



THURBER ENGINEERING LTD.

**FINAL
FOUNDATION INVESTIGATION AND DESIGN REPORT
HIGHWAY 17 CULVERT AT STATION 14+332
DULHUT TOWNSHIP, ONTARIO
ASSIGNMENT NO.: 5020-E-0025
GWP 5207-18-00**

GEOCRES NO.: 41N00-038

Location: Lat: 47.845380°, Long: -84.886579°

Client Name: AECOM Canada Ltd.

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PART 1. FACTUAL INFORMATION

1. INTRODUCTION

This section of the report presents the factual findings obtained from a foundation investigation conducted by Thurber Engineering Ltd. (Thurber) for the replacement of the culvert that crosses Highway 17 at Sta. 14+332 in Dulhut Township, Ontario. Thurber carried out the foundation investigation as a subconsultant to AECOM Canada Ltd. (AECOM) under Agreement No. 5020-E-0025, Change Order 1.

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

It is a condition of this report that Thurber's performance of its professional services is subject to the attached Statement of Limitations and Conditions.

2. SITE DESCRIPTION

2.1 General

The culvert site crosses Highway 17 approximately 17.6 km south of the junction between Highway 17 and Highway 101. For project purposes, Highway 17 is herein described as oriented north-south, and the culvert is described as oriented east-west.

In the area of the culvert, 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 290.0 m. The culvert is located within a section of highway with a superelevated curve, and the highway alignment continues to curve east both north and south of the culvert site. The shoulders to the highway

are paved and steel cable guiderails on wooden posts are present along both northbound and southbound shoulders of the highway. The 2023 traffic volume projection for this section of Highway 17 is understood to be 2,200 AADT.

The existing culvert is reported in AutoCAD drawings provided by AECOM to be 2,400 mm diameter, 37.28 m long, corrugated steel pipe (CSP) culvert with a slight skew to the highway alignment. The culvert has a relatively flat gradient with the invert of the culvert near elevations 281.55 m and 281.20 m at the inlet and outlet, respectively. The water flows through the culvert from east to west. Approximately 0.6 m of ponded water was present at the culvert outlet and a beaver dam was present east of the culvert inlet. Ponded water, presumably due to observed beaver activity, was also observed at the culvert located approximately 90 m to the north of the site.

The cover above the existing culvert is approximately 6.1 m at the highway centerline. Embankment side slopes, in the vicinity of the culvert, are generally inclined at approximately 1.9H:1V but are locally as steep as 1.3H:1V. The existing highway 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 area adjacent to the highway is undeveloped and densely vegetated with mixed forests of coniferous and some deciduous trees and shrubs. Overhead utility lines were not present. Bedrock outcrops are present northwest of the culvert. Lake Superior is located approximately 1.9 km west of the highway alignment.

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

According to Crins et al. 2009¹ the project area is described as Ecoregion 4E (Lake Temagami Ecoregion) within the Ontario Shield Ecozone. According to Wester et al. 2018² the ecoregion is subdivided into Ecodistrict 4E-1 (Michipicoten Ecodistrict). The project area is located in the north part of the ecodistrict, which is characterized by glaciofluvial material sediments and morainal deposits overlying Precambrian bedrock. Bedrock Geology Map (MRD126)³ indicates the site is underlain by mafic to intermediate metavolcanic rocks: basalt and andesite.

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

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

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

2.3 Existing Information

A historical foundation investigation report was not available for this site within the online Geocres Library.

Base plan mapping was provided by AECOM for the preparation of this report.

3. SITE INVESTIGATION AND FIELD TESTING

The foundation investigation and field-testing program was carried out between May 9 and July 7th, 2023, and consisted of two on-road boreholes identified as 23-401 and 23-402 and two off-road boreholes identified as 23-403 and 23-404. The on-road boreholes were advanced with a CME 75 truck mounted drill rig utilizing NW casing and coring techniques. The off-road boreholes were advanced with portable drilling equipment. Prior to commencement of drilling, utility clearances were obtained in the vicinity of the borehole locations.

A summary of the borehole coordinates, elevations, and termination depths is provided within the table below. The as-drilled borehole elevations were surveyed by Thurber with a surveyor's level with a reported vertical accuracy of +/- 1.5 mm and were measured relative to BM HCP 178 (Elevation 288.9 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 AECOM. 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 13.

Table 3-1 Borehole Summary

BOREHOLE NO.	DRILLED LOCATION	NORTHING (m)	EASTING (m)	GROUND SURFACE ELEVATION (m)	TERMINATION DEPTH (m)
23-401	Northbound lane	5 301 090.8	238 450.4	289.7	13.2
23-402	Southbound lane	5 301 074.2	238 446.1	290.3	14.7
23-403	West embankment toe	5 301 092.9	238 425.9	282.0	4.0
23-404	East embankment toe	5 301 092.9	238 469.1	283.5	4.2

The boreholes were advanced to depths ranging from 4.0 to 14.7 m (base elev. 279.3 to 275.6 m). 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. A third-weight hammer was used for SPT testing within Boreholes 23-403 and 23-404, and a hammer weight correction has been applied for the N-values on the borehole record. It is noted that an automatic

hammer could not be used with the portable drill thus the SPT N-values from the portable drilling equipment are considered to be less reliable.

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

A 32 mm diameter well was installed in each of Boreholes 23-403 and 23-404 to allow for measurements of the groundwater level after drilling. The details for the well are illustrated on the respective Record of Borehole sheets provided in Appendix B.

Following completion of the field investigation, the wells and boreholes were decommissioned in general in accordance with O.Reg. 903, as amended. Boreholes 23-401 and 23-402 were capped with cold patch asphalt to reinstate the pavement surface.

4. LABORATORY TESTING

Laboratory testing was selected in general 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. Recovered soil samples were selected for grain size distribution and, where appropriate, Atterberg Limit testing in accordance with MTO and ASTM standards. The rock cores were photographed, and the total core recovery (TCR), solid core recovery (SCR), and rock quality designation (RQD) were measured, and unconfined compressive strength (UCS) testing was carried out on select intact bedrock cores. The results of these tests are summarized on the Record of Borehole sheets included in Appendix B.

One soil sample was selected and submitted for analytical testing of corrosivity parameters and sulphate content.

All laboratory test results from the field investigation are provided in Appendix C.

5. DESCRIPTION OF SUBSURFACE CONDITIONS

Details of the encountered soil stratigraphy are presented on the Record of Borehole sheets included in Appendix B and on 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 Record

of Borehole sheets takes precedence over this general description for interpretation of the site conditions. It must be recognized that the soil and groundwater conditions will vary between and beyond borehole locations. Soil classification is in general accordance with ASTM D2487 with the description of cohesive soils and secondary components as outlined in the MTO Guideline for Foundation Engineering Services Manual (April 2022).

In general, the encountered stratigraphy consists of sand and gravel fill overlaying rockfill overlying silty sand to sand fill over bedrock. Native deposits of silty sand to gravelly sand were encountered at the ground surface in the off-road boreholes.

5.1 Surficial Materials

5.1.1 Asphalt

Asphalt was encountered at the ground surface in both on-road boreholes. The asphalt was measured to have a thickness of 115 mm to 125 mm.

5.2 Fill

5.2.1 Sand and Gravel Fill

A fill layer consisting of sand and gravel was encountered beneath the asphalt in Boreholes 23-401 and 23-402. The fill layer was 0.6 to 0.7 m thick (base elev. 289.5 m to 289.0 m). An SPT N-value of 78 blows was recorded, indicating a very dense relative density.

Moisture contents ranging from 4 to 7% were recorded. The results of gradation analyses completed on two samples of the layer are illustrated in Figure C1 of Appendix C. The results of the tests are summarized in the table below and on the Record of Borehole sheets in Appendix B.

SOIL PARTICLE	PERCENTAGE (%)
Gravel	40 – 52
Sand	45 – 55
Silt	3 – 5
Clay	

5.2.2 Rock Fill

A layer of gravel to cobble sized rockfill with inferred boulders was encountered below the sand and gravel fill in Boreholes 23-401 and 23-402. NQ coring techniques were required to penetrate

the rockfill layer resulting in less retained sample. Boulders were cored in the lower portion of Borehole 23-402. The rockfill layer was 8.0 to 8.6 m thick with an underside depth of 8.7 to 9.4 m (base elev. 281.0 to 280.9 m). SPT testing was attempted and, where obtainable, N-values ranged from 7 blows to refusal blow counts.

The recorded moisture contents of the non-cohesive infill ranged from 1 to 6%. A limited volume of sample was obtained, and the results of gradation analyses completed on two samples of the layer are illustrated in Figure C2 of Appendix C. The results of the tests are summarized in the table below and on the Record of Borehole sheets in Appendix B.

SOIL PARTICLE	PERCENTAGE (%)
Gravel	61 – 76
Sand	21 – 30
Silt	3 – 9
Clay	

5.2.3 Sand to Silty Sand Fill

A layer ranging from sand some gravel to silty sand some gravel was encountered below the rockfill in Boreholes 23-401 and 23-402. NQ coring techniques were required to penetrate this layer in Borehole 23-401, the layer is inferred to contain cobbles and boulders. The fill was 1.1 to 1.7 m thick with an underside depth of 9.8 to 11.1 m (base elev. 279.9 to 279.2 m). SPT N-values in the layer ranged from 14 to 17 blows, indicating a compact relative density. A refusal N-value was obtained in Borehole 23-401 which is attributed to the presence of cobbles/boulders.

The recorded moisture content ranged from 14 to 17%. The results of gradation analyses completed on two samples of the layer are illustrated in Figure C3 of Appendix C. The results of the tests are summarized in the table below and on the Record of Borehole sheets in Appendix B.

SOIL PARTICLE	PERCENTAGE (%)	
Gravel	19 – 20	
Sand	50 – 70	
Silt	29	10
Clay	2	

5.3 Sand

A deposit of sand some silt with organics was encountered below the ground surface in Borehole 23-403. NQ coring techniques were required to penetrate this layer, the layer is inferred to contain cobbles. The layer was 1.0 m thick (base elev. 281.0 m). An SPT N-value of 4 blows was recorded, indicating a loose relative density. A refusal N-value was also obtained but is attributed to the presence of cobbles.

A moisture content of 16% was recorded. The results of gradation analysis completed on a sample of the layer are illustrated in Figure C4 of Appendix C. The results of the tests are summarized in the table below and on the Record of Borehole sheets in Appendix B.

SOIL PARTICLE	PERCENTAGE (%)
Gravel	3
Sand	85
Silt	10
Clay	2

5.4 Gravelly Sand to Gravel

A deposit of gravelly sand some silt transitioning to gravel interbedded with sand was encountered below the ground surface in Borehole 23-404. NQ coring techniques were required to penetrate this layer, the layer is inferred to contain cobbles. The layer was 1.5 m thick (base elev. 282.0 m). An SPT N-value of 9 blows was recorded, indicating a loose relative density. A refusal N-value was obtained but is attributed to the presence of cobbles.

The moisture content of the samples tested ranged from 4 to 6%. The results of gradation analysis completed on a sample of the layer are illustrated in Figure C5 of Appendix C. The results of the tests are summarized in the table below and on the Record of Borehole sheets in Appendix B.

SOIL PARTICLE	PERCENTAGE (%)
Gravel	31
Sand	57
Silt	12
Clay	

5.5 Bedrock

Bedrock outcrops were observed in close proximity to the culvert site. Bedrock was proven by coring in all four boreholes. The depth to bedrock in the on-road boreholes was 9.8 m and 11.1 m (elev. 279.9 m and 279.2 m).

The bedrock encountered consisted of fine grained, dark greenish grey, very strong Greenschist containing calcite inclusions. Photographs of the bedrock cores are provided in Appendix C. The rock core quality measurements are summarized in the following table.

Table 5-1 Bedrock Details

PARAMETER	RANGE
Total Core Recovery (TCR), %	83 - 100
Solid Core Recovery (SCR), %	33 - 92
Rock Quality Designation (RQD), %	22 - 100
Fracture Index (fractures per 0.3 m) ⁽¹⁾	0 - >10
Unconfined Compressive Strength Testing (MPa) ⁽²⁾	152

Notes: (1) Indicated as "FI" on Borehole Logs
(2) Sample tested from Borehole 23-403, Run 1.

An RQD value encountered in the Borehole 23-402 was 22% indicating a very poor quality and the remaining RQD values were 48 to 100% generally indicating a bedrock of fair to excellent quality (CFEM, 2006). The result of an unconfined compressive strength testing (UCS) was 152 MPa, indicating that the tested sample of the bedrock is very strong (CFEM, 2006). The results of the UCS testing are included in Appendix C.

5.6 Groundwater Level

The measured groundwater levels from within the wells installed in Boreholes 23-403 and 23-404 are summarized in the following table.

A representative open-hole groundwater level measurement was not obtained due to the introduction of water during drilling in Boreholes 23-401 and 23-402.



Table 5-2 Measured Water Levels

Borehole	Bottom of Screen Depth /Elevation (m)	Soil in Zone of Screen	Groundwater Level		Date of Measurement
			Depth (mbgs)	Elevation (m)	
23-403	3.9 / 278.1	Bedrock	0.7	281.3	2023 07 11
			0.7	281.3	2023 07 12
			0.7	281.3	2023 07 15
23-404	4.1 / 279.4	Bedrock	0.8	282.7	2023 06 11
			1.7	281.8	2023 07 06
			2.1	281.4	2023 07 08
			1.8	281.7	2023 07 12
			1.8	281.7	2023 07 13
			1.8	281.7	2023 07 15

Ponded water was present near the culvert outlet. The surface water depth was recorded to be 0.6 m at the time of the field investigation.

