FOUNDATION INVESTIGATION AND DESIGN REPORT

Culvert Extension for Highway 69 Station 22+350
Township of Dill
Ministry of Transportation, Ontario
GWP 5219-14-00

Submitted to:

D.M. Wills Associates Ltd.
150 Jameson Drive,
Peterborough, ON
K9J 0B9

Submitted by:

Golder Associates Ltd.
33 Mackenzie Street, Suite 100
Sudbury, Ontario, P3C 4Y1, Canada
+1 705 524 6861

1790361

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PART A

FOUNDATION INVESTIGATION REPORT

CULVERT EXTENSION AT HIGHWAY 69 STA 22+350

TOWNSHIP OF DILL

MINISTRY OF TRANSPORTATION, ONTARIO

GWP 5219-14-00

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 related to the extension of the existing culvert at Station 22+350 on Highway 69 in the Township of Dill, Ontario. The culvert is located approximately 300 m east of Richard Lake Drive. The Key Plan of the general location of this section of Highway 69 and the location of the investigated area are shown on Drawing 1.

The purpose of this exploration is to establish the subsurface conditions at the culvert site by borehole drilling, with laboratory testing carried out on selected soil samples.

The Terms of Reference (TOR) and the scope of work for the foundation investigation are outlined in MTO's Request for Proposal, dated August 2017, and the subsequent clarifications/addenda and change order No. 001, which forms part of the Consultant's Assignment Number 5017-E-0029 for this project. The work has been carried out in accordance with Golder's Supplementary Specialty Plan for foundation engineering services for this project dated April 2018.

2.0 SITE DESCRIPTION

Based on the preliminary GA drawing provided by D.M. Wills, the existing culvert consists of an 800 mm diameter, 48 m long Corrugated Steel Pipe (CSP). The culvert inlet (south end) and outlet (north end) inverts are approximately Elevations 233.9 m and 233.7 m, respectively. In general, the site conditions near the culvert ends consist of a swampy ravine directly at the culvert and to the west and bedrock outcrops to the east. At the location of the culvert along the highway centreline, the highway grade is at approximately Elevation 241.6 m and the embankment is approximately 7.7 m high relative to the invert of the culvert inlet. The embankment side-slopes appear to be constructed predominantly of blasted rock fill which appear to be performing adequately; however, both ends of the culvert could not be located and appeared to be buried by the embankment fill. Surficial erosion of the granular soils adjacent to the paved shoulder in the upper portion of the embankment slope was observed. The site conditions at select locations in the area of the culvert are shown on Photographs 1 and 2.

3.0 INVESTIGATION PROCEDURES

Field work for the subsurface exploration was carried out on March 11,12,18, and 19, 2019, during which time three boreholes (Boreholes RL-1 to RL-3) were advanced at approximately the locations shown on Drawing 1. Borehole RL-3 was advanced through the highway embankment using a track mounted CME 850 drilling rig supplied and operated by Landcore Drilling (Landcore) of Chelmsford, Ontario. Boreholes RL-1 and RL-2 were advanced near the toe of the highway embankment slope adjacent to the culvert inlet using portable tripod equipment supplied and operated by Landcore. A total of four (4) Dynamic Cone Penetration Tests (DCPTs) were advanced in the vicinity of the culvert inlet due to the shallow refusal encountered within the two boreholes advanced with portable tripod equipment. Traffic control, where required, was performed in accordance with MTO's Ontario Traffic Control Manual Book 7 – Temporary Conditions.

Boreholes RL-1 and RL-2 were advanced using NW casing with wash boring techniques and Borehole RL-3 was advanced through the roadway using 76 mm I.D. Hollow Stem Augers and NW casing with wash boring techniques. The coring in Borehole RL-3 was advanced using an NQ-size core barrel. Soil samples were generally 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 or manual (i.e., cathead) hammer in accordance with the Standard Penetration Test (SPT) procedure (ASTM D1586). The portable tripod used a standard weight (63.6 kg) hammer. Field vane shear tests were conducted in cohesive soils for determination of undrained shear strength (ASTM D2573) using an MTO Standard “N” size vane. The groundwater level inside the augers/casing was observed during and upon completion of drilling operations. The boreholes were backfilled in accordance with Ontario Regulation 903. The roadway surface at the borehole drilled through the highway was capped at ground surface using cold patch asphalt.

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; logged the boreholes; and examined the soil and bedrock samples. The soil and rock 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 determination, grain size distribution, and Atterberg limits were carried out on selected soil samples. The geotechnical laboratory testing was completed according to ASTM and MTO LS standards, as applicable.

The as-drilled borehole locations were measured relative to highway chainages/station marked on the pavement by a member of our technical staff and converted into northing/easting coordinates on the plan drawing. The ground surface elevations at the borehole locations were surveyed by Golder relative to the highway/culvert centreline. D.M. Wills provided the site survey with highway centerline elevation (referenced to Geodetic datum) on February 27, 2019. The MTM NAD 83-CSRS CBN v6-2010.0 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)
RL-1	5143569.0 46.431781	311521.2 -80.912546	235.9	2.5
RL-2	5143566.5 46.431758	311521.7 -80.912540	235.9	2.1
RL-3	5143584.6 46.431921	311519.9 -80.912563	241.5	20.2

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

Based on Northern Ontario Engineering Geology Terrain Study (NOEGTS)[1] mapping, the culvert site is reportedly located within a glaciolacustrine plain, and the soils in the area primarily consist of silt and sand.

Based on geological mapping (MNDM)[2], the site is reportedly underlain by Quartz-felspar sandstone, argillite and conglomerate.

4.2 Subsurface Conditions

The detailed subsurface soil and groundwater conditions encountered in the boreholes and the summary results of in situ and laboratory testing are given on the Record of Borehole sheets contained in Appendix A. The detailed results of geotechnical laboratory testing are contained in Appendix B. The results of the in-situ field tests (i.e., SPT 'N' values and Field Vanes) as presented on the Record of Borehole sheets and discussed in Section 4.2 are uncorrected. The stratigraphic boundaries shown on the Record of Borehole sheets and on the interpreted stratigraphic profile shown on Drawing 1 are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change.

The subsurface conditions will vary between and beyond the borehole locations, however, the factual data presented on the Record of Borehole sheets governs any interpretation of the site conditions. A summary description of the soil deposits and groundwater conditions encountered in the boreholes is provided below. It should be noted that the interpreted stratigraphy shown on Drawing 1 is a simplification of the subsurface conditions.

The depth to effective refusal of four dynamic cone penetration tests advanced in close proximity to Boreholes RL-1 and RL-2 are included in the notes on Record of Borehole RL-1 in Appendix A.

A description of the major soil strata and bedrock encountered during the exploration at the site are described below.

4.2.1 Asphalt/Fill

An approximately 350 mm thick layer of asphalt pavement was encountered in Borehole RL-3 at ground surface (i.e., Elevation 241.5 m). Below the asphalt, a 5.4 m thick layer of variable fill was encountered as follows. It is noted that when fill soils were not able to be sampled using the SPT procedure (i.e., when wash boring with casing was used to advance borehole due to presence of cobble to boulder-sized rock fragments), observations of drilling progress and flush water exiting the casing was used to infer the fill type. Directly below the asphalt, a 0.4 m thick layer of sand and gravel was encountered. An approximately 1.3 m thick layer of blast rock fill was encountered below the sand and gravel fill at Elevation 240.7 m, underlain by an approximately 0.9 m thick layer of silty sand fill at Elevation 239.4 m. Below the silty sand fill layer, an inferred cobble was encountered at

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

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

Elevation 238.6 m (based on grinding of casing) followed by an approximately 0.7 m thick layer of clayey gravel fill at Elevation 238.5 m. An approximately 1.4 m thick layer of blast rock fill was encountered below the clayey gravel fill at Elevation 237.7 m. At the bottom of the fill, an approximately 0.5 m thick layer of sand fill was encountered below the blast rock fill layer at Elevation 236.3 m.

Two SPT "N"-values measured within the clayey gravel fill and sand fill layers encountered in Borehole RL-3 at Elevations 238.5 m and 236.3 m, respectively, are 9 blows and 13 blows per 0.3 m of penetration, indicating a loose to compact compactness condition.

A grain size distribution analysis was carried out on one sample of the clayey gravel fill and the result is presented on Figure B-1 in Appendix B. An Atterberg limit test was carried out on the fine portion of the same clayey gravel fill and measured a liquid limit of 18 per cent, a plastic limit of 12 per cent, and a plastic index of 6 per cent. The results, which are presented on Figure B-2 in Appendix B, indicate that the fines portion of the sample is classified as a silt /clayey silt of low plasticity. The natural moisture content measured on the clayey gravel fill sample is 11 per cent. A grain size distribution analysis was carried out on one sample of the sand fill and the result is presented on Figure B-3 in Appendix B. The natural moisture content measured on the sand fill sample is 15 per cent.

4.2.2 Ice/Water

Boreholes RL-1 and RL-2 (located near the toe of the embankment) encountered an 80 mm thick layer of ice at Elevation 235.9 m over about 1.4 m of water at the time of investigation.

4.2.3 Peat

At the bottom of the water in Boreholes RL-1 and RL-2, a 0.6 m to 0.8 m thick layer of fibrous peat was encountered at Elevation 234.4 m. Borehole RL-2 was terminated at the bottom of this fibrous peat layer upon split-spoon refusal during the SPT at Elevation 233.8 m.

The SPT "N"-value measured within the fibrous peat layer encountered in Borehole RL-1 is 1 blow per 0.3 m of penetration suggesting a very soft consistency.

4.2.4 Gravel

Below the peat in Borehole RL-1, an approximately 0.2 m thick layer of coarse gravel was encountered at Elevation 233.6 m. Borehole RL-1 was terminated at the bottom of this coarse gravel layer due to split-spoon refusal during the SPT at Elevation 233.4 m.

4.2.5 Silt

A 7.0 m thick deposit of silt was encountered underlying the fill in Borehole RL-3 at Elevation 235.8 m. The silt typically contained some clay, with trace organics encountered near the interface with the fill.

The SPT “N”-value measured within this silt ranges from 5 blows to 41 blows per 0.3 m of penetration, indicating a loose to dense compactness condition. One in situ field vane test performed within this layer measured a shear strength value greater than 100 kPa.

Three grain size distribution analyses were carried out on samples of the silt layer and the results are presented on Figure B-4 in Appendix B. Three Atterberg limit tests were carried out on samples of the deposit with one test yielding a non-plastic result and the other two tests yielding a liquid limit of 20 per cent and 24 four percent, a plastic limit of 15 per cent and 19 per cent, and a plastic index of 5 per cent. The results are shown on Figure B-5 and confirm that portions of the silt have slight plasticity and are near the transition to being classified as clayey silt with low plasticity. The natural moisture content measured on samples of the deposit ranges between 16 per cent and 26 per cent.

4.2.6 Clayey Silt

An approximately 3.9 m thick layer of clayey silt was encountered underlying the silt layer within Borehole RL-3 at about Elevation 228.9 m.

The STP “N”-values measured within the clayey silt layer range between weight of hammer (WH) per 0.3 m of penetration to 6 blows per 0.3 m of penetration. Two in situ field vane tests performed within the layer measured shear strengths of about 38 kPa, indicating a firm consistency.

One grain size distribution analysis was carried out on a sample within the clayey silt layer and the results are presented on Figure B-6 in Appendix B. One Atterberg limits test was carried out on a sample of the deposit yielding a liquid limit of 21 per cent, a plastic limit of 16 per cent and a plastic index of 5 per cent. The results, which are presented on Figure B-7 in Appendix B, indicate the layer is a clayey silt-silt of low plasticity. The natural moisture content measured on one sample of the deposit is 23 per cent.

4.2.7 Lower Silt

An approximately 0.4 m thick layer of silt was encountered below the clayey silt layer in Borehole RL-3 at approximately Elevation 224.9 m.

An SPT was attempted within this layer and measured 5 blows per 0.13 m of penetration, after which depth the split-spoon achieved effective refusal on bedrock.

4.2.8 Bedrock/Refusal

Bedrock was inferred to be encountered upon refusal of the split-spoon sampler in Boreholes RL-1 and RL-2, and confirmed by coring in Borehole RL-3. Also, bedrock outcrops were confirmed east of the culvert inlet by visual observation and as shown on Drawing 1.

In Boreholes RL-1 and RL-2 (and the four accompanying DCPTs advanced near the culvert inlet), refusal to further penetration was encountered between 1.7 m and 2.9 m below ice surface, corresponding to inferred top of bedrock between Elevations 234.2 m and 233.0 m.

In Borehole RL-3, bedrock was cored from Elevation 224.5 m to 221.3 m (length of 3.2 m). The total core recovery (TCR) of the bedrock core is 100 per cent, solid core recovery (SCR) ranges from 27 percent to 90 per cent and the Rock Quality Designation (RQD) ranges from 27 per cent to 98 per cent. The rock core is described as fine to medium grained, slightly weathered to fresh, grey granitic gneiss. The record of drillhole is displayed in Appendix A.

