



May 23, 2018

## FOUNDATION INVESTIGATION AND DESIGN 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**

**Submitted to:**

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REPORT

**GEOCRES No.: 30M12-418**

**Report Number: 1662333-3**

**Distribution:**

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# **PART A**

**FOUNDATION INVESTIGATION REPORT  
QEW – MISSISSAUGA ROAD OVERPASS REPLACEMENT, SITE NO. 24-196  
QEW WIDENING FROM WEST OF MISSISSAUGA ROAD TO WEST OF  
HURONTARIO STREET, CITY OF MISSISSAUGA  
MINISTRY OF TRANSPORTATION, ONTARIO  
GWP 2002-13-00**



## **1.0 INTRODUCTION**

Golder Associates Ltd. (Golder) has been retained by Morrison Hershfield Limited (MH) on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services for the proposed replacement and widening of the Queen Elizabeth Way (QEW) Overpass at Mississauga Road in support of the widening of the QEW from west of Mississauga Road to west of Hurontario Street in the City of Mississauga, in the Regional Municipality of Peel, Ontario.

The purpose of this investigation is to establish the subsurface soil and bedrock conditions at the proposed replacement structure location, including the associated approach embankments, by borehole drilling, rock coring, and laboratory testing on selected soil and rock core samples.

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

## **2.0 SITE DESCRIPTION**

At this site the QEW is generally oriented in a northeast-southwest direction; for the purpose of this report the QEW is described as being in an east-west orientation. The existing QEW-Mississauga Road Overpass is located approximately 4.1 km east of the QEW-Erin Mills Interchange and approximately 2.1 km west of the QEW - Hurontario Street Interchange and approximately 500 m west of the west bank of the Credit River. Street Interchange, approximately 500 m west of the west bank of the Credit River.

The existing overpass is an approximately 17.3 long and 35.6 m wide single span bridge, with abutments supported on spread footings founded on bedrock at about Elevation 91.6 m. At the time of overpass construction, the surface grade at Mississauga Road was lowered, to allow for the QEW to pass over the roadway, and therefore the heights of the approach embankments is minimal. The natural ground surface, in the immediate vicinity of the overpass is about Elevation 95 m and the grade of the QEW is between about Elevations 99.4 m and 99.8 m. The Mississauga Road grade under the QEW is at about Elevation 93 m.

Land use in the northeast, northwest, southeast and southwest quadrant of the interchange is primarily residential with scattered light commercial development. A golf course is located to the north of QEW and the Credit River and the Credit River valley are located to the east of Mississauga Road.

## **3.0 INVESTIGATION PROCEDURES**

### **3.1 Previous Investigation**

In September 2011, a preliminary foundation investigation for the Mississauga Road Overpass replacement was carried out at the site by Thurber Engineering Ltd. (Thurber) during which time a total of two boreholes, designated as Boreholes MR11-01 and MR11-02, were advanced. The results of the Thurber investigation are contained in their report titled "Foundation Investigation and Design Report, Preliminary Design and Environmental



Assessment, QEW Mississauga Road Overpass, Mississauga Ontario" File No. 19-1351-174, dated May 14, 2012 (GEOCRE 30M12-342).

The locations of the boreholes advanced by Thurber are shown on Drawing 1, and the borehole records and the summary of the laboratory testing results from this investigation are presented in Appendix A. The borehole locations in MTM NAD 83 Zone 10 coordinates, ground surface elevations in Geodetic Datum and the drilled depths as taken from the borehole records are as follows:

Borehole No.	Location (MTM NAD 83)		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing	Easting		
MR11-01	4,823,600.3	295,564.2	94.3	3.1
MR11-02	4,823,601.1	295,645.2	93.3	7.7

### 3.2 Current Investigation

The field work for the current foundation investigation was carried out between August 22 and September 7, 2017 and between December 19 and 21, 2017 during which time twelve sampled boreholes (designated as Boreholes MO-01 to MO-12) were advanced in the area of the structure, at the locations shown on Drawing 1. A cluster of borings was advanced in the area of Boreholes MO-01 and MO-08 (Boreholes are designated as MO-01A to MO-01C and MO-08A to MO-08B) to confirm the depth to bedrock and provide confirmation of subsurface conditions at the locations of the primary boreholes. The Record of Borehole/Drillhole sheets for the current investigation are presented in Appendix B.

The field borehole investigation was carried out using a truck-mounted CME 75 or CME 55 drill rig, supplied and operated by Aardvark Drilling Inc., of Guelph, Ontario, a track-mounted CME 55 drill rig, supplied and operated by Geo-Environmental Drilling Inc., of Acton, Ontario, and using portable drilling equipment supplied and operated by Walker Drilling Ltd., of Utopia, Ontario. The boreholes were advanced through the overburden using 203 mm outer-diameter hollow stem augers and 'HQ' casing in the boreholes advanced by a drill rig and continuous split spoon sampling in the boreholes advanced by the portable drilling equipment (Boreholes MO-01A to MO-01C, MO-08A and MO-08B). In the boreholes where continuous sampling was not carried out, soil samples were obtained at 0.75 m and 1.5 m intervals of depth, using a 50 mm outer diameter (O.D.) split-spoon sampler driven by an automatic hammer in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586-08)<sup>1</sup>. Core samples of the bedrock in select boreholes were obtained using an 'HQ' size rock core barrel and coring techniques.

The boreholes were typically advanced to sampler refusal and bedrock was confirmed by either split spoon sampling or rock coring. The boreholes were advanced to depths of about 1.2 m and 7.0 m below existing ground surface, including coring of bedrock for core lengths of between 3.0 m and 4.7 m in select boreholes. Photographs of the recovered bedrock core samples are provided in Appendix C.

<sup>1</sup> ASTM D1586-08a – Standard Test Method for Standard Penetration Tests and Split Barrel Sampling of the soil.





The water levels in the open boreholes and field moisture content of the recovered soil samples were observed during the drilling operations and are noted on the Record of Borehole sheets in Appendix B. All boreholes were backfilled with bentonite upon completion in accordance with Ontario Regulation 903, Wells (as amended).

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

Selected bedrock core samples were submitted to Maxxam Analytics (Maxxam), a Standards Council of Canada (SCC) accredited laboratory of Mississauga, Ontario for chemical analysis. The samples of bedrock core, specifically collected from Boreholes MO-10 and MO-11 advanced at the west and east abutments, respectively, were crushed and homogenized by Maxxam prior to testing, and analyzed for a suite of corrosivity parameters, including conductivity, resistivity, soluble chloride, soluble sulphate and pH. The results of the chemical analyses are presented in Appendix C.

The as-drilled borehole locations and the ground surface elevations were obtained using a GPS (Trimble XH 3.5G), having an accuracy of 0.1 m in the vertical and 0.1 m in the horizontal directions. The locations given in the Record of Borehole/Drillhole sheets and shown on Drawing 1 are positioned relative to MTM NAD 83 (Zone 10) northing and easting coordinates and the ground surface elevations are referenced to Geodetic datum. The borehole locations, ground surface elevations and drilled depths are summarized below.

Borehole No.	Location (MTM NAD 83)		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing (Latitude, °)	Easting (Longitude, °)		
MO-01A	4,823,580.0 (43.552090)	295,566.4 (-79.614279)	98.9	1.5
MO-01B	4,823,579.3 (43.552084)	295,565.7 (-79.614288)	98.9	1.2
MO-01C	4,823,581.1 (43.552100)	295,567.5 (-79.614265)	99.2	1.2
MO-02	4,823,572.9 (43.552027)	295,595.8 (-79.613914)	99.8	3.3
MO-03	4,823,599.6 (43.552266)	295,570.4 (-79.614229)	94.0	4.3*
MO-04	4,823,583.9 (43.552126)	295,593.2 (-79.613947)	99.7	6.6*
MO-05	4,823,585.1 (43.552136)	295,623.8 (-79.613568)	99.6	7.0*





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Borehole No.	Location (MTM NAD 83)		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing (Latitude, °)	Easting (Longitude, °)		
MO-06	4,823,610.0 (43.552360)	295,581.0 (-79.614098)	93.6	4.7*
MO-07	4,823,616.9 (43.552423)	295,622.5 (-79.613584)	99.4	6.3*
MO-08A	4,823,630.5 (43.552545)	295,614.5 (-79.613684)	98.9	2.2
MO-08B	4,823,632.0 (43.552559)	295,615.8 (-79.613668)	98.9	2.1
MO-09	4,823,634.3 (43.552580)	295,652.9 (-79.613208)	99.2	3.1
MO-10	4,823,597.7 (43.552249)	295,592.3 (-79.613959)	93.1	5.8*
MO-11	4,823,604.7 (43.552313)	295,610.1 (-79.613738)	92.9	5.7*
MO-12	4,823,592.2 (43.552201)	295,628.5 (-79.613511)	92.9	5.7*

\* includes bedrock core between 3.0 m and 4.7 m length

As noted in Section 3.1, the current investigation was supplemented with previous boreholes advanced by Thurber. While the Thurber report does not reference the coordinate system of the borehole locations, it is inferred that they are referenced to the MTM NAD 83 coordinate system based on the plotted position relative to that reference system. The northing and eastings of the boreholes, advanced during the previous investigation and this current investigation are shown in plan and tabulated on Drawing 1.

## 4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

### 4.1 Regional Geology

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

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

<sup>2</sup> 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.)



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 previous and current investigations and the results of the laboratory tests carried out on selected soil and bedrock core samples are presented on the Record of Borehole and Drillhole sheets provided in Appendices A and B, respectively. The results of the in situ field tests (i.e. SPT “N” values) as presented on the Record of Borehole sheets of the current investigation and in sub sections of Section 4.2 are uncorrected. The geotechnical laboratory testing plots for samples from the current investigation are contained in Appendix C.

The stratigraphic boundaries shown on the Record of Borehole sheets and on the stratigraphic profiles on Drawings 1 and 2 are inferred from non-continuous sampling, observations of drilling progress and the results of Standard Penetration Tests. These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. Furthermore, subsurface conditions will vary between and beyond the borehole locations; however, the factual data presented in the record of Borehole and Drillhole sheets governs any interpretation of the site conditions. It should be noted that the interpreted stratigraphy shown on Drawings 1 and 2 is a simplification of the subsurface conditions.

In general, the subsurface conditions in the area of the replacement overpass consist of a layer of asphalt and concrete in places, or topsoil, underlain by a deposit of granular fill associated with the construction of the existing highway and Mississauga Road. The fill is underlain in some locations by a deposit of residual soil comprised of clayey silt or sandy clayey silt. The fill deposits and/or the native deposits are underlain by shale bedrock.

A detailed description of the subsurface conditions encountered in the boreholes from the previous and current investigations are provided in the following sections.

### **4.2.1 Asphalt/Concrete**

Boreholes MO-02, MO-04, MO-05, MO-07 and MO-09 were advanced through the QEW highway surface and encountered a layer of asphalt varying in thickness from about 130 mm to 250 mm. Underlying the asphalt a layer of concrete was encountered in Boreholes MO-02, MO-05, MO-07 and MO-09, ranging in thickness from about 150 mm to 280 mm.

Boreholes MO-03, MO-06, MO-10 to MO-12, MR11-01 and MR11-02 were advanced through the Mississauga Road surface and encountered a layer of asphalt varying in thickness from about 100 mm to 250 mm.

### **4.2.2 Topsoil**

The MO-01 and MO-08 series of boreholes were advanced north of the existing QEW highway surface and encountered a layer of topsoil at ground surface varying in thickness from about 75 mm to 180 mm.

### **4.2.3 Non-Cohesive Fill**

A 0.5 m to 3.3 m thick layer of non-cohesive fill was encountered underlying the asphalt and concrete (where present) or topsoil or cohesive fill (where present) in all the boreholes advanced at the site. The fill is variable in composition and generally consists of silty sand to gravelly sand to sand and gravel. The surface of the fill deposit was encountered between about Elevations 99.4 m and 93.5 m and extends to depths of between about 0.6 m and 3.6 m below ground surface.



The SPT “N” values measured within the non-cohesive fill range from 5 blows to 63 blows per 0.3 m of penetration with one value of 100 blows for 0.13 m of penetration, indicating that the fill layer has a loose to very dense relative density.

Grain size distribution tests were carried out on two samples of the fill material sampled in the boreholes from the previous investigation and the results are presented on the borehole records included in Appendix A. Grain size distribution tests were carried out on thirteen selected samples of the non-cohesive fill recovered during the current investigation and the results are shown on Figures C1A and C1B in Appendix C. One Atterberg limits test was carried out on a sample of the non-cohesive fill from Borehole MO-08B and confirmed that the fill is non-plastic.

The water content measured on samples of the non-cohesive fill ranges between about 3 per cent and 24 per cent.

#### **4.2.4 Cohesive Fill**

A 0.2 m to 0.9 m thick layer of cohesive fill was encountered underlying the topsoil in Borehole MO-08B, underlying the non-cohesive fill in Borehole MO-09, and within the non-cohesive fill layer in Borehole MO-02. The fill is variable in composition and generally consists of clayey silt trace sand to clayey silt with sand and contains trace to some gravel and trace rootlets. The surface of the fill deposit was encountered between about Elevations 98.8 m and 97.9 m and extends to depths of between about 0.6 m and 2.2 m below ground surface.

The SPT “N” values measured within the cohesive fill are 9 blows and 25 blows per 0.3 m of penetration, suggesting a stiff to very stiff consistency.

A grain size distribution test was carried out on a selected sample of the cohesive fill recovered from Borehole MO-09 and the result is shown on Figure C2 in Appendix C. Atterberg limits tests were carried out on two samples of the cohesive fill and measured liquid limits of 31 per cent and 33 per cent, plastic limits of 16 per cent, and plasticity indices of 15 per cent and 17 per cent. These results, which are plotted on a plasticity chart on Figure C3 in Appendix C indicate that the samples tested can be classified as clayey silt of low plasticity.

The water content measured on samples of the cohesive fill ranges between about 14 per cent and 24 per cent.

#### **4.2.5 Clayey Silt**

In Boreholes MO-01A to MO-01C, MO-08A and MO-08B a 0.4 m to 0.9 m thick deposit of clayey silt trace to some sand, trace to some gravel, was encountered underlying the fill deposit at depths between 0.6 m and 1.4 m below ground surface (between Elevations 98.6 m and 97.5 m).

The SPT “N” values measured within the clayey silt deposit range between 20 blows and 77 blows per 0.3 m of penetration and 100 blows for 0.25 m of penetration, suggesting that the cohesive deposit has a very stiff to hard consistency.

A grain size distribution test was carried out on a selected sample of the clayey silt deposit from Borehole MO-08A and the result is shown on Figure C4 in Appendix C. Atterberg limits tests were carried out on three samples of this deposit and measured liquid limits between 32 per cent and 33 per cent, plastic limits between 19 per cent and 20 per cent, and plasticity indices between 12 per cent and 13 per cent. These results are shown on Figure C5 in Appendix C and indicate that the deposit consists of clayey silt of low plasticity.

The natural water content measured on samples of the cohesive deposit are between 11 per cent and 21 per cent.



#### 4.2.6 Residual Soil

In Boreholes MO-02, MO-04, MO-05, MO-07 and MO-09 a 0.2 m to 1.6 m thick deposit of residual soil was encountered underlying the fill deposits at depths between 1.8 m and 3.6 m below ground surface (between Elevations 97.8 m and 96.0 m).

The SPT “N” values measured within the residual soil deposit range between 36 blows and 66 blows per 0.3 m of penetration and up to 100 blows for 0.1 m of penetration, suggesting a hard consistency.

The deposit consists of clayey silt with sand to sandy, some gravel to gravelly and contains trace to some shale fragments, derived from weathering of the underlying shale bedrock. Grain size distribution tests were carried out on three selected samples of the clayey silt residual soil deposit and the results are shown on Figure C6 in Appendix C. Atterberg limits tests were carried out on three samples of this deposit and measured liquid limits between 31 per cent and 33 per cent, plastic limits between 20 per cent and 21 per cent, and plasticity indices between 11 per cent and 13 per cent. These results are shown on Figure C7 in Appendix C and indicate that the deposit consists of clayey silt of low plasticity.

The water content measured on samples of the residual soil deposit range between 4 per cent and 13 per cent.

#### 4.2.7 Bedrock

Bedrock was encountered and core samples were recovered in Boreholes MO-03 to MO-07, MO-10 to MO-12, MR11-01 and MR11-02, and the bedrock surface was inferred from augering and/or split-spoon sampling in Boreholes MO-01A to MO-01C, MO-02, MO-08A, MO-08B and MO-09. The depths to bedrock below ground surface, and the corresponding bedrock surface elevation are summarized below.

Borehole	Depth to Bedrock Surface (m)	Bedrock Surface Elevation (m)	Comments
MO-01A	1.2	97.7	Split Spoon Sample
MO-01B	1.1	97.8	Split Spoon Sample
MO-01C	1.1	98.1	Split Spoon Sample
MO-02	3.3	96.5	Auger Refusal
MO-03	1.0	93.0	Bedrock Cored
MO-04	3.5	96.2	Bedrock Cored
MO-05	3.8	95.9	Bedrock Cored
MO-06	1.6	92.0	Bedrock Cored
MO-07	3.0	96.4	Bedrock Cored
MO-08A	2.2	96.8	Split Spoon Sample
MO-08B	1.8	97.1	Split Spoon Sample
MO-09	3.0	96.2	Split Spoon Sample
MO-10	0.9	92.2	Bedrock Cored
MO-11	0.7	92.2	Bedrock Cored
MO-12	0.9	92.0	Bedrock Cored
MR11-01	1.1	93.3	Bedrock Cored
MR11-02	2.8	90.5	Bedrock Cored



In general, the bedrock surface as encountered or inferred in the area of the proposed bridge replacement slopes gently down towards the south, with the exception of the Mississauga Road alignment which was constructed in a bedrock cut.

Based on a review of the bedrock core samples from the current investigation and descriptions of the bedrock from the previous investigation, the bedrock consists of shale of the Georgian Bay Formation. In general, the bedrock samples are described as moderately to slightly weathered to fresh, thinly to medium bedded, fine grained, faintly porous, very weak to weak, grey, with medium strong limestone interbeds at varying intervals of depth, as presented in the borehole records from the previous investigation in Appendix A and the Record of Drillhole sheets from the current investigation in Appendix B, and shown on the photographs of the recovered core samples on Figures C8 to C15 in Appendix C. The degree of weathering of the bedrock samples (i.e. fresh to moderately weathered – W1 to W3), and the strength classification of the intact rock mass based on field identification (i.e. strong to very strong – R4 to R5) are described in accordance with the International Society for Rock Mechanics (ISRM)<sup>3</sup> standard classification system.

The Rock Quality Designation (RQD) measured on the core samples ranges from about 29 per cent to 100 per cent, and is generally greater than 50 per cent, indicating a rock mass of poor to excellent quality, and generally fair to excellent quality, as per Table 3.10 of CFEM (2006)<sup>4</sup>. The Total Core Recovery (TCR) and Solid Core Recovery (SCR) of samples recovered are between 67 per cent and 100 per cent and between 45 per cent and 100 per cent, respectively.

Unconfined compression uniaxial compressive strengths (UCS) obtained from the UC tests (ASTM D7012)<sup>5</sup> carried out on selected core samples of the shale bedrock and the results are summarised below and the details are presented on the Rock Laboratory Test Result reports from Geomechanica in Appendix C.

Borehole No.	Sample Depth (m)	UCS (MPa)	Bulk Density (g/cm <sup>3</sup> )	Young's Modulus (GPa)
MO-03	3.94 – 4.09	14.8	2.58	2.20
MO-04	6.21 – 6.37	6.4	2.56	0.24
MO-05*	6.08 – 6.16	39.2	2.60	6.53
MO-10	2.68 – 2.83	19.6	2.60	0.86
MO-11	3.66 – 3.79	18.3	2.59	0.97
MO-12	4.15 – 4.27	17.3	2.60	1.00

Note: \*Specimen included 60 mm thick limestone layer at bottom.

Twenty-four axial and eleven diametral point Load tests were carried out on 35 samples of the shale bedrock, and the results are summarized below:

<sup>3</sup> 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.

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

<sup>5</sup> ASTM D7012 – Standard Test Method for Compressive Strength and Elastic Moduli of Intact Rock Core Specimens



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Borehole No.	Sample Depth (m)	Sample Elevation (m)	Orientation	Axial $I_{s(50mm)}$ (MPa)
M0-3	3.35	90.65	Axial	1.84*
M0-3	3.59	90.41	Axial	3.4*
M0-3	3.62	90.38	Axial	1.73
M0-4	5.70	93	Diametral	0.42
M0-4	5.73	92.97	Axial	0.57
M0-4	6.43	93.27	Diametral	0.27
M0-4	6.47	93.23	Axial	1.85*
M0-5	5.68	94.02	Axial	1.38
M0-5	5.97	93.73	Axial	1.56
M0-5	5.87	93.83	Axial	1.62
M0-6	3.40	89.75	Axial	2.14*
M0-6	4.34	89.36	Axial	1.67
M0-6	4.36	89.34	Axial	1.62
M0-7	4.98	94.42	Axial	1.32
M0-7	5.52	93.88	Axial	1.42
M0-7	6.02	93.38	Axial	0.85
MO-10	1.94	91.16	Diametral	0.07
MO-10	2.06	91.04	Axial	0.45
MO-10	2.46	90.64	Axial	0.52
MO-10	2.46	90.64	Diametral	0.62
MO-10	2.59	90.51	Axial	0.36
MO-10	2.89	90.21	Diametral	0.57
MO-10	4.17	88.93	Diametral	0.22
MO-10	4.21	88.89	Axial	0.43
MO-11	1.37	91.53	Axial	6.52*
MO-11	2.25	90.65	Diametral	0.34
MO-11	2.31	90.59	Axial	1.08
MO-11	3.79	89.11	Diametral	0.33
MO-11	3.86	89.04	Axial	0.56
MO-12	2.33	90.57	Diametral	0.38
MO-12	2.33	90.57	Axial	0.46
MO-12	2.47	90.43	Diametral	0.56
MO-12	2.47	90.43	Axial	0.74
MO-12	3.99	88.91	Diametral	0.50
MO-12	3.99	88.91	Axial	0.70

\*test carried out on limestone interlayer



The estimated uniaxial compressive strength (UCS) values for each sample tested for point load strength are based on a relationship between  $Is_{50}$  and UCS which is given by a correlation factor (C) in accordance with ASTM D573108 (*Standard Test Method for Determination of the Point Load Strength Index of Rock and Application to Rock Strength Classification*), which may vary depending on the size of the core sample and the strength of the rock. For this site, the UCS values are based on an estimated average correlation factor (C) of 13.6 calculated from the average  $Is_{50}$ , and ignoring the tests carried out on limestone layers, compared to the average of the UCS test results.

Based on the laboratory UCS and point load tests, in accordance with Table 3.5 in CFEM (2006)<sup>4</sup>, the shale bedrock is generally classified as very weak (R1, 1 MPa < UCS < 5 MPa) to weak (R2, 5 MPa < UCS < 25 MPa) and the limestone interlayers are classified as medium strong (R3, 25 MPa < UCS < 50 MPa) to strong (R4, 50 MPa < UCS < 100 MPa).

#### 4.2.8 Groundwater Conditions

The overburden samples obtained from the borehole investigations were generally moist. The boreholes were observed to be dry upon completion of soil drilling and prior to rock coring and the water level on completion of rock coring varied between about 4.8 m and 5.7 m below ground surface (between Elevations 94.7 m and 93.8 m). The water levels recorded in the piezometers during the 2011 subsurface investigation, about one month after installation is presented below.

Borehole	Stratum Sealed Into	Water Level Depth (m)	Water Elevation (m)	Date
MR11-02	Sand Fill / Bedrock Interface	2.3	91.0	September 30, 2011

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

#### 4.2.9 Analytical Testing Results

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

Parameter	Borehole MO-10 Run 1 (1.65 m to 1.78 m)	Borehole MO-11 Run 2 (1.79 m to 1.89 m)
pH	8.33	8.22
Resistivity (ohm-cm)	1,500	1,800
Electrical Conductivity (umho/cm)	647	566
Chlorides (ug/g)	120	99
Soluble Sulphates (ug/g)	130	150





## 5.0 CLOSURE

This report was prepared by Mr. Matthew Kelly, P.Eng., a geotechnical engineer with Golder. Mr. Jorge Costa, P.Eng., MTO Foundations Designated Contact for Golder and Senior Consultant conducted a technical and quality control review of the report.

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# **PART B**

## **FOUNDATION DESIGN REPORT**

**QEW – MISSISSAUGA ROAD OVERPASS REPLACEMENT, SITE NO. 24-196**  
**QEW WIDENING FROM WEST OF MISSISSAUGA ROAD TO WEST OF**  
**HURONTARIO STREET, CITY OF MISSISSAUGA**  
**MINISTRY OF TRANSPORTATION, ONTARIO**  
**GWP 2002-13-00**



## **6.0 DISCUSSION AND ENGINEERING INVESTIGATION**

This section of the report provides detail foundation engineering recommendations for design of the proposed Mississauga Road overpass replacement structure (Site No. 24-196) as part of the widening of the Queen Elizabeth Way (QEW) from west of Mississauga Road to west of Hurontario Street in the City of Mississauga, in the Regional Municipality of Peel, Ontario. These recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the 2011 and 2017 subsurface investigations. The discussion and recommendations presented are intended to provide the designer with sufficient information to assess the feasible foundation alternatives and carry out the design of the overpass foundations.

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

### **6.1 General**

Based on the General Arrangement (GA) drawing dated January 15, 2018 provided by MH, the proposed 25 m long single-span replacement overpass structure will be 58.3 m wide along Mississauga Road, approximately 22.7 m wider than the existing structure to accommodate six lanes in each of the Toronto bound and Niagara bound directions. The existing Mississauga Road overpass is a single-span structure built circa 1965 and rehabilitated circa 1998. The existing structure is 35.6 m wide and its abutments are supported on shallow foundations founded on bedrock at about Elevation 91.6 m. At the time of overpass construction, Mississauga Road grade was lowered and as such the approach embankments are about 1 m high. The natural ground surface at the grade of the QEW is between about Elevations 99.8 m and 99.4 m, Mississauga Road is constructed in a cut and the roadway surface is about Elevation 95.5 m ( $\pm$ ) at its lowest joint under the overpass; and north and south of the existing overpass are cut slopes at a gradient of about 2 horizontal to 1 vertical.

