DRAFT FOUNDATION INVESTIGATION AND DESIGN REPORT

Extension of Fairbanks Creek Culvert (Site No. 46X-0298/CO), Township of Denison
Highway 17 and Municipal Road 55 West Junction Intersection Improvements
Ministry of Transportation, Ontario
Agreement No. 5019-E-0026, GWP 5032-19-00

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FOUNDATION INVESTIGATION REPORT

EXTENSION OF FAIRBANKS CREEK CULVERT (SITE NO. 46X-0298/CO), TOWNSHIP OF DENISON

HIGHWAY 17 AND MUNICIPAL ROAD 55 WEST JUNCTION INTERSECTION IMPROVEMENTS

MINISTRY OF TRANSPORTATION, ONTARIO

GWP 5032-19-00

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by AECOM Canada Ltd. (AECOM) on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services for the proposed Fairbanks Creek Culvert extension on Highway 17 east bound lane (EBL) alignment at STA 14+384 in the Township of Denison. The proposed work is part of the Highway 17 and Municipal Road 55 West Junction Intersection Improvements. The general location of the culvert is shown on the Key Plan on Drawing 1.

The Terms of Reference and the Scope of Work for the foundation investigation are outlined in MTO's Request for Proposal (RFP), dated May 13, 2020, and subsequent addenda. Golder's proposal for the associated foundation engineering services is contained in Section 7.7 of the AECOM Technical Proposal for this assignment. The work has been carried out in accordance with Golder's Project – Specific Supplementary QC Plan for foundation engineering services for this project, issued on January 12, 2021. The base plan showing the existing horizontal alignment and a drawing showing the existing vertical profile for Highway 17 (and existing culvert invert) were provided to Golder by AECOM in April 2021 and the General Arrangement (GA) for the culvert extension was not available at the time this report was prepared.

This report addresses the investigation carried out for the extension of the Fairbanks Creek Culvert at STA 14+384 only. Separate reports address the foundation investigations for the remaining circular culverts and Highway 17 embankment widening.

Existing subsurface information for this culvert location is available in the previous Foundation Investigation Report for Fairbanks Creek Culvert prepared by Golder under report number 11-1191-0007-07, dated May 6, 2015, GWP 156-98-00, Geocres No. 41I-325 (Golder, 2015) and in the Preliminary Investigation for the Fairbanks Creek Culvert prepared by MTO in January 1975, Geocres No. 41I-092 (MTO,1975).

2.0 SITE DESCRIPTION

The overall project consists of the intersection improvement of Highway 17 at the west junction of Sudbury Municipal Road 55. The existing 6.1 m wide by 3.1 m high by 65.3 m long concrete rigid frame box culvert structure, which was constructed in 1980, crosses the existing Highway 17 EBL at STA 14+384 and is to be extended to the south (downstream side) to accommodate the proposed embankment widening as part of the proposed intersection improvements. Based on the topographic survey provided by AECOM on March 8, 2021, the existing culvert inlet and outlet inverts are at approximately Elevations 238.7 m and 238.6 m, respectively. The highway grade at the culvert location is at approximately Elevation 243.9 m. The existing embankment slopes north and south of the culvert location are generally inclined at about 2 Horizontal and 1 Vertical (2H:1V) with concrete wingwalls present at both the inlet and outlet location. At the time of the current subsurface exploration field work (Winter 2021), the embankment side slopes were generally snow covered and no signs of deep-seated embankment slope instability were observed in the vicinity of the culvert. The ground surface conditions near the culvert outlet and south side of the existing embankment are shown on Photographs 1 to 3.

In general, the topography of this area consists of rolling terrain, numerous bedrock outcrops separated by low-lying swampland with areas of standing water and surficial organic soils. The land use in the general area includes residential developments with scattered rural farm use. The Fairbanks Creek Culvert is located within a low-lying swampland and the ground surface in the vicinity of the culvert extension (outside of the creek) varies between about Elevations 243 m and 241 m, with the creek water level near the outlet measured to be at Elevation 240.9 on January 7, 2021.

3.0 INVESTIGATION PROCEDURES

The investigation for the Fairbanks Creek Culvert at STA 14+384 was carried out on February 8, 2021, during which time one borehole (designated 21-03) was advanced along the shoulder of Highway 17 near the southwest corner of the existing culvert. Previous foundation investigations were carried for the Fairbanks Creek Culvert in 1975 and 2012, during which a total of three boreholes (designated C-5, 1 and 2) were advanced. The locations of the current and previous boreholes are shown on Drawing 1.

The current field investigation was carried out using a track mounted CME-55 drill rig supplied and operated by Landcore Drilling (Landcore) of Sudbury, Ontario. The borehole was advanced using 108 mm hollow stem augers. Similarly, the previous boreholes were advanced using hollow stem augers. Soil samples were generally obtained in the boreholes at 0.75 m and 1.5 m intervals of depth (up to 3 m intervals in the previous boreholes) using 50 mm outer diameter split-spoon samplers driven by an automatic or cathead hammer in general 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 current borehole was backfilled upon completion in accordance with Ontario Regulation 903 Wells (as amended) and capped at the roadway surface using cold patch asphalt.

The groundwater level inside the augers was observed during the drilling operations and is described on the Record of Borehole sheets provided in Appendix A and summarized in Section 4.3.

Field work for the current investigation 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, and Atterberg limits tests were carried out on selected soil samples. The geotechnical laboratory testing was completed according to ASTM and MTO LS standards, as applicable. 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 location, in station and offset, was measured in reference to the centreline alignment staked on the highway shoulder and was subsequently converted into MTM NAD 83 coordinates in AutoCAD. The ground surface elevation at the borehole location was surveyed by Golder, relative to the highway centreline at the culvert centreline, with the elevations provided by AECOM. 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	Location (MTM NAD 83 Zone 12)		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing	Easting		
21-03	5 137 564.6	278 167.4	243.8	15.9

The previous boreholes (1 and 2) drilled by MTO in January 1975 as part of the preliminary investigation for the Fairbanks Creek Culvert and previous borehole (C-5) drilled by Golder in June 2012 are also shown on Drawing 1. The boreholes were positioned relative to northing and easting coordinates determined from the locations shown on Sheet G1-4 (MTO, 1975) and Drawing 2 (Golder, 2015) provided in the GEOCRES reports. The approximate locations, Geodetic ground surface elevations and drilled depths of the boreholes from the previous investigations are as follows:

Borehole	Location (MTM NAD 83 Zone 12)		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing	Easting		
C-5 (2012)	5 137 607.5	278 122.5	242.8	24.4
1 (1975)	5 137 591.3	278 157.7	241.3	27.4*
2 (1975)	5 137 573.4	278 193.0	240.9	37.2**

*DCPT driven from ground surface to a depth of about 24.6 m below ground surface (Elev. 216.7 m)
**DCPT driven from a depth of about 37.2 m (Elev. 203.7 m) to 43.4 m below ground surface (Elev. 197.5 m)

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

As delineated in the NOEGTS[1] Mapping, the ground terrain in this section of Highway 17 is comprised of bedrock knobs, outcrops, and ridges within an undulating to rolling glaciolacustrine plain and alluvial plain containing areas of primarily silt with organic soil deposits. In the lower-lying glaciolacustrine plain and alluvial plain areas, the primary materials consist of wet silts, sands and clays, and the organic terrain deposit primarily consists of peat. The surface water drainage in the area varies from dry to wet, corresponding to areas of moderate to low relief.

Based on geological mapping by the Ministry of Natural Resources (Map 2542)[2], the site is underlain by rocks belonging to the Huronian Supergroup and Elliot Lake Group consisting of siltstone, wacke, and argillite. Areas of mafic and related intrusive rocks comprised of diabase sills, dykes, and related granophyre are also present in the vicinity of the site. Based on geological mapping by the Ontario Department of Mines (Map 2170) [3] this site area is characterized by extensive faults including the Murray Fault, which has been identified to run parallel to the alignment of Highway 17.

[1]Northern Ontario Engineering Geology Terrain Study, Ontario Geological Society Digital Map Reference Number 41ISW.

[2] Ministry of Natural Resources, 1991. Bedrock Geology of Ontario – West Central Sheet, Ontario Geological Survey - Map 2542

[3] Ontario Department of Mines, 1969. Sudbury Mining Area, Sudbury District, Map 2170.

4.2 Subsurface Conditions

The detailed subsurface soil and groundwater conditions as encountered in the boreholes advanced during the current and previous investigations, together with the results of the laboratory tests carried out on selected soil samples are presented on the Record of Borehole sheets in Appendix A. The details of the laboratory tests for the current and previous investigation are provided in Appendices B and C, respectively. The results of the in-situ field tests (i.e., SPT ‘N’-values and undrained shear strengths from the field vanes) as presented on the Record of Borehole sheets and in Section 4.2 are uncorrected. The stratigraphic boundaries shown on the Record of Borehole sheets are inferred from non-continuous sampling, observations of drilling progress, and the results of SPTs and in-situ testing. These boundaries, 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. 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 Peat

Peat/muskeg (0.7 m to 0.9 m thick) was encountered at ground surface in Boreholes 1 and 2 during the previous investigation in 1975 (prior to the highway embankment construction), and below the silty sand to gravelly sand (fill) in Borehole C-5. The peat was encountered between Elevations 239.4 m and 241.3 m.

One SPT ‘N’-value measured at the interface between the peat and the overlying fill was 5 blows per 0.3 m of penetration, suggesting a firm consistency.

4.2.2 Asphalt

A layer of asphalt (100 mm thick) was encountered at the Highway 17 shoulder surface in Borehole 21-03.

4.2.3 Silty Sand to Sand and Gravel (Fill)

A 4.4 m and 3.4 m thick layer of silty sand to sand and gravel (fill) was encountered below the asphalt and at ground surface in Borehole 21-03 and C-5. The top of the fill deposit was encountered at Elevation 243.7 m and 242.8 m in Borehole 21-03 and C-5, respectively. In Borehole C-5, the upper 1.2 m of fill consisted of silty sand intermixed with blast rock. In Borehole 21-03, split-spoon refusal was encountered at 1.0 m depth (within potentially frozen soil) and auger grinding was encountered between 0.8 m and 2.9 m depth, suggesting the potential for obstructions within the fill.

The SPT ‘N’-values measured within the fill generally ranged from 11 blows to 37 blows per 0.3 m of penetration, indicating a compact to dense state of compactness. One SPT test encountered refusal after 0.1 m of penetration, suggesting the fill was frozen or may indicate potential obstructions (e.g., blast rock) within the fill.

Grain size distribution testing was carried out on one sample of the sand fill and the results are presented on Figure B-1 in Appendix B. The natural moisture content measured on two samples of the fill were 5% and 16%.

4.2.4 Sandy Clayey Silt

A 1.1 m thick cohesive deposit of wet, sandy clayey silt was encountered underlying the fill materials in Borehole 21-03 at Elevation 239.3 m.

One SPT ‘N’-value measured within the sandy clayey silt deposit is 2 blows per 0.3 m of penetration, suggesting a soft consistency.

4.2.5 Silty Clay to Clay

A cohesive deposit of silty clay to clay was encountered below the peat in Boreholes C-5, 1, and 2; and below the sandy clayey silt in 21-03. The top of the silty clay to clay was encountered between Elevation 238.2 m and 240.4 m, with a thickness ranging from 10.7 m to 18.3 m. Occasional clayey silt to silt seams/laminations were encountered within the silty clay to clay deposit. Borehole 21-03 was terminated within the deposit after exploring the layer for 10.3 m.

The SPT ‘N’-values measured within this deposit range from 0 blows (i.e., weight of hammer) to 7 blows per 0.3 m of penetration. In-situ field vane tests carried out within this deposit measured undrained shear strengths typically ranging from 29 kPa to 80 kPa with a calculated sensitivity between about 3 and 10. The field vane test results indicate that the deposit has a generally firm to stiff consistency; however, four field vane tests in the upper portion of the deposit measured an undrained shear strength greater than 100 kPa (limit of the measuring device) corresponding to a very stiff consistency.

Atterberg limits tests were carried out on 14 samples of the cohesive deposit and indicate liquid limits of about 35% to 55%, plastic limits of about 19% to 30%, and plasticity indices of about 14% to 29%. The results of the Atterberg limits tests from the current investigation (Borehole 21-03) are shown on the plasticity chart on Figure B-2 in Appendix B and indicate that the material is classified as silty clay of intermediate plasticity. The results of the Atterberg limits tests from the previous investigations (Boreholes C-5, 1 and 2) are shown on the Record of Borehole sheets in Appendix A and on Figures C-1 and C-3 in Appendix C, indicating the deposit ranges from a silty clay of intermediate plasticity to clay of high plasticity.

Five grain size distributions were carried out on samples of the silty clay to clay deposit and the results are shown on the Record of Borehole sheets in Appendix A and summarized on Figure C-2 in Appendix C.

The natural moisture content measured on 17 samples of the silty clay to clay deposit range between about 32% and 65%.

4.2.6 Silt to Silty Sand

During the previous investigations, a deposit of grey silt to silty sand was encountered below the silty clay to clay deposit in Boreholes C-5, 1 and 2. The surface of the deposit was encountered between Elevations 228.0 m and 221.7 m. The deposit was 11.3 m thick in Borehole 2 and was not fully penetrated in Boreholes C-5 or 1 after exploring the deposit for 9.6 m and 11.6 m, respectively.

The SPT ‘N’-values measured within this deposit range between 1 blow and 33 blows per 0.3 m of penetration, indicating a very loose to dense relative density.

The results of grain size distribution tests carried out on six samples of the deposit from the previous investigations are presented on the Record of Borehole sheets in Appendix A. The grain size distributions of two samples of the silt to silt and sand deposit from the 2012 investigation are also presented on Figure C-4 in Appendix C.

An Atterberg limits test completed on a sample of the silt in Borehole C-5 indicates that the material is non-plastic.

The natural moisture content measured on six samples of the deposit range between about 10% and 32%.

4.2.7 Sand and Gravel

During the previous investigation in 1975, a deposit of sand and gravel was encountered below the silt to silty sand deposit in Borehole 2. The surface of the deposit was encountered at Elevation 210.4 m and the borehole was terminated within this deposit after exploring for 6.7 m. A Dynamic Cone Penetration Test (DCPT) was advanced 6.2 m from the bottom of Borehole 2 to Elevation 197.5 m (corresponding to a depth of 43.4 m below ground surface) at which depth effective refusal (greater than 100 blows/0.3 m of penetration) was measured.

The SPT ‘N’-values measured within this deposit range between 28 blows and 47 blows per 0.3 m of penetration, indicating a compact to dense relative density.

A grain size distribution test was carried out on one sample of the deposit and the result is shown on the Record of Borehole sheet in Appendix A.

4.3 Groundwater Conditions

The groundwater levels measured inside the hollow stem augers and/or in the open boreholes relative to ground surface (upon completion of drilling) are summarized below.

Borehole No.	Depth Below Ground Surface to Groundwater Level (m)	Approximate Groundwater Elevation (m)	Notes
21-03	3.2	240.6	Inside augers (unstabilized)
C-5	4.0	238.8	-
1	-(2.4)	243.7	Artesian conditions noted to be encountered in sandy silt to silty sand soil at depth of 25.7 m (El. 215.6 m).
2	0	240.9	-

The water level in Fairbanks Creek was measured at Elevation 240.9 m by others on January 7, 2021. Groundwater and creek water levels in the area are subject to seasonal fluctuations and variations due to precipitation events.

4.4 Analytical Laboratory Testing Results

Analytical testing was carried out on a sample of the sandy clayey silt recovered from Borehole 21-03. 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)	Soluble Chloride (Cl) Content (µg/g)	Sulphide (mg/kg)	pH
21-03	7	4.6-5.2	2,000	502	<20(1)	310	<0.5 (1)	7.12

(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. Matthew Thibeault, P.Eng. This report was prepared by Mr. Tibor Berecz, EIT, and the technical aspects were reviewed by Mr. Matthew Thibeault, P.Eng., a geotechnical engineer. Mr. Kevin Bentley, P.Eng., an MTO Foundations Designated Contact and Associate with Golder, conducted an independent quality control review of this report.

Signature Page

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https://golderassociates.sharepoint.com/sites/128666/project files/6 deliverables/foundations/draft/r2-fairbank creek culvert extension/rev b/20253807-r02-revb-fairbanks creek culvert fidr draft 25aug_2021.docx

PART B

FOUNDATION DESIGN REPORT

EXTENSION OF FAIRBANKS CREEK CULVERT (SITE NO. 46X-0298/CO), TOWNSHIP OF DENISON

HIGHWAY 17 AND MUNICIPAL ROAD 55 WEST JUNCTION INTERSECTION IMPROVEMENTS

MINISTRY OF TRANSPORTATION, ONTARIO

GWP 5032-19-00

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides Foundation design recommendations for the Fairbanks Creek Culvert extension along Highway 17 at Station 14+384. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the subsurface explorations. The discussion and recommendations presented are intended to provide the designer with sufficient information to assess the feasible foundation alternatives and carry out the design of the culvert extension to accommodate the embankment widening, as required. This Foundation Investigation and Design Report, including the discussion and recommendations, are intended for the use of the MTO and shall not be used or relied upon for any other purpose or by any other parties, including the construction or design-build contractor. The contractor must make their own interpretation based on the factual data in the Foundation Investigation Report (Part A) of 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

Golder was retained by AECOM to provide foundation engineering services for the extension of the Fairbanks Creek Culvert crossing the Highway 17 EBL alignment at STA 14+384. The existing 6.1 m wide by 3.1 m high by approximately 65 m long concrete rigid frame box culvert was constructed in 1980. The invert of the existing culvert is at Elevation 238.7 m and 238.6 m at the north (inlet) and south (outlet) side of the culvert, respectively. The existing Highway 17 EBL embankment at the culvert is generally up to about 3 m high relative to the existing ground surface beyond the embankment toe / ditch area. Based on conversations with AECOM, we understand that the existing embankment will be extended by approximately 3.5 m to accommodate the proposed Highway 17 EBL widening to accommodate a new acceleration lane; however, a widening of 5 m may be considered as the design progresses.

