



THURBER ENGINEERING LTD.

**PRELIMINARY
FOUNDATION INVESTIGATION AND DESIGN REPORT
HIGHWAY 17 TWINNING, RENFREW AREA
CULVERTS 20 AND 20N
STA. 24+936, HORTON TOWNSHIP
WP 4068-09-00 / ASSIGNMENT NO. 4018-E-0009**

Geocres No.: 31F07-006

Report to:

Ministry of Transportation Ontario

Latitude: 45.459436°
Longitude: -76.612878°

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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 Highway 17 culvert crossing located near Station 24+936 in Horton Township within the Renfrew County, Ontario. The existing Highway 17 alignment at this site will become the future Highway 17 eastbound lanes and new westbound lanes will be constructed to the north of the existing alignment at this location. The existing culvert (Culvert 20) will be replaced, and a new culvert (Culvert 20N) is required to convey an unnamed tributary of Deil's Creek below the embankment supporting the proposed Highway 17 westbound lanes.

This section of the report presents the factual findings obtained from the foundation investigation conducted by Thurber as part of the current study.

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

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

2 SITE DESCRIPTION

2.1 General

For project purposes, Highway 17 is herein described as oriented east west, and the existing culvert is described as oriented north south. The culvert crosses Highway 17 at Sta. 24+936



Horton which is approximately 370 m west of the Millenium Trail overpass in Horton Township or, alternatively, 1.3 km east of Lochwinnock Road.

In the area of the culvert, the existing Highway 17 is a two-lane highway and has a posted speed limit of 90 km/h. The road surface near the culvert is at approximate elevation 148.6 m. The shoulders have a total width of approximately 2.7 m in the east- and westbound directions with approximately 0.3 m being paved. Steel cable guide rails on wooden posts are present along both shoulders. The traffic volume for this section of Highway 17 is understood to have been 13,900 AADT in 2016.

The existing culverts under the existing Highway 17 embankment are twin 1,220 mm diameter, 49.7 m long, corrugated steel pipe (CSP) culverts with an approximate skew of 55° to the highway alignment. The culvert has an approximate gradient of approximately 0.6% with the invert of the culvert being near elevations 142.5 and 142.2 m at the inlet and outlet, respectively. The cover above the existing culvert is approximately 5.0 m at the highway centerline. The creek flows through the culvert under the highway embankment from north to south. It is understood that the general drainage is near parallel to the south side of the highway alignment to the west of the culvert. The depth of standing water near the inlet and outlet was measured as 1,130 mm and 730 mm, respectively, on July 26, 2024.

Embankment side slopes, in the vicinity of the culvert, are inclined at approximately 2.4H:1V on the north side and 1.9H:1V on the south side. The existing embankment side slopes at the culvert site did not show any visible signs of global instability at the time of the investigation.

The site is in a rural setting, and the terrain along the ditch line is relatively flat. The area near the culvert is mostly farmland with some deciduous trees and shrubs. Temporary silt fences on wooden posts were located immediately near the culvert inlet. A natural gas pipeline crossing under the Highway 17 is located approximate 100 meters west of the culvert site. Overhead utility lines were not present.

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

2.2 Site Geology

Under the same MTO Assignment a Foundation investigation was conducted by Thurber at several high fill locations within the Highway 17 twinning project boundaries. The available information was reviewed prior to this investigation and can be found in the Geocres Library under Geocres Number 31F-235. Borehole NS21-09 from that investigation is relevant to the present report and has been included in Appendix B.

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

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

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



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.

Ontario Geological Survey Map P.3784³ for Precambrian Geology for the Horton Area suggests that the site is underlain by grey to white calcite, fine to medium grained, impure calcite marble.

3 SITE INVESTIGATION AND FIELD TESTING

Borehole NS21-09 was drilled off-road on November 19, 2021, using a Diedrich 50 track mounted drill equipped with hollow stem augers.

The foundation investigation and field-testing program was augmented between March 26 and April 02, 2024, and consisted of one on-road borehole identified as NSC20-3 and two off-road boreholes identified as NSC20-1 and NSC20-2. The on-road borehole was advanced with a CME 75 truck mounted drill rig utilizing hollow stem augers, NW casing, and coring techniques in bedrock. The off-road boreholes were advanced with a CME 75 track mounted drill rig utilizing hollow stem augers, NW casing, and coring techniques in bedrock.

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) |
|---------------------|---------------------------|-----------------------------|----------------------------|-------------------------------------|------------------------------|
| NSC20-1 | Proposed WB Culvert Inlet | 5 035 503.6 (45.459407°) | 296 054.7 (-76.611823°) | 143.6 | 11.3 |
| NSC20-2 | Proposed WB embankment | 5 035 504.8 (45.459417°) | 296 032.8 (-76.612104°) | 143.3 | 15.3 |

³ <http://www.geologyontario.mndm.gov.on.ca/index.html>



| Borehole No. | Drilled Location | Northing (Latitude) | Easting (Longitude) | Ground Surface Elevation (m) | Termination Depth (m) |
|--------------|-----------------------------|-----------------------------|----------------------------|------------------------------|-----------------------|
| NSC20-3 | Existing Eastbound Shoulder | 5 035 481.0 (45.459436°) | 295 972.3 (-76.612878°) | 148.6 | 16.6 |
| NS21-09 | Existing Inlet | 5 035 507.8 (45.459444°) | 296 012.5 (-76.612363°) | 143.1 | 6.7 |

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. Thin-Walled (Shelby) Tube samples were pushed and retrieved in Borehole NSC20-2 to obtain relatively undisturbed cohesive soil samples for further laboratory testing.

A 50 mm diameter monitoring well was installed in each of Boreholes NSC20-1 and NS21-09 to allow for measurements of the groundwater level after drilling. The details of the well installations are illustrated on the respective Record of Borehole sheets provided in Appendix B. The monitoring wells installed as part of the current investigation will be decommissioned by Thurber, as outlined in the Hydrogeological Investigation and Design Report.

Boreholes NSC20-2 and NSC20-3 were backfilled in accordance with MOE requirements (O.Reg 903, as amended).

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.

One-dimensional consolidation testing (ASTM D 2435) was carried out on one relatively undisturbed cohesive sample from Borehole NSC20-2.

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 descriptions is in accordance with 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 topsoil and organic silt over a native deposit of silty clay to clayey silt underlain by silty sand with gravel over bedrock. Sand to silty sand fill was encountered at ground surface in the on-road borehole.

5.1 Embankment Fill

5.1.1 Sand to Silty Sand Fill

A fill layer consisting of sand to silty sand with varying amounts of gravel was encountered at the ground surface in Borehole NSC20-3. The thickness of the layer was 6.1 m (base elev. at 142.5 m). The SPT N values ranged from 11 to 60 blows, indicating a compact to very dense relative density.

The moisture content of the samples tested ranged from 3 to 12%. The results of grain size analyses conducted on two samples of this fill material are summarized in the table below and are illustrated on Figure C1 in Appendix C.

Summary of Grain Size Distribution Testing – Sand to Sandy Silt Fill

| Soil Particle | Percentage (%) |
|----------------------|-----------------------|
| Gravel | 14 – 17 |
| Sand | 62 – 75 |
| Silt & Clay | 11 – 21 |

5.2 Topsoil

A 75 mm thick layer of topsoil was encountered at the ground surface in Borehole NS21-09.

5.3 Organic Silt (MI-OI)

A native deposit of organic silt was encountered at the ground surface in Boreholes NSC20-1 and NSC20-2. Varying amounts of sand and peat inclusions were noted within the layer. The thickness of the layer ranged from 0.4 to 1.0 m (base elev. 143.2 to 142.3 m). The SPT N-values recorded were 1 and 2 blows, indicating a very loose relative density.

The moisture content of the samples tested ranged from 37 to 50%. The results of grain size analysis conducted on one sample of this layer are summarized in the table below and are illustrated on Figure C2 in Appendix C.

Summary of Grain Size Distribution Testing – Organic Silt

| Soil Particle | Percentage (%) |
|---------------|----------------|
| Gravel | 0 |
| Sand | 35 |
| Silt | 46 |
| Clay | 19 |

The results of Atterberg Limits testing carried out on one sample of this material are summarized below and are illustrated on Figure C3 in Appendix C. The laboratory results indicate that the organic silt is of intermediate plasticity (MI-OI).

Summary of Atterberg Limit Testing – Organic Silt

| Parameter | Value |
|------------------|-------|
| Liquid Limit | 40 |
| Plastic Limit | 27 |
| Plasticity Index | 13 |

5.4 Silty Clay (CI) to Clayey Silt (CL)

A native deposit of silty clay to clayey silt was encountered below the organic silt in Boreholes NSC20-1 and NSC20-2, below the sand to silty sand fill in Borehole NSC20-3, and below the topsoil in Borehole NS21-09. Sand partings and seams were encountered throughout the layer. Where fully penetrated, the thickness of the layer ranged from 2.8 to 6.3 m (base elev. 140.4 to 136.0 m). The layer was not fully penetrated in Borehole NS21-09 but was proven to extend to a depth of 6.7 m (elev. 136.4 m).

Where SPT was conducted within the layer, the N-values typically ranged from 4 to 12 blows. Field vane tests were performed within this layer where possible. Undrained shear strengths were obtained and ranged from 75 to greater than 100 kPa. Remolded vane tests recorded sensitivities typically ranging from 5 to 7, indicating a sensitive material (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 20 to 43% but were typically greater than 30%. The results of grain size analysis tests conducted on eight samples of this material are summarized in the table below and are illustrated on Figures C4 and C5 in Appendix C.

Summary of Grain Size Distribution Testing – Silty Clay to Clayey Silt

| Soil Particle | Percentage (%) |
|---------------|----------------|
| Gravel | 0 |
| Sand | 1 – 21 |
| Silt | 53 – 67 |
| Clay | 26 – 38 |

The results of Atterberg Limits testing carried out on eight samples of this material are summarized below and are illustrated on Figure C6 and C7 in Appendix C. The laboratory results indicate that the silty clay to clayey silt is of intermediate to low plasticity (CI to CL).

Summary of Atterberg Limit Testing – Silty Clay to Clayey Silt

| Parameter | Value |
|------------------|---------|
| Liquid Limit | 30 – 39 |
| Plastic Limit | 18 – 25 |
| Plasticity Index | 10 – 17 |

One-dimensional consolidation testing (ASTM D 2435) was carried out on one relatively undisturbed cohesive sample from Borehole NSC20-2. Load increments were maintained for 24 hours. Photographs of the extruded sample are provided in Appendix C. The testing results are presented in Appendix C and are summarized in Table 3-1. The preconsolidation stress summarized in the table was obtained from the end-of-increment void ratios. It should be expected that compressibility characteristics will vary with depth in accordance with the soil index parameters and stress history.

Table 5-1: Advanced Laboratory Test Results

| | |
|---|------------------|
| Borehole | NSC20-2 |
| Sample | TW7 |
| Sample Depth (m) | 4.6 – 5.2 |
| Sample Elevation (m) | 138.4 |
| Soil Layer | Clayey Silt (CL) |
| Load Increment Duration (hrs.) | 24 |
| Moisture Content (%) | 34 |
| Liquidity Index (-) | 1 |
| Initial Void Ratio, e_0 (-) | 0.96 |
| Moist Unit Weight (kN/m^3) | 18.6 |
| In-situ Vertical Effective Stress (kPa) | 50 |
| Preconsolidation Stress, P'_c (kPa) | 246 |
| Overconsolidation Ratio (-) | 5 |
| Recompression Index, C_r (-) | 0.03 |
| Compression Index, C_c (-) | 0.46 |
| Coefficient of Reconsolidation, C_{vr} (cm^2/sec) | 0.007 |
| Coefficient of Consolidation, C_v (cm^2/sec) | 0.002 |

5.5 Silty Sand (SM)

A deposit of silty sand with gravel was encountered beneath the silty clay to clayey silt in Boreholes NSC20-1 through NSC20-3. The thickness of the layer ranged from 0.5 to 2.0 m (base elev. 138.8 to 135.5 m). The SPT N-values ranged from 11 to 65 blows, indicating a compact to very dense relative density. A refusal blow count was encountered at the base of the layer in Borehole NSC20-2 but is attributed to the bedrock surface.

The moisture content of the samples tested ranged from 6 to 16%. The results of gradation analyses completed on two samples of the layer are illustrated in Figure C8 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 | 36 – 39 |
| Sand | 43 – 47 |
| Silt & Clay | 17 – 18 |

5.6 Bedrock

Bedrock was proven by coring in Boreholes NSC20-1, NSC20-2, and NSC20-3. The bedrock surface sloped downwards from north to south with depths ranging from 4.8 to 12.7 m (elevation 138.8 to 135.5 m).

The bedrock encountered consisted of completely to slightly weathered, coarse grained, white to light grey, medium strong to strong marble. Photographs of the bedrock cores are provided in Appendix C. The rock core quality measurements are summarized in Table 5-2.

Table 5-2: Bedrock Details

| Parameter | Range |
|---|----------|
| Total Core Recovery (TCR), % | 15 – 100 |
| Solid Core Recovery (SCR), % | 5 – 100 |
| Rock Quality Designation (RQD), % | 0 – 100 |
| Fracture Index (fractures per 0.3 m) ⁽¹⁾ | 0 – >10 |
| Unconfined Compressive Strength (MPa) | 54 – 65 |

Notes: (1) Indicated as "FI" on Borehole Logs

The RQD values ranged from 0 to 100% but were typically less than 37%, indicating a bedrock of very poor to poor quality (CFEM, 2023). The fracture index was typically greater than 10 fractures per 0.3 m. The results of unconfined compressive strength tests (UCS) were 54 to 65 MPa, indicating that the tested samples of the bedrock are strong (CFEM, 2023). The UCS test results are included in Appendix C.

5.7 Groundwater

Monitoring wells with a diameter of 50 mm were installed in Boreholes NSC20-1 and NS21-09. The recorded groundwater levels are presented in Table 5-3.

Table 5-3: Summary of Groundwater Levels

| Borehole No. | Bottom of Screen Elevation (m) | Groundwater Depth (m) | Groundwater Elevation (m) | Date of Measurement |
|--------------|--------------------------------|-----------------------|---------------------------|---------------------|
| NSC20-1 | 139.6 | 0.3 | 143.3 | April 09, 2024 |
| | | 0.4 | 143.2 | May 01, 2024 |
| | | 0.5 | 143.1 | June 07, 2024 |
| | | 0.5 | 143.1 | June 28, 2024 |
| | | 0.5 | 143.1 | July 12, 2024 |
| | | 0.5 | 143.1 | August 29, 2024 |
| NS21-09 | 137.0 | 0.3 | 142.8 | November 29, 2021 |
| | | 0.5 | 142.6 | December 02, 2021 |
| | | 0.5 | 142.6 | December 13, 2021 |
| | | 0.5 | 142.6 | January 21, 2022 |
| | | 0.4 ^(a) | 142.7 | May 02, 2024 |
| | | 0.5 ^(a) | 142.6 | August 29, 2024 |

Notes: (a) water level taken after borehole log was finalized

The elevation of the ponded water surface in the creek was at approximately 143.6 m and 142.9 m near the culvert inlet and outlet, respectively on July 26, 2024; water depths ranged from 0.7 to 1.1 m.

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 and seasonal fluctuations of the groundwater level are to be expected. In particular, the creek water and groundwater levels may be at a higher elevation after periods of significant and/or prolonged precipitation.

5.8 Analytical Testing

One sample of the native organic 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 5-4. Copies of the test results are provided in Appendix C.

Table 5-4: Results of Chemical Analysis

| Borehole | Sample | Depth (m) | Chloride (µg/g) | Sulphate (µg/g) | Sulphide (%) | pH (-) | Resistivity (Ohm-cm) |
|----------|--------|-----------|-----------------|-----------------|--------------|--------|----------------------|
| NSC20-2 | SS2A | 0.8 – 1.0 | 37 | 21 | < 0.01 | 6.65 | 4,920 |

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 rigs used to drill, test, sample, and decommission the boreholes. Traffic control was performed in accordance with Ontario Book 7 and was provided by C&C Services of Renfrew, Ontario. The field investigation was supervised on a full-time basis by Mr. B. Coote, EIT, and Mr. D. Amorim Pereira, 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 D. Amorim Pereira, Geotechnical Technician, and A. de Oliveira, P.Eng. The report was reviewed by Dr. F. Griffiths, P.Eng., and Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundation Projects.

Thurber Engineering Ltd.
Report Prepared By:

Darlan Amorim Pereira, M.Sc.
Geotechnical Technician



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



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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 a culvert replacement to be located at approximate Station 24+936 on existing Highway 17 Horton Township, Renfrew County and a new culvert crossing located at approximate Station 24+975 on the proposed new westbound lanes of Highway 17. 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 works. The discussion and preliminary recommendations presented in this report are based on information provided by the MTO and the factual data obtained during the current field investigation.

7.1 Background Information

The site is located on Highway 17 approximately 1.3 km east of Lochwinnock Road in Horton Township, Renfrew County. For project purposes, Highway 17 is herein described as oriented east-west and the creek flows from north to south.



The existing culverts under the existing Highway 17 embankment are twin 1,220 mm diameter, 49.7 m long, corrugated steel pipe (CSP) culverts with an approximate skew of 55° to the highway alignment. The invert of the culverts is near elevations 142.5 and 142.2 m at the inlet and outlet, respectively. The road surface near the culvert is at approximate elevation 148.6 m. The ground surface elevation outside the highway embankment footprint ranges from 143.6 m to 143.1 m near the proposed alignment.

The encountered stratigraphy at this site consists of topsoil and organic silt over a native deposit of stiff to very stiff silty clay to clayey silt underlain by silty sand over bedrock. Sand to silty sand fill was encountered below the ground surface in the on-road borehole. It is noted that the ground water level in the monitoring well installed in Borehole NSC20-1 was at elevation 143.1 m on June 28, 2024.

It is noted that a Foundation Investigation and Design Report was previously prepared for several proposed high embankment fills for the overall Highway 17 twinning assignment as documented in Geocres Number 31F-235. The current site is located within High Fill Area G, which encompasses the new westbound alignment from 24+675 to 25+575, Horton Township.

7.2 Proposed Works

The existing Highway 17 alignment at this site will become the future Highway 17 eastbound lanes; new westbound lanes will be constructed approximately 45 m north (centreline to centreline) of the existing alignment at this location. The twin culverts currently present under the existing Highway 17 lanes will require replacement and a new culvert will be required under the proposed westbound lanes. These culverts will convey a tributary of Deil's Creek under the existing and proposed highway embankments.

