



REPORT

Foundation Investigation Report

*MH520-MH591 Storm Sewer Trenchless Crossing East 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

30 Leek Crescent
Richmond Hill, ON L4B 4N4

Submitted by:

WSP Canada Inc.

100 Scotia Court, Whitby, Ontario, L1N 8Y6, Canada

1530382-E-REV1

February 13, 2024

GEOCRES No.: 30M12-526

Latitude: 43.597710°

Longitude: -79.567789°

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Table of Contents

FOUNDATION INVESTIGATION REPORT

1.0 INTRODUCTION1

2.0 SITE DESCRIPTION1

3.0 INVESTIGATION PROCEDURES1

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS.....3

 4.1 Regional Geology.....3

 4.2 Subsurface Conditions3

 4.2.1 Topsoil.....3

 4.2.2 Asphalt3

 4.2.3 Concrete.....3

 4.2.4 Fill.....4

 4.2.5 Silt and Sand to Sand4

 4.2.6 Gravelly Sand to Sand and Gravel4

 4.2.7 Clayey Silt (Residual Soil).....5

 4.2.8 Bedrock.....5

 4.3 Groundwater Conditions7

5.0 CLOSURE7

DRAWINGS

Drawing 1 Borehole Locations and Soil Strata

APPENDICES

APPENDIX A

Borehole and Drillhole Records

APPENDIX B

Geotechnical Laboratory Test Results

- Figure B-1 Grain Size Distribution – SAND to SILT and SAND
- Figure B-2 Grain Size Distribution – GRAVELLY SAND to SAND and GRAVEL
- Figure B-3 Grain Size Distribution – CLAYEY SILT (RESIDUAL SOIL)
- Figure B-4 Plasticity Chart – CLAYEY SILT (RESIDUAL SOIL)
- Figures B-5A/B, B-6, B7 Bedrock Core Photographs – Boreholes 21-35, MP-2, BH-2E
- Rock Laboratory Test Results

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 MH520 and MH591, located east of the QEW-Dixie Road Underpass at approximately Station 12+975 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 the North Service Road.

The natural ground surface in the vicinity of the crossing is at about Elevations 107 m to 108 m. The existing Hanlan 2400 mm concrete pressure pipe feedermain is located mainly parallel to but at a slight skew to the proposed crossing but at a much greater depth, with its obvert at about Elevation 96 m. The sewer is required for overall interchange site drainage.

3.0 INVESTIGATION PROCEDURES

Borehole BH-2E was drilled on February 2, 2024, near the middle of the crossing between MH520 and MH591. Boreholes 21-35 and MP-2 were drilled just north of MH520 in January 2021 and February 2018, respectively. Borehole NW6-3 was drilled in October 2016 southeast of MH591. 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 truck-mounted CME55 or CME75, supplied and operated by specialized drilling contractors. The boreholes were advanced through the overburden using 108 mm outside diameter solid-stem augers or 150 mm to 203 mm outside diameter hollow stem augers. 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 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 layer of bedrock. 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. Samples of the bedrock were also obtained using an 'HQ'-size rock core barrel and coring techniques. The groundwater conditions were noted in the boreholes upon completion of drilling.

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 MP-2, 21-35, and BH-2E in the field based on visual observation and measurement. The bedrock was sequentially photographed, packed, and transported to WSP's Mississauga laboratory for further visual examination. Laboratory testing consisting of Uniaxial Compressive Strength (UCS), CERCHAR Abrasivity, and Slake Durability testing was carried out on a select specimen of the bedrock core samples, by Geomechanica of Mississauga, Ontario, and Point Load Index testing was carried out on selected bedrock samples at Golder's Mississauga Laboratory.

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 and grain size distributions 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 as-drilled borehole location and the ground surface elevation was obtained using a GPS (Trimble XH 3.5G or Geo 7X), 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)
NW6-3	4,828,635.2 (43.597632)	299,356.3 (-79.567427)	107.0	4.7
21-35	4,828,658.3 (43.597847)	299,282.2 (-79.568346)	106.4	10.9
BH-2E	4,828,641.2 (43.597685)	299,333.1 (-79.567711)	107.3	8.0
MP-2	4,828,677.9 (43.598024)	299,287.8 (-79.568277)	106.9	9.3

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, topsoil or fill underlain by a deposit of silt and sand to sand and gravelly sand to sand and gravel which is in turn underlain by shale bedrock. A deposit of clayey silt residual soil overlies the shale bedrock in two boreholes. The groundwater level ranges from 1.5 m to 4.4 m below ground surface. A more detailed description of the soil deposits at the site is provided in the following sub-sections.

4.2.1 Topsoil

A 0.4 m thick layer of topsoil was encountered at the ground surface in Borehole MP-2.

4.2.2 Asphalt

Borehole NW6-3 was advanced through the South Service Road surface and encountered an approximately 180 mm thick layer of asphalt at ground surface. Borehole BH-2E 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 430 mm layer of concrete was encountered beneath the asphalt layer at Borehole BH-2E.

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

A 0.2 m thick deposit of Granular 'A' or sand and gravel fill was encountered underlying the asphalt in Borehole NW6-3 and underlying the concrete in Borehole BH-2E. The surface of the sand and gravel fill deposit was encountered at Elevation 106.8 m in both boreholes and extends to a depth of about 0.4 m below ground surface in Borehole NW6-3 and 0.7 m in Borehole BH-2E (Elevation 106.6 m).

A 1.5 m and 1.4 m thick deposit of silty sand fill was encountered at ground surface in Borehole 21-35 and underlying the Granular 'A' in Borehole BH-2E, respectively. The deposit was noted to have oxidation staining in Borehole BH-2E. The surface of the silty sand fill deposit was encountered at Elevation 106.4 m in Borehole 21-35 and Elevation 106.6 m in Borehole BH-2E and extends to a depth of about 1.5 m below ground surface in Borehole 21-35 and 2.1 m below ground surface in Borehole BH-2E (between Elevations 104.9 m and 105.2 m in the respective boreholes).

The SPT "N"-values measured within the fill range from 5 to 13 blows per 0.3 m of penetration, indicating a loose to compact state of compactness.

The water content measured on two samples of the fill deposit are about 8% and 12%.

4.2.5 Silt and Sand to Sand

A 2.7 m thick deposit of silt and sand was encountered below the fill in Borehole NW6-3. The surface of the granular deposit was encountered at a depth of 0.4 m below ground surface (Elevation 106.6 m) and extends to a depth of 3.1 m below ground surface (Elevation 103.9 m).

In Boreholes 21-35, MP-2, and BH-2E, a 0.7 m to 1.7 m thick deposit of sand, trace silt to silty sand, trace gravel was encountered below the fill in Boreholes 21-35 and BH-2E, and below the topsoil in Borehole MP-2. The deposit was noted to have trace organics in Borehole MP-2. The surface of the sand deposit was encountered at depths ranging from about 0.4 m to 2.1 m below ground surface (between Elevations 106.5 m to 104.9 m), and extends to depths ranging from about 2.1 m to 3.7 m below ground surface (between Elevations 104.8 m to 103.6 m).

The SPT "N"-values measured within the granular deposit range from 17 to 28 blows per 0.3 m of penetration, indicating a compact state of compactness.

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

The water content measured on seven samples of the granular deposit ranges from about 10% to 22%.

4.2.6 Gravelly Sand to Sand and Gravel

A 0.6 m thick deposit of gravelly sand, some silt, trace clay was encountered underlying the silt and sand deposit in Borehole NW6-3. The surface of the granular deposit was encountered at a depth of about 3.1 m below ground surface (Elevation 103.9 m) and extends to a depth of 3.7 m below ground surface (Elevation 103.3 m).

In Borehole MP-2, A 0.5 m thick deposit of sand and gravel, some silt, trace clay was encountered underlying the silty sand deposit. The surface of the granular deposit was encountered at a depth of about 2.1 m below ground surface (Elevation 104.8 m) and extends to a depth of about 2.6 m below ground surface (Elevation 104.3 m).

One SPT “N”-value measured within the granular deposit is 26 blows per 0.3 m of penetration, indicating a compact state of compactness. One “N”-value was 68 blows per 0.23 m of penetration, likely as a result of the underlying shale bedrock and not representative of the sand and gravel.

Grain size distribution testing was carried out on two samples of the deposit and the results are presented on Figure B-2 in Appendix B.

The water content measured on two samples of the granular deposit is 11%.

4.2.7 Clayey Silt (Residual Soil)

A 0.3 m and 0.1 m thick deposit of residual soil was encountered above the shale bedrock, underlying the gravelly sand to silt and sand deposit in Borehole NW6-3 and underlying the silty sand deposit in Borehole BH-2E, respectively. The surface of the residual soil deposit was encountered at a depth of about 3.7 m below ground surface in both boreholes (Elevations 103.3 m and 103.6 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, some sand, some gravel. The deposit was noted to contain shale fragments in Borehole NW6-3.

The SPT “N”-value measured within the residual soil deposit is 100 blows for 0.15 m of penetration, indicating refusal of penetration within the deposit. In Borehole BH-2E, one SPT “N”-value of 18 blows per 0.3 m of penetration was measured suggesting a very stiff consistency, however, this value was likely influenced by the silty sand deposit above.

Grain size distribution testing was carried out on one sample of the residual soil deposit and the results are presented on Figure B-3 in Appendix B. Atterberg limits testing was carried out on one sample of the deposit and the results are presented on Figure B-4 in Appendix B. The Atterberg limits testing measured a liquid limit of about 26%, a plastic limit of about 16%, and a plasticity index of about 10%, indicating the material is a clayey silt of low plasticity.

The water content measured on one sample of the residual soil is about 19%.

4.2.8 Bedrock

Bedrock was encountered underlying the residual soil and clayey silt deposit in Boreholes NW6-3 and BH-2E and underlying the sand and gravel to silty sand deposit in Boreholes MP-2 and 21-35. The bedrock was sampled by split-spoon sampling in all boreholes and was also proven by coring in Boreholes 21-35, MP-2, and BH-2E using HQ sized coring equipment. Photos of the rock core are presented on Figures B-5A/B, B-6, and B-7.

The depth to bedrock below ground surface, as inferred from split spoon sampling and 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)	
NW6-3	107.0	4.0 - 4.7	103.0 – 102.3	0.7	-	-	Split Spoon Sample Only
21-35	106.4	2.2 – 5.0	104.2 – 101.4	0.7	5.0 – 10.9	101.4 – 95.5	8.0

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)	
BH-2E	107.3	3.8 – 4.0	103.5 – 103.3	0.2	4.0 – 8.0	103.3 – 99.3	4.0
MP-2	106.9	2.6 – 4.6	104.3 – 102.3	1.1	4.6 – 9.3	102.3 – 97.6	5.6

The inferred depths to the bedrock surface and thicknesses of the deposits provided in the table above were based on drilling behaviour, observations of drilling cuttings, and split-spoon sampling.

The SPT “N”-values measured within the inferred highly to slightly weathered bedrock obtained by split-spoon sampling range from 28 blows per 0.3 m of penetration and up to 100 blows for 0.13 m of penetration, suggesting a very stiff consistency to refusal of penetration within the rock mass.

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 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 ranges from about 0% to 100%, indicating a rock mass of very poor to excellent quality as per Table 3.10 of CFEM (2006)³. The Total Core Recovery (TCR) ranges from about 18% to 100% and the Solid Core Recovery (SCR) ranges from about 0% to 100% per cent, respectively.

Two Unconfined Compression (UC) tests (ASTM D7012) were carried out on a selected core sample of the shale bedrock obtained in Boreholes 21-35 and MP-2. The results are 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)
21-35	7.4 – 7.6	99.0 – 98.8	7.3	2.6
MP-2	7.8 – 8.0	99.1 – 98.9	20.4	2.6

The core recovered from Borehole BH-2E is considered extremely weak to weak as it was not possible to select a sample for testing without the core breaking. Based on field review of the rock core and the laboratory UCS

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.

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).

The results of CERCHAR abrasivity testing completed on one shale bedrock sample indicates a CERCHAR-Abrasivity-Index (CAI) of 0.282, corresponding to a very low ASTM Classification. The results are 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)	Mean Wear (mm)	CAI (Mean Wear x 10)	ASTM Classification
21-35	RC4	7.6 – 7.9	98.8 – 98.5	0.028	0.282	< Very Low

The results of slake durability testing completed on one shale bedrock sample 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)	Moisture Content (%)	Slake Durability Index (%)	
					1 st Cycle	2 nd Cycle
21-35	RC3	6.4 – 6.6	100.0 – 99.8	3.5	87.6	73.1

The results of two point load index tests completed on two shale bedrock samples from Borehole 21-35 are summarized in the table below and the details are presented in Appendix B.

