specimen preparation and testing along with the test results and photographs of specimens 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:

Ms. Sandra McGaghran
Golder Associates Ltd.
6925 Century Avenue, Suite #100
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December 16, 2016

Project number: 1530382

Abstract

This document summarizes the results of Uniaxial Compressive Strength (UCS) testing of 3 NQ-sized rock core samples for Golder Associates Ltd. (Golder Project No. 1530382). The samples were identified as shale from a drilling investigation near the QEW and Dixie Road in Mississauga Ontario. The results, including the tabulated values of the UCS, bulk density, and elastic modulus along with photos of the test specimens before and after testing, are presented herein.

In this document:

1	Uniaxial Compressive Strength Tests	1
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1 Uniaxial Compressive Strength Tests

1.1 Introduction

This section summarizes the results of rock laboratory testing of NQ-sized shale samples under unconfined uniaxial compression. The tests were performed in Geomechanica's rock testing laboratory in Vaughan, Ontario using a 150 ton (1.3 MN) Forney hydraulic loading frame equipped with pressure-compensated control valve to maintain a nearly constant axial displacement rate of 0.1 mm/min (Figure 1). 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 maintain the moisture content and avoid breakage during handling and preparation.
2. Diamond sawing the core samples to length such that cylindrical specimens with nearly parallel end faces were obtained. When possible, specimens were cut such that they had a length:diameter ratio of at least 2:1. For this project, 1 out of the 3 core samples provided was too short to obtain the desired length to diameter ratio.
3. Surface grinding of specimens to obtain flat and parallel end faces within ± 0.025 mm.
4. Loading the specimens into a stiff hydraulic loading frame and applying a small axial load of 0.5-1.0 kN, removing of the electrical tape, and subsequently loading the specimen to rupture while continuously recording axial force and axial deformation (for select specimens) to determine the peak strength (UCS) and (tangent) Young's modulus (E) (for select specimens).



Figure 1: Forney loading frame used for uniaxial compression testing.

1.2 Results

The results of UCS testing are summarized in Table 1. The stress-strain curve for CV 02/03-1 is shown in Figure 2. The Young's modulus value presented in Table 1 represents 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 UCS.

Table 1: Summary of UCS test results.

Sample	Rock type	Depth from (m)	Depth to (m)	Bulk density (g/cm ³)	UCS (MPa)	Young's modulus, E_{50} (GPa)	Notes
SWM-A-2	Shale	5.10	5.30	2.59	17.7	-	
CV 02/03-1	Shale	7.47	7.70	2.60	17.6	1.2	¹
HML-1	Shale	7.41	7.50	2.59	17.8	-	²
Min				2.59	17.6	1.2	
Max				2.60	17.8	1.2	
Mean				2.59	17.7	1.2	
Standard Deviation				0.01	0.1	-	

¹ Top 25 mm of specimen is limestone
² Specimen length:diameter < 2:1

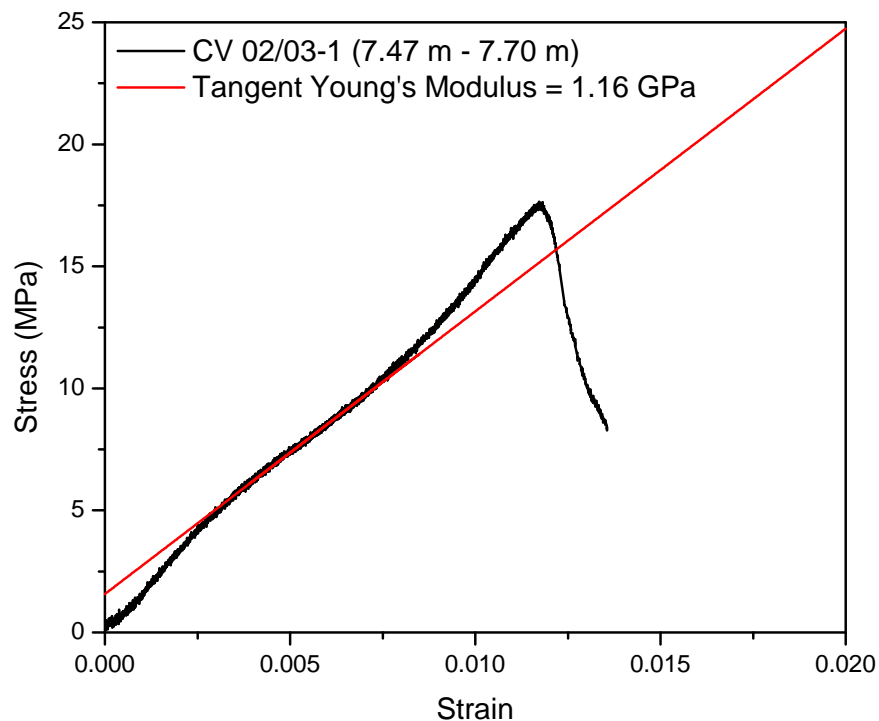


Figure 2: Measured stress-strain curves for samples from different boreholes.

1.3 Specimen photographs

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



Figure 3: Photographs of test specimens before testing (top) and after testing (bottom).

