



THURBER ENGINEERING LTD.

**PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT
NESTOR FALLS BRIDGE REHABILITATION OR REPLACEMENT
HIGHWAY 71, NESTOR FALLS, ONTARIO
AGREEMENT 6021-E-0005, WORK ORDER 2
G.W.P. 6055-18-00, SITE NO. 41S-0074/B0
LATITUDE: 49.115225°, LONGITUDE: -93.926147°**

GEOCRES No.: 52F-69

Report

to

HATCH

Date: August 28, 2023
File: 34988



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PART 1: FACTUAL INFORMATION

1. INTRODUCTION

This report presents the factual data obtained from a foundation investigation carried out by Thurber Engineering Ltd. (Thurber) to support the preliminary design for rehabilitation or replacement of the Nestor Falls Bridge. The Nestor Falls Bridge is located on Highway 71, in Nestor Falls, Ontario.

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

Thurber carried out the investigation as a sub-consultant to Hatch, under the Ministry of Transportation Ontario (MTO) Retainer Agreement Number 6021-E-0005, Work Order 2.

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

The site is located on Highway 71 approximately 300 m south of Arrowhead Road in Nestor Falls, Ontario. Highway 71 and the existing bridge are aligned in a general north-south direction across Nestor Falls, which flows from Kakabikitchiwan Lake on the east side of Highway 71 to Lake of the Woods (Sabaskong Bay) on the west side.

The Nestor Falls bridge is situated within the community of Nestor Falls. The bridge is located adjacent to the Nestor Falls dam spillway structure, which is on the west of Highway 71 (see Photos D3 and D4 in Appendix D). The site is surrounded by lakes to the east and west, with nearby residential and commercial properties located north and south of the bridge. The



surrounding lands are heavily forested with bedrock outcrops visible nearby and along the highway. The existing bridge is an approximately 15 m long, single span concrete bridge supported on spread footings likely founded on bedrock. Photographs of the bridge and surrounding area are presented in Appendix D.

Based on published geological information, the general site area lies within the physiographic region known as the Canadian Shield, characterized by Precambrian bedrock (Foliated tonalite suite; consisting of folded tonalite to foliated to massive granodiorite) exposed at the ground surface or covered by a discontinuous thin layer of drift.

3. INVESTIGATION PROCEDURES

The site investigation and field-testing program for this project was carried out from November 9th to November 11th, 2022. The field program consisted of drilling and sampling two (2) boreholes, 22-01 and 22-02, to depths of 7.2 m and 9.1 m below the existing ground surface (Elevation 324.9 and 322.9 m), respectfully.

Boreholes 22-01 and 22-02 were drilled through the paved portion of Highway 71. The approximate borehole locations are shown on the attached Borehole Locations and Soil Strata Drawing in Appendix A. The Record of Borehole sheets are included in Appendix B.

Utility clearances were obtained prior to the start of drilling. The ground surface elevations for the boreholes were estimated from field measurements and the topographic drawings provided to Thurber by Hatch. The coordinate system MTM NAD 83, Zone 16 was used for the boreholes.

The boreholes for the project were advanced using a truck-mounted CME75 drill rig, using wash boring techniques and NQ coring. In all boreholes, soil samples were obtained at selected intervals with a 50 mm outside diameter split spoon sampler driven in conjunction with the Standard Penetration Test (SPT). Bedrock coring using an NQ size core barrel was used to advance both boreholes into bedrock.

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

The rock cores were logged, and the Total Core Recovery (TCR), Solid Core Recovery (SCR), Rock Quality Designation (RQD) and the Fracture Indices (FI) were measured.

Details of the drilling program, including drilling depths and completion details are summarized in Table 3.1 below.

Table 3.1: Borehole Completion Details

Borehole Number	Borehole Depth / Base Elevation (m)	Completion Details
22-01	7.2 / 324.9	Borehole was backfilled with bentonite holeplug to 0.5 m, then concrete to 0.2 m, then asphalt to surface.
22-02	9.1 / 322.9	Borehole was backfilled with bentonite holeplug to 0.5 m, then concrete to 0.2 m, then asphalt to surface.

4. LABORATORY TESTING

All recovered soil samples were subjected to visual identification and natural moisture content determination. Selected samples were subjected to grain size distribution analyses (sieve and hydrometer). Point load tests and two Unconfined Compressive Strength (UCS) tests were conducted on selected rock core samples. The results of this testing program are summarized on the Record of Borehole sheets in Appendix B and are shown on the figures included in Appendix C.

5. DESCRIPTION OF SUBSURFACE CONDITIONS

Details of the encountered soil stratigraphy are presented on the Record of Borehole sheets in Appendix B and on the Borehole Locations and Soil Strata drawing 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 in the Record of Borehole sheets governs any interpretation of the site conditions. It must be recognized that soil conditions may vary between and beyond the borehole locations.

In general, the subsurface stratigraphy below the pavement structure (asphalt and concrete) typically consists of sand to sand and gravel fill overlying granodiorite bedrock. More detailed descriptions of individual strata are presented below.



5.1 Asphalt and Concrete

Both boreholes were drilled through the paved portion of Highway 71, through the concrete bridge approach slabs. The pavement structure in both locations consisted of an asphalt layer of 75 mm thickness, overlying a 200 mm thick concrete slab.

5.2 Sand and Gravel Fill

Sand and gravel fill was encountered below the concrete in Borehole 22-02. The sand and gravel also contained trace silt and occasional large gravel.

The sand and gravel fill was 1.9 m thick, with an underside depth of 2.2 m below ground surface (Elevation 329.8 m).

SPT 'N' values in the sand and gravel fill ranged from 18 to 30 blows per 0.3 m penetration, indicating a compact to dense relative density.

The measured moisture contents ranged from 8 to 13%.

The results of a grain size analysis conducted on a selected sample of the sand and gravel fill is provided on the Record of Borehole sheets in Appendix B and plotted on Figure C1 in Appendix C. The results are summarized as follows:

Soil Particle	Percentage (%)
Gravel	53
Sand	40
Silt & Clay	7

5.3 Sand Fill

Sand fill was encountered below the concrete slab in Borehole 22-01 and below the sand and gravel fill in Borehole 22-02. The sand fill contained trace to some gravel, trace to some silt, trace clay and occasional large gravel and cobbles.

