



**THURBER** ENGINEERING LTD.

**PRELIMINARY  
FOUNDATION INVESTIGATION AND DESIGN REPORT  
HIGHWAY 17 TWINNING, RENFREW AREA  
CULVERT 7AN  
COUNTY ROAD 20, STA. 9+600, HORTON TOWNSHIP  
WP 4068-09-00 / ASSIGNMENT NO. 4018-E-0009**

Geocres No.: 31F07-002

Report to:

**Ministry of Transportation Ontario**

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**PART 1. FACTUAL INFORMATION**

## **1 INTRODUCTION**

Thurber Engineering Ltd. (Thurber) has been engaged by the Ministry of Transportation Ontario (MTO) to carry out Foundation Investigations to support the design of the Highway 17 Twinning Project which extends from Scheel Drive westerly to 3 km west of Bruce Street within the County of Renfrew, Ontario. Thurber carried out the investigation under Ministry of Transportation (MTO) Assignment No. 4018-E-0009.

This report addresses the foundation investigation for the replacement of an existing CSP culvert with a structural culvert at about Sta. 9+600 on County Road 20 (Castleford Road) in Horton Township, north of the new Bruce Street Interchange with Highway 17.

This section of the report presents the factual findings obtained from the foundation investigation conducted by Thurber as part of the current study. Thurber carried out the investigation under Ministry of Transportation (MTO) Assignment No. 4018-E-0009.

The purpose of this investigation was to explore the subsurface conditions at the site and, based on the data obtained, to provide a borehole location plan, records of boreholes, stratigraphic profile, laboratory test results and a written description of the subsurface conditions.

It should be noted that the use of and reliance on Part 1 of the Report is governed by and limited to the terms and conditions set out in the Report and a reliance letter. The Preferred Proponent remains responsible to assess the need for additional investigations and to complete that work.

## **2 SITE DESCRIPTION**

### **2.1 General**

The culvert crosses County Road 20 approximately 400 m east of the intersection between Highway 17 and County Road 20. Within the project limits County Road 20 is also known as Castleford Road to the north of Highway 17. For project purposes, County Road 20 is herein described as oriented east-west, and the culvert is described as oriented north-south.



In the area of the culvert, the existing County Road 20 is a two-lane road and has a posted speed limit of 80 km/h. The road surface near the culvert is at approximate elevation 146.7 m. The shoulders are not paved and have a width of approximately 2.0 m. The 2016 traffic volume for this section of County Road 20 is understood to have been 2,033 AADT based on a 2016 Survey Count provided by the MTO.

The existing culvert near the site is a corrugated steel pipe (CSP) elliptical culvert with an approximately 3.6 m horizontal span, 2.4 m vertical rise, and 26.1 m length. The culvert is approximately perpendicular to the roadway alignment. The culvert has a relatively shallow gradient with the invert of the culvert at approximately elevation 142.6 m. The cover above the existing culvert is approximately 1.7 m at the roadway centerline. A creek flows through the culvert under the roadway embankment from north to south. In July 2024, the culvert inlet and outlet were partially flooded with a ponded water depth of approximately 550 mm above invert.

Embankment side slopes, in the vicinity of the culvert, are inclined at approximately 2.8H:1V to 3.2H:1V. The existing embankment side slopes at the culvert site did not show any visible signs of global instability at the time of the investigation.

The site is in a rural setting, and the terrain along the ditch line is relatively flat in the vicinity of the culvert site. The area directly adjacent to the culvert is mostly farmland with some deciduous trees and shrubs found along the creek line. Cobbles and boulders are present on the creek bed and banks north of the culvert inlet. Driveway marker 1446 is located approximately 70 m east of the culvert site. Wire fences on wooden posts are located near the culvert inlet and approximately 4 m east from the culvert outlet. Overhead utility lines parallel the westbound ditch line and cross the road near Sta. 9+576.

Photographs of the project area are included in Appendix D. These photographs show the existing condition of the roadway embankment and the culvert at the time of the field investigation.

## 2.2 Site Geology

It is noted that Thurber has completed a Foundation Investigation for the proposed Highway 17 Interchange at County Road 20. The results are presented in Geocres Report 31F-234.

According to Crins et al. 2009<sup>1</sup>, the project area is described as Ecoregion 6E (Lake Simcoe-Rideau Ecoregion) within the Mixedwood Plains Ecozone. According to Wester et al. 2018<sup>2</sup>, the ecoregion is subdivided into Ecodistrict 6E-16 (Pembroke Ecodistrict). The area is characterized by glaciolacustrine dominated landscape overlying a mix of Paleozoic to Precambrian bedrock.

Based on published geological information in *The Physiography of Southern Ontario* by Chapman and Putnam (1984), the site lies within the physiographic region known as the Ottawa Valley Clay

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<sup>1</sup> <https://files.ontario.ca/mnrf-ecosystemspart1-accessible-july2018-en-2020-01-16.pdf>

<sup>2</sup> <https://files.ontario.ca/ecosystems-ontario-part2-03262019.pdf>



Plains. The Ottawa Valley Clay Plains are characterized primarily by clay plains deposited by the Champlain Sea (Leda Clay) interrupted by ridges of rock or sand.

Ontario Geological Survey Map P.3784<sup>3</sup> for Precambrian Geology for the Horton Area suggests the bedrock is dolomitic and calcitic carbonate metasedimentary bedrock including grey to white calcite marble.

### 3 SITE INVESTIGATION AND FIELD TESTING

The foundation investigation and field-testing program was carried out between February 26 and March 18, 2024, and consisted of one on-road borehole identified as SC7-2 and two off-road boreholes identified as SC7-1 and SC7-3. The on-road borehole was advanced with a CME 55 truck mounted drill rig utilizing NW casing and coring techniques in bedrock. The off-road Borehole SC7-1 was advanced with a CME 55 track mounted drill rig utilizing NW casing and coring techniques, and Borehole SC7-3 was advanced with portable drilling equipment. Prior to commencement of drilling, utility clearances were obtained in the vicinity of the borehole locations.

A summary of the borehole coordinates, elevations, and termination depths is provided in Table 3-1. The locations and elevations of the boreholes were surveyed by Thurber with a Trimble Catalyst DA1 antenna with centimeter accuracy and were measured relative to BM HCP 102 (Elevation 129.023 m). Horizontal locations were measured by Thurber relative to existing site features. The elevations and borehole coordinates were reviewed and referenced to the survey data provided by the MTO. The borehole coordinates and elevations are shown on the Borehole Location and Soil Strata drawing included in Appendix A and on the individual Record of Borehole sheets included in Appendix B. The borehole coordinates are referenced to MTM Zone 9.

**Table 3-1: Borehole Summary**

<b>Borehole No.</b>	<b>Drilled Location</b>	<b>Northing (Latitude)</b>	<b>Easting (Longitude)</b>	<b>Ground Surface Elevation (m)</b>	<b>Termination Depth (m)</b>
SC7-1	Near Outlet	5 039 988.2 (45.499694°)	291 758.9 (-76.666871°)	143.7	7.7
SC7-2	Westbound Lane	5 040 011.2 (45.499901°)	291 751.9 (-76.666962°)	146.7	9.8
SC7-3	Near Outlet	5 040 011.9 (45.499907°)	291 737.2 (-76.667149°)	144.4	6.8

The boreholes were advanced to depths ranging from 6.8 to 9.8 m below the existing ground surface (base elev. 137.6 to 136.0 m). Soil samples were obtained at selected intervals using a split spoon sampler in conjunction with Standard Penetration Testing (SPT) in general

<sup>3</sup> <http://www.geologyontario.mndm.gov.on.ca/mines/data/google/mrd126/doc.kml>



accordance with ASTM D 1586. The portable drill used for Borehole SC7-3 was equipped with a full weight hammer, thus no adjustments were necessary for the SPT N values.

The drilling and sampling operations were supervised on a full-time basis by a member of Thurber's technical staff. The drilling supervisor logged the boreholes and processed the recovered soil and rock samples for transport to Thurber's Ottawa laboratory for further examination and testing.

A 50 mm diameter well was installed in each of Boreholes SC7-1 and SC7-3 to allow for measurements of the groundwater level after drilling. The details for the wells are illustrated on the respective Record of Borehole sheets provided in Appendix B. The monitoring wells installed as part of the current investigation will be decommissioned by Thurber, as outlined in the Hydrogeological Investigation and Design Report.

Borehole SC7-2 was backfilled in accordance with MOE requirements (O.Reg 903, as amended) and capped with cold patch asphalt to reinstate the pavement surface

#### **4 LABORATORY TESTING**

Laboratory testing was selected in accordance with the current MTO Guideline for Foundation Engineering Services, Section 5. Geotechnical laboratory testing consisted of natural moisture content determination and visual identification of all retained soil samples. At least 25% of the recovered soil samples were subjected to testing for grain size distribution analysis and, where appropriate, Atterberg Limits in accordance with MTO and ASTM standards. Chemical analysis for determination of pH, conductivity, resistivity, sulphide, sulphate and chloride was carried out on a sample of the soil.

The results of the geotechnical tests are summarized on the Record of Borehole sheets included in Appendix B and all laboratory results are presented on the figures included in Appendix C.

#### **5 GENERAL DESCRIPTION OF SUBSURFACE CONDITIONS**

Details of the encountered soil stratigraphy are presented on the Record of Borehole sheets included in Appendix B and the Borehole Location and Soil Strata Drawing included in Appendix A. A general description of the stratigraphy based on the conditions encountered in the boreholes is given in the following sections. However, the factual data presented on the Borehole Records takes precedence over the Soil Strata Drawing and the general description. It must be recognized that the soil and groundwater conditions may vary between and beyond borehole locations. Soil classification is in accordance with ASTM D2487 with the description of secondary components as outlined in the MTO Guideline for Foundation Engineering Services (GFES) Manual (April 2022) and the 4th Edition of the Canadian Foundation Engineering Manual.

In general, the encountered stratigraphy consists of silty sandy clay to silty sand fill over a native deposit of clayey silt to silty sand over marble bedrock.





## **5.1 Embankment Fill**

### **5.1.1 Asphalt**

Asphalt was encountered at the ground surface in the on-road borehole. The asphalt was measured to have a thickness of 50 mm.

### **5.1.2 Silty Sandy Clay Fill**

A fill layer consisting of silty sandy clay fill was encountered at ground surface in Borehole SC7-1. Organics were noted in the upper 25 mm of the layer. The thickness of the layer was 0.9 m (base elevation at 142.8 m). The layer consistency is described as soft based on tactile evaluations of strength.

The moisture content of the two samples tested were 32% and 49%.

### **5.1.3 Silty Sand Fill**

A fill layer consisting of silty sand with varying amounts of gravel was encountered below the silty sandy clay fill in Borehole SC7-1, below the asphalt in Borehole SC7-2, and at ground surface in Borehole SC7-3. Organics were noted in the layer in Borehole SC7-3. The thickness of the layer ranges from 0.2 to 4.6 m (base elevations ranging from 144.2 to 142.1 m). The SPT N-values ranged from 3 to 53, indicating a very loose to very dense condition.

The moisture content of the samples tested ranged from 3% to 31%. The results of grain size analyses conducted on two samples of this fill material are summarized below and are illustrated on Figure C1 in Appendix C.

**Summary of Grain Size Distribution Testing – Silty Sand Fill**

<b>Soil Particle</b>	<b>Percentage (%)</b>
Gravel	22 – 43
Sand	45 – 62
Silt & Clay	12 – 16

## **5.2 Clayey Silt (CL to CL-ML)**

A native deposit of clayey silt was encountered below the silty sand fill in Boreholes SC7-1 and SC7-2. Varying amounts of sand were noted within the layer. The thickness of the layer ranged from 0.7 to 2.3 m (base elev. 141.4 to 139.9 m). The SPT N-values ranged from 2 to 8 blows, indicating a stiff consistency.



The moisture content of the samples tested ranged from 19 to 32%. The results of two grain size analysis tests conducted on samples of this material are summarized in the table below and are illustrated on Figure C2 in Appendix C.

**Summary of Grain Size Distribution Testing – Clayey Silt**

Soil Particle	Percentage (%)
Gravel	0 – 1
Sand	8 – 39
Silt	38 – 56
Clay	22 – 36

The results of Atterberg Limits testing carried out on two samples of this material are summarized below and are illustrated on Figure C3 in Appendix C. The laboratory results indicate that the clayey silt is of low plasticity (CL to CL-ML).

**Summary of Atterberg Limit Testing – Clayey Silt**

Parameter	Value
Liquid Limit	17 – 28
Plastic Limit	11 – 17
Plasticity Index	6 – 11

### **5.3 Silty Sand**

A deposit of silty sand with varying amounts of gravel was encountered beneath the clayey silt in the Boreholes SC7-1 and SC7-2 and below the silty sand fill in Borehole SC7-3. Some clay was noted in the layer in Borehole SC7-3. The layer in Boreholes SC7-1, SC7-2, and the bottom 100 mm of Borehole SC7-3 were noted to be glacial till. The thickness of the layer ranged from 0.9 to 3.6 m (base elev. 140.6 to 139.0 m). The SPT N-values ranged from 2 to 82 blows, indicating a very loose to very dense relative density. Refusal blow counts were encountered at the base of the layer but are attributed to the bedrock surface. Although not observed within the boreholes, glacial till inherently contains cobbles and boulders.

The moisture content of the sample tested ranged from 7% to 18%. The results of gradation analyses completed on four samples of the layer are illustrated in Figure C4 of Appendix C. The results of the tests are summarized below and on the Record of Borehole sheets in Appendix B.

#### Summary of Grain Size Distribution Testing – Silty Sand

Soil Particle	Percentage (%)	
Gravel	3 – 23	
Sand	52 – 65	
Silt	22 – 27	18 – 28
Clay	6 – 10	

The results of Atterberg Limit testing conducted on the fines portion of a sample of the deposit from Borehole SC7-3 indicate a non-plastic material.

#### 5.4 Bedrock

Bedrock was proven by coring in all boreholes. The depth to bedrock ranged from 3.8 to 6.7 m (elevation 140.6 to 139.0 m). The bedrock surface appears to slope down from the culvert inlet to the outlet.

The bedrock encountered consisted of fine to medium grained, grey, strong to very strong marble. Photographs of the bedrock cores are provided in Appendix C. The rock core quality measurements are summarized in the Table 5-1.

**Table 5-1: Bedrock Details**

Parameter	Range
Total Core Recovery (TCR), %	100
Solid Core Recovery (SCR), %	47 – 100
Rock Quality Designation (RQD), %	30 – 100
Fracture Index (fractures per 0.3 m) <sup>(1)</sup>	0 – >10
Unconfined Compressive Strength (MPa)	75 – 148

Notes: (1) Indicated as “FI” on Borehole Logs

The RQD values in both boreholes ranged from 30% to 100%, indicating a bedrock of poor to excellent quality (CFEM, 2023). The results of unconfined compressive strength tests (UCS) ranged from 75 MPa to 148 MPa, indicating that the tested samples of the bedrock are strong to very strong (CFEM, 2023). The UCS test results are included in Appendix C.

#### 5.5 Groundwater

Monitoring wells with diameters of 50 mm were installed in Boreholes SC7-1 and SC7-3. Groundwater levels recorded in the wells are presented in Table 5-2.

**Table 5-2: Summary of Groundwater Levels**

Borehole No.	Bottom of Screen Elevation (m)	Groundwater Depth (m)	Groundwater Elevation (m)	Date of Measurement
SC7-1	139.1	0.4	143.3	March 07, 2024
		0.5	143.2	March 22, 2024
		0.3	143.4	April 10, 2024
		0.6	143.1	April 24, 2024
		0.9	142.8	June 04, 2024
		0.5	143.2	June 25, 2024
		0.9	142.8	August 30, 2024
SC7-3	141.8	0.5	143.9	March 08, 2024
		1.0	143.4	March 22, 2024
		0.6	143.8	April 10, 2024
		0.8	143.6	April 24, 2024
		1.3	143.1	June 04, 2024
		0.8	143.6	June 25, 2024
		1.1	143.3	July 10, 2024
		1.2	143.2	August 30, 2024

Approximately 0.6 m of ponded water was present near the culvert inlet and outlet in July 2024.

These observations are considered short term as they were recorded at discrete times, and it should be noted that the groundwater level at the time of construction may be different. Seasonal fluctuations of the groundwater level are to be expected. Furthermore, the groundwater level may be at a higher elevation after periods of significant and/or prolonged precipitation.

## 5.6 Analytical Testing

One sample of the native silty clay was submitted to Paracel Laboratories in Ottawa, Ontario for analysis of pH, water soluble sulphate, sulphide and chloride concentrations, resistivity, and conductivity. The analysis results are summarized in Table 5-3. Copies of the test results are provided in Appendix C.

