



THURBER ENGINEERING LTD.

**PRELIMINARY
FOUNDATION INVESTIGATION AND DESIGN REPORT
HIGHWAY 17 TWINNING, RENFREW AREA
CULVERT 105
DUGALD ROAD, STA. 9+890, HORTON TOWNSHIP
WP 4068-09-00 / ASSIGNMENT NO. 4018-E-0009**

Geocres No.: 31F07-003

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 proposed structural culvert at about Sta. 9+890 on the realignment of Dugald Road in Horton Township, north of the new Bruce Street Interchange.

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.

Under the same MTO Assignment a foundation investigation was conducted by Thurber for the new Bruce Street Interchange, which is located approximately 440 m west of the site. The available information was reviewed prior to this investigation and can be found in the Geocres Library under Geocres Number 31F-234.

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

Dugald Road is to be re-aligned as part of the construction of the Bruce Street (County Road 20) Interchange. The realignment will shift the County Road 20 intersection with Dugald Road 315 m to the north. For project purposes, the proposed realignment of Dugald Road is herein described as oriented north-south. A new culvert will be required to convey a creek beneath the newly aligned Dugald Road at approximate Sta. 9+890 with creek flow from west to east.

The land adjacent to the site generally consists of undeveloped agricultural fields. The terrain is relatively flat in the vicinity of the proposed culvert site and generally more rugged along the creek. The area directly adjacent to the proposed culvert is a low-lying marsh dominated with grasses and transitions to mostly farmland with some deciduous trees and shrubs found along the creek line. A driveway crossing an 1,800 mm diameter culvert is currently located approximately 20 m northeast of the site. Overhead utility lines parallel that driveway.

Upstream of the site, a creek crosses beneath County Road 20 through an existing 3600 mm diameter CSP culvert. It is understood that the CSP culvert has an invert elevation at the outlet of 142.6 m.

The water depth in the creek at the proposed location of the culvert was measured to be approximately 600 mm at the time of the field investigation. The elevation of the creek bed near the proposed culvert location was surveyed as 142.3 m on July 26, 2024.

Photographs showing the existing conditions in the area of the site at the time of the field investigation are included in Appendix D for reference.

2.2 Site Geology

According to Crins et al. 2009¹ the project area is described as Ecoregion 6E (Lake Simcoe-Rideau Ecoregion) within the Mixedwood Plains Ecozone. According to Wester et al. 2018² 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 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.

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

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



Ontario Geological Survey Map P.3784³ 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 on February 27 and 28, 2024, and consisted of two off-road boreholes identified as SC105-1 and SC105-2. The boreholes were advanced with a CME 55 track mounted drill rig utilizing hollow stem augers, and NW casing and coring techniques. 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

Borehole No.	Drilled Location	Northing (Latitude)	Easting (Longitude)	Ground Surface Elevation (m)	Termination Depth (m)
SC105-1	Near Inlet	5 039 944.1 (45.499299)	291 820.4 (-76.666084)	143.2	14.1
SC105-2	Near Outlet	5 039 955.1 (45.499398)	291 854.0 (-76.665654)	143.2	7.5

Soil samples were obtained at selected intervals using a split spoon sampler in conjunction with Standard Penetration Testing (SPT) in general accordance with ASTM D 1586. In-situ shear vane testing was carried out within the cohesive layers, where possible, using an MTO 'N' sized vane in general accordance with ASTM D 2573. A Thin-Walled (Shelby) Tube samples were pushed and retrieved in Borehole SC105-1 to obtain a relatively undisturbed cohesive soil sample for further laboratory testing.

A 50 mm diameter well was installed in each of the boreholes 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

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



investigation will be decommissioned by Thurber, as outlined in the Hydrogeological Investigation and Design Report.

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.

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 organic silt over a native deposit of clayey silt followed by silty sand glacial till over bedrock.

5.1 Organic Silt (OI)

A deposit of organic silt containing root material, peat, and various amounts of sand was encountered below the ground surface in the boreholes. The thickness of the layer was 0.6 m (base elev. 142.6 m). The organic silt is described as very loose based on tactile evaluations of strength.

The moisture content of two samples tested were 67 and 153%. The results of a grain size analysis test conducted on a sample of this material are summarized in the table below and are



illustrated on Figure C1 in Appendix C. It is noted that hydrometer testing is less accurate for soils containing organic material.

Summary of Grain Size Distribution Testing – Organic Silt

Soil Particle	Percentage (%)
Gravel	0
Sand	20
Silt	59
Clay	21

The results of Atterberg Limits testing carried out on a sample of this material are summarized below and are illustrated on Figure C2 in Appendix C. The laboratory results indicate that the organic silt is of intermediate plasticity (OI).

Summary of Atterberg Limit Testing – Organic Silt

Parameter	Value
Liquid Limit	43
Plastic Limit	28
Plasticity Index	15

5.2 Clayey Silt (CL)

A glaciomarine native deposit of clayey silt was encountered below the organic silt in the boreholes. Varying amounts of sand were noted within the layer. The thickness of the layer ranged from 2.4 to 6.3 m (base elev. 140.2 to 136.3 m).

Where SPT was conducted within the layer, the N-values typically ranged from weight-of-hammer (WH) to 8 blows. Field vane tests were performed within this layer where possible. All undrained shear strengths obtained were greater than 100 kPa. Remolded vane tests recorded sensitivities typically ranging from less than 5 to 24, indicating that the clayey silt sensitivity varies from sensitive to quick clay (CFEM, 2006). The layer is described as stiff to very stiff in consistency based on N-values, undrained shear strength measurements, and tactile evaluations of strength.

The moisture content of the samples tested ranged from 25 to 34%. The results of three grain size analysis tests conducted on samples of this material are summarized in the table below and are illustrated on Figure C3 in Appendix C.

Summary of Grain Size Distribution Testing – Clayey Silt

Soil Particle	Percentage (%)
Gravel	0
Sand	6 – 11
Silt	57 – 59
Clay	32 – 35

The results of Atterberg Limits testing carried out on three samples of this material are summarized below and are illustrated on Figure C4 in Appendix C. The laboratory results indicate that the clayey silt is of low plasticity (CL). Summary of Atterberg Limit Testing – Clayey Silt

Summary of Atterberg Limit Testing – Clayey Silt

Parameter	Value
Liquid Limit	25 – 27
Plastic Limit	15 – 17
Plasticity Index	9 – 12

5.3 Silty Sand (Glacial Till)

A layer of silty sand till was encountered below the clayey silt deposit in the boreholes. Varying amounts of gravel were encountered throughout the layer. The layer thickness was observed to range from 1.4 to 4.2 m (base elev. 138.8 to 132.1 m). SPT N-values ranged from 11 to 93 blows, indicating a compact to very dense relative density. A refusal blow counted encountered at the base of the layer is attributed to the bedrock surface. Although not observed in the boreholes, it should be anticipated that cobbles and boulders are also present in the glacial till deposit.

The moisture content of the samples tested ranged from 12 to 17%. The results of gradation analyses completed on two samples of the layer are illustrated in Figure C5 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	1 – 13
Sand	61 – 65
Silt	18 – 27
Clay	4 – 11



The results of Atterberg Limit testing conducted on the fines portion of two samples of the deposit indicate a non-plastic material.

5.4 Bedrock

Bedrock was proven by coring in the boreholes. The bedrock surface was sloping with the depth to bedrock observed to be 4.4 and 11.1 m (elevation 138.8 and 132.1 m) in Boreholes SC105-2 and SC105-1 respectively.

The bedrock encountered consisted of fine to medium grained, grey to greyish white, strong marble. Sand seams were encountered within the weathered joints. Photographs of the bedrock cores are provided in Appendix C. The rock core quality measurements are summarized in Table 5-1.

Table 5-1: Bedrock Details

Parameter	Range
Total Core Recovery (TCR), %	78 – 100
Solid Core Recovery (SCR), %	53 – 96
Rock Quality Designation (RQD), %	30 – 96
Fracture Index (fractures per 0.3 m) ⁽¹⁾	0 – >10
Unconfined Compressive Strength (MPa)	67 – 71

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

The RQD values ranged from 30% to 96%, indicating a bedrock of poor to excellent quality (CFEM, 2023). The results of unconfined compressive strength tests (UCS) were 67 and 71 MPa, indicating that the tested samples of the bedrock are 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 SC105-1 and SC105-2. 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 ^(a) (m)	Groundwater Elevation (m)	Date of Measurement
SC105-1	132.2	-0.4	143.6	March 07, 2024
		-0.1	143.3	March 22, 2024
		-0.1	143.3	April 10, 2024
		-0.2	143.4	April 24, 2024
		0.1	143.1	June 04, 2024
		-0.2	143.4	June 26, 2024
		0.1	143.1	August 30, 2024
SC105-2	138.8	-0.1	143.3	March 07, 2024
		0.0	143.2	March 22, 2024
		0.0	143.2	April 10, 2024
		-0.1	143.3	April 24, 2024
		0.0	143.2	June 04, 2024
		-0.2	143.4	June 25, 2024
		-0.1	143.3	July 15, 2024
		0.0	143.2	August 30, 2024

Notes: (a) negative ground water depths indicate artesian conditions

The water depth in the creek was measured to be approximately 600 mm and the elevation of the creek bed near the proposed culvert location was surveyed as 142.3 m on July 26, 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 clayey silt 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)
SC105-2	SS2	0.8 – 1.4	< 10	< 10	< 0.01	7.24	11,100



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. George Downing Estate Drilling Ltd. of Hawkesbury, Ontario, supplied and operated the drill rig used to drill, test, and sample. The field investigation was supervised on a full-time basis by 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., the 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 proposed culvert to be located on realigned Dugald Road near Station 9+890 in Horton Township within the County of Renfrew, Ontario. 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 culvert structure. 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

Dugald Road is to be re-aligned as part of the construction of the Bruce Street (County Road 20) Interchange. The realignment will shift the County Road 20 intersection with Dugald Road 315 m to the north. For project purposes, the proposed realigned Dugald Road is herein described as



oriented north-south. A new culvert will be required to direct creek flow beneath the newly aligned Dugald Road at approximate Sta. 9+890 with creek flow from west to east. Upstream of the site, the creek crosses beneath County Road 20 through an existing 3600 mm diameter CSP culvert. It is understood that the CSP culvert has an invert elevation of approximately 142.6 m.

The proposed culvert is required to convey water from a creek from west to east under a proposed embankment supporting the new Dugald Road. The existing elevation at site is approximately 143.2 m. The elevation of the creek bed near the proposed culvert was surveyed as 142.3 m on July 26, 2024.

The site was found to be underlain by an organic silt deposit overlaying a native deposit of clayey silt followed by silty sand glacial till over sloping bedrock. The bedrock surface was sloping with the depth to bedrock observed to be 4.4 and 11.1 m (elevation 138.8 and 132.1 m) in Boreholes SC105-2 and SC105-1 respectively. It is noted that the water level in the monitoring well SC105-2 was at an elevation of 143.2 m on June 04, 2024. A slight artesian condition was observed in both monitoring wells on earlier dates.

The available foundation information for the nearby, new Highway 17/Bruce Street Interchange was reviewed prior to this investigation and can be found in the Geocres Library under Number 31F-234.

7.2 Proposed Structure

For project purposes, realigned Dugald Road is herein described as oriented north-south.

The Structure and Culvert List of February 23, 2022, for this project indicated that the proposed structural culvert beneath the new Dugald Road alignment is to be a pre-cast, concrete box culvert (CBC) with a length of 29.8 m, a span of 3 m, a rise of 2.4 m, and a slope of 0.84%. It is assumed that the invert of Culvert 105 will be at approximately elevation 142.3 which matches the existing creek bed elevation and is slightly lower than the invert of Culvert 7AN which is approximately 95 m upstream.

As per the preliminary MTO drawings, the proposed final grade of Dugald Road at Station 9+890 is approximate elevation 146.9 m. The proposed embankment has a height of approximately 4.6 m above the original ground surface at the creek. The realigned Dugald Road is to have a total width of 8.5 m comprising two 3.25 m paved lanes with a 1.0m granular shoulder on each side.

