June 01, 2018

FOUNDATION INVESTIGATION AND DESIGN REPORT

RETAINING WALLS NO. 24-887/W AND 24-888/W REPLACEMENT
QEW WIDENING FROM EAST OF CAWTHRA ROAD TO THE EAST MALL
CITIES OF MISSISSAUGA AND ETOBICOKE
MINISTRY OF TRANSPORTATION, ONTARIO
GWP 2102-13-00 & 2432-13-00

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GEOCRES NO.: 30M11-275
(LAT. 43.602532, LONG. -79.562969)

REPORT

Report Number: 1530382-6

Distribution:

1 PDF and 1 Copy - Ministry of Transportation, Ontario (Central Region)
1 PDF and 1 Copy - Ministry of Transportation, Ontario (Foundations Section)
1 PDF - AECOM
1 PDF - Golder Associates Ltd.

Golder Associates

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PART A

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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by AECOM on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services for the widening of Queen Elizabeth Way (QEW) from Cawthra Road to the East Mall in the Cities of Mississauga and Etobicoke, Regional Municipality of Peel/City of Toronto, Ontario.

This report addresses the results of the foundation investigation carried out for the replacement of the following retaining walls:

- Retaining Wall No. 24-887/W, also known as the Brentano retaining wall, located on the north side of the QEW between Laughton Avenue and Etobicoke Creek, extending between about Stations 13+500 and 13+810; and,
- Retaining Wall No. 24-888/W, comprising the eastern section of an existing retaining wall, excluding between about Stations 13+581 and 13+847.5, located on the south side of the QEW between Boxwood Way and Etobicoke Creek, of which the replacement section will extend between about Stations 13+749 and 13+859.

The purpose of this investigation is to establish the subsurface soil and bedrock conditions at the proposed retaining wall locations by borehole drilling, rock coring and laboratory testing on selected soil and rock core samples.

The Terms of Reference (TOR) and the scope of work for the foundation investigation are outlined in MTO's Request for Proposal, dated January 2016, which forms part of the Consultant's Assignment Number (Number 2015-E-0001) for this project. The work has been carried out in accordance with Golder's Supplementary Specialty Plan for foundation engineering services for this project, dated June 6, 2016.

2.0 SITE DESCRIPTION

2.1 Retaining Wall 24-887/W

Retaining Wall No. 24-887/W is located along the north side of QEW between Laughton Avenue and Etobicoke Creek in the City of Mississauga, Ontario. A residential area is located north of the retaining wall. The existing retaining wall generally consists of a concrete cantilever structure founded on shallow foundations, with a noise barrier wall attached on top. An approximately 45.7 m section near the west end of the existing retaining wall, from approximately Stations 13+531.5 to 13+577, consists of a concrete structure founded on cast-in-place piles. The QEW has been constructed in cut in this area, with the existing QEW grade at the site between approximately Elevations 105 m and 106 m, rising from east to west.

2.2 Retaining Wall 24-888/W

Retaining Wall No. 24-888/W is located along the south side of the QEW between Boxwood Way and Etobicoke Creek in the City of Mississauga, Ontario. A residential area is located south of the retaining wall. The QEW has been constructed in a cut in this area, with its grade at approximately Elevation 101 m at the east end of the wall, rising to about Elevation 104.5 m near the west end of the proposed replacement section. The existing concrete cantilever retaining wall extends from about Stations 13+581 to 13+847.5 and is between about 1.6 m and 4.9 m high, founded on shallow foundations. The founding levels of the western portion of the existing retaining wall from about Station 13+581 to 13+749 range from about Elevation 99.974 m to Elevation 102.260 m, respectively, as noted from the contract drawings provided by AECOM. The founding levels of the eastern portion of the existing retaining wall from about Station 13+749 to the east end at about Station 13+847.5 vary between about Elevation 102.3 m at the westernmost extent and about Elevation 100.0 m at the east end. The proposed replacement section is about 110 m long and will extend beyond the east end of the existing wall, to about Station 13+849.

3.0 INVESTIGATION PROCEDURES

The field work for this subsurface investigation was carried out between September 9 and 19, 2016 and between October 30 and December 12, 2017 during which time a total of eleven sampled boreholes, designated as Boreholes RW2-1 to RW2-7, RW3-1 to RW3-3, and STM-10, were advanced immediately adjacent to along the proposed retaining wall replacement alignments. The Record of Borehole and Drillhole sheets and the results of the laboratory testing for the boreholes are presented in Appendices A and B for Retaining Wall Nos. 24-887/W and 24-888/W, respectively.

The details of each replacement retaining wall and the locations of the boreholes advanced at each site are provided below and the borehole locations are shown on Drawings 1 and 2.

Retaining Wall Designation	Approximate Station	Boreholes Advanced	Appendix
Retaining Wall No. 24-887/W	13+500 to 13+810	8 Boreholes (RW2-1 to RW2-7 and STM-10)	A
Retaining Wall No. 24-888/W	13+749 to 13+859 (section from 13+749 to 13+859 will be replaced / extended)	3 Boreholes (RW3-1 to 3-3)	B

The field borehole investigation was carried out using a truck-mounted CME 75 drill rig, supplied and operated by Davis Drilling of Milton, Ontario. The boreholes were advanced through the overburden using 108 mm, 200 mm and 260 mm outside diameter (O.D.) solid stem augers, 160 mm inner diameter (I.D.) hollow stem augers and NW casing. Soil samples were obtained at 0.75 m and 1.5 m intervals of depth, using a 50 mm outer diameter (O.D.) split-spoon sampler driven by an automatic hammer in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586-08)[1]. Samples of the bedrock were obtained using an ‘NQ’ size rock core barrel and coring techniques at all except three of the boreholes.

The boreholes were advanced to depths between 4.1 m and 20.2 m below existing ground surface, including coring of bedrock for a core lengths of between 3.8 m and 11.1 m. Photographs of the recovered rock samples are provided in Appendices A and B.

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

The groundwater conditions and water levels in the open boreholes were observed in all of the boreholes during the drilling operations. Standpipe piezometers were installed in selected boreholes to permit monitoring of the water level pertinent to the retaining wall sites. The installed piezometers consist of a 50 mm diameter PVC pipe, with a 1.5 m slotted screen sealed within a filter sand pack at a select depth within the borehole. The borehole and annulus surrounding the piezometer pipe above the filter sand pack were backfilled to the ground surface with bentonite pellets. Piezometer installation details and water level readings are described on the Record of Borehole sheets included in Appendices A and B. All boreholes in which standpipe piezometers were not installed were backfilled to ground surface with bentonite upon completion, in accordance with Ontario Regulation 903, Wells (as amended) and a 0.1 m to 0.2 m thick asphalt cap was placed in the boreholes drilled on roadways/shoulders.

The field work was observed by members of Golder's engineering and technical staff, who located the boreholes, arranged for the clearance of underground services, observed the drilling, sampling and in situ testing operations, logged the boreholes, and examined and cared for the soil and rock samples. The samples were identified in the field, placed in appropriate containers, labelled and transported to our Mississauga geotechnical laboratory where the samples underwent further visual examination and laboratory testing. All of the laboratory tests were carried out to MTO and/or ASTM Standards, as appropriate. Classification testing (water content, Atterberg limits and grain size distribution) was carried out on selected soil samples. Unconfined compression (uniaxial) strength (UCS), Young's modulus, bulk density, slake durability and CERCHER abrasivity testing was carried out on selected specimens of the bedrock core. The results of the geotechnical (soil and bedrock samples) laboratory testing are included in Appendix A.

One selected bedrock core sample and five soil samples was submitted to Maxxam Analytics (Maxxam) of Mississauga, Ontario which is a Standards Council of Canada (SCC) accredited laboratory for chemical analysis. The sample of bedrock core was crushed and homogenized by Maxxam prior to testing and the homogenized samples were analyzed for corrosivity testing (parameters include conductivity, resistivity, soluble chloride, soluble sulphate and pH). The chemical analyses results are presented in Appendix C.

The borehole locations and the ground surface elevations at the as-drilled locations were 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. The locations given in the Record of Borehole/Drillhole sheets and shown on Drawings 1 and 2 are positioned relative to MTM NAD 83 (Zone 10) northing and easting coordinates and the ground surface elevations are referenced to Geodetic datum. The borehole locations, ground surface elevations and drilled depths are summarized below.

Borehole No.	**Location (MTM NAD 83)**		**Ground Surface Elevation (m)**	**Borehole Depth (m)**
	Northing (Latitude)	**Easting (Longitude)**		
RW2-1	4,829,078.9 (43.601628)	299,630.9 (-79.564027)	107.9	18.8*
RW2-2	4,829,107.3 (43.601883)	299,651.6 (-79.563771)	108.1	18.7*
RW2-3	4,829,144.6 (43.602219)	299,684.5 (-79.563364)	107.5	17.7*
RW2-4	4,829,204.7 (43.602532)	299,747.0 (-79.562969)	108.1	18.6*

Borehole No.	Location (MTM NAD 83)		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing (Latitude)	Easting (Longitude)		
RW2-5	4,829,306.2 (43.602761)	299,845.3 (-79.562591)	102.1	7.7
RW2-6	4,829,243.2 (43.602871)	299,769.4 (-79.561693)	108.0	12.7*
RW2-7	4,829,276.6 (43.603107)	299,794.8 (-79.562314)	107.8	20.2*
STM-10	4,829,179.3 (43.603675)	299,716.4 (-79.561374)	107.6	17.4*
RW3-1	4,829,216.9 (43.603220)	299,819.4 (-79.561302)	104.3	6.4
RW3-2	4,829,255.6 (43.603408)	299,851.0 (-79.561999)	103.0	4.1
RW3-3	4,829,295.7 (43.603581)	299,881.0 (-79.560932)	102.0	7.6*

* Includes bedrock core lengths between about 3.8 m and 11.1 m

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

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

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

The Georgian Bay Formation which 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 General Overview of Subsurface Conditions

The detailed subsurface soil, bedrock and groundwater conditions as encountered in the boreholes advanced during the current investigation and the results of the laboratory tests carried out on selected soil and bedrock core samples are presented on the Record of Borehole and Drillhole sheets provided in Appendices A and B, for the respective retaining wall sections. The results of the in situ field tests (i.e., SPT "N" values) as presented on the Record of Borehole sheets and in Section 4.2 are uncorrected. The results of the geotechnical laboratory testing

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

on soil and bedrock core samples are also presented in Appendices A and B, for the respective retaining wall sections.

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

In general, the stratigraphy encountered at the various borehole locations typically consists of surficial layers of asphalt, topsoil and non-cohesive fill underlain by a cohesive till deposit and/or a cohesive residual soil deposit, in turn underlain by shale bedrock.

Detailed descriptions of the subsurface conditions at each investigated retaining wall are provided in the following sections of this report. Where relatively significant thicknesses of overburden were encountered, the various soil types are described in detail for each main deposit.

4.2.1 Retaining Wall No. 24-887/W

The plan and profile along the proposed retaining wall showing the borehole locations and interpreted stratigraphy between about Station 13+500 and 13+810 are shown on Drawings 1 and 2. The Record of Borehole and Drillhole sheets (Boreholes RW2-1 to RW2-7 and STM-10) and the laboratory test results for this area are presented in Appendix A. Not all of the laboratory testing has been completed to date; the report will be updated once the laboratory testing is completed.

In general, the subsurface conditions in the area of the proposed retaining wall consist of a layer of topsoil, except at the eastern end of the alignment where the boreholes were drilled from the highway surface asphalt pavement, underlain by a layer of sand to gravelly sand to sand and gravel fill or sand and gravelly sand in presently treed areas, further underlain by a sandy clayey silt till deposit. The sandy clayey silt till deposit is underlain by a clayey silt residual soil deposit in some of the boreholes. The cohesive till and residual soil deposits are underlain by shale bedrock, which was encountered between depths of 7.1 m to 8.5 m below ground surface, approximately between Elevations 100.9 m to 94.4 m.

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

4.2.1.1 Topsoil

An approximately 50 mm to 200 mm thick layer of topsoil was encountered immediately below ground surface in Boreholes RW2-1 to RW2-4, RW2-6 and STM-10, which where advanced within the park located north of the existing retaining wall.

4.2.1.2 Asphalt/Concrete

An approximately 100 mm and 150 mm thick layer of asphalt pavement was encountered immediately below ground surface in Boreholes RW2-5 and RW2-7, which were advanced along the shoulder of QEW road surface.

A 200 mm thick layer of concrete was encountered underlying the asphalt pavement in Borehole RW2-5.

4.2.1.3 Fill

Fill was encountered underlying the topsoil and asphalt pavement in Boreholes RW2-2, RW2-4, RW2-5, RW2-7 and STM-10, located along most of the extent of the retaining wall. The fill layer ranges in total thickness from 0.6 m to 1.1 m, and the surface of the fill extends from about Elevation 108.0 m and 107.4 m, with the exception of Borehole RW2-5, where the surface is at about Elevation 101.8 m.

The fill consists of silty sand to sand to sand and gravel. In Boreholes RW2-5 and RW2-7, the fill is associated with the local road structure; in the remaining boreholes, the fill is associated with both the construction of the QEW and the adjacent residential properties.

The Standard Penetration Test (SPT) “N”-values measured within the fill range from 7 blows to 18 blows per 0.3 m of penetration, indicating a loose to compact level of compactness.

The natural water content measured on two selected samples of the fill are about 3 per cent and 6 per cent.

4.2.1.4 Sandy Silt to Silt and Sand to Silty Sand to Sand

A 0.4 m to 3.0 m thick deposit of sandy silt to silty sand to silt and sand to sand was encountered underlying the topsoil and fill layers in all boreholes, with the exception of Borehole RW2-5. The surface of the granular deposit was encountered between Elevation 107.9 m and 106.5 m.

The SPT “N”-values measured within the sandy silt to silty sand to silt and sand to sand deposit range from 2 blows to 38 blows per 0.3 m of penetration, indicating a very loose to dense level of compactness. The SPT “N”-values generally increased with depth, with the lower SPT “N”-values encountered near the surface of the deposit.

Grain size distribution testing was carried out on five selected samples of the sandy silt to silty sand to silt and sand to sand deposit and the results are shown on Figure A1 in Appendix A. The natural water content measured on nine selected samples of the sandy silt to silty sand to silt and sand to sand deposit range between about 2 per cent and 13 per cent.

4.2.1.5 Gravelly Sand to Sand and Gravel

A 0.4 m to 1.5 m thick deposit of gravelly sand to sand and gravel was encountered underlying the sandy silt to silty sand to silt and sand to sand deposit in all boreholes except Boreholes RW2-2 and RW2-5, at between Elevations 106.0 m and 104.7 m.

The SPT “N”-values measured within the gravelly sand to sand and gravel deposit range from 11 blows to 45 blows per 0.3 m of penetration, indicating a compact to dense level of compactness. The SPT “N”-values increased with depth, with the sand and gravel portion of the deposit indicating a dense level of compactness.

Grain size distribution testing was carried out on three samples of the gravelly sand to sand and gravel deposit and the results are shown on Figure A2 in Appendix A. The natural water content measured on six selected samples of this granular range between 4 percent and 9 percent.

4.2.1.6 Clayey Silt

A 0.5 m thick clayey silt deposit was encountered underlying the gravelly sand deposit in Borehole RW2-7 at approximately Elevation 105.2 m and a 4.2 m thick clayey silt deposit was encountered underlying the till deposit (described below) in Borehole RW2-5 at approximately Elevation 99.1 m.

The SPT "N"-values measured within the clayey silt deposit range from 23 blows to 45 blows per 0.3 m of penetration, with one "N"-value of 50 blows per 0.1 m of penetration, suggesting a very stiff to hard consistency.

Atterberg limits testing was carried out on one sample of the clayey silt and measure a liquid limit of 33 per cent, a plastic limit of 21 per cent, and a corresponding plasticity index of 12 per cent. The result, which is plotted on a plasticity chart on Figure A3 in Appendix A, indicates that the deposit consists of clayey silt of low plasticity.

Grain size distribution testing was carried out on one sample of the clayey silt deposit and the result is shown on Figure A4 in Appendix A.

A natural water content measured on two samples of the clayey silt deposit is 10 per cent and 11 per cent.

4.2.1.7 Clayey Silt to Sandy Clayey Silt to Clayey Silt with Sand (Till)

A cohesive till deposit varying in composition from clayey silt to sandy clayey silt to clayey silt with sand to sandy clayey gravel was encountered underlying the sand and gravel and sand deposits in all boreholes, except Borehole RW2-5 where it was encountered underlying the silty sand fill layer, and underlying the clayey silt deposit in Borehole RW2-7. The top of the till deposit was encountered between Elevations 104.9 m and 103.6 m, with the exception of Borehole RW2-5, where it was encountered at Elevation 100.7 m. The deposit ranges in thickness from about 1.4 m to 5.1 m.

The SPT "N"-values measured within the cohesive till deposit range from 12 blows to 40 blows per 0.3 m of penetration with one "N"-value of 50 blows per 0.08 m of penetration at the bottom of the deposit, suggesting a stiff to hard consistency.

Atterberg limits testing was carried out on eight samples of the till deposit and measured liquid limits between 23 per cent and 31 per cent, plastic limits between 15 per cent and 20 per cent, and corresponding plasticity indices between 7 per cent and 12 per cent. These results, which are plotted on a plasticity chart on Figure A5 in Appendix A, indicate that the till deposit consists primarily of clayey silt of low plasticity.

Grain size distribution testing was carried out on seven selected samples of the till deposit and the results are shown on Figure A6 in Appendix A. The natural water content measured on ten samples of the cohesive till deposit range between about 9 per cent and 14 per cent.

4.2.1.8 Clayey Silt to Sandy Clayey Gravel (Residual Soil)

A cohesive residual soil deposit composed of clayey silt to sandy clayey silt to sandy gravelly clayey silt to sandy clayey gravel was encountered underlying the till deposit in Boreholes RW2-1, RW2-2, RW2-4, RW2-7 and STM-10, and underlying the clayey silt deposit Borehole RW2-5. The surface of the deposit was encountered between Elevations 103.3 m and 102.0 m, with the exception of Borehole RW2-5, where it was encountered at Elevation 94.9 m. The deposit ranges in thickness from about 1.5 m to 2.9 m, and is 0.5 m thick in Borehole RW2-5.

The SPT "N"-values measured within the residual soil deposit range from 48 blows to 84 blows per 0.3 m of penetration with one "N"-value of 50 blows per 0.8 m of penetration at the bottom of the deposit, suggesting a hard consistency.

Atterberg limits testing was carried out on one sample of the residual soil deposit and measured a liquid limit of 24 per cent, a plastic limit of 16 per cent, and a corresponding plasticity index of 8 per cent. This result, which is plotted on a plasticity chart on Figure A7 in Appendix A, indicates that the residual soil deposit consists of clayey silt of low plasticity.

The natural water content measured on selected samples of the residual soil deposit range between 5 per cent and 9 per cent.

A grain size distribution test was carried out on one sample of the residual soil deposit and the result is shown on Figure A8 in Appendix A.

4.2.1.9 Shale Bedrock

Bedrock was encountered in all boreholes except in Borehole RW2-5 where refusal was encountered at the bottom of the residual soil deposit either on inferred bedrock or a boulder, and core samples were recovered in the boreholes.

The depths to bedrock below ground surface, as determined from coring and inferred from the augering and split spoon sampling, and the corresponding bedrock surface elevation are summarized below.

Borehole	**Depth to Bedrock Surface (m)**	**Bedrock Surface (m)**	**Comments**
RW2-1	7.2	100.7	Split Spoon Sampling / Bedrock Cored
RW2-2	7.7	100.4	Split Spoon Sampling / Bedrock Cored
RW2-3	7.8	99.7	Bedrock Cored
RW2-4	8.5	99.6	Bedrock Cored
RW2-5	7.7	94.4	Refusal to Split Spoon Sampling / Augering
RW2-6	8.3	99.7	Bedrock Cored
RW2-7	7.2	100.6	Split Spoon Sampling / Bedrock Cored
STM-10	7.1	100.5	Split Spoon Sampling / Bedrock Cored

Based on a review of the bedrock core samples, the bedrock consists of shale of the Georgian Bay Formation. In general, the bedrock core samples are described as highly weathered to fresh, thinly laminated to medium bedded, very fine to fine grained, non-porous to faintly porous, very weak to weak, grey, containing medium strong limestone interbeds as presented in the Record of Drillhole sheets in Appendix A, and shown on the photograph of the recovered core samples on Figures A9 to A15 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., very weak to weak – R1 to R2) are described in accordance with the International Society for Rock Mechanics (ISRM[3]) standard classification system.

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

The Rock Quality Designation (RQD) measured on the core samples ranges from about 12 per cent to 100 per cent, but is generally greater than 51 per cent, indicating a rock mass of very poor to excellent quality as per Table 3.10 of CFEM (2006)[4]. The Total Core Recovery (TCR) and Solid Core Recovery (SCR) of samples recovered are between 87 per cent and 100 per cent and between 38 per cent and 100 per cent, respectively.

Unconfined Compression Strength (UCS) testing (ASTM D7012)[5] was carried out on five selected core sample of the shale bedrock obtained in Boreholes RW2-1, RW2-3, RW2-6, RW2-7 and STM-10 and measured uniaxial compressive strength ranging from about 15 MPa to 32 MPa, as detailed in Appendix A. The Young's modulus was measured to be between 530 MPa and 4,430 MPa.

Based on the laboratory UCS test, in accordance with Table 3.5 in CFEM (2006)[6], the shale bedrock is classified as weak (R2, 5 MPa < UCS < 25 MPa) to medium-strong (R3, 25 MPa < UCS < 50 MPa).

The results of the slake durability testing carried out on three selected core samples of the shale bedrock obtained in Boreholes RW2-2, RW2-3 and STM-10 are presented below.

Borehole Number / Run Number	Depth (m)	Moisture content (%)	Slake Durability Index (1st Cycle), *Id*1 (%)	Slake Durability Index (2nd Cycle), *Id*2 (%)
RW2-2 / 3	11.78 - 12.00	0.80	94.5	82.4
RW2-3 / 2	10.11 - 10.20	0.70	84.4	52.9
STM-10 / 4	13.16 - 13.34	0.83	90.9	78.5

The results of the CERCHAR abrasivity index (CAI) testing carried out on two selected core samples of the shale bedrock obtained in Boreholes RW2-7 and STM-10 are presented below.

Borehole Number / Run Number	Depth (m)	Mean Wear (mm)	CAI	Standard Deviation of CAI	ISRM Classification
RW2-7* / 2	9.86 - 9.99	0.107	1.07	0.18	Low
STM-10	12.80 - 13.01	0.017	0.17	0.07	Extremely Low

* Limestone sample

4.2.1.10 Groundwater Conditions

Details of the water levels observed in the open boreholes at the time of drilling are summarized on the records for Boreholes RW2-1 to RW2-6 in Appendix A of this report and below. The majority of the boreholes were noted to be dry upon completion of overburden drilling. A standpipe piezometer was installed in Boreholes RW2-7 and STM-10 to monitor the groundwater level at the site. The water levels measured in the open boreholes and the piezometers are summarized below:

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

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

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

Borehole No.	Ground Surface Elevation (m)	Depth to Water Level (m)	Groundwater Elevation (m)	Date	Comments
RW2-1	107.9	Dry	-	17-Nov-17	Open borehole - on completion of drilling and prior to rock coring
RW2-2	108.1	Dry	-	15-Nov-17	Open borehole - on completion of drilling and prior to rock coring
RW2-3	107.5	7.7	99.8	13-Nov-17	Open borehole - on completion of drilling and prior to rock coring
RW2-4	108.1	Dry	-	30-Oct-17	Open borehole - on completion of drilling and prior to rock coring
RW2-5	102.1	Dry	-	9-Sept-16	Open borehole - on completion of drilling
RW2-6	108.0	Dry	-	30-Oct-17	Open borehole - on completion of drilling and prior to rock coring
RW2-7	107.8	8.5	99.3	12-Dec-17	Open borehole - on completion of coring
		10.3	97.5	2-Apr-18	Piezometer
STM-10	107.6	Dry	-	10-Nov-17	Dry upon completion of drilling.
		4.8	102.8	28-Mar-17	Piezometer

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

4.2.1.11 Corrosivity Testing Results

As discussed in Section 3.0 four soil samples taken from Boreholes RW2-1, RW2-3, RW2-4 and RW2-7 were submitted for analysis of parameters used to assess the potential corrosivity of the site soils to steel and concrete. The analytical lab test report are presented in Appendix C and summarized as follows:

Parameter	Borehole RW2-1	Borehole RW2-3	Borehole RW2-4	Borehole RW2-7
pH	8.05	8.08	8.11	8.04
Resistivity (ohm-cm)	8,400	1,700	2,500	1,700
Conductivity (umho/cm)	119	584	395	588
Chlorides (ug/g)	<20	<20	<20	32
Soluble Sulphate (ug/g)	<20	580	280	550

4.2.2 Retaining Wall No. 24-888/W

The plan and profile along the proposed retaining wall showing the borehole locations and interpreted stratigraphy between about Station 13+749 and 13+859 (the eastern extent of wall to be replaced) are shown on Drawings 1 and 2, respectively. The Record of Borehole and Drillhole sheets (Boreholes RW3-1 to RW3-3) and the laboratory test results for this area are presented in Appendix B.

In general, the subsurface conditions in the boreholes consist of the QEW roadway pavement structure, which is underlain by sand to silty sand and gravel fill, which extends to the bedrock surface in the easternmost borehole. The pavement structure/fill in the boreholes near the central and western portions of the wall are underlain by a till deposit consisting of clayey silt to clayey silt with sand and gravel. The fill near the east end of the wall and the till deposit in the central and western portions of the wall are underlain by shale bedrock, the surface of which was encountered at between about Elevation 98.8 m and 99.7 m.

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

4.2.2.1 Asphalt / Concrete

An approximately 50 mm to 200 mm thick layer of asphalt pavement was encountered immediately below ground surface in Boreholes RW3-1 to RW3-3.

A 180 mm thick layer of concrete was encountered underlying the asphalt pavement in Boreholes RW3-1 and RW3-2.

4.2.2.2 Fill

A layer of non-cohesive fill was encountered underlying the asphalt pavement in all boreholes. The fill layer is approximately 0.4 m and 0.3 m thick in Boreholes RW3-1 and RW3-2, respectively, and about 2.1 m thick in Borehole RW3-3.

In Boreholes RW3-1 and RW3-2, the fill consists of sand and gravel and is associated with the road structure. In Borehole RW3-3, the fill consists of an upper layer of sand, some silt, trace gravel, underlain by a layer of gravelly silty sand containing trace clay and shale fragments.

The SPT "N"-value within the sand fill measured 8 blows per 0.3 m of penetration, indicating a loose level of compactness; while the SPT "N"-value within the silt sand and gravel layer measured 28 blows per 0.3 m of penetration, indicating a compact level of compactness.

The natural water content measured on one sample of the silt, sand and gravel fill was about 8 per cent.

A grain size distribution test carried out on one sample of the silt sand and gravel fill layer from Borehole RW3-3 and the result is shown on Figure B1 in Appendix B.

4.2.2.3 Clayey Silt to Clayey Silt with Sand and Gravel (Till)

A cohesive till deposit comprised of clayey silt to clayey silt with sand to clayey silt with sand and gravel was encountered underlying the sand and gravel fill in Boreholes RW3-1 and RW3-2. The top of the till deposit was encountered at about Elevation 103.7 m and 102.5 m, and extends for a thickness of about 4.9 m and 3.1 m in Boreholes RW3-1 and RW3-2, respectively.

The SPT "N"-values measured within the till deposit range from 21 blows to 43 blows per 0.3 m of penetration, with one "N"-value of 135 blows per 0.25 m of penetration, suggesting a very stiff to hard consistency.

Atterberg limits testing was carried out on four samples of the cohesive till deposit and measured liquid limits between 19 per cent and 27 per cent, plastic limits between 14 per cent and 18 per cent, and corresponding plasticity indices between 6 per cent and 10 per cent. These results, which are plotted on a plasticity chart on Figure B2 in Appendix B, indicate that the till deposit consists of clayey silt of low plasticity.

The natural water content measured on four samples of the till deposit range between about 6 per cent and 9 per cent.

Grain size distribution testing was carried out on two samples of the till deposit and the results are shown on Figure B3 in Appendix B.

4.2.2.4 Shale Bedrock

Bedrock was encountered in all boreholes and core samples were recovered in Borehole RW3-3. Bedrock samples were obtained by split-spoon sampling in Boreholes RW3-1, RW3-2 and RW3-3 over a bedrock thickness of between 0.4 m and 1.6 m, and was cored for a thickness of 3.7 m in Borehole RW3-3.

The depths to bedrock below ground surface, as determined from coring and inferred from the augering and split spoon sampling, and the corresponding bedrock surface elevation are summarized below.

Borehole	Depth to Bedrock Surface (m)	Bedrock Surface (m)	Comments
RW3-1	5.5	98.8	Auger / Split Spoon Sampling
RW3-2	3.7	99.3	Split Spoon Sampling
RW3-3	2.3	99.7	Split Spoon Sampling / Bedrock Cored

Based on a review of the bedrock core samples, the bedrock consists of shale of the Georgian Bay Formation. In general, the bedrock core samples are described as moderately to slightly weathered, thinly laminated, very fine to fine grained, non-porous, weak, grey, with medium strong limestone interbeds at varying intervals, as presented in the Record of Drillhole sheets in Appendix B, and shown on the photograph of the recovered core samples on Figure B4 in Appendix B. The degree of weathering of the bedrock samples (i.e., fresh to slightly weathered – W1 to W2), and the strength classification of the intact rock mass based on field identification (i.e., strong to very strong – R4 to R5) are described in accordance with the International Society for Rock Mechanics (ISRM[3]) standard classification system.

The Rock Quality Designation (RQD) measured on the core samples ranges from about 15 per cent to 87 per cent, indicating a rock mass of very poor to good quality as per Table 3.10 of CFEM (2006)[4]. The Total Core Recovery (TCR) and Solid Core Recovery (SCR) of samples recovered are between 72 per cent and 100 per cent and between 13 per cent and 94 per cent, respectively.

4.2.2.5 Groundwater Conditions

The overburden samples obtained from the boreholes were generally moist. All boreholes were dry upon completion of drilling and prior to rock coring.

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

4.2.2.6 Corrosivity Testing Results

As discussed in Section 3.0 one bedrock core sample taken from Borehole RW3-3 and one soil sample taken from Borehole RW3-1 was submitted for analysis of parameters used to assess the potential corrosivity of the site soils to steel and concrete. The analytical laboratory test report is presented in Appendix C and the results are summarized as follows:

Parameter	Borehole RW3-1	Borehole RW3-3
pH	8.02	8.18
Resistivity (ohm-cm)	670	2000
Conductivity (umho/cm)	1,500	499
Chlorides (ug/g)	140	<20
Soluble Sulphate (ug/g)	1,400	250

5.0 CLOSURE

This report was prepared by Ms. Nikol Kochmanová, P.Eng, a geotechnical engineer with Golder. Mr. Jorge M.A. Costa, P.Eng., a MTO Foundations Designated Contact and Senior Consultant with Golder, conducted a technical and quality control review of the report.

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PART B

FOUNDATION DESIGN REPORT
RETAINING WALLS NO. 24-887/W AND 24-888/W REPLACEMENT
QEW WIDENING FROM EAST OF CAWTHRA ROAD TO THE EAST MALL
CITIES OF MISSISSAUGA AND ETOBICOKE
MINISTRY OF TRANSPORTATION, ONTARIO
GWP 2102-13-00 & 2432-13-00

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides detail foundation engineering design recommendations for the proposed retaining wall replacements associated with the widening of the Queen Elizabeth Way (QEW) from Cawthra Road to the East Mall, Mississauga/Etobicoke, Ontario. These recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the subsurface investigation. The discussion and recommendations presented are intended to provide the designer with sufficient information to assess the feasible retaining wall alternative types and carry out the design of the retaining wall foundations.

The foundation investigation report, discussion and recommendations are intended for the use of the Ministry of Transportation, Ontario (MTO) and shall not be used or relied upon for any other purpose or by any other parties, including the construction or design-build contractor. The contractor must make their own interpretation based on the factual data in Part A (Foundation Investigation) of the report. Where comments are made on construction, they are provided to highlight those aspects that could affect the design of the project and for which special provisions may be required in the Contract Documents. Those requiring information on the aspects of construction must make their own interpretation of the factual information provided as such interpretation may affect equipment selection, proposed construction methods, scheduling, and the like.

6.1 General

6.1.1 Retaining Wall No. 24-887/W

Based on the design drawings provided to Golder by AECOM as part of the 60% design level submission on August 8, 2017, and subsequently the 90% Executive Review submission on January 17, 2018, the existing noise barrier wall and existing retaining wall, extending along the north side of the QEW between Stations 13+500 and 13+815, west of Etobicoke Creek, including the foundations, will be removed and replaced with a new retaining wall extending between about Stations 13+500 and 13+810, approximately 10 m (east end) to 15 m (west end) to the north of the existing wall. The proposed QEW grade in front of the new retaining wall will be between approximately Elevation 102 m and 107 m, rising from the east to the west, with the ground surface above/behind the new retaining wall at approximately Elevation 108 m; the new retaining wall will therefore be between approximately 1.8 m and 4.5 m high (exposed height along the QEW), except at the end where the exposed height is about 0.8 m.

Due to property constraints between Stations 13+650 and 13+750, a “narrow footprint” retaining wall system that is conducive to overall narrower construction footprint is desirable, such as a soldier pile and concrete panel wall, or a secant pile wall (also referred to as a caisson wall); both of these types of wall are feasible at this site from a geotechnical/foundations perspective. Based on the design drawings dated February 2018, a secant caisson wall is proposed between Stations 13+650 and 13+750, and will consist of structural caissons (i.e., steel-reinforced concrete caissons) spaced between 1.4 m and 1.8 m apart, with filler caissons (i.e., no reinforcing) in between. The Durisol Narrow Footprint system, a proprietary design by Armtec that is included on MTO’s DSM list, has also been contemplated by AECOM as a feasible alternative; this system consists of a steel post and precast concrete panel retaining wall system, similar to a soldier pile and concrete panel wall, which can accommodating the noise barrier wall constructed on top of the retaining wall.

Alternative retaining wall options are feasible where there are no property restrictions behind the wall, such as a concrete retaining wall founded on shallow foundations. Based on the 90% design drawings, a concrete toe wall is proposed between Stations 13+500 and 13+600 and a cantilever concrete retaining wall is proposed between

Stations 13+600 and 13+650, and between Stations 13+750 and 13+810; a secant pile (caisson) wall is proposed between Stations 13+650 and 13+750.

6.1.2 Retaining Wall No. 24-888/W

Based on the design drawings provided to us by AECOM as part of the 60% design level submission on August 8, 2017, and subsequently the 90% Executive Review submission on January 17, 2018, approximately 110 m of the easterly portion of the existing Retaining Wall 24-888/W, extending along the south side of the QEW to the east of Boxwood Way in Mississauga, from about Stations 13+749 to 13+859, is proposed to be replaced. The proposed replacement section will consist of an approximately 3.1 m to 3.4 m high retaining wall (exposed height along the QEW/W-N Ramp) except at the east end where it is approximately 1.3 m high; the alignment of the proposed retaining wall will be maintained at the west end (i.e., connecting to the existing retaining wall) and will be shifted approximately 3 m to the south at the east end.

Based on conversations with AECOM, it is understood that the final wall type for Retaining Wall No. 24-888/W will be dependent on the type of wall constructed at Retaining Wall No. 24-887/W, to achieve similar aesthetic appearances on both sides of the highway corridor at this location. As noted above, from a geotechnical perspective, both a secant pile wall (caisson wall) and a post and concrete precast panel wall are feasible for the new section of this retaining wall. Where there are no property restrictions, alternative retaining wall options, such as a concrete retaining wall founded on shallow foundations, is also feasible.

6.2 Consequence and Site Understanding Classification

In accordance with Section 6.5 of the 2014 Canadian Highway Bridge Design Code and its Commentary (CHBDC, 2014), the retaining wall replacements and their foundation systems are considered to be classified as having a "typical consequence level" associated with exceeding limits states design. In addition, given the level of foundation investigation completed to date in comparison to the degree of site understanding in Section 6.5 of CHBDC (2014), the level of confidence for design is considered to be a "typical degree of site and prediction model understanding." Accordingly, the appropriate corresponding ULS and SLS consequence factor, Ψ, from Table 6.1 and geotechnical resistance factors, ϕ_{gu} and ϕ_{gs}, from Table 6.2 of the CHBDC (2014) have been used for design, as indicated in the sections below.

6.3 Seismic Design

6.3.1 Seismic Site Classification

Subsurface ground conditions for seismic site characterization were established based on the results of the field investigation and laboratory testing. The SPT "N"-values measured in the soil layers and the interpreted shear wave velocity of soils up to 30 m below founding level were used to define the seismic site classification in accordance with Table 4.1 of the CHBDC (2014). Based on this methodology it is considered that a Site Class C would be applicable for the design of the replacement retaining wall structures.

6.3.2 Spectral Response Values and Seismic Performance Category

In accordance with Section 4.4.3.4 of the CHBDC (2014), the peak ground acceleration (PGA) values and design spectral acceleration (Sa) values for Site Class C based on the National Resource Canada (NRC) website are presented below.

Seismic Hazard Values	10% Exceedance in 50 years (475-year return period)	5% Exceedance in 50 years (975-year return period)	2% Exceedance in 50 years (2,475 return period)
PGA (g)	0.042	0.075	0.144
PGV (m/s)	0.031	0.052	0.093
Sa (0.2) (g)	0.069	0.120	0.224
Sa (0.5) (g)	0.042	0.067	0.117
Sa (1.0) (g)	0.023	0.036	0.059
Sa (2.0) (g)	0.011	0.017	0.028
Sa (5.0) (g)	0.0023	0.0039	0.0067
Sa (10.0) (g)	0.001	0.0016	0.0028

6.4 Retaining Wall and Foundation Options

This section of the report presents a comparison of alternative retaining wall / foundation types based on advantages and disadvantages of the alternative retaining wall types and provides geotechnical recommendations for the various types of walls and foundation alternatives.

It should be noted that the selection of the type of walls and foundation alternative will depend on many factors beyond geotechnical / foundation recommendations. From a geotechnical/foundations perspective the type of retaining wall considered suitable for the replacement of the existing retaining walls given the soil conditions as encountered in the various boreholes drilled at the retaining wall sites include the following:

- **Soldier Pile and Concrete Panel Walls:** A soldier pile and concrete panel system (or similar, including proprietary post-and pre-cast panel wall systems) is considered appropriate for Retaining Wall Nos. 24-887/W and 24-888/W, as this type of wall is generally more advantageous in “top-down” construction applications as part of a cut widening, as is the case at these locations along the QEW, rather than for an embankment widening. This type of wall system would decrease the excavation zone and potentially decrease the need for temporary excavation support along the retaining walls. Lateral restraint may be required in the form of soil anchors at some locations. Easements may be required to accommodate the soil anchors depending on the distance from the wall to the property limits. It is considered that construction of a soldier pile and concrete panel wall would be more time-consuming than the construction of a concrete cantilever wall due to the various steps involved (i.e., augering holes; placing and concreting soldier piles; placing backfill in lifts and installing concrete panels; and installing and pre-stressing tie-backs, including testing of selected tie-backs).
- **Drilled Shaft (Caisson) Foundations in a Secant Pile Wall Configuration:** Drilled shaft (caisson) foundations used in a secant pile wall configuration are feasible and considered highly suitable for Retaining Wall Nos. 24-887/W and 24-888/W where property restrictions are present. Similar to soldier pile and concrete panel walls, this wall system is more advantageous in “top-down” construction applications and consists of king piles or soldier piles socketted to sufficient depth within the native soils or the shale bedrock to provide the necessary axial and passive (lateral) resistance for the retained soil height.
- **Concrete Retaining Wall on Deep Foundations:** A concrete wall supported on deep foundations (driven piles or caissons) is considered feasible from a geotechnical/foundations perspective for Retaining Wall Nos.

24-887/W and 24-888/W, provided that sufficient space is available. A concrete retaining wall supported on deep foundations may potentially reduce the excavation zone, require less of a protection system and less backfill compared to a concrete retaining wall supported on shallow foundations. Temporary or permanent liners may be required to construct the deep foundations (i.e., caissons) depending on the soil conditions encountered during construction.

- **Concrete Retaining Wall on Shallow Foundations:** A concrete cantilever retaining wall supported on shallow foundations (concrete strip footing) is also geotechnically feasible for Retaining Wall Nos. 24-887/W and 24-888/W, although depending on the proximity of the excavation to adjacent property/structures/highway limits, temporary excavation support may be required to accommodate excavations to allow for construction of the strip footings. Removal of the existing Retaining Wall No. 24-888/W will require excavation to the base of the existing foundation(s), which would allow for construction of the shallow foundations for the replacement structure. Concrete cantilever retaining walls supported on shallow foundations are typically less tolerable to post construction settlements.
- **Reinforced Soil System (RSS) Wall:** RSS walls are geotechnically feasible but not considered practical at the Retaining Wall Nos. 24-887/W and 24-888/W sites due to space and property restrictions and are not discussed further.
- **Conventional Earth Embankment or Reinforced Earth Slope:** A conventional earth slope constructed at an inclination of 2 Horizontal to 1 Vertical (2H:1V) is geotechnically feasible but not considered practical at the Retaining Wall Nos. 24-887/W and 24-888/W sites due to space and property restrictions and are not discussed further..

A comparison of the various retaining wall options based on advantages, disadvantages and relative cost is presented in Table 1. Based on a comparison of the advantages/disadvantages between the various wall types and supporting foundation alternatives and given the subsurface conditions as encountered at the boreholes, the preferred retaining wall alternative from a geotechnical perspective for the two retaining walls may be summarized as:

- Retaining Wall No. 24-887/W – Soldier Pile and Concrete Panel Wall or similar (e.g., secant caisson wall) where property restrictions exist; Soldier Pile and Concrete Panel Wall or Concrete Cantilever Retaining Wall on Shallow Foundations in other locations
- Retaining Wall No. 24-888/W – Soldier Pile and Concrete Panel Wall or Concrete Cantilever Retaining Wall on Shallow Foundations

The following sections of this report present the results of the assessment/analyses of settlement and global stability for Retaining Wall Nos. 24-887/W and 24-888/W, comparison of the wall/foundation alternatives and provide geotechnical recommendations for the preferred options.