**Table 5-3: Results of Chemical Analysis**

Borehole	Sample	Depth (m)	Chloride (µg/g)	Sulphate (µg/g)	Sulphide (%)	pH (-)	Resistivity (Ohm-cm)
SC7-1	SS3A	1.5 – 1.8	194	215	0.69	7.11	1,470

## 6 MISCELLANEOUS

The borehole locations reflect existing site features and access constraints. The as-drilled locations and ground surface elevation were measured by Thurber following completion of the



field program. Limitless Drilling Ltd. Renfrew, Ontario, supplied and operated the portable equipment, and George Downing Estate Drilling Ltd. of Hawkesbury, Ontario, supplied and operated the drill rigs used to drill, test, sample, and decommission the boreholes. Traffic control was performed in accordance with Ontario Book 7 and was provided by C&C Services of Renfrew, Ontario. The field investigation was supervised on a full-time basis by Mr. I. Khan, EIT, Mr. D. Amorim Pereira, Geotechnical Technician, and Mr. R. Howarth, Geotechnical Technician. Overall supervision of the field investigation program was provided by Mr. J. Gray, P.Eng.

Routine geotechnical laboratory testing were completed by Thurber's laboratory in Ottawa. UCS testing were completed by Thurber's laboratory in Oakville. Analytical testing was completed by Paracel Laboratories Ltd. in Ottawa.

Interpretation of the factual data and preparation of this report was completed by A. de Oliveira, P.Eng. The report was reviewed by Dr. F. Griffiths, P.Eng. and Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundation Projects.

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**PART 2. ENGINEERING DISCUSSION AND RECOMMENDATIONS**

**7 INTRODUCTION**

Part 2 of the report provides an interpretation of the factual data from Part 1 and presents preliminary geotechnical recommendations to assist the project team in designing the foundations for the replacement of a culvert located on County Road 20 near Station 9+600 in Horton Township within the County of Renfrew, Ontario and associated retaining walls. Thurber carried out the investigation under Ministry of Transportation (MTO) Assignment No. 4018-E-0009.

This preliminary foundation investigation and design report with the interpretation and recommendations are intended for the use of the Ministry of Transportation and shall not be used or relied upon for any other purposes or by any other parties including design-build contractors. It should be noted that the use of and reliance on Part 1 of the Report is governed by and limited to the terms and conditions set out in the Report and a reliance letter. The Preferred Proponent remains responsible to assess the need for additional investigations and to complete that work. The Preferred Proponent must make their own interpretation based on the factual data in Part 1 of the report. The information included in Part 2 is not to be relied upon for design purposes and foundation design is the sole responsibility of the Preferred Proponent. No use shall be made of Part 2 or any part thereof. The Preferred Proponent must make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction methods, and scheduling.

The following sections provide preliminary geotechnical recommendations for the construction of foundation elements for the proposed structures. The discussion and preliminary recommendations presented in this report are based on information provided by the MTO and the factual data obtained during the current field investigation.

**7.1 Background Information**

The culvert site is approximately 400 m west of the intersection between Highway 17 and Country Road 20. The existing road surface is near elevation 146.7 m. The existing culvert is a corrugated steel pipe (CSP) elliptical culvert with a span of 3.6 m, a rise of 2.4 m, and a length of 26.1 m. The invert of the existing culvert is near elevation 142.6 m. The cover above the existing culvert is



approximately 1.7 m at the highway centerline. The creek flows through the culvert under the highway embankment from north to south.

In general, the encountered stratigraphy consists of silty sandy clay to silty sand fill over a native deposit of clayey silt to silty sand over bedrock at elevations ranging from 140.6 to 139.0 m. The bedrock surface appears to slope down from the culvert inlet to the outlet. Groundwater was recorded in the monitoring wells at elevations ranging from 143.9 to 142.8 m. Approximately 550 mm of water was ponding near the culvert inlet and outlet in July 2024.

## 7.2 Proposed Structure

The Structure and Culvert List of February 23, 2022, for this project indicated that the existing culvert is to be replaced with a structural culvert along a similar alignment and with a similar invert. The new culvert will be a concrete box with a span of 3 m and a rise of 2.4 m.

Available AutoCAD drawings provided by the MTO indicate that County Road 20 is to be realigned as part of the Bruce Street Interchange construction with the realignment centreline transitioning back to existing at approximately 9+500. The proposed location of the culvert is within the transition zone, with the new centreline near the north edge of pavement of the existing roadway. The new cross section will include 3.5 m wide lanes and 2.5 m wide paved shoulders for a total width of 12 m which is approximately 1.0 m wider than the existing platform. The design profile is also being altered with a grade raise near Station 9+600 of approximately 2.0 m (finished grade near elevation 148.7 m). A right-of-way constraint on the north side of County Road 20 has resulted in the inclusion of a 40 m long retaining wall on that side of the embankment. Details on the retaining wall are not available; however, it is anticipated that it will have a free height of approximately 4 to 5 m in the area of the culvert.

**It is noted that preliminary GA drawings are not available at the time of writing. The preliminary recommendations presented herein must be reassessed once the type, configuration, location, elevation, and orientation of the proposed works are established.**

## 7.3 Applicable Codes and Design Considerations

The geotechnical assessment presented herein has been prepared based on the available data regarding the proposed work, existing ground conditions documented in Part 1 of this report, and in accordance with the Canadian Highway Bridge Design Code (CHBDC), version CSA S6-19.

In accordance with the CHBDC, the analysis and design of the structure takes into consideration the importance of the structure and the consequence associated with exceeding limit states. The importance category and consequence classification are defined by the Regulatory Authority which, in this case, is the Ministry of Transportation, Ontario (MTO).

It is understood that the structures are to be designed to the “Other” importance category (pending confirmation by MTO). It is understood that the new structures have been assigned a “Low” Consequence classification in accordance with Section 6.5.1 of the CHBDC (pending



confirmation by MTO). Accordingly, a consequence factor ( $\Psi$ ) of 1.15, as per Table 6.1 of the CHBDC, has been used in assessing factored geotechnical resistances. If this consequence classification changes, the geotechnical assessment and recommendations provided within this report will need to be reviewed and revised.

As per Section 6.5.3.2 of the CHBDC, the degree of site prediction model understanding is considered to be “Typical” based on the current information.

The frost penetration depth and associated recommendations are provided in Section 10.6.

## 8 SEISMIC CONSIDERATIONS

### 8.1 Spectral and Peak Acceleration Hazard Values

The seismic hazard data for the CHBDC is based on the fifth-generation seismic model developed by the Geological Survey of Canada (GSC)<sup>4</sup>. The GSC seismic hazard calculation data sheet for this site for the *reference* ground condition (Site Class C) is presented in Appendix E. The site coefficients used to determine the design spectral acceleration values are a function of the Site Class, PGA, and  $S_a$  (0.2). The PGA value at this site provided by GSC for a *reference* Site Class C with a 2% probability of exceedance in 50 years (2475-year event) is 0.229g. This value is to be scaled by the  $F(PGA)$  based on the *site-specific* Site Class, as discussed in Section 8.3. As per Section 6.14.2.3.c of the CHBDC, an “Other” geotechnical system beyond an approach embankment bridge interface zone has no seismic performance requirements for travelled lanes.

### 8.2 Seismic Liquefaction Potential

It is anticipated that the foundations for the retaining wall will be supported on bedrock which is not susceptible to liquefaction.

The clayey silt observed in Borehole SC7-1 behaves as “sand-like” and, based on the SPT values following the simplified method for cohesionless soil as outlined in Boulanger and Idriss (2014)<sup>5</sup>, is susceptible to liquefaction under a 1 in 2475 year earthquake. It is anticipated that this material will be removed from beneath the culvert structure as part of the subgrade preparation.

The observed embankment fill has also been assessed using the SPT data, and the soils are considered not susceptible to liquefaction during a 1 in 475 year design earthquake.

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<sup>4</sup> <https://earthquakescanada.nrcan.gc.ca/hazard-alea/interpolat/calc-en.php>

<sup>5</sup> Boulanger, R. W., and Idriss, I. M. (2014). *CPT and SPT based liquefaction triggering procedures*, Report No. UCD/CGM-14/01, Center for Geotechnical Modeling, Department of Civil and Environmental Engineering, University of California, Davis, CA, 134 pp.



The susceptibility of the cohesive soils at this site to experience liquefaction/cyclic softening was assessed following the Boulanger and Idriss (2007)<sup>6</sup> criteria which utilizes the measured undrained shear strengths. Based on the results of the analysis, the cohesive materials at this site are not susceptible to liquefaction or cyclic mobility under the design earthquake.

### 8.3 CHBDC Seismic Site Classification and Performance Category

In accordance with Section 4.4.3.2 of the CHBDC, the selection of the seismic site classification is based on the nature of the soil deposits within the upper 30 m of the stratigraphy. As per Table 4.1 within Section 4.4.3.2 of the CHBDC, the site has been classified as a Seismic Site Class B as it is anticipated that all structure foundations will be within 3 m of bedrock.

The  $F(PGA)$ , as per Table 4.8 within Section 4.4.3.3 of the CHBDC, is equal to 0.87 for this site yielding a scaled *site-specific* Site Class B PGA of 0.199g.

As per Section 4.4.4 of the CHBDC, the Seismic Performance Category is assigned based on the fundamental period, the importance category, and the spectral accelerations scaled to the site class. The  $F(0.2)$  and  $F(1.0)$ , as per Tables 4.2 and 4.4 within Section 4.4.3.3 of the CHBDC, are equal to 0.77 and 0.63 respectively for this site, yielding a scaled *site-specific*  $S_a(0.2)$  of 0.275 and  $S_a(1.0)$  of 0.062. A Seismic Performance Category of 3 is applicable to this site based on Table 4.10 of the CHBDC assuming the fundamental period of the structure is less than 0.5 seconds. The seismic performance category should be confirmed by the culvert designer.

## 9 DESIGN OPTIONS

### 9.1 Culvert Type and Foundation Alternatives

Selection of the replacement culvert type must typically consider the proposed construction procedures, staging requirements, geotechnical resistance available in the foundation soils, depth to suitable bearing stratum, and post-construction settlement. It is understood that for this structure a concrete box culvert has been identified as preferred due to its increased durability. From a geotechnical perspective, the following culvert types were considered:

- Circular Pipes (Concrete, HDPE, Steel)

Although, from a foundation engineering perspective, a pipe culvert is a technically feasible alternative, the proposed pipe must meet the required flow capacity and hydraulic requirements.

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<sup>6</sup> Boulanger, R. W. and Idriss, I. M. (2007). *Evaluation of cyclic softening in silts and clays*, ASCE, *Journal of Geotechnical and Geoenvironmental Engineering*, 133(6), 641-652.

- Open-Bottom Culvert (Box, Arch)

The construction of an open-bottom culvert will have greater construction concerns due to the shallow water table and requirement for greater excavation depths to construct the culvert footings to satisfy frost depth requirements. It is anticipated that the underside of the footings would be at approximately elevation 140.7 m which is more than 2 m below the observed groundwater level and approximately 6 m below the existing road surface. The use of an open bottom culvert would require greater dewatering and excavation efforts. It is also noted that the supporting soil would consist of clayey silt for portions of the culvert; the clayey silt will have limited geotechnical resistance.

- Closed-Bottom Culvert (Box)

A pre-cast, segmental, closed-bottom, box culvert is considered a feasible option from a foundation engineering perspective. Precast sections, rather than cast-in-place construction, can be installed expediently with less potential for disturbance of the subgrade during installation require less excavation depth than open bottom culvert and more manageable dewatering efforts. It is anticipated that the underside of the bedding layer would be at approximately elevation 142.0 m which is approximately 1m below the groundwater level observed on June 4, 2024.

Given the depth to bedrock and the soil and groundwater conditions observed on site, a closed bottom culvert is recommended. A concrete pipe culvert is also considered as a viable alternative at this site.

## **9.2 Construction Methodology Alternatives**

Construction methods that were considered for the proposed culvert replacement under the existing County Road 20 are presented below.

- Open Cut with Full Road Closure and Temporary Detour

Installation of the new culvert and retaining wall using open cut techniques and a full road closure would allow for an expedited construction schedule and could reduce costs associated with roadway protection as well as surface and ground water control. However, an acceptable detour route must be available before this option is carried forward.

- Open Cut with Staged Construction and Temporary Widening

Installation of a new culvert using an open cut with a temporary widening to accommodate passage of traffic during construction is considered feasible from a foundation perspective. However, the limited available right-of-way, the presence of overhead utilities, and the ponded water near the outlet limit the feasibility of significant embankment widening and

would need to be considered in design. Depending on the temporary alignment and length, an additional borehole investigation program may be required to determine the subsurface conditions along the potential detour alignment.

- Open Cut with Staged Construction and Temporary Protection System

Installation of a new culvert using an open cut staged replacement is considered feasible from a foundation perspective. The option would require a Temporary Protection System (TPS), as discussed further in Section 11.2, installed along the embankment centerline to maintain a single lane of traffic flow along the current roadway alignment. The Contractor would need to consider the potential for encountering obstructions in the glacial till layer during the design and installation of roadway protection as well as the presence of shallow bedrock. To reduce lateral deflections, the TPS may need to include anchoring and/or bracing or have the TPS drilled into the bedrock. The height of the TPS could be reduced if the road alignment constraints allowed for a temporary grade lowering to be included.

- Open Cut with Staged Construction with Temporary Grade Lowering

Installation of a new culvert using an open cut staged replacement with grade lowering to maintain movement of traffic within the existing embankment footprint is considered a feasible option from a foundation perspective. If TPS is still required after grade lowering is included, the height of the TPS could be reduced. Over excavation of the existing fills will be required to allow placement of granulars for a temporary pavement structure.

- Open Cut with Temporary Modular Bridge

Installation of a new culvert using an open cut with a temporary modular bridge (TMB) to provide traffic passage over the open excavation is not considered feasible due to the limited clearance which would be available under a modular bridge at this site.

- Trenchless Techniques

Installation of a new culvert using trenchless techniques is not considered feasible due to the potential for encountering a mixed face (including possible bedrock), and insufficient soil cover for both the existing and proposed profiles.

### **9.3 Recommended Approach for Culvert Replacement**

From a foundation engineering perspective, it is recommended that the existing culvert be replaced with a precast segmental closed bottom box culvert (or concrete pipe culvert) using open cut staged construction and full road closure, if possible. If traffic must be maintained during construction, a temporary protection system (TPS) will be required. The height of the TPS could



be reduced if the road alignment constraints allow for a temporary grade lowering to be included. It is noted that a soldier pile and lagging approach will be required for the TPS due to the presence of shallow bedrock: additional efforts will likely be required in terms of bracing, rakers or drilled in piles. Similar techniques will likely be required for the permanent retaining wall.

## 10 PRELIMINARY FOUNDATION DESIGN RECOMMENDATIONS

From a foundation engineering perspective, a concrete closed box culvert is recommended. The following bullets summarize the relevant elevations for this site:

- |   |                    |
|---|--------------------|
| • Existing top of pavement County Road 20       | 146.7 m            |
| • Proposed top of pavement County Road 20       | 148.7 m            |
| • Culvert invert                                | 142.6 m            |
| • Approximate elevation of underside of bedding | 142.0 m            |
| • Groundwater elevation                         | 143.2 m            |
| • Clayey Silt /Glacial Till interface           | 141.4 m to 139.9 m |
| • Bedrock surface                               | 140.6 m to 139.0 m |

### 10.1 Concrete Pipe Culvert Foundation

It is anticipated that the base of the excavation for the replacement culvert will be within the clayey silt and silty sand. Bearing resistance values are not required for pipe culverts. The culvert should be founded on a granular bedding layer (see Section 10.4). Subgrade preparation should follow the recommendations provided in Section 10.4 to provide a suitable subgrade for the bedding. Surface water diversion and dewatering will be required to place the bedding material and install the culvert in the dry (see Section 11.3).

If a concrete pipe is selected, resistance to lateral forces/sliding resistance between concrete and the underlying granular 'A' bedding (see Section 10.4) should be evaluated based on the recommendations in Section 10.4.

### 10.2 Closed-Bottom Box Concrete Culvert

It is understood that the replacement culvert will have the same invert elevation as the existing culvert (approximately 142.6 m). Subgrade preparation should follow the recommendation provided in Section 10.4. Surface water diversion and dewatering may be required to prepare the subgrade and install the culvert in the dry (see Section 11.3).