The Structure and Culvert List of February 23, 2022, for this project indicated that the proposed culvert replacement (Culvert 20) beneath the new eastbound lanes is to be a non-structural, pre-cast, closed-bottom concrete box culvert (CBC) with a length of 39.5 m, a span of 2.5 m, a rise of 1.83 m, and a 1.05% slope. It is assumed that the invert of the replacement culvert will be near elevations 142.5 and 142.2 m at the inlet and outlet to match current conditions.

The proposed culvert (Culvert 20N) to be constructed for the westbound lanes will also be a non-structural, pre-cast, closed-bottom concrete box culvert (CBC) and will have a length of 37.6 m, a span of 2.5 m, a rise of 1.83 m, and a 1.05% slope. It is assumed that the invert of the new westbound embankment culvert will be near elevations 142.8 and 142.5 m at the inlet and outlet. As per the preliminary MTO drawings, the proposed final grade at Station 24+975 of the new westbound embankment is approximate elevation 151.0 m. The proposed embankment has a height of approximately 7.7 m above the original ground surface.

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



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

7.3 Applicable Codes and Design Considerations

The geotechnical assessment presented herein has been prepared based on the available data regarding the proposed work, existing ground conditions document in Part 1 of this report, and in accordance with the Canadian Highway Bridge Design Code (CHBDC), version CSA S6-19. It is noted that the proposed culverts are non-structural; however, the embankments are considered high fills.

In accordance with the CHBDC, the analysis and design take into consideration the importance 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 works are being designed as a “Major Route” importance category. As per Section 6.14.2.1.b and 6.14.2.3.b of the CHBDC, a Major-Route geotechnical system is required to have a seismic performance criterion that meets a return period of 475-years.

It is understood that the works have been assigned a Typical Consequence Classification, in accordance with Section 6.5.1 of the CHBDC. Accordingly, a consequence factor (Ψ) of 1.0, as per Table 6.1 of the CHBDC, has been used in assessing factored geotechnical resistances. If the consequence classification changes, the geotechnical assessment and recommendations provided within this report may need to be reviewed and revised.

The degree of site and prediction model understanding for this site has been assessed to be typical understanding (Section 6.5.3 of CHBDC).

The frost penetration depth and associated recommendations are provided in Section 10.5.

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 0.226 g. Similarly, the PGA value at this site provided by GSC for a reference Site Class C with a 10% probability of

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



exceedance in 50 years (475-year event) is 0.075 g. These values are to be scaled by $F(PGA)$ based on the *site-specific* Site Class, as discussed in Section 8.3.

8.2 Seismic Liquefaction Potential

Based on the assessment using the SPT data following the simplified method for cohesionless soil as outlined in Boulanger and Idriss (2014)⁵, the soils are considered not 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 first 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 1.14 for this site yielding a scaled *site-specific* Site Class D PGA of 0.257 g for a seismic event with a 2% probability of exceedance in 50 years (2475-year event). Similarly, the $F(PGA)$ is 1.29 and the Site Class D PGA is 0.097 g for a seismic event with a 10% probability of exceedance in 50 years (475-year event).

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 1.12 and 1.42 for this site, yielding a scaled *site-specific* $S_a(0.2)$ of 0.393 and $S_a(1.0)$ of 0.139. A Seismic Performance Category of 3 is applicable to this site based on Table 4.10 of the CHBDC. It is noted that the proposed culverts are non-structural; however, the embankments are considered high fills. Seismic design of the embankment is discussed in Section 10.8.

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.

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. 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 open-bottom culverts will have greater construction concerns due to the shallow water table and requirement for greater excavation depths to construct the culvert footings to satisfy frost depth requirements. It is anticipated that the underside of the footings for the replacement culvert would be as deep as approximately elevation 140.3 m which is 2.8 m below the observed groundwater level on June 28, 2024, and approximately 8.3 m below the existing road surface. The use of open bottom culverts 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 culverts, and allow for more manageable dewatering efforts.

A comparison of these alternatives, based on their respective advantages and disadvantages, is included in Appendix F. It is not considered to be economical or practical to support a culvert on deep foundations at this site and therefore this option is not presented in this report.

9.2 Construction Methodology Alternatives

At the time of the field investigation, ponded water was observed at elevations of approximately 143.6 m and 142.9 m near the existing culvert inlet and outlet at respectively. Water depths ranged from 0.7 to 1.1 m. The groundwater level was measured to be at approximate elevation 143.1 m in the monitoring well installed in Borehole NSC20-1 on June 28, 2024. 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 culvert in the dry (See Section 11.3).

It is noted that the preliminary profile for the new westbound lanes includes an embankment approximately 7.7 m above existing grade at Culvert 20N. Mitigation of the settlement induced by the new westbound embankment will require a culvert designed to accommodate the movements. Embankment design recommendations were developed as part of Geocres Report 31F-235 and are provided in Section 10.7.

At the time of preparation of this report, a construction staging plan has not yet been developed. The foundation recommendations presented herein have been prepared based on the assumption that construction of the new culvert (Culvert 20N) and the new westbound embankment will be carried out while traffic remains on the existing alignment. Upon completion of the construction of the new lanes, all traffic would be temporarily directed onto those new lanes to allow culvert replacement (Culvert 20) for the eastbound lanes under a road closure of the existing alignment.

9.3 Recommended Approach

From a geotechnical perspective, closed-bottom, box culverts are recommended at this site. It is anticipated that construction for the new westbound lanes would be carried out while traffic remains on the existing alignment. Once the new lanes are open, all traffic would be rerouted onto the new lanes, while the culvert under the existing lanes is replaced.

Pipe culverts would also be considered a feasible alternative. Construction staging would be similar to that for the closed bottom box culvert option.

Mitigation of the settlement induced by the new westbound embankment will require a culvert designed to accommodate the movements (see Section 10.7.3).

10 PRELIMINARY FOUNDATION DESIGN RECOMMENDATIONS

From a foundation engineering perspective, pre-cast concrete box culverts are recommended. The following bullets summarize the relevant elevations near each culvert:

Existing Highway 17 / Proposed Eastbound Lanes (Culvert 20)

- | | |
|---|------------------|
| • Existing top of pavement | 148.6 m |
| • Culvert invert at outlet | 142.2 m |
| • Groundwater elevation June 28, 2024 | 143.1 m |
| • Underside elevation of Silty Clay/Clayey Silt | 137.9 to 136.0 m |
| • Bedrock surface elevation | 135.9 to 135.5 m |

Proposed Highway 17 Westbound Lanes (Culvert 20N)

- | | |
|----------------------------|---------|
| • Top of pavement | 151.0 m |
| • Culvert invert at outlet | 142.5 m |



- Groundwater elevation June 28, 2024 143.1 m
- Underside elevation of Silty Clay/Clayey Silt 140.4 to 136.0 m
- Bedrock surface elevation 138.8 to 135.5 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 10.3). Subgrade preparation should follow the recommendations provided in Section 10.3 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 10.3) should be evaluated based on the recommendations in Section 10.3.

10.2 Precast Closed Box Concrete Culvert

It is anticipated that the subgrade soils beneath the culvert footprint of the westbound lanes will be subjected to the additional loads from the proposed embankment with a height of approximately 7.7 m above existing grades. Further discussion on the potential settlement of the underlying soils is provided in Section 10.7.3. The subgrade should be prepared as described in Section 10.3.

The recommended geotechnical resistances for 3.1 m wide (outside) pre-cast, closed-bottom, box culverts (both Culverts 20 and 20N) with the culvert base at or below approximate elevation 142.5 m, installed on a bedding layer prepared as described in Section 10.3, and placed on an undisturbed native clayey silt subgrade are as follows:

- Factored Geotechnical Resistance at ULS of 225 kPa
- Factored Geotechnical Resistance at SLS of 150 kPa (provided settlement mitigation is included for Culvert 20N – see Section 10.7.3)

The factored geotechnical resistances include the following factors:

- Consequence factor (Ψ) of 1.0 (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 no greater than 50 mm for Culvert 20N, see further discussion in Section 10.7.3.

Resistance to lateral forces/sliding resistance between the precast concrete and underlying granular a bedding (Section 10.3) should be evaluated in accordance with the CHBDC assuming an unfactored coefficient of friction of 0.45. 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. An unfactored coefficient of friction of 0.35 can be assumed for the interface between the Granular 'A' and the clayey silt. A resistance factor of 0.6 (as per CHBDC Table 6.2) should be used to estimate the sliding resistance between the Granular A and the clayey silt subgrade.

Surface water diversion and dewatering will be required to place the bedding material and install the culvert in the dry (Section 11.3).

10.3 Subgrade Preparation, Bedding and Backfilling

"Granular A" in this section refer to OPSS Granular A 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).

Subgrade preparation for the replacement of Culvert 20 should include excavation and removal of the existing culvert if replaced along the same alignment. If the replacement culvert is placed on a new alignment, the existing culvert may be decommissioned in place (see Section 10.7.3 for further details).

Topsoil and organic silt with a thickness of approximately 0.4 to 1.0 m was encountered in the area of the proposed Culvert 20N during the drilling investigation and must be removed from beneath the culverts and embankment footprints.

For a pipe culvert, at the founding level existing fill, soft/loose soils (including topsoil and organic silt), disturbed soils, or otherwise deleterious materials encountered will need to be removed down to competent inorganic soils. Granular A should be used in dewatered excavations to backfill any sub-excavations required for subgrade improvement. Foundation preparation for a pipe culvert should be as per OPSD 802.031 and OPSD 803.031 with bedding extending to at least 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 clayey silt 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 culvert should be placed on a granular pad with a minimum thickness of 0.3 m consisting of Granular A material. The top of the Granular A pad must extend to 0.5 m beyond the outside edge of all sides of the culvert and sloped away from the footing at 1H:1V, or flatter. The granular bedding shall be compacted as per OPSS.PROV 501.

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 clayey silt 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 and the excavation should be backfilled to the top of the bedding elevation to protect the subgrade from disturbance from both construction traffic and weather. Alternatively, the geotextile and granular pad could be replaced with a 200 mm thick, concrete working slab placed on the prepared subgrade. The dewatering system must be designed to prevent groundwater from being trapped beneath the working slab and must be operational prior to pouring the slab. 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.

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

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

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

10.4 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 within this area provided the effective vertical pressure on the material is less than 150 kPa (Geocres Memorandum 31F-213). An Operational Constraint will be required in the contract restricting the source of Granular A to 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 as generally shown on OPSD 3101.150. 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 OPSD 3101.150.

Lateral earth pressure parameters provided in Table 10-1 and Table 10-2 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.

10.4.1 Static Lateral Earth Pressure

Lateral earth pressures 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_o 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 | Quarry Sourced OPSS Granular A | Quarry Sourced OPSS Granular B Type II |
|---|---|---|---|
| | $\phi = 35^\circ, \gamma = 22.8 \text{ kN/m}^3$ | $\phi = 40^\circ, \gamma = 22.8 \text{ kN/m}^3$ | $\phi = 42^\circ, \gamma = 22.8 \text{ kN/m}^3$ |
| Coefficient of at Rest Earth Pressure, K_o (Restrained Wall) | 0.43 | 0.36 | 0.33 |
| Coefficient of Active Earth Pressure, K_A (Unrestrained Wall) | 0.27 | 0.22 | 0.20 |

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.

10.4.2 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:

- $k_h = \frac{1}{2} * F(\text{PGA}) * \text{PGA}$, for structures that allow 25 to 50 mm of movement, and
- $k_h = F(\text{PGA}) * \text{PGA}$, 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 | Quarry Sourced OPSS Granular A | Quarry Sourced OPSS Granular B Type II |
|--|---|---|---|
| | $\phi = 35^\circ, \gamma = 22.8 \text{ kN/m}^3$ | $\phi = 40^\circ, \gamma = 22.8 \text{ kN/m}^3$ | $\phi = 42^\circ, \gamma = 22.8 \text{ kN/m}^3$ |
| 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_o 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) |

10.5 Frost Penetration Depth

The depth of frost penetration at this site is estimated to be 1.9 m (as per OPSD 3090.101); shallow foundations, if any, should be founded at or below this depth or provided with equivalent insulation. Closed-bottom box culverts are not typically provided with frost protection. The earth cover should be measured perpendicular to the ground surface. Thermally equivalent frost protection could be in the form of insulation provided it is placed *above* the high-water level. It should be noted that open graded materials, such as rock protection, do not have the same thermal protection as soils.

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

10.6 Cement Type and Corrosion Potential

Chemical analysis for determination of pH, water soluble sulphate, sulphides, chloride concentrations, resistivity and electrical conductivity was carried out on samples of the native materials. The analysis results are summarized in Section 5.8 and a copy of the test results is provided in Appendix C.

The pH, resistivity, and chloride concentration provide a preliminary indication of the degree of corrosiveness of the sub-surface environment. The test results provided in Section 5.8 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 5.8 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 provided in Section 5.8 were compared with Table 3 of Canadian Standards Association Standards A23.1-19 (CSA A23.1) and indicate a low degree of sulphate attack potential on concrete structures at this site.

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

Additional testing is recommended during a subsequent design stage.

10.7 Embankments

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

10.7.1 Eastbound Embankment Reinstatement

The existing highway embankment side slopes are inclined at approximately 2.4H:1V on the north side and 1.9H:1V on the south side. The existing slopes did not show any visible signs of global instability at the time of the investigation.

It is understood that no grade raise or embankment widening is anticipated along the Highway 17 alignment.

Embankment reinstatement after construction of the replacement culvert should be carried out in accordance with OPSS.PROV 206. If constructed using Select Subgrade Material (SSM) or Granular B Type I, the embankment should be constructed with side slopes of 2H:1V (or flatter). The granular fill should be placed and compacted in accordance with OPSS.PROV 501.

Where newly placed embankment fill is placed against existing embankment slopes or on a sloping ground surface steeper than 3H:1V, benching of the existing slope should be carried out in accordance with OPSD 208.010.

10.7.2 Embankment Stability

Eastbound – Culvert 20

As part of the current report, embankment stability has been assessed perpendicular to the roadway alignment for the reinstatement of the eastbound embankment at Station 24+936. 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 analysis 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.
- Maximum fill height of 6.4 m for the eastbound embankment reinstatement.
- Eastbound embankment reinstatement with 2H:1V slopes using Select Subgrade Material.
- A site adjusted PGA value for ground motions with a return period of at least 475 years of 0.049 g, equal to ½ of the site adjusted PGA value (0.097 g), was used for seismic analysis, as per Sections 4.4.3.3 and 6.14.2.3 of the CHBDC and outlined in Section 8.2.
- 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 reinstatement:

Table 10-3: Slope Stability Analysis Results for Eastbound Embankment, Sta. 24+936

| Condition | Case | Factor of Safety |
|-----------------------------------|--------------------------------------|-----------------------------|
| | | 2H:1V [SSM/Granular B I] |
| Temporary (traffic loading) | Short Term (Undrained) | 2.0 (Fig G1-1) |
| Permanent (no traffic loading) | Long Term (Drained) | 1.6 (Fig G1-2) |
| Temporary (includes seismic) | Pseudo-Static Seismic (Undrained) | 1.8 (Fig G1-3) |

The geotechnical resistance factors provided in Table 6.2 of the CHBDC for embankment fills with a typical degree of understanding and a consequence factor (Ψ) of 1.0 generates minimum target Factors of Safety of 1.5 and 1.3 for permanent and temporary conditions, respectively. All the static results presented in Table 10-3 meet or exceed the target Factors of Safety. The pseudo-static results, presented in Tables Table 10-3, also meet the target Factor of Safety for seismic design.

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 1.0, a target Factor of Safety of 1.1 is considered appropriate for the pseudo-static seismic analysis. However, it is noted that some displacement of the embankment can occur where the pseudo-static Factor of Safety is less than 1.3. As the culvert proposed is non-structural, there is no “bridge approach embankment interface zone” as defined in Section 6.14.2.2 of the CHBDC. Section 6.14.2.3 of the CHBDC describes the seismic performance



criteria for geotechnical systems outside this zone for major routes as: at least 50% of the travelled lanes are available for use following ground motions with a return period of at least 475 years.

Westbound – Culvert 20N

As noted in Section 7.3, a Foundation Investigation and Design Report was previously prepared for several proposed high embankment fills for the overall Highway 17 twinning assignment as documented in Geocres Report 31F-235. The current site is located within High Fill Area G, which encompasses the new westbound alignment from 24+675 to 25+575, Horton Township. Stability analyses were completed for the new westbound embankment and are presented in that report. The analyses assessed a maximum embankment height of 8.1 m over existing grade which will occur at Station 25+330. All of the static and pseudo-static cases generated acceptable factors of safety.

It was recommended that the embankment should be constructed with side slopes of 2H:1V (or flatter) if constructed using Granular B Type I or Select Subgrade Material (SSM) meeting the requirement of OPSS.PROV 1010. Alternatively, the embankments could be constructed of rock fill with slopes at 1.25H:1V. Further, mid-height berms comprising 2 m wide benches should be incorporated along the length of embankments with heights at or exceeding 8 m of granular fill. Similarly for rock fill embankments, mid-height berms comprising 2 m wide benches should be incorporated along the length of embankments with heights at or exceeding 10 m.

10.7.3 Embankment Settlement

Embankments must be overbuilt to compensate for the estimated settlements.

It is noted that the addition of a widened platform to accommodate future grade raises has not been included in the design assumptions. Similarly, the placement of slope flattening material on rock fill slopes has not been included in the analyses. Inclusion of these modifications to the cross-sections will affect the settlement magnitudes presented herein.

Eastbound – Culvert 20

The reinstated eastbound embankment will have a similar height and footprint to the existing. The proposed opening for the Culvert 20 replacement is greater than the existing, thus, the construction represents a net unloading. No additional settlement is expected along the existing alignment. If the existing culvert is to be decommissioned by filling with grout or removed and backfilled, it is estimated that this would induce further settlement of less than 10 mm beneath the existing culvert alignment as a result of the increased load imposed by the grout/fill. Settlement should be reviewed if the embankment is widened or reinstated to design grades greater than the existing grades.

Self-settlement of the 6.4 m high embankment required to reinstate the eastbound lanes after installation of Culvert 20 will also occur. For an embankment constructed of compacted SSM material, approximately 35 mm of self settlement will occur with the majority of that complete during construction.

