GOLDER

FOUNDATION INVESTIGATION AND DESIGN REPORT

CULVERT 2 - STA. 10+097, HIGHWAY 539A LATITUDE 46.612665; LONGITUDE -80.204985 TOWNSHIP OF CRERAR, ONTARIO MINISTRY OF TRANSPORTATION, ONTARIO GWP 5066-17-00, WP 5162-16-01

GEOCRES NO.: 41I-354

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PART A

FOUNDATION INVESTIGATION REPORT CULVERT 2 – STA 10+097, HIGHWAY 539A TOWNSHIP OF CRERAR, ONTARIO MINISTRY OF TRANSPORTATION, ONTARIO GWP 5066-17-00, WP 5162-16-01

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by D.M. Wills Associates Ltd. (D.M. Wills), on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services for the replacement of the Culvert 2 located on Highway 539A at about Station 10+097 approximately 4.75 km west of the 539/539A junction near River Valley, in the North Bay District in the Township of Crerar, Ontario. The key plan of the general location of this section of Highway 539A and the location of the investigated area are shown on Drawing 1.

2.0 SITE DESCRIPTION

The existing Culvert 2 consists of a structural plate corrugated steel pipe arch (SPCSPA). The dimensions (i.e., diameter, length, etc.) of the existing culvert 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 purpose of this report, Highway 539A is oriented in a north-south direction with the culvert on a slight skew from perpendicular to the highway generally in a west-east orientation.

In general, the topography within the vicinity of the culvert consists of relatively flat terrain, which is heavily forested beyond the highway right-of-way (ROW). There is also a relatively low-lying swampy area southwest of the culvert location. At the culvert location, the highway grade is at approximately Elevation 234.0 m and the embankments are approximately 1.8 m to 2.1 m high relative to the ground surface at the toe of embankment (or about 3.7 m relative to the existing culvert invert). The existing culvert invert is at Elevation 230.3 m at the both the inlet (east end) and outlet (west end). The creek water level, as surveyed by Golder on October 25, 2017, was Elevation 231.8 m. The ground surface conditions at the culvert location are shown on Photographs 1 to 4.

3.0 INVESTIGATION PROCEDURES

The field work for this subsurface investigation was carried out between October 16 and November 11, 2017, during which time six boreholes (Boreholes C2-1 to C2-6) were advanced at approximately the locations shown on Drawing 1. Boreholes C2-1, C2-2, C2-5, and C2-6 were advanced near the toes of the Highway 539A embankment slopes at the culvert inlet/outlet ends. Boreholes C2-3 and C2-4 were advanced from the roadway platform. All boreholes were advanced using a track-mounted CME-55 drill rig, with the exception of Borehole C2-5, which was advanced using with a portable tripod. Both drill rigs were supplied and operated by George Downing Estate Drilling of Grenville-sur-la-rouge, Quebec. Traffic control was performed by Barlett Towing's of North Bay, Ontario, in accordance with the Ontario Traffic Control Manual Book 7 – Temporary Conditions.

The boreholes were advanced using 108 mm inside diameter hollow-stem augers and/or NW casing with wash boring techniques using water from the local creek for wash boring operations. 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 or a cathead hammer (for boreholes advanced using the portable tripod) in accordance with the Standard Penetration Test (SPT) procedure (ASTM D1586). Field vane shear tests were conducted in cohesive soils for determination of undrained shear strengths (ASTM D2573) using an MTO Standard 'N' size vane. Samples of the cohesive soils were obtained using 76 mm O.D. thin walled Shelby Tubes (ASTM D1587) for relatively undisturbed samples. The groundwater levels in the open boreholes were observed during the drilling operations as described on the borehole records in Appendix A. All 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 a member 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; and logged the boreholes. 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 tests were carried out on selected soil samples. The geotechnical laboratory testing was completed according to ASTM and MTO LS standards, as applicable.

A soil sample was obtained during the field investigation at the Culvert 2 location on October 17, 2017, 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 results of the analytical testing are presented in Table B1 in Appendix B.

The as-drilled borehole locations were measured by a member of our technical staff (relative to the existing culvert and roadway centreline) and converted into northing/easting coordinates on the plan drawing. The ground surface elevations at the borehole locations were surveyed relative to a nearby benchmark and the benchmark elevation was obtained from the plan drawing (B-270-539A.pdf) provided by D.M. Wills. The MTM NAD 83 (Zone 10) northing and easting coordinates, geographical coordinates, ground surface elevations referenced to Geodetic datum, and borehole depths at each borehole location are presented on the borehole records in Appendix A and summarized below.

Borehole Number	MTM NAD 83 Northing (m) (Latitude)	MTM NAD 83 Easting (m) (Longitude)	Ground Surface Elevation (m)	Borehole Depth (m)
C2-1	5163904.7 (46.612589)	250801.3 (-80.204979)	233.2	11.3
C2-2	5163914.1 (46.612673)	250791.5 (-80.205108)	232.2	11.3
C2-3	5163905.6 (46.612597)	250807.5 (-80.204898)	233.9	15.8
C2-4	5163916.7 (46.612697)	250799.9 (-80.204998)	234.0	15.8
C2-5	5163914.8 (46.612681)	250810.5 (-80.204859)	231.9	8.2
C2-6	5163924.6 (46.612768)	250803.1 (-80.204957)	232.2	9.8

Note: Borehole C2-5 was terminated due to refusal to further casing advancement.

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

Based on the Northern Ontario Engineering Geology Terrain (NOEGTS) mapping by the Ministry of Natural Resources[1], the subsoils in the vicinity of the Culvert 2 site are comprised of outwash plain, valley train deposits, consisting primarily of sand and silt materials bordered by knobby/hummocky bedrock knobs.

Based on geological mapping by the Ministry of Northern Development and Mines[2], the site is underlain by mafic and ultramafic intrusive bedrock comprised of gabbro and anorthosite.

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 borehole records 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 vane values) as presented on the borehole records and in Section 4 are uncorrected. The stratigraphic boundaries shown on the boreholes records and on the interpreted stratigraphic section, profile and cross-section 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 roadway platform) and topsoil/peat (for boreholes advanced near the embankment toe of slope) overlying deposits of silt, clayey silt, sand and silt to silt and sand, and sand. A more detailed description of the soil deposits and groundwater conditions encountered in the boreholes is provided below.

Deposit/Layer Description	Boreholes	Stratum Surface Elevation (m)	Stratum Thickness (m)	SPT N Values (blows/0.3 m) Shear Strength (kPa) Relative Density or Consistency	Laboratory Testing
Asphalt[1]	C2-3 & C2-4	233.9 & 234.0	0.050	n/a	n/a
(FILL) Sand to **Sand and Gravel**, trace to some silt, trace organics, brown to grey, moist to wet	C2-1 to C2-6	233.9 – 231.5	0.7 – 3.9	N = 3 – 26 **Very Loose to Compact**	w = 11% – 18% 4 – M (Fig. B1)
		233.2 – 232.2	0.6 - 0.7	N = 4 – 11	n/a

[1] Ministry of Natural Resources, Northern Ontario Engineering Geology Terrain Study. Ontario Geological Society Electronic Mapping. Map 41INE

[2] Ministry of Northern Development of Mines. Bedrock Geology of Ontario – East Central Sheet, Ontario Geological Survey - Map 2543

Deposit/Layer Description	Boreholes	Stratum Surface Elevation (m)	Stratum Thickness (m)	SPT N Values (blows/0.3 m) Shear Strength (kPa) Relative Density or Consistency	Laboratory Testing
Silty Sand **Topsoil / Peat,** dark brown to black, moist to wet	C2-1 C2-2 & C2-6			**Very Loose to Compact**	
Silt, trace gravel, trace to some sand, trace to some clay, grey, wet	C2-1 to C2-6	230.9 – 228.7	0.8 – 3.1	N = 0 (weight of hammer) – 8 **Very Loose to Loose**	w = 28% & 29% 2 – MH (Fig. B2) 2 – NP
Clayey Silt trace sand, grey to brown, wet	C2-1 to C2-6	229.5 – 227.8	2.0 – 3.7	N = 2 – 4 Su = 14 – 67 **Soft to Stiff**	w = 28% – 36% 5 – ATT (Fig. B3) w_l = 26% - 30% w_p = 19% - 23% I_p = 6% - 8% 4 – MH (Fig. B4)
Silt and Sand, trace to some gravel, trace clay; grey to brown, wet	C2-1 to C2-5	226.4 – 224.1	1.4 – 4.1	N = 4 - 22 **Loose to Compact**	w = 22% – 26% 4 – MH (Fig. B5)
Sand, trace to some silt, tract gravel; grey, wet	C2-2 to C2-6	225.5 – 222.0	1.0 – 4.1 (not fully penetrated in C2-2, C2-3, C2-4 or C2-6)	N = 4 – 41 **Loose to Dense**	w = 18% - 28% 2 – M (Fig. B6)

Where:
N = SPT 'N'-value; number of blows for 0.3 m of penetration
s_u = Undrained Shear Strength (kPa)
w = Natural Moisture Content (%)
M = Sieve analysis for particle size
MH = Combined sieve and hydrometer analysis
ATT = Atterberg Limits Testing
w_p = Plastic Limit (%)
w_l = Liquid Limit (%)
I_p = Plasticity Index (%)
NP = Non-Plastic Atterberg Limits Test

[1.] A 50 mm layer of buried asphalt was encountered in Boreholes C2-3 and C2-4 between the upper sand and gravel fill and lower gravelly sand to sand fill layers at about 0.3 m depth.

4.3 Groundwater Conditions

The unstabilized groundwater levels measured in the open boreholes upon completion of drilling are summarized below. The creek water level, as surveyed by Golder on October 25, 2017, was at Elevation 231.8 m. Groundwater and creek water levels in the area are subject to seasonal fluctuations and variations due to precipitation events.

Borehole No.	Depth to Unstabilized Groundwater Level (m)	Approximate Groundwater Elevation (m)
C2-1	1.4	231.8
C2-2	0	232.2
C2-3	0.9	233.0
C2-4	1.2	232.8
C2-5	0.2	231.7
C2-6	0.4	231.8

Note: A full head of water was introduced into the hollow stem augers while advancing Borehole C2-2 in order to mitigate heaving at the base of the borehole. In addition, Boreholes C2-4 and C2-5 were advanced using NW casing and wash boring techniques. As such, these groundwater levels may not be representative of in situ groundwater conditions.

5.0 CLOSURE

The field drilling program was supervised by Mr. Mat Riopelle. This Foundation Investigation 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., a Designated MTO Foundations Contact and Senior Consultant for Golder conducted an independent quality control review of this report.

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PART B

FOUNDATION DESIGN REPORT
CULVERT 2 – STA 10+097, HIGHWAY 539A
TOWNSHIP OF CRERAR, ONTARIO
MINISTRY OF TRANSPORTATION, ONTARIO
GWP 5066-17-00, WP 5162-16-01

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides foundation design recommendations for the proposed replacement of the Culvert 2 at Station 10+097 on Highway 539A. 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 designer with sufficient information to assess the feasible foundation alternatives and carry out the design of the structure foundations, as may be required. The Foundation Investigation Report, discussion and recommendations are intended for the use of the MTO and shall not be used or relied upon for any other purpose or by any other parties, including the construction or design-build contractor. The contractor must make their own interpretation based on the factual data in the Foundation Investigation Report (Part A of this report). Where comments are made on construction, they are provided to highlight those aspects that could affect the design of the project and for which special provisions may be required in the Contract Documents. Those requiring information on the aspects of construction must make their own interpretation of the factual information provided as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

6.1 General

Culvert 2 is located in the North Bay District in the Township of Crerar on Highway 539A at about Station 10+097, approximately 4.75 km west of the Highway 539/539A junction near River Valley, Ontario. The highway embankment is comprised of granular fill material and is about 1.8 m to 2.1 m high relative to the natural ground surface near the toe of the embankment slope (or about 3.7 m relative to the existing culvert invert). The existing culvert consists of an SPCSPA with the dimensions provided in Table 1.

A box culvert, open footing culvert or pipe culvert are all considered feasible alternatives for replacement of the existing culvert at this site. Although feasible, an open footing culvert presents greater challenges as it will extend the construction schedule and increase the depth of the excavation, dewatering and shoring requirements compared with a box culvert. Given the relatively low embankment height and limited soil cover, multiple pipe culverts would likely be required to provide a similar flow-through capacity compared to a box or open footing culvert and, if constructed from steel, a CSP culvert will likely have a shorter design life as the replacement structure for this site. From a foundation perspective, a closed-bottom box culvert sufficiently wide to handle the creek flow is preferred. A different culvert type may be preferred due to 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.

Based on the preliminary General Arrangement (GA) drawing provided by D.M. Wills, we understand that the replacement culvert is to consist of a precast, single-cell concrete box culvert approximately 2.4 m wide by 1.8 m high (interior dimensions) with the inverts at Elevations 231.0 m and 230.9 m at the inlet and outlet ends, respectively. We also understand that the culvert will be placed on a slightly revised alignment from the existing culvert such that the replacement culvert runs perpendicular to the highway rather than on the slight skew of the existing culvert alignment.

6.2 Consequence and Site Understanding Classification

As the proposed replacement culvert crosses Highway 539A and has the potential to impact alternative transportation corridors, a “typical consequence level” is considered appropriate for this structure as outlined in Section 6.5 of the Canadian Highway Bridge Design Code (CHBDC 2014) and its Commentary. Further, given the scope of work of the foundation field investigation and laboratory testing program as presented in Sections 3.0 and 4.0, a “typical degree of site and prediction model understanding” has been utilized. Accordingly, the appropriate corresponding ULS and SLS consequence factor, Ψ, and geotechnical resistance factors, Φgu and Φgs, from Tables 6.1 and 6.2 of the CHBDC (2014) have been used for design.

6.3 Culvert Foundation Design Recommendations

6.3.1 Founding Level and Geotechnical Resistance

Prior to placing the bedding/levelling pad and replacement culvert, it is recommended that all organic material (i.e., topsoil, peat and/or mixed organic soils) and existing fill encountered below the culvert footprint be sub-excavated and replaced with Ontario Provincial Standard Specification, Provincial Oriented (OPSS.PROV) 1010 Granular ‘A or Granular ‘B’ Type II fill.

For a proposed 2.8 m wide box culvert (2.4 m wide interior) founded on a properly prepared granular bedding/levelling pad overlying the native subgrade soils or layered zone of the existing gravelly sand fill at Elevation 230.3 m and 230.2 m at the inlet and outlet, respectively (taking into account the invert Elevations noted in Section 6.1, a 0.3 m thick concrete bottom slab, a 75 mm levelling course and a 0.3 m bedding layer), a factored ultimate geotechnical axial resistance of 115 kPa and a factored serviceability geotechnical resistance of 50 kPa (for 25 mm of settlement) may be used for design.