Borehole	Sample	Depth (m)	Elevation (m)	Orientation	Axial Is (50mm) (MPa)
21-35	RC3	5.5 – 5.6	100.9 – 100.8	Axial	5.091
21-35	RC3	6.1 – 6.2	100.3 – 100.2	Axial	2.477

4.3 Groundwater Conditions

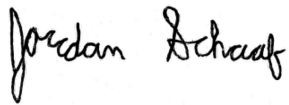
Details of the water levels observed in the boreholes upon completion of drilling are summarized on the Record of Boreholes in Appendix A. In general, the overburden samples obtained were moist and transitioned to wet at depth of about 1.5 m to 2.7 m (Elevation 105.4 m to 104.4 m) in Boreholes 21-35, NW6-3, MP-2, and BH-2E. The water level inside the boreholes upon completion of drilling or prior to rock coring was measured to range from 1.5 m to 4.4 m (Elevation 104.9 m to 102.6 m) in 21-35 and NW6-3 respectively and Borehole MP-2 was dry upon completion of drilling and prior to start of coring. These observations are not considered representative of the stabilized groundwater level at the site. 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.

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 an independent quality control review of the report.

Signature Page

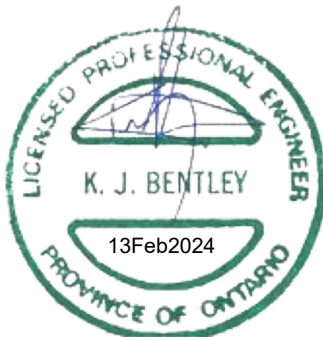
WSP Canada Inc.



Jordan Schaaf
Associate Geotechnical Analyst



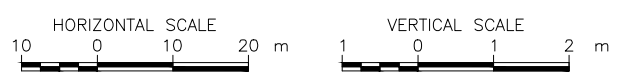
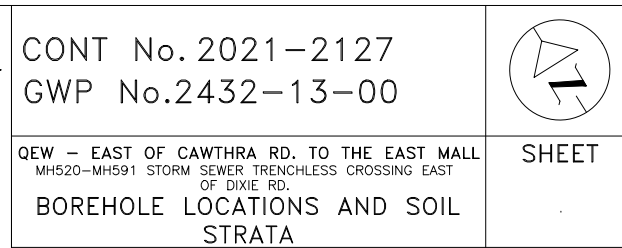
Sarah E M. Poot, P.Eng.
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JNS/KJB/SEMP/al

[https://golderassociates.sharepoint.com/sites/19542g/1 foundations/09 - reports/26 - trenchless sewer 2024/4 - rev1 final/3.east mh520-mh591/1530382 fir rev1 2024/02'13-mh520-mh591 trenchless.docx](https://golderassociates.sharepoint.com/sites/19542g/1%20foundations/09%20-%20reports/26%20-%20trenchless%20sewer%202024/4%20-%20rev1%20final/3.east%20mh520-mh591/1530382%20fir%20rev1%202024/02'13-mh520-mh591%20trenchless.docx)



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

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
21-35	106.4	4828658.3	299282.2
BH-2E	107.3	4828641.2	299333.1
MP-2	106.9	4828677.9	299287.8
NW6-3	107.0	4828635.2	299356.3

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 24-8XW .dwg, received April 19, 2018.
 Design plans provided in digital format by AECOM, drawing file nos. QE_W_Dixie_Cont1_plan.dwg and QE_W_Dixie_Cont2_plan.dwg, received July 21, 2017.
 Watermain plan provided in digital format by AECOM, drawing file no. QE_W_DixielC_UTL_PROP_WATERMAIN.dwg, received March 16, 2021.
 Base plans provided in digital format by AECOM, drawing file nos. QE_W_DixielC_base.dwg and QE_W_DixielC_plan.dwg, dated July 20, 2016, received Dec. 06, 2016.
 Existing ground contours provided in digital format by AECOM, drawing file no. QE_W_DixielC_Contours3D.dwg, received Nov. 08, 2016, contour interval 0.5 m.
 Key plan base data – MNRF LIO, obtained 2015.

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.



NO.	DATE	BY	REVISION	
Geocres No. 30M12-526. Latitude 43.597710, Longitude -79.567789				
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.

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

PROJECT: 1530382-7000

RECORD OF DRILLHOLE: MP-2

SHEET 1 OF 1

LOCATION: N 4828677.9 ;E 299287.8

DRILLING DATE: February 1, 2018

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 55 (Track Mounted)

DRILLING CONTRACTOR: Aardvark Drilling Inc.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	NOTE: For abbreviations, symbols and descriptions refer to LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY														FEATURES																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																													
						FLUSH RETURN	RECOVERY		R.Q.D. %	FRACT. INDEX PER 0.25m	DIP w.r.t CORE AXIS	DISCONTINUITY DATA		HYDRAULIC CONDUCTIVITY K, cm/sec				WEATH- ERING INDEX																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																															
							TOTAL CORE %	SOLID CORE %				Jr	Js	10 ⁻⁶	10 ⁻⁵	10 ⁻⁴	10 ⁻³	W1	W2		W3	W4	W5	W6																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																									
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DEPTH SCALE

1 : 50

LOGGED: AJ

CHECKED: AKV

PROJECT		1530382		RECORD OF BOREHOLE No NW6-3		SHEET 1 OF 1		METRIC						
G.W.P.		2102-13-00; 2432-13-00		LOCATION		N 4828635.2; E 299356.3 MTM NAD 83 ZONE 10 (LAT. 43.597632; LONG. -79.567427)		ORIGINATED BY						
DIST		Central HWY QEW		BOREHOLE TYPE		108 mm O.D. Continuous Flight Solid Stem Augers		COMPILED BY						
DATUM		Geodetic		DATE		October 14, 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						
107.0	GROUND SURFACE													
0.0	ASPHALT (180 mm)													
106.8														
106.6	Sand and gravel (FILL) Brown Moist													
0.4														
	SILT and SAND Compact Brown Moist to wet below 2.6 m depth													
			1	SS	24									
			2	SS	17									
			3	SS	28									
103.9														
3.1	Gravelly SAND, some silt, trace clay Compact Brown Wet		4	SS	26									
103.3														
3.7	CLAYEY SILT, some sand, some gravel, shale fragments (RESIDUAL SOIL)													
103.0			5	SS	100/0.15									
4.0	Hard Brown Wet													
	SHALE (BEDROCK)													
102.3			6	SS	100/0.13									
4.7	END OF BOREHOLE													
	NOTE: 1. Water level in open borehole at a depth of 4.4 m below ground surface (Elev. 102.6 m) upon completion of drilling.													

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 1530382-7000		RECORD OF BOREHOLE No 21-35		SHEET 1 OF 2		METRIC	
G.W.P. 2102-13-00; 2432-13-00		LOCATION N 4828658.3; E 299282.2 MTM NAD 83 ZONE 10 (LAT. 43.597847; LONG. -79.568346)		ORIGINATED BY LM			
DIST Central HWY QEW		BOREHOLE TYPE CME 55, 150 mm O.D. Hollow Stem Augers (Auto Hammer)		COMPILED BY SK			
DATUM Geodetic		DATE January 13, 2021		CHECKED BY KCP			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)							
								20	40	60	80	100	W _P	W	W _L					
106.4	GROUND SURFACE																			
0.0	SILTY SAND (SM) (FILL) Loose to compact Brown Moist		1	SS	5	▽	106													
			2	SS	13		105													
105.0																				
1.5	SAND (SP-SM), trace silt Compact Brown Wet		3	SS	26		104													
104.2																				
2.2	Inferred highly weathered, grey SHALE		4	SS	28		103													
103.5																				
2.9	Grey SHALE (BEDROCK) Bedrock cored from depths of 2.9 m to 10.9 m (between Elev. 103.5 m and 95.5 m). For bedrock coring details refer to Record of Drillhole BH21-35.		1	RC	REC 75%	102														
			2	RC	REC 97%	101														
			3	RC	REC 100%	100														
			4	RC	REC 100%	99														
			5	RC	REC 100%	98														
			6	RC	REC 100%	97														

Continued Next Page

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

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

SHEET 1 OF 1

DATUM: Geodetic

DRILLING CONTRACTOR: Davis Drilling Ltd.

CHECKED: SK



PROJECT		RECORD OF BOREHOLE		No BH-2E		SHEET 1 OF 1		METRIC								
G.W.P. 2102-13-00; 2432-13-00		LOCATION		N 4828641.2; E 299333.1 MTM NAD 83 ZONE 10 (LAT. 43.597685; LONG. -79.567711)		ORIGINATED BY		AM								
DIST Central HWY QEW		BOREHOLE TYPE		CME 75 truck; 203 mm O.D. Hollow Stem Auger		COMPILED BY		DP								
DATUM Geodetic		DATE		February 2, 2024		CHECKED BY		JS								
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)
107.3	GROUND SURFACE						20	40	60	80	100					
0.0	ASPHALT (70 mm)															
0.1	CONCRETE (430 mm)															
106.8	Granular A (180 mm)															
106.6																
0.7	SILTY SAND (SM), trace gravel, oxidation staining (FILL) Loose to compact Brown Moist		1	SS	5											
			2	SS	11											
105.2	SILTY SAND (SM), trace gravel Compact Brown Moist to wet		3	SS	24											
2.1			4	SS	22											
			5A	SS	18											
103.6	CLAYEY SILT (CL), some sand, trace gravel (RESIDUAL SOIL) Very stiff Brown Wet		5B													
4.0	Inferred highly weathered grey SHALE (BEDROCK) (Georgian Bay Formation) Fresh		1	RC	REC 100%											
	Bedrock cored from depths of 4.0 m to 8.0 m (between Elev. 103.3 m and 99.3 m).															
	For bedrock coring details refer to Record of Drillhole BH-2E.															
			2	RC	REC 100%											
			3	RC	REC 100%											
99.3	END OF BOREHOLE															
8.0																

GTA-MTO 001 S:\CLIENTS\MTQ\QEW-DIXIE02_DATA\INT\QEW-DIXIE.GPJ GAL-GTA.GDT 2/12/24

[illegible]

APPENDIX B

Geotechnical Laboratory Test Results

SAND to SILT and SAND

The graph displays the grain size distribution for three different soil samples. The x-axis is labeled 'GRAIN SIZE, mm' and ranges from 0.0001 to 100 on a logarithmic scale. The y-axis is labeled 'PERCENT FINER THAN' and ranges from 0 to 100 on a linear scale. The top x-axis also shows U.S.S. Sieve size, meshes/inch, and the size of openings in inches.

The three curves represent different soil types:

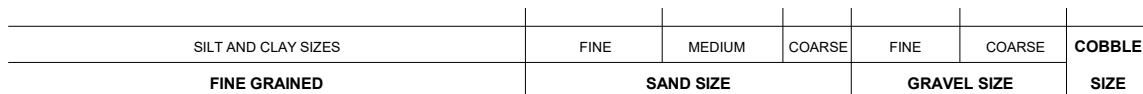
- Top Curve (Solid line with circles):** This curve represents a fine-grained soil. It shows that approximately 100% of the soil is finer than 0.075 mm (No. 20 sieve) and approximately 95% is finer than 0.06 mm (No. 25 sieve).
- Middle Curve (Dashed line with diamonds):** This curve represents a medium-grained soil. It shows that approximately 100% of the soil is finer than 0.425 mm (No. 40 sieve) and approximately 80% is finer than 0.25 mm (No. 60 sieve).
- Bottom Curve (Solid line with squares):** This curve represents a coarse-grained soil. It shows that approximately 100% of the soil is finer than 0.85 mm (No. 20 sieve) and approximately 20% is finer than 0.075 mm (No. 20 sieve).