4.3 Groundwater Conditions

The unstabilized groundwater levels relative to ground surface measured inside the casing or augers upon completion of drilling are summarized below. The ice level of the watercourse near the culvert inlet, as surveyed by Golder on March 19, 2019, was Elevation 235.9 m. Groundwater and watercourse levels in the area are subject to seasonal fluctuations and variations due to precipitation events.

Borehole No.	Ground/Ice Surface Elevation (m)	Depth to Unstabilized Groundwater Level (m)	Approximate Groundwater Elevation (m)
RL-1	235.9	0.0	*235.9
RL-2	235.9	0.0	*235.9
RL-3	241.5	5.3	236.3

Note: *Ice surface elevation of watercourse near culvert inlet/swamp

4.4 Analytical Laboratory Testing Results

Analytical testing was carried out on a sample of the silt deposit near the invert level recovered from Borehole RL-3. The soil sample was submitted to Maxxam Analytics of Sudbury, Ontario for corrosivity testing. The analytical laboratory test results are summarized below, and the detailed analytical laboratory test report is included in Appendix B.

Borehole No.	Sample No.	Depth (m)	Parameters					
			Resistivity (ohm-cm)	Electrical Conductivity (µmho/cm)	Soluble Sulphate (SO_4) Content (µg/g)	Sulphide (S^-) (µg/g)	Soluble Chloride (Cl) Content (µg/g)	pH
RL-3	2b	5.7-5.8	2,000	506	<20[1]	<0.50[1]	240	7.69

Note:
1. Below the reportable detection limit.

5.0 CLOSURE

The field exploration program was carried out under the supervision of Mr. Mathew Riopelle, under the overall direction of Mr. André Bom, P.Eng. This Foundation Investigation Report was prepared by Mr. Gavin Mundry, and Mr. André Bom, P.Eng. provided a technical review of the report. Mr. Kevin Bentley, P.Eng., an MTO Foundations Designated Contact and Associate for Golder, conducted an independent quality control review of this report.

Signature Page

Golder Associates Ltd.

Gavin Mundry, EIT
Geotechnical Engineer-in-Training

LICENSED PROFESSIONAL ENGINEER
Sept 23/19
A. J. K. BOM
100075715
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
K. J. BENTLEY
SEPT. 23/2019
PROVINCE OF ONTARIO

André Bom, P.Eng.
Senior Geotechnical Engineer, Associate

Kevin Bentley, P.Eng.
MTO Foundations Designated Contact, Associate

GM/AB/KB/sb/ca

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PART B

FOUNDATION DESIGN REPORT

CULVERT EXTENSION AT HIGHWAY 69 STA 22+350

TOWNSHIP OF DILL

MINISTRY OF TRANSPORTATION, ONTARIO

GWP 5219-14-00

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides foundation design and recommendations related to the extension of the culvert at Station 22+350 on Highway 69 in the Township of Dill, Ontario. These recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the current subsurface investigation. The discussion and recommendations presented are intended to provide the designer with sufficient information to assess feasible foundation alternatives to design the proposed culvert extension.

The foundation investigation report, discussion and recommendations are intended for the use of the Ministry of Transportation, Ontario (MTO) and shall not be used or relied upon for any other purpose or by any other parties, including the construction or design-build contractor. The contractor must make their own interpretation based on the factual data in Part A (Foundation Investigation) of the report. Where comments are made on construction, they are provided to highlight those aspects that could affect the design of the project, and for which special provisions may be required in the Contract Documents. Those requiring information on 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 Proposed Culvert Extension and Installation Options

The existing culvert consists of an 800 mm diameter, 48 m long Corrugated Steel Pipe (CSP) that is buried under embankment fill at both ends. The culvert inlet invert (south end) is at approximately Elevation 233.9 m and the embankment side-slopes near the culvert inlet location (south side) are sloped at about 1.6H:1V. Based on the drawing provided by D.M. Wills (via email on February 27, 2019) and Golder's site observations during the foundation exploration work, the existing culvert crosses Highway 69 on a skew alignment. During our site investigation, the embankment was observed to be performing well, with some surficial erosion evident along the upper portion of the existing side slopes, where granular soils were observed adjacent to the paved shoulder. It is understood from D.M. Wills that the culvert inlet was previously extended about 3.7 m to the south as part of a previous contract. The current assignment proposes to remove the previous 3.7 m extension, and replace with a 5 m long concrete non-rigid frame box (NRFB) culvert (1200 mm wide by 1200 mm high) extension, such that the culvert inlet will project beyond the existing embankment toe of slope, and the remaining portion of the culvert will be left in place. The design should incorporate appropriate details at the connection of the existing CSP and proposed concrete box to reduce to potential for soil / water containing fines migration into the culvert. As referenced in Section 2.0, the existing embankment at the inlet / south side is approximately 7.7 m high relative to the inlet / south culvert invert. It is understood from D.M. Wills that the proposed culvert extension will follow the existing culvert alignment. Based on Golder's Borehole RL-3 and visual observation of the embankment slope, the embankment is generally constructed of a mixture of granular fill and blast rock fill, with no visual sign of embankment instability.

It is understood from D.M. Wills that open cut excavation is being considered for the replacement and extension of the culvert section near the inlet. It is understood the side-slopes are to remain at about 1.6H:1V and there is no grade raise or widening of the roadway embankment. It is also understood that the south lane (eastbound lane) could be closed during culvert extension and that temporary shoring during construction is not preferred due to anticipated challenges due to the presence of the blast rock fill on the sides and within the embankment.

6.2 Consequence and Site Understanding Classification

In accordance with Section 6.5 of the *Canadian Highway Bridge Design Code* (CHBDC, 2014) and its *Commentary*, the proposed culvert extension is located adjacent to Highway 69 which carries high traffic volumes and its performance will have potential impacts on other transportation corridors; hence, the structure/foundation system is classified as having a “typical consequence level” associated with exceeding limits states design. In addition, given the typical project-specific foundation investigation carried out at this site (as presented in Part A of the report), in comparison to the degree of site understanding in Section 6.5 of *CHBDC* (2014), the level of confidence for design is considered to be a “typical degree of site and prediction model understanding”. Accordingly, the appropriate corresponding ultimate limit state (ULS) and serviceability limit state (SLS) consequence factor, Ψ, and geotechnical resistance factors, ϕ_{gu} and ϕ_{gs}, from Tables 6.1 and 6.2 of the *CHBDC* have been used for design, as applicable.

6.3 Partial Culvert Replacement and Extension by Open Cut Excavation

6.3.1 Global Stability and Settlement

The existing south embankment slope has an inclination of approximately 1.6 horizontal to 1 vertical (~1.6H:1V) near the culvert location. Limit equilibrium slope stability analyses was performed on the existing south slope at the culvert extension replacement using GeoStudio 2019 software, employing the Morgenstern-Price method of analysis. For the analyses, the Factor of Safety (FoS) is defined as the ratio of the forces tending to resist failure to the driving forces tending to cause failure. A target minimum FoS of 1.54 is required for long term, permanent embankment slopes with a Typical or High consequence factor (e.g., near structures) when referencing CHBDC (2014).

The idealized soil profile and associated soil parameters (based on boreholes RL-1 to RL-3) used for the stability analyses are shown in Figure 1.

The calculated FoS for the existing south embankment slope is about 1.31, as shown on Figure 1, which is generally considered acceptable for a Low consequence level when referencing CHBDC (2014), as the culvert is not considered a structure and the existing embankment has been in operation for many years with no indication of instability. In order to achieve a target FoS of 1.54 for this site, a 3 m wide rock fill berm with surface Elevation 237 m extending from the existing toe of slope would be required. The construction of the berm would require a longer culvert extension.

Alternatively, the existing FoS = 1.31 may be considered acceptable to MTO if the site is considered to have a Low consequence level, given that the embankment has been in place for many years and provided the current slope configuration (1.6H:1V) is re-established using rock fill after construction.

It is understood from discussion with D.M. Wills that a temporary or permanent widening or grade raise will not be required for the partial culvert replacement and extension. Due to the presence of the cohesive deposit below the existing embankment, a settlement analysis of the culvert/embankment foundation should be carried out if a temporary or permanent widening or grade raise is required (and update to stability assessment discussed above).

6.3.2 Bedding/Embedment and Cover

The concrete culvert extension should be completed in accordance with the applicable Ontario Provincial Standards for the chosen box size and material type. Considering the bottom of the new extension will be located near the bedrock surface, the extension should consist of a material type that will limit the required depth of excavation.

The extension should be designed in accordance with the MTO Gravity Pipe Design Guidelines (2014), if applicable.

If the culvert extension replacement is to consist of a CSP or plastic pipe, it should be constructed in accordance with OPSD 802.014 (Flexible Pipe Embedment in Embankment) and OPSD 802.010 (Flexible Pipe Embedment and Backfill – Type 3 Soil Earth Excavation). Alternatively, if concrete pipe is used, reference should be made to OPSD 802.031 (*Rigid Pipe Bedding, Cover and Backfill - Excavation) and OPSD 802.034 (Rigid Pipe Bedding and Cover in Embankment – Original Ground Earth or Rock).*

The proposed structural design is to use a concrete box (1.2 m wide x 1.2 m high) for the extension and reference should be made to OPSD 803.010 (*Backfill and Cover for Concrete Culverts with Span Less Than or Equal to 3 m*).

It is understood the extended culvert will have an invert at Elevation 233.8 m. The results of the investigation indicate the bedrock surface may range from Elevation 233.1 m to 234 m in the vicinity of the proposed extension footprint. All unsuitable, deleterious, organic materials, fibrous peat, and fill materials are to be removed from the base/below the culvert extension footprint along its entire alignment. The bedding (minimum 150 mm thick) should be compatible with the class of pipe (if applicable), the surrounding subsoil and anticipated loading conditions and should consist of OPSS.PROV 1010 (Aggregates) Granular ‘B’ Type II or Granular ‘A’ soil. Depending on the success of the contractor’s groundwater control methods, and the quality of the bearing stratum exposed at the base of the excavation, a thicker bedding layer may be required at some locations where wet and softened soil conditions, unsuitable fill, or organic material are present near the base of the excavation. Therefore, the Contract Documents should include a provision for additional thickness of bedding, if required. Alternatively, bedrock may be present above the design subgrade level. In order to avoid the requirement to remove / sub-excavate the granitic gneiss, it is recommended that the hydraulic design be checked for an invert level of about 234.3 m for the new extension, such that design subgrade can allow for some undulation of the bedrock surface and a minimum 150 mm bedding thickness be placed (plus thickness of precast base slab).

From the top of the bedding to 300 mm above the top of the culvert, Granular “B” Type II or ‘A’ should be used around the culvert. All bedding, embedment and cover materials should be placed, and culvert construction carried out in accordance with OPSS 422 (*Precast Reinforced Concrete Box Culverts in Open Cut*) and OPSS 401 (*Trenching, Backfilling and Compacting*), and the bedding/embedment/cover soil should be compacted in accordance with OPSS.PROV 501 (*Compacting*).

As the bottom of the excavation is expected to be below the surrounding water level, and may be susceptible to wet saturated conditions, especially if dewatering is not satisfactorily maintaining the water level sufficiently below the base of the excavation to allow compaction, it is recommended that OPSS.PROV 1010 (*Aggregates*) Granular ‘B’ Type II material be used for bedding and as additional sub-excavation backfill below the bedding, as may be required.

Embankment (trench) backfill / reconstruction above the culvert is discussed in Section 6.5.5.

6.4 Analytical Testing for Construction Materials

The results of analytical tests on one sample of native silt recovered in Borehole RL-3 is summarized in Section 4.4. The potential for sulphate attack and corrosion are discussed in the following paragraphs; however, it is ultimately up to the designer to determine the appropriate construction materials, including the exposure class, and ensuring that all aspects of CSA A23.1-14 (2014) Section 4.1.1 “Durability Requirements” are followed when designing concrete elements, as applicable.

6.4.1 Potential for Sulphate Attack

The analytical test results were compared to CSA A23.1-14 Table 3 ("Additional requirements for concrete subjected to sulphate attack”) for the potential sulphate attack on concrete. The water soluble-sulphate concentration measured in the soil sample is less than the reportable detection limit of 0.002 per cent, which is below the exposure class of S-3 (Moderate) and is considered Negligible according to Table 7.2 in the MTO Gravity Pipe Design Guidelines (2014). Therefore, based on the test result for the sample, when the designer is selecting the exposure class for the structure, the effects of sulphates from within the near surface/culvert invert native soil(s) may not need to be considered.

6.4.2 Potential for Corrosion

The soil has a pH of 7.7 and according to the MTO Gravity Pipe Design Guidelines (2014), the pH is not considered detrimental to culvert durability. The resistivity is 2,000 ohm-cm, which indicates that the soil corrosiveness is Severe to Moderate, as per Table 3.2 of the MTO Gravity Pipe Design Guidelines (2014). It is also noted that sulphide at a concentration of <0.50 µg/g was detected in the analyzed test sample; and sulphide is considered very corrosive to cast iron/steel materials (Cashman and Preene, 2001). As the culvert extension may be exposed to de-icing salt, concrete should be designed for a “C” type exposure class as defined by CSA A23.1-14 Table 1. The culvert should be designed with consideration given to Table 7.1 of the MTO Gravity Pipe Design Guidelines (2014).