6.5 Soldier Pile and Concrete Panel Wall or Drilled Shaft (Caisson) Foundations in a Secant Pile Wall Configuration

A soldier pile and concrete panel wall (including proprietary post and pre-cast panel wall systems) and drilled shaft (caisson) (foundations used in a secant pile wall configuration) are feasible for Retaining Wall Nos. 24-887/W and 24-888/W where property restrictions are present as a constraint to other types of feasible wall construction. These walls are advantageous in this area, since it would minimize temporary excavation into the cut slope compared to the other wall types (i.e., for construction of spread footings for concrete cantilever or reinforced soil masses). For this project, a secant caisson wall is proposed for the section of Retaining Wall No. 24-887/W between Stations 13+650 and 13+750.

These wall systems consist of king piles or soldier piles socketted to sufficient depth within the native soils or the shale bedrock to provide the necessary axial and passive (lateral) resistance for the retained soil height. Axial geotechnical resistance recommendations for the king piles or soldier piles (i.e., steel H-piles installed in concrete caissons) are provided in Section 6.6.2 (Caisson Foundations) of this report. If required, additional lateral support to the wall system could be provided in the form of permanent soil or bedrock anchors located at strategic locations along the retaining walls; however, based on the property limits, such anchors would encroach within the private property and would require a permanent easement.

The concrete lagging panels should be installed as the excavation for the cut progresses such that the unsupported height does not exceed 1.2 m at any time, and the space behind the lagging should be immediately packed with granular material to ensure intimate contact of the soil with the back of the wall and to aid in achieving proper drainage. If sufficient thickness of free-draining granular soil is not provided behind the concrete panels to provide adequate drainage and frost protection, consideration should be given to using a drainage sheet. An insulation layer could also be provided immediately behind the wall to provide frost protection, if required.

6.5.1 Passive Resistance for King Pile or Soldier Pile Sockets

The ultimate passive lateral pressure in front of the soldier piles may be assessed using Brom's equation (1964) using the design parameters / values as follows:

K_p	the coefficient of passive earth pressure, which may be taken as 3.3 for the clayey silt till and clayey silt residual soil. This K_p value must be reduced by an appropriate factor that considers the allowable wall movement in accordance with Figure C6.16 of CHBDC
γ'	the effective unit weight of the soil in front of the soldier pile socket, which may be taken as 10 kN/m3 below the groundwater level

The upper 1.2 m of soil in front of the secant piles/soldier piles should be ignored in the calculation of the passive resistance, to account for disturbance during installation, and for frost effects as interpreted from OPSD 3090.101 (*Frost Penetration Depths for Southern Ontario*).

The factored passive lateral resistance (f_{horiz}) for the fresh rock mass may be taken as 2 MPa.

6.5.2 Permanent Soil Anchors

If required, additional lateral support to the wall system could be provided in the form of permanent soil or bedrock anchors; however, such anchors would encroach beyond the MTO right-of-way into private property, and would require a permanent easement. On this basis, it is assumed that the wall design will endeavour to avoid the use of permanent anchors.

If required, a soil anchor support system can be designed to accommodate the loads applied from lateral earth pressures and surcharge pressures from area, line or point loads and take into account any sloping ground behind the retaining wall system. For design, the soil anchors may be sized based on the following unfactored bond stresses acting between the grout and native soil:

Soil Deposit	Estimated Ultimate Load Transfer (kN/m)
Hard clayey silt till and clayey silt residual soil	60
Shale Bedrock	145

In accordance with the CHBDC (2014), a factor of 0.4 should be applied to the unfactored bond stress value for ULS conditions. The SLS value for 25 mm of displacement will not govern and may be greater than the ULS value. For design purposes an SLS value equal to the ULS value should be used.

The sustained working load should not be greater than 60 per cent of the ultimate tensile strength of the anchor tendons or bars. Soil tie-back anchors should have their fixed length (bond zone) formed within the native very stiff to hard clayey silt till deposit, and should be installed at a downwards angle of 20 degrees or steeper. The first row of anchors should be installed not less than 1.5 m below the top of the wall face. A minimum of 4.5 m of overburden is required above the center of the fixed length (bond zone) to provide the necessary overburden pressure to develop anchor capacity in gravity-grouted anchors; to prevent grout leakage during installation of pressure grouted anchors and to prevent heaving of the ground surface for higher grout pressure operations (FHWA, 1999). The fixed length (bond zone) of the anchors should be at least 3 m (and may be up to 8 m) and should be maintained behind a line drawn upward at 45 degrees from the toe of the proposed wall. The horizontal spacing between anchors will be dependent of the spacing of the soldier piles but should be greater than four times the diameter of the anchor diameter (grouted section) or 1.2 m. The permanent soil anchors should be provided with suitable corrosion protection.

Lateral earth pressures for design are discussed in Section 6.9. Anchor installation, grouting and testing should be carried out in accordance with OPSS 942 (*Pre-Stressed Soil and Rock Anchors*).

6.5.3 Global Stability

Slope stability analyses have been performed for the proposed retaining walls using the commercially available program SLIDE V7 produced by Rocscience Inc., employing the Morgenstern-Price method of analysis. For all analyses, the Factor of Safety (FoS) of numerous potential failure surfaces was computed in order to establish the minimum FoS. The FoS is defined as the ratio of the forces tending to resist failure to the driving forces tending to cause failure. A target minimum factored FoS of 1.54 is adopted for the design of retaining wall height and geometries under static conditions at the end of construction as per the CHBDC (2014). This FoS is considered adequate for the retaining walls at this site considering the design requirements and the field data available. In general, circular slip surfaces were analysed in the design.

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

Soil Deposit	Bulk Unit Weight (kN/m^3)	Undrained Shear Strength (kPa)	Cohesion (c') kPa	Effective Friction Angle
Loose granular fill	19	-	-	28°
Very stiff to hard clayey silt till	21	150	-	32°
Hard clayey silt residual soil	21	200	-	33°

A maximum retained wall height of 4.5 m was assumed for the retaining walls. The groundwater level was inferred from the highest water levels shown on the borehole records.

The stability analysis result indicates that the proposed soldier pile and concrete panel wall at both Retaining Wall Nos. 24-887/W and 24-888/W will have a FoS greater than 1.54 against global instability. An example of the static global stability results is provided on Figure 1. It should be noted that since the soldier piles are recommended to be socketed into the bedrock, no failure surfaces were found based on the assumption that the shale bedrock and the soldier piles have infinite strength.

6.6 Concrete Retaining Wall Founded on Deep Foundations

6.6.1 Driven Steel H-Pile or Steel Pipe (Tube) Pile Foundations

6.6.1.1 Founding Elevations

Driven steel H-Pile or steel tube (pipe) pile foundations are feasible for Retaining Wall Nos. 24-887/W and 24-888/W, but may not be practical where the depth to bedrock is encountered at shallow depths, as is the case for the eastern limit of Retaining Wall No. 24-888/W. If driven piles are the preferred alternative, pre-augering/coring would be required at both retaining wall locations to socket the piles adequately into bedrock.

For steel HP 310 x 110 piles or steel tube piles (324 mm diameter x 6.4 mm thickness) driven to refusal on or in the shale bedrock at Retaining Wall Nos. 24-887/W and 24-888/W, the design pile tip elevations provided below may be used for design of the pile foundations. The founding elevations are based on the strength and weathering observed in the recovered core samples and these recommendations assume only nominal penetration (up to 0.5 m) into the bedrock. The structural designers should assess whether the pile lengths are sufficient from a structural perspective. If longer pile lengths are required from a structural perspective, deeper pre-augering/coring would be needed to penetrate further into the shale bedrock.

Retaining Wall Site	Approximate Station (Reference Boreholes)	Estimated Pile Cap Elevation (m)	Bedrock Surface Elevation (m)	Estimated Design Pile Tip Elevation (m)	Approximate Pile Length (below pile cap) (m)
Retaining Wall No. 24-887/W	13+500 to 13+600 (RW2-1 to RW2-3)	105.3	99.9 to 100.7	99.6 to 100.4	4.9 to 5.7
	13+600 to 13+750 (STM-10, RW2-4 and RW2-6)	104.8 to 103.8	100.0 to 100.9	99.7 to 100.7	3.9 to 4.1
	13+750 to 13+770 (RW2-7)	102.3	100.5	100.2	2.1
	13+770 to 13+810 (RW2-5)	101.8	94.4	94.2	7.6
Retaining Wall No. 24-888/W	13+749 to 13+859 (RW3-1to RW3-3)	103.3	98.8 to 100.0	98.5	4.8

It is recommended that provision be made in the Contract Documents to deal with piles of varying lengths due to the variations in the bedrock surface elevation as shown above.

If the piles are to be driven, consideration must be given to the potential presence of cobbles and boulders within the fill and glacially-derived soils at this site, as well as the potential for damage to the pile tips during seating on the bedrock. In this regard, steel H-piles are preferred over steel tube piles given that steel tubes are considered to pose a higher risk of “hanging up” or being deflected from their vertical or battered orientation during installation, due to their larger end area. The piles should be reinforced at the tip for protection during driving to reduce the potential for damage to the piles in the event that cobbles/boulders and/or very dense layers are encountered within the till deposits. The steel H-piles should be reinforced with flange plates as per OPSD 3000.100 (*Foundation Piles Steel H-Pile Driving Shoe*) or driving shoes such as Titus Standard “H” Bearing Pile Point design for protection during driving. Similarly, if steel tube piles are being considered, driving shoes should be in accordance with OPSD 3001.100 Type II (*Steel Tube Pile Driving Shoe*). The requirement for driving shoes should be included in the Contract Drawings.

The pile caps for the new retaining walls should be provided with a minimum of 1.2 m of soil cover to provide adequate protection against frost penetration as interpreted from OPSD 3090.101 (*Foundation Frost Depths for Southern Ontario*).

6.6.1.2 Factored Geotechnical Axial Resistances

For steel HP 310 x 110 piles (or steel tube piles) (324 mm diameter x 6.4 mm thickness) driven to/into shale bedrock to the design pile tip elevations provided in Section 6.6.1.1, the factored ultimate geotechnical resistance of 1,600 kN per pile may be used for design. The factored serviceability geotechnical resistance at SLS for 25 mm of settlement (for the length of piles required at this site) will be greater than the factored ultimate geotechnical resistance at ULS, as such, the factored ultimate geotechnical resistance at ULS will govern for this foundation type.

The following note, or similar notation, should be shown on the Contract Drawings assuming that a resistance factor of 0.5 is applied to the use of the Hiley calculation based on MTO experience in the Southern Ontario region (refer to the Structural Manual Section 3.3.3 (MTO, 2016)) for Retaining Walls No. 24-887/W and 24-888/W:

> *“Piles to be driven to bedrock.”*

Pile installation should be in accordance with OPSS.PROV 903 *(Deep Foundations).* The pile termination or set criteria will be dependent on the pile driving hammer type, helmet, selected pile and length of pile; the criteria must therefore be established at the time of construction after the piling equipment is known to ensure that the piles are not overdriven and to avoid possible damage to the piles. As the upper portion of the shale bedrock demonstrates some weathering, it is anticipated that the piles will penetrate nominally (up to 0.5 m) into the bedrock. Assuming a hammer energy of 60 kJ, a set criterion of 10 blows (or greater) per 25 mm (or less) penetration is recommended for driving into the shale bedrock to achieve the geotechnical resistances given above. Alternatively, if or when “refusal” is met on stronger shale or limestone interbeds, it is a generally accepted practice to reduce the hammer energy after abrupt peaking is met in the bedrock, and then to gradually increase the energy over a series of blows to seat the pile in the bedrock. An NSSP which outlines the above criteria for seating the piles on bedrock, should be included in the Contact Documents; an example is provided in Appendix C.

6.6.1.3 Resistance to Lateral Loads

6.6.1.3.1 Fill and Native Soil Materials

Resistance to lateral loading can be derived using vertical piles, with enhanced support offered by battered piles, if required. If vertical piles are used, the resistance to lateral loading will have to be derived solely from the soil in front of the piles, whereas battered piles derive lateral resistance from the soil in front of the piles as well as the horizontal component of the axial load present in the inclined pile. For piles, the resistance to lateral loading will be derived from the soil and bedrock in front of the king piles/soldier piles, for the secant pile wall/soldier pile and lagging wall.

Where ground conditions are generally competent and the lateral loads on piles are relatively small such that the maximum lateral pile deflections will be relatively small, the resistance to lateral loading in front of a single pile can be estimated using subgrade reaction theory (as outlined below). However, if it be noted that the response of a pile to lateral loads is highly nonlinear and methods that assume linear behavior (such as subgrade reaction theory) are only appropriate where the maximum pile deflections are less than 1 per cent of the pile diameter, where the loading is static (no cycling) and where the pile material is linear (CFEM, 2006). Where these conditions are not met, the non-linear lateral behavior of the soil should be considered by the use of P-y curves.

The factored serviceability geotechnical response of the soil in front of the piles under lateral loading at this site may be calculated using subgrade reaction theory suggested in CHBDC (2014) Commentary (Section C6.11.2.2), where the coefficient of horizontal subgrade reaction, k_h, (kPa/m) is based on the equation given below, as described by Terzaghi (1955) and the Canadian Foundation Engineering Manual (CFEM 1992).

For non-cohesive soils:

$$k_h = \frac{n_h z}{B}$$

Where

k_h is the coefficient of horizontal subgrade reaction (kPa/m);

n_h is the constant of subgrade reaction (kPa/m);

z is the depth (m); and

B is the pile diameter or width (m).

For cohesive soils:

$$k_h = \frac{67 s_u}{B}$$

Where

k_h is the coefficient of horizontal subgrade reaction (kPa/m);
s_u is the undrained shear strength of the soil (kPa); and
B is the pile diameter or width (m).

The following values of n_h and s_u (Terzaghi, 1995) may be incorporated into the calculations of horizontal subgrade reaction (k_h) for structural analyses for a single vertical pile

Soil Unit	n_h (kPa/m)	s_u (kPa)
Existing fill (assuming engineered non-cohesive fill)	5,000	-
Very stiff to hard clayey silt to silty clay till	-	200
Hard clayey silt residual soil	-	250

Both the structural and geotechnical resistances of the piles should be evaluated to establish the governing case at ULS. At SLS, the horizontal reaction of the piles will be controlled by deflections and the horizontal resistance of the pile should be calculated based on the coefficient of horizontal subgrade reaction (k_h) of the soil as discussed above.

The upper zone of the soil (down to a depth below the pile cap equal to about 1.5xB (where B is the pile diameter) should be neglected in the calculation of lateral resistance of the pile to account for disturbance effects during installation.

Group action for lateral loading should be considered where the pile spacing in the direction of the loading is less than six to eight pile diameters. Group action can be evaluated by reducing the coefficient of horizontal subgrade reaction in the direction of loading by a reduction factor, R (NAVFAC DM-7.02, 1986) as follows:

Pile Spacing in direction of Loading (d = Pile Diameter)	Subgrade Reaction Reduction Factor (R)
8d	1.00
6d	0.70
4d	0.40
3d	0.25

The subgrade reaction reduction factor should be interpolated for pile spacings in between those provided above.

6.6.1.3.2 Bedrock

For the proposed wall heights and loads on the retaining wall, the rock response is expected to remain in the elastic range; therefore, closed form solutions have been used for the preliminary estimation of the ground spring constant.

The lateral rock mass spring constant can be based on the equation given below:

$$k_h = \frac{4\pi(1-\nu)E_h}{(3-4\nu)(1+\nu)} \frac{1}{\ln(r_o/r_i)}$$

Where

k_h is the lateral rock mass spring constant (MPa/m);

E_h is the lateral rock mass elastic modulus (MPa);

ν is Poisson's ratio, which can be taken as 0.2;

r_i = radius of caisson; and,

r_o = radius of 'zero' deformation; typically 10 to 15 caisson diameters (m).

The following lateral rock mass elastic moduli can be used for preliminary purposes and will be confirmed following laboratory testing:

Bedrock	Lateral Rock Mass Elastic Modulus, E_h (MPa)
Weathered shale (assume minimum 2 m thickness)	100
Fresh Shale	400

6.6.2 Caisson Foundations

6.6.2.1 Caisson Founding Elevations

It is anticipated that king piles for a secant pile wall, or soldier piles for a soldier pile and panel wall, will extend into the bedrock over much of the wall length in order to satisfy the lateral loading requirements; however, where the depth to bedrock is deeper and/or the wall height is lower, some vertical elements may terminate within the overlying till/residual soil deposits.

6.6.2.1.1 Foundations in Overburden Soils

For secant piles/soldier piles founded/socketted within the clayey silt till or clayey silt residual soil the following design base elevations may be used:

Retaining Wall	Approximate Station	Reference Boreholes	Design Base Elevation (m)
24-887/W	13+500 to 13+640	RW2-1 to RW2-3 and STM-10	101.5
	13+640 to 13+770	RW2-4, RW2-6 and RW2-7	101
	13+770 to 13+810	RW2-5	95

Retaining Wall	Approximate Station	Reference Boreholes	Design Base Elevation (m)
24-888/W	13+749 to 13+859	RW3-1 to RW3-2	99.5

The performance of caissons will depend upon the final cleaning and verification of the subgrade at the base of the caissons. Each caisson excavation should be carefully cleaned to remove all loosened debris to ensure that the concrete is in intimate contact with the competent bearing stratum.

6.6.2.1.2 Sockets in Shale Bedrock

As the degree of weathering in the upper portion of the bedrock varies, socketting a minimum of approximately 2 m into the good quality bedrock is recommended for design purposes. The final socket depth will depend on the height of soil retained, lateral loads on the retaining wall, secant pile/soldier pile socket diameter and the strength of the bedrock. The following base elevations may be used for preliminary design, and these elevations will be refined following initial design by AECOM's structural engineers based on the anticipated pile length/socket depth to accommodate lateral loading.

Retaining Wall	Reference Boreholes	Bedrock Surface Elevation (m)	Surface of Good Quality Bedrock (RQD > 50%) (m)	Design Base Elevation (m)
24-887/W	RW2-1 to RW2-3	99.9 to 100.7	97.0 to 99.0	95.0
	STM-10, RW2-4, RW2-6 and RW2-7	100.0 to 100.9	98.0 to 99.0	96.0
	RW2-5	94.4	-*	91.5
24-888/W	RW3-1 to RW3-3	98.8 to 100.0	97.3	95.3

* Bedrock was not cored at this location.

The shale bedrock is weak to medium-strong as assessed by UCS testing within the project limits (with calculated UCS values ranging between about 15 MPa to 32 MPa), but in this area of the overall project (Cawthra Road to The East Mall) the shale is generally considered to be very weak to weak with unconfined compressive strengths in the range of 5 MPa to 7 MPa; and with medium strong to strong limestone layers, and therefore the sockets may likely be advanced into the bedrock by churn drilling. If caissons are adopted as the foundation alternative, it is recommended that an NSSP be included in the Contract Documents to describe to the Contractor the strength and character of the bedrock; an NSSP is included in Appendix C for this purpose.

6.6.2.2 Factored Geotechnical Axial Resistances

For caissons designed for end bearing and shaft friction combined, the performance of the caissons in compression will depend to a large degree upon the final cleaning and verification of the condition of the subgrade rock at the base of the caisson. For caissons acting in compression, the base of each caisson excavation must be cleaned to remove all loose cuttings to ensure that the tremied concrete is in intimate contact with the competent shale bedrock. The inspection of the base of the rock sockets can be accomplished after flushing and cleaning of the base by means of a Shaft Inspection Device (SID) such as a video camera. Should the camera inspection indicate that loosened/unacceptable soil or rock is present at the base the caisson, the socket base would need to be re-cleaned and re-inspected. A geotechnical engineer must confirm that the conditions encountered are consistent with the information obtained from the boreholes and that the required minimum socket geometry and cleanliness has been obtained.

The centre-to-centre spacing between proposed caissons within a group founded in bedrock should be greater than 2.5 times the caisson diameter to limit interaction between caissons. So long as this minimum caisson spacing within a group is maintained, the efficiency factor for the pile group is expected to be 1.0 (i.e. no reduction for group effects is required).

6.6.2.2.1 Foundations in Overburden Soils

The recommended design values for the factored ultimate geotechnical axial resistance at ULS and factored serviceability geotechnical resistance (for 25 mm of settlement) for caissons founded at the elevations given in Section 6.6.2.1.1 are provided below.

Retaining Wall	Approximate Station	Diameter (m)	Factored Ultimate Geotechnical Resistance (kN)	Factored Serviceability Geotechnical Resistance, for 25 mm of Settlement (kN)
24-887/W	13+500 to 13+640	0.9	1,500	1,250
		1.2	2,500	2,000
	13+640 to 13+770	0.9	1,500	1,250
		1.2	2,500	2,000
	13+770 to 13+810	0.9	1,500	1,250
		1.2	2,500	2,000
24-888/W	13+749 to 13+859	0.9	1,100	925
		1.2	2,000	1,650

6.6.2.2.2 Sockets in Shale Bedrock

The secant pile or soldier pile sockets can be designed based on shaft resistance in the rock socket and a proportion of end-bearing on the base, based on the length-to-diameter ratio. The following values for factored ultimate geotechnical resistance may be used in design for caissons socketed into bedrock extending to the base excavation (s) given in Section 6.6.2.1.2:

Diameter (m)	Factored Ultimate Geotechnical Resistance (kN)
0.75	650
0.9	950
1.2	1,700

The factored serviceability geotechnical resistance for 25 mm of settlement is greater than the values given for the factored ultimate geotechnical resistance, and therefore the serviceability condition does not apply. To achieve these design values, the base of the rock sockets must be adequately cleaned, and the base conditions verified with a down-hole camera.

The shale in this area of the overall project (Cawthra Road to The East Mall) is generally considered to be very weak to weak with unconfined compressive strengths in the range of 5 MPa to 7 MPa; and with medium strong to strong limestone layers, and therefore the sockets may likely be advanced into the bedrock by churn drilling.

6.6.2.3 Resistance to Lateral Loads

Resistance to lateral loading will be derived from the soil in front of the caissons. The resistance to lateral loading in front of the caisson may be calculated using subgrade reaction theory and the equations and soil parameters provided in Section 6.6.1.3.1 or 6.6.1.3.2 may be used for design.

6.6.2.4 Filler Caissons

Based on the design drawings provided by AECOM, filler caissons will be constructed between and overlapping with the adjacent structural caissons for the secant caisson wall section of Retaining Wall No. 24-887/W between about Sta 13+650 and Sta 13+750. We understand that the filler caissons will not carry any vertical load nor be subjected to lateral loads, with the bearing pressure exerted due to self-weight only. The filler caissons should be founded below any existing fill materials within the native soils, and should be founded a minimum of 1.2 m below the lowest final grade, that is, below the depth of frost penetration, as interpreted from OPSD 3090.101 (*Foundations, Frost Penetration Depth for Southern Ontario*).

6.6.3 Global Stability

Slope stability analyses have been performed for the proposed retaining walls. The global stability analysis outlined in Section 6.5.3 may be used for design. As with the soldier pile and lagging retaining walls, concrete cantilever retaining walls founded on deep foundations driven to or drilled into the shale bedrock will have a factor of safety greater than 1.5 against global instability. An example of the static global stability results is provided on Figure 1. It should be noted that since the piles are recommended to be driven/drilled into the bedrock, no failure surfaces were found based on the assumption that the shale bedrock and the soldier piles have infinite strength.

6.7 Concrete Cantilever Wall Founded on Shallow Foundations

6.7.1 Founding Elevations

Strip footing (shallow) foundations are feasible for the support of Retaining Wall Nos. 24-887/W and 24-888/W and should be founded below any fill or softened/loosened surficial soils. For Retaining Wall No. 24-887/W, strip footings founded on the dense sand to gravelly sand or very stiff to hard clayey silt till can be considered. Based on the design drawings for Retaining Wall No. 24-887/W, the proposed footing elevations for the wall between Station 13+600 and 13+650 are within the existing fill material and the founding level will either have to be lowered, and the wall height or the footing thickness increased, to found within the underlying clayey silt till deposit, or the fill will need to be subexcavated and replaced with a granular pad (see below). For Retaining Wall No. 24-888/W, strip footings should be founded on the very stiff to hard clayey silt till at the west end and center of the retaining wall, and on shale bedrock at the east end of the retaining wall.

All footings should be founded at a minimum depth of 1.2 m below the adjacent final grade to provide adequate protection against frost penetration, in accordance with OPSD 3090.101 (*Foundation, Frost Penetration Depths for Southern Ontario).*

The sand to gravelly sand, clayey silt till and shale bedrock subgrades will be susceptible to disturbance and degradation on exposure to water and construction traffic. It is recommended that a 100 mm thick 20 MPa concrete working slab be placed on the prepared subgrade if footing construction is not carried out within four hours following inspection and approval of the subgrade, to protect the subgrade from softening; this requirement can either be added as a note on the Contract Drawings or included as a Non-Standard Special Provision (NSSP) in the Contract Documents. A sample NSSP is included for this item in Appendix C.

The following founding elevations for the retaining walls are recommended for strip footings founded on competent native materials.

Retaining Wall Site	Approximate Station / Reference Boreholes	Founding Stratum	Sub-excavation Required?	Highest Founding Elevation*
Retaining Wall No. 24-887/W	13+500 to 13+575 RW2-1 and RW2-2	Compact to dense sand to gravelly sand	No	105.3
	13+575 to 13+650 RW2-3 and STM-10	Hard sandy clayey silt till / Compact gravelly sand	No	104.5
	13+650 to 13+725 RW2-4 and RW2-6	Very stiff clayey silt to sandy clayey silt till	No	104.8
	13+725 to 13+770 RW2-7	Very stiff to hard sandy gravelly clayey silt till	No	104.5
	13+770 to 13+810 RW2-5	Very stiff clayey silt till	No	100.7
		Compact Granular Backfill	Yes to Elevation 100.7 m	101.8
Retaining Wall No. 24-888/W	13+749 to 13+775 RW3-1	Very stiff to hard clayey silt till	No	103.0
	13+775 to 13+800 RW3-2	Hard clayey silt till	No	101.8
	13+800 to 13+859 RW3-3	Shale Bedrock	No	99.7 m

* The highest founding elevations provided are based on the soil conditions and may be higher than the minimum required depth of 1.2 m at some locations and should be lowered to be founded below the depth of frost penetration.

A continuous strip footing constructed of sections at different founding elevations must include a sloping base on native ground (or granular pad) between sections, inclined no steeper than 1H:1V (i.e., not vertical)

The footing subgrade should be inspected by qualified geotechnical personnel following excavation, in accordance with OPSS 902 (*Excavating and Backfilling Structures*) to check that all existing fill and/or other unsuitable material have been removed. Where subexcavation of fill is required along Retaining Wall No. 24-887/W, the sub-excavated area could be backfilled with granular material meeting OPSS.PROV 1010 (*Aggregates*) Granular ‘A’ or Granular ‘B’ Type II that is placed and compacted in accordance with OPSS.PROV 501 (*Compacting*), or the thickness of the footing increased to the full excavation depth. If replacement of unsuitable materials with engineered fill is being considered, the area to be subexcavated should be defined by a line extending from the top of the engineering fill pad outward and downward at 1H:1V. The top of the granular engineered fill should extend at least 1 m beyond the plan limits of the footing. Temporary shoring would be required and is discussed in Section 6.10.

6.7.2 Factored Geotechnical Resistance

Strip footings constructed about 3 m to 5 m wide (based on the design drawings) on the properly prepared subgrade, at or below the design elevations given in the Section 6.7.1, should be designed based on the factored ultimate geotechnical resistances and the factored serviceability geotechnical resistance (for 25 mm of settlement) given below.

Retaining Wall Site	Approximate Station / Reference Boreholes	Founding Stratum	Footing Width (m)	Factored Ultimate Geotechnical Resistance (kPa)	Factored Serviceability Geotechnical Resistance (kPa) (for 25 mm of Settlement)
Retaining Wall No. 24-887/W	13+500 to 13+575 RW2-1 and RW2-2	Compact to dense sand to gravelly sand	3.0	525	300
	13+575 to 13+650 RW2-3 and STM-10	Hard sandy clayey silt till	3.0	425*	300*
	13+650 to 13+725 RW2-4 and RW2-6	Very stiff clayey silt to sandy clayey silt till	3.0	325	300
	13+725 to 13+770 RW2-7	Very stiff to hard sandy gravelly clayey silt till	4.5	550	450
	13+770 to 13+810 RW2-5	Very stiff clayey silt till*	5	525*	300*
Retaining Wall No. 24-888/W	13+749 to 13+775 RW3-1	Very stiff to hard clayey silt till	3.0	700	350
	13+775 to 13+800 RW3-2	Hard clayey silt till	3.0	700	550
	13+800 to 13+859 RW3-3	Shale Bedrock	3.0	975	750

* If the strip footings are founded on compacted Granular 'A' or Granular 'B' Type II at higher elevations, a factored geotechnical ultimate resistance of 750 kPa and a factored serviceability geotechnical resistance for 25 mm of settlement of 350 kPa could be employed for the design of the retaining wall foundations, assuming the granular pad is at least 2 m thick.

The ULS resistance and settlement are dependent on the footing size (assumed to be at least 3 m wide), configuration and applied loads; the geotechnical resistances should, therefore, be reviewed if the selected footing width or founding elevation differs from those given above.

The geotechnical resistances provided above are given for loads applied perpendicular to the surface of the footing. Where the load is not applied perpendicular to the surface of the footing, inclination of the load should be taken into account in accordance with Sections 6.10.4 of the CHBDC (2014).

6.7.3 Resistance to Lateral Loads

Resistance to lateral forces / sliding between the concrete footings and the subgrade should be calculated in accordance with Section 6.10.5 of the CHBDC (2014). For cast-in-place concrete footings constructed on a concrete working slab that is cast on top of the dense sand to gravelly sand, very stiff to hard till, hard residual soil, shale bedrock or on the Granular ‘A’ or Granular ‘B’ Type II engineered fill, the coefficient of friction, tan δ or tan ϕ’, can be taken as follows:

- Cast-in-place footing to concrete working slab: tan δ = 0.7
- Cast-in-place concrete footing or working slab to native deposits: tan ϕ’ = 0.5
- Cast-in-place concrete footing or working slab to shale bedrock: tan ϕ’ = 0.56
- Cast-in-place concrete footing or working slab to granular pad: tan ϕ’ = 0.56

6.7.4 Global Stability

The static global stability analyses for the proposed concrete retaining walls supported on shallow foundations were completed using the parameters outlined in Section 6.5.3. A maximum retained soil height of 4.5 m was assumed in the analyses. Groundwater levels were inferred from the highest water levels shown on the borehole records.

The stability analysis results indicate that the proposed concrete retaining wall founded on shallow foundations at Retaining Wall Nos. 24-887/W and 24-88/W will have a factor of safety greater than 1.5 against global instability. An example of the static global stability results is provided on Figure 2.

6.8 Concrete Toe Wall

Based on the 90% design drawings provided to us by AECOM on January 17, 2018, a Type III concrete toe wall is proposed at Retaining Wall No. 24-887/W between Stations 13+500 and 13+600. The concrete toe wall is proposed to be founded between Elevations 106.3 m and 105.7 m, from west to east, and will have a base width ranging from 1.0 m to 1.4 m. The concrete toe wall should be designed and constructed in accordance with OPSD 3120.100 (*Concrete Toe Wall*), where it is noted that the walls should be founded on undisturbed soil having a bearing capacity at ultimate limits states of 300 kPa for a Type III wall. Based on the proposed founding elevations show on the 90% Design Drawings as noted above, the minimum bearing capacity will not be achieved. It is recommended that the toe wall either be founded on the dense sand to gravelly sand deposit at Elevation 105.5 m, or be founded on a minimum 1 m thick granular pad composed of compacted Granular ‘A’ or Granular ‘B’ Type II that is placed at a founding level at or below Elevation 105.5m and compacted in accordance with OPSS.PROV 501 (*Compacting*). If replacement of unsuitable materials with engineered fill is being considered, the area to be subexcavated should be defined by a line extending from the top of the engineering fill pad outward and downward at 1H:1V. The engineering fill pad should extend a minimum of 500 mm beyond the footing boundary.

6.9 Lateral Earth Pressures for Design

The lateral earth pressures acting on the retaining walls will depend on the type and method of placement of the backfill materials, the nature of the soils behind the backfill, the magnitude of the surcharge including construction loadings, the freedom of lateral movement of the structure, and the drainage conditions behind the walls.

The following recommendations are made concerning the design of the walls. These design recommendations and parameters assume level backfill and ground surface behind the walls. Where there is sloping ground behind the walls, the coefficient of lateral earth pressure must be adjusted to account for the slope.

- If the walls are to be constructed by temporarily excavating behind the wall, select, free draining granular fill meeting the specifications of OPSS.PROV 1010 (*Aggregates*) Granular ‘A’ or Granular ‘B’ Type II should be used as backfill behind the walls. Compaction (including type of equipment, target densities, etc.) should be carried out in accordance with OPSS.PROV 501 (*Compacting*).
- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the walls, in accordance with CHBDC (2014) Section 6.12.3 and Figure 6.6. Care must be taken during the compaction operation not to overstress the wall. Heavy construction equipment should be maintained at a distance of at least 1 m away from the walls while the backfill soils are being placed. Hand-operated compaction equipment should be used to compact the backfill soils within a 1 m wide zone adjacent to the walls. Other surcharge loadings should be accounted for in the design, as required.
- For unrestrained walls, the granular backfill should be placed within the wedge-shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the base of the walls (Figure C6.20(b) of the *Commentary* to the CHBDC 2014).

6.9.1 Static Lateral Earth Pressures for Design

The following guidelines and recommendations are provided regarding the lateral earth pressures for static (i.e., not earthquake) loading conditions. These lateral earth pressures assume that the ground above the wall will be flat, not sloping. If the inclination of the slope above the wall changes then new lateral earth pressures will need to be calculated.

- The earth pressures acting on the wall will depend on the material behind the wall (i.e., whether there is a zone of granular backfill as described above, or whether the wall is constructed top-down with the existing soils remaining behind the wall. The following parameters (unfactored) may be used:

Material	Granular A	Granular B Type II	Existing Native Materials
Soil Unit Weight:	22 kN/m^3	21 kN/m^3	20 kN/m3
Coefficients of static lateral earth pressure:			
Active, K_a	0.27	0.27	0.33
At rest, K_o	0.43	0.43	0.50

- If the retaining wall structures do not allow lateral yielding, at-rest earth pressures should be assumed for the foundation design. If the retaining wall structure allows for lateral yielding, active earth pressures should be used in the foundation design. The movement required to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure for design, should be calculated in accordance with Section C6.12.1 and Table C6.6 of the Commentary to the CHBDC (2014).
- Where space is restricted and the walls are constructed in a top-down fashion, with a thinner or absent zone of granular backfill behind the wall, it is recommended that drainage measures (e.g., pre-fabricated sheets) be incorporated on the back of the walls, before or concurrent with the panel installation, to promote drainage and minimize the risk of frost action during freezing temperatures. The wall system and facing should also incorporate subdrains and weep holes at intervals through the wall face.

6.9.2 Seismic Lateral Earth Pressures for Design

Seismic (earthquake) loading must also be taken into account in the design of retaining walls in accordance with Section 4.6.5 of the CHBDC (2014). In this regard, the following should be included in the assessment of lateral earth pressures:

- Seismic loading will result in increased lateral earth pressures acting on the retaining walls. The walls should be designed to withstand the combined lateral loading for the appropriate static pressure conditions given above, plus the earthquake-induced dynamic earth pressure.
- In accordance with Sections 4.6.5 and C.4.6.5 of the CHBDC (2014) and its Commentary, for structures which allow lateral yielding, the horizontal seismic coefficient, k_h, used in the calculation of the seismic active pressure coefficient, is taken as 0.5 times the site-specific PGA. For structures that do not allow lateral yielding, k_h is taken as equal to the site-specific PGA. For both cases the value of the vertical seismic coefficient k_v is taken as zero.
- The following seismic active pressure coefficients (K_{AE}) may be used in design; these coefficients reflect the maximum K_{AE} obtained for each of the earthquake design periods and backfill conditions. It should be noted that these seismic earth pressure coefficients assume that the back of the wall is vertical and the ground surface behind the wall is level. Where sloping backfill is present above the top of the wall, the lateral earth pressures under seismic loading conditions should be calculated by treating the weight of the backfill located above the top of the wall as a surcharge.

	Design Earthquake	**Site PGA**	**Seismic Active Pressure Coefficients, K_{AE}**		
			Granular A	**Granular B Type II**	**Earth Fill**
Yielding Wall	**475-Yr**	0.042g	0.26	0.26	0.31
	975-Yr	0.075g	0.27	0.27	0.32
	2,475 Yr	0.144g	0.29	0.29	0.35
Non-Yielding Wall	**475-Yr**	0.042g	0.27	0.27	0.33
	975-Yr	0.075g	0.29	0.29	0.35
	2,475 Yr	0.144g	0.34	0.34	0.40

- The K_{AE} value for a yielding wall is applicable provided that the wall can move up to $250k_h$ mm, where k_h is the site specific PGA as given in the table above. This corresponds to displacements of 10 mm, 19 mm, and 36 mm for the 475-year, 975-year, and 2,475-year design earthquakes at this site.
- The earthquake-induced dynamic pressure distribution, which is to be added to the static earth pressure distribution, is a linear distribution with maximum pressure at the top of the wall and minimum pressure at its toe (i.e. an inverted triangular pressure distribution). The total pressure distribution (static plus seismic) may be determined per Section C4.6.5 of the *Commentary to CHBDC* (2014).

6.10 Tunnelled Sanitary Sewer Beneath Retaining Wall 24-887/W at Station 13+625

It is understood that prior to the construction of the retaining wall at approximately Station 13+625, a sanitary sewer will be installed using trenchless methods. The sewer will extend from a proposed entry shaft located about 15 m north of the proposed retaining wall, and will cross obliquely under the wall to exit at the proposed shaft on Boxwood Way on the south side of the QEW. The retaining wall between Station 13+600 to 13+650 will then be constructed; it is proposed to be a concrete cantilever wall supported on a 3.0 m wide strip footing founded at approximately Elevation 104.1 m.

Based on the updated drawings for the sanitary sewer as provided by AECOM on May 17, 2018, the obvert of the tunnel bore at this location is proposed at Elevation 103.6 m; therefore, the distance between the proposed tunnel obvert and the proposed underside of the retaining wall footing is about 0.5 m. The soil conditions at the tunnel face are anticipated to consist of gravelly sand to sand and gravel at the obvert and clayey silt till in the remainder of the tunnel face. Considering the potential for mixed face conditions, there is potential for deviation in the vertical alignment during tunneling and, in addition, there is also the potential for loss of ground in the granular soils above the obvert. One of the following mitigation options must be incorporated to address these potential risks and minimize negative interactions between the retaining wall foundation and the previously tunneled sewer:

1. The soil between the proposed underside of the footing and the obvert of the primary liner must be sub-excavated (after tunnelling and prior to construction of the footing) and replaced with either unshrinkable fill or engineered granular fill, as follows:
    - 0.4 MPa "unshrinkable" fill (controlled low-strength mix of sand and cement): This will assist with reducing the potential for differential settlement of the retaining wall foundation to less than about 10 mm, and redistribute stresses to the ground surrounding the pipe rather than the pipe itself, although some small deflection of the pipe would be necessary for stress redistribution.
    - Granular material meeting OPSS.PROV 1010 (*Aggregates*) Granular 'A' or Granular 'B' Type II: This granular fill must be placed and compacted in accordance with OPSS.PROV 501 (*Compacting*). The area to be subexcavated should be defined by a line extending from the top of the engineered fill pad outward and downward at 1H:1V.
    - For either option, the top of the unshrinkable fill or granular engineered fill should extend at least 1 m beyond the plan limits of the footing in all directions, then downward and outward at 1H:1V. A Notice to Contractor addressing this requirement is included in Appendix D.

2. The retaining wall structure can be structurally designed to span the pipe so that if relatively small pipe deformations occur or if the ground has been adversely disturbed by the microtunnelling, foundation stresses will be transferred away from the pipe. In this case, the foundation engineer recommends that wall control joints be located approximately equidistant from the pipe centreline and specifically not over the pipe.

6.11 Corrosion Assessment and Protection

Soil corrosivity may affect the concrete foundations and reinforced steel and other concrete elements buried in the soil. The long-term performance and durability of the foundations are directly related to their respective corrosion resistance. Generally, the corrosivity potential to a structure depends on the soil resistivity / electrical conductivity, hydrogen ion concentration, and salts (chloride and sulphate) concentrations. The analytical results for the samples submitted for testing are summarized in Sections 4.2.1.11 and 4.2.2.6 and the analytical laboratory test reports are included in Appendix C.

6.11.1 Potential for Sulphate Attack

The analytical test results were compared to CSA Standard, CAN/CSA-A23.1-14 Table 3 *("Additional requirements for concrete subjected to sulphate attack")* for potential sulphate attack on concrete. The sulphate concentrations measured in the samples are less than 0.1 per cent, with the exception of one sample with was under 0.2 per cent, which is an exposure class of “Moderate”.

6.11.2 Potential for Corrosion

The test results indicate a pH of between 8.0 and 8.2 and a resistivity of between about 670 ohm-cm and about 8,400 ohm-cm, but generally less than 2,000 ohm-cm. According to the Gravity Pipe Design Guidelines (MTO, 2014), the pH is not considered detrimental to concrete durability. However, the resistivity of 2,000 ohm-cm indicates that the soil corrosiveness is between “Moderate” (4,500 ohm-cm < R < 2,000 ohm-cm) and “Severe” (R < 2,000 ohm-cm), as per Table 3.2 of the Gravity Pipe Design Guidelines (MTO, 2014), and some level of corrosion protection should be applied to the foundation element / materials. Further, given that the retaining wall foundations are located adjacent to the roadway shoulder and will be exposed to de-icing salt, consideration should be given to selection of a “C” type exposure class as defined by CSA A23.1 Table 1.

It is ultimately up to the structural designer to determine the appropriate exposure class and to ensure that all aspects of CSA A23.1 Section 4.1.1 “Durability Requirements” are followed.

6.12 Construction Considerations

The following subsections identify pertinent construction related issues that should be considered at this stage of the design as they may impact the design. Where applicable, Non-Standard Special Provisions (NSSP) should be included in the Contract Documents.

6.12.1 Excavation and Temporary Roadway Protection

The foundation excavations for strip footings at Retaining Wall Nos. 24-887/W and 24-888/W will extend through topsoil and fill materials of varying consistency and level of compactness. Open-cut excavations into these materials should be carried out in accordance with the guidelines outlined in the latest edition of the Occupational Health and Safety Act (OHSA) and Regulation 213 for Construction Activities. The existing fill materials are classified as Type 3 soil and the native soils are is classified as Type 2 soil, according to the OHSA. Temporary excavations (i.e., those which are open for a relatively short time period) should be made with side slopes no steeper than 1H:1V through the Type 3 soils and to within 1.2 m of the bottom of the excavation in Type 2 soils only.