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

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	NW6-3	2	105.2
■	21-35	3	104.9
◆	BH-2E	4	104.3

Date: 12-Feb-24

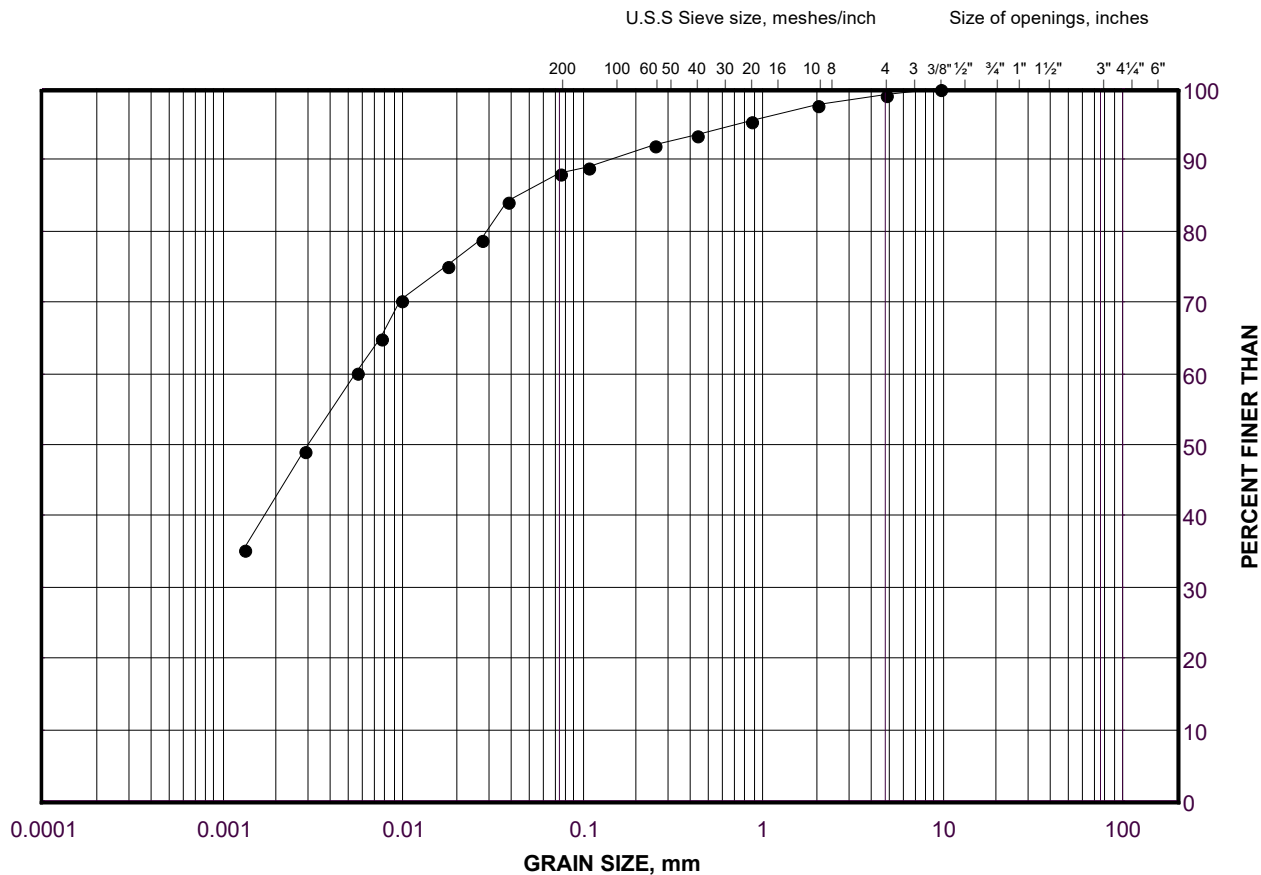
FIGURE B-2

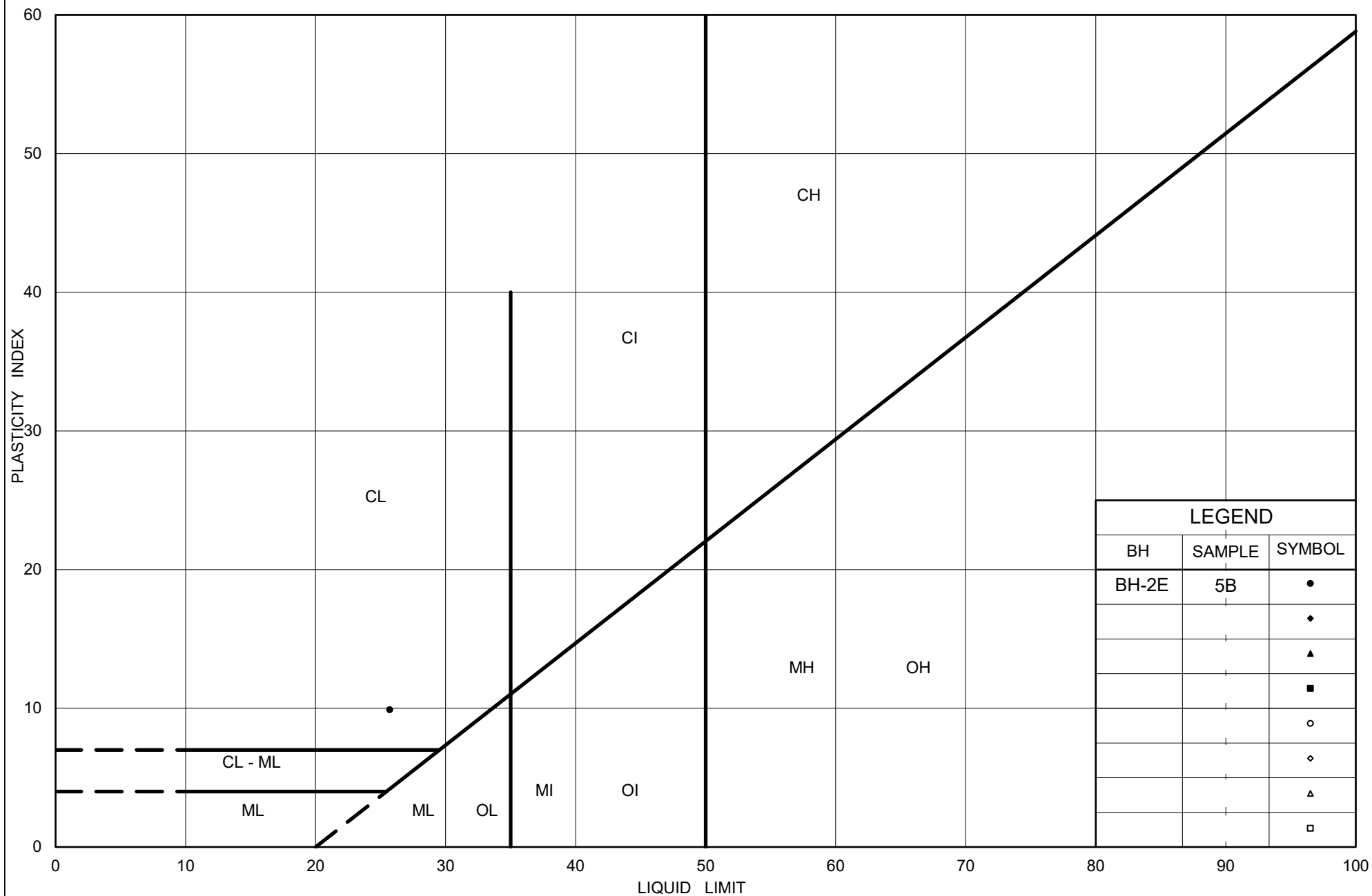


SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	NW6-3	4	103.6
■	MP2	4A	104.3

GRAIN SIZE DISTRIBUTION
CLAYEY SILT (RESIDUAL SOIL)

FIGURE B-3





Ministry of Transportation

Ontario

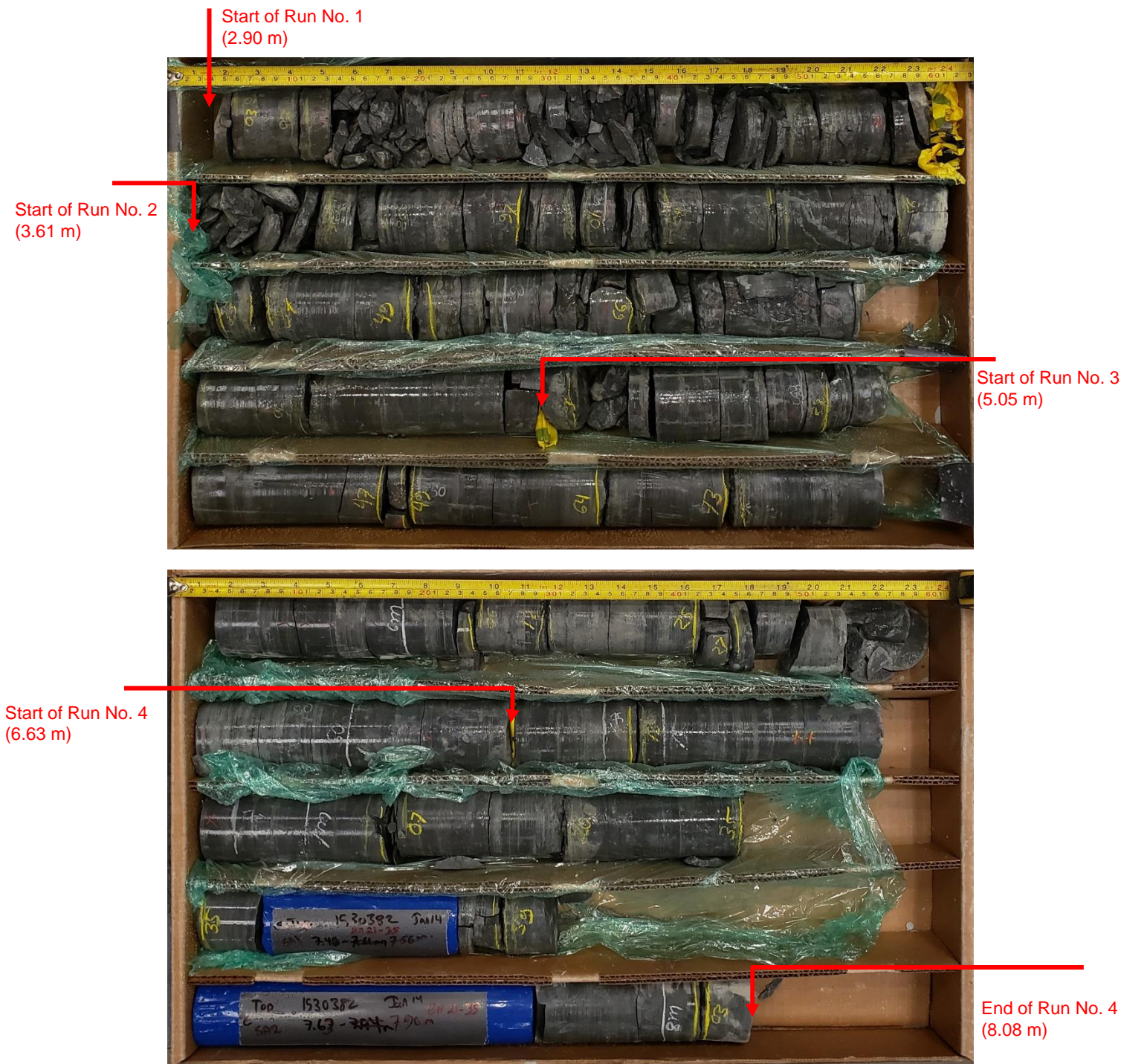
PLASTICITY CHART CLAYEY SILT (RESIDUAL SOIL)

Figure No. B-4

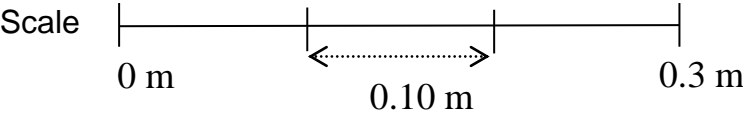
Project No. 1530382


Checked By: SEMP

REVISION DATE: 2021-03-22 BY: KNER Project: 1530382.7000



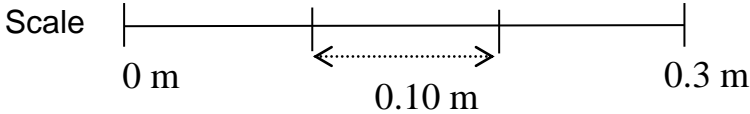
Borehole 21-35: Bedrock cored between depths of about 2.90 m to 8.08 m (continued)




PROJECT					
QEW Widening from East Cawthra Road to the East Mall Foundation Investigation Trenchless Watermain Crossing of the QEW West of Dixie Road					
TITLE					
BEDROCK CORE PHOTOGRAPHS BOREHOLE 21-35					
 GOLDER MEMBER OF WSP	PROJECT No. 1530382.7000			FILE No. ----	
	DESIGN	KN	21/03/22	SCALE	NTS
	CADD	--	--	FIGURE B-5A	
	CHECK	KCP	21/06/08		
	REVIEW	WC	21/06/09		
			VER. 1.		