6.5 Construction Considerations

6.5.1 Temporary Open Cut Excavation

The proposed temporary open cut excavation through the embankment and into the subgrade/bedding of the existing culvert section to be replaced will generally advance through granular fill (sand and gravel, silty sand, clayey gravel, and sand) and blast rock fill, and into the native silt deposit. The proposed extension will result in open cut through the fibrous peat soils, with the potential for excavation into the bedrock if the assumed invert levels do not change.

The excavation must be carried out in accordance with the guidelines outlined in the Occupation Health and Safety Act (OHSA) for Construction Activities in Ontario. Above the water table, the existing fill materials and underlying native granular and cohesive soils are classified as Type 3 soil, according to OHSA and temporary excavations (i.e., those which are open for a relatively short time period) should be made with side slopes no steeper than 1 horizontal to 1 vertical (1H:1V). Below the water table, the existing sandy fill materials, fibrous peat and silt may be classified as Type 4 soil according to OHSA, and temporary excavations (i.e., those which are open for a relatively short time period) into this soil type should be made with side slopes no steeper than

3 horizontal to 1 vertical (3H:1V). Alternatively, temporary protection systems may be used for cuts below the water table through these silty or sandy soils. Temporary Protection systems must be in accordance with OPSS539 (Protection Systems) meeting Performance Level 2.

Due to the height of the embankment at this site, the existing subsurface conditions encountered at the boreholes and the proposed 1H:1V temporary excavated side slope, a conceptual slope stability analysis was carried out using the similar methods and procedures described in Section 6.3.1. Assuming plain strain conditions (which is not the case for the anticipated relatively narrow temporary cut), the calculated FoS for the temporary conditions is near one and the slip surface fails through near the crest of the slope. For conceptual design purposes, a 25 kPa surcharge was added near the crest of the slope (on the highway shoulder) in the stability model to represent a typical excavator during construction, the resulting FoS was calculated to be less than 1 (see Figure 2).

As a result, it is recommended that the width of the temporary excavation be limited, and any equipment/vehicles be restricted to areas outside the temporary cut slope face (at the highway level). The following constraints must be included in the Contract package (see example Operational Constraint in Appendix C) for the replacement of the culvert extension to mitigate potential instability of the slope during excavation:

- Close the most southerly/outside eastbound lane adjacent to the proposed excavation during and while the temporary excavation remains open.
- Limit the length of time that the excavation can be left open, for example a maximum of 3 days and not over the weekend. If a longer period is required, the excavated slopes should be protected from erosion and surface water during rain events.
- The 1H:1V excavated side slopes should be limited to no wider than 3 m directly above the culvert extension replacement location.
- Construction equipment and vehicular traffic should be not positioned closer than 5 m from the crest of the embankment above the limits of the temporary excavation.
- Stockpiles should not be placed on the highway grade or on the existing embankment slope in the vicinity of the existing / excavated embankment near the culvert.

Depending on the construction procedures adopted by the contractor, groundwater seepage conditions, and weather conditions at the time of construction, some local flattening of the open cut excavation slopes may be required, especially where looser/softer zones or where localized seepage is encountered. Further, variable layering of soils and the effectiveness of the contractor’s dewatering systems could alter the OHSA classification and, therefore, the classification of soils for OHSA purposes should be confirmed during construction and an allowance for temporary protection systems be made, especially near the toe of the embankment, where fibrous peat and silts were present below approximately 1.5 m of open water.

6.5.2 Groundwater Control

The groundwater/watercourse level (about Elevation 236 m) was measured to be about 2 m above the culvert invert based on the March 2019 investigation, but could be lower or higher depending on the water level at the time of construction. The temporary excavation is expected to extend below the groundwater level. The groundwater should be lowered to at least below the base of the excavation to maintain basal stability and allow adequate placement and compaction of bedding, embedment and cover soils. Groundwater may be controlled by providing an active dewatering system installed and operated in advance of the excavation, and/or in combination with a cofferdam system (e.g., isolated cut-off wall/sand bags) for the replacement/extension.

The contractor is responsible for the design and installation of all groundwater control measures giving due consideration to the type of cofferdam system, temporary shoring selected as well as the requirements for maintaining the stability/integrity of the foundation subgrade and/or requirement for the replacement options to be performed in-the-dry. Dewatering should be carried out in accordance with OPSS.PROV 517 (*Dewatering).*

Surface water should be directed away from the excavation / work area(s) to prevent ponding of water that could result in disturbance and loosening/softening of the foundation subgrade and/or excessive moisture that could compromise placement and compaction of bedding and embedment / backfill methods of construction. A turbidity curtain (OPSD 219.260) may also be required depending on environmental requirements.

Dewatering operations must be in accordance with OPSS.PROV 517 (*Construction Specification for Dewatering*) and MTO's Special Provision 517F01 (*Dewatering System / Temporary Flow Passage System*) recommending that a design engineer be required. A copy of SP 517F01 is included in Appendix C and includes fill-ins provided by D.M. Wills and Golder. Depending on the design of the temporary cofferdam and dewatering systems, if construction water pumping rates are anticipated to exceed 50 m^3/day, an Environmental Activity Section Registry (EASR) or Permit to Take Water (PTTW) will be required as per the recently introduced changes to the Environmental Protection Act by the Ontario Ministry of Environment and Climate Change (MOECC)/Ontario Ministry of the Environment, Conservation and Parks (MECP).

Erosion protection at the new inlet will need to be designed by the hydraulic engineer to control erosion around the culvert opening, near the embankment toe and scour below the invert that could lead to instability of the highway embankment and/or culvert bedding materials.

6.5.3 Obstructions

Rock fill (cobble to boulder-sized) was visually confirmed on the embankment side-slopes and was encountered in Borehole RL-3. These obstructions will affect temporary excavation into the embankment side slope, equipment, and sequencing, and may impact selection of any temporary protections systems or cofferdam systems used. A Notice to Contractor to identify the presence of cobbles and boulders (rock fill) should be included in the Contract Documents; a copy of which is included in Appendix C.

6.5.4 Subgrade Protection

For open cut culvert installation, the subgrade soils will be susceptible to disturbance from construction traffic and/or ponded water. To limit this degradation, it is recommended that the granular bedding layer be placed immediately after preparation and approval of the subgrade.

6.5.5 Embankment Reconstruction

The existing embankment fill, consisting of granular fill (sand and gravel, silty sand) and blast rock fill may be used for embankment reconstruction/trench backfill. The embankment backfill should be free of organic and cohesive soil (including the existing clayey gravel fill) and should be placed and compacted in accordance with OPSS.PROV 501 (Compacting) and OPSS.PROV 206 (Grading). Embankment side slopes should be constructed no steeper than the existing slope of 1.6H to 1V with the combined granular/blast rock fill. Any excavated clayey fill will need to be removed/wasted and is not considered suitable for re-use as embankment backfill, a Notice to Contractor is included in Appendix C.

6.5.6 Surficial Embankment Stability and Erosion Protection

Depending on the actual embankment fill material type that is removed and is to be re-instated, the final slope geometry, surface treatment and weather conditions (i.e., precipitation, cycles of wetting-drying and/or freezing-thawing), surficial instability of the embankment side slopes may occur, which could include localized sloughing and erosion. As such, in order to maintain the integrity of the reconstructed embankments, erosion protection measures may be required depending on the fill type selected for construction.

Based on the reconstructed embankment using the existing granular and blast rock fill, these materials have a low potential for erosion. However, it is anticipated that the proposed reconstructed slope will be generally consistent with the existing slope at about 1.6H:1V, such that a culvert extension greater than 5 m is not required. As noted in Section 6.1, surficial erosion was observed on the existing slope and it is anticipated that erosion control consisting of hydro-seeding and vegetation following the reconstruction of the embankment is not practical due to the granular / rock fill slope steeper than 2H:1V. Consideration should be given to placement of a minimum 400 mm thick layer of OPSS.PROV 1004 (Aggregates-Miscellaneous) R-10 Rip-Rap for erosion protection and to limit future maintenance of the slopes.

7.0 CLOSURE

This foundation design report was prepared by Mr. Gavin Mundry and Mr. André Bom, P.Eng., a senior geotechnical engineer and Associate of Golder. Mr. Kevin Bentley, P.Eng., an MTO Foundations Designated Contact and Associate with Golder conducted an independent and quality control review of the report.

Signature Page

Golder Associates Ltd.

Gavin Mundry, EIT
Geotechnical Engineer-in-Training

LICENSED PROFESSIONAL ENGINEER
Sept 23/19
A. J. K. BOM
100075715
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
K. J. BENTLEY
SEPT. 23/2019
PROVINCE OF ONTARIO

André Bom, P.Eng.
Senior Geotechnical Engineer, Associate

Kevin Bentley, P.Eng.
MTO Foundations Designated Contact, Associate

GM/AB/KB/sb/ca

https://golderassociates.sharepoint.com/sites/22732g/deliverables/foundations/2. reporting/3. final/1790361 r-rev0 mto deep fill culvert hwy 69 fidr 23sept_19.docx

REFERENCES

Canadian Standards Associations (CSA) Group 2014. Canadian Highway Bridge Design Code and Commentary S6-14

Canadian Standards Association (CSA), 2014. CSA A23.1-14 Concrete Materials and Methods of Construction (R2014)

Cashman, P.M. and Preene M. (2001) Groundwater Lowering in Construction, A Practical Guide. Spoon Press Publisher.

Ministry of Transportation, Ontario, MTO Gravity Pipe Design Guidelines, April 2014

Ontario Ministry of Natural Resources and Forestry. Northern Ontario Engineering Geology Terrain Study. Ontario Geological Society Electronic Mapping. Map 41PNE

Ontario Ministry of Northern Development and Mines. Bedrock Geology of Ontario, East-Central Sheet. Map 2543

Ontario Regulation 903 (Wells)

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

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 Saturated Fine-Grained Soils

Ontario Provincial Standard Drawings (OPSD)

OPSD 802.010	Flexible Pipe Embedment and Backfill – Type 3 Soil Earth Excavation
OPSD 802.014	Flexible Pipe Embedment in Embankment Original Ground: Earth or Rock
OPSD 802.031	Rigid Pipe Bedding, Cover, And Backfill, Type 3 Soil - Earth Excavation
OPSD 803.034	Rigid Pipe Bedding and Cover in Embankment – Original Ground Earth or Rock
OPSD 803.010	Backfill and Cover for Concrete Culverts with Span Less Than or Equal to 3.0 m

Ontario Provincial Standard Specifications (OPSS) – Provincial Oriented

OPSS.PROV 206	Construction Specification for Grading
OPSS.PROV 401	Construction Specification for Trenching, Backfilling and Compacting
OPSS.PROV 421	Construction Specification for Pipe Culvert Installation in Open Cut
OPSS 422	Construction Specification for Precast Reinforced Concrete Box Culverts in Open Cut
OPSS.PROV 501	Construction Specification for Compacting
OPSS.PROV 517	Construction Specification for Dewatering of Pipeline, Utility, and Associated Structure Excavation
OPSS.PROV 539	Construction Specification for Temporary Protection Systems
OPSS.PROV 1004	Material Specification for Aggregates - Miscellaneous
OPSS.PROV 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material

