Foundation Investigation and Design Report

Stavebank Creek Culvert and Kenollie Creek Culvert Replacements, QEW Widening from West of Mississauga Road to West of Hurontario Street, Mississauga

Ministry of Transportation, Ontario, GWP 2002-13-0

Submitted to:

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GEOCRES NO.: 30M12-441
Stavebank Creek Culvert
Latitude: 43.558913 **Longitude**: -79.608430
Kenollie Creek Culvert
Latitude: 43.562222 **Longitude**:-79.60604

Distribution List

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

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PART A

FOUNDATION INVESTIGATION REPORT
STAVEBANK CREEK CULVERT AND KENOLLIE CREEK CULVERT REPLACEMENTS
QEW WIDENING FROM WEST OF MISSISSAUGA ROAD TO WEST OF HURONTARIO STREET, CITY OF MISSISSAUGA
MTO, G.W.P. 2002-13-00

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by Morrison Hershfield Limited (MH) on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services for two culvert replacements and temporary protection systems at Stavebank Creek (approximately Station 17+100) and Kenollie Creek (approximately Station 17+500). This investigation is associated with the widening of the Queen Elizabeth Way (QEW) and interchanges improvements in the City of Mississauga, Ontario. The general areas of the site investigations are shown on the Key Plan on Drawings 1 and 2.

The purpose of this investigation is to establish the subsurface soil, bedrock and groundwater conditions at the existing and proposed culverts, by borehole drilling / bedrock coring and geotechnical / analytical laboratory testing on selected soil and rock samples.

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

2.0 SITE DESCRIPTION

The existing Stavebank Creek and Kenollie Creek Culverts are located approximately 200 m and 650 m east, respectively, of the Credit River in the City of Mississauga, Ontario. The QEW and Premium Way are oriented in a northeast-southwest direction which for the purpose of this report implied as west-east orientation, and the existing culverts are oriented in a northwest-southeast direction, from north of Premium Way to south of the QEW. The creeks' flow direction is essentially northwest to southeast. The QEW consists of three eastbound lanes (Toronto) and three westbound (Hamilton) lanes, while Premium Way consists of one lane in each direction.

The culverts cross under both the QEW and Premium Way, which parallels the northside of the QEW. The existing Stavebank Creek Culvert is approximately 109 m long and is comprised of two longitudinally separate 30 m long and 79 m long concrete boxes1.8 m wide by 1.2 m high. The existing Kenollie Creek Culvert is comprised of one longitudinally continuous concrete box section approximately 70 m long and is 3 m wide by 1.2 m high. Site Photographs 1 to 8 are appended to this report.

Both the existing Stavebank Creek Culvert and Kenollie Creek Culvert and highway embankment in the vicinity of the existing culverts appear to be performing appropriately, from a geotechnical perspective. No settlement or cracking of either culvert is apparent from the field reconnaissance completed as part of the investigation. The nearby embankment side slopes are heavily vegetated with grasses, low shrubs and small diameter trees, there is no apparent seepage on the face and adjacent platform between the QEW and the local streets or at the toes of the embankment and there are no signs of sloughing or erosion.

There are residential areas located north and south of the culvert sites, along the north side of the Premium Way north of the QEW, and along both sides of Pinetree Way south of the QEW. The existing ground surface along the Stavebank Creek Culvert at Station 17+100 varies from about Elevations 95 m to 87 m along its alignment; and at the Kenollie Creek Culvert at Station 17+500 the ground surface varies from about Elevations 95 m to 89 m along its alignment.

3.0 INVESTIGATION PROCEDURES

The field work for the foundation investigation was carried out in a number of separate periods / phases from September 13 to December 21, 2018, during which time a total of thirteen boreholes were advanced:

- Boreholes S1 to S7 were advanced near the location of the existing Stavebank Creek Culvert; and
- Boreholes K1 to K6 were advanced near the location of the existing Kenollie Creek Culvert.

These boreholes are supplemented with nine boreholes drilled between August 23, 2017 and August 9, 2018 for other immediately adjacent structures, such as noise barrier walls and the North-South AT Pedestrian bridge:

- Boreholes NW3-2, NW3-2A, NW3-3, PED-02, PED-03, PED-03A and PED-03B were advanced in the vicinity of Stavebank Creek Culvert; and,
- Boreholes NRW3-6 and NRW7-3 were advanced in the vicinity of Kenollie Creek Culvert.

The locations of the boreholes advanced at the Stavebank Creek Culvert site and the Kenollie Creek Culvert site are shown on Drawings 1 and 2, respectively.

Boreholes S1 and S7 were advanced using a Portable Tripod rig and a manual hammer drive system supplied and operated by Walker Drilling Ltd. of Utopia, Ontario. Boreholes S2 to S6 and K1 to K6 were advanced by a CME-55 track-mounted drill rig supplied and operated by Geo-Environmental Drilling Inc. of Halton Hills, a CME-55 truck-mounted drill rig supplied and operated by Tri-Phase Environmental Inc. of Mississauga, a CME-75 truck-mounted drill and a CME-55 track-mounted drill rig supplied and operated by Davis Drilling Ltd. of Milton, and a CME-55 truck-mounted drill rig supplied and operated by Aardvark Drilling Inc. of Guelph, Ontario. The supplemental boreholes at adjacent structures were advanced by a CME-55 truck mounted and a CME-850 truck-mounted drill rig supplied and operated by Aardvark Drilling Inc. of Guelph, Ontario.

The boreholes were advanced through the overburden using 203 mm outer diameter hollow stem augers, with the exception of Boreholes NW3-2A, and PED-03B, which were advanced using a 156 mm Tricone with drilling mud. 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 in accordance with the Standard Penetration Test (SPT) procedures outlined in ASTM D1586-08[1].

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 S3, PED-03A at the Stavebank Creek Culvert site and in Borehole K2 at the Kenollie Creek Culvert site to permit monitoring of the groundwater level at the 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 filter sand. The section of borehole below the standpipe piezometer was backfilled with bentonite to the underside of the sand pack level, and the remainder of the borehole above the sand pack was backfilled with bentonite to near the ground surface and topped with cold patch asphalt or sand and gravel to match the adjacent ground surface material. All boreholes were backfilled with bentonite upon completion in accordance with Ontario Regulation 903 Wells (as amended).

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

The field work was observed by members of Golder's engineering and technical staff, who located the boreholes, arranged for the clearance of underground services including both public and, where applicable, private locates, observed the drilling, sampling and in-situ testing operations, logged the boreholes, and examined the soil 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 geotechnical laboratory testing. All of the geotechnical laboratory tests were carried out in accordance with MTO and/or ASTM Standards, as appropriate. Classification testing (water content, Atterberg limits, grain size distribution and organic content) was carried out on selected soil samples. Unconfined compression (uniaxial) strength (UCS), Young's modulus, bulk density, was carried out on selected specimens of the bedrock core.

Six selected soil samples and one selected rock core sample were submitted, under chain-of-custody procedures, to Maxxam Analytics of Mississauga, Ontario (a Standards Council of Canada (SCC) accredited laboratory) for corrosivity testing. The soil samples and rock core samples were analyzed for a suite of parameters, including conductivity, resistivity, soluble chloride concentration, soluble sulphate concentration and pH.

The borehole locations and ground surface elevations were measured using a GPS unit (Trimble XH 3.5G), having an accuracy of 0.1 m in the vertical and horizontal directions, with the exception of Borehole S7 which had a vertical accuracy of 1.9 m and a horizontal accuracy of 0.7 m due to heavy tree cover in the area. The as-drilled locations of Boreholes S2, S4 to S6, PED-02, K3 to K6 and NRW7-3, were referenced to site features and then plotted on the borehole location drawing to obtain the coordinates of the locations; and the ground surface elevations were obtained by plotting the coordinates on the digital terrain model and interpreting the elevation. The locations provided on the borehole records and shown on Drawings 1 and 2 are positioned relative to MTM NAD 83 (Zone 10) coordinates system, and the ground surface elevations are referenced to Geodetic datum. The borehole locations, ground surface elevations and drilled depths are summarized below.

Borehole No.	Location (MTM NAD 83)		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing (m) (Latitude, °)	Easting (m) (Longitude, °)		
Stavebank Creek Culvert				
S1	4,824,356.1 (43.559090)	295,982.6 (-79.609144)	91.7	3.4
S2	4,824,357.2 (43.559092)	296,001.4 (-79.608907)	94.9	17.4
S3	4,824,337.3 (43.558912)	296,021.0 (-79.608665)	90.0	16.6
S4	4,824,336.4 (43.558913)	296,040.2 (-79.608430)	95.2	18.4
S5	4,824,341.1 (43.558955)	296,053.5 (-79.608265)	95.3	16.9
S6	4,824,318.8 (43.558755)	296,059.4 (-79.608192)	95.2	14.8

Borehole No.	Location (MTM NAD 83)		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing (m) (Latitude, °)	Easting (m) (Longitude, °)		
S7	4,824,321.2 (43.558768)	296,076.9 (-79.607973)	90.1	4.6
NW3-02	4,824,342.4 (43.558958)	295,994.3 (-79.608996)	95.3	12.3
NW3-02A	4,824344.2 (43.558975)	295,993.8 (-79.609002)	95.3	27.6
NW3-03	4,824,329.2 (43.558840)	296,002.3 (-79.608895)	90.6	8.1
PED-02	4,824,321.8 (43.558773)	296,032.3 (-79.608524)	95.2	16.7
PED-03	4,824,305.3 (43.558625)	296,063.0 (-79.608144)	93.7	13.8
PED-03A	4,824,308.4 (43.558653)	296,062.1 (-79.608155)	94.1	6.1
PED-03B	4,824,309.6 (43.558664)	296,062.8 (-79.608146)	94.1	17.8 (including 3.0 m of bedrock core)
Kenollie Creek Culvert				
K1	4,824,728.9 (43.562439)	296,200.2 (-79.606453)	90.1	5.1 (including 3.8 m of bedrock core)
K2	4,824,716.6 (43.562329)	296216.3 (-79.606253)	93.2	9.4 (including 3.1 m of bedrock core)
K3	4,824,703.7 (43.562222)	296,236.7 (-79.606004)	95.0	14.1 (including 3.4 m of bedrock core)
K4	4,824,692.4 (43.562120)	296,229.9 (-79.606087)	95.0	13.9 (including 3.0 m of bedrock core)
K5	4,824,683.3 (43.562038)	296,242.1 (-79.605937)	95.0	13.5
K5A	4,824,681.0 (43.562017)	296,241.1 (-79.605949)	95.0	16.5 (including 3.7 m of bedrock core)
K6	4,824,688.5 (43.562085)	296,254.9 (-79.605778)	94.9	15.0 (including 0.7 m of bedrock core)
NRW3-6	4,824,701.8 (43.562195)	296,220.4 (-79.606203)	92.9	11.4 (including 3.9 m of bedrock core)

Borehole No.	Location (MTM NAD 83)		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing (m) (Latitude, °)	Easting (m) (Longitude, °)		
NRW7-3	4,824,696.6 (43.562158)	296,259.1 (-79.605727)	94.9	12.3

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Region Geology

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

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

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

4.2 Subsurface Conditions

The detailed subsurface soil, bedrock and groundwater conditions as encountered in the boreholes advanced during the current investigation, the details of the piezometer installations and the summary results of the geotechnical laboratory testing are presented on the Records of Borehole and Drillhole sheets provided in Appendix A for Stavebank Creek Culvert and Appendix B for Kenollie Creek Culvert. Lists on abbreviations and symbols and lithological, geotechnical rock description terminology, field estimation of rock hardness and rock weathering classification are also included in Appendix A and B to assist in the interpretation of the borehole and drillhole records. Plots of the grain size distribution and Atterberg limits tests results are presented on Figures A-1 to A-10D, provided in Appendix A for Stavebank Creek Culvert and on Figures B-1A to B-8, provided in Appendix B for Kenollie Creek Culvert. The results of the in-situ field tests (i.e. SPT "N" values) as presented on the Record of Borehole Sheets and in sub-sections of Section 4.2 are uncorrected. Photographs of the bedrock core samples recovered from the boreholes at the Stavebank Creek Culvert site and the Kenollie Creek Culvert site are presented on Figures A-11 and A-12 and on Figures B-9 to B-15, included in Appendix A and B, respectively. The results of the laboratory tests carried out on selected bedrock core samples and the analytical laboratory test reports are contained in Appendices C and D, respectively for both Stavebank Creek Culverts and Kenollie Creek Culverts.

The stratigraphic boundaries shown on the borehole records are inferred from non-continuous sampling, observations of drilling progress and the results of Standard Penetration Tests. These boundaries, therefore,

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

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.

4.2.1 Stavebank Creek Culvert

In general, the subsurface conditions at the proposed culvert consist of a layer of topsoil, asphalt or concrete, underlain by fill material consisting of clayey silt, silt and sand, and sand and gravel. The fill is underlain by native clayey silt with sand to silt and sand, sand and gravel deposits, and a glacial till deposit consisting of clayey silt, some sand to with sand, some gravel to with gravel. Shale bedrock was encountered underlying the native soil deposits in one borehole.

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

4.2.1.1 Asphalt

A layer of asphalt was encountered in Boreholes S2, S5, S6, NW3-2 and PED-02 at ground surface and ranges in thickness from about 100 mm to 300 mm.

4.2.1.2 Concrete

A layer of concrete was encountered in Borehole S4 at ground surface and has a thickness of 450 mm.

4.2.1.3 Topsoil

Topsoil was encountered in Boreholes S3, S7 and PED-03 at ground surface and the thickness of the layer ranges from about 200 mm to 300 mm.

4.2.1.4 Fill

A 1.2 m to 6.0 m thick fill material, comprised predominately of non-cohesive soil, with cohesive soil in places, was encountered in all boreholes at the Stavebank Creek Culvert site, from ground surface or underlying the asphalt, concrete or topsoil surface layer. In boreholes advanced through Premium Way and the QEW, a thin layer of sand and gravel was encountered underlying the asphalt and/or concrete pavement. The non-cohesive fill generally consists of silt and sand to silty sand with the exception of Borehole S2 advanced through Premium Way where the fill is variable in composition and is interlayered, as described below. The depth and elevation of the top and bottom of the fill material and the corresponding thickness and soil type are summarized below.

Borehole No.	Top of Layer (below ground/pavement surface)		Bottom of Layer		Thickness (m)	Fill Soil Type
	Depth (m)	Elevation (m)	Depth (m)	Elevation (m)		
S1	0.0	91.7	0.6	91.1	0.6	Clayey Silt with Sand
	0.6	91.1	1.2	90.5	0.6	Silty Sand
S2	0.2	94.7	0.3	94.6	0.1	Sand and Gravel

Borehole No.	Top of Layer (below ground/pavement surface)		Bottom of Layer		Thickness (m)	Fill Soil Type
	Depth (m)	Elevation (m)	Depth (m)	Elevation (m)		
S2	0.3	94.6	0.7	94.2	0.4	Sandy Silt
	0.7	94.2	0.9	94.0	0.2	Gravelly Sand
	0.9	94.0	3.7	91.2	2.8	Gravelly Clayey Silt with Sand
	3.7	91.2	4.5	90.4	0.8	Sand
	4.5	90.4	5.6	89.3	1.1	Silty Sand
S3	0.2	89.8	2.7	87.3	2.5	Silt and Sand
S4	0.5	94.7	1.5	93.7	1.0	Sand and Gravel
	1.5	93.7	5.6	89.6	4.1	Silt and Sand
S5	0.3	95.0	5.6	89.7	5.3	Silt and Sand
S6	0.2	95.0	0.7	94.5	0.5	Sand and Gravel
	0.7	94.5	5.6	89.6	4.9	Silt and Sand
S7	0.3	89.8	2.3	87.8	2.0	Silt and Sand
NW3-2	0.2	95.1	2.6	92.7	2.4	Silty Sand
	2.6	92.7	3.7	91.6	1.1	Sandy Clayey Silt
	3.7	91.6	5.3	90.0	1.6	Sand and Gravel
	5.3	90.0	7.8	87.5	2.5	Silty Sand
NW3-3	0.0	90.6	1.5	89.2	1.5	Silty Sand
PED-02	0.1	95.1	3.7	91.5	3.6	Silty Sand to Sand
PED-03	0.2	93.5	6.2	87.6	6.0	Silt and Sand to Silty Sand

The SPT “N”-values measured within the layers of sand and gravel fill range from 8 blows to 34 blows per 0.3 m of penetration, indicating a loose to dense compactness condition. The SPT “N”-values measured within the sandy silt to sand fill layers range from 1 blow to 39 blows per 0.3 m of penetration, indicating a very loose to dense compactness condition. The SPT “N”-values measured within the clayey silt to gravelly clayey silt to clayey silt with sand fill range from 3 blows to 29 blows per 0.3 m of penetration, suggesting a soft to very stiff consistency.

A grain size distribution test was carried out on one sample of the sand and gravel fill and the result is shown on Figure A-1 in Appendix A. The water content measured on a sample of the sand and gravel fill is 10 per cent.

Grain size distribution tests were carried out on thirteen samples of the sandy silt to sand fill layers and the results are shown on Figures A-2A and A-2B in Appendix A. The sandy silt and sand fill contain some asphalt fragments, concrete chips, and in Borehole NW3-3 a hydrocarbon odour was noted at a depth of about 0.8 m below ground surface. In Boreholes S5 and S2 the augers were grinding within the non-cohesive fill material at depths of 0.3 m to 0.6 m and from 1.5 m to 2.7 m, respectively. The water content measured on samples of the silt and sand fill ranges between about 3 per cent and 38 per cent.

A grain size distribution test was carried out on one sample of the cohesive fill material and the result is shown on Figure A-3 in Appendix A. Atterberg limits tests were carried out on three samples of the cohesive fill layers and measured liquid limits ranging from about 19 per cent to 24 per cent, plastic limits ranging from about 14 per cent to 15 per cent and plasticity indices ranging from about 6 per cent to 10 per cent. The results of the Atterberg limits tests are plotted on the plasticity chart on Figure A-4 in Appendix A and indicate that the cohesive fill material consists of clayey silt of low plasticity. The water content measured on samples of the clayey silt with sand fill ranges between about 12 per cent and 16 per cent.

4.2.1.5 Sand and Gravel to Gravel

A deposit of sand and gravel to gravel, trace sand, trace clay was encountered in Borehole S1 underlying a silty sand layer (see Section 4.2.1.6 for discussion), and in Borehole PED-02 underlying the non-cohesive fill material. The surface of the deposit was encountered at depths of 2.7 m and 3.7 m below ground surface (Elevations 89.0 m and 91.5 m), respectively, and extends to a depth of 6.5 m below ground surface (Elevation 88.7 m) in Borehole PED-02. Borehole S1 terminated in the sand and gravel deposit at a depth of 3.4 m below ground surface (Elevation 88.3 m).

The SPT “N”-values measured within the sand and gravel deposit range from 5 blows to 35 blows per 0.3 m of penetration with an “N”-value of 100 blows per 0.05 m of penetration at the bottom of Borehole S1, indicating a generally loose to dense compactness condition and an inferred obstruction (cobble or boulder) at the bottom of Borehole S1.

Grain size distribution tests were carried out on two samples of the sand and gravel deposit and the results are shown on Figure A-5 in Appendix A. The water content measured on two samples of the sand and gravel deposit is about 13 per cent and 22 per cent.

4.2.1.6 Silt to Silt and Sand to Sand

Boreholes S1, S2, S4 to S6, and PED-02, penetrated a deposit consisting of silt to silt and sand to silty sand to sand, trace to some gravel was encountered underlying the fill material in all boreholes, except in Borehole PED-02 where it was encountered underlying the sand and gravel layer. The surface of the deposit was encountered at depths of between about 1.2 m and 6.5 m (between Elevations 90.5 m and 88.7 m) below ground surface, and ranges in thickness from about 0.5 m to 3.1 m.

A lower layer of this deposit (silt, silty sand, sand and gravel) was also encountered underlying the till deposit in Boreholes S2 and NW3-2A and interlayered within the till deposit in PED-03B. The surface of the deposit was encountered at depths of about 14.6 m, 20.3 m and 11.6 m below ground surface (at Elevations 80.3 m, 75.0 m and 82.5 m), respectively, and its thickness is about 1.1 m in the Borehole PED-03B. Borehole S2 terminated within the silty sand deposit at a depth of 17.4 m below ground surface (Elevation 77.5 m), after penetrating about

2.8 m into the deposit. Borehole NW3-2A terminated within the sand and gravel deposit at a depth of 27.6 m (Elevation 67.7 m), after penetrating about 7.3 m into the deposit.

The SPT “N”-values measured within the silt and sand to sand deposit range from 0 blows (weight of hammer) to 16 blows per 0.3 m of penetration, indicating that the deposit has a very loose to compact compactness condition. The SPT “N”-values measured within the non-cohesive interlayers within the till deposit (silt, silty sand) range from 35 blows to 54 blows per 0.3 m of penetration, indicating a dense to very dense compactness condition of the interlayers.

Grain size distribution tests were carried out on three samples of the silty sand layer of the deposit and the results are shown on Figure A-6 in Appendix A. Two organic content tests were completed on samples of the silt and sand to sand layer of the deposit from Borehole S5 and the results are 1.5 per cent and 2.2 per cent. The water content measured on samples of the silt to silt and sand deposit ranges between about 7 per cent and 28 per cent.

4.2.1.7 Clayey Silt with Sand

In Borehole S7 a deposit of clayey silt with sand, trace gravel was encountered underlying the silt and sand fill. The surface of the deposit was encountered at a depth of about 2.3 m below ground surface (Elevation 87.8 m) and the deposit is 0.4 m thick.

An SPT “N”-value measured within the clayey silt with sand deposit is 7 blows per 0.3 m of penetration, suggesting that the clayey silt with sand deposit has a firm consistency.

One grain size distribution test was carried out on the clayey silt with sand deposit and the result is shown on Figure A-7 in Appendix A. One Atterberg limits test was carried out on this same sample and measured a liquid limit of about 22 per cent, a plastic limit of about 16 per cent and a plasticity index was about 6 per cent. The result of the Atterberg limits test is plotted on the plasticity chart on Figure A-8 in Appendix A and indicates that the deposit consists clayey silt to silt of low plasticity. The water content measured on one sample of the clayey silt with sand deposit was about 24 per cent.

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

In all boreholes except S1 an interlayered till deposit consisting of clayey silt, some sand to sandy clayey silt to clayey silt with sand, some gravel to gravelly clayey silt to clayey silt with sand and gravel to sandy gravelly clayey silt, was encountered underlying the silt and sand deposit or underlying the silt and sand fill. Within the till deposit non-cohesive layers of silt and sand to gravelly silt and sand to gravelly sand, trace to some clay, were encountered. The surface of the till deposit was encountered at depths of between about 1.5 m and 8.7 m below ground surface (between Elevations 89.2 m and 86.6 m), and the thickness of the deposit ranges from about 1.6 m to 13.9 m.

Limestone fragments were encountered underlying the till at the bottom of Borehole S7 at a depth of 4.3 m below ground surface (Elevation 85.8 m), inferred to be a cobble/ boulder size slab. The till deposit in Borehole PED-03 was cored from a depth of about 10.7 m to 13.8 m below ground surface (Elevation 83.0 to 79.9 m) as a result of split-spoon refusal and encountered a 0.5 m thick limestone slab at a depth of 12.1 m below ground surface (Elevation 81.6 m), underlain by a 0.3 m thick zone of gravel and cobbles at a depth of 12.9 m below ground surface (Elevation 80.8 m). A photograph of the soil core is presented on Figure A-11in Appendix A.

The SPT “N”-values measured within the cohesive till deposit generally range from 4 blows to 99 blows per 0.3 m of penetration with “N”-values up to 131 blows per 0.08 m of penetration, suggesting a firm to hard consistency; and the SPT “N”-values measured within the non-cohesive till deposit generally range from 17 blows to 46 blows per 0.3 m of penetration, with “N”-values up to 100 blows for 0.08 m of penetration, indicating a compact to very dense compactness condition.

Grain size distribution tests were carried out on twenty-three samples of the till deposit and the results are shown on Figures A-9A to A-9E in Appendix A. Atterberg limits tests were carried out on twenty-two samples of the till deposit and measured liquid limits ranging from about 18 per cent to 30 per cent, plastic limits ranging from about 13 per cent to 17 per cent and plasticity indices ranging from about 5 per cent to 13 per cent for the cohesive till layers; and liquid limits ranging from about 16 per cent to 19 per cent, plastic limits ranging from about 13 per cent to 16 per cent and plasticity indices ranging from about 3 per cent to 4 per cent for the non-cohesive till layers. The results of the Atterberg limits test are plotted on the plasticity charts on Figures A-10A to A-10D in Appendix A and indicate the till deposit consists of clayey silt layers of low plasticity and silt to silty sand of slight plasticity. The water content measured on samples of the cohesive and non-cohesive portions of the till deposit ranges between about 6 per cent and 21 per cent.

4.2.1.9 Sandy Gravelly Clayey Silt (Residual Soil)

A 0.6 m thick layer of residual soil comprised of sandy gravelly clayey silt, containing some shale fragments was encountered underlying the cohesive till layer in Borehole PED-02 at a depth of about 16.1 m below ground surface (Elevation 79.1 m). Residual soil is a heterogeneous mix of fully weathered bedrock that is disintegrated into a soil like material that no longer retains the structure of parent bedrock.

The SPT “N”-value measured within the residual soil deposit at the bedrock contact is 100 blows per 0.13 m of penetration. The water content measured on a sample of the residual soil is 15 per cent.

4.2.1.10 Shale Bedrock

Shale bedrock was encountered in Boreholes PED-02 and was cored in PED-03B. In Borehole PED-02 shale was encountered at a depth of 16.7 m below ground surface (Elevation 78.5 m) and is inferred by a 0.1 m split-spoon sample. In Borehole PED-03B shale was encountered at a depth of 14.8 m below ground surface (Elevation 79.3 m), and 3 m of rock were cored (from 14.8 m to 17.8 m below ground surface).

Based on a review of the bedrock core samples the bedrock consists of shale of the Georgian Bay Formation. In general, the bedrock samples are described as slightly weathered to fresh, laminated to thinly bedded, fine grained, non-porous, very weak, grey shale, with slightly weathered to fresh, laminated, grey, fine grained, non-porous, medium strong limestone interbeds at varying intervals of depth.

The strong limestone layers range in thickness from about 10 mm to 400 mm, with an average thickness of about 20 mm. The stronger layers generally make up about 10 per cent by thickness of the rock encountered during the investigation. The details of the bedrock descriptions are presented on the Record of Drillhole PED-03B sheet and a photograph of the recovered bedrock core samples is presented on Figure A-12 in Appendix A. 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)[3] standard classification system.

The Rock Quality Designation (RQD) measured on the core samples obtained from Borehole PED-03B ranges from about 38 per cent to 78 per cent, indicating a rock mass of poor to good quality, as per Table 3.10 of CFEM (2006)[4]. The Total Core Recovery (TCR) and Solid Core Recovery (SCR) of samples recovered are 100 per cent and between 13 per cent and 43 per cent, respectively.

An Unconfined Compression (UC) test (ASTM D7012)[5] was carried out on a selected core sample of the shale bedrock and the uniaxial compressive strength (UCS), bulk density and tangent Young's modulus of the intact sample 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)
PED-03B	16.0 – 16.3	78.1 – 77.8	6.7	2.57	0.29

Based on the laboratory UCS, in accordance with Table 3.5 in CFEM (2006)[4], the shale bedrock of the core sample tested is classified as weak (R2, 5 MPa < UCS < 25 MPa).

4.2.1.11 Groundwater Conditions

The overburden samples obtained from the borehole investigations were generally moist to wet. The depth to the water level observed in the boreholes upon completion of drilling (and prior to rock coring (where applicable)) is presented below.

Borehole	Upon Completion of Drilling		Comment
	Water Level Depth (m)	Water Level Elevation (m)	
S1	0.7	91.0	Upon completion of drilling.
S2	Dry	--	
S3	15.9	74.1	
S4	6.1	89.1	
S5	7.0	88.3	
S6	11.9	83.3	

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

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

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

Borehole	Upon Completion of Drilling		Comment
	Water Level Depth (m)	Water Level Elevation (m)	
S7	--	--	Water used during borehole advancement
NW3-2	Dry	--	Upon completion of drilling
NW3-2A	3.4	91.9	
NW3-3	Dry	--	
PED-02	1.1*	94.1	Prior to start drilling on Dec 6, 2017
PED-03	Dry	--	Upon completion of drilling
PED-03A	Dry	--	Upon completion of drilling
PED-03B	Dry	--	Upon completion of overburden drilling and prior to rock coring.

Notes:

* Water level not considered to be representative due to the introduction of water to advance the drilling.

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

Borehole	Stratum Well Sealed Into	Water Level Depth (m)	Water Elevation (m)	Date of Piezometer Reading
S3	Fill/Silty Clay Till	0.8	89.2	November 6, 2018
PED-03A	Silt and Sand to Silty Sand Fill	Dry	-	October 10, 2017
		4.3	89.8	November 14, 2017
		4.4	89.7	November 21, 2017
		4.1	90.0	November 6, 2018
		4.4	89.7	November 28, 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.

4.2.1.12 Analytical Testing Results

Four soil samples were submitted for analysis of parameters used to assess the potential corrosivity of the site soil to steel and concrete. The following summarizes the results of the testing:

Parameter	Borehole S2 (SA #9 – Till)	Borehole S4 (SA #9A – Silt and Sand)	Borehole S5 (SA #9 – Silt and Sand)	Borehole S6 (SA #9 – Till)
pH	7.77	7.62	7.04	7.19
Resistivity (ohm-cm)	1500	720	680	840
Electrical Conductivity (umho/cm)	661	1390	1480	1190
Chlorides (ug/g)	37	600	760	630
Soluble Sulphates (ug/g)	550	260	<20*	<20*

Notes:

* Lower than Reportable Detection Limit

4.2.2 Kenollie Creek Culvert

In general, the subsurface conditions at the proposed culvert consist of a layer of topsoil or asphalt and/ or concrete underlain at most borehole locations by fill varying in composition from sand and gravel to gravelly sand in places, underlain by sandy silt to silt and silty sand to sand. The fill deposits are underlain by interlayered deposits of clayey silt with sand, sand to silty sand, and clayey silt with sand, which are in turn underlain by a till deposit consisting of clayey silt to sandy clayey silt and/ or silty sand, which is underlain by residual soil consisting of clayey silt, in places, underlain by shale bedrock. A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.2.1 Asphalt

A layer of asphalt pavement was encountered in Boreholes K2, K3, K4, K5 and K6 at ground surface and ranges in thickness from about 150 mm to 200 mm.

4.2.2.2 Concrete

A layer of concrete was encountered in Boreholes K5 and K6 underlying the asphalt pavement and in Borehole NRW7-3 at ground surface, and ranges in thickness from 300 mm to 430 mm.

4.2.2.3 Topsoil

A layer of topsoil was encountered in Borehole K1 at ground surface and has a thickness of 0.8 m. The water content measured on one sample of the topsoil is about 55 per cent.

4.2.2.4 Fill

A 0.4 m to 0.6 m thick layer of gravelly sand to sand and gravel fill was encountered underlying the asphalt or concrete in Boreholes K3, K4 and NRW7-3. The granular fill was encountered in Boreholes K3 and K4 at a depth of 0.2 m below ground surface (Elevation 94.8 m), concrete in Borehole NRW7-3 at a depth of 0.4 m below ground surface (Elevation 94.5 m).

In all boreholes, with the exception of Borehole K1, non-cohesive fill material consisting of sandy silt to silt and sand to silty sand, trace clay was generally encountered underlying the gravelly sand / sand and gravel fill, asphalt, and/ or concrete, or at ground surface at Borehole NRW3-6. The sandy silt to silty sand fill contains asphalt debris, wood chips, and 0.1 m of black organic silt. The surface of the deposit was encountered at depths ranging between about 0.0 m and 1.0 m below ground surface (between Elevations 94.5 m and 92.9 m), and the thickness of the overall fill material ranges from about 3.6 m to 5.1 m.

The SPT “N”-values measured within the gravelly sand / sand and gravel fill range from 25 blows to 49 blows per 0.3 m of penetration, indicating a compact to dense compactness condition. The SPT “N”-values measured within the sandy silt to silty sand fill range from 0 blows (weight of hammer) to 36 blows per 0.3 m of penetration, indicating a very loose to dense compactness condition.

Grain size distribution tests were carried out on eleven samples of the non-cohesive fill material and the results are shown on Figures B-1A and B-1B in Appendix B. An organic content test was completed on a sample of silty sand fill from Borehole K6 and the result is 1.2 per cent. Atterberg limits tests were carried out on two samples of the silt and sand to silty sand fill and indicate the material to be non-plastic.

The water content measured on samples of the silt and sand to silty sand fill ranges between about 4 per cent and 33 per cent. The water content measured on one sample of the sand and gravel fill is about 12 per cent.

4.2.2.5 Clayey Silt to Clayey Silt with Sand

A deposit of clayey silt, some sand to clayey silt with sand, some gravel was encountered below the non-cohesive fill deposit in Boreholes K4, K5, K6 and NRW7-3. The surface of the clayey silt deposit was encountered at a depth of about 4.9 m and 5.6 m below ground surface (between Elevations 90.1 m and 89.3 m), and the thickness of the deposit ranges from about 1.6 m to 4.6 m.

The SPT “N”-values measured within the clayey silt with sand deposit range from 6 blows to 48 blows per 0.3 m of penetration, suggesting a firm to hard consistency.

Grain size distribution tests were carried out on four samples of the clayey silt with sand and the results are shown on Figure B-2 in Appendix B. An organic content test was completed on a sample of the clayey silt with sand material underlying the fill deposit in Borehole K5 and the result is 3.2 per cent.

Atterberg limits tests were carried out on six samples of the clayey silt with sand deposit and measured liquid limits ranging from about 17 per cent to 27 per cent, plastic limits ranging from about 12 per cent to 20 per cent and plasticity indices ranging from about 5 per cent to 11 per cent. The results of the Atterberg limits test are plotted on the plasticity chart on Figure B-3 in Appendix B and indicate that the deposit consists of clayey silt of low plasticity.

The water content measured on samples of the clayey silt with sand deposit ranges between about 11 per cent and 26 per cent.

4.2.2.6 Silty Sand to Sand

A deposit of silty sand to sand, trace to some gravel was encountered below the clayey silt with sand deposit in Boreholes K4, K5, K6 and NRW7-3. The surface of the silty sand to sand deposit was encountered at depths about 7.2 m and 10.2 m below ground surface (between Elevations 87.8 m and 84.8 m), and the deposit ranges in thickness from about 2.6 m to 4.5 m.

The SPT “N”-values measured within the silty sand to sand deposit range from 16 blows and 53 blows per 0.3 m of penetration, with two “N”-values of 100 blows for 0.13 m of penetration, indicating a compact to very dense compactness condition.

Grain size distribution tests were carried out on three samples of the silty sand to sand deposit and the results are shown on Figure B-4 in Appendix B.

The water content measured on samples of the silty sand to sand deposit ranges between about 11 per cent and 21 per cent.

4.2.2.7 Till

A till deposit comprised of clayey silt to clayey silt with sand, trace gravel to with gravel and an interlayer of silty sand till was encountered below the silt and sand fill deposit in Boreholes K3 and NRW3-6, and below the sand deposit in Boreholes K4 and K6. The surface of the till deposit was encountered at depths between about 3.6 m and 11.7 m below ground surface (Elevations 89.4 m to 83.2 m), and the thickness of the deposit ranges from about 0.6 m to 4.4 m.

Two SPT “N”-values measured within the cohesive till deposit are 4 blows and 17 blows per 0.3 m of penetration and with “N”-values up to 100 blows for 0.10 m of penetration, suggesting that the cohesive till deposit has a firm to hard consistency. One SPT “N”-value measured within the silty sand till interlayer is 32 blows per 0.3 m of penetration, indicating a dense compactness condition.

Grain size distribution tests were carried out on three samples of the till deposit and the results are shown on Figure B-5 in Appendix B.

Atterberg limits tests were carried out on three samples of the till deposit and measured liquid limits ranging from about 15 per cent to 25 per cent, plastic limits ranging from about 12 per cent to 16 per cent and plasticity indices ranging from about 3 per cent to 9 per cent. The results of the Atterberg limits test are plotted on the plasticity chart on Figure B-6 in Appendix B and indicate the till consists of silt of slight plasticity to clayey silt of low plasticity. The water content measured on samples of the till deposit ranges between about 11 per cent and 22 per cent.

4.2.2.8 Sandy Clayey Silt to Clayey Silt (Residual Soil)

A deposit of residual soil comprised of clayey silt, some sand to sandy to clayey silt, trace gravel to with gravel, some shale fragments was encountered underlying the topsoil in Borehole K1, underlying the silty sand fill in Borehole K2, underlying the till deposit in Borehole NRW3-6, and underlying the silty sand layer in Borehole NRW7-3. The surface of the residual soil deposit was encountered at depths between about 0.8 m and 10.8 m below ground surface (between Elevations 89.3 m and 84.1 m), and the thickness of the deposit ranges from about 0.3 m to 1.5 m. Residual soil is a heterogeneous mix of fully weathered bedrock that is disintegrated into a soil like material that no longer retains the structure of parent bedrock.

The SPT “N”-values measured within the residual soil deposit generally range from 16 blows and 43 blows per 0.3 m of penetration, with “N”-values of 50 blows for 0.25 m and to 100 blows for 0.08 m of penetration at the interface with the overlying silty sand layer and underlying shale bedrock, respectively, suggesting a very stiff to hard consistency.

Grain size distribution tests were carried out on two samples of the residual soil deposit and the results are shown on Figure B-7 in Appendix B.

Atterberg limits tests were carried out on two samples of the residual soil deposit and measured liquid limits of about 30 per cent and 34 per cent, plastic limits of about 20 per cent and 21 per cent, and plasticity indices of about 10 per cent and 13 per cent. The results of the Atterberg limits test are plotted on the plasticity chart on Figure B-8 in Appendix B and indicate the residual soil consists of clayey silt of low plasticity.

The water content measured on samples of the residual soil deposit ranges between about 7 per cent and 19 per cent.

4.2.2.9 Shale Bedrock

Bedrock was encountered and core samples were recovered in all boreholes with the exception of Borehole NRW7-3, where the presence of bedrock is inferred by refusal to further split-spoon advancement. The depths to bedrock or refusal below ground surface, the corresponding bedrock surface elevation or refusal elevation and the cored depths are summarized below.

Borehole	Depth to Bedrock Surface / Refusal (m)	Bedrock Surface / Refusal Elevation (m)	Comments
K1	1.1	89.0	Bedrock cored 3.8 m
K2	5.3	87.9	Bedrock cored 3.1 m
K3	10.0	85.0	Bedrock cored 3.4 m
K4	10.8	84.2	Bedrock cored 3.0 m
K5 / K5A	13.4 / 12.8	81.6 / 82.2	Bedrock cored 3.7 m
K6	12.3	82.6	Bedrock cored 0.7 m[1]
NRW3-6	6.2	86.7	Bedrock cored 3.9 m
NRW7-3	12.3	82.6	Refusal to split-spoon advancement

In general, the bedrock surface as encountered along the alignment of the proposed culvert replacement slopes downwards from north-west to south-east.

Based on a review of the bedrock core samples the bedrock consists of shale of the Georgian Bay Formation. In general, the bedrock samples are described as slightly weathered, thinly bedded, fine grained, faintly porous, weak, grey shale, with medium strong to strong limestone interbeds at varying intervals of depth. It is also considered that the shale bedrock is completely to highly weathered in the upper 0.1 m to 1.5 m, based on the encountered resistance to auger advancement and penetration by the split-spoon.

The strong limestone layers range in thickness from 10 mm to 90 mm, with an average thickness of about 20 mm. The stronger layers generally make up about 5 per cent by thickness of the rock encountered during the investigation. The details of the bedrock descriptions are presented on the Record of Drillhole sheets and the photographs of the recovered bedrock core are shown on Figures B-9 to B-15 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)[3] standard classification system.

The Rock Quality Designation (RQD) measured on the core samples obtained from the current investigation ranges from about 28 per cent to 100 per cent with two short runs (less than 0.1 m) measuring an RQD of 0 per cent, indicating a rock mass of poor to excellent quality, and generally an RQD greater than 62 per cent below the upper weathered zone, indicating fair to excellent quality, as per Table 3.10 of CFEM (2006)[4]. The Total Core Recovery (TCR) and Solid Core Recovery (SCR) of samples recovered are between 54 per cent and 100 per cent and between 3 per cent and 99 per cent, respectively, with very small TCR and SCR values measured in highly weathered zone and short core runs.

Unconfined Compression (UC) tests (ASTM D7012)[5] 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. The core sample from Borehole K1 consisted of shale with limestone inclusions, whereas the core sample from Boreholes K2 and K3 consisted of slightly weathered shale.

Borehole No./ Run	Sample Depth Interval (m)	Sample Elevation Interval (m)	Uniaxial Compressive Strength (UCS) (MPa)	Bulk Density (g/cm^3)	Tangent Young's Modulus (GPa)
K1/ Run #2	3.1 – 3.3	87.0 – 86.8	6.4	2.58	0.9
K2/ Run #2	8.1 – 8.2	85.1 – 85.0	13.0	2.55	1.5
K3/ Run #2	11.9 – 12.1	83.1 – 82.9	18.2	2.58	2.3

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).

4.2.2.10 Groundwater Conditions

The overburden samples obtained from the borehole investigations were generally moist to wet. The depths and elevation of the water level observed in the boreholes upon completion of drilling and prior to rock coring is presented below.

Borehole	Upon Completion of Drilling		Comment
	Water Level Depth (m)	Water Elevation (m)	
K1	1.2	88.9	Upon completion of overburden drilling and prior to rock coring.
K2	5.2	88.0	
K3	9.8	85.2	
K4	7.3	87.7	

Borehole	Upon Completion of Drilling		Comment
	Water Level Depth (m)	Water Elevation (m)	
K5	2.4	92.6	Water used during soil drilling.
K6	12.3	82.6	
NRW3-6	5.2	87.7	
NRW7-3	6.4	88.5	Upon completion of soil drilling

The water level recorded in the standpipe piezometer installed in one borehole of the current investigation are presented below.

Borehole	Stratum Well Sealed Into	Water Level Depth (m)	Water Elevation (m)	Date of Piezometer Reading
K2	Fill / Residual Soil / Bedrock	2.1	91.1	December 17, 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.

4.2.2.11 Analytical Testing Results

As noted in Section 3.0, one sample of crushed and homogenized shale bedrock core from Borehole K1 and two samples of fill (from Boreholes K3 and K6) were submitted for analysis of parameters used to assess the potential corrosivity of the site soil and bedrock to steel and concrete. The following summarizes the results of the testing:

Parameter	Borehole K1 (Run #1 Shale)	Borehole K3 (SA#7 FILL)	Borehole K6 (SA#5 FILL)
pH	7.73	7.10	7.65
Resistivity (ohm-cm)	2700	810	640
Electrical Conductivity (umho/cm)	372	1230	1550
Chlorides (ug/g)	53	600	830
Soluble Sulphates (ug/g)	97	210	46

5.0 CLOSURE

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

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PART B

FOUNDATION DESIGN REPORT
STAVEBANK CREEK CULVERT AND KENOLLIE CREEK CULVERT REPLACEMENTS
QEW WIDENING FROM WEST OF MISSISSAUGA ROAD TO WEST OF HURONTARIO STREET, CITY OF MISSISSAUGA
MTO, G.W.P. 2002-13-00

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides foundation engineering design recommendations for the proposed replacement of Stavebank Creek Culvert and Kenollie Creek Culvert, which will be constructed under the existing Premium Way roadway embankment, the existing QEW highway platform and the new embankment associated with the widening of the QEW highway platform along the north side of the QEW. These recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the subsurface investigation. The discussion and recommendations presented are intended to provide the designers with sufficient information to assess the feasible culvert foundation alternatives and carry out the design of the Culvert foundations, and to provide the designers with sufficient information to assess the feasible protection system alternatives. The foundation investigation report, discussion and recommendations are intended for the use of the Ministry of Transportation Ontario (MTO) and shall not be used or relied upon for any other purpose or by any other parties, including the construction or design-build contractor. The contractor must make their own interpretation based on the factual data in Part A (Foundation Investigation) of the report. Where comments are made on construction, they are provided to highlight those aspects that could affect the design of the project and for which special provisions may be required in the Contract Documents. Those requiring information on the aspects of construction must make their own interpretation of the factual information provided, as such interpretation may affect equipment selection, proposed construction methods, scheduling, and the like.

6.1 General

Plan and profiles drawings for the Stavebank Creek and Kenollie Creek Culverts, submitted as part of the revised 90 per cent design completion review package, were provided to Golder by MH on January 4, 2019. Digital AutoCADD files of the plan and profile for both Culverts was also provided to Golder by MH on January 14, 2019. The following summarizes the details regarding the proposed culverts:

Culvert Location	Proposed Structure Dimensions[1]	Approximate Existing Maximum Embankment Height[2] (m)	Culvert Invert Elevation (m)	
			Upstream	Downstream
Stavebank Creek – Under Premium Way	1800 mm wide x 1200 mm high Concrete Box (30 m long)	5.5	89.2	88.2
Stavebank Creek – Under QEW and Widened Platform	1800 mm wide x 1200 mm high Concrete Box (79 m long)	6.5	88.1	87.8
Kenollie Creek	3000 mm wide x 2100 mm high Concrete Box (82 m long)	4.0	89.0	88.8

Notes:

1. Interior dimension.
2. Maximum embankment height above top of culvert and average surrounding natural ground surface.

At Stavebank Creek, the existing culverts which carry the creek under the QEW and Premium Way will be removed by open cut and replaced on the same horizonal alignment by two longitudinal culverts under the widened QEW highway and Premium Way, connected with a Ontario Provincial Standard Drawing (OPSD) 1101.012 1800 mm by 2400 mm wide and about by 6.3 m high precast concrete chamber, on the north side of the proposed QEW westbound lanes. The proposed invert of the concrete chamber is at Elevation 86.0 m. A plunge pool, with dimensions of 3 m by 3 m and bottom at Elevation 87.5 m, is proposed at the outlet of the new Stavebank Creek Culvert.

At Kenollie Creek, the existing culvert will be removed by open cut and replaced by a single culvert, on a new alignment shifted slightly to the east of the existing structure, extending under the widened QEW, Premium Way and the proposed multi-use pathway along the north side of the proposed QEW westbound lanes. Similar to the proposed Stavebank Creek Culvert, a plunge pool with dimensions of 3 m by 3 m and bottom at Elevation of 88.5 m is proposed at the outlet of the new Kenollie Creek culvert.

Temporary protection systems will be required to facilitate the open cut removal and replacement of the Stavebank Creek and Kenollie Creek Culverts and to accommodate the proposed traffic staging plans.

6.2 Consequence and Site Understanding Classification

In accordance with Section 6.5 of the *Canadian Highway Bridge Design Code* and its *Commentary* (CHBDC, 2014), the proposed culvert foundation systems at each site are 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 these locations in comparison to the degree of site understanding in Section 6.5 of the CHBDC (2014), the level of confidence for foundation design of the culverts 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

Both the Stavebank Creek Culvert site and the Kenollie Creek Culvert site may be classified as Site Class C in accordance with Table 4.1 of the CHBDC (2014), wherein the Seismic Site Classification was determined by energy corrected average penetration resistance. Geophysics testing, if carried out, can often provide a more favourable Site Class designation, but this may not be feasible at these sites. For example, Table 4.1 of the CHBDC (2014) indicates that Site Class A and B are not to be used if there is more than 3 m of soil between the underside of the foundations and bedrock.

6.3.2 Spectral Response Values and Seismic Performance Category

In accordance with Section 4.4.3.4 of the CHBDC (2014), the peak ground acceleration (PGA) values and design spectral acceleration (Sa) values for Site Class C 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)	0.043	0.080	0.156
PGV (m/s)	0.032	0.053	0.097
Sa (0.2) (g)	0.072	0.127	0.242
Sa (0.5) (g)	0.043	0.069	0.123

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)
Sa (1.0) (g)	0.023	0.037	0.061
Sa (2.0) (g)	0.011	0.017	0.029
Sa (5.0) (g)	0.0024	0.0040	0.0068
Sa (10.0) (g)	0.0004	0.0016	0.0028

6.4 Foundations Options - Culverts

Either box culverts or “open footing” (shallow foundation) concrete culverts are feasible for replacement of the existing at Stavebank Creek and Kenollie Creek Culverts. Both pre-cast concrete elements (box culvert segments or footing elements) and cast-in-place concrete elements are also feasible from a foundations perspective.

From a foundation perspective, pre-cast concrete box culverts are preferred as replacement structures for both sites over cast-in-place open footing culverts based on the following:

- Pre-cast concrete box culvert construction minimizes the depth of excavation and groundwater control requirements as compared with open footing culverts.
- Pre-cast concrete box culvert segments can usually be installed more expeditiously than cast-in-place open footing culverts, resulting in shorter durations for dewatering, surface water pumping and traffic staging.
- Pre-cast concrete box culvert segments are more tolerant of total and differential settlement, although this is not considered a significant concern at these culvert sites.

Table 1, following the text report, identifies and presents an assessment of the advantages, disadvantages, relative costs and risks/consequences of box culverts and open footing culvert options for these two sites.

Although box culverts may not satisfy fisheries requirements in some applications, it is understood that the design team has adopted box culverts for these two sites; however, recommendations for both the box culvert option and open footing option are provided in the following sections of this report.

6.5 Founding Elevations and Subexcavation Requirements

6.5.1 Box Culverts

It is not necessary to found new box culverts at the standard depth for frost protection purposes, which at this site is 1.2 m as interpolated from OPSD 3090.101 (*Frost Penetration Depths for Southern Ontario*), as the box structure sections are tolerant of small magnitudes of movement related to freeze-thaw cycles, should these occur. Box culverts should, however, be founded below any existing fill/softened soils and surficial / near surface organic materials. The following summarizes the recommended founding levels and subexcavation requirements for new box culverts, based on the inverts of the proposed culverts noted in Section 6.1 and an assumed base slab thickness of 300 mm at both the Stavebank Creek and Kenollie Creek Culvert locations.

Culvert	Proposed Underside of Culvert (m) Upstream / Downstream	Subexcavation Required? Inlet / Outlet	Excavation/Sub-Excavation Elevation (m) Upstream / Downstream	Subgrade Stratum
Stavebank Creek – Under Premium Way	88.9 / 87.9	Yes, about 0.3 m depth to accommodate bedding material	88.6 / 87.6	Compact / very dense Silty Sand / Sand and Gravel; and, Stiff to hard Clayey Silt with Sand (Till).
Stavebank Creek – Under QEW	87.8 / 87.5	Yes, up to about 0.5 m depth below base of culvert where fill is present under the proposed QEW WBL	87.3 / 87.0	Compact Silt and Sand / Silty Sand; and, Stiff to hard Clayey Silt with Sand (Till).
Kenollie Creek	88.7 / 88.5	Yes, about 0.3 m depth to accommodate bedding material	88.4 / 88.2	Slightly weathered shale bedrock; Very stiff to hard Sandy Clayey silt with Gravel (residual soil); Firm Clayey Silt with Sand; and, Stiff to hard Clayey Silt with Sand (Till).

The box culvert subgrade should be inspected by geotechnical personnel to ensure that all existing topsoil and fill/softened soils or other unsuitable material have been removed. Following inspection, any subexcavated areas should be backfilled with granular material meeting OPSS.PROV 1010 (*Aggregates)* Granular ‘A’ or Granular ‘B’ Type II, placed and compacted in accordance with OPSS.PROV 501 (*Compacting*).

The till, shale, residual soil, and clayey silt subgrade will be susceptible to loosening/softening and degradation on exposure to water and construction traffic. As discussed further in Section 6.8.6, if the subexcavation backfill or bedding for the culvert is not placed within four hours after preparing the subgrade a concrete working slab having a minimum thickness of 100 mm and a minimum 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. A Non-Standard Special Provision (NSSP) to address this item is included in Appendix E, which should be included in the Contract Documents.

Box culverts may need to be comprised of will be articulated sections to accommodate differential settlement which may occur due to frost action or due to settlement of the foundation soils resulting from the placement of new embankment fill between the QEW and Premium Way and along the north side of Premium Way (Section 6.11).

6.5.2 Open Footing Culverts

Strip footings for open footing culvert replacements should be founded at a minimum depth of 1.2 m below the lowest surrounding grade to provide adequate protection against frost penetration, as per Ontario Provincial Standard Drawing (OPSD) 3090.101 (*Foundation, Frost Penetration Depths for Southern Ontario*). In addition, the footings should extend below any existing fill and surficial organic materials, where present. The following summarizes the recommended founding levels and subexcavation requirements for new open footing culverts, based on the inverts of the proposed culverts noted in Section 6.1:

Culvert	Proposed Culvert Invert (m) Upstream / Downstream	Subexcavation Required? Inlet / Outlet	Underside of Footing Elevation (m) Upstream / Downstream	Subgrade Stratum
Stavebank Creek – Under Premium Way	89.2 / 88.2	No	88.0 / 87.0	Very dense Sand and Gravel; and, Very stiff to hard Clayey Silt with Sand (Till).
Stavebank Creek – Under QEW	88.1 / 87.8	No	86.9 / 86.6	Compact Silt and Sand / Silty Sand; and, Compact Silt and Sand (Till); Stiff to hard Clayey Silt with Sand (Till).
Kenollie Creek	89.0 / 88.8	No	87.8 / 87.6	Slightly weathered shale bedrock; Hard Sandy Clayey silt with Gravel (residual soil); Hard/Dense Clayey Silt with Sand to Silty Sand (Till); Hard Clayey Silt with Sand; Compact to Dense Silty Sand to Sand.

The footing subgrade should be inspected following excavation, in accordance with OPSS 902 *(Excavating and Backfilling Structures)*, as amended by SP 109S12, to check that all existing fill and surficial organic soils or other unsuitable material have been removed. Where sub-excavation is required to remove unsuitable materials, the sub-excavated area should be backfilled with granular material meeting OPSS.PROV 1010 *(Aggregates)* Granular ‘A’ or Granular ‘B’ Type II that is placed and compacted in accordance with OPSS.PROV 501 *(Compacting)* as amended by SP 105S12.

Dewatering and surface water control will be required for excavation and construction of open footing culverts. The groundwater level must be lowered to a minimum of 1 m below the base of the excavation prior to carrying out any excavation for the culvert. At Stavebank Creek Culvert the groundwater level is about 2.2 m above the base of the excavation; however, at Kenollie Creek Culvert the groundwater is under some hydrostatic pressure. As discussed further in Section 6.15.4, it is recommended that an NSSP be included in the Contract Documents to address groundwater control requirements for the culvert sites

The footing subgrade will be susceptible to loosening/softening and degradation on exposure to water and construction traffic. As discussed further in Section 6.8, if the subexcavation backfill or bedding for the culvert is not placed within four hours after preparing the subgrade a concrete working slab should be placed to protect the integrity of the subgrade. An example NSSP for the working slab is included in Appendix E and should be included in the Contract Documents.

6.6 Geotechnical Resistance

6.6.1 Box Culverts

The box culvert at Stavebank Creek constructed on compacted bedding placed on the native silt and sand / silty sand / sand and gravel and clayey silt with sand till deposits and the box culvert at Kenollie Creek constructed on shale bedrock, residual soil, sandy clayey silt with gravel till or clayey silt, and founded at or below the design elevations given in the Section 6.5.1, may be designed based on the factored ultimate geotechnical resistances and factored serviceability geotechnical resistances (for 25 mm of settlement) given below.