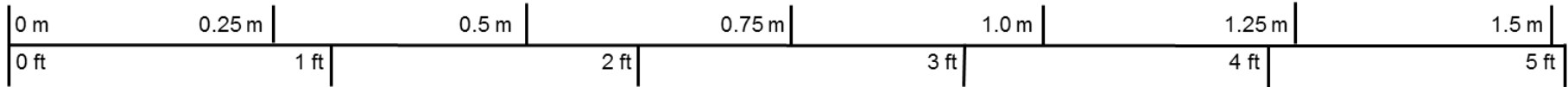
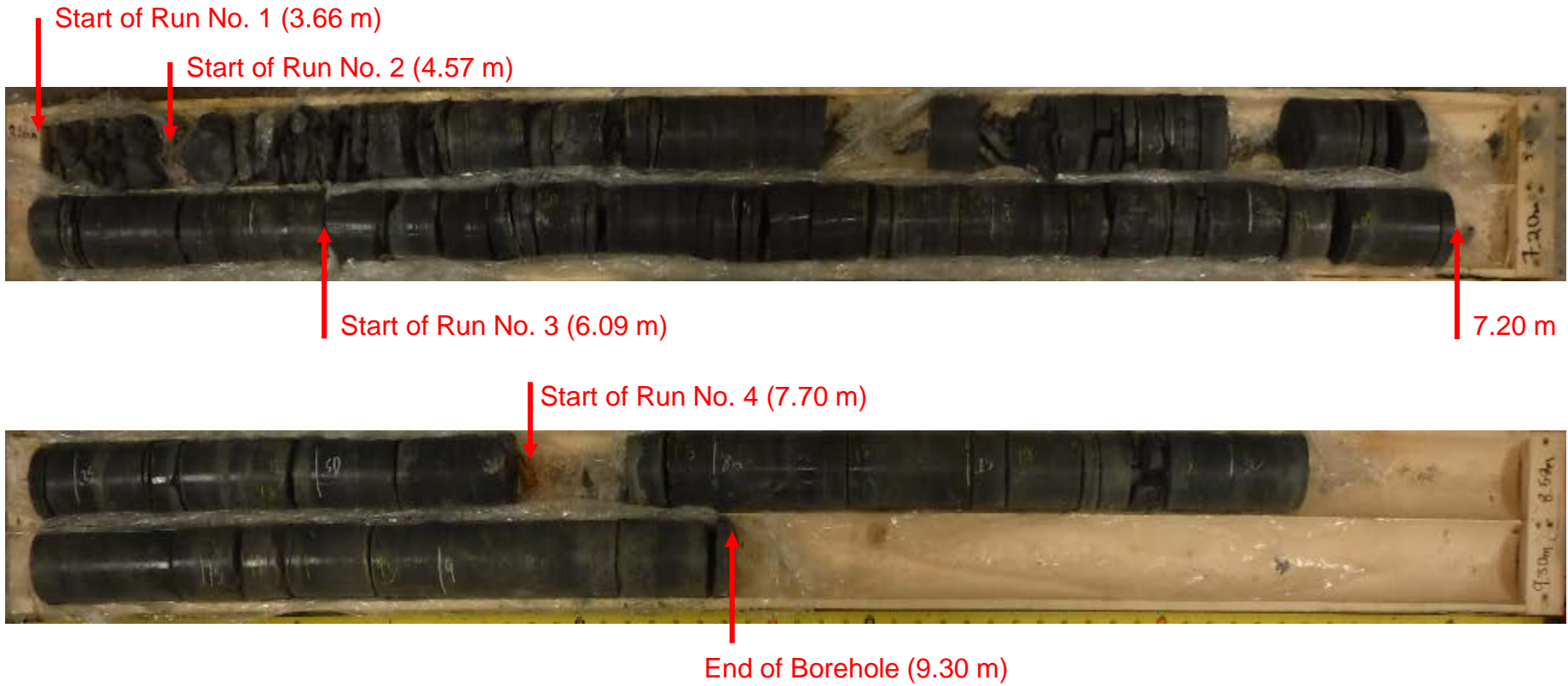



Borehole 21-35: Bedrock cored between depths of about 8.08 m to 10.86 m

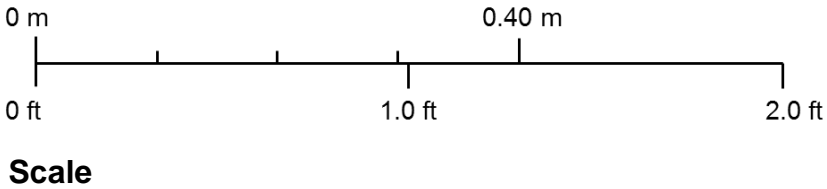
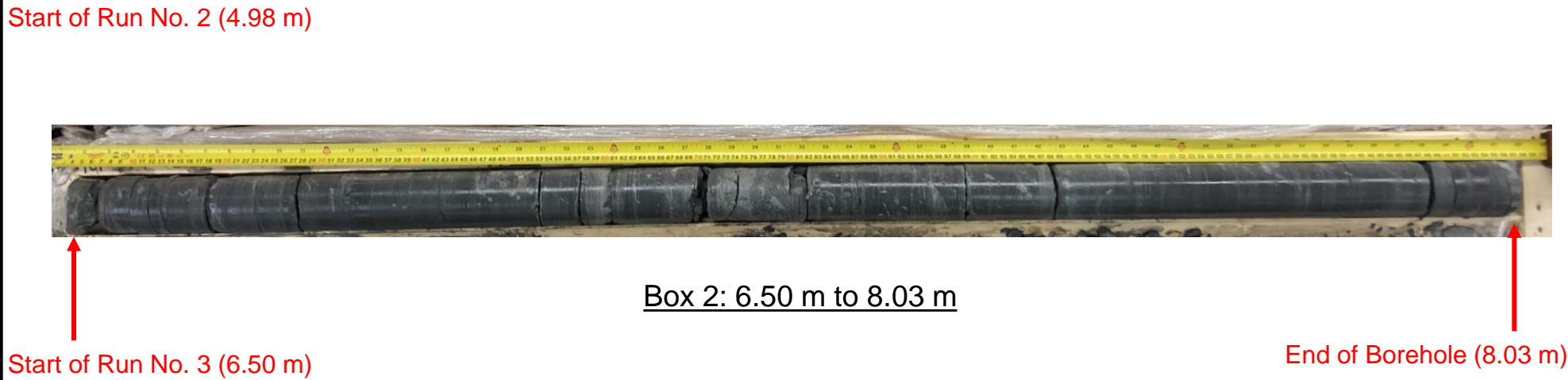
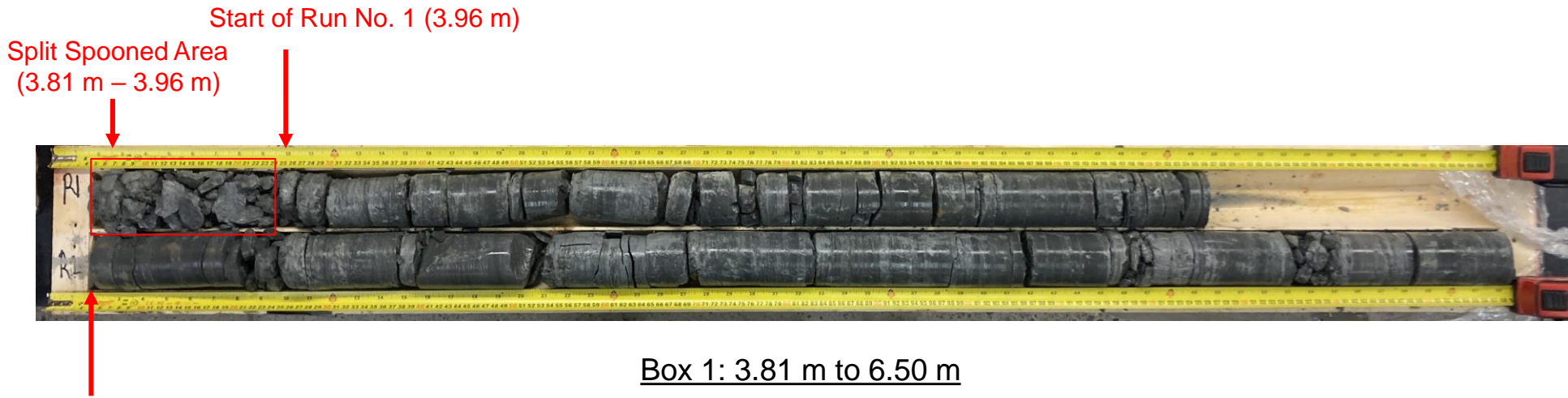


PROJECT QEW Widening from East Cawthra Road to the East Mall Foundation Investigation Trenchless Watermain Crossing of the QEW West of Dixie Road						
TITLE BEDROCK CORE PHOTOGRAPHS BOREHOLE 21-35						
 GOLDER <small>MEMBER OF WSP</small>	PROJECT No. 1530382.7000			FILE No. ----		
	DESIGN	KN	21/03/22	SCALE	NTS	VER. 1.
	CADD	--	--	FIGURE B-5B		
	CHECK	KCP	21/06/08			
	REVIEW	WC	21/06/09			

REVISION DATE: January 23, 2018 BY: DCB Project: 1530382



PROJECT					
QEW IMPROVEMENT FROM EAST OF CAWTHRA ROAD TO EAST MALL					
TITLE					
Bedrock Core Photographs Borehole MP-2					
	PROJECT No. 1530382			FILE No. ----	
	DRAFT	SK	20180406	SCALE	NTS
	CADD	--		FIGURE B-6	
	CHECK	SMM			
	REVIEW	JMAC			
			VER. 1.		



PROJECT					
QEW IMPROVEMENT FROM EAST OF CAWTHRA TO EAST MALL					
TITLE					
Bedrock Core Photographs Borehole BH-2E					
wsp	PROJECT No. 1530382			FILE No. ----	
	DRAFT	AM	20240202	SCALE	NTS
	CADD	--		FIGURE B-7	
	CHECK	DP	20240206		
	REVIEW	JS	20240207		

April 8, 2021

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

Re: UCS, BD, CERCHAR, and Slake testing (Golder Project No. 1530382-7000)

Dear Ms. Nero

On March 19th, 2021, a total of seventeen (17) HQ-sized rock core samples were received by Geomechanica Inc. via drop-off by Golder personnel. These samples were identified as being from project 1530382-7000. From these samples, six (6) UCS, three (3) BD, five (5) CERCHAR Abrasivity, and three (3) Slake Durability were prepared and tested.

Details regarding the steps of specimen preparation and testing are presented in the accompanying laboratory report and summary spreadsheets.

Sincerely,



Bryan Tatone Ph.D., P. Eng.

Geomechanica Inc.
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Rock Laboratory Testing Results

A report submitted to:

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April 8, 2021

Project number: 1530382-7000

Abstract

This document summarizes the results of rock laboratory testing, including 6 Uniaxial Compression Strength (UCS) tests, 3 Brazilian Disc (BD) tensile strength tests, 5 CERCHAR Abrasivity tests, and 3 Slake Durability tests. The results for each test type presented in separate sub-sections herein.

In this document:

1	Uniaxial Compressive Strength Tests	1
2	Brazilian Disc Tests	4
3	CERCHAR Abrasivity Tests	6
4	Slake Durability	10
	Appendices	12
A	UCS specimen sheets	12
B	BD specimen sheets	19

1 Uniaxial Compressive Strength Tests

1.1 Overview

This section summarizes the results of uniaxial compressive strength (UCS) testing of HQ3-sized specimens. The testing was performed in Geomechanica's rock testing laboratory using a 150 ton (1.3 MN) Forney loading frame equipped with pressure-compensated control valve to maintain an axial displacement rate of approximately 0.150 mm/min (Figure 1). The preparation and testing procedure for each specimen included the following:

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



Figure 1: Forney loading frame setup for UCS testing.