No special mitigation measures for settlement are anticipated for the replacement of Culvert 20.

Settlement of the eastbound lanes due to the construction of the new westbound embankment is expected to be negligible.

Westbound – Culvert 20N

Settlement of the proposed embankment to support the westbound lanes was assessed as part of Geocres Report 31F-235. The current site is located within High Fill Area G which encompasses the new westbound alignment from 24+675 to 25+575, Horton Township. Please refer to the previous report for details on the soil parameters used in the settlement analyses. The calculated settlements for High Fill Area G are presented in the Table 10-4 and Table 10-5.

Table 10-4: Predicted Settlement of Underlying Soil

| Site | Height of Embankment (m) | Thickness of Compressible Clay (m) | Cumulative Settlement (Rock Fill/SSM Fill, mm) | | | | |
|------|--------------------------|------------------------------------|--|----------|-----------|-----------|----------|
| | | | 3 Months | 6 Months | 12 Months | 24 Months | 20 Years |
| G | 6.8 | 9.8 | 40/50 | 40/50 | 40/50 | 40/50 | 40/50 |

Table 10-5: Predicted Embankment Fill Compression

| Site | Fill Height (m) | Compression (mm) | |
|------|-----------------|------------------------------------|--------------------------------|
| | | Compacted Granular Fill Embankment | Compacted Rock Fill Embankment |
| G | 7.7 | 40 | 115 |

The maximum amount of settlement in the underlying soils will typically occur at the location where the compressible soil deposit is thickest. The maximum self-settlement of fill will occur where the embankment fill height is greatest. The two locations as well as the corresponding height of fill may not be synchronous.

The estimated settlement of soils underlying the new westbound embankment exceeds the typical SLS limit of 25 mm for a structural foundation. In this case, it is recommended that Culvert 20N be designed to accommodate this additional movement either by oversizing the opening or by including a camber.

In accordance with MTO's document "Embankment Settlement Criteria for Design" (March 2, 2010), the relevant criterion for the embankment is that the settlement of an embankment should be limited to less than 100 mm for a 20 year post construction period. Full height preloads could be applied for a duration of 6 months to facilitate that post-construction settlement meets the guidelines. It is recommended that final grading of the pavement base layer and placement of asphalt be delayed for six months for both the eastbound embankment reinstatement and the westbound embankment construction. Note that a settlement monitoring program should be carried out to confirm the duration of the delay.

In addition to the settlement described above, there will also be self-settlement of the 8.5 m high eastbound embankment material itself. For embankments constructed with compacted rockfill the short term settlement will be approximately 65 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 10 mm. Similarly, an embankment constructed of compacted SSM material will undergo approximately 45 mm of self settlement with the majority of that complete during construction. The placement of asphalt should be delayed for several months after installation of Culvert 20N and the associated backfilling to reestablish grades for the eastbound embankment.

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 organic silt may be classified as a Type 4 soil. The native clayey silt and silt sand materials may be classified as Type 3 soil. **Side slopes for excavations through more than one soil type must be entirely based on the highest soil type number.**

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

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

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

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

11.2 Temporary Protection Systems

Temporary Protection Systems (TPS) could be used for excavation support or groundwater control. They must be implemented in accordance with OPSS.PROV 539. Performance Level 2 (maximum 25 mm horizontal deflection) is considered appropriate where the protection supports an existing roadway. More stringent performance levels may be required if the protection system is intended to support existing structures or utilities. 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.

Steel sheet piles are considered a suitable option for this site; however, the selection and design of protection systems are the responsibility of the Contractor. All protection systems should be designed by a licensed Professional Engineer experienced in such designs and retained by the Contractor. The design of the roadway protection system must incorporate traffic loading and surcharge loading due to construction equipment and operations. In Boreholes NSC20-1 and NSC20-3 the bedrock elevation was 138.8 m and 135.9. The bedrock surface appears to slope down from the westbound culvert inlet to the eastbound culvert outlet. It is noted that shallow sloping bedrock could impact sheet pile installation at this site; suggested wording for a Contract Provision is provided in Appendix H. An anchoring and/or internal bracing system may need to be incorporated into the temporary protection design to resist lateral earth pressure loadings.

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

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/m ³) | K _A | K _P | K ₀ | S _u (kPa) |
|-------------------------------|--|----------------|----------------|----------------|-------------------------|
| Existing Granular Fills | 20 | 0.33 | 3.0 | 0.50 | - |
| Native Cohesive Clayey Silt | 17.5 | - | - | - | 75 |
| Native Silty Sand with Gravel | 21 | 0.27 | 3.7 | 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.

When designing roadway protection systems, the Contractor should consider the potential for obstructions such as cobbles and boulders in existing embankment. Although not encountered in the on-road boreholes at this site, cobbles and boulders have been noted in embankment fill and rockfill embankments have been noted along Highway 17 within the project limits. Suggested wording for an NSSP for obstructions is included in Appendix H.

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 SP 517F01 which amends OPSS.PROV 517. The contractor's design should include an assessment of any adverse effects the dewatering method, construction layout and staging may have on adjacent structures, utilities, and facilities. Given the site conditions (potential for bottom heave from underlying silty sand seams) and anticipated works (excavating to more than 1.5 m below groundwater level), the Designer Fill-In (Note 2) in SP 517F01 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 below the final excavation level; Note 5 of SP 517F01 Table 1 should be a minimum of 0.5 m below the underside of the planned excavation base prior to each stage of excavation.

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

The dewatering plan should be coordinated with 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 or granular 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 earth or granular 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 drawings for the proposed culverts are not available at the time of writing. The preliminary recommendations provided herein will need to be re-evaluated once the culvert details are confirmed.

The planned construction methodology includes open cut excavations for the installation of foundation elements of new culverts. 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/silty clay which will be exposed beneath the 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.
- The bedrock elevation is variable across the site. Sloping bedrock may be encountered and may affect installation of temporary protection systems. A Notice to Contractor has been included in Appendix H.
- Mitigation of the settlement induced by the new westbound embankment will require a culvert designed to accommodate the movements.

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

13 CLOSURE

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

Thurber Engineering Ltd.
Report Prepared By:



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



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

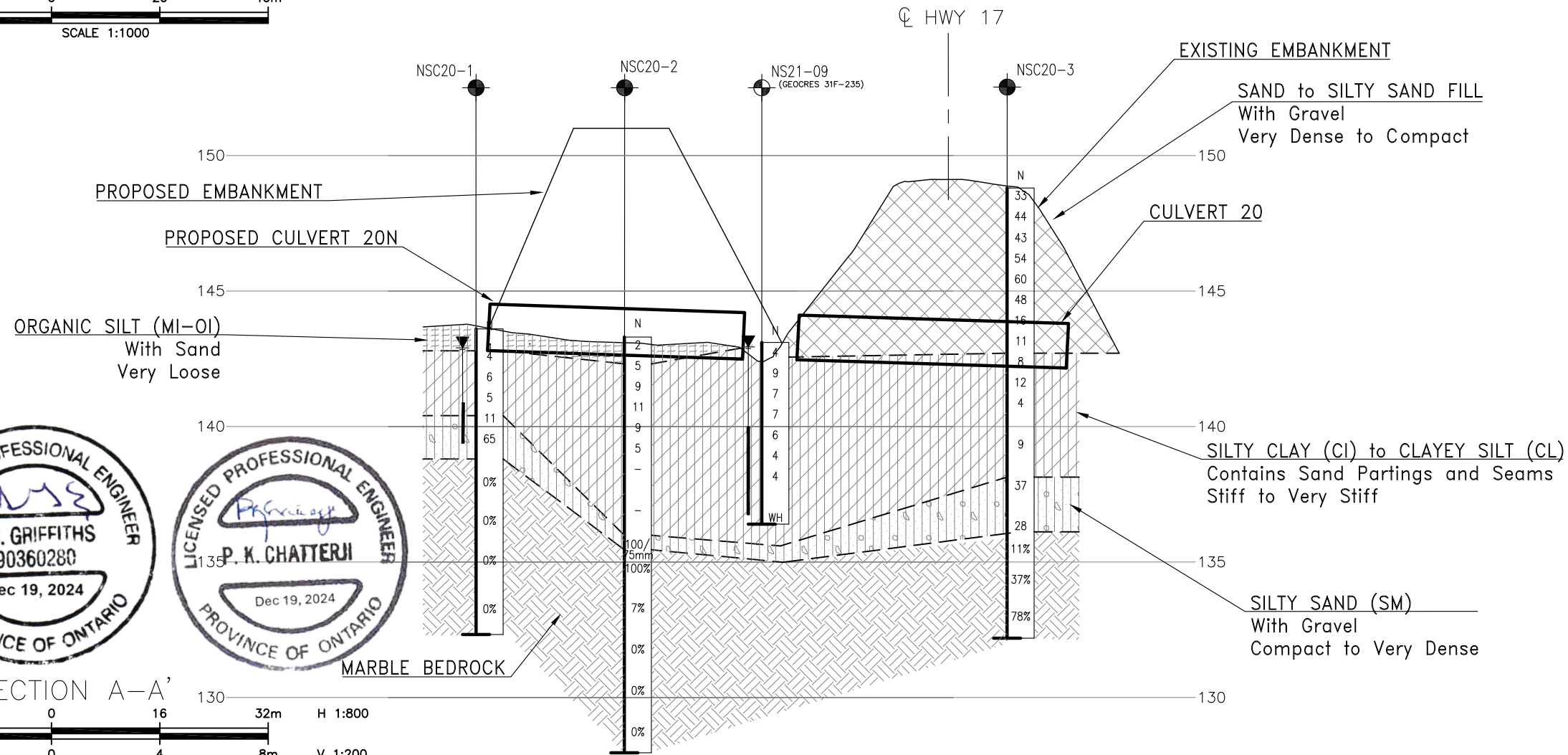
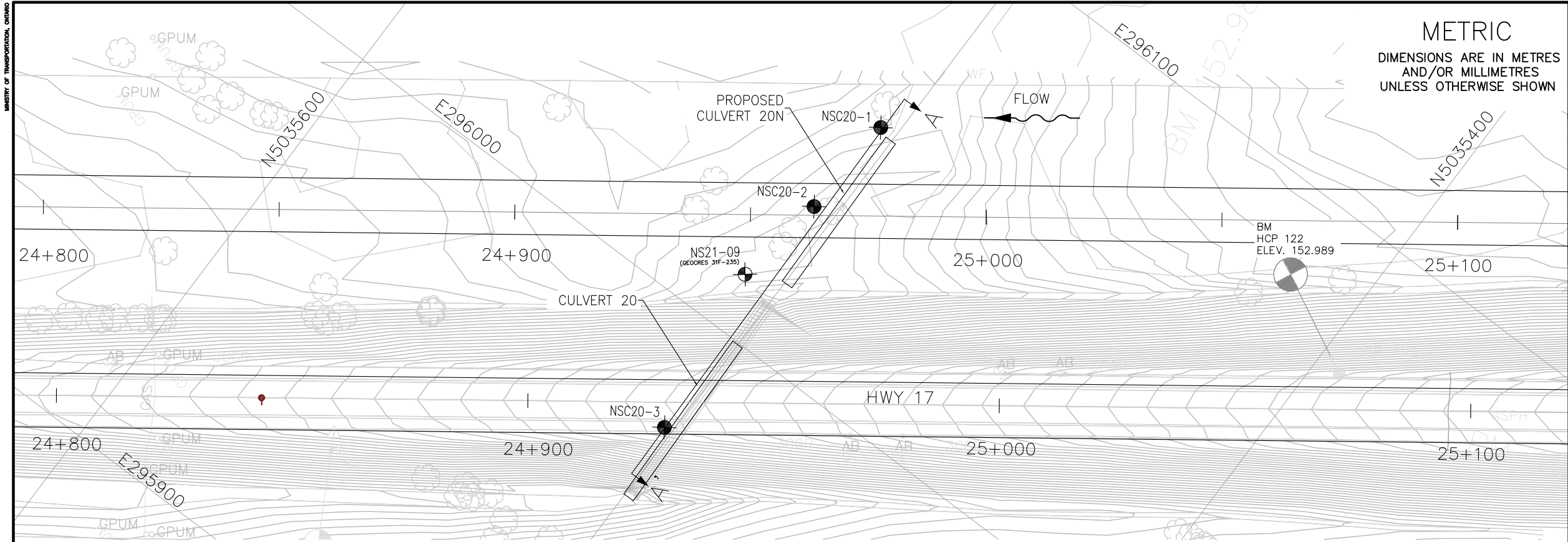


Dr. P.K. Chatterji, P.Eng.
Designated Principal Contact,
Principal, Senior Geotechnical Engineer



Appendix A.

Borehole Location Plan and Stratigraphic Drawings



CONT No
GWP No 4068-09-00

HIGHWAY 17 TWINNING
STA. 24+936, HORTON TWP
CULVERT 20/20N
BOREHOLE LOCATION PLAN AND SOIL STRATA

Ontario

SHEET
1

THURBER

KEYPLAN

LEGEND

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

| NO | ELEVATION | NORTHING | EASTING |
|---------|-----------|-------------|-----------|
| NCS20-1 | 143.6 | 5 035 503.6 | 296 054.7 |
| NCS20-2 | 143.3 | 5 035 504.8 | 296 032.8 |
| NSC20-3 | 148.6 | 5 035 481.0 | 295 972.3 |
| NS21-09 | 143.1 | 5 035 507.8 | 296 012.5 |

-NOTES-

1) The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.

2) This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

3) Coordinate system is MTM NAD 83 Zone 9.

GEOCRES No. 31F07-006

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



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).

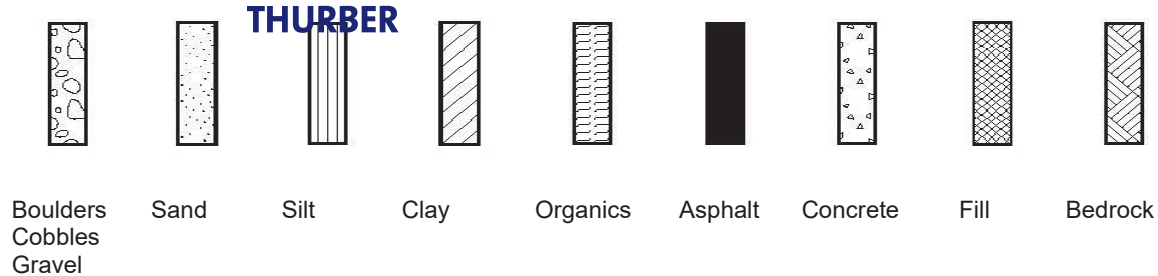
DYNAMIC CONE PENETRATION TEST (DCPT):

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 of the strata symbols are not indicative of the particle size, layer thickness, etc.



| sification | 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 NSC20-1

1 OF 2

METRIC

WP# 4068-09-00 LOCATION Lat: 45.459407°, Long: -76.611823° Culvert 20/20N; Horton Township; MTM z9: N 5 035 503.6 E 296 054.7 ORIGINATED BY BC
 HWY 17 BOREHOLE TYPE CME 75 Trackmount / HSA / NW Casing / NQ Coring COMPILED BY AO
 DATUM Geodetic DATE 2024.04.01 - 2024.04.01 CHECKED BY JG

| 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 (%) | | | | |
| 143.6 | Ground Surface | | | | | | 20 40 60 80 100 | ○ UNCONFINED + FIELD VANE | W _p W W _L | | | | | |
| 0.0 | Sandy ORGANIC SILT (MI-OI) contains peat inclusions very loose brown to black | | 1 | SS | 1 | | 20 40 60 80 100 | ● QUICK TRIAXIAL × LAB VANE | | | | | | |
| 143.2 | | | | | | | | | | | | | | |
| 0.4 | SILTY CLAY (CI) contains sand partings and seams very stiff greyish brown to brown | | 2 | SS | 4 | | | | | | | | 0 5 59 36 | |
| | | | | | | | | | | | | | | |
| | | | 3 | SS | 6 | | | | | | | | | |
| | | | | | | | | | | | | | | |
| | | | 4 | SS | 5 | | | | | | | | 0 4 61 35 | |
| | | | | | | | | | | | | | | |
| 140.4 | | | | | | | | | | | | | | |
| 3.2 | SILTY SAND (SM) with gravel compact to very dense grey to brown | | 5 | SS | 11 | | | | | | | | | |
| | | | | | | | | | | | | | | |
| | | | 6 | SS | 65 | | | | | | | | 39 43 18 (SI+CL) | |
| | | | | | | | | | | | | | | |
| 138.8 | | | | | | | | | | | | | | |
| 4.8 | MARBLE BEDROCK completely to slightly weathered light grey coarse grained medium strong to strong | | 1 | RUN | - | | | | | | | FI | | |
| | | | | | | | | | | | | >10 | | |
| | | | | | | | | | | | | >10 | RUN #1 TCR=23% SCR=5% RQD=0% | |
| | | | | | | | | | | | | >10 | | |
| | | | | | | | | | | | | >10 | | |
| | | | | | | | | | | | | >10 | | |
| | | | 2 | RUN | - | | | | | | | >10 | RUN #2 TCR=15% SCR=5% RQD=0% | |
| | | | | | | | | | | | | >10 | | |
| | | | | | | | | | | | | >10 | | |
| | | | | | | | | | | | | >10 | | |
| | | | 3 | RUN | - | | | | | | | >10 | RUN #3 TCR=51% SCR=7% RQD=0% | |
| | | | | | | | | | | | | >10 | | |
| | | | | | | | | | | | | >10 | | |
| | | | | | | | | | | | | >10 | | |
| | | | | | | | | | | | | >10 | | |