It should be noted that the values shown above are considered short-term readings and may not reflect groundwater levels at the time of construction. Seasonal fluctuations of the groundwater level are to be expected. In particular, the groundwater level may be at a higher elevation after periods of significant and/or prolonged precipitation events. The water level should also be expected to change based on the presence or removal of the beaver dam.

A Single Well Response Test (SWRT), or “slug test”, was carried out on July 15, 2023 in the monitoring wells installed in BH 23-403 and 23-404 by lowering the water level within the monitoring well and recording the recovery of the water level over time with a data logger. The slug tests were completed and analyzed using the Hvorslev method and the plots of the slug test results are included in Appendix B. The hydraulic conductivity values calculated from the in-situ slug tests are summarized in the following table.

Table 5-3 Single Well Response Test Results

Borehole /Monitoring Well	Bottom of Screen Depth /Elevation (m)	Soil in Zone of Screen	Estimated Hydraulic Conductivity (m/s)
23-403	3.9 / 278.1	Bedrock	3.1×10^{-8}
23-404	4.1 / 279.4	Bedrock	2.0×10^{-7}

The overburden thicknesses were too thin to adequately allow for the installation of a well; however, it should be expected that variations in hydraulic conductivity will exist within the various soil deposits that were encountered.

Both wells were decommissioned following the completion of the testing on July 15, 2023.

5.7 Analytical Testing

One soil sample was submitted for analytical testing. The analysis results are included in Appendix C and are summarized in Table 5-4.

Table 5-4 Analytical Test Results

BOREHOLE	23-404
SAMPLE	SS1
DEPTH (ft/m)	0'0" – 1'0" 0 – 0.3
ELEVATION (m)	283.2
SOIL TYPE	Gravelly Sand
CONDUCTIVITY (µS/cm)	75
pH	4.16
RESISTIVITY (Ohm-cm)	13,300
CHLORIDE (µg/g)	12
SULPHATE (µg/g)	15
SULPHIDE (%)	< 0.04

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, and Ohlmann Geotechnical Services Inc. of Almonte, 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 J. Provost Contracting of Wawa, Ontario. The field investigation was supervised on a full-time basis by Mr. I. Khan, EIT and Mr. A. Simpson, EIT. Overall supervision of the field investigation program was provided by Mr. A. de Oliveira, EIT.



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Routine geotechnical laboratory testing was completed by Thurber's laboratory in Ottawa. Analytical testing was completed by Paracel Laboratories Ltd. in Ottawa. UCS testing was carried out by Stantec.

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

Thurber Engineering Ltd.
Report Prepared By:

Anderson de Oliveira, M.A.Sc., EIT
Engineering Intern



Stephen Peters, M.A.Sc., P.Eng.
Associate | Geotechnical Engineer



Fred Griffiths, Ph.D., P.Eng.
Designated Principal Contact
Senior Geotechnical Engineer



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PART 2. ENGINEERING DISCUSSION AND RECOMMENDATIONS

7. GENERAL

This section of the report provides an interpretation of the factual data from Part 1 of this report and presents foundation design recommendations to assist the project team in the design of the replacement of the culvert located on Highway 17 near Station 14+332 in the Township of Dulhut within the District of Michipicoten, Ontario. Thurber Engineering Limited (Thurber) carried out the current field investigation as a sub-consultant to AECOM Canada Ltd. (AECOM) under Agreement No. 5020-E-0025, Change Order 1. The discussion and recommendations presented in this report are based on information provided by AECOM and the factual data obtained during the current field investigation.

This foundation investigation and design report with the interpretation and recommendations are intended for the use of the Ministry of Transportation Ontario and their designer, AECOM Canada Ltd., and shall not be used or relied upon for any other purposes or by any other parties including the construction or design-build contractor. Contractors must make their own interpretation based on the factual data in Part 1 of the report. Where comments are made on construction, they are provided only in order to highlight those aspects which could affect the design of the project. Those requiring information on aspects of construction must make their own interpretation of the factual information provided as such interpretation may affect equipment selection, proposed construction methods, and scheduling and the like.

It is a condition of this report that Thurber's performance of its professional services is subject to the attached Statement of Limitations and Conditions.

7.1 Background Information

The culvert site is approximately 17.6 km south of the junction between Highway 17 and Highway 101 in Dulhut Township, Ontario. The road surface near the culvert is near elevation 290.0 m, and the invert of the culvert is near elevations 281.55 and 281.20 m at the inlet and

outlet, respectively. The cover above the existing culvert is approximately 6.1 m at the highway centerline. The ditch drainage flows through the culvert under the highway embankment from east to west. The existing culvert is reported in AutoCAD drawings by AECOM to be a 2400 mm diameter, 37.3 m long corrugated steel pipe (CSP) culvert.

In general terms, the encountered stratigraphy consists of sand with gravel fill overlying rockfill overlying silty sand to sand fill over bedrock. Native deposits of silty sand to gravelly sand were encountered at the ground surface in the off-road boreholes. Ponded surface water was present near the culvert outlet. Groundwater was recorded between elev. 281.8 to 281.3 m.

7.2 Proposed Work

The proposed works for this non-structural culvert are indicated in the AECOM Foundations Investigations Program Summary Memorandum dated July 2022 with the preferred approach to be insertion of a liner. However, should a liner not satisfy project needs, consideration should be given to culvert replacement with half and half staging. AECOM indicated that a new culvert alignment would be located to the south of existing.

Contract Drawings were not available at the time of writing this report but it is assumed the replacement culvert will have similar alignment, invert and length as the existing.

7.3 Applicable Codes and Design Considerations

The geotechnical assessment presented below has been prepared based on the available data regarding the proposed work, existing ground conditions and in accordance with the Canadian Highway Bridge Design Code (CHBDC), version CSA S6-19. The importance category and consequence classification are defined by the Regulatory Authority which, in this case, is the Ministry of Transportation, Ontario (MTO).

It is understood that the culvert is to be designed to the “Major Route” importance category.

It is understood that the new culvert would have a consequence classification of *Typical Consequence*, 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 this consequence classification changes, the geotechnical assessment and recommendations provided within this report will need to be reviewed and revised.

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

The frost penetration depth and associated recommendations are provided in Section 10.4.

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 0.035g. This value is to be scaled by the $F(PGA)$ based on the *site-specific* Site Class, as discussed in Section 8.3.

8.2 Liquefaction Potential

The susceptibility of the cohesionless soils below the culvert invert to experience liquefaction was assessed using the SPT data following the simplified method for cohesionless soil as outlined in Boulanger and Idriss (2014)⁵. The cohesionless foundation soils are not considered to be susceptible to liquefaction under the design earthquake.

8.3 CHBDC Seismic Site Classification and Performance Category

In accordance with the CHBDC, the selection of the seismic site classification is based on the nature of the soil deposits within the upper 30 m of the stratigraphy. As per Table 4.1 within Section 4.4.3.2 of the CHBDC, the site has been classified as a Seismic Site Class B.

The $F(PGA)$, as per Table 4.8 within Section 4.4.3.3 of the CHBDC, is equal to 0.87 for this site yielding a scaled *site-specific* Site Class B PGA of 0.031g.

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 $F(0.2)$, as per Table 4.2 within Section 4.4.3.3 of the CHBDC, is equal to 0.77 for this site yielding a scaled site specific $S_a(0.2)$ of 0.047. A Seismic Performance Category of 1 is applicable to this site based on Table 4.10 of the CHBDC.

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

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.

9. DESIGN OPTIONS

9.1 Culvert Type and Foundation Alternatives

Selection of the replacement 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. It is understood that lining the existing culvert is also being considered for this site. The options that have been considered from a foundation perspective are presented below:

- Closed Pipe (Concrete, HDPE, Steel)
Pipe culverts are considered a feasible option from a foundation engineering perspective. Open cut or trenchless installation methods are typically considered for pipe culverts.
- Open Bottom Culvert (Box, Arch)
An open bottom culvert would require a greater excavation depth than a closed bottom culvert to satisfy frost protection requirements, however footings constructed on the shallow bedrock would not be required to be founded at the depth of frost. Excavation to bedrock would lead to greater dewatering efforts to construct the culvert in the dry when compared to a closed bottom culvert. This option may not be considered appropriate for the anticipated size of culvert required at this site.
- Closed Bottom Culvert (Box)
A precast segmental 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 founding soils during installation, require less excavation depth than open bottom culverts leading to more manageable dewatering conditions.

Given the size of the replacement culvert and the presence of frequent cobbles and boulder sized rockfill and the relatively shallow and undulating bedrock, a culvert supported on deep foundations or a culvert consisting of precast concrete cap panels supported on parallel alignments of contiguous sheet piles are not feasible for this site.

A comparison of the alternatives, based on their respective advantages and disadvantages, is included in Appendix F. This report focuses on providing foundation recommendations on the design and construction of a pipe culvert or closed bottom concrete culvert replacement.

9.2 Construction Methodology Alternatives

For the proposed culvert replacement, construction methods that were considered are presented below. Common to all techniques discussed below is excavation through rockfill materials. It is understood that only a single lane of traffic is required to be maintained during replacement of the existing culvert. For an open cut through the embankment fill the side slopes of the open excavation for the culvert replacement should follow the recommendations of OSHA as outlined in Section 11.1. Alternatively, if space restrictions prohibit the use of slopes, a temporary protection system as per Section 11.2 should be used.

- Open Cut with Full Road Closure and Temporary Detour
Installation of a new culvert using open cut techniques and a full road closure would allow for an expedited construction schedule and could reduce costs associated with roadway protection. However, it is anticipated that an acceptable detour route is not available and therefore this option is not carried forward.
- Open Cut with Staged Temporary Widening or Detour Embankment
Installation of a new culvert using an open cut with a temporary widening to accommodate passage of traffic during construction is considered feasible from a foundation perspective. However, the available right-of-way, the curved highway alignment and the presence of adjacent rock cuts limit the feasibility of significant roadway widening and would need to be considered in design. Depending on the temporary alignment and length, an additional borehole investigation program may be required to determine the subsurface conditions along the potential detour alignment.
- Open Cut with Staged Construction and Temporary Protection System
Installation of a new culvert using an open cut staged replacement is considered feasible from a foundation perspective. The option would require roadway protection, as discussed further in Section 11.2, installed near the embankment centerline to maintain a single lane of traffic along the current highway embankment. The Contractor would need to consider the rockfill and shallow bedrock during the installation of roadway protection. To reduce lateral deflections, the TPS may need to include anchoring and/or bracing. The height of the TPS could be reduced if the road alignment constraints allowed for a temporary grade lowering to be included.
- Open Cut with Staged Construction and Temporary Grade Lowering
Installation of a new culvert using an open cut staged replacement with grade lowering to maintain movement of traffic within the existing embankment footprint is considered a



feasible option from a foundation perspective. It is noted that grade lowering in the order of 3.5 m would be required which would expose a rockfill surface. Over excavation would be required to allow placement of granulars for a temporary pavement structure. The excavation equipment should be selected based on the presence of rockfill. The curved highway alignment and the presence of adjacent rock cuts may limit the feasibility of significant roadway lowering and would need to be considered in design.

- Open Cut with Temporary Modular Bridge

Installation of a new culvert using an open cut with a temporary modular bridge (TMB) to provide a single lane of traffic passage over the open excavation is generally considered feasible. Additional boreholes would be required at the TMB abutment locations to provide foundation design recommendations. It is expected that this alternative would be more expensive than the other open cut options, therefore it is not recommended.

- Trenchless Techniques

Installation of a new culvert using trenchless techniques is not considered feasible due to the presence of cobble and boulder sized rockfill and the potential for encountering mixed face conditions comprising rock fill and sand to silty sand fill as well as possible bedrock. It is also noted that the groundwater level is partially within the tunnel zone adding further complications and risk to a trenchless approach.

9.3 Recommended Approach for Culvert Replacement

From a foundation engineering perspective, it is recommended that the existing culvert be replaced with either a pipe (flexible or rigid) or precast segmental closed box culvert using open cut staged construction and temporary protection systems (TPS). The height of the TPS could be reduced if the road alignment constraints allow a temporary grade lowering to be included.

10. OPEN CUT FOUNDATION DESIGN RECOMMENDATIONS

10.1 Foundation Bearing Resistances

It is understood that the replacement pipe or closed box culvert will be founded near the same invert elevation as the existing culvert. Therefore, it is anticipated that the underside of a culvert will be founded within rockfill and/or the underlying silty sand to sand fill. There is a potential that the culvert ends could encounter native silt sand to gravelly sand. Relatively shallow bedrock was encountered at all of the boreholes, therefore it is also possible that bedrock could be encountered along the alignment during culvert subgrade preparation.

The replacement culvert should be founded on a bedding layer (see Section 10.2). Subgrade preparation should follow the recommendations provided in Section 10.2.

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

10.1.1 Pipe Culvert

Bearing resistance values are not required for pipe culverts. However, a modulus of subgrade reaction of 30 MN/m³ can be used for a pipe culvert at this site if required. The value should be divided by the pipe diameter when estimating the soil's spring constant.

If a concrete pipe is selected, resistance to lateral forces/sliding resistance between concrete and the underlying granular bedding layer should be evaluated following the recommendations presented in Section 10.1.2.

10.1.2 Closed Box Culvert

A closed box culvert would not need to be founded below the depth of frost (see Section 10.4). For a box culvert with an exterior width of as much as 2.4 m founded on a properly prepared granular bedding layer, the design can be based on factored geotechnical resistance values as follows:

- Factored Geotechnical Resistance at ULS of 400 kPa
- Factored Geotechnical Resistance at SLS of 250 kPa

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)
 - $\phi_{gu} = 0.50$ (static analysis; *typical* degree of understanding)
 - $\phi_{gs} = 0.80$ (static analysis; *typical* degree of understanding)

The bearing resistance values are for vertical, concentric loading. In the case of eccentric or inclined loading, the bearing resistance must be adjusted in accordance with CHBDC Clause 6.10.2. Foundation settlement, based on the supplied SLS resistance, is expected to be as much as 25 mm. The bearing resistances provided above are based on the assumption that subgrade is prepared as recommended in Section 10.2.

