



May 16, 2016

DETAIL FOUNDATION INVESTIGATION AND DESIGN REPORT

**BRULE CREEK CULVERT - SITE NO. 48W-249/C
HIGHWAY 11/17, DISTRICT OF THUNDER BAY
TOWNSHIP OF CONMEE
MINISTRY OF TRANSPORTATION, ONTARIO
G.W.P 6943-10-00 WP 6943-10-01**

Submitted to:

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REPORT





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PART A

**DETAIL FOUNDATION INVESTIGATION REPORT
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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by Hatch, on behalf of the Ministry of Transportation, Ontario (MTO) to provide detail foundation engineering services for the replacement of the twin Brule Creek culvert (Site No. 48W-249). The Brule Creek culverts are located in the District of Thunder Bay in the Township of Conmee on Highway 11/17 at about STA 15+304, approximately 30 m south of the Highway 17 and Mokomon Road junction in Mokomon, Ontario (approximately 39 km Northwest of Thunder 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 existing Brule Creek culvert consists of two Structural Plate Corrugated Steel Pipes (SP CSP), the details of which (i.e., diameter, 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 11/17 is oriented in a north-south direction for this section of roadway and the culvert is oriented perpendicular to the highway in an east-west orientation.

In general, the topography in the area of the culvert is undulating terrain with moderate to dense tree cover beyond the highway right-of-way, and the land use is essentially rural with some rural residences located on the southwest side of site. The highway to the west of Mokomon Road and to the east of the culvert site appears to be within a cut section, and the culvert site is within a fill section. Brule Creek parallels Mokomon Road at the culvert location and flows west-east, draining into the Kaministiquia River which flows southeast and drains into Lake Superior. At the culvert location, the highway grade is at approximately Elevation 382.5 m. The existing culvert inverts, as provided by MTO, are approximately Elevation 374.3 m at the inlet and Elevation 374.2 m at the outlet. The existing embankment is approximately 8.2 m to 8.3 m high, with side slopes inclined at approximately 1.5 Horizontal to 1 Vertical (1.5H:1V) on the west side and approximately 2H:1V on the east side. The creek water level was at Elevation 375.6 m at the inlet (west), as measured by others on May 6, 2013, and measured at Elevations 375.4 m and 374.6 at the inlet and outlet ends, respectively, on December 17, 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 between December 8 and 21, 2015, January 16, and February 20, 2016, during which time eight boreholes (Boreholes BR-1 to BR-7 and BR-6A) were advanced at approximately the locations shown on Drawing 1.

The field investigation was carried out using a variety of drilling equipment due to the nature of the terrain and access constraints at the Brule creek site. The details of the drilling equipment and suppliers are listed below.



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Drilling Equipment	Borehole (s)	Supplied and Operated By
Track Mounted - CME 55	BR-1, BR-3 and BR-6	RPM Drilling Ltd. of Thunder Bay, Ontario
All-Terrain Vehicle - CME 75	BR-7	
Simco SK1 – 2400	BR-2	
Portable Tripod Equipment	BR-4	
Track Mounted - CME 850	BR-5, BR-6A	Cartwright Drilling Ltd. of Thunder Bay, Ontario

The boreholes were advanced using 83 mm or 108 mm inside diameter hollow stem augers and/or and using NW casing and wash boring techniques. In general, 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 and cathead hammers, in accordance with the Standard Penetration Test (SPT) procedures (ASTM D1586). The borehole advanced with portable equipment (Borehole BR-4) employed the use of a half weight hammer and the 'N'-values were corrected for the lower energy drive. Samples 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 monitored 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 December 17, 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 profile drawing provided by MTO (drawing E47311172.dwg). 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)
BR-1	5 371 776.2	331 572.8	375.1	9.8
BR-2	5 371 757.9	331 570.9	374.7	9.8



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Borehole Number	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface Elevation (m)	Borehole Depth (m)
BR-3	5 371 766.7	331 550.2	382.6	18.4
BR-4	5 371 789.7	331 521.5	375.9	7.7
BR-5	5 371 773.5	331 526.6	375.9	9.4
BR-6	5 371 789.9	331 539.7	382.8	10.4
BR-6A	5 371 788.4	331 539.3	382.8	12.7
BR-7	5 371 739.0	331 577.9	375.9	9.8

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 Brule Creek culverts site generally consist of glaciolacustrine plain deposits consisting primarily of clay bordered closely by bedrock to the west and a hummocky ground moraine of silt and tills to the south.

Based on geological mapping by the Ministry of Northern Development and Mines (MNDM)², the site is underlain by metasedimentary rocks; more specifically wacke, arkose, slate, marble, chert, iron formations and bordered by massive granodiorite to granite formations to the northeast.

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, except that the 'N'-values obtained by the use of the half-weight hammer have been corrected as noted in Section 3.0. 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 fill soils of granular, clayey gravel and clay composition (for boreholes advanced through the embankment), peat/topsoil (for boreholes advanced along the toe of slope) underlain by deposits of sand and gravel to sandy gravel, silty clay to clay, silty sand to silt, and sandy silt (Till). A more detailed description of the soil deposits and groundwater conditions encountered in the boreholes is provided below.

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



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Deposit/Layer Description	Boreholes	Deposit Thickness (m)	Deposit Surface Elevation (m)	N Values (blows)/ S_u Shear Strength (kPa)	Laboratory Testing
				Relative Density or Consistency	
Asphalt	BR-3, BR-6 and BR-6A	0.2	382.8 – 382.6	n/a	n/a
(FILL) Sand¹ , trace gravel, trace silt; brown; moist to wet		1.2 – 1.4	382.6 – 382.4	N = 11 Compact	w = 3%, 24% 1 – M (Fig. B1)
(FILL) Clayey Gravel¹ trace to some sand; brown; wet		1.7 – 3.0	381.4 – 381.0	N = 4 – 37 Loose to Dense/ Firm to Hard	n/a
(FILL) Clay , trace to some sand some gravel; reddish brown; wet		2.6 – 4.1	379.7 – 378.0	N = 4 – 21 $S_u = 91$ S = 4 Firm to Stiff	w = 35% - 39% $w_p = 66\%$ $w_l = 27\%$ $I_p = 38\%$ 2 – MH (Fig. B2) 2 – AL (Fig. B3)
Silty Peat/Peat/Topsoil ; brown to black, wet	BR-1, BR-2, BR-4, BR-5 and BR-7	0.7 – 1.4	375.9 – 374.7	N = 2 Very Soft	w = 61% to 70%
Sand and Gravel to Sandy Gravel ; brown; wet	BR-2, BR-5 and BR-7	0.7 – 1.1	375.2 – 373.9	N = 8 – 100 / 0.13 Loose to Very Dense	n/a
Silty Clay to Clay^{2, 3} , trace sand; reddish brown; wet	BR-1 to BR-7 and BR6-A	2.5 – >7.6	375.6 – 373.2	N = 1 – 19 ⁴ $S_u = 51 - > 100$ S = 2 – 3 Stiff to Very Stiff	w = 20% - 63% $w_l = 38\% - 83\%$ $w_l = 16\% - 31\%$ $I_p = 22\% - 53\%$ 9 – MH (Fig. B4) 14 – AL (Fig. B5)
Silt to Silty Sand , trace to some gravel, trace clay; grey; wet	BR-1, BR-2 and BR-7	>3.4 – >5.5 where fully penetrated	371.9 – 369.5	N = 37 – 111/0.23 Dense to Very Dense	w = 21% - 27% 3 – MH (Fig. B6) 2 – AL – N.P.
Sandy Silt (TILL) to Sand and Gravel (TILL)⁵ ; trace to some gravel, trace to some clay; grey; wet	BR-1, BR-3 and BR-6A	>1.1 – >3.6, 1.4	371.5 – 366.4	N = 64 – 111 Very Dense	w = 15% 2 – MH (Fig. B7)

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 (%)



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w_L = Liquid Limit (%)
I_p = Plasticity Index (%)
AL = Atterberg Limits Test
N.P. = Non-Plastic Atterberg Limits Test Result

Notes:

¹ 75 mm cobbles were encountered within the sand and clayey gravel portions of the fill deposit in Boreholes BR-3 and BR-6.

² Trace organics (wood and roots) were encountered in Borehole BR-1 within the upper portion of the silty clay to clay deposit.

³ A 0.6 m thick clayey silt seam was encountered within the Silty Clay to Clay deposit at 7.0 m depth in Borehole BR-5. One Atterberg Test result on the clayey silt seam indicated a liquid limit of 24 per cent, a plastic limit of 14 per cent and a plasticity index of 10 per cent, indicating the material is a clayey silt of low plasticity. The results of the Atterberg Limit test are shown on Figure B5 in Appendix B.

⁴ SPT "N"-values of 50 blows per 0.08 m and 100 blows per 0.08 m of penetration inferred to be indicative of the split-spoon refusing on an obstruction or bedrock and not representative of the consistency of the clay deposit.

⁵ 120 mm and 230 mm size cobbles were encountered at 16.6 m and 18.1 m depth within the sandy silt till deposit in Borehole BR-3.

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 375.4 m and 374.6 m at the inlet and outlet, respectively, on December 17, 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)
BR-1	0.3	374.8
BR-2	0.1	374.6 ¹
BR-3	3.8	378.8 ¹
BR-4	Dry	Inferred ~ 375.4 ²
BR-5	0.4	375.4
BR-6	2.5	380.3 ¹
BR-6A	7.5	375.3
BR-7	0.7	375.2 ¹

Notes:

¹ Borehole was advanced using NW casing and wash boring techniques; water level likely not representative of stabilized groundwater conditions.

² Adjacent creek water level.



Analytical Testing of Creek Water

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 include pH, sulphate, chloride, resistivity and conductivity.

5.0 CLOSURE

The field drilling program was carried out under the supervision of Mr. Randy Axford and Mr. Mathew Riopelle, under the overall direction of Mr. Adam Core, P.Eng. This Preliminary Foundation Investigation Report was prepared by Mr. Adam Core, P.Eng., and Ms. Sarah E. M. Poot, P.Eng., and Associate of Golder provided a technical review of the report. Mr. Jorge M. A. Costa, P.Eng., a Senior Consultant with and Designated MTO Foundations Contact for Golder, conducted an independent quality control review of this report.