6.12.2 Temporary Excavation Support and Vibration Monitoring

Temporary excavation support is likely required to facilitate the removal of the existing retaining wall foundations and construction of the new retaining walls in order to maintain traffic on the QEW and to reduce the extent of sub-excavation required for the project. The temporary excavation support systems should be designed and constructed in accordance with OPSS.PROV 539 *(Temporary Protection Systems).* The lateral movement of the temporary shoring system should meet Performance Level 2 as specified in OPSS.PROV 539, provided that the existing structure, as well as any adjacent utilities, can tolerate this magnitude of deformation.

It is considered that either a driven, interlocking sheetpile system or a soldier pile and timber lagging system would be suitable for the temporary excavation support at this site, based on the inferred subsurface soil conditions and groundwater conditions. An interlocking sheetpile system would contribute to both ground and, where applicable, groundwater control should seepage from non-cohesive zones or interlayers/lenses within the cohesive deposits to occur. For a soldier pile and lagging system, it would be necessary to control seepage and due to the presence of the high groundwater level at this site (approximately Elevation 103.9 m) and also to include measures to mitigate loss of soil particles through the lagging boards in the event that water-bearing non-cohesive soils are encountered.

Existing residential properties are located in close proximity to the north of Retaining Wall No. 24-887/W and south of Retaining Wall No. 24-888/W. The use of vibratory equipment for protection system installation should be minimized where possible, and vibration monitoring of residences within a zone of influence of 200 m should be considered. Vibration levels less than a maximum peak particle velocity (PPV) of 50 mm/s are generally considered applicable for buildings in good condition. An NSSP for vibration monitoring all aspects of the construction for Contact 1 is included in Appendix D.

If deep excavation is required in relation to the surrounding ground surface, specifically with respect to the property on the north side of Retaining Wall No. 24-887/W, a more elaborate and robust excavation support system will be required. Lateral support to the sheetpiles or soldier piles could be provided in the form of rakers, temporary anchors or cross-bracing.

Consideration could be given to either partial or full removal of the protection system upon completion of construction or each stage of construction (as required). Where possible, full removal of the protection system should be considered to mitigate potential impediments to future rehabilitation/reconstruction work at the underpass sites, or to the road structure above. An NSSP is included in Appendix D which addresses the removal or cut-off of the protection system.

The selection and design of the protection system will be the responsibility of the Contractor.

6.12.3 Groundwater and Surface Water Control

The groundwater level at the site was generally at about Elevation 104 m encountered at about Elevation 96.1 m, at about the surface of the bedrock, with one location encountering it within the clayey silt till. Excavations for construction of the retaining walls may extend below the water level; however, it is expected that water inflow from granular zones of fill or present within the native material, or through the till and shale bedrock, can be handled by pumping from well filtered sumps located outside the foundation footprint.

Surface water seepage into the excavations should be expected and will be heavier during periods of sustained precipitation and all surface water should be directed away from the excavations.

6.12.4 Removal of Existing Retaining Wall Foundations

The widening of the QEW will result in the complete replacement and realignment of Retaining Wall No. 24-887/W and the partial replacement of the eastern section of Retaining Wall No. 24-888/W, as described in Sections 6.1.1 and 6.1.2. The replacement of the retaining walls will require removal of the existing shallow and deep foundations.

The existing shallow foundations can be removed during the widening of the QEW and during the construction of the new shallow foundations. Temporary excavation support will be required, as discussed in Section 6.12.2.

It is recommended that the existing deep foundations be cut off at a depth not less than 1.5 m, below final grade (frost depth) to mitigate the potential for frost jacking (adhesion) due to frost penetration. Extraction of the cast-in-place piles is not required at this site.

6.12.5 Bedrock Excavation and/or Socket Formation

The upper portion of the shale bedrock as encountered in some of the boreholes is weathered. The shale in this area is generally considered to be very weak to weak with unconfined compressive strengths in the range of 5 MPa to 16 MPa; however UCS values of up to 32 MPa were obtained on the unconfined compressive strengths test specimen and medium strong to strong limestone layers are present in the shale. Where excavation into bedrock is required, such as for strip footings at the east end of Retaining Wall No. 24-888/W, it is expected that hoe-ramming techniques may be required. It is recommended that an NSSP be included in the Contract Documents to warn the contractor that the bedrock at the site is very weak to weak. An NSSP is provided in Appendix D.

Alternatively, if caissons are adopted and rock sockets are required, or if rock sockets are required for toe support for soldier pile and lagging systems, it is recommended that an NSSP be included in the Contract Documents to warn the Contractor that the shale bedrock is very weak to weak and contains medium strong to strong limestone interbeds, which will require socket formation using coring or churn drilling to advance the hole. An NSSP is provided in Appendix D.

6.12.6 Subgrade Protection

The native soils and shale bedrock that will be exposed at the foundation subgrade level will be susceptible to disturbance from construction traffic, groundwater infiltration and/or ponded water. To limit this degradation, it is recommended that a concrete working slab be placed on the subgrade within four hours after preparation, inspection and approval of the footing subgrade. This requirement can be addressed with a note on the General Arrangement drawing and/or with an NSSP. An NSSP is included in Appendix D.

6.12.7 Obstructions During Pile Driving / Caisson Installation

Although not encountered in the boreholes, the existing fill materials and native soil deposits may contain cobbles and boulders, which may affect the installation of steel H-piles/tube piles or caissons. It is recommended that driving shoes be used on all steel H-piles or tube piles to facilitate driving into / through the overburden soils and seating the piles on shale bedrock. In addition it is recommended that an NSSP be included in the Contract

Documents to warn the Contractor of the possible presence of cobbles and/or boulders within the overburden soils and an example NSSP is presented in Appendix D.

6.12.8 Enbridge Pipeline

It is understood that an NPS 8 Enbridge Pipeline is to be installed by the horizontal directional drill (HDD) method prior to the construction and the widening of the QEW at about Station 13+648. At this location, a concrete cantilever retaining wall is to be constructed / supported on a strip footing with a width of 3 m, with the underside of the footing at Elevation 103.7 m. Based on the design drawings provided by AECOM, the proposed pipeline, which will be approximately perpendicular to the footing, will be installed at about Elevation 101.4 m. The obvert of the NPS 8 pipeline is estimated to be at about Elevation 101.6 m. Therefore, there is an approximately 2.1 m vertical separation between the underside of the proposed footing and the obvert of the proposed pipeline.

The increase in stress due to the retaining wall at the elevation of the obvert of the proposed pipeline is estimated to be about 35 kPa. The pipeline should be designed to accommodate this additional vertical stress.

It is noted that there will be a horizontal separation of about 3 m between the alignment of the pipeline and the start of the proposed contiguous caisson wall (north of the QEW) at about Station 13+651. The contractor who will be installing the caissons should be made aware of the exact location and depth of the as-installed pipe. Caissons in that area will have to be installed with a temporary liner to ensure there is no loss of soil, and concrete should be placed in the caisson using tremie methods as the liner is removed, always ensuring to maintain the surface of the fresh concrete inside the liner (i.e. higher than the bottom of the liner) so that there is no loss of soil.

7.0 CLOSURE

This report was prepared by Ms. Nikol Kochmanová, P.Eng, a geotechnical engineer with Golder, with technical input from Ms. Lisa Coyne, P.Eng., and from Dr. Storer Boone, P.Eng., who provided technical input related to the interaction between the retaining wall and the tunnelled sewer and gas main. Mr. Jorge M.A. Costa, P.Eng., a MTO Foundations Designated Contact and Senior Consultant with Golder, conducted a technical and quality control review of the report.

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Ontario Provincial Standard Specifications (OPSS)

OPSS.PROV 501	Construction Specification for Compacting
OPSS.PROV 539	Construction Specification for Temporary Protection Systems
OPSS 902	Construction Specification for Excavating and Backfilling Structures
OPSS.PROV 903	Construction Specification for Deep Foundations
OPSS 942	Construction Specification for Prestressed Soil and Rock Anchors
OPSS.PROV 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material

Ontario Provincial Standard Drawings (OPSD)

OPSD 3000.100	Foundation, Piles, Steel H-Pile Driving Shoe
OPSD 3001.100	Foundation, Piles, Steel Tube Pile Driving Shoe
OPSD 3090.101	Foundation, Frost Penetration Depths for Southern Ontario
OPSD 3120.100	Walls, Retaining, Concrete Toe Wall

ASTM International

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

Ontario Water Resources Act

Ontario Regulation 903	Wells (as amended)

Ontario Occupational Health and Safety Act

Ontario Regulation 213	Construction Projects (as amended)

LIST OF SYMBOLS

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

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

II.	**STRESS AND STRAIN**
γ	shear strain
Δ	change in, e.g. in stress: $\Delta\sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
υ	Poisson’s ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	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 $= \sigma'_p / \sigma'_{vo}$

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

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

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

LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

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

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

The number of blows by a 63.5 kg. (140 lb.) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) drive open sampler for a distance of 300 mm (12 in.)

Dynamic Cone Penetration Resistance; N_d:

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

PH: Sampler advanced by hydraulic pressure
PM: Sampler advanced by manual pressure
WH: Sampler advanced by static weight of hammer
WR: Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)

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

III. SOIL DESCRIPTION

(a) Non-Cohesive (Cohesionless) Soils

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

(b) Cohesive Soils

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

IV. SOIL TESTS

w	water content
w_p	plastic limit
w_l	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test[1]
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement[1]
D_R	relative density (specific gravity, G_s)
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO_4	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (LV-laboratory vane test)
γ	unit weight

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

V. MINOR SOIL CONSTITUENTS

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

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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 90º angle is horizontal.

Description and Notes

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

Abbreviations

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

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TABLE 1 – COMPARISON OF RETAINING WALL AND FOUNDATION ALTERNATIVES RETAINING WALL NOS. 24-887/W AND 24-888/W

Foundation Option	Feasibility		Advantages	Disadvantages	Relative Costs	Risks/Consequences
	Retaining Wall 24-887/W	Retaining Wall 24-888/W				
Soldier Pile and Concrete Panel Wall or similar (e.g. secant caisson wall)	Feasible and preferred alternative due to the space restrictions • If tie-backs are required, sufficient right-of-way will be required	Feasible – requires temporary protection systems for removal of the existing retaining wall.	• Most advantageous in "top-down" construction applications, such as the cut section at Retaining Wall 24-887/W • Retaining Wall No. 24-887/W - minimizes excavation and requirement for temporary excavation support	• Easement for soil anchors may be required at Retaining Wall No. 24-887/W, depending on distance from wall to property limits • Likely more time-consuming to install than other wall types due to steps involved (pre-augering for socket holes, placing soldier piles, placing backfill in lifts, installing concrete panels, installing, pre-stressing and testing tie-backs)	• Comparable costs to concrete retaining wall, but higher than RSS wall • Cost of temporary protection system combined with RSS wall is comparable	• Need adequate right-of-way for tie back anchors, if such are required • Least demanding on right-of-way space if tie-backs not required
Concrete Retaining Wall on Deep Foundations	Feasible – requires pile cap below frost penetration depth	Feasible – requires pile cap below frost penetration depth and requires temporary protection systems for removal of the existing retaining wall.	• Potentially reduced excavation, protection system and backfill requirements compared to RSS wall	• Temporary/permanent liners may be required to allow for construction of caissons • If refusal (100-blow) stratum or obstructions are encountered, can get piles to hang-up, requiring pre-drilling • If tie-backs are required, significant length required to anchor into competent till soils and design will need to account for settlement of embankment	• Higher cost relative to RSS wall	• Need adequate right-of-way for tie back anchors, if such are required • Least demanding on right-of-way space if tie-backs not required

Foundation Option	Feasibility		Advantages	Disadvantages	Relative Costs	Risks/Consequences
	Retaining Wall 24-887/W	Retaining Wall 24-888/W				
Concrete Cantilever Wall on Shallow Foundations	Feasible provided sufficient space is available during construction and/or for temporary shoring if used. Construction must not interfere with private property.	Feasible provided sufficient space is available during construction and requires temporary protection systems for removal of the existing retaining wall.	• Conventional excavation and construction techniques • Suitable founding stratum below depth of frost penetration at Retaining Wall Nos. 24-887/W and 24-888/W	• Less tolerable to post construction settlements • Temporary excavation support will be required • A temporary construction easement may be required • Footings must be founded below depth of frost penetration	• Higher cost relative to RSS wall	• More susceptible for visible distortion if differential settlement occurs
RSS Walls	Not feasible due to space restrictions.	Not feasible due to space restrictions.	• More tolerable to post construction settlements • Lowest cost alternative where feasible	• Potentially larger amount of excavation required to install reinforcing strips; temporary protection systems required	• Lower cost than concrete retaining wall or walls supported on deep foundations	• Requires wider right-of-way footprint • Can better accommodate some degree of differential settlement
Reinforced Earth Slope Embankment	Not feasible due to space restrictions.	Not feasible due to space restrictions.	• Relative ease of construction but proprietary product required • Vegetated surfaces could be used to improve aesthetics	• Proprietary product design • Special treatment of reinforced earth slope surfaces required to allow vegetation to grow and minimize erosion	• Lower cost than RSS wall	• Requires wider right-of-way footprint • Can accommodate some degree of differential settlement but susceptible to surface erosion

MINISTRY OF TRANSPORTATION, ONTARIO

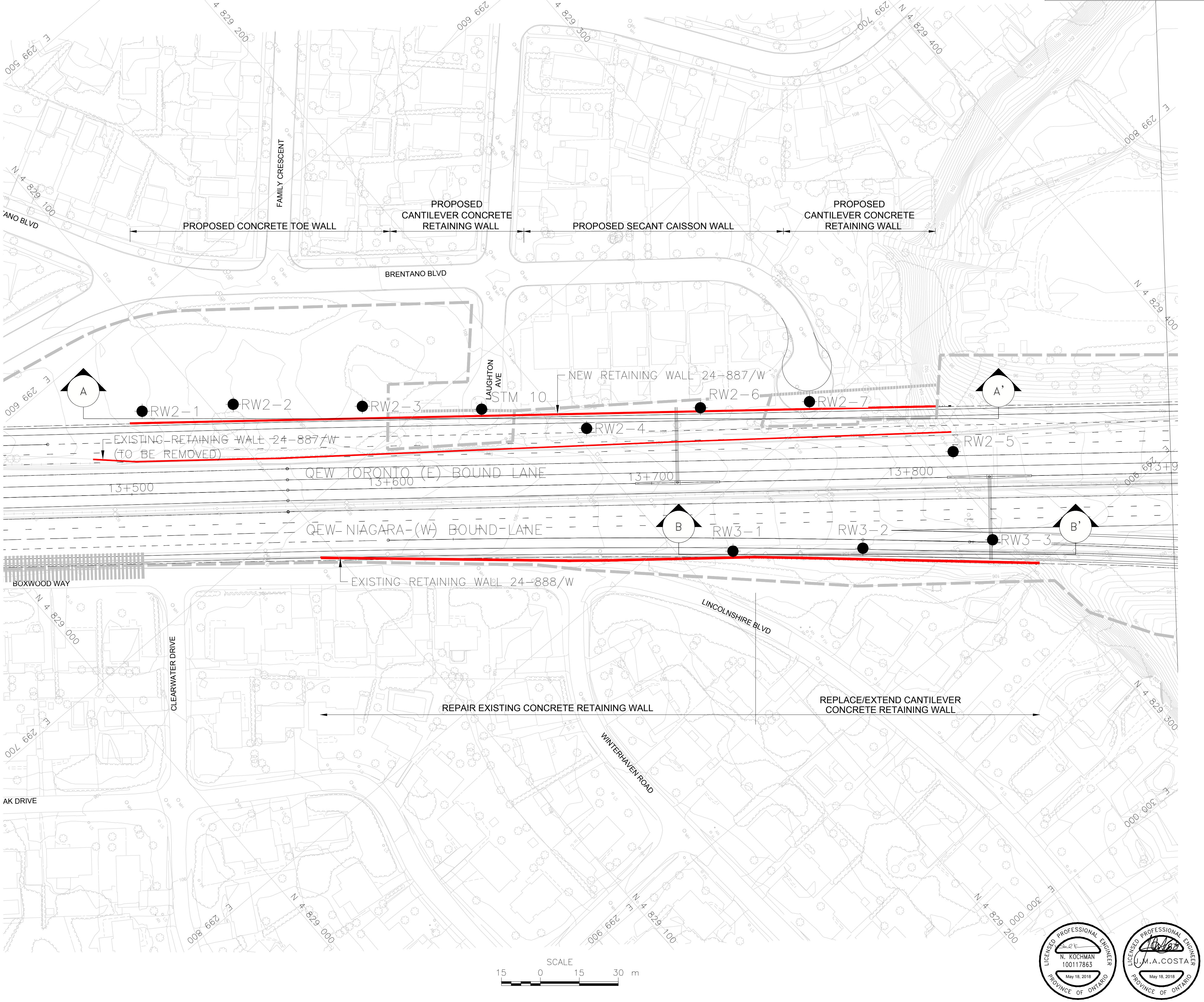
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

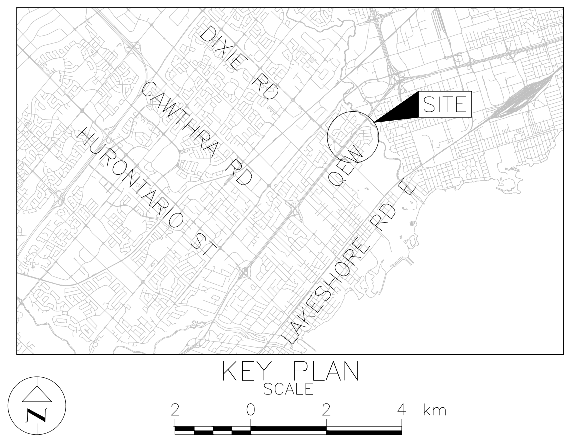
CONT No. 2018–2001
GWP No. 2432–13–00

SHEET

QEW–CAWTHRA TO EAST MALL WIDENING
RETAINING WALL NOS. 24–887/W AND 24–888/W REPLACEMENT
BOREHOLE LOCATIONS

Golder Associates





KEY PLAN
SCALE
2 0 2 4 km

LEGEND

● Borehole – Current Investigation

BOREHOLE CO–ORDINATES (MTM NAD83 ZONE 10)			
No.	ELEVATION	NORTHING	EASTING
RW2–1	107.9	4829078.9	299630.9
RW2–2	108.0	4829107.3	299651.6
RW2–3	107.6	4829144.6	299684.5
RW2–4	108.0	4829204.7	299747.0
RW2–5	102.1	4829306.2	299845.3
RW2–6	108.3	4829243.2	299769.4
RW2–7	107.7	4829276.6	299794.8
RW3–1	104.3	4829216.9	299819.4
RW3–2	103.0	4829255.6	299851.0
RW3–3	102.0	4829295.7	299881.0
STM 10	107.6	4829178.8	299715.1

NOTES

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

REFERENCE

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.
Base plans provided in digital format by AECOM, drawing file nos. QEW_DixieIC_base.dwg and QEW_DixieIC_plan.dwg, dated July 20, 2016, received Dec. 06, 2016.
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.
Existing ground contours provided in digital format by AECOM, drawing file no. QEW_DixieIC_Contours3D.dwg, received Nov. 08, 2016, contour interval 0.5 m.
Key plan base data – MNRF LIO, obtained 2015.

N. KOCHMAN
100117863
May 18, 2018

J.M.A. COSTA
May 18, 2018

NO.	DATE	BY	REVISION

Geocres No. 30M11–275

HWY. QEW		PROJECT NO. 1530382	DIST. CENTRAL
SUBM'D. DB	CHKD. SMM	DATE: 05/18/2018	SITE:
DRAWN: DD	CHKD. NK	APPD. JMAC	DWG. 1

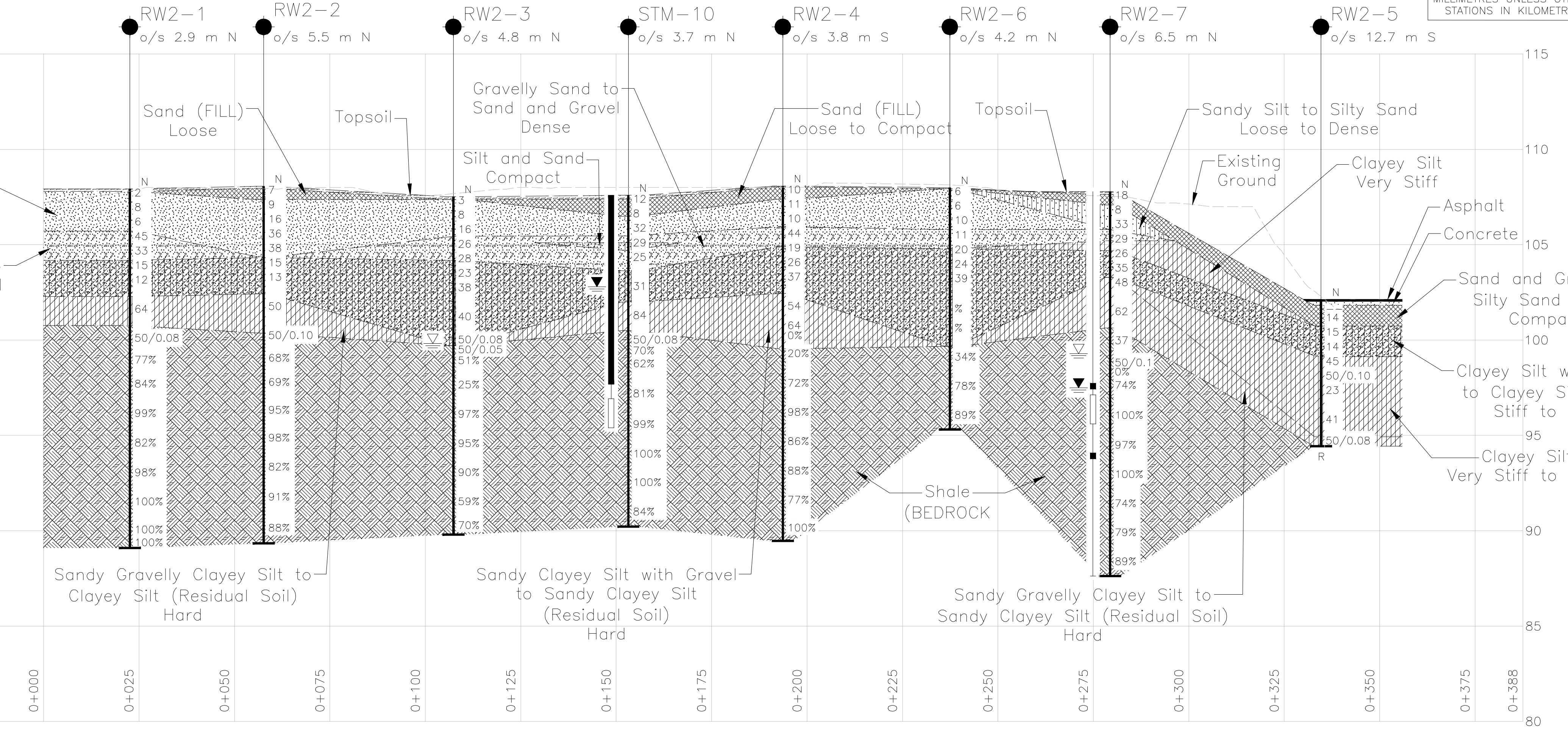
PLOT DATE: May 22, 2018
FILENAME: S:\Clients\MTO\QEW–Dixie\99_PROJ\1530382\40_PROD\0020_Retaining_Walls_24–887W_&_24–888W\1530382–0020–BG–0001.dwg

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

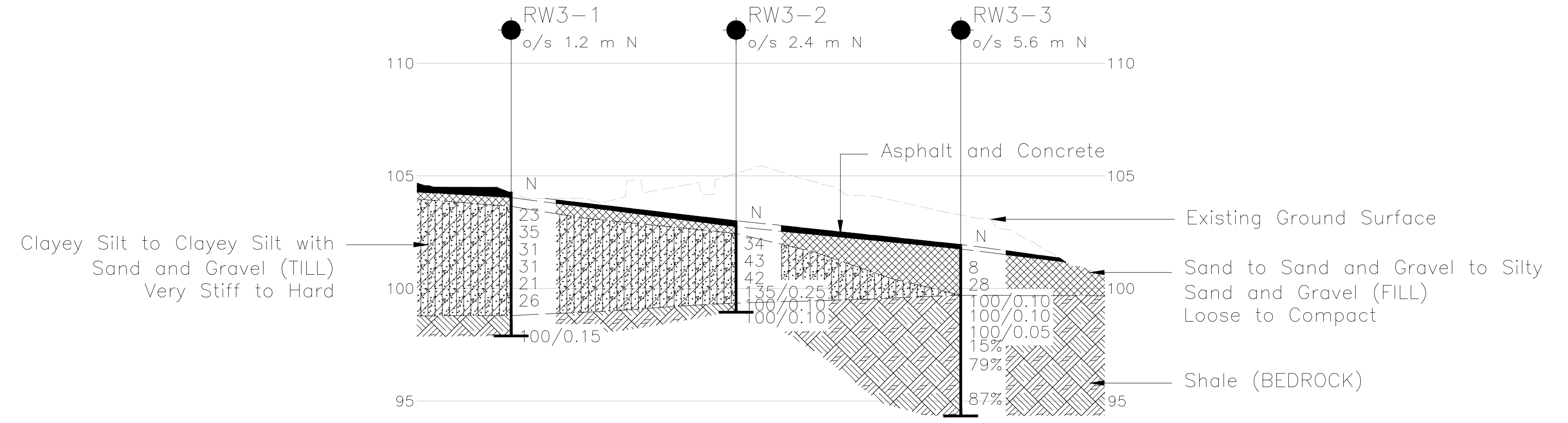
CONT No. 2018-2001
GWP No. 2432-13-00

QEW-CAWTHRA TO EAST MALL WIDENING
RETAINING WALL NOS. 24-887/W AND 24-888/W REPLACEMENT
SOIL STRATA

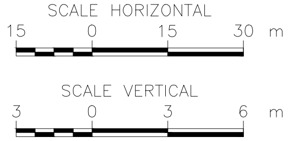
SHEET

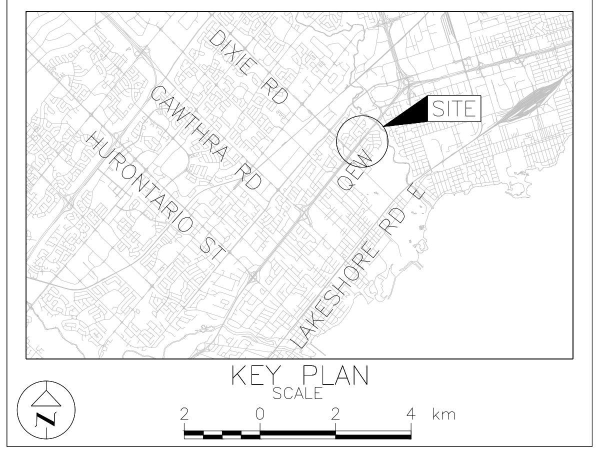


PROFILE A-A'
RETAINING WALL 24-887/W



PROFILE B-B'
RETAINING WALL 24-888/W





KEY PLAN

LEGEND

●	Borehole – Current Investigation
▪	Seal
▯	Piezometer
N 16	Standard Penetration Test Value Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
100%	Rock Quality Designation (RQD)
R	Refusal
▼	WL in piezometer, measured on MAR 28 and APR 2, 2018
▽	WL upon completion of drilling

BOREHOLE CO-ORDINATES (MTM NAD83 ZONE 10)

No.	ELEVATION	NORTHING	EASTING
RW2-1	107.9	4829078.9	299630.9
RW2-2	108.0	4829107.3	299651.6
RW2-3	107.6	4829144.6	299684.5
RW2-4	108.0	4829204.7	299747.0
RW2-5	102.1	4829306.2	299845.3
RW2-6	108.3	4829243.2	299769.4
RW2-7	107.7	4829276.6	299794.8
RW3-1	104.3	4829216.9	299819.4
RW3-2	103.0	4829255.6	299851.0
RW3-3	102.0	4829295.7	299881.0
STM 10	107.6	4829178.8	299715.1

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.

REFERENCE

Existing ground contours provided in digital format by AECOM, drawing file no. QEW_DixieIC_Contours3D.dwg, received Nov. 08, 2016, contour interval 0.5 m.
Key plan base data – MNRF LIO, obtained 2015.

N. KOCHMAN
100117863
May 18, 2018

J.M.A. COSTA
May 18, 2018

NO.	DATE	BY	REVISION

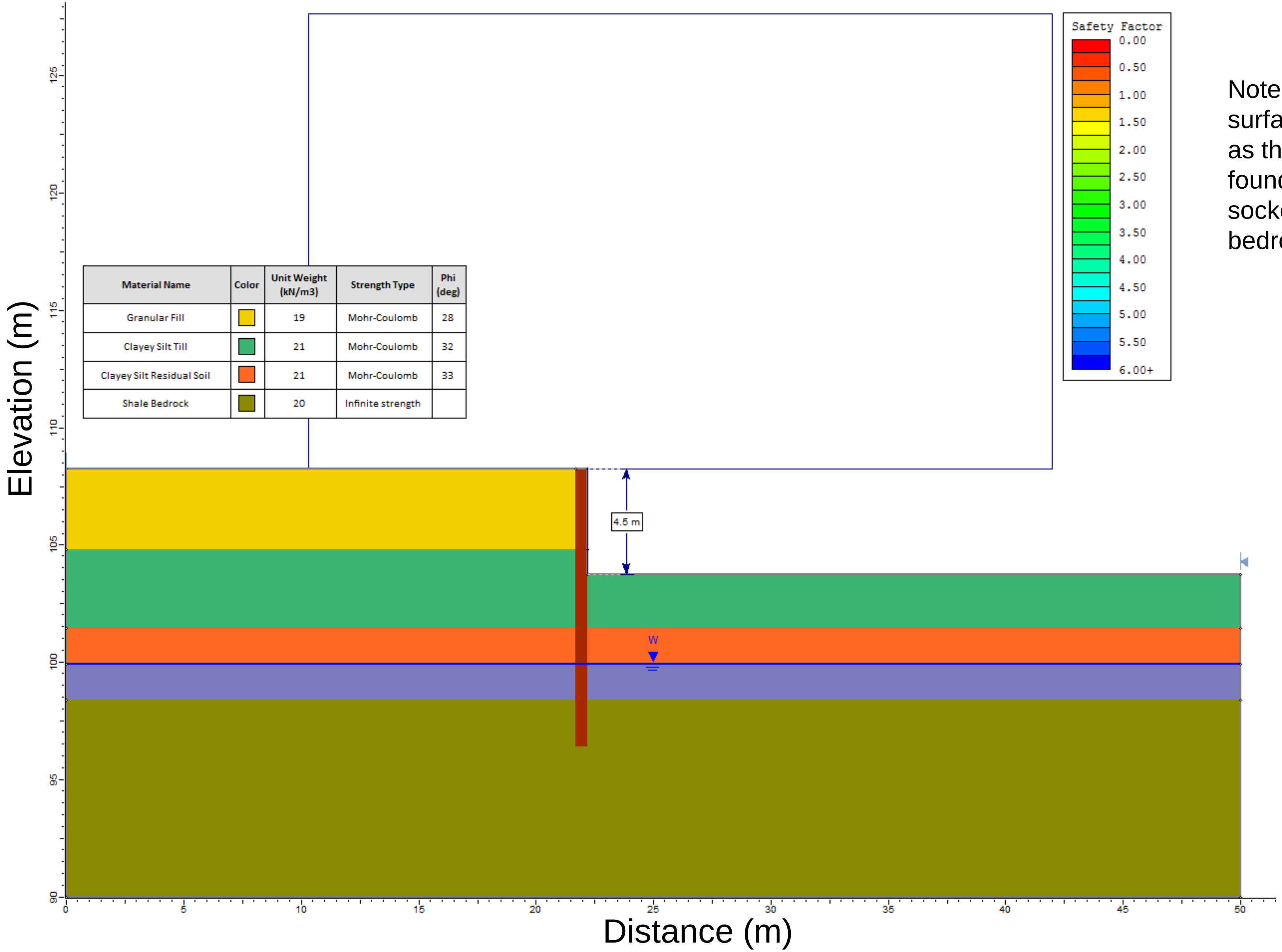
Geocres No. 30M11-275

HWY. QEW		PROJECT NO. 1530382	DIST. CENTRAL
SUBM'D. DB	CHKD. SMM	DATE: 05/18/2018	SITE:
DRAWN: SMD	CHKD. NK	APPD. JMAC	DWG. 2

PLOT DATE: May 22, 2018
FILENAME: S:\Clients\MTO\QEW-Dixie\99_PROJ\1530382\40_PROD\0020_Retaining_Walls_24-887W_&_24-888W\1530382-0020-BG-0002.dwg

STATIC GLOBAL STABILITY
SOLDIER PILE AND CONCRETE PANEL WALL, SECANT WALL OR CONCRETE RETAINING WALL ON DEEP FOUNDATIONS

Figure 1



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Phi (deg)
Granular Fill	🟨	19	Mohr-Coulomb	28
Clayey Silt Till	🟩	21	Mohr-Coulomb	32
Clayey Silt Residual Soil	🟧	21	Mohr-Coulomb	33
Shale Bedrock	🟫	20	Infinite strength	

Note: no failure surfaces were found as the deep foundations are socketed into the bedrock

Date: March 2018

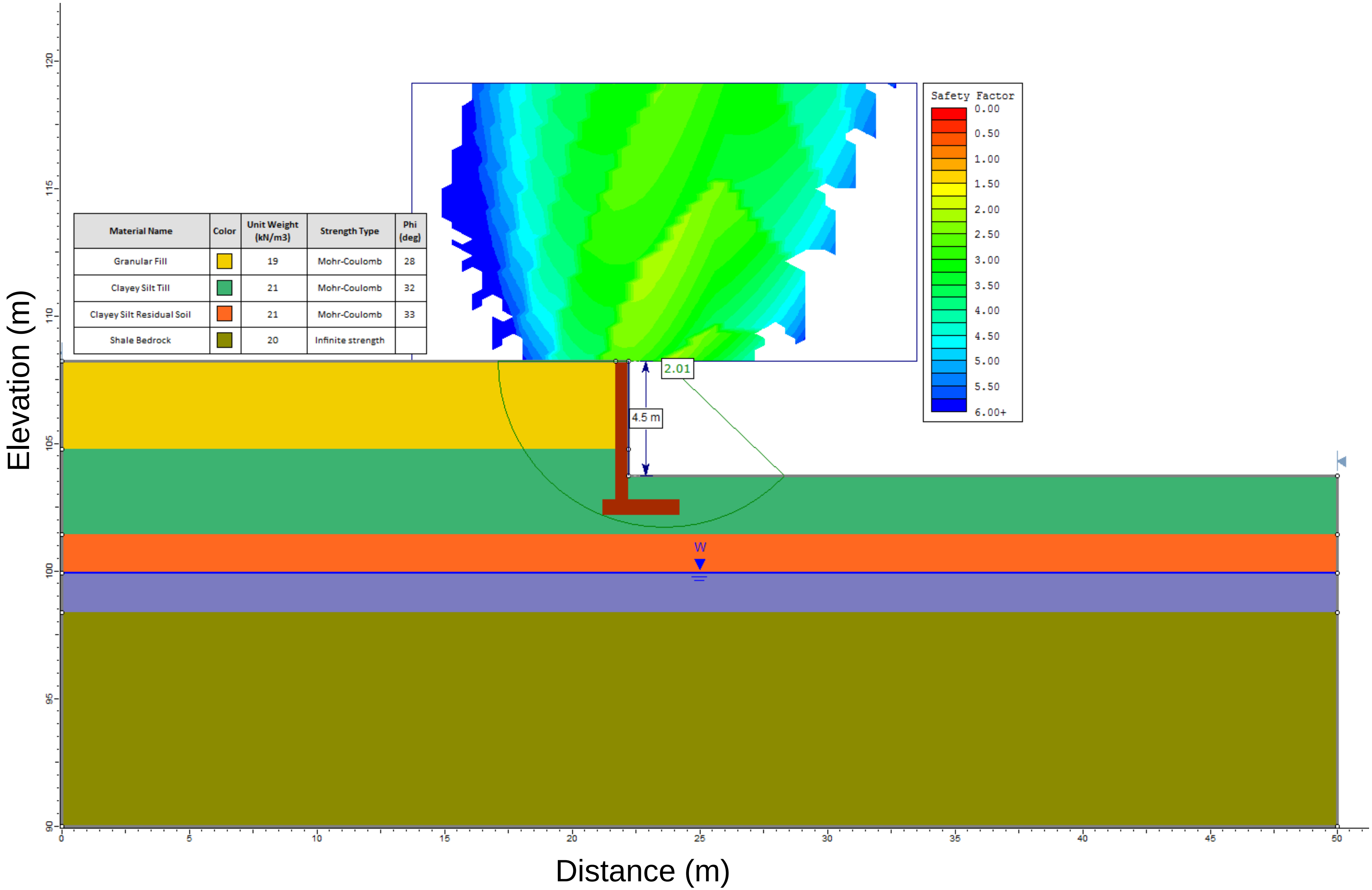
Project No: 1530382

Analysis By: NK

Reviewed By: JMAC

STATIC GLOBAL STABILITY
CONCRETE CANTILEVER WALL ON SHALLOW FOUNDATIONS

Figure 2



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Phi (deg)
Granular Fill	🟨	19	Mohr-Coulomb	28
Clayey Silt Till	🟩	21	Mohr-Coulomb	32
Clayey Silt Residual Soil	🟧	21	Mohr-Coulomb	33
Shale Bedrock	🟫	20	Infinite strength	

Date: March 2018

Project No: 1530382

Analysis By: NK

Reviewed By: JMAC

APPENDIX A

Retaining Wall No. 24-887/W, QEW – Station 13+500 to 13+810 Record of Borehole/Drillhole Sheets, Laboratory Test Results and Bedrock Core Photographs

RECORD OF BOREHOLE No RW2-1

SHEET 1 OF 2

METRIC

PROJECT 1530382

G.W.P. 2102-13-00; 2432-13-00

LOCATION N 4829078.9; E 299630.9 MTM NAD 83 ZONE 10 (LAT. 43.601628; LONG. -79.564027)

ORIGINATED BY AJ

DIST Central HWY QEW

BOREHOLE TYPE Power Auger, 160 mm I.D. and 250 mm O.D. Hollow Stem Augers

COMPILED BY ACK

DATUM Geodetic

DATE November 17, 2017

CHECKED BY NK

ELEV DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLES – NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
107.9	GROUND SURFACE				
0.0	TOPSOIL				
0.1	SAND, some silt, trace gravel Very loose to loose Brown Moist	1	SS	2	
		2	SS	8	
		3	SS	6	
105.7 2.2	SAND and GRAVEL, trace to some silt, trace clay Dense Brown Moist	4	SS	45	43 46 9 2
		5	SS	33	
104.2 3.7	Sandy CLAYEY SILT, trace to some gravel (TILL) Stiff to very stiff Grey with oxidation staining between a depth of 3.7 m and 4.5 m Moist	6	SS	15	
		7	SS	12	10 20 45 25
102.3 5.6	Sandy gravelly CLAYEY SILT, trace shale fragments (RESIDUAL SOIL) Hard Grey Moist	8	SS	64	
100.7 7.2	SHALE (BEDROCK) Bedrock cored from 8.5 m to 18.8 m. Refer to Record of Drillhole RW2-1 for rock coring details.	9	SS	50/0.08	
		1	RC	REC 96%	RQD = 77%
		2	RC	REC 100%	RQD = 84%

GROUND WATER CONDITIONS

ELEVATION SCALE: 107, 106, 105, 104, 103, 102, 101, 100, 99, 98

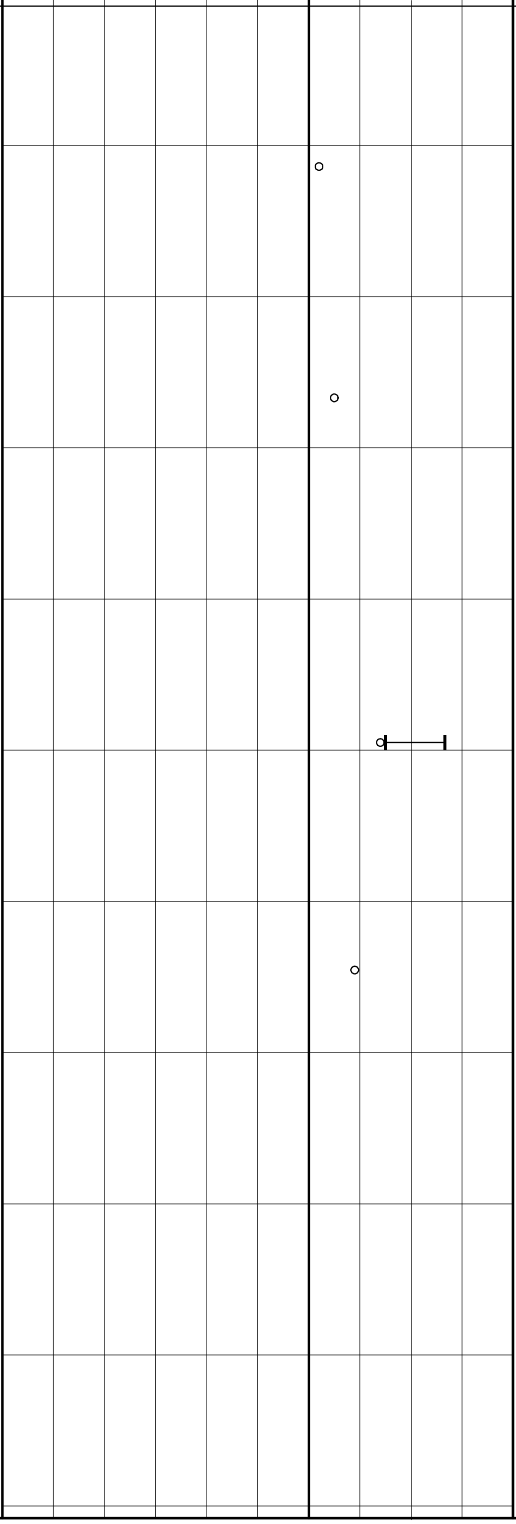
DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L

WATER CONTENT (%): 10 20 30

UNIT WEIGHT γ kN/m³



Continued Next Page

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

RECORD OF BOREHOLE No RW2-1

SHEET 2 OF 2

METRIC

PROJECT 1530382
G.W.P. 2102-13-00; 2432-13-00
DIST Central HWY QEW
DATUM Geodetic
LOCATION N 4829078.9; E 299630.9 MTM NAD 83 ZONE 10 (LAT. 43.601628; LONG. -79.564027)
BOREHOLE TYPE Power Auger, 160 mm I.D. and 250 mm O.D. Hollow Stem Augers
DATE November 17, 2017
ORIGINATED BY AJ
COMPILED BY ACK
CHECKED BY NK

SOIL PROFILE		SAMPLES					DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION --- CONTINUED FROM PREVIOUS PAGE ---	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE				
	SHALE (BEDROCK) Bedrock cored from 8.5 m to 18.8 m. Refer to Record of Drillhole RW2-1 for rock coring details.	2	RC	REC 100%		97				RQD = 84%
		3	RC	REC 100%		96				RQD = 99%
		4	RC	REC 87%		95 94				RQD = 82%
		5	RC	REC 100%		93				RQD = 98%
		6	RC	REC 100%		92 91				RQD = 100%
		7	RC	REC 100%		90				RQD = 100%
89.1		8	RC	REC 100%						RQD = 100%
18.8	END OF BOREHOLE NOTE: 1. Borehole dry upon completion of drilling and prior to rock coring.									