Continued Next Page

+³, ×³: Numbers refer to
Sensitivity

20
15
10
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No NSC20-1

2 OF 2

METRIC

WP# 4068-09-00 LOCATION Lat: 45.459407°, Long: -76.611823°
Culvert 20/20N; Horton Township; MTM z9: N 5 035 503.6 E 296 054.7 ORIGINATED BY BC
HWY 17 BOREHOLE TYPE CME 75 Trackmount / HSA / NW Casing / NQ Coring COMPILED BY AO
DATUM Geodetic DATE 2024.04.01 - 2024.04.01 CHECKED BY JG

| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT | | | | | PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT | | | UNIT WEIGHT γ | REMARKS & GRAIN SIZE DISTRIBUTION (%) |
|---------------|--|------------|---------|------|------------|----------------------------|-----------------|--|----|----|-----|----------------|---|----------------|-------------------|---|---|
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | | SHEAR STRENGTH kPa | | | | | WATER CONTENT (%) | | | | |
| | | | | | | | | ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE | | | | | | | | | |
| | | | | | | | 20 | 40 | 60 | 80 | 100 | W _p | W | W _L | kN/m ³ | GR SA SI CL | |
| | Continued From Previous Page | | | | | | | | | | | | | | | | |
| | MARBLE BEDROCK completely to slightly weathered light grey coarse grained medium strong to strong | | 4 | RUN | - | | 133 | | | | | | | | | >10 >10 >10 >10 >10 | RUN #4 TCR=46% SCR=5% RQD=0% |
| 132.3 | | | | | | | | | | | | | | | | | |
| 11.3 | 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) 2024/04/09 0.3 143.3 2024/05/01 0.4 143.2 2024/06/07 0.5 143.1 2024/06/28 0.5 143.1 2024/07/12 0.5 143.1 2024/08/29 0.5 143.1 | | | | | | | | | | | | | | | | |

RECORD OF BOREHOLE No NSC20-2

1 OF 2

METRIC

WP# 4068-09-00 LOCATION Lat: 45.459417°, Long: -76.612104° Culvert 20/20N; Horton Township; MTM z9: N 5 035 504.8 E 296 032.8 ORIGINATED BY BC
HWY 17 BOREHOLE TYPE CME 75 Trackmount / HSA / NW Casing / NQ Coring COMPILED BY AO
DATUM Geodetic DATE 2024.04.02 - 2024.04.02 CHECKED BY JG

| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT | | | PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT | | | UNIT WEIGHT 7 kN/m ³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) | | | | | |
|---------------|---|------------|---------|------|--------------|----------------------------|-----------------|---|----|----|--|----|----|--|---|----|----|----|----|----|
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | | SHEAR STRENGTH kPa | | | WATER CONTENT (%) | | | | | | | | | |
| 143.3 | Ground Surface | | | | | | 20 | 40 | 60 | 80 | 100 | 20 | 40 | 60 | GR | SA | SI | CL | | |
| 0.0 | Sandy ORGANIC SILT (MI-OI) very loose brown | | 1 | SS | 2 | | | | | | | | 4 | | | 0 | 35 | 46 | 19 | |
| 142.3 | | | | | | | | | | | | | o | | | | | | | |
| 1.0 | CLAYEY SILT (CL) contains sand partings and seams very stiff to stiff brown | | 2 | SS | 5 | | | | | | | | o | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | |
| | | | 3 | SS | 9 | | | | | | | | o | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | |
| | | | 4 | SS | 11 | | | | | | | | 4 | | | | 0 | 3 | 62 | 35 |
| | | | | | | | | | | | | | | | | | | | | |
| | | | 5 | SS | 9 | | | | | | | | o | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | |
| | - becomes grey at a depth of 3.8 m (elev. 139.5 m) | | 6 | SS | 5 | | | | | | | | o | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | |
| | | | 7 | TW | - | | | | | | | | 4 | | | | 0 | 1 | 63 | 36 |
| | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | |
| | | | 8 | TW | - | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | |
| | - unable to push vane | | | | | | | | | | | | | | | | | | | |
| 136.0 | | | | | | | | | | | | | | | | | | | | |
| 7.3 | SILTY SAND (SM) with gravel very dense grey | | 9 | SS | 100/ 75mm | | | | | | | | o | | | | | | | |
| 135.5 | | | | | | | | | | | | | | | | | | | | |
| 7.8 | MARBLE BEDROCK completely to highly weathered white to light grey coarse grained medium strong to strong | | 1 | RUN | - | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | |
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Continued Next Page

+³ × 3: Numbers refer to
Sensitivity

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(%) STRAIN AT FAILURE

DOUBLE LINE CULVERT 20 GINT LOGS.GPJ 2012TEMPLATE(MTO).GDT 12-17-24

METRIC

[illegible]

RECORD OF BOREHOLE No NSC20-3

1 OF 2

METRIC

WP# 4068-09-00 LOCATION Lat: 45.459436°, Long: -76.612878° Culvert 20/20N; Horton Township; MTM z9: N 5 035 481.0 E 295 972.3 ORIGINATED BY DAP
 HWY 17 BOREHOLE TYPE CME 75 Truckmount / HSA / NW Casing / NQ Coring COMPILED BY AO
 DATUM Geodetic DATE 2024.03.26 - 2024.03.26 CHECKED BY JG

| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT | | | | PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT | | | UNIT WEIGHT kN/m ³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) | | | |
|---------------|--|------------|---------|------|------------|----------------------------|-----------------|---|----|----|----|---|----------------|---|---|---|----|----|----|
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | | SHEAR STRENGTH kPa | | | | WATER CONTENT (%) | | | | GR | SA | SI | CL |
| | | | | | | | | 20 | 40 | 60 | 80 | 100 | W _p | W | | | | | |
| 148.6 | Ground Surface | | | | | | | | | | | | | | | | | | |
| 0.0 | SAND with silt, some gravel to SILTY SAND with gravel compact to very dense brown to grey FILL | | 1 | SS | 33 | | | | | | | | | | | | | | |
| | | | 2 | SS | 44 | | | | | | | | | | | | | | |
| | | | 3 | SS | 43 | | | | | | | | | | | | | | |
| | | | 4 | SS | 54 | | | | | | | | | | | | | | |
| | | | 5 | SS | 60 | | | | | | | | | | | | | | |
| | | | 6 | SS | 48 | | | | | | | | | | | | | | |
| | | | 7 | SS | 16 | | | | | | | | | | | | | | |
| | | | 8 | SS | 11 | | | | | | | | | | | | | | |
| 142.5 | | | | | | | | | | | | | | | | | | | |
| 6.1 | SILTY CLAY (Cl) contains sand partings and seams very stiff grey to light brown | | 9 | SS | 8 | | | | | | | | | | | | | | |
| | | | 10 | SS | 12 | | | | | | | | | | | | | | |
| | | | 11 | SS | 4 | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | |
| | | | 12 | SS | 9 | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | |

Continued Next Page

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

RECORD OF BOREHOLE No NSC20-3

2 OF 2

METRIC

WP# 4068-09-00 LOCATION Lat: 45.459436°, Long: -76.612878° Culvert 20/20N; Horton Township; MTM z9: N 5 035 481.0 E 295 972.3 ORIGINATED BY DAP
HWY 17 BOREHOLE TYPE CME 75 Truckmount / HSA / NW Casing / NQ Coring COMPILED BY AO
DATUM Geodetic DATE 2024.03.26 - 2024.03.26 CHECKED BY JG

| 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 (%) GR SA SI CL | |
|---------------|--|---------|------|------------|----------------------------|-----------------|--|----|----|----|--|----|----|--|--|--|
| ELEV DEPTH | DESCRIPTION | NUMBER | TYPE | "N" VALUES | | | SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE | | | | WATER CONTENT (%) w _p w w _L | | | | | |
| | Continued From Previous Page | | | | | | 20 | 40 | 60 | 80 | 100 | 20 | 40 | 60 | | |
| 137.9 | SILTY CLAY (CI) contains sand partings and seams very stiff grey to light brown | | | | | 138 | | | | | | | | | | |
| 10.7 | SILTY SAND (SM) with gravel dense to compact light brown | 13 | SS | 37 | | 137 | | | | | | | | | | |
| | | | | | | | | | | | | | | | | |
| 135.9 | | 14 | SS | 28 | | 136 | | | | | | | | | | |
| 12.7 | MARBLE BEDROCK completely to highly weathered light grey coarse grained medium strong to strong | 1 | RUN | - | | 135 | | | | | | | | | | |
| | | 2 | RUN | - | | 134 | | | | | | | | | | |
| | | 3 | RUN | - | | 133 | | | | | | | | | | |
| 132.0 | | | | | | | | | | | | | | | | |
| 16.6 | End of Borehole | | | | | 132 | | | | | | | | | | |

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

RECORD OF BOREHOLE No NS21-09

1 OF 1

METRIC

WP# 4068-09-00 LOCATION Lat: 45.459444°, Long: -76.612363°
Culvert 24+950 Horton N 5 035 507.8 E 296 012.5 ORIGINATED BY NW
HWY 17 BOREHOLE TYPE Diedrich 50 (D-50) Trackmount / HSA COMPILED BY AO
DATUM Geodetic DATE 2021.11.19 - 2021.11.19 CHECKED BY JG

| 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 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE | | | WATER CONTENT (%) w _P w w _L | | | | GR | SA | SI | CL | | | | |
| 143.1 | Ground Surface | | | | | | | 20 | 40 | 60 | 80 | 100 | | | | | | | | | | |
| 0.0 | TOPSOIL (75 mm) | | | | | | 143 | | | | | | | | | | | | | | | |
| 0.1 | SILTY CLAY (CI) very stiff brown WEATHERED CRUST | | 1 | SS | 4 | | | | | | | | | | | | | | | | | |
| | | | 2 | SS | 9 | | 142 | | | | | | | | | | | | | | | |
| | | | 3 | SS | 7 | | 141 | | | | | | | | | | 0 | 5 | 67 | 28 | | |
| | | | 4 | SS | 7 | | | | | | | | | | | | | | | | | |
| | | | 5 | SS | 6 | | 140 | | | | | | | | | | | | | | | |
| 139.4 | | | | | | | | | | | | | | | | | | | | | | |
| 3.7 | CLAYEY SILT (CL) very stiff to stiff brown | | 6 | SS | 4 | | 139 | | | | | | | | | | | | | | | |
| | | | 7 | SS | 4 | | 138 | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | |
| | - becomes grey | | | | | | 137 | | | | | | | | | | | | | | | |
| 136.4 | | | 8 | SS | WH | | | | | | | | | | | | | | 0 | 1 | 62 | 37 |
| 6.7 | End of Borehole | | | | | | | | | | | | | | | | | | | | | |
| | Monitoring well consists of 50 mm diameter Schedule 40 PVC pipe with a 3.0 m slotted screen | | | | | | | | | | | | | | | | | | | | | |
| | Water Level Readings: DATE DEPTH (m) ELEV. (m) 2021/11/29 0.3 142.8 2021/12/02 0.5 142.6 2021/12/13 0.5 142.6 2022/01/21 0.5 142.6 | | | | | | | | | | | | | | | | | | | | | |

DOUBLE LINE CULVERT 20 GINT LOGS.GPJ 2012TEMPLATE(MTO).GDT 11-20-24

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



Appendix C.

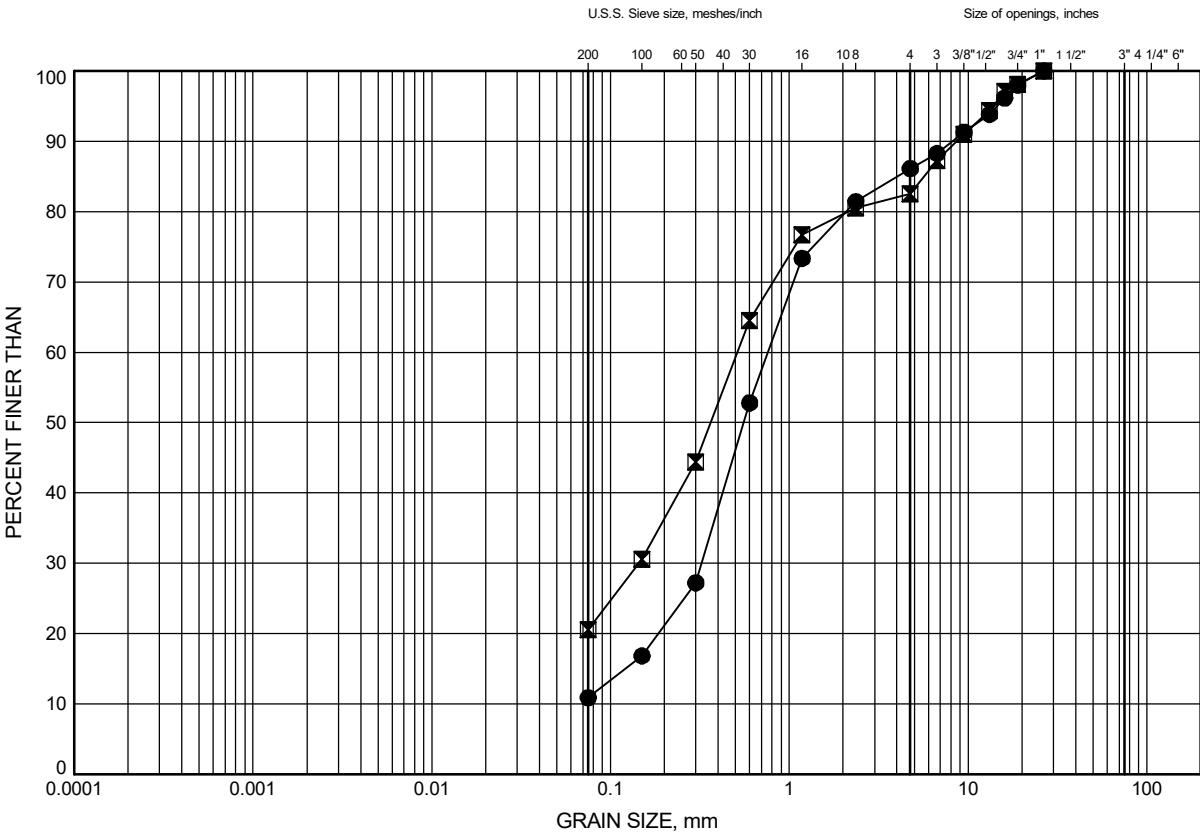
Laboratory Testing



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

GRAIN SIZE DISTRIBUTION

FILL: Sand to Silty Sand



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

LEGEND

| SYMBOL | BOREHOLE | DEPTH (m) | ELEV. (m) |
|--------|----------|-----------|-----------|
| ● | NSC20-3 | 2.6 | 146.0 |
| ⊠ | NSC20-3 | 4.9 | 143.7 |

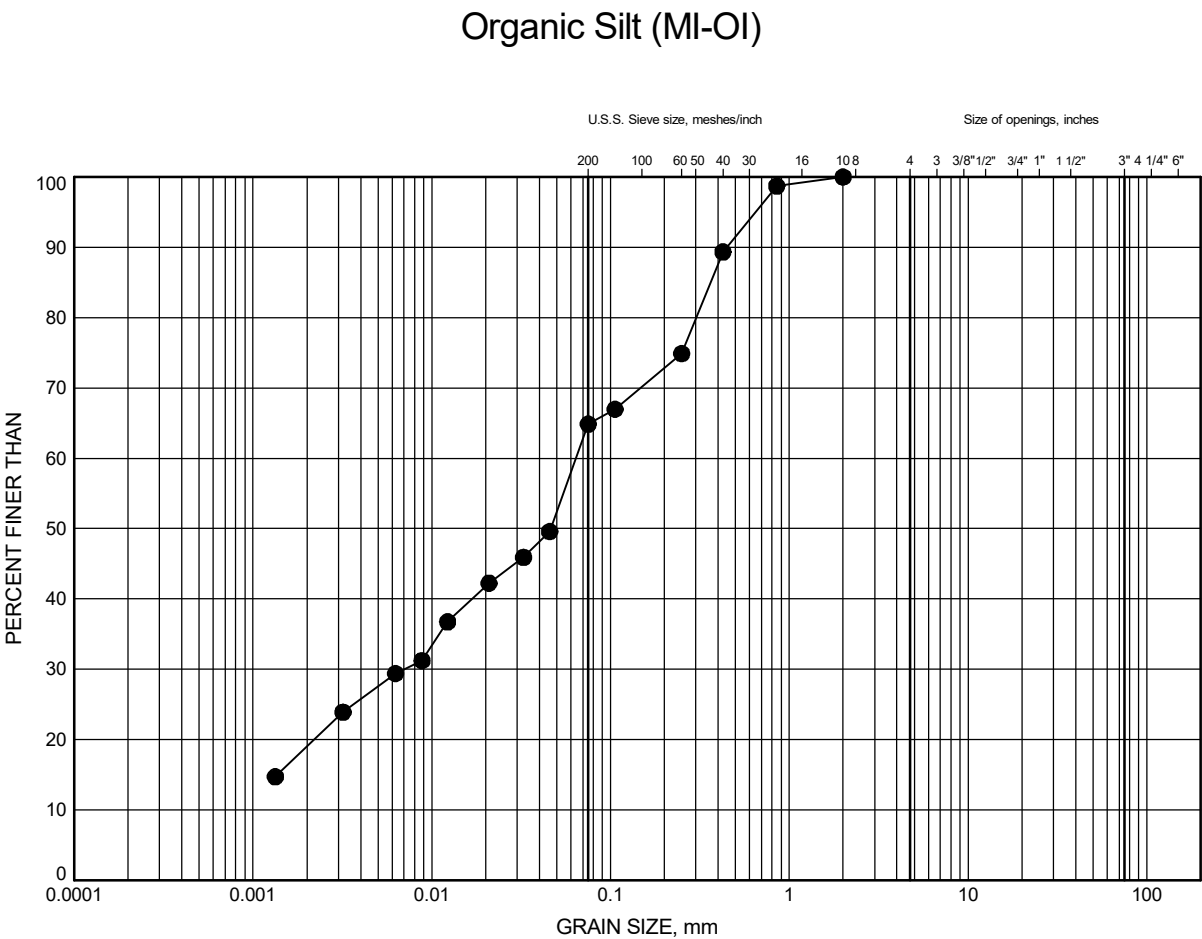
GRAIN SIZE DISTRIBUTION - THURBER CULVERT 20 GINT LOGS.GPJ 7-30-24



Highway 17 Twinning, Sta. 24+936, Culvert 20

GRAIN SIZE DISTRIBUTION

FIGURE C2



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

LEGEND

| | | | |
|--------|----------|-----------|-----------|
| SYMBOL | BOREHOLE | DEPTH (m) | ELEV. (m) |
| ● | NSC20-2 | 0.3 | 143.0 |

