FOUNDATION INVESTIGATION AND DESIGN REPORT

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

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PART A

FOUNDATION INVESTIGATION REPORT
QUEEN ELIZABETH WAY (QEW) - CREDIT RIVER BRIDGE, STRUCTURE SITE NO. 24-203, QEW WIDENING FROM WEST OF MISSISSAUGA ROAD TO WEST OF HURONTARIO STREET, MISSISSAUGA,
MINISTRY OF TRANSPORTATION, ONTARIO,
GWP 2002-13-00

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by Morrison Hershfield Limited (MH) on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services for the proposed twinning of the existing bridge carrying the Hamilton bound traffic on the Queen Elizabeth Way (QEW) over the Credit River in support of the widening of the QEW from west of Mississauga Road to west of Hurontario Street in the City of Mississauga, in the Regional Municipality of Peel, Ontario.

The purpose of this investigation is to establish the subsurface soil and bedrock conditions at the proposed bridge structure location, including the associated approach embankments, by borehole drilling, rock coring, packer testing 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 July 2016, and the approved Change Request letter dated September 6, 2017, which forms part of the Consultant's Assignment Number (2015-E-0033) for this project. The work has been carried out in accordance with Golder's Supplementary Specialty Plan for foundation engineering services for this project, dated February 3, 2017.

2.0 SITE DESCRIPTION

At this site the QEW is generally oriented in a northeast-southwest direction; for the purpose of this report the QEW is described as being in an east-west orientation. The existing QEW Credit River Bridge is located approximately 400 m east of the QEW-Mississauga Road Interchange and approximately 1.4 km west of the QEW - Hurontario Street Interchange and crosses the Credit River Valley over the floodplain and river channel.

The existing bridge is an approximately 256 m long and 29 m wide, seven-span spandrel arch structure, with concrete arches at the piers, supporting six lanes of traffic. The Credit River Valley is about 19 m below the surrounding plateau. The ground surface at the base of the floodplain is at about Elevation 76 m, while the surface of the Credit River, as provided by MH, is at about Elevation 75 m (Sept. 1986). The ground surface at the east and west plateau at the top of the valley (and on the highway near the abutments) is at about Elevation 95 m. The existing bridge is supported on two abutments and six piers all supported on shallow foundations. The east and west abutment are atop the valley plateau and the founding elevation of the supporting footings range from about Elevation 88.9 m to 87.5 m. The piers are located on the valley slopes, within the flood plain and within the Credit River itself, and the founding elevation of the supporting footings range from Elevations 84.5 m (at Pier 6, located on the west valley slope) to 68.2 m.

The east slope of the valley is vegetated with tall grass, shrubs and trees and descends to meet the east bank of the Credit River at an overall slope of about 3.5 Horizontal to 1 Vertical (3.5H:1V) but there are locally steeper sections. Rip-rap erosion protection is present on the on the surface of the east slope and east bank of the river, in some areas. The slope configuration may have been modified by fill placement on the existing valley slope. The east plateau is relatively heavily vegetated and relatively flat. Residential dwellings, now demolished and removed, existed within the footprint of the proposed bridge at the east plateau.

At the west side of the valley, a construction access road was constructed in 2006 and it is understood that the access road alignment was cut through the shale bedrock and shotcrete was applied to the exposed rock faces. The access road splits into an upper access road (which leads to the under-bridge maintenance deck) and a lower access road (which extends down to the base of the valley). The upper access road runs parallel to the west abutment and the surface of the road is at about Elevation 89 m. Above the upper access road, shotcrete was applied to the rock face. The downslope side of the upper access road is supported by a concrete block retaining wall which protects the lower access road. The access road(s) were cut through the west valley slope and constructed to provide access to the underside of the existing bridge and the valley floor. The west valley slopes (between the abutment and access roads) descend near vertically to meet the flood plain of the Credit River Valley. The west plateau is relatively flat but less densely vegetated than the east plateau, consisting mainly of tall grass and some shrubs.

The proposed bridge will be located immediately to the north of the existing bridge. The land use at the east and west plateau of the valley, north of the proposed bridge is residential. A Hydro One Right-of-Way, containing high voltage transmission lines and local utility owned transmission lines, is located within the footprint of the proposed bridge and crosses the Credit River Valley just north of the existing bridge. Additionally, two buried oil pipelines, owned by Trans-Northern Pipeline Inc. are located immediately to the north of the existing bridge and within the footprint of the proposed bridge.

3.0 INVESTIGATION PROCEDURES

The following Sections 3.1 and 3.2 outline the investigations carried out by others (previous) and by Golder (current) that are relevant to the proposed new bridge structure foundation design, respectively. Section 3.3 describes the available subsurface investigation information that may also be relevant to the design of temporary works, such as foundations to support falsework.

3.1 Previous Investigations

From September to November 2010, a foundation investigation for the west access road was carried out by Thurber Engineering Ltd. (Thurber) during which time a total of seven boreholes were drilled in two phases, designated as Boreholes 10-01 to 10-05, 10-03A and 10-03B. The results of the Thurber investigation are contained in their report titled,

- “Foundation Investigation and Design Report, Construction Access Road for Bridge Rehabilitation, QEW Bridge over Credit River, Mississauga, Ontario” File No. 19-92-92-174, dated April 8, 2011 (GEOCRES 30M12-324). Borehole 10-02 was advanced about 7 m north of the west abutment.

In May and June 2011, a preliminary foundation investigation for the new Credit River Bridge was carried out by Thurber, during which time a total of two boreholes, designated as Boreholes 11-01 and 11-02, were advanced at the proposed west abutment and east pier, respectively. The results of the Thurber investigation are contained in their report titled,

- “Foundation Investigation and Design Report, Preliminary Design and Environmental Assessment, QEW Bridge Twinning Over Credit River, Mississauga, Ontario” File No. 19-1351-174, dated May 18, 2012 (GEOCRES 30M12-341).

While the above noted Thurber reports do not reference the coordinate system of the borehole locations, it is inferred that they are referenced to the MTM NAD 83 (Zone 10) coordinate system based on the plotted position relative to that reference system. The locations of the Thurber boreholes (i.e. 10-02, 11-01, and 11-02) that are relevant to the subsurface conditions at or near the proposed new structure foundation units are summarized in the table below along with the geographic coordinates, ground surface elevations (in Geodetic Datum), and the drilled depths based on the Thurber borehole records. These borehole locations are also shown in plan on Drawing 1 and the borehole records and the summary of the relevant laboratory testing results from the Thurber investigation are presented in Appendix A.

Borehole No.	Location (MTM NAD 83, Zone 10)		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing (Latitude, °)	Easting (Longitude, °)		
10-02	4,823,966.3 (43.555579)	295,798.0 (-79.611422)	94.4	24.4*
11-01	4,823,959.1 (43.555514)	295,814.8 (-79.611214)	94.6	7.1*
11-02	4,824,026.4 (43.556210)	295,840.4 (-79.610898)	75.7	8.4*

Note(s):

* Includes bedrock coring lengths of 19.8 m (10-02), 3.5 m (11-01), and 2.1 m (11-02).

3.2 Current Investigation

The field work for the current foundation investigation was carried out between October 10 and October 23, 2017, between January 26 and February 16, 2018 and between July 5 and July 10, 2018, during which time fourteen boreholes, designated as Boreholes CRB-1 to CRB-8, CRB-2A, CRB-2B, CRB-3A, CRB-3C, CRB-5A and NW6-1 were advanced near or within the footprint of the foundation, as follows:

Foundation Element	Nearest Relevant Boreholes
West Approach	CRB-1
West Abutment	CRB-2, CRB-2A and CRB-2B
West Pier	CRB-3, CRB-3A and CRB-3C
East Pier	CRB-4, CRB-5 and CRB-5A
East Abutment	CRB-6 and CRB-7
East Approach	CRB-8
East Retaining Wall	CRB-7, NW6-1

The locations of the boreholes are shown on Drawing 1 and the Records of Boreholes and Drillholes are provided in Appendix B. Lists of abbreviations and symbols and lithological and geotechnical rock description terminology are also provided in Appendix B to assist in the interpretation of the borehole and drillhole records.

The field borehole investigation was carried out using a track-mounted CME 850 drill rig, supplied and operated by Aardvark Drilling Inc., of Guelph, Ontario, a track-mounted CME 55 drill rig, supplied and operated by Geo-Environmental Drilling Inc., of Acton, Ontario, and using portable drilling equipment supplied and operated by OGS Drilling Inc. of Almonte, Ontario. The boreholes were advanced through the overburden using 210 mm and

159 mm outer-diameter hollow-stem augers and ‘HQ’ casing (in the boreholes advanced by a drill rig) and continuous split-spoon sampling, wash-boring, and 57 mm thin-walled casing (in the boreholes advanced by the portable drilling equipment, Boreholes CRB-2A and CRB-3C). In the boreholes where continuous sampling was not carried out, soil samples were obtained at 0.75 m and 1.5 m intervals of depth, using a 50 mm outer diameter split-spoon sampler driven by an automatic hammer, or cathead/safety hammer (for Boreholes CRB-2A, CRB-2B and CRB-3C), in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586-08)[1]. Field vane shear tests were completed using a MTO standard “N”-sized vane and calibrated torque wrench to obtain an estimate of undrained and remolded shear strengths in selected cohesive soil deposits in accordance with ASTM D2573-15[2]. Core samples of the bedrock in all boreholes located at the proposed bridge abutments and piers were obtained using an ‘HQ’ size rock core barrel, or 57 mm outer diameter thin-walled core barrel (Boreholes CRB-2A and CRB-3C), and coring techniques.

All boreholes were advanced to sampler refusal and bedrock was confirmed by either split-spoon sampling or bedrock coring. The boreholes were advanced to depths ranging from about 3.3 m to 17.2 m below existing ground surface, including coring of bedrock for core lengths of between 7.5 m and 9.6 m in select boreholes. Photographs of the recovered bedrock core samples are provided in Appendix B.

The groundwater conditions and water levels in the open boreholes were observed during and immediately following drilling operations. A standpipe piezometer was installed in Boreholes CRB-2, CRB-3A, CRB-5A and CRB-6 to permit monitoring of the groundwater level at these borehole locations. The standpipe piezometers consist of a 50 mm diameter PVC pipe with a slotted screen sealed at a selected depth within the borehole. The borehole annulus surrounding the piezometer screen was backfilled with sand and the remainder of the borehole was then backfilled with bentonite to or to near the ground surface. Details of the piezometer installation and water level readings are presented on the borehole records in Appendix B. Boreholes located at proposed abutments and piers were backfilled with a bentonite cement grout while boreholes at the approach embankments were backfilled with bentonite upon completion in accordance with Ontario Regulation 903, Wells (as amended).

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 both public and private, observed the drilling, sampling and in-situ testing operations, logged the boreholes, and examined and cared for the soil and bedrock core samples. The samples were identified in the field, placed in appropriate containers, labelled and transported to Golder’s Mississauga geotechnical laboratory where the samples underwent further visual examination and laboratory testing. All of the laboratory tests were carried out to MTO and/or ASTM Standards, as appropriate. Classification testing (water content, Atterberg limits and grain size distribution) was carried out on selected soil samples. The results of the laboratory testing for the current investigation are included in Appendix C. Unconfined compression (UC) tests (including assessment of Young’s modulus, Poisson’s ratio, and core density) were carried out on selected specimens of the bedrock core samples by Geomechanica Inc. on behalf of Golder. The results of the laboratory testing on the rock core samples from the current investigation are included in Appendix C.

Selected bedrock core samples were submitted to Maxxam, a Standards Council of Canada (SCC) accredited laboratory of Mississauga, Ontario for chemical analysis. The samples of bedrock core, specifically collected from Boreholes CRB-2, CRB-3A, CRB-5, CRB-6 advanced at the west abutment and pier and east abutment and pier, respectively, were crushed and homogenized by Maxxam prior to testing, and analyzed for a suite of corrosivity

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

[2] ASTM D2573 – Standard Test Method for Field Vane Shear test in Saturated Fine Grained Soils

parameters, including conductivity, resistivity, soluble chloride, soluble sulphate and pH. The results of the chemical analyses are presented in Appendix C.

The as-drilled borehole locations and the ground surface elevations 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 directions. The locations given on the Record of Borehole/Drillhole sheets and shown on Drawing 1 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, geographic coordinates, ground surface elevations and drilled depths are summarized below.

Borehole No.	Location (MTM NAD 83, Zone 10)		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing (Latitude, °)	Easting (Longitude, °)		
CRB-1	4,823,940.2 (43.555344)	295811.1 (-79.611250)	94.9	3.3
CRB-2	4,823,949.7 (43.555430)	295,828.3 (-79.611047)	95.6	12.8 (incl. 9.6 m rock core)
CRB-2A	4,823,960.1 (43.555523)	295,808.0 (-79.611298)	94.5	9.0 (incl. 7.9 m rock core)
CRB-2B	4,823,955.2 (43.555479)	295,818.7 (-79.611165)	94.7	12.7 (incl. 1.5 m soil core and 9.1 m rock core)
CRB-3	4,824,016.8 (43.556034)	295,862.2 (-79.610628)	75.9	15.3 (incl. 8.1 m rock core)
CRB-3A	4,824,025.6 (43.556113)	295,844.6 (-79.610847)	75.7	15.8 (incl. 8.8 m rock core)
CRB-3C	4,824,028.3 (43.556138)	295,837.7 (-79.610932)	75.3	14.1 (incl. 7.7 m rock core)
CRB-4	4,824,135.1 (43.557099)	295,902.0 (-79.610138)	79.1	15.3 (incl. 8.1 m rock core)
CRB-5	4,824,128.9 (43.557044)	295,914.2 (-79.609986)	79.2	15.5 (incl. 8.3 m rock core)
CRB-5A	4,824,130.9 (43.557062)	295,910.6 (-79.610032)	79.3	17.2 (incl. 9.5 m rock core)
CRB-6	4,824,196.7 (43.557650)	295,929.5 (-79.609801)	91.7	13.3 (incl. 8.2 m rock core)
CRB-7	4,824,189.6 (43.557590)	295,951.1 (-79.609531)	94.7	16.0 (incl. 7.5 m rock core)
CRB-8	4,824,211.5 (43.557788)	295,953.7 (-79.609499)	94.7	8.5

Borehole No.	Location (MTM NAD 83, Zone 10)		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing (Latitude, °)	Easting (Longitude, °)		
NW6-1	4,824,163.1 (43.557371)	295,975.2 (-79.609278)	95.3	7.5

3.2.1 Packer Testing

In-situ hydrogeological testing, in general accordance with the procedures defined in ASTM D4630, was conducted at one depth interval in Boreholes CRB-3 and CRB-3A (at the west pier) and in Boreholes CRB-4 and CBR-5 (at the east pier), using a dual pneumatic packer setup connection to an on-surface nitrogen tank through an inflation line. Upon completion of drilling, the packer assembly was lowered into the borehole to isolate a select depth interval within the bedrock and a constant pressure head test was performed. The test results were then used to evaluate the hydraulic conductivity within the isolated packer interval. A pressure gauge data logger, manufactured by In Situ Inc., was used to monitor water pressure responses in the isolated interval during the tests. Flow rates and test pressures in the isolated interval were recorded during the constant pressure head tests as well as being recorded by the data logger. The water pressure profiles obtained were used to calculate estimates of hydraulic conductivity using standard steady-state analysis methods.

3.2.2 Televiewer

The original borehole program consisted of advancing two boreholes at the west abutment; however, following review of the bedrock core information that included zones of lost core it was recommended that an additional borehole be advanced at about the mid-line of the abutment and that geophysics be carried out. Borehole CRB-2B was advanced for this purpose at the location shown on Drawing 1. The borehole was advanced on July 5, 2018 and upon completion of coring the borehole was flushed with water and left overnight to allow for any suspended sediment to settle prior to carrying out the televiewer testing on the next day (July 6, 2018). The following geophysical methods were carried out along the length of the cored borehole and the results of these methods are plotted on the Geophysical Record of Borehole CRB-2B in Appendix B:

- **Mechanical Caliper:** This measurement records the borehole diameter as indicated by the average deflection of three spring-loaded arms pressed against the wall of the borehole. Abrupt shifts to larger diameter (kicks) can indicate the locations where fractures intersect the borehole wall. However, the thickness of the caliper arms and the mechanical enlargement of fractures that can occur during drilling result in an approximate, qualitative relation between fracture aperture and the size of the caliper deflection.
- **Acoustic Televiewer:** This measurement produces an image of the pattern of reflection of an ultrasonic pulse from a source that scans the borehole wall as the logging probe is slowly pulled upwards. The televiewer probe also records telemetry so that the azimuth of the scan and the deviation of the borehole can be measured during logging. The reflection is uniformly bright wherever the borehole wall is solid and smooth. The reflected pulses are scattered wherever a fracture or other irregular opening intersects the borehole wall. Planar features such as fractures produce a linear feature in the borehole image such that the strike and dip of the feature can be estimated.
- **Optical Televiewer:** This measurement produces a continuous oriented 360° image of the borehole wall using an optical imaging system as the logging probe is slowly pulled upwards. The televiewer probe is

magnetically orientated so that the azimuth of the scan and the deviation of the borehole can be measured during logging. As noted above the borehole was flushed with water the day prior to carrying out the optical televiewer test; however, when carrying out the test the water inside the borehole, below an Elevation of 86.4 m was cloudy and the optical televiewer could only collect data above this elevation.

3.3 Investigations for Support of Falsework Design

Several boreholes have been advanced in the vicinity of the proposed Credit River bridge as part of the overall project by Golder that may provide subsurface information relevant to the design of temporary works and/or the falsework supporting the bridge during construction. These include boreholes for the temporary East Access Road (Borehole AR-2), for the East-to-West Active Transportation (Boreholes EW-1 and EW-2), as well as specifically for the falsework support between the proposed west abutment and west pier (Borehole FW-1).

In addition, boreholes 10-03A, 10-03B and 10-04 drilled by Thurber (2011) per the reference in Section 3.1 (GEOCRES 30M12-324) also provide information on the subsurface conditions that may be relevant to the falsework design.

MTO has also provided records of boreholes advanced on behalf of others in the vicinity of the proposed bridge including BH3 (drilled by Stantec Consulting Limited for Trans-Northern Pipeline Inc.) and BH17-2 (drilled by Golder, in a separate investigation, for Alectra Utilities Corporation).

The reference for the above noted reports are listed as follows:

- "Draft Foundation Investigation and Design Report, Temporary East Access Road, East of the Credit River, QEW Widening from West of Mississauga Road to West of Hurontario Street, City of Mississauga Ministry of Transportation, Ontario" G.W.P. 2002-13-00, dated January 15, 2019, prepared by Golder Associates Ltd. (GEOCRES to be assigned). Report for Borehole AR-2.
- "Draft Foundation Investigation and Design Report, East-West Active Transport Bridge Along Credit River Bridge, QEW Widening from West of Mississauga Road to West of Hurontario Street Ministry of Transportation, Ontario" GWP 2002-13-00, dated January 21, 2019, prepared by Golder Associates Ltd. (GEOCRES to be assigned). Report for Boreholes EW-1 and EW-2.
- "Foundation Investigation and Design Report, Construction Access Road for Bridge Rehabilitation, QEW Bridge over Credit River, Mississauga, Ontario" File No. 19-92-92-174, dated April 8, 2011, prepared by Thurber Engineering Ltd. (GEOCRES 30M12-324). Report for Boreholes 10-03A, 10-03B and 10-04.
- "Geotechnical Investigation, Proposed Pipeline Installations, Crossings of Credit River Project No. 160950937 Queen Elizabeth Way & Credit River Mississauga, ON" Document No. TAJ-C-GEO-002, dated December 18, 2018, prepared by Stantec Consulting Ltd. Report for Borehole BH3.
- "Draft Geotechnical Investigation Report, Proposed Electrical Transmission Line Monopoles at Credit River and QEW" dated September 5, 2018, Report No. 1789831, prepared by Golder Associates Ltd. Report for Borehole 17-2.

The borehole records for each of these boreholes are included in Appendix A. The locations of each borehole are shown in plan on Drawing 1. Selected boreholes are included in the profile on Drawing 1. Bedrock core photos, and the results of geotechnical laboratory testing conducted on samples from Borehole FW-1 are included Appendix A

These boreholes are provided for the contractor’s information and use as he may deem appropriate for design of temporary works (including falsework). The majority of these boreholes (i.e. Boreholes AR-2, EW-1, EW-2, FW-1, 10-03A, 10-03B, and 10-4) are publicly available in the MTO GEOCRES system; however, some of the boreholes (i.e. Boreholes BH3 and BH17-2) were advanced by or on the behalf of others for purposes other than MTO works and cannot be relied upon.

The locations, geographic coordinates, ground surface elevations and drilled depths for each of the boreholes noted above are summarized in the table below.

Borehole No.	Location (MTM NAD 83, Zone 10)		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing (Latitude, °)	Easting (Longitude, °)		
AR-2	4,824,172.2 (43.557434)	295,921.4 (-79.609899)	88.4	11.6 (incl. 7.0 m rock core)
EW-1	4,823,955.5 (43.555482)	295,849.5 (-79.610784)	88.5	11.2 (incl. 6.3 m rock core)
EW-2	4,824,156.8 (43.557295)	295,956.2 (-79.609467)	89.1	9.7 (incl. 6.7 m rock core)
FW-1	4,823,994.8 (43.555836)	295,856.0 (-79.610704)	76.1	10.5 (incl. 3.5 m rock core)
10-03A	4,823,979.03 (43.555694)	295,865.10 (-79.610591)	76.2	4.3
10-03B	4,823,983.23 (43.555732)	295,856.15 (-79.610703)	76.3	4.0
10-04	4,824,005.69 (43.555934)	295,823.89 (-79.611102)	76.3	3.7
BH3	4,824,021.6 (43.556077)	295,864.5 (-79.610600)	75.8	30.9 (incl. 23.4 m rock core)
BH17-2	4,824,182.6 (43.557527)	295,900.9 (-79.610151)	89.2	17.1 (incl. 11.0 m rock core)

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)[3].

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

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

4.2 Subsurface Conditions

The detailed subsurface soil, bedrock and groundwater conditions as encountered in the boreholes advanced during the current investigation are presented on the Records of Borehole and Drillhole sheets provided in Appendix B and the results of the laboratory tests carried out on selected soil and bedrock core samples are provided in Appendix C. The subsurface conditions as encountered in the relevant boreholes advanced during the previous investigation, along with the results of the laboratory testing, are included in Appendix A. The results of the in-situ field tests (i.e. SPT "N" values and field vane undrained shear strengths) as presented on the Record of Borehole sheets and in sub-sections of Section 4.2 are uncorrected.

The stratigraphic boundaries shown on the borehole records and on the stratigraphic profiles on Drawings 1 and 2 are inferred from non-continuous sampling, observations of drilling progress and the results of Standard Penetration Tests and in-situ field vane 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 Records of Borehole and Drillhole sheets governs any interpretation of the site conditions. It should be noted that the interpreted stratigraphy shown on Drawings 1 and 2 is a simplification of the subsurface conditions.

In general, the subsurface conditions in the area of the proposed bridge vary from the west plateau, to the west and east bank of the river, to the east plateau. On the west plateau, at the proposed location of the west abutment, the subsurface conditions generally consist of topsoil, overlying fill of various gradations, which in turn overlies silty clay to clayey silt (till) soil. The overburden soil is underlain by weathered shale bedrock at relatively shallow depths, between 1.1 m and 3.6 m below ground surface.

In the area of the west bank and flood plain, at the proposed location of the west pier, the subsurface conditions consist of fill and/or clayey silt to silty clay with organics underlain by sand and gravel and clayey silt (residual soil) at some locations. Weathered shale bedrock underlies the sand and gravel or clayey silt (residual soil) at depths ranging from 6.1 m to 6.3 m below ground surface. In the area of the east bank of the river and flood plain, at the proposed location of the east pier, the subsurface conditions consist of fill underlain by either clayey silt or silty

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[3] 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.)

sand, which in-turn is underlain by organic soils. Silty sand containing organics (shells and wood fragments) and clayey silt pockets underlie the organic soils at some locations. Weathered shale bedrock was encountered underlying the overburden soils at depths ranging from 7.0 m to 7.2 m below ground surface.

On the east plateau, at the location of the proposed east abutment and retaining wall, the subsurface conditions generally consist of fill overlying mixed soils of silt to sandy silt/silty sand to clayey silt. Clayey silt (residual) soil was encountered overlying the weathered shale bedrock, which was encountered at depths ranging from 4.8 m to 8.1 m below ground surface.

A more detailed description of the subsurface conditions encountered in the boreholes from the previous and current investigations at and/or in the immediate vicinity of the proposed foundation units for the new bridge and retaining wall are provided in the following sections.

4.2.1 Asphalt and Concrete

Approximately 150 mm of apshalt underlain by approximately 150 mm of concrete was encountered in Borehole NW6-1 at ground surface.

4.2.2 Topsoil

Topsoil was encountered in Boreholes CRB-2, CRB-2A, CRB-2B (west abutment) and CRB-7, and CRB-8 (east abutment and east approach) at ground surface and ranged in thickness from about 80 mm to 200 mm.

4.2.3 Fill

Fill was encountered in all boreholes except Boreholes CRB-3A and CRB-3C, which were advanced near the west bank of the Credit River at the west abutment. The fill was generally encountered below the topsoil and concrete, or at the ground surface and is variable in composition ranging from non-cohesive to cohesive soils.

Non-cohesive fill consisting of silt and sand to silty sand to gravelly sand was encountered in Boreholes CRB-2, CRB-3, CRB-5 to CRB-8 and NW6-1 and the thickness generally ranges from about 0.7 m to 2.7 m, with the exception of Borehole CRB-7 where the non-cohesive fill extended to a depth of 4.5 m below ground surface.

The cohesive fill ranges from sandy clayey silt, to clayey silt and was encountered in Boreholes CRB-1 to CRB-5, CRB-2A, CRB-2B, CRB-5A underlying the topsoil/non-cohesive fill or at ground surface and the thickness ranges from 0.5 m to 1.7 m, where encountered with the exception of Boreholes CRB-4 and CRB-5A, where the cohesive fill extends to depths of 3.7 m and 4.0 m below ground surface, respectively.

It is noted that in the vicinity of the east pier the ground surface is currently covered with rip-rap and other cobble and boulder sized rock.

The SPT “N” values measured within the non-cohesive fill generally range from 3 blows to 60 blows per 0.3 m of penetration, indicating that the fill layer has a very loose to very dense compactness condition. One SPT “N” value of 54 blows per 0.3 m of penetration was recorded in Borehole CRB-7, but it is in inferred that the value may be affected by the split-spoon penetrating pieces of brick/brick fragments. The SPT “N” values measured within the cohesive fill generally range from 6 blows to 31 blows per 0.3 m of penetration, suggesting that the cohesive fill layers have a firm to hard consistency.

Grain size distribution tests were carried out on four samples of the non-cohesive fill material and the results are shown on Figure C-1 in Appendix C. The non-cohesive fill contains trace gravel to gravelly, trace clay and brick fragments. The presence or organic material including wood fragments and rootlets were noted within the non-cohesive fill at boreholes located at the proposed west abutment (Borehole CRB-2) and at the proposed east pier (Boreholes CRB-5 and CRB-5A). The water content measured on samples of the non-cohesive fill range between about 4 per cent and 25 per cent.

Grain size distribution tests were carried out on three samples of the cohesive fill material and the results are shown on Figure C-2 in Appendix C. The cohesive fill contains shale, limestone and brick fragments, pieces of wood, organics and rootlets. Atterberg limits tests were carried out on three samples of the cohesive fill and measured liquid limits ranging from about 23 per cent to 28 per cent, plastic limits ranging from about 14 per cent to 20 per cent and plasticity indices ranging from about 8 per cent to 11 per cent. The results of the Atterberg limits test are plotted on the plasticity chart on Figure C-3 in Appendix C and indicate the cohesive deposit consists of low plasticity clayey silt. The water content measured on samples of the cohesive fill range between about 8 per cent and 42 per cent.

4.2.4 Clayey Silt with Sand to Silty Clay

In all boreholes with the exception of Boreholes CRB-2 (west abutment), CRB-5 and CRB-5A (east pier), CRB-7 (east abutment) and NW6-1 (east retaining wall), a cohesive deposit ranging in variability from silty clay, to clayey silt with sand, to sandy clayey silt, to silty clay was generally encountered underlying the fill materials, but was located at ground surface (in Boreholes CRB-3A and CRB-3C on the west bank of the river), and underlying the native sand and silt deposit (in Borehole CRB-8 at the east approach) at thicknesses ranging from 0.1 m to 2.7 m. The surface of the deposit was encountered at ground surface in Boreholes CRB-3A and CRB-3C and at depths ranging from about 0.7 m to 6.4 m below ground surface in the other boreholes.

The SPT “N” values measured within the cohesive deposit range from 0 blows (weight of hammer) to 14 blows per 0.3 m of penetration. Two in-situ field vane tests carried out within the cohesive deposit in Borehole CRB-3A (west pier) measured undrained shear strengths of about 22 kPa and 58 kPa. The calculated sensitivities were about 1.5 and 3.2. The field vane test results together with the SPT “N” values indicate that the cohesive deposit has a very soft to stiff consistency.

Grain size distribution test were carried out on eight selected samples of the cohesive deposit and the results are presented on Figure C-4A and C-4B in Appendix C. The cohesive deposit was noted to contain organics and rootlets. Cobbles were encountered within this cohesive deposit in Borehole CRB-3C, while advancing the wash boring casing at depths of 1.2 m and 2.4 m below ground surface. Atterberg limits tests were carried out on seven samples of this deposit and measured liquid limits ranging from about 23 per cent to 42 per cent, plastic limits ranging from about 14 per cent to 25 per cent, and plasticity indices ranging from about 9 per cent to 21 per cent. These laboratory results are plotted on the plasticity chart on Figure C-5 in Appendix C and confirm that the cohesive deposit is comprised of clayey silt (with sand) of low plasticity to silty clay of intermediate plasticity.

The natural water content measured on thirteen samples of the cohesive deposit ranges from about 14 per cent to 42 per cent.

4.2.5 Silt to Silt and Sand to Sand

In Boreholes CRB-5 and CRB-5A (east pier), CRB-7 and CRB-8 (east abutment/approach) and NW6-1 (East Retaining Wall) an approximately 0.5 m to 4.9 m thick granular deposit that varies in composition from silt to silt and sand to sand was encountered underlying the fill. This deposit contains a layer of organic clayey silt in Boreholes CRB-5 and CRB-5A (at the east pier). The depth and elevation of the top and bottom of this granular deposit and the corresponding thickness and soil type are summarized below.

Foundation Unit	Borehole No.	Top of Layer		Bottom of Layer		Thickness (m)	Soil Type
		Depth (m)	Elevation (m)	Depth (m)	Elevation (m)		
East Pier	CRB-5A	4.0	75.3	4.5	74.8	0.5	Silty Sand
		6.4	72.9	7.2	72.1	0.8	
	CRB-5	2.4	76.8	4.7	74.5	2.3	Silty Sand to Silt and Sand
		5.7	73.5	7.2	72.0	1.5	Silty Sand
East abutment and Approach	CRB-7	4.5	90.2	6.5	88.2	2.0	Sandy Silt
	CRB-8	1.5	93.3	3.7	91.0	2.2	Sand
		3.7	91.0	6.4	88.3	2.7	Silt
East Retaining Wall	NW6-1	3.0	92.3	5.3	90.0	2.3	Silty Sand
		5.3	90.0	7.3	88.0	2.0	Silt

In Boreholes CRB-5A and CRB-5 (east pier), the SPT “N” values measured within the granular deposit generally range from zero (weight of rods) to 7 blows per 0.3 m of penetration, indicating a very loose to loose compactness condition. In Boreholes CRB-7 and CRB-8, advanced at the east abutment and approach, the SPT “N” values measured within the granular deposit generally range from 17 blows to 67 blows per 0.3 m of penetration, indicating a compact to very dense compactness condition. In Borehole NW6-1, advanced at the south side of the existing QEW east of the credit river, the SPT “N” values measured within the granular deposit generally range from 3 blows to 41 blows per 0.3 m of penetration, indicating a very loose to dense compactness condition.

Grain size distribution tests were carried out on eight selected samples of the granular deposit and the results are presented on Figure C-6A and Figure C-6B in Appendix C. In Boreholes CRB-5A and CRB-5, advanced at the east pier, the granular deposit contains rootlets, shell fragments and pieces of wood. The organic content measured on a sample of this deposit was about 1 per cent. Atterberg limits tests were carried out on two samples of the silt to sandy silt deposit; one test result indicates the soil to be non-plastic and the results of the second test measured a liquid limit of about 19 per cent, a plastic limit of about 16 per cent, and a plastic index of about 3 per cent. These test results are plotted on the plasticity chart on Figure C-7 in Appendix C and confirm that the granular deposit is comprised of silt of slight plasticity to non-plastic. The water content measured on thirteen samples the silty sand to sandy silt deposit ranges from about 15 per cent to 28 per cent.

4.2.6 Sand and Gravel

In Boreholes CRB-3, CRB-3A and CRB-3C (advanced at the west side of the Credit River), a deposit of sand and gravel was encountered underlying the cohesive deposits. The surface of the sand and gravel deposit was encountered at depths of between about 2.6 m and 3.0 m below ground surface (between Elevations between 73.1 m and 72.7 m) and ranges in thickness from about 2.6 m to 3.6 m.

The SPT “N” values measured within the sand and gravel deposit generally range from 14 blows to 55 blows per 0.3 m of penetration, indicating a compact to very dense compactness condition. One SPT “N” value of 72 blows for 100 mm of penetration was measured in Borehole CRB-3C, which is inferred to be due to the presence of cobbles at the sampling depth.

Five grain size distribution tests were carried out on selected samples of the sand and gravel deposit and the results are shown on Figure C-8 in Appendix C. The sand and gravel contain some silt, trace to some clay and shell fragments were noted within the deposit in Borehole CRB-3. A clayey silt pocket was encountered within the granular deposit in Borehole CRB-3A at a depth of 4.7 m below ground surface and clayey silt layers/pockets were encountered within samples SS6 and SS8 in Borehole CRB-3C, between depths of 3.5 m and 4.7 m below ground surface.

Atterberg limits tests were carried out on the clayey silt layers/pockets from two samples of this deposit from Borehole CRB-3C and measured liquid limits of 23 per cent and 25 per cent, plastic limits of 16 per cent and 17 per cent, and plasticity indices of 6 per cent and 9 per cent. These results are plotted on the plasticity chart on Figure C-9 in Appendix C and confirm that these pockets are clayey silt of low plasticity.

The water content measured on eight samples of the sand and gravel deposit ranges between 12 per cent and 14 per cent.

4.2.7 Organic Clayey Silt with Sand

In Boreholes CRB-4, CRB-5 and CRB-5A (east pier), a deposit of organic clayey silt with to some sand, was encountered at depths of between about 4.5 m and 6.2 m (between Elevations 74.8 m and 73.0 m) below ground surface and ranges in thickness from about 0.8 m to 1.9 m.

The SPT “N” values measured within the organic deposit range from 1 blow to 7 blows per 0.3 m of penetration, suggesting a very soft to firm consistency.

Three grain size distribution tests were carried out on selected samples of the organic clayey silt deposit and the results are shown on Figure C-10 in Appendix C. The organic soil contains trace gravel, sand lenses and pockets of cohesive clayey silt as well as wood and shell fragments. Atterberg limits tests were carried out on three samples of this deposit and measured liquid limits ranging from about 38 per cent to 46 per cent, plastic limits ranging from about 26 per cent to 35 per cent, and plasticity indices ranging from about 6 per cent to 17 per cent. These results are plotted on the plasticity chart on Figure C-11 in Appendix C and indicate that the deposit consists of organic clayey silt of intermediate plasticity.

Organic content tests completed on two samples from this deposit measured 5.5 per cent and 7.1 per cent. The water content measured on samples from the organic deposit ranged from 27 per cent to 47 per cent.

4.2.8 Clayey Silt (Till)

In Borehole CRB-2 (west abutment), in Borehole CRB-7 (east abutment) and in Borehole NW6-1 (east retaining wall) a deposit of clayey silt to clayey silt with sand (till) was encountered underlying the fill and just above the bedrock in Borehole CRB-2; underlying the fill and sandy silt deposit and overlying residual soil and bedrock in Borehole CRB-7; and underlying the silt deposit in Borehole NW6-1 The surface of the clayey silt till was encountered at depths of 2.6 m, 6.5 m, and 7.3 m (Elevations 93.0 m, 88.2 m and 88.0 m) and has a thickness of 0.6 m and 1.3 m at these borehole locations. Borehole NW6-1 terminated in this till deposit at a depth of 7.5 m below ground surface (Elevation 87.8 m).

The SPT “N” values measured within the clayey silt till deposit are 10 blows per 0.3 m of penetration and 50 blows for 150 mm of penetration, suggesting a stiff to hard consistency.

The deposit consists of clayey silt with sand to some sand, trace to some gravel. Two grain size distribution tests were carried out on selected samples of the cohesive till deposit = and the results are shown on Figure C-12 in Appendix C. Atterberg limits tests were carried out on three samples of this cohesive till deposit and measured liquid limits of about 23 per cent to 26 per cent, plastic limits of about 14 per cent to 15 per cent, and plasticity indices of about 9 per cent to 11 per cent. These results are plotted on the plasticity chart on Figure C-13 in Appendix C and indicate that the cohesive till deposit consists of clayey silt of low plasticity.

The natural water content measured on three samples of the clayey silt till are about 14 per cent.

4.2.9 Residual Soil

In Boreholes CRB-2B (west abutment), CRB-3 (west pier), and CRB-6, CRB-7 and CRB-8 (east abutment/approach), a deposit of residual soil was encountered, immediately overlying the bedrock. The surface of the residual soil deposit was encountered at depths of between about 1.4 m and 7.8 m (between about Elevations 93.3 m and 70.3 m) and ranges in thickness from about 0.3 m to 2.2 m. Residual soil represents a heterogeneous mix of severely weathered rock where, in some zones it is indistinguishable from sedimentary materials, while in other zones it retains a degree of the original parent bedrock structure and strength. While advancing Borehole CRB-2B auger refusal was encountered at a depth of 2.1 m (Elevation 92.6 m) below ground surface and the borehole was advanced beyond this depth by HQ coring. At the top of the 1.5 m long core run a 0.25 m thick layer of limestone was encountered; this is possibly a boulder or a limestone layer within the residual soil.

The SPT “N” values measured within the residual soil deposit range from 61 blows per 0.3 m of penetration to 150 mm of penetration, suggesting a hard consistency. The deposit is described as a clayey silt to a sandy clayey silt and contains shale fragments, derived from weathering of the underlying shale bedrock.

Grain size distribution tests were carried out on one selected sample of the clayey silt residual soil deposit and the results are shown on Figure C-14 in Appendix C, although it is noted that the SPT sample tested could not contain larger fragments of rock on account of the sampler size and, within residual soils in general, larger fragments of unweathered rock must be expected. Atterberg limits test were carried out on two samples of this deposit and measured liquid limits of about 23 per cent and 24 per cent, plastic limits of about 15 per cent and 16 per cent, and plasticity indices of about 7 per cent and 8 per cent. These results are plotted on the plasticity chart on Figure C-15 in Appendix C and indicate that the residual soil consists of clayey silt of low plasticity.

The natural water content measured on four samples of the clayey silt residual soil deposit range between 9 per cent and 12 per cent.

4.2.10 Bedrock

Bedrock was encountered and core samples were recovered in all boreholes advanced during the current investigation for this bridge with the exception of Boreholes CRB-1 (west approach) and CRB-8 (east approach), where the bedrock surface was inferred from auguring and/or split-spoon sampling, and with the exception of Borehole NW6-1 where the borehole was terminated prior to encountering refusal. Bedrock was also encountered and cored in Boreholes 10-02, 11-01 and 11-02, advanced during the previous (2010 and 2011) foundation investigations, by others. The depths to bedrock or refusal below ground surface, and the corresponding bedrock surface elevation or refusal elevation, and the cored depth are summarized below.

Foundation Element / Approach	Borehole	Depth to Bedrock Surface /Refusal (m)	Bedrock Surface / Refusal Elevation (m)	Comments
West Approach	CRB-1	1.7	93.2	Augering and split-spoon sampling; 1.6 m penetration
West Abutment	CRB-2	3.2	92.4	Bedrock cored 9.6 m
	11-01[1]	3.3	91.3	0.3 m auger penetration; bedrock cored 3.5 m
	CRB-2A	1.1	93.4	Bedrock cored 7.9 m
	10-02[2]	3.2	91.2	1.4 m augering penetration and split-spoon sampling; bedrock cored 19.7 m
	CRB-2B	3.6	91.1	Soil cored 1.5 m, from Elevation 92.6 to 91.1 m; Bedrock cored 9.1 m
West Pier	CRB-3	6.3	69.6	0.8 m split-spoon penetration; bedrock cored 8.1 m
	CRB-3A	6.1	69.6	0.9 m split-spoon penetration; bedrock cored 8.8 m
	11-02[1]	6.3	69.4	Bedrock cored 2.1 m
	CRB-3C	6.2	69.1	0.2 m split-spoon penetration; bedrock cored 7.7 m
East Pier	CRB-4	7.0	72.1	0.2 m split-spoon penetration; bedrock cored 8.1 m
	CRB-5	7.2	72.0	Bedrock bored 8.3 m
	CRB-5A	7.2	72.1	0.5 m split-spoon penetration; bedrock cored 9.5 m
East Abutment	CRB-6	4.8	86.9	0.3 m split-spoon penetration; bedrock cored 8.2 m
	CRB-7	8.1	86.6	0.4 m split-spoon penetration; bedrock cored 7.5 m
East Approach	CRB-8	8.1	86.7	Split-spoon sampling; 0.4 m penetration

Note(s):

1. Thurber Engineering Ltd. File No. 19-1351-174, dated May 18, 2012 (GEOCRES 30M12-341).
2. Thurber Engineering Ltd. File No. 19-92-92-174, dated April 8, 2011 (GEOCRES 30M12-324)

In general, the bedrock surface as encountered or inferred in the area of the proposed bridge replacement slopes relatively steeply downwards towards the Credit River, on either side of the valley. At the locations of the boreholes drilled within the west and east river valley, and at the east valley plateau, the bedrock surface appears to be relatively flat. At the west valley plateau, the shallow bedrock surface appears to undulate across the proposed west abutment foundation unit. The bedrock surface at the west abutment as presented on the borehole records and on Drawing 2 indicates that the bedrock surface is generally at between about Elevation 92.4 m and 91.1 m; however, it was encountered at a higher elevation (93.4 m) at Borehole CRB-2A, advanced at the north end of the proposed abutment. It is likely that the upper portion of the bedrock is highly to completely weathered and the difference in elevation is a result of different interpretation of the contact between the residual soil and the highly to completely weathered bedrock.

Based on a review of the bedrock core samples from the current investigation and descriptions of the bedrock from the previous investigation, the bedrock consists of shale of the Georgian Bay Formation. In general, the bedrock samples are described as completely to moderately weathered to slightly weathered to fresh, thinly laminated to medium bedded, fine grained, non-porous to faintly porous, very weak to weak, grey shale, with strong limestone interbeds at varying intervals of depth. The strong limestone layers range in thickness from about 5 mm to as much as 510 mm, with an average thickness of about 50 mm. The stronger layers can comprise up to about 25 per cent by thickness of the rock encountered during the investigation, but generally make up between about 5 per cent to 19 per cent by thickness. The details of the bedrock descriptions are presented in the borehole records from the previous investigation in Appendix A, and on the Record of Drillhole sheets and photographs of the recovered bedrock core samples on Figures B-1 to B-11 from the current investigation in Appendix B. The degree of weathering of the bedrock samples (i.e. fresh to completely weathered – W1 to W5), and the strength classification of the intact rock mass based on field identification (i.e. very weak to strong – R1 to R4) are described in accordance with the International Society for Rock Mechanics (ISRM)[4] standard classification system. The surface and upper portion of the shale bedrock, generally had higher degrees of weathering, (completely to highly weathered) as shown on the Record of Drillhole sheets.

The Rock Quality Designation (RQD) measured on the core samples obtained from the current investigation ranges from about 0 per cent to 100 per cent, indicating a rock mass of very poor to excellent quality, but below the completely to highly weathered upper zone, the RQD is generally greater than 54 per cent, indicating fair to excellent quality of the bedrock at depth, as per Table 3.10 of CFEM (2006)[5]. The Total Core Recovery (TCR) and Solid Core Recovery (SCR) of samples recovered are between 11 per cent and 100 per cent and between 0 per cent and 100 per cent, respectively. Similar to RQD, both TCR and SCR increase below the upper completely to highly weathered zone of the shale bedrock. The Thurber report (GEOCRES No. 30M12-341) indicates that the RQD in Borehole 11-01 (west abutment) was practically zero over the depth of the cored borehole (between Elevation 91.0 m and 87.5 m), but Thurber notes that the low RQD values may be a result of the coring equipment used in the tri-pod setup. At the west abutment, 0.5 m and 0.3 m zones of lost core were noted when logging the core samples retrieved from Boreholes CRB-2 and CRB-2A at Elevations of 86.0 m and 88.4 m, respectively. In addition, between Elevation 89.1 m and 88.1 m the RQD was about 40 per cent.

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[4] 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.

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

However, as discussed in Section 3.2.2, an acoustic and optical televiewer test was carried out and based on these results a “soft” zone having a thickness of about 0.1 m was identified at about Elevation 86.1 m.

Unconfined Compression (UC) tests (ASTM D7012)[6] were carried out on selected core samples of the shale bedrock and one sample of the interbedded limestone and the uniaxial compressive strength (UCS), bulk density and tangent Young’s modulus of the intact samples are summarised below and the details are presented on the Rock Laboratory Test Results report from Geomechanica in Appendix C.

Borehole No.	Sample Depth Interval (m)	Sample Elevation Interval (m)	Uniaxial Compressive Strength (UCS) (MPa)	Bulk Density (g/cm^3)	Tangent Young’s Modulus (GPa)
CRB-2	7.8 – 7.9	87.8 to 87.7	11.2	2.58	0.83
CRB-2	11.4 – 11.5	84.2 – 84.1	13.0	2.61	2.19
CRB-2A	4.3 – 4.5	90.2 – 90.0	18.2	2.59	0.75
CRB-2A	4.9 – 5.1	89.6 – 89.4	17.1	2.60	0.76
CRB-2B	6.9 – 7.1	87.8 – 87.6	12.1	2.59	0.59
CRB-2B	9.1 – 9.25	85.6 – 85.4	15.5	2.60	0.63
CRB-3	11.4 – 11.7	64.5 – 64.2	9.4	2.61	2.10
CRB-3A[1]	10.2 – 10.3	65.5 – 65.4	8.9	2.60	0.48
CRB-3A	13.0 - 13.3	62.7 – 62.4	16.9	2.62	0.67
CRB-3C[2]	7.9 – 8.0	67.4 – 67.3	114.1	2.61	22.91
CRB-4	13.6 – 13.8	65.5 – 65.3	18.6	2.61	0.84
CRB-5	13.7 – 13.9	65.5 – 65.3	15.5	2.61	0.61
CRB-5A	12.4 – 12.6	66.9 - 66.7	14.2	2.60	0.96
CRB-5A	15.3 – 15.6	64.0 – 63.7	22.7	2.64	0.93
CRB-6	6.1 - 6.2	85.6 – 85.5	14.6	2.17	0.63
CRB-7	9.2 - 9.4	85.5 – 85.3	15.5	2.59	0.65
CRB-7	12.1 – 12.4	82.6 – 82.3	7.4	2.59	1.28

Note(s):

1. Specimen included two 5 mm to 10 mm thick interbedded limestone layers.
2. Specimen consists of limestone.

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

Thirty-two axial and thirty-four diametral Point Load Tests (PLTs) were carried out on sixty-six samples of the shale bedrock, and the results are summarized in Table C1 in Appendix C.

Based on the laboratory UCS, in accordance with Table 3.5 in CFEM (2006)[4], the shale bedrock is generally classified as weak (R2, 5 MPa < UCS < 25 MPa) and the limestone interlayers are classified as very strong (R5, 100 MPa < UCS < 250 MPa).

4.2.11 Groundwater Conditions

The overburden samples obtained from the borehole investigations were generally moist to wet. Boreholes CRB-1, CRB-2, CRB-2A, CRB-2B, CRB-6 and CRB-7 were observed to be dry upon completion of drilling operations (and prior to start of rock coring operations, where relevant). The depths to the water level observed in the boreholes upon completion of drilling and prior to rock coring is presented below.

Foundation Unit	Borehole	Water Level Depth (m)	Water Elevation (m)	Comment
West Approach	CRB-1	Dry	Dry	Upon completion of drilling.
West Abutment	CRB-2	Dry	Dry	Upon completion of overburden drilling and prior to rock coring.
	CRB-2A	Dry	Dry	
	CRB-2B	Dry	Dry	
West Pier	CRB-3	2.7	73.2	
	CRB-3A	0.9	74.8	
	CRB-3C	0.7	74.6	
East Pier	CRB-4	3.8	75.4	
	CRB-5	4.3	74.9	
	CRB-5A	3.6	75.7	
East Abutment	CRB-6	Dry	Dry	
	CRB-7	Dry	Dry	
East Approach	CRB-8	1.5	93.2	Upon completion of drilling.
East Retaining Wall	NW6-1	6.3	89.0	Upon completion of drilling.

The water levels recorded in the standpipe piezometers installed during the current investigation are presented below:

Foundation Unit	Borehole	Stratum Well Sealed Into	Water Level Depth (m)	Water Elevation (m)	Date of Piezometer Reading
West Abutment	CRB-2	Fill/Clayey Silt Till	2.6	93.0	March 12, 2018
			2.6	93.0	April 30, 2018
			2.5	93.1	November 6, 2018
West Pier	CRB-3A	Sand and Gravel	1.0	74.7	March 12, 2018
			0.7	75.0	April 30, 2018
East Pier	CRB-5A	Silty Sand/Organic Clayey Silt	1.6	77.7	March 12, 2018
			4.0	75.3	April 30, 2018
			4.6	74.7	November 6, 2018
East Abutment	CRB-6	Shale Bedrock	5.6	86.1	November 12, 2017
			5.0	86.7	March 12, 2018
			4.9	86.8	April 30, 2018
			4.9	86.8	November 6, 2018

The water levels recorded in the standpipe piezometers during the previous (2010 and 2011) foundation investigation, are presented below. Additional water levels were measured in the standpipe piezometers in Boreholes 11-01 and 11-02, as part of this current investigation and are also presented in the table below.

Borehole	Stratum Well Sealed Into	Water Level Depth (m)	Water Elevation (m)	Date of Piezometer Reading
10-02	Silty Clay / Shale bedrock	Dry	Dry	December 17, 2018
	Shale bedrock	8.5	85.9	October 4, 2010
		8.5	85.9	October 5, 2010
		8.7	85.7	October 12, 2010
		9.3	85.1	December 17, 2018

Borehole	Stratum Well Sealed Into	Water Level Depth (m)	Water Elevation (m)	Date of Piezometer Reading
11-01	Shale bedrock	3.2	91.4	September 30, 2011
		Dry	Dry	February 12, 2018
		Dry	Dry	March 12, 2018
		Dry	Dry	April 30, 2018
11-02	Sand / Shale bedrock	0.7	75.0	June 8, 2011
		1.5	74.2	October 4, 2011
		0.0	75.7	February 12, 2018
		0.8	74.9	March 12, 2018
		0.9	74.8	April 30, 2018

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

4.2.11.1 Packer Test Results

As described in Section 3.2.1 packer testing was carried out during the borehole drilling program in selected boreholes at the piers. The following summarizes the hydraulic conductivity based on the results of the testing:

Borehole	Depth (Elevation)		Estimated Hydraulic Conductivity (m/s)
	From (m)	To (m)	
CRB-3	8.2 (67.7)	11.5 (64.4)	3.7×10^{-7}
CRB-3A	7.6 (68.1)	9.2 (66.5)	7.2×10^{-8}
CRB-4	9.5 (69.6)	11.7 (67.4)	8.3×10^{-7}
CRB-5	9.7 (69.5)	12.0 (67.2)	6.2×10^{-7}

4.2.12 Analytical Testing Results

As discussed in Section 3.2, four samples of crushed and homogenized shale bedrock core were submitted for analysis of parameters used to assess the potential corrosivity of the site bedrock to steel and concrete. The following summarizes the results of the testing:

Parameter	Borehole CRB-2	Borehole CRB-3A	Borehole CRB-5	Borehole CRB-6
pH	8.09	8.07	8.08	8.11
Resistivity (ohm-cm)	3,800	3,100	1,700	5,000
Electrical Conductivity (umho/cm)	263	321	582	201
Chlorides (ug/g)	<20	44	46	<20
Soluble Sulphates (ug/g)	56	49	160	30

5.0 CLOSURE

This report was prepared by Mr. David Marmor, E.I.T., a geotechnical engineer-in-training with Golder. Ms. Sandra McGaghran, M.Eng., P.Eng. a senior geotechnical engineer and Associate conducted a technical review of the report and Mr. Paul Dittrich, Ph.D., P.Eng., a MTO Foundations Designated Contact and Principal of Golder conducted a quality control review of the report.

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PART B

FOUNDATION DESIGN REPORT
QUEEN ELIZABETH WAY (QEW) - CREDIT RIVER BRIDGE, STRUCTURE SITE NO. 24-203, QEW WIDENING FROM WEST OF MISSISSAUGA ROAD TO WEST OF HURONTARIO STREET, MISSISSAUGA,
MINISTRY OF TRANSPORTATION, ONTARIO,
GWP 2002-13-00

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides detailed foundation engineering recommendations for design of the foundation units supporting the abutments and piers of the proposed new Queen Elizabeth Way (QEW) Credit River Bridge (Site 24-203 WB) as part of the widening of the Queen Elizabeth Way (QEW) from west of Mississauga Road to west of Hurontario Street in the City of Mississauga, in the Regional Municipality of Peel, Ontario. These recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the 2010/2011 (by Thurber) and 2017/2018 (by Golder) subsurface investigations. The discussion and recommendations presented herein are intended to provide the designers with sufficient information to assess the feasible foundation alternatives and carry out the design of the bridge foundations. The Foundation Investigation and Design 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 the Part A (Foundation Investigation) portion 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

Based on the General Arrangement (GA) drawing provided by MH on April 23, 2018, the proposed bridge will be constructed immediately to the north of the existing QEW Credit-River bridge with a separation distance of ±2.0 m between the old and new structures. The new bridge will consist of a three-span structure with piers on each side of the Credit River and abutments at the crests of the valley, which are approximately 18 m above the base of the valley. The centre-span between the piers will be 118 m long and the end-spans from each abutment to the closest pier will be 68 m long. Upon completion, the existing bridge will remain in operation and will carry eastbound traffic while the new bridge will carry westbound traffic. It is understood that the two bridges will not be structurally connected. Furthermore, based on email and telephone conversations between MH and Golder in January 2019, it is our understanding that the new east bridge abutment and associated approach embankment will be slightly lower (by about 1 m) than the existing bridge/roadway. Given this and considering that the new east abutment will be set-back from (i.e. further to the east of) the existing east abutment by about 5 m, a short section of retaining wall supporting the north side of the new east-bound lanes will be required to separate the two structures and approaches in this area. The retaining wall will be installed after the new bridge is in operation and all traffic has been temporarily shifted onto it. The retaining wall will be approximately 12 m long, 7.5 m high (the lower portion of the wall will be set about 4.2 m below the finished grade), with a maximum retained soil height of about 3.3 m, and have its footing aligned with the north wing wall of the existing east abutment and founded at an elevation equal to the underside of the existing east abutment footing at approximately Elevation 88.0 m (as discussed in the paragraphs below).

The elevation at the base of the flood plain is about Elevation 76 m, while the water level in the Credit River, as provided by MH, is at about Elevation 75 m (Sept. 1986). The elevation of the east and west plateaus at the top of the valley is at about Elevation 95 m. The proposed grade of the new westbound lanes will slope downward across the length of the bridge from west to east from about Elevation 96.1 m to 94.8 m.

Based on the 1934 and 1959 Department of Highways contract drawings, the existing bridge is a seven-span structure supported on shallow foundations. The original bridge (circa 1934) was constructed with a width of

15.8 m, but in 1959 the structure was widened to 26.8 m. The existing foundations for the West Abutment and Piers #1 to #5, appear to have been designed to be supported on the shale bedrock. However, the design founding conditions for the existing most easterly pier (Pier #6) and the East Abutment are not clear; the information shown on the available design drawings suggests that these elements were to be founded on ‘shale and clay’ and/or ‘blue clay’. As part of the design for the 1959 widening, Trow, Soderman and Associates (1958) carried out an investigation to determine the as-built elevation(s) of the underside of the original (circa 1934) bridge footings. Based on the drilling carried out, the underside elevation of the West and East Abutments and Piers #1 to #6 were calculated from the lowest depth of sound concrete recovered in the cores. The borehole records from the investigation also indicate the material encountered below the sound concrete which in most cases is identified as shale with no mud seams noted.

The summary borehole records from Trow, Soderman and Associates (1958) for the boreholes advanced through the original (1934) east abutment indicate that either no core recovery or very low recovery (<17%) was obtained below the sound concrete interface and the possibility of the footing being founded on shale bedrock was only inferred based on observations of the drilling process and the elevation of the exposed shale bedrock face on the adjacent valley bank/slopes. The elevation of the as-constructed underside of the original (1934) east abutment footing (north side) is at Elevation 88.0 m (288.8 feet) as reported by Trow, Soderman and Associates (1958). In addition, the underside of the east abutment footing (north side) is identified on the March 1959 drawing of the Credit River Bridge widening on QEW to be at Elevation 88.0 m (288.8 feet). It is noted that Elevation 88.0 m is approximately 1.4 m above the top of bedrock as encountered in Borehole CRB-7, which is the closest borehole advanced adjacent to the north side of the existing east abutment as part of the current investigation. As such, the founding condition of the existing east abutment is not certain and there is a risk that the existing footing is founded on the stiff till or hard residual soil overlying the bedrock.

6.2 Consequence and Site Understanding Classification

The proposed Credit River bridge will carry large volumes of traffic with the potential to impact alternative transportation corridors. In accordance with Section 6.5 of the 2014 Canadian Highway Bridge Design Code and its Commentary (CHBDC 2014), the proposed bridge structure and its foundation system is 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 at this location in comparison to the degree of site understanding in Section 6.5 of the 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.

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 borehole investigations. Based on the anticipated foundation levels on/within the bedrock, the site may be classified as Site Class C (very dense soil and soft rock) in accordance with Table 4.1 of the CHBDC (2014), in the absence of any geophysical testing. Geophysics testing, if carried out, may provide a more favourable Site Class designation.

6.3.2 Spectral Response Values and Seismic Performance Category

Based on the location of the QEW-Credit River Bridge (Latitude: 43.556034° ; Longitude: -79.610628°), the reference Site Class C spectral acceleration values were obtained based on the 5th generation seismic hazard maps published by the Geological Survey of Canada (GSC).

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 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)	0.041	0.076	0.146
PGV (m/s)	0.031	0.051	0.093
Sa (0.2) (g)	0.069	0.121	0.227
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 Foundation Options

Based on the information obtained from the boreholes advanced at the foundation elements, the overburden soils are not considered suitable to support the foundations for the new Credit River bridge. Both shallow and deep foundations founded on shale bedrock are considered suitable for support of the new bridge and these options have been considered in the following sections.

Steel H-piles have been considered for support of the foundations however, due to the relatively thin overburden at the foundation elements (i.e. 1.1 m to 7.0 m) and the requirements to embed pile caps at least 1.2 m deep (for frost protection) or deeper (i.e. at the east pier to ensure a minimum cover below the riverbed), the pile lengths at most locations would be too short to develop sufficient lateral resistance requiring that H-piles be embedded in concrete filled sockets drilled into the bedrock. While such a foundation option is technically feasible it is not preferred as it is not seen to offer any significant advantage over the drilled shaft (caisson) or shallow foundation options.

Based on the GA drawing, to maintain the integrity of the existing bridge during construction, temporary protection systems will be required along the north side of the existing bridge where the new foundation elements are in close proximity to existing foundation elements, to allow for excavation for shallow footings or pile caps and construction of new abutment walls supported on new footings. The recommendations for temporary protection systems and requirements to support existing footings are discussed further in Section 6.11.3.

A comparison of the foundation options based on feasibility, advantages, disadvantages, constructability / risks / consequences and relative costs is provided in Table 1 following the text of this report and is summarized below.

- **Shallow foundations founded on native soil deposits – spread/strip footings:** Given the low strength of the overburden soils and relatively shallow depth to bedrock, shallow footings founded on native soils are not considered to be practical for this site from a foundations perspective.
- **Shallow foundations founded on shale bedrock – spread/strip footings:** Shallow foundations comprised of strip/spread footings founded on the slightly weathered to fresh shale bedrock are feasible for support of the abutments and piers. However, significant depths of excavation (ranging from about 8 m to 9 m deep at the abutments, and 9 m to 10 m deep at the piers) through existing fills, native soils and into the bedrock will be required. The deep excavations at the piers would require cofferdams extending to and sealed into bedrock to reduce groundwater/river water inflows. Excavations at the abutments will be shallower and groundwater control will be easier to manage; however, temporary protection systems will be required to support the adjacent highway approach embankments and, at the east abutment, additional protection measures may be required to ensure adequate support of the existing abutment foundation. Based on the above, shallow footings on bedrock are considered the preferred foundation alternative for support of the new abutments, but not for the piers.
- **Deep foundations – drilled shafts (caissons) socketed into shale bedrock:** Drilled shafts (caissons) socketed into the shale bedrock are considered feasible and the preferred option for the support the new piers. At the west pier, although a cofferdam will still be required for construction, the pile cap can be located within the overburden thereby significantly reducing the excavation depth. At the east pier, because of the constraint requiring the top of pile cap to be embedded a minimum 0.6 m below the riverbed, a cofferdam and significant depth of excavation (up to about 8 m including extending about 1 m into bedrock) will still be required, although the total excavation depth will be less than that needed for the shallow foundation option. It is also anticipated that the requirements for groundwater control and for sealing the cofferdam will be reduced since a portion of the work at the piers (including installing the caissons) could be constructed in-the-wet.
- **Deep foundations – H-Piles socketed into shale bedrock:** Given the relatively shallow depth to bedrock and low strength of the overburden soils, H-piles would have to be embedded in concrete filled sockets drilled into the bedrock and as such, this foundation option is not preferred as it is not seen to offer any significant advantage over the drilled shaft (caisson) or shallow foundation options described above.

Based on discussions with the structural designers, it is understood that there is a requirement that the differential settlement between the abutment(s) and pier(s) (on the east and west side of the Credit River) be limited to less than 7 mm. MH provided the unfactored permanent and live loads for the foundation elements on July 6, 2018 and the load acting on the pier is about 7 times that acting on the abutment. If the abutments and piers are all supported on spread footings, it is estimated that the differential settlement will be greater than 7 mm. Therefore, it is recommended that the piers be supported on drilled piers socketed into the shale bedrock in order to reduce the differential settlement(s) to the design tolerance (see Section 6.6.2 for further discussion).

Based on the above considerations and as detailed in the comparison of foundation alternatives in Table 1, we recommend that the abutments be supported on spread/strip footings founded on bedrock and the piers be supported on drilled shafts socketed into the shale bedrock, as the most technically feasible and cost-effective alternative from a foundations perspective.

6.5 Spread Footings

6.5.1 Footing Elevations

The abutments may be supported on strip or spread footings founded on shale bedrock at the recommended highest founding elevations provided below:

Foundation Element	Reference Borehole(s)	Bedrock Surface Elevation (m)	Highest Founding Elevation on Bedrock (m)
West Abutment	CRB-2 11-01 CRB-2A CRB-2B	92.4 91.3 93.4 91.1	87.4[1]
East Abutment	CRB-7 CRB-6	86.6 86.9	85.4[2]

Note(s):

1. Founding elevation for west abutment selected based on quality of bedrock and desire to maintain new footing at similar elevation as underside of existing west abutment footing.
2. Founding elevation for east abutment selected based on quality of bedrock. Founding elevation of existing east abutment footing is unclear, possibly ranging from approximately Elevation 88 m to 89 m. Depending on actual founding elevation and founding stratum of existing east abutment, protection measures may be required to support existing footing and prevent undermining during new construction.

As discussed in Section 6.1, the existing structure is supported on shallow foundations the majority of which are anticipated to be founded on the shale bedrock. Based on the information in Trow, Soderman (1958), the existing west abutment is anticipated to be founded at Elevation 87.4 m, while the existing east abutment may be founded at Elevation 88 m. The new footings must be founded on undisturbed, competent, slightly weathered to fresh bedrock, which is present at a lower elevation than the anticipated founding elevation of the existing east abutment. Given that the excavations for construction of the new east abutment may extend below the existing east abutment footing, a temporary protection system with Level 1b performance will be required, which may include pre-support of the shale bedrock excavation face, as discussed further in Section 6.11.3.

If the excavation in the bedrock is required to extend to below the highest founding elevations noted above in order to remove all loose, shattered and/or fractured rock within the area of the footing, or where bedrock excavation results in the creation of steps or troughs in the bedrock, the founding stratum could be levelled or raised using mass concrete. In this case the mass concrete, having a minimum thickness of 100 mm and a minimum of 28-day compressive strength of 20 MPa, shall be placed in the excavation within four hours of exposure of the founding level to protect the integrity of the subgrade. An Non-Standard Special Provision (NSSP) to address this item is included in Appendix D, which should be included in the Contract Documents.

The shale bedrock at the site contains horizontal bedding and will tend to break in slabs if removed with an excavator. Where bedrock excavation is required adjacent to the existing footings, it must be carried out using saw-cutting or line drilling techniques to avoid the unintentional removal of bedrock from below the existing footing. An NSSP to address this item is included in Appendix D, which should be included in the Contract Documents.

6.5.2 Geotechnical Resistance

Strip or spread footings constructed on the properly prepared shale bedrock excavation base founded at or below the design elevations given in the preceding section, should be designed based on the factored ultimate geotechnical resistances and factored serviceability geotechnical resistances (for 25 mm of settlement) given below.

Foundation Element	Founding Stratum	Footing Width	Factored Ultimate Geotechnical Resistance	Factored Serviceability Geotechnical Resistance
East and West Abutment	Slightly weathered to fresh shale bedrock	4 m	3,000 kPa	1,500 kPa
		6 m		1,000 kPa

Based on the design bridge loadings provided by MH on July 6, 2018, we understand that the unfactored permanent and live load acting on each abutment strip footing will be 23,000 kN and based on a strip footing having dimensions of 5.5 m by 27.6 m, the settlement of the strip footing(s) founded on the shale bedrock at the elevations noted in Section 6.5.1, is estimated to be less than about 5 mm. The factored geotechnical resistances provided above are given for loads that will be applied perpendicular to the surface of the footings. Where the load is not applied perpendicular to the footing, inclination of the load should be taken into account in accordance with Section 6.10.4 of the CHBDC (2014).

The footing subgrade should be inspected by a Foundation Engineer, in accordance with OPSS 902 *(Excavating and Backfilling Structures),* as amended by SP109S12, to check that all existing fill(s), concrete and native soils have been removed. Furthermore, following excavations into the bedrock, it will be necessary to clean, scale and remove all loose, shattered and/or fractured rock within the area of the footing to ensure proper concrete bond to the bedrock surface.

The shale bedrock subgrade will be susceptible to disturbance from ponded water, precipitation from inclement weather, wetting/drying, and/or construction traffic. If the concrete for the footings cannot be poured immediately after excavation and inspection, it is recommended that a concrete working slab having a minimum thickness of 100 mm and a minimum of 28-day compressive strength of 20 MPa be placed in the excavation within four hours of exposure of the founding level to protect the integrity of the subgrade. An NSSP to address this item is included in Appendix D, which should be included in the Contract Documents.

The point load and UC test results of the shale bedrock core samples, and field description of the recovered bedrock core samples indicate that the shale is weak but contains strong to very strong limestone / siltstone / dolostone inter-layers, and where excavations for the abutment foundations extend into this formation, appropriate construction equipment and procedures (such as hoe-ramming) will be required. It is recommended that an NSSP be included in the Contract Documents to warn the contractor of the strength and characteristics of the bedrock, and an example of such an NSSP is included in Appendix D. As further discussed in Section 6.11.5, vibration monitoring of the existing bridge structure is not anticipated to be required during hoe-ramming; however, it may be prudent to monitoring the surrounding residential structures.

6.5.3 Resistance to Lateral Loads

Resistance to lateral forces / sliding resistance between the new 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 or concrete working slab constructed directly on bedrock, or concrete footing on a concrete working slab, the sliding resistance may be calculated based on the unfactored coefficient of friction, tan δ, which can be taken as follows:

- Cast-in-place footing or working slab to shale bedrock: tan $\delta = 0.5$
- Cast-in-place footing to concrete working slab: tan $\delta = 0.7$

If necessary, the sliding resistance between the concrete footing and/or working slab and the bedrock at the abutments can be supplemented by dowelling into the bedrock. The horizontal resistance of the dowels is dependent on the strength of the bedrock, grout and steel. The dowels should have a minimum length within the slightly weathered to fresh bedrock of 1 m, and the structural strength of the dowels and compressive strength of the grout should not be exceeded. For uplift of the dowels, a factored resistance value of 250 kPa may be assumed for the grout-to-rock bond stress for ULS design. The actual bond stress along the rock-grout interface may vary from the design value given and it should therefore be verified in the field by pull-out testing. These values assume that construction is carried out in dry conditions. If dowels into bedrock are required, an NSSP should be included in the Contract Documents, and such an NSSP is included in Appendix D. The dowels should be comprised of corrosion-resistant steel based on the bedrock and groundwater conditions.

6.5.4 Frost Protection

The spread / strip footing should be founded at a minimum depth of 1.2 m below the lowest surrounding grade, including measured perpendicular to the sloping ground surface, to provide adequate protection against frost penetration (as interpreted from OPSD 3090.101 – *Foundation Frost Penetration Depths for Southern Ontario)*. As a guide, the MTO has adopted the use of 25 mm (1 inch) of rigid polystyrene foam insulation as equivalent to a 0.3 m reduction in soil cover, if required.

6.6 Caissons (Drilled Shafts)

6.6.1 Founding Elevations

Caisson foundations socketed into the shale bedrock are recommended for support of the bridge piers. Depending on the magnitude of the bridge loads, consideration can be given to design of the caissons based on end bearing and/or shaft resistance.

For caissons designed based on a combination of end-bearing and shaft friction, it is recommended that they be socketed two times the caisson diameter (L/D=2), or a minimum of 2 m whichever is greater, into Fair to Good Quality bedrock (an RQD greater than 50 per cent).

At the proposed west pier, given the location of the pier cap within the overburden (i.e. above the bedrock surface), liners extending into the upper weathered portion of the bedrock will be required during construction of drilled shafts to prevent “necking” of the concrete and ensure the integrity of the cross-section. At the proposed east pier, the use of liners for construction of the drilled shafts may or may not be necessary depending on the design of the cofferdam and the sequence of construction. All liners, where required for construction at the piers, shall be permanent and not be withdrawn. Water inflow into the drilled shafts at the pier locations should be expected given the proximity to the Credit River; therefore, placement of concrete by the tremie method will be required to install drilled shafts.

The following outlines the recommended founding elevation for the base of the drilled shafts at the piers based on end-bearing and shaft friction (L/D=2) design:

Foundation Unit	Borehole	Bedrock Surface Elevation (m)	Recommended[1] Base of Drilled Shaft Elevation (m)		
			Diameter = 1.2 m	Diameter = 1.5 m	Diameter = 1.8 m
West Pier	CRB-3	69.6	66.5	66.0	65.5
	CRB-3A	69.6			
	11-02	69.4			
	CRB-3C	69.1			
East Pier	CRB-5	72.0	69.5	69.0	68.5
	CRB-5A	72.1			
	CRB-4	72.1			

Note(s):

1. Based on a minimum ratio of rock socket length (L) / diameter (D) = 2.

Alternatively, to reduce the requirements for cleaning and inspection of the base of the rock sockets, the caissons may be designed based on shaft resistance only. Based on the RQD of the bedrock core samples obtained from the boreholes, the fractures per 0.3 m and observations of the quality of the bedrock, it is recommended that the top of the caisson rock sockets commence at or below the maximum top of rock socket elevations as summarized below for each pier:

Foundation Unit	Borehole	Bedrock Surface Elevation (m)	Recommended Highest Top of Rock Socket Elevation (m)
West Pier	CRB-3	69.6	68.0
	CRB-3A	69.6	
	11-02	69.4	
	CRB-3C	69.1	
East Pier	CRB-5	72.0	70.5
	CRB-5A	72.1	
	CRB-4	72.1	

The shale bedrock is weak with unconfined compressive strengths generally in the range of 10 MPa to 20 MPa, but it contains strong to very strong limestone/siltstone/dolostone layers, and as such the rock sockets would likely have to be advanced into the bedrock by churn drilling or rock coring. It is recommended that an NSSP be included in the Contract Documents to describe to the Contractor the strength and characteristics of the bedrock; an NSSP is included in Appendix D for this purpose.

6.6.2 Geotechnical Axial Resistances

Based on the bridge loadings provided by MH on January 14, 2019, we understand that the unfactored combined permanent and live load acting on each caisson will be 21,000 kN and that the maximum permissible settlement of the caissons will be 10 mm with a maximum differential of 7 mm. Considering this target load and settlement tolerance, the following caissons alternatives are provided. For caissons designed based on a combination of shaft resistance and a portion of the end-bearing resistance within the rock socket and founded at the elevations provided in Section 6.6.1 for various caisson diameters, the following values of factored ultimate geotechnical resistance and factored serviceability geotechnical resistances (for 25 mm and 10 mm of settlement) given below may be used for design:

Length of rock socket to diameter of caisson = 2

Foundation Unit	Diameter (m)	Factored Ultimate Geotechnical Resistance (kN)	Factored Serviceability Geotechnical Resistance (kN)	
			(25 mm Settlement)	(10 mm Settlement)
Piers	1.2	5,500	--[1]	--[1]
	1.5	8,700	--[1]	--[1]
	1.8	12,500	--[1]	--[1]

Note(s):

1. The factored serviceability geotechnical resistance at SLS (for 25 mm of settlement) is greater than the factored ultimate geotechnical resistance at ULS therefore the ULS will govern the design.

The above values are only valid providing that the side walls (shaft) and base of the rock socket are properly cleaned and inspected with a video camera, as per the discussion below and as per the NSSP in Appendix D.

For caissons designed based on rock socket shaft resistance only and constructed with the maximum top of rock socket at the elevations provided in Section 6.6.1, the following values of factored ultimate geotechnical resistance and factored serviceability geotechnical resistances (for 25 mm and 10 mm of settlement) given below, based on shaft resistance of the rock socket only, may be used for design:

Length of rock socket to diameter of caisson = 3

Foundation Unit	Diameter (m)	Factored Ultimate Geotechnical Resistance (kN)	Factored Serviceability Geotechnical Resistance (kN)	
			(25 mm Settlement)	(10 mm Settlement)
Piers	1.2	6,500	--[1]	--[1]
	1.5	10,000	--[1]	--[1]
	1.8	14,500	--[1]	--[1]

Note(s):

1. The factored serviceability geotechnical resistance at SLS (for 25 mm of settlement) is greater than the factored ultimate geotechnical resistance at ULS therefore the ULS will govern the design.

Length of rock socket to diameter of caisson = 4

Foundation Unit	Diameter (m)	Factored Ultimate Geotechnical Resistance (kN)	Factored Serviceability Geotechnical Resistance (kN)	
			(25 mm Settlement)	(10 mm Settlement)
Piers	1.2	8,500	--[1]	--[1]
	1.5	13,500	--[1]	--[1]
	1.8	19,500	--[1]	19,000

Note(s):

1. The factored serviceability geotechnical resistance at SLS (for 25 mm of settlement) is greater than the factored ultimate geotechnical resistance at ULS therefore the ULS will govern the design.

Length of rock socket to diameter of caisson = 5

Foundation Unit	Diameter (m)	Factored Ultimate Geotechnical Resistance (kN)	Factored Serviceability Geotechnical Resistance (kN)	
			(25 mm Settlement)	(10 mm Settlement)
Piers	1.2	11,000	--[1]	--[1]
	1.5	17,000	--[1]	18,000
	1.8	24,000	--[1]	21,500

Note(s):

1. The factored serviceability geotechnical resistance at SLS (for 25 mm of settlement) is greater than the factored ultimate geotechnical resistance at ULS therefore the ULS will govern the design.

The above values are only valid providing that the side walls (shaft) of the rock socket are properly cleaned and inspected with Shaft Inspection Device (SID) such as a video camera, as per the discussion below and as per the NSSP in Appendix D.

The centre-to-centre spacing between the caissons should be greater than 2.5 times the caisson diameter to limit interaction between caissons. Provided this minimum drilled shaft 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).

During advancement of the rock socket, it must be filled with water at all times, until concrete is placed in the rock socket using tremie methods. Following drilling of the rock sockets, the walls of the socket and the base will be covered by a “cake” of rock powder/mud from the drilling operations. Therefore, flushing and cleaning of the rock sockets walls and base are required and this requirement must be included in the Contract Documents. For drilled shafts designed based on a combination of end-bearing resistance and a portion of the shaft resistance within the rock socket, the base of each rock socket must be thoroughly cleaned to remove all loose cuttings to ensure that the tremied concrete is in intimate contact with the competent shale bedrock. Upon completion of flushing and cleaning of the rock socket, the base will need to be inspected by means of a Shaft Inspection Device (SID) such as a video camera to confirm that the base is free of debris. For the caissons that are designed based on shaft resistance only, the rock socket walls of the caissons are required to be free from mud slurry; therefore, flushing and cleaning of the rock sockets walls are required and this requirement must be included in the Contract Documents. Following acceptance of the rock socket by the Foundation Engineer, concrete must be placed using tremie methods within 6 hours of the final cleaning and within seven days from when the rock socket was completed (i.e. prior to cleaning). An NSSP for cleaning requirements and concreting of the caisson is provided in Appendix D. All drilled installation should be carried out in accordance with OPSS.PROV 903 (*Deep Foundations*), as amended by SP 109F57.

6.6.3 Resistance to Lateral Loads

The resistance to lateral loading will be derived solely from the soil and bedrock in front of the vertical drilled shafts. Where ground conditions are generally competent and the lateral loads on the drilled shafts are relatively small such that the maximum lateral drilled shaft deflections will be relatively small, the resistance to lateral loading in front of a single drilled shafts can be estimated using subgrade reaction theory (as outlined below). However, it should be noted that the response of a caisson to lateral loads is highly nonlinear and methods that assume linear behavior (such as subgrade reaction theory) are only appropriate where the maximum horizontal drilled shafts deflections are less than 1 percent of the caisson diameter, where the loading is static (no cycling) and where the caisson 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 drilled shafts 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: n_h is the constant of horizontal subgrade reaction (kPa/m), as given below;

Z is the depth (m) below the in-ground drilled shaft cap; and, B is the drilled shaft diameter/width (m)

For cohesive soils:

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

Where: s_u is the undrained shear strength of the soil (kPa); and,

B is the drilled shaft diameter/width (m)

For completely to moderately weathered shale bedrock:

$$k_h = \frac{50{,}000}{B}$$

Where: k_h is the coefficient of horizontal subgrade reaction in kPa/m

B is the drilled shaft diameter/width (m)

For slightly weathered to fresh shale bedrock:

$$k_h = \frac{350{,}000}{B}$$

Where: k_h is the coefficient of horizontal subgrade reaction in kPa/m

B is the drilled shaft diameter/width (m)

The following values of n_h and s_u (Terzaghi, 1955 and American Petroleum Institute (API), 2002) may be incorporated into the calculations of horizontal subgrade reaction (k_h) for structural analyses for a single vertical drilled shaft. The ranges in values reflect the variability in the subsurface conditions, the soil properties and the approximate nature of the analysis and the non-linear nature of the soil behaviour (such that k_h is a function of deflection).

Foundation Element	Soil Unit	Elevation Interval (m)	n_h (kPa/m)	s_u (kPa)
West Pier (Boreholes CRB-3, CRB-3A, 11-02 and CRB-3C)	Soft clayey silt with sand	72.9[1] to 72.2	--	20
	Compact to dense sand and gravel (below the water table)	72.4 to 69.6	7,500	--
	Completely to Moderately Weathered Shale Bedrock	69.6 to 68.0	See above equations	
	Slightly Weathered to Fresh Shale Bedrock	Below 68.0		
East Pier (Boreholes CRB-5, CRB-5A and CRB-4)	Completely to Moderately Weathered Shale Bedrock	71.2[1] to 69.5	See above equations	
	Slightly Weathered to Fresh Shale Bedrock	Below 69.5		

Note(s):

1. Underside of caisson cap elevation.

Based on the above, both the structural and geotechnical resistances of the piles (i.e. caissons/drilled shafts) should be evaluated to establish the governing case at ULS. At SLS, the horizontal resistance of the piles will be controlled by deflections and the horizontal resistance of the piles should be calculated based on the coefficient of horizontal subgrade reaction (k_h) of the soil as discussed above. The SLS resistance should be taken as that corresponding to a horizontal deflection of 10 mm at the underside of the pile cap for units supporting the abutments (see Section C6.11.2.2.2 of the Commentary to the CHBDC, 2014).

The upper zone of the soil (down to a depth below the caisson cap equal to about 1.5 times B (after Broms, 1964, where B is the pile diameter) should be neglected in the calculation of lateral resistance of the caisson to account for disturbance effects during installation.

Group action for lateral loading should also be evaluated by reducing the coefficient of horizontal subgrade reaction either in the direction of loading or perpendicular to the direction of loading by relevant group pile efficiency factors as outlined in Section C6.11.3.4 of the *Commentary to the CHBDC* (2014).

6.6.4 Frost Protection

Pile caps on the caisson foundations, if constructed below ground surface must be constructed not less than 1.2 m below the surrounding finished grade including measured perpendicular to the sloping ground surface to provide for protection from frost penetration, as interpreted from OPSD 3090.101 (*Frost Penetration Depths for Southern Ontario*).

6.7 RSS (Retained Soil System) Wall Separating Existing and Proposed East Embankments

It is understood that a mechanically-reinforced soil retaining system (retained soil system or RSS wall) is proposed for the retaining wall to be constructed between the existing and the proposed east abutments. The RSS retaining walls are to be designed for high performance and appearance in accordance with MTO's *RSS Wall Design Guidelines* (September 2008).

6.7.1 Founding Elevations

A typical RSS wall has a front facing panel system that is supported on a strip footing placed at a shallow depth below the ground surface in front of the wall. The facing footing (typically 200 mm thick) should be placed within a 500 mm thick levelling pad comprised of OPSS.PROV 1010 (*Aggregates*) Granular 'A', placed in accordance with OPSS.PROV 501 (*Compacting*), as detailed in Figure 5.2 of MTO's *RSS Wall Design Guidelines* (September 2008). There should be a minimum thickness of 300 mm of Granular 'A' below the facing footing. The compacted granular levelling pad should extend at least 1.0 m beyond the outside edge of the facing footing, then downward and outward at 1H:1V. If bedrock is encountered above the recommended founding elevation (as discussed below) then the bedrock should be subexcavated to the design elevations and replaced with OPSS.PROV 1010 (*Aggregates*) Granular 'A'. A Notice to Contractor addressing this requirement is included in Appendix D.

As detailed in Figure 5.22 of MTO's *RSS Wall Design Guidelines*, it is recommended that the underside of the levelling pad be founded at a minimum depth of 1.0 m below the final finished grade at the base of (i.e. in front of and measured perpendicular to the slope face) the RSS wall. As such, the minimum soil cover to the base of the wall and top of the footing/levelling pad should be 0.5 m below the final finished grade in front of the base of the RSS wall. Prior to placement of the levelling pad and the reinforced soil mass, all existing topsoil and loose or deleterious materials must be removed and the existing fill is required to be proof-rolled to identify any softened/disturbed areas for sub-excavation and replacement, where applicable, with compacted OPSS.PROV 1010 (*Aggregates*) Granular 'A', placed in accordance with OPSS.PROV 501 (*Compacting*).

As discussed in Section 6.1, the underside of the east abutment for the existing Credit River bridge is reportedly at about Elevation 88.0 m. It is recommended that the base of the RSS wall and reinforced soil mass be founded coincident with this elevation, in order to minimize the additional load imposed from the RSS wall onto the existing east abutment footing. If the underside of the east abutment is at a higher or lower elevation, this should be communicated to the Contract Administrator so that adjustments to the design can be made if necessary. This requirement must be noted on the Contract Drawings. Based on the closest borehole information, and the available information regarding the elevation of the underside of the existing abutment (as described in Section 6.1), the top of the 500 mm thick Granular 'A' levelling pad/facing footing and bottom of the reinforced soil mass are recommended to be founded no higher than the maximum founding elevation noted in the table below. Depending on the final grade at the base of the RSS wall, the levelling pad may need to be installed below this recommended elevation to achieve the minimum embedment depth of 1.0 m between the underside of the levelling pad and the final finished grade.

Section of Retaining Wall and Corresponding Boreholes	Maximum (Highest) Founding Elevation (m)	Anticipated Founding Soil
RSS Wall Separating Existing and Proposed East Approach Embankments (Boreholes CRB-7 and NW6-1)	88.0[1]	Clayey Silt with Sand Till

Note(s):

1. Elevation of top of RSS wall footing / underside of RSS mass is to be coincident with the underside of the adjacent existing east abutment footing / wing wall footing.

6.7.2 Geotechnical Resistances

Assuming that the RSS wall acts as a unit and applies load across the full width of the reinforced soil mass, the factored ultimate and serviceability geotechnical resistances given below may be used for assessment of the reinforced mass founded on the properly prepared compacted granular fill, or on the proof-rolled fill subgrade at the sub-excavation elevations given above.