No headwalls or retaining walls are proposed at this site.

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 document 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 culvert is to be designed to the “Other” importance category (pending confirmation by MTO). It is understood that the new culvert would have been assigned a consequence classification of Low Consequence, in accordance with Section 6.5.1 of the CHBDC (pending confirmation by MTO). Accordingly, a consequence factor (Ψ) 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)⁴. 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 0.229 g. 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 outside an approach embankment bridge interface zone has no seismic performance requirements for travelled lanes.

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



8.2 Seismic Liquefaction Potential

Based on an assessment using the SPT data following the simplified method for cohesionless soil as outlined in Boulanger and Idriss (2014)⁵, the well graded silty sand (till) soils are not considered susceptible to liquefaction during a 1 in 2475yr design earthquake.

The susceptibility of the cohesive soils at this site to experience liquefaction/cyclic softening was assessed following the Boulanger and Idriss (2007)⁶ 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 D.

The $F(PGA)$, as per Table 4.8 within Section 4.4.3.3 of the CHBDC, is equal to 1.13 for this site yielding a scaled *site-specific* Site Class D PGA of 0.259 g.

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, is equal to 1.115 and 1.417 for this site, yielding a scaled *site-specific* $Sa(0.2)$ of 0.398 and $Sa(1.0)$ of 0.139. 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 structural engineer.

9 DESIGN OPTIONS

9.1 Culvert Type and Foundation Alternatives

Selection of the culvert type must consider the proposed construction procedures, staging requirements, geotechnical resistance available in the foundation soils, depth to suitable bearing stratum and post-construction settlement criteria. It is understood that for this structure a concrete

⁵ 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.

⁶ 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.



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, navigation and hydraulic requirements.

- Open-Bottom Culvert (Box, Arch)

The construction of an open-bottom culvert will have greater construction concerns due to the high water table (small artesian condition) 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 140.4 m which is nearly 3 m below the observed groundwater level. The use of an open bottom culvert would require greater dewatering efforts and has the potential for larger settlement following construction when compared to other culvert options.

- 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 141.7 m which is approximately 1.5 m below the observed groundwater level.

A comparison of these alternatives, based on their respective advantages and disadvantages, is included in Appendix F. Given the soil and groundwater conditions observed on site, a closed bottom culvert is recommended. A concrete pipe culvert is also considered a viable alternative at this site.

9.2 Construction Methodology Alternatives

At the time of the field investigation, the creek flowed from west to east with a water depth measured to be approximately 600 mm near the proposed culvert site. Water levels were measured to be at approximate elevation 143.2 m in the monitoring wells. Excavations will likely extend below the water level of the creek. An adequate and effective dewatering plan including surface water management, cofferdams, creek diversion and excavation dewatering will be required to enable excavation to the required founding elevation and construction of the foundations in the dry (See Section 11.3).



At the time of preparation of this report, a construction staging plan has not yet been developed. Culvert 105 will be on the realigned Dugald Road, thus the foundation recommendations presented herein have been prepared based on the assumption that construction of the culvert will be carried out early in the construction of the realignment without the need to consider traffic staging and no requirement for temporary roadway protection.

9.3 Recommended Approach for Culvert Replacement

A closed-bottom, box culvert is recommended at this site. It is anticipated that construction of the new embankment would be carried out while traffic remains on the existing alignment. A preload period, 1 to 2 months in duration, would be required prior to carrying out the open cut excavation for the culvert installation (See Section 10.8.2). A temporary culvert will be required during the preload period.

Alternatively, the culvert could be designed with a camber or be over-sized to accommodate the anticipated settlement without compromising hydraulic capacity. In this case, the placement of asphalt should be delayed for several months.

A pipe culvert would also be considered a feasible alternative provided a similar preload period is utilized.

10 PRELIMINARY FOUNDATION DESIGN RECOMMENDATIONS

From a foundation engineering perspective, a concrete box culvert is recommended.

• Proposed top of pavement Dugald Road	146.9 m
• Culvert invert	142.3 m
• Approximate elevation of underside of bedding	141.7 m
• Groundwater elevation	143.2 m
• Clayey Silt /Glacial Till interface	140.2 m to 136.3 m

10.1 Concrete Pipe Culvert Foundation

It is anticipated that the invert of the replacement culvert will be within the clayey silt layer. 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 anticipated that the subgrade soils within the culvert footprint will be subjected to the additional loads from the proposed embankment with a height of approximately 4.6 m. Further discussion on the potential settlement of the subgrade soils is provided in Section 10.8. The subgrade should be prepared as described in Section 10.4. Surface water diversion and dewatering will be required to place the bedding material and install the culvert in the dry (Section 11.3).

The recommended geotechnical resistances for a 3.6 m wide (outside dimension) pre-cast, closed-bottom, box culvert with the culvert base at or below approximate elevation 142.0 m, installed on a bedding layer with a minimum thickness of 0.3 m placed on an undisturbed native clayey silt subgrade are as follows:

- Factored Geotechnical Resistance at ULS of 200 kPa
- Factored Geotechnical Resistance at SLS of 100 kPa (provided a preload period is included in the schedule – see Section 10.8)

The factored geotechnical resistances include the following factors:

- Consequence factor (Ψ) of 1.15 (as per CHBDC Table 6.1)
- Geotechnical resistance factors (as per CHBDC Table 6.2):
 - $\phi_{gu} = 0.5$ (static analysis; typical degree of understanding)
 - $\phi_{gs} = 0.8$ (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 reduced in accordance with CHBDC Clause 6.10.2. Foundation settlement, based on the supplied SLS resistance, is expected to be less than 25 mm for culverts constructed on subgrades prepared with good workmanship and in accordance with Sections 10.4 and 10.8.

Resistance to lateral forces/sliding resistance between the precast concrete and underlying granular bedding (Section 10.4) should be evaluated in accordance with the CHBDC assuming an unfactored coefficient of friction of 0.45 for precast concrete. A resistance factor of 0.8 (as per CHBDC Table 6.2) should be used to estimate the sliding resistance between the culvert and Granular A.

10.3 Wingwalls / Retaining Walls

Based on preliminary information provided by MTO, no retaining walls or headwalls are proposed at this location.



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 SP 110S06. “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. The culvert should be constructed following OPSS.PROV 401 and either OPSS.PROV 421 (pipe culvert) or OPSS.PROV 422 (box culvert).

All organics, soft or loose deposits, disturbed soils, alluvial deposits and deleterious materials must be stripped from the footprint of the culvert foundation to expose competent native subgrade material at or below the desired founding elevations. Organic silt with a thickness of approximately 0.6 m (base elevation of 142.6 m) was encountered across the area of the proposed culvert during the drilling investigation and must be removed from beneath the culvert and embankment footprint. The geotechnical resistance values provided above assume that organic material and very soft to soft deposits, where encountered at the subgrade level within the culvert footprint, are removed.

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 μm (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 OPSD 802.031 and OPSD 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 culvert base 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.

10.5 Backfill and Lateral Earth Pressures

Structural backfill material should consist of Granular A or Granular B Type II meeting the OPSS.PROV 1010 and SP 110S06 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 in 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 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 OPSS 3101.150 for the culvert and wingwalls/headwalls. 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/headwalls, where required, must incorporate a subdrain as shown in OPSS 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:

- σ_h = horizontal pressure on the wall at depth h (kPa)
- K = earth pressure coefficient (see table below)
(K_A for unrestrained walls, K_0 for restrained walls)
- γ = 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_0 (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)	3.69	4.60	5.04

The parameters in Table 10-1 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 D. 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 D (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)	0.44	0.37	0.34
Coefficient of Active Earth Pressure, K_{AE} (Unrestrained Wall)	0.35	0.28	0.26

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:

σ_{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)
γ	=	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 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 a sample of the native clayey silt 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 low 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 in 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

Embankments shall be constructed in accordance with OPSS.PROV 206. It is recommended that local marine clay not be used as embankment fill.

10.8.1 Embankment Stability

Embankment stability has been assessed perpendicular to the roadway alignment. Analyses were completed for the proposed embankment at Dugald Road near Station 9+890.

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 outputs provided in Appendix G. The following additional parameters and assumptions were used in the analysis:

- The soil stratigraphy is based on the nearest boreholes.
- A maximum fill height of 4.6 m.
- Embankment fill consisting of Select Subgrade Material (SSM) placed with side slopes as steep as 2H:1V or rockfill placed with side slopes as steep as 1.25H:1V.
- A site adjusted PGA value of 0.13g, equal to $\frac{1}{2}$ of the site adjusted PGA value (0.259g), was used for seismic analysis, as per Section 4.4.3.3, of the CHBDC and outlined in Section 8.3.
- 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 following factor of safety values for the proposed eastbound embankment design:

Table 10-3: Slope Stability Analysis Results for Dugald Road, Sta. 9+890

Condition	Case	Factor of Safety	
		2H:1V [SSM]	1.25H:1V [Rockfill]
Temporary (traffic loading)	Short Term (Undrained)	1.9 (Fig G1-1)	1.8 (Fig G2-1)
Permanent (no traffic loading)	Long Term (Drained)	1.5 (Fig G1-2)	1.5 (Fig G2-2)
Temporary (includes seismic)	Pseudo-Static Seismic (Undrained)	1.1 (Fig G1-3)	1.1 (Fig G2-3)

The geotechnical resistance factors provided in Table 6.2 of the CHBDC for embankment fills with a typical degree of understanding and a Ψ 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 6.3 in Section 6.14.4.1 of the CHBDC indicates a minimum seismic resistance factor of 0.95 for force-based design and 1.0 for performance-based design. Based on these values and consequence factor (Ψ) of 1.15, a target Factor of Safety of 1.0 for this temporary condition with a typical degree of understanding is considered appropriate for the pseudo-static seismic analysis. The pseudo-static results, presented in Table 10-3 above, meet the target Factor of Safety for seismic design. However, it is noted that some displacement of the embankment can occur where the pseudo-static Factor of Safety is less than 1.3. It is noted that 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.

Should slope flattening of the rockfill embankments be used onsite with surplus excavated material, slope protection and drainage measures will be required to ensure the long-term surficial stability of the embankment slopes, see Section 11.4. Slope flattening should meet the requirements of OPSD 202.010 and OPSD 202.020.

10.8.2 Embankment Settlement

The future Dugald Road alignment will result in a fill height of about 4.6 m at the proposed culvert. 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 shown in Table 10-4.

Table 10-4: Summary of MTO Settlement Criteria

Distance from Structure	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. We have utilized consolidation parameters for the clayey silt layer encountered in boreholes drilled for the nearby Highway 17/Bruce Street Interchange as described in Geocres Report 31F-234. The soil parameters used in the models are summarized in Table 10-5, below.

Table 10-5: Summary of Material Parameters

Soil Type	Thickness (m)	Unit Weight (kN/m ³)	Settlement Parameters							
			P _c ' (kPa)	e ₀	Primary				Secondary	
					C _c	C _r	C _v (cm ² /s)	C _{vr} (cm ² /s)	C _α	C _{αr}
Clayey Silt	2.4 – 6.3	17.5	350+	1.1	0.50	0.05	0.041	0.063	0.012	0.004
Silty Sand (Till)	1.4 – 4.2	21.0	E _s = 80 MPa							

Analyses were carried out to calculate the predicted settlement with time, considering SSM embankments and a unit weight of 21 kN/m³. Settlement in the underlying soils from rockfill (unit weight of 20 kN/m³) placement is expected to be slightly less than that generated from SSM fill.



The estimated settlement of the underlying native soils is approximately 120 mm (115 mm of recompression from the clayey silt and 5 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 complete a month after construction. It is recommended that the site be preloaded to minimize post-construction settlement of the culvert. A preload period of 1 to 2 months should be sufficient. A temporary culvert will need to be placed to allow creek flow during the preload period.

Alternatively, the culvert could be designed with a camber or be over-sized to accommodate the anticipated settlement without compromising hydraulic capacity. In this case, the placement of asphalt should be delayed for several months.