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Report Signature Page

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PART B

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6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

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

6.1 General

The Brule Creek twin culverts are located in the District of Thunder Bay in the Township of Conmee on Highway 11/17 at about STA 15+304, approximately 30 m south of the Highway 11/17 and Mokomon Road junction in Mokomon, Ontario, approximately 39 km northwest of Thunder Bay. The highway embankment is constructed of granular and cohesive fill material and is about 8.1 m to 8.3 m high relative to the existing culvert invert, with approximately 5.3 m of soil cover over the existing culverts. The details (i.e., width, height, length, etc.) of the existing twin steel plate corrugated steel pipe culverts are summarized in Table 1.

A box culvert, open footing culvert or pipe culvert(s) are considered feasible alternatives as the replacement culvert at this site, however from a foundation perspective a box type culvert sufficiently wide to handle the creek flow is preferred. An open footing culvert although feasible, presents additional challenges as it will extend the construction schedule and increase the excavation, dewatering and shoring requirements, when compared to the box type culvert. Additionally, pipe culverts similar to the existing culvert configuration, while considered feasible at this site, provide less flow-through capacity compared to box a culvert or open footing culvert with a similar span and if constructed from steel may have a shorter design life. Other culvert types may be preferred due to construction staging or other considerations such as fisheries requirements related to natural channel substrate. A comparison of culvert types based on advantages, disadvantages and risks/consequences is presented in Table 2.

As outlined in the request for proposal and based on discussions with HMM, we understand that it is proposed that the new culvert be comprised of a pre-cast single cell box culvert approximately 6 m wide by 3 m high. The proposed culvert inverts are at Elevations 373.8 m and 373.7 m at the inlet and outlet ends, respectively. We understand that there will not be an embankment grade raise, however temporary widening of the embankment, benched in the upper portion below the embankment crest, may be required for traffic staging such that detours or modification to the existing embankment toes are not required. We understand that permanent Retained Soil System (RSS) walls, up to 3.2 m high, are to be constructed on both sides of the culvert inlet end, recommendations.

6.2 Geotechnical Resistance

For the box culvert replacement, it is recommended that any organic materials encountered below the culvert footprint be sub-excavated and replaced with Granular 'B' Type II. The factored geotechnical axial resistance at Ultimate Limit States (ULS) and the geotechnical axial resistance at Serviceability Limit States (for 25 mm of settlement) for a 6 m wide box culvert founded on a properly prepared subgrade / granular bedding overlying the



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native stiff to very stiff silty clay to clay deposit at Elevation 373.2 and 371.1 m (taking into account invert Elevations, a 0.3 m thick bottom of concrete box culvert and a 0.3 m bedding layer) may be taken as 200 kPa and 165 kPa, respectively.

The geotechnical resistance/reaction provided 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 it's Commentary.

The loading on the foundation soils below the culvert and the associated settlement at the culvert location will be governed by the thickness/height of the overlying and adjacent embankment fill. As such, it is recommended that the structural engineer exercise caution when utilizing the values for the geotechnical reaction at SLS in the design of the culvert and that consideration be given to the sequence and staging of the construction. We understand that there will not be a grade raise.

6.2.1 Frost Protection

It is not necessary to found a box culvert at the standard depth for frost protection purposes, as box structures are tolerant of small magnitudes of movement related to freeze-thaw cycles, should these occur.

6.2.2 Resistance to Lateral Loads/Sliding Resistance

Resistance to lateral forces/sliding resistance between the base of the box culvert and granular bedding material should be calculated in accordance with Section 6.7.5 of the CHBDC. For a pre-cast concrete box culvert founded on a compacted granular fill (Bedding/Levelling layer), $\tan \delta = 0.45$.

6.3 Stability, Settlement and Horizontal Strain

6.3.1 Stability

Limit equilibrium slope stability analysis for the reconstructed embankment adjacent to the culvert 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 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)
New Granular Embankment Fill	21	35	-
Clayey Silt to Silt (Stiff to Very Stiff)	18	-	60
Silty Sand to Silt (Dense to Very Dense)	18	30	-
Sandy Silt (Till) (Very Dense)	19	33	-



The results of the analysis indicate that a FoS of 1.3 is achieved for the above reconstructed embankment and subsurface conditions with the proposed embankments height up to about 8.3 m relative to the culvert invert and granular fill embankments constructed with side slopes inclined 2H:1V at or flatter. This assumes that there is no grade raise or platform widening at the culvert site.

6.3.2 Settlement

Given that an embankment grade raise or widening is not proposed as part of the culvert replacement and highway embankment reconstruction, the existing native soils will not experience additional load, and therefore, settlement of the culvert is estimated to be less than 25 mm.

6.3.3 Horizontal Strain

Horizontal strain is not expected to occur as the permanent embankment geometry is not changing from the current (existing) geometry. As a result, culvert construction concurrent with the embankment construction can be carried out without the need for any foundation mitigation measures or culvert camber.

6.4 Foundation Recommendations for Permanent RSS Walls

6.4.1 Founding Elevations

A typical RSS wall has front facing panels supported on a concrete footing (i.e., levelling pad) constructed on a compacted granular pad founded at a shallow depth below the ground surface at the front of the wall. The granular pad should consist of a minimum 0.5 m thick compacted OPSS.PROV 1010 (*Aggregates*) Granular 'A' material that should extend at least 0.5 m beyond the outside edge of the facing panels, then outward/downward at 1 horizontal to 1 vertical (1H:1V) to the bottom of the pad, consistent with MTO's RSS Design Guideline (2008).

The facing panels footing (levelling pad), granular pad and the reinforced soil mass should be founded below any existing topsoil or unsuitable fill soils. In addition, to mitigate differential settlement and improve the performance of the walls at this site, it is recommended that the upper, very loose zone of organics and fill materials (where present) be sub-excavated and replaced with compacted granular fill, prior to constructing the reinforced soil mass and facing panels. Once sub-excavation and backfilling is completed, the RSS wall facing footing (levelling pad) should be founded at a minimum depth of 1.0 m below the final backfilled grade, on the 0.5 m thick granular pad. The reinforcement can be placed directly over the sub-excavation backfill at grade.

The proposed founding levels for the north RSS wall front facing panels (i.e., levelling pad) are Elevations 372.8 m and 374.2 m for the two (approximately 2.95 m and 2.9 m long) lower and upper sections of the walls. The proposed founding level for the south RSS wall front facing panels is Elevation 372.8 m (with no steps) for the approximately 4.85 m long section of wall. Excavations for the installation of the granular pad will therefore be required to Elevation 372.3 m and 373.7 m for the lower and upper portions of each wall, respectively and as such, based on the soils encountered in BR-4 and BR-5, the granular pads will be founded within the silty clay to clay deposit. The retained soil mass does not need to be founded at the same level as the levelling pad (i.e., facing footing) but should be founded on a competent subgrade (i.e., granular pad is not required). We



understand that excavation will be required to the depth of the levelling pad at Elevations 372.8 m and 374.2 m (full height) for the installation of the RSS wall reinforcing strips, although competent deposits were encountered at Elevations 374.9 m and 375.2 m on the north and south sides of the culvert, respectively, should it be desired to found the RSS wall mass at a higher elevation. Due to the proximity to the adjacent creek, it is anticipated that the excavation for installation of the RSS wall levelling pad, granular pad and RSS wall mass (at some locations) will extend to below the groundwater table. It is recommended that the sub-excavation be backfilled with OPSS.PROV 1010 (*Aggregates*) Granular 'B' Type II material below the groundwater table as the excavation progresses. The RSS wall should be constructed in a similar manner as discussed for the culvert installation in Section 6.6, Construction Considerations.

For this site, from a foundations perspective, the RSS walls should be designed for medium performance and appearance in accordance with MTO Special Provision (SP) 599S22 (Retained Soil System).

6.4.2 Geotechnical Resistances

For the RSS facing panels supported on a concrete footing (levelling pad) constructed on compacted granular pad, the wall design may be completed based on a factored geotechnical resistance at ULS of 160 kPa and a geotechnical reaction at SLS (for 25 mm of settlement) of 140 kPa, assuming a minimum embedment depth of about 1 m.

Assuming that the RSS wall acts as a unit and uses the full width of the reinforced soil mass (which can be taken as 0.8 times the wall height, or a width of 2.6 m for the maximum 3.2 m high RSS wall above final surrounding grade at this site), the RSS wall may be designed based on a factored geotechnical resistance at ULS of 160 kPa and a geotechnical reaction at SLS (for 25 mm of settlement) of 140 kPa, placed over the properly prepared subgrade (native silty clay or sandy gravel) as discussed above.

6.4.3 Resistance to Lateral Loads/Sliding Resistance

Resistance to lateral forces/sliding resistance between the compacted fill of the RSS wall and the subgrade should be calculated in accordance with Section 6.7.5 of the CHBDC. The coefficient of friction, $\tan \phi'$, between the compacted granular fill of the RSS wall and the compacted granular backfill may be taken as 0.62. This represents an unfactored value. The actual values used should be reviewed and revised, if necessary, by the proprietary RSS wall designer.

6.4.4 Global Stability

The static global stability analyses for the RSS wall were completed using the parameters outlined in Section 6.3, and assume that all existing topsoil and organics are completely removed prior to constructing the RSS walls. The results of the static global stability analyses, presented on Figure 1, indicate that a minimum factor of safety of 1.5 is achieved for an RSS wall up to 3.2 m high (retained soil height).

It should be noted that the internal stability of the reinforced earth structure is to be designed and ensured by the proprietary product designer/supplier.



6.5 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 replacement culvert and any wing walls 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 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 maximum 200 mm loose lift thickness and compacted. Weep holes should be installed to provide positive drainage of the granular backfill.
- Granular fill may be placed in a zone with the width equal or greater than the equivalent depth of frost penetration (as per OPSD 3090.100 (Foundation Frost Penetration Depths for Northern Ontario), which for this site is 2.4 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 horizontal to 1 vertical (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 lateral earth pressures acting against the culvert walls and wing/head walls are based on the proposed backfill material against the walls 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 structures allow for lateral yielding, active earth pressures may be used in the design of the structure(s). If the structures do not allow 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.



6.6 Culvert Construction Considerations

6.6.1 Construction Staging and Temporary Roadway Protection

We understand that staged construction is being considered at this site for replacement of the culvert. The sketches of the proposed staging operation provided by HMM indicate staging considerations generally as follows:

Option 1 – 2 Lane Staged Traffic

- Stage 1: a temporary widening and grade lowering of the east side of the upper portion of the embankment for traffic use and the installation of roadway protection system to facilitate replacement of the west portion of the culvert and allow for two lanes of traffic. Steepening of the east side slope or construction of a low temporary retaining wall would be required.
- Stage 2: roadway protection to facilitate the replacement of the east portion of the culvert.