RSS Wall Height, H[1] (m)	Recommended Minimum Strip Length[2] (m)	Factored Ultimate Geotechnical Resistance	Factored Serviceability Geotechnical Resistance (for 25 mm of Settlement)
7.5 m	5.2	400 kPa	225 kPa

Note(s):

1. H represents the height of the RSS wall from top of leveling pad/facing footing to top of wall. It is understood the proposed wall will retain 3.3 m of soil and the bottom of the RSS wall will be placed at Elevation 88.0 m.
2. The recommended minimum strip length to satisfy global stability is discussed in Section 6.7.4

6.7.3 Resistance to Lateral Loads

The resistance to lateral forces / sliding resistance between the compacted granular backfill of the RSS wall and the subgrade should be calculated in accordance with Section 6.10.5 of the CHBDC (2014). The coefficient of friction, tan φ', between the compacted granular fill of the RSS wall and the properly prepared subgrade may be taken as 0.6. The actual values used should be reviewed and revised, if necessary, by the proprietary RSS wall designer during detail design.

6.7.4 Global Stability

Slope stability analyses have been performed for the proposed RSS wall using the commercially available program SLIDE, version 8.0 produced by Rocscience Inc., employing the GLE (General Limit Equilibrium) / 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 has been adopted for the design of the RSS wall under static, permanent conditions. This FoS is considered adequate for RSS walls at this site considering the design requirements and the field data available. In general, circular slip surfaces were analysed in the design.

The global stability analyses for the proposed RSS wall was carried out using the parameters outlined in Section 6.9.2 for the approach embankment stability assessment. A maximum retained soil height of 3.3 m was assumed for the RSS wall. The groundwater level was inferred from the highest water levels shown on the borehole records and was assumed to be at Elevation 89 m. The analysis was carried out using the wall dimensions discussed in Section 6.7.2. The results of the stability analysis indicate that the proposed RSS wall at this site, with the dimensions and reinforcing lengths listed in Section 6.7.2, will have a FoS greater than 1.54 (long-term) for global stability.

The internal stability of the RSS wall is to be designed and assessed by the proprietary product designer. This design and assessment should include a check of all failure surfaces that may fall within the RSS reinforced soil mass as it was assumed to have infinite strength in Golder’s global stability assessment. Extending the reinforcement zone beyond the minimum length identified in Section 6.7.2 may be required to ensure adequate internal stability.

6.8 Lateral Earth Pressure for Design of Abutments, Wing Walls, and RSS Wall

The lateral earth pressures acting on the abutment walls and any associated wing 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 surcharge including construction loadings, the freedom of lateral movement of the structure, and the drainage conditions behind the walls.

The following recommendations are made regarding the design of the abutment/wing walls:

- 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. Longitudinal drains or weep holes should be installed to provide positive drainage of the granular backfill. Compaction (including type of equipment, target densities, etc.) should be carried out in accordance with OPSS.PROV 501 (*Compacting*). Other aspects of the granular backfill requirements with respect to subdrains and frost taper should be in accordance with OPSD 3101.150 (*Walls, Abutment, Backfill, Minimum Granular Requirement*), OPSD 3121.150 (*Walls, Retaining, Backfill, Minimum Granular Requirement), and OPSD 3190.100 (Walls, Retaining and Abutment, Wall Drain*).
- 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 immediately behind the walls as per OPSS.PROV 501 (*Compacting*). Other surcharge loadings should be accounted for in the design, as required.

- For restrained walls, granular fill should be placed in a zone with the width equal to at least 1.2 m (equivalent to the depth of frost penetration at this site as interpreted from OPSD 3090.101, (*Frost Penetration Depths for Southern Ontario*)) behind the back of the wall on Figure C6.20(a) of the *Commentary to the* CHBDC (2014). For unrestrained walls, fill should be placed within the wedge-shaped zone defined by a line drawn flatter than1 horizontal to 1 vertical (1H:1V) extending up and back from the rear face of the footing or pile cap on Figure C6.20(b) of the *Commentary to the* CHBDC (2014).

6.8.1 Lateral Earth Pressures for Static Design

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

- For a restrained wall, the pressures are based on the proposed embankment fill and the following parameters (unfactored) may be used assuming the use of earth fill:

Material	Earth Fill
Soil Unit Weight:	20 kN/m^3
Coefficients of static lateral earth pressure: Active, K_a At rest, K_o Passive, K_P	 0.33 0.50 3.0

- For an unrestrained wall, the pressures are based on using engineered granular fill and the following parameters (unfactored) may be used:

Material	Granular 'A'	Granular 'B' Type II
Soil Unit Weight:	22 kN/m^3	21 kN/m^3
Coefficients of static lateral earth pressure: Active, K_a At rest, K_o Passive, K_P	 0.27 0.43 3.7	 0.27 0.43 3.7

- If the wall support and superstructure allow lateral yielding, active earth pressures may be used in the geotechnical design of the structure. The movement required to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure for design, should be calculated in accordance with Section C6.12.1 and Table C6.6 of the *Commentary to the CHBDC*, 2014.
- If the wall does not allow lateral yielding (i.e., restrained structure where the rotational or horizontal movement is not sufficient to mobilize an active earth pressure condition), at-rest earth pressures (plus any compaction surcharge) should be assumed for geotechnical design.

6.8.2 Seismic Lateral Earth Pressures for Design

Seismic (earthquake) loading must also be taken into account in the design of retaining / wing 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 abutment stem and/or 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**	0.041g	0.26	0.26	0.31
	975-Yr	0.076g	0.27	0.27	0.33
	2,475 Yr	0.146g	0.29	0.29	0.35
Non-Yielding Wall	**475-Yr**	0.041g	0.28	0.28	0.34
	975-Yr	0.076g	0.32	0.32	0.39
	2,475 Yr	0.146g	0.43	0.43	0.51

- 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).

6.9 Approach Embankment Design and Construction

Based on the General Arrangement (GA) drawing provided by MH on April 23, 2018, the proposed bridge will be constructed immediately to the north of the existing QEW-Credit River bridge with the new abutments located at the crest of the valley slopes, which is approximately 18 m above the base of the valley.

The elevation at both the east and west plateaus of the valley (i.e. at the crest of the valley slopes) is about Elevation 95 m. The proposed grade of the westbound lanes will slope downward across the length of the bridge from west to east from about Elevation 96.1 m to 94.8 m.

The east front slope of the valley is vegetated with tall grass, shrubs and trees and descends to meet the east bank of the Credit River at an average slope of about 3.3H:1V but locally steeper. The west front slope of the valley (within the footprint of the proposed bridge) and the northerly side slope has been engineered (by others) with a combination micropile supported concrete block RSS retaining wall, soil and/or rock anchored shotcrete slope protection, and vegetated slope. This combination of retaining wall and slope protection supports and protects the upper and lower access roads that provide access to the underside of the existing bridge and to the valley floor. The slopes below the upper access road descend at about 1H:1V to meet the flood plain of the Credit Valley.

At the east approach, the existing ground surface slopes downward from southeast to northwest from about Elevation 94.7 m to Elevation 91.7 m. The boreholes advanced in the area generally encountered a thin layer of topsoil underlain by a deposit of sand fill which is underlain by deposits of compact to very dense silt to sand and firm to hard clayey silt to clayey silt with sand till and residual soil/shale bedrock.

At the west approach the existing ground surface is relatively flat and ranges between Elevations 94.5 m and 95.6 m. The boreholes advanced in the area generally encountered a thin layer of topsoil underlain by deposits of sandy clayey silt fill and sand fill which is underlain by a deposit of stiff to hard silty clay or very stiff clayey silt till and shale bedrock.

The proposed grade at the highway center line is Elevation 94.8 m and 96.1 m, resulting in approach embankments that are up to 3.1 m and 1.5 m high at the east and west approaches, respectively.

6.9.1 Subgrade Preparation and Embankment Construction

Prior to construction of the new approach embankments it is recommended that any loosened/softened fill and topsoil/organic soils be removed from the footprint of the approach embankments.

It is recommended that the fill for construction of the new approach embankment widening(s) as well as the front and side slopes consist of Granular ‘A’ or Granular ‘B’ Type I or Type II meeting the specifications of OPSS.PROV 1010 (*Aggregates*). The embankment fill should be placed and compacted in accordance with OPSS.PROV 501 (*Compacting*) and OPSS.PROV 206 (*Grading*). For granular fill, the embankment front and side slopes should be constructed no steeper than 2 Horizontal to 1 Vertical (2H:1V) in granular fill.

Consideration may be given to constructing the fill supporting the West Multi-Use Trail/Access Road along the north side of the new approach with compacted earth fill meeting the requirements of OPSS.PROV 212 (*Earth Borrow)* placed and compacted in accordance with OPSS.PROV 501 (*Compacting*) and OPSS.PROV 206 (*Grading*). It is noted that the use of earth fill to support the West Multi-Use Trail may result in some differential settlement along the alignment of the trail/access road. For earth fill, the side slopes must be inclined at 3H:1V or flatter. The use of earth fill is not recommended for construction of the front slope of the embankment and any slopes extending into the river valley and floodplain.

Inspection and field density testing should be carried out by qualified personnel during fill placement operations to ensure that appropriate materials are used and that adequate levels of compaction have been achieved.

To reduce surface water erosion on the granular embankment side slopes, topsoil and seeding as per OPSS 802 (*Topsoil*) and OPSS.PROV 804 (*Seed and Cover*) should be carried out as soon as possible after construction of the embankments. If this slope protection is not in place before winter, then alternate protection measures, such as covering the slope with straw, or gravel sheeting as per OPSS 511 (*Rip Rap, Rock Protection and Granular Sheeting*), and OPSS.PROV 1004 (*Aggregates – Miscellaneous*) will be required to reduce the potential for erosion and to reduce the potential for the requirement of remedial works on the side slopes in the spring prior to topsoil dressing and seeding.

6.9.2 Stability

Limit equilibrium slope stability analyses were performed on the east and west approach embankment side slopes and front slopes extending into the river valley using the commercially available program Slide (Version 8.0) 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 embankment slopes under static conditions at for the long-term, permanent condition as per the CHBDC (2014). This FoS is considered appropriate for the embankments at this site considering the design requirements and the field data available. The stability analyses were performed to assess if the target minimum FoS was achieved for the design embankment height and geometries. In general, circular slip surfaces were analysed in the design.

For the non-cohesive soils present at the site, the effective stress parameters employed in the analysis were estimated from empirical correlations based on the results of the in-situ Standard Penetration Tests (SPT). The correlations proposed by Peck et al (1974) and U.S. Navy (1986) were employed and the results were adjusted by engineering judgment based on precedent experience in similar soil conditions.

For the cohesive deposits, total stress parameters were employed in the analyses of the short-term, undrained conditions (i.e., temporary conditions). The total stress parameters (i.e., average mobilized undrained shear strength – s_u) for the cohesive soils were estimated from correlations with the SPT results and other laboratory test data (i.e., natural water content), where appropriate. Effective stress parameters were also assigned to the cohesive deposits to evaluate the stability based on long-term, drained conditions (i.e., permanent conditions). The effective stress parameters (i.e., effective friction angle (ϕ’) for the cohesive deposits were estimated from empirical correlations based on the plasticity index. The correlations proposed by Mitchell (1993), Kulhawy and Mayne (1990), and Ladd et al. (1977) were employed and the results were adjusted using engineering judgment based on precedent experience in similar soil conditions.

For the bedrock, the effective stress parameters employed in the analysis were estimated based on precedent experience in the shale bedrock in the area of the site.

Summarized below are the simplified stratigraphy and the associated strengths and unit weights employed for the different soil and rock types in the approach embankment areas.

Soil Deposit	Bulk Unit Weight (kN/m^3)	Effective Friction Angle (°)	Effective Cohesion (kPa)	Undrained Shear Strength (kPa)
Granular A or Granular B Type I and II Fill	22	35°	0	--
Loose to compact silt and sand fill	20	30°	0	--
Compact sandy silt	20	32°	0	--
Firm to stiff sandy clayey silt	19	30°	0	35
Stiff to hard clayey silt till	20	32°	0	75
Hard residual soil	20	34°	0	--
Shale bedrock	23	50°	175	--

For this site, where filling is required, it is recommended that the approach embankments slopes (front and side) be constructed of Granular A or Granular B Type I or II with fill slopes no steeper than 2H:1V; with this geometry the resulting factored FoS against global instability for the short-term (undrained) and long-term (drained) cases is greater than 1.54. If the fill supporting the multi-use trail/access road along the north side of the new approach is constructed with compacted earth fill (meeting the requirements of OPSS.PROV 212 (*Earth Borrow)),* the side slopes must be inclined at 3H:1V or flatter.

It is understood that the front slope of the east approach (i.e. in front of the east abutment) will remain at the same inclination as the existing slope (approximately 3.3H:1V) and will include an unloading on the upper portion of the slope to accommodate the proposed West Multi-Use Trail. The limit equilibrium analysis (LEA) indicates that for this geometry the factored FoS is greater than 1.54 for both the temporary (short-term) and permanent (long-term) conditions.

As described in Section 6.9, the existing front slope of the west approach (i.e. the valley slope in front of the west abutment) and the northerly side slope has been engineered (by others) with a combination micropile supported concrete block RSS retaining wall, soil and/or rock anchored shotcrete slope protection, and vegetated slope.

It is our understanding that the slope protection and RSS retaining wall system located below the upper access road at the west abutment is to remain in place, while the slope protection above the upper access road (i.e. between the upper access road and the new abutment) is to be removed as part of the new abutment construction. Based on the drawings and cross-sections of the west approach area provided to Golder by MH on January 8, 2019, it appears that relatively large changes to the geometry of the existing front and side slopes will be required (including up to 3 m of cut, up to 7 m of fill, and the construction of a low, 2 m high, gabion retaining wall) in some areas in order to accommodate the alignment of the new West Multi-Use Trail in this area.

The sides slopes for the filling (where required) are proposed to be constructed at 2H:1V. Limit equilibrium analyses have been carried out to evaluate the effect of the proposed grade changes on the existing west abutment area front and side slopes. The subsurface stratigraphy (i.e. thickness and type of overburden and depth to bedrock) have been estimated based on the limited information available for this area. The dimensions, capacity and location(s) of the soil/rock anchors supported the existing slopes in this area have been based on the information shown on the design drawings prepared by TH O'Rourke Structural and RWH Engineering dated August 2011; the actual as-built locations and condition of these structural features are unknown. The results of the LEA stability analysis indicate a FoS≥1.54 for 2H:1V side slopes for the conditions and geometry described above assuming backfill is comprised of Granular B Type I or Type II material and assuming the gabion retaining wall (required over a short distance along the existing upper access road i.e. just west of the north end of the proposed west abutment) is no more than 2 m in height. We note that there is some risk associated with the long-term behaviour of the proposed fill slopes and gabion retaining wall given the lack of specific information on the existing conditions in this area. The internal stability of the gabion wall is the responsibility of the designer retained by the contractor. We note that the width of the base of the gabion wall should be equal to the height of the gabion wall and given that the gabion wall will be perched atop the shotcrete slope it may be required to be anchored into the bedrock.

As described above, the slope protection above the upper access road (i.e. between the upper access road and the new west abutment) which reportedly includes soil/rock anchors, as detailed on the RWH drawings, is to be removed as part of the new abutment construction. Consequently, construction equipment, should be capable of excavating through such obstructions. It is recommended that a Notice to Contractor be included in the Contract Documents to address obstructions (refer to Appendix D).

6.9.3 Settlement

Settlement of the subgrade under the east and west approach embankment areas can be expected as a result of the loading from the new fills on the existing fill and native soil deposits. Settlement of new granular fill that is properly placed and compacted for construction of the embankments will be small and occur during construction.

To estimate the magnitude of the expected settlements due to the construction of the new approach fills, analyses were carried out using hand and spreadsheet calculations. The immediate compression of the existing fill and native soil deposits was modelled by estimating an elastic modulus of deformation based on the SPT 'N'-values and using correlations proposed by Bowles (1984) and Kulhawy and Mayne (1990).

The simplified stratigraphy, together with the associated stiffness and unit weights employed for the different foundation soil types at the east and west approaches, as encountered in the nearest boreholes, are summarized below.

Borehole/Approach	**Soil Type**	**Approximate Thickness (m)**	**Bulk Unit Weight (kN/m^3)**	**Elastic Modulus (MPa)**
West Approach	Firm sandy clayey silt fill	0.7	19	8
	Loose to compact sand fill	1.9	19	10
	Very stiff clayey silt till	0.6	20	50
East Approach	Loose to compact silt and sand fill	1.7	20	7
	Firm to stiff sandy clayey silt	2.7	19	12
	Hard residual soil	0.4	20	80

6.9.3.1 Settlement Performance Requirements

The settlement performance criterion for design of high fill embankments is in accordance with MTO's Guideline "Embankment Settlement Criteria for Design" (2010).

Where new embankments approach structural elements, the following post-construction settlement and differential settlement criteria are considered acceptable for settlements to occur within 20 years post-paving for the bridge approach embankments at this site.

Location	Maximum Limits During Pavement Design Life	
	Distance from Transition Point (i.e., Abutment)	Total Post-Construction Settlement
Transition/Taper to Bridge Abutments	0 m to 20 m	25
	20 m to 50 m	50
	50 m to 75 m	75
	>75 m	>100

The above total settlements are considered to be applicable over a 20-year period following completion of construction (i.e., final paving). These performance criteria form part of the overall design performance for the embankment in the vicinity of the approach embankments.

6.9.3.2 Results of Analysis

Based on the analysis using the above parameters, the settlements of the up to 1.5 m and 3.1 m high approach embankments are expected to be less than 25 mm, meeting the requirements of the above noted embankment settlement criteria for design. In addition, the factored settlement of the existing approach embankments resulting from the construction of the new approach embankments is expected to be less than about 5 mm.

For fill slopes constructed out of OPSS.PROV (*Aggregates*) Granular 'A' or Granular 'B' Type I or II, the settlement of the fill itself is expected to be negligible and is anticipated to take place during construction. If earth fill is used to support the West Multi-Use Trail/Access Road on the north side of the approach, settlements of up to about 50 mm may occur). Given the variability of earth fill types allowed in OPSS.PROV 212 (*Earth Borrow*), this settlement may not be uniform and portions of the settlement may occur post-construction, which may affect the performance of the asphalt placed on the surface of the West Multi-Use Trail/Access Road.

6.10 Analytical Testing of Construction Materials

The results of an analytical test on four samples of the shale bedrock are presented in Section 4.2.17 and in Appendix C. The analytical test results were compared to CSA A23.1 Table 3 *("Additional requirements for concrete subjected to sulphate attack")* for potential sulphate attack on concrete. The sulphate concentrations measured in the tested samples are below the exposure class of S-3 (Moderate). Therefore, based on the four shale bedrock samples tested, when the designer is selecting the exposure class for the structure, the effects of sulphates may not need to be considered.

The analytical test results of the bedrock samples were also compared to Table 2 of the U.S. Criteria for Assessing Ground Corrosion Potential (as derived from Federal Highways Administration (FHWA) 2003) for the potential attack on buried steel. The sulphate and chloride concentrations measured in the bedrock samples indicate “Mild to no Corrosion Potential” and the resistivity in the samples indicates “Strong corrosion potential” to “Mild to no Corrosion Potential”. Based on the results of the samples tested, and given that the structure is located adjacent to the roadway and will be exposed to de-icing salt, consideration should be given by the designer to designing for a “C” type exposure class as defined by CSA A23.1 Table 1.

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

6.11 Construction Considerations

6.11.1 Overburden Excavation and Control of Groundwater and Surface Water

For spread/strip footings founded directly on competent bedrock at the abutments at the highest founding elevations provided in Section 6.5.1, the excavations will extend to a depths of about 8 m to 9 m below the surrounding grade, and potentially up to about 3 m to 4 m below the footing of the adjacent existing bridge at the east abutment. At the west abutment, the proposed elevation of the new footing is at about the same elevation as the existing adjacent footing.

For the drilled shaft/caisson foundations at the piers, excavation to the pile cap level will be up to about 8 m deep, including extending about 1 m into bedrock, at the east pier. At the west pier, the required excavation to the underside of pile cap will be about 3 m deep.

Excavations for shallow footings and for pile caps will extend through the existing fill materials and the compact to very dense silt to sand to sand and gravel, soft to stiff clayey silt and stiff to hard clayey silt to silty clay to clayey silt with sand till.

All open-cut (unsupported) excavations must be carried out in accordance with the guidelines outlined in the latest edition of Occupational Health and Safety Act (OHSA) and Regulation for Construction Activities (O.Reg. 213/91). The existing fill is classified as Type 3 soil, while the native deposits are generally classified as Type 3 soils, according to the OHSA. Temporary excavations (i.e. those that are open for a relatively short time period) should be made with side slopes no steeper than 1H:1V, in Type 3 soil. Excavated material must be stockpiled at a distance away from the excavation equal to or greater than the height of the open cut excavation.

Temporary protection systems will likely be required along the north side of the existing highway approach embankments and existing foundation units that are in close proximity to the proposed new abutment foundations. Recommendations for temporary protection systems are provided in Section 6.11.3.

The elevation of the groundwater table at the site is discussed in detail in Section 4.2.11, but is generally anticipated to be at about the bedrock surface on the plateau above the river valley and at about the natural ground surface in the river valley. Excavations for shallow foundations at the abutments will extend below the groundwater level; it is expected that groundwater inflow through the shale bedrock can be handled by pumping from well filtered sumps located outside the foundation footprint.

Excavations at the piers will extend below the groundwater and river water levels. Due to the proximity of the piers to the edge of the Credit River, a groundwater cut-off (cofferdam) will be required to minimize dewatering requirements as well as the occurrence of potential environmental impacts. A cut-off/cofferdam could consist of construction of a steel (i.e. sheet pile wall) box installed to or below the bedrock surface. There is potential for groundwater seepage in the upper weathered and fractured portions of the shale bedrock. The method and extent of ground water control required will ultimately depend on the type of temporary cofferdam(s) selected by the contractor. The contractor is responsible for the design and installation of the temporary cofferdams, of all groundwater control measures, as well as the requirements for maintaining the stability / integrity of the base of the excavation.

Unwatering, cofferdam construction and groundwater seepage control / management for the pier foundations and pile caps should be carried out in accordance with OPSS.PROV 517 (*Dewatering*) as amended by SP No. 517F01 and as modified by the Non-Standard Special Provision FOUN0003 (*Dewatering of Structure Excavation*) and an NSSP for Cofferdams. A copy of FOUN0003 has been provided in Appendix D, for inclusion in the Contract Documents.

Water takings in excess of 50 m^3/day are regulated by the Ministry of the Environment, Conservation and Parks (MECP). Certain takings of groundwater and stormwater for construction dewatering purposes with a combined total less than 400 m^3/day qualify for self-registration on the MECP's Environmental Activity and Sector Registry (EASR). Registry on the EASR replaces the need to obtain a PTTW for water taking less than 400 m^3/day and a Section 53 approval for discharge of water to the environment. A "Water Taking Plan" and a "Discharge Plan" are required by the MECP if water is taken in accordance with an EASR. In all cases, discharge under the EASR must be in accordance with a Discharge Plan. A Category 3 PTTW would be required for water takings in excess of 400 m^3/day.

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. Surface water should be directed away from the excavation area(s).

It is noted that in the vicinity of the east pier the ground surface is currently covered with rip-rap and other cobble and boulder sized rock. An Notice to Contractor should be included in the Contract Documents to warn the contractor about these conditions, as it may affect their selection of equipment and methodologies and may impact their schedule. An example Notice to Contractor is included in Appendix D.

6.11.2 Bedrock Excavation

Spread/strip footings at abutment locations will be founded at or below Elevations 85.4 m and 87.4 m at the east and west abutments, respectively. Up to about 1.5 m and 6 m of bedrock excavation will be required at the east and west abutments, respectively.

The shale bedrock contains horizontal bedding and will tend to break in slabs if removed with an excavator. Therefore, where bedrock excavation is required adjacent to a footing that is supporting the existing bridge it must be carried out using saw-cutting or line drilling techniques to avoid the unintentional removal of bedrock from below the existing footing. An NSSP to address this item is included in Appendix D and should be included in the Contract Documents. If the excavation in the bedrock is required to extend below the recommended elevations provided in Section 6.5.1 to remove all loose, shattered and/or fractured rock within the area of the footing, the over-excavated portions must be replaced with mass concrete having a minimum thickness of 100 mm and a minimum of 28-day compressive strength of 20 MPa, shall be placed in the excavation within four hours of exposure of the founding level to protect the integrity of the subgrade. An NSSP to address this item is included in Appendix D, which should be included in the Contract Documents.

The shale bedrock at the site is weak (corresponding to unconfined compressive strengths generally in the range of 10 MPa to 20 MPa), but contains strong to very strong limestone/dolostone/siltstone interbeds and possible boulders or interbeds within the residual soil, above the bedrock. It is recommended that an NSSP be included in the Contract Documents to warn the Contractor of the bedrock characteristics, that excavation into the bedrock will require appropriate equipment and construction procedures, and that the bedrock excavation shall not disturb the existing bridge footings. An NSSP is provided in Appendix D for inclusion in the Contract Documents.

6.11.3 Temporary Protection Systems

Temporary protection systems will be required along the north side of the existing QEW lanes and existing bridge foundations to facilitate safe construction of the new abutment foundations and maintain operation of the existing adjacent highway and bridge.

The temporary protection systems should be designed and constructed in accordance with OPSS.PROV 539 (*Temporary Protection Systems) and Special Provision SP105S09.* The design of the temporary shoring systems for lateral movement should meet the following requirements, as specified in OPSS.PROV 539, as amended by SP105S09:

- Performance Level 2 - where the temporary shoring system is supporting the existing highway embankment; and,
- Performance Level 1b - where the temporary shoring system is supporting the existing bridge foundations.

The above recommended performance levels should be checked to confirm that the existing adjacent structure and any utilities can tolerate the associated magnitude of deformation. If the excavated material is to be stockpiled adjacent to the temporary protection systems, the systems must be designed for this additional loading.

To construct the proposed shallow footings on sound bedrock at the abutments at the elevations given in Section 6.5.1, the protection systems are required for an estimated excavation depth of up to approximately 9 m relative to the adjacent QEW highway grade. Due to the presence of relatively shallow bedrock and the performance requirements of the temporary protection system(s), temporary shoring could be comprised of either soldier pile and lagging, sheet pile wall, or possibly a contiguous caisson wall depending on the founding condition and elevation of the existing footings (in particular at the east abutment) and the separation distance between the edge of existing footing and the face of the temporary excavation. It is possible that elements of the protection system will need to be cored into or otherwise fixed within the bedrock to obtain sufficient lateral resistance and toe fixity for the system to ensure the integrity of the existing structure and fill behind the abutments.

At the east abutment, where the design elevation of the underside of the new footing is anticipated to be below the level of the adjacent existing footing, if the temporary protection system is designed to terminate at the top of the bedrock surface, there may an additional requirement to pre-support the bedrock face prior to excavation to minimize the chance of undermining the existing footing. Pre-support could be in the form of vertical dowels grouted into the bedrock at an offset between the toe of the temporary protection and the face of the rock excavation.

At the pier locations, given the soil conditions and shallow depth to the groundwater table as well as the proximity to the Credit River, an interlocking sheet pile wall system will be required for the temporary excavation support to allow for construction of the pile caps in-the-dry. In addition, given the chance for water seepage through fractures in the shale bedrock and/or near the interface of the sheet pile wall/bedrock surface, it is anticipated that seepage control (in the form of properly filtered sumps) and/or installation of a concrete plug at the base of the cofferdam may be required to control water and allow construction of the pile caps in-the-dry.

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

6.11.4 Bedrock Subgrade Inspection and Protection

Immediately following completion of excavation of the bedrock to the founding level(s), the footing subgrade should be inspected by a Foundation Engineer in accordance with OPSS 902 *(Excavating and Backfilling Structures)* as amended by SP109S12, to check that all existing fill materials, concrete and fractured, softened or loosened portions of the shale bedrock are removed prior to construction of the footings for the abutments.

The shale bedrock that will be exposed at the foundation subgrade level will be susceptible to weathering (wetting/drying) and/or disturbance from water and construction traffic. To limit this degradation, it is recommended that a minimum 100 mm thick concrete working slab having a minimum thickness of 100 mm and a minimum of 28-day compressive strength of 20 MPa be placed on the subgrade within four hours after excavation, 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 provided in Appendix D for inclusion in the Contract Documents.

6.11.5 Vibration Monitoring During Construction

A maximum peak particle velocity (PPV) of 100 mm/s is generally considered applicable for bridge structures in good condition. Based on vibration monitoring experience, it is considered unlikely that vibrations induced by conventional construction activities such as line drilling and/or hoe-ramming or caisson installation in bedrock will reach this threshold level and, therefore, vibration monitoring for the existing bridge is not expected to be required during construction at this site.

Residential homes are located within about 100 m of the proposed abutment locations. A lower PPV threshold of 25 mm/s is generally considered applicable for vibration impacts on buildings, and the zone of influence could extend to about 250 m. Therefore, vibration monitoring should be carried out at the existing structures near the bridge site during shoring/cofferdam installation, bedrock excavation and drilled shaft construction operations. An NSSP describing the requirements for vibration monitoring is presented in Appendix D.

6.11.6 Scour Protection and Erosion Control

The proposed pier foundations are located in close proximity to the Credit River and below the normal river water level and as such, the pier foundations could experience some erosion/scour throughout the design life for the structure if the backfilled soils are eroded.

Scour protection should be provided around the pier foundations for the bridge structure. Riprap should be provided on the river channel banks and around the piers. The riprap should extend from the channel banks to 1 m above the design flood level at the structure site.

6.11.7 Obstructions

The residual soil and glacially derived soils at the site above the bedrock surface contain rock fragments, particularly immediately above the bedrock, as noted on the borehole records, which could affect the installation of temporary shoring and deep (drilled shaft) foundations. In addition, it is noted that in the vicinity of the east pier the ground surface is currently cover with rip-rap and other cobble and boulder sized rock. Further, the existing slope protection above the upper access road at the west abutment contains soil/rock anchors that are to be removed as part of the new abutment construction.

A Notice to Contractor should be included in the Contract Documents to identify to the contractor the possible presence of rock fragments, or slabs of cobbles / boulders, rip-rap sizes at ground surface and within the overburden soils or immediately above the bedrock, as well as the presence of soil/rock anchors in the vicinity of the west abutment; an example Notice to Contractor is provided in Appendix D.

6.11.8 Falsework

It is understood that the contractor may be required to erect falsework in order to support the bridge spans during construction. The contractor is responsible for the design and installation of all temporary works including the falsework and any foundations required to support the falsework. Section 3.3 of the FIDR refers to the available factual subsurface information that may be of use to the contractor for their design of these temporary systems.

The subsurface conditions across the site are variable. In places where the overburden is competent or where the bedrock is located at shallow depth, it should be possible to support falsework on shallow foundations underlain by granular pads. However, in areas where the overburden is comprised of organics and/or soft clays, some form of ground improvement (i.e. subexcavation of poor soils and replacement with granular materials or construction of aggregate piers) may be required to provide sufficient geotechnical resistance for shallow foundations. Where support of falsework is required to be located on slopes, it should be possible to design shallow foundations capable of providing a factored geotechnical resistance on the order of about 100 kPa so long as near surface organics or fills are removed and the footings are constructed on the stiff to hard clayey silt (till) over bedrock.

Based on our discussions with MH, they have indicated that the displacement / distortion limits for falsework are as specified in OPSS 919 Section 919.04.01.07. In addition, Sections 919.04.02.01.01 and 919.04.02.01.02 provide the requirements of the design engineer / design checking engineer who is to be retained by the contractor.

7.0 CLOSURE

This report was prepared by Mr. Matthew Kelly, P.Eng., a Geotechnical Engineer with Golder. Ms. Sandra McGaghran, M.Eng., P.Eng. a senior geotechnical engineer and Associate conducted a technical review of the report and Mr. Paul Dittrich, Ph.D., P.Eng., a MTO Foundations Designated Contact and Principal of Golder conducted an independent quality control review of this report.

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ASTM International:

ASTM D1586	Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils

ASTM D7012	Standard Test Method for Compressive Strength and Elastic moduli of Intact Rock Core Specimens under Varying States of Stress and Temperature

Commercial Software:

Slide (Version 8) by Rocscience Inc.

Ontario Provisional Standard Drawing:

OPSD 202.010	Slope Flattening using Surplus Excavated Material on Earth or Rock Embankment
OPSD 208.010	Benching of Earth Slopes
OPSD 3000.100	Foundation, Piles, Steel H-Pile Driving Shoe
OPSD 3000.100	Foundation, Piles, Tube Pile Driving Shoe
OPSD 3090.101	Foundation Frost Penetration Depths for Southern Ontario
OPSD 3101.150	Walls, Abutment, Backfill Minimum Granular Requirement
OPSD 3121.150	Walls, Retaining, Backfill Minimum Granular Requirement
OPSD 3190.100	Walls, Retaining and Abutment, Wall Drain

Ontario Provincial Standard Specification:

OPSS.PROV 206	Construction Specification for Grading
OPSS.PROV 212	Construction Specification for Earth Borrow
OPSS.PROV 501	Construction Specification for Compacting
OPSS 511	Construction Specification for Rip-Rap, Rock Protection, and Granular Sheeting
OPSS.PROV 539	Construction Specification for Temporary Protection Systems
OPSS.PROV 802	Construction Specification for Topsoil
OPSS.PROV 804	Construction Specification for Seed and Cover
OPSS.PROV 902	Construction Specification for Excavating and Backfilling Structures
OPSS.PROV 903	Construction Specification for Deep Foundations
OPSS.PROV 904	Construction Specifications for Concrete Structures
OPSS 919	Formwork and Falsework
OPSS.PROV 1004	Material Specification for Aggregates - Miscellaneous
OPSS.PROV 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade, and Backfill Material

Ontario Water Resources Act:

Ontario Regulation 903	Wells (as amended)

Ontario Occupational Health and Safety Act:

Ontario Regulation 213/91	Construction Projects (as amended)

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Special Provision:

Special Provision No. 105S09 Amendment to OPSS 539

Special Provision No. 109S12 Amendment to OPSS 902

Special Provision No. 109F57 Amendment to OPSS 903

TABLE 1 - COMPARISON OF FOUNDATION ALTERNATIVES, CREDIT RIVER BRIDGE, SITE No. 24-203, GWP 2002-13-00

Foundation Option	Feasibility	Advantages	Disadvantages	Constructability/Risk	Relative Costs
Spread/strip footings founded on native soil deposits	• Not feasible for support of piers or abutments due to low strength of the overburden soils and impractical considering the relatively shallow depth to bedrock.				

Spread/strip footings founded on competent shale bedrock	• Feasible for support of the piers; however, requires cofferdams to complete excavation and footing construction in the dry adjacent to/within the river.
	• Feasible for support of the abutments; however, requires temporary roadway protection and may require temporary protection of existing bridge footings at the east abutment in particular, where the new footings are not constructed at the same elevation as the existing adjacent footings.
		• Higher geotechnical resistance than for shallow foundations bearing on native soil deposits.
		• At existing bridge, the west abutment and piers are supported on shallow foundations on bedrock; east abutment founding condition is unclear (may be founded on till or residual soil/upper weathered bedrock).
		• Straight forward method of construction; however, extensive excavation, including into bedrock, required.
		• At abutments, only minor groundwater seepage anticipated, so pumping from filtered sumps expected to provide adequate groundwater control.
			• Significant excavation depths through existing fills and native soils and into bedrock required:
			  • Up to 8.2 m deep at west abutment.
			  • Up to 7.5 m deep at west pier.
			  • Up to 9.8 m deep at east pier.
			  • Up to 9.3 m deep at east abutment.
			• Above depths are greater than what is required for pile cap construction for caisson (drilled shaft) foundation option.
			• The differential settlement between foundation units is estimated to be greater than 7 mm if both the abutment and piers are founded on spread footings, which exceeds the structural requirement.
			• At the abutments, temporary roadway protection systems required along north edge of existing bridge approach embankments
			• At east abutment, since existing footing is estimated to be founded about 2.5 m to 4 m above the recommended founding elevation of the new footing, installation of dowelling to pre-support the rock face and support the existing footing during excavation may be required if minimum separation distance cannot be achieved.
			• Groundwater control (cofferdams) required at pier locations due to proximity to Credit River; may be difficult to “seal” cofferdam at the bedrock surface or prevent upward seepage inflows; mitigation measures such as concrete plug placed by tremie methods may be required.
			• Precludes use of integral abutments, although may permit semi-integral abutments; potentially greater maintenance required at abutments.
				• Conventional excavation and construction techniques.
				• Risk of groundwater / river water inflow through gaps at the cofferdam-bedrock interface or through bedrock fractures at the piers.
					• Estimated cost is approximately $\$600/\text{m}^3$ for construction of shallow foundations, excluding deeper excavation and protection systems and dowelling at the abutments, and cofferdams at the piers.

Foundation Option	Feasibility	Advantages	Disadvantages	Constructability/Risk	Relative Costs
			• Greater volume of excavation spoil; and concrete than for deep foundations. • More groundwater control required associated with deeper excavations than for deep foundations option.		
Steel H-piles founded on shale bedrock; Steel tube (pipe) piles on shale bedrock	• Not feasible for support of the piers or abutments due to relatively shallow depth to bedrock and low strength of the overburden soils. • Would require socketing steel piles into concrete filled rock sockets in bedrock.				
Caissons (drilled shafts) socketed into shale bedrock	• Feasible for support of abutments and piers.	• Higher bearing resistances than for steel H-piles, requiring fewer pile elements. • Will result in less depth of excavation (including less depth of bedrock excavation) than for spread footing option. • At abutments, may reduce temporary protection system requirements, in particular, the need for dowelling to pre-support excavation face. Minor groundwater seepage anticipated within pile cap excavation, so pumping from filtered sumps will provide adequate groundwater control. • At piers, may result smaller footprint/working area than for spread footing option and so overall volume of excavation may be less; may also reduce cofferdam and groundwater control requirements. • Pile caps could potentially be eliminated if the pier columns extended directly up from the top of the drilled shafts.	• Significant depth of excavation (including about 1 m of bedrock excavation) required at east pier due to constraint to have top of pier cap embedded a minimum of 0.6 m below riverbed. Although total excavation depth will be less than that required for shallow foundation option, a deep cofferdam will still be required. • Permanent liners would be required during construction to control potential ground loss in overburden soils and to mitigate groundwater (and river water) inflows. • Shale bedrock contains strong to very strong limestone layers, so more expensive coring/churn drilling required to form bedrock socket through these layers. • Precludes use of integral abutments. • The rock socket is required to be cleaned (potentially by airlift methods) and inspection with a video camera would be required. • Concrete would have to be placed by tremie methods, below the water level at piers.	• Conventional construction methods for caisson foundations; temporary or permanent liners required for ground and groundwater control. • Access for large caisson drill rig may be difficult at east pier given steepness of slope, weak soil conditions and restriction of use of barge/floating platform in the river.	• Estimated cost is approximately \$1000/m length for caisson installation and \$600/m^3 for pile cap construction; this cost expected to be higher to account for pre-drilling/coring through harder limestone layers and for temporary or permanent liners.

CONT No. 2019–2016
GWP No. 2002–13–00

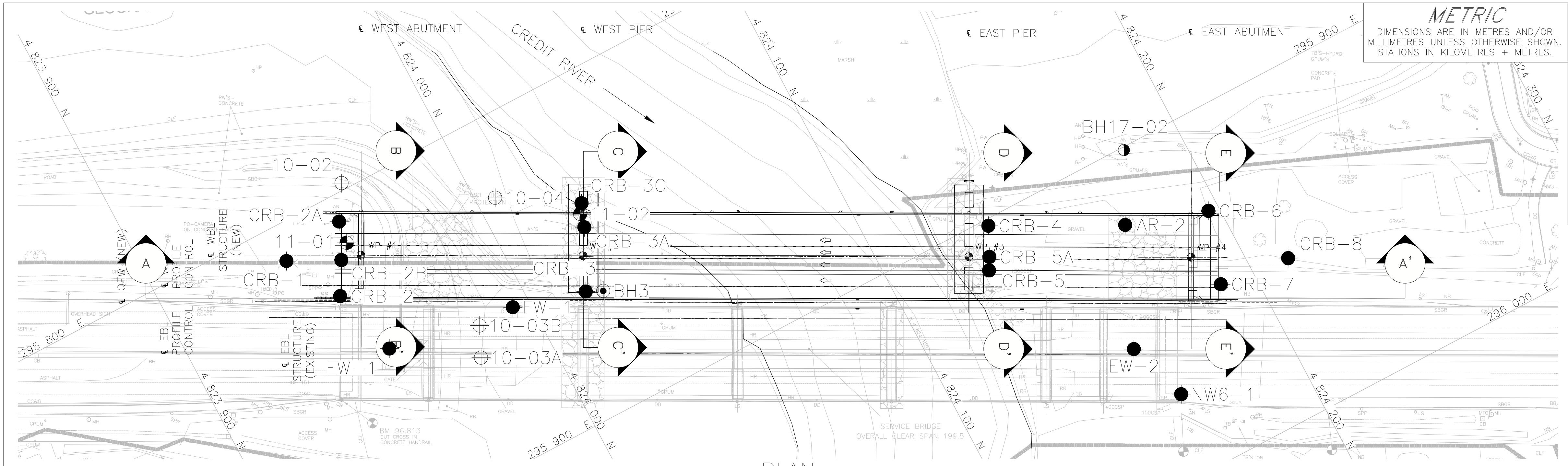
SHEET

QEW WIDENING – MISSISSAUGA RD TO HURONTARIO ST
QEW – CREDIT RIVER BRIDGE

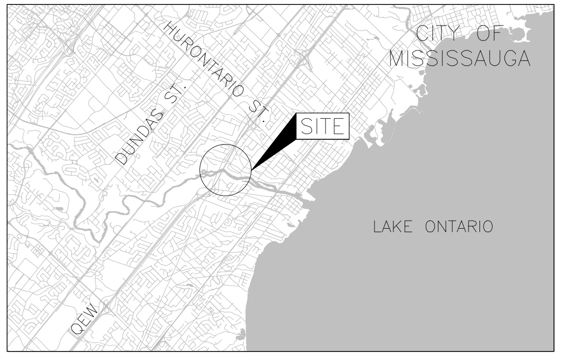
BOREHOLE LOCATIONS AND SOIL STRATA

GOLDER

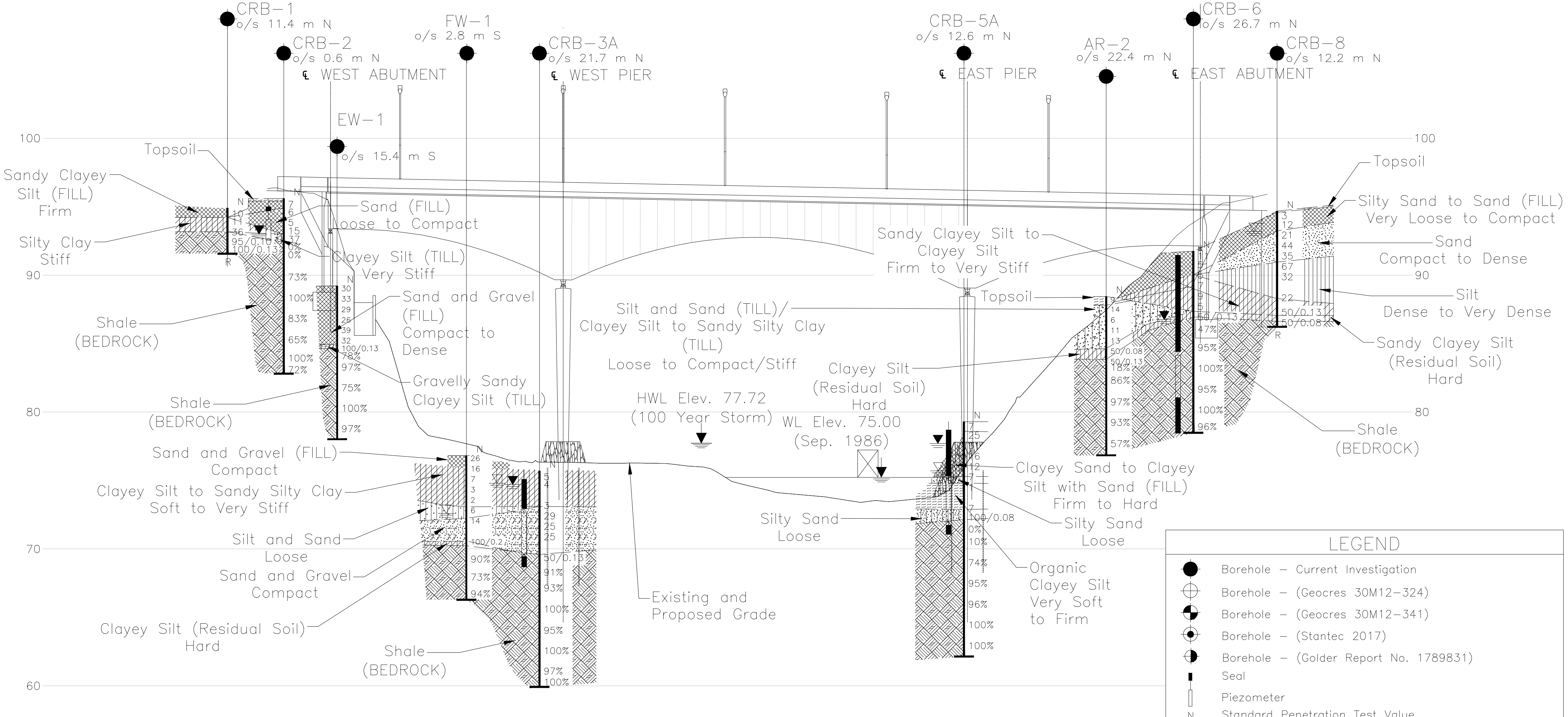
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.



PLAN
SCALE
15 0 15 30 m



KEY PLAN
SCALE
2 0 2 4 km



PROFILE A–A'

HORIZONTAL SCALE
15 0 15 30 m

VERTICAL SCALE
3.75 0 3.75 7.5 m

LEGEND

●	Borehole – Current Investigation
⊕	Borehole – (Geocres 30M12–324)
◉	Borehole – (Geocres 30M12–341)
⊙	Borehole – (Stantec 2017)
◐	Borehole – (Golder Report No. 1789831)
▮	Seal
▯	Piezometer
N	Standard Penetration Test Value
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
R	Split–Spoon Refusal
100%	Rock Quality Designation (RQD)
▼	WL in piezometer, measured on November 6, 2018
▽	WL upon completion of drilling

BOREHOLE CO–ORDINATES (MTM NAD 83 ZONE 10)			
No.	ELEVATION	NORTHING	EASTING
10–02	94.4	4823966.3	295798.0
10–03A	76.2	4823979.0	295865.1
10–03B	76.3	4823983.2	295856.2
10–04	76.3	4824005.7	295823.9
11–01	94.6	4823959.1	295814.8
11–02	75.7	4824026.4	295840.4
AR–2	88.4	4824172.2	295921.4
BH17–02	89.2	4824182.6	295900.9
CRB–1	94.9	4823940.2	295811.1
CRB–2	95.6	4823949.7	295828.3
CRB–2A	94.5	4823960.1	295808.0
CRB–2B	94.7	4823955.2	295818.7
CRB–3	75.9	4824016.8	295862.2
CRB–3A	75.7	4824025.6	295844.6
CRB–3C	75.3	4824028.3	295837.7
CRB–4	79.1	4824135.1	295902.0
CRB–5	79.2	4824128.9	295914.2
CRB–5A	79.3	4824130.9	295910.6
CRB–6	91.7	4824196.7	295929.5
CRB–7	94.7	4824189.6	295951.1
CRB–8	94.7	4824211.5	295953.7
EW–1	87.7	4823955.5	295849.5
EW–2	89.1	4824156.8	295956.2
FW–1	76.1	4823994.8	295856.0
NW6–1	95.3	4824163.1	295975.2

NOTES

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

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

REFERENCE

Base plans provided in digital format by Morrison Hershfield, drawing file no. X11609340Base.dwg, received April 12, 2018.
General Arrangement plan and profile provided in digital format by Morrison Hershfield, drawing file no. 01.GENERAL ARRANGEMENT (for Golder).dwg, received April, 13, 2018 and 01.GENERAL ARRANGEMENT.dwg, received January 29, 2019.

J.P. DITTRICH
Feb. 05, 2019

S. McGaghran
Feb. 05, 2019

Geocres No. 30M12–426

HWY. QEW		PROJECT NO. 1662333	DIST. CENTRAL
SUBM'D. JIL	CHKD. DM	DATE: 2/5/2019	SITE: 24–203
DRAWN: DD/TB	CHKD. SMM	APPD. JPD	DWG. 1

PLOT DATE: February 11, 2019
FILENAME: S:\Clients\MTO\QEW–Credit_River\99_PROJ\1662333_MH_P&F\40_PROD\0001_Credit_River_Bridge_Twin\1662333–0001–BG–0001.dwg

MINISTRY OF TRANSPORTATION, ONTARIO

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

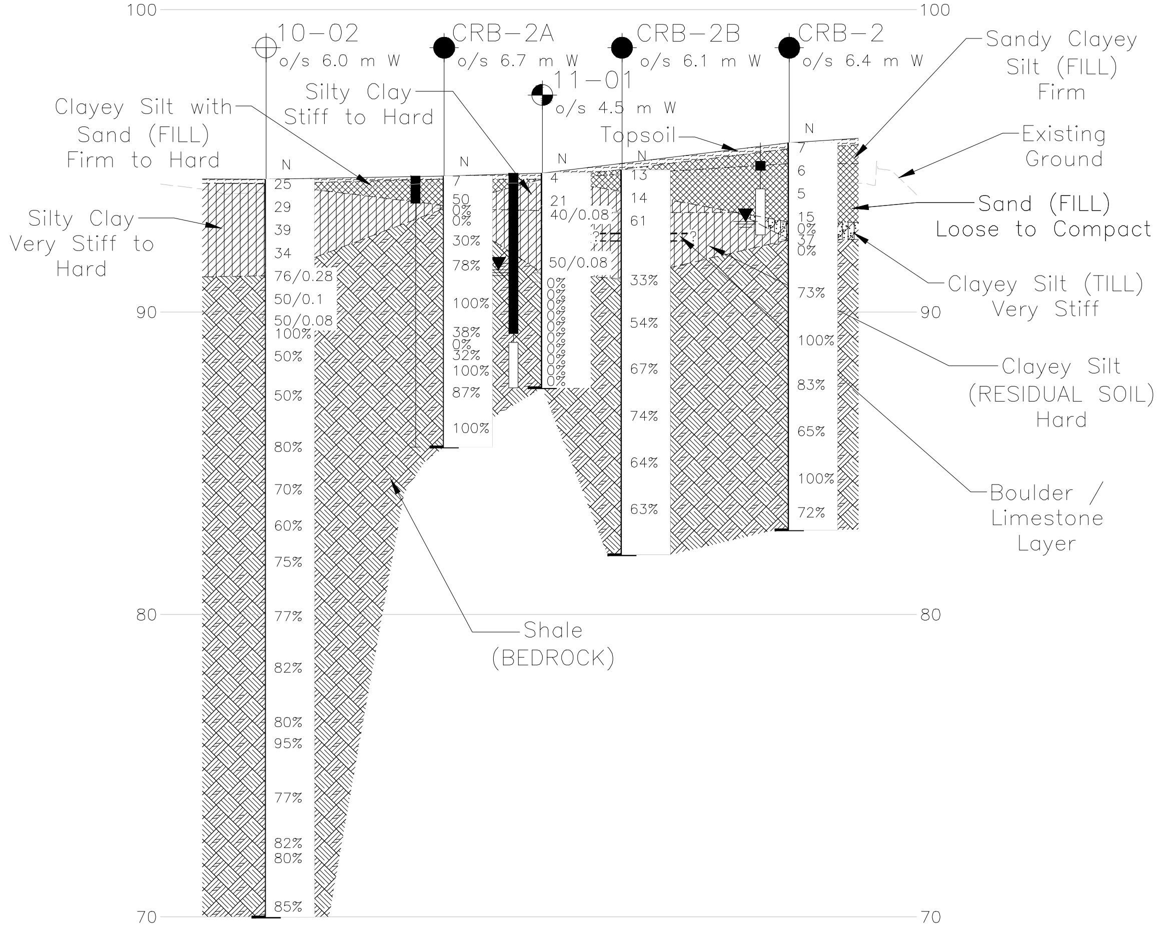
CONT No. 2019–2016
GWP No. 2002–13–00

SHEET

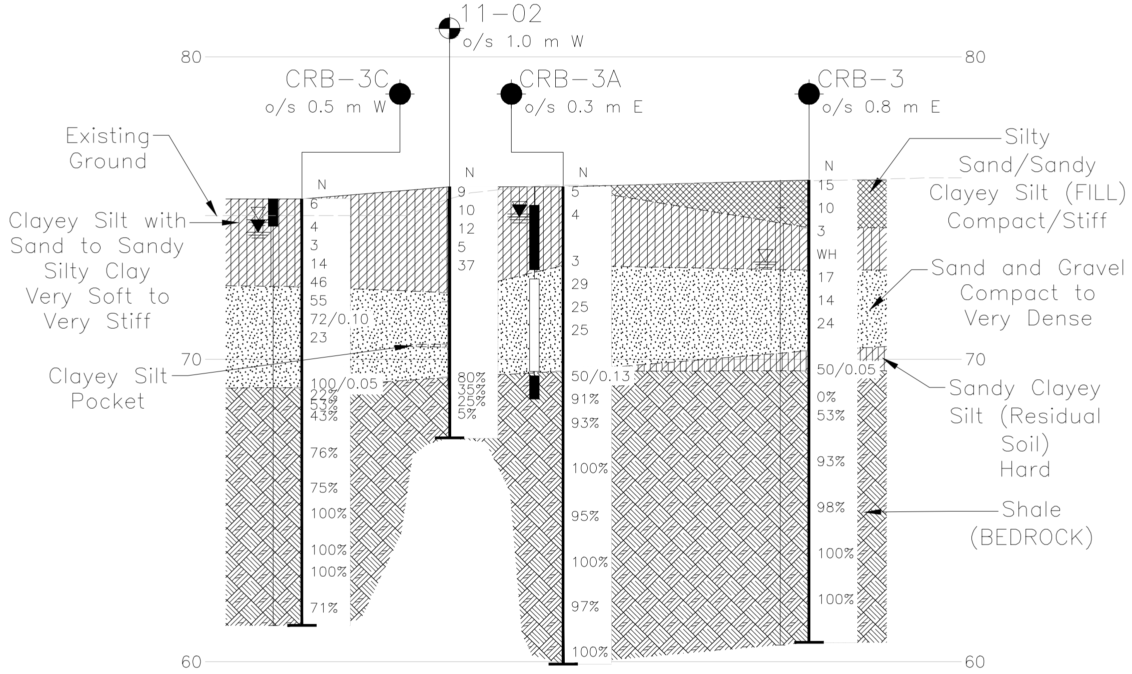
QEW WIDENING – MISSISSAUGA RD TO HURONTARIO ST
QEW – CREDIT RIVER BRIDGE

SOIL STRATA

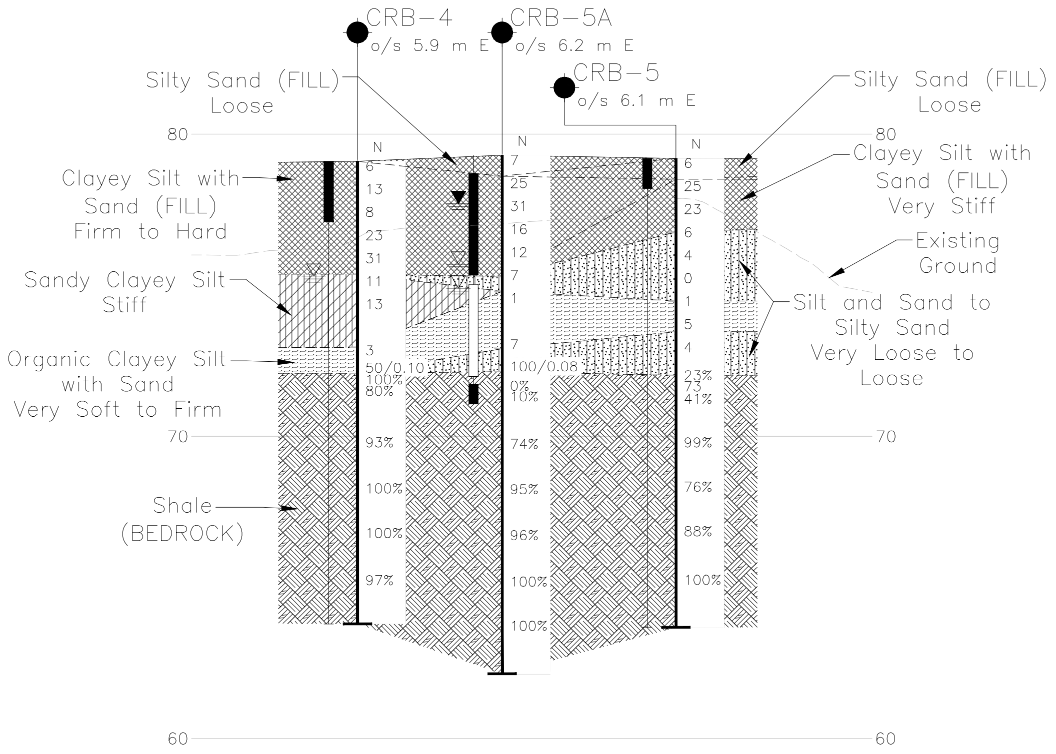
GOLDER



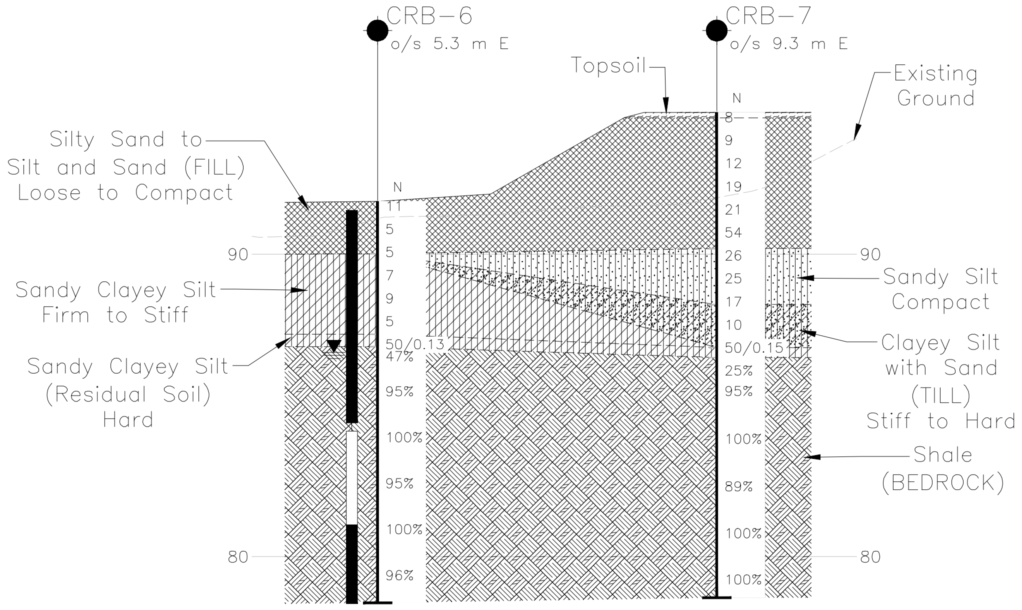
SECTION B–B' WEST ABUTMENT



SECTION C–C' WEST PIER



SECTION D–D' EAST PIER



SECTION E–E' EAST ABUTMENT

SCALE HORIZONTAL
5 0 5 10 m

VERTICAL SCALE
2.5 0 2.5 5 m

LEGEND

●	Borehole – Current Investigation
◐	Borehole – (Geocres 30M12–341)
⊕	Borehole – (Geocres 30M12–324)
▮	Seal
▯	Piezometer
N	Standard Penetration Test Value
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
100%	Rock Quality Designation (RQD)
▼	WL in piezometer, measured on November 6, 2018
▽	WL upon completion of drilling

BOREHOLE CO-ORDINATES (MTM NAD 83 ZONE 10)

No.	ELEVATION	NORTHING	EASTING
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11–02	75.7	4824026.4	295840.4
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CRB–3	75.9	4824016.8	295862.2
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CRB–5	79.2	4824128.9	295914.2
CRB–5A	79.3	4824130.9	295910.6
CRB–6	91.7	4824196.7	295929.5
CRB–7	94.7	4824189.6	295951.1

NOTES

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J.P. DITTRICH
Jan. 04, 2019

S. McGaghran
Jan. 04, 2019

NO.	DATE	BY	REVISION

Geocres No. 30M12–426

HWY. QEW		PROJECT NO. 1662333	DIST. CENTRAL
SUBM'D. JIL	CHKD. DM	DATE: 1/4/2019	SITE: 24–203
DRAWN: DD/TB	CHKD. SMM	APPD. JPD	DWG. 2

PLOT DATE: February 6, 2019
FILENAME: S:\Clients\MTO\QEW-Credit_River\99_PROJ\1662333_MH_P&F\40_PROD\0001_Credit_River_Bridge_Twin\1662333-0001-BC-0002.dwg

APPENDIX A

Previous By Others and Relevant Borehole and Drillhole Records, Bedrock Core Photographs Geotechnical Laboratory Test Results and Analytical Test Results

Draft Geotechnical Investigation Report, Proposed Electrical Transmission Line Monopoles at Credit River and QEW” dated September 5, 2018, Report No. 1789831, prepared by Golder Associates Ltd.

Borehole 17-2

PROJECT: 1789831

RECORD OF BOREHOLE: BH17-2

SHEET 1 OF 2

LOCATION: SEE FIGURE 1

BORING DATE: June 25 and 26, 2018

DATUM: Geodetic

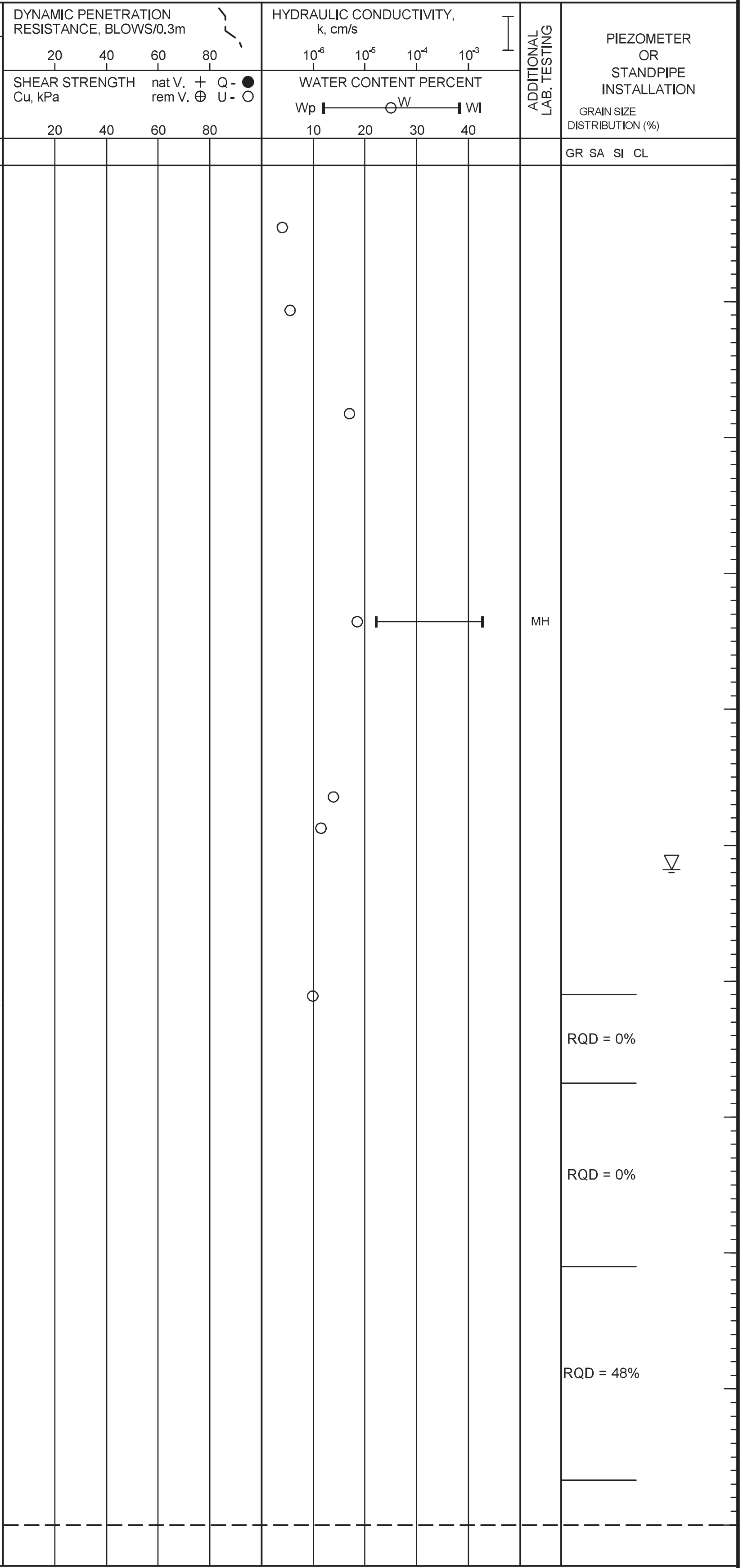
SPT/DCPT HAMMER: MASS, 64kg; DROP, 760mm

DRILL RIG: MARL-MT5

HAMMER TYPE: AUTOMATIC

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)
		GROUND SURFACE		89.19
0	Power Auger 153 mm O.D. Solid Stem Auger	FILL - (SM) CLAYEY SILTY SAND, some gravel, trace organics	▩	0.00
		FILL - (SW/GW) SAND and GRAVEL; dark brown to grey; moist, compact	▩	88.89 0.30
		FILL - (SM) SILTY SAND; brown; moist, non-cohesive, compact	▩	87.67 1.52
		(CI) SILTY CLAY, trace sand; varved, brown; w<PL, firm to very stiff	▨	87.06 2.13
		SHALE (BEDROCK); grey		84.34 4.85
	HQ Core -	- Bedrock cored from a depth of 6.1 m to 17.1 m. - For bedrock coring details, refer to record of Drillhole BH17-2.		
10		*CONTINUED NEXT PAGE*		

SAMPLES NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
1A	SS	16		
1B				
2	SS	25		
3	SS	14		
4	SS	8		
5	SS	19	MH	
6A	SS	50/ 0.08		
6B				
7	SS	50/ 0.02		
1	RC			RQD = 0%
2	RC			RQD = 0%
3	RC			RQD = 48%
4	RC			



DEPTH SCALE
1 : 50

GOLDER

LOGGED: SM/BC
CHECKED: AK

PROJECT: 1789831

RECORD OF BOREHOLE: BH17-2

SHEET 2 OF 2

LOCATION: SEE FIGURE 1

BORING DATE: June 25 and 26, 2018

DATUM: Geodetic

SPT/DCPT HAMMER: MASS, 64kg; DROP, 760mm

DRILL RIG: MARL-MT5

HAMMER TYPE: AUTOMATIC

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE			SAMPLES			DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m 20 40 60 80 SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, cm/s 10^{-6} 10^{-5} 10^{-4} 10^{-3} WATER CONTENT PERCENT Wp ⊢ ○W ⊣ Wl 10 20 30 40	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	NUMBER	TYPE	BLOWS/0.3m				GRAIN SIZE DISTRIBUTION (%)
		--- CONTINUED FROM PREVIOUS PAGE ---									GR SA SI CL
10	HQ Core	SHALE (BEDROCK); grey			4	RC					RQD = 64%
11											UC = 21.4 MPa
12					5	RC					RQD = 55%
13					6	RC					RQD = 72%
14											
15					7	RC					RQD = 78%
16					8	RC					RQD = 99%
17				72.09 17.10							
		END OF HOLE NOTES: 1. Water level was measured to be at a depth of 5.2 m below ground surface in open borehole prior to rock coring.									
18											
19											
20											

DEPTH SCALE

1 : 50

GOLDER

LOGGED: SM/BC

CHECKED: AK

PROJECT: 1789831

RECORD OF DRILLHOLE: BH17-2

SHEET 1 OF 2

LOCATION: SEE FIGURE 1

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: June 25 and 26, 2018

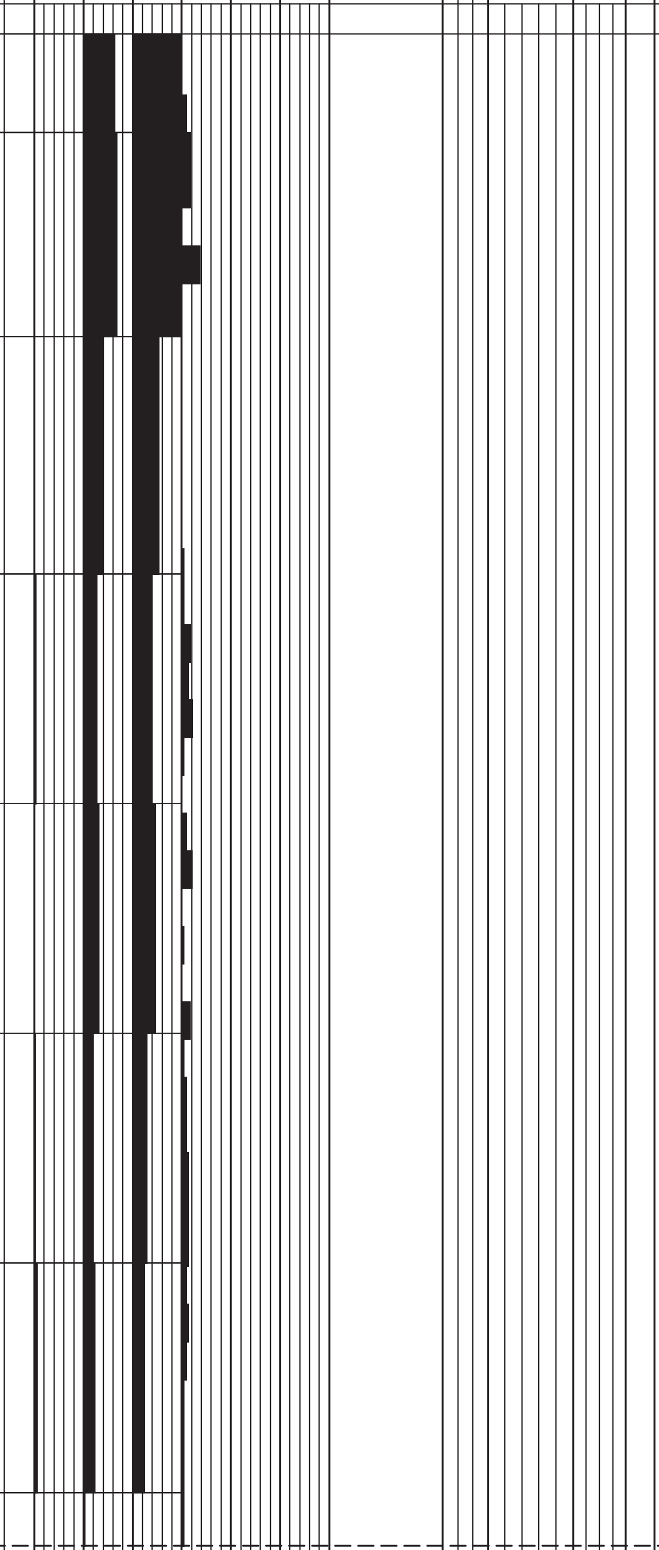
DRILL RIG: MARL-MT5

DRILLING CONTRACTOR: Drilltech

DATUM: Geodetic

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	NOTES
		Continued from Record of Borehole 17-2		83.09		
7 – 8	HQ CORE	Highly weathered, thinly bedded, grey, fine grained, very weak, SHALE (Georgian Bay Formation) with LIMESTONE interbeds		6.10	1 2	
8 – 15	HQ CORE	Moderately weathered, thinly bedded, grey, fine grained, weak SHALE (Georgian Bay Formation) with LIMESTONE interbeds		81.09 8.10	3 4 5 6 7	UC = 21.4 MPa
16	HQ CORE			73.44 15.75	8	
		CONTINUED NEXT PAGE				



DEPTH SCALE

1 : 50

GOLDER

LOGGED: SM/BC

CHECKED: AK

PROJECT: 1789831

RECORD OF DRILLHOLE: BH17-2

SHEET 2 OF 2

LOCATION: SEE FIGURE 1

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: June 25 and 26, 2018

DRILL RIG: MARL-MT5

DRILLING CONTRACTOR: Drilltech

DATUM: Geodetic

JN - Joint
FLT - Fault
SH - Shear
VN - Vein
CJ - Conjugate
BD - Bedding
FO - Foliation
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OR - Orthogonal
CL - Cleavage
PL - Planar
CU - Curved
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ST - Stepped
IR - Irregular
PO - Polished
K - Slickensided
SM - Smooth
RO - Rough
VR - Very 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.	FLUSH	COLOUR % RETURN	RECOVERY TOTAL CORE % 80 60 40 20	RECOVERY SOLID CORE % 80 60 40 20	R.Q.D. % 80 60 40 20	FRACT. COUNT PER 0.25 m 5 10 15 20	DISCONTINUITY DATA B Angle 0 90 180 270	DISCONTINUITY DATA DIP w.r.t. CORE AXIS 0 30 60 90	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn	HYDRAULIC CONDUCTIVITY K, cm/sec 10^{-6} 10^{-5} 10^{-4} 10^{-3}	Diametral Point Load Index (MPa) 2 4 6	RMC -Q' AVG.	NOTES
		--- CONTINUED FROM PREVIOUS PAGE ---																			
17	HQ CORE -	Slightly weathered to fresh, thinly bedded, grey, fine grained, weak SHALE (Georgian Bay Formation) with LIMESTONE interbeds		72.09	8																
		END OF BOREHOLE		17.10																	
18																					
19																					
20																					
21																					
22																					
23																					
24																					
25																					
26																					

GTA-RCK 018 S:\CLIENTS\ALECTRA_UTILITIES_INC\HYDRO_POLES_QEW-CREDIT_RIVER\02_DATA\GINT\HYDRO_POLES_QEW.GPJ GAL-MISS.GDT 8/7/18

DEPTH SCALE
1 : 50

GOLDER

LOGGED: SM/BC
CHECKED: AK

PROJECT

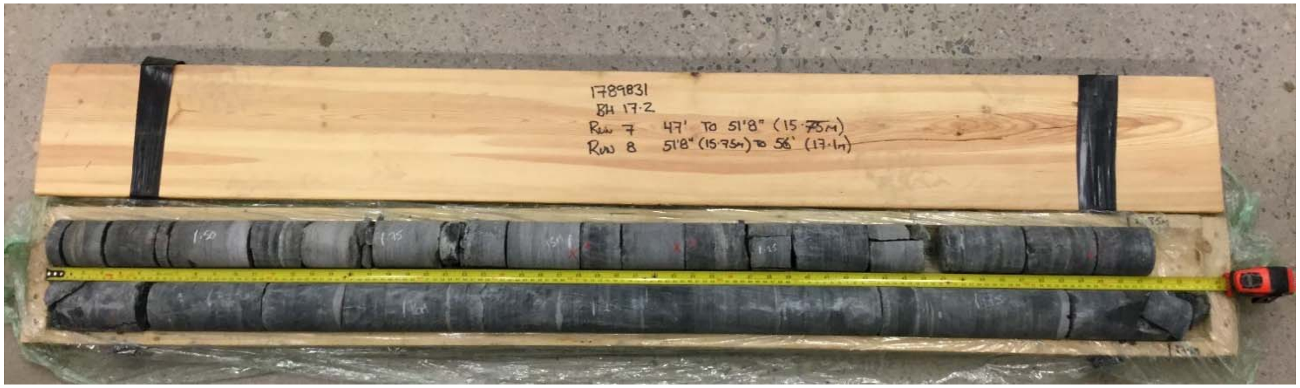
Proposed Electrical Transmission Line Monopoles at Credit River and QEW

TITLE

Bedrock Core Photograph
Borehole BH17-2 (6.1 m to 12.1 m)

GOLDER

PROJECT No.	1789831		FILE No.	----
DRAFT	SM	20180801	SCALE	AS SHOWN
CADD	--			
CHECK	AK	20180801	**FIGURE**	
REVIEW	--			



PROJECT

Proposed Electrical Transmission Line Monopoles at Credit River and QEW

TITLE

Bedrock Core Photograph
Borehole BH17-2 (12.1 m to 17.1 m)

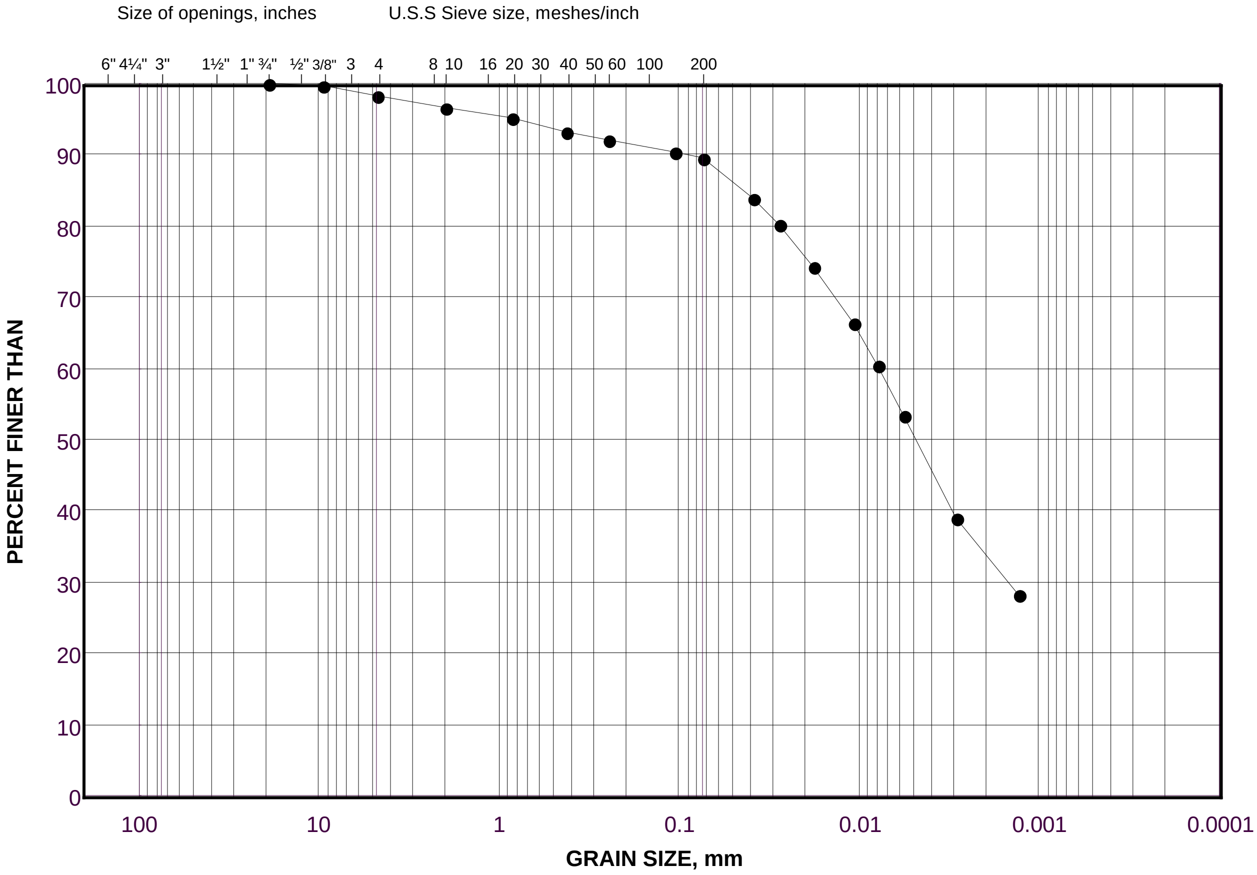
GOLDER

PROJECT No.	1789831		FILE No.	----
DRAFT	SM	20180801	SCALE	AS SHOWN
CADD	--			
CHECK	AK	20180801	FIGURE	
REVIEW	--			

GRAIN SIZE DISTRIBUTION

(CI) SILTY CLAY

FIGURE



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

LEGEND

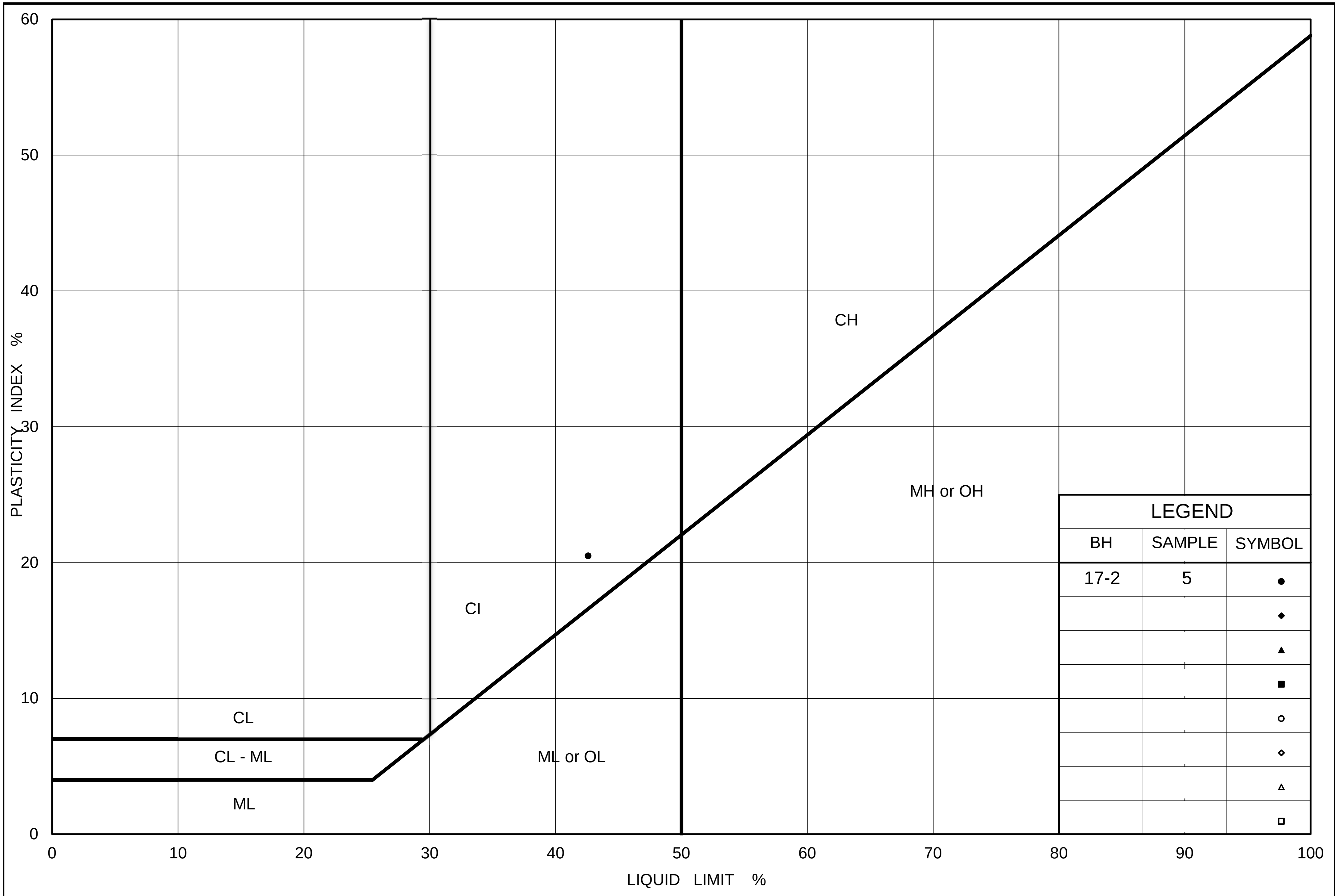
SYMBOL	Borehole	SAMPLE
●	17-2	5

Project Number: 1789831

Checked By: AK

Golder Associates

Date: 19-Jul-18



GOLDER

PLASTICITY CHART
(CI) - SILTY CLAY

Figure No.

Project No. 1789831

Checked By: AK

Your Project #: 1789831
Site Location: QEW/CREDIT RIVER
Your C.O.C. #: 112658

Attention: Jeff Tolton

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

Report Date: 2018/07/17
Report #: R5299625
Version: 1 - Final

CERTIFICATE OF ANALYSIS

MAXXAM JOB #: B8G6725
Received: 2018/07/05, 15:33

Sample Matrix: Soil
Samples Received: 1

Analyses	Quantity	Date Extracted	Date Analyzed	Laboratory Method	Reference
Chloride (20:1 extract)	1	N/A	2018/07/10	CAM SOP-00463	EPA 325.2 m
Conductivity	1	N/A	2018/07/11	CAM SOP-00414	OMOE E3530 v1 m
pH CaCl2 EXTRACT	1	2018/07/12	2018/07/12	CAM SOP-00413	EPA 9045 D m
Resistivity of Soil	1	2018/07/05	2018/07/11	CAM SOP-00414	SM 23 2510 m
Sulphate (20:1 Extract)	1	N/A	2018/07/09	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 #: 1789831
Site Location: QEW/CREDIT RIVER
Your C.O.C. #: 112658

Attention: Jeff Tolton

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

Report Date: 2018/07/17
Report #: R5299625
Version: 1 - Final

CERTIFICATE OF ANALYSIS

MAXXAM JOB #: B8G6725
Received: 2018/07/05, 15:33

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.