In addition to the settlement described above, there will be self-settlement of the 4.6 m high embankment material itself. For embankments constructed with compacted rockfill the short term settlement will be approximately 25 mm (up to 1 year after completion of construction with 90% of this value occurring in the first six months). In addition, rockfill embankments will continue to settle after the first year with an estimate of an additional 5 mm. Similarly, an embankment constructed of compacted SSM material will undergo approximately 25 mm of self settlement with the majority of that complete during construction.

Embankments must be overbuilt to compensate for the estimated settlement.

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 native clayey silt and silty sand materials may be classified as Type 3 soil above the water table and Type 4 below. **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). 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. 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 into the underlying native soil deposits.

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.

Although not anticipated, at locations where there are space restrictions, the excavations could 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

Although not anticipated, Temporary Protection Systems may be required during various stages of construction and must be implemented in accordance with OPSS.PROV 539 as amended by SP 105S09. Performance Level 2 (maximum 25 mm horizontal deflection) is considered appropriate where the protection supports the existing roadway.

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. When designing roadway protection systems, the Contractor should consider the potential for obstructions such as cobbles and boulders (inherent in glacial tills) as well as minor artesian conditions that were noted during groundwater measurements in the glacial till layer. It may be necessary to predrill for the soldier piles. Lateral support may need to be enhanced by socketing the soldier piles into bedrock and/or by using bracing or rakers. Suggested wording for an NSSP for obstructions is included in Appendix H. 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 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^(*) (kN/m³)	K_A	K_P	K_o	Su (kPa)
Native Cohesive Clayey Silt	17.5	-	-	-	75
Native Silty Sand (Glacial Till)	21	0.27	3.69	0.43	-

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

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

11.3 Surface and Groundwater Control

Culvert subgrade preparation and placement and compaction of granular bedding/pads and culvert placement must be carried out in the dry. 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. 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 SP517F01 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 (slight artesian condition, and potential for basal heave) and anticipated works, the Designer Fill-In (Note 2) in SP517F01 Table 1 should be "Yes" for dewatering systems; the design Engineer and design-checking Engineer 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 a minimum of 0.5 m below the underside of the planned excavation base prior to each stage of excavation; Note 5 of SP517F01 Table 1 should be 0.5m.

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 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. It is anticipated that sump pumps will likely be sufficient to extract water from the excavation for the culverts. Pumping from within a sandbag cofferdam system is likely sufficient. More than one pump may be required. A sheet pile cofferdam enclosure driven into the foundation clayey silt may also be considered. The groundwater level within the work zone should be lowered by pumping from sumps to a minimum of 0.5 m below the underside of the planned excavation base prior to each stage of excavation.

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 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 new 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 drawings for the culvert are not available at the time of writing. The preliminary recommendations provided herein will need to be re-evaluated once the culvert invert elevations are confirmed.

The planned construction methodology includes open cut excavations for the installation of foundation elements of a new culvert. 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 foundations in the dry.
- The clayey silt which will be exposed beneath a culvert bedding layer 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.
- Mitigation of the settlement induced by the new embankment fill may require a preload. An instrumentation and monitoring program will need to be implemented to assess the progress of the preload. Given the limited project length (9+870 to 9+930), the monitoring program would include approximately three settlement rods located on the new alignment with a nominal spacing of less than 25 m. The base plates should be installed prior to fill placement and the rods will require extension as fill is placed around them. The top of the settlement rods should be surveyed every week during preload construction and every two weeks for the anticipated 1 to 2 month preload period. The installation of the monitoring equipment and surveying would typically be carried out by the Contractor, with the results evaluated by the Contract Administration team.

Alternatively, the culvert could be designed with a camber or be over-sized to accommodate the anticipated settlement without compromising hydraulic capacity. In this case, the placement of asphalt should be delayed for several months.

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., the Designated Principal Contact for MTO Foundation Projects.

Thurber Engineering Ltd.
Report Prepared By:



Anderson de Oliveira, M.A.Sc., P.Eng.
Geotechnical Engineer



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

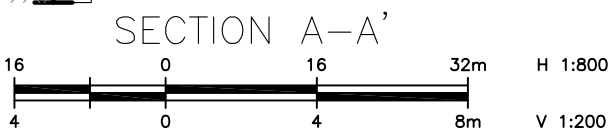
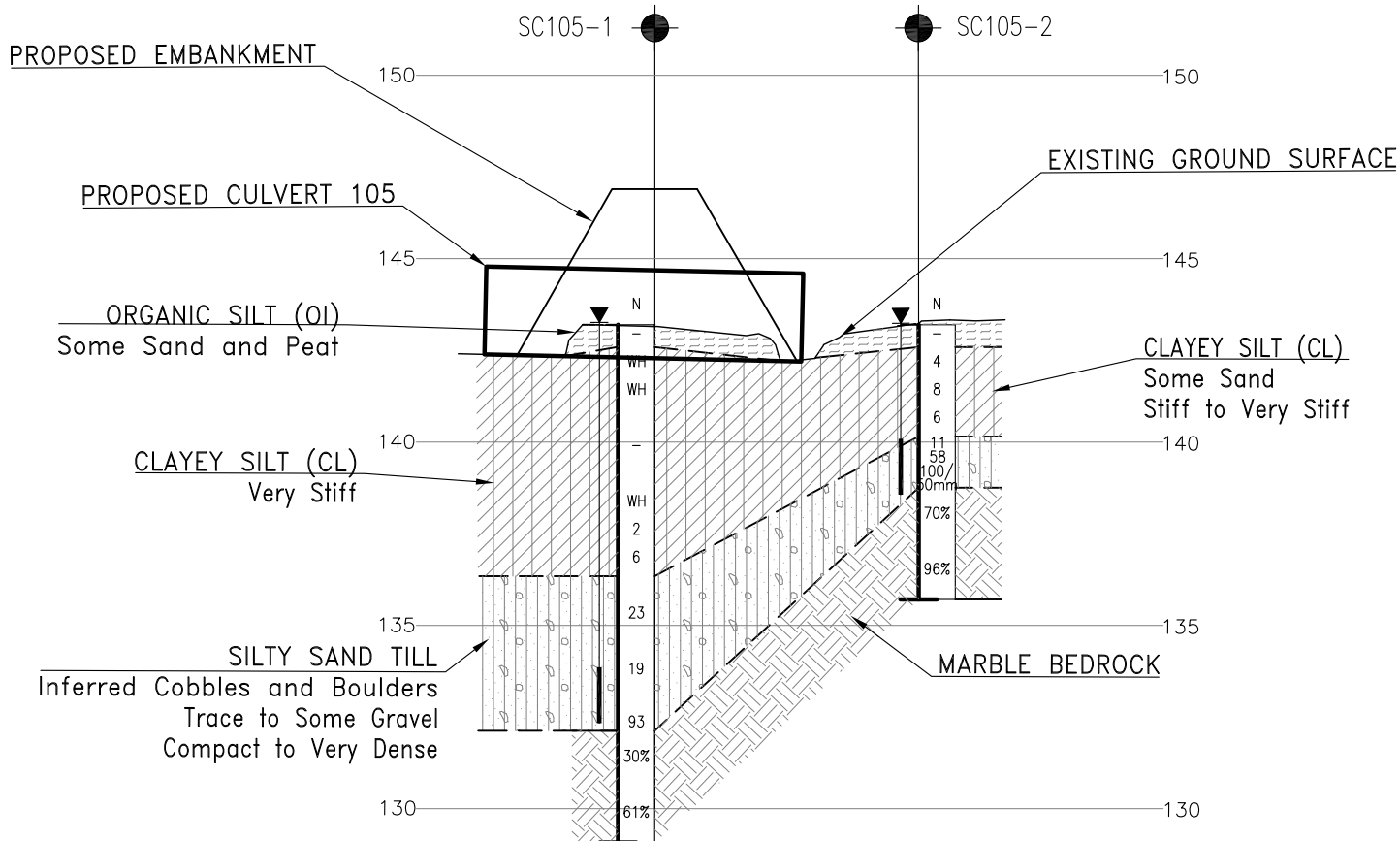
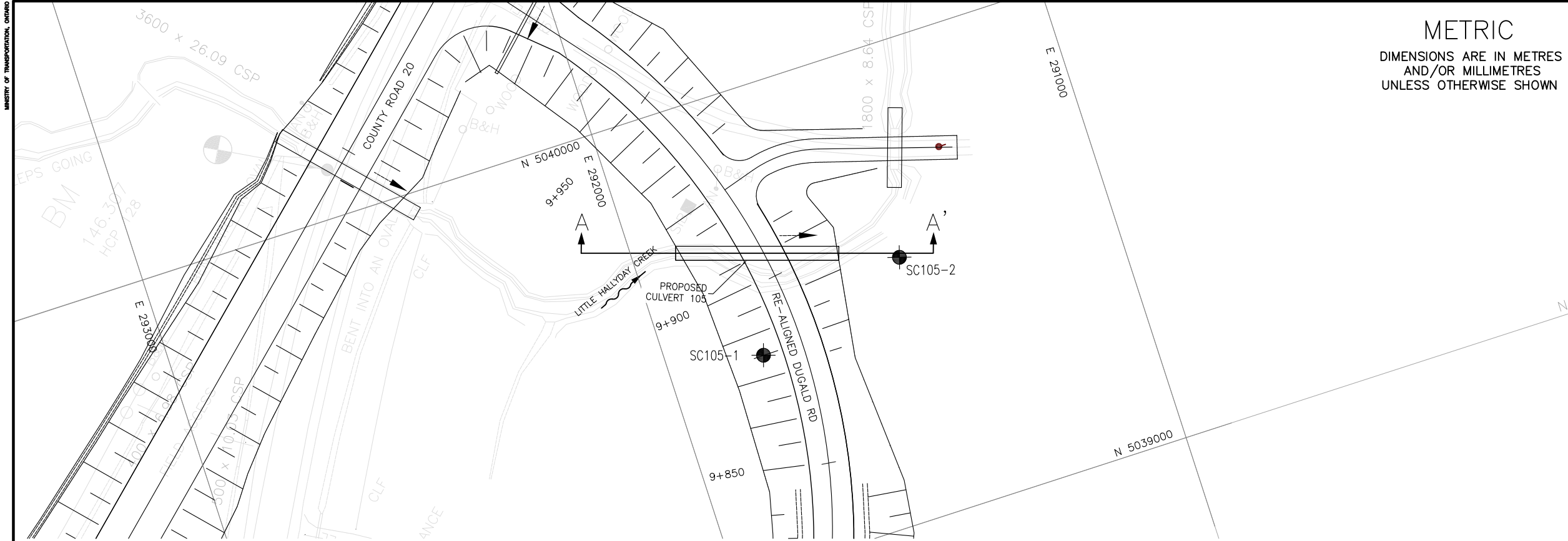


Dr. P.K. Chatterji, P.Eng.
Designated Principal Contact,
Principal, Senior Geotechnical Engineer



Appendix A.

Borehole Location Plan and Stratigraphic Drawings



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

CONT No
GWP No 4068-09-00

HIGHWAY 17 TWINNING
RE-ALIGNED DUGALD RD
STA. 9+890, CULVERT 105
BOREHOLE LOCATION PLAN AND SOIL STRATA

Ontario

SHEET
1

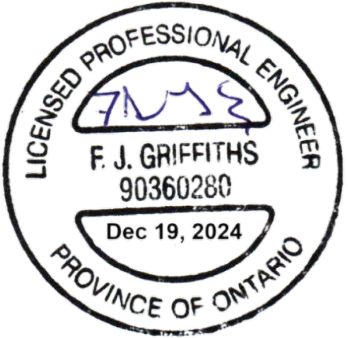


LEGEND			
	Borehole		
	Historic Borehole		
N	Blows /0.3m (Std Pen Test, 475J/blow)		
CONE	Blows /0.3m (60' Cone, 475J/blow)		
PH	Pressure, Hydraulic		
	Water Level Upon Completion of Drilling		
	Water Level in Mointoring Well/Piezometer		
	Monitoring Well/Piezometer Screen		
90%	Rock Quality Designation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

NO	ELEVATION	NORTHING	EASTING
SC105-1	143.2	5 039 944.1	291 820.4
SC105-2	143.2	5 039 955.1	291 854.0

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

GEOCRES No. 31F07-003



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



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).