The new abutments are proposed to be constructed on the same alignment as the existing abutments, but founded at a lower level. Based on the GA, the grade of QEW will remain at the existing elevation with the exception of a slight grade raise towards the south end of the structure. The grade of Mississauga Road grades will be lowered by about 1.6 m, to about Elevation 94 m. The QEW will be widened to the north, which will result in fill embankments having a thickness of about 4 m. A concrete wing wall is proposed at the toe of the fill embankment on the four corners of the overpass replacement structure.

### **6.2 Consequences and Site Understanding Classification**

The proposed QEW overpass structure at Mississauga Road will carry large volumes of traffic with the potential to impact alternative transportation corridors. In accordance with Section 6.5 of the 2014 Canadian Highway Bridge Design Code and its Commentary (CHBDC 2014), the proposed overpass structure and its foundation system is considered to be classified as having a “typical consequence level” associated with exceeding limits states design. In addition, given the level of foundation investigation completed to date at this location in comparison to the degree of site understanding in Section 6.5 of the CHBDC (2014), the level of confidence for design is considered to be a “typical degree of site and prediction model understanding.” Accordingly, the



appropriate corresponding ULS and SLS consequence factor,  $\psi$ , from Table 6.1 and geotechnical resistance factors,  $\phi_{gu}$  and  $\phi_{gs}$ , from Table 6.2 of the CHBDC (2014) have been used for design.

### 6.3 Foundation Options

Given the presence of bedrock at relatively shallow depths at this site, the overpass abutments may be founded on shallow foundations directly on/into bedrock. Considering that bedrock is present at shallow depth, supporting the abutments on driven H-piles is considered not practical from a foundations perspective, as relatively deep excavations / drilling into bedrock would be necessary to allow for the minimum pile lengths required for an integral abutment design. Alternatively, consideration could be given to supporting the abutments on drilled shafts (caissons). Drilled shafts socketed into bedrock are feasible and may be practicable from a construction staging perspective.

Based on the GA drawing and to maintain QEW traffic flow during construction, staged construction with temporary protection systems would be required along the north side of the abutment areas to allow for excavation and removal of the existing abutments and shallow footings and for construction of new abutment walls supported on new footings founded at about Elevations 91 m or 90 m (see Section 6.4.1). Elements of the protection system will need to be cored into or otherwise fixed within the bedrock to obtain sufficient lateral resistance and toe fixity for the system to ensure the integrity of the existing structure and fill behind the abutments. Therefore, it is recommended that the temporary protection system be designed to Performance Level 1b (per OPSS.PROV 539 Temporary Protection Systems) as amended by *SP105S09*, adjacent to the existing footings.

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

- **Shallow foundations –strip footings:** Shallow foundations comprised of strip footings founded on bedrock are feasible for support of the abutments, although this foundation type will preclude the use of integral abutments. To support the new overpass abutments on shallow foundations directly on/into undisturbed bedrock, excavations to a depth of approximately 10 m below the highway grade will be required. Such excavations will extend below the founding elevations of the existing footing and as such temporary protection systems will be required. Further, excavations are anticipated to extend below the groundwater level at this site, and as such some level of groundwater control will be required to maintain dry conditions at the base of the footings.
- **Deep foundations – drilled shafts (caissons):** Drilled shafts (caissons) are considered feasible for the support of the abutments. Similar to shallow foundations, this option would preclude integral abutment design. This option would be more expensive than shallow foundations, but would need smaller excavation areas and as such would not require a stiff temporary protection system. Given the size of the drilled shafts, the groundwater control efforts are anticipated to be less than that for shallow foundations. If adopted, temporary liners would be required to control the groundwater flow and any fill during the construction and to allow for proper inspection of the drilled shafts base and side walls.
- **Deep foundations – driven steel H-Piles socketed into Shale Bedrock:** Given the relatively shallow depth to bedrock, driven steel H-piles are considered not practical from a foundations perspective.

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



## 6.4 Strip Footings

### 6.4.1 Footing Elevations

As discussed in Section 6.1, the existing structure are founded on shallow foundations at about Elevation 91.6 m. Based on the GA drawing the proposed abutments would be supported by shallow foundations at Elevation 90.2 m. The new footings must be founded on undisturbed competent bedrock, which is present at lower levels than the founding elevation of the existing footings. Given that the excavations for construction of the new footings extend below the existing footings, temporary protection system with Level 1b performance will be required.

The proposed abutments are to be constructed on the existing abutment alignments, and as such the existing abutments and footings would have to be removed and the new footings have to be founded below the level of the existing footing to be placed on undisturbed bedrock. The highest founding elevations on undisturbed competent bedrock for each foundation element are provided below:

Structural Element	Reference Boreholes	Bedrock Surface Elevation (m)	Highest Founding Elevation on Bedrock (m)
West Abutment	MR11-01	93.3	91.0
	MO-03	93.0	
	MO-04	96.2*	
	MO-10	92.2	
	MO-05	95.9	
	MO-12	92.0	
East Abutment	MO-06	92.0	90.0
	MO-11	92.2	
	MO-07	96.4*	
	MR11-02	90.5	

\* The bedrock surface outside of the existing overpass structure footprint; represents the bedrock surface prior to grade cut for construction of the overpass.

If the excavation in bedrock is required to extend to below the highest founding elevations provided to remove all loose, shattered and/or fractured rock within the area of the footing, or where bedrock excavation results in the creation of steps or troughs in the bedrock, the founding stratum could be levelled or raised using mass concrete. In this case the mass concrete should be of the same composition and compressive strength as the concrete used for the footing construction.

To maintain traffic flow, the new overpass will be required to be constructed in stages (the GA drawing indicates three stages of construction). The shale bedrock contains horizontal bedding and will tend to break in slabs if removed with an excavator. Where bedrock excavation is required adjacent to a footing that is supporting the existing overpass structure, it must be carried out using saw-cutting or line drilling techniques to avoid the unintentional removal of bedrock from below the existing footing. A Non-Standard Special Provision (NSSP) to address this item is included in Appendix D, which should be included in the Contract Documents.

Geotechnical resistances for strip footings founded on bedrock at the elevations recommended above are provided in Section 6.4.2.



## **6.4.2 Geotechnical Resistances**

Strip footings (2 m wide as indicated on the GA) placed on the properly prepared bedrock excavation base may be designed based on a factored ultimate geotechnical resistance of 6,000 kPa. The factored serviceability geotechnical resistance for 25 mm of settlement will be greater than the factored ultimate geotechnical resistance; as such, the factored ultimate geotechnical resistance will govern for this foundation type.

The factored ultimate geotechnical resistance provided above is given for loads that will be applied perpendicular to the surface of the footings. Where the load is not applied perpendicular to the footing, inclination of the load should be taken into account in accordance with Section 6.10.4 of the CHBDC (2014).

The footing subgrade should be inspected by a Foundation Engineer, in accordance with OPSS 902 (*Excavating and Backfilling Structures*), *Special Provision SP109S12* and *NSSP FOUN0003*, to check that all existing fill(s), concrete and native soils have been removed. Furthermore, following excavations into the bedrock, it will be necessary to clean, scale and remove all loose, shattered and/or fractured rock within the area of the footing to ensure proper concrete bond to the bedrock.

The shale bedrock subgrade will be susceptible to disturbance from ponded water, precipitation from inclement weather and/or construction traffic. If the concrete for the footings cannot be poured immediately after excavation and inspection, it is recommended that a concrete working slab (100 mm thick of 20 MPa compressive strength concrete) be placed in the excavation within four hours to protect the integrity of the subgrade. Where bedrock excavation results in the creation of steps or troughs in the bedrock, the surface of the founding stratum could be levelled by the use of dental or mass concrete. In this case the dental or mass concrete should be of the same composition and compressive strength as the concrete used for the footing construction. An NSSP to address each of a concrete working slab and dental concrete is included in Appendix D, which should be included in the Contract Documents.

The UC test results of the shale bedrock core samples indicate that the shale is weak with medium strong limestone/siltstone/dolostone layers, and where excavations for the abutment foundations extend into this formation, appropriate construction equipment and procedures (such as hoe-ramming) will be required. It is recommended that an NSSP, as noted in Section 6.4.1 be included in the Contract Documents to warn the contractor of such obstructions. As further discussed in Section 6.9.5, vibration monitoring is not anticipated to be required during hoe-ramming activities, neither on the existing bridges nor on the nearest residences which are 115 m away.

## **6.4.3 Resistance to Lateral Loads**

Resistance to lateral forces/sliding resistance between the new concrete footings and the subgrade should be calculated in accordance with Section 6.10.5 of the CHBDC (2014). For cast-in-place concrete footings constructed directly on bedrock, or on a concrete working slab that is cast on top of the shale bedrock, the sliding resistance may be calculated based on the coefficient of friction,  $\tan \delta$ , of 0.5, as interpreted from NAVFAC (1986).

If necessary, the sliding resistance between the concrete footing and/or working slab and the bedrock at the abutments can be supplemented by dowelling into the bedrock. The horizontal resistance of the dowels is dependent on the strength of the bedrock, grout and steel. The dowels should have a minimum bond length of 1 m into Fair to Good Quality rock (i.e. exhibiting RQD greater than 50 per cent), and the structural strength of the dowels and compressive strength of the grout should not be exceeded. For uplift of the dowels, a factored value of 175 kPa may be assumed for the grout-to-rock bond stress for ULS design. The actual bond stress along the rock-grout interface may vary from the design value given and it should therefore be verified in the field by pull-



out testing. If dowelling into bedrock is adopted at this site, an NSSP should be included in the Contract Documents to specify the installation, materials and testing of the dowels; an example is provided in Appendix D. These values assume that construction is carried out in dry conditions.

#### 6.4.4 Frost Protection

The abutment footings founded directly on/into the bedrock do not require soil cover for frost protection, provided that the footings are founded into the bedrock at or below the levels recommended in Section 6.4.1.

### 6.5 Caissons

#### 6.5.1 Founding Elevations

Caissons foundations could also be considered for support of the bridge abutments. Caissons should be socketed a minimum of two times the caisson diameters, or a minimum of 2 m whichever is greater, into Fair to Good Quality bedrock (an RQD greater than 50 per cent). The following details the top of the estimated rock socket elevation:

Foundation Element	Location Along Element	Borehole	Bedrock Surface Elevation (m)	Estimated Top of Rock Socket Elevation (m)	Highest Base of Socket Elevation (m)
West Abutment	North	MR11-01 MO-03	92.8 92.8	91.7	89.5
	Centre	MO-04 MO-10	96.2* 92.0		
	South	MO-05 MO-12	95.7 91.7		
East Abutment	North	MO-06	91.9	90.2	88.0 to 85.0
	Centre	MO-11 MO-07	91.7 96.1*		
	South	MR11-02	90.2		

\* The bedrock surface outside of the existing overpass structure footprint; represents the bedrock surface prior to grade cut for construction of the overpass.

The shale bedrock is weak with unconfined compressive strengths generally in the range of 15 MPa to 20 MPa, but it contains medium strong limestone/siltstone/dolostone layers, and as such the sockets would likely have to be advanced into the rock by churn drilling or rock coring. If caissons are adopted, it is recommended that an NSSP be included in the Contract Documents to describe to the Contractor the strength and characteristics of the bedrock; an NSSP is included in Appendix D for this purpose. Consideration should be given to using temporary liners during construction to control groundwater flow and any fill in the caisson holes to allow for inspection of the base and sides of the caissons.

#### 6.5.2 Geotechnical Axial Resistances

If the caisson foundation option is adopted, additional deeper boreholes would need to be completed to obtain bedrock quality / strength information at or below the base of the proposed caisson(s), prior to completion of detail design. For drilled shafts with the diameters indicated below, designed for shaft friction and end bearing resistance and socketed a minimum length equal to 2 pile diameters, but not less than 2 m long into the good to excellent quality bedrock (i.e. exhibiting an RQD greater than 75 per cent), with the socket extending below the top of the





rock socket relative to the elevations provided in Section 6.5.1 or lower, the following provides the factored ultimate geotechnical resistance for various caisson diameters:

**For Socket length equal to 2 caisson diameters:**

Foundation Element	Caisson Diameter (m)	Factored Ultimate Geotechnical Resistance (kN)	Factored Serviceability Geotechnical Resistance (kN)
West and East Abutments	1.2	7,500	Not applicable*
	1.5	10,800	

\* 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 governs.

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

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

### **6.5.3 Resistance to Lateral Loads**

Drilled shafts socketed into bedrock will provide resistance to lateral loading. The resistance to lateral loading in front of a single vertical drilled shaft may be estimated using subgrade reaction theory and the rock mass spring constant. For drilled shafts socketed into the bedrock, it is anticipated that the rock will remain in the elastic range under the lateral loading. Based on this assumption, closed form solutions have been used for the estimation of the bedrock lateral spring constant.

Based on the estimated assessed lateral rock mass elastic modulus of the shale bedrock,  $E_h = 500$  MPa, and a Poisson's ratio of 0.2, the lateral rock mass spring constant,  $k_h$ , of 825 MN/m/m may be used.

For rock sockets and when the centre to centre caisson spacing is greater than 3 times caisson diameter (D), subgrade reaction reduction factor is negligible.

### **6.5.4 Frost Protection**

Pile caps on the caisson foundations, if constructed below ground surface must be constructed not less than 1.2 m below the surrounding grade for protection from frost penetration, as interpreted from OPSP 3090.101 (Frost Penetration Depths for Southern Ontario).



## 6.6 Lateral Earth Pressure for Design of Abutments and Wing Walls

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

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

- Select, free draining granular fill meeting the specifications of OPSS.PROV 1010 (Aggregates) Granular 'A' or Granular B Type II, should be used as backfill behind the walls. Longitudinal drains or weep holes should be installed to provide positive drainage of the granular backfill. Compaction (including type of equipment, target densities, etc.) should be carried out in accordance with OPSS.PROV 501 (Compacting). Other aspects of the granular backfill requirements with respect to subdrains and frost taper should be in accordance with OPSD 3101.150 (Walls, Abutment, Backfill, Minimum Granular Requirement), OPSD 3121.150 (Walls, Retaining, Backfill, Minimum Granular Requirement), and OPSD 3190.100 (Walls, Retaining and Abutment, Wall Drain).
- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the walls, in accordance with CHBDC (2014) Section 6.12.3 and Figure 6.6. Care must be taken during the compaction operation not to overstress the wall. Heavy construction equipment should be maintained at a distance of at least 1 m away from the walls while the backfill soils are being placed. Hand operated compaction equipment should be used to compact the backfill soils immediately behind the walls as per OPSS.PROV 501. Other surcharge loadings should be accounted for in the design, as required.
- For restrained walls, granular fill should be placed in a zone with the width equal to at least 1.2 m (equivalent to the depth of frost penetration as interpreted from OPSD 2090.101, Frost Penetration Depths for Southern Ontario) behind the back of the wall on Figure C6.20(a) of the Commentary to the CHBDC (2014). For unrestrained walls, fill should be placed within the wedge-shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing or pile cap on Figure C6.20(b) of the Commentary to the CHBDC (2014).

### 6.6.1 Lateral Earth Pressures for Design

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

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

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



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

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

- If the wall support and superstructure allow lateral yielding, active earth pressures may be used in the geotechnical design of the structure. The movement to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure, may be taken as:
  - Rotation of approximately 0.002 about the base of a vertical wall (where the rotation is calculated as the horizontal displacement divided by the height of the wall);
  - Horizontal translation of 0.001 times the height of the wall; or,
  - A combination of both.
- If the wall does not allow lateral yielding (i.e., restrained structure where the rotational or horizontal movement is not sufficient to mobilize an active earth pressure condition), at-rest earth pressures (plus any compaction surcharge) should be assumed for geotechnical design.

## 6.7 Approach Embankment Design and Construction

It is our understanding that the replacement structure will be widened by about 23 m and new wing walls and retaining walls will be constructed at the corners of the new structure. Mississauga Road was constructed in a cut and the widening at this site will require placement of embankment fill on the north side of the existing overpass structure on the order of 4 m thick on top of the existing slope and up to 6.5 m thick at a limited location at the toe of the existing side slopes.

At the north side of the west approach, Boreholes MO-01A to MO-01C were advanced within the northerly widening footprint and encountered a layer of loose to compact silty sand to sand fill underlain by a 0.6 m thick layer of very stiff to hard clayey silt extending to shale bedrock at a depth of 1.2 m below ground surface (Elevations 98 m to 97.5 m).

At the north side of the east approach, Boreholes MO-08A and MO-08B were advanced closest within the widening footprint and encountered a layer about 0.8 m and 1.3 m thick of compact/stiff fill underlain by a 1.0 m and 0.4 m thick deposit of very stiff to hard clayey silt, respectively.

### 6.7.1 Subgrade Preparation and Embankment Construction

Prior to construction of the new widening sections of the approach embankments it is recommended that any topsoil and loosened/softened fill be removed.

Fill for construction of the new embankment widenings should consist of Granular 'B' Type I or Type II meeting the specifications of OPSS.PROV 1010 (Aggregates). The embankment fill should be placed and compacted in accordance with OPSD 208.010 (Benching of Earth Slopes), OPSS.PROV 501 (Compacting) and



OPSS.PROV 206 (Grading). If adopted, embankment side slopes should be constructed no steeper than 2H:1V in granular fill.

All granular fill should be placed in lifts with loose thickness not exceeding 300 mm and compacted to at least 95 per cent of the Standard Proctor Maximum Dry Density of the material. Inspection and field density testing should be carried out by qualified personnel during fill placement operations to ensure that appropriate materials are used and that adequate levels of compaction have been achieved.

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

## 6.7.2 Concrete Wing Walls

Based on the GA drawing, it is understood that concrete wing walls are proposed for all corners of the overpass structure, and are considered suitable from a foundations perspective. Consideration could be given to the use of a retained soil system (RSS) wall at the corners of the overpass structure; however, this would require excavation into the existing cut slope, which may require temporary protection systems in order to support the cut slope. For this reason, RSS walls were not considered to be practical and are not discussed further in this report.

The highest founding levels of the wing walls footings, the corresponding factored ultimate geotechnical resistance and the factored serviceability geotechnical resistance are provided below.

Wall Location	Reference Boreholes	Highest Founding Elevation (m)	Founding Stratum	Factored Ultimate Geotechnical Resistance (kPa)	Factored Serviceability Geotechnical Resistance (25 mm settlement) (kPa) <sup>1</sup>
Northwest Corner	MO-01A to C	98.3	Very Stiff to Hard Clayey Silt	250 <sup>2</sup>	>250 <sup>2</sup>
Southwest Corner	MO-05	96.0	Residual Soil	500	>500
Northeast Corner	MO-08A and MO-08B	97.5	Very Stiff to Hard Clayey Silt	250 <sup>2</sup>	>250 <sup>2</sup>
Southeast Corner	MR11-02	90.5	Shale Bedrock	600	>600

Notes:

- 1- The factored serviceability geotechnical resistance is greater than the factored ultimate geotechnical resistance and as such the factored ultimate geotechnical resistance governs.
- 2- A factored ultimate geotechnical resistance and factored serviceability geotechnical resistance of 500 kPa can be used for footings founded on residual soil/shale bedrock at/below approximate Elevation of 96.5 m, if required.

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



The footing subgrade should be inspected by a Foundation Engineer, in accordance with OPSS 902 (*Excavating and Backfilling Structures*), *Special Provision SP109S12* and *NSSP FOUN0003*, to check that all existing fill(s), deleterious soils and construction debris have been removed.

Resistance to lateral forces/sliding resistance between the base of the footings for the concrete wing walls and the subgrade should be calculated in accordance with Section 6.10.5 of the CHBDC (2014). The coefficient of friction,  $\tan \delta$ , between the properly prepared subgrade on native cohesive soils / shale bedrock as noted above, and the concrete wing and wall footings may be taken as 0.4, as interpreted from NAVFAC (1986).

### 6.7.3 Global Stability

Limit equilibrium slope stability analyses for the embankment side slopes was carried out using the commercially available program Slide (version 7.0), developed by Rocscience Inc., employing the Morgenstern Price method of analysis. For all analyses, the Factors of Safety (FoS) of numerous potential failure surfaces were computed for the critical embankment cross section in order to establish the minimum FoS. Based on the results of the analysis for deep seated global failure surfaces, the FoS for the anticipated widened embankment side slopes, for the short-term (undrained) and long-term (drained) cases are greater than 1.3 and 1.5, respectively; an example of the stability analysis results for the long-term embankment slope are shown on Figure 1.

### 6.7.4 Settlement

Settlement of the subgrade soils beneath the widened west and east approach embankment areas can be expected as a result of the loading from the new fills on the fill material and underlying relatively thin deposits of non-cohesive gravelly sand to sand soils. Settlement of new granular fill that is properly placed and compacted for construction of the widened embankments would occur during construction.

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

The simplified stratigraphy, together with the associated strengths and unit weights employed for the different foundation soil types at the west and east abutments, as encountered in Boreholes MO-01A to C and MO-08A and B, respectively, are summarized below.

Borehole/Approach	Soil Type	Approximate Thickness (m)	Bulk Unit Weight (kN/m <sup>3</sup> )	Elastic Modulus (MPa)
Boreholes MO-01A to C West Approach	Loose to Compact silty sand to sand (fill)	0.5	20	10
	Very Stiff to Hard Clayey Silt	0.6	19	50
Boreholes MO-08A and B East Approach	Compact Silty Sand/Stiff Clayey Silt (fill)	1.3	20	15
	Very Stiff to Hard Clayey Silt	0.9	19	50

### 6.7.5 Settlement Performance Requirements

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



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

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

These performance criteria form part of the overall design performance for the embankment in the vicinity of the approach embankments.

### 6.7.6 Results of Analysis

Based on the analysis using the parameters prevented in Section 6.7.4 the anticipated settlement of the approach embankment widening fill is expected to be less than 25 mm, which meets the requirements of the above noted embankment settlement criteria for design.

## 6.8 Corrosion Assessment and Protection

The results of an analytical test on two samples of the shale bedrock are presented in Section 4.2.9 and in Appendix C. The analytical test results were compared to CSA A23.1 Table 3 (*"Additional requirements for concrete subjected to sulphate attack"*) for potential sulphate attack on concrete. The sulphate concentrations measured in the tested samples are below the exposure class of S-3 (Moderate). Therefore, based on the two shale bedrock samples tested, when the designer is selecting the exposure class for the structure, the effects of sulphates may not need to be considered.

The analytical test results of the bedrock samples were also compared to Table 2 of the U.S. Criteria for Assessing Ground Corrosion Potential (as derived from Federal Highways Administration (FHWA) 2003) for the potential attack on buried steel. The sulphate and chloride concentrations and the resistivity measured in the bedrock samples "Strong corrosion potential". Based on the results of the samples tested, and given that the structure is located adjacent to the roadway and will be exposed to de-icing salt, consideration should be given by the designer to designing for a "C" type exposure class as defined by CSA A23.1 Table 1.

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

## 6.9 Construction Considerations

### 6.9.1 Overburden Excavation and Control of Groundwater and Surface Water

For the option of spread/strip foundations founded directly on competent bedrock at the maximum founding elevations provided in Section 6.4.1, the excavations for foundations will extend to a depth of about 10 m below the QEW grade and 3 m below the Mississauga Road grade at the west and east abutments. The excavations will extend through the existing fill materials, the very stiff to hard clayey silt deposit and into bedrock

Open-cut excavations must be carried out in accordance with the guidelines outlined in the latest edition of Occupational Health and Safety Act (OHSA) and Regulation for Construction Activities (O.Reg. 213/91). The





existing fill is classified as Type 3 soil, while the native deposits are classified as Type 2 soils, according to the OHSA. Temporary excavations (i.e. those that are open for a relatively short time period) should be made with side slopes no steeper than 1H:1V, in Type 3 soil, and may be cut vertically for the lower 1.2 m depth of excavation in Type 2 soil. Excavated material must be stockpiled at a distance away from the excavation equal to or greater than the height of the open cut excavation.

It is expected that for construction staging, temporary protection systems will be required along the north and south sides of the excavations for the abutments. Recommendations for temporary protection systems are provided in Section 6.9.3.

The groundwater level at the site was measured at about Elevation 91 m, in the piezometer installed in borehole MR11-02 and water level on completion of rock coring was between Elevations 94.7 m and 93.8 m and will likely be higher during wetter periods of the year. For construction considerations, the excavations at the west and east abutments should be assumed to extend to about the groundwater level. It is expected that groundwater seepage inflow into the excavation can be handled by pumping from well filtered sumps located outside the foundation footprint.

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

### **6.9.2 Bedrock Excavation**

The new spread/strip footings at abutment locations will be founded at or below Elevation 90 m and up to approximately 2.3 m and 2.2 m of bedrock excavations will be required at the west and east abutments, respectively.

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

If the excavation in the bedrock is required to extend below the recommended elevations provided in Section 6.4.1 to remove all loose, shattered and/or fractured rock within the area of the footing, the over-excavated portions can be replaced with dental concrete, having the same composition and compressive strength as the concrete used for the strip footings. An NSSP for dental concrete is provided in Appendix D for inclusion in the Contract Documents. In addition, if the concrete footing is not constructed within four hours after the bedrock excavation to the footing founding level, the exposed subgrade should be protected with a concrete working slab. An NSSP for a dental concrete is provided in Appendix D for inclusion in the Contract Documents.

The shale bedrock at the site is weak (corresponding to unconfined compressive strengths generally in the range of 15 MPa to 20 MPa), but contains medium strong limestone/dolostone/siltstone interbeds. It is recommended that an NSSP be included in the Contract Documents to warn the Contractor of the bedrock characteristics, that excavation into the bedrock will require appropriate equipment and construction procedures, and that the bedrock excavation shall not disturb the existing bridge footings during construction staging. An NSSP is provided in Appendix D for inclusion in the Contract Documents.





### **6.9.3 Temporary Protection Systems**

Temporary protection systems will be required along the north side and median of the QEW lanes to facilitate the staged removal of the existing bridges foundations and the construction of the new abutment foundations and overpass structure.

The temporary protection systems should be designed and constructed in accordance with OPSS.PROV 539 (*Temporary Protection Systems*) as amended by SP105S09. The lateral movement of the temporary shoring system should meet Performance Level 2 as specified in OPSS.PROV 539, as amended by SP105S09, provided that any existing adjacent structures or utilities can tolerate this magnitude of deformation. If the excavated material is to be stockpiled adjacent to the temporary protection system, the system must be designed for this additional loading.

To handle removal of existing abutment footings, sub-excavation of the fill material and construction of new footings, the protection systems are required for an estimated excavation depth of up to approximately 10 m relative to the QEW grade at the site. A soldier pile and timber lagging system would be considered suitable for the temporary excavation support at the abutments, based on the subsurface soil and groundwater conditions. For a soldier pile and lagging system, it would be necessary to control seepage or include measures to mitigate loss of soil particles through the lagging boards.