The existing subgrade soils at the founding elevation were observed to be clayey silt or silty sand. For a box culvert with an exterior width of as much as 3.6 m founded on a properly prepared and compacted granular bedding layer, the design can be based on factored geotechnical resistance values as follows:



- Factored Geotechnical Resistance at ULS of 300 kPa
- Factored Geotechnical Resistance at SLS of 100 kPa

The factored geotechnical resistances include the following factors:

- Consequence factor ( $\Psi$ ) of 1.15 (as per CHBDC, Table 6.1)
- Geotechnical resistance factors (as per CHBDC, Table 6.2)
  - $\phi_{gu} = 0.50$  (static analysis; typical degree of understanding)
  - $\phi_{gs} = 0.80$  (static analysis; typical degree of understanding)

The bearing resistance values are for vertical, concentric loading. In the case of eccentric or inclined loading, the bearing resistance must be adjusted in accordance with CHBDC Clause 6.10.2. Foundation settlement, based on the supplied SLS resistance, is expected to be as much as 25 mm. The bearing resistances provided above are based on the assumption that subgrade is prepared as recommended in Section 10.4 and 10.6.

Resistance to lateral forces/sliding resistance between precast concrete and underlying granular bedding (see Section 10.4) should be evaluated in accordance with the CHBDC assuming an unfactored coefficient of friction of 0.45 for precast concrete. A geotechnical resistance factor of 0.8 ( $\phi_{gu}$ ), as per Table 6.2 of the CHBDC (static analysis – typical understanding) should be applied to the sliding frictional capacity between concrete and granular bedding.

### 10.3 Retaining Walls

Based on preliminary information provided by MTO, a retaining wall is to be built near the inlet. The retaining wall foundations if supported on soil will require frost protection of 1.9 m of soil cover. At the low point, (at the culvert) the ground surface will be at 142.6, thus the footings should be set at elevation 140.7 m. The top of bedrock was observed at elevation 140.6 m in Borehole SC7-3 which was drilled near the inlet. Thus, it is recommended that concrete retaining walls be supported on spread footings founded directly on bedrock.

**It is noted that the bedrock surface is variable and may be encountered at varying depth along the length of the proposed retaining wall. Additional boreholes should be drilled along the length of the retaining wall during a subsequent design stage to confirm bedrock elevations.**

The recommended geotechnical resistances for a 3 m wide footing cast on sound bedrock or on mass concrete on bedrock are as follows:

- Factored Geotechnical Resistance at ULS of 3,000 kPa
- Factored Geotechnical Resistance at SLS will not govern for footings on bedrock



The factored geotechnical resistances include the following factors:

- Consequence factor ( $\Psi$ ) of 1.15 (as per CHBDC, Table 6.1)
- Geotechnical resistance factors (as per CHBDC, Table 6.2)
  - $\phi_{gu} = 0.50$  (static analysis; typical degree of understanding)
  - $\phi_{gs} = 0.80$  (static analysis; typical degree of understanding)

The bearing resistance values are for vertical, concentric loading. In the case of eccentric or inclined loading, the bearing resistance must be adjusted in accordance with CHBDC Clause 6.10.2.

Resistance to lateral forces/sliding resistance between the concrete and the underlying bedrock (Section 10.4) should be evaluated in accordance with the CHBDC assuming an unfactored coefficient of friction of 0.7. A geotechnical resistance factor of 0.8 ( $\phi_{gu}$ ), as per Table 6.2 of the CHBDC (static analysis – typical understanding) should be applied to the sliding frictional capacity. If sufficient lateral resistance is not available, rock dowels could be considered.

A retained soil system (RSS) for a retaining wall is not recommended at this site as it would be located within a watercourse and could be affected by fluctuating water levels.

## **10.4 Subgrade Preparation, Bedding and Backfilling**

“Granular A” and “Granular B Type II” in this section refer to OPSS Granular A or Granular B Type II meeting the specifications of OPSS.PROV 1010 and SP110S06. “Granular A” is further defined as “Quarry-Source Granular A” unless specifically described as “Pit-Source Granular A”. Fills should be placed and compacted as per OPSS.PROV 501 and OPSS.PROV 206.

### **10.4.1 Culvert**

The culvert should be constructed following OPSS.PROV 401 and either OPSS.PROV 421 (pipe culvert) or OPSS.PROV 422 (box culvert).

Subgrade preparation for the culvert replacement should include excavation and removal of the existing culvert if replaced along the same alignment. If the replacement culvert is placed on a new alignment, the existing culvert may be decommissioned in place (see Section 10.8.2 for further details).

At the founding level existing fill, soft/loose soils, disturbed soils, or otherwise deleterious materials encountered will need to be removed down to competent inorganic soils. Construction traffic should not travel on the exposed subgrade. Granular A should be used in dewatered excavations to backfill any sub-excavations required for subgrade improvement. Given the sensitive subgrade clayey silt soils anticipated at the founding level of the culvert, construction equipment should not be permitted to travel on the exposed subgrade. The compaction of



granular directly above the subgrade may result in disturbance of the material with pumping of fines into the granular and difficulty achieving the specified degree of compaction. After inspection and approval of the subgrade, protection of the subgrade should include installation of a Class II, non-woven geotextile with a maximum FOS of 150  $\mu\text{m}$  (OPSS.PROV 1860) installed beneath the Granular A material. The geotextile should be placed as soon as possible after preparation of the final subgrade level. Alternately, the geotextile and granular pad could be replaced with a 200 mm thick, concrete working slab placed on the prepared subgrade. The working slab should extend at least 0.5 m beyond the outside dimensions of the culvert. An NSSP is provided in Appendix H to include in the contract documents to alert the Contractor of the sensitive nature of the foundation soils.

Foundation preparation for a pipe culvert should be as per OPSS 802.031 and OPSS 803.031 with bedding extending to 300 mm below the pipe. It is recommended that culvert cover and bedding materials consist of OPSS.PROV 1010 Granular A.

The closed box culvert will be founded on existing soils, the foundation subgrade should be prepared as per OPSS.PROV 902 using Granular A material as backfill of over-excavated areas, where required. The box culvert should be placed on a granular pad with a minimum thickness of 0.3 m consisting of Granular A material. The top of the Granular A pad must extend to 0.5 m beyond the outside edge of all sides of the culvert and sloped away from the footing at 1H:1V, or flatter. The granular bedding shall be compacted as per OPSS.PROV 501.

Backfill and cover for concrete box culverts should be as per OPSS 803.010 with cover material consisting of OPSS.PROV 1010 Granular A. Backfill above the granular cover material for a box or rigid pipe culvert should be in accordance with OPSS.PROV 902 and consist of materials meeting the requirements of OPSS Select Subgrade Material (SSM) or better.

Heavy compaction equipment, used adjacent to or directly above the culvert, must be restricted in accordance with OPSS.PROV 501 to protect the culvert from damage.

It is noted that construction will extend below the observed water level. Dewatering will be required to place the granular bedding and/or concrete in the dry. Please review Section 11.3 for additional comments on groundwater and surface water control.

#### **10.4.2 Retaining Wall**

The geotechnical resistance recommendations provided in Section 10.3 for the retaining wall are applicable for foundation elements placed directly on sound, unfractured bedrock or on mass concrete on sound, unfractured bedrock. The top of bedrock elevation is variable; sloping bedrock will likely be encountered within the excavation footprint. Bedrock excavation and/or mass concrete should be used to provide a flat surface for the footings. The foundation subgrade should be prepared as per OPSS.PROV 902 using mass concrete as backfill, where required. The mass concrete should be the same class and strength as the footing concrete. All shattered and loosened rock fragments should be removed from the footprint of the footing. The bedrock surface





shall be cleaned with a hydrovac or air-lance prior to the placement of concrete to create a clean bedrock/concrete interface.

It is noted that construction will extend below groundwater elevation. Creek diversion and dewatering will be required to prepare the subgrade in the dry. Please refer to Section 11.3 for additional comments on groundwater and surface water control.

The limits of structural backfill should be in accordance with OPSD 3101.150. Structural backfill adjacent to the wingwall/retaining wall should consist of OPSS Granular A or Granular B Type II placed and compacted in accordance with OPSS.PROV 501. Heavy compaction equipment used adjacent to the retaining wall must be restricted in accordance with OPSS.PROV 501.07.02a.

## **10.5 Backfill and Lateral Earth Pressures**

Structural backfill material should consist of Granular A or Granular B Type II meeting the OPSS.PROV 1010 and SP110S06 specifications. Large scale direct shear box testing on samples of Granular A and Granular B Type II from several nearby aggregate sources was completed for this project. The results indicate that for design of structural backfill for this project, an internal angle of friction of 40 degrees and 42 degrees can be used for quarry-sourced Granular A and Granular B Type II, respectively, generated within this area provided the effective vertical pressure on the material is less than 150 kPa (Geocres Memorandum 31F-213). An Operational Constraint will be required in the contract restricting the source of Granular A to local quarries. Throughout this report, the term "Granular A" is defined as "Quarry-Source Granular A" unless specifically described as "Pit-Source Granular A".

The backfill must be in accordance with OPSS.PROV 902 and placed to the extents shown on OPSD 3101.150 for the culvert and retaining walls. Structural backfill should consist of Granular A or Granular B Type II placed and compacted in accordance with OPSS.PROV 501. Heavy compaction equipment used adjacent to the walls must be restricted in accordance with OPSS.PROV 501.07.02a). The design of the retaining walls must incorporate a subdrain as shown in OPSD 3101.150.

Lateral earth pressure parameters provided in the sections below are based on the assumptions that the wall is vertical and the backfill is fully drained so that there are no unbalanced hydrostatic pressures above the permanent groundwater level. If adequate drainage cannot be confirmed, the potential for buildup of hydrostatic pressures should be considered in design.

Where back slopes are horizontal, the corresponding coefficients provided in Table 10-1 and Table 10-2 should be used. For other backfill and wall geometries, Thurber will need to calculate the appropriate earth pressure coefficients once the final geometry is confirmed.

### **10.5.1 Static Lateral Earth Pressure**

Lateral earth pressures acting on structures should be computed in accordance with the CHBDC. Under drained conditions the lateral earth pressure is generally given by the following expression:



$$\sigma_h = K * (\gamma h + q)$$

where:

$\sigma_h$	=	horizontal pressure on the wall at depth h (kPa)
K	=	earth pressure coefficient (see table below) ( $K_a$ for unrestrained walls, $K_o$ for restrained walls)
$\gamma$	=	unit weight of retained soil (see table below), use submerged unit weight below groundwater level
h	=	depth below top of fill where pressure is computed (m)
q	=	value of any surcharge (kPa)

A lateral earth pressure due to backfill compaction should be added to the calculated lateral earth pressure in accordance with Clause 6.12.3 of the CHBDC. Typical earth pressure coefficients for OPSS Granular A and OPSS Granular B Type II backfill are shown in Table 10-1.

**Table 10-1: Static Earth Pressure Coefficients**

Condition	Pit Sourced OPSS Granular A $\phi = 35^\circ, \gamma = 22.8 \text{ kN/m}^3$	Quarry Sourced OPSS Granular A $\phi = 40^\circ, \gamma = 22.8 \text{ kN/m}^3$	Quarry Sourced OPSS Granular B Type II $\phi = 42^\circ, \gamma = 22.8 \text{ kN/m}^3$
Coefficient of at Rest Earth Pressure, $K_o$ (Restrained Wall)	0.43	0.36	0.33
Coefficient of Active Earth Pressure, $K_a$ (Unrestrained Wall)	0.27	0.22	0.20
Coefficient of Passive Earth Pressure, $K_p$ (Movement toward soil)	3.7	4.60	5.0

The parameters in the table correspond to full mobilization of active and passive earth pressures and require certain relative movements between the wall and adjacent soil to produce these conditions. The movement required can be assessed from Table C6.12 of the Commentary to the CHBDC. Active earth pressures should be used for unrestrained walls. For rigid structures, at-rest horizontal earth pressures would apply for design.

### 10.5.2 Combined Static and Seismic Lateral Earth Pressure

In accordance with Clause 6.14 of the CHBDC, retaining structures should be designed using dynamic earth pressure coefficients that incorporate the effects of earthquake loading. The following recommendations are per Section C6.14 of the Commentary of the CHBDC which states

that seismically induced lateral soil pressures may be calculated using Mononobe Okabe Method with:

- $k_h = \frac{1}{2} * F(PGA) * PGA$ , for structures that allow 25 to 50 mm of movement, and
- $k_h = F(PGA) * PGA$ , for restrained walls

The coefficients of horizontal earth pressure for seismic loading presented in Table 10-2 may be used for vertical walls. The provided earth pressure coefficients are based on a Seismic Site Class B. Please see Section 8.3 for the respective PGA and F(PGA) values.

**Table 10-2: Combined Static and Seismic Earth Pressure Coefficients – Site Class B (2,475-year)**

Condition	Pit Sourced OPSS Granular A $\phi = 35^\circ, \gamma = 22.8 \text{ kN/m}^3$	Quarry Sourced OPSS Granular A $\phi = 40^\circ, \gamma = 22.8 \text{ kN/m}^3$	Quarry Sourced OPSS Granular B Type II $\phi = 42^\circ, \gamma = 22.8 \text{ kN/m}^3$
Coefficient of Active Earth Pressure, $K_{AE}$ (Restrained Wall)	0.39	0.33	0.30
Coefficient of Active Earth Pressure, $K_{AE}$ (Unrestrained Wall)	0.33	0.27	0.25

The total pressure due to combined static and seismic loads acting at a specific depth below the top of the wall/soil may be determined using the following equation that includes consideration of material properties and the soils profile.

$$\sigma_{hAE} = K * \gamma * d + (K_{AE} - K_A) * \gamma * (H - d)$$

where:

$\sigma_{hAE}$	=	combined static and seismic lateral earth pressure on wall at depth d (kPa)
d	=	depth below the top of the wall where pressure is computed (m)
K	=	static earth pressure coefficient ( $K_A$ for unrestrained walls, $K_0$ for restrained walls)
$\gamma$	=	unit weight of retained soil, adjusted below water level
$K_{AE}$	=	combined static and seismic earth pressure coefficient
H	=	total height of the wall (m)

## 10.6 Frost Penetration Depth

The depth of frost penetration at this site is estimated to be 1.9 m (as per OPSD 3090.101); shallow foundations, if any, should be founded at or below this depth or provided with equivalent insulation. Closed-bottom box culverts are not typically provided with frost protection. The earth



cover should be measured perpendicular to the ground surface. Thermally equivalent frost protection could be in the form of insulation provided it is placed *above* the high-water level. It should be noted that open graded materials, such as rock protection, do not have the same thermal protection as soils.

Please also refer to the pavement design report for frost taper recommendations for the pavement.

## **10.7 Cement Type and Corrosion Potential**

Chemical analysis for determination of pH, water soluble sulphate, sulphides, chloride concentrations, resistivity and electrical conductivity was carried out on samples of the native materials. The analysis results are summarized in Section 5.6 and a copy of the test results is provided in Appendix C.

The pH, resistivity and chloride concentration provide an indication of the degree of corrosiveness of the sub-surface environment. The test results provided in Section 5.6 were compared with Table 3.2 of the MTO Gravity Pipe Design Guideline and generally indicate a severe corrosive environment. The test results provided in Section 5.6 may be used to aid in the selection of coatings and corrosion protection systems for buried steel objects.

The concentration of soluble sulphate provides an indication of the degree of sulphate attack that is expected for concrete in contact with the soil and groundwater at the site. The sulphate results were compared with Table 3 of Canadian Standards Association Standards A23.1-19 (CSA A23.1) and indicate a less than moderate degree of sulphate attack potential on concrete structures at this site.

The corrosive effects of road de-icing salts should also be considered.

## **10.8 Embankment Design and Reinstatement**

Embankments should be constructed in accordance with OPSS.PROV 206. Local marine clay must not be used as embankment fill.

### **10.8.1 Embankment Stability**

The existing highway embankment side slopes are generally sloped at approximately 2.8H:1V to 3.2H:1V. The existing slopes did not show visible signs of global instability at the time of investigation.

It is understood that a grade raise and centreline shift are proposed along the County Road 20 alignment. It is also understood that right-of-way constraints require the inclusion of a retaining wall on the inlet side of the highway.

Embankment reinstatement after construction of the replacement culvert should be carried out in accordance with OPSS.PROV 206. The fill should be placed and compacted in accordance with OPSS.PROV 501. Where newly placed embankment fill is placed against existing embankment slopes or on a sloping ground surface steeper than 3H:1V, benching of the existing slope should be carried out in accordance with OPSD 208.010.

Embankment stability has been assessed perpendicular to the highway alignment. Analyses were completed for the outlet side of the reinstated embankment. The slope stability analyses were carried out using GeoStudio 2024 Slope/W software for limit equilibrium analysis. Input parameters, soil model and groundwater conditions for the analyses are based on the in-situ testing and the results of laboratory testing and are shown on the stability analyses output figures provided in Appendix G. The following additional parameters and assumptions were used in the analysis:

- An existing maximum fill height of 4.5 m based on the existing road elevation of 146.7 m and the top of native soils below elevation 142.2 in Borehole SC7-1.
- The embankment reinstatement/grade raise is constructed of Select Subgrade Material (SSM) sloped at 2H:1V. The grade raise is approximately 2 m above existing fill (finished road elevation of 148.7 m).
- The retaining wall on the inlet side of the culvert is founded on bedrock.
- As discussed in Section 8.1 and as per Section 6.14.2.3.c of the CHBDC, an “Other” geotechnical system outside an approach embankment bridge interface zone has no seismic performance requirements for travelled lanes. Therefore, seismic loading has not been included in the assessment.
- A traffic surcharge of 17 kPa has been applied as a temporary load.

