FOUNDATION INVESTIGATION AND DESIGN REPORT

Highway 101, Culvert at Station 12+820, Township of Beatty
Ministry of Transportation, Ontario
GWP 5217-13-00

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PART A

FOUNDATION INVESTIGATION REPORT

HIGHWAY 101, CULVERT AT STA 12+820, TOWNSHIP OF BEATTY

MINISTRY OF TRANSPORTATION, ONTARIO

GWP 5217-13-00

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by D.M. Wills Associates Limited (DM Wills) on behalf of the Ministry of Transportation, Ontario (MTO), to provide foundation engineering services related to the culvert on Highway 101 at Station 12+820, in the Township of Beatty, approximately 6 km east of Highway 11. The Key Plan of the general location of this section of Highway 101 and the location of the investigated area are shown on Drawing 1.

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

2.0 SITE DESCRIPTION

The existing culvert consists of an approximate 1.2 m diameter, 61 m long Corrugated Steel Pipe. Based on DM Will’s survey, the culvert inlet (north end) and outlet (south end) inverts are at approximately Elevations 263.7 m and 262.5 m, respectively. The highway grade at the culvert location is at approximately Elevation 270.9 m. In general, the topography within the vicinity of the culvert consists of generally flat farmland with some hilly terrain.

Based on the survey provided by DM Wills, the embankment slopes at the culvert location are generally inclined at 2.5 Horizontal and 1 Vertical (2.5H:1V). At the time of the subsurface exploration field work, the embankment side slopes were generally grass covered. No signs of deep-seated embankment slope instability were observed in the vicinity of the culvert. Along the south embankment slope in the vicinity of the culvert, rock fragments (cobble and boulder sized) were observed at ground surface mixed with vegetation, which we understand (from discussions with MTO) were placed to remediate a previous surficial slope failure at this location (see Drawing 1). The ground surface conditions at select locations of the culvert area are shown on Photographs 1 to 4.

3.0 INVESTIGATION PROCEDURES

Field work for this subsurface exploration was carried out between June 2 and 30, 2020, during which time seven boreholes (Boreholes C12-1 to C12-7) were advanced at the approximate locations shown on Drawing 1. Boreholes C12-1 to C12-6 were advanced using a track mounted CME-55 drilling rig supplied and operated by Landcore Drilling of Sudbury, Ontario, and Borehole C12-7 was advanced using portable drilling equipment supplied and operated by Landcore Drilling. Traffic control, where required, was performed in accordance with MTO’s Ontario Traffic Control Manual Book 7 – Temporary Conditions.

The boreholes were advanced using 108 mm I.D. Hollow Stem Augers or NW casing and wash boring. 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 or cathead hammer in accordance with the Standard Penetration Test (SPT) procedures (ASTM D1586). Select samples of the cohesive soils were obtained using 76 mm O.D. thin-walled Shelby Tubes (ASTM D1587). In-situ vane shear tests were carried out in cohesive soils for determination of undrained shear strengths in accordance with Standard Test Method for Field Vane Shear Test in Saturated Fine Grained Soils (ASTM 2573), using an MTO standard ‘N’-size vane.

The water level inside the augers was observed during and upon completion of drilling operations and a standpipe piezometer was installed in Borehole C12-4 to permit monitoring of the groundwater level. The piezometer consisted of a 50 mm diameter polyvinyl chloride (PVC) pipe, with a slotted screen, sealed within a sand filter pack at a selected depth interval within the borehole. Above the sand filter pack and piezometer screen, the annulus surrounding the piezometer pipe was backfilled with bentonite to create a seal above the screen. The piezometer installation details, and water level readings are indicated on the borehole records contained in Appendix A. The boreholes and piezometer were backfilled in accordance with Ontario Regulation 903 (as amended). The boreholes drilled through the roadway were capped at the roadway 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 samples. The soil samples were identified in the field, placed in labelled containers, and transported to Golder's geotechnical laboratory in Sudbury for further examination and laboratory testing. Index and classification testing consisting of water content determinations, grain size distributions, Atterberg limits, and consolidation (oedometer) was carried out on selected soil samples. The geotechnical laboratory testing was completed according to ASTM and MTO LS standards, as applicable. In addition, one soil sample was submitted to Bureau Veritas Laboratories in Sudbury, Ontario, an accredited analytical laboratory, for testing of a suite of corrosivity indicator parameters.

The as-drilled borehole locations were measured relative to the highway chainage/station marked on the pavement or relative to the end of culvert by a member of our technical staff and converted into northing/easting coordinates on the plan drawing. The ground surface elevation at each borehole location was surveyed by Golder, relative to the highway centreline or culvert end, with the elevations provided by DM Wills. The northing and easting 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. The latitude/longitude coordinates of the borehole locations are also shown on the borehole records.

Borehole Number	MTM NAD 83 Northing (m)	MTM NAD 83 Easting (m)	Ground Surface Elevation (m)	Borehole Depth (m)
C12-1	5377901.3	350443.8	270.8	20.4
C12-2	5377901.1	350458.8	270.9	15.9
C12-3	5377891.9	350430.8	270.7	15.9
C12-4	5377924.7	350463.6	265.6	10.5
C12-5	5377922.7	350472.6	264.5	10.4
C12-6	5377874.3	350432.8	264.3	11.3
C12-7	5377877.0	350424.2	264.8	2.9 (DCPT to 9.1)

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 located within a glaciolacustrine plain, with the soils consisting primarily of clay.

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 plotted results of geotechnical laboratory testing are contained in Appendix B. The results of the in-situ field tests (i.e., SPT ‘N’-values and in-situ (field) vane undrained shear strengths), 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 results of the analytical laboratory testing by Bureau Veritas Laboratories (BVL) are summarized in Section 4.4.

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.

4.2.1 Fill

A 2.4 m to 5.6 m thick layer of sand to sand and gravel (fill) was encountered at ground surface in the roadway boreholes (Boreholes C12-1 to C12-3) between Elevations 270.9 m and 270.7 m. Auger grinding was encountered in Borehole C12-1 from surface to 2.1 m depth and throughout the fill in Borehole C12-3. In Borehole C12-2, split-spoon refusal was encountered on an inferred cobble at 1.1 m depth and on an inferred boulder at 3.8 m depth; also, in this borehole, the augers deflected out of vertical alignment due to the cobbles/boulders within the fill and was abandoned at a depth of 3.8 m. Borehole C12-2 was continued by advancing NW casing adjacent to the original borehole location and soil sampling operations were resumed below a depth of 4.6 m.

Below the sand to sand and gravel (fill) in Boreholes C12-1 to C12-3, a 2.9 m to 3.2 m thick layer of cohesive fill was encountered, consisting of silt to clayey silt, silty clay or clay. This cohesive fill contained trace organics and trace wood fragments.

In Borehole C12-6, a 0.1 m thick layer of rock fill (cobble to boulder sized in vicinity of borehole) was encountered at ground surface and was underlain by 0.7 m of sand fill. In Borehole C12-7, below the topsoil, a 2.7 m thick layer

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[1] Ontario Ministry of Natural Resources and Forestry. Northern Ontario Engineering Geology Terrain Study. Ontario Geological Society Electronic Mapping. Map 42ANE.

of sand to gravelly sand (fill) was encountered to the termination of the borehole at a depth of 2.9 m. A DCPT was driven from the bottom of the borehole and was terminated at a depth of 9.1 m below ground surface.

The SPT ‘N’-values measured within the sand to sand and gravel (fill) typically range from 13 blows to greater than 100 blows per 0.3 m of penetration, indicating a compact to very dense compactness condition. One SPT ‘N” value of 4 was measured in the sand fill (below the rock fill) in Borehole C12-6. The SPT ‘N’-values measured within the cohesive fill range from 8 blows to 22 blows per 0.3 m of penetration, suggesting a stiff to very stiff consistency.

Grain size distribution testing was carried out on four samples of the sand to sand and gravel (fill) and the results are presented on Figure B-1 in Appendix B. In summary, the fill samples contained 1-38% gravel, 51-92% sand, and 7-13% fines. The natural moisture content measured on samples of the sand to sand and gravel (fill) range from about 2% to 5%.

Atterberg limit testing carried out on two samples of the cohesive silty clay to clay fill measured a liquid limit of 40 and 53, plastic limit of 13 and 21, and corresponding plasticity index of 26 and 32. The test results, which are plotted on Figure B-2, indicate that the cohesive fill consists of silty clay of intermediate plasticity to clay of high plasticity. The natural moisture content measured on two samples of the cohesive fill are about 26% and 55%.

4.2.2 Peat/Topsoil

An approximately 1 m thick deposit of amorphous peat was encountered below the fill in Borehole C12-1 at Elevation 264.6 m.

A 150 mm to 700 mm thick layer of topsoil was encountered at ground surface in Boreholes C12-4, C12-5, and C12-7. In Borehole C12-5, wood pieces were encountered at a depth of 0.7 m. Materials designated as topsoil were classified solely based on visual and textural evidence. Testing of organic content, or for other nutrients, was not carried out and therefore, the use of materials classified as topsoil cannot be relied upon for support and growth of landscaping vegetation.

The SPT ‘N’-value measured within the peat was 13 blows per 0.3 m of penetration, suggesting a stiff consistency. The SPT ‘N’-value measured within the topsoil ranged from 0 blows (weight of hammer) to 5 blows per 0.3 m of penetration indicating a very loose to loose compactness condition.

4.2.3 Clayey Silt

A 1.4 m thick deposit of clayey silt was encountered below the fill in Borehole C12-6 at Elevation 263.5 m. Trace organics and trace wood pieces were encountered in the samples of this deposit.

The SPT ‘N’-value measured within the clayey silt deposit was 4 and 8 blows for 0.3 m of penetration, suggesting a firm consistency.

4.2.4 Sandy Silt

A 0.8 m thick deposit of sandy silt was encountered below the topsoil in Borehole C12-5. Wood pieces were encountered at the bottom of the deposit at 1.5 m depth.

The SPT ‘N’-value measured within the sandy silt was 0 blows (weight of hammer) for 0.3 m of penetration, indicating a very loose compactness condition.

4.2.5 Silty Clay to Clay

A cohesive deposit of silty clay to clay was encountered in Boreholes C12-1 to C12-6 and the boreholes were terminated in the deposit after exploring for 7.2 m to 13.2 m. Silt to clayey silt seams, layers, and laminations were generally encountered throughout the deposit. Shelby tubes obtained from Boreholes C12-5 and C12-6 were extruded and silt to clayey silt zones were measured to be 10 mm to 40 mm thick between silty clay to clay zones measured to be 10 mm to 75 mm thick. As a result, portions of the deposit are considered to be varved, although a regular repeating pattern of clay and silt was not evident within many of the samples collected.

The SPT ‘N’-values measured within the silty clay to clay range from 0 blows (weight of hammer) to 10 blows per 0.3 m of penetration. In-situ field vane tests carried out within the deposit measured undrained shear strengths ranging from about 25 kPa to 60 kPa and sensitivity ranging from about 2 to 13 (but typically 2 to 6). The SPN “N”-values, together with the field vane test results, suggest that the deposit generally has a firm consistency.

Atterberg limit testing was carried out on 11 samples of the combined silty clay to clay and silt to clayey silt portions of the deposit and two samples of the clay layer from the Shelby tubes (C12-5/7A and C12-6/6A); the test results measure a liquid limit ranging from 37 to 67, plastic limit ranging from 16 to 23 and plasticity index ranging from 21 to 44, which are plotted on Figure B-3 and indicate that the combined soil deposit and clay layers from the Shelby tubes consist of silty clay of intermediate plasticity to clay of high plasticity. An Atterberg limit test was also carried out on two samples of the clayey silt interlayers within the deposit from the Shelby tubes (C12-5/7B and C12-6/6B) and measured a liquid limit of 29 and 33, plastic limit of 17 and 18, and plasticity index of 11 and 15, which are plotted on Figure B-4 and indicate a clayey silt of low plasticity. The natural moisture content measured on the samples of the deposit range between about 30% and 60%.

One laboratory consolidation (oedometer) test was carried out on a sample of the silty clay to clay obtained from Borehole C12-6. The results of the consolidation test are provided on Figure B-5 in Appendix B and are summarized below.

Borehole / Sample No.	Sample Elevation (m)	w_n (%)	γ (kN/m^3)	σ_{vo}' (kPa)	σ_p' (kPa)	OCR	e_o	C_c	C_r	c_v (cm^2/s)
C12-6 / Sample 6	259.3	58	16.4	44	135	3.1	1.58	0.73	0.04	0.005

Notes:

Coefficient of consolidation value given for effective stress ranging from about 30 kPa to 80 kPa (representative of effective overburden pressure for current conditions).

Where:	
w_n	Natural Moisture content (%)
γ	Unit weight (kN/m^3)
σ_{vo}'	Effective overburden pressure (kPa)
σ_p'	Preconsolidation pressure (kPa)
OCR	Overconsolidation Ratio
e_o	Initial void ratio
C_c	Compression index
C_r	Recompression index
c_v	Coefficient of consolidation in the normally consolidated range (cm^2/s)

4.3 Groundwater Conditions

The unstabilized groundwater levels relative to ground surface measured inside the open boreholes upon completion of drilling are summarized below. The groundwater levels measured in the piezometer installed in Borehole C12-4 are also summarized below. Groundwater and creek water levels in the area are subject to seasonal fluctuations and variations due to precipitation events.

Borehole No.	Depth to Groundwater Level (m)	Approximate Groundwater Elevation (m)	Notes
C12-1	Dry	-	Open borehole (unstabilized)
C12-2	Dry	-	Open borehole (unstabilized)
C12-3	Dry	-	Open borehole (unstabilized)
C12-4	Dry 0 - 0.3*	- 265.6 265.9	Open borehole (unstabilized) Piezometer (June 6, 2020) Piezometer (June 10, 2020)
C12-5	Dry	-	Open borehole (unstabilized)
C12-6	Dry	-	Open borehole (unstabilized)
C12-7	0	264.8	Open borehole (unstabilized)

*Artesian condition (measured above ground surface)

4.4 Analytical Laboratory Testing Results

Analytical testing was carried out on a sample of the clay recovered from Borehole C12-5. The soil sample was submitted to Bureau Veritas Laboratories 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)	Chloride (Cl) Content (µg/g)	Sulphide (mg/kg)	pH
C12-5	3	1.5-2.1	3,300	306	<20[1]	83	<0.5 (1)	7.80

(1) The sulphate and sulphide concentrations are below the reportable detection limit of 20 µg/g and 0.5 mg/kg, respectively.

5.0 CLOSURE

The field drilling program was carried out under the supervision of Mr. Tibor Berecz EIT, under the overall direction of Mr. André Bom, P.Eng. This Foundation Investigation Report was prepared by Mr. Andre Bom, P.Eng. Mr. Kevin Bentley, P.Eng., an MTO Foundations Designated Contact and Associate with Golder, conducted an independent quality control review of this report.

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PART B

FOUNDATION DESIGN REPORT

HIGHWAY 101, CULVERT AT STA 12+820, TOWNSHIP OF BEATTY

MINISTRY OF TRANSPORTATION, ONTARIO

GWP 5217-13-00

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides a discussion on stability and settlement with regards to the existing embankment geometry. We understand that the culvert on Highway 101 at Station 12+820, in the Township of Beatty, was initially planned to be replaced as part of this assignment; however, D.M. Wills has indicated that a culvert replacement is no longer required. The discussion and recommendations presented are intended to provide the designer with sufficient information to make informed decisions related to the short and long-term stability of the existing embankment. These recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the current subsurface exploration. 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 General

The existing culvert consists of a 1.2 m diameter, 61 m long Corrugated Steel Pipe. Based on the survey provided by D.M. Wills via email on July 10, 2020, and our site observations during the foundation exploration work, the existing culvert crosses the existing Highway 101 embankment on a skew. The height of the embankment based on the survey is 7.2 m relative to the invert at the inlet (north end) and 8.4 m relative to the invert at the outlet (south end).

Based on information provided to Golder by MTO on June 30, 2020, consisting of a memorandum by MTO dated June 2, 2003, and site photographs, we understand that during previous construction activity, there was a previous slope failure and remediation carried out at the site as summarized below:

1) An approximately 1.5 m grade raise was constructed in the early 2000s.
2) A slope failure and sudden settlement occurred in 2003, on the south side of the embankment in the vicinity of the culvert, at which point, MTO carried out a site visit and prepared recommendations for rehabilitation of the embankment side slopes that included installing geotextile and backfilling with rip rap.
3) From photographs during MTO’s 2003 site visit, the grade raise appeared to consist of a uniform fine sand.
4) Based on MTO’s 2003 memorandum, the cause of the slope failure/sudden settlement at the site was attributed to piping around the culvert (“water flowing through the embankment fill, adjacent to the culvert”), which washed away finer material.
5) It was noted that the condition of the culvert was not impacted by the ‘settlement’.
6) Between Station 12+425 to 12+450 (about 400 m west of the culvert), after a grade lowering of the embankment at this location, a slope failure of the cut on the south side of the highway was observed, extending approximately 500 mm into the underlying clay.