The soldier piles would have to be driven or socketed to sufficient depth to provide the necessary passive resistance for the retained soil height, including any surcharge loads behind the protection system within at least a 1H:1V zone relative to the base of the excavation. Additional lateral support to the soldier piles may need to be provided in the form of rakers or temporary anchors.

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

### **6.9.4 Bedrock Subgrade Inspection and Protection**

Immediately following completion of excavation of the bedrock to the founding level, the footing subgrade should be inspected by a Foundation Engineer in accordance with OPSS 902 (*Excavating and Backfilling Structures*) as amended by SP109S12 and NSSP FOUN0003, to check that all existing fill materials, concrete and fractured, softened or loosened portions of the shale bedrock are removed prior to construction of the footings for the abutments.

The shale bedrock that will be exposed at the foundation subgrade level will be susceptible to weathering and/or disturbance from water and construction traffic. To limit this degradation, it is recommended that a minimum 100 mm thick concrete working slab of 20 MPa concrete be placed on the subgrade within four hours after preparation, inspection and approval of the footing subgrade. This requirement can be addressed with a note on the General Arrangement drawing and/or with an NSSP. An NSSP is provided in Appendix D for inclusion in the Contract Documents.

### **6.9.5 Vibration Monitoring During Construction**

It is anticipated that line drilling and/or hoe-ramming will be required to excavate into the bedrock for the proposed strip footings at the proposed abutment locations.

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



at this site, but it would be prudent for the contractor to monitor vibrations at the existing structure during the bedrock excavation operations. A vibration monitoring plan should be included in the Contractor's work plan for the bedrock excavation consistent with the NSSP presented in Appendix D.

Residential homes are located about 100 m east and 125 m west of the existing structure. Although a lower PPV threshold of 50 mm/s is generally considered applicable for vibration impacts on buildings, the zone of influence is anticipated to be less than 300 m and potentially even less than 100 m based on the encountered subsurface conditions. Therefore, vibration monitoring is not expected to be required at the existing structures near the bridge site. However, it would be prudent to carry out such monitoring during critical stages of the construction, such as during bedrock excavation operations. An NSSP describing the requirements for vibration monitoring is presented in Appendix D.

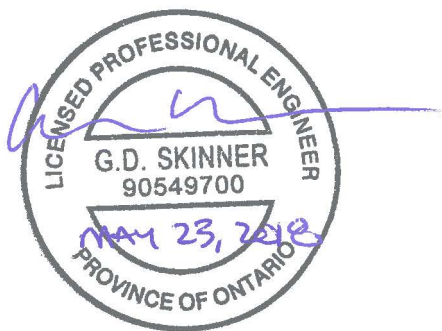


## 7.0 CLOSURE

This report was prepared by Mr. Al Varshoi, P.Eng., a Geotechnical Engineer with Golder and the technical aspects were reviewed by Dr. Graeme Skinner, P.Eng., a Senior Geotechnical Engineer and Principal of Golder. Mr. Jorge Costa, P.Eng., a Senior Consultant with Golder and MTO Foundations Designated Contact conducted an independent quality control review of this report.

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- Canadian Geotechnical Society. 2006. Canadian Foundation Engineering Manual (CFEM), 4<sup>th</sup> Edition. The Canadian Geotechnical Society, BiTech Publisher Ltd., British Columbia.
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### **ASTM International:**

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

### **Commercial Software:**

- Slide (Version 6) by Rocscience Inc.

### **Ontario Provisional Standard Drawing:**

- |               |   |
|---------------|---|
| OPSD 202.010  | Slope Flattening using Surplus Excavated Material on Earth or Rock Embankment |
| OPSD 208.010  | Benching of Earth Slopes  |
| OPSD 3000.100 | Foundation, Piles, Steel H-Pile Driving Shoe                                  |



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## FOUNDATION REPORT

### QEW-MISSISSAUGA ROAD OVERPASS REPLACEMENT, GWP 2002-13-00

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OPSD 3000.100	Foundation, Piles, Tube Pile Driving Shoe
OPSD 3090.101	Foundation Frost Penetration Depths for Southern Ontario
OPSD 3101.150	Walls, Abutment, Backfill Minimum Granular Requirement
OPSD 3121.150	Walls, Retaining, Backfill Minimum Granular Requirement
OPSD 3190.100	Walls, Retaining and Abutment, Wall Drain

#### Ontario Provincial Standard Specification:

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

#### Ontario Water Resources Act:

Ontario Regulation 903      Wells (as amended)

#### Ontario Occupational Health and Safety Act:

Ontario Regulation 213/91      Construction Projects (as amended)

#### Ministry of Transportation, Ontario

Structural Manual, Provincial Highways Management Division, Highway Standards Branch, Bridge Office, August 2014.

MTO Foundations Guideline, Embankment Settlement Criteria for Design, July 2010.

#### Special Provision:

Special Provision No. 105S09	Amendment to OPSS 539
Special Provision No. 109S12	Amendment to OPSS 902
Special Provision No. P109F57	Amendment to OPSS 903



# TABLE





## FOUNDATION REPORT QEW-MISSISSAUGA ROAD OVERPASS REPLACEMENT, GWP 2002-13-00

**TABLE 1 - COMPARISON OF FOUNDATION ALTERNATIVES, MISSISSAUGA OVERPASS, SITE No. 24-196, G.W.P. 2002-13-00**

Foundation Option	Feasibility	Advantages	Disadvantages	Constructability	Estimated Costs
Spread/strip footings founded on competent shale bedrock	<ul style="list-style-type: none"> <li>Feasible for support of the abutments; however, requires temporary roadway protection for staged construction of spread footings.</li> </ul>	<ul style="list-style-type: none"> <li>Existing abutments are supported on shallow foundations and have performed well.</li> <li>Lower vibration impacts on existing structures than for drilled shafts (caissons) or steel H-pile installation in pre-drilled holes.</li> <li>Only minor groundwater seepage anticipated at the abutments, so pumping from filtered sumps expected to provide adequate groundwater control.</li> <li>Appropriate bedrock quality/strength at shallow depth for support of shallow foundations.</li> </ul>	<ul style="list-style-type: none"> <li>At the abutments temporary roadway protection systems required along median edges of QEW EBL and WBL.</li> <li>Due to the required excavation depths to found on competent bedrock, excavations will extend deeper than the founding level of the existing footings and therefore temporary roadway protections systems would be required to have a high degree of stiffness.</li> <li>Lower bearing geotechnical resistances compared to deep foundation options.</li> <li>Precludes use of integral abutments; potentially greater maintenance required at abutments.</li> </ul>	<ul style="list-style-type: none"> <li>Conventional excavation and construction techniques.</li> </ul>	<ul style="list-style-type: none"> <li>Estimated cost is approximately \$600/m<sup>3</sup> for construction of shallow foundations, excluding deeper excavation and protection system at west and east abutments.</li> </ul>
Caissons socketed into shale bedrock	<ul style="list-style-type: none"> <li>Feasible for support of abutments.</li> <li>If adopted, additional deeper boreholes would need to be completed to obtain bedrock quality / strength information at or below the base of the caisson(s).</li> </ul>	<ul style="list-style-type: none"> <li>Higher bearing resistances than for shallow foundations or steel H-piles, requiring fewer pile elements.</li> </ul>	<ul style="list-style-type: none"> <li>Temporary liners would be required during construction to control potential ground losses in existing embankment fill.</li> <li>Shale bedrock is weak with medium strong limestone layers, so more expensive coring required to form bedrock socket through these layers.</li> <li>Precludes use of integral abutments.</li> <li>The rock socket is required to be cleaned using airlift methods and if the design relies on resistance from the base, inspection with a video camera would be required.</li> <li>Concrete would have to be placed by tremie methods below the water level.</li> </ul>	<ul style="list-style-type: none"> <li>Conventional construction methods for caisson foundations; temporary liners required for ground and groundwater control.</li> </ul>	<ul style="list-style-type: none"> <li>Estimated cost is approximately \$1000/m length for caisson installation and \$600/m<sup>3</sup> for caisson cap construction; this cost expected to be higher to account for pre-drilling/coring through harder limestone layers and for temporary liners.</li> </ul>
Steel H-piles founded on competent shale bedrock; Steel tube (pipe) piles on competent shale bedrock	<ul style="list-style-type: none"> <li>Not practical due to shallow depth to bedrock.</li> </ul>	<ul style="list-style-type: none"> <li>Abutment pile caps could be maintained higher than footings founded on shale bedrock at the abutments, reducing excavation depth and associated temporary roadway protection system requirements.</li> </ul>	<ul style="list-style-type: none"> <li>Piles would need to be installed into pre-drilled holes to achieve the minimum 5 m pile length for integral abutment design</li> <li>Temporary protection systems likely required along median edge of QEW EBL and WBL to facilitate excavation to pile cap level.</li> </ul>	<ul style="list-style-type: none"> <li>Conventional construction methods for steel H-pile.</li> <li>Socketing of steel H-piles or tube piles required into bedrock in pre-drilled holes.</li> </ul>	<ul style="list-style-type: none"> <li>Estimated cost is approximately \$250/m length for pile installation and \$600/m<sup>3</sup> for pile cap construction; the cost may be</li> </ul>



## FOUNDATION REPORT QEW-MISSISSAUGA ROAD OVERPASS REPLACEMENT, GWP 2002-13-00

Foundation Option	Feasibility	Advantages	Disadvantages	Constructability	Estimated Costs
		<ul style="list-style-type: none"><li>Minor groundwater seepage anticipated in pile cap excavations from possible perched water table, so pumping from filtered sumps will provide adequate groundwater control.</li><li>Allows for integral abutment construction, but to achieve minimum 5 m pile length, piles must be installed in drilled sockets in bedrock to accommodate the 3 m CSPs/sand fill element.</li><li>Higher geotechnical resistance than those for shallow foundations.</li></ul>	<ul style="list-style-type: none"><li>At this site, socketing of the piles up to 5 m into competent bedrock is required to achieve minimum pile length and penetrate through the highly fractured bedrock near surface zone.</li><li>The base of the pre-drilled hole for the steel H-pile must be cleaned of all cuttings and loosened material; otherwise the piles must be driven from the base of the pre-drilled holes to ensure they are seated within the bedrock.</li><li>Large diameter pre-drilled holes required to accommodate a portion of the CSP for integral abutment construction.</li></ul>	<ul style="list-style-type: none"><li>Temporary liners required for ground and groundwater control.</li><li>Structural design specifies that there must be a 1 m long section of concrete within the annular space at the base of the pre-drilled rock socket; this will require placement of concrete using tremie methods.</li><li>In addition, the annulus around the steel H-pile above the bedrock surface and below the CSP is to be backfilled with Granular 'B' Type II, which must be installed through the temporary casing in lifts.</li></ul>	<ul style="list-style-type: none"><li>higher to account for use of temporary liners.</li><li>Potentially less costly maintenance over life of the structure than semi-integral abutment structures.</li></ul>

Prepared By: ARV

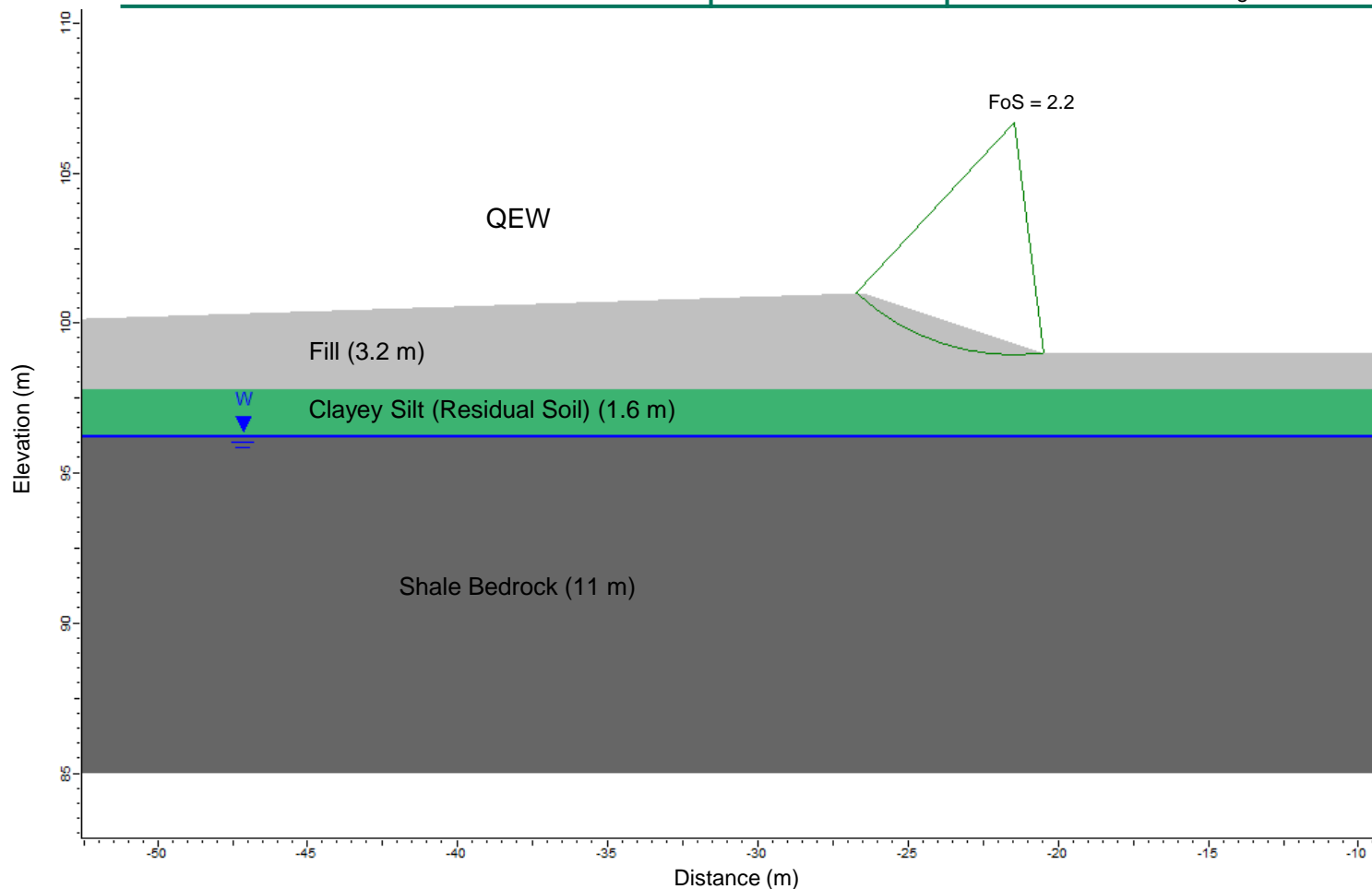
Reviewed By: JMAC



# QEW-Mississauga Road Overpass Slope Stability – QEW Widened Embankment Side Slope

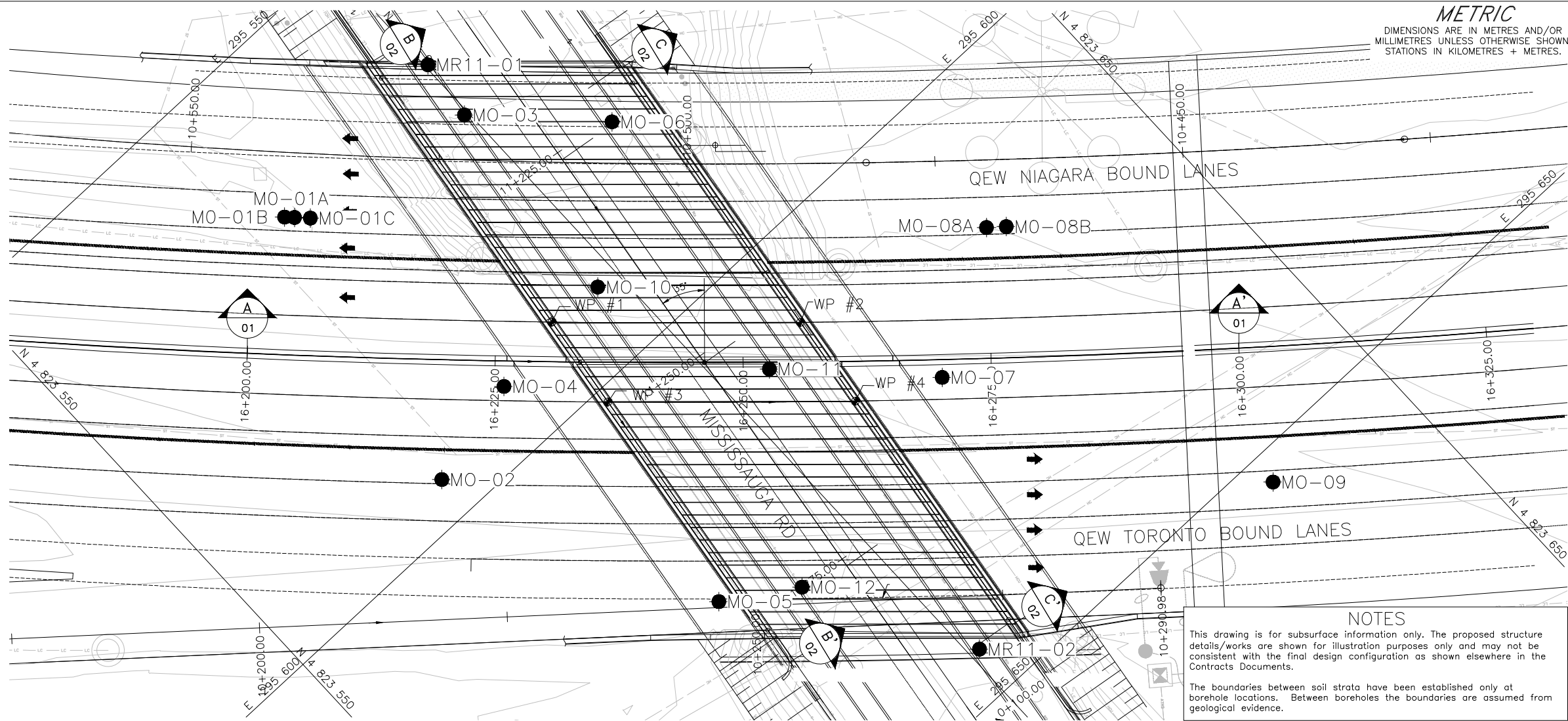
Figure 1

Material Name	Unit Weight (kN/m <sup>3</sup> )	Cohesion (kPa)	Friction Angle (degrees)
Fill	20	1	30
Clayey Silt (Residual Soil)	21	150	-
Shale	24	Infinite Strength	





# DRAWINGS



PLAN

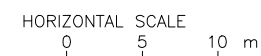
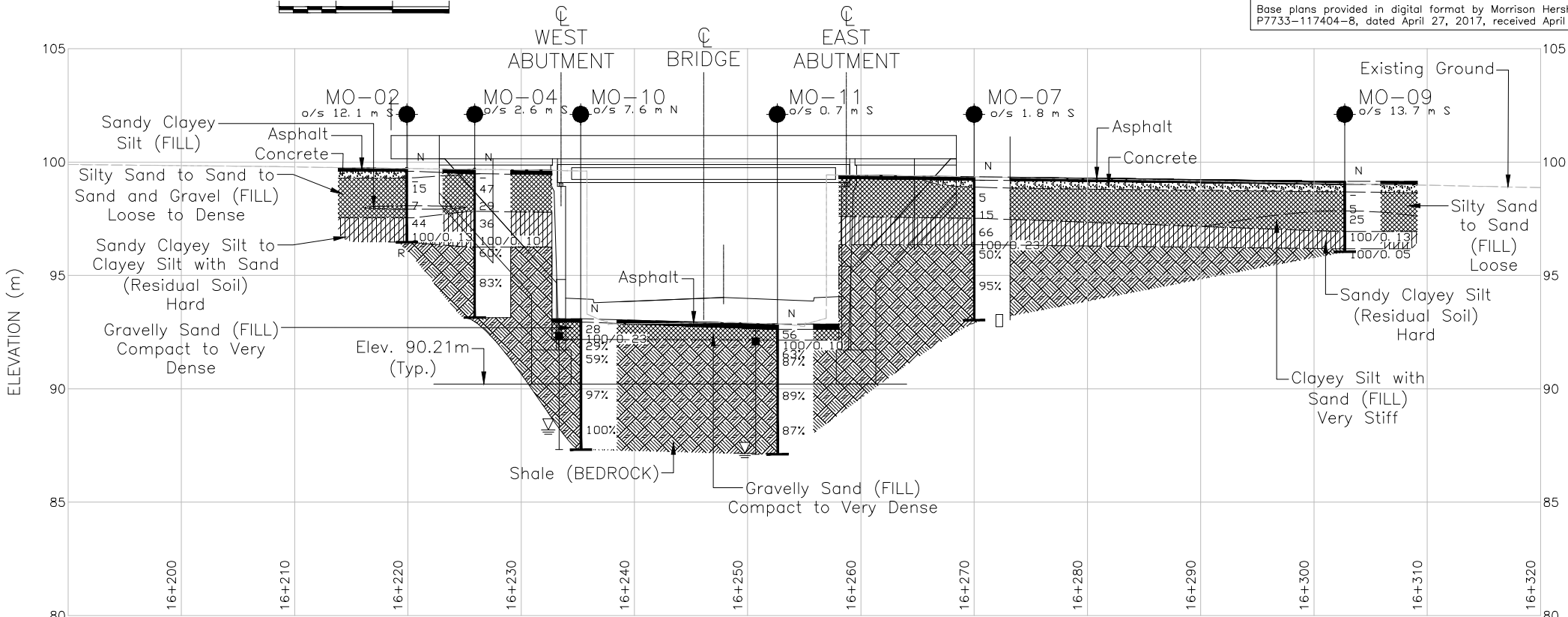
**NOTES**

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.

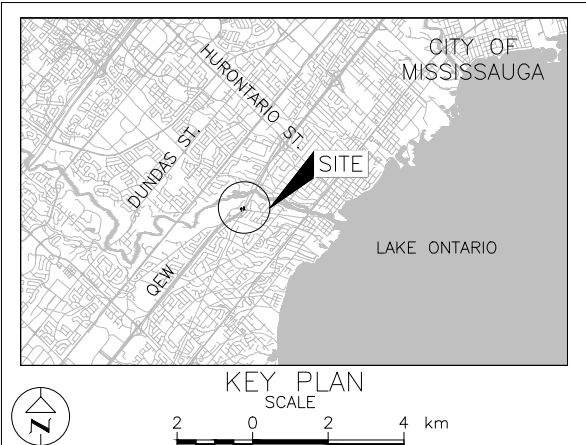
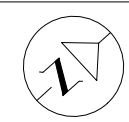
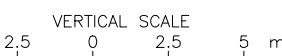
**REFERENCE**

Base plans provided in digital format by Morrison Hershfield, drawing file P7733-117404-8, dated April 27, 2017, received April 27, 2016.