Culvert	Founding Stratum	Factored Ultimate Geotechnical Resistance	Factored Serviceability Geotechnical Resistance (for 25 mm settlement)
Stavebank Creek – 1.8 m wide under Premium Way	Compacted granular bedding on: - very stiff to hard clayey silt with sand (Till); and, - compact, silty sand very dense sand and gravel	200 kPa	200 kPa
Stavebank Creek - 1.8 m wide under QEW	Compacted granular bedding on: - stiff to hard clayey silt with sand (Till) - compact silt and sand (Till); and, - compact silt and sand to silty sand	200 kPa	200 kPa
Kenollie Creek 3.0 m wide	Compacted granular bedding on: - very stiff to hard gravelly sandy clayey; silt (Residual Soil)	350 kPa	300 kPa
	Compacted granular bedding on: - stiff to hard clayey silt with sand (Till); and, - firm to very stiff clayey silt with sand	200 kPa	200 kPa

Notes:

1. Refer to Section 6.5.7 for discussion on settlement of culverts under new embankment loading.

The geotechnical resistances and settlement are dependent on the box culvert span, configuration and applied loads, including the loads imparted by the new embankment construction; the geotechnical resistances/reactions, therefore, must be reviewed if the culvert span/footing size or founding elevation differs significantly from that given above. The geotechnical resistances provided above are based on loading applied perpendicular to the surface of the footings. Where the load is not applied perpendicular to the surface of the footing, inclination of the load should be taken into account in accordance with Section 6.10.4 of the CHBDC (2014).

It should be noted that at the Kenollie Creek Culvert site, the proposed culvert will be founded on variable subgrade conditions. Though, it is not expected that settlement will exceed the serviceability limits state at any point along the alignment, differential settlement (less than 25 mm) should be expected based on the varying composition and quality of the founding stratum along the culvert alignment (see Section 6.5.7 for further details).

6.6.2 Open Footing Culverts

Strip footings placed on the properly prepared subgrade at or below the founding elevations recommended in Section 6.5.2, should be designed based on the factored ultimate geotechnical resistance values and the factored serviceability geotechnical resistance values (for 25 mm of settlement) as given below. These recommendations are based on an assumed footing width of 1 m to 2 m.

Culvert	Founding Stratum	Factored Ultimate Geotechnical Resistance	Factored Serviceability Geotechnical Resistance (for 25 mm settlement)
Stavebank Creek – under Premium Way	- very stiff to hard clayey silt with sand (Till); and, - compact, silty sand / sand and gravel.	325 kPa	300 kPa
Stavebank Creek - under QEW	- stiff to hard clayey silt with sand (Till) Compact Silt and Sand (Till); and, - compact silt and sand to sand.	325 kPa	300 kPa
	- very stiff to hard gravelly sandy clayey silt (Residual Soil); - hard/dense clayey silt with sand to Silty Sand (Till); - hard clayey silt with sand; and, - compact to dense silt sand to sand.	200 kPa	175 kPa

Notes:

1. The factored serviceability geotechnical resistance (for 25 mm of settlement) is greater than the factored ultimate geotechnical resistance, and therefore the factored ultimate geotechnical resistance should be used in design.
2. Refer to Section 6.11 for discussion on settlement of culverts under new embankment loading.

The geotechnical resistances and settlement are dependent on the footing size, configuration and applied loads, including the loads imparted by the existing and widened embankment construction; the geotechnical resistances/reactions, therefore, must be reviewed if the footing size or founding elevation differs significantly from that given above. The geotechnical resistances provided above are based on loading applied perpendicular to the surface of the footings. Where the load is not applied perpendicular to the surface of the footing, inclination of the load should be taken into account in accordance with Section 6.10.4 of the CHBDC (2014).

Similar to the box culvert, it should be noted that at the Kenollie Creek site, the proposed footing will be founded on variable subgrade conditions. Though, it is not expected that settlement will exceed the serviceability limits state at any point along the alignment, differential settlement (less than 25 mm) should be expected based on the varying composition and quality of the founding stratum along the culvert alignment (see Section 6.11).

6.7 Resistance to Lateral Loads / Sliding Resistance

Resistance to lateral forces / sliding resistance between the base slab/footing for the new culverts and the subgrade should be calculated in accordance with Section 6.10.5 of the CHBDC (2014). A coefficient of friction, tan δ, (unfactored) which may be used for design at each site is provided below.

Culvert	Founding Stratum	Coefficient of friction, tan δ (unfactored)
Box Culverts		
Stavebank Creek – Under Premium Way	Compacted granular bedding on: - compact silty sand / sand and gravel; - stiff to hard clayey silt with sand (Till)	0.40
Stavebank Creek – Under QEW	Compacted granular bedding on: - stiff to hard clayey silt with sand (Till) - compact silt and sand to sand	0.40

Culvert	Founding Stratum	Coefficient of friction, tan δ (unfactored)
Kenollie Creek	Compacted granular bedding on: - shale bedrock; - very stiff to hard sandy clayey silt with gravel (Residual Soil); - stiff to hard gravelly clayey silt with sand (Till); - firm to very stiff clayey silt with sand	0.40
Pre-Cast Concrete Culverts	Compacted Granular 'A' Bedding	0.45

Culvert	Founding Stratum	Coefficient of friction, tan φ' (unfactored)
Open Footing - Cast In-place Concrete Footing		
Stavebank Creek – Under Premium Way	- stiff to hard clayey silt with sand (Till); and, - compact, silty sand / sand and gravel.	0.35
Stavebank Creek – Under QEW	- stiff to hard clayey silt with sand (Till); and, - compact silt and sand to sand.	0.35
Kenollie Creek	- shale bedrock; - very stiff to hard gravelly sandy clayey silt (Residual Soil); - hard/dense clayey silt with sand to Silty Sand (Till); - hard clayey silt with sand; and, - compact to dense silt sand to sand.	0.35

6.8 Culvert Bedding and Backfill

For the new box culverts, the bedding/levelling course and backfill requirements should be in accordance with OPSS 422 (*Precast Reinforced Concrete Box Culverts and Box Sewers in Open Cut*). New box culverts should be provided with at least 300 mm of OPSS.PROV 1010 (*Aggregates*), Granular 'A' material for bedding purposes, or alternatively a 100 mm thick concrete working slab. The levelling course may consist of OPSS.PROV 1010 (*Aggregates*), Granular 'A' or OPPS.PROV 1002 (*Aggregates - Concrete*) Fine Aggregate.

Granular bedding is not required for footings for open footing culverts. Footings can be placed directly on properly prepared subgrade, as described in Section 6.5.2.

Culvert construction, backfill and cover for all concrete culverts (either box culvert or open footing) should be completed in accordance with OPSD 803.010 (*Backfill and Cover for Concrete Culverts*), including the placement of a 75 mm thick levelling course. Backfill to culvert walls and cover should consist of granular fill meeting the requirements of OPSS.PROV 1010 (*Aggregates*), Granular 'A' or Granular 'B' Type II. The backfill and cover should be placed and compacted in accordance with OPSS.PROV 501 (*Compacting*) as amended by SP 105S22. The new culverts should be designed for the full overburden and hydrostatic pressures, and live load, assuming that the embankment fill has a unit weight of 22 kN/m^3 for OPSS.PROV 1010 (*Aggregates*) Granular 'A', 21 kN/m^3 for Granular 'B' Type II and 19 kN/m^3 for earth fill above the cover comprised of Granular 'B' Type I, Select Subgrade Material (SSM) or earth borrow.

Excavated fill material from the existing embankment may be used to backfill above the culvert cover material within the footprint of the existing highway embankment. Excavated fill material should meet the specifications for suitable earth borrow material as per OPSS.PROV 212 (*Earth Borrow*) and in accordance with OPSS.PROV 206 (*Grading*) and placed and compacted in accordance with OPSS.PROV 501 (*Compacting*) as amended by SP 105S22. The existing fill material from above the groundwater level is expected to near its optimum moisture content for compacting. Fill material from below the groundwater level will likely require drying in order to reach optimum moisture content, prior to placement and compact.

Backfill placement for the reconstruction of the roadway embankments placed along and over the culverts should be carried out as per OPSD 208.010 (Benching of Earth Slopes) to integrate the existing embankment fill and the new fill along the cut faces.

6.9 Embankment Reconstruction and Widening

Reconstruction of the roadway embankments immediately adjacent to the culverts and construction of the new roadway embankment platforms for the widening sections of the roadway along the north side of Premium Way and along the north side of the QEW (between the QEW WBL and the existing Premium Way) should be carried out in accordance with OPSS.PROV 206 (*Grading*) using suitable earth borrow material as per OPSS.PROV 212 (*Earth Borrow*) and / or OPSS.PROV 1010 (*Aggregates*) Granular ‘A’, Granular ‘B’ Type I or Select Subgrade Material (SSM), and in accordance with OPSD 208.010 (*Benching of Earth Slopes*) to integrate the existing embankment fill and the new fill along the existing roadway embankment side slopes. The final embankment side slopes should be protected against erosion by surface water runoff as soon as practicable after completion of slope grading using a combination of materials in accordance with OPSS.PROV 802 (*Topsoil*), OPSS 803 (Sodding) and / or OPSS.PROV 804 (*Seed and Cover*) as applicable for the QEW roadway and Premium Way municipal street.

6.10 Culvert Erosion Protection

Provision should be made for scour and erosion protection at the inlet and outlet of both Stavebank Creek and Kenollie Creek Culverts. In order to prevent surface water from flowing either beneath the culvert (i.e., in the case of a box culverts), potentially causing undermining and scouring, or around the culvert, creating seepage through the embankment fill and potentially causing erosion and loss of fine soil particles, a clay seal or concrete cut-off wall should be provided at the upstream and downstream end of each culvert. If a clay seal is adopted, the clay material should meet the requirements of OPSS.PROV 1205 (*Clay Seal*), and the seal should extend from a depth of 1 m below the scour level to a minimum horizontal distance of 2 m on either side of the culvert inlet opening, and a minimum vertical height equivalent to the high water level including along the embankment slope.

If the creek flow velocities are sufficiently high, provision should be made for scour and erosion protection (suitable non-woven geotextiles and/or rip-rap) at the culvert inlet and outlet, including in front of any wing walls/retaining walls adjacent to the channel. The requirements for and design of erosion protection measures for the inlet and outlet of the culvert should be assessed by the hydraulic design engineer. As a minimum, rip-rap treatment for the outlet of the culvert should be consistent with the standard presented in OPSD 810.010 (*Rip-Rap Treatment*) using OPSS.PROV 1004 (*Aggregates – Miscellaneous*) R-10 or R-50 size rip-rap material as may be required by the hydraulic design engineer. Erosion protection for the inlet of the culverts should also follow the standard presented in OPSD 810.010 (*Rip-Rap Treatment*) similar to the outlet, but with the rip-rap should be placed up to the toe of slope level, in combination with the cut-off measures noted above.

6.11 Settlement

Generally, the new culverts will be installed under existing embankment fill of the QEW and Premium Way. However, as a result of widening the QEW along the north side of the existing roadway, a new section of embankment between the existing QEW and Premium Way will be constructed to accommodate the QEW widening.

At the Stavebank Creek Culvert, fill heights of up to about 7 m above existing ground surface will be required to construct the embankment for the proposed QEW widening westbound lanes. The additional fill height will extend from the connection chamber southerly for about 27 m to the existing QEW embankment. The remaining portion of the new culvert will be constructed under existing embankments and settlement of the culvert due to the re-placement of fill material is not expected at these locations.

At the Kenollie Creek Culvert, fill heights of about 4.5 m will be required to widen the existing Premium Way embankment northerly to accommodate the proposed multi-use pathway. The additional fill height will extend from about the inlet of the culvert to about 15 m downstream. Fill heights of approximately 2 m will be required to widen the QEW embankment north to accommodate the proposed QEW westbound lanes. The additional fill height will extend from about 25 m downstream of the culvert inlet to about 35 m downstream. The proposed grade raises along the culvert alignments are summarised below.

Culvert	Approximate Location	Existing Ground Surface Elevation (m)	Proposed Pavement Surface Elevation (m)	Approximate Maximum Grade Raise (m)
Stavebank Creek – Under QEW Widening	Stations 0+035 to 0+065	88.0	95.0	7.0
Kenollie Creek (at inlet) - Under Premium Way Widening	Stations 0+002 to 0+017	89.5	94.0	4.5
Kenollie Creek – Under QEW Widening	Stations 0+023 to 0+037	93.0	95.0	2.0

The settlement analysis for the culvert sites was carried out using the commercially available program Settle-3D by Rocscience and hand calculations using estimated elastic deformation moduli as given below for each culvert, based on correlations with SPT “N”-values and engineering judgement from experience with similar soils in this region of Ontario.

Soil Deposit	Bulk Unit Weight	Elastic Modulus
New Embankment Fill and Granular Bedding	21 kN/m^3	Not Applicable
Stiff to hard clayey silt with sand (Till)	20 kN/m^3	50 MPa (Stiff to very stiff)
		100 MPa (Hard)
Residual Soil	20 kN/m^3	100 MPa
Bedrock	23 kN/m^3	1000 MPa

The approximately 7.0 m thick new embankment section fill loading at the Stavebank Creek Culvert under the proposed QEW Westbound lanes, is expected to induce settlement of the underlying foundation soils in the range of 15 mm to 20 mm. The estimated settlement will be gradual over the length of the culvert, and roughly proportional to the change in fill thickness along the 2H:1V (2 horizontal to 1 vertical) side slopes of the existing QEW embankment north slopes and therefore no settlement mitigation measures are considered required for the culvert. The estimated settlement assumes that the subexcavation recommendations provided is Section 6.5.1 are completed and subexcavated material is replaced with compacted granular bedding below the culvert footprint as may be applicable. The settlement of the foundation soils under the approximately 4.5 m thick new embankment section along the north side of Premium Way and the 2.0 m thick additional fill along the north side of the QEW embankment at the Kenollie Creek Culvert is estimated to be negligible (i.e. less than 5 mm), and therefore no settlement mitigation measures are required for the culvert. This assumes that the subexcavation recommendations provided in Section 6.5.1 are completed and subexcavated material is replaced with compacted granular bedding below the culvert footprint.

6.12 Stability of Widening Embankment

At the Stavebank Creek Culvert site the existing embankment at the inlet (i.e., at Premium Way) is being widened to the north to accommodate the new Multi-Use Trail. In addition, new fill is being placed between the existing Premium Way embankment and QEW embankment to accommodate the new QEW westbound lanes. At the Kenollie Creek Culvert site the existing embankment as the inlet will also be widened to the north. Limit equilibrium slope stability analyses were performed for the widened embankment associated with the Stavebank Creek Culvert and Kenollie Creek Culvert construction 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 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 shale bedrock, the effective stress parameters employed in the analysis were estimated based on similar bedrock conditions in the areas adjacent to the Kenollie Creek Culvert site as encountered for the overall project.

Summarized below are the simplified stratigraphy and the associated strengths and unit weights employed for the different soil types in the embankment area at Stavebank Creek Culvert.

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 Type II Fill (New Embankment Fill)	22	35°	0	--
Select Subgrade Material - SSM (New Embankment Fill)	20	32°	0	
Compacted Earth Fill (New Embankment Fill)	20	28°	0	
Existing Soft to Very Stiff / Loose to Compact Earth Fill	20	28°	0	--
Very Loose to Compact Silty Sand to Silt and Sand to Sand	18	28°	0	--
Very Dense Sand and Gravel	21	35°	0	--
Firm to Hard / Compact to Dense Clayey Silt with Sand (TILL) / Silt and Sand (TILL)	21	33°	0	

Below are the simplified stratigraphy and the associated strengths and unit weights employed for the different soil and rock types in the embankment area at Kenollie Creek Culvert.

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 Type II Fill (New Embankment Fill)	22	35°	0	--
Select Subgrade Material SSM (New Embankment Fill)	20	32°	0	
Compacted Earth Fill (New Embankment Fill)	20	28°	0	
Existing Very Loose to Dense Earth Fill	20	28°	0	--
Hard sandy clayey silt (Residual soil)	20	34°	0	--
Shale bedrock	23	50°	175	--

At the Stavebank Creek Culvert site, in order to achieve a factored global (failure of existing embankment foundation soils) Factor of Safety (FoS) of 1.33 in the short-term/temporary condition and 1.54 in the long-term/permanent condition, the existing embankment fill should be removed starting at the toe of the existing embankment and extending a horizontal distance of not less than 3.0 m and sloping back to meet the top of existing embankment at a slope of 2H:1V. The widened embankment may then be constructed using OPSS.PROV 1010 (*Aggregates)* Granular 'A' or Granular 'B' Type I or Type II material inclined at a side slope of 2H:1V. Alternatively, consideration could be given to using OPSS.PROV 1010 (*Aggregates*) Select Subgrade Material (SSM); however, in order to achieve the factored global FoS, as detailed above, the embankment must be inclined at 2.25H:1V or flatter. Consideration could also be given to the use of compacted earth fill meeting the requirements of OPSS.PROV 212 (*Earth Borrow*); however, in order to meet the requirements for global slope stability, the side slope of the widened embankment would have to be constructed at an inclination of 2.5H:1V or flatter. The result of a stability analysis for the embankment side slopes constructed of Granular 'A' or Granular 'B' Type I or II, inclined at 2H:1V is shown on Figure 1.

If it is considered preferable to leave existing fill in-place (instead of excavating and removing it as recommended above), the new embankment side slopes must inclined at 2.5H:1V, provided the embankment is constructed of OPSS.PROV 1010 (*Aggregates)* Granular 'A', Granular 'B' Type I or Type II, or SSM. Compacted earth fill meeting the requirements of OPSS.PROV 212 (*Earth Borrow*) may also be considered; however, the side slopes of the widened embankment would have to be constructed at an inclination of 2.75H:1V or flatter to achieve the required factored Global FoS. As discussed in Section 6.9, if the existing embankment is left in place the side slopes should be benched in accordance with OPSD 208.010 (*Benching of Earth Slopes*) to integrate the existing embankment fill and the new fill along the existing roadway embankment side slopes.

At the Kenollie Creek Culvert site, in order to achieve a factored global (failure of embankment foundation soils) FoS of 1.33 in the short-term/temporary condition and 1.54 in the long-term/permanent condition, the proposed widened embankment may be constructed at side slope of 2H:1V, provided the embankment is constructed of OPSS.PROV 1010 (*Aggregates)* Granular 'A' or Granular 'B' Type I or Type II material. Alternatively, consideration could be given to the use of OPSS.PROV 1010 (*Aggregates*) Select Subgrade Material (SSM), provided the embankment is inclined at 2.5H:1V or flatter to achieve the required factored global FoS. Consideration could also be given to the use of compacted earth fill meeting the requirements of OPSS.PROV 212 (*Earth Borrow*); however, in order to meet the requirements for the factored Global FoS, the side slopes of the widened embankment will have to be constructed at an inclination of 2.75H:1V or flatter. The result of a stability analysis for the embankment side slopes constructed of Granular A' or Granular 'B' Type I or Type II material, inclined at 2H:1V is shown on Figure 2.

It is noted that surficial stability of the embankment at both the Stavebank Creek Culvert and Kenollie Creek Culvert sites is less than the factored global FoS of 1.54, but greater than 1.3. Erosion protection and on-going maintenance of the slope may be required, depending on the selected embankment fill type. Further discussion on these aspects is provided in Section 6.15.2.

6.13 Lateral Earth Pressures for Design of Culvert Walls

The lateral earth pressures acting on the culvert 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. Seismic (earthquake) loading must also be taken into account in the design.

The following recommendations are made concerning the design of the culvert walls:

- 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 culvert walls. Compaction (including type of equipment, target densities, etc.) should be carried out in accordance with OPSS.PROV 501 (*Compacting*) as amended by SP 105S22. Other aspects of the granular backfill requirements with respect to frost taper should be in accordance with OPSD 803.010 (*Backfill and Cover for Concrete Culverts*).
- 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. 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 (estimated vertical frost penetration depth as interpreted from OPSD 3090.101 *(Frost Penetration Depths*)) behind the back of the wall, per Figure C6.20(a) of the Commentary to the CHBDC (2014).

6.13.1 Static Lateral Earth Pressures for Design

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

- 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 behind the granular zone:

Material	**Earth Fill**	**OPPS.PROV1010 Granular 'B' Type I or Select Subgrade Method**
Soil Unit Weight:	20 kN/m^3	21
Coefficients of static lateral earth pressure: Active, K_a At rest, K_o	 0.33 0.50	 0.31 0.47

- If the culvert walls support allow for 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).
- As the culvert walls likely do not allow for 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.

6.13.2 Seismic Lateral Earth Pressures for Design

Seismic (earthquake) loading may also have been taken into account in the design of culvert 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 culvert 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 that 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 culvert wall is level. Where sloping backfill is present above / beyond 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**	0.043g	0.29	0.29	0.35
	975-Yr	0.080g	0.30	0.30	0.36
	2,475 Yr	0.156g	0.32	0.32	0.39
Non-Yielding Wall	**475-Yr**	0.043g	0.30	0.30	0.37
	975-Yr	0.080g	0.33	0.33	0.40
	2,475 Yr	0.156g	0.37	0.37	0.45

- 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 11, 20, and 39 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).

6.14 Analytical Testing for Construction Material

The results of an analytical test on four samples of native soil from boreholes advanced at Stavebank Creek Culvert, and two samples of native soil and one sample of shale bedrock from boreholes advanced at Kenollie Creek Culvert are summarized in Sections 4.2.1.12 and 4.2.2.11, respectively and the analytical laboratory test reports are presented in Appendix D. The potential for sulphate attack and corrosion are discussed in the following sub-sections. However, it is ultimately up to the designer to determine the appropriate construction materials, including the exposure class and ensuring that all aspects of CSA A23.1-14 Section 4.1.1 “Durability Requirements” are followed when designing concrete elements.

6.14.1 Potential for Sulphate Attack

The analytical test results were compared to CSA A23.1 14 Table 3 ("Additional requirements for concrete subjected to sulphate attack”) for the potential sulphate attack on concrete. The sulphate concentrations measured in all samples of the native soils and shale bedrock range from less than 0.002 per cent to about 0.06 per cent, which are below the exposure class of “S-3” (Moderate - 0.1 – 0.2 per cent; the sulphate concentrations are considered negligible according to the Gravity Pipe Design Guidelines Table 7.2 (MTO, 2014). Therefore, based on the samples tested, when the designer is selecting the exposure class for the structure, the effects of sulphates from within the native soil deposits and bedrock around the culvert may not need to be considered.

6.14.2 Potential for Corrosion

Based on the test results from the soil sample and bedrock core sample the pH ranges from about 7.0 to 7.8 and the resistivity ranges from 640 to 2,700 ohm-cm. According to the MTO Gravity Pipe Design Guidelines (2014), the pH is not considered detrimental to culvert durability. The resistivity is less than 2,000 ohm-cm except for one test result, which indicates that the soil corrosiveness is generally severe (R < 2,000 ohm-cm), as per Table 3.2 of the MTO Gravity Pipe Design Guidelines (2014). As the culverts will also be located under the roadway / highway shoulders and will be exposed to de-icing salt, concrete should be designed for a “C” type exposure class as defined by CSA A23.1-14 Table 1. All culverts should be designed with consideration given to Table 7.1 of the MTO Gravity Pipe Design Guidelines (2014).

6.15 Construction Considerations

6.15.1 Open Cut Excavations

The foundation excavations for construction of the box culverts or footings will extend through existing fill and into the underlying native soil and / potentially into the slightly weathered bedrock near the inlet of the Kenollie Creek Culvert. Groundwater was generally encountered above the proposed excavation depth within the fill and native soils encountered at the site. Where space permits, open-cut excavations into these materials must be carried out in accordance with the guidelines outlined in the Occupational Health and Safety Act (OHSA) for Construction Activities. The existing fill materials at both culvert sites are classified as Type 3 soil.

At the Stavebank Creek Culvert site the native silty sand to silt and sand is classified as Type 3 soil, while the gravelly clayey silt with sand till is classified as Type 2 soil above the water table and Type 3 soil below the water table.

At the Kenollie Creek Culvert site the clayey silt with sand is classified as a Type 3 soil, the gravelly sandy clayey silt residual soil and till deposits and the silty sand to sand deposits are classified as Type 2 soils above the water table and as Type 3 soils below the water table, according to the OHSA. Temporary excavations (i.e. those which are open for a relatively short time period) should be made with side slopes no steeper than 1 horizontal to 1 vertical (1H:1V).

Excavated material must be stockpiled at a distance away from the excavation equal to or greater than the depth of the open cut excavation.

A Notice to Contractor is included in Appendix E to warn the contractor about the potential for encountering bedrock at the inlet at Kenollie Creek Culvert.

6.15.2 Surficial Embankment Stability and Erosion Protection

The roadway highway embankments proposed Stavebank Creek Culvert and Kenollie Creek Culvert replacement sites is to be widened to accommodate the widened QEW and re-alignment of Premium Way and new Multi-Use Trail to the north of the existing alignment. It is understood that different fill materials are being considered for the embankment widening construction, including: OPSS.PROV 1010 (*Aggregates)* Granular ‘A’: Granular ‘B’ Type I or Type II; Select Subgrade Material (SSM); and compacted earth meeting the requirements of OPSS.PROV 212 (*Earth Borrow*).

Section 6.12 provides design recommendations for side slope geometry to achieve a factored Factor of Safety (FoS) that satisfies the requirements of the CHBDC (2014) with respect to global stability for temporary and permanent conditions. However, depending on the selected embankment fill material type, slope geometry, surface treatment and weather (i.e. precipitation, cycles of wetting-drying and/or freezing-thawing), surficial instability of the embankment side slopes may occur, which could include localized sloughing and erosion. As such, in order to maintain the integrity of the new embankments, erosion protection measures may be required depending on the fill type selected for construction.

The potential for erosion of embankment fill types can be estimated using the Wischmeier Nomograph (1978). The silt, sand and organic content of the fill material, as well as the soil structure and permeability, influence the erosion potential of a soil. The Wischmeier Nomograph generates a ‘K’ Factor between 0 and 1.0, which categorizes the erodibility of the soil. The higher the K value, the greater the erodibility; for example, highly erodible silty soils may have a K factor exceeding 0.6, while relatively non-erodible soils may have a K factor less than 0.2.

Based on the specified gradation, granular fill such as OPSS.PROV 1010 (*Aggregates)* Granular ‘A’, or Granular ‘B’ Type I or Type II, have a low potential for erosion. For embankments constructed of granular fill, erosion control can be limited to hydro-seeding and vegetation. On-going maintenance for embankments constructed of this material is not expected to be required.

The specification for OPSS.PROV 1010 *(Aggregates)* SSM allows for much more variation in the gradation of the material compared to Granular ‘A’, or Granular ‘B’ Type I or Type II, and therefore has the potential to be low - erodible to moderate - erodible. Erosion protection for slopes constructed of SSM should consist of erosion control blankets and hydro-seeding. Slopes constructed of SSM and properly protected from erosion should require limited on-going maintenance.

The specification for earth borrow as provided in OPSS.PROV 212 *(Earth Borrow)* allows for a wide variability of soil types with a wide range of gradations. As such, the potential for surficial instability and erosion of earth borrow material may range from low - to severe – erodibility depending on the soil type. Based on the potential range in gradations, and variability and uncertainty in soil types for embankments constructed of earth borrow, flattening of side slopes may be required and robust erosion protection such as the application of a minimum 300 mm thick layer of granular sheeting meeting the specification in OPSS.PROV 1004 *(Aggregates – Miscellaneous)* is recommended to be placed on the slopes. Even with appropriate erosion protection, on-going maintenance of embankment slopes constructed of earth borrow may be required depending on the side slope geometry as well as the final gradation and soil type of the earth borrow used for construction. In some cases, in particular for clayey earth borrow with intermediate to high plasticity, flatter side slopes than 2.75H:1V as specified for compacted earth follow in Section 6.12 will be necessary to maintain surficial stability.

6.15.3 Temporary Protection Systems

Based on information provided by MH, it is understood that temporary excavation support systems will be used to install the culverts and accommodate the traffic staging plan. Based on the General Arrangement drawings provided, temporary excavation support systems will be required along Premium Way, as well as along and across the existing QEW. The temporary excavation support systems should be designed and constructed in accordance with OPSS.PROV 539 (*Temporary Protection Systems*), as amended by SP 105S09. The lateral movement of the protection systems should meet Performance Level 2 as specified in OPSS.PROV 539, provided that any utilities, if present, can tolerate this magnitude of deformation.

It is anticipated that a driven interlocking sheet pile system may be constructable where temporary protection systems are required south of about Station 0+035 to the outlet at Kenollie Creek Culvert site and at the Stavebank Creek Culvert site; however, where SPT “N” values greater than about 50 blows per 0.3 m of penetration were encountered installation of driven sheet piles is expected to be more challenging and potentially unfeasible at these sites. North of Station 0+035 to the inlet at Kenollie Creek Culvert residual soil and shale bedrock were encountered at depths of about 2 m to 3 m below the culvert base. Cobbles and/or boulders are anticipated to be encountered within the till deposit, based on observations of drilling progress (auger grinding) in Boreholes NRW3-6, NRW7-3, K3, NW3-3 and S6 and recovered core samples of shale / limestone slabs, cobbles and/or boulders in Boreholes PED-03 and S7. For the protection system required along Premium Way and the north side of the existing QEW at Kenollie Creek Culvert and potentially for the entire length of protection system required for the Stavebank Creek Culvert site, the contractor may elect to use a soldier pile and lagging system. The advantages, disadvantages, relative costs and risks/consequences associated with different Temporary Protection System options are compared in Table 2 and the options are discussed in further detail in the sections below.

The sheet piles or soldier piles will need to extend/be socketed to a sufficient depth to provide the necessary passive resistance for the retained soil height, plus any surcharge loads behind the protection system. Lateral support to the sheet pile wall or soldier pile wall could be provided in the form of rakers or temporary anchors, if and as required.

While the selection and design of the temporary protection system will be the responsibility of the contractor, the following information is provided to MTO and its designers to aid in assessment of the approximate construction costs during detail design.

Soil Type	Unit Weight	Internal Angle of Friction	Undrained Shear Strength	Coefficient of Lateral Earth Pressure[1]		
	(γ, kN/m^3)	(ϕ, degrees)	(S_u, kPa)	Active K_a	At Rest K_o	Passive K_p [2]
Stavebank Creek Culvert Site						
Existing Gravelly Sand to Sand and Gravel (Fill) (Loose to Dense)	20	30	--	0.33	0.50	3.00
Existing Sandy Silt to Silty Sand to Silt and Sand (Fill) (Very Loose to Compact)	20	28	--	0.36	0.53	2.77
Existing Clayey Silt (Fill) (Soft to Very Stiff)	19	28	50	0.36	0.53	2.77
Silt and Sand to Silty Sand to Sand (Very Loose to Compact)	20	28	--	0.36	0.53	2.77
Sand and Gravel (Very Dense)	21	35	--	0.27	0.42	3.69
Clayey Silt with Sand (Till) (Firm to Hard)	20	34	150	0.29	0.46	3.39
Kenollie Creek Culvert Site						
Existing Gravelly Sand to Sand and Gravel (Fill) (Compact to Dense)	20	30	--	0.33	0.50	3.00
Existing Sandy Silt to Silty Sand (Fill) (Very Loose to Dense)	20	28	--	0.36	0.53	2.77
Clayey Silt with Sand (Firm to Very Stiff)	19	28	50	0.36	0.53	2.77
Silty Sand to Sand (Compact to Very Dense)	20	32	--	0.31	0.47	3.25
Clayey Silt with Sand Till (Very Stiff to Hard)	20	34	200	0.29	0.46	3.39
Clayey Silt (Residual Soil) (Hard)	20	34	200	0.28	0.44	3.54

Notes:

1. The earth pressure coefficients noted above are based on a horizontal surface adjacent to the excavation. If sloped surfaces are present, the coefficient of earth pressure should be adjusted accordingly.
2. The total passive resistance below the base of the excavation (i.e. adjacent to the temporary protection system) may be calculated based on the values of K_p indicated above but reduced by an appropriate factor that considers the allowable wall movement in accordance with Figure C6.16 of the CHBDC (2014) to account for the fact that a large strain would be required for mobilization of the full passive resistance.

It should be noted that the pressure distributions given above are the minimum for the ultimate stress condition; a stiffer design may be required than predicted by these distributions in order to maintain displacements within an acceptable range. In addition, the earth pressure coefficients provided above are based on a horizontal surface adjacent to the top of the excavation; if sloped surfaces are present, the coefficient of earth pressure should be adjusted accordingly.

For the temporary protection system at the Stavebank Creek Culvert site, a design groundwater level at approximately Elevations 90.0 m should be assumed based on the groundwater level measured in the standpipe piezometers installed in Boreholes S3 and PED-03A. For the temporary protection system at the Kenollie Creek Culvert site, a design groundwater level at approximately Elevation 91.1 m should be assumed based on the groundwater level measured in the standpipe piezometer installed in Borehole K2. If a soldier pile and lagging system is adopted, it would be necessary to control seepage or include measures to mitigate loss of soil particles through lagging boards.

Design of the temporary support systems should include an evaluation of base stability, soil squeezing stability and hydraulic uplift stability as defined in the CFEM (2006). Provided appropriate dewatering systems, possibly in combination with temporary cut-off walls, are utilized, the excavation bases should be stable with respect to soil strength considerations. Since the excavations are generally below the static groundwater level there is potential for basal instability to occur (e.g. ‘piping’, ‘quick conditions’ or ‘boiling’) should an unbalanced hydrostatic head result in large upward seepage gradients at the base of the excavation. The hydrostatic head should be drawn down to at least 1 m below the base of the excavation to prevent the occurrence of base heave, discussed further in Section 6.15.4.

Consideration should be given to either partial or full removal of the protection system upon completion of construction. Where possible, full removal of the protection system should be considered to mitigate potential impediments to future rehabilitation/reconstruction work. An NSSP is included in Appendix E which addressed the removal or cut off of the protection system.

6.15.4 Groundwater Control

At the Stavebank Creek Culvert site, the groundwater level measured in the standpipe piezometer installed in the fill and till deposits in Boreholes S3 and PED-03A is at about Elevation 90 m which is about 0.4 m to 3.4 m above the proposed base of the excavations, depending on the selected culvert foundation option (i.e., open footing or box culvert). The excavations at the Stavebank Creek Culvert site will extend through non-cohesive fill, silty sand to silt and sand deposits and into the clayey silt till deposit. Samples of the fill and silty sand to silt and sand deposits were generally characterized as moist to wet and unstabilized groundwater levels (measured in open boreholes after drilling) were generally encountered within the fill and/or silty sand to silt and sand deposits, above the till deposit. Given the relatively high permeability of the non-cohesive fill material and the native silty sand to silt and sand deposit, it is likely that pumping from filtered sumps placed at the base of the excavation will not be sufficient to handle the groundwater inflows from these layers into the excavation.

At the Kenollie Creek Culvert site, the groundwater level measured in the standpipe piezometer installed in the fill and till deposits and into the shale bedrock in Borehole K2 is about Elevation 91.1 m, which is about 2.7 m to 2.9 m above the proposed base of the excavation for the box culvert. The excavations at the Kenollie Creek Culvert site will extend through the non-cohesive fill and into residual soil, and through / into clayey silt till along the north portion of the culvert and through non-cohesive fill, silty sand to sand and clayey silt along the southern portion of the culvert. Samples of the fill were generally characterized as moist to wet and, given the relatively high permeability of the fill material, it is likely that pumping from filtered sumps placed at the base of the excavation also will not be sufficient to handle the groundwater inflows from the layer into the excavation. In addition, under the existing QEW at the Kenollie Creek Culvert site, a low permeability clayey silt layer was encountered overlying a wet silty sand to sand deposit. When the silty sand to sand deposit was penetrated during the drilling operations, there was “blow back” into the casing, indicating the groundwater within the silty sand to sand deposit is under pressure, which could cause basal instability of the excavation for the box culvert at this location.

The groundwater within the silty sand to sand deposit must be lowered to a minimum of 1 m below the base of the excavation for the Stavebank Creek Culvert and Kenollie Creek Culvert prior to any excavation for the culvert. If the groundwater level is not lowered to below the base of the excavation prior to the excavation being made to that level there is the potential for basal instability of the clayey silt with sand deposit. It is recommended that an NSSP be included in the Contract Documents to warn the Contractor of the water-bearing soil conditions under pressure and the requirement for dewatering prior to any excavation for the Kenollie Creek Culvert, an example is included in Appendix E.

Further, where granular fill or native non-cohesive / granular soils overlay less permeable materials (e.g., clayey silt till), such is the case at the Kenollie Creek Culvert site, groundwater will tend to accumulate at this interface as a perched condition and will be difficult to remove with conventional dewatering systems and/or sumps and pumps. Use of widely-spaced sump pits and pumps, shallow drilled wells with submersible pumps or drainage that relies on gravity flow of water may not be adequate to lower groundwater below subgrade / sub-excavation levels. Therefore, we recommend that active dewatering be carried out in advance of excavation using systems such as closely-spaced well points or eductors / vacuum well points to depressurize / actively draw down the groundwater level to approximately 1 m below the base of the excavation along the culvert alignment at both culvert sites. Dewatering should be designed / carried out in accordance with OPSS.PROV 517 (*Dewatering*), as amended by MTO Special Provision SP 517F01, a copy of which is included in Appendix E.

Based on hydrogeological data provided by MH we understand that the hydraulic conductivity of the silt and sand deposit at both Stavebank Creek Culvert and Kenollie Creek Culvert is about 4.9×10^{-3} cm/s and that the maximum drawdown of about 3.8 m will be required to lower the groundwater level to 1 m below the base of the excavation at both sites. As a result of drawdown, settlement will occur due to the change in effective stress conditions in response to lowering of the groundwater level estimated to be in the range of between 10 mm and 15 mm at the dewatering source / extraction point (i.e., maximum drawdown location). The settlements are expected to decrease away from the extraction point to less than 5 mm at a distance of about 25 m from the dewatering point. The dewatering system developed by the contractor must be designed and constructed in a way to avoid loss of soil particles, as this loss of material could lead to further settlements, to magnitudes greater then the estimated settlements due only to the change in the effective stress of the soil. It is noted that a TransNorthern Pipeline Inc. pipeline is located along the north side of Premium Way near the inlet at Stavebank Creek Culvert, therefore an assessment should be carried out to determine if the pipeline and any other utilities that are located in the area can tolerate this magnitude of settlement. If there are utilities or structures within the area where settlement is expected, consideration should be given to installing settlement monitoring points above the utilities and installing settlement rods adjacent to structures. A pre- and post-construction condition survey of any structures within the area where settlement is expected should be carried out prior to construction.

Consideration should be given to providing a groundwater cut-off system in conjunction with the temporary protection systems, such as sheet pilling, installed to an appropriate tip depth to cut off and reduce groundwater flow through the sides of the excavation and reduce the risk of basal instability.

Construction 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. The construction water taking permit and registration should be prepared adequately in advance of site excavation works so as not to unduly affect the construction schedule.

Surface water should be directed away from open excavation areas to prevent ponding of water that could result in disturbance and weakening of the subgrade.

6.15.5 Obstructions During Installation of Temporary Protection Systems

It is anticipated that cobble and/or boulder size materials may be encountered within the till deposits at both the Stavebank Creek and Kenollie Creek Culvert sites. Also, at the Kenollie Creek Culvert site cobbles and/or boulders may also be encountered within the silty sand deposit. Along with the potential presence of cobbles and/or boulders, debris within the fill material, such as concrete and asphalt fragments and wood chips, were encountered during the geotechnical investigation. The presence of these obstructions may affect the installation of protection system elements. It is recommended that an NSSP be included in the Contract Documents to warn the Contractor of the possible presence of cobbles and/or boulders within the overburden soils; an NSSP is provided in Appendix E.

6.15.6 Vibration Monitoring During Temporary Protection System Installation

If the temporary protection systems are installed using vibratory methods, significant vibrations are not anticipated, given the generally very stiff to hard / dense nature of the native soil deposits. Residential/commercial buildings are present in the vicinity of the site, at distances of approximately 50 m and 60 m from the replacement culvert locations at Stavebank Creek and Kenollie Creek, respectively. A lower PPV threshold of 25 mm/s is generally considered applicable for buildings. While it is expected that vibration levels will not reach these thresholds at the structures, MTO has requested pre- and post-construction condition surveys and vibration monitoring at or near the buildings, to defend against potential damage claims associated with vibration-inducing activities at similar sites. A sample NSSP is provided in Appendix E, to address vibration monitoring condition surveys at residences located within 100 m of the culverts construction operations both along the north side and south side of the QEW.

6.15.7 Subgrade Protection

The subgrade soils at the base level of the excavations for the Stavebank Creek Culvert and Kenollie Creek Culvert will be susceptible to disturbance from construction traffic and/or ponded water. To limit this degradation, it is recommended that the subgrade at the Culvert sites be protected within four hours of preparation, inspection, and approval of the subgrade for the box culvert. As discussed in Section 6.5.1, subgrade protection for box culverts could be provided by granular bedding or a concrete working slab. This requirement can be addressed with a note on the General Arrangement drawing and/or with an NSSP, such as the sample NSSP for the working slab included in Appendix E.

7.0 CLOSURE

This Foundation Design Report was prepared by Mr. David Marmor, E.I.T. a geotechnical engineering intern with Golder and the technical aspects were reviewed by Ms. Sandra McGaghran, M.Eng., P.Eng., an Associate and Senior Geotechnical Engineer with Golder. Mr. Jorge Costa, P.Eng., Senior Consultant and MTO Foundations Designated Contact for Golder, conducted an independent technical and quality control review of the report.

Golder Associates Ltd.

LICENSED PROFESSIONAL ENGINEER
S.M. McGAGHRAN
May 8/19
PROVINCE OF ONTARIO

David Marmor

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Geotechnical Engineering Intern

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May 8/2019
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DPM/SMM/JMAC/rb

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https://golderassociates.sharepoint.com/sites/11176g/shared documents/07-reporting/foundations/11 - culverts/3 - final/1662333 fidr stavebank kenollie culverts 2019may8.docx

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Ladd, C.C, Foott, R., Ishihara, K., Schlosser, F., and Poulos, H.G. 1977. Stress deformation and strength characteristics. Proceedings of the 9th International Conference on Soil Mechanics and Foundation Engineering, Tokyo, Vol. 2, pp. 421494.

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Wischmeier, W.H., and Smith, D.D. 1978. Predicting Rainfall Erosion Loses – A Guide to Conservation Planning. U.S. Department of Agriculture, Agriculture Handbook No. 537.

ASTM International:

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

Commercial Software:

Slide (Version 8) by Rocscience Inc.

Settle 3D (Version 4) Rocscience Inc.

Ontario Provisional Standard Drawing:

OPSD 208.010	Benching of Earth Slopes
OPSD 810.010	Rip-Rap Treatment for Sewer and Culvert Outlets
OPSD 803.010	Backfill and Cover for Concrete Culverts
OPSD 1101.012	Precast Concrete Valve Chamber with Poured-in-Place Thrust Blocks, 1800 x 2400 mm Components

OPSD 3090.101	Foundation Frost Penetration Depths for Southern Ontario

Ontario Provincial Standard Specification:

OPSS.PROV 206	Construction Specification for Grading
OPSS.PROV 212	Construction Specification for Earth Borrow
OPSS 422	Construction Specification for Precast Reinforced Concrete Box Culverts and Box Sewers in Open Cut
OPSS.PROV 501	Construction Specifications for Compacting
OPSS.PROV 517	Construction Specification for Dewatering
OPSS.PROV 539	Construction Specification for Temporary Protection Systems
OPSS 802	Construction Specification for Topsoil
OPSS 803	Construction Specification for Sodding
OPSS.PROV 804	Construction Specification for Seed and Cover
OPSS 902	Construction Specification for Excavating and Backfilling Structures
OPSS.PROV 1002	Material Specifications for Aggregates – Concrete
OPSS.PROV 1004	Material Specification for Aggregates – Miscellaneous
OPSS.PROV 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material
OPSS.PROV 1205	Material Specifications for Clay Seal
SP 105S22	Special Provision – Amendment to OPSS 501, June 2016
SP 105S09	Special Provision – Amendment to OPSS 539, November 2014
SP 109S12	Special Provision – Amendment to OPSS 902, August 2018
SP 517F01	Special Provision - Amendment to OPSS 517, July 2017

Ontario Water Resources Act:

Ontario Regulation 903	Wells (as amended)

Ontario Occupational Health and Safety Act:

Ontario Regulation 213/91	Construction Projects (as amended)

Ministry of Transportation, Ontario

Highway Drainage Design Standards, January 2008

Table 1: Comparison of Foundation Alternatives for Stavebank Creek Culvert and Kenollie Creek Culvert

Options	Advantages	Disadvantages	Relative Costs	Risks / Consequences
Box Culvert Replacement	▪ Minimizes depth of excavation, excavation support and dewatering requirements compared to open footing option. ▪ Pre-cast box sections expected to allow faster construction than cast-in-place open footings, with shorter duration for dewatering and surface water pumping / diversion. ▪ Can better accommodate differential settlement caused by the placement of new embankment fill.	▪ Will require excavation below the water table and dewatering.	▪ Less overall cost relative to open footing culvert replacement because shorter period of excavation, support and dewatering systems are required for culvert installation.	▪ May not satisfy specific fisheries requirements related to natural channel substrate, if applicable.
Open Footing Culvert Replacement	▪ Would satisfy fisheries requirements related to natural channel substrate, if applicable. ▪ May be feasible to build culvert replacements on pre-cast footing sections, to accelerate construction schedule and reduce time for dewatering and surface water pumping.	▪ Excavation depths are greater than for box culvert option to found footings at/below depth of frost penetration, resulting in increased excavation support and dewatering requirements. ▪ Cast-in-place footings may require a longer duration for construction, including dewatering and surface water pumping, as compared with pre-cast culvert segments or footing elements. ▪ Less accommodation of differential settlement, although estimate of settlement is within tolerable limits.	▪ Greater overall cost relative to box culvert replacement because deeper excavations are required which will also result in additional time period of temporary support systems and dewatering system operation.	▪ Longer construction time and deeper excavations introduce greater risk to the installation of the culvert replacement. ▪ Excavations and support systems will have to penetrate deeper into hard till, residual soil, and/or bedrock. ▪ Groundwater levels will have to be lowered to greater depths.

Table 2: Comparison of Temporary Protection System Options

Options	Advantages	Disadvantages	Relative Costs	Risks / Consequences
Soldier Pile and Lagging	▪ Better able to penetrate cobbles, boulders or other potential obstructions, and better able to penetrate denser soils where present. ▪ Relatively straightforward construction.	▪ May require pre-drilling through cobble nests, boulders or other obstructions as encountered at the site. ▪ May require socket penetration in boulders, limestone slabs and into weak shale bedrock with strong limestone interlayers. ▪ Longer installation time compared to installation of sheet piles. ▪ Additional measures required to control groundwater / surface water seepage through lagging boards to avoid ground loss.	▪ Higher cost compared to sheet piles walls, especially if obstructions are encountered.	▪ Low risk that equipment won’t penetrate obstructions in order to achieve required depth. ▪ Risk of soil loss behind lagging if seepage not adequately / properly controlled.
Sheet Pile Wall	▪ Relatively straight forward installation provided that obstructions are not encountered. ▪ Easier to remove compared to soldier pile and lagging. ▪ Can also provide for groundwater seepage control.	▪ Cannot penetrate hard till, cobbles and boulders, or into bedrock. ▪ May not be feasible to install in denser soils, such as those present along the north portion of Stavebank Creek Culvert and Kenollie Creek Culvert sites.	▪ Typically less expensive than soldier pile and lagging.	▪ Risk of sheet piles encountering obstructions and not achieving required depth.

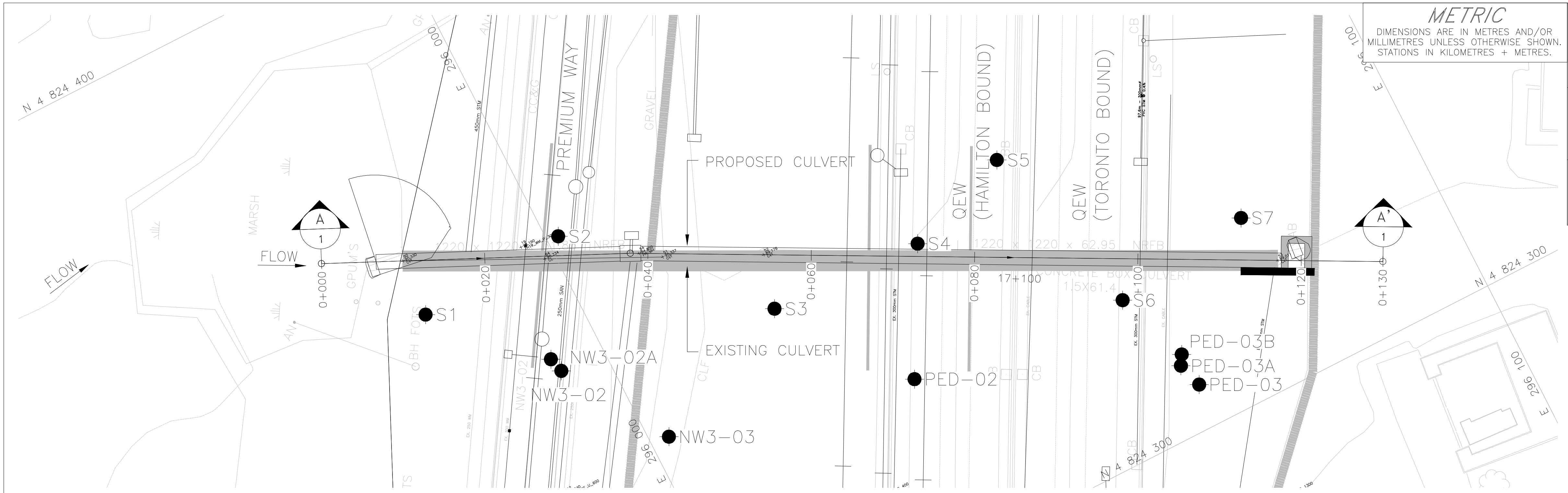
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No. 2019–2016
GWP No. 2002–13–00

SHEET

QEW WIDENING–MISSISSAUGA RD TO HURONTARIO ST
STAVEBANK CREEK CULVERT
BOREHOLE LOCATIONS
AND SOIL STRATA

GOLDER



PLAN

SCALE
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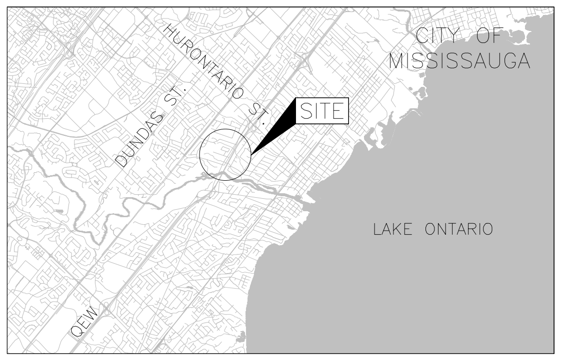


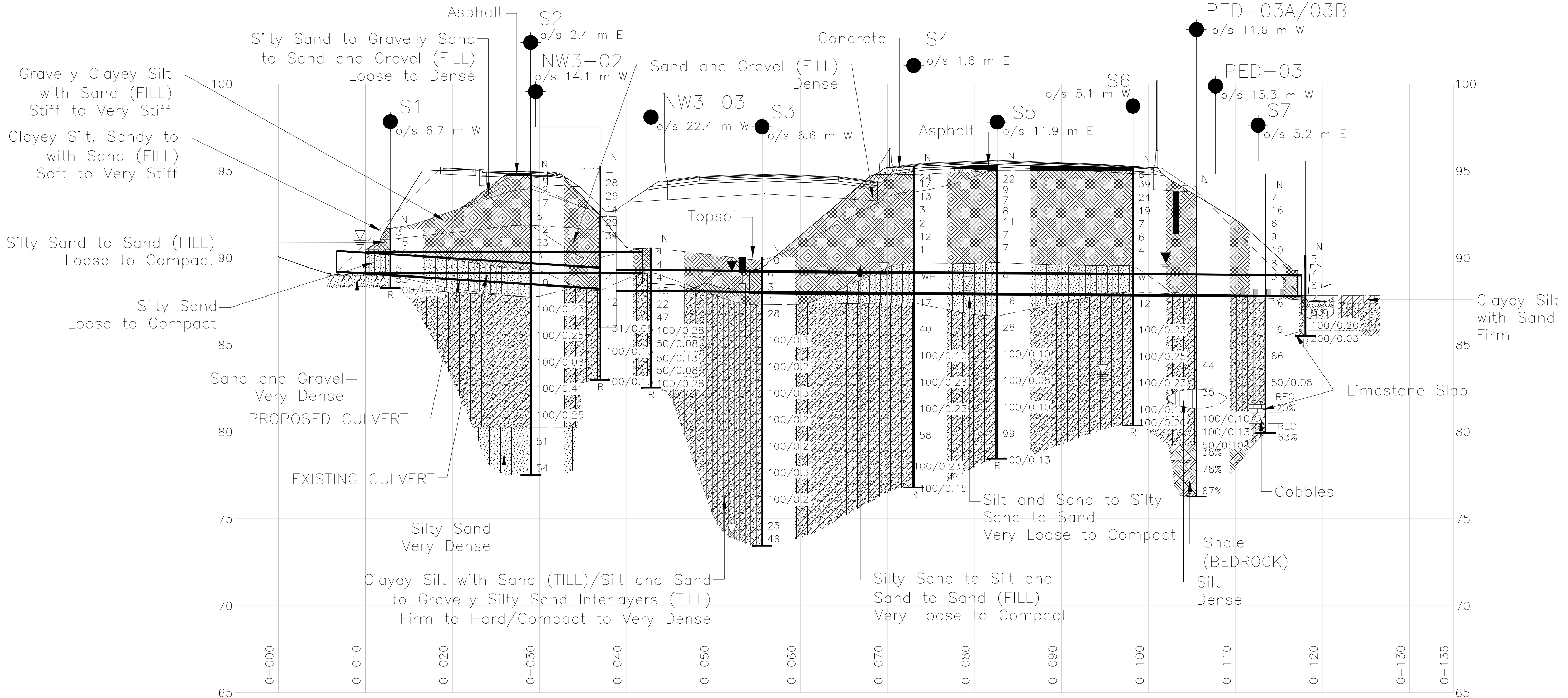
KEY PLAN
SCALE
2 0 2 4 km

LEGEND

●	Borehole – Current Investigation
▮	Seal
▯	Piezometer
N	Standard Penetration Test Value
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
100%	Rock Quality Designation (RQD)
▼	WL in piezometer, measured on NOV 28, 2017 or NOV 6, 2018
▽	WL upon completion of drilling
R	Split–spoon Refusal
REC/%	Recovery

BOREHOLE CO–ORDINATES (MTM NAD 83 ZONE 10)

No.	ELEVATION	NORTHING	EASTING
S7	90.1	4824321.2	296076.9
S6	95.2	4824318.8	296059.4
S5	95.3	4824341.1	296053.5
S4	95.2	4824336.4	296040.2
S3	90.0	4824337.3	296021.0
S2	94.9	4824357.2	296001.4
S1	91.7	4824356.1	295982.6
PED–03B	94.1	4824309.6	296062.8
PED–03A	94.1	4824308.4	296062.1
PED–03	93.7	4824305.3	296063.0
PED–02	95.2	4824321.8	296032.3
NW3–03	90.6	4824329.2	296002.3
NW3–02A	95.3	4824344.2	295993.8
NW3–02	95.3	4824342.4	295994.3



A–A PROFILE
1

SCALE HORIZONTAL
6 0 6 12 m

SCALE VERTICAL
6 0 6 12 m

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.
Culverts plan provided in digital format by Morrison Hershfield, drawing file no. Culverts–and–Protection.dwg, received August 28, 2018,
General arrangement plan provided by Morrison Hershfield, drawing file 11609340 – QEW Culvert – C3D 2017.dwg, received January 11, 2019.