Based on discussions with D.M. Wills, we understand that a factored ultimate geotechnical resistance of 115 kPa and a factored serviceability geotechnical resistance of 110 kPa are required for design of the proposed replacement box culvert. Based on the soil conditions and proposed invert elevations, settlements of about 50 mm are estimated for the anticipated loading conditions, for the culvert foundation conditions as noted above, and provided there is no grade raise or widening of the embankment. Provided that settlements of about 50 mm can be tolerated by the new culvert structure, a factored serviceability geotechnical resistance of 110 kPa may be used for design. Actual settlements will likely be less than the calculated settlements, as the serviceability geotechnical resistance has been factored in accordance with the CHBDC (2014) and considering that the existing highway grade is being maintained (i.e. no additional embankment loading).

If settlements up to about 50 mm cannot be tolerated, consideration must be given to partial sub-excavation and replacement of the lower layer of existing fill and the generally very loose silt deposit; however, we understand that this may not be feasible given the limited extent of separation to the current MTO ROW. However, such excavation could be carried out to the ROW limit if a temporary protection system such as a sheet pile wall, is first installed t along the ROW.

In the event that an open footing culvert is chosen as the replacement option, a factored ultimate geotechnical resistances of 95 kPa and a factored serviceability geotechnical resistance of 75 kPa (based on 25 mm total settlement) may be used for design of an assumed 1.0 m wide footing founded at/or below Elevation 229.0 m and 228.9 m at the inlet and outlet ends, respectively, to provide for a minimum of 2.0 m of soil cover for protection against frost penetration, as interpreted from OPSD 3090.100.

The factored geotechnical resistances provided above are based on the loading applied perpendicular to the base of the culvert/footings; where applicable, inclination of the load should be taken into account in accordance with Section 6.10.4 and Section C6.10.4 of CHBDC (2014) and it’s Commentary. The factored geotechnical resistances should be reviewed if the founding elevation and/or the foundation widths differ from those given above.

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. The factored geotechnical serviceability resistances provided above assume there will not be any temporary and/or permanent grade raise at the culvert location (including during the course of construction).

6.3.2 Frost Protection

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

6.3.3 Resistance to Lateral Loads/Sliding Resistance

Resistance to lateral forces / sliding resistance should be calculated in accordance with Section 6.10.5 of CHBDC (2014), applying the appropriate consequence and degree of site understanding factors as noted above in Section 6.2. A coefficient of friction, tan δ_i', of 0.45 may be used at the interface between the base of the pre-cast box culvert and the granular bedding material.

6.4 Stability, Settlement and Horizontal Strain

6.4.1 Embankment Stability

Based on the survey cross-section drawings provided by D.M. Wills, the existing embankment side slopes are relatively steep with embankment west and east side slopes oriented at about 1.5 horizontal to 1 vertical (1.5H:1V) and 1H:1V. Based on our site observations of the condition of the embankment in the immediate area of the culvert, the available photographic information and discussions with MTO Technical Services, we understand that the existing embankment is currently, and has historically performed adequately.

Based on discussions with D.M. Wills, we understand that given the limited available space to the MTO ROW, the embankments are to be re-constructed at a side slope of about 1.4H:1V, which we understand is the flattest side slope that can be accommodated while still providing sufficient space along the toes (about 1.2 m) for the Contractor to carry out the construction works and to accommodate construction equipment for future maintenance purposes. Based on discussions with D.M. Wills, we understand that an approximately 1 m narrowing of presently available space will be required relative to the west toe of slope to accommodate the proposed slightly steeper 1.4H:1V side slope. On the east side, we understand that an approximately 1m widening will be required relative to the east toe of the embankment along with a slight reduction in shoulder width to accommodate the flatter 1.4H:1V side slope.

Limit equilibrium slope stability analysis was carried out for the both the existing and proposed re-constructed highway embankments 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.

For the purpose of the stability analysis, the FoS is equal to the inverse of the product of the consequence factor, Ψ, and the geotechnical resistance factor, Φ_{gu} (i.e., FoS = 1 / ($\Psi * \Phi_{gu}$)). Accordingly, a target minimum FoS of 1.33 and 1.54 has been used for the design of the embankment slopes for temporary (short-term) and permanent (long-term) conditions, respectively, as per Table 6.2 of CHBDC (2014). The stability analyses assume that all organics and other deleterious materials are removed below the final embankment footprint.

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

Soil Deposit	Bulk Unit Weight (kN/m^3)	Short-Term Analysis		Long-Term Analysis
		Effective Friction Angle (°)	Undrained Shear Strength (kPa)	Effective Friction Angle (°)
New Granular Fill (compacted Granular A or Granular B Type II)	21	35-40	-	35-40
Existing Granular Fill (very loose to compact)	20	32	-	32
Silt (very loose to loose)	18	28	-	28
Clayey Silt (soft to stiff)	17	-	35	27
Sand and Silt to Silt and Sand (loose to compact)	19	29	-	29
Sand (loose to dense)	20	30	-	30

Based on the analyses, the existing east side slope generally has a lower factor of safety compared to the west slope and the critical slip surfaces are relatively shallow extending through the very loose to loose cohesionless silt deposit. As such, the FoS for both the drained and undrained cases are identical. Therefore, the long-term (drained) analyses, with the higher target FoS, will govern for embankment design.

The results of the analysis indicate that the existing east side slope at an inclination of 1H:1V has a FoS for global stability of 1.08 and as such, does not satisfy the minimum target FoS for either temporary or permanent conditions. For the highway embankment reconstructed at a proposed side slope of 1.4H:1V utilizing an OPSS.PROV 1010 Granular B Type I fill material with an internal friction angle of 35°, a FoS of 1.28 is anticipated for the re-constructed embankment (see Figure 1). This embankment reconstruction strategy does not satisfy the stability requirements for temporary conditions, and hence is also not for long-term, permanent conditions. However, if the subgrade is sub-excavated to at least Elevation 230.0 m and the backfill under the culvert and reconstructed embankment is comprised of Granular B Type I material, a FoS of about 1.36 is obtained, which still does not satisfy the permanent condition requirement (See Figure 2).

In order to a achieve a minimum target FoS of 1.5 for long-term stability, the sub-excavated area to Elevation 230.0 m and the proposed 1.4H:1V embankment would need to be re-constructed using OPSS.PROV 1010 Granular A or Granular B Type II material, with a minimum friction angle of at least 40° being utilized for design (see Figure 3). Based on historic shear box testing performed by Golder and on published information, it is our opinion that an internal friction angle of 45° (or even higher) can be achieved for either a Granular A or Granular B Type II backfill material. Note that Golder has not conducted shear box texting on Granular B Type I materials and as such, we would not suggest that a friction angle of 40° be adopted for an embankment re-constructed using Granular B Type I material.

From a foundations perspective, it is our opinion that reconstructing the existing highway embankment at a side slope of 1.4H:1V using Granular A or Granular B Type II fill (with an internal friction angle of 40° for design) is the most cost-effective approach to achieving the minimum target FoS of greater than 1.5 for global stability.

Given the relatively low traffic volumes along Highway 539A, which is currently a gravel-surfaced roadway and the limited height of the embankments, consideration could be given by MTO to utilizing a "low" consequence factor, which would reduce the target minimum FoS against global stability to about 1.16 and 1.34 for the short-term and long-term conditions, respectively. As indicated above, these minimum target FoS can be achieved if the subgrade is sub-excavated to at least Elevation 230.0 m and the backfill under the culvert and re-constructed embankment is comprised of Granular B Type I material. Based on discussions with MTO, we understand that this is their preferred option, given their observations over time of the satisfactory performance and low level of maintenance requirements of the road.

If a "low" consequence factor is not adopted by the MTO and if MTO considers that a less conservative internal friction angle of 40° is not appropriate for an embankment re- constructed using OPSS.PROV 1010 Granular A or Granular B Type II material, then stability mitigation measures will be required as discussed in Section 6.4.2.

6.4.2 Stability Mitigation Measures

Embankment stability mitigation measures for this site (if required) may include the following:

- flattening of the embankment side slope to 2H:1V in the culvert/embankment reconstruction area
- partial sub-excavation and replacement of the very loose to loose silt deposit to greater depths (i.e., lower Elevation than 230.0 m)
- retaining/head wall systems
- use of a modified Granular B Type II material with attendant laboratory testing to verify that an internal friction angle of Φ>40° can be achieved

Based on discussions at the Design Team Review Meeting (DTRM) held January 24, 2018, it is our understanding that the consensus of the D.M. Wills and MTO team is that the above noted stability mitigation measures are likely not practical from a scheduling and/or cost perspective, particularly given that this culvert is located on a secondary highway with relatively low traffic volumes, the fairly low embankment heights and considering that the existing embankment side slopes (including the steeper 1H:1V east side slope) are performing satisfactorily. The stability mitigation measures and associated constraints (as discussed at the DTRM) are presented below for consideration. A comparison of culvert types based on advantages, disadvantages and risks/consequences are also presented in Table 3.

Based on discussions with D.M. Wills, we understand that given the limited space to the ROW limits at this site, property acquisitions would be required to accommodate 2H:1V side slopes and/or to allow for partial sub-excavation and replacement of the very loose to loose silt deposit. Based on discussions with MTO, we also understand that property acquisitions could potentially delay the project by a period of about 12 months to 18 months; however, it has been recognized that the adjacent property is crown land, which could potentially result in a shorter acquisition period.

Alternatively, consideration can be given to the use of a retaining wall system along the ROW at both the culvert inlet and outlet ends. If a retaining wall system is to be considered, given that post-construction settlements are anticipated at this site (as discussed in Section 6.3), we recommend the use of a gabion wall as gabion walls are more tolerant to differential settlement compared to concrete wingwalls and/or RSS walls. Further, it is our understanding that MTO typically does not recommend the use of RSS wall systems near flowing creeks/streams due to risks associated with erosion/scour, which could impact long-term wall performance.

Consideration was also given to the use of a modified Granular B Type II, with a customized (coarser) gradation to further justify the use of a less conservative (higher) internal friction angle for this material beyond that typical utilized/accepted by MTO. However, based on discussions with MTO, we understand that the additional costs associated with a purchase of a limited quantity of a modified Granular B Type II material may not be practical from a financial perspective and further consideration should be given to the use of conventional OPSS.PROV 1010 materials.

6.4.3 Settlement and Horizontal Strain

Given the presence of the very loose to loose silt and soft to stiff clayey silt deposits below the proposed culvert and the anticipated loading conditions provided by D.M. Wills, factored settlements of up to about 50 mm are anticipated for the proposed replacement box culvert. The settlement will be relatively uniform across the roadway platform given the fairly uniform embankment loading conditions and considering the limited widening being proposed as part of the culvert replacement works. As such a culvert camber is not considered necessary. However, some differential settlement may occur at the culvert inlet end as a result of the proposed flattening/widening of the east side slope of the re-constructed embankment.

The estimated 50 mm of total culvert settlement is comprised of about 30 mm of immediate settlement within the cohesionless silt, sand and silt to silt and sand, and underlying sand deposits and about 20 mm of short-term consolidation settlement of the cohesive clayey silt deposit. Although concrete box culverts are tolerant of small magnitudes of differential settlement, consideration could be given to the use of dowels and/or steel strapping to mitigate differential movement between the pre-cast box culvert units.

Given that the settlement associated with compression of the cohesionless deposits will occur almost immediately (i.e. during culvert backfilling and embankment re-construction), post-construction settlement of the finished road surface is anticipated to be limited to the estimated 20 mm of consolidation settlement noted above. Based on an inferred/estimated coefficient of consolidation (c_v) equal to 4.0×10^{-2} cm^2/s, for the clayey silt deposit it is estimated that a majority (i.e., 90%) of the consolidation settlement within the up to 3.7 m thick clayey silt deposit will occur over a period of approximately one to two weeks following completion of backfilling.

Based on MTO's "*Embankment Settlement Criteria for Design*" (MTO, July 2010), the maximum total settlement for a non-freeway longitudinal transition within 20 m of the culvert structure is 25 mm within 20 years following final paving. As such, it is considered that embankment settlement mitigation measures are not required at this site; however, it would be prudent to incorporate an operational constraint (OC) into the tender documents to delay final paving of the roadway by a period of two weeks to mitigate risks associated with post-construction settlement impacts to the final paved surface. However, it is recognized that this may not be practical given the proposed rapid culvert replacement strategy, which will utilize a 72-hour, full road closure period.

If is estimated that about 5 mm of total settlement will occur at the toe of the existing east embankment slope as a result of the small wedge of embankment widening/flatting at the east toe of embankment slope. This could result in a similar magnitude of differential settlement occurring at the east (inlet) end of the culvert but should not impact the existing roadway platform (i.e. driving lanes and shoulders).

6.5 Lateral Earth Pressures

The lateral earth pressures acting on the side walls of the culvert and retaining/head walls will depend on the type and method of placement of backfill materials, the nature of the 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.

Select, free draining, non-frost susceptible granular fill meeting the requirements of OPSS.PROV 1010 (Aggregates) Granular 'A' or Granular 'B' Type II should be used as backfill for the sub-excavated areas as well as behind the culvert walls, and on top of the culvert for a minimum thickness of 300 mm in a similar configuration to OPSD 803.010 (Backfill and Cover for Concrete Culverts). Backfill should be placed and compacted in accordance with OPSS.PROV 501 (Compacting). The following parameters and coefficients may be used in the wall design:

Fill Type	Internal Angle of Friction (ϕ)	Unit Weight (kN/m^3)	Coefficients of Static Lateral Earth Pressure		
			Active, K_a	At-Rest, K_o	Passive, K_p
Granular 'A'	35°*	22	0.27	0.43	3.69
Granular 'B' Type II	35°*	21	0.27	0.43	3.69

*Conservative value for the purposes of lateral earth pressure assessments.

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.

The total passive resistance below the base of the excavation (i.e. within the sheet pile cofferdam and/or adjacent to the temporary protection system) may be calculated based on the values of K_p indicated above but reduced by an appropriate factor that considers the allowable wall movement in accordance with Figure C6.16 of the CHBDC (2014) to account for the fact that a large strain would be required for mobilization of the full passive resistance.

6.6 Construction Considerations

6.6.1 Construction Staging and Temporary Roadway Protection

Based on discussions with D.M. Wills, it is understood that a full road closure with an open cut excavation is being proposed to allow for an accelerated construction schedule. The temporary excavation for the culvert replacement will be made through the existing embankment granular fill and into the very loose to loose silt deposit, potentially extending into the soft to stiff clayey silt stratum. The granular fill and native soils are considered to be Type 3 soil above the groundwater table and Type 4 soil below the groundwater table. 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. All excavations must be carried out in accordance with Ontario Regulation 213, Ontario Occupational Health and Safety Act for Construction Projects (as amended).