A concrete box or open footing culvert extension could be considered as feasible alternatives; however, from a foundation perspective, a box culvert is preferred to match the existing foundation type and limit the depth of the proposed excavation for foundations. The additional excavation depth for open footings (below frost depth) will result in more rigorous dewatering and temporary shoring efforts. The artesian groundwater conditions encountered in the granular layer below silty clay deposit will also pose a higher risk to foundation subgrade stability during construction if an open footing option is considered. In addition, an open footing culvert increases the risks associated with the compressible and relatively low resistance foundation soils and will likely require additional installation time during construction.

For the box culvert extension, a cast-in-place or precast extension could be considered, a comparison of advantages and disadvantages for each culvert alternative is provided in Table 1 following the text of the report. Based on conversations with the designer, we understand that a cast-in-place open footing culvert extension may still be preferred given the constructability concerns related to creek flow diversion / dewatering and to expediate the overall construction schedule depending on the availability/delays associated with the manufacture of precast units. Therefore, discussions on a box or open footing culvert extension are provided in the following sections.

6.1.1 Consequence and Site Understanding Classification

As Highway 17 carries a relatively large volume of traffic, and has the potential to impact alternative transportation corridors, a “typical consequence level” is considered appropriate for the foundation design at this site, as outlined in Section 6.5 of the Canadian Highway Bridge Design Code (CHBDC 2019) and its Commentary. Further, given the scope of work of the foundation field investigation and laboratory testing program, as presented in Sections 3.0 and 4.0 and Section 6.1.2, a “typical degree of site and prediction model understanding” has been utilized. Accordingly, the appropriate corresponding ULS and SLS consequence factor, Ψ, and geotechnical resistance factors, Φ_{gu} and Φ_{gs}, from Tables 6.1 and 6.2 of the CHBDC have been used for design.

6.1.2 Interpretation of Compressible Cohesive Deposit

This foundation report is specific to the Fairbanks Creek Culvert extension; however, Golder also prepared a separate Foundation Investigation and Design Report for the overall Highway 17 Embankment Widening (encompassing the area of Fairbanks Creek Culvert) as part of the overall assignment. A detailed assessment of the foundation engineering soil parameters for the area of the Fairbanks Creek Culvert was developed as part of the Highway 17 Embankment Widening FIDR and a summary is presented in Figure 1. Figure 1 includes all data and laboratory testing from the surrounding boreholes advanced as part of the overall foundation investigation for the Highway 17/MR 55 intersection improvement and widening project (including previous investigations in the area). The reference data/information used to develop Figure 1 are included in the Highway 17 Embankment Widening FIDR. Figure 1 is used to model the behaviour of the cohesive deposits for the stability and settlement models at the Fairbanks Creek Culvert location.

6.2 Embankment Stability

Based on our site observations at the time of the field investigation and a review of the available satellite images, the existing highway embankment in the culvert area appears to be performing satisfactorily with no visual evidence of instability (i.e., soil movement) on the southern embankment side slope and no tension cracks near the embankment crest that would be indicative of instability.

Referring to the base topographic plan drawings provided by AECOM, the existing south embankment side slope in the vicinity of the culvert is inclined at about 2H:1V to 3H:1V, with locally steeper slopes (1H:1V) directly adjacent to and above the headwall of the culvert. From discussions with AECOM, we understand that the embankment at this location will be widened by approximately 3.5 m with a final side slope inclined at 2H:1V or shallower.

6.2.1 Methodology

The stability analyses were carried out using the embankment geometry at approximately Sta. 14+380 along Highway 17 EBL, which was based on a cross section developed from the topographic drawings provided by AECOM and assuming a 3.5 m to 5 m embankment widening. The subsurface conditions are based on the closest boreholes advanced near the culvert on the south side. Due to the transient nature of traffic loading, traffic loads have not been included in the slope stability analyses which is considered to be typical practice considering the target factors of safety. The stability analyses assume that the organic deposits within the proposed new embankment widening footprint have been removed and replaced in general accordance with OPSD 203.020 (Embankments Over Swamp) prior to construction of the new embankment.

The limit equilibrium analyses were performed using the commercially available program GeoStudio 2021 (Version 11.0.1.21429), produced by GEOSLOPE International Ltd., by employing the Morgenstern-Price method to assess the short-term (undrained) conditions and long-term (drained) conditions. For all analyses, the Factor of Safety (FoS) of numerous potential failure surfaces was computed in order to establish the minimum FoS against global instability. The FoS is defined as the ratio of the forces tending to resist failure to the driving forces tending to cause failure.

For the purpose of the stability analysis, the FoS is equal to the inverse of the product of the consequence factor, Ψ, and the geotechnical resistance factor, Φ_{gu} (i.e., FoS = 1 / [Ψ * Φ_{gu}]). A minimum FoS of 1.33 in the short-term condition is required, based on a typical consequence level and a typical degree of site understanding, as per the CHBDC (2019). Similarly, a minimum FoS of 1.54 in the long-term condition is required.

For the analyses, it is assumed that the new embankment fill is free-draining, and that the groundwater level is located near the bottom of the fill/top of the native subgrade (i.e., measured creek water level). The stability analysis was carried out to check if the proposed embankment widening design meets the required minimum FoS at the culvert location in both short-term and long-term conditions.

6.2.1.1 Parameter Selection

The founding soils at the location of the culvert include a combination of organic soils, cohesive deposits (clayey silt to silty clay), and granular soils. A summary of the foundation engineering soil parameters employed in the stability models for the cohesive deposit encountered (i.e., clayey silt to silty clay) is presented on Figure 1. The existing and proposed new granular fill was assumed to have an effective friction angle of 35° with a compacted unit weight of 21 kN/m^3. For the granular foundation soils, effective stress parameters were employed in the analyses assuming drained conditions for both short-term and long-term analyses. For cohesive deposits, total stress or effective stress parameters were employed in the analyses, as appropriate.

The effective stress parameters (effective friction angle and effective cohesion) for the organic and granular soils were estimated from the measured in-situ compactness and laboratory results combined with engineering judgement based on experience in similar soil conditions.

As summarized in Figure 1, the total stress parameters (i.e., mobilized undrained shear strength) for the cohesive soils were assessed based on the results of in-situ field vane shear tests, inferred from the laboratory consolidation test results, and estimated from correlations with the SPT results and other laboratory test data (i.e., natural water content), where appropriate. For the consolidation tests performed in the clayey soils, the following correlation proposed by Mesri (1975) was employed to estimate the mobilized undrained shear strength:

$$s_{u(FV-uncorrrected)} = 0.22\sigma_p{}'$$

where: $s_{u(FV-uncorrrected)}$ = average mobilized undrained shear strength (kPa)

$\sigma_p{}'$ = preconsolidation pressure (kPa)

With respect to the overconsolidated cohesive crust encountered below the fill or near ground surface, the design line for the mobilized undrained shear strength presented on Figure 1 was adjusted to account for potential fissuring after Tavenas and Leroueil (1980).

The Standard Test Method for Field Vane Shear Test in Saturated Fine-Grained Soils (ASTM D2573) states that the peak undrained shear strength from the field vane test needs to be multiplied by a vane correction factor (μ) to give a mobilized field value of undrained strength for geotechnical analysis. It also includes the following expression:

$$\tau_{mobilized} = \mu_v (s_u)_{FV}$$

where: $\tau_{mobilized}$ = the mobilized shear strength ($s_{u(mob)}$) for geotechnical analysis

μ_v = an empirical correction factor that has been related to plasticity index (PI) and/or liquid limit (w_L) and/or other parameters based on back calculation from failure case history records of full-scale projects.

Given the presence of clayey silt to silt laminations in Borehole 21-03, 1, and 2 (and the surrounding boreholes), the silty clay to clay deposit is considered to be varved. For a horizontally layered varved clay stratum, a maximum correction factor (μ_{max}) can applied over a range of failure surface angles relatively close to the horizontal (e.g., approximately $i = 0^o$ +/-5^o to +/-15^o), while the minimum correction factor (i.e., μ=1 or no correction) is applied over a range of failure surface angles oblique to the horizontal (e.g., - $45^o < i > +45^o$). Ladd and Foott (1977) suggest that the near horizontal failure surface mobilizing the minimum shear strength should (i.e., along-shear) be defined by $i = 0^o$ +/-10^o, while the portions of the slip surface oblique to the horizontal mobilizing the maximum shear strength (i.e., cross-shear) be defined by $i = 30^o$ to 60^o.

For the stability analyses presented herein, a simplified μ_{avg} correction factor was applied to the undrained shear strength design line, where applicable, to account for the affect of varves on the lower shear strength mobilized 'along-shear' in the field. Figure 2 presents data available from literature for both non-varved and varved clay sites and a proposed correlation based on plasticity index to select a μ_{avg}. A μ_{avg} correction factor of 0.85 was used for both the stability and settlement analyses.

The effective parameters for the cohesive soils were assessed based on a combination of engineering judgement and empirical correlations. In particular, the effective friction angle was based on correlations to Atterberg limit testing (i.e. [Mitchell, 1993], [Ladd, 1977] and [Kulhawy and Mayne, 1990]). The effective cohesion was conservatively assumed to be negligible.

6.2.1.2 Results of Unmitigated Stability Analysis

The results of the global stability analyses carried out at STA 14+380 (i.e., immediately adjacent to the structural culvert) are presented on Figures 3 to 5 for the short-term total stress analysis, short-term effective stress analysis, and long-term effective stress analysis, which calculated Factors of Safety equal to 1.98, 1.57, and 1.84, respectively. The proposed design includes a 3.5 m embankment widening; however, based on discussions with AECOM, we further understand that a 5 m embankment widening may be considered as the design progresses. Therefore, the stability analyses were carried out for a 5 m widening, which is considered to provide conservative results for the currently proposed 3.5 m widening. Based on the results of the stability analysis, a widening of 5 m with a side slope of 2H:1V will satisfy the global stability requirements outlined in the CHBDC. We further understand that 3H:1V embankment slopes might be considered for the widening, which would further increase the global stability of the proposed embankment.

6.2.2 Culvert Settlement

The following sections outline the methods used to carry out the analyses, interpretation of the geotechnical parameters and results of analysis associated with settlement.

6.2.2.1 Methodology

The settlement performance criteria for embankment widenings are outlined in Section 1.3 of the MTO Foundation Guideline, "Embankment Settlement Criteria for Design", dated July 2010 (MTO Guideline, 2010). The guideline indicates that the total settlements for an embankment / structure transition over a 20-year period following completion of construction for a "freeway" shall not exceed the limits in the table below.

Distance from Transition Point	20-year Post Construction Settlement (mm)			
	0-20 m	20-50 m	50-75 m	>75 m
Freeways	25	50	75	100

Various settlement mitigation alternatives and the methodology used for the settlement assessment are discussed in the separate Embankment Widening FIDR for this section of highway as part of the current assignment. The recommended settlement mitigation alternative from a foundations perspective for the widening was the use of a preload period in combination with lightweight fill. As the Fairbanks Creek Culvert will be within the area treated with lightweight fill, the discussions herein are limited to the unmitigated and preferred mitigation (i.e., 6-month preload period with subsequent lightweight fill replacement) settlement results. In addition, the settlement results provided herein are for a 3.5 m widening, if a 5 m widening is deemed necessary, Golder should be provided the opportunity to review and revise the analyses to confirm estimated settlement magnitudes and durations, as appropriate.

6.2.3 Results of Unmitigated Settlement Analysis

For the settlement analyses, the proposed embankment widening of 3.5 m (to match existing Highway 17 grade), and approximate ramp alignment/interchange grading was modelled as external loads based on the conceptual design drawings provided by AECOM. Settlements were estimated along the proposed Highway 17 embankment widening shoulder (south side) at the approaches to the culvert structure (parallel to Highway 17), as well as perpendicular to Highway 17 along the culvert centreline alignment.

The results of the settlement analyses along the Highway 17 embankment widening shoulder (south side) are presented in the table below.

Distance from Transition Point	20-year Post Construction Settlement (mm)			
	0-20 m	20-50 m	50-75 m	>75 m
Fairbanks Creek Culvert	~115	~225	~225	~225

Along the culvert centreline alignment, the total maximum settlement is anticipated to be approximately 75 mm (near widened shoulder), with a maximum differential settlement of approximately 50 mm (relative to the existing culvert). The structural engineer/precast culvert manufacturer will need to check if these settlements are tolerable for the proposed culvert structure extension and connection to the existing culvert. Regardless, the total settlement of the proposed Highway 17 widening at the culvert location exceeds the recommended tolerable value of 50 mm for embankment widenings (MTO Guideline, 2010), as discussed in the separate Highway 17 Embankment Widening FIDR.

Based on the calculated settlement results along the Highway 17 shoulder from the transition to the Fairbanks Creek Culvert and for the general embankment widening, the settlement is anticipated to exceed the limits established by MTO for a freeway; therefore, settlement mitigation will be required.

6.2.4 Results of 6-month (i.e., 180 day) Preload Period with Earth Fill followed by Lightweight Fill Replacement

A combination of a 6-month preload period with earth fill followed by a partial replacement of the earth fill with lightweight fill (cellular concrete) was identified as the recommended settlement mitigation alternative for the embankment widening. For the purpose of the analysis, cellular concrete with a unit weight of 5 KN/m^3 was assumed. A summary of the settlement results for the preload followed by an up to 3 m thick cellular concrete replacement are provided in the table below.

Distance from Transition Point	20-year Post Construction Settlement (mm)			
	0-20 m	20-50 m	50-75 m	>75 m
Fairbanks Creek Culvert	~25	~50	~50	~50

Based on the results of the settlement analyses, the post construction settlement tolerances near the culvert structure are achieved for the proposed mitigation option.

Should a preload with lightweight fill be selected as the preferred settlement mitigation alternative, Golder could prepare an example specification to supply and install the cellular concrete for incorporation in the Contract.

When the general arrangement drawing and high water levels for Fairbanks Creek become available, Golder will need to review the design assumptions and practical limits that lightweight fill can be placed immediately adjacent to the culvert (e.g., effective drainage) and along the embankment widening. If an alternative mitigation option is selected for the overall embankment widening, Golder will need to check that settlements along and adjacent to the culvert structure transition are tolerable.

6.3 Geotechnical Axial Resistance

For a 6.1 m wide box culvert extension at this site, the culvert should be designed on the basis of a factored geotechnical axial resistance at Ultimate Limit States (ULS) of 100 kPa based on the culvert being founded on a properly prepared subgrade/granular bedding (as discussed in Section 6.6). Similarly, for an open footing culvert extension with 1 m to 3 m wide footings founded at the frost depth below creek bottom, the footings should be designed on the basis of a factored geotechnical axial resistance at Ultimate Limit States (ULS) of 85 kPa based on the footings being founded on a properly prepared subgrade (as discussed in Section 6.6). The geotechnical resistance is applicable for loads that will be applied perpendicular to the base of the culvert. Where loads are not applied perpendicular to the base of the culvert, inclination of the loads should be taken into account in accordance with Section 6.10.2 and Section C6.10.5 of the CHBDC and its Commentary.

With regards to Serviceability Limit States (SLS), the loading and resistance of the foundation soils below the culvert (and the associated total settlement) at the culvert location will be governed by the design height of the overlying embankment fill, and more specifically adjacent embankment fill (for the proposed widening of the EBL embankment), sequencing during construction, and the chosen settlement mitigation option. As such, it is recommended that the structural engineer exercise caution when assessing/utilizing the values of the geotechnical axial resistance at SLS in the design of the culvert. As discussed in Section 6.2.4 for the proposed mitigation option, the culvert is anticipated to experience a total of about 50 mm of settlement assuming a 3.5 m embankment widening east and west of the culvert extension location. Approximately 25 mm of the settlement (and 25 mm differential settlement relative to existing culvert) will occur during the 6-month preload period (with extension in place) and the remaining 25 mm of settlement (and additional 25 mm differential settlement) is expected to occur during the 20 years following construction.

6.3.1 Frost Protection

The estimated frost penetration depth in the vicinity of the Highway 17 and MR 55 intersection is 2.1 m, as interpreted from OPSD 3090.100 (Foundation Frost Penetration Depths for Northern Ontario). As the majority of the culvert extension is to be founded below the estimated 2.1 m depth of frost penetration for this site (within the roadway) and the recommended granular backfill materials (including the existing granular embankment fill) are classified as having a low susceptibility to frost heaving (as per the MTO Pavement Design and

Rehabilitation Manual), a frost taper is not considered to be required as per OPSD 803.030 (Frost Treatment – Pipe Culverts).

Where the risk of differential heaving at the culvert ends is high, which is not considered the case at this culvert site due to significant size of the culvert and water levels in the creek, consideration can be given to sub-excavating and replacing the frost susceptible soils with non-frost susceptible fill materials (i.e., OPSS.PROV 1010 Granular ‘A’ or Granular ‘B’ Type II) and/or incorporating polystyrene insulation into the design. However, these measures are typically not considered to be practical or cost effective. As such, measures to mitigate the risk of differential heaving occurring at the culvert ends at these sites are not considered necessary.