RESULTS OF ANALYSES OF SOIL

Maxxam ID			HDH112			HDH112		
Sampling Date			2018/06/25			2018/06/25		
COC Number			112658			112658		
	UNITS	**Criteria**	**BH17-2 SA4-5**	**RDL**	**QC Batch**	**BH17-2 SA4-5 Lab-Dup**	**RDL**	**QC Batch**
Calculated Parameters								
Resistivity	ohm-cm	-	4500		5615167			
Inorganics								
Soluble (20:1) Chloride (Cl-)	ug/g	-	<20	20	5618714			
Conductivity	umho/cm	**470**	224	2	5621364	222	2	5621364
Available (CaCl2) pH	pH	**-**	7.54		5624965			
Soluble (20:1) Sulphate (SO4)	ug/g	**-**	70	20	5618691			

RDL = Reportable Detection Limit
QC Batch = Quality Control Batch
Lab-Dup = Laboratory Initiated Duplicate
Criteria: Ontario Reg. 153/04 (Amended April 15, 2011)
Table 1: Full Depth Background Site Condition Standards
Soil - Agricultural or Other Property Use

TEST SUMMARY

Maxxam ID: HDH112
Sample ID: BH17-2 SA4-5
Matrix: Soil

Collected: 2018/06/25
Shipped:
Received: 2018/07/05

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	5618714	N/A	2018/07/10	Deonarine Ramnarine
Conductivity	AT	5621364	N/A	2018/07/11	Tahir Anwar
pH CaCl2 EXTRACT	AT	5624965	2018/07/12	2018/07/12	Gnana Thomas
Resistivity of Soil		5615167	2018/07/11	2018/07/11	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	5618691	N/A	2018/07/09	Deonarine Ramnarine

Maxxam ID: HDH112 Dup
Sample ID: BH17-2 SA4-5
Matrix: Soil

Collected: 2018/06/25
Shipped:
Received: 2018/07/05

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Conductivity	AT	5621364	N/A	2018/07/11	Tahir Anwar

GENERAL COMMENTS

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

Package 1	10.0°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**
5618691	Soluble (20:1) Sulphate (SO4)	2018/07/09	NC	70 - 130	99	70 - 130	<20	ug/g	9.3	35
5618714	Soluble (20:1) Chloride (Cl-)	2018/07/10	105	70 - 130	102	70 - 130	<20	ug/g	NC	35
5621364	Conductivity	2018/07/11			101	90 - 110	<2	umho/cm	0.89	10
5624965	Available (CaCl2) pH	2018/07/12			101	97 - 103			0.0019	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 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).

Brad Newman, Scientific Service 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

6740 Campobello Road, Mississauga, Ontario L5N 2L8
Phone: 905-817-5700 Fax: 905-81 Toll Free: 800-563-6266
CAM FCD-01191/3

IMMEDIATE

For: Ema Gitej

CHAIN OF CUSTODY RECORD 112658

Page ____ of ____

Invoice Information

Company Name: Golder Associates
Contact Name: Erti Mansaku
Address: 6925 Century Avenue,
Phone: Fax:
Email: erti_mansaku@golder.com

Report Information (if differs from invoice)

Company Name: Golder Associates
Contact Name: Erti Mansaku / Jeff Tolton
Address:
Phone: Fax:
Email: jeff_tolton@golder.com

Project Information (where applicable)

Quotation #:
P.O. #/ AFE#:
Project #: 1789831
Site Location: QEW / Credit River
Site #:
Sampled By: Brad Crowe

Turnaround Time (TAT) Required

☒ Regular TAT (5-7 days) Most analyses

PLEASE PROVIDE ADVANCE NOTICE FOR RUSH PROJECTS

Rush TAT (Surcharges will be applied)
☐ 1 Day ☐ 2 Days ☐ 3-4 Days

Date Required:
Rush Confirmation #:

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

Regulation 153

☒ Table 1 ☐ Res/Park ☒ Med/ Fine
☐ Table 2 ☐ Ind/Comm ☐ Coarse
☐ Table 3 ☒ Agri/ Other
☐ Table ____
FOR RSC (PLEASE CIRCLE) Y / [✓ N]

Other Regulations

☐ CCME ☐ Sanitary Sewer Bylaw
☐ MISA ☐ Storm Sewer Bylaw
☐ PWQO Region ____
☐ Other (Specify) ____
☐ REG 558 (MIN. 3 DAY TAT REQUIRED)

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

	SAMPLE IDENTIFICATION	DATE SAMPLED (YYYY/MM/DD)	TIME SAMPLED (HH:MM)	MATRIX	# OF CONTAINERS SUBMITTED	FIELD FILTERED (CIRCLE) Metals / Hg / CrVI	BTEX/ PHC F1	PHCs F2 - F4	VOCs	REG 153 METALS & INORGANICS	REG 153 ICPMS METALS	REG 153 METALS (Hg, Cr VI, ICPMS Metals, HWS - B)	Corrositivity package							HOLD- DO NOT ANALYZE
1	BH17-2 SA4-5	Jun 25, 2018	PM	SOIL									X							
2																				
3																				
4																				
5																				
6																				
7																				
8																				
9																				
10																				

05-Jul-18 15:33
Ema Gitej
B8G6725
GK1 ENV-676

LABORATORY USE ONLY

CUSTODY SEAL Y / [✓ N]		COOLER TEMPERATURES
Present	Intact	
		10 | 10 | 10

COOLING MEDIA PRESENT: Y / [✓ N]

COMMENTS

RELINQUISHED BY: (Signature/Print)	DATE: (YYYY/MM/DD)	TIME: (HH:MM)	RECEIVED BY: (Signature/Print)	DATE: (YYYY/MM/DD)	TIME: (HH:MM)
Erti Mansaku	2018/07/05	PM		2018/07/05	15:33

MAXXAM JOB #

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. Sample container, preservation, hold time and packages information can be viewed at http://maxxam.ca/wp-content/uploads/Ontario-COC.pdf.

White: Maxxam ~ Yellow: Client

COC-1004 (03/17)

Current Investigation - Boreholes Advanced for Other Structures

Boreholes AR-2, FW-1, EW-1 and EW-2

RECORD OF BOREHOLE No AR-2

SHEET 1 OF 1

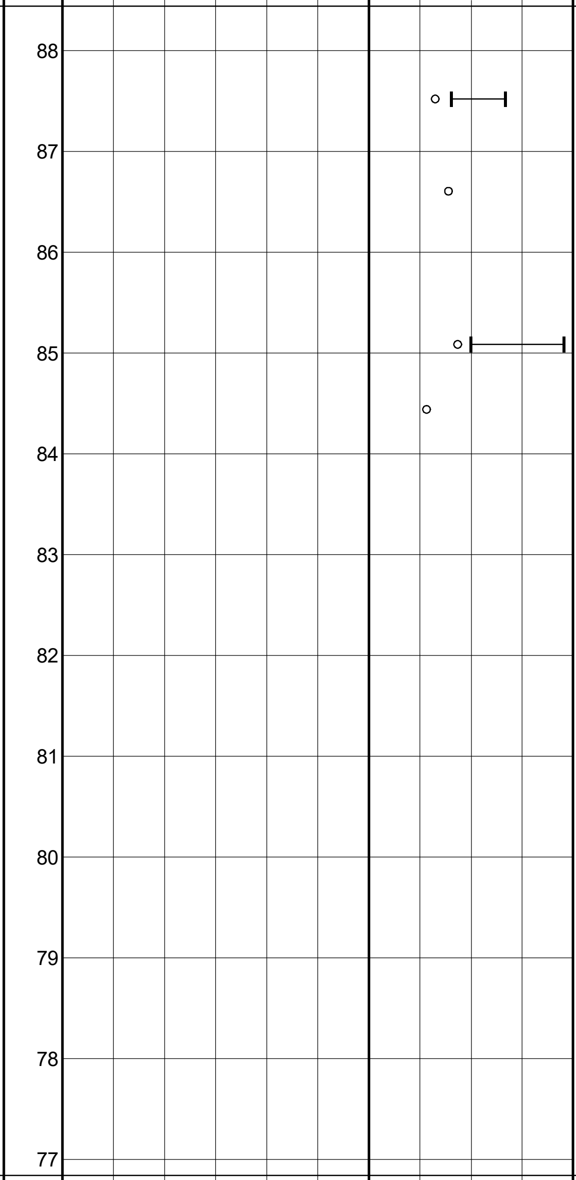
METRIC

PROJECT 1662333
G.W.P. 2002-13-00
DIST Central HWY QEW
DATUM Geodetic
LOCATION N 4824172.2; E 295921.4 MTM NAD 83 ZONE 10 (LAT. 43.557434; LONG. -79.609899)
BOREHOLE TYPE CME 75, 114 mm I.D. Hollow Stem Augers, HQ Casing
DATE July 30, 2018
ORIGINATED BY ACM
COMPILED BY CC
CHECKED BY SMM

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
88.4	GROUND SURFACE					
0.0	TOPSOIL (600 mm)	1	SS	9		
87.8 0.6	CLAYEY SILT, some sand, some gravel, trace rootlets, shale fragments (TILL) Stiff Brown to grey Moist	2A 2B	SS	14		
87.3 1.1	SILT and SAND, trace to some clay, trace to some gravel, clayey silt pockets, shale fragments (TILL) Loose to compact Brown Moist	3	SS	6		6 56 31 7
86.2 2.2	Sandy SILTY CLAY, trace to some gravel, trace shale fragments (TILL) Stiff Brown grey with oxidation staining Moist	4	SS	11		
		5	SS	13		7 22 44 27
84.6 3.8	CLAYEY SILT, some sand, some shale fragments (RESIDUAL SOIL) Hard Brown grey Moist	6	SS	50/0.08		
83.8 4.6	SHALE (BEDROCK) Grey Bedrock cored from a depth of 4.6 m to 11.6 m For bedrock coring details, refer to Record of Drillhole AR-2	7	SS	50/0.13		
		1	RC	REC 100%		RQD = 18%
		2	RC	REC 100%		RQD = 86%
		3	RC	REC 100%		RQD = 97%
		4	RC	REC 100%		RQD = 93%
		5	RC	REC 100%		RQD = 57%
76.8 11.6	END OF BOREHOLE NOTES: 1. Borehole dry prior to rock coring.					

STRAT PLOT

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



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

PROJECT: 1662333

RECORD OF DRILLHOLE: AR-2

SHEET 1 OF 1

LOCATION: N 4824172.2 ;E 295921.4
INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: July 30, 2018
DRILL RIG: CME 75
DRILLING CONTRACTOR: Davis Drilling

DATUM: Geodetic

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

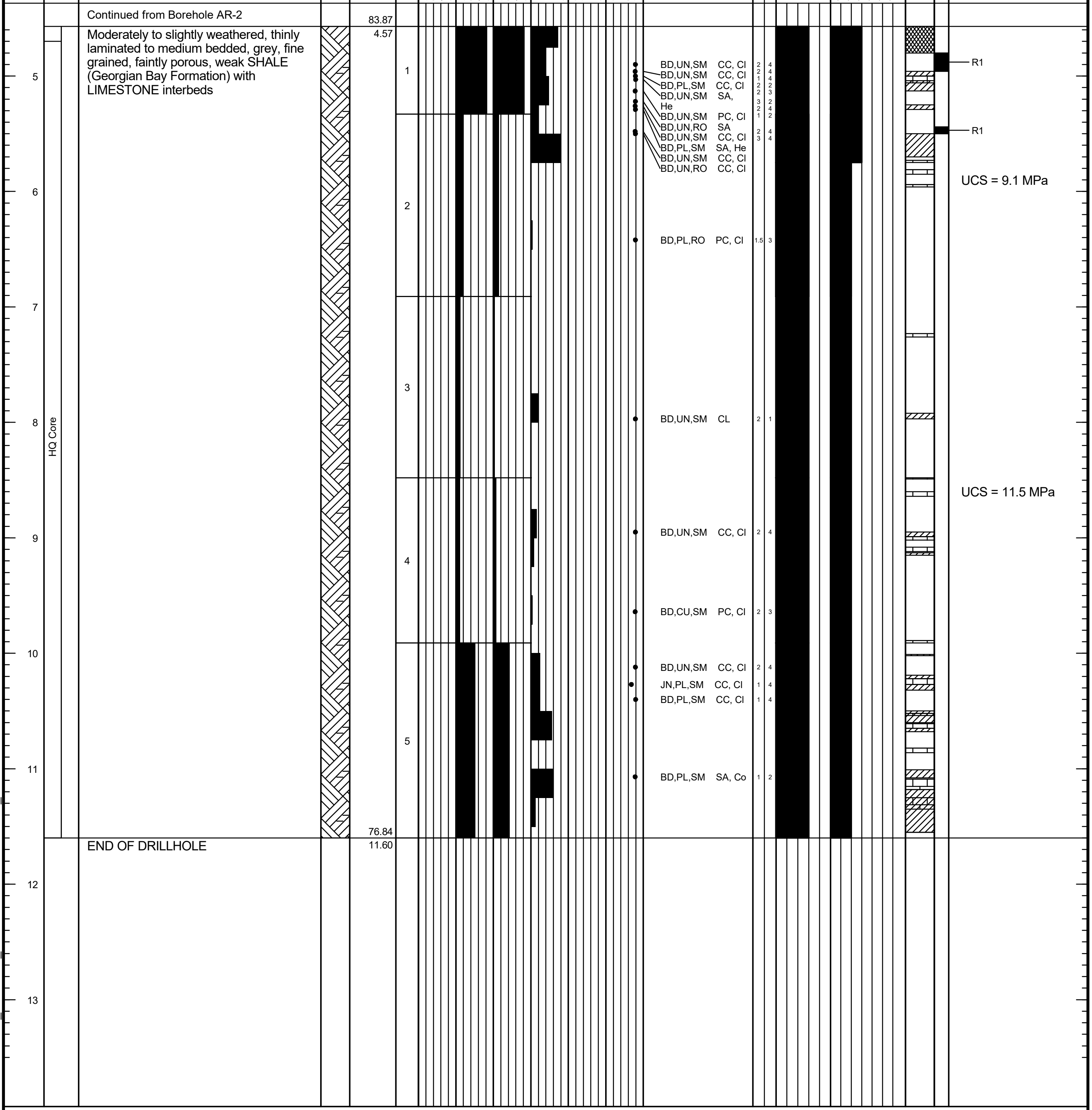
BD- Bedding
FO- Foliation
CO- Contact
OR- Orthogonal
CL - Cleavage

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

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

BR - Broken Rock

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



Continued from Borehole AR-2 — 83.87

Moderately to slightly weathered, thinly laminated to medium bedded, grey, fine grained, faintly porous, weak SHALE (Georgian Bay Formation) with LIMESTONE interbeds — 4.57

76.84
END OF DRILLHOLE — 11.60

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

UCS = 9.1 MPa

UCS = 11.5 MPa

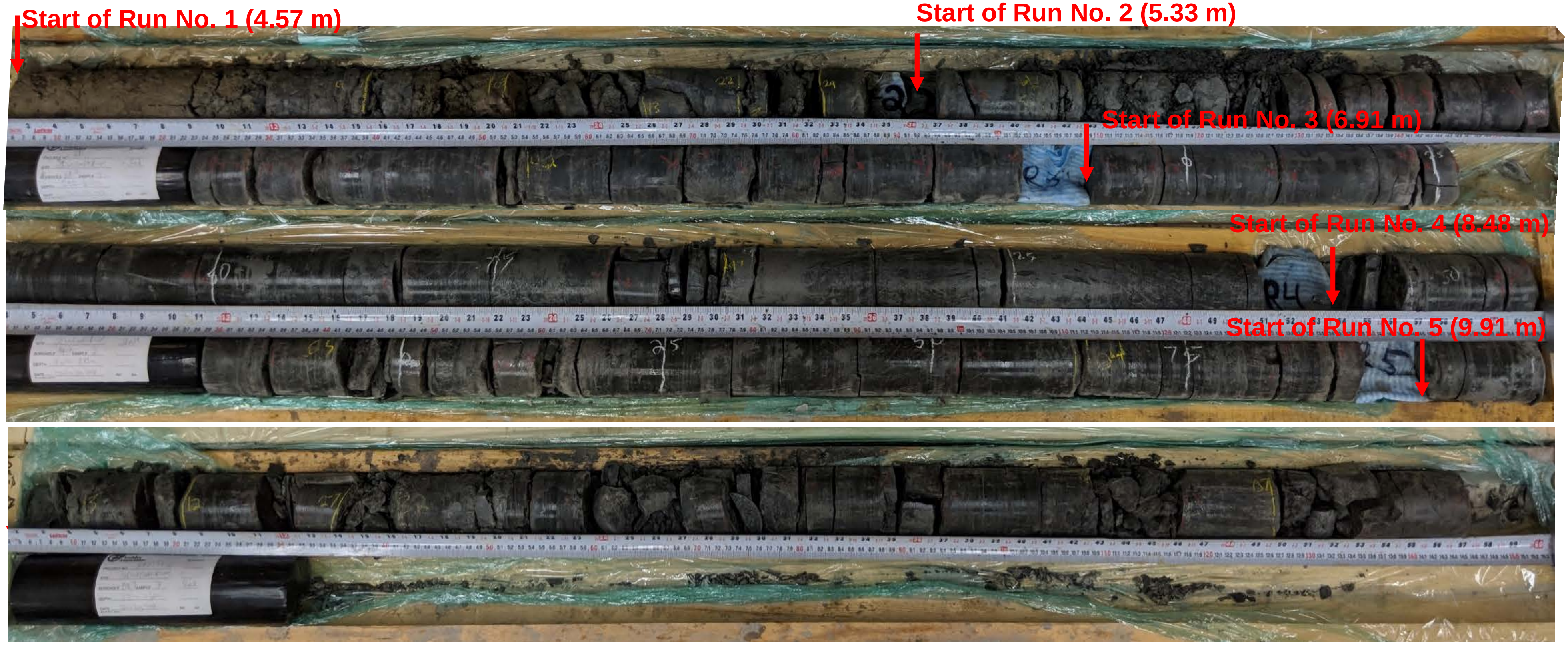
FEATURES LEGEND

▨	BROKEN CORE
▩	CLAY SEAM
▦	LIMESTONE
■	LOST CORE

DEPTH SCALE
1 : 50

GOLDER

LOGGED: ACM
CHECKED: AB

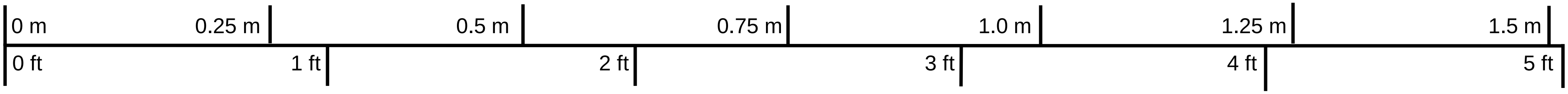

Start of Run No. 1 (4.57 m)

Start of Run No. 2 (5.33 m)

Start of Run No. 3 (6.91 m)

Start of Run No. 4 (8.48 m)

Start of Run No. 5 (9.91 m)



Scale

PROJECT

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

TITLE

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

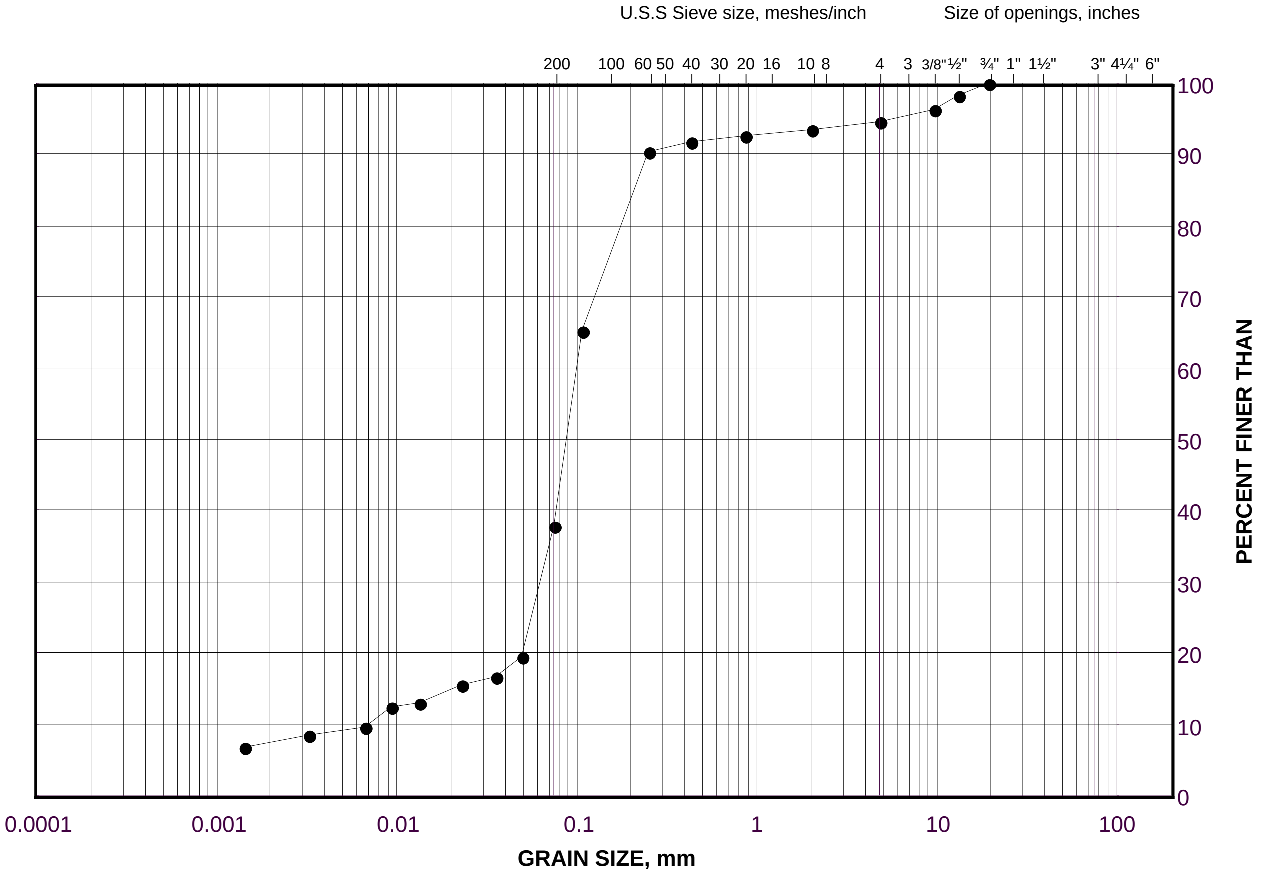
GOLDER

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

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

GRAIN SIZE DISTRIBUTION
Silt and Sand (Till)

FIGURE



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

LEGEND

SYMBOL	Borehole	SAMPLE	DEPTH(m)
●	AR-2	3	1.52 - 2.13

Project Number: 1662333

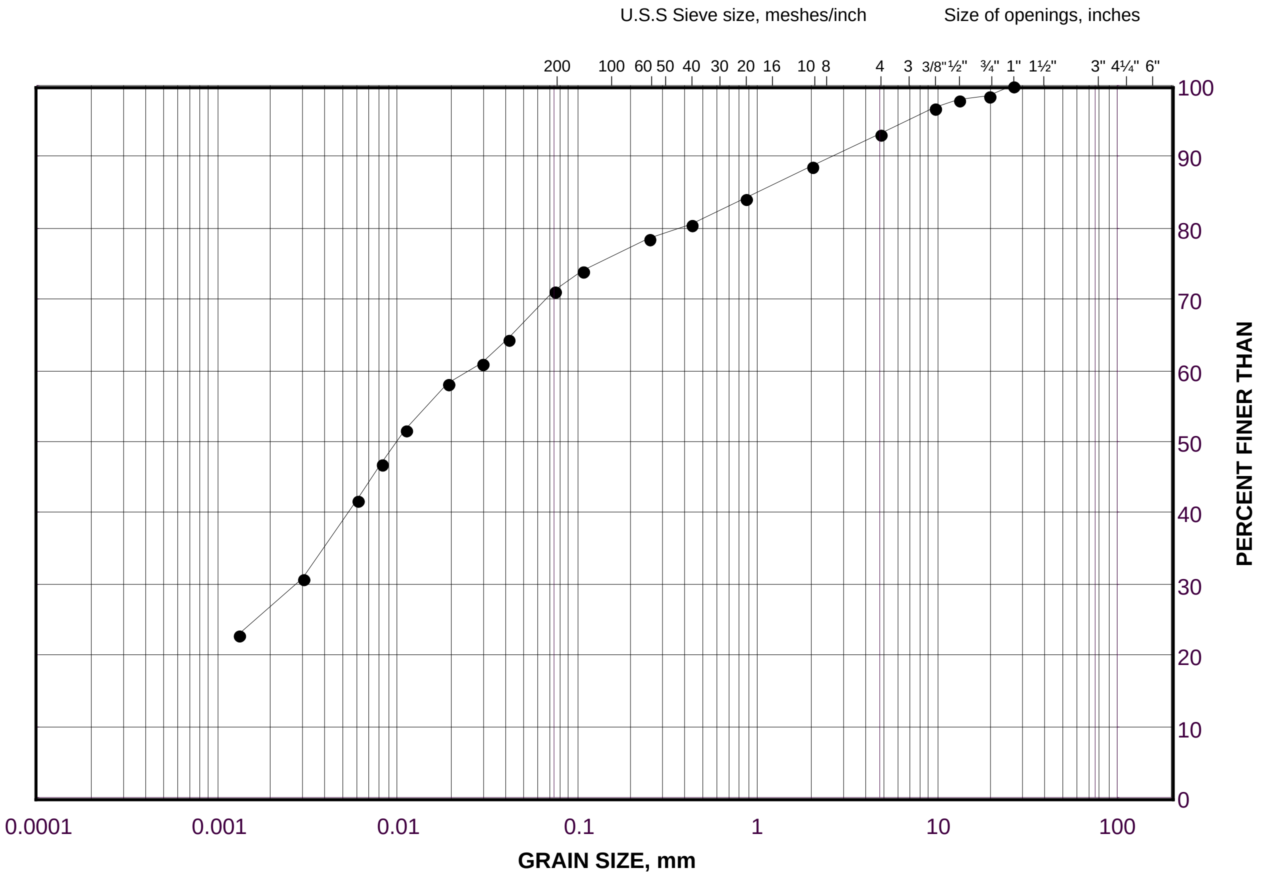
Checked By: SMM

Golder Associates

Date: 09-Aug-18

GRAIN SIZE DISTRIBUTION
Sandy Silty Clay (Till)

FIGURE



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

LEGEND

SYMBOL	Borehole	SAMPLE	DEPTH(m)
●	AR-2	5	3.05 - 3.66

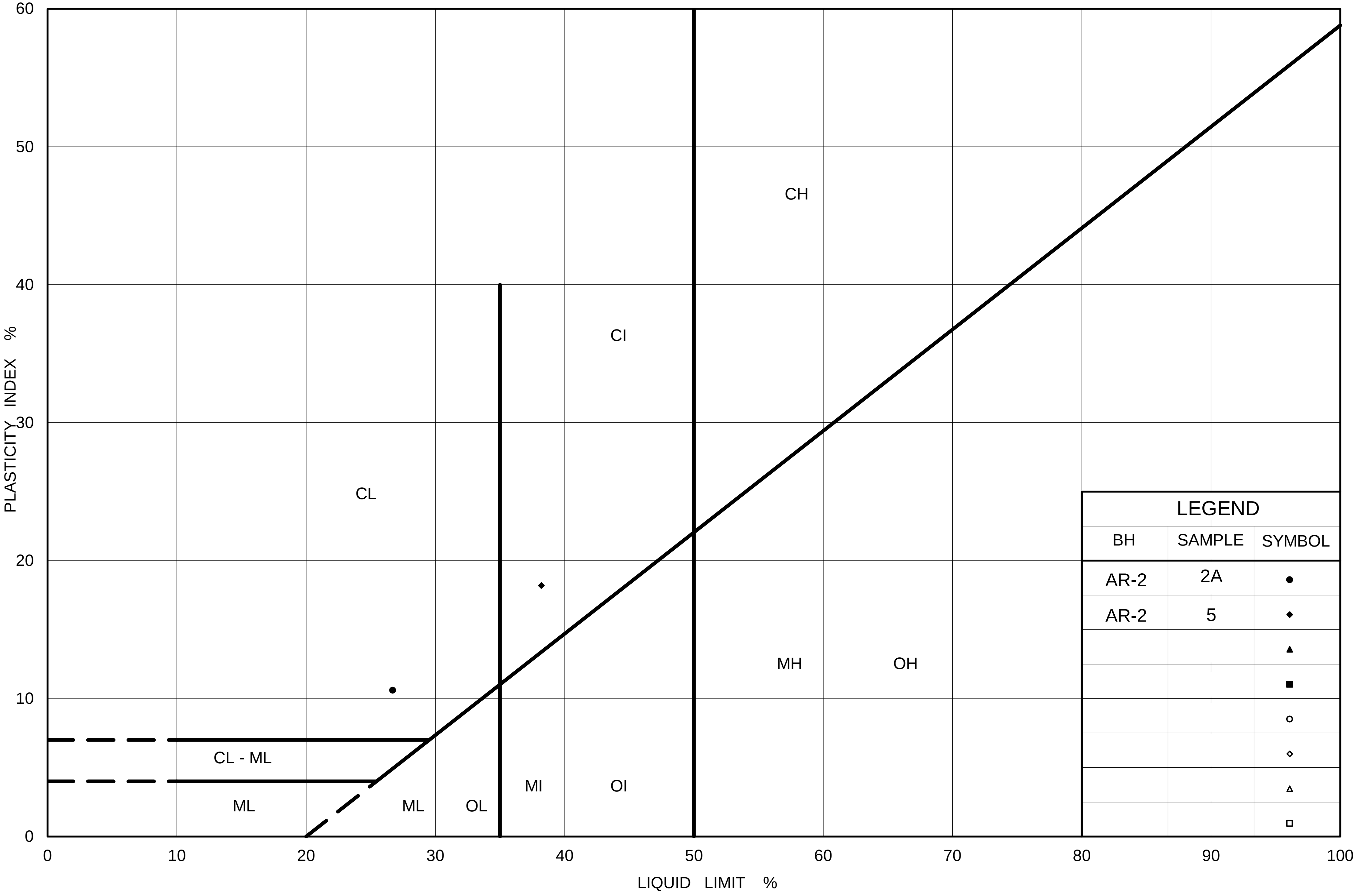
Project Number: 1662333

Checked By: SMM

Golder Associates

Date: 09-Aug-18

Oct 75, FF-S-21



Ministry of Transportation

Ontario

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

Figure No.

Project No. 1662333

Checked By: SMM

RECORD OF BOREHOLE No FW-1

SHEET 1 OF 1

METRIC

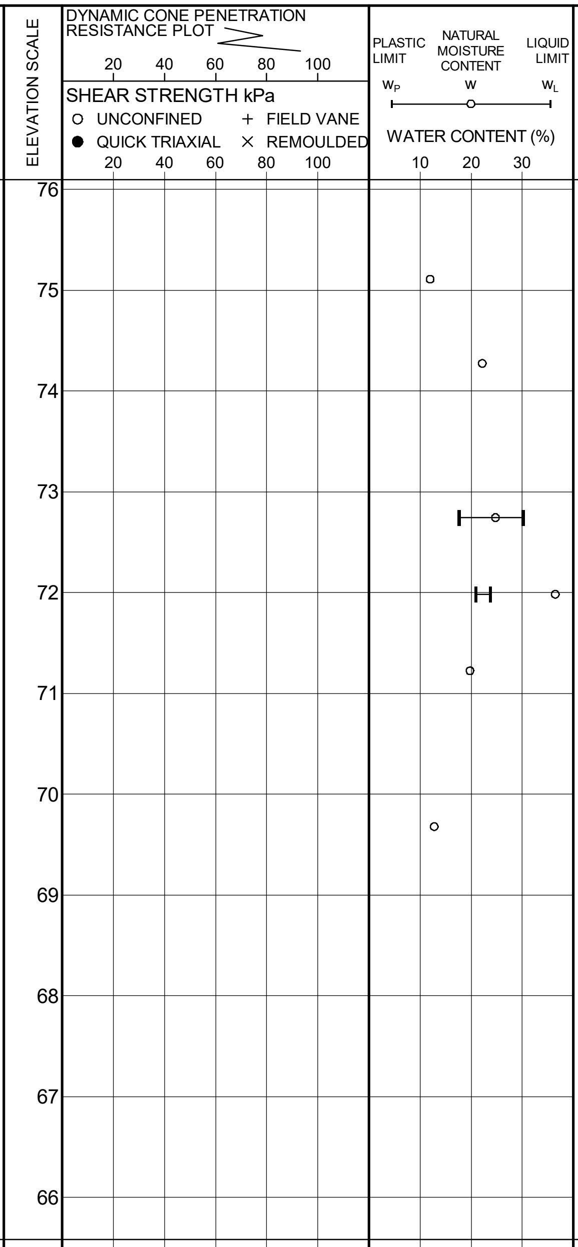
PROJECT 1662333
G.W.P. 2002-13-00
DIST Central HWY QEW
DATUM Geodetic
LOCATION N 4823994.8; E 295856.0 MTM NAD 83 ZONE 10 (LAT. 43.555836; LONG. -79.610704)
BOREHOLE TYPE CME 55, 114 mm O.D., Hollow Stem Augers
DATE October 10, 2018
ORIGINATED BY ACM
COMPILED BY JMP
CHECKED BY SMM

SOIL PROFILE

ELEV DEPTH	DESCRIPTION
76.1	GROUND SURFACE
0.0	Sand and gravel, some silt (FILL) Compact Brown Moist
75.4 0.7	CLAYEY SILT, interbedded with SILT and SAND, some clay, trace gravel Soft to very stiff Grey-brown to dark brown Moist
73.1 3.1	CLAYEY SILT, some sand, some shell fragments, trace organics Soft Grey-brown Moist
72.4 3.7	SILT and SAND, some gravel, trace to some clay, some shell fragments Loose Dark brown Wet
71.5 4.6	SAND and GRAVEL, trace to some silt, trace clay, some shell fragments from 6.1 m to 6.2 m Compact Dark brown to grey Wet
69.9	CLAYEY SILT, some gravel, some shale fragments (RESIDUAL SOIL) Hard Grey Moist
69.5 6.6	SHALE (BEDROCK) Grey Bedrock cored from a depth of 7.0 m to 10.5 m For bedrock coring details, refer to Record of Drillhole FW-1
65.6 10.5	END OF BOREHOLE NOTES: 1. Water level measured at 3.5 m (Elev. 72.6 m) below ground surface upon completion of drilling.

SAMPLES

NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
1	SS	26					
2A 2B	SS	16					
3	SS	7		1	31	53	15
4	SS	3					
5	SS	2	∇				
6	SS	6		12	47	33	8
7	SS	14		49	39	10	2
8A 8B 8C	SS	100/0.2					
1	RC	REC 90%		RQD = 90%			
2	RC	REC 91%		RQD = 73%			
3	RC	REC 100%		RQD = 94%			



UNIT WEIGHT γ kN/m³

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

PROJECT: 1662333

RECORD OF DRILLHOLE: FW-1

SHEET 1 OF 1

LOCATION: N 4823994.8 ;E 295856.0

INCLINATION: -90°   AZIMUTH: ---

DRILLING DATE: October 10, 2018

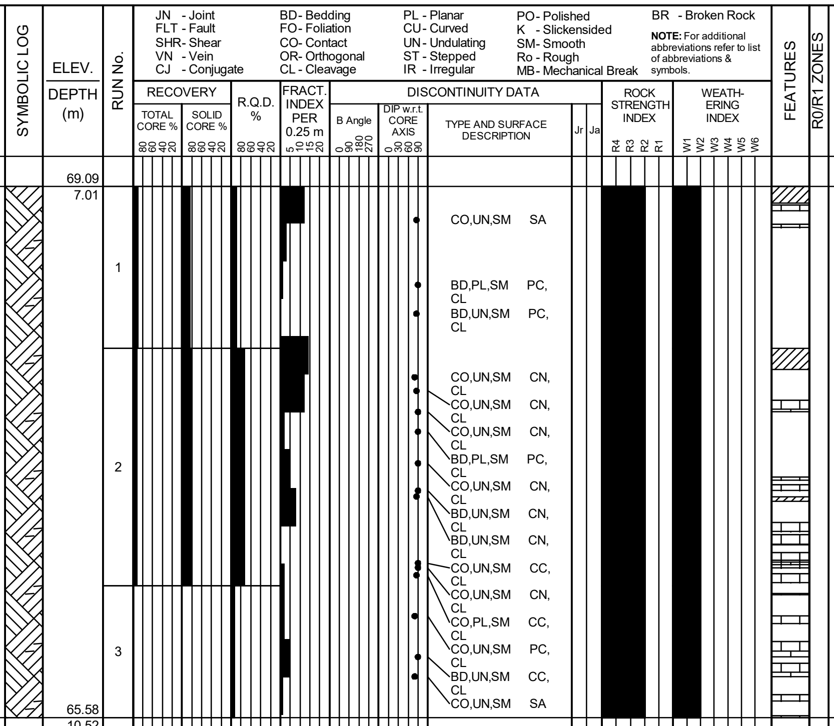
DRILL RIG: CME 55

DRILLING CONTRACTOR: Geo-Environmental Drilling

DATUM: Geodetic

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. DEPTH (m)	RUN No.
		Continued from Record of Borehole FW-1	69.09	
		Moderate to slighly weathered, thinly bedded, grey, fine grained, faintly porous, weak SHALE (Georgian Bay Formation) with limestone interbeds.	7.01	1 2 3
			65.58 10.52	
		END OF DRILLHOLE		



TYPE AND SURFACE DESCRIPTION

- CO,UN,SM SA
- BD,PL,SM PC, CL
- BD,UN,SM PC, CL
- CO,UN,SM CN, CL
- CO,UN,SM CN, CL
- CO,UN,SM CN, CL
- BD,PL,SM PC, CL
- CO,UN,SM CN, CL
- BD,UN,SM CN, CL
- BD,UN,SM CN, CL
- CO,UN,SM CC, CL
- CO,UN,SM CN, CL
- CO,PL,SM CC, CL
- CO,UN,SM PC, CL
- BD,UN,SM CC, CL
- CO,UN,SM SA

NOTES WATER LEVELS INSTRUMENTATION

DEPTH SCALE METRES: 8 9 10 11 12 13 14 15 16

FEATURES LEGEND

▨	BROKEN CORE
▩	CLAY SEAM
▦	LIMESTONE
■	LOST CORE

GTA-RCK 054 S:\CLIENTS\MTO\QEW-CREDIT_RIVER\QEW-CREDIT_RIVER\02_DATA\GINT\QEW-CREDIT_RIVER.GPJ GAL-MISS.GDT 19-1-4

DEPTH SCALE
1 : 50

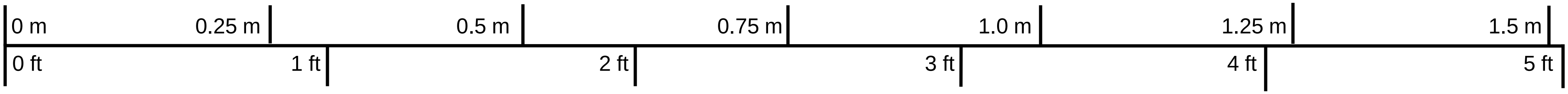
GOLDER

LOGGED: ACM
CHECKED: SK

Start of Run No. 1 (7.01 m)

Start of Run No. 2 (8.08 m)

Start of Run No. 3 (9.65 m)



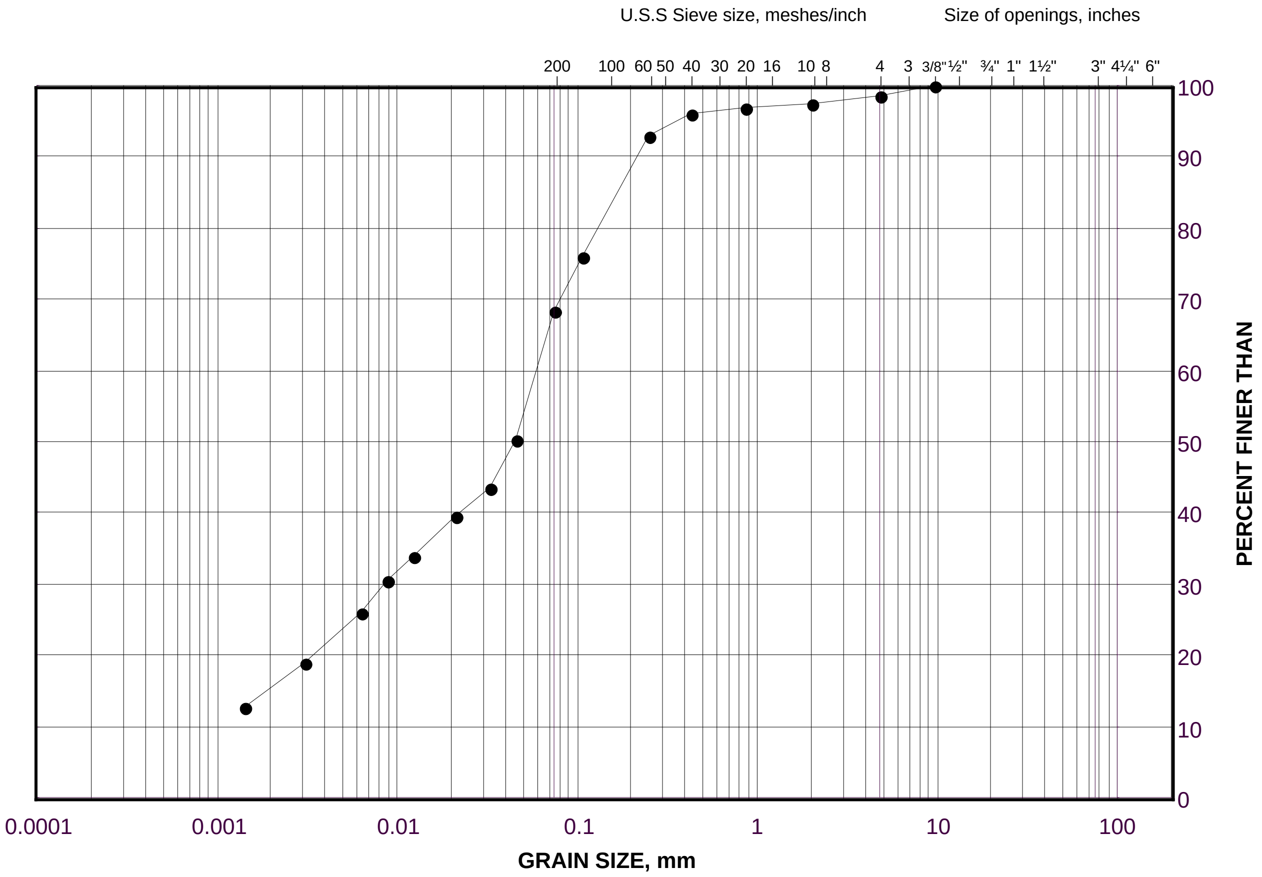
Scale

PROJECT	**MTO Assignment 2015-E-0033: Detail Design for the widening/rehab/realignment of QEW Between Mississauga Road and Hurontario Street**		
TITLE	**Bedrock Core Photograph Borehole FW-1 (7.01 m to 10.52 m)**		
GOLDER	PROJECT No. 1662333		FILE No. ----
	DRAFT	ACM	Jan. 2019	SCALE AS SHOWN VER. 1.
	CADD	--		
	CHECK			**FIGURE**
	REVIEW			

GRAIN SIZE DISTRIBUTION

Clayey Silt

FIGURE



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

LEGEND

SYMBOL	Borehole	SAMPLE	DEPTH(m)
●	FW-1	3	1.52 - 2.13

Project Number: 1662333

Checked By: SMM

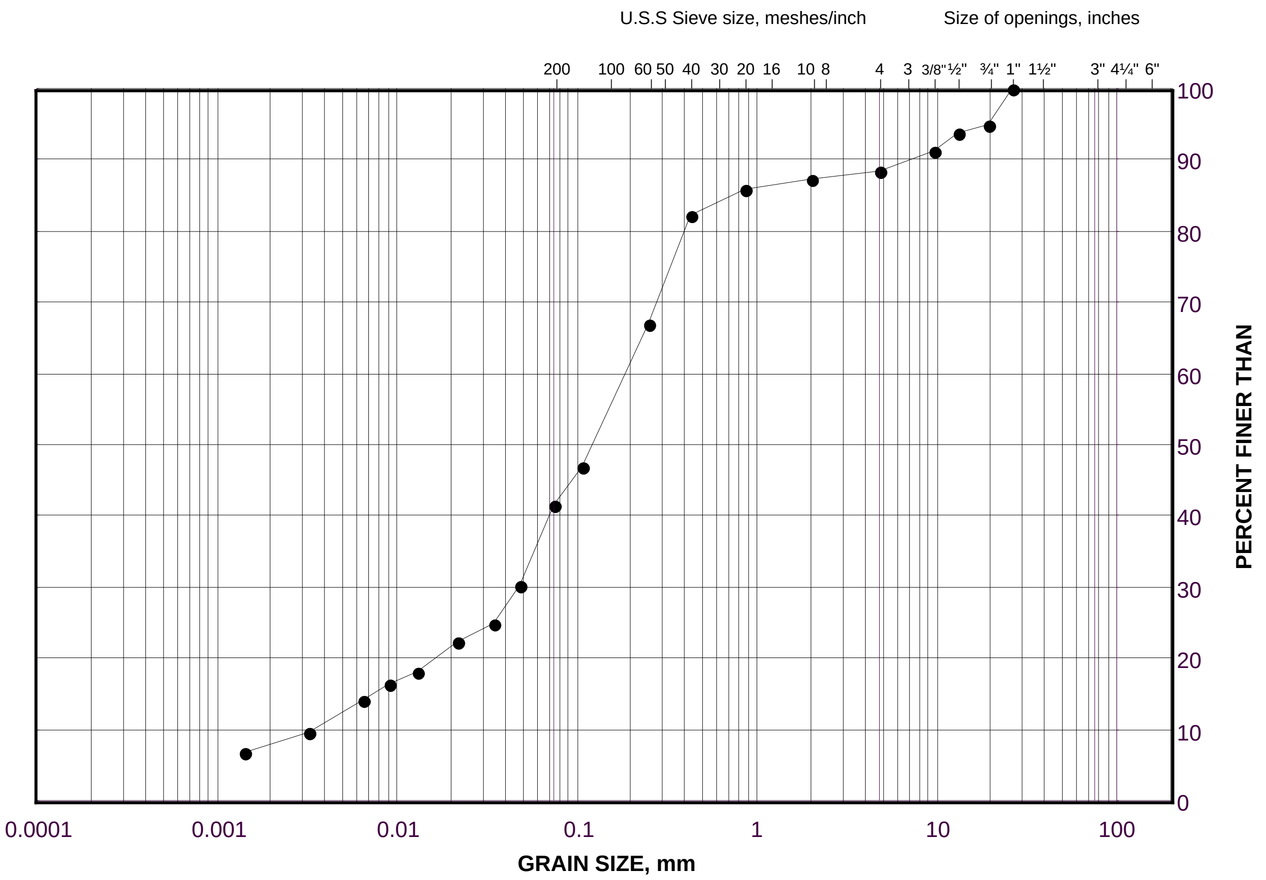
Golder Associates

Date: 09-Nov-18

GRAIN SIZE DISTRIBUTION

Silt and Sand

FIGURE



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

LEGEND

SYMBOL	Borehole	SAMPLE	DEPTH(m)
●	FW-1	6	3.81 - 4.42

Project Number: 1662333

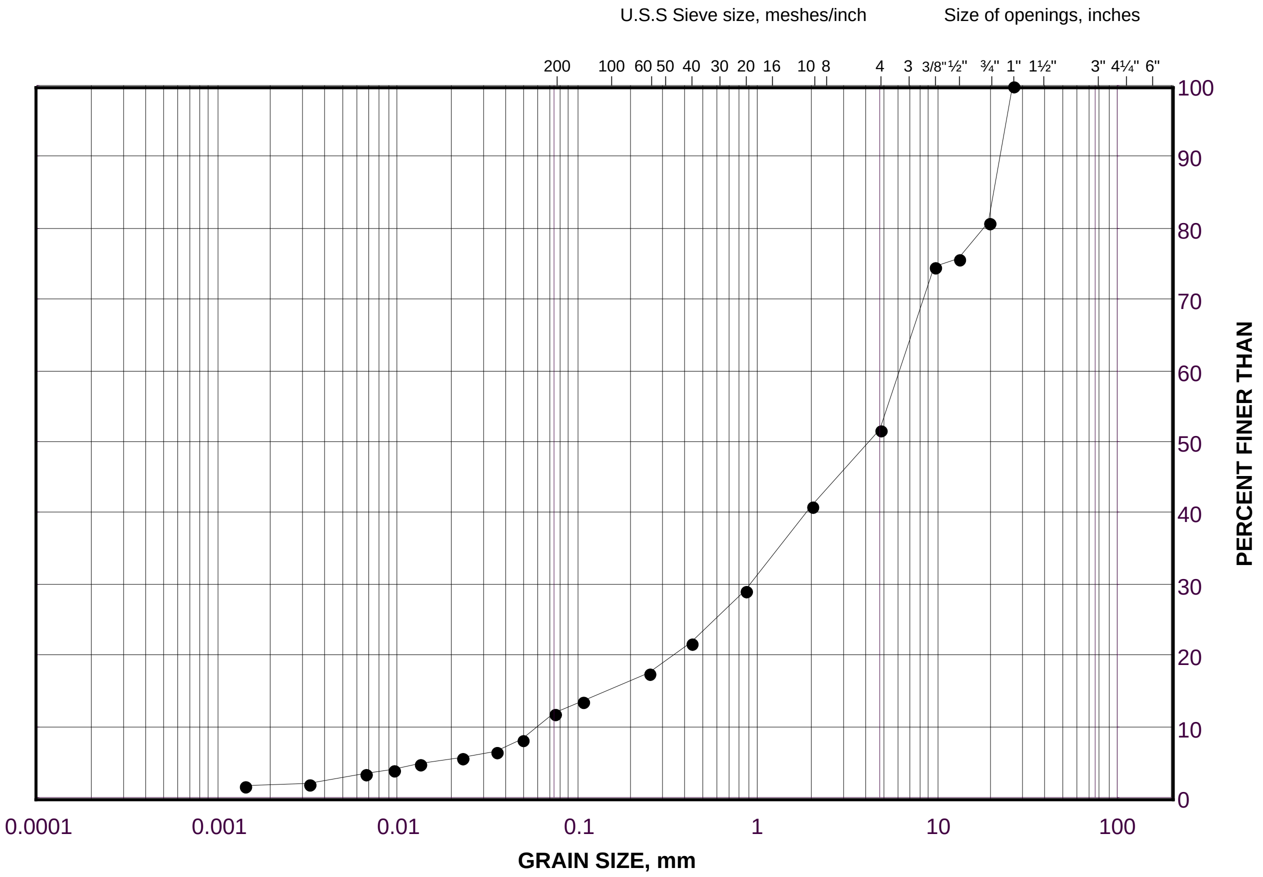
Checked By: SMM

Golder Associates

Date: 09-Nov-18

GRAIN SIZE DISTRIBUTION
Sand and Gravel

FIGURE



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

LEGEND

SYMBOL	Borehole	SAMPLE	DEPTH(m)
●	FW-1	7	4.57 - 5.18

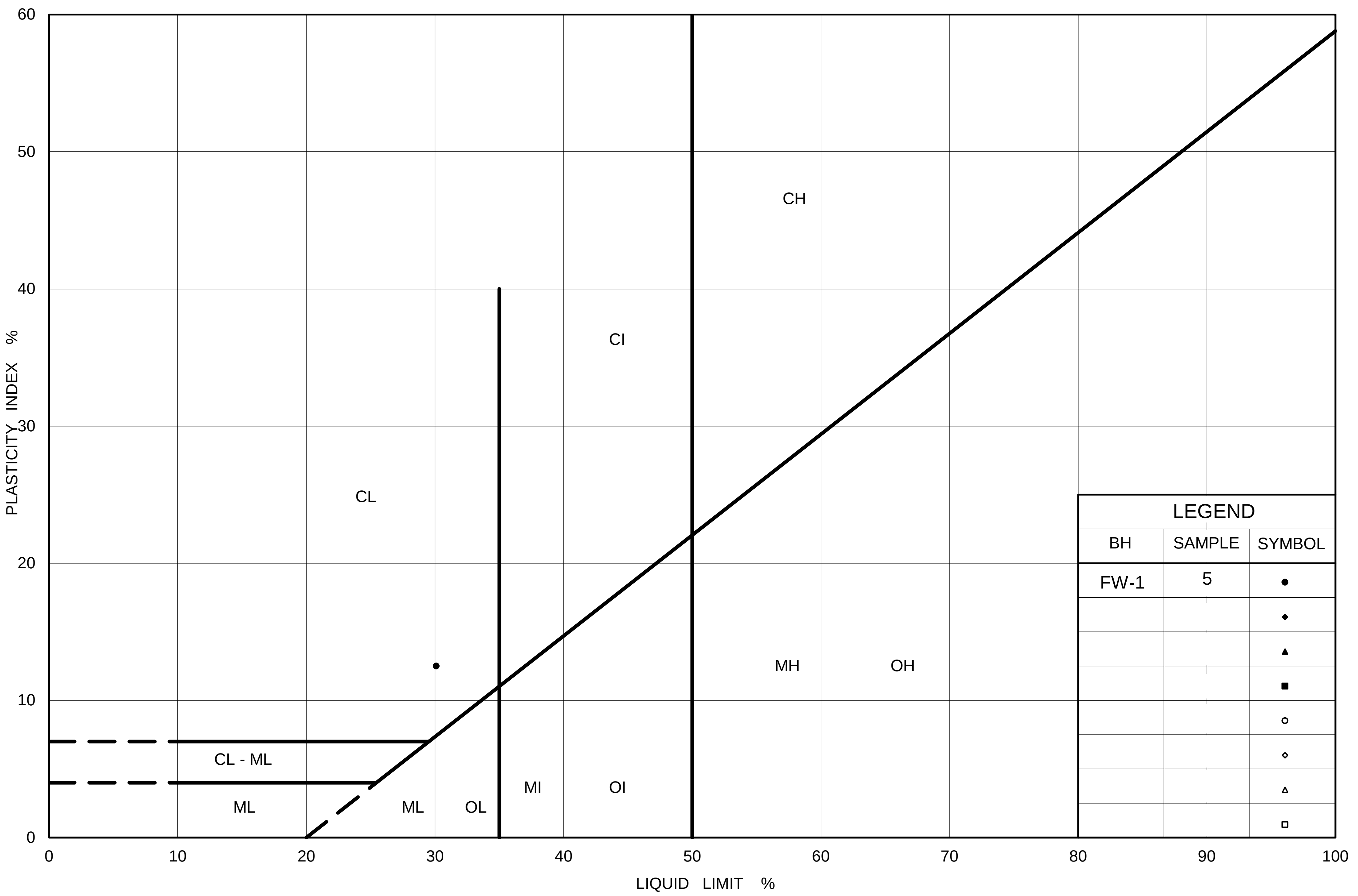
Project Number: 1662333

Checked By: SMM

Golder Associates

Date: 09-Nov-18

Oct 75, FF-S-21



Ministry of Transportation

Ontario

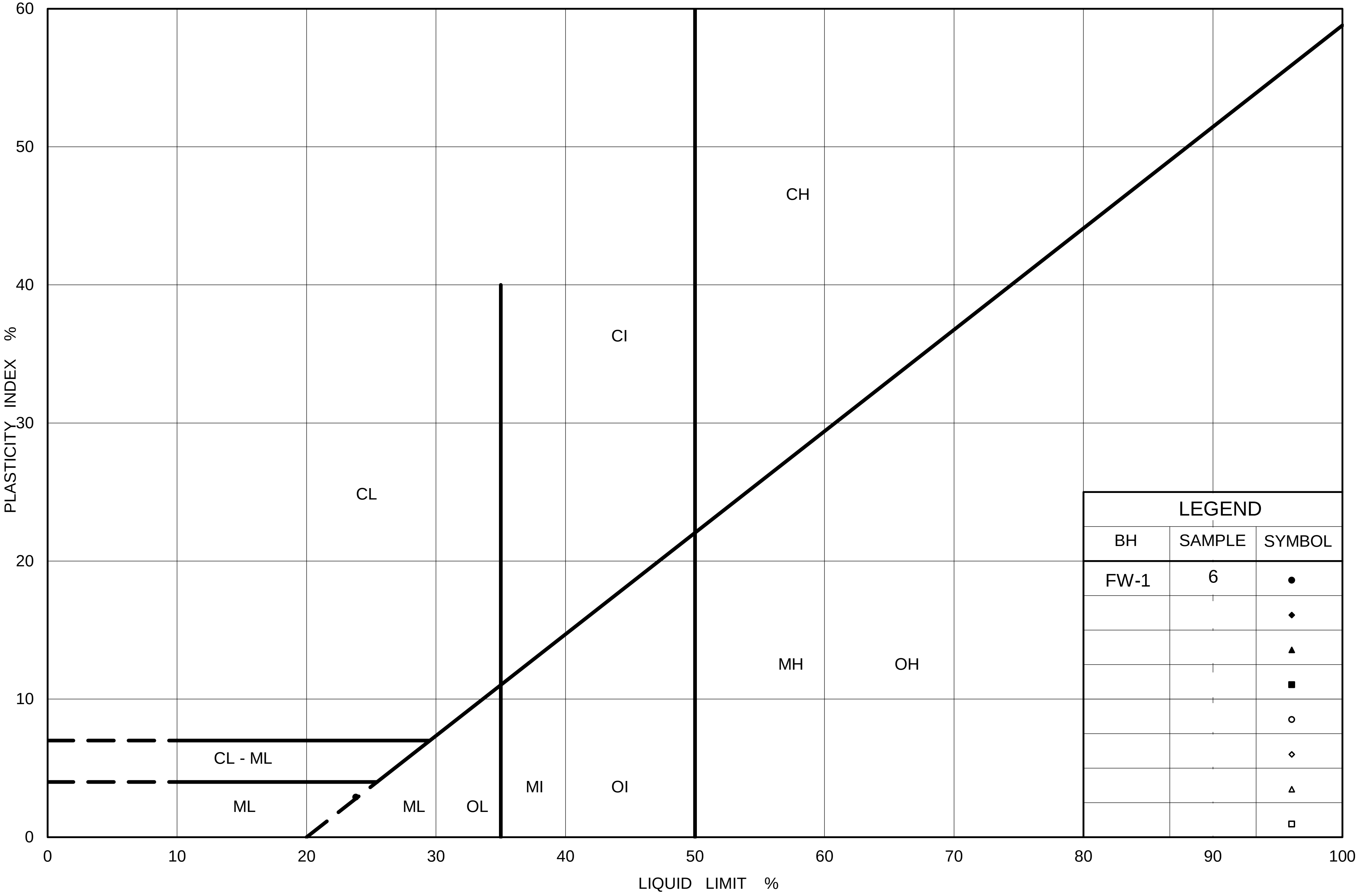
PLASTICITY CHART
Clayey Silt

Figure No.

Project No. 1662333

Checked By: SMM

Oct 75, FF-S-21



Ministry of Transportation

Ontario

PLASTICITY CHART
Silt and Sand

Figure No.

Project No. 1662333

Checked By: SMM

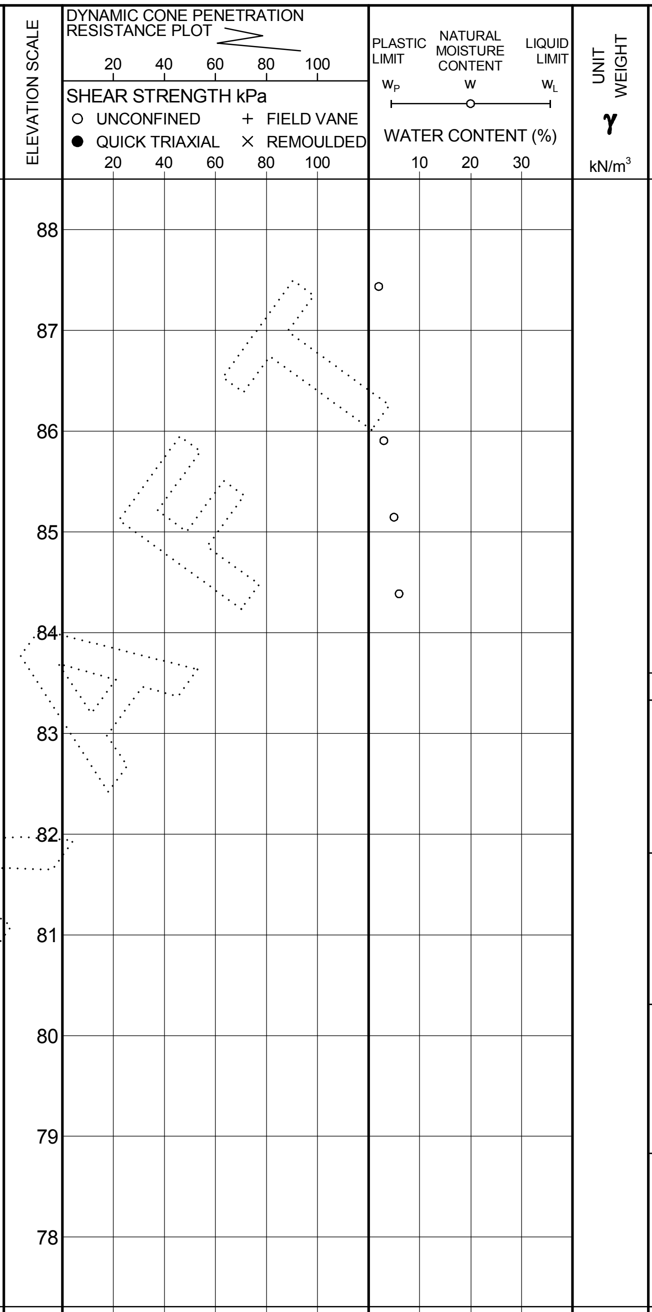
RECORD OF BOREHOLE No EW-1

SHEET 1 OF 1

METRIC

PROJECT	1662333		
G.W.P.	2002-13-00	LOCATION	N 4823955.5; E 295849.5 MTM NAD 83 ZONE 10 (LAT. 43.555482; LONG. -79.610784)
ORIGINATED BY	JL		
DIST	Central	HWY	QEW
BOREHOLE TYPE	CME 55, 159 mm O.D., Hollow Stem Augers, HQ Casing	COMPILED BY	KN
DATUM	Geodetic	DATE	May 1, 2018
CHECKED BY	SMM		

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES		GR	SA	SI	CL
88.5	GROUND SURFACE									
0.0	Sand and gravel, some silt, trace clay (FILL) Compact to dense Grey to grey-brown Dry		1	SS	30					
			2	SS	33		37	46	15	2
	- Geotextile grid fragments recovered in split-spoon sample SA#3 at 1.8 m depth and SA#4 at 2.7 m depth		3	SS	29					
			4	SS	26		32	50	16	2
			5	SS	39					
84.2			6	SS	32					
83.9 4.6	Gravelly Sandy CLAYEY SILT (TILL) Grey to brown Moist to wet		7	SS	100/0.13					
	SHALE (BEDROCK) Grey		1	RC	REC 100%		RQD = 78%			
	Bedrock cored from a depth of 4.9 m to 11.2 m For bedrock coring details, refer to Record of Drillhole EW-1		2	RC	REC 100%		RQD = 97%			
			3	RC	REC 100%		RQD = 75%			
			4	RC	REC 100%		RQD = 100%			
77.3			5	RC	REC 100%		RQD = 97%			
11.2	END OF BOREHOLE NOTES: 1. Borehole dry prior to rock coring. 2. Borehole backfilled with bentonite cement grout to 3.0 m depth, and bentonite (Hole Plug) to ground surface.									



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

GTA-MTO 001 S:\CLIENTS\MTO\QEW-CREDIT_RIVER\02_DATA\GINT\QEW-CREDIT_RIVER.GPJ GAL-GTA.GDT 11/15/18

PROJECT: 1662333

RECORD OF DRILLHOLE: EW-1

SHEET 1 OF 1

LOCATION: N 4823955.5 ;E 295849.5

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: May 1, 2018

DRILL RIG: CME 55

DRILLING CONTRACTOR: Geo-Environmental Drilling Inc.

DATUM: Geodetic

JN - Joint	BD- Bedding	PL - Planar	PO- Polished
FLT - Fault	FO- Foliation	CU- Curved	K - Slickensided
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

BR - Broken Rock

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	RECOVERY TOTAL CORE % (80 60 40 20)	RECOVERY SOLID CORE % (80 60 40 20)	R.Q.D. % (80 60 40 20)	FRACT. INDEX PER 0.25 m (5 10 15 20)	DISCONTINUITY DATA B Angle (0 90 180 270)	DIP w.r.t. CORE AXIS (0 30 60 90)	TYPE AND SURFACE DESCRIPTION	Jr	Ja	ROCK STRENGTH INDEX (R4 R3 R2 R1)	WEATHERING INDEX (W1 W2 W3 W4 W5 W6)	FEATURES	R0/R1 ZONES	NOTES WATER LEVELS INSTRUMENTATION
		Continued from Record of Borehole EW-1		83.60															
5	HQ Core	Slightly weathered to fresh, thinly laminated to medium bedded, grey, very fine to fine grained, faintly porous, weak SHALE (Georgian Bay Formation) with LIMESTONE interbeds		4.90	1							BD,UN,SM CL	2	1					
												CO,UN,SM CL	2	1					
6					2							CO,UN,SM CL	2	1					
												BD,UN,SM PC, CI	2	3					
												CO,UN,SM SA, CI	2	2				R1 R0	
7					3							CO,UN,SM CC, CI	2	4					
												JN,PL,SM CL	1	1					
												CO,UN,SM CL	2	1					UCS=156.8 MPa (Limestone)
												BD,UN,SM SA, CI	2	2					
												BD,UN,SM CC, CI	2	4					
												JN,PL,SM CL	1	1					
												JN,UN,SM CL	2	1					
												BD,UN,SM IN, CI	2	6					
												BD,UN,SM CL	2	1					
8												BD,UN,SM CL	2	1					
												CO,UN,SM SA, CI	2	2					
					4							BD,UN,SM PC, CI	2	3					
9												JN,UN,SM CL	2	1					
												JN,UN,SM CL	2	1					
10												BD,UN,SM SA, CI	2	2					
					5							BD,UN,SM SA, CI	2	2					
												BD,UN,SM SO, CI	2	1					
11												BD,UN,SM PC, CI	2	3					
												BD,UN,SM IN, CI	2	6					
		END OF DRILLHOLE		77.31 11.19															
12																			
13																			

FEATURES LEGEND

▨ BROKEN CORE	▩ CLAY SEAM	▦ LIMESTONE	■ LOST CORE

DEPTH SCALE

1 : 50

GOLDER

LOGGED: JL

CHECKED: AKV

Start of Run No. 1 (4.90 m)

Start of Run No. 2 (5.17 m)

Start of Run No. 3 (6.69 m)

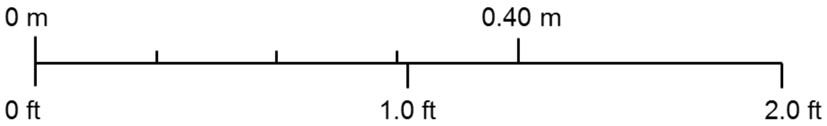
Box 1: 4.90 m to 8.19 m

Start of Run No. 4 (8.19 m)

Start of Run No. 5 (9.67 m)

End of Borehole (11.19 m)

Box 2: 8.19 m to 11.19 m



Scale

PROJECT

EAST-WEST ACTIVE TRANSPORT PEDESTRIAN BRIDGE ALONG CREDIT RIVER BRIDGE

TITLE

Bedrock Core Photographs
Borehole EW-1 (4.90 m to 11.19 m)

GOLDER

PROJECT No. 1662333			FILE No. ----		
DRAFT	SK	20180628	SCALE	NTS	VER. 1.
CADD	--		**FIGURE**		
CHECK	SMM				
REVIEW	JMAC	20181108			

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

RECORD OF BOREHOLE No EW-2

SHEET 1 OF 1

METRIC

PROJECT 1662333

G.W.P. 2002-13-00

LOCATION N 4824156.8; E 295956.2 MTM NAD 83 ZONE 10 (LAT. 43.557295; LONG. -79.609467)

ORIGINATED BY JL

DIST Central HWY QEW

BOREHOLE TYPE CME 55, 159 mm O.D., Hollow Stem Augers, HQ Casing

COMPILED BY KN

DATUM Geodetic

DATE May 2, 2018

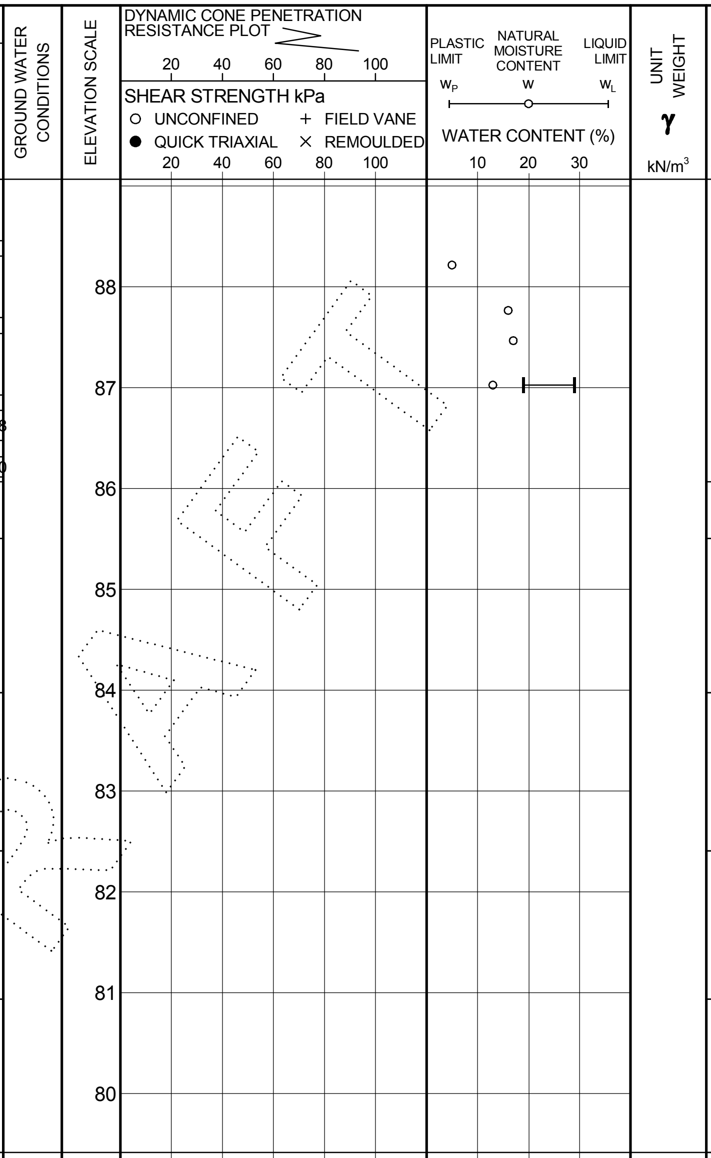
CHECKED BY SMM

ELEV DEPTH	SOIL PROFILE — DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
89.1	GROUND SURFACE				
0.0	Sandy clayey silt, trace gravel (FILL) Stiff Brown Dry	1	SS	13	
88.4 0.9	Sand and gravel, trace to some silt, trace clay (FILL) Brown Moist	2A	SS	9	36 50 12 2
		2B			
	Gravelly CLAYEY SILT with SAND (TILL) Stiff to very stiff Brown Moist	3A	SS	17	
		3B			24 38 26 12
86.9 2.2	SHALE (BEDROCK), with limestone interbeds Grey	4	SS	100/0.28	
		5	SS	100/0.20	
	Bedrock cored from a depth of 3.0 m to 9.7 m	1	RC	REC 100%	RQD = 84%
	For bedrock coring details, refer to Record of Drillhole EW-2	2	RC	REC 100%	RQD = 31%
		3	RC	REC 100%	RQD = 78%
		4	RC	REC 100%	RQD = 93%
		5	RC	REC 100%	RQD = 100%
79.4 9.7	END OF BOREHOLE				

NOTES:

1. Borehole dry prior to rock coring.

2. Borehole backfilled with bentonite cement grout to 1.5 m depth, and bentonite (Hole Plug) to ground surface.



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

PROJECT: 1662333

RECORD OF DRILLHOLE: EW-2

SHEET 1 OF 1

LOCATION: N 4824156.8 ;E 295956.2

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: May 2, 2018

DRILL RIG: CME 55

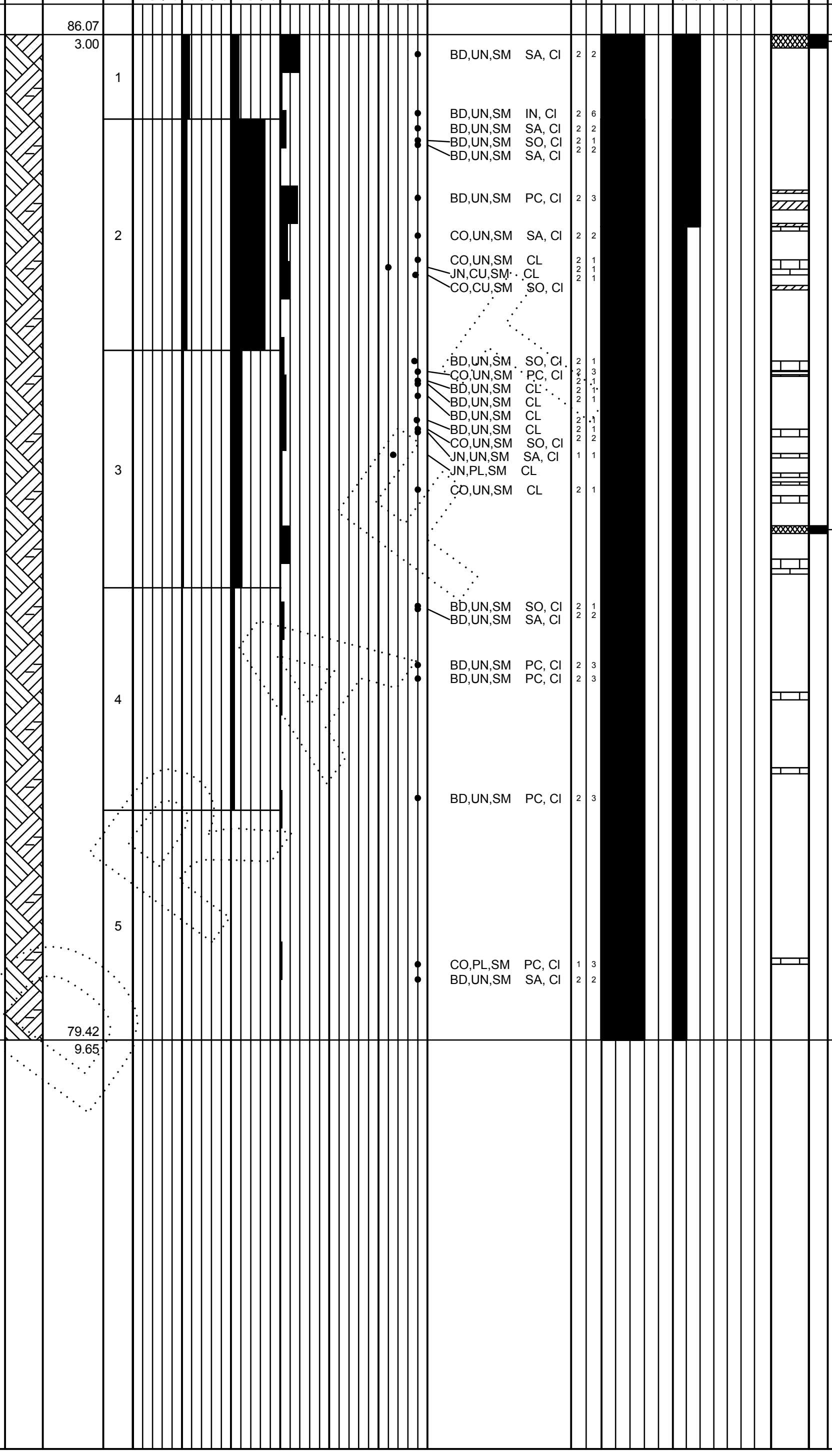
DRILLING CONTRACTOR: Geo-Environmental Drilling Inc.

DATUM: Geodetic

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

DEPTH SCALE METRES | DRILLING RECORD | DESCRIPTION | SYMBOLIC LOG | ELEV. DEPTH (m) | RUN No. | RECOVERY (TOTAL CORE % 80 60 40 20; SOLID CORE % 80 60 40 20) | R.Q.D. % 80 60 40 20 | FRACT. INDEX PER 0.25 m 5 10 15 20 | DISCONTINUITY DATA (B Angle 0 90 180 270; DIP w.r.t. CORE AXIS 0 30 60 90; TYPE AND SURFACE DESCRIPTION; Jr; Ja) | ROCK STRENGTH INDEX R4 R3 R2 R1 | WEATHERING INDEX W1 W2 W3 W4 W5 W6 | FEATURES | R0/R1 ZONES | NOTES WATER LEVELS INSTRUMENTATION

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. DEPTH (m)
		Continued from Record of Borehole EW-2	86.07
3	HQ Core	Slightly weathered to fresh, thinly laminated to medium bedded, grey, very fine to fine grained, faintly porous, weak SHALE (Georgian Bay Formation) with LIMESTONE interbeds	3.00
9.65		END OF DRILLHOLE	79.42 9.65



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

UCS=16.1 MPa

FEATURES LEGEND

▨	BROKEN CORE
▩	CLAY SEAM
▤	LIMESTONE
■	LOST CORE

DEPTH SCALE

1 : 50

GOLDER

LOGGED: JL

CHECKED: AKV

Start of Run No. 1 (3.00 m)

Start of Run No. 2 (3.56 m)

Start of Run No. 3 (5.09 m)

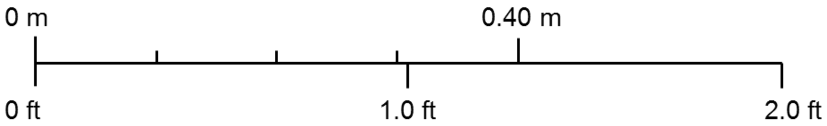
Box 1: 3.00 m to 6.66 m

Start of Run No. 4 (6.66 m)

Start of Run No. 5 (8.13 m)

End of Borehole (9.65 m)

Box 2: 6.66 m to 9.65 m



Scale

PROJECT

EAST-WEST ACTIVE TRANSPORT PEDESTRIAN BRIDGE ALONG CREDIT BRIDGE

TITLE

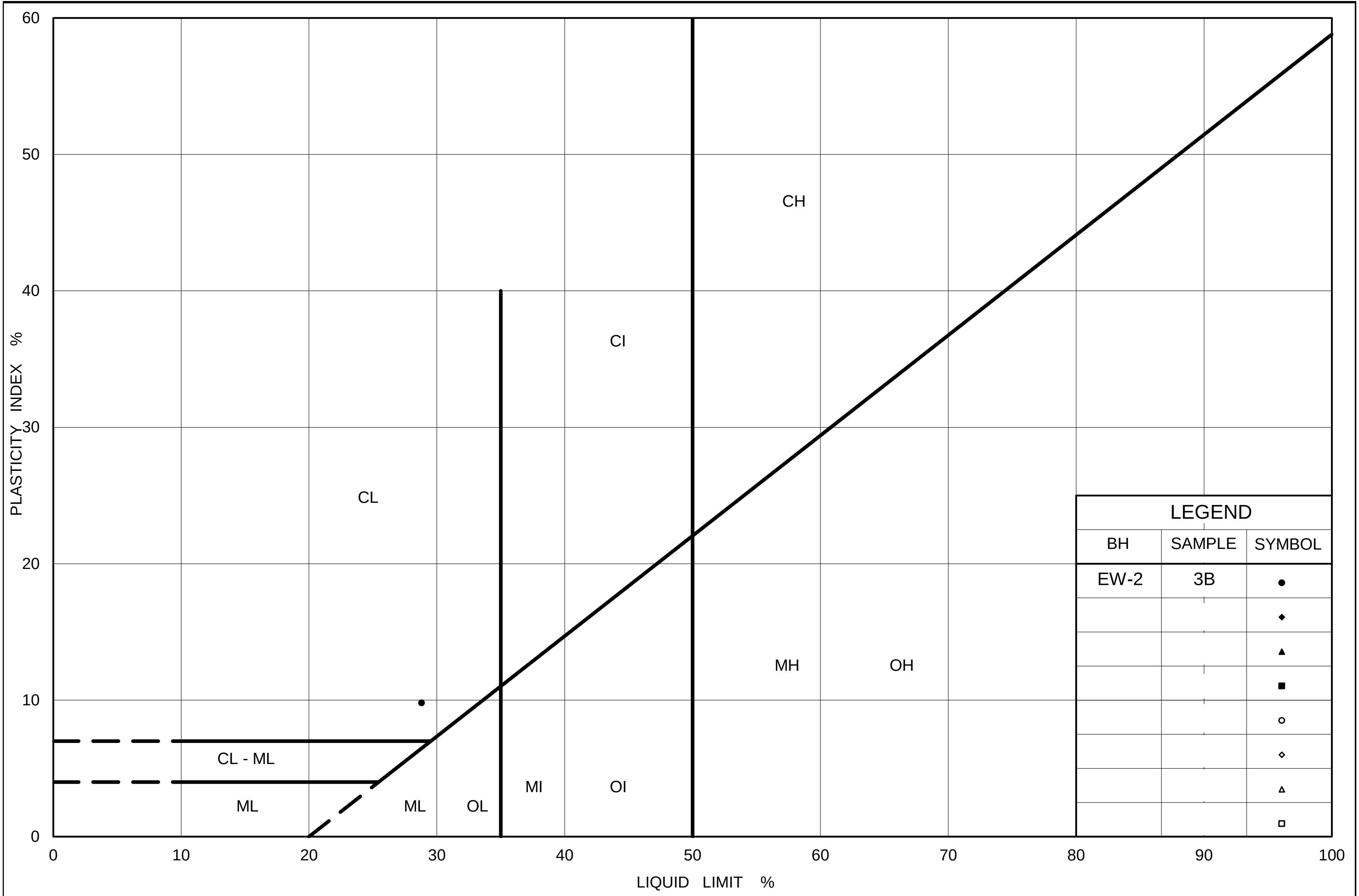
Bedrock Core Photographs
Borehole EW-2 (3.00 m to 9.65 m)

GOLDER

PROJECT No. 1662333			FILE No. ----		
DRAFT	SK	20180628	SCALE	NTS	VER. 1.
CADD	--		**FIGURE**		
CHECK	SMM				
REVIEW	JMAC	20181108			

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

Oct 75, FF-S-21



Ministry of Transportation

Ontario

PLASTICITY CHART
Gravelly CLAYEY SILT with Sand (TILL)

Figure No.

