

**FINAL
FOUNDATION INVESTIGATION AND DESIGN REPORT
OLD WOMAN RIVER BRIDGE
HIGHWAY 17
AGREEMENT NO. 5020-E-0025
GWP 5207-18-00**



THURBER ENGINEERING LTD.

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OLD WOMAN RIVER BRIDGE
HIGHWAY 17
AGREEMENT NO. 5020-E-0025
GWP 5207-18-00**

**SITE NO. 38C-0009/B0
GEOCRES NO.: 41N00-036**

**Report
to
AECOM Canada Ltd.**

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Longitude: -84.894241°

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PART 1. FACTUAL INFORMATION

1. INTRODUCTION

This section of the report presents the factual findings obtained from a foundation investigation completed at the Highway 17 crossing of Old Woman River in the Township of LaRonde within the District of Algoma, Ontario. Thurber Engineering Limited (Thurber) carried out the assignment as a sub-consultant to AECOM Canada Ltd. (AECOM) under Agreement No. 5020-E-0025 and as part of Change Order 1.

The purpose of this investigation was to explore the subsurface conditions at the site and, based on the data obtained, to provide a borehole location plan, records of boreholes, a stratigraphic profile, laboratory test results, and a written description of the subsurface conditions. A model of the subsurface conditions influencing design and construction was developed in the course of the current investigation.

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

2. SITE DESCRIPTION

2.1 General

The existing Highway 17 bridge crosses Old Woman River approximately 24.3 km south of the junction of Highway 17 and Highway 101. The bridge site is situated within Lake Superior Provincial Park at approximate Sta. 15+250 LaRonde Township which is about 170 m south of the driveway entrance to the Old Woman Bay Scenic Lookout site. For project purposes, the bridge is herein described as oriented north-south, and the river's flow is described as oriented east to west. Passing lanes are present approximately 500 m north and south of the bridge.



The existing bridge is a two-lane, concrete deck on steel plate girders bridge, comprising one span built with a northeast to southwest skew to the river. Concrete parapet walls are placed along the east and west edges of the bridge deck. Steel guiderails supported with wood posts extend from both ends of the bridge. The existing roadway embankment side slopes at the site did not show any visible signs of global instability at the time of the investigation, but active surficial water erosion was observed at the side slopes near the approaches. The embankments near the approaches are sloped at 2.9H:1V to 2.0H:1V. Traffic volumes on this section of Highway 17 are understood to have been 2,300 AADT in 2019.

Drawings provided by AECOM indicate that the road surface is at approximate elevation 187.8 m. The river's water level was surveyed by Thurber at elev. 183.3 m on August 5, 2022. The depth of water at that time was 1.6 m. The river flows from east to west, toward Lake Superior which is approximately 300 m east of the site. The water flow was noted to be stronger near the north abutment where the water depth was observed to be deeper. Sand sediments, tree trunks and branches were observed near the south abutment. Beyond the bridge the river flows in a meandering manner. Frequent sand banks were noted, and rounded cobbles are present on the riverbed and on the riverbanks.

The site is in a rural setting and the area directly adjacent to the roadway is undeveloped and densely vegetated with coniferous and deciduous trees and shrubs. An entrance to a Scenic Lookout is located north of the bridge. The terrain is slightly undulating in the vicinity of the site. Overhead utility lines were not present.

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

2.2 Existing Structure Information

The Terms of Reference (TOR) describe the bridge as constructed in 1958 (Contract 58-24) with a span length of 30 m. The structure is 10.4 m wide with a travelled width of 9.5 m and consists of a concrete deck on steel plate girders on conventional reinforced concrete abutments with expansion joints. Wingwalls are approximately 9.5 m long at each abutment and are at 54 deg from the abutment face. The abutment and wingwalls are founded on trapezoidal shaped spread footings which extend down to approximate elev. 181.6 m.

Historical construction drawings by Proctor & Redfern (W.P. 958-57) show that sheet piles were installed at each abutment to elev. 179.2 m to aid construction. Thurber observed the sheet piles in the water near the east side of the north abutment, see Photo 9 in Appendix D.



The bridge was rehabilitated in 1997 (Contract 1997-0240).

The 2019 OSIM report indicates that the existing structure is in fair condition overall but with some elements, such as girder ends and abutment walls, in poor condition. The OSIM report recommends structure rehabilitation in 1-5 years.

2.3 Existing Subsurface Information

The following historical foundation investigation report was available for this site within the Online Geocres library:

- Geocres Report No. 41N00-008 (e. m. peto associates, 1957) presents the results of the foundation investigations carried out for the design and construction of the existing bridge structure. This investigation included 4 boreholes: 2 on the north side and 2 on the south side of the proposed Old Woman River bridge. All 4 boreholes indicated the presence of topsoil underlain by fine to coarse sand. Soils with organics were encountered 15.2 m below the ground surface in all the boreholes. The boreholes were terminated within the sand deposit at a depth of 30.7 m (approx. elev. 154.2 to 153.1 m).

The coordinates of the boreholes are not provided in the historical investigation and the boreholes were advanced prior to the bridge construction. For this reason, it is difficult to know the borehole locations. Nonetheless, the historical borehole plan and borehole records are included in the report for the reference in Appendix A.

2.4 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 area is characterized by glaciofluvial, morainal material, and Precambrian bedrock. The main substrate type in the Ecodistrict is bedrock covered by a discontinuous thin layer of mineral material.

Regional Geology Map MRD126³ indicates the site is on the boundary between a unit to the north consisting of granodiorite to granite and a unit consisting of gneiss to tonalite to granodiorite to the south.

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>



3. SITE INVESTIGATION AND FIELD TESTING

The site investigation and field-testing program was carried out between August 4 and 6, 2022 as part of the original scope, and between May 25 and June 11, 2023 as part of Change Order 1, and consisted of two on-road boreholes identified as 22-01 and 22-02, and two off-road boreholes identified as OW-23-01 and OW-23-02. Multiple attempts were required to drill at the off-road borehole OW-23-02 due to difficult drilling conditions which included cobbles and flowing sands. The borehole record included herein and the plotted borehole location, document the conditions for the deepest of these borehole attempts.

The on-road boreholes were advanced with a CME 75 truck mounted drill rig utilizing HW Casing and coring techniques. The off-road boreholes were advanced with portable drilling techniques and utilized a third-weight hammer for SPT advancement. A hammer weight correction has been applied to the reported N-Values for the SPT carried out with the portable equipment. Prior to commencement of drilling, utility clearances were obtained in the vicinity of the borehole locations.

A summary of the borehole coordinates, elevations, and termination depths is provided in Table 3-1, below. The as-drilled borehole elevations were surveyed by Thurber with an auto-level relative to BM HCP 150 (Elevation 186.656 m). The elevations and borehole coordinates were reviewed and referenced to the survey provided by AECOM. Horizontal locations were measured by Thurber relative to existing site features. 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)
22-01	North Abutment/ Southbound Lane	5 295 028.9	237 784.3	187.6	15.8
22-02	South Abutment/ Northbound Lane	5 295 006.9	237 825.5	187.7	15.8 (DCPT 19.2)
OW-23-01	Embankment Toe/Northwest Quadrant	5 295 018.7	237 785.2	184.4	4.0 (DCPT 4.8)
OW-23-02	Embankment Toe/Southeast Quadrant	5 295 016.9	237 826.0	185.1	4.1

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. The boreholes were advanced to sampled depths ranging from 4.0 m to 15.8 m below the existing ground surface (elev. 181.0 m to 171.8 m). A Dynamic Cone Penetration (DCPT) was completed below the sampled depth in Boreholes 22-02 and OW-23-01 to a tip elevation at 168.5 and 179.6 m (19.2 and 4.8 m below the ground surface), respectively. Bedrock was not encountered within the depth of investigation.

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 samples for transport to the laboratory for further examination and testing.

A 32 mm diameter monitoring well was installed in each of Boreholes OW-23-01 and OW-23-02 to observe the groundwater level upon completion of drilling. The details for the well are illustrated on the respective Record of Borehole sheets provided in Appendix B. The wells were decommissioned in July 2023.

Following completion of the field investigation, the boreholes were decommissioned in general accordance with O.Reg. 903, as amended. Boreholes 22-01 and 22-02 were capped with asphalt patch 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. Selected soil samples were chosen for grain size distribution and tested in accordance with MTO and ASTM standards. The results of these tests are summarized on the Record of Borehole sheets included in Appendix B.

Four soil samples were selected and submitted for analytical testing of corrosivity parameters and sulphate content.

All laboratory test results from the 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 paragraphs. 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 accordance with ASTM D 2487 as per current MTO Guidelines for Foundation Engineering Services.

In general terms, the encountered stratigraphy consisted of granular fill over native deposits of sand with varying amounts of silt and gravel.

5.1 Asphalt

Asphalt was encountered at the ground surface in both on-road boreholes with a recorded thickness of 110 and 140 mm.

5.2 Fill - Silty Sand with Gravel to Sand with Silt and Gravel

Silty sand with gravel to sand with silt and gravel fill was encountered beneath the asphalt in Boreholes 22-01 and 22-02. The granular fill layer was 2.5 to 4.3 m thick (base elevation 185.0 to 183.3 m). SPT N-values in the fill materials ranged from 10 to 70 blows, indicating a compact to very dense relative density. Borehole 22-01 had higher blow counts of 100 blows/125 mm which could represent a cobble within the fill. Coring was required to advance past a boulder.



The recorded moisture content of samples of the fill layer ranged from 4 to 16%. The results of three gradation analyses completed on samples of the fill are illustrated on Figure C1 of Appendix C. The results of the test are summarized below and on the Record of Borehole sheets in Appendix B.

Soil Particle	Percentage (%)	
Gravel	18 to 40	
Sand	52 to 78	
Silt	4 to 8	23
Clay		3

5.3 Sand (SP / SP-SM / SW-SM) to Gravel (GW) with Sand

A native layer of sand to gravel with sand was encountered below the granular fill. Varying amounts of gravel, silt, and cobbles were encountered within the sand, and wood fragments were encountered in Boreholes 22-01 and 22-02 approximately 15.2 m below the ground surface (elev. 172.5 m). In all the historical borehole logs, organic content was also encountered as high as elev. 174.2 m extending to the base of the investigation. The layer was not fully penetrated but was extended to depths ranging from 4.0 m to 15.8 m below the existing ground surface (base elev. 181.0 to 171.8 m) in the current investigation. SPT N-values ranged from 1 to 49 blows but were typically greater than 14 blows, indicating a compact to dense relative density. Borehole OW-23-02 had higher blow counts of 100 blows/150 mm which could infer cobbles within the layer. Coring was required to advance past the cobbles. Flowing sand was encountered near elev. 182.1 to 175.0 m in boreholes 22-01, 22-02, and OW-23-01.