GRAIN SIZE DISTRIBUTION - THURBER CULVERT 20 GINT LOGS.GPJ 7-30-24

Date July 2024

GWP# 4018-E-0009



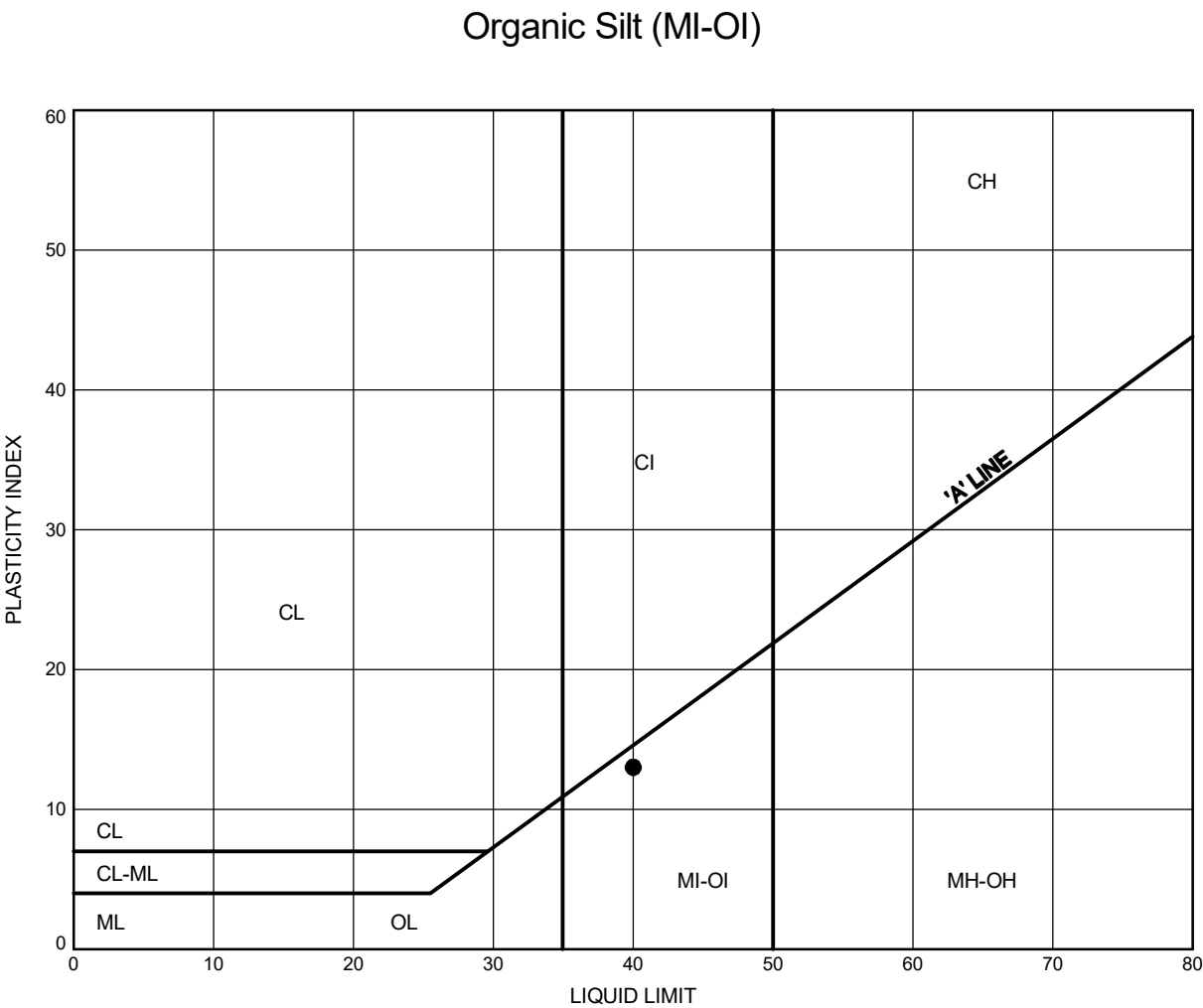
Prep'd RH

Chkd. AO

Highway 17 Twinning, Sta. 24+936, Culvert 20

ATTERBERG LIMITS TEST RESULTS

FIGURE C3



LEGEND

| SYMBOL | BOREHOLE | DEPTH (m) | ELEV. (m) |
|--------|----------|-----------|-----------|
| ● | NSC20-2 | 0.3 | 143.0 |

THURBALT CULVERT 20 GINT LOGS.GPJ 7-30-24

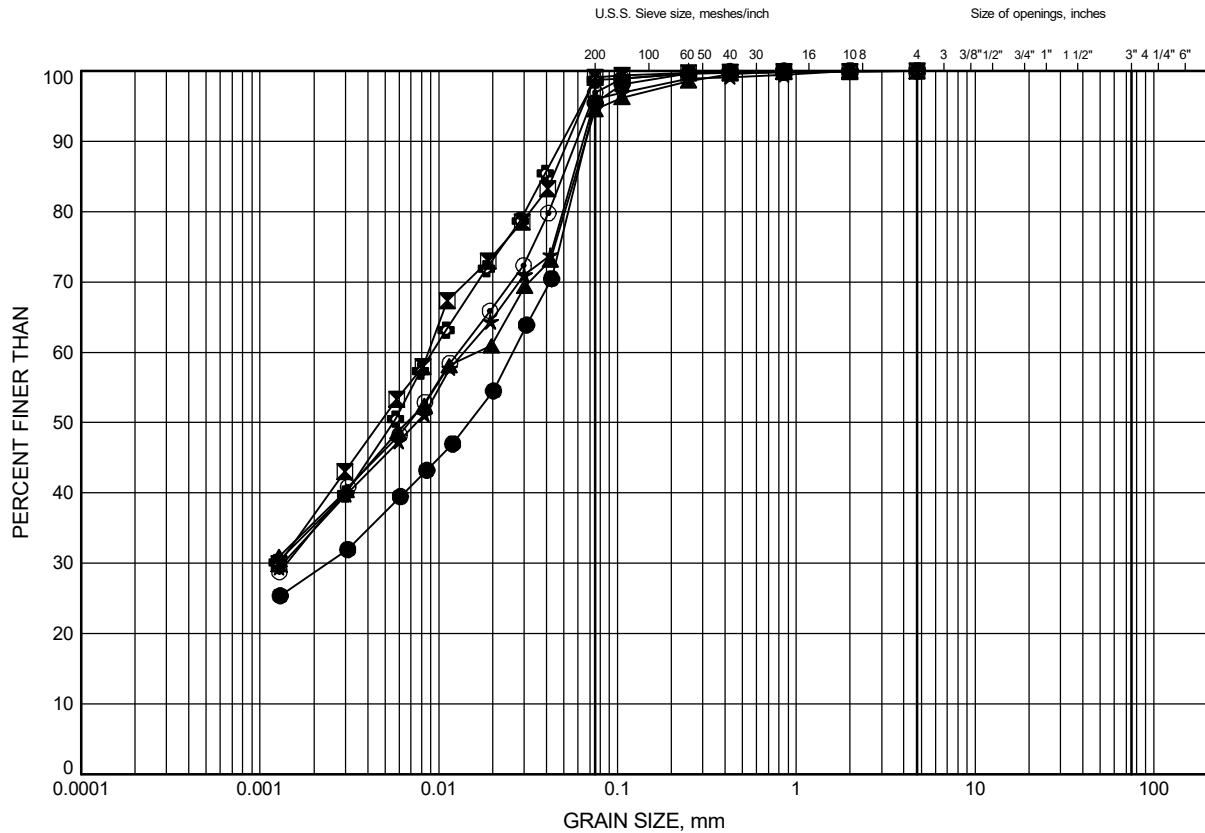
Date July 2024
GWP# 4018-E-0009



Prep'd RH
Chkd. AO

GRAIN SIZE DISTRIBUTION

Silty Clay (CI) to Clayey Silt (CL)



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

LEGEND

| SYMBOL | BOREHOLE | DEPTH (m) | ELEV. (m) |
|--------|----------|-----------|-----------|
| ● | NS21-09 | 1.8 | 141.3 |
| ⊠ | NS21-09 | 6.4 | 136.7 |
| ▲ | NSC20-1 | 1.1 | 142.5 |
| ★ | NSC20-1 | 2.6 | 141.0 |
| ⊙ | NSC20-2 | 2.6 | 140.7 |
| ⊕ | NSC20-2 | 4.9 | 138.4 |

Date July 2024

GWP# 4018-E-0009

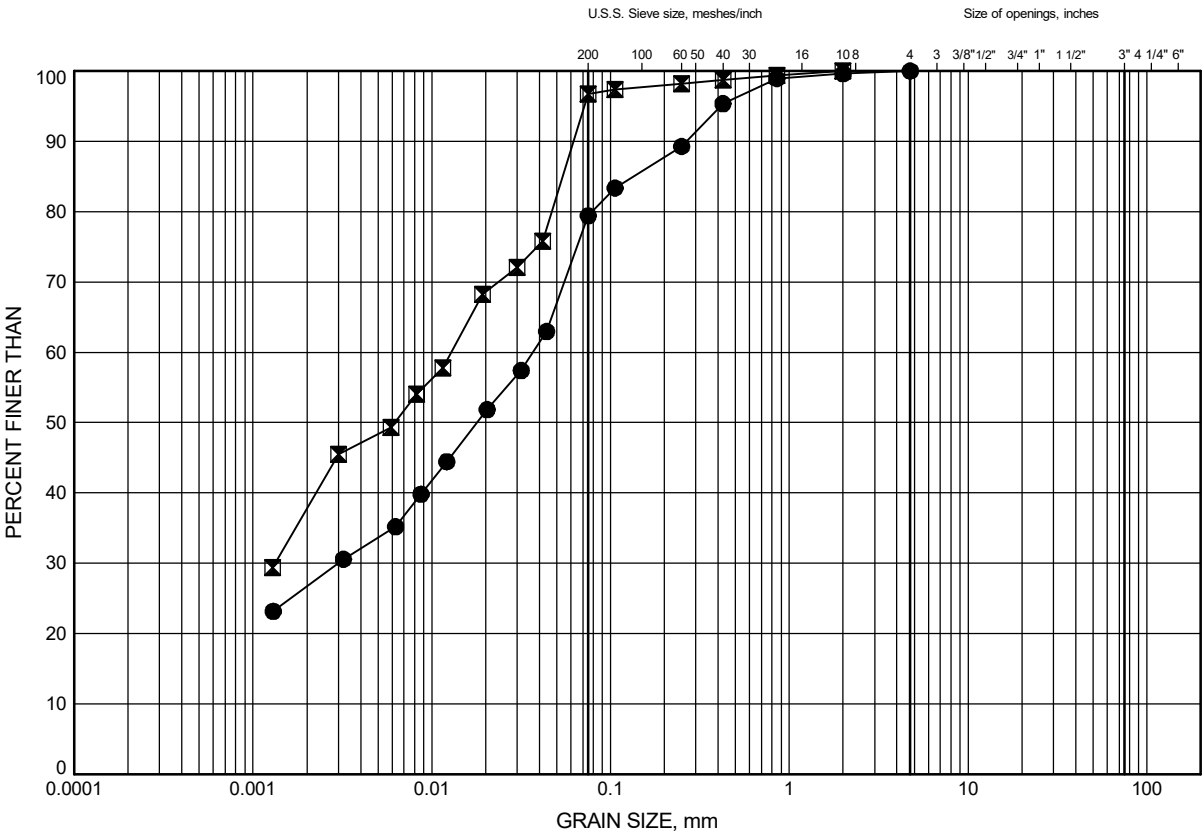


Prep'd RH

Chkd. AO

GRAIN SIZE DISTRIBUTION

Silty Clay (CI) to Clayey Silt (CL)



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

LEGEND

| SYMBOL | BOREHOLE | DEPTH (m) | ELEV. (m) |
|--------|----------|-----------|-----------|
| ● | NSC20-3 | 7.2 | 141.4 |
| ⊠ | NSC20-3 | 9.4 | 139.2 |

GRAIN SIZE DISTRIBUTION - THURBER CULVERT 20 GINT LOGS.GPJ 7-30-24

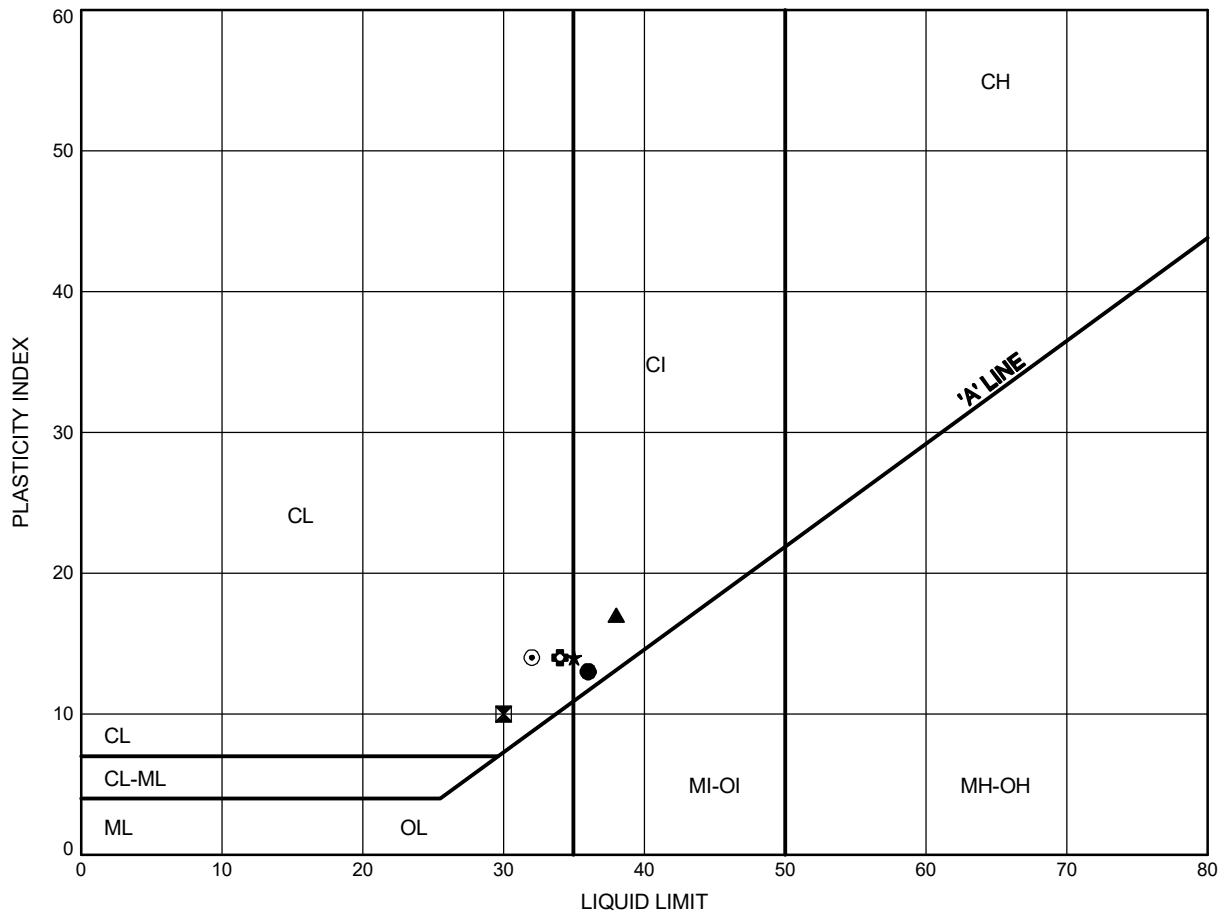


Highway 17 Twinning, Sta. 24+936, Culvert 20

ATTERBERG LIMITS TEST RESULTS

FIGURE C6

Silty Clay (CI) to Clayey Silt (CL)



LEGEND

| SYMBOL | BOREHOLE | DEPTH (m) | ELEV. (m) |
|--------|----------|-----------|-----------|
| ● | NS21-09 | 1.8 | 141.3 |
| ⊠ | NS21-09 | 6.4 | 136.7 |
| ▲ | NSC20-1 | 1.1 | 142.5 |
| ★ | NSC20-1 | 2.6 | 141.0 |
| ⊙ | NSC20-2 | 2.6 | 140.7 |
| ⊕ | NSC20-2 | 4.9 | 138.4 |

Date July 2024

GWP# 4018-E-0009



Prep'd RH

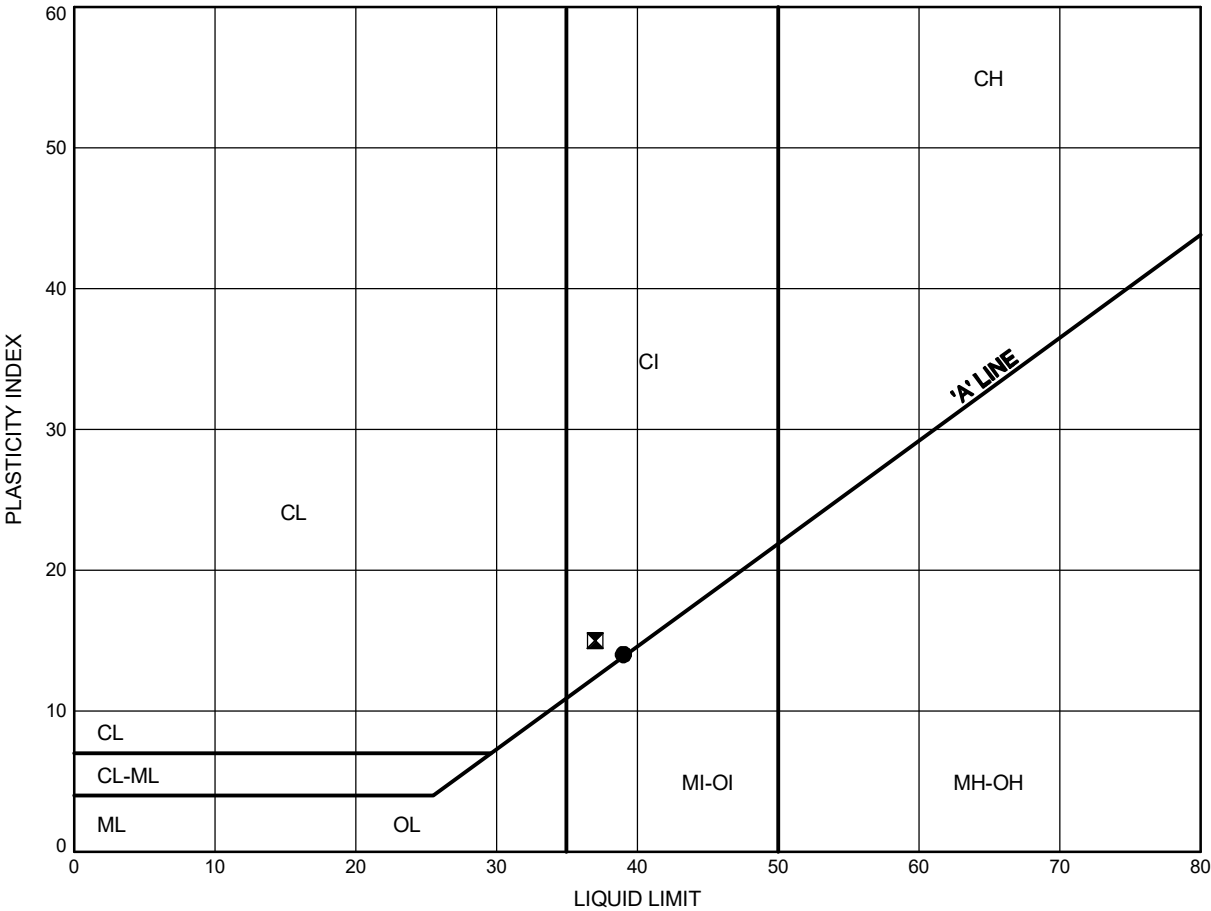
Chkd. AO

Highway 17 Twinning, Sta. 24+936, Culvert 20

ATTERBERG LIMITS TEST RESULTS

FIGURE C7

Silty Clay (CI) to Clayey Silt (CL)