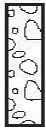
DYNAMIC CONE PENETRATION TEST (DCPT):

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 SC105-1

1 OF 2

METRIC

WP# 4068-09-00 LOCATION Lat: 45.499299°, Long: -76.666084°
Culvert 105; Horton Township; MTM z9: N 5 039 944.1 E 291 820.4 ORIGINATED BY RH
HWY 17 BOREHOLE TYPE CME 55 Trackmount / HSA / NW Casing / NQ Coring COMPILED BY AO
DATUM Geodetic DATE 2024.02.27 - 2024.02.27 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 ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)			
143.2	Ground Surface							20 40 60 80 100		W _P W W _L			GR SA SI CL
0.0	ORGANIC SILT (OI), some sand contains root material and peat very loose brownish black		1	GS	-		143	○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE					0 20 59 21
142.6													
0.6	CLAYEY SILT (CL) very stiff grey		2	SS	WH		142						
			3	SS	WH		141		24.0 + > 115 kPa +				0 6 59 35
			4	TW	-		140						
									4.8 + 8.0 +				
							139						
			5	SS	WH		138						
	- unable to push vane		6	SS	2		137						
			7	SS	6								0 7 59 34
136.3	- unable to push vane												
6.9	SILTY SAND (SM), trace gravel inferred cobbles and boulders compact to very dense grey GLACIAL TILL						136						
			8	SS	23		135						
			9	SS	19		134						1 61 27 11 Non-plastic

Continued Next Page

+³, ×³: Numbers refer to
Sensitivity 20
15 10 5 0
(%) STRAIN AT FAILURE

DOUBLE LINE 24726 CULVERT 105.GPJ 2012TEMPLATE(MTO).GDT 12-16-24

METRIC

SOIL PROFILE					
ELEV DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES	GROUND WATER CONDITIONS	ELEVATION SCALE
			NUMBER	TYPE	"N" VALUES
	Continued From Previous Page				
132.1	SILTY SAND, trace gravel inferred cobbles and boulders compact to very dense grey GLACIAL TILL		10	SS	93
11.1	MARBLE BEDROCK slightly weathered to fresh jointed grey fine to medium grained strong - sand seam from a depth of 11.9 to 12.3 m		1	RUN	-
129.1			2	RUN	-
14.1	End of Borehole Monitoring Well installed: Schedule 40 PVC standpipe with 50-mm diameter and 3.0-m slotted screen. Stick-up cover installed at ground surface. Water Level Readings: DATE DEPTH (m) ELEV. (m) 2024/03/07 -0.4 143.6 2024/03/22 -0.1 143.3 2024/04/10 -0.1 143.3 2024/04/24 -0.2 143.4 2024/06/04 0.1 143.1 2024/06/26 -0.2 143.4 2024/08/30 0.1 143.1				

+³, ×³: Numbers refer to Sensitivity

RECORD OF BOREHOLE No SC105-2

1 OF 1

METRIC

WP# 4068-09-00 LOCATION Lat: 45.499398°, Long: -76.665654°
Culvert 105; Horton Township; MTM z9: N 5 039 955.1 E 291 854.0 ORIGINATED BY RH
HWY 17 BOREHOLE TYPE CME 55 Trackmount / HSA / NW Casing / NQ Coring COMPILED BY AO
DATUM Geodetic DATE 2024.02.28 - 2024.02.28 CHECKED BY CM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							
143.2	Ground Surface							20 40 60 80 100							
0.0	ORGANIC SILT (OI) contains root material and peat very loose brownish black		1	GS	-		143							153	
142.6															
0.6	CLAYEY SILT (CL), some sand stiff to very stiff grey		2	SS	4		142								
			3	SS	8		141								0 11 57 32
			4	SS	6										
140.2							140								
3.0	SILTY SAND (SM), some gravel inferred cobbles and boulders compact to very dense grey GLACIAL TILL		5	SS	11										13 65 18 4 Non-plastic
			6	SS	58		139								
138.8			7	SS	100/ 0mm										
4.4	MARBLE BEDROCK slightly weathered to fresh jointed greyish white fine to medium grained strong		1	RUN	-		138								RUN #1 TCR=90% SCR=83% RQD=70%
							137								
			1	RUN	-										RUN #1 TCR=100% SCR=96% RQD=96% UCS=67MPa
135.7							136								
7.5	End of Borehole														
Monitoring Well installed: Schedule 40 PVC standpipe with 50-mm diameter and 3.0-m slotted screen. Stick-up cover installed at ground surface.															
Water Level Readings:															
DATE	DEPTH (m)	ELEV. (m)													
2024/03/07	-0.1	143.3													
2024/03/22	0.0	143.2													
2024/04/10	0.0	143.2													
2024/04/24	-0.1	143.3													
2024/06/04	0.0	143.2													
2024/06/25	-0.2	143.4													
2024/07/15	-0.1	143.3													
2024/08/30	0.0	143.2													

DOUBLE LINE 24726 CULVERT 105.GPJ 2012TEMPLATE(MTO).GDT 12-16-24



Appendix C.

Laboratory Testing

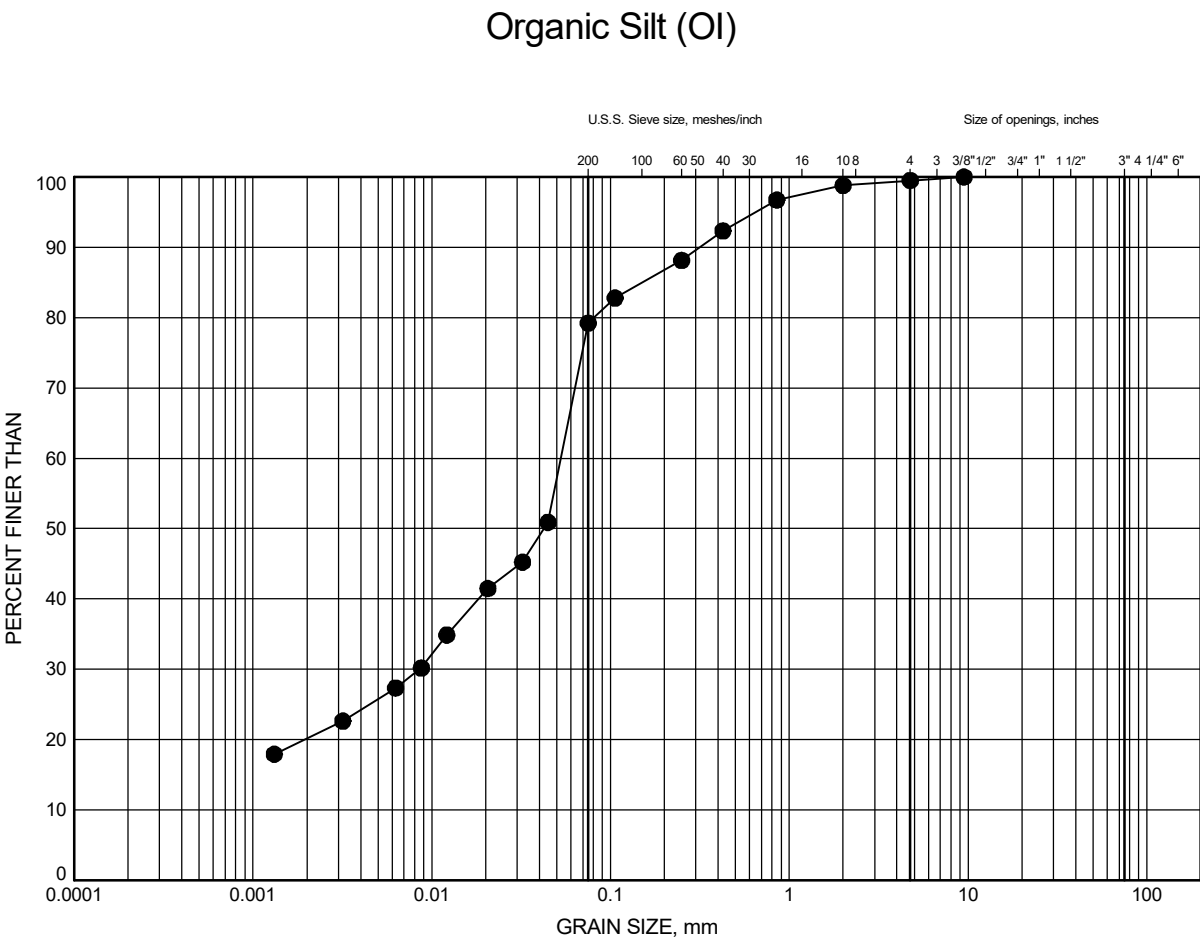


Appendix C.1
Particle Size Analysis Figures
Atterberg Limit Test Results
Unconfined Compressive Strength Testing Results
Rock Core Photos

Highway 17 Twinning, Culvert 105 (Dugald Road)

GRAIN SIZE DISTRIBUTION

FIGURE C1



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	SC105-1	0.3	142.9

GRAIN SIZE DISTRIBUTION - THURBER 24726 CULVERT 105.GPJ 7-11-24

Date July 2024

WP# 4068-09-00



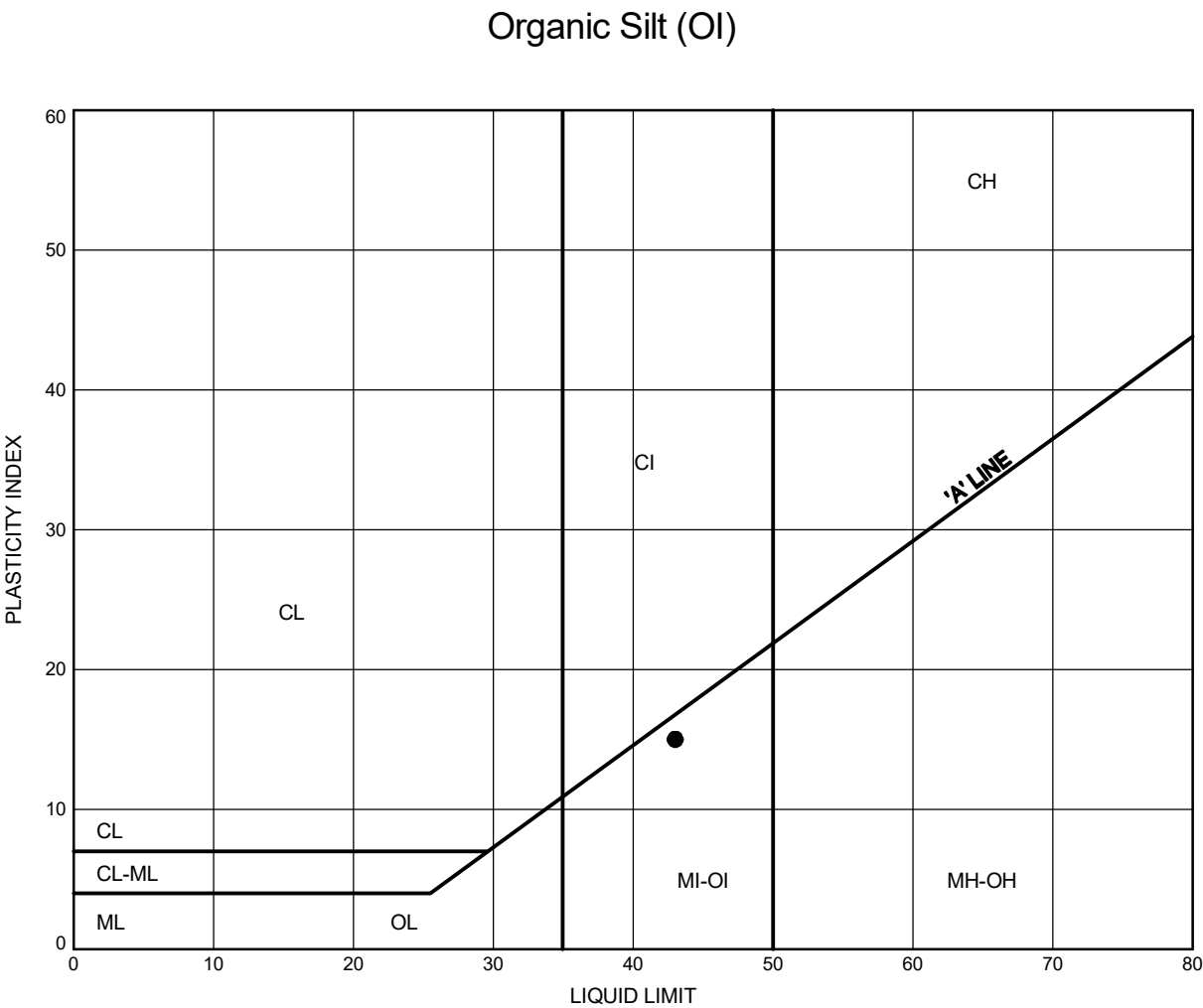
Prep'd RH

Chkd. MJK

Highway 17 Twinning, Culvert 105 (Dugald Road)