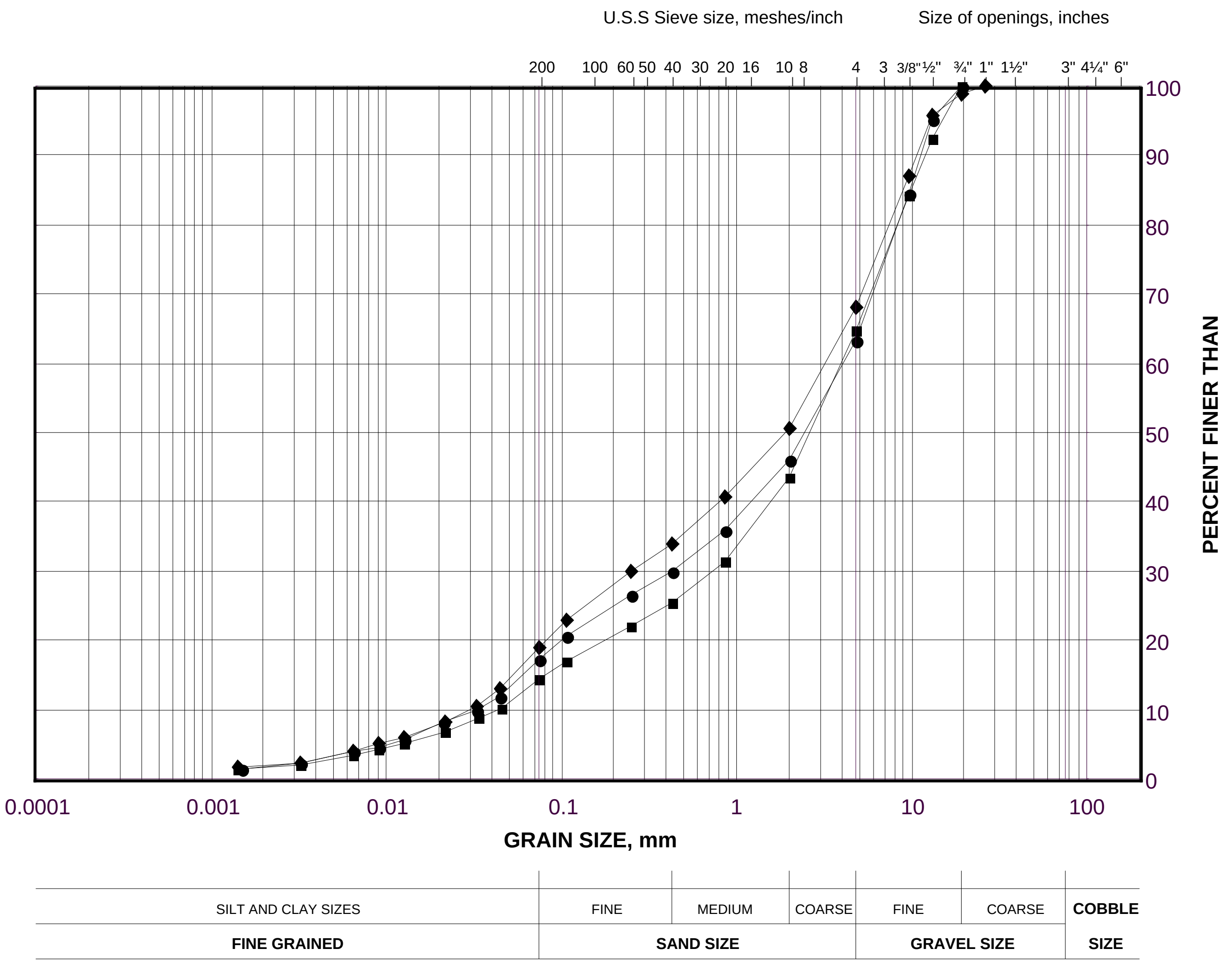
Project No. 1662333

Checked By: SMM

GRAIN SIZE DISTRIBUTION

Sand and Gravel (FILL)

FIGURE



LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	EW-1	2	87.5
■	EW-2	2A	88.2
◆	EW-1	4	85.9

Project Number: 1662333

Checked By: SMM

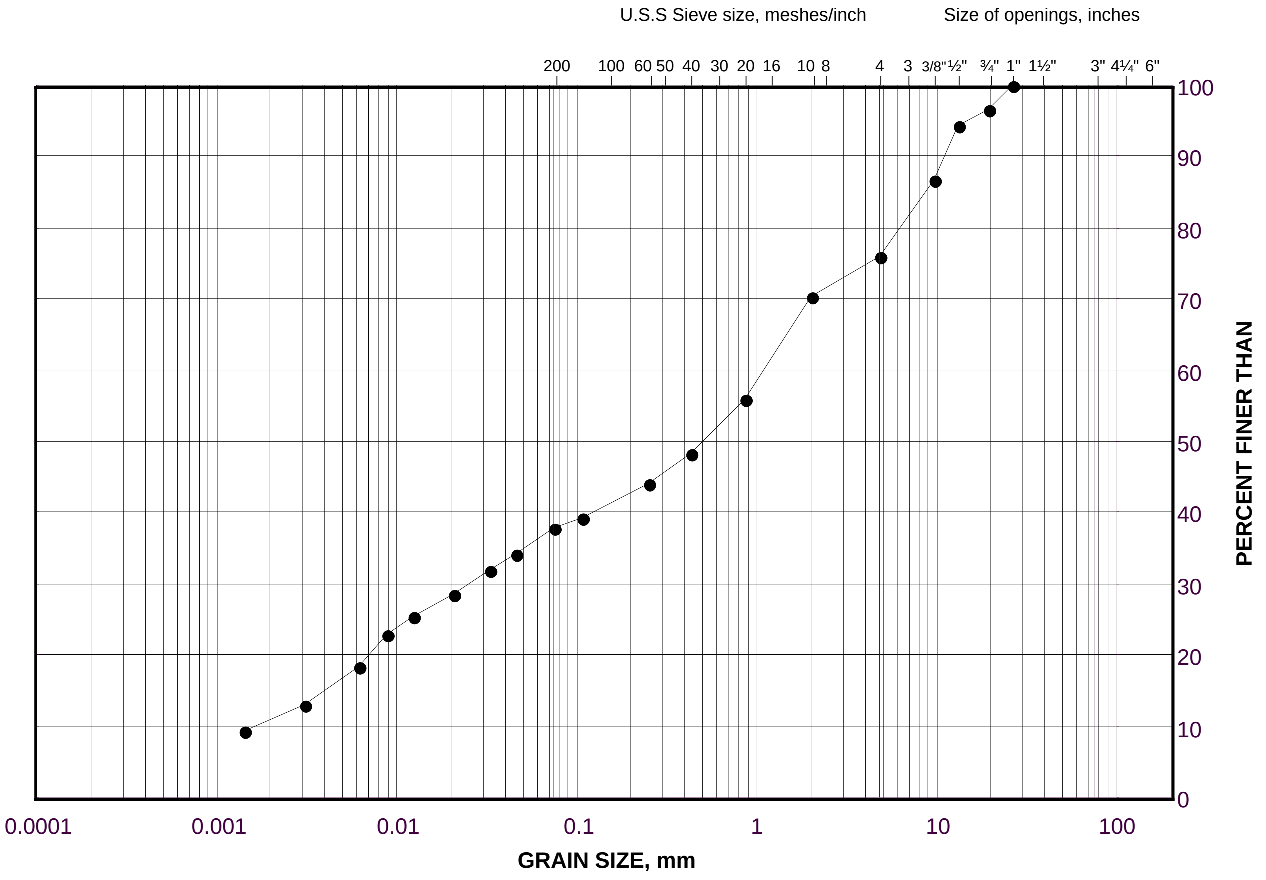
Golder Associates

Date: 25-Jun-18

GRAIN SIZE DISTRIBUTION

Gravelly Clayey Silt with Sand (TILL)

FIGURE



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	EW-2	3B	87.1

Project Number: 1662333

Checked By: SMM

Golder Associates

Date: 25-Jun-18

GEOMECHANICA

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

June 05, 2018

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

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

Dear Mr. Marmor:

On May 22, 2018 two (2) HQ-sized core samples were received by Geomechanica Inc. via drop-off by Golder personnel. These samples were identified as being from boreholes drilled as part of Golder project 1662333. A uniaxial compressive strength (UCS) specimen was prepared and tested from each of these samples (2 tests total).

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

Sincerely,

Bryan Tatone Ph.D., P. Eng.

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

Rock Laboratory Testing Results

A report submitted to:

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

Prepared by:

Bryan Tatone, PhD, PEng
Omid Mahabadi, PhD, PEng

Geomechanica Inc
#900-390 Bay St
Toronto ON
M5H 2Y2 Canada
Tel: +1-647-478-9767
info@geomechanica.com

June 5, 2018
Project number: 1662333

Abstract

This document summarizes the results of rock laboratory testing of 2 uniaxial compression tests. Results, including uniaxial compressive strength (UCS) and Young's modulus along with photographs of samples before and after testing are presented. Additional specimen information is included in an accompaning summary spreadsheet.

In this document:

1 Uniaxial Compressive Strength (UCS) testing

This report summarizes the results of 2 uniaxial compression tests. The testing was performed in Geomechanica's rock testing laboratory using a 150 ton (1.3 MN) Forney loading frame equipped with pressure-compensated control valve to maintain an axial displacement rate of approximately 0.15 mm/min for shale and inter-bedded limestone/shale and 0.075 mm/min for limestone samples (Figure 1).

The specimen preparation and testing procedure included the following:

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

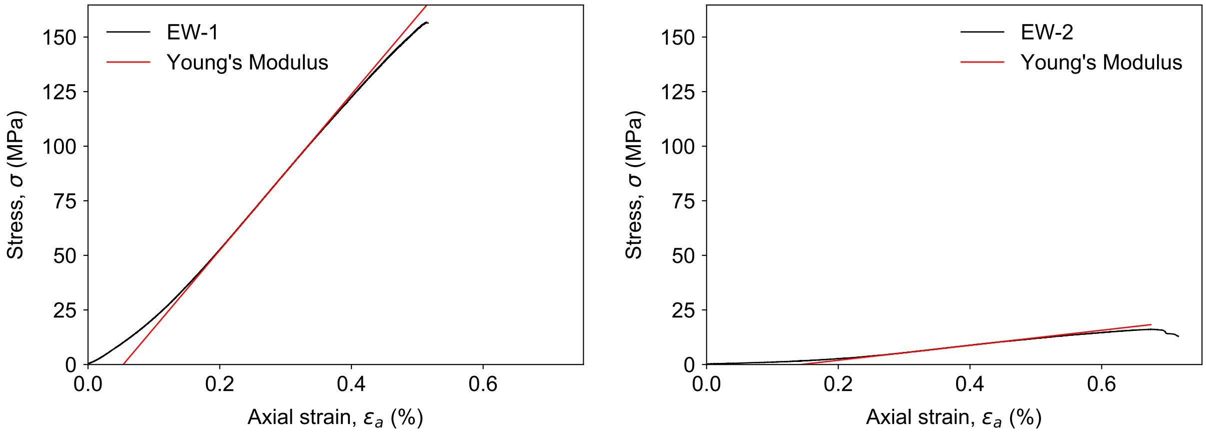
Figure 1: UCS test setup.

1.1 Results

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

Table 1: Summary of laboratory test results.

Sample	Depth (m)	Lithology description	Bulk density ρ (g/cm^3)	UCS (MPa)	Young's Modulus E (GPa)	Failure description
EW-1	7.26 - 7.43	Limestone	2.66	156.8	35.7	Axial splitting
EW-2	4.89 - 5.09	Inter-bedded shale & limestone	2.62	16.1	3.4	Axial splitting



(a) EW-1 - Limestone

(b) EW-2 - Inter-bedded shale/limestone

Figure 2: Measured stress-strain curves.

1.2 Specimen photographs

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

Figure 3: Photographs of specimens before and after testing.

Your Project #: 1662333
Site Location: QEW/CREDIT
Your C.O.C. #: 655260-05-01

Attention: Sandra McGaghran

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

Report Date: 2018/05/30
Report #: R5183725
Version: 1 - Final

CERTIFICATE OF ANALYSIS

MAXXAM JOB #: B8C0582
Received: 2018/05/22, 19:45

Sample Matrix: ROCK
Samples Received: 2

Analyses	Quantity	Date Extracted	Date Analyzed	Laboratory Method	Reference
Chloride (20:1 extract)	2	N/A	2018/05/29	CAM SOP-00463	EPA 325.2 m
Conductivity	2	N/A	2018/05/29	CAM SOP-00414	OMOE E3530 v1 m
pH CaCl2 EXTRACT	2	2018/05/29	2018/05/29	CAM SOP-00413	EPA 9045 D m
Resistivity of Soil	2	2018/05/23	2018/05/29	CAM SOP-00414	SM 23 2510 m
Sulphate (20:1 Extract)	2	N/A	2018/05/29	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 #: 1662333
Site Location: QEW/CREDIT
Your C.O.C. #: 655260-05-01

Attention: Sandra McGaghran

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

Report Date: 2018/05/30
Report #: R5183725
Version: 1 - Final

CERTIFICATE OF ANALYSIS

MAXXAM JOB #: B8C0582
Received: 2018/05/22, 19:45

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.

Maxxam Job #: B8C0582
Report Date: 2018/05/30

Golder Associates Ltd
Client Project #: 1662333
Site Location: QEW/CREDIT
Sampler Initials: JL

RESULTS OF ANALYSES OF ROCK

Maxxam ID		GTG829	GTG830			GTG830		
Sampling Date		2018/05/01	2018/05/02			2018/05/02		
COC Number		655260-05-01	655260-05-01			655260-05-01		
	UNITS	**EW1-R3-7.20 TO 7.26**	**EW2-R1-3.38 TO 3.51**	**RDL**	**QC Batch**	**EW2-R1-3.38 TO 3.51 Lab-Dup**	**RDL**	**QC Batch**
Calculated Parameters								
Resistivity	ohm-cm	1800	1300		5543388			
Inorganics								
Soluble (20:1) Chloride (Cl)	ug/g	110	130	20	5550731			
Conductivity	umho/cm	561	794	2	5552520	870	2	5552520
Available (CaCl2) pH	pH	8.28	8.00		5552937			
Soluble (20:1) Sulphate (SO4)	ug/g	250	630	20	5550732	630	20	5550732

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

TEST SUMMARY

Maxxam ID: GTG829
Sample ID: EW1-R3-7.20 TO 7.26
Matrix: ROCK

Collected: 2018/05/01
Shipped:
Received: 2018/05/22

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	5550731	N/A	2018/05/29	Deonarine Ramnarine
Conductivity	AT	5552520	N/A	2018/05/29	Tahir Anwar
pH CaCl2 EXTRACT	AT	5552937	2018/05/29	2018/05/29	Gnana Thomas
Resistivity of Soil		5543388	2018/05/29	2018/05/29	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	5550732	N/A	2018/05/29	Alina Dobreanu

Maxxam ID: GTG830
Sample ID: EW2-R1-3.38 TO 3.51
Matrix: ROCK

Collected: 2018/05/02
Shipped:
Received: 2018/05/22

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	5550731	N/A	2018/05/29	Deonarine Ramnarine
Conductivity	AT	5552520	N/A	2018/05/29	Tahir Anwar
pH CaCl2 EXTRACT	AT	5552937	2018/05/29	2018/05/29	Gnana Thomas
Resistivity of Soil		5543388	2018/05/29	2018/05/29	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	5550732	N/A	2018/05/29	Alina Dobreanu

Maxxam ID: GTG830 Dup
Sample ID: EW2-R1-3.38 TO 3.51
Matrix: ROCK

Collected: 2018/05/02
Shipped:
Received: 2018/05/22

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Conductivity	AT	5552520	N/A	2018/05/29	Tahir Anwar
Sulphate (20:1 Extract)	KONE/EC	5550732	N/A	2018/05/29	Alina Dobreanu

GENERAL COMMENTS

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

Package 1	8.7°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
5550731	Soluble (20:1) Chloride (Cl)	2018/05/29	NC	70 - 130	108	70 - 130	<20	ug/g	3.1	35
5550732	Soluble (20:1) Sulphate (SO4)	2018/05/29	NC	70 - 130	99	70 - 130	<20	ug/g	0.89	35
5552520	Conductivity	2018/05/29			100	90 - 110	<2	umho/cm	9.2	10
5552937	Available (CaCl2) pH	2018/05/29			100	97 - 103			0.39	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 spike amount was too small to permit a reliable recovery calculation (matrix spike concentration was less than 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
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 1 of 1

IMMEDIATE

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: Fax:
Email: Sandra-Mcgaghran@golder.com

PROJECT INFORMATION:

Quotation #: B80683
P.O. #:
Project: ~~1674130 WO#0~~ 1662333
Project Name: QEW / CREDIT
Site #:
Sampled By: JL

Laboratory Use Only:

Maxxam Job #:	Bottle Order #:
	655260
COC #:	Project Manager:
C#655260-05-01	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~~ Date	Matrix	Field Filtered (please circle):	Metals / Hg / Cr VI	Standard Corrosivity pkg (No Sulphide and Redox Potential)										# of Bottles	Comments
1		EW1-R3-7.20 to 7.26		2018/05/01	ROCK			X										1	sample in Bag
2		EW2-R1-3.38 to 3.51		2018/05/02	ROCK			X										1	Sample in Bag
3																			
4																			
5																			
6																			
7																			
8																			
9																			
10																			

22-May-18 19:45
Ema Gitej
B8C0582
GK1 ENV-632

* RELINQUISHED BY: (Signature/Print)	Date: (YY/MM/DD)	Time	RECEIVED BY: (Signature/Print)	Date: (YY/MM/DD)	Time	# jars used and not submitted
Kati [illegible]	18/05/22	7:45PM	[illegible]	2018/05/22	19:45	

Laboratory Use Only				
Time Sensitive	Temperature (°C) on Recei	Custody Seal	Yes	No
	7/10/9	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

"Geotechnical Investigation, Proposed Pipeline Installations, Crossings of Credit River Project No. 160950937, Document No. TAJ-C-GEO-002, dated December 18, 2018, prepared by Stantec Consulting Ltd.

Record of Borehole BH3

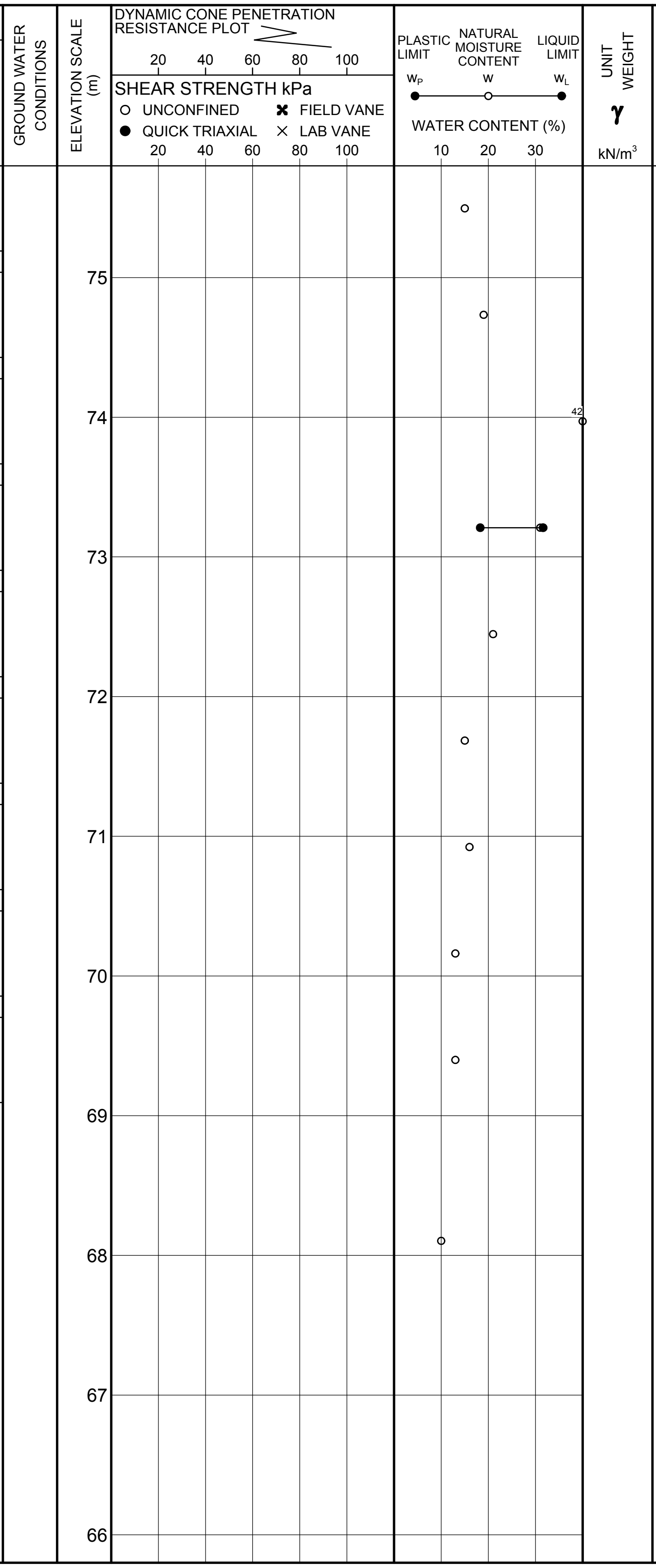
Stantec

RECORD OF BOREHOLE No BH3

1 OF 4 **METRIC**

PROJECT # 160950937	PROJECT ESA Drilling Credit River & QEW		
W.P.	LOCATION Credit River and QEW, Mississauga, ON	N: 4 823 506 E: 612 222	ORIGINATED BY RB
DIST HWY	BOREHOLE TYPE Hollow Stem Auger		COMPILED BY DR
DATUM Geodetic	DATE November 13, 2017		CHECKED BY JJB

ELEV / DEPTH (m)	DESCRIPTION (SOIL PROFILE)	NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
75.8	Grass				
	Sandy CLAY (CL) Brown Very soft to soft - moist to wet	1	SS	4	
		2	SS	4	
		3	SS	3	
73.5 2.3	Grey - wet	4	SS	0	1 30 48 21
72.8 3.0	Silty Gravel with Sand (GM) Grey Compact - trace clay - wet	5	SS	14	
		6	SS	16	
		7	SS	29	
		8	SS	16	40 37 14 9
		9	SS	14	
68.6 7.2	Inferred BEDROCK (Georgian Bay Formation) Grey - highly to completely weathered shale (soil-like consistency)	1	HQ	100/ 100/	TCR=89% SCR=56% RQD=0%
67.8 8.0	SHALE with limestone interbedding (Georgian Bay Formation) Dark grey shale with light grey limestone interbedding - very poor quality - moderately weathered - occassional clay seams up to 25mm	2	HQ	100/	TCR=95% SCR=86% RQD=19%
65.8					



Continued Next Page

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

Stantec

RECORD OF BOREHOLE No BH3

2 OF 4

METRIC

PROJECT # 160950937
PROJECT ESA Drilling Credit River & QEW
W.P.
LOCATION Credit River and QEW, Mississauga, ON N: 4 823 506 E: 612 222
ORIGINATED BY RB
DIST HWY
BOREHOLE TYPE Hollow Stem Auger
COMPILED BY DR
DATUM Geodetic
DATE November 13, 2017
CHECKED BY JJB

SOIL PROFILE: ELEV DEPTH (m)	SOIL PROFILE: DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: "N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE (m)	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED ✖ FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
10.0	SHALE with limestone interbedding (Georgian Bay Formation) Dark grey shale with light grey limestone interbedding - poor to good quality - freshly to moderately weathered - occassional clay seams up to 25mm		3	HQ	100/						TCR=100% SCR=91% RQD=48%
							65				
			4	HQ	100/		64				TCR=99% SCR=96% RQD=84%
							63				
			5	HQ	100/						TCR=98% SCR=98% RQD=80%
							62				
			6	HQ	100/		61				TCR=95% SCR=82% RQD=57%
							60				
			7	HQ	100/						TCR=99% SCR=93% RQD=25%
							59				
			8	HQ	100/		58				TCR=93% SCR=78% RQD=49%
							57				
			9	HQ	100/						TCR=99% SCR=98% RQD=77%
							56				
55.8											

Continued Next Page

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

Stantec

RECORD OF BOREHOLE No BH3

METRIC

PROJECT # 160950937
PROJECT ESA Drilling Credit River & QEW
W.P.
LOCATION Credit River and QEW, Mississauga, ON N: 4 823 506 E: 612 222
ORIGINATED BY RB
DIST HWY
BOREHOLE TYPE Hollow Stem Auger
COMPILED BY DR
DATUM Geodetic
DATE November 13, 2017
CHECKED BY JJB

SOIL PROFILE			SAMPLES					DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED ✖ FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE (m)				
20.0	SHALE with limestone interbedding (Georgian Bay Formation) Dark grey shale with light grey limestone interbedding - very poor to excellent quality - freshly to slightly weathered - occassional clay seams										
							55				
			10	HQ	100/						TCR=98% SCR=97% RQD=82%
							54				
			11	HQ	100/						TCR=100% SCR=98% RQD=21%
							53				
							52				
			12	HQ	100/						TCR=100% SCR=100% RQD=91%
							51				
			13	HQ	100/						TCR=100% SCR=95% RQD=68%
							50				
							49				
			14	HQ	100/						TCR=100% SCR=100% RQD=93%
							48				
			15	HQ	100/						TCR=100% SCR=100% RQD=94%
							47				
							46				
45.8											

Continued Next Page

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

Stantec

RECORD OF BOREHOLE No BH3

4 OF 4

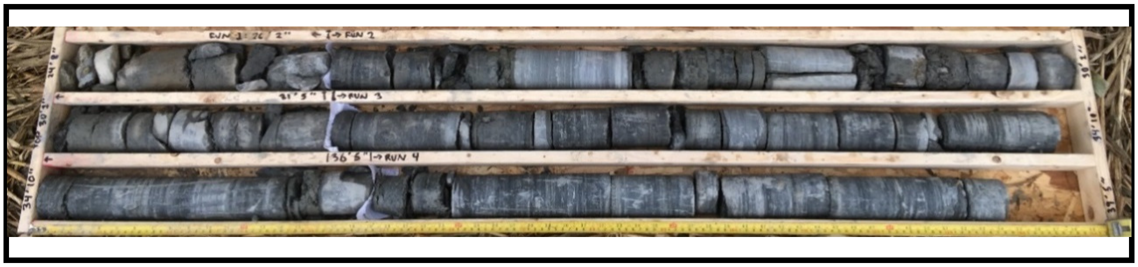
METRIC

PROJECT # 160950937
PROJECT ESA Drilling Credit River & QEW
W.P.
LOCATION Credit River and QEW, Mississauga, ON N: 4 823 506 E: 612 222
ORIGINATED BY RB
DIST
HWY
BOREHOLE TYPE Hollow Stem Auger
COMPILED BY DR
DATUM Geodetic
DATE November 13, 2017
CHECKED BY JJB

SOIL PROFILE			SAMPLES								
ELEV DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE (m)	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED ✖ FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m^3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
30.0 44.9	SHALE with limestone interbedding (Georgian Bay Formation) Dark grey shale with light grey limestone interbedding - good quality - freshly weathered		16	HQ	100/		45				TCR=99% SCR=96% RQD=77%
30.9	END OF BOREHOLE at approximately 30.9 m below existing grade. Groundwater level not measured in open borehole due to the introduction of water for rock coring.										

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

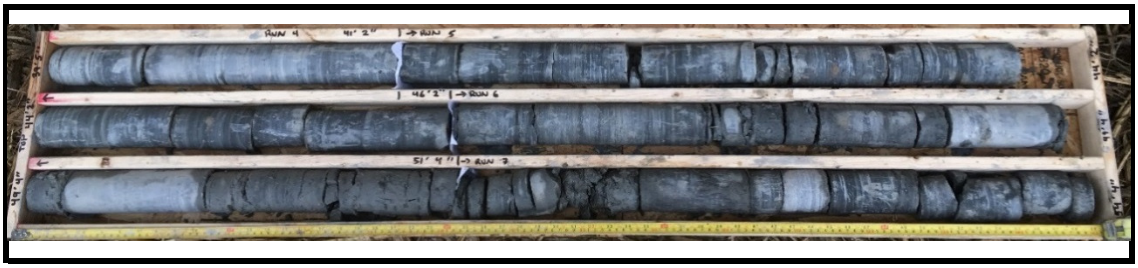
Project:	ESA Drilling Credit River & QEW
Project Number:	160950937
Location:	Mississauga, ON
Borehole:	BH3
Depth (m):	7.52 - 30.94



7.52m — RUN 1 → | RUN 2 — 9.17m

9.17m — RUN 2 → | RUN 3 — 10.62m

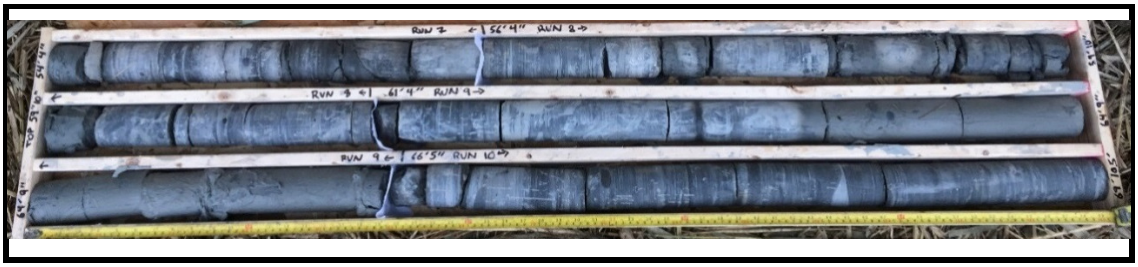
10.62m — RUN 3 → | RUN 4 — 12.01m



12.01m — RUN 4 → | RUN 5 — 13.46m

13.46m — RUN 5 → | RUN 6 — 15.04m

15.04m — RUN 6 → | RUN 7 — 16.56m

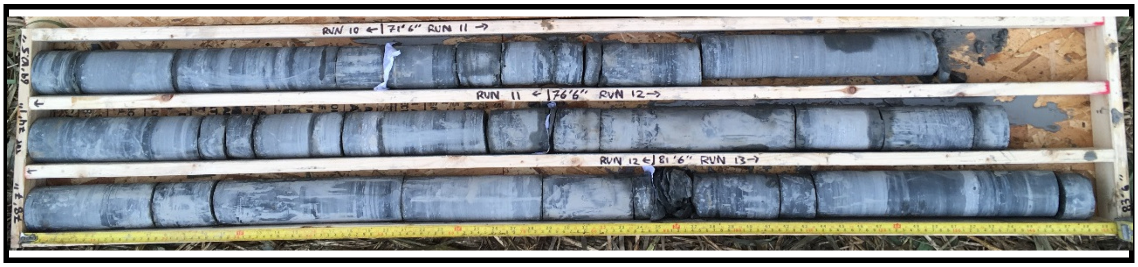


16.56m — RUN 7 → | RUN 8 — 18.24m

18.24m — RUN 8 → | RUN 9 — 19.74m

19.74m — RUN 9 → | RUN 10 — 21.30m

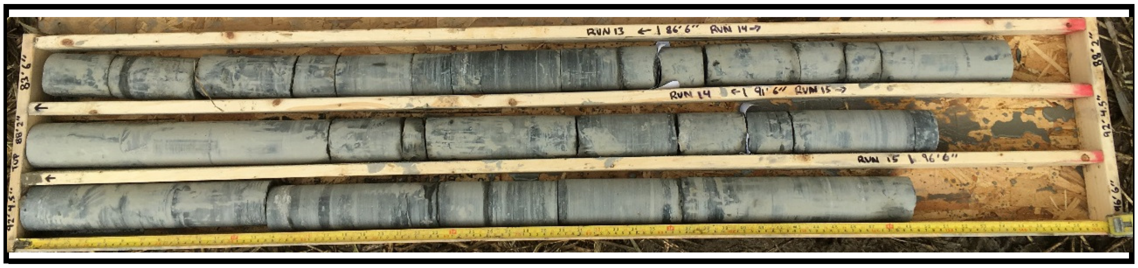
Project:	ESA Drilling Credit River & QEW		
Project Number:	160950937		
Location:	Mississauga, ON		
Borehole:	BH3	**Depth (m):**	7.52 - 30.94



21.30m — RUN 10 → | RUN 11 — 22.58m

22.58m — RUN 11 → | RUN 12 — 23.95m

23.95m — RUN 12 → | RUN 13 — 25.45m

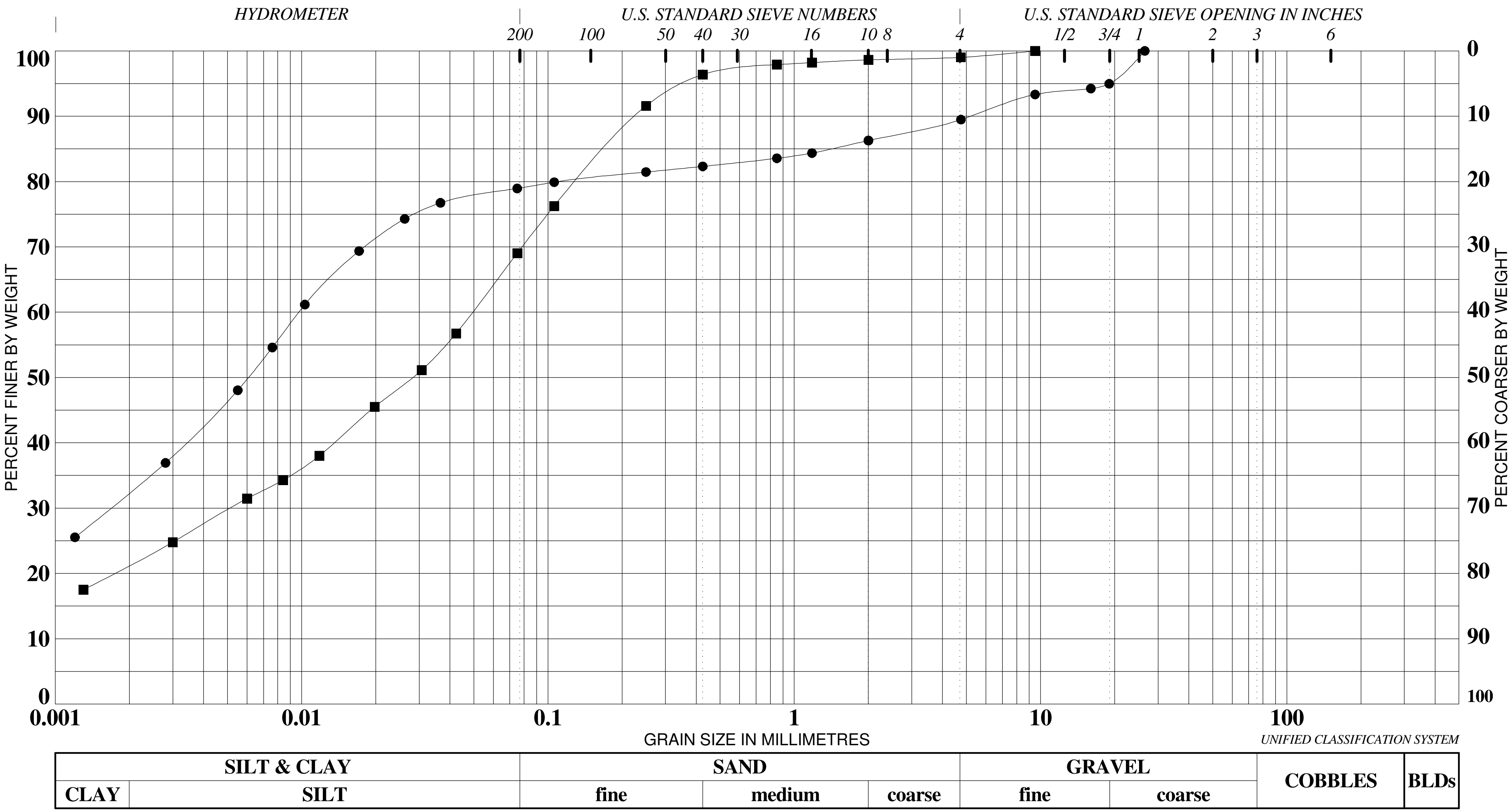


25.45m — RUN 13 → | RUN 14 — 26.87m

26.87m — RUN 14 → | RUN 15 — 28.16m

28.16 — RUN 15 → 29.42m

29.42m — RUN 16 → 30.94m



Sample		Depth (m)	Description	W%	W_L	W_P	I_P	%Gravel	%Sand	%Silt	%Clay
●	BH2	1.1	Clay with gravel (CL)	12	37	22	15	11	10	47	32
■	BH3	2.6	Sandy CLAY (CL)	31	32	18	14	1	30	48	21

Stantec

Project: ESA Drilling Credit River & QEW

Location: Credit River and QEW, Mississauga, ON

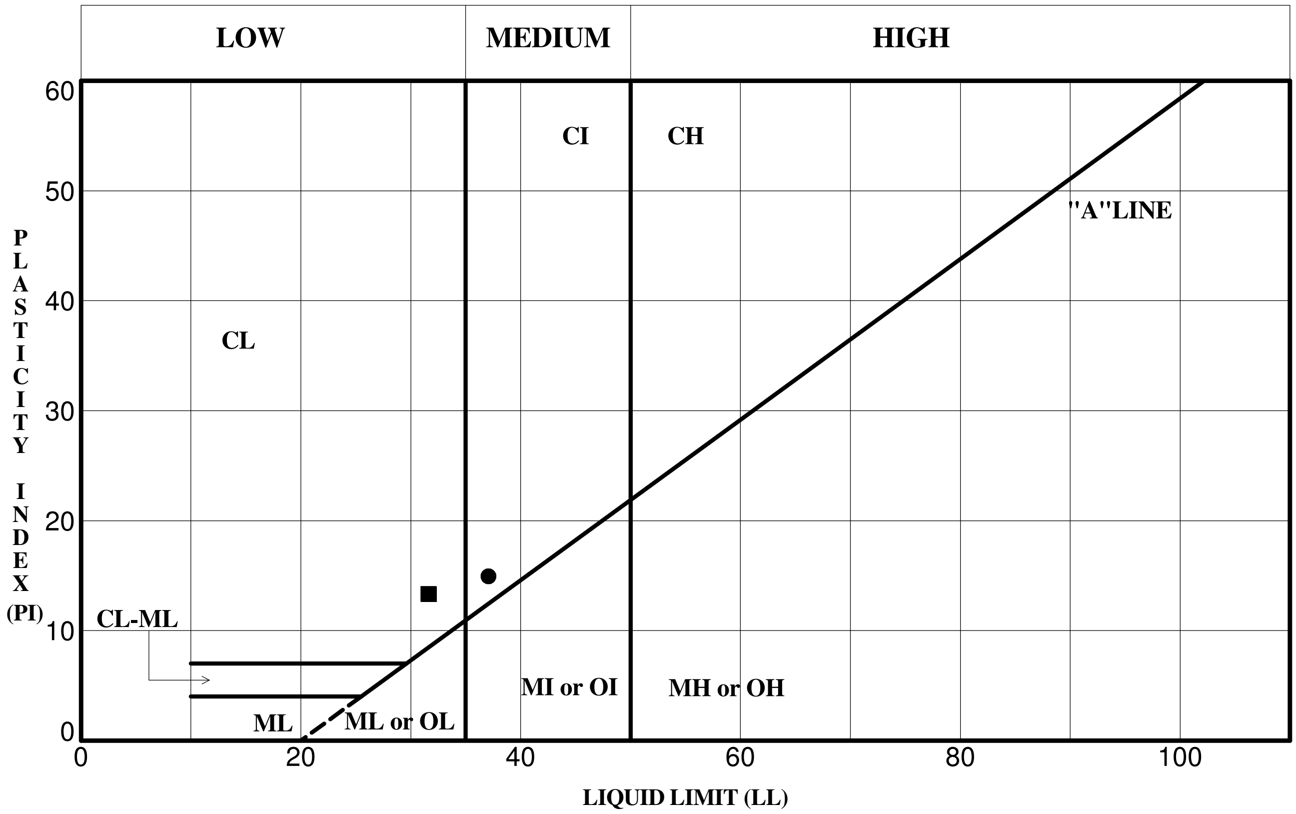
Project No.: 160950937

GRADATION CURVE (ASTM D422)

Figure: 1

Remarks:

PLASTICITY CHART



Specimen		Depth (m)	LL	PL	PI	Fines	W%	Classification
●	BH2	1.1	37	22	15	79	12	Clay with gravel (CL)
■	BH3	2.6	32	18	14	69	31	Sandy CLAY (CL)

Stantec

Project: ESA Drilling Credit River & QEW

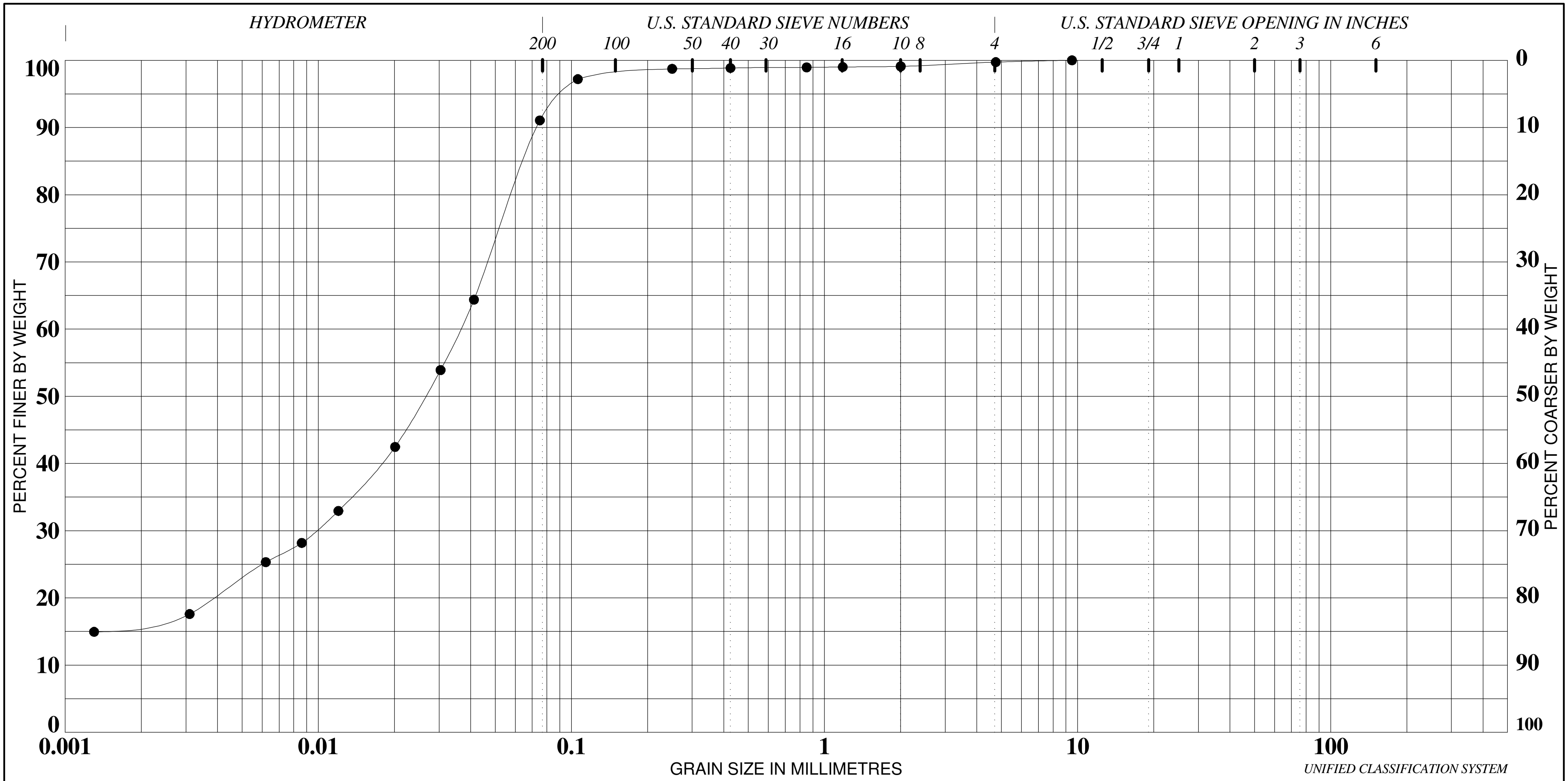
Location: Credit River and QEW, Mississauga, ON

Project No.: 160950937

ATTERBERG LIMITS
(ASTM D4318)

Figure: 2

Remarks:



SILT & CLAY		SAND			GRAVEL		COBBLES	BLDs
CLAY	SILT	fine	medium	coarse	fine	coarse		

Sample	Depth (m)	Description	W%	W_L	W_P	I_P	%Gravel	%Sand	%Silt	%Clay
● BH4	3.4	SILT (ML)	16				0	9	75	16

Stantec

Project: ESA Drilling Credit River & QEW

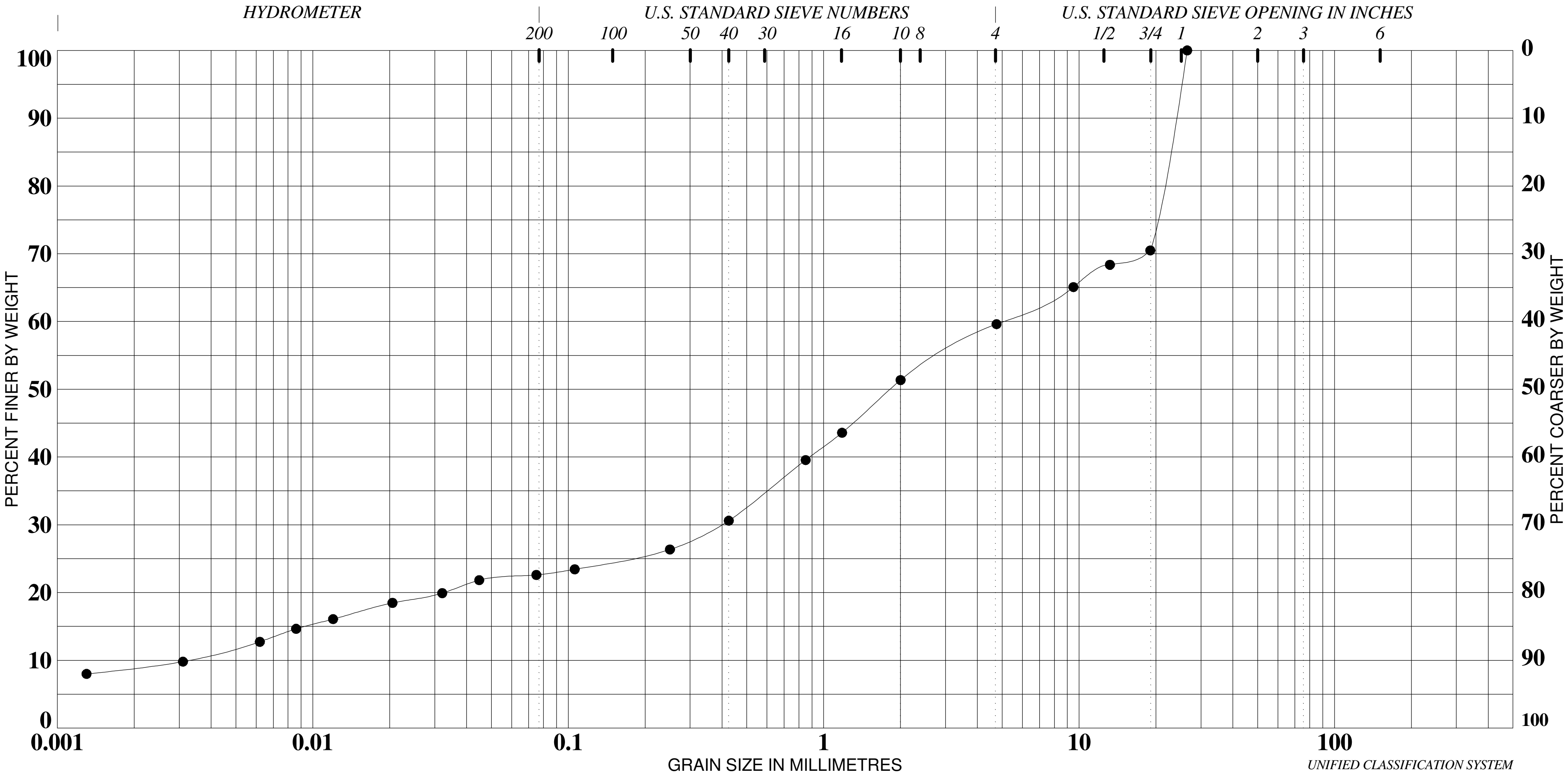
Location: Credit River and QEW, Mississauga, ON

Project No.: 160950937

GRADATION CURVE (ASTM D422)

Figure: 3

Remarks:



SILT & CLAY		SAND			GRAVEL		COBBLES	BLDs
CLAY	SILT	fine	medium	coarse	fine	coarse		

	Sample	Depth (m)	Description	W%	W_L	W_P	I_P	%Gravel	%Sand	%Silt	%Clay
●	BH3	5.6	Silty Gravel with Sand (GM)	13				40	37	14	9

Project: ESA Drilling Credit River & QEW

Location: Credit River and QEW, Mississauga, ON

Project No.: 160950937

GRADATION CURVE (ASTM D422)

Figure: 4

Remarks:

Foundation Investigation and Design Report, Preliminary Design and Environmental Assessment, QEW Bridge Twinning Over Credit River, Mississauga, Ontario” File No. 19-1351-174, dated May 18, 2012 (GEOCRES 30M12-341)

Boreholes 11-01 and 11-02

Foundation Investigation and Design Report, Construction Access Road for Bridge Rehabilitation, QEW Bridge over Credit River, Mississauga, Ontario” File No. 19-92-92-174, dated April 8, 2011, prepared by Thurber Engineering Ltd. (GEOCRES 30M12-324).

Boreholes 10-03A, 10-03B and 10-04.

SYMBOLS, ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES

1. TEXTURAL CLASSIFICATION OF SOILS

CLASSIFICATION	PARTICLE SIZE	VISUAL IDENTIFICATION
Boulders	Greater than 200mm	same
Cobbles	75 to 200mm	same
Gravel	4.75 to 75mm	5 to 75mm
Sand	0.075 to 4.75mm	Not visible particles to 5mm
Silt	0.002 to 0.075mm	Non-plastic particles, not visible to the naked eye
Clay	Less than 0.002mm	Plastic particles, not visible to the naked eye

2. COARSE GRAIN SOIL DESCRIPTION (50% greater than 0.075mm)

TERMINOLOGY	PROPORTION
Trace or Occasional	Less than 10%
Some	10 to 20%
Adjective (e.g. silty or sandy)	20 to 35%
And (e.g. sand and gravel)	35 to 50%

3. TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

DESCRIPTIVE TERM	UNDRAINED SHEAR STRENGTH (kPa)	APPROXIMATE SPT(1) 'N' VALUE
Very Soft	12 or less	Less than 2
Soft	12 to 25	2 to 4
Firm	25 to 50	4 to 8
Stiff	50 to 100	8 to 15
Very Stiff	100 to 200	15 to 30
Hard	Greater than 200	Greater than 30

NOTE: Hierarchy of Soil Strength Prediction

1) Laboratory Triaxial Testing
2) Field Insitu Vane Testing
3) Laboratory Vane Testing
4) SPT value
5) Pocket Penetrometer

4. TERMS DESCRIBING DENSITY (COHESIONLESS SOILS ONLY)

DESCRIPTIVE TERM	SPT "N" VALUE
Very Loose	Less than 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	Greater than 50

5. LEGEND FOR RECORDS OF BOREHOLES

SYMBOLS AND ABBREVIATIONS FOR SAMPLE TYPE

SS Split Spoon Sample	WS Wash Sample	AS Auger (Grab) Sample
TW Thin Wall Shelby Tube Sample	TP Thin Wall Piston Sample	
PH Sampler Advanced by Hydraulic Pressure	PM Sampler Advanced by Manual Pressure	
WH Sampler Advanced by Self Static Weight	RC Rock Core	SC Soil Core

$$\text{Sensitivity} = \frac{\text{Undisturbed Shear Strength}}{\text{Remoulded Shear Strength}}$$

▼ Water Level

C_{pen} Shear Strength Determination by Pocket Penetrometer

(1) SPT 'N' Value Standard Penetration Test 'N' Value – refers to the number of blows from a 63.5kg hammer free falling a height of 0.76m to advance a standard 50 mm outside diameter split spoon sampler for 0.3 m depth into undisturbed ground.

(2) DCPT Dynamic Cone Penetration Test – Continuous penetration of a 50 mm outside diameter, 60° conical steel point attached to "A" size rods driven by a 63.5 kg hammer free falling a height of 0.76 m. The resistance to cone penetration is the number of hammer blows required for each 0.3 m advance of the conical point into undisturbed ground.

UNIFIED SOILS CLASSIFICATION

MAJOR DIVISIONS		GROUP SYMBOL	TYPICAL DESCRIPTION
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	GW	Well-graded gravels or gravel-sand mixtures, little or no fines.
		GP	Poorly-graded gravels or gravel-sand mixtures, little or no fines.
		GM	Silty gravels, gravel-sand-silt mixtures.
		GC	Clayey gravels, gravel-sand-clay mixtures.
	SAND AND SANDY SOILS	SW	Well-graded sands or gravelly sands, little or no fines.
		SP	Poorly-graded sands or gravelly sands, little or no fines.
		SM	Silty sands, sand-silt mixtures.
		SC	Clayey sands, sand-clay mixtures.
FINE GRAINED SOILS	SILTS AND CLAYS $W_L < 50\%$	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays. ($W_L < 30\%$).
		CI	Inorganic clays of medium plasticity, silty clays. ($30\% < W_L < 50\%$).
		OL	Organic silts and organic silty-clays of low plasticity.
	SILTS AND CLAYS $W_L > 50\%$	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
		CH	Inorganic clays of high plasticity, fat clays.
		OH	Organic clays of medium to high plasticity, organic silts.
HIGHLY ORGANIC SOILS		Pt	Peat and other highly organic soils.
CLAY SHALE			
SANDSTONE			
SILTSTONE			
CLAYSTONE			
COAL			

EXPLANATION OF ROCK LOGGING TERMS

ROCK WEATHERING CLASSIFICATION

Fresh (FR)	No visible signs of weathering.
Fresh Jointed (FJ)	Weathering limited to the surface of major discontinuities.
Slightly Weathered (SW)	Penetrative weathering developed on open discontinuity surfaces, but only slight weathering of rock material.
Moderately Weathered (MW)	Weathering extends throughout the rock mass, but the rock material is not friable.
Highly Weathered (HW)	Weathering extends throughout the rock mass and the rock is partly friable.
Completely Weathered (CW)	Rock is wholly decomposed and in a friable condition, but the rock texture and structure are preserved.

SYMBOLS

▨	CLAYSTONE
▤	SILTSTONE
░	SANDSTONE
■	COAL
▩	Bedrock (general)

DISCONTINUITY SPACING

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

TERMS

Total Core Recovery: (TCR)	Core recovered as a percentage of total core run length.
Solid Core Recovery: (SCR)	Percent Ratio of solid core of full cylindrical shape recovered. Expressed with respect to the total length of core run.
Rock Quality Designation: (RQD)	Total length of sound core recovered in pieces 0.1m in length or larger as a percentage of total core run length.
Uniaxial Compressive Strength (UCS)	Axial stress required to break the specimen
Fracture Index: (FI)	Frequency of natural fractures per 0.3m of core run.

STRENGTH CLASSIFICATION

Rock Strength	**Approximate Uniaxial Compressive Strength**		**Field Estimation of Hardness***
	(MPa)	**(psi)**	
Extremely Strong	Greater than 250	Greater than 36,000	Specimen can only be chipped with a geological hammer
Very Strong	100-250	15,000 to 36,000	Requires many blows of geological hammer to break
Strong	50-100	7,500 to 15,000	Requires more than one blow of geological hammer to break
Medium Strong	25.0 to 50.0	3,500 to 7,500	Breaks under single blow of geological hammer.
Weak	5.0 to 25.0	750 to 3,500	Can be peeled by a pocket knife with difficulty
Very Weak	1.0 to 5.0	150 to 750	Can be peeled by a pocket knife, crumbles under firm blows of geological pick.
Extremely Weak (Rock)	0.25 to 1.0	35 to 150	Indented by thumbnail

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 11-01

1 OF 1

METRIC

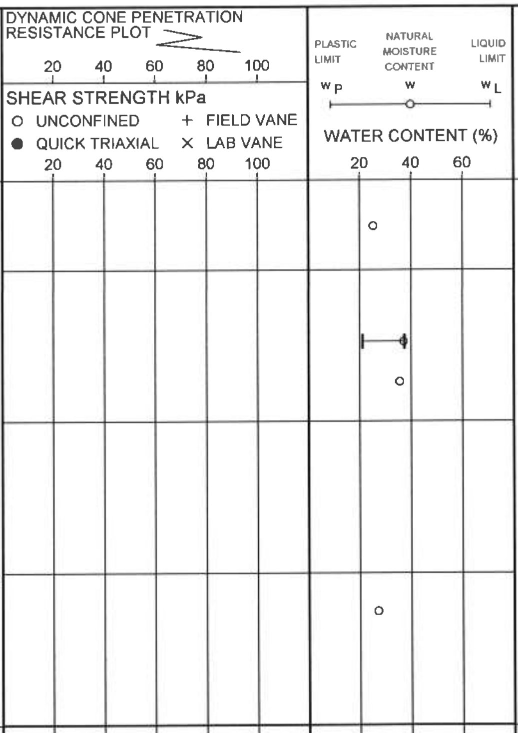
W.P. W.O. 08-20008
LOCATION N 4 823 959.1 E 295 814.8 QEW Bridge at Credit River
ORIGINATED BY SLD
HWY QEW
BOREHOLE TYPE Tripod (Hilti) - Wash Boring and Coring
COMPILED BY AN
DATUM Geodetic
DATE 2011.05.30 - 2011.06.02
CHECKED BY SKP

ELEV DEPTH	DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
94.6						
0.0	**TOPSOIL**, with roots and organics: (200mm)					
0.2	Silty **CLAY**, trace sand Very Stiff Grey Moist	1	SS	4	94	
		2	SS	21		2 15 49 34
	trace shale fragments	3	SS	40/ 0.075		
		1	RUN		93	
		2	RUN			
		3	RUN			
		4	RUN		92	
		5	RUN			
		4	SS	50/ 0.075		
91.3 3.3	**SHALE**, weathered, grey					
91.0 3.6	END OF SPT SAMPLING TO 3.6m AND START CORING. FOR ROCK DETAILS PLEASE REFER TO 11-01R.				91	

Piezometer installation consists of 38mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen.

WATER LEVEL READINGS:

DATE	DEPTH (m)	ELEV. (m)
Sep.30/11	3.2	91.4



UNIT WEIGHT γ kN/m³

+ 3, × 3: Numbers refer to Sensitivity

20 15 5 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE 11-01R

PROJECT : QEW Mississauga Rd. Overpass
LOCATION : Mississauga, ON
STARTED : May 30, 2011
COMPLETED : June 2, 2011

INCLINATION: Vertical AZIMUTH:

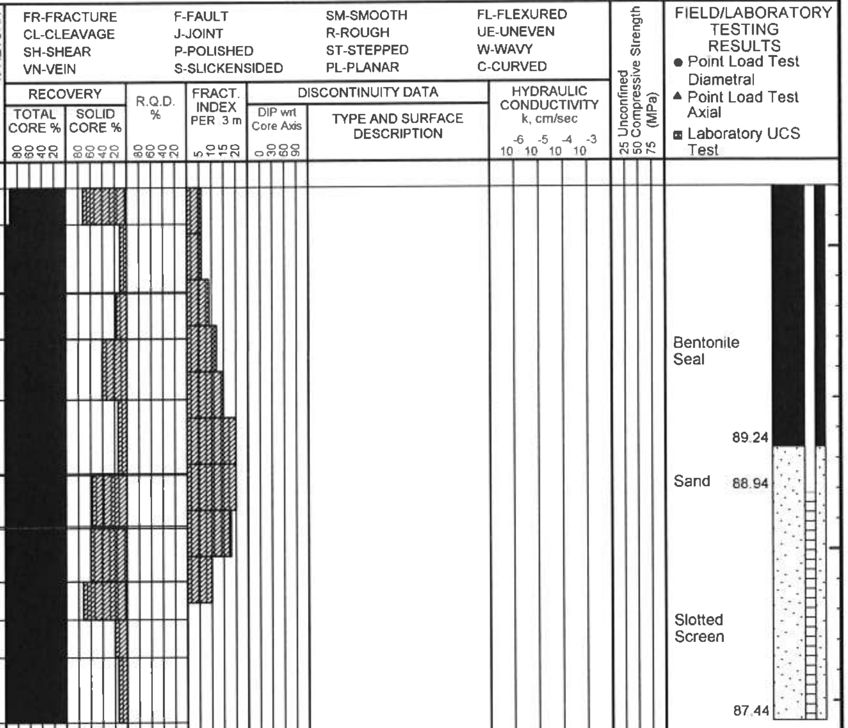
Project No. W.O. 08-20008

SHEET 1 OF 1
DATUM Geodetic

FR-FRACTURE
CL-CLEAVAGE
SH-SHEAR
VN-VEIN
F-FAULT
J-JOINT
P-POLISHED
S-SLICKENSIDED
SM-SMOOTH
R-ROUGH
ST-STEPPED
PL-PLANAR
FL-FLEXURED
UE-UNEVEN
W-WAVY
C-CURVED

FIELD/LABORATORY TESTING RESULTS
● Point Load Test Diametral
▲ Point Load Test Axial
⊠ Laboratory UCS Test

DEPTH SCALE (metres)	DRILLING RECORD	DESCRIPTION	ELEV. / DEPTH (m)	RUN No.	FLUSH COLOUR % RETURN
			91.0		
	RUN	**SHALE**, slightly to moderately weathered, fine grained, thinly bedded, grey, with limestone interbeds	3.6	1	100
4	RUN	20mm clay seam at 3.7m		2	100
	RUN	30mm clay seam at 3.76m		3	90
	RUN	10mm clay seams at 3.8m, 3.9m, 4.0m and 4.2m		4	100
	RUN	Frequent clay seams (20mm to 100mm) from 4.3m to 5.0m 350mm clay seam at 5.0m 200mm rubble zone at 5.4m		5	100
	RUN	Clay seams (25mm to 75mm) at 5.5m, 5.6m, 5.7m, 5.9m, 6.0m		6	100
6	RUN			7	100
	RUN	Clay seams (25mm to 75mm) at 6.2m, 6.3m, 6.4m, 6.6m, 6.7m and 6.8m		8	100
	RUN			9	100
	RUN		87.5	10	100
		END OF BOREHOLE AT 7.1m	7.1		
8					
10					
12					



GROUNDWATER ELEVATIONS

∇ SHALLOW/SINGLE INSTALLATION
WATER LEVEL (date)

▼ DEEP/DUAL INSTALLATION
WATER LEVEL (date)

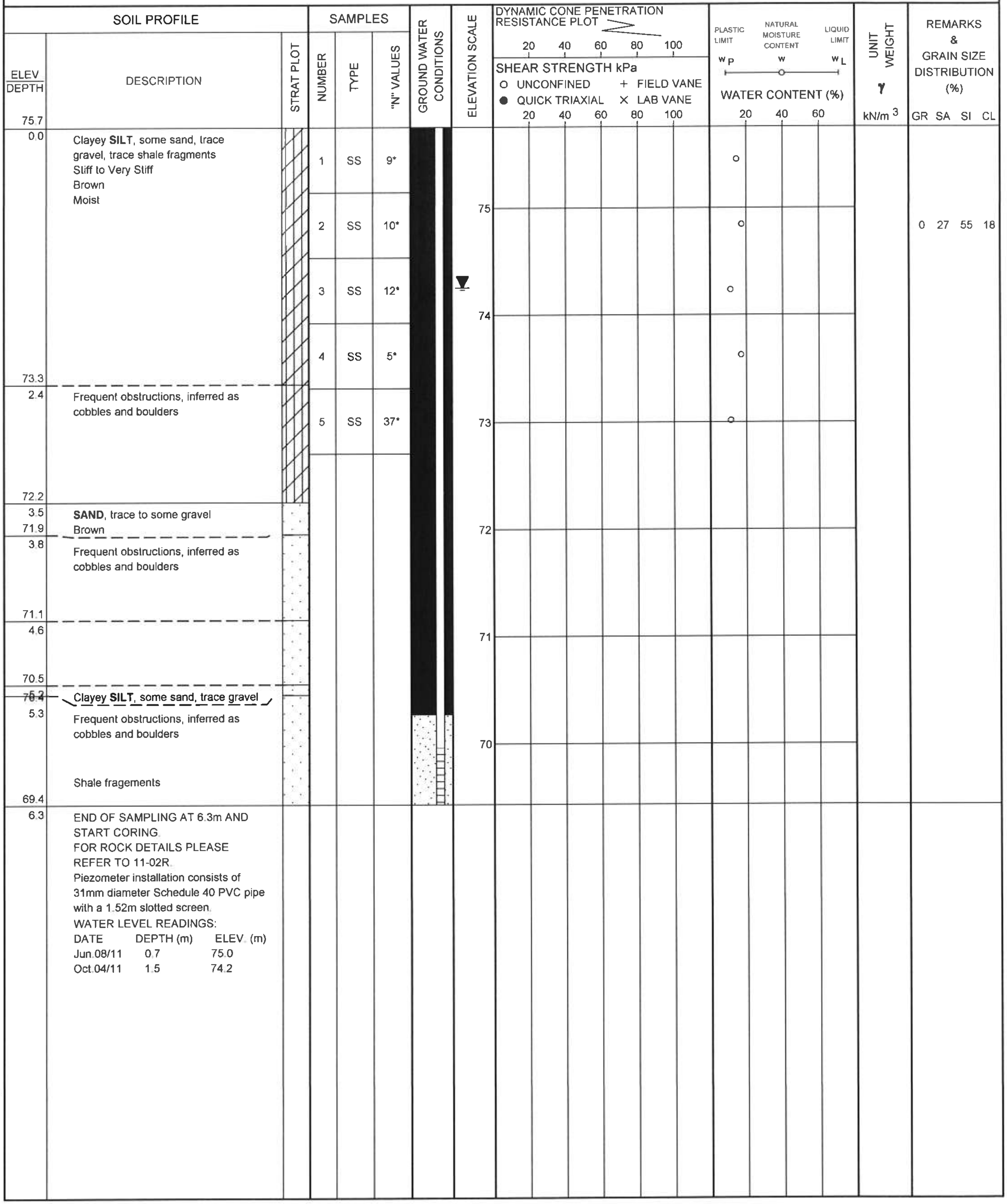
LOGGED : SLD
CHECKED : SKP

RECORD OF BOREHOLE No 11-02

1 OF 1

METRIC

W.P. W.O. 08-20008
LOCATION N 4 824 026.4 E 295 840.4 QEW Bridge at Credit River
ORIGINATED BY SLD/ES
HWY QEW
BOREHOLE TYPE Tripod (Hilti) - Wash Boring and Coring
COMPILED BY AN
DATUM Geodetic
DATE 2011.06.07 - 2011.06.09
CHECKED BY SKP



ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
75.7								
0.0	Clayey **SILT**, some sand, trace gravel, trace shale fragments Stiff to Very Stiff Brown Moist	1	SS	9*				
		2	SS	10*	0	27	55	18
		3	SS	12*				
		4	SS	5*				
73.3 2.4	Frequent obstructions, inferred as cobbles and boulders	5	SS	37*				
72.2 3.5	**SAND**, trace to some gravel Brown							
71.9 3.8	Frequent obstructions, inferred as cobbles and boulders							
71.1 4.6								
70.5 5.2	Clayey **SILT**, some sand, trace gravel							
70.4 5.3	Frequent obstructions, inferred as cobbles and boulders Shale fragements							
69.4 6.3	END OF SAMPLING AT 6.3m AND START CORING. FOR ROCK DETAILS PLEASE REFER TO 11-02R. Piezometer installation consists of 31mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS:							

DATE	DEPTH (m)	ELEV. (m)
Jun.08/11	0.7	75.0
Oct.04/11	1.5	74.2

+ 3, × 3: Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE 11-02R

PROJECT : QEW Mississauga Rd. Overpass
LOCATION : Mississauga, ON
STARTED : June 7, 2011
COMPLETED : June 9, 2011

INCLINATION: Vertical AZIMUTH:

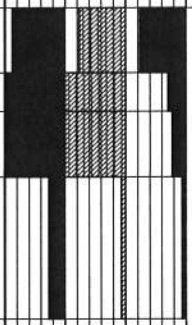
Project No. W.O. 08-20008

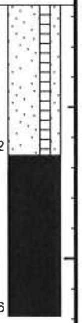
SHEET 1 OF 1
DATUM Geodetic

FR-FRACTURE F-FAULT SM-SMOOTH FL-FLEXURED
CL-CLEAVAGE J-JOINT R-ROUGH UE-UNEVEN
SH-SHEAR P-POLISHED ST-STEPPED W-WAVY
VN-VEIN S-SLICKENSIDED PL-PLANAR C-CURVED

FIELD/LABORATORY TESTING RESULTS
● Point Load Test Diametral
▲ Point Load Test Axial
▨ Laboratory UCS Test

DEPTH SCALE (metres)	DRILLING RECORD		DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATRATION RATE (m/min)	COLOUR / % RETURN FLUSH	RECOVERY TOTAL CORE % 80 60 40 20	RECOVERY SOLID CORE % 80 60 40 20	R.Q.D. % 80 60 40 20	FRACT. INDEX PER 0.3 m 5 10 15 20	DISCONTINUITY DATA DIP wrt Core Axis 0 30 60 90	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY k, cm/sec 10⁻⁶ 10⁻⁵ 10⁻⁴ 10⁻³	25 50 75 Unconfined Compressive Strength (MPa)	FIELD/LABORATORY TESTING RESULTS
					69.4												
	NQ Coring	RUN	**SHALE**, highly weathered, fine grained, thinly bedded, with strong limestone interbeds 75mm clay seam at 6.5m		6.3	1	0.006	10									
		RUN	50mm sand seam at 6.7m			2	0.008	0									Slotted Screen
		RUN	Limestone interbed (up to 25mm thick) at 6.5m 125mm sand seam at 6.7m			3	0.01	0									68.42
8		RUN	50mm sand seam at 6.9m 50mm clay seam at 6.8m 50mm clay seam at 7.0m 50mm sand and gravel layer at 7.1m		67.3	4	0.008	0									Bentonite 67.36
			END OF BOREHOLE AT 8.3m. NOTE: SAND BLEW BACK UP THE HOLE AT 7.0m.		8.4												
10																	
12																	
14																	
16																	





GROUNDWATER ELEVATIONS

▽ SHALLOW/SINGLE INSTALLATION
WATER LEVEL (date)

▼ DEEP/DUAL INSTALLATION
WATER LEVEL (date)

LOGGED : SLD
CHECKED : SKP

Appendix B

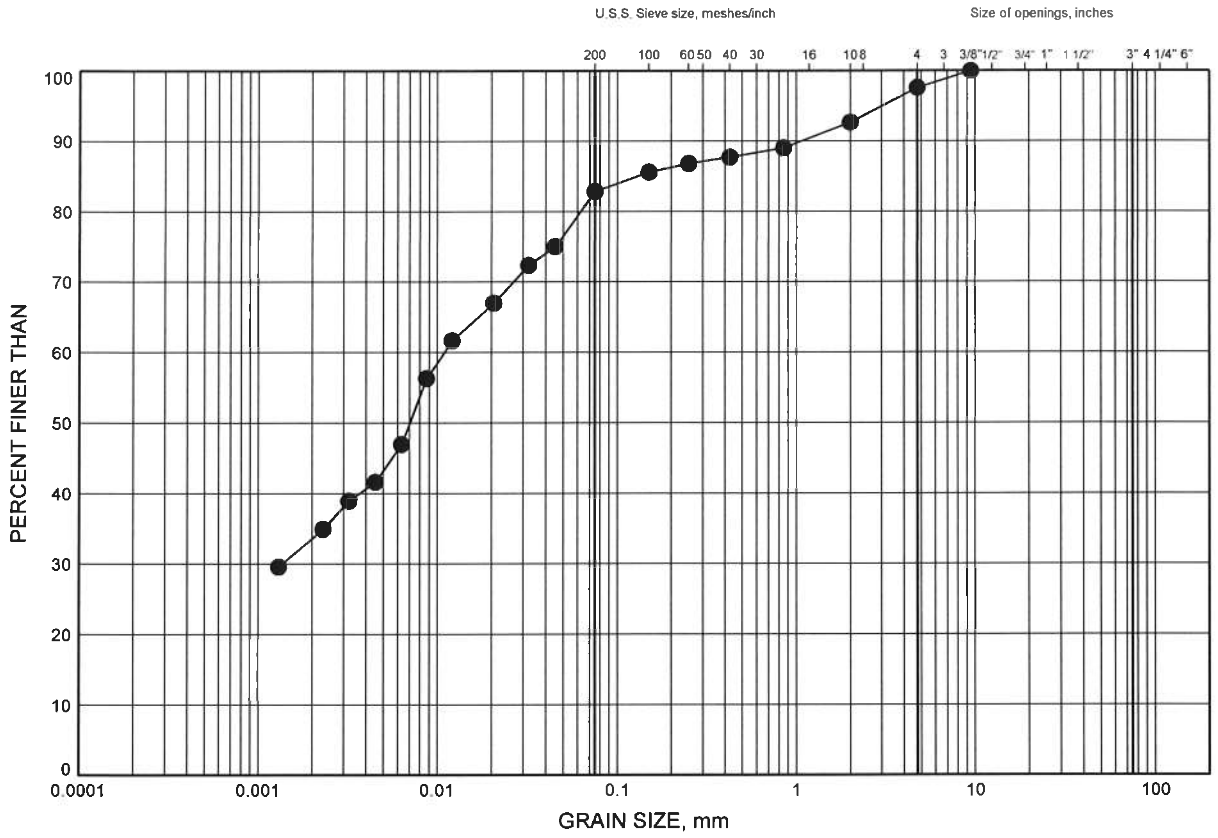
Laboratory Test Results

QEW Bridge at Credit River

GRAIN SIZE DISTRIBUTION

FIGURE B1

SILTY CLAY



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	11-01	1.07	93.50

Date May 2012

W.P.# W.O. 08-20008

THURBER

Prep'd MFA

Chkd. SKP

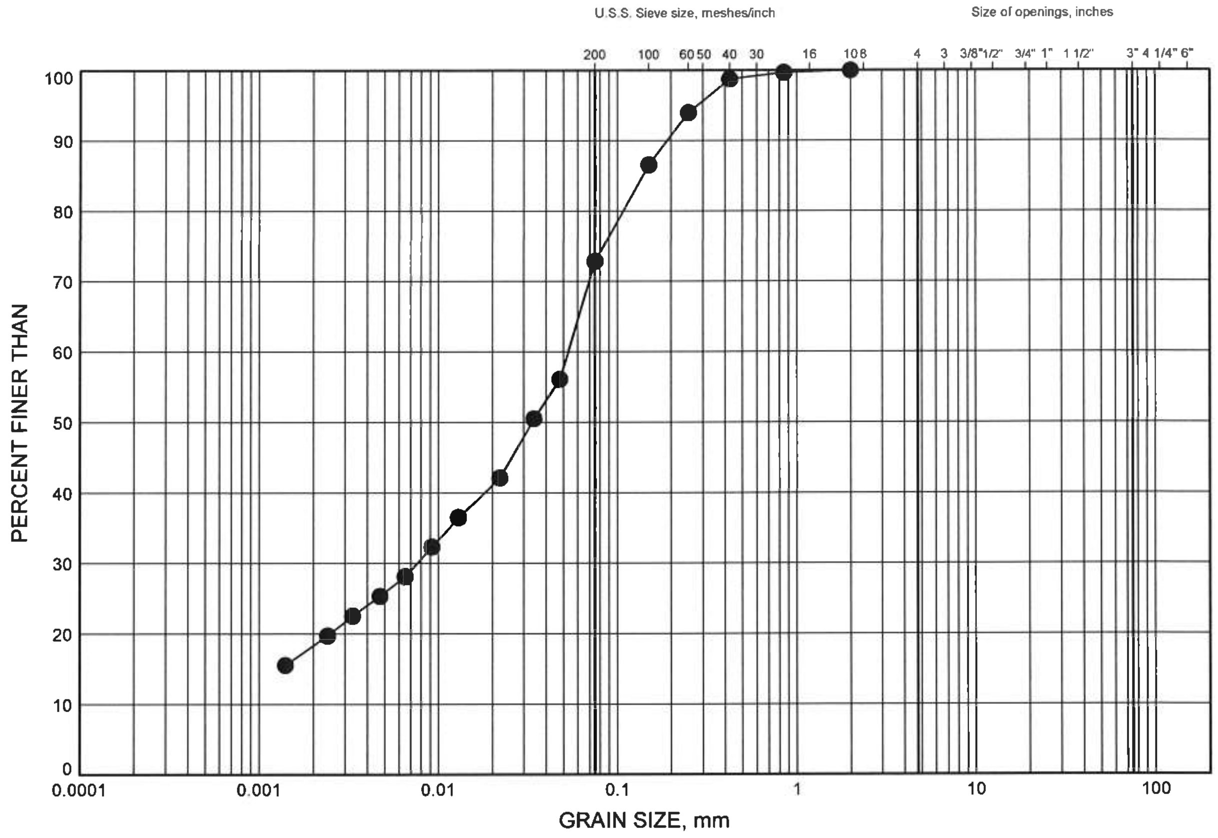
GRAIN SIZE DISTRIBUTION - THURBER 1174.GPJ 5/17/12

QEW Bridge at Credit River

GRAIN SIZE DISTRIBUTION

FIGURE B2

CLAYEY SILT



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	11-02	0.91	74.83

Date May 2012

W.P.# W.O. 08-20008

THURBER

Prep'd MFA

Chkd. SKP

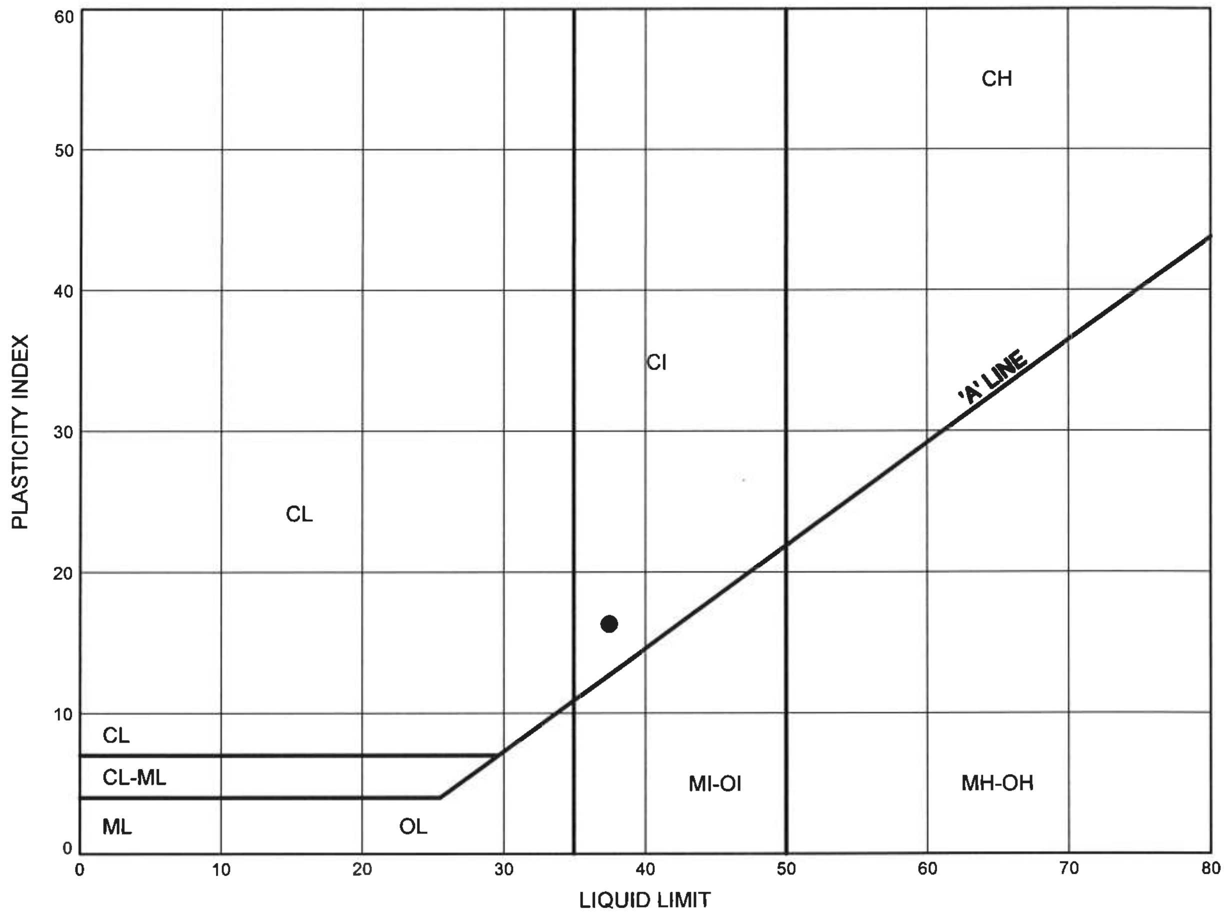
GRAIN SIZE DISTRIBUTION - THURBER 1174.GPJ 5/17/12

QEW Bridge at Credit River

ATTERBERG LIMITS TEST RESULTS

FIGURE B3

SILTY CLAY



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	11-01	1.07	93.50

THURBALT 1174.GPJ 5/17/12

Date May 2012

W.P.# W.O. 08-20008

THURBER

Prep'd MFA

Chkd. SKP

TABLE 2 - Point Load and Unconfined Compressive Strength Test Results
Credit River Access Road

Run	Depth (m)	UCS (MPa)	Rock Type	Test
Borehole		**10-01**		
2	3.4	0.7	Shale	
2	3.6	0.6	Shale	
2	3.9	2.7	Shale	
2	4.4	3.4	Shale	
3	4.7	10.7	Shale	
3	5.1	3.0	Shale	
3	5.4	0.7	Shale	
3	5.7	6.8	Shale	
3	5.8	3.4	Shale	
3	6.2	3.4	Shale	
4	6.5	2.0	Shale	
4	6.9	0.7	Shale	
4	7.3	0.7	Shale	
4	7.6	4.8	Shale	
5	7.9	0.7	Shale	
5	8.3	2.7	Shale	
5	8.7	5.4	Shale	
5	8.9	2.9	Shale	
5	9.2	2.0	Shale	
6	9.6	9.4	Shale	
6	10.0	1.3	Shale	
6	10.2	6.8	Shale	
6	10.7	1.4	Shale	
7	11.0	4.8	Shale	
7	11.6	0.7	Shale	
7	11.7	3.4	Shale	
7	12.2	49.6	Limestone	
Borehole		**10-02**		
1	4.4	0.5	Shale	
1	4.4	0.5	Shale	
2	5.0	0.7	Shale	
2	5.0	0.5	Shale	
2	5.7	0.5	Shale	
2	6.0	0.5	Shale	
2	6.0	8.9	Shale	
3	6.4	1.7	Shale	
3	6.4	3.1	Shale	
3	6.9	4.1	Shale	
3	7.4	0.7	Shale	
3	7.4	8.6	Shale	

Total Rock Core

Rock Type	**Shale**	**Limestone**
Average (MPa)	3.3	49.6
Minimum (MPa)	0.6	
Maximum (MPa)	10.7	

Run #	Average (MPa)
2	1.9
3	4.7
4	2.0
5	2.8
6	4.7
7	14.6

Total Rock Core

Rock Type	**Shale**	**Limestone**	**Shale/Limestone**
Average (MPa)	4.2	109.5	26.6
Minimum (MPa)	0.5	95.2	5.1
Maximum (MPa)	13.8	120.4	48.1

Run #	Average (Mpa)
2	2.2
3	3.6
4	2.8
5	1.9
6	5.0
7	19.7

TABLE 2 - Point Load and Unconfined Compressive Strength Test Results
Credit River Access Road

Run	Depth (m)	UCS (MPa)	Rock Type	Test
4	7.8	0.7	Shale	
4	7.8	4.5	Shale	
4	8.7	1.7	Shale	
4	8.7	4.4	Shale	
5	9.5	0.7	Shale	
5	9.5	5.4	Shale	
5	10.1	0.5	Shale	
5	10.1	3.2	Shale	
5	10.5	0.5	Shale	
5	10.5	1.4	Shale	
6	11.0	0.5	Shale	
6	11.0	13.8	Shale	
6	11.8	0.7	Shale	
6	11.8	7.6	Shale	
6	12.4	2.6	Shale	
7	12.5	1.7	Shale	
7	12.5	6.5	Shale	
7	12.9	112.9	Limestone	
7	13.2	1.7	Shale	
7	13.2	4.1	Shale	
7	13.6	10.7	Shale	UC
7	13.7	0.7	Shale	
8	14.0	0.7	Shale	
8	14.0	8.5	Shale	
8	14.8	1.7	Shale	
8	14.8	9.5	Shale	
8	15.1	3.4	Shale	
8	15.1	13.0	Shale	
9	15.5	0.7	Shale	
9	15.5	2.6	Shale	
9	16.0	0.7	Shale	
9	16.0	1.5	Shale	
9	16.4	120.4	Limestone	
10	17.0	0.7	Shale	
10	17.0	3.9	Shale	
10	17.6	0.7	Shale	
10	17.6	2.5	Shale	
10	18.0	0.7	Shale	
10	18.0	3.8	Shale	
11	18.6	10.1	Shale	
11	18.6	12.9	Shale	

8	6.1
9	25.2
10	2.0
11	6.6
12	20.0
13	3.3
14	7.2
15	9.2

TABLE 2 - Point Load and Unconfined Compressive Strength Test Results
Credit River Access Road

Run	Depth (m)	UCS (MPa)	Rock Type	Test
11	19.1	0.7	Shale	
11	19.1	7.3	Shale	
11	19.3	11.9	Shale	UC
11	19.6	0.7	Shale	
11	19.6	2.7	Shale	
12	19.9	3.4	Shale	
12	19.9	11.5	Shale	
12	20.3	1.7	Shale	
12	20.3	7.2	Shale	
12	21.0	95.2	Limestone	
12	21.1	1.7	Shale	
12	21.1	6.3	Shale	
12	21.3	5.1	Shale/Lime	
12	21.3	48.1	Shale/Lime	
13	21.9	0.7	Shale	
13	21.9	5.9	Shale	
14	22.4	5.0	Shale	
14	22.4	9.4	Shale	
15	23.0	5.1	Shale	
15	23.0	8.7	Shale	
15	23.9	6.7	Shale	
15	23.9	12.4	Shale	
15	24.3	13.4	Shale	
Borehole		**10-05**		
1	3.2	1.4	Shale	
2	3.5	0.9	Shale	
2	4.0	0.8	Shale	
2	4.2	1.1	Shale	
2	4.8	4.1	Shale	
3	5.0	0.7	Shale	
3	5.3	8.4	Shale	
3	5.6	1.5	Shale	
3	6.0	1.3	Shale	
3	6.3	1.3	Shale	
4	6.4	1.3	Shale	
4	6.7	8.5	Shale	
4	7.1	2.7	Shale	
4	7.4	10.8	Shale	UC
4	7.5	2.0	Shale	
4	7.8	2.0	Shale/Lime	
5	8.1	1.4	Shale	

Total Rock Core

Rock Type	Shale	Limestone	Shale/Limestone
Average (MPa)	3.0	73.1	3.7
Minimum (MPa)	0.7	27.9	2.0
Maximum (MPa)	10.8	200.5	5.4

Run #	Average (Mpa)
1	1.4
2	1.7
3	2.6
4	4.6
5	9.6
6	3.8
7	7.7
8	4.7
9	61.2

TABLE 2 - Point Load and Unconfined Compressive Strength Test Results
Credit River Access Road

Run	Depth (m)	UCS (MPa)	Rock Type	Test
5	8.5	1.4	Shale	
5	8.9	0.7	Shale	
5	9.0	35.0	Limestone	
6	9.4	3.4	Shale	
6	9.8	0.7	Shale	
6	10.0	2.9	Shale	
6	10.3	5.4	Shale	
6	10.7	6.8	Shale	
7	11.0	2.0	Shale	
7	11.3	27.9	Limestone	
7	11.6	1.3	Shale	
7	11.9	0.7	Shale	
7	12.2	6.8	Shale	
8	12.5	4.7	Shale	
8	13.0	2.7	Shale	
8	13.5	5.9	Shale	
8	13.7	5.4	Shale/Lime	
9	14.1	37.7	Limestone	
9	14.4	200.5	Limestone	
9	14.6	64.3	Limestone	
9	15.0	1.3	Shale	
9	15.2	2.0	Shale	

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No 10-02

1 OF 1

METRIC

W.P.	2186-07-00	LOCATION	QEW - Credit River Access Road (N 4 823 966.28 E 295 797.95)	ORIGINATED BY	SLL
HWY	QEW	BOREHOLE TYPE	Hollow Stem Augers/HQ Rock Coring	COMPILED BY	AN
DATUM	Geodetic	DATE	2010.09.30 - 2010.09.30	CHECKED BY	SKP

SOIL PROFILE		SAMPLES			REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
94.4								
0.0	**TOPSOIL:** (150mm)							
0.2	Silty **CLAY**, with sand, trace gravel, roots and rootlets Very Stiff to Hard Brown Moist	1	SS	25	3	59	22	16
		2	SS	29				
	With shale fragments	3	SS	39	3	16	46	35
		4	SS	34	1	26	47	26
91.2 3.2	**SHALE**, weathered Grey Moist	5	SS	76/ 0.275				
		6	SS	50/ 0.100				
89.7 4.6	END OF SPT SAMPLING AT 4.6m. START CORING AT 4.6m. FOR ROCK DETAILS PLEASE REFER TO 10-02R.	7	SS	50/ 0.075				

Other column headings: STRAT PLOT; GROUND WATER CONDITIONS; ELEVATION SCALE (94, 93, 92, 91, 90); DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE (20, 40, 60, 80, 100); PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) (20, 40, 60); UNIT WEIGHT γ kN/m^3

Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen.