Option 2 – Single Lane Staged Traffic Signals using Temporary Traffic Signals

- An alternative staging operation consisting of a single lane with traffic signals is also being considered, which would eliminate the need for temporary embankment widening/grade lowering on the east side of the roadway but would still require temporary roadway protection as per Option 1.

The temporary excavation for the culvert replacement will be made through the existing granular and cohesive embankment fill and into native soils, which are comprised of a deposit of sandy gravel in places, underlain by/or into stiff to very stiff 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 and cohesive fills and native soils are considered to be Type 3 soil above the groundwater table and Type 4 soil below. 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.

Installation of sheet-piles for temporary shoring may be impeded by the presence of cobbles inferred to be present within the embankment fill in Boreholes BR-3 and BR-6 and within the sandy silt till in Borehole BR-3. It may be necessary to excavate and replace the existing fill material (where practical) in the areas of sheet-pile installation in a series of narrow trenches of limited length to remove or avoid cobble size materials. 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. Excavation and replacement fill placement should be carried out in the same day to avoid leaving any trench open overnight. It is recommended that an NSSP be included in the contract documents to address obstructions; a sample NSSP is included in Appendix C.

The design of the temporary support systems, and temporary retaining walls as may be required for the temporary widening for staging, may be designed using the following parameters:



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Soil Type	Internal Angle of Friction	Unit Weight	Undrained Shear Strength	Coefficient of Earth Pressure		
	(ϕ , degrees)	(γ , kN/m ³)	(kPa)	Active, K_a	At Rest, K_o	Passive, K_p
Sand - FILL (Compact)	33	20	-	0.29	0.46	3.39
Clayey Gravel - FILL (Loose to Dense/ Firm to Hard)	30	19	-	0.33	0.50	3.00
Clay - FILL (Stiff)	23	18	60	0.44	0.61	2.28
Peat/Topsoil (Very Soft)	27	12	1	0.38	0.55	2.66
Sand and Gravel to Sandy Gravel	32	20	-	0.31	0.47	3.25
Clay (Stiff to Very Stiff)	23	18	60	0.44	0.61	2.28
Silty Sand to Silt (Dense)	30	18	-	0.33	0.50	3.00
Sand and Gravel to Sandy Silt - TILL (Dense to Very Dense)	33	19	-	0.29	0.46	3.39

The temporary shoring design should be assessed for both the drained (ϕ) and undrained (c_u) cases and the design should be based on the more conservative earth pressure conditions. Further, the total passive resistance of the temporary protection system below the base of the excavation should be calculated based on the values of K_p given above and then reduced by an appropriate factor of safety that considers; the allowable wall movement as extrapolated from Figure C6.16 of the CHBDC (2006) to account for the fact that a large strain would be required for full mobilization of the passive resistance.

The earth pressure coefficients noted above are based on a horizontal surface adjacent to the excavation. If sloped surfaces are present, the coefficient of earth pressure should be adjusted accordingly.

The subsurface soils at this site are potentially sensitive to disturbance from vibration and/or driving, which should be considered in the design and installation of the temporary protection systems.

Where required, temporary protection systems should be designed and constructed in accordance with OPSS.PROV 539 (Temporary Protection Systems). Temporary excavation support should be designed to Performance Level 2 for any excavation adjacent to the existing roadway. Design of the temporary excavation and roadway support system should include an evaluation of base stability ("base heave" or soil squeezing stability) and hydraulic uplift stability as defined in the Canadian Foundation Engineering Manual (CFEM 2006).



6.6.2 Temporary Retaining Walls/Slopes

Temporary retaining walls may be required for the widening if localized slope steepening is not permitted. The contractor will be responsible for design of the temporary works.

A temporary RSS wall footing may be designed based on a factored geotechnical resistance at ULS and a geotechnical reaction at SLS (for 25 mm of settlement) of 150 kPa as placed on a compacted 150 mm thick granular pad constructed over the clayey embankment fill a minimum of 0.6 m below the lowest surrounding grade.

Resistance to lateral forces/sliding resistance between the temporary RSS wall footing and the subgrade should be calculated in accordance with Section 6.7.5 of the bedding *CHBDC*. The coefficient of friction, $\tan \delta$ can be taken as 0.58 between the wall footing and compacted granular fill.

For temporary retaining walls up to about 3 m high constructed on the east side of the embankments, a factor of safety greater than 1.5 is achieved for global stability of the wall. The internal stability will need to be checked by the designer.

As an alternative to a temporary retaining wall, consideration could be given to temporary steepening the embankment side slopes for Stage 1 construction (assume additional right-of-way is not available at the toe of slope). The temporary steepened slope should not be inclined steeper than 1.5H:1V. The temporary steepened slope should be constructed of OPSS.PROV 1010 Granular 'B' Type II material and the fill should be properly benched into the existing slope as per OPSD 208.010 (Benching of earth slopes). A temporary side slope with the above configuration will achieve a factor of safety greater than 1.3.

6.6.3 Control of Groundwater and Surface Water

Excavation along the culvert alignment will be required to remove the existing embankment fill, organic materials where present, through the sandy gravel deposit where present and into the native clay stratum to achieve the required invert/bedding level prior to placement of bedding, the actual culvert, backfill and roadway pavement structure. Groundwater flow into the excavation can be expected due to the depth of the excavations as well as the potential presence of perched water within the embankment fill, and the presence of the sandy gravel deposit overlying the silty clay to clay deposit. 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 during the construction period, surface water flow conditions, perched groundwater levels and groundwater level at the time of construction, water flow could be passed through the area by means of a temporary culvert, using one of the existing CSP culverts or diverted by pumping from behind temporary cofferdam.

Excavations for the box culvert will extend below the creek water level and through the sandy gravel deposit, and may therefore require temporary shoring with unwatering to allow for construction/placement of the footings and/or placement of bedding material in dry conditions, where required. It is likely that the groundwater level is near ground surface, although groundwater perched within the embankment fill may also be present, but



discharge of groundwater into open excavations will be slow and of limited volume, and the need for dewatering will be limited. Temporary shoring and groundwater control could be in the form of a sheet-pile cut off wall or cofferdam advanced to an appropriate depth to control groundwater inflow from the creek and to prevent base heaving of the foundation subgrade.

Provided that the creek flow is diverted and the unwatering system is installed to a suitable depth to mitigate groundwater inflows, pumping volumes are not anticipated to exceed 50 m³/day. As such, it is anticipated that a Permit to Take Water (PTTW) would not be required.

6.6.4 Excavation and Replacement Fill Below Culvert

Prior to placement of any bedding material, granular fill (or concrete working slab if required), all organic materials (including peat, topsoil and mixed organic soil) and any softened or disturbed soils, should be sub-excavated from below the plan limits of the proposed works.

The culvert subgrade should be inspected by a Quality Verification Engineer following sub-excavation to ensure that all organics and other unsuitable materials have been removed, in accordance with OPSS 422 (Precast Reinforced Concrete Box Culverts) for a pre-cast box culvert. Following inspection, the sub-excavated area should be backfilled with granular material meeting the requirements of an OPSS.PROV 1010 (Aggregates) Granular 'A' or Granular 'B' Type I, II or III that is placed and compacted in accordance with OPSS.PROV 501 (Compacting). For backfilling below the water level, if required, we recommend that only Granular B Type II be utilized.

6.6.5 Culvert Bedding and Backfill

The bedding, levelling pad and granular backfill 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, seepage from the perched groundwater in the embankment fill and groundwater seepage through the native sandy gravel stratum during excavation to the invert and bedding level, it is recommended that a minimum 300 mm thick layer of OPSS.PROV 1010 (Aggregates) Granular 'B' Type II material be used for bedding and sub excavation backfilling purposes. We do not recommend that Granular B Type I or III be used for bedding purposes. As the native soil below the bedding is generally fine grained, it is also recommended that a non-woven geotextile be placed between the native soil and the bottom of the granular 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. The bedding should be placed in maximum 200 mm thick loose lifts and compacted to at least 98 per cent of the Standard Proctor Maximum Dry Density (SPMDD) of the materials, consistent with 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 similar to that presented on OPSP 803.010 (Backfill and Cover for Concrete Culverts) for culvert construction in dry conditions.

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. The granular backfill should be placed and compacted



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.

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.

It is recommended that the backfill above the culvert consist of Granular 'A' or 'B' (Type I, II or III) rather than re-using the cohesive fill that will be excavated from above the culvert. Given that the top of the proposed box culvert will be approximately 3.3 m below the estimated frost penetration depth (about 2.4 m below finished grade), Section A-A "Frost Penetration Line At or Above The Top of Culvert" of OPSD 803.01010 (Backfill and Cover for Concrete Culverts) would apply and a frost taper would typically not be required. Grain size analyses on two samples of the existing clay fill material indicate that the clay fill has silt contents of 26 per cent and 39 per cent, which suggests that the material has a "low" to borderline "moderate" frost susceptibility based on the classification systems provided in the MTO Pavement Design and Rehabilitation Manual (2013) and the Chamberlain (1982) criteria given in the TAC Pavement Design and Management Guide (1997).

As the existing embankment beyond the new culvert backfill area will likely consist of an upper layer of granular pavement structure underlain by clayey gravel fill and clay fill, it would be prudent to incorporate a frost taper (similar to Section B-B "Frost Penetration Line Below Top of Culvert" on OPSD 803.010) into the existing highway embankment adjacent to the culvert to minimize frost heave between the new free-draining, non-frost susceptible, granular backfill over the culvert and the existing poorly draining, "low" frost susceptible cohesive fill of the adjacent embankment. However, we also understand from a staging perspective, a frost taper may be difficult (and potentially costly to construct due to the proximity of the culvert in relation to the Mokomon Road intersection (located approximately 30 m north of the culvert).

As the clayey gravel fill and clay fill are not considered to be a free-draining material, there is a potential for heaving to occur due to the expansion of pore water should surface water infiltrate into these cohesive layers, on the order of about 30 mm relative to the section of granular backfill above the culvert, which is considered free-draining, and not susceptible to frost heave. Given that the risk for frost heaving due to formation of ice lenses within the clayey soils of the embankment is anticipated to be relatively low, a frost taper is not required, provided a roughly 30 mm deep sag (i.e. reverse frost heave) at the culvert location is considered acceptable from a roadway design perspective and from a traffic performance perspective.

6.6.6 Subgrade Protection

The native soils (silty clay to clay) will be susceptible to disturbance from construction traffic and/or ponded water. To limit the effect of this disturbance and as an alternative to the 300 mm compacted bedding layer, a concrete working slab could be placed on the subgrade if the box culvert (bedding/levelling course and structure) is not placed within four hours after preparation, inspection, and approval of the foundation subgrade. The minimum thickness of the concrete working slab should be 100 mm and the concrete should have a minimum 28 day compressive strength of 20 MPa. It is recommended that an NSSP be included in the contract for a concrete working slab for the culvert site; a sample NSSP is included in Appendix C.