The gravel fill ranged in thickness from 3.6 to 4.2 m, with the underside depth ranging from 4.5 to 5.8 m below ground surface (Elevation 327.6 m to 326.2 m).

SPT 'N' values in the sand fill ranged from 4 to 47 blows per 0.3 m penetration, indicating a loose to dense relative density (typically compact to dense).

The measured moisture contents ranged from 5 to 27%.



The results of grain size analyses conducted on selected samples of the sand fill are provided on the Record of Borehole sheets in Appendix B and plotted on Figure C2 in Appendix C. The results are summarized as follows:

Soil Particle	Percentage (%)
Gravel	5 to 17
Sand	68 to 84
Silt & Clay	11 to 15

5.4 Bedrock

The overburden soils described above are underlain by bedrock. The bedrock is described as granodiorite, is white and grey in colour with pink intrusions, and is slightly weathered to fresh. Bedrock was proven by coring 2.7 to 3.3 m at both borehole locations.

Table 5.1 summarizes the depths and elevations to the top of the bedrock at the borehole locations. Photographs of the rock cores are included in Appendix C.

Table 5.1 - Depths and Elevations of Top of Bedrock

Borehole	Top of Bedrock	
	Depth Below Existing Grade Level (m)	Elevation (m)
22-01	4.5	327.6
22-02	5.8	326.2

Total Core Recovery (TCR) in the bedrock was 100% throughout all runs, and Solid Core Recovery (SCR) ranged between 93% to 100%. The Rock Quality Designation (RQD) determined from the recovered cores ranged between 75% and 100%, which indicates good to excellent rock quality. The Fracture Index (FI) of the rock, expressed as fractures per 0.3 m of core ranged from 0 to 5.

Average unconfined compressive strengths (UCS) of the rock ranged between 175 and 204 MPa. These estimated rock strength values are interpreted from point load tests that were conducted on rock cores recovered from the boreholes. Unconfined Compression Strength (UCS) tests were also conducted on 1 specimen from each of the bedrock core samples. The UCS test results

ranged from 112 to 123 MPa. Based on the test results, the bedrock is typically very strong. The UCS and point load test results are presented in Appendix C.

5.5 Groundwater Conditions

Groundwater conditions were observed during drilling operations and groundwater levels were measured in the open boreholes upon completion of drilling. The measured groundwater levels are summarized in Table 5.2 below.

Table 5.2: Groundwater Measurements

Borehole	Date	Water Level (m)		Remark
		Depth	Elevation	
22-01	November 11, 2022	3.3	328.8	Open Borehole
22-02	November 9, 2022	3.4	328.6	Open Borehole

The groundwater level is likely to reflect the local surface water level at the dam spillway structure. Based on existing survey drawings provided by MTO, in August 2012 the local surface water level was measured at Elevation 328.7 m upstream and Elevation 322.8 m downstream of the dam.

It should also be noted that groundwater levels are short term observations and 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 and spring snow melts. The water level will also vary due to dam control near Nestor Falls.

6. CORROSIVITY AND SULPHATE TEST RESULTS

Samples of the sand fill from Boreholes 22-01 and 22-02, and a sample of surface water collected from upstream of the dam were submitted for analytical testing of corrosivity parameters and sulphate. The laboratory certificates of analysis are presented in Appendix C. The results of the analytical tests are summarized below in Table 6.1.

Table 6.1: Analytical Test Results

Parameter	Units (Soil)	Units (Water)	Sample ID, Depth, Type and Test Results		
			22-01 CORR	22-02 CORR	Nestor Falls
			SS2 1.5 – 2.1 m deep	SS6 4.6 – 5.2 m deep	Surface Water
			Sand Fill, above Groundwater	Sand Fill, below Groundwater	Water
Redox Potential	mV	mV	131	213	224
Sulphide	%	µg/L	<0.04	<0.04	<6
pH	-	-	10.1	11.1	7.31
Chloride	µg/g	mg/L	84	1400	3
Sulphate	µg/g	mg/L	52	100	1.3
Conductivity	µS/cm	µS/cm	294	2120	80
Resistivity	ohm-cm	ohm-cm	3400	471	12,500*

*Calculated based on conductivity result

7. MISCELLANEOUS

Eastern Ontario Diamond Drilling Ltd. of Hawkesbury, Ontario supplied a rubber truck-mounted CME75 drill rig to conduct the drilling, sampling and in-situ testing operations for the boreholes. Traffic control services conforming to Ontario Book 7 were provided by ML Judson Trucking Ltd., of Emo, Ontario.

Geotechnical laboratory testing was carried out in Thurber's geotechnical laboratory. Analytical testing was carried out by SGS Canada Inc.

The field investigation was supervised on a full-time basis by Mr. George Azzopardi of Thurber. Overall supervision of the field program was provided by Ms. Madisan Chiarotto, E.I.T. and Mr. Mark Farrant, P.Eng. of Thurber.

Interpretation of the field data and preparation of this report was carried out by Ms. Madisan Chiarotto, E.I.T. and Mr. Mark Farrant, P.Eng. The report was reviewed by Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.



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PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS

8. GENERAL

This report provides an interpretation of the factual data from Part 1 of the report and presents geotechnical recommendations to support the preliminary design for rehabilitation or replacement of the Nestor Falls Bridge located on Highway 71 in Nestor Falls, Ontario. The discussion and recommendations presented in this report are based on the information provided by Hatch and on the factual data obtained during the course of the investigation.

This foundation investigation and design report with the interpretation and recommendations are intended for the use of the Ministry of Transportation and Hatch, 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.

The existing bridge on Highway 71 is aligned in a general north-south direction across Nestor Falls, which flows from Kakabikitchiwan Lake on the east side of Highway 71 to Lake of the Woods (Sabaskong Bay) on the west side. The bridge is located adjacent to the Nestor Falls dam spillway structure, which is on the west of Highway 71 (see Photos D3 and D4 in Appendix D).