A-A'

QEW CENTERLINE PROFILE



**LEGEND**

●	Borehole
⊥	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)
R	Refusal
≡	WL upon completion of drilling

**BOREHOLE CO-ORDINATES (MTM NAD83 ZONE 10)**

No.	ELEVATION (m)	NORTHING	EASTING
MO-01A	98.9	4823580.0	295566.4
MO-01B	98.9	4823579.3	295565.7
MO-01C	99.2	4823581.1	295567.5
MO-02	99.8	4823572.9	295595.8
MO-03	94.0	4823599.6	295570.4
MO-04	99.7	4823583.9	295593.2
MO-05	99.6	4823585.1	295623.8
MO-06	93.6	4823610.0	295581.0
MO-07	99.4	4823616.9	295622.5
MO-08A	98.9	4823630.5	295614.5
MO-08B	98.9	4823632.0	295615.8
MO-09	99.2	4823634.3	295652.9
MO-10	93.1	4823597.7	295592.3
MO-11	92.9	4823604.7	295610.1
MO-12	92.9	4823592.2	295628.5
MR11-01	94.3	4823600.3	295564.2
MR11-02	93.3	4823601.1	295645.2

NO.	DATE	BY	REVISION
1	May 22, 2018	SMD	Issue for Review
2	May 22, 2018	MWK	Reviewed
3	May 22, 2018	JMAC	Approved

Geocres No. 30M12-418

HWY. QEW	PROJECT NO. 1662333	DIST. CENTRAL
SUBM'D. JIL	CHKD. SM	DATE: 5/7/2018
DRAWN: SMD	CHKD. MWK	APPD. JMAC
		DWG. 01

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



LEGEND

Borehole

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 Sept. 30, 2011

WL upon completion of drilling

BOREHOLE CO-ORDINATES (MTM NAD83 ZONE 10)			
No.	ELEVATION (m)	NORTHING	EASTING
MO-01A	98.9	4823580.0	295566.4
MO-01B	98.9	4823579.3	295565.7
MO-01C	99.2	4823581.1	295567.5
MO-03	94.0	4823599.6	295570.4
MO-04	99.7	4823583.9	295593.2
MO-05	99.6	4823585.1	295623.8
MO-06	93.6	4823610.0	295581.0
MO-07	99.4	4823616.9	295622.5
MO-08A	98.9	4823630.5	295614.5
MO-08B	98.9	4823632.0	295615.8
MO-10	93.1	4823597.7	295592.3
MO-11	92.9	4823604.7	295610.1
MO-12	92.9	4823592.2	295628.5
MR11-01	94.3	4823600.3	295564.2
MR11-02	93.3	4823601.1	295645.2



NOTES

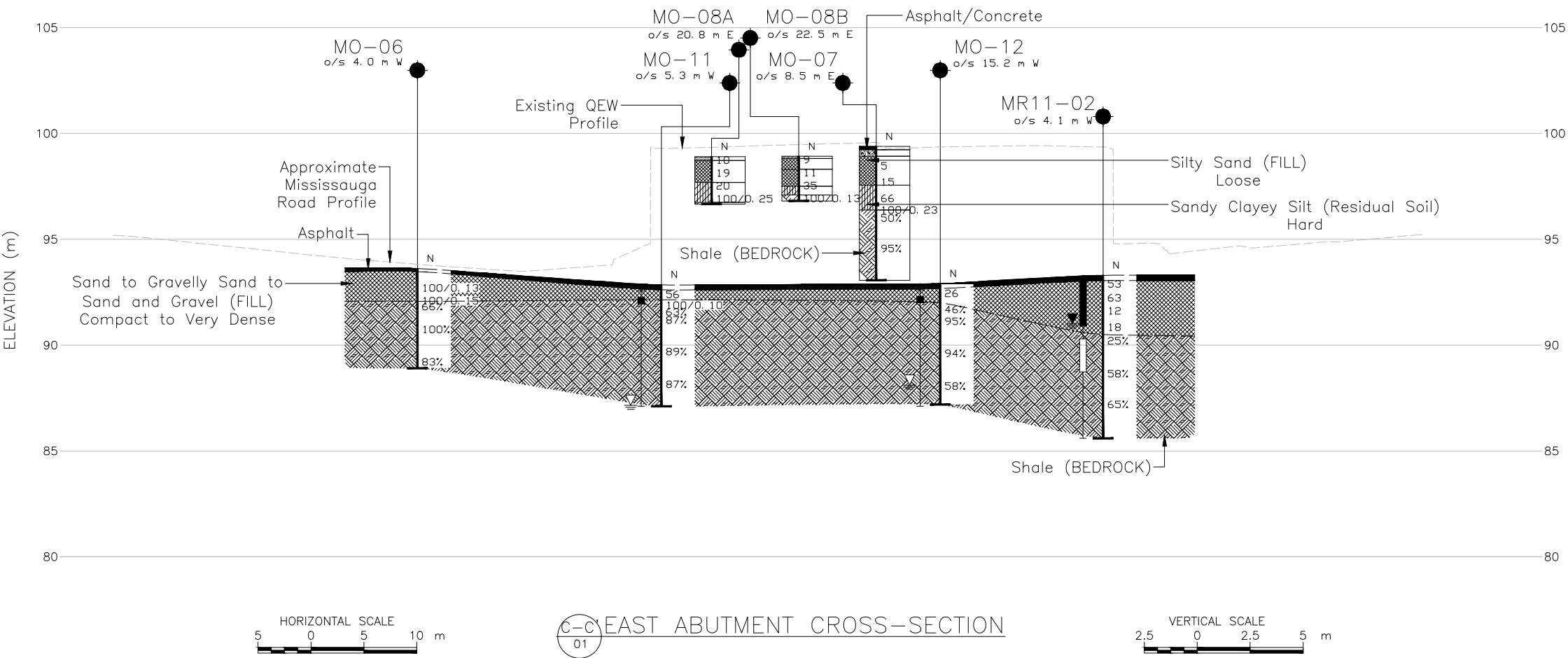
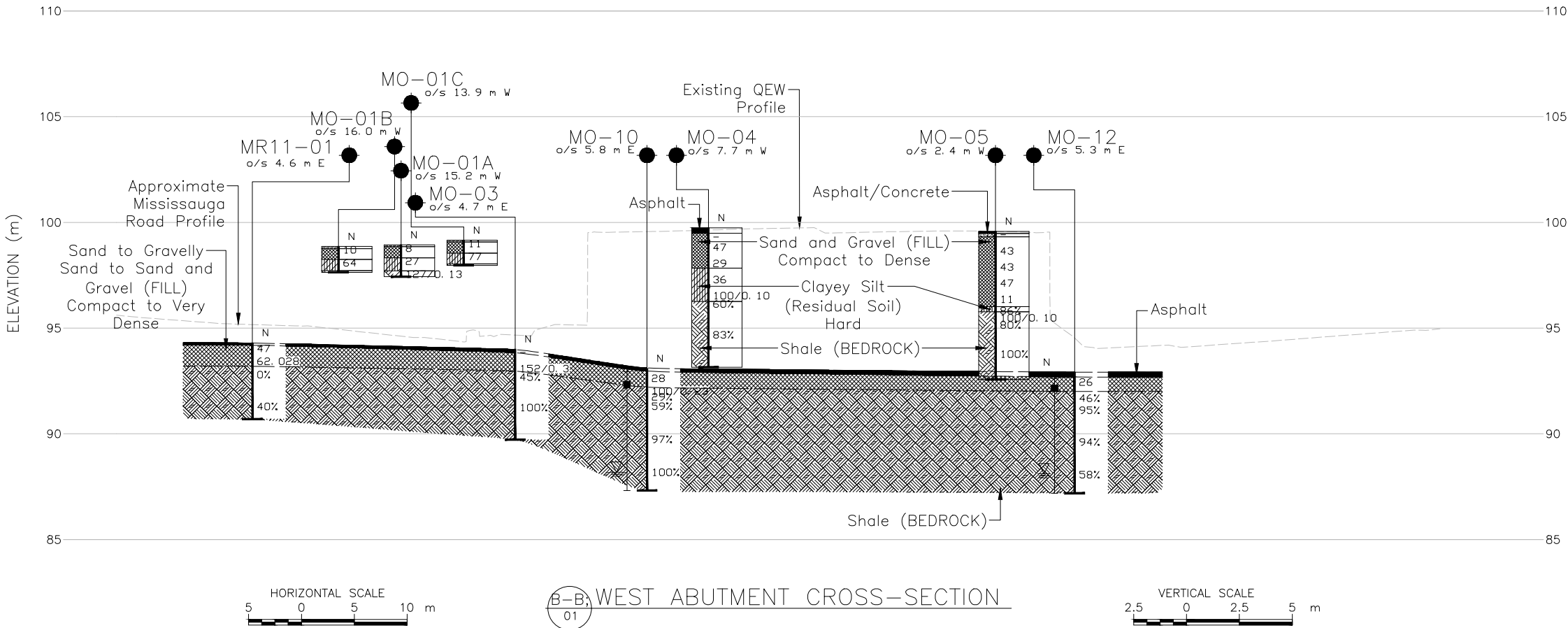
This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

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

REFERENCE

Base plans provided in digital format by Morrison Hershfield, drawing file P7733-117404-8, dated April 27, 2017, received April 27, 2016.

NO.	DATE	BY	REVISION
Geocres No. 30M12-418			
HWY.	QEW	PROJECT NO.	1662333
SUBM'D. JIL	CHKD. SM	DATE:	5/7/2018
DRAWN: SMD	CHKD. MWK	APPD. JMAC	SITE: 24-196
		DWG. 02	







# APPENDIX A

## Record of Boreholes - Previous Investigation (GEOCRES No. 30M12-342)

## SYMBOLS, ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES

### 1. TEXTURAL CLASSIFICATION OF SOILS

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

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

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

### 3. TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

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

NOTE: Hierarchy of Soil Strength Prediction

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



### 4. TERMS DESCRIBING DENSITY (COHESIONLESS SOILS ONLY)

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

### 5. LEGEND FOR RECORDS OF BOREHOLES

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

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






 Water Level  
 Shear Strength Determination by Pocket Penetrometer

- (1) SPT 'N' Value Standard Penetration Test 'N' Value – refers to the number of blows from a 63.5kg hammer free falling a height of 0.76m to advance a standard 50 mm outside diameter split spoon sampler for 0.3 m depth into undisturbed ground.
- (2) DCPT Dynamic Cone Penetration Test – Continuous penetration of a 50 mm outside diameter, 60° conical steel point attached to "A" size rods driven by a 63.5 kg hammer free falling a height of 0.76 m. The resistance to cone penetration is the number of hammer blows required for each 0.3 m advance of the conical point into undisturbed ground.

# UNIFIED SOILS CLASSIFICATION

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

## EXPLANATION OF ROCK LOGGING TERMS

<u>ROCK WEATHERING CLASSIFICATION</u>		<u>SYMBOLS</u>	
<b>Fresh (FR)</b>	No visible signs of weathering.		CLAYSTONE
<b>Fresh Jointed (FJ)</b>	Weathering limited to the surface of major discontinuities.		SILTSTONE
<b>Slightly Weathered (SW)</b>	Penetrative weathering developed on open discontinuity surfaces, but only slight weathering of rock material.		SANDSTONE
<b>Moderately Weathered (MW)</b>	Weathering extends throughout the rock mass, but the rock material is not friable.		COAL
<b>Highly Weathered (HW)</b>	Weathering extends throughout the rock mass and the rock is partly friable.		Bedrock (general)
<b>Completely Weathered (CW)</b>	Rock is wholly decomposed and in a friable condition, but the rock texture and structure are preserved.		

<u>DISCONTINUITY SPACING</u>		<u>STRENGTH CLASSIFICATION</u>			
Bedding	Bedding Plane Spacing	Rock Strength	Approximate Uniaxial Compressive Strength		Field Estimation of Hardness*
			(MPa)	(psi)	
Very thickly bedded	Greater than 2m	Extremely Strong	Greater than 250	Greater than 36,000	Specimen can only be chipped with a geological hammer
Thickly bedded	0.6 to 2m				
Medium bedded	0.2 to 0.6m	Very Strong	100-250	15,000 to 36,000	Requires many blows of geological hammer to break
Thinly bedded	60mm to 0.2m				
Very thinly bedded	20 to 60mm	Strong	50-100	7,500 to 15,000	Requires more than one blow of geological hammer to break
Laminated	6 to 20mm				
Thinly Laminated	Less than 6mm	Medium Strong	25.0 to 50.0	3,500 to 7,500	Breaks under single blow of geological hammer.
<u>TERMS</u>		Weak	5.0 to 25.0	750 to 3,500	Can be peeled by a pocket knife with difficulty
Total Core Recovery: (TCR)	Core recovered as a percentage of total core run length.	Very Weak	1.0 to 5.0	150 to 750	Can be peeled by a pocket knife, crumbles under firm blows of geological pick.
Solid Core Recovery: (SCR)	Percent Ratio of solid core of full cylindrical shape recovered. Expressed with respect to the total length of core run.	Extremely Weak (Rock)	0.25 to 1.0	35 to 150	Indented by thumbnail
Rock Quality Designation: (RQD)	Total length of sound core recovered in pieces 0.1m in length or larger as a percentage of total core run length.				
Uniaxial Compressive Strength (UCS)	Axial stress required to break the specimen				
Fracture Index: (FI)	Frequency of natural fractures per 0.3m of core run.				

RECORD OF BOREHOLE No MR11-01

1 OF 1

METRIC

W.P. W.O. 08-20008 LOCATION N 4 823 600.3 E 295 564.2 QEW Mississauga Road Overpass ORIGINATED BY DA  
 HWY QEW BOREHOLE TYPE Solid Stem Augers/NQ Core Barrel COMPILED BY AN  
 DATUM Geodetic DATE 2011.09.17 - 2011.09.17 CHECKED BY MEF

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								
94.3								20	40	60	80	100				
0.0	ASPHALT: (100mm)		1	AS												
0.1	SAND, some gravel, some silt, trace clay Dense Brown Moist (FILL)		1	SS	47		94									17 62 16 5
93.3	Some limestone fragments		2	SS	62/ 0.28											
1.1	SHALE, weathered, grey						93									
92.8	END OF SPT SAMPLING AT 1.2m. AUGER TO 1.5m AND START CORING. FOR ROCK DETAILS PLEASE REFER TO MR11-01R.															
1.5																

+<sup>3</sup> . X<sup>3</sup> : Numbers refer to  
Sensitivity 20  
15 5  
10 (%) STRAIN AT FAILURE

# RECORD OF BOREHOLE MR11-01R

PROJECT : QEW Mississauga Rd. Overpass  
LOCATION : Mississauga, ON  
STARTED : September 17, 2011  
COMPLETED : September 17, 2011

Project No. W.O. 08-20008

INCLINATION: Vertical AZIMUTH:

SHEET 1 OF 1  
DATUM Geodetic

[illegible]

## GROUNDWATER ELEVATIONS

▽ SHALLOW/SINGLE INSTALLATION  
WATER LEVEL (date)

 DEEP/DUAL INSTALLATION  
WATER LEVEL (date)

LOGGED : DA  
CHECKED : MEF



# RECORD OF BOREHOLE No MR11-02

1 OF 1

METRIC

W.P. W.O. 08-20008 LOCATION N 4 823 601.1 E 295 645.2 QEW Mississauga Road Overpass ORIGINATED BY DA  
 HWY QEW BOREHOLE TYPE Solid Stem Augers/NQ Core Barrel COMPILED BY AN  
 DATUM Geodetic DATE 2011.09.17 - 2011.09.17 CHECKED BY MEF

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
93.3														
0.0 93.1	ASPHALT: (250mm)													
0.3	SAND, some gravel Very Dense Brown Moist (FILL)		1	SS	53		93							
0.6 92.6														
0.7	Gravelly SAND, trace organics Very Dense Brown Moist (FILL)		2	SS	63		92							
1.3														
92.0														
1.3	SAND, fine grained, trace silt, some limestone fragments, trace shale fragments Compact Moist (FILL)		3	SS	12		91							
90.5														
2.8 90.2	SHALE, weathered, grey		5	SS	50/									49 37 14 (SI+CL)
3.1	END OF SPT SAMPLING AT 3.1m AND START CORING. FOR ROCK DETAILS PLEASE REFER TO MR11-02R.  Piezometer installation consists of 25mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen.  WATER LEVEL READINGS: DATE      DEPTH (m)      ELEV. (m) Sep.30/11      2.3      91.0				0.100									

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to  
Sensitivity

20  
15  
10  
(%) STRAIN AT FAILURE

# RECORD OF BOREHOLE MR11-02R

PROJECT : QEW Mississauga Rd. Overpass  
 LOCATION : Mississauga, ON  
 STARTED : September 17, 2011  
 COMPLETED : September 17, 2011

Project No. W.O. 08-20008

INCLINATION: Vertical AZIMUTH:

SHEET 1 OF 1  
 DATUM Geodetic

DEPTH SCALE (metres)	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV.	RUN No.	PENETRATION RATE (mm/min)	COLOUR % RETURN	FR-FRACTURE CL-CLEAVAGE SH-SHEAR VN-VEIN	F-FAULT J-JOINT P-POLISHED S-SLICKENSIDED	SM-SMOOTH R-ROUGH ST-STEPPED PL-PLANAR	FL-FLEXURED UE-UNEVEN W-WAVY C-CURVED	Unconfined Compressive Strength (MPa)	FIELD/LABORATORY TESTING RESULTS ● Point Load Test Diametral ▲ Point Load Test Axial ■ Laboratory UCS Test								
				DEPTH										FLUSH	RECOVERY		R.Q.D. %	FRACT. INDEX PER .3 m	DISCONTINUITY DATA		HYDRAULIC CONDUCTIVITY k, cm/sec
				(m)											TOTAL CORE %	SOLID CORE %			DIP wrt Core Axis	TYPE AND SURFACE DESCRIPTION	
															80 60 40 20	80 60 40 20			0 30 60 90		
				93.31 3.1																	
4	NQ Coring RUN	SHALE, highly to moderately weathered, fine grained, thinly bedded, grey, very weak to weak, with strong to very strong limestone interbeds: (GEORGIAN BAY FORMATION)		1	0.06	100															
6	RUN	becoming slightly weathered		2	0.09	100															
		limestone interbeds from 5.7m to 5.8m																			
8	RUN	vertical joint from 6.6m to 7.0m		3	0.07	100															
8		END OF BOREHOLE AT 7.7m.			85.6 7.7																
10																					
12																					

## GROUNDWATER ELEVATIONS

▽ SHALLOW/SINGLE INSTALLATION  
 WATER LEVEL (date)

▼ DEEP/DUAL INSTALLATION  
 WATER LEVEL (date)

LOGGED : DA  
 CHECKED : MEF



# **APPENDIX B**

## **Record of Borehole and Record of Drillhole Sheets - Current Investigation**



## LIST OF SYMBOLS

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

### I. GENERAL

$\pi$	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

$\gamma$	shear strain
$\Delta$	change in, e.g. in stress: $\Delta \sigma$
$\varepsilon$	linear strain
$\varepsilon_v$	volumetric strain
$\eta$	coefficient of viscosity
$\nu$	Poisson's ratio
$\sigma$	total stress
$\sigma'$	effective stress ( $\sigma' = \sigma - u$ )
$\sigma'_{vo}$	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
$\sigma_{oct}$	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
$\tau$	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

### III. SOIL PROPERTIES

<b>(a)</b>	<b>Index Properties</b>
$\rho(\gamma)$	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
$\gamma'$	unit weight of submerged soil ( $\gamma' = \gamma - \gamma_w$ )
$D_R$	relative density (specific gravity) of solid particles ( $D_R = \rho_s / \rho_w$ ) (formerly $G_s$ )
e	void ratio
n	porosity
S	degree of saturation

### (a) Index Properties (continued)

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

### (b) Hydraulic Properties

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

### (c) Consolidation (one-dimensional)

$C_c$	compression index (normally consolidated range)
$C_r$	recompression index (over-consolidated range)
$C_s$	swelling index
$C_\alpha$	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
$\sigma'_p$	pre-consolidation stress
OCR	over-consolidation ratio = $\sigma'_p / \sigma'_{vo}$

### (d) Shear Strength

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

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

Notes: 1  
2

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

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



## LIST OF ABBREVIATIONS

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

### I. SAMPLE TYPE

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

### II. PENETRATION RESISTANCE

#### Standard Penetration Resistance (SPT), N:

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

#### Dynamic Cone Penetration Resistance; $N_d$ :

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

**PH:** Sampler advanced by hydraulic pressure

**PM:** Sampler advanced by manual pressure

**WH:** Sampler advanced by static weight of hammer

**WR:** Sampler advanced by weight of sampler and rod

#### Piezo-Cone Penetration Test (CPT)

A electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm<sup>2</sup> 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

Density Index	N
Relative Density	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 <sup>1</sup>
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement <sup>1</sup>
$D_R$	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 <sub>4</sub>	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (LV-laboratory vane test)
$\gamma$	unit weight

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

### V. MINOR SOIL CONSTITUENTS

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



## LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

### WEATHERINGS STATE

**Fresh:** no visible sign of weathering

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

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

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

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

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

### BEDDING THICKNESS

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

### JOINT OR FOLIATION SPACING

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

### GRAIN SIZE

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

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

### CORE CONDITION

#### Total Core Recovery (TCR)

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

#### Solid Core Recovery (SCR)

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

#### Rock Quality Designation (RQD)

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

### DISCONTINUITY DATA

#### Fracture Index

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

#### Dip with Respect to Core Axis

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

#### Description and Notes

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

#### Abbreviations

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



PROJECT		RECORD OF BOREHOLE				No MO-01A		SHEET 1 OF 1		METRIC				
G.W.P. 2002-13-00		LOCATION				N 4823580.0; E 295566.4 MTM NAD		ZONE 10 (LAT. 43.552090; LONG. -79.614279)		ORIGINATED BY AJ				
DIST Central HWY QEW		BOREHOLE TYPE				64 mm O.D. 51 mm I.D. Split Spoon Sampler				COMPILED BY JL				
DATUM Geodetic		DATE				December 20, 2017				CHECKED BY GDS				
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
98.9	GROUND SURFACE													
0.0	TOPSOIL (75 mm)		1A											
98.3	Silty gravelly sand, some clay (FILL)		1B	SS	8									29 34 25 12
0.6	Loose Brown Moist		2	SS	27									
97.7	CLAYEY SILT, trace to some sand													
97.4	Very stiff Brown to grey Moist		3	SS	27/0.13									
1.5	SHALE (BEDROCK)													
	Grey													
	END OF BOREHOLE													
NOTE: 1. Borehole dry upon completion of drilling.														

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PROJECT		1662333		RECORD OF BOREHOLE No MO-01B		SHEET 1 OF 1		METRIC										
G.W.P.		2002-13-00		LOCATION		N 4823579.3; E 295565.7 MTM NAD ZONE 10 (LAT. 43.552084; LONG. -79.614288)		ORIGINATED BY										
DIST		Central HWY QEW		BOREHOLE TYPE		64 mm O.D. 51 mm I.D. Split Spoon Sampler		COMPILED BY										
DATUM		Geodetic		DATE		December 20, 2017		CHECKED BY										
								GDS										
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV	DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa					WATER CONTENT (%)			γ	GR SA SI CL
								20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					W <sub>p</sub> W W <sub>L</sub> 10 20 30					
98.9		GROUND SURFACE																
0.0		TOPSOIL (100 mm)		1A														
0.1		Gravelly sand, trace clay, some silt to silty (FILL)		1B	SS	10												28 54 14 4
98.3		Loose Grey Moist		2A	SS	64		98										
0.6																		
97.8		CLAYEY SILT, trace to some sand		2B														
1.2		Hard Brown to grey Moist																
		SHALE (BEDROCK)																
		Grey																
		END OF BOREHOLE																
NOTE: 1. Borehole dry upon completion of drilling.																		

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PROJECT		RECORD OF BOREHOLE				No MO-01C		SHEET 1 OF 1		METRIC							
G.W.P. 2002-13-00		LOCATION				N 4823581.1; E 295567.5 MTM NAD		ZONE 10 (LAT. 43.552100; LONG. -79.614265)		ORIGINATED BY AJ							
DIST Central HWY QEW		BOREHOLE TYPE				64 mm O.D. 51 mm I.D. Split Spoon Sampler				COMPILED BY JL							
DATUM Geodetic		DATE				December 20, 2017				CHECKED BY GDS							
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
99.2	GROUND SURFACE							20	40	60	80	100					
98.9	TOPSOIL (75 mm)		1A	SS	11		99										
98.6	Sand, trace clay, some silt to silty, some gravel, trace organics (FILL)		1B	SS													
98.1	Compact Grey Moist		2A	SS	77												
98.1	CLAYEY SILT, trace to some sand		2B				98										
1.2	Hard Brown Moist																
	SHALE (BEDROCK) Grey																
	END OF BOREHOLE																
NOTE: 1. Borehole dry upon completion of drilling.																	

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PROJECT 1662333		RECORD OF BOREHOLE No MO-02				SHEET 1 OF 1		METRIC						
G.W.P. 2002-13-00		LOCATION N 4823572.9; E 295595.8 MTM NAD 83 ZONE 10 (LAT. 43.552027; LONG. -79.613914)				ORIGINATED BY FC								
DIST Central HWY QEW		BOREHOLE TYPE CME 75, 203 mm O.D. Hollow Stem Augers				COMPILED BY KN								
DATUM Geodetic		DATE August 24, 2017				CHECKED BY SMM								
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
99.8	GROUND SURFACE													
0.0	ASPHALT (150 mm)													
99.3	CONCRETE (300 mm)													
0.5	Silty sand, trace to some gravel, trace to some clay (FILL) Compact Grey to brown Moist		1	AS	-									4 68 23 5
			2	SS	15									
98.1	Sandy clayey silt, some gravel (FILL) Mottled brown Moist		3A 3B 3C	SS	7									
97.6	Sand, trace clay, some silt (FILL) Loose Brown Moist		4	SS	44									19 27 43 11
96.5	Sandy CLAYEY SILT, some gravel, some shale fragments (RESIDUAL SOIL) Hard Brown Moist		5	SS	100/0.13									
3.3	END OF BOREHOLE - AUGER REFUSAL													
NOTE: 1. Borehole dry upon completion of drilling.														

PROJECT 1662333		RECORD OF BOREHOLE No MO-03				SHEET 1 OF 1		METRIC									
G.W.P. 2002-13-00		LOCATION N 4823599.6; E 295570.4 MTM NAD 83 ZONE 10 (LAT. 43.552266; LONG. -79.614229)				ORIGINATED BY FC											
DIST Central HWY QEW		BOREHOLE TYPE CME 55, 203 mm O.D. Hollow Stem Augers, NW Casing				COMPILED BY KN											
DATUM Geodetic		DATE August 24, 2017				CHECKED BY SMM											
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
94.0	GROUND SURFACE							20	40	60	80	100					
0.0	ASPHALT (150 mm)																
0.2	Sand and gravel, trace to some fines (FILL) Brown Moist		1	AS	-												45 45 (10)
93.0			2A	SS	152/0.3												
1.0	SHALE (BEDROCK) Grey  Bedrock cored from a depth of 1.2 m to 4.3 m  For bedrock coring details, refer to Record of Drillhole MO-03		2B														
			1	RC	REC 88%												RQD = 45%
			2	RC	REC 100%												RQD = 100%
89.7																	
4.3	END OF BOREHOLE																
	NOTES:  1. Borehole dry prior to rock coring.																

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SHEET 1 OF 1

DATUM: Geodetic

DRILLING CONTRACTOR: Aardvark Drilling

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CHECKED: AC

PROJECT 1662333		RECORD OF BOREHOLE No MO-04		SHEET 1 OF 1		METRIC											
G.W.P. 2002-13-00		LOCATION N 4823583.9; E 295593.2 MTM NAD 83 ZONE 10 (LAT. 43.552126; LONG. -79.613947)		ORIGINATED BY FC													
DIST Central HWY QEW		BOREHOLE TYPE CME 75, 203 mm O.D. Hollow Stem Augers, HQ Casing		COMPILED BY KN													
DATUM Geodetic		DATE September 7, 2017		CHECKED BY SMM													
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa					WATER CONTENT (%)			γ	GR SA SI CL
							20 40 60 80 100	20 40 60 80 100	W <sub>p</sub>	W	W <sub>L</sub>	10 20 30	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100		
99.7	GROUND SURFACE																
0.0	ASPHALT (250 mm)																
0.3	Sand and gravel, trace to some silt, trace clay (FILL) Compact to dense Brown Moist		1	AS	-		99									32 60 6 2	
			2	SS	47												
97.8			3A	SS	29		98										
1.9	CLAYEY SILT with SAND, some gravel, some shale fragments (RESIDUAL SOIL) Hard Brown to grey Moist		3B														
			4	SS	36		97									20 38 35 7	
			5	SS	100/0.10												
96.2			6	SS	100/0.10												
3.5	SHALE (BEDROCK) Grey						96										
	Bedrock cored from a depth of 3.5 m to 6.6 m		1	RC	REC 87%		95									RQD = 60%	
	For bedrock coring details, refer to Record of Drillhole MO-04		2	RC	REC 95%		94									RQD = 83%	
93.1	END OF BOREHOLE																
6.6	NOTES:  1. Borehole dry prior to rock coring.																