6.6.7 Erosion Protection

Provision should be made for scour and erosion protection at the box culvert location. In order to prevent surface water from flowing either beneath the box 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 clay seal or concrete cut-off wall should be provided at the upstream end of the box 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 opening. 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 blanket, including the creek side slopes and fill slope over the culvert if this clay seal is adopted.

As outlined in the CHBDC (2006) Section 6.12 and C6.1.2 and referenced in Clause 11.10.1 of AASHTO (2012) LFRD Bridge Design Specifications, special consideration should be taken for mechanically stabilized earth walls (such as RSS walls) where floodplain erosion or scour has the potential to undermine the reinforced fill zone or facing panel or any supporting footing (i.e. the concrete levelling pad and granular pad). Consideration should be given to lowering the base of the RSS wall and/or or to provide alternative methods of scour protection such as rip rap of sufficient size, placed to a sufficient depth to preclude scour. We understand that the granular pad/levelling pad of the RSS wall installation is being provided with a minimum of about 1.6 m of soil cover with rip rap protection similar to that described above. The erosion/scour protection for the RSS wall should be carefully reviewed by the hydraulics design engineer, design engineer and the proprietary RSS wall designer.

6.6.8 Obstructions

The contractor should be alerted to the presence of cobbles within the embankment fill and within the native till deposits as encountered in Boreholes BR-3 and BR-6. A sample NSSP is included in Appendix C.

6.6.9 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.



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For potential sulphate attack on concrete, the results of the water analysis were compared to Table 3 in CSA A23-1-09, and indicate that the relative degree of sulphate attack is low (less than the moderate range). However, given that the location of the culvert location is on Highway 11/17 and will be exposed to de-icing salts it is recommended that C-1 class exposure concrete be considered for the pre-cast culvert units. Further, the resistivity results indicate that the creek water has a low corrosiveness potential based on the Transportation Research Board Guidelines (Transportation Research Board, National Research Council, 1998 as referenced in the MTO Gravity Pipe Manual, 2014). It should be noted that the creek water levels in the area are subject to seasonal fluctuations and variations due to precipitation events and the water chemistry could also be variable. These recommendations are provided as guidance only; the structural designer should take the results of the laboratory testing, the potential for corrosion and the ultimate selection of materials into consideration.

7.0 CLOSURE

This Preliminary Foundation Design Report was prepared by Mr. Adam Core, P.Eng., and the technical aspects were reviewed by Ms. Sarah E. M. Poot, P.Eng, an Associate of Golder. Mr. Jorge M. A. Costa, P.Eng., a Senior Consultant with and Designated MTO Foundations Contact of Golder, conducted an independent quality control review of this report.



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Report Signature Page

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Transportation Research Board, National Research Council, 1998, "Service Life of Drainage Pipe", National Cooperative Highway Research Program (NCHRP) Synthesis 254.

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 1205 Material Specification for Clay Seal

OPSS 1860 Material Specification for Geotextiles

Ontario Provincial Standard Specifications (OPSS) – Provincial Oriented

OPSS.PROV 501 Construction Specification for Compacting

OPSS.PROV 539 Construction Specification for Temporary Protection Systems

OPSS.PROV 1002 Material Specification for Aggregates - Concrete



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OPSS.PROV 1010 Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material

Ontario Provincial Standard Drawings (OPSD)

OPSD 208.010 Benching of Earth Slopes

OPSD 803.010 Backfill and Cover for Concrete Culverts with Spans Less Than or Equal to 3.0 m

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)



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Table 1: Summary Details of Existing Culvert

Culvert Location (Twp)	Site #	Approximate Height of Embankment ¹ (m)	Existing Culvert			Approximate Invert Elevation ²	
			Type	Approximate Dimension ²	Approximate Length (m)	West (Inlet) End of Culvert (m)	East (Outlet) End of Culvert (m)
Hwy 11/17 STA 15+304 (Township of Conmee)	48W-249	8.1 – 8.3	Twin CSP	2.7 m diameter	45 – 46	374.3	374.2

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 E47311172.dwg).

Prepared by: AC
Checked by: SEMP
Reviewed by: JMAC



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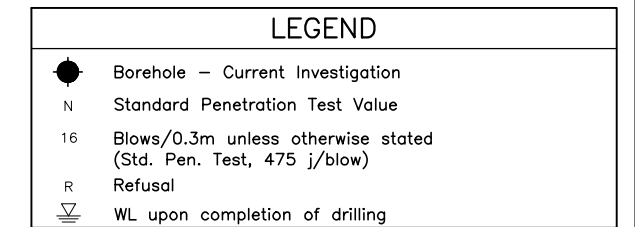
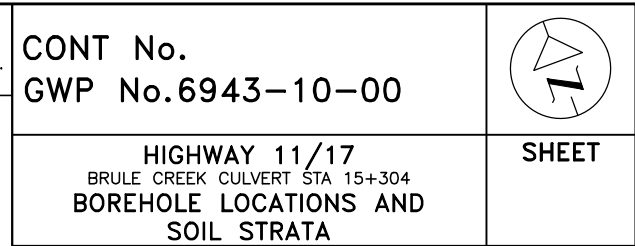
Table 2: Comparison of Foundation Alternatives

Option	Advantages	Disadvantages	Risks/Consequences
Pre-Cast Box Culvert	<ul style="list-style-type: none"> ■ Minimizes depth of excavation, protection system and dewatering requirements compared to open footing option. Minor excavation required for sub-excavation of existing fill/organics and for the bedding/levelling course. ■ Allows faster construction resulting in shorter duration for dewatering and surface water pumping. ■ More tolerant of total and differential settlement if the highway embankment is raised or widened at the culvert site or if heave/settlement occurs resulting from freeze/thaw of the subgrade. ■ Backfill/bedding under the culvert may be placed underwater (i.e. Granular 'B' Type II) minimizing or eliminating water pumping requirements. 	<ul style="list-style-type: none"> ■ May not satisfy fisheries requirements related to natural channel substrate, if applicable. ■ Cut-off wall (or clay seal/blanket) 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. 	<ul style="list-style-type: none"> ■ Some risk of disturbance of the native clay deposit during construction; can be mitigated with use of a tremie concrete working slab or Granular B Type II working pad. ■ Low risk related to settlement performance.
Open Footing Culvert	<ul style="list-style-type: none"> ■ 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. ■ Existing culvert can likely be used for water diversion while new footings are being constructed adjacent to the culvert depending on the width of the new culvert. ■ Readily suitable for construction using concrete or metal sections. ■ Would likely satisfy fisheries requirements related to natural channel substrate, if applicable. 	<ul style="list-style-type: none"> ■ Excavation depths are greater than for a concrete box or pipe culvert option, resulting in increased excavation support and dewatering/unwatering requirements and additional spoil material to be disposed off-site. ■ 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, and would require a tremie concrete plug to allow for subsequent unwatering and footing construction. ■ Less tolerant of total and differential settlement if the highway embankment is raised or widened at the culvert site. ■ Concrete or metal arch sections supported on concrete open (strip) footings may not allow for adequate soil cover to be placed including the roadway pavement structure and the long-term performance of such structures is not known. 	<ul style="list-style-type: none"> ■ High risk of disturbance of the native clay deposit during construction; can be mitigated with use of a concrete working slab or Granular 'B' Type II pad. ■ Would likely require greater depth of dewatering/ unwatering for footing construction. ■ Culvert joints may be required to accommodate total and differential settlement (if required).



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BRULE CREEK CULVERT - SITE NO. 48W-249/C

Option	Advantages	Disadvantages	Risks/Consequences
Pipe Culvert(s)	<ul style="list-style-type: none">■ Allows for faster construction resulting in shorter duration for dewatering and surface water pumping compared to an open footing culvert.■ More tolerant of total and differential settlement if the highway embankment is raised or widened, or if heave/settlement occurs due to freeze/thaw of the subgrade.■ Backfill under the culvert may be placed underwater (i.e. Granular 'B' Type II) minimizing or eliminating water pumping requirements.	<ul style="list-style-type: none">■ Reduced flow-through capacity compared to box culvert options with a similar span - additional flow capacity may have to be provided by multiple pipes.■ Cut-off wall (or clay seal/blanket) may be required at inlet to mitigate potential scour under culvert(s).■ Difficult to compact backfill materials to level of culvert springline.■ CSP does not have as long of design life compared to concrete options.	<ul style="list-style-type: none">■ Some risk of disturbance of the native clay deposit during construction; can be mitigated with use of a tremie concrete working slab or Granular B Type II working pad.■ Limited risk related to settlement performance.



NOTES

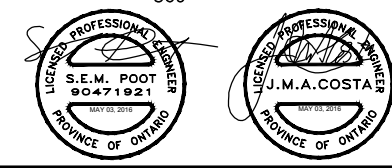
This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

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

The complete Foundation Investigation and Design Report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of OPS General Conditions.

REFERENCE

Base plans provided in digital format by MTO, drawing file nos. E47311172.dwg received Dec. 11, 2015. GA provided in digital format by MTO file nos. ST-358767-BRULE CREEK CULVERT-01-GENERAL ARRANGEMENT.dwg received Apr. 6, 2016.