At the time of Golder's 2020 subsurface exploration field work, the embankment side slopes were grass covered with coarser cobble to boulder sized fill evident on the ground surface at the approximate location of the 2003 south slope failure (shown on Drawing 1), consistent with the recommendations for rehabilitation outlined in MTO's June 2, 2003, memorandum.

We are not aware of any additional failures that have occurred at the site since 2003, and D.M. Wills have confirmed that there are no proposed changes to the existing embankment (i.e., no grade raise/lowering and/or widening) in the area of the culvert, except potentially a minor grade raise for an extra lift of pavement.

6.2 Consequence and Site Understanding Classification

In accordance with Section 6.5 of the *Canadian Highway Bridge Design Code* (CHBDC, 2019) and its *Commentary*, we understand that the culvert along Highway 101 at Station 12+820 in the Township of Beatty is expected to carry medium traffic volumes and the performance will have potential impacts on other transportation corridors; hence, the culvert foundation system is classified as having a "typical consequence level" associated with exceeding limits states design. Accordingly, the appropriate corresponding ultimate limit state (ULS) and serviceability limit state (SLS) consequence factor, Ψ, from Table 6.1 of the *CHBDC* has been used for design, as applicable. Given the 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* (2019), the level of confidence for design is considered to be a "typical degree of site and prediction model understanding". Therefore, the corresponding ultimate limit state (ULS) and serviceability limit state (SLS) geotechnical resistance factors, ϕ_{gu} and ϕ_{gs}, from Table 6.2 of the *CHBDC* would generally be used for design, as applicable.

Based on discussions with MTO Foundations in July 2020, relating to preliminary stability analyses, MTO Foundations provided approval to use the geotechnical resistance factor exceptions outlined in Table 1 of the Provincial Engineering Memorandum titled "*Materials Engineering and Research Office (MERO) # 2020-01"* (MERO#2020-1), dated March 23, 2020, for this project.

6.3 Embankment Stability

Based on our site observations at the time of the field investigation, the existing embankment in the area of the culvert appears to be performing satisfactorily. There was no visual evidence of instability (i.e., soil movement) on the embankment side slopes, nor tilted guide rails, nor tension cracks near the embankment crest that would be indicative of instability. As shown on the survey provided by D.M. Wills, the existing embankment side slopes are inclined at about 2.5H:1V.

6.3.1 Methodology

Limit equilibrium slope stability analysis was carried out for the existing (and proposed) highway embankment (at the location of the culvert) using the commercially available program GeoStudio 2019 (Version 10.0.2.18035), 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, and in the context of the CHBDC (2019), the target FoS is defined as being equal to the inverse of the product of the consequence factor, Ψ, and the geotechnical resistance factor, Φ_{gu} (i.e., FoS = 1 / (Ψ * Φ_{gu})). In particular, for the assessment of the embankment slopes at this culvert location, a "Typical consequence level" from the CHBDC and a "Typical degree of site and prediction model understanding" from MERO #2020-01 were utilized, corresponding to a target minimum FoS of 1.25 and 1.43 considering global stability for temporary (short-term) and permanent (long-term) conditions, respectively.

The stability analyses have been carried out based on the existing embankment geometry provided by D.M. Wills and based on the subsurface conditions as encountered in Boreholes C12-1 and C12-6.

6.3.2 Parameter Selection

For the cohesive soils, including the silty clay embankment fill and native clayey silt-clay deposit, total stress parameters were employed for analysing the short-term, undrained condition. The total stress parameters for the cohesive soils have been based on the results of the in-situ field vane test data (in the native soils) and based on the results of the SPT testing (in the fill soils). The uncorrected field vane measurements are shown over the depth of the deposit in Figure 1. As per ASTM D2573, the in-situ measured values from the in-situ vane shear testing were corrected for mobilized shear strength to be utilized in stability analyses with the following equation:

$$s_{u(mob)} = \mu \times s_{u(fv)}$$

where: $s_{u(fv)}$ = uncorrected measured in-situ shear vane test

$s_{u(mob)}$ = mobilized shear strength value

μ = vane correction factor

The corrections proposed in the ASTM were developed for homogeneous clays and provide an average mobilized undrained shear strength to be applied over the entirety of the modeled slip surface. As silty clay-clay soils with interlayers of silt to clayey silt (i.e., resembling varved clays) were encountered at this site, the corrections identified in ASTM D2573 may not be applicable. In particular, it is evident from various results in literature, including laboratory testing identified in New Liskeard clays (Lacasse et al., 1977), that the lowest shear strength is mobilized along the near horizontal portions of the slip surface (i.e., along the weaker varve laminae). Therefore, for the stability modeling, a maximum vane correction factor ($\mu_{max} = 0.64$) was developed based on literature information from varved clay sites and was applied only to the near horizontal portions of the modeled slip surfaces (i.e., where the slip surface inclination is equal to or less than 15°). For inclination angles greater than 30°, the design undrained shear strength was uncorrected ($\mu = 1$) and the correction factor was interpolated between these values for inclination angles between 15° and 30° accordingly. A summary of the anisotropic behavior modeled for the undrained shear strength, using the correction factor is shown in Figure 2.

As a change to the embankment geometry is not required for this site (i.e., no widening or no grade raise), the long-term, drained condition, effective stress parameters (i.e., c' and ϕ') were assigned to the cohesive soils using the same geometry as the existing embankment. The selection of these parameters for the analysis considered empirical correlations with index properties (Mitchell, 1993) and the results of laboratory triaxial testing carried out on clayey soils from a previous project in the Township of Beatty (Geocres No. 42A00-052), as shown in the table below.

Reference	Plasticity Index	OCR	Effective Cohesion (c')	Effective Friction Angle (°)
Mitchell, 1993	29 (1)	1	0	27
Geocres No. 42A00-052	33	~3.5 (2)	8	31

(1) Average of Atterberg Limits testing results completed as part of the current investigation.

(2) Estimated from preconsolidation pressure correlated from in-situ shear vane testing carried out adjacent to the tested triaxial specimens.

Given that the correlated shear strength parameters and those obtained from nearby sites represent cross shear values (i.e., failures across both silty and clayey laminae), a correction factor was applied to the drained parameters (i.e., c' and ϕ') for the varved clay stratigraphy in the stability model, using a similar approach as outlined above, for the total stress parameters.

For the non-cohesive soils, including the granular embankment fill, effective stress parameters were employed for analysing the long-term drained conditions. The effective stress parameters (i.e., c' and ϕ') were based on precedent experience in similar soils.

Summarized below are the simplified stratigraphy and the associated short-term and long-term soil uncorrected parameters and unit weights employed for the different soil types in the area of the culvert.

Soil Deposit	Bulk Unit Weight (kN/m^3)	Short-Term Condition		Long-Term Condition	
		Effective Friction Angle (°)	Undrained Shear Strength (kPa)	Effective Friction Angle (°)	Effective Cohesion (c') (kPa)
Existing Granular Fill (compact to very dense)	20	35	-	35	0
Existing Clayey Silt to Clay Fill (stiff to very stiff)	19	-	75	-	75
Peat	12	27	1	27	1
Clayey Silt to Silty Clay-Clay (including "varved" zones) (firm)	17	-	30 (1)	29 (2)	4 (2)

(1) The shear strength of the cohesive stratum was corrected to mobilized shear strength along the near horizontal portions of the slip surface.

(2) Effective friction angle and effective cohesion of the cohesive deposit has been estimated based on an average of a literature correlation (Mitchell, 1991) and triaxial test data from a similar cohesive soil at a site located in the Township of Beatty Township (Geocres No. 42A00-052).

6.3.3 Results of Analyses

The results of global stability analyses carried out on the existing embankment are summarized in the table below.

Factor of Safety – Existing Embankment			
North Side Slope		South Side Slope	
Short Term Condition	Long Term Condition	Short Term Condition	Long Term Condition
1.66	1.68	1.19 (see Figure 3A)	1.42 (see Figure 4)

Further to the request by MTO, an additional stability analysis using an alternative parameter selection method (i.e., a lower-bound design line of raw field vane measurements in lieu of mobilized shear strength) was carried out for comparison purposes for the south side slope (short-term condition). If a lower-bound raw undrained shear strength value of 25 kPa is used for design, the FoS against global instability is calculated to be 1.21, as shown in Figure 3B. Although not preferred for a higher risk option, if a more liberal raw undrained shear strength value of 30 kPa is used for design, the FoS against global instability is calculated to be 1.40.

Based on the results of the analyses, the south side slope has the lowest factor of safety for this embankment. The short-term stability analyses fall slightly below the acceptable FoS of 1.25 presented in the MERO#2020-1 guidelines, whereas the stability analysis for the long-term condition is approximately at the acceptable FoS of 1.43 (i.e., within the margin of error of the analysis). Based on our review of available information, no historical performance issues have been documented with regards to global (i.e., deep seated) stability of the side slopes of the embankment since 2003. The currently proposed rehabilitation along this portion of the highway is not anticipated to change the existing embankment geometry; therefore, it is unlikely that an undrained (i.e., short-term condition) would occur as a result of the construction without an external influence (e.g., temporary vibrations, unanticipated external loading, such as stockpiles, seismic event, extreme rainfall, etc.).

Given that the FoS for the short-term stability analysis (using the preferred parameter selection method) for the south slope does not meet the acceptable criteria specified in the MERO guidelines, consideration can be given to the following options:

- Accept risk (i.e., do nothing) – Based on the historical satisfactory performance of the side slope since 2003 and the fact that the embankment geometry will not change as a result of the proposed rehabilitation, the risk of future instability is considered low. MTO would need to accept the lower factor of safety or higher strength parameter selection method, as the highway owner and no additional site investigation (e.g., shear strength testing) and/or stability mitigation is required.
- Adopt a ‘Low consequence factor’ for the embankment at this location, which we understand from previous discussions with MTO Foundations on other projects, may require input/approval from the MTO Bridge Office. If a low consequence factor is adopted, the target factor of safety for short-term stability reduces to 1.16 and the short-term stability results meet the requirements of CHBDC (2019).
- Additional high complexity laboratory testing and refinement of soil parameters. With this approach, existing soil samples from the site can be used to carry out complex laboratory testing (e.g., triaxial tests, direct shear tests, direct simple shear test, etc.) to refine the geotechnical strength parameters. The high complexity

laboratory testing may support the use of a "high" degree of site and prediction model understanding geotechnical resistance factor, which would further reduce the minimum required target FoS and/or the laboratory results may confirm that higher soil strength parameters can be used. There is a risk that the laboratory results may indicate lower shear strength parameters should be used, that will lower the FoS.

- Development of slope stability mitigation measures based on the available information (or the enhanced laboratory program, if completed). Stability mitigation techniques could involve slope flattening (2.8H:1V side-slopes based on preliminary slope stability assessment), berms, grade lowering, geosynthetic reinforcement, or ground improvement techniques. Based on the culvert location relative to the embankment, slope flattening, or toe berms will result in long extensions to the culvert and/or realignment of the watercourse onto private property. If geosynthetic reinforcement is considered, significant excavation and reconstruction of the embankment and road surface would be required, and a proprietary designer would need to be retained to confirm the internal stability of the reinforced slope.

Further to the discussion with MTO Foundations and D.M. Wills on July 21, 2020, regarding preliminary options, we understand that the preferred approach is to accept the risk and consequences of the short-term condition of the embankment side slope in the vicinity of the culvert, with no additional laboratory testing or stability mitigation; however, MTO will need to confirm this option is acceptable. Based on the satisfactory performance of the embankment for the past 18 years and acceptable FoS for global slope stability in the long-term condition, the "do nothing" is considered to be the preferred alternative, provided regular inspections and maintenance of the culvert and surrounding embankment are performed.

6.4 Construction Considerations

6.4.1 Surficial Embankment Stability and Erosion Protection

As part of the highway rehabilitation in the vicinity of the culvert, we understand that any construction will be limited to pavement rehabilitation, with the possibility of some regrading of pavement subgrade. Based on the cohesive foundation soils at this site, it is recommended that the existing embankment geometry be maintained (i.e., no widening or grade raise) at all times.

Stockpiles on the highway/shoulder should be placed well away from the culvert area, in order to prevent surcharging the embankment and further reducing the Factor of Safety against global instability of the side slopes.

If the side slope vegetation is disturbed or excess surface water runoff is directed to the side slopes, the embankment may be susceptible to surficial instability, which could include localized sloughing and erosion. The existing vegetation on the slopes should be maintained as much as practical and/or reinstated if disturbed. Alternatively, gravel sheeting could be placed to reduce the potential for erosion.

For portions of the embankment that are disturbed and/or need to be re-constructed, granular fill should be used, followed by topsoil and seeding (for erosion control) following the construction specifications of OPSS 802 (*Topsoil*) and OPSS.PROV 804 (*Seed and Cover*).

MTO's June 2, 2003, memorandum attributed the sudden settlement observed at that time to "water flowing through the embankment fill, adjacent to the culvert, and washing out some of the fine-grained embankment material." We understand that no further "piping" or settlement due to erosion of finer materials has been observed since 2003. In order to reduce the risk of further settlements, consideration could be given to reducing the potential for seepage around the culvert (i.e., piping) by designing and constructing cut-off walls at the upstream and downstream side of the culvert. A hydraulics engineer would be required to determine an appropriate depth and width for a cut-off wall at this site. It should also be noted that the potential for piping at the culvert location is increased if water flow is obstructed through the culvert; therefore, regardless of whether a cut-off wall is constructed, maintaining the culvert clear of debris through ongoing maintenance will be critical to prevent further ground movements/settlement at this location.

7.0 CLOSURE

This Foundation Design report was prepared by Mr. Matthew Thibeault, P.Eng., a geotechnical engineer of Golder. Mr. Andre Bom, P.Eng., a senior geotechnical engineer and Associate, reviewed the report. 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.

Matt Thibeault

Matthew Thibeault, P.Eng.

Geotechnical Engineer

LICENSED PROFESSIONAL ENGINEER
Mar 2/21
A. J. K. BOM
100075715
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LICENSED PROFESSIONAL ENGINEER
K. J. BENTLEY
02/03/2021
PROVINCE OF ONTARIO

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MTO Foundations Designated Contact, Associate

AB/KJB/sm

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https://golderassociates.sharepoint.com/sites/111953/project files/6 deliverables/foundations/1. reporting/1-c12/final/19126505-r-reva-culvert 12 hwy 101 fidr final ___2021.docx

REFERENCES

Golder Associates Ltd. 1999. Foundation Investigation and Design, Highway 101 Re-alignment, W.P. 258-96-00, Geocres No. 42A00-052. Golder Project No. 991-1145.

Lacasse, M.S., Ladd, C.C. and Barsvary, A.K. 1977. Undrained behaviour of embankments on New Liskeard varved clays. Canadian Geotechnical Journal, 14, pp. 367-388.

Mitchell, J. K., 1993, Fundamentals of Soil Behavior, Second Edition, John Wiley & Sons, Inc.

ASTM International:

ASTM D1586 Standard Test Method for Standard Penetration Test and Split-Barrel Sampling of Soils.

ASTM D1587 Standard Practice for Thin-Walled Tube Sampling of Soils for Geotechnical Purposes

ASTM D2573 Standard Test Method for Field Vane Shear Test in Saturated Fine Grained Soils.