LEGEND

| SYMBOL | BOREHOLE | DEPTH (m) | ELEV. (m) |
|--------|----------|-----------|-----------|
| ● | NSC20-3 | 7.2 | 141.4 |
| ⊠ | NSC20-3 | 9.4 | 139.2 |

THURBALT CULVERT 20 GINT LOGS.GPJ 7-30-24

Date July 2024
GWP# 4018-E-0009



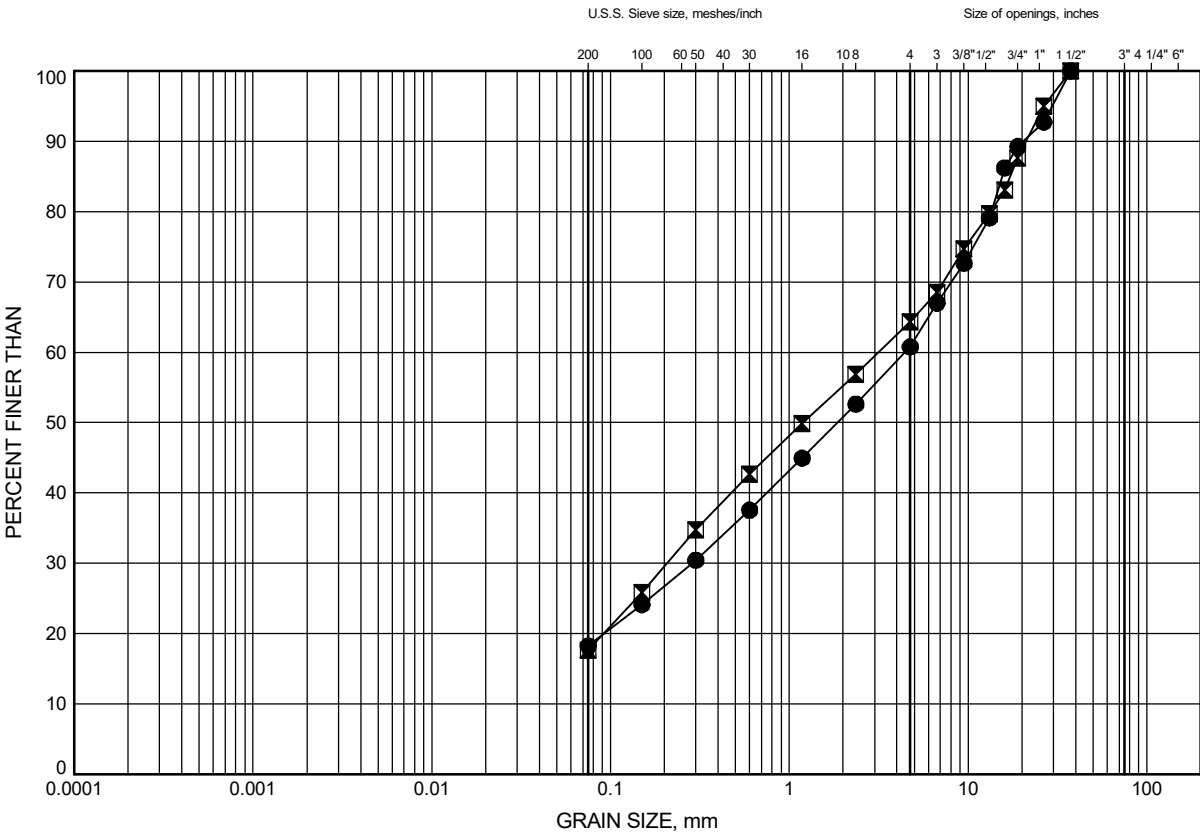
Prep'd RH
Chkd. AO

Highway 17 Twinning, Sta. 24+936, Culvert 20

GRAIN SIZE DISTRIBUTION

FIGURE C8

Silt Sand (SM)



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

LEGEND

| SYMBOL | BOREHOLE | DEPTH (m) | ELEV. (m) |
|--------|----------|-----------|-----------|
| ● | NSC20-1 | 4.2 | 139.4 |
| ⊠ | NSC20-3 | 12.5 | 136.1 |

GRAIN SIZE DISTRIBUTION - THURBER CULVERT 20 GINT LOGS.GPJ 7-30-24

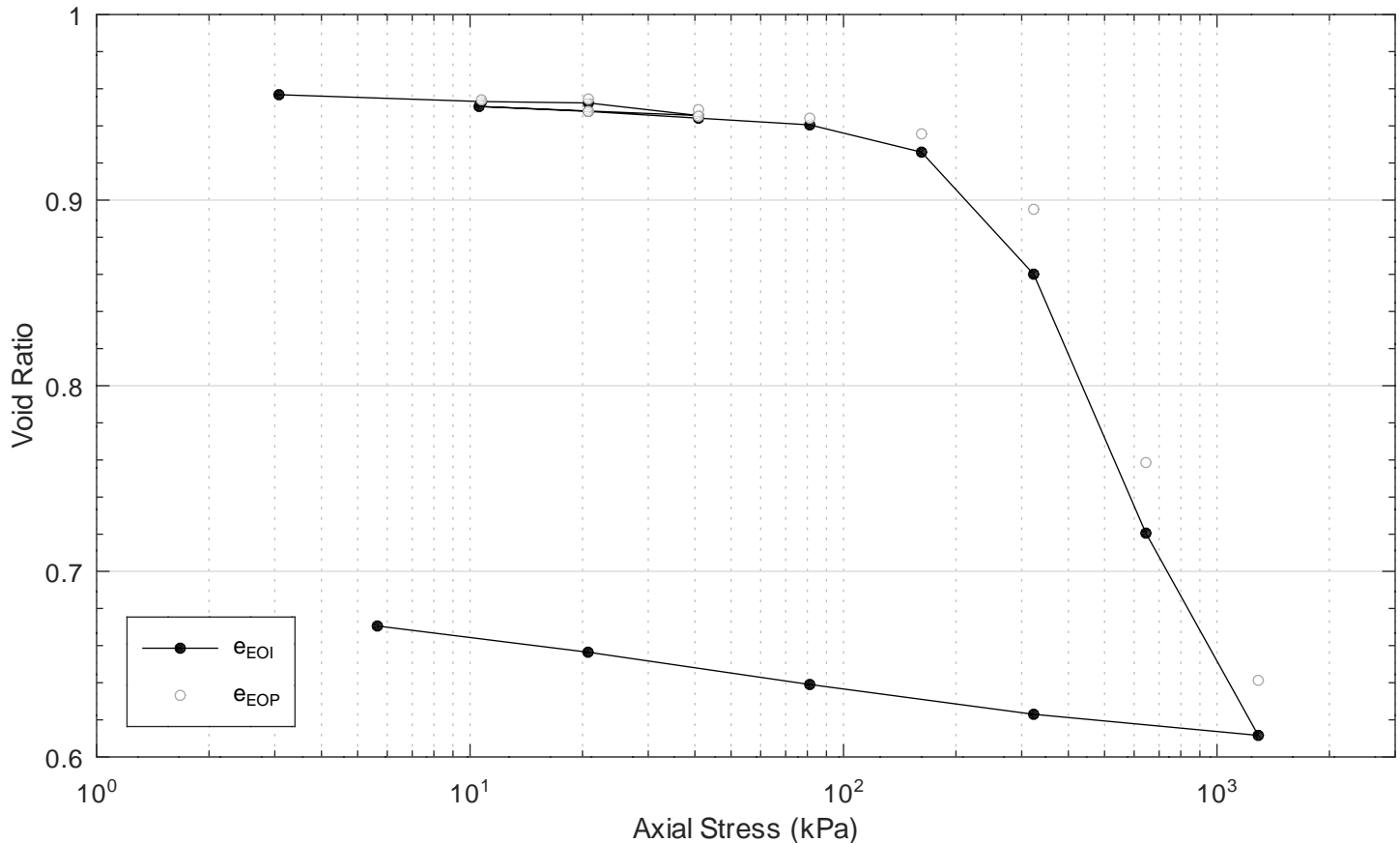
Date July 2024
GWP# 4018-E-0009



Prep'd RH
Chkd. AO



Project: 24726
 Hwy 17 Twinning
 Borehole: NSC20-2
 Sample: TW7
 Depth: 4.6m
 Client: MTO



Start of Test 2024-06-07

| | | | |
|----------------------|-------------------|-----------------|-------|
| Diameter of Sample | cm | D | 6.327 |
| Height of Sample | cm | H _o | 2.540 |
| Height of Solids | cm | H _s | 1.298 |
| Water Content | % | w _o | 33.75 |
| Dry Density | g/cm ³ | ρ _d | 1.42 |
| Moist Unit Weight | kN/m ³ | γ | 18.6 |
| Void Ratio | - | e _o | 0.956 |
| Degree of Saturation | - | S _{ro} | 0.98 |
| Specific Gravity | - | G _s | 2.771 |

End of Test 2024-06-24

| | | | |
|------------------|----|----------------|-------|
| Height of Sample | cm | H _f | 2.169 |
| Water Content | % | w _f | 24.71 |
| Void Ratio | - | e _f | 0.671 |

TRIMMING: the specimen was manually trimmed to the size of the consolidation ring, then mounted in a fixed ring consolidometer

LOADING: the consolidometer was flooded with water with the seating load adjusted to limit swelling

CALCULATIONS: coefficients of consolidation were calculated by the square root time method, secondary consolidation was calculated based on the available duration of the time step

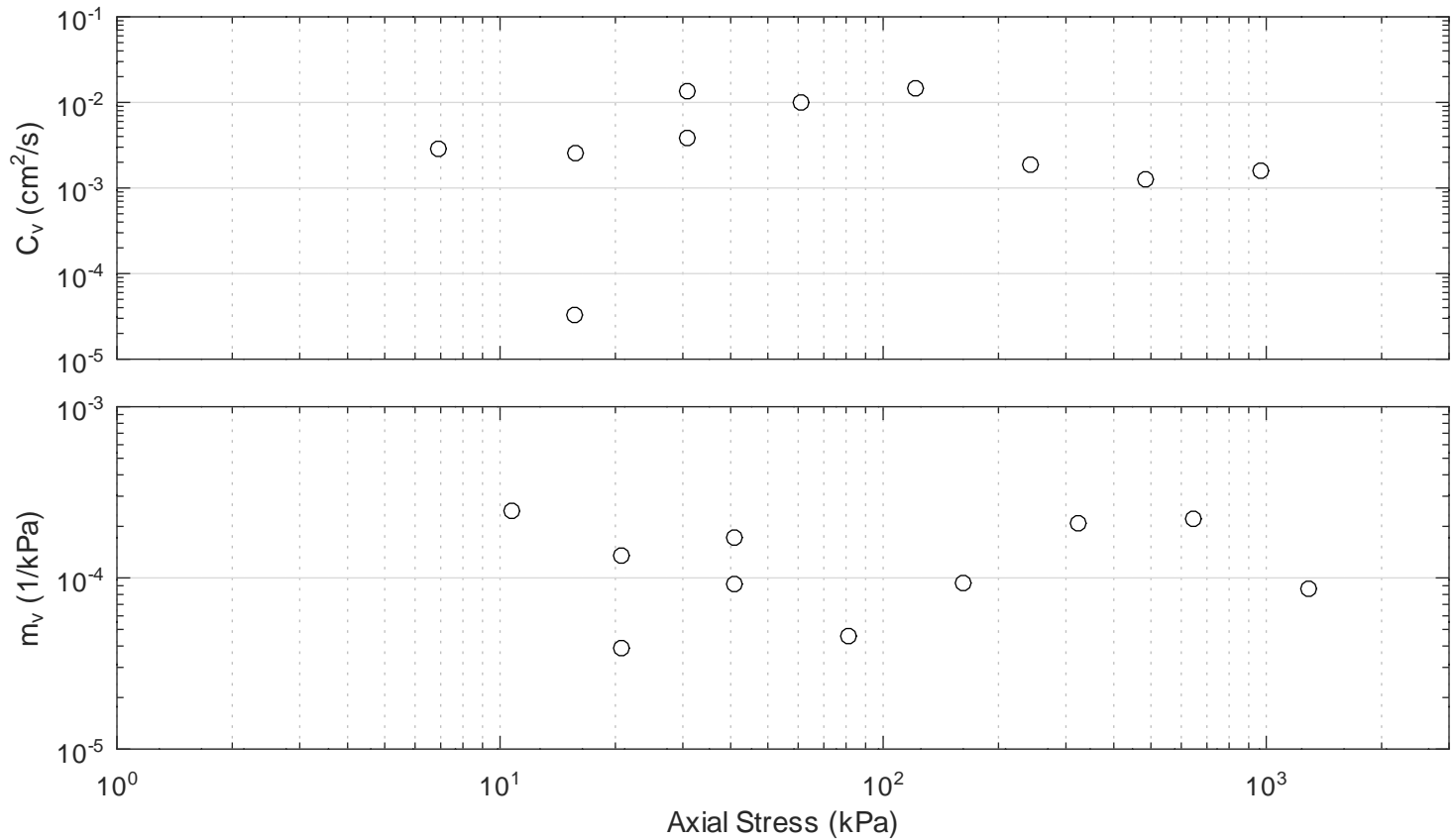
Interpreted Results

| | | | |
|------------------------------------|-----|-----------------|-------|
| Recompression Index (reloading) | - | C _r | 0.031 |
| Compression Index | - | C _c | 0.464 |
| Recompression Index (unloading) | - | C _r | 0.027 |
| Probable Preconsolidation Pressure | kPa | p' _c | 246 |

Check: AO/SP Review: KS/PK



Project: 24726
 Hwy 17 Twinning
 Borehole: NSC20-2
 Sample: TW7
 Depth: 4.6m
 Client: MTO



| Load No. | Axial Stress | Load Duration | System Deflec. | Dial | Sample Height | Axial Strain | Void Ratio | Void Ratio | Time U(0.99) | C_v | k_v | C_{ae} |
|----------|--------------|---------------|----------------|--------|---------------|--------------|------------|------------|--------------|--------------------|----------|----------|
| | kPa | min | mm | mm | cm | % | (EOI) | (EOP) | min | cm ² /s | cm/s | - |
| 0 | | | | 10.000 | 2.540 | 0.00 | 0.956 | | | | | |
| 1 | 3.1 | 1440.1 | 0.008 | 10.002 | 2.541 | -0.04 | 0.957 | | | | | |
| 2 | 10.7 | 1440.1 | 0.041 | 9.920 | 2.536 | 0.15 | 0.953 | 0.954 | 16.5 | 2.87e-03 | 6.93e-08 | 0.0002 |
| 3 | 20.7 | 1440.4 | 0.102 | 9.849 | 2.535 | 0.19 | 0.952 | 0.954 | 18.6 | 2.56e-03 | 9.75e-09 | 0.0007 |
| 4 | 40.9 | 1440.5 | 0.141 | 9.722 | 2.526 | 0.53 | 0.946 | 0.949 | 3.5 | 1.35e-02 | 2.28e-07 | 0.0006 |
| 5 | 10.6 | 1440.5 | 0.115 | 9.811 | 2.532 | 0.28 | 0.950 | | | | | |
| 6 | 20.7 | 1440.2 | 0.122 | 9.770 | 2.529 | 0.42 | 0.948 | 0.948 | 1440.1 | 3.28e-05 | 4.35e-10 | 0.0046 |
| 7 | 40.9 | 1440.3 | 0.141 | 9.704 | 2.524 | 0.61 | 0.944 | 0.945 | 12.3 | 3.84e-03 | 3.47e-08 | 0.0004 |
| 8 | 81.2 | 1440.0 | 0.213 | 9.585 | 2.520 | 0.79 | 0.941 | 0.944 | 4.7 | 9.98e-03 | 4.48e-08 | 0.0008 |
| 9 | 161.8 | 1440.2 | 0.295 | 9.312 | 2.500 | 1.54 | 0.926 | 0.936 | 3.1 | 1.46e-02 | 1.34e-07 | 0.0020 |
| 10 | 322.9 | 1440.3 | 0.382 | 8.372 | 2.415 | 4.90 | 0.860 | 0.895 | 21.2 | 1.87e-03 | 3.83e-08 | 0.0103 |
| 11 | 645.2 | 1440.3 | 0.484 | 6.459 | 2.234 | 12.04 | 0.721 | 0.759 | 17.8 | 1.26e-03 | 2.74e-08 | 0.0109 |
| 12 | 1290.0 | 1440.5 | 0.601 | 4.927 | 2.093 | 17.61 | 0.612 | 0.641 | 13.6 | 1.59e-03 | 1.35e-08 | 0.0086 |
| 13 | 322.9 | 1440.4 | 0.466 | 5.210 | 2.107 | 17.03 | 0.623 | | | | | |
| 14 | 81.2 | 1440.2 | 0.347 | 5.538 | 2.128 | 16.21 | 0.639 | | | | | |
| 15 | 20.7 | 1440.4 | 0.280 | 5.831 | 2.151 | 15.32 | 0.656 | | | | | |
| 16 | 5.6 | 2760.1 | 0.223 | 6.072 | 2.169 | 14.59 | 0.671 | | | | | |