Based on available archival drawings and reports provided by Hatch and MTO, the existing bridge is an approximately 15.7 m long, single span concrete bridge, built in approximately 1939 and rehabilitated in approximately 2009. The bridge is supported on concrete spread footings, which based on the 1939 design drawing are likely founded on bedrock at a depth of approximately 5 m (Elevation 327 to 326 m). The 1939 drawing indicates that rock fill may be present in the approach



embankments behind the abutments. The ground surface elevation on Highway 71 at the bridge is approximately 332 m. The lake level was measured in August 2012 at Elevation 328.7 m upstream of the dam and Elevation 322.8 m downstream of the dam.

9. BRIDGE DESIGN

9.1 Bridge Alternatives

This section presents discussions on various options for rehabilitation or replacement of the existing bridge.

Several options considered for this site are listed below:

- Minor rehabilitation including localized concrete repairs, waterproofing, paving and installation of deck drains. No foundation modifications or significant excavations are anticipated for this option. There are no foundation considerations associated with this option.
- Major rehabilitation including replacement of the existing bridge deck, bearings, and approach slabs. This option would utilize the existing bridge foundations and would require temporary excavations to replace the bridge bearings and approach slabs.
- Complete replacement with a new widened single-span bridge, including construction of new foundations. Foundation options considered include concrete spread footings on bedrock or steel H-Piles driven to or socketed into bedrock.

Preliminary draft General Arrangement (GA) drawings were provided by Hatch for each of the above bridge options. Based on discussions with Hatch, we understand that MTO prefers the minor rehabilitation option, with a future full replacement of the bridge to be considered. Preliminary foundation recommendations for each of the bridge options are presented below.

9.2 Summary of Subsurface Conditions

In general, the subsurface stratigraphy encountered in the boreholes consisted of pavement structure and bridge approach slab (asphalt and concrete), underlain by typically compact to dense sand and gravel fill and sand fill, overlying very strong granodiorite bedrock at 4.5 to 5.8 m depth.

The groundwater level in the open boreholes ranged from approximate Elevation 328.6 to 328.8 m, which are similar to the local upstream surface water level of 328.7 m, recorded in August 2012.



9.3 Minor Bridge Rehabilitation

The minor rehabilitation option does not include modification of the existing bridge foundations and no significant excavations are anticipated to conduct the rehabilitation work. In the event that temporary excavations are required, foundation recommendations are provided in Sections 10 and 11 below.

9.4 Major Bridge Rehabilitation

The major rehabilitation option includes replacement of the existing bridge deck with a new deck supported on the existing foundations. This option will also require temporary excavations to replace the bridge bearings and bridge approach slabs. The preliminary GA drawing shows that the new bridge deck will be approximately 15.7 m long and will retain the same width and grade level as the existing structure. Foundation considerations for this option include the capacity of the existing foundations to support the heavier bridge deck, temporary excavations through the roadway for replacement of the bearings and approach slabs, and temporary roadway protection systems (TPS) to allow for staged construction during roadway excavation. Temporary excavation and roadway protection are discussed in Sections 10 and 11 below.

9.4.1 Bearing Capacity of Existing Footings on Bedrock

The boreholes advanced at this site encountered bedrock at approximately 4.5 m deep (Elevation 327.6 m) at the south abutment (BH 22-01) and 5.8 m deep (Elevation 326.2 m) at the north abutment (BH 22-02). This is relatively consistent with the bedrock level shown on the 1939 pre-construction bridge drawings. Drilling through the spread footings would be required to confirm the founding conditions; however the borehole information from approximately 4 m behind the abutments suggests that the spread footings are likely founded on bedrock as intended on the original 1939 design.

Based on the presence of very strong igneous bedrock (granodiorite) at the founding level, the factored geotechnical resistance of the bedrock at ULS is estimated to be 5,000 kPa. The SLS condition does not govern for footings bearing on bedrock. The assessed geotechnical resistance is based on the footings being subjected to vertical concentric loading. If there are eccentric or inclined loads applied by the replacement superstructure, the resistance values used in design must be reduced in accordance with the CHBDC 2019 Clause 6.10.5.3.

The lateral resistance developed along the base of concrete footings founded on sound bedrock may be computed using a factored friction coefficient of 0.7 at ULS.



We understand that the new bridge deck will apply a bearing pressure of approximately 440 kPa on the existing footings. Although this is understood to be an approximately 20% increase in load, it does not exceed the geotechnical capacity of the existing spread footings on bedrock. Provided that the existing foundations and footing concrete are structurally sound, with any required structural repairs completed (if needed), then the existing spread footings founded on bedrock are expected to be suitable to support the major rehabilitation option without augmentation of the existing foundations.

9.5 Full Bridge Replacement

The full bridge replacement option will also include construction of new abutment foundations. The preliminary GA drawing shows that the replacement bridge will be approximately 15.7 m long and 14.7 m wide, which is approximately 2.7 m wider than the existing bridge. To accommodate the proposed widening to the east side of the highway, the centreline of the new bridge will be shifted approximately 0.9 m to the east. No grade raise is proposed for the new bridge, however some embankment widening to the east is anticipated to accommodate the wider bridge platform. The GA drawing shows the proposed new abutments to be located behind the existing abutments, with the abutments supported on H-pile foundations socketed into bedrock. New RSS walls are proposed to retain the abutment fill. Based on the presence of relatively shallow bedrock at the site, consideration may be given to founding the new bridge on concrete spread footings supported on bedrock, or on steel H-Piles driven to or socketed into bedrock as shown on the GA drawing. Preliminary recommendations for both foundation options are presented below.

Should the replacement bridge option be selected, additional boreholes are recommended to be advanced during the detailed design stage in order to collect sufficient subsurface information for design of the bridge foundations, RSS walls, and embankment widening (refer to Section 17 below).

9.5.1 Concrete Spread Footings Founded on Bedrock

Construction of new concrete spread footings founded on bedrock at approximate Elevation 326 to 327 m is a feasible option to support a replacement bridge at this site.

Based on the presence of very strong igneous bedrock (granodiorite) at the founding level, spread footings founded on the bedrock may be designed using a Factored Geotechnical Resistance at ULS of 5,000 kPa. The SLS condition will not govern design for footings bearing on bedrock. Considering the high available resistance, the footing design is expected to be governed by minimum foundation width considerations and not by the geotechnical resistance. Additional foundation investigation is recommended for detailed design of this option, as discussed in



Section 17, including sufficient boreholes to further define the elevation of the bedrock surface at the proposed footing locations.

