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REVISED FOUNDATION INVESTIGATION AND DESIGN REPORT

STORMWATER MANAGEMENT POND QEW WIDENING FROM WEST OF MISSISSAUGA ROAD TO WEST OF HURONTARIO STREET CITY OF MISSISSAUGA MINISTRY OF TRANSPORTATION, ONTARIO ASSIGNMENT NO. 2015-E-0033, G.W.P. 2002-13-00

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REPORT

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PART A

FOUNDATION INVESTIGATION REPORT
STORMWATER MANAGEMENT (SWM) POND
QEW WIDENING FROM WEST OF MISSISSAUGA ROAD TO WEST OF HURONTARIO STREET
CITY OF MISSISSAUGA, REGION OF PEEL
MINISTRY OF TRANSPORTATION, ONTARIO
ASSIGNMENT NO. 2015-E-0033, G.W.P. 2002-13-00

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by Morrison Hershfield Limited (MH) on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services in support of the detail design for 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.

The purpose of this investigation is to establish the subsurface soil, bedrock and groundwater conditions at the location of the proposed Stormwater Management (SWM) Pond by borehole drilling and laboratory testing on selected soil 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 (Number 2015-E-0033) for this project. The work has been carried out in accordance with Golder's Supplementary Specialty Plan for foundation engineering services for this project, dated February 3, 2017.

2.0 SITE DESCRIPTION

The existing QEW-Mississauga Road overpass is located approximately 2.0 km west of the QEW-Hurontario Street interchange, in the City of Mississauga. The QEW alignment in the project area is oriented generally in a southwest-northeast direction; for the purposes of this report, the QEW alignment is described as being in an east-west orientation.

The current ground surface in the vicinity of the interchange is grass covered with some trees and is at between about Elevations 97 m and 101 m. In the immediate area of the SWM Pond the ground surface grade is between about Elevation 99.4 m and 100.5 m, sloping down towards the southeast.

Land use to the south of the interchange is primarily residential, and a golf course is located immediately to the north of the interchange and northeast of Mississauga Road.

3.0 INVESTIGATION PROCEDURES

Field work for the foundation investigation was carried out on August 14 and 16, 2017, during which time a total of four sampled boreholes (designated as Boreholes SWMW-01 to SWMW-04) were advanced within the outline of the proposed SWM Pond. The location of the boreholes are shown on Drawing 1 and the Records of Boreholes and Drillholes are included in Appendix A.

The field borehole investigation was carried out using a track-mounted CME 55 drill rig, supplied and operated by Aardvark Drilling Inc. of Guelph, Ontario. The boreholes were advanced using 150 mm or 108 mm outside diameter solid-stem augers through the overburden, and NW casing and an NQ core barrel through the bedrock in two of the boreholes. Soil and weathered bedrock samples were obtained at 0.75 m intervals of depth, using a 50 mm outer diameter split-spoon sampler driven by an automatic hammer in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586-08)[1].

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

The boreholes were either advanced to auger and/or sampler refusal (i.e. on inferred bedrock) or cored into bedrock, to depths ranging from about 2.6 m to 7.7 m below existing ground surface. Samples of the bedrock were obtained using an 'NQ'-size rock core barrel and coring techniques in Boreholes SWMW-03 and SWMW-04. Photographs of the recovered bedrock core samples are provided in Appendix B.

The groundwater conditions and water levels in the open boreholes were observed during and immediately following drilling operations. A standpipe piezometer was installed in Borehole SWMW-04 to permit monitoring of the groundwater level at the borehole location. The standpipe piezometer consists of 50 mm diameter PVC pipe, with a slotted screen within a sand filter pack sealed within the bedrock. Above the sand filter pack and piezometer screen, the annulus surrounding the piezometer pipe was backfilled to the ground surface with bentonite pellets. The piezometer installation details and water level readings are shown on the borehole records contained in Appendix A. All remaining boreholes were backfilled with bentonite upon completion, in accordance with Ontario Regulation 903 (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 the soil and bedrock samples. The samples were identified in the field, placed in appropriate containers, labelled and transported to Golder's Mississauga geotechnical laboratory where the samples underwent further visual examination and laboratory testing. All of the laboratory tests were carried out to MTO and/or ASTM Standards, as appropriate. Classification testing (water content, Atterberg limits and grain size distribution) was carried out on selected soil samples. The results of the geotechnical laboratory testing are included in Appendix B.

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

Borehole No.	Location (MTM NAD 83)		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing (m) (Latitude, °)	**Easting (m) (Longitude, °)**		
SWMW-01	4,823,494.9 (43.551320)	295,272.4 (-79.617900)	100.5	2.6
SWMW-02	4,823,554.0 (43.551850)	295,325.1 (-79.617300)	100.3	2.6
SWMW-03	4,823,513.7 (43.551490)	295,424.7 (-79.616000)	99.4	6.9*
SWMW-04	4,823,507.5 (43.551430)	295,332.3 (-79.617200)	99.8	7.7*

* includes bedrock core of 4.5 m length in Boreholes SWMW-03 and SWMW-04.

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

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

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

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

4.2 Subsurface Conditions

The detailed subsurface soil, bedrock and groundwater conditions as encountered in the boreholes and the results of the laboratory tests carried out on selected soil and bedrock core samples are presented on the Records of Boreholes and Drillholes provided in Appendix A. The results of the in-situ field tests (i.e. SPT “N” values) as presented on the borehole records and in sub-sections of Section 4.2 are uncorrected. The geotechnical laboratory testing results and test data are contained in Appendix B.

The stratigraphic boundaries shown on the borehole records and the stratigraphic cross-section on Drawing 1 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 and soil/bedrock 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 borehole and drillhole records govern any interpretation of the site conditions. It should be noted that the interpreted stratigraphy shown on Drawing 1 is a simplification of the subsurface conditions.

In general, the subsurface conditions in the area of the proposed SWM Pond consist of a layer of topsoil, underlain by a deposit of sand at one borehole location, or by a deposit of silty clay to sandy silty clay, which are all in turn underlain by residual soil consisting of sandy clayey silt. The native soil deposits are underlain by shale bedrock. A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Topsoil

A 80 mm to 200 mm thick layer of topsoil was encountered at the ground surface in all of the boreholes.

4.2.2 Sand

Underlying the topsoil in Borehole SWMW-02 a deposit of sand was encountered. The surface of this non-cohesive deposit was encountered at a depth of about 0.2 m (about Elevation 100.1 m) below ground surface and extends to about 0.8 m (Elevation 99.5 m) below ground surface.

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

The single SPT “N”-value measured within the granular deposit is 5 blows per 0.3 m of penetration, indicating a loose compactness condition.

The water content measured on a sample of the sand deposit is about 7 per cent.

4.2.3 Silty Clay to Sandy Silty Clay

Underlying the topsoil in Boreholes SWMW-01, SWMW-03 and SWMW-04 a deposit of sandy silty clay to silty clay, some sand, was encountered. The surface of the silty clay deposit was encountered at depths of between about 0.1 m and 0.2 m below ground surface (between about Elevation 100.3 and 99.3 m) and extends to between about 0.7 m and 1.4 m below ground surface (between Elevation 99.1 m and 98.7 m). The thickness of this deposit varies from about 0.5 m to 1.2 m.

SPT “N”-values measured within the silty clay deposit are between 9 blows and 64 blows per 0.3 m of penetration, suggesting a stiff to hard consistency.

The results of grain size distribution tests completed on two selected samples of the silty clay deposit are shown on Figure B1 in Appendix B.

Atterberg limits tests were carried out on two samples of the silty clay deposit and measured liquid limits of about 36 per cent and 45 per cent, plastic limits of about 20 per cent and 21 per cent, and plasticity indices of about 16 per cent and 24 per cent. These test results, which are plotted on a plasticity chart on Figure B2 in Appendix B, indicate that the deposit can be classified as a silty clay of medium plasticity.

The natural water content measured on samples of the silty clay deposit ranges between 9 per cent and 16 per cent.

4.2.4 Clayey Silt (Residual Soil)

Underlying the sand deposit in Borehole SWMW-02, and the silty clay deposit in Boreholes SWMW-01, SWMW-03 and SWMW-04, a clayey silt (residual soil) was encountered at depths between about 0.7 m and 1.4 m below ground surface (between Elevations 99.5 m and 98.7 m). The base of the residual soil was encountered at depths of between about 2.2 m and 2.5 m below ground surface (between Elevation 98.0 m and 97.2 m). This deposit is interpreted to be derived from weathering of the underlying shale bedrock, and consists of sandy clayey silt trace to some gravel, containing varying amounts of shale and limestone fragments.

SPT “N”-values measured within the residual soil are between 18 blows and 93 blows per 0.3 m of penetration, suggesting a very stiff to hard consistency.

The results of a grain size distribution test completed on a selected sample of the residual soil are shown on Figure B3 in Appendix B.

Atterberg limits tests were carried out on two samples of the residual soil and measured liquid limits of about of 32 per cent and 34 per cent, plastic limits of about 20 per cent, and plasticity indices of about 12 per cent and 14 per cent. These test results, which are plotted on a plasticity chart on Figure B4 in Appendix B, indicate that the fines portion of the residual soil can be classified as a clayey silt of low plasticity.

The natural water content measured on samples of the residual soils ranges between 7 per cent and 16 per cent.

4.2.5 Shale Bedrock

Bedrock was encountered and confirmed by split-spoon sampling in Boreholes SWMW-01 and SWMW-02, and bedrock core samples were obtained in Boreholes SWMW-03 and SWMW-04. The depths to bedrock below ground surface, and the corresponding bedrock surface elevation are summarized below.

Borehole	Depth to Bedrock Surface / Refusal (m)	Bedrock Surface / Refusal Elevation (m)	Comments
SWMW-01	2.5	98.0	Split-Spoon Sample
SWMW-02	2.5	97.8	Split-Spoon Sample
SWMW-03	2.2	97.2	Bedrock Cored 4.5 m
SWMW-04	2.2	97.6	Bedrock Cored 4.5 m

In general, the bedrock surface as encountered or inferred in the area of the proposed stormwater management pond is relatively horizontal to gently sloping towards the south.

Based on a review of the bedrock core samples, the bedrock consists of shale of the Georgian Bay Formation. In general, the bedrock samples are described as slightly weathered, thinly laminated to bedded, fine grained, slightly porous, weak, grey, with medium strong to strong limestone interbeds at varying intervals, as presented in the drillhole records in Appendix A, and shown on the photographs of the recovered core samples on Figures B5 and B6 in Appendix B. The degree of weathering of the bedrock samples (i.e. slightly weathered –W2), and the strength classification of the intact rock mass based on field identification (i.e. weak – R2) are described in accordance with the International Society for Rock Mechanics (ISRM[3]) standard classification system.

The Rock Quality Designation (RQD) measured on the core samples is between 67 per cent and 100 per cent, indicating a rock mass of generally fair to excellent quality, as per Table 3.10 of CFEM (2006)[4]. The Total Core Recovery (TCR) and Solid Core Recovery (SCR) of samples recovered are 100 per cent and between 95 per cent and 100 per cent, respectively.

4.2.6 Groundwater Conditions

The overburden samples obtained from the boreholes were generally moist. The open boreholes were observed to be dry upon completion of drilling; however, these observations are not necessarily representative of the stabilized groundwater level at the site. A standpipe piezometer was installed in Borehole SWMW-04, sealed within the shale bedrock, and the recorded water level is summarized below:

Borehole	Stratum Sealed Into	Depth to Water Level (m)	Water Level Elevation (m)	Date
SWMW-04	Bedrock	2.4	97.4	November 28, 2017

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

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

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.

5.0 CLOSURE

This report was prepared by Mr. Matthew Kelly, P.Eng., a Geotechnical Engineer with Golder. Ms. Sandra McGaghran, M.Eng., P.Eng., a Geotechnical Engineer and Associate with Golder, and Mr. Paul Dittrich, P.Eng., a Principal with Golder reviewed the technical aspects of the report. Mr. Jorge Costa, P.Eng., MTO Foundations Designated Contact for Golder and Senior Consultant, conducted a quality control audit of the report.

