FOUNDATION INVESTIGATION AND DESIGN REPORT

Stormwater Management Ponds

Highway 401/Kingston Road 38 Interchange Improvements, Kingston, Ontario

MTO GWP 4049-11-00

Agreement No. 9015-E-0007, Assignment No. 2

Submitted to:

Ministry of Transportation, Ontario

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GEOCRES No.: 31C-271

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SWM#2 - Lat: 44.288332, Long: -76.576359

1664176-2

June 29, 2018

Distribution List

1 PDF and 1 Copy - Ministry of Transportation, Ontario (Foundations Section)

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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services for two proposed Stormwater Management Ponds (SWMP) as part of the proposed improvements to the Highway 401 interchange at Kingston Road 38, in the City of Kingston. The location of each SWMP is shown on the Key Plan on Drawings 1 and 2. This work has been carried out under Retainer Agreement No. 9015-E-0007, Assignment No. 2.

The purpose of this investigation is to establish the subsurface conditions at the locations of the two proposed SWMPs by borehole drilling, rock coring and laboratory testing on selected samples. The Terms of Reference (TOR) and scope of work for the foundation investigation are outlined in MTO's Work Order No. 2016-11046, signed July 25, 2017, which forms part of the Consultant's Assignment No. 2 under Agreement No. 9016-E-0007 for this project.

2.0 SITE DESCRIPTION

SWMP #1 is located southwest of the of the intersection of McIvor Road and Jackson Mills Road, approximately 250 m north of County Road (Kingston Road) 38 / Highway 401 interchange. SWMP #2 is located to the south of the Highway 401 eastbound lanes, approximately 0.5 km west of the County Road (Kingston Road) 38/Highway 401 interchange and east of the bridge at Collins Creek.

Land use north, south and east of the proposed SWMP #1 is primarily residential and the topography is relatively flat. Kingston Road 38 and Jackson Mills Road are oriented in the north-south direction and are located along the west side and east edge, respectively of the proposed SWMP #1.

Land use south of the proposed SWMP #2 is forested private property and the natural topography of the site slopes downward from east to west in the location of the proposed SWMP, towards Collins Creek . West of the SWMP #2 is Collins Creek, which flows southerly from Collins Lake located north of the City of Kingston, and into the township of South Frontenac towards Lake Ontario. The Collins Creek wetland is designated as a provincially significant wetland by the Cataraqui Region Conservation Authority, and extends to the north and south of Collins Creek Bridge. An existing SWMP is located immediately adjacent to the south side of Highway 401. Photographs of the general site conditions are presented in Photographs 1 to 4 following the text of this report.

3.0 INVESTIGATION PROCEDURES

The field work for this foundation investigation was carried out on January 4, 18 and 19, 2018 during which time a total of four boreholes (designated Boreholes 9 to 12) were advanced: Boreholes 11 and 12 at SWMP #1; and Boreholes 9 and 10 at SWMP #2, at the locations shown on Drawings 1 and 2, respectively.

Field work was completed using a CME-55 track-mounted drill rig supplied and operated by Marathon Drilling Co. Ltd. of Greely, Ontario. The boreholes were advanced through the overburden using 210 mm outside diameter hollow stem augers. In general, soil 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]. Bedrock coring at the boreholes was carried out using an 'NQ' core barrel.

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

Groundwater conditions and water levels in the open boreholes were observed during the drilling and immediately following drilling operations, and standpipe piezometers were installed in Boreholes 9 and 11. The standpipe piezometers consist of a 50 mm diameter PVC pipe, with a slotted screen sealed at a selected depth interval within the bedrock. The piezometer installation details and water level readings are shown on the borehole records contained within Appendix A. The piezometers have been left in place to permit water level readings up to construction. All remaining boreholes were backfilled with bentonite upon completion, in accordance with Ontario Regulation 903 (Wells, as amended).

The field work was observed by a member of Golder's engineering and technical staff, who located the boreholes, arranged for the clearance of underground utility services, observed the drilling, sampling and in-situ testing operations, and logged the boreholes. The samples were identified in the field, placed in appropriate containers, labelled and transported to Golder's Whitby and Mississauga geotechnical laboratories where the samples underwent further visual examination. Classification testing (water content, Atterberg limits and grain size distribution) was carried out on selected soil samples and Unconfined Compression (UC) tests were carried out on selected specimens of the bedrock core to determine the uniaxial compressive strength of the bedrock (UCS), all in accordance with MTO and/or ASTM standards as applicable.

The borehole locations and 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 borehole locations given on the borehole records and shown on Drawings 1 and 2 are positioned relative to MTM NAD 83 (Zone 9) northing and easting coordinates, and the ground surface elevations referenced to Geodetic Datum. The borehole locations in MTM and geographic coordinates, ground surface elevations and borehole depths are summarized below.

Borehole No.	Pond Location	Location (MTM NAD 83)		Ground Surface Elevation (m)	Borehole Depth (m)
		Northing (m) (Latitude, °)	Easting (m) (Longitude, °)		
11	SWM Pond 1	4,905,368.7 (44.288301)	299,362.8 (-76.568128)	103.4	6.1*
12		4,905,371.9 (44.288331)	299,390.6 (-76.567780)	103.1	2.2
9	SWM Pond 2	4,905,372.7 (44.288332)	298,705.9 (-76.576359)	88.0	4.5*
10		4,905,328.5 (44.287935)	298,758.3 (-76.575701)	92.8	4.5*

* Includes bedrock coring to depths between 3.0 m and 4.1 m.

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

As delineated in *The Physiography of Southern Ontario*[2], this section of Highway 401 lies within the physiographic region known as the Napanee Plain. The Napanee Plain consists of a flat to undulating limestone plain, and is characterized by relatively shallow soil deposits overlying bedrock. Overburden soils of the Napanee Plain generally consist of glacial till and alluvium in river and stream valleys. In the southern portion of the Napanee Plain, low-lying areas are typically covered with deposits of stratified clay. Bedrock in the area typically consists of grey limestone/dolostone of the Gull River Formation.

4.2 Subsurface Conditions

The detailed subsurface soil and groundwater conditions as encountered in the boreholes advanced during the investigation and results of the laboratory tests carried out on selected soil and bedrock core samples/specimens are presented on the borehole records 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 Section 4.2 are uncorrected. The geotechnical laboratory testing plots, photographs of the bedrock core samples and photographs of the UC test specimens are contained in Appendix B.

The stratigraphic boundaries shown on the borehole records and on the stratigraphic profile on Drawings 1 and 2 are inferred from non-continuous sampling, observations of drilling progress and the results of Standard Penetration Tests and in-situ field 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 borehole records 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 at SWMP #1 consist of a layer of topsoil underlain by a cohesive deposit consisting of silty clay to clay further underlain by limestone bedrock. The subsurface conditions at SWMP #2 consist of clay encountered at ground surface at the west end of the SWMP and clayey silt topsoil at the east end, both underlain by limestone bedrock. A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Topsoil

A 200 mm thick layer of topsoil was encountered from ground surface in Borehole 11 and 12 (SWMP #1) and a 400 mm thick layer of clayey silt topsoil was encountered from ground surface in Borehole 10 (at the eastern portion of SWMP #2).

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

4.2.2 Clayey Silt to Silty Clay to Clay

A 2.2 m and 2.1 m thick deposit of silty clay was encountered underlying the topsoil in Boreholes 11 and 12 (SWMP #1), respectively, and a 1.5 m thick deposit of clay was encountered from ground surface in Borehole 9 (SWMP #2). The cohesive strata are described as moist to wet, brown and containing trace sand and trace organics. The surface of the deposit was encountered at Elevations 103.3 m and 103.0 m at SWMP #1 and at Elevation 88.0 m at SWMP #2.

The SPT 'N'-values measured within the clayey silt to silty clay to clay deposit range from 4 blows to 23 blows per 0.3 m of penetration, indicating a firm to very stiff consistency. In general, excluding the upper sample of this deposit, the SPT 'N'-values indicate a stiff to very stiff consistency.

The results of grain size distribution tests carried out on three samples of the cohesive deposits are shown on Figure B1 in Appendix B. Atterberg limits tests were carried out on three samples of the cohesive deposits and measured liquid limits between about 44 per cent and 56 per cent, plastic limits between about 21 per cent and 26 per cent, and plasticity indices between about 23 per cent and 30 per cent. These results, which are plotted on a plasticity chart on Figure B2 in Appendix B, indicate that the cohesive deposit can be classified as silty clay and clay, in the areas of SWMP #1 and SWMP #2, respectively. The natural water content measured on samples of the silty clay and clay deposits ranges between 32 per cent and 45 per cent.

4.2.3 Bedrock

Bedrock was encountered underlying the silty clay and clay deposit and cored at three borehole locations, and refusal to auger and split-spoon advancement on inferred bedrock was recorded at one borehole location. A summary of the bedrock encountered at each borehole location is outlined in the table below:

SWM Pond No.	Borehole No.	Depth Bedrock Encountered (m)	Bedrock Elevation Encountered (m)	Length Bedrock Cored (m)
1	11	2.3	101.1	3.8
	12	2.2*	100.9*	N/A
2	9	1.5	86.5	3.0
	10	0.4	92.4	4.1

*Inferred bedrock based on split-spoon and auger refusal.

Based on review of the bedrock core samples, the bedrock is described as moderately to slightly weathered, fine grained, thinly to medium bedded, slightly porous, grey limestone. An approximately 60 mm clay seam was noted to be present within Borehole 9 at 2.2 m depth. Photographs of the bedrock cores are shown on Figures B3 and B4 in Appendix B.

The Rock Quality Designation (RQD) measured on the recovered bedrock core samples generally ranges from 56 per cent to 95 per cent, and is 10 per cent for a 1.2 m core run from just below the bedrock surface in Borehole 10 (SWMP #2), indicating that the rock is generally of fair to excellent quality in accordance with Table 3.10 of the

Canadian Foundation Engineering Manual (CFEM), 2006[3] with the upper 1.1 m zone of bedrock in one borehole considered to be of very poor quality.

Three UC tests were performed on select samples of the rock core in accordance with ISRM[4] and measured Uniaxial Compressive Strengths ranging from 47 MPa to 160 MPa, indicating that the rock is medium strong (R3, 25<UCS<50 MPa) to very strong (R5, 100<UCS<250 MPa), in accordance with Table 3.5 of CFEM (2006)[3]. The laboratory UC test results and photographs of the condition of the bedrock core specimens tested are presented on Figures B5A to B7A and Figures B5B to B7B, respectively, in Appendix B.

4.2.4 Groundwater Conditions

The overburden samples obtained from the boreholes were generally moist to wet. Water levels were recorded in the open boreholes prior to rock coring and a standpipe piezometer was installed in each of Boreholes 9 and 11, sealed within the limestone bedrock to allow for monitoring of the groundwater level of the site. The recorded water levels are summarized below.

Borehole No. (Pond Location)	Depth to Water Level (m)	Water Level Elevation (m)	Date	Notes
9 (SWMP #2)	0.9	87.1	January 18, 2018	Open borehole prior to bedrock coring
	0.4	87.6	May 2, 2018	Standpipe piezometer
10 (SWMP #2)	Dry	-	January 19, 2018	Open borehole prior to bedrock coring
11 (SWMP #1)	2.0	101.4	January 4, 2018	Open borehole prior to bedrock coring
	0.0	103.4	January 19, 2018	Standpipe piezometer; frozen at ground surface
	0.0	103.4	May 2, 2018	Standpipe piezometer
12 (SWMP #1)	2.1	101.0	January 4, 2018	Open borehole

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.

[3] Canadian Geotechnical Society. 2006. Canadian Foundation Engineering Manual, 4th Edition, BiTech Publications.

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

5.0 CLOSURE

The field work was carried out by Lindsay Palmer, under the direction of Ms. Sarah Poot, P.Eng., geotechnical engineer and Associate with Golder. This report was prepared by Ms. Katelyn Nero and Mr. Adam Core, P. Eng. and the technical aspects were reviewed by Ms. Poot.. Ms. Lisa C. Coyne, P.Eng., an MTO Foundations Designated Contact and Principal of Golder, conducted an independent quality control review of the report.

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PART B

FOUNDATION DESIGN REPORT
STORMWATER MANAGEMENT PONDS
HIGHWAY 401/KINGSTON ROAD 38 INTERCHANGE IMPROVEMENTS
KINGSTON, ONTARIO
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6.0 DISCUSSION AND RECOMMENDATIONS

6.1 General

This section of the report provides geotechnical recommendations for the design of the proposed stormwater management ponds (SWMPs) designated SWMP #1 and SWMP #2. These recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the subsurface investigation at the proposed SWMP locations. The 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.

6.2 Pond Configurations

The proposed design elements for SWMPs #1 and #2 are outlined in the table below, based on the information provided via email correspondence from Stantec Consulting Ltd. on March 15, 2018 and subsequent design drawings received June 13, 2018, in conjunction with the approximate locations and perimeter outline of the SWMPs provided during the proposal stages.

SWMP (Relevant Boreholes)	Design Pond Bottom Elevation (m)	Design Top of Pond Elevation (m)	Design High Water Level Elevation (m)	Permanent Pool Elevation (m)
1 (Boreholes 11 and 12)	103.7	105.0	104.7	N/A – dry pond
2 (Boreholes 9 and 10)	86.3	88.8	88.7	87.4

Based on the above, the corresponding estimated cut/fill depths to achieve the proposed SWMP configurations are summarized in the table below.

SWMP (Relevant Boreholes)	Excavation Depth / Fill Thickness Relative to Pond Base Elevation	Excavation Depth / Fill Thickness Relative to Top of Pond Elevation
1 (Boreholes 11 and 12)	0.4 m to 0.7 m fill (includes compensation fill to replace topsoil excavation)	Up to 1.3 m of additional berm fill
2 (Boreholes 9 and 10*)	1.7 m to 6.5 m excavation (includes 0.2 m to 6.1 m of bedrock excavation*)	Up to 0.9 m additional berm fill (west side) and up to 6.5 m deep cut along east side

* Depth of bedrock cut extends to below bottom of Borehole 10.