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

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

1.2 Results

The results of UCS testing 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 600 data points defining the stress-strain curve. Typically the modulus is defined at 50% of the UCS strength. However, due to prevalent non-linear stress-strain behaviour, custom stress ranges (where specimens deformed linearly) were selected for moduli determination. These stress ranges are provided in the summary spreadsheet that accompanies this report. Please note that additional specimen details and measurements are provided in the summary spreadsheet that accompanies this report.

Table 1: Summary of Uniaxial Compression test results.

Sample	Depth (m)	Bulk density ρ (g/cm ³)	UCS (MPa)	Young's modulus E (GPa)	Lithology	Failure description
21-34-7.82to7.95m	7.82 - 7.95	2.620	19.1	2.5	Siltstone, Limestone and Shale	1
21-35-7.4to7.56m	7.40 - 7.56	2.585	7.3	0.4	Shale and Siltstone	2, 3, 4
21-37-7.08to7.3m	7.08 - 7.30	2.627	20.1	2.0	Siltstone, Limestone, and Shale	5, 3
21-36-9.92to10.07m	9.92 - 10.07	2.541	8.5	0.6	Siltstone, Limestone, and Shale	1, 3, 6
21-3-6.47to6.7m	6.47 - 6.70	2.621	6.3	1.2	Siltstone and Shale	1
21-4-7.44to7.58m	7.44 - 7.58	2.632	23.0	3.4	Siltstone, Limestone, and Shale	5

¹ Axial splitting failure

² Inclined shear failure

³ Specimen emitted saline pore water upon loading

⁴ Failure partly along pre-existing structure

⁵ Partial hourglass failure

⁶ Failure localized in softer shale layer

1.3 Specimen photographs

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

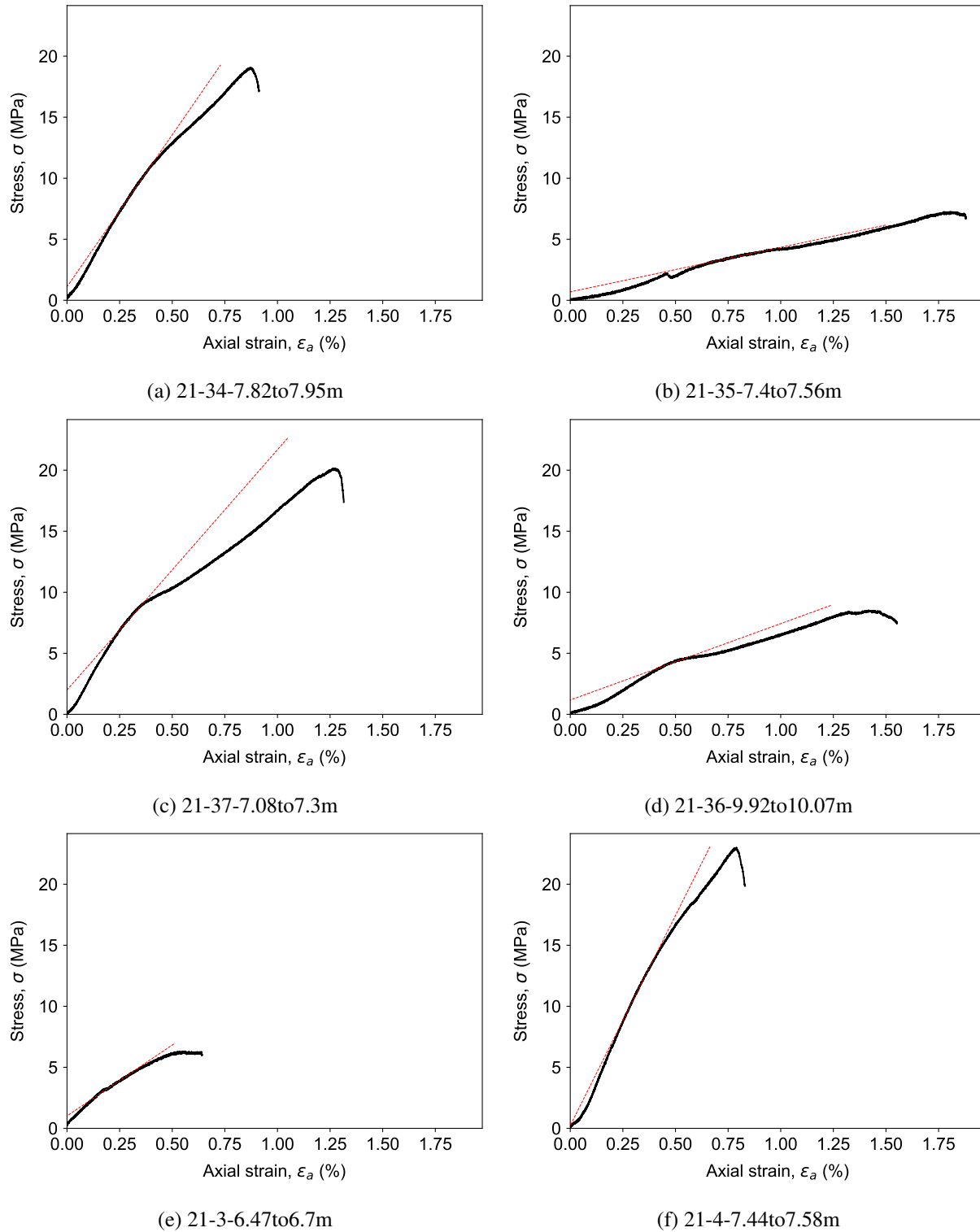


Figure 2: Measured stress-strain curves.

2 Brazilian Disc Tests

2.1 Overview

This section summarizes the results of Brazilian disc testing. The tests were performed using a 12 ton Carver hydraulic loading frame coupled to a SPX hydraulic pump fitted with a pressure-compensated flow control valve (Figure 3). A consistent displacement rate of approximately 0.175 mm/min was employed for all tests. 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 and possible damage during subsequent specimen preparation.
2. Diamond cutting of core samples to obtain disc specimens with nearly flat (within 0.5 mm) and parallel (within 0.5° end faces and a thickness approximately equal to the core radius. From each core sample as many discs as possible (given the available core sample length) were prepared and tested.
3. Diametric loading of disc specimens to rupture using a hydraulic loading frame equipped with fixed flat loading platens. The applied force and diametric displacement were continuously measured to calculate the indirect tensile strength. Note that a strip of tape and cardboard was placed on the specimens at the platen contact points to act as a cushion to distribute the applied load over the thickness of the sample.

The above Brazilian disc testing procedure adhered to ASTM D3967-16.

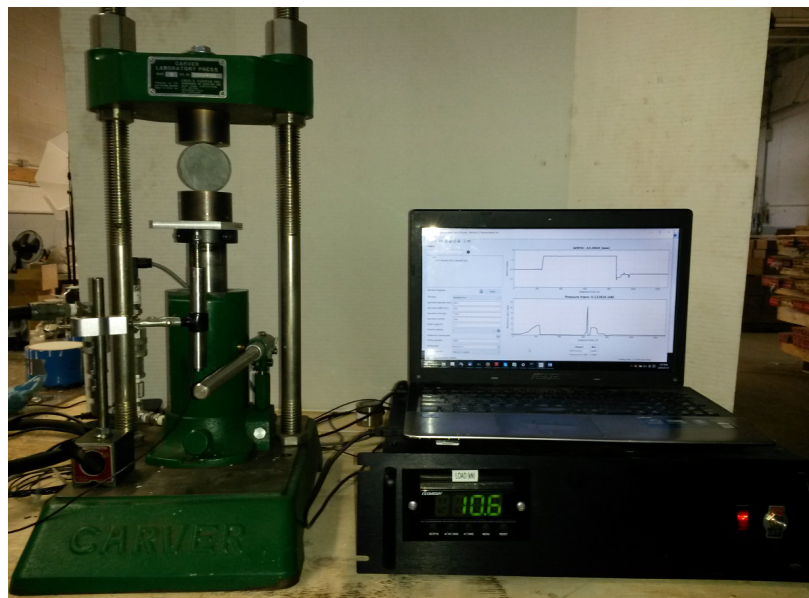


Figure 3: Brazilian disc testing setup.

2.2 Results

A summary of the Brazilian disc testing results are provided in Table 2. Additional details and measurements for the test specimens are included in the accompanying summary spreadsheet. The indirect tensile strength, σ_t , was calculated, as:

$$\sigma_t = \frac{2P}{\pi Dt} \quad (1)$$

where P is the peak diametric load; D is the specimen diameter; and t is the specimen thickness.

It must be noted that some Brazilian disc specimen did not fail via diametric splitting. Some failed via a combination of diametric splitting and fissility delamination, while others simply failed at the platen contact points or by fissility delamination alone. The failure mode of each disc is noted in the last column of the summary table.

Table 2: Summary of Brazilian Disc test results.

Borehole	Sample	Disc	Depth (m)	Bulk density ρ (g/cm ³)	Tensile strength (MPa)	Lithology	Failure description
21-34	21-34-7.29to7.4m	1	7.29 - 7.40	2.578	2.7	Siltstone and Limestone	1
		2		2.602	2.4	Siltstone and Limestone	1
				Average	2.5		
				Standard deviation	0.1		
21-37	21-37-8.42to8.56m	1	8.42 - 8.56	2.609	5.0	Limestone and Siltstone	2, 3
		2		2.609	1.7	Siltstone	4
		3		2.580	3.0	Siltstone and Limestone	1
		4		2.611	3.3	Limestone	4, 3
				Average	3.3		
				Standard deviation	1.2		
21-4	21-4-8.04to8.14m	1	8.04 - 8.14	2.608	2.1	Siltstone and Limestone	1
		2		2.638	3.0	Siltstone and Limestone	1
				Average	2.5		
				Standard deviation	0.4		

¹ Diametric failure

² Partial diametric failure

³ Failure along pre-existing structure

⁴ Non-diametric failure

2.3 Specimen photographs

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

3 CERCHAR Abrasivity Tests

3.1 Overview

This section summarizes the results of CERCHAR abrasivity testing. The tests were performed using a Type-2 CERCHAR apparatus as shown in Figure 4a. The tips of the styluses were sharpened to a conical angle of 90° using the setup shown in Figure 4b. The styluses used to perform the tests are shown in Figure 4c-d (Rockwell hardness 55 ± 1). A static force of 70 N was applied on top of the stylus by using a combination of weights. Details of the testing procedure for each sample, which followed ASTM D7625-10, proceeded as follows:

1. The tips of the five styluses were sharpened using the grinding apparatus (Figure 4b).
2. The styluses were placed under a microscope (60x magnification) and three scaled photos (120° apart) are captured before the test is conducted to ensure the 90° point has been properly formed.
3. The test specimens consisted of pieces HQ core with fresh fracture surface perpendicular to the core axis.
4. The specimen was secured in the cross-slide vise of the testing apparatus and the stylus carefully lowered on to the surface of the rock.
5. A scratch measuring 10 mm in length was created over a duration of 10 seconds. This process was repeated with all five styluses on undisturbed parts of the specimen surface (e.g., Figure 5a).
6. Lastly, the worn tips were re-examined under the microscope. From three scaled photos (120° apart), the wear flat, d , was measured (e.g., Figure 5c).

The length or the diameter of the wear flat, d , was measured from scaled microscope images using the image processing software Fiji (e.g., Figure 5b-c). The mean wear of the tip is calculated by taking the average d of all tests. The CERCHAR-Abrasivity-Index (CAI) of the sample is subsequently calculated by taking the mean wear and multiplying it by 10.

3.2 Results

The results of the CERCHAR abrasivity tests are summarized in Table 3. Further specimen and testing details are included in the summary spreadsheet that accompanies this report.