6.3.2 Resistance to Lateral Loads/Sliding Resistance

Resistance to lateral forces/sliding resistance between the base of the concrete and the granular fill/bedding placed following sub-excavation of organic deposits for a box extension or native silty clay for an open footing extension should be calculated in accordance with Section 6.10.4 of the CHBDC. The following summarizes the unfactored values of coefficient of friction for the interface materials.

Interface Materials	Coefficient of Friction
Precast Concrete Box Culvert on Compacted Granular ‘B’ Type II or Granular ‘A’	tan δ = 0.45
Cast-in-Place Concrete Box Culvert on Compacted Granular ‘B’ Type II or Granular ‘A’	tan δ = 0.55
Cast-in-Place Opening Footing Culvert on Native Silty Clay	tan δ = 0.35

6.4 Lateral Earth Pressures

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

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

- Select, free draining granular fill meeting the specifications of OPSS PROV. 1010 (Aggregates) Granular ‘A’ or Granular ‘B’ Type II should be used as backfill behind the culvert walls, and head wall, if applicable, and on top of the culvert for a thickness of 300 mm. Backfill should be placed in a maximum of 200 mm loose lift thickness and nominally compacted. Weep holes should be installed in the walls, as appropriate, to provide positive drainage of the granular backfill. Compaction (including type of equipment, target densities, etc.) should be carried out in accordance with OPSS.PROV 501.
- Granular fill (where utilized) should be placed in a zone with the width not less than 2.1 m behind the back of the culvert. The pressures are based on the proposed embankment fill materials and the following parameters (unfactored) may be used:

Fill Type	Unit Weight	Coefficients of Static Lateral Earth Pressure	
		At-Rest, K_o	Active, K_a
Granular 'A'	22 kN/m^3	0.43	0.27
Granular 'B' Type II	21 kN/m^3	0.43	0.27

If the culvert structure (or head wall/wing wall) allows for lateral yielding, active earth pressures may be used in the foundation design. If the culvert structure does not allow lateral yielding, at-rest earth pressures should be assumed for foundation design. The movement to allow active pressures to develop within the backfill, and thereby assume a restrained structure, may be taken as per Figure C6.27 of the CHBDC Commentary.

6.5 Seismic Requirements

As per Section 7.5.8.1 of the CHBDC, buried structures shall be designed to resist inertial forces associated with a seismic event having a 2% chance of exceedance in 50 years (i.e., 2,475-year return period). Using the information obtained from the NRCan (2015) Hazard Calculator for the culvert site located at latitude 46.377296° and longitude -81.346209°, a peak ground acceleration (PGA) of 0.057 g, where "g" is the acceleration due to gravity (9.81 m/s^2), was obtained for a return period of 2,475 years.

As further indicated in Section 7.5.8.1 of the CHBDC, a full seismic analysis is only required for buried structures where: the site is classified as Site Class F "Other Soils" (i.e., liquifiable, highly organic, highly plastic, etc.) in accordance with Table 4.1 of the CHBDC; and the design spectral response acceleration, Sa(0.2), is greater than 0.7 g for a 2,475-year return period. Based on the subsurface conditions encountered in Borehole 21-03, the Fairbanks Creek Culvert may be classified as Site Class "E" in accordance with Table 4.1 of the CHBDC. Using the information from the NRCan (2015) Hazard Calculator, a spectral acceleration [Sa(0.2)] of 0.098 g for a 2,475-year return period was obtained for the culvert site. As such, a full seismic analysis is not required.

6.6 Constructions Considerations

6.6.1 Temporary Excavations / Support Systems

All excavations must be carried out in accordance with Ontario Regulation 213, Ontario *Occupational Health and Safety Act* for Construction Projects (OHSA), as amended.

Based on the encountered subsurface conditions (compact to dense sand to sand and gravel fill and stiff to very stiff silty clay) and anticipated excavation extents/depths (typically less than 2.5 m below adjacent ground surface but up to 5.5 m below road shoulder ground surface) required to facilitate the box culvert extension installation, temporary open cut excavations are considered feasible. For an open footing culvert, assuming the base of the footings are founded at about Elevation 236.5 m (i.e. below frost depth from the creek bottom), the anticipated excavation depths will be about 4.5 m below adjacent ground surface and up to 7.5 m below the road shoulder ground surface, making this option more challenging as it will require a more robust temporary support system. Excavations for the culvert extension are anticipated to extend through a portion of the existing granular embankment fill materials (i.e., within the existing shoulder) and the upper portion of the native soil deposits. The granular fill and native soils within the anticipated excavation depths can be classified as Type 3 soil above the groundwater table and Type 4 soil below the groundwater table as per the OHSA. Temporary open-cut excavations in Type 3 and Type 4 soils can be sloped no steeper than 1H:1V and 3H:1V, respectively.

Temporary shoring systems are likely required (especially adjacent to Highway 17 to accommodate the connection detail to the existing culvert and possibly to remove the existing wing walls) and could consist of sheet piles and/or solider piles and lagging and could be incorporated into a cofferdam enclosure for dewatering purposes (as discussed in the next section). Consideration should be given to the potential for cobble and/or boulder sized obstructions (e.g., blast rock), as identified and inferred to be present within the existing embankment fill materials and/or near the native soil interface. Horizontal support to the system could be in the form of struts, walers, rakers, or anchors if a cantilevered system is not sufficient. Temporary protection/ dewatering systems (if utilized) are the responsibility of the Contractor and should be designed and constructed in accordance with OPSS.PROV 539 (Temporary Protection Systems), as amended by SSP 105S09. Temporary protection systems should be designed to Performance Level 2 for any excavation adjacent to an existing roadway. Design of the temporary support system should include an evaluation of base stability, soil squeezing stability, and hydraulic uplift stability, as defined in the Canadian Foundation Engineering Manual (CFEM 2006). Special consideration should be given the artesian groundwater conditions identified to be present in Borehole 1, especially if a deeper open footing foundation is being considered.

Consideration could be given to either partial or full removal of the temporary protection system(s) upon completion of construction, as noted in OPSS.PROV 539. As noted above, there is a risk that the installation and/or subsequent removal of the temporary protection system(s) could result in subgrade disturbance/softening of the clayey silt portions of the cohesive deposits at these sites depending on type of system and installation methodology utilized. There is also a risk of soil adhesion along the piles (CFEM 2006), which could create a void in the subsoil after removal. Considering the presence of artesian groundwater pressures in the underlying granular layers, the presence of a void would create a preferential pathway for the underlying groundwater pressures. In addition, if an open footing culvert extension is selected, the close proximity of the shoring to the footings may cause disturbance in the underlying soils. From our perspective, depending on the depth of the temporary protection system, there are associated risks and full removal is not preferred unless mitigation measures to seal any artesian groundwater source are incorporated into the work plan and an adequate distance between the footing and shoring is maintained. The Contractor will need to evaluate these risks based on the type of system and installation methodology ultimately adopted as part of their temporary protection system design. Further, the Contractor will need to re-evaluate these risks prior to removing the temporary protection system based on site observations during installation of the temporary protection system related to subgrade, culvert, and embankment performance.

Although the design of the temporary protection and/or dewatering (i.e., cofferdam) system(s), if required, will be carried out by the Contactor, the following soil parameters are provided to enable the structural designer to develop a conceptual design and asses the approximate construction costs for the project system, if adopted at this site.

Fill / Soil Type	Bulk Unit Weight, γ (kN/m^3)	Effective Stress Parameters (1)				Total Stress Parameters (1)
		Internal Angle of Friction, ϕ (degrees)	Lateral Earth Pressure Coefficients (2)			Undrained Shear Strength, s_u (kPa)
			Active, K_a	At Rest, K_o	Passive, K_p (3)	
New Granular Fill	21	35	0.27	0.43	3.69	-
Existing Granular Fill	20	35	0.27	0.43	3.69	-

Fill / Soil Type	Bulk Unit Weight, γ (kN/m³)	Effective Stress Parameters [1]				Total Stress Parameters [1]
		Internal Angle of Friction, ϕ (degrees)	Lateral Earth Pressure Coefficients [2]			Undrained Shear Strength, s_u (kPa)
			Active, K_a	At Rest, K_o	Passive, K_p [3]	
Peat / Muskeg	12.5	27.5	0.37	0.54	2.72	-
Silt to Silty Sand	18	28	0.36	0.53	2.77	-
Clayey Silt to Silty Clay above Elev. 237 m	18	31	0.32	0.48	3.12	55
Clayey Silt to Silty Clay Elev. 237-236 m	18	29	0.35	0.52	2.88	43 (average)
Clayey Silt to Silty Clay below Elev. 236 m	18	29	0.35	0.52	2.88	30

Notes:

(1) The temporary shoring design should be assessed for both the effective stress, drained (ϕ') and total stress, undrained (s_u) cases and the design should be based on the more conservative earth pressure conditions.

(2) The lateral earth pressure coefficients presented above are based on a horizontal surface adjacent to the excavation. If sloped surfaces are expected, the coefficients should be corrected accordingly.

(3) The total passive resistance below the base of the excavation adjacent to the temporary protection system may be calculated based on the values of K_p indicated above but reduced by an appropriate factor that considers the allowable wall movement in accordance with Figure C6.27 of the CHBDC (2019) to account for the fact that a large strain would be required for mobilization of the full passive resistance.

6.6.2 Subgrade Preparation

Prior to placing the levelling pad/bedding layer for the box culvert option and before pouring concrete for the open footing option, all existing fill, exposed organic materials (including topsoil, peat, and/or mixed organic soil with excessive organics), and any disturbed/softened native soils should be sub-excavated from below the plan limits of the proposed works to expose the undisturbed native subgrade soil within the plan limits of the box culvert or open footing footprint.

The subgrade shall be inspected following sub-excavation, to ensure that all organics (if encountered) and other unsuitable materials have been removed, in accordance OPSS 902 (Excavating and Backfilling – Structures) and OPSS.PROV 206 (Grading). Following inspection and approval of the exposed subgrade, any additional fill material required to raise the grade up to the underside of the proposed bedding layer or founding level shall consist of granular material meeting the requirements of an OPSS.PROV 1010 Granular 'A' or Granular 'B' Type II, as amended by SSP 110S06. As the native silty clay at the culvert invert elevation on Highway 17 is generally fine grained, a non-woven geotextile shall be placed between the native soil and the granular backfill/bedding material(s) for the box culvert option. The geotextile shall meet the specifications for OPSS.PROV 1860 (Geotextiles) Class II and have a filtration opening size (FOS) not greater than 212 µm. The granular fill shall be placed in maximum 300 mm thick loose lifts and compacted to at least 98% of the Standard Proctor Maximum Dry Density (SPMDD) of the material in accordance with OPSS.PROV 501 (Compacting), as amended by SSP 105S22. Sub-aqueous fill placement is not recommended at this site.

6.6.3 Bedding / Backfill / Cover

If a box culvert extension is utilized, bedding, backfill and cover for the culvert extension should be in general accordance with OPSS 422 (Construction Specification for Precast Reinforced Concrete Box Culverts in Open Cut), as applicable.

For a precast box culvert extension, a granular bedding layer should be incorporated into the design. In dry conditions, we recommend that a minimum 300 mm thick layer of OPSS.PROV 1010 (Aggregates) Granular ‘A’ material be used for bedding purposes or per the manufacturer’s recommendation. In wet conditions, we recommend a minimum 300 mm thick layer of OPSS.PROV 1010 Granular ‘B’ Type II material be used for bedding purposes.

For an open footing culvert extension, the footings can be founded directly on the firm to stiff native silty clay.

The backfill and cover for embankment re-instatement / widening and between the top of the culvert and the pavement structure could consist of OPSS.PROV 1010 (Aggregates) Granular ‘A’ or Granular ‘B’ (Type I, II, or III) and/or the excavated non-frost susceptible granular embankment fill materials.

The bedding, backfill, and cover should be placed in general accordance with OPSS.PROV 401 and in accordance with OPSS.PROV 501 (Compacting), as amended by SSP 105S22. Further, if precast units are utilized for the extension, compaction of the bedding should be completed in accordance with OPSS 422, which indicates that bedding under the middle third of the box unit base shall be loosely placed and uncompacted. We do not recommend the use of clear stone for bedding purposes.

Inspection of the subgrade and of the placed/compacted bedding/backfill/cover shall be carried out by qualified geotechnical personnel during all engineered fill placement operations, to ensure that appropriate materials are used and that adequate levels of compaction by the construction equipment have been achieved.

Embankment restoration after completion of the culvert extension should be carried out in accordance with OPSS.PROV 206, as amended by SSP 102S05, 206F04, and 206F06. Further, it is recommended that the widened embankment fill be benched into the existing embankment as per OPSD 208.010 (Benching).

As discussed in Section 6.3.1 “Frost Protection”, given that the culvert extension is to be founded below the estimated 2.1 m depth of frost penetration for this site, and the recommended backfill and existing embankment fill materials are generally classified as having a low susceptibility to frost heaving (as per the MTO Northern Region Pavement Design Practices and Guidelines), a frost taper is not required.

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

6.6.4 Erosion Protection

Provision should be made for erosion protection of the embankment side slopes near the outlet of the culvert extension. The requirements for, and design of, erosion protection measures for the widened embankment side slope and new culvert outlet should be assessed by the Hydrology and Drainage Engineer(s). As a minimum, the exposed embankment side slope near the culvert extension should be seeded and covered in accordance with OPSS.PROV 804 (Temporary Erosion Control), as amended by SSP 804F02 (if applicable). If additional erosion protection is required, consideration could be given to the use rip-rap, rock protection, or granular sheeting, meeting the requirements of OPSS.PROV 1004 (Aggregates – Miscellaneous), as amended by SSP 110S16, which is placed/constructed in accordance with OPSS.PROV 511 (Rip-Rap, Rock Protection, and Granular Sheeting).

The requirements for and design of erosion protection measures for the inlet and outlet of the culvert should be assessed by the Hydrology and Drainage Engineer(s). As a minimum, rip-rap treatment for the outlet of the culvert extensions should be consistent with the standard presented in OPSD 810.010 (Rip-Rap Treatment). For an open footing culvert, sufficient erosion protection should be provided along the bottom of the creek (within the culvert) such that the base of the open footing is founded below the design scour depth.

6.6.5 Control of Groundwater and Surface Water

Temporary excavations to reach the design founding level (i.e., bottom of box culvert bedding or bottom of open footing) will extend below the watercourse (i.e., creek water level), and surface water and groundwater flow/seepage into the excavation should be expected. Therefore, control of the surface water (measured to be more than 2 m deep at culvert outlet in January 2021) and groundwater will be required to facilitate the culvert extension as the open footing or box culvert bedding and culvert placement (and any associated wing walls) is recommended to be carried out in-the-dry.

Surface water should be directed away from the excavation areas to prevent ponding of water that could result in disturbance and weakening of the foundation subgrade and to allow for placement and compaction of bedding and backfill soils. Depending on the water level/groundwater level at the time of construction, cofferdams (e.g., sheet pile box) will likely be required and are considered feasible at the culvert site. Given the deeper excavation required for an open footing option, a more elaborate dewatering/cofferdam system will be required for this option, although temporary flow diversion of the creek may be reduced compared to the box culvert option. Consideration should be given to the potential for rock fill (cobble and/or boulder sized) obstructions, as identified and inferred to be present within the existing embankment fill materials and/or near the native soil interface. Provided the cofferdam is relatively watertight, continuous, and is installed with adequate penetration/depth into the native clayey soils, water pumping volumes within the excavation are anticipated to be manageable. Depending on the water flows at the time of construction, the water could potentially be pumped from behind the cofferdam near the inlet or be diverted through a temporary diversion pipe/channel. For the open footing option, the construction could be staged to divert and allow passage of the creek (half-and-half construction).

Unwatering/dewatering of all excavations should be carried out in accordance with OPSS.PROV 902, as modified by SSP FOUN0003, a copy of which is included in Appendix D. The fill-in information related to the minimum design storm return period and preconstruction survey distance have been input and should be reviewed by AECOM's Hydrology and Drainage Engineer(s).

An Environmental Activity Section Registry (EASR) may not be required to temporarily pump surface water flows from behind a cut-off wall or cofferdam system, provided the water is returned back to the same watercourse and the prescribed discharge requirements are met. However, an EASR will be required to unwater/dewater the excavation area if pumping volumes are anticipated to be greater than 50 m^3/day and a Permit to Take Water (PTTW) will be required if pumping volumes are anticipated to be greater than 400 m^3/day. Based on the soil conditions at this site and the anticipated culvert invert elevation, pumping volumes to unwater/dewater the excavation areas are anticipated to be less than 50 m^3/day if an appropriate watertight cofferdam system with sufficient embedment into the underlying cohesive deposit is utilized. The Contractor will need to evaluate the estimated seepage and groundwater removal quantity, based on their proposed construction methods/procedures and the groundwater conditions expected at the time of construction, to make the final assessment/determination whether an EASR or PTTW is ultimately required.

6.6.6 Obstructions

The Contractor shall be alerted to the potential for cobble and boulder obstructions (i.e., blast rock) within the embankment fill as identified in Borehole C-5 and inferred to be present in Borehole 21-03. It is recommended that a Notice to Contractor be included in the Contract Documents to alert the Contractor to the potential presence of these obstructions. A sample Notice to Contractor is included in Appendix D. Note that the extent and depth of the obstruction(s) may vary beyond and between the borehole locations.

6.7 Corrosion Assessment and Protection

The results of analytical testing on a soil sample recovered in Borehole 21-03 are summarized in Section 4.4 “Analytical Laboratory Testing Results” and are included in Appendix B. The potential for sulphate attack and corrosion are discussed in the following sub-sections; however, it is ultimately up to the designer to determine the appropriate construction materials, including the appropriate type of cement for concrete elements (if required) and/or the need for corrosion protection for steel elements (if required).