The recorded moisture content ranged from 3 to 35% but was typically less than 23%. The results of gradation analyses completed on ten samples of the native sand illustrated on Figures C2 and C3 of Appendix C. The results of the tests are summarized below and on the Record of Borehole sheets in Appendix B.

Soil Particle	Percentage (%)	
Gravel	0 to 13	
Sand	78 to 96	
Silt	1 to 5	6 to 8
Clay		0 to 3



Three samples from the layer in Boreholes 22-01, 22-02, and OW-23-01 had a higher gravel content. The results of gradation analyses completed on those samples are summarized below and are illustrated on Figure C4 of Appendix C. The results tests are also summarized on the Record of Borehole sheets in Appendix B

Soil Particle	Percentage (%)
Gravel	35 to 72
Sand	27 to 59
Silt	1 to 6
Clay	

5.4 Refusal

Bedrock was not encountered within the depth of the borehole investigation. However, a Dynamic Cone Penetration Test (DCPT) was carried out below the sampled depth in Borehole 22-02, and a refusal blow count was encountered at a tip elevation of 168.5 m. A DCPT was also carried out below the sampled depth in Borehole OW-23-01 to a refusal blow count of 100 blows at a tip elevation of 179.6 m. However, this DCPT refusal is inferred to be refusal on a cobble or boulder within the layer.

Historical boreholes from Geocres 41N00-008 also did not encounter bedrock within a depth of investigation of 30.8 m (elev. 153.2 m). The historical boreholes did not carry out SPT testing below a depth of 18.3 m (elev. 165.7 m).

5.5 Groundwater Level

The measured groundwater levels within the wells installed in Borehole OW-23-01 and OW-23-02 are summarized in Table 5-1, below. An unstabilized groundwater level was recorded in Borehole 22-01 in the open borehole however, water was used during the drilling operations thus this reading may not be representative.



Table 5-1 Measured Water Levels

Borehole	Bottom of Screen Depth / Elevation (m)	Soil in Zone of Screen	Groundwater Level		Date of Measurement
			Depth (mbgs)	Elevation (m)	
OW-23-01	3.2 181.2	Sand with Silt	1.1	183.3	2023/06/02
			1.1	183.3	2023/06/09
			1.1	183.3	2023/06/15
			1.1	183.3	2023/07/06
			1.1	183.3	2023/07/14
OW-23-02	3.4 181.7	Sand with Silt	1.7	183.4	2023/07/08
			1.7	183.4	2023/07/11
			1.7	183.4	2023/07/13
			1.7	183.4	2023/07/14

The river water level was surveyed by Thurber at elev. 183.3 m on August 5, 2022.

These observations are considered short term, and it should be noted that the groundwater and river water level at the time of construction may be different. Seasonal fluctuations of the groundwater level are to be expected. In particular, the groundwater and river water level may be at a higher elevation after periods of significant and/or prolonged precipitation and spring snow melts.

A Single Well Response Test (SWRT), or “slug test”, was carried out on July 14, 2023 in the monitoring wells 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 Table 5-2, below.

Table 5-2 Single Well Response Test Results

Borehole /Monitoring Well	Bottom of Screen Depth /Elevation (m)	Soil in Zone of Screened	Estimated Hydraulic Conductivity (m/s)
OW-23-01	3.2 / 181.2	Sand with Silt	4.2×10^{-4}
OW-23-02	3.4 / 181.7	Sand with Silt	2.7×10^{-4}

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 14, 2023.

5.6 Analytical Testing

Three soil samples were submitted for analytical testing. The analysis results are included in Appendix C and are summarized in Table 5-3.

Table 5-3 Summary of Analytical Test results

Borehole	22-01	22-02	22-02	OW-23-02
Sample	SS5	SS6	SS9	SS8
Depth (ft/m)	10 – 12 3.0 – 3.6	12.5 – 14.5 3.8 – 4.4	20 – 22 6.1 – 6.7	10 – 12 3.0 – 3.6
Elevation (m)	184.3	183.6	181.3	181.8
Soil Type	Sand	Sand with Silt and Gravel Fill	Sand with Silt	Sand with Silt
Conductivity ($\mu\text{S}/\text{cm}$)	86	158	45	18
pH	7.38	6.88	7.15	6.84
Resistivity (Ohm-cm)	11,600	6,340	22,000	54,700
Chloride ($\mu\text{g}/\text{g}$)	5	39	<5	<10
Sulphate ($\mu\text{g}/\text{g}$)	<5	29	6	<10
Sulphide (%)	<0.04	<0.04	-	<0.04



6. MISCELLANEOUS

The borehole locations reflect existing site features and access constraints. The as-drilled locations and ground surface elevations were measured by Thurber. George Downing Estate Drilling Ltd. of Hawkesbury, Ontario, and Ohlmann Geotechnical Services Inc. of Almonte, Ontario, supplied and operated the drill rig used to drill, test, sample, and decommission the boreholes. Traffic control was performed in accordance with Ontario Book 7 and was provided by Leroy Construction of Blind River, Ontario, and J. Provost Contracting Ltd. of Wawa, Ontario. The field work was supervised on a full-time basis by Mr. I. Khan, EIT and Mr. A. de Oliveira, EIT, under the direction of Mr. S. Peters, P.Eng.

Geotechnical laboratory testing was carried out by Thurber's geotechnical laboratory in Ottawa, Ontario. Analytical laboratory testing was carried out by Paracel Laboratories Ltd. in Ottawa, Ontario.



Interpretation of the data and preparation of this report were carried out by A. de Oliveira, E.I.T., and S. Peters, P.Eng. The report was reviewed by F. Griffiths, P.Eng., and P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundation Projects.

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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 assessment of the existing footings and design of the temporary protection and coffer dam systems for the rehabilitation of Old Woman River bridge located at the Highway 17 crossing of Old Woman River in the township of Laronde within the District of Algoma, Ontario. Thurber Engineering Limited (Thurber) carried out the assignment as a sub-consultant to AECOM Canada Ltd. (AECOM) under Agreement No. 5020-E-0025. The discussion and recommendations presented in this report are based on the 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 and 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. The construction or design-build contractor 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. Contractors must make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction methods and scheduling.

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

7.1 Background

The bridge site is located on Highway 17 approximately 24.3 km south of the junction of Highway 17 and Highway 101. Highway 17 is a two-lane road with a width of 10.4 m across the



bridge. The single span bridge is 30 m long and was constructed in 1958. The bridge abutments and wing walls are founded on spread footings.

In general terms, the encountered stratigraphy consisted of granular fill over a native deposit of sand with varying amounts of silt and gravel. The river water level was at elev. 183.3 m on August 5, 2022.

7.2 Proposed Structure

It is indicated in the Terms of Reference that the proposed rehabilitation work includes superstructure replacement, abutment modification and refacing, retaining wall refacing and north abutment footing rehabilitation. AECOM's Foundation Investigation Program Summary Memorandum dated July 2022 indicated that resurfacing of submerged portions of the abutments would be required. A preliminary General Arrangement (GA) drawing was provided on Sheet 84 in the 30% design package, copy included in Appendix A. It is understood that the overall width of the structure is under review which could affect the number of girders and girder size as well as superstructure abutment loading. The anticipated work does not include a grade raise or alteration to the existing embankment geometry.

It is understood that the work will require excavations into the approach fills. Temporary protection systems could be needed to allow a single lane of traffic around the work area.

7.3 Applicable Codes and Design Considerations

The geotechnical assessment presented below has been prepared based on the available data regarding the proposed foundations, existing ground surface conditions and in accordance with the Canadian Highway Bridge Design Code (CHBDC) version CSA S6-19.

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

It is understood that the structure is designated as a "Major-Route Bridge" importance category.

It is understood that the bridge has 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 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 9.3.

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 0.035 g. 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 at the site 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 Section 4.4.3.2 of the CHBDC, the selection of the seismic site classification is based on the nature of soil deposit within the upper 30 m of the stratigraphy. As per Table 4.1 of the CHBDC, the site has been assessed to be Seismic Site Class D based on the standard penetration resistance values.

The $F(PGA)$ as per Table 4.8 within Section 4.4.3.3 of the CHBDC is equal to 1.29 for a Site Class D yielding a scaled *site-specific* PGA of 0.045g.

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

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.



class. The $F(0.2)$, as per Table 4.2 within Section 4.4.3.3 of the CHBDC, is equal to 1.24 for this site yielding a scaled *site-specific* $S_a(0.2)$ of 0.08. A Seismic Performance Category of 1 would be applicable for this site based on Table 4.10 of the CHBDC.

9. FOUNDATION DESIGN RECOMMENDATIONS

9.1 Existing Abutment Footings

It is understood that the existing abutments/footings may be reused. The existing footings are shown on Contract Drawing 58-24 and are trapezoidal in shape with a width of 3.88 m (12' 8.75") and a maximum and minimum length at the front and back of the footing of 15.89 m (52' 1.5") and 8.13 m (26' 8"), respectively. The south and north abutments are founded at elev. 181.58 and 181.66 m (596' and 595.75'), respectively. The footings are indicated to be 0.91 m (3') below the riverbed and 6.1 m below the pavement. To evaluate the design of the rehabilitated structure, the existing footings can be based on factored geotechnical resistance value as follows:

- Factored Geotechnical Resistance at ULS: 325 kPa

The factored geotechnical resistance includes the following factors:

- Consequence factor (Ψ) of 1.0 (as per CHBDC, Table 6.1)
- Geotechnical resistance factor (as per CHBDC, Table 6.2)
 - $\phi_{gu} = 0.50$ (static analysis; typical degree of understanding)

The geotechnical resistance presented is for vertical concentric loading only and will need to be adjusted for the effects of inclined or eccentric loadings, where applicable, in accordance with CHBDC Clause 6.10.2. *The geotechnical resistance values presented herein assume the existing footings are structurally sound and the founding soils are stable and will not be disturbed.*

As the ULS value is close to the design bearing capacity of 3 tsf (Geocres 41N00-008) negligible settlement would occur and therefore the SLS condition will not govern if the loads are not increased. *If the loading is increased the ULS and SLS should be reevaluated to assess if the footing may need to be augmented.*

Resistance to lateral forces/sliding resistance between the base of existing concrete footing and the underlying soil should be evaluated in accordance with the CHBDC assuming an unfactored coefficient of base friction of 0.45. 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 pad.



9.2 Backfill and Lateral Earth Pressure

Structural backfill material should consist of Granular A or Granular B Type II meeting the OPSS.PROV 1010 specifications and SP110S06. The backfill must be in accordance with OPSS.PROV 902 and placed and compacted in accordance with OPSS.PROV 501 to the extents shown on OPSD 3121.150. The backfill should be compacted and compaction equipment to be used adjacent to the structure must be restricted in accordance with OPSS.PROV 501.07.02.

Lateral earth pressure provided in the equations in the sections 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 wall design.