SHALLOW PIEZOMETER
WATER LEVEL READINGS:

DATE	DEPTH (m)	ELEV. (m)
Oct.04/10	Dry	-
Oct.05/10	Dry	-
Oct.12/10	Dry	-
Dec.17/10	Dry	-

DEEP PIEZOMETER
WATER LEVEL READINGS:

DATE	DEPTH (m)	ELEV. (m)
Oct.04/10	8.5	85.9
Oct.05/10	8.5	85.9
Oct.12/10	8.7	85.7
Dec.17/10	9.3	85.1

ONTMT4S 9292.GPJ 1/7/11

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

$15 \overset{20}{\underset{10}{\oplus}} 5$ (%) STRAIN AT FAILURE

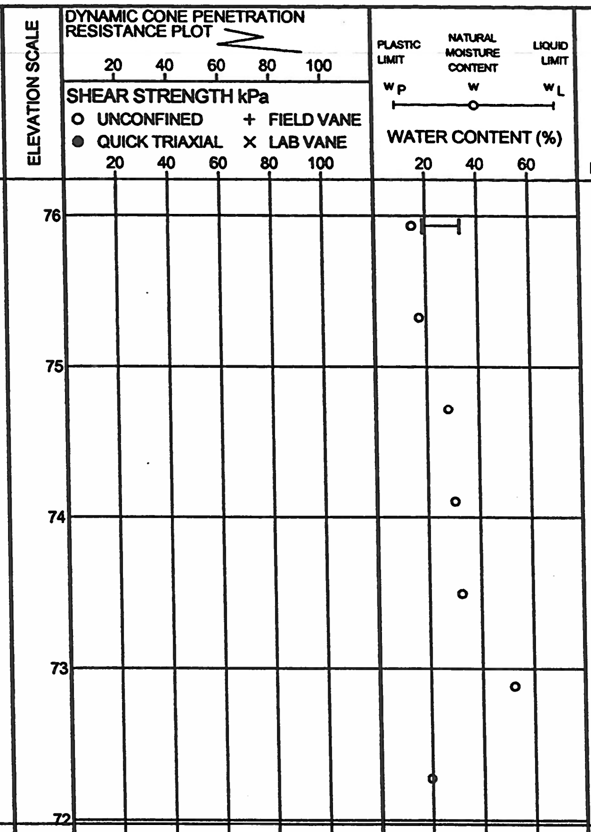
Ministry of Transportation
Ontario

RECORD OF BOREHOLE No 10-03A

1 OF 1 **METRIC**

W.P. 2186-07-00	LOCATION QEW - Credit River Access Road (N 4 823 979.03 E 295 865.10)		ORIGINATED BY GA
HWY QEW	BOREHOLE TYPE Tri-pod		COMPILED BY AN
DATUM Geodetic	DATE 2010.11.04 - 2010.11.04		CHECKED BY SKP

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	GR	SA	SI	CL
76.2									
0.0	Clayey SILT, with sand, trace gravel, trace shale fragments Very Stiff Brown/Grey Dry	1	SS	17		0	44	36	20
75.6 0.6	Frequent shale fragments, trace silt and sand	2	SS	24					
75.0 1.2	Clayey SILT, with sand, trace gravel Stiff to Firm Grey Wet	3	SS	14	∇	3	33	47	17
		4	SS	8					
73.5 2.7	PEAT, amorphous, occasional rootlets Firm Brown Moist	5	SS	10		1	38	46	15
		6	SS	4					
72.6 3.7	SHALE, weathered Grey Moist	7	SS	35					
72.0 4.3	END OF BOREHOLE AT 4.3m UPON SPLIT SPOON SAMPLER REFUSAL. BOREHOLE OPEN TO 4.3m AND WATER LEVEL AT 1.5m. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG TO SURFACE.								



UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%)

+³, ×³: Numbers refer to Sensitivity

15-◆-5 (20 above, 10 below) (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

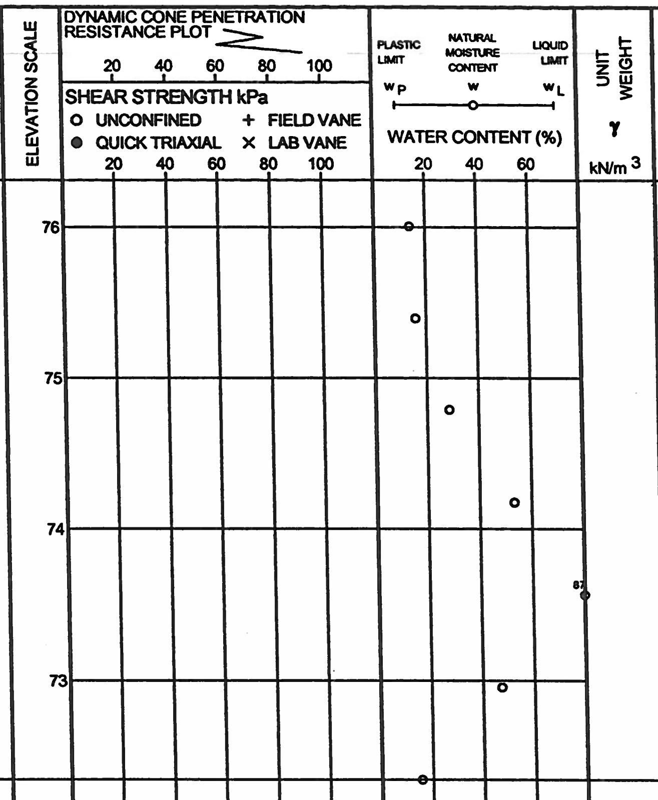
THURBER

RECORD OF BOREHOLE No 10-03B

1 OF 1 **METRIC**

W.P. 2186-07-00	LOCATION QEW - Credit River Access Road (N 4 823 983.23 E 295 856.15)	ORIGINATED BY GA
HWY QEW	BOREHOLE TYPE Tri-pod	COMPILED BY AN
DATUM Geodetic	DATE 2010.10.04 - 2010.11.04	CHECKED BY SKP

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES		GR	SA	SI	CL
76.3 0.0	Clayey SILT, with sand, trace gravel Very Stiff Brown Dry	1	SS	21					
75.7 0.6	Frequent shale fragments	2	SS	28					
75.0 1.3	Clayey SILT, with sand Stiff Grey Moist	3	SS	14		0	36	46	18
		4	SS	11	∇				
73.9 2.4	PEAT, amorphous, occasional rootlets Firm to Stiff Brown Moist	5	SS	7					
73.2 3.0	Clayey SILT, some clay, mixed with organics Stiff	6	SS	10		0	25	57	18
72.6 3.7	SHALE, weathered Grey Wet	7	SS	50/ 0.150					
72.3 4.0	END OF BOREHOLE AT 4.0m UPON SPLIT SPOON SAMPLER REFUSAL. BOREHOLE OPEN TO 4.0m AND WATER LEVEL AT 2.1m. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG TO SURFACE.								



+ 3, × 3: Numbers refer to Sensitivity

20
15-⊕-5 (%) STRAIN AT FAILURE
10

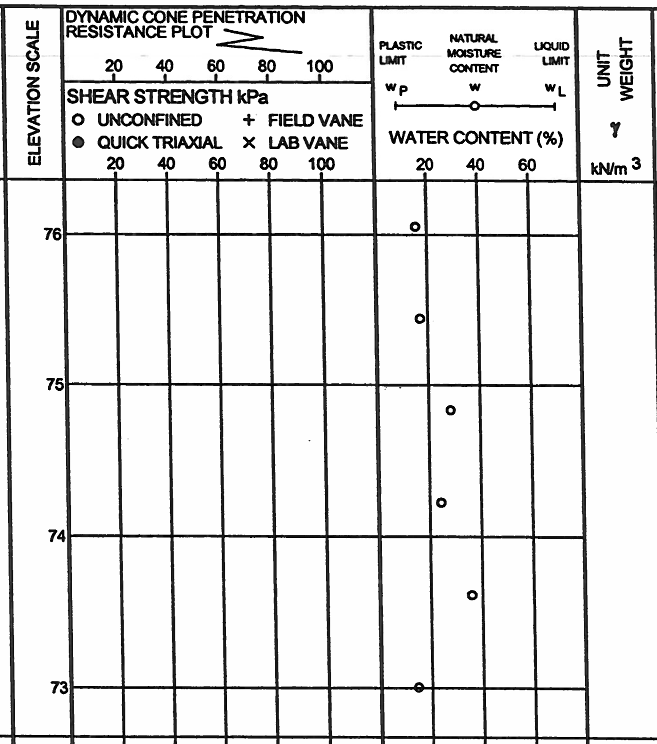
Ministry of Transportation
Ontario

RECORD OF BOREHOLE No 10-04

1 OF 1 **METRIC**

W.P. 2186-07-00
LOCATION QEW - Credit River Access Road (N 4 824 005.69 E 295 823.89)
ORIGINATED BY GA
HWY QEW
BOREHOLE TYPE Tri-pod
COMPILED BY AN
DATUM Geodetic
DATE 2010.11.05 - 2010.11.05
CHECKED BY SKP

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES		GR	SA	SI	CL
76.3									
0.0	TOPSOIL: (50mm)				⊻				
	Clayey SILT, some sand, trace gravel Stiff to Very Stiff Brown to Grey Dry to Wet	1	SS	17					
		2	SS	19		2	30	46	22
		3	SS	12					
		4	SS	8					
		5	SS	10		0	23	59	18
72.8		6	SS	26					
72.[illegible] 3.5	SHALE, weathered	7	SS	50/ 0.025					
3.7	END OF BOREHOLE AT 3.7m. BOREHOLE OPEN TO 3.7m AND WATER LEVEL AT 0.1m. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG TO SURFACE.								



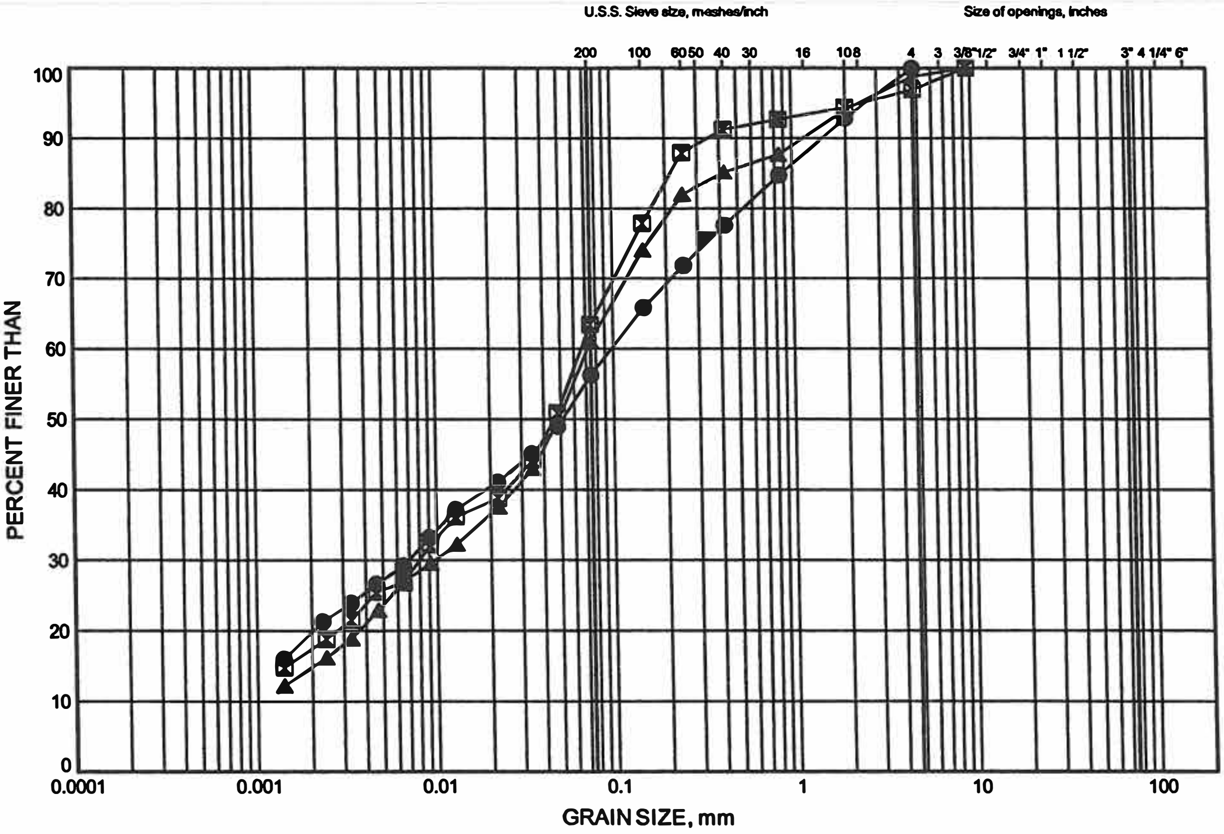
+ 3, × 3: Numbers refer to Sensitivity

15-⊕-5 (20 above, 10 below) (%) STRAIN AT FAILURE

QEW - Credit River Access Road
GRAIN SIZE DISTRIBUTION

FIGURE B1

CLAYEY SILT



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	10-03A	0.30	75.92
☒	10-03A	1.52	74.70
▲	10-03A	2.74	73.48

W.P.# 2186-07-00

Prepared By AN

Checked By SKP

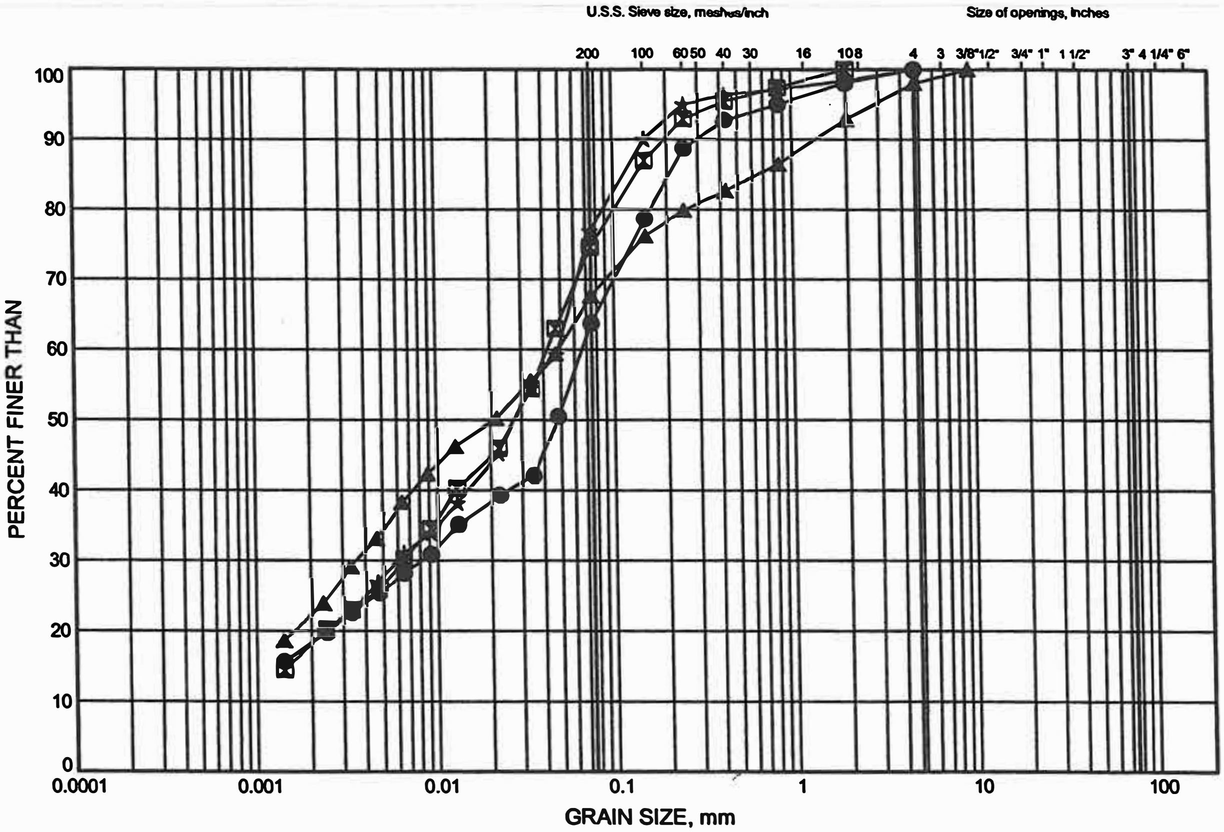
THURBER

QEW - Credit River Access Road

GRAIN SIZE DISTRIBUTION

FIGURE B2

CLAYEY SILT



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	10-03B	1.52	74.77
☒	10-03B	3.35	72.94
▲	10-04	0.91	75.43
★	10-04	2.74	73.60

W.P.# 2186-07-00

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Checked By SKP

THURBER

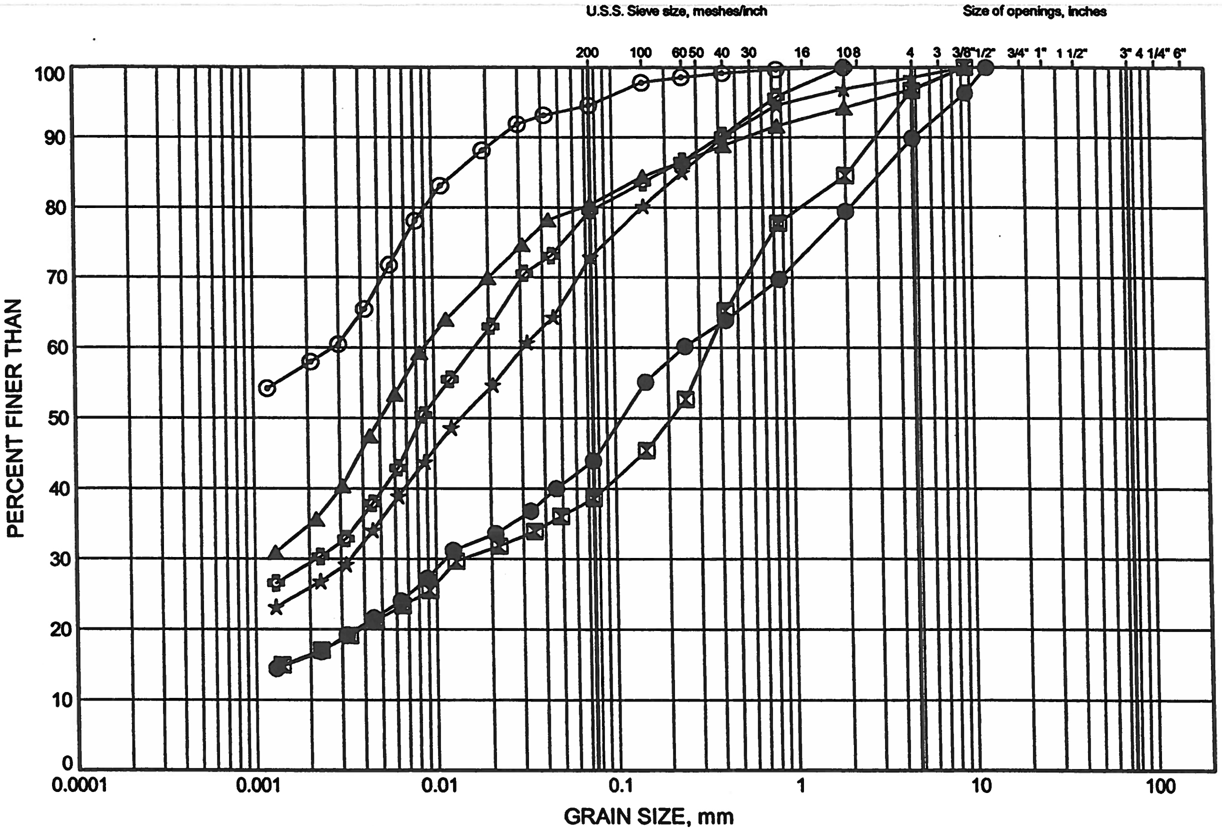
GRAIN SIZE DISTRIBUTION - THURBER 9292.GPJ 1/7/11

QEW - Credit River Access Road

GRAIN SIZE DISTRIBUTION

FIGURE B3

SILTY CLAY



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	10-01	0.30	95.88
☒	10-02	0.30	94.09
▲	10-02	1.83	92.56
★	10-02	2.59	91.80
⊙	10-05	0.91	94.33
✪	10-05	1.83	93.41

W.P.# 2186-07-00
Prepared By AN
Checked By SKP

THURBER

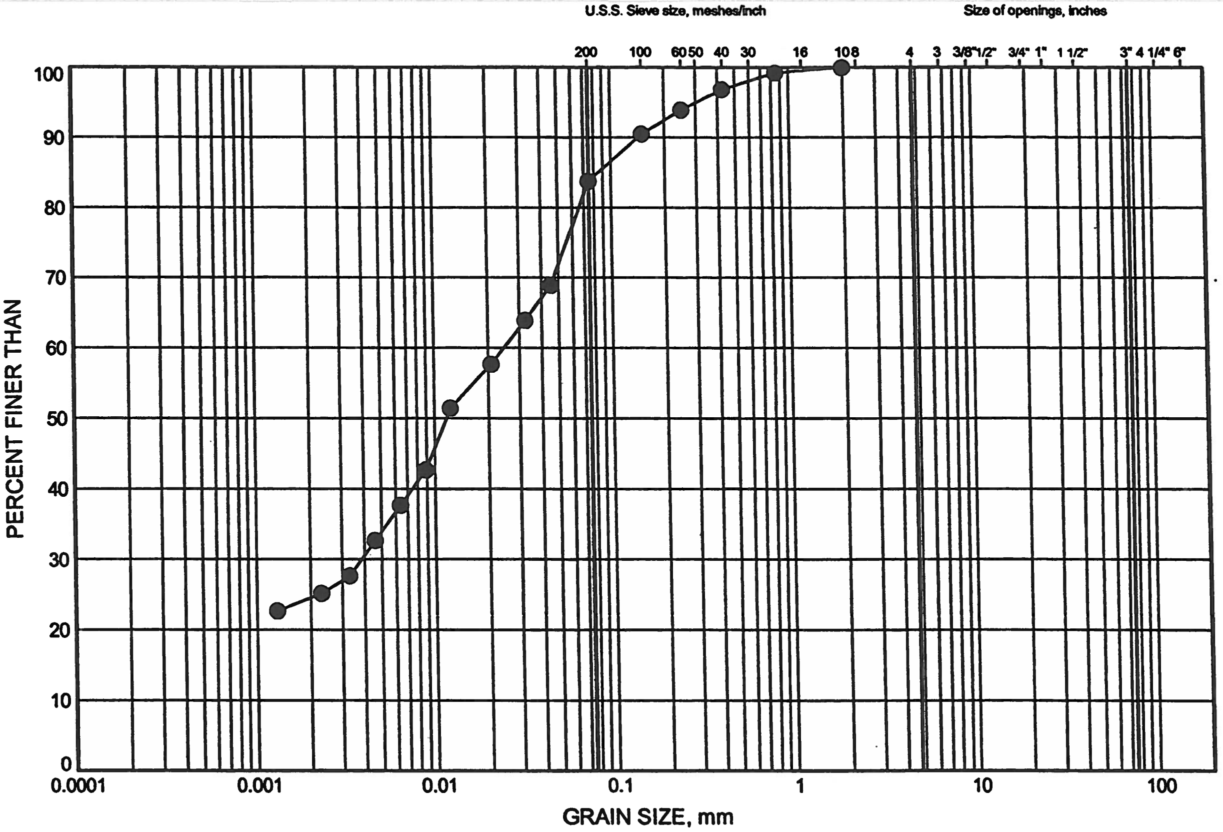
GRAIN SIZE DISTRIBUTION - THURBER 9292.GPJ 1/7/11

QEW - Credit River Access Road

GRAIN SIZE DISTRIBUTION

FIGURE B4

WEATHERED SHALE



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	10-01	1.83	94.35

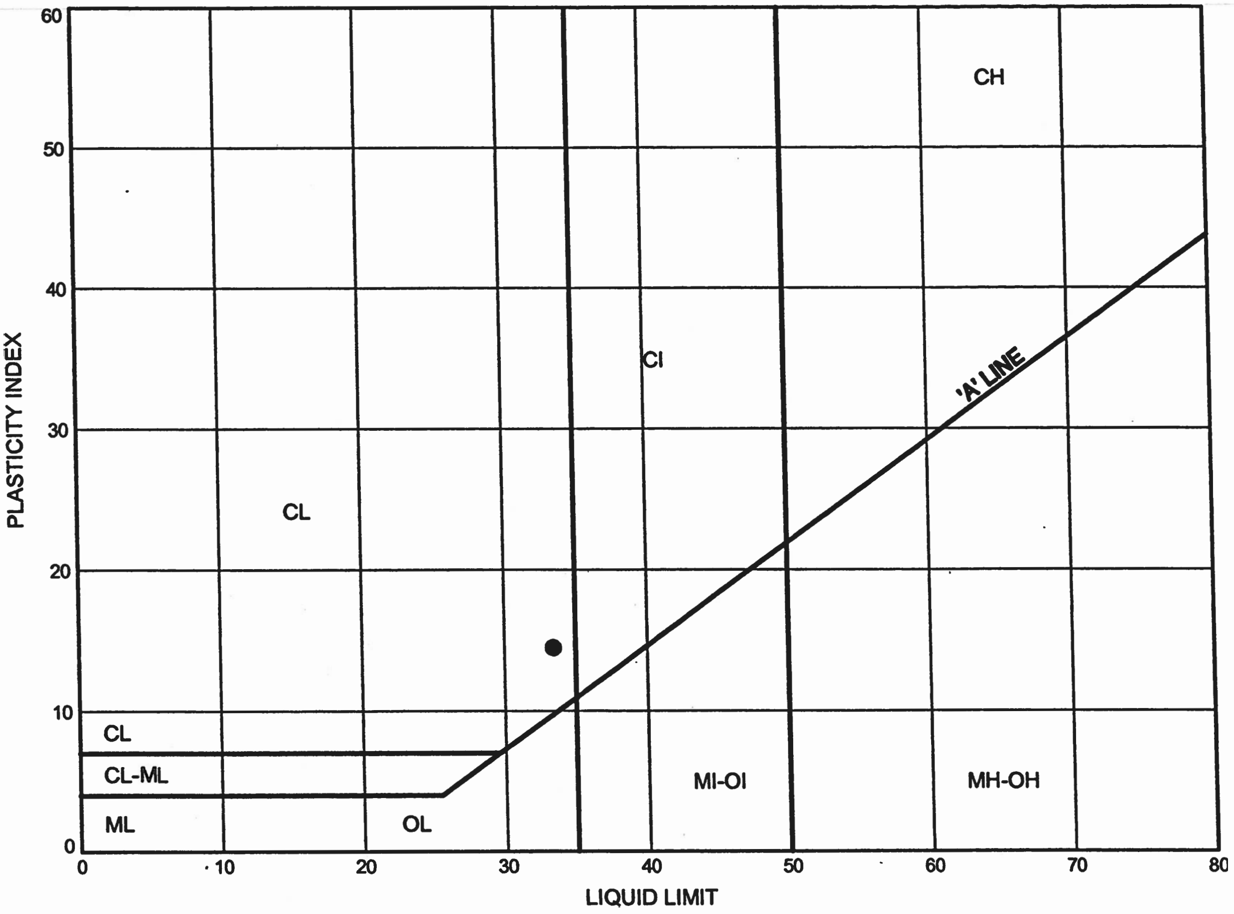
W.P.# 2186-07-00
Prepared By AN
Checked By SKP

QEW - Credit River Access Road

ATTERBERG LIMITS TEST RESULTS

FIGURE B5

CLAYEY SILT



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	10-03A	0.30	75.92

Date January 2011

Project 2186-07-00

THURBER

Prep'd AN

Chkd. SKP

THURBALT 9292.GPJ 1/7/11

APPENDIX B

Current Investigation – Borehole Records, Drillhole Records and Bedrock Core Photographs

LIST OF SYMBOLS

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

I. GENERAL

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

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta\,\sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
υ	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
σ_1, σ_2, σ_3	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

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

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

(a) Index Properties (continued)

w	water content
w_l or LL	liquid limit
w_p or PL	plastic limit
I_p or PI	plasticity index = $(w_l - w_p)$
w_s	shrinkage limit
I_L	liquidity index = $(w - w_p) / I_p$
I_C	consistency index = $(w_l - w) / I_p$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
I_D	density index = $(e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(d) Shear Strength

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

Notes:

1 $\tau = c' + \sigma' \tan \phi'$

2 shear strength = (compressive strength)/2

LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

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

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

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

Dynamic Cone Penetration Resistance; N_d:

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

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

Piezo-Cone Penetration Test (CPT)

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

III. SOIL DESCRIPTION

(a) Non-Cohesive (Cohesionless) Soils

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

(b) Cohesive Soils

Consistency	c_u, s_u kPa	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	Sand and Gravel
	With (cohesive)	Silty Clay with sand / Clayey Silt with sand

WEATHERINGS STATE

Fresh: no visible sign of weathering

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

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

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

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

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

BEDDING THICKNESS

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

JOINT OR FOLIATION SPACING

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

GRAIN SIZE

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

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

CORE CONDITION

Total Core Recovery (TCR)

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

Solid Core Recovery (SCR)

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

Rock Quality Designation (RQD)

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

DISCONTINUITY DATA

Fracture Index

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

Dip with Respect to Core Axis

The angle of the discontinuity relative to the axis (length) of the core. In a vertical borehole a discontinuity with a 90° 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
FLT	Fault
SH	Shear
VN	Vein
FR	Fracture
SY	Stylolite
BD	Bedding
FO	Foliation
CO	Contact
AXJ	Axial Joint
KV	Karstic Void
MB	Mechanical Break

PL	Planar
CU	Curved
UN	Undulating
IR	Irregular
K	Slickensided
PO	Polished
SM	Smooth
SR	Slightly Rough
RO	Rough
VR	Very Rough

RECORD OF BOREHOLE No CRB-1

SHEET 1 OF 1

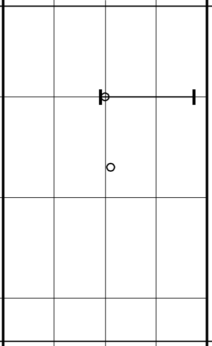
METRIC

PROJECT 1662333
G.W.P. 2002-13-00
DIST Central HWY QEW
DATUM Geodetic
LOCATION N 4823940.2; E 295811.1 MTM NAD 83 ZONE 10 (LAT. 43.555344; LONG. -79.611250)
BOREHOLE TYPE CME 55, 210 mm O.D., 108 mm I.D. Hollow Stem Augers (Auto Hammer)
DATE February 7, 2018
ORIGINATED BY JL
COMPILED BY KN
CHECKED BY SMM

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES			GR	SA	SI	CL
94.9	GROUND SURFACE									
0.0	Sandy clayey silt, some gravel, contains rootlets / organics (FILL) Stiff Brown Dry to moist	1	SS	10						
94.2 0.7	SILTY CLAY some sand, trace gravel Stiff Mottled brown to grey Moist	2A 2B	SS	11		94	4	15	49	32
		3A 3B	SS	36		93				
93.2 1.7	SHALE (BEDROCK) Grey	4	SS	95/0.10						
						92				
		5	SS	100/0.13						
91.6 3.3	END OF BOREHOLE - SPLIT-SPOON REFUSAL NOTE: 1. Borehole dry upon completion of drilling.									

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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



UNIT WEIGHT γ kN/m³

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

GTA-MTO 001 S:\CLIENTS\MTO\QEW-CREDIT_RIVER\02_DATA\GINT\QEW-CREDIT_RIVER.GPJ GAL-GTA.GDT 2/11/19

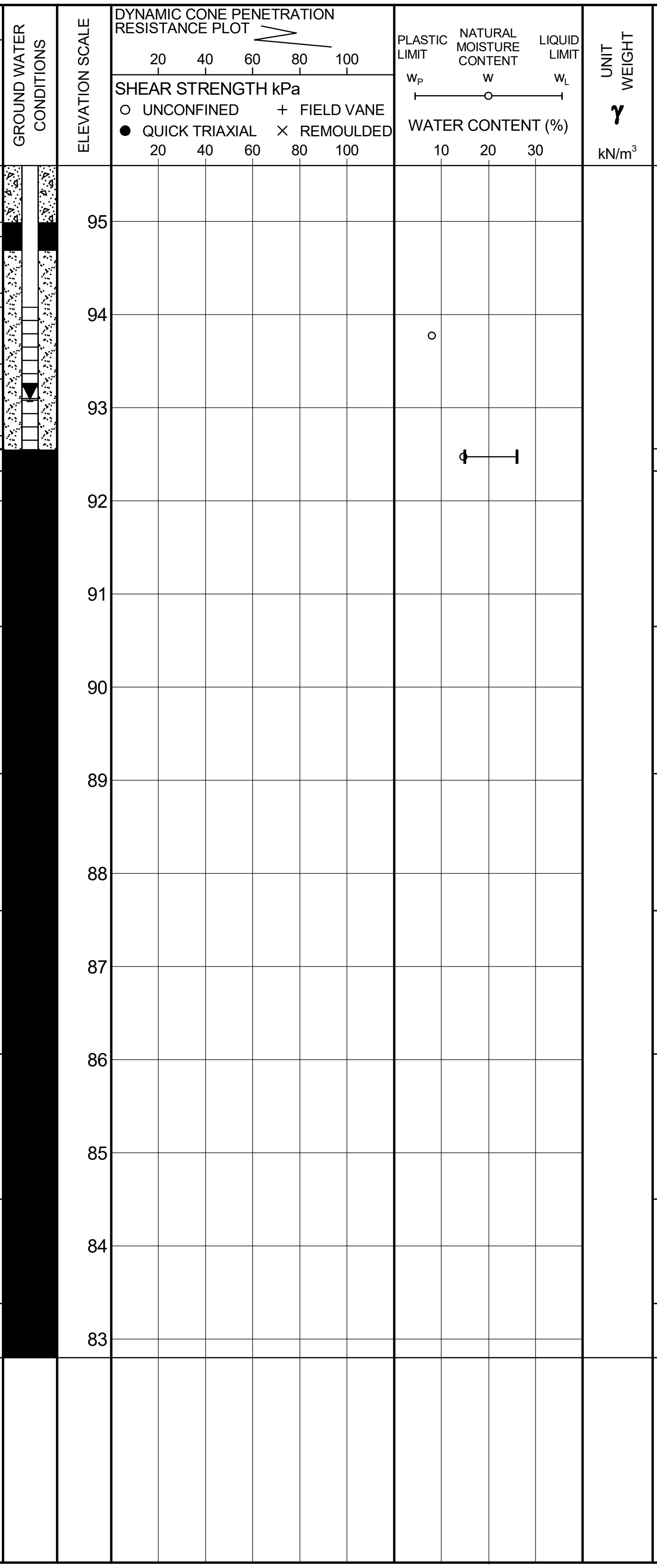
RECORD OF BOREHOLE No CRB-2

SHEET 1 OF 2

METRIC

PROJECT 1662333
G.W.P. 2002-13-00
DIST Central HWY QEW
DATUM Geodetic
LOCATION N 4823949.7; E 295828.3 MTM NAD 83 ZONE 10 (LAT. 43.555430; LONG. -79.611047)
BOREHOLE TYPE CME 55, 210 mm O.D., 108 mm I.D. Hollow Stem Augers (Auto Hammer)
DATE February 6, 2018
ORIGINATED BY JL
COMPILED BY KN
CHECKED BY SMM

SOIL PROFILE		SAMPLES			REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR SA SI CL
95.6	GROUND SURFACE				
0.0	TOPSOIL (200 mm)				
0.2	Sandy clayey silt, some gravel (FILL) Firm Grey Moist	1	SS	7	
94.9 0.7	Sand, some silt, trace to some gravel, trace clay, contains rootlets / organics (FILL) Loose to compact Brown to grey Moist, becoming wet at a depth of about 2.3 m	2	SS	6	6 12 47 35
		3	SS	5	10 67 19 4
93.0 2.6	CLAYEY SILT, some sand, some gravel (TILL) Very stiff Grey Moist to wet	4A 4B	SS	15	
		5A 5B	SS	37	
92.4 3.2	SHALE (BEDROCK) Grey Bedrock cored from a depth of 3.2 m to 12.8 m For bedrock coring details, refer to Record of Drillhole CRB-2	1	RC	REC 37.5%	RQD = 0%
		2	RC	REC 23%	RQD = 2%
		3	RC	REC 100%	RQD = 73%
		4	RC	REC 100%	RQD = 100%
		5	RC	REC 100%	RQD = 83%
		6	RC	REC 71%	RQD = 65%
		7	RC	REC 100%	RQD = 100%
		8	RC	REC 100%	RQD = 72%
82.8 12.8					



Continued Next Page

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

PROJECT 1662333

RECORD OF BOREHOLE No CRB-2

SHEET 2 OF 2

METRIC

G.W.P. 2002-13-00

LOCATION N 4823949.7; E 295828.3 MTM NAD 83 ZONE 10 (LAT. 43.555430; LONG. -79.611047)

ORIGINATED BY JL

DIST Central HWY QEW

BOREHOLE TYPE CME 55, 210 mm O.D., 108 mm I.D. Hollow Stem Augers (Auto Hammer)

COMPILED BY KN

DATUM Geodetic

DATE February 6, 2018

CHECKED BY SMM

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/m^3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	END OF BOREHOLE NOTES: 1. Borehole dry upon completion of drilling. 3. Groundwater level measurements in piezometer: Date Depth (m) Elev. (m) 12/03/18 2.6 93.0 30/04/18 2.6 93.0 06/11/18 2.5 93.1										

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

PROJECT: 1662333

RECORD OF DRILLHOLE: CRB-2

SHEET 1 OF 2

LOCATION: N 4823949.7 ;E 295828.3

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: February 6, 2018

DRILL RIG: CME 55 Track

DRILLING CONTRACTOR: Geo-Environmental Drilling

DATUM: Geodetic

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

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

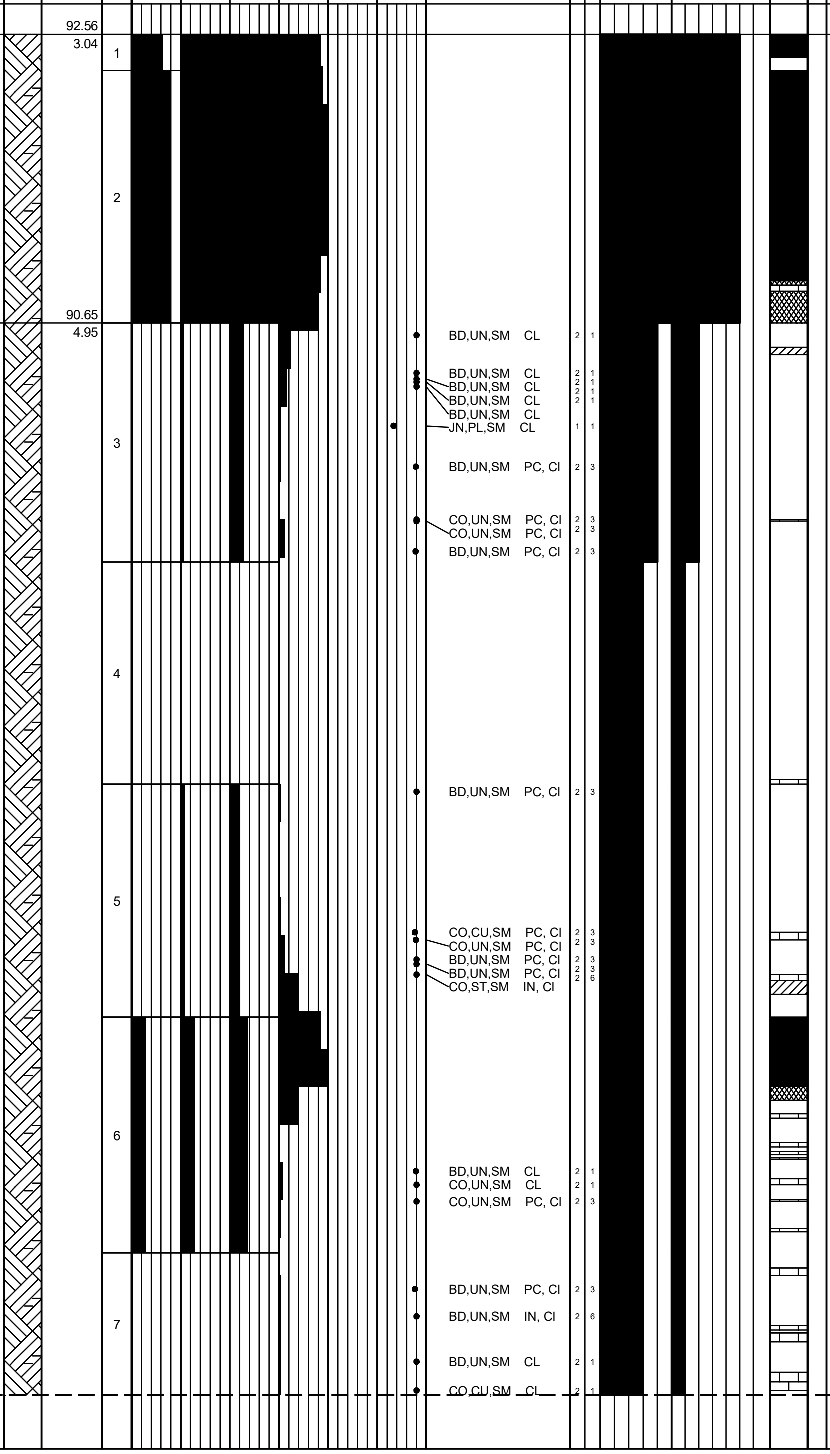
PL - Planar
CU- Curved
UN- Undulating
ST - Stepped
IR - Irregular

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

BR - Broken Rock

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. DEPTH (m)	NOTES WATER LEVELS INSTRUMENTATION
		Continued from Borehole CRB-2	92.56	
	NQ Rock Core	Completely weathered, thinly laminated to medium bedded, brown to grey, very fine to fine grained, faintly porous, extremely weak SHALE (Georgian Bay Formation) with LIMESTONE interbeds	3.04	
4				
5		Slightly weathered to fresh, thinly laminated to medium bedded, grey, very fine to fine grained, faintly porous, very weak to weak SHALE (Georgian Bay Formation) with LIMESTONE interbeds	90.65 4.95	
6				
7				
8				UC = 11.2 MPa
				PLT(D) = 0.04 PLT(A) = 0.82
9				
10				
				PLT(D) = 0.19
11				
				UC = 13.0 MPa PLT(A) = 0.33
12		*CONTINUED NEXT PAGE*		



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

FEATURES LEGEND

▨ BROKEN CORE

▩ CLAY SEAM

▦ LIMESTONE

■ LOST CORE

DEPTH SCALE
1 : 50

GOLDER

LOGGED: JL
CHECKED: AB

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

PROJECT: 1662333

RECORD OF DRILLHOLE: CRB-2

SHEET 2 OF 2

LOCATION: N 4823949.7 ;E 295828.3

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: February 6, 2018

DRILL RIG: CME 55 Track

DRILLING CONTRACTOR: Geo-Environmental Drilling

DATUM: Geodetic

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

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	Jr	Ja	NOTES WATER LEVELS INSTRUMENTATION
		--- CONTINUED FROM PREVIOUS PAGE ---						
	NQ Rock Core	Slightly weathered to fresh, thinly laminated to medium bedded, grey, very fine to fine grained, faintly porous, very weak to weak SHALE (Georgian Bay Formation) with LIMESTONE interbeds		7				PLT(D) = 0.10 PLT(A) = 0.24
				8	BD,UN,SM CC, CI	2	4	
					CO,CU,SM PC, CI	2	3	
					CO,UN,SM PC, CI	2	3	
			82.80		CO,UN,SM IN, CI	2	6	
		END OF DRILLHOLE	12.80					
13								
14								
15								
16								
17								
18								
19								
20								
21								

FEATURES LEGEND

▨ BROKEN CORE
▩ CLAY SEAM
▤ LIMESTONE
■ LOST CORE

DEPTH SCALE
1 : 50

GOLDER

LOGGED: JL
CHECKED: AB

RECORD OF BOREHOLE No CRB-2A

SHEET 1 OF 1

METRIC

PROJECT 1662333

G.W.P. 2002-13-00

DIST Central HWY QEW

DATUM Geodetic

LOCATION N 4823960.1; E 295808.0 MTM NAD 83 ZONE 10 (LAT. 43.555523; LONG. -79.611298)

BOREHOLE TYPE 64 mm O.D. 51 mm I.D. Continuous Split Spoon Sampling (Cathead/Safety Hammer)

DATE January 28, 2018

ORIGINATED BY JL

COMPILED BY KN

CHECKED BY SMM

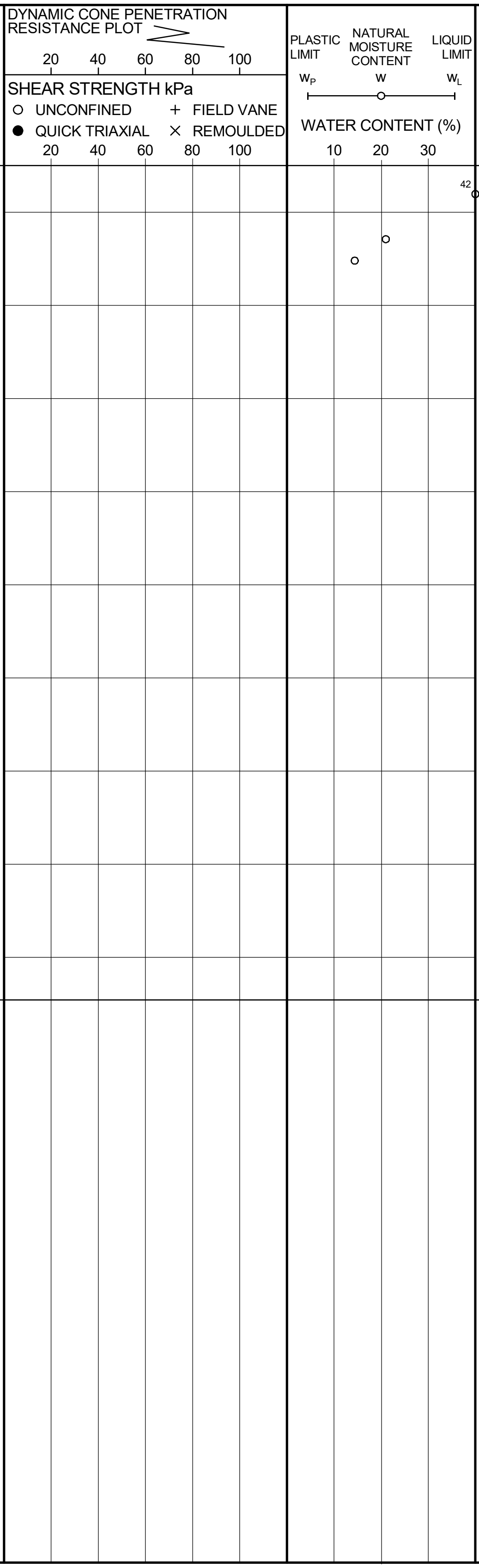
SOIL PROFILE

ELEV DEPTH	DESCRIPTION
94.5	GROUND SURFACE
0.0	TOPSOIL (100 mm)
0.1	Clayey silt, sandy to with sand, contains organics / rootlets, oxidation staining, trace gravel (FILL) Firm to hard Brown to grey to black Moist
93.5 1.1	SILTY CLAY, trace sand Hard Mottled brown to grey Dry
	SHALE (BEDROCK) Grey Bedrock cored from a depth of 1.1 m to 9.0 m For bedrock coring details, refer to Record of Drillhole CRB-2A
85.5 9.0	END OF BOREHOLE NOTE: 1. Borehole dry prior to rock coring.

SAMPLES

NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	7		
2A	SS	50		
2B				0 4 65 31
2C				
1	RC	REC 100%		RQD = 0%
2	RC	REC 86%		RQD = 0%
3	RC	REC 100%	▼	RQD = 30%
4	RC	REC 100%		RQD = 78%
5	RC	REC 100%		RQD = 100%
6	RC	REC 100%		RQD = 38%
7	RC	REC 100%		RQD = 42%
8	RC	REC 54%		RQD = 32%
9	RC	REC 100%		RQD = 100%
10	RC	REC 100%		RQD = 89%
11	RC	REC 100%		RQD = 100%

ELEVATION SCALE: 94, 93, 92, 91, 90, 89, 88, 87, 86



UNIT WEIGHT γ kN/m³

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

$○^{3\%}$ STRAIN AT FAILURE

PROJECT: 1662333

RECORD OF DRILLHOLE: CRB-2A

SHEET 1 OF 1

LOCATION: N 4823960.1 ;E 295808.0

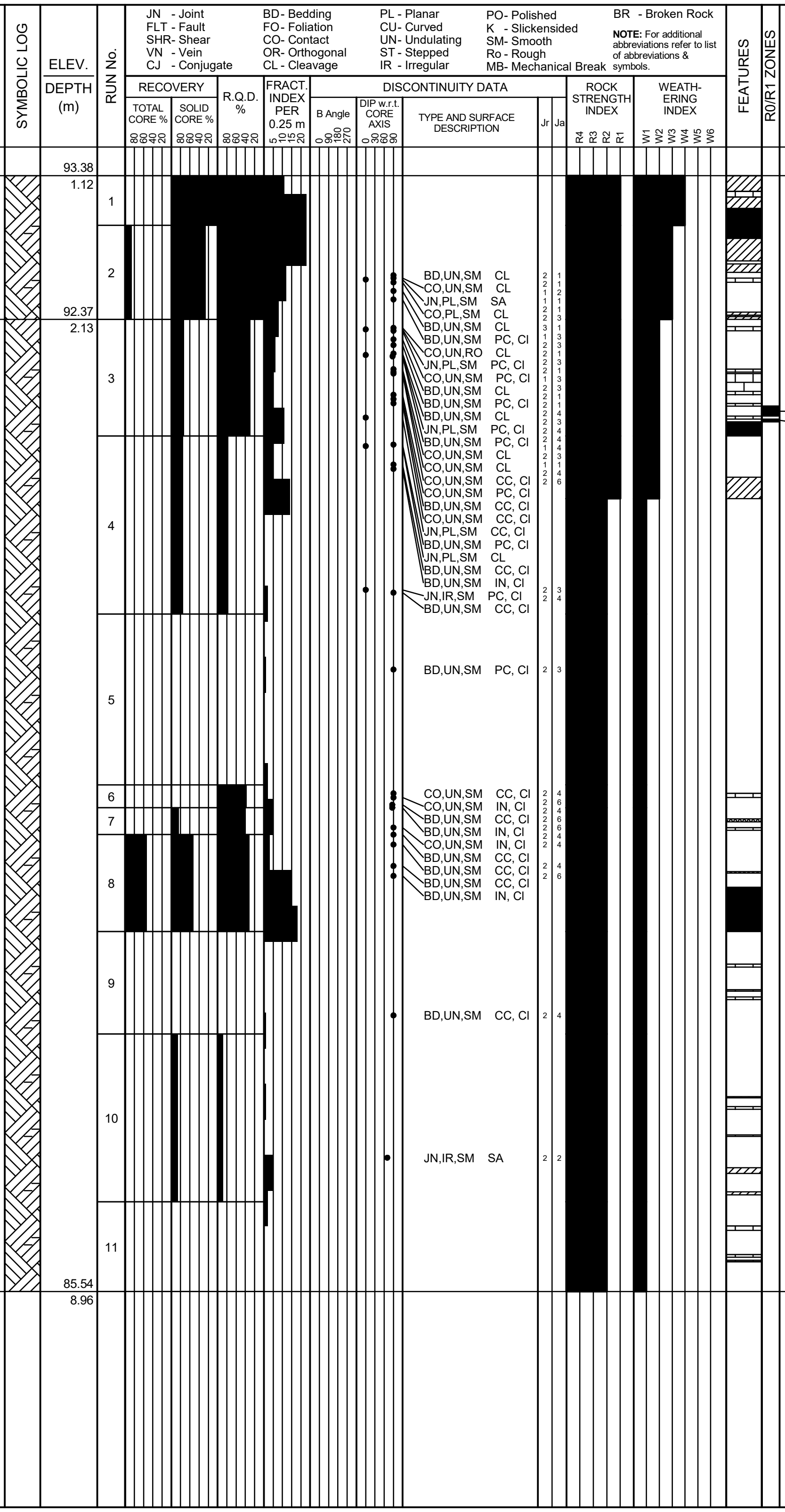
INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: January 28, 2018
DRILL RIG: Portable/Tripod
DRILLING CONTRACTOR: OGS Drilling

DATUM: Geodetic

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

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



DEPTH SCALE METRES	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	NOTES WATER LEVELS INSTRUMENTATION
	Continued from Borehole CRB-2A	93.38		
	Highly weathered to moderately weathered, thinly laminated to medium bedded, grey, very fine to fine grained, faintly porous, very weak SHALE (Georgian Bay Formation) with LIMESTONE interbeds	1.12	1 2	
2	Slightly weathered to fresh, thinly laminated to medium bedded, grey, very fine to fine grained, faintly porous, very weak to weak SHALE (Georgian Bay Formation) with LIMESTONE interbeds	92.37 2.13	3	
3			4	
4				UC = 18.2 MPa
5			5	PLT(D) = 0.18 MPa UC = 17.1 MPa
6			6 7 8	PLT(D) = 0.28 MPa PLT(A) = 0.40 MPa
7			9	PLT(A) = 0.37 MPa
8			10 11	PLT(D) = 0.44 MPa PLT(A) = 0.13 MPa
9	END OF DRILLHOLE	85.54 8.96		
10				

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

FEATURES LEGEND

▨	BROKEN CORE
▩	CLAY SEAM
▦	LIMESTONE
■	LOST CORE

DEPTH SCALE
1 : 50

GOLDER

LOGGED: JL
CHECKED: MPL

RECORD OF BOREHOLE No CRB-2B

SHEET 1 OF 1

METRIC

PROJECT 1662333

G.W.P. 2002-13-00

LOCATION N 4823955.2; E 295818.7 MTM NAD 83 ZONE 10 (LAT. 43.555479; LONG. -79.611166)

ORIGINATED BY SK

DIST Central HWY QEW

BOREHOLE TYPE CME 55, 210 mm O.D., 108 mm I.D. Hollow Stem Augers (Auto Hammer)

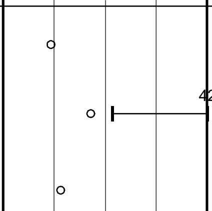
COMPILED BY

DATUM Geodetic

DATE July 5, 2018

CHECKED BY

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						
	GROUND SURFACE									
0.0	TOPSOIL (150 mm)	1A								
0.2	Sandy clayey silt, some gravel (FILL) Stiff Brown Moist	1B	SS	13						
0.7	SILTY CLAY, some sand, trace to some gravel Stiff Brown Moist	2	SS	14				42		6 13 46 35
1.4	CLAYEY SILT, some sand, some gravel, some shale fragments (RESIDUAL SOIL) Hard Brown Moist - Auger grinding at 1.5 m depth - A 0.3 m thick boulder / limestone layer at 2.1	3	SS	61						
2.4										
3.6	SHALE (BEDROCK) Bedrock cored from a depth of 2.0 m to 12.7 m For bedrock coring details, refer to Record of Drillhole CRB-2B	1	RC	REC 100%						RQD = 33%
		2	RC	REC 97%						RQD = 55%
		3	RC	REC 100%						RQD = 67%
		4	RC	REC 100%						RQD = 74%
		5	RC	REC 100%						RQD = 64%
		6	RC	REC 100%						RQD = 63%
12.7	END OF BOREHOLE NOTE: 1. Borehole dry prior to rock coring.									



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

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

PROJECT: 1662333

RECORD OF DRILLHOLE: CRB-2B

SHEET 1 OF 2

LOCATION: N 4823955.2 ;E 295818.7

INCLINATION: -90° AZIMUTH: ---

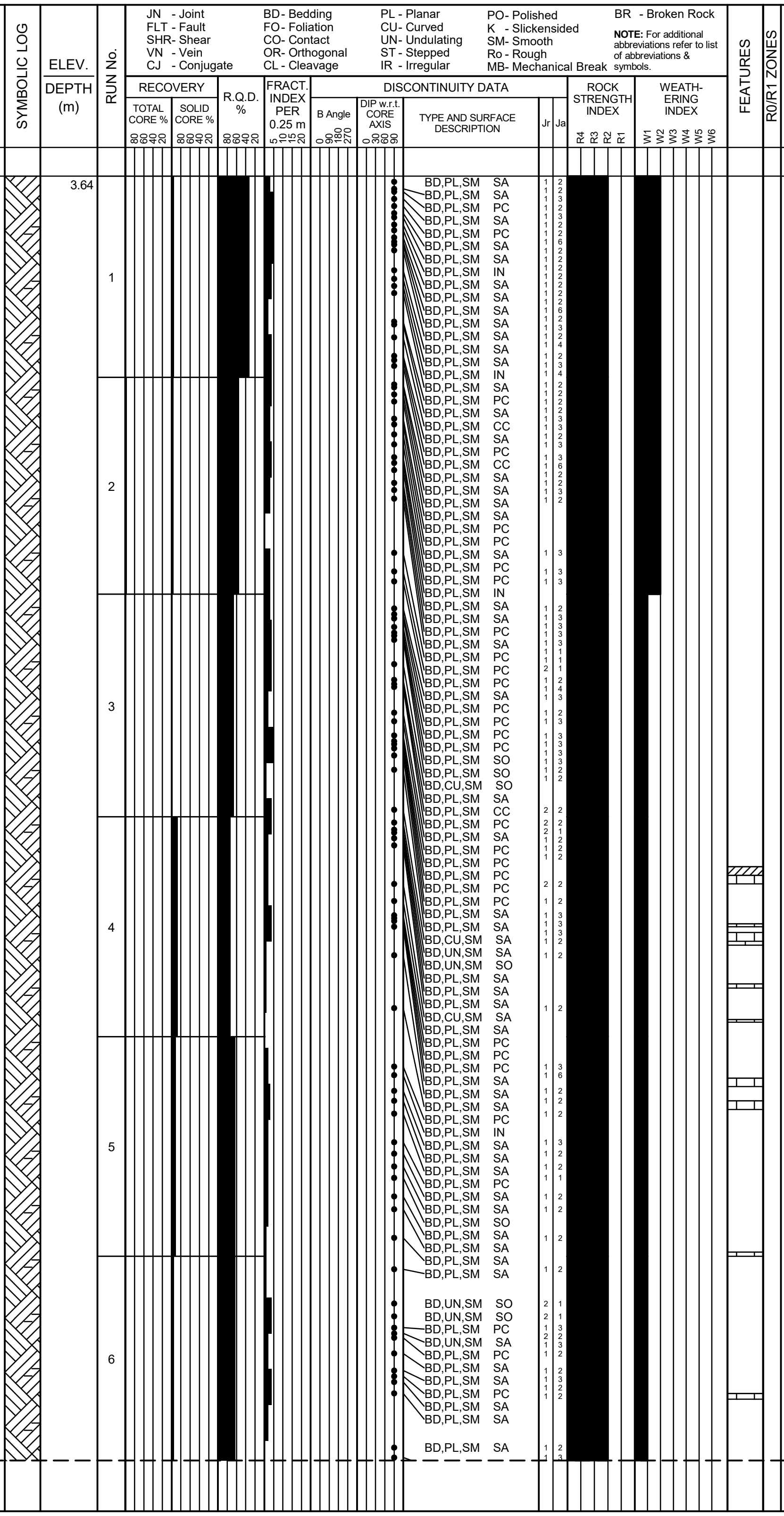
DRILLING DATE: July 5, 2018

DRILL RIG: CME 55 Track

DRILLING CONTRACTOR: Geo-Environmental Drilling

DATUM: Geodetic

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	NOTES WATER LEVELS INSTRUMENTATION
		Continued from Borehole CRB-2B	
4 5 6		Slightly weathered to fresh, thinly laminated to medium bedded, very fine to fine grained, faintly porous, weak SHALE (Georgian Bay Formation) with LIMESTONE interbeds	
7			UCS = 12.1 MPa E = 0.59 MPa
8 9			UCS = 15.5 MPa E = 0.63 MPa
10 11 12			
		CONTINUED NEXT PAGE	



FEATURES LEGEND

▨ BROKEN CORE

▩ CLAY SEAM

▦ LIMESTONE

■ LOST CORE

DEPTH SCALE

1 : 50

GOLDER

LOGGED:

CHECKED:

PROJECT: 1662333

RECORD OF DRILLHOLE: CRB-2B

SHEET 2 OF 2

LOCATION: N 4823955.2 ;E 295818.7

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: July 5, 2018

DRILL RIG: CME 55 Track

DRILLING CONTRACTOR: Geo-Environmental Drilling

DATUM: Geodetic

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

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

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

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

BR - Broken Rock

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	RECOVERY TOTAL CORE % 80 60 40 20	RECOVERY SOLID CORE % 80 60 40 20	R.Q.D. % 80 60 40 20	FRACT. INDEX PER 0.25 m 5 10 15 20	DISCONTINUITY DATA B Angle 0 90 180 270	DISCONTINUITY DATA DIP w.r.t. CORE AXIS 0 30 60 90	DISCONTINUITY DATA 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 ---																	
			▨					■				BD,PL,SM PC			■	■			
		END OF BOREHOLE		12.72															
13																			
14																			
15																			
16																			
17																			
18																			
19																			
20																			
21																			

FEATURES LEGEND

▨	BROKEN CORE	▦	CLAY SEAM	▤	LIMESTONE	■	LOST CORE

DEPTH SCALE

1 : 50

GOLDER

LOGGED:

CHECKED:

RECORD OF BOREHOLE No CRB-3

SHEET 1 OF 2

METRIC

PROJECT 1662333

G.W.P. 2002-13-00

LOCATION N 4824016.8; E 295862.2 MTM NAD 83 ZONE 10 (LAT. 43.556034; LONG. -79.610628)

ORIGINATED BY JL

DIST Central HWY QEW

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

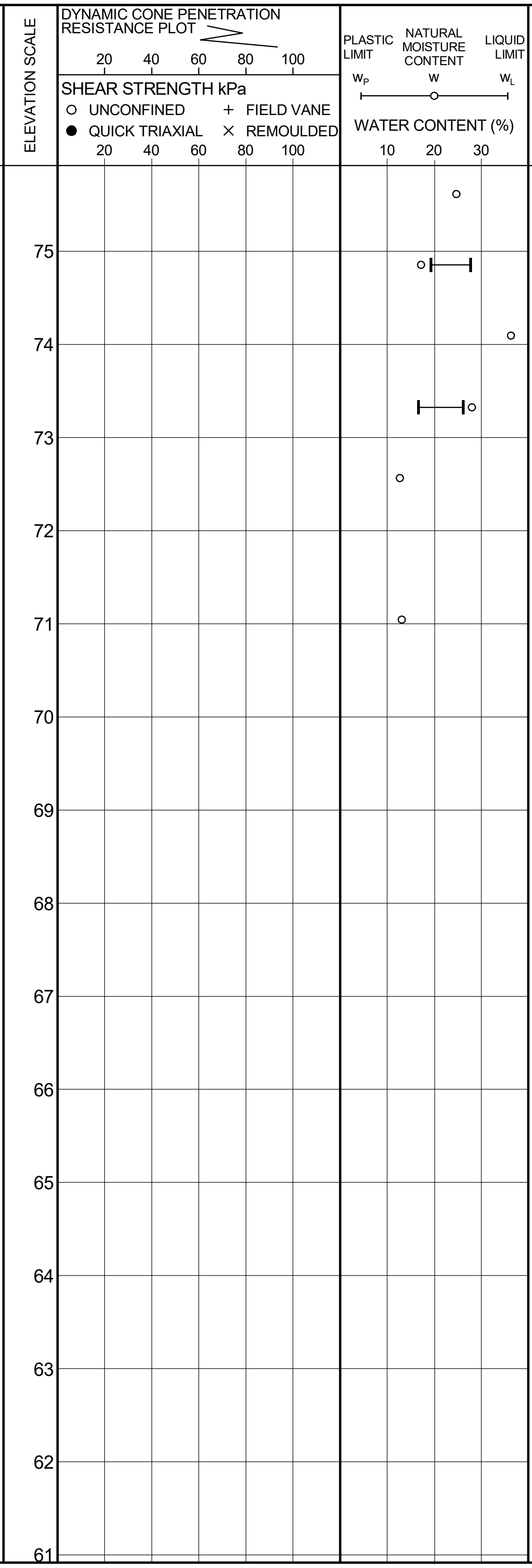
COMPILED BY MPL

DATUM Geodetic

DATE October 10-13, 2017

CHECKED BY SMM

ELEV DEPTH	DESCRIPTION (SOIL PROFILE)	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
75.9	GROUND SURFACE									
0.0	Silty sand, some gravel (FILL) Compact Brown to grey Moist to wet	1	SS	15						
75.2 0.7	Sandy clayey silt, trace gravel (FILL) Stiff Brown Moist to wet	2	SS	10						
74.3 1.6	CLAYEY SILT with SAND, trace gravel, contains organics Soft to very soft Brown to black Moist to wet	3	SS	3						
		4	SS	WH	▽	OC = 1.7%	1	41	46	12
72.9 3.0	SAND and GRAVEL, trace to some silt, trace clay Compact Brown to grey Wet - Shell fragments from a depth of about 3.8 m to a depth of 5.2 m	5	SS	17			50	35	13	2
		6	SS	14						
		7	SS	24			42	48	(10)	
70.3 5.6	Sandy CLAYEY SILT, contains shale fragments (RESIDUAL SOIL) Hard Brown Moist	8A 8B	SS	50/0.05						
69.6 6.3	SHALE (BEDROCK) Grey Bedrock cored from a depth of 7.2 m to 15.3 m For bedrock coring details, refer to Record of Drillhole CRB-3	1	RC	REC 11%			RQD = 0%			
		2	RC	REC 100%			RQD = 53%			
		3	RC	REC 98%			RQD = 93%			
		4	RC	REC 100%			RQD = 98%			
		5	RC	REC 100%			RQD = 100%			
		6	RC	REC 100%			RQD = 100%			



Continued Next Page

+³, ×³: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

GTA-MTO 001 S:\CLIENTS\MTO\QEW-CREDIT_RIVER\02_DATA\GINT\QEW-CREDIT_RIVER.GPJ GAL-GTA.GDT 2/11/19

RECORD OF BOREHOLE No CRB-3

SHEET 2 OF 2

METRIC

PROJECT 1662333

G.W.P. 2002-13-00

DIST Central HWY QEW

DATUM Geodetic

LOCATION N 4824016.8; E 295862.2 MTM NAD 83 ZONE 10 (LAT. 43.556034; LONG. -79.610628)

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

DATE October 10-13, 2017

ORIGINATED BY JL

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						
60.6	SHALE (BEDROCK)	▨	6	RC							RQD = 100%
15.3	END OF BOREHOLE NOTE: 1. Water level measured at a depth of about 2.7 m (Elev. 73.2 m) below ground surface prior to rock coring.										

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

$○^{3\%}$ STRAIN AT FAILURE

PROJECT: 1662333

RECORD OF DRILLHOLE: CRB-3

SHEET 1 OF 1

LOCATION: N 4824016.8 ;E 295862.2

INCLINATION: -90° AZIMUTH: ---

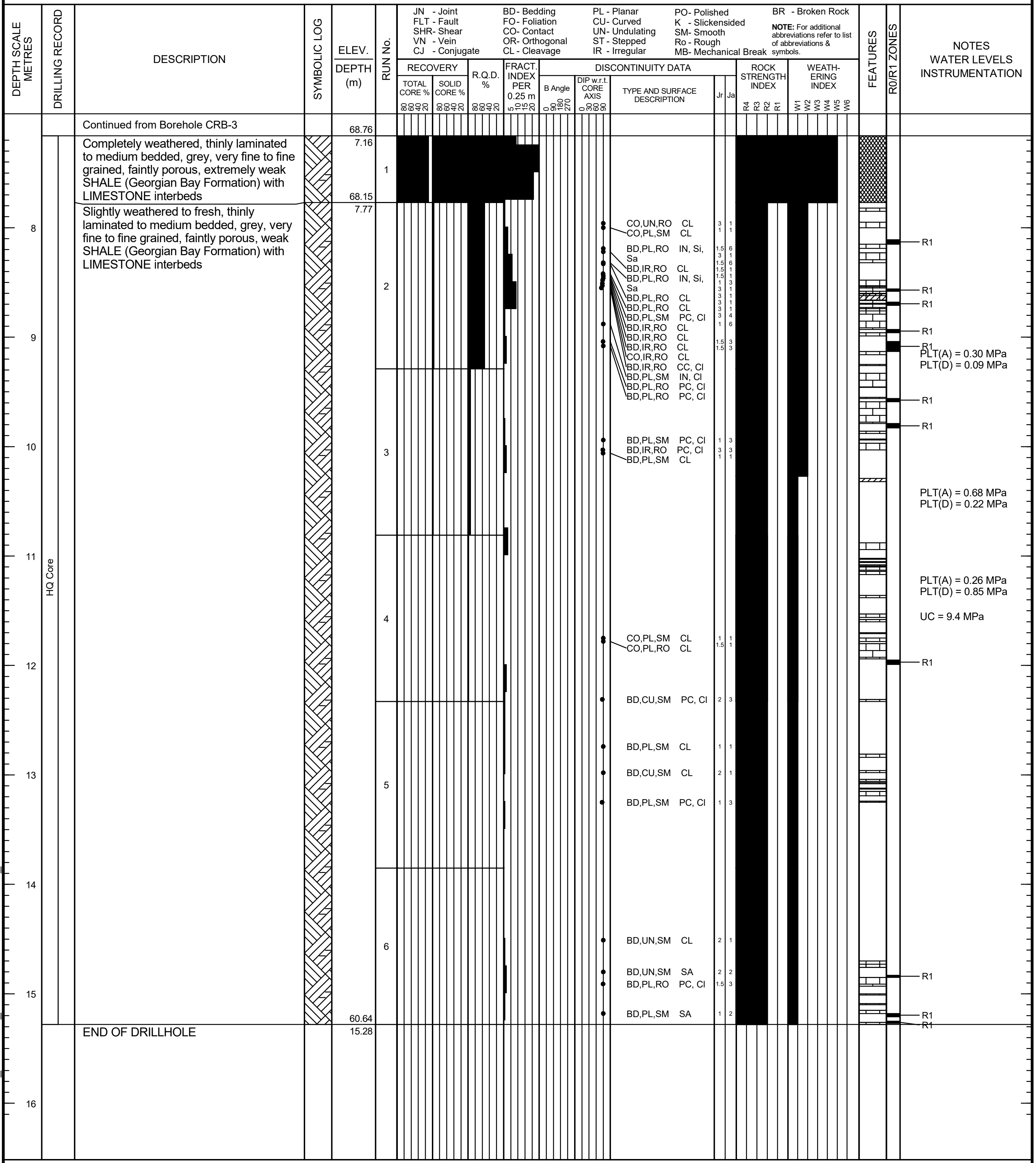
DRILLING DATE: October 10-13, 2017

DRILL RIG: CME 850 Track

DRILLING CONTRACTOR: Aardvark Drilling

DATUM: Geodetic

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



ELEV. DEPTH (m)	DESCRIPTION
68.76	Continued from Borehole CRB-3
7.16	Completely weathered, thinly laminated to medium bedded, grey, very fine to fine grained, faintly porous, extremely weak SHALE (Georgian Bay Formation) with LIMESTONE interbeds
68.15 7.77	Slightly weathered to fresh, thinly laminated to medium bedded, grey, very fine to fine grained, faintly porous, weak SHALE (Georgian Bay Formation) with LIMESTONE interbeds
60.64 15.28	END OF DRILLHOLE

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

R1
R1
R1
R1
R1
PLT(A) = 0.30 MPa
PLT(D) = 0.09 MPa
R1
R1
PLT(A) = 0.68 MPa
PLT(D) = 0.22 MPa
PLT(A) = 0.26 MPa
PLT(D) = 0.85 MPa
UC = 9.4 MPa
R1
R1
R1
R1

FEATURES LEGEND

▨ BROKEN CORE	▩ CLAY SEAM	▦ LIMESTONE	■ LOST CORE

DEPTH SCALE

1 : 50

GOLDER

LOGGED: JL

CHECKED: ML

RECORD OF BOREHOLE No CRB-3A

SHEET 1 OF 2

METRIC

PROJECT 1662333

G.W.P. 2002-13-00

DIST Central HWY QEW

DATUM Geodetic

LOCATION N 4824025.6; E 295844.6 MTM NAD 83 ZONE 10 (LAT. 43.556113; LONG. -79.610847)

BOREHOLE TYPE CME 55, 210 mm O.D., 108 mm I.D. Hollow Stem Augers (Auto Hammer)

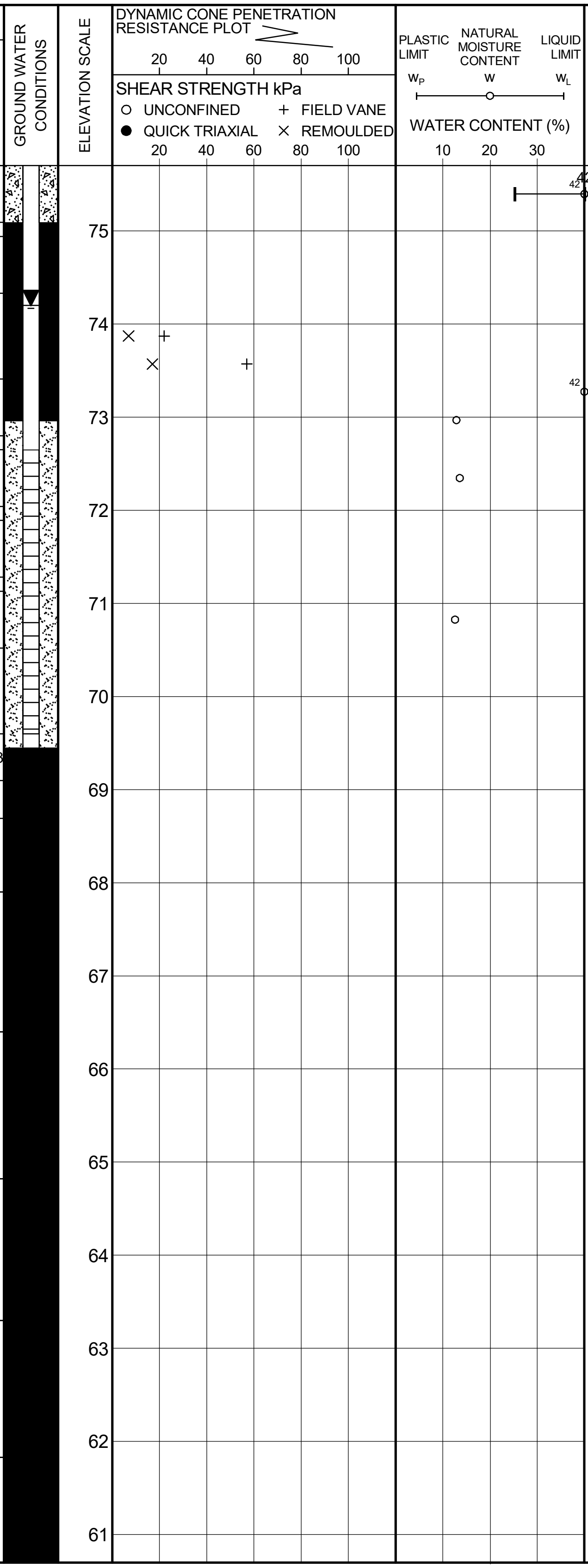
DATE February 9, 2018

ORIGINATED BY JL

COMPILED BY KN

CHECKED BY SMM

SOIL PROFILE ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES "N" VALUES	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
75.7	GROUND SURFACE					
0.0	Sandy SILTY CLAY, contains rootlets / organics Soft to stiff Brown Moist, becoming wet at a depth of about 2.3 m	1	SS	5		0 27 57 16
		2	SS	4		
		3A	SS	3	OC= 3.1%	
73.1 2.6	SAND and GRAVEL, some silt, trace clay Compact Brown to grey Wet	3B				
		4	SS	29		
		5	SS	25		
	- Clayey silt pocket encountered at a depth of about 4.7 m below ground surface	6	SS	25		43 39 14 4
69.6 6.1	SHALE (BEDROCK) Grey Bedrock cored from a depth of 7.0 m to 15.8 m For bedrock coring details, refer to Record of Drillhole CRB-3A	7	SS	50/0.13		
		1	RC	REC 100%		RQD = 85%
		2	RC	REC 100%		RQD = 67%
		3	RC	REC 100%		RQD = 89%
		4	RC	REC 98%		RQD = 83%
		5	RC	REC 100%		RQD = 99%
		6	RC	REC 100%		RQD = 90%



Continued Next Page

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

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

GOLDER

Foundation Design

PROJECT 1662333	**RECORD OF BOREHOLE No CRB-3A** SHEET 2 OF 2		**METRIC**
G.W.P. 2002-13-00	LOCATION N 4824025.6; E 295844.6 MTM NAD 83 ZONE 10 (LAT. 43.556113; LONG. -79.610847)		ORIGINATED BY JL
DIST Central HWY QEW	BOREHOLE TYPE CME 55, 210 mm O.D., 108 mm I.D. Hollow Stem Augers (Auto Hammer)		COMPILED BY KN
DATUM Geodetic	DATE February 9, 2018		CHECKED BY SMM

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/m^3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SHALE (BEDROCK) Grey	▨	6	RC	REC 100%	■					RQD = 90%
59.9		▨	7	RC	REC 100%	■	60				RQD = 93%
15.8	END OF BOREHOLE										

NOTES:

1. Water level measured at a depth of about 0.9 m below ground surface prior to rock coring.

2. Water level measured in standpipe piezometer:

Date	Depth (m)	Elev. (m)
12/03/18	1.0	74.7
30/04/18	0.7	75.0
06/11/18	1.5	74.2

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

PROJECT: 1662333

RECORD OF DRILLHOLE: CRB-3A

SHEET 1 OF 1

LOCATION: N 4824025.6 ;E 295844.6
INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: February 9, 2018
DRILL RIG: CME 55 Track
DRILLING CONTRACTOR: Geo-Environmental Drilling

DATUM: Geodetic

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

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. DEPTH (m)	RUN No.
		Continued from Borehole CRB-3A	68.69	
	HQ Core	Slightly weathered to fresh, thinly laminated to medium bedded, grey, very fine to fine grained, faintly porous, weak SHALE (Georgian Bay Formation) with LIMESTONE interbeds	7.01	1
8				2
9		Fresh, thinly to medium bedded, grey, fine grained, faintly porous, weak SHALE (Georgian Bay Formation) with LIMESTONE interbeds		
10				3
11				4
12				
13				5
14				6
15				7
		END OF DRILLHOLE	59.92 15.78	
16				

Column headings: RECOVERY (TOTAL CORE %, SOLID CORE %; 80 60 40 20), R.Q.D. % (80 60 40 20), FRACT. INDEX PER 0.25 m (5 10 15 20), DISCONTINUITY DATA (B Angle 0 90 180 270; DIP w.r.t. CORE AXIS 0 30 60 90; TYPE AND SURFACE DESCRIPTION; Jr; Ja), ROCK STRENGTH INDEX (R4 R3 R2 R1), WEATHERING INDEX (W1 W2 W3 W4 W5 W6), FEATURES, R0/R1 ZONES, NOTES WATER LEVELS INSTRUMENTATION

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

R0/R1 ZONES: R1, R0, R1, R1, R1, R0, R0, R1, R1

NOTES:
PLT(D) = 0.62
PLT(A) = 0.46
UC = 8.9 MPa
PLT(D) = 0.06
PLT(A) = 1.35
UC = 16.9 MP
PLT(A) = 0.78
PLT(D) = 0.36

FEATURES LEGEND

▨ BROKEN CORE ▩ CLAY SEAM ▤ LIMESTONE ■ LOST CORE

DEPTH SCALE
1 : 50

GOLDER

LOGGED: JL
CHECKED: DM

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

PROJECT 1662333

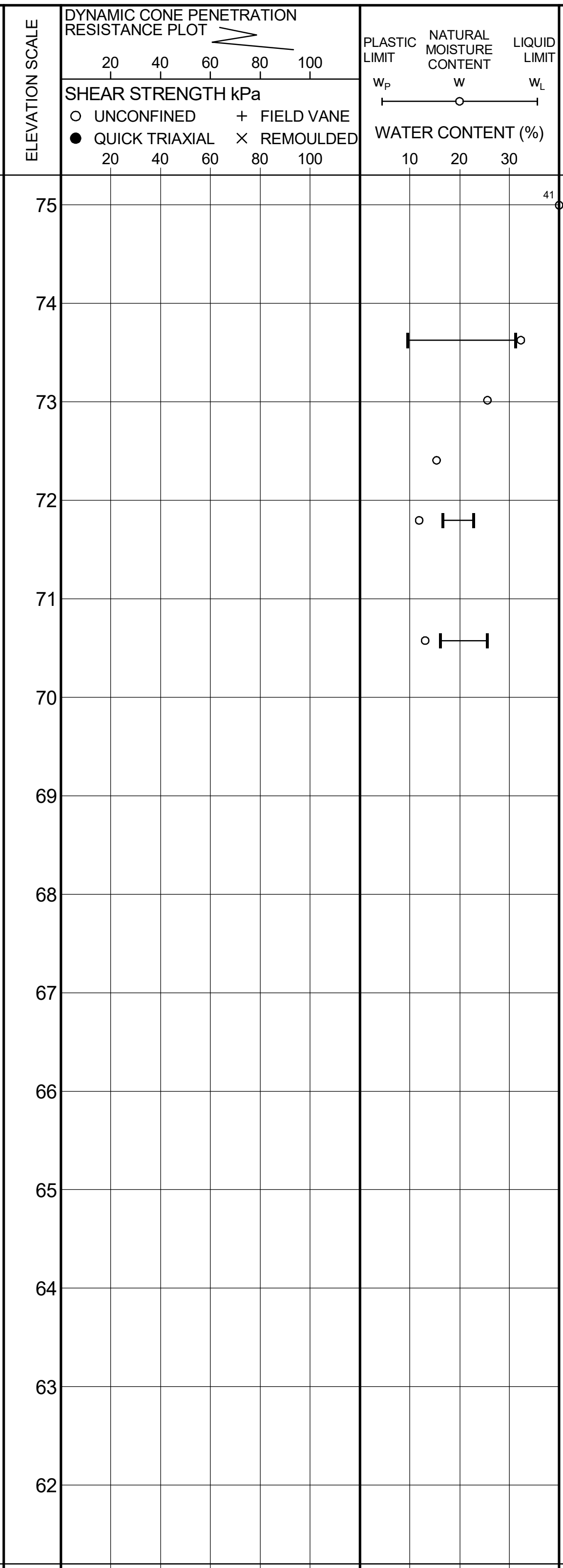
RECORD OF BOREHOLE No CRB-3C

SHEET 1 OF 2

METRIC

G.W.P. 2002-13-00
LOCATION N 4824028.3; E 295837.7 MTM NAD 83 ZONE 10 (LAT. 43.556138; LONG. -79.610932)
ORIGINATED BY JL
DIST Central HWY QEW
BOREHOLE TYPE Cont. Split Spoon & 73 mm O.D., 60 mm I.D., Washbore Casing (Cathead/Safety Hammer)
COMPILED BY MPL
DATUM Geodetic
DATE January 26, 2018
CHECKED BY SMM

SOIL PROFILE 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
75.3	GROUND SURFACE						
0.0	CLAYEY SILT with SAND, some gravel, contains organics / rootlets Soft to stiff Brown to grey Wet	1	SS	6	▽		
	- Cobble encountered at 1.2 m depth	2	SS	4			
		3	SS	3		OC= 2.4%	13 35 41 11
72.7	- Cobble encountered at 2.4 m depth	4	SS	14			
2.6	SAND and GRAVEL to SAND, some gravel, some silt, trace to some clay, contains cobbles Compact to very dense Brown and grey to grey Wet	5	SS	46			
		6	SS	55			40 36 18 6
	- Clayey silt pockets / layers encountered in samples SS6 and SS8	7	SS	72/0.10			
		8	SS	23			15 57 20 8
69.1		9	SS	100/0.05			
6.2	SHALE (BEDROCK) Grey	1	RC	REC 67%			RQD = 22%
	Bedrock cored from a depth of 6.4 m to 14.1 m	2	RC	REC 100%			RQD = 53%
	For bedrock coring details, refer to Record of Drillhole CRB-3C	3	RC	REC 100%			RQD = 43%
		4	RC	REC 91%			RQD = 76%
		5	RC	REC 100%			RQD = 75%
		6	RC	REC 100%			RQD = 100%
		7	RC	REC 100%			RQD = 100%
		8	RC	REC 100%			RQD = 100%
61.2		9	RC	REC 100%			RQD = 71%
14.1	END OF BOREHOLE						



Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No CRB-3C

SHEET 2 OF 2

METRIC

PROJECT 1662333

G.W.P. 2002-13-00

LOCATION N 4824028.3; E 295837.7 MTM NAD 83 ZONE 10 (LAT. 43.556138; LONG. -79.610932)

ORIGINATED BY JL

DIST Central HWY QEW

BOREHOLE TYPE Cont. Split Spoon & 73 mm O.D., 60 mm I.D., Washbore Casing (Cathead/Safety Hammer)

COMPILED BY MPL

DATUM Geodetic

DATE January 26, 2018

CHECKED BY SMM

SOIL PROFILE			SAMPLES					DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION --- *CONTINUED FROM PREVIOUS PAGE* ---	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE				GR	SA	SI	CL
	NOTES: 1. Water level measured at ground surface (Elev. 75.3 m) at start of shift on January 25, 2018 2. Water level measured at a depth of 0.7 m below ground surface (Elev. 74.6 m) prior to rock coring on January 26, 2018													

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

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

PROJECT: 1662333

RECORD OF DRILLHOLE: CRB-3C

SHEET 1 OF 1

LOCATION: N 4824028.3 ;E 295837.7

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: January 26, 2018
DRILL RIG: Portable/Tripod
DRILLING CONTRACTOR: OGS Drilling

DATUM: Geodetic

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

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

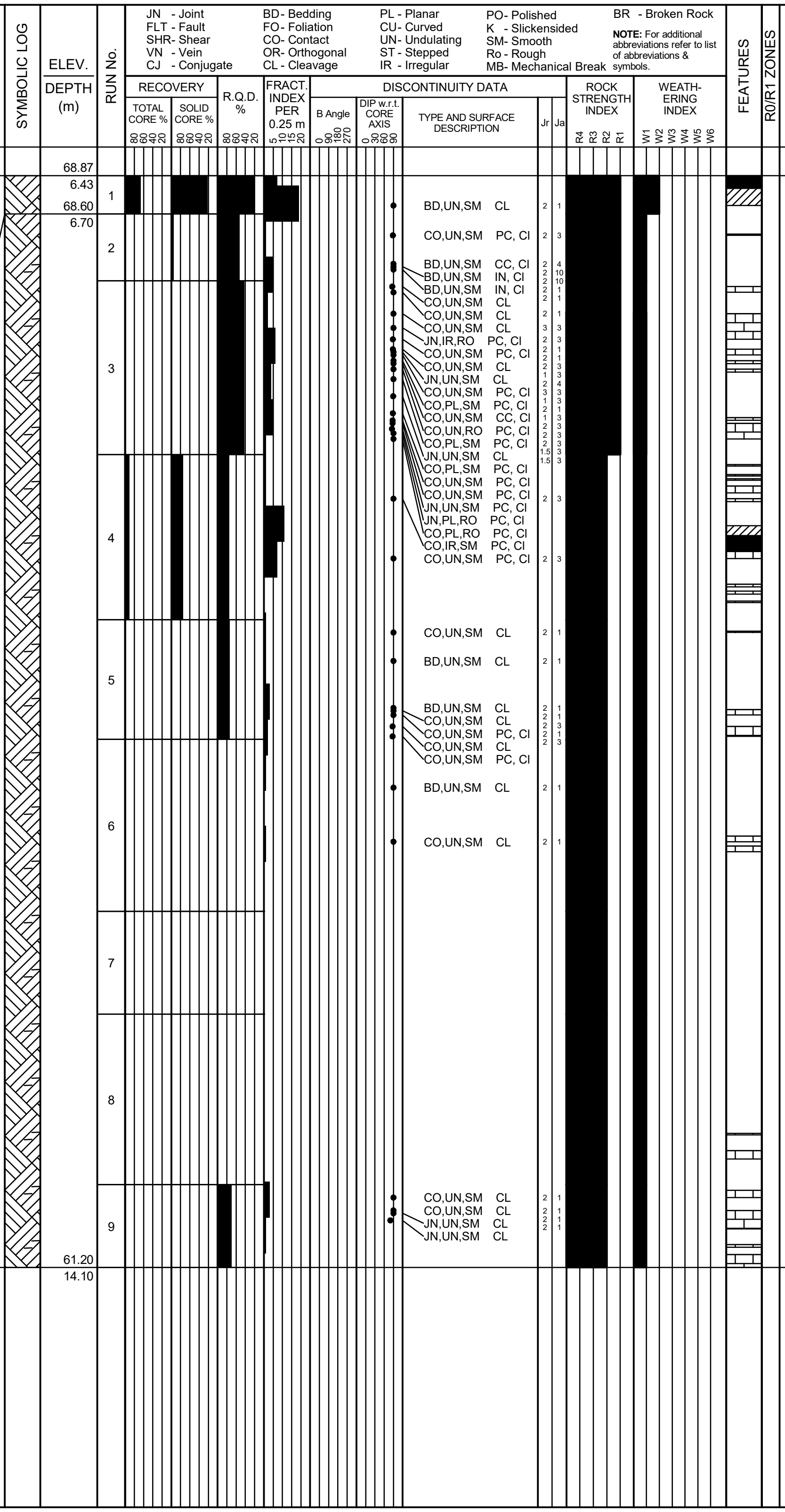
PL - Planar
CU- Curved
UN- Undulating
ST - Stepped
IR - Irregular

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

BR - Broken Rock

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. DEPTH (m)	RUN No.
		Continued from Borehole CRB-3C	68.87	
	NQ Rock Core	Slightly weathered to fresh, thinly laminated to medium bedded, grey, very fine to fine grained, faintly porous, very weak to weak SHALE (Georgian Bay Formation) with LIMESTONE interbeds	6.43	1
7		Slightly weathered to fresh, thinly bedded, grey, fine grained, non-porous, very weak to weak SHALE (Georgian Bay Formation) with LIMESTONE interbeds	68.60 6.70	2
8				3
9				4
10				5
11				6
12				7
13				8
14			61.20 14.10	9
		END OF DRILLHOLE		
15				



DISCONTINUITY DATA — TYPE AND SURFACE DESCRIPTION

BD,UN,SM CL
CO,UN,SM PC, CI
BD,UN,SM CC, CI
BD,UN,SM IN, CI
BD,UN,SM IN, CI
CO,UN,SM CL
CO,UN,SM CL
CO,UN,SM CL
JN,IR,RO PC, CI
CO,UN,SM PC, CI
CO,UN,SM CL
JN,UN,SM CL
CO,UN,SM PC, CI
CO,PL,SM PC, CI
CO,UN,SM CC, CI
CO,UN,RO PC, CI
CO,PL,SM PC, CI
JN,UN,SM CL
CO,PL,SM PC, CI
CO,UN,SM PC, CI
CO,UN,SM PC, CI
JN,UN,SM PC, CI
JN,PL,RO PC, CI
CO,PL,RO PC, CI
CO,IR,SM PC, CI
CO,UN,SM PC, CI
CO,UN,SM CL
BD,UN,SM CL
BD,UN,SM CL
CO,UN,SM CL
CO,UN,SM PC, CI
CO,UN,SM CL
CO,UN,SM PC, CI
BD,UN,SM CL
CO,UN,SM CL
CO,UN,SM CL
CO,UN,SM CL
JN,UN,SM CL
JN,UN,SM CL

Jr	Ja
2	1
2	3
2	4
2	10
2	10
2	1
2	1
2	1
3	3
2	3
2	1
2	1
2	3
1	3
2	4
3	3
1	3
2	1
1	3
2	3
2	3
2	3
1.5	3
1.5	3
2	3
2	3
2	1
2	1
2	1
2	1
2	3
2	1
2	3
2	1
2	1
2	1
2	1
2	1
2	1

NOTES WATER LEVELS INSTRUMENTATION

UC = 114.1 MPa

PLT(D) = 0.25 MPa
PLT(A) = 0.28 MPa

PLT(D) = 0.24 MPa
PLT(A) = 0.60 MPa

PLT(D) = 0.28 MPa

PLT(A) = 0.58 MPa

FEATURES LEGEND

▨ BROKEN CORE
▩ CLAY SEAM
▦ LIMESTONE
■ LOST CORE

DEPTH SCALE
1 : 50

GOLDER

LOGGED: JL
CHECKED: AB

GOLDER

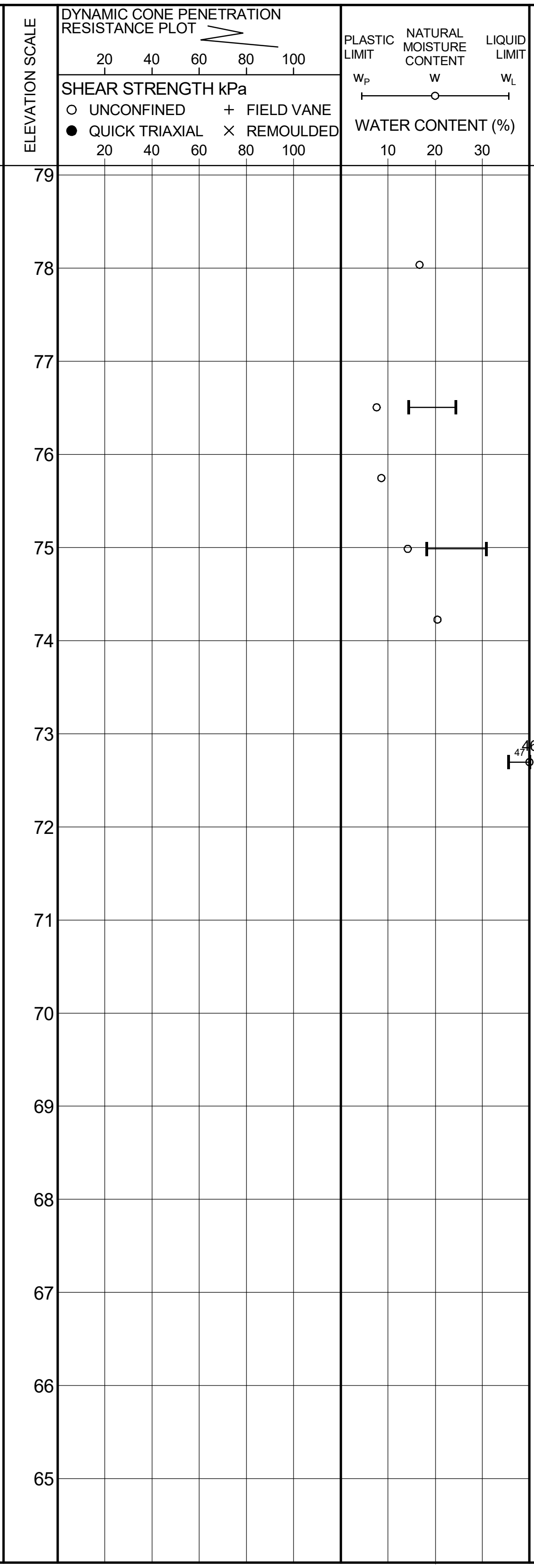
RECORD OF BOREHOLE No CRB-4

SHEET 1 OF 2

METRIC

PROJECT 1662333			
G.W.P. 2002-13-00	LOCATION N 4824135.1; E 295902.0 MTM NAD 83 ZONE 10 (LAT. 43.557099; LONG. -79.610138)	ORIGINATED BY JL
DIST Central HWY QEW	BOREHOLE TYPE CME 55, 159 mm O.D., 70 mm I.D. Hollow Stem Augers (Auto Hammer)	COMPILED BY KN
DATUM Geodetic	DATE February 16, 2018	CHECKED BY SMM

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
79.1	GROUND SURFACE							
0.0	Clayey silt with sand, trace to some gravel, contains rootlets, contains shale fragments (FILL) Firm to hard Brown, becoming grey at 3.1 m Moist		1	SS	6			
			2	SS	13			
			3	SS	8			
			4	SS	23			7 41 37 15
			5	SS	31			
75.4 3.7	Sandy CLAYEY SILT, some gravel, contains shale fragments Stiff Moist, becoming wet at 4.6 m		6	SS	11	▽		14 27 42 17
			7	SS	13			
73.0 6.2	ORGANIC CLAYEY SILT with SAND, trace gravel, contains sand lenses Soft Brown Wet		8	SS	3			2 32 56 10
72.1 7.0	SHALE (BEDROCK) Grey Bedrock cored from a depth of 7.2 m to 15.3 m For bedrock coring details, refer to Record of Drillhole CRB-4		1	RC	REC 100%			RQD = 0%
			9	SS	50/0.10			
			2	RC	REC 87%			RQD = 53%
			3	RC	REC 97%			RQD = 77%
			4	RC	REC 100%			RQD = 79%
			5	RC	REC 100%			RQD = 86%
			6	RC	REC 100%			RQD = 97%



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

RECORD OF BOREHOLE No CRB-4

SHEET 2 OF 2

METRIC

PROJECT 1662333			
G.W.P. 2002-13-00	LOCATION N 4824135.1; E 295902.0 MTM NAD 83 ZONE 10 (LAT. 43.557099; LONG. -79.610138)	ORIGINATED BY JL	
DIST Central	HWY QEW	BOREHOLE TYPE CME 55, 159 mm O.D., 70 mm I.D. Hollow Stem Augers (Auto Hammer)	COMPILED BY KN
DATUM Geodetic	DATE February 16, 2018	CHECKED BY SMM	

SOIL PROFILE			SAMPLES					DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION --- CONTINUED FROM PREVIOUS PAGE ---	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE				
63.8	SHALE (BEDROCK)	▨	6	RC			64				RQD = 97%
15.3	END OF BOREHOLE NOTES: 1. Water level encountered during drilling at a depth of about 4.6 m (Elev. 74.5 m) below ground surface. 2. Water level measured in open borehole at a depth of about 3.8 m (Elev. 75.4 m) below ground surface prior to rock coring.										

GTA-MTO 001 S:\CLIENTS\MTO\QEW-CREDIT_RIVER\02_DATA\GINT\QEW-CREDIT_RIVER.GPJ GAL-GTA.GDT 2/11/19

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

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

PROJECT: 1662333

RECORD OF DRILLHOLE: CRB-4

SHEET 1 OF 1

LOCATION: N 4824135.1 ;E 295902.0

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: February 16, 2018

DRILL RIG: CME 55 Track

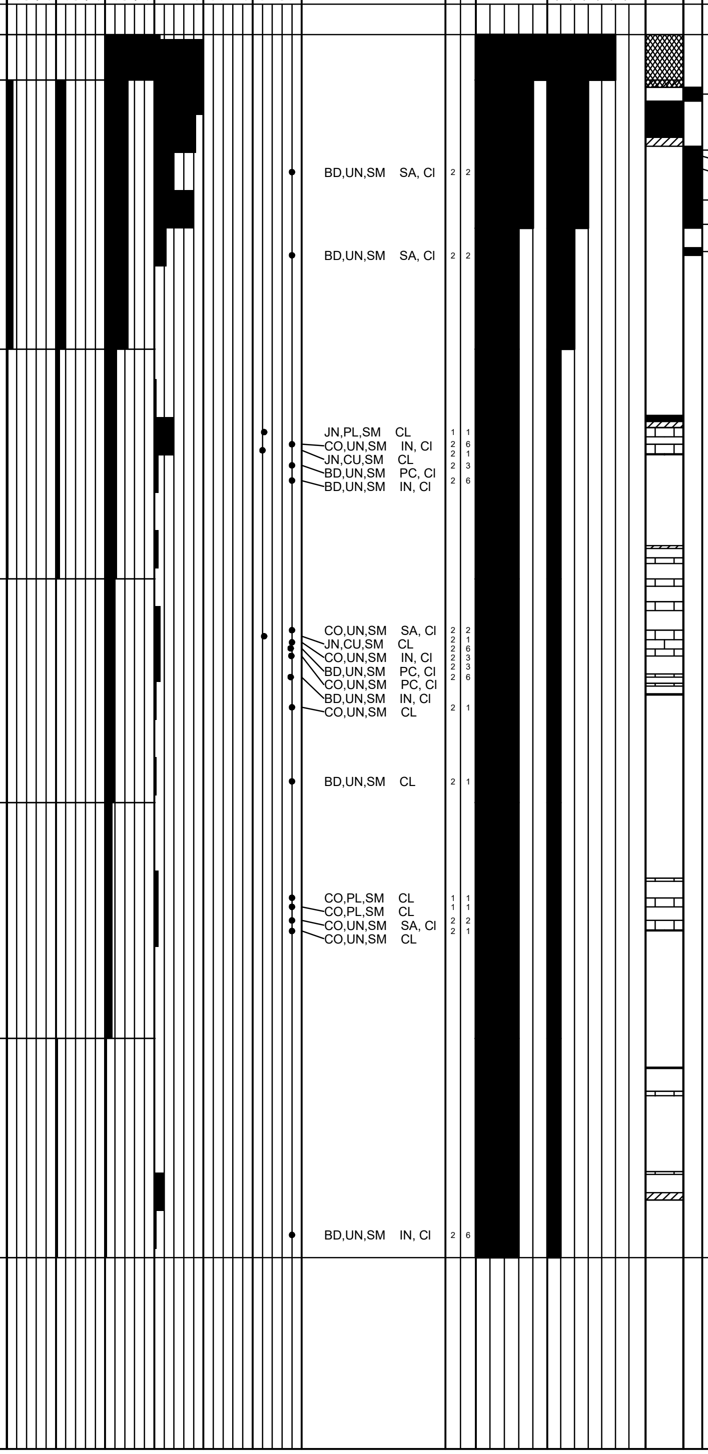
DRILLING CONTRACTOR: Geo-Environmental Drilling

DATUM: Geodetic

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

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. DEPTH (m)	RUN No.
		Continued from Borehole CRB-4	71.88	
	HQ Core	Completely weathered, thinly laminated to medium bedded, brown-grey, very fine to fine grained, faintly porous, extremely weak SHALE (Georgian Bay Formation)	71.58 7.22	1
8		Moderately weathered to fresh, thinly to medium bedded, brown-grey, fine grained, faintly porous, very weak to weak SHALE (Georgian Bay Formation) with LIMESTONE interbeds	71.58 7.52	2
9		Fresh, thinly to medium bedded, grey, fine grained, faintly porous, weak SHALE (Georgian Bay Formation) with LIMESTONE interbeds	69.80 9.30	3, 4, 5, 6
15		END OF DRILLHOLE	63.79 15.31	



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

NOTES WATER LEVELS INSTRUMENTATION

PLT(D) = 0.04 MPa
PLT(A) = 1.02 MPa
PLT(A) = 2.14 MPa

PLT(D) = 0.89 MPa

UC = 15.5 MPa

PLT(D) = 0.08 MPa

PLT(A) = 0.23 MPa

FEATURES LEGEND

▨ BROKEN CORE

▩ CLAY SEAM

▦ LIMESTONE

■ LOST CORE

DEPTH SCALE
1 : 50

GOLDER

LOGGED: JL
CHECKED: DM

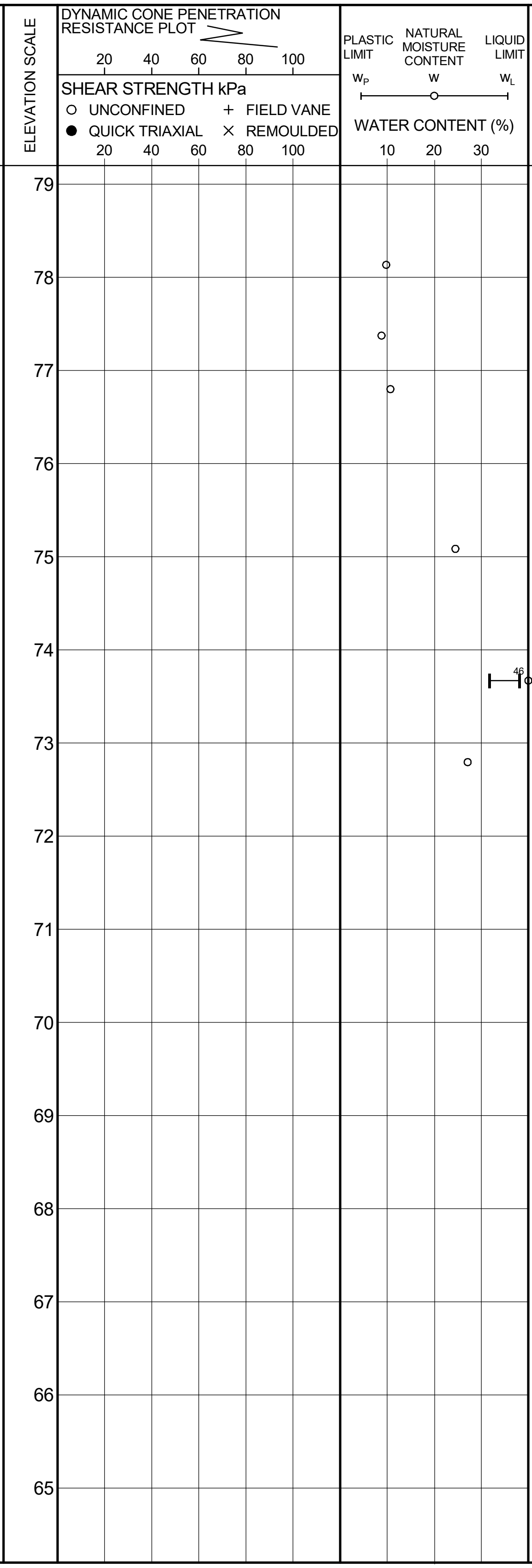
RECORD OF BOREHOLE No CRB-5

SHEET 1 OF 2

METRIC

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

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



Continued Next Page

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

GOLDER

RECORD OF BOREHOLE No CRB-5

SHEET 2 OF 2

METRIC

PROJECT 1662333

G.W.P. 2002-13-00

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

ORIGINATED BY JL

DIST Central HWY QEW

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

COMPILED BY KN

DATUM Geodetic

DATE February 13, 2018

CHECKED BY SMM

SOIL PROFILE			SAMPLES			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 ---										
63.7	SHALE (BEDROCK) Grey	▨	6	RC	REC 100%		64				RQD = 99%
15.5	END OF BOREHOLE NOTES: 1. Water level encountered during drilling at a depth of about 3.7 m (Elev. 75.5 m)below ground surface. 2. Water level measured in open borehole at a depth of about 4.3 m (Elev. 74.9 m) below ground surface prior to rock coring.										

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

○$^{3\%}$ STRAIN AT FAILURE

PROJECT: 1662333

RECORD OF DRILLHOLE: CRB-5

SHEET 1 OF 1

LOCATION: N 4824128.9 ;E 295914.2

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: February 13, 2018

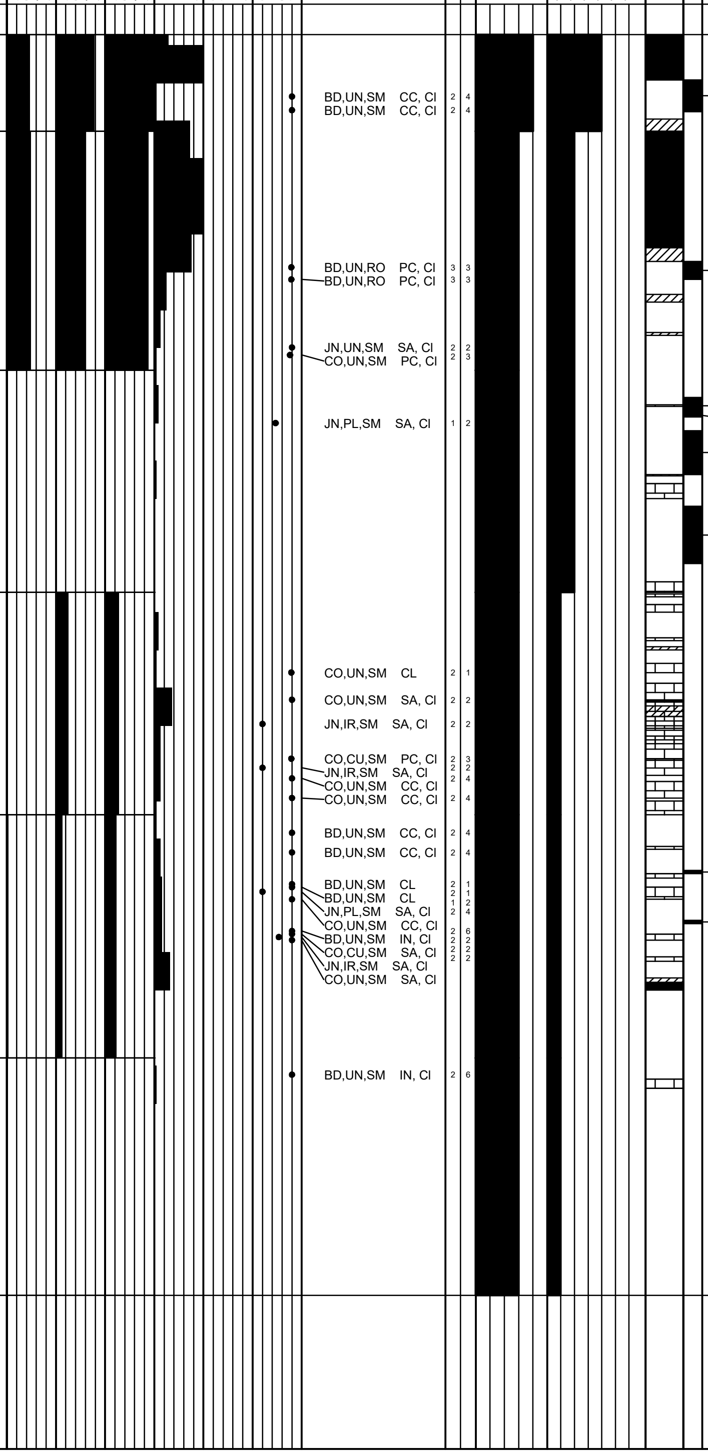
DRILL RIG: CME 55 Track

DRILLING CONTRACTOR: Geo-Environmental Drilling

DATUM: Geodetic

JN - Joint	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.
		Continued from Borehole CRB-5	72.02	
	HQ Core	Highly weathered, thinly laminated to medium bedded, brown to grey, very fine to fine grained, faintly porous, very weak SHALE (Georgian Bay Formation)	7.18 71.38	1
8	HQ Core	Slightly weathered to fresh, thinly laminated to medium bedded, grey, very fine to fine grained, faintly porous, weak SHALE (Georgian Bay Formation) with LIMESTONE interbeds	7.82 69.80	2
9–15	HQ Core	Slightly weathered to fresh, thinly to medium bedded, grey, fine grained, faintly porous, very weak to weak SHALE (Georgian Bay Formation) with LIMESTONE interbeds	9.40 63.68	3, 4, 5, 6
		END OF DRILLHOLE	15.52	



DISCONTINUITY DATA

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

NOTES WATER LEVELS INSTRUMENTATION

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

PLT(A) = 1.45 MPa

PLT(D) = 0.07 MPa

UC = 18.6 MPa

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

FEATURES LEGEND

▨ BROKEN CORE	▦ CLAY SEAM	▤ LIMESTONE	■ LOST CORE

DEPTH SCALE

1 : 50

GOLDER

LOGGED: JL

CHECKED: DM

GOLDER

RECORD OF BOREHOLE No CRB-5A

SHEET 1 OF 2

METRIC

PROJECT 1662333

G.W.P. 2002-13-00

LOCATION N 4824130.9; E 295910.6 MTM NAD 83 ZONE 10 (LAT. 43.557062; LONG. -79.610032)

ORIGINATED BY JL

DIST Central HWY QEW

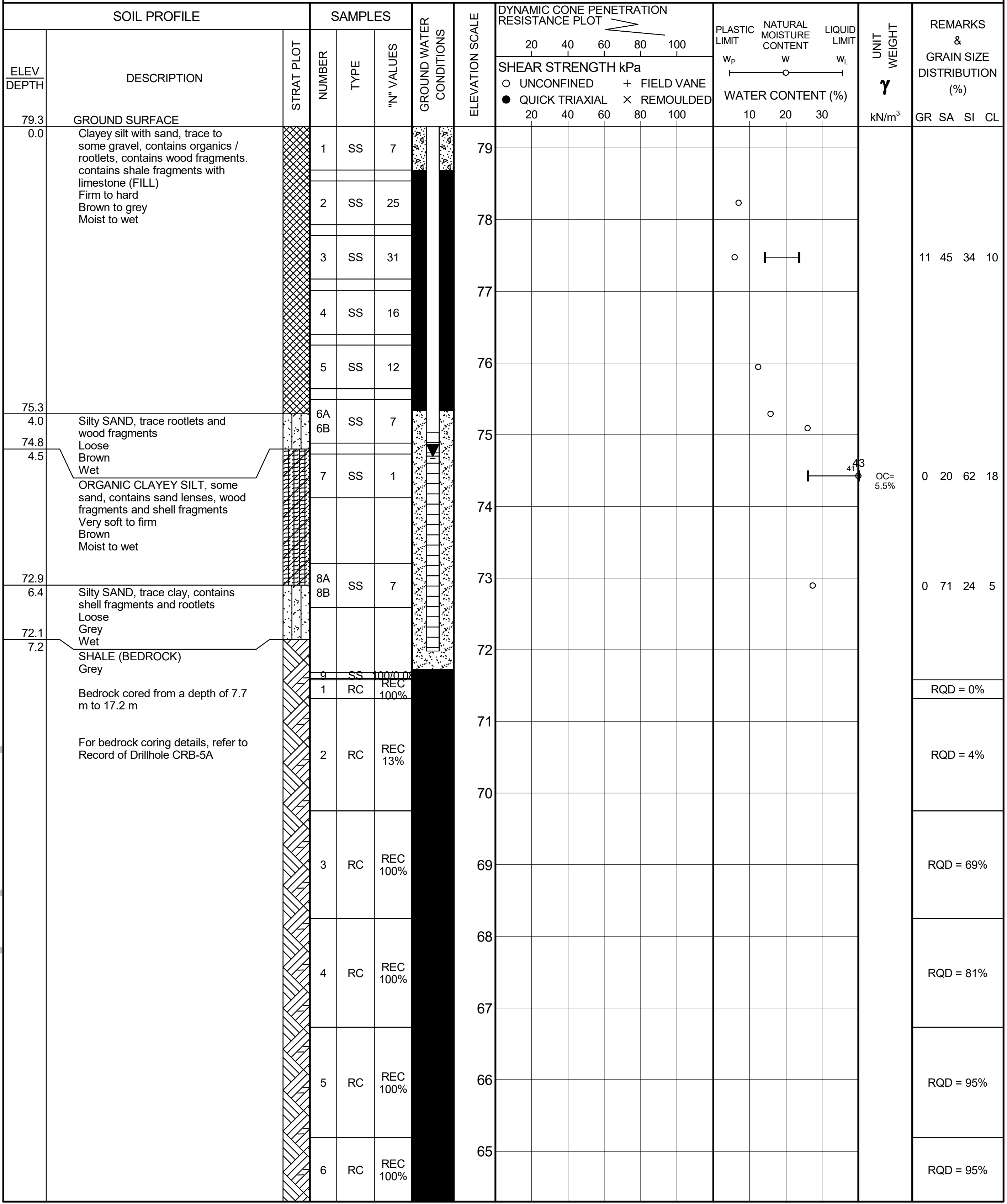
BOREHOLE TYPE CME 55, 159 mm O.D., 70 mm I.D. Hollow Stem Augers (Auto Hammer)

COMPILED BY KN

DATUM Geodetic

DATE February 15 and 16, 2018

CHECKED BY SMM



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

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

Continued Next Page

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

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

GTA-MTO 001 S:\CLIENTS\MTO\QEW-CREDIT_RIVER\02_DATA\GINT\QEW-CREDIT_RIVER.GPJ GAL-GTA.GDT 2/21/19

PROJECT 1662333

RECORD OF BOREHOLE No CRB-5A

SHEET 2 OF 2

METRIC

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

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

NOTES:

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

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

3. Groundwater level measurements in piezometer:

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

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

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

PROJECT: 1662333

RECORD OF DRILLHOLE: CRB-5A

SHEET 1 OF 2

LOCATION: N 4824130.9 ;E 295910.6

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: February 15 and 16, 2018

DRILL RIG: CME 55 Track

DRILLING CONTRACTOR: Geo-Environmental Drilling

DATUM: Geodetic

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

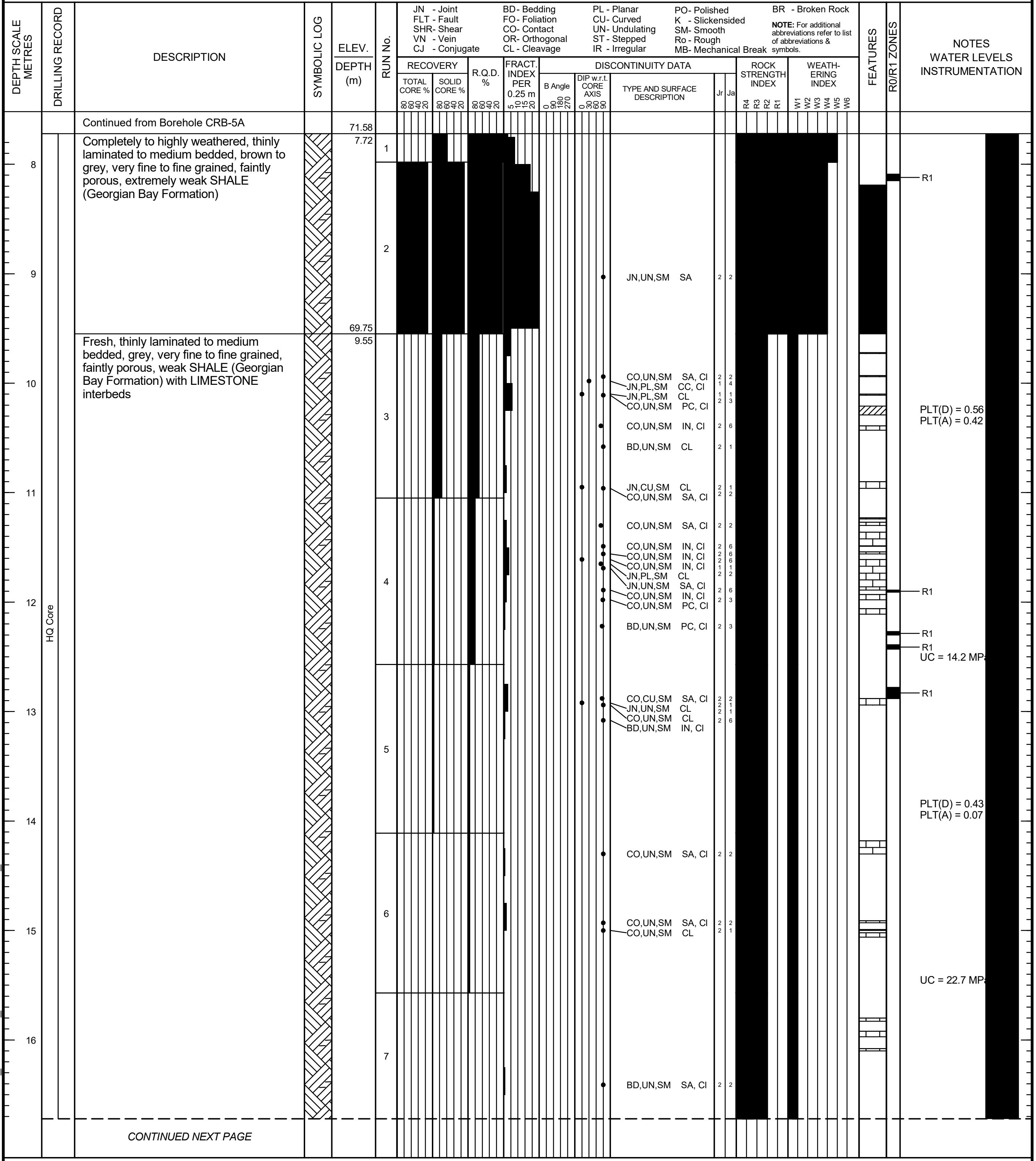
BD- Bedding
FO- Foliation
CO- Contact
OR- Orthogonal
CL - Cleavage

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

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

BR - Broken Rock

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



ELEV. / DEPTH (m)	DESCRIPTION
	Continued from Borehole CRB-5A
71.58 7.72	Completely to highly weathered, thinly laminated to medium bedded, brown to grey, very fine to fine grained, faintly porous, extremely weak SHALE (Georgian Bay Formation)
69.75 9.55	Fresh, thinly laminated to medium bedded, grey, very fine to fine grained, faintly porous, weak SHALE (Georgian Bay Formation) with LIMESTONE interbeds

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

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

UC = 14.2 MP

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

UC = 22.7 MP

CONTINUED NEXT PAGE

FEATURES LEGEND

▨ BROKEN CORE	▩ CLAY SEAM	▦ LIMESTONE	■ LOST CORE

DEPTH SCALE
1 : 50

GOLDER

LOGGED: JL
CHECKED: DM

PROJECT: 1662333

RECORD OF DRILLHOLE: CRB-5A

SHEET 2 OF 2

LOCATION: N 4824130.9 ;E 295910.6

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: February 15 and 16, 2018

DRILL RIG: CME 55 Track

DRILLING CONTRACTOR: Geo-Environmental Drilling

DATUM: Geodetic

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

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

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

FEATURES LEGEND

▨	BROKEN CORE	▩	CLAY SEAM	▦	LIMESTONE	■	LOST CORE

DEPTH SCALE
1 : 50

GOLDER

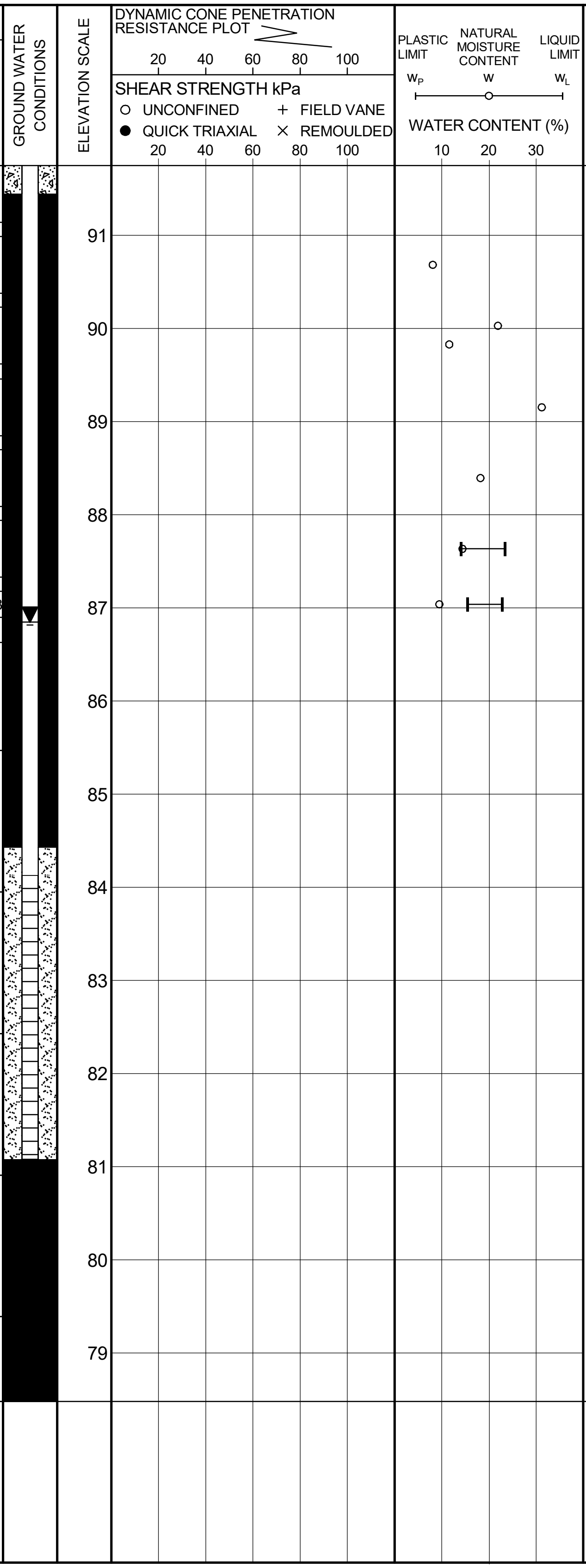
LOGGED: JL
CHECKED: DM

RECORD OF BOREHOLE No CRB-6

SHEET 1 OF 2

METRIC

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



ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
91.7	GROUND SURFACE					
0.0	Silty sand, trace to some gravel, trace clay, contains brick fragments (FILL) Loose to compact Brown Moist	1	SS	11		
		2	SS	5		7 67 24 2
90.0 / 1.7	Sandy CLAYEY SILT, trace to some gravel Firm to stiff brown Moist to wet - Mottled brown-grey below a depth of about 2.3 m	3A 3B	SS	5		
		4	SS	7		
		5	SS	9		
	- Becoming gravelly at a depth of about 3.7 m - Auger grinding at a depth of about 3.7 m	6	SS	5		10 26 44 20
87.3 / 4.4	Sandy CLAYEY SILT, some shale fragments (RESIDUAL SOIL) Hard Grey Moist	7	SS	50/0.13		
86.9 / 4.8	SHALE (BEDROCK) Grey Bedrock cored from a depth of 5.1 m to 13.3 m For bedrock coring details, refer to Record of Drillhole CRB-6	1	RC	REC 86%		RQD = 47%
		2	RC	REC 100%		RQD = 95%
		3	RC	REC 100%		RQD = 100%
		4	RC	REC 100%		RQD = 95%
		5	RC	REC 100%		RQD = 100%
		6	RC	REC 96%		RQD = 96%
78.4 / 13.3	END OF BOREHOLE					

Continued Next Page

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

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

PROJECT 1662333

RECORD OF BOREHOLE No CRB-6

SHEET 2 OF 2

METRIC

G.W.P. 2002-13-00

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

ORIGINATED BY JL

DIST Central HWY QEW

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

COMPILED BY MPL

DATUM Geodetic

DATE October 18-20, 2017

CHECKED BY SMM

SOIL PROFILE			SAMPLES								
ELEV DEPTH	DESCRIPTION	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/m^3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---										

NOTES:

1. Borehole dry prior to rock coring.

2. Water level measured in standpipe piezometer:

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

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

○ $^{3\%}$ STRAIN AT FAILURE

PROJECT: 1662333

RECORD OF DRILLHOLE: CRB-6

SHEET 1 OF 1

LOCATION: N 4824196.7 ;E 295929.5

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: October 18-20, 2017
DRILL RIG: CME 850 Track
DRILLING CONTRACTOR: Aardvark Drilling

DATUM: Geodetic

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

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

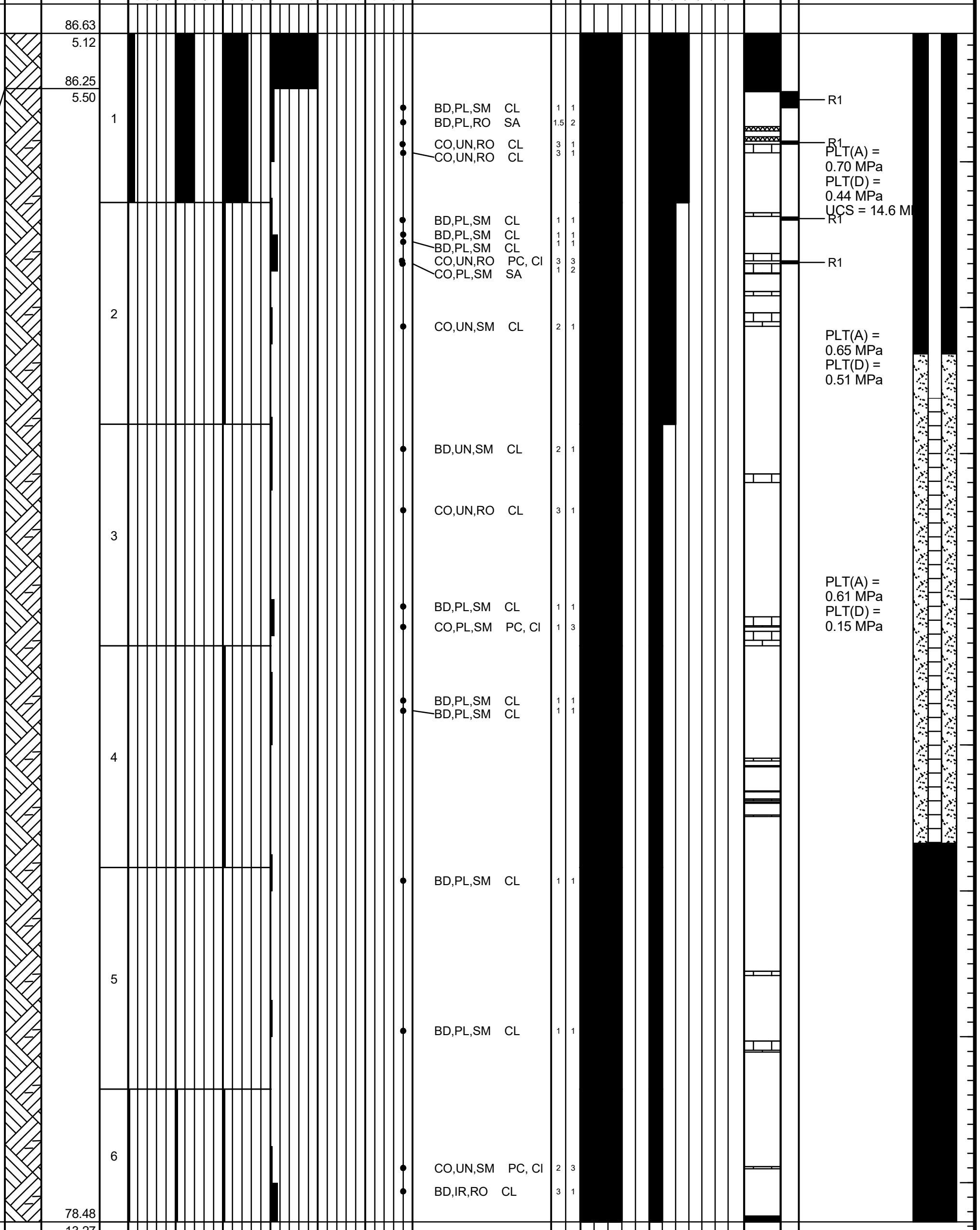
PL - Planar
CU- Curved
UN- Undulating
ST - Stepped
IR - Irregular

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.
		Continued from Borehole CRB-6		86.63	
	HQ Casing	Moderately weathered to fresh, thinly laminated to medium bedded, grey, very fine to fine grained, faintly porous, weak SHALE (Georgian Bay Formation) with LIMESTONE interbeds		5.12	
6–13	HQ Core	Moderately weathered to fresh, thinly laminated to medium bedded, grey, fine grained, slightly porous, weak SHALE (Georgian Bay Formation) with slightly weathered to fresh, thinly bedded, grey, fine grained, non-porous, medium strong, LIMESTONE interbeds.		86.25 5.50	1 2 3 4 5 6
				78.48	
		END OF DRILLHOLE		13.27	
14					



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

NOTES
WATER LEVELS
INSTRUMENTATION

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

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

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

FEATURES LEGEND

▨	BROKEN CORE
▩	CLAY SEAM
▦	LIMESTONE
■	LOST CORE

DEPTH SCALE
1 : 50

GOLDER

LOGGED: JL
CHECKED: MPL

RECORD OF BOREHOLE No CRB-7

SHEET 1 OF 2

METRIC

PROJECT 1662333

G.W.P. 2002-13-00

DIST Central HWY QEW

DATUM Geodetic

LOCATION N 4824189.6; E 295951.1 MTM NAD 83 ZONE 10 (LAT. 43.557590; LONG. -79.609531)

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

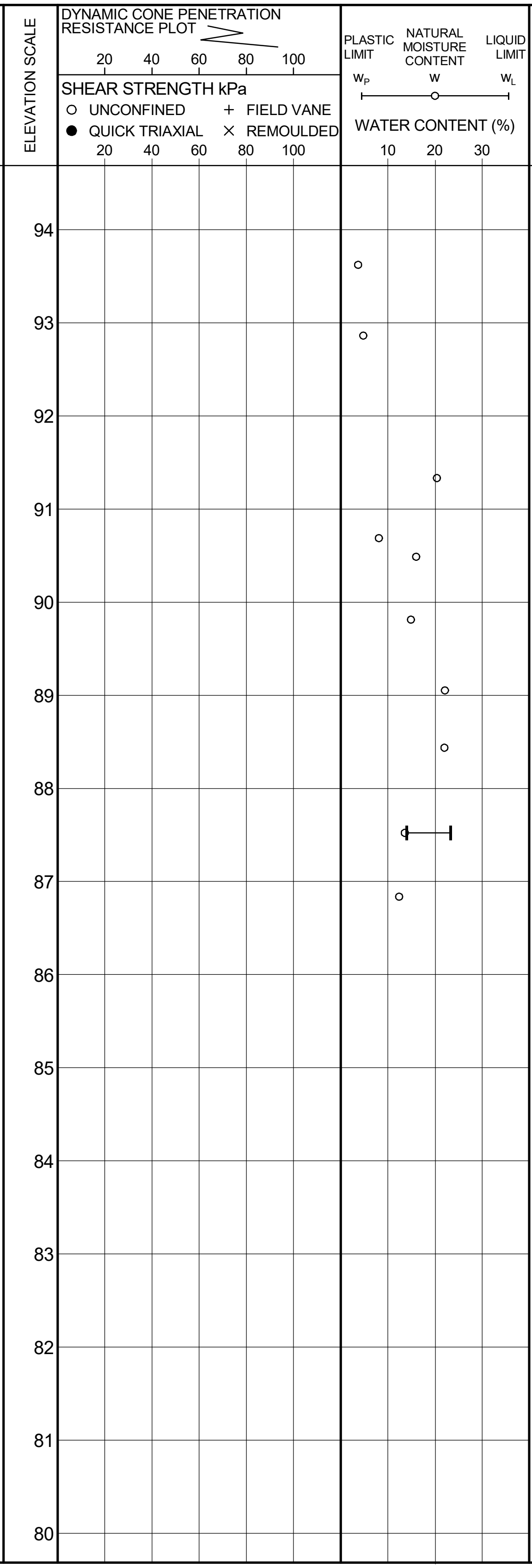
DATE October 23, 2017

ORIGINATED BY JL

COMPILED BY MPL

CHECKED BY SMM

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
94.7	GROUND SURFACE					
0.0	TOPSOIL (180mm)					
0.2	Silt and sand, trace gravel, trace clay (FILL) Loose to compact Brown Moist	1	SS	8		
		2	SS	9		
		3	SS	12		3 62 33 2
	- Becoming wet at a depth of about 2.6 m	4	SS	19		
		5	SS	21		
	- Brick fragments at a depth of about 4.0 m	6A 6B	SS	54		
90.2 4.5	Sandy SILT, trace clay Compact Brown Wet - Clayey silt layer between 4.6 m to 4.7 m - Becoming grey at a depth of about 4.9 m	7	SS	26		
		8	SS	25	NP	0 22 75 3
	- Clayey silt layer between 6.4 m and 6.5 m	9A 9B 9C	SS	17		
88.2 6.5	CLAYEY SILT with SAND, trace gravel (TILL) Stiff to hard Grey Moist	10	SS	10		4 30 47 19
86.9	Sandy CLAYEY SILT, containing shale fragments (RESIDUAL SOIL) Hard Grey Moist	11	SS	50/0.15		
86.6 8.1	SHALE (BEDROCK) Grey Bedrock cored from a depth of 8.5 m to 16.0 m	1	RC	REC 79%		RQD = 25%
	For bedrock coring details, refer to Record of Drillhole CRB-7	2	RC	REC 100%		RQD = 95%
		3	RC	REC 100%		RQD = 100%
		4	RC	REC 100%		RQD = 89%
		5	RC	REC 100%		RQD = 100%



Continued Next Page

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

GOLDER

Foundation Design

RECORD OF BOREHOLE No CRB-7

SHEET 2 OF 2

METRIC

PROJECT 1662333
G.W.P. 2002-13-00
LOCATION N 4824189.6; E 295951.1 MTM NAD 83 ZONE 10 (LAT. 43.557590; LONG. -79.609531)
ORIGINATED BY JL
DIST Central HWY QEW
BOREHOLE TYPE CME 850, 210 mm O.D. Hollow Stem Augers, HQ Casing (Auto Hammer)
COMPILED BY MPL
DATUM Geodetic
DATE October 23, 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) Grey	▨	5	RC	REC 100%						RQD = 100%
		▨	6	RC	REC 100%		79				RQD = 100%
78.7 16.0	END OF BOREHOLE NOTE: 1. Borehole dry prior to rock coring.										

GTA-MTO 001 S:\CLIENTS\MTO\QEW-CREDIT_RIVER\02_DATA\GINT\QEW-CREDIT_RIVER.GPJ GAL-GTA.GDT 2/11/19

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

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

PROJECT: 1662333

RECORD OF DRILLHOLE: CRB-7

SHEET 1 OF 1

LOCATION: N 4824189.6 ;E 295951.1

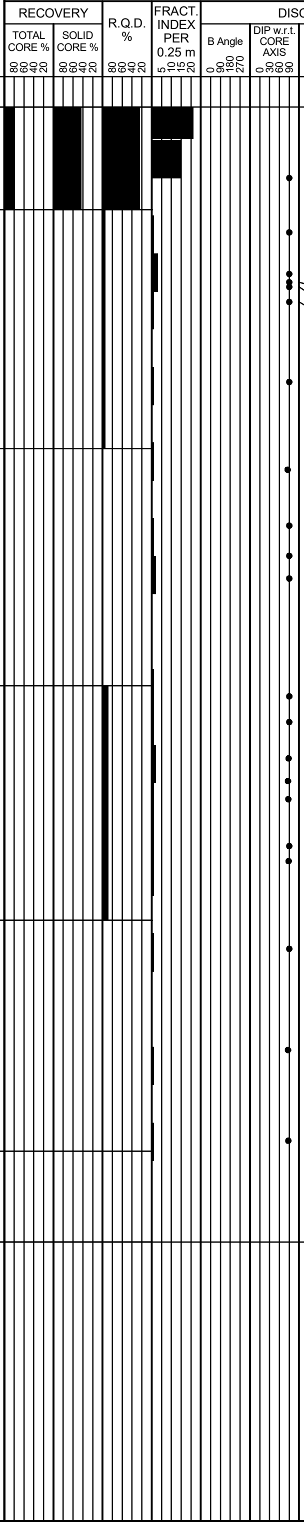
INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: October 23, 2017
DRILL RIG: CME 850 Track
DRILLING CONTRACTOR: Aardvark Drilling

DATUM: Geodetic

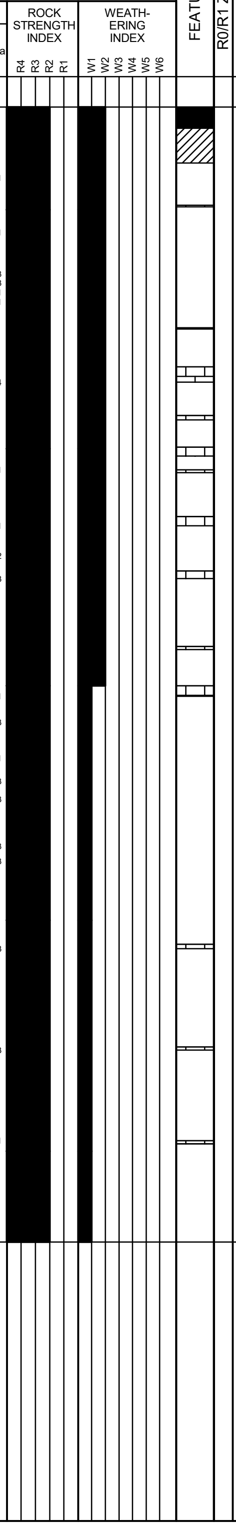
JN - Joint	BD - Bedding	PL - Planar	PO - Polished	BR - Broken Rock
FLT - Fault	FO - Foliation	CU - Curved	K - Slickensided	**NOTE:** For additional abbreviations refer to list of abbreviations & symbols.
SHR - Shear	CO - Contact	UN - Undulating	SM - Smooth	
VN - Vein	OR - Orthogonal	ST - Stepped	Ro -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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.
		Continued from Borehole CRB-7		86.16	
9	HQ Core	Slightly weathered to fresh, thinly laminated to medium bedded, grey, very fine to fine grained, faintly porous, weak SHALE (Georgian Bay Formation) with LIMESTONE interbeds		8.53 85.48	1
10–15	HQ Core	Slightly weathered to fresh, thinly bedded, grey, fine grained, non-porous, weak to medium strong, SHALE (Georgian Bay Formation) with slightly weathered to fresh, thinly bedded, grey, fine grained, non-porous, medium strong, LIMESTONE interbeds.		9.21 78.65	2 3 4 5 6
16		END OF DRILLHOLE		16.04	
17					



DISCONTINUITY DATA

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



NOTES
WATER LEVELS
INSTRUMENTATION

PLT(A) = 0.47 MPa
PLT(D) = 0.26 MPa

UC = 15.5 MPa
PLT(A) = 0.56 MPa
PLT(D) = 0.42 MPa

PLT(A) = 0.85 MPa
PLT(D) = 0.32 MPa
UC = 7.4 MPa

FEATURES LEGEND

▨ BROKEN CORE	▩ CLAY SEAM	▦ LIMESTONE	■ LOST CORE

DEPTH SCALE
1 : 50

GOLDER

LOGGED: JL
CHECKED: MPL

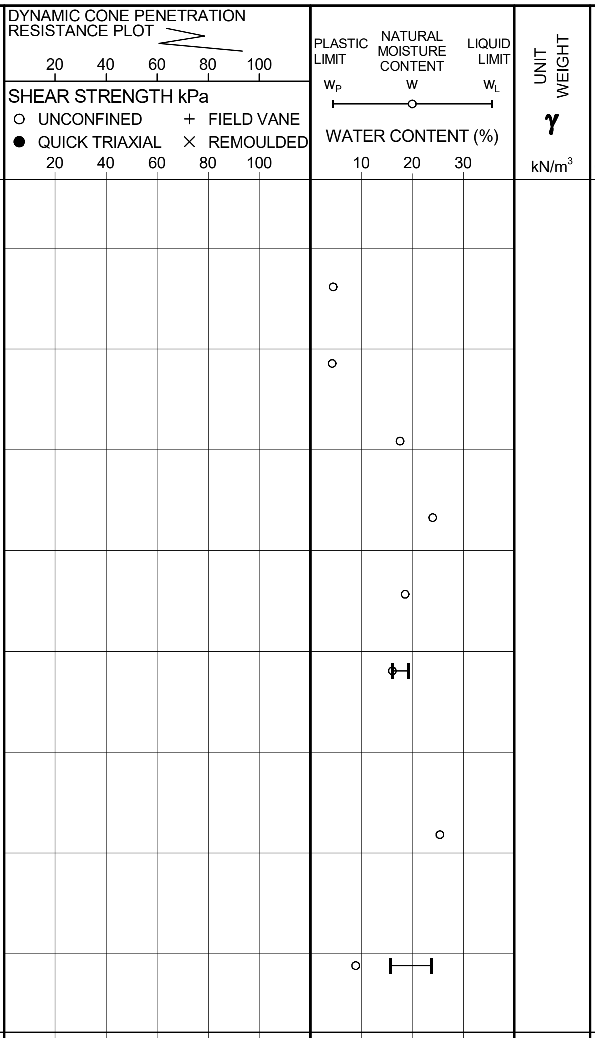
RECORD OF BOREHOLE No CRB-8

SHEET 1 OF 1

METRIC

PROJECT 1662333
G.W.P. 2002-13-00
LOCATION N 4824211.5; E 295953.7 MTM NAD 83 ZONE 10 (LAT. 43.557788; LONG. -79.609499)
ORIGINATED BY JL
DIST Central HWY QEW
BOREHOLE TYPE CME 850, 210 mm O.D. Hollow Stem Augers (Auto Hammer)
COMPILED BY MPL
DATUM Geodetic
DATE October 17, 2017
CHECKED BY SMM

SOIL PROFILE: ELEV DEPTH	SOIL PROFILE: DESCRIPTION	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: "N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%): GR	SA	SI	CL
94.7	GROUND SURFACE									
0.0 0.1	TOPSOIL (80 mm)									
	Sand, some silt, trace clay (FILL) Very loose to compact Brown Moist	1	SS	3		94				
		2	SS	12	∇					
93.3 1.5	SAND, some silt, trace clay Compact to dense Brown Moist	3	SS	21		93				
		4	SS	44		92	0	84	14	2
	- Becoming wet below a depth of about 3.1 m - Clayey silt pockets at a depth of about 3.1 m	5	SS	35		91				
91.0 3.7	SILT, trace to some sand, trace to some clay Slight plasticity Dense to very dense Grey Wet	6	SS	67						
		7	SS	32		90	0	7	85	8
	- Becoming grey and brown at a depth of about 5.6 m					89				
88.3 6.4	CLAYEY SILT, some sand Very stiff Grey Wet	8A 8B	SS	22		88				
87.1 7.6	Sandy CLAYEY SILT with shale fragments (RESIDUAL SOIL) Hard Grey Moist - Auger grinding at a depth of about 7.6 m	9A	SS	50/0.13		87	31	27	34	8
86.7 8.1	SHALE (BEDROCK) Grey	9B								
86.2 8.5	END OF BOREHOLE - SPLIT-SPOON REFUSAL	~~10~~	~~SS~~	~~50/0.08~~						



NOTE:

1. Water level measured at a depth of about 1.5 m (Elev. 93.2 m) below ground surface upon completion of drilling.

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

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

GOLDER

Foundation Design

RECORD OF BOREHOLE No NW6-1

SHEET 1 OF 1

METRIC

PROJECT 1662333
G.W.P. 2002-13-00
DIST Central HWY QEW
DATUM Geodetic
LOCATION N 4824163.1; E 295975.2 MTM NAD 83 ZONE 10 (LAT. 43.557371; LONG. -79.609278)
BOREHOLE TYPE CME 55, 108 mm I.D., Hollow Stem Augers
DATE July 10, 2018
ORIGINATED BY ACM
COMPILED BY SK
CHECKED BY SMM

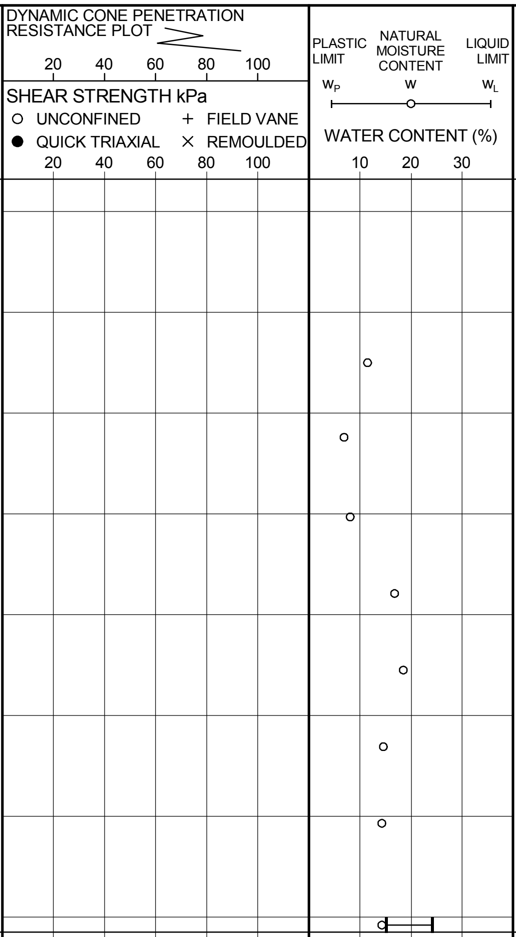
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
95.3	GROUND SURFACE										
0.0	ASPHALT (150 mm)										
	CONCRETE (150 mm)					95					
0.3	Gravelly sand, some fines (FILL) Compact to very dense Brown Moist	1	SS	60				21	60	(19)	
		2	SS	47		94					
		3	SS	40		93					
		4	SS	10							
92.3 3.0	Silty SAND, trace to some clay Very loose to dense Brown Moist	5	SS	4		92					
		6	SS	3		91		1	67	25	7
		7	SS	34							
90.0 5.3	SILT, trace to some sand, trace to some clay Dense Brown to grey below 6.6 m Moist	8	SS	32		90		0	8	80	12
		9	SS	41	▽	89					
		10A	SS	35							
88.0 7.5	Sandy CLAYEY SILT, trace gravel (TILL) Hard Grey Moist	10B				88		3	22	50	25

END OF BOREHOLE

NOTES:

1. Borehole caved to a depth of 6.4 m below ground surface upon removal of hollow stem augers.

2. Water level measured at a depth of 6.3 m (Elev. 89.0 m) below ground surface upon completion of soil drilling.



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

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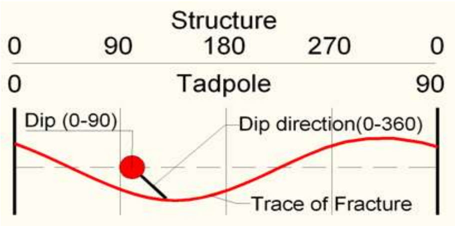
GOLDER

GEOPHYSICAL RECORD OF BOREHOLE: CRB-2B

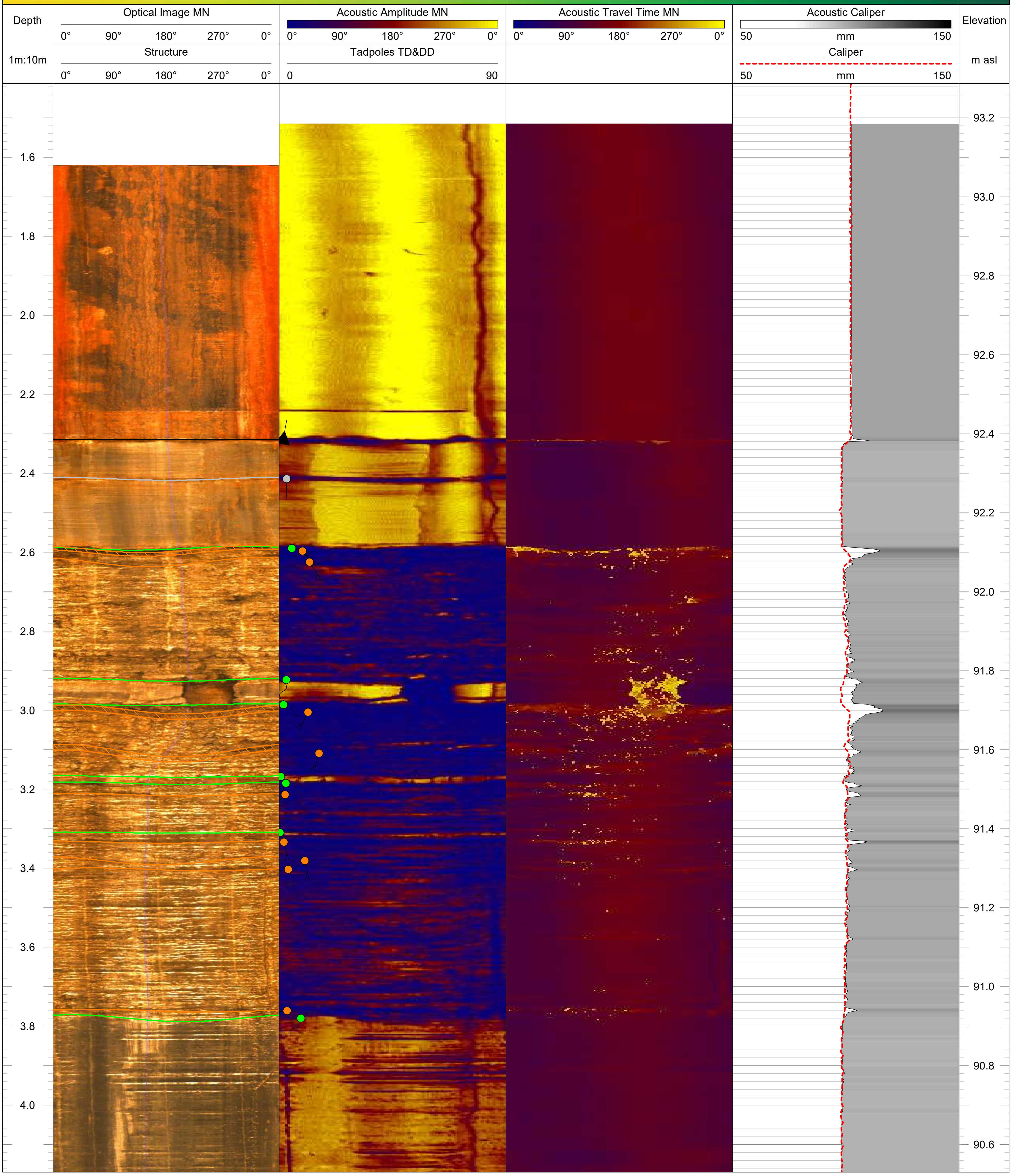
Project Number: 1662333
Client: Morrison Hershfield Ltd.
Date: July 2018

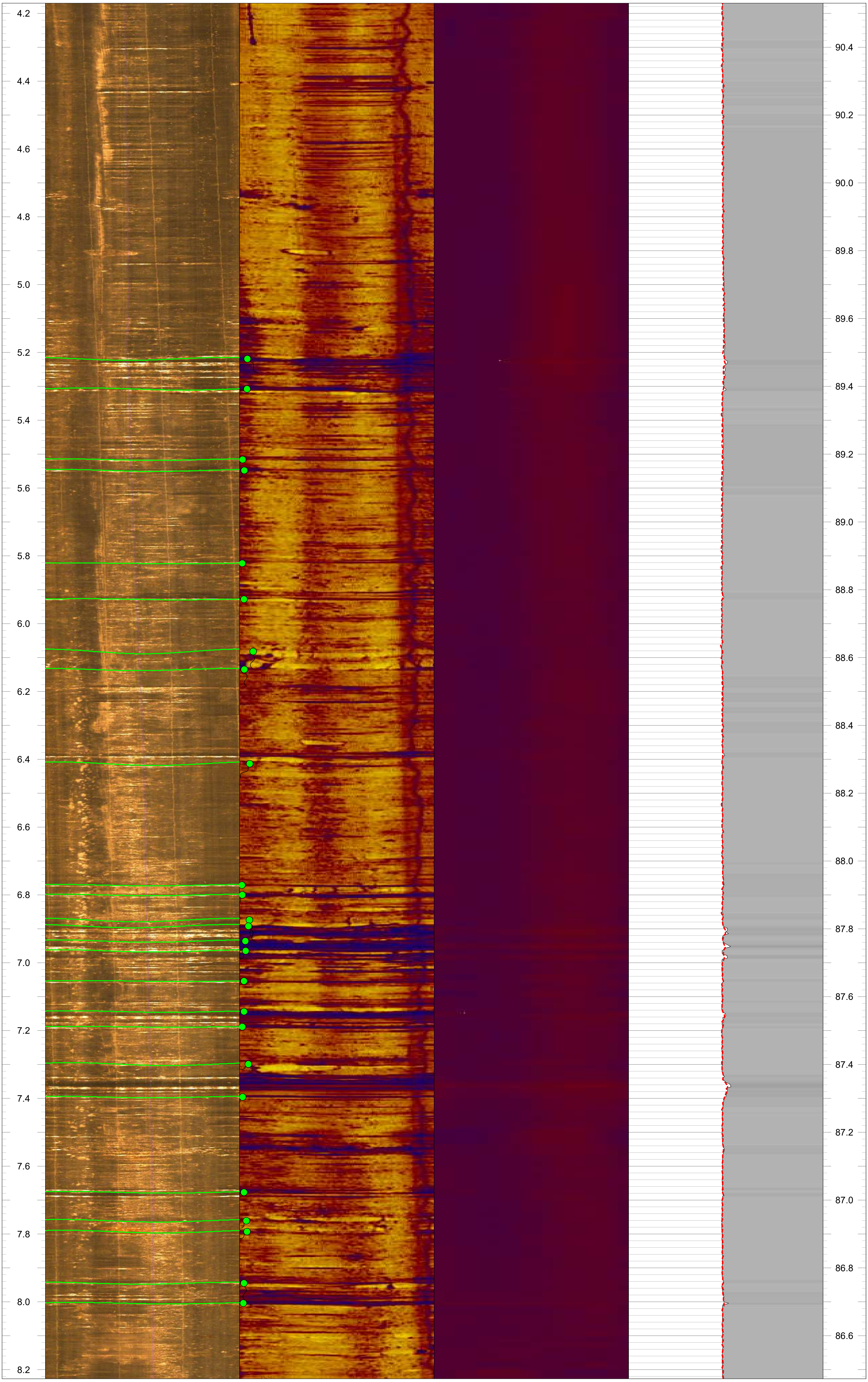
Datum:	NAD83, UTM Zone 17N	**Depth Reference:**	"0" at Ground	**Casing Depth:**	2.5 m bgs	**Location:**	QEW Credit River
Easting:	295,818.7 m E	**Drilled Depth:**	13 m bgs	**Water Level:**	8.2 m bgs	**Log Date:**	6-Jul-18
Northing:	4,823,955.2 m N	**Borehole Diameter:**	97 mm	**Borehole Inclination:**	0 deg, Vertical		
Elevation:	94.7 m asl	**Casing Diameter:**	100 mm	**Borehole Azimuth:**	N/A	**Logged By:**	AR



● Partially Open Joint / Fracture
○ Filled Fracture / Joint
● Bedding / Banding / Foliation
◀ Casing
◆ Water Table

Notes:




4.2
4.4
4.6
4.8
5.0
5.2
5.4
5.6
5.8
6.0
6.2
6.4
6.6
6.8
7.0
7.2
7.4
7.6
7.8
8.0
8.2
90.4
90.2
90.0
89.8
89.6
89.4
89.2
89.0
88.8
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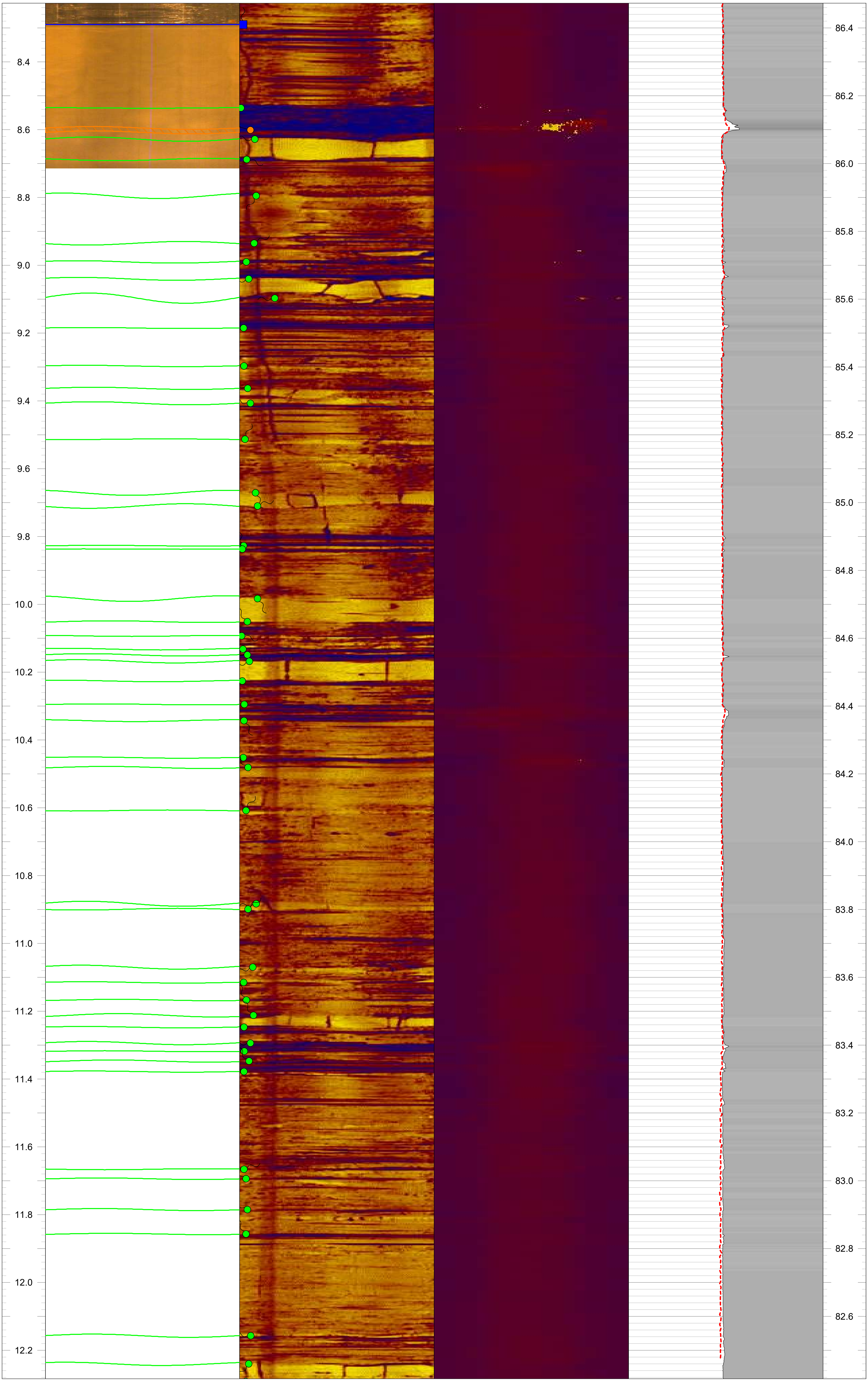
87.0
86.8
86.6


8.4
8.6
8.8
9.0
9.2
9.4
9.6
9.8
10.0
10.2
10.4
10.6
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11.0
11.2
11.4
11.6
11.8
12.0
12.2
86.4
86.2
86.0
85.8
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85.4
85.2
85.0
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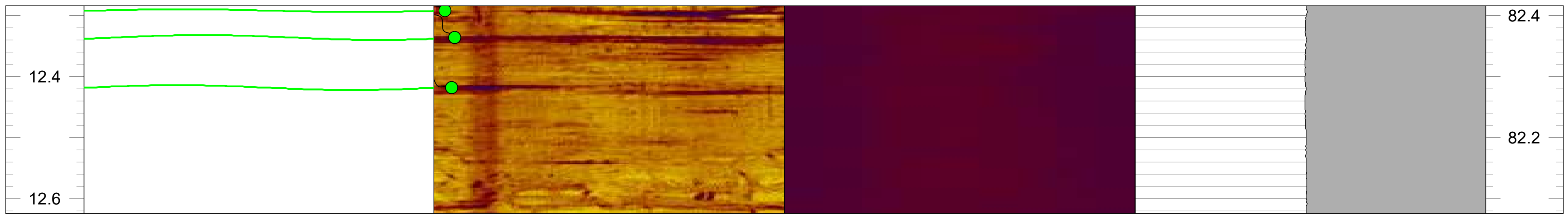
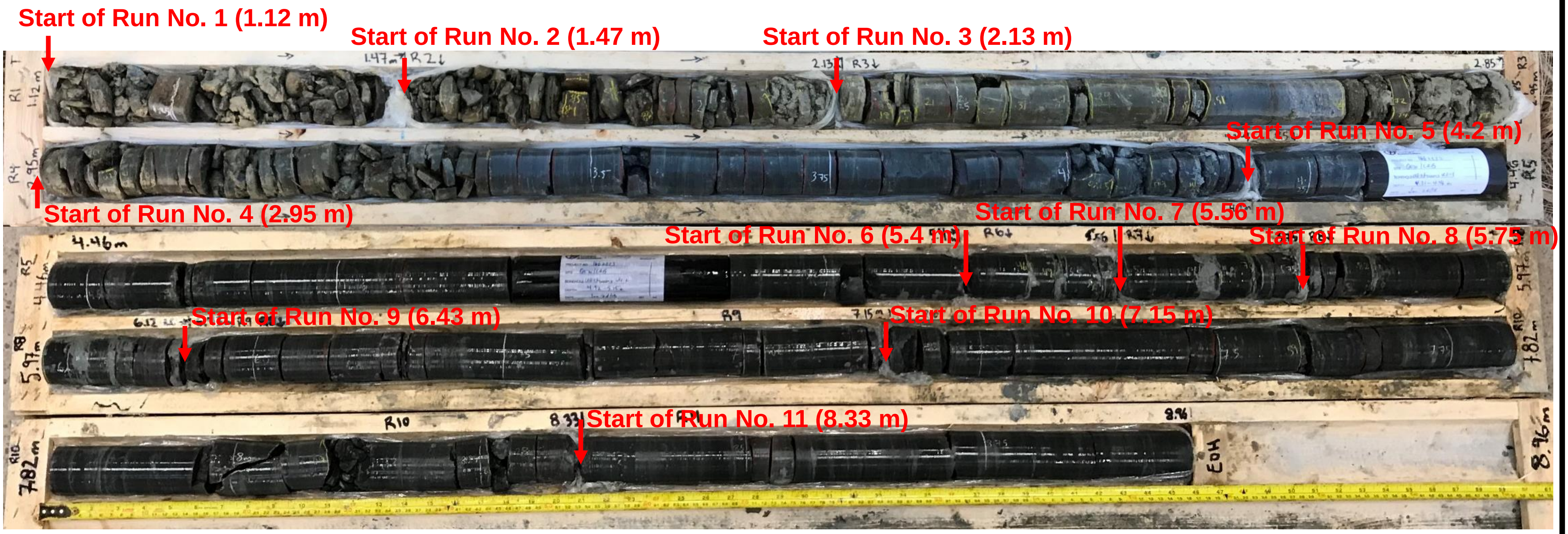
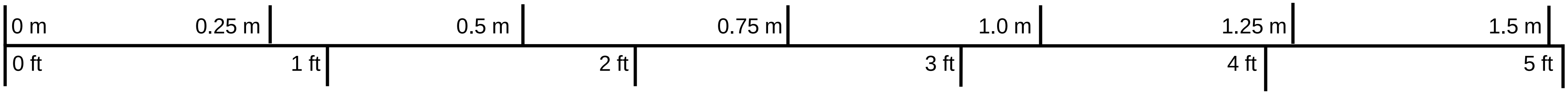
83.6
83.4
83.2
83.0
82.8
82.6


12.4
12.6
82.4
82.2

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

PROJECT	**MTO Assignment 2015-E-0033: Detail Design for the widening/rehab/realignment of QEW Between Mississauga Road and Hurontario Street**				
TITLE	**Bedrock Core Photograph Borehole CRB-2 (3.04 m to 12.8 m)**				
GOLDER	PROJECT No. 1662333			FILE No. ----	
	DRAFT	JIL	Mar 2018	SCALE AS SHOWN	VER. 1.
	CADD	--		**FIGURE B-1**	
	CHECK	DM	June 2018		
	REVIEW	SMM	June 2018		





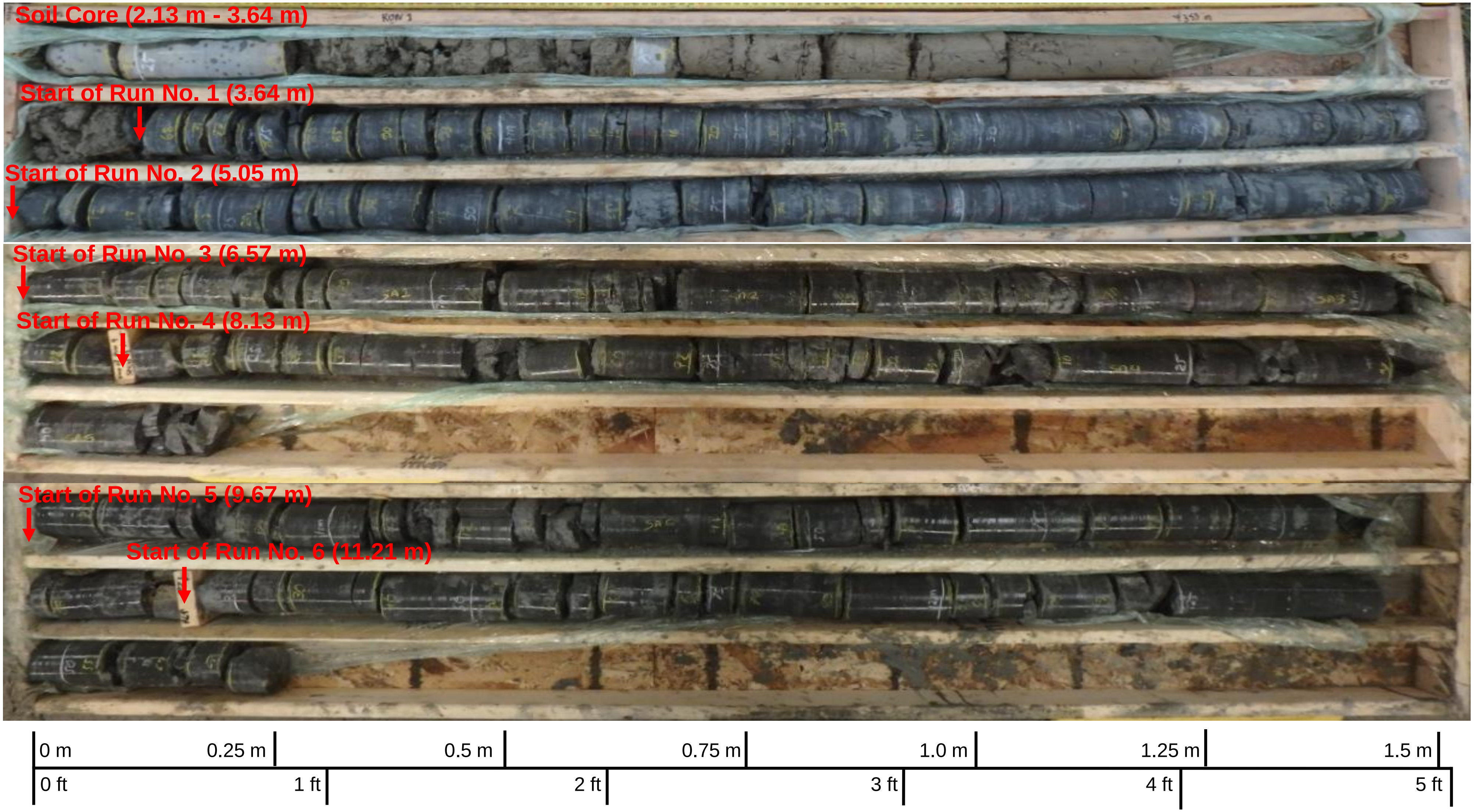
Scale

PROJECT	**MTO Assignment 2015-E-0033: Detail Design for the widening/rehab/realignment of QEW Between Mississauga Road and Hurontario Street**
TITLE	**Bedrock Core Photograph Borehole CRB-2A (1.12 m to 8.96 m)**