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

S. McGaghran
Feb. 15, 2019

NO.	DATE	BY	REVISION

Geocres No. 30M 12–441

HWY. QEW	PROJECT NO. 1662333		DIST. CENTRAL
SUBM'D. ACM	CHKD. DM	DATE: 04/22/2019	SITE:
DRAWN: DD	CHKD. SMM	APPD. JMAC	DWG. 1

MINISTRY OF TRANSPORTATION, ONTARIO

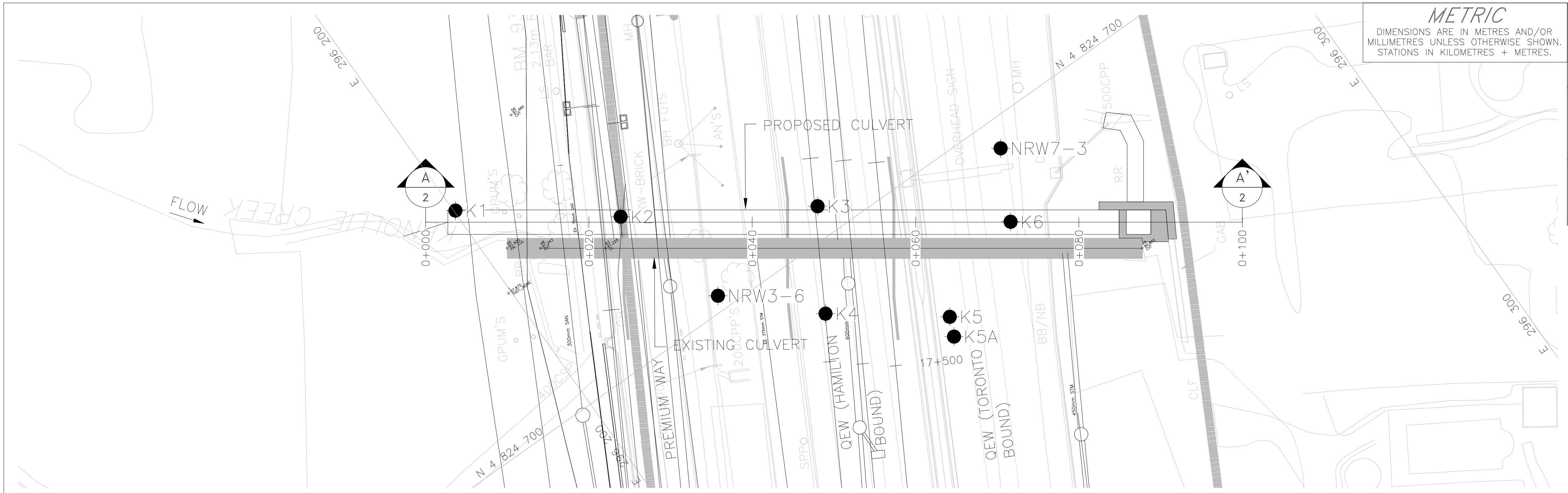
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No. 2019–2016
GWP No. 2002–13–00

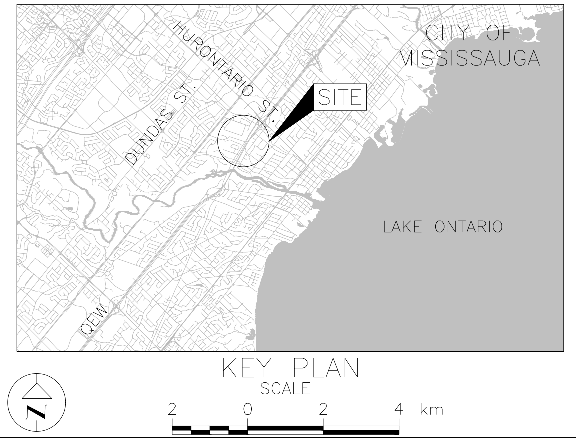
SHEET

QEW WIDENING MISSISSAUGA RD TO HURONTARIO ST
KENOLLIE CREEK CULVERT
BOREHOLE LOCATIONS
AND SOIL STRATA

GOLDER



PLAN
SCALE
6 0 6 12 m



KEY PLAN
SCALE
2 0 2 4 km

LEGEND

●	Borehole – Current Investigation
▮	Seal
▯	Piezometer
N	Standard Penetration Test Value
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
100%	Rock Quality Designation (RQD)
▼	WL in piezometer, measured on DEC 17, 2018
▽	WL upon completion of drilling
R	Split–Spoon Refusal

BOREHOLE CO–ORDINATES (MTM NAD 83 ZONE 10)			
No.	ELEVATION	NORTHING	EASTING
K1	90.1	4824728.9	296200.2
K2	93.2	4824716.6	296216.3
K3	95.0	4824703.7	296236.7
K4	95.0	4824692.4	296229.9
K5	95.0	4824683.3	296242.1
K5A	95.0	4824681.0	296241.1
K6	94.9	4824688.5	296254.9
NRW3–6	92.9	4824701.8	296220.4
NRW7–3	94.9	4824696.6	296259.1

NOTES

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

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

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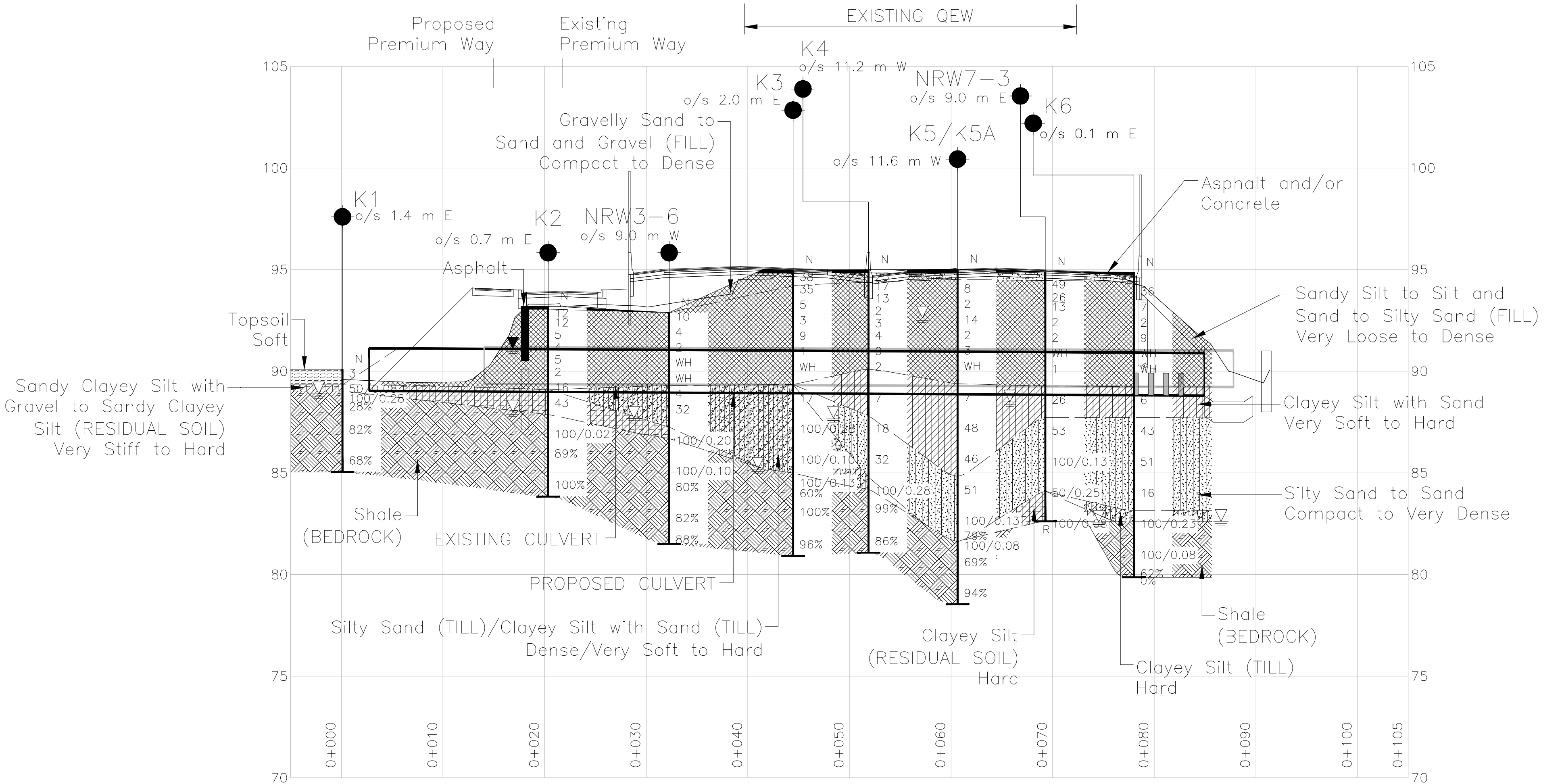
Base plans provided in digital format by Morrison Hershfield, drawing file no. X11609340Base.dwg, received April 12, 2018.
Culverts plan provided in digital format by Morrison Hershfield, drawing file no. Culverts–and–Protection.dwg, received August 28, 2018,
General arrangement plan provided by Morrison Hershfield, drawing file 11609340 – QEW Culvert – C3D 2017.dwg, received January 11, 2019.



A–A PROFILE
2

SCALE HORIZONTAL
6 0 6 12 m

SCALE VERTICAL
6 0 6 12 m

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

S. McGaghran
Feb. 15, 2019

NO.	DATE	BY	REVISION

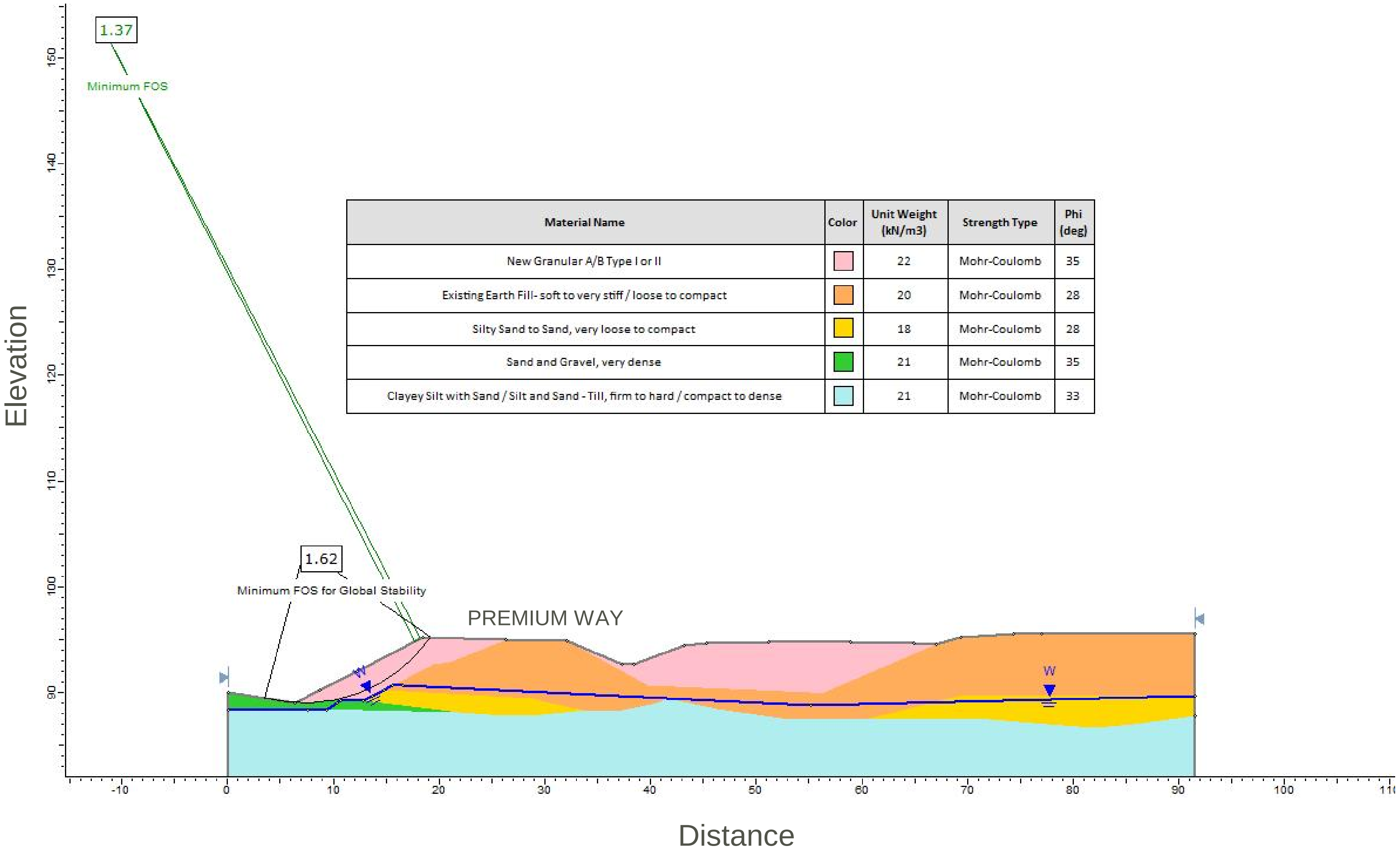
Geocres No. 30M 12–441

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

PLOT DATE: May 6, 2019
FILENAME: S:\Clients\MTO\QEW–Credit_River\99_PROJ\1662333_MH_P&F\40_PROD\0019_Kenollie_Culvert\1662333–0019–BG–0001.dwg

Static Global Stability Analysis – Embankment at Stavebank Creek Culvert

Figure 1



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Phi (deg)
New Granular A/B Type I or II	🟥	22	Mohr-Coulomb	35
Existing Earth Fill - soft to very stiff / loose to compact	🟧	20	Mohr-Coulomb	28
Silty Sand to Sand, very loose to compact	🟨	18	Mohr-Coulomb	28
Sand and Gravel, very dense	🟩	21	Mohr-Coulomb	35
Clayey Silt with Sand / Silt and Sand - Till, firm to hard / compact to dense	🟦	21	Mohr-Coulomb	33

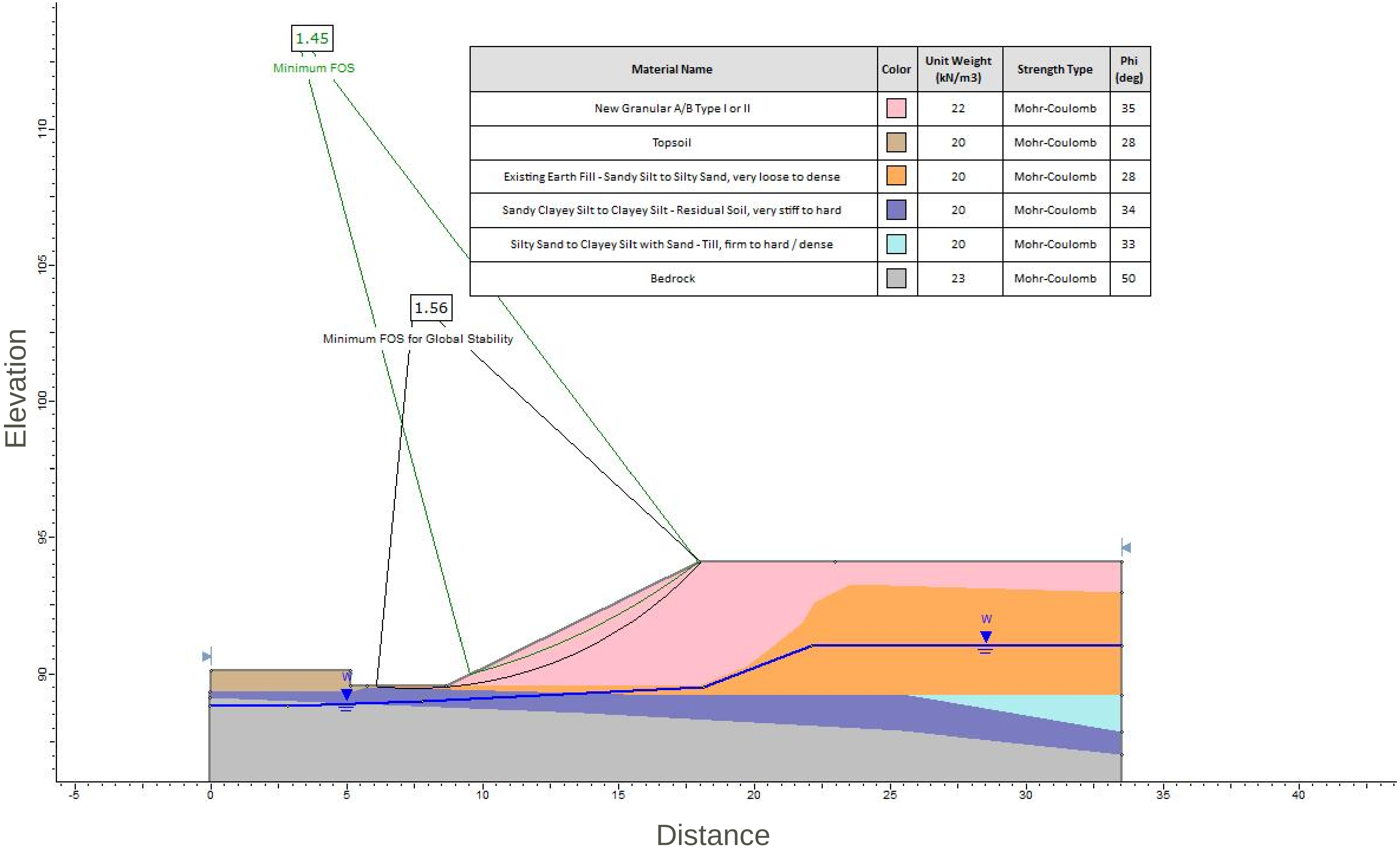
GOLDER

Date: May 2019
Project No.: 1662333

Analysis by: KNN
Checked by: SMM

Static Global Stability Analysis – Embankment at Kenollie Creek Culvert

Figure 2



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Phi (deg)
New Granular A/B Type I or II	🟥	22	Mohr-Coulomb	35
Topsoil	🟫	20	Mohr-Coulomb	28
Existing Earth Fill - Sandy Silt to Silty Sand, very loose to dense	🟧	20	Mohr-Coulomb	28
Sandy Clayey Silt to Clayey Silt - Residual Soil, very stiff to hard	🟪	20	Mohr-Coulomb	34
Silty Sand to Clayey Silt with Sand - Till, firm to hard / dense	🟦	20	Mohr-Coulomb	33
Bedrock	⬜	23	Mohr-Coulomb	50

GOLDER

Date: May 2019
Project No.: 1662333

Analysis by: KNN
Checked by: SMM

PROJECT

Stavebank Creek Culvert and Kenollie Creek Culvert Replacements, QEW Widening from West of Mississauga Road to West of Hurontario Street

TITLE

Looking East toward Borehole S1 and Inlet of Proposed Stavebank Culvert

GOLDER

PROJECT No. 1662333			FILE No. ----	
DRAFT	DPM	08/04/19	SCALE AS SHOWN	VER. 1.
CADD	--		**Photograph 1**	
CHECK	SMM			
REVIEW	JMAC			

CREATED: April 8, 2019 8   BY: DPM   Project: 1662333

PROJECT

Stavebank Creek Culvert and Kenollie Creek Culvert Replacements, QEW Widening from West of Mississauga Road to West of Hurontario Street

TITLE

Looking West at Borehole S2 on Stavebank Road

GOLDER

PROJECT No. 1662333			FILE No. ----		
DRAFT	DPM	08/04/19	SCALE	AS SHOWN	VER. 1.
CADD	--		**Photograph 2**		
CHECK	SMM				
REVIEW	JMAC				

CREATED: April 8, 2019 8 BY: DPM Project: 1662333

PROJECT

Stavebank Creek Culvert and Kenollie Creek Culvert Replacements, QEW Widening from West of Mississauga Road to West of Hurontario Street

TITLE

Looking East towards Borehole S3 between Stavebank Road and the QEW

GOLDER

PROJECT No. 1662333			FILE No. ----	
DRAFT	DPM	08/04/19	SCALE AS SHOWN	VER. 1.
CADD	--		**Photograph 3**	
CHECK	SMM			
REVIEW	JMAC			

CREATED: April 8, 2019 8 BY: DPM Project: 1662333

PROJECT
Stavebank Creek Culvert and Kenollie Creek Culvert Replacements, QEW Widening from West of Mississauga Road to West of Hurontario Street

TITLE
Looking West towards Borehole S4, Located in QEW Erie-Bound Lanes

GOLDER

PROJECT No. 1662333			FILE No. ----	
DRAFT	DPM	08/04/19	SCALE AS SHOWN	VER. 1.
CADD	--		**Photograph 4**	
CHECK	SMM			
REVIEW	JMAC			

CREATED: April 8, 2019 8 BY: DPM Project: 1662333

PROJECT

Stavebank Creek Culvert and Kenollie Creek Culvert Replacements, QEW Widening from West of Mississauga Road to West of Hurontario Street

TITLE

Looking South-East towards Borehole S7 at Outlet of Proposed Stavebank Culvert

GOLDER

PROJECT No. 1662333			FILE No. ----		
DRAFT	DPM	08/04/19	SCALE	AS SHOWN	VER. 1.
CADD	--		**Photograph 5**		
CHECK	SMM				
REVIEW	JMAC				

CREATED: April 8, 2019 8 BY: DPM Project: 1662333

PROJECT

Stavebank Creek Culvert and Kenollie Creek Culvert Replacements, QEW Widening from West of Mississauga Road to West of Hurontario Street

TITLE

Looking West towards Borehole K1 at Inlet of Proposed Kenolie Culvert

GOLDER

PROJECT No. 1662333			FILE No. ----	
DRAFT	DPM	08/04/19	SCALE AS SHOWN	VER. 1.
CADD	--		**Photograph 6**	
CHECK	SMM			
REVIEW	JMAC			

CREATED: April 8, 2019 8 BY: DPM Project: 1662333

CREATED: April 8, 2019 8 BY: DPM Project: 1662333

PROJECT

Stavebank Creek Culvert and Kenollie Creek Culvert Replacements, QEW Widening from West of Mississauga Road to West of Hurontario Street

TITLE

Looking East towards Borehole K2 from Premium Way

GOLDER

PROJECT No. 1662333			FILE No. ----		
DRAFT	DPM	08/04/19	SCALE	AS SHOWN	VER. 1.
CADD	--		**Photograph 7**		
CHECK	SMM				
REVIEW	JMAC				

PROJECT

Stavebank Creek Culvert and Kenollie Creek Culvert Replacements, QEW Widening from West of Mississauga Road to West of Hurontario Street

TITLE

Looking South towards Borehole K3 from QEW Erie-Bound Lanes

GOLDER

PROJECT No. 1662333			FILE No. ----	
DRAFT	DPM	08/04/19	SCALE AS SHOWN	VER. 1.
CADD	--		**Photograph 8**	
CHECK	SMM			
REVIEW	JMAC			

APPENDIX A

Record of Borehole and Drillhole Sheets, Bedrock Core Photographs and Geotechnical Laboratory Results for Stavebank Creek Culvert

LIST OF SYMBOLS

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

I. GENERAL

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

II. STRESS AND STRAIN

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

III. SOIL PROPERTIES

(a) Index Properties

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

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

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(d) Shear Strength

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

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

LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

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

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

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

Dynamic Cone Penetration Resistance; N_d:

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

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

Piezo-Cone Penetration Test (CPT)

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

III. SOIL DESCRIPTION

(a) Non-Cohesive (Cohesionless) Soils

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

(b) Cohesive Soils

Consistency

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

IV. SOIL TESTS

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

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

V. MINOR SOIL CONSTITUENTS

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

WEATHERINGS STATE

Fresh: no visible sign of weathering

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

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

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

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

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

BEDDING THICKNESS

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

JOINT OR FOLIATION SPACING

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

GRAIN SIZE

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

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

CORE CONDITION

Total Core Recovery (TCR)

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

Solid Core Recovery (SCR)

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

Rock Quality Designation (RQD)

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

DISCONTINUITY DATA

Fracture Index

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

Dip with Respect to Core Axis

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

Description and Notes

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

Abbreviations

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

FIELD ESTIMATION OF ROCK HARDNESS

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

Notes:

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

Reference:

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

Hoek, E., Kaiser, P.K., Bawden, W.F., 1995. "Support of Underground Excavations in Hard Rock", Balkema, Rotterdam.

ROCK WEATHERING CLASSIFICATION

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

Reference:

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

RECORD OF BOREHOLE No S1

SHEET 1 OF 1

METRIC

PROJECT 1662333

G.W.P. 2002-13-00

LOCATION N 4824356.1; E 295982.6 MTM NAD 83 ZONE 10 (LAT. 43.559090; LONG. -79.609144)

ORIGINATED BY ACM

DIST Central HWY QEW

BOREHOLE TYPE Portable Tripod - NW Casing and Wash Boring

COMPILED BY SK

DATUM Geodetic

DATE December 19, 2018

CHECKED BY SMM

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

SOIL PROFILE ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES "N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION / SHEAR STRENGTH	WATER CONTENT (%)	UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
91.7	GROUND SURFACE													
0.0	Clayey silt with sand (FILL) Soft Brown Moist		1	SS	3									
91.1 0.6	Silty sand, some gravel (FILL) Compact Brown Moist		2	SS	15	▽	91		○					
90.5 1.2	Silty SAND, some gravel, trace clay Loose to compact Grey-brown Wet		3	SS	10		90		○		18	53	25	4
			4	SS	5									
89.0 2.7	SAND and GRAVEL, trace to some silt, trace clay Very dense Grey Wet		5	SS	53		89		○		35	57	6	2
			6	SS	100/0.05									
88.3 3.4	END OF BOREHOLE													

NOTES:

1. Water level measured at a depth of about 0.7 m below ground surface (Elev. 91.0 m) upon completion of drilling.

2. Flowing sands encountered between a depth of about 2.4 m to 3.4 m.

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

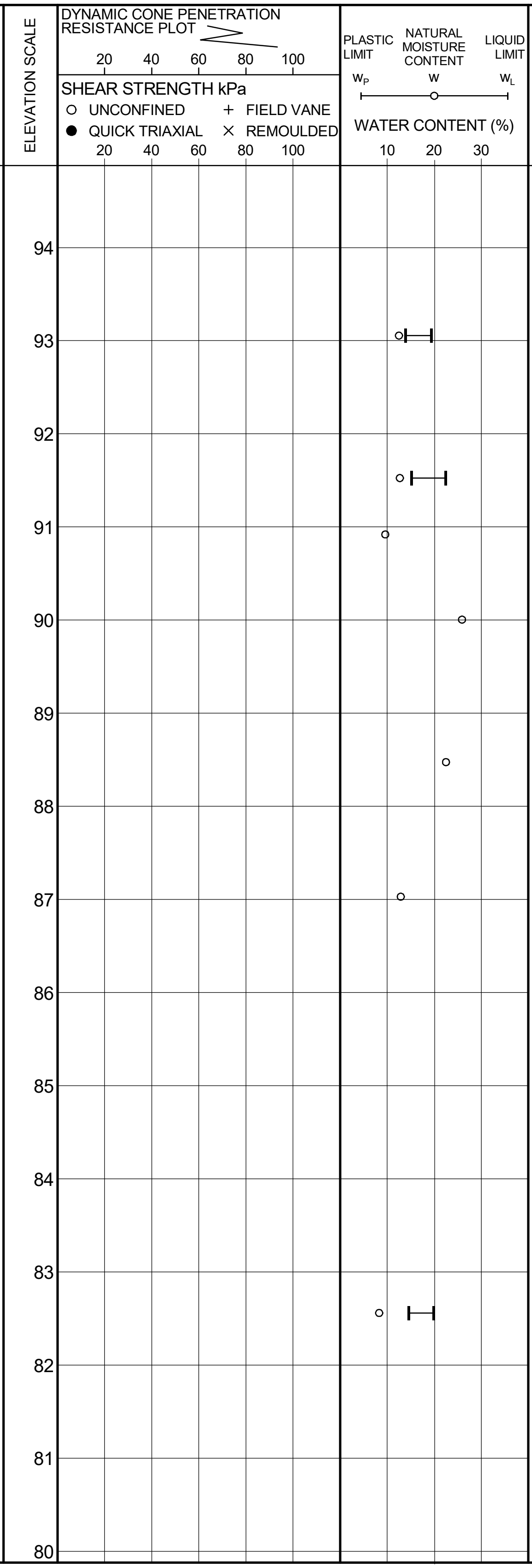
RECORD OF BOREHOLE No S2

SHEET 1 OF 2

METRIC

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

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



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

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

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RECORD OF BOREHOLE No S2

SHEET 2 OF 2

METRIC

PROJECT 1662333

G.W.P. 2002-13-00

LOCATION N 4824357.2; E 296001.4 MTM NAD 83 ZONE 10 (LAT. 43.559092; LONG. -79.608907)

ORIGINATED BY ACM

DIST Central HWY QEW

BOREHOLE TYPE CME 75, 210 mm O.D. Hollow Stem Augers

COMPILED BY JMP

DATUM Geodetic

DATE September 13, 2018

CHECKED BY SMM

SOIL PROFILE			SAMPLES					DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE				GR	SA	SI	CL
	--- *CONTINUED FROM PREVIOUS PAGE* ---													
	Silty SAND, some gravel, trace to some clay Very dense Grey Moist		14	SS	51									
							79							
							78							
77.5			15	SS	54				○		13	55	25	7
17.4	END OF BOREHOLE NOTE: 1. Borehole dry upon completion of drilling.													

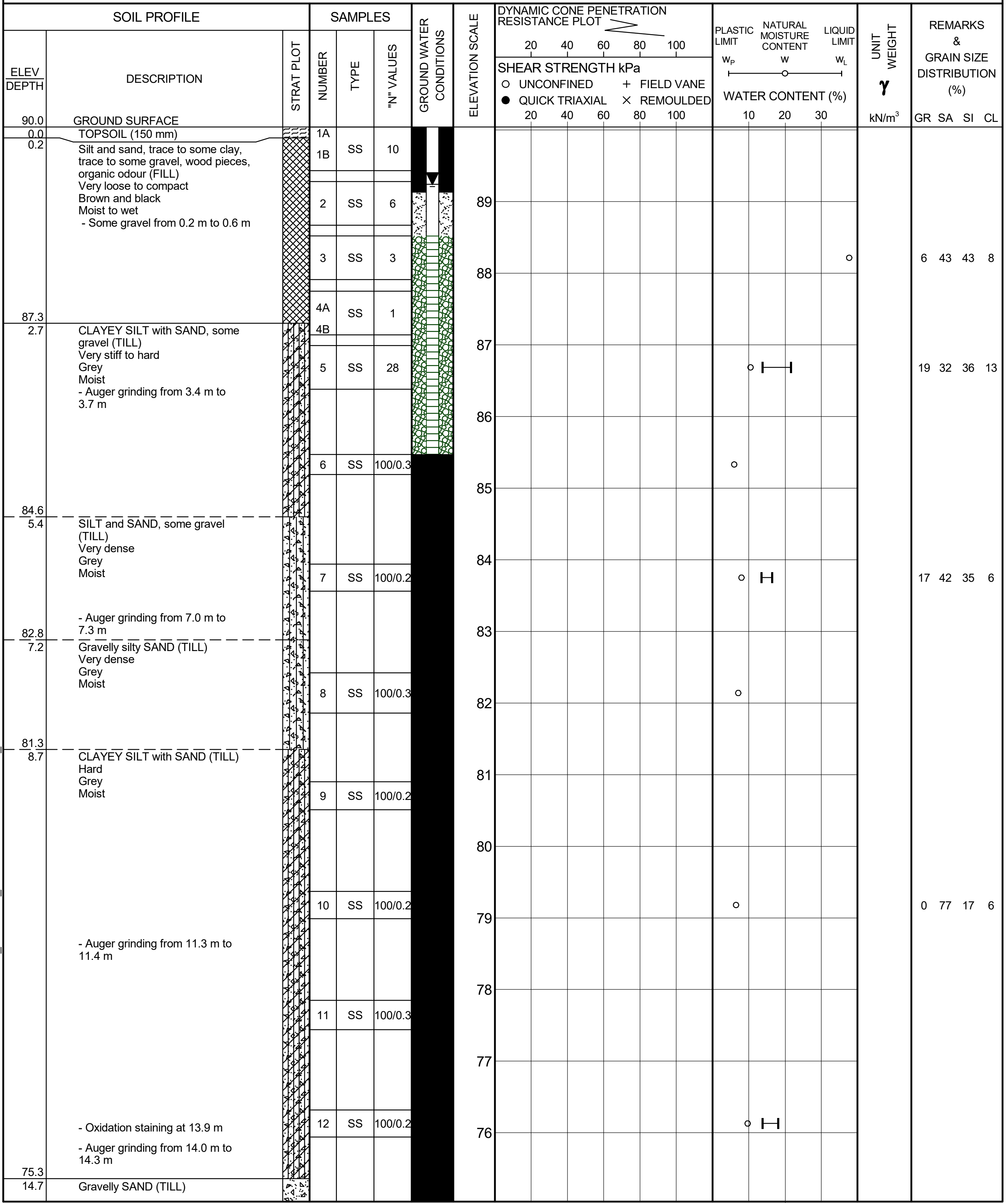
+³, ×³: Numbers refer to Sensitivity ○³% STRAIN AT FAILURE

RECORD OF BOREHOLE No S3

SHEET 1 OF 2

METRIC

PROJECT 1662333
G.W.P. 2002-13-00
DIST Central HWY QEW
DATUM Geodetic
LOCATION N 4824337.3; E 296021.0 MTM NAD 83 ZONE 10 (LAT. 43.558912; LONG. -79.608665)
BOREHOLE TYPE CME 55, 108 mm I.D., Hollow Stem Augers
DATE October 9, 2018
ORIGINATED BY ACM
COMPILED BY JMP
CHECKED BY SMM



ELEV / DEPTH	DESCRIPTION
90.0	GROUND SURFACE
0.0	TOPSOIL (150 mm)
0.2	Silt and sand, trace to some clay, trace to some gravel, wood pieces, organic odour (FILL) Very loose to compact Brown and black Moist to wet - Some gravel from 0.2 m to 0.6 m
87.3 / 2.7	CLAYEY SILT with SAND, some gravel (TILL) Very stiff to hard Grey Moist - Auger grinding from 3.4 m to 3.7 m
84.6 / 5.4	SILT and SAND, some gravel (TILL) Very dense Grey Moist - Auger grinding from 7.0 m to 7.3 m
82.8 / 7.2	Gravelly silty SAND (TILL) Very dense Grey Moist
81.3 / 8.7	CLAYEY SILT with SAND (TILL) Hard Grey Moist - Auger grinding from 11.3 m to 11.4 m - Oxidation staining at 13.9 m - Auger grinding from 14.0 m to 14.3 m
75.3 / 14.7	Gravelly SAND (TILL)

NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
1A 1B	SS	10				
2	SS	6				
3	SS	3	6	43	43	8
4A 4B	SS	1				
5	SS	28	19	32	36	13
6	SS	100/0.3				
7	SS	100/0.2	17	42	35	6
8	SS	100/0.3				
9	SS	100/0.2				
10	SS	100/0.2	0	77	17	6
11	SS	100/0.3				
12	SS	100/0.2				

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

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

RECORD OF BOREHOLE No S3

SHEET 2 OF 2

METRIC

PROJECT	1662333				
G.W.P.	2002-13-00	LOCATION	N 4824337.3; E 296021.0 MTM NAD 83 ZONE 10 (LAT. 43.558912; LONG. -79.608665)	ORIGINATED BY	ACM
DIST	Central HWY QEW	BOREHOLE TYPE	CME 55, 108 mm I.D., Hollow Stem Augers	COMPILED BY	JMP
DATUM	Geodetic	DATE	October 9, 2018	CHECKED BY	SMM

SOIL PROFILE			SAMPLES								REMARKS & GRAIN SIZE DISTRIBUTION (%)			
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	GR	SA	SI	CL
	--- *CONTINUED FROM PREVIOUS PAGE* ---													
	Gravelly SAND, trace to some silt, trace clay (TILL) Compact to dense Grey Wet		13	SS	25				○		20	72	6	2
							74							
73.4			14	SS	46									
16.6	END OF BOREHOLE													

NOTES:

1. Water level measured at a depth of 15.9 m below ground surface (Elev. 74.1 m) upon completion of drilling.

2. Groundwater level measurements in piezometer:

Date	Depth (m)	Elev. (m)
06/11/18	0.8	89.2

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

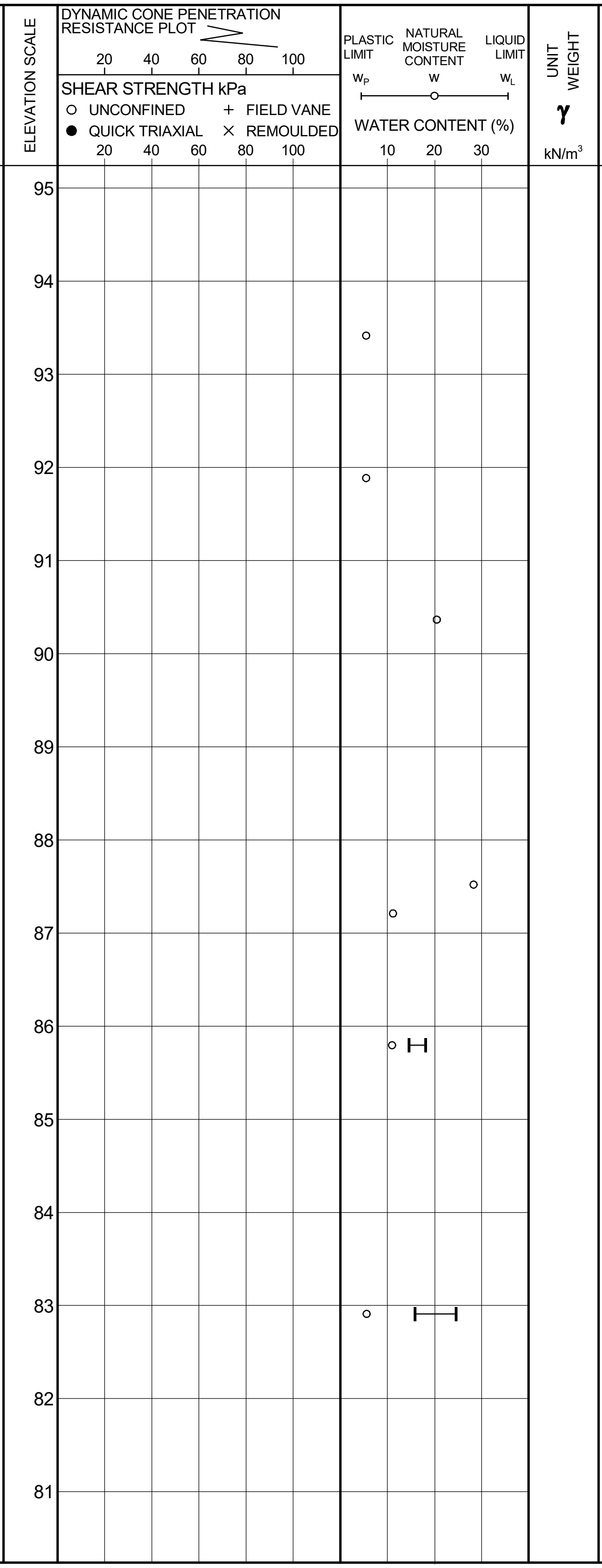
RECORD OF BOREHOLE No S4

SHEET 1 OF 2

METRIC

PROJECT 1662333
G.W.P. 2002-13-00
DIST Central HWY QEW
DATUM Geodetic
LOCATION N 4824336.4; E 296040.2 MTM NAD 83 ZONE 10 (LAT. 43.558913; LONG. -79.608430)
BOREHOLE TYPE CME 55, 108 mm I.D., Hollow Stem Augers
DATE September 16, 2018
ORIGINATED BY ACM
COMPILED BY JMP
CHECKED BY SMM

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
95.2	GROUND SURFACE								
0.0	CONCRETE (450 mm)								
94.7 0.5	Sand and gravel, trace to some silt (FILL) Compact Brown Moist	1	SS	24					
		2	SS	17					
93.7 1.5	Silt and sand, trace clay (FILL) Very loose to compact Brown Moist to wet at a depth of 5.0 m - Trace to some gravel from 2.3 m to 2.9 m	3	SS	13					
		4	SS	3					
		5	SS	2		0	65	33	2
		6	SS	12					
		7	SS	1					
89.6 5.6	SILT and SAND Very loose Grey Wet - Silty seam at a depth of 6.6 m, 1 mm thick - Trace clay layer between 7.6 m to 7.8 m				∇				
		8	SS	WH					
87.4 7.8	SILT and SAND, some gravel, trace to some clay (TILL) Compact to very dense Grey Moist to wet	9A 9B	SS	17					
		10	SS	40		16	42	37	5
		11	SS	100/0.10					
83.7 11.5	Gravelly CLAYEY SILT with SAND (TILL) Hard Grey Moist - Auger grinding from 12.8 m to 13.4 m	12	SS	100/0.28		19	43	27	11
		13	SS	100/0.23					



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+3, ×3: Numbers refer to Sensitivity

○3% STRAIN AT FAILURE

RECORD OF BOREHOLE No S4

SHEET 2 OF 2

METRIC

PROJECT 1662333

G.W.P. 2002-13-00

LOCATION N 4824336.4; E 296040.2 MTM NAD 83 ZONE 10 (LAT. 43.558913; LONG. -79.608430)

ORIGINATED BY ACM

DIST Central HWY QEW

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

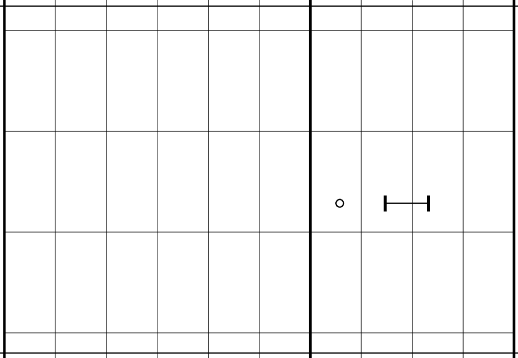
COMPILED BY JMP

DATUM Geodetic

DATE September 16, 2018

CHECKED BY SMM

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
	--- CONTINUED FROM PREVIOUS PAGE ---										
	Gravelly CLAYEY SILT with SAND (TILL) Hard Grey Moist					80					
		14	SS	58							
						79					
		15	SS	100/0.23				29	45	19	7
						78					
	- Auger grinding from 17.4 m to 17.7 m										
						77					
76.8 18.4	END OF BOREHOLE	16	SS	100/0.15							



NOTES:

1. Water level recorded at a depth of about 6.1 m below ground surface (Elev. 89.1 m) upon completion of drilling.

2. Borehole caved to a depth of about 6.1 m upon removal of augers.

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

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

GOLDER

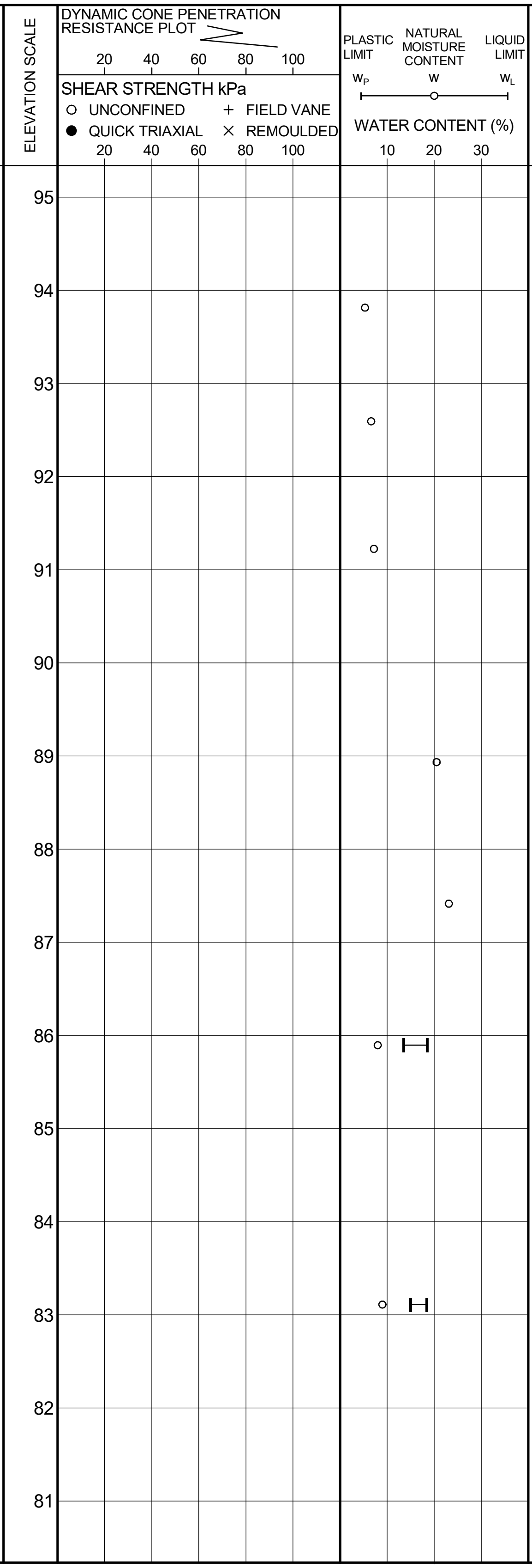
RECORD OF BOREHOLE No S5

SHEET 1 OF 2

METRIC

PROJECT 1662333
G.W.P. 2002-13-00
DIST Central HWY QEW
DATUM Geodetic
LOCATION N 4824341.1; E 296053.5 MTM NAD 83 ZONE 10 (LAT. 43.558955; LONG. -79.608265)
BOREHOLE TYPE CME 75, 184 mm O.D., Hollow Stem Augers
DATE November 23, 2018
ORIGINATED BY SK
COMPILED BY JMP
CHECKED BY SMM

ELEV DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLES – NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
95.3	GROUND SURFACE									
0.0 95.0	ASPHALT (300 mm)									
0.3	Silt and sand, trace clay (FILL) Loose to compact Brown Moist - Auger grinding from 0.3 m to 0.6 m - Some oxidation staining at 2.4 m to 3.1 m	1	SS	22						
		2	SS	9						
		3	SS	7						
		4	SS	8			0	58	41	1
		5	SS	11						
		6	SS	7						
		7	SS	7						
89.7 5.6	SILT and SAND to SAND, some organics, some wood pieces, trace clay Loose to compact Grey Moist	8	SS	8		Org = 1.5%				
					∇					
		9	SS	16		Org = 2.2%				
86.6 8.7	CLAYEY SILT with SAND, some gravel (TILL) Very stiff to hard Grey Moist - Auger grinding at 10.4 m	10	SS	28			18	34	38	10
		11	SS	100/0.10						
83.6 11.7	SILT and SAND, some gravel, trace to some clay, trace organics (TILL) Very dense Grey Moist	12	SS	100/0.08			18	30	45	7
82.0 13.3	CLAYEY SILT with SAND, some gravel, shale fragments (TILL) Hard Grey Moist	13	SS	100/0.10						



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

GOLDER

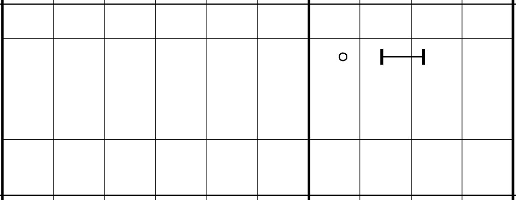
RECORD OF BOREHOLE No S5

SHEET 2 OF 2

METRIC

PROJECT 1662333
G.W.P. 2002-13-00
LOCATION N 4824341.1; E 296053.5 MTM NAD 83 ZONE 10 (LAT. 43.558955; LONG. -79.608265)
ORIGINATED BY SK
DIST Central HWY QEW
BOREHOLE TYPE CME 75, 184 mm O.D., Hollow Stem Augers
COMPILED BY JMP
DATUM Geodetic
DATE November 23, 2018
CHECKED BY SMM

ELEV DEPTH	SOIL PROFILE – DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	SAMPLES 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
	--- CONTINUED FROM PREVIOUS PAGE ---													
	CLAYEY SILT with SAND, some gravel, shale fragments (TILL) Hard Grey Moist						80							
			14	SS	99						15	55	23	7
							79							
78.4	- Oxidation staining at 16.8 m		15	SS	100/0.13									
16.9	END OF BOREHOLE													



NOTES:

1. Borehole caved to a depth of 7.6 m below ground surface upon completion of drilling.

2. Water level measured in caved borehole at a depth of 7.0 m below ground surface (Elev 88.3 m) upon completion of drilling.

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

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

RECORD OF BOREHOLE No S6

SHEET 1 OF 2

METRIC

PROJECT	1662333		
G.W.P.	2002-13-00	LOCATION	N 4824318.8; E 296059.4 MTM NAD 83 ZONE 10 (LAT. 43.558755; LONG. -79.608192)
DIST	Central HWY QEW	BOREHOLE TYPE	CME 55, 108 mm I.D., Hollow Stem Augers
DATUM	Geodetic	DATE	October 2, 2018
ORIGINATED BY	ACM	COMPILED BY	JMP
CHECKED BY	SMM		

SOIL PROFILE

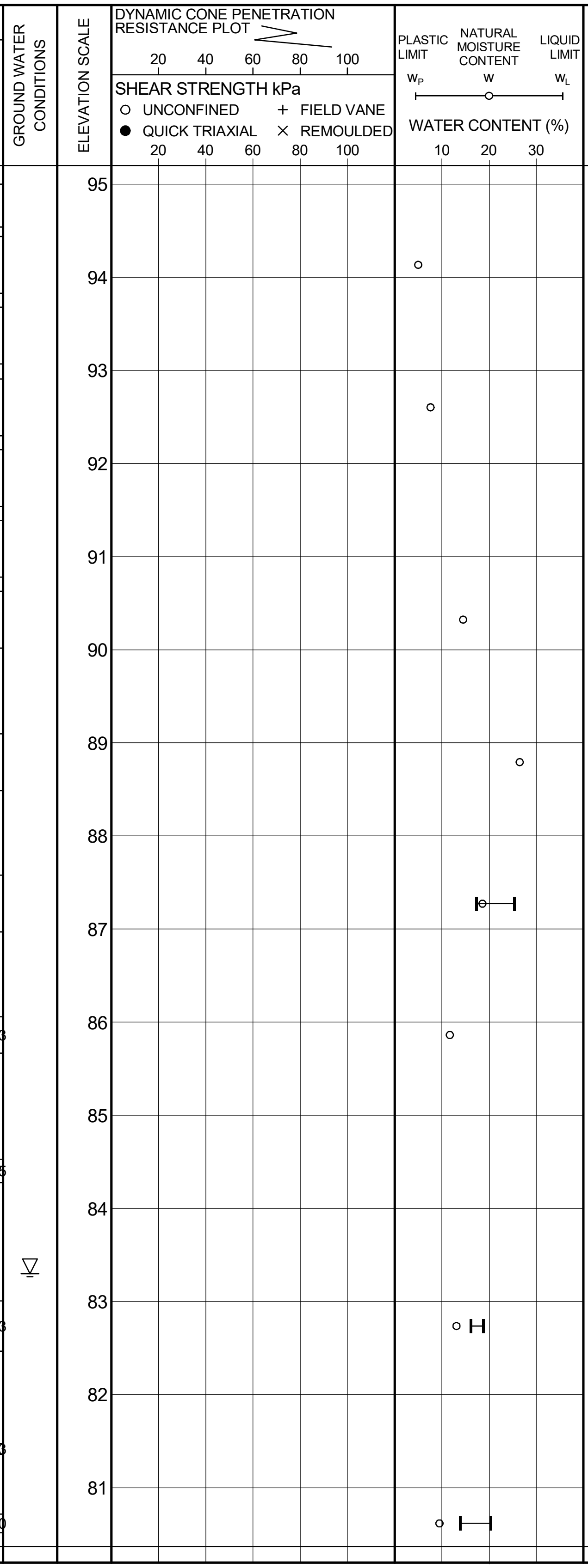
ELEV DEPTH	DESCRIPTION
95.2	GROUND SURFACE
0.0	ASPHALT (200 mm)
0.2	Sand and gravel (FILL) Very dense Brown Moist
94.5 0.7	Silt and sand, trace clay, trace gravel (FILL) Loose to dense Brown Moist
89.6 5.6	Silty SAND, trace to some clay, some gravel Very loose Brown to grey at 6.3 m Wet
88.0 7.2	CLAYEY SILT with SAND, trace gravel (TILL) Stiff to hard Grey Moist - Sand pocket at 7.9 m and 10.8 m
83.6 11.6	SILT and SAND, trace to some clay, trace gravel (TILL) Hard Grey Moist
81.8 13.4	CLAYEY SILT with SAND with GRAVEL, containing shale fragments (TILL) Hard Grey Moist to wet - Auger grinding from 13.4 m to 14.5 m
80.4 14.8	- Auger bouncing at 14.5 m

Continued Next Page

SAMPLES

NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
1	SS	8					
2	SS	39					
3	SS	24					
4	SS	19		2	63	31	4
5	SS	7					
6	SS	6					
7	SS	4		0	56	41	3
8	SS	WH					
9	SS	12		4	33	46	17
10	SS	100/0.23					
11	SS	100/0.25					
12	SS	100/0.23		2	30	58	10
13	SS	100/0.13					
14	SS	100/0.20		44	36	16	4

REMARKS & GRAIN SIZE DISTRIBUTION (%)



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

RECORD OF BOREHOLE No S6

SHEET 2 OF 2

METRIC

PROJECT 1662333

G.W.P. 2002-13-00

LOCATION N 4824318.8; E 296059.4 MTM NAD 83 ZONE 10 (LAT. 43.558755; LONG. -79.608192)

ORIGINATED BY ACM

DIST Central HWY QEW

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

COMPILED BY JMP

DATUM Geodetic

DATE October 2, 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 *--- CONTINUED FROM PREVIOUS PAGE ---*	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	END OF BOREHOLE NOTES: 1. Water level measured at a depth of 11.9 m (Elev. 83.3 m) below ground surface upon completion of drilling.										