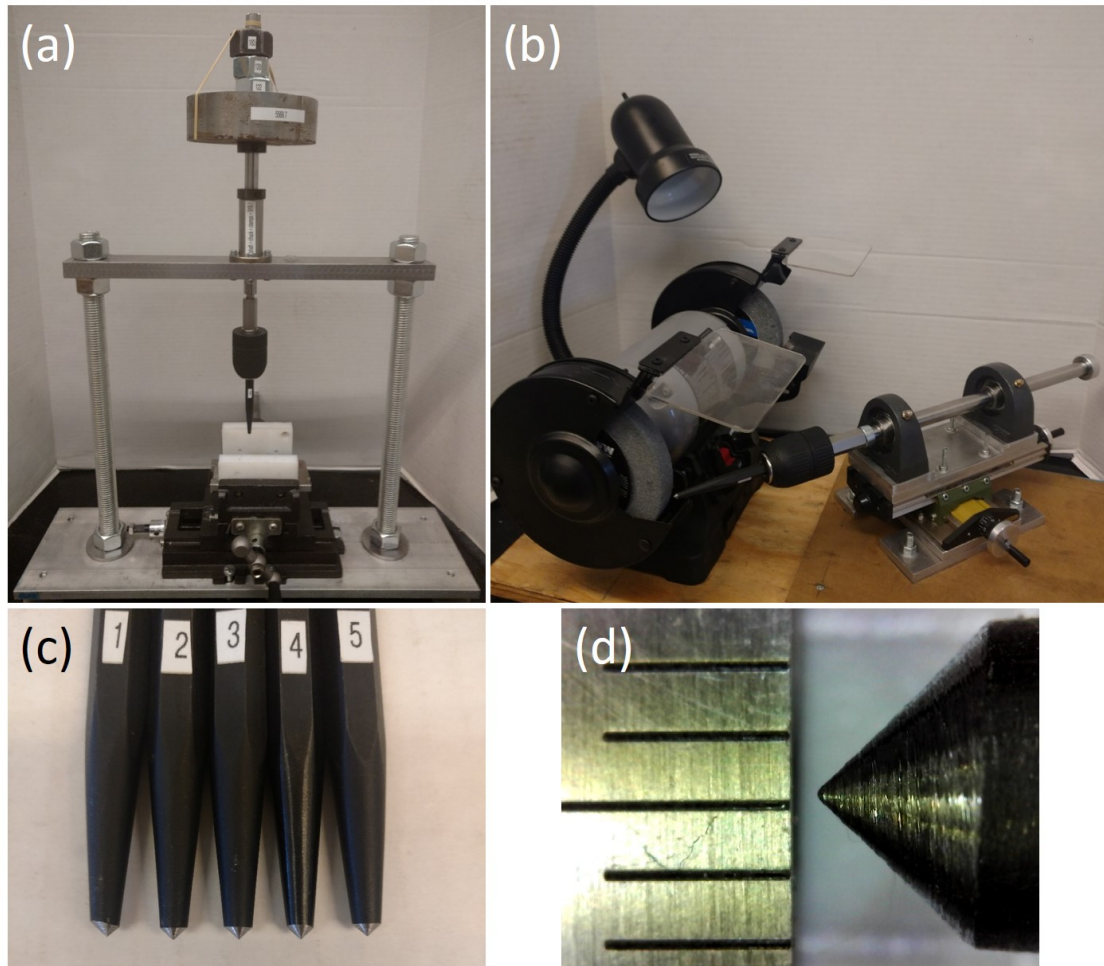


Figure 4: Photos showing (a) the CERCHAR apparatus, (b) tip sharpening setup, (c) the five styluses used to perform the test and (d) a microscope image of one of the stylus tips.

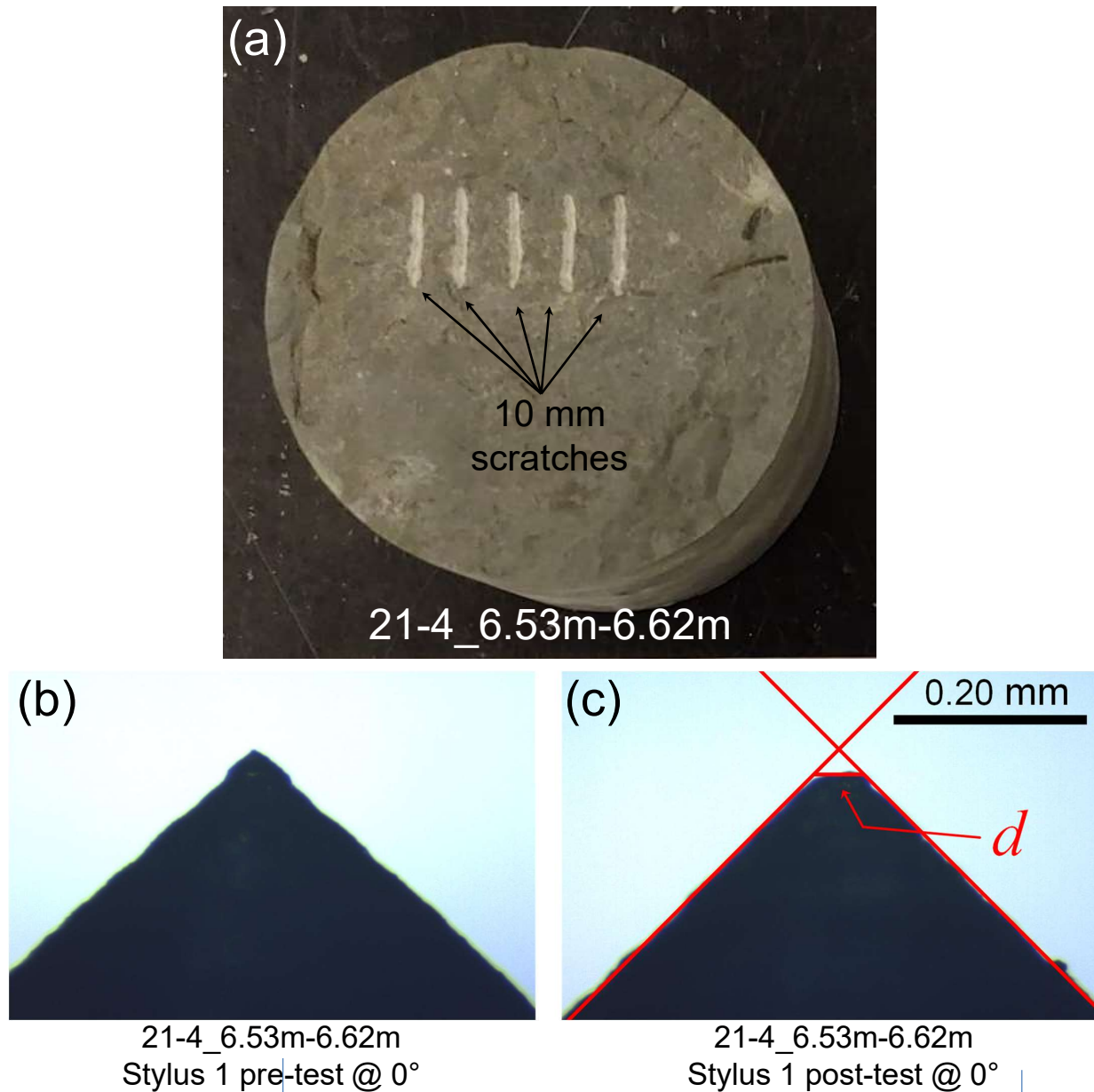


Figure 5: (a) Photograph showing an example of the five 10 mm scratches on a select test specimen; (b) microscope image of select stylus prior to testing at the noted position; and (c) microscope image of the same stylus at the same position following testing with the wear flat, d , denoted.

Table 3: Summary of CERCHAR abrasivity test results.

Sample	Depth (m)	Test 1 Mean (mm)	Test 2 Mean (mm)	Test 3 Mean (mm)	Test 4 Mean (mm)	Test 5 Mean (mm)	Mean Wear (mm)	CAI	Lithology	ASTM Classification
21-4-6.53m-6.62m	6.53 - 6.62	0.056	0.066	0.053	0.034	0.073	0.057	0.566	Siltstone	Low
21-34-6.38m-6.48m	6.38 - 6.48	0.036	0.028	0.032	0.024	0.030	0.030	0.299	Shale	< Very Low
21-35-7.63m-7.90m	7.63 - 7.90	0.028	0.026	0.026	0.034	0.028	0.028	0.282	Shale	< Very Low
21-36-8.97m-9.09m	8.97 - 9.09	0.050	0.047	0.033	0.043	0.041	0.043	0.430	Shale	Very Low
21-37-7.30m-7.45m	7.30 - 7.45	0.032	0.028	0.028	0.059	0.039	0.037	0.373	Shale	Very Low

4 Slake Durability

4.1 Overview

This section summarizes the results of slake durability testing. The tests were performed using an M&L Testing Equipment Slake Durability apparatus capable of simultaneously performing four slake durability tests (Figure 6). The test was conducted using the following procedure:

1. The core was broken using a hammer and point load testing apparatus into 40-60 g lumps. The sharp edges of the lumps were removed by lightly hammering and/or filing the edges.
2. Approximately 10 lumps weighing 450-550 g were inserted into the drum and dried in the oven at 110 °C until reaching a constant mass.
3. The drum was removed from the oven and allowed to cool to room temperature, weighed, and subsequently rotated in room temperature distilled water at 20 revolutions per minute for 10 minutes.
4. The drum was returned to the oven to dry for approximately one day and weighed again.
5. Steps 3 and 4 were then repeated for a second cycle.
6. The drum was thoroughly cleaned, dried, and weighed.

The above slake durability testing procedure adhered to ASTM D4644-16.

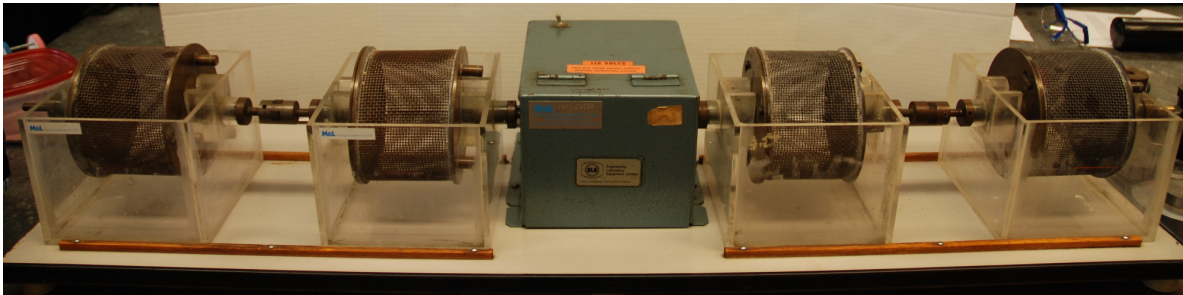


Figure 6: Test setup showing the slake durability apparatus.

4.2 Results

The results of the tests are summarized in Table 4. Additional measurements and sample descriptions are provided the summary spreadsheet that accompanies this report. The slake durability index after one and two cycles was calculated as follows, respectively:

$$I_{d1} = \frac{B - D}{A - D} \times 100\% \quad (2)$$

$$I_{d2} = \frac{C - D}{A - D} \times 100\% \quad (3)$$

where A is the mass of the specimen and drum before the first test cycle, B is the mass of the specimen and drum after oven drying the first cycle, C is the mass of the specimen and drum after oven drying the second cycle and D is the mass of the drum.

Table 4: Summary of slake durability testing results.

Sample	Depth (m)	Moisture content (%)	Pre-First Cycle, A (g)	Post-First Cycle, B (g)	Post-Second Cycle, C (g)	Mass of Drum, D (g)	Slake Durability Index, (1st Cycle) I_{d1} (%)	Slake Durability Index (2nd Cycle), I_{d2} (%)	Lithology
21-34-8.53to8.79	8.53 - 7.79	2.54	2439.01	2388.61	2303.35	1895.48	90.73	75.04	Shale & Limestone
21-35-6.39to6.62	6.39 - 6.62	3.46	2357.54	2295.54	2223.12	1857.24	87.61	73.13	Shale & Limestone
21-37-7.62to7.83	7.62 - 7.83	3.54	2282.26	2225.27	2149.97	1847.18	86.90	69.59	Shale

4.3 Specimen Photographs

Photographs of the specimens before testing and after testing are shown in Figure 7.