GOLDER

PROJECT No. 1662333			FILE No. ----		
DRAFT	JIL	Mar 2018	SCALE	AS SHOWN	VER. 1.
CADD	--		**FIGURE B-2**		
CHECK	DM	June 2018			
REVIEW	SMM	June 2018			

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



Scale

PROJECT

MTO Assignment 2015-E-0033: Detail Design for the widening/rehab/realignment of QEW Between Mississauga Road and Hurontario Street

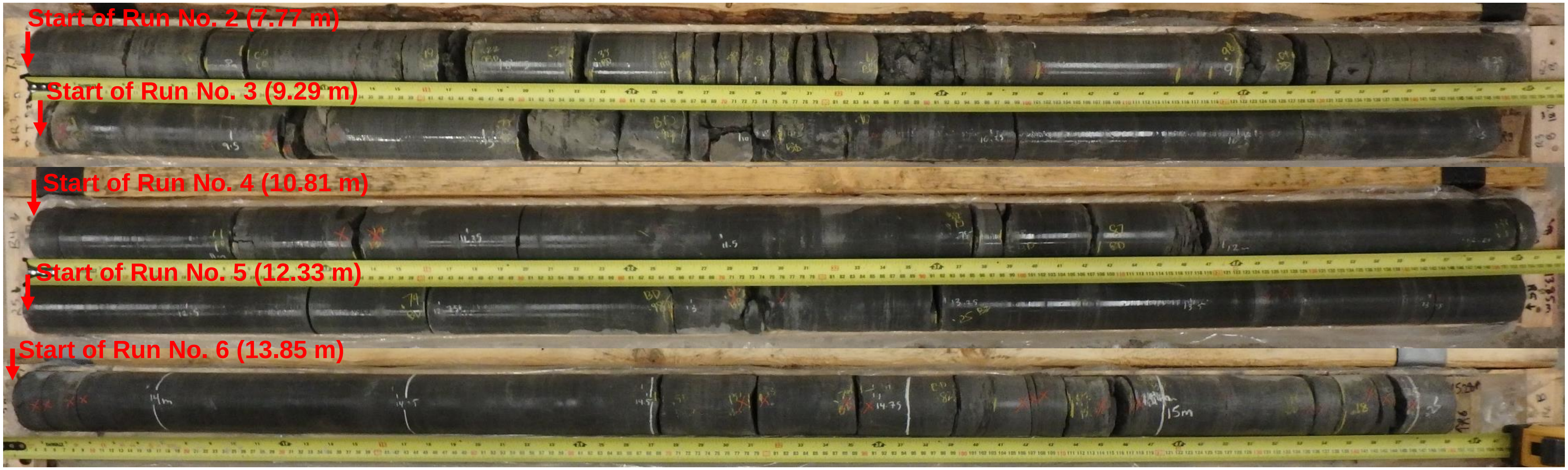
TITLE

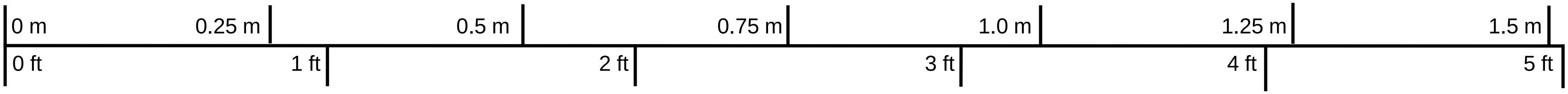
Bedrock Core Photograph
Borehole CRB-2B (3.64 m to 12.72 m)

GOLDER

PROJECT No. 1662333			FILE No. ----	
DRAFT	SK	July 2018	SCALE AS SHOWN	VER. 1.
CADD	--		**FIGURE B-3**	
CHECK				
REVIEW				

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





Scale

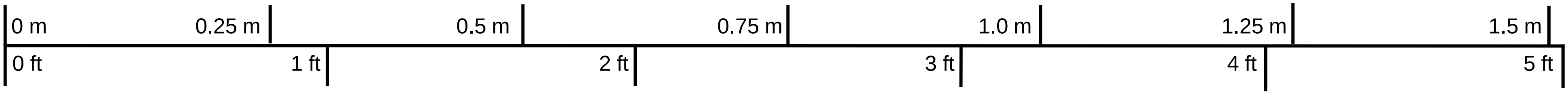
PROJECT
MTO Assignment 2015-E-0033: Detail Design for the widening/rehab/realignment of QEW Between Mississauga Road and Hurontario Street

TITLE
Bedrock Core Photograph
Borehole CRB-3 (7.77 m to 15.28 m)

GOLDER

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

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



Scale

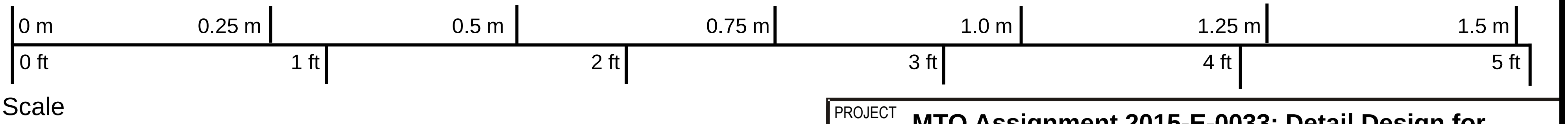
PROJECT	**MTO Assignment 2015-E-0033: Detail Design for the widening/rehab/realignment of QEW Between Mississauga Road and Hurontario Street**
TITLE	**Bedrock Core Photograph Borehole CRB-3A (7.01 m to 15.78 m)**

GOLDER

PROJECT No. 1662333			FILE No. ----	
DRAFT	JIL	Mar 2018	SCALE AS SHOWN	VER. 1.
CADD	--		**FIGURE B-5**	
CHECK	DM	June 2018		
REVIEW	SMM	June 2018		

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

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

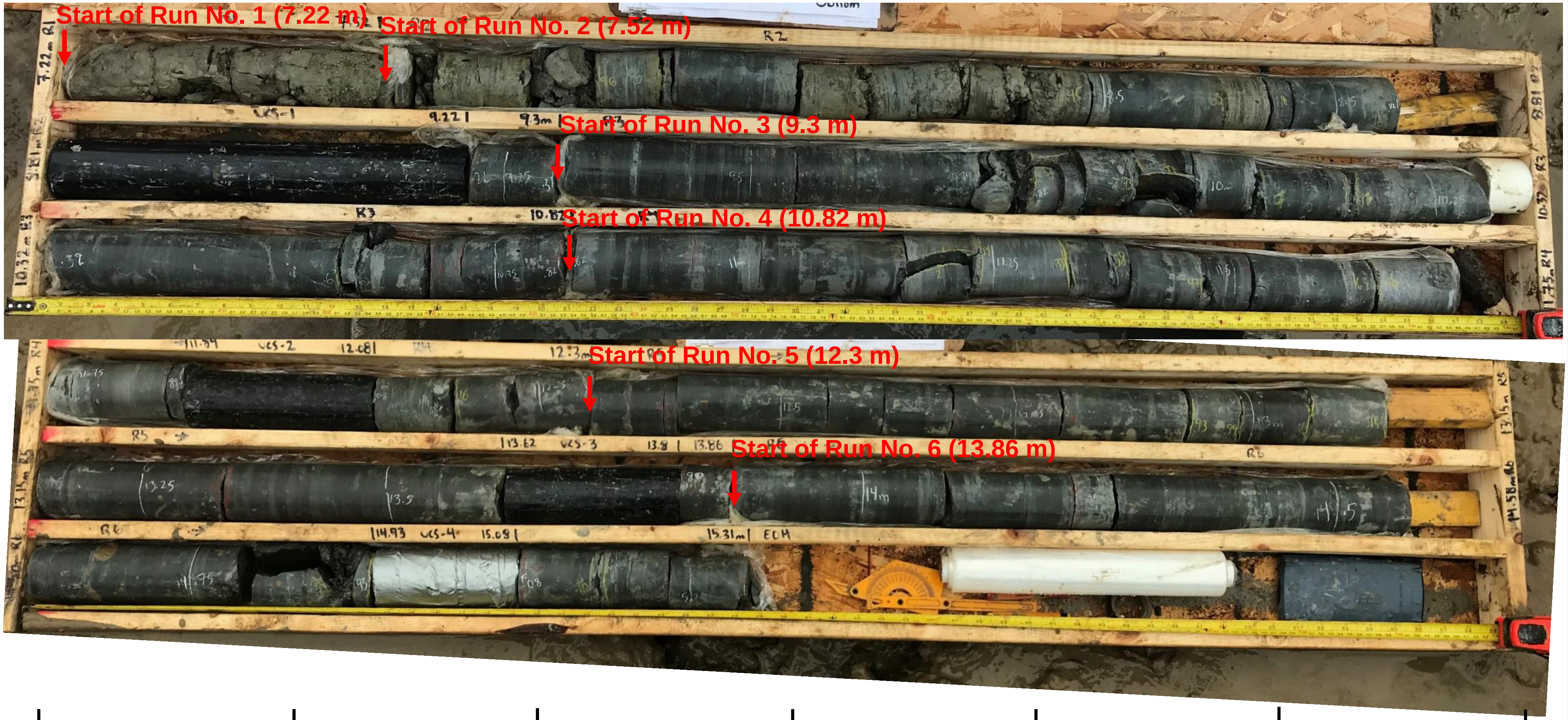


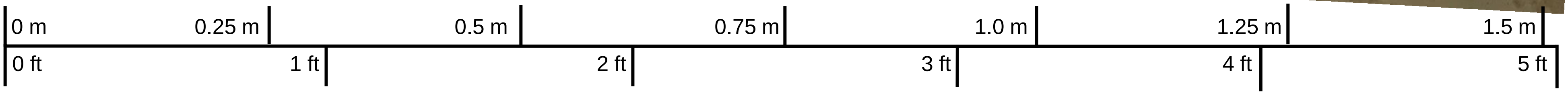
PROJECT
MTO Assignment 2015-E-0033: Detail Design for the widening/rehab/realignment of QEW Between Mississauga Road and Hurontario Street

TITLE
Bedrock Core Photograph
Borehole CRB-3C (6.43 m to 14.1 m)

GOLDER

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





Scale

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

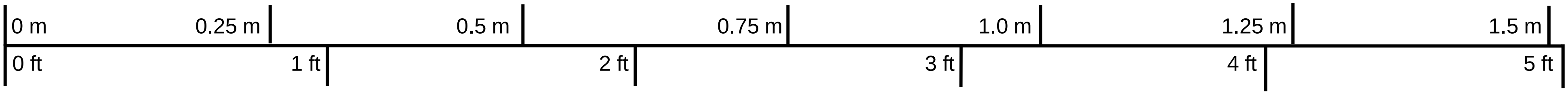
PROJECT **MTO Assignment 2015-E-0033: Detail Design for the widening/rehab/realignment of QEW Between Mississauga Road and Hurontario Street**

TITLE

Bedrock Core Photograph
Borehole CRB-4 (7.22 m to 15.31 m)

GOLDER

PROJECT No. 1662333			FILE No. ----		
DRAFT	JIL	Mar 2018	SCALE	AS SHOWN	VER. 1.
CADD	--		**FIGURE B-7**		
CHECK	DM	June 2018			
REVIEW	SMM	June 2018			



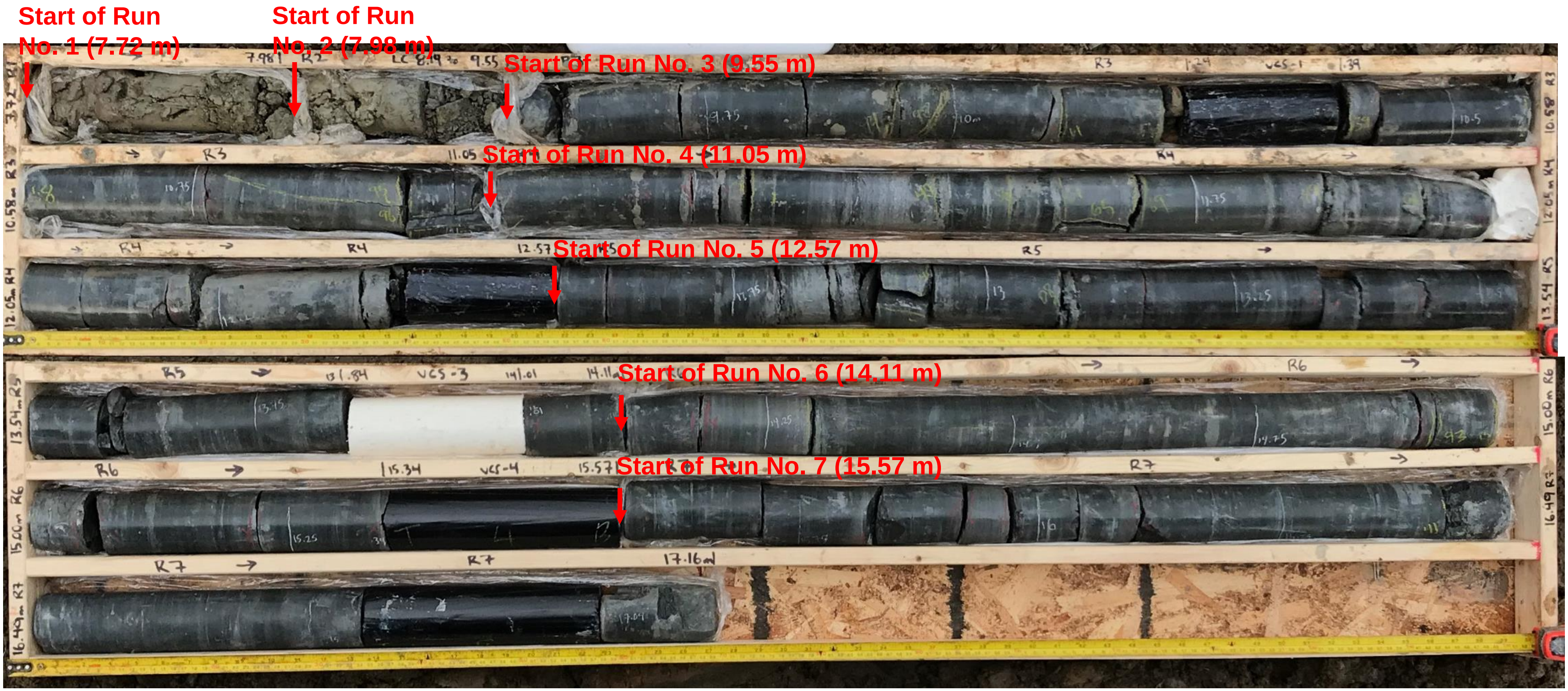
Scale

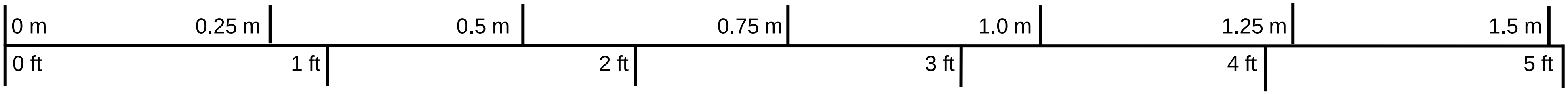
PROJECT	**MTO Assignment 2015-E-0033: Detail Design for the widening/rehab/realignment of QEW Between Mississauga Road and Hurontario Street**
TITLE	**Bedrock Core Photograph Borehole CRB-5 (7.18 m to 15.52 m)**

GOLDER

PROJECT No. 1662333			FILE No. ----	
DRAFT	JIL	Mar 2018	SCALE AS SHOWN	VER. 1.
CADD	--		**FIGURE B-8**	
CHECK	DM	June 2018		
REVIEW	SMM	June 2018		

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





Scale

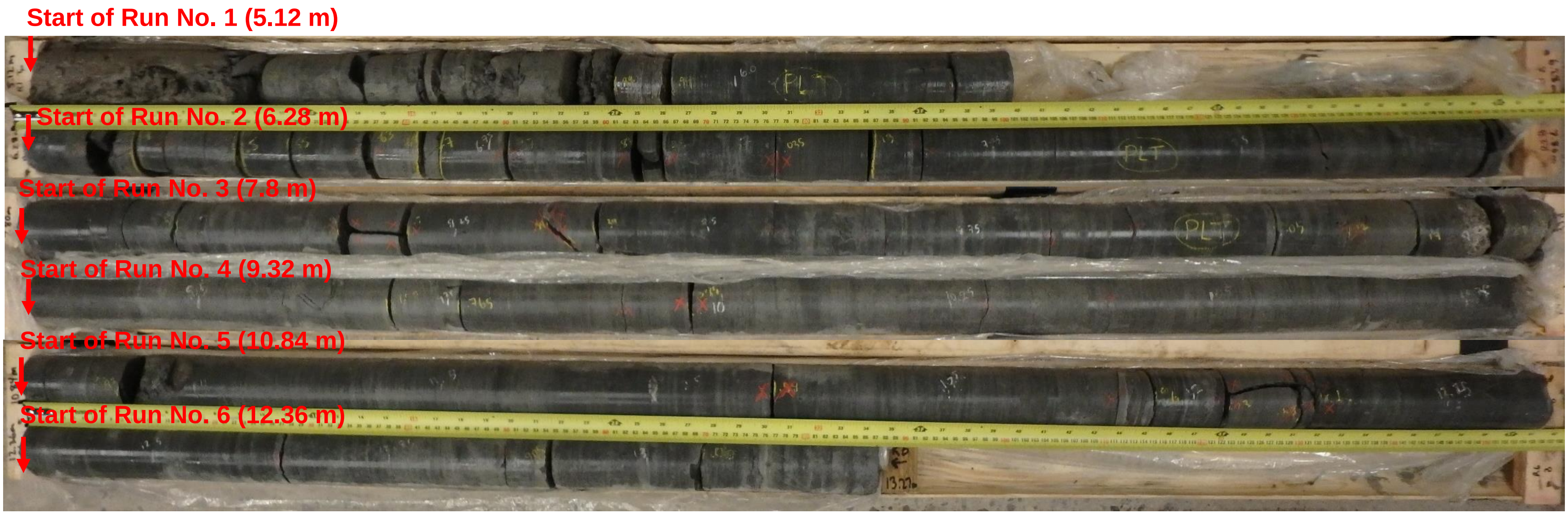
PROJECT
MTO Assignment 2015-E-0033: Detail Design for the widening/rehab/realignment of QEW Between Mississauga Road and Hurontario Street

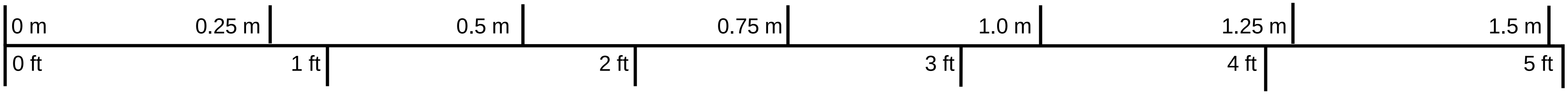
TITLE
Bedrock Core Photograph
Borehole CRB-5A (7.72 m to 17.16 m)

GOLDER

PROJECT No. 1662333			FILE No. ----		
DRAFT	JIL	Mar 2018	SCALE	AS SHOWN	VER. 1.
CADD	--		**FIGURE B-9**		
CHECK	DM	June 2018			
REVIEW	SMM	June 2018			

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





Scale

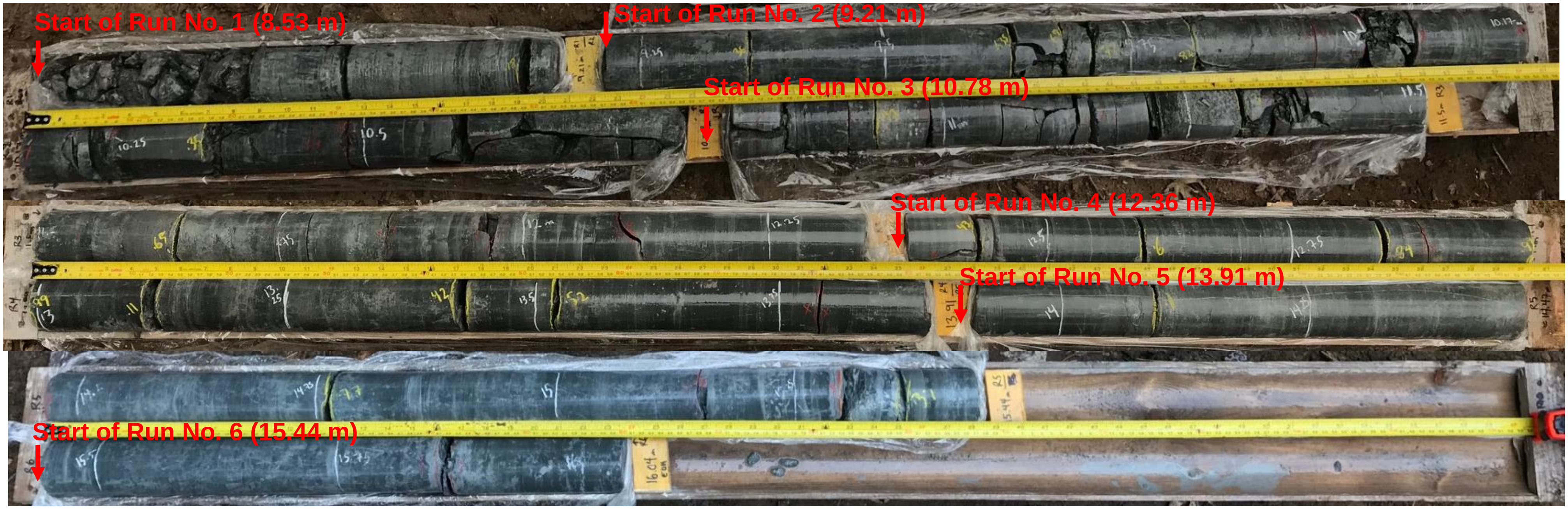
PROJECT
MTO Assignment 2015-E-0033: Detail Design for the widening/rehab/realignment of QEW Between Mississauga Road and Hurontario Street

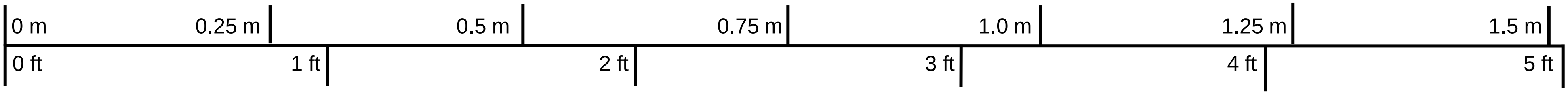
TITLE
Bedrock Core Photograph
Borehole CRB-6 (5.12 m to 13.27 m)

GOLDER

PROJECT No. 1662333			FILE No. ----	
DRAFT	JIL	Mar 2018	SCALE AS SHOWN	VER. 1.
CADD	--			
CHECK	DM	June 2018	**FIGURE B-10**	
REVIEW	SMM	June 2018		

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





Scale

PROJECT **MTO Assignment 2015-E-0033: Detail Design for the widening/rehab/realignment of QEW Between Mississauga Road and Hurontario Street**

TITLE **Bedrock Core Photograph**
Borehole CRB-7 (8.53 m to 16.04 m)

GOLDER

PROJECT No. 1662333			FILE No. ----		
DRAFT	JIL	Mar 2018	SCALE	AS SHOWN	VER. 1.
CADD	--		**FIGURE B-11**		
CHECK	DM	June 2018			
REVIEW	SMM	June 2018			

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

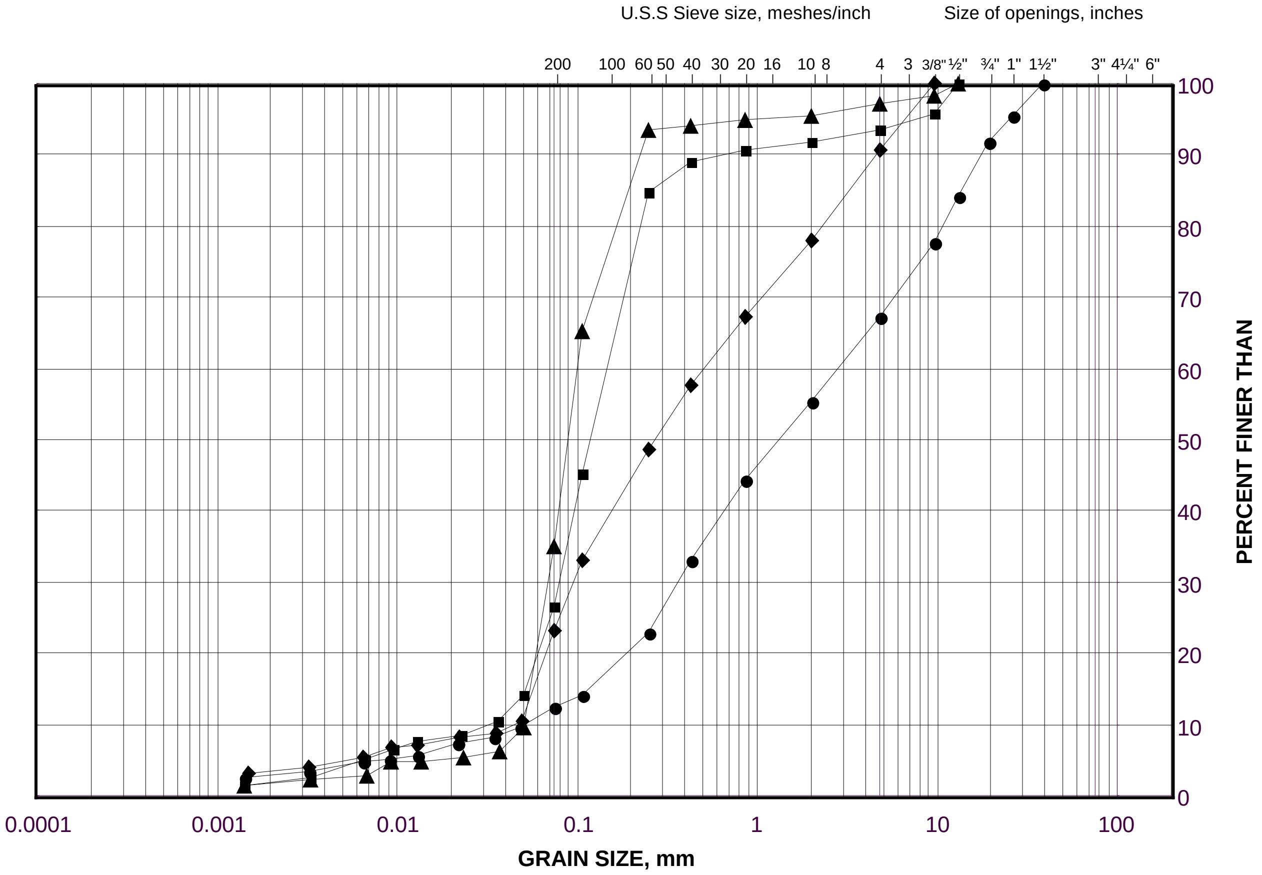
APPENDIX C

Geotechnical Laboratory Test Results and Analytical Test Results

GRAIN SIZE DISTRIBUTION

Silt and Sand to Sand to Gravelly Sand (Fill)

FIGURE C-1



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	NW6-1	1	94.8
■	CRB-6	2	90.6
◆	CRB-2	3	93.8
▲	CRB-7	3	92.9

Project Number: 1662333

Checked By: SMM

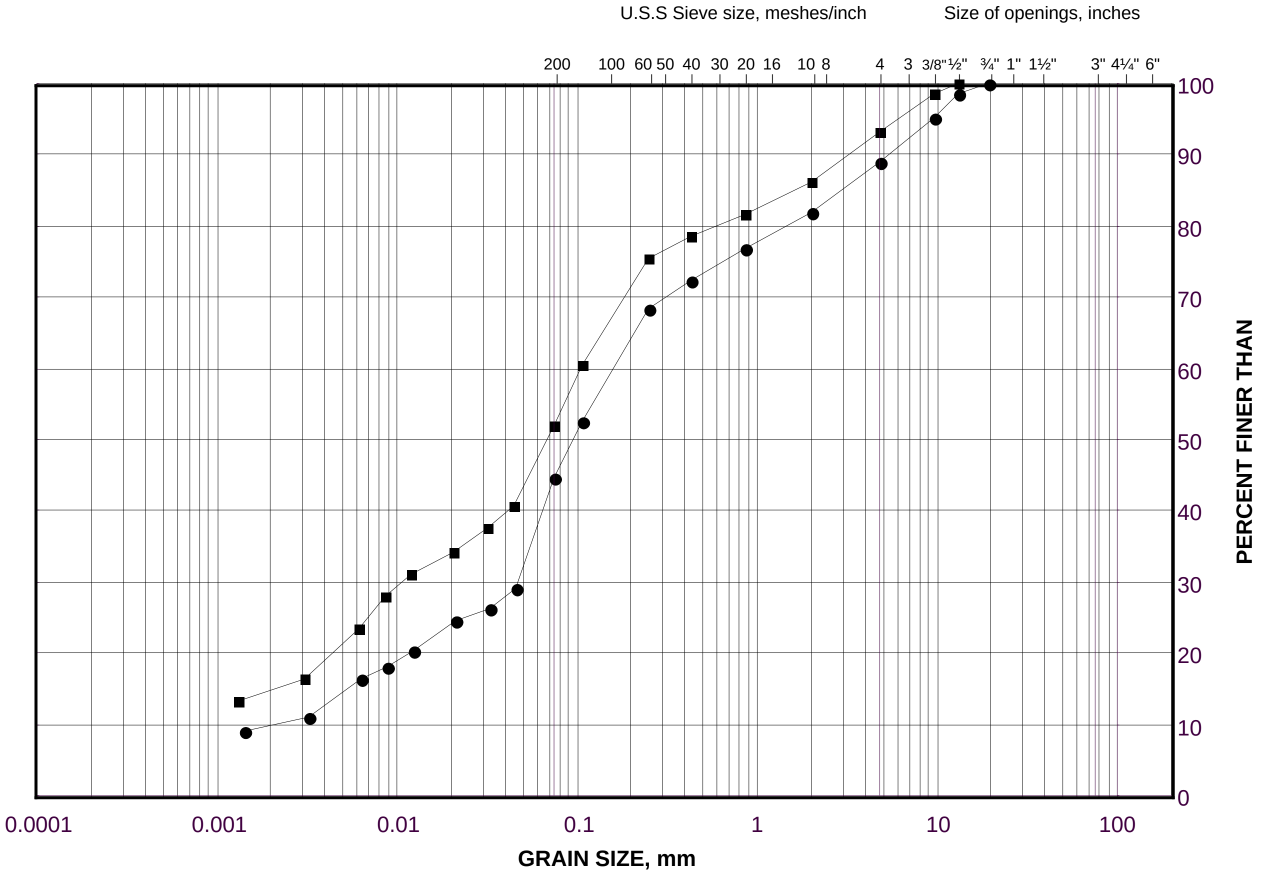
Golder Associates

Date: 01-Feb-19

GRAIN SIZE DISTRIBUTION

Clayey Silt with Sand (Fill)

FIGURE C-2



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	CRB-5A	3	77.5
■	CRB-4	4	76.5

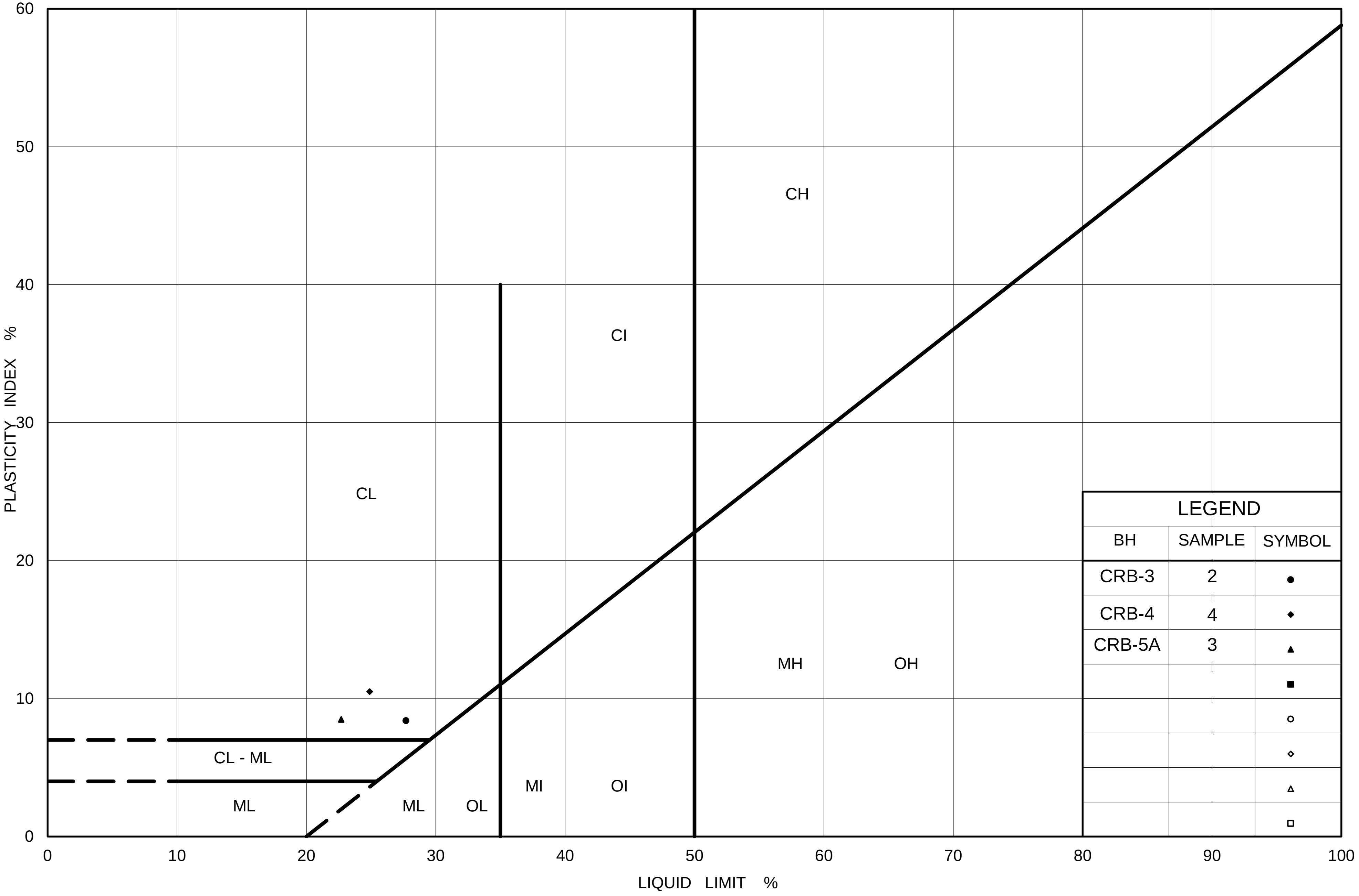
Project Number: 1662333

Checked By: SMM

Golder Associates

Date: 24-May-18

Oct 75, FF-S-21



Ministry of Transportation

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PLASTICITY CHART
Clayey Silt with Sand to Sandy Clayey Silt (Fill)

Figure No. C-3

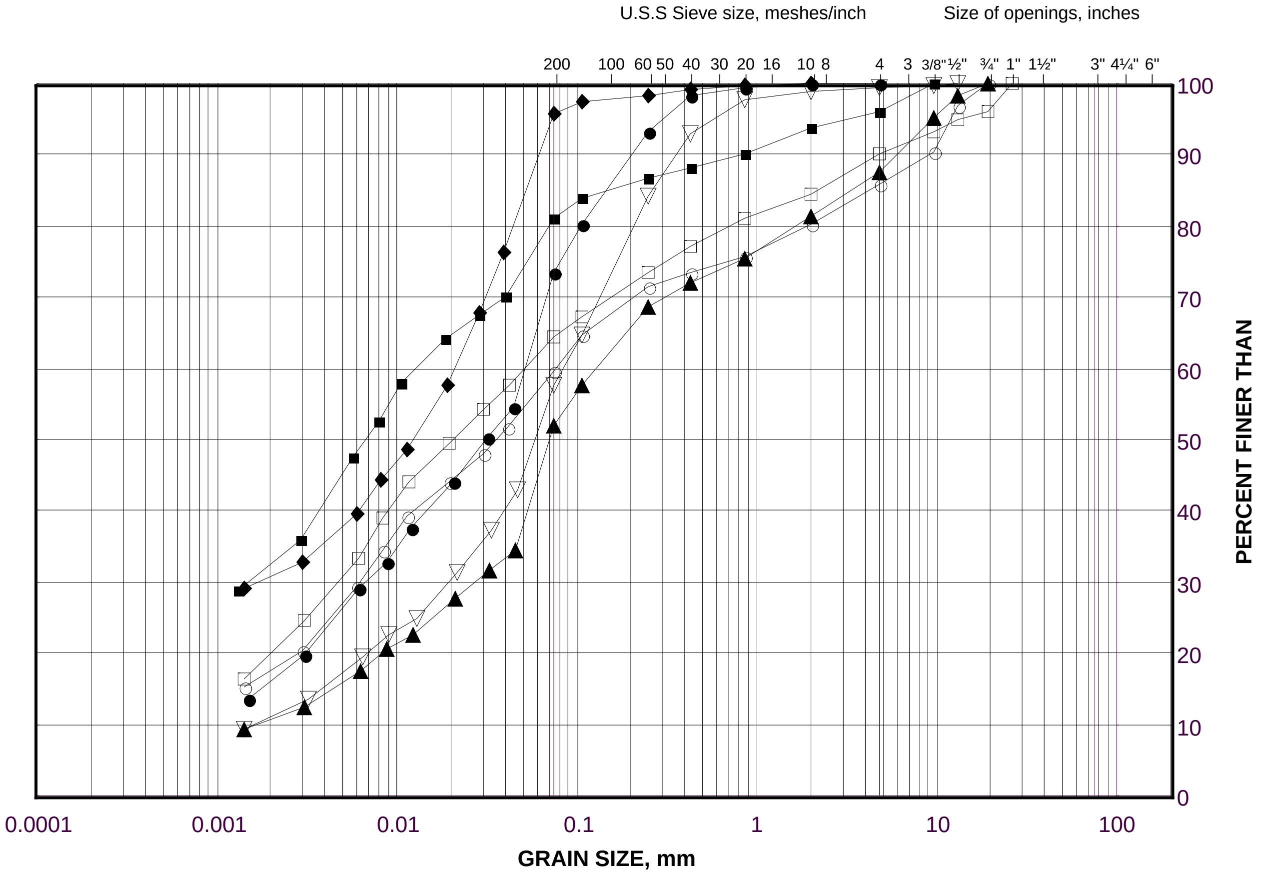
Project No. 1662333

Checked By: SMM

GRAIN SIZE DISTRIBUTION

Clayey Silt with Sand to Silty Clay

FIGURE C-4A



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	CRB-3A	1	75.4
■	CRB-1	2A	94.0
◆	CRB-2A	2B	93.3
▲	CRB-3C	3	73.6
▽	CRB-3	4	73.3
❍	CRB-4	6	75.0
□	CRB-6	6	87.6

Project Number: 1662333

Checked By: SMM

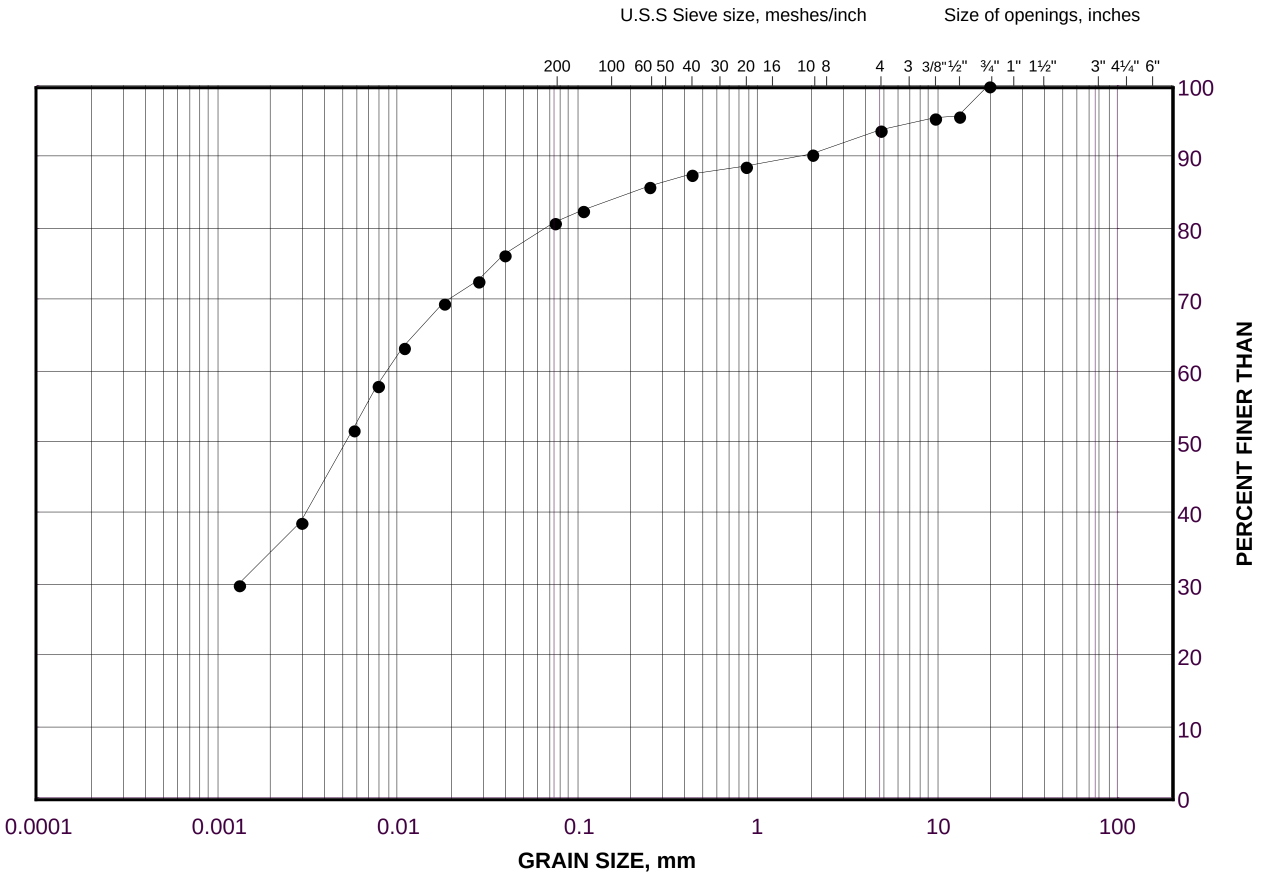
Golder Associates

Date: 24-May-18

GRAIN SIZE DISTRIBUTION

Silty Clay

FIGURE C-4B



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	CRB-2B	2	93.6

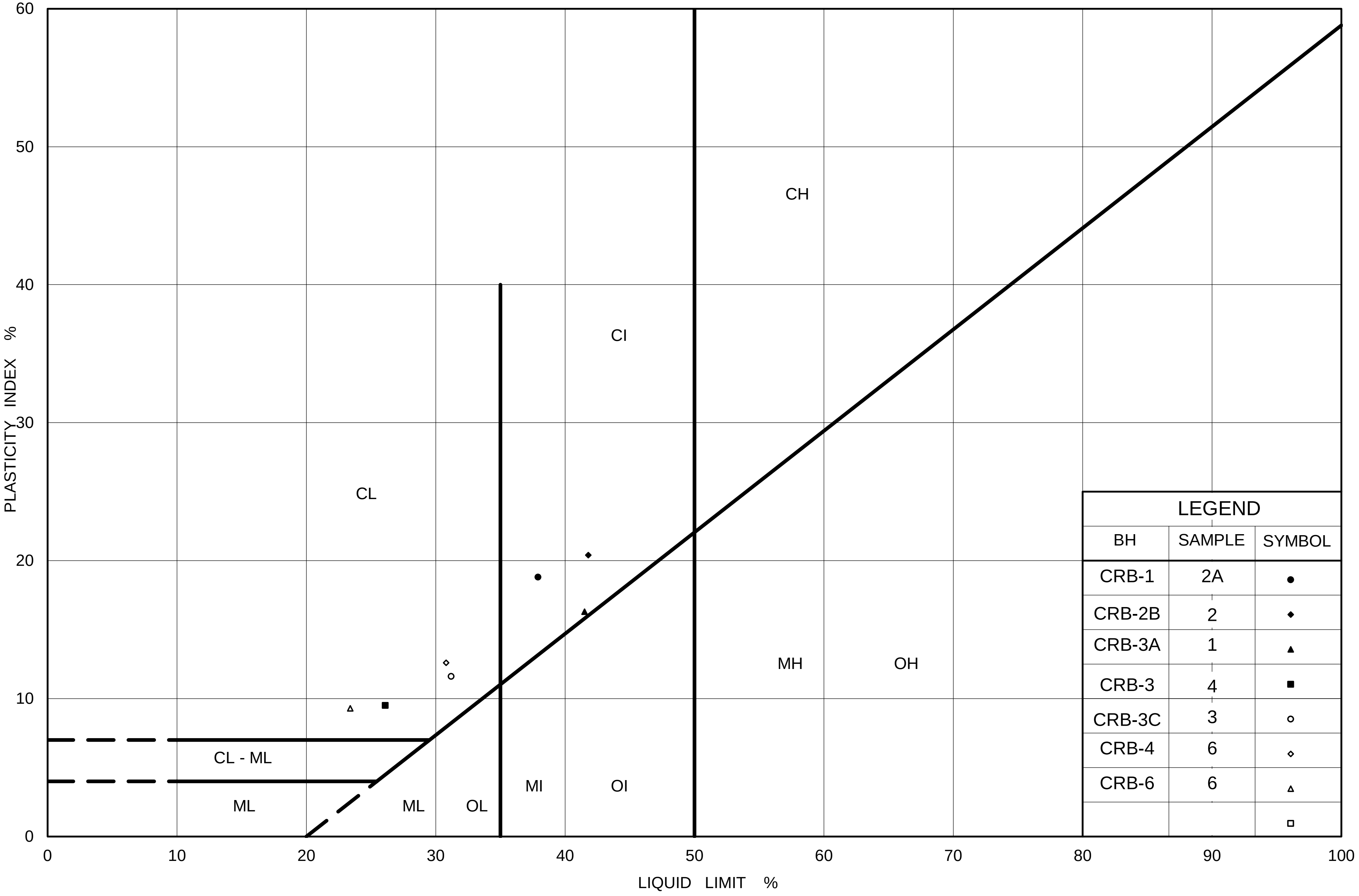
Project Number: 1662333

Checked By: SMM

Golder Associates

Date: 24-Aug-18

Oct 75, FF-S-21



Ministry of Transportation

Ontario

PLASTICITY CHART
Clayey Silt with Sand to Silty Clay

Figure No. C-5

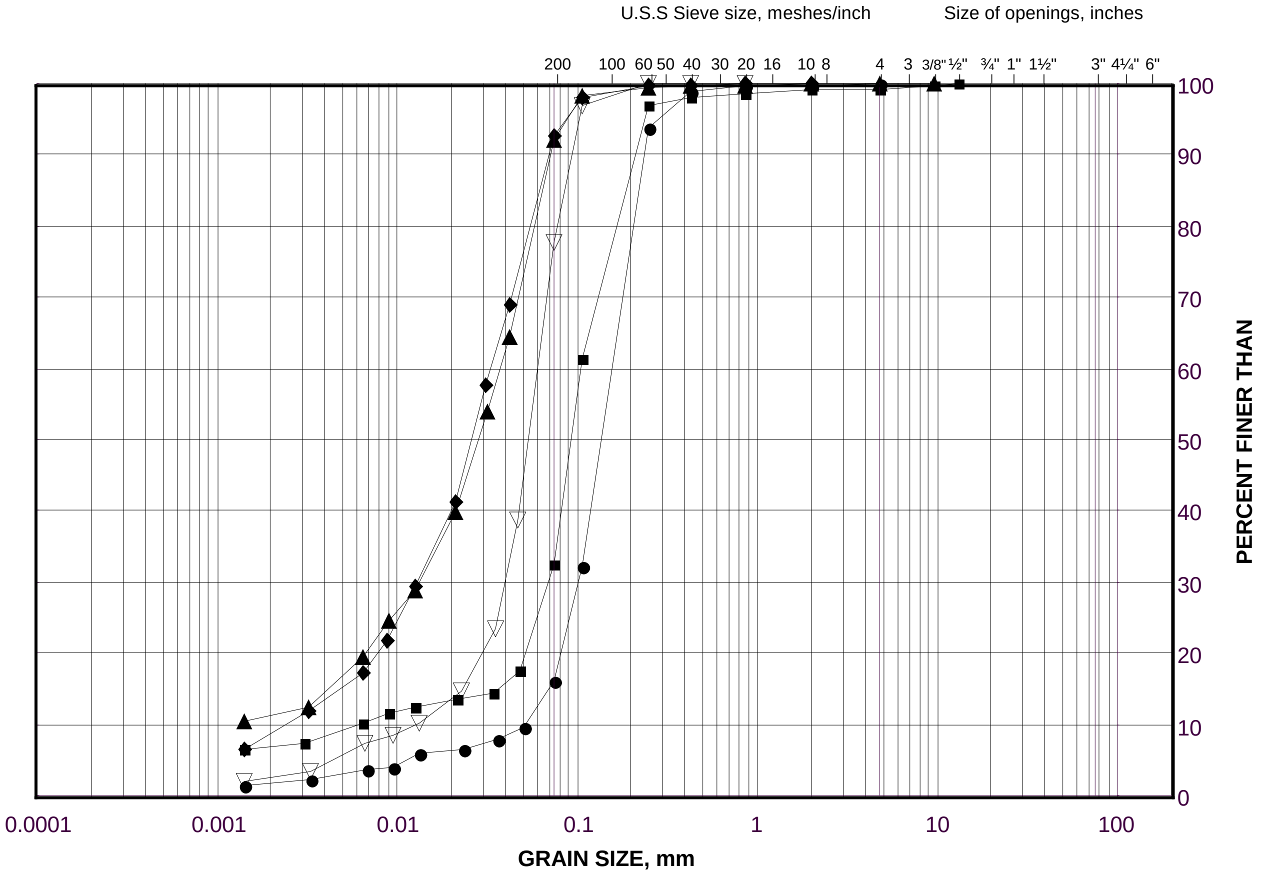
Project No. 1662333

Checked By: SMM

GRAIN SIZE DISTRIBUTION

Silt to Silty Sand to Sand

FIGURE C-6A



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	CRB-8	4	92.1
■	NW6-1	6	91.2
◆	CRB-8	7	89.8
▲	NW6-1	8	89.4
▽	CRB-7	8	89.1

Project Number: 1662333

Checked By: SMM

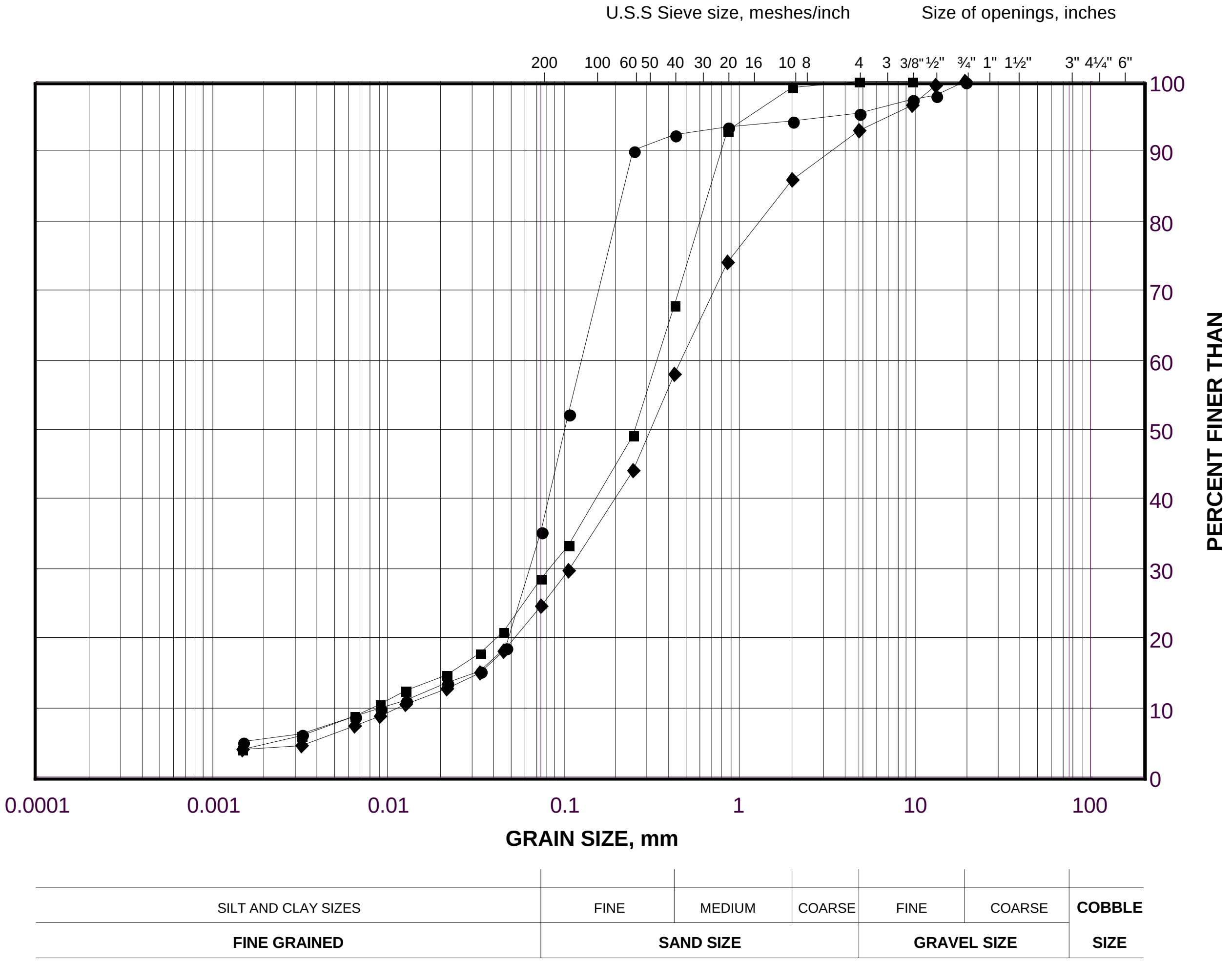
Golder Associates

Date: 01-Feb-19

GRAIN SIZE DISTRIBUTION

Silt to Silty Sand to Sand

FIGURE C-6B



LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	CRB-5	6	75.1
■	CRB-5A	8B	72.9
◆	CRB-5	9	72.8

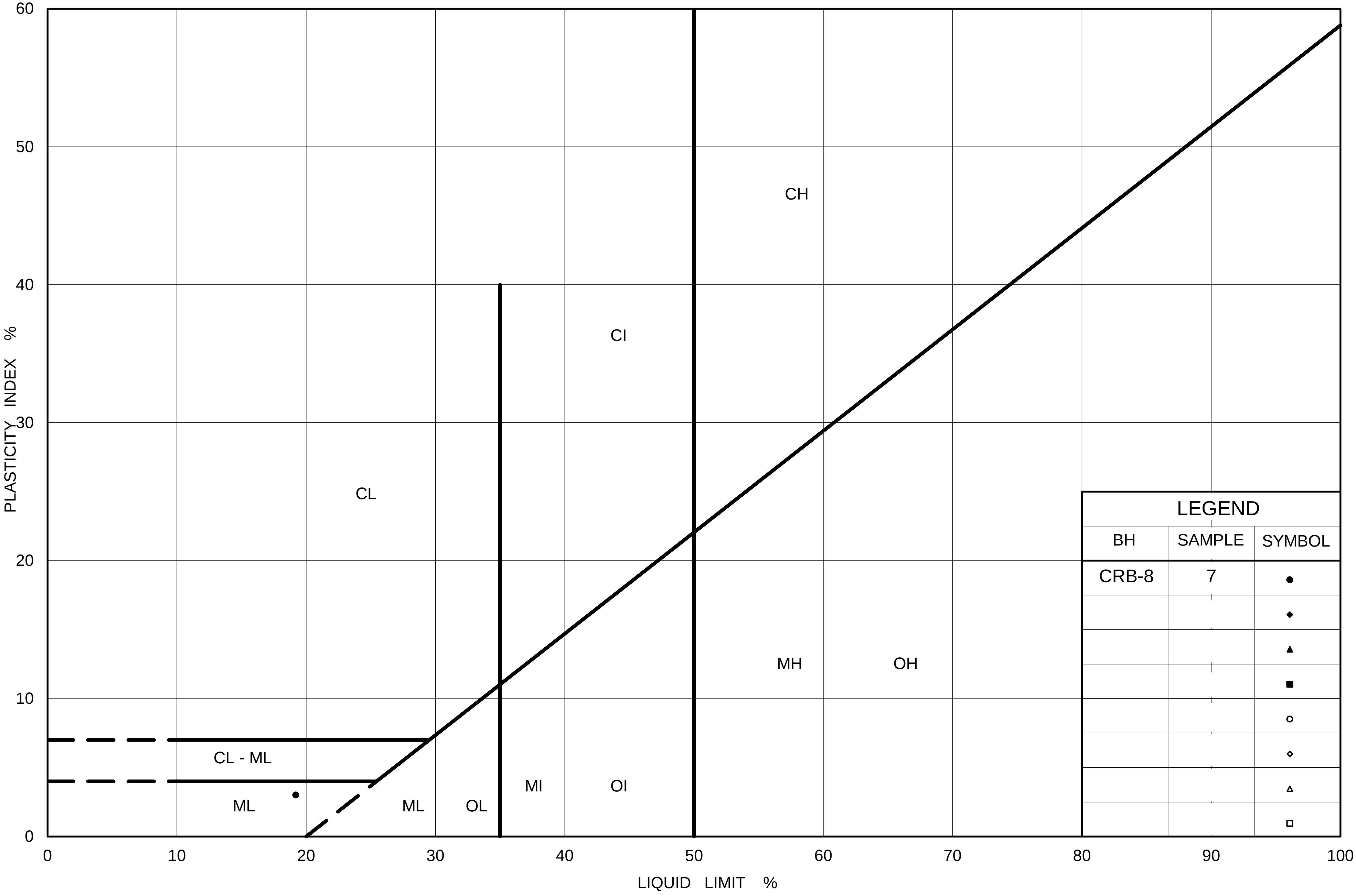
Project Number: 1662333

Checked By: SMM

Golder Associates

Date: 21-Feb-19

Oct 75, FF-S-21



Ministry of Transportation

Ontario

PLASTICITY CHART
Silt (Slight Plasticity)

Figure No. C-7

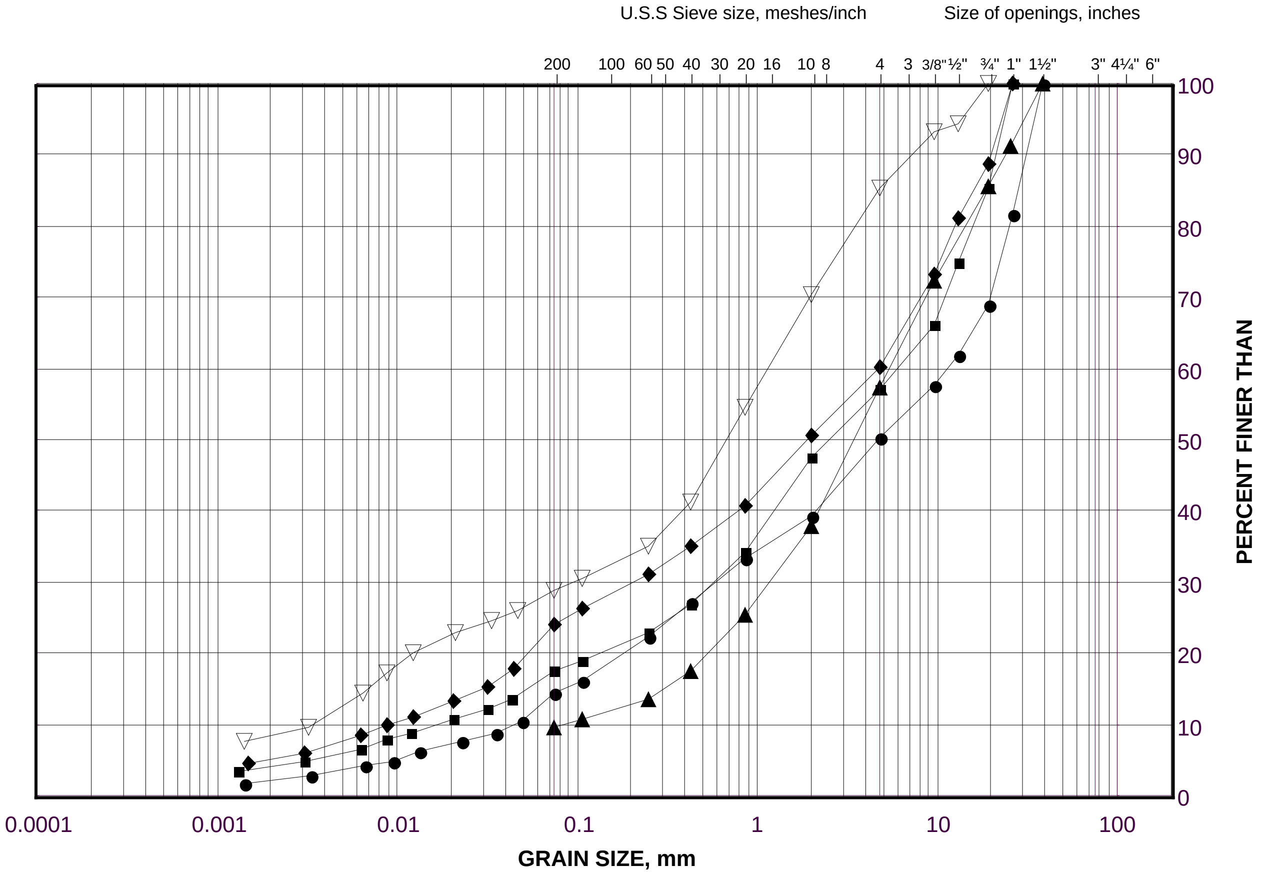
Project No. 1662333

Checked By: SMM

GRAIN SIZE DISTRIBUTION

Sand and Gravel to Sand

FIGURE C-8



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	CRB-3	5	72.6
■	CRB-3A	6	70.8
◆	CBR-3C	6	71.8
▲	CRB-3	7	71.1
▽	CRB-3C	8	70.6

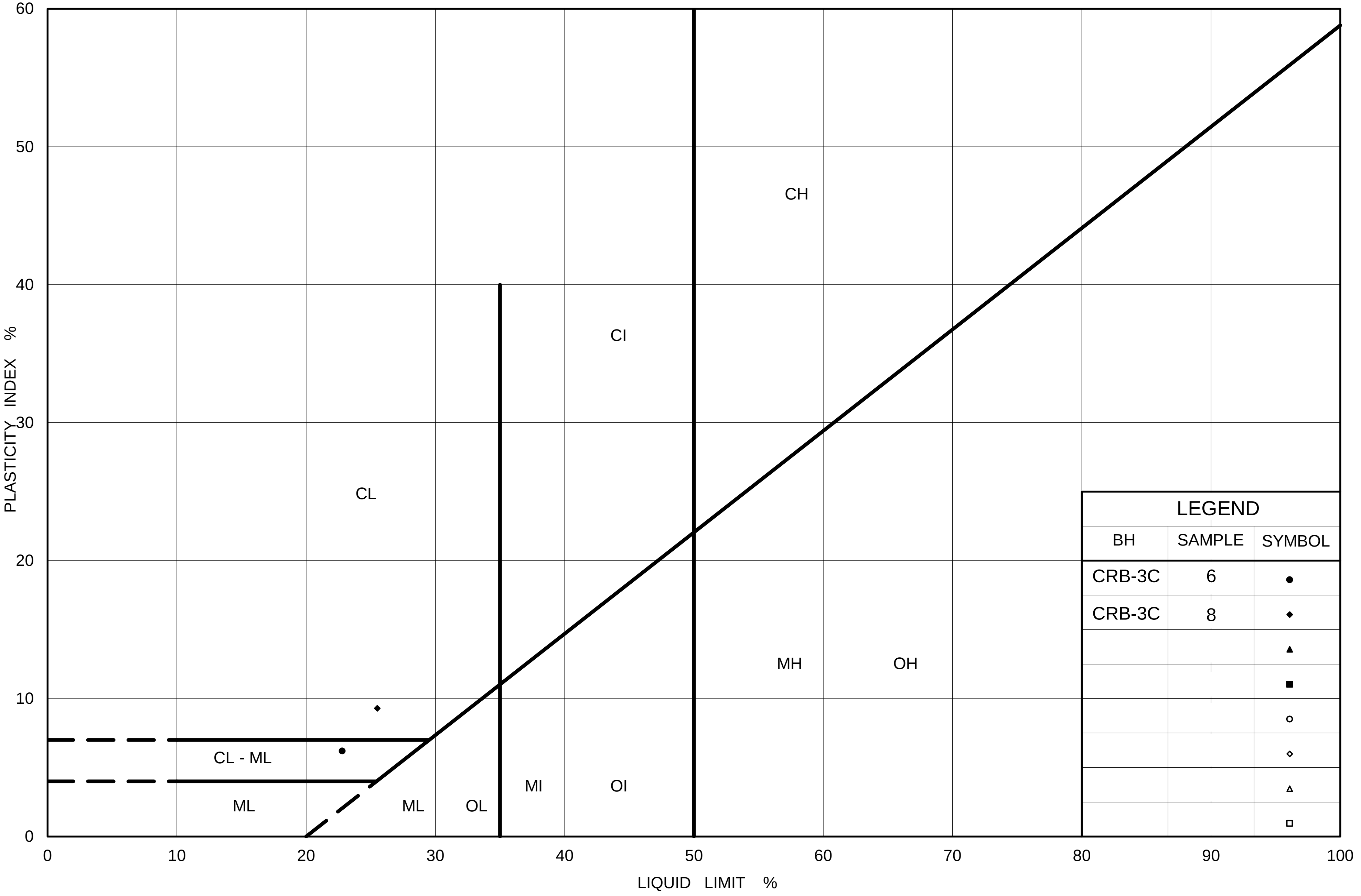
Project Number: 1662333

Checked By: SMM

Golder Associates

Date: 24-May-18

Oct 75, FF-S-21



Ministry of Transportation

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PLASTICITY CHART
Clayey Silt (Pockets)

Figure No. C-9

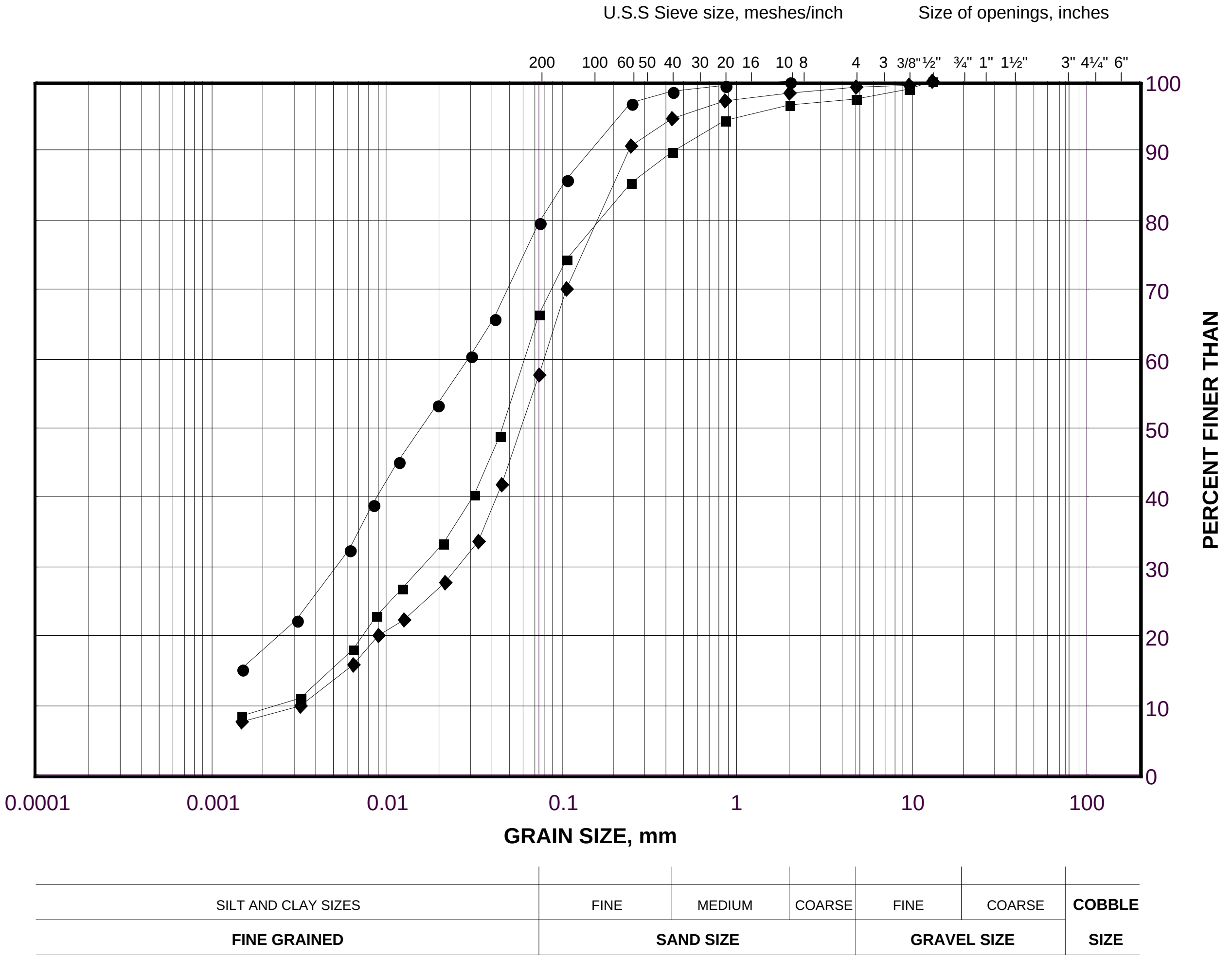
Project No. 1662333

Checked By: SMM

GRAIN SIZE DISTRIBUTION

Organic Clayey Silt to Organic Clayey Silt with Sand

FIGURE C-10



LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	CRB-5A	7	74.4
■	CRB-4	8	72.7
◆	CRB-5	8A	73.7

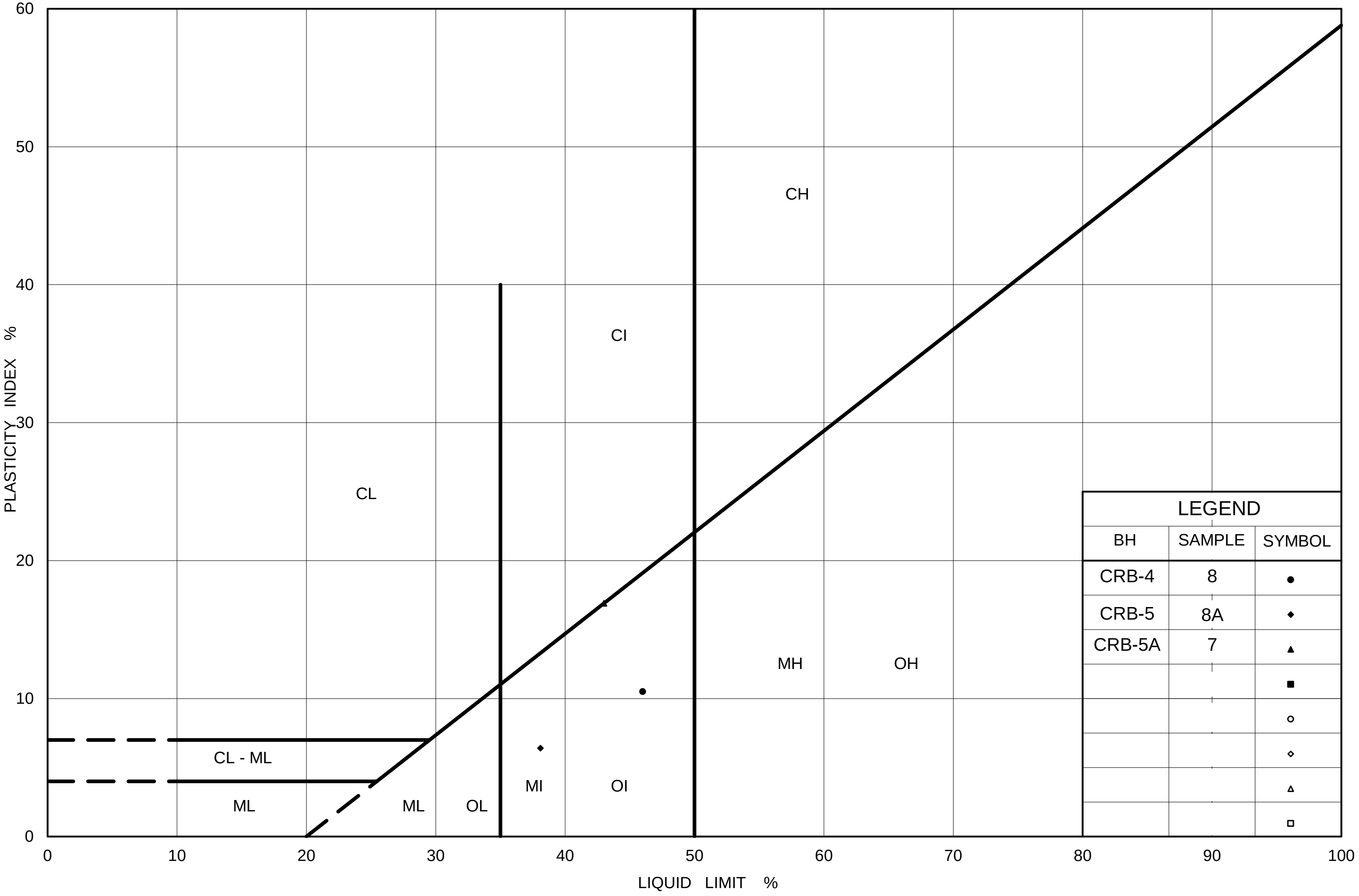
Project Number: 1662333

Checked By: SMM

Golder Associates

Date: 23-May-18

Oct 75, FF-S-21



BH	SAMPLE	SYMBOL
CRB-4	8	●
CRB-5	8A	◆
CRB-5A	7	▲
		■
		○
		◇
		△
		□

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PLASTICITY CHART
Organic Clayey Silt

Figure No. C-11

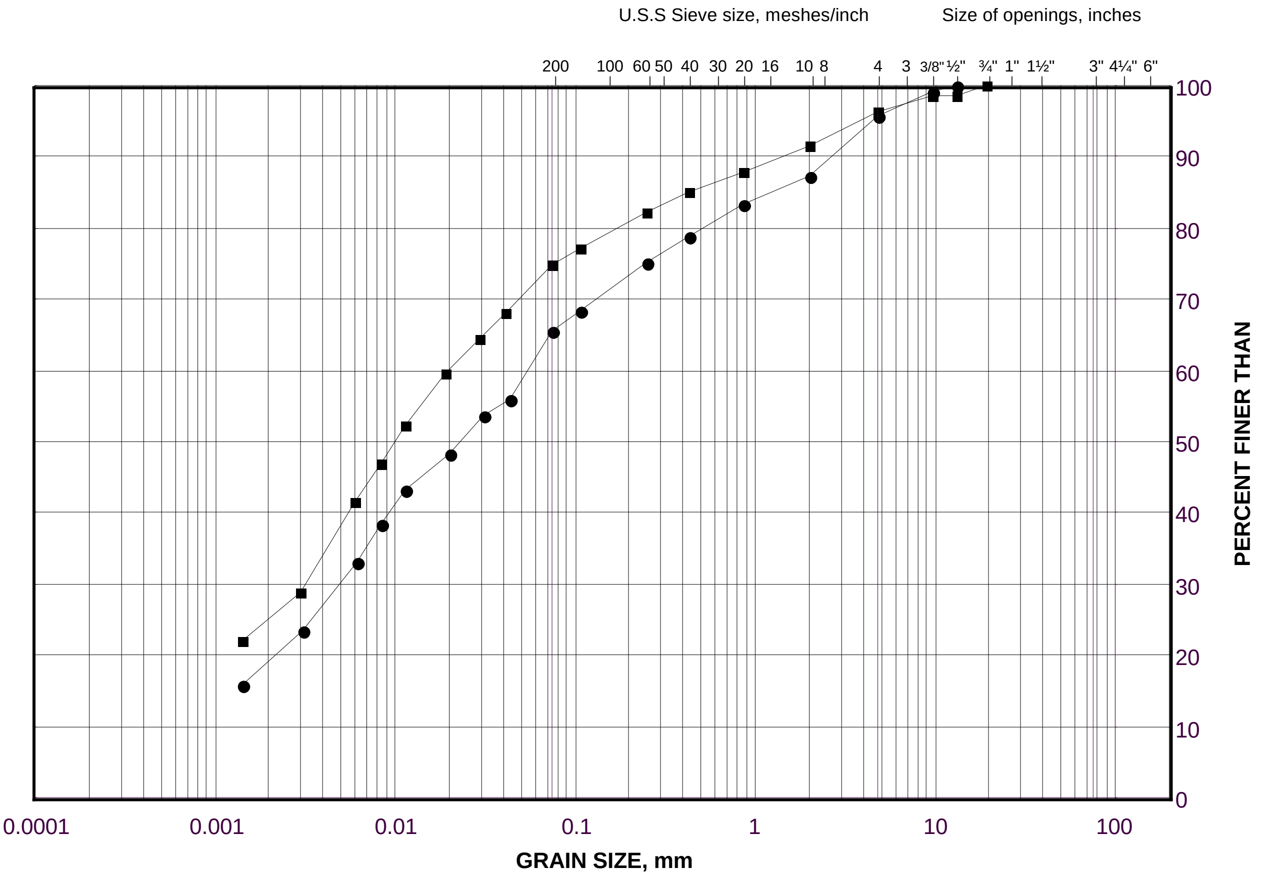
Project No. 1662333

Checked By: SMM

GRAIN SIZE DISTRIBUTION

Sandy Clayey Silt to Clayey Silt with Sand (Till)

FIGURE C-12



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	CRB-7	10	87.5
■	NW6-1	10B	87.9

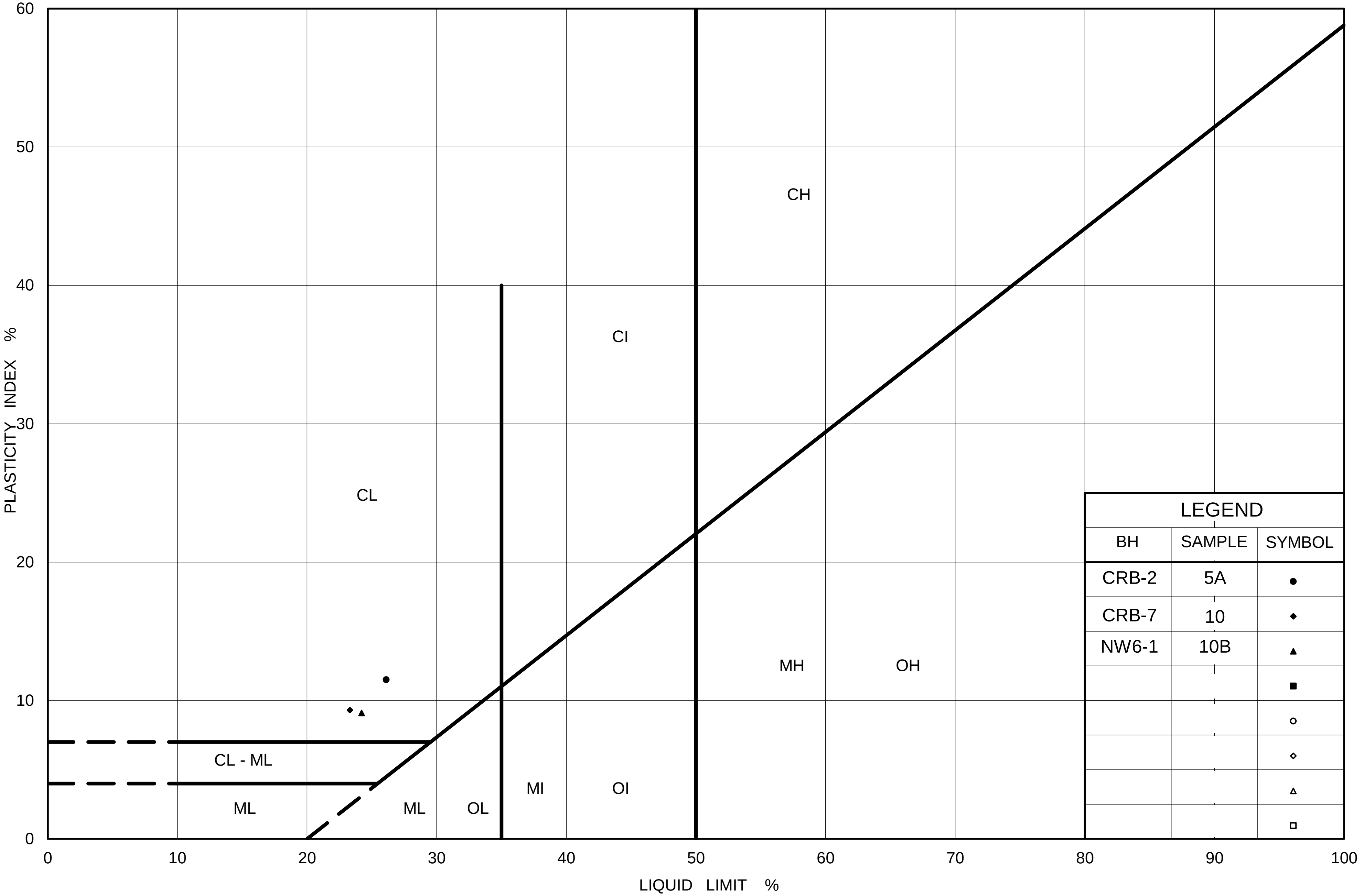
Project Number: 1662333

Checked By: SMM

Golder Associates

Date: 01-Feb-19

Oct 75, FF-S-21



LEGEND		
BH	SAMPLE	SYMBOL
CRB-2	5A	●
CRB-7	10	◆
NW6-1	10B	▲
		■
		○
		◇
		△
		□

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PLASTICITY CHART

Sandy Clayey Silt to Clayey Silt with Sand (Till)

Figure No. C-13

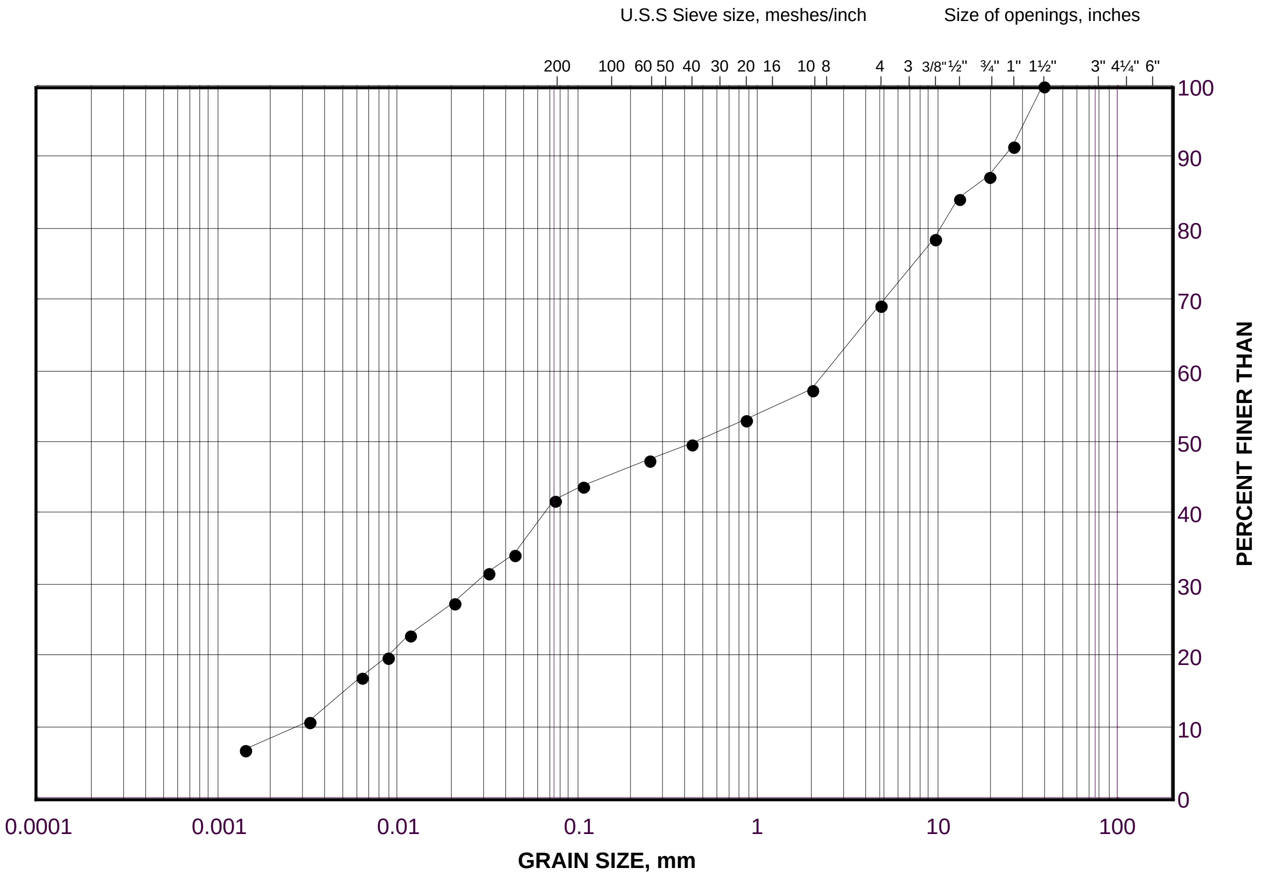
Project No. 1662333

Checked By: SMM

GRAIN SIZE DISTRIBUTION

Sandy Clayey Silt (Residual Soil)

FIGURE C-14



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	CRB-8	9A	86.9

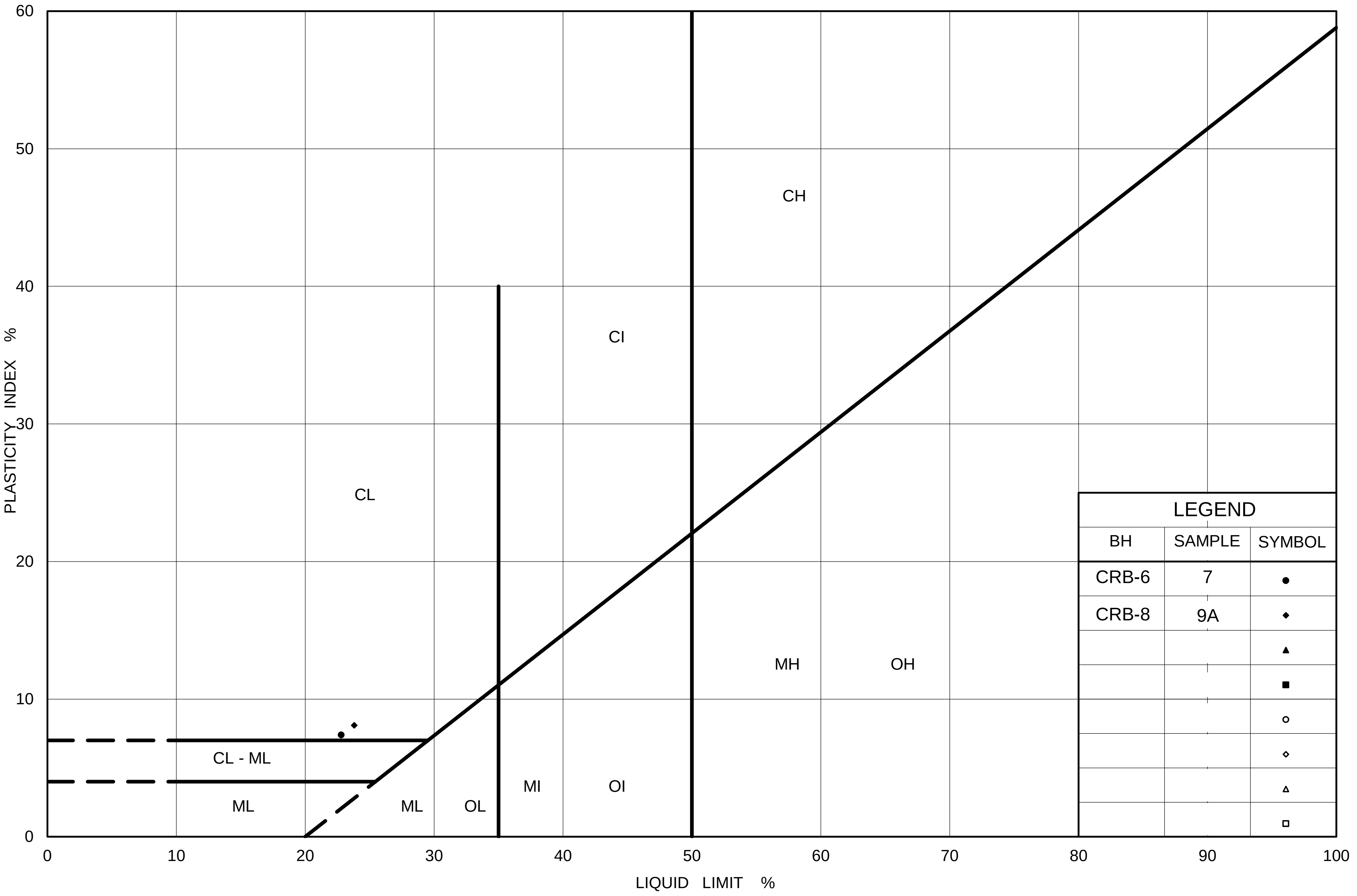
Project Number: 1662333

Checked By: SMM

Golder Associates

Date: 24-May-18

Oct 75, FF-S-21



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PLASTICITY CHART
Sandy Clayey Silt (Residual Soil)

Figure No. C-15

Project No. 1662333

Checked By: SMM

Table C1: Summary of Point Load Test Results

Borehole No.	Sample Depth (m)	Sample Elevation (m)	Orientation	Corrected Is (50 mm) (MPa)
CRB-2	8.47	87.13	Diametral	0.04
CRB-2	8.5	87.10	Axial	0.82
CRB-2	10.48	85.12	Diametral	0.19
CRB-2	11.61	83.99	Axial	0.33
CRB-2	12.22	83.38	Diametral	0.10
CRB-2	12.26	83.34	Axial	0.24
CRB-2A	7.07	87.43	Axial	0.37
CRB-2A	7.5	87.00	Diametral	0.44
CRB-2A	7.55	86.95	Axial	0.13
CRB-2A	4.81	89.69	Diametral	0.18
CRB-2A	5.82	88.68	Diametral	0.28
CRB-2A	5.87	88.63	Axial	0.40
CRB-2B	7.27	87.43	Diametral	0.44
CRB-2B	7.89	86.81	Diametral	0.15
CRB-2B	8.00	86.70	Axial	0.46
CRB-2B	9.49	85.21	Diametral	0.06
CRB-2B	10.27	84.43	Diametral	0.50
CRB-2B	10.27	84.43	Axial	0.33
CRB-3	9.2	66.70	Axial	0.30
CRB-3	9.2	66.70	Diametral	0.09
CRB-3	10.47	65.43	Axial	0.68
CRB-3	10.47	65.43	Diametral	0.22

Borehole No.	Sample Depth (m)	Sample Elevation (m)	Orientation	Corrected Is (50 mm) (MPa)
CRB-3	11.27	64.63	Axial	0.26
CRB-3	11.27	64.63	Diametral	0.85
CRB-3A	7.85	67.85	Diametral	0.62
CRB-3A	7.85	67.85	Axial	0.46
CRB-3A	10.33	65.37	Diametral	0.06
CRB-3A	10.41	65.29	Axial	1.35
CRB-3A	13.4	62.30	Diametral	0.36
CRB-3A	13.36	62.34	Axial	0.78
CRB-3C	9.12	66.18	Diametral	0.25
CRB-3C	9.24	66.06	Axial	0.28
CRB-3C	11.38	63.92	Diametral	0.24
CRB-3C	11.42	63.88	Axial	0.60
CRB-3C	13.18	62.12	Diametral	0.28
CRB-3C	13.9	61.40	Axial	0.58
CRB-4	13.5	65.60	Diametral	0.89
CRB-4	9.22	69.88	Diametral	0.04
CRB-4	9.22	69.88	Axial	1.02
CRB-4	9.28	69.82	Axial	2.14
CRB-4	14.93	64.17	Diametral	0.08
CRB-4	14.93	64.17	Axial	0.23
CRB-5	8.95	70.25	Diametral	0.76
CRB-5	9.03	70.17	Axial	0.37
CRB-5	14.48	64.72	Diametral	0.52

Borehole No.	Sample Depth (m)	Sample Elevation (m)	Orientation	Corrected Is (50 mm) (MPa)
CRB-5	14.51	64.69	Axial	0.27
CRB-5	13.31	65.89	Diametral	0.07
CRB-5	9.53	69.67	Axial	1.45
CRB-5A	10.24	69.06	Diametral	0.56
CRB-5A	10.29	69.01	Axial	0.42
CRB-5A	13.84	65.46	Diametral	0.43
CRB-5A	13.89	65.41	Axial	0.07
CRB-5A	16.8	62.50	Diametral	0.82
CRB-5A	17	62.30	Axial	0.24
CRB-6	6.03	85.67	Axial	0.70
CRB-6	6.03	85.67	Diametral	0.44
CRB-6	7.34	84.36	Axial	0.65
CRB-6	7.34	84.36	Diametral	0.51
CRB-6	9.03	82.67	Axial	0.61
CRB-6	9.03	82.67	Diametral	0.15
CRB-7	9.06	85.64	Axial	0.47
CRB-7	9.06	85.64	Diametral	0.26
CRB-7	9.46	85.24	Axial	0.56
CRB-7	9.46	85.24	Diametral	0.42
CRB-7	12.04	82.66	Axial	0.85
CRB-7	12.04	82.66	Diametral	0.32

GEOMECHANICA

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

November 22, 2017

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

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

Dear Mr. Marmor:

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

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

Sincerely,

Giovanni Grasselli Ph.D., P. Eng.