The recommended geotechnical resistance is based on a footing subjected to vertical concentric loading. Where eccentric or inclined loads are applied, the resistance values used in design must be reduced in accordance with the CHBDC 2019 Clause 6.10.5.3.

The lateral resistance developed along the base of concrete footings founded on sound bedrock may be computed using a factored friction coefficient of 0.7 at ULS.

Construction of footings on bedrock would require large temporary excavations carried out through cohesionless fill and below the groundwater and surface water level, which will be difficult at this site. Therefore, although geotechnical suitable, the footing option is not recommended from a constructability perspective. Hence, further recommendations for this option were not developed.

9.5.2 Steel H-Piles Driven to or Socketed into Bedrock

As shown on the preliminary GA drawing, supporting the new abutments on steel H-piles, driven to or socketed into bedrock is also a feasible foundation option for the replacement bridge. For this option, the new abutments could be located behind the existing abutments, with the existing abutments remaining in place or cut off just above the water level. This option would significantly reduce excavation below water.

As the depth to bedrock ranges from 4.5 to 5.8 m, driven piles may not be feasible to achieve adequate pile lengths for lateral stability purposes. Furthermore, there may be rock fill present in the approach embankments, which would make pile driving difficult. Therefore, it may be necessary to install drilled-in piles installed in sockets cored into the bedrock to satisfy lateral pile stability requirements. Installation of the piles would then involve either excavation or drilling through the sand to sand and gravel fill and possible rock fill, then coring an adequate depth into bedrock to form a rock socket, inserting the pile, and grouting the annular space in the socket with concrete. A socket diameter approximately 200 mm larger than the largest dimension (corner to corner) of the pile will be required.

The factored geotechnical resistances and the estimated tip elevations recommended for HP 310x110 piles driven to or socketed into bedrock are presented below in Table 9.1.

Table 9.1 – Recommended Axial Geotechnical Resistances for Steel HP 310x110 Piles

Foundation Element	Approximate Pile Tip Depth/Elevation Below Existing Ground (m)	Factored ULS Geotechnical Resistance Per Pile (kN)	SLS Resistance (kN)
South Abutment (Borehole 22-01)	4.5 / 327.6	2,800	Does not govern
North Abutment (Borehole 22-02)	5.8 / 326.2	2,800	Does not govern

The actual pile tip elevations may vary during installation if a sloping bedrock surface exists. Additional foundation investigation is recommended to confirm the bedrock surface elevation at both abutment locations for detailed design of this option, as discussed in Section 17.

The axial resistances based on the bedrock strength are expected to exceed the factored structural capacity of the pile. Accordingly, the structural capacity of the HP 310x110 piles (2000 kN per pile) will govern the design.

The SLS condition will not govern the design of piles founded on bedrock.

The structural resistance of the pile must be checked by the structural designer.

Pile installation must be in accordance with OPSS.PROV 903.

9.5.3 Pile Lateral Resistance in Soil

The geotechnical lateral resistance acting on an H-pile in cohesionless soil may be calculated using a value for the coefficient of horizontal subgrade reaction (k_s) and ultimate lateral resistance (p_{ult}) as follows:

$$k_s = n_h z / D \quad (\text{kN/m}^3)$$

$$p_{ult} = 3 \gamma' z K_p \quad (\text{kPa})$$

Where z = depth of embedment of pile (m)

D = pile width or diameter (m)

n_h = coefficient related to soil relative density (kN/m^3)

γ' = effective unit weight (kN/m^3)

K_p = passive earth pressure coefficient



For analysis of the interaction between a pile and the surrounding soil, the above equations and parameters recommended in Table 9.2 below, may be used. The lateral pressures obtained from the analysis should not exceed the ultimate lateral resistance.

Table 9.2 – Soil Parameters for Lateral Pile Resistance

Abutment	Elevation	n_h (kN/m ³)	K_p	Unit Weight* (kN/m ³)	Soil Conditions
South (22-01)	329.5 to 328.8	5,500	3.3	21	Compact to dense sand fill
	328.8 to 327.6	3,500	3.3	11*	Compact sand fill
North (22-02)	329.4 to 328.6	5,500	3.3	21	Compact sand and gravel to sand fill
	328.6 to 326.2	3,500	3.3	11*	Compact to dense sand fill

*Buoyant unit weight below groundwater level

The lateral resistance within the depth of frost (2.3 m) should not be included.

The spring constant, K_s , for analysis may be obtained by the expression, $K_s = k_s L D$ (kN/m), where k_s is the coefficient of horizontal subgrade reaction (kN/m³), D is the pile width (m), and L is the length (m) of the pile segment or element used in the analysis. The ultimate lateral resistance, P_{ult} , may be obtained from the expression, $P_{ult} = p_{ult} L D$. This represents the ultimate load at which the pile fails and will not support any additional load at greater displacements.

The modulus of subgrade reaction and ultimate lateral resistance may have to be reduced, based on the pile spacing. The reduction factors to be used for a pile group oriented perpendicular or parallel to the direction of loading are provided in Section C.6.11.3.4 of CHBDC 2019.

Horizontal loads may be resisted by means of battered piles if load requirements exceed the available lateral pile resistances.

9.5.4 Pile Lateral Resistance in Rock Socket

The ultimate passive resistance that can be mobilized by the embedded portion of a pile socketed in the bedrock may be assumed to be a constant with depth and is given by:



$$P_p = 6 C D L$$

Where $C = 5,000 \text{ kPa}$ (undrained shear strength of rock mass)

$D =$ Socket diameter (m)

$L =$ Depth of socket in rock (m)

The depth of the pile socket should be calculated by the design engineer but should be socketed into bedrock a minimum of 1.5 m. Deeper sockets may be required to meet lateral resistance requirements.

9.5.5 Frost Cover

The depth of frost penetration at this site is approximately 2.3 m based on OPSD 3090.100. Typically, the base of all footings and pile caps, if employed, must be provided with a minimum of 2.3 m of earth cover as protection against frost action. With shallow bedrock at this site, the minimum earth cover may not be achievable for pile caps, and therefore insulation could be considered to reduce the depth of earth cover. Concrete spread footings founded on bedrock do not need to be placed below the depth of frost.