If temporary protection systems are required along the highway to facilitate a staged construction approach while maintaining traffic during the culvert replacement work, the temporary protection system could consist of either driven sheet-piling or soldier piles and lagging where H-piles would be driven to a suitable depth, with horizontal lagging installed as the excavation proceeds. Support to the system could be in the form of struts, wales, and rakers or anchors. Where required, the temporary protection system shall be designed and constructed in accordance with OPSS.PROV 539 (Temporary Protection Systems). Temporary protection systems should be designed to Performance Level 2 for any excavation adjacent to existing roadway.

Although the design of the temporary protection system will be completed by the contractor, the following parameters are provided to enable the structural designer to develop a conceptual design and assess the approximate construction costs for the protection systems, if adopted at this site:

Soil Type	Unit Weight (γ, kN/m^3)	Internal Angle of Friction (ϕ, degrees)	Cohesion (c_u, kPa)	Coefficient of Earth Pressure		
				Active, K_a	At Rest, K_o	Passive, K_p
New Granular Fill (compact)	21	35	-	0.27	0.43	3.69
Existing Granular Fill (very loose to compact)	20	32	-	0.31	0.47	3.25
Silt (Very Loose to Loose)	18	28	-	0.36	0.53	2.77
Clayey Silt (Soft to Stiff)	17	27	35	0.38	0.55	2.66
Sand and Silt to Silt and Sand (Loose to Compact)	19	29	-	0.35	0.50	2.88
Sand (Loose to Dense)	20	30	-	0.33	0.50	3.0

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 (2014) 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 above the top of the protection system, the coefficient of earth pressure should be adjusted accordingly.

The silt and/or clayey silt subgrade at this site is sensitive to disturbance from vibration and/or sheet pile/pile driving operations for wall installation, which should be considered in the design and installation of the temporary protection systems. Additionally, the 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 CHBDC (2014).

6.6.2 Control of Groundwater and Surface Water

Excavation at the culvert alignment will be required to remove the existing embankment fill, native soils and the existing SPCSPA culvert prior to placement of the engineered backfill and bedding materials, the new culvert structure, backfill/cover material and placement of the roadway pavement structure. Excavations for the box culvert will extend below the creek water level and will therefore require temporary protection systems with unwatering to allow for placement of the bedding material. Groundwater flow into the excavation can be expected due to the relatively permeable nature of the adjacent granular embankment fill. 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.

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. Based on the GA drawing provided by D.M. Wills, we understand that the creek water will be diverted via a sheet-pile cut off wall in combination with a temporary bypass.

Dewatering of all excavations should be carried out in accordance with OPSS.PROV 517 (Dewatering), as modified by Special Provision (SP) 517F01, and Non-Standard Special Provision (NSSP) FOUND003 (Dewatering of Structure Excavation). A copy of NSSP FOUND003 is included in Appendix C. Consideration should also be given to including an NSSP in the Contract Documents to alert the Contractor to the requirements for unwatering and potential impacts to excavation stability and subgrade disturbance; a sample NSSP is included in Appendix C.

Provided that the creek flow is diverted away from the proposed excavation and the unwatering system is installed to a suitable depth to mitigate groundwater inflows, construction site dewatering pumping volumes are not anticipated to exceed 50 m^3/day. As such, it is anticipated that, under recently introduced changes to the Environmental Protection Act by the Ontario Ministry of the Environment and Climate Change, an Environmental Activity Section Registry (EASR) would not be required, however the contractor should be required to evaluate the estimated seepage and groundwater removal quantity, which depends on the construction methods/procedures and decide whether an EASR is required.

6.6.3 Excavation and Replacement below Culvert

Prior to placement of any bedding material or engineered fill as backfill for sub-excavation of unsuitable soils (if required), the existing embankment fill, organics (if encountered) and any disturbed soils should be sub-excavated from below the plan limits of the proposed works.

The culvert subgrade should be inspected 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). Following inspection, the sub-excavated area should be backfilled with granular material meeting the requirements of an OPSS.PROV 1010 Granular ‘A’ or Granular ‘B’ Type II that is placed and compacted in accordance with OPSS.PROV 501 (Compacting). The use of Granular ‘B’ Type II fill is recommended in wet conditions or below water.

6.6.4 Culvert Bedding

The bedding and levelling pad requirements for a pre-cast box culvert should be accordance with OPSS 422 (Pre-cast Reinforced Concrete Box Culverts). Given the potential for surface water flow and some groundwater seepage through the fill 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 purposes. Given the potential presence of groundwater/surface water, we do not recommend that Granular ‘B’ Type I or III, nor any materials from the Group II list in OPSS 422, be used for bedding purposes. As the native soil below the bedding is generally fine grained (silt and/or clayey silt), it is also 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. The bedding should be placed in maximum 200 mm thick loose lifts and where possible compacted to at least 98 per cent of the Standard Proctor Maximum Dry Density (SPMDD) of the materials 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 fine concrete aggregate meeting the grading requirements specified in OPSS.PROV 1002 (Aggregates – Concrete) should be provided with a geometry similar to that provided on OPSD 803.010 (Backfill and Cover for Concrete Culverts) and should be placed in dry conditions.

As an alternative to the bedding layer noted above, consideration could be given to using a 100 mm thick concrete working slab outlined further in Section 6.6.6; the concrete working slab would be covered with a minimum 75 mm thick levelling pad to facilitate placement of the box culvert segments.

Based on discussions with D.M. Wills, we understand that the cost of Granular B Type II fill material in the North Bay District is currently quite high due limited availability and MTO has requested that the use of Granular B Type II material be limited where possible. As such, consideration could also be given to the use of a Granular A bedding material but given the anticipated wet conditions, consideration must be given to the use of a fully enclosed cofferdam system (installed to a sufficient depth) to allow construction to proceed in dry conditions.

6.6.5 Backfill

Backfill above/behind the culvert walls should consist of granular fill meeting the specifications for OPSS.PROV 1010 (Aggregates) Granular ‘A’ or Granular ‘B’ Type II. The granular 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 as per OPSS 422 (Precast Reinforced Concrete Box Culverts).

As the existing granular embankment fill material, which is considered to have a low susceptibility to frost heaving (based on MTO Pavement Design and Rehabilitation Manual, 2013), extends below the estimated 2.0 m depth of frost penetration, a frost taper as per OPSD 803.010 is not required at this site.

Backfill placement for reconstruction of the roadway embankments along and over the 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.

6.6.6 Subgrade Protection

The native silt and clayey silt subgrade will be susceptible to disturbance from construction traffic and/or ponded water. To limit the effect of this disturbance and in the event that the granular backfill/bedding is not placed within a timely manner, once the foundation subgrade has been inspected and approved, a concrete working slab could be placed on the subgrade followed subsequently by the remaining Granular A or Granular B Type II sub-excavation backfill and bedding. 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. An NSSP should be included in the Contract to address subgrade protection and concrete working slab at this site; a sample NSSP has been included in Appendix C.

6.6.7 Erosion Protection

Provision should be made for erosion protection of the re-constructed embankment side slopes at the culvert location, particularly given the proposed 1.4H:1V side slopes at this site. Further, 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 and/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.PROV 1205 (Clay Seal), and the seal should be a minimum of 1 m thick, whether constructed of natural clay or soil bentonite mix, or alternatively a geosynthetic clay liner (GCL). The clay seal/GCL should extend from a depth of 1 m below the scour level to a minimum vertical height on the embankment side slopes equivalent to the high-water level. The seal/GCL should also extend a minimum horizontal distance of 2 m on either side of the culvert inlet opening. If a geosynthetic clay liner (GCL) is utilized in lieu of the clay seal on the embankment side slopes, the GCL should be constructed within the embankment slope to allow for a minimum 0.3 m thick granular (embankment) fill cover to be placed over the GCL to provide for protection from the requisite overlying erosion protection material. If required, any rip-rap/rock fill slope protection material should be placed on the granular cover layer and not directly on the GCL.

At this site, the use a geosythetic clay liner (GCL), rather than a clay seal, is considered the preferred alternative for the following reasons: it’s much thinner (only a few millimeters thick) than the standard natural clay (or soil-bentonite) seal layer; requires a shallower excavation to the slope subgrade; and is much easier to install. It is anticipated that the contractor/installer should be able to install the GCL within the 72-hr full-road closure allowable period. If the road closure or permitted work period is short/tight, the contractor/installer can likely complete the installation afterwards with a single-lane closure; however, the GCL should not be left exposed overnight/weekend and must be covered immediately after installation. Given the steep proposed side slopes (i.e. 1.4H:1V) at this site, we suggest that a narrow/shallow trench be made near the top of the embankment to anchor the GCL in place.

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. Given the steep proposed side slopes, it is recommended that the rip rap be placed across the entire exposed granular surface of the re-constructed embankments at both the inlet and outlet ends. 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 or GCL.

6.6.8 Analytical Testing for Construction Materials

The results of an analytical test on a soil sample taken at the Culvert 2 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.

For potential sulphate attack on concrete, the results of the soil analysis were compared to Table 3 in CSA A23-1-09, which indicate that the relative degree of sulphate attack is low (less than the moderate range). However, given that the culvert location is on Highway 539A 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 soil 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 soil 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 Detail 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., a Designated MTO Foundations Contact and Senior Consultant for Golder, conducted an independent quality control review and technical audit of this report.

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Canadian Standards Association (CSA), 2014. *Canadian Highway Bridge Design Code* and *Commentary on CAN/CSA S6-14*.

Canadian Standards Association (CSA), 2014. CSA A23.1-09 *Concrete Materials and Methods of Construction* (R2014).

Ministry of Natural Resources, *Northern Ontario Engineering Geology Terrain Study*. Ontario Geological Society Electronic Mapping. Map 41INE.

Ministry of Northern Development of Mines. *Bedrock Geology of Ontario – East Central Sheet*, Ontario Geological Survey – Map 2543.

Ministry of Transportation, *MTO Gravity Pipe Design Guidelines*, MTO Drainage and Hydrology Design and Contract Standards Office, May 2014

Ministry of Transportation, “MTO Pavement Design and Rehabilitation Manual”, MTO Materials Engineering and Research Office, Second Edition 2013.

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

Transportation Research Board, National Research Council, 1998. *Service Life 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 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.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
OPSS 1860	Material Specification for Geotextiles

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 3090.010 Foundation, Frost Penetration Depths for Northern Ontario

Ontario Water Resource Act

Regulation 903Wells (as amended)

Table 1: Summary Details of Existing Culvert

Culvert Location	Site #	Approximate Height of Embankment [1]	Existing Culvert			Approximate Invert Elevation[2]	
			Type	Approximate Dimension[2]	Approximate Length	East End (Inlet)	West End (Outlet)
Hwy 539A, Sta. 10+097 Twp of Crerar	N/A	3.7 m	Structural Plate Corrugated Steel Pipe Arch	2.8 m span	13 m	230.3	230.3

1. Embankment height is relative to existing ground surface at the centreline of the roadway and the existing culvert invert.
2. Culvert dimensions and invert elevations are based on the plan and profile drawings provided by D.M. Wills/MTO (Drawing C-270-539A-1).

Table 2: Comparison of Alternative Culvert Types

Option	Advantages	Disadvantages	Risks/Consequences
Pre-Cast Box Culvert	■ 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. ■ More tolerant of total and differential settlement. ■ Backfill/bedding under the culvert may be placed underwater (i.e., Granular 'B' Type II) minimizing or eliminating water pumping requirements. ■ Allows for greater flow volume than circular/arch CSP.	■ May not satisfy fisheries requirements related to natural channel substrate, if applicable. ■ Cut-off wall (or clay seal) likely required at inlet to mitigate potential scour under the culvert. ■ Transportation to and on-site lifting of large pre-cast sections will be required.	■ Moderate risk of disturbance of the native silt and/or clayey silt deposits during construction; can be mitigated with use of a tremie concrete working slab or Granular 'B' Type II working pad/bedding or working slab. ■ Low risk related to settlement performance as box segments can accommodate some total and differential settlement.
Open Footing 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. ■ Readily suitable for construction using concrete or metal sections. ■ Would likely satisfy fisheries requirements related to natural channel substrate, if applicable. ■ Allows for greater flow volume than circular/arch CSP.	■ Excavation depths are greater than for an open footing culvert option, resulting in increased excavation support and dewatering 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. ■ Less tolerant of total and differential settlement if the highway embankment is raised or widened at the culvert site.	■ Moderate risk of disturbance of the native silt and/or clayey silt deposits during construction; can be mitigated with use of a tremie concrete working slab or Granular 'B' Type II working pad. ■ May require greater depth of dewatering for footing construction. ■ Culvert joints may be required to accommodate the anticipated total and differential settlement.

Option	Advantages	Disadvantages	Risks/Consequences
Pipe Culvert(s)	■ Allows for faster construction resulting in shorter duration for dewatering and surface pumping compared to an open footing culvert. ■ More tolerant of total and differential settlement. ■ Backfill under the culvert may be placed underwater (i.e., Granular ‘B’ Type II) minimizing or eliminating water pumping requirements.	■ Reduced flow-through capacity compared to box culvert and open footing options with a similar span – additional flow through capacity may have to be provided by multiple pipes. ■ Cut-off wall or clay seal may be required at inlet to mitigate potential scour under the culvert(s). ■ Difficult to compact backfill materials to level of culvert springline if not done in the dry. ■ CSP does not have as long of design life compared to concrete options.	■ Moderate risk of disturbance of the native silt and/or clayey silt deposits during construction; can be mitigated with use of a tremie concrete working slab or Granular ‘B’ Type II working pad. ■ Lower risk related to anticipated total and differential settlement compared to box or open footing option.

Table 3: Comparison of Alternative Stability Mitigation Measures

Option	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Reconstruction side slopes at 1.4H:1V using Gran. A, Gran. B Type II or Modified Gran. B Type II	■ Satisfies minimum target factor of safety for global stability within the current limits of the MTO ROW. ■ Readily available material; except Modified Granular B Type II, which may not be as readily available	■ Utilizes a less conservative friction angle for design. ■ Potentially more prone to erosion and potential additional maintenance costs given the steeper side slopes. ■ Potentially requires additional erosion protection although Granular B Type II is considered to have a low erodibility factor.	■ Least expensive option overall. ■ Gran. A or Gran. B Type II is more expensive than Gran. B Type I fill material. ■ Modified Gran. B Type II would be the most expensive material but would also be more conservative for design.	■ Higher risk related to stability performance. ■ Low risk related to erosion and can be further mitigated with the use of rip rap protection at the culvert inlet (and potentially outlet). ■ Low risk related to scheduling delays
Partial Sub-Excavation of the Silt Deposit and Reconstruction of side slopes at 1.4H:1V using Gran. B Type I	■ Satisfies minimum target factor of safety for global stability within the current limits of the MTO ROW. ■ Utilizes a more conservative friction angle for design.	■ Requires property acquisition to extend current MTO ROW or requires temporary protection system along ROW to allow for vertical excavation. ■ Will result in a significant schedule delay required to obtain additional property. ■ Requires additional excavation and dewatering during construction. ■ Potentially requires additional erosion protection and/or maintenance costs if Gran. B Type I is utilized.	■ Additional costs for property acquisitions, additional fill material and potentially a temporary protection system. ■ Potentially cheaper for Gran. B Type I placed above the groundwater level. ■ Potentially more costly due to delayed schedule.	■ Lower risk related to stability performance. ■ Moderate risk related to erosion if Granular B Type I is utilized but can mitigated with the use of rip rap protection at the culvert inlet and outlet. ■ High risk related to scheduling delays.