ATTERBERG LIMITS TEST RESULTS

FIGURE C2



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	SC105-1	0.3	142.9

Date July 2024
WP# 4068-09-00

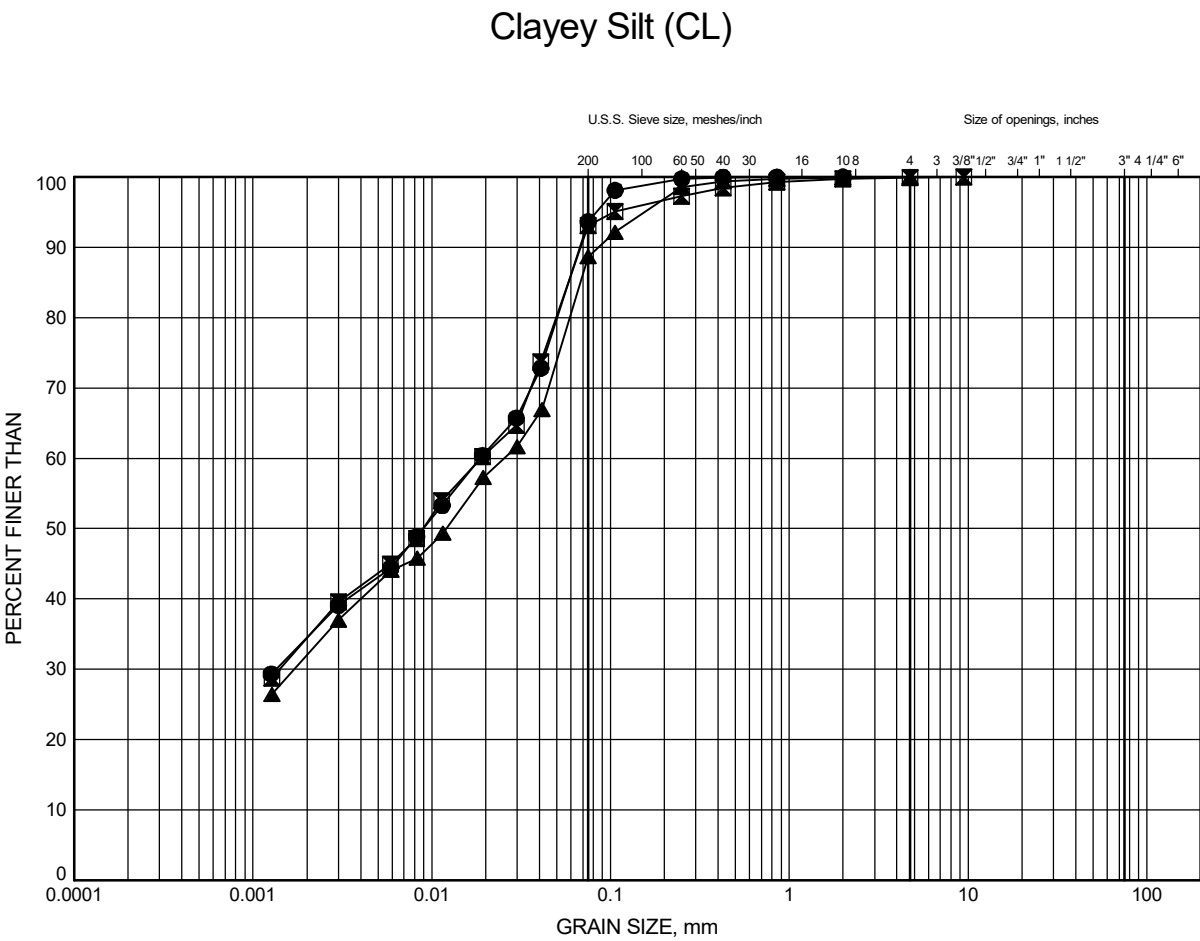


Prep'd RH
Chkd. MJK

Highway 17 Twinning, Culvert 105 (Dugald Road)

GRAIN SIZE DISTRIBUTION

FIGURE C3



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	SC105-1	1.8	141.4
⊠	SC105-1	6.4	136.8
▲	SC105-2	1.8	141.4

GRAIN SIZE DISTRIBUTION - THURBER 24726 CULVERT 105.GPJ 7-11-24

Date July 2024

WP# 4068-09-00



Prep'd RH

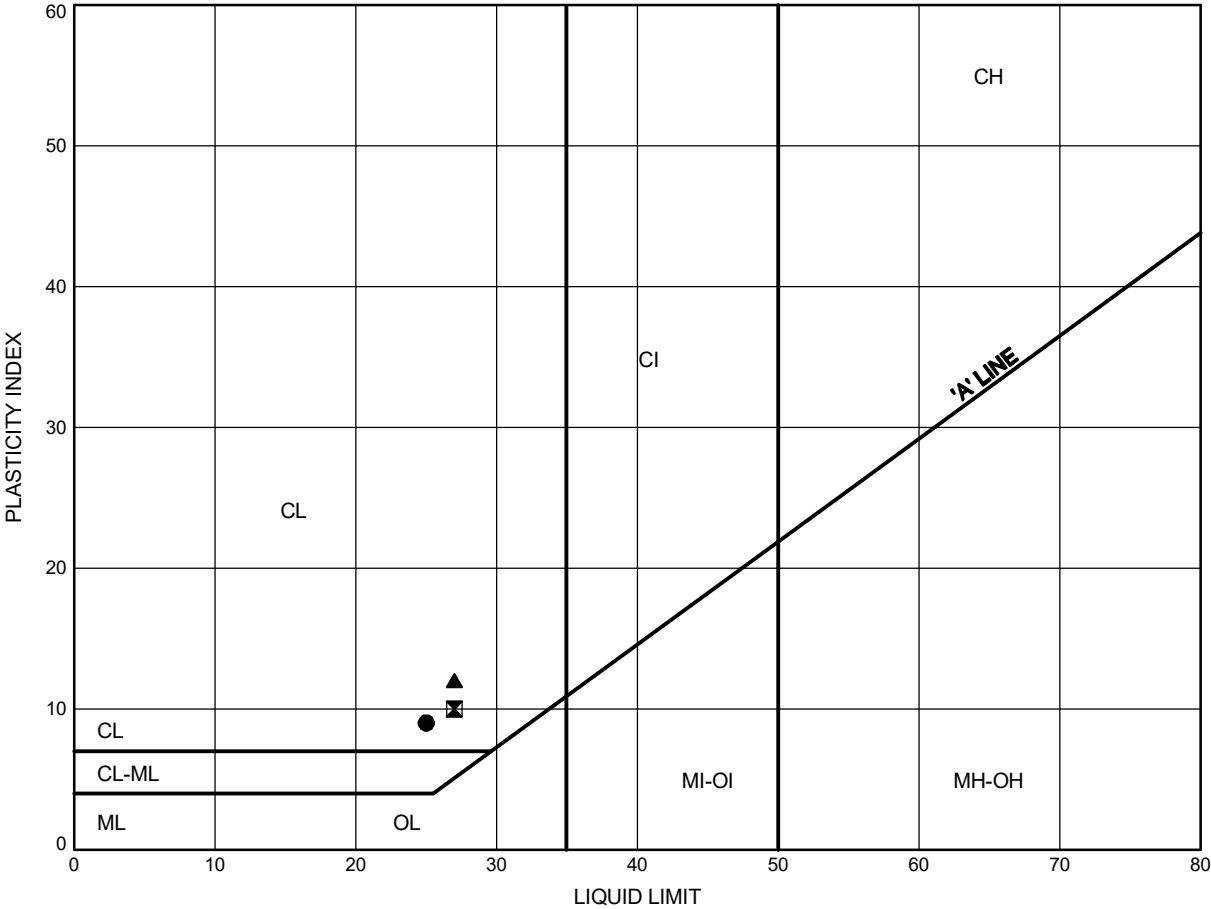
Chkd. MJK

Highway 17 Twinning, Culvert 105 (Dugald Road)

ATTERBERG LIMITS TEST RESULTS

FIGURE C4

Clayey Silt (CL)



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	SC105-1	1.8	141.4
⊠	SC105-1	6.4	136.8
▲	SC105-2	1.8	141.4

THURBALT 24726 CULVERT 105.GPJ 7-11-24

Date July 2024
WP# 4068-09-00



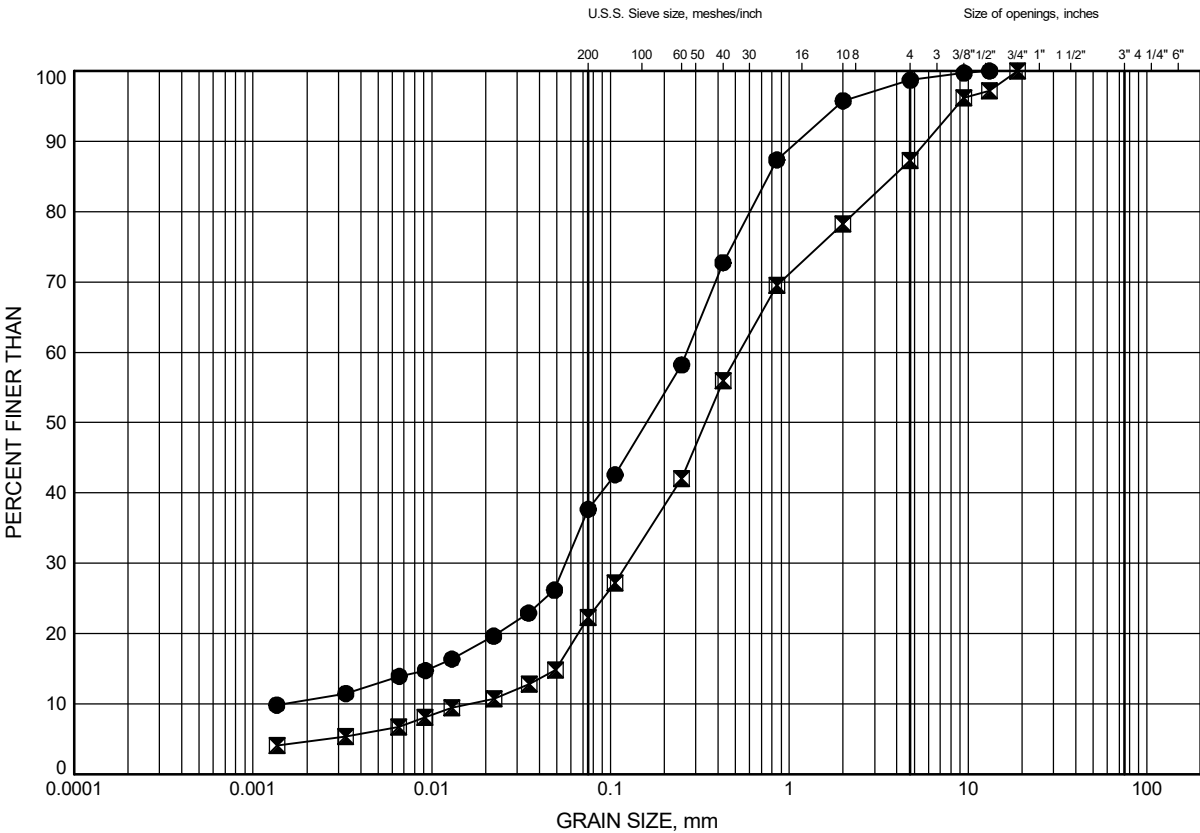
Prep'd RH
Chkd. MJK

Highway 17 Twinning, Culvert 105 (Dugald Road)