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

Rock Laboratory Testing Results

A report submitted to:

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

Prepared by:

Bryan Tatone, PhD
Omid Mahabadi, PhD
Giovanni Grasselli, PhD, PEng

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

November 22, 2017
Project number: 1662333

Abstract

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

In this document:

1 Overview

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

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

Figure 1: UCS Test setup.

2 Results

The results of the tests are summarized in Table 1. The corresponding stress-strain curves for the uniaxial compression tests are presented in Figure 2. The Young's modulus is the tangent modulus, calculated as the slope of the best fit line through ±300 data points on either side of the point representing 50% of the peak strength.

Table 1: Summary of laboratory test results.

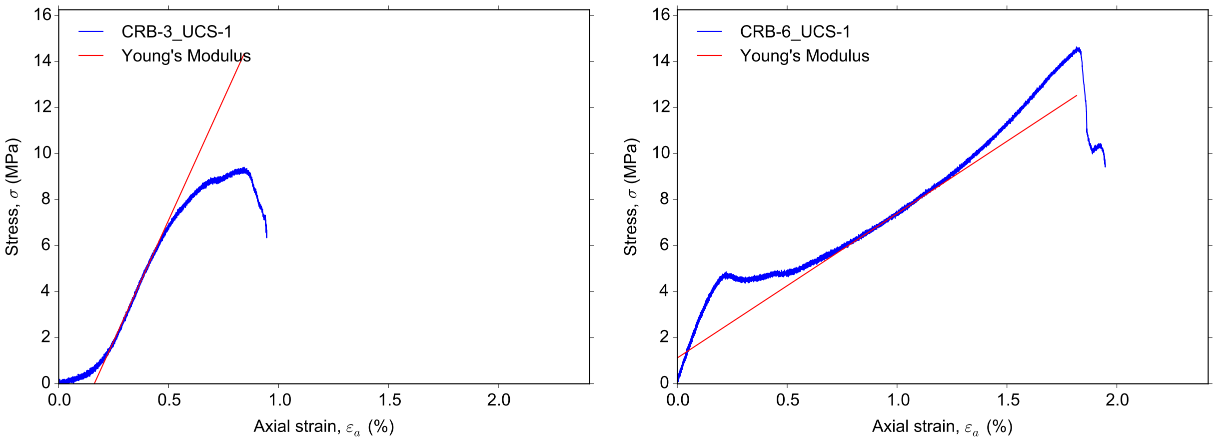
Sample	Depth (m)	Bulk density ρ (g/cm^3)	UCS (MPa)	Young's Modulus E (GPa)	Notes
CRB-3, UCS-1	11.44 - 11.66	2.61	9.4	2.10	[1]
CRB-6, UCS-1	6.06 - 6.17	2.17	14.6	0.63	[1,2]
CRB-7, UCS-1	9.21 - 9.369	2.59	15.5	0.65	[1,2]
CRB-7, UCS-3	12.11 - 12.36	2.59	7.4	1.28	
Mean		2.49	11.7	1.2	
Standard Deviation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

[1] Specimen emitted fresh pore water upon loading

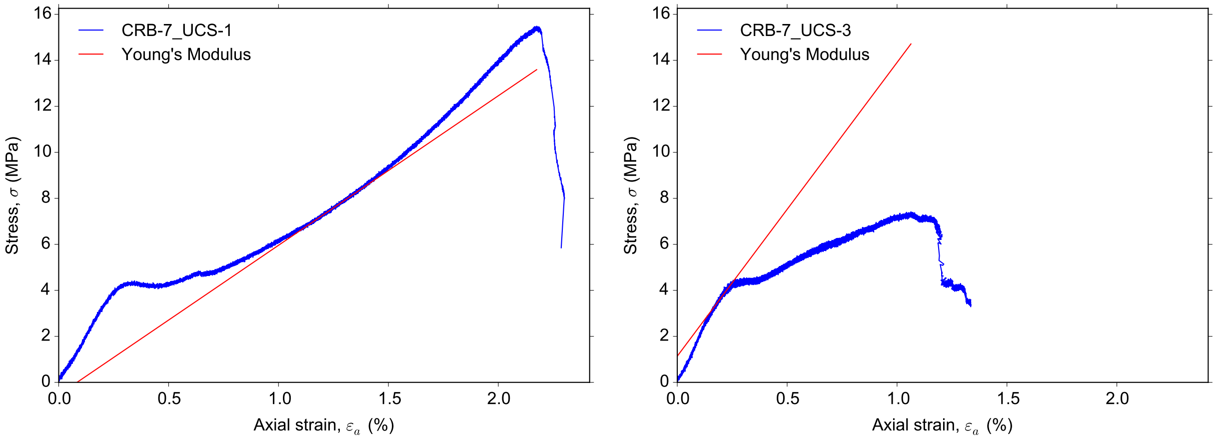
[2] length:diameter ratio < 2:1.

2.1 Specimen photographs

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



(a) CRB-3, UCS-1 (b) CRB-6, UCS-1



(c) CRB-7, UCS-1 (d) CRB-7, UCS-3

Figure 2: Measured stress-strain curves.

CRB-3, UCS-1
11.44 m – 11.66 m

CRB-6, UCS-1
6.06 m – 6.17 m

CRB-7, UCS-1
9.21 m – 9.36 m

CRB-7, UCS-3
12.11 m – 12.36 m

Figure 3: Photographs of specimens prior to testing.

GEOMECHANICA

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

April 09, 2018

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

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

Dear Mr. Marmor:

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

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

Sincerely,

Bryan Tatone Ph.D., P. Eng.

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

Rock Laboratory Testing Results

A report submitted to:

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

Prepared by:

Bryan Tatone, PhD, PEng
Omid Mahabadi, PhD, PEng

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

April 9, 2018
Project number: 1662333

Abstract

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

In this document:

1 Overview

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

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

2 Results

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

Table 1: Summary of laboratory test results.

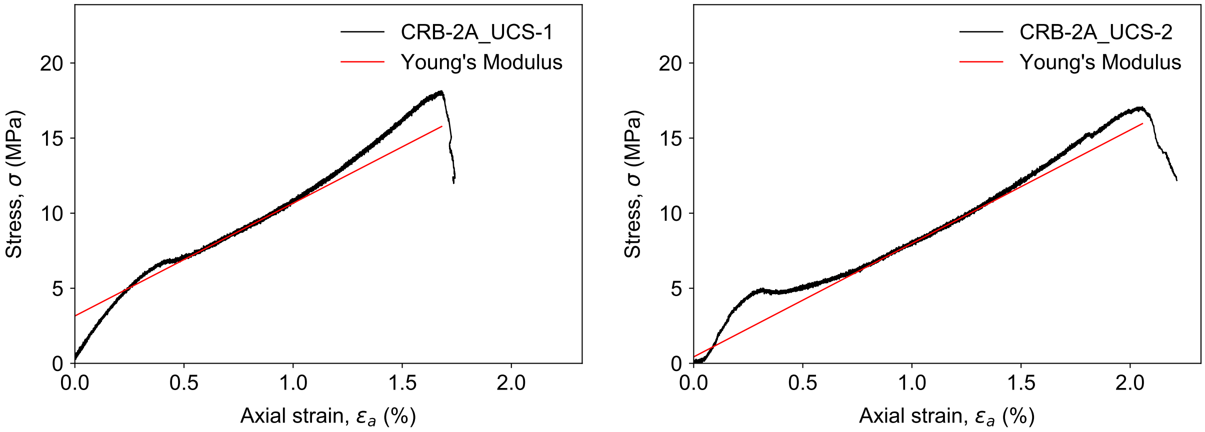
Sample	Rock (m)	Depth type	Bulk density ρ (g/cm^3)	UCS (MPa)	Young's Modulus E (GPa)	Notes
CRB-2A, UCS-1	Shale	4.31 - 4.46	2.59	18.2	0.75	1, 2
CRB-2A, UCS-2	Shale	4.92 - 5.15	2.60	17.1	0.76	1
CRB-3C, UCS-3	Limestone	7.87 - 7.98	2.61	114.1	22.91	2, 3
CRB-2, UCS-2	Shale	7.75 - 7.92	2.58	11.2	0.83	1
CRB-2,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

[1] Upon loading specimen emitted pore water

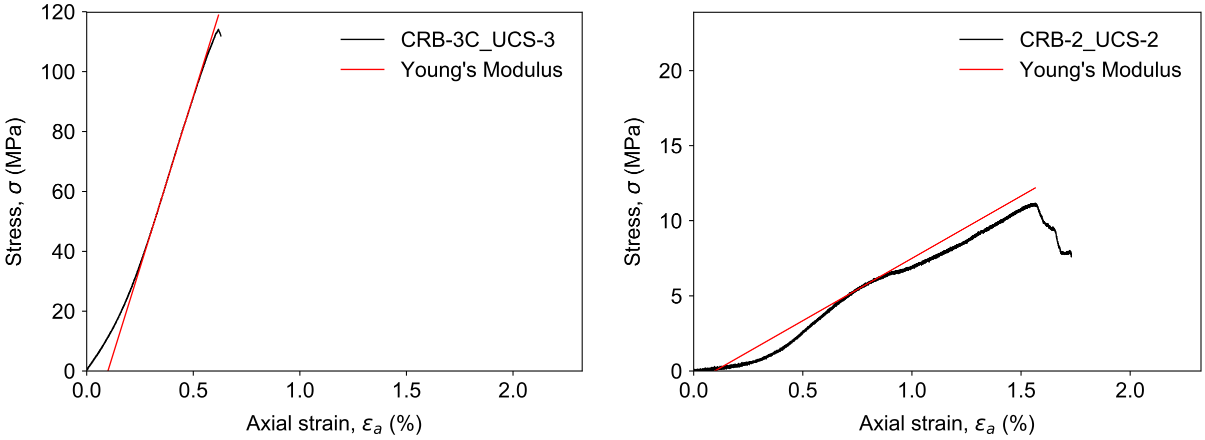
[2] Irregular diameter > 0.5 mm

[3] Length:Diameter ratio less than 2

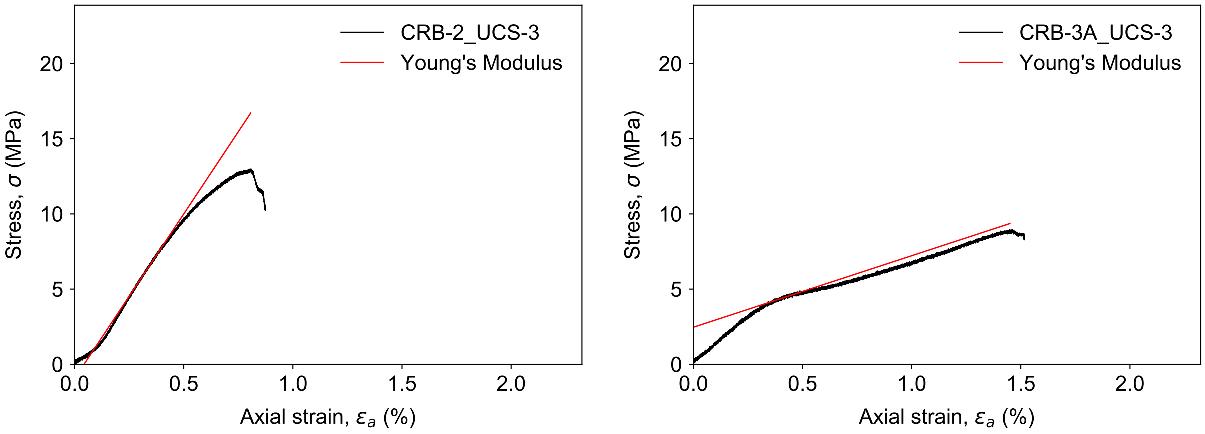
[4] Inter-bedded limestone and shale



(a) CRB-2A, UCS-1 (b) CRB-2A, UCS-2

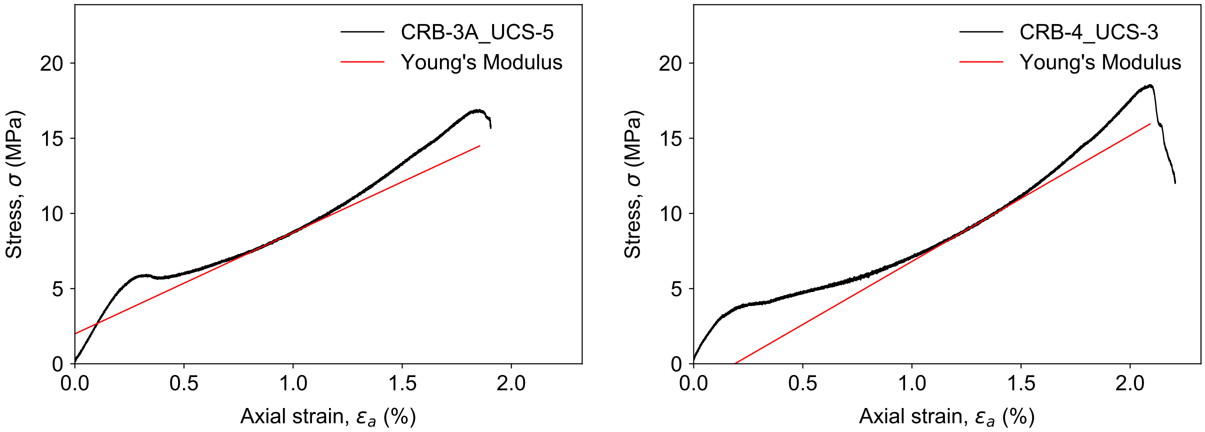


(c) CRB-3C, UCS-3 (d) CRB-2, UCS-2

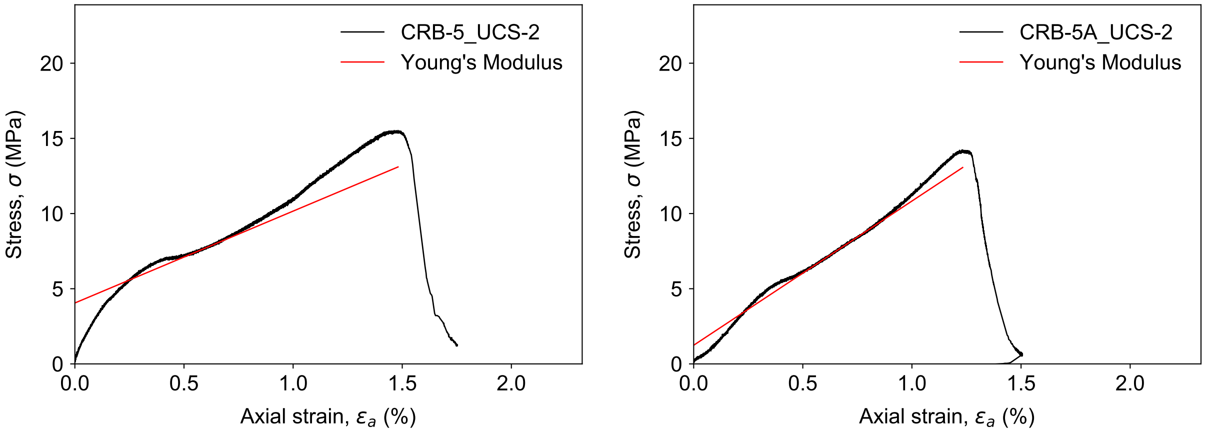


(e) CRB-2, UCS-3 (f) CRB-3A, UCS-3

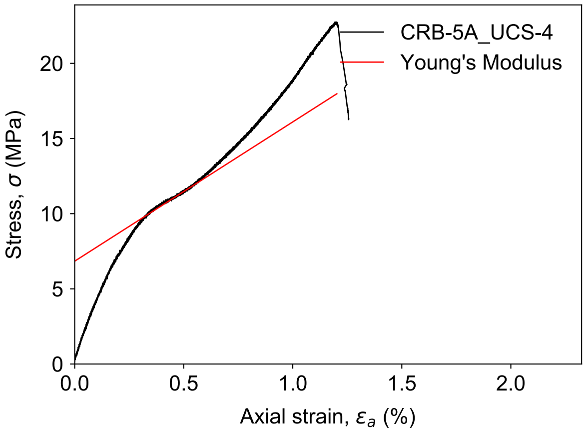
Figure 1: Measured stress-strain curves.



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



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



(e) CRB-5A, UCS-4

Figure 2: Measured stress-strain curves.

2.1 Specimen photographs

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

Figure 3: Photographs of specimens prior to testing.

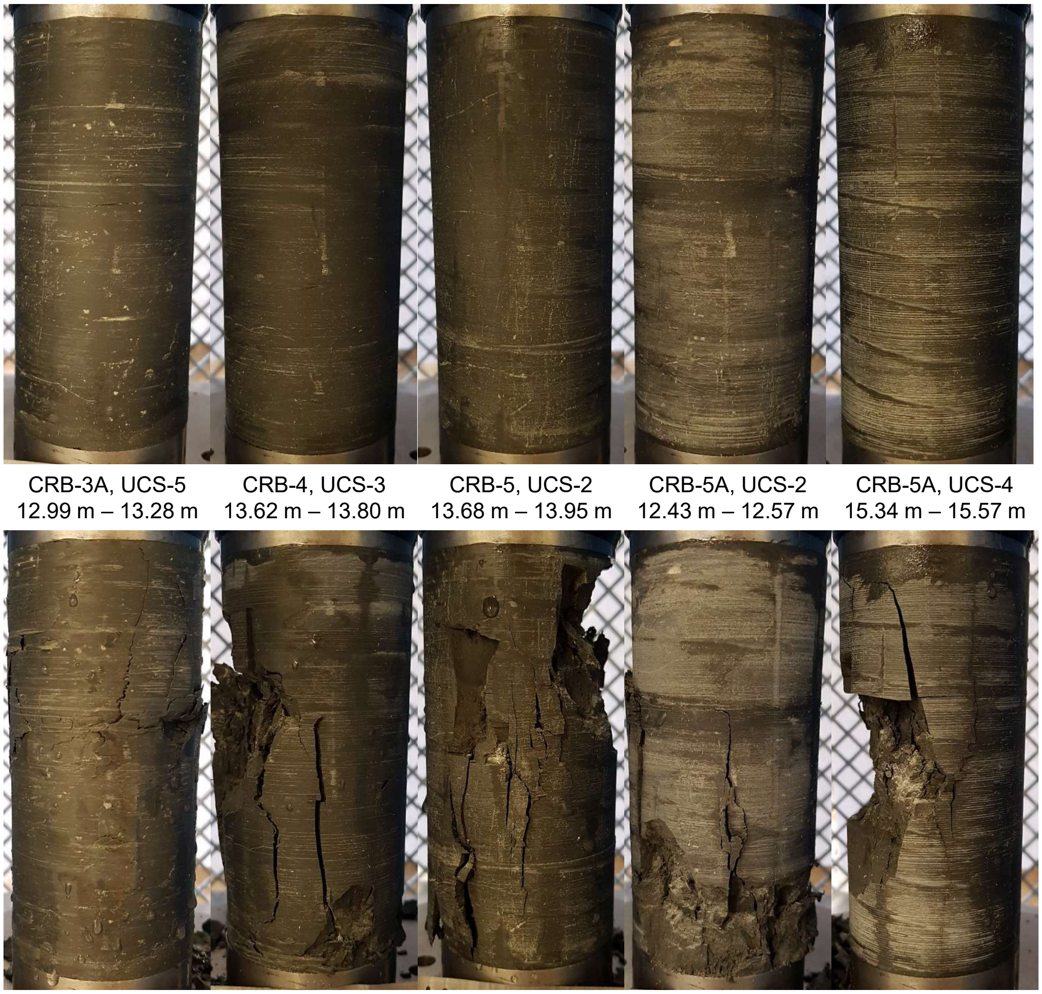


Figure 4: Photographs of failed specimens after testing.

GEOMECHANICA

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

July 26, 2018

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

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

Dear Mr. Marmor:

On July 18, 2018 two (2) HQ3-sized core samples were received by Geomechanica Inc. via drop-off by Golder personnel. These samples were identified as being from boreholes drilled as part of Golder project 1662333. A uniaxial compressive strength (UCS) specimen was prepared and tested from each of these samples (2 tests total).

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

Sincerely,

Bryan Tatone Ph.D., P. Eng.

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

GEOMECHANICA

Rock Laboratory Testing Results

A report submitted to:

David Marmor

Golder Associates Limited
6925 Century Avenue, Suite #100
Mississauga, Ontario
Canada L5N 7K2

Prepared by:

Bryan Tatone, PhD, PEng
Omid Mahabadi, PhD, PEng

Geomechanica Inc
#900-390 Bay St
Toronto ON
M5H 2Y2 Canada
Tel: +1-647-478-9767
info@geomechanica.com

July 26, 2018
Project number: 1662333

Abstract

This document summarizes the results of rock laboratory testing of 2 uniaxial compressive strength (UCS) tests. Results, including uniaxial compressive strength (UCS) and Young's modulus along with photographs of samples before and after testing are presented. Additional specimen information is included in an accompaning summary spreadsheet.

In this document:

1 Uniaxial Compressive Strength (UCS) testing

This report summarizes the results of 2 uniaxial compressive strength (UCS) tests. The testing was performed in Geomechanica's rock testing laboratory using a 150 ton (1.3 MN) Forney loading frame equipped with pressure-compensated control valve to maintain an axial displacement rate of approximately 0.15 mm/min (Figure 1). This displacement rate was selected to target specimen failure to occur within 2 - 15 minutes.

The specimen preparation and testing procedure included the following:

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

Figure 1: UCS test setup.

1.1 Results

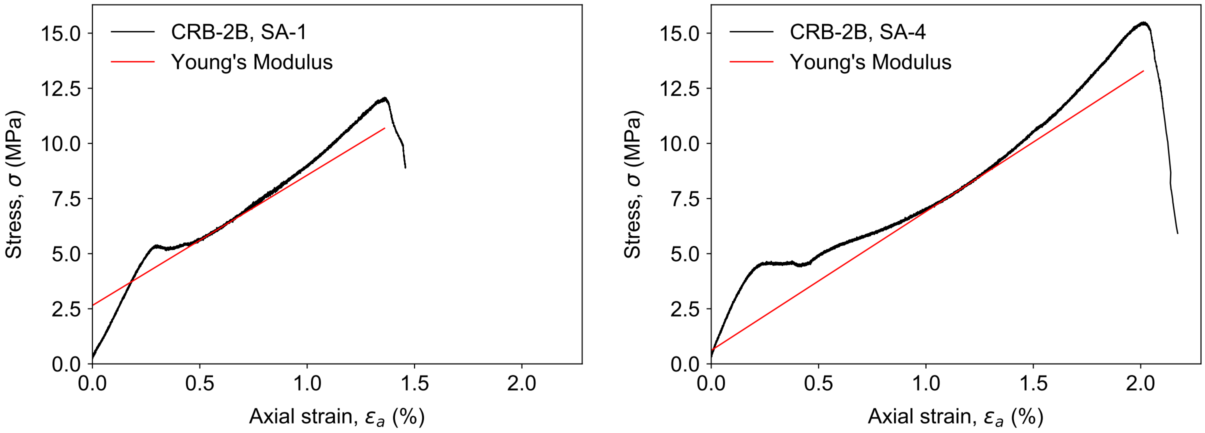
The results of the tests are summarized in Table 1. The corresponding stress-strain curves for the uniaxial compression tests are presented in Figure 2. Young's modulus is the tangent modulus, calculated as the slope of the best fit line through ±300 data points on either side of the point representing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

Table 1: Summary of laboratory test results.

Sample	Depth (m)	Lithology description	Bulk density ρ (g/cm^3)	UCS (MPa)	Young's Modulus E (GPa)	Failure description
CRB-2B-SA-1	6.89 - 7.06	Shale	2.593	12.1	0.59	Diffuse axial splitting[1, 2]
CRB-2B-SA-4	9.10 - 9.25	Shale	2.601	15.5	0.63	Inclined shear band[1, 2]

[1] Specimen side straightness (i.e., diameter) varied up to 0.6 mm

[2] Specimen emitted pore water upon loading



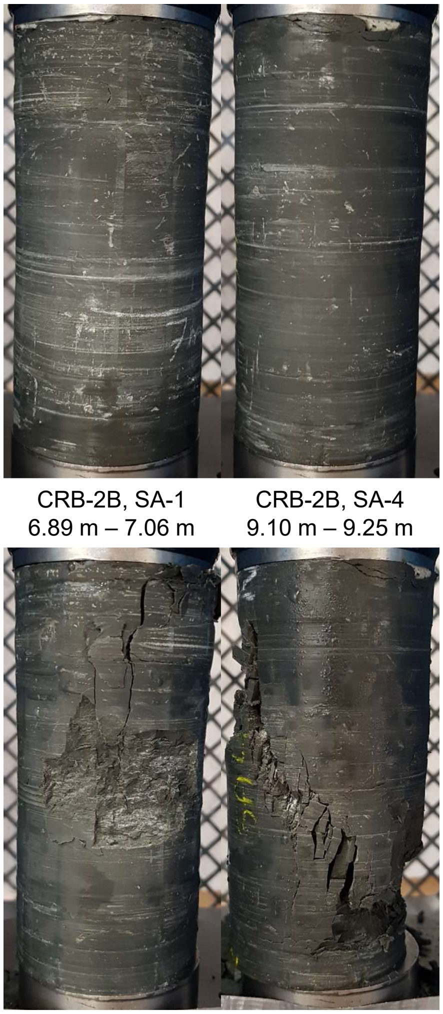
(a) CRB-2B, SA-1

(b) CRB-2B, SA-4

Figure 2: Measured stress-strain curves.

1.2 Specimen photographs

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



CRB-2B, SA-1
6.89 m – 7.06 m

CRB-2B, SA-4
9.10 m – 9.25 m

Figure 3: Photographs of specimens before and after testing.

Your Project #: 1662333
Site Location: QEW/CREDIT RIVER
Your C.O.C. #: 51329

Attention:David Marmor

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

Report Date: 2017/11/21
Report #: R4869236
Version: 1 - Final

CERTIFICATE OF ANALYSIS

MAXXAM JOB #: B7P4571
Received: 2017/11/13, 12:50

Sample Matrix: Soil
Samples Received: 3

Analyses	Quantity	Date Extracted	Date Analyzed	Laboratory Method	Reference
Chloride (20:1 extract)	3	N/A	2017/11/17	CAM SOP-00463	EPA 325.2 m
Conductivity	3	N/A	2017/11/20	CAM SOP-00414	OMOE E3530 v1 m
pH CaCl2 EXTRACT	3	2017/11/17	2017/11/17	CAM SOP-00413	EPA 9045 D m
Resistivity of Soil	3	2017/11/13	2017/11/20	CAM SOP-00414	SM 22 2510 m
Sulphate (20:1 Extract)	3	N/A	2017/11/17	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.

Your Project #: 1662333
Site Location: QEW/CREDIT RIVER
Your C.O.C. #: 51329

Attention:David Marmor

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

Report Date: 2017/11/21
Report #: R4869236
Version: 1 - Final

CERTIFICATE OF ANALYSIS

MAXXAM JOB #: B7P4571
Received: 2017/11/13, 12:50

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 #: B7P4571
Report Date: 2017/11/21

Golder Associates Ltd
Client Project #: 1662333
Site Location: QEW/CREDIT RIVER
Sampler Initials: JC

RESULTS OF ANALYSES OF SOIL

Maxxam ID		FNR708			FNR708		FNR709	FNR710		
Sampling Date		2017/10/16 16:00			2017/10/16 16:00		2017/10/20 10:00	2017/10/26 13:30		
COC Number		51329			51329		51329	51329		
	UNITS	**NW3-01 SA7**	**RDL**	**QC Batch**	**NW3-01 SA7 Lab-Dup**	**QC Batch**	**CRB-06 RC-01 6.00-6.05**	**PED-03 SA8**	**RDL**	**QC Batch**
Calculated Parameters										
Resistivity	ohm-cm	490		5263307			5000	1300		5263307
Inorganics										
Soluble (20:1) Chloride (Cl)	ug/g	1000	40	5268736			<20	350	20	5268736
Conductivity	umho/cm	2040	2	5273678			201	762	2	5273678
Available (CaCl2) pH	pH	7.86		5270614	7.93	5270614	8.11	7.73		5270614
Soluble (20:1) Sulphate (SO4)	ug/g	69	20	5268737			30	70	20	5268737

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

TEST SUMMARY

Maxxam ID: FNR708
Sample ID: NW3-01 SA7
Matrix: Soil

Collected: 2017/10/16
Shipped:
Received: 2017/11/13

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	5268736	N/A	2017/11/17	Deonarine Ramnarine
Conductivity	AT	5273678	N/A	2017/11/20	Neil Dassanayake
pH CaCl2 EXTRACT	AT	5270614	2017/11/17	2017/11/17	Tahir Anwar
Resistivity of Soil		5263307	2017/11/20	2017/11/20	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	5268737	N/A	2017/11/17	Deonarine Ramnarine

Maxxam ID: FNR708 Dup
Sample ID: NW3-01 SA7
Matrix: Soil

Collected: 2017/10/16
Shipped:
Received: 2017/11/13

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
pH CaCl2 EXTRACT	AT	5270614	2017/11/17	2017/11/17	Tahir Anwar

Maxxam ID: FNR709
Sample ID: CRB-06 RC-01 6.00-6.05
Matrix: Soil

Collected: 2017/10/20
Shipped:
Received: 2017/11/13

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	5268736	N/A	2017/11/17	Deonarine Ramnarine
Conductivity	AT	5273678	N/A	2017/11/20	Neil Dassanayake
pH CaCl2 EXTRACT	AT	5270614	2017/11/17	2017/11/17	Tahir Anwar
Resistivity of Soil		5263307	2017/11/20	2017/11/20	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	5268737	N/A	2017/11/17	Deonarine Ramnarine

Maxxam ID: FNR710
Sample ID: PED-03 SA8
Matrix: Soil

Collected: 2017/10/26
Shipped:
Received: 2017/11/13

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	5268736	N/A	2017/11/17	Deonarine Ramnarine
Conductivity	AT	5273678	N/A	2017/11/20	Neil Dassanayake
pH CaCl2 EXTRACT	AT	5270614	2017/11/17	2017/11/17	Tahir Anwar
Resistivity of Soil		5263307	2017/11/20	2017/11/20	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	5268737	N/A	2017/11/17	Deonarine Ramnarine

GENERAL COMMENTS

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

Package 1	5.3°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**
5268736	Soluble (20:1) Chloride (Cl)	2017/11/17	NC	70 - 130	103	70 - 130	<20	ug/g	14	35
5268737	Soluble (20:1) Sulphate (SO4)	2017/11/17	NC	70 - 130	107	70 - 130	<20	ug/g	13	35
5270614	Available (CaCl2) pH	2017/11/17			99	97 - 103			0.85	N/A
5273678	Conductivity	2017/11/20			100	90 - 110	<2	umho/cm	0	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)

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

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Phone: 905-817-5700 Fax: 905-817-5779 Toll Free: 800-563-6266

CHAIN OF CUSTODY RECORD

51329

Page 1 of 1

INVOICE INFORMATION

Company Name: Golder Associates
Contact Name: David Marmor
Address: 6925 Century Ave
Suite #1000 Mississauga
Phone: 705-792-8203 Fax: 9055676561
Email: david-marmor@golder.com

REPORT INFORMATION (if differs from invoice)

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

PROJECT INFORMATION

Quotation #:
P.O. #: 1662333
Project #: 1662333
Site Location: QEW/Credit River
Site #:
Sampled By: Jeremy Lebow

TURNAROUND TIME (TAT) REQUIRED

☒ Regular TAT (5-7 days)

PLEASE PROVIDE ADVANCE NOTICE FOR RUSH PROJECTS

Rush TAT (Applicable Surcharge)
☐ 1 Day (100%)
☐ 2 Days (50%)
☐ 3-4 Days (25%)

Rush Confirmation #:
Date Required:

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

FOR RSC (PLEASE CIRCLE) YES / NO

OTHER REGULATIONS

☐ CCME
☐ MISA
☐ PWQO
☐ Other (Specify):
☐ REG 558 (MINIMUM 3 DAY TAT REQUIRED)
☐ Sanitary Sewer Bylaw
☐ Storm Sewer Bylaw
Municipality:

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

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

ANALYSIS REQUESTED

	SAMPLE IDENTIFICATION	DATE SAMPLED	TIME SAMPLED	MATRIX	# OF CONT.	FIELD FILTERED (PLEASE CIRCLE) Metals / Hg / CrVI	Standard Corrosivity pH, Chloride, Sulphates, Conductivity/Resistivity
1	NW3-01 Sa 7	17/10/16	4 pm	Soil	1		X
2	CRB-06 RC-01 6.00-6.05	17/10/20	10 am	Soil/Rock	1		X
3	PED-03 Sa 8	17/10/26	1:30pm	Soil	1		X
4							
5							
6							
7							
8							
9							
10							

LABORATORY USE ONLY

CUSTODY SEAL (Y/N) N
Present
Intact
COOLING MEDIA PRESENT (Y / N) Y
Temperature (°C) on Receipt: 4/5/7

COMMENTS / TAT COMMENTS

13-Nov-17 12:50
Ema Gitej
B7P4571
TSP ENV-980

RELINQUISHED BY: (Signature/Print)	DATE: (YYYY/MM/DD)	TIME:	RECEIVED BY: (Signature/Print)	DATE: (YYYY/MM/DD)	TIME:	# JARS USED AND NOT SUBMITTED	MAXXAM JOB #
	2017/11/13	1250		2017/11/13	12:50		

COC-1004 (11/13) - ENV ENG.

Maxxam Analytics International Corporation o/a Maxxam Analytics

White: Maxxam - Yellow: Client

Your Project #: 1662333
Site Location: QEW/CRB
Your C.O.C. #: 35606

Attention: David Marmor

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

Report Date: 2018/04/26
Report #: R5092302
Version: 1 - Final

CERTIFICATE OF ANALYSIS

MAXXAM JOB #: B886277
Received: 2018/04/17, 14:35

Sample Matrix: Soil
Samples Received: 3

Analyses	Quantity	Date Extracted	Date Analyzed	Laboratory Method	Reference
Chloride (20:1 extract)	3	N/A	2018/04/22	CAM SOP-00463	EPA 325.2 m
Conductivity	3	N/A	2018/04/19	CAM SOP-00414	OMOE E3530 v1 m
pH CaCl2 EXTRACT	3	2018/04/20	2018/04/20	CAM SOP-00413	EPA 9045 D m
Resistivity of Soil	3	2018/04/17	2018/04/19	CAM SOP-00414	SM 23 2510 m
Sulphate (20:1 Extract)	3	N/A	2018/04/20	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 #: 1662333
Site Location: QEW/CRB
Your C.O.C. #: 35606

Attention: David Marmor

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

Report Date: 2018/04/26
Report #: R5092302
Version: 1 - Final

CERTIFICATE OF ANALYSIS

MAXXAM JOB #: B886277
Received: 2018/04/17, 14:35

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.

Maxxam Job #: B886277
Report Date: 2018/04/26

Golder Associates Ltd
Client Project #: 1662333
Site Location: QEW/CRB
Sampler Initials: JL

SOIL CORROSIVITY PACKAGE (SOIL)

Maxxam ID		GLY689	GLY690	GLY691		
Sampling Date		2018/02/14 15:00	2018/02/09 14:00	2018/02/06 12:00		
COC Number		35606	35606	35606		
	UNITS	**CRB_5_13.31-13.41**	**CRB_3A_11.10-11.28**	**CRB_2_6.10-6.24**	**RDL**	**QC Batch**
Calculated Parameters						
Resistivity	ohm-cm	1700	3100	3800		5488254
Inorganics						
Soluble (20:1) Chloride (Cl)	ug/g	46	44	<20	20	5493401
Conductivity	umho/cm	582	321	263	2	5491303
Available (CaCl2) pH	pH	8.08	8.07	8.09		5492017
Soluble (20:1) Sulphate (SO4)	ug/g	160	49	56	20	5493410

RDL = Reportable Detection Limit
QC Batch = Quality Control Batch

Maxxam ID		GLY691		
Sampling Date		2018/02/06 12:00		
COC Number		35606		
	UNITS	**CRB_2_6.10-6.24 Lab-Dup**	**RDL**	**QC Batch**
Inorganics				
Soluble (20:1) Chloride (Cl)	ug/g	<20	20	5493401
Soluble (20:1) Sulphate (SO4)	ug/g	58	20	5493410

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

TEST SUMMARY

Maxxam ID: GLY689
Sample ID: CRB_5_13.31-13.41
Matrix: Soil

Collected: 2018/02/14
Shipped:
Received: 2018/04/17

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	5493401	N/A	2018/04/22	Deonarine Ramnarine
Conductivity	AT	5491303	N/A	2018/04/19	Tahir Anwar
pH CaCl2 EXTRACT	AT	5492017	2018/04/20	2018/04/20	Gnana Thomas
Resistivity of Soil		5488254	2018/04/19	2018/04/19	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	5493410	N/A	2018/04/20	Alina Dobreanu

Maxxam ID: GLY690
Sample ID: CRB_3A_11.10-11.28
Matrix: Soil

Collected: 2018/02/09
Shipped:
Received: 2018/04/17

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	5493401	N/A	2018/04/22	Deonarine Ramnarine
Conductivity	AT	5491303	N/A	2018/04/19	Tahir Anwar
pH CaCl2 EXTRACT	AT	5492017	2018/04/20	2018/04/20	Gnana Thomas
Resistivity of Soil		5488254	2018/04/19	2018/04/19	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	5493410	N/A	2018/04/20	Alina Dobreanu

Maxxam ID: GLY691
Sample ID: CRB_2_6.10-6.24
Matrix: Soil

Collected: 2018/02/06
Shipped:
Received: 2018/04/17

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	5493401	N/A	2018/04/22	Deonarine Ramnarine
Conductivity	AT	5491303	N/A	2018/04/19	Tahir Anwar
pH CaCl2 EXTRACT	AT	5492017	2018/04/20	2018/04/20	Gnana Thomas
Resistivity of Soil		5488254	2018/04/19	2018/04/19	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	5493410	N/A	2018/04/20	Alina Dobreanu

Maxxam ID: GLY691 Dup
Sample ID: CRB_2_6.10-6.24
Matrix: Soil

Collected: 2018/02/06
Shipped:
Received: 2018/04/17

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	5493401	N/A	2018/04/22	Deonarine Ramnarine
Sulphate (20:1 Extract)	KONE/EC	5493410	N/A	2018/04/20	Alina Dobreanu

GENERAL COMMENTS

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

Package 1	8.7°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**
5491303	Conductivity	2018/04/19			101	90 - 110	<2	umho/cm	0.69	10
5492017	Available (CaCl2) pH	2018/04/20			99	97 - 103			0.37	N/A
5493401	Soluble (20:1) Chloride (Cl)	2018/04/22	116	70 - 130	106	70 - 130	<20	ug/g	NC	35
5493410	Soluble (20:1) Sulphate (SO4)	2018/04/20	NC	70 - 130	101	70 - 130	<20	ug/g	3.1	35

N/A = Not Applicable

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

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

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

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

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

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).

Cristina Carriere

Cristina Carriere, Scientific Service 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 6740 Campobello Road, Mississauga, ON L5N 2L8
Phone: 905-817-5700 Fax: 905-817-5778 Toll Free: (800) 563-6266

CHAIN OF CUSTODY RECORD

35606

Page 1 of 1

INVOICE INFORMATION

Company Name: Golder Associates
Contact Name: David Marmor
Address: 6925 Century ave. #100
Mississauga, ON L5N7K2
Phone: 905-567-4444 Fax:
Email: david_marmor@golder.com

REPORT INFORMATION (if differs from invoice)

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

PROJECT INFORMATION

Quotation #:
P.O. #:
Project #: 1662333
Project Name: QEW/CRB
Location:
Sampled By: Jeremy Lebow

MAXXAM JOB NUMBER

CHAIN OF CUSTODY #

00

REGULATORY CRITERIA

Note: For regulated drinking water samples - please use the Drinking Water Chain of Custody Form.

☐ MISA
☐ PWQO
☐ Reg. 558

Reg. 153
☐ Table 1
☐ Table 2
☐ Table 3
☐ Table 6

☐ Residential / Parkland
☐ Industrial / Commercial
☐ Medium / Fine
☐ Coarse

Sewer Use
☐ Sanitary
☐ Storm
Municipality:

Reg. 153
☐ 2004 ☐ 2011

Report Criteria on C of A? ☐

Other (specify):

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 to 7 Working Days

Rush TAT: Rush Confirmation #: (call Lab for #)
☐ 1 day ☐ 2 days ☐ 3 days

DATE Required:
TIME Required:

Please note that TAT for certain tests such as BOD and Dioxins/Furans are > 5 days - contact your Project Manager for details.

	Sample Identification	Date Sampled	Time Sampled	Matrix (GW, SW, Soil, etc.)	Regulated Drinking Water? (Y / N)	Metals Field Filtered? (Y / N)	Corrosivity	# of Cont.	COMMENTS / TAT COMMENTS
1	CRB_5_13.31-13.41	Feb. 14/18	3:00	rock			X	1	
2	CRB_3A_11.10-11.28	Feb. 9/18	2:00	rock			X	1	
3	CRB_2_6.10-6.24	Feb. 6/18	12:00	rock			X	1	
4									
5									
6									
7									
8									
9									
10									
11									
12									

17-Apr-18 14:35
Ema Gitej
B886277
URE ENV-2012

RELINQUISHED BY (Signature/Print)	RECEIVED BY (Signature/Print)	Date	Time	# JARS USED AND NOT SUBMITTED	Laboratory Use Only Temperature (°C) on Receipt
Andrea Begin	Parneet K. Purewal	2018/04/17	14:35		8/10/8

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

COC-1004 (03/10) - ENV. ENG. Maxxam Analytics International Corporation o/a Maxxam Analytics White: Maxxam Yellow: Mail Pink: Client

APPENDIX D

Non-Standard Special Provisions, Special Provisions and Notice to Contractor

<u>DEWATERING SYSTEM</u> - Item No.
<u>TEMPORARY FLOW PASSAGE SYSTEM</u> - Item No.

Special Provision No. 517F01 July 2017

Amendment to OPSS 517, November 2016

Design Storm Return Period and Preconstruction Survey Distance

517.01 SCOPE

Section 517.01 of OPSS 517 is deleted in its entirety and replaced with the following:

This specification covers the requirements for the design, operation, and removal of a dewatering or temporary flow passage system or both to control water during construction, and the control of the water prior to discharge to the natural environment and sewer systems.

517.04 DESIGN AND SUBMISSION REQUIREMENTS

517.04.01 Design Requirements

Subsection 517.04.01 of OPSS 517 is amended by deleting the first paragraph in its entirety and replacing it with the following:

A dewatering or temporary flow passage system or both shall be designed to control water at the locations specified in the Contract Documents and at any other location where a system is necessary to complete the work. The design of the system shall be sufficient to permit the work at each location to be carried out as specified in the Contract Documents.

Subsection 517.04.01 of OPSS 517 is further amended by deleting the second last paragraph in its entirety and replacing it with the following:

Temporary flow passage systems shall be designed, as a minimum, for a 2 year design storm return period and groundwater discharge, except for the work specified in Table A. For the work specified in Table A, the temporary flow passage system shall be designed, as a minimum, for the design storm return period specified in Table A and groundwater discharge. A longer return period shall be used when determined appropriate for the work.

Intensity-Duration Factor (IDF) curve location, site specific minimum return period, return period flow estimates, and other information is provided in Table A. The IDF information can be accessed through the MTO IDF Curve Look up Tool on the Drainage and Hydrology page of MTO's website. The return period flow estimates do not include flow volumes from groundwater discharge. The Owner specifically excludes these flow estimates from the warranty in the Reliance on Contract Documents subsection of OPSS 100, MTO General Conditions of Contract.

Table A

IDF Curve Location	Latitude: 43.554167				Longitude: -79.612500	
Temporary Flow Passage Systems						
Site Name / Station Reference	Minimum Return Period (Years)	Return Period Flow Estimates (m^3/s)				Design Engineer Requirements (Note 1)
		2 Year	5 Year	10 Year	25 Year	
Credit River Bridge	2	120.0	223.0	291.0	369.0	Yes
Stavebank Creek	2	0.7	1.1	1.6	2.0	No
Kenollie Creek	2	3.1	4.7	5.4	10.0	No
Dewatering Systems						
Site Name / Station Reference	Preconstruction Survey Distance (Note 2) (m)					Design Engineer Requirements (Note 1)
Credit River Bridge	50					Yes

Note:

1. "Yes" means the design Engineer and design-checking Engineer shall have a minimum of 5 years of experience in designing systems of similar nature and scope to the required work. "No" means a minimum experience level is not required for the design Engineer and design-checking Engineer.
2. "N/A" indicates a preconstruction survey is not required.

Table A (Sample)

<table>
<tr><td>IDF Curve Location</td><td colspan="3">Latitude: 44.974844</td><td colspan="3">Longitude: -79.769339</td></tr>
<tr><td colspan="7">Temporary Flow Passage Systems</td></tr>
<tr><td rowspan="2">Site Name / Station Reference</td><td rowspan="2">Minimum Return Period (Years)</td><td colspan="4">Return Period Flow Estimates (m^3/s)</td><td rowspan="2">Design Engineer Requirements (Note 1)</td></tr>
<tr><td>2 Year</td><td>5 Year</td><td>10 Year</td><td>25 Year</td></tr>
<tr><td>Woods Creek Culvert Rehabilitation</td><td>2</td><td>0.7</td><td>3.5</td><td>7.5</td><td>10.9</td><td>N/A</td></tr>
<tr><td>Site 32-145
Robbs Creek Culvert Replacement</td><td>10</td><td>1.6</td><td>7.6</td><td>17.4</td><td>25.2</td><td>Yes</td></tr>
<tr><td colspan="7">Dewatering Systems</td></tr>
<tr><td>Site Name / Station Reference</td><td colspan="5">Preconstruction Survey Distance (Note 2) (m)</td><td>Design Engineer Requirements (Note 1)</td></tr>
<tr><td>Site 32-145
Robbs Creek Culvert Replacement</td><td colspan="5">300</td><td>Yes</td></tr>
<tr><td colspan="7">Note:
1. “Yes” means the design Engineer and design-checking Engineer shall have a minimum of 5 years of experience in designing systems of similar nature and scope to the required work. “No” means a minimum experience level is not required for the design Engineer and design-checking Engineer.
2. “N/A” indicates a preconstruction survey is not required.</td></tr>
</table>

WARRANT: Always with these tender items.

AMENDMENT TO OPSS 539, NOVEMBER 2014

Special Provision No. 105S09 March 2018

539.03 DEFINITIONS

Section 539.03 of OPSS 539 is amended by the deletion of the definitions for **Certificate of Conformance** and **Quality Verification Engineer**.

539.04 DESIGN AND SUBMISSION REQUIREMENTS

539.04.02.05 Milestone Inspections

Clause 539.04.02.05 of OPSS 539 is deleted in its entirety.

539.07 CONSTRUCTION

539.07.03.03.02 Excavation Depths Less Than or Equal to Three Metres

Clause 539.07.03.03.02 of OPSS 539 is amended with the following:

The Contractor's Engineer shall inspect the following Work:

a) Installation of the protection system, including excavation to dredge line.

b) Removal of the protection system.

539.07.03.03.03 Excavation Depths Exceeding Three Metres

Clause 539.07.03.03.03 of OPSS 539 is amended with the following:

The Contractor's Engineer shall inspect the following Work:

a) Layout and extent of protection system.

b) Piling.

c) Installation of protection system, including excavation to dredge line.

d) Removal of protection system.

539.07.03.04 Inspection of Protection Systems

539.07.03.04.01 Excavation Depths Less Than or Equal to Three Metres

Clause 539.07.03.04.01 of OPSS 539 is deleted in its entirety and replaced with the following:

For protection systems to facilitate excavation depths less than or equal to 3 m and provided that surcharge loading due to vehicular traffic, construction equipment and materials, or other is beyond a horizontal distance defined by a 1H : 2V line projected from the dredge line at the face of the protection system to the roadway surface, the Contractor’s Engineer shall inspect and verify that the that the protection system was installed, monitored, and subsequently removed according to the Contract Documents.

A Certificate of Conformance shall be submitted to the Contractor Administrator upon completion of the installation of the protection system.

A Certificate of Conformance shall be submitted to the Contractor Administrator upon completion of the removal of the protection system.

Should the traffic be within a horizontal distance defined by a 1H: 2V line projected from the dredge line at the face of the protection system to the roadway surface, the Certificate of Conformance requirements as specified in the Excavation Depths Exceeding Three Metres clause shall apply.

539.07.03.04.02 Excavation Depths Exceeding Three Metres

Clause 539.07.03.04.02 of OPSS 539 is deleted in its entirety and replaced with the following:

For protection systems to facilitate excavation depths that exceed 3 m or should traffic, construction equipment and materials, or other be within a horizontal distance defined by a 1H:1V line projected from the dredge line at the face of the protection system to the roadway surface.

The Contractor’s Engineer shall inspect and verify that the materials have been supplied and installed according to the Contract Documents. A Certificate of Conformance shall be submitted to the Contract Administrator upon completion of the installation of the materials.

The Contractor’s Engineer shall inspect and verify and that the protection system was installed, monitored, and subsequently removed according to the Contract Documents. A Certificate of Conformance shall be submitted to the Contract Administrator upon completion of the removal of the protection system.

AMENDMENT TO OPSS 903, APRIL 2016

Special Provision No. 109F57 April 2018

903.03 DEFINITIONS

Section 903.03 of OPSS 903 is amended by the deletion of the definitions for Certificate of Conformance and Quality Verification Engineer.

903.04 DESIGN AND SUBMISSION REQUIREMENTS

903.04.02.04.02.01 Milestone Inspections

Clause 903.04.02.04.02.01 of OPSS 903 is deleted in its entirety.

903.04.02.06 Review of Splice Test Results and Permission to Proceed

Clause 903.04.02.06 of OPSS 903 is deleted in its entirety.

903.07 CONSTRUCTION

903.07.02.07.01 General

Clause 903.07.02.07.01 of OPSS 903 is amended by deleting the first paragraph in its entirety and replacing it with the following:

The driving of piles shall be carefully monitored and controlled and pile driving records produced for each pile under the direction of the Contractor. A pile driving record shall be submitted to the Contract Administrator.

903.07.02.07.03 Driving to a Specified Ultimate Resistance

903.07.02.07.03.01 General

Clause 903.07.02.07.03.01 of OPSS 903 is deleted in its entirety and replaced with the following:

When piles are specified to be driven to a specified ultimate resistance, the specified ultimate resistance shall be determined using the [N/A] at end of initial driving as specified in the Contract Documents. If the specified ultimate resistance is not achieved, retap/restrike shall be conducted after initial driving as specified in the Contract Documents.

A Request to Proceed shall be submitted to the Contract Administrator after the design ultimate resistance is achieved.

The next operation shall not proceed until a Notice to Proceed has been received from the Contract Administrator.

903.07.02.07.03.03 Driving to Bedrock

Clause 903.07.02.07.03.03 of OPSS 903 is amended by deleting the last sentence in its entirety.

903.07.02.07.04 Wave Equation Analysis

Clause 903.07.02.07.04 of OPSS 903 is deleted in its entirety and replaced with the following:

When requested by the Contract Administrator, all equipment, material, and personnel shall be supplied to conduct the wave equation analysis procedure.

903.07.03.07 Concrete

903.07.03.07.01 General

Clause 903.07.03.07.01 of OPSS 903 is deleted in its entirety and replaced with the following:

A Request to Proceed shall be submitted to the Contract Administrator before the concrete placement.

The reinforcement shall not be displaced or distorted during the construction of the caisson.

The placement of concrete shall not proceed until the Contract Administrator has inspected the caisson hole and issued to the Contractor a Notice to Proceed.

Concrete shall be placed immediately after the Notice to Proceed has been received and shall be placed in the caisson according to OPSS 904 and as specified herein.

Arching of concrete during casing withdrawal shall be prevented.

903.07.03.07.05 Founding Elevation

Clause 903.07.03.07.05 of OPSS 903 is amended by deleting the last paragraph in its entirety and replacing it with the following:

Complete access to inspect the bearing area of the caisson pile prior to the placement of concrete shall be given to the Contract Administrator.

903.07.06 Load Test

Subsection 903.07.06 of OPSS 903 is amended by deleting the first paragraph in its entirety and replacing it with the following:

When a load test is specified in the Contract Documents, the testing shall be according to ASTM D 1143M for piles under vertical static load, ASTM D 3689 for piles under tensile load, and ASTM D 3966 for piles under lateral loads. The Contract Administrator shall witness the pile load test. All records and results of the pile load test shall be submitted to the Contract Administrator.

903.07.08.01.02 Visual Inspection of Welds

Clause 903.07.08.01.02 of OPSS 903 is deleted in its entirety and replaced with the following:

Complete access to visually inspect the welds shall be given to the Contract Administrator.

A representative sample of not less than 30% of the welds, as determined by the Contract Administrator, shall be visually inspected for conformance to the requirements of CSA W59 and the Contract Documents.

903.07.08.01.03 Non-Destructive Testing of Welds

Clause 903.07.08.01.03 of OPSS 903 is deleted in its entirety and replaced with the following:

Radiographic or ultrasonic testing shall be carried out using procedures according to CSA W59.

Ultrasonic or radiographic testing shall be carried out on the entire length of selected splice welds chosen at random by the Contractor’s welding inspector assigned to carry out visual inspections.

Selection shall be based on the following criteria:

a) For pile groups other than at integral abutments, 10% of the splice welds, rounded to the next highest number, but no fewer than two.

b) For pile groups at integral abutments, 10% of the splice welds, rounded to the next highest number, but no fewer than two of when the welds are below 6 m of the pile cut-off elevation.

c) For pile groups at integral abutments, all splice welds within 6 m of the pile cut-off elevation.

903.07.08.03 Certificate of Conformance

Clause 903.07.08.03 of OPSS 903 is deleted in its entirety.

903.10 BASIS FOR PAYMENT

903.10.01 Supply Equipment for Installing Driven Piles - Item
Supply Equipment for Installing Caisson Piles - Item
Supply Equipment for Installing Displacement Caisson Piles - Item

Subsection 903.10.01 of OPSS 903 is amended by deleting the second paragraph in its entirety and replacing it with the following:

For payment purposes, 50% of the work under this item shall be paid when the satisfactory performance of the equipment has been demonstrated to the Contract Administrator by the installation of 1% of piles.

Another 40% shall be paid by progress payments proportional to the work completed. The remaining 10% shall be paid on the satisfactory completion of the installation of piles.

AMENDMENT TO OPSS 902, NOVEMBER 2010

Special Provision No. 109S12 March 2018

902.03 DEFINITIONS

Section 902.03 of OPSS 902 is amended by the deletion of the definitions for **Certificate of Conformance** and **Quality Verification Engineer**.

Section 902.03 is further amended by the addition of the following definition:

Dewatering System means as defined in OPSS 517.

902.04 DESIGN AND SUBMISSION REQUIREMENTS

902.04.02.02 Milestone Inspections

Clause 902.04.02.02 of OPSS 902 is deleted in its entirety.

902.07 CONSTRUCTION

902.07.04 Dewatering Structure Excavation

Subsection 902.07.04 of OPSS 902 is deleted in its entirety and replaced with the following:

902.07.04 Dewatering Systems

902.07.04.01 General

Until backfilling has been completed and to permit the placing of concrete in the dry, all work necessary to control the flow of water into the excavation and to prevent disturbance of the founding material shall be carried out.

902.07.06 Backfilling

902.07.06.02 Compaction

Clause 902.07.06.02 of OPSS 902 shall be deleted in its entirety and replaced by the following:

Backfill shall be placed according to OPSS 206, except the Modified Layer Compaction Method shall not apply, and compacted according to OPSS 501.

Only hand operated vibratory type compaction equipment shall be used for compaction of fill material within the restricted zone behind all earth retaining structures.

902.07.08 Certificate of Conformance

Subsection 902.07.08 of OPSS 905 is deleted in its entirety and replaced with the following:

902.07.08 Inspection for Dewatering, Excavation and Backfilling

A Request to Proceed shall be submitted to the Contract Administrator prior to the commencement of dewatering of the excavation for the structure and completion of the excavation for the foundation.

The next operation after the completion of the excavation for the foundation shall not proceed until a Notice to Proceed has been received from the Contract Administrator.

A Request to Proceed shall be submitted to the Contract Administrator upon completion of the excavation for backfill and frost tapers and prior to the commencement of backfilling of excavation.

The next operation prior to the commencement of backfilling of excavation shall not proceed until a Notice to Proceed has been received from the Contract Administrator.

<u>DEWATERING STRUCTURE EXCAVATIONS</u> – Item No.

Non-Standard Special Provision

1.0 SCOPE

As part of the work under this item, the Contractor shall design, supply, and install cofferdams to construct the foundations for the new piers at the QEW-Credit River bridge.

All work as shown on the Contract Drawings.

2.0 REFERENCES – Not Used

3.0 DEFINITIONS

Stamped means drawings or details that have been reviewed and stamped "Conforms With Contract Documents". The stamp shall include the date and signature of the Contractor's Engineer.

Contractor's Engineer means an Engineer licensed to practice in the Province of Ontario who has a minimum of five (5) years of experience in the field of design and/or construction of cofferdams. The Contractor shall retain the Contractor's Engineer to ensure conformance with the contract document.

Cofferdam Design Engineer means an Engineer licensed to practice in the Province of Ontario who has a minimum of five (5) years of experience in the field of design and/or construction of bridges. In addition, the Cofferdam Design Engineer shall have had responsible experience in the design of at least 5 other cofferdams. The Contractor shall retain the Cofferdam Design Engineer to ensure conformance with the contract documents and issue certificate(s) of conformance for the design

4.0 DESIGN AND SUBMISSION REQUIREMENTS

The design of concrete cofferdams shall be in accordance with the Canadian Highway Bridge Design Code CAN/CSA-S6-14.

Submission of Shop Drawings

All shop drawings submissions shall bear the seal and signature of the Cofferdam Design Engineer.

The Contractor shall submit to the Contractor's Engineer shop drawings for review and stamping.

At least two weeks prior to the commencement of cofferdam construction, the Contractor shall submit to the Contract Administrator, for information purposes only, four (4) sets of stamped drawings and calculations of the cofferdam system.

The Contractor shall, at least three (3) weeks prior to the commencement of the cofferdam installation, submit to the Contractor's Engineer for review, four (4) sets of drawings and calculations indicating:

- the cofferdam design;
- the location, type and dimensions of each cofferdam to be used;
- a schematic showing the configuration of all cofferdams;

- the thickness of the tremie plug to ensure stability of the design excavation and cofferdam and the pour sequence of the tremie concrete for which the cofferdam was designed to accommodate unbalanced loading from staged placement and variable heights of the tremie concrete.

The Contractor’s Engineer shall review all calculations, construction details, shop drawings and procedures.

All submissions shall bear the seal and signature of the Cofferdam Design Engineer and Contractor’s Engineer.

5.0 MATERIALS – Not Used

6.0 EQUIPMENT – Not Used

7.0 CONSTRUCTION

The soils at the site are glacially- and fluvially-derived and should be expected to contain cobbles and boulders. In addition, obstructions may be present at or near the surface (i.e. existing rip-rap) and/or within the existing fill materials at the site. Appropriate equipment and procedures will be required to penetrate these obstructions to allow for installation of the cofferdam and for construction of the pile caps in dry conditions. The shale bedrock is found at relatively shallow depth below ground surface at the pier locations, and the surface of the bedrock will vary and undulate across each foundation element. The shale bedrock is weak, but contains strong to very strong limestone layers, although the upper portion of the bedrock does exhibit some degree of weathering; appropriate equipment and procedures will be required to seat the cofferdams onto or into the surface of the shale bedrock and mitigate groundwater seepage at the soil-bedrock interface.

Footing/pile cap construction below the groundwater and/or Credit River water levels must be carried out in dry conditions. The excavation shall be kept stable during the work.

The Contractor shall cut the cofferdam at the limits indicated on the Contract Drawings at the completion of the construction of the footings.

8.0 QUALITY ASSURANCE

Certificates of Conformance

The Cofferdam Design Engineer shall inspect the installation of each cofferdam prior to the placing of the tremie concrete in that cofferdam. After the installation of each of the cofferdam has been completed, but before placing the tremie concrete, the Contractor shall submit a Certificate of Conformance for each cofferdam to the Contract Administrator, sealed and signed by the Cofferdam Design Engineer. The Certificates of Conformance shall state that the cofferdam is in place, and has been installed in conformance with the stamped shop drawings and the Contract Drawings.

9.0 MEASUREMENT FOR PAYMENT

Measurement for cofferdams shall be by length in metres of cofferdam installed.

10.0 BASIS OF PAYMENT

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

DOWELS INTO ROCK - Item No.

Non-Standard Special Provision

Scope of Work

Where required at the abutments for the QEW Credit River Bridge, the Contractor shall provide dowels into the bedrock at the foundations for the spread footings founded on bedrock.

Construction

Concrete shall be of the same strength as the footing concrete and placed in accordance with OPSS 904. All reinforcing steel supplied shall be in accordance with OPSS.PROV 1440 (dowel bars conforming to CAN/CSA G30.18, Grade 400).

Where dowels are to be placed in rock, hole shall be drilled to the required depth and size. Hole diameter shall be two times the nominal diameter of the dowel. Each hole shall be cleaned out, grouted and the dowel set in place. Grout shall be of the same strength as the footing concrete or at least 25 MPa at 28 days.

If the dowel drill hole contains water, the Contractor shall remove the water, otherwise a tremie procedure shall be used to completely fill the hole with grout. The dowel shall be forced into the hole after the grout has been placed and while it is still fresh.

Rock Dowel Testing

All proposed testing procedures shall be in general conformance with ASTM D3689-07, ASTM D1143-07 and ASTM D4435-08. Field testing must be carried out in the presence of, and the results reviewed and approved by, the Contract Administrator.

Performance Tests

Performance testing shall be carried out at two dowels at each foundation element to confirm that the design load of the rock dowels can be achieved. The Contract Administrator will select the rock dowels to be tested.

Performance test shall be by axial tensioning using a hydraulic jack with a capacity of at least 1.5 times the ultimate strength of the dowels.

Rock dowels shall be loaded and unloaded in 3 cycles and measurements of the displacement of the dowel shall be carried out at each load increment (step) in accordance with the following schedule:

Cycle-Step	1-1	1-2	1-3	2-1	2-2	2-3	2-4
% Design Load	50	75	25	50	75	100	25

Cycle-Step	3-1	3-2	3-3	3-4	3-5
% Design Load	50	75	100	110	25

The design load shall be taken as 360 kN for 35M dowels, 252 kN for 30M dowels, 180 kN, for 25M dowels, and 108 kN for 20M dowels.

Displacement measurements shall be carried out at each load increment using calibrated displacement gauges capable of measuring movements of 0.0025 cm. Measurements shall be referenced to an independent fixed referenced pint.

Rock dowels which fail to meet the acceptance criteria shall be replaced at the Contractor’s expense and re-tested. If a rock dowel fails, three (3) additional rock dowels shall be tested at the same abutment and pier footing as directed by the Contract Administrator.

Acceptance criteria for the rock dowels will be in accordance with the Post-Tensioning Institute (1985) as follows:

- The dowels are acceptable if the total elastic movement is greater than 80 percent of the theoretical elastic elongation of the free stressing and is less than the theoretical elongation of the free stressing length plus 50 percent of the bond length.

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.

CAISSON PILES – Item No.

Non-Standard Special Provision

Amendment to OPSS.PROV 903, April 2016

903.01 SCOPE

Section 903.01 of OPSS.PROV 903 is deleted in its entirety and replaced with the following:

This specification covers the requirements for the supply and installation of caisson piles (caisson foundations) for the west pier and east pier foundations of the QEW-Credit River Bridge WB.

903.07 CONSTRUCTION

Section 903.07.03.02.01 General

Section 903.07.03.02.01 of OPSS.PROV 903 is amended by the addition of the following:

Caisson foundations for support of the QEW-Credit River Bridge WB piers will extend through the overburden, below the groundwater/river water level and into the shale bedrock, which is weak and contains clay seams and strong to very strong limestone interlayers at varying depths/elevations. Appropriate equipment and construction procedures will be required to penetrate the overburden and advance sockets into the bedrock to reach the design founding level, including the use of permanent liners to provide support to the overburden soils and the upper zone of weathered bedrock.

The caisson foundations, including the rock sockets, at the Credit River Bridge pier foundation elements must be constructed in the wet at all times and the rock sockets must be maintained in a wet condition including during placement of reinforcing steel and placement of concrete using tremie methods.

The liners used to support the ground during construction must be advanced sufficiently into bedrock to prevent the overburden and weathered shale bedrock from falling into the caisson/rock socket.

Construction of a rock socket or installation and seating of a steel liner shall not be permitted within 5 m of any caisson into which concrete has been placed within the preceding 24 hours.

Section 903.07.03.03 Inspection of the Excavation

Section 903.07.03.03 of OPSS.PROV 903 is amended by the addition of the following:

Within 24 hours following the completion of the excavation for each caisson (including excavation of the rock socket), the walls and base of each rock socket shall be thoroughly cleaned and inspected immediately thereafter. Cleaning shall be by airlift or other suitable means such that the water issuing from the caisson on flushing or pumping is clean and free of soil, rock cuttings and any other material. Every reasonable step shall be taken to remove all soil and rock cuttings and other materials from the caisson and from the walls and base of the rock sockets.

Prior to placing reinforcement steel and concrete, the Contractor shall provide the Contract Administrator with conclusive evidence that the walls and base of the rock sockets are clean and free of debris, by a method satisfactory to the Foundation Engineer retained by the Contract Administrator.

Section 903.07.03.07.03 Concrete Placed Under Water or Under Slurry

Section 903.07.03.07.03 of OPSS.PROV 903 is amended by the addition of the following:

Each caisson shall be completed (including concrete placement) within seven days from when drilling of the rock socket first commenced.

Section 903.07.03.07.04 Withdrawal of Liners

Section 903.07.03.07.04 of OPSS.PROV 903 is deleted in its entirety and replaced with the following:

The liners at the piers for the QEW Credit River Bridge WB shall not be removed.

ROCK EXCAVATION FOR STRUCTURE – Item No.

Non-Standard Special Provision

Amendment to OPSS 902, November 2010

902.07 Construction

902.07.05.02 Excavation for Foundations

Section 902.07.05.02 of OPSS 902 is amended by the deletion of the fifth paragraph and the addition of the following:

Excavations for foundations for the new QEW Credit River Bridge WB (Site No. 24X-0203/B2), the QEW Mississauga Road Overpass (Site No. 24X-0196/B0) and the QEW Credit River E-W Pedestrian bridge (Site No. 24X-0204/B0) will extend into the shale bedrock, which is very weak to weak, contains clay seams and strong to very strong limestone interlayers at varying depths/elevations. Appropriate equipment and construction procedures will be required to penetrate the overburden and excavate the bedrock to reach the design founding level.

If any bedrock excavation is required adjacent to and/or below an existing foundation that is supporting the existing QEW Credit River bridge or QEW Mississauga Road overpass, it shall be carried out using sawcutting or line drilling techniques. During construction, if removal of bedrock extends to 0.3 m or deeper below the base of the adjacent existing foundation supporting the existing QEW Credit River Bridge or QEW Mississauga Road overpass, then temporary roadway protection systems will be required to support the existing foundation. Any over-excavation of the bedrock below the design foundation level must be replaced immediately with mass concrete, having a minimum 28-day compressive strength of 20 MPa.

If bedrock is encountered above the design elevation of the underside of the granular levelling pad supporting the RSS wall footing or above the design elevation of the base of the reinforced soil zone as shown on the Contract Documents, the bedrock must be subexcavated and replaced with OPSS.PROV 1010 Granular ‘A’ and placed in accordance with OPSS.PROV 501.

DEWATERING STRUCTURE EXCAVATIONS - Item No.

Special Provision No. FOUN0003 March 8, 2018

Amendment to OPSS 902, November 2010

OPSS 902, November 2010, Construction Specification for Excavating and Backfilling - Structures is amended as follows:

902.02 REFERENCES

Section 902.02 of OPSS 902 is amended by the addition of the following:

Ontario Provincial Standard Specifications, Construction

OPSS 517	Dewatering
OPSS 805	Temporary Erosion and Sediment Control Measures

902.03 DEFINITIONS

Section 903.03 of OPSS 902 is amended by the addition of the following:

Automatic Transfer Switch means as defined in OPSS 517.

Cofferdam means as defined in OPSS 539.

Cut-Off Wall means as defined in OPSS 517.

Design Storm Return Period means as defined in OPSS 517.

Dewatering System means as defined in OPSS 517.

Groundwater Control System means as defined in OPSS 517.

Plug means as defined in OPSS 517.

Sediment means as defined in OPSS 517.

Sediment Control Measure means as defined in OPSS 517.

Temporary Flow Passage System means as defined in OPSS 517.

Unwatering means as defined in OPSS 517.

Vegetated Discharge Area means as defined in OPSS 517.

Waterbody means as defined in OPSS 517.

Watercourse means as defined in OPSS 517.

902.04 DESIGN AND SUBMISSION REQUIREMENTS

902.04.01 Design Requirements

902.04.01.01 Dewatering

Clause 902.04.01.01 of OPSS 902 is deleted in its entirety and replaced with the following:

A dewatering system shall be designed to control water and the flow of water into the excavation, prevent disturbance of the foundation, permit the placing of concrete in the dry, and complete the excavating and backfilling for structures work.

When the system includes temporary flow passage system, the system shall be designed, as a minimum, for a 2 year design storm return period, and groundwater discharge. A longer return period shall be used when determined appropriate for the work.

The dewatering system shall be according to the design requirements specified in OPSS 517.

902.04.02 Submission Requirements

Subsection 902.04.02 of OPSS 902 is deleted in its entirety and replaced with the following:

902.04.02.01 Working Drawings

Working Drawings for the dewatering system shall be according to OPSS 517.

902.04.02.02 Preconstruction Survey

When a groundwater control system by wells or a well point system will be used, a condition survey of property and structures that may be affected by the work shall be carried out. The condition survey shall include the location and condition of adjacent properties, buildings, underground structures, water wells, Utilities, and structures, within a distance of 50 metres from the groundwater control system. In addition, all water wells used as a supply of drinking water and located within this distance shall be tested for compliance with Ontario Drinking Water Quality Standards.

Water wells within the preconstruction survey distance can be located using the website https://www.ontario.ca/environment-and-energy/map-well-records or its successor site.

Copies of the condition survey and water quality test results shall be submitted to the Contract Administrator prior to the operation of the groundwater control system.

902.04.02.03 Milestone Inspections

Clause 902.04.02.03 of OPSS 902 is deleted in its entirety.

902.07 CONSTRUCTION

Subsection 902.07.04 of OPSS 902 is deleted in its entirety and replaced with the following:

902.07.04 Dewatering Structure Excavation

902.07.04.01 General

The dewatering systems shall be constructed and operated according to the Working Drawings.

Activation and deactivation of a temporary flow passage system, if applicable, shall be according to OPSS 517.

The dewatering system shall be continuously operational to control buoyancy forces until such forces can be resisted by backfill and structure self-weight, to keep excavations stable, to avoid erosion impacts from the release of accumulated water, and to keep the work area in the condition required to complete the associated work as specified in the Contract Documents.

When a temporary flow passage system is to remain operational through a seasonal shutdown period, the Contractor shall be responsible for any maintenance or repair costs due to the system during the seasonal shutdown period.

Temporary erosion and sediment control measures, including controlling the discharge of water, shall be according to OPSS 805. Measures not specified in OPSS 805 shall be according to the Working Drawings. Temporary erosion and sediment control measures and cover material to protect exposed soils, as required by the Working Drawings, shall be installed as soon as is practical.

Stranded fish shall be managed as specified in the Contract Documents.

Unwatering shall be carried out as necessary.

Water suspected of being contaminated as indicated by visual or olfactory observations shall be reported to the Contract Administrator.

Dewatering and temporary flow passage systems shall be discontinued in a manner that does not disturb any structure, pipeline, or flow channel. Operation of the dewatering system shall be shut down according to the procedures specified in the Working Drawings, where applicable.

902.07.04.02 Discharge of Water

The discharge of water shall be according to OPSS 517.

902.07.04.03 Monitoring

Monitoring shall be according to OPSS 517.

902.07.04.04 System Amendments

Amendments to stop any displacement, damage, soil loss or erosion due to the operation of the dewatering system shall be according to OPSS 517.

902.07.04.05 Removal

Removal of dewatering system and temporary flow passage system components shall be according to OPSS 517.

NOTES TO DESIGNER:

Designer Fill-Ins

* Fill in the design storm return period according to MTO Drainage Design Standard TW-1.

** Fill in the preconstruction survey distance as recommended by the foundation engineer.

WARRANT: Include with this standard tender item **only** on the recommendation of a foundation engineer.

CUSTODIAN: Tony Sangiuliano, MERO - Foundation Group.

TEMPORARY PROTECTION SYSTEM – Item No.

Non-Standard Special Provision

Amendment to OPSS 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 surface or as indicated on the Contract Drawings.

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.

VIBRATION MONITORING – Item No.

Special Provision

TABLE OF CONTENTS

1.0 SCOPE

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

2.0 REFERENCES

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

> FOUNDATION INVESTIGATION REPORT
> Queen Elizabeth Way (QEW) - Credit River Bridge, Structure Site No. 24-203, QEW Widening from West of Mississauga Road to West of Hurontario Street, Mississauga, Ministry of Transportation, Ontario, GWP 2002-13-00
>
> FOUNDATION INVESTIGATION REPORT
> Queen Elizabeth Way (QEW) – Mississauga Road Overpass Replacement, Structure Site No. 24-196, QEW Widening from West of Mississauga Road to West of Hurontario Street, Mississauga, Ministry of Transportation, Ontario, GWP 2002-13-00
>
> FOUNDATION INVESTIGATION REPORT

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

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

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

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

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

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

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

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

3.0 DEFINITIONS

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

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

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

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

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

4.0 DESIGN AND SUBMISSION REQUIREMENTS

4.1 Submission Requirements

The Contractor/Contractor's Engineer shall submit details of the vibration monitoring plan to the Contract Administrator for information purposes at least 2 weeks prior to any work related to strip footing, deep foundation, cofferdam and temporary protection system installation. The submittals shall satisfy the specifications and at a minimum contain the following specific information:

a) Equipment and methods used by the Contractor to perform the work that may cause undue vibration.
b) Qualifications of vibration monitoring specialist.
c) Details regarding proposed instrumentation.
d) Proposed location of instruments adjacent to the on the residences, structures, utilities, wells, or other potentially vibration-sensitive structures within a 250 m radius from the excavation and installation of spread/strip footings, cofferdams, deep foundations and temporary protection systems, as applicable.
e) Proposed frequency of readings.
f) Action plan to be taken to adjust excavation, deep foundation and protection system installation methods if readings show vibrations exceeding tolerable levels.

6.0 EQUIPMENT

6.1 Vibration Monitoring Equipment

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

7.0 CONSTRUCTION

7.1 Pre- and Post-Construction Condition Surveys

A Pre-Construction Condition Survey and Post-Construction Condition Survey shall be prepared for all buildings, utilities, structures, water wells, and facilities within 250 m of excavation and installation of spread/strip footings, cofferdams, deep foundations and temporary protection systems.

7.1.1 Pre-Construction Condition Surveys

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

The Pre-Construction Condition Survey, at each structure/well within a 250 m radius of excavation and installation of spread/strip footings, cofferdams, deep foundations and temporary protection systems, shall be completed a minimum of two (2) weeks prior to commencement of excavation and installation of shallow and deep foundations and temporary protection systems. Only one Pre-Construction Condition Survey per structure or facility is required to be carried out in advance of excavation and installation of spread/strip footings, cofferdams, deep foundations or temporary protection system installation, unless more than six (6) months will elapse between these operations, in which case an interim inspection will be required.

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

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

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

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

7.1.2 Post-Construction Condition Surveys

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

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

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

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

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

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

7.2 Monitoring

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

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

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

7.3 Records

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

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

10.0 BASIS OF PAYMENT

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

WORKING SLAB - Item No.

Special Provision

1.0 Scope

This Special Provision covers the requirements for the supply and placement of a concrete working slab under foundations where necessary for the QEW - Credit River bridge, Mississauga overpass, North-South Active Transport bridge, East-West Active Transport bridge and the culverts.

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 thickness of 100 mm and a minimum of 28 day compressive 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 bedrock shall be exposed by removing all fill, existing concrete and native soil and then 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. Any over-excavated portions of the bedrock must be replaced with dental concrete, having the same composition and compressive strength as the concrete used for the foundation construction. If the concrete for the footings cannot be poured within four hours after excavation and inspection, a concrete working slab must be placed in the excavation immediately to protect the integrity of the subgrade.

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

NOTICE TO CONTRACTOR – Subsurface Obstructions

Special Provision

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

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

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

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

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

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