July 14, 2017

REVISED PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT

SAWMILL CREEK CULVERT - SITE NO. 48E-50/C
HIGHWAY 17, DISTRICT OF THUNDER BAY
TOWNSHIP OF SYINE
MINISTRY OF TRANSPORTATION, ONTARIO
G.W.P 6366-14-00

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REPORT

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PART A

REVISED PRELIMINARY FOUNDATION INVESTIGATION REPORT
SAWMILL CREEK CULVERT – SITE NO. 48E-50/C
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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) was originally retained by Hatch Ltd. (Hatch)(previously Hatch Mott MacDonald), on behalf of the Ministry of Transportation, Ontario (MTO) to provide preliminary foundation engineering services for the replacement of the Sawmill Creek culvert (Site No. 48E-50/C). The final Preliminary Foundation Investigation and Design Report (FIDR) (GWP 6331-14-00, Golder Report No. 1411523-R21 dated September 15, 2015) was submitted to MTO (via Hatch) as part of MTO RFP Assignment Number 6014-E-0009. Based on discussions with Hatch and MTO, we understand that the original highway centreline data provided by the MTO was incorrect. MTO subsequently retained Golder to provide this revised Preliminary FIDR (based on updated highway centreline survey data provided by MTO) under Work Order Assignment #7 of the MTO Northwestern Foundations Retainer Assignment Number 6015-E-0023 and 6015-E-0024.

The Sawmill Creek culvert is located in the District of Thunder Bay in the Township of Syine on Highway 17 at STA 19+312, approximately 18 km generally east of the Highway 17 and Mill Road junction in Terrace Bay, Ontario. The key plan showing the general location of this section of Highway 17 and the location of the investigated area are shown on Drawing 1.

2.0 SITE DESCRIPTION

The Sawmill Creek culvert consists of concrete box the details of which (i.e., width, height, length, etc.) are summarized in Table 1 following the text of the report.

It should be noted that the orientation (i.e., north, south, east, west) stated in the text of the report is typically referenced to project north and therefore may differ from magnetic north shown on the drawing. For the purposes of this report Highway 17 is oriented in a north-south direction for this section of roadway with the culvert perpendicular to the highway in an east-west orientation. In general, the topography in the area of the culvert is relatively flat with steep embankments at the creek location. There is moderate to dense tree cover beyond the highway right-of-way. The area is generally forested with a few private residences and two motels within close proximity to the creek. Sawmill Lake is located to the east of Highway 17 and drains westerly via Sawmill Creek into Jackfish Lake located to the west of Highway 17. At the culvert location, the highway grade is at Elevation 188.2 m and the culvert invert, as provided by MTO, is at Elevation 182.9 m at the inlet and at Elevation 182.5 m at the outlet. The creek water level was at Elevation 183.6 m and 182.9 m at the inlet (east) and outlet (west) ends, respectively, as measured by others on May 12, 2014. The creek water level was at Elevation 183.0 m on the west (outlet) side, as measured by Golder on March 13, 2015. Surface conditions in the culvert inlet and outlet areas are shown on Photographs 1 to 4, attached.

3.0 INVESTIGATION PROCEDURES

The field work for this subsurface investigation was carried out on March 13 and between March 22 and 24, 2015, during which time four boreholes (Boreholes SW-1 to SW-4) were advanced at approximately the locations shown on Drawing 1. Boreholes SW-1 and SW-4 were advanced at the toe of slope near the culvert outlet/inlet, respectively, and Boreholes SW-2 and SW-3 were advanced from the existing highway platform. The boreholes were advanced using a track-mounted CME 55 drill rig, which was supplied and operated by George Downing Estate Drilling Ltd. of Grenville-Sur-La-Rouge, Quebec.

The boreholes were advanced using 108 mm inside diameter hollow stem augers and using NW casing and wash boring techniques. Soil samples were obtained in the boreholes at 0.75 m and 1.5 m intervals of depth using 50 mm outer diameter split-spoon samplers driven by an automatic hammer, in accordance with the Standard Penetration Test (SPT) procedures (ASTM D1586). Sampled of the cohesive soils were obtained using 76 mm O.D. thin walled Shelby Tubes (ASTM D1587) for relatively undisturbed samples. Field vane shear tests were conducted in cohesive soils for determination of undrained shear strengths (ASTM D2573) using MTO Standard ‘N’ size vanes. The groundwater level in the open boreholes was observed during the drilling operations as described on the Record of Borehole sheets in Appendix A. The boreholes were backfilled upon completion in accordance with Ontario Regulation 903 Wells (as amended).

The field work was supervised on a full-time basis by members of Golder’s technical staff who: located the boreholes in the field; arranged for the clearance of underground services; supervised the drilling and sampling operations; logged the boreholes; and examined and cared for the soil samples. The soil samples were identified in the field, placed in labelled containers and transported to Golder’s geotechnical laboratory in Sudbury for further examination and laboratory testing. Index and classification testing consisting of water content determinations, grain size distributions and Atterberg limits were carried out on selected soil samples. The geotechnical laboratory testing was completed according to MTO LS standards.

A sample of the creek water was obtained during the field investigation (on March 24, 2015) using appropriate sampling protocols and submitted to a specialist analytical laboratory under chain of custody procedures for testing for a suite of parameters including pH, resistivity, conductivity, sulphates and chlorides.

The as-drilled borehole locations and ground surface elevations were measured and surveyed by members of our technical staff, referenced to the highway centerline and existing culvert and converted into northing/easting coordinates on the plan drawing. The ground surface elevation of the highway centerline was obtained from the revised profile drawing [drawing E581171 (Revised).dwg] provided by MTO on June 21, 2017. The MTM NAD83 northing and easting coordinates, ground surface elevations referenced to Geodetic datum, and borehole depths at each borehole location are presented on the Record of Borehole sheets in Appendix A and summarized below.

Borehole Number	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface Elevation m)	Borehole Depth (m)
SW-1	5409633.2	309041.7	185.0	11.8
SW-2	5409645.6	309029.9	188.1	10.5
SW-3	5409637.1	309024.8	188.1	10.1
SW-4	5409648.4	309012.3	184.0	15.8

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

Based on Northern Ontario Engineering Geology Terrain (NOEGTS)[1] mapping, the subsoils in the vicinity of the Sawmill Creek culvert site generally consist of glaciolacustrine delta deposits consisting of sand and gravel bordering with areas of bedrock knobs.

Based on geological mapping by the Ministry of Northern Development and Mines (MNDM)[2], the site is underlain by massive granodiorite to granite bedrock of the Archean Era bordering with mafic to intermediate metavolcanic rocks consisting of basaltic and andesite flows, tuffs and breccias, chert and iron formations.

4.2 Subsurface Conditions

The detailed subsurface soil and groundwater conditions encountered in the boreholes and the results of in situ and laboratory testing are given on the Record of Borehole sheets contained in Appendix A. The detailed results of geotechnical laboratory testing are contained in Appendix B. The results of the in situ field tests (i.e., SPT ‘N’ values and undrained shear strengths from field vanes) as presented on the Record of Borehole sheets and in Section 4 are uncorrected. The stratigraphic boundaries shown on the Record of Borehole sheets and on the interpreted stratigraphic profile on Drawing 1 are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. The subsoil conditions will vary between and beyond the borehole locations.

In summary, the subsoil conditions encountered at the site consist of asphalt and granular fill (for boreholes advanced through the embankment) underlain by deposits of peat, silty sand, silt to sandy silt, silty clay to clay, silt to sandy silt, and gravelly silty sand. A more detailed description of the soil deposits and groundwater conditions encountered in the boreholes is provided below.

Deposit/Layer Description	Boreholes	Deposit Thickness (m)	Deposit Surface Elevation (m)	N Values (blows)/ Shear Strength Relative Density or Consistency	Laboratory Testing
Asphalt	SW-2, SW-3	0.140	188.1	n/a	n/a
(FILL)[1] Silt and Sand to Silty Sand, trace to some gravel, trace clay, trace to some organics; brown to grey; frozen/wet	SW-1 to SW-4	1.1 – 5.5	188.0 – 184.0	N = 1 – 33 [2] **Very loose to dense**	w = 6% – 35% 4 – M/MH (Fig. B1)
Silt to Sandy Silt, **Silty Sand;** trace to some clay; grey; wet	SW-1, SW-3, SW-4	0.3 – 1.5	182.9 - 182.8	N = 5 – 7 **Loose**	w = 24% 1 – MH (Fig. B2)

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[1] Northern Ontario Engineering Geology Terrain Study. Ontario Geological Society Electronic Mapping. Map 42DNE

[2] Ministry of Northern Development of Mines. Bedrock Geology of Ontario – West Central Sheet, Ontario Geological Survey – Map 2542

Deposit/Layer Description	Boreholes	Deposit Thickness (m)	Deposit Surface Elevation (m)	N Values (blows)/ Shear Strength	Laboratory Testing
				Relative Density or Consistency	
Peat; black, wet	SW-2	0.2	182.5	n/a	n/a
Silty Clay to Clay, trace sand, trace organics; grey; wet	SW-1 to SW-4	3.3 – 5.5 (Boreholes SW-2 and SW-3 terminated in this deposit)	182.6 – 181.3	N = 0 (weight of rods) – 6 s_u = 12 – >100 [3] S = 1 – 2	w = 53% – 65% w_p = 49% - 69% w_l = 18% - 24% I_p = 31% - 47% 1 – MH (Fig. B3) 5 – AL (Fig. B4)
				Soft to stiff	
Silt to Sandy Silt, trace to some sand, trace to some clay; grey; wet	SW-1, SW-4	5.4 - 7.4	178.6 – 177.1	N = 1 – 14 [4]	w = 20% - 25% 3 – MH (Fig. B5)
				Very loose to compact	
Gravelly Silty Sand; grey; wet	SW-4	1.5 (terminated in this deposit)	169.7	N = 114	n/a
				Very dense	

Where:

N = SPT ‘N’-value; number of blows for 0.3 m of penetration
s_u = Undrained Shear Strength (kPa)
S = Sensitivity
M = Sieve analysis
MH = Combined Sieve and Hydrometer analysis
w = Natural Moisture Content (%)
w_p = Plastic Limit (%)
w_l = Liquid Limit (%)
I_p = Plasticity Index (%)
AL = Atterberg Limits Test

Notes:

[1] In Borehole SW-1, the upper 100 mm of fill consists of topsoil.

[2] In four sampling events within the granular fill, the split spoon did not penetrate the entire SPT depth due to the frozen state of the material. Augers were noted to be grinding on inferred cobbles near bottom of the fill deposit.

[3] The field vane undrained shear strengths measured within the silty clay to clay deposit in Boreholes SW-1 and SW-4 advanced beyond the embankment toe of slope range from about 12 kPa to 24 kPa (soft); whereas the field vane undrained shear strengths measured in Boreholes SW-2 and SW-3 advanced below the existing embankment, generally range from about 31 to 35 kPa (firm). Field vane undrained shear strengths of 62 kPa and >100 kPa were measured at or near the borehole termination depth in Boreholes SW-2 and SW-3. These values are likely due to the proximity of the underlying silt to sandy silt deposit (encountered in Boreholes SW-1 and SW-4) and are not considered representative of the consistency of the silty clay to clay deposit.

[4] In one sampling instance within the silt deposit, the split spoon did not penetrate the entire depth due likely due to the presence of cobbles within the underlying gravelly silty sand deposit as inferred by auger grinding.

Groundwater Conditions

Unstabilized groundwater levels measured in the open boreholes upon completion of drilling are summarized below. The creek water level was measured at Elevation 183.0 m at the west (outlet) side on March 13, 2015. Groundwater and creek water levels in the area are subject to seasonal fluctuations and variations due to precipitation events.

Borehole No.	Depth to Groundwater Level (m)	Groundwater Elevation (m)
SW-1	2.6	182.4
SW-2	5.3	182.8
SW-3	5.3	182.8
SW-4	1.2	182.8

5.0 CLOSURE

The field drilling program was carried out under the supervision of Mr. Rob Ireland under the overall direction of Mr. David Muldowney, P.Eng. This revised Preliminary Foundation Investigation Report was prepared by Mr. Tibor Berecz, and Mr. David Muldowney, P.Eng., provided a technical review of the report. Mr. Jorge M. A. Costa, P.Eng., the Designated MTO Foundations Contact and Senior Consultant, conducted an independent quality control review of this report.

Report Signature Page

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PART B

REVISED PRELIMINARY FOUNDATION DESIGN REPORT
SAWMILL CREEK CULVERT – SITE NO. 48E-50/C
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6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides preliminary foundation design recommendations for the proposed replacement of the Sawmill Creek culvert. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during this subsurface investigation.

The discussion and recommendations presented are intended to provide the designers with sufficient information to assess the feasible foundation alternatives and to carry out the preliminary design of the culvert replacement. Further investigation and analysis may be required during detail design, once the configuration of the proposed culvert and replacement strategy is finalized, to confirm and expand on the preliminary foundation recommendations provided in this report.

Where comments are made on construction, they are provided to highlight those aspects that could affect the future detail design of the project and for which special provisions may be required in the Contract Documents. Those requiring information on the 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.