Resistance to lateral forces/sliding resistance between precast concrete and the underlying Granular bedding (see Section 10.2) should be evaluated in accordance with the CHBDC

assuming an unfactored coefficient of 0.45 for precast concrete. A geotechnical resistance factor of 0.8 (ϕ_{gu}), as per Table 6.2 of the CHBDC (static analysis – typical understanding) should be applied to the sliding frictional capacity between concrete and Granular bedding.

10.2 Subgrade Preparation, Embedment, Bedding, Cover and Backfilling

“Granular A” and “Granular B Type II” in this section refer to OPSS Granular A or Granular B Type II meeting the specifications of OPSS.PROV 1010 and SP110S06. 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 culvert replacement should include excavation and removal of the existing culvert if replaced along the same alignment. It is understood that the replacement culvert may be placed south of the existing alignment, and therefore the existing culvert may be abandoned in place.

At the founding level, existing fill, soft/loose soils, disturbed soils, or otherwise deleterious materials encountered will need to be removed down to competent inorganic soils. Construction traffic should not travel on the exposed subgrade. As soon as practical, the excavation should be backfilled to the underside of the bedding elevation to protect the subgrade from disturbance from both construction traffic and weather. Granular A should be used in dewatered excavations to backfill any sub-excavations required for subgrade improvement, see further comments below for excavations in the wet.

In areas where shallow bedrock is encountered and the subgrade material transitions from soil to bedrock, a subgrade taper sloped no steeper than 5H:1V shall be constructed in the surface of the bedrock. In areas with bedrock subgrade, care must be taken to avoid fracturing or disturbing the bedrock below the culvert. The exposed bedrock subgrade must be inspected to confirm that loose pieces have been removed and that subgrade is suitable and uniformly competent. Mass concrete can be used to fill any uneven bedrock surface to achieve the design bedding/embedment level for the culvert. If mass concrete is used, it should extend at least 300 mm beyond all edges of the culvert.

Foundation preparation for a pipe culvert should be as per OPSS.PROV 421 and OPSD 802.010 (flexible pipe) and OPSD 802.031 (rigid pipe) with bedding extending to 300 mm below the pipe in both cases. It is recommended that culvert cover, embedment and bedding materials consist of OPSS.PROV 1010 Granular A.

In order to provide a more uniform foundation subgrade condition for a closed box culvert, bedding and cover material conforming to OPSS.PROV 1010 Granular A requirements must be provided under the base of the culvert as per OPSS 422 and OPSD 803.010. The Granular bedding layer should be a minimum of 300 mm thick and covered with a 75 mm levelling course of Granular A.

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

Backfill above the granular cover material for a closed box or rigid pipe culvert or the embedment layer for a flexible 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. Please refer to Section 10.6 for additional details on embankment reconstruction and side slope inclinations.

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.

10.3 Lateral Earth Pressure

The equations for lateral earth pressure provided below are based on the assumption that the backfill is fully drained so that there are no unbalanced hydrostatic pressures. If adequate drainage cannot be confirmed, the potential for buildup of hydrostatic pressures should be considered in design. 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.

Lateral earth pressures acting on vertical walls should be computed in accordance with the Section 6.12 of the CHBDC but under fully drained conditions, the lateral pressures are generally given by the following expression:

$$\begin{aligned}\sigma_h &= K * (\gamma d + q) && \text{[static]} \\ \sigma_{hAE} &= K * \gamma * d + (K_{AE} - K_A) * \gamma * (H - d) && \text{[combined static and seismic]}\end{aligned}$$

where:

$$\begin{aligned}\sigma_h &= \text{static lateral earth pressure on the wall at depth } d \text{ (kPa)} \\ \sigma_{hAE} &= \text{combined static and seismic lateral earth pressure on wall at depth } d \text{ (kPa)} \\ K &= \text{static earth pressure coefficient (see table below)} \\ &\quad (K_A \text{ for yielding walls, } K_o \text{ for non-yielding walls)} \\ K_{AE} &= \text{combined static and seismic earth pressure coefficient}\end{aligned}$$



γ	=	unit weight of retained soil (kN/m ³), see table below adjusted to submerged unit weight below water level
d	=	depth below top of fill where pressure is computed (m)
H	=	total height of the wall (m)
q	=	value of any surcharge (kPa)

Static Lateral Earth Pressure

Typical earth pressure coefficients for vertical walls for backfill material are shown in Table 10-1.

Table 10-1: Static Earth Pressure Coefficients

MATERIAL	UNIT WEIGHT (kN/m ³)	K _A (YIELDING WALL)	K ₀ (NON-YIELDING WALL)	K _p (MOVEMENT TOWARD SOIL)	GROUND SURFACE BEHIND WALL
OPSS Granular A	22.8	0.27	0.43	3.7	Horizontal
		0.40	-	-	2H:1V
OPSS Granular B Type II	22.0	0.27	0.43	3.7	Horizontal
		0.40	-	-	2H:1V

The parameters in the table correspond to full mobilization of active and passive earth pressures and require certain relative movements between the wall and adjacent soil to produce these conditions. Figure C6.27 and Table C6.12 of the Commentary to the CHBDC indicates the relative movement required to fully mobilize the active earth pressure. Where ground surfaces are sloped at 2H:1V behind the walls, the corresponding coefficients provided in Table 10-1 should be used.

If lateral movement is not permissible and/or the wall is restrained, the at rest earth pressure coefficient should be used. If the wall design allows lateral movement, the active earth pressures should be used.

A geotechnical resistance factor of 0.5 (ϕ_{gu}) should be applied in static design to the passive earth pressures in accordance with Table 6.2 of the CHBDC (static analysis typical understanding). The soils within the depth of frost should be ignored from providing passive lateral resistance, however the equivalent surcharge loading from the weight of the soils above the frost depth should be incorporated into the lower soils layers.

Combined Static and Seismic Lateral Earth Pressure

In accordance with Clause 6.14 of the CHBDC, structures should be designed using dynamic earth pressure coefficients that incorporate the effects of earthquake loading. The following recommendations are per Section C6.14.7.2 of the Commentary of the CHBDC which states that seismically induced lateral soil pressures may be calculated using Mononobe Okabe Method with:

- $k_h = \frac{1}{2} * F(PGA) * PGA$, for structures that allow 25 to 50 mm of movement, and
- $k_h = F(PGA) * PGA$, for non-yielding 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 1 in 2475yr seismic event and a Seismic Site Class B.

Table 10-2: Combined Static and Seismic Earth Pressure Coefficients

MATERIAL	UNIT WEIGHT (kN/m ³)	K_{AE} (YIELDING WALL)	K_{AE} (NON-YIELDING WALL)	GROUND SURFACE BEHIND WALL
OPSS Granular A	22.8	0.28	0.29	Horizontal
		0.41	0.43	2H:1V
OPSS Granular B Type II	22.0	0.28	0.29	Horizontal
		0.41	0.43	2H:1V

10.4 Frost Depth

The frost penetration depth at this site is 2.2 m as per OPSD 3090.100. It is not necessary to found a pipe or a closed box culvert below the depth of frost penetration.

Given the depth of cover at this site, it is not anticipated that frost tapers will be required for pavement purposes.

10.5 Cement Type and Corrosion Potential

Analytical tests were completed to determine the potential for degradation of concrete in the presence of soluble sulphates and the potential for corrosion of exposed steel used in buried infrastructure. The concentration of soluble sulphate provides an indication of the degree of sulphate attack that is expected for concrete in contact with soil and groundwater at the site. Soluble sulphate concentrations less than 1000 µg/g generally indicate that a low degree of sulphate attack is expected for concrete in contact with soil and groundwater. The sulphate

content in the soils is 15 µg/g, see Section 5.7. The selection for class of concrete should include consideration of the effects of road de-icing salts.

The pH, resistivity, and chloride concentration provide an indication of the degree of corrosiveness of the sub-surface environment. The tests results provided in Section 5.7 may be used to aid in the selection of coatings and corrosion protection systems for buried steel objects. The corrosive effects of road de-icing salts should also be considered.

10.6 Embankment Design and Reinstatement

10.6.1 Embankment Reinstatement

The existing highway embankment side slopes are generally sloped at approximately 1.9H:1V and locally as steep as 1.3H:1V. 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 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 with materials similar to the existing. If constructed using rock fill, the embankment could be reconstructed with side slopes of 1.5H:1V (or flatter). 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.

As the permanent embankment envelope is to remain unchanged, the settlement beneath the embankment is expected to be negligible.

The magnitude of the embankment self-compression constructed with granular fill is in the order of 0.5% of the newly reconstructed embankment height and is expected to occur predominately during fill placement.

If the existing culvert is to be abandoned and fully grouted or removed and backfilled, it is estimated that this would induce settlements of less than 5 mm beneath the existing culvert alignment as a result of the increased load imposed by the grout/fill.

10.6.2 Temporary Grade Lowering

It is anticipated that grade lowering of approximately 3.5 m would be required to create a working surface wide enough to replace the culvert in two stages without temporary protection systems. The existing embankment is generally constructed with rockfill, thus additional excavation of the embankment fill will be required to prepare a temporary driving surface. The existing embankment side slopes should remain stable if the slopes are not altered or subject to significant loadings. The slopes are the responsibility of the Contractor and should be constructed following the recommendations described in Section 11.1. After culvert installation, the embankment should be reinstated as described in Section 10.6.1.

11. CONSTRUCTION CONSIDERATIONS

11.1 Excavation

All excavation must be conducted in accordance with the requirements of the Occupational Health & Safety Act & Regulations (OHSA) for Construction Projects. The fill materials and native soils above the groundwater level may be classified as Type 3 soil and native soils below the groundwater level are classified as Type 4 soils. *If an excavation penetrates more than one soil type, the entire excavation must be completed in accordance with the more stringent requirement as per the requirements of the regulation.*

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 the 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. Temporary embankment and cut slope stability is the responsibility of the Contractor.

Excavation for culvert replacement must be carried out in accordance with OPSS.PROV 401, OPSS.PROV 421 and OPSS 422 and will be carried out through existing embankment fill and into the underlying native soils. 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 slopes (both existing and temporary).

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

11.2 Temporary Protection Systems

Temporary Protection Systems may be required during various stages of construction and must be implemented in accordance with OPSS.PROV 539 as amended by SP 105S09. Performance Level 2 (maximum 25 mm horizontal deflection) is considered appropriate where the protection supports the existing highway. More stringent performance levels may be required if the protection system is intended to support existing structures or utilities. The actual 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.

It will be difficult to drive sheet piles at this site due to the presence of rockfill and shallow bedrock. A suggested contract provision concerning obstructions is provided in Appendix G. Drilled in soldier piles with lagging are considered suitable at this site. However, the selection and design of roadway protection is the responsibility of the Contractor. All protection systems should be designed by a licensed Professional Engineer experienced in such designs and retained by the Contractor. The design of the roadway protection system must incorporate traffic loading and surcharge loading due to construction equipment and operations. An anchoring and/or internal bracing system may need to be incorporated into the temporary protection design to resist lateral earth pressure loadings.

Lateral earth pressure coefficients, under fully mobilized conditions, that can be used in design of the protection system installed through new granular fill material consisting of Granular A or Granular B Type II are provided in Table 10-1 for static conditions. The lateral earth pressure coefficients for the existing fill and native soils are given below for a vertical wall and a horizontal backslope. Unit weights provided herein are to be adjusted for applications below the groundwater level. Unbalanced hydrostatic pressures should be considered in the design of the protection systems.



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

MATERIAL	UNIT^(*) WEIGHT (kN/m³)	K_A (-)	K_p (-)	S_u (kPa)	GROUND SURFACE BEHIND WALL
Existing Sand Fill	20	0.33	3.0	-	Horizontal
Existing Rockfill	19	0.24	4.2	-	Horizontal
Native Silty Sand	19	0.33	3.0	-	Horizontal
Native Gravelly Sand	20	0.31	3.3	-	Horizontal

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

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

11.3 Surface and Groundwater Control

Excavations that extend below the groundwater level without prior dewatering are not recommended since the inflow of groundwater will make it difficult to maintain a dry, sound base on which to work. Disturbance of the subgrade soils is considered to be a risk without groundwater lowering. The presence of rockfill and cohesionless subgrade soils could allow for increased seepage.

Typically, subgrade preparation, placement and compaction of granular bedding, and culvert installation must be carried out in the dry. Based on the groundwater elevation at the time of the investigation, the site will require dewatering to lower the groundwater (see Section 10.2). Furthermore, surface runoff will tend to seep into and accumulate into the excavations. The Contractor must control groundwater, perched groundwater, and surface water flow at the site to permit construction in a dry and stable excavation. Typically, the groundwater level within the work zone should be lowered to a minimum of 0.5 m below the underside of the planned excavation base prior to each stage of excavation.

A properly designed dewatering system to control groundwater and ditch/surface water is required and may include cofferdams, ditch diversion, pumping etc. If required, the temporary flow diversion pipe should be placed outside the construction area. If the replacement culvert is installed on a new alignment the existing culvert could be used for ditch flow diversion until the new culvert is completed. Alternatively, an existing culvert located approximately 90 m north of the site could be utilized as a flow diversion pipe if the flow can be diverted/pumped. The design of flow passage systems is the responsibility of the Contractor. Given the site conditions and

anticipated works, the Designer Fill-In ***** in SP 517F01 Table A for flow passage systems should be “Yes”; the design Engineer and design-checking Engineer do need a minimum of 5 years of experience in designing similar flow passage systems.