It should be noted that significant bedrock excavation is required for SWMP #2. Consideration should be given by the designer to moving the location of the pond to the west to limit the amount of bedrock excavation to avoid the higher costs associated with this item. Consideration may also be given to lowering the base for the SWMP#1 dry pond, to minimize the requirements for additional fill placement at this location.

6.3 Pond Base Stability – Construction and Maintenance Conditions

The design groundwater levels indicated below have been considered in developing the design recommendations for the proposed SWMPs #1 and #2.

The groundwater level in the SWMP #1 area is assumed for design purposes to be at ground surface as recorded in the piezometer in Borehole 11. In the SWMP #2 area, the groundwater level is assumed to be at Elevation 87.6 m (at a depth of 0.4 m below ground surface) as recorded in the piezometer in Borehole 9. The groundwater levels are considered reasonable for design based on the topography of the sites and the potential for a high groundwater level in the Spring or during/following heavy precipitation and limited infiltration. Given the proposed design base elevation for SWMP #1 , which requires fill above the existing ground surface profile to achieve the pond bottom design level, and given that the design base elevation for SWMP #2 is below the bedrock surface, no base instability is anticipated at either location during both the shorter-term construction period, and during the short periods of the longer-term maintenance operations, although there may be localized softening / loosening of the soil to bedrock transition in the western portion of the SWMP #2 cut slope during construction. The elevation of the pond base relative to the high water level for SWMP#2 is provided in the table below.

SWM Pond	Design Pond Base Elevation (m)	High Water Level Elevation (m)	High Water Level Relative to Pond Base
2	86.3	88.7	2.4 m above

As the design groundwater level is approximately 1.7 m higher than the design base elevation at SWMP #2, there is potential for groundwater seepage during construction. The clay deposit has a relatively low permeability, although seams or interlayers of water-bearing silts and sands could be encountered within the deposit. A higher seepage rate/volume should be anticipated during construction at the bedrock interface and from the weathered/fractured zone of the limestone bedrock. Further, given the natural topography of the area, there may be a slight hydraulic gradient from east to west towards the creek following the natural bedrock slope and seepage may occur through natural fissures in the excavated bedrock cut faces.

6.4 Permanent Pool Design and Pond Liner Consideration

6.4.1 SWMP #1

It is understood that SWMP #1 will operated as a “dry pond” and the pool design water level is to be accommodated with perimeter berms constructed above existing ground surface. Consideration could be given to raising/levelling the pond footprint to the design bottom-of-pond elevation using cohesive or granular fill, or to revising the design to maintain the existing ground surface without additional filling. Notwithstanding the materials used for any grade raise to design bottom-of-pond elevation, t is recommended that a 0.3 m thick layer of granular bedding be placed over the base of the pond to reduce the potential for desiccation of the clay bottom, and for protection of the base during maintenance and removal of sediment. The above noted protection layer would need to be taken into account when determining the design pond bottom elevation. The pond perimeter berms should be constructed with granular fill and the use of a clay liner, or alternatively fully constructed using a silty clay or clay material, if locally available. Details of the excavation and construction are discussed in further detail in Section 6.8.1.

6.4.2 SWMP #2

It is understood that SWMP #2 will operate as a “wet pond”. If site grading and stormwater storage requirements permit, it is recommended that the permanent pool level (i.e. operating water level) for SWMP #2 be designed to be close to the groundwater level (Elevation 87.1 m to 87.6 m as measured in the piezometer in Borehole 9 in January 2018 and May 2018, respectively), to minimize inflow or recharge of groundwater during the normal operating conditions.

At the east end of SWMP #2, bedrock excavation up to 6.1 m will be required to meet the proposed bottom-of-pond of Elevation of 86.3 m. At the west end, where native soil is present to the pond bottom, perimeter berms are required (until they meet with the excavated rock) and could be constructed using granular fill or by constructing the perimeter berm from silty clay or clay material if locally available.

It is understood that the permanent pool design level is proposed to be about Elevation 87.4 m (approximately 0.3 m higher to 0.2 m lower than the measured groundwater level at Borehole 9), and as such, there will be some net groundwater outflow from the pond in response to the fluctuation of the local groundwater regime. This will require a water control system to actively discharge pond water during precipitation events and runoff inflow as well as to minimize discharge of accumulated groundwater inflow during periods of higher groundwater levels. For this site, given the proposed pond design excavation depth, permanent pool water level and groundwater levels, we consider that a liner is not required on the base and side slopes of the excavation from a geotechnical perspective. However, given the natural variability of the limestone bedrock (natural fissures, fractures and bedding planes, as well as the potential for some disturbance during construction) and the permanent (operating) pond level (depending on the time of year), there may be a requirement to minimize surface water / groundwater infiltration interaction stipulated by the Ministry of the Environment and Climate Change (MOECC).

To address the MOECC requirements (if necessary) and to reduce the volume of seepage into and outflow from the pond, a clay liner with a thickness of not less than 0.45 m could be constructed on the bedrock subgrade and along the side slopes up to or above the design high water level. In addition to a liner, the bedrock surface should be “slush” grouted or covered by a 50 mm thick layer of unshrinkable fill/concrete as per OPSS 1359 (Unshrinkable Backfill) to fill in any voids/fractures near the bedrock surface.

If a full liner is deemed necessary to fulfill MOECC requirements, there is the potential for heave or softening/loosening of the base of the pond liner at times when the pond is either empty for maintenance purposes or the pond water level is lower than the groundwater level outside of the pond. To mitigate this for a full liner approach, it is recommended that a minimum 1.85 m thick layer of granular fill or small size rock fill (150 mm minus size material) be placed over the pond bottom liner and grouted surface. This will require additional bedrock excavation to Elevation 84.0 m (i.e., an additional 2.3 m of excavation to accommodate the 0.45 m thick liner and the 1.85 m of ballast) to maintain a design bottom Elevation 86.3 m.

As an alternative to a full liner across the entire SWMP #2 footprint, and in an attempt to limit (without fully eliminating) groundwater inflow/outflow, consideration could be given to placing a liner along the face of the berm as noted above and benching a key into the bedrock at the toe; the key should have minimum dimensions of

1.5 m deep and 1.5 m width. The liner and key should extend easterly along the berm on both sides of the pond, a minimum of 3 m past the point at which the top of the berm is fully into bedrock. This assumes that most of the groundwater losses will likely occur at the west end of the pond, since the natural hydraulic gradient is east to west or downslope towards the creek. The exact transition point would need to be determined by observation during construction. With this option there is still potential for groundwater inflow/outflow; however it will be greatly reduced compared with the option of no liner.

6.5 Settlement/Global Stability of Berm/Cut Slopes

6.5.1 Settlement of Perimeter Pond Berms

The proposed design for SWMPs #1 and #2 implies that perimeter berms up to 1.3 m and 0.9 m high, respectively, are required, either fully or partially around the perimeter of the ponds. Time-dependent consolidation settlement of the cohesive deposit under the berm fill will occur; however this settlement is expected to primarily occur during construction as the cohesive deposit is considered to be overconsolidated.

To estimate the magnitude of the expected settlements, analyses were carried out on the critical sections of the proposed berms. The rate of settlement of the cohesive foundation soils was assessed using Terzaghi's one-dimensional consolidation theory and assumes that berm fill settlement is negligible.

The immediate compression of the non-cohesive deposits was modelled by estimating an elastic modulus of deformation based on the SPT "N"-values measured in the boreholes and using correlations proposed by Bowles (1984) and Kulhawy and Mayne (1990).

The following correlation relating in-situ undrained shear strength to pre-consolidation stress (Mesri, 1975) was employed:

$$\sigma p' = s_u(mob) / 0.22$$

where: $\sigma p'$ = pre-consolidation stress (kPa)

$s_u(mob)$ = average mobilized undrained shear strength (kPa)

The consolidation settlement of the cohesive deposits (silty clay) was assessed using the results of the laboratory index testing to estimate the deformation parameters (i.e. recompression and compression indices) using empirical correlations proposed in literature by Koppula (1986). The shear strength parameters were correlated to SPT 'N'-values.

The simplified stratigraphy together with the associated strengths and unit weights employed for the different soil types in the areas of the ponds are summarized below.

Soil Type	γ (kN/m^3)	Settlement Parameters
Silty Clay to Clay*	19	s_u = 25 - 100 kPa σ_p' = 115 - 450 kPa, Cr = 0.04 e_0 = 0.6

*The surface of the silty clay to clay deposit should be proof rolled, as discussed further in Section 6.7.1.

Given the above-noted perimeter berm thicknesses (i.e., 1.3 m and 1.2 m), the total post-construction settlement of the native overconsolidated cohesive deposit at SWMPs #1 and #2 berm locations is expected to be less than 10 mm, and to occur during construction; thus settlement mitigation is not required below the SWMP side berms.

6.5.2 Global Stability of Pond Cut/Berm Slopes

Based on discussions with Stantec, it is understood that from a safety and maintenance perspective, the berms for the SWMPs are to have side slopes oriented at 5 horizontal to 1 vertical (5H:1V) from pond bottom to 3 m above the high water level, and 3H:1V above this point.

The SWMP #1 design implies that fill up to 1.3 m high/thick is required for construction of a perimeter berm. It is recommended that the proposed side slopes for a granular berm incorporated with a clay liner, or for a clay fill berm, be constructed at 3 horizontal to 1 vertical (3H:1V) inclination or flatter as may be required for safety and maintenance considerations, as noted above. It is further understood that the SWMP #2 perimeter cut (and fill on the west side) slopes are proposed to be constructed at a 3H:1V inclination or flatter. The exterior (or downstream side) of the berms for both SWMP #1 and #2 should be constructed at 3H:1V or flatter. The bedrock cut slope may be formed at an inclination of 3H:1V or flatter where liners are required on the pond interior, or steeper as noted below in Section 6.7 above the liner. It should be noted that from a geotechnical perspective, permanent slopes in limestone bedrock can be constructed at a near-vertical orientation, although such a configuration would require greater thicknesses of lining material to be placed at the orientations noted above.

Slope stability analyses have been performed using the commercially available program SLOPEW (Version 7.23), developed by Geostudios Inc., at critical sections to verify that the cut/fill slopes have a global factor of safety under static conditions equal to or greater than 1.3 in short-term conditions, and 1.5 in long-term conditions. These minimum factors of safety are considered appropriate for the proposed SWMP 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 (°) (Drained Analysis)	Undrained Shear Strength (kPa) (Undrained Analysis)
Compacted Clayey Silt to Silty Clay (Fill)	19	30	-
Topsoil	14	27	1
Silty Clay to Clay, Firm to Very Stiff	19	28	25 to 100
Limestone Bedrock	23	40	-

The piezometric level during normal operating conditions used in the stability analyses is based on a design groundwater level at Elevation 103.4 m at SWMP #1 and Elevation 87.6 m at SWMP #2, measured on May 2, 2018 in the standpipe piezometers. In the stability analyses, the groundwater level has been assumed to be depressed to the base of the pond for unwatered (dry) pond conditions (assuming the ponds may be fully unwatered) for maintenance purposes.

The results of the static global stability analyses indicate that a factor of safety greater than 1.3 and 1.5 is achieved for the global stability short (undrained) condition and long term (drained) condition, respectively, for permanent fill and/or cut slopes inclined at 3H:1V on the downstream side for the high water level condition and 3H:1V berm slopes on the pond side of SWMPs #1 and #2, both under normal operating conditions and during drained (unwatered level) conditions. The 3H:1V cut slope inclination in soil at SWMP #2 is considered appropriate even where the pond will be excavated into limestone bedrock, due to the potential for weathering of the limestone over time. The results of selected stability analyses carried out for SWMPs #1 and #2 are shown on Figures 1 to 3 for the long term (drained) condition.

A maximum (steepest) cut slope inclination of 3H:1V is also recommended to promote surficial stability of the cut/berm slopes under changes in the operating water level and to reduce the potential surface erosion of the cut/berm slopes above the water level. Recommendations for protection and enhancement of the surficial stability of the pond side slopes are provided in Section 6.6.

6.6 Surficial Stability and Erosion Protection

The requirements for design of erosion protection measures for the stormwater inlet and outlet works should be assessed by the hydraulic design engineer, taking into consideration hydraulic elements and erodibility of the subgrade soils. 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. Rip-rap should be provided over the full extent of the side slopes and base grade below and adjacent to the 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) plus 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 of 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.

6.7 Inlet/Outlet Structure Foundations

Based on the SWMP #2 drawings provided by Stantec, it is understood that two 1200 mm diameter maintenance holes (as per OPSD 701.010, Precast Concrete Maintenance Hole – 1200 mm Diameter) and two headwalls for the inlet and outlet (as per OPSD 804.030 and 804.050) are to be constructed in the southeast quadrant of the pond. It is further understood that the Maintenance Holes, MH-1 and MH-2, are proposed to be founded at approximately Elevation 86.3 m and 87.2 m, respectively and the Headwalls, HW-1 and HW-2, are to be founded at approximately Elevation 87.4 m and 86.9 m, respectively. Based on the variable bedrock surface between Boreholes 9 and 10 and the proximity of the maintenance holes and headwalls to the borehole locations, the founding conditions may range from requiring bedrock excavation, to being founded on the surface of the bedrock, to being founded on native silty clay over bedrock. In all cases, a granular bedding layer should be placed over the soil or bedrock and consist of a minimum 300 mm thick layer of OPSS 1010 - Granular ‘A’, extending laterally a minimum 300 mm beyond the edge of maintenance hole and/or headwall structure(s).