It is assumed that the existing concrete box culvert is to be replaced with a culvert of similar dimensions, on the same alignment as well as at invert elevations similar to those of the existing culvert. In addition, it is assumed that there will be no embankment grade raise or widening in the area of the culvert as part of the Highway 17 reinstatement.

6.2 Foundations

The Sawmill Creek culvert is located in the District of Thunder Bay in the Township of Syine on Highway 17 at STA 19+312, approximately 18.3 km generally east of Highway 17 and Mill Road junction in Terrace Bay, Ontario. The highway embankment is constructed of granular fill material and is about 4.2 m high relative to the ground surface at the toe of slope adjacent to the culvert (or about 5.5 m relative to the culvert invert) with approximately 3.4 m of soil cover over the existing culvert. The existing culvert consists of a concrete box, the details of which (i.e., width, height, length, etc.) are summarized in Table 1.

Based on the structural inspection report, we understand that about 400 mm of settlement has occurred at the midpoint of the culvert. Since this amount of settlement is not currently reflected at the pavement surface, it is likely that re-grading has occurred in the past. This settlement is likely due to loading of the relatively high embankment over the approximately 5.5 m thick, soft to firm, silty clay to clay deposit encountered at this site. It is the expected that the embankment loading induced long-term consolidation settlements in the cohesive deposit as evidenced by both the observed culvert settlement and the measured strength gain of the cohesive deposit (i.e., higher undrained shear strength measured in the boreholes that penetrate below the existing embankment compared to that measured in the boreholes at the toe of the embankment slopes). Although the primary consolidation settlement of the silty clay to clay stratum under the existing embankment geometry has likely already occurred (based on the date of construction of the existing culvert), secondary (creep) settlement will continue in the long-term, regardless of the culvert replacement, unless settlement mitigation measures are implemented (see Section 6.3.2). The future settlement will occur differentially with the highest settlements occurring at the culvert midpoint. As such, the culvert replacement strategy adopted at this site will need to consider the potential settlement of both the embankment and the replacement culvert.

6.2.1 Foundation Options

Based on discussions with Hatch and a review of the preliminary General Arrangement (GA) drawings, we understand that the following culvert types are being considered at this location:

- A pre-cast concrete box; and
- A pre-cast open footing structure with a pre-cast concrete arch or metal box.

In this report we have considered the following culvert options:

- A pre-cast concrete box;
- An open footing structure supported on either cast-in-place or pre-cast footings; and
- A pipe culvert(s).

We understand that a sheet-pile abutment and concrete cap option is not being considered for culvert replacements on the Highway 17 corridor based on discussions with Hatch and MTO. A pipe culvert, including an elliptical culvert and/or flexible arch culvert is considered feasible at this site but would provide less flow-through capacity compared to a box culvert or open footing culvert with a similar span and would likely require multiple, parallel pipes. If a pipe culvert is selected, a corrugated steel pipe would be preferred over concrete as CSPs are more tolerant of differential settlements. Open footing arch culverts could be considered but would provide less flow-through capacity (depending on the geometry of the arch and the creek water flow) compared to a box or open footing culvert with a similar span and the overall performance of such structures over the longer term is not known.

From a foundation perspective, a box culvert sufficiently wide to handle the creek flow is preferred. Given, the presence of the soft to firm upper zones of the silty clay to clay deposit at this site, and the anticipated settlement associated with the embankment loading, a pre-cast concrete box is recommended over an open footing culvert as box culvert segments are typically more tolerant of different settlement and can also be constructed using a camber. Further, box culverts can accommodate an accelerated construction schedule and there are reduced excavation, dewatering and shoring requirements. If higher geotechnical resistances are required, consideration could be given to founding an open footing culvert on replacement fill after sub-excavation of the full thickness of the cohesive deposit or to founding the culvert on deep foundations in order to eliminate consolidation settlement below the culvert. However, this option would result in differential settlement along the roadway between the embankment over the culvert and the embankment beyond both sides of the culvert, which will continue to settle. Should the anticipated settlement or strain not be tolerable by the preferred culvert type, then a deep foundation alternative or other mitigative measure (see Section 6.3.2) would need to be considered at the detail design stage, when a more detailed settlement/strain analysis is carried out. A comparison of culvert types based on advantages, disadvantages and risks/consequences is presented in Table 2.

6.2.1.1 Box Culvert

It is not necessary to found a box culvert at the standard depth for frost protection purposes, as a box structure is tolerant of small magnitudes of movement related to freeze-thaw cycles, should these occur. However, as indicated above and discussed further in Section 6.3, mitigation of consolidation settlement should be carried out prior to culvert construction. We recommend that the replacement box culvert, if adopted, be founded on the upper zone of the soft to firm silty clay to clay deposit underlying the existing embankment. Recommended foundation elevation and foundation conditions for a replacement box culvert are provided in Table 3.

6.2.1.2 Open Footing Culvert

Strip footings for an open footing culvert should be founded at a minimum depth of 2.2 m below the lowest surrounding grade to provide adequate protection against frost penetration, as per OPSD 3090.100 (Foundation Frost Penetration Depths for Northern Ontario). We recommend placing the footings on the loose silty sand and/or soft to firm silty clay to clay deposit underlying the existing embankment and avoid excavating deeper into the very loose to compact silt to sandy silt stratum. As indicated above and discussed further in Section 6.3, mitigation of consolidation settlement should be carried out prior to culvert construction. Recommended founding elevations and foundation conditions for the replacement open footing culvert are provided in Table 3.

6.2.1.3 Pipe Culvert(s)

It is not necessary to found a pipe culvert at the standard depth for frost protection purposes, as such a culvert is tolerant to small magnitudes of movement related to freeze-thaw cycles, should these occur. However, as indicated above and discussed further in Section 6.3, mitigation of consolidation settlement should be carried out prior to culvert construction. We recommend that the replacement pipe culvert(s), if adopted, be founded on the soft to firm silty clay to clay stratum. Recommended founding elevations and foundation conditions for a pipe culvert are provided in Table 3.

6.2.2 Geotechnical Resistances

The factored geotechnical axial resistance at Ultimate Limit States (ULS) are provided in the following sections. For the replacement culverts founded on the soft to firm silty clay to clay stratum, the geotechnical axial reaction at the Serviceability Limit State (SLS) does not strictly apply as settlement under the culvert will be governed by embankment loading rather than culvert loading. The SLS values given in Table 3 are applicable to loadings from the soil cover over the existing culvert, however the overall settlement will be the result from the full thickness of embankment fill immediately adjacent to the culvert wall or over the footings, as the case may be. Embankment settlement is discussed in Section 6.3.

6.2.2.1 Box Culvert

A box culvert, placed on the properly prepared subgrade and/or properly placed granular pad/backfill at or below the founding elevation identified in Table 3, should be designed based on the recommended factored geotechnical axial resistance at ULS and geotechnical axial reaction at SLS provided in Table 3. For a box culvert, with dimensions similar to those of the existing culvert, the ultimate bearing resistance is governed by the soft to firm silty clay to clay deposit and the undrained shear strength of this deposit is fairly consistent with depth. As such, the width of the culvert will not influence the geotechnical resistance at ULS. As indicated in Section 6.2.2, the geotechnical reaction at SLS does not strictly apply as settlement under the culvert will be governed by embankment loading rather than culvert loading.

The factored geotechnical axial resistance at ULS and geotechnical reaction at SLS are dependent on the configuration and applied loads; the geotechnical resistance/reaction should, therefore, be reviewed if the culvert configuration founding elevation differs from those given in Table 3. The geotechnical resistance/reaction provided in Table 3 are based on loading applied perpendicular to the base of the culvert; where applicable, inclination of the load should be taken into account in accordance with Section 6.7.4 and Section C6.7.4 of the Canadian Highway Bridge Design Code (CHBDC 2006) and its Commentary.

6.2.2.2 Open Footing Culvert

Strip footings placed on the properly prepared subgrade, at or below the founding elevation recommended in Table 3, should be designed based on the factored geotechnical axial resistance at ULS and geotechnical axial reaction at SLS as provided in Table 3. For open footings, the ultimate bearing resistance is provided by/within the silty clay to clay deposit and the undrained shear strength of this deposit is fairly consistent with depth. As such, the width of the footing (i.e., 0.6 m or 1.2 m) will not influence the factored geotechnical axial resistance at ULS. As indicated in Section 6.2.2, the geotechnical reaction at SLS does not strictly apply as settlement under the culvert will be governed by embankment loading rather than culvert loading.

The factored geotechnical axial resistance/reaction are dependent on the configuration and applied loads; the geotechnical axial resistance/reaction should, therefore, be reviewed if the culvert configuration and founding elevation differs from those given in Table 3. The geotechnical resistance/reaction provided in Table 3 are based on loading applied perpendicular to the base of the footings; where applicable, inclination of the load should be taken into account in accordance with Section 6.7.4 and Section 6.7.4 of the CHBDC and its Commentary.

6.2.3 Resistance to Lateral Loads/Sliding Resistance

Resistance to lateral forces/sliding resistance between the base of the box culvert and granular bedding material or between the base of the strip footing and subgrade soil should be calculated in accordance with Section 6.7.5 of the CHBDC. Table 4 provides the coefficients of friction between the base of the culvert/footing and potential interface materials.

6.3 Stability, Settlement and Horizontal Strain

6.3.1 Stability

Limit equilibrium slope stability analysis was carried out using the commercially available program GeoStudio 2007 (Version 7.23), produced by Geo-Slope International Ltd., employing the Morgenstern-Price method of analysis. For the analyses, the Factor of Safety (FoS) of numerous potential failure surfaces was computed in order to establish the minimum FoS. The FoS is defined as the ratio of the forces tending to resist failure to the driving forces tending to cause failure.

The associated strengths and unit weights employed for the slope stability analysis are summarized below.

Soil Deposit	Bulk Unit Weight (kN/m^3)	Effective Friction Angle (°)	Undrained Shear Strength (kPa)
Sand and Silt to Silty Sand (Fill)	19	30	-
Silt to Sandy Silt, Silty Sand	18	28	-
Silty Clay to Clay (at the toe of slope)	17	-	15
Silty Clay to Clay (below the embankment)	17	-	30
Silt to Sandy Silt	18	28	-
Gravelly Silty Sand	19	32	-

The results of the preliminary analyses indicate a FoS of 1.3 is achieved for the existing and reconstructed embankment, constructed with granular fill at a side slope of 2 Horizontal to 1 Vertical (2H:1V) or flatter. Given the presence of the soft zone of the silty clay to clay deposit at the toe of the embankment slopes and the potential for stability issues at this site, a grade raise or widening is not recommended, unless stability mitigation measures are incorporated into the embankment geometry or a ground improvement strategy is adopted.

6.3.2 Settlement

Given that an embankment grade raise or widening is not proposed as part of the culvert replacement, the existing native soils will not experience additional load (provided the culvert soil cover does not exceed 3.4 m); however, as long term consolidation settlement is anticipated to occur due to the existing embankment loading, (and any re-grading as part of maintenance operations) corresponding culvert settlement is also anticipated. As noted in Section 6.3, this settlement will likely occur differentially with the majority of the settlement occurring near the midpoint of the culvert.

To estimate the expected magnitude of settlement of the silty clay to clay deposit under the reconstructed embankment loading, the deformation parameters (i.e., preconsolidation pressure, recompression and compression indices, etc.) presented below were used in the analysis, as determined using empirical correlations based on the laboratory index and soil classification test results (i.e., moisture content, liquid limit, etc.).

σ_p' (kPa)	e_o	C_r	C_c	C_v* (cm^2/s)	$C_{\alpha\varepsilon}$
65	1.52	0.082	0.82	8.3×10^{-4}	5.5×10^{-3}

*in the normally consolidated range

where:	σ_p'	is the preconsolidation stress
	e_o	is the initial void ratio
	C_r	is the recompression index
	C_c	is the compression index
	c_v	is the coefficient of consolidation
	$C_{\alpha\varepsilon}$	is the modified secondary compression index

Based on the results of the preliminary settlement analysis, the estimated consolidation settlement for the existing embankment height of 4.2 m (relative to the ground surface at the toe of the embankment slope adjacent to the culvert) is estimated to be about 680 mm, comprised of 650 mm of primary consolidation and 30 mm of secondary (creep) settlement. These values assume the original embankment was constructed to the full height of 4.2 m after installation of the existing concrete box culvert. As the existing culvert was constructed prior to 1980 (actual date unknown), and about 400 mm of settlement has been experienced at the existing culvert midpoint (based on available information), the silty clay to clay deposit has consolidated since installation of the existing culvert.

Based on the assumed coefficient of consolidation (c_v), it is estimated that about 90 per cent of the primary consolidation settlement would have been be completed in about 2.4 years following embankment construction or following any subsequent grade raise. As such, it is expected that 100 per cent of primary settlement associated has likely already taken place. Therefore, we would not anticipate any additional primary settlement provided the existing embankment grade is maintained or lowered. If re-grading has taken place since the original construction, then additional primary settlement may have occurred.