The dewatering system will be required to remain operational and effective until the temporary excavations are backfilled and then should be decommissioned and removed. 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 FOUN0003 which amends OPSS.PROV 902 and SP517F01 which amends OPSS.PROV 517. Given the site conditions and anticipated works, the Designer Fill-In ***** in SP517F01 Table A should be “No” for dewatering systems; the design Engineer and design-checking Engineer do not need a minimum of 5 years of experience in designing similar dewatering systems. A preconstruction survey is not recommended, thus Designer Fill-In ** in this SP should be “N/A”.

The water level will fluctuate and the minimum groundwater elevation for the site at the time of the excavation should be taken as the expected highwater level defined in SP517F01 and SP FOUN0003.

The dewatering plan should be coordinated with the TPS design; however, it will be difficult to install a watertight sheet pile enclosure. It is anticipated that sump pumps will likely be sufficient to extract water from an excavation carried out behind sandbag coffer dams. Pumping should continue until control of inflow is achieved and the Granular bedding and culvert can be placed and backfilled in a dry stable environment.

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.

11.4 Scour and Erosion Protection

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

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

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

OPSS.PROV 804. Slope vegetation should be established as soon as possible after completion of construction in order to limit surficial erosion and water should be prevented from running down an unprotected slope.

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

It is recommended that a clay seal be used to minimize the potential for piping and erosion around the inlet of the culvert. The clay seal must extend to approximately 300 mm above the high water level and laterally for the width of the granular material and have a minimum thickness of 500 mm. The material requirements should be in accordance with OPSS.PROV 1205. A geosynthetic clay liner could be considered for use as a clay seal.

12. CONSTRUCTION CONCERNS

Potential construction concerns include, but are not necessarily limited to:

- Obstructions
Buried obstructions (i.e., rockfill, cobbles, boulders, shallow bedrock) may be encountered during construction and will interfere with excavations and installation of temporary protection/dewatering systems. 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.
- Dewatering & Temporary Flow Passage Systems
It will be necessary to divert the ditch flow around the excavation to place the bedding and construct the culvert in the dry. Excavations and placement of bedding material must be completed in the dry. The presence of rockfill and cohesionless native soils may increase seepage rates. Suitable dewatering and flow passage systems must be employed to enable control of groundwater seepage and inflow. The flow passage system will be critical for culvert construction at this site. The Contractor should be prepared to take appropriate measures to construct the bedding layer and place the culvert in a dry and stable environment.



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The successful performance of the project will depend largely upon good workmanship and quality control during construction. Subgrade examination and field density testing should be carried out by qualified personnel during construction to confirm that foundation recommendations are correctly implemented and material specifications are met.

13. CLOSURE

Engineering analysis and preparation of this report were carried out by Mr. S. Peters, P.Eng. The report was reviewed by Dr. F. Griffiths, P.Eng., a Designated Principal Contact for MTO Foundation Projects.

Thurber Engineering Ltd.
Report Prepared By:



Stephen Peters, M.A.Sc., P.Eng.
Associate
Geotechnical Engineer



Fred Griffiths, Ph.D., P.Eng.
Senior Associate
Senior Geotechnical Engineer

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- a) Nature and Exactness of Soil and Contaminant Description: Classification and identification of soils, rocks, geological units, contaminant materials and quantities have been based on investigations performed in accordance with the standards set out in Paragraph 1. Classification and identification of these factors are judgmental in nature. Comprehensive sampling and testing programs implemented with the appropriate equipment by experienced personnel may fail to locate some conditions. All investigations utilizing the standards of Paragraph 1 will involve an inherent risk that some conditions will not be detected and all documents or records summarizing such investigations will be based on assumptions of what exists between the actual points sampled. Actual conditions may vary significantly between the points investigated and the Client and all other persons making use of such documents or records with our express written consent should be aware of this risk and the Report is delivered subject to the express condition that such risk is accepted by the Client and such other persons. Some conditions are subject to change over time and those making use of the Report should be aware of this possibility and understand that the Report only presents the conditions at the sampled points at the time of sampling. If special concerns exist, or the Client has special considerations or requirements, the Client should disclose them so that additional or special investigations may be undertaken which would not otherwise be within the scope of investigations made for the purposes of the Report.
- b) Reliance on Provided Information: The evaluation and conclusions contained in the Report have been prepared on the basis of conditions in evidence at the time of site inspections and on the basis of information provided to Thurber. Thurber has relied in good faith upon representations, information and instructions provided by the Client and others concerning the site. Accordingly, Thurber does not accept responsibility for any deficiency, misstatement or inaccuracy contained in the Report as a result of misstatements, omissions, misrepresentations, or fraudulent acts of the Client or other persons providing information relied on by Thurber. Thurber is entitled to rely on such representations, information and instructions and is not required to carry out investigations to determine the truth or accuracy of such representations, information and instructions.
- c) Design Services: The Report may form part of design and construction documents for information purposes even though it may have been issued prior to final design being completed. Thurber should be retained to review final design, project plans and related documents prior to construction to confirm that they are consistent with the intent of the Report. Any differences that may exist between the Report's recommendations and the final design detailed in the contract documents should be reported to Thurber immediately so that Thurber can address potential conflicts.
- d) Construction Services: During construction Thurber should be retained to provide field reviews. Field reviews consist of performing sufficient and timely observations of encountered conditions in order to confirm and document that the site conditions do not materially differ from those interpreted conditions considered in the preparation of the report. Adequate field reviews are necessary for Thurber to provide letters of assurance, in accordance with the requirements of many regulatory authorities.

6. RELEASE OF POLLUTANTS OR HAZARDOUS SUBSTANCES

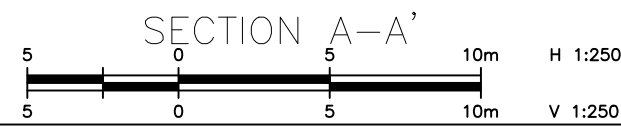
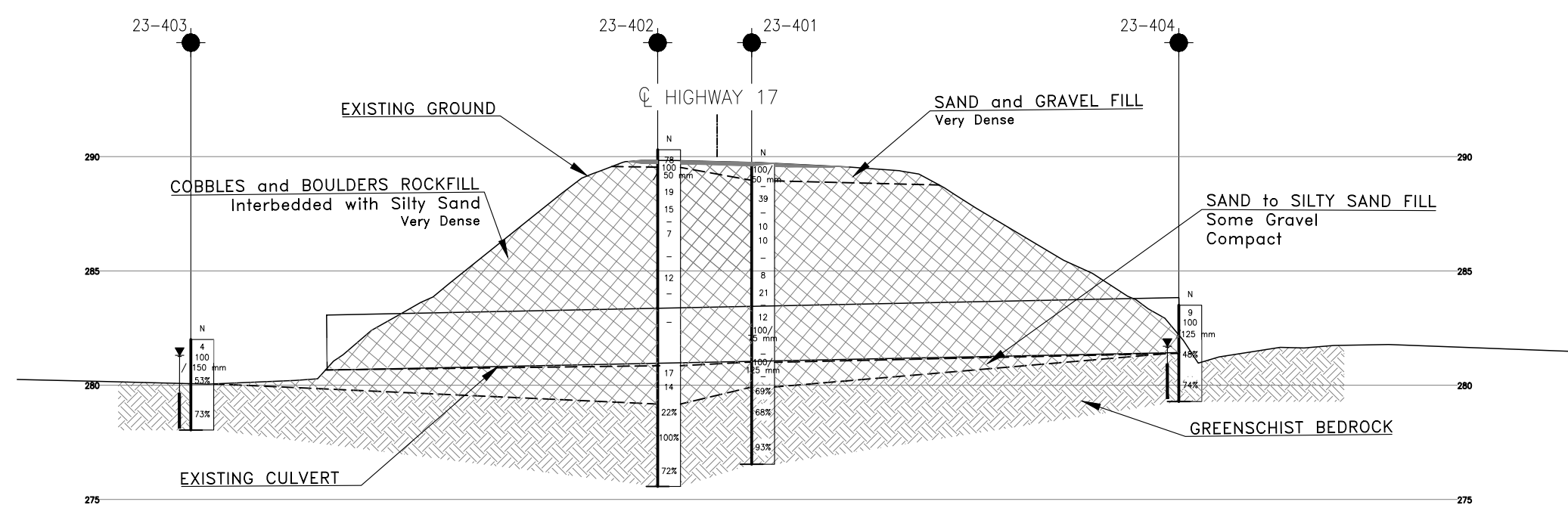
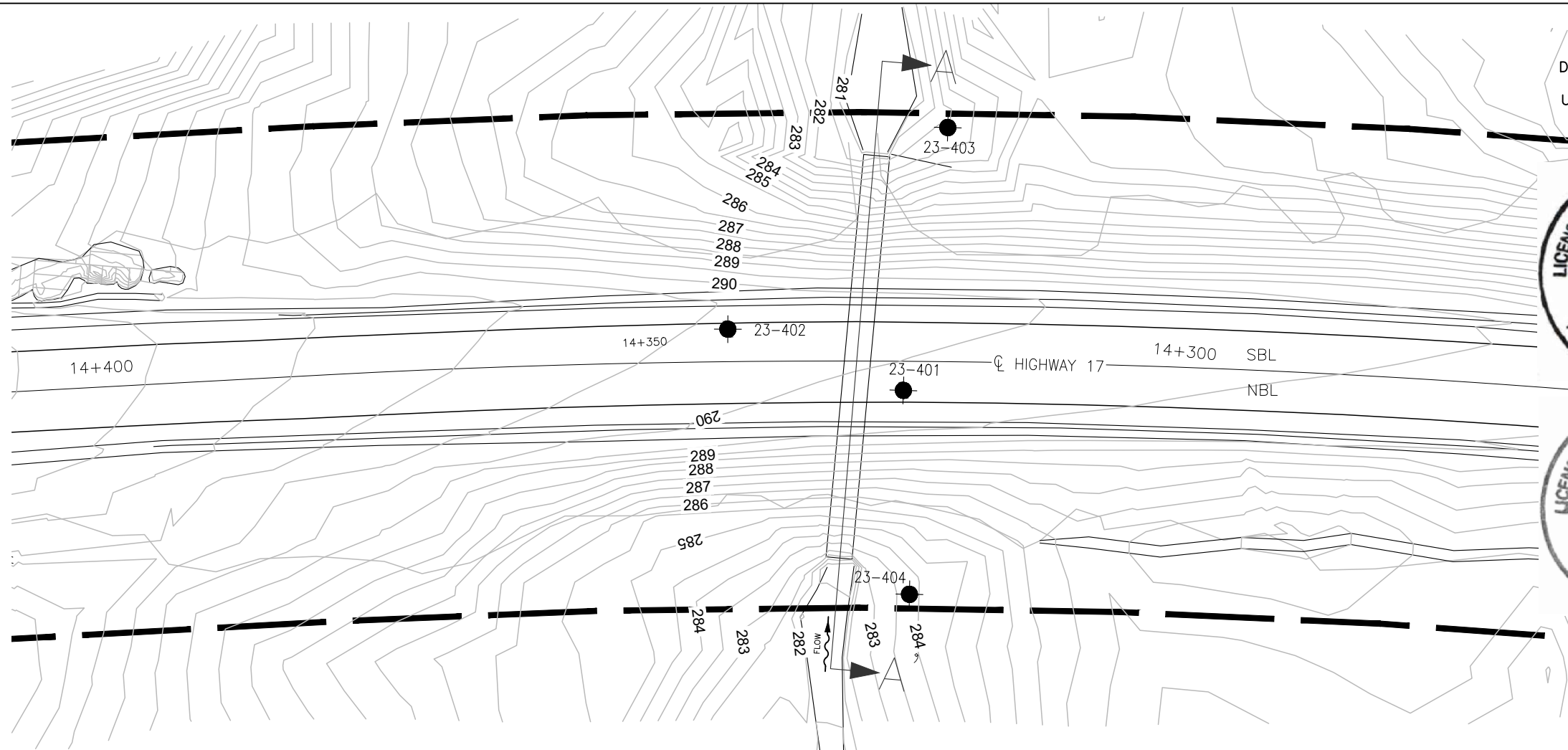
Geotechnical engineering and environmental consulting projects often have the potential to encounter pollutants or hazardous substances and the potential to cause the escape, release or dispersal of those substances. Thurber shall have no liability to the Client under any circumstances, for the escape, release or dispersal of pollutants or hazardous substances, unless such pollutants or hazardous substances have been specifically and accurately identified to Thurber by the Client prior to the commencement of Thurber's professional services.

7. INDEPENDENT JUDGEMENTS OF CLIENT

The information, interpretations and conclusions in the Report are based on Thurber's interpretation of conditions revealed through limited investigation conducted within a defined scope of services. Thurber does not accept responsibility for independent conclusions, interpretations, interpolations and/or decisions of the Client, or others who may come into possession of the Report, or any part thereof, which may be based on information contained in the Report. This restriction of liability includes but is not limited to decisions made to develop, purchase or sell land.