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PROJECT 1662333		RECORD OF BOREHOLE No MO-05		SHEET 1 OF 1		METRIC															
G.W.P. 2002-13-00		LOCATION N 4823585.1; E 295623.8 MTM NAD 83 ZONE 10 (LAT. 43.552136; LONG. -79.613568)		ORIGINATED BY FC																	
DIST Central HWY QEW		BOREHOLE TYPE CME 75, 203 mm O.D. Hollow Stem Augers, HQ Casing		COMPILED BY KN																	
DATUM Geodetic		DATE August 22, 2017		CHECKED BY SMM																	
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)						
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa					WATER CONTENT (%)			γ			GR SA SI CL		
99.6	GROUND SURFACE							20 40 60 80 100					10 20 30			kN/m³					
0.0	ASPHALT (100 mm)							○ UNCONFINED + FIELD VANE					W <sub>p</sub> W W <sub>L</sub>								
0.3	CONCRETE (150 mm)		1	AS	-			● QUICK TRIAXIAL × REMOULDED													
	Sand and gravel, trace to some silt, trace clay (FILL) Compact to dense Brown Moist		2	SS	43		99						○								
			3	SS	43		98														
			4	SS	47		97						○						40 50 8 2		
			5A	SS	11		96						○						39 35 19 7		
96.0	Sandy CLAYEY SILT, some shale fragments (RESIDUAL SOIL) Brown-grey Moist		6	SS	100/10 10																
3.8	SHALE (BEDROCK) Grey		1	RC	REC 100%		95												RQD = 86%		
	Bedrock cored from a depth of 3.9 m to 6.4 m		2	RC	REC 100%		94												RQD = 80%		
	For bedrock coring details, refer to Record of Drillhole MO-05		3	RC	REC 100%														RQD = 100%		
93.2	END OF BOREHOLE																				
6.4	NOTE: 1. Borehole dry prior to rock coring.																				

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PROJECT: 1662333

**RECORD OF DRILLHOLE: MO-05**

SHEET 1 OF 1

LOCATION: N 4823585.1 ;E 295623.8

DRILLING DATE: August 22, 2017

DATUM: Geodetic


INCLINATION: -90° AZIMUTH: —


DRILL RIG: CME 75

DRILLING CONTRACTOR: Aardvark Drilling


DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	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.													FEATURES	R0/R1 ZONES																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																							
						RECOVERY		R.Q.D. %	FRACT. INDEX PER Meter	DISCONTINUITY DATA						ROCK STRENGTH INDEX					WEATH- ERING INDEX																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																						
						TOTAL CORE %	SOLID CORE %			B Angle 0 to 90 N to S	DIP w.r.t. CORE AXIS 0 to 90 S to N	TYPE AND SURFACE DESCRIPTION	Jr	Ja	R4 R3 R2 R1	W1 W2 W3 W4 W5 W6																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																											
4	HQ Casing	Continued from Borehole MO-05		95.70 3.90	1																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																						

## FEATURES LEGEND

 BROKEN CORE

 CLAY SEAM

 LIMESTONE

 LOST CORE

DEPTH SCALE

1 : 50



LOGGED: FC

CHECKED: AC

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PROJECT 1662333		RECORD OF BOREHOLE No MO-06				SHEET 1 OF 1		METRIC									
G.W.P. 2002-13-00		LOCATION N 4823610.0; E 295581.0 MTM NAD 83 ZONE 10 (LAT. 43.552360; LONG. -79.614098)				ORIGINATED BY FC											
DIST Central HWY QEW		BOREHOLE TYPE CME 75, 203 mm O.D. Hollow Stem Augers, HQ Casing				COMPILED BY KN											
DATUM Geodetic		DATE August 24, 2017				CHECKED BY SMM											
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
93.6	GROUND SURFACE							20	40	60	80	100					
0.0	ASPHALT (150 mm)																
0.2	Sand and gravel, trace to some silt, trace clay (FILL) Very dense Brown Moist		1	AS	-												
			2	SS	100/0.13												32 55 10 3
92.0	SHALE (BEDROCK) Grey		3	SS	100/0.15												
1.6	Bedrock cored from a depth of 1.7 m to 4.7 m  For bedrock coring details, refer to Record of Drillhole MO-06		1	RC	REC 93%												RQD = 66%
			2	RC	REC 100%												RQD = 100%
88.9	END OF BOREHOLE		3	RC	REC 100%												RQD = 83%
4.7	NOTE:  1. Borehole dry prior to rock coring.																

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[illegible]

## FEATURES LEGEND



BROKEN CORE



CLAY SEAM



LIMESTONE



LOST CORE

DEPTH SCALE

1 : 50



# GOLDER

LOGGED: FC

CHECKED: AC

PROJECT 1662333		RECORD OF BOREHOLE No MO-07				SHEET 1 OF 1		METRIC								
G.W.P. 2002-13-00		LOCATION N 4823616.9; E 295622.5 MTM NAD 83 ZONE 10 (LAT. 43.552423; LONG. -79.613584)				ORIGINATED BY FC										
DIST Central HWY QEW		BOREHOLE TYPE CME 75, 203 mm O.D. Hollow Stem Augers, HQ Casing				COMPILED BY KN										
DATUM Geodetic		DATE August 30, 2017				CHECKED BY SMM										
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								
99.4	GROUND SURFACE															
0.0	ASPHALT (150 mm)															
98.9	CONCRETE (300 mm)		1	AS	-											
0.5	Silty sand, trace clay (FILL) Loose Brown Moist		2	SS	5											0 68 28 4
97.6	- Clayey silt, containing organics from a depth of about 1.3 m to 1.4 m		3A	SS	15											
1.8	Sandy CLAYEY SILT, some gravel, some shale fragments (RESIDUAL SOIL) Hard Brown Moist		3B													13 29 43 15
96.4	SHALE (BEDROCK) Grey		4	SS	66											
3.0	Bedrock cored from a depth of 3.3 m to 6.3 m  For bedrock coring details, refer to Record of Drillhole MO-07		5	SS	100/0.23											
			1	RC	REC 78%											RQD = 50%
			2	RC	REC 100%											RQD = 95%
93.1	END OF BOREHOLE															
6.3	NOTE:  1. Borehole dry prior to rock coring.															

GTA-MTO 001 \GOLDER\GDS\GAL\MISSISSAUGA\CLIENTS\MTQEQW-CREDIT\_RIVER\02\_DATA\GINTQEQW-CREDIT\_RIVER.GPJ GAL-GTA.GDT 5/7/18

PROJECT: 1662333

## RECORD OF DRILLHOLE: MO-07

SHEET 1 OF 1

LOCATION: N 4823616.9 ;E 295622.5

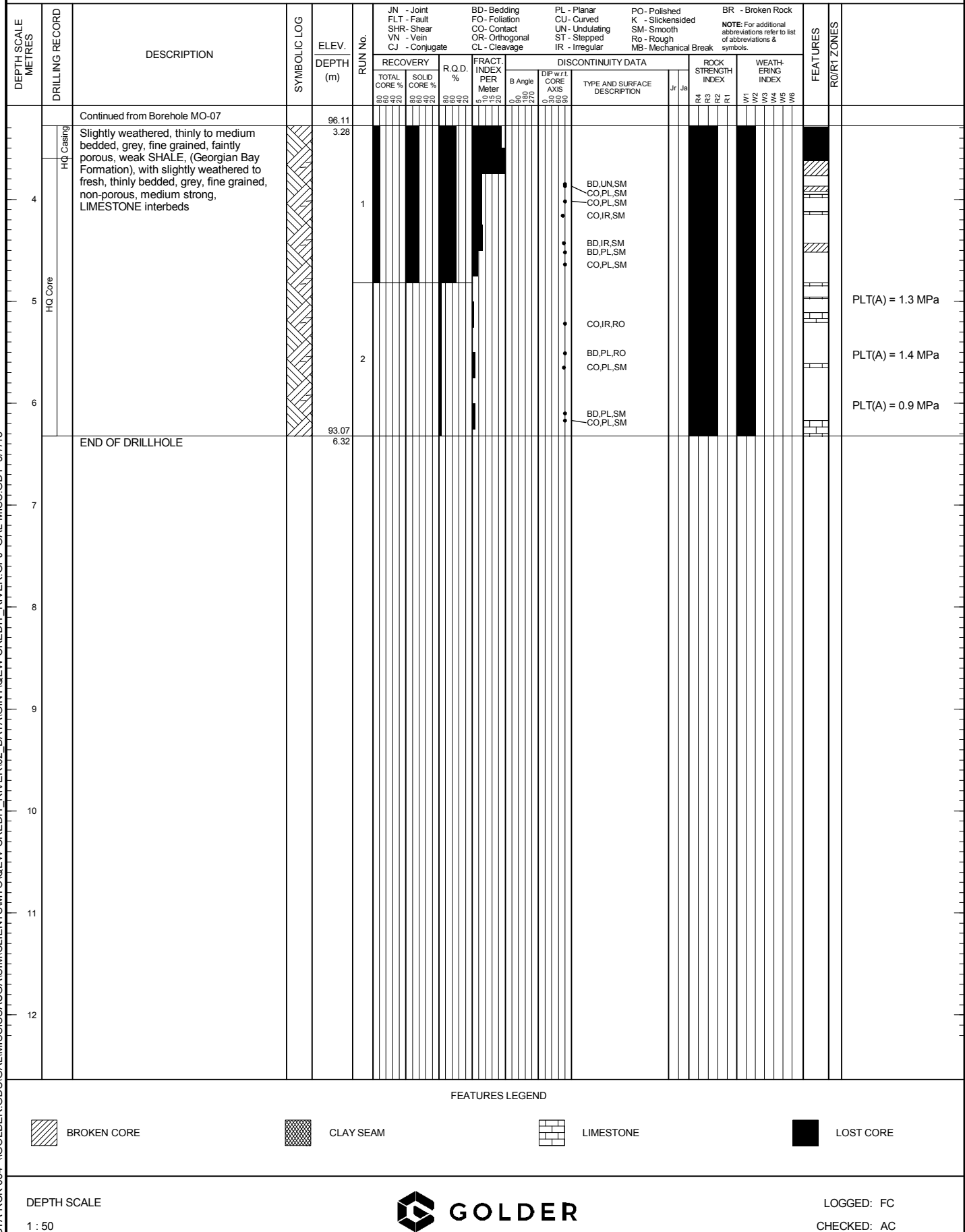
DRILLING DATE: August 30, 2017

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: CME 75

DRILLING CONTRACTOR: Aardvark Drilling



GTA-RCK 054 \GOLDER.GDS\GALMISSAUGAISIMCLIENTS\MTQEQW-CREDIT\_RIVER\02\_DATA\GINTQEQW-CREDIT\_RIVER.GPJ GAL-MISS.GDT 5/7/18

PROJECT		1662333		RECORD OF BOREHOLE No MO-08A		SHEET 1 OF 1		METRIC							
G.W.P.		2002-13-00		LOCATION		N 4823630.5; E 295614.5 MTM NAD 83 ZONE 10 (LAT. 43.552545; LONG. -79.613684)		ORIGINATED BY							
DIST		Central		HWY		QEW		BOREHOLE TYPE							
64 mm O.D. 51 mm I.D. Split Spoon Sampler		COMPILED BY		DM		DATE		December 21, 2017							
DATUM		Geodetic		CHECKED BY		GDS									
SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS		ELEVATION SCALE		DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT		REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES										
98.9	GROUND SURFACE														
0.0	TOPSOIL (180 mm)		1A		10										
0.2	Silty sand, trace gravel, some organics (FILL) Compact Brown Moist to wet		1B	SS											
			2	SS	19										
97.7															
1.2	CLAYEY SILT, trace to some sand, trace to some gravel Very stiff to hard Grey to brown Moist		3A	SS	20										
			3B												
96.8			4A	SS	100/0.25										
			4B												
2.2	SHALE (BEDROCK) Grey END OF BOREHOLE														
	NOTE: 1. Borehole dry upon completion of drilling.														

GTA-MTO 001 \GOLDER\GDS\GAL\MISSISSAUGA\CLIENTS\IMTO\QEW-CREDIT\_RIVER\02\_DATA\GINTQEW-CREDIT\_RIVER.GPJ GAL-GTA.GDT 5/7/18

PROJECT <u>1662333</u>		<b>RECORD OF BOREHOLE No MO-08B</b>				SHEET 1 OF 1		<b>METRIC</b>	
G.W.P. <u>2002-13-00</u>		LOCATION <u>N 4823632.0; E 295615.8 MTM NAD 83 ZONE 10 (LAT. 43.552559; LONG. -79.613668)</u>				ORIGINATED BY <u>JL</u>			
DIST <u>Central</u> HWY <u>QEW</u>		BOREHOLE TYPE <u>64 mm O.D. 51 mm I.D. Split Spoon Sampler</u>				COMPILED BY <u>DM</u>			
DATUM <u>Geodetic</u>		DATE <u>December 21, 2017</u>				CHECKED BY <u>GDS</u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
						○ UNCONFINED    + FIELD VANE ● QUICK TRIAXIAL    × REMOULDED					WATER CONTENT (%)						
						20	40	60	80	100	10	20	30				
98.9	GROUND SURFACE																
0.0	TOPSOIL (100 mm)																
98.3	Clayey silt, trace sand, trace gravel, some rootlets (FILL)		1	SS	9							○					
0.6	Stiff Brown to black Moist		2	SS	11							○					
97.5	Silty sand, trace to some clay, trace gravel, some rootlets (FILL)		3A	SS	35							○					
1.4	Compact		3B	SS								○					
97.1	Brown Moist to wet																
96.8	CLAYEY SILT		4	SS	100/0.13												
2.1	Hard Brown to grey Moist SHALE (BEDROCK) Grey END OF BOREHOLE																
NOTE: 1. Borehole dry upon completion of drilling.																	

GTA-MTO 001 \GOLDER\GDS\GAL\MISSISSAUGA\CLIENTS\IMTO\QEW-CREDIT\_RIVER\02\_DATA\GINTQEW-CREDIT\_RIVER.GPJ GAL-GTA.GDT 5/7/18



PROJECT 1662333		RECORD OF BOREHOLE No MO-09		SHEET 1 OF 1		METRIC													
G.W.P. 2002-13-00		LOCATION N 4823634.3; E 295652.9 MTM NAD 83 ZONE 10 (LAT. 43.552580; LONG. -79.613208)		ORIGINATED BY FC															
DIST Central HWY QEW		BOREHOLE TYPE CME 75, 203 mm O.D. Hollow Stem Augers, HQ Casing		COMPILED BY KN															
DATUM Geodetic		DATE August 29, 2017		CHECKED BY SMM															
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa			WATER CONTENT (%)			γ			GR SA SI CL		
99.2	GROUND SURFACE							20 40 60 80 100	20 40 60 80 100	10 20 30									
0.0	ASPHALT (130 mm)						99												
98.7	CONCRETE (280 mm)																		
0.5	Sand, some silt, trace clay (FILL) Brown Moist		1	AS	-														
97.9			2A	SS	5		98											0 86 12 2	
1.3	Clayey silt with sand, some gravel, trace rootlets (FILL) Very stiff Black to brown Moist		3	SS	25													18 35 30 17	
97.0							97												
2.2	Sandy CLAYEY SILT, some shale fragments (RESIDUAL SOIL) Hard Brown Moist		4	SS	100/0.13														
96.2			5	SS	100/0.05														
3.1	SHALE (BEDROCK) Grey END OF BOREHOLE - AUGER REFUSAL																		
NOTE: 1. Borehole dry upon completion of drilling.																			

PROJECT 1662333		RECORD OF BOREHOLE No MO-10				SHEET 1 OF 1		METRIC									
G.W.P. 2002-13-00		LOCATION N 4823597.7; E 295592.3 MTM NAD 83 ZONE 10 (LAT. 43.552249; LONG. -79.613959)				ORIGINATED BY JL											
DIST Central HWY QEW		BOREHOLE TYPE CME 75, 203 mm O.D. Hollow Stem Augers				COMPILED BY DM											
DATUM Geodetic		DATE December 20, 2017				CHECKED BY GDS											
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
93.1	GROUND SURFACE																
0.0	ASPHALT (150 mm)																
0.2	Gravelly sand, trace clay, trace to some silt (FILL) Compact Brown Moist		1	SS	28												26 60 11 3
92.2	SHALE (BEDROCK) Grey		2	SS	100/0.23												
1.1	Bedrock cored from a depth of 1.1 m to 5.8 m.  For bedrock coring details, refer to Record of Drillhole MO-10		1	RC	REC 67%												RQD = 29%
			2	RC	REC 97%												RQD = 59%
			3	RC	REC 100%												RQD = 97%
			4	RC	REC 100%												RQD = 100%
87.3	END OF BOREHOLE																
5.8	NOTES:  1. Borehole dry prior to rock coring.  2. Water level measured at a depth of about 4.9 m below ground surface following completion of rock coring.																

GTA-MTO 001 \GOLDER\GDS\GAL\MISSISSAUGA\CLIENTS\MTQEW-CREDIT\_RIVER\02\_DATA\GINTQEW-CREDIT\_RIVER.GPJ GAL-GTA.GDT 5/7/18

PROJECT: 1662333

## RECORD OF DRILLHOLE: MO-10

SHEET 1 OF 1

LOCATION: N 4823597.7 ;E 295592.3

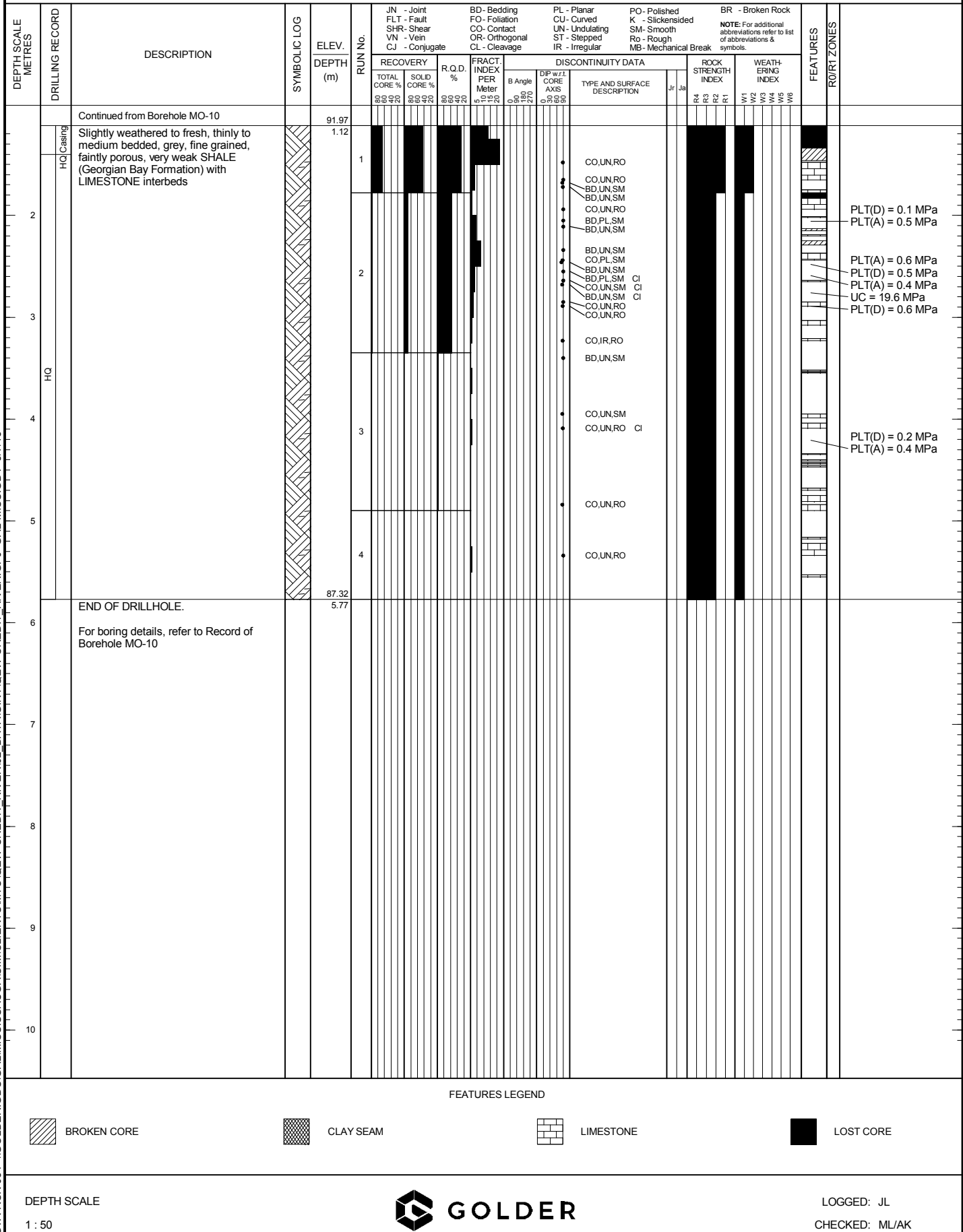
DRILLING DATE: December 20, 2017

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: CME 75

DRILLING CONTRACTOR: Geo-Environmental Drilling



GTA-RCK 054 \\GOLDER.GDS\GALMISSAUGA\SIMCLIENTS\MTQIQEW-CREDIT\_RIVER\02\_DATA\GINTQEW-GDT\_5/7/18

PROJECT 1662333		RECORD OF BOREHOLE No MO-11				SHEET 1 OF 1		METRIC									
G.W.P. 2002-13-00		LOCATION N 4823604.7; E 295610.1 MTM NAD 83 ZONE 10 (LAT. 43.552313; LONG. -79.613738)				ORIGINATED BY JL											
DIST Central HWY QEW		BOREHOLE TYPE CME 75, 203 mm O.D. Hollow Stem Augers				COMPILED BY DM											
DATUM Geodetic		DATE December 19, 2017				CHECKED BY GDS											
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
92.9	GROUND SURFACE																
0.0	ASPHALT (230 mm)																
0.2	Gravelly sand, trace clay, trace to some silt (FILL)		1A	SS	56												29 56 11 4
92.2	Very dense Brown Moist SHALE (BEDROCK)		1B	SS	100/0.10												
0.7	Grey		2	SS	100/0.10												RQD = 63%
	Bedrock cored from a depth of 1.1 m to 5.7 m.		1	RC	REC 100%												
	For bedrock coring details, refer to Record of Drillhole MO-11.		2	RC	REC 100%												RQD = 87%
			3	RC	REC 100%												RQD = 89%
			4	RC	REC 100%												RQD = 87%
87.2	END OF BOREHOLE																
5.7	NOTES: 1. Borehole dry prior to rock coring. 2. Water level measured at a depth of 5.7 m below ground surface following completion of rock coring.																




PROJECT 1662333		RECORD OF BOREHOLE No MO-12				SHEET 1 OF 1		METRIC									
G.W.P. 2002-13-00		LOCATION N 4823592.2; E 295628.5 MTM NAD 83 ZONE 10 (LAT. 43.552201; LONG. -79.613511)				ORIGINATED BY JL											
DIST Central HWY QEW		BOREHOLE TYPE CME 75, 203 mm O.D. Hollow Stem Augers				COMPILED BY DM											
DATUM Geodetic		DATE December 19, 2017				CHECKED BY GDS											
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
92.9	GROUND SURFACE																
0.0	ASPHALT																
0.2	Gravelly sand to sand and gravel, trace fines (FILL) Compact Brown Moist		1	SS	26												
92.0	SHALE (BEDROCK) Grey		1	RC	REC 100%												RQD = 46%
0.9	Bedrock cored from a depth of 1.2 m to 5.7 m.  For bedrock coring details, refer to Record of Drillhole MO-12.		2	RC	REC 100%												RQD = 95%
			3	RC	REC 100%												RQD = 94%
			4	RC	REC 100%												RQD = 58%
87.2	END OF BOREHOLE																
5.7	NOTES:  1. Borehole dry prior to rock coring.  2. Water level measured at a depth of 4.8 m below ground surface following completion of rock coring.																