Option	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Reconstruction of side slopes at 2H:1V using Gran. A or Gran. B (Type I or II)	■ Utilizes a more conservative friction angle for design. ■ Less prone to erosion and potentially reduced maintenance costs. ■ Standard practice. ■ Can use more readily available Granular B Type I material.	■ Requires property acquisition to extend current MTO ROW or requires temporary protection system along ROW to allow for vertical excavation. ■ Will result in a significant schedule delay required to obtain additional property.	■ Additional costs for property acquisitions and additional fill material and potentially a temporary protection system. ■ Granular B Type I is less expensive than Gran A or Granular B Type II fill material. ■ Potentially more costly due to delayed schedule.	■ Low risk related to stability performance. ■ Lower risk related to erosion. ■ High risk related to scheduling delays.
Retaining Walls with 2H:1V Upper Side Slopes.	■ Satisfies minimum target factor of safety for global stability within the current limits of the MTO ROW. ■ Variety of systems including gabion walls, which are more tolerant to differential settlement. ■ Standard practice.	■ Requires additional costs associate with construction and materials. ■ Additional excavation and dewatering requirements for wall installation. ■ Additional construction time required, which may extend the full road-closure period.	■ Higher cost option. ■ More risks associated with subgrade disturbance and potential construction issues. ■ Likely requires additional maintenance and replacement costs.	■ Low risk related to stability performance. ■ Moderate risk related to erosion and/or settlement issues depending on the selected retaining wall system. ■ Higher risk of scheduling delays.

MINISTRY OF TRANSPORTATION, ONTARIO

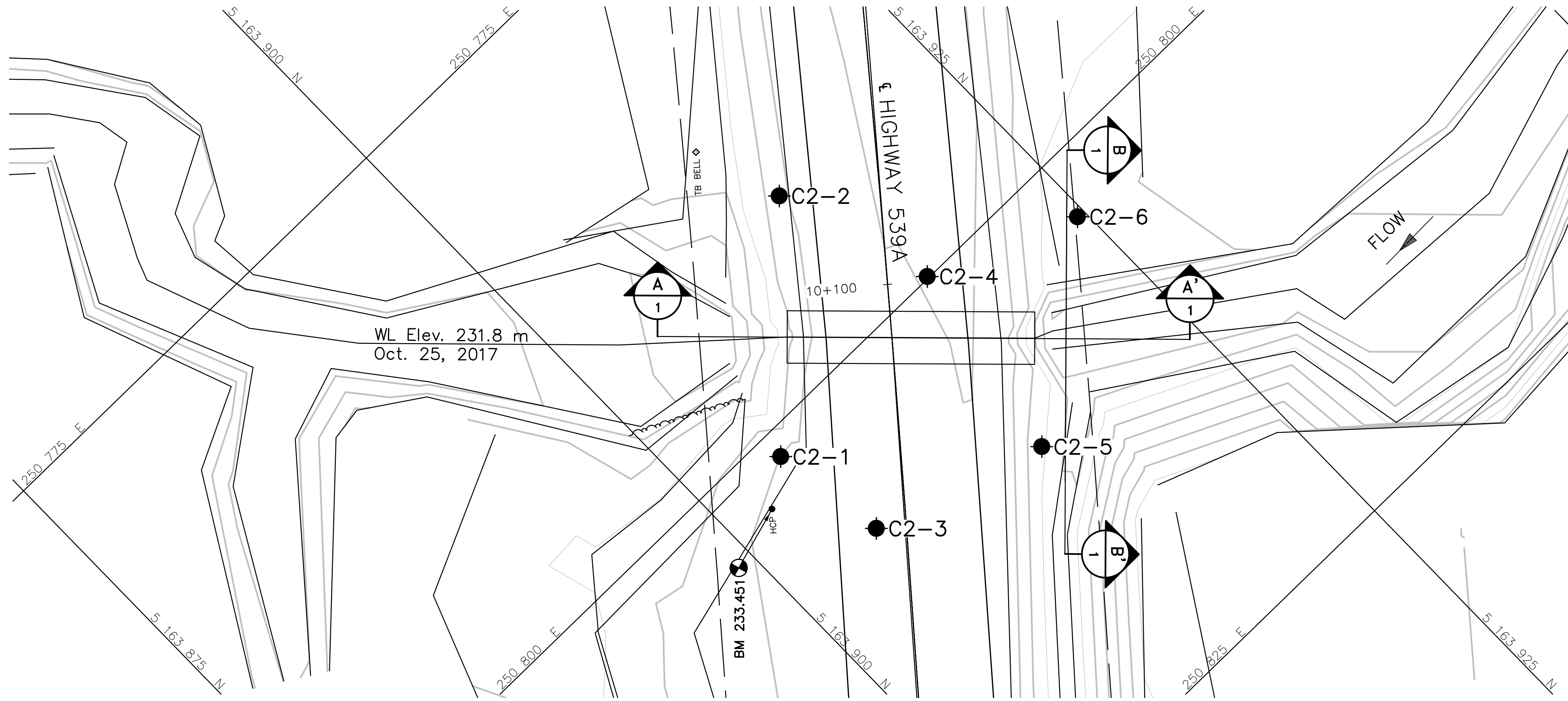
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
WP No. 5162-16-01

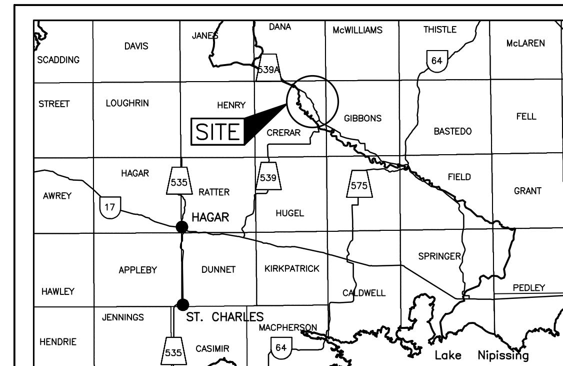
SHEET

HIGHWAY 539A
CULVERT 2 AT STA 10+097
LAT. 46.612665; LONG. -80.204985
BOREHOLE LOCATIONS AND SOIL STRATA

GOLDER



PLAN
SCALE
3 0 3 6 m



KEY PLAN
SCALE
10 0 10 20 km

LEGEND

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

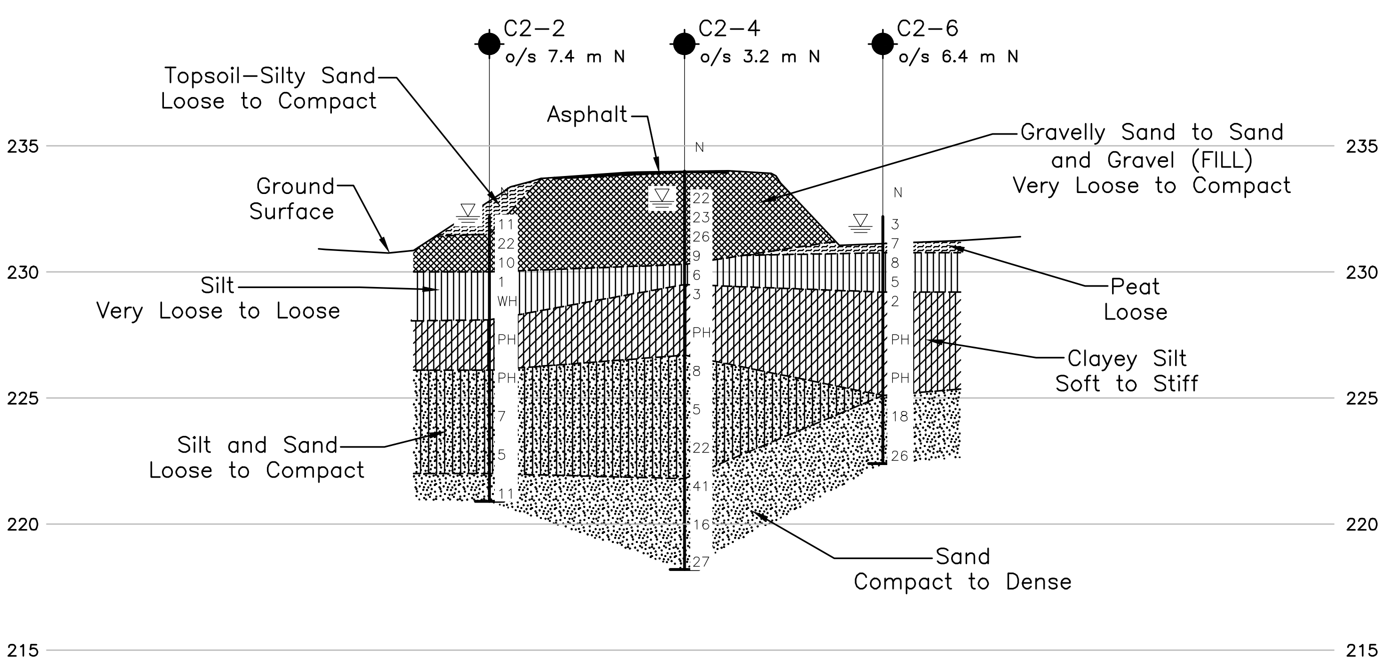
BOREHOLE CO-ORDINATES (NAD 83 MTM ZONE 10)

No.	ELEVATION	NORTHING	EASTING
C2-1	233.2	5163904.7	250801.3
C2-2	232.2	5163914.1	250791.5
C2-3	233.9	5163905.6	250807.5
C2-4	234.0	5163916.7	250799.9
C2-5	231.9	5163914.8	250810.5
C2-6	232.2	5163924.6	250803.1

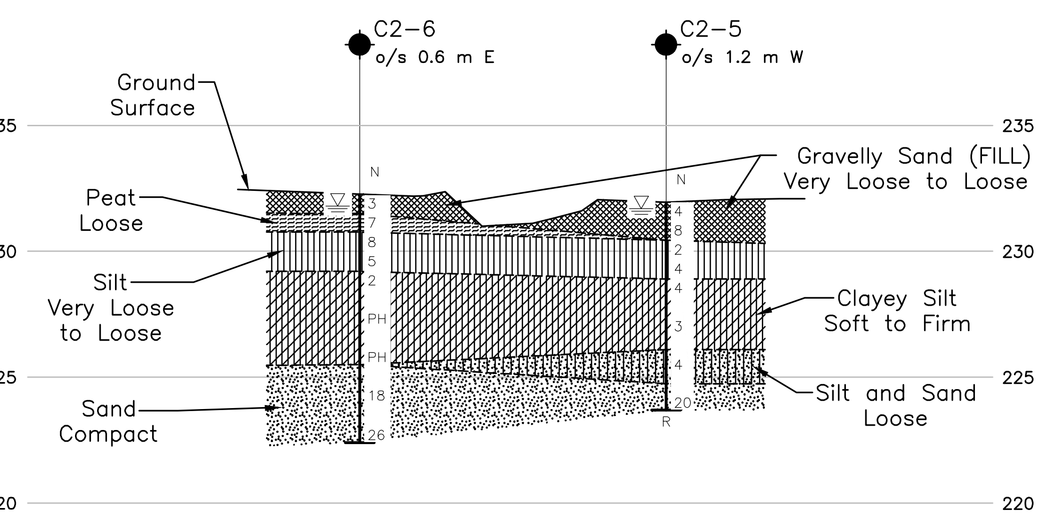
NOTES
This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

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 DM WILLS, drawing file no. WP-5130-16-00.dwg, received OCT 30, 2017.



A-A' 1 PROFILE
HORIZONTAL SCALE
3 0 3 6 m
3 0 3 6 m
VERTICAL SCALE



B-B' 1 CROSS-SECTION
HORIZONTAL SCALE
3 0 3 6 m
3 0 3 6 m
VERTICAL SCALE

LICENSED PROFESSIONAL ENGINEER D.A. MULDOWNEY 100164716 PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER J.M.A. COSTA PROVINCE OF ONTARIO

NO.	DATE	BY	REVISION

Geocres No. 41I-354

HWY. 539A		PROJECT NO. 1777318	DIST.
SUBM'D.	CHKD. AC	DATE: 3/15/2018	SITE: NA
DRAWN: TB	CHKD. DAM	APPD. JMAC	DWG. 1

PLOT DATE: March 15, 2018
FILENAME: S:\Clients\MTO\Hwy539&539A\99_PROJ\1777318\40_PROD\Culvert-2\1777318_Culvert-2.dwg

Photograph 1: Culvert 2 – Hwy 539A – Sta 10+097 Crerar Twp., Facing South (taken October 16, 2017)

Photograph 2: Culvert 2 – Hwy 539A – Sta 10+097 Crerar Twp., Facing North (taken October 16, 2017)

Photograph 3: Culvert 2 – Hwy 539A – Sta 10+097 Crerar Twp., Inlet End Facing South (taken October 16, 2017)