APPENDIX A

Borehole Locations and Strata Drawing

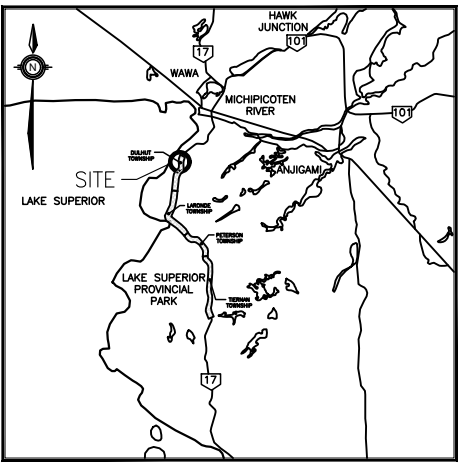


METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN







CONT No
GWP No 5207-18-00

HIGHWAY 17
DULHUT TOWNSHIP
Culvert 14+332
BOREHOLE LOCATIONS AND SOIL STRATA



KEYPLAN

LEGEND

	Borehole
	Borehole (Previous Investigation)
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
23-401	289.7	5 301 090.8	238 450.4
23-402	290.3	5 301 074.2	238 446.1
23-403	282.0	5 301 092.9	238 425.9
23-404	283.5	5 301 092.9	238 469.1

-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 13.

GEOCRES No. 41N00-038

[illegible]

APPENDIX B

Symbols and Terms
Record of Boreholes Sheets
Single Well Response Test



SYMBOLS, ABBREVIATIONS AND TERMS USED ON TEST HOLE RECORDS

TERMINOLOGY DESCRIBING COMMON SOIL GENESIS

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

TERMINOLOGY DESCRIBING SOIL STRUCTURE:

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

RECOVERY:

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

N-VALUE:

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

DYNAMIC CONE PENETRATION TEST (DCPT):

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



STRATA PLOT:

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



Boulders
Cobbles
Gravel Sand Silt Clay Organics Asphalt Concrete Fill Bedrock

TEXTURING CLASSIFICATION OF SOILS

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

TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

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

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

SAMPLE TYPES

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

TERMS DESCRIBING CONSISTENCY (COHESIONLESS SOILS ONLY)

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

MODIFIED UNIFIED SOIL CLASSIFICATION

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

Note - W_L = Liquid Limit



EXPLANATION OF ROCK LOGGING TERMS

ROCK WEATHERING CLASSIFICATION

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

TERMS

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

DISCONTINUITY SPACING

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

STRENGTH CLASSIFICATION

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

RECORD OF BOREHOLE No 23-401

1 OF 2

METRIC

GWP# 5207-18-00 LOCATION Lat: 47.845423°, Long: -84.886551° Sta. 14+332, Duhiut Township, MTM z13: N 5 301 090.8 E 238 450.4 ORIGINATED BY APS
 HWY 17 BOREHOLE TYPE CME 75 Truck Mounted / NW Casing / NQ Coring COMPILED BY RH
 DATUM Geodetic DATE 2023.05.11 - 2023.05.11 CHECKED BY AO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							
289.7	Ground Surface							<div>20406080100</div> <div>○ UNCONFINED + FIELD VANE</div> <div>● QUICK TRIAXIAL × LAB VANE</div>							
0.0	ASPHALT (115 mm)							<div>204060</div> <div>W_P W W_L</div> <div>PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT</div> <div>WATER CONTENT (%)</div>							
0.1	SAND and GRAVEL very dense light brown		1	SS	100/ 50 mm									52 45 3 (SI+CL)	
289.0	FILL														
0.7	GRAVEL and COBBLES inferred boulders ROCKFILL		2	NQ	-										
			3	SS	39										76 21 3 (SI+CL)
			4	NQ	-										
			5	SS	10										
			6	SS	10										
			7	NQ	-										
			8	SS	8										
			9	SS	21										
			10	NQ	-										
			11	SS	12										
			12	SS	100/ 75 mm										
		13	NQ	-											
281.0															
8.7	SILTY SAND, some gravel contains cobbles and boulders very dense light brown FILL		14	SS	100/ 125 mm									19 50 29 2	
			15	NQ	-										
279.9															
9.8	GREENSCHIST BEDROCK		1	RUN											

Continued Next Page

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

METRIC

[illegible]

+³, ×³: Numbers refer to Sensitivity

RECORD OF BOREHOLE No 23-402

1 OF 2

METRIC



GWP# 5207-18-00 LOCATION Lat: 47.845273°, Long: -84.886604° Sta. 14+332, Dulhut Township, MTM z13: N 5 301 074.2 E 238 446.1 ORIGINATED BY APS
 HWY 17 BOREHOLE TYPE CME 75 Truck Mounted / NW Casing / NQ Coring COMPILED BY RH
 DATUM Geodetic DATE 2023.05.09 - 2023.05.10 CHECKED BY AO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						WATER CONTENT (%)
								20 40 60 80 100						
290.3	Ground Surface													
0.0	ASPHALT (125 mm)													
0.1	SAND and GRAVEL very dense blackish brown		1	SS	78		290						40 55 5 (SI+CL)	
289.5	FILL		2	SS	100									
0.8	GRAVEL and COBBLES interbedded with silty sand ROCKFILL				50 mm		289							
			3	SS	19								61 30 9 (SI+CL)	
			4	SS	15		288							
			5	NQ	-		287							
			6	SS	7									
			7	NQ	-		286							
			8	SS	12		285							
284.5	COBBLES and BOULDERS ROCKFILL		9	NQ	-		284							
5.8	- 1.0 m boulder at a depth of 6.1 m													
	- 508 mm boulder at a depth of 7.0 m		10	NQ	-		283							
							282							
280.9	SAND, some gravel compact greyish brown FILL		10	SS	17		281							
9.4														

Continued Next Page

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

METRIC

ELEV. DEPTH	SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						WATER CONTENT (%)
								○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE × LAB VANE					
	Continued From Previous Page							20 40 60 80 100		20 40 60				

[illegible]

DOUBLE LINE 31653-400.GPJ 2012TEMPLATE(MTO).GDT 11-24-23

+³, ×³: Numbers refer to Sensitivity

RECORD OF BOREHOLE No 23-403

1 OF 1

METRIC

GWP# 5207-18-00 LOCATION Lat: 47.845439°, Long: -84.886878° Sta. 14+332, Dulhut Township, MTM z13: N 5 301 092.9 E 238 425.9 ORIGINATED BY IK
 HWY 17 BOREHOLE TYPE Portable / NW Casing / NQ Coring COMPILED BY RH
 DATUM Geodetic DATE 2023.07.06 - 2023.07.07 CHECKED BY AO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa													
								○ UNCONFINED + FIELD VANE													
								● QUICK TRIAXIAL × LAB VANE													
						20	40	60	80	100	20	40	60	WATER CONTENT (%)		GR	SA	SI	CL		
282.0	Ground Surface																				
0.0	SAND , some silt contains cobbles and organics loose brown		1	SS	4																
			2	NQ	-																
			3	SS	100																
281.0																					
1.0	GREENSCHIST BEDROCK contains calcite seams fresh jointed fine grained dark greenish grey very strong rusty colouration at joints				/ 150 mm																
			1	RUN																	

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

RECORD OF BOREHOLE No 23-404

1 OF 1

METRIC

GWP# 5207-18-00 LOCATION Lat: 47.845444°, Long: -84.886301° Sta. 14+332, Dulhut Township, MTM z13: N 5 301 092.9 E 238 469.1 ORIGINATED BY IK
 HWY 17 BOREHOLE TYPE Portable / NW Casing / NQ Coring COMPILED BY RH
 DATUM Geodetic DATE 2023.06.09 - 2023.06.11 CHECKED BY AO

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa							
283.5	Ground Surface														
0.0	GRAVELLY SAND, some silt contains organics loose brown		1	SS	9										31 57 12 (SI+CL)
282.9			2	NQ	-										
0.6	GRAVEL and COBBLES with silty sand infills very dense brown to grey		3	SS	100 /125 mm										
282.0															
1.5	GREENSCHIST BEDROCK slightly weathered to fresh fine grained greenish grey very strong		1	RUN											RUN #1 TCR=100% SCR=38% RQD=48%
			2	RUN											RUN #2 TCR=100% SCR=66% RQD=74%
279.3															
4.2	End of Borehole														
	Monitoring well installed: Schedule 40 PVC standpipe with 32-mm diameter and 1.5-m slotted screen. Water Level Readings: DATE DEPTH (m) ELEV. (m) 2023/06/11 0.8 282.7 2023/07/06 1.7 281.8 2023/07/08 2.1 281.4 2023/07/12 1.8 281.7 2023/07/13 1.8 281.7 2023/07/15 1.8 281.7 Note: A third-weight hammer was used to advance the split-spoon sampler. The "N" values presented above have been adjusted to provide an estimate of the "N" value that would have been obtained with a standard hammer.														

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



THURBER ENGINEERING LTD.

Slug Test Analysis Report

Project: Highway 17 and Old Woman River Bridge

Number: 31653

Client: AECOM

Location: Laronde Township, Ontario

Slug Test: 23-403

Test Well: 23-403

Test Conducted by: SM & IK

Test Date: 2023-07-15

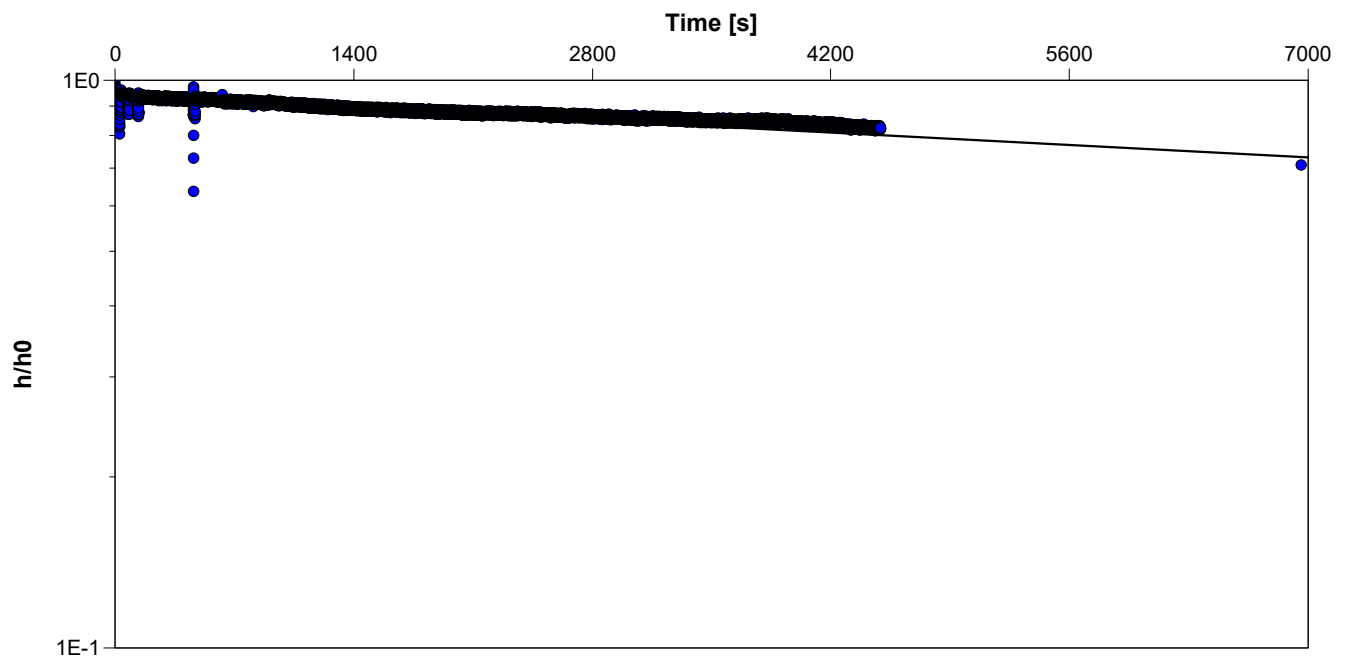
Analysis Performed by: SM

SWRT Analysis

Analysis Date: 2023-07-19

Aquifer Thickness:

Checked by: AH



Calculation using Hvorslev

Observation Well

Hydraulic Conductivity
[m/s]

23-403

3.1×10^{-8}



THURBER ENGINEERING LTD.