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

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

PROJECT: 1530382

RECORD OF DRILLHOLE: RW2-1

SHEET 1 OF 2

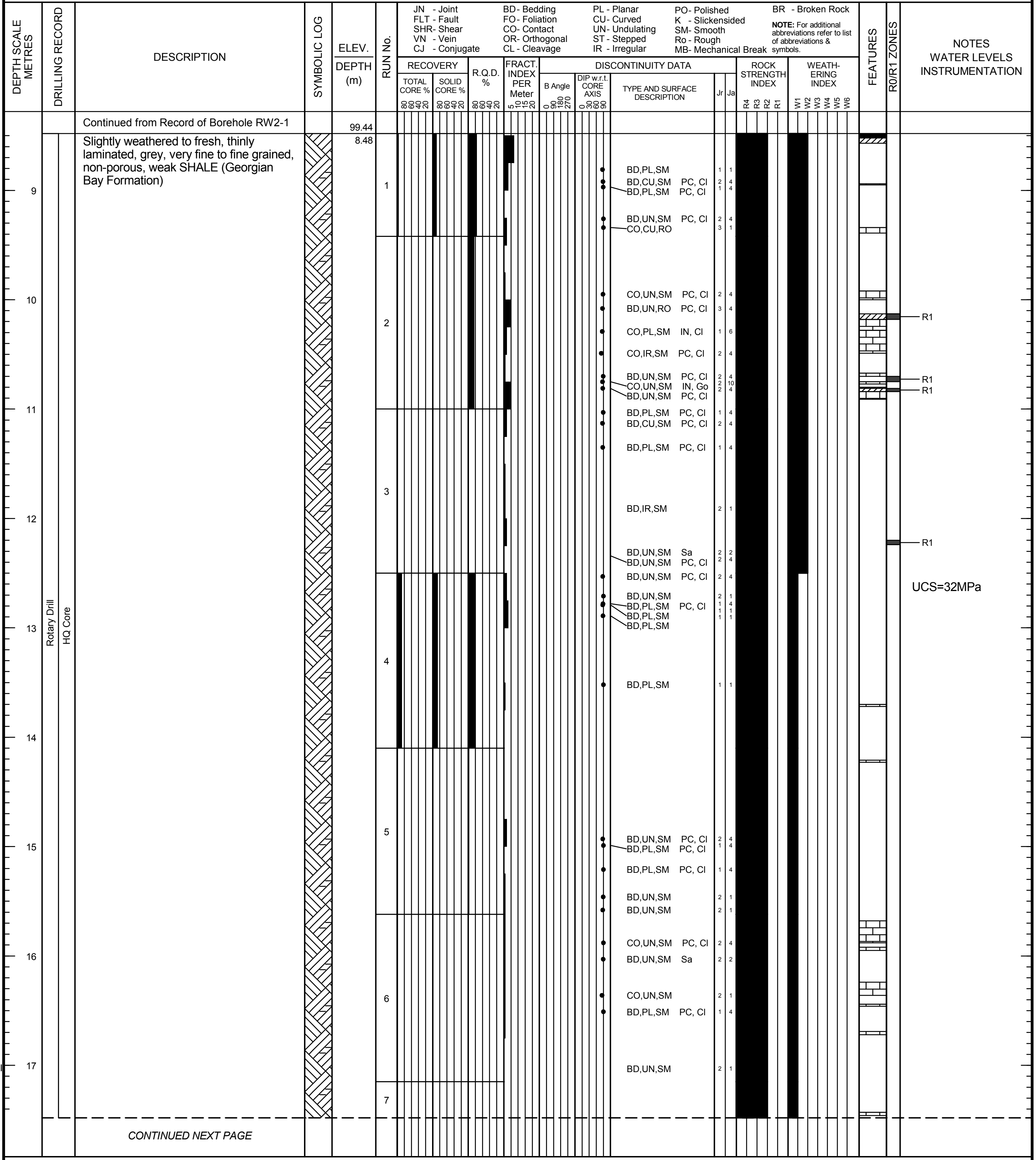
LOCATION: N 4829078.9 ;E 299630.9

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: November 17, 2017
DRILL RIG: CME 75 (Track Mounted)
DRILLING CONTRACTOR: Davis Drilling Ltd.

DATUM: Geodetic

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



Continued from Record of Borehole RW2-1 — 99.44

8.48 — Slightly weathered to fresh, thinly laminated, grey, very fine to fine grained, non-porous, weak SHALE (Georgian Bay Formation)

TYPE AND SURFACE DESCRIPTION		Jr	Ja
BD,PL,SM		1	1
BD,CU,SM	PC, Cl	2	4
BD,PL,SM	PC, Cl	1	4
BD,UN,SM	PC, Cl	2	4
CO,CU,RO		3	1
CO,UN,SM	PC, Cl	2	4
BD,UN,RO	PC, Cl	3	4
CO,PL,SM	IN, Cl	1	6
CO,IR,SM	PC, Cl	2	4
BD,UN,SM	PC, Cl	2	4
CO,UN,SM	IN, Go	2	10
BD,UN,SM	PC, Cl	2	4
BD,PL,SM	PC, Cl	1	4
BD,CU,SM	PC, Cl	2	4
BD,PL,SM	PC, Cl	1	4
BD,IR,SM		2	1
BD,UN,SM	Sa	2	2
BD,UN,SM	PC, Cl	2	4
BD,UN,SM	PC, Cl	2	4
BD,UN,SM		2	1
BD,PL,SM	PC, Cl	1	4
BD,PL,SM		1	1
BD,PL,SM		1	1
BD,PL,SM		1	1
BD,UN,SM	PC, Cl	2	4
BD,PL,SM	PC, Cl	1	4
BD,PL,SM	PC, Cl	1	4
BD,UN,SM		2	1
BD,UN,SM		2	1
CO,UN,SM	PC, Cl	2	4
BD,UN,SM	Sa	2	2
CO,UN,SM		2	1
BD,PL,SM	PC, Cl	1	4
BD,UN,SM		2	1

UCS=32MPa

CONTINUED NEXT PAGE

FEATURES LEGEND

▨ BROKEN CORE	▩ CLAY SEAM	▦ LIMESTONE	■ LOST CORE

DEPTH SCALE
1 : 50

GOLDER

LOGGED: AJ
CHECKED: MPL

PROJECT: 1530382

RECORD OF DRILLHOLE: RW2-1

SHEET 2 OF 2

LOCATION: N 4829078.9 ;E 299630.9
INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: November 17, 2017
DRILL RIG: CME 75 (Track Mounted)
DRILLING CONTRACTOR: Davis Drilling Ltd.

DATUM: Geodetic

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

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	RECOVERY TOTAL CORE % 80 60 40 20	RECOVERY SOLID CORE % 80 60 40 20	R.Q.D. % 80 60 40 20	FRACT. INDEX PER Meter 5 10 15 20	DISCONTINUITY DATA B Angle 0 90 180 270	DISCONTINUITY DATA DIP w.r.t. CORE AXIS 0 30 60 90	TYPE AND SURFACE DESCRIPTION	Jr	Ja	ROCK STRENGTH INDEX R4 R3 R2 R1	WEATHERING INDEX W1 W2 W3 W4 W5 W6	FEATURES	R0/R1 ZONES	NOTES WATER LEVELS INSTRUMENTATION
		--- CONTINUED FROM PREVIOUS PAGE ---																	
18	Rotary Drill HQ Core	Slightly weathered to fresh, thinly laminated, grey, very fine to fine grained, non-porous, weak SHALE (Georgian Bay Formation)			7						●	CO,PL,SM	1	1					
											●	CO,CU,SM	2	1					
				89.10	8														
19		END OF DRILLHOLE		18.82															
20																			
21																			
22																			
23																			
24																			
25																			
26																			

FEATURES LEGEND

▨ BROKEN CORE
▩ CLAY SEAM
▦ LIMESTONE
■ LOST CORE

DEPTH SCALE
1 : 50

GOLDER

LOGGED: AJ
CHECKED: MPL

RECORD OF BOREHOLE No RW2-2

SHEET 1 OF 2

METRIC

PROJECT 1530382

G.W.P. 2102-13-00; 2432-13-00

LOCATION N 4829107.3; E 299651.6 MTM NAD 83 ZONE 10 (LAT. 43.601883; LONG. -79.563771)

ORIGINATED BY AJ

DIST Central HWY QEW

BOREHOLE TYPE Power Auger, 160 mm I.D. and 250 mm O.D. Hollow Stem Augers

COMPILED BY ACK

DATUM Geodetic

DATE November 15, 2017

CHECKED BY NK

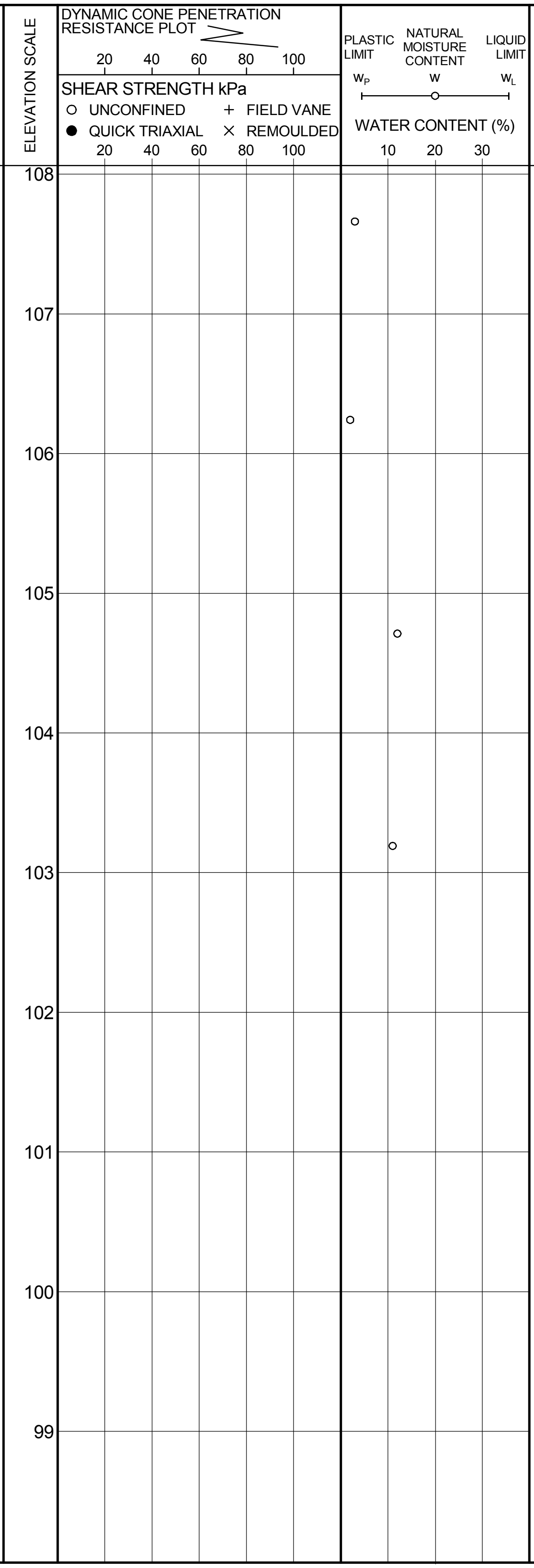
SOIL PROFILE

ELEV DEPTH	DESCRIPTION
108.1	GROUND SURFACE
0.0	TOPSOIL
0.1	Sand, trace to some silt, trace clay, trace gravel (FILL) Loose Brown Moist
107.4 0.7	SAND, trace to some gravel, some silt Loose to dense Brown to grey Moist
104.4 3.7	CLAYEY SILT, some sand, some gravel (TILL) Stiff to very stiff Grey Moist
102.5 5.6	CLAYEY SILT, some sand, some gravel, trace shale fragments (RESIDUAL SOIL) Hard Grey Moist
100.4 7.7	SHALE (BEDROCK) Bedrock cored from 8.5 m to 18.7 m. Refer to Record of Drillhole RW2-2 for rock coring details.

Continued Next Page

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1A			
1B	SS	7	
2	SS	9	
3	SS	16	0 85 14 1
4	SS	36	
5	SS	38	
6	SS	15	
7	SS	13	
8	SS	50	
9	SS	50/0.10	
1	RC	REC 97%	RQD = 68%
2	RC	REC 100%	RQD = 69%



UNIT WEIGHT γ kN/m³

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

RECORD OF BOREHOLE No RW2-2

SHEET 2 OF 2

METRIC

PROJECT 1530382

G.W.P. 2102-13-00; 2432-13-00

LOCATION N 4829107.3; E 299651.6 MTM NAD 83 ZONE 10 (LAT. 43.601883; LONG. -79.563771)

ORIGINATED BY AJ

DIST Central HWY QEW

BOREHOLE TYPE Power Auger, 160 mm I.D. and 250 mm O.D. Hollow Stem Augers

COMPILED BY ACK

DATUM Geodetic

DATE November 15, 2017

CHECKED BY NK

SOIL PROFILE			SAMPLES								
ELEV DEPTH	DESCRIPTION --- *CONTINUED FROM PREVIOUS PAGE* ---	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SHALE (BEDROCK) Bedrock cored from 8.5 m to 18.7 m. Refer to Record of Drillhole RW2-2 for rock coring details.		2	RC	REC 100%		98				RQD = 69%
			3	RC	REC 100%		97 96				RQD = 95%
			4	RC	REC 100%		95				RQD = 98%
			5	RC	REC 92%		94 93				RQD = 82%
			6	RC	REC 99%		92 91				RQD = 91%
89.4			7	RC	REC 95%		90				RQD = 88%
18.7	END OF BOREHOLE NOTE: 1. Borehole dry upon completion of drilling and prior to rock coring.										

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

PROJECT: 1530382

RECORD OF DRILLHOLE: RW2-2

SHEET 1 OF 2

LOCATION: N 4829107.3 ;E 299651.6

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: November 15, 2017
DRILL RIG: CME 75 (Truck Mounted)
DRILLING CONTRACTOR: Davis Drilling Ltd.

DATUM: Geodetic

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

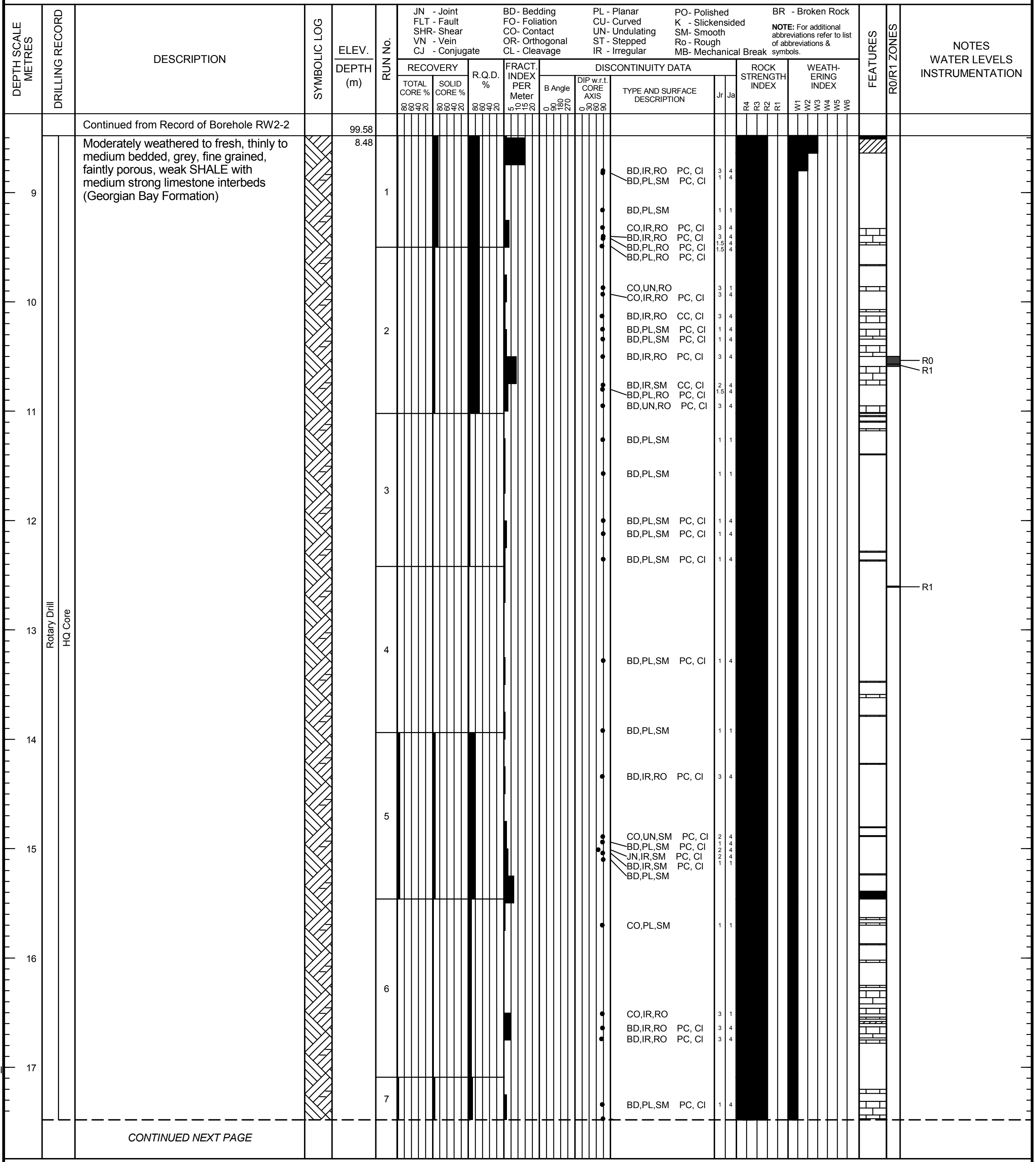
BD- Bedding
FO- Foliation
CO- Contact
OR- Orthogonal
CL - Cleavage

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

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

BR - Broken Rock

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



Continued from Record of Borehole RW2-2 — 99.58

8.48 — Moderately weathered to fresh, thinly to medium bedded, grey, fine grained, faintly porous, weak SHALE with medium strong limestone interbeds (Georgian Bay Formation)

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

CONTINUED NEXT PAGE

FEATURES LEGEND

▨	BROKEN CORE	▩	CLAY SEAM	▦	LIMESTONE	■	LOST CORE

DEPTH SCALE
1 : 50

GOLDER

LOGGED: AJ
CHECKED: MPL

PROJECT: 1530382

RECORD OF DRILLHOLE: RW2-2

SHEET 2 OF 2

LOCATION: N 4829107.3 ;E 299651.6

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: November 15, 2017

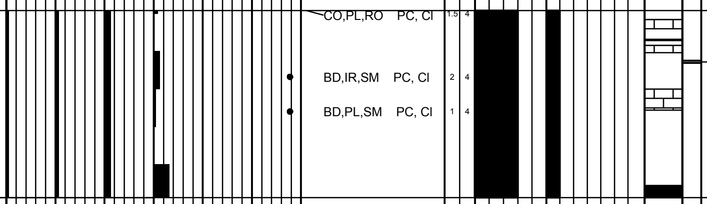
DRILL RIG: CME 75 (Truck Mounted)

DRILLING CONTRACTOR: Davis Drilling Ltd.

DATUM: Geodetic

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	Jr	Ja	FEATURES	R0/R1 ZONES	NOTES WATER LEVELS INSTRUMENTATION
		--- CONTINUED FROM PREVIOUS PAGE ---									
	Rotary Drill HQ Core	Moderately weathered to fresh, thinly to medium bedded, grey, fine grained, faintly porous, weak SHALE with medium strong limestone interbeds (Georgian Bay Formation)			7	CO,PL,RO PC, Cl	1.5	4			
18						BD,IR,SM PC, Cl	2	4		R1	
						BD,PL,SM PC, Cl	1	4			
		END OF DRILLHOLE		89.34 18.72							
19											
20											
21											
22											
23											
24											
25											
26											



FEATURES LEGEND

▨ BROKEN CORE	▩ CLAY SEAM	▦ LIMESTONE	■ LOST CORE

DEPTH SCALE

1 : 50

GOLDER

LOGGED: AJ

CHECKED: MPL

RECORD OF BOREHOLE No RW2-3

SHEET 1 OF 2

METRIC

PROJECT 1530382

G.W.P. 2102-13-00; 2432-13-00

LOCATION N 4829144.6; E 299684.5 MTM NAD 83 ZONE 10 (LAT. 43.602219; LONG. -79.563364)

ORIGINATED BY JL

DIST Central HWY QEW

BOREHOLE TYPE Power Auger, 160 mm I.D. and 250 mm O.D. Hollow Stem Augers

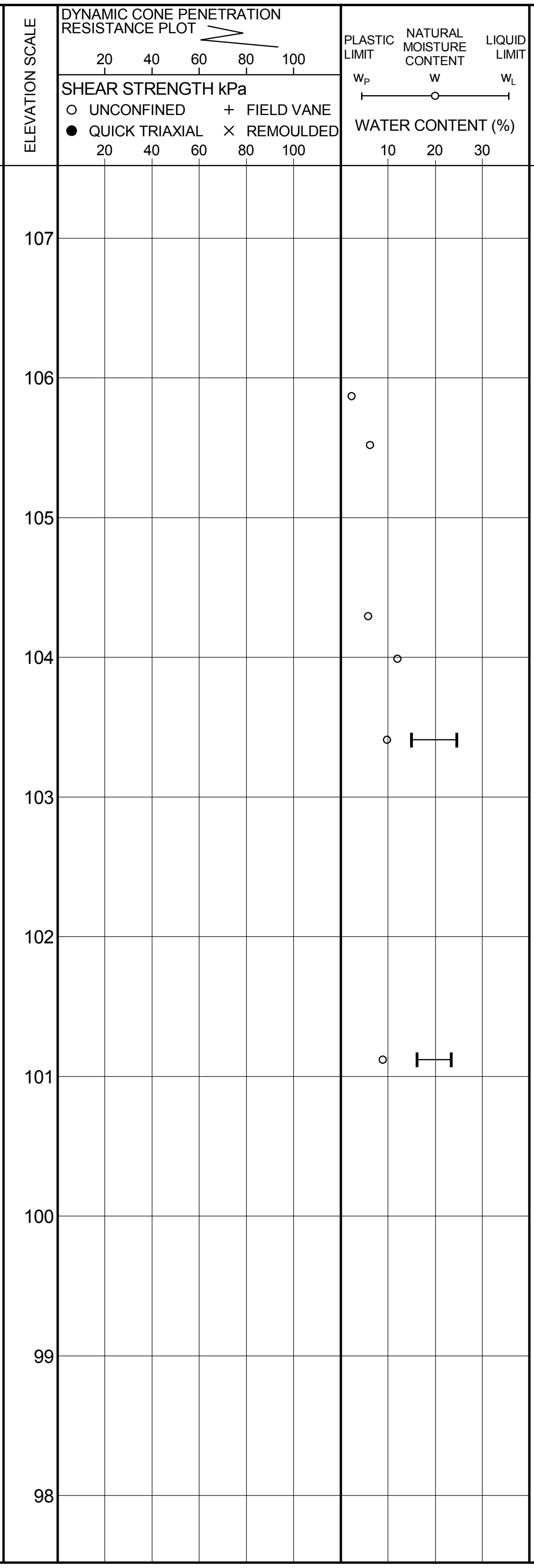
COMPILED BY ACK

DATUM Geodetic

DATE November 13, 2017

CHECKED BY SMM

ELEV DEPTH	SOIL PROFILE: DESCRIPTION	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: "N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
107.5	GROUND SURFACE									
0.0	TOPSOIL									
0.2	SAND, some silt to silty, trace organics Very loose to compact Brown Moist	1	SS	3						
		2	SS	8			0	81	18	1
		3A 3B	SS	16						
105.4 2.1	Gravelly SAND, trace to some fines, containing clayey silt pockets Compact Grey Moist	4	SS	26			26	60	12	2
	- Becoming wet at a depth of 3.1 m below ground surface	5A 5B	SS	28						
104.2 3.4	Sandy CLAYEY SILT to CLAYEY SILT with SAND, trace gravel to gravelly (TILL) Very stiff to hard Grey Moist	6	SS	23			10	30	45	15
	- Cobbles/boulders inferred from augers grinding at depths of 3.0 m to 3.7 m, 4.0 m, and 5.5 m to 6.1 m	7	SS	38						
		8	SS	40			22	31	41	6
		9	SS	50/0.08	▽					
		10	SS	50/0.05						
99.7 7.8	SHALE (BEDROCK) Bedrock cored from 7.8 m to 17.7 m. Refer to Record of Drillhole RW2-3 for rock coring details.	1	RC	REC 100%			RQD = 51%			
		2	RC	REC 98%			RQD = 25%			



Continued Next Page

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

GOLDER

Foundation Design

RECORD OF BOREHOLE No RW2-3

SHEET 2 OF 2

METRIC

PROJECT 1530382

G.W.P. 2102-13-00; 2432-13-00

LOCATION N 4829144.6; E 299684.5 MTM NAD 83 ZONE 10 (LAT. 43.602219; LONG. -79.563364)

ORIGINATED BY JL

DIST Central HWY QEW

BOREHOLE TYPE Power Auger, 160 mm I.D. and 250 mm O.D. Hollow Stem Augers

COMPILED BY ACK

DATUM Geodetic

DATE November 13, 2017

CHECKED BY SMM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m^3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	--- CONTINUED FROM PREVIOUS PAGE ---										
	SHALE (BEDROCK) Bedrock cored from 7.8 m to 17.7 m. Refer to Record of Drillhole RW2-3 for rock coring details.		2	RC	REC 98%		97				RQD = 25%
			3	RC	REC 100%		96				RQD = 97%
			4	RC	REC 98%		95 94				RQD = 95%
			5	RC	REC 100%		93				RQD = 90%
			6	RC	REC 97%		92 91				RQD = 59%
			7	RC	REC 100%		90				RQD = 70%
89.8 17.7	END OF BOREHOLE NOTE: 1. Water level in borehole at 7.7 m depth below ground surface (Elev. 99.8 m) upon completion of drilling and prior to rock coring.										

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

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

PROJECT: 1530382

RECORD OF DRILLHOLE: RW2-3

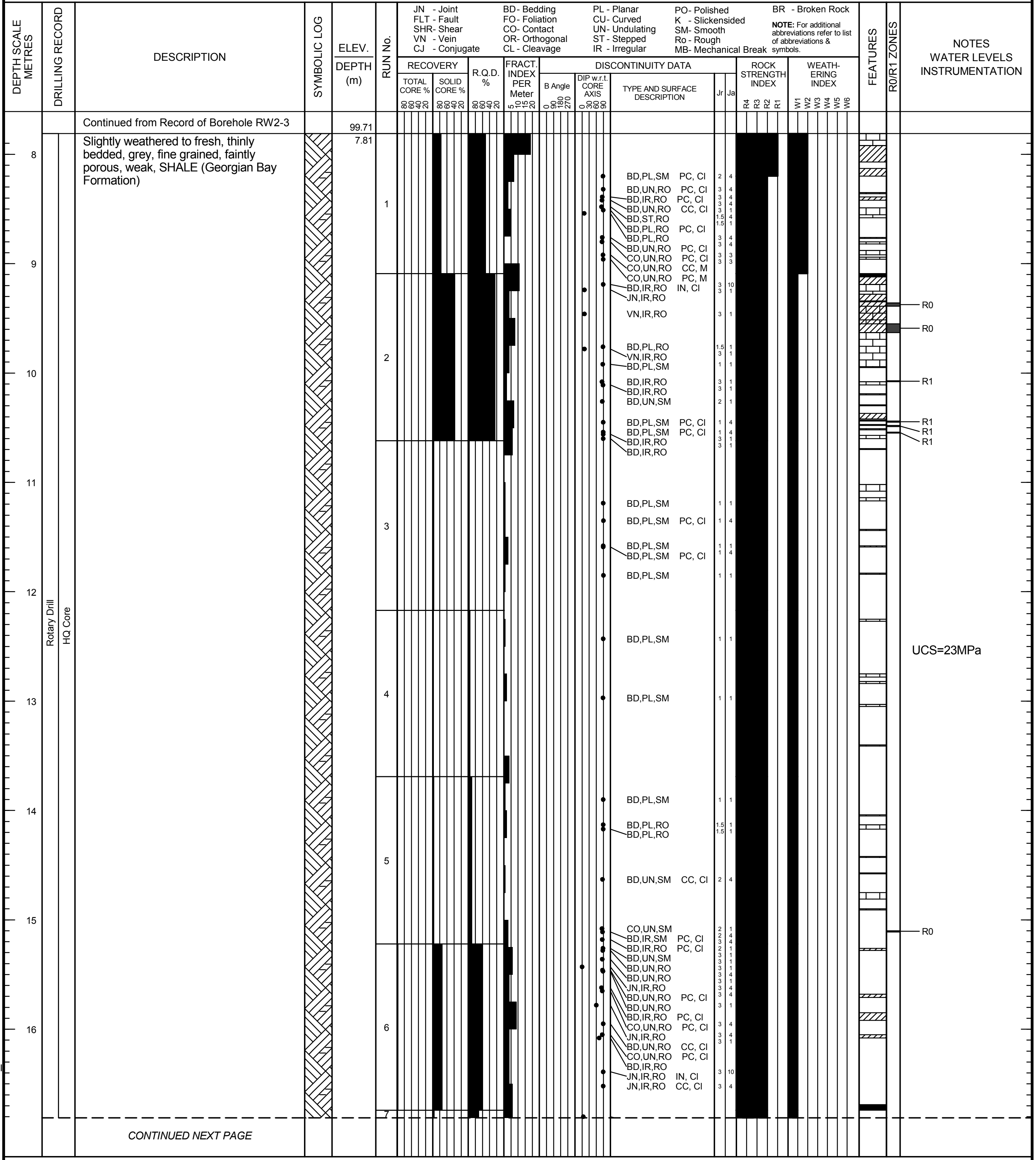
SHEET 1 OF 2

LOCATION: N 4829144.6 ;E 299684.5

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: November 13, 2017
DRILL RIG: CME 75 (Truck Mounted)
DRILLING CONTRACTOR: Davis Drilling Ltd.

DATUM: Geodetic



Continued from Record of Borehole RW2-3

Slightly weathered to fresh, thinly bedded, grey, fine grained, faintly porous, weak, SHALE (Georgian Bay Formation)

CONTINUED NEXT PAGE

FEATURES LEGEND

▨	BROKEN CORE	▩	CLAY SEAM	▦	LIMESTONE	■	LOST CORE

DEPTH SCALE
1 : 50

GOLDER

LOGGED: AK
CHECKED: MPL

PROJECT: 1530382

RECORD OF DRILLHOLE: RW2-3

SHEET 2 OF 2

LOCATION: N 4829144.6 ;E 299684.5

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: November 13, 2017
DRILL RIG: CME 75 (Truck Mounted)
DRILLING CONTRACTOR: Davis Drilling Ltd.

DATUM: Geodetic

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

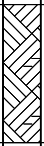
BD- Bedding
FO- Foliation
CO- Contact
OR- Orthogonal
CL - Cleavage

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

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

BR - Broken Rock

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. / DEPTH (m)	RUN No.	RECOVERY TOTAL CORE % (80 60 40 20)	RECOVERY SOLID CORE % (80 60 40 20)	R.Q.D. % (80 60 40 20)	FRACT. INDEX PER Meter (5 10 15 20)	B Angle (0 90 180 270)	DIP w.r.t. CORE AXIS (0 30 60 90)	TYPE AND SURFACE DESCRIPTION	Jr	Ja	ROCK STRENGTH INDEX (R4 R3 R2 R1)	WEATHERING INDEX (W1 W2 W3 W4 W5 W6)	FEATURES	R0/R1 ZONES	NOTES WATER LEVELS INSTRUMENTATION
		--- CONTINUED FROM PREVIOUS PAGE ---																	
17	Rotary Drill HQ Core	Slightly weathered to fresh, thinly bedded, grey, fine grained, faintly porous, weak, SHALE (Georgian Bay Formation)		89.80 17.72	7							JN,IR,RO BD,IR,RO PC, Cl CO,UN,RO PC, Cl CO,UN,RO PC, Cl CO,UN,RO PC, Cl CO,UN,RO PC, Cl CO,UN,RO PC, Cl	3 3 3 3 3 3 3	1 4 4 4 4 4 4			▨		
18		END OF DRILLHOLE																	
19																			
20																			
21																			
22																			
23																			
24																			
25																			

FEATURES LEGEND

▨	BROKEN CORE
▩	CLAY SEAM
▦	LIMESTONE
■	LOST CORE

DEPTH SCALE
1 : 50

GOLDER

LOGGED: AK
CHECKED: MPL

RECORD OF BOREHOLE No RW2-4

SHEET 1 OF 2

METRIC

PROJECT 1530382

G.W.P. 2102-13-00; 2432-13-00

LOCATION N 4829204.7; E 299747.0 MTM NAD 83 ZONE 10 (LAT. 43.602532; LONG. -79.562969)

ORIGINATED BY EN

DIST Central HWY QEW

BOREHOLE TYPE Power Auger, 160 mm I.D. and 250 mm O.D. Hollow Stem Augers

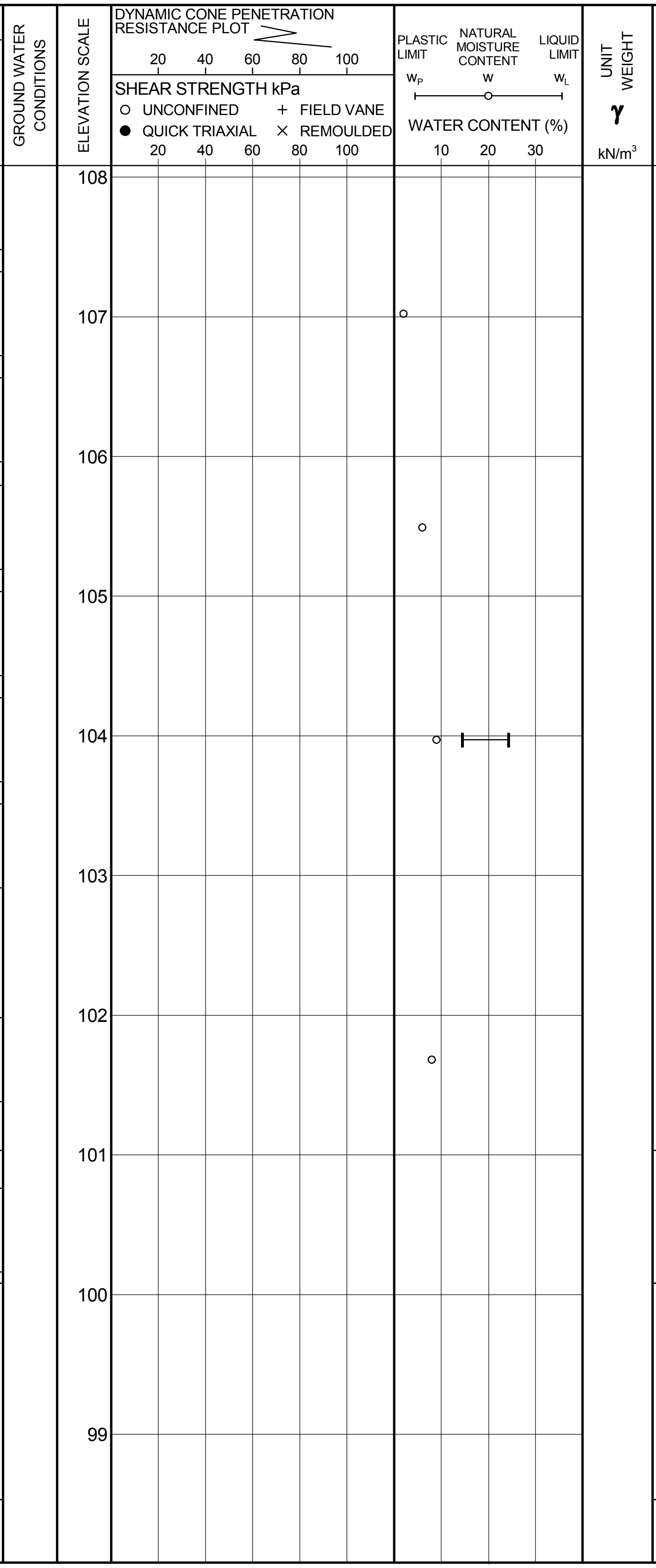
COMPILED BY MPL

DATUM Geodetic

DATE October 30, 2017

CHECKED BY SMM

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
108.1	GROUND SURFACE				
0.0	TOPSOIL				
0.1	Sand, trace brick fragments, trace clay, trace gravel (FILL) Compact Brown Moist	1	SS	10	
107.4 / 0.7	SAND, trace to some silt Compact Brown Moist	2	SS	11	0 87 12 1
		3	SS	10	
106.0 / 2.1	Gravelly SAND, some silt Dense Brown Moist	4	SS	44	
104.9 / 3.2	Sandy CLAYEY SILT, some gravel (TILL) Very stiff to hard Grey Moist	5A 5B	SS	19	
		6	SS	26	9 24 49 18
		7	SS	37	
102.5 / 5.6	Sandy CLAYEY SILT, some gravel, trace shale fragments (RESIDUAL SOIL) Hard Grey Moist	8	SS	54	
		1 9	RC SS	REC 100% 64	RQD = 0%
99.6 / 8.5	SHALE (BEDROCK) Bedrock cored from depths of 8.5 m to 18.6 m. Refer to Record of Drillhole RW2-4 for rock coring details.	2	RC	REC 100%	RQD = 20%
		3	RC	REC 100%	RQD = 72%



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

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

RECORD OF BOREHOLE No RW2-4

SHEET 2 OF 2

METRIC

PROJECT 1530382
G.W.P. 2102-13-00; 2432-13-00
DIST Central HWY QEW
DATUM Geodetic
LOCATION N 4829204.7; E 299747.0 MTM NAD 83 ZONE 10 (LAT. 43.602532; LONG. -79.562969)
BOREHOLE TYPE Power Auger, 160 mm I.D. and 250 mm O.D. Hollow Stem Augers
DATE October 30, 2017
ORIGINATED BY EN
COMPILED BY MPL
CHECKED BY SMM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION --- *CONTINUED FROM PREVIOUS PAGE* ---	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	SHALE (BEDROCK) Bedrock cored from depths of 8.5 m to 18.6 m. Refer to Record of Drillhole RW2-4 for rock coring details.	▨	3	RC	REC 100%		98 97				RQD = 72%
		▨	4	RC	REC 100%		96				RQD = 98%
		▨	5	RC	REC 100%		95 94				RQD = 86%
		▨	6	RC	REC 100%		93				RQD = 88%
		▨	7	RC	REC 100%		92 91				RQD = 77%
		▨	8	RC	REC 100%		90				RQD = 100%
89.5 18.6	END OF BOREHOLE NOTES: 1. Borehole dry upon completion of drilling and prior to rock coring. 2. Run 1 of Rotary Drilling was advanced through residual soil from 7.1 m to 8.0 m (Elev. 100.9 m to Elev. 100.1 m).										

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

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

PROJECT: 1530382

RECORD OF DRILLHOLE: RW2-4

SHEET 1 OF 2

LOCATION: N 4829204.7 ;E 299747.0

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: October 30, 2017
DRILL RIG: CME 75 (Truck Mounted)
DRILLING CONTRACTOR: Davis Drilling Ltd.