Copies of the output from the stability analyses are provided in Appendix G. Each output figure shows the slope geometry, groundwater conditions, soil stratigraphy and soil strength parameters utilized in the analysis.

The stability analyses generated the factor of safety values shown below in Table 10-3.

The geotechnical resistance factors provided in Table 6.2 of the CHBDC for embankment fills with a typical degree of understanding and a consequence factor ( $\Psi$ ) of 1.15, generates minimum target Factors of Safety of 1.34 and 1.16 for permanent and temporary conditions, respectively. All the static results presented in Table 10-3 meet or exceed the target Factors of Safety.

**Table 10-3: Slope Stability Results for County Road 20 - Near Outlet**

Condition	Case	Factor of Safety 2H:1V Select Subgrade Material
Permanent	Long Term	1.9 (Fig G1-1)
Temporary (traffic loading)	Short Term	1.9 (Fig G1-2)

## 10.8.2 Embankment Settlement

The proposed grade raise will increase the height of fill from 4.5 m to 6.5 m. The loading imposed from the new fill will increase the effective stress in underlying soil deposits and induce consolidation settlement in the clayey silt layer and elastic settlement in the granular deposits at the site.

In accordance with MTO's document "Embankment Settlement Criteria for Design" (March 2, 2010), the criteria adopted for embankment design at this site is as per Table 10-4:

**Table 10-4: Summary of MTO Settlement Criteria**

Distance from Abutment	0-20 m	20-50 m	50-75 m	>75 m	Post Construction Settlement Period
Settlement Limits Non-Freeway	25 mm	50 mm	100 mm	200 mm	15 years

Representative site stratigraphy was developed based on the Record of Borehole logs with material properties based on the results of in-situ field testing and laboratory testing. The consolidation parameters utilized were those for the clayey silt layer encountered in boreholes closer to Highway 17 as described in Geocres Report 31F-234. The soil parameters used in the models are summarized in Table 10-5, below.

Analyses were carried out to calculate the predicted settlement with time, considering SSM embankments (unit weight of 21 kN/m<sup>3</sup>) and soil stratigraphy based in Borehole SC7-1.

**Table 10-5: Summary of Material Parameters**

Soil Type	Thickness (m)	Unit Weight (kN/m <sup>3</sup> )	Settlement Parameters					
			P' <sub>c</sub> (kPa)	e <sub>0</sub>	C <sub>c</sub>	C <sub>r</sub>	C <sub>v</sub> (cm <sup>2</sup> /s)	C <sub>vr</sub> (cm <sup>2</sup> /s)
Clayey Silt	2.3	17.5	350+	1.1	0.50	0.05	0.041	0.063
Silty Sand (Till)	0.9	21	E <sub>s</sub> = 5 MPa					

Settlement was calculated near the culvert inlet and outlet using details from Boreholes SC7-3 and SC7-1, respectively. The estimated settlement of the underlying native cohesionless soils near the inlet is 70 mm. This settlement will occur during construction. The estimated settlement of the underlying native cohesive and cohesionless materials near the outlet is 25 mm (15 mm of recompression from the clayey silt and less than 10 mm of elastic settlement in the silty sand). Given the limited thickness of the native soils, time dependent recompression is expected to occur rapidly and should be predominantly completed at the end of construction.



If the existing culvert is decommissioned by filling it with grout or removed and backfilled, it is estimated that this would induce further settlement of less than 10 mm beneath the existing culvert alignment as a result of the increased load imposed by the grout/fill.

In addition to the settlement described above, there will be self-settlement of the embankment reinstatement fill material. Adjacent to the culvert, the 6.5 m embankment constructed of compacted SSM material will undergo approximately 35 mm of self settlement with the majority of that complete during construction.

Embankments must be overbuilt to compensate for the estimated settlement.

### 10.8.3 Temporary Grade Lowering

It is estimated that grade lowering of more than 1.7 m would be required to create a working surface wide enough to replace the culvert in two stages without temporary protection systems. It is unlikely that this would leave sufficient cover over the culvert. However, a lesser temporary grade lowering could offer a benefit in reducing the height of TPS and should be considered.

### 10.8.4 Temporary Widening or Detour Embankment

A foundation investigation was not completed for a temporary detour embankment as part of the current assignment. Further assessment of the existing County Road 20 embankment should be carried out where construction staging dictates that a temporary detour embankment is needed. A temporary culvert extension may also be required in the area of the embankment widening. The drainage impacts will need to be assessed. Additional foundation field investigation may be required.

## 11 CONSTRUCTION CONSIDERATIONS

### 11.1 Temporary Excavations

All temporary excavation must be conducted in accordance with the requirements of the Occupational Health & Safety Act & Regulations (OHSA) for Construction Projects. The fill and native clayey silt and silt sand materials may be classified as Type 3 soil. Unsupported excavations made in Type 3 soils must have side slopes no steeper than 1H:1V from the base of the excavation. **Side slopes for excavations through more than one soil type must be entirely based on the highest soil type number.**

Excavation should occur in a dewatered environment (see Section 11.3). Excavations must be planned and carried out in a manner that does not impact on the stability of existing roadway. The temporary cut slopes may have to be protected from precipitation and runoff to avoid surficial instabilities. The duration of temporary open excavations and cut slopes should be minimized to



reduce the likelihood of causing instability concerns. Embankment and cut slope stability is the responsibility of the Contractor.

Excavation should be carried out in accordance OPSS.PROV 902, OPSS.PROV 421 and OPSS.PROV 422. The management and disposal of excess material shall be in accordance with OPSS.PROV 180. Excavations will extend through existing fills and into the underlying native soil deposits to bedrock.

Bedrock excavation may be required for the retaining wall installation; however, the quantities are anticipated to be limited. It is anticipated that blasting may not be permitted at this site and that an excavator equipped with a bucket would not be sufficient to remove the bedrock observed at this site. Rock removal may require a hoe ram or ripper; however, it is the contractor's responsibility to determine the preferred excavation approach.

Selection of the equipment and methodology to excavate and prepare the founding surface is the responsibility of the Contractor. Material stockpiling is a temporary construction measure, and the associated stability implications are the responsibility of the Contractor. The selection and placement of construction equipment (such as cranes) and construction of temporary construction access roads are also the Contractor's responsibility. Placement of the crane or temporary stockpiling must not destabilize the embankment.

At locations where there are space restrictions or where a slope must be retained, the excavations will need to be carried out within a protection system. Further discussion on temporary protection systems (TPS) is presented in Section 11.2.

## **11.2 Temporary Protection Systems**

Temporary Protection Systems may be required during various stages of construction and must be implemented in accordance with OPSS.PROV 539. Performance Level 2 (maximum 25 mm horizontal deflection) is considered appropriate where the protection supports an existing roadway. More stringent performance levels may be required if the protection system is intended to support existing structures or utilities.

It will be difficult to drive sheet piles at this site due to the presence of shallow sloping bedrock; suggested wording for a Contract Provision is provided in Appendix H. Drilled in soldier piles with lagging is considered suitable for this site; however, the selection and design of roadway protection is the responsibility of the Contractor. All protection systems should be designed by a licensed Professional Engineer experienced in such designs and retained by the Contractor. The design of the roadway protection system must incorporate traffic loading and surcharge loading due to construction equipment and operations. An anchoring and/or internal bracing system may need to be incorporated into the temporary protection design to resist lateral earth pressure loadings. The pressure distribution acting on the shoring system is a function of the construction sequence and the relative flexibility of the wall, and these factors must be considered when



designing the shoring system. The design of the TPS support of the fill above the existing culvert may need to incorporate additional structural elements.

The lateral earth pressure coefficients and undrained strengths for the existing soils are given in Table 11-1 for a vertical wall and a horizontal backslope. Unit weights provided herein are to be adjusted for applications below the groundwater level. Unbalanced hydrostatic pressures should be considered in the design of the protection systems.

**Table 11-1: Static Earth Pressure Coefficients for Existing Soils**

Material	Unit Weight <sup>(*)</sup> (kN/m <sup>3</sup> )	K <sub>A</sub>	K <sub>P</sub>	K <sub>0</sub>	Su (kPa)
Existing Silty Sandy Clay Fill	17.5	-	-	-	25
Existing Silty Sand Fill	20	0.33	3.0	0.50	-
Native Clayey Silt	17.5	-	-	-	75
Native Silty Sand	19	0.33	3.0	0.50	-

Note: (\*) to be adjusted when below water level

It is recommended that the protection systems within 3 m from the edges of the new culvert should be left in place and cut off in accordance with OPSS.PROV 539.

### 11.3 Surface and Groundwater Control

Subgrade preparation and placement and compaction of granular bedding/pads or mass concrete for culvert, footing and retaining wall construction must be carried out in the dry. Furthermore, surface runoff will tend to seep into and accumulate into the excavations.

The Contractor must control groundwater, perched groundwater and surface water flow at the site with a flow passage system and a dewatering system to permit construction in a dry and stable excavation.

The temporary flow diversion pipe should be placed outside the construction area. If the replacement culvert is installed on a new alignment the existing culvert could be used for flow diversion until the new culvert is completed. The design of flow passage systems is the responsibility of the Contractor. Given the site conditions and anticipated works, the Designer Fill-In (Note 2) in SP 517F01 Table 1 for flow passage systems should be "No; the design Engineer and design-checking Engineer do not need a minimum of 5 years of experience in designing similar flow passage systems.

The design of dewatering systems is the responsibility of the Contractor. The Contract Documents must alert the Contractor to this responsibility and to design the system in accordance with SP





517F01 which amends OPSS.PROV 517. The contractor's design should include an assessment of any adverse effects the dewatering method, construction layout and staging may have on adjacent structures, utilities and facilities. Given the site conditions and anticipated works, the Designer Fill-In (Note 2) in SP 517F01 Table 1 should be "No" for dewatering systems; the design Engineer and design-checking Engineer do not need a minimum of 5 years of experience in designing similar dewatering systems. A preconstruction survey is not recommended; thus, Designer Fill-In Note 4 in this SP should be "N/A". Based on the groundwater elevation at the time of the investigation, it is anticipated that the site will require dewatering to lower the groundwater to below the final excavation or footing level; Note 5 of SP517F01 Table 1 should be a minimum of 0.5 m below the underside of the planned excavation base prior to each stage of excavation.

The water level will fluctuate and the minimum design groundwater elevation for the site at the time of the excavation should be no lower than the highwater level in the creek generated by the return period flow estimates defined in SP 517F01.

The dewatering plan should be coordinated with the TPS design. The dewatering system will be required to remain operational and effective until the temporary excavations are backfilled and then should be decommissioned and removed.

A sheet pile cofferdam enclosure might be difficult to install at this site (discussion in Section 11.2). Alternative dewatering methods such as a sandbag cofferdam with sump pumps to extract water from the excavation are likely sufficient. More than one pump may be required.

Further assessment of dewatering requirements and the need for registration on the Environmental Activity and Sector Registry (EASR) or a Permit to take Water (PTTW) should be carried out by specialists experienced in this field.

Please refer to Hydrogeological Investigation and Design Report for additional discussion on dewatering with respect to this assignment.

#### **11.4 Erosion and Scour Control**

The Contractor should provide silt fences and erosion control blankets as per OPSS.PROV 805 and OPSD 219.110 throughout the duration of construction to prevent transport of silt/sediment.

Particle size analysis on samples of the existing embankment fill and native materials indicate that the soils have a low to medium potential for soil erodibility (Wischmeier Nomograph factor, K).

Slope protection and drainage measures will be required to ensure the long-term surficial stability of the embankment slopes. A vegetation cover should be established on exposed earth surfaces to protect against surficial erosion in general accordance with OPSS.PROV 803 and OPSS.PROV 804. Slope vegetation should be established as soon as possible after completion



of construction in order to limit surficial erosion and water should be prevented from running down an unprotected slope.

Scour and erosion protection must be provided for the culvert inlet and outlet areas. Effective scour and erosion protection should be provided along the waterline and ditches. Design of the erosion protection measures must consider hydrologic and hydraulic factors and shall be carried out by specialists experienced in this field. Typically, rock protection should be provided over all earth surfaces subjected to flowing water in accordance with OPSS.PROV 511. Treatment at the outlet should be in accordance with OPSD 810.010.

Liaison between the Foundations Consultant, Structural Engineer and Hydraulic/Drainage Engineer will be required in design to ensure that scour protection, if required, is adequately addressed.

## 12 DESIGN AND CONSTRUCTION CONCERNS

The preliminary recommendations presented herein must be reassessed once the type, location, elevation and orientation of the works are established.

The seismic hazard data considered for the preliminary design recommendations provided in this report were obtained from the fifth-generation seismic model developed by the Geological Survey of Canada (GSC). Additional seismic analyses will be required to reflect the reference seismic hazard available at the time of detailed design.

**The DB Contractor must review the existing factual information and determine the extent of additional field investigations and laboratory testing required to support the foundation design of the proposed works. It is noted that preliminary GA drawings are not available at the time of writing. The preliminary recommendations provided herein will need to be re-evaluated once the culvert invert foundation elevations are confirmed.**

Of particular note at this site is the identification of potentially liquefiable soils at the culvert outlet. It has been assumed in the preliminary design that this material will be removed from beneath the culvert structure as part of the subgrade preparation; additional investigation and design is required.

A right-of-way constraint on the north side of County Road 20 has resulted in the inclusion of a 40 m long retaining wall on that side of the reinstated embankment. Details on the retaining wall are not available. It has been assumed that the retaining wall will be founded on bedrock, however, the bedrock surface is variable and may be encountered at varying depth along the length of the proposed wall. Additional boreholes should be drilled along the length of the retaining wall during a subsequent design stage to confirm bedrock elevations.

The planned construction methodology includes open cut excavations for the installation of foundation elements of a new culvert and inlet retaining wall. Potential construction concerns may include, but are not necessarily limited to:



- Construction will extend below the water level in the creek. An adequate and effective surface water management and dewatering plan must be implemented to construct the culvert and retaining wall foundations in the dry.
- The native soil which could be exposed beneath the culvert bedding is sensitive and readily disturbed. A suggested Notice to Contractor is provided in Appendix H.
- The Contractor's selection of construction equipment and methodology must include assessment of the capability of the existing soils to support the proposed construction equipment and supplies.
- The bedrock elevation is variable across the site. Sloping bedrock will be encountered. A Notice to Contractor has been included in Appendix H.

The successful performance of the structure installations will depend largely upon good workmanship and quality control during construction. Observation of the excavation and backfilling operations will be required as per OPSS.PROV 902 during construction to confirm that the foundation recommendations are correctly implemented, and material specifications are met.



### 13 CLOSURE

Engineering analysis and preparation of this report was carried out by A. de Oliveira, P.Eng. The report was reviewed by Dr. F. Griffiths, P.Eng., and Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundation Projects.