PHOTOGRAPHS

**Photograph 1: Brule Creek Culvert
Looking North at Culvert (December 2015)**



**Photograph 2: Brule Creek Culvert
Looking South at Culvert (December 2015)**





PHOTOGRAPHS

**Photograph 3: Brule Creek Culvert
Looking South at West Side Inlet (December 2015)**



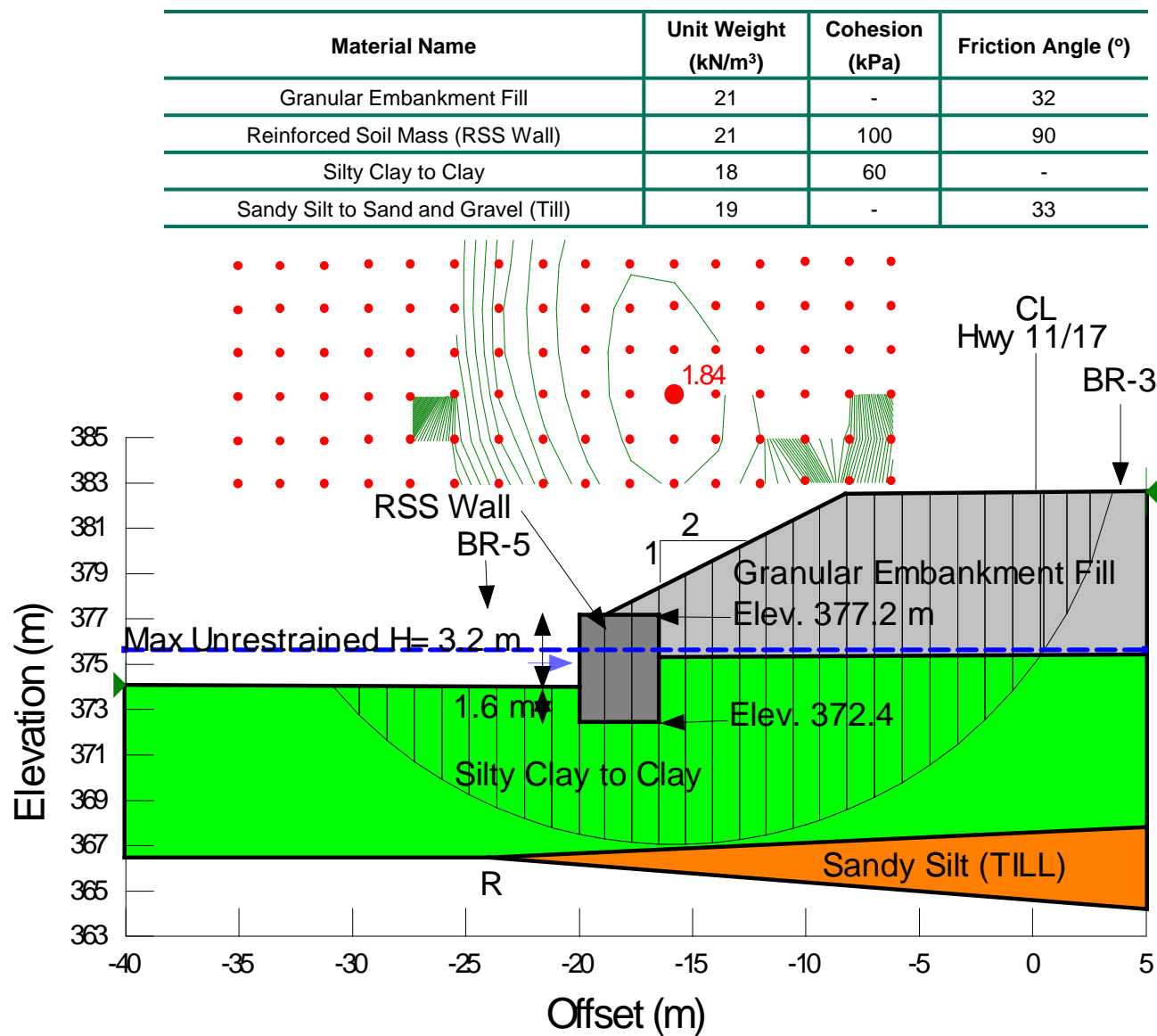
**Photograph 4: Brule Creek Culvert
Looking West at East Side Outlet (December 2015)**





Stability Analysis RSS Wall – Brule Creek Culvert Site No. 48W-249/C

Figure 1





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
ν	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	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 = σ'_p / σ'_{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

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

Notes: 1
2

$$\tau = c' + \sigma' \tan \phi'$$
$$\text{shear strength} = (\text{compressive strength})/2$$



LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

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

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

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

Dynamic Cone Penetration Resistance; N_d :

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

PH: Sampler advanced by hydraulic pressure

PM: Sampler advanced by manual pressure

WH: Sampler advanced by static weight of hammer

WR: Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)

A electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm² pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (Q_t), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

III. SOIL DESCRIPTION

(a) Non-Cohesive (Cohesionless) Soils

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

(b) Cohesive Soils Consistency

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

IV. SOIL TESTS

w	water content
w_p	plastic limit
w_l	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test 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

PROJECT 1533879		RECORD OF BOREHOLE No BR-1				1 OF 1 METRIC								
G.W.P. 6943-10-00		LOCATION N 5371776.2; E 331572.8				ORIGINATED BY MR								
DIST _____ HWY 11/17		BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers				COMPILED BY AC								
DATUM GEODETIC		DATE December 8, 2015				CHECKED BY SEMP								
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
375.1	GROUND SURFACE							20 40 60 80 100	20 40 60					
0.0	TOPSOIL, some clay, trace sand, trace gravel Brown Wet		1	AS	-		375							
374.4														
0.7	SILTY CLAY to CLAY, trace to some sand, trace to some gravel Stiff to very stiff Reddish brown Wet		2	SS	19		374							
	Trace organics (wood / roots) in Samples 2 and 3.		3	SS	17		373							
			4	SS	13		372							
371.9			5	SS	50/0.08		371							
3.2	SILTY SAND, trace to some gravel Compact to very dense Grey Wet		6	SS	58		370							
			7	SS	25		369							
369.5							368							
5.6	SILT, trace to some clay, trace sand Very dense Grey Wet		8	SS	66		367							
			9	SS	52		366							
366.4														
8.7	Sandy SILT, trace to some gravel, trace to some clay (TILL) Very dense Grey Wet		10	SS	66									
365.3														
9.8	END OF BOREHOLE													
	Note: 1. Water level at a depth of 0.3 m below ground surface (Elev. 374.8 m) upon completion of drilling.													


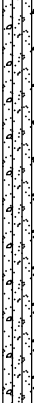

SUD-MTO 001 1533879.GPJ GAL-MISS.GDT 23/03/16 DATA INPUT:

PROJECT		1533879		RECORD OF BOREHOLE No BR-2		1 OF 1		METRIC	
G.W.P.		6943-10-00		LOCATION		N 5371757.9; E 331570.9		ORIGINATED BY	
DIST		HWY 11/17		BOREHOLE TYPE		108 mm I.D. Hollow Stem Augers, NW Casing and Wash Boring		COMPILED BY	
DATUM		GEODETIC		DATE		December 18, 2015		CHECKED BY	
SEMP		SEMP		SEMP		SEMP		SEMP	
SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS		DYNAMIC CONE PENETRATION RESISTANCE PLOT		UNIT WEIGHT	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L
374.7	GROUND SURFACE								
0.0	TOPSOIL, some peat, trace sand, trace gravel Brown Wet								
373.9									
0.8	SAND and GRAVEL Very dense Brown Wet		1	SS	100/0.13				
373.2									
1.5	CLAY, trace sand, trace gravel Stiff Reddish brown Wet		2	SS	11				
			3	SS	9				
			4	SS	7				
370.4									
4.3	SILT, trace sand, trace to some gravel, trace clay Dense to very dense Grey Wet One piece of gravel recovered in Sample 5. One piece of gravel in Sample 6.		5	SS	37				
			6	SS	38				
			7	SS	72				
			8	SS	40				
364.9									
9.8	END OF BOREHOLE Note: 1. Water level at a depth of 0.1 m below ground surface (Elev. 374.6 m) upon completion of drilling.								

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

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

SUD-MTO 001 1533879.GPJ GAL-MISS.GDT 23/03/16 DATA INPUT:

PROJECT 1533879		RECORD OF BOREHOLE No BR-3				2 OF 2 METRIC							
G.W.P. 6943-10-00		LOCATION N 5371766.7; E 331550.2				ORIGINATED BY MR							
DIST _____ HWY 11/17		BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers, NW Casing and Wash Boring				COMPILED BY AC							
DATUM GEODETIC		DATE December 11 and 12, 2015				CHECKED BY SEMP							
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					
	--- CONTINUED FROM PREVIOUS PAGE ---						20 40 60 80 100						
367.8	CLAY, trace to some sand, trace gravel Stiff to very stiff Reddish brown Wet		12	SS	7								0 3 36 61
14.8	Sandy SILT, trace to some gravel, trace to some clay (TILL) Very dense Grey Wet		14	SS	64/0.15								
	A 120 mm cobble encountered at 16.6 m depth.		15	SS	64								
364.2	A 230 mm cobble encountered at 18.1 m depth.		16	SS	68/0.13								13 27 53 7
18.4	END OF BOREHOLE												
	Note: 1. Water level at a depth of 3.8 m below ground surface (Elev. 378.8 m) upon completion of drilling.												

PROJECT 1533879		RECORD OF BOREHOLE No BR-4				1 OF 1 METRIC							
G.W.P. 6943-10-00		LOCATION N 5371789.7; E 331521.5		ORIGINATED BY MR/RA									
DIST _____ HWY 11/17		BOREHOLE TYPE Portable Equipment		COMPILED BY AC									
DATUM GEODETIC		DATE December 20 and 21, 2015		CHECKED BY SEMP									
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT		REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED		W _p W W _L	WATER CONTENT (%)	γ kN/m ³	GR SA SI CL
375.9	GROUND SURFACE							20 40 60 80 100					
0.0	PEAT (Fibrous), trace sand Very soft Black Wet		1	AS	-	▽	375						
			2	SS	2								
374.5													
1.4	CLAY, trace sand, trace gravel Stiff to very stiff Reddish brown Wet		3	SS	8		374						
			4	SS	12								
			5	SS	10		373						
			6	SS	10								
			7	SS	14		372						
			8	SS	16								
			9	SS	100/10.08		371						
							370						
							369						
368.2	END OF BOREHOLE SPLIT-SPOON REFUSAL												
7.7	Note: 1. Borehole dry upon completion of drilling, inferred water level at about 0.5 m (Elev. 375.4 m) below ground surface as per adjacent creek water level on December 17, 2015. 2. Split Spoon samples obtained by driving with a 1/2 weight hammer. SPT 'N' values have been adjusted to the inferred values that would be obtained using a standard weight hammer.												

[illegible]

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

PROJECT 1533879		RECORD OF BOREHOLE No BR-6				1 OF 1 METRIC								
G.W.P. 6943-10-00		LOCATION N 5371789.9; E 331539.7				ORIGINATED BY MR								
DIST _____ HWY 11/17		BOREHOLE TYPE 83 mm I.D. Hollow Stem Augers, NW Casing and Wash Boring				COMPILED BY AC								
DATUM GEODETIC		DATE December 12, 2015				CHECKED BY SEMP								
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
382.8	GROUND SURFACE													
0.0	ASPHALT (200 mm)													
0.2	Sand, trace to some gravel (FILL) Compact Brown Moist		1	AS	-									
			2	SS	11									
381.4														
1.4	Clayey gravel, trace to some sand (FILL) Very stiff to hard / compact to dense Reddish brown Moist to wet		3	SS	24									
			4	SS	37									
379.7	A 75 mm cobble encountered at 3.0 m depth.													
3.1	Clay, some sand, trace gravel (FILL) Stiff to very stiff Reddish brown Wet		5	SS	13									
			6	SS	14									
			7	SS	14									
			8	SS	21									
375.6														
7.2	CLAY, trace sand, trace gravel Firm to very stiff Reddish brown Wet		9	SS	18									
			10	SS	7									
372.4														
10.4	END OF BOREHOLE													
	Note: 1. Water level at a depth of 2.5 m below ground surface (Elev. 380.3 m) upon completion of drilling.													