DATUM: Geodetic

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

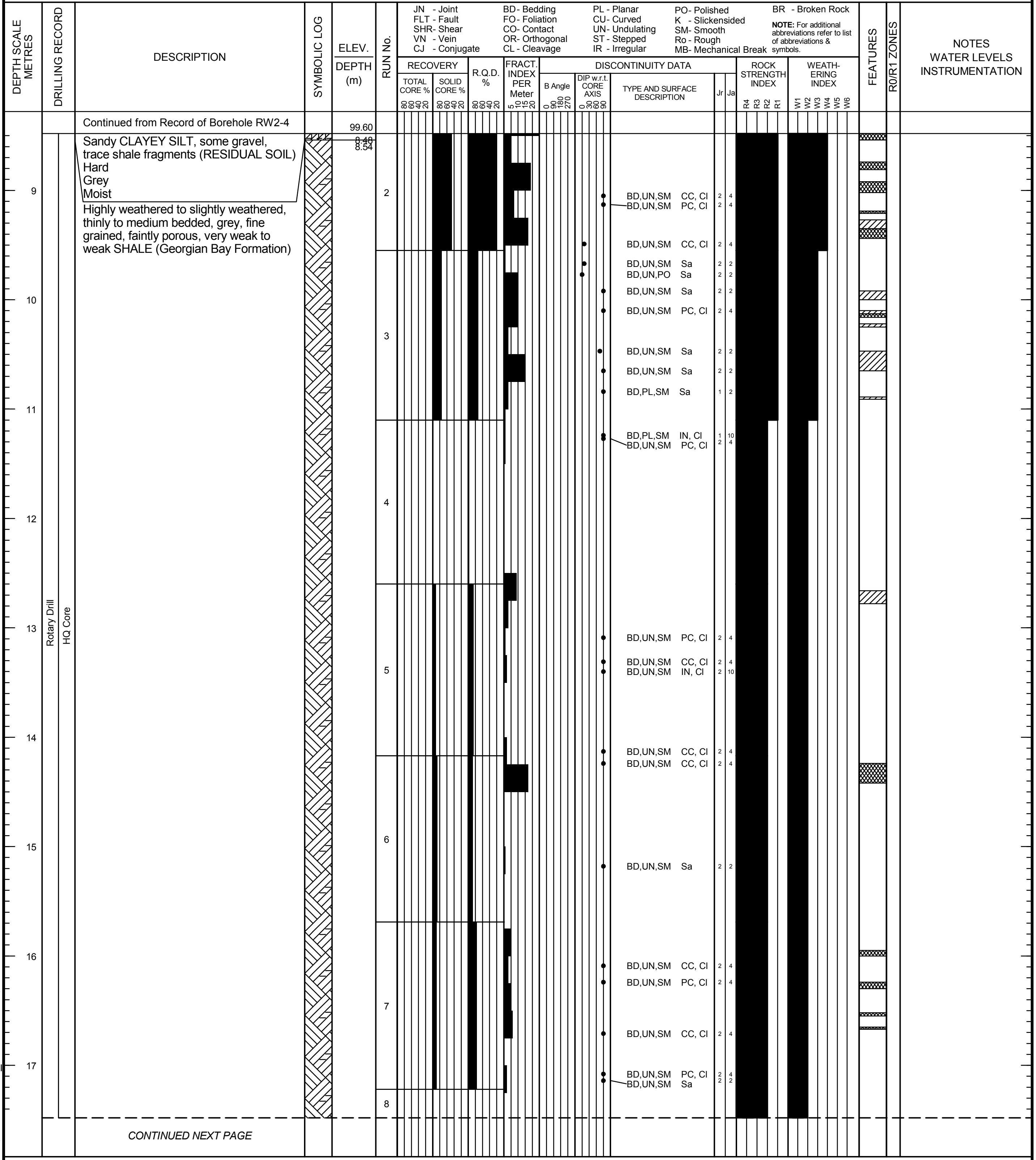
BD- Bedding
FO- Foliation
CO- Contact
OR- Orthogonal
CL - Cleavage

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

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

BR - Broken Rock

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



DESCRIPTION

Continued from Record of Borehole RW2-4

99.60

8.48

8.54

Sandy CLAYEY SILT, some gravel, trace shale fragments (RESIDUAL SOIL)
Hard
Grey
Moist

Highly weathered to slightly weathered, thinly to medium bedded, grey, fine grained, faintly porous, very weak to weak SHALE (Georgian Bay Formation)

TYPE AND SURFACE DESCRIPTION	Jr	Ja
BD,UN,SM CC, Cl	2	4
BD,UN,SM PC, Cl	2	4
BD,UN,SM CC, Cl	2	4
BD,UN,SM Sa	2	2
BD,UN,PO Sa	2	2
BD,UN,SM Sa	2	2
BD,UN,SM PC, Cl	2	4
BD,UN,SM Sa	2	2
BD,UN,SM Sa	2	2
BD,PL,SM Sa	1	2
BD,PL,SM IN, Cl	1	10
BD,UN,SM PC, Cl	2	4
BD,UN,SM PC, Cl	2	4
BD,UN,SM CC, Cl	2	4
BD,UN,SM IN, Cl	2	10
BD,UN,SM CC, Cl	2	4
BD,UN,SM CC, Cl	2	4
BD,UN,SM Sa	2	2
BD,UN,SM CC, Cl	2	4
BD,UN,SM PC, Cl	2	4
BD,UN,SM CC, Cl	2	4
BD,UN,SM PC, Cl	2	4
BD,UN,SM Sa	2	2

CONTINUED NEXT PAGE

FEATURES LEGEND

▨	BROKEN CORE	▩	CLAY SEAM	▦	LIMESTONE	■	LOST CORE

DEPTH SCALE
1 : 50

GOLDER

LOGGED: EN
CHECKED: JL

GTA-RCK 054 S:\CLIENTS\MTO\QEW-DIXIE\02_DATA\GINT\QEW-DIXIE.GPJ GAL-MISS.GDT 17/5/18 GPK

PROJECT: 1530382

RECORD OF DRILLHOLE: RW2-4

SHEET 2 OF 2

LOCATION: N 4829204.7 ;E 299747.0

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: October 30, 2017

DRILL RIG: CME 75 (Truck Mounted)

DRILLING CONTRACTOR: Davis Drilling Ltd.

DATUM: Geodetic

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

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

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

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

BR - Broken Rock

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. / DEPTH (m)	RUN No.	RECOVERY TOTAL CORE % 80 60 40 20	RECOVERY SOLID CORE % 80 60 40 20	R.Q.D. % 80 60 40 20	FRACT. INDEX PER Meter 5 10 15 20	B Angle 0 90 180 270	DIP w.r.t. CORE AXIS 0 30 60 90	TYPE AND SURFACE DESCRIPTION	Jr	Ja	ROCK STRENGTH INDEX R4 R3 R2 R1	WEATHERING INDEX W1 W2 W3 W4 W5 W6	FEATURES	R0/R1 ZONES	NOTES WATER LEVELS INSTRUMENTATION
		--- CONTINUED FROM PREVIOUS PAGE ---																
18	Rotary Drill HQ Core	Highly weathered to slightly weathered, thinly to medium bedded, grey, fine grained, faintly porous, very weak to weak SHALE (Georgian Bay Formation)	89.46 18.62	8							BD,PL,SM PC, Cl	1	4					
											BD,UN,SM Sa	2	2					
											BD,UN,SM Sa	2	2					
		END OF DRILLHOLE																
19																		
20																		
21																		
22																		
23																		
24																		
25																		
26																		

FEATURES LEGEND

▨	BROKEN CORE
▩	CLAY SEAM
▦	LIMESTONE
■	LOST CORE

DEPTH SCALE

1 : 50

GOLDER

LOGGED: EN

CHECKED: JL

RECORD OF BOREHOLE No RW2-5

SHEET 1 OF 1

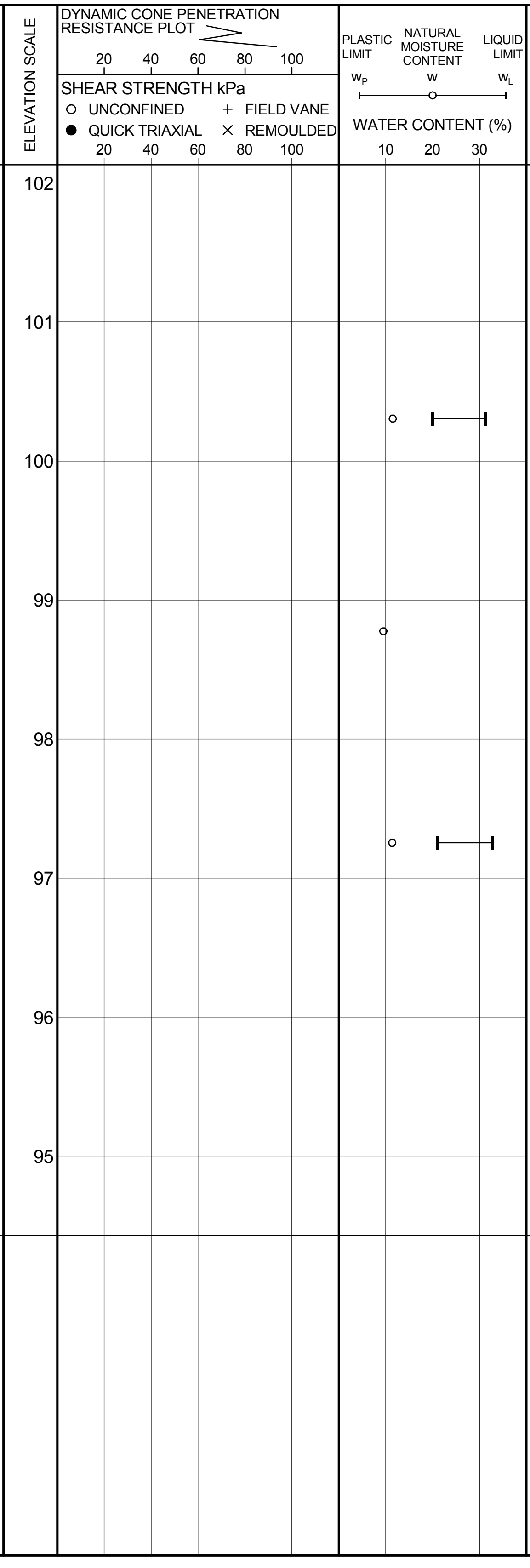
METRIC

PROJECT 1530382			
G.W.P. 2102-13-00; 2432-13-00	LOCATION N 4829306.2; E 299845.3 MTM NAD 83 ZONE 10 (LAT. 43.602761; LONG. -79.562591)	ORIGINATED BY MK	
DIST Central HWY QEW	BOREHOLE TYPE Power Auger, 160 mm I.D. and 250 mm O.D. Hollow Stem Augers	COMPILED BY ACK	
DATUM Geodetic	DATE September 9, 2016	CHECKED BY SMM	

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
102.1	GROUND SURFACE							
0.0	ASPHALT (100 mm)							
	CONCRETE (180 mm)							
101.8 0.3	Silty sand, some gravel to silty sand and gravel, containing clayey silt pockets (FILL) Compact Brown Moist	1	AS	--				
		2	SS	14				
100.7 1.4	Sandy CLAYEY SILT, some gravel (TILL) Stiff to very stiff Grey Moist	3	SS	15	18	21	50	11
		4	SS	14				
99.1 3.0	CLAYEY SILT, trace to some sand, trace to some gravel Very stiff to hard Grey Moist	5	SS	45				
	- Shale fragments at a depth of 3.8 m below ground surface	6	SS	50/0.10				
		7	SS	23	8	11	66	15
		8	SS	41				
94.9 7.2	Sandy CLAYEY SILT, trace gravel, trace shale fragments (RESIDUAL SOIL) Hard Grey Moist - Shale fragments at a depth of 7.6 m below ground surface	9	SS	50/0.08				
94.4 7.7	END OF BOREHOLE SPLIT SPOON/AUGER REFUSAL ON PROBABLE BEDROCK							

NOTE:

1. Open borehole dry upon completion of drilling.



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

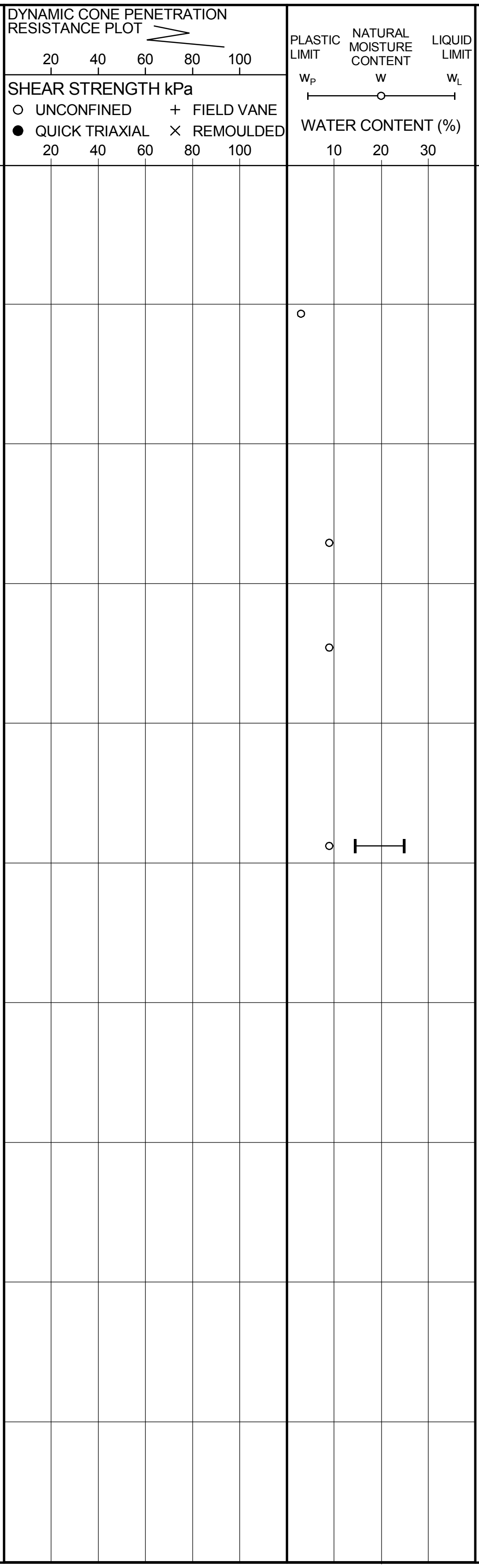
RECORD OF BOREHOLE No RW2-6

SHEET 1 OF 2

METRIC

PROJECT 1530382
G.W.P. 2102-13-00; 2432-13-00
DIST Central HWY QEW
DATUM Geodetic
LOCATION N 4829243.2; E 299769.4 MTM NAD 83 ZONE 10 (LAT. 43.602871; LONG. -79.561693)
BOREHOLE TYPE Power Auger, 160 mm I.D. and 250 mm O.D. Hollow Stem Augers
DATE October 30, 2017
ORIGINATED BY EN
COMPILED BY MPL
CHECKED BY SMM

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLES – NUMBER	SAMPLES – TYPE	SAMPLES – "N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
108.0	GROUND SURFACE							
0.0	TOPSOIL							
0.1	SAND, trace to some silt, trace gravel Loose to compact Brown Moist	1	SS	6				
		2	SS	6		107		
		3A 3B	SS	10		106		
105.8 2.2	Gravelly SAND, some silt to silty, containing silty clay pockets Compact Grey to brown Moist	4A 4B	SS	11				25 50 20 5
						105		
104.8 3.2	Sandy CLAYEY SILT, trace to some gravel, trace shale fragments (TILL) Very stiff to hard Grey Moist	5A 5B	SS	20				
		6	SS	24		104		
		7	SS	39		103		8 27 47 18
						102		
		1	RC	-				
						101		
		2	RC	-				
						100		
99.7 8.3	SHALE (BEDROCK) Coring beginning at 6.1 m. Bedrock cored from 8.3 m to 12.7 m. Refer to Record of Drillhole RW2-6 for rock coring details.	3	RC	REC 100%		99		RQD = 34%
		4	RC	REC 100%				RQD = 78%



Continued Next Page

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

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

PROJECT 1530382

RECORD OF BOREHOLE No RW2-6

SHEET 2 OF 2

METRIC

G.W.P. 2102-13-00; 2432-13-00

LOCATION N 4829243.2; E 299769.4 MTM NAD 83 ZONE 10 (LAT. 43.602871; LONG. -79.561693)

ORIGINATED BY EN

DIST Central HWY QEW

BOREHOLE TYPE Power Auger, 160 mm I.D. and 250 mm O.D. Hollow Stem Augers

COMPILED BY MPL

DATUM Geodetic

DATE October 30, 2017

CHECKED BY SMM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m^3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION *--- CONTINUED FROM PREVIOUS PAGE ---*	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	SHALE (BEDROCK) Coring beginning at 6.1 m. Bedrock cored from 8.3 m to 12.7 m. Refer to Record of Drillhole RW2-6 for rock coring details.		4	RC	REC 100%		97				RQD = 78%
			5	RC	REC 100%		96				RQD = 89%
95.3 12.7	END OF BOREHOLE NOTES: 1. Borehole dry upon completion of drilling and prior to rock coring. 2. Run 1 and Run 2 of Rotary Drilling was advanced through residual soil from 6.1 m to 6.6 m (Elev. 101.9 m to Elev. 101.4 m).										

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

PROJECT: 1530382

RECORD OF DRILLHOLE: RW2-6

SHEET 1 OF 1

LOCATION: N 4829243.2 ;E 299769.4

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: October 30, 2017

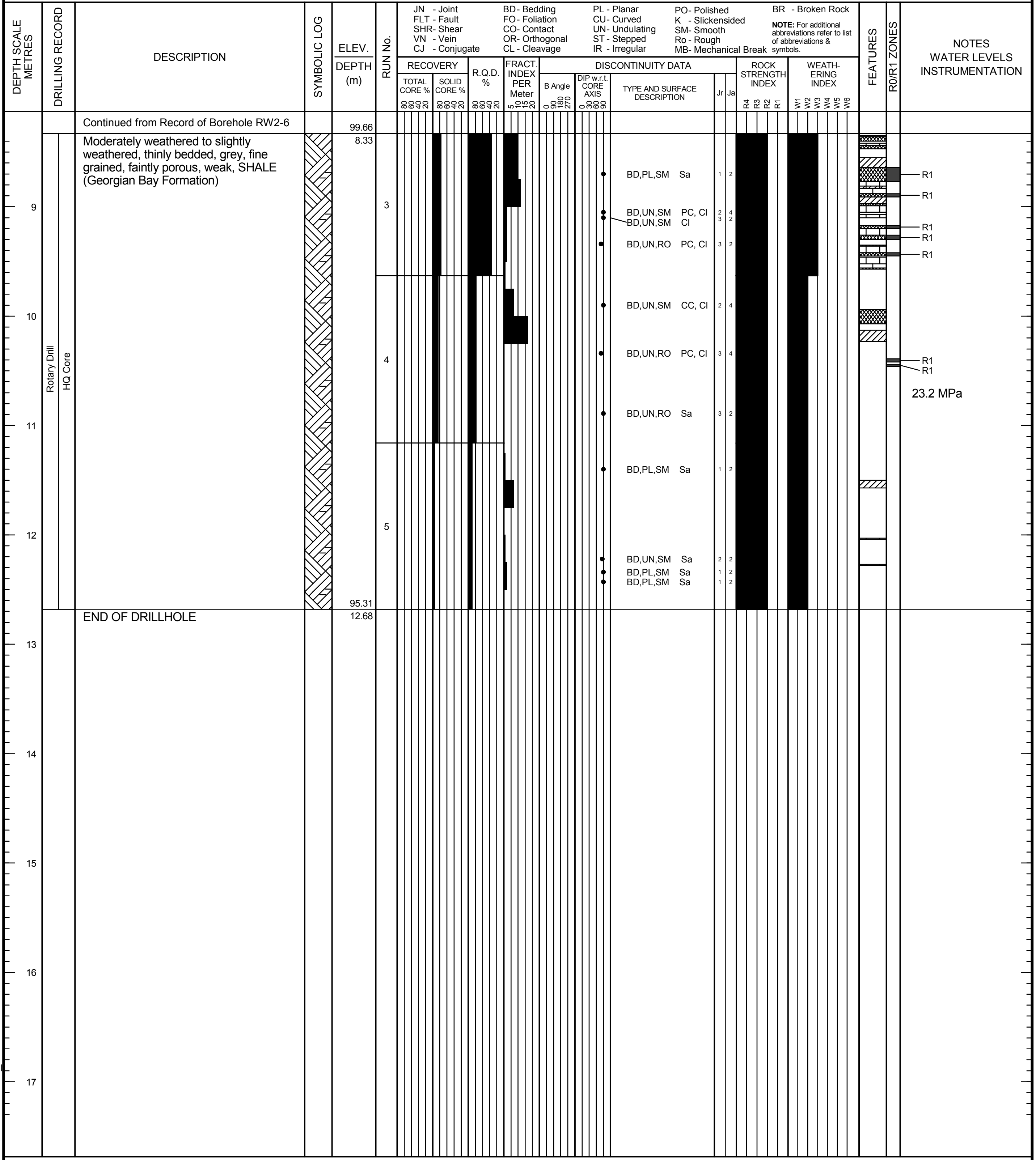
DRILL RIG: CME 75 (Truck Mounted)

DRILLING CONTRACTOR: Davis Drilling Ltd.

DATUM: Geodetic

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

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



DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	Jr	Ja	NOTES WATER LEVELS INSTRUMENTATION
		Continued from Record of Borehole RW2-6	99.66					
	Rotary Drill HQ Core	Moderately weathered to slightly weathered, thinly bedded, grey, fine grained, faintly porous, weak, SHALE (Georgian Bay Formation)	8.33					
					BD,PL,SM Sa	1	2	
9				3	BD,UN,SM PC, Cl	2	4	
					BD,UN,SM Cl	3	2	
					BD,UN,RO PC, Cl	3	2	
10					BD,UN,SM CC, Cl	2	4	
				4	BD,UN,RO PC, Cl	3	4	23.2 MPa
11					BD,UN,RO Sa	3	2	
					BD,PL,SM Sa	1	2	
12				5	BD,UN,SM Sa	2	2	
					BD,PL,SM Sa	1	2	
			95.31		BD,PL,SM Sa	1	2	
		END OF DRILLHOLE	12.68					
13								
14								
15								
16								
17								

FEATURES LEGEND

▨ BROKEN CORE ▩ CLAY SEAM ▦ LIMESTONE ■ LOST CORE

DEPTH SCALE

1 : 50

GOLDER

LOGGED: EN

CHECKED: ACK

RECORD OF BOREHOLE No RW2-7

SHEET 1 OF 3

METRIC

PROJECT	1530382		
G.W.P.	2102-13-00; 2432-13-00	LOCATION	N 4829276.6; E 299794.9 MTM NAD 83 ZONE 10 (LAT. 43.603107; LONG. -79.562314)
DIST	Central	HWY	QEW
BOREHOLE TYPE	Power Auger, 160 mm I.D. and 250 mm O.D. Hollow Stem Augers		
DATUM	Geodetic	DATE	December 7, 2017
ORIGINATED BY	DCB	COMPILED BY	ACK
CHECKED BY	NK		

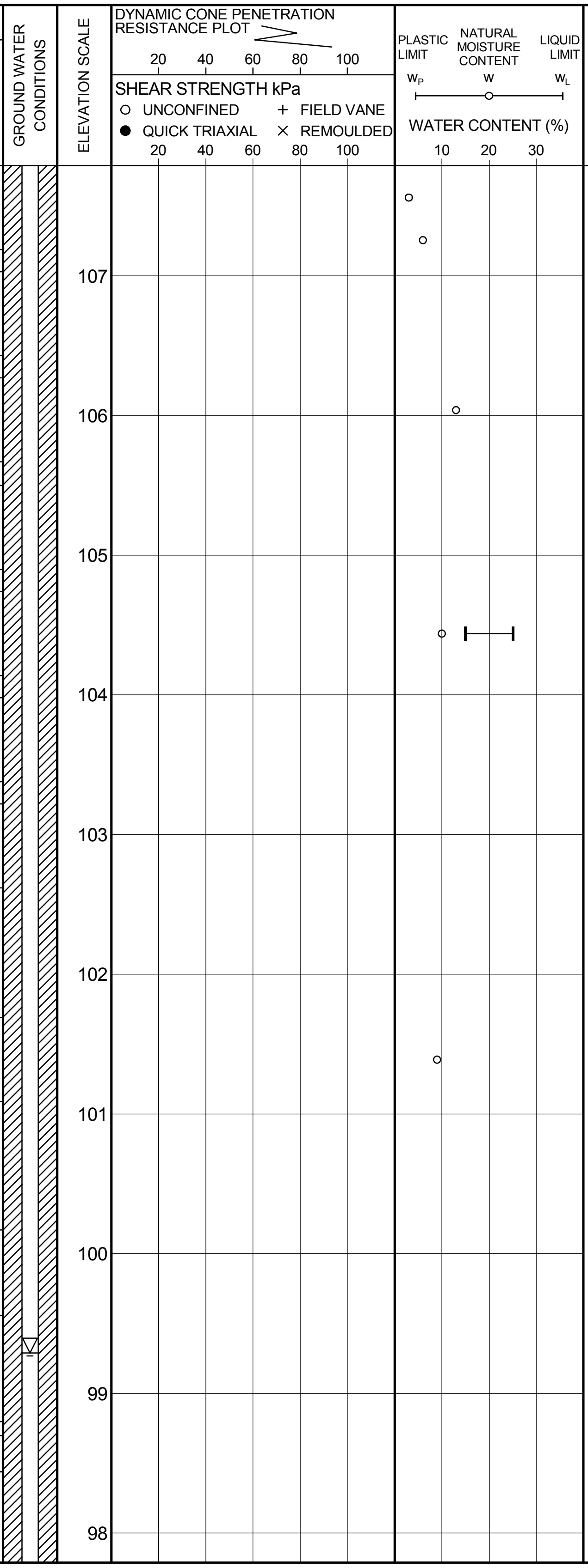
SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
107.8	GROUND SURFACE
0.0	ASPHALT (150 mm)
0.2	Sand and gravel to sand, some gravel (FILL) Brown Moist Compact
107.1 0.7	Silty SAND Loose Brown Moist
106.4 1.4	Sandy SILT, trace clay Dense Brown Moist
105.8 2.0	Gravelly SAND, trace silt Compact Brown Moist
105.2 2.6	CLAYEY SILT, trace sand Very stiff Brown to grey Moist
104.7 3.1	CLAYEY SILT with SAND (TILL) Very stiff to hard Grey Moist
103.3 4.5	Sandy gravelly CLAYEY SILT, containing shale fragments (RESIDUAL SOIL) Hard Grey Moist
100.6 7.2	SHALE (BEDROCK) Bedrock cored from 9.1 m to 20.2 m. Refer to Record of Drillhole RW2-7 for rock coring details.

Continued Next Page

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1A	SS	18	
1B			
2	SS	8	
3A	SS	33	
3B			
4A	SS	29	
4B			
5	SS	26	17 45 26 12
6A	SS	35	
6B			
7	SS	48	
8	SS	62	
9	SS	37	
10	SS	50/0.1	
1	RC	REC 100%	RQD = 0%
2	RC	REC 97%	RQD = 74%



UNIT WEIGHT γ kN/m³

+³, ×³: Numbers refer to Sensitivity

○³% STRAIN AT FAILURE

RECORD OF BOREHOLE No RW2-7

SHEET 2 OF 3

METRIC

PROJECT 1530382

G.W.P. 2102-13-00; 2432-13-00

DIST Central HWY QEW

DATUM Geodetic

LOCATION N 4829276.6; E 299794.9 MTM NAD 83 ZONE 10 (LAT. 43.603107; LONG. -79.562314)

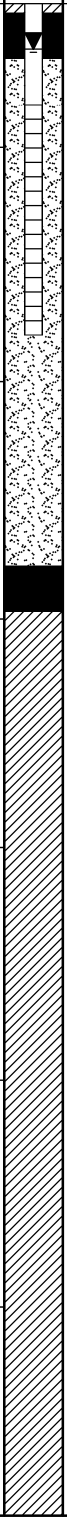
BOREHOLE TYPE Power Auger, 160 mm I.D. and 250 mm O.D. Hollow Stem Augers

DATE December 7, 2017

ORIGINATED BY DCB

COMPILED BY ACK

CHECKED BY NK



SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	--- CONTINUED FROM PREVIOUS PAGE ---										
	SHALE (BEDROCK) Bedrock cored from 9.1 m to 20.2 m. Refer to Record of Drillhole RW2-7 for rock coring details.		2	RC	REC 97%		97				RQD = 74%
			3	RC	REC 100%		96				RQD = 100%
			4	RC	REC 100%		95 94				RQD = 97%
			5	RC	REC 100%		93				RQD = 100%
			6	RC	REC 100%		92 91				RQD = 74%
			7	RC	REC 98%		90				RQD = 79%
			8	RC	REC 100%		89 88				RQD = 89%

Continued Next Page

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

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

RECORD OF BOREHOLE No RW2-7

SHEET 3 OF 3

METRIC

PROJECT	1530382		
G.W.P.	2102-13-00; 2432-13-00	LOCATION	N 4829276.6; E 299794.9 MTM NAD 83 ZONE 10 (LAT. 43.603107; LONG. -79.562314)
ORIGINATED BY	DCB		
DIST	Central	HWY	QEW
BOREHOLE TYPE	Power Auger, 160 mm I.D. and 250 mm O.D. Hollow Stem Augers	COMPILED BY	ACK
DATUM	Geodetic	DATE	December 7, 2017
CHECKED BY	NK		

SOIL PROFILE			SAMPLES								
ELEV DEPTH	DESCRIPTION *--- CONTINUED FROM PREVIOUS PAGE ---*	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
87.6		▨	8	RC		▨					RQD = 89%
20.2	END OF BOREHOLE										

NOTES:

1. Water level measured at a depth of 8.5 m below ground surface (Elev. 99.2 m) upon completion of soil drilling.

2. Water level in standpipe piezometer measured as follows:

Date	Depth (m)	Elev. (m)
Feb 22/18	4.3	103.5
Feb 23/18	11.7	96.1
Mar 01/18	11.3	96.5
Mar 07/18	11.2	96.6
Mar 16/18	10.9	96.9
Apr 02/18	10.3	97.5

GTA-MTO 001 S:\CLIENTS\MTO\QEW-DIXIE\02_DATA\GINT\QEW-DIXIE.GPJ GAL-GTA.GDT 17/5/18 GPK

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

$○^{3\%}$ STRAIN AT FAILURE

PROJECT: 1530382

RECORD OF DRILLHOLE: RW2-7

SHEET 1 OF 2

LOCATION: N 4829276.6 ;E 299794.9

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: December 7, 2017

DRILL RIG: CME 75 (Truck Mounted)

DRILLING CONTRACTOR: Davis Drilling Ltd.

DATUM: Geodetic

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

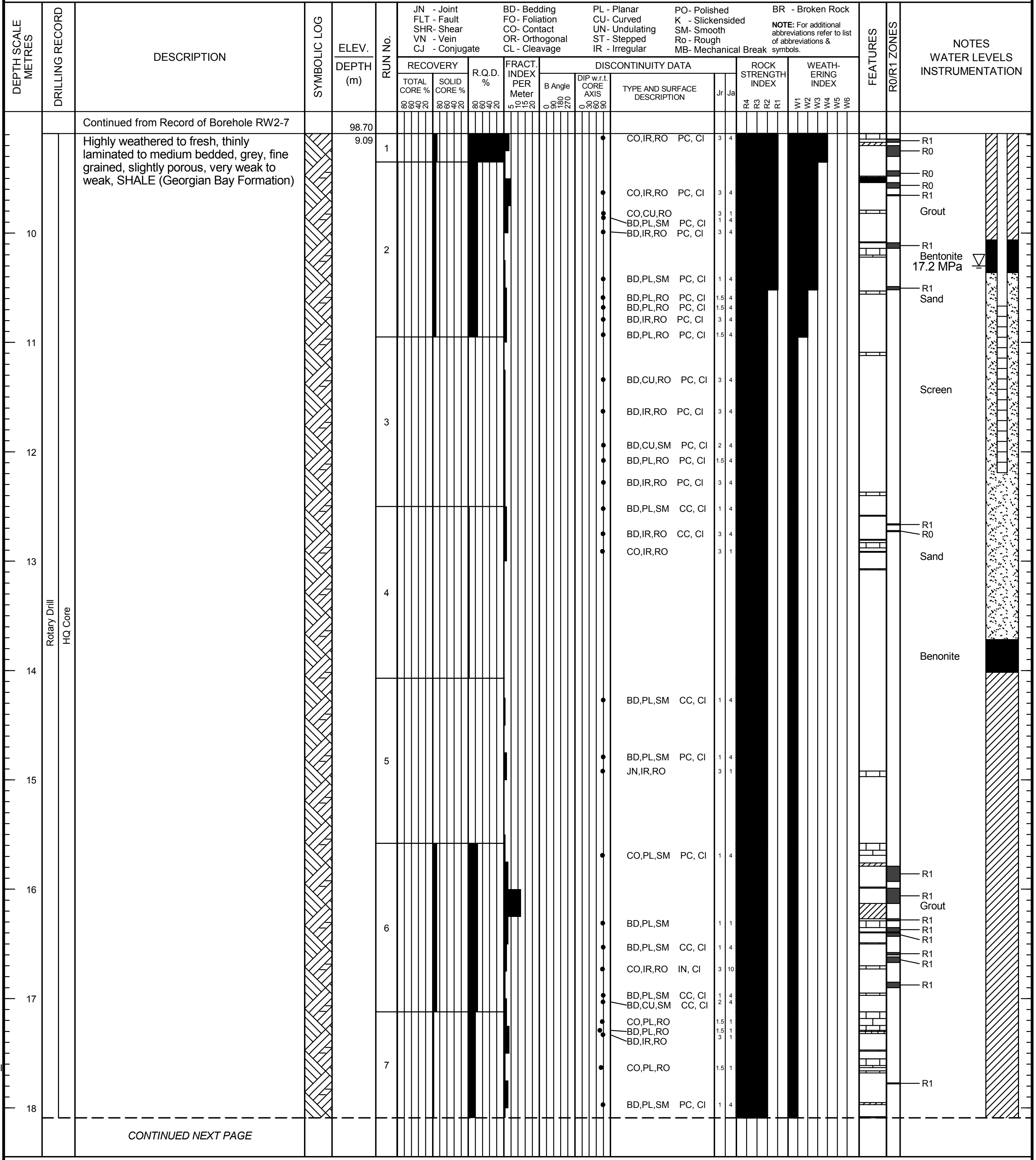
BD- Bedding
FO- Foliation
CO- Contact
OR- Orthogonal
CL - Cleavage

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

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

BR - Broken Rock

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



Continued from Record of Borehole RW2-7 — 98.70

Highly weathered to fresh, thinly laminated to medium bedded, grey, fine grained, slightly porous, very weak to weak, SHALE (Georgian Bay Formation) — 9.09

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

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FEATURES LEGEND

▨	BROKEN CORE
▩	CLAY SEAM
▦	LIMESTONE
■	LOST CORE

DEPTH SCALE

1 : 50

GOLDER

LOGGED: DCB

CHECKED: ACK

PROJECT: 1530382

RECORD OF DRILLHOLE: RW2-7

SHEET 2 OF 2

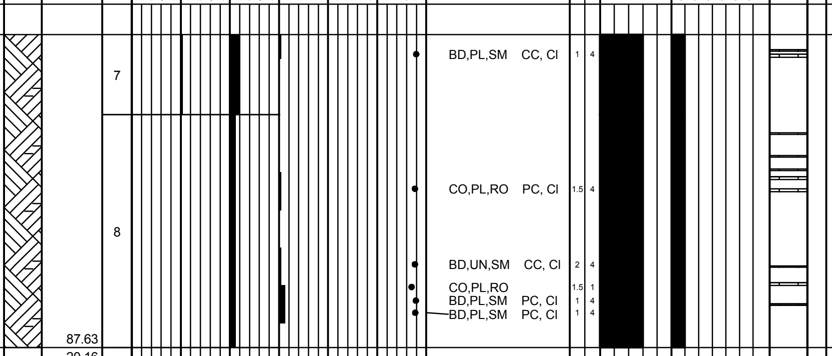
LOCATION: N 4829276.6 ;E 299794.9

INCLINATION: -90°　AZIMUTH: ---

DRILLING DATE: December 7, 2017
DRILL RIG: CME 75 (Truck Mounted)
DRILLING CONTRACTOR: Davis Drilling Ltd.

DATUM: Geodetic

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



DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. / DEPTH (m)	RUN No.	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	Jr	Ja	NOTES WATER LEVELS INSTRUMENTATION
		--- CONTINUED FROM PREVIOUS PAGE ---						
	Rotary Drill HQ Core	Highly weathered to fresh, thinly laminated to medium bedded, grey, fine grained, slightly porous, very weak to weak, SHALE (Georgian Bay Formation)		7	BD,PL,SM CC, CI	1	4	
19				8	CO,PL,RO PC, CI	1.5	4	Grout
					BD,UN,SM CC, CI	2	4	
					CO,PL,RO	1.5	1	
					BD,PL,SM PC, CI	1	4	
20			87.63 20.16		BD,PL,SM PC, CI	1	4	
		END OF DRILLHOLE						
21								
22								
23								
24								
25								
26								
27								

NOTES:

1. Water level measured at a depth of 8.5 m below ground surface (Elev. 99.2 m) upon completion of soil drilling.

2. Water level in standpipe piezometer measured at a depth of 11.2 m below ground surface (Elev. 96.5 m) on March 7, 2018.

3. Water level in standpipe piezometer measured at a depth of 10.9 m below groundsurface (Elev. 96.8 m) on March 16, 2018.

4. Water level in standpipe piezometer measured at a depth of 10.3 m below groundsurface (Elev. 97.4 m) on April 2, 2018.

FEATURES LEGEND

▨ BROKEN CORE	▩ CLAY SEAM	▦ LIMESTONE	■ LOST CORE

DEPTH SCALE
1 : 50

GOLDER

LOGGED: DCB
CHECKED: ACK

RECORD OF BOREHOLE No STM-10

SHEET 1 OF 2

METRIC

PROJECT 1530382

G.W.P. 2102-13-00; 2432-13-00

LOCATION N 4829178.8; E 299715.1 MTM NAD 83 ZONE 10 (LAT. 43.603675; LONG. -79.561374)

ORIGINATED BY EN

DIST Central HWY QEW

BOREHOLE TYPE Power Auger, 160 mm I.D. and 250 mm O.D. Hollow Stem Augers

COMPILED BY ACK

DATUM Geodetic

DATE November 10, 2017

CHECKED BY SMM

SOIL PROFILE | SAMPLES | GROUND WATER CONDITIONS | ELEVATION SCALE

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

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

PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L

WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ kN/m³

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

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
107.6	GROUND SURFACE							
0.0	TOPSOIL							
107.4 0.2	Sand, trace silt to silty, trace to some gravel, trace clay, trace rootlets, silty clay pockets from 0.2 m to 1.1 m (FILL) Loose to compact Brown Moist	1	SS	12				
		2A 2B	SS	8				
106.5 1.1	SAND, trace to some silt, trace gravel Loose to dense Brown Moist	3	SS	32	2	86	11	1
105.5 2.1	SAND and GRAVEL, some silt Brown Moist	4A						
105.1 2.5	SILT and SAND, trace gravel, trace clay Compact Brown Moist	4B	SS	29	1	35	56	8
104.7 2.9	Gravelly SAND, some rock fragments, trace silt Compact Brown Moist	5	SS	25				
103.6 4.0	Sandy CLAYEY SILT, some gravel (TILL) Hard Grey Moist	6	SS	31				
102.0 5.6	Sandy CLAYEY GRAVEL, trace shale fragments (RESIDUAL SOIL) Hard Grey Moist	7	SS	84	55	25	15	5
100.5 7.1	SHALE (BEDROCK) Bedrock cored from 7.6 m to 17.4 m Refer to Record of Drillhole STM-10 for rock coring details.	8	SS	50/0.08				
		1	RC	REC 100%	RQD = 70%			
		2	RC	REC 100%	RQD = 62%			
		3	RC	REC 100%	RQD = 81%			

ELEVATION SCALE: 107, 106, 105, 104, 103, 102, 101, 100, 99, 98

Continued Next Page

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

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

RECORD OF BOREHOLE No STM-10

SHEET 2 OF 2

METRIC

PROJECT 1530382

G.W.P. 2102-13-00; 2432-13-00

DIST Central HWY QEW

DATUM Geodetic

LOCATION N 4829178.8; E 299715.1 MTM NAD 83 ZONE 10 (LAT. 43.603675; LONG. -79.561374)

BOREHOLE TYPE Power Auger, 160 mm I.D. and 250 mm O.D. Hollow Stem Augers

DATE November 10, 2017

ORIGINATED BY EN

COMPILED BY ACK

CHECKED BY SMM

SOIL PROFILE		SAMPLES			ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES		
	--- CONTINUED FROM PREVIOUS PAGE ---					
	SHALE (BEDROCK) Bedrock cored from 7.6 m to 17.4 m Refer to Record of Drillhole STM-10 for rock coring details.	3	RC	REC 100%	97	RQD = 81%
		4	RC	REC 100%	96 95	RQD = 99%
		5	RC	REC 100%	94	RQD = 100%
		6	RC	REC 100%	93 92	RQD = 100%
		7	RC	REC 100%	91	RQD = 84%
90.2 17.4	END OF BOREHOLE					

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ kN/m³

NOTES:

1. Borehole dry upon completion of drilling and prior to rock coring.

2. Water level in standpipe piezometer measured as follows:

Date	Depth (m)	Elev. (m)
Nov 15/17	4.1	103.5
Feb 02/18	3.4	104.2
Feb 22/18	4.5	103.1
Feb 23/18	4.9	102.7
Mar 28/18	4.8	102.8

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

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

PROJECT: 1530382

RECORD OF DRILLHOLE: STM-10

SHEET 1 OF 2

LOCATION: N 4829178.8 ;E 299715.1

DRILLING DATE: November 10, 2017

DATUM: Geodetic

DRILL RIG:

INCLINATION: -90° AZIMUTH: ---

DRILLING CONTRACTOR: Davis Drilling Ltd.