Ontario Provincial Standard Specifications (OPSS) – Provincial Oriented

OPSS 802	Construction Specification for Topsoil
OPSS.PROV 804	Construction Specification for Seed and Cover

MINISTRY OF TRANSPORTATION, ONTARIO

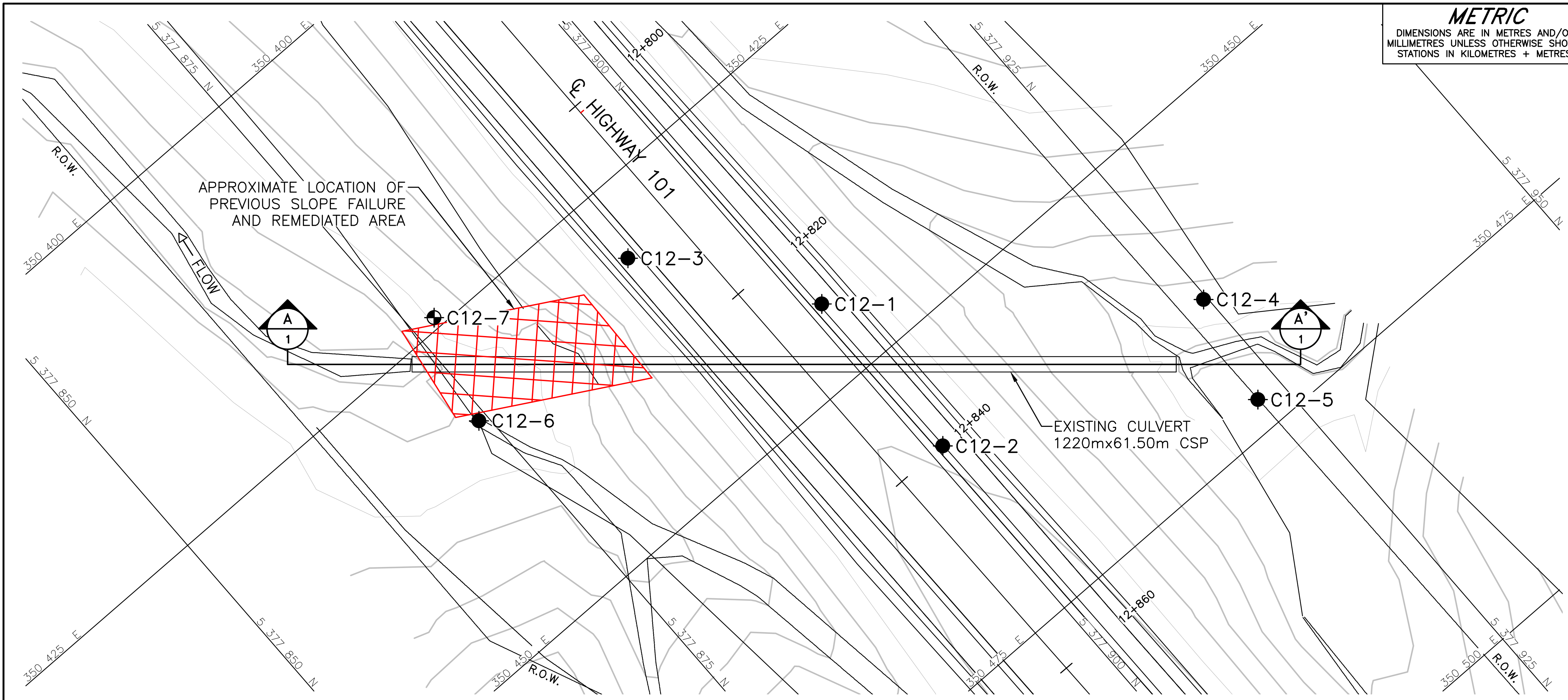
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
GWP No. 5217–13–00

SHEET

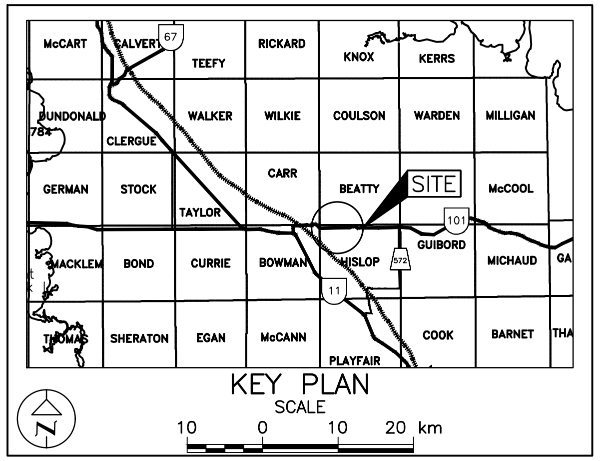
HIGHWAY 101
CULVERT 12 AT STA. 12+820
BOREHOLE LOCATIONS AND SOIL STRATA

GOLDER



PLAN

SCALE
4 0 4 8 m



KEY PLAN

SCALE
10 0 10 20 km

LEGEND

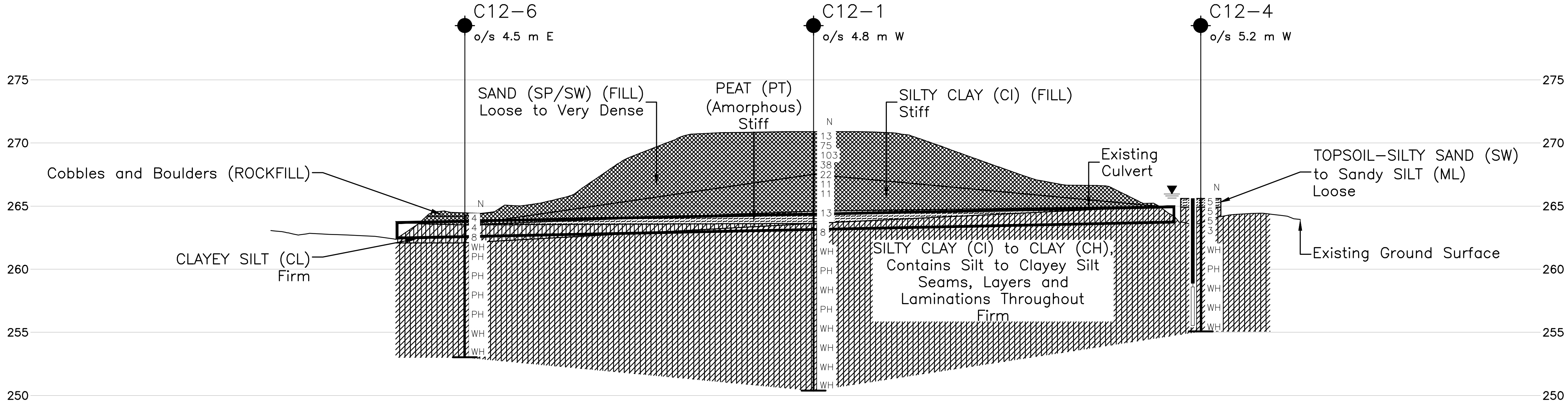
●	Borehole – Current Investigation
⊕	Borehole and Dynamic Cone Penetration Test
▮	Seal
▯	Piezometer
N	Standard Penetration Test Value
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
▼	WL in piezometer, measured on JUNE 10, 2020

A.J.K. BOM
100075715
MAR 02, 2021

K. J. BENTLEY
MAR 02, 2021

BOREHOLE CO-ORDINATES (NAD 83 MTM ZONE 12)

No.	ELEVATION	NORTHING	EASTING
C12–1	270.8	5377901.3	350443.8
C12–2	270.9	5377901.1	350458.8
C12–3	270.7	5377891.9	350430.8
C12–4	265.6	5377924.7	350463.6
C12–5	264.5	5377922.7	350472.6
C12–6	264.3	5377874.3	350432.8
C12–7	264.8	5377877.0	350424.2



A–A'
1

CULVERT CENTERLINE PROFILE

HORIZONTAL SCALE
4 0 4 8 m

4 0 4 8 m
VERTICAL SCALE

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 D.M. WILLS, drawing file no. ACAD–gwp52171300b.dwg, received SEPTEMBER 8, 2020.

NO.	DATE	BY	REVISION
.	.	.	.

Geocres No. 42A–139

HWY. 101		PROJECT NO. 19126505	DIST. .
SUBM'D. .	CHKD. .	DATE: 3/1/2021	SITE: .
DRAWN: TR	CHKD. AB	APPD. KB	DWG. 1

PLOT DATE: March 1, 2021
FILENAME: S:\Clients\MTO\Hwy11&101\99_PROJ\19126505\40_PROD\0001_C12\19126505-0001-BG-0001.dwg

Photograph 1: North embankment slope, drill rig set up at Borehole C12-4, looking east (June 2020)

Photograph 2: South embankment slope, looking west (June 2020)

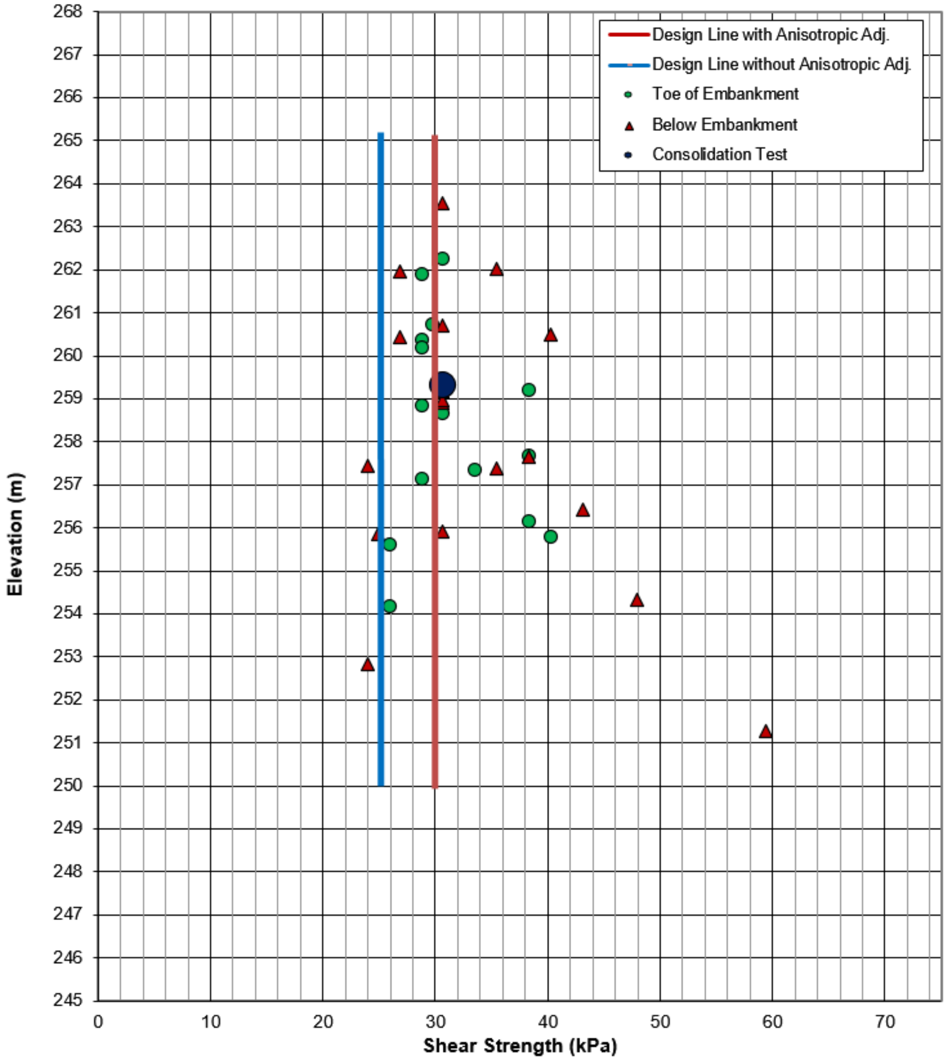
Photograph 3: South embankment, looking up the slope at previous slope failure and rehabilitated area (June 2020)

Photograph 4: South embankment, looking down the slope at drill rig set up at Borehole C12-6 (June 2020)

Undrained Shear Strength vs Elevation

Silty Clay to Clay Deposit

Figure 1

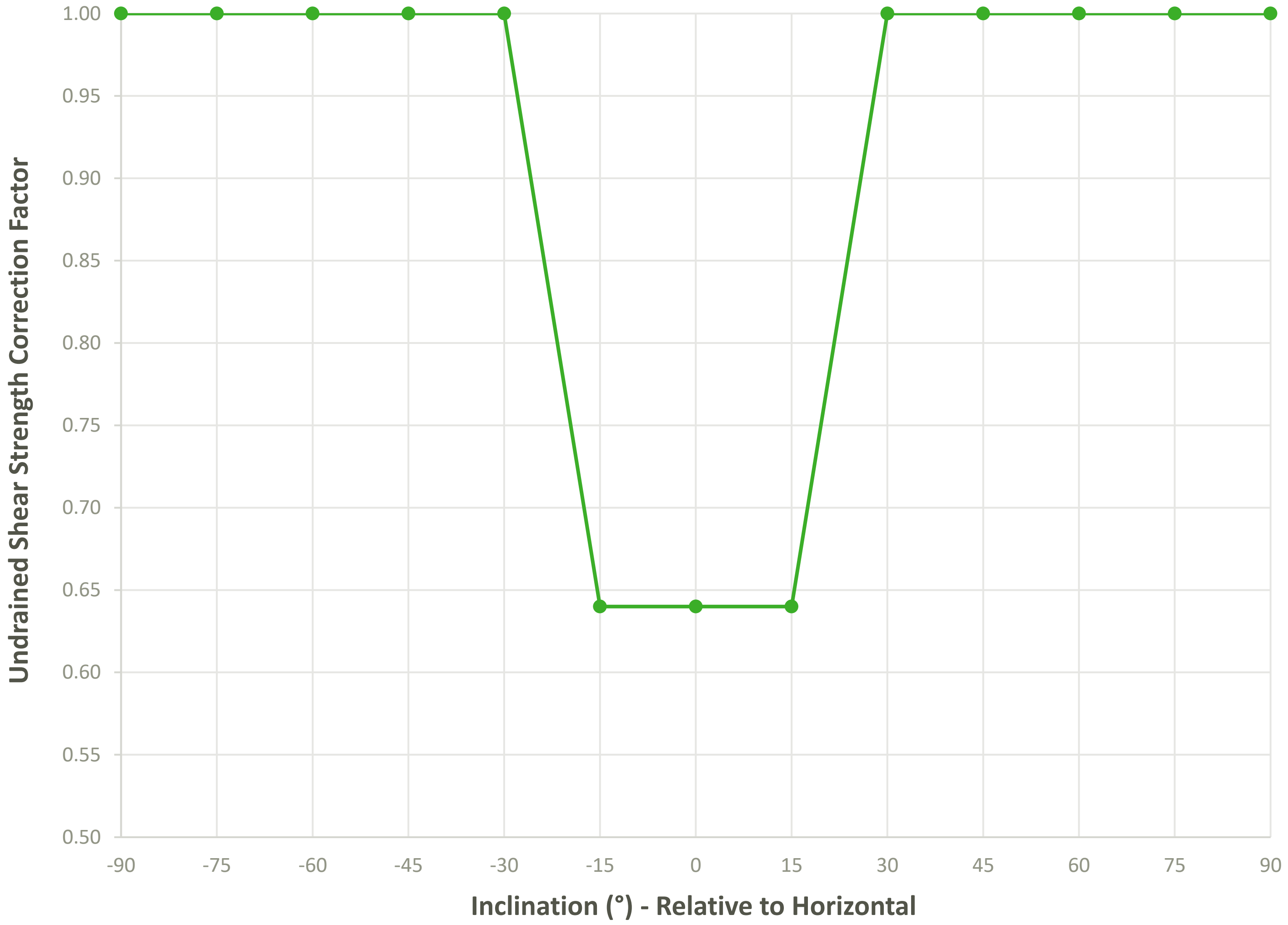


Date: January 2021
Project Number: 19126505

Analysis By: KJ
Reviewed By: AB

Anisotropic Undrained Shear Strength Correction Function for Silty Clay to Clay Deposit

Figure 2



Date: January 2021
Project Number: 19126505

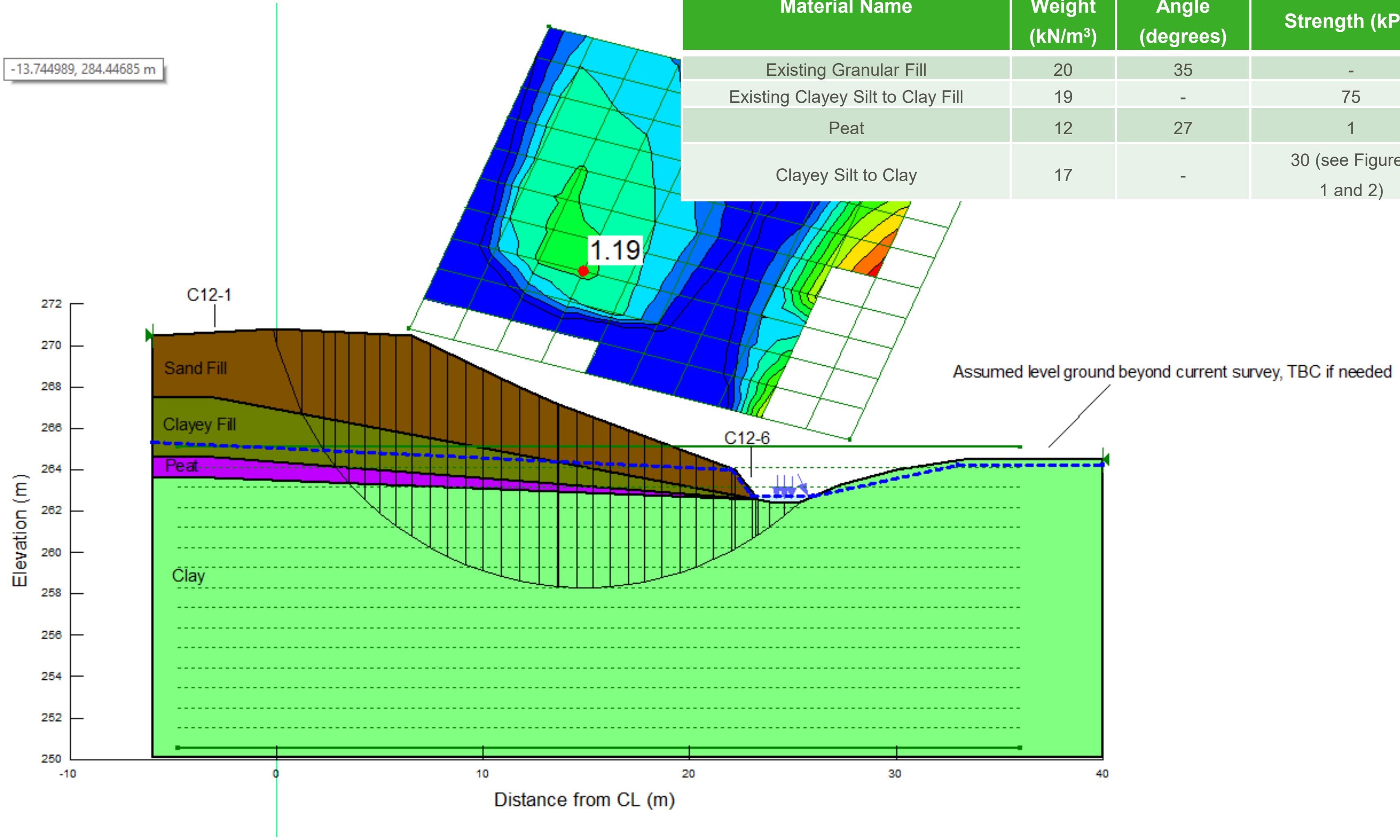
Analysis By: KJ
Reviewed By: MT/AB

Global Stability Analysis

Figure 3A

South Side Slope – Existing Embankment
Short-Term Condition – With Anisotropic Adjustment