9.2.1 Static Lateral Earth Pressure

Lateral earth pressures acting on vertical structures 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:

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

where:

σ_h	=	static lateral earth pressure on the wall at depth d (kPa)
K	=	static earth pressure coefficient (see table below)
γ	=	unit weight of retained soil (see table below) adjusted below water level
d	=	depth below top of fill where pressure is computed (m)
q	=	value of any surcharge (kPa)

A lateral earth pressure due to backfill compaction should be added to the calculated lateral earth pressure in accordance with Clause 6.12.3 of the CHBDC. Typical earth pressure coefficients for vertical walls for backfill material are shown in Table 9-1.

Table 9-1. 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 <i>Behind Wall</i>
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
Existing Granular Fill	21.0	0.33	0.5	3.0	Horizontal
		0.52	-	-	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.

If lateral movement is not permissible and/or the wall is restrained, the at-rest/non-yielding earth pressure coefficient should be used. If the wall design allows lateral movement, the active/yielding earth pressures should be used. Where ground surfaces are sloped at 2H:1V behind the walls, the corresponding coefficients provided in Table 9-1 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 soil layers.

9.2.2 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 9-2 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 D.

Table 9-2. 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	0.30	Horizontal
		0.42	0.45	2H:1V
OPSS Granular B Type II	22.0	0.28	0.30	Horizontal
		0.42	0.45	2H:1V
Existing Granular Fill	21.0	0.35	0.36	Horizontal
		0.56	0.60	2H:1V

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

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

where:

- σ_{hAE} = combined static and seismic lateral earth pressure on wall at depth d (kPa)
- d = depth below the top of the wall where pressure is computed (m)
- K = static earth pressure coefficient
(K_A for yielding walls, K_o for non-yielding walls)
- γ = unit weight of retained soil, adjusted below water level
- K_{AE} = combined static and seismic earth pressure coefficient
- H = total height of the wall (m)

9.3 Frost Depth

The frost penetration depth at this site is 2.2 m as per OPSD 3090.100.



For all footings a minimum of 2.2 m of earth cover, or thermal equivalent, must be provided above the base of the footings to serve as protection against frost. The earth cover should be measured perpendicular to the ground surface. Thermally equivalent frost protection could be in the form of insulation provided it is placed *above* the high-water level. It should be noted that porous materials, such as rock protection, does not have the same thermal protection as soil.

9.4 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 low ranging from <5 to 29 µg/g, see Section 5.6. 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 5.6 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.

9.5 Embankment Design and Reinstatement

9.5.1 Embankment Reinstatement

Structural backfill must be in accordance with OPSS.PROV 902 and placed and compacted in accordance with OPSS.PROV 501 to the extents shown on OPSD 3121.150.

Embankment reinstatement beyond the limits of OPSD 3121.150 should be carried out in accordance with OPSS.PROV 206. If constructed using Select Subgrade Material (SSM) or Granular B Type I, the embankment should be reconstructed with side slopes of 2H:1V, or flatter. The fill should be placed and compacted in accordance with OPSS.PROV 501.

Prior to placement of fill, topsoil organic or otherwise deleterious soils should be removed. 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.



9.5.2 Settlement

As the permanent embankment envelope is to remain unchanged and there will be no grade raise, there will be negligible settlement of the underlying soils as a result of embankment reinstatement.

The magnitude of the self-compression of an embankment constructed with granular materials is in the order of 0.5% of the reinstated height and is expected to occur predominantly during fill placement.

9.5.3 Global Stability

The existing grades are to be maintained. Provided the subgrade preparation and construction of the rehabilitation works are carried out in accordance with recommendations provided within this report and OPSS.PROV 206, the global stability of slope will be the same as the current conditions.

10. CONSTRUCTION CONSIDERATIONS

10.1 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 water table Type 3 soil. The non-cohesive 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 O.Reg. 213/91, s. 227 (3).*

Excavation should occur in a dewatered environment (see Section 10.3). 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 the structure must be carried out in accordance with OPSS.PROV 902 as amended by SP FOUN0003 and will be carried out through the existing fill. Please refer to Section 10.3 for designer fill-in recommendations. 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 the construction of temporary construction access roads are also the Contractor's responsibility. Placement of the crane or temporary stockpiling must not destabilize the embankment.

At locations where there are space restrictions or where a slope has to be retained or where the excavation are near the river, the excavations will need to be carried out within a protection system. Further discussion on temporary protection systems (TPS) is presented in Section 10.2.

10.2 Temporary Protection Systems

Temporary Protection Systems (TPS) 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.

Steel sheet piles are considered a suitable option, however, cobbles and boulders were noted in the fill and native soils. A suitable anchoring and/or bracing system may need to be incorporated into the temporary protection design to resist lateral earth pressure loadings. Note that the selection and design of roadway protection systems are the responsibility of the Contractor. All protection systems should be designed by a licensed Professional Engineer experienced in such designs and retained by the Contractor. The design of the roadway protection system must incorporate traffic loading and surcharge loading due to construction equipment and operations. As noted in Section 2.2, the sheet piles installed as part of Contract 58-24 may still be present near each abutment and need to be considered in TPS design. A notice to contractor should be provided in the contract concerning the presence of obstructions, see recommended text in Appendix F.

The TPS design must be coordinated with the dewatering plans to ensure the basal stability of the excavation. Sheet piles may require additional embedment below the base of the excavation.

Lateral earth pressure coefficients, under fully mobilized conditions, that can be used in design of the protection system installed through granular fill materials are provided in the Table 9-1 for static conditions. The lateral earth pressure coefficients for the existing soils are given below. Unit weights provided herein are to be adjusted when applied to depths below the water level.

Unbalanced hydrostatic pressures should also be considered in the design of the protection systems.

Table 10-1 Static Earth Pressure Coefficients for Existing Soil

Material	Unit Weight ^(*) (kN/m ³)	K _A	K _P	Ground Surface Behind Structure
Existing Sand to Gravel with Sand	21	0.33	3.0	Horizontal

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

The use of vibration for installation and/or removal of sheet pile protection systems/cofferdams could result in settlement and must be carried out cautiously. It is recommended that the protection systems within 3 m from the edges of the wingwalls/structure should be left in place and cut off in accordance with OPSS.PROV 539.

10.3 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 groundwater should be lowered to 0.5 m below the base of excavation prior to excavating.

Construction, subgrade preparation and placement and compaction of granular materials must be carried out in the dry. It is anticipated that the depth of excavation for the superstructure will be above the normal river water level. However, excavation depths of 500 mm are indicated on the GA Drawing (Sheet 84) to resurface portions of the abutments and the excavations will extend below the river and groundwater level. Surface water will tend to seep into and accumulate in the excavations. The Contractor must be prepared to control groundwater and surface water at the site. Water from either precipitation and/or surface flow must be diverted away from the excavation at all times. Dewatering and surface water diversion must remain operational and effective until the temporary excavations are backfilled.

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 dewatering systems in accordance with SP FOUN0003 which amends OPSS.PROV 902. Given the site conditions and anticipated works, the design Engineer and design-checking Engineer do need a minimum of 5 years of experience in designing similar dewatering systems and the Designer Fill-In ***** in SP517F01 Table A should be a "Yes". A preconstruction survey is required, thus Designer Fill-In ** in SP FOUN0003 should be "200 m".



The surface water level will fluctuate and the minimum water elevation for the site at the time of the excavation should be taken as the expected high-water level defined in the contract documents.

The dewatering plan must be coordinated with the TPS design. Construction of cofferdams will be required to divert flow away from the work on the wing wall and abutment faces. Excavations should be carried out in watertight sheet pile enclosure driven to sufficient depth to cut off the groundwater flow. Sheet piles can be designed following the recommendations provided in Section 10.2. The sheet piles should be installed with sufficient embedment below the base of the excavation to maintain basal stability of the excavation. Pumping should continue until control of inflow is achieved and the excavations are backfilled. Multiple pumps may be required. Pumping should be carried out to not cause settlement of the abutments.

Assessment of the 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.

10.4 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 materials indicate that the soils have a low potential for soil erodibility (Wischmeier Nomograph factor, K ranging from 0 to 0.15).

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 all 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. Existing erosion protection material at this location must be reinstated.

Scour protection must be provided along the waterline to protect the footings. Design of the protection measure 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.



11. CONSTRUCTION CONCERNS

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

- Obstructions (ie: cobbles, boulders, buried debris, old structure, old sheet piles)

Buried obstructions will be encountered during construction and will interfere with excavations and installation of temporary protection/dewatering systems. Sheet piles from the original construction are also present around the footings and shall not be removed. 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 foundation elements.

- Water Inflow

Excavation near the river level, where required, will involve lowering the groundwater level prior to excavation. It will be necessary to divert river flow away from excavation that extend below the river level. Dewatering may be difficult and a suitable dewatering / unwatering scheme will be critical for construction at this site.

- Equipment Selection

The Contractor's selection of construction equipment and methodology must include assessment of the capability of the existing soils to support the proposed construction equipment and supplies.

The 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.



12. CLOSURE

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

Thurber Engineering Ltd.
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STATEMENT OF LIMITATIONS AND CONDITIONS

1. STANDARD OF CARE

This Report has been prepared in accordance with generally accepted engineering or environmental consulting practices in the applicable jurisdiction. No other warranty, expressed or implied, is intended or made.

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All documents, records, data and files, whether electronic or otherwise, generated as part of this assignment are a part of the Report, which is of a summary nature and is not intended to stand alone without reference to the instructions given to Thurber by the Client, communications between Thurber and the Client, and any other reports, proposals or documents prepared by Thurber for the Client relative to the specific site described herein, all of which together constitute the Report.

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The Report has been prepared for the specific site, development, design objectives and purposes that were described to Thurber by the Client. The applicability and reliability of any of the findings, recommendations, suggestions, or opinions expressed in the Report, subject to the limitations provided herein, are only valid to the extent that the Report expressly addresses proposed development, design objectives and purposes, and then only to the extent that there has been no material alteration to or variation from any of the said descriptions provided to Thurber, unless Thurber is specifically requested by the Client to review and revise the Report in light of such alteration or variation.

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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.