Based on the modified secondary compression index ($C_{\alpha\varepsilon}$), the magnitude of secondary consolidation (creep) settlement for the cohesive deposit is expected to be about 30 mm per log-cycle of time; however, literature suggests that this value can be underestimated if based on empirical correlations as creep settlement has not been studied extensively. This corresponds to at least 30 mm of creep over the 37 year period since construction of the existing culvert, or more, since the actual date of construction is not known. The proposed replacement culvert is estimated to be subject to an additional 10 mm of creep settlement (for a 20 year design life) as a result of the reconstructed (existing) embankment loading. The estimated creep settlement is based on assumed deformation parameters with no consolidation testing and no information regarding the construction history at this site. As such, a more conservative estimate of 25 mm to 50 mm of creep settlement should be anticipated to occur due to the reconstructed embankment loading at the replacement culvert location over a 20 year design period. The magnitude of creep will become less over time relative to the date of the last loading increase (i.e., re-grading/maintenance). Constructing a box culvert with a camber could mitigate potential culvert settlement over the long term.

Should a grade raise or widening be required, additional long-term settlement (both primary and secondary) would occur and the magnitude of the settlement will be directly related to the magnitude of the grade raise/widening. Further, the additional settlement will likely occur differentially across the length of the culvert. Additional laboratory testing (i.e., consolidation testing) and settlement analysis should be performed at the detailed design stage to more accurately estimate the long-term settlements associated with the selected replacement culvert at this site and especially if an embankment widening or grade raise is proposed. Other options to reduce the magnitude of long-term settlement under the existing or widened/raised embankment could include replacing existing fill with lightweight embankment fill or surcharging prior to culvert replacement. Surcharging may not be practical due to the impact on the roadway.

6.3.3 Horizontal Strain

Based on the anticipated vertical settlements at this site (up to about 50 mm), horizontal strain along the culvert is expected to occur. As a result, the culvert may need to be constructed with a camber to accommodate horizontal strains estimated to be about 0.036 resulting in a total horizontal opening along the length of the culvert up to about 100 mm unless settlement mitigation measures are implemented.

6.4 Lateral Earth Pressures

The lateral earth pressures acting on the side walls (or head/wing walls if required) of the culvert will depend on the type and method of placement of backfill materials, the nature of soils/embankment fill behind the backfill, the magnitude of surcharge including construction loadings, the freedom of lateral movement of the structure, and the drainage conditions behind the walls.

The following recommendations are made concerning the design of the culverts and any wing or head walls. It should be noted that these design recommendations and parameters are applicable to level backfill and ground surface behind the walls. Where there is sloping ground behind the walls, the coefficient of lateral earth pressure must be adjusted to account for the slope.

- Select, free draining granular fill meeting the requirements of OPSS.PROV 1010 (Aggregates) Granular 'A' or Granular 'B' Type I, II or III, but with less than 5 per cent passing the 200 sieve (0.075 mm) should be used as backfill behind the culvert walls, and on top of the culvert for a thickness of up to 300 mm. Backfill should be placed in a maximum of 200 mm loose lift thickness. Weep holes should be installed to allow for positive drainage of the granular backfill. Compaction (including type of equipment, target densities, etc.) should be carried out in accordance with OPSS.PROV 501 (Compacting).
- The granular fill may be placed either in a zone with the width equal to at least 2.2 m behind the back of the walls for a restrained wall (see Figure C6.20(a) of the Commentary to the CHBDC), or within the wedge shaped zone defined by a line drawn at 1.5 H:1V extending up and back from the rear face of the base of the walls for an unrestrained wall (see Figure C6.20(b) of the Commentary to the CHBDC). The pressures are based on the proposed embankment fill material and the following parameters (unfactored) may be used:

Fill Type	**Internal Angle of Friction (ϕ)**	**Unit Weight**	**Coefficients of Static Lateral Earth Pressure**	
			At-Rest, K_o	**Active, K_a**
Granular 'A'	35°	22 kN/m^3	0.43	0.27
Granular 'B' Type II	35°	21 kN/m^3	0.43	0.27
Granular 'B' Type I or III	32°	21 kN/m^3	0.47	0.30

If the structure allows for lateral yielding, active earth pressures may be used in the design of the structure. If the structure does not allow for lateral yielding, at-rest earth pressures should be assumed for design. The movement to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure, may be taken as presented in Table C6.6 of the Commentary to the CHBDC.

6.5 Construction Considerations

6.5.1 Temporary Roadway Protection

The temporary excavation for the culvert replacement will be made through the very loose to dense granular embankment fill and into the native soils, which are comprised of very loose to loose silt to sandy silt, loose silty sand and soft to firm silty clay to clay. All excavations must be carried out in accordance with Ontario Regulation 213, Ontario Occupational Health and Safety Act for Construction Projects (as amended). The granular fill, silt to sandy silt, and silty sand are considered to be Type 3 soil above the groundwater table and Type 4 soil below the groundwater table. The soft to firm silty clay to clay is considered to be Type 4 soil. Temporary open-cut excavations in Type 3 soils should remain stable if side slopes are formed no steeper than 1H:1V. In Type 4 soils, the side slopes should be formed no steeper than 3H:1V.

Temporary protection support systems may be required along the highway to facilitate construction staging and maintain traffic during culvert replacement work. The temporary support systems could consist of driven sheet-piling extended to a suitable depth, or may also consist of soldier piles and lagging where H-piles are driven to a suitable depth and horizontal lagging is installed as the excavation proceeds. Support to the system could be in the form of struts and walers and rakers or anchors. Where required, temporary protection systems should be designed and constructed in accordance with OPSS.PROV 539 (Temporary Protection Systems). Temporary excavation support systems should be designed to Performance Level 2 for any excavation adjacent to existing roadways.

The installation of the sheet-piles for temporary shoring may be impeded by the presence of cobbles, as inferred to be present within the fill material in Borehole SW-2. It may be necessary to excavate and replace the existing fill material in the areas of sheet-pile installation in a series of limited length and narrow trenches. In general, the narrowest suitable excavator bucket should be used. The replacement fill could consist of excavated fill material or imported granular material such as OPSS.PROV 1010 Granular ‘A’ or Granular ‘B’ Type I, II or III provided that 100 per cent of the material passes the 75 mm size and less than 5 per cent passes the 75 µm size. The excavated spoil pile may be re-used to backfill the excavation after removing any cobbles encountered that may otherwise impede the sheet-pile installation. Excavation and replacement should be carried out on the same day to avoid leaving any trench open overnight. Consideration should be given to include an NSSP in the contract to address obstructions; a sample NSSP should be provided at the detail design stage, if required depending on final culvert design and construction staging.

6.5.2 Excavation and Replacement Below Culvert

Prior to placement of any bedding material or concrete, the existing embankment fill, all organics (including peat, topsoil or mixed organic materials) and any disturbed soils, should be sub-excavated from below the plan limits of the proposed works to the founding levels provided in Table 3.

The culvert subgrade should be inspected by a Quality Verification Engineer following sub-excavation to ensure that unsuitable materials have been removed as noted above, in accordance with OPSS 422 (Precast Reinforced Concrete Box Culverts) for a pre-cast box culvert and OPSS 902 (Excavating and Backfilling Structures) for an open footing culvert. Following inspection, the sub-excavated area should be backfilled with granular material meeting the requirements of OPSS.PROV 1010 (Aggregates) Granular ‘A’ or Granular ‘B’ Type II or III that is placed and compacted in accordance with OPSS.PROV 501 (Compacting). The use of Granular

'B' Type II is recommended in wet ground conditions or below water and placement should be in accordance with OPSS.PROV 209 (Embankments over Swamps).

6.5.3 Culvert Bedding and Backfill

6.5.3.1 Box Culvert

The bedding and levelling pad requirements for a pre-cast box culvert should be accordance with OPSS 422 (Precast Reinforced Concrete Box Culverts). Given the potential for surface water flow and groundwater seepage through the relatively permeable embankment fill during excavation to the invert level (including any required bedding/levelling material), and hence potential for subgrade softening, a minimum 300 mm thick layer of OPSS.PROV 1010 (Aggregates) Granular 'B' Type II material should be used for bedding purposes. As the native soil below the bedding is generally fine grained, it is recommended that a non-woven geotextile be placed between the native soil and the bottom of the bedding. The geotextile should meet the specifications for OPSS 1860 (Geotextiles) Class II and have a fabric opening size (FOS) not greater than 212 µm. Above water, the bedding should be placed in maximum 200 mm thick loose lifts and compacted to at least 95 per cent of the standard Proctor maximum dry density (SPMDD) as specified in OPSS.PROV 501 (Compacting). In addition, a 75 mm thick uncompacted levelling pad consisting of OPSS.PROV 1010 (Aggregates) Granular 'A' or concrete fine aggregate meeting the grading requirements specified in OPSS.PROV 1002 (Aggregates – Concrete) should be provided with a geometry similar to that presented in OPSD 803.010 (Backfill and Cover for Concrete Culverts).

If the top of the box culvert is located above the frost penetration depth, depending on the embankment thickness over the culvert and the final size and founding elevation of the culvert, a frost taper should be constructed in accordance with OPSD 803.010 (Backfill and Cover for Concrete Culverts). Although OPSD 803.010 relates to box culverts with spans less than or equal to 3.0 m, a similar frost taper at 10H:1V is considered acceptable for the assumed 6 m wide box culvert replacement option.

If the top of the box culvert is located below the frost penetration depth, a frost taper may not be required from a foundation perspective but may be required from a pavement restoration perspective due to the presence of the moderate to highly frost susceptible silt and sand to silty sand embankment fill.

6.5.3.2 Open Footing Culvert

The excavation and backfilling requirements for the open footing culvert replacement should be in accordance with OPSS 902 (Excavating and Backfilling – Structures). The open footing culvert should be provided with at least 2.2 m of soil cover for frost protection.

Should a pre-cast open footing culvert be the selected replacement option, a bedding layer and levelling pad will be required above the native soil or replacement fill material. The bedding layer and levelling pad for the pre-cast open footings should follow the recommendations as discussed further in Section 6.5.3.1 for the box culvert replacement option.

If the top of the open footing culvert is located above the frost penetration depth, depending on the embankment thickness over the culvert and the final size and founding elevation of the culvert, a frost taper should be constructed in accordance with OPSD 803.010 (Backfill and Cover for Concrete Culverts). Although OPSD

803.010 also relates to an open footing culvert with spans less than or equal to 3.0 m, a similar frost taper at 10H:1V is considered acceptable for the assumed 9 m to 11 m wide open footing culvert replacement options shown on the preliminary GA drawings.

If the top of the open footing culvert is located below the frost penetration depth, a frost taper may not be required from a foundation perspective but may be required from a pavement restoration perspective due to the presence of the moderate to highly frost susceptible silty sand to silt and sand embankment fill.

6.5.3.3 Pipe Culvert(s)

The bedding, levelling and backfill for a concrete pipe, CSP or SP CSPA culvert should be in accordance with OPSD 802.034 (Rigid Pipe Bedding and Cover in Embankment), OPSD 802.014 (Flexible Pipe Embedment in Embankment) or OPSD 802.024 (Flexible Pipe Arch, Embedment in Embankment), respectively, and culvert construction should be in accordance with OPSS 421 (Pipe Culvert Installation in Open Cut). It is important that the backfill at the haunches be well compacted. The pipe culvert should be constructed on a minimum 300 mm thick layer of OPSS.PROV 1010 Granular ‘A’ or Granular ‘B’ Type II material for bedding purposes, however this layer thickness should be confirmed at the detail design stage for the actual culvert type and size selected.

If the top of the pipe culvert(s) is located above the frost penetration depth, depending on the embankment thickness over the culvert and the final size and founding elevation, a frost taper should be constructed with geometry similar to that provided on OPSD 803.031 (Frost Treatment).

If the top of the pipe culvert is located below the frost penetration depth, a frost taper may not be required from a foundation perspective but may be required from a pavement restoration perspective due to the presence of the moderate to highly frost susceptible silty sand to silt and sand embankment fill.

6.5.3.4 Backfill

Backfill behind the culvert walls should consist of granular fill meeting the specifications for OPSS.PROV 1010 (Aggregates) Granular ‘A’ or Granular ‘B’ Type I, II or III, but with less than 5 per cent passing the No. 200 (0.075 mm) sieve. The backfill should be placed in maximum 200 mm thick loose lifts and be compacted to at least 98 per cent of the SPMDD of the materials in accordance with OPSS.PROV 501 (Compacting). The fill should also be placed concurrently on both sides of the culvert, ensuring that the backfill depth on one side does not exceed the other side by more than 400 mm.

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

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

6.5.4 Subgrade Protection

The native sandy silt and/or silty clay to clay subgrade will be susceptible to disturbance from construction traffic and/or ponded water. To limit the effect of this disturbance, a 300 mm compacted bedding layer or 100 mm concrete working slab should be placed immediately after preparation, inspection and approval of the foundation subgrade. The concrete should have a minimum 28 day compressive strength of 20 MPa. Consideration should be given to include an NSSP in the contract to address subgrade protection at this site. An NSSP should be provided at the detail design stage, if required depending on final culvert design and construction staging.