Thurber Engineering Ltd.  
Report Prepared By:



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Geotechnical Engineer



Dr. Fred Griffiths, P.Eng.  
Principal, Senior Geotechnical Engineer

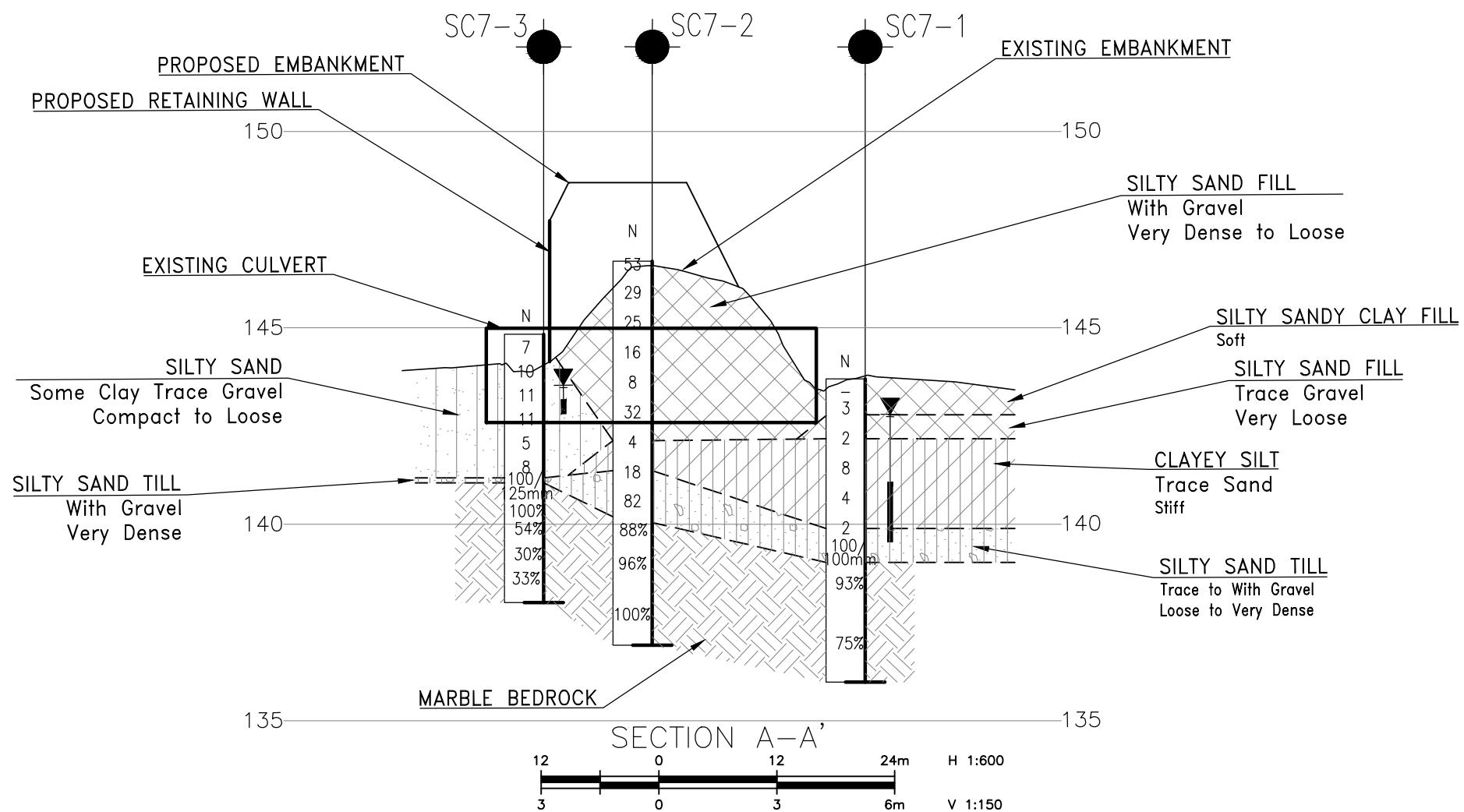
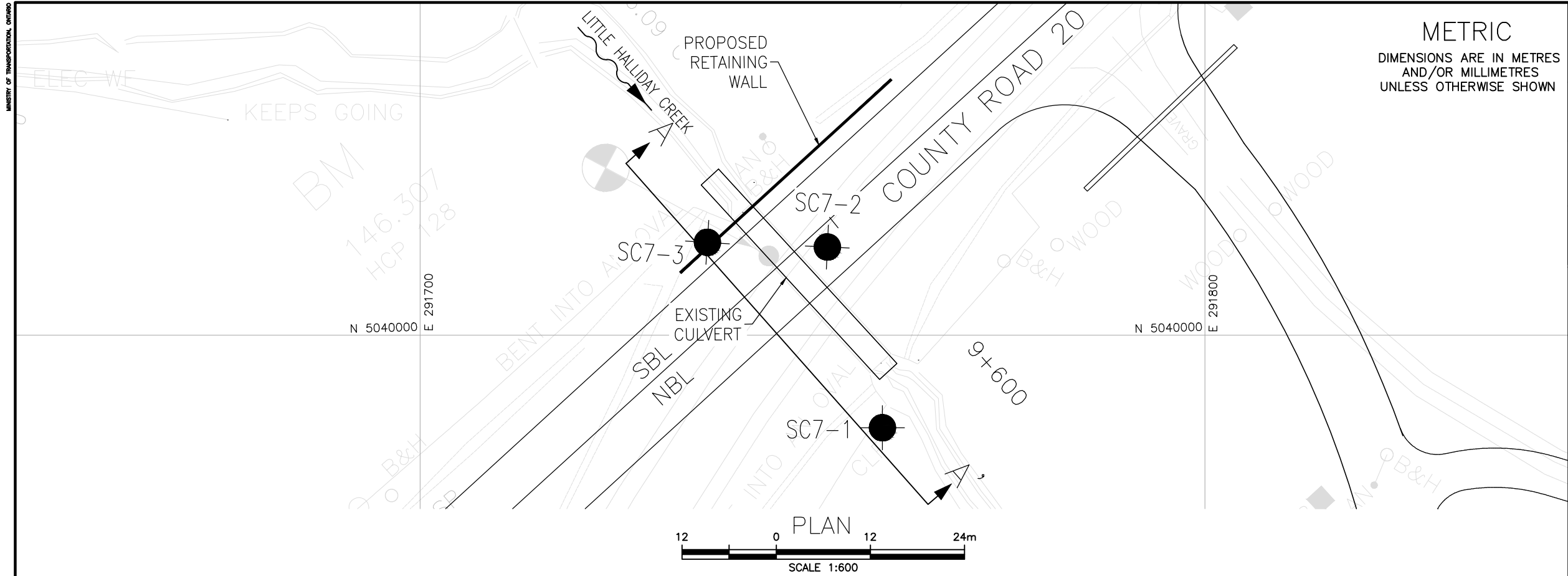


Dr. P.K. Chatterji, P.Eng.  
Designated Principal Contact,  
Principal, Senior Geotechnical Engineer



## **Appendix A.**

### **Borehole Location Plan and Stratigraphic Drawings**



METRIC  
DIMENSIONS ARE IN METRES  
AND/OR MILLIMETRES  
UNLESS OTHERWISE SHOWN

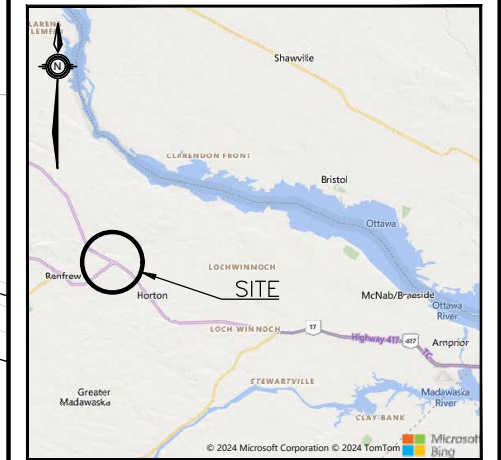
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GWP No 4068-09-00	



HIGHWAY 17 TWINNING COUNTY ROAD 20, STATION 9+600 CULVERT 7AN BOREHOLE LOCATION PLAN AND SOIL STRATA	
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


SHEET  
1

**Ontario** 


**THURBER**

## KEYPLAN

### LEGEND

	Borehole
	Historic Borehole
N	Blows /0.3m (Std Pen Test, 475J/blow)
CONE	Blows /0.3m (60° Cone, 475J/blow)
PH	Pressure, Hydraulic
	Water Level Upon Completion of Drilling
	Water Level in Monitoring Well/Piezometer
	Monitoring Well/Piezometer Screen
90%	Rock Quality Designation (RQD)
A/R	Auger Refusal

NO	ELEVATION	NORTHING	EASTING
SC7-1	143.7	5 039 988.2	291 758.9
SC7-2	146.7	5 040 011.2	291 751.9
SC7-3	144.4	5 040 011.9	291 737.2

-NOTES-

- 1) The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.
- 2) This drawing is for subsurface information only. Surface details and features are for conceptual illustration.
- 3) Coordinate system is MTM NAD 83 Zone 9.

**GEOCRES No. 31F07-002**

REVISIONS									
	DATE	BY	DESCRIPTION					DATE	NOV 2024
DESIGN	AO	CHK -	CODE	LOAD					
DRAWN	RH	CHK FG	SITE	STRUCT	DWG	1			

LOTDATE: 12/17/2024 9:00 AM



## **Appendix B.**

### **Record of Borehole Sheets**



## SYMBOLS, ABBREVIATIONS AND TERMS USED ON TEST HOLE RECORDS

### TERMINOLOGY DESCRIBING COMMON SOIL GENESIS

Topsoil	mixture of soil and humus capable of supporting vegetative growth
Peat	mixture of fragments of decayed organic matter
Till	unstratified glacial deposit which may include particles ranging in sizes from clay to boulder
Fill	material below the surface identified as placed by humans (excluding buried services)

### TERMINOLOGY DESCRIBING SOIL STRUCTURE:

Desiccated	having visible signs of weathering by oxidization of clay materials, shrinkage cracks, etc.
Fissured	having cracks, and hence a blocky structure
Varved	composed of alternating layers of silt and clay
Stratified	composed of alternating successions of different soil types, e.g. silt and sand
Layer	> 75 mm in thickness
Seam	2 mm to 75 mm in thickness
Parting	< 2 mm in thickness

### RECOVERY:

For soil samples, the recovery is recorded as the length of the soil sample recovered.

### N-VALUE:

Numbers in this column are the field results of the Standard Penetration Test: the number of blows of a 63.5 kg hammer falling 0.76 m, required to drive a 50 mm O.D. split spoon sampler 0.3 m into undisturbed soil. For samples where insufficient penetration was achieved and N-value cannot be presented, the number of blows are reported over the sampler penetration in millimetres (e.g. 50/75).

### DYNAMIC CONE PENETRATION TEST (DCPT):

Dynamic cone penetration tests are performed using a standard 60 degree apex cone connected to an "A" size drill rods with the same standard fall height and weight as the Standard Penetration Test. The DCPT value is the number of blows of the hammer required to drive the cone 0.3 m into the soil. The DCPT is used as a probe to assess soil variability.





### STRATA PLOT:

Strata plots symbolize the soil and bedrock description. They are combinations of the following basic symbols. The dimensions within the strata symbols are not indicative of the particle size, layer thickness, etc.



Boulders  
Cobbles  
Gravel      Sand      Silt      Clay      Organics      Asphalt      Concrete      Fill      Bedrock

### TEXTURING CLASSIFICATION OF SOILS

Classification	Particle Size
Boulders	Greater than 200 mm
Cobbles	75 – 200 mm
Gravel	4.75 – 75 mm
Sand	0.075 – 4.75 mm
Silt	0.002 – 0.075 mm
Clay	Less than 0.002 mm

### TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

Descriptive Term	Undrained Shear Strength (kPa)
Very Soft	12 or less
Soft	12 – 25
Firm	25 – 50
Stiff	50 – 100
Very Stiff	100 – 200
Hard	Greater than 200

NOTE: Clay sensitivity is defined as the ratio of the undisturbed strength over the remolded strength.

### SAMPLE TYPES

SS	Split spoon samples
ST	Shelby tube or thin wall tube
DP	Direct push sample
PS	Piston sample
BS	Bulk sample
WS	Wash sample
HQ, NQ, BQ etc.	Rock core sample obtained with the use of standard size diamond coring equipment

### TERMS DESCRIBING CONSISTENCY (COHESIONLESS SOILS ONLY)

Descriptive Term	SPT “N” Value
Very Loose	Less than 4
Loose	4 – 10
Compact	10 – 30
Dense	30 – 50
Very Dense	Greater than 50

### MODIFIED UNIFIED SOIL CLASSIFICATION

Major Divisions		Group Symbol	Typical Description
COARSE GRAINED SOIL	GRAVEL AND GRAVELLY SOILS	GW	Well-graded gravels or gravel-sand mixtures, little or no fines.
		GP	Poorly-graded gravels or gravel-sand mixtures, little or no fines.
		GM	Silty gravels, gravel-sand-silt mixtures.
		GC	Clayey gravels, gravel-sand-clay mixtures.
	SAND AND SANDY SOILS	SW	Well-graded sands or gravelly sands, little or no fines.
		SP	Poorly-graded sands or gravelly sands, little or no fines.
		SM	Silty sands, sand-silt mixtures.
		SC	Clayey sands, sand-clay mixtures.
FINE GRAINED SOILS	SILT AND CLAY SOILS $W_L < 35\%$	ML	Inorganic silts, very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
		OL	Organic silts and organic silty-clays of low plasticity.
	SILT AND CLAY SOILS $35\% < W_L < 50\%$	MI	Inorganic compressible fine sandy silt with clay of medium plasticity, clayey silts.
		CI	Inorganic clays of medium plasticity, silty clays.
		OI	Organic silty clays of medium plasticity.
	SILT AND CLAY SOILS $W_L > 50\%$	MH	Inorganic silts, micaceous or diatomaceous fine sandy of silty soils, elastic silts.
		CH	Inorganic clays of high plasticity, fat clays.
		OH	Organic clays of high plasticity, organic silts.
HIGHLY ORGANIC SOILS		Pt	Peat and other organic soils.

Note -  $W_L$  = Liquid Limit



## EXPLANATION OF ROCK LOGGING TERMS

### ROCK WEATHERING CLASSIFICATION

Fresh (FR)	No visible signs of weathering.
Fresh Jointed (FJ)	Weathering limited to surface of major discontinuities.
Slightly Weathered (SW)	Penetrative weathering developed on open discontinuity surfaces, but only slight weathering of rock materials.
Moderately Weathered (MW)	Weathering extends throughout the rock mass, but the rock material is not friable.
Highly Weathered (HW)	Weathering extends throughout the rock mass and the rock is partly friable.
Completely Weathered (CW)	Rock is wholly decomposed and in a friable condition, but the rock texture and structures are preserved.

### TERMS

Total Core Recovery: (TCR)	Core recovered as a percentage of total core run length.
Solid Core Recovery: (SCR)	Percent ratio of solid core of full cylindrical shape recovered. Expressed with respect to the total length of core run.
Rock Quality Designation: (RQD)	Total length of sound core recovered in pieces 0.1 m in length or larger, as a percentage of total core length
Unconfined Compressive Strength: (UCS)	Axial stress required to break the specimen.
Fracture Index: (FI)	Frequency of natural fractures per 0.3 m of core run.

### DISCONTINUITY SPACING

Bedding	Bedding Plane Spacing
Very thickly bedded	Greater than 2 m
Thickly bedded	0.6 to 2 m
Medium bedded	0.2 to 0.6 m
Thinly bedded	60 mm to 0.2 m
Very thinly bedded	20 to 60 mm
Laminated	6 to 20 mm
Thinly laminated	Less than 6 mm

### STRENGTH CLASSIFICATION

Rock Strength	Approximate Uniaxial Compressive Strength (MPa)
Extremely Strong	Greater than 250
Very Strong	100 – 250
Strong	50 – 100
Medium Strong	25 – 50
Weak	5 – 25
Very Weak	1 – 5
Extremely Weak	0.25 – 1

RECORD OF BOREHOLE No SC7-1

1 OF 1

METRIC

WP# 4068-09-00 LOCATION Lat: 45.499694°, Long: -76.666871°  
Culvert 7AN; Horton Township; MTM z9: N 5 118 066.2 E 300 655.1 ORIGINATED BY RH  
HWY 17 BOREHOLE TYPE CME 55 Trackmount / HSA / NW Casing / NQ Coring COMPILED BY AO  
DATUM Geodetic DATE 2024.02.26 - 2024.02.26 CHECKED BY CM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				
								○ UNCONFINED + FIELD VANE					w <sub>p</sub> w w <sub>L</sub>				
								● QUICK TRIAXIAL × LAB VANE									
143.7	Ground Surface						20	40	60	80	100	20	40	60		GR SA SI CL	
0.0	SILTY SANDY CLAY soft brown FILL - organics noted in the upper 25 mm of the layer		1	GS	-								○			0 8 56 36	
142.8													○				
0.9	SILTY SAND, trace gravel very loose grey FILL		2	SS	3								○				
142.2													○				
1.5	CLAYEY SILT (CL), trace sand stiff grey		3	SS	2								○				
													○				
			4	SS	8								○				
			5	SS	4								○				
139.9																	
3.8	SILTY SAND, trace to with gravel very loose to very dense grey GLACIAL TILL		6	SS	2								○			7 65 22 6	
139.0			7	SS	100/ 100mm								○				
4.7	MARBLE BEDROCK fresh jointed grey fine to medium grained strong		1	RUN	-											RUN #1 TCR=100% SCR=93% RQD=93% UCS=78MPa	
			2	RUN	-											RUN #2 TCR=100% SCR=75% RQD=75%	
136.0																	
7.7	End of Borehole																
	Monitoring Well installed: Schedule 40 PVC standpipe with 50-mm diameter and 1.5-m slotted screen. Stick-up cover installed at ground surface.																
	Water Level Readings: DATE DEPTH (m) ELEV. (m)																
	2024/03/07 0.4 143.3																
	2024/03/22 0.5 143.2																
	2024/04/10 0.3 143.4																
	2024/04/24 0.6 143.1																
	2024/06/04 0.9 142.8																
	2024/06/25 0.5 143.2																
	2024/08/30 0.9 142.8																