MINISTRY OF TRANSPORTATION, ONTARIO

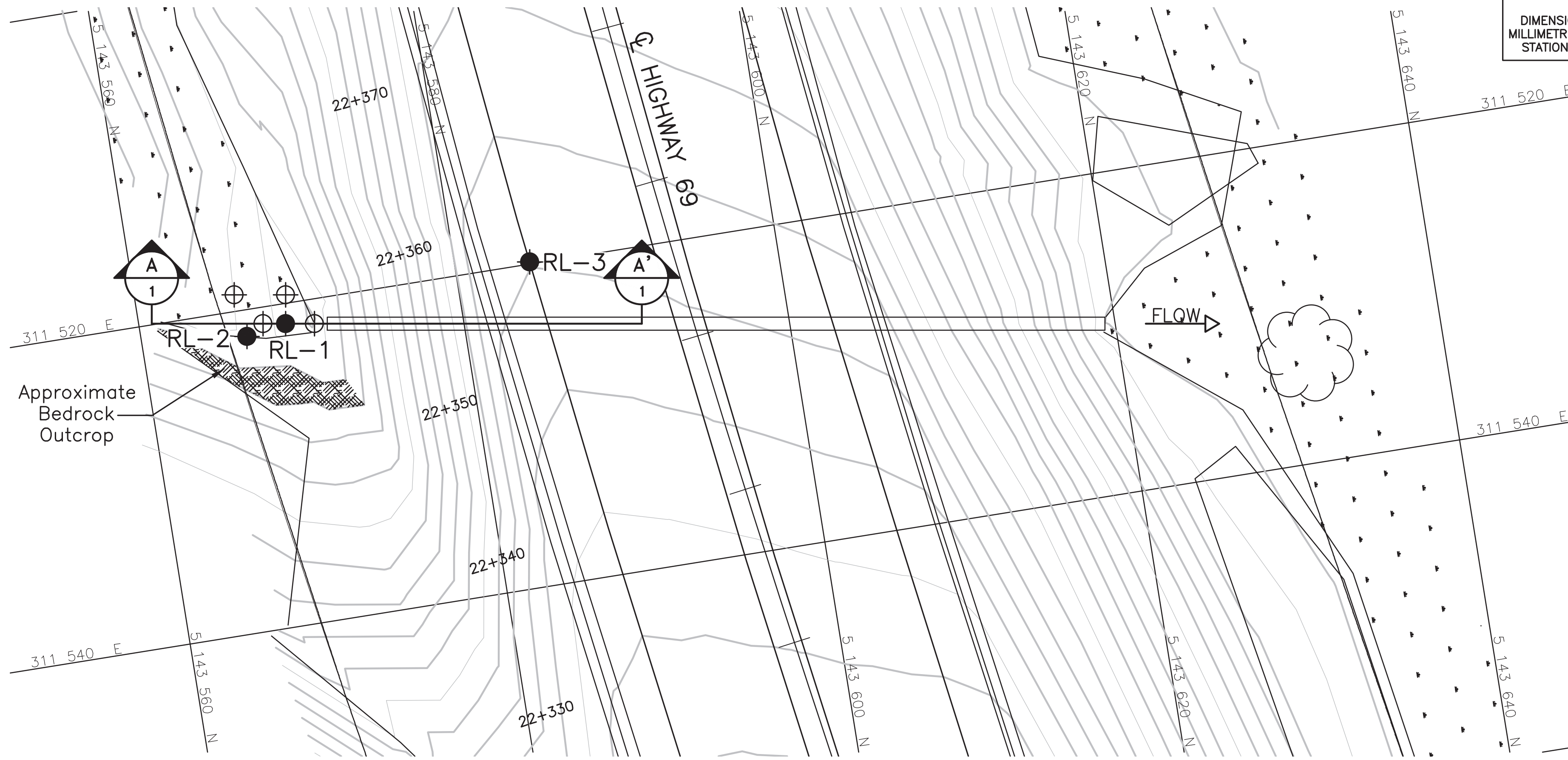
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
GWP No. 5219-14-00

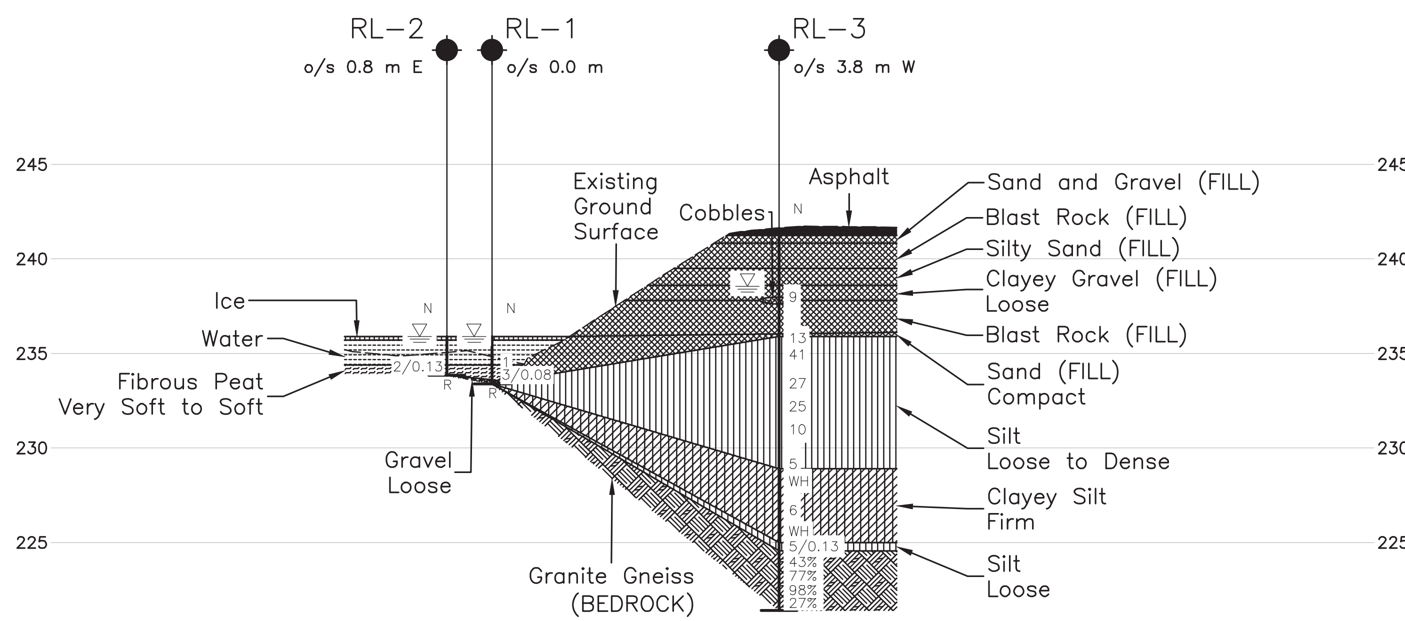
SHEET

HIGHWAY 69
DEEP FILL CULVERT
BOREHOLE LOCATIONS AND SOIL STRATA

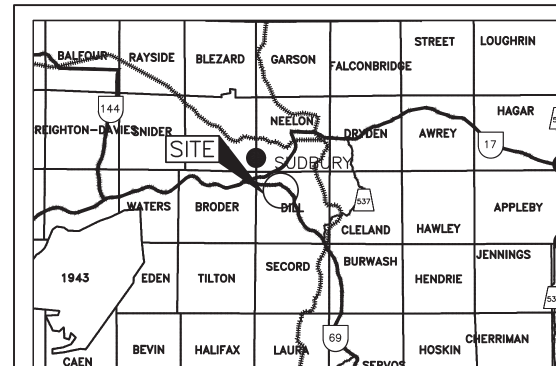
GOLDER



PLAN
SCALE
4 0 4 8 m



A-A
1
CROSS-SECTION
SCALE
4 0 4 8 m



KEY PLAN
SCALE
10 0 10 20 km

LEGEND

●	Borehole – Current Investigation
⊕	Dynamic Cone Penetration Test
N	Standard Penetration Test Value
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
R	Split-spoon refusal
100%	Rock Quality Designation (RQD)
▽	WL upon completion of drilling

BOREHOLE CO-ORDINATES (NAD 83 MTM ZONE 12)

No.	ELEVATION	NORTHING	EASTING
RL-1	235.9	5143569.0	311521.2
RL-2	235.9	5143566.5	311521.7
RL-3	241.5	5143584.6	311519.9

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 CALLON DIETZ INC. drawing file no. gwp52191400b.dwg, received MAY 16, 2019.

NO.	DATE	BY	REVISION

Geocres No. 41I-363

HWY. 69		PROJECT NO. 1790361	DIST.
SUBM'D.	CHKD. GM	DATE: 9/23/2019	SITE:
DRAWN: TR	CHKD. AB	APPD. KB	DWG. 1

PLOT DATE: September 23, 2019
FILENAME: S:\Clients\MTO\Hwy69\99_PROJ\1790361\40_PROD\0001_Richard_Lake\1790361-0001-RG-0001.dwg

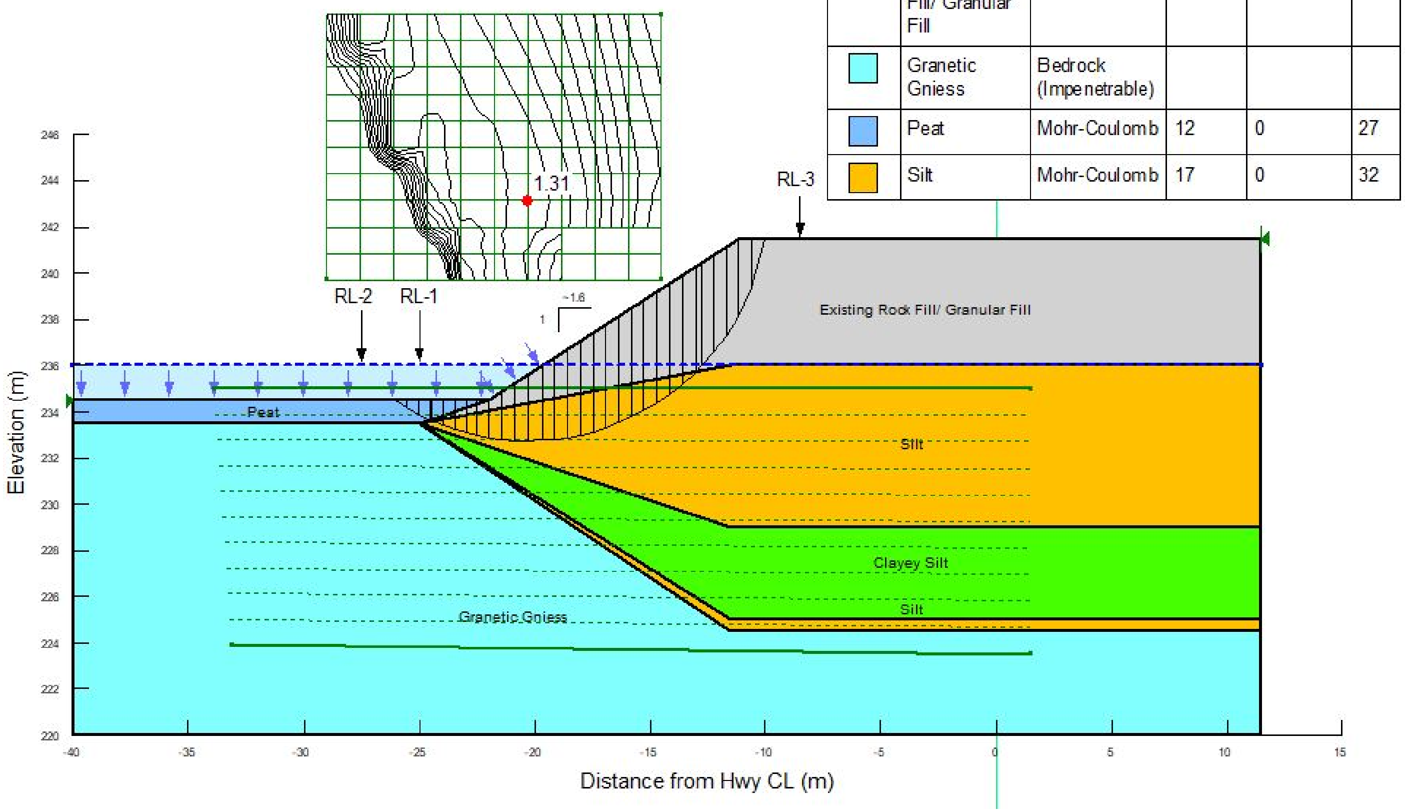
Photograph 1: South Embankment (Inlet End), Looking South-West (from D.M. Wills)

Photograph 2: South Embankment (Inlet End), Looking South-East (from D.M. Wills)

Global Stability Analysis
Highway 69 Station 22+350 Culvert
Existing/Proposed Embankment South Slope

Figure 1

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
Green	Clayey Silt	Mohr-Coulomb	17	40	0
Grey	Existing Rock Fill/ Granular Fill	Mohr-Coulomb	20	0	37
Cyan	Granetic Gniess	Bedrock (Impenetrable)			
Blue	Peat	Mohr-Coulomb	12	0	27
Orange	Silt	Mohr-Coulomb	17	0	32