SUD-MTO 001 1533879.GPJ GAL-MISS.GDT 23/03/16 DATA INPUT:

PROJECT 1533879		RECORD OF BOREHOLE No BR-6A		1 OF 2 METRIC	
G.W.P. 6943-10-00		LOCATION N 5371788.4; E 331539.3		ORIGINATED BY MR	
DIST _____ HWY 11/17		BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers		COMPILED BY AC	
DATUM GEODETIC		DATE February 20, 2016		CHECKED BY SEMP	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100	W _p	W	W _L		GR	SA	SI	CL
SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED																				
382.8	GROUND SURFACE																			
0.0	Advanced from ground surface to 6.1 m depth without sampling (see Borehole BR-6 for stratigraphy).																			
376.7																				
6.1	Clay, some sand, trace organics (FILL) Very stiff Reddish brown Wet		1	SS	17															
375.6																				
7.2	CLAY Stiff to very stiff Reddish brown Wet		2	SS	10															
			3	SS	3															
			4	SS	3															
371.5																				
11.3	SAND and GRAVEL, some silt, trace clay (TILL) Very dense Grey Wet																			

SUD-MTO 001 1533879.GPJ GAL-MISS.GDT 06/04/16 DATA INPUT:

Continued Next Page

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



SUD-MTO 001 1533879.GPJ GAL-MISS.GDT 06/04/16 DATA INPUT:

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

PROJECT		1533879		RECORD OF BOREHOLE No BR-7		1 OF 1		METRIC	
G.W.P.		6943-10-00		LOCATION		N 5371739.0; E 331577.9		ORIGINATED BY	
DIST		HWY 11/17		BOREHOLE TYPE		Solid Stem Augers, NW Casing and Wash Boring		COMPILED BY	
DATUM		GEODETIC		DATE		December 16, 2015		CHECKED BY	
SEMP		SEMP		SEMP		SEMP		SEMP	
SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS		DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	20 40 60 80 100	W _p W W _L	UNIT WEIGHT
375.9	GROUND SURFACE								
0.0	PEAT (Fibrous) Black Wet								
375.2									
0.7	SANDY GRAVEL, some organics Compact Brown Wet		1	SS	16				
374.5									
1.4	SILTY CLAY to CLAY, some sand, trace gravel Firm to very stiff Reddish brown Wet		2	SS	4				
			3	SS	6				
			4	SS	7				
			5	SS	10				
369.5			6A	SS	11/0.20				
6.4	SILTY SAND, some gravel Very dense Grey Wet		6B						
			7	SS	108				
367.3									
8.6	SILT, trace to some clay, trace sand Dense Grey Wet								
			8	SS	41				
366.1									
9.8	END OF BOREHOLE								
	Note: 1. Water level at a depth of 0.7 m below ground surface (Elev. 375.2 m) upon completion of drilling.								

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



APPENDIX B

Laboratory Test Results



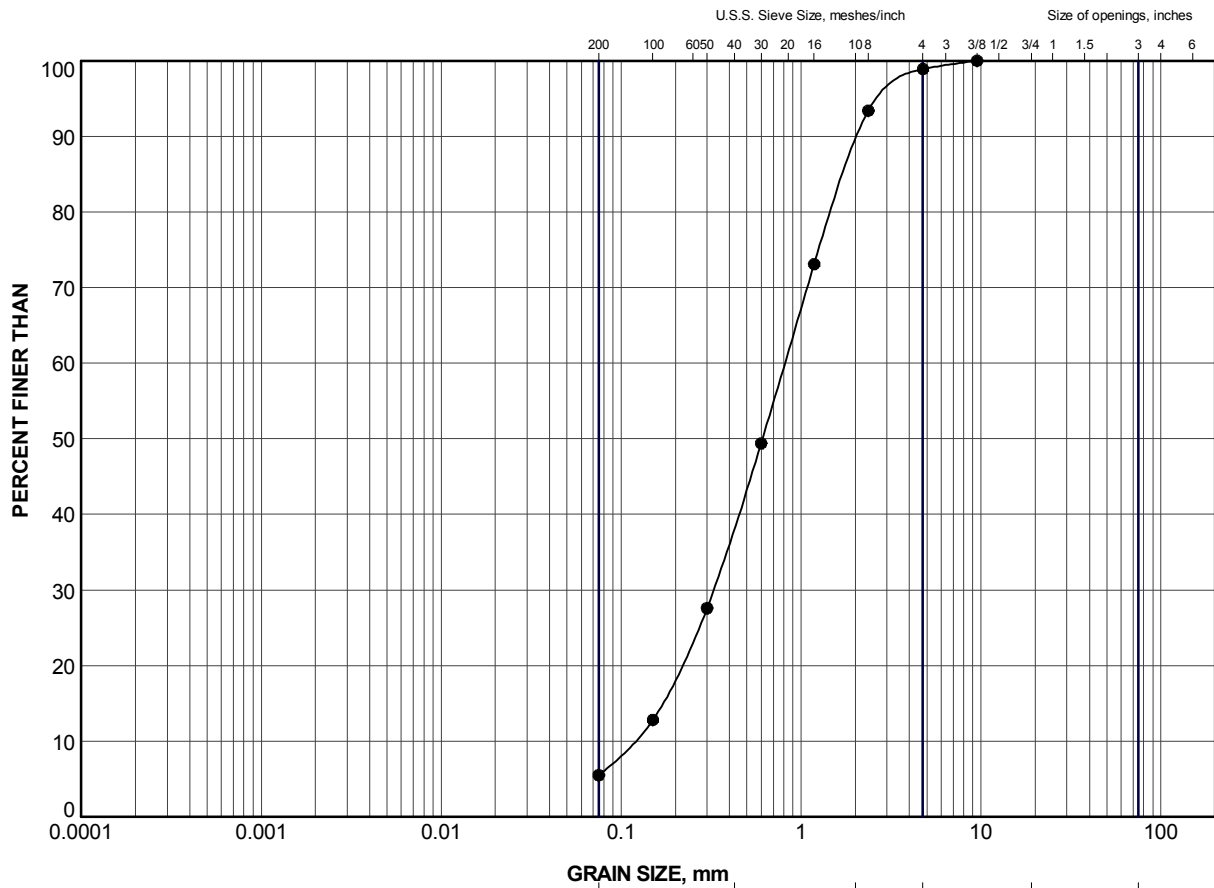
DETAIL FOUNDATION REPORT BRULE CREEK CULVERT - SITE NO. 48W-249/C

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

Parameter	Units	Result
Chloride (CL)	mg/L	1.59
Sulphate (SO4)	mg/L	3.19
Conductivity (EC)	µS/cm	170
Resistivity	ohm-cm	5882
pH	n/a	8.00


Notes: 1. Sample obtained on December 17, 2015.
2. Analytical testing carried out by ALS Canada Ltd.

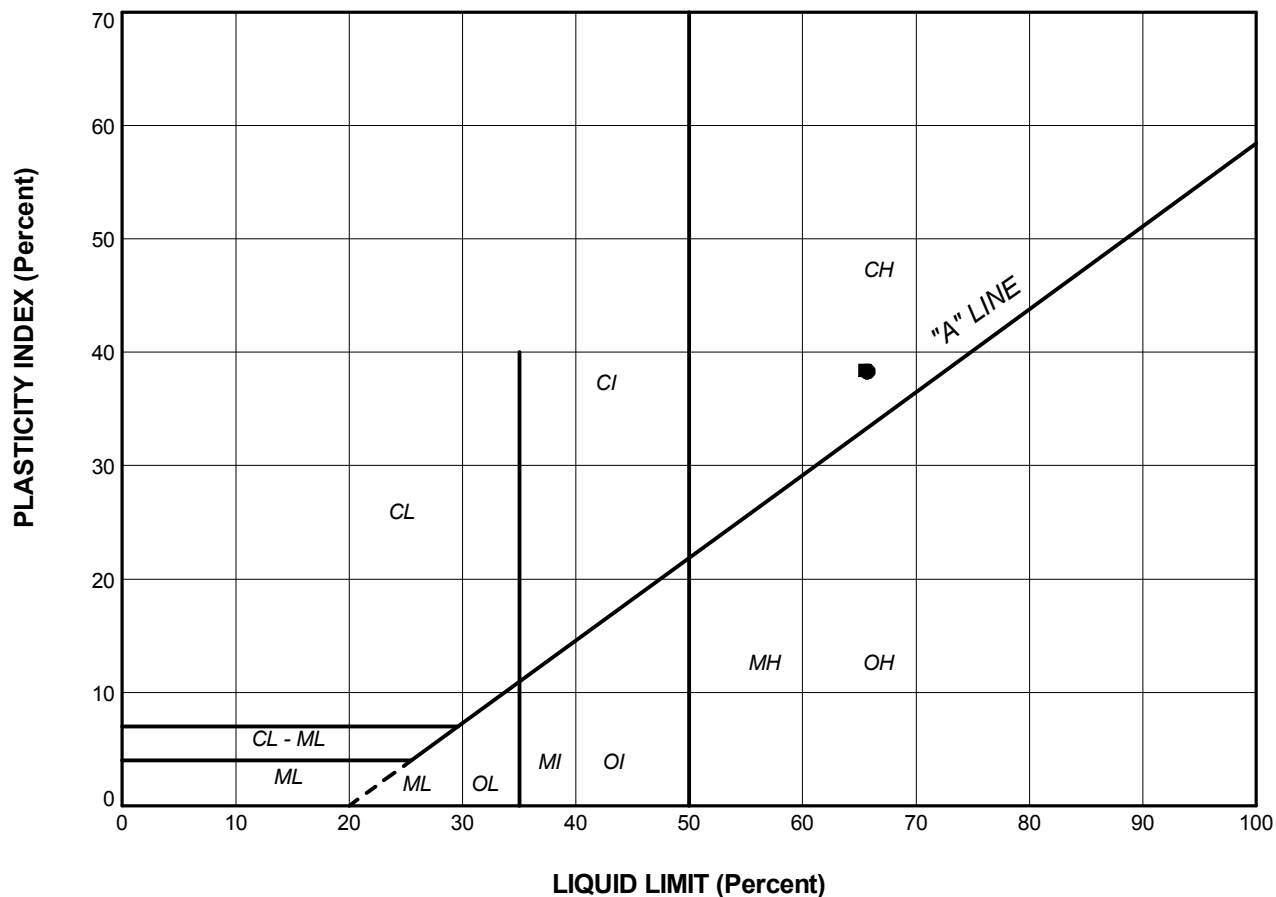
Prepared by: AC
Checked by: SEMP
Reviewed by: JMAC