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to  
Sensitivity

20  
15  
10  
(%) STRAIN AT FAILURE

DOUBLE LINE 24726 CULVERT 7AN.GPJ 2012TEMPLATE(MTO).GDT 12-16-24

RECORD OF BOREHOLE No SC7-2

1 OF 1

METRIC

WP# 4068-09-00 LOCATION Lat: 45.499901°, Long: -76.666962°  
Culvert 7AN; Horton Township; MTM z9: N 5 118 072.5 E 300 646.5 ORIGINATED BY DAP  
HWY 17 BOREHOLE TYPE CME 55 Truckmount / HSA / NW Casing / NQ Coring COMPILED BY AO  
DATUM Geodetic DATE 2024.03.18 - 2024.03.18 CHECKED BY CM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)						
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									WATER CONTENT (%)			GR	SA	SI	CL
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL	× LAB VANE	20					40	60	80				
146.7	Ground Surface																						
0.0	ASPHALT (50 mm)																						
0.8	SILTY SAND with gravel loose to very dense grey FILL		1	SS	53								○										
			2	SS	29								○										
			3	SS	25								○					22 62 16 (SI+CL)					
			4	SS	16								○										
			5	SS	8								○										
			6	SS	32								○					43 45 12 (SI+CL)					
142.1	Sandy CLAYEY SILT (CL-ML) stiff grey		7	SS	4								H ○					1 39 38 22					
141.4	SILTY SAND with gravel compact to very dense grey GLACIAL TILL		8	SS	18								○										
			9	SS	82								○					23 59 18 (SI+CL)					
140.0	MARBLE BEDROCK fresh jointed grey fine to medium grained very strong		1	RUN	-													RUN #1 TCR=100% SCR=66% RQD=88%					
			2	RUN	-													RUN #2 TCR=100% SCR=90% RQD=96% UCS=148MPa					
			3	RUN	-													RUN #3 TCR=100% SCR=100% RQD=100%					
136.9																							
9.8	End of Borehole																						

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to  
Sensitivity

20  
15  
10

(%) STRAIN AT FAILURE

DOUBLE LINE 24726 CULVERT 7AN.GPJ 2012TEMPLATE(MTO).GDT 12-16-24

# RECORD OF BOREHOLE No SC7-3

1 OF 1

METRIC

WP# 4068-09-00 LOCATION Lat: 45.499907°, Long: -76.667149°  
Culvert 7AN; Horton Township; MTM z9: N 5 118 064.1 E 300 651.8 ORIGINATED BY IK  
HWY 17 BOREHOLE TYPE Portable / HW Casing / NQ Coring COMPILED BY AO  
DATUM Geodetic DATE 2024.03.07 - 2024.03.07 CHECKED BY CM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>P</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT  γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
144.4	Ground Surface							20	40	60	80	100					GR SA SI CL
0.0	SILTY SAND with organics and gravel compact dark brown FILL		1	SS	7		144										20 52 28 (SI+CL)
143.5	SILTY SAND with gravel loose to compact brown		2	SS	10												
0.9	SILTY SAND, some clay trace gravel compact to loose grey		3	SS	11		143										
			4	SS	11		142										3 60 27 10 Non-plastic
			5	SS	5												
			6	SS	8		141										
140.7			7	SS	100/											FI	RUN #1 TCR=100% SCR=100% RQD=100%
140.6	SILTY SAND with gravel very dense grey GLACIAL TILL		1	RUN	125mm											7	
3.8	MARBLE BEDROCK slightly weathered to fresh jointed grey fine to medium grained strong		2	RUN	-		140									8	RUN #2 TCR=100% SCR=63% RQD=54%
			3	RUN	-		139									5	RUN #3 TCR=100% SCR=53% RQD=30%
			4	RUN	-		138									9	RUN #4 TCR=100% SCR=47% RQD=33% UCS=75MPa
137.6	End of Borehole															8	
6.8	Monitoring Well installed: Schedule 40 PVC standpipe with 50-mm diameter and 1.5-m slotted screen. Stick-up cover installed at ground surface.  Water Level Readings: DATE DEPTH (m) ELEV. (m) 2024/03/08 0.5 143.9 2024/03/22 1.0 143.4 2024/04/10 0.6 143.8 2024/04/24 0.8 143.6 2024/06/04 1.3 143.1 2024/06/25 0.8 143.6 2024/07/10 1.1 143.3 2024/08/30 1.2 143.2															7	
																3	
																3	

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity 20 15 10 5 10 (%) STRAIN AT FAILURE



## **Appendix C.**

### **Laboratory Testing**

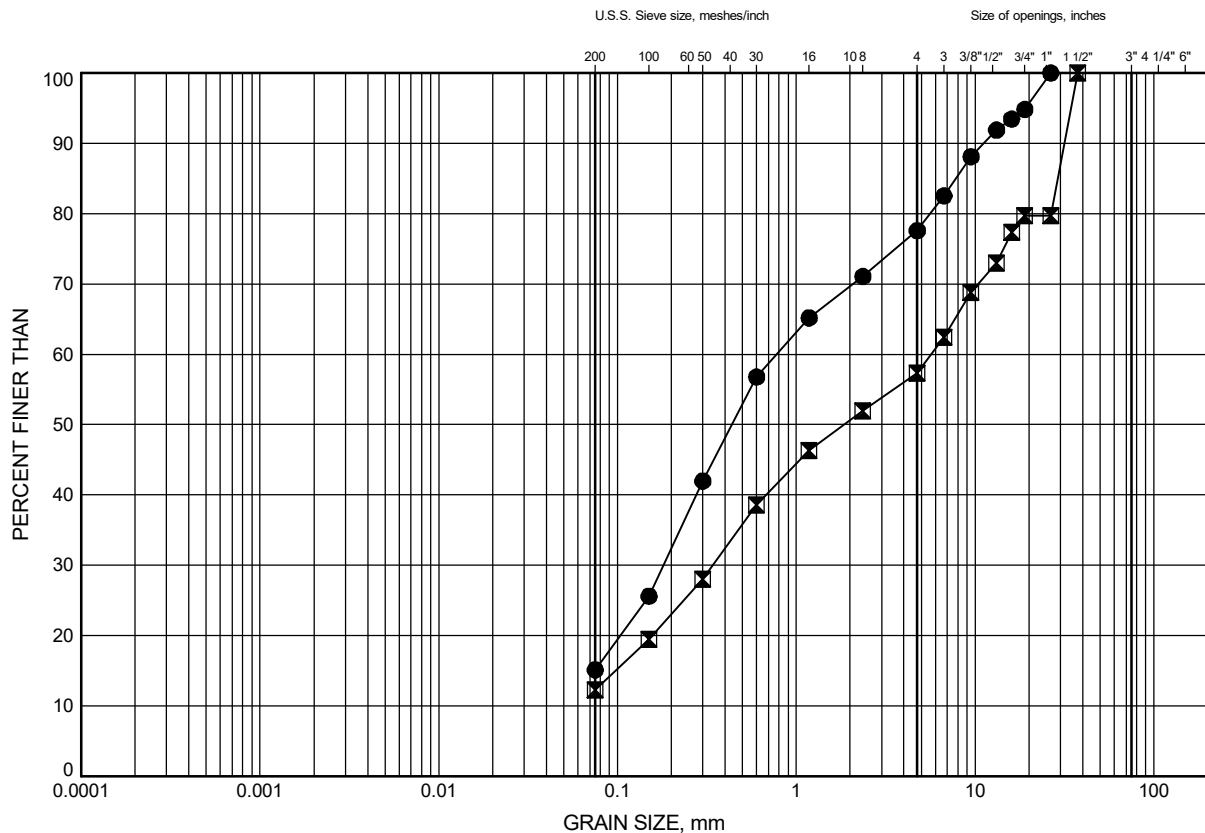


**Appendix C.1**  
**Particle Size Analysis Figures**  
**Atterberg Limit Test Results**  
**Unconfined Compressive Strength Testing Results**  
**Rock Core Photos**



# GRAIN SIZE DISTRIBUTION

FILL: Silty Sand



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

## LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	SC7-2	1.8	144.9
⊠	SC7-2	4.1	142.6

Date August 2024

WP# 4068-09-00

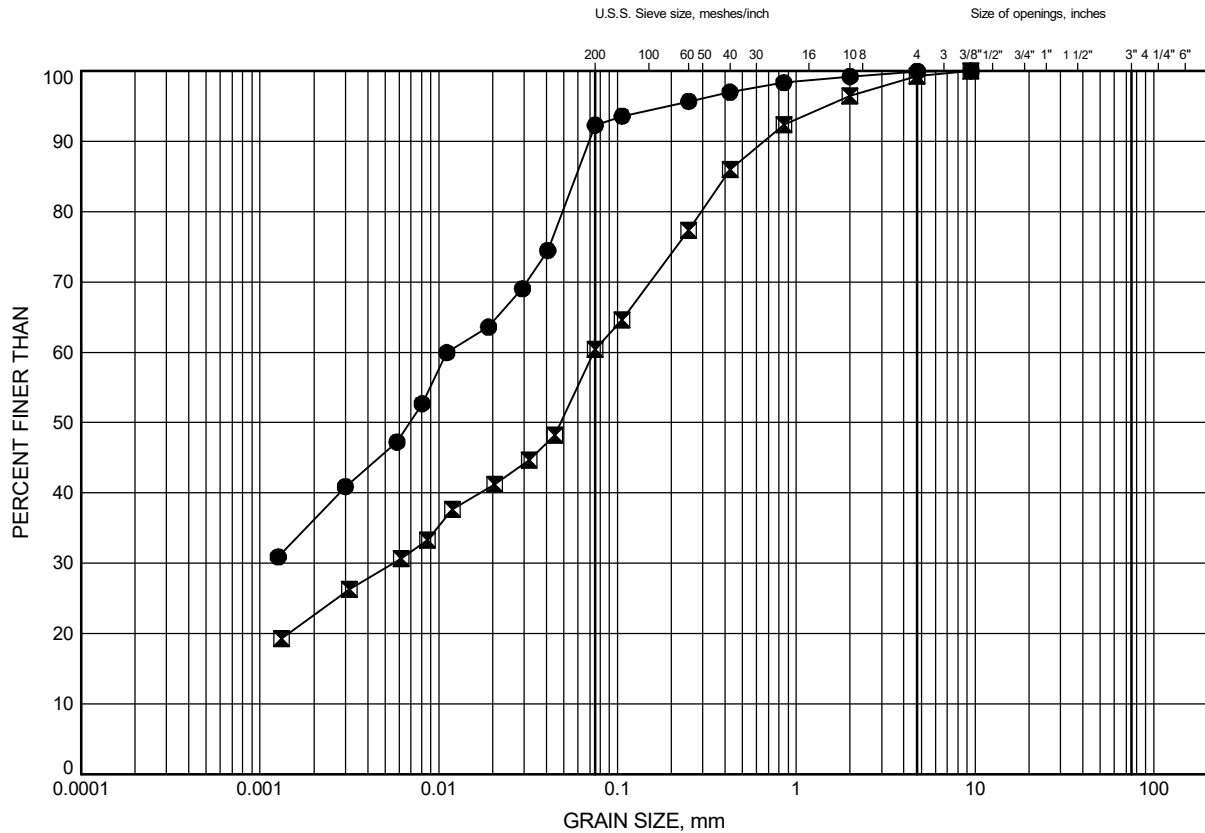


Prep'd RH

Chkd. AO

# GRAIN SIZE DISTRIBUTION

Clayey Silt (CL to CL-ML)



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

## LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	SC7-1	2.0	141.7
⊠	SC7-2	4.9	141.8

Date August 2024

WP# 4068-09-00

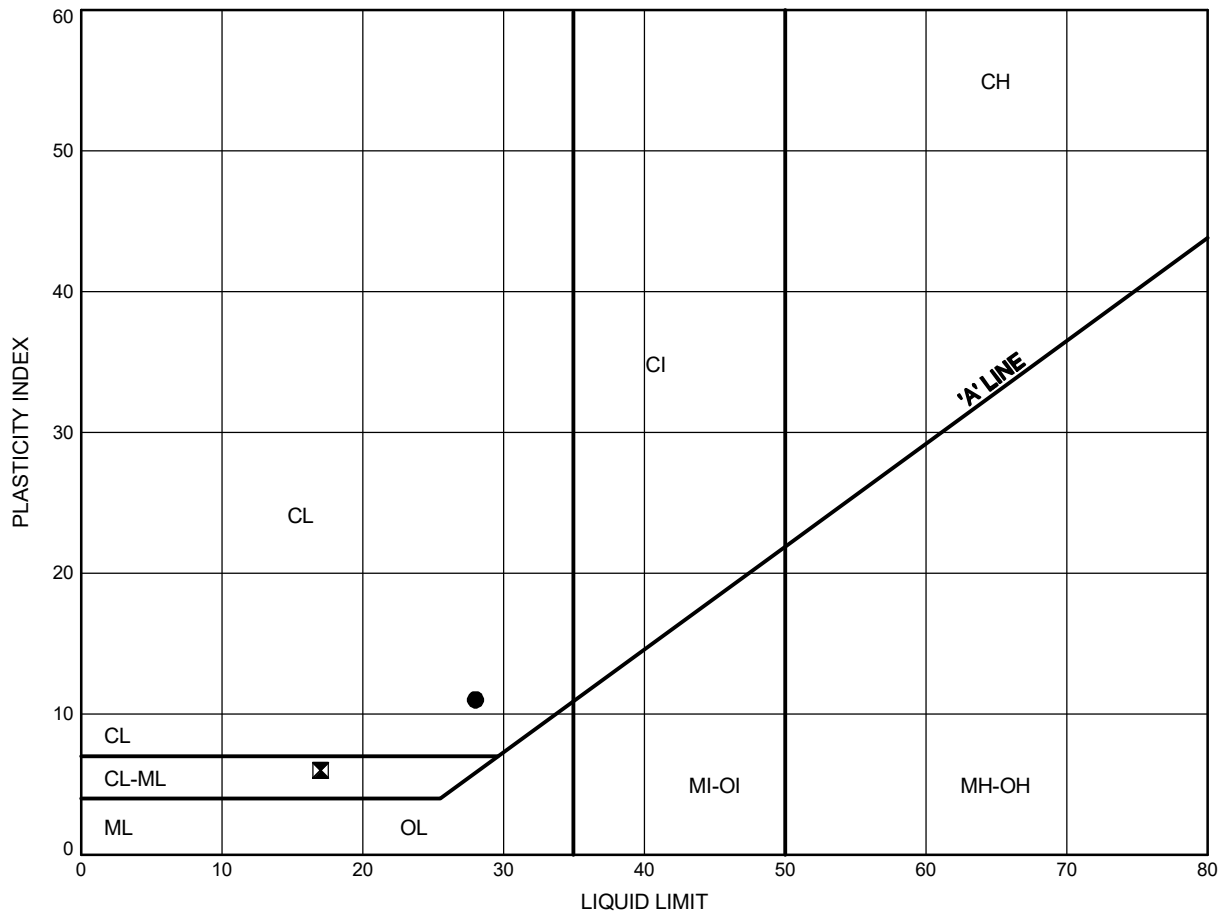


Prep'd RH

Chkd. AO

# ATTERBERG LIMITS TEST RESULTS

Clayey Silt (CL to CL-ML)



## LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	SC7-1	2.0	141.7
⊠	SC7-2	4.9	141.8

Date August 2024

WP# 4068-09-00

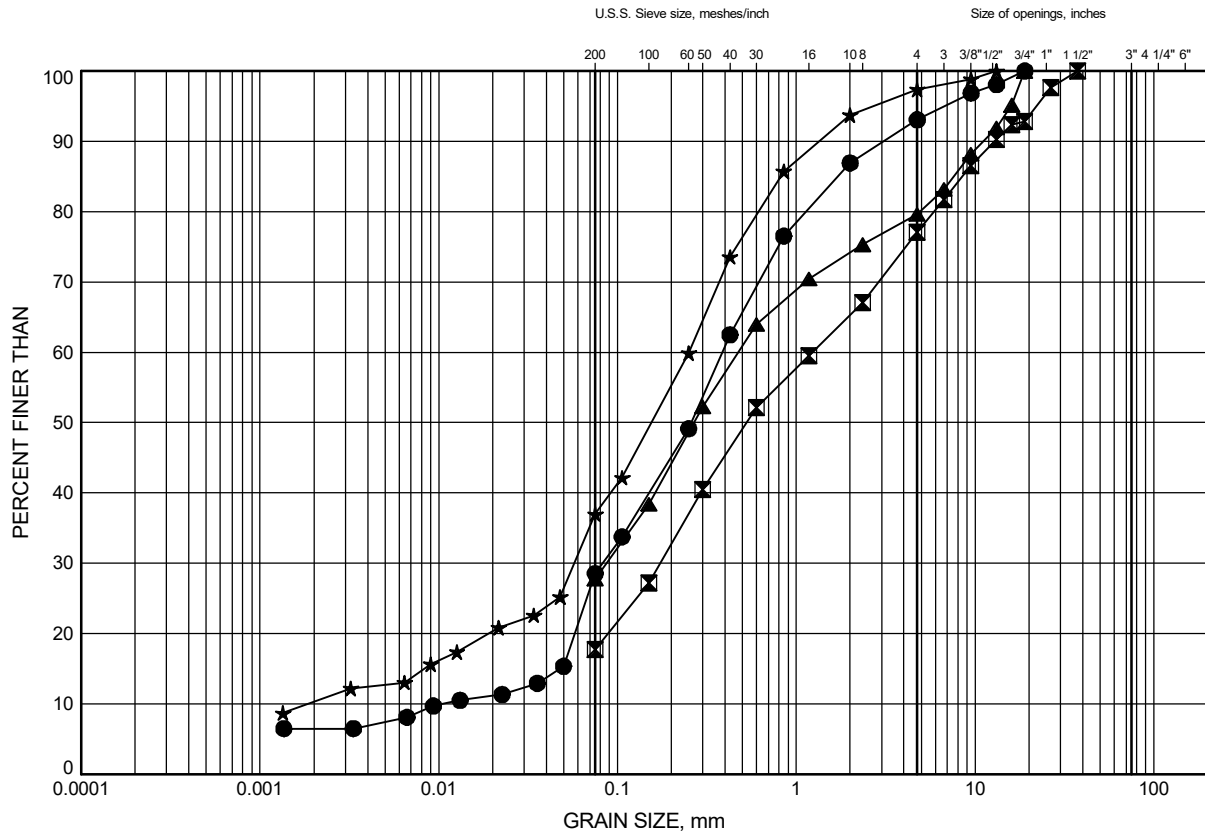


Prep'd RH

Chkd. AO

# GRAIN SIZE DISTRIBUTION

## Silty Sand



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

## LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	SC7-1	4.1	139.6
⊠	SC7-2	6.4	140.3
▲	SC7-3	0.4	144.0
★	SC7-3	2.1	142.3

Date August 2024

WP# 4068-09-00



Prep'd RH

Chkd. AO

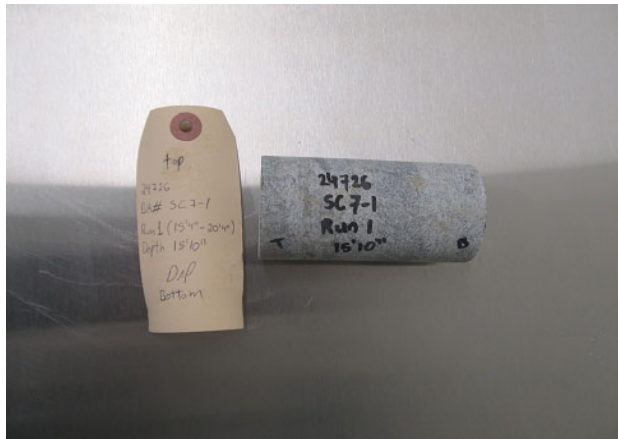
## UNCONFINED COMPRESSION TEST REPORT

### ASTM D7012-14

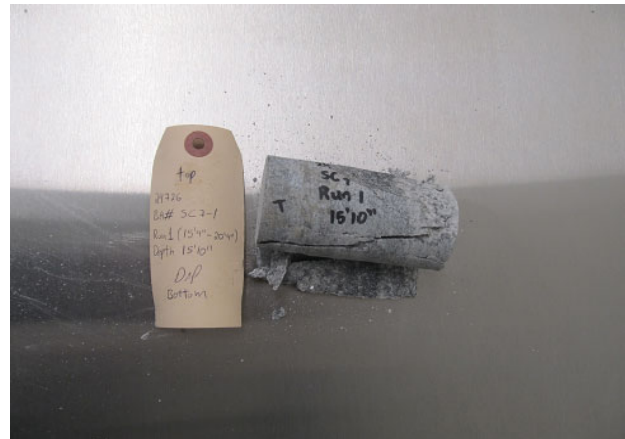
CLIENT:	Thurber Engineering (Ottawa)	FILE NUMBER:	24726
PROJECT NAME:	Highway 17 Twinning - Renfrew	REPORT DATE:	10-Jul-24
BOREHOLE No.:	SC7-1	TEST DATE:	9-May-24
SAMPLE No.:	Run 1		
SAMPLE DEPTH:	4.83 m		
DESCRIPTION:	Marble		

Avg. Height (cm):	9.5	Weight (g):	448.0
Avg. Diameter (cm):	4.7	Wet Density (kg/m <sup>3</sup> ):	2,718
H. to Dia. Ratio**:	2:1	Dry Density (kg/m <sup>3</sup> ):	2,718
Cross Sectional Area (cm <sup>2</sup> ):	17.35	Moisture Content* (%):	N/A
Sample Volume (cm <sup>3</sup> ):	164.82		

ORIGINAL SPECIMEN



FRACTURED SPECIMEN



AVG. RATE OF STRAIN TO FAILURE:	0.250 MPa/s
MAXIMUM COMPRESSIVE LOAD:	135.5 kN
UNCONFINED COMPRESSIVE STRENGTH:	78.1 MPa

Note: \* The moisture content was obtained before the test.  
 \*\* Dimensions of Specimen conform to ASTM D 4543-04.

TEST DONE BY: GF  
 REVIEWED BY: WM

UCS SC7-1 Run 1

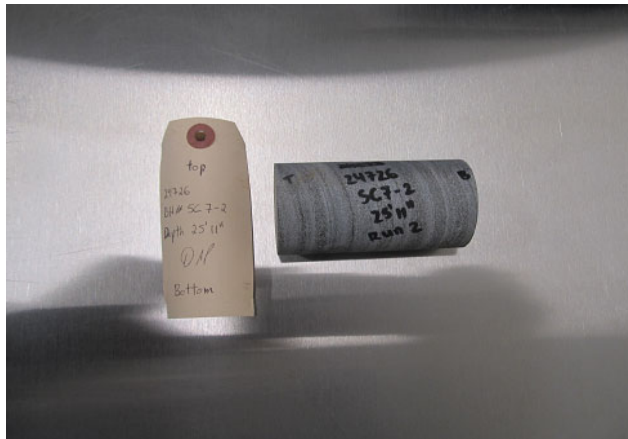
## UNCONFINED COMPRESSION TEST REPORT

### ASTM D7012-14

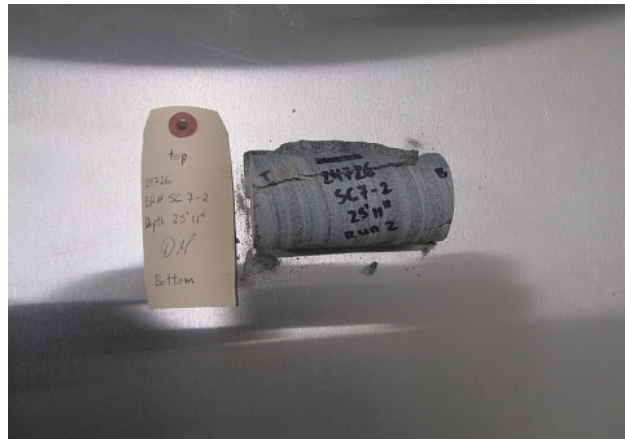
CLIENT:	Thurber Engineering (Ottawa)	FILE NUMBER:	24726
PROJECT NAME:	Highway 17 Twinning - Renfrew	REPORT DATE:	10-Jul-24
BOREHOLE No.:	SC7-2	TEST DATE:	9-May-24
SAMPLE No.:	Run 2		
SAMPLE DEPTH:	7.90 m		
DESCRIPTION:	Marble		

Avg. Height (cm):	9.5	Weight (g):	453.7
Avg. Diameter (cm):	4.7	Wet Density (kg/m <sup>3</sup> ):	2,753
H. to Dia. Ratio**:	2:1	Dry Density (kg/m <sup>3</sup> ):	2,753
Cross Sectional Area (cm <sup>2</sup> ):	17.35	Moisture Content* (%):	N/A
Sample Volume (cm <sup>3</sup> ):	164.82		

ORIGINAL SPECIMEN



FRACTURED SPECIMEN



AVG. RATE OF STRAIN TO FAILURE:	0.250 MPa/s
MAXIMUM COMPRESSIVE LOAD:	257.3 kN
UNCONFINED COMPRESSIVE STRENGTH:	148.3 MPa

Note: \* The moisture content was obtained before the test.  
 \*\* Dimensions of Specimen conform to ASTM D 4543-04.

TEST DONE BY: GF  
 REVIEWED BY: WM

UCS SC7-2 Run 2

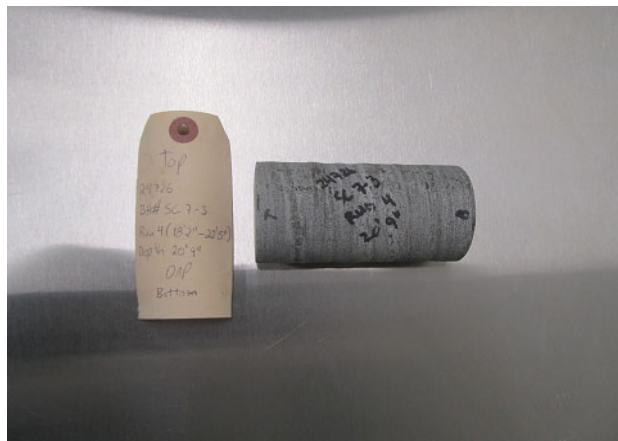
## UNCONFINED COMPRESSION TEST REPORT

### ASTM D7012-14

CLIENT:	Thurber Engineering (Ottawa)	FILE NUMBER:	24726
PROJECT NAME:	Highway 17 Twinning - Renfrew	REPORT DATE:	10-Jul-24
BOREHOLE No.:	SC7-3	TEST DATE:	9-May-24
SAMPLE No.:	Run 4		
SAMPLE DEPTH:	6.32 m		
DESCRIPTION:	Marble		

Avg. Height (cm):	10.0	Weight (g):	521.6
Avg. Diameter (cm):	5.0	Wet Density (kg/m <sup>3</sup> ):	2,656
H. to Dia. Ratio**:	2:1	Dry Density (kg/m <sup>3</sup> ):	2,656
Cross Sectional Area (cm <sup>2</sup> ):	19.63	Moisture Content* (%):	N/A
Sample Volume (cm <sup>3</sup> ):	196.35		

ORIGINAL SPECIMEN



FRACTURED SPECIMEN



AVG. RATE OF STRAIN TO FAILURE:	0.250 MPa/s
MAXIMUM COMPRESSIVE LOAD:	147.6 kN
UNCONFINED COMPRESSIVE STRENGTH:	75.2 MPa

Note: \* The moisture content was obtained before the test.  
 \*\* Dimensions of Specimen conform to ASTM D 4543-04.

TEST DONE BY: GF  
 REVIEWED BY: WM

UCS SC7-3 Run 4

# Borehole SC7-1

Run 1 and 2

Depth 4.7 to 7.7 m

Elevation 139.0 to 136.0 m

Dry Sample

Run 1 Start  
elev. 139.0 m

Run 1 End  
elev. 137.5 m



Run 2 Start  
elev. 137.5 m

Run 2 End  
elev. 136.0 m



**THURBER** ENGINEERING LTD.

Foundation Investigation  
Culvert 7AN (County Road 20, Sta. 9+600)  
Renfrew, Ontario

W.P. 4068-09-00  
Project No.: 24726



**Borehole SC7-1**  
**Run 1 and 2**  
**Depth 4.7 to 7.7 m**  
**Elevation 139.0 to 136.0 m**  
**Wet Sample**

Run 1 Start  
elev. 139.0 m

Run 1 End  
elev. 137.5 m



Run 2 Start  
elev. 137.5 m

Run 2 End  
elev. 136.0 m

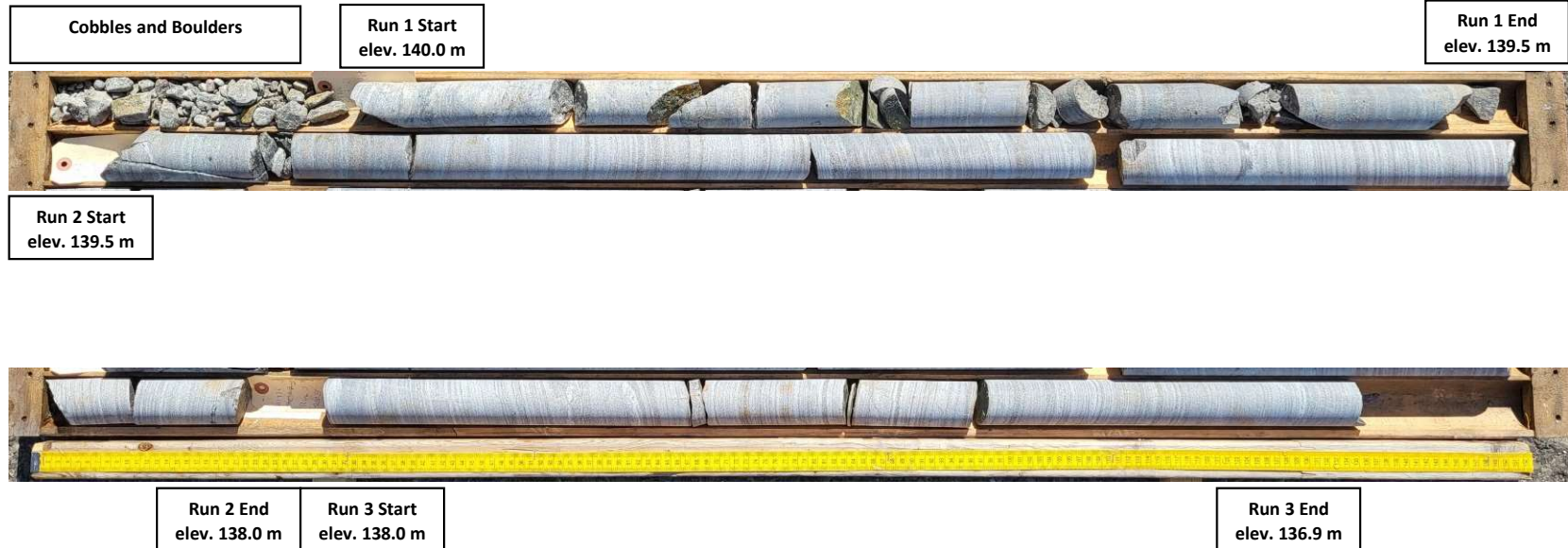
## Borehole SC7-2

Run 1, 2, and 3

Depth 6.7 to 9.8 m

Elevation 140.0 to 136.9 m

Dry Sample



**THURBER** ENGINEERING LTD.

Foundation Investigation  
Culvert 7AN (County Road 20, Sta. 9+600)  
Renfrew, Ontario

W.P. 4068-09-00  
Project No.: 24726

## Borehole SC7-2

Run 1, 2, and 3

Depth 6.7 to 9.8 m

Elevation 140.0 to 136.9 m

Wet Sample

Cobbles and Boulders

Run 1 Start  
elev. 140.0 m

Run 1 End  
elev. 139.5 m



Run 2 Start  
elev. 139.5 m



Run 2 End  
elev. 138.0 m

Run 3 Start  
elev. 138.0 m

Run 3 End  
elev. 136.9 m

**Borehole SC7-3**  
**Run 1, 2, 3 and 4**  
**Depth 3.8 to 6.8 m**  
**Elevation 140.6 to 137.6 m**  
**Dry Sample**

Run 1 Start elev. 140.6 m	Run 1 End elev. 140.5 m	Run 2 Start elev. 140.5 m
------------------------------	----------------------------	------------------------------

Run 2 End elev. 139.6 m	Run 3 Start elev. 139.6 m
----------------------------	------------------------------



Run 3 End elev. 138.9 m	Run 4 Start elev. 138.9 m
----------------------------	------------------------------

Run 4 End elev. 137.6 m
----------------------------



**Borehole SC7-3**  
**Run 1, 2, 3 and 4**  
**Depth 3.8 to 6.8 m**  
**Elevation 140.6 to 137.6 m**  
**Wet Sample**

Run 1 Start elev. 140.6 m	Run 1 End elev. 140.5 m	Run 2 Start elev. 140.5 m
------------------------------	----------------------------	------------------------------

Run 2 End elev. 139.6 m	Run 3 Start elev. 139.6 m
----------------------------	------------------------------



Run 3 End elev. 138.9 m	Run 4 Start elev. 138.9 m
----------------------------	------------------------------

Run 4 End elev. 137.6 m
----------------------------



## **Appendix C.2**

### **Analytical Testing Results**

Certificate of Analysis

Report Date: 11-Mar-2024

Client: Thurber Engineering Ltd.