6.7.1 Potential for Sulphate Attack

The analytical test results were compared to CSA Standard, CAN/CSA-A23.1-14 Table 3 “Additional requirements for concrete subjected to sulphate attack” for potential sulphate attack on concrete. The measured soluble sulphate concentrations on the soil sample from Borehole 21-03 were less than the detectible limit (i.e., <20 µg/g), which is below the S-3 (Moderate) exposure class and is considered negligible according to Table 7.2 in the MTO Gravity Pipe Guidelines (2014).

However, given that the culvert location will be exposed to de-icing salts, it is recommended that a C-1 (reinforced concrete) or C-2 (non-structurally reinforced concrete) class exposure concrete be considered for any concrete elements required at these sites.

6.7.2 Potential for Corrosion

The pH measured from the sample obtained in Borehole 21-03 was 7.1. The MTO Gravity Pipe Design Guidelines (2014) indicate soil pH levels between 5.5 and 8.5 are generally not considered detrimental to culvert durability. The measured resistivity was 2000 ohm-cm, which indicates that the soil has a “moderate” to “severe” corrosiveness potential, as per Table 3.2 of the MTO Gravity Pipe Design Guidelines (2014).

It should be noted that the water levels are subject to seasonal fluctuations and variations, due to precipitation events, and the soil chemistry could also be variable. These recommendations are provided as guidance only. The culvert designer should take the results of the laboratory testing and the potential for corrosion into consideration as part of the ultimate material selection process.

7.0 CLOSURE

This report was prepared by Mr. Matthew Thibeault, P.Eng., a geotechnical engineer with Golder. Mr. Kevin Bentley, P.Eng., Golder’s Designated MTO Foundations Contact for this project and an Associate of Golder, conducted an independent quality review of the report.

Signature Page

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Https://golderassociates.sharepoint.com/sites/128666/Project Files/6 Deliverables/Foundations/Draft/R2-Fairbank Creek Culvert Extension/Rev B/20253807-R02-RevB-Fairbanks Creek Culvert FIDR Draft 25Aug_2021.docx

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ASTM D1587	Standard Practice for Thin-Walled Tube Sampling of Fine-Grained Soils for Geotechnical Purposes
ASTM D2573	Standard Test Method of Field Vane Shear Test in Saturated Fine-Grained Soils
ASTM D2435	Standard Test Method for One-Dimensional Consolidation Properties of Soils Using Incremental Loading

Ontario Provincial Standard Specifications (OPSS)

OPSS.PROV 206	Construction Specification for Grading
OPSS.PROV 401	Construction Specification for Trenching, Backfilling, and Compacting
OPSS 422	Construction Specification for Precast Reinforced Concrete Box Culverts in Open Cut
OPSS.PROV 501	Construction Specification for Compacting
OPSS.PROV 511	Construction Specification for Rip-Rap, Rock Protection, and Granular Sheeting
OPSS.PROV 517	Construction Specification for Dewatering
OPSS.PROV 539	Construction Specification for Temporary Protection Systems

OPSS.PROV 804	Construction Specification for Temporary Erosion Control
OPSS.PROV 1004	Material Specification for Aggregates – Miscellaneous
OPSS.PROV 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material
OPSS.PROV 1860	Material Specification for Geotextiles

OPSS Standard Special Provisions

SSP 102S05	Amendment to OPSS 206
SSP 105S09	Amendment to OPSS 539
SSP 105S22	Amendment to OPSS 501
SSP 110S06	Amendment to OPSS 1010
SSP 110S16	Amendment to OPSS 1004
SSP 206F04	Amendment to OPSS 206
SSP 206F06	Amendment to OPSS 206
SSP 517F01	Amendment to OPSS 517
SSP 804F02	Amendment to OPSS 804
SSP FOUND0003	Amendment to OPSS 902

Ontario Provincial Standard Drawings (OPSD)

OPSD 203.020	Embankments Over Swamps, Existing Slope Excavated
OPSD 208.010	Benching of Earth Slopes
OPSD 810.010	General Rip-Rap Layout for Sewer and Culvert Outlets
OPSD 3090.100	Foundation Frost Penetration Depths for Northern Ontario

Ontario Water Resource Act

Regulation 903	Wells (as amended)

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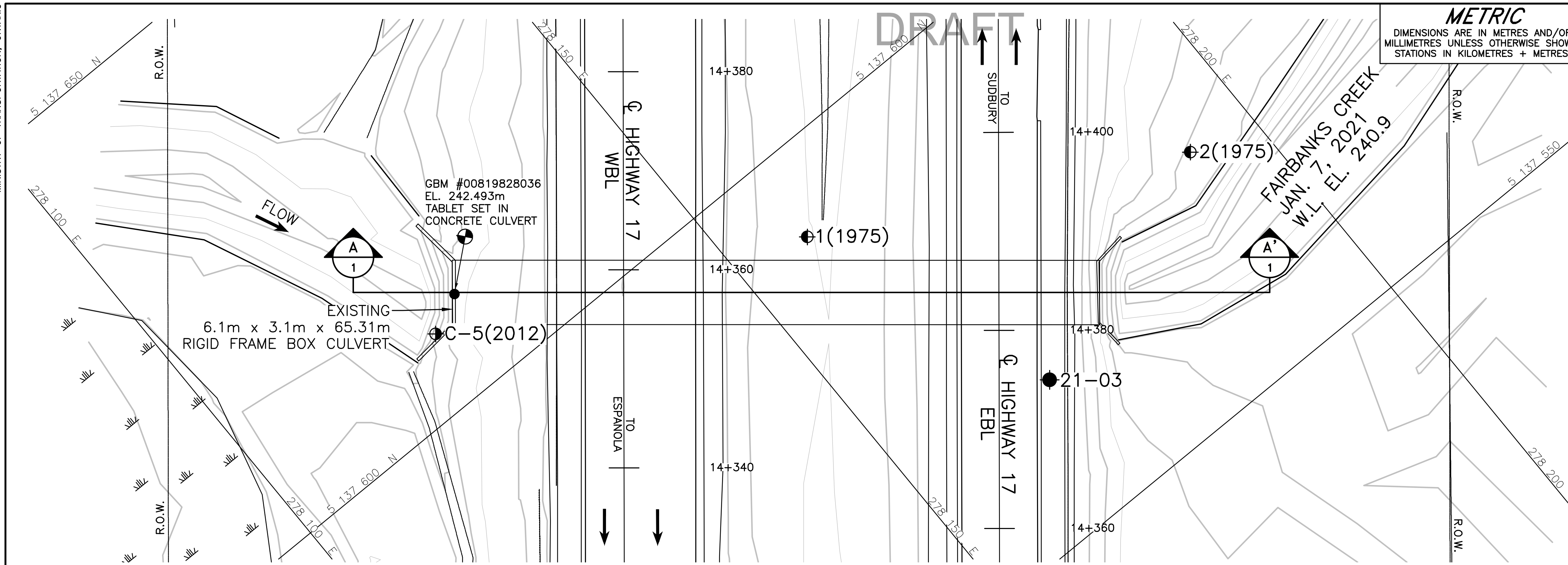
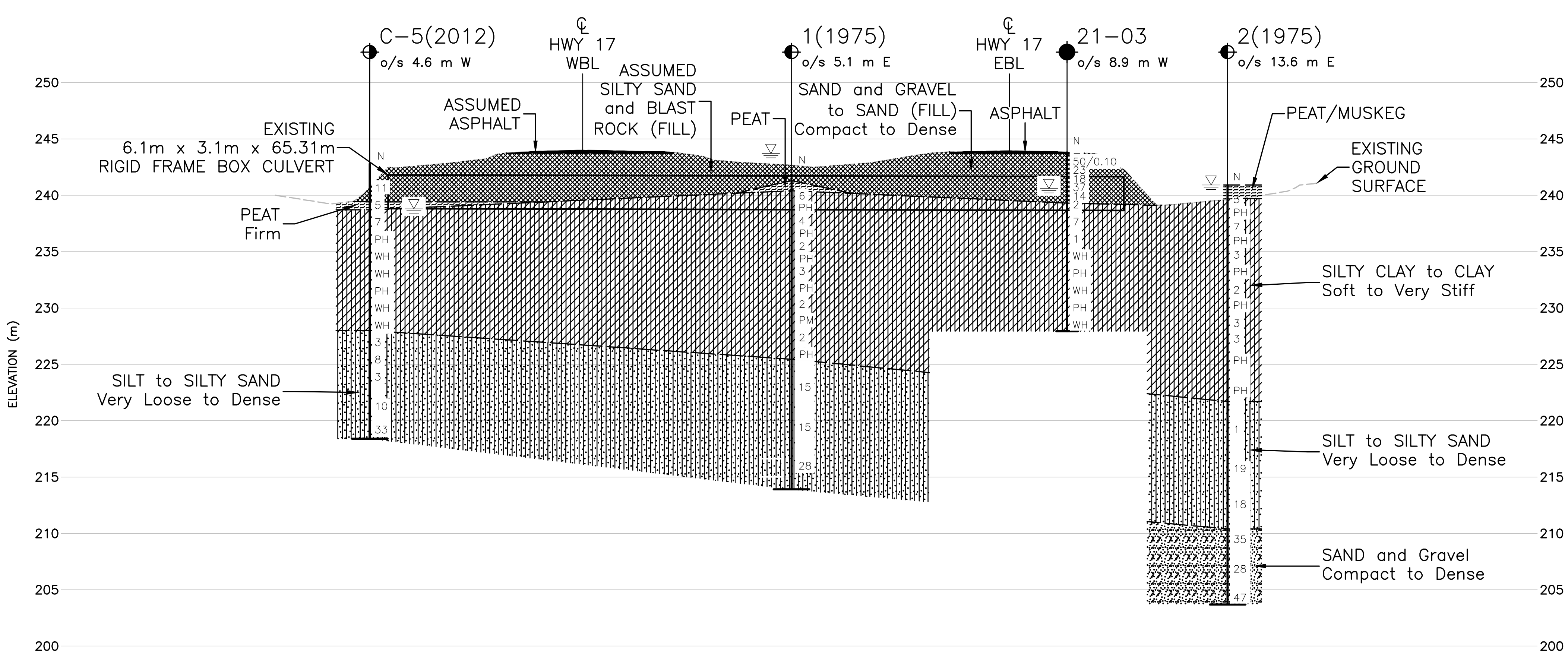


PLAN–FAIRBANKS CREEK CULVERT

SCALE 5 0 5 10 m



A-A CENTRELINE PROFILE–FAIRBANKS CREEK CULVERT

SCALE 5 0 5 10 m

METRIC

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

CONT No.
GWP No. 5032-19-00

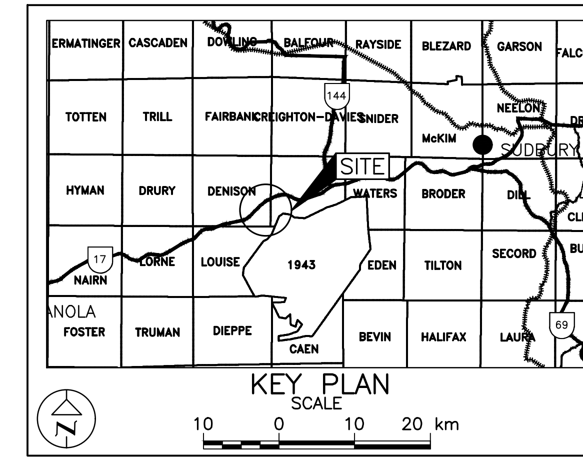
SHEET

HIGHWAY 17

FAIRBANKS CREEK CULVERT STA. 14+384

BOREHOLE LOCATIONS AND SOIL STRATA

GOLDER MEMBER OF WSP



KEY PLAN

SCALE 10 0 10 20 km

LEGEND

- Borehole – Current Investigation
- Borehole – Previous Investigation (MTO–1975)
- Borehole – Previous Investigation (Golder–2012)
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- WL upon completion of drilling

BOREHOLE CO-ORDINATES (NAD83 MTM ZONE 12)

No.	ELEVATION	NORTHING	EASTING
C-5(2012)	242.8	5137607.5	278122.5
1(1975)	241.3	5137591.3	278157.7
2(1975)	240.9	5137573.4	278193.0
21-03	243.8	5137564.6	278167.4

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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 AECOM CANADA LTD., drawing file no. Hwy 17-MR55.dwg, received MARCH 8, 2021.

NO.	DATE	BY	REVISION

Geocres No.

HWY. 17		PROJECT NO. 20253807	DIST.
SUBM'D.	CHKD.	DATE: 7/30/2021	SITE:
DRAWN: TR	CHKD.	APPD.	DWG. 1

PLOT DATE: July 30, 2021

Table 1: Comparison of Alternative Culvert Extension Types

Option	Advantages	Disadvantages	Risks/Consequences
Pre-Cast Box Culvert Extension	■ Reduces depth of excavation, protection system, and dewatering requirements compared to an open-footing option. ■ Allows for faster construction, resulting in shorter duration for dewatering and surface water pumping compared to an open-footing culvert. ■ More tolerant of total and differential settlement compared to an open-footing culvert and cast-in-place box culvert. ■ Straight forward construction procedure. ■ Lower foundation geotechnical resistance required compared to open footings.	■ Transportation to and on-site lifting of pre-cast sections will be required. ■ Specialized connection/tie-in to existing concrete culvert required if exact size/ dimensions cannot be matched. ■ Potential construction delays in ordering precast units.	■ Lower risk of dewatering concerns/ issues as box culvert segments can be placed in relatively wet conditions (although not preferred) compared to cast-in-place box or cast-in-place open footings. ■ Lower risk of future settlement/ differential settlement concerns to structure due to more tolerable segmental system compared to rigid structures.
Cast-in-Place Box Culvert Extension	■ Reduces depth of excavation, protection system, and dewatering system requirements compared to an open-footing option. ■ If adequately reinforced, more tolerant of total and differential settlement compared to an open-footing culvert but less tolerant compared to a precast box segments. ■ Easiest construction procedure. ■ Cast-in-place tie-in detail can be continuous with extension construction/concrete pour. ■ Lower foundation geotechnical resistance required compared to open footings.	■ Weather and season dependent for concrete pour and curing operations. ■ Additional time/schedule to erect formwork, reinforcing steel placement, and concrete pours compared to precast installation.	■ Higher risk of dewatering concerns/ issues affecting construction sequencing and pouring of concrete compared to precast units. ■ Higher risk related to settlement performance compared to precast units, but lower risk compared to open footing culvert.

Option	Advantages	Disadvantages	Risks/Consequences
Open Footing Culvert	■ Preferred for environmental/fisheries and/or constructability perspective. ■ May be feasible to construct the culvert on pre-cast footing sections to accelerate construction schedule and reduce time for dewatering/ unwatering (pumping), although not conventional and will required special provisions. ■ Readily suitable for construction using concrete or steel sections (although steel not preferred as it is not compatible with existing structure).	■ Highest foundation stresses and least tolerant to total and differential settlement. ■ Excavation depths/extents are greater than for a box culvert, resulting in increased effort for temporary excavation support systems and dewatering system/cofferdam requirements ■ Additional spoil material generated and will need to be disposed on or off-site. ■ Longer anticipated schedule for construction and dependence on weather season for cast-in-place concrete; however, precast footings and/or open box segments could be considered to expedite the schedule. ■ Different foundation type compared to existing box culvert structure will complicate connection detail and increase temporary protection efforts.	■ Higher risk of disturbance to the native subgrade soils (that weaken with depth) during construction. ■ Higher risk of disturbing foundation soils below existing box culvert at transition. ■ Highest risk related to settlement performance; culvert joints may be required to accommodate the total and differential settlement. Highest risk of differential settlement at transition to existing culvert due to different foundation system. ■ Deeper excavation/excavation support system increases risk of artesian groundwater conditions affecting foundation soils during and/or after construction.

Photograph 1: Culvert South End (Outlet), Looking Northeast

Photograph 2: Culvert South End (Outlet) and Highway 17 South Embankment Slope, Looking Southwest towards MR55.