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

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

RECORD OF BOREHOLE No S7

SHEET 1 OF 1

METRIC

PROJECT 1662333

G.W.P. 2002-13-00

DIST Central HWY QEW

DATUM Geodetic

LOCATION N 4824321.2; E 296076.9 MTM NAD 83 ZONE 10 (LAT. 43.558768; LONG. -79.607973)

BOREHOLE TYPE Portable Tripod - NW Casing and Wash Boring

DATE December 20-21, 2018

ORIGINATED BY ACM

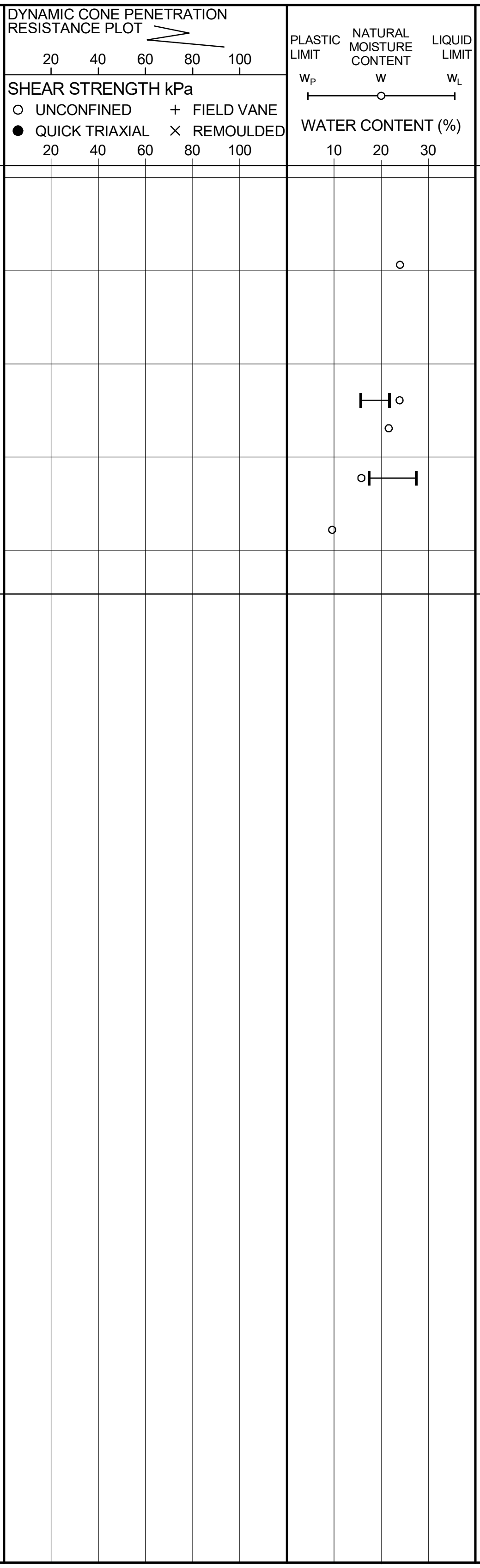
COMPILED BY SK

CHECKED BY SMM

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES				GR	SA	SI	CL
90.1	GROUND SURFACE										
0.0	TOPSOIL	1A	SS	5		90					
89.8 0.3	Silt and sand, trace to some gravel, trace clay (FILL) Loose Brown with oxidation staining Moist to wet at 8.7 m	1B									
		2	SS	7		89		7	56	34	3
	- 0.03 m concrete piece at 1.5 m	3	SS	6		88					
87.8 2.3	CLAYEY SILT with SAND, trace gravel Firm Grey Wet - Oxidation staining from 2.7 m to 3.7 m	4A	SS	7				4	43	37	16
87.4 2.7		4B									
	CLAYEY SILT with SAND, trace gravel (TILL) Hard Brown-grey to grey Moist	5	SS	33		87		3	33	44	20
		6	SS	100/0.20		86					
85.8	LIMESTONE	7	SS	200/0.03							
85.5 4.6	END OF BOREHOLE SPLIT-SPOON REFUSAL										

NOTES:

1. Water level not taken due to water added during drilling.



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

$○^{3\%}$ STRAIN AT FAILURE

PROJECT 1662333	**RECORD OF BOREHOLE No NW3-2**	SHEET 1 OF 1	**METRIC**
G.W.P. 2002-13-00	LOCATION N 4824342.4; E 295994.3 MTM NAD 83 ZONE 10 (LAT. 43.558958; LONG. -79.608996)		ORIGINATED BY FC
DIST Central HWY QEW	BOREHOLE TYPE CME 55, 203 mm O.D. Hollow Stem Augers		COMPILED BY KN
DATUM Geodetic	DATE August 23, 2017		CHECKED BY MWK

SOIL PROFILE

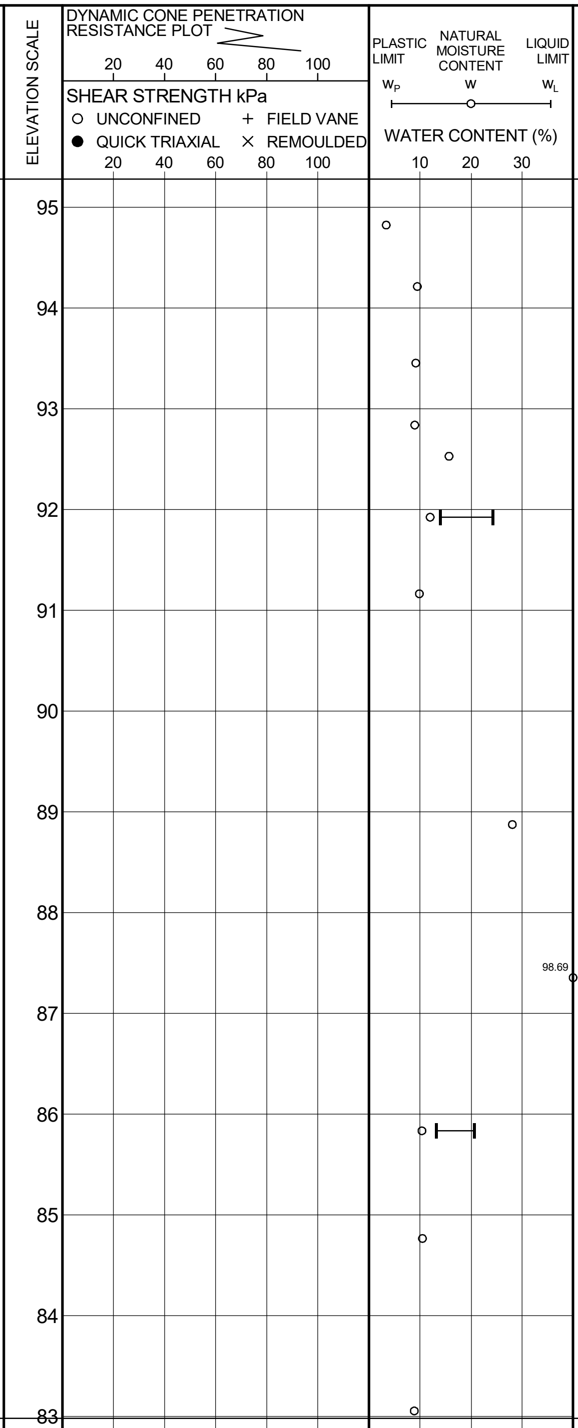
ELEV DEPTH	DESCRIPTION
95.3	GROUND SURFACE
0.0	ASPHALT (150 mm)
0.2	Silty sand, some gravel to gravelly, trace to some clay (FILL) Compact Brown Moist - Asphalt fragments at a depth of about 1.8 m
92.7 2.6	Sandy clayey silt, trace to some gravel (FILL) Stiff to very stiff Brown to grey, mottled Moist
91.6 3.7	Sand and gravel, some silt, trace clay (FILL) Dense Grey to brown Moist to wet - Trace asphalt fragments at a depth of about 4.0 m
90.0 5.3	Silty sand, trace clay, trace gravel, trace organics, trace asphalt fragments (FILL) Very loose Brown Moist to wet
87.5 7.8	- 100 mm silty sand, organic layer and pieces of wood at a depth of 7.6 m CLAYEY SILT with SAND, some gravel (TILL) Stiff to hard Grey Moist to wet - Trace organics from a depth of about 8.5 m
83.0 12.3	END OF BOREHOLE NOTE: 1. Borehole dry prior to tricone drilling below a depth of 3.4 m and introduction of wash water.

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
1	AS	-				
2	SS	28				
3	SS	26	16	58	20	6
4A 4B	SS	14				
5	SS	29				
6	SS	34	43	41	12	4
7	SS	2	1	70	26	3
8	SS	12				
9	SS	131/0.08	12	38	38	12
10	SS	100/0.13				
11	SS	100/0.13				

GROUND WATER CONDITIONS

UNIT WEIGHT γ kN/m³



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

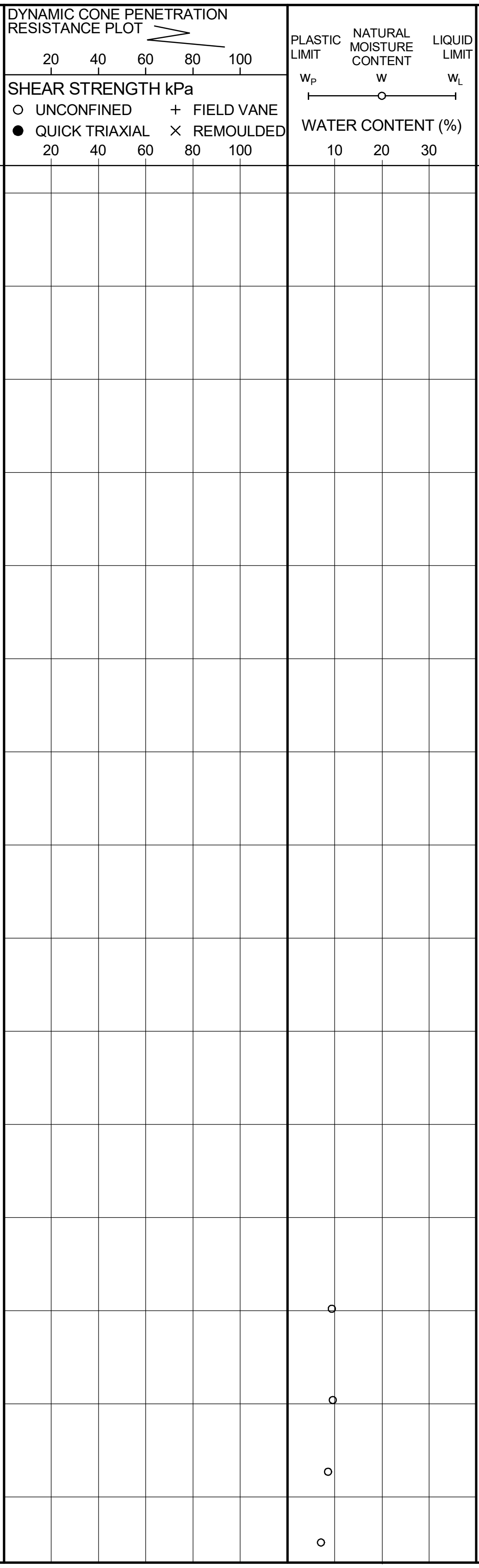
RECORD OF BOREHOLE No NW3-2A

SHEET 1 OF 2

METRIC

PROJECT 1662333
G.W.P. 2002-13-00
LOCATION N 4824344.2; E 295993.8 MTM NAD 83 ZONE 10 (LAT. 43.558975; LONG. -79.609002)
ORIGINATED BY FC
DIST Central HWY QEW
BOREHOLE TYPE CME 850, 156 mm Tricone with Drilling Mud
COMPILED BY KN
DATUM Geodetic
DATE November 20-21, 2017
CHECKED BY MWK

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
95.3	GROUND SURFACE								
0.0	Refer to Record of Borehole NW3-2 for soil profile details. Borehole NW3-2A locates about 2 m NE of Borehole NW3-2.					∇ (at 92)	95–84		
83.1 12.2	CLAYEY SILT with SAND, some gravel (TILL) Hard Grey Moist to wet		1	SS	100/0.10		83		
			2	SS	100/0.28		82		
			3	SS	92		81		
			4	SS	77				



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

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

RECORD OF BOREHOLE No NW3-2A

SHEET 2 OF 2

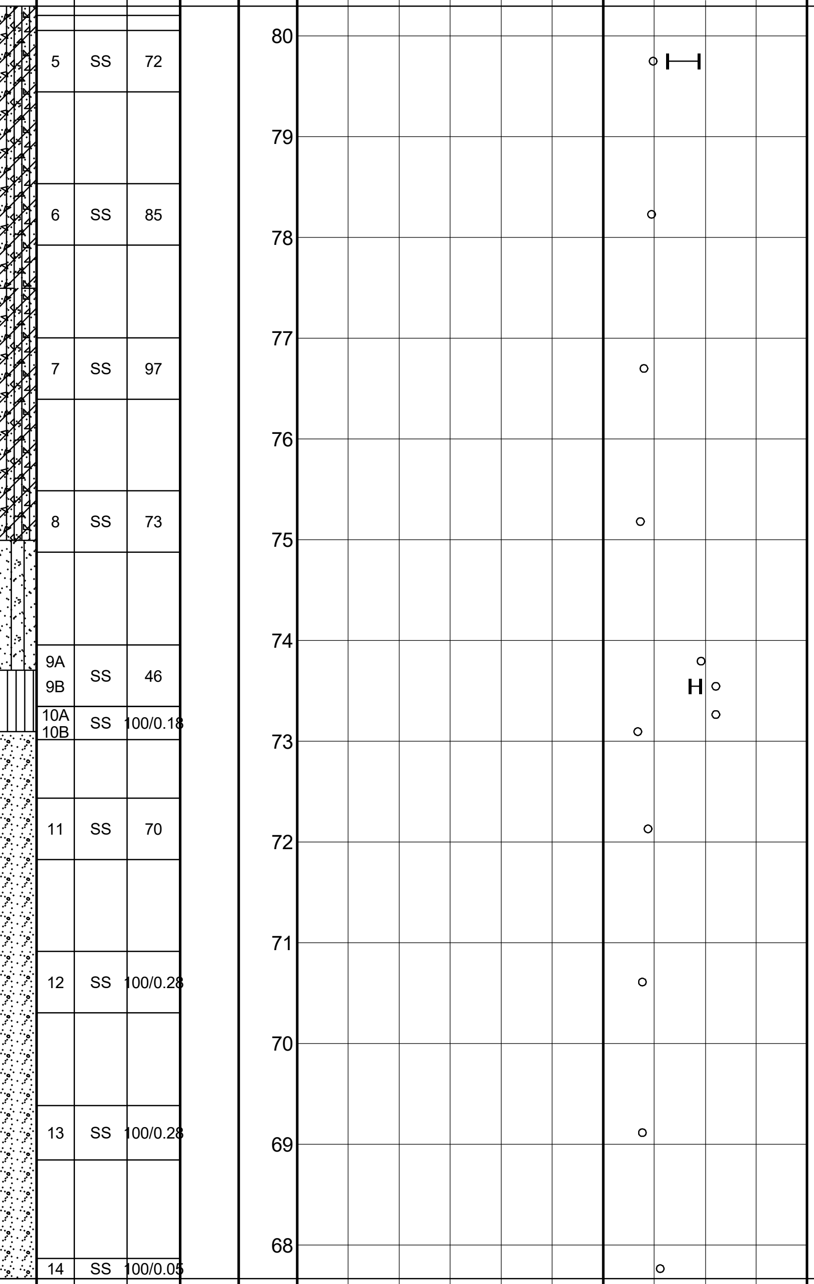
METRIC

PROJECT	1662333		
G.W.P.	2002-13-00	LOCATION	N 4824344.2; E 295993.8 MTM NAD 83 ZONE 10 (LAT. 43.558975; LONG. -79.609002)
DIST	Central	HWY	QEW
BOREHOLE TYPE	CME 850, 156 mm Tricone with Drilling Mud		
DATUM	Geodetic	DATE	November 20-21, 2017
ORIGINATED BY	FC	COMPILED BY	KN
CHECKED BY	MWK		

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV / DEPTH	DESCRIPTION
	CLAYEY SILT with SAND, some gravel (TILL) Hard Grey Moist to wet
77.5 17.8	Gravelly CLAYEY SILT with SAND (TILL) Hard Grey Wet
75.0 20.3	Silty SAND, trace to some clay, trace gravel Grey Moist to wet
73.7 21.6	SILT, some sand, trace clay Dense Grey Wet - Clayey silt pocket at a depth of about 21.9 m
73.1 22.2	SAND and GRAVEL, trace to some silt, trace clay Very dense Grey Moist to wet
67.7 27.6	END OF BOREHOLE

NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
5	SS	72	18	64	14	4
6	SS	85				
7	SS	97	26	57	14	3
8	SS	73				
9A 9B	SS	46	1	70	24	5
10A 10B	SS	100/0.18				
11	SS	70				
12	SS	100/0.28				
13	SS	100/0.28	37	44	16	3
14	SS	100/0.05				



UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%)

NOTES:

1. Water level measured at a depth of about 3.4 m below ground surface (Elev. 91.9 m) on November 21, 2017 before start of drilling when the borehole was at a depth of about 23.5 m, after the introduction of drilling mud/water during borehole drilling operations on Nov 20, 2017.

+³, ×³: Numbers refer to Sensitivity

○^3% STRAIN AT FAILURE

GOLDER

Foundation Design

RECORD OF BOREHOLE No NW3-3

SHEET 1 OF 1

METRIC

PROJECT 1662333

G.W.P. 2002-13-00

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

ORIGINATED BY ACM

DIST Central HWY QEW

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

COMPILED BY SK

DATUM Geodetic

DATE August 9, 2018

CHECKED BY SMM

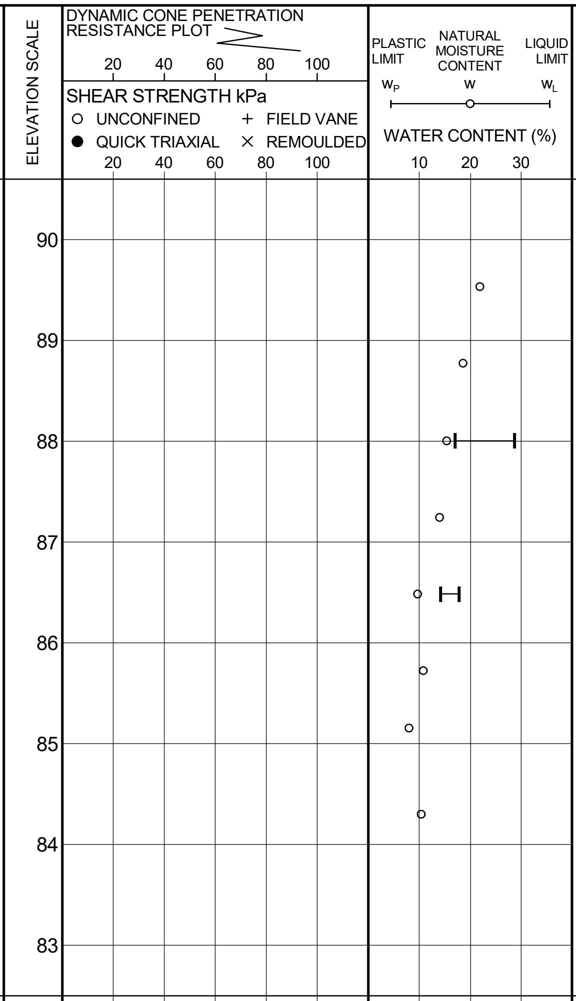
SOIL PROFILE

ELEV DEPTH	DESCRIPTION
90.6	GROUND SURFACE
0.0	Silty sand, some gravel, trace to some clay, trace organics (FILL) Very loose Dark brown Moist to wet below 0.7 m - Hydrocarbon odour at 0.8 m
89.2 1.5	CLAYEY SILT with SAND, trace to some gravel (TILL) Soft to stiff Grey-brown to grey at 3.7 m with oxidation staining Moist
87.2 3.4	SILT and SAND, some gravel. trace to some clay (TILL) Compact to very dense Grey Moist - Auger grinding from 3.4 m to 3.8 m - Auger grinding from 6.1 m to 6.9 m - Limestone layer between depths of 6.2 m and 6.3 m
82.5 8.1	END OF BOREHOLE NOTES: 1. Open borehole dry upon completion of drilling.

SAMPLES

NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
1	SS	4						
2	SS	4			18	49	27	6
3	SS	4						
4	SS	15			8	31	44	17
5	SS	22						
6	SS	47						
7	SS	100/0.28						
8	SS	50/0.08						
9A 9B	SS	50/0.13						
10	SS	50/0.08						
11	SS	100/0.28						

REMARKS & GRAIN SIZE DISTRIBUTION (%)



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

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

RECORD OF BOREHOLE No PED-02

SHEET 1 OF 2

METRIC

PROJECT 1662333

G.W.P. 2002-13-00

DIST Central HWY QEW

DATUM Geodetic

LOCATION N 4824321.8; E 296032.3 MTM NAD 83 ZONE 10 (LAT. 43.558773; LONG. -79.608524)

BOREHOLE TYPE CME 75, 203 mm O.D. Hollow Stem Augers, HQ Casing

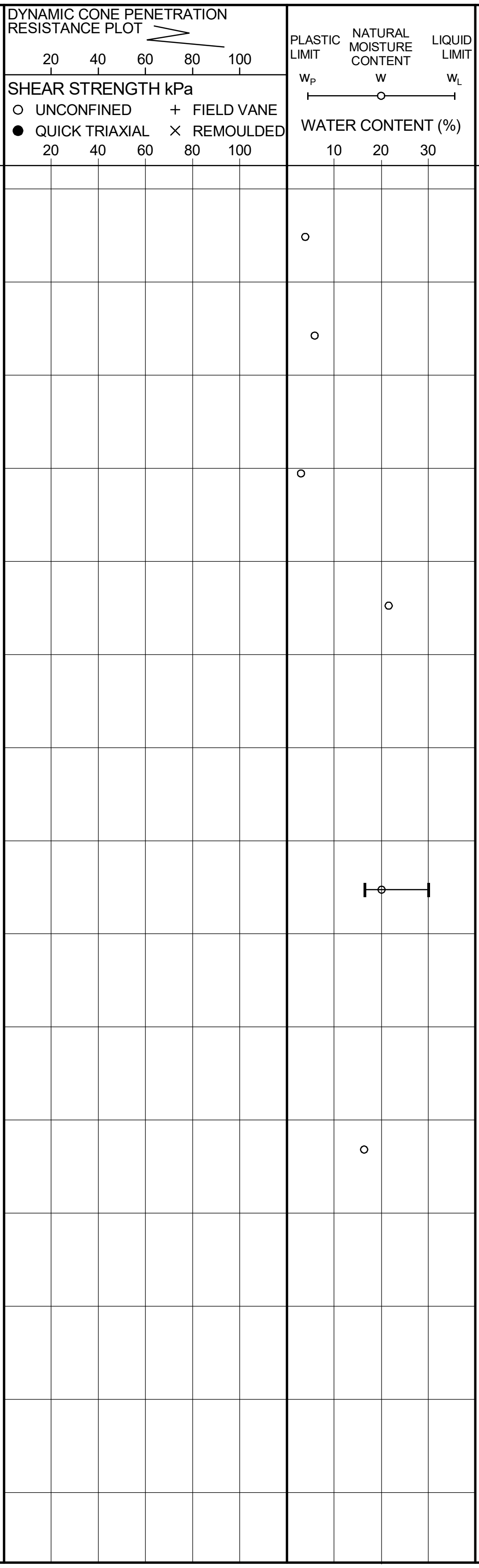
DATE December 4-6 2017

ORIGINATED BY JL

COMPILED BY DH

CHECKED BY MWK

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES				GR	SA	SI	CL
95.2	GROUND SURFACE										
0.0	ASPHALT (100 mm)										
0.1	Silt sand to sand, some gravel to gravelly sand, some silt, trace clay, some asphalt fragments (FILL) Loose to Compact Brown Moist	1	AS	-		95					
		2	SS	25	∇						
						94					
		3	SS	14				19	56	22	3
						93					
		4	SS	12							
						92					
		5	SS	9							
91.5 3.7	SAND and GRAVEL to GRAVEL trace sand, trace fines Loose Grey Moist to wet	6	SS	5		91					
		7	SS	7				66	32	(2)	
						90					
		8	SS	7		89					
88.7 6.5	SILT and SAND to Silty SAND Brown Moist to wet										
88.2 7.0	CLAYEY SILT, some sand to with SAND, trace to some gravel (TILL) Very stiff to hard Grey Moist to wet - SAND to SILTY SAND pockets/zones - Contains some shale fragments throughout					88					
		9	SS	20				6	27	45	22
						87					
		10	SS	100/0.08							
						86					
						85					
		11	SS	100/0.10							
						84					
		12	SS	100/0.03							
						83					
						82					
		13	SS	100/0.25							
						81					



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

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

RECORD OF BOREHOLE No PED-02

SHEET 2 OF 2

METRIC

PROJECT 1662333

G.W.P. 2002-13-00

DIST Central HWY QEW

DATUM Geodetic

LOCATION N 4824321.8; E 296032.3 MTM NAD 83 ZONE 10 (LAT. 43.558773; LONG. -79.608524)

BOREHOLE TYPE CME 75, 203 mm O.D. Hollow Stem Augers, HQ Casing

DATE December 4-6 2017

ORIGINATED BY JL

COMPILED BY DH

CHECKED BY MWK

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
			14	SS	100/0.28		80				17	51	24	8
79.1 16.1	Sandy gravelly CLAYEY SILT, some shale fragments (RESIDUAL SOIL) Very dense Grey Moist to wet						79							
			15	SS	100/0.13									
78.5 16.7	SHALE (BEDROCK) Grey													
	END OF BOREHOLE SPLIT-SPOON REFUSAL													

NOTES:

1. Borehole dry to 3.0 m depth prior to tricone drilling.

2. Water level measured at a depth of about 1.9 m below ground surface (Elev. 93.3 m) on December 5, 2017 before start of drilling when the borehole was at a depth of about 10.5 m.

3. Water level measured at a depth of about 1.1 m below ground surface (Elev. 94.1 m) on December 6, 2017 before start of drilling when the borehole was at a depth of about 16.6 m.

4. The water level measurement is not considered to be representative of the groundwater level due to the introduction of drilling mud/water during borehole drilling operations.

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

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

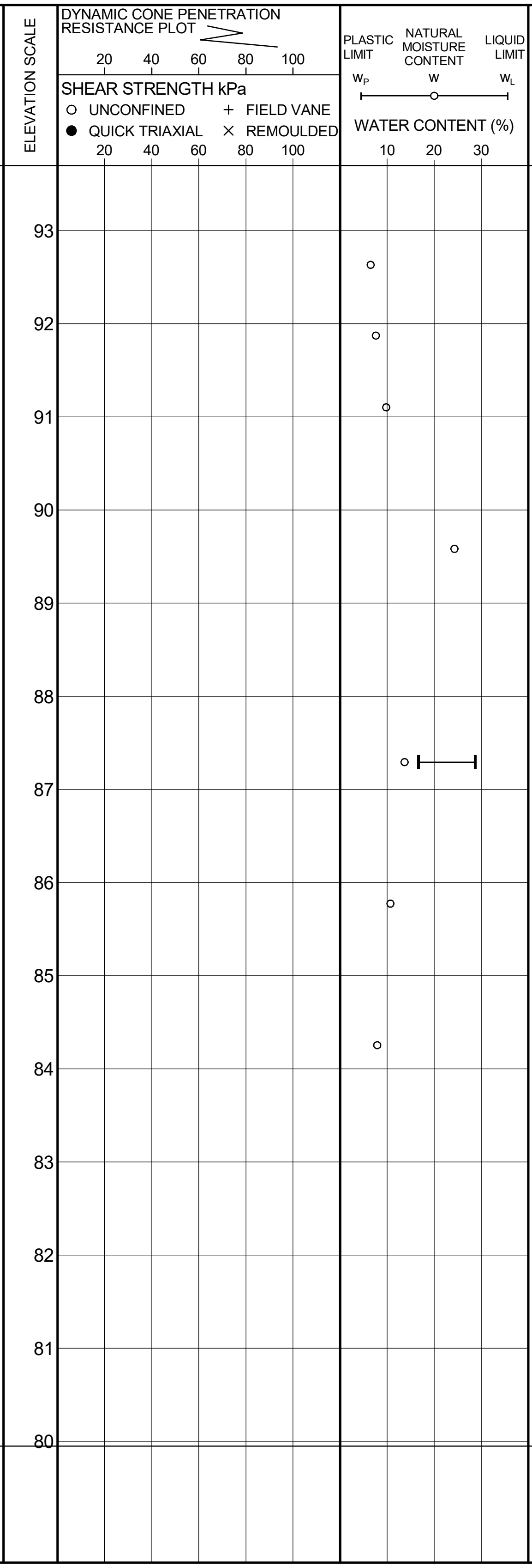
RECORD OF BOREHOLE No PED-03

SHEET 1 OF 1

METRIC

PROJECT 1662333
G.W.P. 2002-13-00
DIST Central HWY QEW
DATUM Geodetic
LOCATION N 4824305.3; E 296063.0 MTM NAD 83 ZONE 10 (LAT. 43.558625; LONG. -79.608144)
BOREHOLE TYPE CME 850, 210 mm O.D. Hollow Stem Augers, HQ Casing
DATE October 26-27, 2017
ORIGINATED BY JL
COMPILED BY DPM
CHECKED BY MWK

ELEV DEPTH	DESCRIPTION (SOIL PROFILE)	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
93.7	GROUND SURFACE									
0.0	TOPSOIL (200mm)									
0.2	Silt and sand to silty sand, trace clay, trace organics (FILL) Very loose to compact Brown Moist	1	SS	7						
		2	SS	16						
		3	SS	6			0	66	31	3
		4	SS	9						
		5	SS	10						
	- Becoming grey at a depth of about 3.5 m - Auger grinding at a depth of about 3.7 m - PHC odour between depths of about 3.8 m and 6.1 m - Becoming black at a depth of about 4.1 m - Some asphalt fragments at a depth of about 4.1 m - Becoming wet at a depth of about 4.6 m - Some gravel at a depth of about 4.6 m	6	SS	8			14	57	27	2
		7	SS	3						
87.6 6.2	Sandy CLAYEY SILT, trace to some gravel to gravelly (TILL) Very stiff to hard Brown and grey Moist to wet - Mottled grey at a depth of about 7.2 m	8	SS	16			18	26	41	15
		9	SS	19						
		10	SS	66						
		11	SS	50/0.08						
		1	SC	REC 20%			RQD = 0%			
	- Limestone slab cored from a depth of about 12.1 m to 12.6 m - Red- grey below a depth of about 12.2 m - Cobbles and gravel from a depth of about 12.9 m to 13.2 m	2	SC	REC 63%			RQD = 0%			
80.0 13.8	END OF BOREHOLE NOTE: 1. Borehole dry prior to rock coring.									



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

RECORD OF BOREHOLE No PED-03A

SHEET 1 OF 1

METRIC

PROJECT 1662333

G.W.P. 2002-13-00

LOCATION N 4824308.4; E 296062.1 MTM NAD 83 ZONE 10 (LAT. 43.558653; LONG. -79.608155)

ORIGINATED BY JL

DIST Central HWY QEW

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

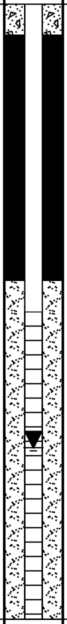
COMPILED BY DPM

DATUM Geodetic

DATE October 27, 2017

CHECKED BY MWK

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
94.1	GROUND SURFACE										
0.0	Refer to Record of Borehole PED-03 for soil profile details						94 93 92 91 90 89				
88.0 6.1	END OF BOREHOLE										



NOTES:

1. Groundwater level measurements in piezometer:

Date	Depth (m)	Elev. (m)
27/10/17	DRY	
14/11/17	4.3	89.8
21/11/17	4.4	89.7
28/11/18	4.4	89.7
06/11/18	4.1	90.0

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

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

RECORD OF BOREHOLE No PED-03B

SHEET 1 OF 2

METRIC

PROJECT 1662333

G.W.P. 2002-13-00

LOCATION N 4824309.6; E 296062.8 MTM NAD 83 ZONE 10 (LAT. 43.558664; LONG. -79.608146)

ORIGINATED BY JL

DIST Central HWY QEW

BOREHOLE TYPE CME 850, 156 mm Tricone with Drilling Mud

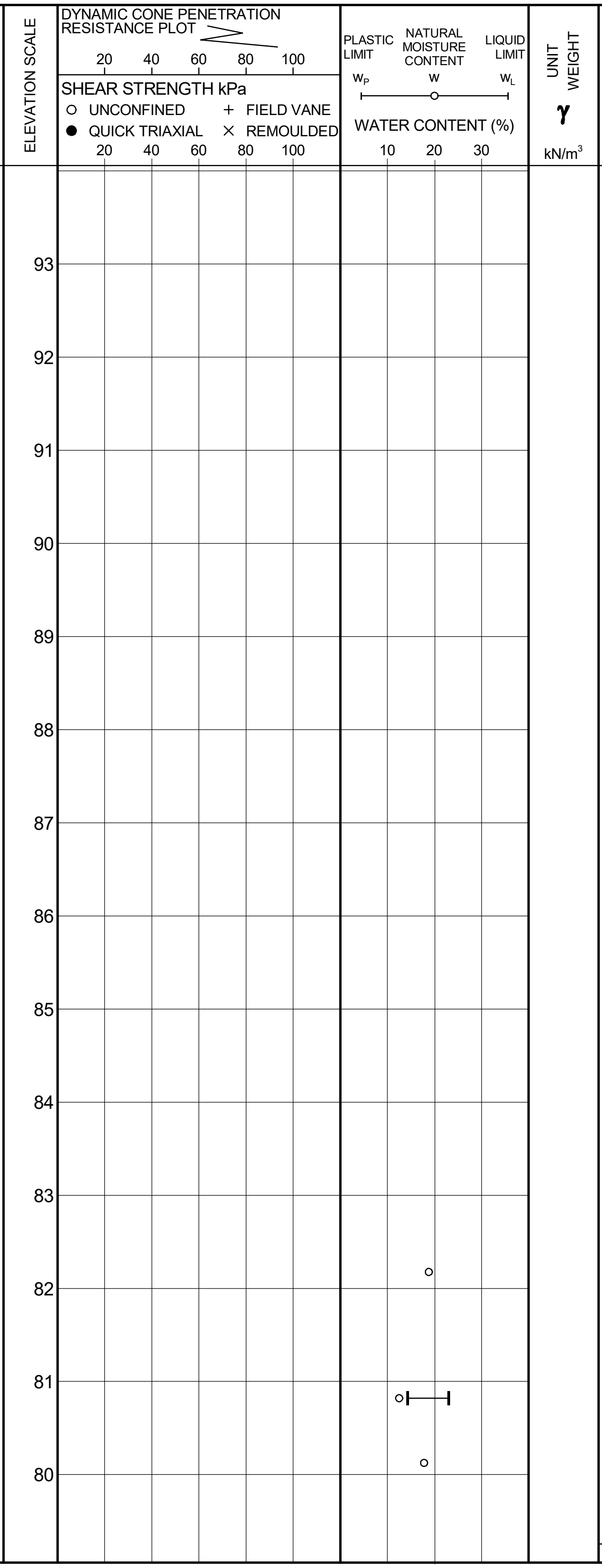
COMPILED BY KN

DATUM Geodetic

DATE November 15-16, 2017

CHECKED BY MWK

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
94.1	GROUND SURFACE									
0.0	Refer to Record of Borehole PED-03 for soil stratigraphy between ground surface and a depth of about 10.1 m									
84.0 10.1	Sandy gravelly CLAYEY SILT (TILL) Hard Grey Moist to wet		1	SS	44					
82.5 11.6	SILT, some clay, some sand Dense Grey Moist to wet		2	SS	35					
81.4 12.7	Gravelly CLAYEY SILT with SAND (TILL) Hard Red/grey Moist to wet		3	SS	100/0.10		24	48	21	7
			4	SS	100/0.13					
			5	SS	50/0.10					
79.3 14.8			1	RC			RQD = 38%			



Continued Next Page

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

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

RECORD OF BOREHOLE No PED-03B

SHEET 2 OF 2

METRIC

PROJECT 1662333

G.W.P. 2002-13-00

LOCATION N 4824309.6; E 296062.8 MTM NAD 83 ZONE 10 (LAT. 43.558664; LONG. -79.608146)

ORIGINATED BY JL

DIST Central HWY QEW

BOREHOLE TYPE CME 850, 156 mm Tricone with Drilling Mud

COMPILED BY KN

DATUM Geodetic

DATE November 15-16, 2017

CHECKED BY MWK

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION --- *CONTINUED FROM PREVIOUS PAGE* ---	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	Shale BEDROCK Grey Bedrock cored from a depth of 14.8 m to 17.8 m For bedrock coring details, refer to Record of Drillhole PED-03B		1	RC	REC 96%						RQD = 38%
			2	RC	REC 100%		78				RQD = 78%
76.3			3	RC	REC 100%		77				RQD = 67%
17.8	END OF BOREHOLE NOTE: 1. Borehole dry prior to tricone drilling.										

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

PROJECT: 1662333

RECORD OF DRILLHOLE: PED-03B

SHEET 1 OF 1

LOCATION: N 4824309.6 ;E 296062.8

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: November 15-16, 2017

DRILL RIG: CME 850 Track

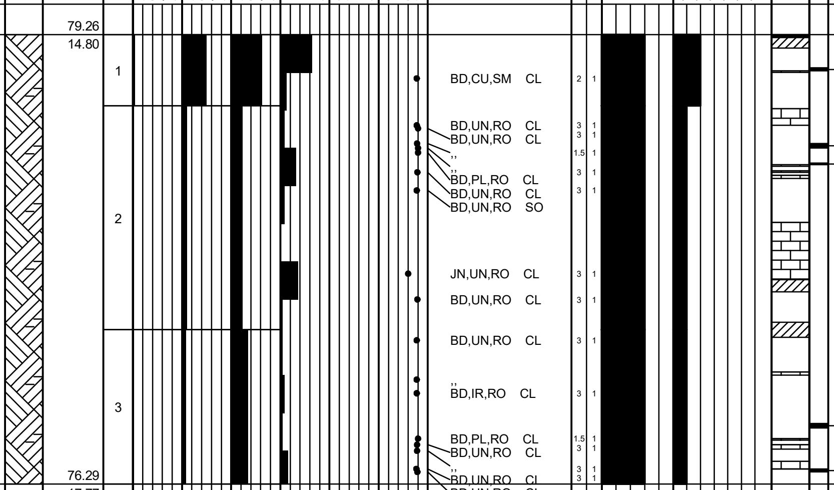
DRILLING CONTRACTOR: Aardvark Drilling

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.	NOTES WATER LEVELS INSTRUMENTATION
		Continued from Borehole PED-03		79.26		
15	NW Casing NQ Rock 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		14.80	1	R0
						R1
						R1
16					2	UC = 6.7 MPa
17					3	R1
				76.29		R1
		END OF DRILLHOLE		17.77		
18						
19						
20						
21						
22						
23						



TYPE AND SURFACE DESCRIPTION — Jr Ja

BD,CU,SM CL 2 1
BD,UN,RO CL 3 1
BD,UN,RO CL 3 1
,, 1.5 1
3 1
,,
BD,PL,RO CL
3 1
BD,UN,RO CL
BD,UN,RO SO
JN,UN,RO CL 3 1
BD,UN,RO CL 3 1
BD,UN,RO CL 3 1
,,
BD,IR,RO CL 3 1
BD,PL,RO CL 1.5 1
BD,UN,RO CL 3 1
,, 3 1
BD,UN,RO CL 3 1
BD,UN,RO CL

FEATURES LEGEND

▨ BROKEN CORE	▩ CLAY SEAM	▤ LIMESTONE	■ LOST CORE

DEPTH SCALE
1 : 50

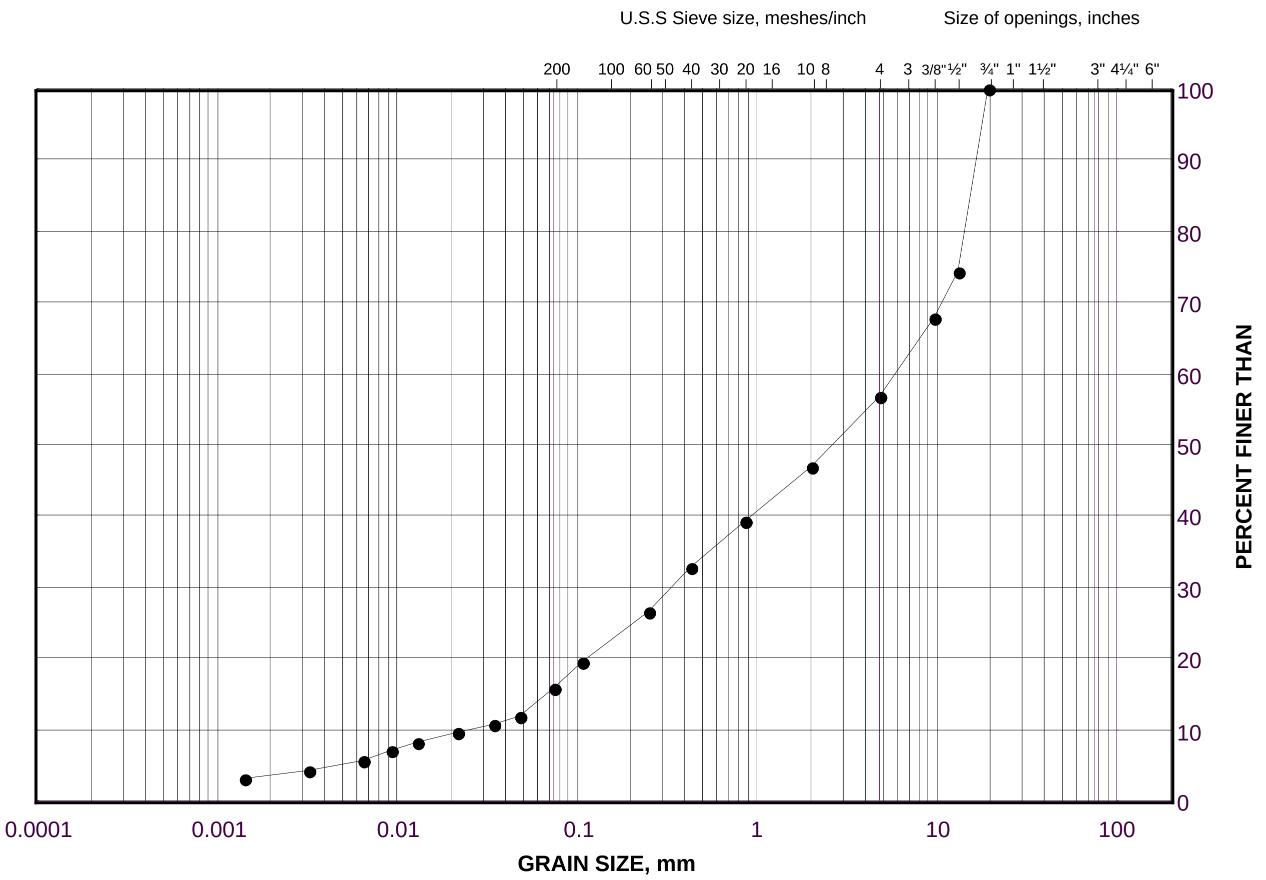
GOLDER

LOGGED: JL
CHECKED: SMM

GRAIN SIZE DISTRIBUTION

Sand and Gravel (Fill)

FIGURE A-1



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	NW3-2	6	91.2

Project Number: 1662333

Checked By: SMM

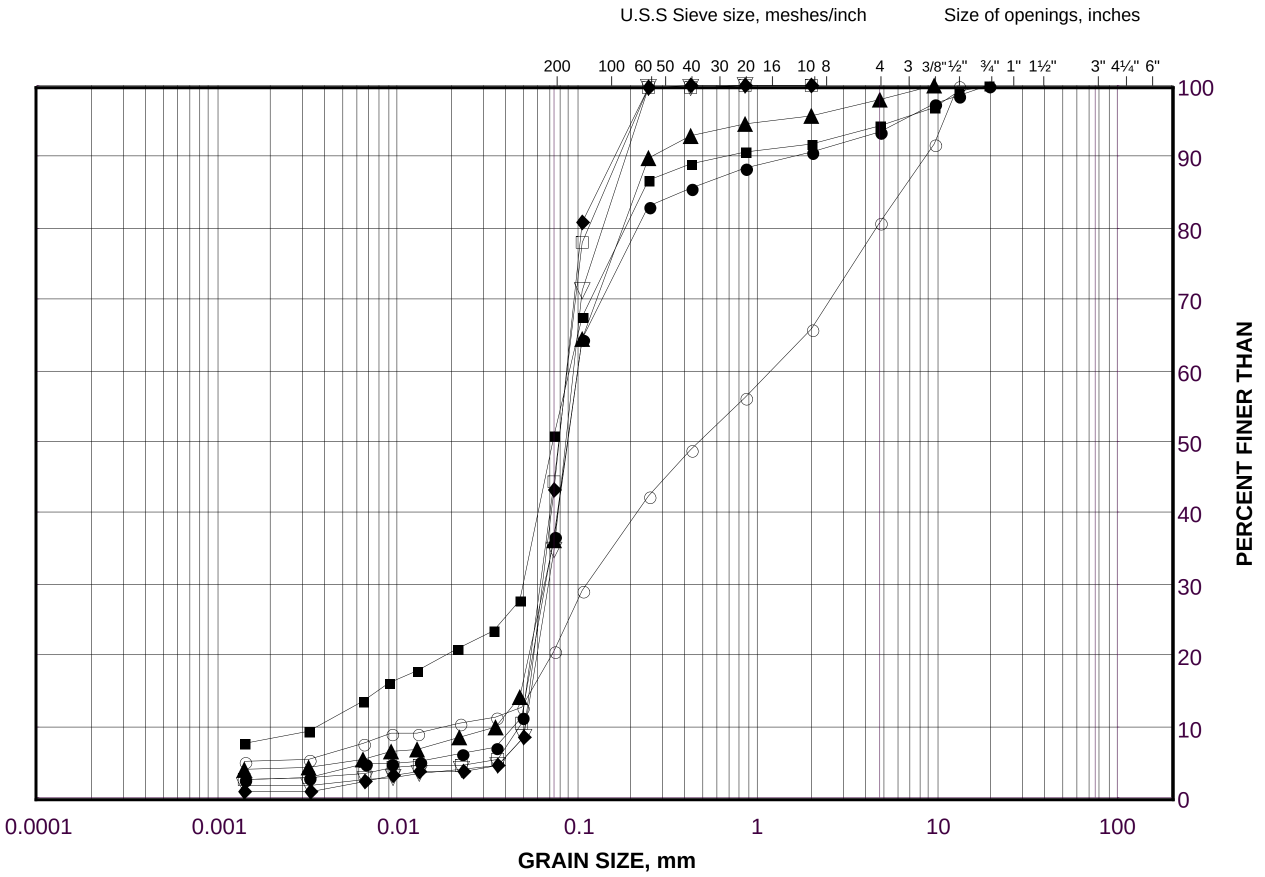
Golder Associates

Date: 05-Feb-19

GRAIN SIZE DISTRIBUTION

Silt and Sand to Sand (Fill)

FIGURE A-2A



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	S7	2	89.1
■	S3	3	88.2
◆	S5	4	92.6
▲	S6	4	92.6
▽	S4	5	91.8
❍	S2	6A	90.9
□	S6	7	90.3

Project Number: 1662333

Checked By: SMM

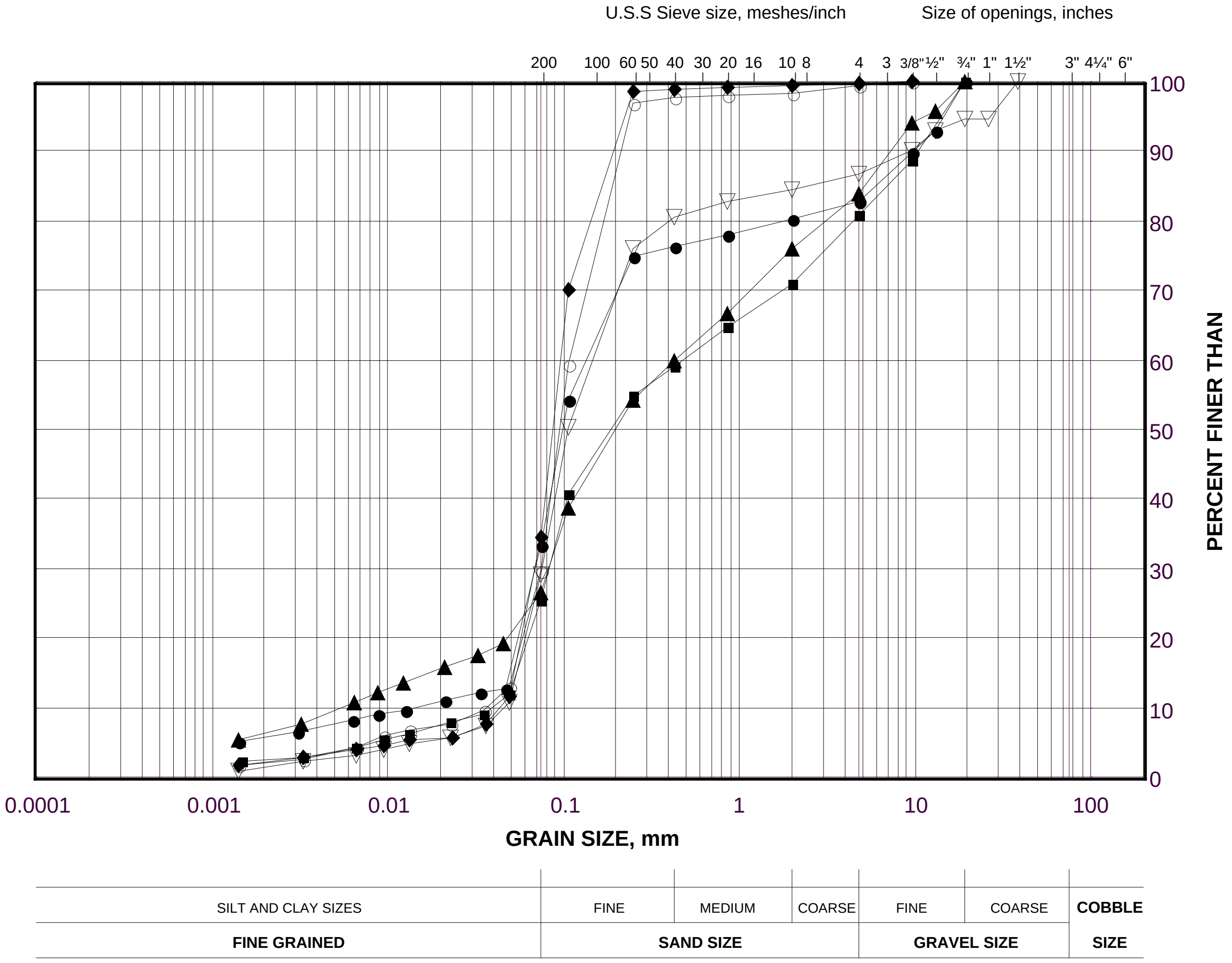
Golder Associates

Date: 05-Feb-19

GRAIN SIZE DISTRIBUTION

Silt and Sand to Silty Sand to Sand (Fill)

FIGURE A-2B



LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	NW3-3	2	89.5
■	PED-02	3	93.4
◆	PED-03	3	91.9
▲	NW3-2	3	93.4
▽	PED-03	6	89.6
❍	NW3-2	7	88.9

Project Number: 1662333

Checked By: SMM

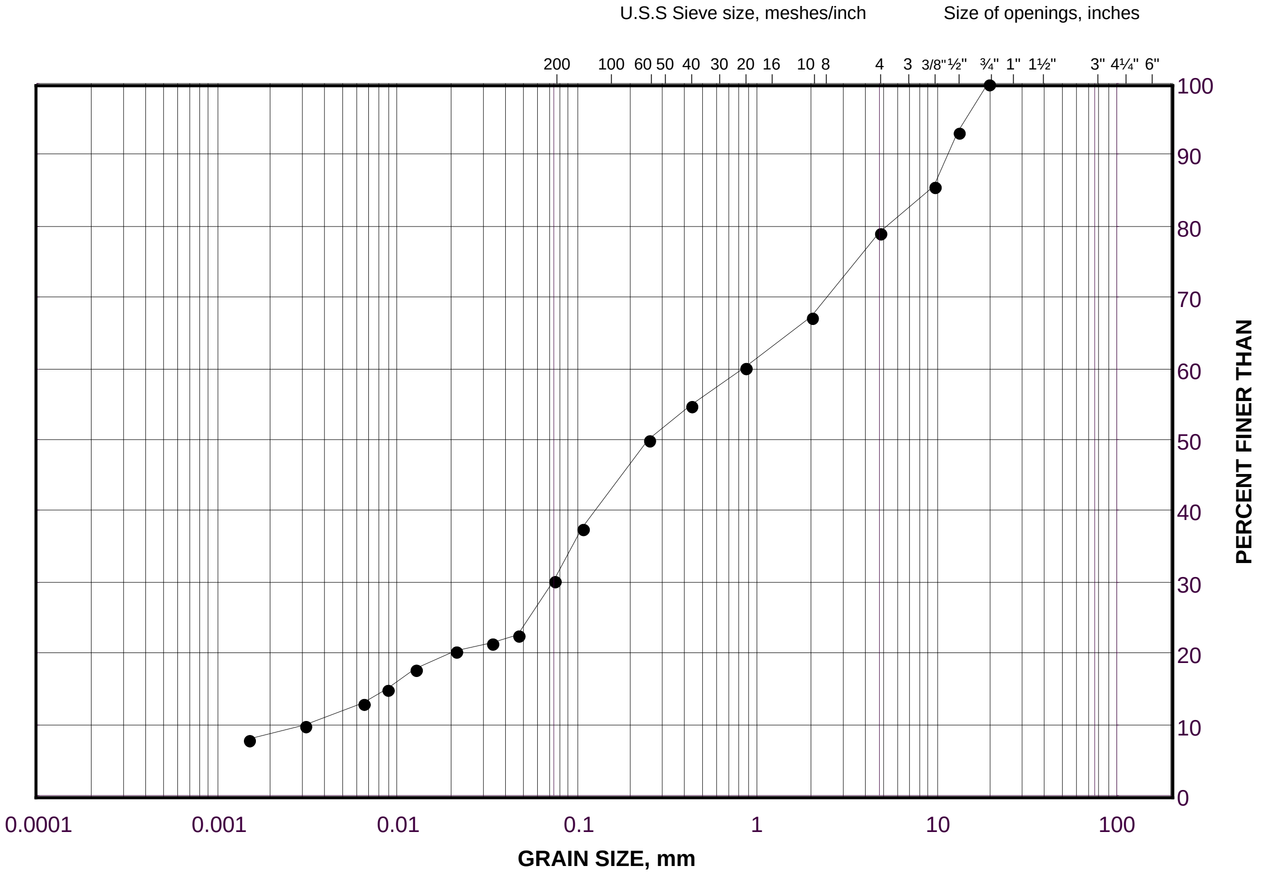
Golder Associates

Date: 05-Feb-19

GRAIN SIZE DISTRIBUTION

Gravelly Clayey Silt with Sand (Fill)