6.7.1 Geotechnical Axial Resistances

The 1200 mm diameter maintenance hole foundations bearing on the native clay soils a minimum of 1.5 m below the ground surface may be designed with a factored ultimate geotechnical resistance of 200 kPa and a factored serviceability geotechnical resistance of 100 kPa. The headwall foundations (assumed to be approximately 0.3 m to 0.5 m wide) bearing on the native clay soils may be designed with a factored ultimate geotechnical resistance 150 kPa and a factored serviceability geotechnical resistance of 100 kPa (for 25 mm of settlement).

For footings (maintenance hole or headwall) placed on a level and properly cleaned and prepared excavated bedrock surface, the factored ultimate geotechnical axial resistance may be taken as 1,000 kPa. The factored serviceability geotechnical resistance for 25 mm of settlement will be greater than the factored ultimate geotechnical axial resistance, because the bedrock is considered to be an unyielding material; as such, ULS conditions will govern.

6.7.2 Resistance to Lateral Loads

Resistance to lateral forces / sliding resistance should be calculated using a coefficient of friction, tan δ_i'. A tan δ_i' of 0.45 may be used for design at the interface between the base of pre-cast footings and the granular bedding material. A tan δ_i', of 0.7 may be used for cast-in-place footings directly on bedrock. If bedrock at greater than 10 degrees is encountered, dowels should be incorporated into the design. The horizontal resistance of the dowels is dependent on the strength of the bedrock, grout and steel. Where the rock mass is stronger than the concrete, the design of the dowels into the rock may be handled in the same way as the dowel embedment into the concrete for uniaxial compressive strength of the grout is similar to that of the concrete. The dowels should have a minimum embankment length within the very strong bedrock of 1 m, and the structural strength of the grout should not be exceeded.

6.7.3 Frost Protection

All manholes/headwalls should be provided with a minimum of 1.5 m of conventional soil cover for frost protection, in accordance with OPSD 3090.101 *(Foundation Frost Penetration Depths for Southern Ontario).* If the footings are founded on the limestone bedrock, frost protection is not required.

6.8 Construction Considerations

6.8.1 Perimeter Berm Construction and Excavation for Ponds

SWMP #1

As noted in Section 6.2, the proposed dry SWMP #1 does not require excavation based on the design pond base elevation; however filling for grading of the pond bottom to the design base level and for perimeter berm construction to thicknesses/heights of 1.3 m and 1.2 m (or 1.6 m to 1.9 m including base fill grading) is required to achieve the desired pond/berm levels noted in Section 6.2. The native topsoil or any mixed organic soils should be removed prior to the berm construction, and the native silty clay subgrade under the berm footprint should be proof-rolled with a peg foot (also known as sheepsfoot) roller prior to berm construction. An NSSP has been included in Appendix C to address the need for proof-rolling prior to berm construction, and to indicate that a peg foot-type roller should be utilized for compaction of the cohesive materials.

Fill for the base of pond grading and berm construction should consist of Granular ‘A’ or Granular ‘B’ Type I, II or III meeting the specifications of OPSS.PROV 1010 (Aggregates), or alternatively the berms may be constructed using clay material, having a plasticity index of not less than 10 per cent, and containing a minimum 15 per cent clay sizes smaller than 2 µm (0.002 mm). The berm fill should be placed and compacted in accordance with OPSS.PROV 501 (Compacting) and OPSS.PROV 206 (Grading) to not less than 95 per cent of the Standard Proctor Maximum Dry Density of the materials.

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.

Temporary or permanent excavations required within or adjacent to the proposed SWMP #1 (including drainage structures such as for drainage pipes, drainage structures or headwalls), would extend into the silty clay stratum underlying the site. The silty clay to clay 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 (OSHA) for Construction Projects. As such, temporary open-cut excavations should be completed with side slopes no steeped than 3H:1V in Type 3 soil below the water table and 1H:1V above the water table. All excavations must be carried out in accordance with the latest edition of the OSHA.

SWMP #2

Fill berm construction should consist of Granular ‘A’ or Granular ‘B’ Type I, II or III meeting the specifications of OPSS.PROV 1010 (Aggregates), or alternatively the berms may be constructed using clay material having a plasticity index of not less than 10 per cent, and containing a minimum 15 per cent clay sizes smaller than 2 µm (0.002 mm). The berm fill should be placed and compacted in accordance with OPSS.PROV 501 (Compacting) and OPSS.PROV 206 (Grading) to not less than 95 per cent of the Standard Proctor Maximum Dry Density of the materials.

As noted in Section 6.2, the proposed wet SWMP #2 will required excavations ranging from 2.1 m to 6.9 m deep below the present ground surface, including 0.6 m to 6.5 m into the bedrock at the west and east ends of the pond respectively. Temporary side slopes in the bedrock can be constructed at near vertical, that is 1H:2V (i.e. 0.5H:1V), in accordance with MTO’s Special Provision 206S03 for Rock Faces; however the permanent pond side slopes will be formed at 3H:1V or flatter to satisfy MOECC requirements. Along the west side of the pond, up to 0.9 m of berm fill is required. Where berm fill is required, the native topsoil or any mixed organic soils should be removed from the footprint of the berm and the native clay subgrade should be proof-rolled with a peg foot (also known as sheepsfoot) roller prior to berm construction, and the above-noted NSSP will also apply.

At the east end of the pond and following the natural slope, a small berm or ditch should be created to direct or control surface water inflows from the uphill side of the pond, to minimize erosion on the slope face(s).

Permanent and temporary excavations for the pond and any associated drainage structures, if required, will be made through topsoil and a clay deposit and into the underlying limestone bedrock. For temporary or permanent excavations required within or adjacent to the proposed SWMP #2 (including drainage structures such as for drainage pipes, drainage structures or headwalls), the topsoil and clay is considered to be Type 3 soil according to the Occupational Health & Safety Act & Regulation (OSHA) for Construction Projects, and may behave as Type 4 soil below the water table. As such, temporary open-cut excavations should be completed with side slopes no steeped than 3H:1V in Type 3 and Type 4 soils. All excavations must be carried out in accordance with the latest edition of the OSHA.

The limestone bedrock at the site is medium strong to strong (corresponding to unconfined compressive strengths in the range of 47 MPa to 72 MPa). The use of carefully controlled drill and blast excavation techniques may be required in order to ensure a neat excavation line and minimize face instabilities and long-term maintenance problems; if blasting is permitted on this project, it should be controlled in accordance with OPSS.PROV 120 (Use of Explosives). Alternatively, the rock faces could be excavated mechanically using large hydraulic hoe-ramming equipment; it is understood that this technique has been used successfully on previous MTO Eastern Region projects. Line drilling of the rock face prior to mechanical excavation could be used to produce a neat face with minimal overbreak. Mechanical scaling will be required to remove loose rock on the face as a result of some overbreak or which may be created due to the blocky nature of the rock mass and the potential presence of joint sets sub-parallel to the cut face. Rock excavation to the depths proposed at this site will have a slow production rate.

Where blasting is permitted and employed, inspection of the rock cut face immediately after blasting should be carried out by qualified geotechnical personnel retained by the Contract Administrator in order to assess where scaling or loosened rock should be carried out.

6.8.2 Liner Construction

SWMP #2

If a liner is required as outlined in Section 6.4.2, the liner should be constructed using a clay material, having a plasticity index of not less than 10 per cent, and containing a minimum 15 per cent clay sizes smaller than 2 µm (0.002 mm). The clay liner should have a compacted thickness of not less than 0.45 m, constructed in equal lifts and compacted to not less than 95 per cent of the Standard Proctor Maximum Dry Density of the material.

Alternatively, a geosynthetic clay liner (GCL) could be considered, in which case a minimum 0.15 m thick layer of winter sand of a gradation stipulated in OPSS.PROV 1004 (Aggregates – Miscellaneous) should be placed over the prepared bedrock surface as bedding for the GCL. If a GCL is used with rock fill adopted for ballast, the bottom 200 mm (immediately above the GCL) should be comprised of OPSS.PROV 1010 Granular B Type I material to prevent the rock fill from puncturing the GCL.

6.8.3 Groundwater Control During and Following Construction

As discussed in Section 6.2, the groundwater level at SWMP #1 is approximately 1.0 m below the design pond base elevation, and the groundwater level at SWMP #2 is approximately 1.4 m above the design pond base elevation. Relatively minor groundwater seepage is anticipated from the relatively low permeability silty clay and clay strata. However, more significant groundwater inflows should be anticipated form discontinuities and fracture zones within the upper zone of the limestone bedrock at SWMP #2, in particular the east rock cut face where the hydraulic gradient may be toward the west following the natural topography. Dewatering should be carried out in accordance with OPSS.PROV 517 (Dewatering) as amended by Special Provision 517F01, a sample of which is included in Appendix C.

Is it noted that under the Environmental Protection Act (MOECC), water taking for construction site dewatering for volumes greater than 50,000 L/day) but less than 400,000 L/day qualify for the Environmental Activity Section Registry (EASR). Under the EASR, a Permit to Take Water is not required for water taking of construction site dewatering for volumes less than 400,000 L/day. It is recommended that groundwater control measures constructed at the site be turned off progressively to allow the groundwater level to recover in a controlled manner to prevent loosening/softening of the pond base and perimeter berm 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.

6.8.4 Decommissioning of Piezometers

The piezometers at both SWMP #1 and #2, Boreholes 11 and 9, respectively have been left in place to permit further groundwater monitoring. An NSSP has been included in Appendix C to address the decommissioning of the piezometers which is to be carried out during construction.

7.0 CLOSURE

This report was prepared by Mr. Adam Core, P.Eng. and the technical aspects were reviewed by Ms. Sarah E.M. Poot, P.Eng. a senior geotechnical engineering and associate with Golder, with technical input from Mr. Jorge Costa, P.Eng., and Mr. Frank Barone, Ph.D., P.Eng. Ms. Lisa C. Coyne, P.Eng., MTO Foundations Designated Contact and Principal of Golder, conducted an independent quality control review of the report.

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Principal, MTO Foundations Designated Contact

KN/AC/JMAC/SEMP/LCC/cr

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Koppula, S.D., 1986. Discussion: Statistical Estimation of Compression Index, Geotechnical Testing Journal, ASTM, Vol. 4, No. 2, pp. 68-73.

Mesri, G., 1975. Discussion on New Design Procedure for Stability of Soft Clays. ASCE Journal of the Geotechnical Engineering Division, Vol. 101, GT4, pp. 409-412.

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:

SlopeW by Geostudios (Version 7.23).

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 120	General Specification for the use of Explosives
OPSS.PROV 206	Construction Specification for Grading
OPSS.PROV 501	Construction Specification for Compacting
OPSS.PROV 517	Construction Specification for Dewatering
OPSS.PROV 804	Construction Specification for Seed and Cover
OPSS.PROV 1004	Material Specification for Aggregates – Miscellaneous
OPSS.PROV 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade, and Backfill Material

Ontario Provincial Special Provision:

SP206S03	Amendment to OPSS 206
SP517F01	Dewatering System

Ontario Water Resources Act:

Ontario Regulation 903	Wells (as amended)

Ontario Occupational Health and Safety Act:

Ontario Regulation 213/91	Construction Projects (as amended)

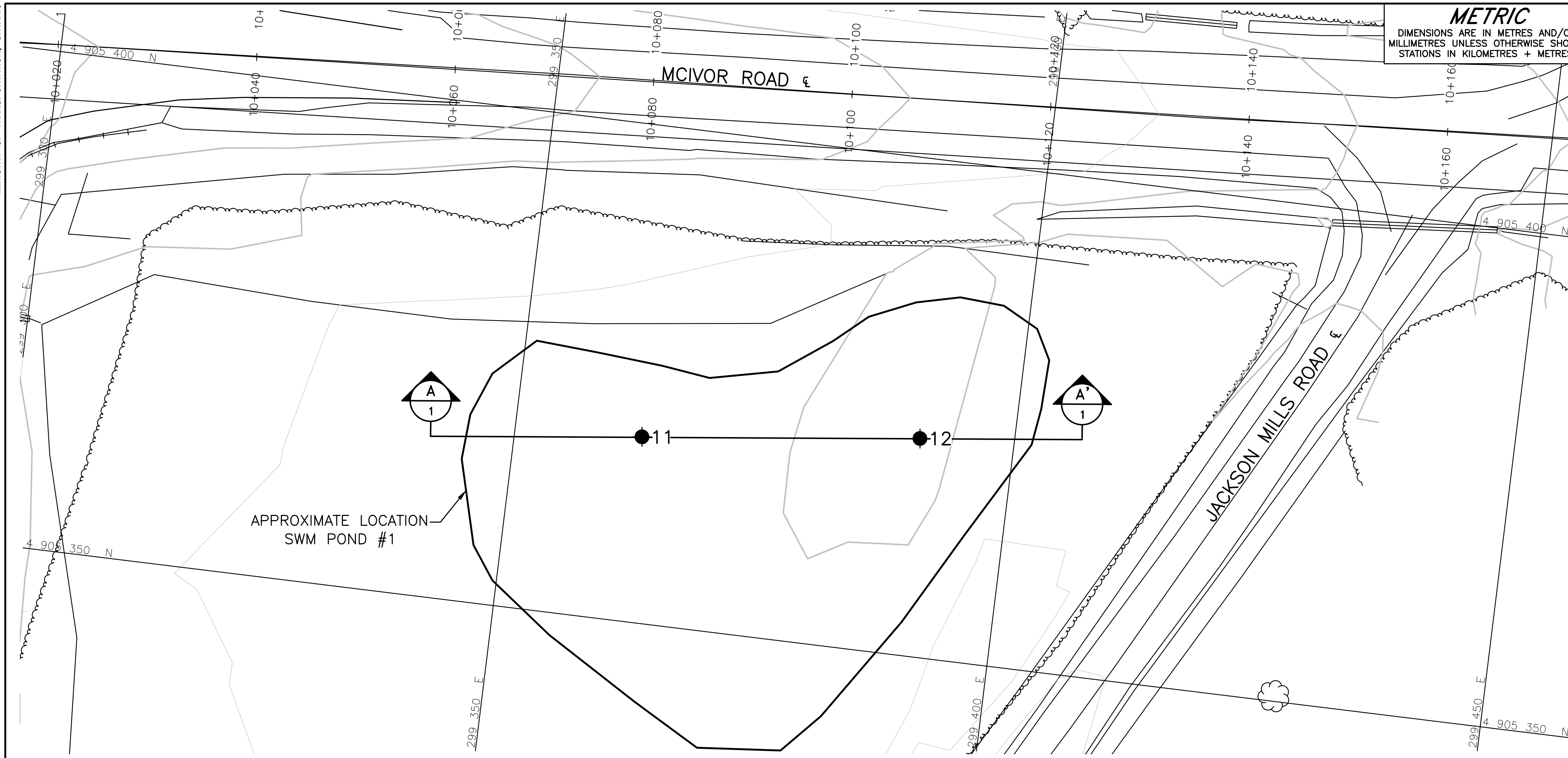
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
WP No. 4049–11–00