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PART B

FOUNDATION DESIGN REPORT
STORMWATER MANAGEMENT (SWM) POND
QEW WIDENING FROM WEST OF MISSISSAUGA ROAD TO WEST OF HURONTARIO STREET
CITY OF MISSISSAUGA, REGION OF PEEL
MINISTRY OF TRANSPORTATION, ONTARIO
ASSIGNMENT NO. 2015-E-0033, G.W.P. 2002-13-00

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides geotechnical recommendations for the design of the proposed Storm Water Management (SWM) Pond in the northwest quadrant of the QEW / Mississauga Road interchange. These recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the current subsurface investigation at the proposed SWM Pond location. The discussion and recommendations contained in this report are intended to provide the designers with sufficient information to complete the detail design of the proposed SWM Pond. 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 undertaking the work 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 aspects of construction should make their own interpretation of the factual information provided as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

The following summarizes the proposed SWM Pond design elements based on the Proposed Wet Pond Plan and Sections drawings provided to us by Morrison Hershfield Limited (MH) on December 14, 2018 and the subsurface conditions encountered in Boreholes SWMW-01 to SWMW-04:

Design Pond Base Elevation (m)	Design Top of Pond Elevation (m)	Approximate Excavation / Cut Depth (m)	Approximate Excavation / Cut Depth into Shale Bedrock (m)
96.0	100.0	3.4 to 4.5	1.2 to 2.0

6.2 Pond Base Stability – Construction and Maintenance Conditions

The design groundwater level indicated below has been considered in developing the design recommendations for the proposed SWM Pond. This groundwater level is considered reasonable based on the groundwater level measured in the piezometer installed in Borehole SWMW-04 and the potential for higher groundwater levels in the spring or during/following periods of heavy precipitation and infiltration. The factor of safety against base instability is greater than 1.5 based on the estimated strength properties of the shale bedrock, and the design groundwater level and the proposed pond base elevation, during both the short-term construction period, and long-term during maintenance conditions. It is however noted that there may be localized softening / loosening of the pond bottom and sloughing of the lower portion of the pond banks during construction.

SWM Pond	Pond Base Elevation (m)	Design Groundwater Elevation (m)	Groundwater Level Relative to Pond Base	Potential for Base Instability
SMW Pond	96.0	97.4	1.4 m above	No

As the design groundwater level is approximately 1.4 m above the design pond base elevation it is anticipated that groundwater seepage will occur from the weathered zone/fractured shale bedrock into the excavation during construction. Additional seepage should also be anticipated during construction where "perched" groundwater may be encountered in the near-surface sand deposits overlying the cohesive deposits.

6.3 Permanent Pool Design and Pond Liner Considerations

If site grading and stormwater storage requirements permit, it is recommended that the permanent pool level (i.e., operating water level in the SWM Pond) be designed to be close to the groundwater level, as given in Section 6.2, to minimize inflow of groundwater or recharge of the groundwater regime. It is understood that the permanent pool design operating level is proposed to be about Elevation 97.4 m (i.e. approximately equivalent with the measured groundwater level), which will require a water control system to actively discharge pond water during precipitation events and runoff inflow, as well as to minimize accumulation of groundwater inflow if the groundwater level rises above the level measured in the piezometer in Borehole SWMW-04.

If the operating water level in the pond is set to below the groundwater level, there will be some net groundwater inflow to the SWM Pond, resulting in drawdown of the groundwater table in the immediate vicinity of the pond; however, this drawdown will be localized and is not expected to impact the performance of adjacent green space, roadways or utilities.

For the currently proposed operating condition (i.e. operating water level to be approximately equal to or lower than the groundwater level), a pond liner (geosynthetic or compacted clay) is not recommended for this site because incorporation of a liner into the system would also require a ballast layer over the liner which could otherwise be uplifted due to differences in head between the higher groundwater level and the lower water level in the pond. A liner/ballast to counterbalance the upward groundwater pressures would also require excavations into the shale bedrock to maintain the pond bottom elevation at the design level.

However, if the operating pond level is to be maintained higher than the groundwater level, a pond liner is recommended over the exposed sections of bedrock, as the pond base will be formed within the near surface, fractured/weathered zone of the shale bedrock potentially resulting in higher exfiltration (i.e. outward seepage) of storm water.

6.3.1 Compacted Clay Liner

If the operating pond level is to be maintained higher than the groundwater level, a compacted natural clay liner, or a geosynthetic clay liner (GCL) is recommended over the exposed bedrock on the base and side slopes of the pond. The natural clay soil material for the pond liner should have a minimum clay content of 15 per cent, and a plasticity index greater than 10 per cent. The clay liner should consist of a minimum thickness of at least 450 mm of compacted clay, placed in three equal thickness loose lifts and compacted to at least 95 per cent of the material's Standard Proctor maximum dry density. A Non-Standard Special Provision to address the supply and placement of the compacted clay liner is provided in Appendix C, for inclusion in the Contract Documents.

The clay liner must be covered with ballast fill of sufficient thickness to resist hydrostatic uplift forces. Based on a design pond base level at Elevation 96.0 m and a groundwater level at Elevation 97.4 m, the 0.45 m thick natural clay liner or GCL for the bottom of the pond should be constructed with its base at Elevation 94.3 m (1.7 m below the design pond base level); the 450 mm thick natural clay liner or GCL should then be covered with ballast fill up to the design pond base at Elevation 96.0 m. The critical condition (lowest factor of safety) will occur when the pond is dry (i.e. unwatered for maintenance purposes), at which time, for this proposed liner and ballast fill geometry, the factor of safety against uplift forces on the base of the liner would be approximately 1.1 relative to the groundwater level at the site.

The ballast fill could consist of OPSS 1010 Granular “B” Type I; or alternatively, the ballast fill could consist of the shale that is excavated during the pond construction, crushed to 150 mm minus sizes. The ballast fill should be placed and nominally compacted by the construction equipment.

The clay liner should extend up the side slopes of the pond to at least the maximum operating water level during the design storm event, in order to control surface water exfiltration into the soil above the groundwater level. The clay liner thickness should be maintained at 450 mm on the pond side slopes; however, the thickness of the ballast fill can be reduced on the side slopes above the pond base by interpolating linearly between that thickness required to resist the full hydrostatic pressure at the base of the pond (1.7 m thickness of combined liner and ballast fill at Elevation 96.0 m), decreasing to a minimum thickness of 300 mm of ballast fill the elevation of the maximum design storm operating water level to provide protection to the clay liner from desiccation and possible damage during periodic pond clean-out operations.

The pond side slopes should be formed no steeper than 3H:1V in order to allow operation of the construction equipment for placement and compaction of the clay liner and ballast fill. It should be noted that for safety considerations, some municipalities stipulate that the perimeter slopes be benched at 7H:1V over the length of the slope from 1 m below to 1 m above the operating pond level.

6.4 Global Stability of Pond Cut Slopes

We understand that the SWM Pond perimeter cut slopes are proposed to be constructed at a 4 horizontal to 1 vertical (4H:1V) inclination. Slope stability analyses have been performed using the commercially available program *SLIDE*, developed by Rocscience Inc., at critical sections to check that the cut slopes have a global factor of safety under static conditions equal to or greater than 1.5. This minimum factor of safety is considered appropriate for the proposed SWM Pond side slopes on this project, considering the design requirements and the available field and laboratory testing data.

The following parameters have been used in the static global stability analyses, based on field and laboratory test data as well as accepted correlations (CHBDC, 2006; Bowles, 1984; and Kulhawy and Mayne, 1990):

Soil Deposit	Bulk Unit Weight (kN/m^3)	Effective Friction Angle	Undrained Shear Strength (kPa)
Silty clay deposit	20	28°	50
Clayey silt (residual soil)	21	32°	100
Shale bedrock	23	40°	-

For the normal operating conditions case, the piezometric level used in the stability analyses is based on a design groundwater level that has been assumed to be at the “stabilized” groundwater conditions measured on November 14, 20147 in the standpipe piezometer (i.e. at Elevation 97.4 m). For the dewatered (dry) pond conditions during maintenance, the piezometric level in the stability analyses has been assumed to be depressed to the base of the pond.

The results of the static global stability analyses indicate that a factor of safety greater than 1.5 is achieved for the global stability of permanent cut slopes inclined at an overall profile of 4H:1V at the SWM Pond, both under normal operating conditions and during drained (unwatered level) conditions. The overall 4H:1V profile for the pond slopes is considered appropriate even where the pond will be excavated into shale bedrock, due to the potential for weathering of the shale bedrock over time. Results of the global static stability analyses are included on Figures 1 and 2 for selected critical locations.

A maximum (steepest) cut slope inclination of 4H:1V is also recommended to promote surficial stability of the cut slopes under changes in the operating water level and to reduce the potential surface erosion of the cut slopes above the water level. Recommendations for protection and enhancement of the surficial stability of the pond side slopes are provided in Section 6.5.

6.5 Surficial Stability and Erosion Protection

The requirements for design of erosion protection measures for the water inlet and outlet works should be assessed by the hydraulic design engineer. As a minimum, rip-rap treatment for the inlet and outlet of the storm sewer pipes and/or ditches/channels should be consistent with the standard presented in OPSD 810.010 (General Rip-Rap Layout for Sewer and Culvert Outlets) Rip-Rap Treatment Type A, with the rip-rap placed to above the pipe obvert, in combination with cut-off headwalls if these are adopted. Rip-rap should be provided over the full extent of the side slopes and base grade below and adjacent to the sewer inlet / outlet locations.

The pond slopes above the operating water level should be vegetated as soon as practical after construction to minimize the potential for erosion due to surface water run-off, either by placement of topsoil as per OPSS 802 (topsoil) and seeding as per OPSS.PROV 804 (Seed and cover) or pegged sod in accordance with OPSS 803 (sodding). Consideration could also be given to protecting the active water line zone (i.e., from the low water level to the high water level) with a minimum 150 mm thick layer of OPSS.PROV 1004 (Aggregates) R-10 rip-rap, constructed in accordance with OPSS 511 (rip-Rap, Rock Protection); however, this may not be necessary if appropriate vegetation can be established in this zone.

In addition, a granular drainage blanket may be required to control surficial sloughing of cut slopes through saturated cohesionless soil (sand) zones or layers, if these are encountered perched above the residual soil deposit. Determination of the frequency, extent and exact locations of such seepage zones from the limited borehole data is not possible. Therefore, an observational approach is recommended involving examination of the cut slopes during and following construction to identify any areas of water-bearing cohesionless soils; where present, a granular drainage blanket minimum 0.3 m thick should be placed on the pond cut slopes where lenses or layers or water-bearing cohesionless soils are observed to minimize surficial sloughing and/or erosion.

6.6 Hydraulic Conductivity

The hydraulic conductivity of the soils anticipated to be present along the side slopes of the proposed SWM Pond as encountered in Boreholes SWMW-01 to SWMW-04 has been estimated based on the grain size distribution test results from the recent borehole investigation using the following empirical correlation developed by Hazen as referenced in Freeze and Cherry (1979):

$$K=Ad^2_{10}$$

Where: K = Hydraulic conductivity (cm/s)

A = constant equal to 1

d_{10} = grain size for which 10 per cent of the particles are finer (mm)

The hydraulic conductivity of the sandy silty clay to silty clay and of the sandy silty clay residual soils is anticipated to range between about 2×10^{-6} cm/s and 2×10^{-7} cm/s; and for the sand deposit encountered in one borehole is estimated to be about 1×10^{-2} cm/s (Freeze and Cherry, 1979).