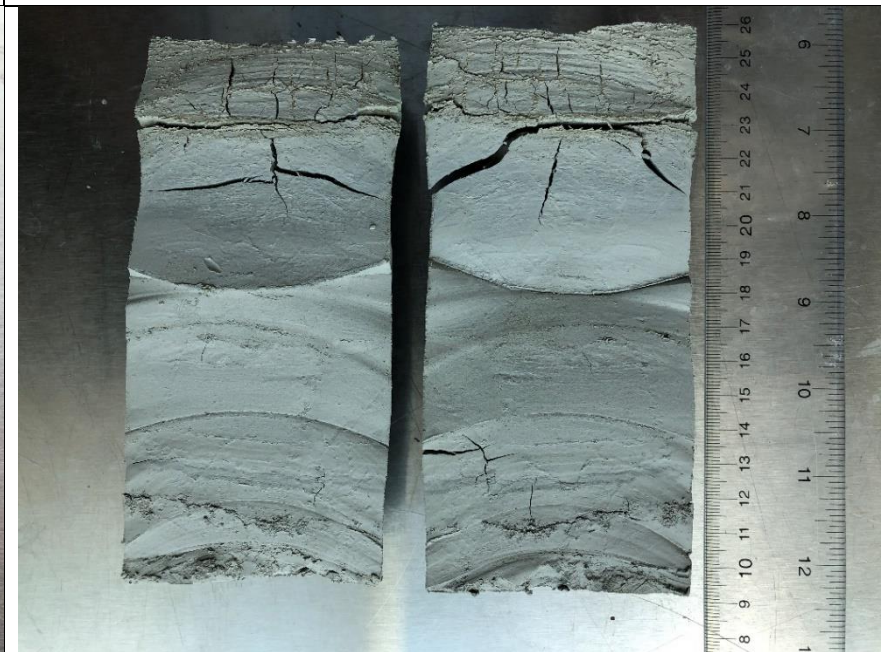
Borehole NSC20-2, Sample TW7, Depth 4.9 m

(sample width approximately equal to diameter of Thin-Walled sample tube, ~70 mm)

“Wet”



“Dry”



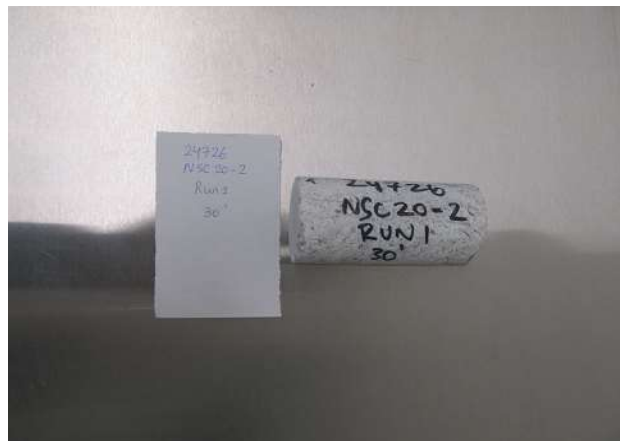
UNCONFINED COMPRESSION TEST REPORT

ASTM D7012-14

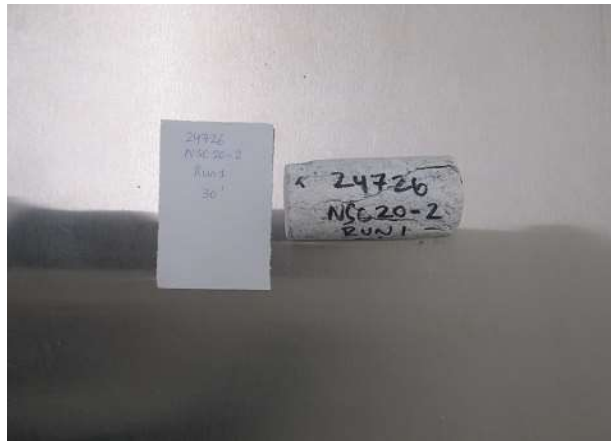
| | | | |
|---------------|-------------------------------|--------------|-----------|
| CLIENT: | Thurber Engineering (Ottawa) | FILE NUMBER: | 24726 |
| PROJECT NAME: | Highway 17 Twinning - Renfrew | REPORT DATE: | 18-Jul-24 |
| BOREHOLE No.: | NSC20-2 | TEST DATE: | 9-May-24 |
| SAMPLE No.: | Run 1 | | |
| SAMPLE DEPTH: | 9.14 m | | |
| DESCRIPTION: | Marble | | |

| | | | |
|--|--------|-----------------------------------|-------|
| Avg. Height (cm): | 9.7 | Weight (g): | 462.3 |
| Avg. Diameter (cm): | 4.7 | Wet Density (kg/m ³): | 2,747 |
| H. to Dia. Ratio**: | 2.1:1 | Dry Density (kg/m ³): | 2,747 |
| Cross Sectional Area (cm ²): | 17.35 | Moisture Content* (%): | N/A |
| Sample Volume (cm ³): | 168.29 | | |

ORIGINAL SPECIMEN



FRACTURED SPECIMEN



| | |
|----------------------------------|-------------|
| AVG. RATE OF STRAIN TO FAILURE: | 0.250 MPa/s |
| MAXIMUM COMPRESSIVE LOAD: | 94.3 kN |
| UNCONFINED COMPRESSIVE STRENGTH: | 54.4 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 NSC20-2 Run 1

UNCONFINED COMPRESSION TEST REPORT

ASTM D7012-14

| | | | |
|---------------|-------------------------------|--------------|-----------|
| CLIENT: | Thurber Engineering (Ottawa) | FILE NUMBER: | 24726 |
| PROJECT NAME: | Highway 17 Twinning - Renfrew | REPORT DATE: | 18-Jul-24 |
| BOREHOLE No.: | NSC20-3 | TEST DATE: | 9-May-24 |
| SAMPLE No.: | Run 3 | | |
| SAMPLE DEPTH: | 15.42 m | | |
| DESCRIPTION: | Marble | | |

| | | | |
|--|--------|-----------------------------------|-------|
| Avg. Height (cm): | 9.5 | Weight (g): | 470.3 |
| Avg. Diameter (cm): | 4.7 | Wet Density (kg/m ³): | 2,853 |
| H. to Dia. Ratio**: | 2:1 | Dry Density (kg/m ³): | 2,853 |
| Cross Sectional Area (cm ²): | 17.35 | Moisture Content* (%): | N/A |
| Sample Volume (cm ³): | 164.82 | | |

ORIGINAL SPECIMEN



FRACTURED SPECIMEN



| | |
|----------------------------------|-------------|
| AVG. RATE OF STRAIN TO FAILURE: | 0.250 MPa/s |
| MAXIMUM COMPRESSIVE LOAD: | 113.4 kN |
| UNCONFINED COMPRESSIVE STRENGTH: | 65.4 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 NSC20-3 Run 3

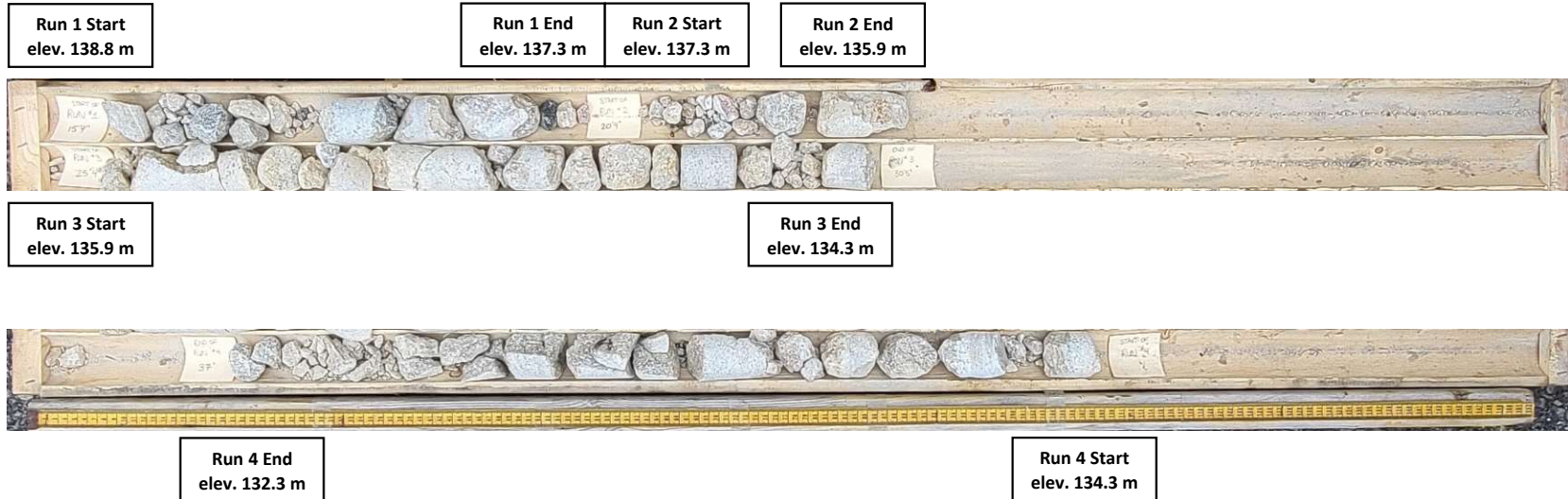
Borehole NSC20-1

Run 1, 2, 3 and 4

Depth 4.8 to 11.3 m

Elevation 138.8 to 132.3 m

Dry Sample

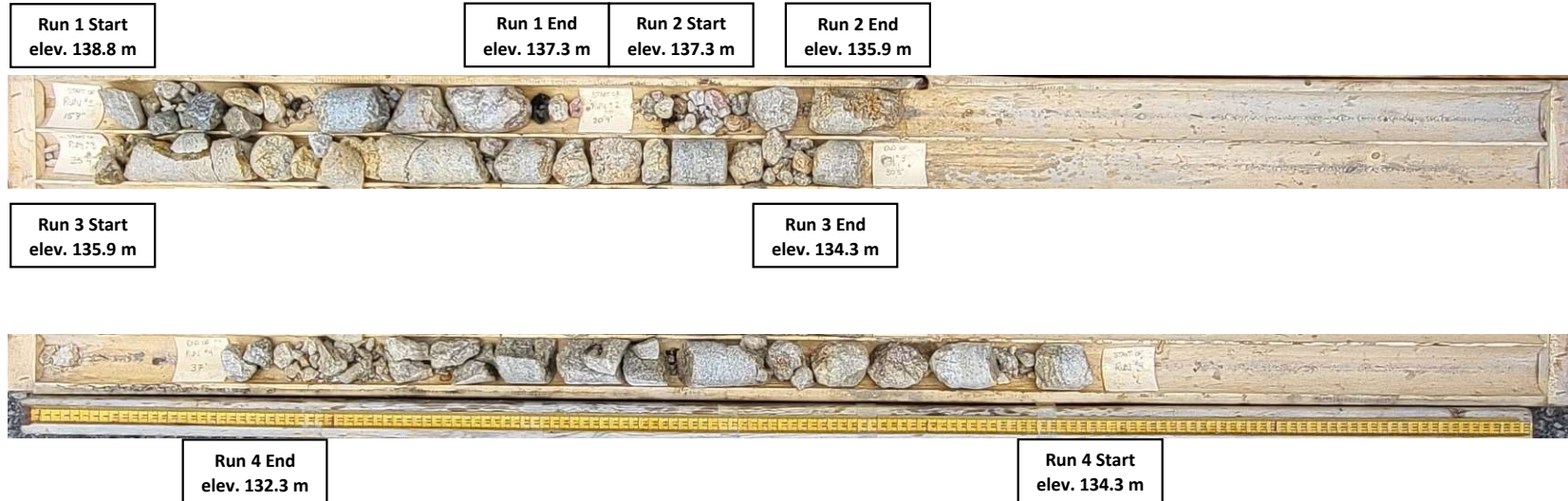


THURBER ENGINEERING LTD.

Foundation Investigation
Culvert 20.20N (Hwy 17, Sta. 24+900)
Renfrew, Ontario

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

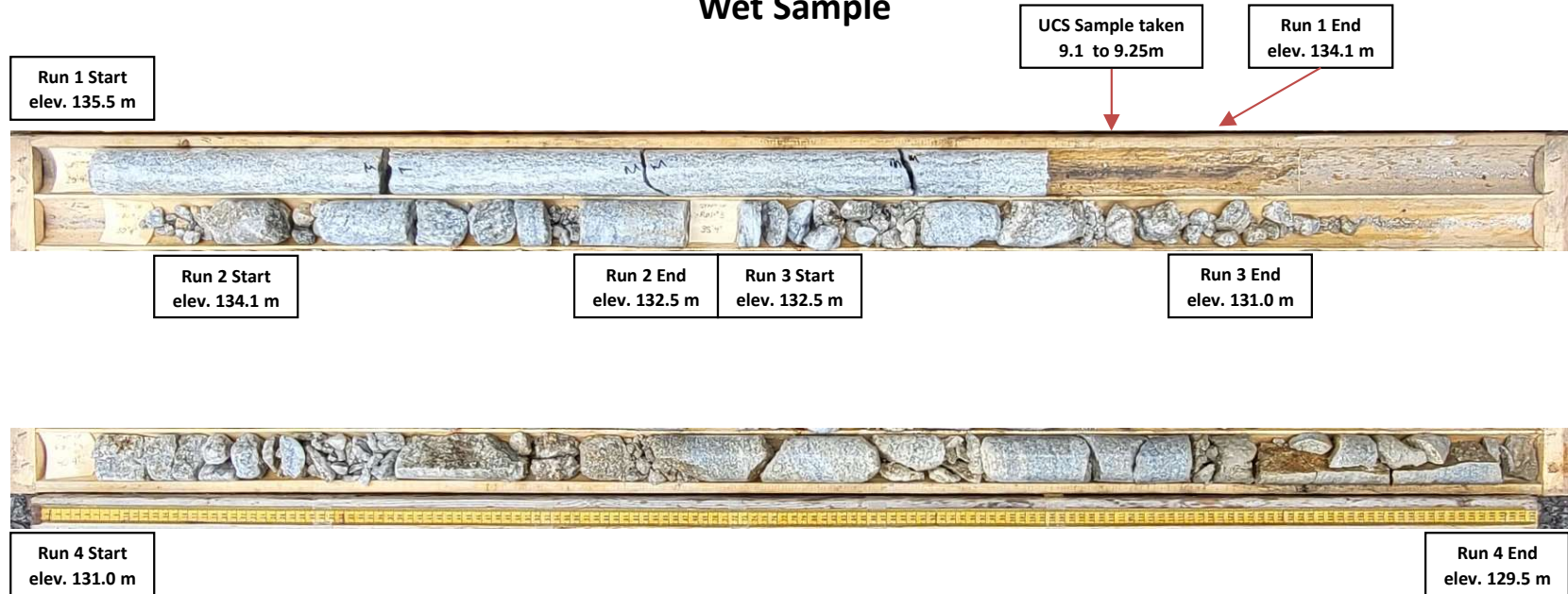
Borehole NSC20-1
Run 1, 2, 3 and 4
Depth 4.8 to 11.3 m
Elevation 138.8 to 132.3 m
Wet Sample



Borehole NSC20-2
Run 1, 2, 3 and 4
Depth 7.8 to 13.8 m
Elevation 135.5 to 129.5 m
Dry Sample



Borehole NSC20-2
Run 1, 2, 3 and 4
Depth 7.8 to 13.8 m
Elevation 135.5 to 129.5 m
Wet Sample



Borehole NSC20-2

Run 5

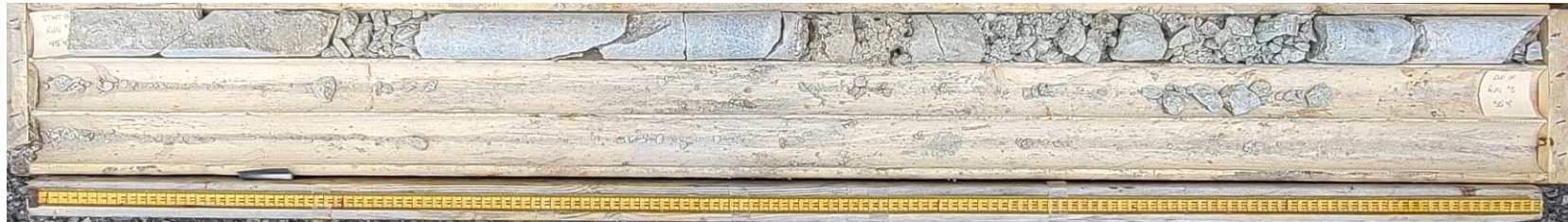
Depth 13.8 to 15.3 m

Elevation 129.5 to 128.0 m

Dry Sample

Run 5 Start
elev. 129.5 m

Run 5 End
elev. 128.0 m



THURBER ENGINEERING LTD.

Foundation Investigation
Culvert 20.20N (Hwy 17, Sta. 24+900)
Renfrew, Ontario

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

Borehole NSC20-2

Run 5

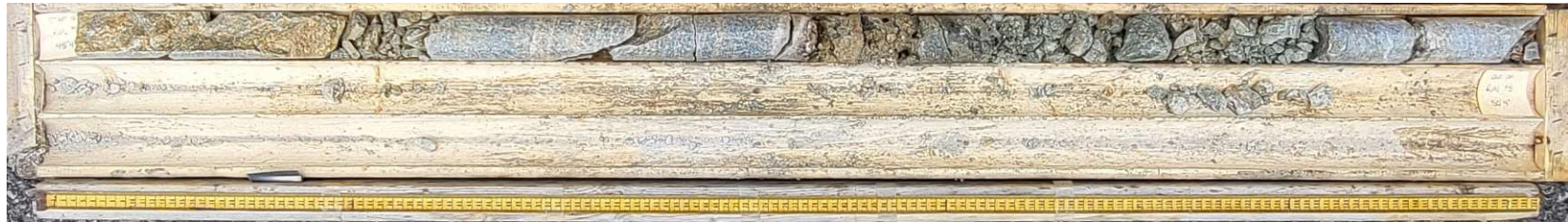
Depth 13.8 to 15.3 m

Elevation 129.5 to 128.0 m

Wet Sample

Run 5 Start
elev. 129.5 m

Run 5 End
elev. 128.0 m



THURBER ENGINEERING LTD.