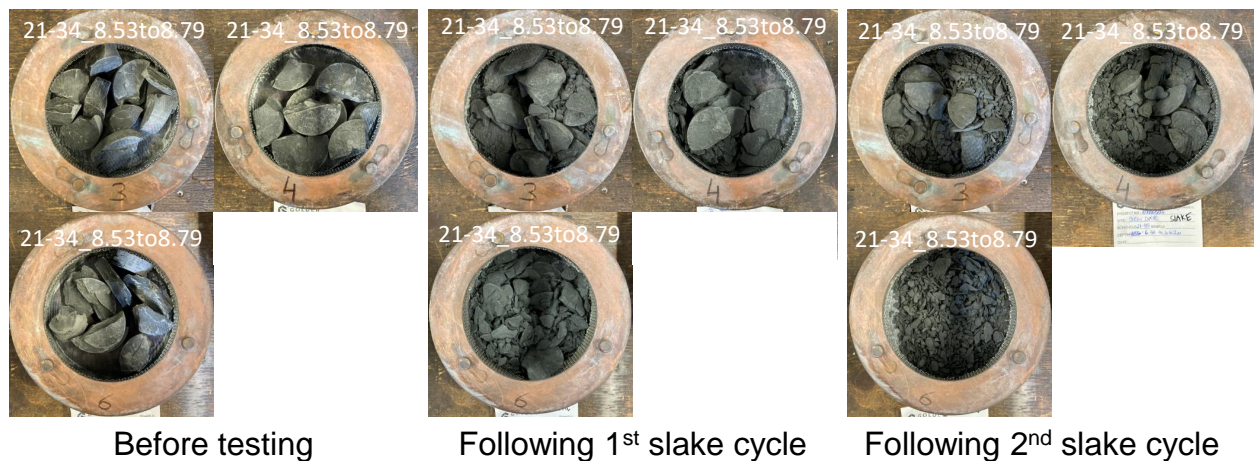




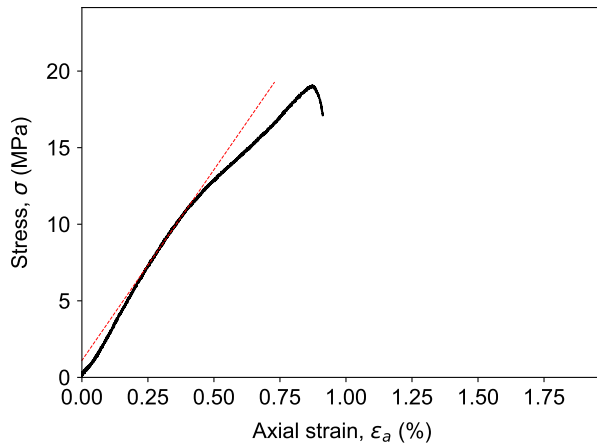
Figure 7: Photographs of slake durability specimens before and after testing.

Appendices



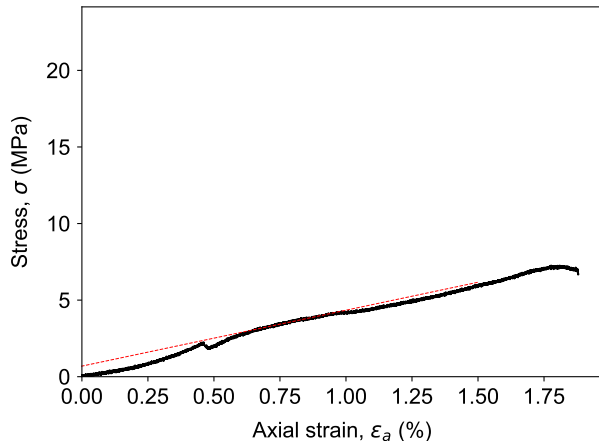
A UCS specimen sheets

- 21-34-7.82to7.95m
- 21-35-7.4to7.56m
- 21-37-7.08to7.3m
- 21-36-9.92to10.07m
- 21-3-6.47to6.7m
- 21-4-7.44to7.58m



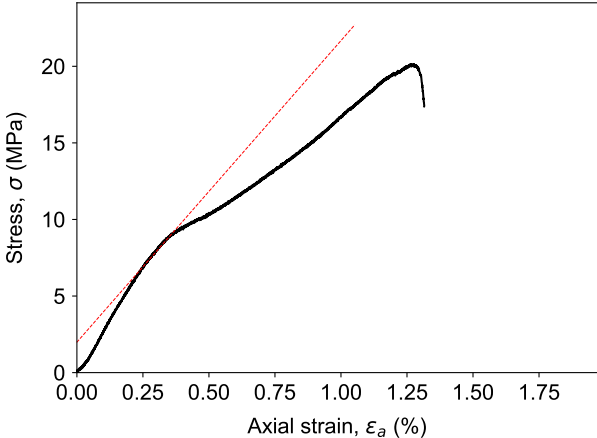
Uniaxial Compression Test

Client	Golder Associates Ltd.	Project	1530382-7000														
Sample	21-34-7.82to7.95m	Depth	7.82 - 7.95														
<div>Specimen parameters</div> <table><tr><td>Diameter (mm)^a</td><td>59.76</td></tr><tr><td>Length (mm)^a</td><td>121.00</td></tr><tr><td>Bulk density ρ (g/cm³)</td><td>2.620</td></tr><tr><td>UCS (MPa)</td><td>19.1</td></tr><tr><td>Young's modulus E (GPa)^b</td><td>2.5</td></tr><tr><td>Lithology</td><td>Siltstone, Limestone and Shale</td></tr><tr><td>Failure description^c</td><td>1</td></tr></table>		Diameter (mm) ^a	59.76	Length (mm) ^a	121.00	Bulk density ρ (g/cm ³)	2.620	UCS (MPa)	19.1	Young's modulus E (GPa) ^b	2.5	Lithology	Siltstone, Limestone and Shale	Failure description ^c	1	<div>Prior to testing</div> 	<div>After testing</div> 
Diameter (mm) ^a	59.76																
Length (mm) ^a	121.00																
Bulk density ρ (g/cm ³)	2.620																
UCS (MPa)	19.1																
Young's modulus E (GPa) ^b	2.5																
Lithology	Siltstone, Limestone and Shale																
Failure description ^c	1																
<div><div><div><div>^a Additional specimen measurement/details provided in accompanying summary spreadsheet.</div><div>^b Tangent modulus, calculated as the slope of the best fit line through ± 300 data points on either side of the point representing 50.0% of the peak strength.</div><div>^c Failure description: ¹ Axial splitting failure;</div></div></div><div></div></div>																	
Remarks:																	
Remarks: Displacement Rate: 0.15mm/min																	
Performed by	PL/SL	Date	2021-03-30														



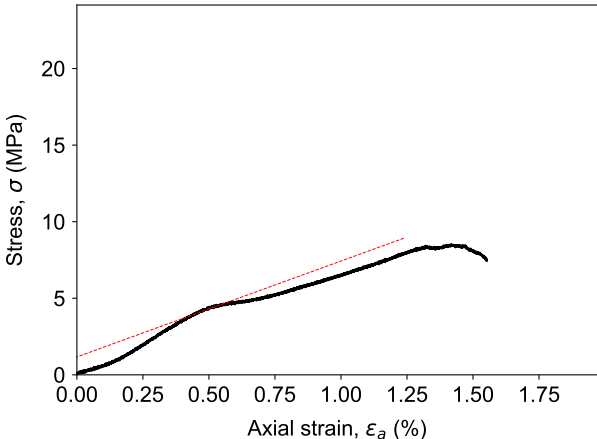
Uniaxial Compression Test

Client	Golder Associates Ltd.	Project	1530382-7000														
Sample	21-35-7.4to7.56m	Depth	7.40 - 7.56														
<div>Specimen parameters</div> <table><tr><td>Diameter (mm) ^a</td><td>59.97</td></tr><tr><td>Length (mm) ^a</td><td>126.74</td></tr><tr><td>Bulk density ρ (g/cm³)</td><td>2.585</td></tr><tr><td>UCS (MPa)</td><td>7.3</td></tr><tr><td>Young's modulus E (GPa) ^b</td><td>0.4</td></tr><tr><td>Lithology</td><td>Shale and Siltstone</td></tr><tr><td>Failure description ^c</td><td>2, 3, 4</td></tr></table>		Diameter (mm) ^a	59.97	Length (mm) ^a	126.74	Bulk density ρ (g/cm ³)	2.585	UCS (MPa)	7.3	Young's modulus E (GPa) ^b	0.4	Lithology	Shale and Siltstone	Failure description ^c	2, 3, 4	<div>Prior to testing</div> 	<div>After testing</div> 
Diameter (mm) ^a	59.97																
Length (mm) ^a	126.74																
Bulk density ρ (g/cm ³)	2.585																
UCS (MPa)	7.3																
Young's modulus E (GPa) ^b	0.4																
Lithology	Shale and Siltstone																
Failure description ^c	2, 3, 4																
<div><div><div>^a Additional specimen measurement/details provided in accompanying summary spreadsheet.</div><div>^b Tangent modulus, calculated as the slope of the best fit line through ± 300 data points on either side of the point representing 50.0% of the peak strength.</div><div>^c Failure description: ² Inclined shear failure; ³ Specimen emitted saline pore water upon loading; ⁴ Failure partly along pre-existing structure;</div></div><div></div></div>																	
Remarks:																	
Performed by	PL/SL	Date	2021-03-30														



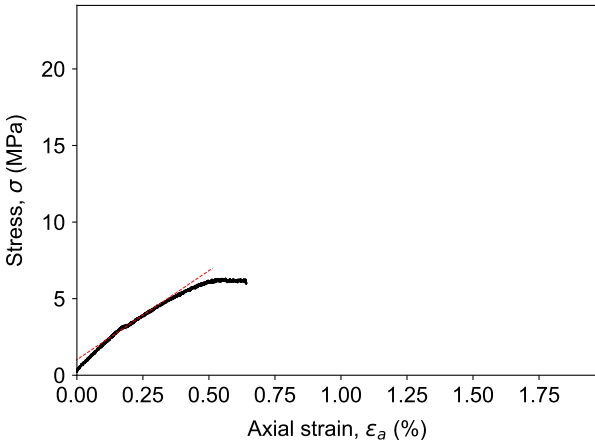
Uniaxial Compression Test

Client	Golder Associates Ltd.	Project	1530382-7000														
Sample	21-37-7.08to7.3m	Depth	7.08 - 7.30														
<div>Specimen parameters</div> <table><tr><td>Diameter (mm) ^a</td><td>59.90</td></tr><tr><td>Length (mm) ^a</td><td>126.69</td></tr><tr><td>Bulk density ρ (g/cm³)</td><td>2.627</td></tr><tr><td>UCS (MPa)</td><td>20.1</td></tr><tr><td>Young's modulus E (GPa) ^b</td><td>2.0</td></tr><tr><td>Lithology</td><td>Siltstone, Limestone, and Shale</td></tr><tr><td>Failure description ^c</td><td>5, 3</td></tr></table>		Diameter (mm) ^a	59.90	Length (mm) ^a	126.69	Bulk density ρ (g/cm ³)	2.627	UCS (MPa)	20.1	Young's modulus E (GPa) ^b	2.0	Lithology	Siltstone, Limestone, and Shale	Failure description ^c	5, 3	<div>Prior to testing</div> 	<div>After testing</div> 
Diameter (mm) ^a	59.90																
Length (mm) ^a	126.69																
Bulk density ρ (g/cm ³)	2.627																
UCS (MPa)	20.1																
Young's modulus E (GPa) ^b	2.0																
Lithology	Siltstone, Limestone, and Shale																
Failure description ^c	5, 3																
<div><div><div>^a Additional specimen measurement/details provided in accompanying summary spreadsheet.</div><div>^b Tangent modulus, calculated as the slope of the best fit line through ± 300 data points on either side of the point representing 40.0% of the peak strength.</div><div>^c Failure description: ⁵ Partial hourglass failure; ³ Specimen emitted saline pore water upon loading;</div></div><div></div></div>																	
Remarks:																	
Performed by	PL/SL	Date	2021-03-30														