6.5.5 Erosion Protection

Provision should be made for scour and erosion protection at the culvert location. In order to prevent surface water from flowing either beneath the culvert (potentially causing undermining and scouring) or around the culvert (creating seepage through the embankment fill, and potentially causing erosion and loss of fine soil particles), a concrete cut-off wall or clay seal should be provided at the upstream end of the culvert. If a clay seal is adopted, the clay material should meet the requirements of OPSS 1205 (Clay Seal), and the seal should be a minimum thickness of 1 m, if constructed of natural clay or soil bentonite mix. The clay seal should extend from a depth of 1 m below the scour level to a minimum vertical height equivalent to the high water level. The seal should also extend a minimum horizontal distance of 2 m on either side of the culvert inlet. Alternatively, a 0.6 m thick clay blanket may be constructed, extending upstream three times the culvert height and along the adjacent slopes to a height of two times the culvert height or the high water level, whichever is greater.

The requirements for and design of erosion protection measures for the inlet and outlet of the culvert should be assessed by the hydraulics design engineer. As a minimum, rip rap treatment for the outlet of the culvert should be consistent with the standard presented in OPSD 810.010 (Rip Rap Treatment). Erosion protection for the inlet of the culvert should also follow the standard presented in OPSD 810.010 (Rip Rap Treatment) similar to the outlet but with the rip rap placed up to the toe of slope level, in combination with the cut off measures noted above. Similarly, rip rap should be provided over the full extent of the clay seal/blanket, including the creek side slopes and fill slope over the culvert, if a clay seal/blanket is adopted.

6.5.6 Control of Groundwater and Surface Water

Excavation along the culvert alignment will be required to remove the existing embankment fill and potentially a portion of the native soils to achieve the required founding level (or an adequate subgrade level for the sub-excavation backfill) prior to placement of bedding, the actual culvert, culvert backfill and roadway pavement structure. Groundwater flow into the excavation can be expected due to the depth of the excavations and the presence of relatively permeable embankment fill and native silty sand to sandy silt deposits. Therefore, control of groundwater will be necessary to allow for construction to be carried out in dry conditions, where required. Surface water should be directed away from the excavation areas to prevent ponding of water that could result in disturbance and weakening of the foundation subgrade.

Depending on the creek flow, local surface water flow conditions and groundwater level at the time of construction, water flow could be passed through the area by means of a temporary culvert, using a portion of the existing culvert or diverted by pumping from behind temporary cofferdams.

Excavations for box, open footing and pipe culvert options will extend below the creek water level, and the groundwater level, and will require temporary shoring with dewatering to allow for construction/placement of the footings in dry conditions, where required. Temporary shoring and dewatering could be in the form of a cut-off wall or cofferdam advanced to an appropriate depth to control groundwater inflow from the creek. As discussed in Section 6.5.2, Granular 'B' Type II replacement fill/bedding material can be placed sub-aqueously, however, dewatering may still be required as the culvert invert is below the creek water level.

Dewatering of all excavations should be carried out in accordance with OPSS 517 (Dewatering). Consideration should be given to include an NSSP in the contract to address unwatering at this site. An NSSP should be provided at the detail design stage, if required depending on final culvert design and construction staging.

At this preliminary stage, an accurate prediction of the groundwater pumping volumes cannot be made, as the flow rate would be dependent on construction methods adopted by the contractor. However, given the subsurface conditions at the culvert site, it is considered that with prudent control of surface water flows and adequate diversion around the work area minimizing surface water filtration, groundwater pumping volumes likely would not exceed 50 m^3/day during initial drawdown stages and/or during periods of precipitation. For this site, a Permit to Take Water (PTTW) is likely not required.

6.5.7 Obstructions

The contractor should be alerted to the potential presence of cobble size material within the embankment fill material as encountered in Borehole SW-2. An NSSP should be developed at the detail design stage for inclusion into the Contract Documents to alert the contractor to the presence of such obstructions.

6.5.8 Analytical Testing for Construction Materials

The results of an analytical test on a sample of creek water taken at the culvert site are presented in Table B1 in Appendix B. The suite of parameters tested is intended to allow the design engineer to assess the requirements for the appropriate type of cement to be used in construction and the need for corrosion protection of steel reinforcing elements.

6.6 Recommendations for Further Work During Detail Design

During the detail design phase, additional field investigation and testing should be carried out, based on the final configuration and/or alignment of the culvert and the replacement strategy (i.e., staging). In particular, additional laboratory testing (i.e., consolidation testing) and settlement/stability analysis should be performed, even if a grade raise/widening is not being considered. In addition, consideration should be given to drilling additional boreholes advanced to a suitable depth to confirm the thickness of the cohesive deposit and for design of temporary protection works and groundwater control (cut-off) system. The scope and results of this investigation must be reviewed at the time of the detail design to determine if they meet the then-current MTO requirements for the culvert type, groundwater cut-off system or staging strategy under consideration, and if additional investigation and analysis is necessary. It is recommended that a supplemental investigation be carried out at the detail design stage, comprised of two boreholes through the roadway further away from the culvert. Additional Shelby tube samples from the cohesive deposit below the existing highway embankment should also be obtained as part of the investigation. Further, the assessment of and need for an application for a PTTW should be defined early in the detail design phase of the project as not to delay the start of construction.

7.0 CLOSURE

This revised Preliminary Foundation Design Report was prepared by Mr. Adam Core, P.Eng. and the technical aspects were reviewed by Mr. David Muldowney, P.Eng. Mr. Jorge M. A. Costa, P.Eng., Designated MTO Foundations Contact and Senior Consultant, conducted an independent quality control review of this report.

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REFERENCES

Canadian Standards Association (CSA), 2006. Canadian Highway Bridge Design Code and Commentary on CAN/CSA S6 06. CSA Special Publication, S6.1 06.

Occupational Health and Safety Act and Regulation for Construction Projects (as amended).

Ministry of Northern Development of Mines. Bedrock Geology of Ontario – West Central Sheet, Ontario Geological Survey – Map 2542.

Northern Ontario Engineering Geology Terrain Study. Ontario Geological Society Electronic Mapping. Map 42DNE.

ASTM International:

ASTM D1586	Standard Test Method for Standard Penetration Test and Split-Barrel Sampling of Soils
ASTM D1587	Standard Practice for Thin-Walled Tube Sampling for Soils for Geotechnical Purposes
ASTM D2573	Standard Test Method for Field Vane Shear Test in Cohesive Soil

Commercial Software

GeoStudio (Version 7.23) by Geo-Slope International Ltd.

Ontario Provincial Standard Specifications (OPSS)

OPSS 422	Construction Specification for Precast Reinforced Concrete Box Culverts and Box Sewers in Open Cut
OPSS 902	Construction Specification for Excavating and Backfilling – Structures
OPSS 1860	Material Specification for Geotextiles

Ontario Provincial Standard Specifications (OPSS) – Provincial Oriented

OPSS.PROV 421	Construction Specification for Pipe Sewer Installation in Open Cut
OPSS.PROV 209	Construction Specification for Embankments over Swamps and Compressible Soils
OPSS.PROV 501	Construction Specification for Compacting
OPSS.PROV 517	Construction Specification for Dewatering
OPSS.PROV 539	Construction Specification for Temporary Protection Systems
OPSS.PROV 1002	Material Specification for Aggregates - Concrete
OPSS.PROV 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material
OPSS.PROV 1205	Material Specification for Clay Seal

Ontario Provincial Standard Drawings (OPSD)

OPSD 208.010	Benching of Earth Slopes
OPSD 802.014	Flexible Pipe, Embedment in Embankment, Original Ground: Earth or Rock

OPSD 802.024	Flexible Pipe Arch, Embedment in Embankment, Original Ground: Earth or Rock
OPSD 802.034	Rigid Pipe Bedding and Cover in Embankment, Original Ground: Earth or Rock
OPSD 803.010	Backfill and Cover for Concrete Culverts with Spans Less Than or Equal to 3.0 m
OPSD 803.031	Frost Treatment – Pipe Culverts, Frost Penetration Line between Top of Pipe and Bedding Grade
OPSD 810.010	General Rip-Rap Layout for Sewer and Culvert Outlets
OPSD 3090.100	Foundation, Frost Penetration Depths for Northern Ontario

Ontario Water Resource Act:

Regulation 903	Wells (as amended)

Table 1: Summary of Culvert Details

Culvert Location	Site #	Approximate Height of Embankment[1] (m)	Existing Culvert			Approximate Invert Elevation[2]	
			Type	Approximate Dimension[2]	Approximate Length (m)	East End of Culvert (m)	West End of Culvert (m)
Hwy 17 STA 19+312	48E-50/C	4.2	concrete box	6.1 m wide x 1.8 m high	27	182.9	182.5

Notes: 1. Embankment height is relative to existing ground surface at the centreline of the roadway and the ground surface at the toe of the embankment slope (i.e., original ground surface).
2. Culvert dimensions and invert elevations are based on the plan and profile drawings provided by MTO [Drawing E581171 (Revised).dwg].

Prepared by: TB
Checked by: DAM
Reviewed by: JMAC

Table 2: Comparison of Foundation Alternatives

Option	Advantages	Disadvantages	Risks/Consequences
Pre-Cast Box Culvert	■ More tolerant of total and differential settlement due to embankment loading, and can be constructed with a camber to accommodate ongoing/future consolidation settlement of the cohesive deposit. ■ Minimizes depth of excavation, protection system (if required) and dewatering requirements compared to open footing option. ■ Allows faster construction resulting in shorter duration for dewatering and surface water pumping. ■ Backfill/bedding under the culvert may be placed underwater (i.e., Granular ‘B’ Type II) minimizing or eliminating water pumping requirements.	■ Relatively low bearing resistance on the soft to firm silty clay to clay. If higher bearing resistance is required, the silty clay to clay deposit could be sub-excavated and replaced with engineered fill or deep foundations could be considered, however, differential settlement will occur along the roadway embankment above and outside the culvert. ■ May not satisfy fisheries requirements related to natural channel substrate, if applicable. ■ Cut-off wall (or clay seal/blanket) likely required at inlet to mitigate potential scour under culvert. ■ Transportation to and on-site lifting of large pre-cast sections will be required. ■ May require water diversion of a relatively wide creek channel.	■ High risk of disturbance of the native silty clay to clay during construction; can be mitigated with use of a tremie concrete working slab or Granular B Type II working pad. ■ Moderate risk related to anticipated differential settlement; but lower risk compared to open footing option.
Open Footing Culvert	■ Future settlement may be reduced, compared to box culvert option, due to partial sub-excavation of the silty clay to clay deposit to reach the lower founding level. ■ Existing culvert can likely be used for water diversion while new footings are being constructed adjacent to the culvert. ■ May be feasible to construct the culvert on pre-cast footing sections, to accelerate construction schedule and reduce time for dewatering/unwatering (pumping) of surface water. ■ Would likely satisfy fisheries requirements related to natural channel substrate, if applicable.	■ Less tolerant of total and differential settlement due to embankment loading. ■ Relatively low bearing resistance on the soft to firm silty clay to clay. If higher bearing resistance is required, the silty clay to clay deposit could be fully sub-excavated and replaced with engineered fill or deep foundations could be considered, however, differential settlement will occur along the roadway embankment above and outside the culvert. ■ Excavation depths are greater than for a box culvert option, resulting in increased excavation support requirements and additional spoil material to be disposed off-site. ■ Would likely require a tremie plug within a parallel set of sheet pile walls to allow for footing construction in dry conditions. ■ Constructing footings in the dry will take longer due to requirements for installation of a groundwater and surface water control system, dewatering and surface water pumping and excavation in a confined space.	■ High risk of disturbance of the native silty clay to clay during construction; can be mitigated with use of a tremie concrete working slab or Granular B Type II working pad. ■ High risk related to anticipated differential settlement.

Option	Advantages	Disadvantages	Risks/Consequences
Pipe Culvert(s)	■ More tolerant of total and differential settlement due to embankment loading. ■ Minimizes depth of excavation, protection system (if required) and dewatering requirements compared to open footing option. ■ Allows for faster construction resulting in shorter duration for dewatering and surface pumping. ■ Backfill under the culvert may be placed underwater (i.e., Granular 'B' Type II) minimizing or eliminating water pumping requirements.	■ Reduced flow-through capacity, unless multiple pipe culverts are considered. ■ May not satisfy fisheries requirements related to natural channel substrate, if applicable. ■ Cut-off wall or clay seal/blanket may be required at inlet to mitigate potential scour under culvert. ■ CSP does not have as long of design life compared to concrete options. ■ Difficulty in compacting backfill materials to level of culvert springline.	■ High risk of disturbance of the native silty clay to clay during construction; can be mitigated with use of a tremie concrete working slab or Granular B Type II working pad. ■ Moderate risk related to anticipated differential settlement; but lower risk compared to open footing option.