Material Name	Unit Weight (kN/m^3)	Friction Angle (degrees)	Undrained Shear Strength (kPa)
Existing Granular Fill	20	35	-
Existing Clayey Silt to Clay Fill	19	-	75
Peat	12	27	1
Clayey Silt to Clay	17	-	30 (see Figures 1 and 2)



Date: January 2021
Project Number: 19126505

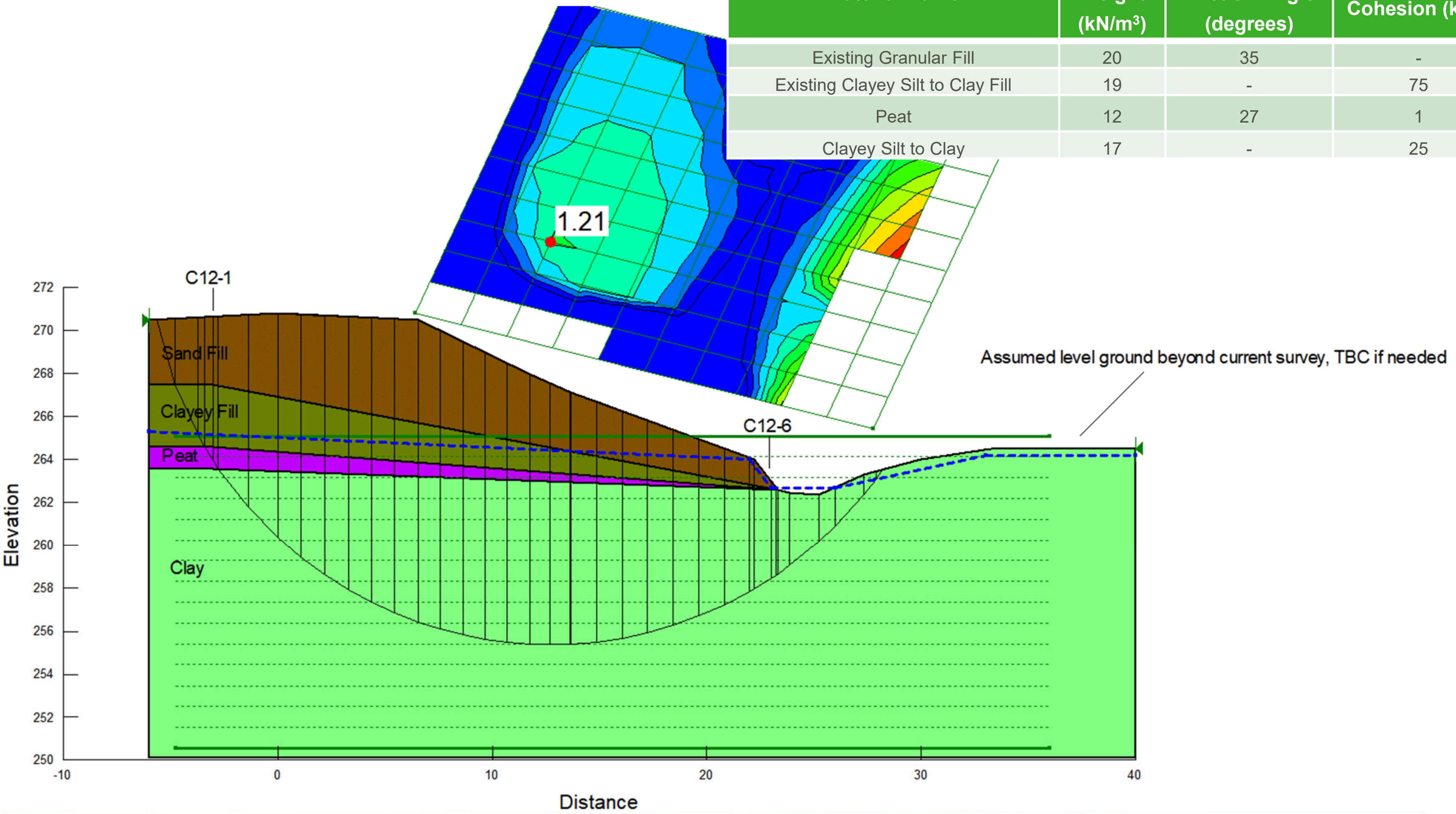
Analysis By: AB
Reviewed By: MT

Global Stability Analysis

Figure 3B

South Side Slope – Existing Embankment
Short-Term Condition – Without Anisotropic Adjustment

Material Name	Unit Weight (kN/m³)	Effective Friction Angle (degrees)	Effective Cohesion (kPa)
Existing Granular Fill	20	35	-
Existing Clayey Silt to Clay Fill	19	-	75
Peat	12	27	1
Clayey Silt to Clay	17	-	25



Assumed level ground beyond current survey, TBC if needed

Date: January 2021
Project Number: 19126505

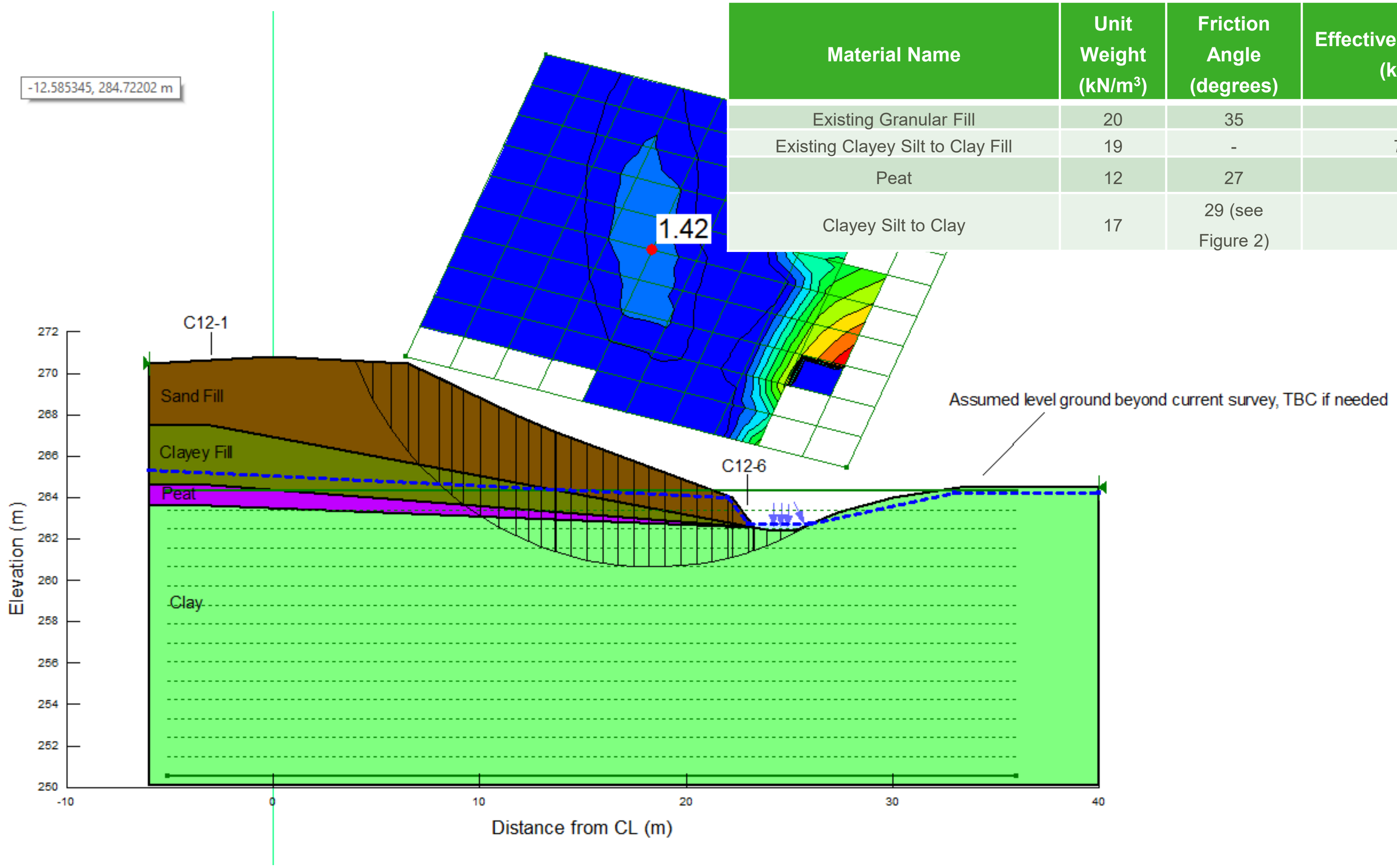
Analysis By: AB
Reviewed By: MT

Global Stability Analysis

Figure 4

South Side Slope – Existing Embankment
Long Term Condition

Material Name	Unit Weight (kN/m^3)	Friction Angle (degrees)	Effective Cohesion (kPa)
Existing Granular Fill	20	35	-
Existing Clayey Silt to Clay Fill	19	-	75
Peat	12	27	1
Clayey Silt to Clay	17	29 (see Figure 2)	4



Assumed level ground beyond current survey, TBC if needed

Date: January 2021
Project Number: 19126505

Analysis By: AB
Reviewed By: MT

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	>200	>8
COBBLES	Not Applicable	75 to 200	3 to 8
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

1. 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.
2. 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.

LIST OF SYMBOLS
MINISTRY OF TRANSPORTATION, ONTARIO

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 $\gamma = \rho \cdot g$ (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
$C_{\alpha(e)}$	secondary compression index
C_α	rate of secondary compression
$C_{\alpha(\varepsilon)}$	modified 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
c′	effective cohesion
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction $= \tan \delta$
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 or 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

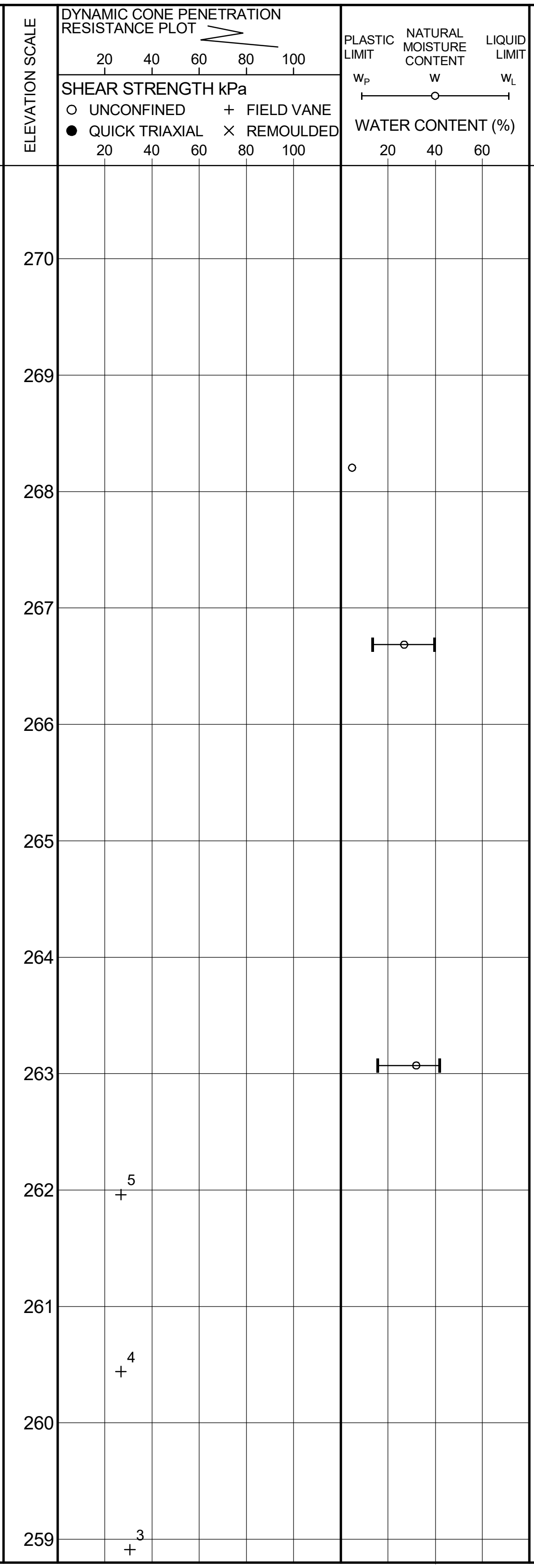
GOLDER

RECORD OF BOREHOLE No. C12-1

1 OF 2 **METRIC**

PROJECT 19126505
G.W.P. 5217-13-00
LOCATION N 5377901.3; E 350443.8 NAD83 MTM ZONE 12 (LAT. 48.538043; LONG. -80.381847)
ORIGINATED BY TB
DIST
HWY 101
BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers
COMPILED BY TR
DATUM GEODETIC
DATE June 2, 2020
CHECKED BY AB

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES		GR	SA	SI	CL
270.8	GROUND SURFACE								
0.0	SAND (SP), some silt, trace gravel (FILL) Compact to very dense Brown to grey Moist - Auger grinding from 0.0 m to 2.1 m depth	1	SS	13					
		2	SS	75					
		3	SS	103					
	- Trace silt pockets in Sample No. 4	4	SS	38		9	78	(13)	
267.5 3.3	SILTY CLAY (CI), trace sand, trace organics, trace wood pieces (FILL) Stiff Brown to grey Moist to wet	5A 5B	SS	22					
		6	SS	11					
		7	SS	11					
264.6 6.2	PEAT (PT) (Amorphous) Stiff Black Wet	8	SS	13					
263.6 7.2	SILTY CLAY (CI) to CLAY (CH) Firm Grey Wet - Silt to clayey silt seams, layers and laminations encountered throughout deposit; portions of the deposit are considered to be varved.	9A 9B	SS	8					
		10	SS	WH					
		11	TO	PH					



UNIT WEIGHT γ kN/m³

Continued Next Page

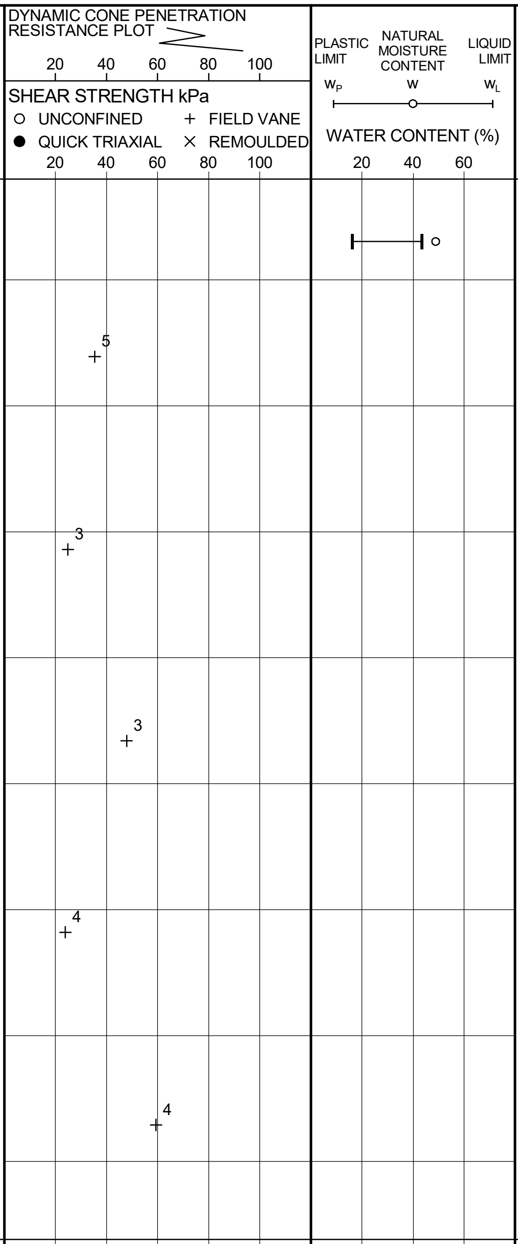
+3, ×3: Numbers refer to Sensitivity ○3% STRAIN AT FAILURE

RECORD OF BOREHOLE No. C12-1

2 OF 2 **METRIC**

PROJECT 19126505
G.W.P. 5217-13-00
LOCATION N 5377901.3; E 350443.8 NAD83 MTM ZONE 12 (LAT. 48.538043; LONG. -80.381847)
ORIGINATED BY TB
DIST ____ HWY 101
BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers
COMPILED BY TR
DATUM GEODETIC
DATE June 2, 2020
CHECKED BY AB

SOIL PROFILE			SAMPLES						REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---								
	SILTY CLAY (CI) to CLAY (CH) Firm Grey Wet - Silt to clayey silt seams, layers and laminations encountered throughout deposit; portions of the deposit are considered to be varved.	▨	12	SS	WH				
		▨					258		
		▨	13	TO	PH		257		
		▨					256		
		▨	14	SS	WH		255		
		▨					254		
		▨	15	SS	WH		253		
		▨	16	SS	WH		252		
	- Transition to stiff.	▨	17	SS	WH		251		
250.4 20.4	END OF BOREHOLE NOTE: 1. Borehole dry upon completion of drilling.								