SHEET

MCIVOR ROAD
STORMWATER MANAGEMENT POND #1
BOREHOLE LOCATIONS AND SOIL STRATA

GOLDER



PLAN
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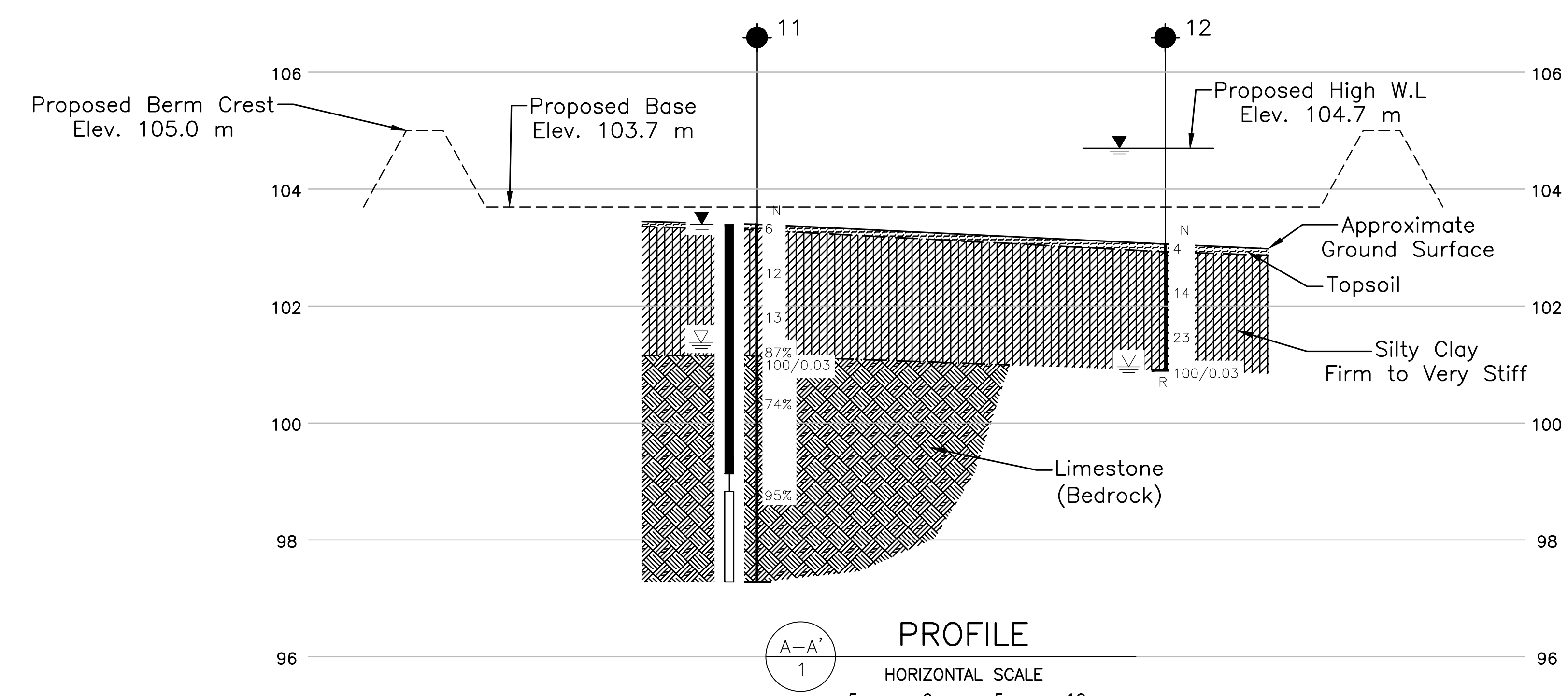
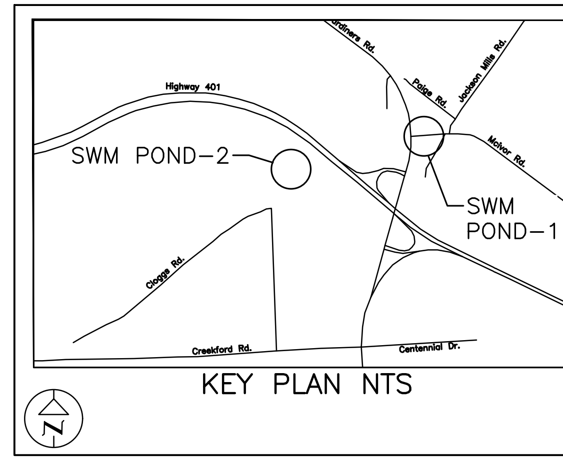


PROFILE
HORIZONTAL SCALE
5 0 5 10 m
1.25 0 1.25 2.5 m
VERTICAL SCALE



KEY PLAN NTS

LEGEND

- Borehole – Current Investigation
- Seal
- Piezometer
- N — Standard Penetration Test Value
- 16 — Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- 100% — Rock Quality Designation (RQD)
- R — Refusal
- WL in piezometer, measured on May 02, 2018
- WL upon completion of drilling

BOREHOLE CO–ORDINATES MTM NAD 83 ZONE 9

No.	ELEVATION	NORTHING	EASTING
11	103.4	4905368.7	299362.8
12	103.1	4905371.9	299390.6

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 Stantec, drawing file nos. x–401–38lc.dgw, received DEC 07, 2017, and 1033_Pond_Boreholes Coords.pdf.

LICENSED PROFESSIONAL ENGINEER L.C. COYNE June 29, 2018 PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER S.E.M. POOT 90471921 June 29, 2018 PROVINCE OF ONTARIO

NO.	DATE	BY	REVISION

Geocres No. 31C–271

HWY. 401		PROJECT NO. 1664176	DIST.
SUBM'D. KN	CHKD. KN/AC	DATE: 06/29/2018	SITE:
DRAWN: TR	CHKD. SEMP	APPD. JMAC/LCC	DWG. 1

MINISTRY OF TRANSPORTATION, ONTARIO

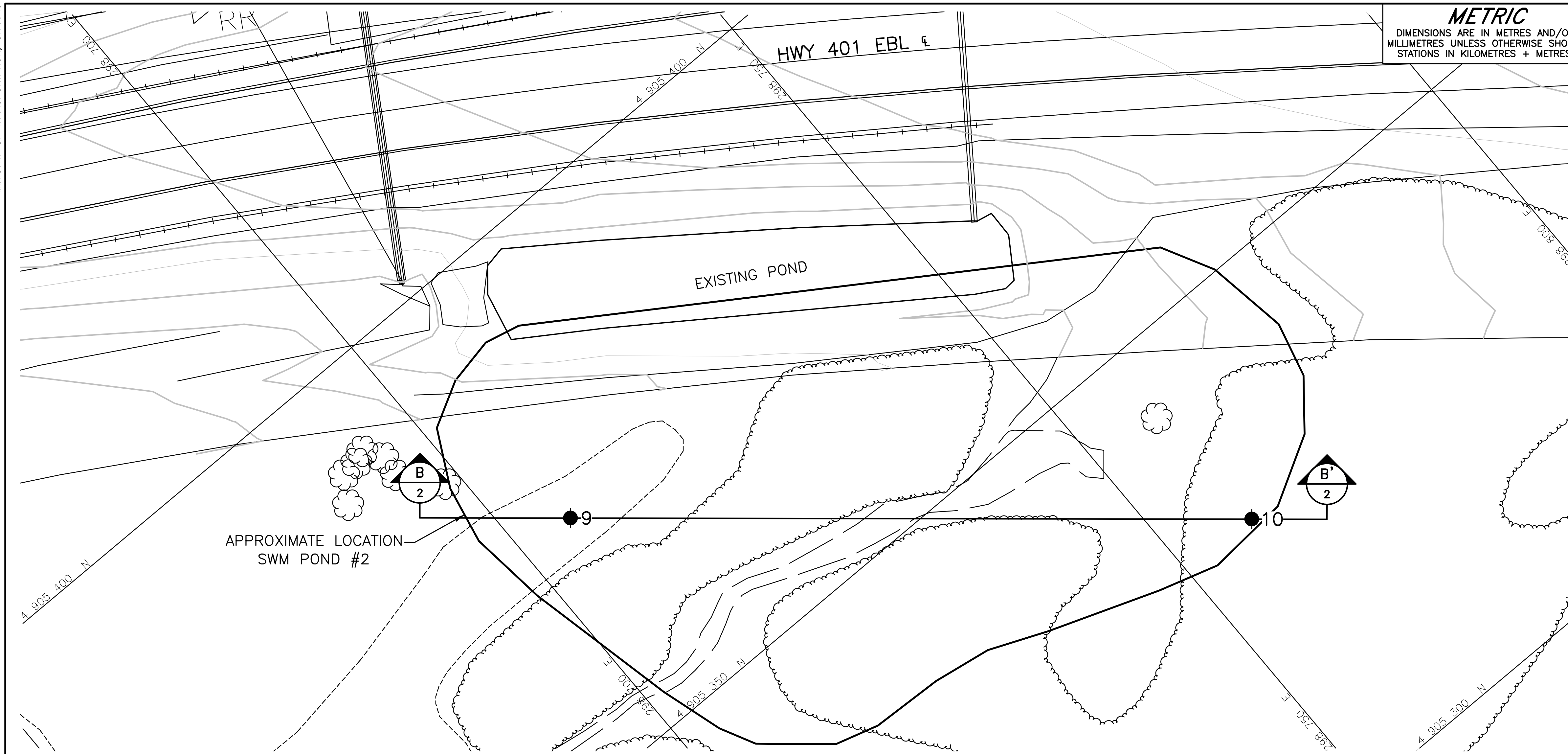
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
WP No. 4049-11-00

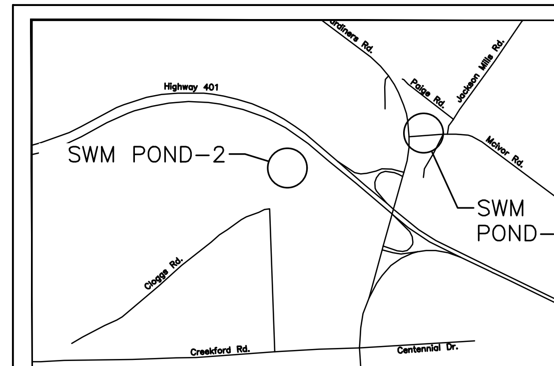
SHEET

HIGHWAY 401 EBL
STORMWATER MANAGEMENT POND #2
BOREHOLE LOCATIONS AND SOIL STRATA

GOLDER



PLAN
SCALE 5 0 5 10 m



KEY PLAN NTS

LEGEND

- Borehole – Current Investigation
- Seal
- Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- 100% Rock Quality Designation (RQD)
- WL in piezometer, measured on MAY 2, 2018
- WL upon completion of drilling

BOREHOLE CO-ORDINATES NAD 83 ZONE 9

No.	ELEVATION	NORTHING	EASTING
9	88.6	4905572.7	298701.6
10	92.8	4905538.8	298578.4

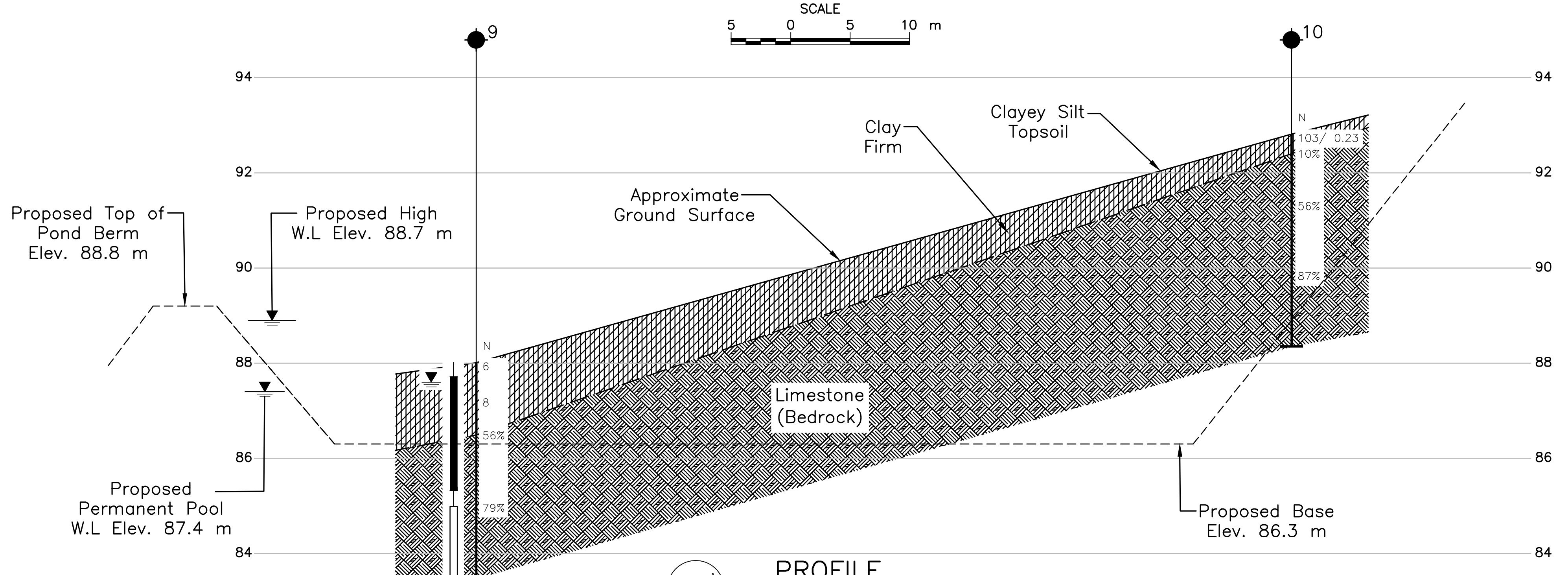
NOTES

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 Stantec, drawing file nos. x-401-38lc.dgw, received DEC 07, 2017, and 1033_Pond_Boreholes Coords.pdf.