GRAIN SIZE DISTRIBUTION

FIGURE C5

Silty Sand (Glacial Till)



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	SC105-1	9.4	133.8
⊠	SC105-2	3.4	139.8

GRAIN SIZE DISTRIBUTION - THURBER 24726 CULVERT 105.GPJ 7-11-24

Date July 2024
WP# 4068-09-00



Prep'd RH
Chkd. MJK

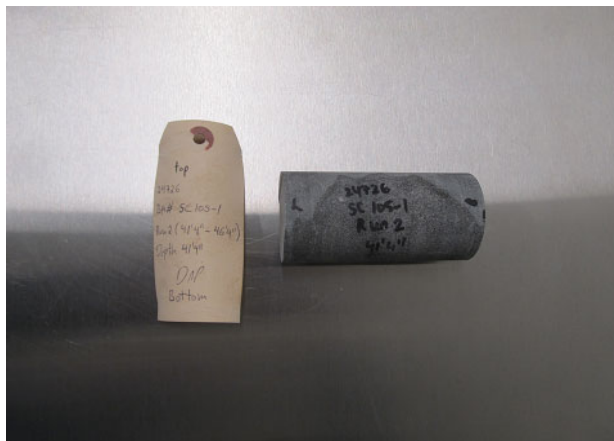
UNCONFINED COMPRESSION TEST REPORT

ASTM D7012-14

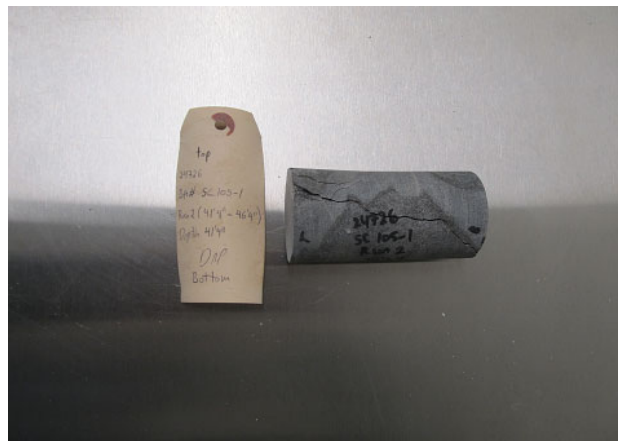
CLIENT:	Thurber Engineering (Ottawa)	FILE NUMBER:	24726
PROJECT NAME:	Highway 17 Twinning - Renfrew	REPORT DATE:	11-Jul-24
BOREHOLE No.:	SC105-1	TEST DATE:	9-May-24
SAMPLE No.:	Run 2		
SAMPLE DEPTH:	12.6 m		
DESCRIPTION:	Marble		

Avg. Height (cm):	9.6	Weight (g):	457.4
Avg. Diameter (cm):	4.7	Wet Density (kg/m ³):	2,746
H. to Dia. Ratio**:	2:1	Dry Density (kg/m ³):	2,746
Cross Sectional Area (cm ²):	17.35	Moisture Content* (%):	N/A
Sample Volume (cm ³):	166.55		

ORIGINAL SPECIMEN



FRACTURED SPECIMEN



AVG. RATE OF STRAIN TO FAILURE:	0.250 MPa/s
MAXIMUM COMPRESSIVE LOAD:	123.6 kN
UNCONFINED COMPRESSIVE STRENGTH:	71.2 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 SC105-1 Run 2

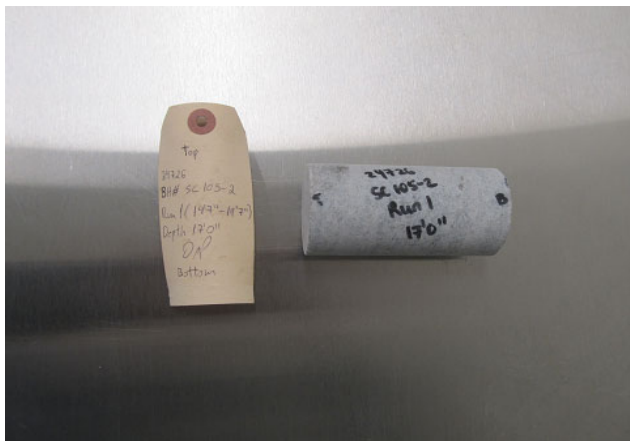
UNCONFINED COMPRESSION TEST REPORT

ASTM D7012-14

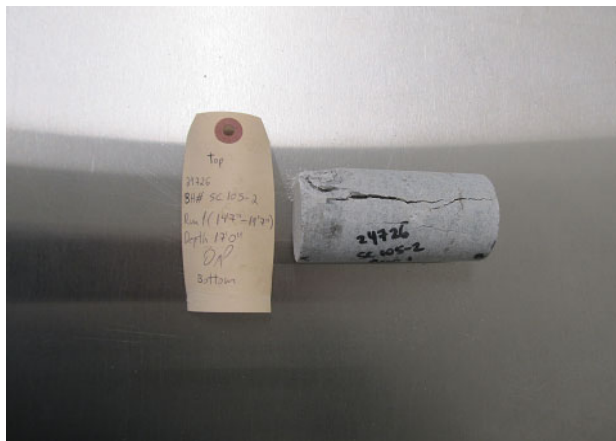
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PROJECT NAME:	Highway 17 Twinning - Renfrew	REPORT DATE:	11-Jul-24
BOREHOLE No.:	SC105-2	TEST DATE:	9-May-24
SAMPLE No.:	Run 1		
SAMPLE DEPTH:	5.18 m		
DESCRIPTION:	Marble		

Avg. Height (cm):	9.6	Weight (g):	454.8
Avg. Diameter (cm):	4.7	Wet Density (kg/m ³):	2,731
H. to Dia. Ratio**:	2:1	Dry Density (kg/m ³):	2,731
Cross Sectional Area (cm ²):	17.35	Moisture Content* (%):	N/A
Sample Volume (cm ³):	166.55		

ORIGINAL SPECIMEN



FRACTURED SPECIMEN



AVG. RATE OF STRAIN TO FAILURE:	0.250 MPa/s
MAXIMUM COMPRESSIVE LOAD:	116.4 kN
UNCONFINED COMPRESSIVE STRENGTH:	67.1 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 SC105-2 Run 1

Borehole SC105-1

Runs 1 and 2

Depth 11.1 to 14.1 m

Elevation 132.1 to 129.1 m

Dry Sample

Run 1 Start
elev. 132.1 m

Sand Seam from 39'2" to 40'3"

Run 1 End
elev. 130.6 m



Run 2 Start
elev. 130.6 m

Run 2 End
elev. 129.1 m

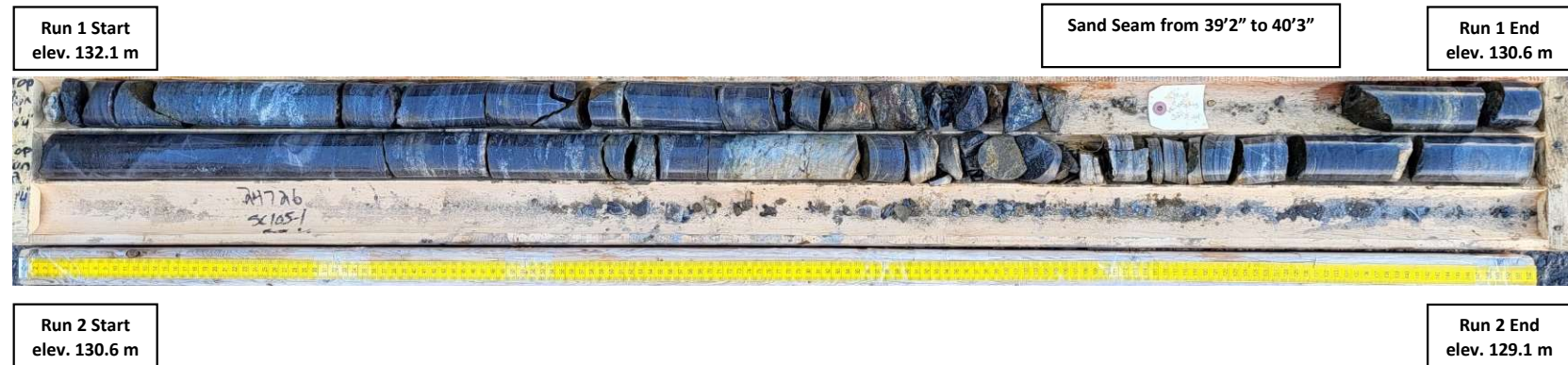


THURBER ENGINEERING LTD.

Foundation Investigation
Culvert 105 (Dugald Road, Sta. 9+890)
Horton Township, Ontario

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

Borehole SC105-1
Runs 1 and 2
Depth 11.1 to 14.1 m
Elevation 132.1 to 129.1 m
Wet Sample



THURBER ENGINEERING LTD.

Foundation Investigation
Culvert 105 (Dugald Road, Sta. 9+890)
Horton Township, Ontario

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

Borehole SC105-2
Runs 1 and 2
Depth 4.4 to 7.5 m
Elevation 138.8 to 135.7 m
Dry Sample

Run 1 Start
elev. 138.8 m

Run 1 End
elev. 137.2 m



Run 2 Start
elev. 137.2 m

Run 2 End
elev. 135.7 m

Borehole SC105-2
Runs 1 and 2
Depth 4.4 to 7.5 m
Elevation 138.8 to 135.7 m
Wet Sample

Run 1 Start
elev. 138.8 m

Run 1 End
elev. 137.2 m



Run 2 Start
elev. 137.2 m

Run 2 End
elev. 135.7 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.	81.5	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 ₂ CO ₃) %
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
6: SC105-2 SS#2 2'6"-4'6"	28-Feb-24 09:00	< 0.01
7: SC7-1 SC#3A 5' 6"	26-Feb-24 15:00	0.60

RL - SGS Reporting Limit

Method Descriptions

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

Kimberley Didsbury
Project Specialist,
Environment, Health & Safety



Appendix D.

Site Photographs



Photo 1. Looking northeast near Station 9+890 (March 22, 2024)
Note: private driveway and CSP to northeast of the site.



Photo 2. Looking northwest near Station 9+890 (March 22, 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: 45.499N 76.666W

User File Reference: Culvert 105, Dugald Road, Sta. 9+890

2024-07-15 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 m/s^2). Peak ground velocity is given in m/s . Values are for "firm ground" (NBCC2015 Site Class C, average shear wave velocity 450 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 and www.nationalcodes.ca for more information



Natural Resources
Canada

Ressources naturelles
Canada

Canada



Appendix F.