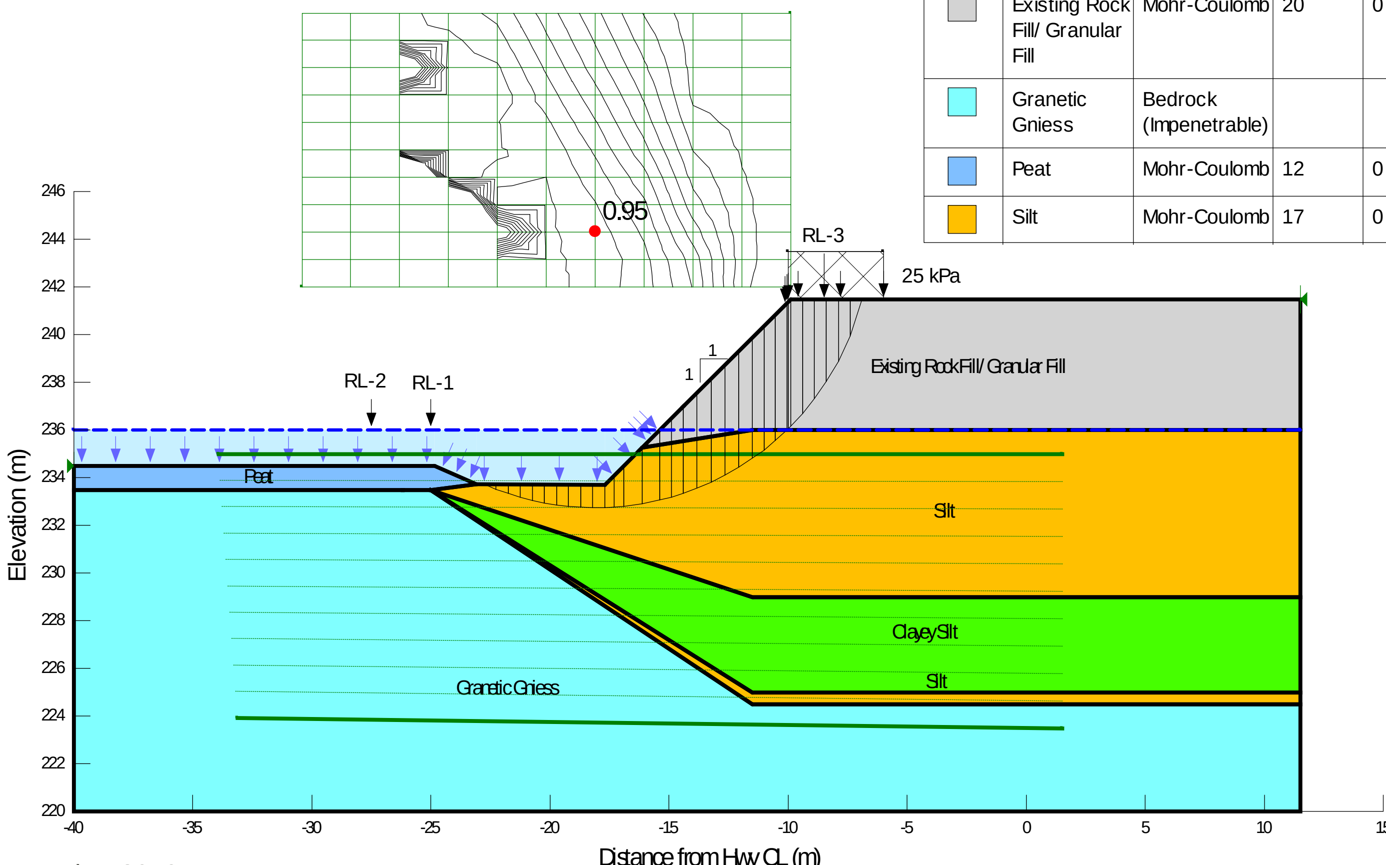
Date: June 2019
Project No: 1790361

Analysis By: GM **Reviewed By:** AB

Global Stability Analysis
Highway 69 Station 22+350 Culvert
Embankment South Slope-Temporary Excavation

Figure 2

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
(green)	Clayey Silt	Mohr-Coulomb	17	40	0
(grey)	Existing Rock Fill/ Granular Fill	Mohr-Coulomb	20	0	37
(cyan)	Granetic Gniess	Bedrock (Impenetrable)			
(blue)	Peat	Mohr-Coulomb	12	0	27
(orange)	Silt	Mohr-Coulomb	17	0	32



Date: September 2019
Project No: 1790361

Analysis By: GM Reviewed By: AB

GOLDER

APPENDIX A

Record of Boreholes

ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES AND TEST PITS
MINISTRY OF TRANSPORTATION, ONTARIO

PARTICLE SIZES OF CONSTITUENTS

Soil Constituent	Particle Size Description	Millimetres	Inches (US Std. Sieve Size)
BOULDERS	Not Applicable	>300	>12
COBBLES	Not Applicable	75 to 300	3 to 12
GRAVEL	Coarse Fine	19 to 75 4.75 to 19	0.75 to 3 (4) to 0.75
SAND	Coarse Medium Fine	2.00 to 4.75 0.425 to 2.00 0.075 to 0.425	(10) to (4) (40) to (10) (200) to (40)
FINES	Classified by plasticity	<0.075	< (200)

MODIFIERS FOR SECONDARY COMPONENTS[1,2]

Percentage by Mass	Modifier
> 35	Use 'and' to combine primary and secondary component (*i.e.,* SAND and gravel)
> 20 to 35	Primary soil name prefixed with "gravelly, sandy" as applicable
> 10 to 20	some (*i.e.,* some sand)
≤ 10	trace (*i.e.,* trace fines)

1. Only applicable to components not described by Primary Group Name.
2. Classification of Primary Group Name based on Unified Soil Classification System (ASTM D2487) for coarse-grained soils; fine-grained soils described per current MTO Soil Classification System.

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.) split-spoon sampler for a distance of 300 mm (12 in.). Values reported are as recorded in the field and are uncorrected.

Cone Penetration Test (CPT)
An 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 (u) and sleeve friction (f_s) are recorded electronically at 25 mm penetration intervals.

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

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

SAMPLES

AS	Auger sample
BS	Block sample
CS	Chunk sample
DD	Diamond Drilling
DO or DP	Seamless open ended, driven or pushed tube sampler – note size
DS	Denison type sample
GS	Grab Sample
MC	Modified California Samples
MS	Modified Shelby (for frozen soil)
RC / SC	Rock core / Soil core
SS	Split spoon sampler – note size
ST	Slotted tube
TO	Thin-walled, open – note size (Shelby tube)
TP	Thin-walled, piston – note size (Shelby tube)
WS	Wash sample
OD / ID	Outer Diameter / Inner Diameter
HSA / SSA	Hollow-Stem Augers / Solid-Stem Augers

SOIL TESTS

w	water content
PL , w_p	plastic limit
LL , 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, Gs)
DS	direct shear test
GS	specific gravity
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 (FV)	field vane (LV-laboratory vane test)
γ	unit weight

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

COARSE-GRAINED SOILS

Compactness[1]

Term	SPT 'N' (blows/0.3m)[2]
Very Loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	➢ 50

3. Definition of compactness terms are based on SPT 'N' ranges as provided in Terzaghi, Peck and Mesri (1996). Many factors affect the recorded SPT 'N' value, including hammer efficiency (which may be greater than 60% in automatic trip hammers), overburden pressure, groundwater conditions, and grainsize. As such, the recorded SPT 'N' value(s) should be considered only an approximate guide to the soil compactness. These factors need to be considered when evaluating the results, and the stated compactness terms should not be relied upon for design or construction.
4. SPT 'N' in accordance with ASTM D1586, uncorrected for the effects of overburden pressure.

FINE-GRAINED SOILS

Consistency

Term	Undrained Shear Strength (kPa)	SPT 'N'[1,2] (blows/0.3m)
Very Soft	< 12	0 to 2
Soft	12 to 25	2 to 4
Firm	25 to 50	4 to 8
Stiff	50 to 100	8 to 15
Very Stiff	100 to 200	15 to 30
Hard	> 200	> 30

1. SPT 'N' in accordance with ASTM D1586, uncorrected for overburden pressure effects; approximate only.
2. SPT 'N' values should be considered ONLY an approximate guide to consistency; for sensitive clays (e.g., Champlain Sea clays), the N-value approximation for consistency terms does NOT apply. Rely on direct measurement of undrained shear strength or other manual observations.

Field Moisture Condition

Term	Description
Dry	Soil flows freely through fingers.
Moist	Soils are darker than in the dry condition and may feel cool.
Wet	As moist, but with free water forming on hands when handled.

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
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
U	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

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

* 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 = $(w_l - w_p)$
NP	non-plastic
w_s	shrinkage limit
I_L	liquidity index = $(w - w_p) / I_p$
I_C	consistency index = $(w_l - w) / I_p$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
I_D	density index = $(e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(d) Shear Strength

τ_p, τ_r	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction = tan δ
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

GOLDER

RECORD OF BOREHOLE No RL-1

 METRIC

PROJECT 1790361

G.W.P. 5219-14-00 LOCATION N 5143569.0; E 311521.2 NAD83 MTM ZONE 12 (LAT. 46.431781; LONG. -80.912546) ORIGINATED BY MR

DIST HWY 69 BOREHOLE TYPE Portable Equipment, NW Casing and Wash Boring COMPILED BY GM/TR

DATUM GEODETIC DATE March 11, 2019 CHECKED BY AB

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)	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
235.9	GROUND SURFACE					▽					
0.0 / 0.1	ICE (80 mm) WATER						235				
234.4 / 1.5	Fibrous PEAT, some silt, trace to some sand, trace gravel. Very soft. Dark brown. Wet		1	SS	1		234				
233.6 / 233.4 / 2.5	Coarse GRAVEL, trace sand, trace wood. Grey. Wet		2	SS	3/0.08						
	END OF BOREHOLE Split-Spoon Refusal (Hammer Bouncing)										

NOTES:

1. Water level at ice surface (Elev. 235.9 m) upon completion of drilling.
2. Advanced DCPT 1.4 m south of borehole and refusal at a depth of 1.9 m below ice surface.
3. Advanced DCPT 1.8 m north of borehole and refusal at a depth of 2.8 m below ice surface.
4. Advanced DCPT 1.8 m west of borehole and refusal at a depth of 1.7 m below ice surface.
5. Advanced DCPT 3.2 m south and 1.8 m west of borehole and refusal at a depth of 2.9 m below ice surface.

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

PROJECT 1790361

RECORD OF BOREHOLE No RL-2

 METRIC

G.W.P. 5219-14-00 | LOCATION N 5143566.5; E 311521.7 NAD83 MTM ZONE 12 (LAT. 46.431758; LONG. -80.91254) | ORIGINATED BY MR

DIST | HWY 69 | BOREHOLE TYPE Portable Equipment, NW Casing and Wash Boring | COMPILED BY GM/TR

DATUM GEODETIC | DATE March 11, 2019 | CHECKED BY AB

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)	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
235.9	GROUND SURFACE										
0.0	ICE (80 mm)										
0.1	WATER						235				
234.4 1.5	Fibrous PEAT, trace gravel Soft Dark brown Wet		1	SS	2/0.13		234				
233.8 2.1	END OF BOREHOLE Split-Spoon Refusal (Hammer Bouncing) NOTE: 1. Water level at ice surface (Elev. 235.9 m) upon completion of drilling.										

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

GOLDER

Foundation Design

RECORD OF BOREHOLE No RL-3

 METRIC

PROJECT 1790361

G.W.P. 5219-14-00

LOCATION N 5143584.6; E 311519.9 NAD83 MTM ZONE 12 (LAT. 46.431921; LONG. -80.912563)

ORIGINATED BY MR

DIST ___ HWY 69

BOREHOLE TYPE 76 mm I.D. Hollow Stem Augers, NW Casing and Wash Boring

COMPILED BY GM/TR

DATUM GEODETIC

DATE March 18-19, 2019

CHECKED BY AB

ELEV / DEPTH (m)	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ (kN/m³)	GR	SA	SI	CL
241.5	GROUND SURFACE									
241.2 / 0.0	ASPHALT (350 mm)									
240.7 / 0.4	Sand and gravel (FILL)									
239.4 / 0.8	Blast rock (FILL) - Inferred cobble to boulder size fragments	-	NQ	-						
238.5 / 2.1	Silty sand (FILL) - Cobble at 3.0 m depth.									
237.7 / 3.0	Clayey gravel (FILL) Loose Brown to grey Wet	1	SS	9			38	23	26	13
236.3 / 3.8	Blast rock (FILL) - Inferred cobble to boulder size fragments	-	NQ	-						
235.8 / 5.2	Sand, some gravel, trace silt (FILL) Compact Brown Wet	2a / 2b	SS	13	▽		16	75	(10)	
234.3 / 5.7	SILT, trace to some gravel, trace organics Dense Grey to brown Wet	3	SS	41		NP	20	7	64	9
7.2	SILT, some clay, trace sand Loose to compact Brown to grey Wet	4	SS	27						
		5	SS	25			0	1	84	15
		6	SS	10						

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

Field vane at approx. elev. 230.2: >100

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

ELEVATION SCALE: 241, 240, 239, 238, 237, 236, 235, 234, 233, 232, 231, 230

REMARKS & GRAIN SIZE DISTRIBUTION (%)

Continued Next Page

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

RECORD OF BOREHOLE No RL-3

 METRIC

PROJECT 1790361
G.W.P. 5219-14-00
LOCATION N 5143584.6; E 311519.9 NAD83 MTM ZONE 12 (LAT. 46.431921; LONG. -80.912563)
ORIGINATED BY MR
DIST ___ HWY 69
BOREHOLE TYPE 76 mm I.D. Hollow Stem Augers, NW Casing and Wash Boring
COMPILED BY GM/TR
DATUM GEODETIC
DATE March 18-19, 2019
CHECKED BY AB

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION / SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m^3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SILT, some clay, trace sand Loose to compact Brown to grey Wet	7	SS	5		229				0 1 83 16
228.9 12.7	CLAYEY SILT, trace sand Firm Grey Wet	8	SS	WH		228				
							+ 2			
		9	SS	6		227				
						226	+ 2			
		10	SS	WH						0 1 78 21
224.9 16.6	SILT, trace plastic fines Loose Grey Wet	11	SS	5/0.13		225				
224.5 17.0	GRANITIC GNEISS (BEDROCK) For coring details see Record of Drillhole RL-3.	1	RC	REC 100%		224				RQD = 43%
		2	RC	REC 100%		223				RQD = 77%
		3	RC	REC 100%		222				RQD = 98%
		4	RC	REC 100%						RQD = 27%
221.3 20.2	END OF BOREHOLE									

NOTE:

1. Water level at a depth of 5.3 m below ground surface (Elev. 236.3 m) upon completion of drilling.

2. Drilling through fill soils started with hollow stem augers but switched to casing / coring techniques to penetrate rockfill.