Photograph 3: Culvert South End (Outlet), Looking North

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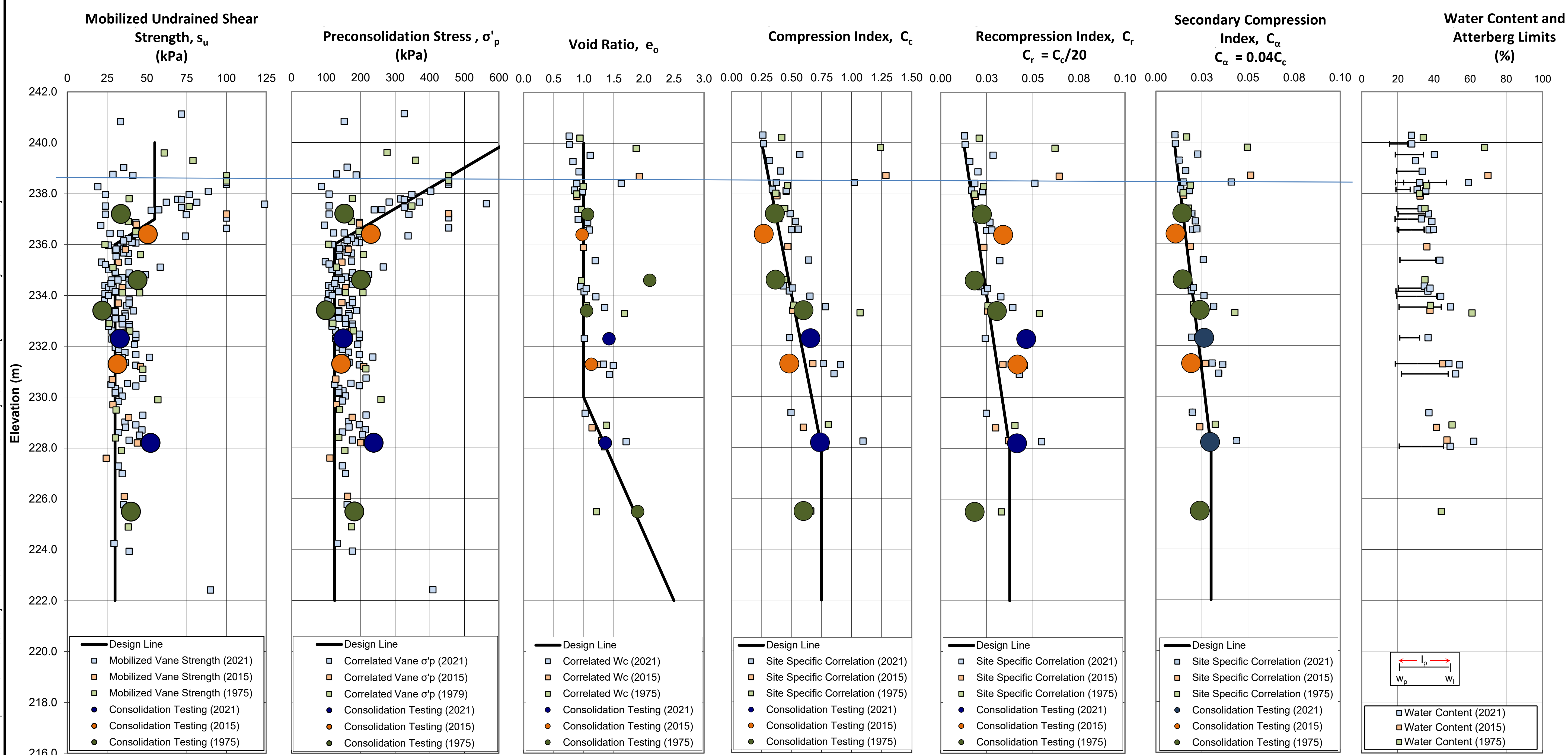
SUMMARY OF ENGINEERING PARAMETERS
FOR COHESIVE DEPOSITS
Highway 17 and MR 55 Widening

FIGURE 1



https://golderassociates.sharepoint.com/sites/128666/Project Files/5 Technical Work/Foundations/1400 - Analysis/Parameters/[Param Sum Hwy 17 & MR 55.xlsm]Final Plots

Date: July 2, 2021
Project No: 20253807

Prepared By: TB/MT
Checked By: KB

GOLDER MEMBER OF WSP

Golder Associates Ltd.

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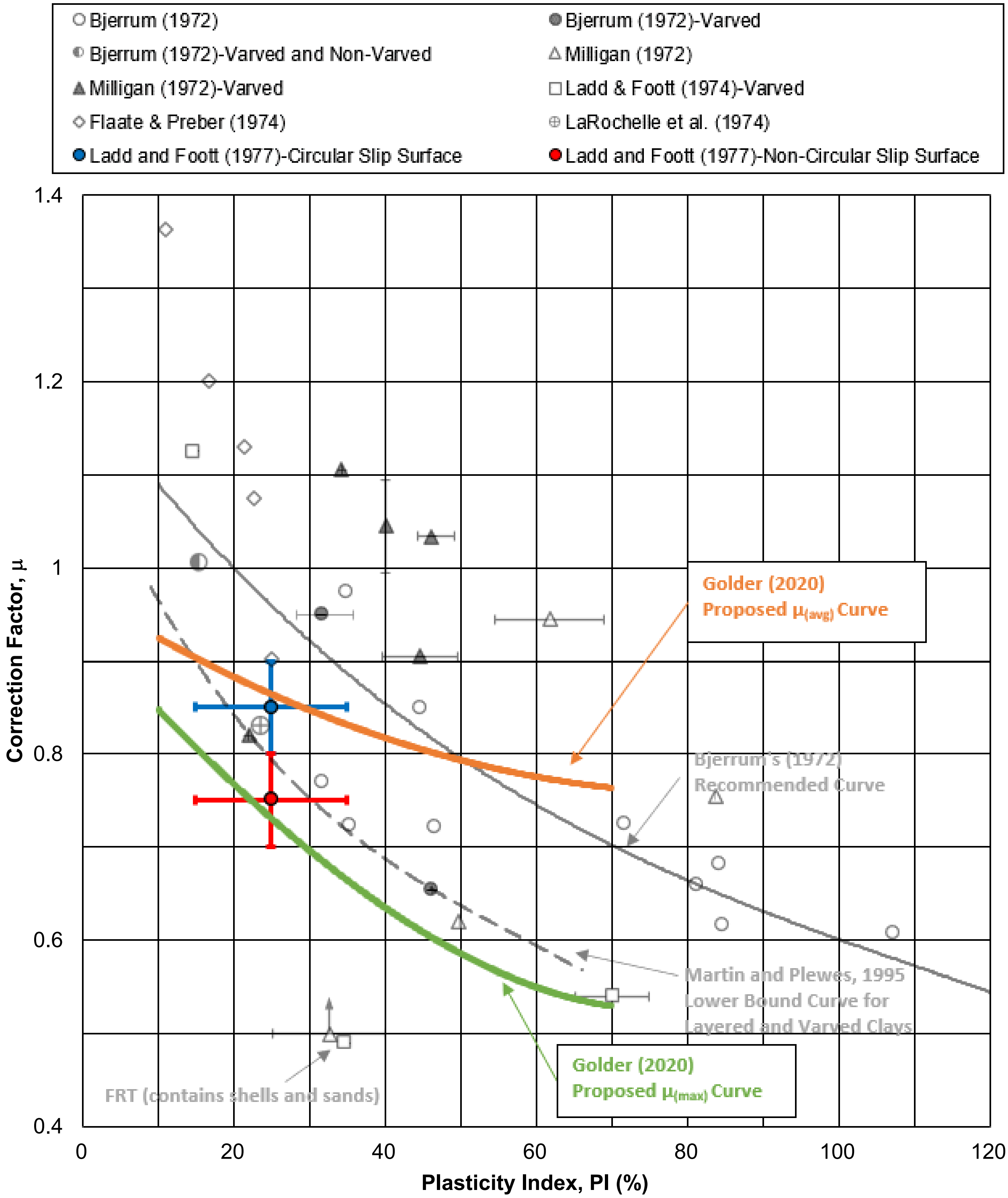
PROPOSED FIELD VANE CORRECTION FACTORS FOR VARVED CLAYS	FIGURE 2



(after Ladd et al., 1977; from Ladd and DeGroot, 2003)

Project No. 1651997 WO#12

Prepared By: TZ

Checked By: JPD

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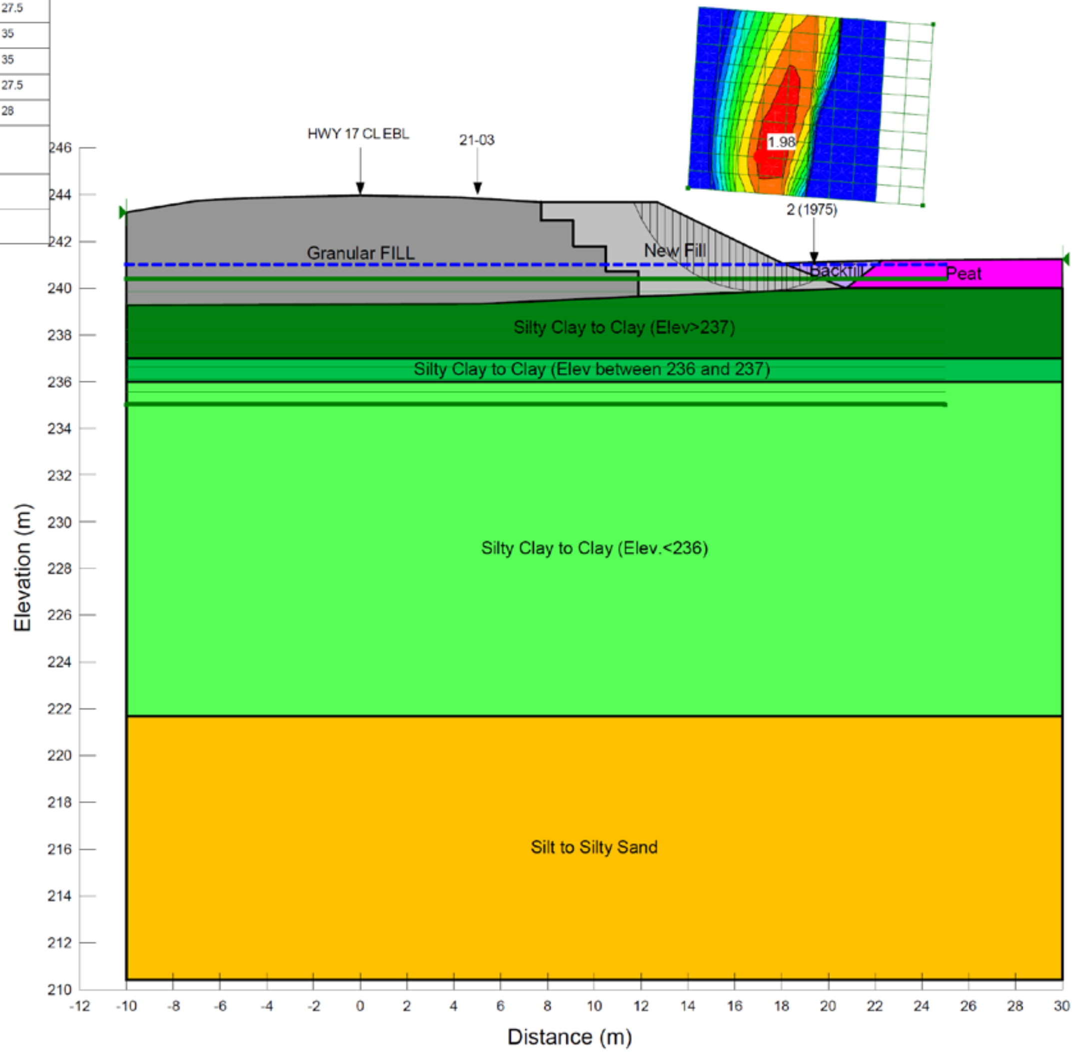
Stability Analysis

Figure 3

Station 14+380, Short-term Condition (Total Stress Parameters)

Color	Name	Unit Weight (kN/m³)	Cohesion (kPa)	C-Datum (kPa)	C-Rate of Change ((kN/m²)/m)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	Backfill	12.5				1	27.5
	Granular FILL	21				0	35
	New Fill	21				0	35
	Peat	12.5				1	27.5
	Silt to Silty Sand	18				0	28
	Silty Clay to Clay (Elev between 236 and 237)	18		55	-25		
	Silty Clay to Clay (Elev.<236)	18	30				
	Silty Clay to Clay (Elev>237)	18	55				



Date: August 2021
Project Number: 20253807-R02

Analysis By: TB
Reviewed By: MT

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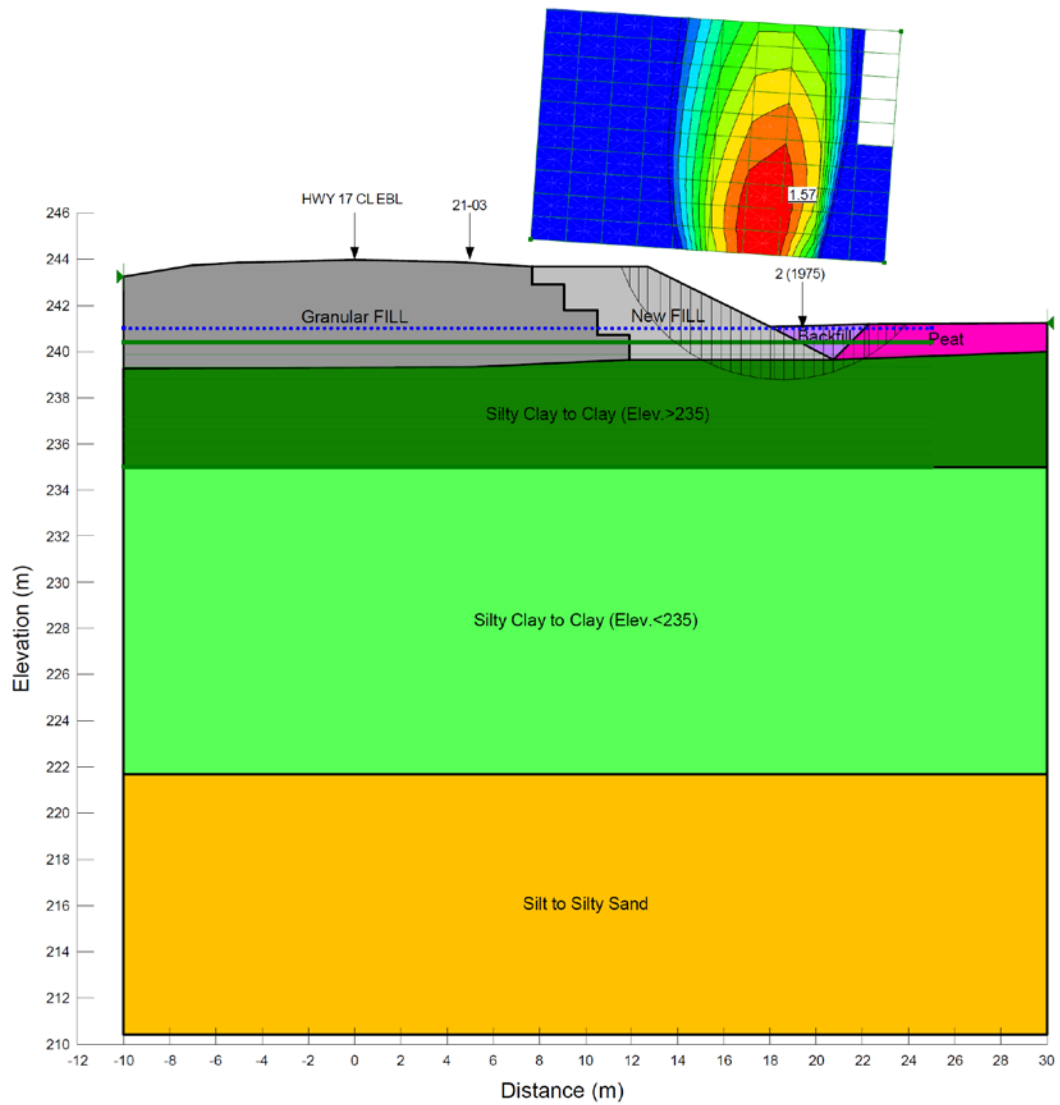
Stability Analysis

Figure 4

Station 14+380, Short-term Condition (Effective Stress Parameters)

Color	Name	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	B-bar	Add Weight
	Backfill	12.5	1	27.5	0	No
	Granular FILL	21	0	35	0	No
	New FILL	21	0	35	0	Yes
	Peat	12.5	1	27.5	0	No
	Silt to Silty Sand	18	0	28	0	No
	Silty Clay to Clay (Elev.<235)	18	0	29	1	No
	Silty Clay to Clay (Elev.>235)	18	0	31	0.3	No



Date: August 2021
Project Number: 20253807-R02

Analysis By: TB
Reviewed By: MT

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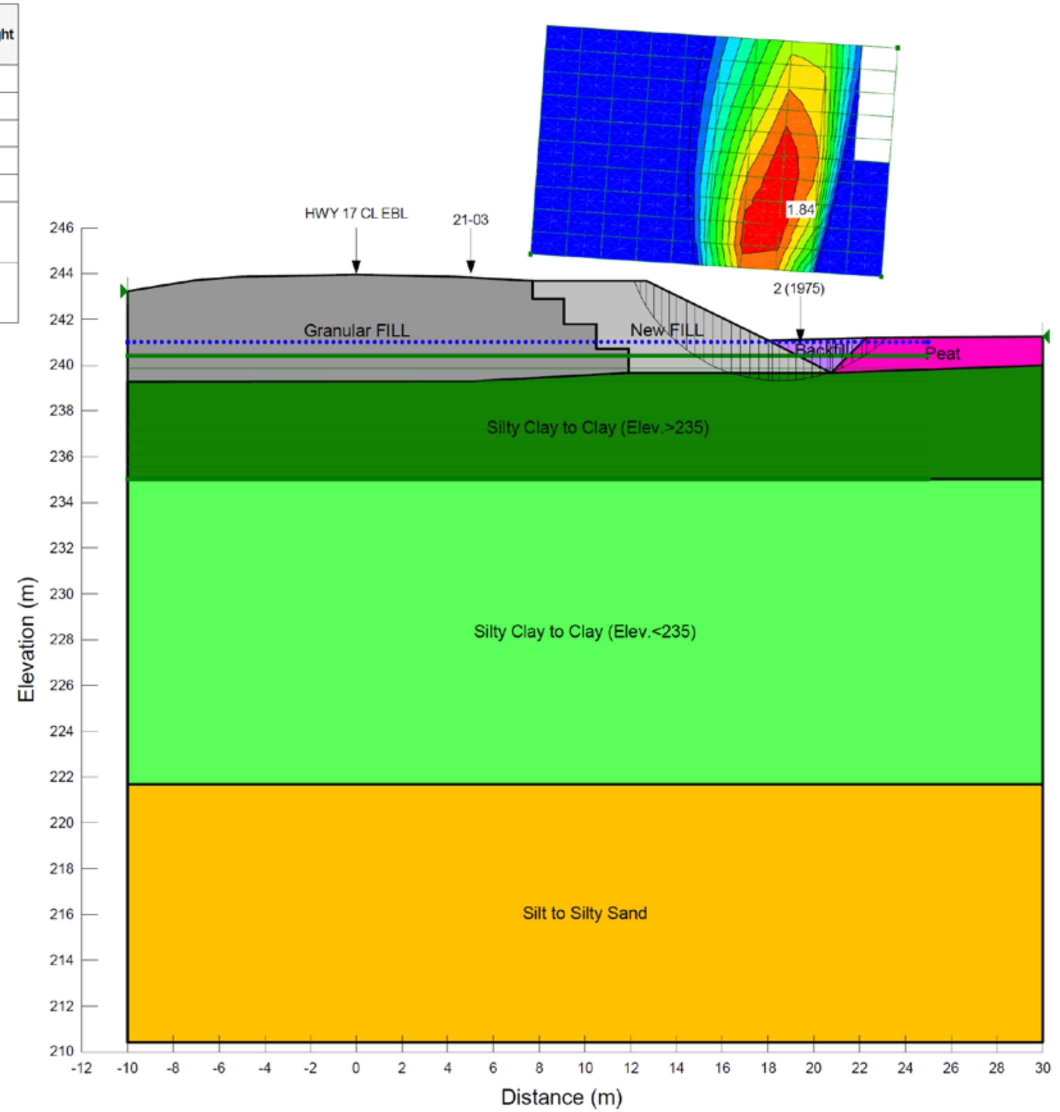
Stability Analysis