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

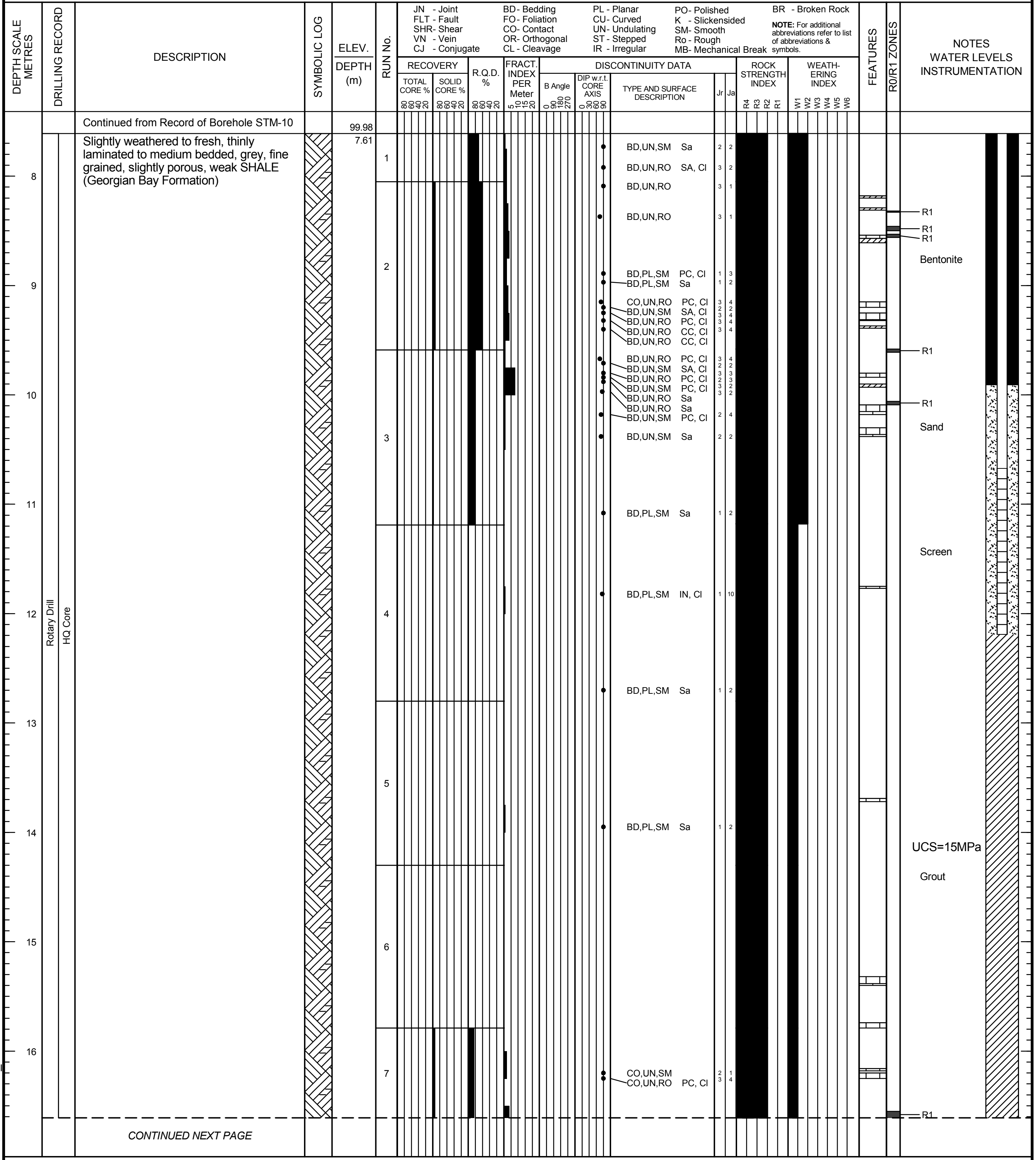
BD- Bedding
FO- Foliation
CO- Contact
OR- Orthogonal
CL - Cleavage

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

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

BR - Broken Rock

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



Continued from Record of Borehole STM-10 — 99.98

7.61 — Slightly weathered to fresh, thinly laminated to medium bedded, grey, fine grained, slightly porous, weak SHALE (Georgian Bay Formation)

Type and Surface Description		Jr	Ja
BD,UN,SM	Sa	2	2
BD,UN,RO	SA, Cl	3	2
BD,UN,RO		3	1
BD,UN,RO		3	1
BD,PL,SM	PC, Cl	1	3
BD,PL,SM	Sa	1	2
CO,UN,RO	PC, Cl	3	4
BD,UN,SM	SA, Cl	2	2
BD,UN,RO	PC, Cl	3	4
BD,UN,RO	CC, Cl	3	4
BD,UN,RO	CC, Cl	3	4
BD,UN,RO	PC, Cl	3	4
BD,UN,SM	SA, Cl	2	2
BD,UN,RO	PC, Cl	3	3
BD,UN,SM	PC, Cl	2	3
BD,UN,RO	Sa	3	2
BD,UN,RO	Sa	3	2
BD,UN,SM	PC, Cl	2	4
BD,UN,SM	Sa	2	2
BD,PL,SM	Sa	1	2
BD,PL,SM	IN, Cl	1	10
BD,PL,SM	Sa	1	2
BD,PL,SM	Sa	1	2
CO,UN,SM		2	1
CO,UN,RO	PC, Cl	3	4

CONTINUED NEXT PAGE

FEATURES LEGEND

▨ BROKEN CORE

▩ CLAY SEAM

▦ LIMESTONE

■ LOST CORE

DEPTH SCALE
1 : 50

GOLDER

LOGGED: EN
CHECKED:

PROJECT: 1530382

RECORD OF DRILLHOLE: STM-10

SHEET 2 OF 2

LOCATION: N 4829178.8 ;E 299715.1

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: November 10, 2017

DRILL RIG:

DRILLING CONTRACTOR: Davis Drilling Ltd.

DATUM: Geodetic

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	Jr	Ja	R0/R1 ZONES	NOTES WATER LEVELS INSTRUMENTATION
		--- CONTINUED FROM PREVIOUS PAGE ---							
17	Rotary Drill HQ Core	Slightly weathered to fresh, thinly laminated to medium bedded, grey, fine grained, slightly porous, weak SHALE (Georgian Bay Formation)		7	BD,PL,SM IN, Go CO,UN,SM PC, Cl BD,PL,SM IN, Go BD,UN,SM Sa BD,UN,SM Sa	1 2 1 2 2	5 4 5 2 2	R1 R1 R1	Grout
		END OF DRILLHOLE	90.21 17.38						
18									
19									
20									
21									
22									
23									
24									
25									

RECOVERY: TOTAL CORE % 80 60 40 20; SOLID CORE % 80 60 40 20 — R.Q.D. % 80 60 40 20 — FRACT. INDEX PER Meter 5 10 15 20 — B Angle 0 90 180 270 — DIP w.r.t. CORE AXIS 0 30 60 90 — ROCK STRENGTH INDEX R4 R3 R2 R1 — WEATHERING INDEX W1 W2 W3 W4 W5 W6 — FEATURES — SYMBOLIC LOG

FEATURES LEGEND

▨ BROKEN CORE

▩ CLAY SEAM

▦ LIMESTONE

■ LOST CORE

DEPTH SCALE

1 : 50

GOLDER

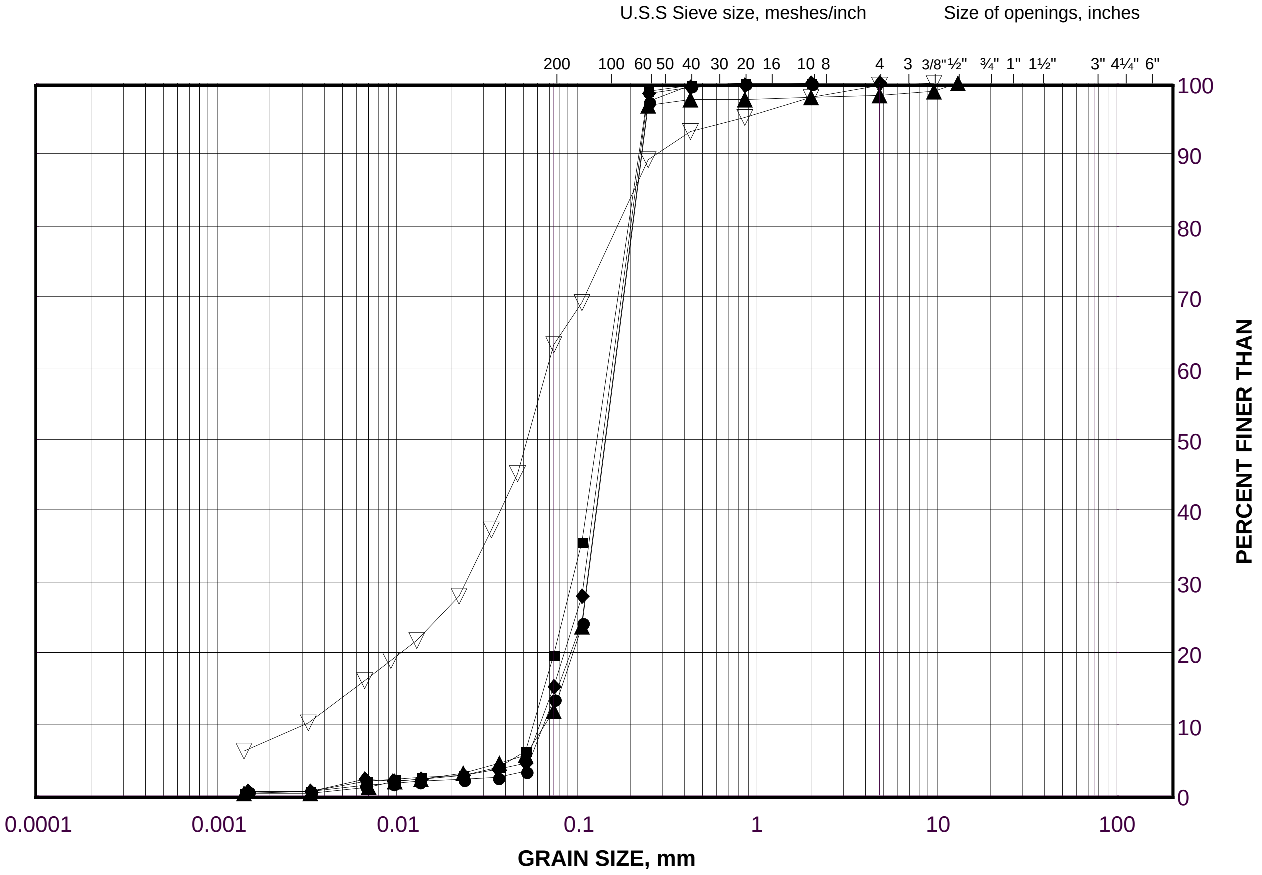
LOGGED: EN

CHECKED:

GRAIN SIZE DISTRIBUTION

Silt and Sand to Sand

FIGURE A1



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	RW2-4	2	107.0
■	RW2-3	2	106.5
◆	RW2-2	3	106.3
▲	STM-10	3	105.8
▽	STM-10	4B	104.9

Project Number: 1530382

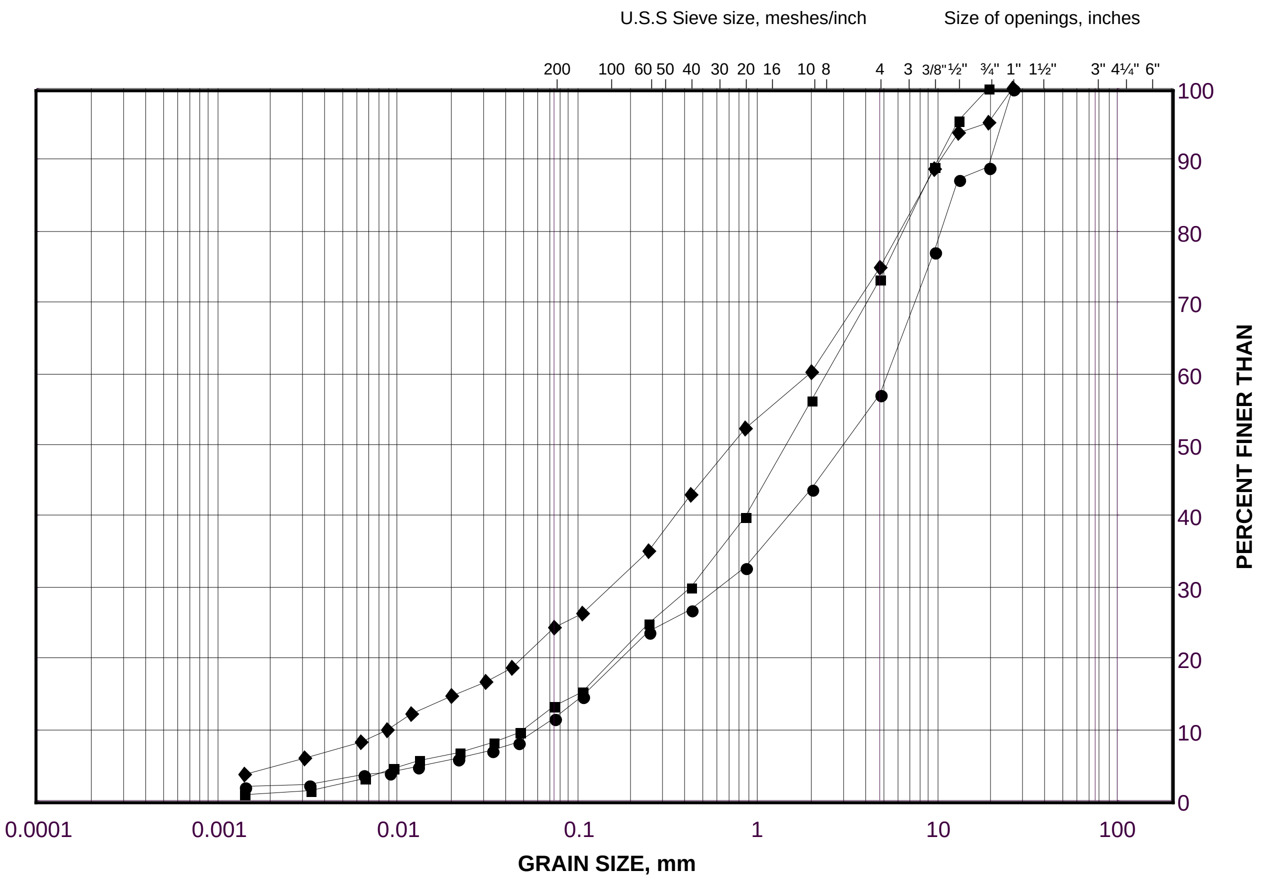
Checked By: NK

Golder Associates

Date: 16-May-18

GRAIN SIZE DISTRIBUTION
Gravelly Sand to Sand and Gravel

FIGURE A2



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

LEGEND

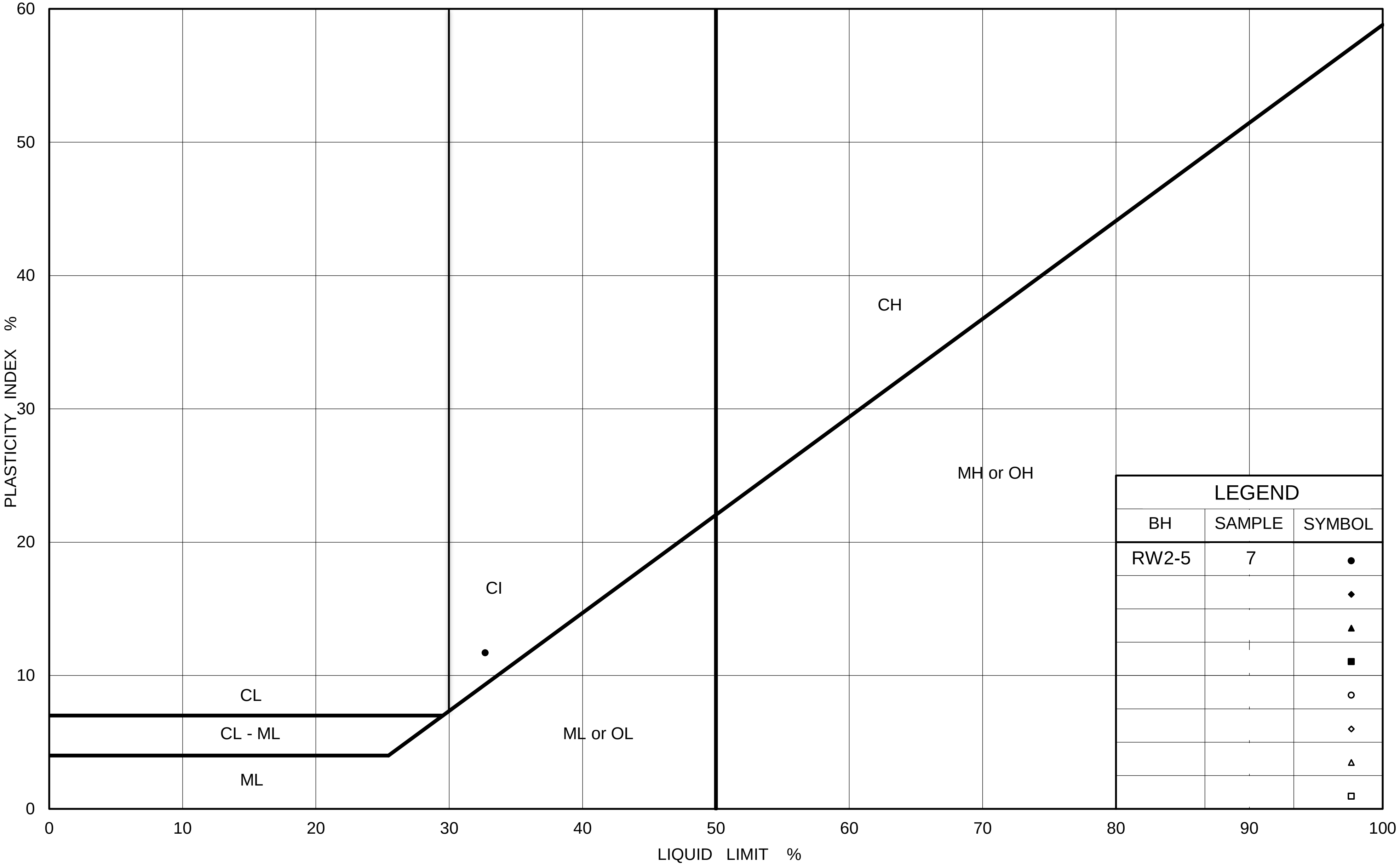
SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	RW2-1	4	105.3
■	RW2-3	4	104.9
◆	RW2-6	4B	105.3

Project Number: 1530382

Checked By: NK

Golder Associates

Date: 16-May-18



Golder Associates

PLASTICITY CHART
Clayey Silt

Figure No. A3

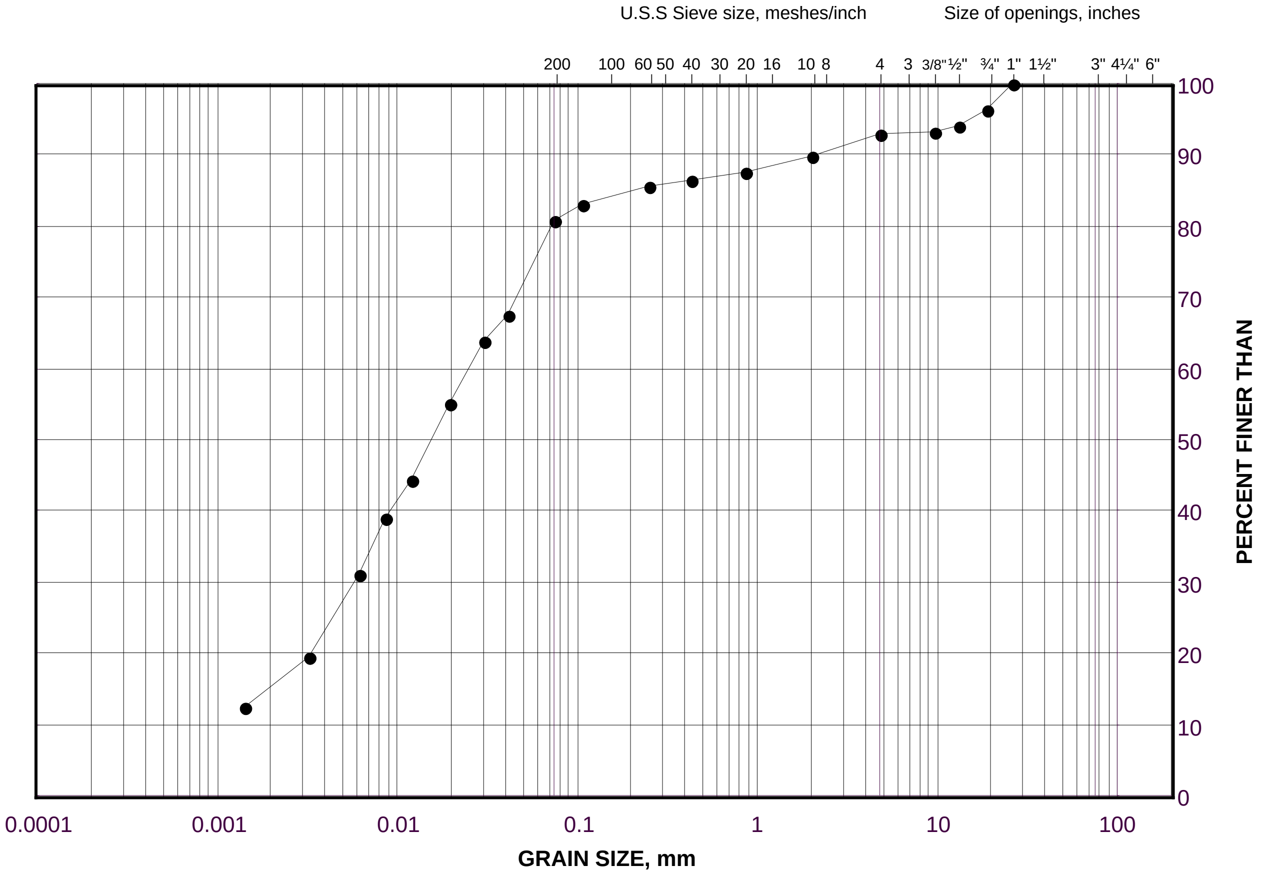
Project No. 1530382

Checked By: NK

GRAIN SIZE DISTRIBUTION

Clayey Silt

FIGURE A4



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

LEGEND

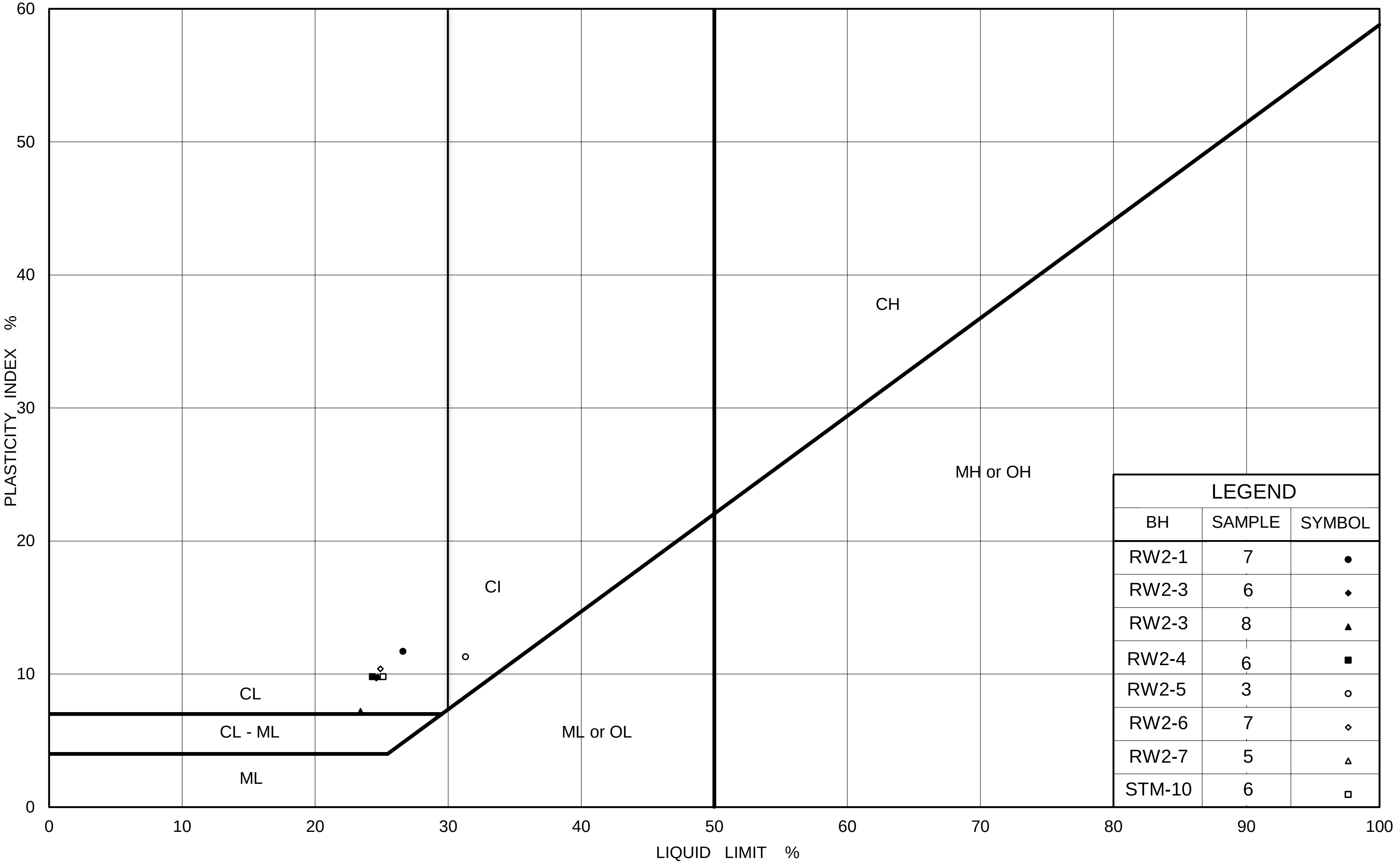
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	RW2-5	7	97.2

Project Number: 1530382

Checked By: ____________________

Golder Associates

Date: 16-May-18



Golder Associates

PLASTICITY CHART

Clayey Silt to Sandy Clayey Silt to Clayey Silt with Sand (Till)

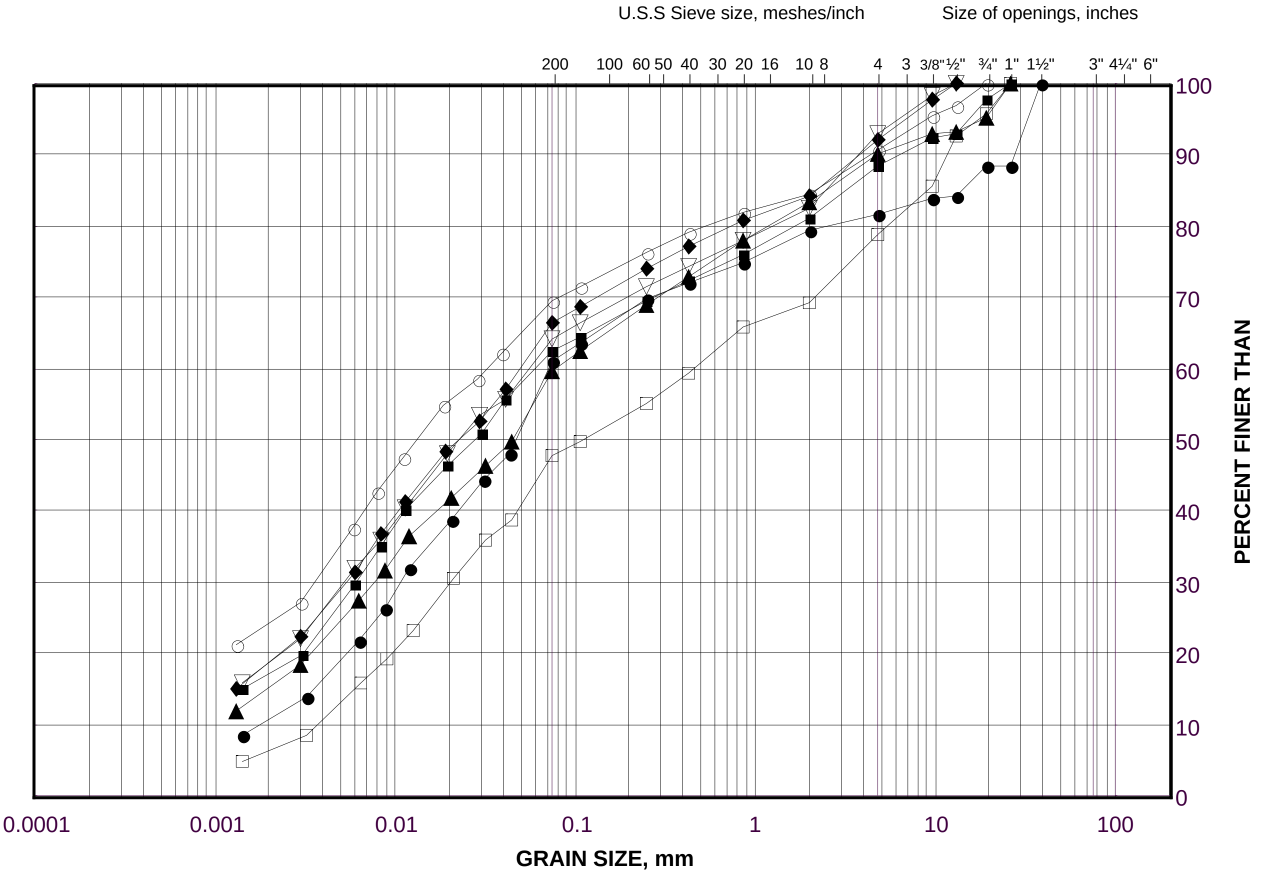
Figure No. A5

Project No. 1530382

Checked By: NK

GRAIN SIZE DISTRIBUTION
Clayey Silt to Sandy Clayey Silt to Clayey Silt with Sand (Till)

FIGURE A6



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

LEGEND

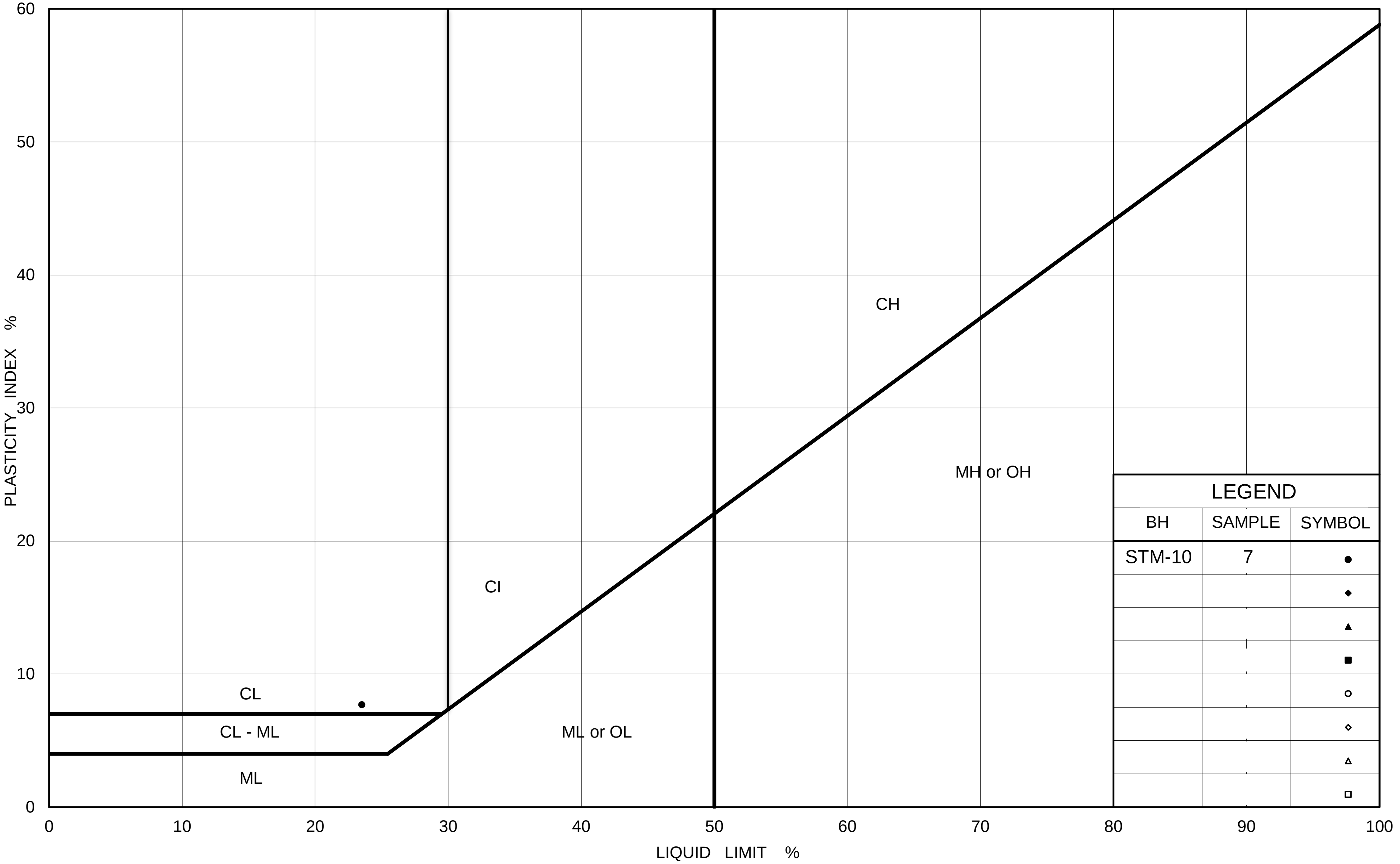
SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	RW2-5	3	100.3
■	RW2-7	5	104.4
◆	RW2-4	6	103.9
▲	RW2-3	6	103.4
▽	RW2-6	7	103.1
❍	RW2-1	7	103.1
□	RW2-3	8	101.1

Project Number: 1530382

Checked By: NK

Golder Associates

Date: 16-May-18



Golder Associates

PLASTICITY CHART
Sandy Clayey Gravel (Residual Soil)

Figure No. A7

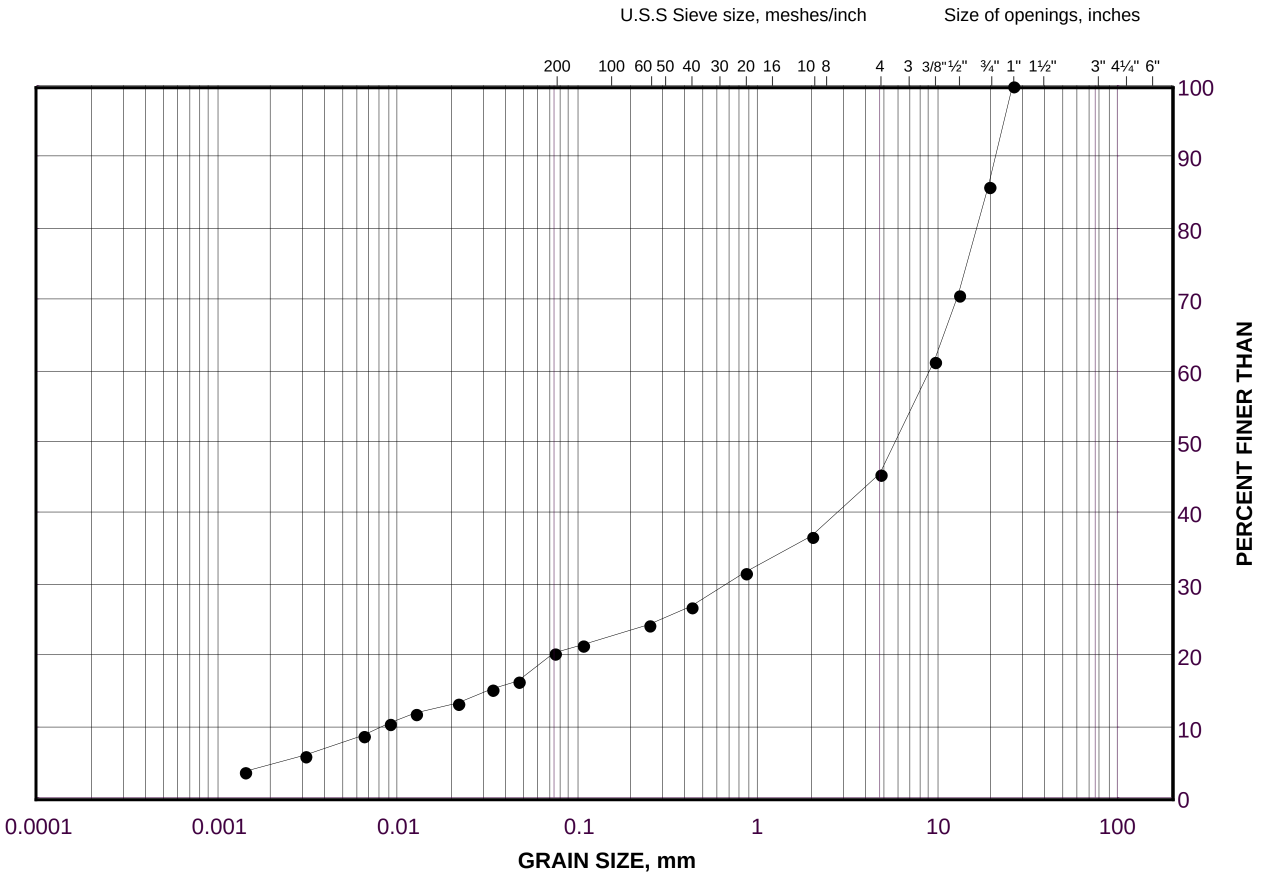
Project No. 1530382

Checked By: NK

GRAIN SIZE DISTRIBUTION

Sandy Clayey Gravel (Residual Soil)

FIGURE A8



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	STM-10	7	101.2

Project Number: 1530382

Checked By: NK

Golder Associates

Date: 16-May-18

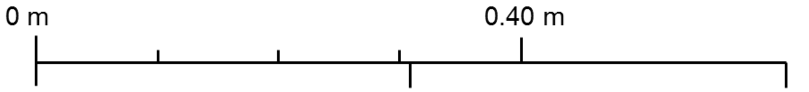
Start of Run No. 1 (8.48 m)

Start of Run No. 2 (9.42 m)

Box 1: 8.48 m to 11.00 m

Start of Run No. 3 (11.00 m)

Box 2: 11.00 m to 12.50 m



Scale

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

PROJECT

QEW IMPROVEMENT FROM EAST OF CAWTHRA TO EAST MALL

TITLE

Bedrock Core Photographs
Borehole RW2-1 (8.48 m to 12.50 m)

Golder Associates

PROJECT No. 1530382			FILE No. ----		
DRAFT	DCB	20180129	SCALE	NTS	VER. 1.
CADD	--		**FIGURE A9A**		
CHECK	ACK	20180227			
REVIEW	NK	20180306			

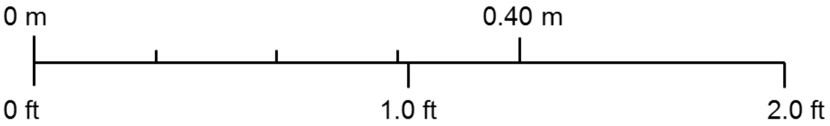
Start of Run No. 4 (12.50 m)

Start of Run No. 5 (14.10 m)

Box 3: 12.50 m to 15.62 m

Start of Run No. 6 (15.62 m)

Box 4:15.62 m to 17.15 m



Scale

PROJECT

QEW IMPROVEMENT FROM EAST OF CAWTHRA TO EAST MALL

TITLE

Bedrock Core Photographs
Borehole RW2-1 (12.50 m to 17.15 m)

Golder Associates

PROJECT No. 1530382			FILE No. ----		
DRAFT	DCB	20180129	SCALE	NTS	VER. 1.
CADD	--		**FIGURE A9B**		
CHECK	ACK	20180227			
REVIEW	NK	20180306			

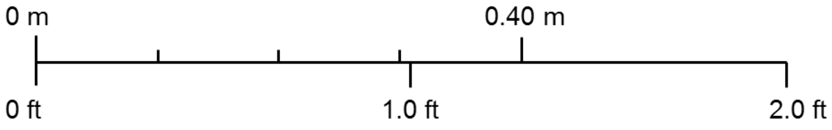
REVISION DATE: January 23, 2018 BY: DCB Project: 1530382

Start of Run No. 7 (17.15 m)

Start of Run No. 8 (18.56 m)

End of Borehole (18.82 m)

Box 5: 17.15 m to 18.82 m (End of Borehole)



Scale

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

PROJECT

QEW IMPROVEMENT FROM EAST OF CAWTHRA TO EAST MALL

TITLE

Bedrock Core Photographs
Borehole RW2-1 (17.15 m to 18.82 m)

Golder Associates

PROJECT No. 1530382			FILE No. ----		
DRAFT	DCB	20180129	SCALE	NTS	VER. 1.
CADD	--		**FIGURE A9C**		
CHECK	ACK	20180227			
REVIEW	NK	20180306			

Start of Run No. 1 (8.48 m)

Start of Run No. 2 (9.50 m)

Start of Run No. 3 (11.02 m)

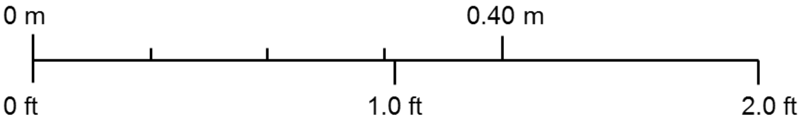
Box 1: 8.48 m to 11.32 m

Start of Run No. 4 (12.42 m)

Start of Run No. 5 (13.94 m)

Box 2: 11.32 m to 14.33 m

Scale



PROJECT

QEW IMPROVEMENT FROM EAST OF CAWTHRA TO EAST MALL

TITLE

Bedrock Core Photographs
Borehole BHRW2-2 (8.48 m to 14.33 m)

PROJECT No. 1530382			FILE No. ----		
DRAFT	DCB	20180123	SCALE	NTS	VER. 1.
CADD	--		**FIGURE A10A**		
CHECK	ACK	20180227			
REVIEW	NK	20180306			

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

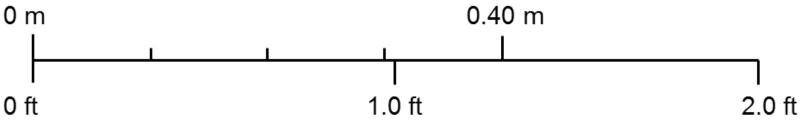
Start of Run No. 6 (15.46 m)

Box 3: 14.33 m to 7.70 m

Start of Run No. 7 (17.09 m)

Box 4: 17.09 m to 18.72 m

Scale



PROJECT

QEW IMPROVEMENT FROM EAST OF CAWTHRA TO EAST MALL

TITLE

Bedrock Core Photographs
Borehole BHRW2-2 (14.33 m to 18.72 m)

Golder Associates

PROJECT No. 1530382			FILE No. ----		
DRAFT	DCB	20180123	SCALE	NTS	VER. 1.
CADD	--		**FIGURE A10B**		
CHECK	ACK	20180227			
REVIEW	NK	20180306			

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

Start of Run No. 1 (7.81 m)

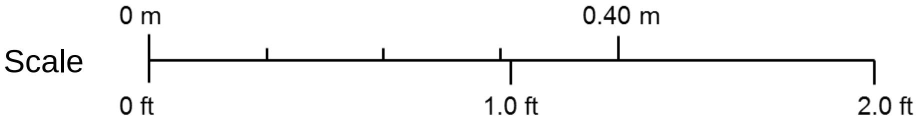
Start of Run No. 2 (9.09 m)

Box 1: 7.81 m to 10.62 m

Start of Run No. 3 (10.62 m)

Start of Run No. 4 (12.17 m)

Box 2: 10.62 m to 13.45 m



PROJECT

QEW IMPROVEMENT FROM EAST OF CAWTHRA TO EAST MALL

TITLE

Bedrock Core Photographs
Borehole BHRW2-3 (7.81 m to 13.45 m)