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

RECORD OF BOREHOLE No. C12-2

1 OF 2 **METRIC**

PROJECT 19126505

G.W.P. 5217-13-00

DIST ____ HWY 101

DATUM GEODETIC

LOCATION N 5377901.1; E 350458.8 NAD83 MTM ZONE 12 (LAT. 48.538041; LONG. -80.381644)

BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers and NW Casing with Wash Boring

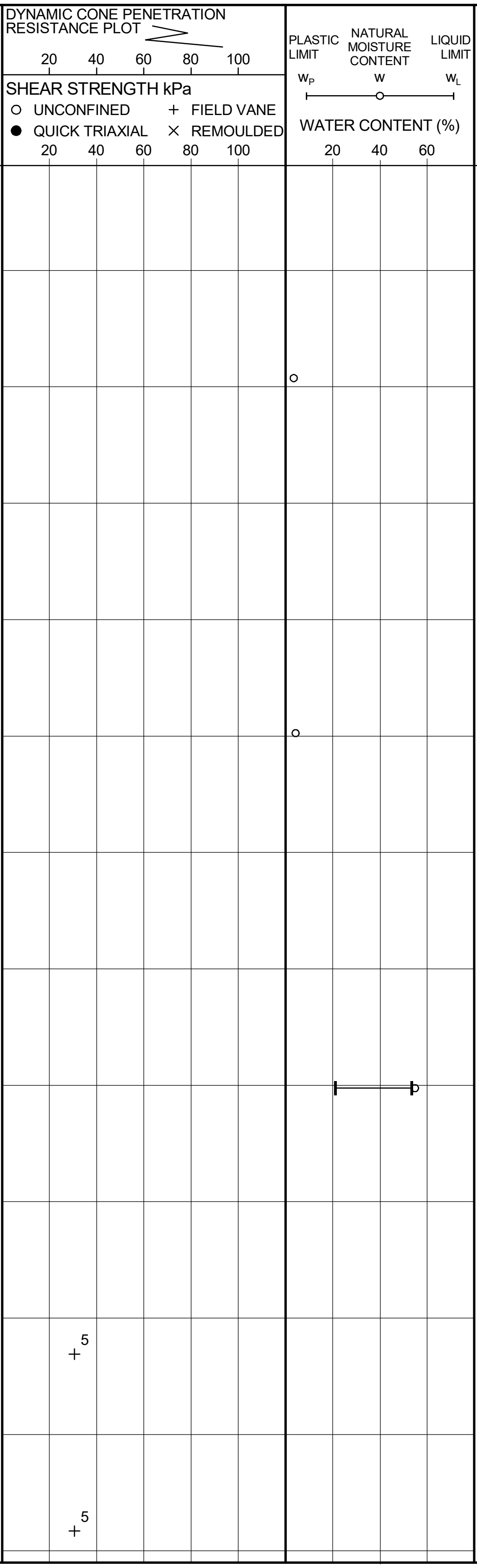
DATE June 3, 2020

ORIGINATED BY TB

COMPILED BY TR

CHECKED BY AB

SOIL PROFILE ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES "N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
270.9	GROUND SURFACE											
0.0	SAND (SP) and gravel, some silt (FILL) Compact to very dense Brown Moist		1	SS	13							
	- Split-spoon refusal on inferred cobble at 1.1 m depth		2	SS	94/0.23		270					
			3	SS	97		269		38	51	(11)	
			4	SS	40		268					
			5	SS	118							
	- Split-spoon refusal on inferred boulder at 3.8 m depth. No recovery in Sample No. 6. - Auger deflected out of alignment and borehole abandoned at 3.8 m depth. Drilled new borehole adjacent to C12-2 using NW Casing and continued sampling below 4.6 m depth.		~~6~~	~~SS~~	~~100/0.00~~		267					
			7	SS	24		266		37	53	(10)	
265.3 5.6	CLAY (CH), trace gravel, trace sand (FILL) Stiff Grey Wet						265					
			8	SS	9		264					
	- Trace wood pieces in Sample No. 9		9	SS	8		263					
262.2 8.7	SILTY CLAY (CI) to CLAY (CH) Firm Grey Wet - Silt to clayey silt seams, layers and laminations encountered throughout deposit; portions of the deposit are considered to be varved.						262					
			10	SS	WH		261					
			11	SS	WH		260					
							259					



Continued Next Page

+3, ×3: Numbers refer to Sensitivity

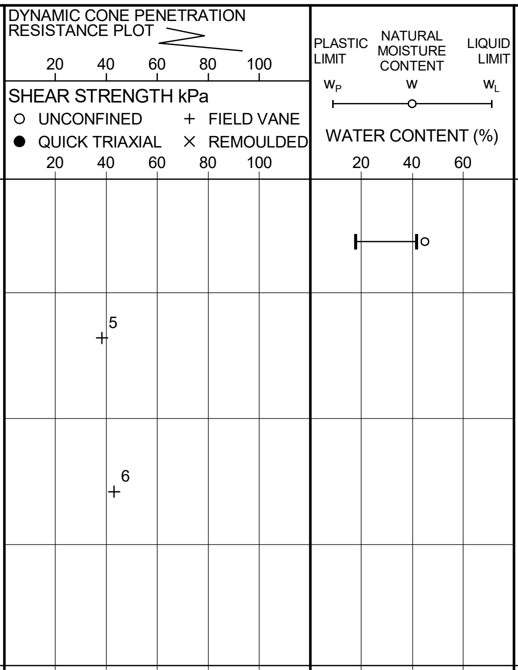
○3% STRAIN AT FAILURE

RECORD OF BOREHOLE No. C12-2

2 OF 2 **METRIC**

PROJECT 19126505
G.W.P. 5217-13-00
LOCATION N 5377901.1; E 350458.8 NAD83 MTM ZONE 12 (LAT. 48.538041; LONG. -80.381644)
ORIGINATED BY TB
DIST ____ HWY 101
BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers and NW Casing with Wash Boring
COMPILED BY TR
DATUM GEODETIC
DATE June 3, 2020
CHECKED BY AB

SOIL PROFILE			SAMPLES							
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---								
	SILTY CLAY (CI) to CLAY (CH) Firm Grey Wet - Silt to clayey silt seams, layers and laminations encountered throughout deposit; portions of the deposit are considered to be varved.	▨	12	SS	WH				
		▨					258		
		▨	13	SS	WH		257		
		▨					256		
255.0 15.9	END OF BOREHOLE NOTE: 1. Borehole dry upon completion of drilling.	▨	14	SS	WH				



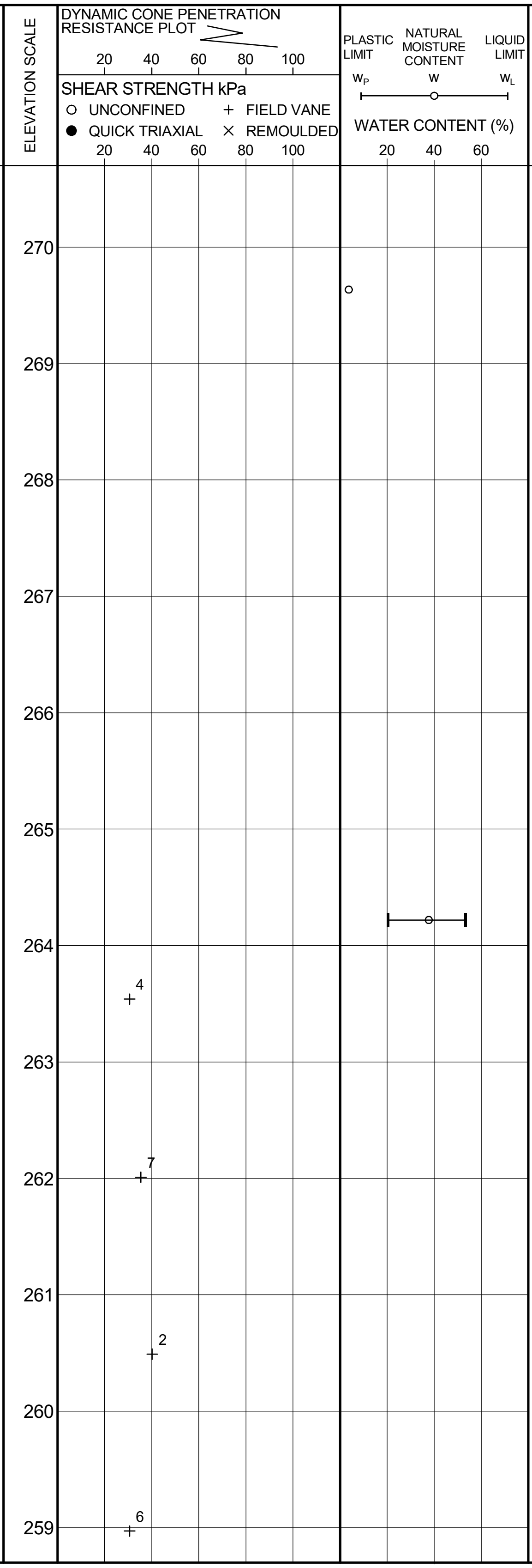
+3, ×3: Numbers refer to Sensitivity   ○3% STRAIN AT FAILURE

RECORD OF BOREHOLE No. C12-3

1 OF 2 **METRIC**

PROJECT 19126505
G.W.P. 5217-13-00
LOCATION N 5377891.9; E 350430.8 NAD83 MTM ZONE 12 (LAT. 48.537959; LONG. -80.382024)
ORIGINATED BY TB
DIST
HWY 101
BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers
COMPILED BY TR
DATUM GEODETIC
DATE June 6, 2020
CHECKED BY AB

ELEV DEPTH	SOIL PROFILE – DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
270.7	GROUND SURFACE										
0.0	SAND (SP), trace gravel, trace silt (FILL) Compact to very dense Brown Moist - Auger grinding through fill	▩	1	SS	19						
			2	SS	78			1	92	(7)	
			3	SS	60						
268.3 2.4	SILT (ML) to CLAYEY SILT (CL), trace gravel, trace organics, trace wood (FILL) Stiff to very stiff Grey Moist	▩	4A 4B	SS	13						
			5	SS	21						
			6	SS	14						
			7	SS	11						
265.1 5.6	SILTY CLAY (CI) to CLAY (CH) Firm Grey Wet - Silt to clayey silt seams, layers and laminations encountered throughout deposit; portions of the deposit are considered to be varved.	▨	8A 8B	SS	10						
			9	SS	WH						
			10	SS	WH						
			11	SS	WH						



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

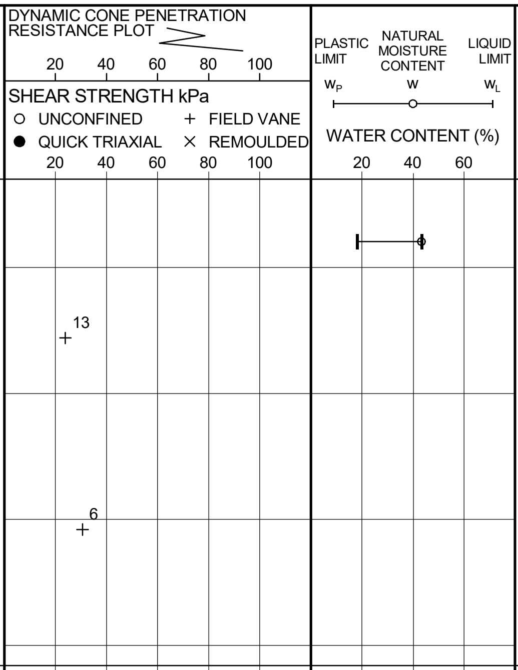
PROJECT 19126505

RECORD OF BOREHOLE No. C12-3

2 OF 2 **METRIC**

G.W.P. 5217-13-00
LOCATION N 5377891.9; E 350430.8 NAD83 MTM ZONE 12 (LAT. 48.537959; LONG. -80.382024)
ORIGINATED BY TB
DIST
HWY 101
BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers
COMPILED BY TR
DATUM GEODETIC
DATE June 6, 2020
CHECKED BY AB

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---								
	SILTY CLAY (CI) to CLAY (CH) Firm Grey Wet - Silt to clayey silt seams, layers and laminations encountered throughout deposit; portions of the deposit are considered to be varved.	▨	12	SS	WH		258		
		▨	13	SS	WH		257 256		
254.8		▨	14	SS	WH		255		
15.9	END OF BOREHOLE NOTE: 1. Borehole dry upon completion of drilling.								



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

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

RECORD OF BOREHOLE No. C12-4

1 OF 2 **METRIC**

PROJECT 19126505

G.W.P. 5217-13-00

LOCATION N 5377924.7; E 350463.6 NAD83 MTM ZONE 12 (LAT. 48.538252; LONG. -80.381576)

ORIGINATED BY TB

DIST HWY 101

BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers

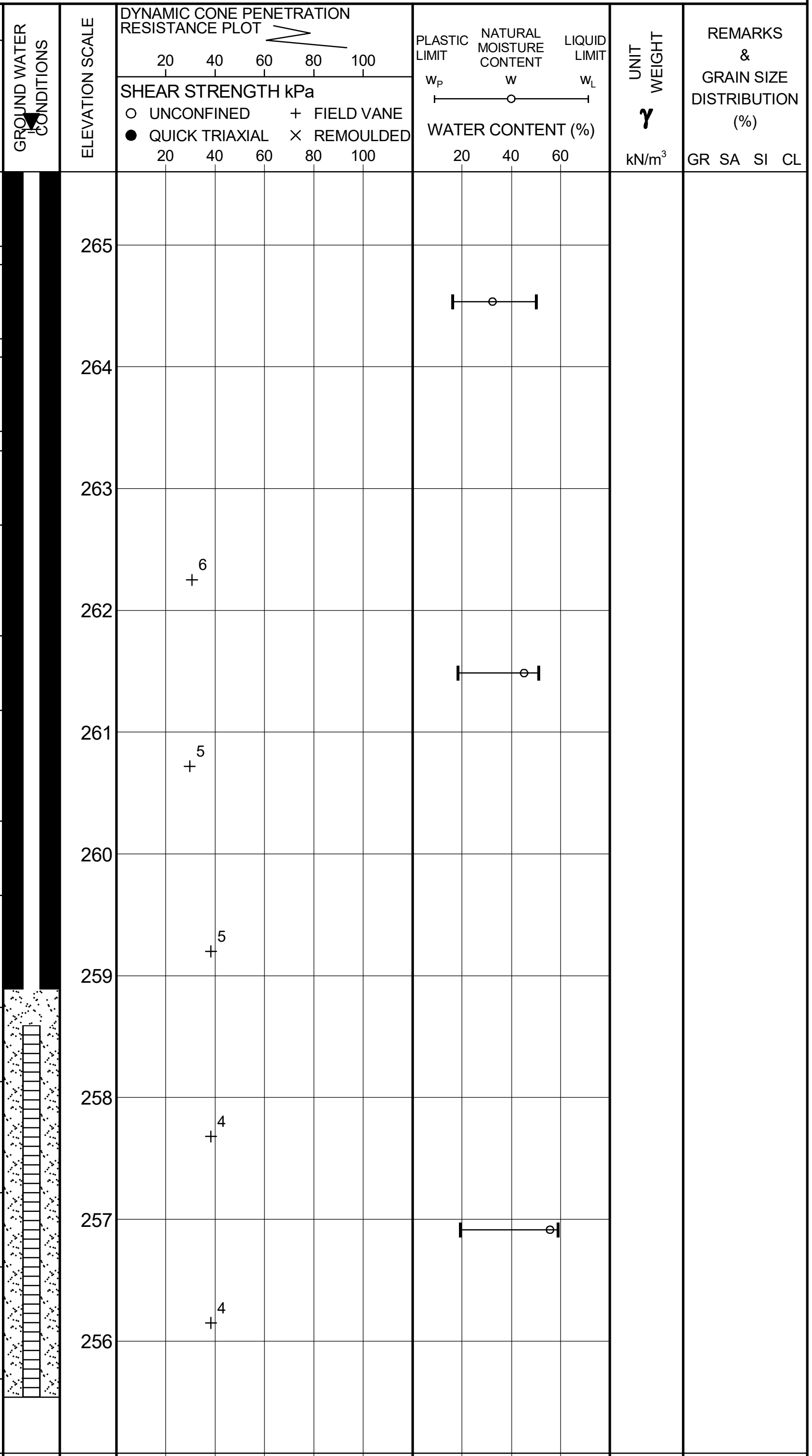
COMPILED BY TR

DATUM GEODETIC

DATE June 4, 2020

CHECKED BY AB

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES
265.6	GROUND SURFACE			
0.0	TOPSOIL - SILTY SAND (SM) to sandy SILT (ML) Loose Brown to black Moist	1	SS	5
264.9 0.7	SILTY CLAY (CI) to CLAY (CH) Firm Grey Wet - Silt to clayey silt seams, layers and laminations encountered throughout deposit; portions of the deposit are considered to be varved.	2	SS	5
		3	SS	5
		4A 4B	SS	3
		5	SS	WH
		6	TO	PH
		7	SS	WH
		8	SS	WH
		9	SS	WH
255.1 10.5	END OF BOREHOLE			



Continued Next Page

+³, ×³: Numbers refer to Sensitivity

○³% STRAIN AT FAILURE

PROJECT 19126505

RECORD OF BOREHOLE No. C12-4

2 OF 2 **METRIC**

G.W.P. 5217-13-00
LOCATION N 5377924.7; E 350463.6 NAD83 MTM ZONE 12 (LAT. 48.538252; LONG. -80.381576)
ORIGINATED BY TB
DIST
HWY 101
BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers
COMPILED BY TR
DATUM GEODETIC
DATE June 4, 2020
CHECKED BY AB

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						
	NOTES: 1. Borehole dry upon completion of drilling. 2. Water level in piezometer at ground surface (Elev. 265.6 m) on June 6, 2020. 3. Water level measured in piezometer at a depth of 0.3 m above ground surface (Elev. 265.9 m) on June 10, 2020.										