Table 3: Geotechnical Axial Resistance and Reaction for Pre-Cast Box and Open Footing Replacement Culverts

Culvert Location	Approximate Invert Elevation [1] (East End/West End)	Culvert Type	Approximate Backfill/Bedding/Footing Founding Elevation (East End/West End)	Founding Condition	Factored Geotechnical Axial Resistance at ULS [2]	Geotechnical Reaction at SLS for 25 mm of Settlement [2]
Hwy 17 STA 19+312	182.9 m/182.5 m	Pre-Cast Box	182.5 m/182.1 m	Bedding/Levelling Pad over Loose Sandy Silt and/or Firm Silty Clay to Clay	135 kPa	70 [3] kPa
		Open Footing (0.6 m and 1.2 m wide footing)	180.3 m	Firm Silty Clay to Clay	95 kPa	70 [3] kPa
		Pipe Culvert [4]	182.6 m/182.2 m	Bedding Layer over Loose Sandy Silt and/or Stiff to Very Stiff Clay Stratum	N/A	N/A

Notes:

1. Culvert invert elevations are based on the profile drawings provided by MTO [Drawing E581171 (Revised).dwg].
2. The factored geotechnical axial resistance at ULS and geotechnical reaction at SLS (where applicable) based on an assumed 6.0 m wide box culvert and a 0.6 m or 1.2 m wide open footings at the foundation elevations provided above. The recommended geotechnical resistance/reaction should be reviewed if the foundation configuration and/or founding elevations differ from those given above.
3. Mitigation of consolidation (creep) settlement of the silty clay to clay deposit may be required prior to culvert construction. The SLS values given above are associated with 25 mm settlement of culvert due to the loading from the overlying soil cover (i.e., 3.4 m of soil cover) and do not include long-term secondary (creep) settlement associated with the embankment loading, which could be in the order of about 50 mm and is expected to occur under the existing embankment.
4. The founding elevation may need to be adjusted based on the type and size of the pipe culvert and required bedding thickness.

Prepared by: AC
Checked by: DAM
Reviewed by: JMAC

Table 4: Resistance to Lateral Loads/Sliding Resistance for Pre-Cast Box and Open Footing Replacement Culverts

Culvert Location	Pre-Cast Box Culvert or Open Footing		Cast-in-place Open Footing	
	Interface Material	Coefficient of Friction[1] (tan δ)	Interface Material	Coefficient of Friction[1] (tan δ)
Hwy 17 STA 19+312	Compacted Granular Bedding/Levelling Pad	0.45	Soft to Firm Silty Clay to Clay	0.30

Notes: 1. These values are unfactored. In accordance with CHBDC, a factor of 0.8 is to be applied in calculating the horizontal resistances.

Prepared by: AC
Checked by: DAM
Reviewed by: JMAC

MINISTRY OF TRANSPORTATION, ONTARIO

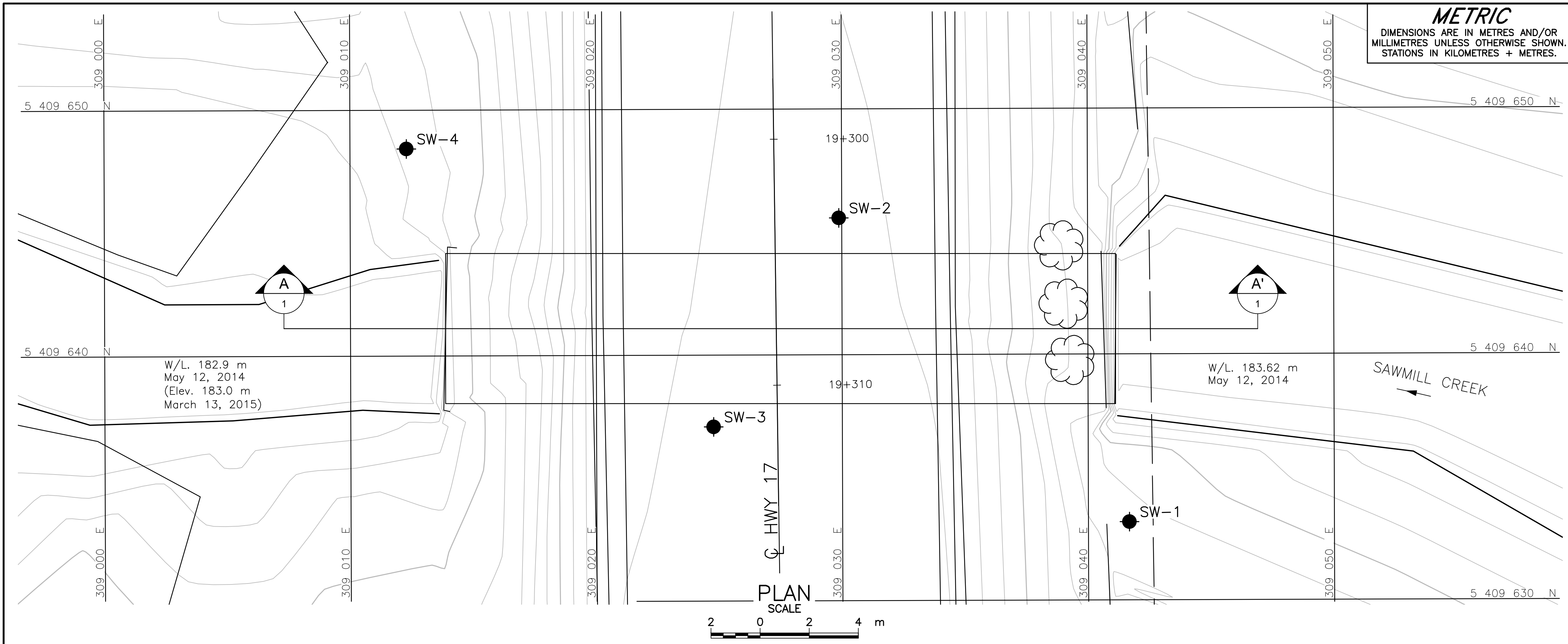
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

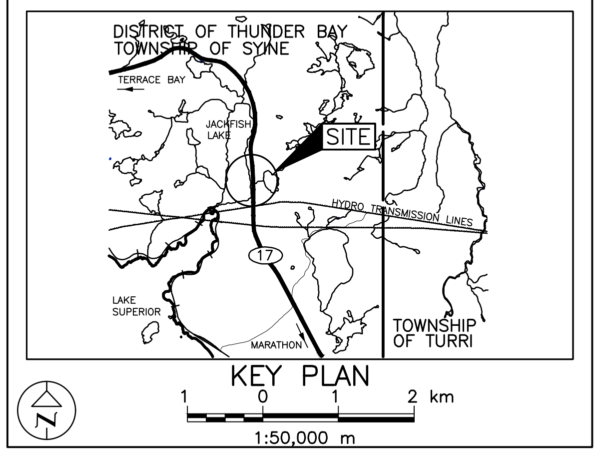
CONT No.
GWP No. 6366-14-00

SHEET
.

HIGHWAY 17
SAWMILL CREEK CULVERT STA 19+312
BOREHOLE LOCATIONS AND SOIL STRATA

Golder Associates





LEGEND

●	Borehole
N	Standard Penetration Test Value
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
▽	WL upon completion of drilling
R	Refusal

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
SW-1	185.0	5409633.2	309041.7
SW-2	188.1	5409645.6	309029.9
SW-3	188.1	5409637.1	309024.8
SW-4	184.0	5409648.4	309012.3

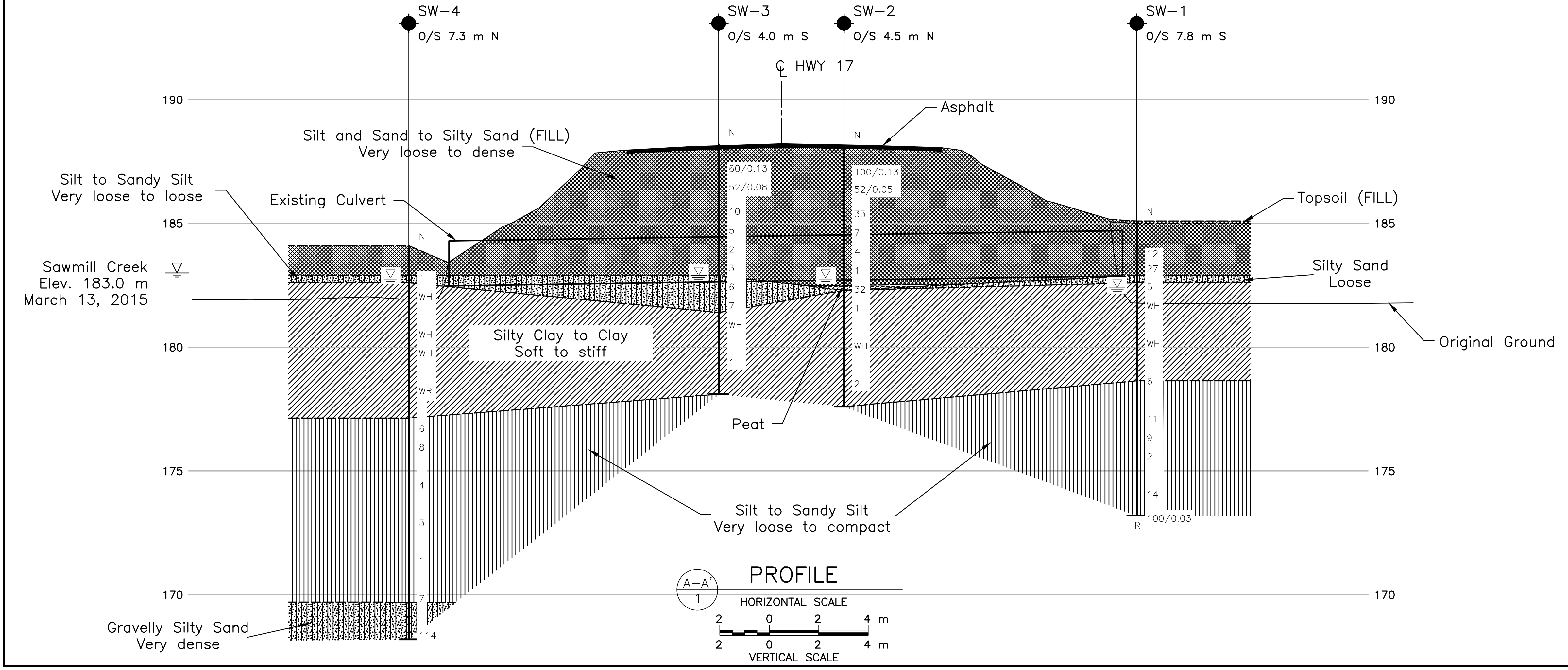
NOTES

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 MTO, drawing file no. E581171 (Revised).dwg, dated MAY 2017, received JUN 21 2017.



D.A.MULDOWNEY
100164716
JUL 14, 2017

J.M.A.COSTA
JUL 14, 2017

NO.	DATE	BY	REVISION
.	.	.	.

Geocres No. 42D-39

HWY. 17		PROJECT NO. 1411523	DIST. .
SUBM'D. AC	CHKD. .	DATE: 7/14/2017	SITE: 48E-50/C
DRAWN: TB	CHKD. DAM	APPD. JMAC	DWG. 1

PLOT DATE: July 14, 2017
FILENAME: H:\Projects\2014\1411523 hmm 26 culverts thunder bay\CIVIL 3D\1411523_Sawmill Creek.dwg

Photograph 1: Sawmill Creek Culvert
East Side - Inlet (Golder – February 24, 2015)

Photograph 2: Sawmill Creek Culvert
West Side - Outlet (Golder – February 24, 2015)

Photograph 3: Sawmill Creek Culvert
East Side - Inlet (Taken from MTO, OSIM_08-29-2012)

Photograph 4: Sawmill Creek Culvert
West Side - Outlet (Taken from MTO, OSIM_08-29-2012)

APPENDIX A

Record of Boreholes

LIST OF SYMBOLS

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

I. GENERAL

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

II. STRESS AND STRAIN

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

III. SOIL PROPERTIES

(a) Index Properties

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

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(d) Shear Strength

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

Notes:

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

2 shear strength = (compressive strength)/2

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

LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

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

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

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

Dynamic Cone Penetration Resistance (DCPT); 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