The hydraulic conductivity of the shale bedrock is controlled by and highly dependent on the extent of weathering, amount of clay infilling within the beds and the extensiveness of horizontal and vertical fractures. Based on recent experience at nearby sites along the QEW, the horizontal hydraulic conductivity of the shale bedrock in the area of the pond is expected to range between about 1×10^{-5} cm/s and 1×10^{-7} cm/s, and the vertical hydraulic conductivity is expected to range between 1×10^{-6} cm/s and 1×10^{-9} cm/s. These hydraulic conductivity range of values will not necessarily be representative of the transition zone between the overburden and the rock or in zones of faults/ fractures where the hydraulic conductivity could be orders of magnitude higher.

As the SWM Pond will be excavated into the shale bedrock and the groundwater level in the piezometer installed in Borehole SWMW-04 was measured at the overburden/weathered bedrock contact (i.e. at Elevation 97.4 m), which is the proposed operating water level in the pond, the seepage inflow/outflow rate will be controlled by decreases/increases in the groundwater level (assuming that the water level control system is a design feature of the SMW Pond).

6.7 Construction Considerations

6.7.1 Excavation for Pond Construction

The proposed SWM Pond will require excavation to depths of up to about 3.6 m below the current ground surface. Permanent and temporary excavations for the pond and any associated drainage structures, if required, will be made through topsoil, loose sand, silty clay deposits and clayey silt residual soil and into the underlying shale bedrock. Based on the results of Unconfined Compression (UC) tests carried out on selected bedrock core samples obtained from boreholes advanced for the nearby Mississauga Road overpass, the shale bedrock at the site is weak (corresponding to uniaxial compressive strengths in the range of 6 MPa to 15 MPa), but the shale bedrock contains medium strong to strong limestone interbeds. Hoe-ramming techniques will likely be required to penetrate through the harder limestone interbeds and into the bedrock to reach the design base elevation. It is recommended that a Non-Standard Special Provision (NSSP) be included in the Contract Documents to warn the Contractor of the bedrock characteristics, and that excavation into the bedrock will require appropriate equipment and construction procedures. An NSSP is provided in Appendix C for inclusion in the Contract Documents.

For temporary or permanent excavations required within or adjacent to the proposed SWM pond (including for drainage structures (e.g. for drainage pipes, drainage structures or headwalls)), the sand and silty clay deposit are considered to be Type 3 soils and the clayey silt residual soil is considered to be Type 2 soil above the water table and Type 3 soil below the water table, according to the Occupational Health & Safety Act & Regulation (OHSA) for Construction Projects. As such, temporary open-cut excavations should be completed with side slopes no steeper than 1H:1V in Type 3 soil and Type 2 soil below the water table. All excavations must be carried out in accordance with the latest edition of the OHSA.

6.7.2 Groundwater Control During and Following Construction

As discussed in Section 6.2, the groundwater level is approximately 1.4 m above the design pond base elevation. Relatively minor groundwater seepage is anticipated from the relatively low permeability clayey silt residual soil. During wet periods of the year or during periods of precipitation there is the potential that perched water conditions may be present in the sand deposit overlying the cohesive deposits at site; some groundwater seepage from the sand deposit should be expected during wet periods of the year or during periods of precipitation. More significant groundwater inflows are anticipated from discontinuities and fracture zones within the shale bedrock and/or near the weathered zone/bedrock interface. Dewatering should be carried out in accordance with OPSS 517 (Dewatering) as amended by Special Provision No. 517F01, a copy of which is included in Appendix C.

Due to the subsurface and groundwater conditions relative to the proposed pond geometry, groundwater inflows are anticipated to occur during construction. Lowering of the groundwater table to a minimum of 0.3 m below the base of excavation prior to the start of excavation activities is recommended to allow excavation to be carried out in-the-dry during construction. If construction water pumping volumes are anticipated to exceed 50 m^3/day, an Environmental Activity Section Registry (EASR) will be required as per the recently introduced changes to the Environmental Protection Act by the Ontario Ministry of Environment and Climate Change (MOECC)

It is recommended that the groundwater control measures be turned off progressively to allow the groundwater levels to recover in a controlled manner to prevent loosening/softening of the pond base and side slopes. A Non-Standard Special Provision (NSSP), provided in Appendix C, should be included in the Contract Documents to address the groundwater control requirements during construction.

7.0 CLOSURE

This report was prepared by Mr. Matthew Kelly, P.Eng., a Geotechnical Engineer with Golder. Ms. Sandra McGaghran, M.Eng., P.Eng., a Geotechnical Engineer and Associate with Golder, and Mr. Paul Dittrich, P.Eng., a Principal with Golder reviewed the technical aspects of the report. Mr. Jorge Costa, P.Eng., MTO Foundations Designated Contact for Golder and Senior Consultant, conducted a quality control audit of the report.

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MWK/SMM/JPD/JMAC/sm/rb

https://golderassociates.sharepoint.com/sites/11176g/shared documents/07-reporting/foundations/1 - swm pond wet/final/rev 1/1662333 fidr - swm pond 2018may09.docx

REFERENCES

Bowles, J.E., 1984. *Physical and Geotechnical Properties of Soils*, Second Edition. McGraw Hill Book Company, New York.

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Chapman, L.J. and Putnam, D.F. 1984. *The Physiography of Southern Ontario*, Ontario Geological Survey, Special Volume 2, Third Edition. Accompanied by Map P.2715, Scale 1:600,000.

Freeze, R.A. and Cherry, J.A. 1979. Groundwater. Prentice Hall Publishers

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Kulhawy, F.H. and Mayne, P.W., 1990. *Manual on Estimating Soil Properties for Foundation Design*. EL 6800, Research Project 1493 6. Prepared for Electric Power Research Institute, Palo Alto, California.

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 Temperatures

Commercial Software:

Slide (Version 6) by Rocscience Inc.

Ontario Provisional Standard Drawing:

OPSD 810.010	General Rip-Rap Treatment Layout for sewer and culvert outlets

Ontario Provincial Standard Specification:

OPSS 511	Construction Specification for Rip Rap, Rock Protection and Granular Sheeting
OPSS 802	Construction Specification for Topsoil
OPSS 803	Construction Specification for Sodding
OPSS.PROV 804	Construction Specification for Seed and Cover
OPSS.PROV 1004	Material Specification for Aggregates - Miscellaneous

Ontario Water Resources Act:

Ontario Regulation 903	Wells (as amended)

Ontario Occupational Health and Safety Act:

Ontario Regulation 213/91	Construction Projects (as amended)

DRAWINGS

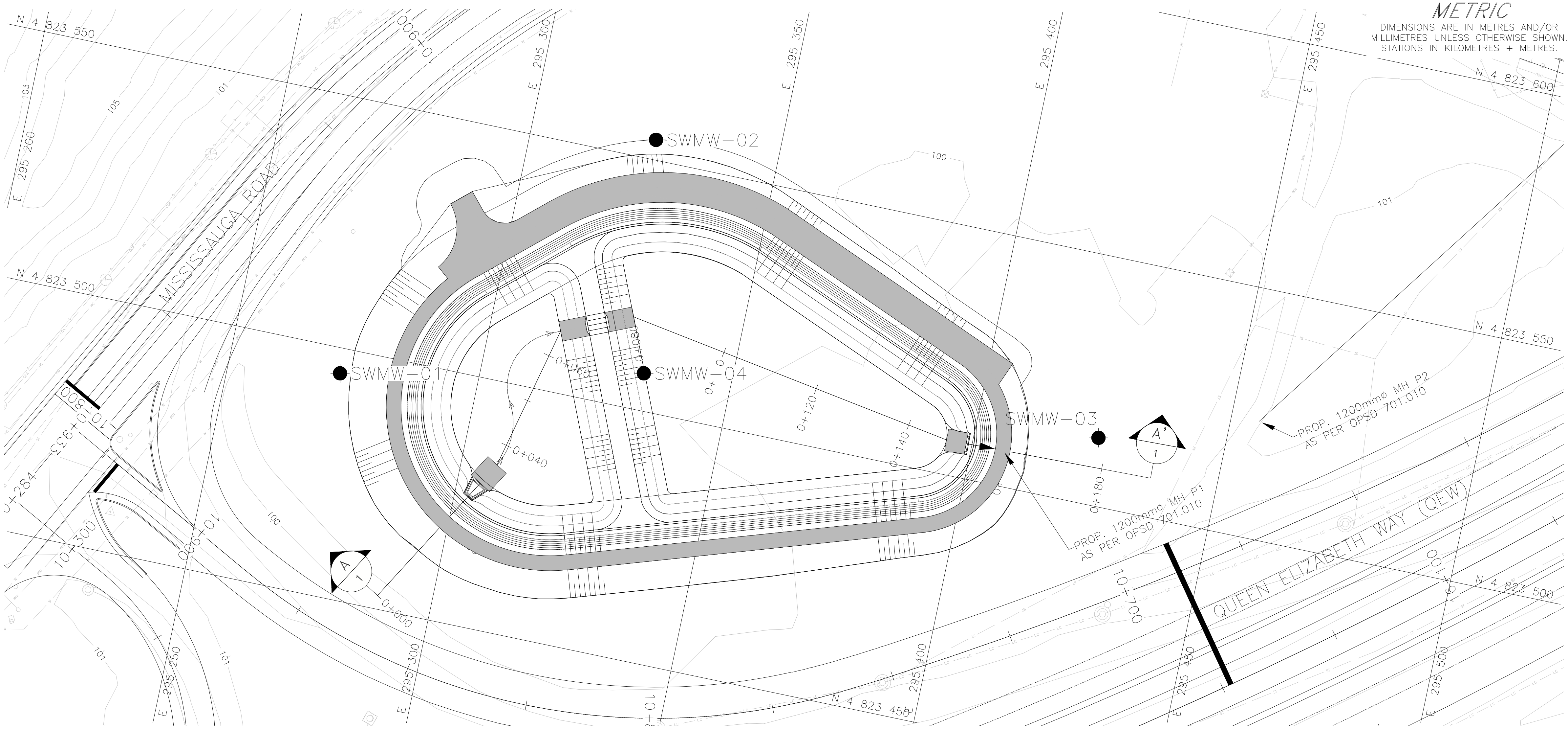
CONT No.
GWP No.2002–13–00

SHEET

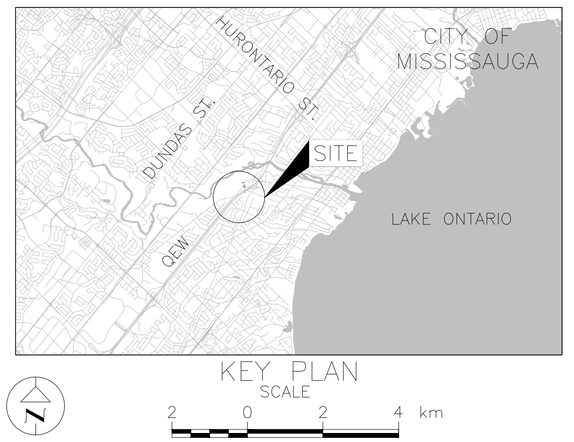
QEW STORMWATER MANAGEMENT POND
BOREHOLE LOCATIONS AND
SOIL STRATA

Golder Associates

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



PLAN
VERTICAL SCALE
10 0 10 20 m



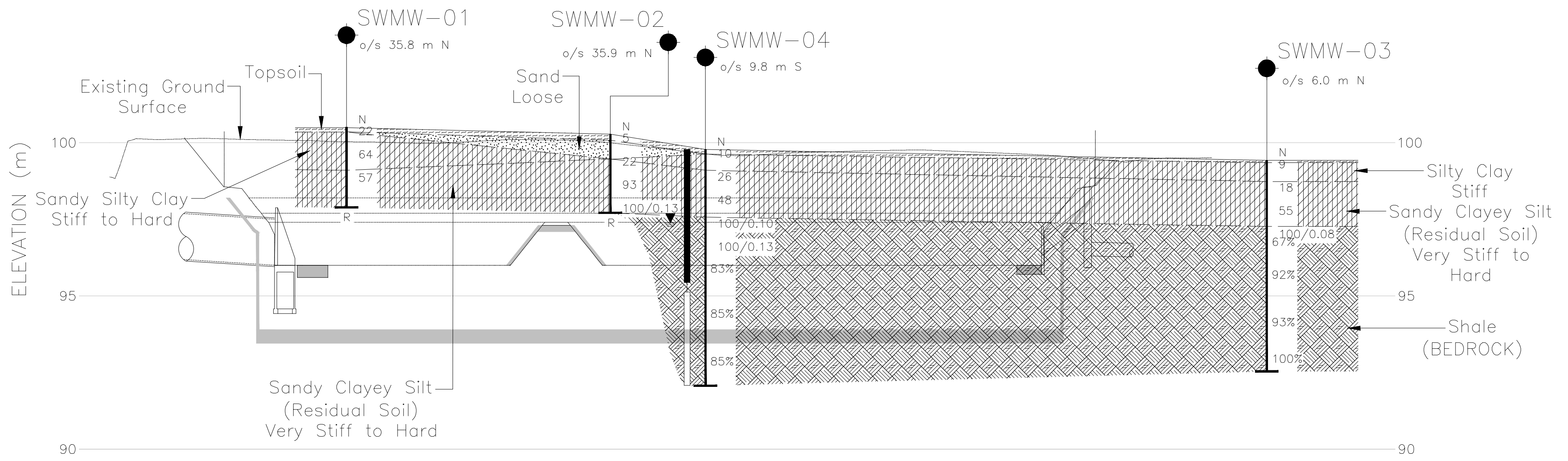
KEY PLAN
SCALE
2 0 2 4 km

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 in piezometer, measured on Nov. 14, 2017