3. Fill descriptions (where not sampled) are inferred from observation during drilling / casing advancement and observing flush water returns.

Legend: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED

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

PROJECT: 1790361

RECORD OF DRILLHOLE: RL-3

SHEET 1 OF 1

LOCATION: N 5143584.6; E 311519.9
NAD83 MTM ZONE 12 (LAT. 46.431921; LONG. -80.912563)
INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: March 19, 2019
DRILL RIG: CME 850 Track Mount
DRILLING CONTRACTOR: Landcore Drilling

DATUM: GEODETIC

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

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	FLUSH COLOUR % RETURN	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION
		GROUND SURFACE		224.5			
17	NW	GRANITIC GNEISS Fine to medium grain Slightly weathered to fresh Grey		17.0	1	Red/brown 100	JIR JCR
18	NQ Coring March 19, 2019				2	Red/brown 100	JIR
19					3	Red/brown 100	JPLSM
20				221.3 20.2	4	Red/brown 100	
		END OF DRILLHOLE					

Column headings: RECOVERY (TOTAL CORE %, SOLID CORE %), R.Q.D. %, FRACT. INDEX METRES, B Angle, DIP w.r.t. CORE AXIS, Jr, Ja, Jn, HYDRAULIC CONDUCTIVITY k, cm/s (10^{-6}, 10^{-5}, 10^{-4}, 10^{-3}), Diametral Point Load Index (MPa), RMC -Q' AVG.

Depth scale: 17, 18, 19, 20, 21, 22, 23, 24, 25, 26, 27, 28, 29

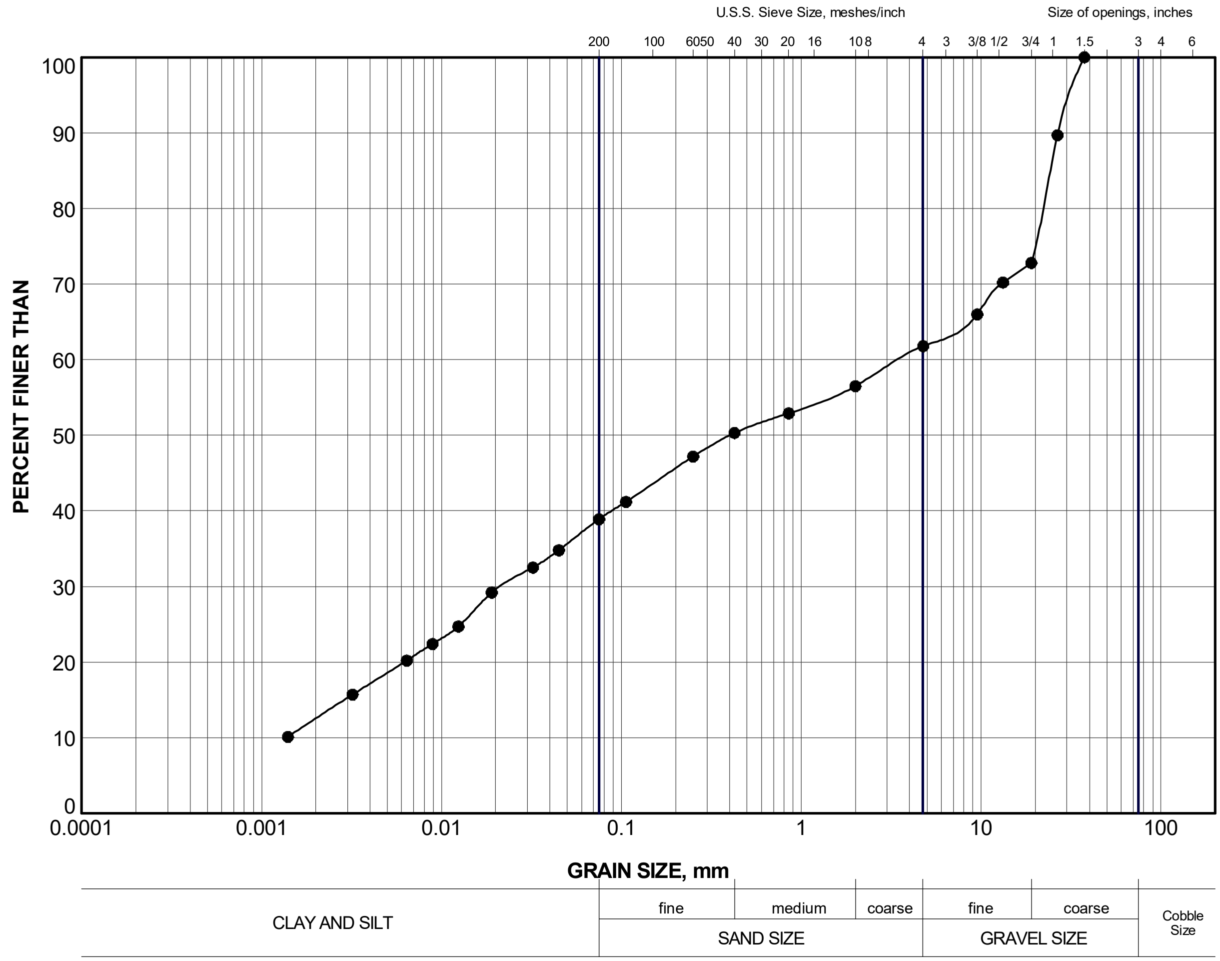
DEPTH SCALE
1 : 60

GOLDER

LOGGED: MR
CHECKED: AB

APPENDIX B

Laboratory Test Results



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	RL-3	1	238.2

PROJECT

HIGHWAY 69
DEEP FILL CULVERT

TITLE

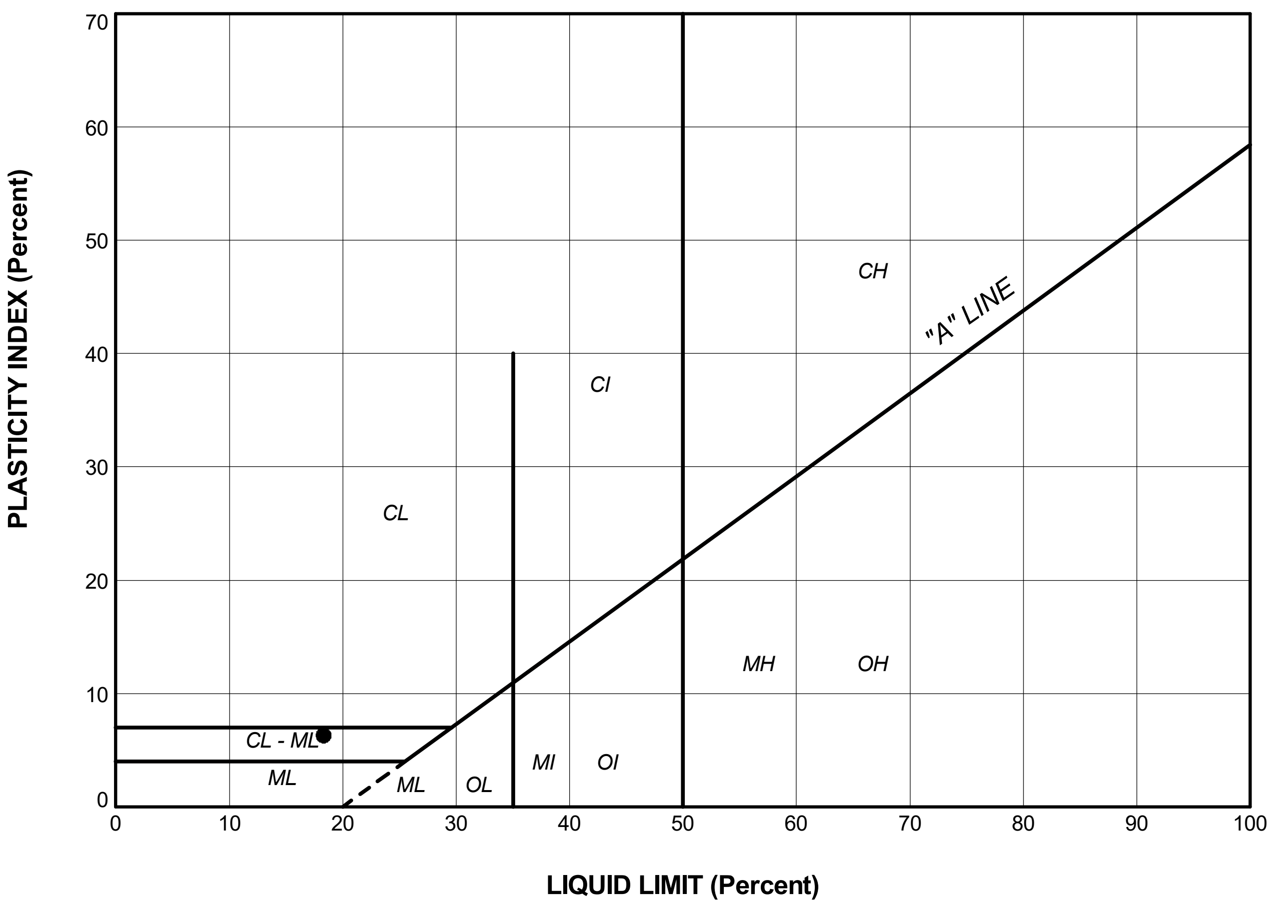
GRAIN SIZE DISTRIBUTION

Clayey Gravel (FILL)

GOLDER
SUDBURY, ONTARIO

PROJECT No.		1790361	FILE No.	1790361.GPJ
DRAWN	TR	Sep 2019	SCALE N/A	REV.
CHECK	AB	Sep 2019	**FIGURE B-1**	
APPR	KB	Sep 2019		

SUD-MTO GSD GLDR_LDN.GDT



SOIL TYPE
C = Clay
M = Silt
O = Organic

PLASTICITY
L = Low
I = Intermediate
H = High

LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	RL-3	1	18.3	12.0	6.3

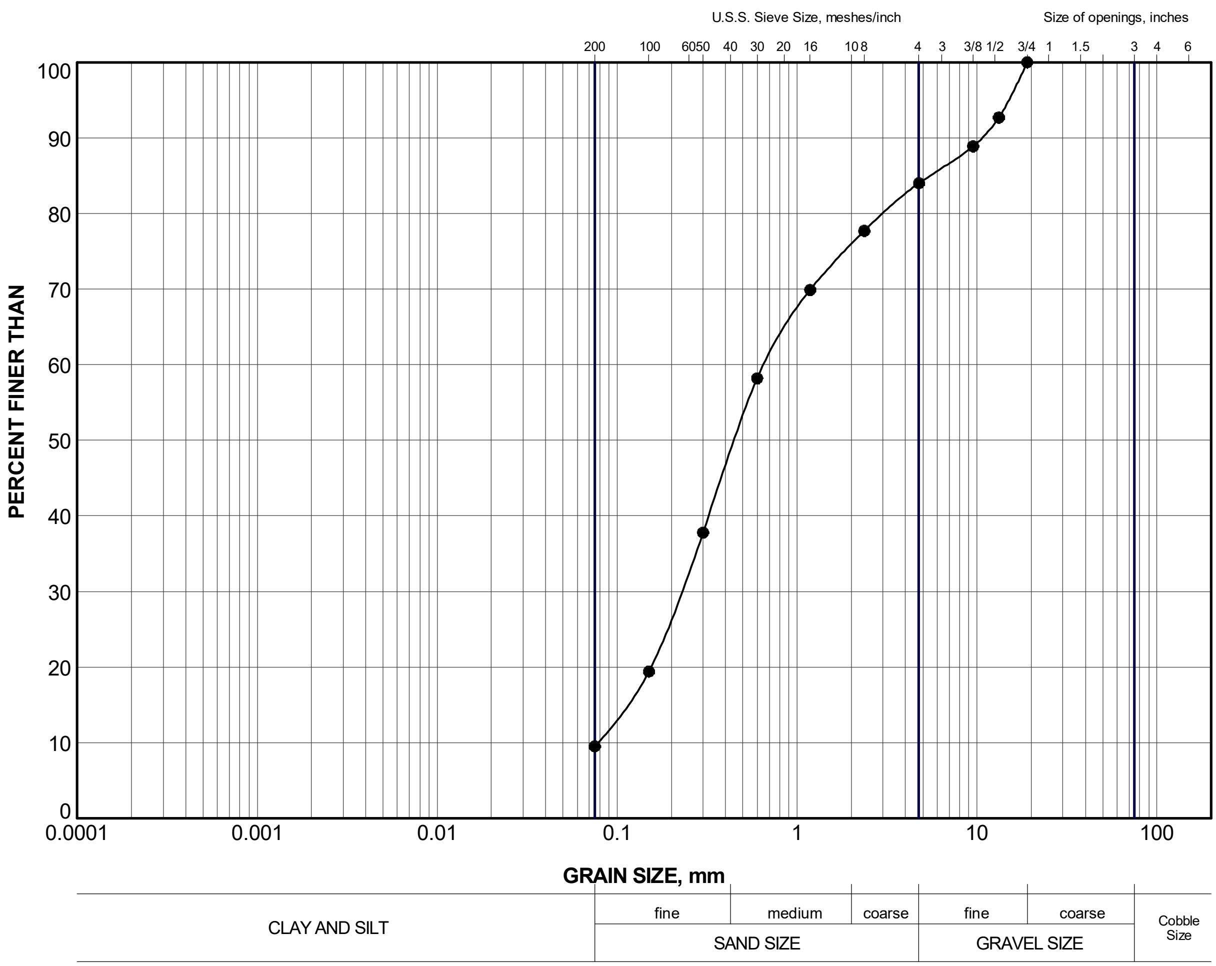
PROJECT
HIGHWAY 69
DEEP FILL CULVERT

TITLE
PLASTICITY CHART
Clayey Gravel (FILL)