Foundation Comparison



COMPARISON OF ALTERNATIVE FOUNDATION TYPES

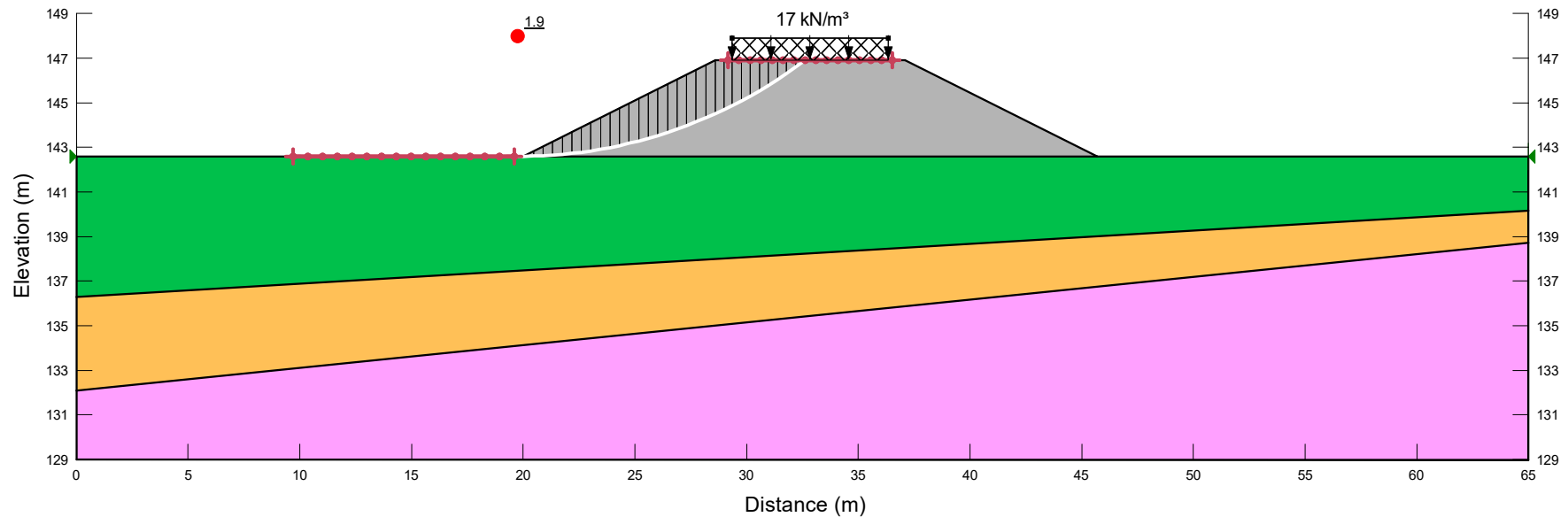
	Pipe Culverts	Open-Bottom Box Culverts	Closed-Bottom Box Culverts
Advantages	<p>Relatively expedient installation if precast units are used</p> <p>Smaller magnitude of settlement than open footing culvert due to lower bearing stress on subgrade</p>	<p>Possibility to maintain work zone outside of existing waterway</p> <p>Readily encompasses natural substrate. Preferable from environmental perspective</p>	<p>Relatively expedient installation if precast units are used</p> <p>Smaller magnitude of settlement than open footing culvert due to lower bearing stress on subgrade</p>
Disadvantages	<p>Requires a temporary by-pass to maintain waterflow</p> <p>Several parallel pipes may be required to provide hydraulic opening equivalent to box culvert</p> <p>Protection system will require bracing, anchors and/or rakers</p> <p>Difficult to include natural substrate</p>	<p>May require protection system for construction of foundations</p> <p>Protection system would require bracing, anchors and/or rakers</p> <p>Deepest excavation increases quantities and dewatering concerns</p> <p>Less expedient installation as cast-in-place footings needed prior to placement of precast units</p>	<p>Requires a temporary by-pass to maintain waterflow</p> <p>Requires deeper concrete box with increased rise to include natural substrate</p> <p>Dewatering will likely be completed using sump and pump methods from behind sandbag coffer dams. Sheet piles could be utilized as an alternative coffer dam; however, their design may need to incorporate bracing, anchors or rakers due to the presence of shallow and sloping bedrock.</p>
Risks/Consequences	Some risk of basal instability during footing excavation due to depth of excavation below water table.	Increased risk of basal instability during footing excavation due to depth of excavation below water table.	Some risk of basal instability during footing excavation due to depth of excavation below water table.
Relative Cost	Low to Moderate	Moderate	Moderate
Recommendation	Feasible	Not Recommended	Recommended



Appendix G.

Slope Stability Analysis Figures

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Undrained Shear Strength (kPa)
■	a) Clayey Silt (Undrained)	Undrained (Phi=0)	17.5			75
■	c) Silty Sand (Glacial Till)	Mohr-Coulomb	21	0	35	
■	d) Bedrock	Bedrock (Impenetrable)				
■	e) SSM	Mohr-Coulomb	21	0	32	

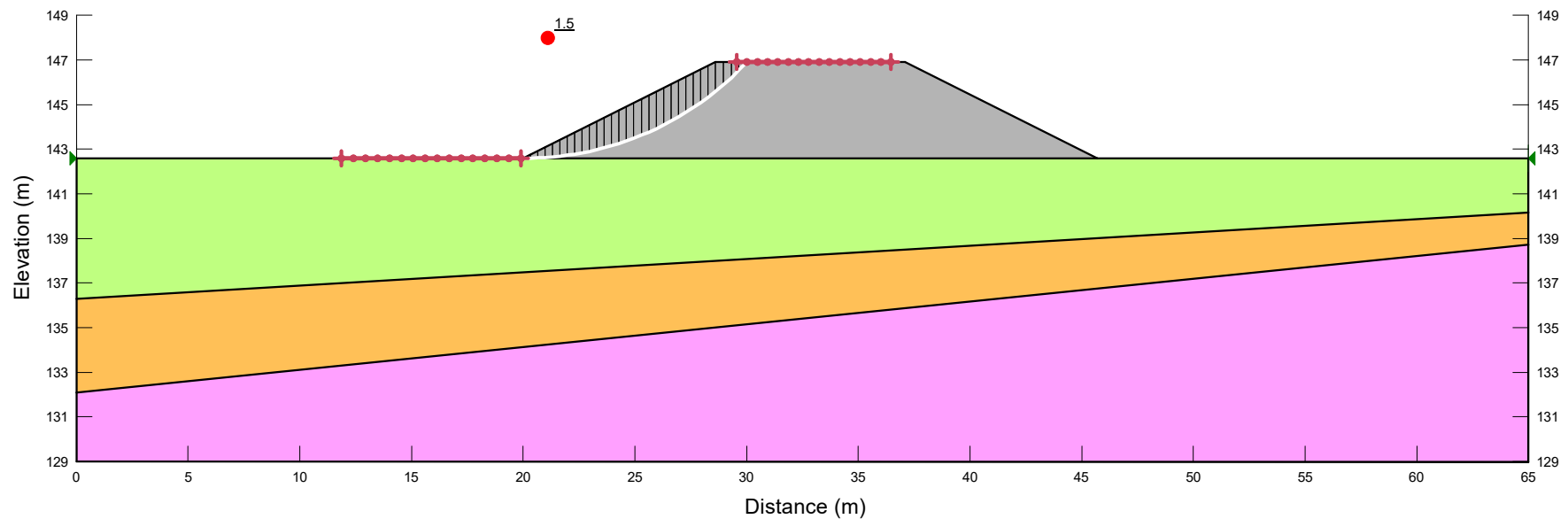


Project 24726 - Hwy 17 Sta 9+890, Culvert 105		
Analysis a1) Temporary (traffic), short term, static, undrained		
Seismic Coefficient H: g, V: g	Last Run 2024-11-28, 02:35:22 PM	Scale 1:300

Additional Details
 Name: a) 2.0H:1V SSM embankment
 Comments:
 Method: Morgenstern-Price, Half-Sine
 Minimum Slip Surface Depth: 1.5 m
 Entry: (19.593, 142.6) m, Exit: (32.607667, 146.9) m
 Center: (19.812381, 163.78154) m, Radius: 21.182673 m

Figure G1-1

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
■	b) Clayey Silt (Drained)	Mohr-Coulomb	17.5	5	27
■	c) Silty Sand (Glacial Till)	Mohr-Coulomb	21	0	35
■	d) Bedrock	Bedrock (Impenetrable)			
■	e) SSM	Mohr-Coulomb	21	0	32

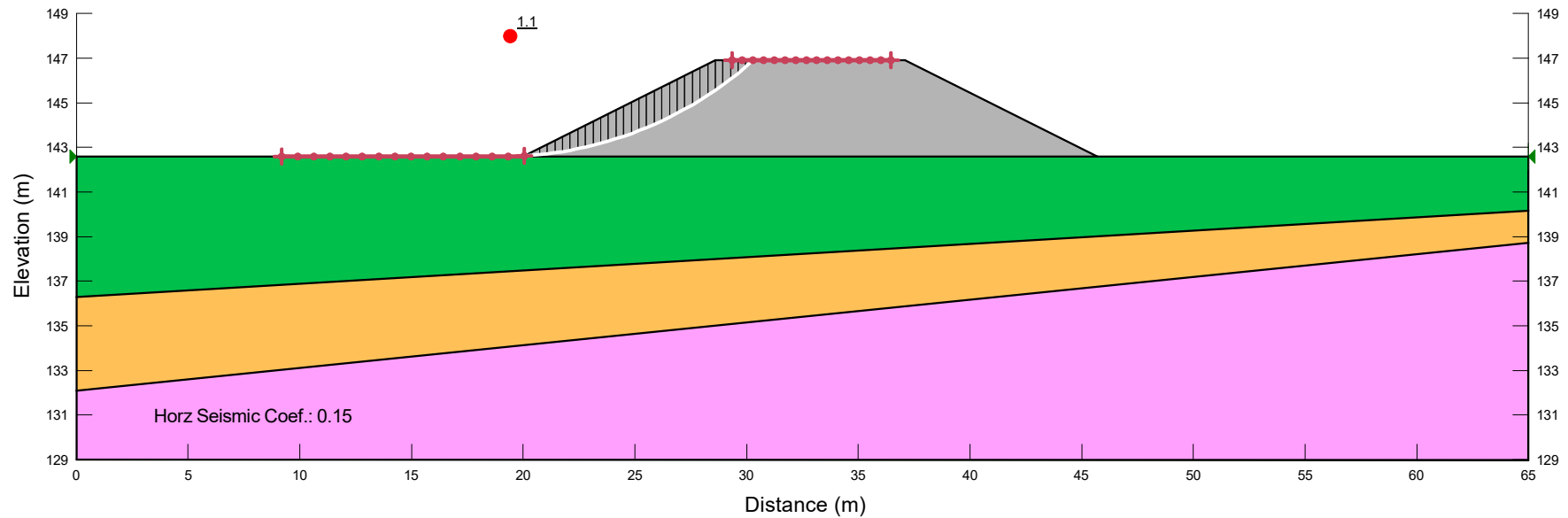


Project 24726 - Hwy 17 Sta 9+890, Culvert 105		
Analysis a2) Permanent, long term, static, drained		
Seismic Coefficient H: g, V: g	Last Run 2024-11-28, 02:35:22 PM	Scale 1:300

Additional Details
 Name: a) 2.0H:1V SSM embankment
 Comments:
 Method: Morgenstern-Price, Half-Sine
 Minimum Slip Surface Depth: 1.5 m
 Entry: (19.893, 142.6) m, Exit: (30.014333, 146.9) m
 Center: (20.104085, 156.16494) m, Radius: 13.56658 m

Figure G1-2





Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Undrained Shear Strength (kPa)
■	a) Clayey Silt (Undrained)	Undrained (Phi=0)	17.5			75
■	c) Silty Sand (Glacial Till)	Mohr-Coulomb	21	0	35	
■	d) Bedrock	Bedrock (Impenetrable)				
■	e) SSM	Mohr-Coulomb	21	0	32	