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

(b) Cohesive Soils

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

IV. SOIL TESTS

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

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

V. MINOR SOIL CONSTITUENTS

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

RECORD OF BOREHOLE No SW-1

1 OF 1 **METRIC**

PROJECT 1411523

G.W.P. 6366-14-00

LOCATION N 5409633.2; E 309041.7

ORIGINATED BY RI

DIST HWY 17

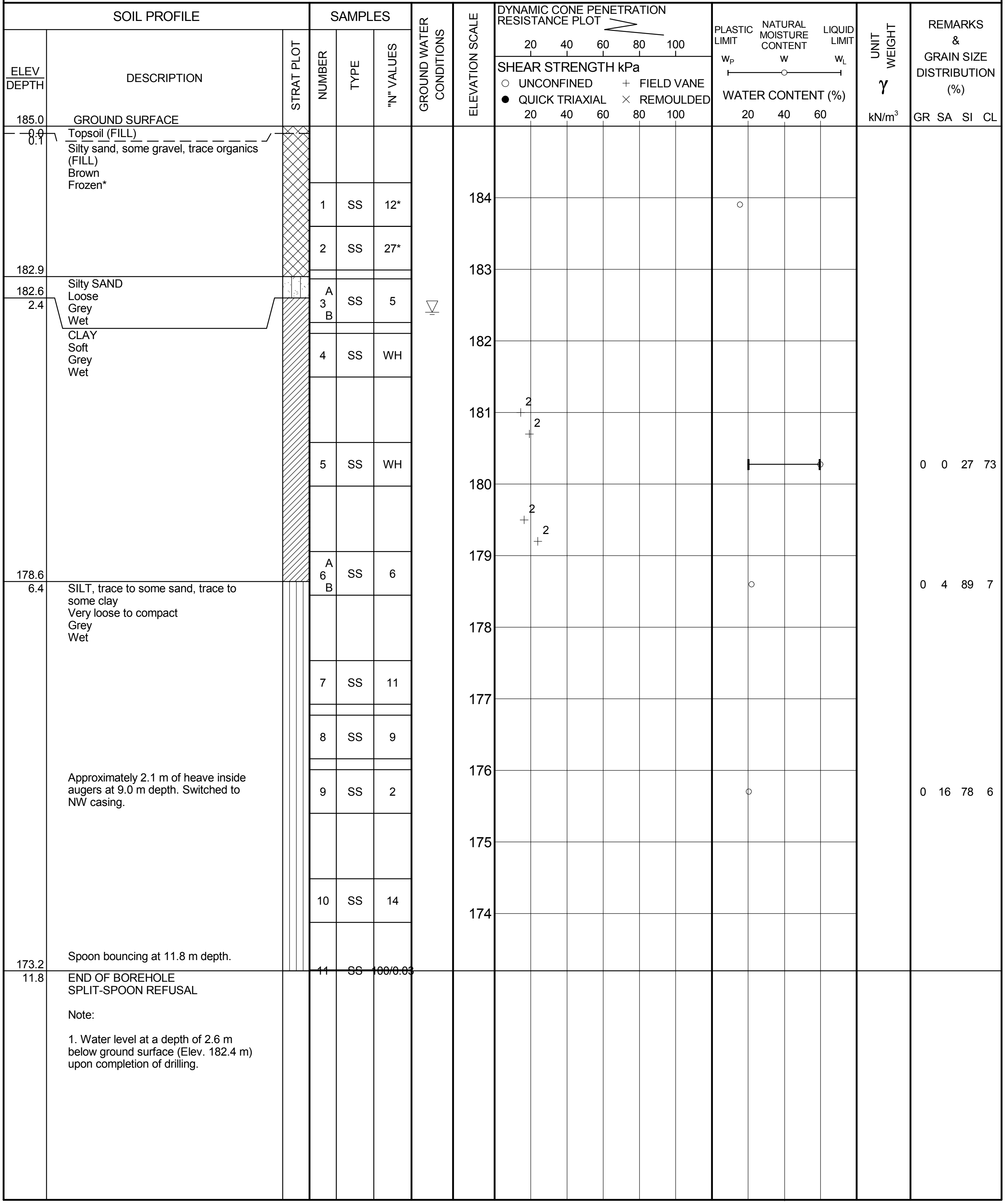
BOREHOLE TYPE 108 mm I. D. Continuous Flight Hollow Stem Augers, NW Casing

COMPILED BY MT

DATUM GEODETIC

DATE March 22 and 23, 2015

CHECKED BY SEMP



ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
185.0	GROUND SURFACE							
0.0	Topsoil (FILL)							
0.1	Silty sand, some gravel, trace organics (FILL) Brown Frozen*	1	SS	12*				
		2	SS	27*				
182.9	Silty SAND Loose Grey Wet	3 A / 3 B	SS	5				
182.6 / 2.4	CLAY Soft Grey Wet	4	SS	WH				
		5	SS	WH	0	0	27	73
		6 A / 6 B	SS	6				
178.6 / 6.4	SILT, trace to some sand, trace to some clay Very loose to compact Grey Wet				0	4	89	7
		7	SS	11				
		8	SS	9				
	Approximately 2.1 m of heave inside augers at 9.0 m depth. Switched to NW casing.	9	SS	2	0	16	78	6
		10	SS	14				
	Spoon bouncing at 11.8 m depth.	11	SS	100/0.03				
173.2 / 11.8	END OF BOREHOLE SPLIT-SPOON REFUSAL							

Note:

1. Water level at a depth of 2.6 m below ground surface (Elev. 182.4 m) upon completion of drilling.

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

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

RECORD OF BOREHOLE No SW-2

1 OF 1 **METRIC**

PROJECT 1411523
G.W.P. 6366-14-00
LOCATION N 5409645.6; E 309029.9
ORIGINATED BY RI
DIST ____ HWY 17
BOREHOLE TYPE 108 mm I. D. Continuous Flight Hollow Stem Augers
COMPILED BY MT
DATUM GEODETIC
DATE March 13, 2015
CHECKED BY SEMP

SOIL PROFILE ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES "N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
188.1	GROUND SURFACE								
0.0	ASPHALT (140 mm)								
0.1	Silty sand, trace to some gravel, trace clay, trace organics (FILL) Very loose to dense Brown Frozen* to wet	1	SS	100/ 0.13*					
		2	SS	52/ 0.05*					
		3	SS	33		3	66	(31)	
		4	SS	7					
		5	SS	4					
		6	SS	1		6	62	28	4
	Auger grinding at 5.3 m depth on inferred cobbles.				▽				
182.5	PEAT Black Wet	A 7 B C	SS	32					
5.8	CLAY, trace sand, trace organics Firm to stiff Grey Wet	8	SS	1					
		9	SS	WH					
		10	SS	2					
177.6 10.5	END OF BOREHOLE								

Note:

1. Water level at a depth of 5.3 m below ground surface (Elev. 182.8 m) upon completion of drilling.

STRAT PLOT

ELEVATION SCALE

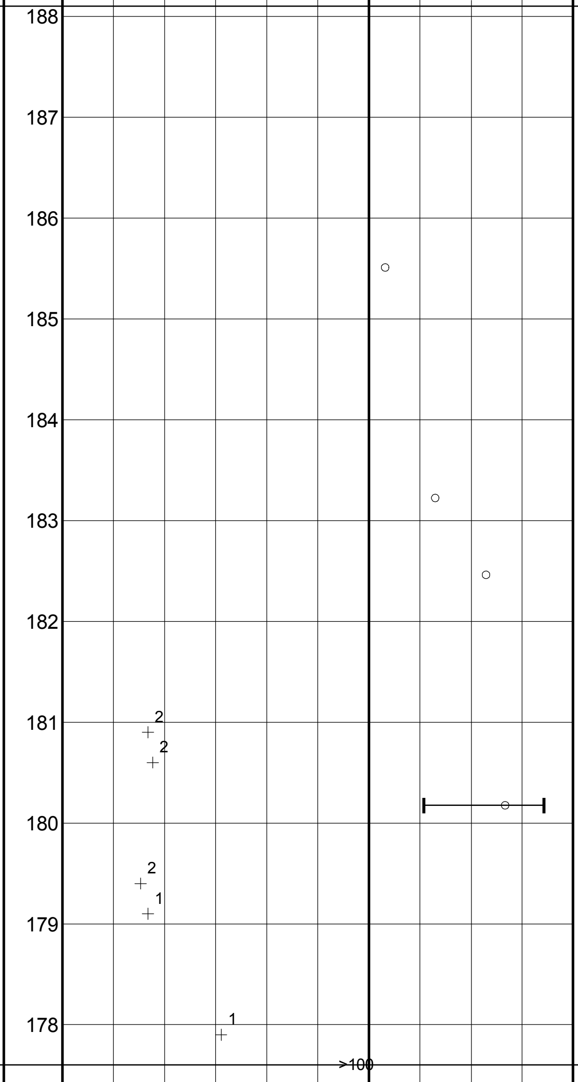
DYNAMIC CONE PENETRATION RESISTANCE PLOT
20 40 60 80 100

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

PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L

WATER CONTENT (%)
20 40 60

UNIT WEIGHT γ kN/m³



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

RECORD OF BOREHOLE No SW-3

1 OF 1 **METRIC**

PROJECT 1411523

G.W.P. 6366-14-00

LOCATION N 5409637.1; E 309024.8

ORIGINATED BY RI

DIST

HWY 17

BOREHOLE TYPE 108 mm I. D. Continuous Flight Hollow Stem Augers

COMPILED BY MT

DATUM GEODETIC

DATE March 13, 2015

CHECKED BY SEMP

SOIL PROFILE

ELEV DEPTH	DESCRIPTION
188.1	GROUND SURFACE
0.0	ASPHALT (140 mm)
0.1	Silt and sand, trace gravel, trace clay, trace organics (FILL) Very loose to compact Brown to grey Frozen* to wet
182.8 5.3	Sandy SILT, trace to some clay Loose Grey Wet
181.3 6.8	CLAY Firm Grey Wet
178.0 10.1	END OF BOREHOLE

Note:

1. Water level at a depth of 5.3 m below ground surface (Elev. 182.8 m) upon completion of drilling.

SAMPLES

NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
1	SS	60/ 0.13*				
2	SS	52/ 0.08*				
3	SS	10				
4	SS	5				
5	SS	2	1	35	60	4
6	SS	3				
7	SS	6				
8	SS	7	0	25	65	10
9	SS	WH				
10	SS	1				

GROUND WATER CONDITIONS

ELEVATION SCALE

DYNAMIC CONE PENETRATION RESISTANCE PLOT
20 40 60 80 100

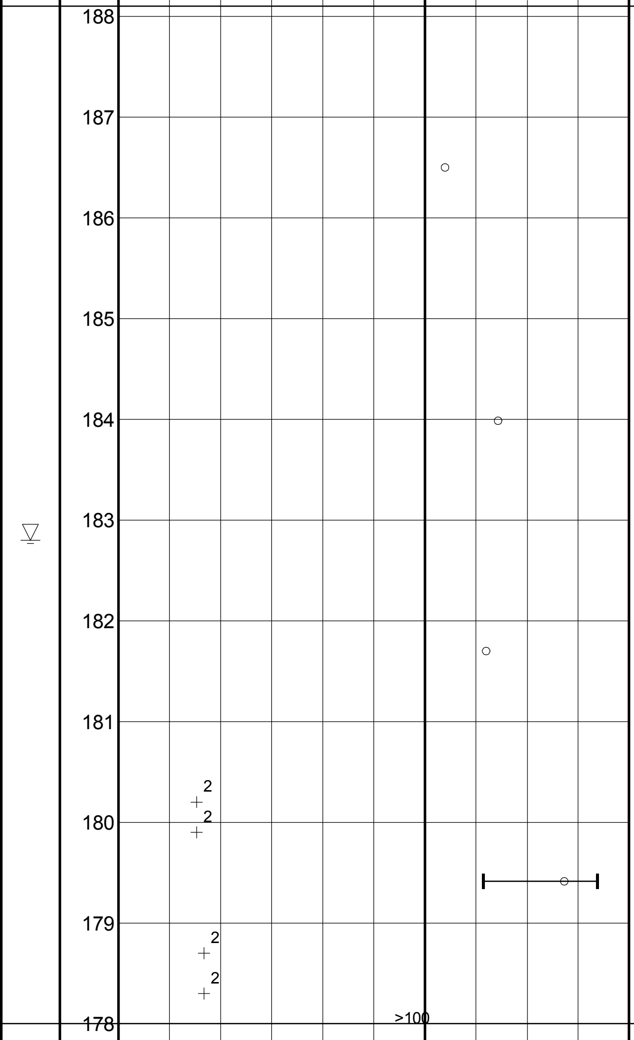
SHEAR STRENGTH kPa
○ UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × REMOULDED
20 40 60 80 100

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, ×3: Numbers refer to Sensitivity

○3% STRAIN AT FAILURE

RECORD OF BOREHOLE No SW-4

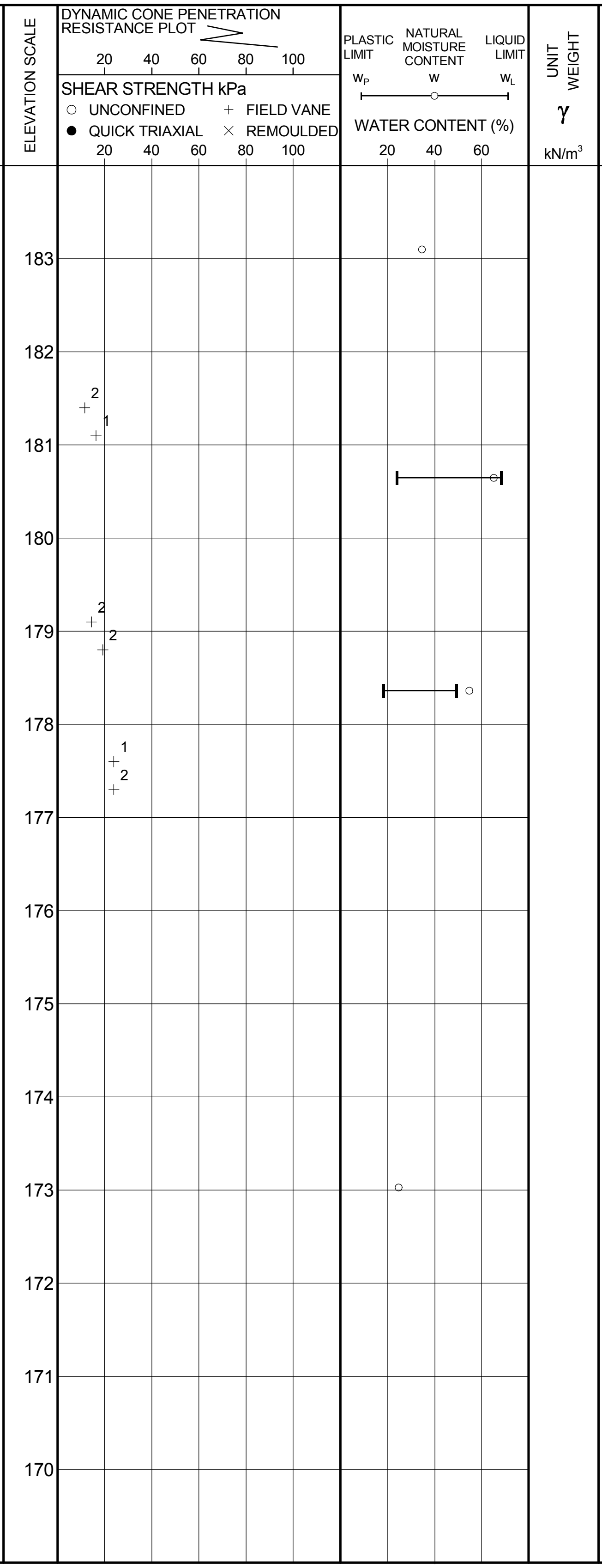
1 OF 2 **METRIC**

PROJECT 1411523
G.W.P. 6366-14-00
DIST ____ HWY 17
DATUM GEODETIC

LOCATION N 5409648.4; E 309012.3
BOREHOLE TYPE 108 mm I. D. Continuous Flight Hollow Stem Augers
DATE March 23 and 24, 2015