FIGURE A-3



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	S2	5	91.5

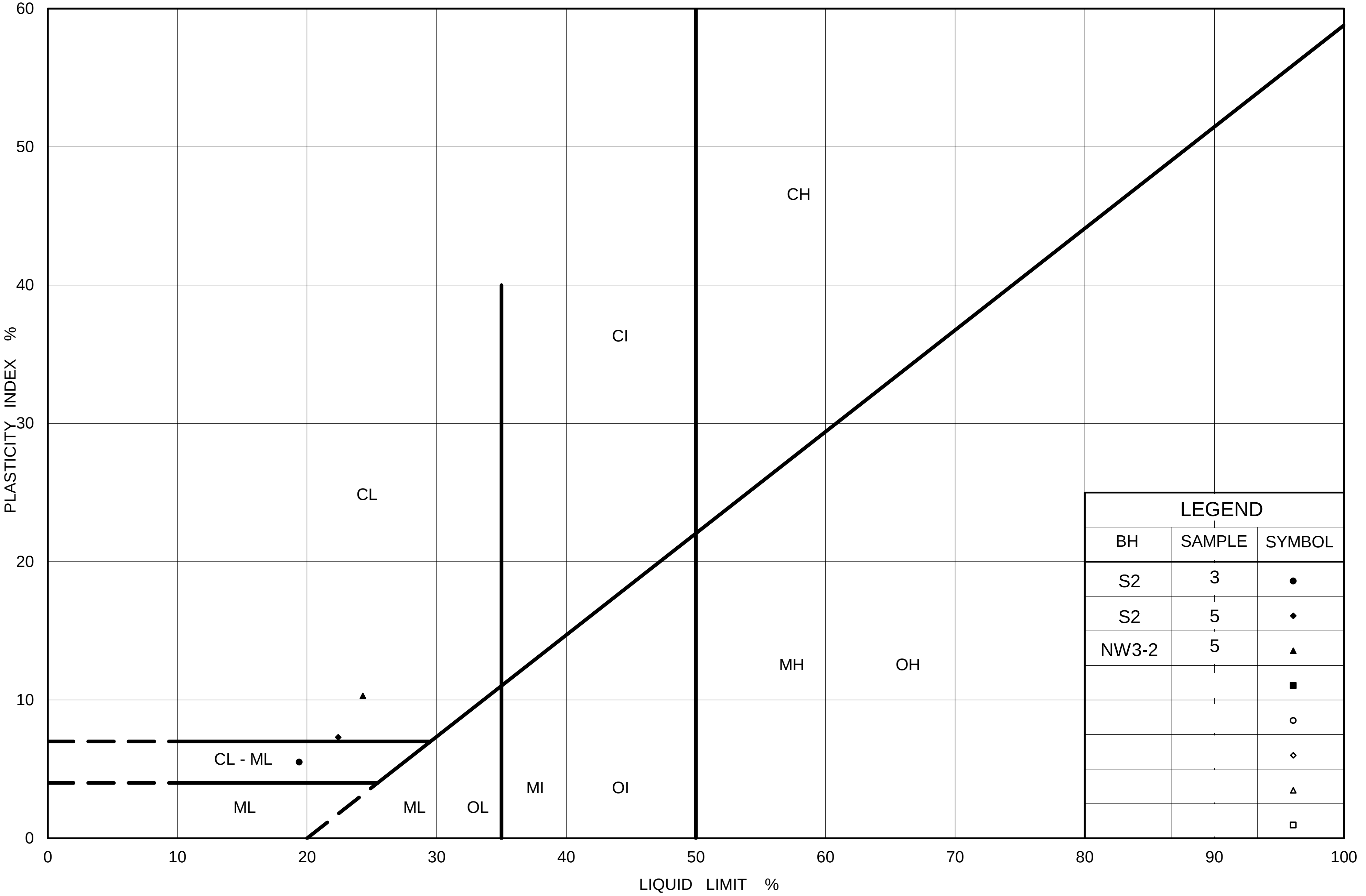
Project Number: 1662333

Checked By: SMM

Golder Associates

Date: 05-Feb-19

Oct 75, FF-S-21



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PLASTICITY CHART
Gravelly Clayey Silt with Sand to Clayey Silt with Sand (Fill)

Figure No. A-4

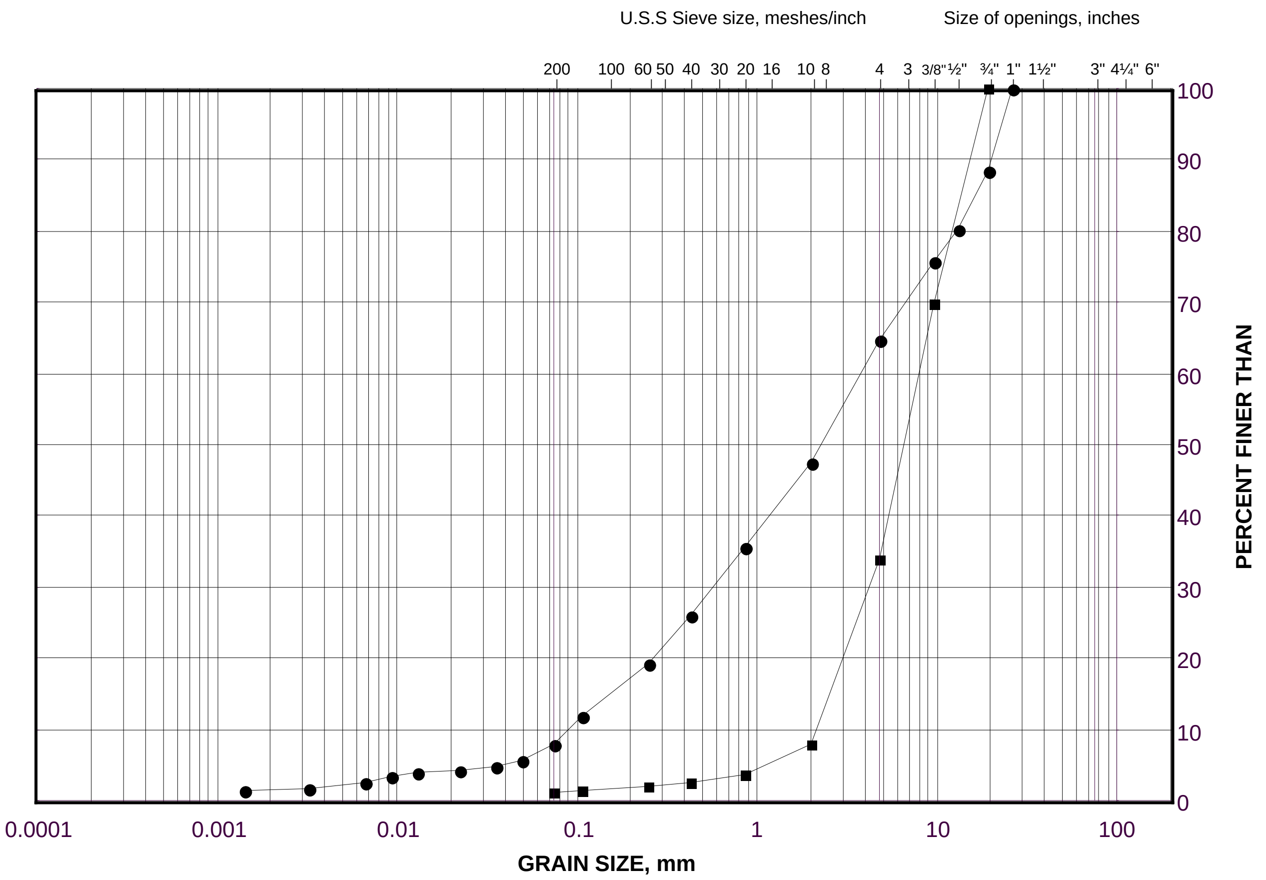
Project No. 1662333

Checked By: SMM

GRAIN SIZE DISTRIBUTION

Sand and Gravel

FIGURE A-5



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	S1	5	88.7
■	PED-02	7	90.5

Project Number: 1662333

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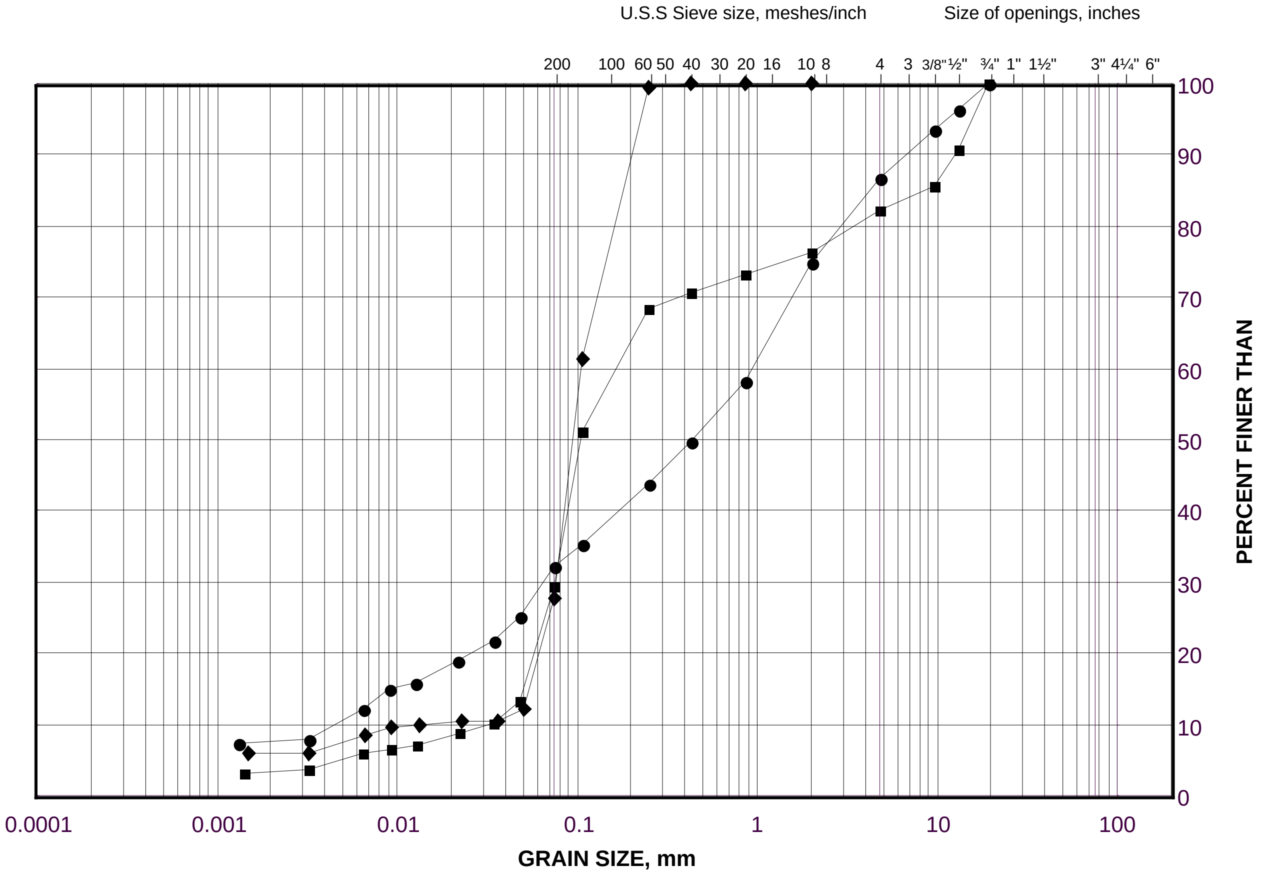
Date: 05-Feb-19

GRAIN SIZE DISTRIBUTION

Silty Sand

FIGURE A-6



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	S2	15	77.8
■	S1	3	90.2
◆	S2	8	88.5

Project Number: 1662333

Checked By: SMM

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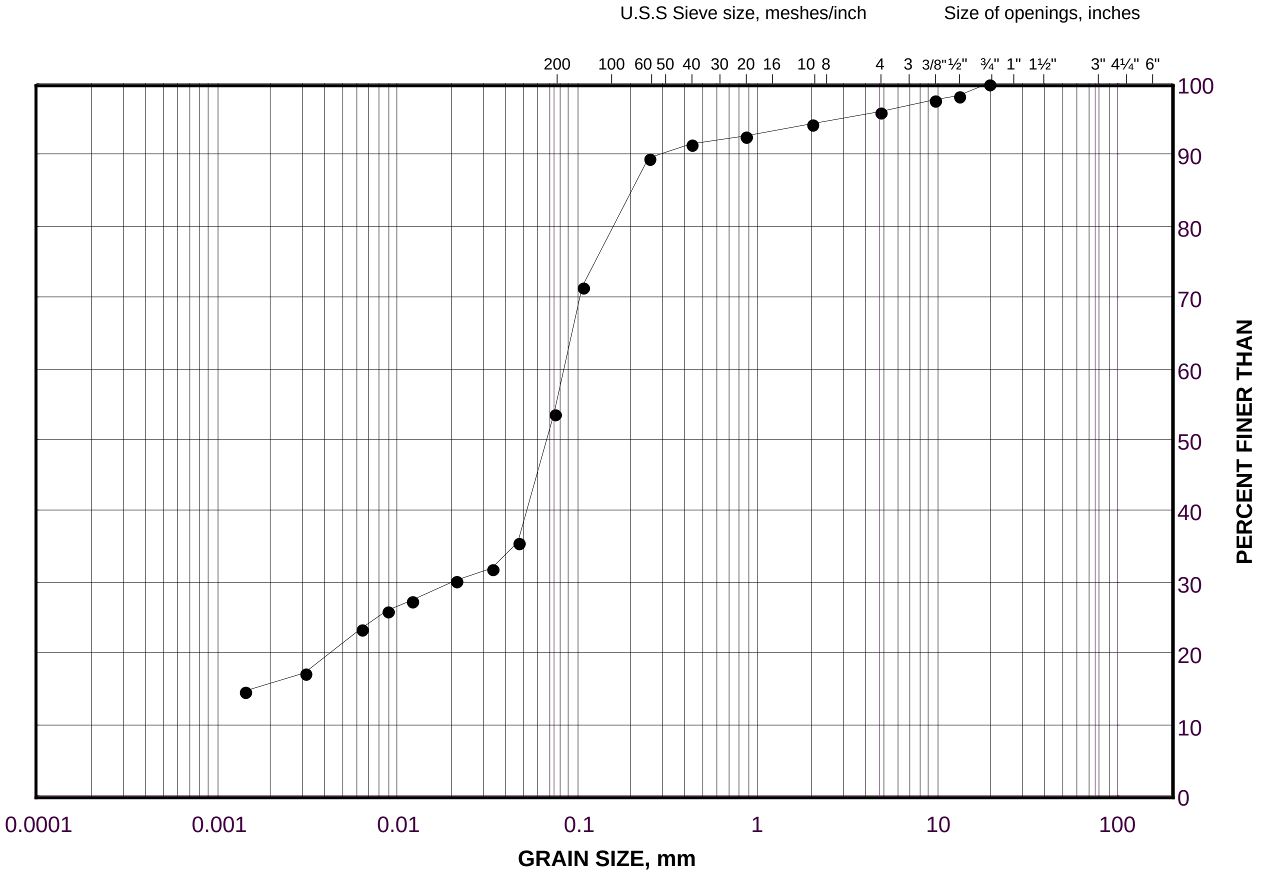
Date: 05-Feb-19

GRAIN SIZE DISTRIBUTION

Clayey Silt with Sand

FIGURE A-7



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	S7	4A	87.6

Project Number: 1662333

Checked By: SMM

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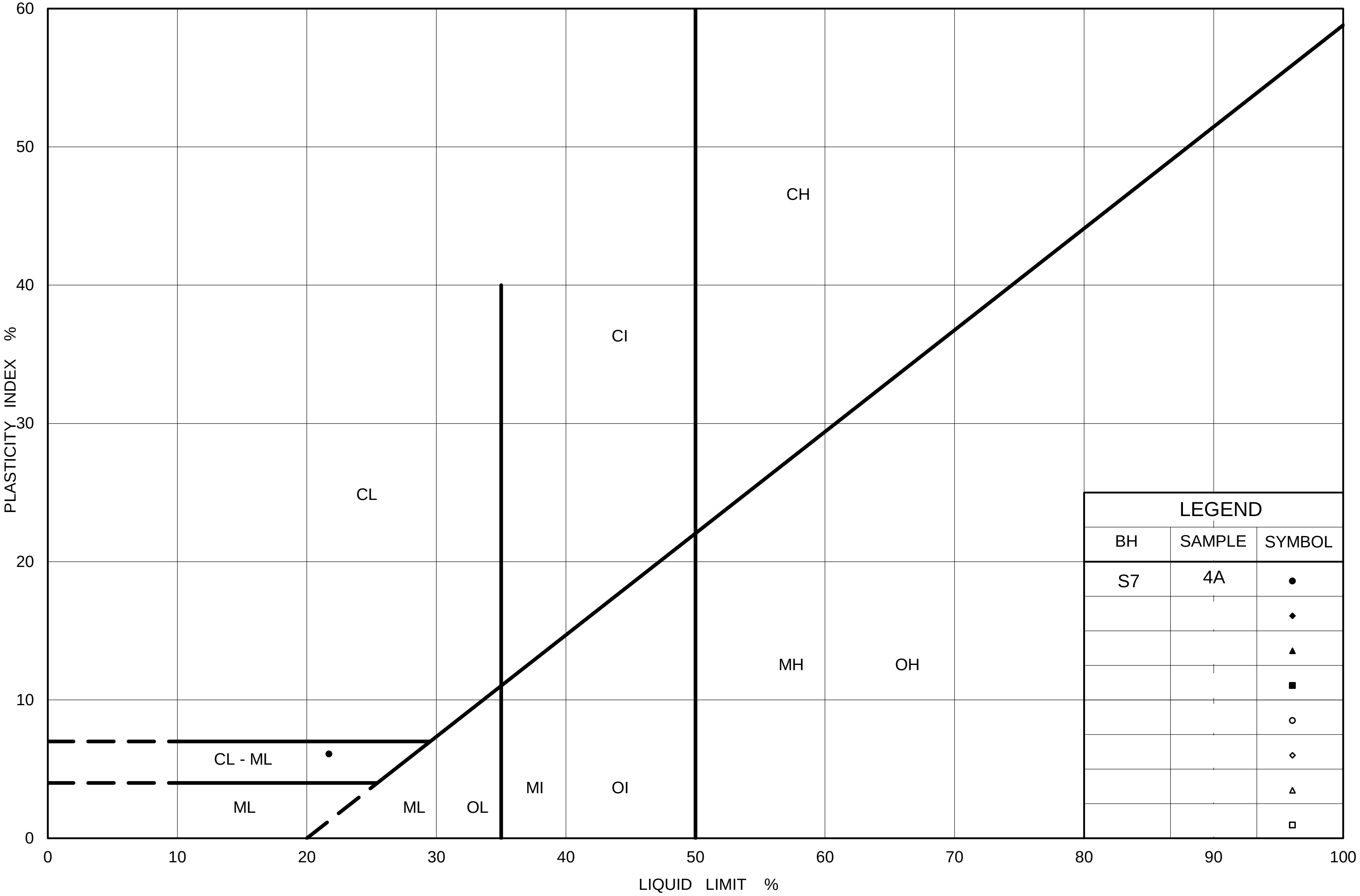
Date: 05-Feb-19

Oct 75, FF-S-21



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Ontario

PLASTICITY CHART
Clayey Silt with Sand

Figure No. A-8

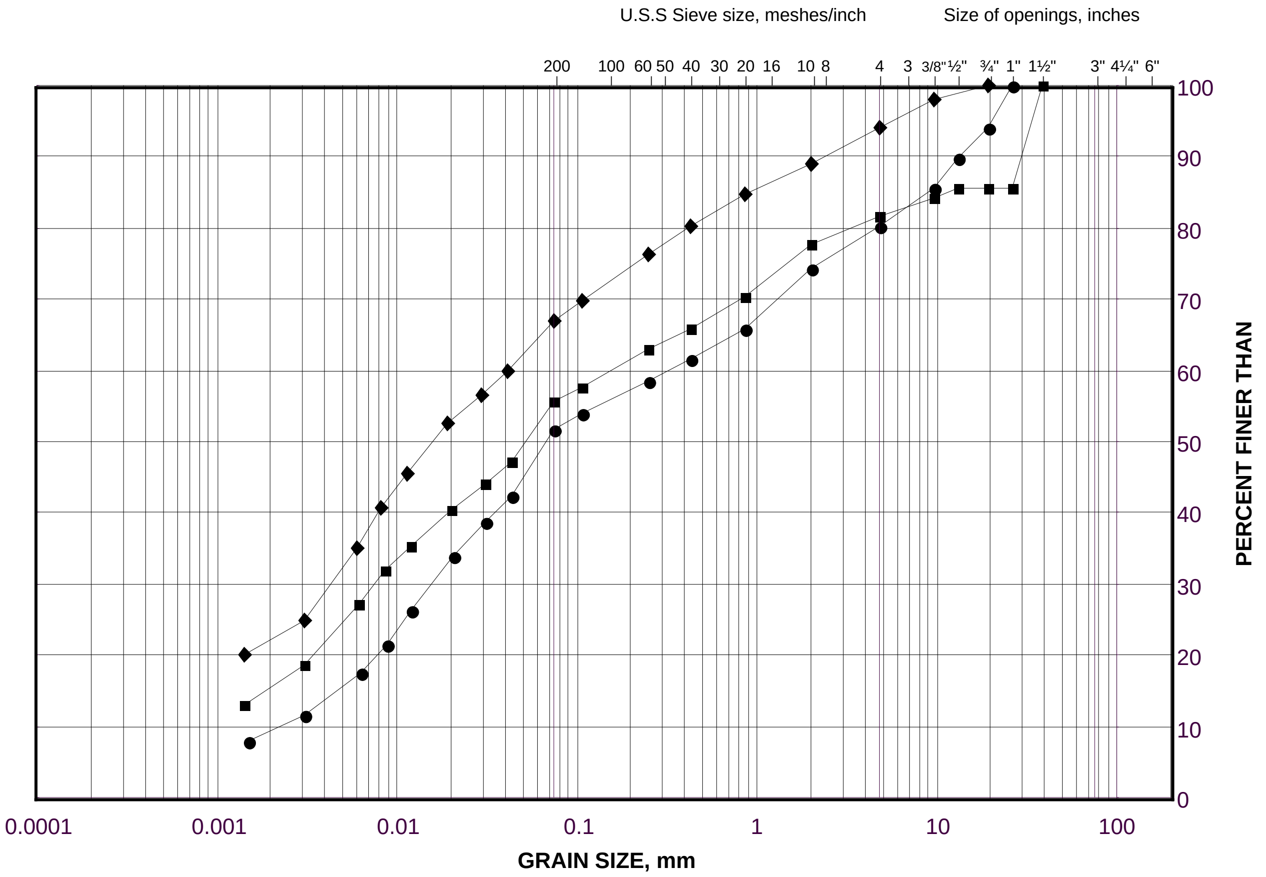
Project No. 1662333

Checked By: SMM

GRAIN SIZE DISTRIBUTION

Sandy Clayey Silt (Till)

FIGURE A-9A



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	S2	12	82.6
■	PED-03	8	87.3
◆	PED-02	9	87.5

Project Number: 1662333

Checked By: SMM

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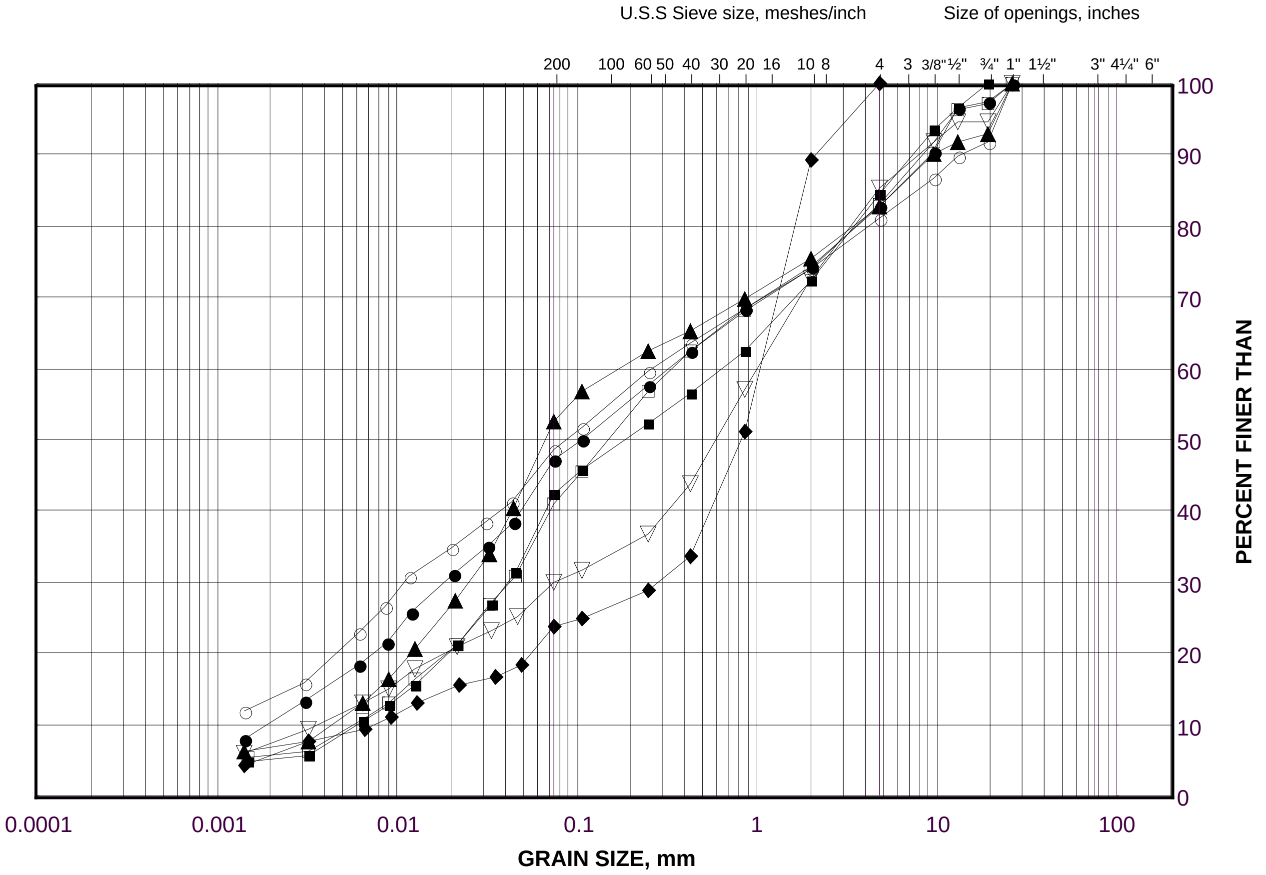
Date: 05-Feb-19

GRAIN SIZE DISTRIBUTION

Silt and Sand to Clayey Silt with Sand (Till)

FIGURE A-9B



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	S5	10	85.9
■	S4	10	85.8
◆	S3	10	79.1
▲	S5	12	83.1
▽	S5	14	79.8
○	S3	5	86.7
□	S3	7	83.8

Project Number: 1662333

Checked By: SMM

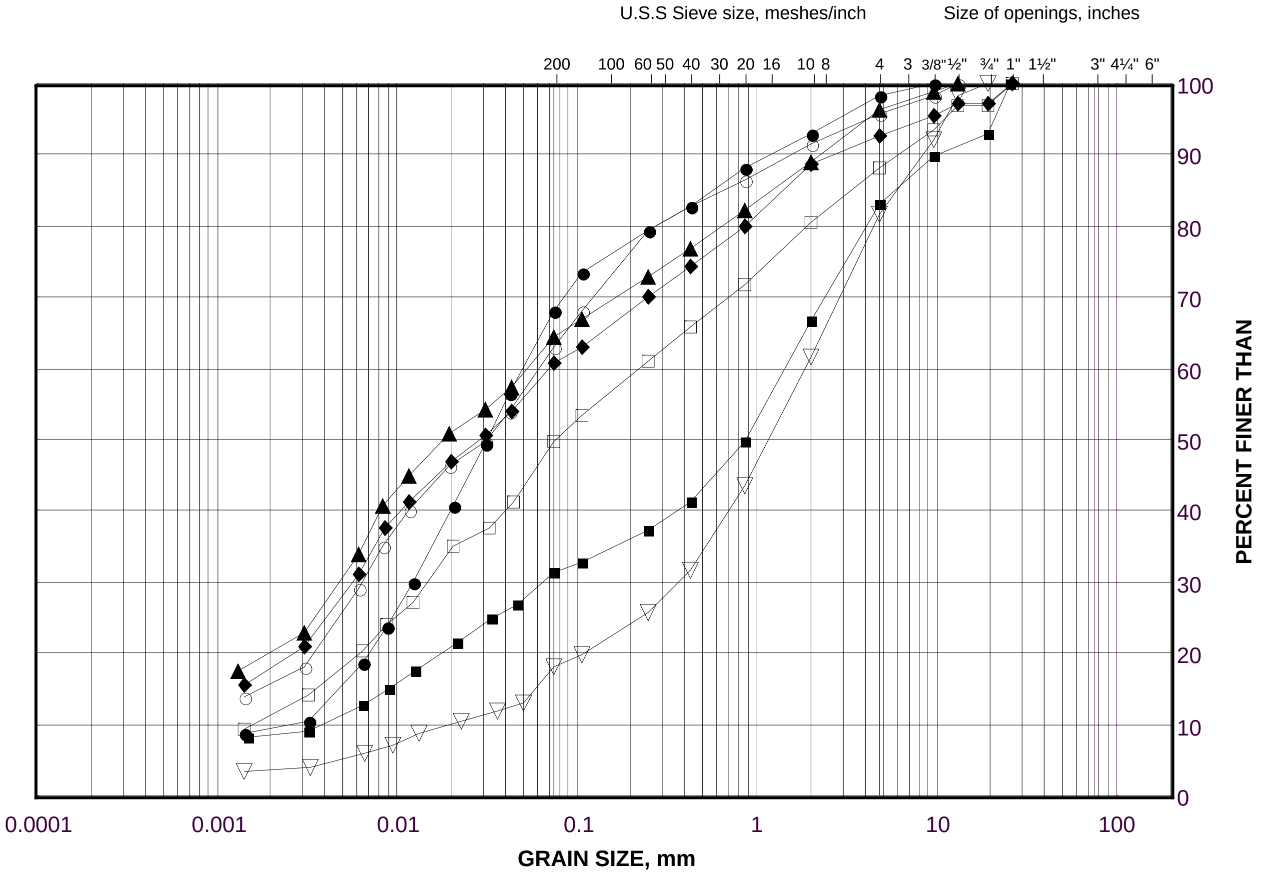
Golder Associates

Date: 05-Feb-19

GRAIN SIZE DISTRIBUTION

Silt and Sand to Clayey Silt with Sand (Till)

FIGURE A-9C



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	S6	12	82.7
■	PED-02	14	80.0
◆	NW3-3	4	88.0
▲	S7	5	86.8
▽	NW3-2A	5	79.8
○	S6	9	87.3
□	NW3-2	9	85.9

Project Number: 1662333

Checked By: SMM

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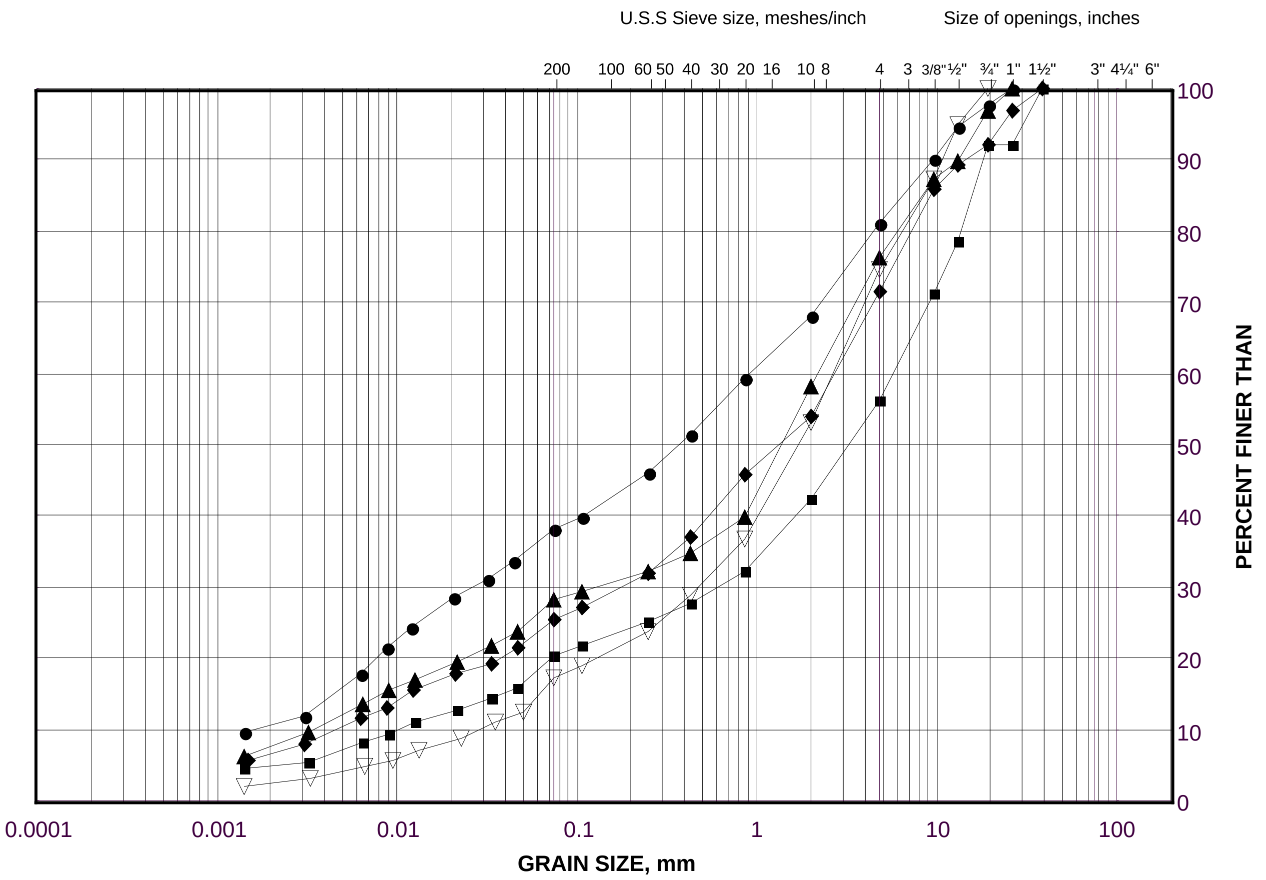
Date: 05-Feb-19

GRAIN SIZE DISTRIBUTION

Clayey Silt with Sand, Gravelly to with Gravel (Till)

FIGURE A-9D



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	S4	12	82.9
■	S6	14	80.6
◆	S4	15	78.3
▲	PED-03B	3	80.9
▽	NW3-2A	7	76.7

Project Number: 1662333

Checked By: SMM

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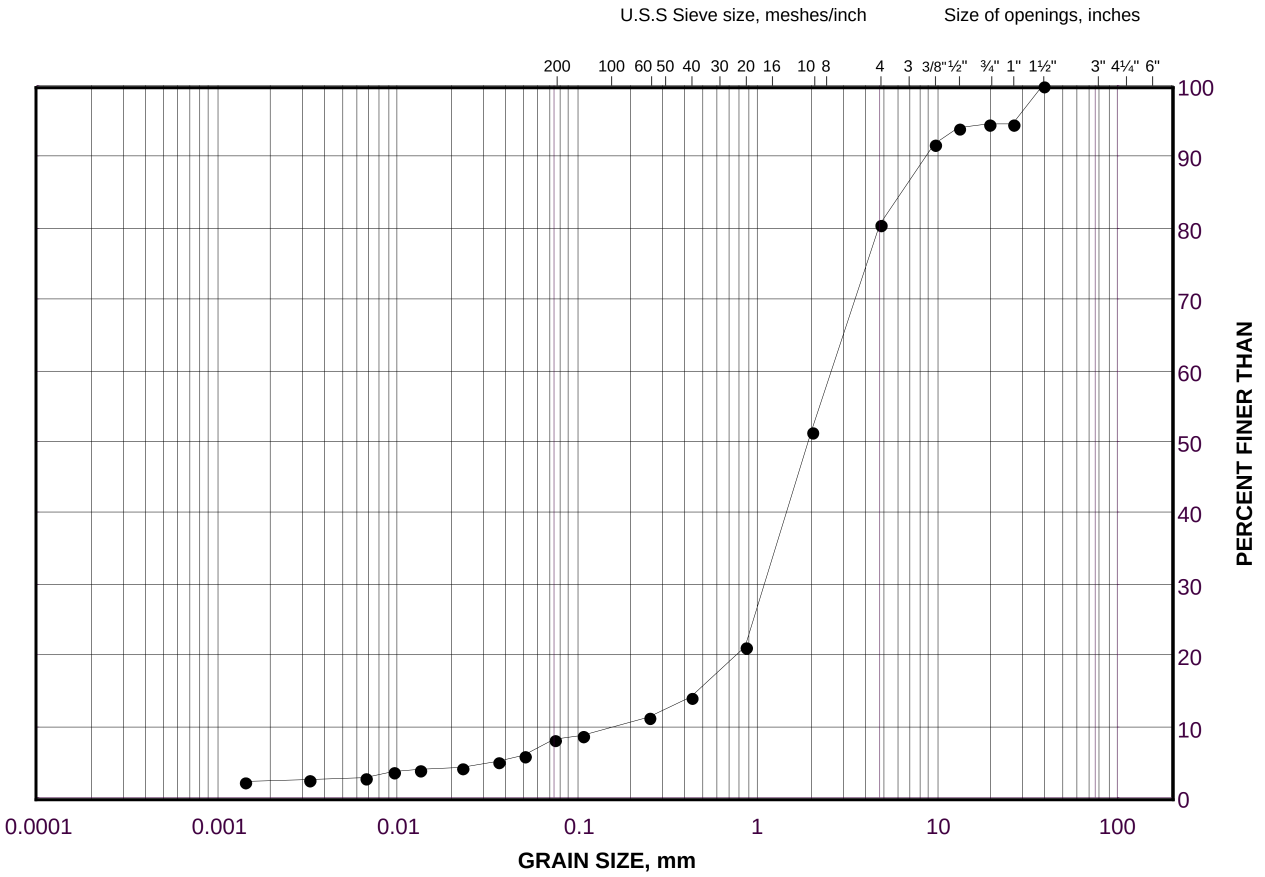
Date: 05-Feb-19

GRAIN SIZE DISTRIBUTION

Gravelly Sand (Till)

FIGURE A-9E



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	S3	13	74.5

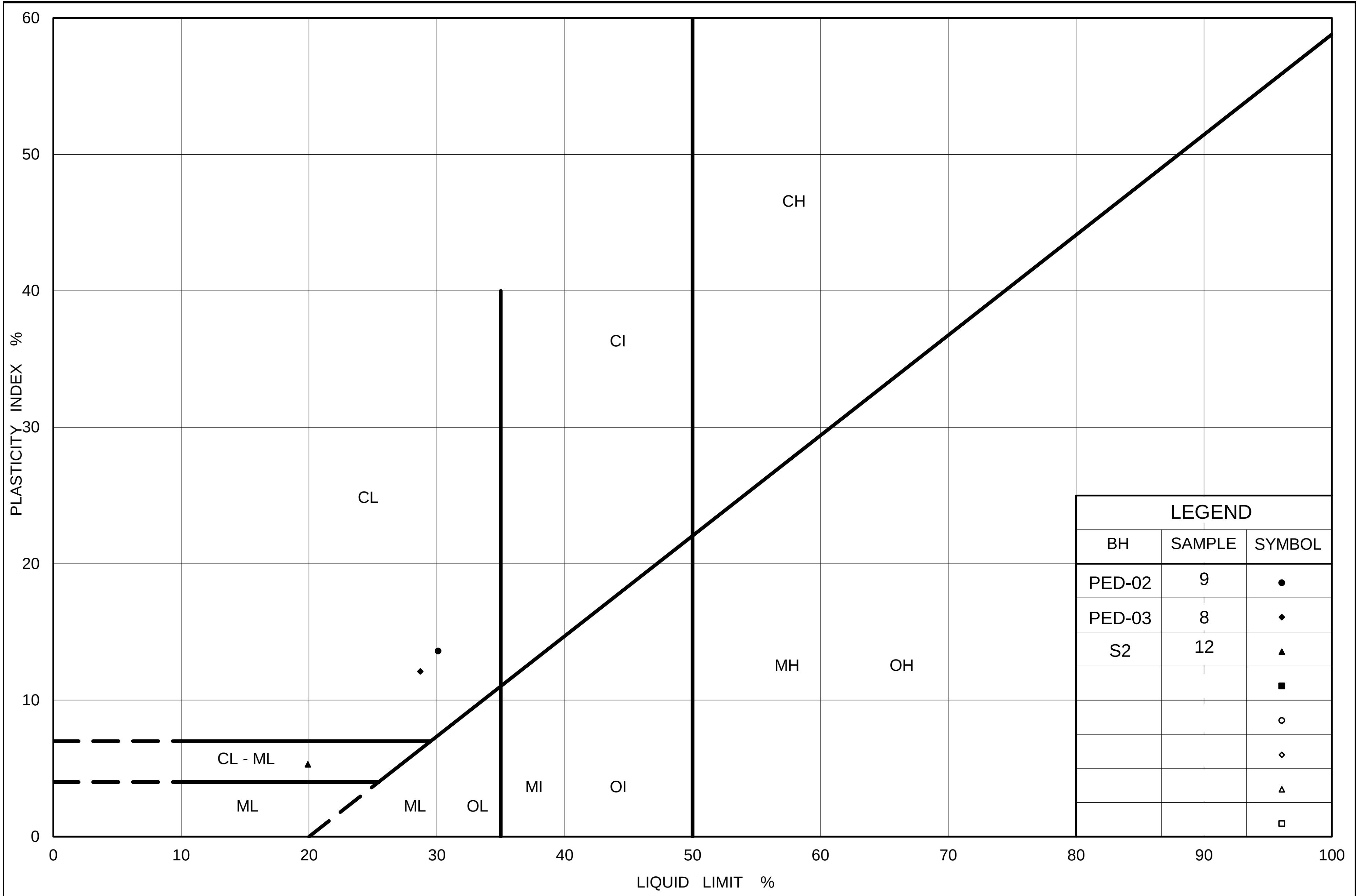
Project Number: 1662333

Checked By: SMM

Golder Associates

Date: 05-Feb-19

Oct 75, FF-S-21



BH	SAMPLE	SYMBOL
PED-02	9	●
PED-03	8	◆
S2	12	▲
		■
		○
		◇
		△
		□

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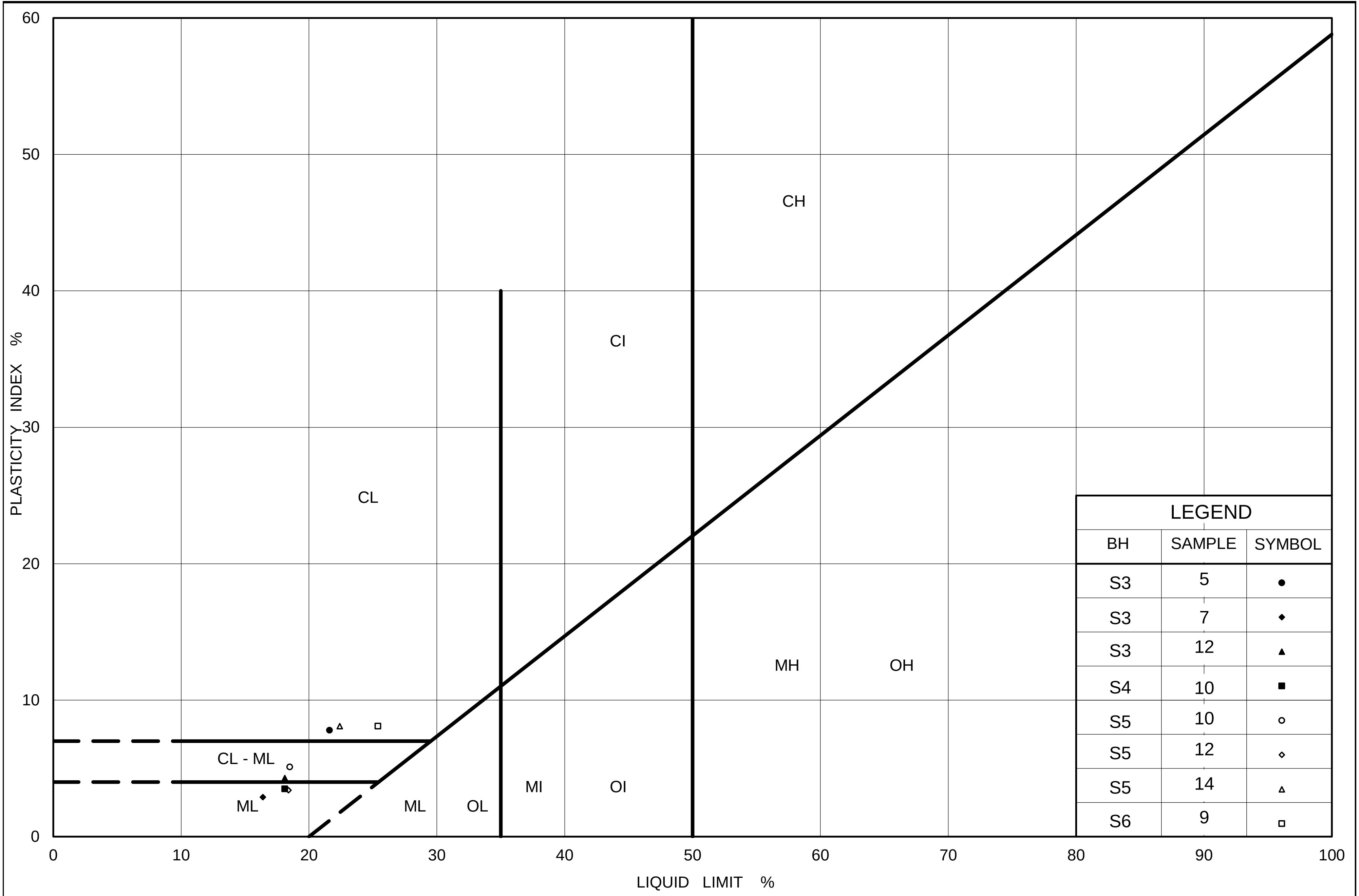
PLASTICITY CHART
Sandy Clayey Silt (Till)

Figure No. A-10A

Project No. 1662333

Checked By: SMM

Oct 75, FF-S-21



BH	SAMPLE	SYMBOL
S3	5	●
S3	7	◆
S3	12	▲
S4	10	■
S5	10	○
S5	12	◇
S5	14	△
S6	9	□

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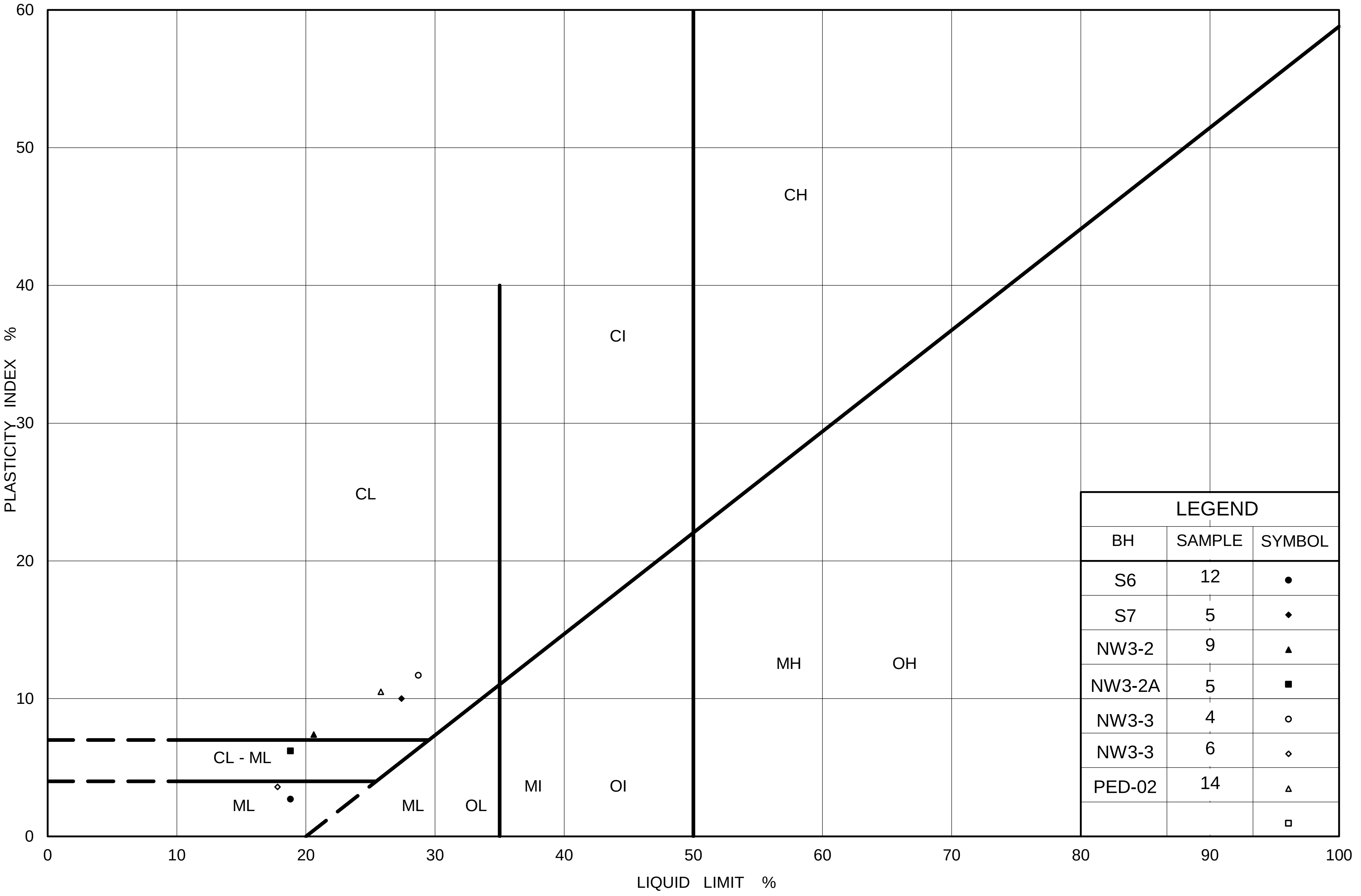
PLASTICITY CHART
Silt and Sand to Clayey Silt with Sand (Till)

Figure No. A-10B

Project No. 1662333

Checked By: SMM



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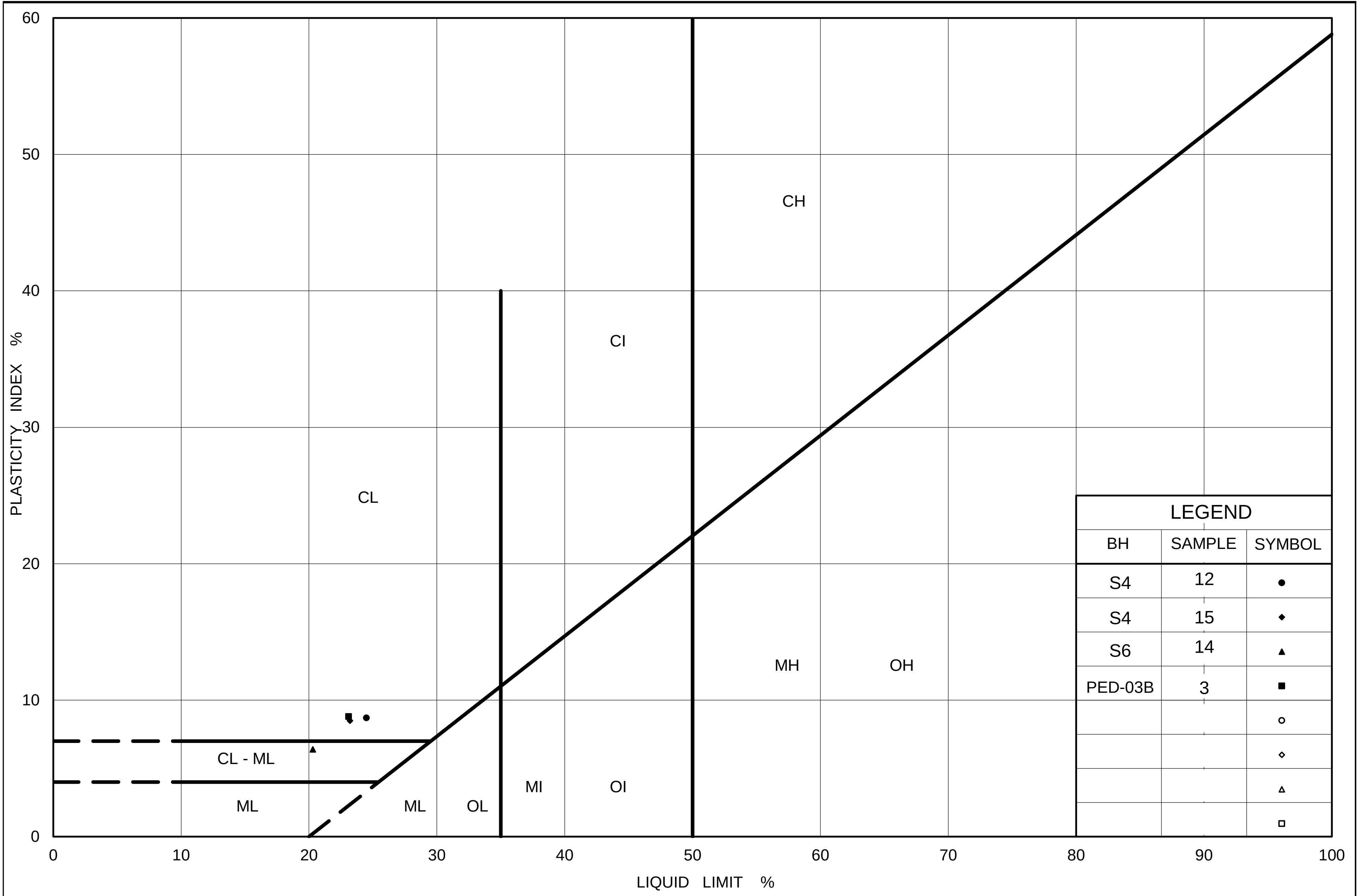
PLASTICITY CHART
Silt and Sand to Clayey Silt with Sand (Till)

Figure No. A-10C

Project No. 1662333

Checked By: SMM

Oct 75, FF-S-21



BH	SAMPLE	SYMBOL
S4	12	●
S4	15	◆
S6	14	▲
PED-03B	3	■
		○
		◇
		△
		□

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PLASTICITY CHART
Clayey Silt with Sand, Gravelly to with Gravel (Till)

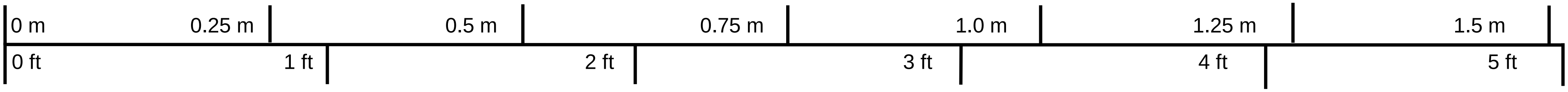
Figure No. A-10D

Project No. 1662333

Checked By: SMM

Start of Run No. 1 (10.72 m)

Start of Run No. 2 (12.23 m)



Scale

PROJECT **MTO Assignment 2015-E-0033: Detail Design for the widening/rehab/realignment of QEW Between Mississauga Road and Hurontario Street**

TITLE
Core Photograph
Borehole PED-03 (10.72 m to 13.75 m)

GOLDER

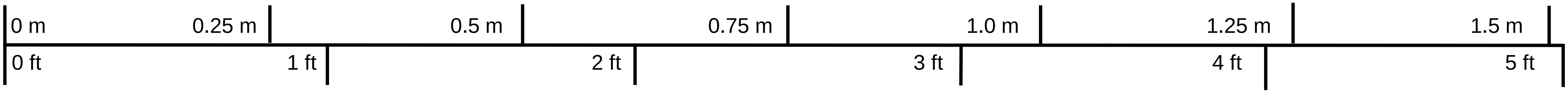
PROJECT No. 1662333			FILE No. ----		
DRAFT	KMG	Feb 2019	SCALE	AS SHOWN	VER. 1.
CADD	--		**FIGURE A-11**		
CHECK	SMM	Apr 2019			
REVIEW	JMAC	Apr 2019			

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

Start of Run No. 1 (14.80 m)

Start of Run No. 2 (15.27 m)

Start of Run No. 3 (16.75 m)



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 PED-03B (14.80 m to 17.77 m)

GOLDER

PROJECT No. 1662333		
DRAFT	KMG	Feb 2019
CADD	--	
CHECK	SMM	Apr 2019
REVIEW	JMAC	Apr 2019

FILE No. ----

SCALE AS SHOWN VER. 1.