Slug Test Analysis Report

Project: Highway 17 and Old Woman River Bridge

Number: 31653

Client: AECOM

Location: Laronde Township, Ontario

Slug Test: 23-404

Test Well: 23-404

Test Conducted by: SM & IK

Test Date: 2023-07-15

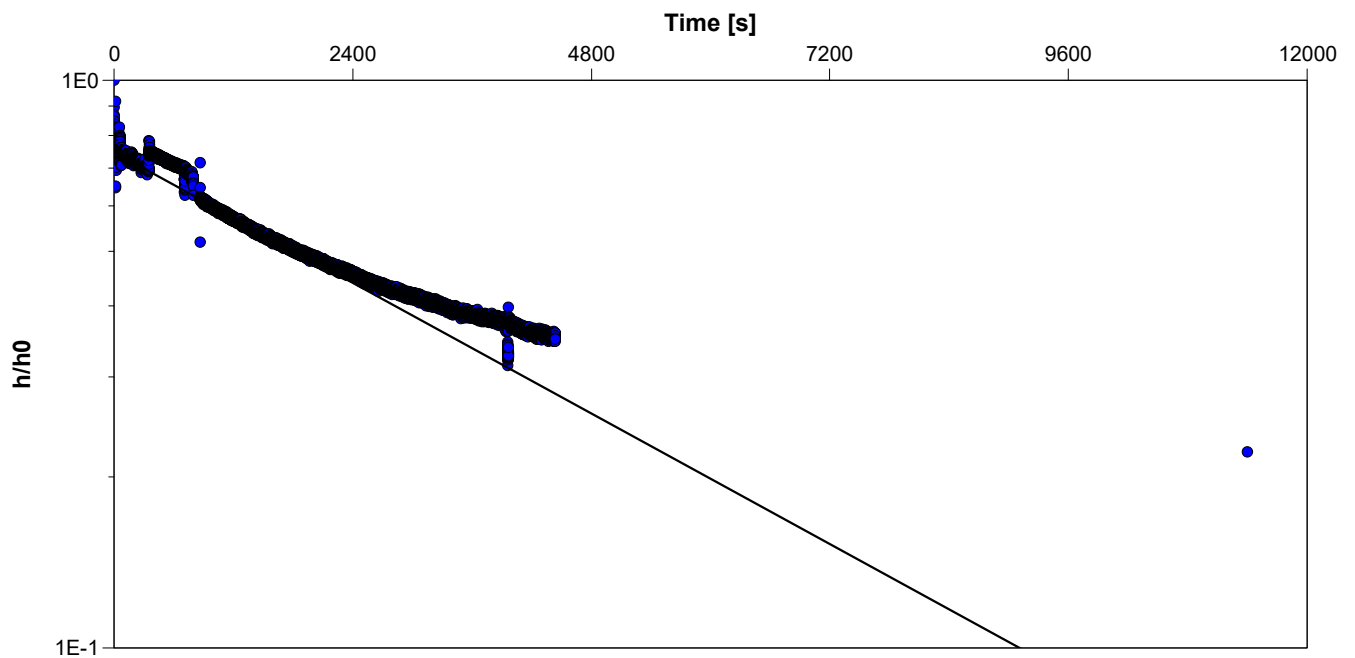
Analysis Performed by: SM

SWRT Analysis

Analysis Date: 2023-07-19

Aquifer Thickness:

Checked by: AH



Calculation using Hvorslev

Observation Well

Hydraulic Conductivity
[m/s]

23-404

2.0×10^{-7}

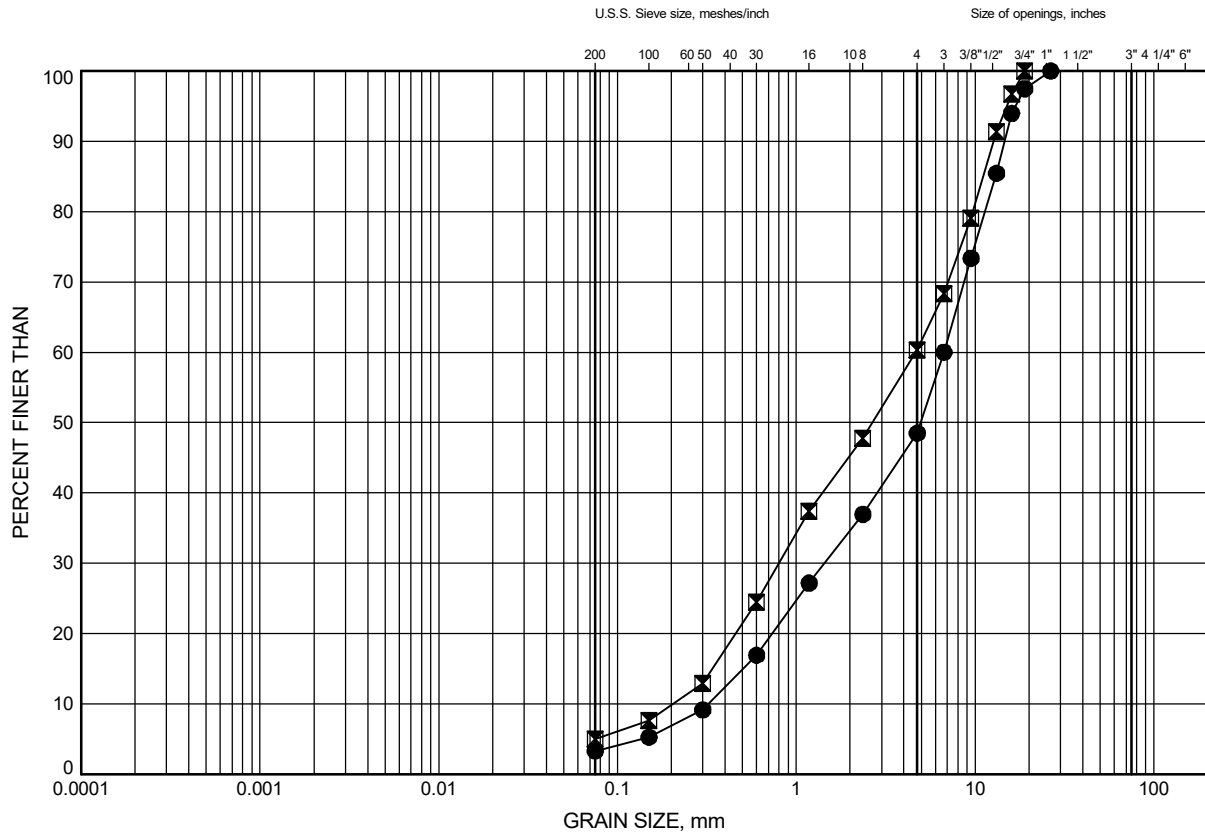
APPENDIX C

Particle Size Analysis Figures
Unconfined Compressive Strength Testing Results
Bedrock Core Photographs
Analytical Testing Results

Hwy 17 Old Woman River GRAIN SIZE DISTRIBUTION

FIGURE C1

FILL: Sand and Gravel



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	23-401	0.2	289.5
⊠	23-402	0.4	289.9

Date August 2023

GWP# 5207-18-00



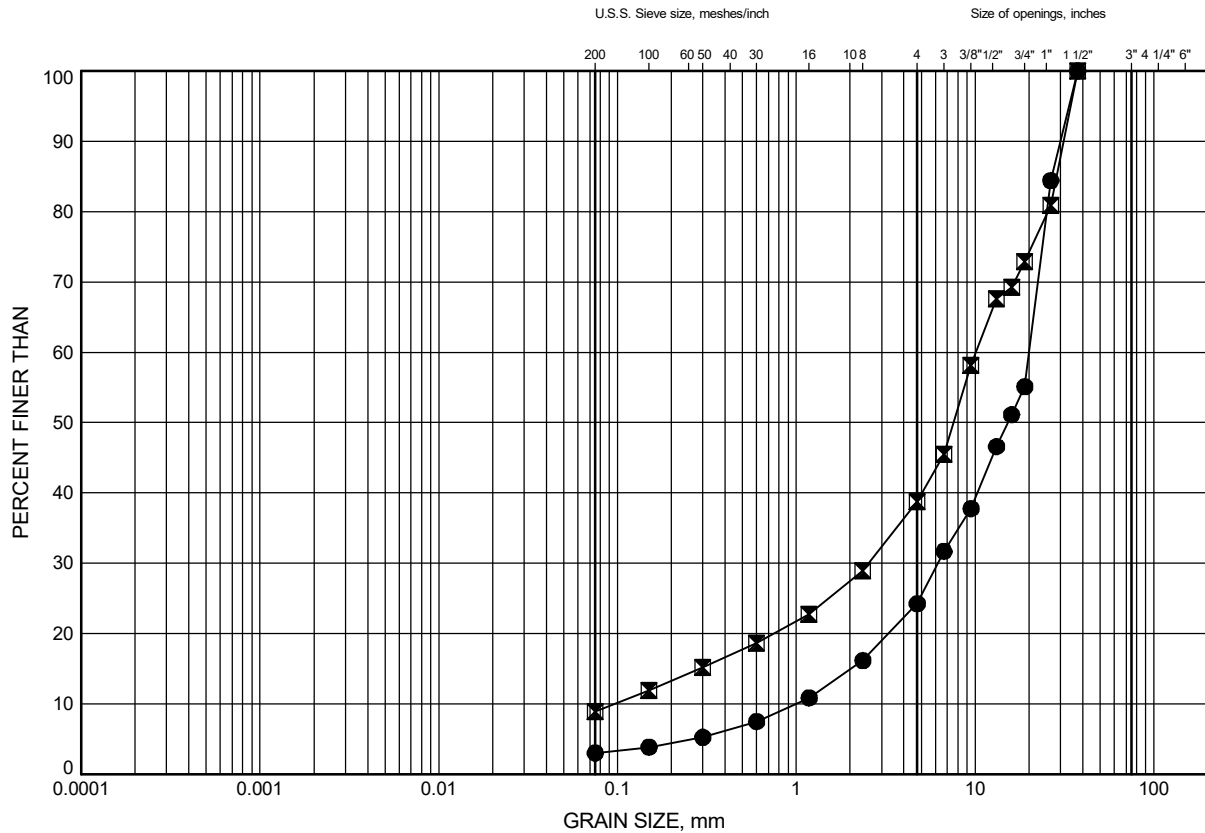
Prep'd RH

Chkd. AO

Hwy 17 Old Woman River GRAIN SIZE DISTRIBUTION

FIGURE C2

Rock Fill



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	23-401	1.5	288.2
◻	23-402	1.8	288.5

Date August 2023
GWP# 5207-18-00

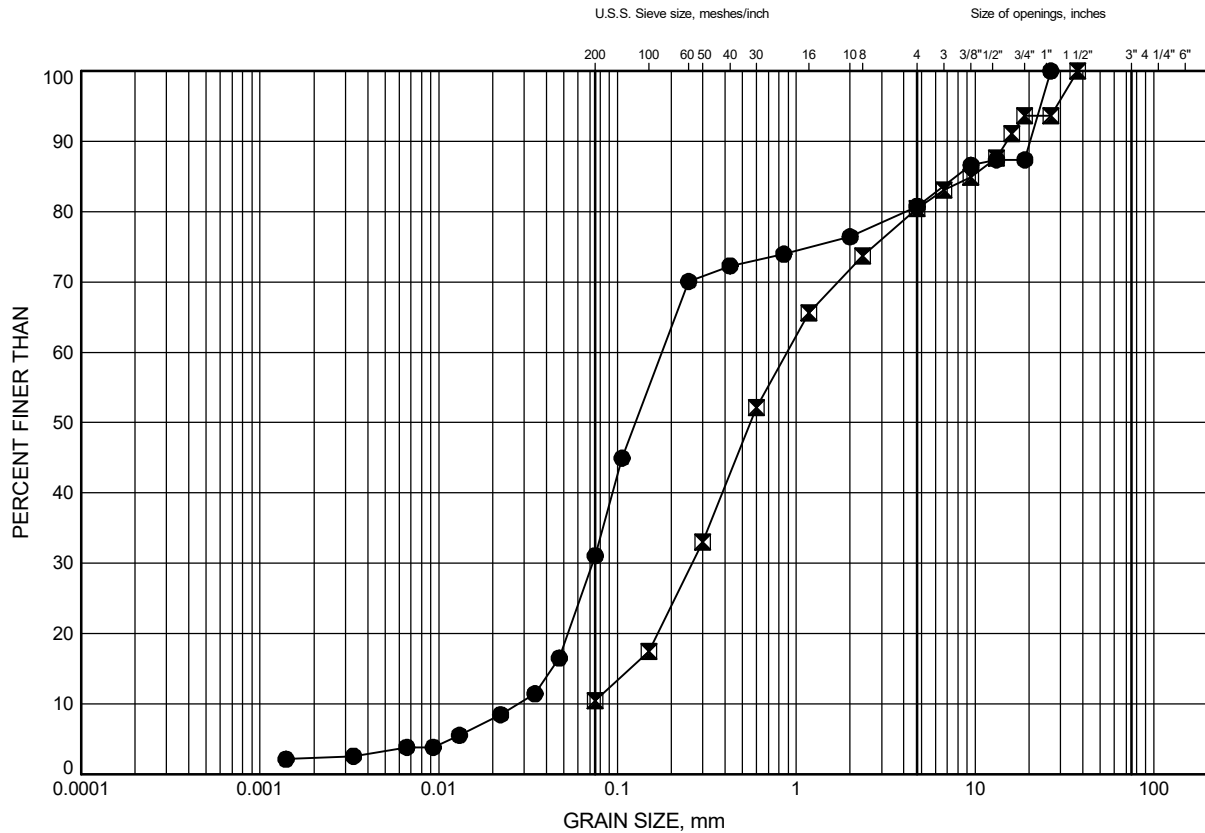


Prep'd RH
Chkd. AO

Hwy 17 Old Woman River GRAIN SIZE DISTRIBUTION

FIGURE C3

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)
●	23-401	8.8	280.9
⊠	23-402	10.4	279.9