LEGEND

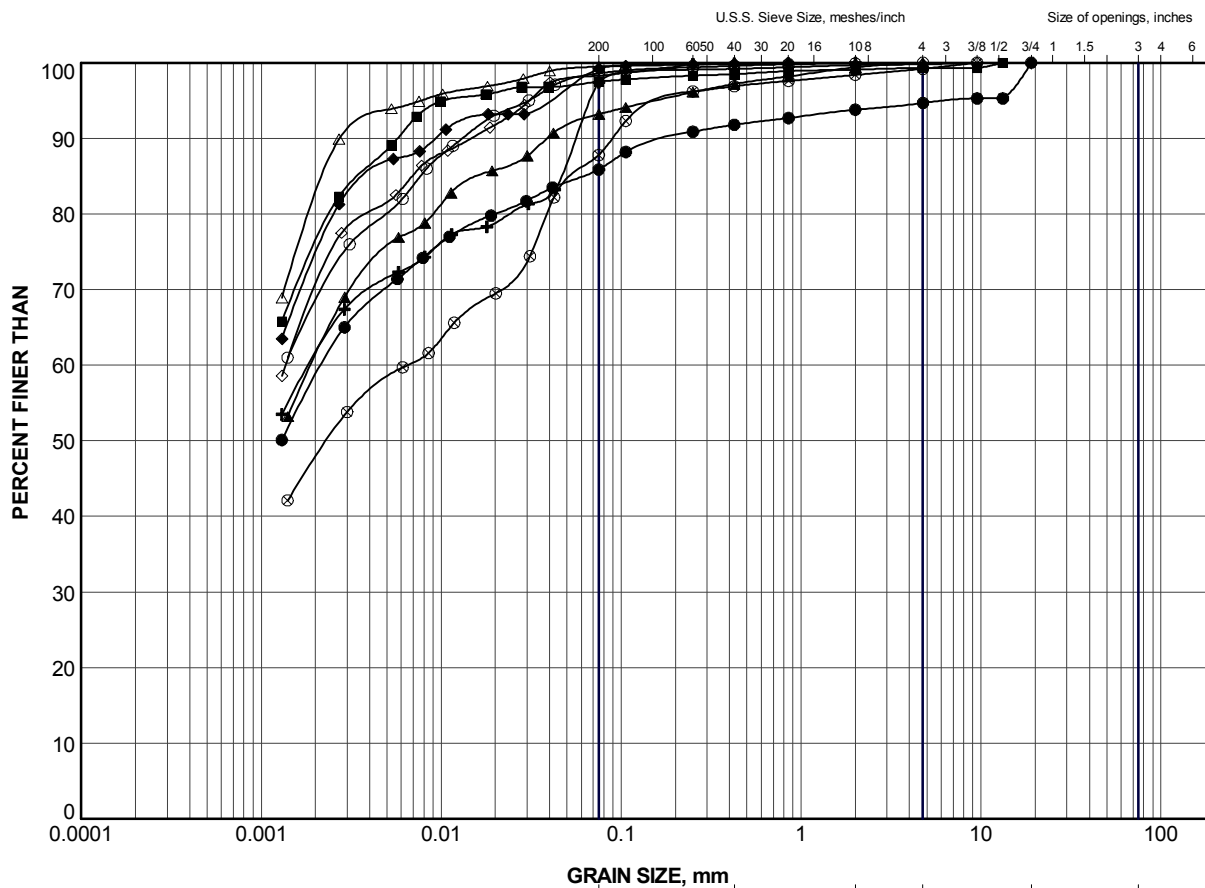
SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	BR-3	2	381.5

PROJECT					
HIGHWAY 11/17 BRULE CREEK CULVERT STA 15+304					
TITLE					
GRAIN SIZE DISTRIBUTION SAND (FILL)					
PROJECT No.		1533879		FILE No. 1533879.GPJ	
DRAWN	JJL	Feb 2016	SCALE	N/A	REV.
CHECK	SEMP	Feb 2016			
APPR	JMAC	Feb 2016			
 Golder Associates SUDBURY, ONTARIO			FIGURE B1		



PROJECT					
HIGHWAY 11/17 BRULE CREEK CULVERT STA 15+304					
TITLE					
PLASTICITY CHART CLAY (FILL)					
PROJECT No. 1533879			FILE No. 1533879.GPJ		
DRAWN	JJL	Feb 2016	SCALE	N/A	REV.
CHECK	SEMP	Feb 2016	FIGURE B3		
APPR	JMAC	Feb 2016			




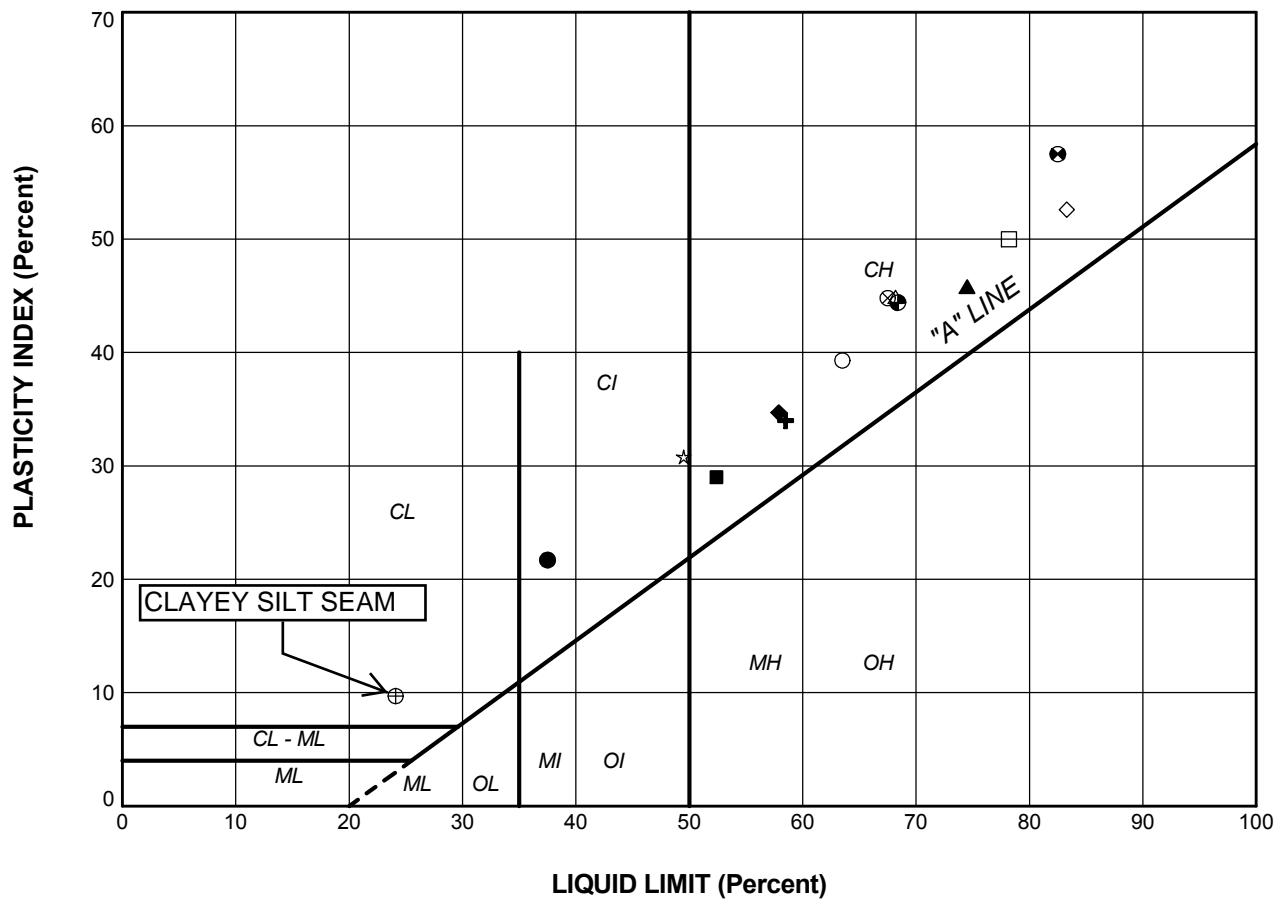


CLAY AND SILT	GRAVEL SIZE, mm					Cobble Size
	fine	medium	coarse	fine	coarse	
	SAND SIZE			GRAVEL SIZE		

LEGEND


SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	BR-1	4	372.5
■	BR-2	2	372.9
▲	BR-3	10	373.2
+	BR-3	12	370.1
◆	BR-4	4	373.3
◇	BR-4	7	371.0
○	BR-5	4	372.6
△	BR-6	9	374.9
⊗	BR-7	5	370.7