Figure 5

Station 14+380, Long-term Condition (Effective Stress Parameters)

Color	Name	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	B-bar	Add Weight
	Backfill	12.5	1	27.5	0	No
	Granular FILL	21	0	35	0	No
	New FILL	21	0	35	0	Yes
	Peat	12.5	1	27.5	0	No
	Silt to Silty Sand	18	0	28	0	No
	Silty Clay to Clay (Elev.<235)	18	0	29	0	No
	Silty Clay to Clay (Elev.>235)	18	0	31	0	No



Date: August 2021
Project Number: 20253807-R02

Analysis By: TB
Reviewed By: MT

APPENDIX A

Record of Borehole

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
σ_1, σ_2, σ_3	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

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

* Density symbol is ρ. Unit weight symbol is γ. where $\gamma = \rho \cdot g$ (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 = σ'_p / σ'_{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 δ
c_u, s_u	undrained shear strength (ϕ = 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

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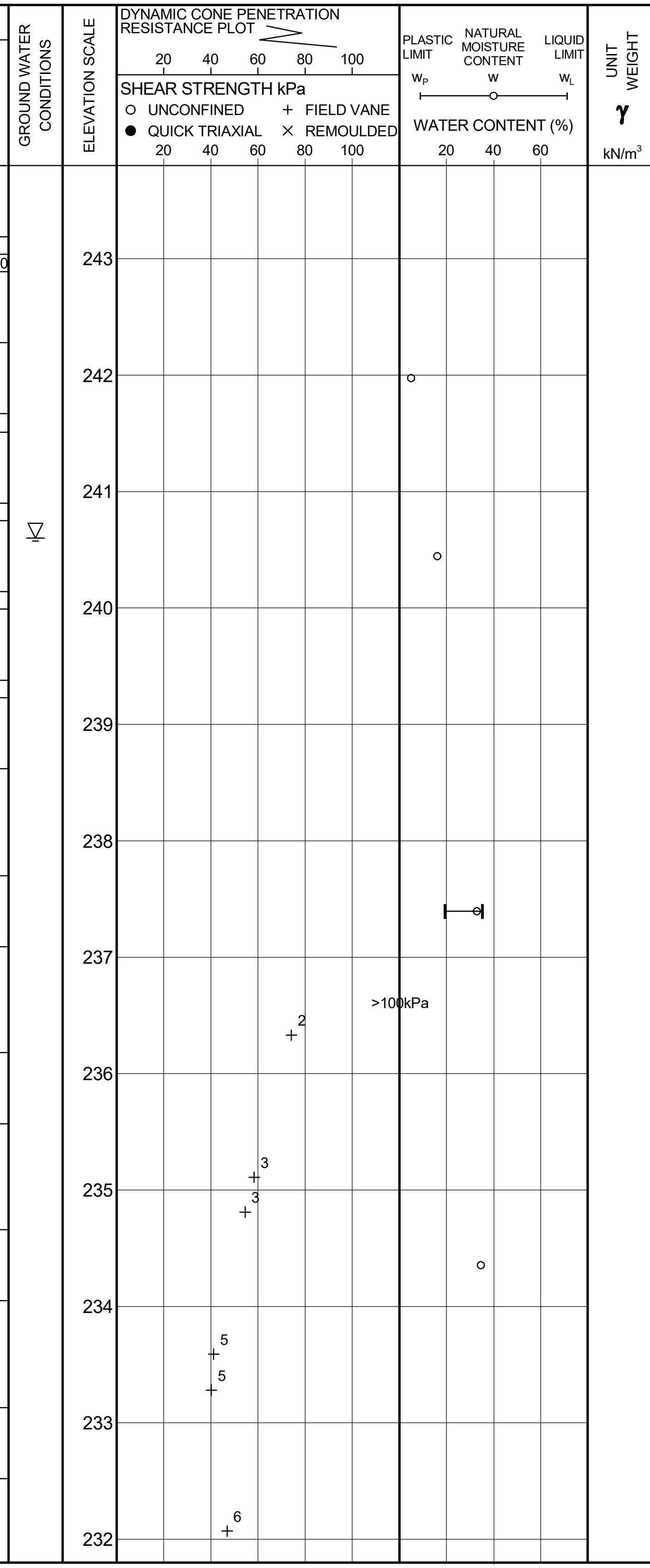
RECORD OF BOREHOLE No. 21-03

 METRIC

PROJECT 20253807			
G.W.P. 5032-19-00	LOCATION N 5137564.6; E 278167.4 NAD83 MTM ZONE 12 (LAT. 46.377267; LONG. -81.346208)	ORIGINATED BY TB/NP
DIST ___ HWY 17	BOREHOLE TYPE CME 55 Track Mount, 108 mm I.D. Hollow Stem Augers	COMPILED BY TR
DATUM GEODETIC	DATE February 8, 2021	CHECKED BY MT

SOIL PROFILE / SAMPLES

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
243.8	GROUND SURFACE				
0.0	ASPHALT (100 mm)				
0.1	SAND (SP), some fines, trace gravel to SAND and gravel (SP) (FILL) Compact to dense Brown Moist to wet - Split-spoon refusal at 1.0 m depth (potentially frozen). - Auger grinding between 0.8 m and 2.9 m depth.	1	AS	-	
		2	SS	50/0.10	
		3	SS	23	8 81 (11)
		4	SS	18	
		5	SS	37	
		6	SS	14	
239.3 4.5	Sandy CLAYEY SILT (CL) Soft Grey Wet	7	SS	2	
238.2 5.6	SILTY CLAY (CI) Firm to very stiff Grey Wet	8	SS	7	
		9	SS	1	
		10	SS	WH	
		11	TO	PH	



Continued Next Page

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

SUD-MTO 001 R:\SUDBURY\SIM\CLIENTS\MTO\HWY17_MR_55\02_DATA\GINT\20253807\20253807.GPJ GAL-MISS.GDT 7/16/21 TR

GOLDER
MEMBER OF WSP

DRAFT

Foundation Design

PROJECT 20253807	RECORD OF BOREHOLE No. 21-03	2 OF 2	METRIC
G.W.P. 5032-19-00	LOCATION N 5137564.6; E 278167.4 NAD83 MTM ZONE 12 (LAT. 46.377267; LONG. -81.346208)	ORIGINATED BY TB/NP	
DIST ___ HWY 17	BOREHOLE TYPE CME 55 Track Mount, 108 mm I.D. Hollow Stem Augers	COMPILED BY TR	
DATUM GEODETIC	DATE February 8, 2021	CHECKED BY MT	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED (20 40 60 80 100)	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L; WATER CONTENT (%) (20 40 60)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	--- CONTINUED FROM PREVIOUS PAGE ---										
	SILTY CLAY (CI) Firm to very stiff Grey Wet - Laminations of clayey silt observed in split-spoon sample No. 12.	▨	12	SS	WH		231	+ 6; + 4; + 4	├──┤ ○		
			13	TO	PH		230				
							229	+ 4; + 6			
227.9			14	SS	WH		228		○		
15.9	END OF BOREHOLE NOTES: 1. Water level measured at a depth of 3.2 m below ground surface (Elev. 240.6 m) inside augers upon completion of drilling.										

SUD-MTO 001 R:\SUDBURY\SIM\CLIENTS\MTO\HWY17_MR_55\02_DATA\GINT\20253807\20253807.GPJ GAL-MISS.GDT 7/16/21 TR

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

Golder Associates

DRAFT

Foundation Design

RECORD OF BOREHOLE No C-5

 METRIC

PROJECT 11-1191-0007
G.W.P. 156-98-00
DIST
HWY 17
DATUM Geodetic

LOCATION N 5137607.5; E 278122.5
BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing and Wash Boring
DATE June 29 and July 3, 2012

ORIGINATED BY LK
COMPILED BY EC
CHECKED BY SEMP

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
242.8	GROUND SURFACE
0.0	Silty Sand containing blast rock (FILL) Grey Moist
241.6 / 1.2	Gravelly Sand, some silt (FILL) Compact Grey Moist Trace organics below 3.0 m depth.
239.4 / 3.4	PEAT (Fibrous) Firm Black Wet
238.7 / 4.1	SILTY CLAY, trace sand Firm Grey Wet Very stiff zone
228.0 / 14.8	

SAMPLES

NUMBER	TYPE	"N" VALUES
1	SS	11
2	SS	5
3	SS	7
4	TO	PH
5	SS	WH
6	SS	WH
7	SS	PH
8	SS	WH
9	SS	WH

GROUND WATER CONDITIONS

ELEVATION SCALE: 242, 241, 240, 239, 238, 237, 236, 235, 234, 233, 232, 231, 230, 229, 228

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

>100

Field vane sensitivity values: 5, 6, 4, 6, 4, 4

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L

WATER CONTENT (%): 20 40 60

UNIT WEIGHT γ kN/m³

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

Continued Next Page

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

SUD-MTO 001 11-1191-0007.GPJ GAL-MISS.GDT 06/05/15 DATA INPUT:

RECORD OF BOREHOLE No C-5

 METRIC

PROJECT 11-1191-0007
G.W.P. 156-98-00
LOCATION N 5137607.5; E 278122.5
ORIGINATED BY LK
DIST HWY 17
BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing and Wash Boring
COMPILED BY EC
DATUM Geodetic
DATE June 29 and July 3, 2012
CHECKED BY SEMP

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
	SILT, trace to some clay, trace sand Very loose to loose Grey Wet	10	SS	3							
					227						
					226						
		11	SS	8		○	NP	0	1	89	10
					225						
		12	SS	3							
					224						
					223						
222.7 20.1	SAND and SILT, trace clay Compact to dense Grey Wet										
					222						
		13	SS	10		○		0	57	41	2
					221						
					220						
					219						
		14	SS	33							
218.4 24.4	END OF BOREHOLE Note: 1. Water level at a depth of 4.0 m below ground surface (Elev. 238.8 m) upon completion of drilling.										

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

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

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

ENGINEERING SERVICES BRANCH–GEOTECHNICAL OFFICE–SOIL MECHANICS SECTION

RECORD OF BOREHOLE Nº 1

W.P. 61-74-02/03 LOCATION CO-ORDS. 16,854,900N; 912,585E. ORIGINATED BY ~~JDI~~

DIST. 17 HWY. 17, LINE 'D' BORING DATE JANUARY 14, 1975 COMPILED BY C.McK.

DATUM GEODETIC BOREHOLE TYPE HOLLOW STEM AUGER AND CONE TEST CHECKED BY

SOIL PROFILE

ELEV. / DEPTH	DESCRIPTION
241.3 / 791.6	GROUND LEVEL
240.4 / 788.6 / 0.9 / 3.0	Muskeg
	Silty clay to clay, Occasional Silt seams — Stiff, Firm, Soft, Firm
225.4 / 739.6 / 15.8 / 52.0	Sandy Silt to Silty sand, Traces of clay — Compact, Dense
213.8 / 701.6 / 27.4 / 90.0	End of Borehole

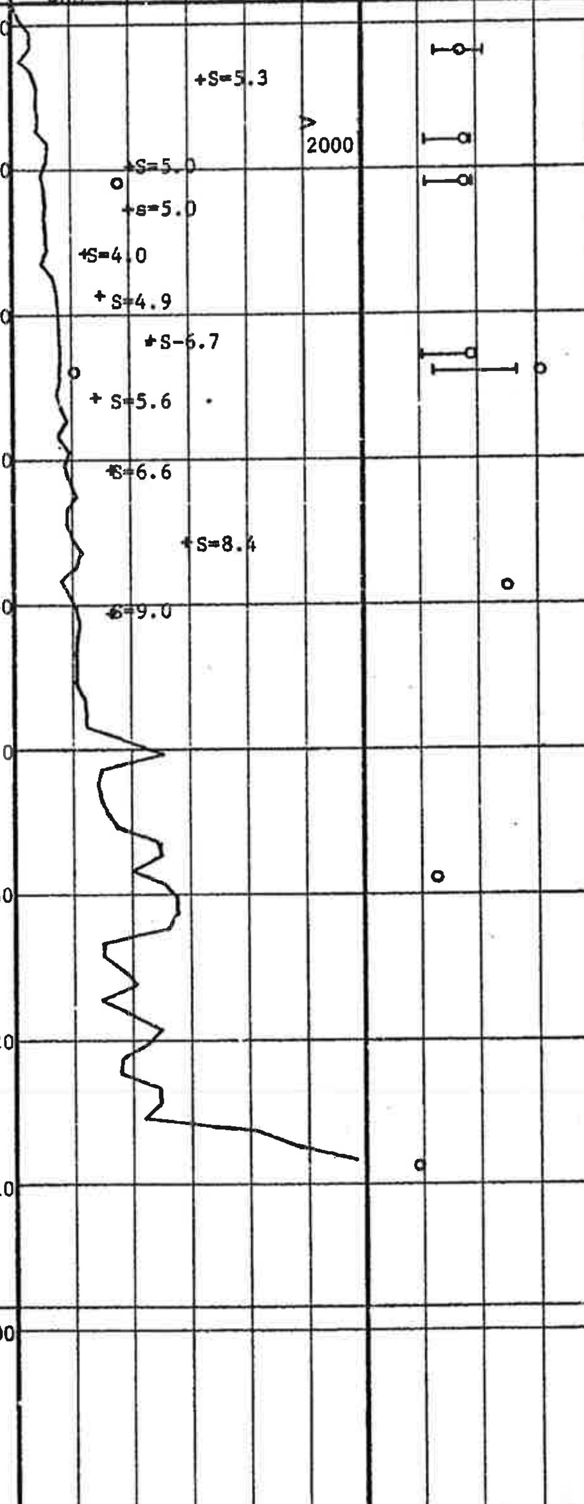
SAMPLES

NUMBER	TYPE	'N' VALUES
1	SS	6
2	TW	PH
3	SS	4
4	TW	PH
5	SS	2
6	TW	PH
	SS	3
8	TW	PH
9	SS	2
10	TW	PH
11	SS	2
12	TW	PH
13	SS	15
14	SS	15
15	SS	28