ORIGINATED BY RI
COMPILED BY MT
CHECKED BY SEMP

SOIL PROFILE		SAMPLES				REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	GR SA SI CL
184.0	GROUND SURFACE					
0.0	Silt and sand, trace clay, trace gravel, some organics (FILL) Grey Frozen					
182.9		A 1 B	SS	1	▽	2 40 53 5
182.6	Silty SAND Very loose Grey Wet					
1.4	SILTY CLAY to CLAY Very soft to soft Grey Wet Trace organics in Sample 2.	2	SS	WH		
		3	SS	WH		
		4	SS	WH		
		5	SS	WR		
177.1						
6.9	SILT to Sandy SILT, trace clay Very loose to loose Grey Wet	6	SS	6		
		7	SS	8		
		8	SS	4		
		9	SS	3		0 24 72 4
		10	SS	1		
		11	SS	7		
169.7						
14.3						



Continued Next Page

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

RECORD OF BOREHOLE No SW-4

2 OF 2 **METRIC**

PROJECT 1411523
G.W.P. 6366-14-00
DIST ____ HWY 17
DATUM GEODETIC
LOCATION N 5409648.4; E 309012.3
BOREHOLE TYPE 108 mm I. D. Continuous Flight Hollow Stem Augers
DATE March 23 and 24, 2015
ORIGINATED BY RI
COMPILED BY MT
CHECKED BY SEMP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION --- *CONTINUED FROM PREVIOUS PAGE* ---	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	Gravelly Silty SAND Very dense Grey Wet		12	SS	114						
168.2 15.8	Approximately 1.8 m of heave noted in augers at a depth of 15.2 m. END OF BOREHOLE Notes: 1. Water level at a depth of 1.2 m below ground surface (Elev. 182.8 m) upon completion of drilling. 2. Moved 1.2 m west of borehole and retrieved Shelby Tube samples at depths of 2.4 m, 3.0 m and 4.9 m below existing ground surface.										

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

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

APPENDIX B

Laboratory Test Results

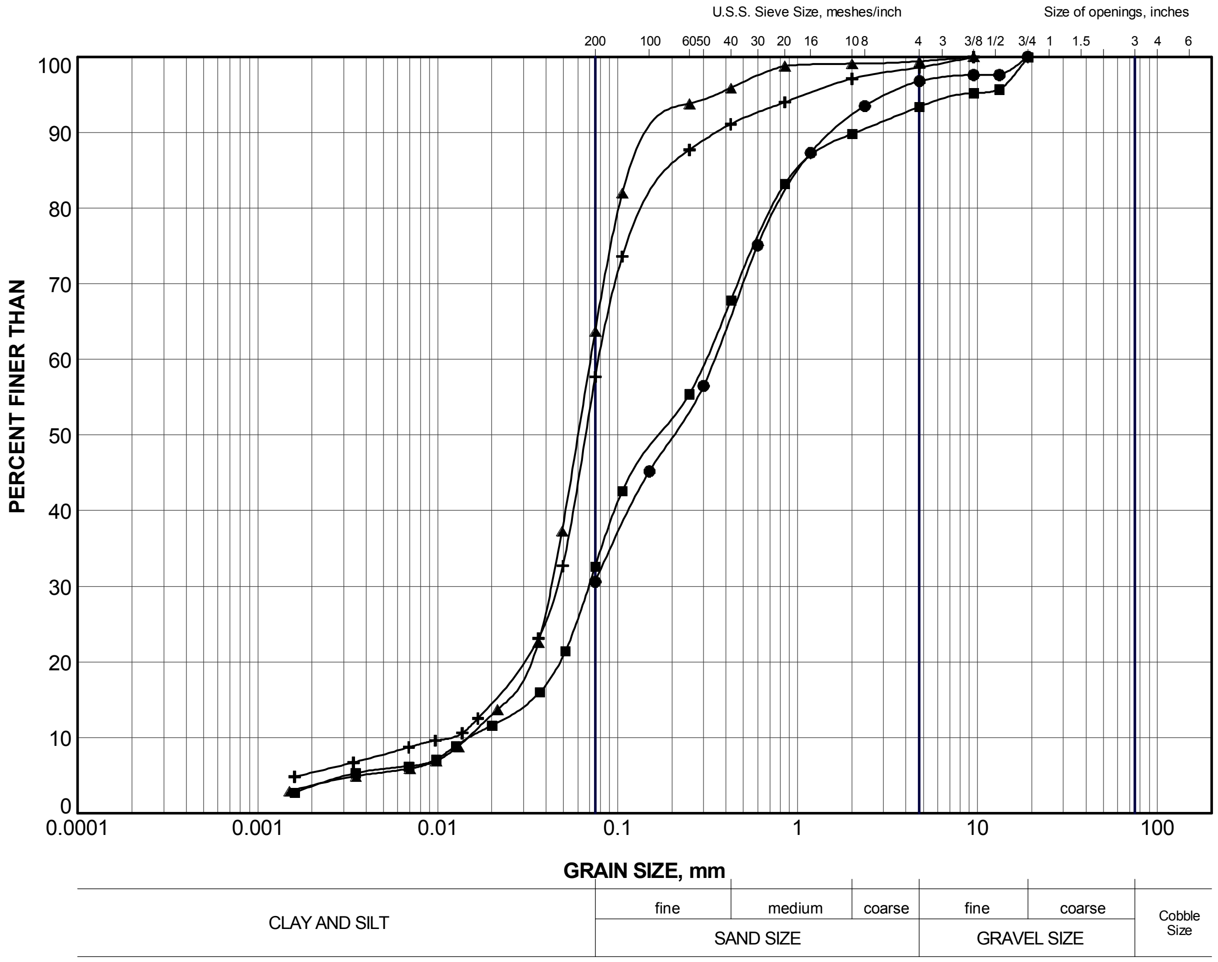
Table B1: Summary of Analytical Testing of Sawmill Creek Water Sample

Parameter	Units	Result
Chloride (CL)	mg/L	0.71
Sulphate (SO4)	mg/L	4.45
Conductivity (EC)	µS/cm	73.9
Resistivity	µohm-cm	<0.33
pH	n/a	7.44

Notes:

1. Sample obtained on March 24, 2015.
2. Analytical testing carried out by ALS Canada Ltd.

Prepared by:	TB
Checked by:	DAM
Reviewed by:	JMAC



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	SW-2	3	185.5
■	SW-2	6	183.2
▲	SW-3	5	184.0
+	SW-4	1A	183.1

PROJECT

HIGHWAY 17
SAWMILL CREEK CULVERT STA 19+312

TITLE

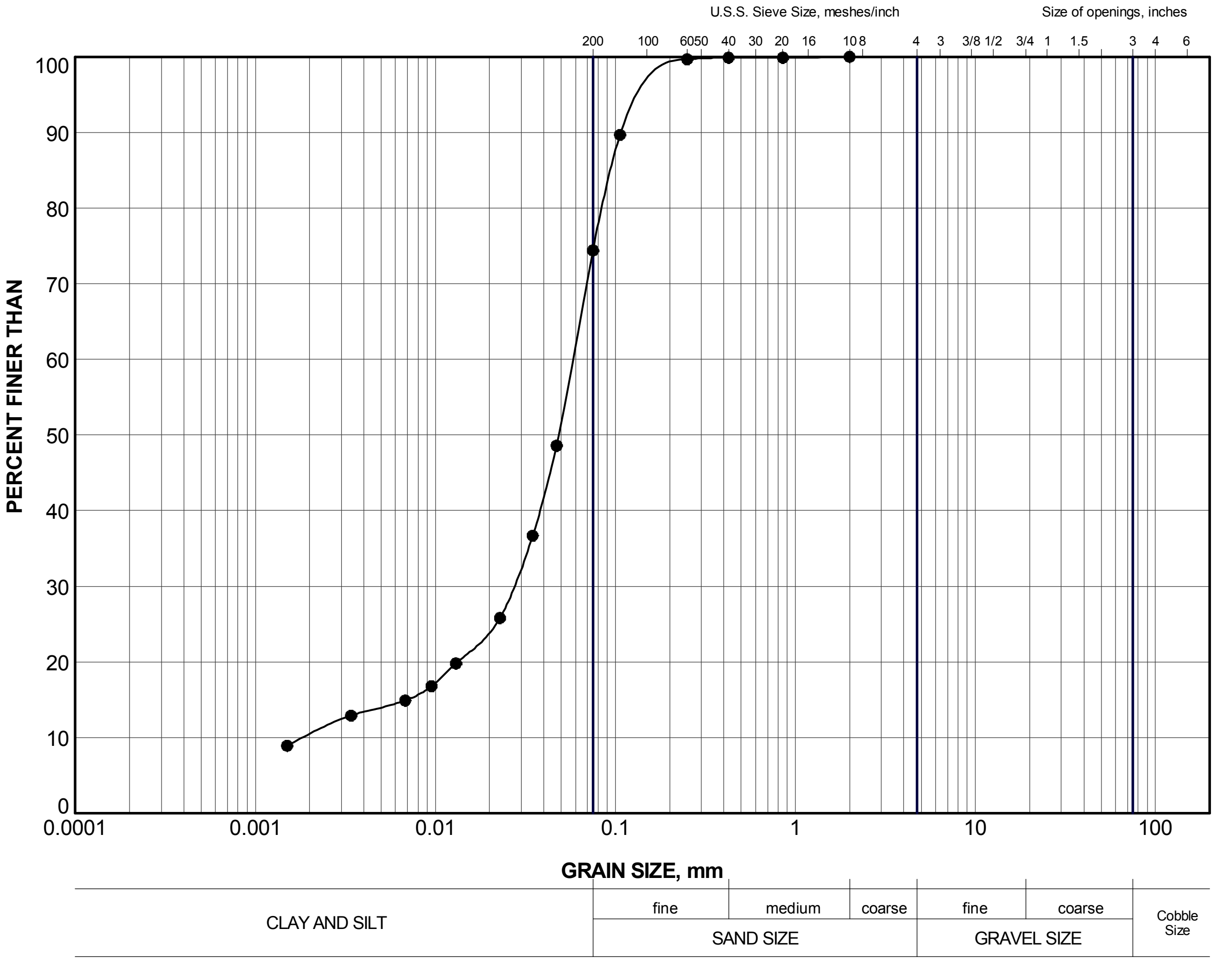
GRAIN SIZE DISTRIBUTION

SILT and SAND to SILTY SAND (FILL)

Golder Associates
SUDBURY, ONTARIO

PROJECT No.		1411523	FILE No.	1411523.GPJ
DRAWN	TB	Jul 2017	SCALE N/A	REV.
CHECK	DAM	Jul 2017	**FIGURE B1**	
APPR	JMAC	Jul 2017		