Golder Associates

PROJECT No. 1530382			FILE No. ----		
DRAFT	DCB	20180123	SCALE	NTS	VER. 1.
CADD	--		**FIGURE A11A**		
CHECK	ACK	20180227			
REVIEW	NK	20180306			

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

Start of Run No. 5 (13.62 m)

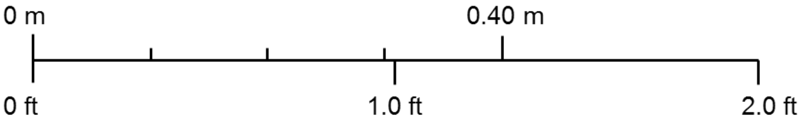
Start of Run No. 6 (15.22 m)

Box 3: 13.45 m to 16.32 m

Start of Run No. 7 (16.74 m)

Box 4: 16.32 m to 17.72 m

Scale



PROJECT

QEW IMPROVEMENT FROM EAST OF CAWTHRA TO EAST MALL

TITLE

Bedrock Core Photographs
Borehole BHRW2-3 (13.45 m to 17.72 m)

Golder Associates

PROJECT No. 1530382			FILE No. ----		
DRAFT	DCB	20180123	SCALE	NTS	VER. 1.
CADD	--		**FIGURE A11B**		
CHECK	ACK	20180227			
REVIEW	NK	20180306			

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

Start of Run No. 1 (7.05 m) – After Split Spoon Sampling

Start of Run No. 2 (8.00 m)

Box 1: 7.05 m to 9.55 m

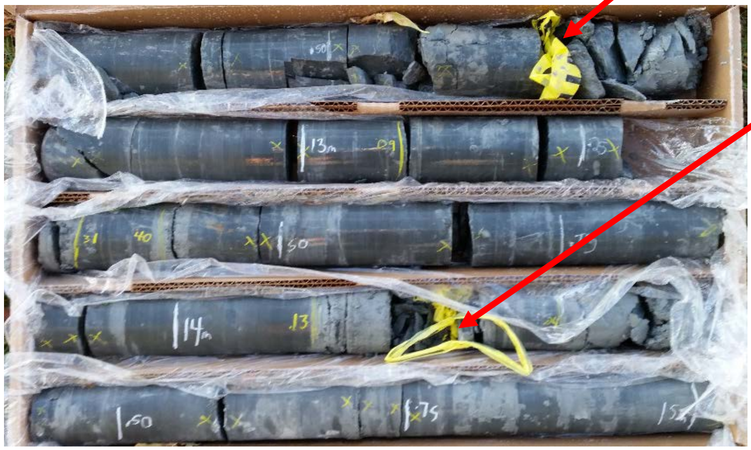
Start of Run No. 3 (9.55 m)

Start of Run No. 4 (11.10 m)

Box 2: 9.55 m to 12.26 m

Start of Run No. 5 (12.66 m)

Start of Run No. 6 (14.17 m)

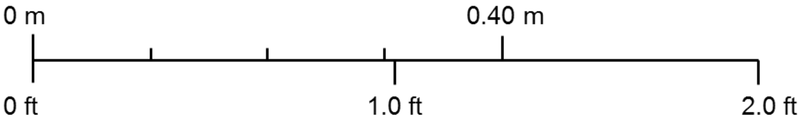


Box 3: 12.26 m to 15.03 m

Start of Run No. 7 (15.69 m)

Box 4: 15.03 m to 17.22 m

Scale



PROJECT

QEW IMPROVEMENT FROM EAST OF CAWTHRA TO EAST MALL

TITLE

Bedrock Core Photographs
Borehole BHRW2-4 (7.05 m to 17.22 m)

Golder Associates

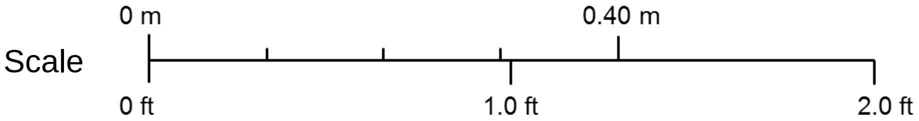
PROJECT No. 1530382			FILE No. ----		
DRAFT	DCB	20180123	SCALE	NTS	VER. 1.
CADD	--		**FIGURE A12A**		
CHECK	ACK	20180227			
REVIEW	NK	20180306			

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

Start of Run No. 8 (17.22 m)

End of Borehole (18.62 m)

Box 5: 17.22 m to 18.62 m (End of Borehole)



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

PROJECT

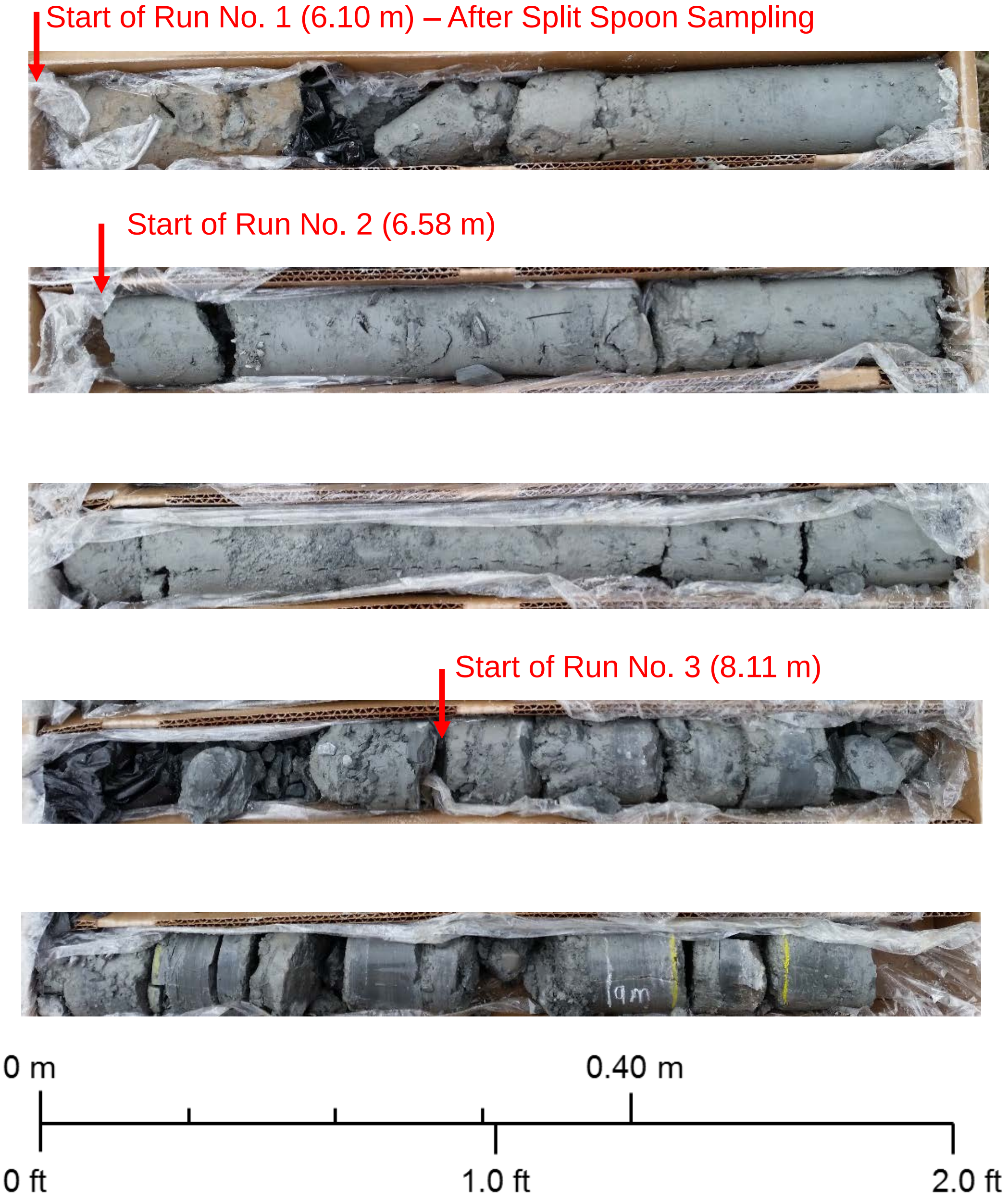
QEW IMPROVEMENT FROM EAST OF CAWTHRA TO EAST MALL

TITLE

Bedrock Core Photographs
Borehole BHRW2-4 (17.22 m to 18.62 m)

Golder Associates

PROJECT No. 1530382			FILE No. ----	
DRAFT	DCB	20180123	SCALE NTS	VER. 1.
CADD	--		**FIGURE A12B**	
CHECK	ACK	20180227		
REVIEW	NK	20180306		



Scale

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

PROJECT

QEW IMPROVEMENT FROM EAST OF CAWTHRA TO EAST MALL

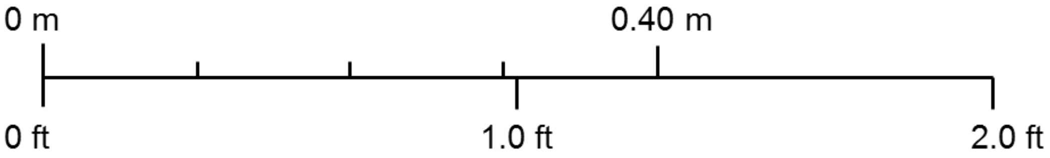
TITLE

Bedrock Core Photographs
Borehole RW2-6 (6.10 m to 9.63 m)

Golder Associates

PROJECT No. 1530382			FILE No. ----		
DRAFT	DCB	20180129	SCALE	NTS	VER. 1.
CADD	--		FIGURE A13A		
CHECK	ACK	20180227			
REVIEW	NK	20180306			

Start of Run No. 4 (9.63 m)



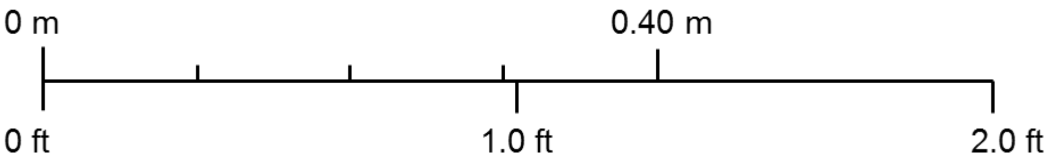
Scale

PROJECT	QEW IMPROVEMENT FROM EAST OF CAWTHRA TO EAST MALL				
TITLE	Bedrock Core Photographs Borehole RW2-6 (9.63 m to 11.16 m)				
Golder Associates	PROJECT No. 1530382			FILE No. ----	
	DRAFT	DCB	20180129	SCALE NTS	VER. 1.
	CADD	--		FIGURE A13B	
	CHECK	ACK	20180227		
	REVIEW	NK	20180306		

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

Start of Run No. 5 (11.16 m)

End of Borehole (12.68 m)



Scale

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

PROJECT

QEW IMPROVEMENT FROM EAST OF CAWTHRA TO EAST MALL

TITLE

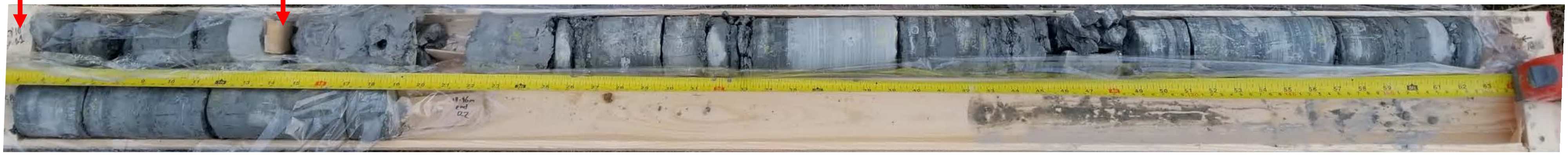
Bedrock Core Photographs
Borehole RW2-6 (11.16 m to 12.68 m)

Golder Associates

PROJECT No. 1530382			FILE No. ----	
DRAFT	DCB	20180129	SCALE NTS	VER. 1.
CADD	--		**FIGURE A13C**	
CHECK	ACK	20180227		
REVIEW	NK	20180306		

Start of Run No. 1 (9.09 m)

Start of Run No. 2 (9.35 m)



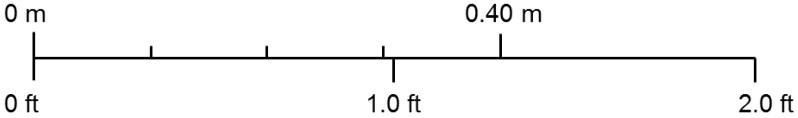
Box 1: 9.09 m to 10.96 m

Start of Run No. 3 (10.96 m)

Start of Run No. 4 (12.50 m)

End of Box 2 (14.07 m)

Box 2: 10.96 m to 14.07 m



Scale

PROJECT **QEW IMPROVEMENT FROM EAST OF CAWTHRA TO EAST MALL**					
TITLE **Bedrock Core Photographs** **Borehole RW2-7 (9.09 m to 14.07 m)**					
PROJECT No. 1530382			FILE No. ----		
DRAFT	DCB	20180129	SCALE	NTS	VER. 1.
CADD	--		**FIGURE A14A**		
CHECK	ACK	20180227			
REVIEW	NK	20180306			

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

Start of Run No. 5 (14.07 m)

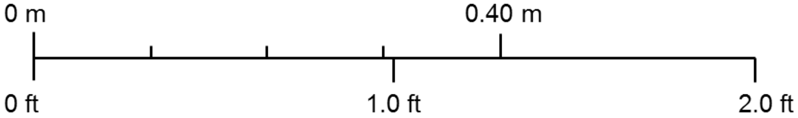
Start of Run No. 6 (15.58 m)

Box 3: 14.07 m to 17.12 m

Start of Run No. 7 (17.12 m)

Start of Run No. 8 (18.62 m)

Box 4: 17.12 m to 20.16 m (End of Borehole)



Scale

PROJECT

QEW IMPROVEMENT FROM EAST OF CAWTHRA TO EAST MALL

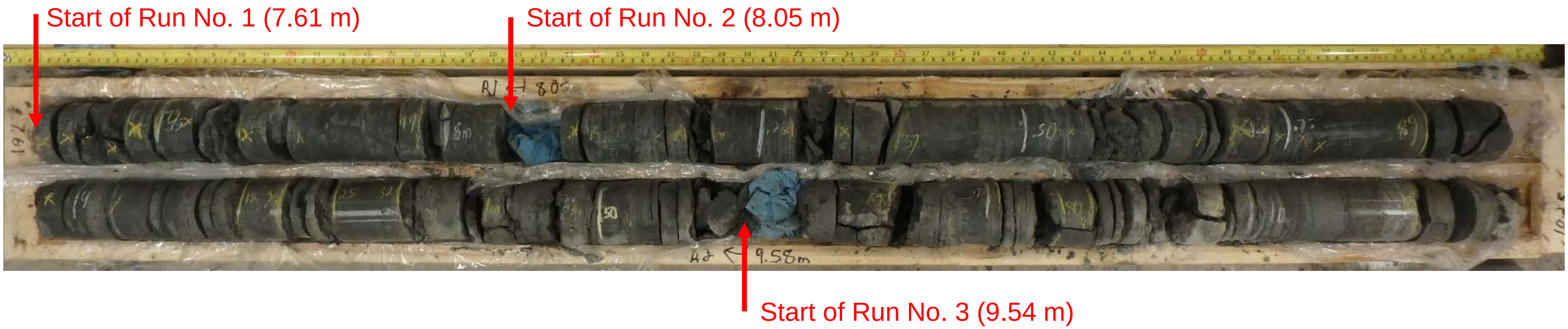
TITLE

Bedrock Core Photographs
Borehole RW2-7 (14.07 m to 20.16 m)

PROJECT No. 1530382			FILE No. ----	
DRAFT	DCB	20180129	SCALE NTS	VER. 1.
CADD	--		**FIGURE A14B**	
CHECK	ACK	20180227		
REVIEW	NK	20180306		

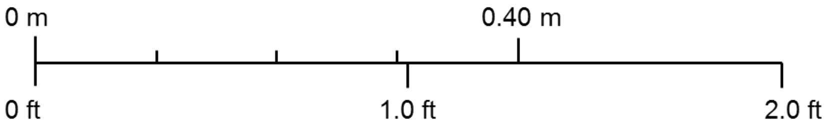
Golder Associates

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



Box 1: 7.61 m to 10.25 m

Box 2: 10.25 m to 13.13 m



Scale

PROJECT

QEW IMPROVEMENT FROM EAST OF CAWTHRA TO EAST MALL

TITLE

Bedrock Core Photographs
Borehole STM-10 (7.61 m to 13.13 m)

PROJECT No. 1530382			FILE No. ----		
DRAFT	DCB	20180129	SCALE	NTS	VER. 1.
CADD	--		**FIGURE A15A**		
CHECK	ACK	20180227			
REVIEW	NK	20180306			

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

Start of Run No. 6 (14.30 m)

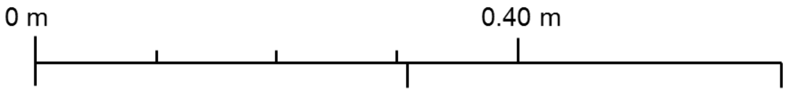
End of Run No. 6 (15.79 m)

Box 3: 13.13 m to 15.79 m

Start of Run No. 7 (15.79 m)

End of Hole (17.38 m)

Box 4: 15.79 m to 17.38 m (End of Borehole)



Scale

PROJECT	QEW IMPROVEMENT FROM EAST OF CAWTHRA TO EAST MALL				
TITLE	Bedrock Core Photographs Borehole STM-10 (13.13 m to 17.38 m)				
Golder Associates	PROJECT No. 1530382			FILE No. ----	
	DRAFT	DCB	20180129	SCALE NTS	VER. 1.
	CADD	--		FIGURE A15B	
	CHECK	ACK	20180227		
	REVIEW	NK	20180306		

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

APPENDIX B

Retaining Wall No. 24-888/W, QEW – Station 13+749 to 13+859 Record of Borehole/Drillhole Sheets, Laboratory Test Results and Bedrock Core Photographs

RECORD OF BOREHOLE No RW3-1

SHEET 1 OF 1

METRIC

PROJECT 1530382

G.W.P. 2102-13-00; 2432-13-00

DIST Central HWY QEW

DATUM Geodetic

LOCATION N 4829216.9; E 299819.4 MTM NAD 83 ZONE 10 (LAT. 43.603220; LONG. -79.561302)

BOREHOLE TYPE 108 mm O.D. Continuous Flight Solid Stem Augers

DATE September 18, 2016

ORIGINATED BY PKS

COMPILED BY AJ

CHECKED BY SMM

SOIL PROFILE			SAMPLES					DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE				GR	SA	SI	CL
104.3	GROUND SURFACE													
0.0	ASPHALT (50 mm)													
104.1	CONCRETE (180 mm)													
0.2	Sand and gravel (FILL) Brown Moist						104							
103.7 0.6	CLAYEY SILT, some sand, trace gravel to CLAYEY SILT with SAND, some gravel (TILL) Very stiff to hard Grey Moist		1	SS	23									
							103							
			2	SS	35						12	32	41	15
							102							
			3	SS	31									
							101							
			4	SS	31									
			5	SS	21									
							100							
	- Shale fragments at a depth of 4.6 m		6	SS	26									
	- Auger grinding at a depth of 5.5 m						99							
98.8 5.5	SHALE (BEDROCK)													
			7	SS	100/0.15		98							
97.9 6.4	END OF BOREHOLE NOTE: 1. Open borehole dry upon completion of drilling.													

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

○ $^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No RW3-2

SHEET 1 OF 1

METRIC

PROJECT 1530382			
G.W.P. 2102-13-00; 2432-13-00	LOCATION N 4829255.6; E 299851.0 MTM NAD 83 ZONE 10 (LAT. 43.603408; LONG. -79.561999)		ORIGINATED BY PKS
DIST Central	HWY QEW	BOREHOLE TYPE 108 mm O.D. Continuous Flight Solid Stem Augers	COMPILED BY AJ
DATUM Geodetic		DATE September 18, 2016	CHECKED BY SMM

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
103.0	GROUND SURFACE													
0.0	ASPHALT (75 mm)													
102.8	CONCRETE (180 mm)													
0.3 102.5	Sand and gravel (FILL) Brown Moist													
0.6	CLAYEY SILT to CLAYEY SILT with SAND and GRAVEL (TILL) Hard Grey Moist		1	SS	34		102							
			2	SS	43		101							
	- Shale fragments below a depth of 2.3 m		3	SS	42		100							
			4	SS	135/0.25						32	35	26	7
99.3 3.7	SHALE (BEDROCK)		5	SS	100/0.10									
98.9			6	SS	100/0.10		99							
4.1	END OF BOREHOLE													

NOTE:

1. Open borehole dry upon completion of drilling.

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

○ 3% STRAIN AT FAILURE

GOLDER

Foundation Design

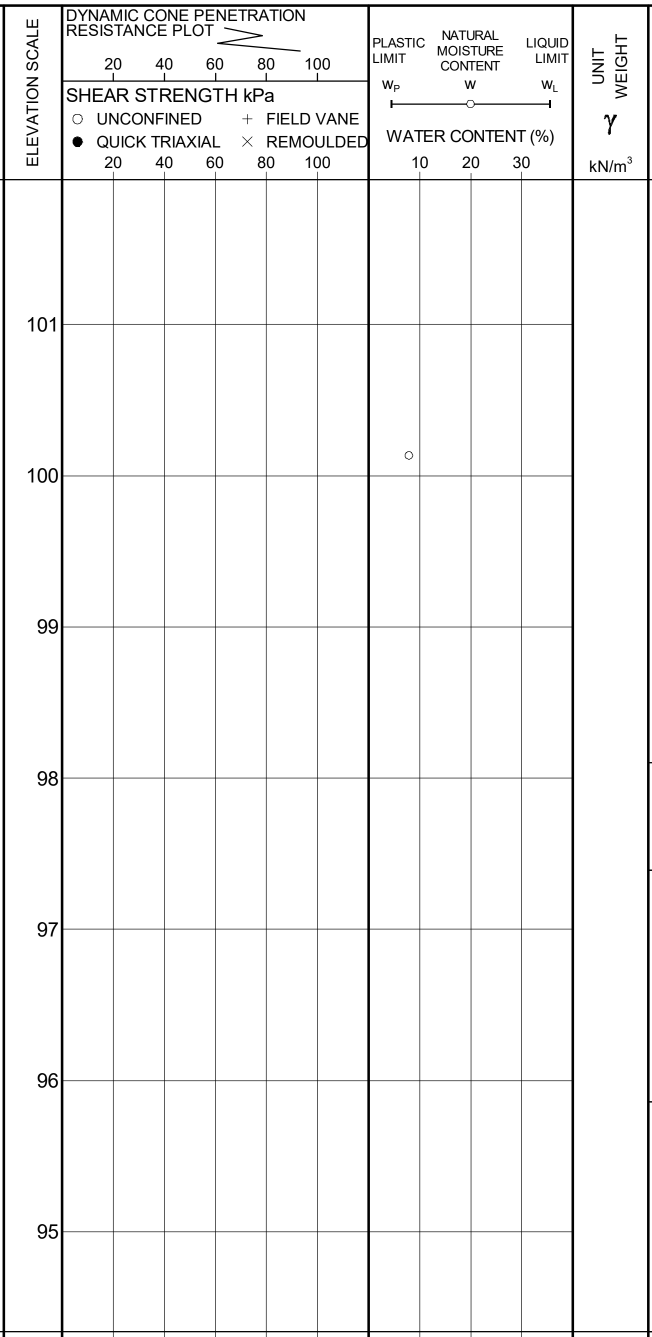
RECORD OF BOREHOLE No RW3-3

SHEET 1 OF 1

METRIC

PROJECT 1530382
G.W.P. 2102-13-00; 2432-13-00
DIST Central HWY QEW
DATUM Geodetic
LOCATION N 4829295.7; E 299881.0 MTM NAD 83 ZONE 10 (LAT. 43.603581; LONG. -79.560932)
BOREHOLE TYPE 108 mm O.D. Solid Stem Augers
DATE September 19, 2016
ORIGINATED BY PKS
COMPILED BY ACK
CHECKED BY SMM

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
102.0	GROUND SURFACE					
0.0	ASPHALT (200 mm)					
101.8 0.2	Sand, some silt, trace gravel (FILL) Loose Brown Moist	1	SS	8		
100.8 1.2	Silt, sand and gravel, trace clay, some shale fragments (FILL) Compact Grey Moist	2	SS	28		32 31 32 5
99.7 2.3	SHALE (BEDROCK)	3	SS	100/0.10		
		4	SS	100/0.10		
		5	SS	100/0.05		
	Bedrock cored from depths of 3.8 m to 7.6 m. For bedrock coring details refer to Record of Drillhole RW3-3.	1	RC	REC 72%		RQD = 15%
		2	RC	REC 100%		RQD = 79%
		3	RC	REC 96%		RQD = 87%
94.4 7.6	END OF BOREHOLE NOTE: 1. Open borehole dry upon completion of drilling prior to rock coring.					



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

GTA-MTO 001 S:\CLIENTS\MTO\QEW-DIXIE\02_DATA\GINT\QEW-DIXIE.GPJ GAL-GTA.GDT 18-5-9 GPK

PROJECT: 1530382

RECORD OF DRILLHOLE: RW3-3

SHEET 1 OF 1

LOCATION: N 4829295.7 ;E 299881.0

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: September 19, 2016

DRILL RIG: CME 75 (Truck Mounted)

DRILLING CONTRACTOR: Davis Drilling Ltd.

DATUM: Geodetic

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

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

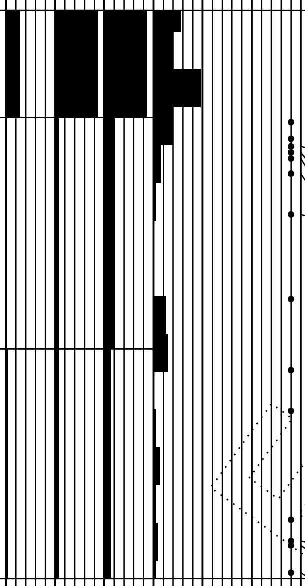
PL - Planar
CU - Curved
UN - Undulating
ST - Stepped
IR - Irregular

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

BR - Broken Rock

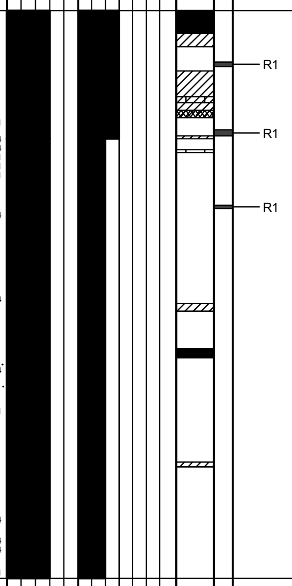
NOTE: For additional abbreviations refer to list of abbreviations & symbols.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. DEPTH (m)	RUN No.
		Continued from Record of Borehole RW3-3	98.10	
4	Rotary Drill NQ Core	Moderately weathered, thinly laminated, grey, very fine grained, non-porous, weak SHALE with medium strong limestone interbeds [Georgian Bay Formation]	3.86	1
5		Slightly weathered, thinly laminated, grey, very fine to fine grained, non-porous, weak SHALE [Georgian Bay Formation]	97.25 4.71	2
6				
7				3
		END OF DRILLHOLE	94.34 7.62	
8				
9				
10				
11				
12				



DISCONTINUITY DATA

TYPE AND SURFACE DESCRIPTION	Jr	Ja
BD,PL,RO	1.5	1
BD,PL,SM CC, Br	1	4
BD,PL,SM PC, Cl+Br	1	4
CO,UN,RO	3	1
BD,PL,SM	1	1
BD,PL,SM	1	1
BD,PL,SM CC, Br	1	4
BD,UN,RO CC, Cl+Br	3	4
BD,PL,SM CC, Br	1	4
BD,UN,SM SA	2	1
BD,PL,SM PC, Cl+Br	1	4
BD,IR,RO PC, Cl	3	4
BD,PL,SM PC, Cl+Br	1	4
BD,IR,SM SA	2	1



NOTES
WATER LEVELS
INSTRUMENTATION

FEATURES LEGEND

▨ BROKEN CORE	▦ CLAY SEAM	▤ LIMESTONE	■ LOST CORE

DEPTH SCALE
1 : 50

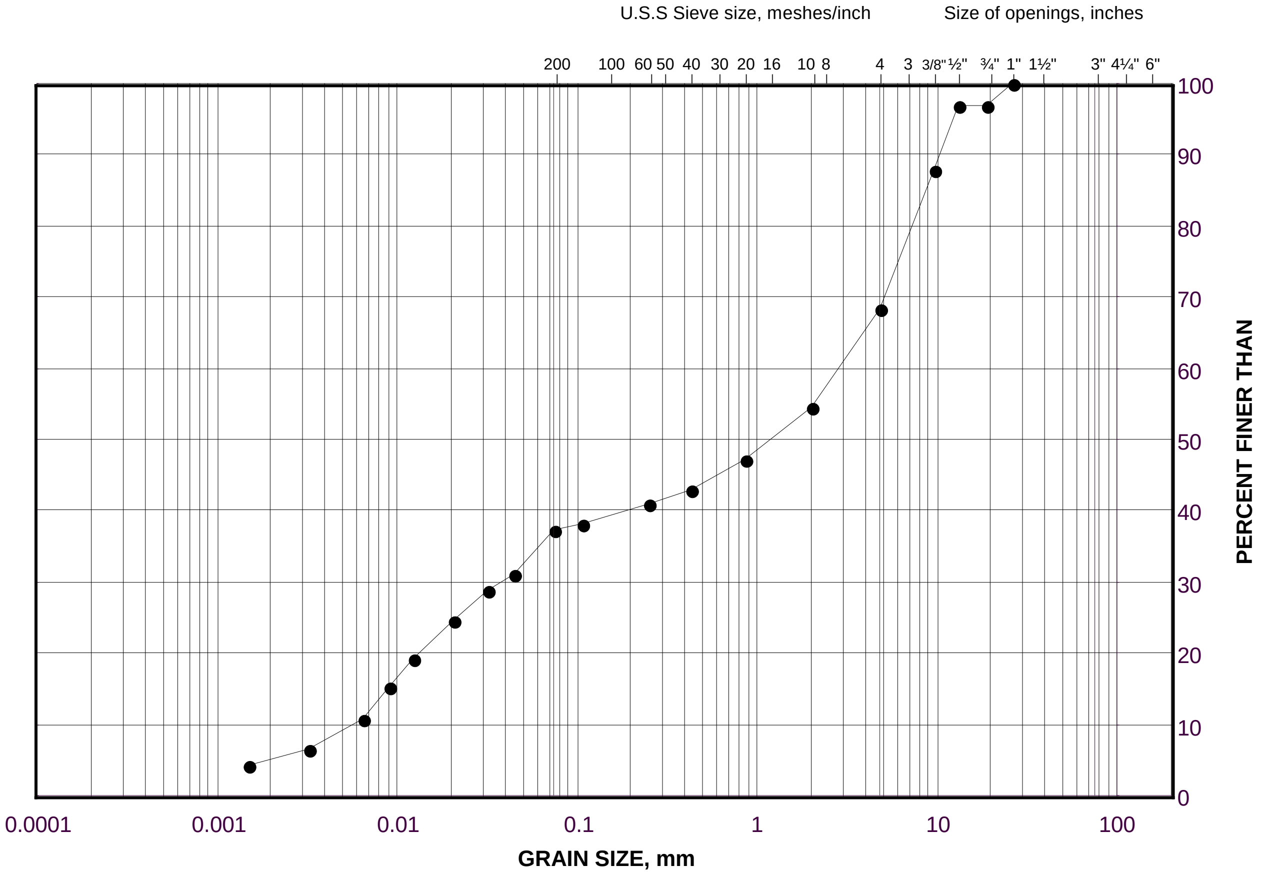
Golder Associates

LOGGED: PKS
CHECKED: CEC/AB

GRAIN SIZE DISTRIBUTION

Gravelly Silty Sand (Fill)

FIGURE B1



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	RW3-3	2	100.2

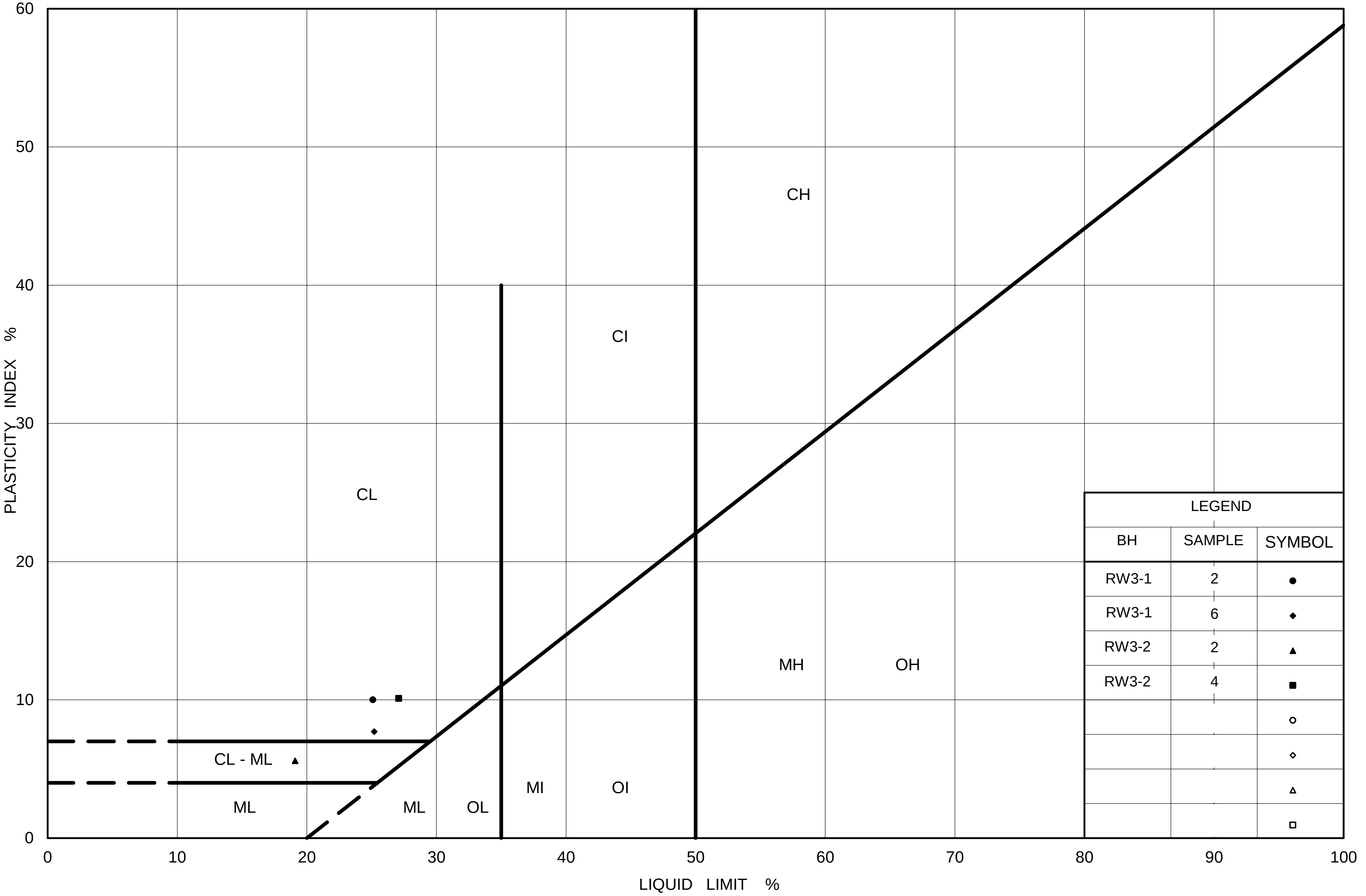
Project Number: 1530382

Checked By: NK

Golder Associates

Date: 27-Feb-18

Oct 75, FF-S-21



BH	SAMPLE	SYMBOL
RW3-1	2	●
RW3-1	6	◆
RW3-2	2	▲
RW3-2	4	■
		○
		◇
		△
		□

Ministry of Transportation

Ontario

PLASTICITY CHART
Clayey Silt with Sand to Clayey Silt with Sand and Gravel (Till)

Figure No. B2

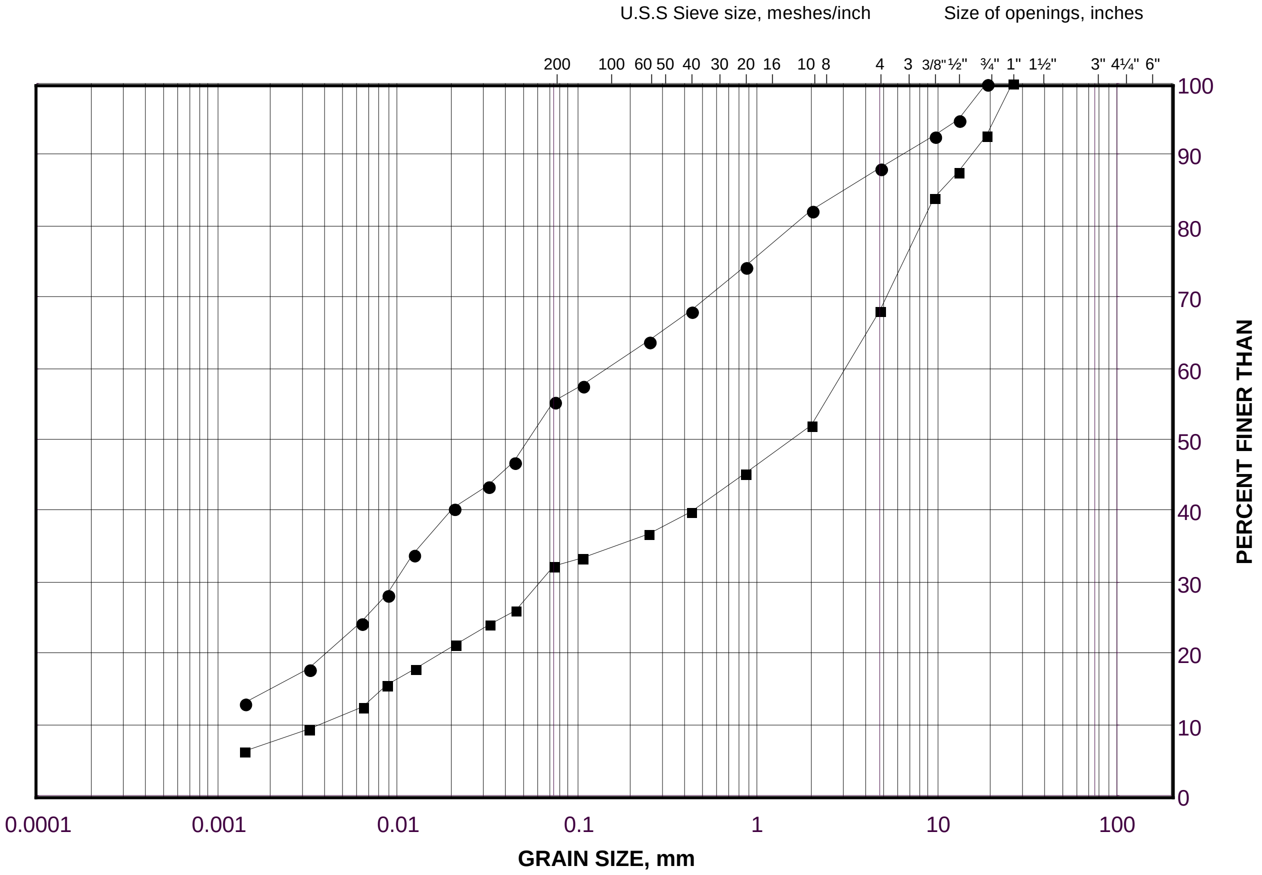
Project No. 1530382

Checked By:NK

GRAIN SIZE DISTRIBUTION

Clayey Silt with Sand to Clayey Silt with Sand and Gravel (Till)

FIGURE B3



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	RW3-1	2	102.5
■	RW3-2	4	99.8

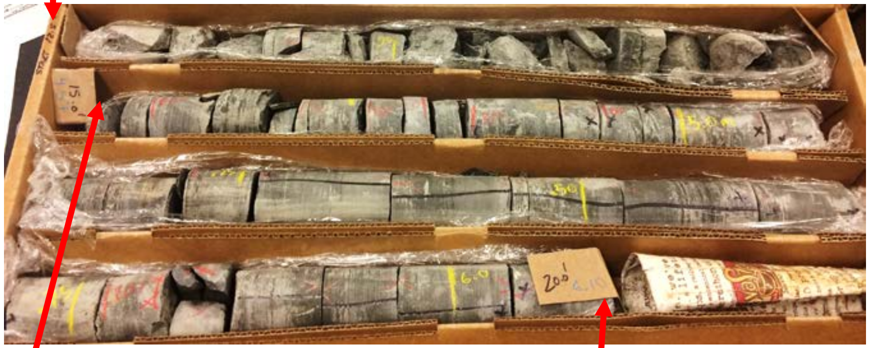
Project Number: 1530382

Checked By: NK

Golder Associates

Date: 27-Feb-18

Start of Run No. 1 (3.86 m)

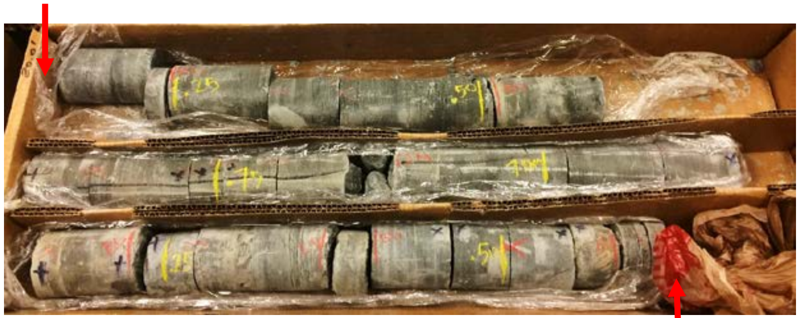


Start of Run No. 2 (4.57 m)

End of Run No. 2 (6.10 m)

Box 1: 3.86 m to 6.10 m

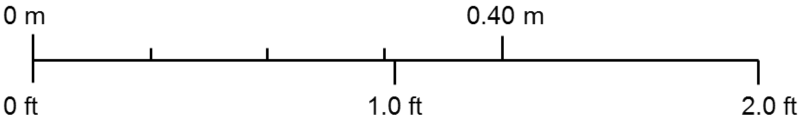
Start of Run No. 3 (6.10 m)



End of Run No. 3 (7.62 m)

Box 2: 6.10 m to 7.62 m

Scale



PROJECT

QEW IMPROVEMENT FROM EAST OF CAWTHRA TO EAST MALL

TITLE

Bedrock Core Photographs
Borehole RW3-3 (7.05 m to 18.62 m)

Golder Associates

PROJECT No. 1530382			FILE No. ----		
DRAFT	DCB	20180123	SCALE	NTS	VER. 1.
CADD	--		**FIGURE B4**		
CHECK	ACK	20180227			
REVIEW	JMAC	201802XX			

REVISION DATE:February 27, 2018 BY: AK Project: 1530382

APPENDIX C

Analytical Test Results

Your Project #: 1530382
Site Location: QEW-CAWTHRA
Your C.O.C. #: 70344

Attention:Alysha Kobylinski

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

Report Date: 2016/11/19
Report #: R4252452
Version: 1 - Final

CERTIFICATE OF ANALYSIS

MAXXAM JOB #: B6O5411
Received: 2016/11/10, 17:14

Sample Matrix: SOLID
Samples Received: 5

Analyses	Quantity	Date Extracted	Date Analyzed	Laboratory Method	Reference
Chloride (20:1 extract)	5	N/A	2016/11/16	CAM SOP-00463	EPA 325.2 m
Conductivity	5	N/A	2016/11/16	CAM SOP-00414	OMOE E3530 v1 m
pH CaCl2 EXTRACT	5	2016/11/16	2016/11/16	CAM SOP-00413	EPA 9045 D m
Resistivity of Soil	5	2016/11/10	2016/11/17	CAM SOP-00414	SM 22 2510 m
Sulphate (20:1 Extract)	5	N/A	2016/11/16	CAM SOP-00464	EPA 375.4 m

Remarks:

Maxxam Analytics' laboratories are accredited to ISO/IEC 17025:2005 for specific parameters on scopes of accreditation. Unless otherwise noted, procedures used by Maxxam are based upon recognized Provincial, Federal or US method compendia such as CCME, MDDELCC, EPA, APHA.