FIGURE A-12

APPENDIX B

Record of Borehole and Drillhole Sheets, Bedrock Core Photographs and Geotechnical Laboratory Results for Kenollie Creek Culvert

LIST OF SYMBOLS

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

I. GENERAL

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

II. STRESS AND STRAIN

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

III. SOIL PROPERTIES

(a) Index Properties

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

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

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(d) Shear Strength

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

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

2 shear strength = (compressive strength)/2

LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

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

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

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

Dynamic Cone Penetration Resistance; N_d:

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

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

Piezo-Cone Penetration Test (CPT)

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

III. SOIL DESCRIPTION

(a) Non-Cohesive (Cohesionless) Soils

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

(b) Cohesive Soils

Consistency

	c_u, s_u	
	kPa	**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^{o} angle is horizontal.

Description and Notes

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

Abbreviations

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

FIELD ESTIMATION OF ROCK HARDNESS

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

Notes:

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

Reference:

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

Hoek, E., Kaiser, P.K., Bawden, W.F., 1995. "Support of Underground Excavations in Hard Rock", Balkema, Rotterdam.

ROCK WEATHERING CLASSIFICATION

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

Reference:

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

RECORD OF BOREHOLE No K1

SHEET 1 OF 1

METRIC

PROJECT 1662333

G.W.P. 2002-13-00

DIST Central HWY QEW

DATUM Geodetic

LOCATION N 4824728.9; E 296200.2 MTM NAD 83 ZONE 10 (LAT. 43.562439; LONG. -79.606453)

BOREHOLE TYPE CME 75, 114 mm I.D., Hollow Stem Augers

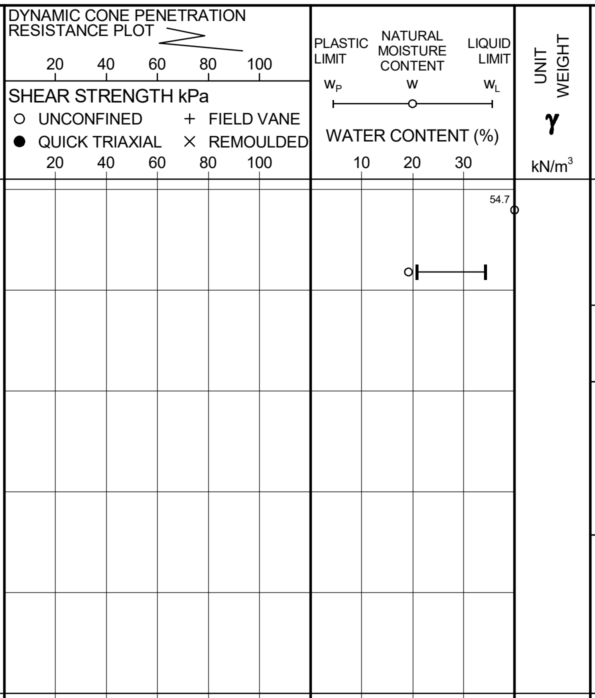
DATE November 29, 2018

ORIGINATED BY ACM

COMPILED BY DM

CHECKED BY SMM

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
90.1	GROUND SURFACE						
0.0	TOPSOIL Soft Brown Moist	1	SS	3		90	
89.3							
89.0 1.1	CLAYEY SILT, some sand, some gravel, some shale fragments (RESIDUAL SOIL) Hard Grey Wet	2A 2B	SS	50/0.08	∇	89	20 19 47 14
		3	SS	100/0.28			
	SHALE (BEDROCK) Grey Bedrock cored from a depth of 1.3 m to 5.1 m For bedrock coring details, refer to Record of Drillhole K1 - Auger grinding from 1.1 m to 1.2 m	1	RC	REC 54%		88	RQD = 28%
		2	RC	REC 100%		87	RQD = 82%
		3	RC	REC 100%		86	RQD = 68%
85.0 5.1	END OF BOREHOLE					85	



NOTES:

1. Water level measured at a depth of 1.2 m below ground surface (Elev 88.9 m) upon completion of soil drilling, prior to bedrock coring.

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

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

PROJECT: 1662333

RECORD OF DRILLHOLE: K1

SHEET 1 OF 1

LOCATION: N 4824728.9 ;E 296200.2

INCLINATION: -90° AZIMUTH: ---

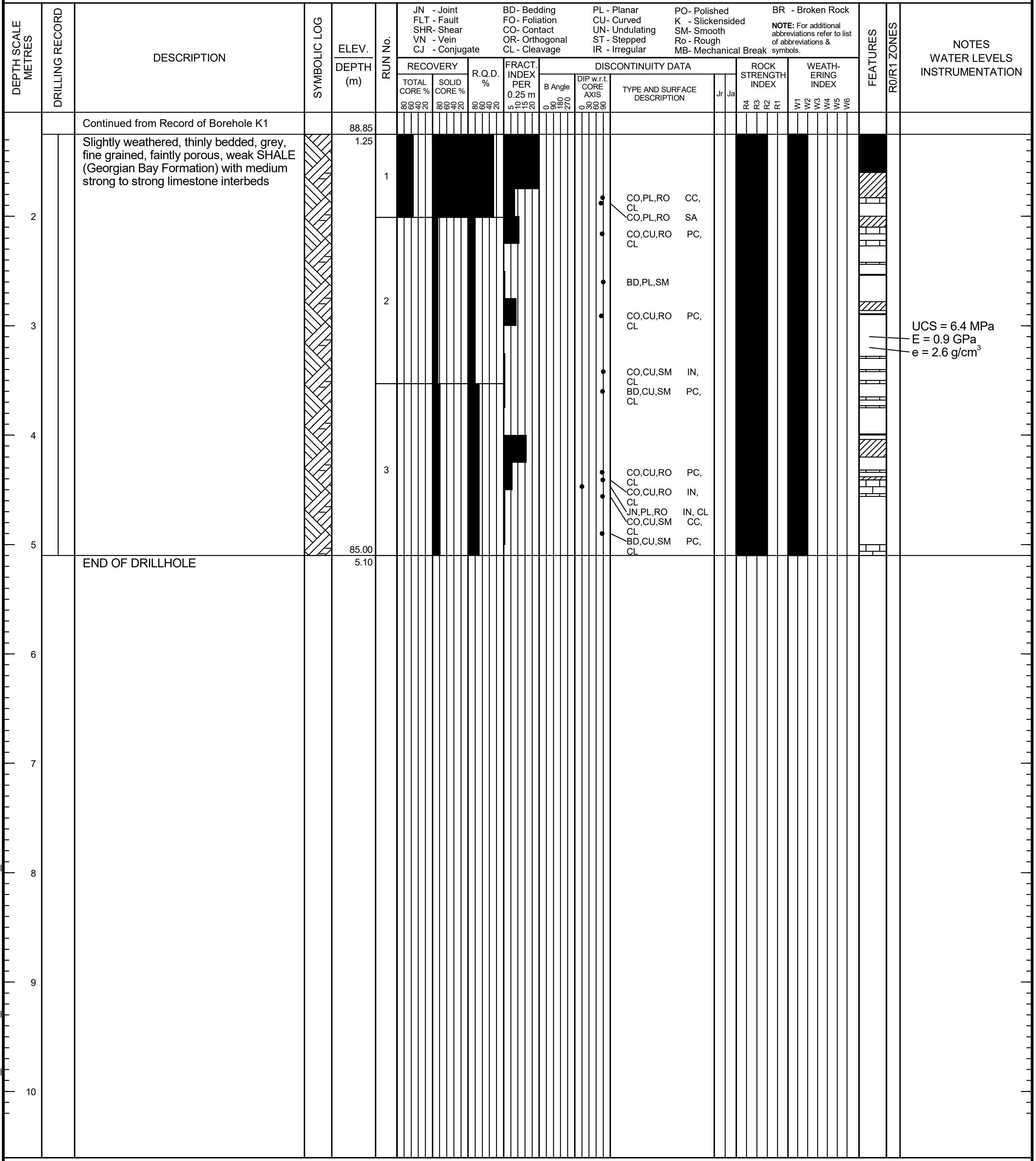
DRILLING DATE: November 29, 2018

DRILL RIG: CME 75

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	



DESCRIPTION	ELEV. DEPTH (m)	TYPE AND SURFACE DESCRIPTION	NOTES WATER LEVELS INSTRUMENTATION
Continued from Record of Borehole K1	88.85		
Slightly weathered, thinly bedded, grey, fine grained, faintly porous, weak SHALE (Georgian Bay Formation) with medium strong to strong limestone interbeds	1.25	CO,PL,RO CC, CL CO,PL,RO SA CO,CU,RO PC, CL BD,PL,SM CO,CU,RO PC, CL CO,CU,SM IN, CL BD,CU,SM PC, CL CO,CU,RO PC, CL CO,CU,RO IN, CL JN,PL,RO IN, CL CO,CU,SM CC, CL BD,CU,SM PC, CL	UCS = 6.4 MPa E = 0.9 GPa e = 2.6 g/cm^3
	85.00		
END OF DRILLHOLE	5.10		

FEATURES LEGEND

▨	BROKEN CORE
▩	CLAY SEAM
▦	LIMESTONE
■	LOST CORE

DEPTH SCALE

1 : 50

GOLDER

LOGGED: ACM

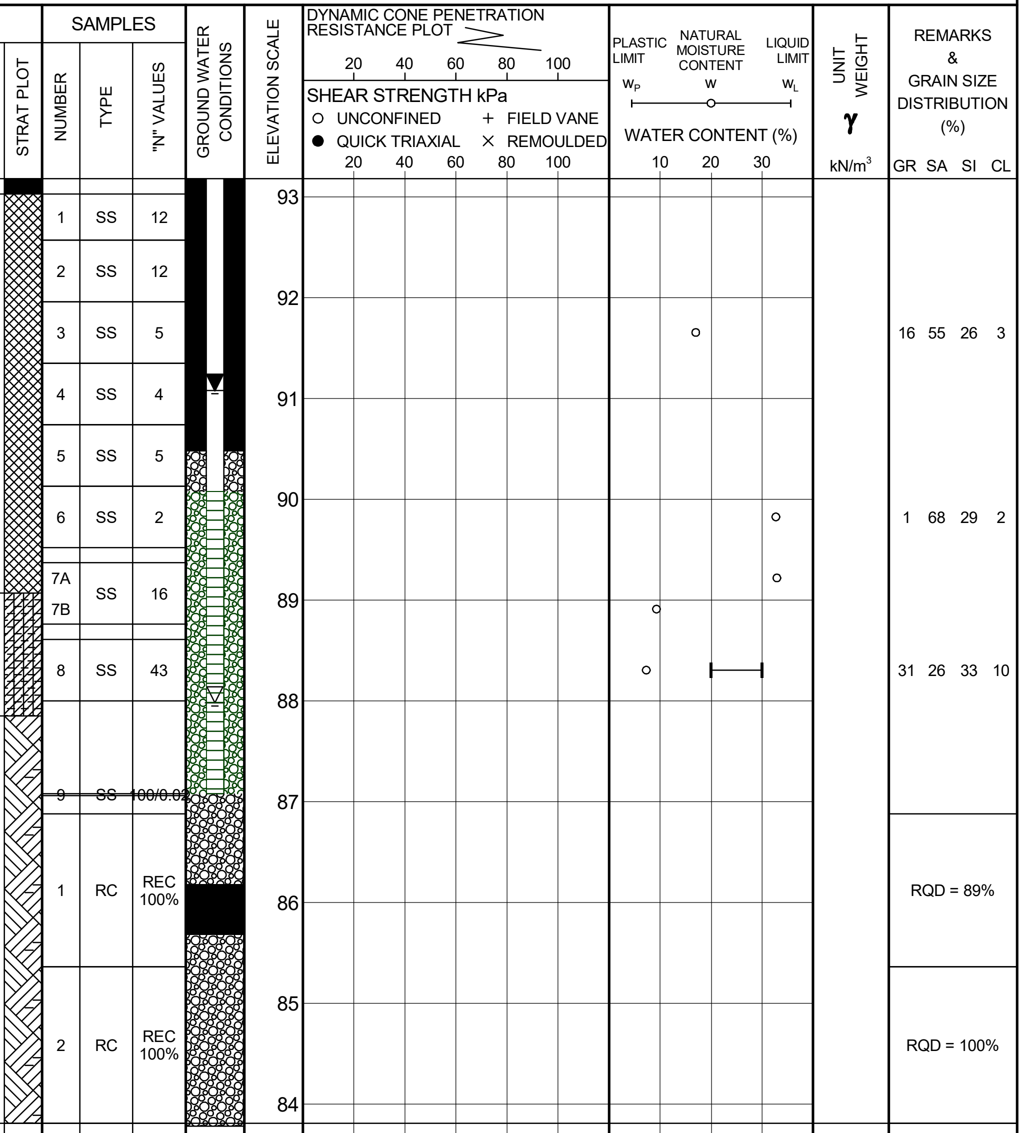
CHECKED: DPM

RECORD OF BOREHOLE No K2

SHEET 1 OF 1

METRIC

PROJECT 1662333
G.W.P. 2002-13-00
LOCATION N 4824716.6; E 296216.3 MTM NAD 83 ZONE 10 (LAT. 43.562329; LONG. -79.606253)
ORIGINATED BY ACM
DIST Central HWY QEW
BOREHOLE TYPE CME 75, 210 mm O.D., Hollow Stem Augers
COMPILED BY JMP
DATUM Geodetic
DATE September 14, 2018
CHECKED BY SMM



ELEV DEPTH	DESCRIPTION
93.2	GROUND SURFACE
0.0	ASPHALT (150 mm)
0.2	Silty sand, trace to some gravel, trace clay (FILL) Very loose to compact Brown Moist to wet - Asphalt fragments encountered at 1.1 m - Wood chips present from 3.7 m to 4.1 m depth
89.1 4.1	Sandy CLAYEY SILT with GRAVEL, some shale fragments (RESIDUAL SOIL) Very stiff to hard Grey Moist
87.9 5.3	SHALE (BEDROCK) Grey Bedrock cored from a depth of 6.3 m to 9.4 m For bedrock coring details, refer to Record of Drillhole K2
83.8 9.4	END OF BOREHOLE

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	12	
2	SS	12	
3	SS	5	16 55 26 3
4	SS	4	
5	SS	5	
6	SS	2	1 68 29 2
7A 7B	SS	16	
8	SS	43	31 26 33 10
9	SS	100/0.02	
1	RC	REC 100%	RQD = 89%
2	RC	REC 100%	RQD = 100%

NOTES:

1. Water level measured at a depth of 5.2 m below ground surface (Elev 88.0 m) upon completion of soil drilling, prior to bedrock coring.

2. Water level measured in piezometer at a depth of 2.1 m below ground surface (Elev. 91.1 m) on December 17, 2018.

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

○$^{3\%}$ STRAIN AT FAILURE

PROJECT: 1662333

RECORD OF DRILLHOLE: K2

SHEET 1 OF 1

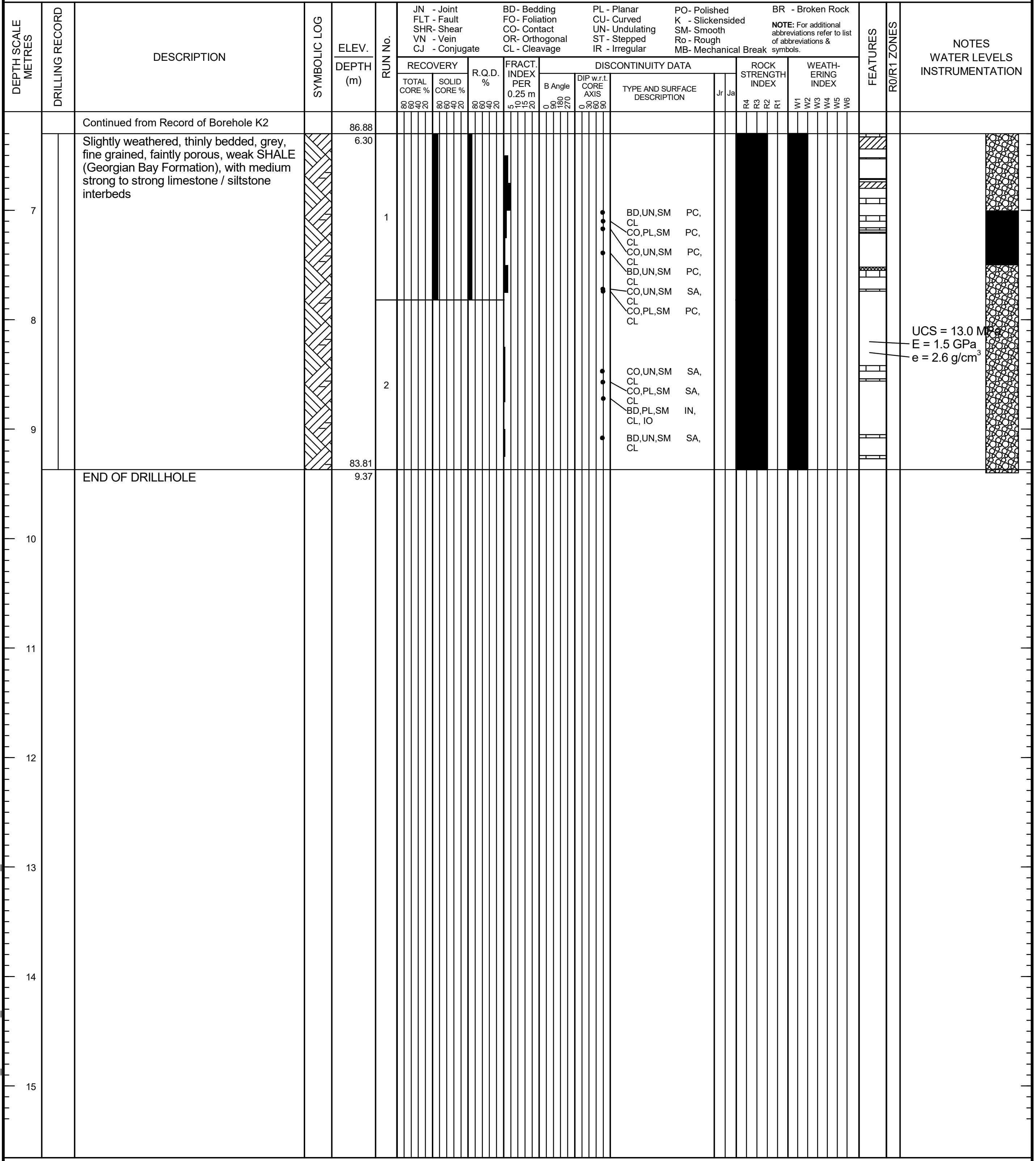
LOCATION: N 4824716.6 ;E 296216.3

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: September 14, 2018
DRILL RIG: CME 75
DRILLING CONTRACTOR: Davis Drilling

DATUM: Geodetic

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



DESCRIPTION	ELEV. / DEPTH (m)	RUN No.	TYPE AND SURFACE DESCRIPTION	NOTES WATER LEVELS INSTRUMENTATION
Continued from Record of Borehole K2	86.88			
Slightly weathered, thinly bedded, grey, fine grained, faintly porous, weak SHALE (Georgian Bay Formation), with medium strong to strong limestone / siltstone interbeds	6.30	1	BD,UN,SM PC, CL CO,PL,SM PC, CL CO,UN,SM PC, CL BD,UN,SM PC, CL CO,UN,SM SA, CL CO,PL,SM PC, CL	
		2	CO,UN,SM SA, CL CO,PL,SM SA, CL BD,PL,SM IN, CL, IO BD,UN,SM SA, CL	UCS = 13.0 MPa E = 1.5 GPa e = 2.6 g/cm^3
	83.81			
END OF DRILLHOLE	9.37			

FEATURES LEGEND

▨ BROKEN CORE	▩ CLAY SEAM	▦ LIMESTONE	■ LOST CORE

DEPTH SCALE
1 : 50

GOLDER

LOGGED: ACM
CHECKED: JP

RECORD OF BOREHOLE No K3

SHEET 1 OF 2

METRIC

PROJECT 1662333

G.W.P. 2002-13-00

DIST Central HWY QEW

DATUM Geodetic

LOCATION N 4824703.7; E 296236.7 MTM NAD 83 ZONE 10 (LAT. 43.562222; LONG. -79.606004)

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

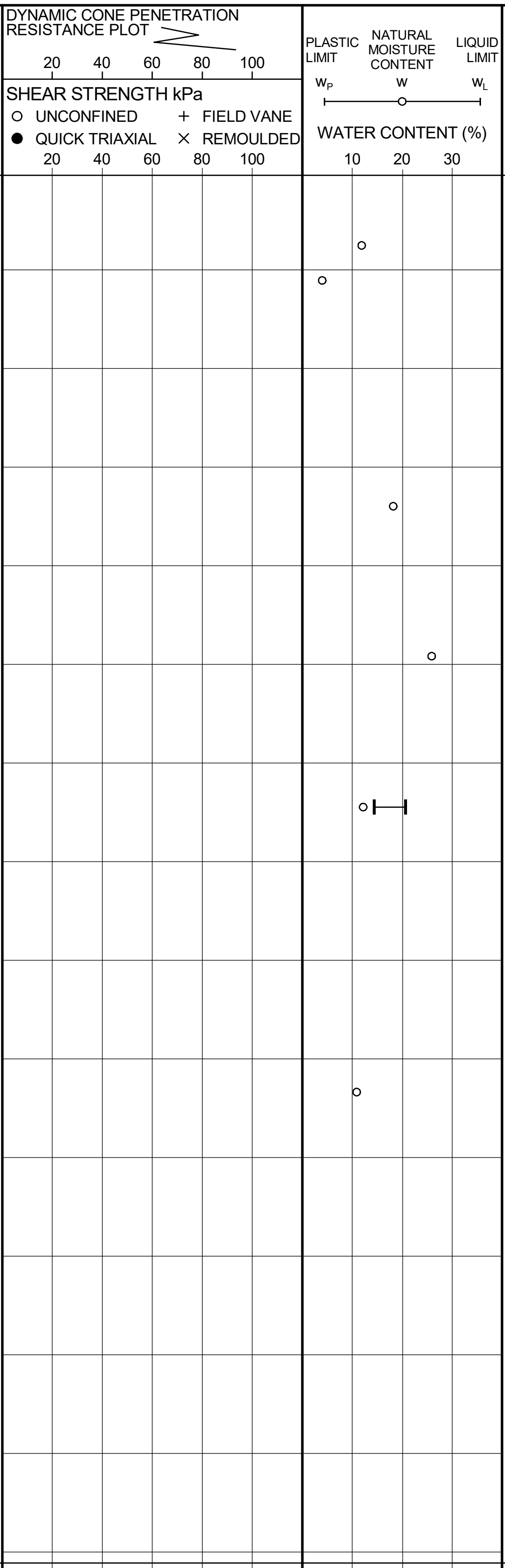
DATE September 17, 2018

ORIGINATED BY ACM

COMPILED BY JMP

CHECKED BY SMM

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL	REMARKS
95.0	GROUND SURFACE											
0.0	ASPHALT (150 mm)											
0.2	Sand and gravel (FILL) Dense Brown Moist	1A 1B	SS	38								
94.2 0.8	Silty sand, trace clay (FILL) Very loose to dense Brown, oxidation staining from 3.1 m to 3.7 m Moist to wet below 3.8 m	2	SS	35		94						
		3	SS	5		93						
		4	SS	3		92						
		5	SS	9		91		0	72	26	2	
		6	SS	1								
		7	SS	WH		90		0	78	20	2	
89.4 5.6	Sandy CLAYEY SILT with GRAVEL (TILL) Very stiff to hard Grey Moist to wet - Auger grinding from 6.4 m to 6.6 m					89						
		8	SS	17		88		34	29	29	8	
		9	SS	100/0.28		87						
	- Trace shale fragments from 9.1 m to 10.0 m - Auger grinding at 9.4 m to 9.8 m	10	SS	100/0.10	∇	86						
85.0 10.0	SHALE (BEDROCK) Grey Bedrock cored from a depth of 10.7 m to 14.1 m For bedrock coring details, refer to Record of Drillhole K3	11	SS	100/0.13		85						
		1	RC	REC 60%		84						RQD = 60%
		2	RC	REC 100%		83						RQD = 100%
		3	RC	REC 100%		82 81						RQD = 96%
80.9 14.1	END OF BOREHOLE											



Continued Next Page

+³, ×³: Numbers refer to Sensitivity

○³% STRAIN AT FAILURE

GOLDER

Foundation Design

RECORD OF BOREHOLE No K3

SHEET 2 OF 2

METRIC

PROJECT 1662333
G.W.P. 2002-13-00
LOCATION N 4824703.7; E 296236.7 MTM NAD 83 ZONE 10 (LAT. 43.562222; LONG. -79.606004)
ORIGINATED BY ACM
DIST Central HWY QEW
BOREHOLE TYPE CME 55, 108 mm I.D., Hollow Stem Augers
COMPILED BY JMP
DATUM Geodetic
DATE September 17, 2018
CHECKED BY SMM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION --- *CONTINUED FROM PREVIOUS PAGE* ---	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	NOTES: 1. Water level measured at a depth of 9.8 m below ground surface (Elev. 85.2 m) upon completion of soil drilling, prior to bedrock coring.										

+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 2/13/19

PROJECT: 1662333

RECORD OF DRILLHOLE: K3

SHEET 1 OF 1

LOCATION: N 4824703.7 ;E 296236.7

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: September 17, 2018

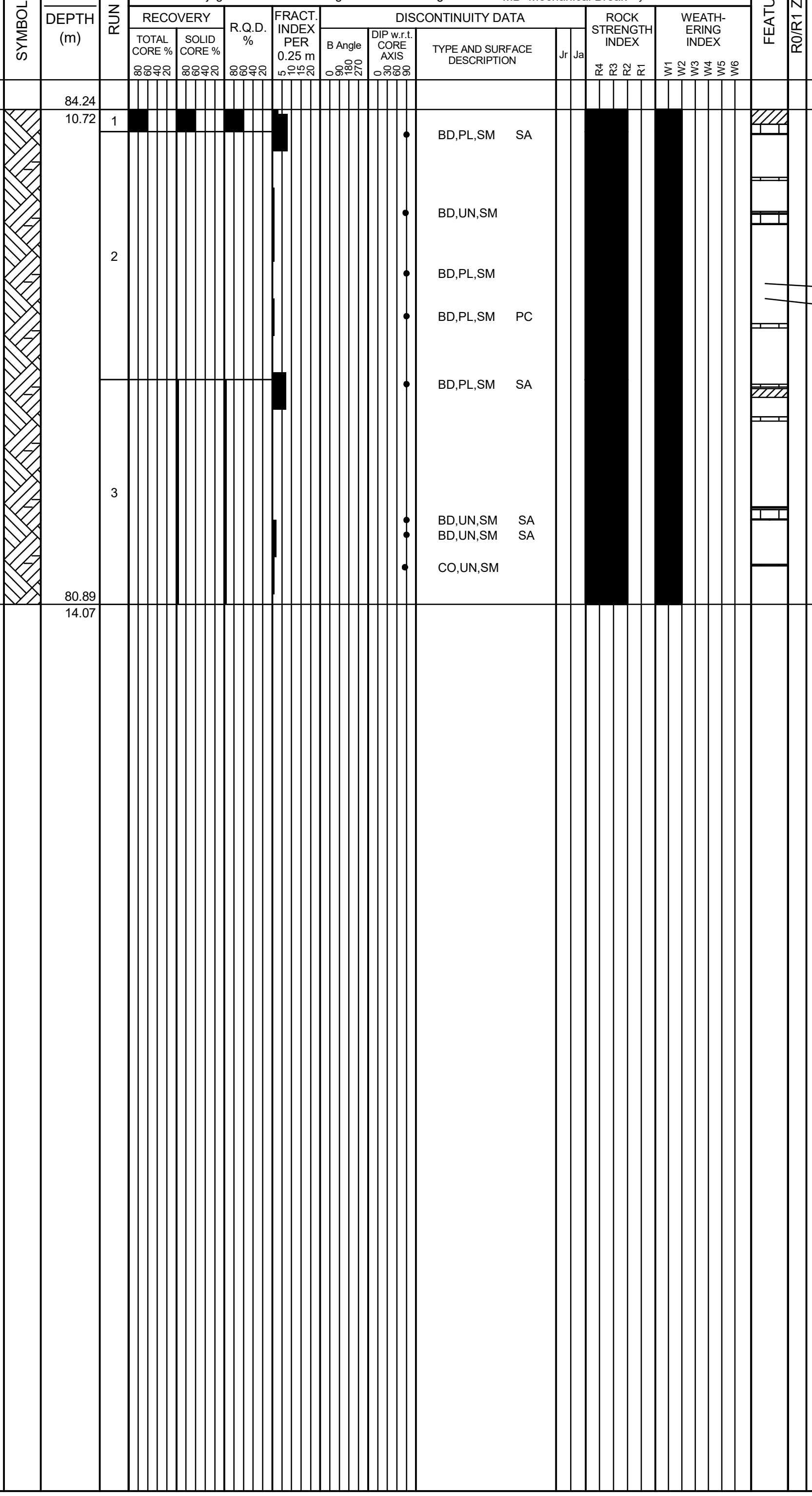
DRILL RIG: CME 55

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.	TYPE AND SURFACE DESCRIPTION	NOTES WATER LEVELS INSTRUMENTATION
		Continued from Record of Borehole K3		84.24			
		Slightly weathered, thinly bedded, grey, fine grained, faintly porous, weak SHALE (Georgian Bay Formation) with medium strong to strong limestone interbeds		10.72	1	BD,PL,SM SA	
11						BD,UN,SM	
					2	BD,PL,SM	UCS = 18.2 MPa E = 2.3 GPa e = 2.6 g/cm^3
12						BD,PL,SM PC	
						BD,PL,SM SA	
13					3		
						BD,UN,SM SA BD,UN,SM SA	
						CO,UN,SM	
14				80.89 14.07			
		END OF DRILLHOLE					
15							
16							
17							
18							
19							



FEATURES LEGEND

▨ BROKEN CORE	▩ CLAY SEAM	▦ LIMESTONE	■ LOST CORE

DEPTH SCALE

1 : 50

GOLDER

LOGGED: ACM

CHECKED: JP

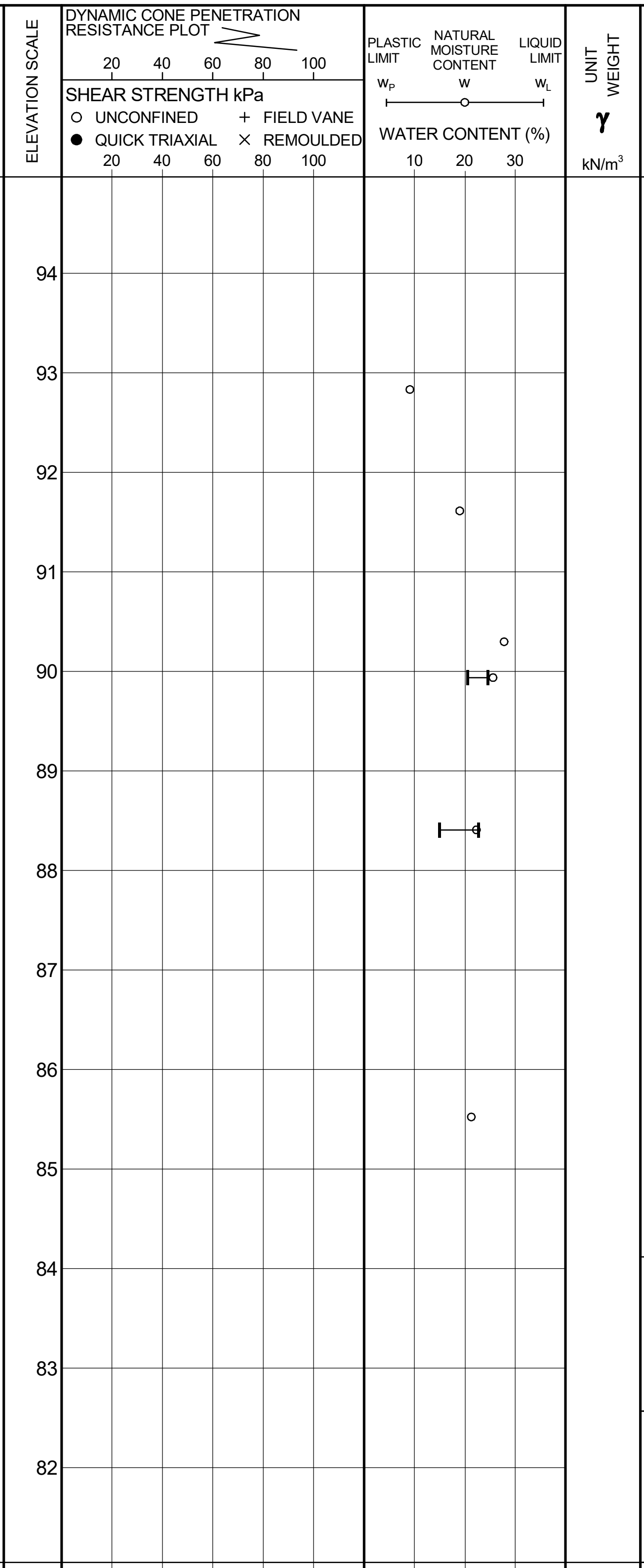
RECORD OF BOREHOLE No K4

SHEET 1 OF 2

METRIC

PROJECT 1662333
G.W.P. 2002-13-00
DIST Central HWY QEW
DATUM Geodetic
LOCATION N 4824692.4; E 296229.9 MTM NAD 83 ZONE 10 (LAT. 43.562120; LONG. -79.606087)
BOREHOLE TYPE CME 55, 210 mm O.D. Hollow Stem Auger
DATE September 18, 2018
ORIGINATED BY ACM
COMPILED BY JMP
CHECKED BY SMM

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
95.0	GROUND SURFACE					
0.0	ASPHALT (150 mm)					
0.2	Sand and gravel, trace silt (FILL) Compact Brown Moist	1	SS	25		
94.4 0.6	Silt and sand, trace clay (FILL) Very loose to compact Brown Moist becoming wet below a depth of 3.8 m	2	SS	17		
		3	SS	13		
		4	SS	2		0 62 37 1
	- Oxidation staining at 2.7 m	5	SS	3		
		6	SS	4		
		7	SS	WH		
		8A 8B 8C	SS	2		0 55 35 10
90.1 4.9	CLAYEY SILT, some sand to with sand, some gravel Very soft to firm Grey to brown, oxidation staining present Moist to wet	9A 9B	SS	7		
87.8 7.2	SAND, some silt, trace clay Compact to dense Brown Wet				∇	
		10	SS	18		
		11	SS	32		0 79 17 4
84.8 10.2	CLAYEY SILT, some sand, some gravel (TILL) Hard Grey Moist	12A 12B	SS	100/0.28		
84.2 10.8	SHALE (BEDROCK) Grey Bedrock cored from a depth of 10.9 m to 13.9 m For bedrock coring details, refer to Record of Drillhole K4	1	RC	REC 99%		RQD = 99%
		2	RC	REC 100%		RQD = 86%
81.1 13.9	END OF BOREHOLE					



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

RECORD OF BOREHOLE No K4

SHEET 2 OF 2

METRIC

PROJECT 1662333
G.W.P. 2002-13-00
LOCATION N 4824692.4; E 296229.9 MTM NAD 83 ZONE 10 (LAT. 43.562120; LONG. -79.606087)
ORIGINATED BY ACM
DIST Central HWY QEW
BOREHOLE TYPE CME 55, 210 mm O.D. Hollow Stem Auger
COMPILED BY JMP
DATUM Geodetic
DATE September 18, 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				
	NOTES: 1. Water level recorded at a depth of about 7.3 m below ground surface (Elev. 87.7 m) upon completion of soil drilling, prior to bedrock coring										

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

PROJECT: 1662333

RECORD OF DRILLHOLE: K4

SHEET 1 OF 1

LOCATION: N 4824692.4 ;E 296229.9

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: September 18, 2018

DRILL RIG: CME 55

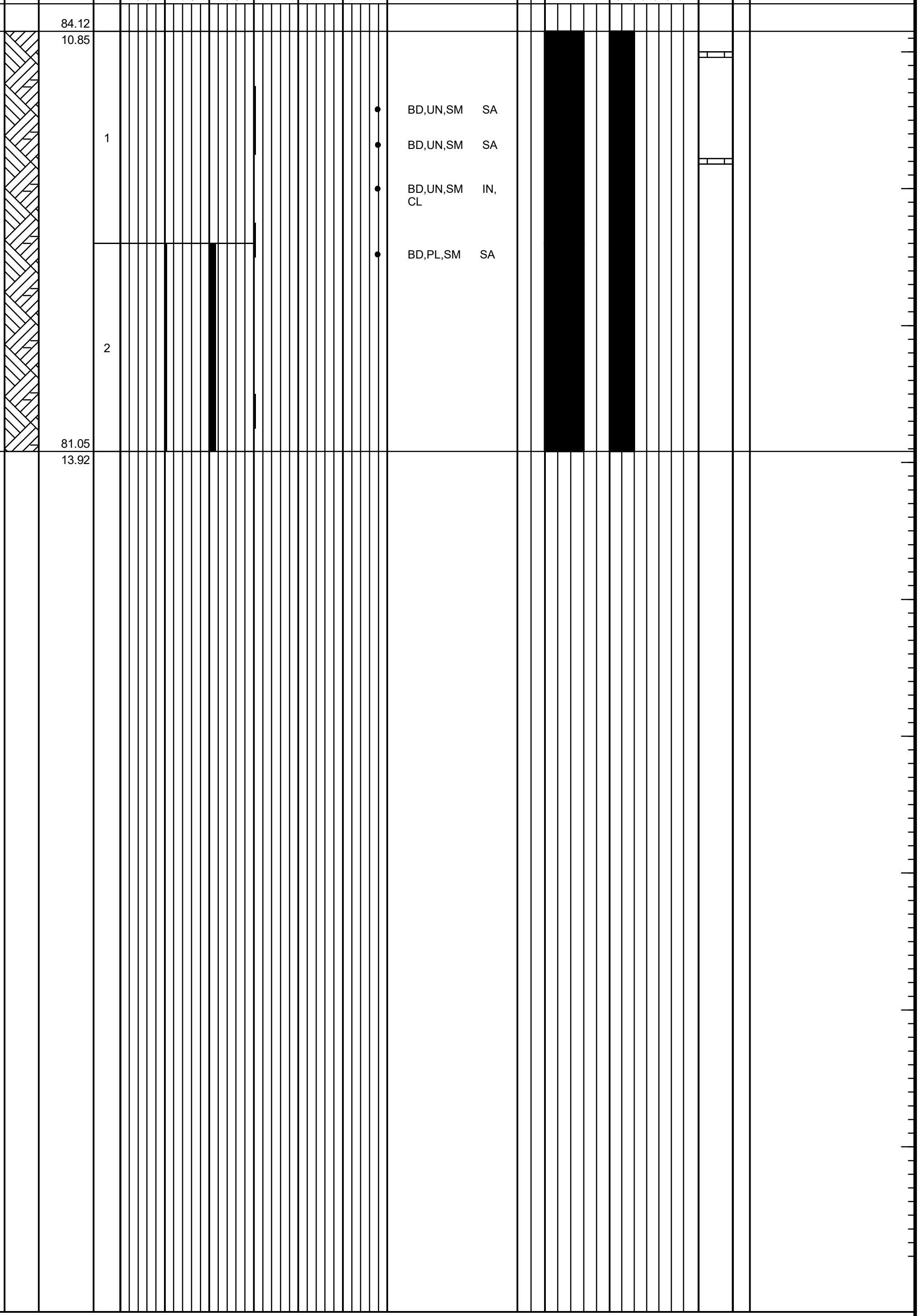
DRILLING CONTRACTOR: Aardvark Drilling

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.	RECOVERY TOTAL CORE %	RECOVERY SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.25 m	DISCONTINUITY DATA B Angle	DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	Jr	Ja	ROCK STRENGTH INDEX (R4 R3 R2 R1)	WEATHERING INDEX (W1 W2 W3 W4 W5 W6)	FEATURES	R0/R1 ZONES	NOTES WATER LEVELS INSTRUMENTATION
		Continued from Record of Borehole K4		84.12															
11		Slightly weathered, thinly bedded, grey, fine grained, faintly porous, weak SHALE (Georgian Bay Formation), with medium strong to strong limestone / siltstone interbeds		10.85	1							BD,UN,SM SA BD,UN,SM SA							
12												BD,UN,SM IN, CL BD,PL,SM SA							
13				81.05 13.92	2														
14		END OF DRILLHOLE																	
15																			
16																			
17																			
18																			
19																			



FEATURES LEGEND

▨ BROKEN CORE

▦ CLAY SEAM

▤ LIMESTONE

■ LOST CORE

DEPTH SCALE

1 : 50

GOLDER

LOGGED: ACM

CHECKED: JP

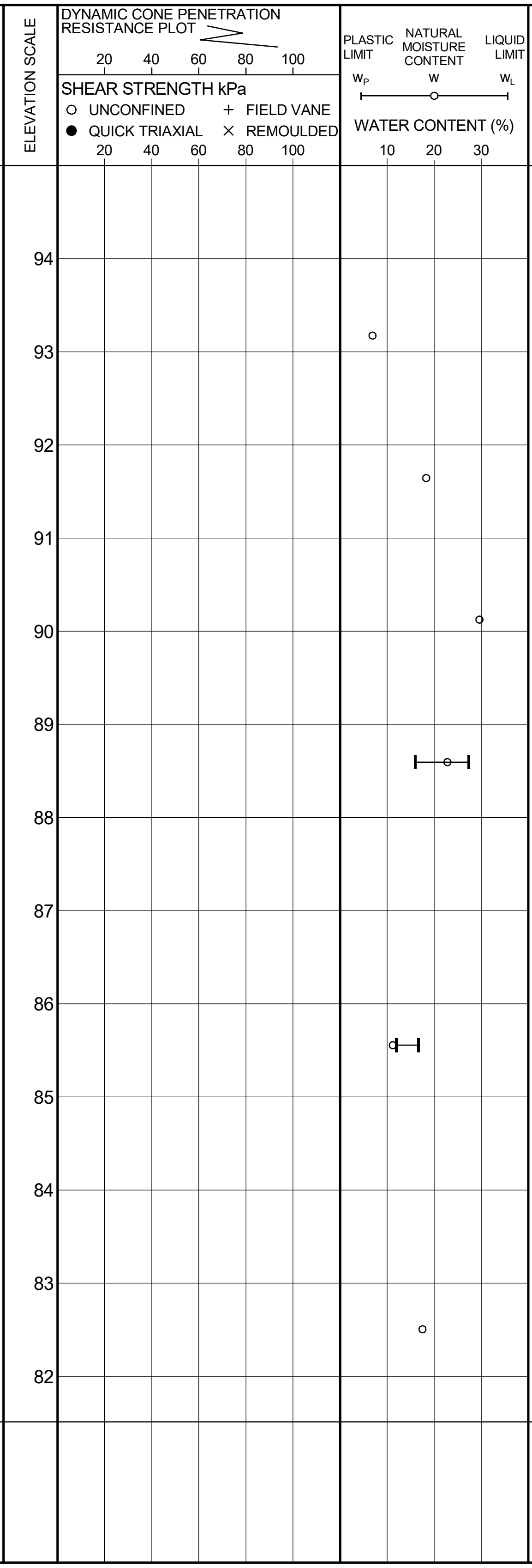
RECORD OF BOREHOLE No K5

SHEET 1 OF 2

METRIC

PROJECT 1662333
G.W.P. 2002-13-00
LOCATION N 4824683.3; E 296242.1 MTM NAD 83 ZONE 10 (LAT. 43.562038; LONG. -79.605937)
ORIGINATED BY SK
DIST Central HWY QEW
BOREHOLE TYPE CME 75, 180 mm O.D., Hollow Stem Augers
COMPILED BY DM
DATUM Geodetic
DATE November 30, 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
95.0	GROUND SURFACE									
0.0	ASPHALT (200 mm)									
	CONCRETE (300 mm)									
94.5 0.5	Silt and sand, trace clay (FILL) Very loose to compact Brown Moist to wet	1	SS	8						
		2	SS	2						
					▽					
		3	SS	14						
		4	SS	2		NP	0	59	39	2
		5	SS	3						
	- Brown to grey at 4.7 m	6	SS	WH						
89.4 5.6	CLAYEY SILT with SAND, trace to some gravel Firm to hard Grey Moist to wet	7	SS	7		Org = 3.2%	13	32	39	16
	- Auger grinding at 7.0 m									
	- Auger grinding from 7.9 m to 8.2 m and from 8.7 m to 9.0 m	8	SS	48						
		9	SS	46			10	61	25	4
84.8 10.2	Silty SAND, trace to some gravel, trace clay Very dense Grey Wet	10	SS	51						
		11	SS	100/0.13						
82.2 12.8	SHALE (BEDROCK) Grey - Auger grinding at 13.1 m	12	SS	100/0.08						
81.5 13.5	END OF BOREHOLE SPLIT-SPOON REFUSAL									



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

○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No K5

SHEET 2 OF 2

METRIC

PROJECT 1662333

G.W.P. 2002-13-00

LOCATION N 4824683.3; E 296242.1 MTM NAD 83 ZONE 10 (LAT. 43.562038; LONG. -79.605937)

ORIGINATED BY SK

DIST Central HWY QEW

BOREHOLE TYPE CME 75, 180 mm O.D., Hollow Stem Augers

COMPILED BY DM

DATUM Geodetic

DATE November 30, 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
	NOTES: 1. Borehole caved to a depth of 6.4 m below ground surface (Elev. 88.6 m) upon removal of Hollow Stem Augers. 2. Water level measured at a depth of 2.4 m below ground surface (Elev. 92.6 m) upon removal of Hollow Stem Augers, but water was added during drilling.													

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

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

RECORD OF BOREHOLE No K5A

SHEET 1 OF 2

METRIC

PROJECT 1662333			
G.W.P. 2002-13-00	LOCATION N 4824683.3; E 296242.1 MTM NAD 83 ZONE 10 (LAT. 43.562017; LONG. -79.605949)	ORIGINATED BY SK
DIST Central HWY QEW	BOREHOLE TYPE CME 75, 180 mm O.D., Hollow Stem Augers	COMPILED BY DM
DATUM Geodetic	DATE November 30, 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	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE				
95.0	GROUND SURFACE										
0.0											
							94				
							93				
							92				
							91				
							90				
							89				
	Refer to Record of Borehole K5 for stratigraphy						88				
							87				
							86				
							85				
							84				
							83				
82.2 12.8	SHALE (BEDROCK) Grey Bedrock cored from a depth of 12.8 m to 16.5 m For bedrock coring details, refer to Record of Drillhole K5A	▨	1	RC	REC 100%		82				RQD = 79%
		▨	2	RC	REC 95%		81				RQD = 69%

Continued Next Page

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

RECORD OF BOREHOLE No K5A

SHEET 2 OF 2

METRIC

PROJECT 1662333

G.W.P. 2002-13-00

LOCATION N 4824683.3; E 296242.1 MTM NAD 83 ZONE 10 (LAT. 43.562017; LONG. -79.605949)

ORIGINATED BY SK

DIST Central HWY QEW

BOREHOLE TYPE CME 75, 180 mm O.D., Hollow Stem Augers

COMPILED BY DM

DATUM Geodetic

DATE November 30, 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 *--- CONTINUED FROM PREVIOUS PAGE ---*	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
78.5	SHALE (BEDROCK) Grey Bedrock cored from a depth of 12.8 m to 16.5 m For bedrock coring details, refer to Record of Drillhole K5A	▨	3	RC	REC 100%		79				RQD = 94%
16.5	END OF BOREHOLE SPLIT-SPOON REFUSAL NOTES: 1. Borehole K5A was cored about 2 m west of Borehole K5.										

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

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

PROJECT: 1662333

RECORD OF DRILLHOLE: K5A

SHEET 1 OF 1

LOCATION: N 4824683.3 ;E 296242.1

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: November 30, 2018

DRILL RIG: CME 75

DRILLING CONTRACTOR: Davis Drilling

DATUM: Geodetic

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

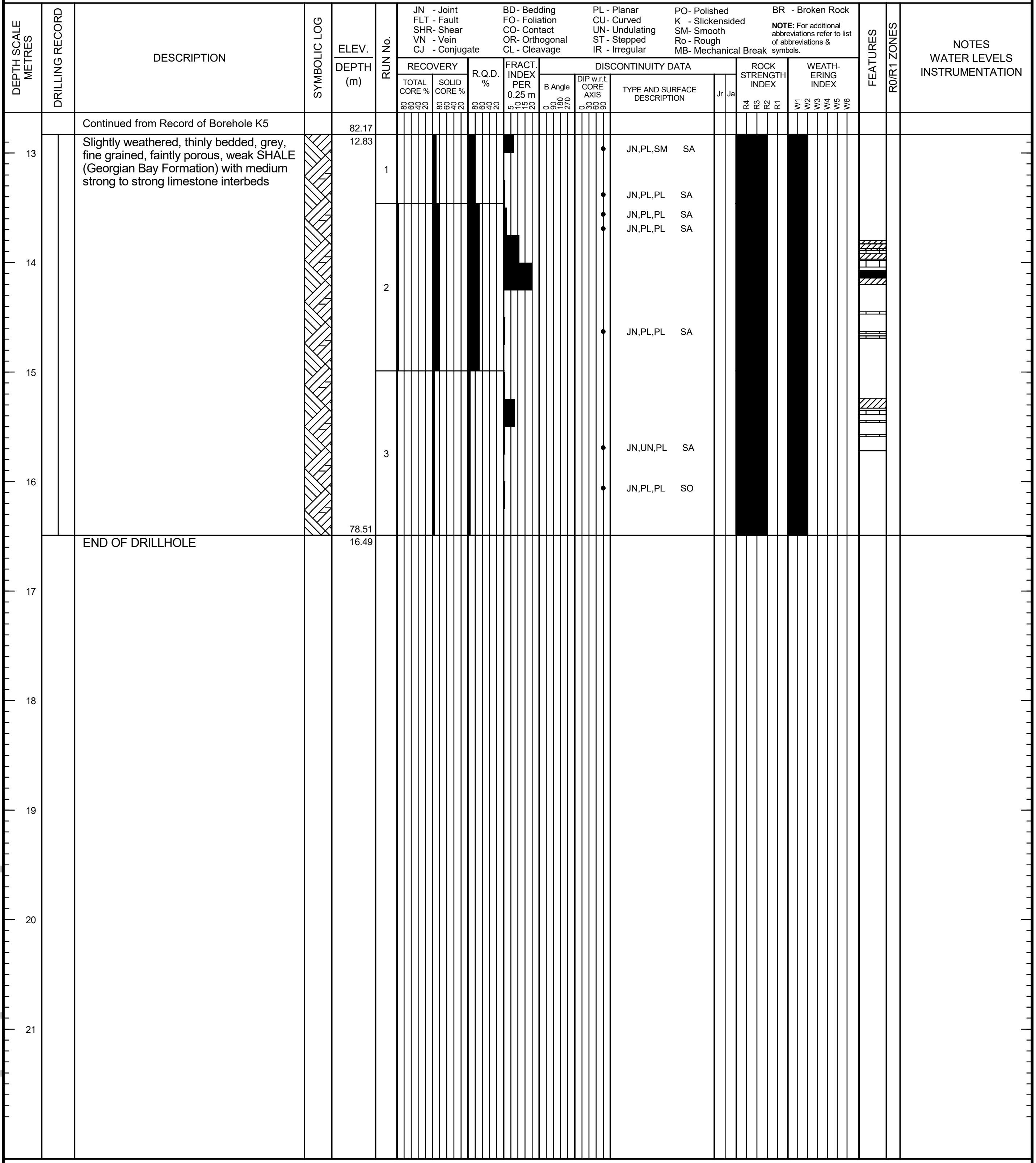
BD- Bedding
FO- Foliation
CO- Contact
OR- Orthogonal
CL - Cleavage

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.