SHEET 1 OF 1

DATUM: Geodetic

DRILLING CONTRACTOR: Geo-Environmental Drilling

[illegible] BROKEN CORE

CLAY SEAM

 Limestone

LOST CORE

DEPTH SCALE

1 : 50

LOGGED: JL

CHECKED: ML/AK



# **APPENDIX C**

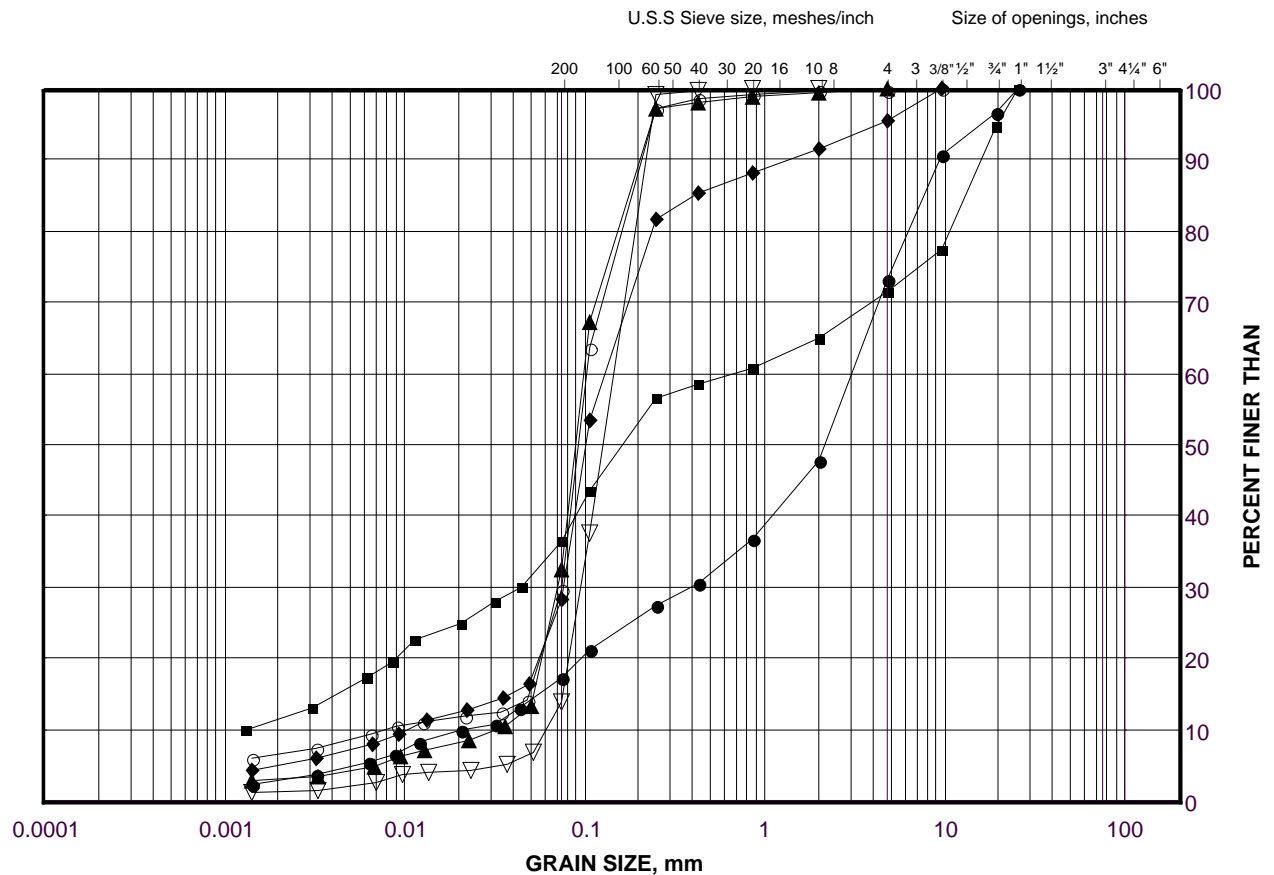
## **Geotechnical Laboratory Test Results, Bedrock Core Photographs and Analytical Test Results**



# GRAIN SIZE DISTRIBUTION

Silty Sand to Gravelly Sand (Fill)

FIGURE C1A



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

## LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	MO-01B	1B	98.6
■	MO-01A	1B	98.6
◆	MO-02	2	98.7
▲	MO-07	2	98.3
▽	MO-09	2A	98.0
○	MO-08B	3A	97.6

Project Number: 1662333

Checked By: MWK

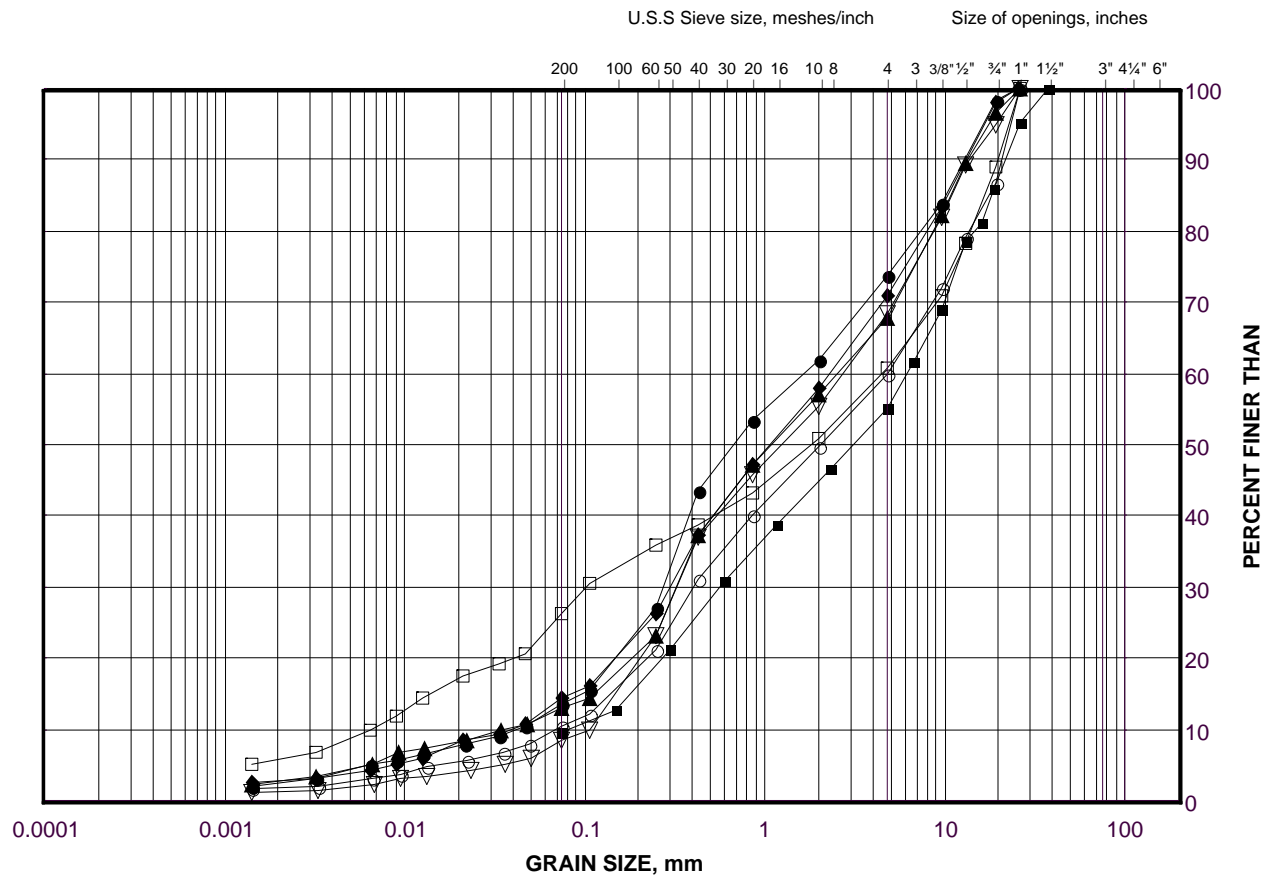
**Golder Associates**

Date: 01-Feb-18

# GRAIN SIZE DISTRIBUTION

Gravelly Sand to Sand and Gravel (Fill)

FIGURE C1B



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

## LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	MO-10	1	92.6
■	MO-03	1	93.6
◆	MO-11	1A	92.3
▲	MO-06	2	92.7
▽	MO-04	2	98.6
○	MO-05	4	97.1
□	MO-05	5B	96.1

Project Number: 1662333

Checked By: MWK

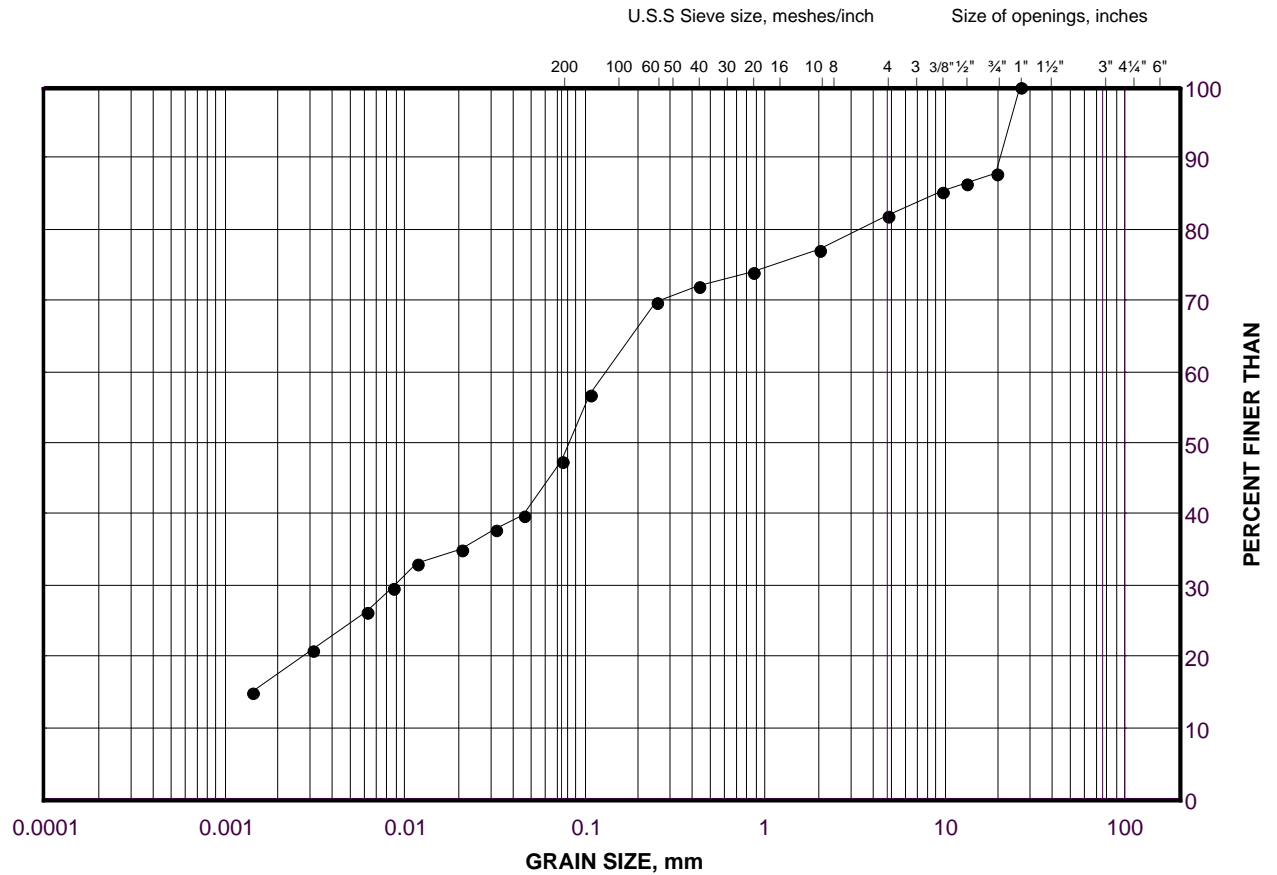
**Golder Associates**

Date: 01-Feb-18

# GRAIN SIZE DISTRIBUTION

Clayey Silt with Sand (Fill)

FIGURE C2



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

## LEGEND

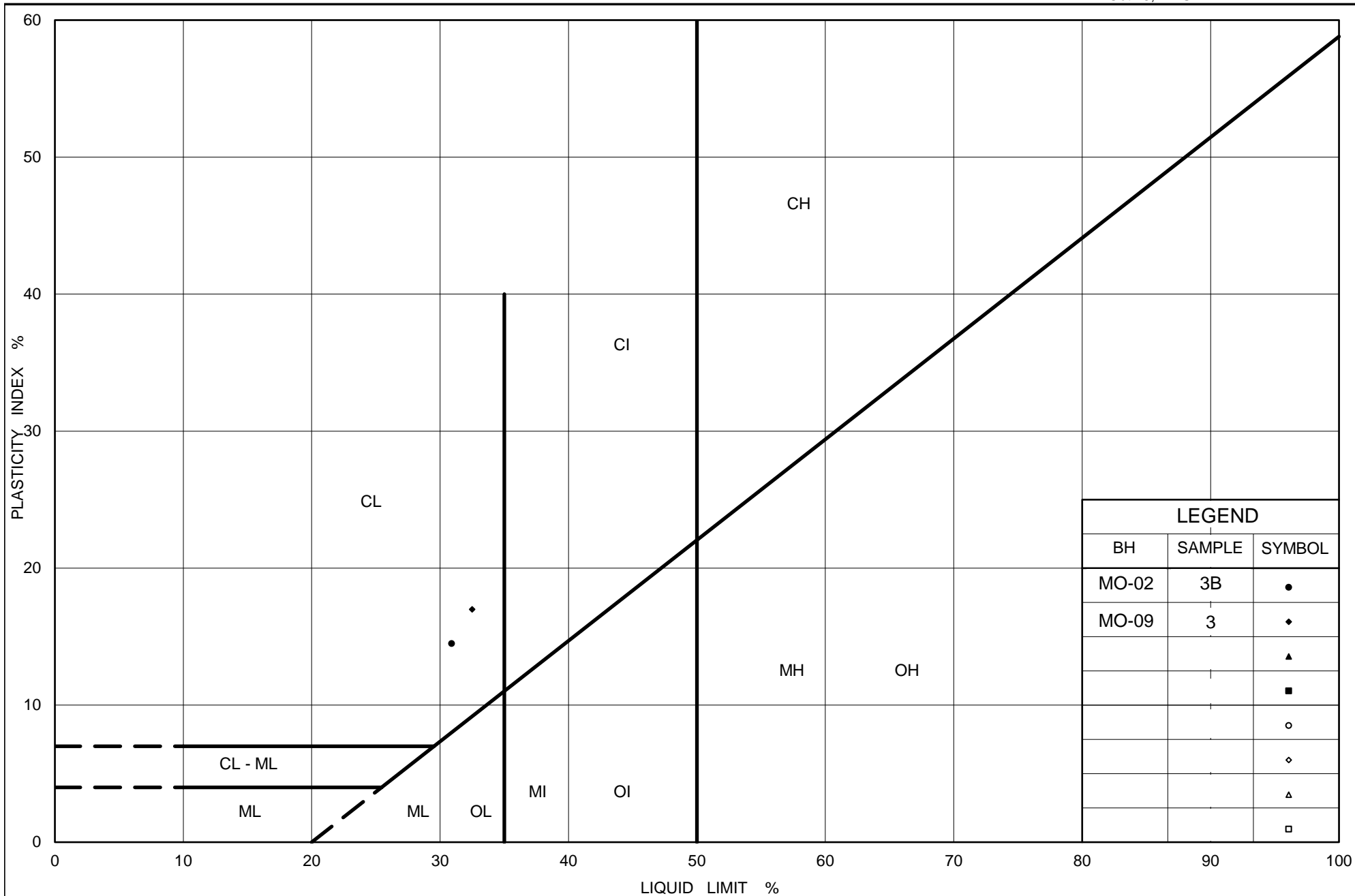
SYMBOL	Borehole	SAMPLE	ELEVATION(m)
•	MO-09	3	97.4

Project Number: 1662333

Checked By: MWK

**Golder Associates**

Date: 01-Feb-18



Ministry of Transportation

Ontario

## PLASTICITY CHART Clayey Silt (Fill)

Figure No. C3

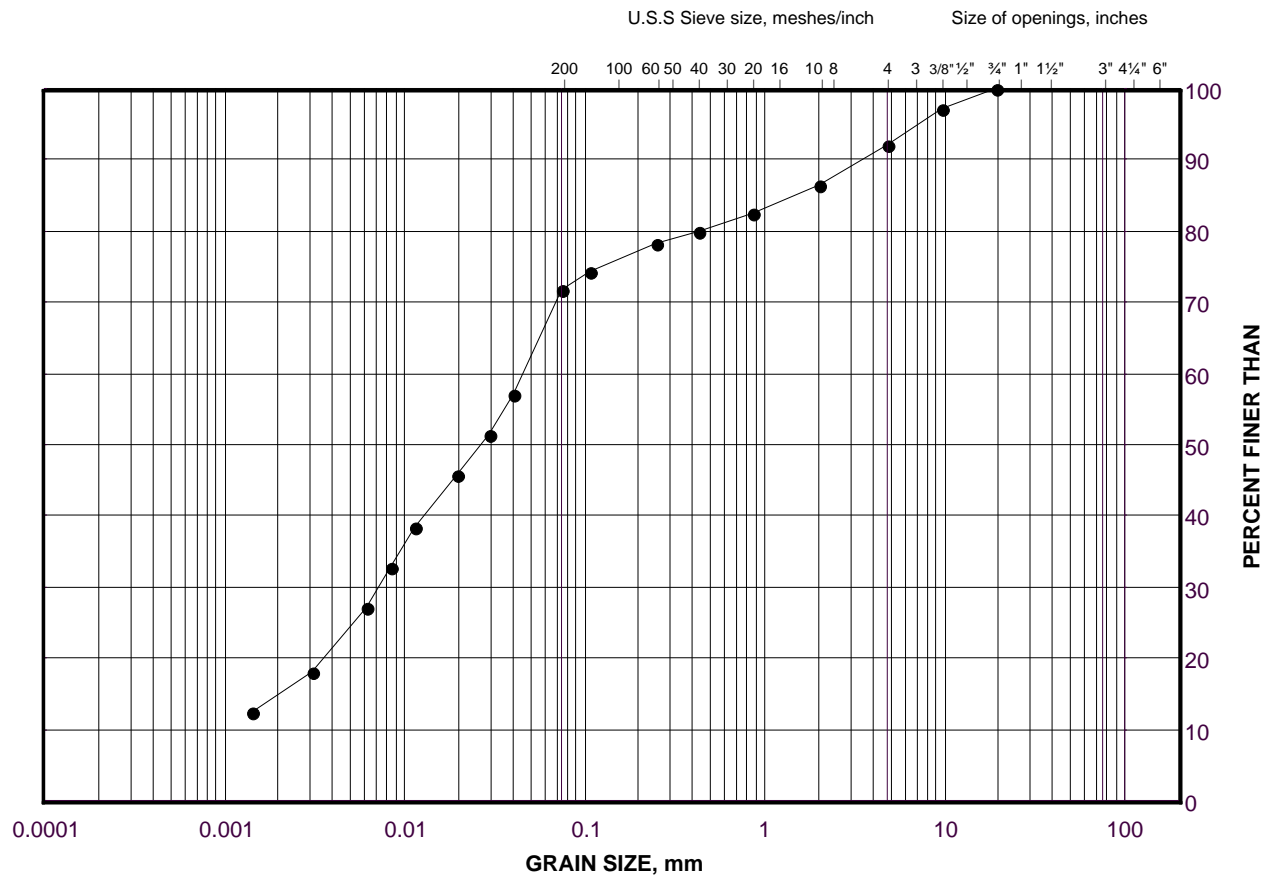
Project No. 1662333

Checked By: MWK

# GRAIN SIZE DISTRIBUTION

Clayey Silt

FIGURE C4



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

## LEGEND

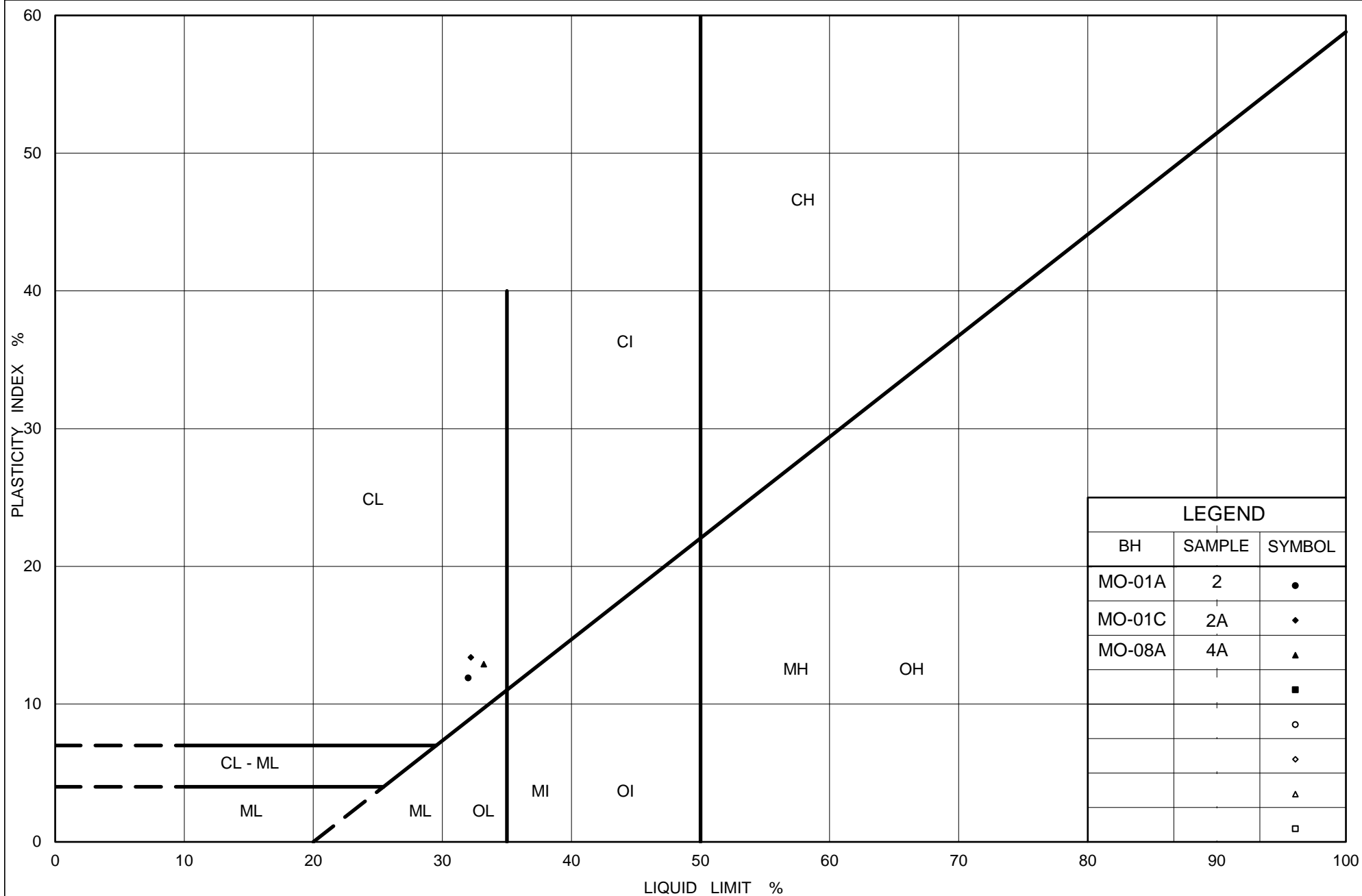
SYMBOL	Borehole	SAMPLE	ELEVATION(m)
•	MO-08A	4A	96.9

Project Number: 1662333

Checked By: MWK

**Golder Associates**

Date: 01-Feb-18



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## PLASTICITY CHART

### Clayey Silt

Figure No. C5

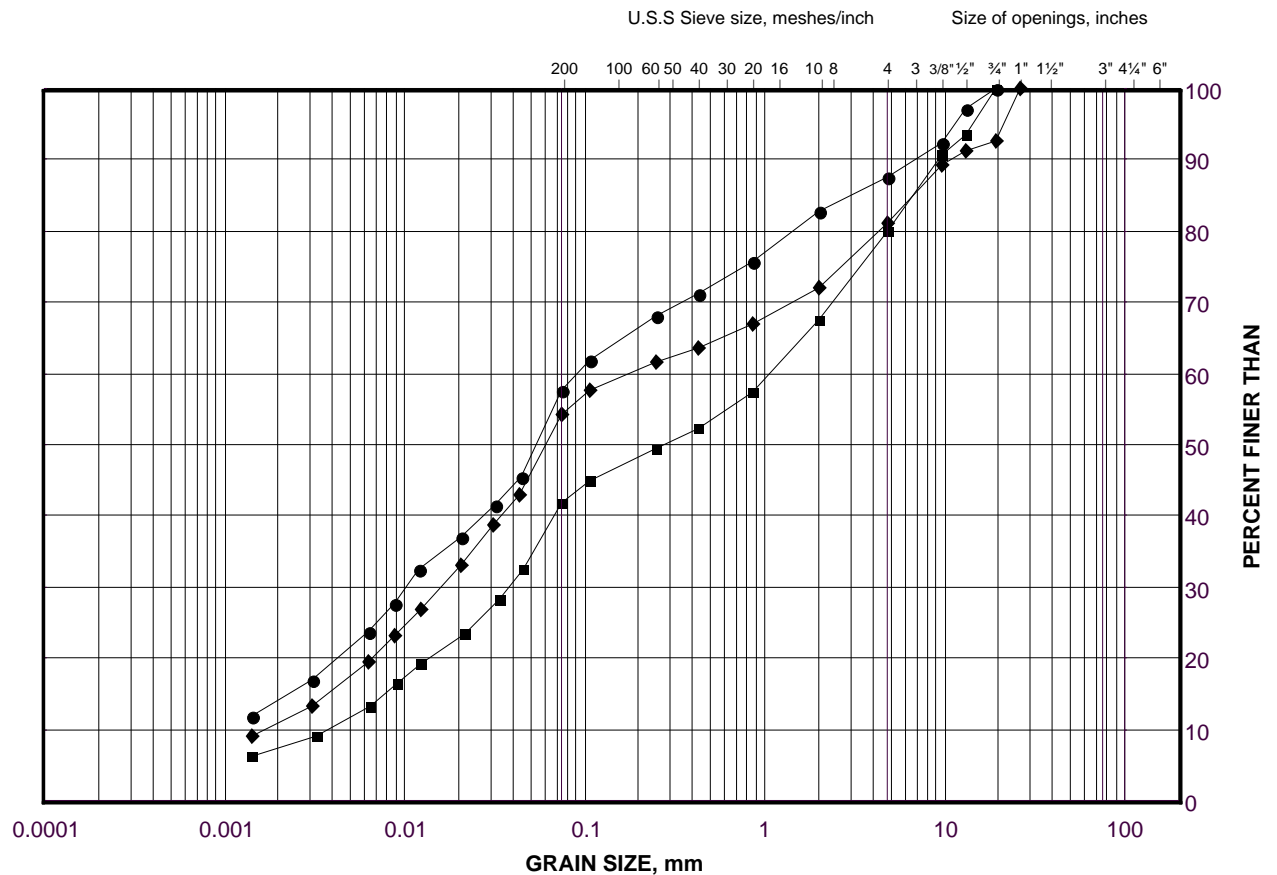
Project No. 1662333

Checked By: MWK

# GRAIN SIZE DISTRIBUTION

Sandy Clayey Silt to Clayey Silt with Sand (Residual Soil)

FIGURE C6



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

## LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	MO-07	3B	97.4
■	MO-04	4	97.1
◆	MO-02	4	97.2

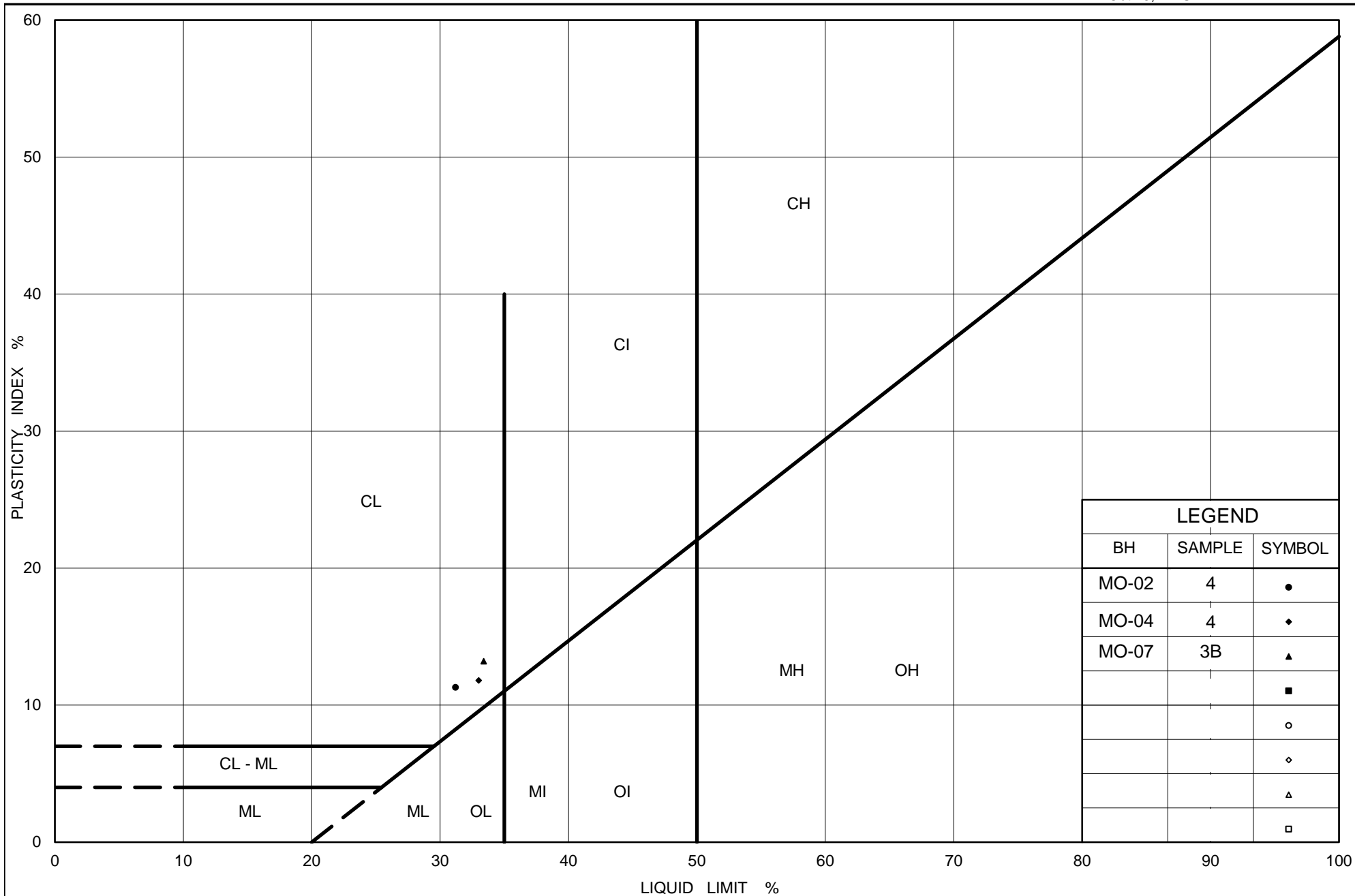
Project Number: 1662333

Checked By: MWK

**Golder Associates**

Date: 01-Feb-18





Ministry of Transportation

Ontario

## PLASTICITY CHART

### Clayey Silt (Residual Soil)

Figure No. C7

Project No. 1662333

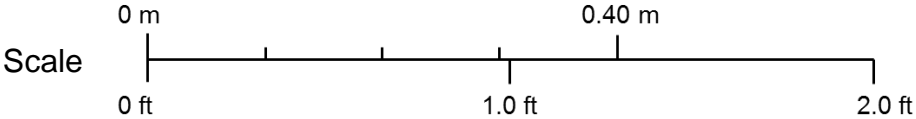
Checked By: MWK


Start of Run No. 1 (1.22 m)



Box 1: 1.22 m to 4.28 m

Start of Run No. 2 (2.76 m)

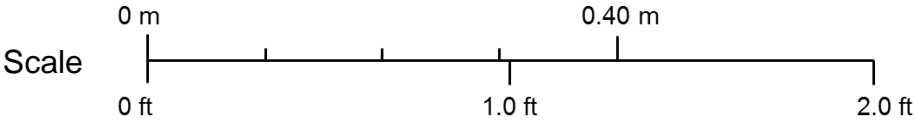



PROJECT					
MTO Assignment 2015-E-0033: Detail Design for the widening/rehab/realignment of QEW Between Mississauga Road and Hurontario Street					
TITLE					
Bedrock Core Photographs Borehole MO-03 (1.22 m to 4.28 m)					
	PROJECT No. 1662333			FILE No. ----	
	DESIGN	MWK	1/31/18	SCALE	NTS
	CADD	--		FIGURE C8	
	CHECK				
	REVIEW	JMAC	2/01/18		
			REV.		