PROFILE B-B' 2
HORIZONTAL SCALE 5 0 5 10 m
1.25 0 1.25 2.5 m VERTICAL SCALE

LICENSED PROFESSIONAL ENGINEER L.C. COYNE June 29, 2018 PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER S.E.M. POOT 90471921 June 29, 2018 PROVINCE OF ONTARIO

NO.	DATE	BY	REVISION

Geocres No. 31C-271

HWY. 401		PROJECT NO. 1664176	DIST.
SUBM'D. KN	CHKD. KN/AC	DATE: 06/29/2018	SITE:
DRAWN: TR	CHKD. SEMP	APPD. JMAC/LCC	DWG. 2

PLOT DATE: June 29, 2018
FILENAME: S:\Clients\MTO\Hwy_401_Collins_Creek_Bridge\99_PROJ\1664176_Collins_Creek\40_PROD\0001_Borehole_Location_Plan\1664176_BC_0001.dwg

Photograph 1: Stormwater Management Pond # 1 area (McIvor Road looking South) (Image from Google, accessed May 2018)

Photograph 2: Stormwater Management Pond # 1 area - McIvor Road looking west toward County Road 38 (taken May 2018)

Photograph 3: Stormwater Management Pond # 2 Area – Looking southeast from approximately 17+350 Highway 401 EBL (photo taken from Google, accessed May 2018)

Photograph 4: Stormwater Management Pond # 2 area – Looking southwest at east end of Pond from Highway 401 EBL shoulder (taken January 2018)

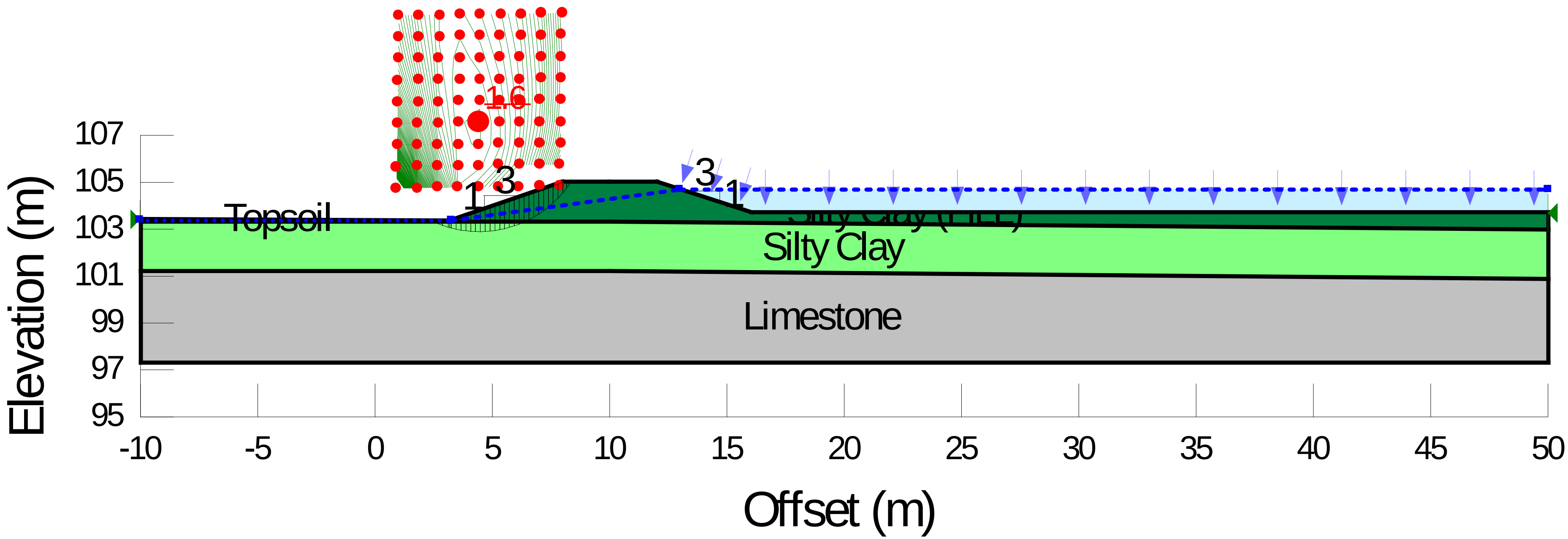
Global Stability Analysis

Figure 1

SWM Pond #1
Downstream Side – 3H:1V – Pond High Water Level
Long Term (Drained) Analysis

Material Name	Unit Weight (kN/m³)	Undrained Shear Strength (kPa)	Friction Angle (degrees)
Compacted Silty Clay Fill	19	-	30
Topsoil	14	27	1
Silty Clay	19	-	30

Note: Short Term (Undrained) Analysis was completed (not shown) using an undrained shear strength of 25 kPa and exceeds the minimum required Factor of Safety of 1.3



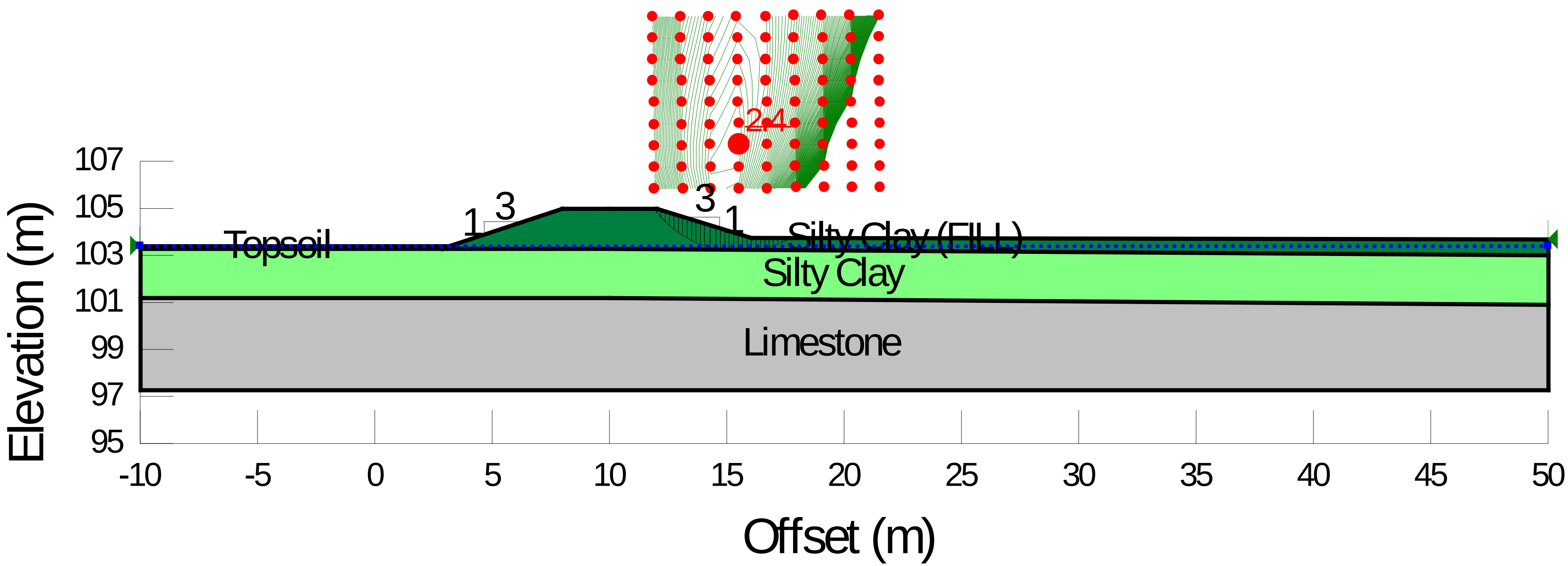
Global Stability Analysis

Figure 2

SWM Pond #1
Interior Side – 3H:1V– Pond Drained
Long Term (Drained) Analysis

Material Name	Unit Weight (kN/m^3)	Undrained Shear Strength (kPa)	Friction Angle (degrees)
Compacted Silty Clay Fill	19	-	30
Topsoil	14	27	1
Silty Clay	19	-	30

Note: Short Term (Undrained) Analysis was completed (not shown) using an undrained shear strength of 25 kPa and exceeds the minimum required Factor of Safety of 1.3



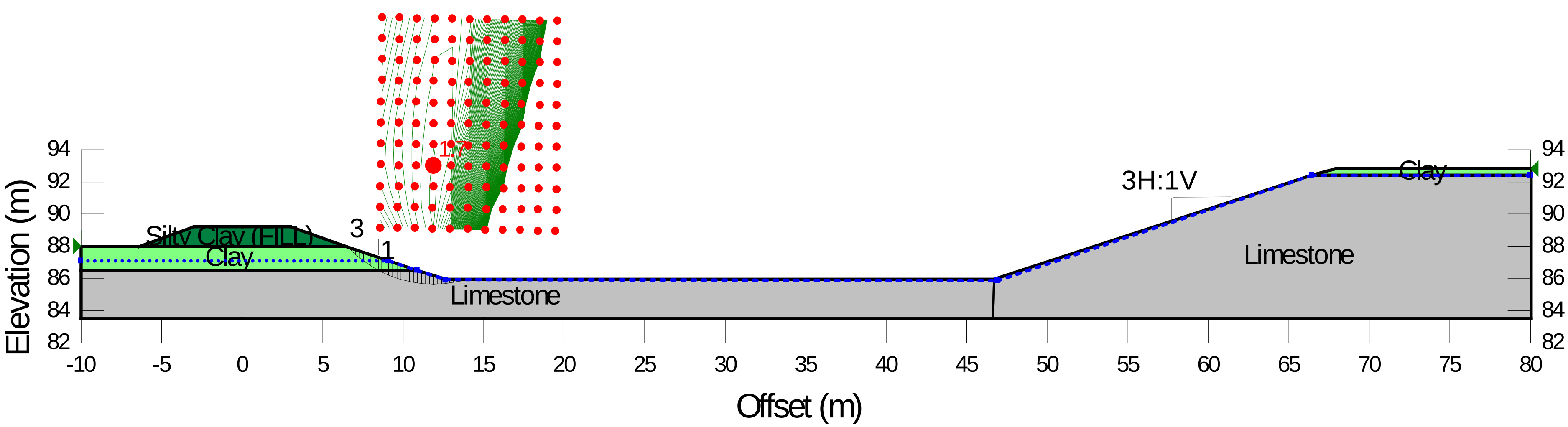
Global Stability Analysis

Figure 3

SWM Pond #2
West Side– 3H:1V – Pond Drained
Long Term (Drained) Analysis

Material Name	Unit Weight (kN/m³)	Undrained Shear Strength (kPa)	Friction Angle (degrees)
Compacted Silty Clay Fill	19	-	30
Topsoil	14	27	1
Clay	19	-	30
Limestone	23	-	40

Note: Short Term (Undrained) Analysis was completed (not shown) using an undrained shear strength of 25 kPa and exceeds the minimum required Factor of Safety of 1.3



APPENDIX A

Records of Boreholes and Drillholes

LIST OF SYMBOLS

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

I. GENERAL

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

II. STRESS AND STRAIN

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

III. SOIL PROPERTIES

(a) Index Properties

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

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

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(d) Shear Strength

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

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

LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

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

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

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

Dynamic Cone Penetration Resistance; N_d:

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

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

Piezo-Cone Penetration Test (CPT)

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

III. SOIL DESCRIPTION

(a) Non-Cohesive (Cohesionless) Soils

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

(b) Cohesive Soils

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

IV. SOIL TESTS

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

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

V. MINOR SOIL CONSTITUENTS

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

WEATHERINGS STATE

Fresh: no visible sign of weathering

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

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

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

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

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

BEDDING THICKNESS

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

JOINT OR FOLIATION SPACING

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

GRAIN SIZE

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

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

CORE CONDITION

Total Core Recovery (TCR)

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

Solid Core Recovery (SCR)

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

Rock Quality Designation (RQD)

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

DISCONTINUITY DATA

Fracture Index

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

Dip with Respect to Core Axis

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

Description and Notes

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

Abbreviations

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

GOLDER

RECORD OF BOREHOLE No 9

SHEET 1 OF 1

METRIC

PROJECT 1664176 / 0001

G.W.P. 4049-11-00

LOCATION N 4905372.7; E 298705.9 MTM NAD 83 ZONE 9 (LAT. 44.288332; LONG. -76.576359)

ORIGINATED BY KN

DIST Eastern HWY 401

BOREHOLE TYPE 210 mm O.D. Hollow Stem Augers and NQ Coring

COMPILED BY SEMP

DATUM GEODETIC

DATE January 18, 2018

CHECKED BY SEMP

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
88.0	GROUND SURFACE						
0.0	CLAY, trace sand, trace gravel, trace rootlets to 0.6 m Firm Mottled brown and grey Moist to wet	1	SS	6			
		2	SS	8	87		0 2 33 65
86.5 1.5	LIMESTONE (BEDROCK) Bedrock cored from a depth of 1.5 m to 4.5 m For bedrock coring details, refer to Record of Drillhole No. 9	1	RC	REC 93%	86		RQD = 56%
		2	RC	REC 100%	85 84		RQD = 79%
83.5 4.5	END OF BOREHOLE NOTES: 1. Borehole dry prior to rock coring. 2. Water level measurements in standpipe piezometer: Date Depth(m) Elev.(m) 18/01/18 0.9 87.1 02/05/18 0.4 87.6						

Legend: DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED (20, 40, 60, 80, 100); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) (20, 40, 60); UNIT WEIGHT γ kN/m³

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

RECORD OF DRILLHOLE: 9

PROJECT: 1664176 / 0001

SHEET 1 OF 1

LOCATION: N 4905372.7 ;E 298705.9

DRILLING DATE: January 18, 2018

DATUM: GEODETIC

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 55

DRILLING CONTRACTOR: Marathon Drilling Co. Ltd.