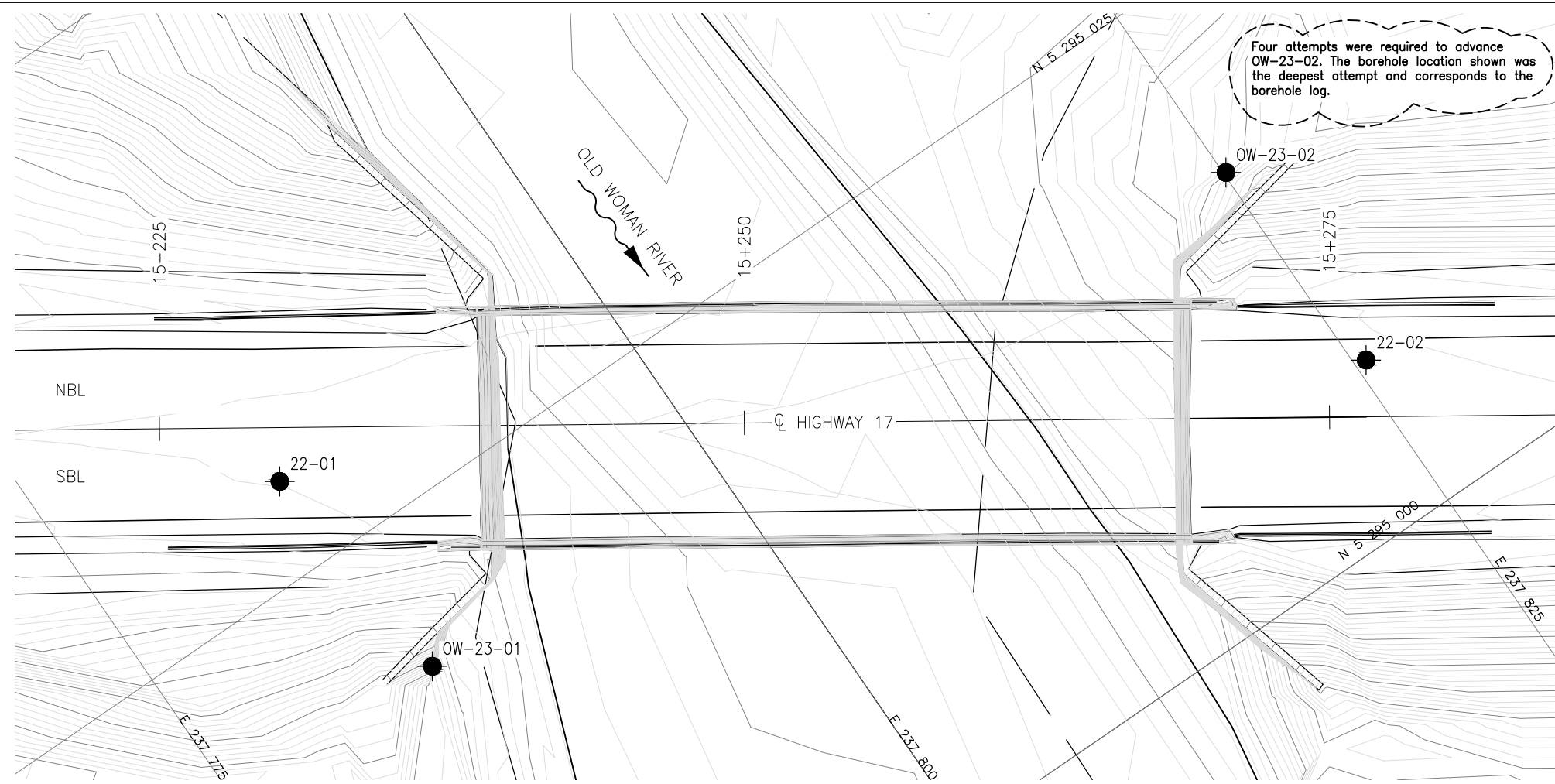
7. INDEPENDENT JUDGEMENTS OF CLIENT

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.



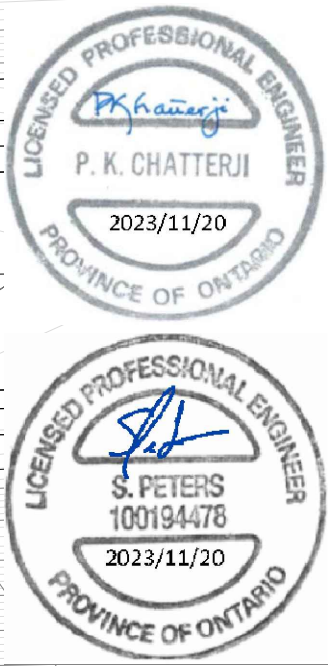
Appendix A Drawings

Borehole Locations and Strata Drawing
Historical Drawings and Borehole Records



Four attempts were required to advance OW-23-02. The borehole location shown was the deepest attempt and corresponds to the borehole log.

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



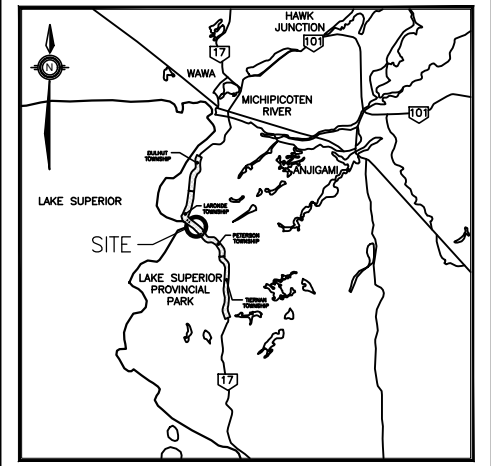
CONT No
GWP No 5207-18-00

HIGHWAY 17
OLD WOMAN RIVER BRIDGE
LARONDE TOWNSHIP
BOREHOLE LOCATIONS AND SOIL STRATA

Ontario

THURBER ENGINEERING LTD.

SHEET



KEYPLAN

LEGEND

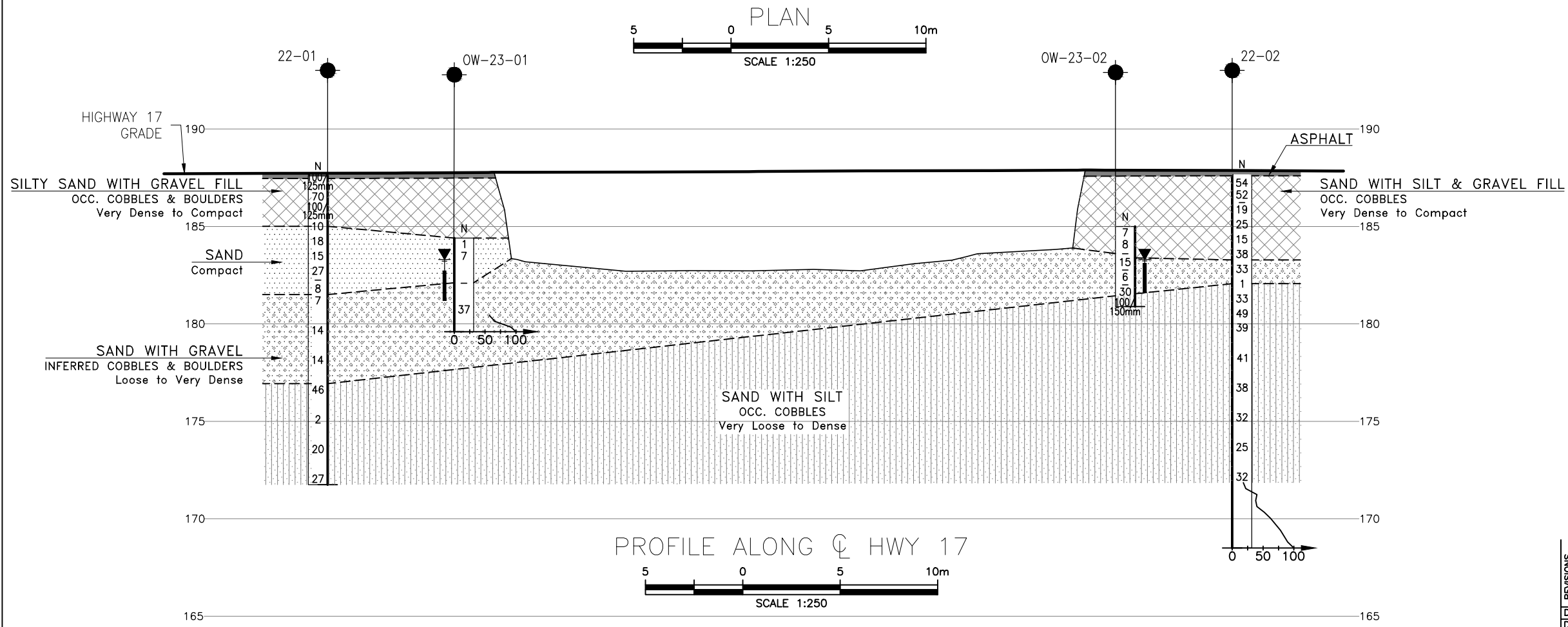
- Borehole
- Borehole (Previous Investigation)
- N Blows /0.3m (Std Pen Test, 475J/blow)
- CONE Blows /0.3m (60° Cone, 475J/blow)
- PH Pressure, Hydraulic
- Water Level
- Head Artesian Water
- Piezometer
- 90% Rock Quality Designation (RQD)
- A/R Auger Refusal

NO	ELEVATION	NORTHING	EASTING
22-01	187.6	5 295 028.9	237 784.3
22-02	187.7	5 295 006.9	237 825.5
OW-23-01	184.4	5 295 018.7	237 785.2
OW-23-02	185.1	5 295 016.9	237 825.1

-NOTES-

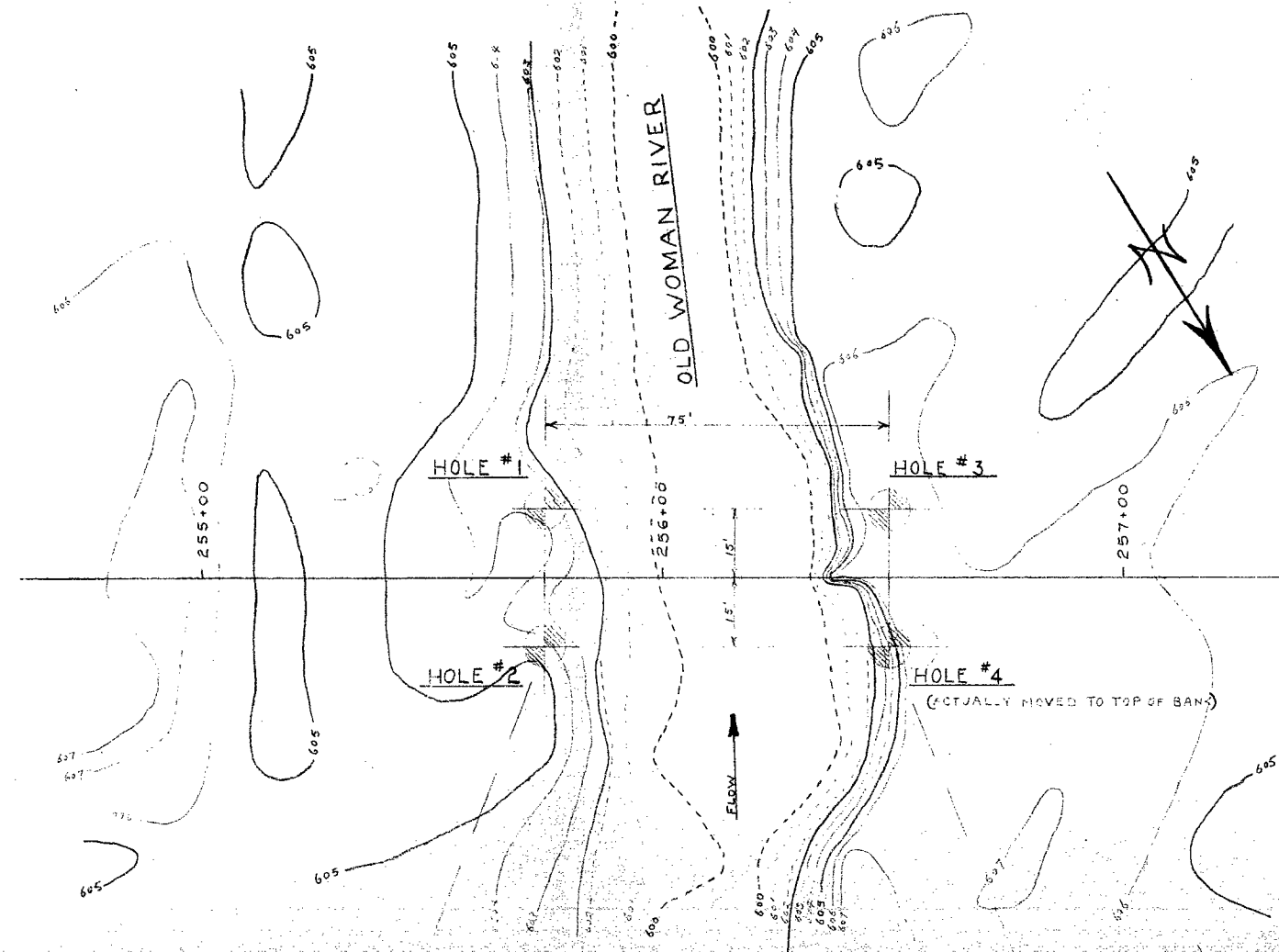
- The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.
- This drawing is for subsurface information only. Surface details and features are for conceptual illustration.
- Coordinate system is MTM NAD 83 Zone 13.