REVISION DATE October 3, 2017 BY: AC Project: 1662333



Box 1: 3.53 m to 6.59 m



PROJECT					
MTO Assignment 2015-E-0033: Detail Design for the widening/rehab/realignment of QEW Between Mississauga Road and Hurontario Street					
TITLE					
Bedrock Core Photographs Borehole MO-04 (3.53 m to 6.59 m)					
	PROJECT No. 1662333			FILE No. ----	
	DESIGN	MWK	1/31/18	SCALE	NTS
	CADD	--		FIGURE C9	
	CHECK				
	REVIEW	JMAC	2/01/18		
			REV.		

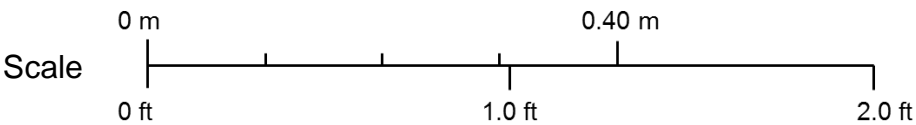
Start of Run No. 1 (3.90 m)

Start of Run No. 2 (4.27 m)



Box 1: 3.90 m to 6.38 m

Start of Run No. 3 (5.80 m)



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

TITLE **Bedrock Core Photographs**  
**Borehole MO-05 (3.90 m to 6.38m)**

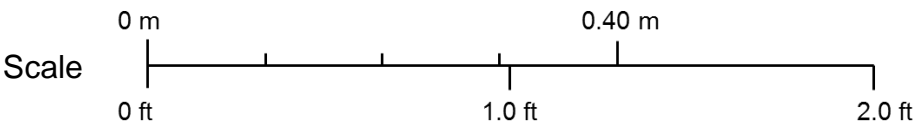


PROJECT No. 1662333			FILE No. ----		
DESIGN	MWK	1/31/18	SCALE	NTS	REV.
CADD	--		<b>FIGURE C10</b>		
CHECK					
REVIEW	JMAC	2/01/18			




Box 1: 1.67 m to 4.71 m

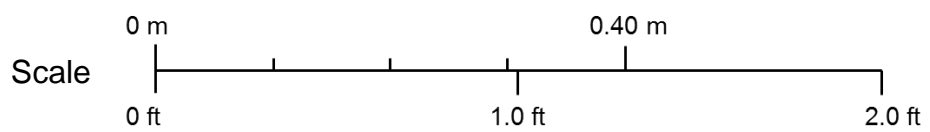
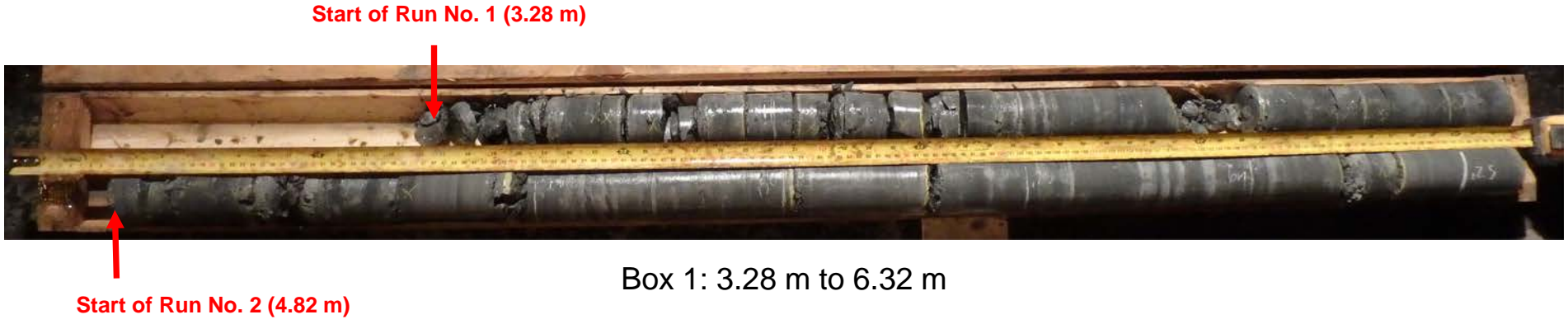
Start of Run No. 3 (4.41 m)




PROJECT	MTO Assignment 2015-E-0033: Detail Design for the widening/rehab/realignment of QEW Between Mississauga Road and Hurontario Street				
TITLE	Bedrock Core Photographs Borehole MO-06 (1.67 m to 4.71m)				

	PROJECT No. 1662333		FILE No. ----		
	DESIGN	MWK	1/31/18	SCALE	NTS
	CADD	--		FIGURE C11	
	CHECK				
	REVIEW	JMAC	2/01/18		

REVISION DATE October 3, 2017 BY: AC Project: 1662333



PROJECT MTO Assignment 2015-E-0033: Detail Design for the widening/rehab/realignment of QEW Between Mississauga Road and Hurontario Street					
TITLE <b>Bedrock Core Photographs</b> <b>Borehole MO-07 (3.28 m to 6.32 m)</b>					
	PROJECT No. 1662333			FILE No. ----	
	DESIGN	MWK	1/31/18	SCALE	NTS
	CADD	--		<b>FIGURE C12</b>	
	CHECK				
	REVIEW	JMAC	2/01/18		
			REV.		



Start of Run No. 1 (1.12 m)

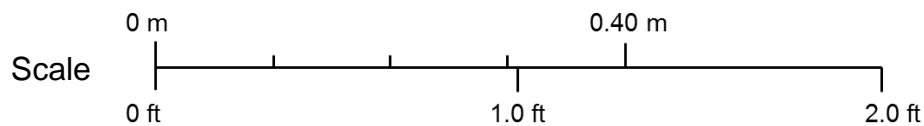
Start of Run No. 2 (1.78 m)



Start of Run No. 3 (3.35 m)

Start of Run No. 4 (4.90 m)

Box 1: 1.12 m to 5.77 m

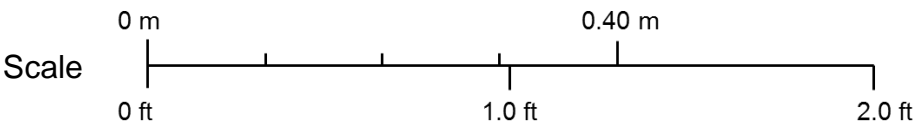
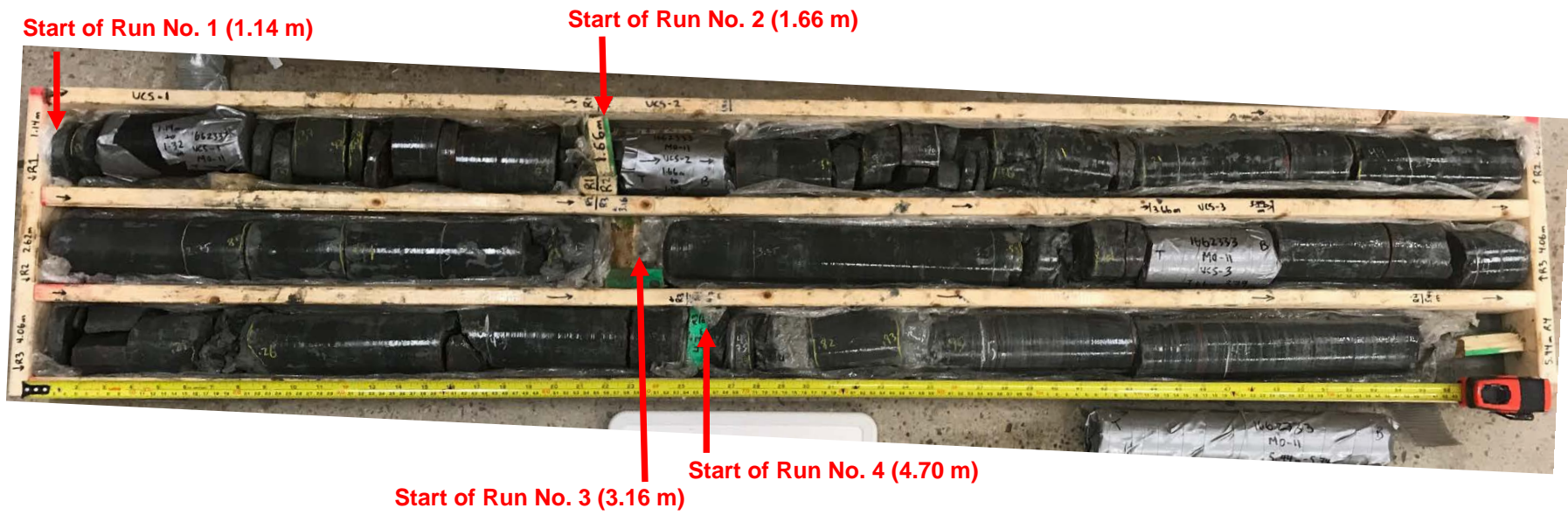



PROJECT MTO Assignment 2015-E-0033: Detail Design for the widening/rehab/realignment of QEW Between Mississauga Road and Hurontario Street

TITLE **Bedrock Core Photographs**  
**Borehole MO-10 (1.12 m to 5.77 m)**

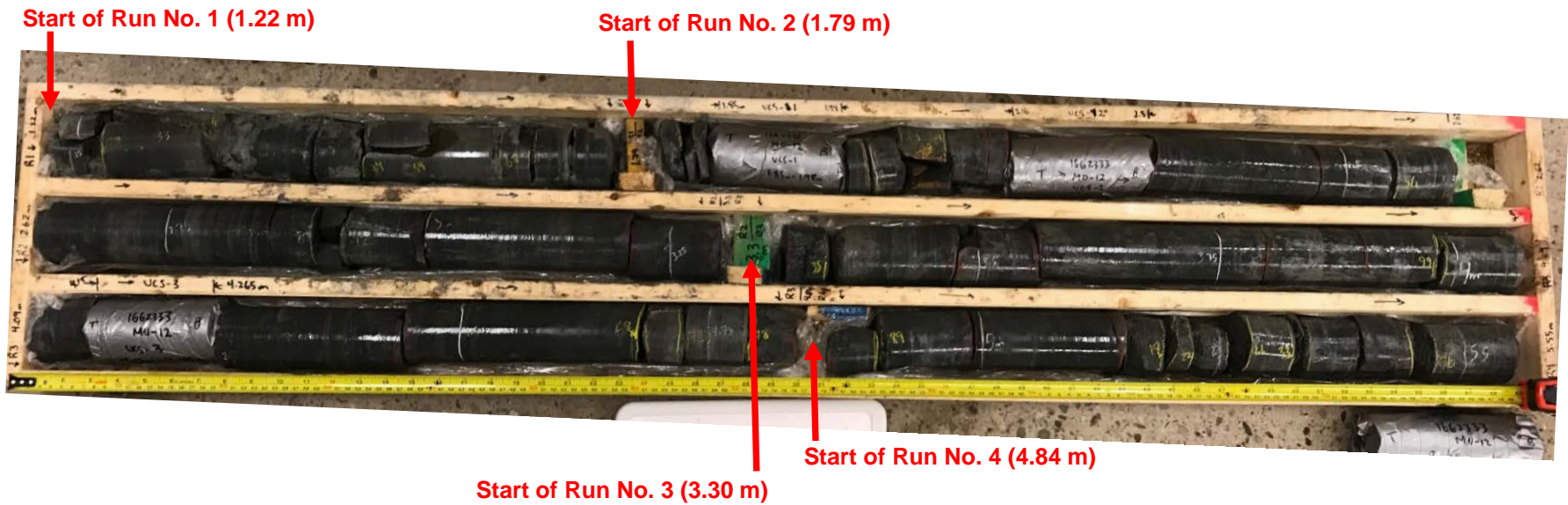


PROJECT No. 1662333			FILE No. ----		
DESIGN	MWK	1/31/18	SCALE	NTS	REV.
CADD	--		<b>FIGURE C13</b>		
CHECK					
REVIEW	JMAC	2/01/18			

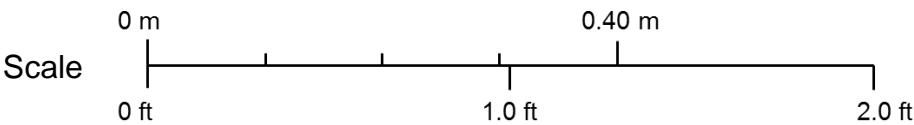



PROJECT MTO Assignment 2015-E-0033: Detail Design for the widening/rehab/realignment of QEW Between Mississauga Road and Hurontario Street					
TITLE Bedrock Core Photographs Borehole MO-11 (1.14 m to 5.74 m)					
	PROJECT No. 1662333			FILE No. ----	
	DESIGN	MWK	1/31/18	SCALE	NTS
	CADD	--		FIGURE C14	
	CHECK				
	REVIEW	JMAC	2/01/18		
			REV.		





Box 1: 1.22 m to 5.72 m



PROJECT					
MTO Assignment 2015-E-0033: Detail Design for the widening/rehab/realignment of QEW Between Mississauga Road and Hurontario Street					
TITLE					
Bedrock Core Photographs Borehole MO-12 (1.22 m to 5.72 m)					
	PROJECT No. 1662333			FILE No. ----	
	DESIGN	MWK	1/31/18	SCALE	NTS
	CADD	--		FIGURE C15	
	CHECK				
	REVIEW	JMAC	2/01/18		
				REV.	

October 23, 2017

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

Re: UCS testing  
(Golder Project No. 166233)

Dear Mr. Zalucki:

On September 25, 2017 three (3) HQ-sized core samples were received by Geomechanica Inc. via dropoff. These samples were identified as being from boreholes drilled as part of Golder project 166233. A uniaxial compressive strength (UCS) specimen was prepared tested from each of these samples (3 tests total).

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

Sincerely,



Giovanni Grasselli Ph.D., P. Eng.

Geomechanica Inc.  
Tel: (647) 478-9767  
Email: [giovanni.grasselli@geomechanica.com](mailto:giovanni.grasselli@geomechanica.com)

# Rock Laboratory Testing Results

**A report submitted to:**

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

**Prepared by:**

Bryan Tatone, PhD  
Omid Mahabadi, PhD  
Giovanni Grasselli, PhD, PEng

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

**October 23, 2017**

Project number: 1662333

**Abstract**

This document summarizes the results of 3 uniaxial compression strength testson 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	1
2	Results	2

## 1 Overview

This report summarizes the results of laboratory testing of 3 uniaxial compression strength testson 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  $\pm 0.025$  mm) and parallel end faces (within  $0.25^\circ$ ).
4. Placement of the specimen into the loading frame, applying a 0.5-1.0 kN axial load, removing the electrical tape, and positioning a radial strain sensor at the mid-height of the specimen.
5. Axial loading to rupture while continuously recording axial force, axial deformation, and radial deformation to determine peak strength (UCS), (tangent) Young's modulus ( $E$ ), and Poisson's ratio ( $\nu$ ).



Figure 1: Uniaxial Compressive Strength (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  $\pm 300$  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 $\rho$ (g/cm <sup>3</sup> )	UCS (MPa)	Young's Modulus $E$ (GPa)	Notes
M05-1	6.08 - 6.16	2.60	39.2	6.53	See <sup>1</sup>
M03-1	3.94 - 4.09	2.58	14.8	2.20	See <sup>2</sup>
M04-1	6.21 - 6.37	2.56	6.4	0.24	See <sup>3</sup>
Mean		2.58	20.1	3.0	
Standard Deviation		0.02	13.9	2.6	

<sup>1</sup> Failure partially along healed fracture, bottom 60 mm of specimen limestone, length:diameter ratio < 2:1.

<sup>2</sup> Specimen very weathered, inter-bedded limestone and shale, length:diameter ratio < 2:1.

<sup>3</sup> Specimen separated along weak plane prior to testing, halves tightly re-fit and tested; specimen emitted water upon loading.

### 2.1 Specimen photographs

Photographs of the specimens prior to and after testing are presented in Figure 3

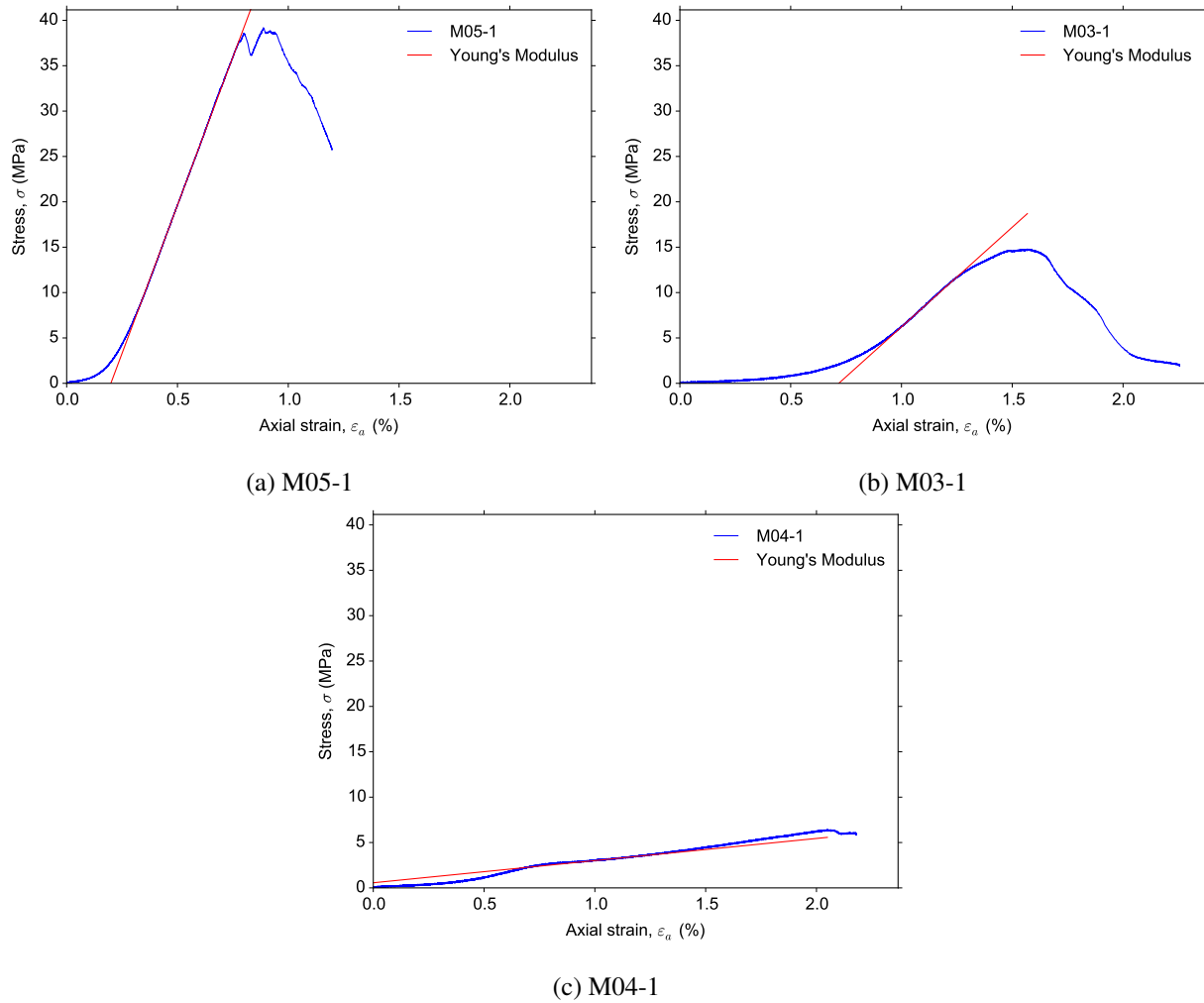


Figure 2: Measured stress-strain curves.

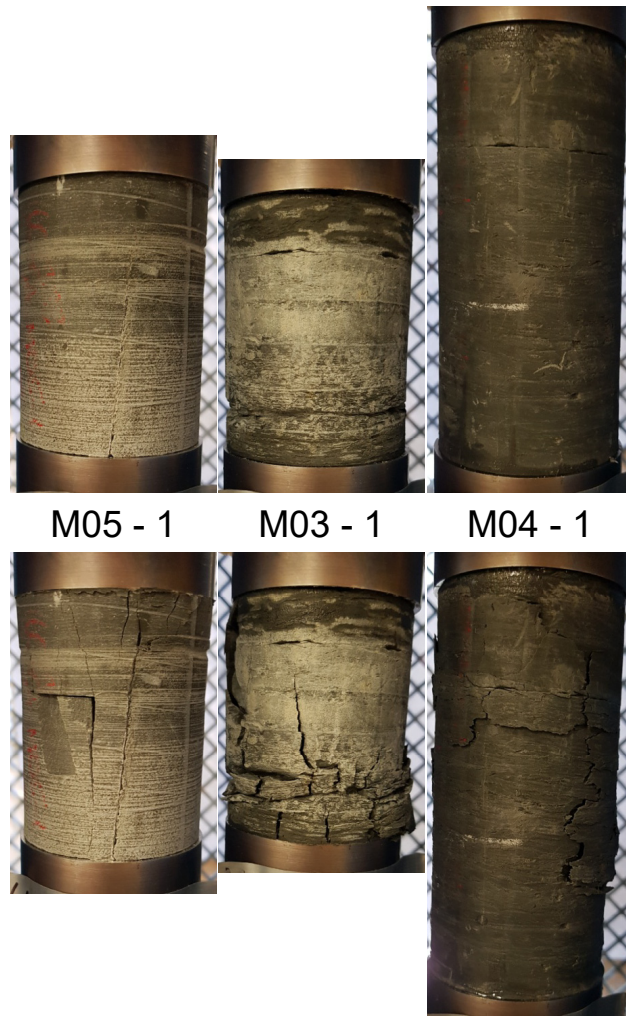


Figure 3: Photographs of specimens prior to testing.



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.

Geomechanica Inc.  
Tel: (647) 478-9767  
Email: [giovanni.grasselli@geomechanica.com](mailto:giovanni.grasselli@geomechanica.com)



# Rock Laboratory Testing Results

**A report submitted to:**

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

**Prepared by:**

Bryan Tatone, PhD  
Omid Mahabadi, PhD  
Giovanni Grasselli, PhD, PEng

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

**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	1
2	Results	2

## 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  $\pm 0.025$  mm) and parallel end faces (within  $0.25^\circ$ ).
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  $\pm 300$  data points on either side of the point representing 50.0% of the peak strength.

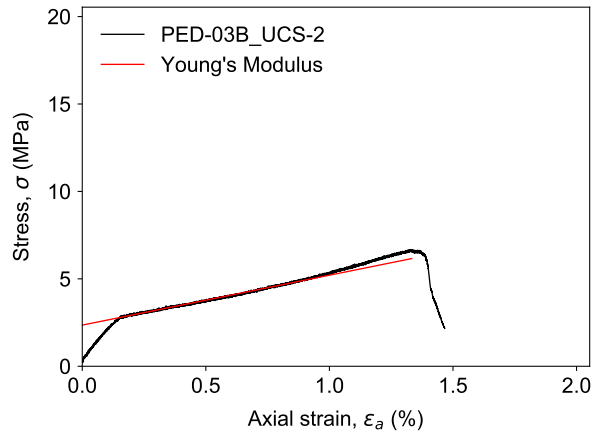
Table 1: Summary of laboratory test results.