BOREHOLE CO-ORDINATES (MTM NAD83, ZONE 10)

No.	ELEVATION	NORTHING	EASTING
SWMW–01	100.5	4823494.9	295272.4
SWMW–02	100.3	4823554.0	295325.1
SWMW–03	99.4	4823513.7	295424.7
SWMW–04	99.8	4823507.5	295332.3



A–A' CROSS–SECTION A–A'

HORIZONTAL SCALE
10 0 10 20 m

VERTICAL SCALE
2 0 2 4 m

NOTES

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

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

REFERENCE

Base plans provided in digital format by Morrison Hershfield, drawing file no. P7733–117404–8, dated April 27, 2017, received April 27, 2016. Wet pond plan provided in digital format by Morrison Hershfield, drawing file no. 11609340–Proposed_PONDS–3D.dwg, received November 22, 2018.

J. M. A. COSTA
Jan. 4, 2019

M.W.KELLY
Jan. 4, 2019

NO.	DATE	BY	REVISION

Geocres No. 30M12–416

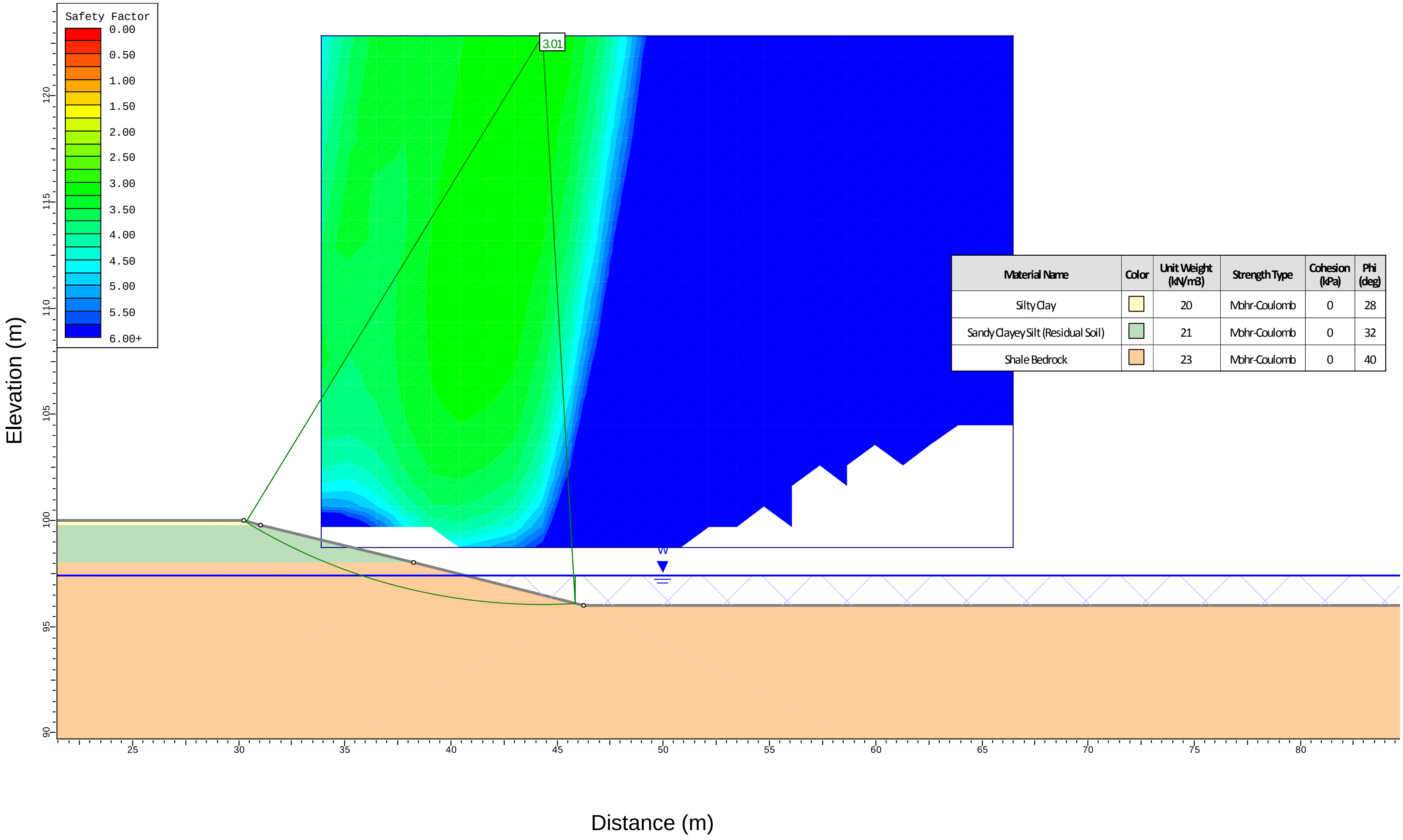
HWY. QEW		PROJECT NO. 1662333	DIST. CENTRAL
SUBM'D. DM	CHKD. MK	DATE: 1/4/2019	SITE:
DRAWN: SMD/DD	CHKD. SMM	APPD. JD/JMAC	DWG. 1

PLOT DATE: January 4, 2019
FILENAME: S:\Clients\MTO\QEW–Credit_River\99_PROJ\1662333_MH_P&F\40_PROD\0004_SWM_Pond_Wet\1662333–0004–BG–0001.dwg

FIGURES

Static Slope Stability Analysis – SWM Pond Operating Conditions

Figure 1



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
Silty Clay	🟨	20	Mohr-Coulomb	0	28
Sandy Clayey Silt (Residual Soil)	🟩	21	Mohr-Coulomb	0	32
Shale Bedrock	🟧	23	Mohr-Coulomb	0	40

Date: December 2018

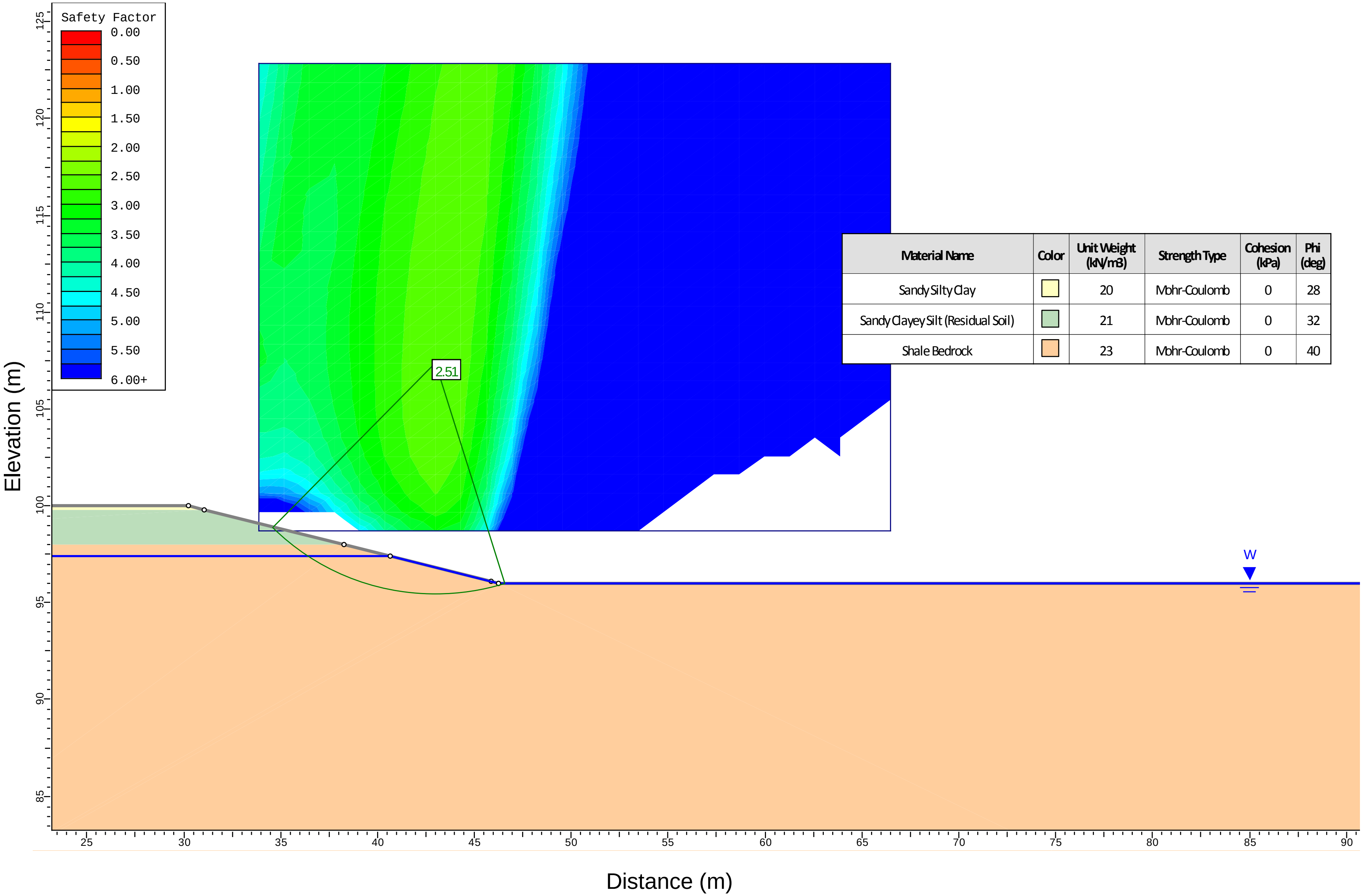
Project No: 1662333

Analysis By: MWK **Reviewed By:** JPD

GOLDER

Static Slope Stability Analysis – SWM Pond
Unwatered Condition for Maintenance

Figure 2



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Sandy Clayey Silt (Residual Soil)	🟩	21	Mohr-Coulomb	0	32
Shale Bedrock	🟧	23	Mohr-Coulomb	0	40

Date: December 2018

Project No: 1662333

Analysis By: MWK **Reviewed By:** JPD

APPENDIX A

Borehole and Drillhole Records

LIST OF SYMBOLS

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

I. GENERAL

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

II. STRESS AND STRAIN

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

III. SOIL PROPERTIES

(a) Index Properties

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

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(d) Shear Strength

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

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

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

LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

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

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

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

Dynamic Cone Penetration Resistance; N_d:

The number of blows by a 63.5 kg (140 lb.) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 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^2 pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (Q_t), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