GEOCRES No. 41N00-036



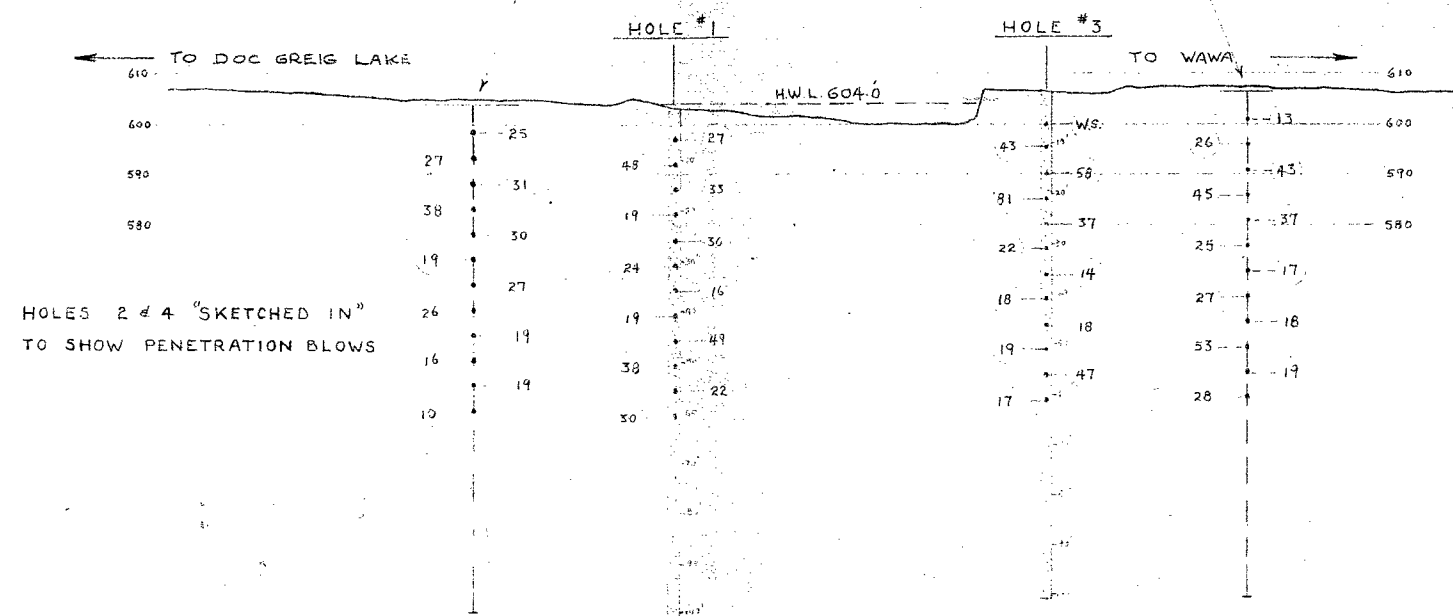
PROFILE ALONG HWY 17

REVISIONS	DATE	BY	DESCRIPTION
DESIGN	AO	CHK	CODE
DRAWN	AN	CHK	SITE
			LOAD
			STRUCT
			DATE
			NOV 2023
			DWG F-1



LEGEND

- 2" O.D. SPLIT BARREL SAMPLE
- 16 STD. PENETRATION TEST BLOWS
- 4200 IN. LBS. BLOWS PER FOOT



HOLES 2 & 4 "SKETCHED IN"
TO SHOW PENETRATION BLOWS

SCALES: HOR. 1" = 20'
VER. 1" = 20'



e.m. peto & associates ltd.	
SOIL SITE INVESTIGATION AT OLD WOMAN RIVER--HWY.17 BRIDGE	
FOR DEPARTMENT OF HIGHWAYS OF ONTARIO	
OUR JOB No. 5790	DATE AUG. 26/57
CLIENTS PLAN No. E-3187	PER. <i>lu</i>



Appendix B Record of Borehole Sheets

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).

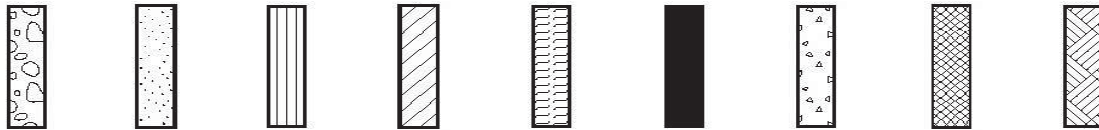
DYNAMIC CONE PENETRATION TEST (DCPT):

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 22-01

1 OF 2

METRIC

GWP# 5207-18-00 LOCATION Lat: 47.790835°, Long: -84.894512° Old Woman River, Laronde Township, MTM z13: N 5 295 028.9 E 237 784.3 ORIGINATED BY AO/AH
 HWY 17 BOREHOLE TYPE CME 75 Truck Mounted / HW Casing / NQ Coring COMPILED BY AO
 DATUM Geodetic DATE 2022.08.04 - 2022.08.05 CHECKED BY SP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL					
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										
								20 40 60 80 100		PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT								
							W P W W L											
							WATER CONTENT (%)											
							○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE											
							20 40 60 80 100 20 40 60											
187.6	Asphalt Surface																	
0.0	ASPHALT (140 mm)																	
0.1	SILTY SAND with Gravel Occasional Cobbles and Boulders Very dense to compact Brownish grey FILL - 240 mm Boulder at a depth of 2.0 m		1	SS	100/ 125mm							○						
			2	SS	70								○					21 53 23 3
			3	SS	100/ 125mm								○					
			1	NQ	-													
			4	SS	10									○				
185.0																		
2.6	SAND (SP) Occasional Cobbles Compact Light brown		5	SS	18							○						
			6	SS	15								○					3 96 1 (SI+CL)
			7	SS	27								○ ○					
			2	NQ	-													
			8	SS	8									○				0 95 5 (SI+CL)
181.5																		
6.1	GRAVEL (GW) with Sand to SAND (SP) with Gravel Inferred Cobbles and Boulders Loose to compact Light greyish brown		9	SS	7							○						
			10	SS	14								○					72 27 1 (SI+CL)
			11	SS	14													

Continued Next Page

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

DOUBLE LINE 31653 HWY 17 OLD WOMAN RIVER.GPJ 2012TEMPLATE(MTO).GDT 11-20-23

METRIC

SOIL PROFILE						GROUND WATER CONDITIONS		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES		ELEVATION SCALE			
			NUMBER	"N" VALUES				
	Continued From Previous Page				DYNAMIC CONE PENETRATION RESISTANCE PLOT 	NATURAL MOISTURE CONTENT LIMIT 	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
176.9	GRAVEL (GW) with Sand to SAND (SP) with Gravel Inferred Cobbles and Boulders Loose to compact Light greyish brown	[Pattern]						
10.7	SAND (SP-SM) with Silt Inferred Cobbles Very loose to dense Light grey	[Pattern]	12	SS 46				0 92 8 0
	- At a depth of 12.8 m, drilling method was switched from NW to HW casing as a result of flowing sands.	[Pattern]	13	SS 2				
		[Pattern]	14	SS 20				
	- Wood fragments at a depth of 15.2 m	[Pattern]	15	SS 27				0 91 8 1
171.8 / 15.8	End of Borehole Water was introduced into the borehole as part of the drilling procedure. An open-hole water level may not be representative of groundwater conditions.							