10. TEMPORARY EXCAVATIONS

Temporary excavations will be required for the major rehabilitation option to repair or replace the bridge bearings and to replace the bridge approach slabs. Temporary excavations will also be required for the full bridge replacement option for construction of new foundations, RSS walls, and other associated works. In both cases, it is recommended that all excavations be kept above the groundwater and upstream surface water elevation of approximately 328.8 m. Please note however, should a replacement bridge founded on spread footings on bedrock be selected, then large excavations below water will be required. Additional investigation and recommendations for groundwater control will be required for detailed design if the spread footing option is selected (refer to Section 17).

All excavations should be carried out in accordance with the Occupational Health and Safety Act (OHSA). For the purposes of the OHSA, the sand and gravel fill and sand fill at this site are both classified as a Type 3 soil above the water table. Below the water table (i.e., if the groundwater flow is not controlled), the fill soils would be classified as Type 4 soils.

Support of excavations such as through the use of temporary roadway protection systems (TPS) will be required to maintain traffic during construction. Options for roadway protection include interlocking sheet piles or a drilled-in soldier pile and lagging system. Given the relatively shallow

depth to bedrock (4.5 to 5.8 m) and the possible presence of rock fill in the approach embankments, it may not be possible to drive sheet piles to adequate depths. Therefore, a drilled-in soldier pile and lagging system is likely the best option.

Temporary roadway protection systems are discussed further in Section 11.

11. TEMPORARY PROTECTION SYSTEM

A temporary roadway protection system for support of excavations near the abutments should be designed and implemented in accordance with OPSS.PROV 539 and designed for Performance Level 2.

Due to the presence of relatively shallow bedrock and possible rock fill in the embankment, a drilled-in soldier pile and lagging system is likely the best option for roadway protection instead of interlocking sheet piles. Depending on the required depth of penetration, the soldier piles may need to be socketed into bedrock.

The soil parameters in Table 11.1 may apply for the design of the temporary roadway protection system with horizontal backfill.

Table 11.1 – Soil Parameters for Temporary Protection System Design

Soil Parameter	Existing Sand and Gravel to Sand Fill
ϕ (angle of internal friction)	32°
γ (total unit weight)	21 kN/m ³
γ_w (submerged unit weight)	11 kN/m ³
K_a	0.31
K_p	3.3

The design of the temporary protection system is the responsibility of the Contractor. The actual pressure distribution acting on the protection/shoring system is a function of the construction sequence and the relative flexibility of the wall, and these factors have to be considered when designing the shoring system. All protection systems should be designed by a Professional Engineer experienced in such designs, who will determine an appropriate support system. The designer of the roadway protection system should consider the presence of a possible sloping

bedrock surface at this site, and check whether the depth of soldier piles, if utilized, is sufficient to provide base fixity.

12. ABUTMENT BACKFILL AND LATERAL EARTH PRESSURES

Backfill behind the rehabilitated or new abutments should consist of free-draining, non-frost susceptible granular materials such as Granular A or B Type II or Type III conforming to the requirements of OPSS.PROV 1010. Reference should be made to the backfill arrangements stipulated in OPSD 3101.150, as appropriate. Compaction equipment to be used adjacent to the walls should be restricted in accordance with OPSS.PROV 501.

Earth pressures acting on the structures may be assumed to be distributed triangularly and to be governed by the characteristics of the abutment backfill. For a fully drained condition, the pressures should be computed in accordance with the CHBDC 2019, but are generally given by the expression:

$$p_h = K (\gamma h + q)$$

where	p_h	=	horizontal pressure on the wall at depth h (kPa)
	K	=	earth pressure coefficient (see table below)
	γ	=	bulk unit weight of retained soil (see table below)
	h	=	depth below top of fill where pressure is computed (m)
	q	=	value of any surcharge (kPa)

Earth pressure coefficients for backfill to the abutment walls are dependent on the material used as backfill. Recommended unfactored values are shown in Table 12.1 below.

Table 12.1 – Lateral Earth Pressure Coefficients (K)

Loading Condition	OPSS Granular A or Granular B Type II $\phi = 35^\circ; \gamma = 22.8 \text{ kN/m}^3$		OPSS Granular B Type I or Type III $\phi = 32^\circ; \gamma = 21.2 \text{ kN/m}^3$	
	Horizontal Backfill	Sloping Backfill (2H:1V)	Horizontal Backfill	Sloping Backfill (2H:1V)
Active K_A (Unrestrained Wall)	0.27	0.38	0.31	0.46
At-rest K_0 (Restrained Wall)	0.43	-	0.47	-
Passive K_P	3.7	-	3.3	-

The use of a material with a high friction angle and low active pressure coefficient (e.g. Granular A, Granular B Type II) is preferred as it results in lower earth pressures acting on the wall.

In accordance with Clause 6.12.3 of the CHBDC 2019, a compaction surcharge should be added. The magnitude of the surcharge should be 12 kPa at the top of fill and decrease to 0 kPa at a depth of 1.7 m for Granular B Type I, or at a depth of 2.0 m for Granular A or B Type II.

13. SEISMIC CONSIDERATIONS

In accordance with the CHBDC 2019, the selection of the seismic site classification is based on the soil conditions encountered in the upper 30 m of the stratigraphy. Based on the presence of generally compact to dense granular fill soils overlying relatively shallow bedrock, the site is considered to be classified as Seismic Site Class C in accordance with Table 4.1, Clause 4.4.3.2 of the CHBDC. The peak ground acceleration, PGA, for a 2,475-year return period seismic event (2% probability of being exceeded in 50 years) at this site is 0.051 g as per the National Building Code of Canada (NBCC 2020).

The coefficients of horizontal earth pressures for seismic loading on walls assuming a level backfill, a Site Class C, and a reference PGA of 0.051 are presented in Table 13.1 below. The vertical acceleration coefficient k_v has been ignored ($k_v = 0$).