III. SOIL DESCRIPTION

(a) Non-Cohesive Soils

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

(b) Cohesive Soils

Consistency

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

IV. SOIL TESTS

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

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

V. MINOR SOIL CONSTITUENTS

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

GOLDER

Foundation Design

RECORD OF BOREHOLE No SWMW-01

SHEET 1 OF 1

METRIC

PROJECT 1662333

G.W.P. 2002-13-00

LOCATION N 4823494.9; E 295272.4 MTM NAD 83 ZONE 10 (LAT. 43.551320; LONG. -79.617900)

ORIGINATED BY FC

DIST Central HWY QEW

BOREHOLE TYPE CME 55, 152 mm O.D., Solid Stem Augers (Auto Hammer)

COMPILED BY KN

DATUM Geodetic

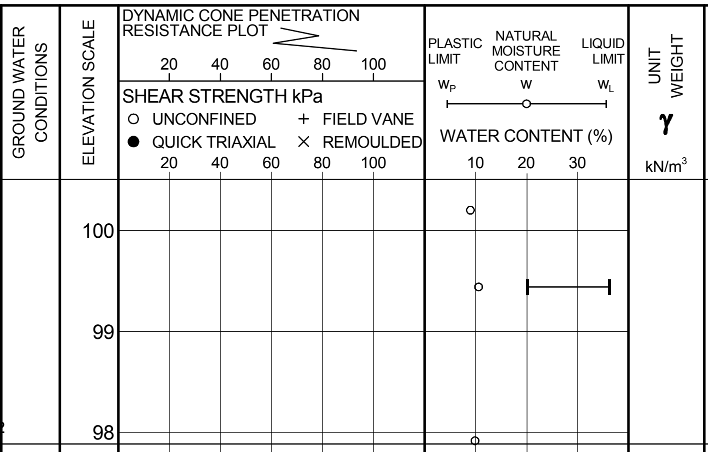
DATE August 16, 2017

CHECKED BY SMM

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
100.5	GROUND SURFACE							
0.0	TOPSOIL (150mm)							
0.2	Sandy SILTY CLAY, trace to some gravel, trace rootlets Very stiff to hard Brown Moist	1	SS	22				
		2	SS	64	8	22	55	15
99.1 1.4	Sandy CLAYEY SILT, trace to some gravel, contains shale fragments (RESIDUAL SOIL) Hard Brown Moist	3	SS	57				
98.0 2.6	SHALE (BEDROCK) Grey	4A 4B	SS	100/0.02				
	END OF BOREHOLE - AUGER REFUSAL							

NOTES:

1. Borehole dry upon completion of drilling.



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

RECORD OF BOREHOLE No SWMW-02

SHEET 1 OF 1

METRIC

PROJECT 1662333
G.W.P. 2002-13-00
DIST Central HWY QEW
DATUM Geodetic
LOCATION N 4823554.0; E 295325.1 MTM NAD 83 ZONE 10 (LAT. 43.551850; LONG. -79.617300)
BOREHOLE TYPE CME 55, 152 mm O.D., Solid Stem Augers (Auto Hammer)
DATE August 16, 2017
ORIGINATED BY FC
COMPILED BY KN
CHECKED BY MWK

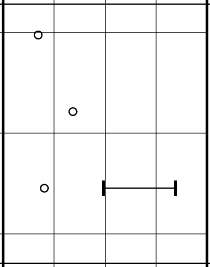
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
100.3	GROUND SURFACE										
0.0	TOPSOIL (200mm)										
0.2	SAND, trace silt, trace rootlets Loose Brown Moist	1	SS	5		100					
99.5 0.8	Sandy CLAYEY SILT, trace to some gravel, contains shale fragments (RESIDUAL SOIL) Very stiff to hard Brown to brown-grey Moist	2	SS	22		99					
		3	SS	93				8	30	49	13
97.8 2.6	SHALE (BEDROCK) Grey	4A 4B	SS	100/0.13		98					
	END OF BOREHOLE - AUGER REFUSAL										

NOTES:

1. Borehole dry upon completion of drilling.

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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



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

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

RECORD OF BOREHOLE No SWMW-03

SHEET 1 OF 1

METRIC

PROJECT 1662333

G.W.P. 2002-13-00

LOCATION N 4823513.7; E 295424.7 MTM NAD 83 ZONE 10 (LAT. 43.551490; LONG. -79.616000)

ORIGINATED BY FC

DIST Central HWY QEW

BOREHOLE TYPE CME 55, 152 mm O.D., Solid Stem Augers (Auto Hammer)

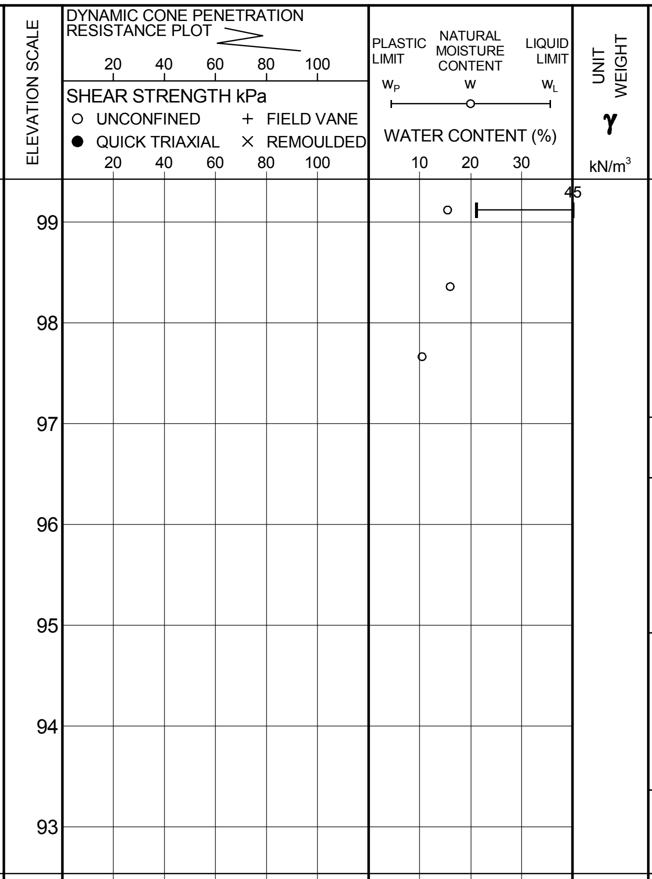
COMPILED BY KN

DATUM Geodetic

DATE August 14, 2017

CHECKED BY MWK

ELEV DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLES – NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
99.4	GROUND SURFACE					
0.0 0.1	TOPSOIL (80mm) SILTY CLAY, some sand, trace gravel, trace rootlets Stiff Brown Moist	1	SS	9		1 14 51 34
98.7 0.7	Sandy CLAYEY SILT, contains shale fragments (RESIDUAL SOIL) Very stiff to hard Brown Moist	2	SS	18		
		3	SS	55		
97.2 2.2	SHALE (BEDROCK) Grey	4	SS	100/0.08		
		1	RC	REC 100%		RQD = 67%
		2	RC	REC 100%		RQD = 92%
	Bedrock cored from depths of 2.4 m to 6.9 m For bedrock coring details refer to Record of Drillhole SWMW-03	3	RC	REC 100%		RQD = 93%
		4	RC	REC 100%		RQD = 100%
92.5 6.9	END OF BOREHOLE NOTES: 1. Borehole dry prior to rock coring.					



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

PROJECT: 1662333

RECORD OF DRILLHOLE: SWMW-03

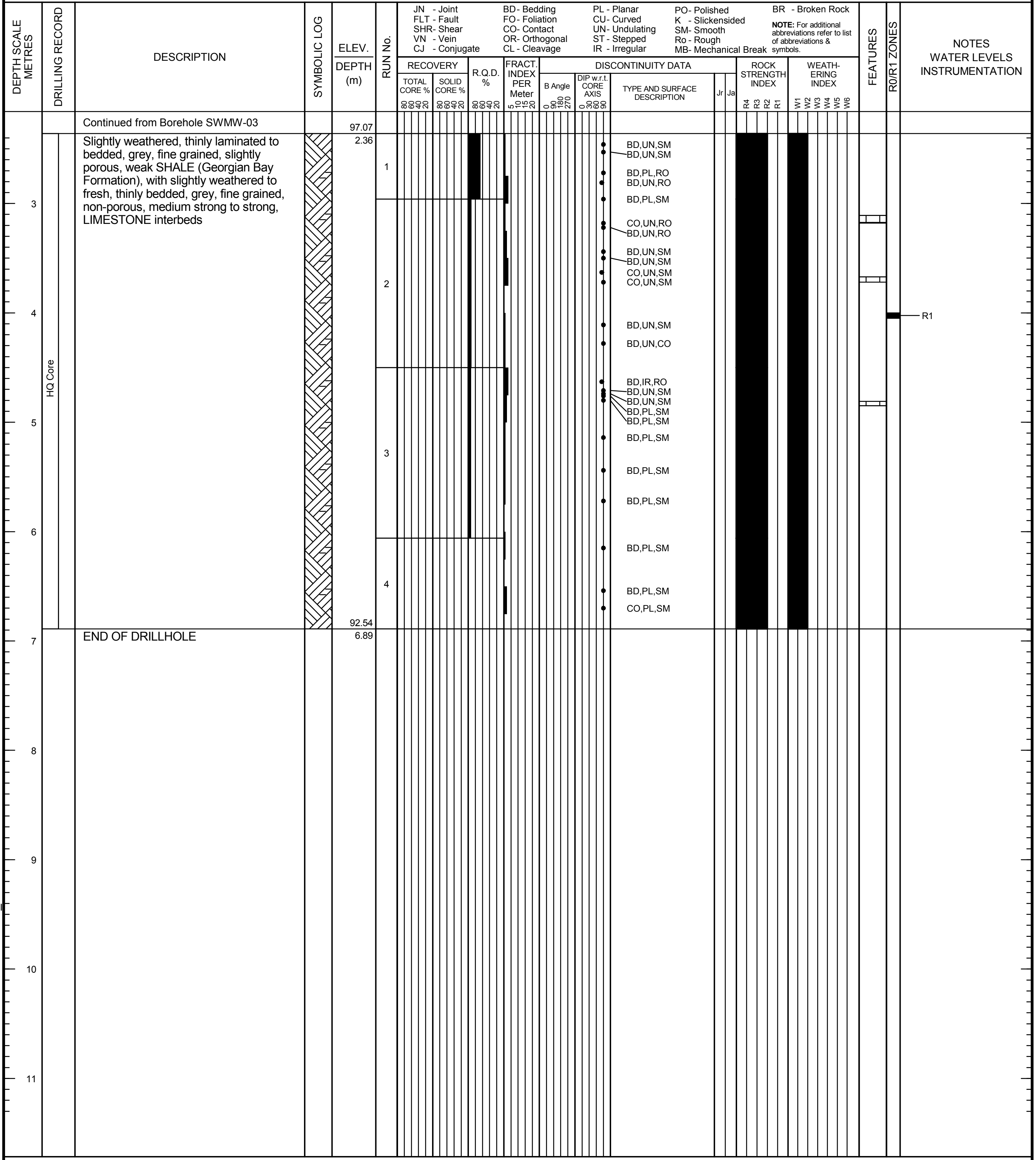
SHEET 1 OF 1

LOCATION: N 4823513.7 ;E 295424.7
INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: August 14, 2017
DRILL RIG: CME 55
DRILLING CONTRACTOR: Aardvark Drilling

DATUM: Geodetic

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



DESCRIPTION	ELEV. DEPTH (m)	TYPE AND SURFACE DESCRIPTION
Continued from Borehole SWMW-03	97.07	
Slightly weathered, thinly laminated to bedded, grey, fine grained, slightly porous, weak SHALE (Georgian Bay Formation), with slightly weathered to fresh, thinly bedded, grey, fine grained, non-porous, medium strong to strong, LIMESTONE interbeds	2.36	BD,UN,SM BD,UN,SM BD,PL,RO BD,UN,RO BD,PL,SM CO,UN,RO BD,UN,RO BD,UN,SM BD,UN,SM CO,UN,SM CO,UN,SM BD,UN,SM BD,UN,CO BD,IR,RO BD,UN,SM BD,UN,SM BD,PL,SM BD,PL,SM BD,PL,SM BD,PL,SM BD,PL,SM BD,PL,SM BD,PL,SM CO,PL,SM
	92.54	
END OF DRILLHOLE	6.89	