+³, ×³: Numbers refer to Sensitivity

RECORD OF BOREHOLE No 22-02

1 OF 2

METRIC

GWP# 5207-18-00 LOCATION Lat: 47.790641°, Long: -84.893961° Old Woman River, Laronde Township, MTM z13: N 5 295 006.9 E 237 825.5 ORIGINATED BY AO/AH
 HWY 17 BOREHOLE TYPE CME 75 Truck Mounted / HW Casing / NQ Coring COMPILED BY AO
 DATUM Geodetic DATE 2022.08.05 - 2022.08.06 CHECKED BY SP

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					
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL	× LAB VANE		
187.7	Asphalt Surface												
0.0	ASPHALT (110 mm)												
0.1	SAND with Silt and Gravel Occasional Cobbles Very dense to compact Brown to grey FILL		1	SS	54								
			2	SS	52								40 52 8 (SI+CL)
			1	NQ	-								
			3	SS	19								
			4	SS	25								
			5	SS	15								18 78 4 (SI+CL)
			6	SS	38								
183.3													
4.4	SAND (SP) with Gravel Inferred Cobbles Dense to very loose Light grey		7	SS	33								45 51 4 (SI+CL)
			8	SS	1								
182.1													
5.6	SAND (SP to SP-SM) with Silt Inferred Cobbles Dense to Compact Light grey		9	SS	33								
			10	SS	49								
			11	SS	39								4 92 4 (SI+CL)
			12	SS	41								

Continued Next Page

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

DOUBLE LINE 31653 HWY 17 OLD WOMAN RIVER.GPJ 2012TEMPLATE(MTO).GDT 11-20-23

METRIC

[illegible]

DOUBLE LINE 31653 HWY 17 OLD WOMAN RIVER.GPJ 2012TEMPLATE(MTO).GDT 11-20-23

+³, ×³: Numbers refer to Sensitivity

RECORD OF BOREHOLE No OW-23-01

1 OF 1

METRIC

GWP# 5207-18-00 LOCATION Lat: 47.790743°, Long: -84.894498° Old Woman River, Laronde Township, MTM z13: N 5 295 018.7 E 237 785.2 ORIGINATED BY IK
 HWY 17 BOREHOLE TYPE Portable / NW Casing / NQ Coring COMPILED BY RH
 DATUM Geodetic DATE 2023.06.26 - 2023.06.27 CHECKED BY SP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa			W P	W	W L			GR	SA
184.4	Ground Surface							20	40	60	80	100					
0.0	SAND (SP-SM to SW-SM) with Silt trace to with Gravel contains cobbles very loose to dense light brown to greyish brown		1	SS	1		184									6	89 5 (SI+CL)
			2	SS	7											35	59 6 (SI+CL)
							183										
	- Flowing sands noted below a depth of 2.3 m		3	NQ	-		182										
							181										
180.4			4	SS	37											13	78 6 3
4.0	End of sampled borehole Borehole advanced with DCPT						180										
179.6																	
4.8	End of Borehole on DCPT refusal																
	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) 2023/06/02 1.1 183.3 2023/06/09 1.1 183.3 2023/06/15 1.1 183.3 2023/07/06 1.1 183.3 2023/07/14 1.1 183.3 Note 1: A third-weight hammer was used to advance the split-spoon sampler. The N values presented above have been adjusted to a standard hammer. Note 2: A full-weight hammer was used to advance the DCPT.																

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

RECORD OF BOREHOLE No OW-23-02

1 OF 1

METRIC

GWP# 5207-18-00 LOCATION Lat: 47.790732°, Long: -84.893954° Old Woman River, Laronde Township, MTM z13: N 5 295 016.9 E 237 826.0 ORIGINATED BY IK
 HWY 17 BOREHOLE TYPE Portable / NW Casing / NQ Coring COMPILED BY RH
 DATUM Geodetic DATE 2023.05.25 - 2023.06.11 CHECKED BY SP

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									WATER CONTENT (%)																		
185.1	Ground Surface							20	40	60	80	100					GR	SA	SI	CL															
0.0	SAND (SP to SP-SM) trace to with Silt, trace Gravel contains cobbles loose to dense light brown to greyish brown		1	SS	7		185										7	91	2	(SI+CL)															
			2	SS	8																														
			3	NQ	-		184																												
			4	SS	15																														
			5	NQ	-		183																												
			6	SS	6																														
			7	NQ	-																														
			8	SS	30		182										9	82	6	3															
			9	SS	100																														
181.0							181																												
4.1	End of Borehole - Practical Portable Refusal Monitoring Well installed: Schedule 40 PVC standpipe with 32-mm diameter and 1.5-m slotted screen. Water Level Readings: <table><tr><th>DATE</th><th>DEPTH (m)</th><th>ELEV. (m)</th></tr><tr><td>2023/07/08</td><td>1.7</td><td>183.4</td></tr><tr><td>2023/07/11</td><td>1.7</td><td>183.4</td></tr><tr><td>2023/07/13</td><td>1.7</td><td>183.4</td></tr><tr><td>2023/07/14</td><td>1.7</td><td>183.4</td></tr></table> Note 1: A third-weight hammer was used to advance the split-spoon sampler. The N values presented above have been adjusted to a standard hammer. Note 2: Three other attempts were made and could not advance the borehole deeper. 4.1 m.	DATE	DEPTH (m)	ELEV. (m)	2023/07/08	1.7	183.4	2023/07/11	1.7	183.4	2023/07/13	1.7	183.4	2023/07/14	1.7	183.4																			
DATE	DEPTH (m)	ELEV. (m)																																	
2023/07/08	1.7	183.4																																	
2023/07/11	1.7	183.4																																	
2023/07/13	1.7	183.4																																	
2023/07/14	1.7	183.4																																	

+³, ×³: Numbers refer to Sensitivity 20 15 10 5 0 (%) 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: OW-23-01

Test Well: OW-23-01

Test Conducted by: SM & IK

Test Date: 2023-07-14

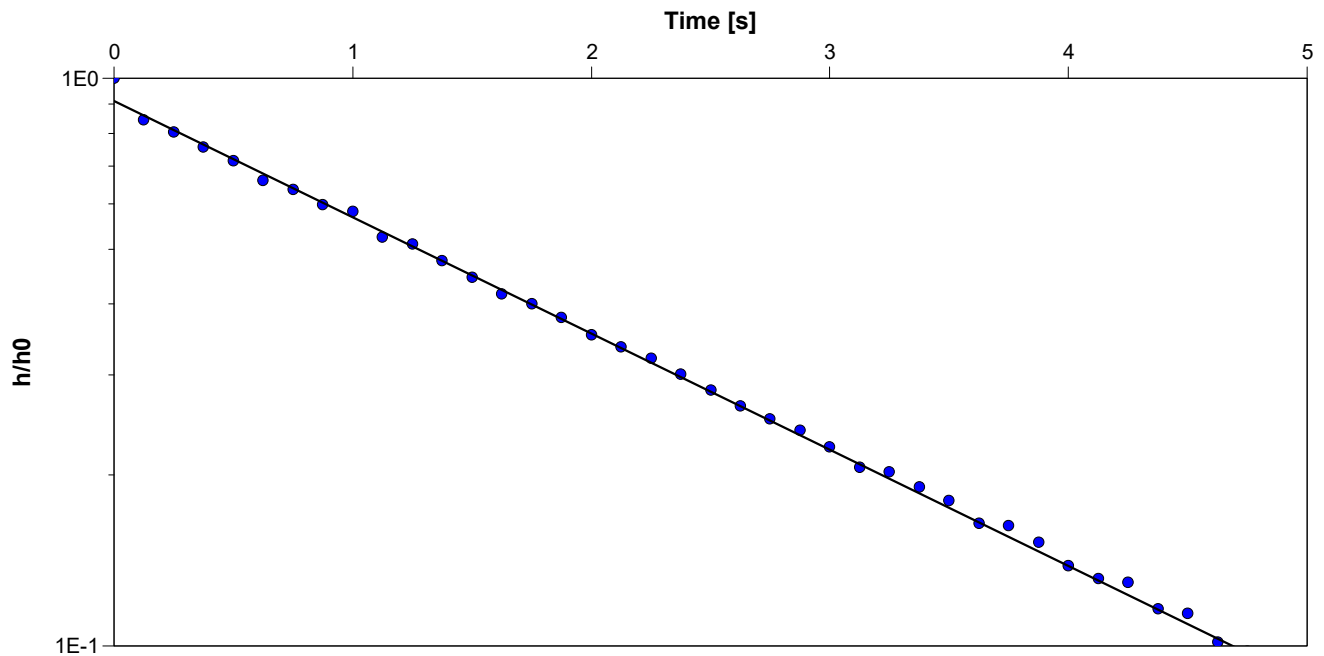
Analysis Performed by: SM

SWRT Analysis

Analysis Date: 2023-07-18

Aquifer Thickness:

Checked by: AH



Calculation using Hvorslev

Observation Well

Hydraulic Conductivity
[m/s]

OW-23-01

4.2×10^{-4}



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: OW-23-02

Test Well: OW-23-02

Test Conducted by: SM & IK

Test Date: 2023-07-14

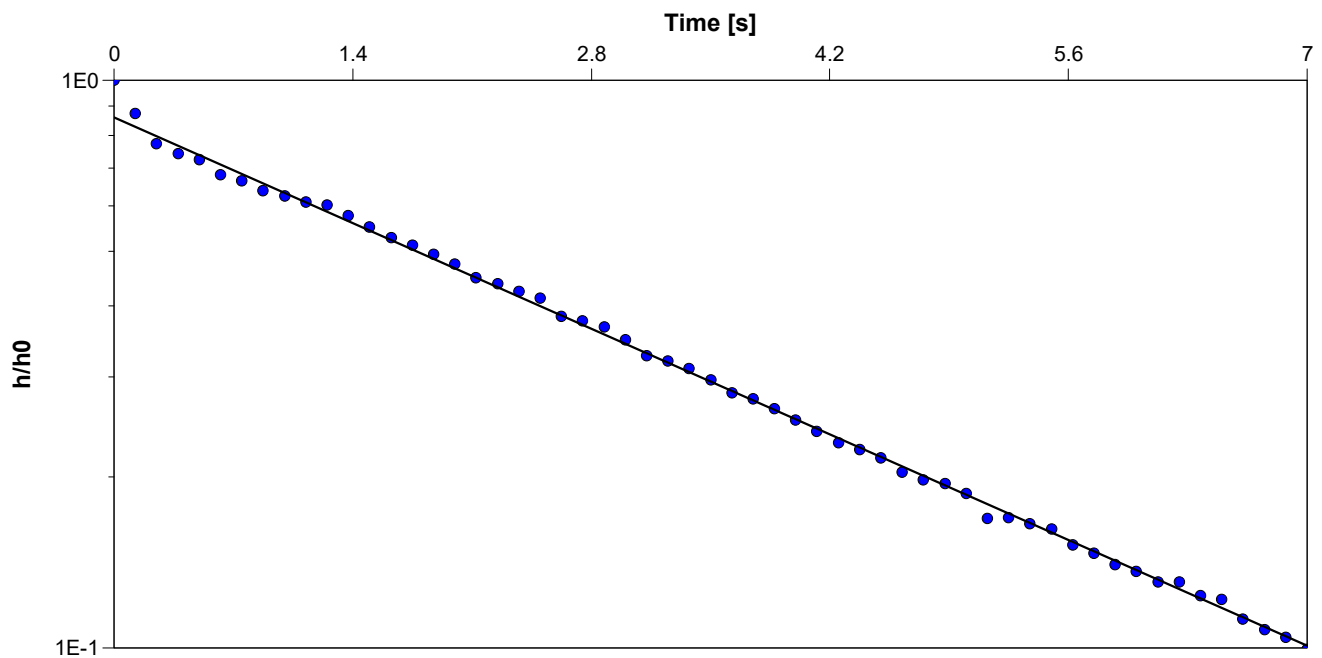
Analysis Performed by: SM

SWRT Analysis

Analysis Date: 2023-07-18

Aquifer Thickness:

Checked by: AH



Calculation using Hvorslev

Observation Well

Hydraulic Conductivity
[m/s]