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

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

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

Drilling Record	Description	Elev. / Depth (m)	Run No.
	Continued From Record of Borehole No. 9	86.48	
NW Casing; NQ Coring	LIMESTONE (GULL RIVER FORMATION) Moderately weathered Thinly to medium bedded Slightly porous Medium strong Grey - A 60 mm clay layer noted at 2.2 m depth.	1.53	1
NQ Coring	LIMESTONE (GULL RIVER FORMATION) Slightly weathered Thinly to medium bedded Slightly porous Medium Strong Grey	84.96 3.05	2
	END OF DRILLHOLE	83.47 4.54	

Discontinuity data:

Type and Surface Description	Jr	Ja
BD,PL,	1	1
BD,UN,Ro	3	1
BD,PL,Ro	1.5	4
BD,PL,Ro	1.5	1
BD,UN,Ro	3	1
BD,UN,Ro	3	1
BD,IR,Ro	3	1
BD,PL,Ro	1.5	1
BD,UN,SM	2	1
BD,PL,Ro	1.5	1
JN,PL,Ro	1.5	1
BD,UN,SM	2	1
BD,PL,Ro	1.5	4
JN,PL,SM	1	1
JN,PL,SM	1	1
BD,PL,Ro	1.5	4
BD,PL,Ro	1.5	1
BD,PL,Ro	1.5	4
BD,PL,Ro	1.5	1
BD,UN,Ro	3	4
BD,UN,Ro	3	1

UCS = 72 MPa

Depth scale: 2, 3, 4, 5, 6, 7, 8, 9, 10

FEATURES LEGEND

BROKEN CORE CLAY SEAM LIMESTONE LOST CORE

DEPTH SCALE

1 : 50

GOLDER

LOGGED: KN

CHECKED: SEMP

GOLDER

Foundation Design

RECORD OF BOREHOLE No 10

SHEET 1 OF 1

METRIC

PROJECT 1664176 / 0002

G.W.P. 4049-11-00

LOCATION N 4905328.5; E 298758.3 MTM NAD ZONE 9 (LAT. 44.287935; LONG. -76.575701)

ORIGINATED BY KN

DIST HWY 401

BOREHOLE TYPE 210 mm O.D. Hollow Stem Augers and NQ Coring

COMPILED BY SEMP

DATUM GEODETIC

DATE January 18 and 19, 2018

CHECKED BY SEMP

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%)
92.8	GROUND SURFACE						
0.0	Clayey silt, some sand, some rootlets, trace gravel (TOPSOIL) Brown Wet	1	SS	103/ 0.23		○ (approx. 45)	
92.4 / 0.4	LIMESTONE (BEDROCK) Bedrock cored from a depth of 0.4 m to 4.5 m For bedrock coring details, refer to Record of Drillhole No. 10	1	RC	REC 68%	92		RQD = 10%
		2	RC	REC 94%	91, 90		RQD = 56%
		3	RC	REC 100%	89		RQD = 87%
88.3 / 4.5	END OF BOREHOLE NOTE: 1. Borehole dry prior to rock coring.						

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L, WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

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

GTA-MTO 001 \\GOLDER.GDS\GAL\MISSISSAUGA\SIM\CLIENTS\MTO\HWY_401_COLLINS_CREEK_BRIDGE\02_DATA\GINT\1664176.GPJ GAL-GTA.GDT 5-10-18 TB

PROJECT: 1664176 / 0002

RECORD OF DRILLHOLE: 10

SHEET 1 OF 1

LOCATION: N 4905328.5 ;E 298758.3

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: January 18 and 19, 2018

DRILL RIG: CME 55

DRILLING CONTRACTOR: Marathon Drilling Co. Ltd.

DATUM: GEODETIC

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

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. / DEPTH (m)	RUN No.
		Continued From Record of Borehole No. 10	92.40	
0–1.51	NW Casing / NQ Coring	LIMESTONE (GULL RIVER FORMATION) Moderately weathered Thinly to medium bedded Slightly porous Medium strong Grey	0.41	1
			91.30 1.51	
1.51–4.46	NQ Coring	LIMESTONE (GULL RIVER FORMATION) Slightly weathered Thinly to medium bedded Slightly porous Medium Strong Grey		2, 3
			88.35 4.46	
		END OF DRILLHOLE		

DISCONTINUITY DATA

TYPE AND SURFACE DESCRIPTION	Jr	Ja
BD,IR,Ro	3	1
BD,IR,Ro	3	1
BD,UN,Ro	3	2
JN,PL,SM	1	1
BD,UN,Ro	3	2
BD,UN,Ro	3	2
JN,PL,Ro	1.5	4
BD,UN,Ro	3	1
BD,UN,Ro	3	1
BD,UN,SM	2	1
BD,UN,Ro	3	1
BD,UN,Ro	3	1
BD,UN,Ro	1	1
BD,IR,Ro	3	1
JN,PL,SM	1	1
BD,IR,Ro	3	1
BD,UN,Ro	3	1
JN,IR,Ro	3	1

UCS = 47 MPa

Column headings: SYMBOLIC LOG; 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; ROCK STRENGTH INDEX R4 R3 R2 R1; WEATHERING INDEX W1 W2 W3 W4 W5 W6; FEATURES; R0/R1 ZONES

Depth scale: 1 2 3 4 5 6 7 8 9

FEATURES LEGEND

BROKEN CORE CLAY SEAM LIMESTONE LOST CORE

DEPTH SCALE

1 : 50

GOLDER

LOGGED: KN

CHECKED: SEMP

GOLDER

Foundation Design

RECORD OF BOREHOLE No 11

SHEET 1 OF 1 **METRIC**

PROJECT 1664176 / 0002

G.W.P. 4049-11-00

LOCATION N 4905368.7; E 299362.8 MTM NAD ZONE 9 (LAT. 44.288301; LONG. -76.568128)

ORIGINATED BY LP

DIST ______ HWY 401

BOREHOLE TYPE 210 mm Hollow Stem Augers and NQ Casing / Coring

COMPILED BY SEMP

DATUM GEODETIC

DATE January 4, 2018

CHECKED BY SEMP

ELEV DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLES – NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
103.4	GROUND SURFACE							
0.0	TOPSOIL (200 mm)							
0.1	SILTY CLAY, trace sand, organic inclusions to 0.7 m, rootlets to 0.7 m Firm to stiff Brown Moist	1	SS	6	103			
		2	SS	12				0 1 45 54
		3	SS	13	102			
		4	SS	100/0.03				
101.1 2.3	LIMESTONE (BEDROCK) Bedrock cored from a depth of 2.3 m to 6.1 m For bedrock coring details, refer to Record of Drillhole No. 11	1	RC	REC 87%	101			RQD = 87%
		2	RC	REC 100%	100			RQD = 74%
		3	RC	REC 100%	99 98			RQD = 95%
97.3 6.1	END OF BOREHOLE							

Plot column headings: STRAT PLOT; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED; 20 40 60 80 100); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60.

NOTES:

1. Water level recorded in open borehole at a depth of about 2.0 m below ground surface (Elev. 101.4 m) upon completion of drilling and prior to rock coring.

2. Water level measurements in standpipe piezometer:

Date	Depth (m)	Elev. (m)
19/01/18	frozen at 0.0	103.4
02/05/18	0.0	103.4

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

RECORD OF DRILLHOLE: 11

PROJECT: 1664176 / 0002
LOCATION: N 4905368.7 ;E 299362.8
INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: January 4, 2018
DRILL RIG: CME 55
DRILLING CONTRACTOR: Marathon Drilling Co. Ltd.

SHEET 1 OF 1
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
		Continued From Record of Borehole No. 11	101.15		
3	NW Casing; NQ Coring	LIMESTONE (GULL RIVER FORMATION) Moderately weathered Thinly bedded Slightly porous Medium strong Grey	2.25	1	BD,IR,Ro PC,M BD,IR,Ro PC,M BD,PL,SM PC,M
4, 5, 6	NQ Coring	LIMESTONE (GULL RIVER FORMATION) Moderately weathered Thinly to medium bedded Slightly porous Very Strong Grey	100.32 3.08	2, 3	BD,UN,Ro BD,IR,Ro BD,UN,Ro BD,IR,Ro BD,UN,Ro BD,UN,Ro BD,CU,Ro BD,UN,Ro JN,IR,Ro BD,IR,Ro
		END OF DRILLHOLE	97.28 6.12		

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

UCS = 160 MPa

Depth scale: 7, 8, 9, 10, 11

FEATURES LEGEND

BROKEN CORE CLAY SEAM LIMESTONE LOST CORE

DEPTH SCALE
1 : 50

GOLDER

LOGGED: LP
CHECKED: SEMP

RECORD OF BOREHOLE No 12

SHEET 1 OF 1 **METRIC**

PROJECT 1664176 / 0002

G.W.P. 4049-11-00 LOCATION N 4905371.9; E 299390.6 MTM NAD ZONE 9 (LAT. 44.288331; LONG. -76.567781) ORIGINATED BY LP

DIST HWY 401 BOREHOLE TYPE 210 mm O.D. Hollow Stem Augers COMPILED BY SMD

DATUM GEODETIC DATE January 4, 2018 CHECKED BY SEMP

ELEV DEPTH	DESCRIPTION (SOIL PROFILE)	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
103.1	GROUND SURFACE							
0.0	TOPSOIL (200 mm)					103		
0.1	SILTY CLAY, trace sand, organic inclusions to a depth of about 1.4 m Firm to very stiff Brown Moist	1	SS	4				
		2	SS	14		102		
		3	SS	23				0 2 38 60
100.9 2.2	END OF BOREHOLE Auger and Split-Spoon Refusal at 2.2 m depth	~~4~~	~~SS~~	~~100/0.03~~	▽	101		

NOTE:

1. Water level recorded in open borehole at a depth of 2.1 m below ground surface (Elev. 101.0 m) 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

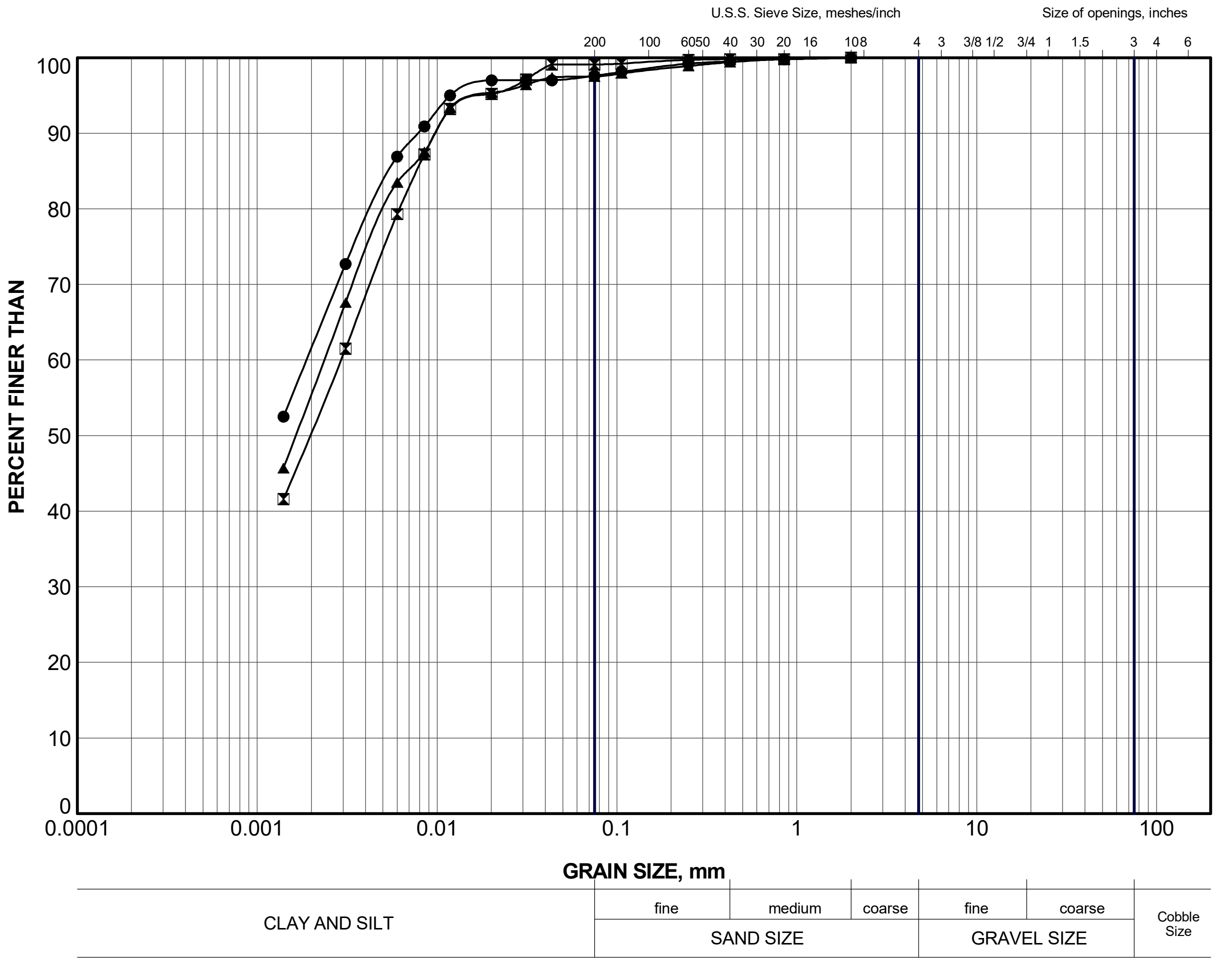
PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m^3