FEATURES LEGEND

▨ BROKEN CORE
▩ CLAY SEAM
▦ LIMESTONE
■ LOST CORE

DEPTH SCALE
1 : 50

GOLDER

LOGGED: FC
CHECKED: AC

GTA-RCK 054 N:\COMMON\SDONOVAN\FOR MATT\20180509\QEW-CREDIT_RIVER.GPJ GAL-MISS.GDT 9/5/18

RECORD OF BOREHOLE No SWMW-04

SHEET 1 OF 1

METRIC

PROJECT 1662333

G.W.P. 2002-13-00

LOCATION N 4823507.5; E 295332.3 MTM NAD 83 ZONE 10 (LAT. 43.551430; LONG. -79.617200)

ORIGINATED BY FC

DIST Central HWY QEW

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

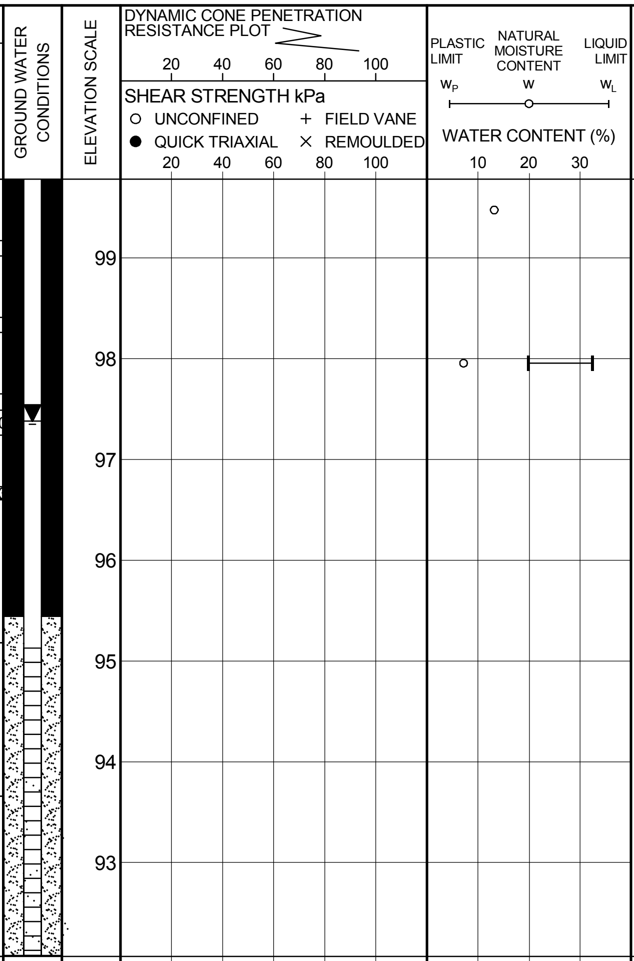
COMPILED BY KN

DATUM Geodetic

DATE August 16, 2017

CHECKED BY MWK

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
99.8	GROUND SURFACE				
0.0	TOPSOIL (150mm)				
99.6 0.2	Sandy SILTY CLAY, trace gravel, trace rootlets Stiff Brown Moist	1	SS	10	
99.1 0.7	Sandy CLAYEY SILT, contains shale fragments (RESIDUAL SOIL) Very stiff to hard Brown to grey Moist	2	SS	26	
		3	SS	48	
97.6 2.2	SHALE (BEDROCK)	4	SS	100/0.10	
		5	SS	100/0.13	
		1	RC	REC 100%	RQD = 83%
	Bedrock cored from depths of 3.2 m to 7.7 m For bedrock coring details refer to Record of Drillhole SWMW-04	2	RC	REC 100%	RQD = 91%
92.1		3	RC	REC 100%	RQD = 85%
7.7	END OF BOREHOLE				



UNIT WEIGHT γ kN/m³

NOTES:

1. Borehole dry prior to rock coring.

2. Groundwater level measurements in piezometer:

Date	Depth (m)	Elev. (m)
14/11/2017	2.4	97.4
21/11/2017	2.4	97.4
28/11/2017	2.4	97.4

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

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

PROJECT: 1662333

RECORD OF DRILLHOLE: SWMW-04

SHEET 1 OF 1

LOCATION: N 4823507.5 ;E 295332.3

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: August 16, 2017

DRILL RIG: CME 55

DRILLING CONTRACTOR: Aardvark Drilling

DATUM: Geodetic

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

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	R0/R1 ZONES
		Continued from Borehole SWMW-04	96.61			
4 5 6 7	HQ Core	Slightly weathered, thinly laminated to bedded, grey, fine grained, slightly porous, weak SHALE (Georgian Bay Formation), with slightly weathered to fresh, thinly bedded, grey, fine grained, non-porous, medium strong to strong, LIMESTONE interbeds	3.17	1	BD,UN,SM BD,UN,RO BD,IR,RO BD,UN,SM BD,PL,SM BD,UN,SM BD,PL,SM BD,UN,RO	
				2	BD,PL,SM BD,UN,RO BD,UN,SM CO,PL,SM CO,PL,SM BD,UN,SM BD,PL,SM	
			92.07	3	BD,PL,SM BD,PL,SM BD,PL,SM BD,IR,RO BD,IR,RO BD,PL,SM CO,UN,SM CO,PL,SM CO,UN,SM	R1 R1
8 9 10 11 12		END OF DRILLHOLE	7.71			

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

FEATURES LEGEND

▨ BROKEN CORE	▩ CLAY SEAM	▦ LIMESTONE	■ LOST CORE

DEPTH SCALE
1 : 50

LOGGED: FC
CHECKED: AC

GTA-RCK 054 N:\COMMON\SDONOVAN\FOR MATT\20180509\QEW-CREDIT_RIVER.GPJ GAL-MISS.GDT 9/5/18

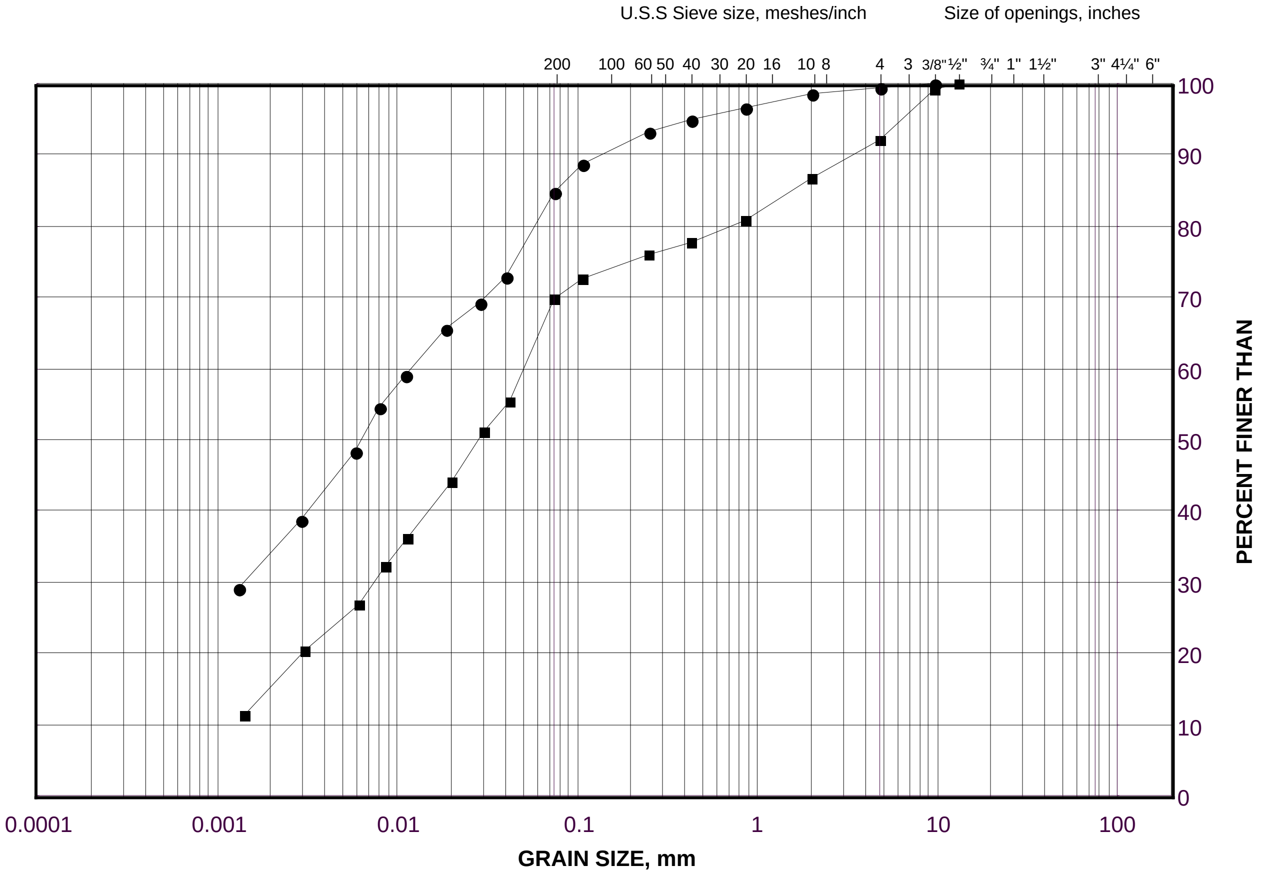
APPENDIX B

Laboratory Test Results, Bedrock Core Photographs

GRAIN SIZE DISTRIBUTION

Silty Clay to Sandy Silty Clay

FIGURE B1



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	SWMW-03	1	99.1
■	SWMW-01	2	99.4