Table 13.1 – Seismic Earth Pressure Parameters

Loading Condition	Horizontal Acceleration Coefficient, k_h	Seismic Earth Pressure Coefficients (K_{AE})	
		OPSS Granular A or Granular B Type II $\phi = 35^\circ$, $\gamma = 22.8 \text{ kN/m}^3$	OPSS Granular B Type I $\phi = 32^\circ$, $\gamma = 21.2 \text{ kN/m}^3$
Active (Unrestrained Wall)	0.026	0.26	0.29
Active (Restrained Wall)	0.051	0.27	0.31

In view of the low potential for seismic activity in the area, liquefaction is not considered to be a concern at this site.

14. EMBANKMENT RESTORATION

Embankment restoration after completion of the bridge rehabilitation or replacement should be carried out in accordance with OPSS.PROV 206. The embankment reconstruction material should consist of imported Granular A, Granular B Type II or Type III, or Rock Fill material. The restored embankment below and beyond the approach walls should be reinstated at the existing



slope inclination, but no steeper than 2H:1V. For the replacement option, additional granular fill or rock fill should be placed as necessary to widen the approach embankments and transition to the existing highway embankment slopes.

In general, surface vegetation, peat, topsoil, organic deposits, disturbed material or otherwise loose/soft soils should be stripped within the embankment reconstruction footprints. Inspection and approval of the foundation surfaces by qualified geotechnical personnel should be conducted.

Disturbed or regraded earth slopes must be provided with erosion protection in accordance with OPSS.PROV 804.

15. CORROSION AND SULPHATE ATTACK POTENTIAL

The results of the corrosivity and sulphate content analytical tests conducted on the soil and surface water samples indicate the following conditions at the locations tested:

- The potential for corrosion on metal or concrete foundations from the shallow sand fill above the groundwater level is considered to be mild to moderate due to the low concentrations of sulphate and chloride in the sample tested from Borehole 22-01. The effect of road deicing salt should be considered while selecting the class of concrete.
- The sand fill below the groundwater and surface water level is considered to be corrosive to metal and concrete, based on the high concentration of chloride and the low resistivity in the sample tested from Borehole 22-02.
- The potential for sulphate attack on concrete from the surrounding soil or surface water is considered to be negligible due to the low sulphate concentration in the samples tested.
- Appropriate protection measures are recommended for metal or concrete structural elements; particularly those located below the groundwater and surface water level. The effect of road deicing salt should also be considered while selecting the corrosion protection measures.

16. CONSTRUCTION CONCERNS

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

- The temporary roadway protection system must be designed to take into account the presence of relatively shallow bedrock, and possible rock fill at the existing abutments or elsewhere beyond the boreholes from this investigation.

- Appropriate corrosion protection measures should be provided; particularly below the water level.
- The water level in the watercourse may fluctuate and be at a higher elevation at the time of construction than indicated in the report.

17. ADDITIONAL INVESTIGATION FOR DETAILED DESIGN

Should the replacement bridge option be selected, additional boreholes are recommended to be advanced during the detailed design stage in order to collect sufficient subsurface information for design. The foundation investigation should satisfy the requirements in the latest version of MTO's Guideline for Foundation Engineering Services. This may include additional boreholes at the new abutment locations, proposed RSS wall locations, widened embankment locations, and temporary roadway protection system locations to investigate the presence of rock fill beyond the boreholes from this report.

18. CLOSURE

Engineering analysis and preparation of the design report was carried out by Mr. Mark Farrant, P.Eng. The report was reviewed by Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

THURBER ENGINEERING LTD.



Mark Farrant, P.Eng.
Associate, Senior Geotechnical Engineer



Dr. P.K. Chatterji, P.Eng.
Designated MTO Principal Contact

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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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- 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.