OW-23-02

2.7×10^{-4}



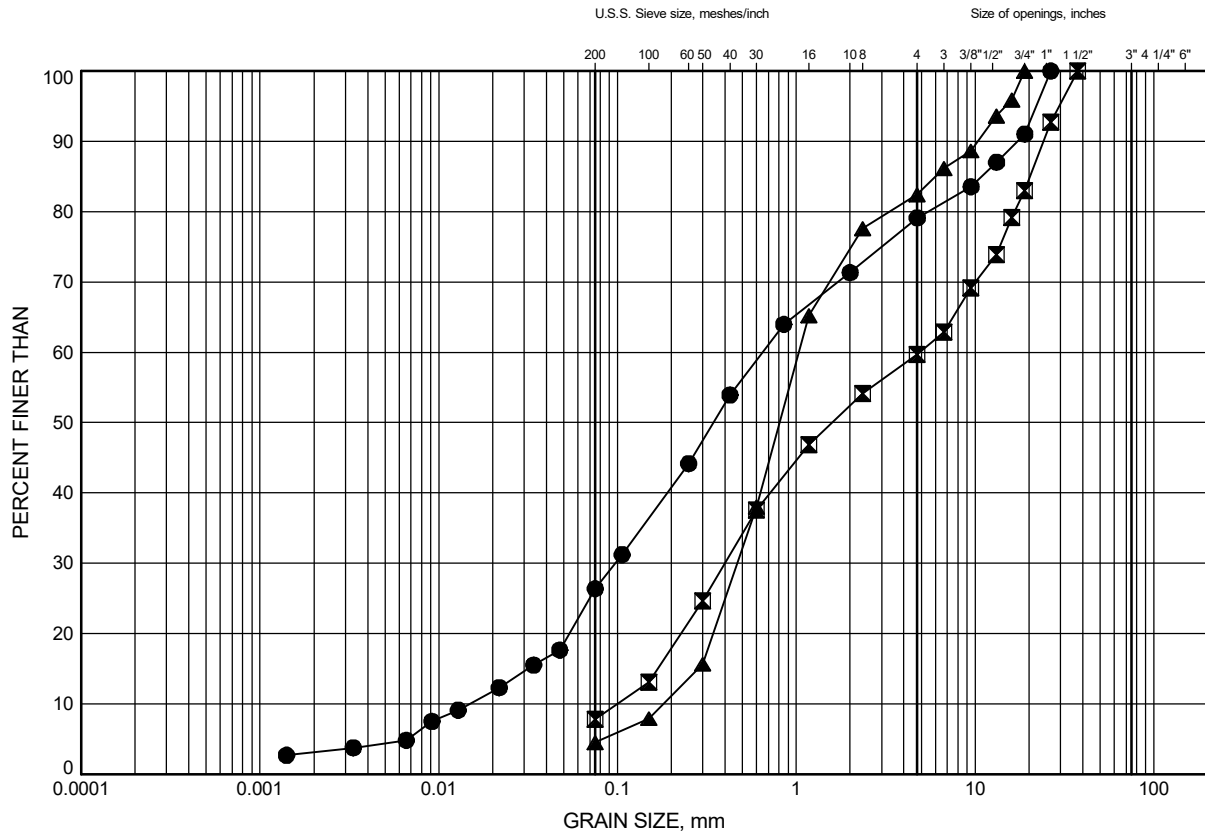
Appendix C Laboratory Testing

Particle Size Analysis Figures Analytical Testing Results

Highway 17 Old Woman River GRAIN SIZE DISTRIBUTION

FIGURE C1

FILL: Silty Sand with Gravel to Sand with Silt and Gravel



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	22-01	1.1	186.5
⊠	22-02	1.1	186.6
▲	22-02	3.4	184.3

Date August 2023
GWP# 5207-18-00

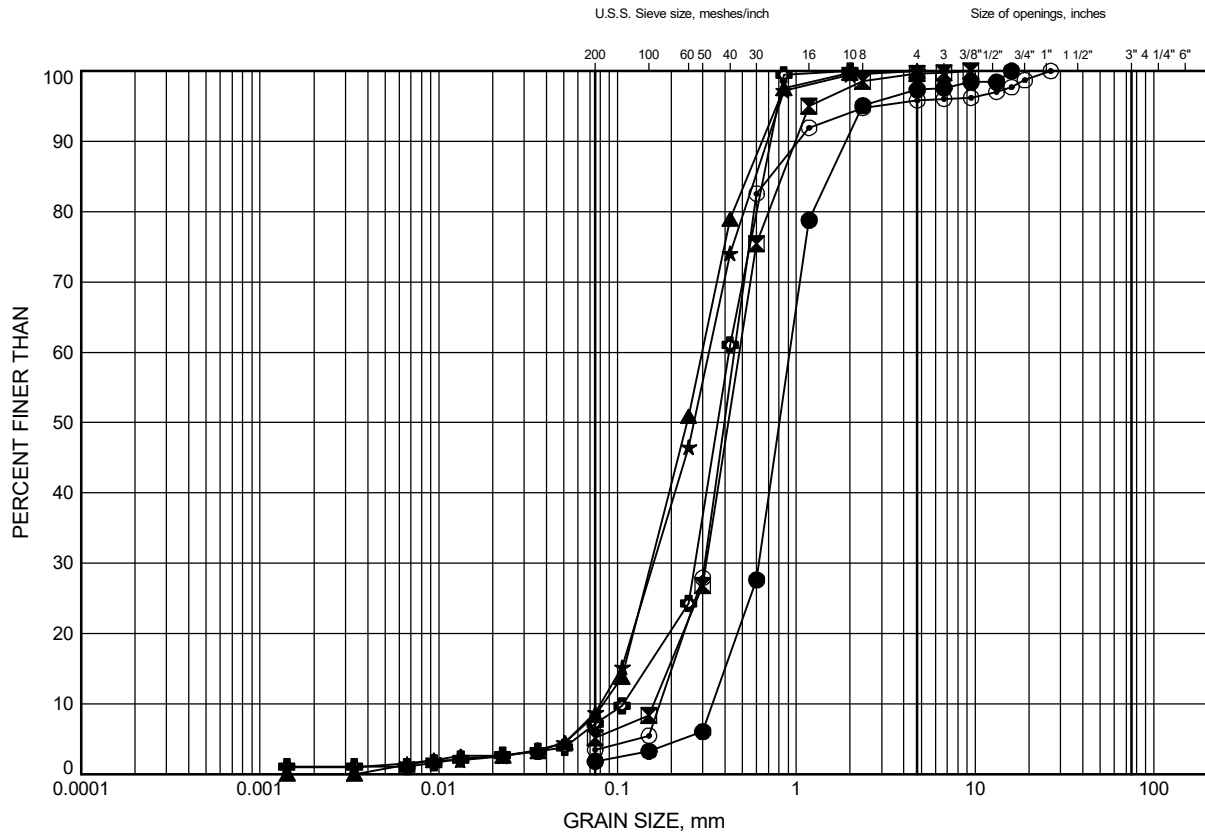


Prep'd RH
Chkd. AO

Highway 17 Old Woman River GRAIN SIZE DISTRIBUTION

FIGURE C2

Sand with Silt



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	22-01	4.1	183.5
⊠	22-01	5.8	181.8
▲	22-01	11.0	176.6
★	22-01	15.5	172.1
⊙	22-02	7.9	179.8
⊕	22-02	14.0	173.7

Date August 2023
GWP# 5207-18-00

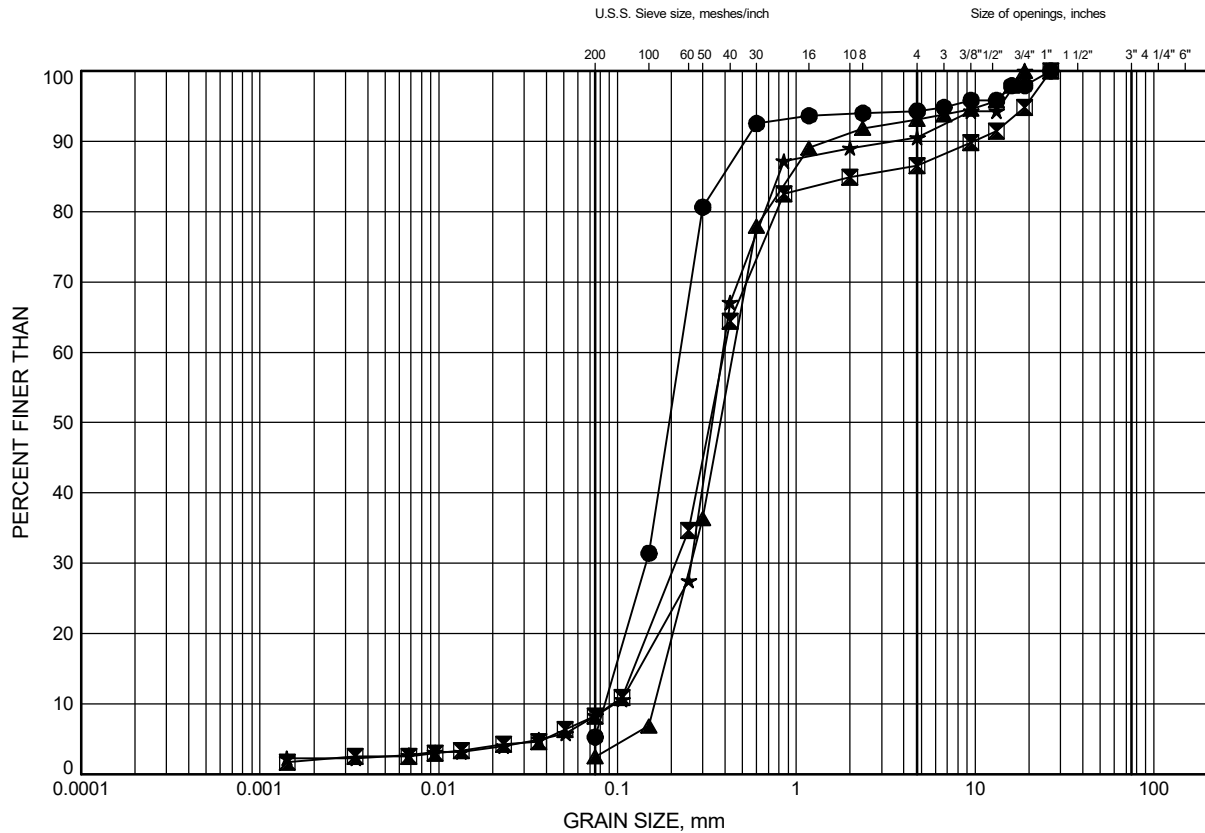


Prep'd RH
Chkd. AO

Highway 17 Old Woman River GRAIN SIZE DISTRIBUTION

FIGURE C3

Sand with Silt



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	OW-23-01	0.3	184.1
⊠	OW-23-01	3.7	180.7
▲	OW-23-02	0.3	184.8
★	OW-23-02	3.4	181.7