Foundation Investigation
Culvert 20.20N (Hwy 17, Sta. 24+900)
Renfrew, Ontario

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

Borehole NSC20-3

Run 1, 2 and 3

Depth 12.7 to 16.6 m

Elevation 135.9 to 132.0 m

Dry Sample

Run 1 Start
elev. 135.9 m

Run 1 End
elev. 134.7 m

Run 2 Start
elev. 134.7 m



Run 2 End
elev. 136.6 m

Run 3 Start
elev. 136.6 m



Run 3 End
elev. 132.0 m



THURBER ENGINEERING LTD.

Foundation Investigation
Culvert 20.20N (Hwy 17, Sta. 24+900)
Renfrew, Ontario

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

Borehole NSC20-3

Run 1, 2 and 3

Depth 12.7 to 16.6 m

Elevation 135.9 to 132.0 m

Wet Sample

Run 1 Start
elev. 135.9 m

Run 1 End
elev. 134.7 m

Run 2 Start
elev. 134.7 m



Run 2 End
elev. 136.6 m

Run 3 Start
elev. 136.6 m



Run 3 End
elev. 132.0 m



THURBER ENGINEERING LTD.

Foundation Investigation
Culvert 20.20N (Hwy 17, Sta. 24+900)
Renfrew, Ontario

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



Appendix C.2

Analytical Testing Results

Certificate of Analysis

Report Date: 18-Apr-2024

Client: Thurber Engineering Ltd.

Order Date: 12-Apr-2024

Client PO: Highway 17 Renfrew, Various Sites

Project Description: 24726 task 700.706a

| | | | | | | | | |
|--------------------------|---------------|--------------|------------------------|---------------------------|-------------------|-------------------------|---|---|
| | | Client ID: | BON24-2 SS4 10'-12' | NSC20-2 SS2A 2'6"-3'3" | SC10-1 SS2B 3'-4' | SC10-4 SS2 2'6"-4'6" | | |
| | | Sample Date: | 09-Apr-24 09:00 | 02-Apr-24 09:00 | 21-Mar-24 09:00 | 04-Apr-24 09:00 | - | - |
| | | Sample ID: | 2415421-05 | 2415421-06 | 2415421-07 | 2415421-08 | | |
| | | Matrix: | Soil | Soil | Soil | Soil | | |
| | | MDL/Units | | | | | | |
| Physical Characteristics | | | | | | | | |
| % Solids | 0.1 % by Wt. | | 72.6 | 69.1 | 73.2 | 72.5 | - | - |
| General Inorganics | | | | | | | | |
| Conductivity | 5 uS/cm | | 286 | 203 | 316 | 247 | - | - |
| pH | 0.05 pH Units | | 6.79 | 6.65 | 6.95 | 6.84 | - | - |
| Resistivity | 0.1 Ohm.m | | 35.0 | 49.2 | 31.6 | 40.5 | - | - |
| Anions | | | | | | | | |
| Chloride | 10 ug/g | | 12 | 37 | 97 | 27 | - | - |
| Sulphate | 10 ug/g | | 24 | 21 | 44 | <10 | - | - |

**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

19-April-2024

Date Rec. : 16 April 2024
LR Report: CA12714-APR24
Reference: Project#: 2415421

Copy: #1

CERTIFICATE OF ANALYSIS

Final Report

| Sample ID | Sample Date & Time | Sulphide (Na ₂ CO ₃) % |
|-------------------------------------|----------------------|---|
| 1: Analysis Start Date | | 19-Apr-24 |
| 2: Analysis Start Time | | 13:06 |
| 3: Analysis Completed Date | | 19-Apr-24 |
| 4: Analysis Completed Time | | 13:12 |
| 5: RL | | 0.01 |
| 6: SC10-3 SS3A 5' 0" | 11-Mar-24 | < 0.01 |
| 7: SC23-2 SS5 10' 12" | 13-Mar-24 | 0.83 |
| 8: DOC23-1 SS7, 15' 17" | 11-Mar-24 | 0.01 |
| 9: OBR23-1 SS16 40' 50" | 27-Mar-24 | < 0.01 |
| 10: DON24-2 SS4 10' 12" | 09-Apr-24 | < 0.01 |
| 11: NSC20-2 SS2A 2'6"-3'3" | 02-Apr-24 | < 0.01 |
| 12: SC10-1 SS2B 3' 4" | 21-Mar-24 | < 0.01 |
| 13: SC10-4 SS2 2'0"-4'0" | 04-Apr-24 | < 0.01 |

RL - SGS Reporting Limit

Note: Samples taken March 11 and 13th were past the 28 day holding time for Sulphide analysis when received; result may be unreliable. Processed past holding time as per client's instructions.

Kimberley Didsbury
Project Specialist,
Environment, Health & Safety



Appendix D.

Site Photographs



Photo 1. Looking north along fence line and trees (March 12, 2024)



Photo 2. Looking south at culvert inlet (March 12, 2024)



Photo 3. Gas line sign at approx. 100 m northwest of the site (July 26, 2024)



Photo 4. Looking south at the well protection for Borehole NSC20-1 (April 02, 2024)



Photo 5. Looking southwest along silt fence near to the culvert site (March 12, 2024)

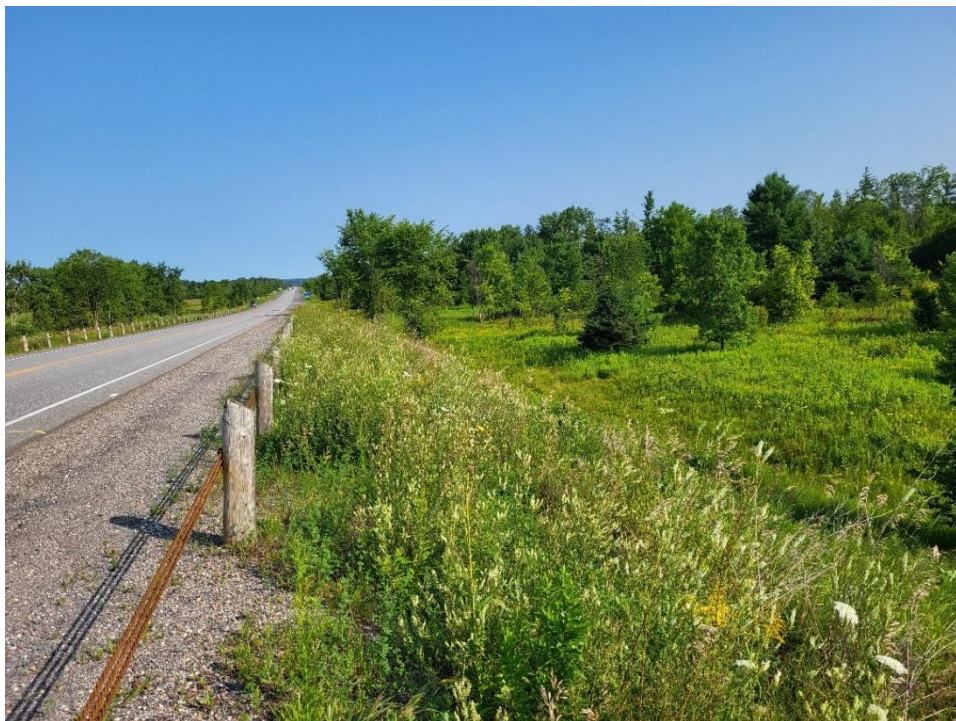


Photo 6. Looking northwest along Highway 17 westbound embankment (July 26, 2024)



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: 45.459N 76.612W

User File Reference: Culvert 20N, Highway 17, Sta. 24+936

2024-07-27 18:08 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) | 0.354 | 0.182 | 0.104 | 0.031 |
| Sa (0.1) | 0.420 | 0.227 | 0.137 | 0.045 |
| Sa (0.2) | 0.351 | 0.197 | 0.123 | 0.043 |
| Sa (0.3) | 0.267 | 0.154 | 0.098 | 0.035 |
| Sa (0.5) | 0.191 | 0.112 | 0.072 | 0.026 |
| Sa (1.0) | 0.098 | 0.059 | 0.038 | 0.013 |
| Sa (2.0) | 0.047 | 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) | 0.226 | 0.124 | 0.075 | 0.025 |
| PGV (m/s) | 0.160 | 0.090 | 0.055 | 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

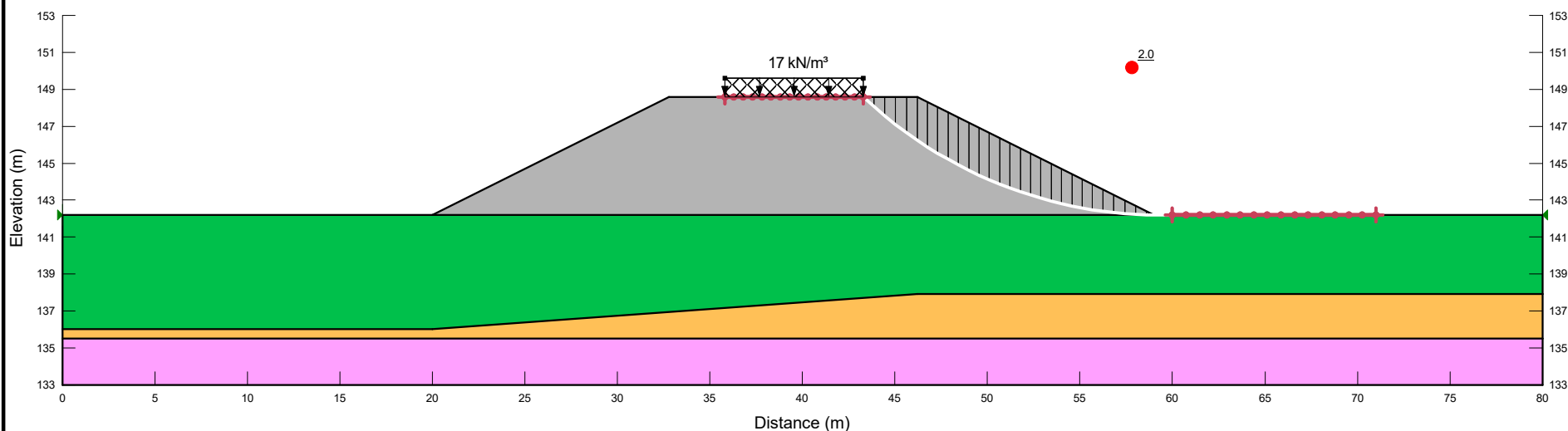
| | Pipe Culverts | Open-Bottom Box Culverts | Closed-Bottom Box Culverts |
|---------------------------|---|--|---|
| Advantages | <p>Relatively expedient installation if precast units are used.</p> <p>Smaller magnitude of settlement than open footing culvert due to lower bearing stress on subgrade</p> | <p>Readily encompasses natural substrate. Preferable from environmental perspective</p> <p>Possibility to maintain work zone to span the existing culvert; however, the replacement would need to be significantly wider than existing to allow for foundation excavation without conflict with existing pipe.</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 may require bracing, anchors and/or rakers</p> <p>Difficult to include natural substrate.</p> | <p>Require protection system for construction of foundations</p> <p>Protection system may 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>Protection system may require bracing, anchors and/or rakers</p> |
| Risks/Consequences | Some risk of basal disturbance during excavation due to depth of excavation below water table. | Increased risk of basal disturbance during footing excavation due to depth of excavation below water table. | Some risk of basal disturbance during 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) | Undrained Shear Strength (kPa) | Piezometric Surface | Effective Cohesion (kPa) | Effective Friction Angle (°) |
|-------|----------------------------|--------------------------------|---------------------|--------------------------------|---------------------|--------------------------|------------------------------|
| ■ | a) Clayey Silt (Undrained) | Undrained (Phi=0) | 17.5 | 75 | 1 | | |
| ■ | c) Silty Sand | Mohr-Coulomb | 19 | | 1 | 0 | 30 |
| ■ | d) Bedrock | Bedrock (Impenetrable) | | | 1 | | |
| ■ | e) SSM | Mohr-Coulomb | 21 | | 1 | 0 | 32 |

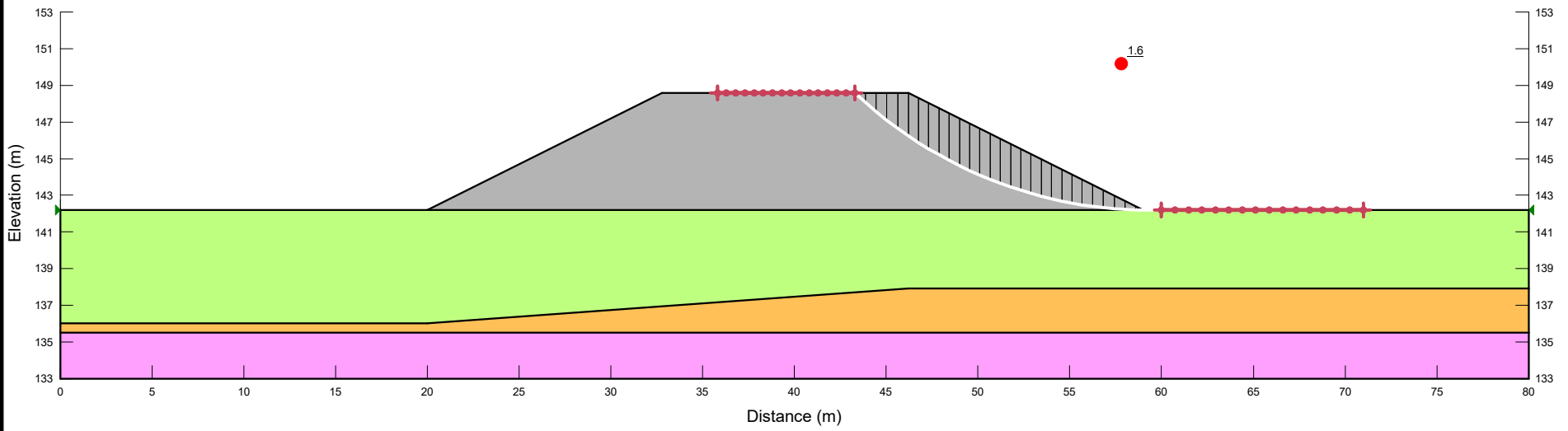


| | | |
|--|-------------------------------------|----------------|
| Project 24726 - Hwy 17, Sta 24+936, Culvert 20N | | |
| Analysis a1) Temporary (traffic), short term, static, undrained | | |
| Seismic Coefficient H: g, V: g | Last Run 2024-11-28, 03:01:59 PM | Scale 1:340 |

Additional Details
 Name: a) 2.0H:1V SSM embankment
 Comments:
 Method: Morgenstern-Price, Half-Sine
 Minimum Slip Surface Depth: 1.5 m
 Entry: (43.3, 148.6) m, Exit: (60, 142.2) m
 Center: (59.353825, 165.50217) m, Radius: 23.311127 m

Figure G1-1

| Color | Name | Slope Stability Material Model | Unit Weight (kN/m³) | Effective Cohesion (kPa) | Effective Friction Angle (°) | Piezometric Surface |
|--|--------------------------|--------------------------------|---------------------|--------------------------|------------------------------|---------------------|
| ■ | b) Clayey Silt (Drained) | Mohr-Coulomb | 17.5 | 5 | 27 | 1 |
| ■ | c) Silty Sand | Mohr-Coulomb | 19 | 0 | 30 | 1 |
| ■ | d) Bedrock | Bedrock (Impenetrable) | | | | 1 |
| ■ | e) SSM | Mohr-Coulomb | 21 | 0 | 32 | 1 |




| | | | | |
|---|---|-------------------------|--------------------------------------|---|
|  THURBER | Project | | Additional Details | |
| | 24726 - Hwy 17, Sta 24+936, Culvert 20N | | Name: a) 2.0H:1V SSM embankment | |
| | Analysis | | Comments: | |
| | a2) Permanent, long term, static, drained | | Method: Morgenstern-Price, Half-Sine | |
| Seismic Coefficient | | Last Run | | Minimum Slip Surface Depth: 1.5 m Entry: (43.3, 148.6) m, Exit: (60, 142.2) m Center: (59.353825, 165.50217) m, Radius: 23.311127 m |
| H: g, V: g | | 2024-11-28, 03:01:58 PM | | |
| | | Scale | | |
| | | 1:340 | | |

Figure G1-2

Figure G1-2



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 Sheeting |
| OPSS.PROV 517 | Construction Specification for Dewatering |
| OPSS.PROV 539 | Construction Specification for Temporary Protection Systems |
| OPSS.PROV 803 | Vegetative Cover |
| OPSS.PROV 804 | Construction Specification for Seed and Cover |
| OPSS.PROV 805 | Construction Specification for Temporary Erosion and Sediment Control Measures |
| OPSS.PROV 902 | Construction Specification for Excavating and Backfilling Structures |
| OPSS.PROV 1010 | Material Specification for Aggregates Base, Subbase, Select Subgrade, and Backfill Material |
| OPSS.PROV 1860 | Material Specification for Geotextiles |
| OPSD 208.010 | Benching of Earth Slopes |
| OPSD 219.110 | Light-Duty Silt Fence Barrier |
| OPSD 802.031 | Rigid Pipe Bedding, Cover and Backfill, Type 3 Soil, Earth Excavation |
| OPSD 803.010 | Backfill and Cover for Concrete Culverts With Spans Less Than or Equal To 3.0 m |
| OPSD 803.031 | Frost Treatment - Pipe Culverts, Frost Penetration Line Between Top of Pipe and Bedding Grade |
| OPSD 810.010 | General Rip-Rap Layout for Sewer and Culvert Outlets |
| OPSD 3090.101 | Foundation Frost Depths for Southern Ontario |
| OPSD 3101.150 | Walls Abutment, Backfill Minimum Granular Requirement |
| SP 110S06 | Amendment to OPSS 1010, April 2013 |
| SP 517F01 | Amendment to OPSS 517 - Construction Specification for Dewatering |



2. Suggested wording for NSSPs

“Protection of Sensitive Foundation Soils”

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

“Structural Backfill”

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

“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 existing fill and 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. 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.