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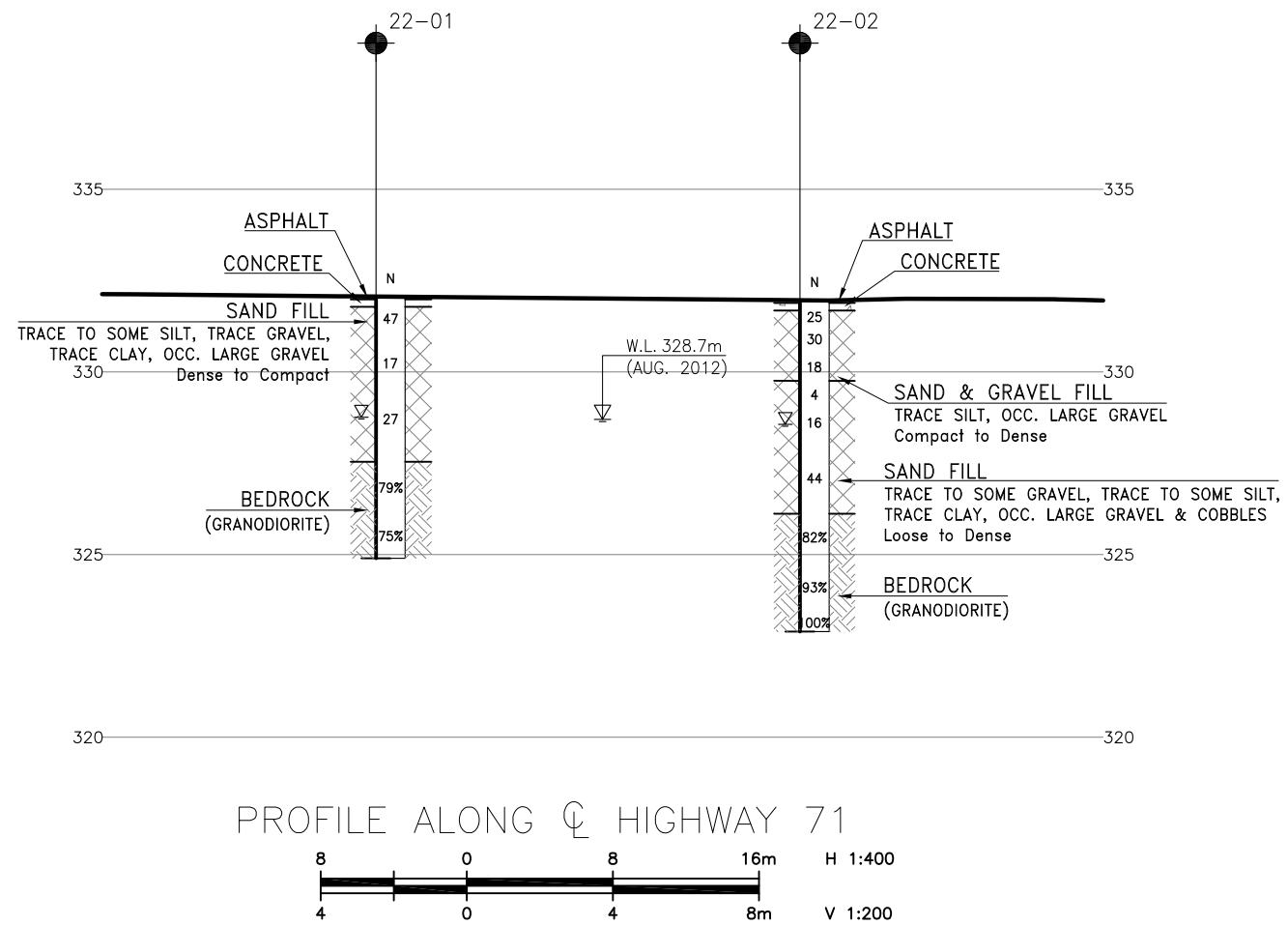
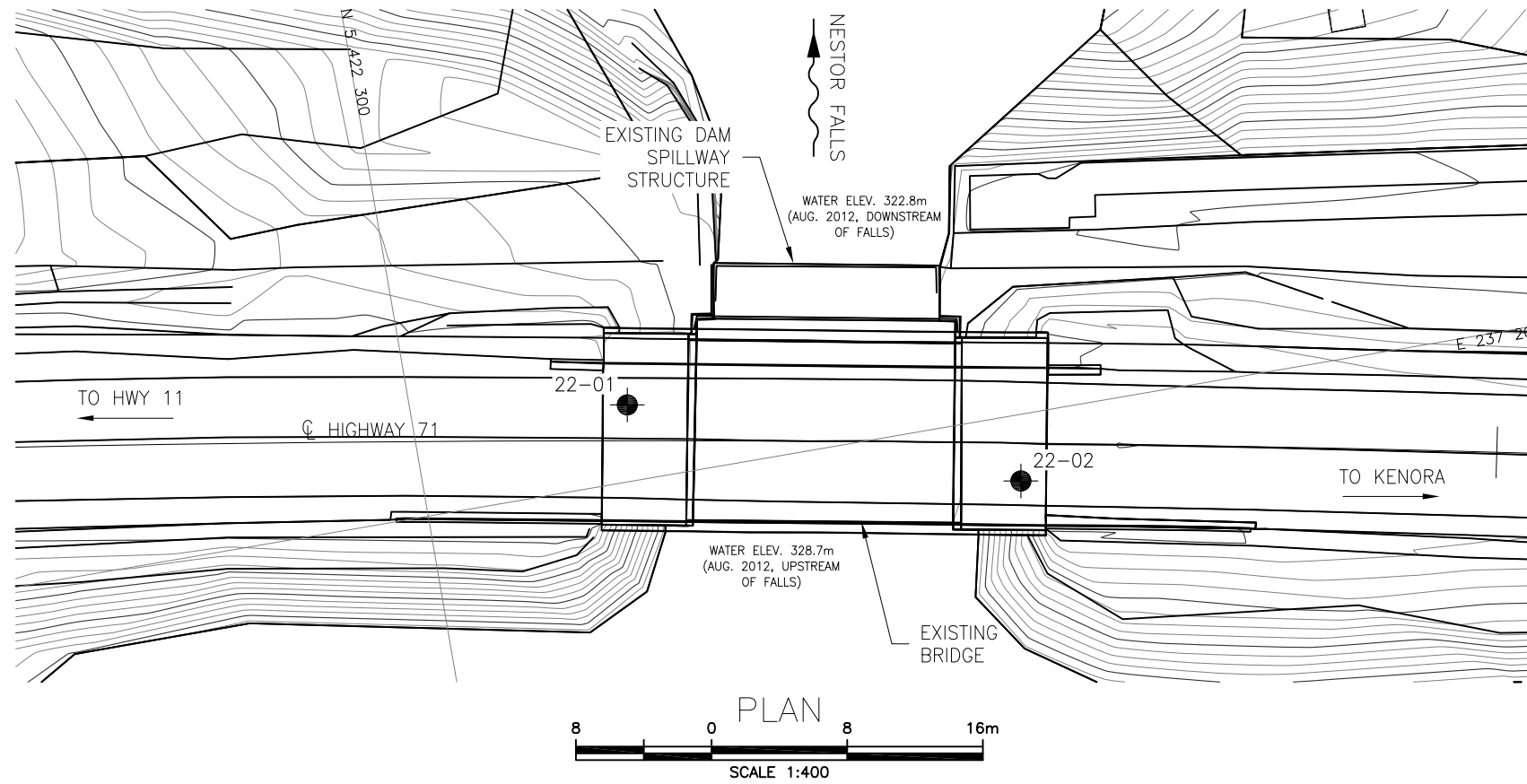
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Appendix A

Borehole Locations and Soil Strata Drawing



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

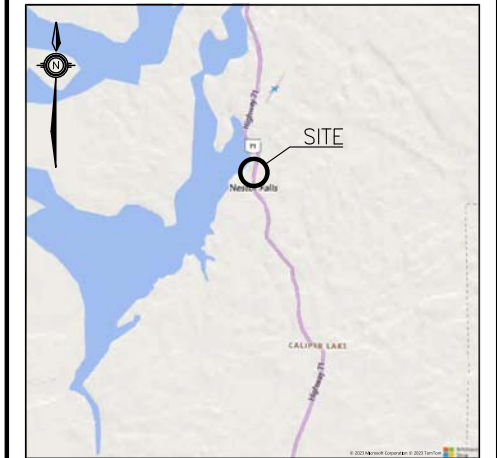


CONT No	
GWP No	6055-18-00

HIGHWAY 71
NESTOR FALLS BRIDGE






BOREHOLE LOCATIONS AND SOIL STRATA

HATCH



KEYPLAN

LEGEND

	Borehole
	Borehole and Cone
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
22-01	332.1	5 442 312.7	237 195.5
22-02	332.0	5 442 334.8	237 203.9

-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 16.