GROUND WATER ELEV.: 790, 780, 770, 760, 750, 740, 730, 720, 710, 700

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20, 40, 60, 80, 100

SHEAR STRENGTH: O UNCONFINED, ● QUICK TRIAXIAL, + FIELD VANE, × LAB VANE; 400, 800, 1200, 1600, 2000



LIQUID LIMIT — W_L; PLASTIC LIMIT — W_P; WATER CONTENT — W; WATER CONTENT %: 20, 40, 60

UNIT WEIGHT γ	REMARKS % GR. SA. SI. CL.
117	0 0 54 46
105	0 0 70 30
	0 27 69 4
	0 55 40 5

Artesian Head El. 799.6 / 243.7

Encount. El. 707.5 / 215.6

20
15 -o- 5 % STRAIN AT FAILURE

DRAFT

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

ENGINEERING SERVICES BRANCH-GEOTECHNICAL OFFICE-SOIL MECHANICS SECTION

RECORD OF BOREHOLE Nº 2

W.P. 61=74-02/03 LOCATION CO-ORDS. 16,854,844N; 912,670E. ORIGINATED BY MM

DIST. 17 HWY 17, Line 'D' BORING DATE January 16, 1975 COMPILED BY MM

DATUM GEODETIC BOREHOLE TYPE HOLLOW STEM AUGER AND CONE TEST CHECKED BY

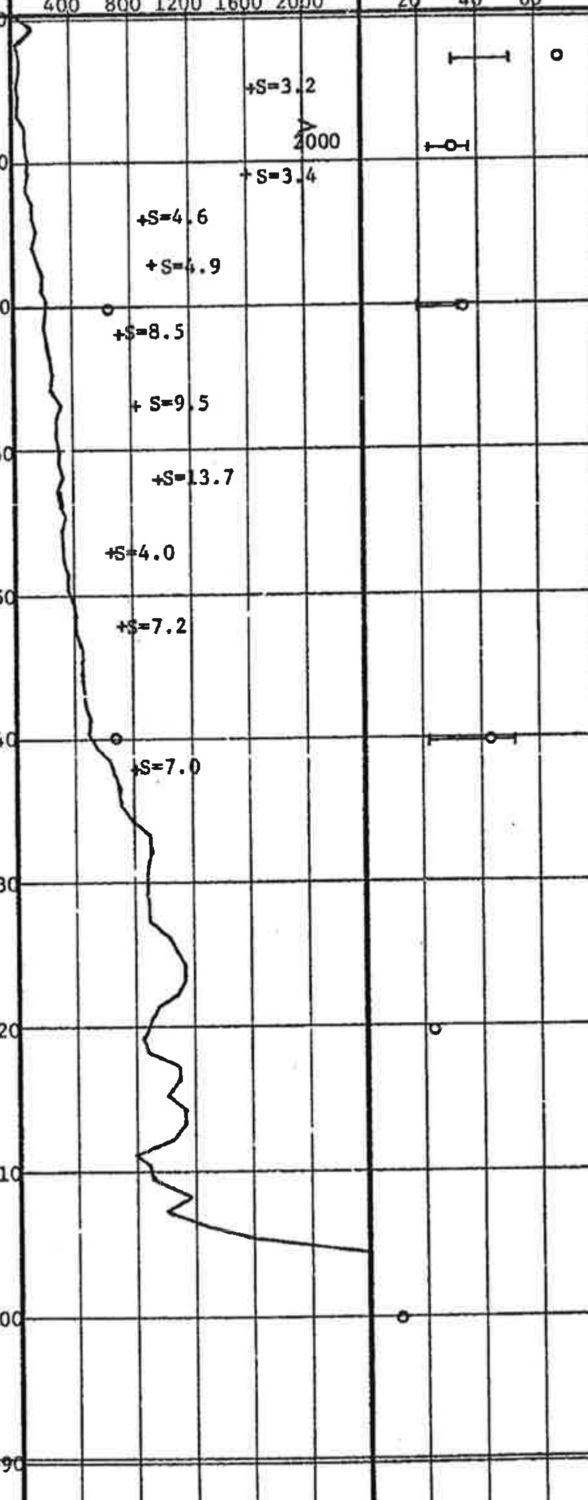
OFFICE REPORT ON SOIL EXPLORATION

Elev. (m) / Depth (m)	ELEV. / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	ELEV.	LIQUID LIMIT / PLASTIC LIMIT / WATER CONTENT	UNIT WEIGHT γ	REMARKS % GR. SA. SI. CL.
240.9	790.4	GROUND LEVEL				790			
240.0 / 0.9	787.4 / 3.0	Muskeg							
		Silty Clay to Clay, Occasional Silt Seams; Stiff / Firm	1	SS	3				
			2	TW	PH				0 1 53 46
			3	SS	7	780			
			4	TW	PH				
			5	SS	3				
			6	TW	PH	770		116	0 0 68 32
			7	SS	2				
			8	TW	PH	760			
			9	SS	3				
			10	SS	3	750			
			11	TW	PH	740		109	0 0 (100)
221.7 / 19.2	727.4 / 63.0		12	TW	PH	730			
		Silt to Silty sand, Traces of clay; V. Loose / Compact	13	SS	1	720			0 5 90 5
			14	SS	19	710			
			15	SS	18	700			0 87 (13)
210.4 / 30.5	690.4 / 100.0					690			

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH o UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 400 800 1200 1600 2000

WATER CONTENT % 20 40 60



DRAFT

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS - ONTARIO

ENGINEERING SERVICES BRANCH - GEOTECHNICAL OFFICE - SOIL MECHANICS SECTION

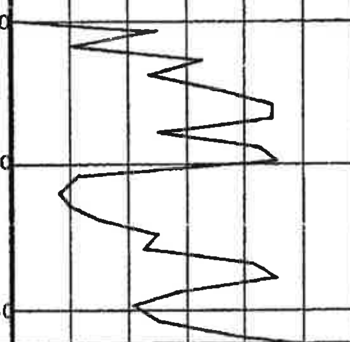
RECORD OF BOREHOLE № 2 (Continued)

W.P. 61-74-02/03 LOCATION CO-ORDS. 16,854,844N; 912,670E. ORIGINATED BY MM

DIST. 17 HWY. 17 Line 'D' BORING DATE January 16th, 1975 COMPILED BY MM

DATUM GEODETIC BOREHOLE TYPE HOLLOW STEM AUGER & CONE TEST CHECKED BY

(metric)	ELEV. / DEPTH	SOIL PROFILE: DESCRIPTION	STRAT. PLOT	SAMPLES: NUMBER	TYPE	'N' VALUES	GROUND WATER / ELEV.	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE) 400 800 1200 1600 2000	LIQUID LIMIT — W_L / PLASTIC LIMIT — W_P / WATER CONTENT — W; WATER CONTENT % 20 40 60	UNIT WEIGHT γ	REMARKS % GR SA SI CL
210.4	690.4						690				
30.5	100.0	Sand and gravel Traces of Silt Compact to dense		16	SS	35					
				17	SS	28	680		o		51 42 (7)
203.7	668.4			18	SS	47	670				
37.2	122.0	End of Borehole					660				
197.5	647.9						650				
43.4	142.5	End of Cone Penetration					640				

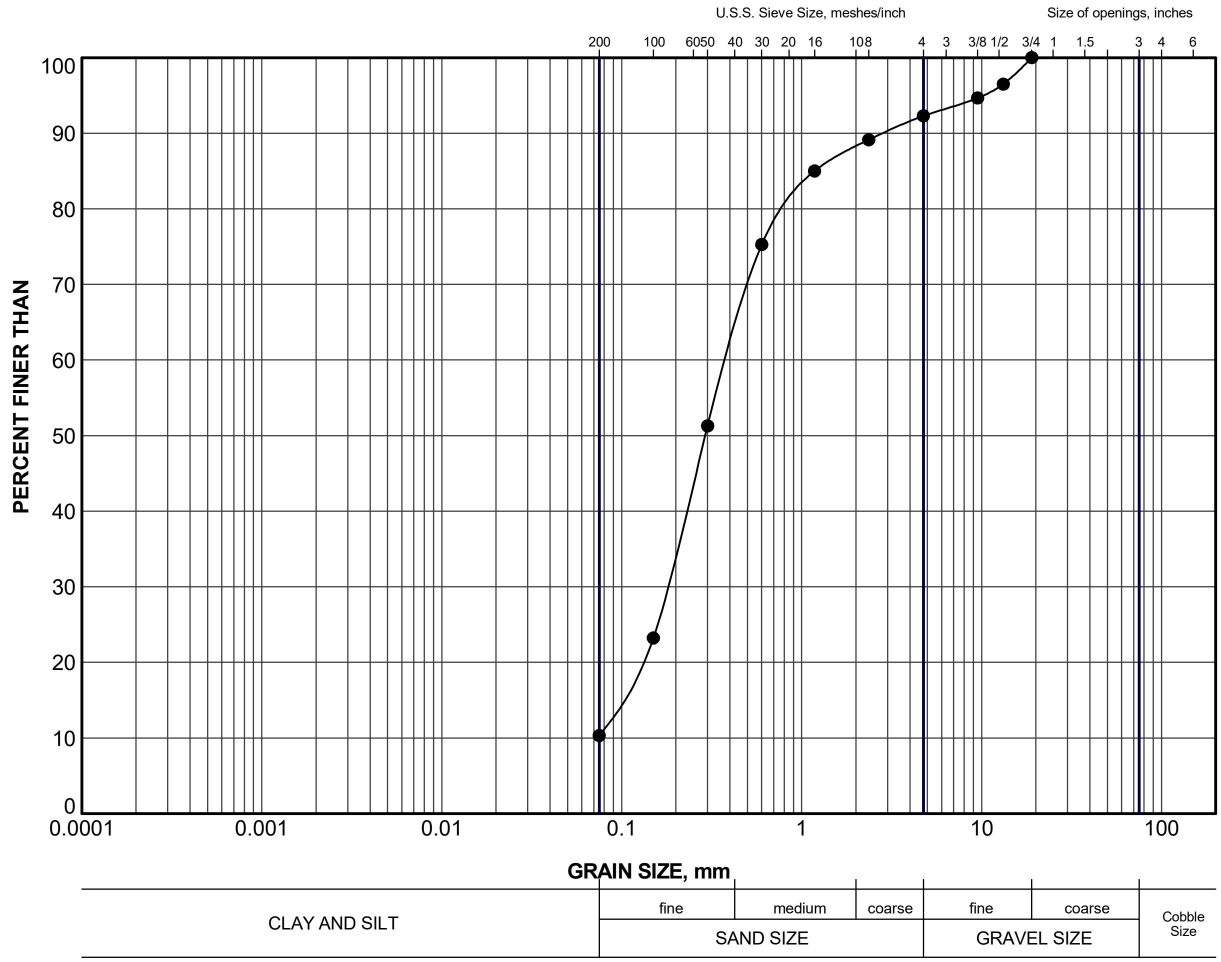


20
15 5 % STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

APPENDIX B

Laboratory Test Results (Current Investigation)

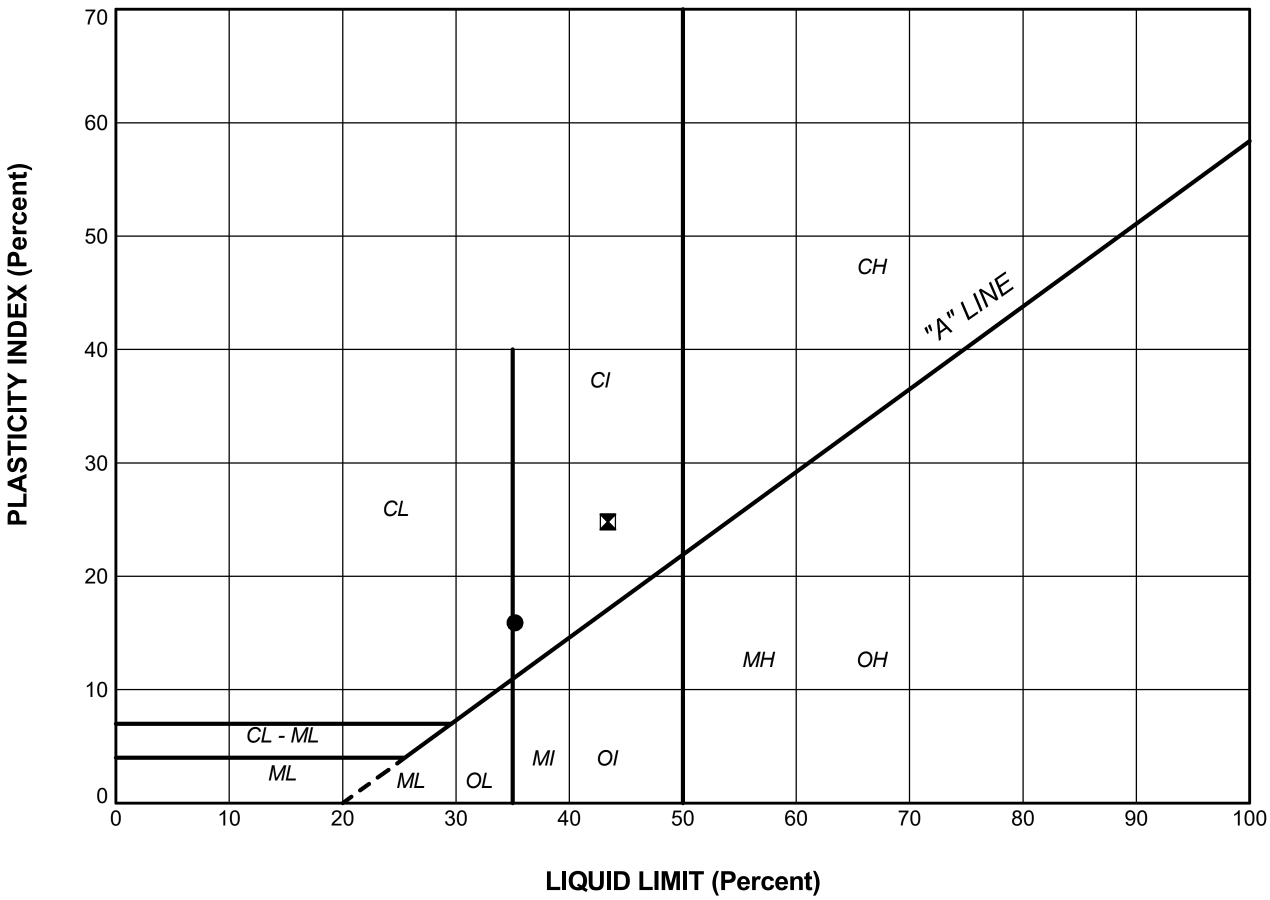


LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	21-03	3	242.0

PROJECT	HIGHWAY 17				
TITLE	**GRAIN SIZE DISTRIBUTION** SAND (SP) (FILL)				
GOLDER SUDBURY, ONTARIO	PROJECT No.		20253807	FILE No.	20253807.GPJ
	DRAWN	TR	Apr 2021	SCALE N/A	REV.
	CHECK	TB	Apr 2021	**FIGURE B-1**	
	APPR	MT	Apr 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
●	21-03	8	35.2	19.3	15.9
⊠	21-03	12	43.4	18.6	24.8

PROJECT
HIGHWAY 17

TITLE
PLASTICITY CHART
SILTY CLAY (CI)

GOLDER
SUDBURY, ONTARIO

PROJECT No.	20253807		FILE No.	20253807.GPJ
DRAWN	TR	Apr 2021	SCALE N/A	REV.
CHECK	TB	Apr 2021	**FIGURE B-2**	
APPR	MT	Apr 2021		

SUD-MTO PL (25) GLDR_LDN.GDT

BUREAU VERITAS

BV Labs Job #: C140122
Report Date: 2021/02/23

Golder Associates Ltd
Client Project #: 20253807
Sampler Initials: TB

RESULTS OF ANALYSES OF SOIL

BV Labs ID		OVO702	OVO703			OVO703		OVO704		
Sampling Date		2021/02/01	2021/02/08			2021/02/08		2021/02/02		
COC Number		na	na			na		na		
	UNITS	BH21-2 SA#5	BH21-3 SA#7	RDL	QC Batch	BH21-3 SA#7 Lab-Dup	QC Batch	BH21-7 SA#6	RDL	QC Batch
Calculated Parameters										
Resistivity	ohm-cm	580	2000		7201555			1600		7201555
Inorganics										
Soluble (20:1) Chloride (Cl-)	ug/g	980	310	20	7206500			410	20	7206500
Conductivity	umho/cm	1720	502	2	7206535			639	2	7206535
Available (CaCl2) pH	pH	6.65	7.12		7212920	7.07	7212920	6.27		7212920
Soluble (20:1) Sulphate (SO4)	ug/g	49	<20	20	7206511			25	20	7206511
Sulphide	mg/kg	<0.5 (1)	<0.5 (1)	0.5	7211691			<0.5 (1)	0.5	7211691
Physical Testing										
Moisture-Subcontracted	%	11	17	0.30	7211690			24	0.30	7211690

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

Bureau Veritas Laboratories 6740 Campobello Road, Mississauga, Ontario, L5N 2L8 Tel: (905) 817-5700 Toll-Free: 800-563-6266 Fax: (905) 817-5777 www.bvlabs.com

Microbiology testing is conducted at 6660 Campobello Rd. Chemistry testing is conducted at 6740 Campobello Rd.