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

APPENDIX B

Laboratory Test Results



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	9	2	86.9
⧗	11	2	102.3
▲	12	3	101.4

PROJECT
HIGHWAY 401 Collins Creek SWM Ponds

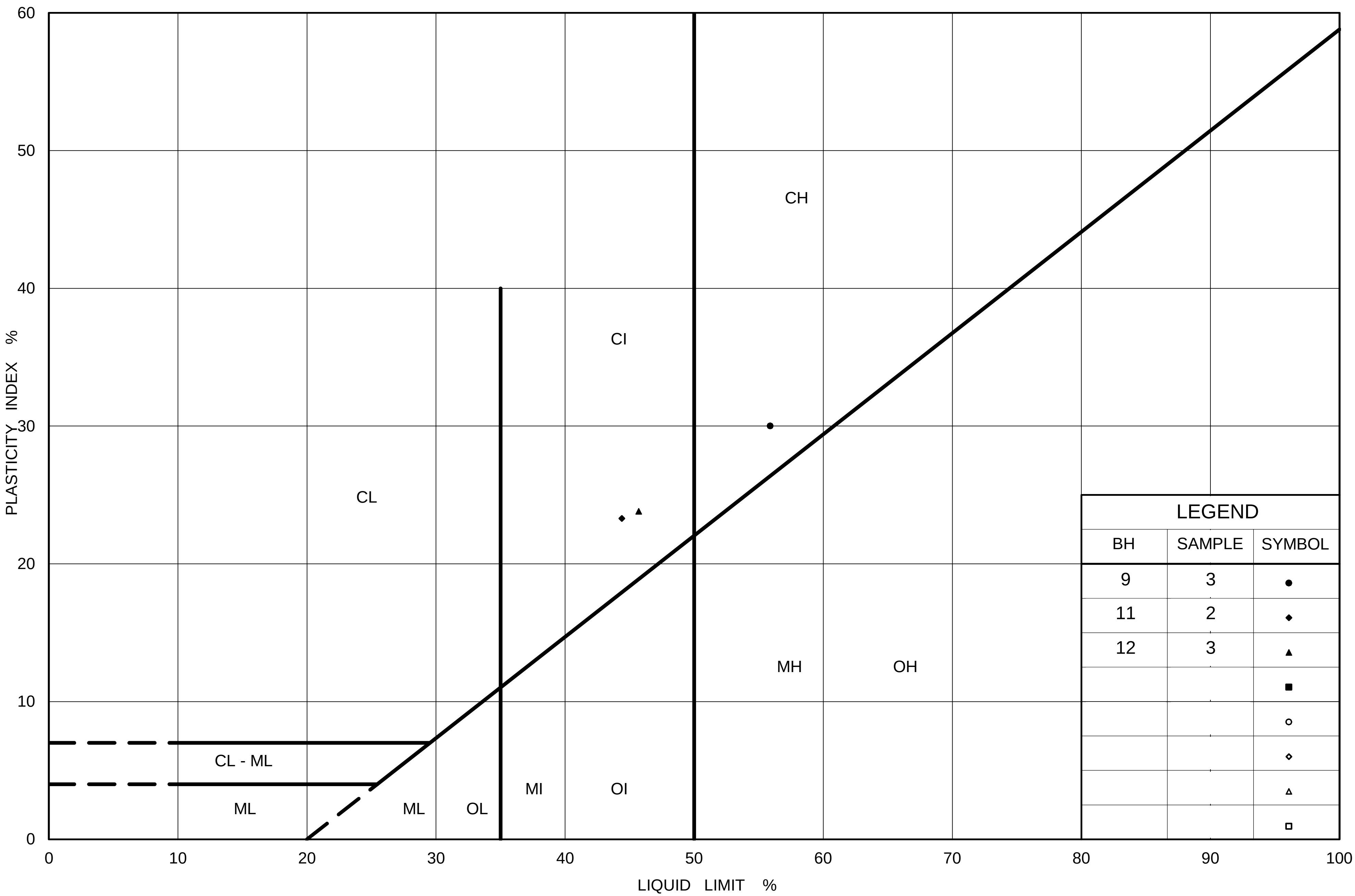
TITLE
GRAIN SIZE DISTRIBUTION
SILTY CLAY

GOLDER
SUDBURY, ONTARIO

PROJECT No.		1664176	FILE No.	1664176.GPJ
DRAWN	TR	May 2018	SCALE N/A	REV.
CHECK	AC	May 2018		
APPR	JMAC	May 2018	**FIGURE B1**	

SUD-MTO GSD (2016) GLDR_LDN.GDT

Oct 75, FF-S-21



Ministry of Transportation

Ontario

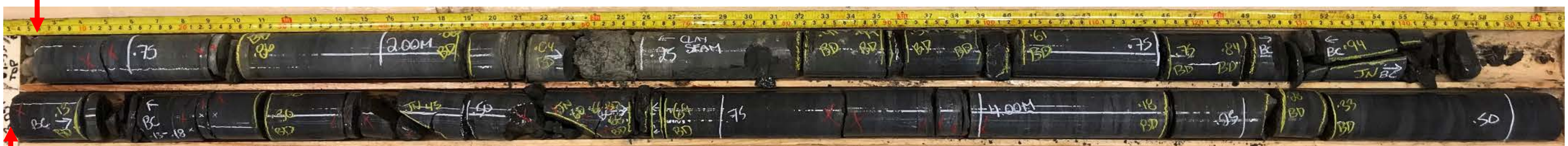
PLASTICITY CHART
SILTY CLAY to CLAY

Figure No. B2

Project No. 1664176

Checked By: SP

Start of Run No. 1 (1.53 m)



Start of Run No. 2 (3.05 m)

End of Borehole (4.54 m)

Borehole 9: Bedrock cored between depths of about 1.53 m to 4.54 m

Start of Run No. 1 (0.41 m)

Start of Run No. 2 (1.51 m)

Start of Run No. 3 (2.98 m)

End of Borehole (4.46 m)

Borehole 10: Bedrock cored between depths of about 0.41 m and 4.46 m

Scale: 0 m — 0.40 m; 0 ft — 1.0 ft — 2.0 ft

PROJECT	**Agreement No.: 9016-E-0007** **Collins Creek Bridge** **Kingston, Ontario**
TITLE	**BEDROCK CORE PHOTOGRAPHS** **BOREHOLES 9 AND 10**

GOLDER

PROJECT No. 1664176			FILE No. ----		
DESIGN	KN	20180507	SCALE	NTS	VER. 1.
CADD	--		**FIGURE B3**		
CHECK	SP				
REVIEW	JMAC	20180507			

Start of Run No. 1 (2.25 m)

Start of Run No. 2 (3.08 m)

Start of Run No. 3 (4.64 m)

End of Borehole (6.12 m)

Borehole 11: Bedrock cored between depths of about 2.25 m and 6.12 m

Scale

0 m — 0.40 m

0 ft — 1.0 ft — 2.0 ft

PROJECT	**Agreement No.: 9016-E-0007** **Collins Creek Bridge** **Kingston, Ontario**				
TITLE	**BEDROCK CORE PHOTOGRAPHS** **BOREHOLE 11**				
GOLDER	PROJECT No. 1664176		FILE No. ----		
	DESIGN	KN	20180507	SCALE NTS	VER. 1.
	CADD	--		**FIGURE B4**	
	CHECK	SP			
	REVIEW	JMAC	20180507		

GOLDER

FIGURE 5A

UNCONFINED COMPRESSION TEST (UC) OF INTACT ROCK CORE SPECIMENS

ASTM D7012

SAMPLE IDENTIFICATION

PROJECT NUMBER	1664176	SAMPLE NUMBER	SA 1
PROJECT NAME	MTO/MERO East Fnd Ret/Ontario	SAMPLE DEPTH, m	3.99-4.18
BOREHOLE NUMBER	BH-9	DATE:	01/29/2018

TEST CONDITIONS

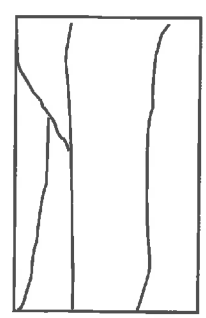
MACHINE SPEED, mm/min	N/A	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.24

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	10.60	WATER CONTENT, (specimen) %	0.17
SAMPLE DIAMETER, cm	4.74	UNIT WEIGHT, kN/m^3	26.01
SAMPLE AREA, cm^2	17.62	DRY UNIT WT., kN/m^3	25.96
SAMPLE VOLUME, cm^3	186.76	SPECIFIC GRAVITY	-
WET WEIGHT, g	495.48	VOID RATIO	-
DRY WEIGHT, g	494.64		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

STRAIN AT FAILURE, %	N/A	COMPRESSIVE STRENGTH, MPa	71.9

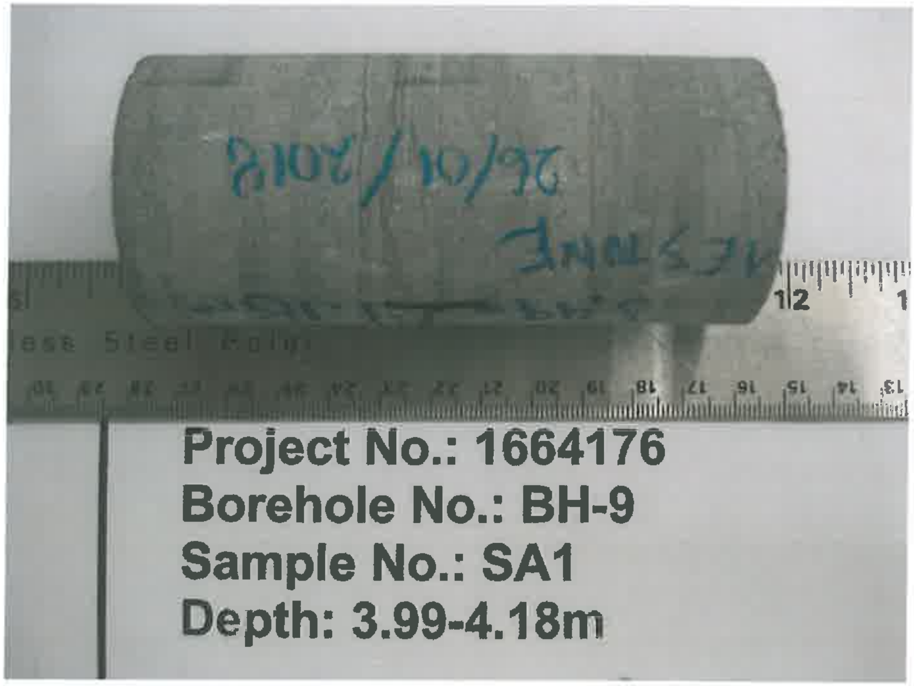
REMARKS:

Checked By:

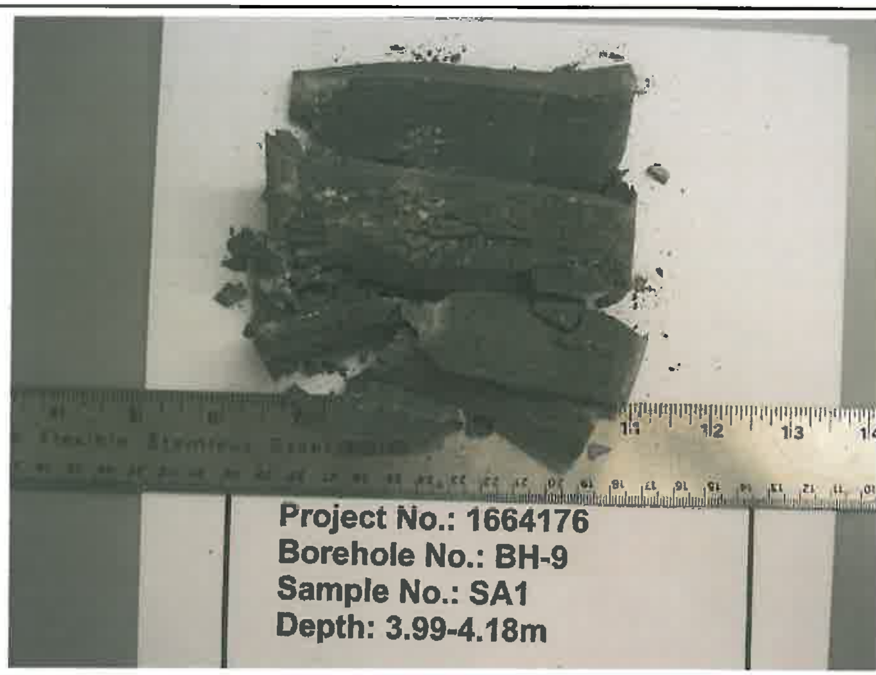
Golder Associates

UNCONFINED COMPRESSION TEST (UC) OF INTACT ROCK CORE SPECIMENS
ASTM D7012

FIGURE 5B



BEFORE COMPRESSION



AFTER COMPRESSION

Date Feb. 5, 2018
Project 1664176

Golder Associates

Drawn Frank
Chkd.