GOLDER
SUDBURY, ONTARIO

PROJECT No.		1790361	FILE No.	1790361.GPJ
DRAWN	TR	Sep 2019	SCALE N/A	REV.
CHECK	AB	Sep 2019		
APPR	KB	Sep 2019	**FIGURE B-2**	

SUD-MTO PL GLDR_LDN.GDT



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	RL-3	2A	236.1

PROJECT

HIGHWAY 69
DEEP FILL CULVERT

TITLE

GRAIN SIZE DISTRIBUTION

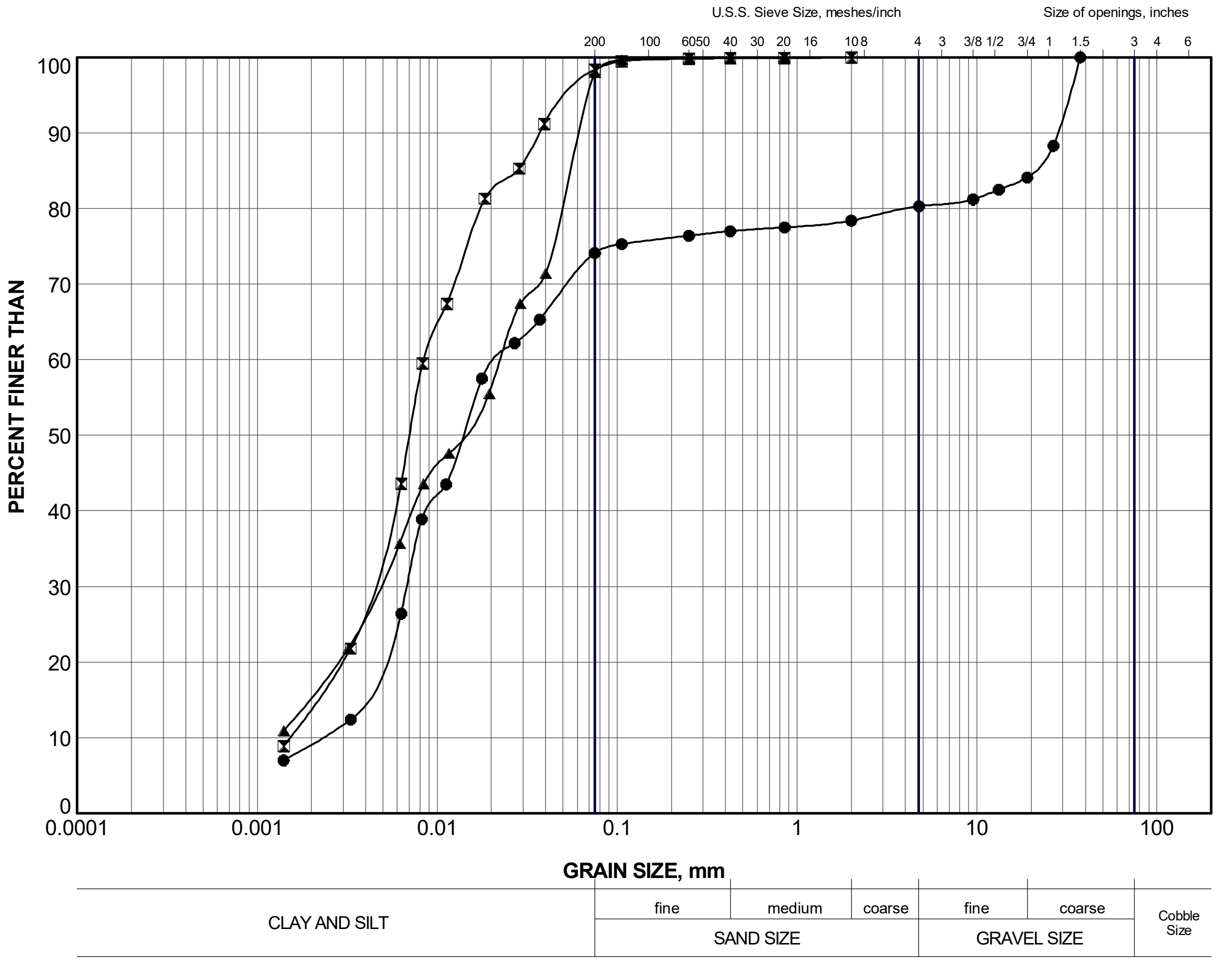
Sand (FILL)

GOLDER

SUDBURY, ONTARIO

PROJECT No.		1790361	FILE No.	1790361.GPJ
DRAWN	TR	Sep 2019	SCALE N/A	REV.
CHECK	AB	Sep 2019	**FIGURE B-3**	
APPR	KB	Sep 2019		

SUD-MTO GSD GLDR_LDN.GDT

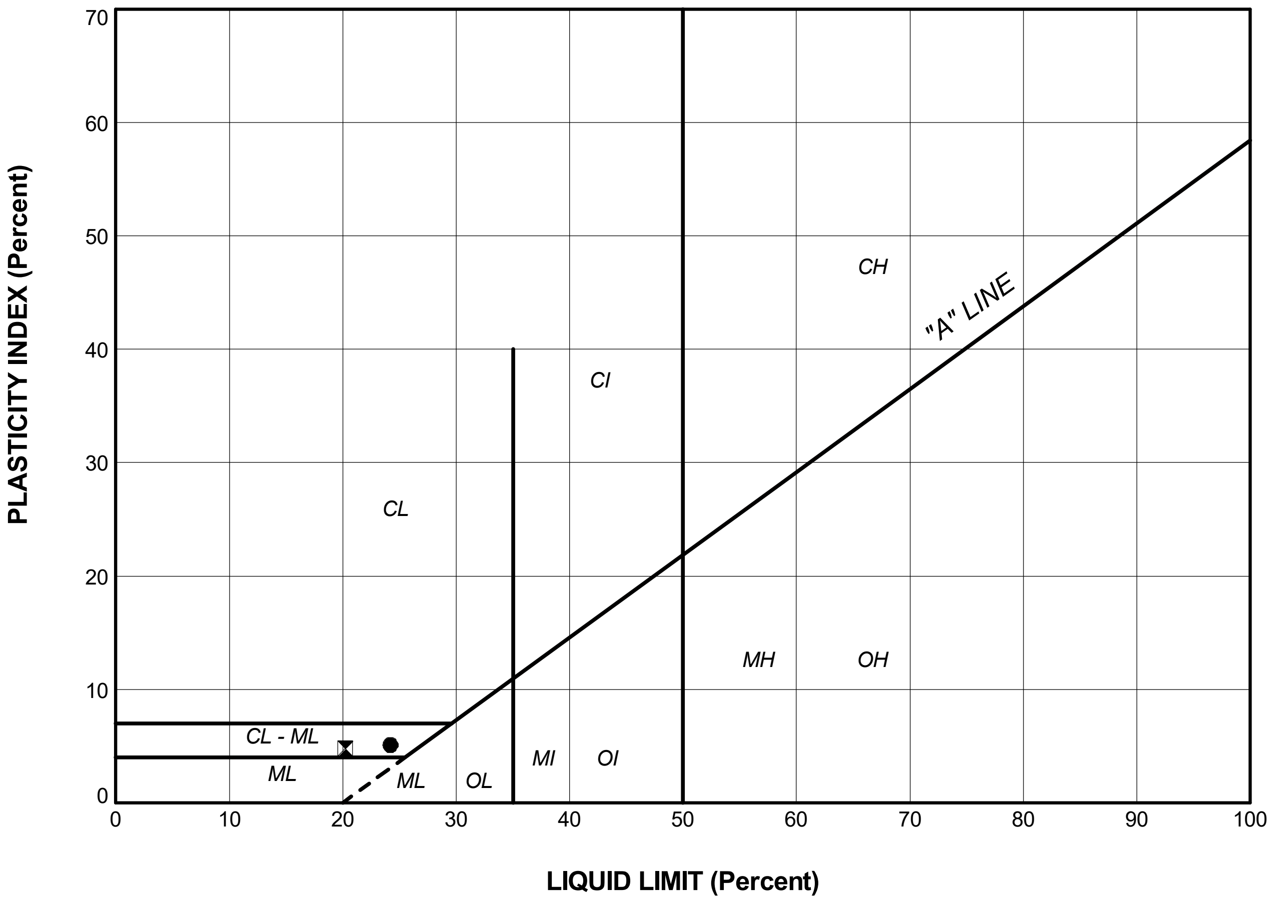


LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	RL-3	3	235.1
⧗	RL-3	5	232.3
▲	RL-3	7	229.2

PROJECT	HIGHWAY 69 DEEP FILL CULVERT				
TITLE	**GRAIN SIZE DISTRIBUTION** Silt				
GOLDER SUDBURY, ONTARIO	PROJECT No.		1790361	FILE No.	1790361.GPJ
	DRAWN	TR	Sep 2019	SCALE N/A	REV.
	CHECK	AB	Sep 2019	**FIGURE B-4**	
	APPR	KB	Sep 2019		

SUD-MTO GSD GLDR_LDN.GDT



SOIL TYPE
C = Clay
M = Silt
O = Organic

PLASTICITY
L = Low
I = Intermediate
H = High

LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	RL-3	5	24.2	19.1	5.1
⧗	RL-3	7	20.2	15.4	4.8