All work recorded herein has been done in accordance with procedures and practices ordinarily exercised by professionals in Maxxam's profession using accepted testing methodologies, quality assurance and quality control procedures (except where otherwise agreed by the client and Maxxam in writing). All data is in statistical control and has met quality control and method performance criteria unless otherwise noted. All method blanks are reported: unless indicated otherwise, associated sample data are not blank corrected.

Maxxam Analytics' liability is limited to the actual cost of the requested analyses, unless otherwise agreed in writing. There is no other warranty expressed or implied. Maxxam has been retained to provide analysis of samples provided by the Client using the testing methodology referenced in this report. Interpretation and use of test results are the sole responsibility of the Client and are not within the scope of services provided by Maxxam, unless otherwise agreed in writing.

Solid sample results, except biota, are based on dry weight unless otherwise indicated. Organic analyses are not recovery corrected except for isotope dilution methods. Results relate to samples tested.
This Certificate shall not be reproduced except in full, without the written approval of the laboratory.
Reference Method suffix "m" indicates test methods incorporate validated modifications from specific reference methods to improve performance.
* RPDs calculated using raw data. The rounding of final results may result in the apparent difference.

Your Project #: 1530382
Site Location: QEW-CAWTHRA
Your C.O.C. #: 70344

Attention:Alysha Kobylinski

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

Report Date: 2016/11/19
Report #: R4252452
Version: 1 - Final

CERTIFICATE OF ANALYSIS

MAXXAM JOB #: B6O5411
Received: 2016/11/10, 17:14

Encryption Key

Please direct all questions regarding this Certificate of Analysis to your Project Manager.
Ema Gitej, Senior Project Manager
Email: EGitej@maxxam.ca
Phone# (905)817-5829
==
Maxxam has procedures in place to guard against improper use of the electronic signature and have the required "signatories", as per section 5.10.2 of ISO/IEC 17025:2005(E), signing the reports. For Service Group specific validation please refer to the Validation Signature Page.

Total Cover Pages : 2

RESULTS OF ANALYSES OF SOLID

Maxxam ID		DKV715	DKV715		DKV716		
Sampling Date		2016/11/03	2016/11/03		2016/11/10		
COC Number		70344	70344		70344		
	UNITS	**RW3-3-4.33M-4.43M**	**RW3-3-4.33M-4.43M Lab-Dup**	**QC Batch**	**OHS-4-SA4-2.29M-2.59M**	**RDL**	**QC Batch**
Calculated Parameters							
Resistivity	ohm-cm	2000		4745989	850		4745989
Inorganics							
Soluble (20:1) Chloride (Cl)	ug/g	<20		4748291	500	20	4748291
Conductivity	umho/cm	499		4749169	1180	2	4749169
Available (CaCl2) pH	pH	8.18		4750330	7.92		4750333
Soluble (20:1) Sulphate (SO4)	ug/g	250	230	4748348	270	20	4748348

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

Maxxam ID		DKV716		DKV717	DKV718		
Sampling Date		2016/11/10		2016/11/10	2016/11/03		
COC Number		70344		70344	70344		
	UNITS	**OHS-4-SA4-2.29M-2.59M Lab-Dup**	**QC Batch**	**OHS-5-SA5-3.81M-4.42M**	**CV01-01-8.74M-8.80M**	**RDL**	**QC Batch**
Calculated Parameters							
Resistivity	ohm-cm		4745989	1400	1000		4745989
Inorganics							
Soluble (20:1) Chloride (Cl)	ug/g		4748291	40	260	20	4748291
Conductivity	umho/cm		4749169	720	965	2	4749169
Available (CaCl2) pH	pH	7.90	4750333	7.86	8.14		4750330
Soluble (20:1) Sulphate (SO4)	ug/g		4748348	560	320	20	4748348

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

RESULTS OF ANALYSES OF SOLID

Maxxam ID		DKV719		
Sampling Date		2016/11/03		
COC Number		70344		
	UNITS	**CV02/3-1-5.27M-5.32M**	**RDL**	**QC Batch**
Calculated Parameters				
Resistivity	ohm-cm	1500		4745989
Inorganics				
Soluble (20:1) Chloride (Cl)	ug/g	100	20	4748291
Conductivity	umho/cm	682	2	4749169
Available (CaCl2) pH	pH	8.01		4750330
Soluble (20:1) Sulphate (SO4)	ug/g	250	20	4748348

RDL = Reportable Detection Limit
QC Batch = Quality Control Batch

TEST SUMMARY

Maxxam ID: DKV715
Sample ID: RW3-3-4.33M-4.43M
Matrix: SOLID

Collected: 2016/11/03
Shipped:
Received: 2016/11/10

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	4748291	N/A	2016/11/16	Alina Dobreanu
Conductivity	AT	4749169	N/A	2016/11/16	Tahir Anwar
pH CaCl2 EXTRACT	AT	4750330	2016/11/16	2016/11/16	Neil Dassanayake
Resistivity of Soil		4745989	2016/11/17	2016/11/17	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	4748348	N/A	2016/11/16	Deonarine Ramnarine

Maxxam ID: DKV715 Dup
Sample ID: RW3-3-4.33M-4.43M
Matrix: SOLID

Collected: 2016/11/03
Shipped:
Received: 2016/11/10

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Sulphate (20:1 Extract)	KONE/EC	4748348	N/A	2016/11/16	Deonarine Ramnarine

Maxxam ID: DKV716
Sample ID: OHS-4-SA4-2.29M-2.59M
Matrix: SOLID

Collected: 2016/11/10
Shipped:
Received: 2016/11/10

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	4748291	N/A	2016/11/16	Alina Dobreanu
Conductivity	AT	4749169	N/A	2016/11/16	Tahir Anwar
pH CaCl2 EXTRACT	AT	4750333	2016/11/16	2016/11/16	Neil Dassanayake
Resistivity of Soil		4745989	2016/11/17	2016/11/17	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	4748348	N/A	2016/11/16	Deonarine Ramnarine

Maxxam ID: DKV716 Dup
Sample ID: OHS-4-SA4-2.29M-2.59M
Matrix: SOLID

Collected: 2016/11/10
Shipped:
Received: 2016/11/10

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
pH CaCl2 EXTRACT	AT	4750333	2016/11/16	2016/11/16	Neil Dassanayake

Maxxam ID: DKV717
Sample ID: OHS-5-SA5-3.81M-4.42M
Matrix: SOLID

Collected: 2016/11/10
Shipped:
Received: 2016/11/10

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	4748291	N/A	2016/11/16	Alina Dobreanu
Conductivity	AT	4749169	N/A	2016/11/16	Tahir Anwar
pH CaCl2 EXTRACT	AT	4750330	2016/11/16	2016/11/16	Neil Dassanayake
Resistivity of Soil		4745989	2016/11/17	2016/11/17	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	4748348	N/A	2016/11/16	Deonarine Ramnarine

TEST SUMMARY

Maxxam ID: DKV718
Sample ID: CV01-01-8.74M-8.80M
Matrix: SOLID

Collected: 2016/11/03
Shipped:
Received: 2016/11/10

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	4748291	N/A	2016/11/16	Alina Dobreanu
Conductivity	AT	4749169	N/A	2016/11/16	Tahir Anwar
pH CaCl2 EXTRACT	AT	4750330	2016/11/16	2016/11/16	Neil Dassanayake
Resistivity of Soil		4745989	2016/11/17	2016/11/17	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	4748348	N/A	2016/11/16	Deonarine Ramnarine

Maxxam ID: DKV719
Sample ID: CV02/3-1-5.27M-5.32M
Matrix: SOLID

Collected: 2016/11/03
Shipped:
Received: 2016/11/10

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	4748291	N/A	2016/11/16	Alina Dobreanu
Conductivity	AT	4749169	N/A	2016/11/16	Tahir Anwar
pH CaCl2 EXTRACT	AT	4750330	2016/11/16	2016/11/16	Neil Dassanayake
Resistivity of Soil		4745989	2016/11/17	2016/11/17	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	4748348	N/A	2016/11/16	Deonarine Ramnarine

GENERAL COMMENTS

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

Package 1	14.0°C

Results relate only to the items tested.

QUALITY ASSURANCE REPORT

			Matrix Spike		SPIKED BLANK		Method Blank		RPD	
QC Batch	Parameter	Date	% Recovery	QC Limits	% Recovery	QC Limits	Value	UNITS	Value (%)	QC Limits
4748291	Soluble (20:1) Chloride (Cl)	2016/11/16	NC	70 - 130	108	70 - 130	<20	ug/g	0.49	35
4748348	Soluble (20:1) Sulphate (SO4)	2016/11/16	NC	70 - 130	107	70 - 130	<20	ug/g	9.4	35
4749169	Conductivity	2016/11/16			99	90 - 110	<2	umho/cm	0.93	10
4750330	Available (CaCl2) pH	2016/11/16			99	97 - 103			0.28	N/A
4750333	Available (CaCl2) pH	2016/11/16			99	97 - 103			0.26	N/A

N/A = Not Applicable

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

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

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

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

NC (Matrix Spike): The recovery in the matrix spike was not calculated. The relative difference between the concentration in the parent sample and the spiked amount was too small to permit a reliable recovery calculation (matrix spike concentration was less than 2x that of the native sample concentration).

VALIDATION SIGNATURE PAGE

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

Eva Pranjic

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

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

Maxxam IMMEDIATE TEST 6740 Campobello Road, Mississauga, ON L5N 2L8
Phone: 905-817-5700 Fax: 905-817-5779 Toll Free: (800) 563-6266

CHAIN OF CUSTODY RECORD

70344

Page 1 of 1

INVOICE INFORMATION

Company Name: Golder Associates
Contact Name: Alysha Kobylinski
Address: 6925 CENTURY AVE, SUITE 100
Mississauga, ON
Phone: 647-618-1364 Fax: 905-567-6561
Email: Alysha_Kobylinski@golder.com

REPORT INFORMATION (if differs from invoice)

Company Name:
Contact Name:
Address:
Phone: Fax:
Email:

PROJECT INFORMATION

Quotation #:
P.O. #:
Project #: 1530382
Site Location: QEW - CAWTHRA
Site #:
Sampled By:

MAXXAM JOB NUMBER

CHAIN OF CUSTODY #

00

Note: For MOE Regulated Drinking Water samples, please use the Drinking Water CofC.

Regulation 153 (2011)

☐ Table 1 ☐ Res/Park ☐ Med/Fine
☐ Table 2 ☐ Ind/Comm ☐ Coarse
☐ Table 3 ☐ Agri/Other
☐ Table ____

For RSC
☐ Yes
☒ No

Other Regulations

☐ CCME ☐ Sanitary Sewer Bylaw
☐ Reg. 558 ☐ Storm Sewer Bylaw
☐ MISA Municipality: ____
☐ PWQO
☐ Other (specify): ____

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

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

ANALYSIS REQUESTED (Please be specific)

TURNAROUND TIME (TAT) REQUIRED

PLEASE PROVIDE ADVANCE NOTICE FOR RUSH PROJECTS.

Regular (Standard) TAT: ☒
(5-7 working days for most tests)

Rush TAT:
Samples must be received by 3pm to guarantee your TAT

Rush Confirmation #: PN
☐ 1 day ☐ 2 days ☐ 3 days
Date Req'd:

TATs for certain tests are > 5 days. Please contact your Project Manager for details.

	Sample Identification	Date Sampled	Time Sampled	Matrix (GW, SW, Soil, etc.)	MOE Regulated Drinking Water? (Y / N)	Metals Field Filtered? (Y / N)	CORROSIVITY PACKAGE	# of Cont.	COMMENTS / TAT COMMENTS
1	RW3-3-4.33m-4.43m	NOV 3,2016	AM	ROCK	N	N	X	1	
2	OHS-4-SA4-2.29m-2.59m	NOV 10,2016	AM	SOIL	N	N	X	1	
3	OHS-5-SA5-3.81m-4.42m	Nov 10,2016	AM	SOIL	N	N	X	1	
4	CV01-01-8.74m-8.80m	NOV 3,2016	AM	ROCK	N	N	X	1	
5	CV02/3-1-5.27m-5.32m	NOV 3,2016	AM	ROCK	N	N	X	1	
6									
7									
8									
9									
10									

10-Nov-16 17:14
Ema Gitej
B6O5411
KP7 ENV-803

*RELINQUISHED BY (Signature/Print)	Date (YYYY/MM/DD)	Time:	RECEIVED BY: (Signature/Print)	Date (YYYY/MM/DD)	Time:
Amelia Jewison	2016/11/10	17:10	UMAN	2016/11/10	17:14

#JARS USED AND NOT SUBMITTED

Laboratory Use Only

Custody Seal	Yes	No	Temperature (°C) on Receipt
Present		/	10/12/14°C
Intact		/	

on ice

*MANDATORY SECTIONS IN GREY MUST BE FILLED OUT. AN INCOMPLETE CHAIN OF CUSTODY MAY RESULT IN ANALYTICAL TAT DELAYS.

COC-1004 (10/11) - ENV. ENG.
Maxxam Analytics International Corporation o/a Maxxam Analytics
White: Maxxam Yellow: Mail Pink: Client

Maxxam
A Bureau Veritas Group Company

Your Project #: 1530382
Site Location: QEW CAWTHRA
Your C.O.C. #: 655260-03-01

Attention: Sandra McGaghran

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

Report Date: 2018/05/17
Report #: R5155109
Version: 1 - Final

CERTIFICATE OF ANALYSIS

MAXXAM JOB #: B8B3986
Received: 2018/05/15, 12:05

Sample Matrix: Soil
Samples Received: 5

Analyses	Quantity	Date Extracted	Date Analyzed	Laboratory Method	Reference
Chloride (20:1 extract)	5	N/A	2018/05/17	CAM SOP-00463	EPA 325.2 m
Conductivity	5	N/A	2018/05/17	CAM SOP-00414	OMOE E3530 v1 m
pH CaCl2 EXTRACT	5	2018/05/17	2018/05/17	CAM SOP-00413	EPA 9045 D m
Resistivity of Soil	5	2018/05/16	2018/05/17	CAM SOP-00414	SM 23 2510 m
Sulphate (20:1 Extract)	5	N/A	2018/05/17	CAM SOP-00464	EPA 375.4 m

Remarks:

Maxxam Analytics' laboratories are accredited to ISO/IEC 17025:2005 for specific parameters on scopes of accreditation. Unless otherwise noted, procedures used by Maxxam are based upon recognized Provincial, Federal or US method compendia such as CCME, MDDELCC, EPA, APHA.

All work recorded herein has been done in accordance with procedures and practices ordinarily exercised by professionals in Maxxam's profession using accepted testing methodologies, quality assurance and quality control procedures (except where otherwise agreed by the client and Maxxam in writing). All data is in statistical control and has met quality control and method performance criteria unless otherwise noted. All method blanks are reported; unless indicated otherwise, associated sample data are not blank corrected.

Maxxam Analytics' liability is limited to the actual cost of the requested analyses, unless otherwise agreed in writing. There is no other warranty expressed or implied. Maxxam has been retained to provide analysis of samples provided by the Client using the testing methodology referenced in this report. Interpretation and use of test results are the sole responsibility of the Client and are not within the scope of services provided by Maxxam, unless otherwise agreed in writing.

Solid sample results, except biota, are based on dry weight unless otherwise indicated. Organic analyses are not recovery corrected except for isotope dilution methods.
Results relate to samples tested.
This Certificate shall not be reproduced except in full, without the written approval of the laboratory.
Reference Method suffix "m" indicates test methods incorporate validated modifications from specific reference methods to improve performance.
* RPDs calculated using raw data. The rounding of final results may result in the apparent difference.

Maxxam
A Bureau Veritas Group Company

Your Project #: 1530382
Site Location: QEW CAWTHRA
Your C.O.C. #: 655260-03-01

Attention: Sandra McGaghran

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

Report Date: 2018/05/17
Report #: R5155109
Version: 1 - Final

CERTIFICATE OF ANALYSIS

MAXXAM JOB #: B8B3986
Received: 2018/05/15, 12:05

Encryption Key

Please direct all questions regarding this Certificate of Analysis to your Project Manager.
Ema Gitej, Senior Project Manager
Email: EGitej@maxxam.ca
Phone# (905)817-5829
==
Maxxam has procedures in place to guard against improper use of the electronic signature and have the required "signatories", as per section 5.10.2 of ISO/IEC 17025:2005(E), signing the reports. For Service Group specific validation please refer to the Validation Signature Page.

Total Cover Pages : 2

Maxxam Job #: B8B3986
Report Date: 2018/05/17

Golder Associates Ltd
Client Project #: 1530382
Site Location: QEW CAWTHRA
Sampler Initials: AJ

SOIL CORROSIVITY PACKAGE (SOIL)

Maxxam ID		GRV507		GRV508			GRV508		
Sampling Date		2016/09/18		2017/11/17			2017/11/17		
COC Number		655260-03-01		655260-03-01			655260-03-01		
	UNITS	**RW3-1-SA1**	**RDL**	**RW2-1-SA5**	**RDL**	**QC Batch**	**RW2-1-SA5 Lab-Dup**	**RDL**	**QC Batch**
Calculated Parameters									
Resistivity	ohm-cm	670		8400		5533603			
Inorganics									
Soluble (20:1) Chloride (Cl)	ug/g	140	20	<20	20	5535716			
Conductivity	umho/cm	1500	2	119	2	5535789			
Available (CaCl2) pH	pH	8.02		8.05		5535614			
Soluble (20:1) Sulphate (SO4)	ug/g	1400	60	<20	20	5535750	<20	20	5535750

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

Maxxam ID		GRV509	GRV510	GRV511		
Sampling Date		2017/11/13	2017/10/31	2017/11/23		
COC Number		655260-03-01	655260-03-01	655260-03-01		
	UNITS	**RW2-3-SA7**	**RW2-4-SA8**	**RW2-7-SA6A**	**RDL**	**QC Batch**
Calculated Parameters						
Resistivity	ohm-cm	1700	2500	1700		5533603
Inorganics						
Soluble (20:1) Chloride (Cl)	ug/g	<20	<20	32	20	5535716
Conductivity	umho/cm	584	395	588	2	5535789
Available (CaCl2) pH	pH	8.08	8.11	8.04		5535614
Soluble (20:1) Sulphate (SO4)	ug/g	580	280	550	20	5535750

RDL = Reportable Detection Limit
QC Batch = Quality Control Batch

TEST SUMMARY

Maxxam ID: GRV507
Sample ID: RW3-1-SA1
Matrix: Soil

Collected: 2016/09/18
Shipped:
Received: 2018/05/15

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	5535716	N/A	2018/05/17	Alina Dobreanu
Conductivity	AT	5535789	N/A	2018/05/17	Tahir Anwar
pH CaCl2 EXTRACT	AT	5535614	2018/05/17	2018/05/17	Gnana Thomas
Resistivity of Soil		5533603	2018/05/17	2018/05/17	Ewa Pranjic
Sulphate (20:1 Extract)	KONE/EC	5535750	N/A	2018/05/17	Alina Dobreanu

Maxxam ID: GRV508
Sample ID: RW2-1-SA5
Matrix: Soil

Collected: 2017/11/17
Shipped:
Received: 2018/05/15

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	5535716	N/A	2018/05/17	Alina Dobreanu
Conductivity	AT	5535789	N/A	2018/05/17	Tahir Anwar
pH CaCl2 EXTRACT	AT	5535614	2018/05/17	2018/05/17	Gnana Thomas
Resistivity of Soil		5533603	2018/05/17	2018/05/17	Ewa Pranjic
Sulphate (20:1 Extract)	KONE/EC	5535750	N/A	2018/05/17	Alina Dobreanu

Maxxam ID: GRV508 Dup
Sample ID: RW2-1-SA5
Matrix: Soil

Collected: 2017/11/17
Shipped:
Received: 2018/05/15

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Sulphate (20:1 Extract)	KONE/EC	5535750	N/A	2018/05/17	Alina Dobreanu

Maxxam ID: GRV509
Sample ID: RW2-3-SA7
Matrix: Soil

Collected: 2017/11/13
Shipped:
Received: 2018/05/15

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	5535716	N/A	2018/05/17	Alina Dobreanu
Conductivity	AT	5535789	N/A	2018/05/17	Tahir Anwar
pH CaCl2 EXTRACT	AT	5535614	2018/05/17	2018/05/17	Gnana Thomas
Resistivity of Soil		5533603	2018/05/17	2018/05/17	Ewa Pranjic
Sulphate (20:1 Extract)	KONE/EC	5535750	N/A	2018/05/17	Alina Dobreanu

Maxxam ID: GRV510
Sample ID: RW2-4-SA8
Matrix: Soil

Collected: 2017/10/31
Shipped:
Received: 2018/05/15

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	5535716	N/A	2018/05/17	Alina Dobreanu
Conductivity	AT	5535789	N/A	2018/05/17	Tahir Anwar
pH CaCl2 EXTRACT	AT	5535614	2018/05/17	2018/05/17	Gnana Thomas
Resistivity of Soil		5533603	2018/05/17	2018/05/17	Ewa Pranjic
Sulphate (20:1 Extract)	KONE/EC	5535750	N/A	2018/05/17	Alina Dobreanu

TEST SUMMARY

Maxxam ID: GRV511
Sample ID: RW2-7-SA6A
Matrix: Soil

Collected: 2017/11/23
Shipped:
Received: 2018/05/15

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	5535716	N/A	2018/05/17	Alina Dobreanu
Conductivity	AT	5535789	N/A	2018/05/17	Tahir Anwar
pH CaCl2 EXTRACT	AT	5535614	2018/05/17	2018/05/17	Gnana Thomas
Resistivity of Soil		5533603	2018/05/17	2018/05/17	Ewa Pranjic
Sulphate (20:1 Extract)	KONE/EC	5535750	N/A	2018/05/17	Alina Dobreanu

GENERAL COMMENTS

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

Package 1	9.0°C

Samples received and analyzed past the recommended hold time as per client request.

Results relate only to the items tested.

QUALITY ASSURANCE REPORT

			Matrix Spike		SPIKED BLANK		Method Blank		RPD	
QC Batch	**Parameter**	**Date**	**% Recovery**	**QC Limits**	**% Recovery**	**QC Limits**	**Value**	**UNITS**	**Value (%)**	**QC Limits**
5535614	Available (CaCl2) pH	2018/05/17			100	97 - 103			0.80	N/A
5535716	Soluble (20:1) Chloride (Cl)	2018/05/17	NC	70 - 130	100	70 - 130	<20	ug/g	1.7	35
5535750	Soluble (20:1) Sulphate (SO4)	2018/05/17	104	70 - 130	106	70 - 130	<20	ug/g	NC	35
5535789	Conductivity	2018/05/17			100	90 - 110	<2	umho/cm	7.6	10

N/A = Not Applicable

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

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

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

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

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

NC (Duplicate RPD): The duplicate RPD was not calculated. The concentration in the sample and/or duplicate was too low to permit a reliable RPD calculation (absolute difference <= 2x RDL).

VALIDATION SIGNATURE PAGE

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

Eva

Eva Pranjic

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

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

Maxxam
A Bureau Veritas Group Company

Maxxam Analytics International Corporation o/a Maxxam Analytics
6740 Campobello Road, Mississauga, Ontario Canada L5N 2L8 Tel:(905) 817-5700 Toll-free:800-563-6266 Fax:(905) 817-5777 www.maxxam.ca

CHAIN OF CUSTODY RECORD

Page of

INVOICE TO:

Company Name: #1326 Golder Associates Ltd
Attention: Accounts Payable
Address: 6925 Century Ave Suite 100
Mississauga ON L5N 7K2
Tel: (905) 567-4444 x Fax: (905) 567-6561 x
Email: AP_CustomerService@golder.com

REPORT TO:

Company Name: Golder Associates
Attention: Sandra McGaghran
Address:
Tel: 905-567-6100 x 1235 Fax:
Email: Sandra_McGaghran@golder.com

PROJECT INFORMATION:

Quotation #: B80683
P.O. #:
Project: ~~1674130 WO#9~~ 1530382
Project Name:
Site #: QEW Cawthra
Sampled By: AJ / JL / DCB

Laboratory Use Only:

Maxxam Job #:
Bottle Order #: 655260
COC #: C#655260-03-01
Project Manager: Ema Gitej

MOE REGULATED DRINKING WATER OR WATER INTENDED FOR HUMAN CONSUMPTION MUST BE SUBMITTED ON THE MAXXAM DRINKING WATER CHAIN OF CUSTODY

Regulation 153 (2011)

☐ Table 1 ☐ Res/Park ☐ Medium/Fine
☐ Table 2 ☐ Ind/Comm ☐ Coarse
☐ Table 3 ☐ Agri/Other ☐ **For RSC**
☐ Table ___

Other Regulations

☐ CCME ☐ Sanitary Sewer Bylaw
☐ Reg 558 ☐ Storm Sewer Bylaw
☐ MISA Municipality ______
☐ PWQO
☐ Other ______

Special Instructions

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

ANALYSIS REQUESTED (PLEASE BE SPECIFIC)

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

Regular (Standard) TAT: ☐
(will be applied if Rush TAT is not specified):
Standard TAT = 5-7 Working days for most tests.
Please note: Standard TAT for certain tests such as BOD and Dioxins/Furans are > 5 days - contact your Project Manager for details.

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

	Sample Barcode Label	Sample (Location) Identification	Date Sampled	Time Sampled	Matrix	Field Filtered (please circle): Metals / Hg / Cr VI	Standard Corrosivity pkg (No Sulphide and Redox Potential)	# of Bottles	Comments
1	—	RW3-1_SA1	18/09/18	AM	SOIL		X	1	
2	—	RW2-1_SA5	17/11/17	↓	↓		X	1	
3	—	RN2-3_SA7	17/11/13				X	1	
4	—	RW2-4_SA8	17/10/31				X	1	
5	—	RN2-7_SA6A	17/11/23				X	1	
6									
7									
8									
9									
10									

15-May-18 12:05
Ema Gitej
B8B3986
PS4 ENV-1215

* RELINQUISHED BY: (Signature/Print)	Date: (YY/MM/DD)	Time	RECEIVED BY: (Signature/Print)	Date: (YY/MM/DD)	Time	# jars used and not submitted
Katie Nero	18/05/15	12:05	Ashwini Sukumar	2018/05/15	12:05	

Laboratory Use Only

Time Sensitive	Temperature (°C) on Recei	Custody Seal	Yes	No
	12/4/11	Present		✓
		Intact		✓

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

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

** SAMPLE CONTAINER, PRESERVATION, HOLD TIME AND PACKAGE INFORMATION CAN BE VIEWED AT HTTP://MAXXAM.CA/WP-CONTENT/UPLOADS/ONTARIO-COC.PDF.

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

White: Maxxa Yellow: Client

on ice

Maxxam Analytics International Corporation o/a Maxxam Analytics

APPENDIX D

Non-Standard Special Provisions

FOUNDATIONS ON BEDROCK - Item No.

Non-Standard Special Provision

Where strip footings, steel piles or caissons for Retaining Wall support extend to or into the shale bedrock, which is very weak to weak in the area of the retaining wall replacements, but which exhibits UCS values up to 32 MPa and which contains medium strong to strong limestone layers at varying depths/elevations, appropriate equipment and construction procedures will be required to penetrate into the bedrock to reach the founding level.

Basis of Payment

Payment at the lump sum contract price for this tender item shall be full compensation for all labour, equipment and materials for completion of the work.

END OF SECTION

H-PILES - Item No.

Non-Standard Special Provision

Amendment to OPSS.PROV 903, April 2016

Deep Foundations

903.07 CONSTRUCTION

903.07.02 Driven Piles

903.07.02.07.03.03 Driving to Bedrock

Section 903.07.02.07.03.03 of OPSS 903 is deleted and replaced with the following:

In order to avoid overdriving and possibly damaging the piles when seating onto bedrock, the piles shall be driven to an initial set equal to or greater than 10 blows per 25 mm of penetration (unless abrupt peaking occurs) using a hammer with rated energy of about 50 kilojoules but not exceeding 60 kilojoules. The bedrock elevation shall be recorded. On reaching the required set, the hammer energy shall be reduced to 75 percent of the maximum energy and the pile shall then be re-driven in 2 sets of 10 blows and the penetration recorded after each set of 10 blows. The hammer energy shall then be increased to 100 percent and the pile re-driven for 10 blows and the penetration recorded. A final set of no less than 10 blows per 25 mm of penetration shall be obtained at the maximum hammer energy.

If unusually excessive penetration per blow is observed, driving shall be stopped and this excessive penetration immediately reported to the Contract Administrator.

The Contractor’s Engineer shall determine when the hammer energy can be increased and when the driving is complete for each pile.

DEEP FOUNDATIONS – Item No.

Non-Standard Special Provision

Amendment to OPSS.PROV 903, April 2016

Deep Foundations

903.07 CONSTRUCTION

Section 903.07.03.02 of OPSS.PROV 903 shall be amended by the addition of the following:

The Contactor shall be alerted to the potential presence of cobbles and boulders within the gravelly sand to sand and gravel, cohesive tills and residual soils. Considerations of the presence of these obstructions must be made in the selection of appropriate equipment and procedures for excavations, driving steel H-piles, or advancing caissons, such that the design tip levels are achieved; or installation of temporary protection systems.

<u>EARTH EXCAVATION FOR STRUCTURE</u> – Item No.

Non-Standard Special Provision

Amendment to OPSS 902, November 2010

Excavating and Backfilling – Structures

902.07 CONSTRUCTION

Section 902.07 of OPSS 902 shall be amended by the addition of the following:

The Contactor is alerted to the potential presence of cobbles and boulders within the sand and gravel, cohesive tills and hard rock slabs within the residual soils. Consideration of the presence of these obstructions shall be made in the selection of appropriate equipment and procedures for excavations and for installation of temporary protection systems.

PROTECTION SYSTEM – Item No.

Special Provision

Amendment to OPSS.PROV 539, November 2014

593.07.02 Removal of Protection Systems

Subsection 539.07.02 of OPSS 539 is deleted in its entirety and replaced with the following:

Protection systems shall be removed from the right-of-way unless it is specified in the Contract Documents that the protection system may be left in place.

Where piles are left in place, the top shall be removed to at least 1.2 m below the finished grade or ground level.

The method and sequence of removal shall be such that there shall be no damage to the new work, existing work and facility being protected.

All disturbed areas shall be restored to an equivalent or better condition than existing prior to the commencement of construction.

WORKING SLAB - Item No.

Non-Standard Special Provision

1.0 Scope

This Special Provision covers the requirements for the supply and placement of a concrete working slab under foundations for the QEW retaining wall replacement structures.

2.0 References

This Special Provision refers to the following standards, specifications or publications:

Ontario Provincial Standard Specifications, Construction
OPSS 902 Excavating and Backfilling - Structures

3.0 Definitions - Not Used

4.0 Design and Submission Requirements - Not Used

5.0 Materials

Concrete for working slabs shall have a minimum 28 day strength of 20 MPa.

6.0 EQUIPMENT - Not Used

7.0 CONSTRUCTION

7.01 Excavation

Excavation for the working slab shall be according to OPSS 902.

7.02 Protection of Founding Soil

Following inspection and approval of the prepared subgrade, a working slab with a minimum thickness of 100 mm shall be placed on the foundation subgrade as specified in the Contract Documents.

7.03 Protection of Founding Bedrock

The surface of the footing founding rock shall be exposed, cleaned and any loose or fractured parts removed so that sound rock is exposed. The working slab shall be placed on the exposed cleaned sound founding rock surface as specified in the Contract Documents. Thickness of the mass concrete pad shall depend on the slope and irregularities in the exposed founding rock surface. A nominal thickness and a footprint plan view area has been specified on the Contract Documents

7.04 Dewatering

Dewatering shall be carried out according to OPSS 902.

8.0 Quality Assurance - Not Used

9.0 Measurement for Payment - Not Used

10.0 Basis of Payment

10.01 Working Slab - Item

Payment at the Contract price for the above tender item shall be full compensation for all labour, Equipment and Material to do the work.

END OF SECTION

VIBRATION MONITORING - Item No.

Special Provision

TABLE OF CONTENTS

1.0 SCOPE

This special provision describes requirements for vibration monitoring for the following components of the Contract:

- Deep foundation and temporary protection system installation for the construction of the Etobicoke Creek bridge
- Temporary protection system for the construction of the retaining wall between Station 13+830 to 13+975 on the north side of the QEW
- Deep foundation installation for a retaining wall between Station 13+650 and 13+750 on the north side of the QEW
- Temporary protection system for the removal of existing retaining walls on the north side of the QEW between Station 13+501 and 13+815 and on the south side of the QEW between Station 13+748.5 and 13+847.5.

2.0 REFERENCES

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

1. Foundation Investigation and Design Report, QEW - Etobicoke Bridge Replacement (Site No. 37-237/1&2), City off Mississauga, Etobicoke, Ministry of Transportation, Ontario, GWP 2102-13-00 and 2432-13-00.

2. Retaining Wall from Station 13+830 to 13+975, QEW Improvements from East of Cawthra Road to The East Mall, Mississauga and Etobicoke, Ministry of Transportation, Ontario, G.W.P. 2102-13-00 & 2432-13-00.

3. Foundation Investigation and Design Report, Retaining Walls No. 24-887/W and 24-888/W Replacement, QEW Widening from East of Cawthra Road to the East Mall, Cities of Mississauga and Etobicoke, Ministry of Transportation, Ontario, GWP 2102-13-00 & 2432-13-00.

4. Sanitary Sewer, QEW Widening from East of Cawthra Road to the East Mall, Cities of Mississauga and Etobicoke, Ministry of Transportation, Ontario, GWP 2102-13-00 & 2432-13-00

3.0 DEFINITIONS

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

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

Peak Particle Velocity (PPV) means the maximum component velocity in millimetres per second that ground particles move as a result of energy released from vibratory construction operations.

Pre-Construction Condition Survey means a detailed record, accompanied by film or video, as necessary, of the condition of private or public property, prior to the commencement of vibratory or vibration-inducing construction operations.

Post-Construction Condition Survey means a detailed record, accompanied by film or video, as necessary, of the condition of private or public property, after completion of vibratory or vibration-inducing construction operations.

4.0 DESIGN AND SUBMISSION REQUIREMENTS

4.1 Submission Requirements

The Contractor/Contractor's Engineer shall submit details of the vibration monitoring plan to the Contract Administrator for information purposes. The submittals shall satisfy the specifications and at a minimum contain the following specific information:

a) Equipment and methods used by the Contractor to perform the work that may cause undue vibration.

b) Qualifications of vibration monitoring specialist.
c) Details regarding proposed instrumentation.
d) Proposed location of instruments adjacent to the on the residences, utilities, wells, or other potentially vibration-sensitive structures within a 250 m radius from the Etobicoke Creek bridge, within 75 m of the protection systems for the removal of the existing retaining wall and deep foundation installation on the north side of the QEW, and within 50 m of the proposed retaining wall alignment and/or protection systems on the south side of the QEW.
e) Proposed frequency of readings.
f) Action plan to be taken to adjust deep foundation and protection system installation methods or if readings show vibrations exceeding tolerable levels.

6.0 EQUIPMENT

6.1 Vibration Monitoring Equipment

All vibration monitoring equipment shall be capable of measuring and recording ground vibration PPV up to 200 mm/s in the vertical, transverse, and radial directions. The equipment shall have been calibrated within the last 12 months either by the manufacturer or other qualified agent. Proof of calibration shall be submitted to the Contract Administrator prior to commencement of any monitoring operations.

7.0 CONSTRUCTION

7.1 Pre- and Post-Construction Condition Surveys

A Pre-Construction Condition Survey and Post-Construction Condition Survey shall be prepared for all buildings, utilities, structures, water wells, and facilities within a 250 m radius from the Etobicoke Creek bridge, within 75 m of the protection systems for the removal of the existing retaining wall and deep foundation installation on the north side of the QEW, and within 50 m of the proposed retaining wall alignment and/or protection systems on the south side of the QEW.

7.1.1 Pre-Construction Condition Surveys

The standard inspection procedure shall include the provision of an explanatory letter to the owner or occupant and owner with a formal request for permission to carry out an inspection.

The Pre-Construction Condition Survey, at each structure/well within a 250 m radius from the Etobicoke Creek bridge, within 75 m of the protection systems for the removal of the existing retaining wall and deep foundation installation on the north side of the QEW, and within 50 m of the proposed retaining wall alignment and/or protection systems on the south side of the QEW, shall be completed a minimum of two (2) weeks prior to commencement of installation of the deep foundations and/or protection system(s). Only one Pre-Construction Condition Survey per structure or facility is required to be carried out in advance of deep foundation and protection system installation, unless more than six (6) months will elapse between these operations, in which case an interim inspection will be required.

The Pre-Construction Condition Survey shall include, as a minimum, the following information:

a) Type of structure, including type of construction and if possible, the date when built.

b) Identification and description of existing differential settlements, including visible cracks in walls, floors, and ceilings, including a diagram, if applicable, room-by-room. All other apparent structural and cosmetic damage or defects shall also be noted. Defects shall be described, including dimensions, wherever possible.
c) Digital photographs or digital video or both, as necessary, to record areas of significant concern.

Photographs and videos shall be clear and shall accurately represent the condition of the property. Each photograph or video shall be clearly labelled with the location and date taken.

A copy of the Pre-Construction Construction Survey limited to a single residence or property, including copies of any photographs or videos that may form part of the report, shall be provided to the owner of that residence or property, upon request.

7.1.2 Post-Construction Condition Surveys

The standard inspection procedure shall include the provision of an explanatory letter to the owner or occupant and owner with a formal request for permission to carry out an inspection.

A Post-Construction Condition Survey at each structure within a 250 m radius from the Etobicoke Creek bridge, within 75 m of the protection systems for the removal of the existing retaining wall and deep foundation installation on the north side of the QEW, and within 50 m of the proposed retaining wall alignment and/or protection systems on the south side of the QEW, is required within two (2) months of completion of the installation of deep foundations and protection systems.

The Post-Construction Condition Survey shall include, as a minimum, the following information:

a) Identification and description of existing differential settlements, including visible cracks in walls, floors, and ceilings, including a diagram, if applicable, room-by-room. All other apparent structural and cosmetic damage or defects shall also be noted. Defects shall be described, including dimensions, wherever possible.
b) Digital photographs or digital video or both, as necessary, to record areas of significant concern.
c) Comparison between pre-condition survey documented concerns and post-condition concerns.

Photographs and videos shall be clear and shall accurately represent the condition of the property. Each photograph or video shall be clearly labelled with the location and date taken.

A copy of the Post-Construction Condition Survey limited to a single residence or property, including copies of any photographs or videos that may form part of the report, shall be provided to the owner of that residence or property, upon request. The report shall confirm that there have been no changes to the property between the Pre-Construction Condition Survey and the Post-Construction Condition Survey as a result of the installation of deep foundations and protection systems.

7.2 Monitoring

The vibration monitoring equipment shall be placed on the ground surface in the vicinity of each retaining wall section requiring deep foundation elements or protection systems, and on the ground surface at radial distances of 25 m, 50 m, and 100 m from these locations toward receptors (e.g., buildings, sensitive utilities). The Contractor shall take readings continuously during construction for the deep foundation elements of retaining walls or associated protection system installation, and shall immediately notify the Contract Administrator if the vibrations exceed the limits specified herein.

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

If the readings are not within the limits stated above, the Contractor must alter the installation procedures until the vibrations at the various locations are within acceptable levels.

7.3 Records

The Contractor/Contractor's Engineer shall submit details of the vibration monitoring to the Contract Administrator as follows:

a) The time/duration of each reading.
b) Construction operations (i.e. installation of sheet piling) and timing of such relative to the readings.
c) Details of exceedances and modifications to operations.
d) Final report containing all relevant data including vibration monitoring and Pre- and Post-Construction Condition Surveys.

10.0 BASIS OF PAYMENT

Payment at the Contract price for the above tender item shall be full compensation for all labour, Equipment and Material required to do the work.

SUBEXCAVATION – Item No.

Notice to Contractor

A sanitary sewer tunnel will be installed prior to the construction of the retaining wall in the vicinity of Station 13+625 on the north side of the QEW. Where the proposed strip footing for the retaining wall intersects the alignment of the sanitary sewer tunnel, the Contractor shall subexcavate below the wall footing founding level to expose the primary liner of the tunnel, and replace the subexcavated area with unshrinkable fill having a compressive strength of 0.4 MPa. The top of the unshrinkable fill should extend at least 1 m beyond the plan limits of the strip footing in all directions, and downward and outward at an orientation of 1 horizontal to 1 vertical (1H:1V).

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