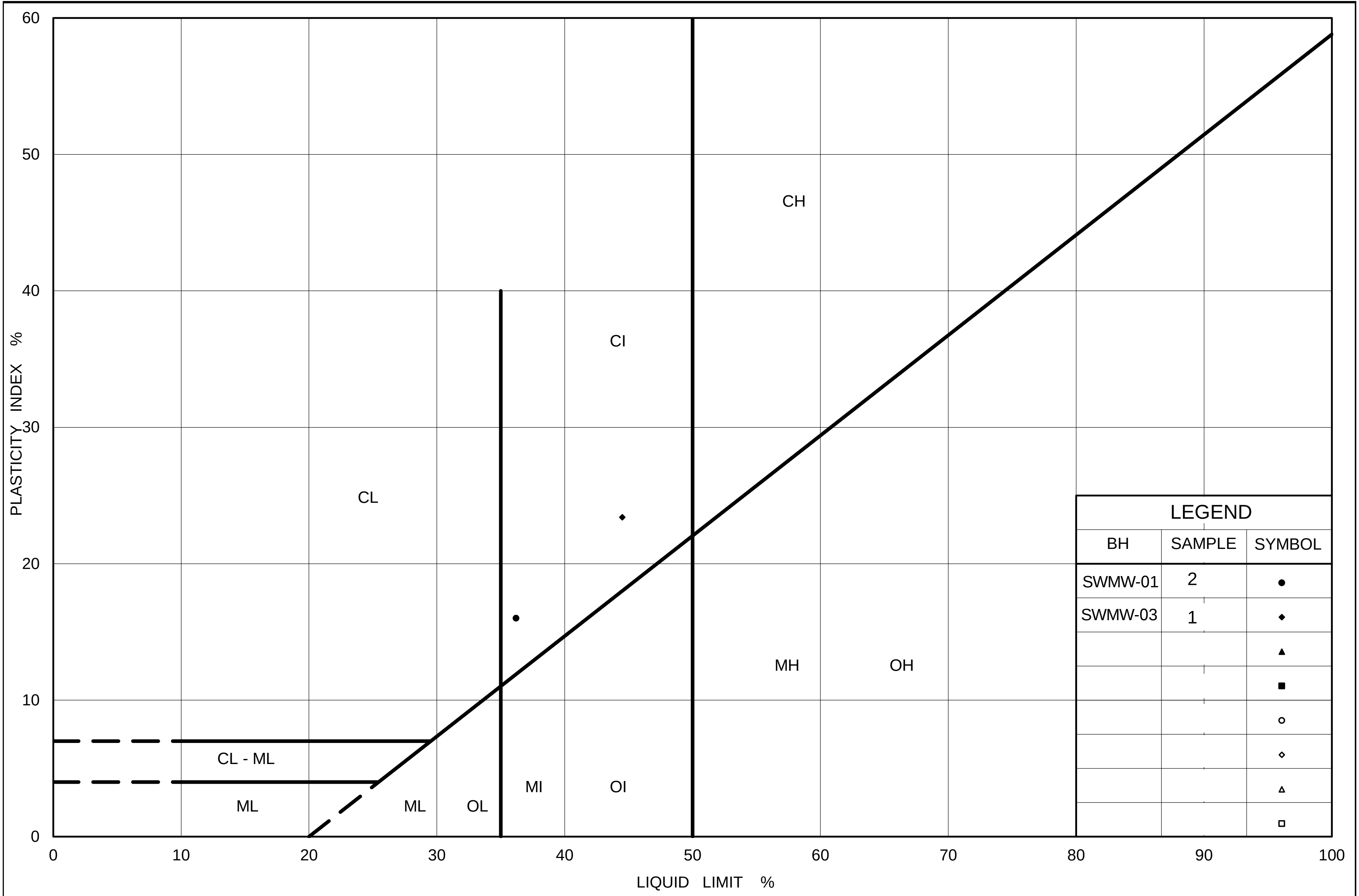
Project Number: 1662333

Checked By: JPD

Golder Associates

Date: 04-Dec-17

Oct 75, FF-S-21



BH	SAMPLE	SYMBOL
SWMW-01	2	●
SWMW-03	1	◆
		▲
		■
		○
		◇
		△
		□

Ministry of Transportation

Ontario

PLASTICITY CHART
Silty Clay

Figure No. B2

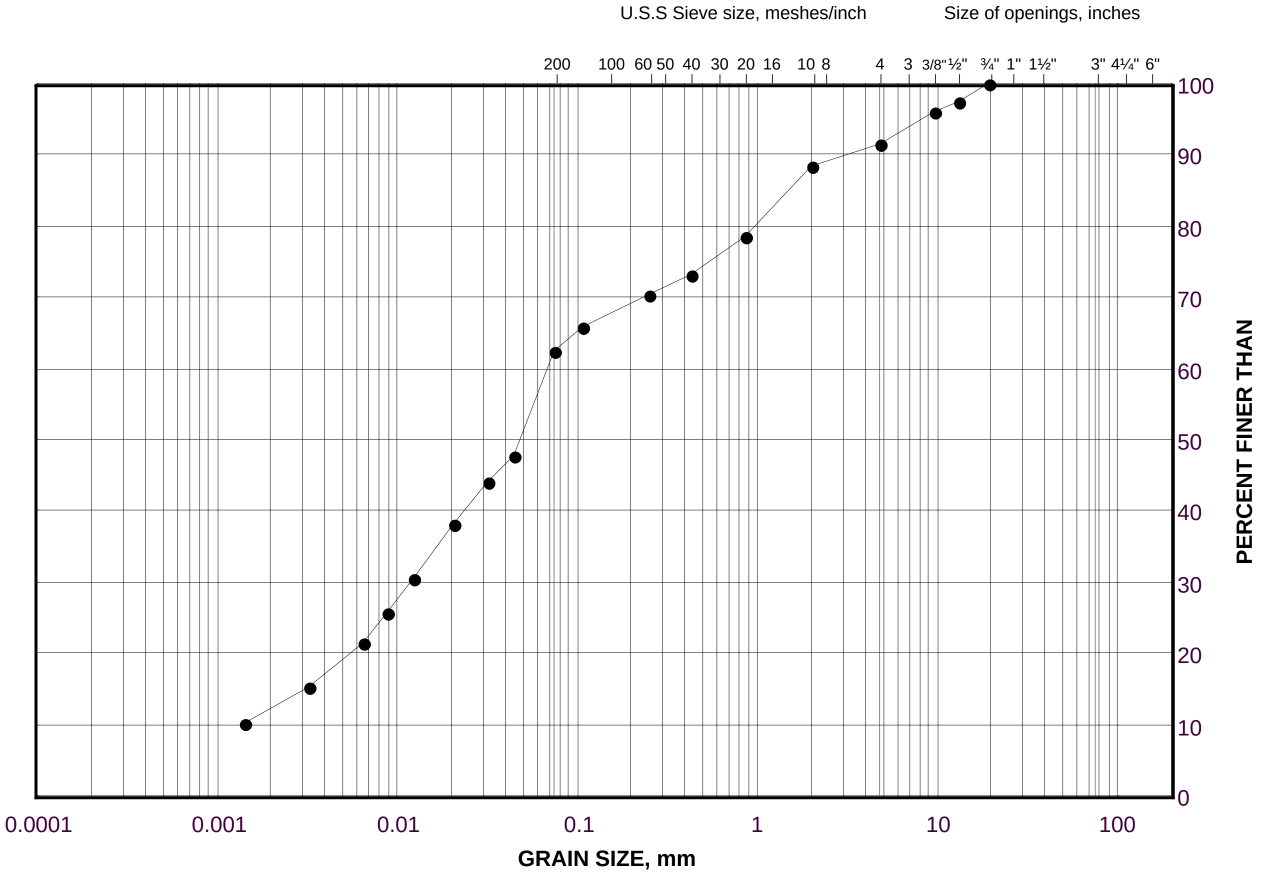
Project No. 1662333

Checked By: JPD

GRAIN SIZE DISTRIBUTION

Sandy Clayey Silt (Residual Soil)

FIGURE B3



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	SWMW-02	3	98.4

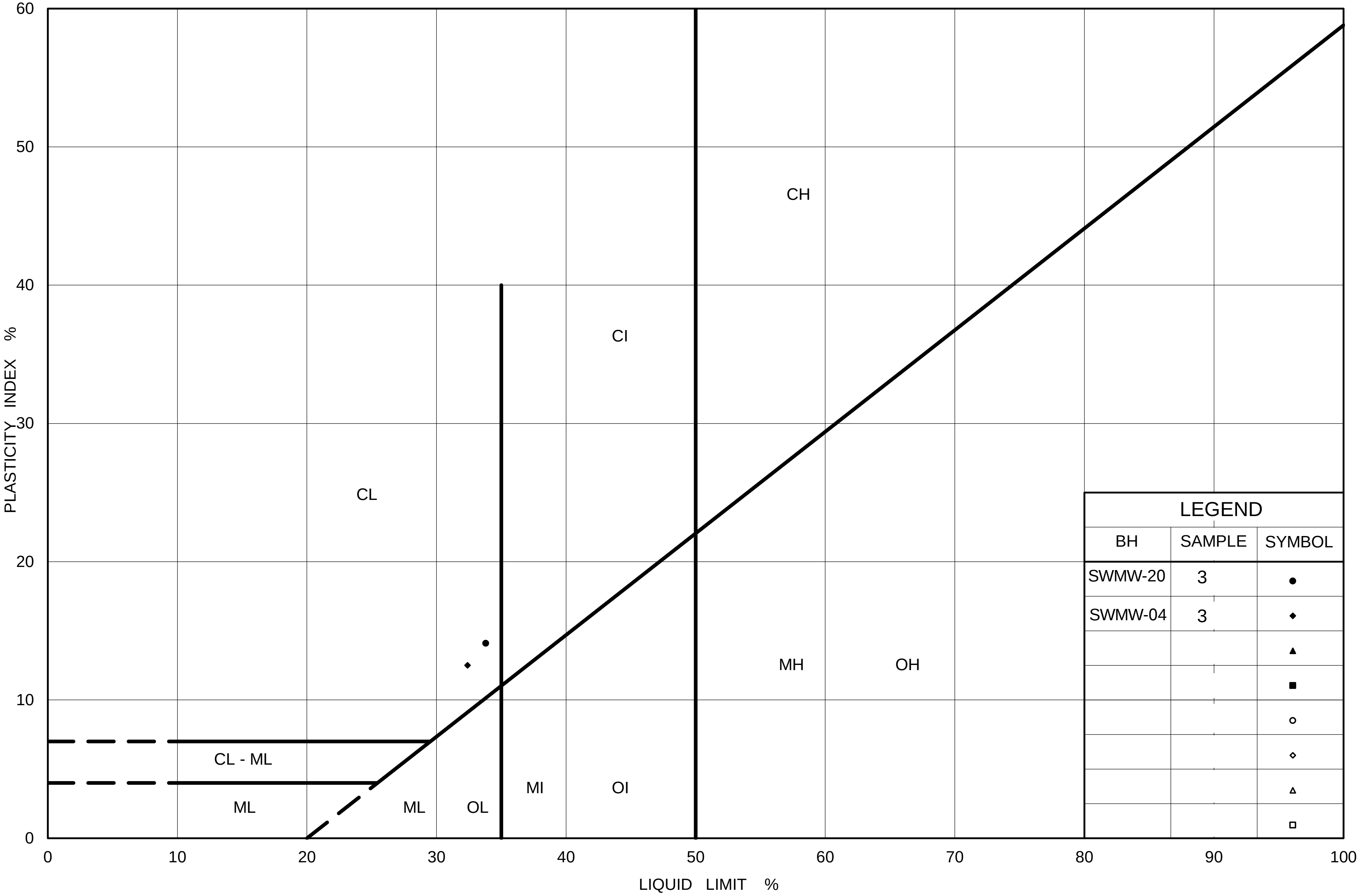
Project Number: 1662333

Checked By: JPD

Golder Associates

Date: 04-Dec-17

Oct 75, FF-S-21



Ministry of Transportation

Ontario

PLASTICITY CHART
Clayey Silt (Residual Soil)

Figure No. B4

Project No. 1662333

Checked By: JPD

Start of Run No. 1 (2.36 m)

Start of Run No. 2 (2.96 m)

Start of Run No. 3 (4.50 m)

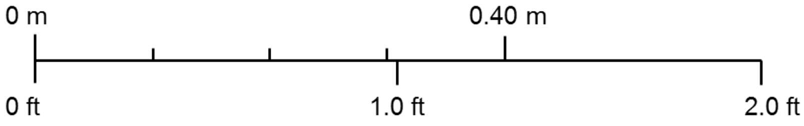
Box 1: 2.36 m to 5.43 m

Start of Run No. 4 (6.06 m)

End of Run No. 4 (6.89 m)

Box 2: 5.43 m to 6.89 m

Scale



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

TITLE
Bedrock Core Photographs
Borehole SWMW-03 (2.36 m to 6.89 m)

Golder Associates

PROJECT No. 1662333			FILE No. ----		
DESIGN	AC	20171003	SCALE	NTS	REV.
CADD	--		**FIGURE B5**		
CHECK	MWK	20170208			
REVIEW	KJB	20170208			

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


Start of Run No. 1 (3.17 m)

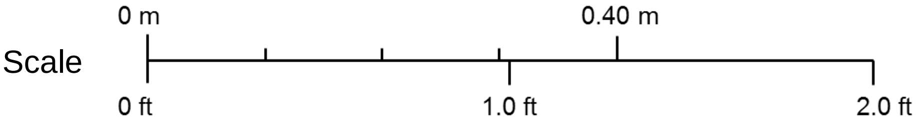
Start of Run No. 2 (4.60 m)

Box 1: 3.17 m to 6.12 m


Start of Run No. 3 (6.12 m)

End of Run No. 3 (7.71 m)

Box 2: 6.12 m to 7.71 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 SWMW-04 (3.17 m to 7.71 m)

Golder Associates

PROJECT No. 1662333			FILE No. ----		
DESIGN	AC	20171003	SCALE	NTS	REV.
CADD	--		**FIGURE B6**		
CHECK	MWK	20170208			
REVIEW	KJB	20170208			

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

APPENDIX C

Non-Standard Special Provisions

BEDROCK EXCAVATION – Item No.