DESCRIPTION	ELEV. DEPTH (m)	RUN No.	TYPE AND SURFACE DESCRIPTION
Continued from Record of Borehole K5	82.17		
Slightly weathered, thinly bedded, grey, fine grained, faintly porous, weak SHALE (Georgian Bay Formation) with medium strong to strong limestone interbeds	12.83	1	JN,PL,SM SA JN,PL,PL SA
		2	JN,PL,PL SA JN,PL,PL SA JN,PL,PL SA
	78.51 16.49	3	JN,UN,PL SA JN,PL,PL SO
END OF DRILLHOLE			

FEATURES LEGEND

▨	BROKEN CORE
▩	CLAY SEAM
▦	LIMESTONE
■	LOST CORE

DEPTH SCALE
1 : 50

GOLDER

LOGGED: SK
CHECKED: KF

RECORD OF BOREHOLE No K6

SHEET 1 OF 2

METRIC

PROJECT 1662333			
G.W.P. 2002-13-00	LOCATION N 4824688.5; E 296254.9 MTM NAD 83 ZONE 10 (LAT. 43.562085; LONG. -79.605778)	ORIGINATED BY ACM
DIST Central HWY QEW	BOREHOLE TYPE CME 55, 38 mm, Solid Stem Augers	COMPILED BY JMP
DATUM Geodetic	DATE November 11, 2018	CHECKED BY SMM

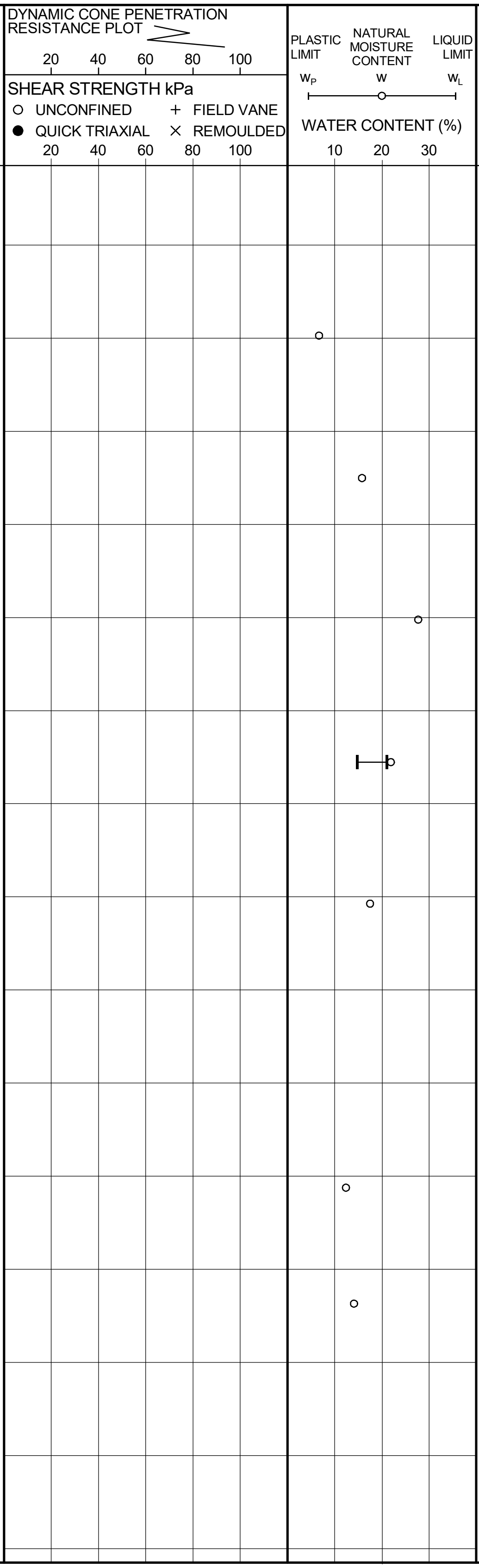
SOIL PROFILE

ELEV DEPTH	DESCRIPTION
94.9	GROUND SURFACE
0.0	ASPHALT (150 mm)
	CONCRETE
94.4 0.5	Silt and sand, trace clay (FILL) Compact to very loose Brown with oxidation staining Moist to wet - Trace to some gravel from 0.8 m to 1.4 m
91.2 3.7	Silty sand, trace clay (FILL) Very Loose Brown Wet - Oxidation staining from 4.6 m to 6.1 m
89.3 5.6	CLAYEY SILT with SAND Firm Brown to grey Wet to moist
87.7 7.2	SAND, some silt, trace clay Very dense to compact Grey Wet
83.2 11.7	CLAYEY SILT, some sand, trace gravel (TILL) Hard Grey Moist
82.6 12.3	SHALE (BEDROCK) Grey Bedrock cored from a depth of 14.3 m to 15.0 m For bedrock coring details, refer to Record of Drillhole K6
79.9	

Continued Next Page

SAMPLES

NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	36			
2	SS	7			
3	SS	2			
4	SS	9			0 61 37 2
5	SS	WH			
6	SS	WH		Org = 1.2%	Non-Plastic 0 78 20 2
7	SS	6			
8	SS	43			
9	SS	51			
10	SS	16			0 79 18 3
11A 11B	SS	100/0.23	∇		
12	SS	100/0.08			
1	RC	REC 62%			RQD = 62%
		REC			



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

RECORD OF BOREHOLE No K6

SHEET 2 OF 2

METRIC

PROJECT 1662333
G.W.P. 2002-13-00
LOCATION N 4824688.5; E 296254.9 MTM NAD 83 ZONE 10 (LAT. 43.562085; LONG. -79.605778)
ORIGINATED BY ACM
DIST Central HWY QEW
BOREHOLE TYPE CME 55, 38 mm, Solid Stem Augers
COMPILED BY JMP
DATUM Geodetic
DATE November 11, 2018
CHECKED BY SMM

SOIL PROFILE			SAMPLES					DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m^3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION --- *CONTINUED FROM PREVIOUS PAGE* ---	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE				
15.0	END OF BOREHOLE NOTES: 1. Water level measured at a depth of 12.3 m below ground surface (Elev. 82.6 m) prior to rock coring.		2	RC	100%						RQD = 0%

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

○$^{3\%}$ STRAIN AT FAILURE

PROJECT: 1662333

RECORD OF DRILLHOLE: K6

SHEET 1 OF 1

LOCATION: N 4824688.5 ;E 296254.9

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: November 11, 2018

DRILL RIG: CME 55

DRILLING CONTRACTOR: Triphase Drilling

DATUM: Geodetic

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

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

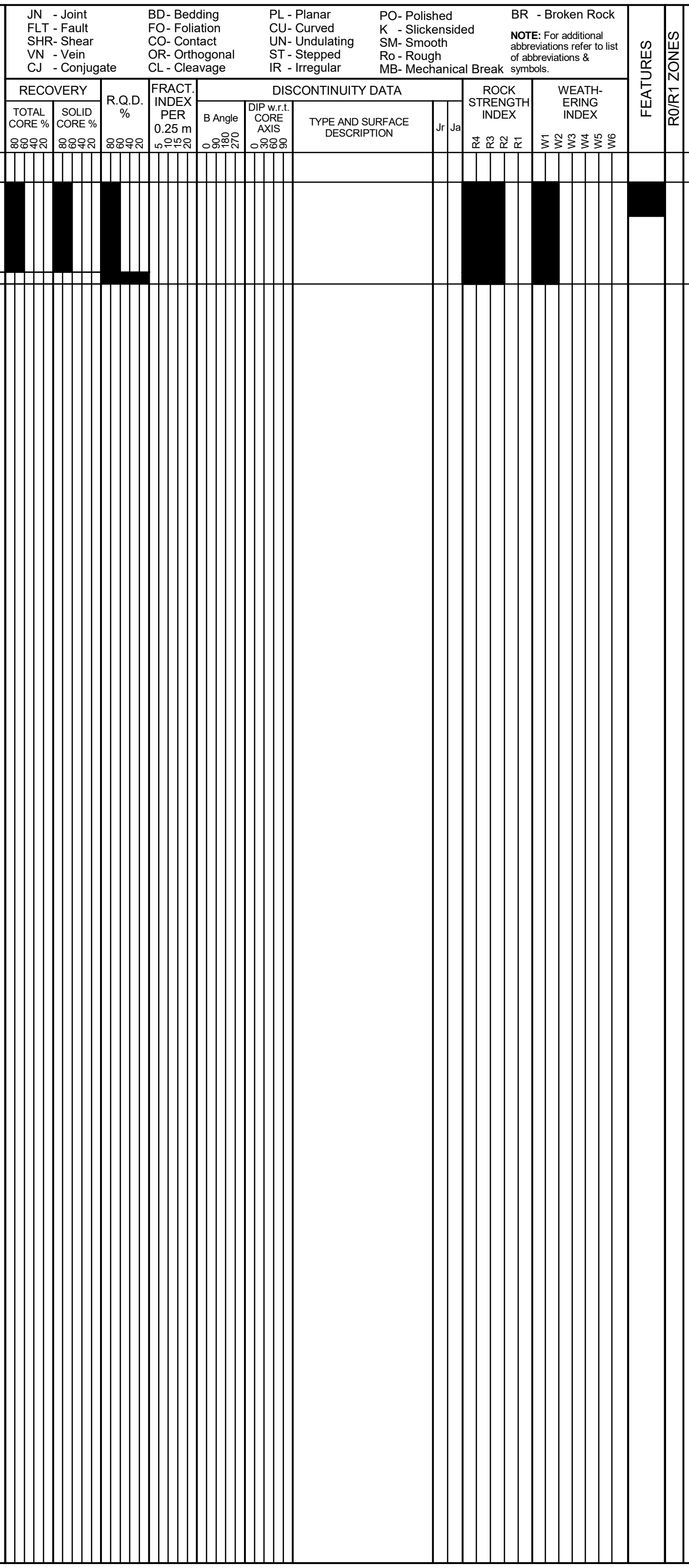
PL - Planar
CU- Curved
UN- Undulating
ST - Stepped
IR - Irregular

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.	NOTES WATER LEVELS INSTRUMENTATION
		Continued from Record of Borehole K6		80.52		
		Slightly weathered, thinly bedded, grey, fined grained, faintly porous, weak SHALE (Georgian Bay Formation)		14.33	1	
15				79.83	2	
		END OF DRILLHOLE		15.02		



Depth scale: 15, 16, 17, 18, 19, 20, 21, 22, 23

FEATURES LEGEND

▨ BROKEN CORE

▩ CLAY SEAM

▦ LIMESTONE

■ LOST CORE

DEPTH SCALE
1 : 50

GOLDER

LOGGED: ACM
CHECKED: DPM

RECORD OF BOREHOLE No NRW3-6

SHEET 1 OF 1

METRIC

PROJECT 1662333

G.W.P. 2002-13-00

DIST Central HWY QEW

DATUM Geodetic

LOCATION N 4824701.8; E 296220.4 MTM NAD 83 ZONE 10 (LAT. 43.562195; LONG. -79.606203)

BOREHOLE TYPE CME 75, 85 mm I.D., 190 mm O.D., Hollow Stem Augers

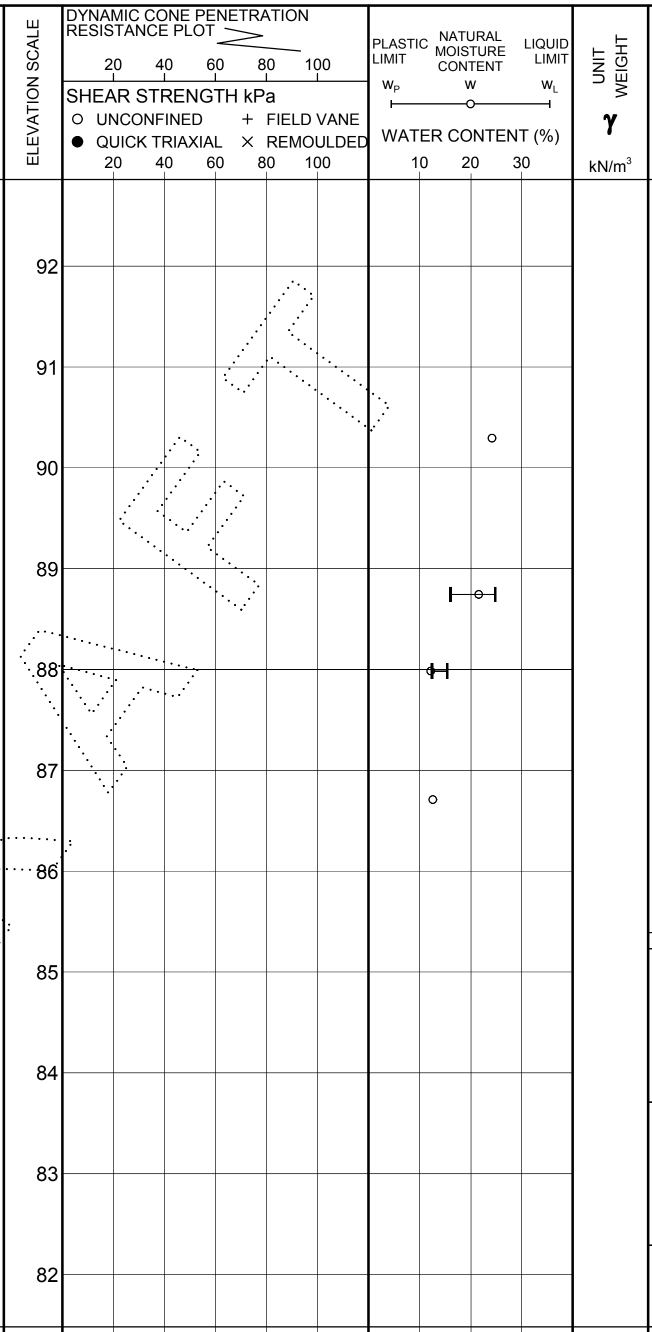
DATE June 22, 2018

ORIGINATED BY CC

COMPILED BY ACM

CHECKED BY SMM

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES		GR	SA	SI	CL
92.9	GROUND SURFACE								
0.0	Sandy silt, some gravel, rootlets, trace organics (FILL) Compact Brown Dry	1	SS	10					
92.2 0.7	Silt and sand, trace clay, trace rootlets (FILL) Very Loose Brown, grey below 2.2 m depth Moist to wet below 2.3 m depth	2	SS	4					
		3	SS	2					
		4	SS	WH		0	52	44	4
		5A 5B	SS	WH					
89.3 3.6	CLAYEY SILT with SAND, trace to some gravel (TILL) Very soft Brown to grey with oxidation staining Moist	6	SS	4		10	35	42	13
88.4 4.5	Silty SAND, some gravel, trace to some clay (TILL) Dense Grey Wet - Augers grinding from 5.2 m to 7.6 m	7	SS	32	∇	16	56	22	6
87.3 5.6	CLAYEY SILT, some sand, some shale fragments (RESIDUAL SOIL) Grey Wet								
86.7 6.2	Inferred completely to moderately weathered, brown to grey, extremely weak to weak SHALE (Georgian Bay Formation)	8A 8B	SS	100/0.20					
85.4 7.5	SHALE (BEDROCK) Grey Bedrock cored from a depth of 7.5 m to 11.4 m For bedrock coring details, refer to Record of Drillhole NRW3-6	1 9	RC SS	REC 100% 100/0.10		RQD = 0%			
		2	RC	REC 94%		RQD = 80%			
		3	RC	REC 96%		RQD = 82%			
81.5		4	RC	REC 100%		RQD = 88%			
11.4	END OF BOREHOLE								



NOTES:

1. Borehole caved to a depth of 6.7 m below ground surface upon completion of soil drilling prior to rock coring.

2. Water level measured at a depth of about 5.2 m below ground surface (Elev. 87.7 m) prior to rock coring.

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

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

PROJECT: 1662333

RECORD OF DRILLHOLE: NRW3-6

SHEET 1 OF 1

LOCATION: N 4824701.8 ;E 296220.4

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE:

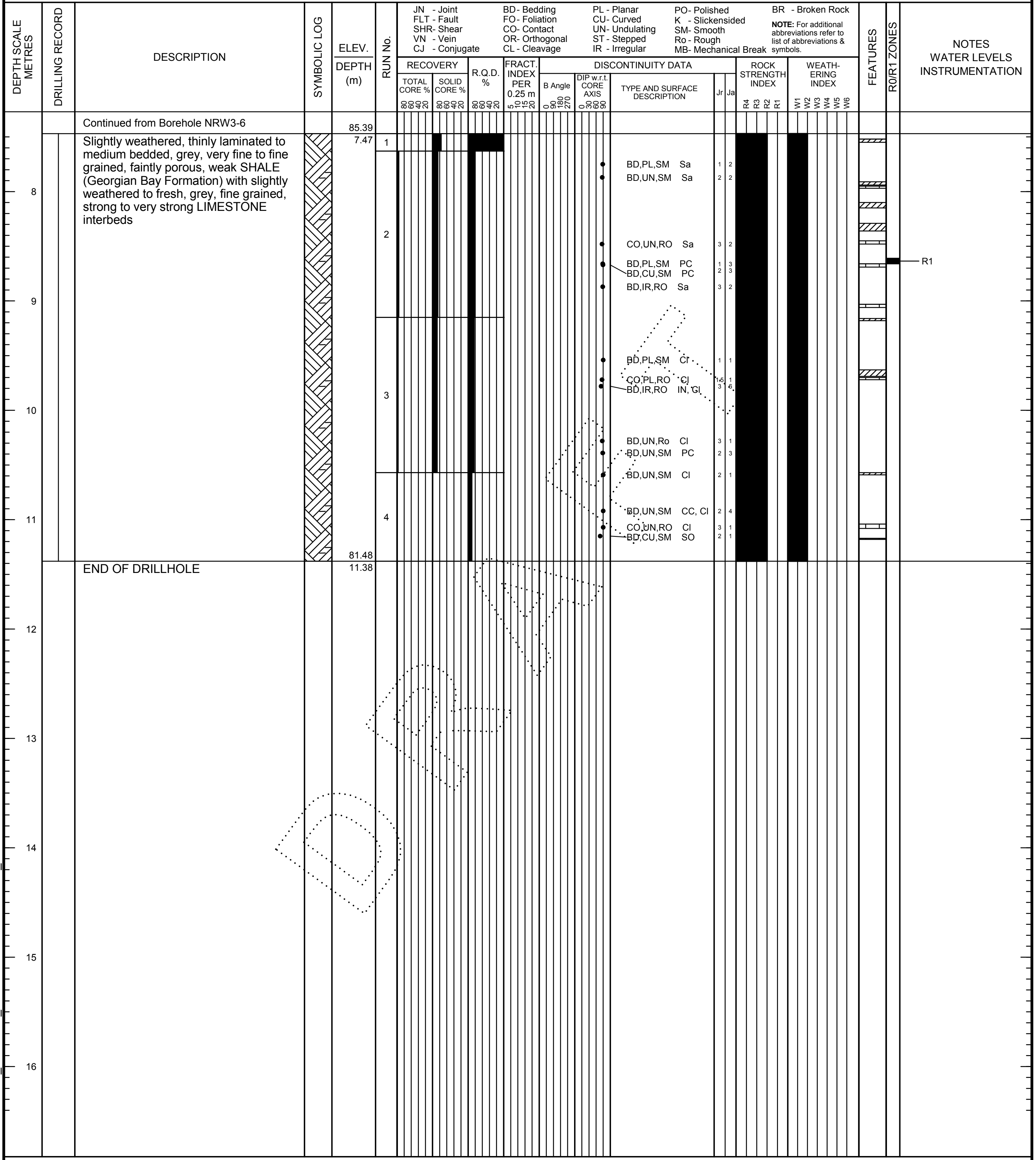
DRILL RIG: CME 75

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.



DESCRIPTION	ELEV. / DEPTH (m)
Continued from Borehole NRW3-6	85.39
Slightly weathered, thinly laminated to medium bedded, grey, very fine to fine grained, faintly porous, weak SHALE (Georgian Bay Formation) with slightly weathered to fresh, grey, fine grained, strong to very strong LIMESTONE interbeds	7.47
	81.48
END OF DRILLHOLE	11.38

DISCONTINUITY DATA

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

NOTES WATER LEVELS INSTRUMENTATION

FEATURES LEGEND

▨	BROKEN CORE
▩	CLAY SEAM
▦	LIMESTONE
■	LOST CORE

GTA-RCK 054 S:\CLIENTS\MTO\QEW-CREDIT_RIVER\02_DATA\GINT\QEW-CREDIT_RIVER.GPJ GAL-MISS.GDT 04/30/19

DEPTH SCALE
1 : 50

GOLDER

LOGGED:
CHECKED:

RECORD OF BOREHOLE No NRW7-3

SHEET 1 OF 1

METRIC

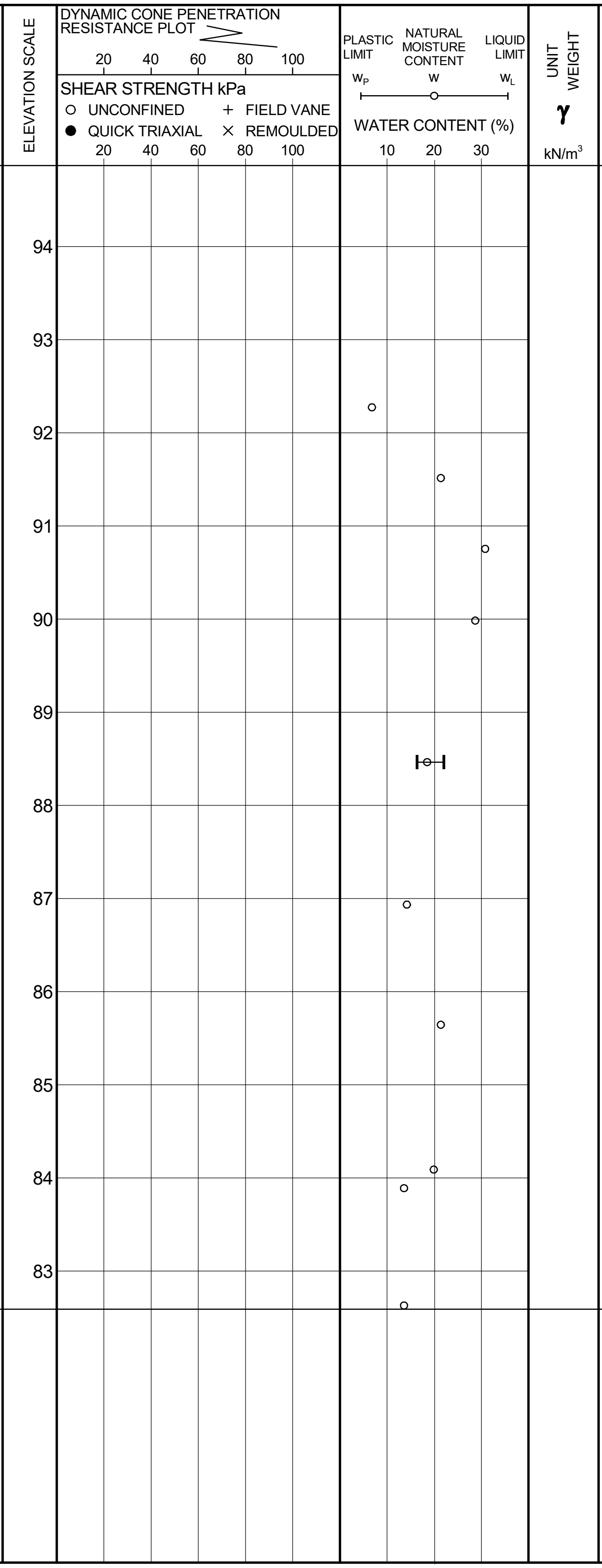
PROJECT 1662333			
G.W.P. 2002-13-00	LOCATION N 4824696.6; E 296259.1 MTM NAD 83 ZONE 10 (LAT. 43.562158; LONG. -79.605727)	ORIGINATED BY ACM	
DIST Central	HWY QEW	BOREHOLE TYPE CME 114 mm O.D. Hollow Stem Auger	COMPILED BY SK
DATUM Geodetic	DATE July 15, 2018	CHECKED BY SMM	

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES		GR	SA	SI	CL
94.9	GROUND SURFACE								
0.0	CONCRETE (430 mm)								
94.5 0.4	Gravelly sand, trace to some silt (FILL) Dense Brown Wet	1	SS	49					
93.9 1.0	Silt and sand, trace clay (FILL) Very loose to compact Brown Moist to wet below 3.6 m	2	SS	26					
		3	SS	13					
	- 0.1 m of black organic silt present at 2.4 m	4	SS	2		0	67	31	2
		5	SS	2					
		6	SS	WH		0	64	34	2
		7	SS	1					
89.3 5.6	CLAYEY SILT with SAND, some gravel Very stiff Grey Wet - Auger grinding from 6.7 m to 7.0 m	8	SS	26	▽	17	40	31	12
87.7 7.2	Silty SAND, trace to some clay, trace gravel Very dense Grey Wet - Auger grinding from 7.6 m to 8.2 m and from 9.1 m to 12.2 m	9	SS	53		2	70	21	7
		10	SS	100/0.13					
84.1 10.8	CLAYEY SILT, some sand, some shale fragments below 11.6 m (RESIDUAL SOIL) Hard Grey Moist	11A 11B	SS	50/0.25					
82.6		12	SS	100/0.08					
12.3	END OF BOREHOLE SPLIT-SPOON REFUSAL								

NOTES:

1. Borehole caved to a depth of 4.3 m below ground surface upon removal of hollow stem augers.

2. Water level measured at a depth of 6.4 m below ground surface (Elev. 88.5 m) within hollow stem augers upon completion of soil drilling.

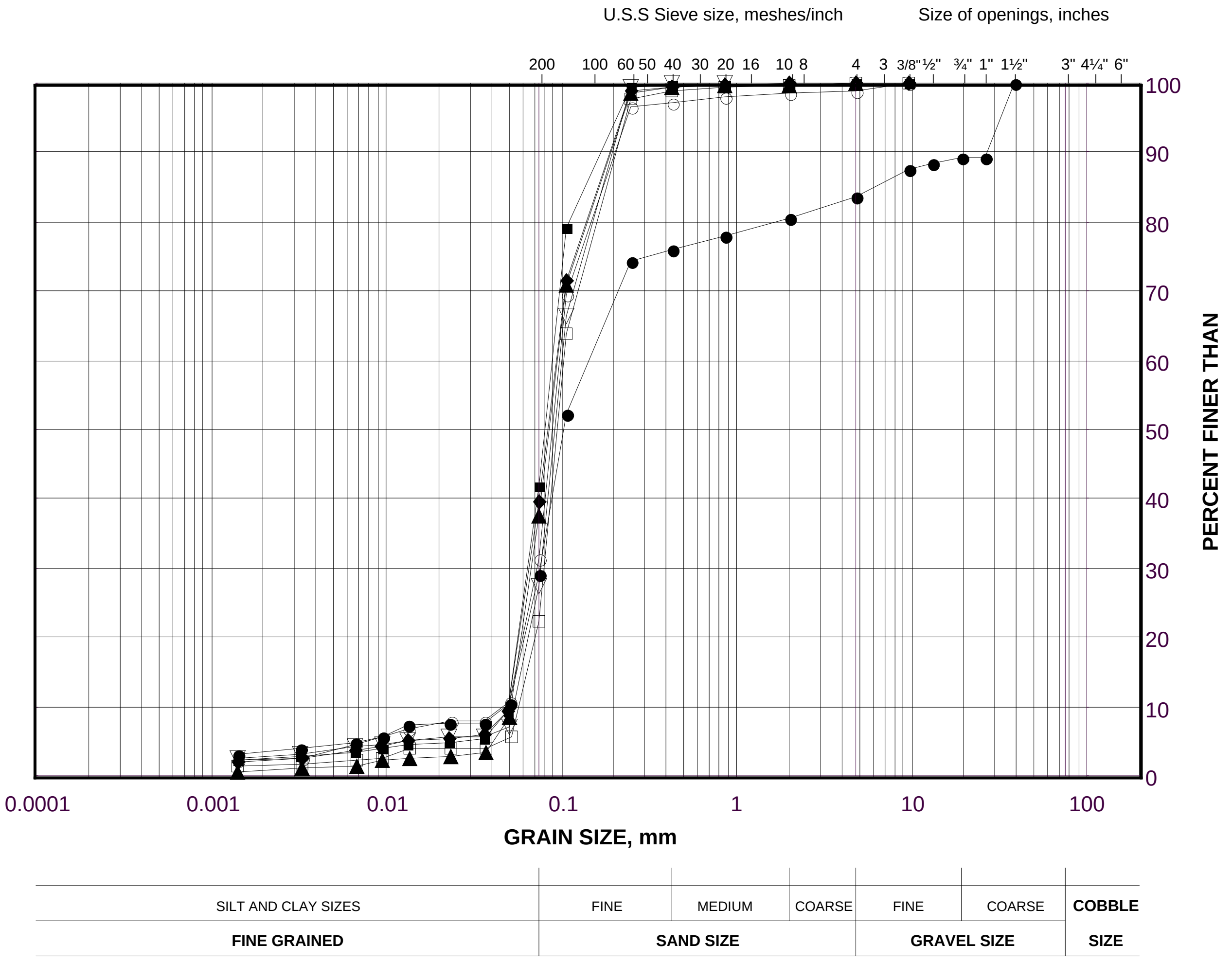


+3, ×3: Numbers refer to Sensitivity ○3% STRAIN AT FAILURE

GRAIN SIZE DISTRIBUTION

Silt and Sand to Silty Sand (Fill)

FIGURE B-1A



LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	K2	3	91.7
■	K5	4	91.6
◆	K6	4	91.5
▲	K4	4	92.9
▽	K3	5	91.6
❍	K2	6	89.8
□	K3	7	90.1

Project Number: 1662333

Checked By: SMM

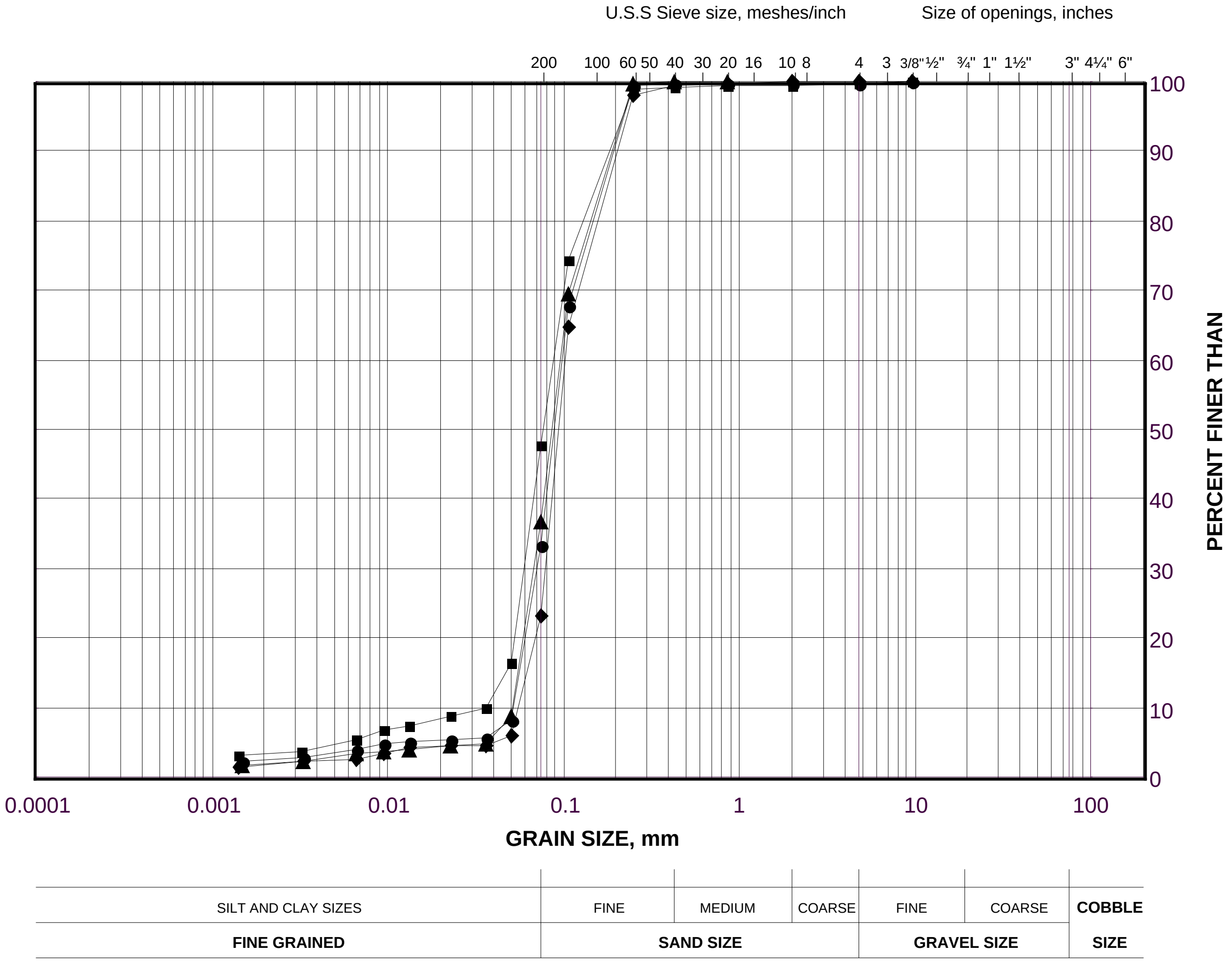
Golder Associates

Date: 05-Feb-19

GRAIN SIZE DISTRIBUTION

Silt and Sand to Silty Sand (Fill)

FIGURE B-1B



LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	NRW7-3	4	92.3
■	NRW3-6	4	90.3
◆	K6	6	90.0
▲	NRW7-3	6	90.8

Project Number: 1662333

Checked By: SMM

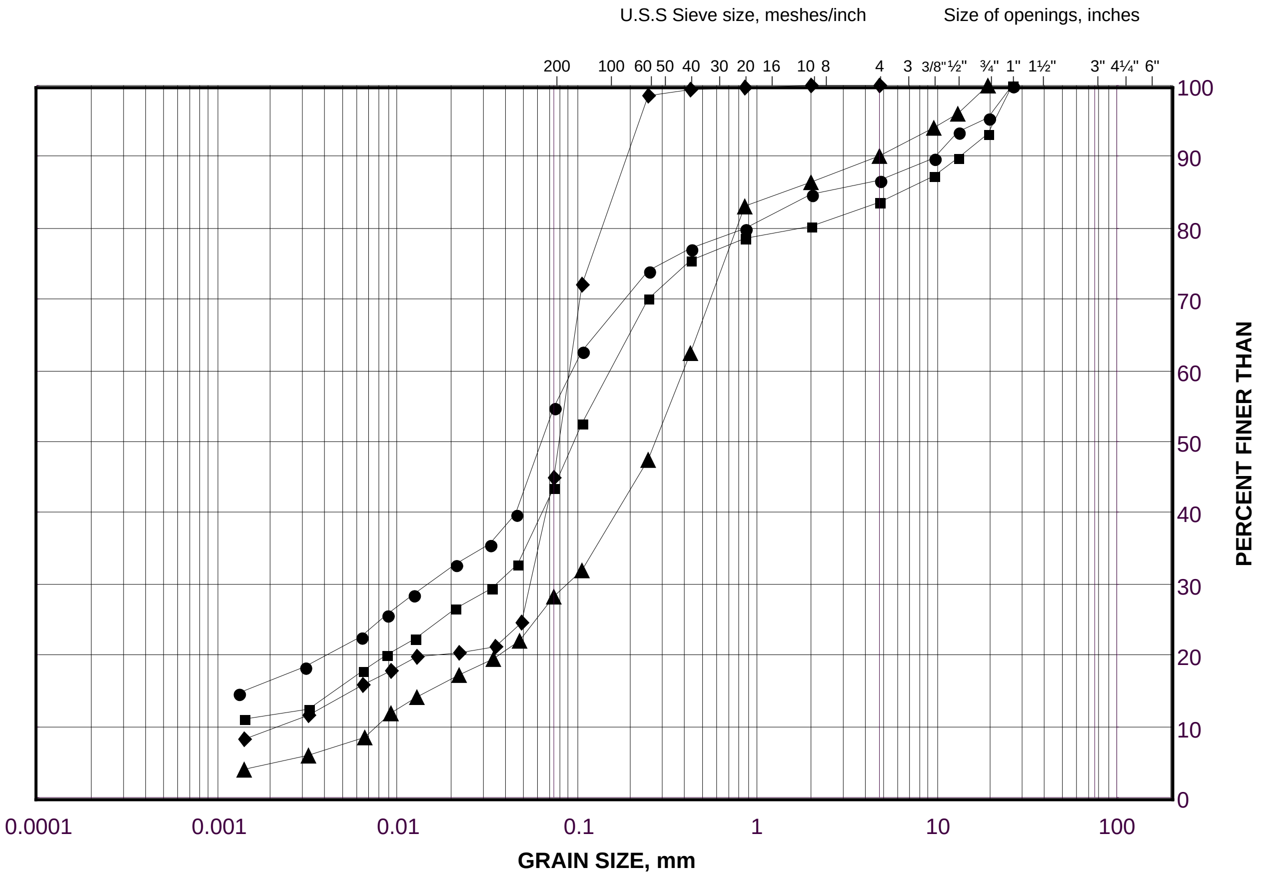
Golder Associates

Date: 05-Feb-19

GRAIN SIZE DISTRIBUTION

Clayey Silt with Sand

FIGURE B-2



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	K5	7	88.6
■	NRW7-3	8	88.5
◆	K4	8C	89.9
▲	K5	9	85.6

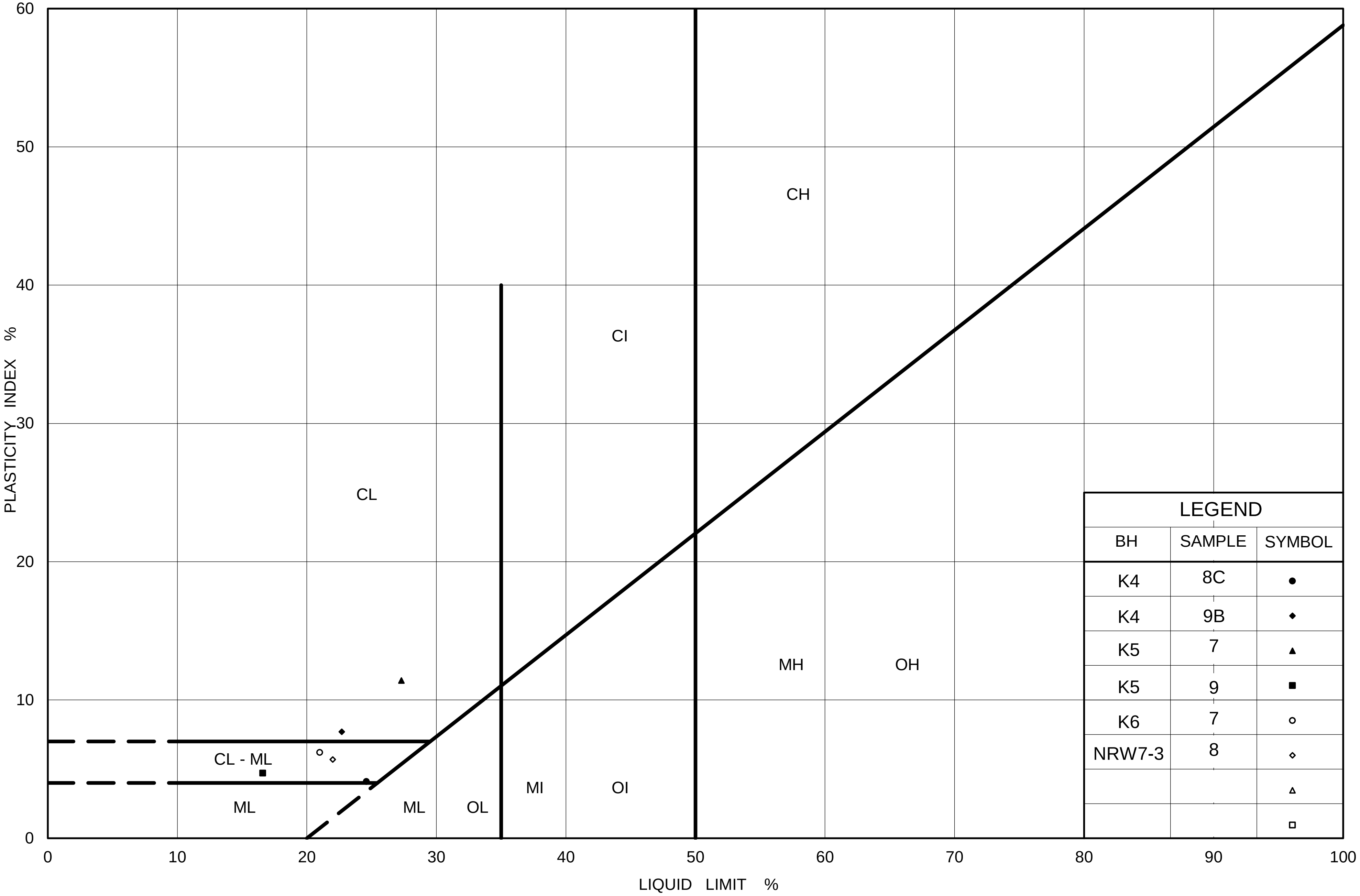
Project Number: 1662333

Checked By: SMM

Golder Associates

Date: 05-Feb-19

Oct 75, FF-S-21



Ministry of Transportation

Ontario

PLASTICITY CHART
Clayey Silt with Sand

Figure No. B-3

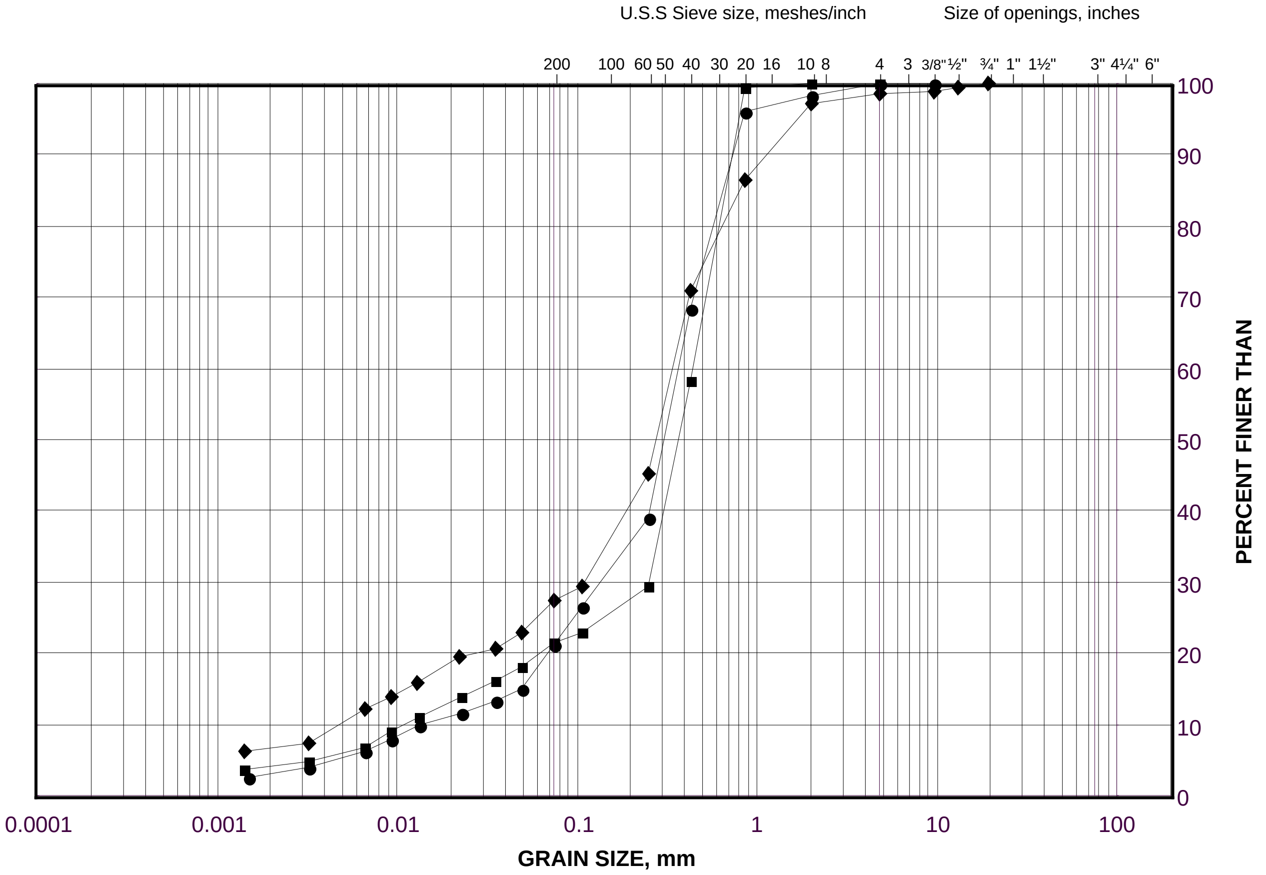
Project No. 1662333

Checked By: SMM

GRAIN SIZE DISTRIBUTION

Silty Sand to Sand

FIGURE B-4



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	K6	10	83.9
■	K4	11	85.6
◆	NRW7-3	9	86.9

Project Number: 1662333

Checked By: SMM

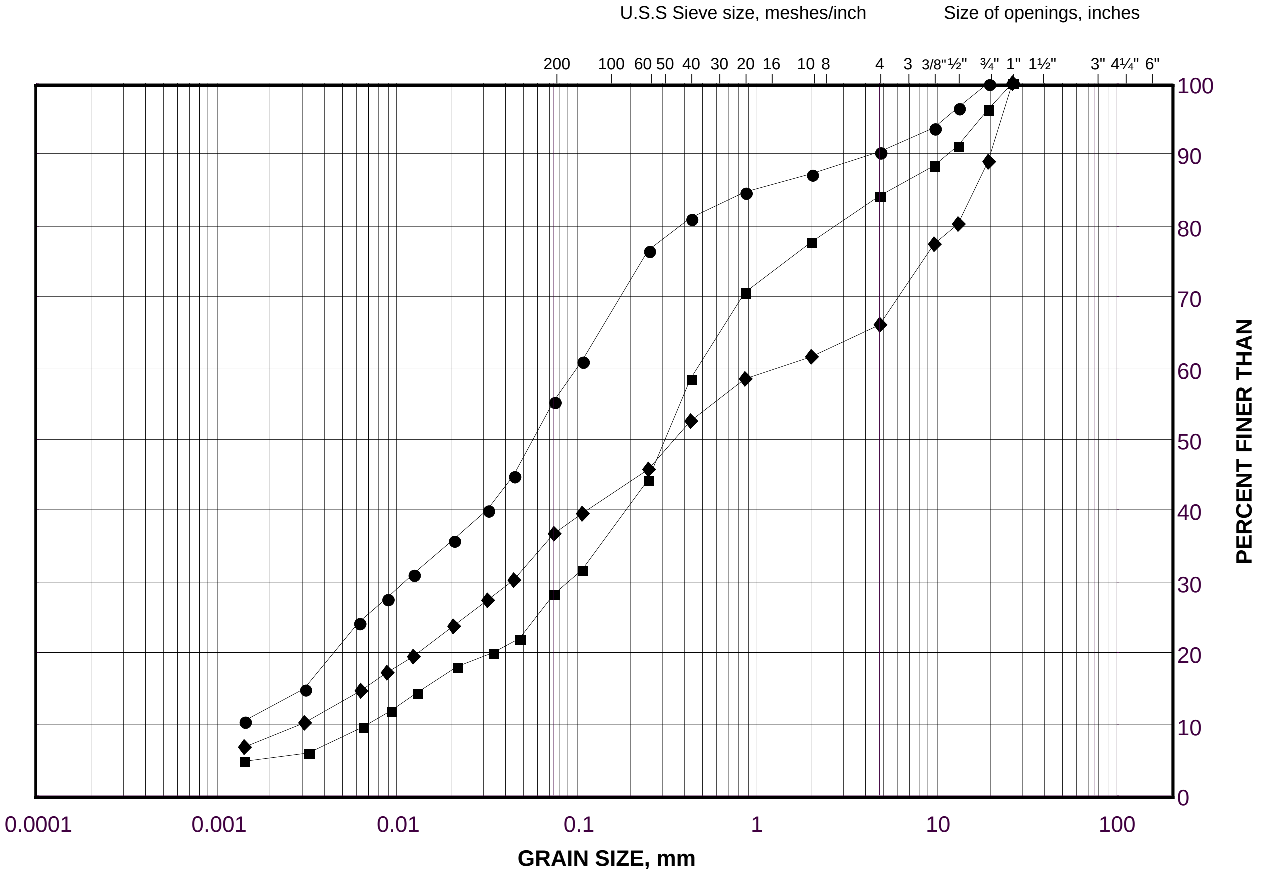
Golder Associates

Date: 05-Feb-19

GRAIN SIZE DISTRIBUTION

Silty Sand / Sandy Clayey Silt with Gravel / Clayey Silt with Sand (Till)

FIGURE B-5



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	NRW3-6	6	88.8
■	NRW3-6	7	88.0
◆	K3	8	88.6

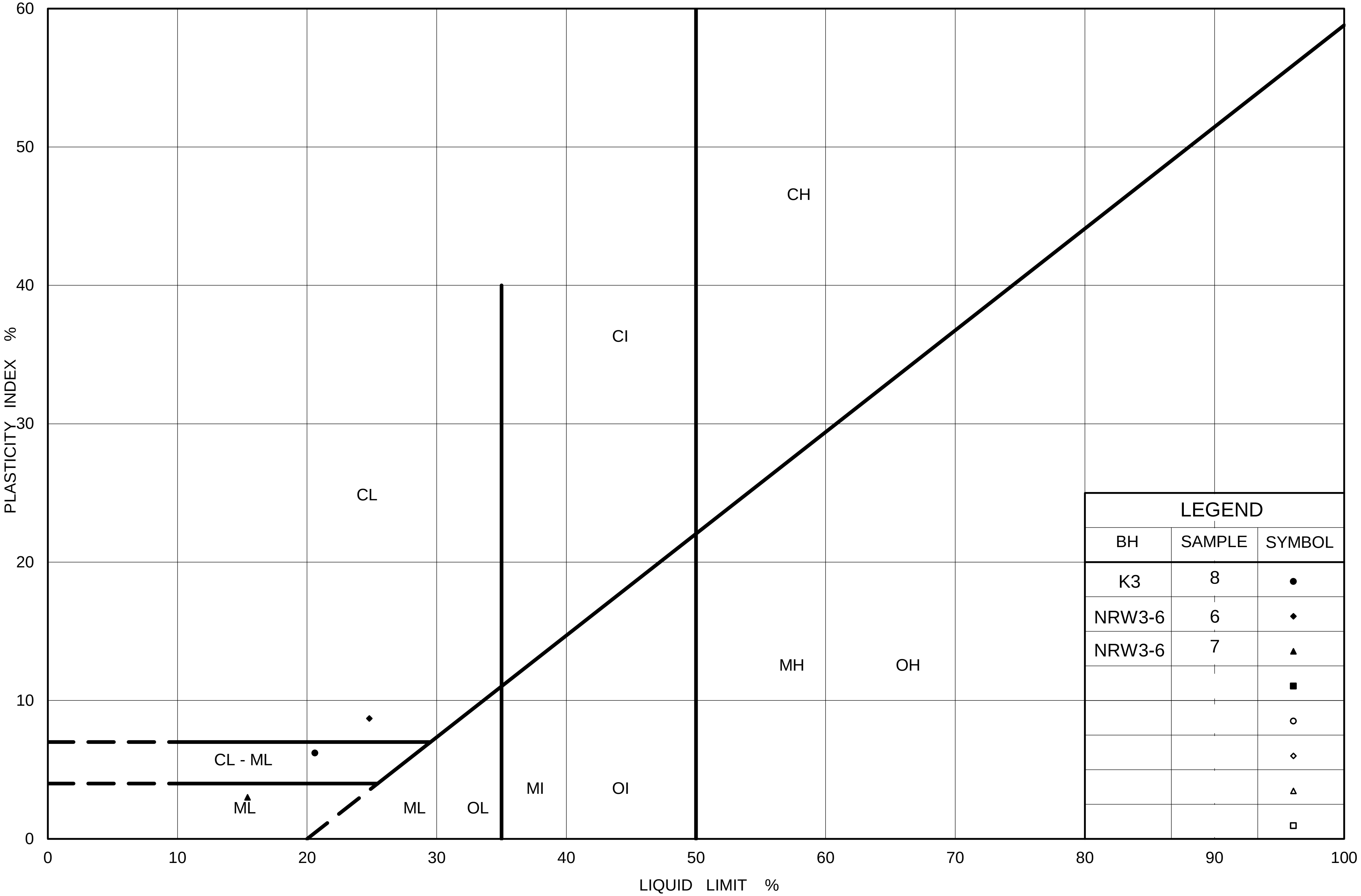
Project Number: 1662333

Checked By: SMM

Golder Associates

Date: 05-Feb-19

Oct 75, FF-S-21



Ministry of Transportation

Ontario

PLASTICITY CHART

Silty Sand to Sandy Clayey Silt with Gravel / Clayey Silt with Sand (Till)

Figure No. B-6

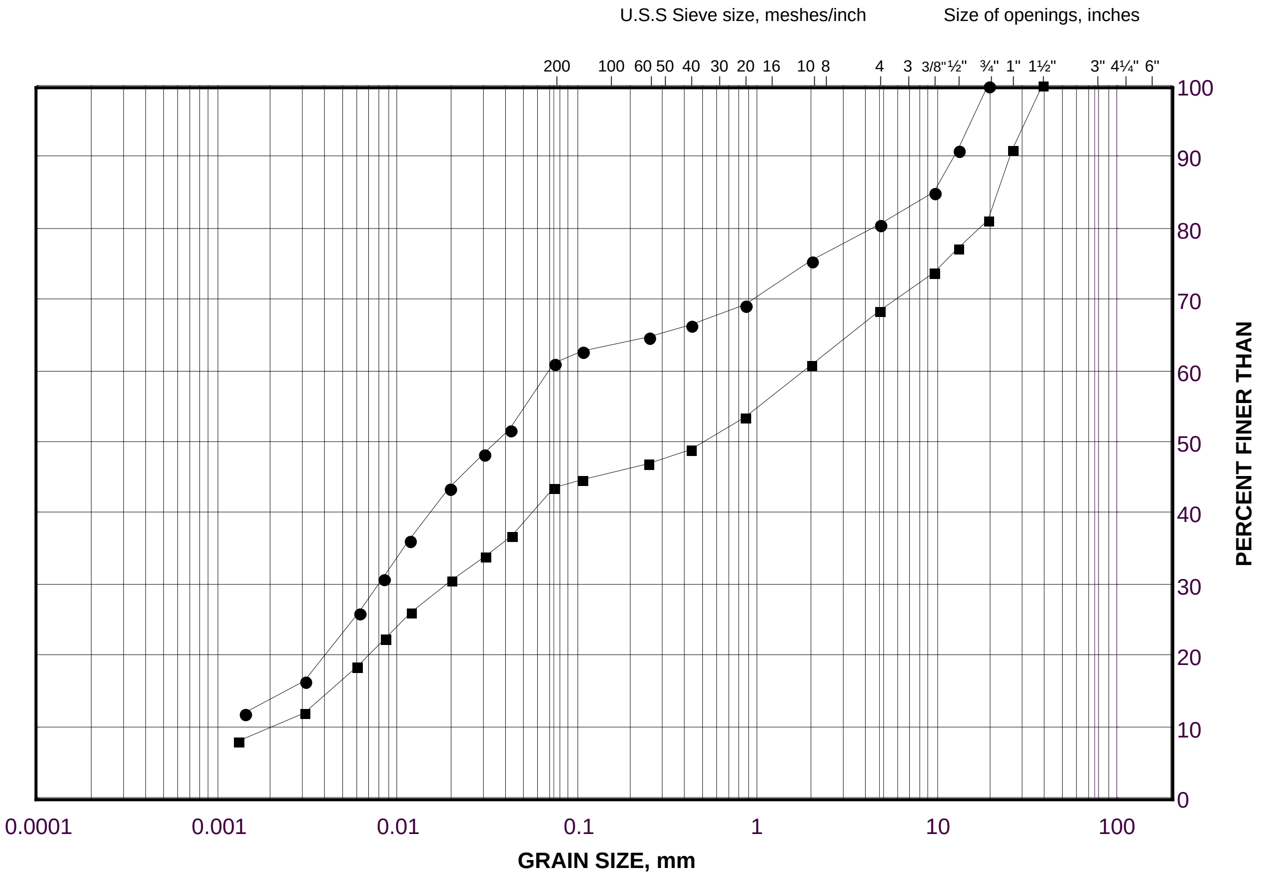
Project No. 1662333

Checked By: SMM

GRAIN SIZE DISTRIBUTION

Sandy Clayey Silt with Gravel / Clayey Silt (Residual Soil)

FIGURE B-7



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	K1	2A	89.1
■	K2	8	88.3

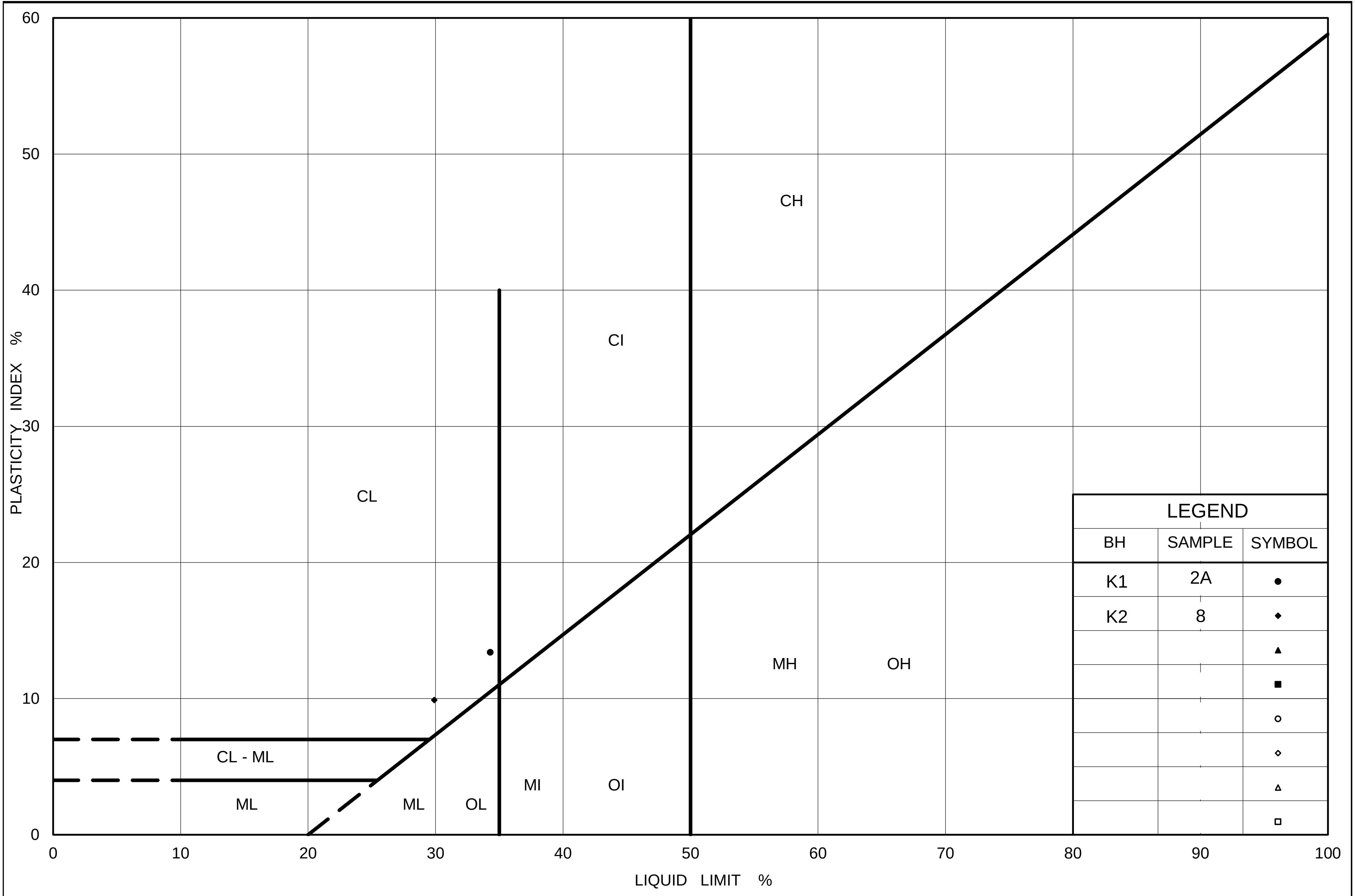
Project Number: 1662333

Checked By: SMM

Golder Associates

Date: 05-Feb-19

Oct 75, FF-S-21



Ministry of Transportation

Ontario

PLASTICITY CHART

Sandy Clayey Silt with Gravel / Clayey Silt (Residual Soil)