Sample	Depth (m)	Bulk density $\rho$ (g/cm <sup>3</sup> )	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	

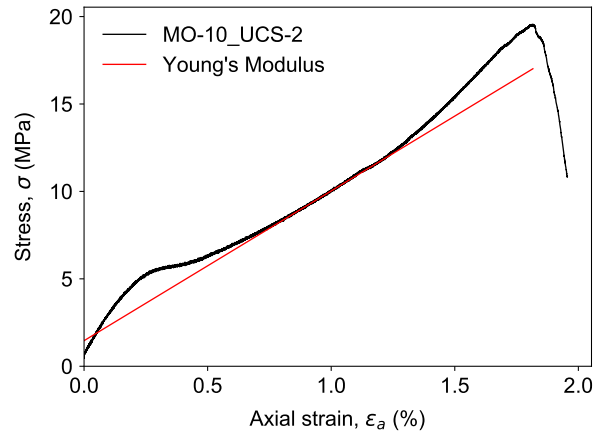
<sup>1</sup> Specimen emitted fresh pore water upon loading  
<sup>2</sup> Length:diameter ratio < 2:1  
<sup>3</sup> Contains limestone layers

### 2.1 Specimen photographs

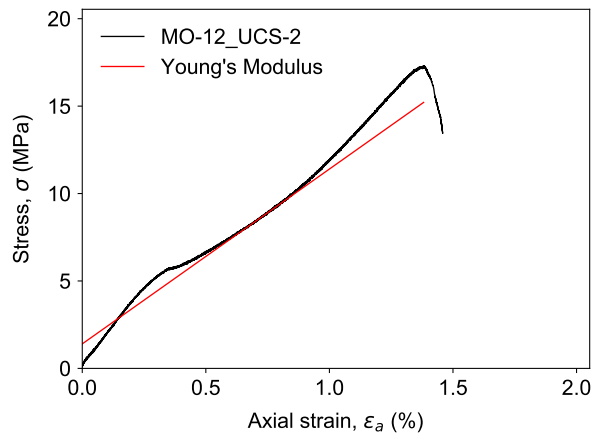
Photographs of the specimens before and after testing are presented in Figure 3.



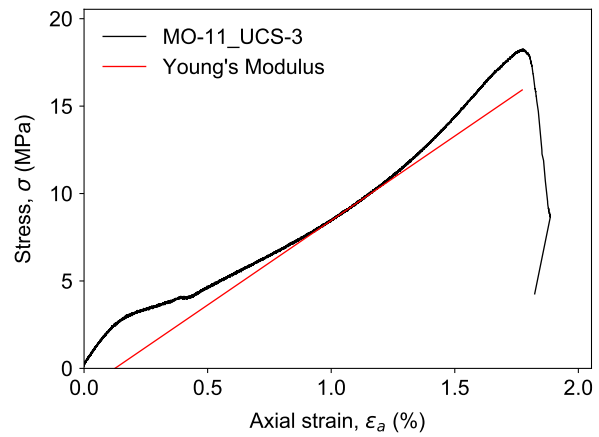
(a) PED-03B, UCS-2



(b) MO-10, UCS-2



(c) MO-12, UCS-2



(d) MO-11, UCS-3

Figure 2: Measured stress-strain curves.

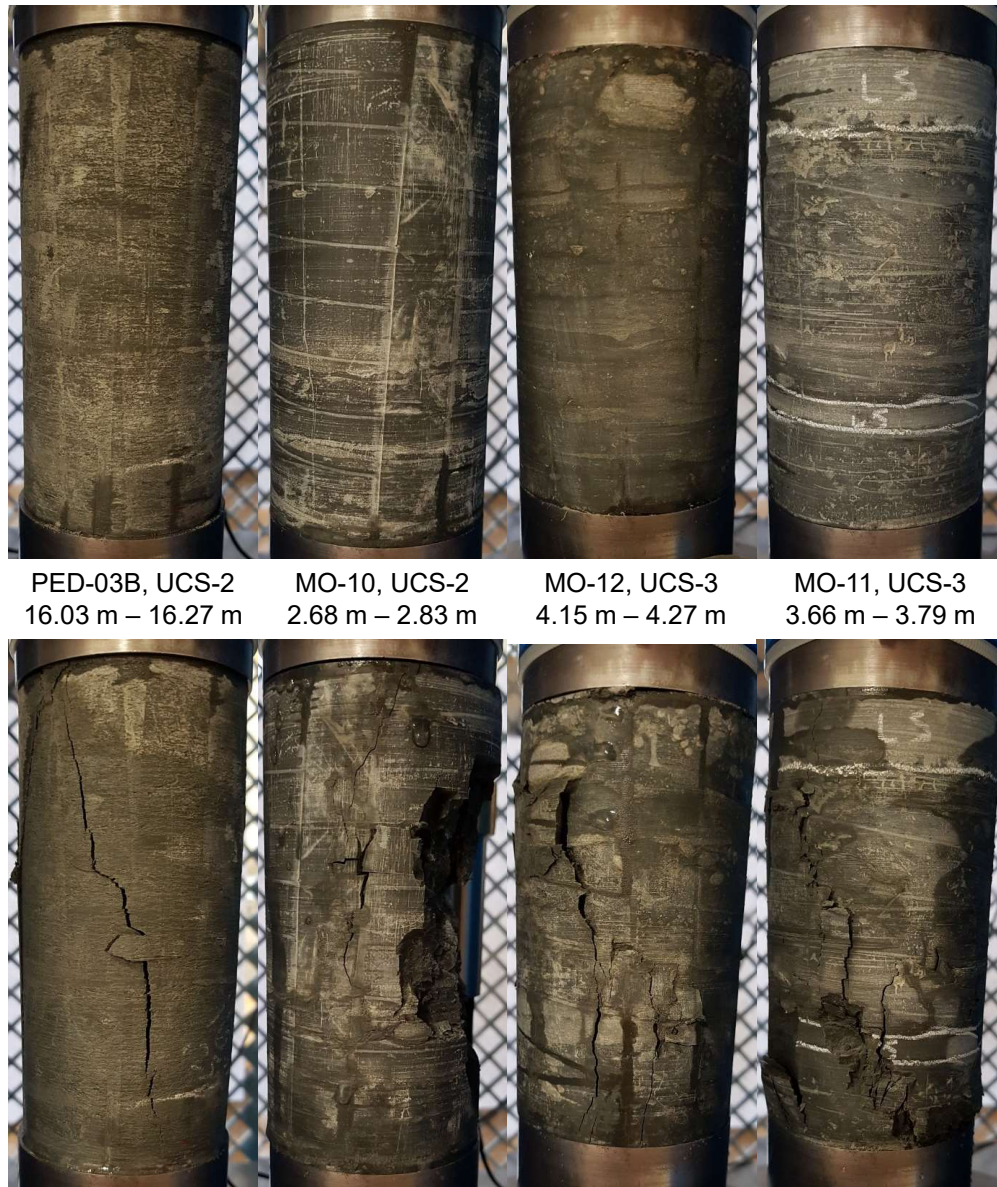


Figure 3: Photographs of specimens prior to testing.

Your Project #: 1662333  
Site Location: MISSISSAUGA RD/QEW  
Your C.O.C. #: 60265

**Attention: Jeremy Lebow**

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

**Report Date: 2018/01/17**  
Report #: R4940243  
Version: 1 - Final

**CERTIFICATE OF ANALYSIS**

**MAXXAM JOB #: B806888**

**Received: 2018/01/11, 09:32**

Sample Matrix: ROCK  
# Samples Received: 2

Analyses	Date		Date Analyzed	Laboratory Method	Reference
	Quantity	Extracted			
Chloride (20:1 extract)	2	N/A	2018/01/17	CAM SOP-00463	EPA 325.2 m
Conductivity	2	N/A	2018/01/16	CAM SOP-00414	OMOE E3530 v1 m
pH CaCl2 EXTRACT	2	2018/01/15	2018/01/15	CAM SOP-00413	EPA 9045 D m
Resistivity of Soil	2	2018/01/11	2018/01/16	CAM SOP-00414	SM 22 2510 m
Sulphate (20:1 Extract)	2	N/A	2018/01/17	CAM SOP-00464	EPA 375.4 m

**Remarks:**

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

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

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

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

Results relate to samples tested.

This Certificate shall not be reproduced except in full, without the written approval of the laboratory.

Reference Method suffix "m" indicates test methods incorporate validated modifications from specific reference methods to improve performance.

\* RPDs calculated using raw data. The rounding of final results may result in the apparent difference.



Your Project #: 1662333  
Site Location: MISSISSAUGA RD/QEW  
Your C.O.C. #: 60265

**Attention: Jeremy Lebow**

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

**Report Date: 2018/01/17**  
Report #: R4940243  
Version: 1 - Final

**CERTIFICATE OF ANALYSIS**

**MAXXAM JOB #: B806888**  
**Received: 2018/01/11, 09:32**

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 ROCK

<b>Maxxam ID</b>		FWV272	FWV273		FWV273		
<b>Sampling Date</b>		2017/12/20	2017/12/19		2017/12/19		
<b>COC Number</b>		60265	60265		60265		
	<b>UNITS</b>	<b>MO-10</b>	<b>MO-11</b>	<b>QC Batch</b>	<b>MO-11 Lab-Dup</b>	<b>RDL</b>	<b>QC Batch</b>
<b>Calculated Parameters</b>							
Resistivity	ohm-cm	1500	1800	5348800			
<b>Inorganics</b>							
Soluble (20:1) Chloride (Cl)	ug/g	120	99	5353048	98	20	5353048
Conductivity	umho/cm	647	566	5354646	569	2	5354646
Available (CaCl <sub>2</sub> ) pH	pH	8.33	8.22	5352490			
Soluble (20:1) Sulphate (SO <sub>4</sub> )	ug/g	130	150	5353049	150	20	5353049
RDL = Reportable Detection Limit							
QC Batch = Quality Control Batch							
Lab-Dup = Laboratory Initiated Duplicate							



Maxxam Job #: B806888  
Report Date: 2018/01/17

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

## TEST SUMMARY

**Maxxam ID:** FWV272  
**Sample ID:** MO-10  
**Matrix:** ROCK

**Collected:** 2017/12/20  
**Shipped:**  
**Received:** 2018/01/11

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	5353048	N/A	2018/01/17	Deonarine Ramnarine
Conductivity	AT	5354646	N/A	2018/01/16	Neil Dassanayake
pH CaCl2 EXTRACT	AT	5352490	2018/01/15	2018/01/15	Tahir Anwar
Resistivity of Soil		5348800	2018/01/16	2018/01/16	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	5353049	N/A	2018/01/17	Alina Dobreanu

**Maxxam ID:** FWV273  
**Sample ID:** MO-11  
**Matrix:** ROCK

**Collected:** 2017/12/19  
**Shipped:**  
**Received:** 2018/01/11

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	5353048	N/A	2018/01/17	Deonarine Ramnarine
Conductivity	AT	5354646	N/A	2018/01/16	Neil Dassanayake
pH CaCl2 EXTRACT	AT	5352490	2018/01/15	2018/01/15	Tahir Anwar
Resistivity of Soil		5348800	2018/01/16	2018/01/16	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	5353049	N/A	2018/01/17	Alina Dobreanu

**Maxxam ID:** FWV273 Dup  
**Sample ID:** MO-11  
**Matrix:** ROCK

**Collected:** 2017/12/19  
**Shipped:**  
**Received:** 2018/01/11

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	5353048	N/A	2018/01/17	Deonarine Ramnarine
Conductivity	AT	5354646	N/A	2018/01/16	Neil Dassanayake
Sulphate (20:1 Extract)	KONE/EC	5353049	N/A	2018/01/17	Alina Dobreanu

### GENERAL COMMENTS

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

Package 1	1.3°C
-----------	-------

**Results relate only to the items tested.**

## QUALITY ASSURANCE REPORT

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

QC Batch	Parameter	Date	Matrix Spike		SPIKED BLANK		Method Blank		RPD	
			% Recovery	QC Limits	% Recovery	QC Limits	Value	UNITS	Value (%)	QC Limits
5352490	Available (CaCl <sub>2</sub> ) pH	2018/01/15			100	97 - 103			1.0	N/A
5353048	Soluble (20:1) Chloride (Cl)	2018/01/17	NC	70 - 130	110	70 - 130	<20	ug/g	0.98	35
5353049	Soluble (20:1) Sulphate (SO <sub>4</sub> )	2018/01/17	NC	70 - 130	109	70 - 130	<20	ug/g	1.4	35
5354646	Conductivity	2018/01/16			100	90 - 110	<2	umho/cm	0.53	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).

*Cristina Carriere*

---

Cristina Carriere, Scientific Service Specialist

---

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

Invoice Information		Report Information (if differs from invoice)		Project Information (where applicable)		Turnaround Time (TAT) Required							
Company Name: <u>Golder Associates Ltd.</u>		Company Name: <u>Golder</u>		Quotation #: _____		<input checked="" type="checkbox"/> Regular TAT (5-7 days) Most analyses							
Contact Name: <u>David Marmor</u>		Contact Name: <u>Jeremy Lebow</u>		P.O. #/ AFE#: _____		PLEASE PROVIDE ADVANCE NOTICE FOR RUSH PROJECTS							
Address: <u>100-6925 Century Ave.</u> <u>Mississauga, ON L5N 7K2</u>		Address: _____		Project #: <u>1062333</u>		Rush TAT (Surcharges will be applied)							
Phone: <u>905-567-4444</u> Fax: <u>905-567-6561</u>		Phone: _____ Fax: _____		Site Location: <u>Mississauga Rd / QEW</u>		<input type="checkbox"/> 1 Day <input type="checkbox"/> 2 Days <input type="checkbox"/> 3-4 Days							
Email: <u>David.Marmor@golder.com</u>		Email: <u>invoice email + jlebow@golder.com</u>		Site #: _____		Date Required: _____							
Sampled By: _____		Sampled By: <u>Jeremy Lebow</u>		Rush Confirmation #: _____									
MOE REGULATED DRINKING WATER OR WATER INTENDED FOR HUMAN CONSUMPTION MUST BE SUBMITTED ON THE MAXXAM DRINKING WATER CHAIN OF CUSTODY													
<b>Regulation 153</b> <input type="checkbox"/> Table 1 <input type="checkbox"/> Res/Park <input type="checkbox"/> Med/ Fine <input type="checkbox"/> Table 2 <input type="checkbox"/> Ind/Comm <input type="checkbox"/> Coarse <input type="checkbox"/> Table 3 <input type="checkbox"/> Agri/ Other <input type="checkbox"/> Table _____ FOR RSC (PLEASE CIRCLE) Y / <u>(N)</u>		<b>Other Regulations</b> <input type="checkbox"/> CCME <input type="checkbox"/> Sanitary Sewer Bylaw <input type="checkbox"/> MISA <input type="checkbox"/> Storm Sewer Bylaw <input type="checkbox"/> PWQO Region _____ <input type="checkbox"/> Other (Specify) _____ <input type="checkbox"/> REG 558 (MIN. 3 DAY TAT REQUIRED)		<b>Analysis Requested</b> REFER TO BACK OF COC REG 153 METALS & INORGANICS REG 153 ICPMS METALS REG 153 METALS (Hg, Cr VI, ICPMS Metals, HWS - B) <u>Cerrosivity Package</u>				<b>LABORATORY USE ONLY</b> CUSTODY SEAL Y / <u>(N)</u> Present <input type="checkbox"/> Intact <input type="checkbox"/> COOLING MEDIA PRESENT: Y / <u>(N)</u> COMMENTS:					
Include Criteria on Certificate of Analysis: <u>(Y)</u> / N SAMPLES MUST BE KEPT COOL (< 10 °C) FROM TIME OF SAMPLING UNTIL DELIVERY TO MAXXAM													
SAMPLE IDENTIFICATION		DATE SAMPLED (YYYY/MM/DD)	TIME SAMPLED (HH:MM)	MATRIX	# OF CONTAINERS SUBMITTED	FIELD FILTERED (CIRCLE) Metals / Hg / CrVI	BTEX / PHC F1	PHCS F2 - F4	VOCs	REG 153 METALS & INORGANICS	REG 153 ICPMS METALS	REG 153 METALS (Hg, Cr VI, ICPMS Metals, HWS - B)	HOLD - DO NOT ANALYZE
1	<u>MO-10</u>	<u>2017/12/20</u>	<u>am</u>	<u>Rock</u>	<u>1</u>	<u>N</u>							<u>X</u>
2	<u>MO-11</u>	<u>2017/12/19</u>	<u>am</u>	<u>Rock</u>	<u>1</u>	<u>N</u>							<u>X</u>
3													
4													
5													
6													
7													
8													
9													
10													
RELINQUISHED BY: (Signature/Print)		DATE: (YYYY/MM/DD)	TIME: (HH:MM)	RECEIVED BY: (Signature/Print)		DATE: (YYYY/MM/DD)	TIME: (HH:MM)						
<u>Jeremy Lebow</u>		<u>2018/01/11</u>	<u>09:30</u>	<u>[Signature]</u>		<u>2018/01/11</u>	<u>09:32</u>						

11-Jan-18 09:32  
 Ema Gitej  
  
**B806888**  
 PS4 ENV-1265



# **APPENDIX D**

## **Non-Standard Special Provisions**



## **FOUNDATIONS ON BEDROCK - Item No.**

---

Non-Standard Special Provision

---

### **1.0 Scope**

Excavations for shallow foundations for the new overpass will extend into the shale bedrock, which is weak and which contains clay seams and medium strong to strong limestone layers at varying depths/elevations. Appropriate equipment and construction procedures will be required to penetrate the overburden and excavate the bedrock to reach the design founding level.

If the concrete for the footings cannot be poured immediately after excavation and inspection, a concrete working slab (100 mm thick having the same composition and compressive strength as the concrete used for the footing) must be placed in the excavation within four hours to protect the integrity of the subgrade.

At the north and/or south limit of the staging, the existing abutment footing(s) must be sawcut as part of the removals to minimize damage to the existing footing and/or underlying bedrock that is supporting the existing bridge. If any bedrock excavation is required adjacent to and/or below a footing that is supporting the bridge, it must be carried using sawcutting or line drilling techniques. During construction, if removal of the bedrock extends to 0.3 m below the adjacent footing supporting the existing overpass structure, then temporary roadway protection systems will be required to support the existing footing and overpass structure. Any over-excavated portions of the bedrock must be replaced with mass concrete, having the same composition and compressive strength as the concrete used for the footing construction.

### **2.0 Basis of Payment**

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



## **CAISSON FOUNDATION – Item No.**

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Non-Standard Special Provision

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### **1.0 Scope**

Where caisson foundations are used for support of the bridge foundation elements, the foundations will extend into the shale bedrock, which is very weak to weak and which contains clay seams and medium strong to strong limestone layers at varying depths/elevations. Appropriate equipment and construction procedures will be required to penetrate the overburden and advance sockets into the bedrock to reach the design founding level.

Immediately following the completion of excavation for each caisson (including the construction of the rock socket) each caisson shall be thoroughly cleaned by airlift or other means such that the water issuing from the caisson on pumping is clean and free from silt and other material. Every reasonable step shall be taken to remove all loose, silty/sandy soil and all cuttings and other materials from the caisson and from the rock socket. Immediately following drilling, cleaning and inspection/approval of the rock socket by a Foundation Engineer (and not longer than 24 hours following completion of socket drilling) the required reinforcement shall be placed and the rock socket shall be immediately concreted using the tremie method.

### **2.0 Basis of Payment**

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

## **DOWELS INTO ROCK - Item No.**

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Non-Standard Special Provision

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### **1.0 Scope**

Where required, the Contractor shall provide dowels into the bedrock at the foundations for the QEW-Mississauga Road replacement structure.

### **2.0 Construction**

Concrete shall be of the same strength as the footing concrete and placed in accordance with OPSS 904. All reinforcing steel supplied shall be in accordance with OPSS.PROV 1440 (dowel bars conforming to CAN/CSA G30.18, Grade 400).

Where dowels are to be placed in rock, hole shall be drilled to the required depth and size. Hole diameter shall be two times the nominal diameter of the dowel. Each hole shall be cleaned out, grouted and the dowel set in place. Grout shall be of the same strength as the footing concrete or at least 25 MPa at 28 days.

If hole contains water, the Contractor shall remove the water, otherwise a tremie procedure shall be used to completely fill the hole with grout. The dowel shall be forced into the hole after the grout has been placed and while it is still fresh.

### **3.0 Rock Dowel Testing**

All proposed testing procedures shall be in general conformance with ASTM D3689-07, ASTM D1143-07 and ASTM D4435-08. Field testing must be carried out in the presence of, and the results reviewed and approved by, the Contract Administrator.

#### **3.01 Performance Tests**

Performance testing shall be carried out at two dowels to confirm that the design load of the rock dowels can be achieved. The Contract Administrator will select the rock dowels to be tested.

Performance test shall be by axial tensioning using a hydraulic jack with a capacity of at least 1.5 times the ultimate strength of the dowels.

Rock dowels shall be loaded and unloaded in 3 cycles and measurements of the displacement of the dowel shall be carried out at each load increment (step) in accordance with the following schedule:

Cycle-Step	1-1	1-2	1-3	2-1	2-2	2-3	2-4
% Design Load	50	75	25	50	75	100	25
Cycle-Step	3-1	3-2	3-3	3-4	3-5		
% Design Load	50	75	100	110	25		

The design load shall be taken as 360 kN for 35M dowels, 252 kN for 30M dowels, 180 kN, for 25M dowels, and 108 kN for 20M dowels.

Displacement measurements shall be carried out at each load increment using calibrated displacement gauges capable of measuring movements of 0.0025 cm. Measurements shall be referenced to an independent fixed referenced pint.

Rock dowels which fail to meet the acceptance criteria shall be replaced at the Contractor's expense and re-tested. If a rock dowel fails, three (3) additional rock dowels shall be tested at the same abutment and pier footing as directed by the Contract Administrator.

Acceptance criteria for the rock dowels will be in accordance with the Post-Tensioning Institute (1985) as follows:

- The dowels are acceptable if the total elastic movement is greater than 80 percent of the theoretical elastic elongation of the free stressing and is less than the theoretical elongation of the free stressing length plus 50 percent of the bond length.

#### **4.0 Basis of Payment**

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

## **VIBRATION MONITORING - Item No.**

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Non-Standard Special Provision

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### **1.0 Scope**

This special provision describes requirements for vibration monitoring during caisson installation works for the construction of the QEW – Mississauga Road bridge structure.

### **2.0 References**

The subsurface conditions at the site are described in the following Foundation Investigation Report for GWP 2002-13-00:

Foundation Investigation and Design 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.

### **3.0 Definitions**

### **4.0 Submission Requirements**

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

- Qualifications of vibration monitoring specialist.
- Details regarding proposed instrumentation.
- Proposed location of instruments.
- Proposed frequency of readings.
- Proposed methods for adjusting piling methods if readings show vibrations exceeding tolerable levels.

### **5.0 Monitoring**

The vibration monitoring equipment shall be placed as close as possible to the piling works. The Contractor/Contractor's Engineer shall take readings on the existing structures during caisson installation, starting with the caisson furthest away from the existing structure.

The vibrations measured at the site shall not exceed 100 mm/s (peak particle velocity).

The results shall be submitted to the Contract Administrator after each caisson has been installed, prior to continuing with the subsequent caisson. As a minimum, the caisson number, location, and drilling log must be submitted with vibration monitoring results.

If the vibration monitoring results are acceptable, the Contractor may continue with the next caisson(s) with readings taken during installation of each caisson. The results of subsequent caissons should be submitted to the Contract Administrator after each caisson has been installed.

If the readings are not within the limits stated above, the Contractor must alter the procedures until the vibrations at the existing structures are within acceptable levels. The above process must be repeated for each caisson.

## **6.0 Basis of Payment**

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

**END OF SECTION**

## **DENTAL CONCRETE - Item No.**

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Non-Standard Special Provision

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Special Provision

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### **1.0 Scope**

This Special Provision covers the requirements for the supply and placement of dental concrete for the QEW Mississauga Road Overpass structure replacement.

### **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**

Dental concrete should have the same composition and compressive strengths as the concrete used for the footing construction.

### **6.0 EQUIPMENT - Not Used**

### **7.0 CONSTRUCTION**

#### **7.01 Excavation**

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

Where bedrock excavation results in the creation of steps or troughs in the bedrock, the surface of the founding stratum could be levelled by the use of dental or mass concrete. In this case the dental or mass concrete should be of the same composition and compressive strength as the concrete used for the footing construction.

**8.0      Quality Assurance - Not Used**

**9.0      Measurement for Payment - Not Used**

**10.0     Basis of Payment**

**10.01   Dental Concrete - 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

## **WORKING SLAB - Item No.**

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Non-Standard Special Provision

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Special Provision

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### **1.0 Scope**

This Special Provision covers the requirements for the supply and placement of a concrete working slab under foundations for the QEW Mississauga Road Bridge replacement structure.

### **2.0 References**

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

#### **Ontario Provincial Standard Specifications, Construction**

SP109S12      Amendment to OPSS 902, 2010 Excavating and Backfilling - Structures  
NSSP FOUN0003

### **3.0 Definitions - Not Used**

### **4.0 Design and Submission Requirements - Not Used**

### **5.0 Materials**

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

### **6.0 EQUIPMENT - Not Used**

### **7.0 CONSTRUCTION**

#### **7.01 Excavation**

Excavation for the working slab shall be according to OPSS 902 as amended by Special Provision SP109S12 and NSSP FOUN0003.

#### **7.03 Protection of Founding Bedrock**

The surface of the footing founding rock 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 concrete, having the same composition and compressive strength as the concrete used for the footing construction. If the concrete for the footings cannot be poured immediately after excavation and inspection, a concrete working slab must be placed in the excavation within four hours to protect the integrity of the subgrade. Thickness of the mass concrete pad shall depend on the slope and irregularities in the exposed founding rock surface. A nominal thickness and a footprint plan view area has been specified on the Contract Documents.



#### **7.04            Dewatering**

Dewatering shall be carried out according to OPSS 902 as amended by Special Provision SP109S12 and NSSP FOUN0003.

#### **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

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