PROJECT
HIGHWAY 69
DEEP FILL CULVERT

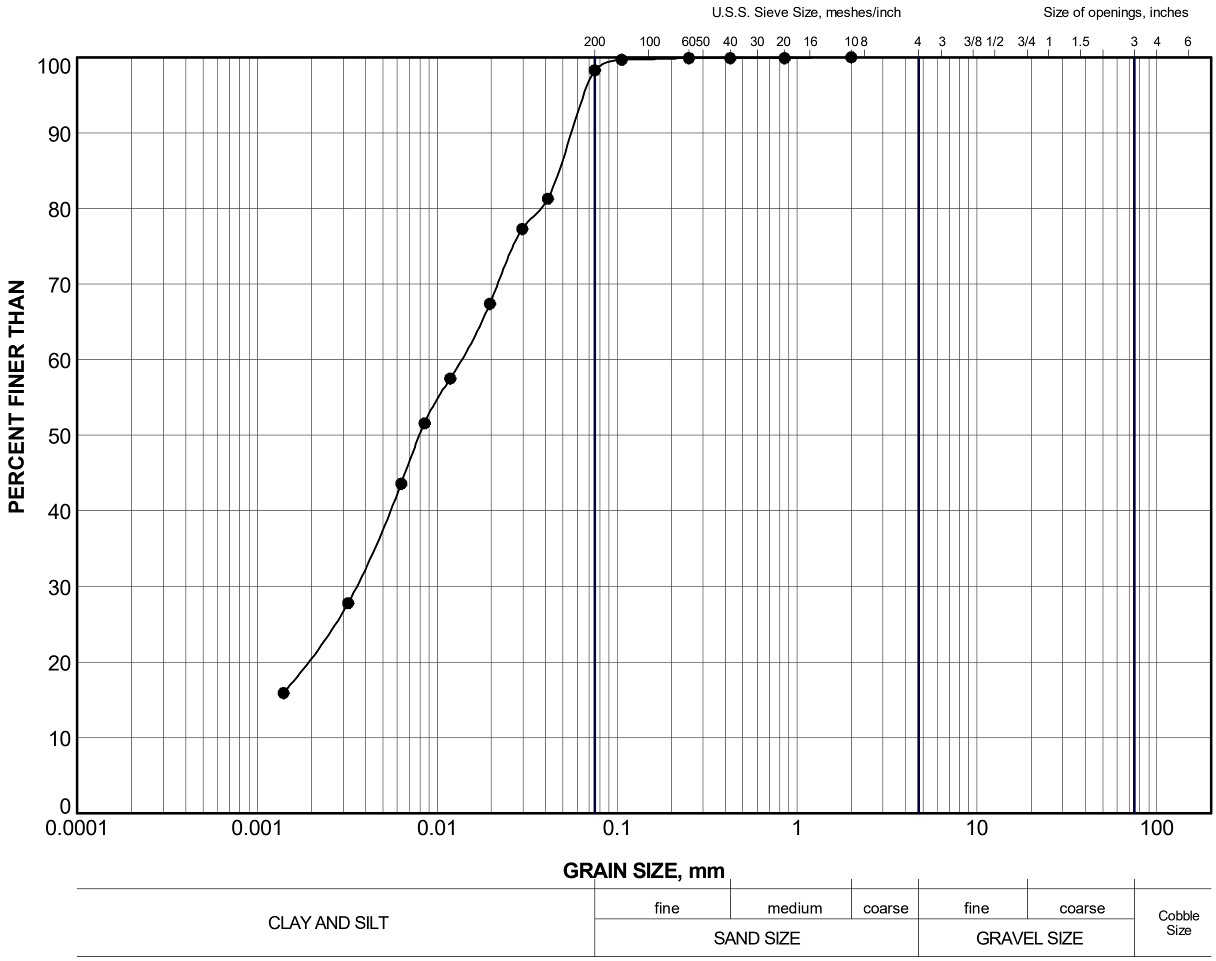
TITLE
PLASTICITY CHART
Silt

GOLDER
SUDBURY, ONTARIO

PROJECT No.		1790361	FILE No.	1790361.GPJ
DRAWN	TR	Sep 2019	SCALE N/A	REV.
CHECK	AB	Sep 2019		
APPR	KB	Sep 2019		

FIGURE B-5

SUD-MTO PL GLDR_LDN.GDT



LEGEND

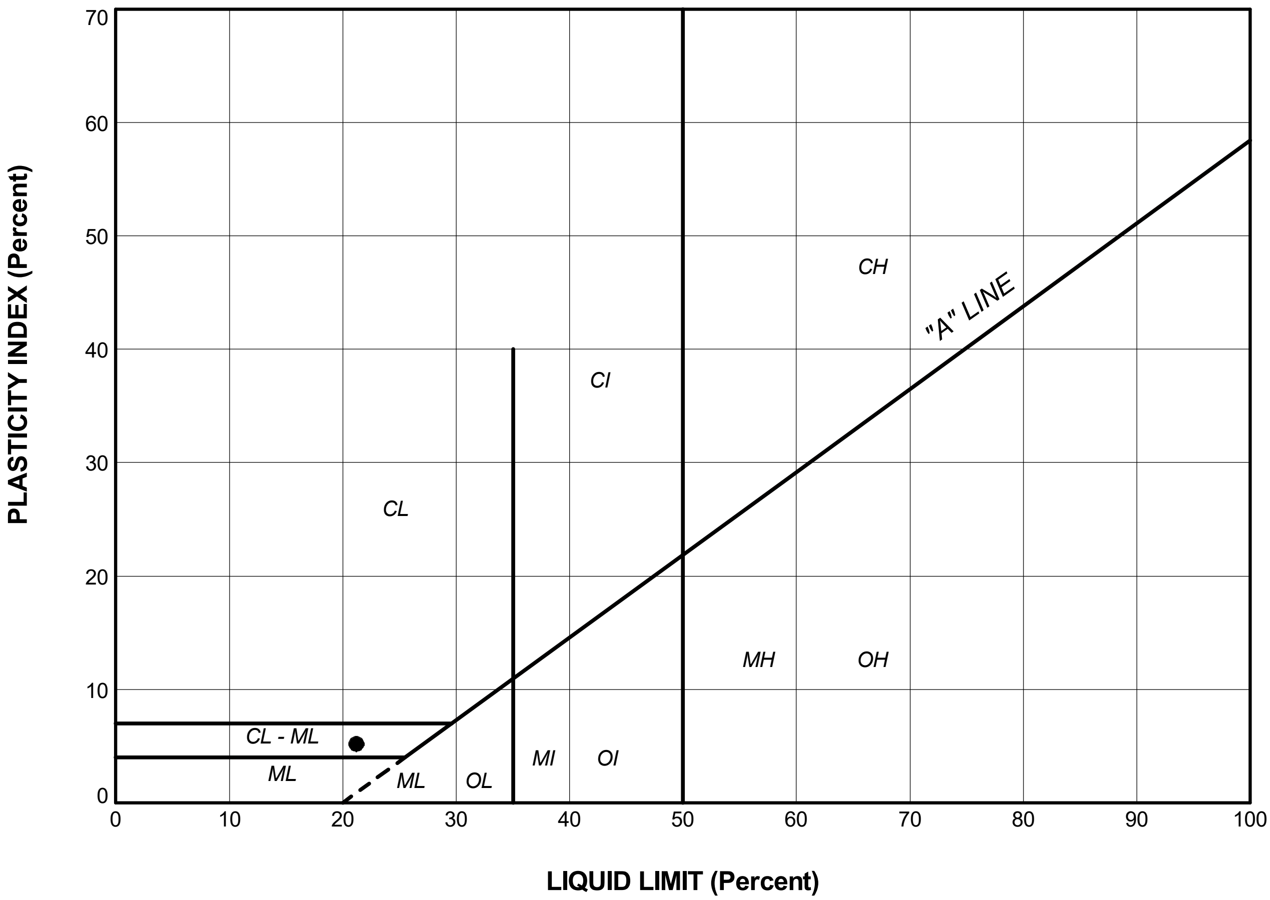
SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	RL-3	10	225.3

PROJECT	HIGHWAY 69 DEEP FILL CULVERT
TITLE	**GRAIN SIZE DISTRIBUTION** Clayey Silt

GOLDER
SUDBURY, ONTARIO

PROJECT No.		1790361	FILE No.	1790361.GPJ
DRAWN	TR	Sep 2019	SCALE N/A	REV.
CHECK	AB	Sep 2019	**FIGURE B-6**	
APPR	KB	Sep 2019		

SUD-MTO GSD GLDR_LDN.GDT



SOIL TYPE
C = Clay
M = Silt
O = Organic

PLASTICITY
L = Low
I = Intermediate
H = High

LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	RL-3	10	21.2	16.0	5.2

PROJECT
HIGHWAY 69
DEEP FILL CULVERT

TITLE
PLASTICITY CHART
Clayey Silt

GOLDER
SUDBURY, ONTARIO

PROJECT No.		1790361	FILE No. 1790361.GPJ	
DRAWN	TR	Sep 2019	SCALE N/A	REV.
CHECK	AB	Sep 2019	**FIGURE B-7**	
APPR	KB	Sep 2019		

SUD-MTO PL GLDR_LDN.GDT

RESULTS OF ANALYSES OF SOIL

Maxxam ID		JGE906			JGE906		
Sampling Date		2019/03/18 15:00			2019/03/18 15:00		
COC Number		127608			127608		
	UNITS	**RL-3 SA1**	**RDL**	**QC Batch**	**RL-3 SA1 Lab-Dup**	**RDL**	**QC Batch**
CONVENTIONALS							
Sulphide	ug/g	<0.50	0.50	6062227			
Calculated Parameters							
Resistivity	ohm-cm	2000		6035108			
Inorganics							
Soluble (20:1) Chloride (Cl-)	ug/g	240	20	6036711	230	20	6036711
Conductivity	umho/cm	506	2	6037167	496	2	6037167
Available (CaCl2) pH	pH	7.69		6036826			
Soluble (20:1) Sulphate (SO4)	ug/g	<20	20	6036712	<20	20	6036712
Physical Testing							
Moisture-Subcontracted	%	20	0.30	6062226			

RDL = Reportable Detection Limit
QC Batch = Quality Control Batch
Lab-Dup = Laboratory Initiated Duplicate

APPENDIX C

Notice to Contractor/ Operational Constraint/ Special Provision

OBSTRUCTIONS – Item No.

Notice to Contractor

The contactor shall be alerted to the presence of cobbles and boulders within the fill along the alignment of the culvert at Highway 69, Station 22+350, Township of Dill. Consideration of the presence of these obstructions must be made in the selection of appropriate equipment and procedures for open cut excavations and installation of temporary protection systems if required.

BACKFILLING EMBANKMENT – Item No.

Notice to Contractor

The existing clayey portion of the fill to be excavated from the embankment and all native subgrade soils that may be sub-excavated from the culvert area at this site are to be stockpiled separately from the excavated existing embankment fill comprised of sand and gravel, silty sand, and blast rock fill, which may be used for embankment reconstruction. The existing clayey fill excavated from the embankment and the native subgrade soils comprised of peat and silt that may be sub-excavated from the culvert area shall not be re-used as backfill to the culvert nor used for embankment reconstruction.

TEMPORARY EXCAVATION – Item No.

Operational Constraint

The Contractor shall be alerted to the following requirements/constraints related to the temporary excavation of the embankment side-slope for the culvert extension at Highway 69, Station 22+350, Township of Dill:

- The most southerly/outside eastbound lane and shoulder of Highway 69 shall be closed during and while the excavation is open.
- The length of time that the excavation can be left open shall be limited to a maximum of 3 days and not over the weekend. If a longer period is required, the temporary excavated slopes must be protected from erosion during rain events and surface water directed away from crest of embankment.
- The 1H:1V excavated side slopes shall be limited to no wider than 3 m directly above the culvert extension replacement location.
- Construction equipment and vehicular traffic shall be not positioned closer than 5 m from the crest of the temporary excavation.
- Stockpiles shall not be placed on the highway embankment where the embankment side slope (temporary or permanent) is sloped steeper than 2H:1V.

DEWATERING SYSTEM - Item No.
TEMPORARY FLOW PASSAGE SYSTEM - Item No.

Special Provision No. 517F01 July 2017

Amendment to OPSS 517, November 2016

Design Storm Return Period and Preconstruction Survey Distance

517.01 SCOPE

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

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

517.04 DESIGN AND SUBMISSION REQUIREMENTS

517.04.01 Design Requirements

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

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

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

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

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

<table>
<caption>Table A</caption>
<tr><td>IDF Curve Location</td><td colspan="3">Latitude: 46.429167</td><td colspan="3">Longitude: 80.937500</td></tr>
<tr><td colspan="7">Temporary Flow Passage Systems</td></tr>
<tr><td rowspan="2">Site Name /
Station Reference</td><td rowspan="2">Minimum
Return Period
(Years)</td><td colspan="4">Return Period Flow Estimates (m^3/s)</td><td rowspan="2">Design Engineer
Requirements
(Note 1)</td></tr>
<tr><td>2
Year</td><td>5
Year</td><td>10
Year</td><td>25
Year</td></tr>
<tr><td>Culvert Extension Replacement
Dill Township, Sta. 22+350</td><td>2</td><td>1.07</td><td>1.43</td><td>1.67</td><td>2.17</td><td>Yes</td></tr>
<tr><td colspan="7">Dewatering Systems</td></tr>
<tr><td>Site Name /
Station Reference</td><td colspan="5">Preconstruction Survey Distance (Note 2)
(m)</td><td>Design Engineer
Requirements
(Note 1)</td></tr>
<tr><td>Culvert Extension Replacement
Dill Township, Sta. 22+350</td><td colspan="5">100</td><td>Yes</td></tr>
<tr><td colspan="7">Note:
1. “Yes” means the design Engineer and design-checking Engineer shall have a minimum of 5 years of experience in designing systems of similar nature and scope to the required work. “No” means a minimum experience level is not required for the design Engineer and design-checking Engineer.
2. “N/A” indicates a preconstruction survey is not required.</td></tr>
</table>

NOTES TO DESIGNER:

Designer Fill-in for Table A:

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

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

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

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

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

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

Table A (Sample)

IDF Curve Location	Latitude: 44.974844				Longitude: -79.769339	
Temporary Flow Passage Systems						
Site Name / Station Reference	Minimum Return Period (Years)	Return Period Flow Estimates (m^3/s)				Design Engineer Requirements (Note 1)
		2 Year	5 Year	10 Year	25 Year	
Woods Creek Culvert Rehabilitation	2	0.7	3.5	7.5	10.9	N/A
Site 32-145 Robbs Creek Culvert Replacement	10	1.6	7.6	17.4	25.2	Yes
Dewatering Systems						
Site Name / Station Reference	Preconstruction Survey Distance (Note 2) (m)					Design Engineer Requirements (Note 1)
Site 32-145 Robbs Creek Culvert Replacement	300					Yes

Note:

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

WARRANT: Always with these tender items.

GOLDER

golder.com