Figure No. B-8

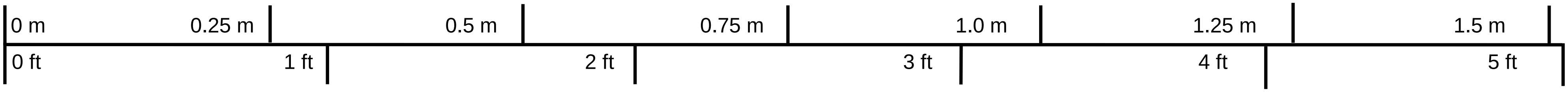
Project No. 1662333

Checked By: SMM

Start of Run No. 1 (1.25 m)

Start of Run No. 2 (2.01 m)

Start of Run No. 3 (3.53 m)



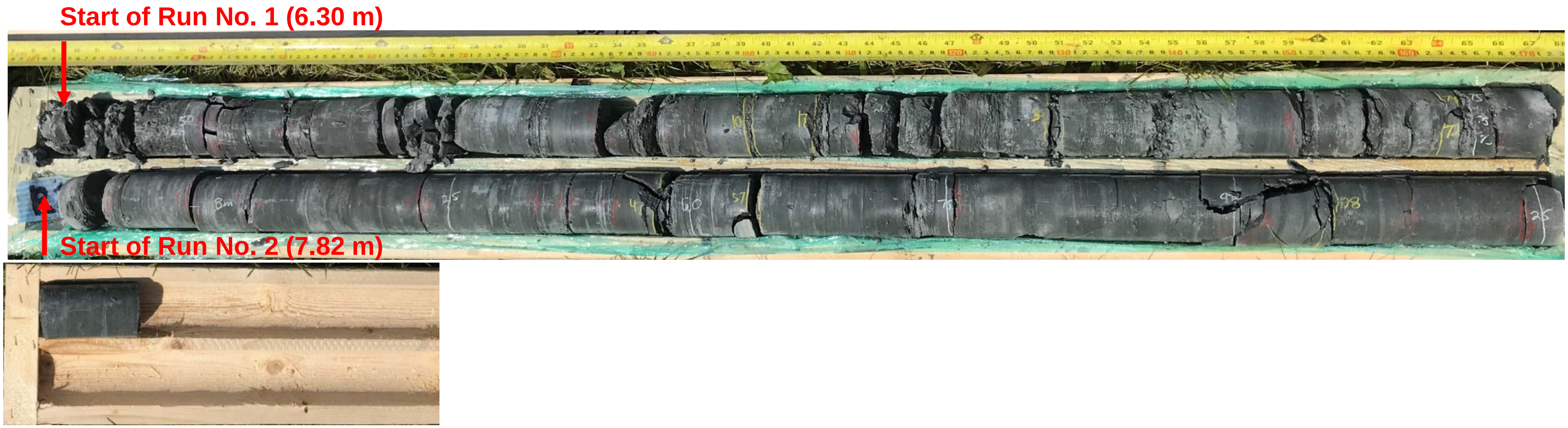
Scale

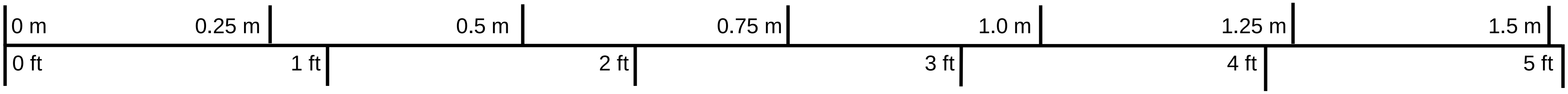
PROJECT	**MTO Assignment 2015-E-0033** **QEW Widening Between** **Mississauga Road and Hurontario Street**
TITLE	**Bedrock Core Photograph** **Borehole K1 (1.25 m to 5.05 m)**

GOLDER

PROJECT No. 1662333			FILE No. ----		
DRAFT	KMG	Jan 2019	SCALE	AS SHOWN	VER. 1.
CADD	--		**FIGURE B-9**		
CHECK	SMM	Apr 2019			
REVIEW	JMAC	Apr 2019			

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





Scale

PROJECT

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

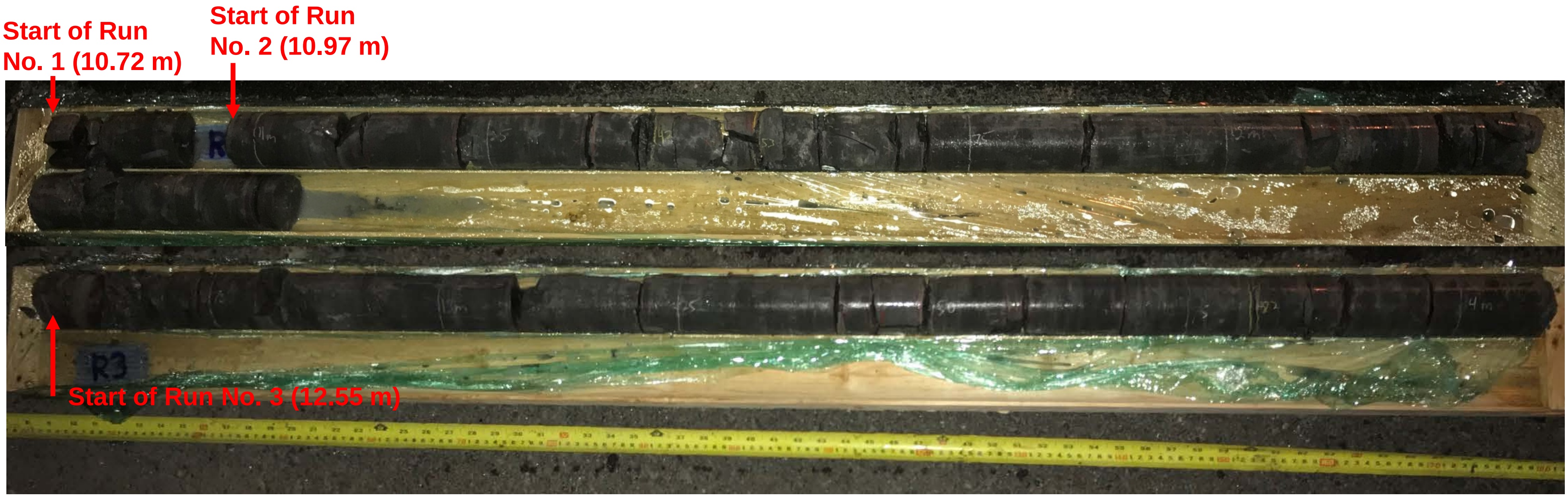
TITLE

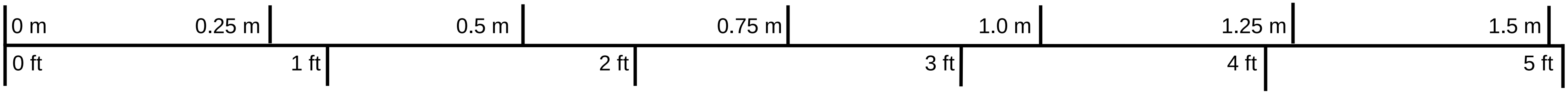
Bedrock Core Photograph
Borehole K2 (6.30 m to 9.37 m)

GOLDER

PROJECT No. 1662333			FILE No. ----	
DRAFT	KMG	Jan 2019	SCALE AS SHOWN	VER. 1.
CADD	--		**FIGURE B-10**	
CHECK	SMM	Apr 2019		
REVIEW	JMAC	Apr/2019		

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



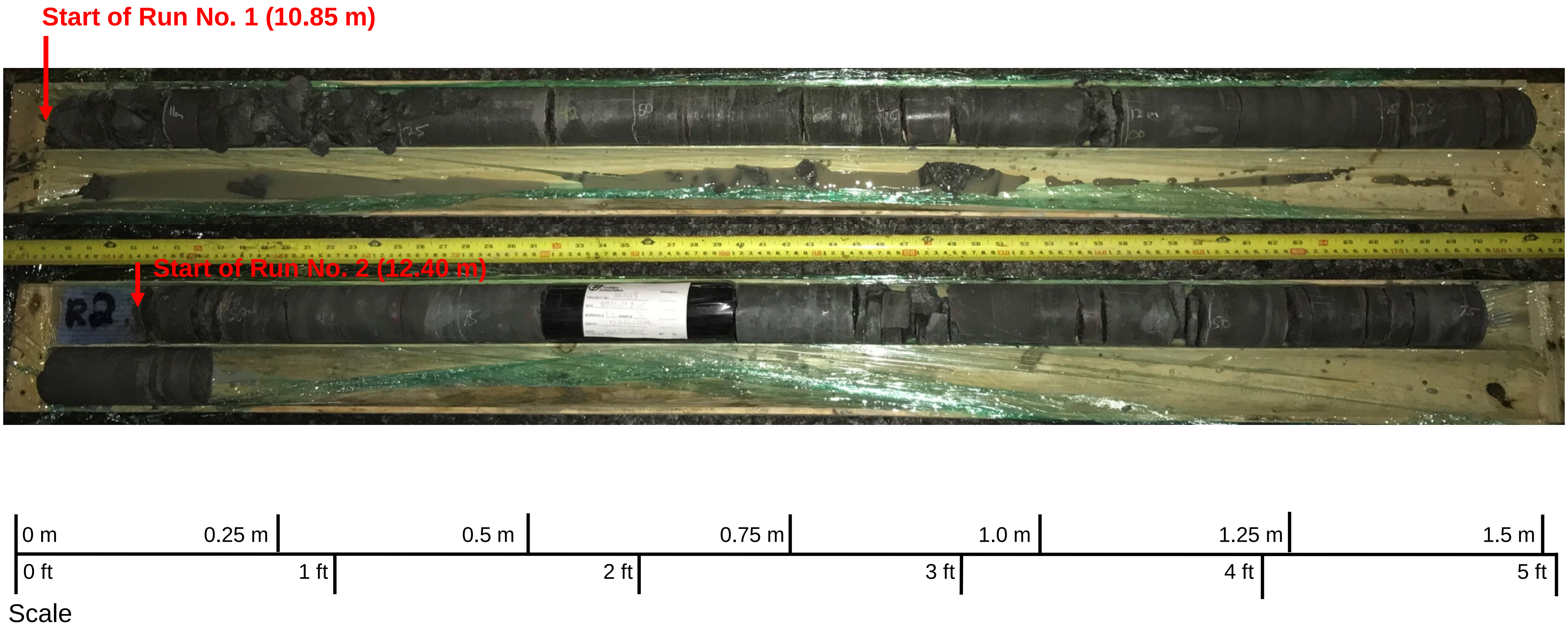


Scale

PROJECT	**MTO Assignment 2015-E-0033** **QEW Widening Between** **Mississauga Road and Hurontario Street**
TITLE	**Bedrock Core Photograph** **Borehole K3 (10.72 m to 14.07 m)**

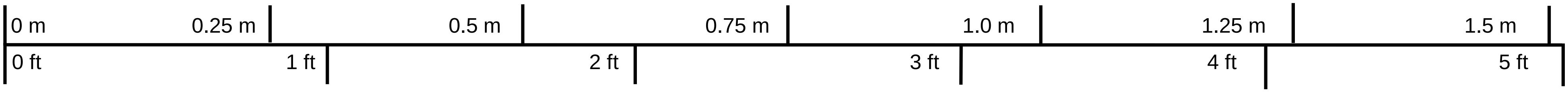
GOLDER

PROJECT No. 1662333			FILE No. ----	
DRAFT	KMG	Jan 2019	SCALE AS SHOWN	VER. 1.
CADD	--		**FIGURE B-11**	
CHECK	SMM	Apr 2019		
REVIEW	JMAC	Apr 2019		



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

PROJECT	**MTO Assignment 2015-E-0033 QEW Widening Between Mississauga Road and Hurontario Street**				
TITLE	**Bedrock Core Photograph Borehole K4 (10.85 m to 13.92 m)**				
GOLDER	PROJECT No. 1662333			FILE No. ----	
	DRAFT	KMG	Jan 2019	SCALE AS SHOWN	VER. 1.
	CADD	--		**FIGURE B-12**	
	CHECK	SMM	Apr 2019		
	REVIEW	JMAC	Apr 2019		



Scale

PROJECT

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

TITLE

Bedrock Core Photograph
Borehole K5 (12.83 m to 16.49 m)

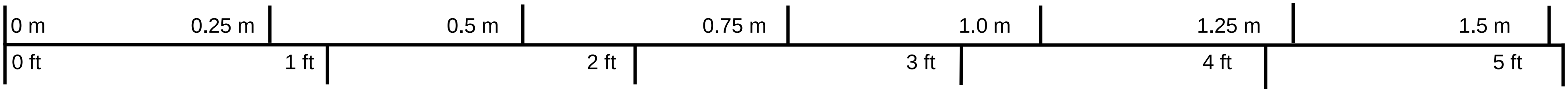
GOLDER

PROJECT No. 1662333			FILE No. ----	
DRAFT	KMG	Jan 2019	SCALE AS SHOWN	VER. 1.
CADD	--		**FIGURE B-13**	
CHECK	SMM	xx/xx/2019		
REVIEW	JMAC	xx/xx/2019		

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

Start of Run No. 1 (14.33 m)

Start of Run No. 2 (14.94 m)



Scale

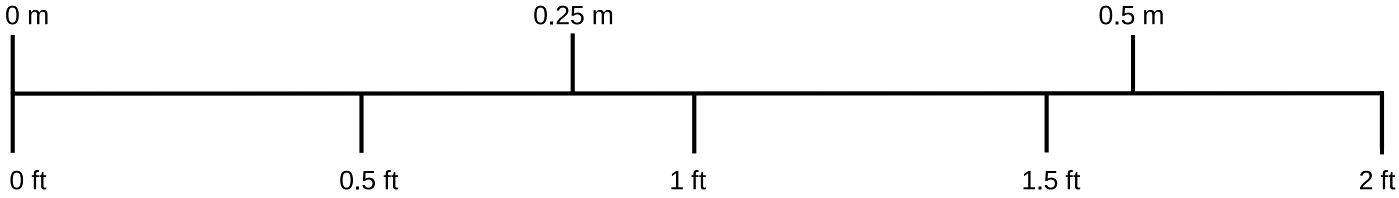
PROJECT	**MTO Assignment 2015-E-0033** **QEW Widening Between** **Mississauga Road and Hurontario Street**
TITLE	**Bedrock Core Photograph** **Borehole K6 (14.33 m to 15.02 m)**

GOLDER

PROJECT No. 1662333			FILE No. ----		
DRAFT	KMG	Jan 2019	SCALE	AS SHOWN	VER. 1.
CADD	--		**FIGURE B-14**		
CHECK	SMM	Apr 2019			
REVIEW	JMAC	Apr 2019			

Borehole NRW3-6

Scale



REVISION DATE: Februrary 13, 2019 BY: 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 NRW3-6 (7.47 m to 10.30)

GOLDER

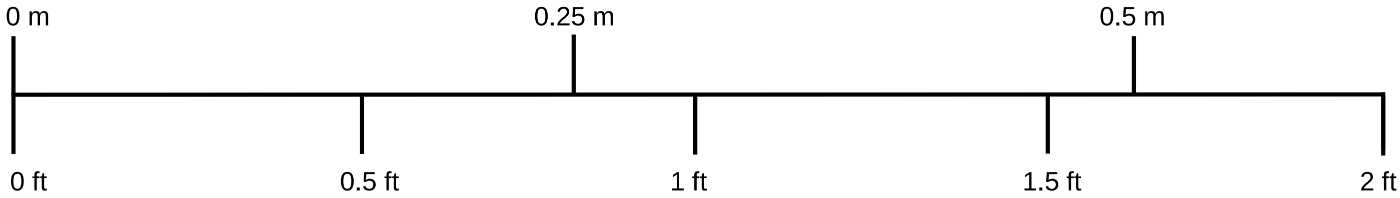
PROJECT No. 1662333			FILE No. ----	
DESIGN	JMP	Feb 2019	SCALE AS SHOWN.	VER. 1
CADD	-- --			
CHECK	SMM	Apr 2019	**FIGURE B-15**	
REVIEW	JMAC	Apr 2019		

Borehole NRW3-6

Start of Run No. 4 (10.57 m)

End of Run No. 4 (11.38 m)

Scale



REVISION DATE: February 13, 2019 BY: 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 NRW3-6 (10.30 m to 11.38)**

GOLDER	PROJECT No.	1662333		FILE No.	----	
	DRAFT	JMP	Feb 2019	SCALE	AS SHOWN.	VER. 1
	CADD	-- --		**FIGURE B-15**		
	CHECK	SMM	Apr 2019			
	REVIEW	JMAC	Apr 2019			

APPENDIX C

Geomechanica Rock Testing Results

GEOMECHANICA

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

December 20, 2018

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

Re: UCS + E testing
(Golder Project No. 1662333-8006)

Dear Mr. Marmor:

On Decmeber 6, 2018 three 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 1662333-8006. A total of 3 uniaxial compressive strength (UCS) specimens were prepared and tested from these samples.

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
lab@geomechanica.com

December 20, 2018
Project number: 1662333-8006

Abstract

This document summarizes the results of rock laboratory testing, including the result of 3 uniaxial compressive strength (UCS) tests. The uniaxial compressive strength (UCS) and tangent Young's modulus along with photographs of samples before and after testing are presented herein.

In this document:

1 Uniaxial Compressive Strength Tests

1.1 Overview

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

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

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

Using a precision V-block mounted on the magnetic chuck of the surface grinder, test specimens met the end flatness, end parallelism, and perpendicularity criteria set out in ASTM D4543-08. The side straightness criteria, as checked with a feeler gauge, was met for all samples and the minimum length:diameter criteria was met for all specimens unless noted otherwise in Table 1. Testing of the specimens followed ASTM D7012-14 with the following exceptions:

- Rather than a spherical seat diameter equal to 1 to 2 times the specimen diameter, the setup used here employed a 25.4 mm diameter high precision ball bearing and seat. Despite the smaller diameter, this seat could move freely to accommodate small angular rotations in any direction, as needed, and therefore did not appreciably influence the results.
- The tests presented herein included the measurement of the UCS and Young's (elastic) modulus, but not the Poisson's ratio. This represents a hybrid between Methods C and D of ASTM D7012-14.

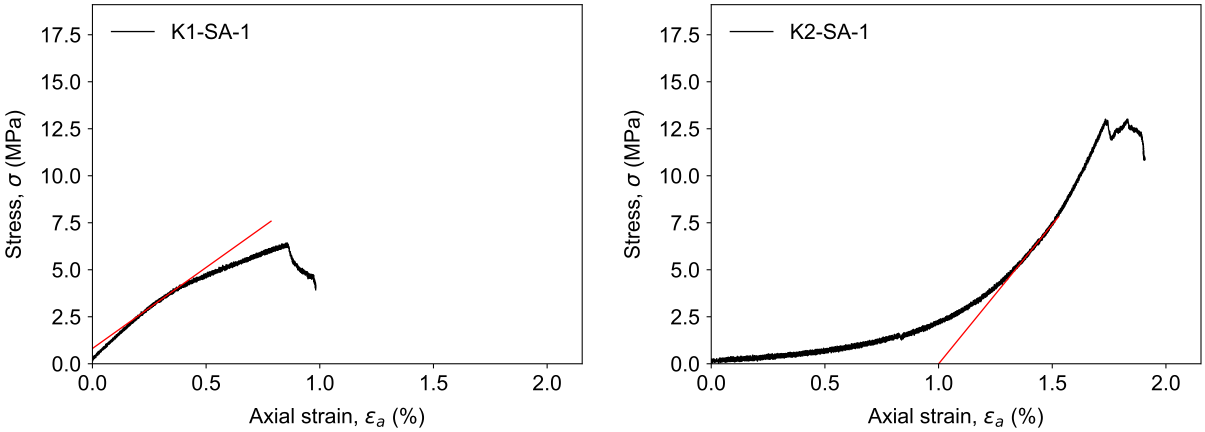
1.2 Results

The results of 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.

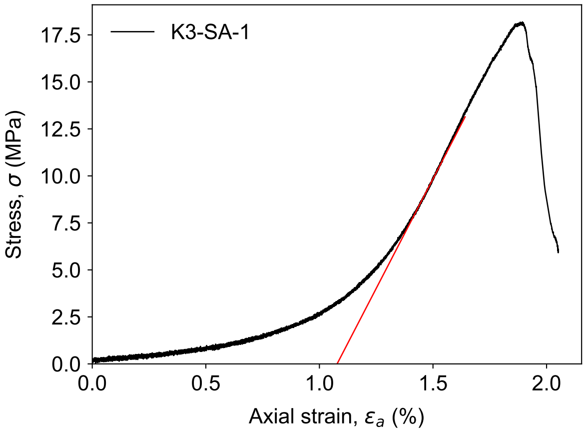
Table 1: Summary of Uniaxial Compression test results.

Sample	Depth (m)	Bulk density ρ (g/cm^3)	UCS (MPa)	Young's modulus E (GPa)	Lithology	Failure description
K1-SA-1	3.14 - 3.28	2.576	6.4	0.9	Georgian Bay Formation - shale with limestone inclusions	1
K2-SA-1	8.05 - 8.22	2.550	13.0	1.5	Georgian Bay Formation - weathered shale	1
K3-SA-1	11.87 - 12.06	2.576	18.2	2.3	Georgian Bay Formation - very weathered shale	1
	Average	2.567	12.5	1.6		
	Standard deviation	0.012	4.8	0.6		

[1] Axial splitting failure



(a) K1-SA-1 (b) K2-SA-1



(c) K3-SA-1

Figure 2: Measured stress-strain curves.

1.3 Specimen photographs

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

Appendices

Specimen sheets

- K1-SA-1
- K2-SA-1
- K3-SA-1

Uniaxial Compression Test

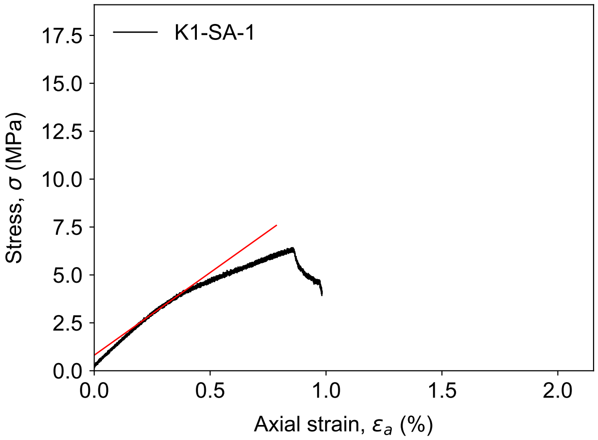
Client	Golder Associates Ltd.	**Project**	1662333-8006
Sample	K1-SA-1	**Depth**	3.14 - 3.28

Specimen parameters	
Diameter (mm) [a]	63.07
Length (mm) [a]	136.54
Bulk density ρ (g/cm^3)	2.576
UCS (MPa)	6.4
Young's modulus E (GPa) [b]	0.9
Lithology	Georgian Bay Formation - shale with limestone inclusions
Failure description [c]	1

[a] Additional specimen measurement/details provides in accompanying summary spreadsheet.
[b] Tangent modulus, calculated as the slope of the best fit line through ±300 data points on either side of the point representing 50.0% of the peak strength.
[c] Failure description: [1] Axial splitting failure;

Prior to testing

After testing



Remarks:

Performed by	BSAT	**Date**	2018-12-19

Uniaxial Compression Test

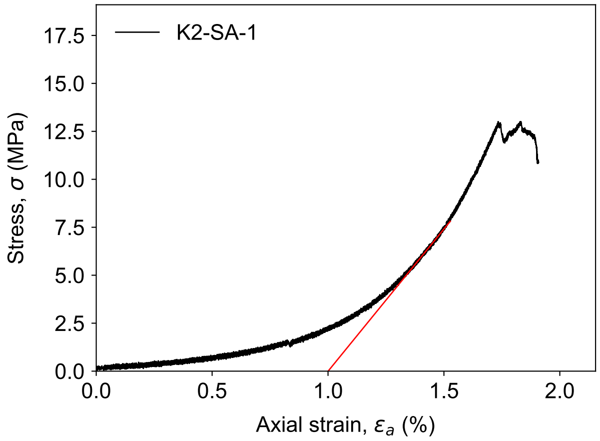
Client	Golder Associates Ltd.	**Project**	1662333-8006
Sample	K2-SA-1	**Depth**	8.05 - 8.22

Specimen parameters	
Diameter (mm) [a]	60.20
Length (mm) [a]	127.31
Bulk density ρ (g/cm^3)	2.550
UCS (MPa)	13.0
Young's modulus E (GPa) [b]	1.5
Lithology	Georgian Bay Formation - weathered shale
Failure description [c]	1

Prior to testing

After testing

[a] Additional specimen measurement/details provides in accompanying summary spreadsheet.
[b] Tangent modulus, calculated as the slope of the best fit line through ±300 data points on either side of the point representing 50.0% of the peak strength.
[c] Failure description: [1] Axial splitting failure;



Remarks:

Performed by	BSAT	**Date**	2018-12-19

Uniaxial Compression Test

Client	Golder Associates Ltd.	**Project**	1662333-8006
Sample	K3-SA-1	**Depth**	11.87 - 12.06

Specimen parameters	
Diameter (mm) [a]	60.19
Length (mm) [a]	123.03
Bulk density ρ (g/cm^3)	2.576
UCS (MPa)	18.2
Young's modulus E (GPa) [b]	2.3
Lithology	Georgian Bay Formation - very weathered shale
Failure description [c]	1

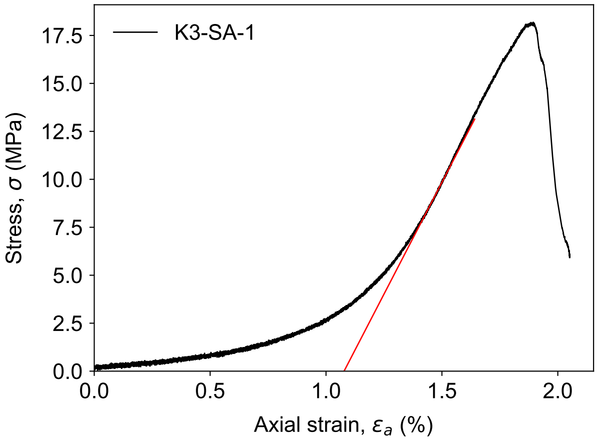
Prior to testing

After testing

[a] Additional specimen measurement/details provides in accompanying summary spreadsheet.

[b] Tangent modulus, calculated as the slope of the best fit line through ±300 data points on either side of the point representing 50.0% of the peak strength.

[c] Failure description: [1] Axial splitting failure;



Remarks:

Performed by	BSAT	**Date**	2018-12-19

GEOMECHANICA

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

January 03, 2018

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 25, 2017 one (1) HQ-sized core sample was received by Geomechanica Inc. via drop-off by Golder personnel. On December 22, 2017 an additional three (3) 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 166233 (denoted as QEW South Ped. Bridge and QEW and Mississauga Road 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.

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Rock Laboratory Testing Results

A report submitted to:

David Marmor
Golder Associates Ltd.
6925 Century Avenue, Suite #100
Mississauga, Ontario
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Prepared by:

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January 3, 2018
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 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. 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.

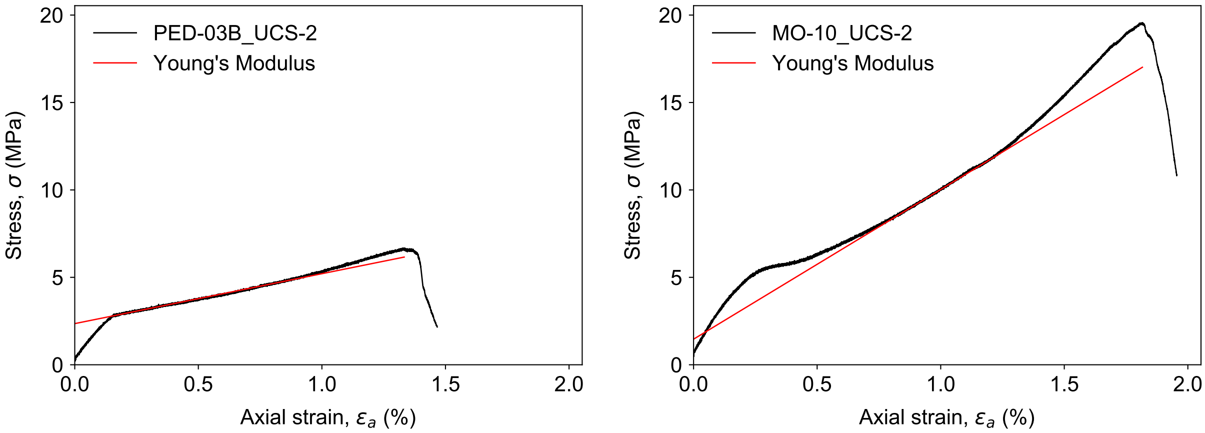
Table 1: Summary of laboratory test results.

Sample	Depth (m)	Bulk density ρ (g/cm^3)	UCS (MPa)	Young's Modulus E (GPa)	Notes
PED-03B, UCS-2	16.03 -16.27	2.57	6.7	0.29	[1]
MO-10, UCS-2	2.68 - 2.83	2.60	19.6	0.86	[1]
MO-12, UCS-2	4.15 - 4.27	2.60	17.3	1.00	[1,2]
MO-11, UCS-3	3.66 - 3.79	2.59	18.3	0.97	[1,2,3] - 2 layers 8 - 20 mm thick
Mean		2.59	15.5	0.8	
Standard Deviation		0.02	5.1	0.3	

[1] Specimen emitted fresh pore water upon loading
[2] Length:diameter ratio $< 2:1$
[3] Contains limestone layers

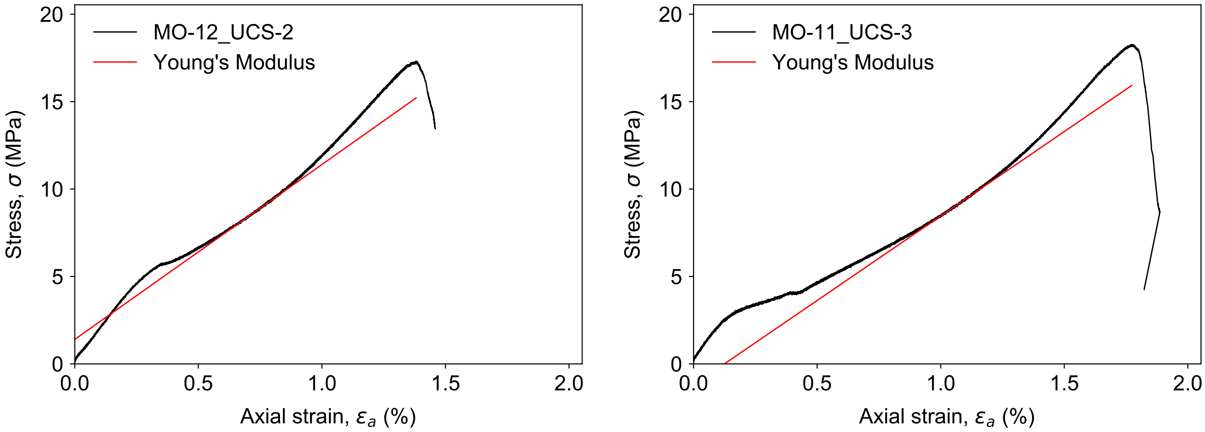
2.1 Specimen photographs

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



(a) PED-03B, UCS-2

(b) MO-10, UCS-2



(c) MO-12, UCS-2

(d) MO-11, UCS-3

Figure 2: Measured stress-strain curves.

PED-03B, UCS-2
16.03 m – 16.27 m

MO-10, UCS-2
2.68 m – 2.83 m

MO-12, UCS-3
4.15 m – 4.27 m

MO-11, UCS-3
3.66 m – 3.79 m

Figure 3: Photographs of specimens prior to testing.

APPENDIX D

Analytical Test Reports (Maxxam Analytics)

Your Project #: 1662333
Site Location: QEW/CREDIT RIVER
Your C.O.C. #: 641804-09-01

Attention: Jane Peter

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

Report Date: 2018/12/12
Report #: R5522742
Version: 1 - Final

CERTIFICATE OF ANALYSIS

MAXXAM JOB #: B8W6758
Received: 2018/12/06, 12:29

Sample Matrix: ROCK
Samples Received: 1

Analyses	Quantity	Date Extracted	Date Analyzed	Laboratory Method	Reference
Chloride (20:1 extract)	1	N/A	2018/12/12	CAM SOP-00463	EPA 325.2 m
Conductivity	1	N/A	2018/12/12	CAM SOP-00414	OMOE E3530 v1 m
pH CaCl2 EXTRACT	1	2018/12/12	2018/12/12	CAM SOP-00413	EPA 9045 D m
Resistivity of Soil	1	2018/12/11	2018/12/12	CAM SOP-00414	SM 23 2510 m
Sulphate (20:1 Extract)	1	N/A	2018/12/12	CAM SOP-00464	EPA 375.4 m

Sample Matrix: Soil
Samples Received: 4

Analyses	Quantity	Date Extracted	Date Analyzed	Laboratory Method	Reference
Chloride (20:1 extract)	4	N/A	2018/12/12	CAM SOP-00463	EPA 325.2 m
Conductivity	4	N/A	2018/12/12	CAM SOP-00414	OMOE E3530 v1 m
pH CaCl2 EXTRACT	4	2018/12/12	2018/12/12	CAM SOP-00413	EPA 9045 D m
Resistivity of Soil	4	2018/12/11	2018/12/12	CAM SOP-00414	SM 23 2510 m
Sulphate (20:1 Extract)	4	N/A	2018/12/12	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. Where applicable, unless otherwise noted, Measurement Uncertainty has not been accounted for when stating conformity to the referenced standard.

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. Maxxam is not responsible for the accuracy or any data impacts, that result from the information provided by the customer or their agent.

Solid sample results, except biota, are based on dry weight unless otherwise indicated. Organic analyses are not recovery corrected except for isotope dilution methods.

Your Project #: 1662333
Site Location: QEW/CREDIT RIVER
Your C.O.C. #: 641804-09-01

Attention: Jane Peter

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

Report Date: 2018/12/12
Report #: R5522742
Version: 1 - Final

CERTIFICATE OF ANALYSIS

MAXXAM JOB #: B8W6758
Received: 2018/12/06, 12:29

Results relate to samples tested. When sampling is not conducted by Maxxam, results relate to the supplied 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.

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 #: B8W6758
Report Date: 2018/12/12

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

SOIL CORROSIVITY PACKAGE (ROCK)

Maxxam ID		IMF969		
Sampling Date		2018/12/06 11:45		
COC Number		641804-09-01		
	UNITS	**1.75M TO 1.83M K1-CORROSIVITY #1**	**RDL**	**QC Batch**
Calculated Parameters				
Resistivity	ohm-cm	2700		5882461
Inorganics				
Soluble (20:1) Chloride (Cl-)	ug/g	53	20	5883825
Conductivity	umho/cm	372	2	5883994
Available (CaCl2) pH	pH	7.73		5883840
Soluble (20:1) Sulphate (SO4)	ug/g	97	20	5883826

RDL = Reportable Detection Limit
QC Batch = Quality Control Batch

SOIL CORROSIVITY PACKAGE (SOIL)

Maxxam ID		IMF965	IMF966	IMF967	IMF968		
Sampling Date		2018/12/04 16:30	2018/12/04 16:30	2018/12/04 16:30	2018/12/04 16:30		
COC Number		641804-09-01	641804-09-01	641804-09-01	641804-09-01		
	UNITS	**15'-17' K3-SS7**	**25'-25'8'' S4-SS9A**	**25'-26'6'' S2-SS9**	**25'-27' S5-SS9**	**RDL**	**QC Batch**
Calculated Parameters							
Resistivity	ohm-cm	810	720	1500	680		5882461
Inorganics							
Soluble (20:1) Chloride (Cl-)	ug/g	600	600	37	760	20	5883825
Conductivity	umho/cm	1230	1390	661	1480	2	5883994
Available (CaCl2) pH	pH	7.10	7.62	7.77	7.04		5883840
Soluble (20:1) Sulphate (SO4)	ug/g	210	260	550	<20	20	5883826

RDL = Reportable Detection Limit
QC Batch = Quality Control Batch

TEST SUMMARY

Maxxam ID: IMF965
Sample ID: 15'-17' K3-SS7
Matrix: Soil

Collected: 2018/12/04
Shipped:
Received: 2018/12/06

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	5883825	N/A	2018/12/12	Deonarine Ramnarine
Conductivity	AT	5883994	N/A	2018/12/12	Kazzandra Adeva
pH CaCl2 EXTRACT	AT	5883840	2018/12/12	2018/12/12	Gnana Thomas
Resistivity of Soil		5882461	2018/12/12	2018/12/12	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	5883826	N/A	2018/12/12	Deonarine Ramnarine

Maxxam ID: IMF966
Sample ID: 25'-25'8'' S4-SS9A
Matrix: Soil

Collected: 2018/12/04
Shipped:
Received: 2018/12/06

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	5883825	N/A	2018/12/12	Deonarine Ramnarine
Conductivity	AT	5883994	N/A	2018/12/12	Kazzandra Adeva
pH CaCl2 EXTRACT	AT	5883840	2018/12/12	2018/12/12	Gnana Thomas
Resistivity of Soil		5882461	2018/12/12	2018/12/12	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	5883826	N/A	2018/12/12	Deonarine Ramnarine

Maxxam ID: IMF967
Sample ID: 25'-26'6'' S2-SS9
Matrix: Soil

Collected: 2018/12/04
Shipped:
Received: 2018/12/06

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	5883825	N/A	2018/12/12	Deonarine Ramnarine
Conductivity	AT	5883994	N/A	2018/12/12	Kazzandra Adeva
pH CaCl2 EXTRACT	AT	5883840	2018/12/12	2018/12/12	Gnana Thomas
Resistivity of Soil		5882461	2018/12/12	2018/12/12	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	5883826	N/A	2018/12/12	Deonarine Ramnarine

Maxxam ID: IMF968
Sample ID: 25'-27' S5-SS9
Matrix: Soil

Collected: 2018/12/04
Shipped:
Received: 2018/12/06

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	5883825	N/A	2018/12/12	Deonarine Ramnarine
Conductivity	AT	5883994	N/A	2018/12/12	Kazzandra Adeva
pH CaCl2 EXTRACT	AT	5883840	2018/12/12	2018/12/12	Gnana Thomas
Resistivity of Soil		5882461	2018/12/12	2018/12/12	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	5883826	N/A	2018/12/12	Deonarine Ramnarine

Maxxam ID: IMF969
Sample ID: 1.75M TO 1.83M K1-CORROSIVITY #1
Matrix: ROCK

Collected: 2018/12/06
Shipped:
Received: 2018/12/06

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	5883825	N/A	2018/12/12	Deonarine Ramnarine
Conductivity	AT	5883994	N/A	2018/12/12	Kazzandra Adeva

TEST SUMMARY

Maxxam ID: IMF969
Sample ID: 1.75M TO 1.83M K1-CORROSIVITY #1
Matrix: ROCK

Collected: 2018/12/06
Shipped:
Received: 2018/12/06

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
pH CaCl2 EXTRACT	AT	5883840	2018/12/12	2018/12/12	Gnana Thomas
Resistivity of Soil		5882461	2018/12/12	2018/12/12	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	5883826	N/A	2018/12/12	Deonarine Ramnarine

GENERAL COMMENTS

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

Package 1	5.3°C

Sample IMF969 [1.75M TO 1.83M K1-CORROSIVITY #1] : Sample analyzed for Corrosivity package to include Chloride, Sulphate, pH and Conductivity as per client request.

Results relate only to the items tested.

QUALITY ASSURANCE REPORT

			Matrix Spike		SPIKED BLANK		Method Blank		RPD	
QC Batch	Parameter	Date	% Recovery	QC Limits	% Recovery	QC Limits	Value	UNITS	Value (%)	QC Limits
5883825	Soluble (20:1) Chloride (Cl-)	2018/12/12	NC	70 - 130	102	70 - 130	<20	ug/g	1.9	35
5883826	Soluble (20:1) Sulphate (SO4)	2018/12/12	NC	70 - 130	103	70 - 130	<20	ug/g	24	35
5883840	Available (CaCl2) pH	2018/12/12			101	97 - 103			1.3	N/A
5883994	Conductivity	2018/12/12			103	90 - 110	<2	umho/cm	0.65	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).

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.

IMMEDIATE

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 COUSTODY RECORD

Page of

INVOICE TO:

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

REPORT TO:

Company Name: GOLDER ASSOCIATES
Attention: JANE PETER
Address: 6925 CENTURY AVE
MISSISSAUGA, ON, L5N7K2
Tel: (613) 9299467 Fax:
Email: Jane_peter@golder.com

PROJECT INFORMATION:

Quotation #: B70916
P.O. #:
Project: 1662333
Project Name: QEW/ Credit River
Site #:
Sampled By: JMP

Laboratory Use Only:

Maxxam Job #:	Bottle Order #:
	641804
COC #:	**Project Manager:**
C#641804-09-01	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~~ Depth:	Sample (Location) Identification	Date Sampled	Time Sampled	Matrix	Field Filtered (please circle):	Metals / Hg / Cr VI	CORROSIVITY (STANDARD PACKAGE)	# of Bottles	Comments
1	15'–17'	K3-SS7	04/12/18	4:30pm	SS			X	1	
2	25'-25'8"	S4-SS9A	04/12/18	4:30pm	SS			X	1	
3	25'-26'6"	S2-SS9	04/12/18	4:30pm	SS			X	1	
4	25'-27'	S5-SS9	04/12/18	4:30pm	SS			X	1	
5	1.75 m to 1.83 m	K1- Corrosivity #7	06/12/18	11:45am	ROCK					
6										
7										
8										
9										
10										

06-Dec-18 12:29
Ema Gitej
B8W6758
URE ENV-1116

* RELINQUISHED BY: (Signature/Print)	Date: (YY/MM/DD)	Time	RECEIVED BY: (Signature/Print)	Date: (YY/MM/DD)	Time	# jars used and not submitted
JANE PETER / Jane	~~18/12/~~ 18/12/05		KAITLYN VANGELVAREN	2018/12/06	12:29	

Laboratory Use Only

Time Sensitive	Temperature (°C) on Recei	Custody Seal	Yes	No
	8/3/5	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

Maxxam Analytics International Corporation o/a Maxxam Analytics

Your Project #: 1662333
Site#: K6
Site Location: QEW-CREDIT RIVER
Your C.O.C. #: 641804-07-01

Attention: Jane Peter

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

Report Date: 2018/12/07
Report #: R5516071
Version: 1 - Final

CERTIFICATE OF ANALYSIS

MAXXAM JOB #: B8V9829
Received: 2018/11/29, 18:29

Sample Matrix: Soil
Samples Received: 2

Analyses	Quantity	Date Extracted	Date Analyzed	Laboratory Method	Reference
Chloride (20:1 extract)	2	N/A	2018/12/06	CAM SOP-00463	EPA 325.2 m
Conductivity	2	N/A	2018/12/06	CAM SOP-00414	OMOE E3530 v1 m
pH CaCl2 EXTRACT	2	2018/12/06	2018/12/06	CAM SOP-00413	EPA 9045 D m
Resistivity of Soil	2	2018/12/01	2018/12/07	CAM SOP-00414	SM 23 2510 m
Sulphate (20:1 Extract)	2	N/A	2018/12/06	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. Where applicable, unless otherwise noted, Measurement Uncertainty has not been accounted for when stating conformity to the referenced standard.

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. Maxxam is not responsible for the accuracy or any data impacts, that result from the information provided by the customer or their agent.

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. When sampling is not conducted by Maxxam, results relate to the supplied 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#: K6
Site Location: QEW-CREDIT RIVER
Your C.O.C. #: 641804-07-01

Attention: Jane Peter

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

Report Date: 2018/12/07
Report #: R5516071
Version: 1 - Final

CERTIFICATE OF ANALYSIS

MAXXAM JOB #: B8V9829
Received: 2018/11/29, 18:29

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		IKS868			IKS868			IKS869		
Sampling Date		2018/11/28 07:24			2018/11/28 07:24			2018/11/28 07:24		
COC Number		641804-07-01			641804-07-01			641804-07-01		
	UNITS	**K6-SS5**	**RDL**	**QC Batch**	**K6-SS5 Lab-Dup**	**RDL**	**QC Batch**	**S6-SS9**	**RDL**	**QC Batch**
Calculated Parameters										
Resistivity	ohm-cm	640		5867074				840		5867074
Inorganics										
Soluble (20:1) Chloride (Cl-)	ug/g	830	20	5874021	790	20	5874021	630	20	5874021
Conductivity	umho/cm	1550	2	5874376				1190	2	5874376
Available (CaCl2) pH	pH	7.65		5873908				7.19		5873908
Soluble (20:1) Sulphate (SO4)	ug/g	46	20	5874022	30	20	5874022	<20	20	5874022

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

TEST SUMMARY

Maxxam ID: IKS868
Sample ID: K6-SS5
Matrix: Soil

Collected: 2018/11/28
Shipped:
Received: 2018/11/29

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	5874021	N/A	2018/12/06	Alina Dobreanu
Conductivity	AT	5874376	N/A	2018/12/06	Kazzandra Adeva
pH CaCl2 EXTRACT	AT	5873908	2018/12/06	2018/12/06	Gnana Thomas
Resistivity of Soil		5867074	2018/12/07	2018/12/07	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	5874022	N/A	2018/12/06	Alina Dobreanu

Maxxam ID: IKS868 Dup
Sample ID: K6-SS5
Matrix: Soil

Collected: 2018/11/28
Shipped:
Received: 2018/11/29

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	5874021	N/A	2018/12/06	Alina Dobreanu
Sulphate (20:1 Extract)	KONE/EC	5874022	N/A	2018/12/06	Alina Dobreanu

Maxxam ID: IKS869
Sample ID: S6-SS9
Matrix: Soil

Collected: 2018/11/28
Shipped:
Received: 2018/11/29

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	5874021	N/A	2018/12/06	Alina Dobreanu
Conductivity	AT	5874376	N/A	2018/12/06	Kazzandra Adeva
pH CaCl2 EXTRACT	AT	5873908	2018/12/06	2018/12/06	Gnana Thomas
Resistivity of Soil		5867074	2018/12/07	2018/12/07	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	5874022	N/A	2018/12/06	Alina Dobreanu

GENERAL COMMENTS

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

Package 1	3.0°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**
5873908	Available (CaCl2) pH	2018/12/06			100	97 - 103			0.98	N/A
5874021	Soluble (20:1) Chloride (Cl-)	2018/12/06	NC	70 - 130	102	70 - 130	<20	ug/g	5.0	35
5874022	Soluble (20:1) Sulphate (SO4)	2018/12/06	NC	70 - 130	109	70 - 130	<20	ug/g	NC	35
5874376	Conductivity	2018/12/06			104	90 - 110	<2	umho/cm	0.44	10

N/A = Not Applicable

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

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

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

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

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

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

VALIDATION SIGNATURE PAGE

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

Eva P

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

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: JANE PETER
Address: 6925 CENTURY AVE
MISSISSAUGA ON, L5N7K2
Tel: (613) 929 9467 Fax:
Email: Jane_peter@golder.com

PROJECT INFORMATION:

Quotation #: B70916
P.O. #:
Project: QEW - Credit River
Project Name: 1662333
Site #: K6
Sampled By: JMP

Laboratory Use Only:

Maxxam Job #:
Bottle Order #: 641804
COC #: C#641804-07-01
Project Manager: Ema Gitej

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

Regulation 153 (2011)

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

Other Regulations

☐ CCME ☐ Sanitary Sewer Bylaw
☐ Reg 558 ☐ Storm Sewer Bylaw
☐ MISA Municipality ________
☐ PWQO
☒ Other O. Reg.

Special Instructions

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

ANALYSIS REQUESTED (PLEASE BE SPECIFIC)

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

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

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

	Sample Barcode Label	Sample (Location) Identification	Date Sampled	Time Sampled	Matrix	Field Filtered (please circle):	Metals / Hg / Cr VI	Corrosivity	# of Bottles	Comments
1	K6-SS5	K6-SS5	08 Nov 2018	7:24	SS			X	1	
2	S6-SS9	S6-SS9	8 ''	''	SS			X	1	limited samples
3										
4										
5										
6										
7										
8										
9										
10										

29-Nov-18 18:29
Ema Gitej
B8V9829
CA2 SRFumeHood

CAUTION
Reason: Matrix Unclear

* RELINQUISHED BY: (Signature/Print)	Date: (YY/MM/DD)	Time	RECEIVED BY: (Signature/Print)	Date: (YY/MM/DD)	Time	# jars used and not submitted
JANE PETER / Jane	18/11/28	7:24pm	KAITLYN VANGRUNSVEN	2018/11/29	18:29	

Laboratory Use Only

Time Sensitive
Temperature (°C) on Recei: 2/4/3

Custody Seal	Yes	No
Present		✓
Intact		

White: Maxxa Yellow: Client

* 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

APPENDIX E

Non-Standard Special Provisions, Notice to Contractor and Special Provisions

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

PROTECTION SYSTEM – Item No.

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 level.

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

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

VIBRATION MONITORING – Item No.

Special Provision

TABLE OF CONTENTS

1.0 SCOPE

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

2.0 REFERENCES

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

> FOUNDATION INVESTIGATION REPORT
> Queen Elizabeth Way (QEW) - Credit River Bridge, Structure Site No. 24-203, QEW Widening from West of Mississauga Road to West of Hurontario Street, Mississauga, Ministry of Transportation, Ontario, GWP 2002-13-00
>
> FOUNDATION INVESTIGATION REPORT
> Queen Elizabeth Way (QEW) – Mississauga Road Overpass Replacement, Structure Site No. 24-196, QEW Widening from West of Mississauga Road to West of Hurontario Street, Mississauga, Ministry of Transportation, Ontario, GWP 2002-13-00
>
> FOUNDATION INVESTIGATION REPORT

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

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

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

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

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

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

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

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

3.0 DEFINITIONS

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

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

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

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

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

4.0 DESIGN AND SUBMISSION REQUIREMENTS

4.1 Submission Requirements

The Contractor/Contractor's Engineer shall submit details of the vibration monitoring plan to the Contract Administrator for information purposes at least 2 weeks prior to any work related to strip footing, deep foundation, cofferdam and temporary protection system installation. The submittals shall satisfy the specifications and at a minimum contain the following specific information:

a) Equipment and methods used by the Contractor to perform the work that may cause undue vibration.
b) Qualifications of vibration monitoring specialist.
c) Details regarding proposed instrumentation.
d) Proposed location of instruments adjacent to the on the residences, structures, utilities, wells, or other potentially vibration-sensitive structures within a 250 m radius from the excavation and installation of spread/strip footings, cofferdams, deep foundations and temporary protection systems, as applicable.
e) Proposed frequency of readings.
f) Action plan to be taken to adjust excavation, deep foundation and protection system installation methods if readings show vibrations exceeding tolerable levels.

6.0 EQUIPMENT

6.1 Vibration Monitoring Equipment

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

7.0 CONSTRUCTION

7.1 Pre- and Post-Construction Condition Surveys

A Pre-Construction Condition Survey and Post-Construction Condition Survey shall be prepared for all buildings, utilities, structures, water wells, and facilities within 250 m of excavation and installation of spread/strip footings, cofferdams, deep foundations and temporary protection systems.

7.1.1 Pre-Construction Condition Surveys

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

The Pre-Construction Condition Survey, at each structure/well within a 250 m radius of excavation and installation of spread/strip footings, cofferdams, deep foundations and temporary protection systems, shall be completed a minimum of two (2) weeks prior to commencement of excavation and installation of shallow and deep foundations and temporary protection systems. Only one Pre-Construction Condition Survey per structure or facility is required to be carried out in advance of excavation and installation of spread/strip footings, cofferdams, deep foundations or temporary protection system installation, unless more than six (6) months will elapse between these operations, in which case an interim inspection will be required.

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

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

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

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

7.1.2 Post-Construction Condition Surveys

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

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

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

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

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

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

7.2 Monitoring

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

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

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

7.3 Records

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

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

10.0 BASIS OF PAYMENT

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

NOTICE TO CONTRACTOR – 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.

<u>NOTICE TO CONTRACTOR</u> – Stability of Excavation Base Near Stavebank and Kenollie Creek Culvert

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Special Provision

The Contactor shall be alerted to the groundwater elevation and conditions at the Stavebank and Kenollie Creek Culvert sites. The subsurface conditions at the site are described in the following report:

> 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

The groundwater in the native silty sand to sand deposit present underlying the clayey silt with sand deposit at the Kenollie Creek Culvert site is under hydrostatic pressure. Prior to any excavation for the culvert, the Contractor shall design and install an appropriate dewatering system and temporary protection system to enable construction of the culvert in such a way as to prevent disturbance to the founding soils. Lowering of the groundwater level to 1 m below the underside the base of the excavation shall be undertaken prior to commencing any excavation.

The native silt and sand to silty sand to sand deposit present underlying the fill materials is wet. Prior to any excavation for the culvert, the Contractor shall design and install an appropriate dewatering system and temporary protection system to enable construction of the culvert in such a way as to prevent disturbance to the founding soils. The groundwater at the Stavebank Creek Culvert site must be lowered to 1 m below the underside the base of the excavation shall be undertaken prior to commencing any excavation.

The dewatering system design shall be completed by a design Engineer and design-checking Engineer, both of whom shall have a minimum 5 years experience in designing systems of similar nature and scope to the required work.

NOTICE TO CONTRACTOR – Rock Excavation

Special Provision

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

DEWATERING SYSTEM - Item No.
TEMPORARY FLOW PASSAGE SYSTEM - 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.

WARRANT: Always with these tender items.

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