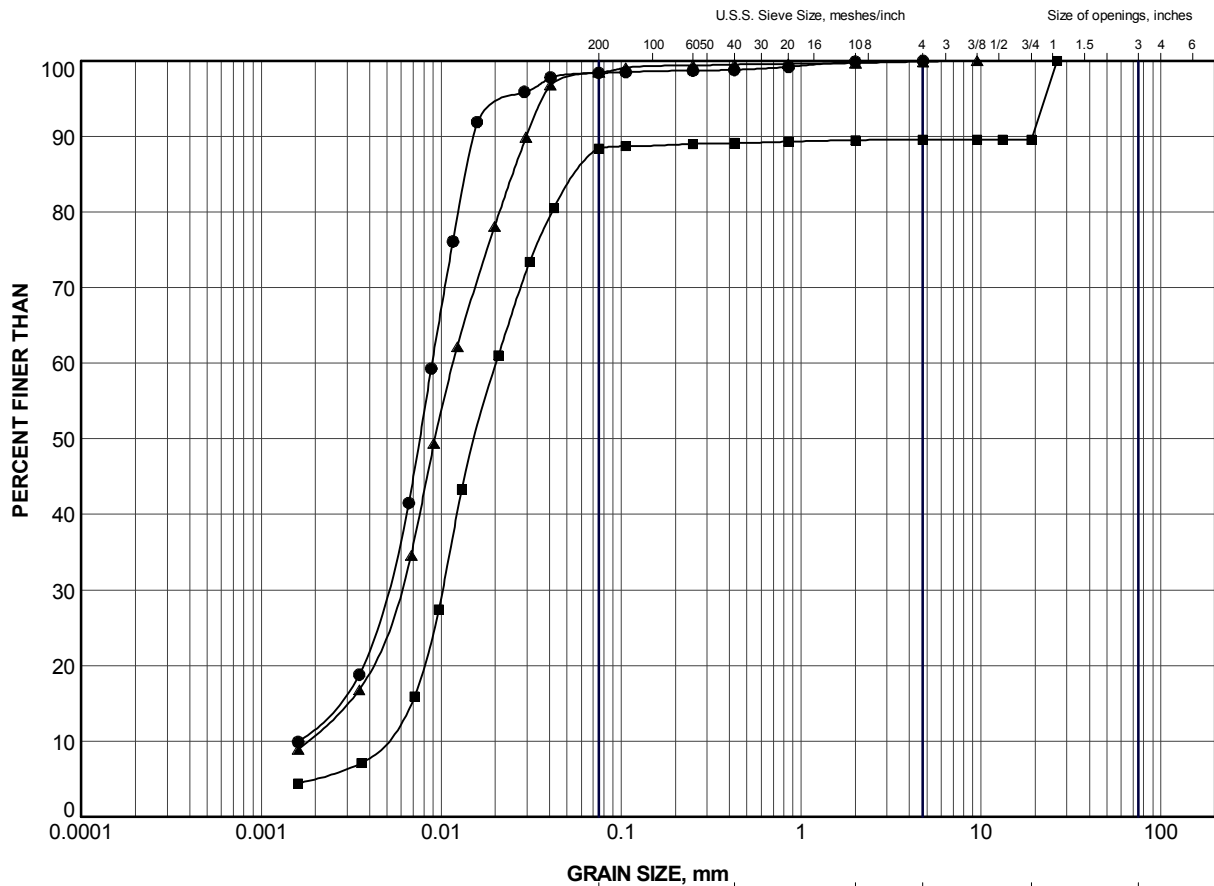
PROJECT					HIGHWAY 11/17 BRULE CREEK CULVERT STA 15+304				
TITLE					GRAIN SIZE DISTRIBUTION SILTY CLAY to CLAY				
PROJECT No.			1533879		FILE No.			1533879.GPJ	
DRAWN	JJL	Feb 2016	SCALE	N/A	REV.				
CHECK	SEMP	Feb 2016							
APPR	JMAC	Feb 2016							
 Golder Associates SUDBURY, ONTARIO					FIGURE B4				



LEGEND


SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	BR-1	2	37.5	15.8	21.7
■	BR-1	4	52.4	23.4	29.0
▲	BR-2	2	74.5	28.7	45.8
+	BR-3	10	58.5	24.5	34.0
◆	BR-3	12	57.9	23.2	34.7
◇	BR-4	4	83.3	30.7	52.6
○	BR-4	7	63.5	24.2	39.3
△	BR-5	3B	68.2	23.3	44.9
⊗	BR-5	4	67.5	22.7	44.8
⊕	BR-5	7	24.1	14.4	9.7
□	BR-6	9	78.2	28.2	50.0
⊗	BR-6A	4	82.5	25.0	57.5
●	BR-7	2	68.4	24.0	44.4
☆	BR-7	5	49.5	18.7	30.8

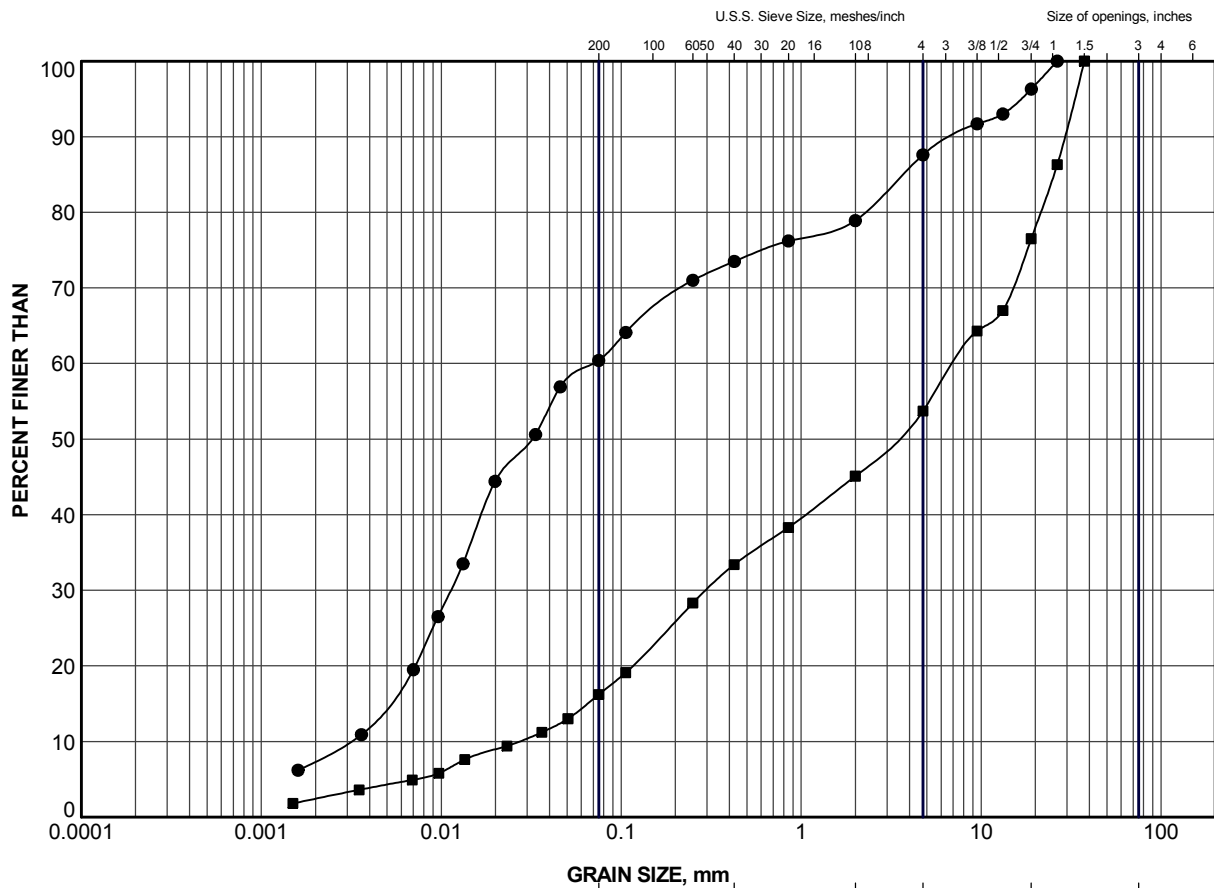
PROJECT				
HIGHWAY 11/17 BRULE CREEK CULVERT STA 15+304				
TITLE				
PLASTICITY CHART SILTY CLAY to CLAY				
PROJECT No.		1533879		FILE No.
DRAWN		JJL	Apr 2016	SCALE
CHECK		SEMP	Apr 2016	REV.
APPR		JMAC	Apr 2016	
 Golder Associates SUDBURY, ONTARIO		FIGURE B5		



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	BR-1	9	367.2
■	BR-2	6	368.3
▲	BR-7	8	366.4


PROJECT						HIGHWAY 11/17 BRULE CREEK CULVERT STA 15+304					
TITLE						GRAIN SIZE DISTRIBUTION SILT					
PROJECT No.				1533879		FILE No.				1533879.GPJ	
DRAWN		JJL		Feb 2016		SCALE		N/A		REV.	
CHECK		SEMP		Feb 2016							
APPR		JMAC		Feb 2016							
 Golder Associates SUDBURY, ONTARIO						FIGURE B6					



GRAIN SIZE, mm						
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	BR-3	16	364.3
■	BR-6A	5	370.3

PROJECT						HIGHWAY 11/17 BRULE CREEK CULVERT STA 15+304					
TITLE						GRAIN SIZE DISTRIBUTION SANDY SILT (TILL) to SAND AND GRAVEL (TILL)					
PROJECT No.				1533879		FILE No.				1533879.GPJ	
DRAWN	JJL	Apr 2016		SCALE	N/A	REV.					
CHECK	SEMP	Apr 2016									
APPR	JMAC	Apr 2016									
 Golder Associates SUDBURY, ONTARIO				FIGURE B7							



APPENDIX C

Non-Standard Special Provisions (NSSPs)

OBSTRUCTIONS

Non-Standard Special Provision

As part of the work for the culvert installation at the Brule Creek culvert, the Contactor shall be alerted to the presence of cobble size material as encountered in Boreholes BR-3 and BR-6 within the fill and in Borehole 3 within the native sandy silt till soils at this site.

WORKING SLAB - Item No.

Non-Standard Special Provision

1.0 SCOPE

This Special Provision covers the requirements for the supply and placement of a concrete working slab under structure foundations.

2.0 REFERENCES

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

Ontario Provincial Standard Specifications, Construction

OPSS 902 Excavating and Backfilling - Structures

3.0 DEFINITIONS - Not Used

4.0 DESIGN AND SUBMISSION REQUIREMENTS - Not Used

5.0 MATERIALS

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

6.0 EQUIPMENT - Not Used

7.0 CONSTRUCTION

7.01 Excavation

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

7.02 Protection of Founding Soil

Following inspection and approval of the prepared subgrade, a working slab with a minimum thickness of 100 mm shall be placed on the foundation subgrade as specified in the Contract Documents.

7.03 Dewatering

Dewatering shall be carried out according to OPSS 902.

8.0 QUALITY ASSURANCE - Not Used

9.0 MEASUREMENT FOR PAYMENT - Not Used

10.0 BASIS OF PAYMENT

10.01 Working Slab - Item

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

UNWATERING OF STRUCTURE EXCAVATION - Item No.

Non-Standard Special Provision

Construction of the culvert will require excavations to extend below the groundwater level. The cohesionless soils that are present below the groundwater table at this site will slough, run, boil or cave into the excavation unless appropriate groundwater controls are in place. The Contractor is to design and install an appropriate excavation protection and unwatering system to enable construction and prevent disturbance to the founding soils.

Basis of Payment

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

END OF SECTION

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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