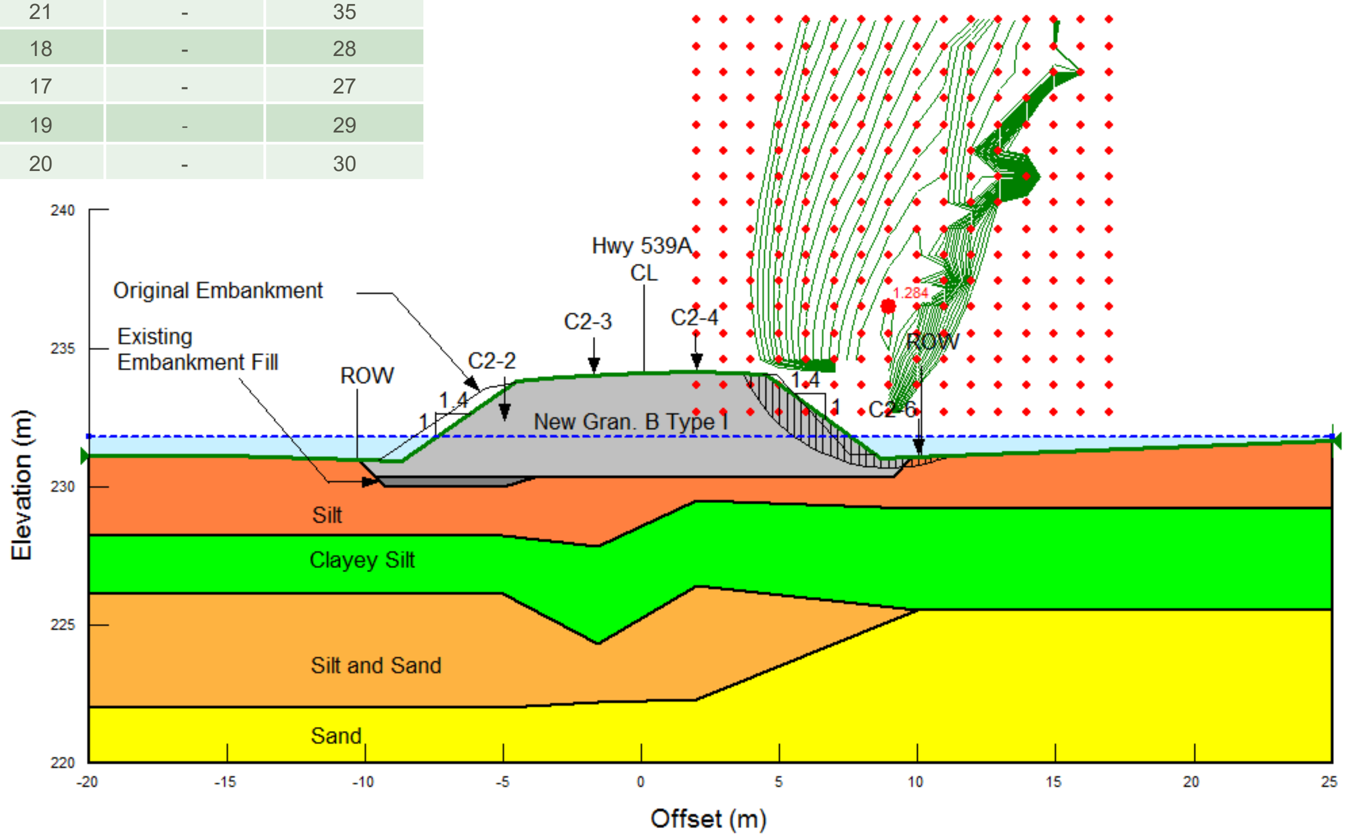
Photograph 4: Culvert 2 – Hwy 539A – Sta 10+097 Crerar Twp., Outlet End Facing Northwest (taken October 16, 2017)

Global Stability Analysis

Figure 1

Proposed East Side Slope- Gran. B Type I Fill
Long-Term (Drained) Analysis

Material Name	Unit Weight (kN/m³)	Undrained Shear Strength (kPa)	Friction Angle (degrees)
Existing Granular Fill	20	-	32
New Gran. B Type I Fill	21	-	35
Silt	18	-	28
Clayey Silt	17	-	27
Silt and Sand	19	-	29
Sand	20	-	30

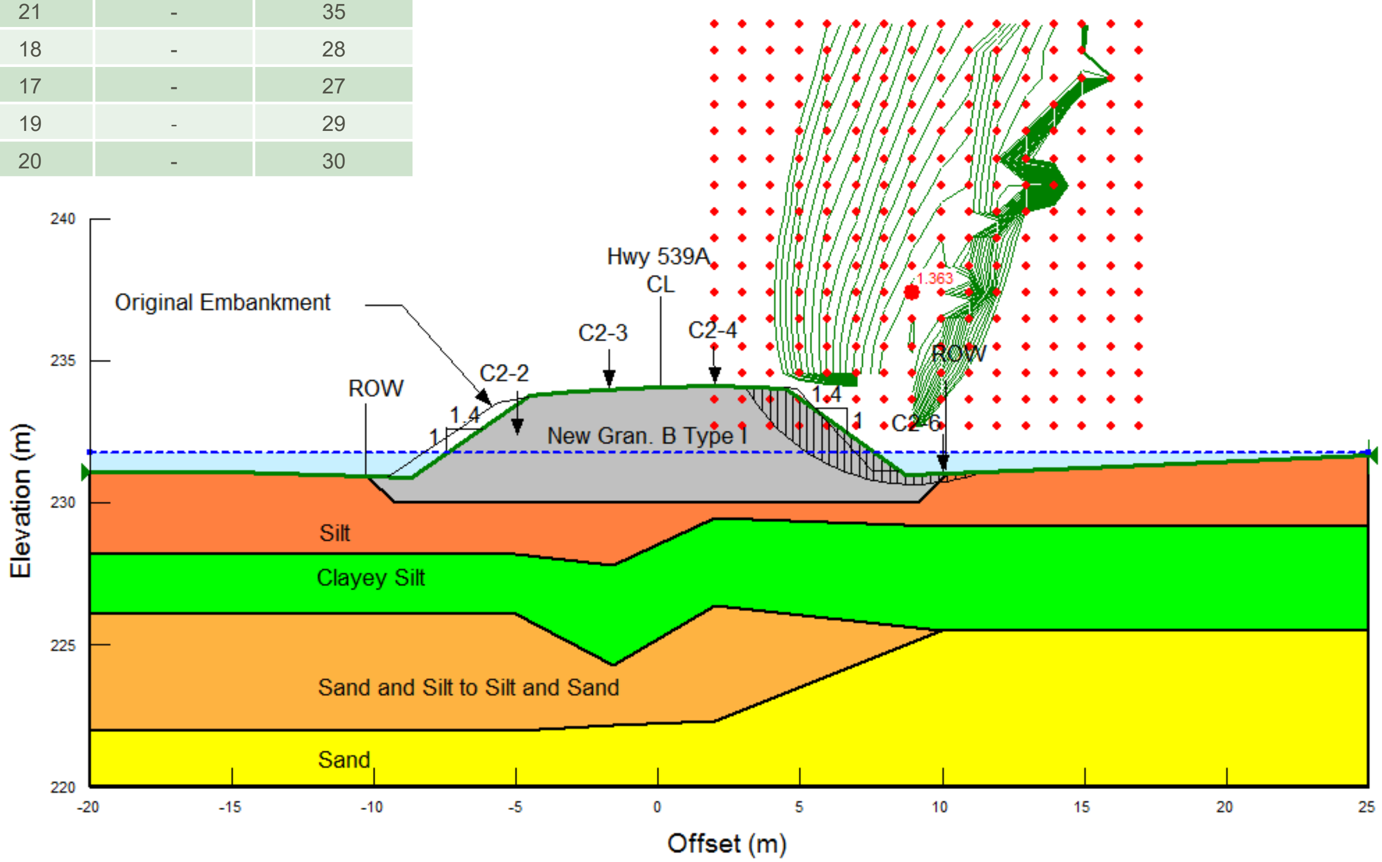


Global Stability Analysis

Figure 2

Proposed East Side Slope
Gran. B Type I Fill and Partial Sub-Excavation to Elev. 230m
Long-Term (Drained) Analysis

Material Name	Unit Weight (kN/m³)	Undrained Shear Strength (kPa)	Friction Angle (degrees)
New Gran. B Type I Fill	21	-	35
Silt	18	-	28
Clayey Silt	17	-	27
Silt and Sand	19	-	29
Sand	20	-	30

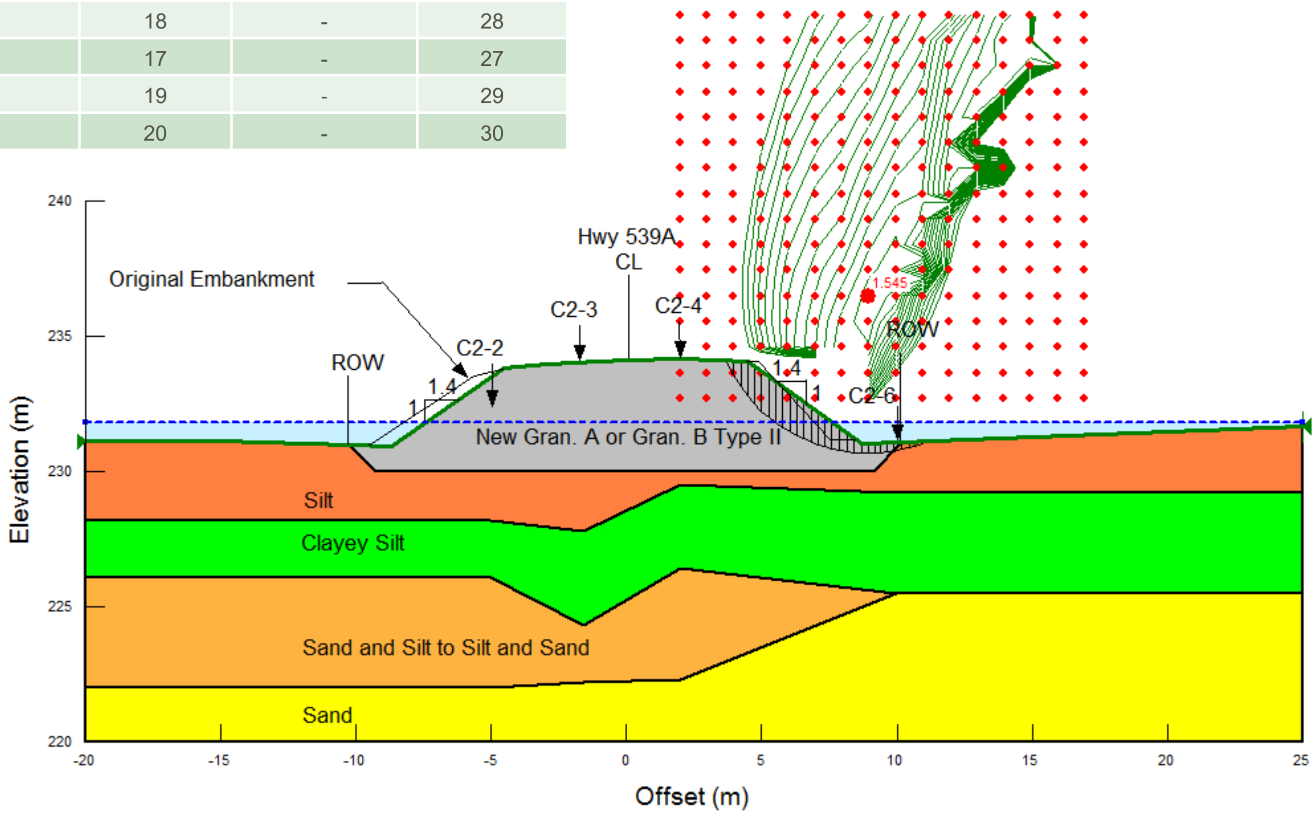


Global Stability Analysis

Figure 3

Proposed East Side Slope
Gran. A or Gran. B Type II Fill
Long-Term (Drained) Analysis

Material Name	Unit Weight (kN/m³)	Undrained Shear Strength (kPa)	Friction Angle (degrees)
New Gran. A or Gran. B Type II Fill	21	-	40
Silt	18	-	28
Clayey Silt	17	-	27
Silt and Sand	19	-	29
Sand	20	-	30



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

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

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(d) Shear Strength

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

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

LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

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

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

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

Dynamic Cone Penetration Resistance; N_d:

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

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

Piezo-Cone Penetration Test (CPT)

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

III. SOIL DESCRIPTION

(a) Non-Cohesive (Cohesionless) Soils

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

(b) Cohesive Soils

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

IV. SOIL TESTS

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

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

V. MINOR SOIL CONSTITUENTS

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

GOLDER

Foundation Design

RECORD OF BOREHOLE No C2-1

1 OF 2 **METRIC**

PROJECT 1777318

W.P. 5162-16-01

LOCATION N 5163904.7; E 250801.3 NAD83 MTM ZONE 10 (LAT. 46.612589; LONG. -80.204979)

ORIGINATED BY MR

DIST ____ HWY 539A

BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers

COMPILED BY AC

DATUM GEODETIC

DATE October 25, 2017

CHECKED BY DAM

ELEV DEPTH (m)	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH (FIELD VANE, kPa / SENSITIVITY)	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
233.2	GROUND SURFACE								
0.0	TOPSOIL - Silty Sand, Loose, Dark brown, Moist	1	SS	4		233			
232.6 / 0.6	Sand, some gravel, trace silt (FILL), Loose, Brown to grey, Moist	2	SS	9	∇	232			
		3	SS	9		231			
		4	SS	4					
		5	SS	6		230		○	12 85 (3)
		6	SS	6		229			
228.7 / 4.5	SILT, trace sand, Very loose, Grey, Wet	7	SS	4		228			
227.8 / 5.4	CLAYEY SILT, silt laminations, Firm to stiff, Grey to brown, Wet						+ (2)		
		8	SS	2		227		H○H	
						226	+ (2), + (2)		
		9	TO	PH		225			
							+ (2)		
224.1 / 9.1	SILT and SAND, trace gravel, trace clay, Compact, Grey to brown, Wet	10	TO	PH		224			
						223			
		11	SS	12		222		○	1 42 53 4
221.9 / 11.3									

Continued Next Page

+3, ×3: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

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

GOLDER

RECORD OF BOREHOLE No C2-1

 METRIC

PROJECT 1777318
W.P. 5162-16-01
LOCATION N 5163904.7; E 250801.3 NAD83 MTM ZONE 10 (LAT. 46.612589; LONG. -80.204979)
ORIGINATED BY MR
DIST HWY 539A
BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers
COMPILED BY AC
DATUM GEODETIC
DATE October 25, 2017
CHECKED BY DAM

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	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	--- CONTINUED FROM PREVIOUS PAGE ---										
	END OF BOREHOLE Note: 1. Water level at a depth of 1.4 m below ground surface (Elev. 231.8 m) upon completion of drilling. 2. Additional borehole advanced 1.0 m south to obtain additional Shelby tube sample from 6.1 m to 6.7 m depth (Elev. 271.1 m to 226.5 m) and additional field vanes at 5.8 m and 7.3 m depths (Elev. 227.4 m and 225.9 m).										