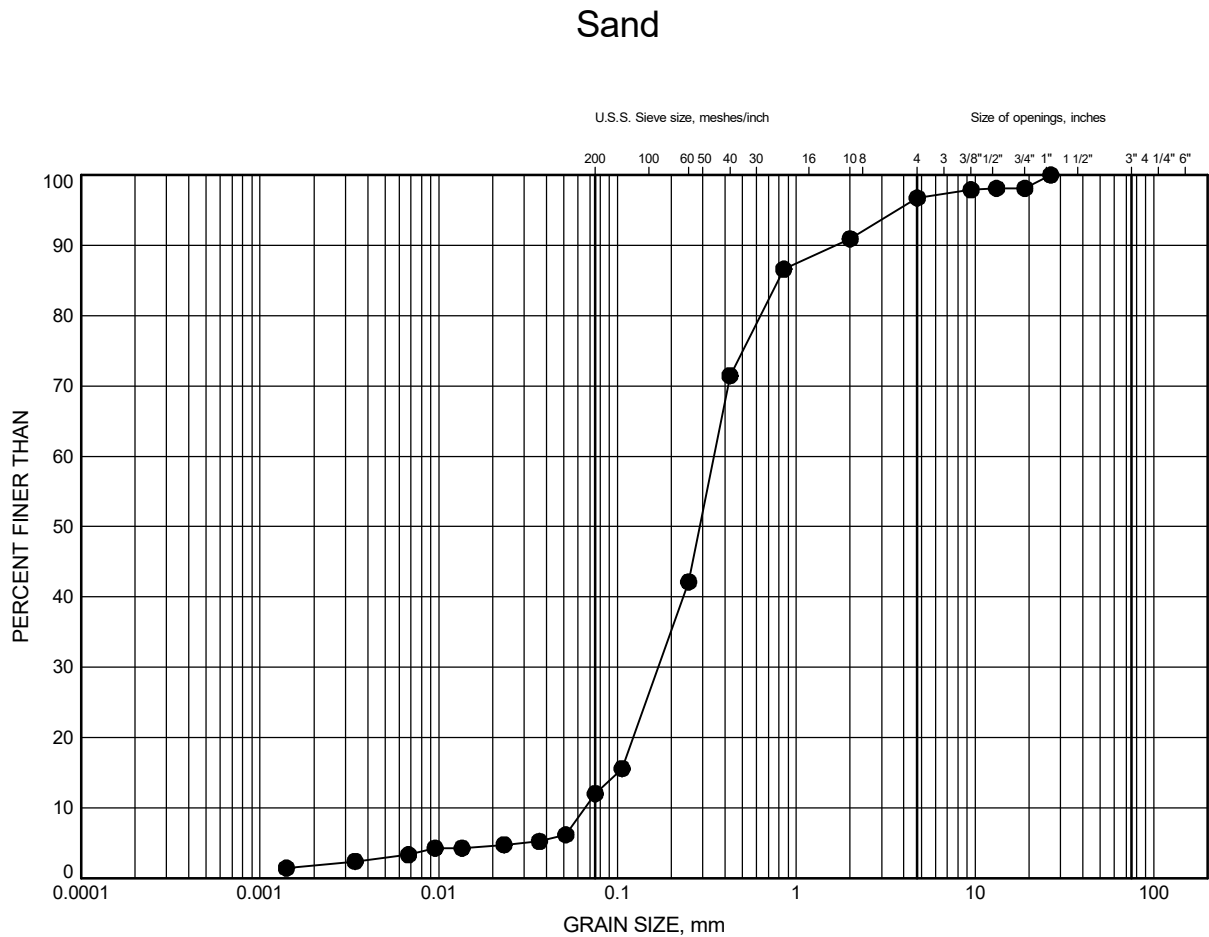
Date August 2023
GWP# 5207-18-00



Prep'd RH
Chkd. AO

Hwy 17 Old Woman River GRAIN SIZE DISTRIBUTION

FIGURE C4



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	23-403	0.3	281.7

Date August 2023
GWP# 5207-18-00

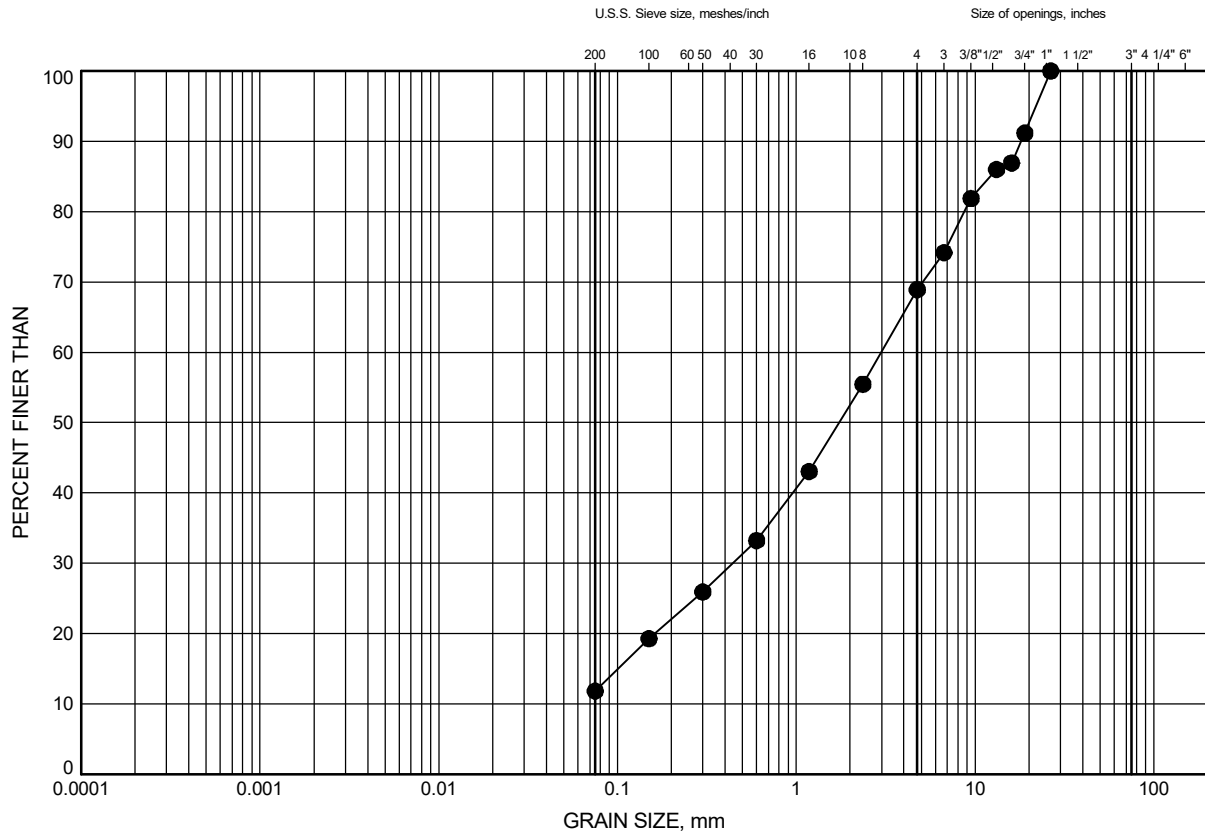


Prep'd RH
Chkd. AO

Hwy 17 Old Woman River GRAIN SIZE DISTRIBUTION

FIGURE C5

Gravelly Sand to Gravel



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	23-404	0.3	283.2

Date August 2023
GWP# 5207-18-00



Prep'd RH
Chkd. AO



Stantec Consulting Ltd.
2781 Lancaster Rd, Suite 100 A&B, Ottawa ON K1B 1A7

August 8, 2023
File: 122410864

Client: Thurber Engineering, File #31653.60

**Reference: ASTM D7012, Method C, Unconfined Compressive Strength of Intact Rock Core
Hwy 17-Old Woman River**

The following table summarizes unconfined compressive strength results for three intact rock cores.

Location	Sample Depth	Compressive Strength (MPa)	Description of Break
23-102 Run-2	35' 35'0"	161.8	Well formed cones at both ends.
23-403 Run-1	4'8"-5'5"	152.3	Vertical cracking
23-502 Run-1	37'4" 38'	93.7	Diagonal fracture

Sincerely,

Stantec Consulting Ltd.

Brian Prevost
Laboratory Supervisor
Tel: 613-738-6075
Fax: 613-722-2799
brian.prevost@stantec.com

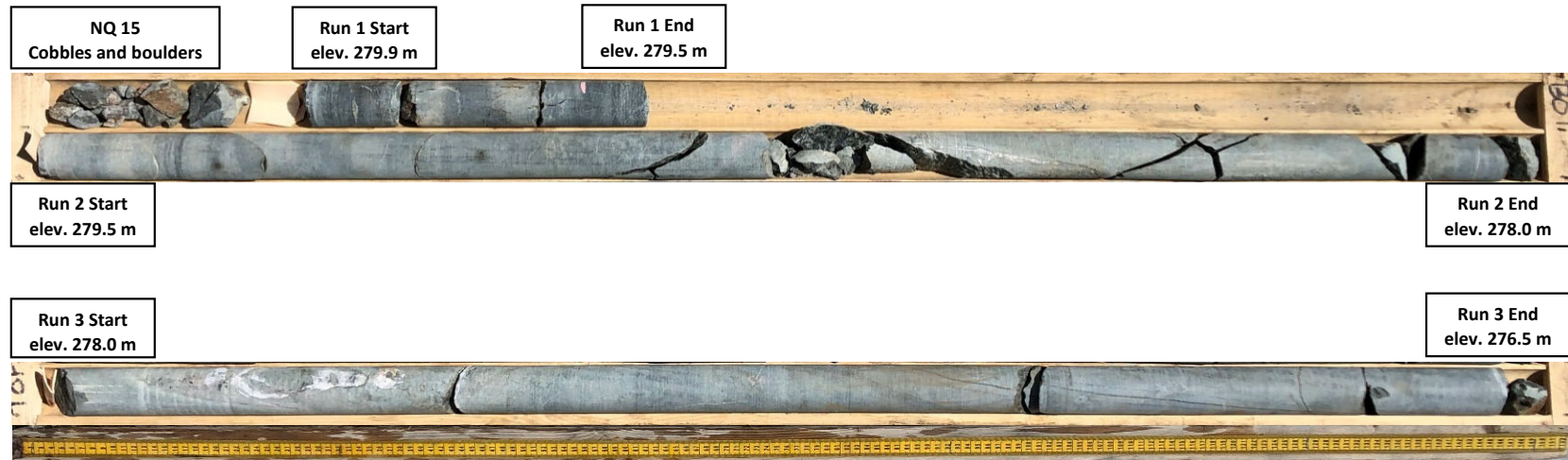
Borehole 23-401

Runs 1 to 3

Depth 9.8 m to 13.2 m

Elevation 279.9 m to 276.5 m

Dry Sample



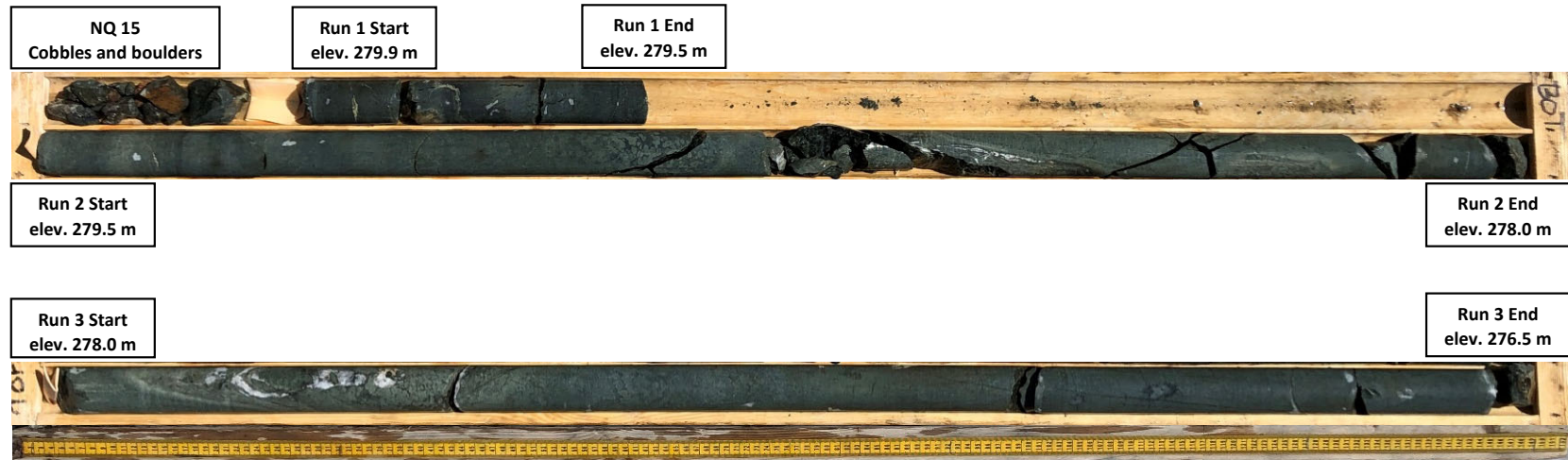
Borehole 23-401

Runs 1 to 3

Depth 9.8 m to 13.2 m

Elevation 279.9 m to 276.5 m

Wet Sample



THURBER ENGINEERING LTD.