Non-Standard Special Provision

The shale bedrock at the site of the Storm Water Management (SWM) Pond is weak, but contains interbeds of medium strong to strong limestone. Appropriate construction equipment and procedures will be required for excavation into the bedrock. Bedrock excavation shall not disturb the adjacent highway facilities or utilities.

BASIS OF PAYMENT

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

DEWATERING – Item No.

Non-Standard Special Provision

SCOPE

The work under this item includes the design, installation, and removal of dewatering systems to facilitate the construction of the Storm Water Management (SWM) Pond. Excavations for the SWM Pond will extend below the groundwater level at the site.

REFERENCES

OPSS 518	Construction Specification for Control of Water from Dewatering Operations

SUBMISSION AND DESIGN REQUIREMENTS

Written details for the proposed dewatering system shall be submitted to the Contract Administrator for information purposes a minimum of ten business days prior to commencing dewatering operations. The Contractor shall reference Borehole and Drillhole records included in the Contract Documents as a guide in determining requirements.

CONSTRUCTION

Dewatering System

The Contractor is responsible for the design, installation, operation, maintenance and removal of an adequate dewatering system in the shale bedrock below the base at the SWM Pond, to lower the groundwater to a minimum of 0.3 m below the base of excavation level to facilitate excavation, and liner construction (if applicable) in-the-dry.

Operation

A dewatering operation shall be provided to maintain the groundwater level below the excavation base at all times during the work. All components of the dewatering system shall be maintained in an effective, functioning and stable condition at all times during the work. Notwithstanding the above, the work shall be completed in accordance with the environmental and operational constraints specified elsewhere in the contract.

Removal

The dewatering system shall be turned off progressively following completion of the SWM Pond construction such that the groundwater is permitted to recover to normal operating levels in a controlled manner.

BASIS OF PAYMENT

Payment at the contract price for the above tender item shall be full compensation for all labour, equipment and material to do the work.

DEWATERING SYSTEM - Item No.

Special Provision No. 517F01 July 2017

Amendment to OPSS 517, November 2016

Design Storm Return Period and Preconstruction Survey Distance

517.01 SCOPE

Section 517.01 of OPSS 517 is deleted in its entirety and replaced with the following:

This specification covers the requirements for the design, operation, and removal of a dewatering or temporary flow passage system or both to control water during construction, and the control of the water prior to discharge to the natural environment and sewer systems.

517.04 DESIGN AND SUBMISSION REQUIREMENTS

517.04.01 Design Requirements

Subsection 517.04.01 of OPSS 517 is amended by deleting the first paragraph in its entirety and replacing it with the following:

A dewatering or temporary flow passage system or both shall be designed to control water at the locations specified in the Contract Documents and at any other location where a system is necessary to complete the work. The design of the system shall be sufficient to permit the work at each location to be carried out as specified in the Contract Documents.

Subsection 517.04.01 of OPSS 517 is further amended by deleting the second last paragraph in its entirety and replacing it with the following:

Temporary flow passage systems shall be designed, as a minimum, for a 2 year design storm return period and groundwater discharge, except for the work specified in Table A. For the work specified in Table A, the temporary flow passage system shall be designed, as a minimum, for the design storm return period specified in Table A and groundwater discharge. A longer return period shall be used when determined appropriate for the work.

Intensity-Duration Factor (IDF) curve location, site specific minimum return period, return period flow estimates, and other information is provided in Table A. The IDF information can be accessed through the MTO IDF Curve

Look up Tool on the Drainage and Hydrology page of MTO's website. The return period flow estimates do not include flow volumes from groundwater discharge. The Owner specifically excludes these flow estimates from the warranty in the Reliance on Contract Documents subsection of OPSS 100, MTO General Conditions of Contract.

Table A

<table>
<tr><td>IDF Curve Location</td><td colspan="3">Latitude: *</td><td colspan="3">Longitude: *</td></tr>
<tr><td colspan="7">Temporary Flow Passage Systems</td></tr>
<tr><td rowspan="2">Site Name /
Station Reference</td><td rowspan="2">Minimum Return Period (Years)</td><td colspan="4">Return Period Flow Estimates (m^3/s)</td><td rowspan="2">Design Engineer Requirements (Note 1)</td></tr>
<tr><td>2 Year</td><td>5 Year</td><td>10 Year</td><td>25 Year</td></tr>
<tr><td>**</td><td>***</td><td>****</td><td>****</td><td>****</td><td>****</td><td>*****</td></tr>
<tr><td colspan="7">Dewatering Systems</td></tr>
<tr><td>Site Name /
Station Reference</td><td colspan="5">Preconstruction Survey Distance (Note 2)
(m)</td><td>Design Engineer Requirements (Note 1)</td></tr>
<tr><td>**</td><td colspan="5">******</td><td>*****</td></tr>
<tr><td colspan="7">Note:
1. "Yes" means the design Engineer and design-checking Engineer shall have a minimum of 5 years of experience in designing systems of similar nature and scope to the required work. "No" means a minimum experience level is not required for the design Engineer and design-checking Engineer.
2. "N/A" indicates a preconstruction survey is not required.</td></tr>
</table>

NOTES TO DESIGNER:

Designer Fill-in for Table A:

* Enter the latitude and longitude co-ordinates of the IDF Curve as obtained using the MTO IDF Curve Look up Tool. Create additional tables, as necessary, if more than one (1) IDF curve was used on the contract (i.e. on a very long contract there may be two IDF curves used to better represent rainfall events for two (2) different sections of the contract).

** Fill-in site name, work, and station reference as appropriate for the dewatering system and/or temporary flow passage system item locations.

*** For temporary flow passage system item locations, fill-in the minimum design storm return period for the site based on MTO Drainage Design Standard TW-1.

**** For temporary flow passage system item locations, fill-in the design flow rate estimates for the various return periods.

***** Insert “Yes” when recommended by the Foundation Engineer. Insert “No” otherwise.

****** Fill-in the required distance for preconstruction survey if recommended by the Foundation Engineer. Fill-in “N/A” if not recommended.

WARRANT: Always with these tender items.

COMPACTED CLAY LINER FOR SWM POND – ITEM NO.

Non-Standard Special Provision

Scope

This special provision describes the requirements for the construction of the 450 mm (minimum) thick compacted clay liner over the base and side slopes of the SWM Pond.

Material

The clay liner material shall have the following properties:

Per cent clay-size material (i.e. per cent finer than 0.002 mm)	15% or greater
Plasticity index	10% or greater
Maximum particle size	100 mm

As an alternative to a natural source, a clay mixture meeting the requirements of OPSS 1205.05.03 could be used for the clay liner, provided that permeability testing (in accordance with ASTM 5084 – Permeability of Saturated Soils Using a Flexible Wall Permeameter) demonstrates that the clay mixture can attain a hydraulic conductivity of $1x10^{-7}$ cm/s or less.

The suitability of the soil or clay mixture for clay liner construction must be confirmed by the Contractor's Engineer.

Clay Liner Construction

The clay liner shall be constructed on the prepared shale bedrock subgrade. No standing water or excessive moisture shall be present on the subgrade surface at the time of clay liner construction. The clay liner is to be constructed over the entire base and the side slopes of the SWM Pond, as shown on the Contract Drawings.

The clay material shall be compacted at a water content that is within the range of 0 to 4 per cent wetter than the Standard Proctor optimum water content, as determined by the Contractor's Engineer. If water content testing by the Contractor's Engineer indicates that the water content needs to be adjusted to meet the above-noted range in compaction water content, then the adjustment (either wetting or drying) shall be carried out after placement of each loose lift and the loose lift shall be tilled to promote moisture uniformity through the full thickness of the lift prior to compaction of the lift.

The soil liner shall have a minimum final compacted thickness of 450 mm, as measured perpendicular to the subgrade surface. The soil liner shall be constructed in three lifts of equal loose thickness. Each lift shall be compacted to achieve an in situ density equal to or greater than 95 per cent of the material's Standard Proctor maximum dry density as determined by the Contractor's Engineer (by ASTM D698). Each lift shall receive a minimum of six one-way passes of the compactor to ensure kneading/bonding of the material.

The Contractor's Engineer shall perform in situ density tests and collect samples of the compacted clay liner at the in situ density test locations.

All perforations in the compacted clay liner shall be backfilled using dry bentonite pellets. Perforations that must be filled include, but are not limited to, the following:

- Nuclear density test probe holes;
- Holes made by a small spade near the nuclear density test locations to obtain a sample for laboratory water content testing; and
- Holes resulting from removal of any foreign material present in the liner material.

The size of the bentonite pellets used for backfill of the perforations shall be less than one-half the diameter of the perforation, or 25 mm, whichever is smaller. For the nuclear density test probeholes, the pellets shall be placed in lifts and compacted using a tamping rod.

The final surface of the compacted clay liner shall be shaped to the specified contours and sealed by at least one pass of a smooth drum roller.

Construction Quality Control

Monitoring of the soil liner construction shall be carried out by the Contractor's Engineer. The Contractor's Engineer's work shall include the following:

- Measurement of water content, grain size distribution, Atterberg limits, and Standard Proctor maximum dry density and optimum water content on representative samples of the clay liner material taken from the borrow area.
- Observation of the lift thickness as placed loose and after compaction.
- Monitoring of the number of passes used to compact each lift.
- Measurement of the in situ density and water content of the clay liner material after compaction.
- Inspection of the condition of the finished surface of the compacted clay liner prior to placement of the overlying ballast fill.

The proposed minimum testing frequencies are presented in Tables 1 and 2. Actual test frequencies may vary. Sampling/testing locations shall be selected by the Contractor's Engineer.

Table 1
8.0 Minimum Construction Quality Assurance Testing Frequencies
***9.0* for Borrow Source**

Test	Method	Minimum Frequency of Testing
Standard Proctor maximum dry density	ASTM D 698	1 per 1,000 m^3
Atterberg Limits	ASTM D 4318	1 per 500 m^3
Water content (Micro-wave Method)	ASTM D 4643	1 per 500 m^3
Clay size content (i.e. percent finer than 0.002 mm)	ASTM D1140	1 per 500 m^3
Maximum particle size	Visual inspection	Continuous

Table 2 Minimum Construction Quality Assurance Testing Frequencies after Compaction of Clay Liner Material		
Test	**Method**	**Minimum Frequency of Testing**
In situ water content test, per lift (except lowermost lift)	ASTM D 3017 (Nuclear Method)	1 per 500 m^2
Laboratory water content test, per lift (all lifts)	ASTM D 4643 (Microwave Method)	1 per 500 m^2
In situ density test, per lift (except lowermost lift)	ASTM D 2922 (Nuclear Method)	1 per 500 m^2

10.0 Construction Quality Assurance

The Construction Quality Control test data collected by the Contractor's Engineer shall be provided to the Contract Administrator's geotechnical/foundations consultant for review and concurrence.

11.0 Basis of Payment

Payment at the contract price for the above tender item shall include full compensation for all materials and labour to complete the work.

END OF SECTION

At Golder Associates we strive to be the most respected global company providing consulting, design, and construction services in earth, environment, and related areas of energy. Employee owned since our formation in 1960, our focus, unique culture and operating environment offer opportunities and the freedom to excel, which attracts the leading specialists in our fields. Golder professionals take the time to build an understanding of client needs and of the specific environments in which they operate. We continue to expand our technical capabilities and have experienced steady growth with employees who operate from offices located throughout Africa, Asia, Australasia, Europe, North America, and South America.

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Europe	+ 356 21 42 30 20
North America	+ 1 800 275 3281
South America	+ 55 21 3095 9500

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