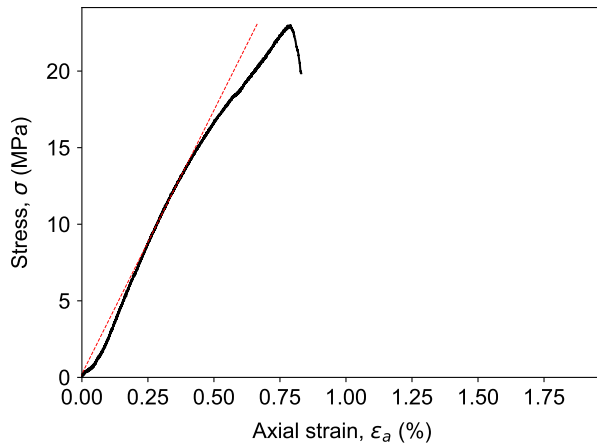
Uniaxial Compression Test

Client	Golder Associates Ltd.	Project	1530382-7000
Sample	21-36-9.92to10.07m	Depth	9.92 - 10.07
Specimen parameters		Prior to testing	After testing
Diameter (mm) ^a	59.99		
Length (mm) ^a	120.65		
Bulk density ρ (g/cm ³)	2.541		
UCS (MPa)	8.5		
Young's modulus E (GPa) ^b	0.6		
Lithology	Siltstone, Limestone, and Shale		
Failure description ^c	1, 3, 6		
^a Additional specimen measurement/details provided in accompanying summary spreadsheet.			
^b Tangent modulus, calculated as the slope of the best fit line through ±300 data points on either side of the point representing 50.0% of the peak strength.			
^c Failure description: ¹ Axial splitting failure; ³ Specimen emitted saline pore water upon loading; ⁶ Failure localized in softer shale layer;			
			
Remarks:			
Performed by	PL/SL	Date	2021-03-30

Uniaxial Compression Test

Client	Golder Associates Ltd.	Project	1530382-7000														
Sample	21-3-6.47to6.7m	Depth	6.47 - 6.70														
<div>Specimen parameters</div> <table><tr><td>Diameter (mm) ^a</td><td>59.81</td></tr><tr><td>Length (mm) ^a</td><td>126.69</td></tr><tr><td>Bulk density ρ (g/cm³)</td><td>2.621</td></tr><tr><td>UCS (MPa)</td><td>6.3</td></tr><tr><td>Young's modulus E (GPa) ^b</td><td>1.2</td></tr><tr><td>Lithology</td><td>Siltstone and Shale</td></tr><tr><td>Failure description ^c</td><td>1</td></tr></table>		Diameter (mm) ^a	59.81	Length (mm) ^a	126.69	Bulk density ρ (g/cm ³)	2.621	UCS (MPa)	6.3	Young's modulus E (GPa) ^b	1.2	Lithology	Siltstone and Shale	Failure description ^c	1	<div>Prior to testing</div> 	<div>After testing</div> 
Diameter (mm) ^a	59.81																
Length (mm) ^a	126.69																
Bulk density ρ (g/cm ³)	2.621																
UCS (MPa)	6.3																
Young's modulus E (GPa) ^b	1.2																
Lithology	Siltstone and Shale																
Failure description ^c	1																
<div><div><div>^a Additional specimen measurement/details provided in accompanying summary spreadsheet.</div><div>^b Tangent modulus, calculated as the slope of the best fit line through ± 300 data points on either side of the point representing 50.0% of the peak strength.</div><div>^c Failure description: ¹ Axial splitting failure;</div></div><div></div></div>																	
Remarks:																	
Performed by	PL/SL	Date	2021-03-30														

Uniaxial Compression Test

Client	Golder Associates Ltd.	Project	1530382-7000														
Sample	21-4-7.44to7.58m	Depth	7.44 - 7.58														
<div>Specimen parameters</div> <table><tr><td>Diameter (mm)^a</td><td>60.05</td></tr><tr><td>Length (mm)^a</td><td>126.77</td></tr><tr><td>Bulk density ρ (g/cm³)</td><td>2.632</td></tr><tr><td>UCS (MPa)</td><td>23.0</td></tr><tr><td>Young's modulus E (GPa)^b</td><td>3.4</td></tr><tr><td>Lithology</td><td>Siltstone, Limestone, and Shale</td></tr><tr><td>Failure description^c</td><td>5</td></tr></table>		Diameter (mm) ^a	60.05	Length (mm) ^a	126.77	Bulk density ρ (g/cm ³)	2.632	UCS (MPa)	23.0	Young's modulus E (GPa) ^b	3.4	Lithology	Siltstone, Limestone, and Shale	Failure description ^c	5	<div>Prior to testing</div> 	<div>After testing</div> 
Diameter (mm) ^a	60.05																
Length (mm) ^a	126.77																
Bulk density ρ (g/cm ³)	2.632																
UCS (MPa)	23.0																
Young's modulus E (GPa) ^b	3.4																
Lithology	Siltstone, Limestone, and Shale																
Failure description ^c	5																
<div><div><div><div>^a Additional specimen measurement/details provided in accompanying summary spreadsheet.</div><div>^b Tangent modulus, calculated as the slope of the best fit line through ± 300 data points on either side of the point representing 50.0% of the peak strength.</div><div>^c Failure description: ⁵ Partial hourglass failure;</div></div></div><div></div></div>																	
Remarks:																	
Performed by	PL/SL	Date	2021-03-30														

B BD specimen sheets

- 21-34-21-34-7.29to7.4m
- 21-37-21-37-8.42to8.56m
- 21-4-21-4-8.04to8.14m

Brazilian Disc Test

Client	Golder Associates Ltd.	Project	1530382-7000																		
Sample	21-34-21-34-7.29to7.4m	Depth	7.29 - 7.40																		
<div>Specimen parameters</div> <table><tr><td>Disc</td><td>1</td><td>2</td></tr><tr><td>Thickness (mm) ^a</td><td>33.41</td><td>33.60</td></tr><tr><td>Diameter (mm) ^a</td><td>59.98</td><td>59.98</td></tr><tr><td>Tensile strength (MPa)</td><td>2.7</td><td>2.4</td></tr><tr><td>Lithology</td><td colspan="2">Siltstone and Limestone</td></tr><tr><td>Failure description ^b</td><td>1</td><td>1</td></tr></table>				Disc	1	2	Thickness (mm) ^a	33.41	33.60	Diameter (mm) ^a	59.98	59.98	Tensile strength (MPa)	2.7	2.4	Lithology	Siltstone and Limestone		Failure description ^b	1	1
Disc	1	2																			
Thickness (mm) ^a	33.41	33.60																			
Diameter (mm) ^a	59.98	59.98																			
Tensile strength (MPa)	2.7	2.4																			
Lithology	Siltstone and Limestone																				
Failure description ^b	1	1																			
<div>^a Additional specimen measurement/details provided in accompanying summary spreadsheet.</div> <div>^b Failure description: ¹ Diametric failure;</div>																					
<div>Average tensile strength (MPa) 2.6</div>																					
<div><div>Prior to testing</div><div>After testing</div></div>																					
<div>Performed by PL/SL</div> <div>Date 2021-03-20</div>																					

Brazilian Disc Test

Client	Golder Associates Ltd.		Project	1530382-7000
Sample	21-37-21-37-8.42to8.56m		Depth	8.42 - 8.56
Specimen parameters				
Disc	1	2	3	4
Thickness (mm) ^a	33.32	33.18	33.72	33.42
Diameter (mm) ^a	59.88	59.83	59.78	59.92
Tensile strength (MPa)	5.0	1.7	3.0	3.3
Lithology	Limestone and Siltstone	Siltstone	Siltstone and Limestone	Limestone
Failure description ^b	2, 3	4	1	4, 3

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

^b Failure description: ² Partial diametric failure; ³ Failure along pre-existing structure; ⁴ Non-diametric failure; ¹ Diametric failure;

Average tensile strength (MPa)

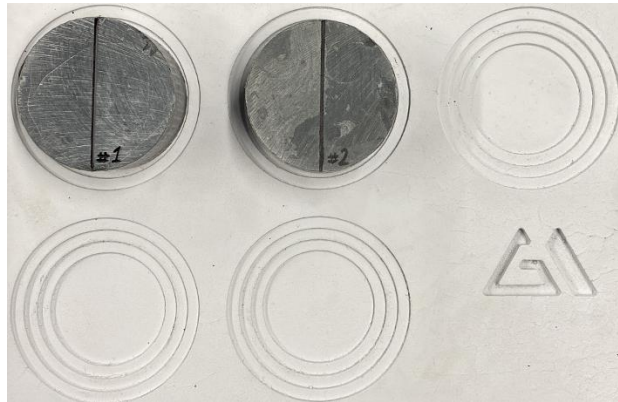
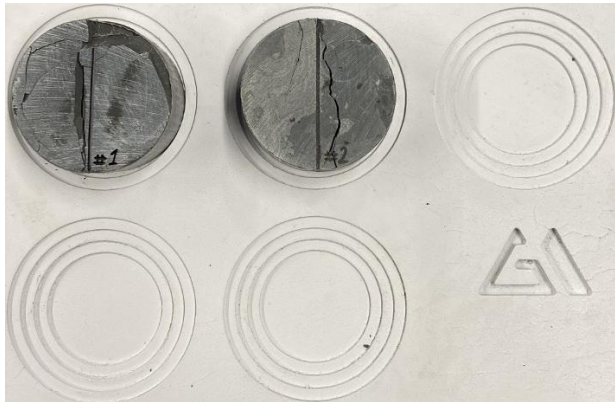
3.2

Prior to testing

After testing

Performed by	PL/SL	Date	2021-03-20
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Brazilian Disc Test

Client	Golder Associates Ltd.	Project	1530382-7000																		
Sample	21-4-21-4-8.04to8.14m	Depth	8.04 - 8.14																		
<div>Specimen parameters</div> <table><tr><td>Disc</td><td>1</td><td>2</td></tr><tr><td>Thickness (mm) ^a</td><td>33.29</td><td>32.80</td></tr><tr><td>Diameter (mm) ^a</td><td>60.21</td><td>60.09</td></tr><tr><td>Tensile strength (MPa)</td><td>2.1</td><td>3.0</td></tr><tr><td>Lithology</td><td colspan="2">Siltstone and Limestone</td></tr><tr><td>Failure description ^b</td><td>1</td><td>1</td></tr></table>				Disc	1	2	Thickness (mm) ^a	33.29	32.80	Diameter (mm) ^a	60.21	60.09	Tensile strength (MPa)	2.1	3.0	Lithology	Siltstone and Limestone		Failure description ^b	1	1
Disc	1	2																			
Thickness (mm) ^a	33.29	32.80																			
Diameter (mm) ^a	60.21	60.09																			
Tensile strength (MPa)	2.1	3.0																			
Lithology	Siltstone and Limestone																				
Failure description ^b	1	1																			
<div><div>^a Additional specimen measurement/details provided in accompanying summary spreadsheet.</div><div>^b Failure description: ¹ Diametric failure;</div></div>																					
<div>Average tensile strength (MPa) 2.6</div>																					
<div><div><div>Prior to testing</div><div></div></div><div><div>After testing</div><div></div></div></div>																					
Performed by	PL/SL	Date	2021-03-20																		

POINT LOAD TEST ON ROCK SAMPLES

ASTM D5731

PROJECT NO. 1530382 (7000)
 TITLE AECOM/2015-E-0001/QEW Cawthra
 DATE March 23, 2021

Borehole Number	Sample Number	Sample Depth (m)	Test Type	Core Length (mm)	Core ⁽²⁾ Diameter (mm)	Equivalent Diameter (mm)	Ram Pressure (kPa)	Load (P) (kN)	Is Axial (MPa)	Is Diametral (MPa)	Is (50mm) (MPa)	Approx. ⁽¹⁾ UCS (MPa)
21-34	-	6.78-6.85	A	21.35	59.04	40.06	4,710.00	4.47	2.782	-	2.518	53
21-34	-	6.22-6.28	A	29.83	60.32	47.86	4,820.00	4.57	1.995	-	1.956	45
21-35	-	6.12-6.17	A	22.64	59.05	41.26	4,850.00	4.60	2.701	-	2.477	52
21-35	-	5.58-5.64	A	22.37	60.17	41.40	10,020.00	9.50	5.543	-	5.091	107
21-36	-	6.41-6.44	A	24.49	60.16	43.31	1,460.00	1.38	0.738	-	0.692	15
21-36	-	7.24-7.27	A	23.53	60.09	42.43	10,850.00	10.29	5.714	-	5.307	111
21-37	-	6.86-6.92	A	22.81	60.17	41.80	3,790.00	3.59	2.056	-	1.897	40
21-37	-	7.00-7.05	A	29.22	60.01	47.25	4,950.00	4.69	2.102	-	2.049	47

⁽¹⁾ $Is_{50} \times C$ from ISRM ("Suggested Methods for Determining Point Load Strength", International Society for Rock Mechanics Commission on Testing Methods, Int. J. Rock. Mech. Min. Sci. and Geomechanical Abstr., Vol 22, No. 2 1985, pp. 51-60.

⁽²⁾ Actual distance between point load cones at time of failure.

Checked By: 

Golder Associates