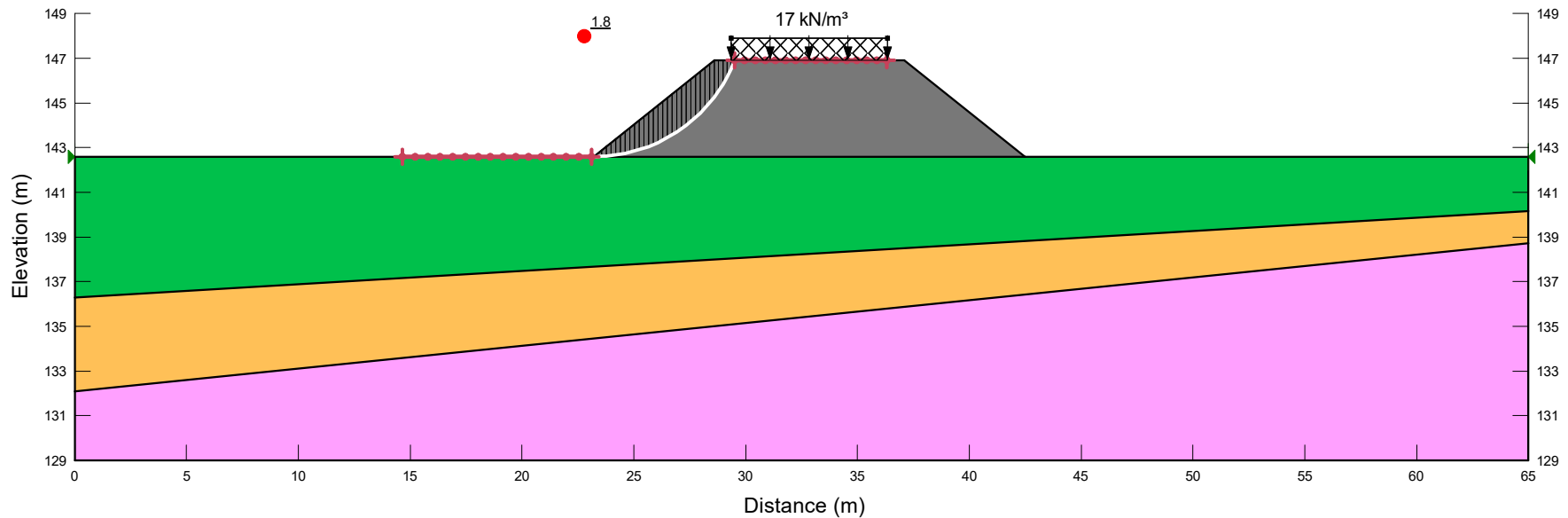


Project 24726 - Hwy 17 Sta 9+890, Culvert 105		
Analysis a3) Temporary (seismic), pseudo-static, undrained		
Seismic Coefficient H: 0.15g, V: g	Last Run 2024-11-28, 02:35:20 PM	Scale 1:300

Additional Details
 Name: a) 2.0H:1V SSM embankment
 Comments:
 Method: Morgenstern-Price, Half-Sine
 Minimum Slip Surface Depth: 1.5 m
 Entry: (20.047699, 142.62385) m, Exit: (30.288044, 146.9) m
 Center: (19.050101, 159.41251) m, Radius: 16.818269 m

Figure G1-3

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Undrained Shear Strength (kPa)
	a) Clayey Silt (Undrained)	Undrained (Phi=0)	17.5			75
	c) Silty Sand (Glacial Till)	Mohr-Coulomb	21	0	35	
	d) Bedrock	Bedrock (Impenetrable)				
	f) Rockfill	Mohr-Coulomb	20	0	42	

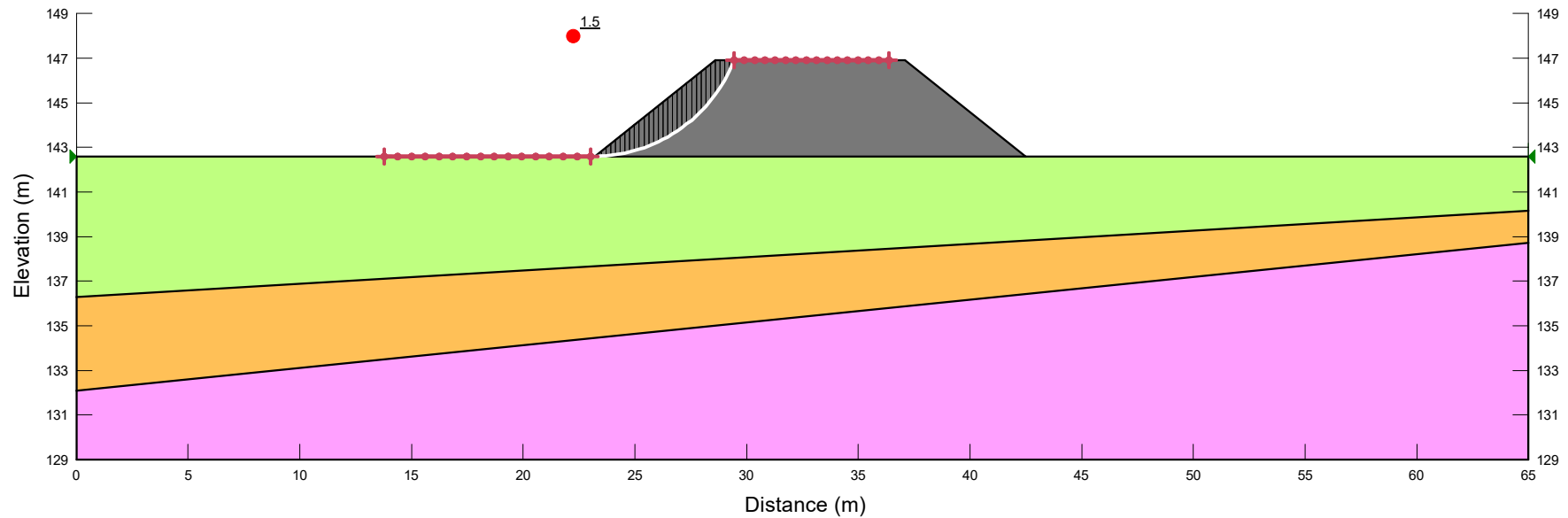


Project 24726 - Hwy 17 Sta 9+890, Culvert 105		
Analysis b1) Temporary (traffic), short term, static, undrained		
Seismic Coefficient H: g, V: g	Last Run 2024-11-28, 02:35:24 PM	Scale 1:300

Additional Details
 Name: b) 1.25H:1V Rockfill embankment
 Comments:
 Method: Morgenstern-Price, Half-Sine
 Minimum Slip Surface Depth: 1.5 m
 Entry: (23.099675, 142.6) m, Exit: (29.5098, 146.9) m
 Center: (23.226752, 149.33844) m, Radius: 6.7396339 m

FigureG2-1

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
■	b) Clayey Silt (Drained)	Mohr-Coulomb	17.5	5	27
■	c) Silty Sand (Glacial Till)	Mohr-Coulomb	21	0	35
■	d) Bedrock	Bedrock (Impenetrable)			
■	f) Rockfill	Mohr-Coulomb	20	0	42




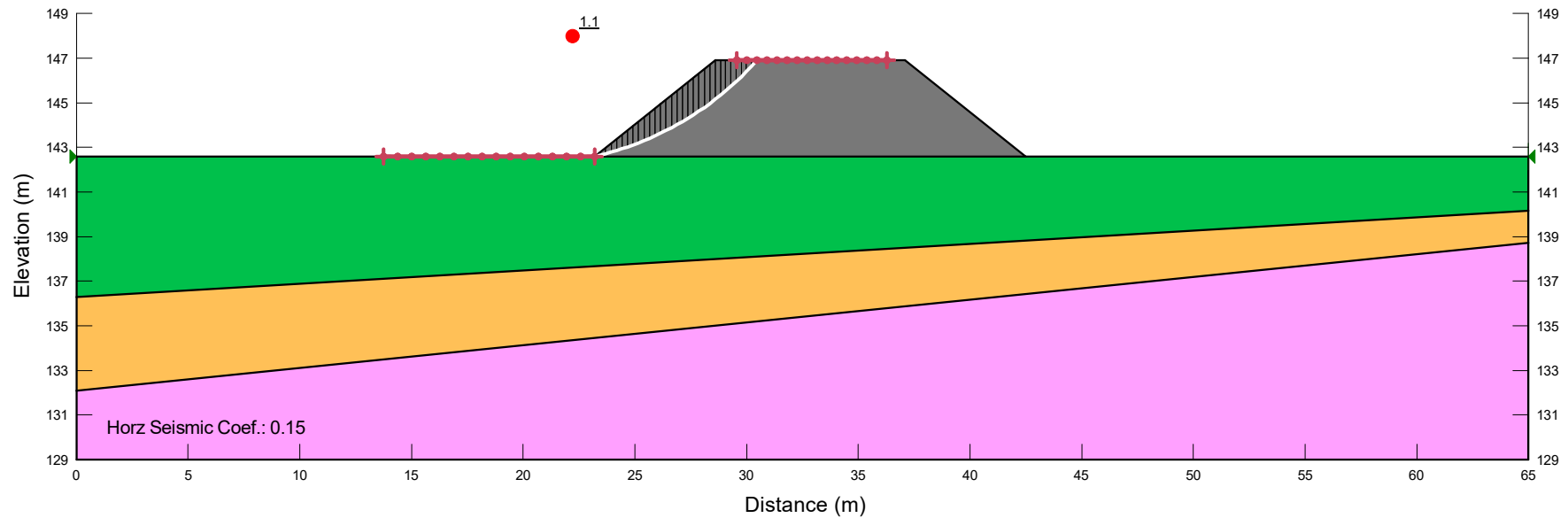
	Project		Additional Details	
	24726 - Hwy 17 Sta 9+890, Culvert 105		Name: b) 1.25H:1V Rockfill embankment	
	Analysis		Comments:	
	b2) Permanent, long term, static, drained		Method: Morgenstern-Price, Half-Sine	
Seismic Coefficient	Last Run	Scale	Minimum Slip Surface Depth: 1.5 m	
H: g, V: g	2024-11-28, 02:35:24 PM	1:300	Entry: (23.022856, 142.6) m, Exit: (29.432981, 146.9) m	
			Center: (23.149933, 149.33844) m, Radius: 6.7396339 m	

Figure G2-2

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Undrained Shear Strength (kPa)
■	a) Clayey Silt (Undrained)	Undrained (Phi=0)	17.5			75
■	c) Silty Sand (Glacial Till)	Mohr-Coulomb	21	0	35	
■	d) Bedrock	Bedrock (Impenetrable)				
■	f) Rockfill	Mohr-Coulomb	20	0	42	



Project 24726 - Hwy 17 Sta 9+890, Culvert 105		
Analysis b3) Temporary (seismic), pseudo-static, undrained		
Seismic Coefficient H: 0.15g, V: g	Last Run 2024-11-28, 02:35:24 PM	Scale 1:300

Additional Details
 Name: b) 1.25H:1V Rockfill embankment
 Comments:
 Method: Morgenstern-Price, Half-Sine
 Minimum Slip Surface Depth: 1.5 m
 Entry: (23.2, 142.6) m, Exit: (30.457805, 146.9) m
 Center: (20.137706, 156.04381) m, Radius: 13.788175 m

Figure G2-3



Appendix H.

List of Referenced Specifications Non-Standard Special Provisions



1. The following Special Provisions and OPS 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 Sheetting
OPSS.PROV 517	Construction Specification for Dewatering and Temporary Flow Passage Systems
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 202.010	Slope Flattening Using Excess Material on Earth or Rock Embankment
OPSD 202.020	Drainage Gap for Slope Flattening on Rock or Granular Embankment
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
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 517F01	Amendment to OPSS 517 - Construction Specification for Dewatering



SP 105S09	Amendment to OPSS 539 - Construction Specification for Temporary Protection Systems
SP 110S06	Amendment to OPSS 1010, April 2013

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 shall consist of OPSS Granular B Type II or Quarry Sourced OPSS Granular A material.

“Notice to Contractor: Obstructions”

Buried obstructions may be encountered during construction and interfere with excavations and installation of temporary protection/dewatering systems. Cobbles and boulders may be encountered within the glacial till layer. The Contractor must be prepared to dislodge or penetrate obstructions. Where obstructions are encountered near the surface, the Contractor may choose to remove such obstructions, provided it does not destabilize the existing embankment or temporary works.

“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.