Date August 2023
GWP# 5207-18-00

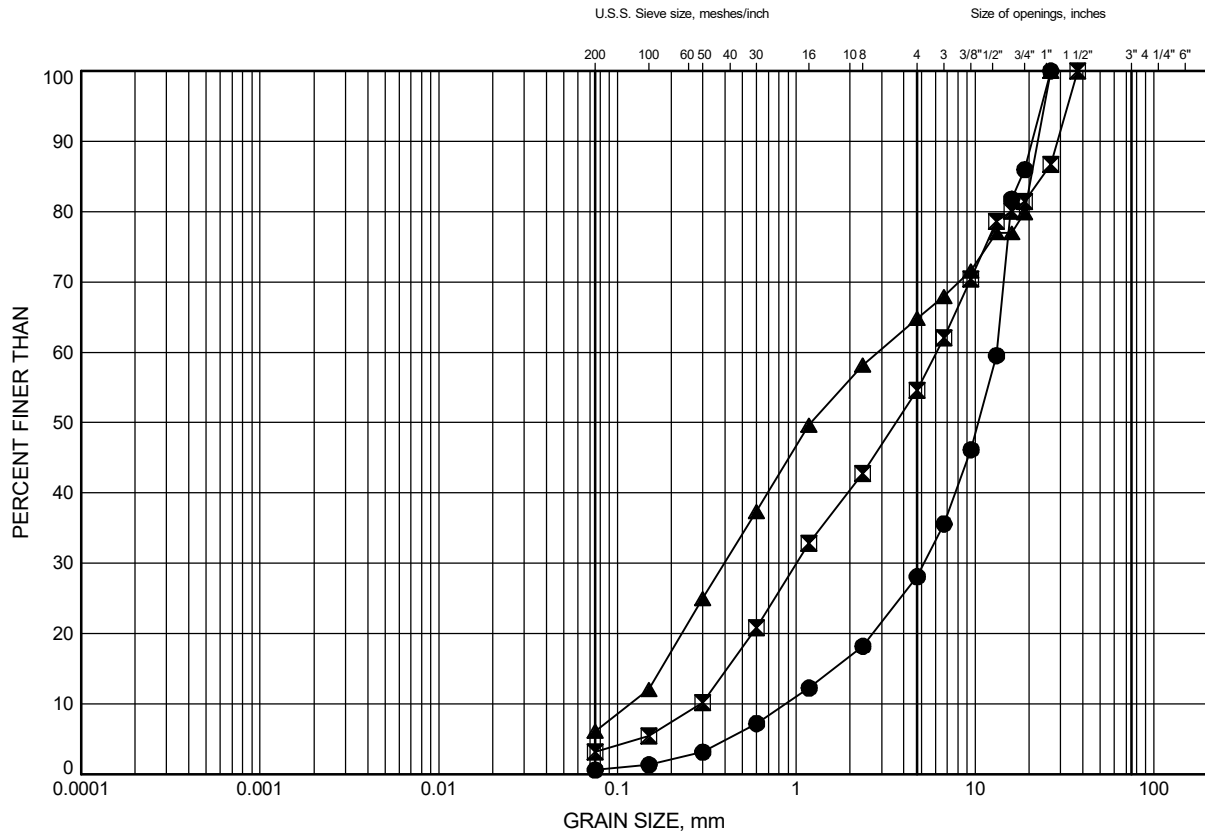


Prep'd RH
Chkd. AO

Highway 17 Old Woman River GRAIN SIZE DISTRIBUTION

FIGURE C4

Sand with Gravel to Gravel with Sand



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	22-01	7.9	179.7
⊠	22-02	4.9	182.8
▲	OW-23-01	0.9	183.5

Date August 2023
GWP# 5207-18-00



Prep'd RH
Chkd. AO

Certificate of Analysis

Report Date: 22-Aug-2022

Client: Thurber Engineering Ltd.

Order Date: 12-Aug-2022

Client PO:

Project Description: 31653 Hwy 17 Old Woman River

Client ID:	22-01 SS5 (10'-12')	22-02 SS6 (12'6"-14'6")	-	-	
Sample Date:	04-Aug-22 09:00	05-Aug-22 09:00	-	-	-
Sample ID:	2233615-01	2233615-02	-	-	-
Matrix:	Soil	Soil	-	-	-
MDL/Units					

Physical Characteristics

% Solids	0.1 % by Wt.	92.7	90.4	-	-	-	-
----------	--------------	------	------	---	---	---	---

General Inorganics

Conductivity	5 uS/cm	86	158	-	-	-	-
pH	0.05 pH Units	7.38	6.88	-	-	-	-
Resistivity	0.1 Ohm.m	116	63.4	-	-	-	-

Anions

Chloride	5 ug/g	5	39	-	-	-	-
Sulphate	5 ug/g	<5	29	-	-	-	-

Certificate of Analysis

Report Date: 24-Aug-2022

Client: Thurber Engineering Ltd.

Order Date: 17-Aug-2022

Client PO:

Project Description: 31653 Hwy 17 Old Woman River

Client ID:	22-02 SS9 (20'-22')	-	-	-	-
Sample Date:	06-Aug-22 09:00	-	-	-	-
Sample ID:	2234368-01	-	-	-	-
Matrix:	Soil	-	-	-	-
MDL/Units					

Physical Characteristics

% Solids	0.1 % by Wt.	81.3	-	-	-	-
----------	--------------	------	---	---	---	---

General Inorganics

Conductivity	5 uS/cm	45	-	-	-	-
pH	0.05 pH Units	7.15	-	-	-	-
Resistivity	0.1 Ohm.m	220	-	-	-	-

Anions

Chloride	5 ug/g	<5	-	-	-	-
Sulphate	5 ug/g	6	-	-	-	-

Certificate of Analysis

Report Date: 06-Jul-2023

Client: Thurber Engineering Ltd.

Order Date: 27-Jun-2023

Client PO:

Project Description: 31653 Hwy 17 Old Woman River

Client ID:	OW-23-02 SS8 (10'-12')	-	-	-
Sample Date:	25-May-23 09:00	-	-	-
Sample ID:	2326229-01	-	-	-
MDL/Units	Soil	-	-	-

Physical Characteristics

% Solids	0.1 % by Wt.	82.3	-	-	-
----------	--------------	------	---	---	---

General Inorganics

Conductivity	5 uS/cm	18 [1]	-	-	-
pH	0.05 pH Units	6.84 [1]	-	-	-
Resistivity	0.1 Ohm.m	547	-	-	-

Anions

Chloride	10 ug/g dry	<10 [1]	-	-	-
Sulphate	10 ug/g dry	<10 [1]	-	-	-

Certificate of Analysis

Client: Thurber Engineering Ltd.

Client PO:

Report Date: 06-Jul-2023

Order Date: 27-Jun-2023

Project Description: 31653 Hwy 17 Old Woman River

Qualifier Notes:

Login Qualifiers :

Sample - One or more parameter received past hold time - Conductivity, chloride, sulphate, pH, and sulphide.

Applies to samples: OW-23-02 SS8 (10'-12')

Sample Qualifiers :

- 1 : Holding time had been exceeded upon receipt of the sample at the laboratory or prior to the analysis being requested.
- 3 : OW-23-02 SS8 (10'-12') - client confirmed sample collected May 25, 2023 as per the collection date on the sample, and not June 1, 2023 as per the COC.

Sample Data Revisions

None

Work Order Revisions / Comments:

None

Other Report Notes:

n/a: not applicable

ND: Not Detected

MDL: Method Detection Limit

Source Result: Data used as source for matrix and duplicate samples

%REC: Percent recovery.

RPD: Relative percent difference.

NC: Not Calculated

Soil results are reported on a dry weight basis when the units are denoted with 'dry'.

Where %Solids is reported, moisture loss includes the loss of volatile hydrocarbons.

**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-August-2022

Date Rec. : 17 August 2022
LR Report: CA13718-AUG22
Reference: Project#: 2233615

Copy: #1

CERTIFICATE OF ANALYSIS

Final Report

Sample ID	Sample Date & Time	Sulphide (Na ₂ CO ₃) %
1: Analysis Start Date		24-Aug-22
2: Analysis Start Time		09:03
3: Analysis Completed Date		25-Aug-22
4: Analysis Completed Time		17:01
5: QC - Blank		< 0.04
6: QC - STD % Recovery		112%
7: QC - DUP % RPD		ND
8: RL		0.02
9: 22-01 SS5 (10'-12')	04-Aug-22	< 0.04
10: 22-02 SS6 (12'6"-14'6")	05-Aug-22	< 0.04

RL - SGS Reporting Limit
ND - Not Detected

Kimberley Didsbury
Project Specialist,
Environment, Health & Safety

**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

18-July-2023

Date Rec. : 30 June 2023
LR Report: CA19636-JUN23
Reference: Project#: 2326229

Copy: #1

CERTIFICATE OF ANALYSIS

Final Report

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: OW-23-02 SS8 (10'-12")	25-May-23 09:00	< 0.04

RL - SGS Reporting Limit
ND - Not Detected

Kimberley Didsbury
Project Specialist,
Environment, Health & Safety



Appendix D Site Photographs



Photograph 1: Looking west of bridge *[taken August 2022]*



Photograph 2: Looking east of bridge *[taken August 2022]*



Photograph 3: Looking south of bridge *[taken August 2022]*



Photograph 4: Looking north of bridge *[taken August 2022]*



Photograph 5: Looking at north wing wall and abutment *[taken August 2022]*



Photograph 6: Looking at south wing wall and abutment *[taken August 2022]*



Photograph 7: Looking north at river alignment *[taken August 2022]*



Photograph 8: Erosion at the south abutment's west side slope *[taken August 2022]*



Photograph 9: Sheet piles below water at the east side of the north abutment *[taken August 2022]*



Photograph 10: Deposited sand and tree debris near the south abutment *[taken August 2022]*



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: 47.791N 84.894W

User File Reference: Old Woman River

2022-12-20 19:57 UT

Probability of exceedance per annum	0.000404	0.001	0.0021	0.01
Probability of exceedance in 50 years	2 %	5 %	10 %	40 %
Sa (0.05)	0.046	0.023	0.014	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	0.022	0.007
Sa (0.5)	0.043	0.028	0.018	0.005
Sa (1.0)	0.026	0.017	0.011	0.003
Sa (2.0)	0.013	0.008	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 List of Referenced Specifications and Contract Provisions



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

- OPSD 208.010
- OPSD 219.110
- OPSD 3090.100
- OPSD 3121.150
- OPSS.PROV 206
- OPSS.PROV 501
- OPSS.PROV 511
- OPSS.PROV 539
- OPSS.PROV 803
- OPSS.PROV 804
- OPSS.PROV 805
- OPSS.PROV 902
- OPSS.PROV 1010
- SP 110S06
- SP FOUN0003

2. Contract Provisions - Obstructions

Buried obstructions will be encountered during construction and interfere with excavations and installation of temporary protection/dewatering systems. Sheet piles from the original construction are also present around the footings and shall not be removed. 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 foundation elements.