SUD-MTO 001 MTM ZN INC LAT/LONG S:\CLIENTS\MTO\HWY539&539A\02_DATA\GINT\1777318.GPJ GAL-MISS.GDT 3/15/18 TB

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

GOLDER

RECORD OF BOREHOLE No C2-2

 METRIC

PROJECT 1777318
W.P. 5162-16-01
LOCATION N 5163914.1; E 250791.5 NAD83 MTM ZONE 10 (LAT. 46.612673; LONG. -80.205108)
ORIGINATED BY MR
DIST HWY 539A
BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers
COMPILED BY AC
DATUM GEODETIC
DATE November 9, 2017
CHECKED BY DAM

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	SHEAR STRENGTH kPa (FIELD VANE, sensitivity)	WATER CONTENT	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
232.2	GROUND SURFACE										
0.0	TOPSOIL - Silty Sand, trace gravel; Compact; Dark brown; Wet	1	SS	11							
231.5 / 0.7	Gravelly sand, trace silt (FILL); Compact; Brown to grey; Wet	2	SS	22							
		3	SS	10							
230.0 / 2.2	SILT, trace gravel, trace to some sand, trace clay; Very loose; Grey; Wet	4	SS	1							
		5	SS	WH		○	NP	3	9	86	2
228.1 / 4.1	CLAYEY SILT, silt laminations; Soft to firm; Grey to brown; Wet				+ 2						
		6	TO	PH							
					+ 3						
226.1 / 6.1	SILT and SAND, trace clay; Loose; Grey; Wet	7	TO	PH							
	Introduced full head of water at 7.6 m depth to mitigate heave inside augers.	8	SS	7		○		0	55	44	1
		9	SS	5							
222.0 / 10.2	SAND, trace gravel; Compact; Grey; Wet	10	SS	11							
220.9 / 11.3											

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE (232–221); 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

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

SUD-MTO 001 MTM ZN INC LAT/LONG S:\CLIENTS\MTO\HWY539&539A\02_DATA\GINT\1777318.GPJ GAL-MISS.GDT 3/15/18 TB

GOLDER

PROJECT 1777318	RECORD OF BOREHOLE No C2-2	2 OF 2	METRIC
W.P. 5162-16-01	LOCATION N 5163914.1; E 250791.5 NAD83 MTM ZONE 10 (LAT. 46.612673; LONG. -80.205108)		ORIGINATED BY MR
DIST HWY 539A	BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers		COMPILED BY AC
DATUM GEODETIC	DATE November 9, 2017		CHECKED BY DAM

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						
	END OF BOREHOLE Note: 1. Water level at ground surface (Elev. 232.2 m) upon completion of drilling.										

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

SUD-MTO 001 MTM ZN INC LAT/LONG S:\CLIENTS\MTO\HWY539&539A\02_DATA\GINT\1777318.GPJ GAL-MISS.GDT 3/15/18 TB

GOLDER

RECORD OF BOREHOLE No C2-3

 METRIC

PROJECT 1777318

W.P. 5162-16-01

LOCATION N 5163905.6; E 250807.5 NAD83 MTM ZONE 10 (LAT. 46.612597; LONG. -80.204898)

ORIGINATED BY MR

DIST ____ HWY 539A

BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers

COMPILED BY AC

DATUM GEODETIC

DATE October 16, 2017

CHECKED BY DAM

ELEV DEPTH (m)	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH (kPa)	WATER CONTENT	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
233.9	GROUND SURFACE									
0.0	ASPHALT (50 mm)									
	Sand and gravel (FILL) Brown Moist									
0.3	ASPHALT (50 mm)									
	Gravelly sand, some silt (FILL) Compact Brown Moist to wet	1	SS	15	∇	233				
		2	SS	12		232		○		21 61 (18)
		3	SS	11						
230.9 / 3.0	SILT, trace to some sand, trace to some clay Very loose Grey Wet	4	SS	3		231				
		5	SS	2		230				
		6	SS	4		229		○	NP	0 7 86 7
	Approximately 0.6 m heave inside auger at 6.1 m depth.									
227.8 / 6.1	CLAYEY SILT, silt laminations Firm Grey Wet	7	SS	2		228				
						227	+ (sensitivity 2)			
		8	SS	3		226		⊢⊣ ○		0 1 73 26
						225	+ (sensitivity 2)			
		9A	SS	4						
224.3 / 9.6	SILT and SAND, trace clay Compact Grey to brown Wet	9B						○		0 50 46 4
						224				
		10	SS	19		223				
222.2 / 11.7						222				

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

Continued Next Page

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

GOLDER

RECORD OF BOREHOLE No C2-3

 METRIC

PROJECT 1777318

W.P. 5162-16-01 | LOCATION N 5163905.6; E 250807.5 NAD83 MTM ZONE 10 (LAT. 46.612597; LONG. -80.204898) | ORIGINATED BY MR

DIST | HWY 539A | BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers | COMPILED BY AC

DATUM GEODETIC | DATE October 16, 2017 | CHECKED BY DAM

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED) (20 40 60 80 100)	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%) (20 40 60)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---										
	SAND, trace gravel, trace to some silt Loose to compact Brown Wet		11	SS	8						
							221				
			12	SS	10		220				
							219				
			13	SS	4				○		0 88 (12)
218.1 15.8	END OF BOREHOLE Note: 1. Water level at a depth of 0.9 m below ground surface (Elev. 233.0 m) upon completion of drilling.										

SUD-MTO 001 MTM ZN INC LAT/LONG S:\CLIENTS\MTO\HWY539&539A\02_DATA\GINT\1777318.GPJ GAL-MISS.GDT 3/15/18 TB

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

GOLDER

RECORD OF BOREHOLE No C2-4

 METRIC

PROJECT 1777318
W.P. 5162-16-01
LOCATION N 5163916.7; E 250799.9 NAD83 MTM ZONE 10 (LAT. 46.612697; LONG. -80.204998)
ORIGINATED BY MR
DIST ___ HWY 539A
BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers, NW Casing and Wash Boring
COMPILED BY AC
DATUM GEODETIC
DATE October 17, 2017
CHECKED BY DAM

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	W.C. / Shear strength	GR SA SI CL
234.0	GROUND SURFACE						
0.0	ASPHALT (50 mm)						
0.3	Sand and gravel (FILL) Brown Moist						
	ASPHALT (50 mm)						
	Gravelly sand, trace silt (FILL) Loose to compact Brown Moist	1	SS	22	▽		
		2	SS	23			
		3	SS	26			
		4	SS	9		w ≈ 15%	23 74 (3)
230.3 / 3.7	SILT, trace sand Loose Grey Wet	5	SS	6			
229.5 / 4.5	NW casing and wash boring below 4.6 m depth. CLAYEY SILT, silt laminations Stiff Grey Wet	6	SS	3		W_P–W_L ≈ 21–27, w ≈ 30%	0 1 62 37
						Field vane +, sensitivity 2	
		7	SS	PH			
						Field vane +, sensitivity 3	
226.4 / 7.6	SILT and SAND, some gravel, trace clay Loose to compact Grey Wet	8	SS	8			
		9	SS	5			
		10	SS	22		w ≈ 20%	15 35 46 4
222.3 / 11.7							

Elevation scale: 233, 232, 231, 230, 229, 228, 227, 226, 225, 224, 223

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 (%)

Continued Next Page

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

SUD-MTO 001 MTM ZN INC LAT/LONG S:\CLIENTS\MTO\HWY539&539A\02_DATA\GINT\1777318.GPJ GAL-MISS.GDT 3/15/18 TB

GOLDER

RECORD OF BOREHOLE No C2-4

 METRIC

PROJECT 1777318			
W.P. 5162-16-01	LOCATION N 5163916.7; E 250799.9 NAD83 MTM ZONE 10 (LAT. 46.612697; LONG. -80.204998)		ORIGINATED BY MR
DIST	HWY 539A	BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers, NW Casing and Wash Boring	COMPILED BY AC
DATUM GEODETIC	DATE October 17, 2017		CHECKED BY DAM

ELEV DEPTH	SOIL PROFILE – DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED (20 40 60 80 100)	PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) (20 40 60)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---										
	SAND, trace gravel, trace silt Compact to dense Grey Wet		11	SS	41						
							221				
			12	SS	16		220				
							219				
			13	SS	27						
218.2 15.8	END OF BOREHOLE Note: 1. Water level at a depth of 1.2 m below ground surface (Elev. 232.8 m) upon completion of drilling.										

SUD-MTO 001 MTM ZN INC LAT/LONG S:\CLIENTS\MTO\HWY539&539A\02_DATA\GINT\1777318.GPJ GAL-MISS.GDT 3/15/18 TB

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

GOLDER

PROJECT 1777318

RECORD OF BOREHOLE No C2-5

METRIC

W.P. 5162-16-01

LOCATION N 5163914.8; E 250810.5 NAD83 MTM ZONE 10 (LAT. 46.612681; LONG. -80.204859)

ORIGINATED BY MR

DIST HWY 539A

BOREHOLE TYPE NW Casing and Wash Boring

COMPILED BY AC

DATUM GEODETIC

DATE November 11, 2017

CHECKED BY DAM

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
231.9	GROUND SURFACE									
0.0	Gravelly sand, trace silt (FILL) Loose Brown Wet	1	SS	4	▽					
		2	SS	8		231		○		27 70 (3)
230.4 / 1.5	SILT Very loose Grey Wet. No recovery in Sample 3.	3	SS	2		230				
		4	SS	4						
228.9 / 3.0	CLAYEY SILT, silt laminations Firm Grey Wet	5	SS	4		229				
						228	+ 2			
		6	SS	3		227		├─┤○		0 0 74 26
							+ 2			
226.1 / 5.8	SILT and SAND, trace gravel, trace clay Loose Grey Wet	7	SS	4		226				
224.7 / 7.2	SAND, trace gravel Compact Grey Wet					225				
		8	SS	20		224				
223.7 / 8.2	END OF BOREHOLE REFUSAL TO FURTHER CASING ADVANCEMENT									

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

Note:

1. Water level at a depth of 0.2 m below ground surface (Elev. 231.7 m) upon completion of drilling.

2. Split-spoon samples obtained by driving with a 1/2 weight hammer. SPT 'N' values have beeen adjusted to the inferred values that would be obtained using a standard weight hammer.

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

GOLDER

RECORD OF BOREHOLE No C2-6

 METRIC

PROJECT 1777318

W.P. 5162-16-01

DIST ____ HWY 539A

DATUM GEODETIC

LOCATION N 5163924.6; E 250803.1 NAD83 MTM ZONE 10 (LAT. 46.612768; LONG. -80.204957)

BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers

DATE November 10, 2017

ORIGINATED BY MR

COMPILED BY AC

CHECKED BY DAM

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
232.2	GROUND SURFACE												
0.0	Gravelly sand, trace organics (FILL) Very loose Brown Wet	1	SS	3	∇	232							
231.5 0.7	PEAT (Amorphous), trace sand, trace gravel Loose Dark brown Wet	2	SS	7		231							
230.8 1.4	SILT Loose Grey Wet No recovery in Samles 3 and 4. Silt noted on inside walls of split-spoon sampler.	3	SS	8		230							
		4	SS	5									
229.2 3.0	CLAYEY SILT, trace sand Soft to firm Grey Wet	5	SS	2		229		H○		0	1	74	25
						228	+ 2 (Field vane)						
		6	TO	PH		227							
							+ 1 (Field vane)						
		7	TO	PH		226							
225.5 6.7	SAND, trace to some gravel. trace silt Compact Grey Wet					225							
		8	SS	18		224		○		6	91	(3)	
		9	SS	26		223							
222.4 9.8	END OF BOREHOLE Note: 1. Water level at a depth of 0.4 m below ground surface (Elev. 231.8 m) upon completion of drilling.												

Legend: DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60; REMARKS & GRAIN SIZE DISTRIBUTION (%)

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

SUD-MTO 001 MTM ZN INC LAT/LONG S:\CLIENTS\MTO\HWY539&539A\02_DATA\GINT\1777318.GPJ GAL-MISS.GDT 3/15/18 TB

APPENDIX B

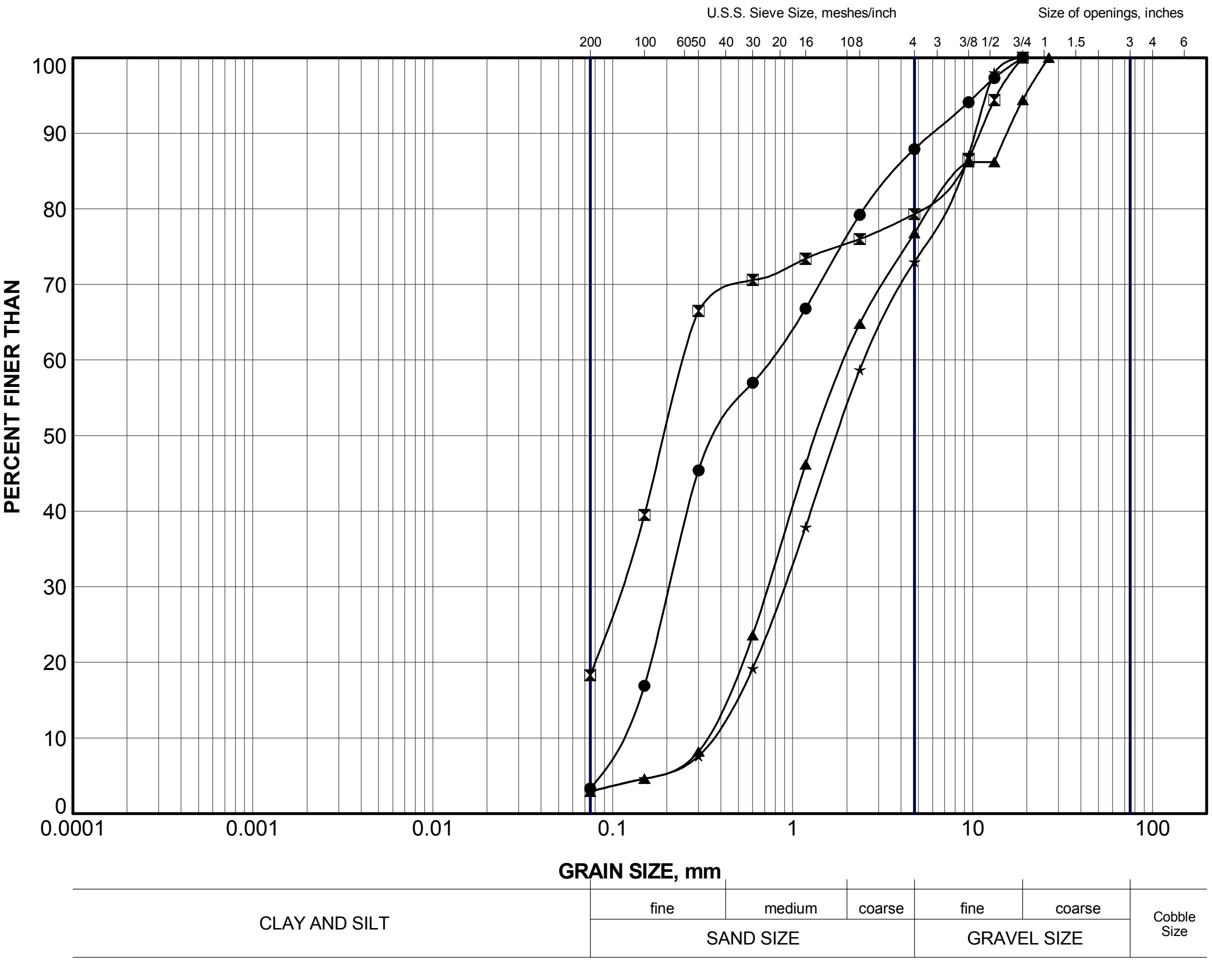
Laboratory Test Results

Table B1 - Summary of Analytical Testing of Culvert 2 Soil Sample

Parameter	Units	Results
Resistivity	ohm-cm	15,000
Conductivity	µmho/cm	69
pH	pH	6.81
Sulphate	µg/g	Not Detected
Chloride	µg/g	Not Detected

Notes:
1. Sample obtained October 17, 2017 (Borehole C2-4, Sample 5)
2. Analytical testing carried out by Maxxam Analytics Inc.