Order Date: 5-Mar-2024

Client PO: Culvert 105 and Culvert 7

Project Description: 24726 task 700.706a

Client ID:	SC105-2 SS#2 2'6"-4'6"	SC7-1 SS#3A 5'-6'	-	-	
Sample Date:	28-Feb-24 09:00	26-Feb-24 15:00	-	-	-
Sample ID:	2410180-01	2410180-02	-	-	-
Matrix:	Soil	Soil	-	-	-
MDL/Units					

**Physical Characteristics**

% Solids	0.1 % by Wt.	87.5	76.1	-	-	-	-
----------	--------------	------	------	---	---	---	---

**General Inorganics**

Conductivity	5 uS/cm	90	679	-	-	-	-
pH	0.05 pH Units	7.24	7.11	-	-	-	-
Resistivity	0.1 Ohm.m	111	14.7	-	-	-	-

**Anions**

Chloride	10 ug/g	<10	194	-	-	-	-
Sulphate	10 ug/g	<10	215	-	-	-	-

**SGS Canada Inc.**

P.O. Box 4300 - 185 Concession St.  
Lakefield - Ontario - K0L 2H0  
Phone: 705-652-2000 FAX: 705-652-6365

**Paracel Laboratories**

Attn : Dale Robertson

300-2319 St.Laurent Blvd.  
Ottawa, ON  
K1G 4K6, Canada

Phone: 613-731-9577  
Fax: 613-731-9064

12-March-2024

**Date Rec. :** 07 March 2024  
**LR Report:** CA13227-MAR24  
**Reference:** Project#: 2410180

**Copy:** #1

## CERTIFICATE OF ANALYSIS

### Final Report

Sample ID	Sample Date & Time	Sulphide (Na <sub>2</sub> CO <sub>3</sub> ) %
1: Analysis Start Date		12-Mar-24
2: Analysis Start Time		07:24
3: Analysis Completed Date		12-Mar-24
4: Analysis Completed Time		09:03
5: RL		0.01
<del>6: SC105-2 SS#2 2'-6" 4'-8"</del>	<del>28-Feb-24 09:00</del>	<del>&lt; 0.01</del>
7: SC7-1 SS#3A 5'-6'	26-Feb-24 15:00	0.69

RL - SGS Reporting Limit

### Method Descriptions

Parameter	Description	SGS Method Code
Sulphide (Na <sub>2</sub> CO <sub>3</sub> )	Sulphide by ECS	ME-CA-[ENV]ARD-LAK-AN-020

Kimberley Didsbury  
Project Specialist,  
Environment, Health & Safety





## **Appendix D.**

### **Site Photographs**



**Photo 1. Looking east along north embankment and culvert inlet (April 24, 2024)**



**Photo 2. Looking south at culvert outlet (March 22, 2024)**





**Photo 3. Looking northeast along south embankment and culvert outlet  
(April 24, 2024)**



**Photo 4. Looking west along north embankment (March 22, 2024)**



**Photo 5. Looking east along County Road 20 eastbound (March 18, 2024)**



## **Appendix E.**

### **GSC Seismic Hazard Calculation**



# 2015 National Building Code Seismic Hazard Calculation

INFORMATION: Eastern Canada English (613) 995-5548 français (613) 995-0600 Facsimile (613) 992-8836  
Western Canada English (250) 363-6500 Facsimile (250) 363-6565

Site: 45.500N 76.667W

User File Reference: Culvert 7 AN, County Road 20, Sta. 9+600

2024-07-05 20:33 UT

Probability of exceedance per annum	0.000404	0.001	0.0021	0.01
Probability of exceedance in 50 years	2 %	5 %	10 %	40 %
Sa (0.05)	0.361	0.185	0.105	0.031
Sa (0.1)	0.427	0.230	0.138	0.045
Sa (0.2)	0.357	0.199	0.124	0.043
Sa (0.3)	0.271	0.155	0.098	0.035
Sa (0.5)	0.193	0.113	0.072	0.026
Sa (1.0)	0.098	0.059	0.038	0.013
Sa (2.0)	0.048	0.028	0.018	0.005
Sa (5.0)	0.013	0.007	0.004	0.001
Sa (10.0)	0.005	0.003	0.002	0.001
PGA (g)	0.229	0.126	0.076	0.025
PGV (m/s)	0.161	0.090	0.056	0.018

**Notes:** Spectral ( $S_a(T)$ , where  $T$  is the period in seconds) and peak ground acceleration (PGA) values are given in units of  $g$  ( $9.81 \text{ m/s}^2$ ). Peak ground velocity is given in  $\text{m/s}$ . Values are for "firm ground" (NBCC2015 Site Class C, average shear wave velocity  $450 \text{ m/s}$ ). NBCC2015 and CSAS6-14 values are highlighted in yellow. Three additional periods are provided - their use is discussed in the NBCC2015 Commentary. Only 2 significant figures are to be used. **These values have been interpolated from a 10-km-spaced grid of points. Depending on the gradient of the nearby points, values at this location calculated directly from the hazard program may vary. More than 95 percent of interpolated values are within 2 percent of the directly calculated values.**

## References

**National Building Code of Canada 2015 NRCC no. 56190;** Appendix C: Table C-3, Seismic Design Data for Selected Locations in Canada

**Structural Commentaries (User's Guide - NBC 2015: Part 4 of Division B)**  
**Commentary J:** Design for Seismic Effects

**Geological Survey of Canada Open File 7893** Fifth Generation Seismic Hazard Model for Canada: Grid values of mean hazard to be used with the 2015 National Building Code of Canada

See the websites [www.EarthquakesCanada.ca](http://www.EarthquakesCanada.ca) and [www.nationalcodes.ca](http://www.nationalcodes.ca) for more information



Natural Resources  
Canada

Ressources naturelles  
Canada

Canada



## **Appendix F.**

### **Foundation Comparison**



### COMPARISON OF ALTERNATIVE FOUNDATION TYPES

	<b>Rigid Pipe Culverts</b>	<b>Open-Bottom Box Culverts</b>	<b>Closed-Bottom Box Culverts</b>
<b>Advantages</b>	Relatively expedient installation if precast units are used.	Culvert will be supported directly on bedrock. Settlement of culvert will be negligible.  Readily encompasses natural substrate. Preferable from environmental perspective	Relatively expedient installation if precast units are used.
<b>Disadvantages</b>	Requires a temporary by-pass to maintain waterflow  Several parallel pipes may be required to provide hydraulic opening equivalent to box culvert  Protection system will require bracing, anchors and/or rakers  Difficult to include natural substrate.	Requires protection system for construction of foundations  Protection system will require bracing, anchors and/or rakers  Deepest excavation increases quantities and dewatering concerns.  Less expedient installation as cast-in-place footings needed prior to placement of precast units	Requires a temporary by-pass to maintain waterflow  Requires deeper concrete box with increased rise to include natural substrate.  Protection system will require bracing, anchors and/or rakers
<b>Risks/Consequences</b>	Some risk of basal instability during excavation due to depth of excavation below water table.	Risk that bedrock surface is higher than anticipated/would require bedrock excavation  Risk that bedrock surface is deeper than anticipated/would require mass concrete to create level founding surface	Some risk of basal instability during excavation due to depth of excavation below water table.
<b>Relative Cost</b>	Low to Moderate	Moderate	Moderate
<b>Recommendation</b>	<b>Feasible</b>	<b>Not Recommended</b>	<b>Recommended</b>

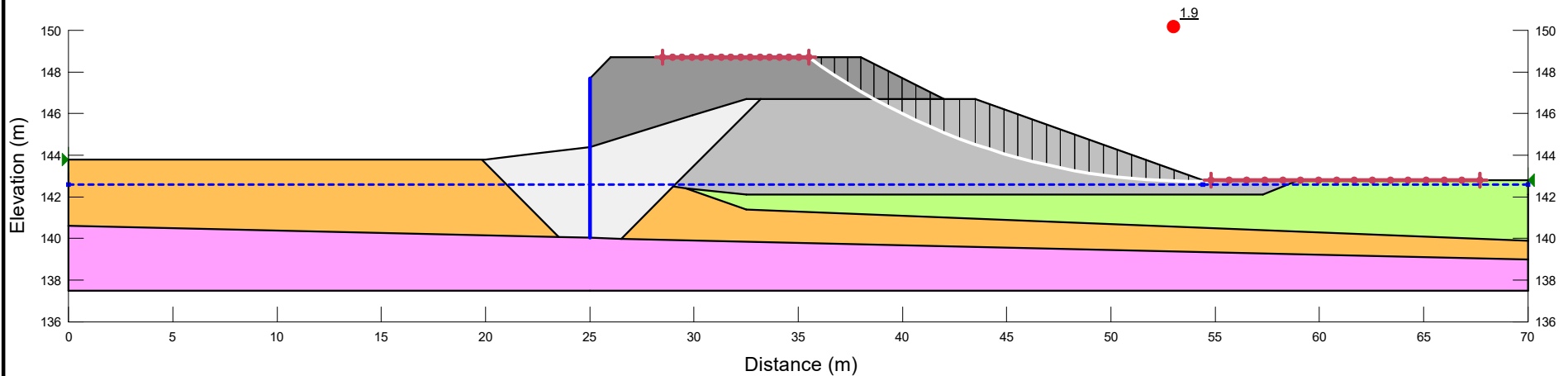




## **Appendix G.**

### **Slope Stability Analysis Figures**

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
<span style="color: green;">■</span>	b) Clayey Silt (Drained)	Mohr-Coulomb	17.5	5	27
<span style="color: orange;">■</span>	c) Silty Sand	Mohr-Coulomb	19	0	30
<span style="color: magenta;">■</span>	d) Bedrock	Bedrock (Impenetrable)			
<span style="color: gray;">■</span>	e) SSM	Mohr-Coulomb	21	0	32
<span style="color: lightgray;">■</span>	g) Existing Fill	Mohr-Coulomb	20	0	30
<span style="color: white;">■</span>	h) Granular A	Mohr-Coulomb	22.8	0	35




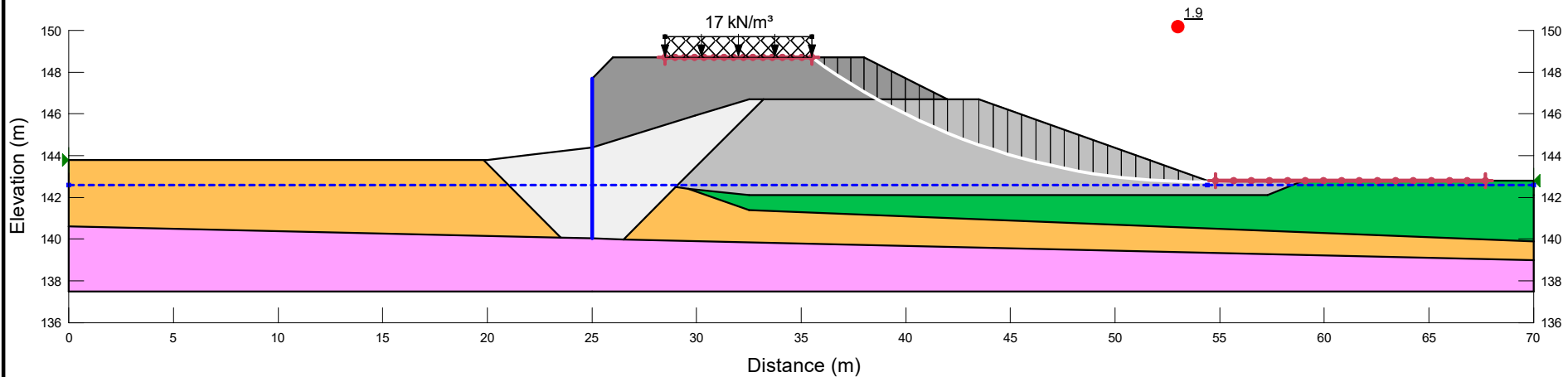
	Project			Additional Details	
	24726 -County Road 20, Sta 9+600, Culvert 7AN			Name: b) 2.0H:1V SSM embankment and wall	
	Analysis			Comments:	
	b1) Permanent, long term, static, drained			Method: Morgenstern-Price, Half-Sine	
Seismic Coefficient		Last Run		Minimum Slip Surface Depth: 1.5 m	
H: g, V: g		2024-11-28, 01:45:16 PM		Entry: (35.5, 148.7) m, Exit: (55.66, 142.8) m	
				Center: (53.884676, 174.12665) m, Radius: 31.376919 m	
		1:300			

Figure G1-1

**Figure G1-1**

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Undrained Shear Strength (kPa)
<span style="color: green;">■</span>	a) Clayey Silt (Undrained)	Undrained (Phi=0)	17.5			75
<span style="color: orange;">■</span>	c) Silty Sand	Mohr-Coulomb	19	0	30	
<span style="color: magenta;">■</span>	d) Bedrock	Bedrock (Impenetrable)				
<span style="color: gray;">■</span>	e) SSM	Mohr-Coulomb	21	0	32	
<span style="color: lightgray;">■</span>	g) Existing Fill	Mohr-Coulomb	20	0	30	
<span style="color: white;">■</span>	h) Granular A	Mohr-Coulomb	22.8	0	35	



Project 24726 -County Road 20, Sta 9+600, Culvert 7AN		
Analysis b2) Temporary (traffic), short term, static, undrained		
Seismic Coefficient H: g, V: g	Last Run 2024-11-28, 01:45:18 PM	Scale 1:300

Additional Details  
 Name: b) 2.0H:1V SSM embankment and wall  
 Comments:  
 Method: Morgenstern-Price, Half-Sine  
 Minimum Slip Surface Depth: 1.5 m  
 Entry: (35.5, 148.7) m, Exit: (55.66, 142.8) m  
 Center: (53.884676, 174.12665) m, Radius: 31.376919 m

**Figure G1-2**



## **Appendix H.**

### **List of Referenced Specifications Non-Standard Special Provisions**



1. The following Special Provisions and OPSS Documents are referenced in this report:

OPSS.PROV 180	Management of Excess Materials
OPSS.PROV 206	Construction Specification for Grading
OPSS.PROV 401	Trenching, Backfilling, and Compacting
OPSS.PROV 421	Pipe Culvert Installation in Open Cut
OPSS.PROV 422	Precast Reinforced Concrete Box Culverts and Box Sewers 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 803	Vegetative Cover
OPSS.PROV 804	Construction Specification for Seed and Cover
OPSS.PROV 805	Construction Specification for Temporary Erosion and Sediment Control Measures
OPSS.PROV 902	Construction Specification for Excavating and Backfilling Structures
OPSS.PROV 1010	Material Specification for Aggregates Base, Subbase, Select Subgrade, and Backfill Material
OPSS.PROV 1860	Material Specification for Geotextiles
OPSD 208.010	Benching of Earth Slopes
OPSD 219.110	Light-Duty Silt Fence Barrier
OPSD 802.031	Rigid Pipe Bedding, Cover and Backfill, Type 3 Soil, Earth Excavation
OPSD 803.010	Backfill and Cover for Concrete Culverts with Spans Less Than or Equal To 3.0 m
OPSD 803.031	Frost Treatment - Pipe Culverts, Frost Penetration Line Between Top of Pipe and Bedding Grade
OPSD 810.010	General Rip-Rap Layout for Sewer and Culvert Outlets
OPSD 3090.101	Foundation Frost Depths for Southern Ontario
OPSD 3101.150	Walls Abutment, Backfill Minimum Granular Requirement
SP 110S06	Amendment to OPSS 1010, April 2013
SP 517F01	Amendment to OPSS 517 - Construction Specification for Dewatering



2. Suggested wording for NSSPs

**“Protection of Sensitive Foundation Soils”**

The Contractor is advised that the native silty and clayey soils that will be exposed at the subgrade are moisture sensitive and may become disturbed or otherwise negatively impacted when subjected to construction or personnel traffic, freeze-thaw actions, ingress or ponding water. The Contractor shall be responsible for selecting appropriate granular compaction equipment, implementing adequate groundwater control measures and to minimize construction and personnel traffic on the founding subgrade.

**“Structural Backfill”**

Structural backfill for the culvert and retaining walls shall consist of OPSS Granular B Type II or Quarry Sourced OPSS Granular A material.

**“Shallow and Sloping Bedrock”**

The contractor is hereby notified that bedrock was encountered at variable elevation in the boreholes drilled at the site. Rock excavation may be required at some locations. Mass concrete may be required to create level surfaces for foundation elements. The Contractor shall be prepared to excavate bedrock to achieve design grades. The presence of shallow bedrock may affect the installation of Temporary Protection Systems. The Contractor's Temporary Protection System design shall include consideration of shallow bedrock.