+³, ×³: Numbers refer to Sensitivity

○³% STRAIN AT FAILURE

RECORD OF BOREHOLE No. C12-5

1 OF 1 **METRIC**

PROJECT 19126505
G.W.P. 5217-13-00
DIST
HWY 101
DATUM GEODETIC
LOCATION N 5377922.7; E 350472.6 NAD83 MTM ZONE 12 (LAT. 48.538233; LONG. -80.381455)
BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers
DATE June 4, 2020
ORIGINATED BY TB
COMPILED BY TR
CHECKED BY AB

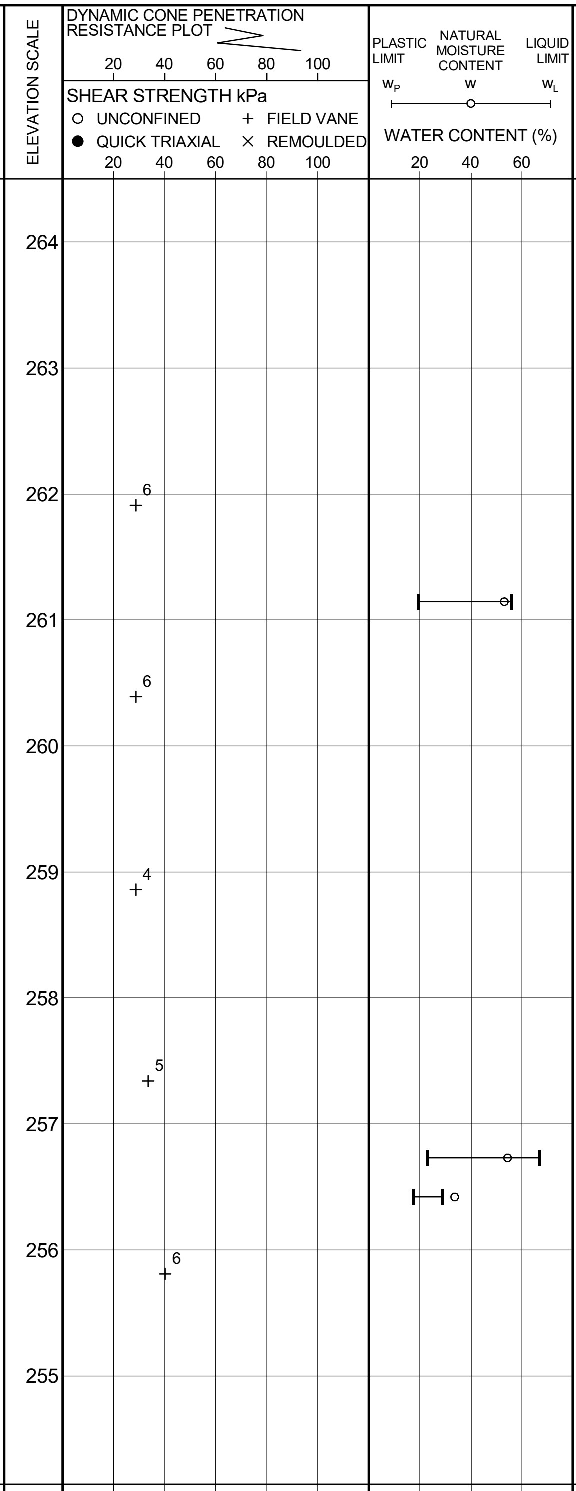
SOIL PROFILE

ELEV DEPTH	DESCRIPTION	STRAT PLOT
264.5	GROUND SURFACE	
0.0	TOPSOIL - Sandy SILT (ML) Very loose Brown Moist	▤
263.8 0.7	- Wood pieces at 0.7 m Sandy SILT (ML), trace organics Very loose Brown Moist	▦
263.0 1.5	- Wood pieces at 1.5 m SILTY CLAY (CI) to CLAY (CH) Firm Grey Wet - Silt to clayey silt seams, layers and laminations encountered throughout deposit; portions of the deposit are considered to be varved. - 25 mm to 40 mm thick light grey clayey silt zones between 10 mm to 75 mm thick dark grey clay	▨
254.1 10.4	NOTES: 1. Borehole dry upon completion of drilling.	

SAMPLES

NUMBER	TYPE	"N" VALUES
1	SS	WH
2	SS	WH
3	SS	WH
4	SS	WH
5	SS	WH
6	SS	WH
7A 7B	TO	PH
8	TO	PH
9	SS	WH

GROUND WATER CONDITIONS



UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL

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

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

GOLDER

Foundation Design

RECORD OF BOREHOLE No. C12-6

1 OF 1 **METRIC**

PROJECT 19126505
G.W.P. 5217-13-00
DIST ______ HWY 101
DATUM GEODETIC
LOCATION N 5377874.3; E 350432.8 NAD83 MTM ZONE 12 (LAT. 48.537801; LONG. -80.381999)
BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers
DATE June 5, 2020
ORIGINATED BY TB
COMPILED BY TR
CHECKED BY AB

SOIL PROFILE

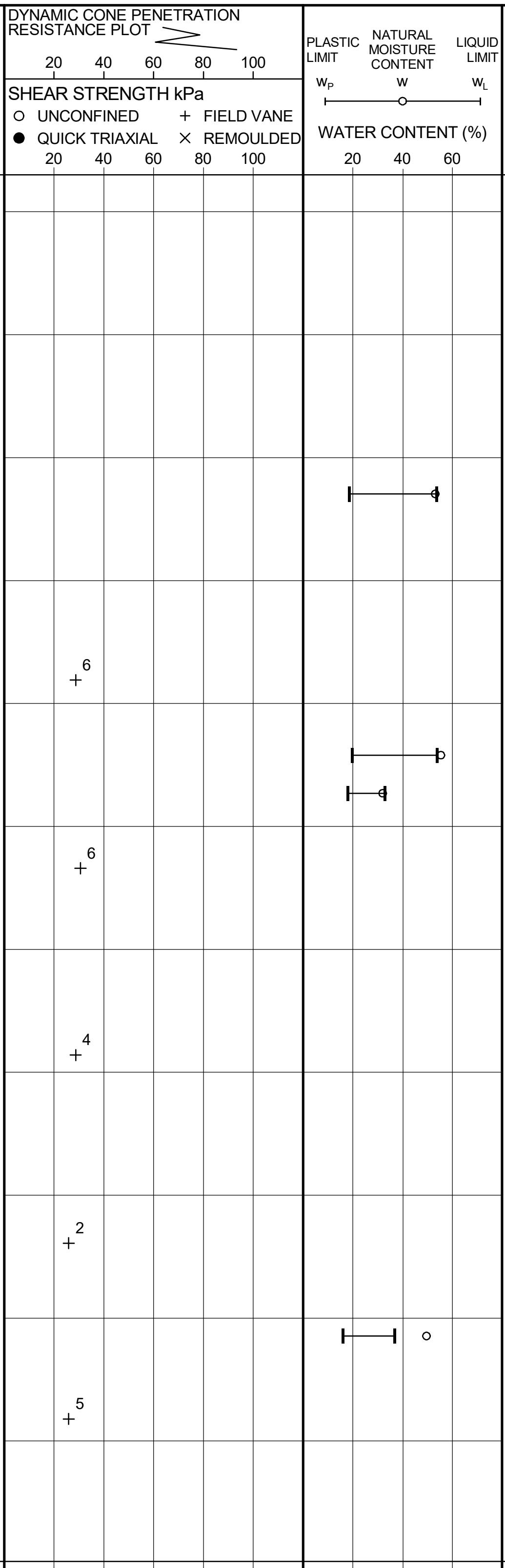
ELEV DEPTH	DESCRIPTION
264.3	GROUND SURFACE
0.0	Cobbles and boulders on surface (ROCKFILL)
0.1	SAND (SW), trace gravel, trace organics (FILL) Loose Brown Wet
263.5 0.8	CLAYEY SILT (CL), trace organics, trace wood pieces Firm Brown Moist
262.1 2.2	SILTY CLAY (CI) to CLAY (CH) Firm Grey Wet - Silt to clayey silt seams, layers and laminations encountered throughout deposit; portions of the deposit are considered to be varved. - 10 mm thick light grey clayey silt zones between 20 mm thick dark grey clay
253.0 11.3	NOTES: 1. Borehole dry upon completion of drilling.

SAMPLES

NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³
1	AS	4	
2	SS	4	
3	SS	8	
4	SS	WH	
5	TO	PH	
6A 6B	TO	PH	16.4
7	TO	PH	
8	TO	PH	
9	SS	WH	
10	SS	WH	

GROUND WATER CONDITIONS

ELEVATION SCALE



REMARKS & GRAIN SIZE DISTRIBUTION (%)
GR SA SI CL

+3, ×3: Numbers refer to Sensitivity

○3% STRAIN AT FAILURE

SUD-MTO 001 S:\SIM\CLIENTS\MTO\HWY11&101\02_DATA\GINT\19126505.GPJ GAL-MISS.GDT 10/2/20 TR

RECORD OF BOREHOLE No. C12-7

1 OF 1 **METRIC**

PROJECT 19126505			
G.W.P. 5217-13-00	LOCATION N 5377877.0; E 350424.2 NAD83 MTM ZONE 12 (LAT. 48.537826; LONG. -80.382115)	ORIGINATED BY AD	
DIST	HWY 101	BOREHOLE TYPE Portable Equipment, NW Casing	COMPILED BY TR
DATUM GEODETIC	DATE June 30, 2020	CHECKED BY AB	

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES			GR SA SI CL
264.8	GROUND SURFACE				▽		
0.0	TOPSOIL (150 mm)						
0.2	SAND (SW), trace organics (FILL) Very loose to compact Brown Moist to wet	1	SS	4			
		2	SS	11		264	
263.4 1.5	Gravelly SAND (SW) (FILL) Loose Brown Wet	3	SS	5		263	
		4	SS	7			
261.9 2.9	END OF BOREHOLE Dynamic Cone Penetration Test (DCPT) started at 2.9 m depth.					262	
						261	
						260	
						259	
						258	
						257	
						256	
255.7 9.1	END OF DCPT NOTE: 1. Water level measured at ground surface (Elev. 264.8 m) upon completion of drilling.						

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH kPa

○ UNCONFINED + FIELD VANE

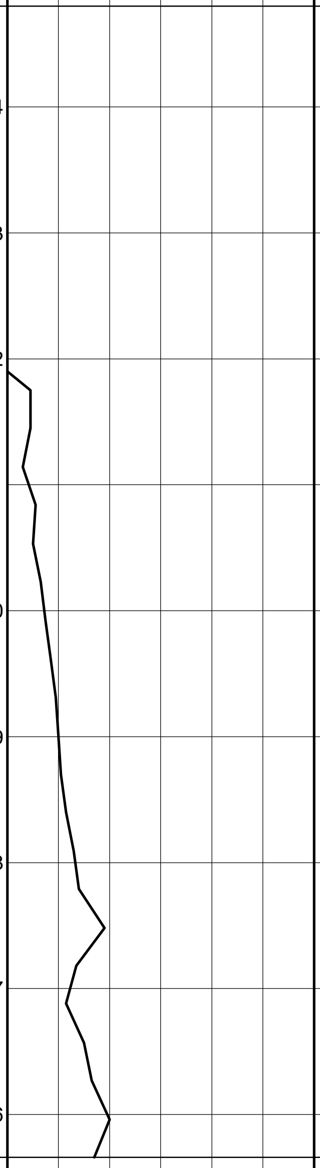
● QUICK TRIAXIAL × REMOULDED

20 40 60 80 100

PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L

WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m³

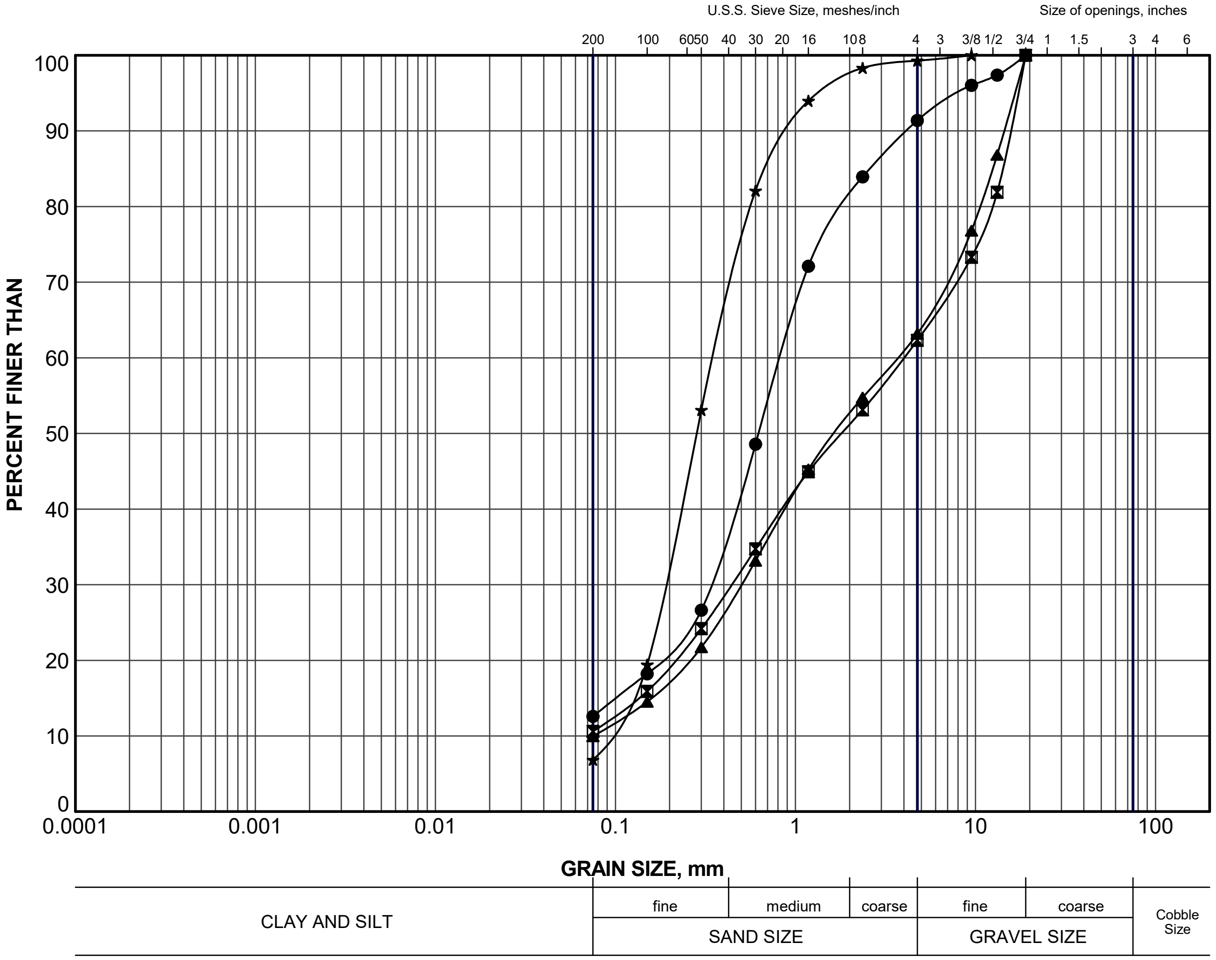


SUD-MTO 001 S:\SIM\CLIENTS\MTO\HWY11&101\02_DATA\GINT\19126505.GPJ GAL-MISS.GDT 10/2/20 TR

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

APPENDIX B

Laboratory Test Results



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	C12-1	4	268.2
☒	C12-2	3	269.1
▲	C12-2	7	266.0
★	C12-3	2	269.6