Prepared by: AC
Reviewed by: DAM



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	C2-1	5	229.8
⧗	C2-3	2	232.1
▲	C2-4	4	230.6
★	C2-5	2	230.8

PROJECT

HIGHWAY 539A
CULVERT 2 AT STA 10+097

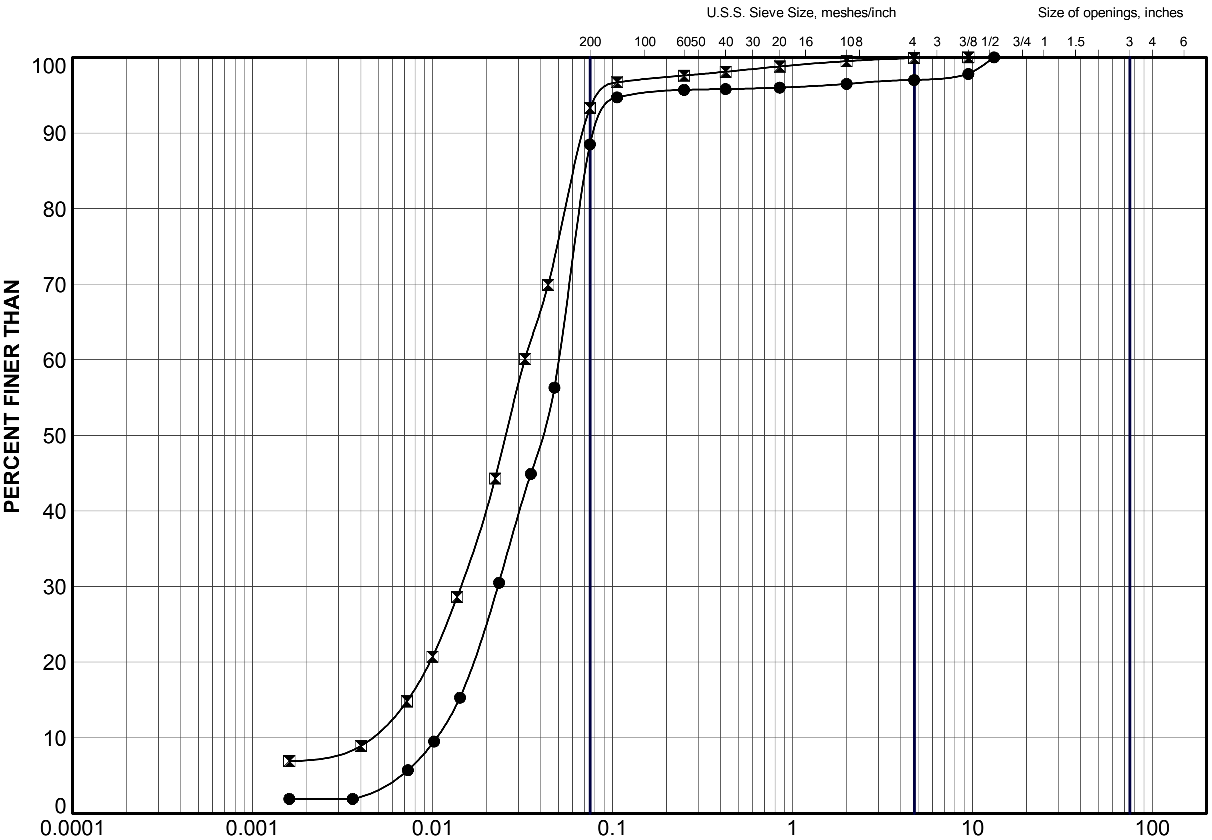
TITLE

GRAIN SIZE DISTRIBUTION
SAND to GRAVELLY SAND (FILL)

Golder Associates
SUDBURY, ONTARIO

PROJECT No.		1777318	FILE No.	1777318.GPJ
DRAWN	TB	Jan 2018	SCALE N/A	REV.
CHECK	DAM	Jan 2018	**FIGURE B1**	
APPR	JMAC	Jan 2018		

SUD-MTO GSD (2016) GLDR_LDN.GDT



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	C2-2	5	228.8
⧗	C2-3	6	229.0

PROJECT

HIGHWAY 539A
CULVERT 2 AT STA 10+097

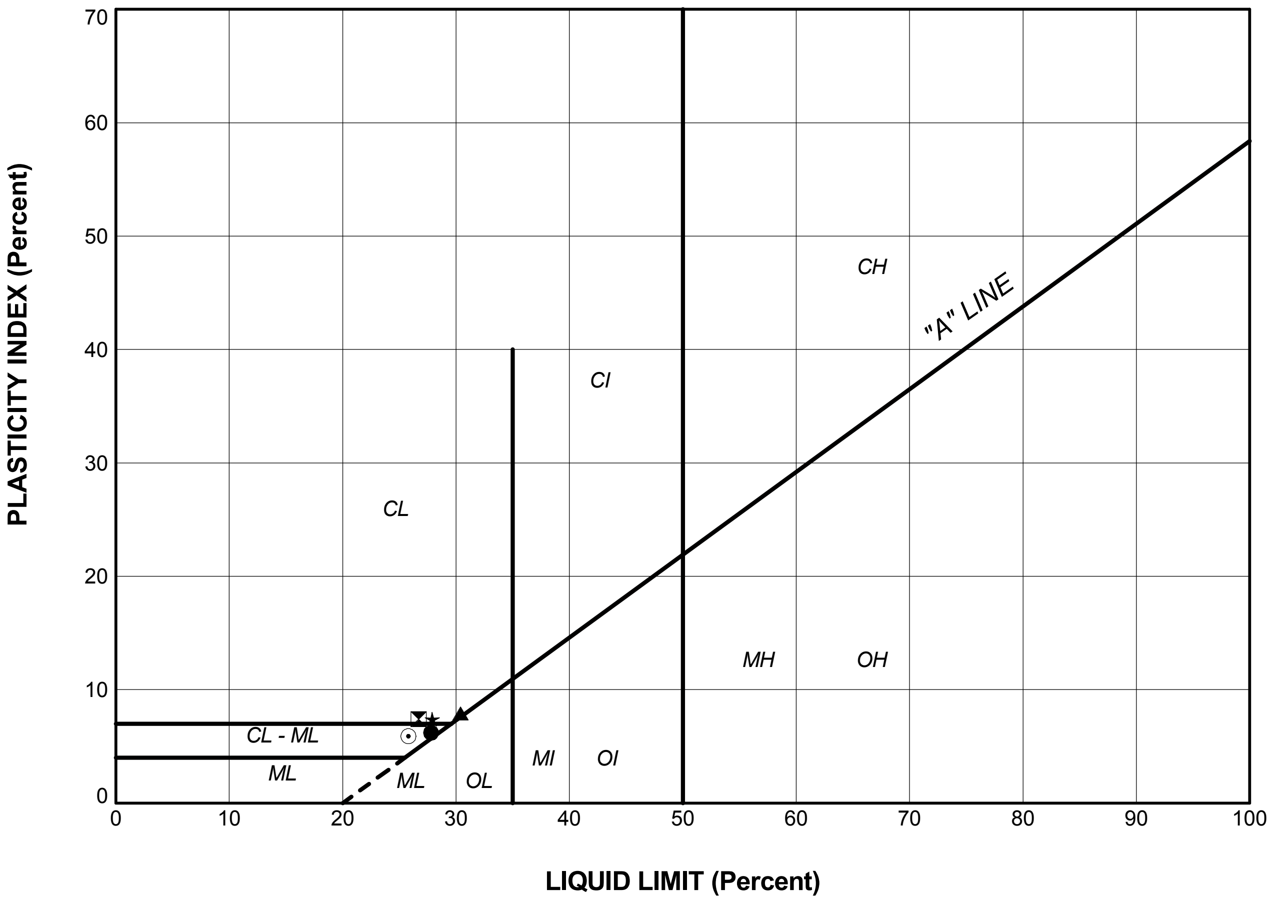
TITLE

GRAIN SIZE DISTRIBUTION

SILT

Golder Associates
SUDBURY, ONTARIO

PROJECT No.		1777318	FILE No. 1777318.GPJ
DRAWN	TB	Jan 2018	SCALE N/A REV.
CHECK	DAM	Jan 2018	
APPR	JMAC	Jan 2018	FIGURE B2



SOIL TYPE
C = Clay
M = Silt
O = Organic

PLASTICITY
L = Low
I = Intermediate
H = High

LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	C2-1	8	27.8	21.6	6.2
⧗	C2-3	8	26.7	19.3	7.4
▲	C2-4	6	30.4	22.5	7.9
★	C2-5	6	27.9	20.5	7.4
⊙	C2-6	5	25.8	19.9	5.9

PROJECT

HIGHWAY 539A
CULVERT 2 AT STA 10+097

TITLE

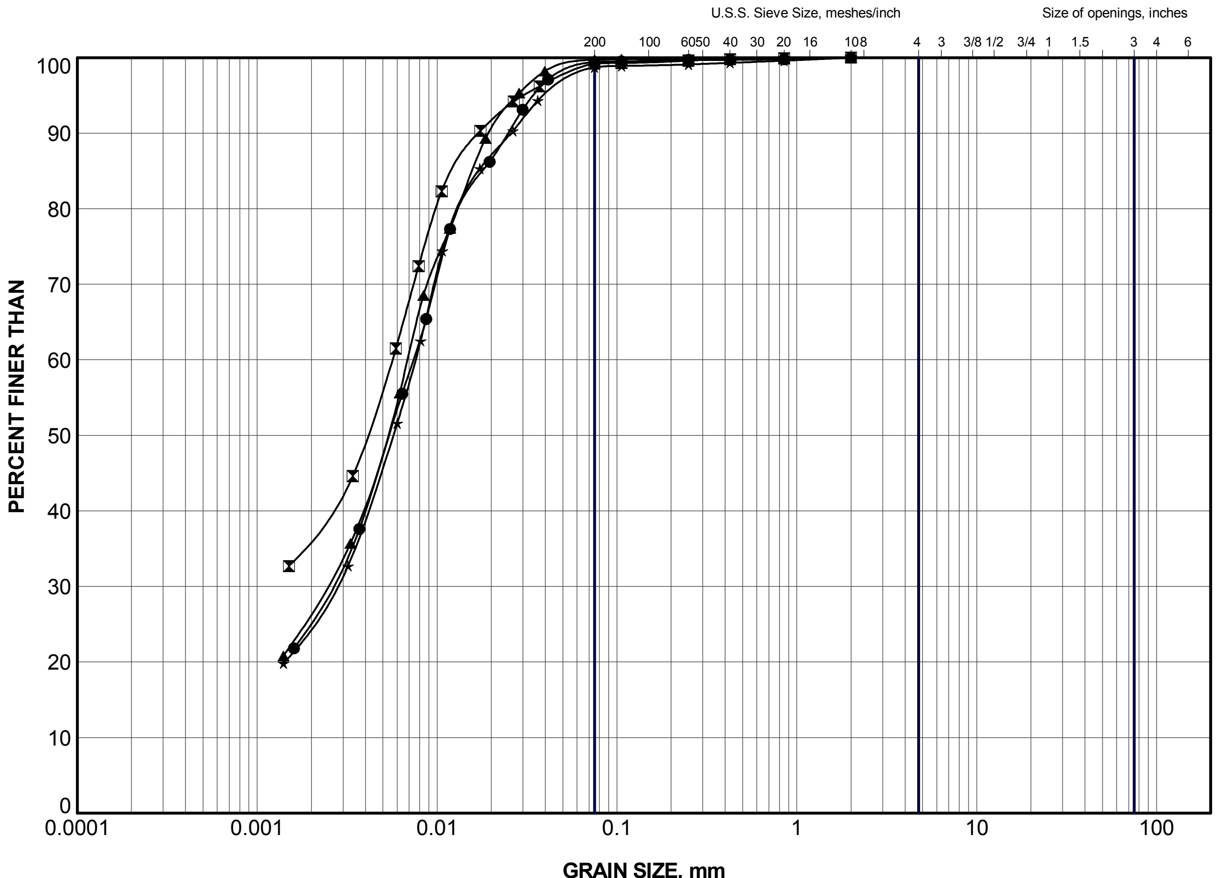
PLASTICITY CHART

CLAYEY SILT

Golder Associates
SUDBURY, ONTARIO

PROJECT No.		1777318	FILE No.	1777318.GPJ
DRAWN	TB	Jan 2018	SCALE N/A	REV.
CHECK	DAM	Jan 2018		
APPR	JMAC	Jan 2018	**FIGURE B3**	

SUD-MTO PL (2016) GLDR_LDN.GDT



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	C2-3	8	226.0
⧗	C2-4	6	229.1
▲	C2-5	6	227.0
★	C2-6	5	228.8