SUD-MTO GSD (NEW) GLDR_LDN.GDT



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	SW-3	8	181.7

PROJECT

HIGHWAY 17
SAWMILL CREEK CULVERT STA 19+312

TITLE

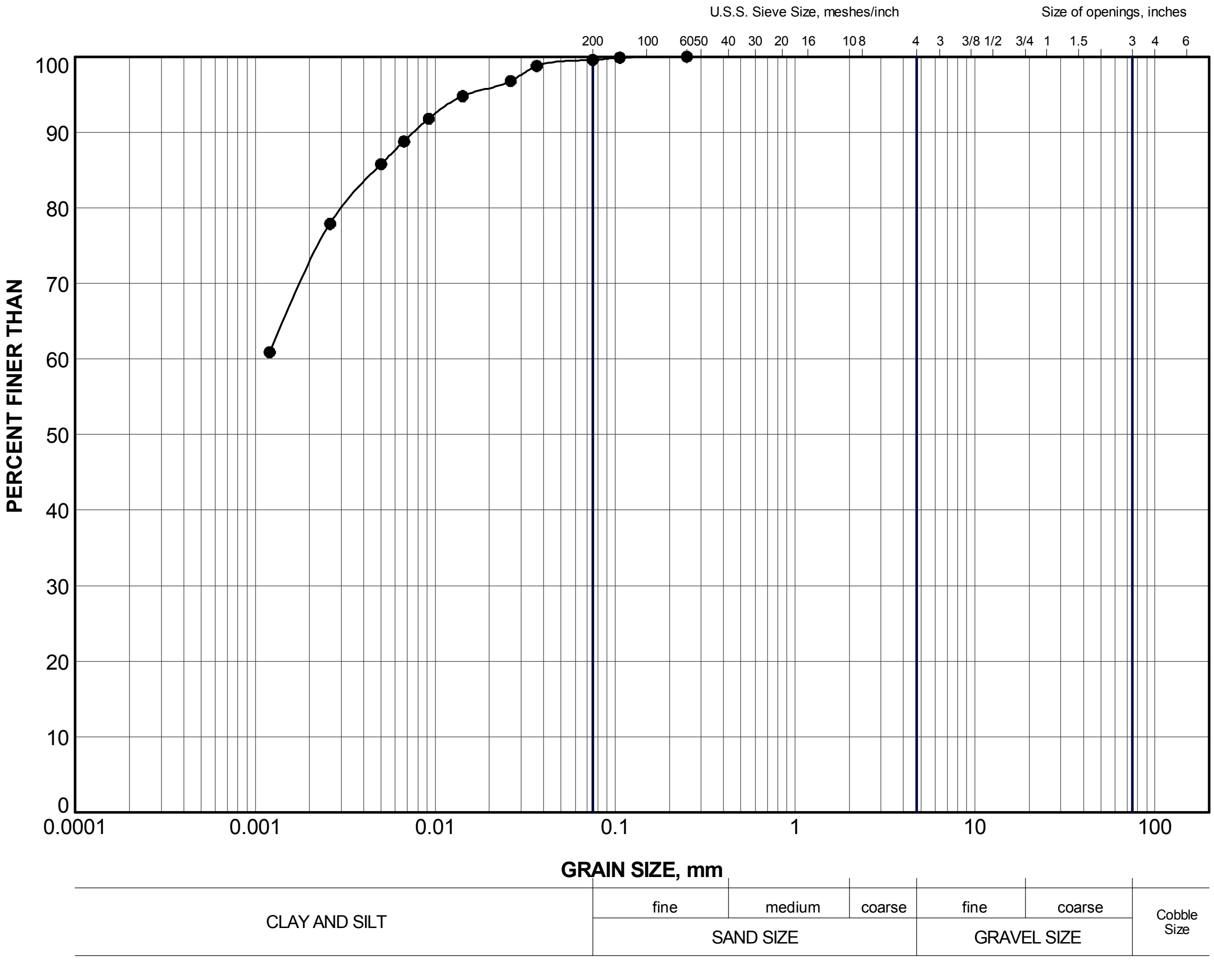
GRAIN SIZE DISTRIBUTION

SANDY SILT

PROJECT No.		1411523	FILE No.	1411523.GPJ
DRAWN	TB	Jul 2017	SCALE N/A	REV.
CHECK	DAM	Jul 2017		
APPR	JMAC	Jul 2017	**FIGURE B2**	

Golder Associates
SUDBURY, ONTARIO

SUD-MTO GSD (NEW) GLDR_LDN.GDT



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	SW-1	5	180.3

PROJECT

HIGHWAY 17
SAWMILL CREEK CULVERT STA 19+312

TITLE

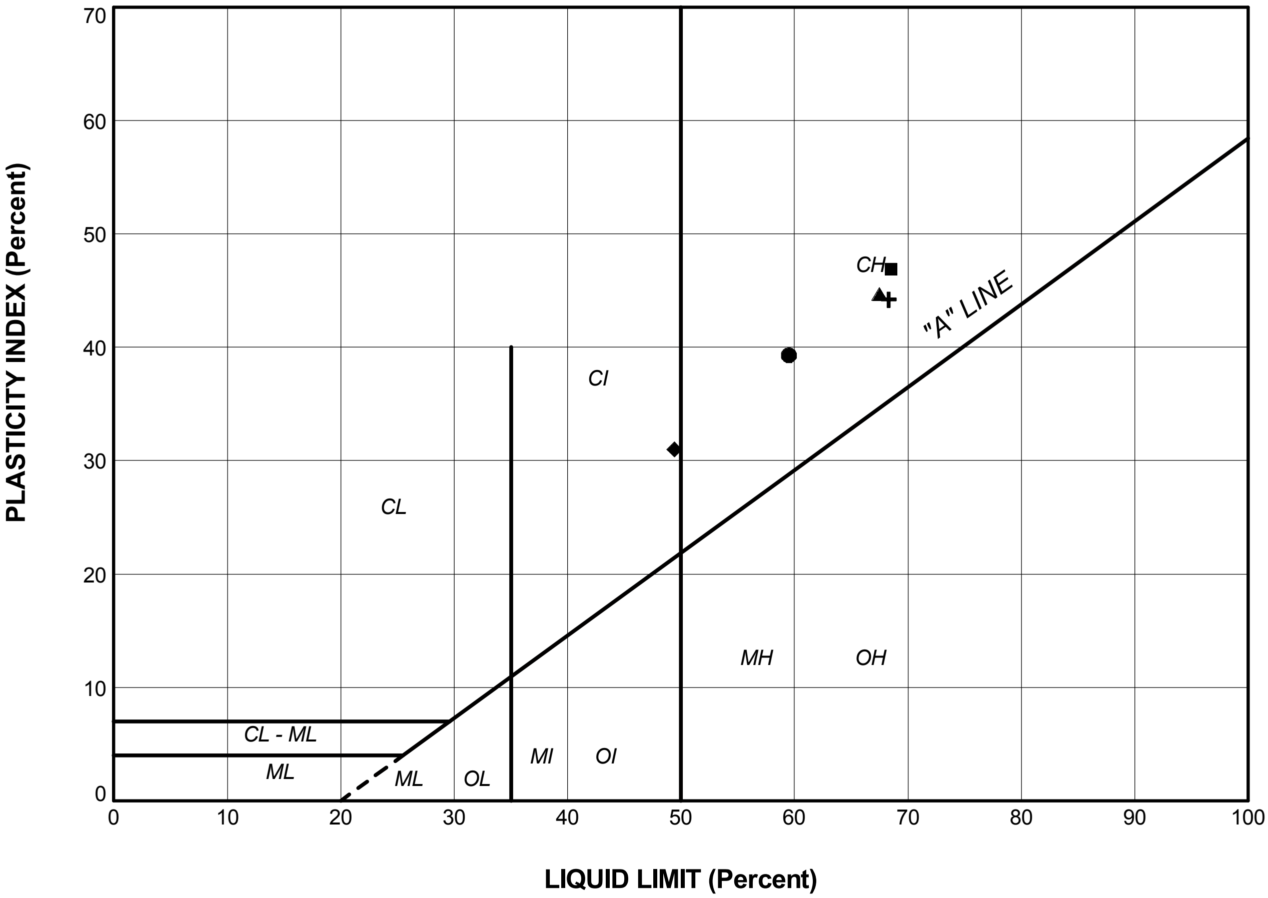
GRAIN SIZE DISTRIBUTION

CLAY

Golder Associates
SUDBURY, ONTARIO

PROJECT No.		1411523	FILE No.	1411523.GPJ
DRAWN	TB	Jul 2017	SCALE N/A	REV.
CHECK	DAM	Jul 2017		
APPR	JMAC	Jul 2017	**FIGURE B3**	

SUD-MTO GSD (NEW) GLDR_LDN.GDT



SOIL TYPE
C = Clay
M = Silt
O = Organic

PLASTICITY
L = Low
I = Intermediate
H = High

LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	SW-1	5	59.5	20.2	39.3
■	SW-2	9	68.5	21.6	46.9
▲	SW-3	10	67.5	22.8	44.7
+	SW-4	3	68.3	24.1	44.2
◆	SW-4	5	49.4	18.4	31.0

PROJECT

HIGHWAY 17
SAWMILL CREEK CULVERT STA 19+312

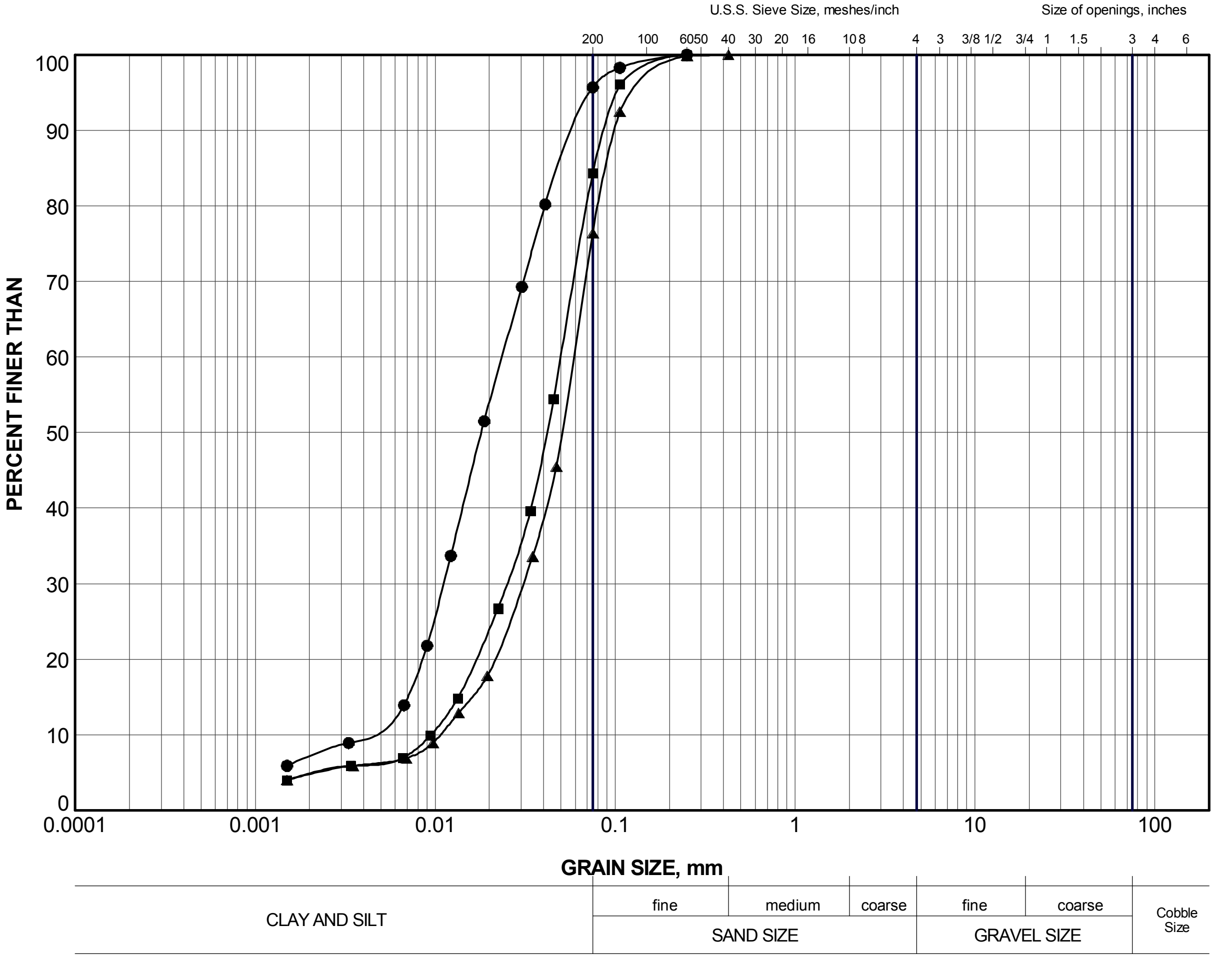
TITLE

PLASTICITY CHART

SILTY CLAY to CLAY

Golder Associates
SUDBURY, ONTARIO

PROJECT No.		1411523	FILE No.	1411523.GPJ
DRAWN	TB	Jul 2017	SCALE N/A	REV.
CHECK	DAM	Jul 2017		
APPR	JMAC	Jul 2017	**FIGURE B4**	



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	SW-1	6B	178.8
■	SW-1	9	175.7
▲	SW-4	9	173.0

PROJECT

HIGHWAY 17
SAWMILL CREEK CULVERT STA 19+312

TITLE

GRAIN SIZE DISTRIBUTION

SILT to SANDY SILT

Golder Associates
SUDBURY, ONTARIO

PROJECT No.		1411523	FILE No. 1411523.GPJ
DRAWN	TB	Jul 2017	SCALE N/A REV.
CHECK	DAM	Jul 2017	
APPR	JMAC	Jul 2017	**FIGURE B5**

SUD-MTO GSD (NEW) GLDR_LDN.GDT

As a global, employee-owned organisation with over 50 years of experience, Golder Associates is driven by our purpose to engineer earth's development while preserving earth's integrity. We deliver solutions that help our clients achieve their sustainable development goals by providing a wide range of independent consulting, design and construction services in our specialist areas of earth, environment and energy.

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