PROJECT

HIGHWAY 101
CULVERT 12 AT STA. 12+820
TOWNSHIP OF BEATTY

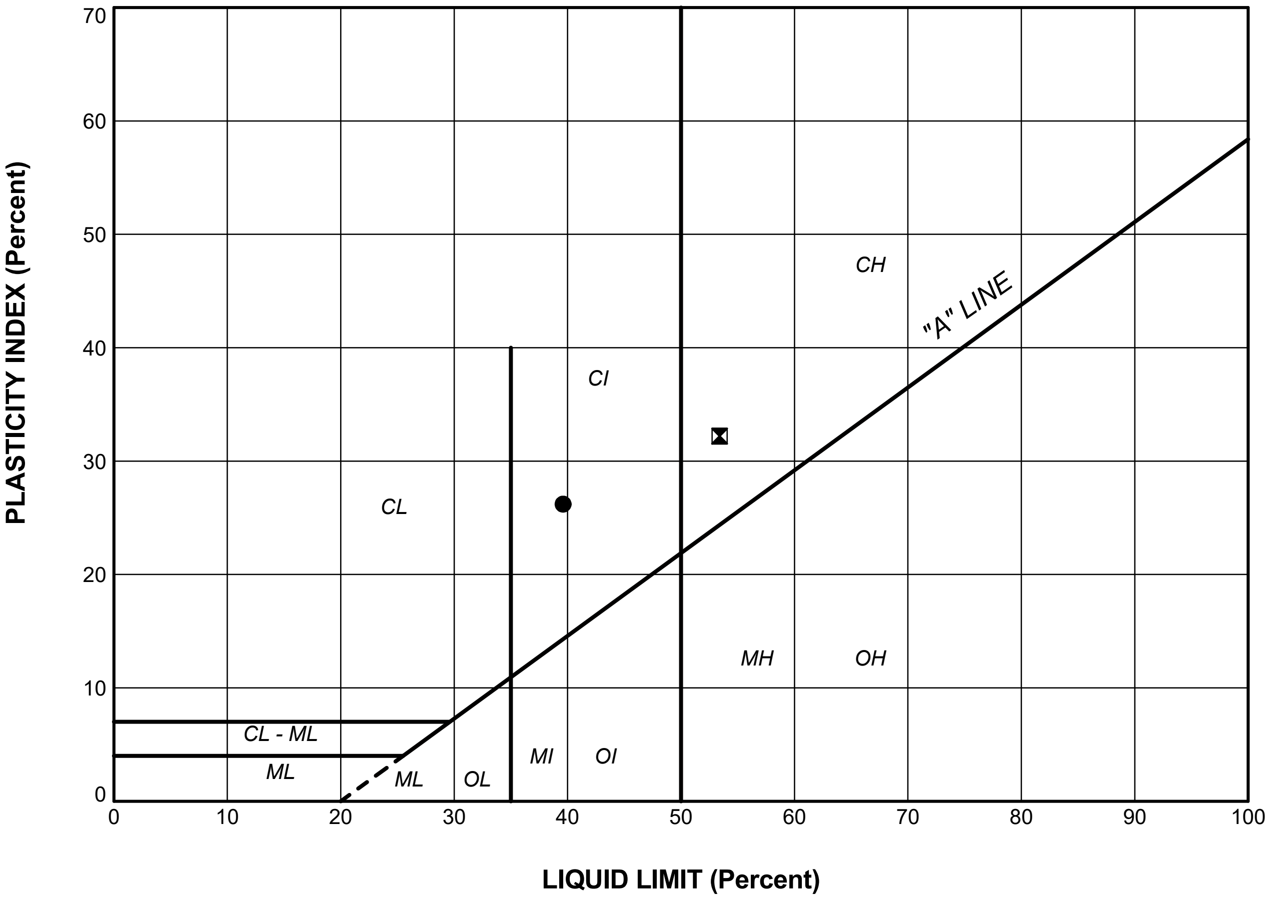
TITLE

GRAIN SIZE DISTRIBUTION
SAND (SP) to SAND (SP) and Gravel (FILL)

GOLDER
SUDBURY, ONTARIO

PROJECT No.		19126505	FILE No.	19126505.GPJ
DRAWN	TR	Jan 2021	SCALE N/A	REV.
CHECK	AB	Jan 2021	**FIGURE B-1**	
APPR	KB	Jan 2021		

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
●	C12-1	6	39.6	13.4	26.2
☒	C12-2	9	53.4	21.2	32.2

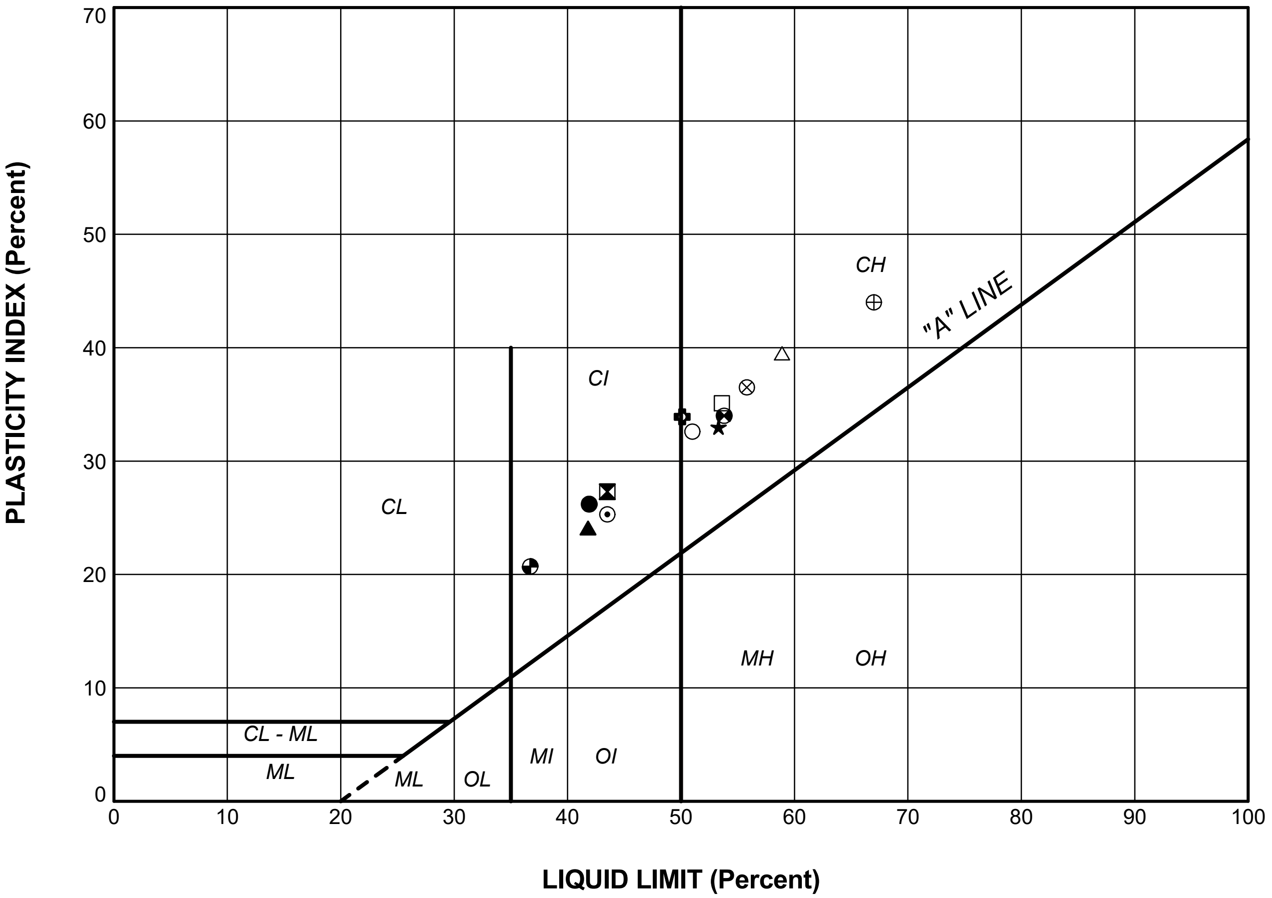
PROJECT
HIGHWAY 101
CULVERT 12 AT STA. 12+820
TOWNSHIP OF BEATTY

TITLE
PLASTICITY CHART
SILTY CLAY (CI) to CLAY (CH) (FILL)

GOLDER
SUDBURY, ONTARIO

PROJECT No.		19126505	FILE No. 19126505.GPJ
DRAWN	TR	Jan 2021	SCALE N/A REV.
CHECK	AB	Jan 2021	
APPR	KB	Jan 2021	**FIGURE B-2**

SUD-MTO PL 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
●	C12-1	9A	41.9	15.7	26.2
⧗	C12-1	12	43.5	16.2	27.3
▲	C12-2	12	41.8	17.7	24.1
★	C12-3	8B	53.3	20.3	33.0
⊙	C12-3	12	43.5	18.2	25.3
✚	C12-4	2	50.1	16.2	33.9
○	C12-4	5	51.0	18.4	32.6
△	C12-4	8	58.9	19.4	39.5
⊗	C12-5	4	55.8	19.3	36.5
⊕	C12-5	7A	67.0	23.0	44.0
□	C12-6	4	53.6	18.5	35.1
◉	C12-6	6A	53.8	19.8	34.0
◑	C12-6	9	36.7	16.0	20.7

PROJECT
HIGHWAY 101
CULVERT 12 AT STA. 12+820
TOWNSHIP OF BEATTY

TITLE
PLASTICITY CHART
SILTY CLAY (CI) to CLAY (CH)

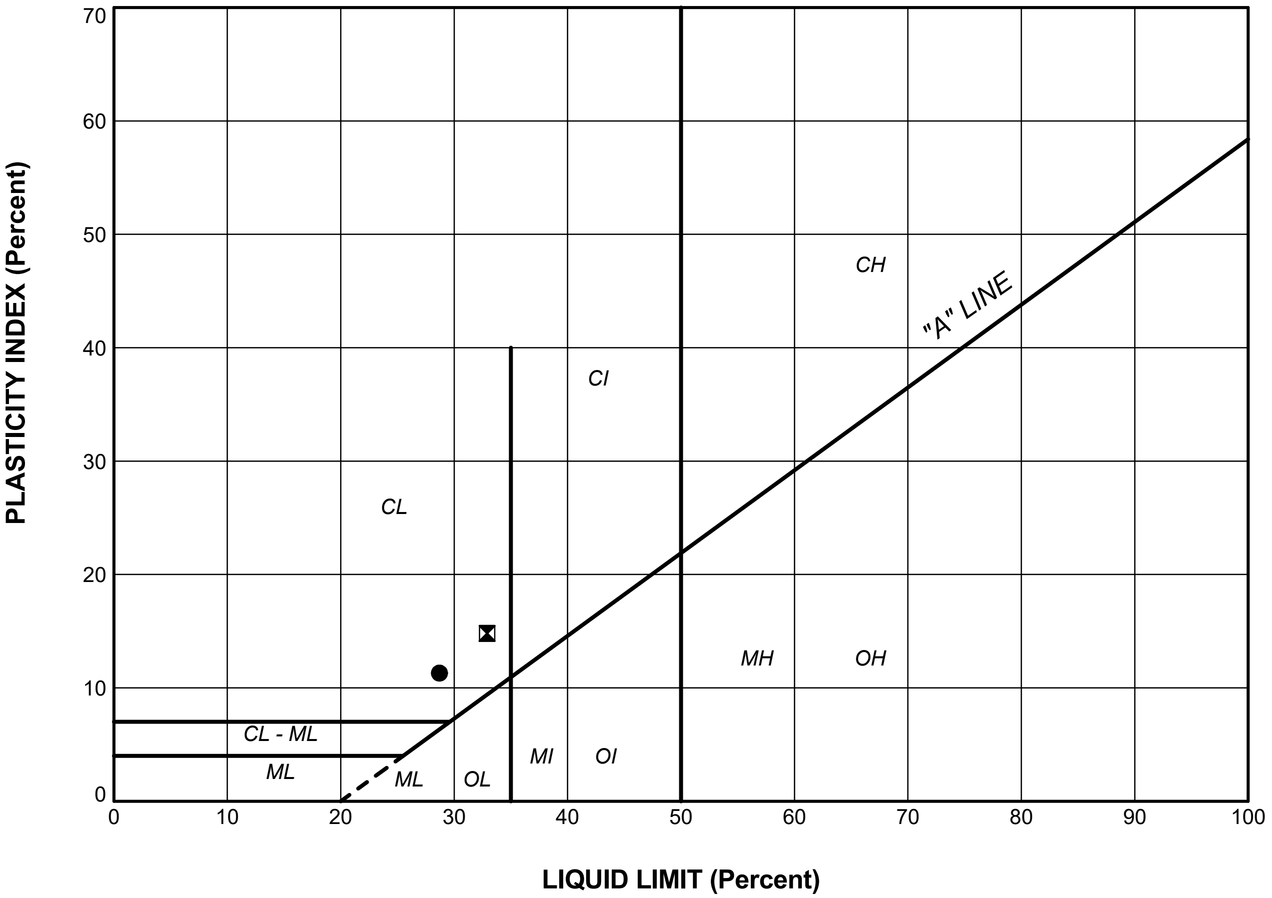
GOLDER
SUDBURY, ONTARIO

PROJECT No.		19126505
DRAWN	TR	Jan 2021
CHECK	AB	Jan 2021
APPR	KB	Jan 2021

FILE No. 19126505.GPJ
SCALE N/A
REV.

FIGURE B-3

SUD-MTO PL (25) 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
●	C12-5	7B	28.7	17.4	11.3
☒	C12-6	6B	32.9	18.1	14.8

PROJECT

HIGHWAY 101
CULVERT 12 AT STA. 12+820
TOWNSHIP OF BEATTY

TITLE

PLASTICITY CHART

CLAYEY SILT (CL)
(INTERLAYER)

PROJECT No.		19126505	FILE No.	19126505.GPJ
DRAWN	TR	Jan 2021	SCALE N/A	REV.
CHECK	AB	Jan 2021	**FIGURE B-4**	
APPR	KB	Jan 2021		

GOLDER
SUDBURY, ONTARIO

SUD-MTO PL GLDR_LDN.GDT

CONSOLIDATION TEST SUMMARY

FIGURE B-5
Pg. 1 of 4

SAMPLE IDENTIFICATION

Project Number	19126505-2000	Sample Number	6
Borehole Number	C12-6	Sample Depth, m	5.0

TEST CONDITIONS

Test Method	B	Load Duration, hr	24
Oedometer Number	1(Calibrated June/2020)	Load Increment Ratio	1
Date Started	June 15, 2020		
Date Completed	June 30, 2020		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	2.53	Unit Weight, kN/m^3	16.37
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m^3	10.35
Area, cm^2	31.61	Specific Gravity, measured	2.725
Volume, cm^3	80.03	Solids Height, cm	0.981
Water Content, %	58.19	Volume of Solids, cm^3	31.00
Wet Mass, g	133.62	Volume of Voids, cm^3	49.04
Dry Mass, g	84.47	Degree of Saturation, %	100.2

TEST COMPUTATIONS

Stress σ_v' kPa	End of Primary Deformation[1] ΔH_{EOP} mm	Specimen Height[2] H_{EOI} cm	End of Primary Void Ratio[3] e_{EOP}	Average Height $(H_p+H_{EOI})/2$ cm	Time[1] t_{90} sec	Coefficient of Consolidation c_v cm^2/s	Modulus of Volume Compressibility m_v m^2/kN	Hydraulic Conductivity[4] k_v cm/s	Total Work w kJ/m^3
0	0.00	2.532	1.582	2.532					
9	0.06	2.528	1.576	2.530	540	2.51E-03	2.50E-04	6.16E-08	0
26	0.06	2.517	1.571	2.523	240	5.62E-03	1.12E-04	6.19E-08	0
34	0.01	2.515	1.565	2.516	217	6.20E-03	2.74E-04	1.66E-07	0
69	0.10	2.498	1.554	2.507	265	5.03E-03	1.24E-04	6.11E-08	0
137	0.17	2.469	1.530	2.483	346	3.78E-03	1.36E-04	5.03E-08	1
273	1.53	2.210	1.361	2.339	6000	1.93E-04	4.79E-04	9.08E-09	15
547	1.65	2.025	1.085	2.117	3937	2.41E-04	3.91E-04	9.25E-09	63
1095	1.15	1.887	0.948	1.956	2160	3.75E-04	9.73E-05	3.58E-09	117
547	-0.13	1.898	0.935	1.893					
137	-0.47	1.939	0.977	1.919					
34	-0.53	1.988	1.027	1.963					
9	-0.44	2.028	1.067	2.008					

Note:

[1] Root Time Method (Taylor, 1942).