APPENDIX C

Laboratory Test Results (Previous Investigations)

DRAFT

Figure C1

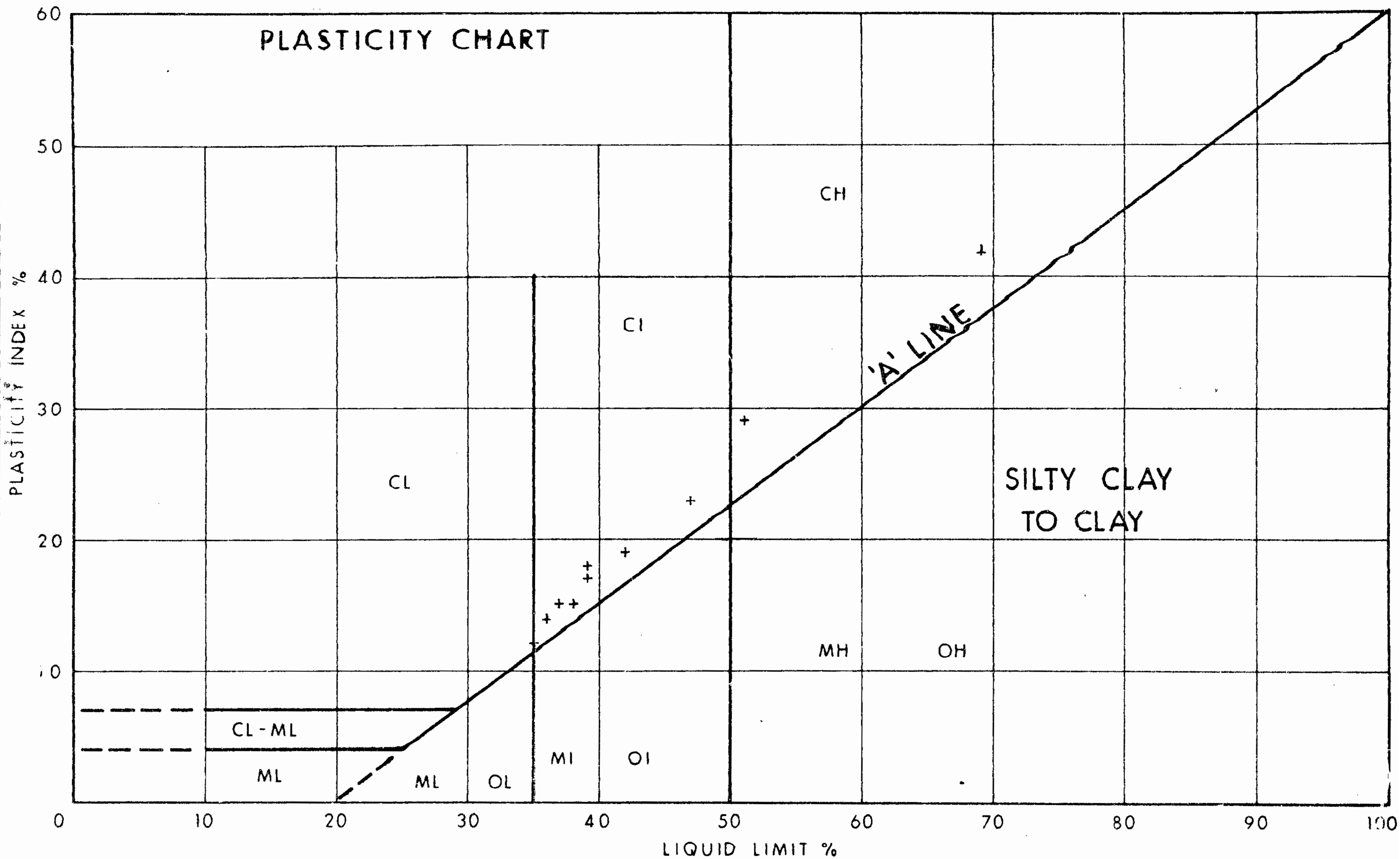


FIG. 2

W.P. 61-74-02/03

Figure C2

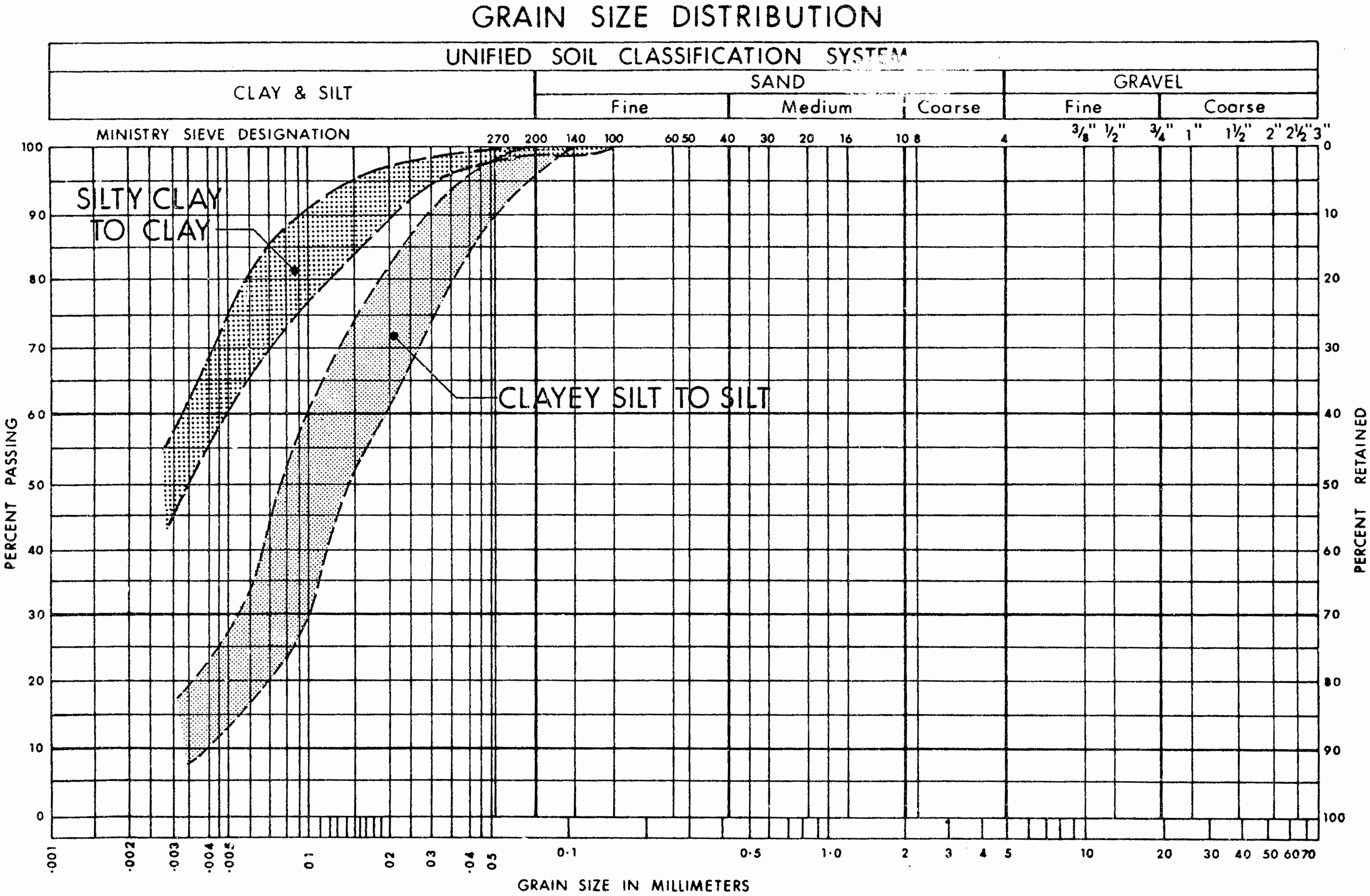
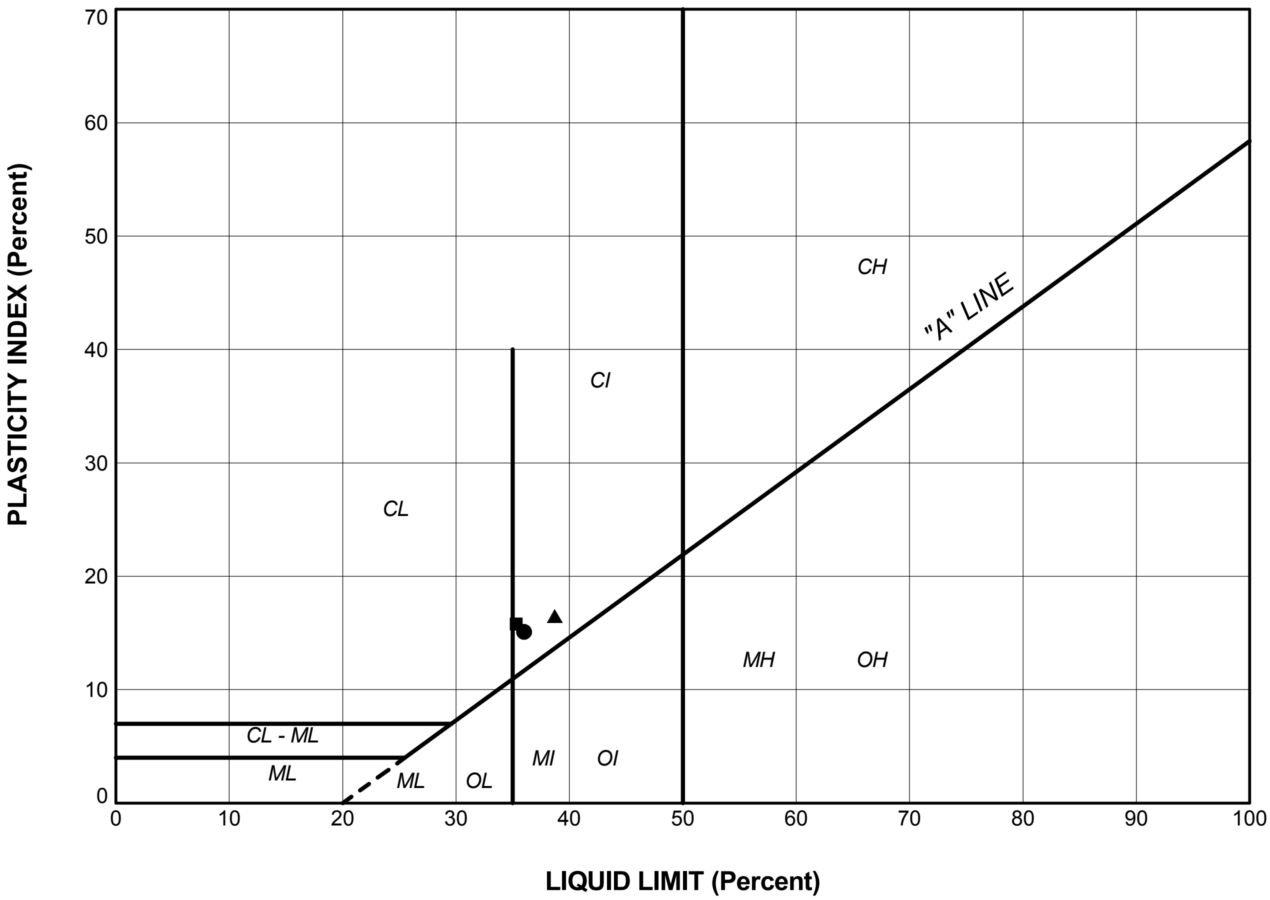


FIG. 4

W.P. 61-74-02/03



SOIL TYPE
C = Clay
M = Silt
O = Organic

PLASTICITY
L = Low
I = Intermediate
H = High

LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	C-5	3	36.0	20.9	15.1
■	C-5	6	35.3	19.5	15.8
▲	C-5	9	38.7	22.2	16.5

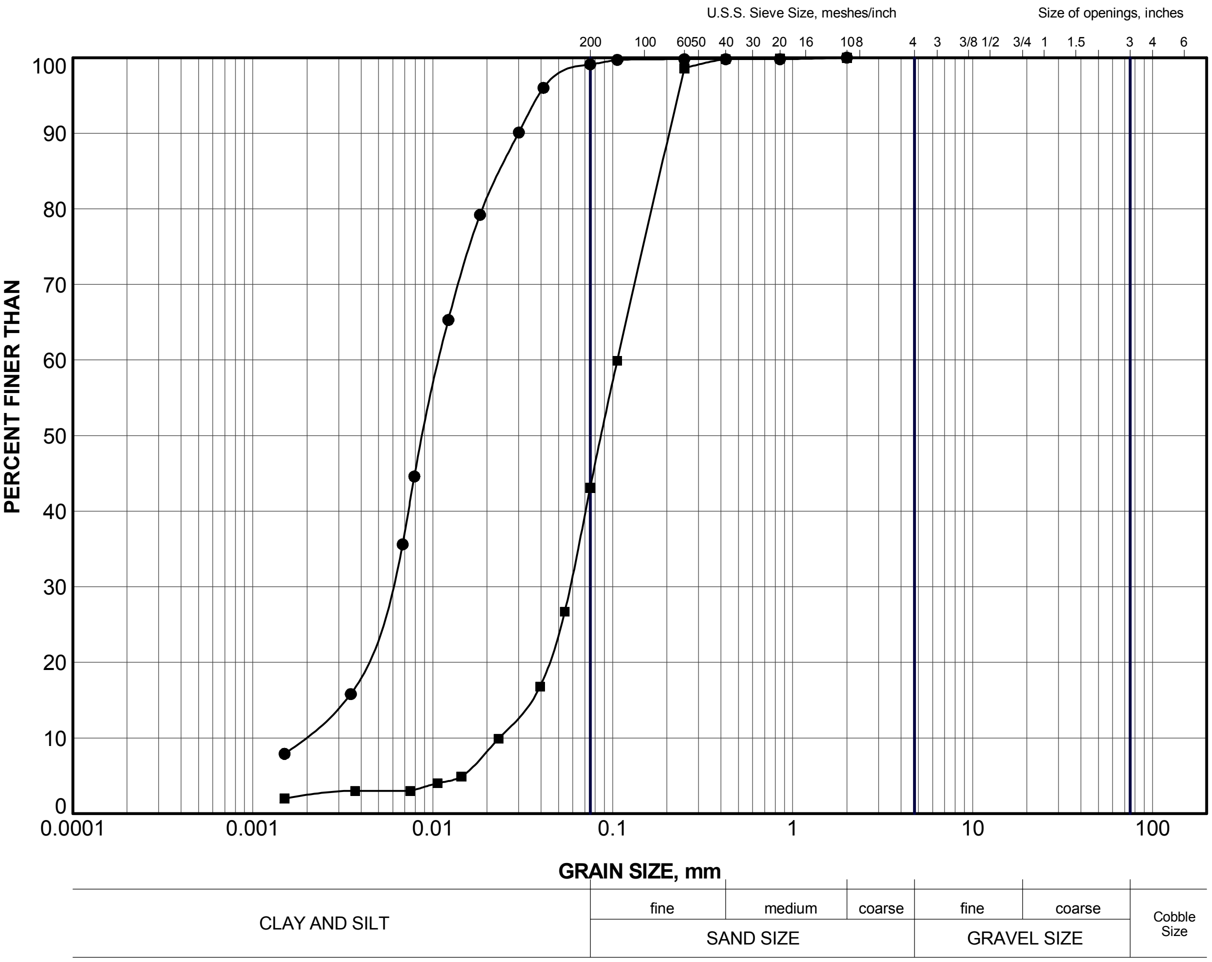
PROJECT
HIGHWAY 17
FAIRBANK CREEK CULVERT STA-14+240

TITLE
PLASTICITY CHART
SILTY CLAY

Golder Associates
SUDBURY, ONTARIO

PROJECT No.	11-1191-0007		FILE No.	1111910007 CULVERTS.GPJ
DRAWN	TB	Oct 2014	SCALE N/A	REV.
CHECK	SEMP	Oct 2014		
APPR		Oct 2014	**FIGURE C3**	

SUD-MTO PL (NEW) GLDR_LDN.GDT



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	C-5	11	225.7
■	C-5	13	221.2

PROJECT

HIGHWAY 17
FAIRBANK CREEK CULVERT STA-14+240

TITLE

GRAIN SIZE DISTRIBUTION

SILT to SILT and SAND

Golder Associates
SUDBURY, ONTARIO

PROJECT No.	11-1191-0007		FILE No. 1111910007 CULVERTS.GPJ
DRAWN	TB	Oct 2014	SCALE N/A REV.
CHECK	SEMP	Oct 2014	
APPR	JMAC	Oct 2014	FIGURE C4

SUD-MTO GSD (NEW) GLDR_LDN.GDT

APPENDIX D

Notice to Contractor and Standard Special Provisions

DRAFT

EXISTING SUBSURFACE CONDITIONS – Item No.

Notice to Contractor

The Contractor is alerted to the potential for cobble and boulder obstructions within the embankment fill as inferred to be present based on instances of auger griding and/or split-spoon refusal as encountered in Borehole 21-03. The extent and depth of obstructions may vary beyond and between the borehole locations.

DEWATERING STRUCTURE EXCAVATIONS - Item No.

Special Provision No. FOUN0003

Amendment to OPSS 902, November 2010

902.02 REFERENCES

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

Ontario Provincial Standard Specifications, Construction

OPSS 517 Dewatering
OPSS 805 Temporary Erosion and Sediment Control Measures

902.03 DEFINITIONS

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

Automatic Transfer Switch means as defined in OPSS 517.

Cofferdam means as defined in OPSS 539.

Cut-Off Wall means as defined in OPSS 517.

Design Storm Return Period means as defined in OPSS 517.

Groundwater Control System means as defined in OPSS 517.

Plug means as defined in OPSS 517.

Sediment means as defined in OPSS 517.

Sediment Control Measure means as defined in OPSS 517.

Temporary Flow Passage System means as defined in OPSS 517.

Unwatering means as defined in OPSS 517.

Vegetated Discharge Area means as defined in OPSS 517.

Waterbody means as defined in OPSS 517.

Watercourse means as defined in OPSS 517.

902.04 DESIGN AND SUBMISSION REQUIREMENTS

902.04.01 Design Requirements

902.04.01.01 Dewatering

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

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

When the system includes temporary flow passage system, the system shall be designed, as a minimum, for a 2 year design storm return period, and groundwater discharge. A longer return period shall be used when determined appropriate for the work.

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

902.04.02 Submission Requirements

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

902.04.02.01 Preconstruction Survey

When a groundwater control system by wells or a well point system will be used, a condition survey of property and structures that may be affected by the work shall be carried out. The condition survey shall include the location and condition of adjacent properties, buildings, underground structures, water wells, Utilities, and structures, within a distance of 50 metres from the groundwater control system. In addition, all water wells used as a supply of drinking water and located within this distance shall be tested for compliance with Ontario Drinking Water Quality Standards.

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

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

902.04.02.02 Working Drawings

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

902.07 CONSTRUCTION

902.07.04 Dewatering Structure Excavation

Subsection 902.07.04 of OPSS 902 is amended by the addition of the following clauses:

902.07.04.01 General

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

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

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

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

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

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

Unwatering shall be carried out as necessary.

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

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

902.07.04.02 Discharge of Water

The discharge of water shall be according to OPSS 517.

902.07.04.03 Monitoring

Monitoring shall be according to OPSS 517.

902.07.04.04 System Amendments

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

902.07.04.05 Removal

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

NOTES TO DESIGNER:

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

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

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

CUSTODIAN:	Tony Sangiuliano, MERO - Foundation Group.

DRAFT

GOLDER

golder.com