Geotechnical Investigation
Highway 17 Old Woman River Bridge
Sta. 14+332, Dulhut Township

BH 23-401
Project No.: 31653

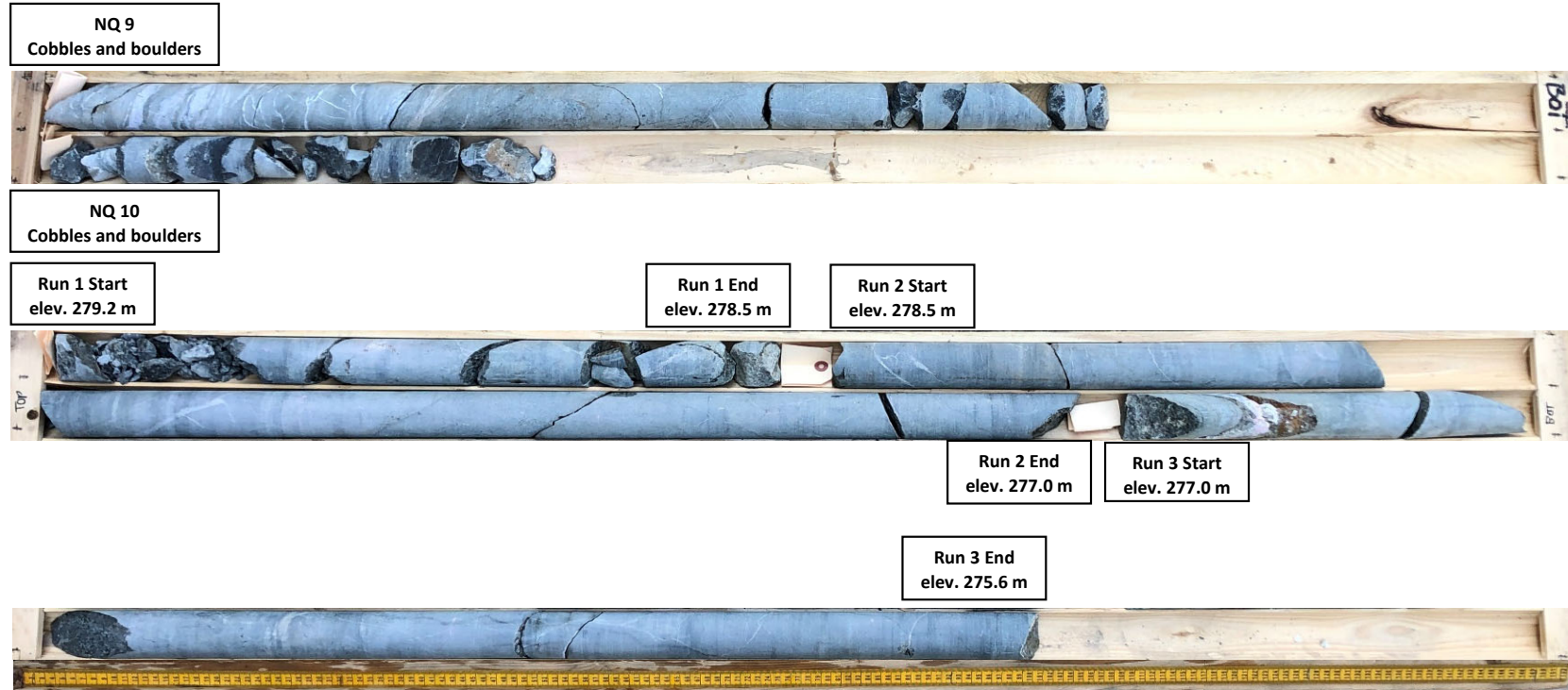
Borehole 23-402

Runs 1 to 3

Depth 11.1 m to 14.7 m

Elevation 279.2 m to 275.6 m

Dry Sample



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Sta. 14+332, Dulhut Township

BH 23-402
Project No.: 31653

Borehole 23-402

Runs 1 to 3

Depth 11.1 m to 14.7 m

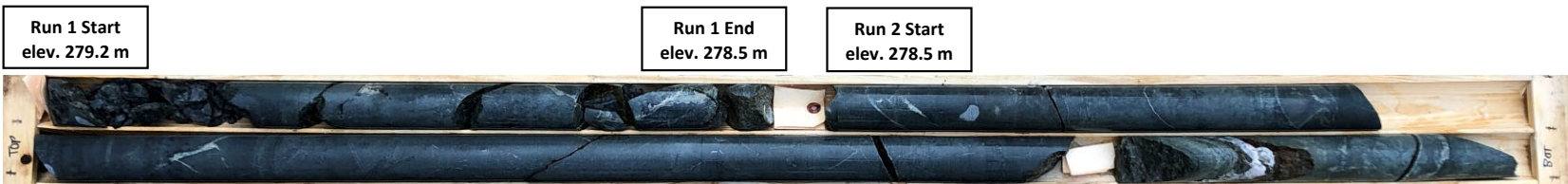
Elevation 279.2 m to 275.6 m

Wet Sample

NQ 9
Cobbles and boulders



NQ 10
Cobbles and boulders



Run 1 Start
elev. 279.2 m

Run 1 End
elev. 278.5 m

Run 2 Start
elev. 278.5 m

Run 2 End
elev. 277.0 m

Run 3 Start
elev. 277.0 m

Run 3 End
elev. 275.6 m



Borehole 23-403

Run 1

Depth 1.0 m to 2.5 m

Elevation 281.0 m to 279.5 m

Dry Sample

NQ 2 Cobbles and boulders

Run 1 Start
elev. 281.0 m



Run 1 End
elev. 279.5 m



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Geotechnical Investigation
Highway 17 Old Woman River Bridge
Sta. 14+332, Dulhut Township

BH 23-403
Project No.: 31653

Borehole 23-403

Run 1

Depth 1.0 m to 2.5 m

Elevation 281.0 m to 279.5 m

Wet Sample

NQ 2 Cobbles and boulders

Run 1 Start
elev. 281.0 m



Run 1 End
elev. 279.5 m



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Highway 17 Old Woman River Bridge
Sta. 14+332, Dulhut Township

BH 23-403
Project No.: 31653

Borehole 23-403

Run 2

Depth 2.5 m to 4.0 m

Elevation 279.5 m to 278.0 m

Dry Sample

Run 2 Start
elev. 279.5 m



Run 2 End
elev. 278.0 m



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BH 23-403
Project No.: 31653

Borehole 23-403

Run 2

Depth 2.5 m to 4.0 m

Elevation 279.5 m to 278.0 m

Wet Sample

Run 2 Start
elev. 279.5 m



Run 2 End
elev. 278.0 m



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Geotechnical Investigation
Highway 17 Old Woman River Bridge
Sta. 14+332, Dulhut Township

BH 23-403
Project No.: 31653

Borehole 23-404

Run 1

Depth 1.5 m to 2.7 m

Elevation 282.0 m to 280.8 m

Dry Sample

NQ 2 and 4 Gravel and Cobbles



Run 1 Start
elev. 282.0 m



Run 1 End
elev. 280.8 m



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Geotechnical Investigation
Highway 17 Old Woman River Bridge
Sta. 14+332, Dulhut Township

BH 23-404
Project No.: 31653

Borehole 23-404

Run 1

Depth 1.5 m to 2.7 m

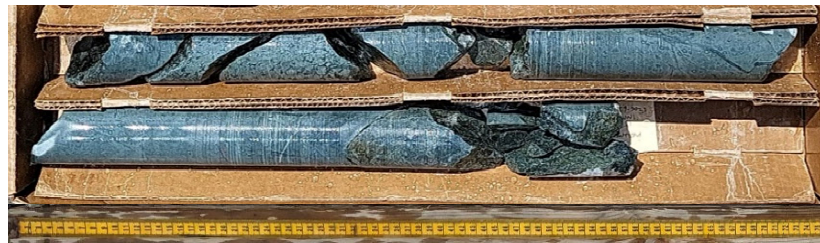
Elevation 282.0 m to 280.8 m

Wet Sample

NQ 2 and 4 Gravel and Cobbles



Run 1 Start
elev. 282.0 m



Run 1 End
elev. 280.8 m



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Geotechnical Investigation
Highway 17 Old Woman River Bridge
Sta. 14+332, Dulhut Township

BH 23-404
Project No.: 31653

Borehole 23-404

Run 2

Depth 2.7 m to 4.2 m

Elevation 280.8 m to 279.3 m

Dry Sample

Run 2 Start
elev. 280.8 m



Run 2 End
elev. 279.3 m



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Geotechnical Investigation
Highway 17 Old Woman River Bridge
Sta. 14+332, Dulhut Township

BH 23-404
Project No.: 31653

Borehole 23-404

Run 2

Depth 2.7 m to 4.2 m

Elevation 280.8 m to 279.3 m

Wet Sample

Run 2 Start
elev. xx m



Run 2 End
elev. xx m



THURBER ENGINEERING LTD.

Geotechnical Investigation
Highway 17 Old Woman River Bridge
Sta. 14+332, Dulhut Township

BH 23-404
Project No.: 31653

Certificate of Analysis

Report Date: 07-Jul-2023

Client: Thurber Engineering Ltd.

Order Date: 28-Jun-2023

Client PO:

Project Description: 31653 Hwy 17 Old Woman River

Client ID:	23-404 GB1 (0'-1')	-	-	-
Sample Date:	09-Jun-23 09:00	-	-	-
Sample ID:	2326353-01	-	-	-
MDL/Units	Soil	-	-	-

Physical Characteristics

% Solids	0.1 % by Wt.	93.9	-	-	-
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General Inorganics

Conductivity	5 uS/cm	75	-	-	-
pH	0.05 pH Units	4.16	-	-	-
Resistivity	0.1 Ohm.m	133	-	-	-

Anions

Chloride	10 ug/g dry	12	-	-	-
Sulphate	10 ug/g dry	15	-	-	-

**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

26-July-2023

Date Rec. : 30 June 2023
LR Report: CA19633-JUN23
Reference: Project#: 2326353

Copy: #2

CERTIFICATE OF ANALYSIS

Final Report - Revised

Sample ID	Sample Date & Time	Sulphide (Na ₂ CO ₃) %
1: Analysis Start Date		18-Jul-23
2: Analysis Start Time		12:08
3: Analysis Completed Date		18-Jul-23
4: Analysis Completed Time		14:07
5: QC - Blank		< 0.04
6: QC - STD % Recovery		112%
7: QC - DUP % RPD		ND
8: RL		0.02
9: 23-404 GB1 (0'-1')	09-Jun-23 09:00	< 0.04

RL - SGS Reporting Limit
ND - Not Detected

Revised July 26, 2023 - Sample ID updated as per client request.

Kimberley Didsbury
Project Specialist,
Environment, Health & Safety

APPENDIX D

Site Photographs



Photo 1: Culvert inlet (*taken on June 11, 2023*)



Photo 2: Culvert outlet (*taken on July 06, 2023*)



Photo 3: Highway 17 north of the culvert alignment (*taken on May 12, 2023*)



Photo 4: Highway 17 south of the culvert alignment (*taken on May 09, 2023*)



Photo 5: Looking north at the northbound embankment *(taken on June 11, 2023)*



Photo 6: Looking south at the southbound embankment *(taken on July 11, 2023)*

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: 47.846N 84.887W

User File Reference: Highway 17 STA 14+242 Dulhut Township

2023-07-20 18:25 UT

Requested by: Thurber Engineering

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)	0.046	0.023	0.013	0.004
Sa (0.1)	0.064	0.035	0.021	0.006
Sa (0.2)	0.061	0.036	0.023	0.007
Sa (0.3)	0.052	0.032	0.021	0.007
Sa (0.5)	0.042	0.027	0.018	0.005
Sa (1.0)	0.026	0.016	0.010	0.003
Sa (2.0)	0.013	0.007	0.004	0.001
Sa (5.0)	0.003	0.002	0.001	0.000
Sa (10.0)	0.001	0.001	0.001	0.000
PGA (g)	0.035	0.020	0.012	0.004
PGV (m/s)	0.032	0.019	0.012	0.003

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 Comparisons



THURBER ENGINEERING LTD.

RIGID OR FLEXIBLE PIPE	OPEN BOTTOM CULVERT	CLOSED BOTTOM CULVERT	TRENCHLESS
Advantages			
<ul style="list-style-type: none"> - Readily available materials and simple installation methods 	<ul style="list-style-type: none"> - Relatively expedient installation if precast units are used - 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. 	<ul style="list-style-type: none"> - Relatively expedient installation if precast units are used 	<ul style="list-style-type: none"> - Avoids open cut and reduces need for roadway protection systems - Allows for two directions of traffic to be maintained throughout construction
Disadvantages			
<ul style="list-style-type: none"> - Requires moderate excavation - Protection system will require bracing, anchors and/or rakers - May require temporary by-pass pumping system 	<ul style="list-style-type: none"> - Requires largest and deepest excavation - Protection system is higher so will require additional bracing, anchors and/or rakers - Dewatering to greater depth - May require temporary by-pass pumping system 	<ul style="list-style-type: none"> - Requires moderate excavation - Protection system will require bracing, anchors and/or rakers - May require temporary by-pass pumping system 	<ul style="list-style-type: none"> - Requires specialized construction equipment and Contractor - Requires construction of entry and exit pits and access to the toes of the slope - very slow progress in rockfill
Risks			
<ul style="list-style-type: none"> - rockfill and cohesionless native soils may increase seepage rates requiring more robust sump pumps 	<ul style="list-style-type: none"> - Potential of encountering undulating bedrock surface - rockfill and cohesionless native soils may increase seepage rates requiring more robust sump pumps 	<ul style="list-style-type: none"> - Potential of encountering undulating bedrock surface - rockfill and cohesionless native soils may increase seepage rates requiring more robust sump pumps 	<ul style="list-style-type: none"> - Obstructions present - Entry and exit pits could require an enclosure excavation - A mixed face is anticipated. - Groundwater may be encountered.
Recommendation			
Recommended	Not recommended	Recommended	Not feasible

APPENDIX G

List of Referenced Specifications and Contract Provisions

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

- OPSS.PROV 206
- OPSS.PROV 401
- OPSS.PROV 421
- OPSS.PROV 422
- OPSS.PROV 501
- OPSS.PROV 511
- OPSS.PROV 517
- OPSS.PROV 539
- OPSS.PROV 803
- OPSS.PROV 804
- OPSS.PROV 805
- OPSS.PROV 902
- OPSS.PROV 1004
- OPSS.PROV 1010
- OPSS.PROV 1205
- OPSS.PROV 1860
- SP 105S09
- SP 110S06
- SP 517F01
- SP FOUN0003
- OPSD 208.010
- OPSD 219.110
- OPSD 802.031
- OPSD 803.010
- OPSD 803.031
- OPSD 810.010
- OPSD 3090.100

2. Contract Provision – Obstructions

Installation of roadway protection systems and coffer dams will encounter obstructions such as rockfill, cobbles and boulders. Such obstructions may impede the work from reaching the design depth of installation. The Contractor shall be prepared to remove, drill through and/or penetrate these obstructions and extend the work to the design depths. The work must not destabilize the culvert(s) or embankment.

3. Contract Provision - Dewatering

It will be necessary to divert the ditch flow around the excavation to place the bedding and construct the culvert in the dry. Excavations and placement of bedding material must be completed in the dry. The presence of rockfill and cohesionless native soils may increase seepage rates. A suitable dewatering / unwatering system must be employed to enable control of groundwater seepage and inflow. The dewatering scheme will be critical for culvert construction at this site. The Contractor should be prepared to take appropriate measures to construct the bedding layer and place the culvert in a dry and stable environment.