[2] Specimen height corrected for apparatus deformation and presented for end of increment.

[3] Void ratio for unloading (i.e. rebound) calculated for the end of increment.

[4] Hydraulic conductivity calculated using coefficient of consolidation based on t_{90} values.

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

Sample Height, cm	2.03	Unit Weight, kN/m^3	18.06
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m^3	12.93
Area, cm^2	31.61	Specific Gravity, measured	2.725
Volume, cm^3	64.09	Solids Height, cm	0.981
Water Content, %	39.74	Volume of Solids, cm ^{3}	31.00
Wet Mass, g	118.04	Volume of Voids, cm ^{3}	33.09
Dry Mass, g	84.47		

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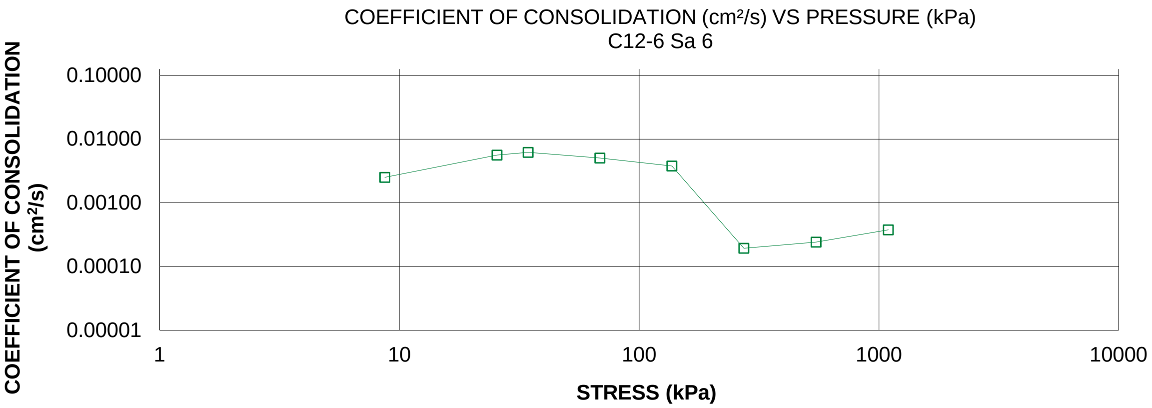
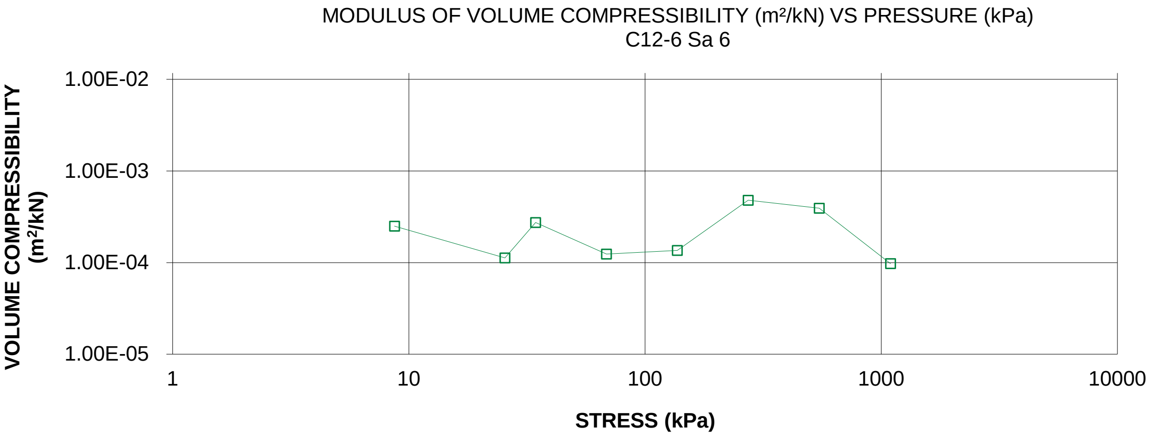
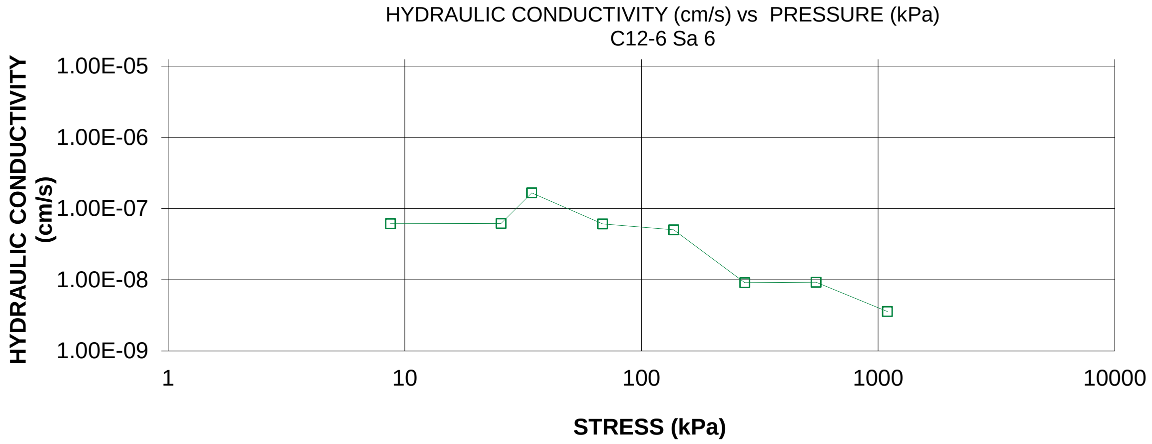
CONSOLIDATION TEST SUMMARY	**FIGURE B-5** **Pg. 2 of 4**







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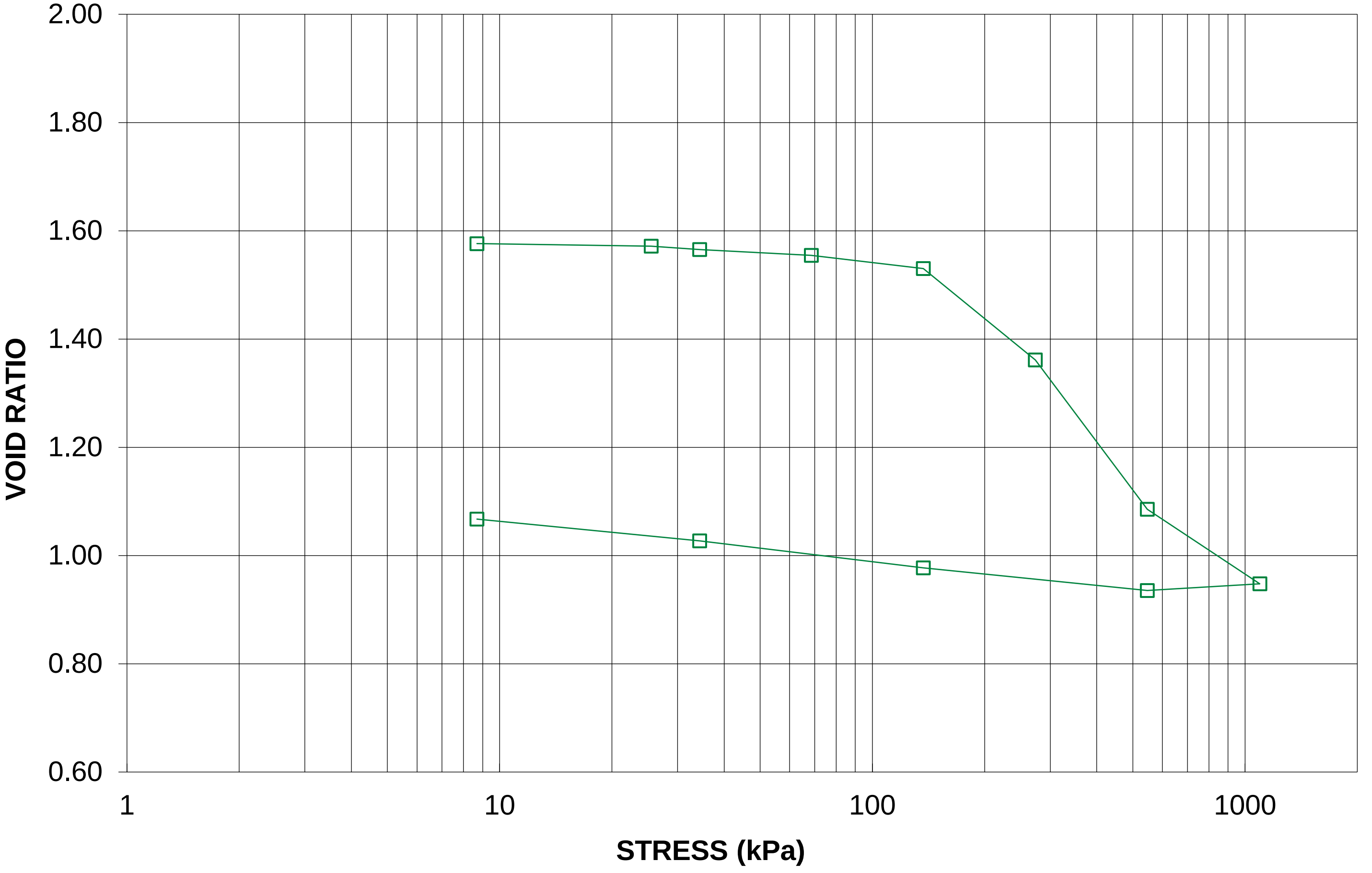
CONSOLIDATION TEST
VOID RATIO VS LOG STRESS

FIGURE B-5
Pg. 3 of 4

TOTAL WORK (kJ/m³/kN) VS PRESSURE (kPa)
C12-6 Sa 6



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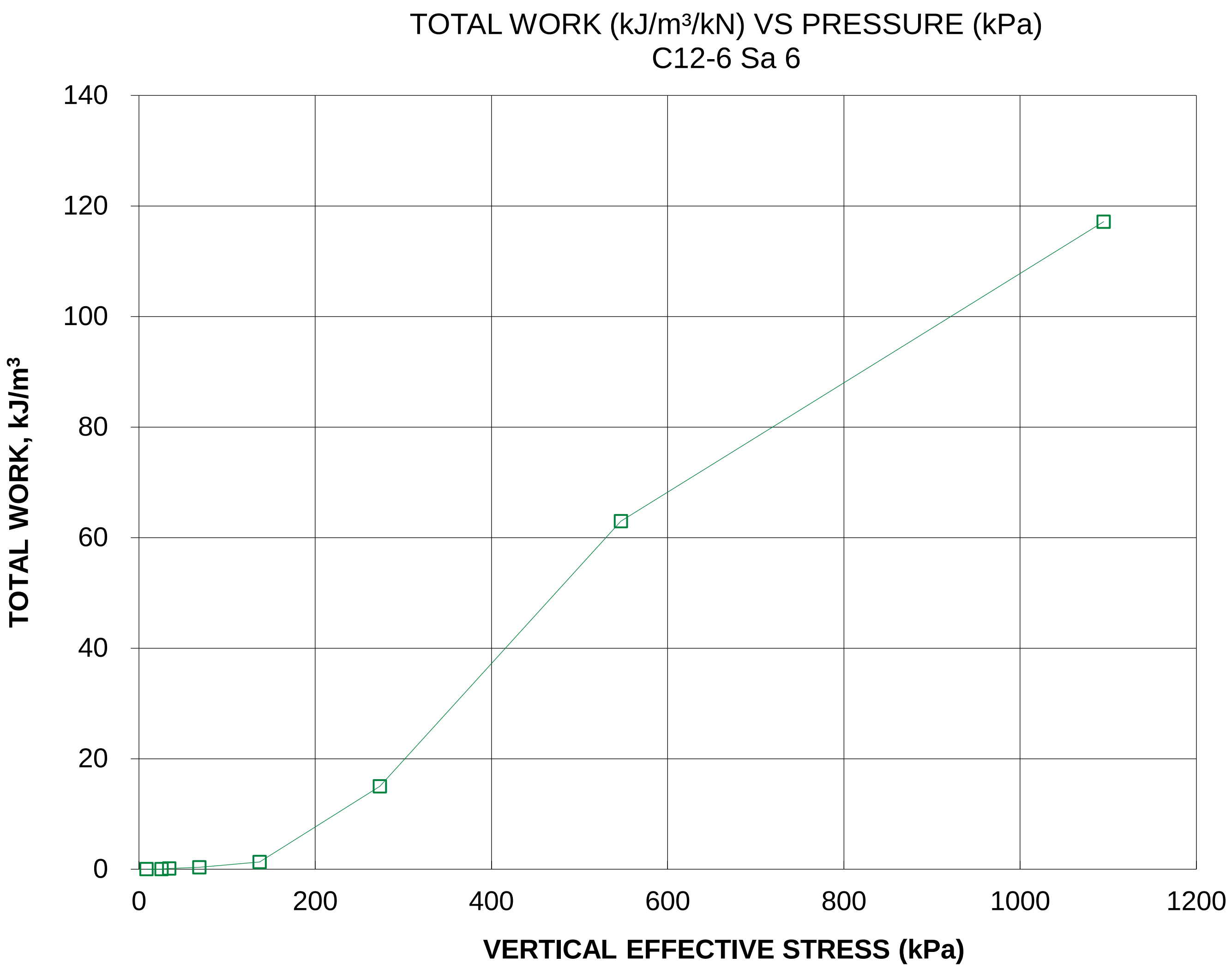
Project No. 19126505-2000

Prepared By: TG



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CONSOLIDATION TEST
TOTAL WORK VS STRESS

FIGURE B-5
Pg. 4 of 4

RESULTS OF ANALYSES OF SOIL

BV Labs ID		MWN864			MWN864			MWN865		
Sampling Date		2020/06/04			2020/06/04			2020/06/08		
COC Number		137483			137483			137483		
	UNITS	**C12-5 SA3**	**RDL**	**QC Batch**	**C12-5 SA3 Lab-Dup**	**RDL**	**QC Batch**	**RC-2 SA3**	**RDL**	**QC Batch**
Calculated Parameters										
Resistivity	ohm-cm	3300		6789098				14000		6789098
Inorganics										
Soluble (20:1) Chloride (Cl-)	ug/g	83	20	6792715				<20	20	6792715
Conductivity	umho/cm	306	2	6793003				74	2	6793003
Available (CaCl2) pH	pH	7.80		6792746				8.21		6792740
Soluble (20:1) Sulphate (SO4)	ug/g	<20	20	6792716				<20	20	6792716
Sulphide	mg/kg	<0.5 (1)	0.5	6816007	<0.5	0.5	6816007	0.5 (1)	0.5	6816007
Physical Testing										
Moisture-Subcontracted	%	38	0.30	6816006				2.8	0.30	6816006

RDL = Reportable Detection Limit
QC Batch = Quality Control Batch
Lab-Dup = Laboratory Initiated Duplicate
(1) Sample contained greater than 10% headspace at time of extraction.

BV Labs ID		MWN865			MWN866		
Sampling Date		2020/06/08			2020/06/07		
COC Number		137483			137483		
	UNITS	**RC-2 SA3 Lab-Dup**	**RDL**	**QC Batch**	**WR-1 SA7**	**RDL**	**QC Batch**
Calculated Parameters							
Resistivity	ohm-cm				770		6789098
Inorganics							
Soluble (20:1) Chloride (Cl-)	ug/g	<20	20	6792715	410	20	6792715
Conductivity	umho/cm	73	2	6793003	1310	2	6793003
Available (CaCl2) pH	pH				11.9		6792740
Soluble (20:1) Sulphate (SO4)	ug/g	<20	20	6792716	490	20	6792716
Sulphide	mg/kg				593 (1)	10	6816007
Physical Testing							
Moisture-Subcontracted	%				9.7	0.30	6816006

RDL = Reportable Detection Limit
QC Batch = Quality Control Batch
Lab-Dup = Laboratory Initiated Duplicate
(1) Detection limits raised due to dilution to bring analyte within the calibrated range. Sample contained greater than 10% headspace at time of extraction.

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