PROJECT

HIGHWAY 539A
CULVERT 2 AT STA 10+097

TITLE

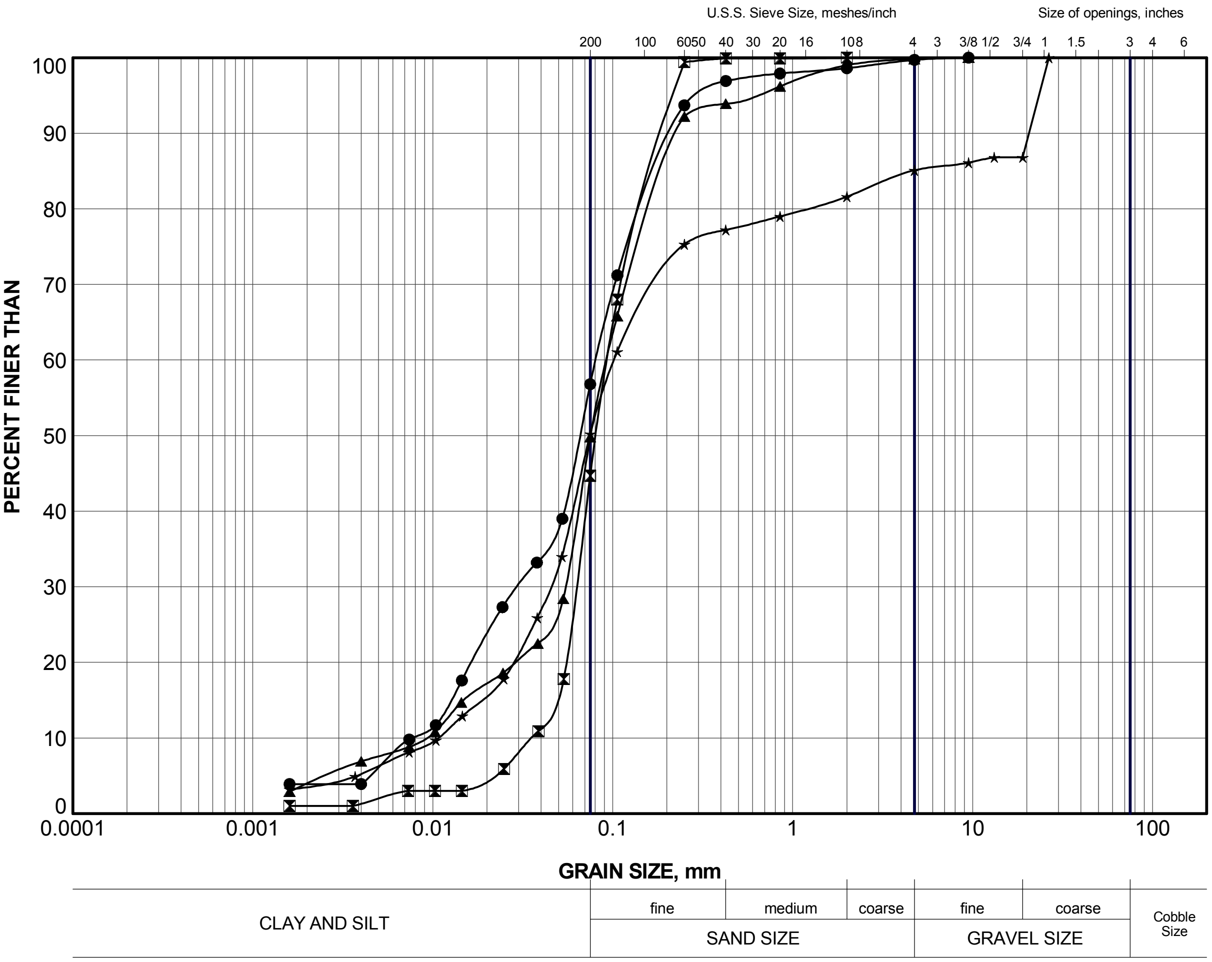
GRAIN SIZE DISTRIBUTION

CLAYEY SILT

PROJECT No.		1777318	FILE No.	1777318.GPJ
DRAWN	TB	Jan 2018	SCALE N/A	REV.
CHECK	DAM	Jan 2018		
APPR	JMAC	Jan 2018	FIGURE B4	

Golder Associates
SUDBURY, ONTARIO

SUD-MTO GSD (2016) GLDR_LDN.GDT



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	C2-1	11	222.2
⧗	C2-2	8	224.3
▲	C2-3	9B	224.2
★	C2-4	10	223.0

PROJECT

HIGHWAY 539A
CULVERT 2 AT STA 10+097

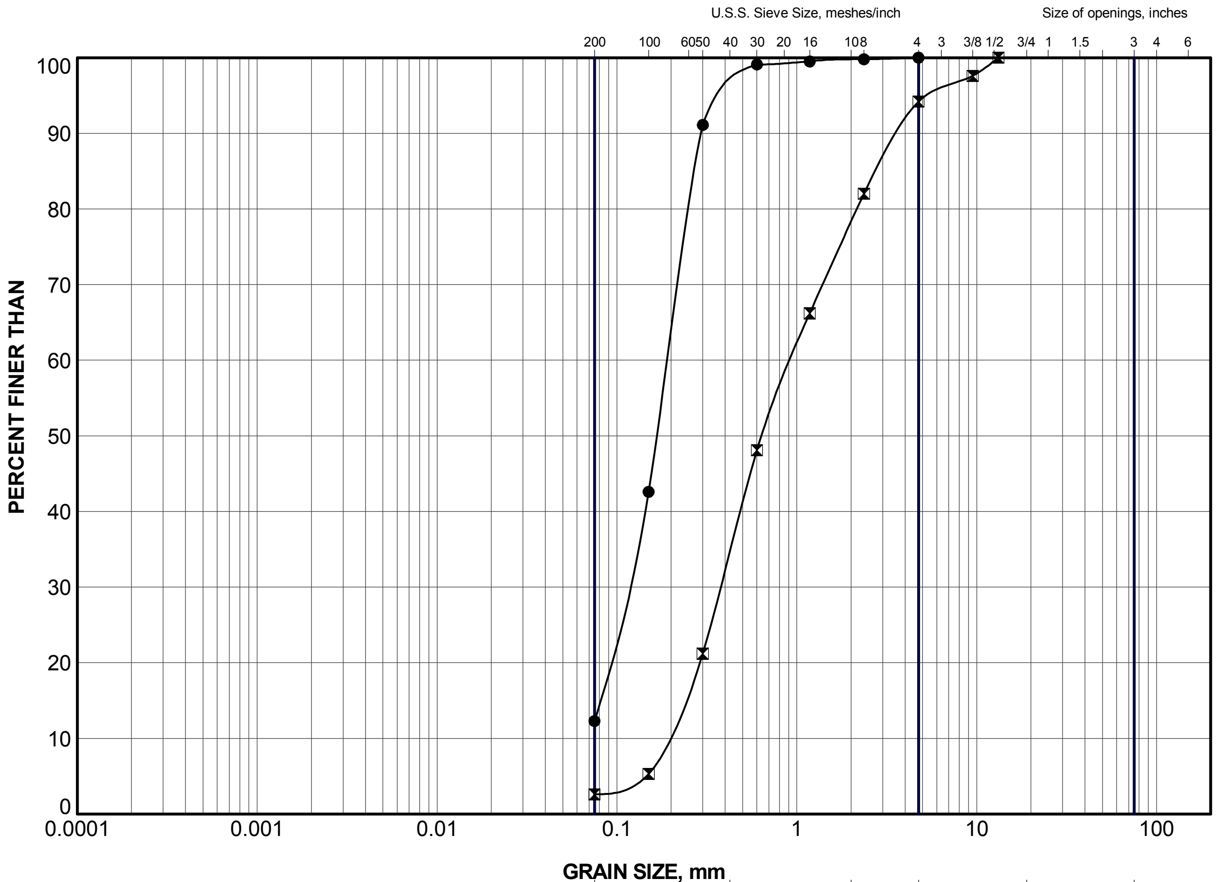
TITLE

GRAIN SIZE DISTRIBUTION
SILT and SAND

Golder Associates
SUDBURY, ONTARIO

PROJECT No.		1777318	FILE No.	1777318.GPJ	
DRAWN	TB	Jan 2018	SCALE	N/A	REV.
CHECK	DAM	Jan 2018			
APPR	JMAC	Jan 2018	FIGURE B5		

SUD-MTO GSD (2016) GLDR_LDN.GDT



CLAY AND SILT	SAND SIZE: fine	SAND SIZE: medium	SAND SIZE: coarse	GRAVEL SIZE: fine	GRAVEL SIZE: coarse	Cobble Size

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	C2-3	13	218.4
⧗	C2-6	8	224.3

PROJECT
HIGHWAY 539A
CULVERT 2 AT STA 10+097

TITLE
GRAIN SIZE DISTRIBUTION
SAND

Golder Associates
SUDBURY, ONTARIO

PROJECT No.		1777318	FILE No.	1777318.GPJ
DRAWN	TB	Jan 2018	SCALE N/A	REV.
CHECK	DAM	Jan 2018		
APPR	JMAC	Jan 2018	**FIGURE B6**	

SUD-MTO GSD (2016) GLDR_LDN.GDT

APPENDIX C

Non-Standard Special Provisions

<u>DEWATERING STRUCTURE EXCAVATIONS</u> - Item No.

Special Provision No. FOUN0003 March 8, 2018

Amendment to OPSS 902, November 2010

OPSS 902, November 2010, Construction Specification for Excavating and Backfilling - Structures is amended as follows:

902.02 REFERENCES

Section 902.02 of OPSS 902 is amended by the addition of the following:

Ontario Provincial Standard Specifications, Construction

OPSS 517	Dewatering
OPSS 805	Temporary Erosion and Sediment Control Measures

902.03 DEFINITIONS

Section 903.03 of OPSS 902 is amended by the addition of the following:

Automatic Transfer Switch means as defined in OPSS 517.

Cofferdam means as defined in OPSS 539.

Cut-Off Wall means as defined in OPSS 517.

Design Storm Return Period means as defined in OPSS 517.

Dewatering System means as defined in OPSS 517.

Groundwater Control System means as defined in OPSS 517.

Plug means as defined in OPSS 517.

Sediment means as defined in OPSS 517.

Sediment Control Measure means as defined in OPSS 517.

Temporary Flow Passage System means as defined in OPSS 517.

Unwatering means as defined in OPSS 517.

Vegetated Discharge Area means as defined in OPSS 517.

Waterbody means as defined in OPSS 517.

Watercourse means as defined in OPSS 517.

902.04 DESIGN AND SUBMISSION REQUIREMENTS

902.04.01 Design Requirements

902.04.01.01 Dewatering

Clause 902.04.01.01 of OPSS 902 is deleted in its entirety and replaced with the following:

A dewatering system shall be designed to control water and the flow of water into the excavation, prevent disturbance of the foundation, permit the placing of concrete in the dry, and complete the excavating and backfilling for structures work.

When the system includes temporary flow passage system, the system shall be designed, as a minimum, for a [* Designer Fill-In, See Notes to Designer] year design storm return period, and groundwater discharge. A longer return period shall be used when determined appropriate for the work.

The dewatering system shall be according to the design requirements specified in OPSS 517.

902.04.02 Submission Requirements

Subsection 902.04.02 of OPSS 902 is deleted in its entirety and replaced with the following:

902.04.02.01 Working Drawings

Working Drawings for the dewatering system shall be according to OPSS 517.

902.04.02.02 Preconstruction Survey

When a groundwater control system by wells or a well point system will be used, a condition survey of property and structures that may be affected by the work shall be carried out. The condition survey shall include the location and condition of adjacent properties, buildings, underground structures, water wells, Utilities, and structures, within a distance of [** Designer Fill-In, See Notes to Designer] metres from the groundwater control system. In addition, all water wells used as a supply of drinking water and located within this distance shall be tested for compliance with Ontario Drinking Water Quality Standards.

Water wells within the preconstruction survey distance can be located using the website https://www.ontario.ca/environment-and-energy/map-well-records or its successor site.

Copies of the condition survey and water quality test results shall be submitted to the Contract Administrator prior to the operation of the groundwater control system.

902.04.02.03 Milestone Inspections

Clause 902.04.02.03 of OPSS 902 is deleted in its entirety.

902.07 CONSTRUCTION

Subsection 902.07.04 of OPSS 902 is deleted in its entirety and replaced with the following:

902.07.04 Dewatering Structure Excavation

902.07.04.01 General

The dewatering systems shall be constructed and operated according to the Working Drawings.

Activation and deactivation of a temporary flow passage system, if applicable, shall be according to OPSS 517.

The dewatering system shall be continuously operational to control buoyancy forces until such forces can be resisted by backfill and structure self-weight, to keep excavations stable, to avoid erosion impacts from the release of accumulated water, and to keep the work area in the condition required to complete the associated work as specified in the Contract Documents.

When a temporary flow passage system is to remain operational through a seasonal shutdown period, the Contractor shall be responsible for any maintenance or repair costs due to the system during the seasonal shutdown period.

Temporary erosion and sediment control measures, including controlling the discharge of water, shall be according to OPSS 805. Measures not specified in OPSS 805 shall be according to the Working Drawings. Temporary erosion and sediment control measures and cover material to protect exposed soils, as required by the Working Drawings, shall be installed as soon as is practical.

Stranded fish shall be managed as specified in the Contract Documents.

Unwatering shall be carried out as necessary.

Water suspected of being contaminated as indicated by visual or olfactory observations shall be reported to the Contract Administrator.

Dewatering and temporary flow passage systems shall be discontinued in a manner that does not disturb any structure, pipeline, or flow channel. Operation of the dewatering system shall be shut down according to the procedures specified in the Working Drawings, where applicable.

902.07.04.02 Discharge of Water

The discharge of water shall be according to OPSS 517.

902.07.04.03 Monitoring

Monitoring shall be according to OPSS 517.

902.07.04.04 System Amendments

Amendments to stop any displacement, damage, soil loss or erosion due to the operation of the dewatering system shall be according to OPSS 517.

902.07.04.05 Removal

Removal of dewatering system and temporary flow passage system components shall be according to OPSS 517.

NOTES TO DESIGNER:

Designer Fill-Ins

*   Fill in the design storm return period according to MTO Drainage Design Standard TW-1.

**   Fill in the preconstruction survey distance as recommended by the foundation engineer.

WARRANT: Include with this standard tender item **only** on the recommendation of a foundation engineer.

CUSTODIAN: Tony Sangiuliano, MERO - Foundation Group.

SUBGRADE PROTECTION – Item No.

Non-Standard Special Provision

Scope of Work

The native silt and/or clayey silt subgrade at this site is susceptible to disturbance and loosening from construction traffic and ponded water. Any loosened or disturbed soils below the plan limits of the proposed works should be sub-excavated and replaced with compacted engineered fill. A 300 mm thick protection layer, or bedding layer, comprised of Granular A or Granular B Type II material should be placed in a timely manner after inspection and approval of the subgrade condition. Any disturbed soils below the plan limits of the proposed works should be sub-excavated and replaced with compacted engineered fill.

Basis of Payment

Payment at the lump sum contract price for the above tender item includes full compensation for all labour, equipment and material for completion of the work.

END OF SECTION

UNWATERING OF STRUCTURE EXCAVATION - Item No.

Non-Standard Special Provision

Construction of Culvert 2 will require excavations to extend below the groundwater level and the adjacent creek water level. The granular embankment fill and native silt and/or clayey silt deposits present below the groundwater table 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

GOLDER

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