FIGURE 6A

UNCONFINED COMPRESSION TEST (UC) OF INTACT ROCK CORE SPECIMENS

ASTM D7012

SAMPLE IDENTIFICATION

PROJECT NUMBER	1664176	SAMPLE NUMBER	SA 1
PROJECT NAME	MTO/MERO East Fnd Ret/Ontario	SAMPLE DEPTH, m	1.17-1.34
BOREHOLE NUMBER	BH-10	DATE:	01/29/2018

TEST CONDITIONS

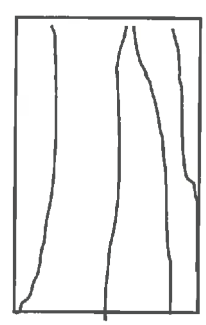
MACHINE SPEED, mm/min	N/A	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.30

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	10.91	WATER CONTENT, (specimen) %	0.01
SAMPLE DIAMETER, cm	4.73	UNIT WEIGHT, kN/m^3	26.34
SAMPLE AREA, cm^2	17.59	DRY UNIT WT., kN/m^3	26.34
SAMPLE VOLUME, cm^3	191.92	SPECIFIC GRAVITY	-
WET WEIGHT, g	515.72	VOID RATIO	-
DRY WEIGHT, g	515.67		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

STRAIN AT FAILURE, %	N/A	COMPRESSIVE STRENGTH, MPa	46.8

REMARKS:

Checked By: [signature]

Golder Associates

UNCONFINED COMPRESSION TEST (UC) OF INTACT ROCK CORE SPECIMENS
ASTM D7012

FIGURE 6B

BEFORE COMPRESSION

AFTER COMPRESSION

Date Feb. 5, 2018
Project 1664176

Golder Associates

Drawn Frank
Chkd. LM

FIGURE 7A

UNCONFINED COMPRESSION TEST (UC) OF INTACT ROCK CORE SPECIMENS

ASTM D7012

SAMPLE IDENTIFICATION

PROJECT NUMBER	1664176	SAMPLE NUMBER	SA 1
PROJECT NAME	MTO/MERO East Fnd Ret/Ontario	SAMPLE DEPTH, m	3.58-3.81
BOREHOLE NUMBER	BH-11	DATE:	01/29/2018

TEST CONDITIONS

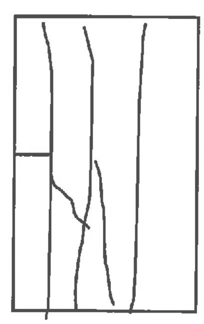
MACHINE SPEED, mm/min	N/A	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.24

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	10.63	WATER CONTENT, (specimen) %	0.01
SAMPLE DIAMETER, cm	4.74	UNIT WEIGHT, kN/m^3	27.25
SAMPLE AREA, cm^2	17.66	DRY UNIT WT., kN/m^3	27.25
SAMPLE VOLUME, cm^3	187.75	SPECIFIC GRAVITY	-
WET WEIGHT, g	521.93	VOID RATIO	-
DRY WEIGHT, g	521.90		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

STRAIN AT FAILURE, %	N/A	COMPRESSIVE STRENGTH, MPa	159.9

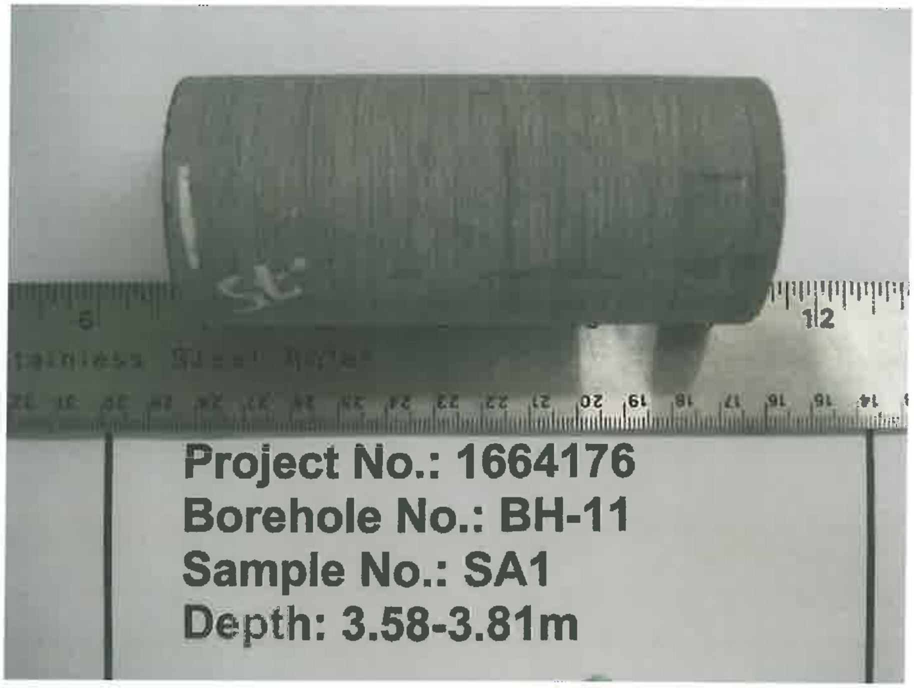
REMARKS:

Checked By: LM

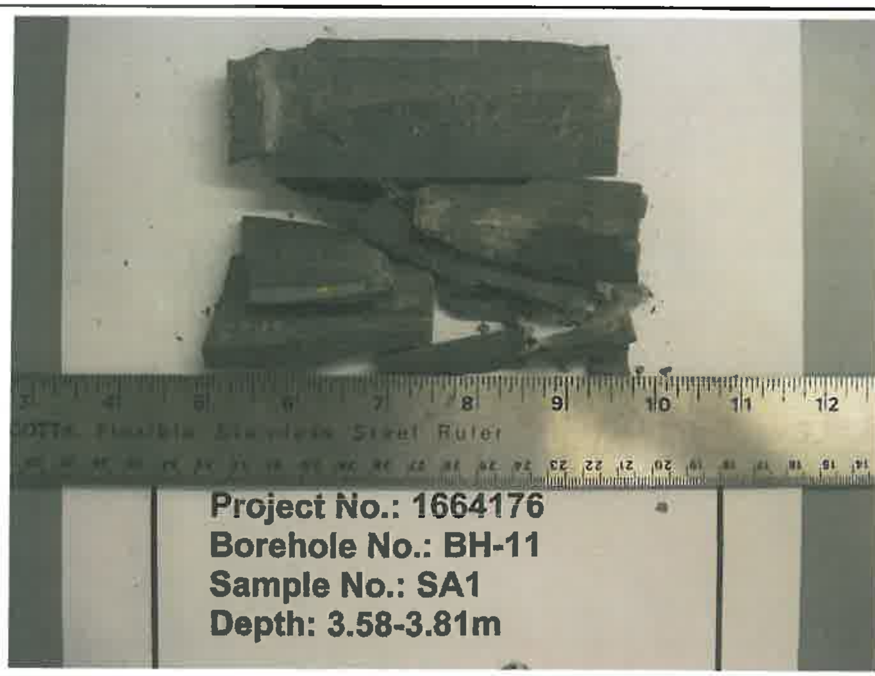
Golder Associates

UNCONFINED COMPRESSION TEST (UC) OF INTACT ROCK CORE SPECIMENS
ASTM D7012

FIGURE 7B



BEFORE COMPRESSION



AFTER COMPRESSION

Date Feb. 5, 2018
Project 1664176

Golder Associates

Drawn Frank
Chkd.

APPENDIX C

Non-Standard and Standard Special Provisions

ROCK EXCAVATION – Item No.

Non-Standard Special Provision

The limestone at Storm water Management Pond #1 and #2 is medium strong and appropriate construction equipment and procedures will be required for excavation into the bedrock. Bedrock excavation shall not disturb the adjacent highway facilities or utilities.

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 Storm Water Management (SWM) Pond #2. The excavation for SWM Pond #2 will extend into the bedrock below the groundwater level at the site. There is a risk of base instability of the constructed liner during the removal of the dewatering system if the system is not turned off progressively (in a controlled manner).

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 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 limestone bedrock below the base at SWM Pond #2 to lower the groundwater to a minimum 0.3 m below the base of the 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 #2 liner 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.

<u>DEWATERING SYSTEM</u> - Item No.
<u>TEMPORARY FLOW PASSAGE SYSTEM</u> - Item No.

Special Provision No. 517F01 July 2017

Amendment to OPSS 517, November 2016

Design Storm Return Period and Preconstruction Survey Distance

517.01 SCOPE

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

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

517.04 DESIGN AND SUBMISSION REQUIREMENTS

517.04.01 Design Requirements

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

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

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

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

Intensity-Duration Factor (IDF) curve location, site specific minimum return period, return period flow estimates, and other information is provided in Table A. The IDF information can be accessed through the MTO IDF Curve Look up Tool on the Drainage and Hydrology page of MTO's website. The return period flow estimates do not include flow volumes from groundwater discharge. The Owner specifically excludes these flow estimates from the warranty in the Reliance on Contract Documents subsection of OPSS 100, MTO General Conditions of Contract.

Table A

IDF Curve Location	Latitude: *					Longitude: *
Temporary Flow Passage Systems						
Site Name / Station Reference	Minimum Return Period (Years)	Return Period Flow Estimates (m^3/s)				Design Engineer Requirements (Note 1)
		2 Year	5 Year	10 Year	25 Year	
**	***	****	****	****	****	*****
Dewatering Systems						
Site Name / Station Reference	Preconstruction Survey Distance (Note 2) (m)					Design Engineer Requirements (Note 1)
**	300					Yes

Note:

1. "Yes" means the design Engineer and design-checking Engineer shall have a minimum of 5 years of experience in designing systems of similar nature and scope to the required work. "No" means a minimum experience level is not required for the design Engineer and design-checking Engineer.
2. "N/A" indicates a preconstruction survey is not required.

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.

Table A (Sample)

<table>
<tr><td colspan="2">IDF Curve Location</td><td colspan="3">Latitude: 44.974844</td><td colspan="3">Longitude: -79.769339</td></tr>
<tr><td colspan="8">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" colspan="2">Design Engineer
Requirements
(Note 1)</td></tr>
<tr><td>2
Year</td><td>5
Year</td><td>10
Year</td><td>25
Year</td></tr>
<tr><td>Woods Creek Culvert
Rehabilitation</td><td>2</td><td>0.7</td><td>3.5</td><td>7.5</td><td>10.9</td><td colspan="2">N/A</td></tr>
<tr><td>Site 32-145
Robbs Creek Culvert Replacement</td><td>10</td><td>1.6</td><td>7.6</td><td>17.4</td><td>25.2</td><td colspan="2">Yes</td></tr>
<tr><td colspan="8">Dewatering Systems</td></tr>
<tr><td>Site Name /
Station Reference</td><td colspan="5">Preconstruction Survey Distance (Note 2)
(m)</td><td colspan="2">Design Engineer
Requirements
(Note 1)</td></tr>
<tr><td>Site 32-145
Robbs Creek Culvert Replacement</td><td colspan="5">300</td><td colspan="2">Yes</td></tr>
<tr><td colspan="8">Note:
1. “Yes” means the design Engineer and design-checking Engineer shall have a minimum of 5 years of experience in designing systems of similar nature and scope to the required work. “No” means a minimum experience level is not required for the design Engineer and design-checking Engineer.
2. “N/A” indicates a preconstruction survey is not required.</td></tr>
</table>

WARRANT: Always with these tender items.

DECOMMISSION OF PIEZOMETERS - Item No.

Non-Standard Special Provision

Standpipe piezometers were installed in boreholes as part of the Foundation Investigation for the Storm Water Management Ponds. The standpipe piezometers installed as part of the Foundation Investigation are listed below; additional information regarding installation details and location are found within the contract documents and the Foundation Investigation Report.

Standpipe Piezometer Identification	Approximate Location		PVC Pipe and Screen diameter / Borehole diameter	Depth (Below Ground Surface) to Tip of Screen
	Northing (m) (Latitude, °)	Easting (m) (Longitude, °)		
9	4,905,372.7 (44.288332	298,705.9 (-76.576359)	50 mm / 210 mm	4.5 m
11	4,905,368.7 (44.288301)	299,362.8 (-76.568128)	50 mm / 210 mm	6.1 m

The standpipe piezometer is registered as Well Tag Number Z289788 (9) and Z280789 (11). The registered owner is the Ministry of Transportation, Ontario.

The standpipe piezometer has been left in place to allow for monitoring of groundwater levels up to construction.

As part of the construction activities the contractor shall properly decommission the standpipe piezometer prior to the start of the trenchless crossing works. The abandonment method for standpipe piezometers must satisfy the minimum requirements of Ontario Regulation 903 Wells, as amended under the Ontario Water Resources Act. In addition, the contractor shall provide a written record of the decommissioning procedure to the Contract Administrator. The record shall include plugging material used, depth of plugging material and limit of the PVC standpipe/screen removal.

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.

Grading - Item No.

Non-Standard Special Provision

After the removal of deleterious materials (topsoil/organics), where berm fill for embankment construction is required, the native silty clay to clay soils shall be proof rolled with a peg foot (also known as Sheepsfoot) type roller prior to berm construction under the extents of the proposed berm footprint. Further all compaction of cohesive soils shall be completed with a peg foot type roller.

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.

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