GEOCRES No. 52F-69

[illegible]



Appendix B

Record of Borehole Sheets

SYMBOLS, ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES

1. TEXTURAL CLASSIFICATION OF SOILS

CLASSIFICATION	PARTICLE SIZE	VISUAL IDENTIFICATION
Boulders	Greater than 200mm	same
Cobbles	75 to 200mm	same
Gravel	4.75 to 75mm	5 to 75mm
Sand	0.075 to 4.75mm	Not visible particles to 5mm
Silt	0.002 to 0.075mm	Non-plastic particles, not visible to the naked eye
Clay	Less than 0.002mm	Plastic particles, not visible to the naked eye

2. COARSE GRAIN SOIL DESCRIPTION (50% greater than 0.075mm)

TERMINOLOGY	PROPORTION
Trace or Occasional	Less than 10%
Some	10 to 20%
Adjective (e.g. silty or sandy)	20 to 35%
And (e.g. sand and gravel)	35 to 50%

3. TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

DESCRIPTIVE TERM	UNDRAINED SHEAR STRENGTH (kPa)	APPROXIMATE SPT ⁽¹⁾ 'N' VALUE
Very Soft	12 or less	Less than 2
Soft	12 to 25	2 to 4
Firm	25 to 50	4 to 8
Stiff	50 to 100	8 to 15
Very Stiff	100 to 200	15 to 30
Hard	Greater than 200	Greater than 30

NOTE: Hierarchy of Soil Strength Prediction

- 1) Laboratory Triaxial Testing
- 2) Field Insitu Vane Testing
- 3) Laboratory Vane Testing
- 4) SPT value
- 5) Pocket Penetrometer



4. TERMS DESCRIBING DENSITY (COHESIONLESS SOILS ONLY)

DESCRIPTIVE TERM	SPT "N" VALUE
Very Loose	Less than 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	Greater than 50

5. LEGEND FOR RECORDS OF BOREHOLES

SYMBOLS AND ABBREVIATIONS FOR SAMPLE TYPE	SS Split Spoon Sample	WS Wash Sample	AS Auger (Grab) Sample
	TW Thin Wall Shelby Tube Sample	TP Thin Wall Piston Sample	
	PH Sampler Advanced by Hydraulic Pressure	PM Sampler Advanced by Manual Pressure	
	WH Sampler Advanced by Self Static Weight	RC Rock Core	SC Soil Core

$$\text{Sensitivity} = \frac{\text{Undisturbed Shear Strength}}{\text{Remoulded Shear Strength}}$$


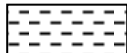



 Water Level
 Shear Strength Determination by Pocket Penetrometer

- (1) SPT 'N' Value Standard Penetration Test 'N' Value – refers to the number of blows from a 63.5kg hammer free falling a height of 0.76m to advance a standard 50 mm outside diameter split spoon sampler for 0.3 m depth into undisturbed ground.
- (2) DCPT Dynamic Cone Penetration Test – Continuous penetration of a 50 mm outside diameter, 60° conical steel point attached to "A" size rods driven by a 63.5 kg hammer free falling a height of 0.76 m. The resistance to cone penetration is the number of hammer blows required for each 0.3 m advance of the conical point into undisturbed ground.

UNIFIED SOILS CLASSIFICATION

MAJOR DIVISIONS		GROUP SYMBOL	TYPICAL DESCRIPTION
COARSE GRAINED SOILS	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	SILTS AND CLAYS W _L < 50%	ML	Inorganic silts and 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. (W _L < 30%).
		CI	Inorganic clays of medium plasticity, silty clays. (30% < W _L < 50%).
		OL	Organic silts and organic silty-clays of low plasticity.
	SILTS AND CLAYS W _L > 50%	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
		CH	Inorganic clays of high plasticity, fat clays.
		OH	Organic clays of medium to high plasticity, organic silts.
HIGHLY ORGANIC SOILS		Pt	Peat and other highly organic soils.
CLAY SHALE			
SANDSTONE			
SILTSTONE			
CLAYSTONE			
COAL			

EXPLANATION OF ROCK LOGGING TERMS

<u>ROCK WEATHERING CLASSIFICATION</u>		<u>SYMBOLS</u>	
Fresh (FR)	No visible signs of weathering.		
Fresh Jointed (FJ)	Weathering limited to the surface of major discontinuities.		CLAYSTONE
Slightly Weathered (SW)	Penetrative weathering developed on open discontinuity surfaces, but only slight weathering of rock material.		SILTSTONE
Moderately Weathered (MW)	Weathering extends throughout the rock mass, but the rock material is not friable.		SANDSTONE
Highly Weathered (HW)	Weathering extends throughout the rock mass and the rock is partly friable.		COAL
Completely Weathered (CW)	Rock is wholly decomposed and in a friable condition, but the rock texture and structure are preserved.		Bedrock (general)

<u>DISCONTINUITY SPACING</u>		<u>STRENGTH CLASSIFICATION</u>			
Bedding	Bedding Plane Spacing	Rock Strength	Approximate Uniaxial Compressive Strength		Field Estimation of Hardness*
			(MPa)	(psi)	
Very thickly bedded	Greater than 2m	Extremely Strong	Greater than 250	Greater than 36,000	Specimen can only be chipped with a geological hammer
Thickly bedded	0.6 to 2m				
Medium bedded	0.2 to 0.6m	Very Strong	100-250	15,000 to 36,000	Requires many blows of geological hammer to break
Thinly bedded	60mm to 0.2m	Strong	50-100	7,500 to 15,000	Requires more than one blow of geological hammer to break
Very thinly bedded	20 to 60mm				
Laminated	6 to 20mm	Medium Strong	25.0 to 50.0	3,500 to 7,500	Breaks under single blow of geological hammer.
Thinly Laminated	Less than 6mm				

<u>TERMS</u>		Extremely Weak (Rock)	0.25 to 1.0	35 to 150	Indented by thumbnail
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 0.1m in length or larger as a percentage of total core run length.				
Uniaxial Compressive Strength (UCS)	Axial stress required to break the specimen				
Fracture Index: (FI)	Frequency of natural fractures per 0.3m of core run.				

RECORD OF BOREHOLE No 22-01

1 OF 1

METRIC

GWP# 6055-18-00 LOCATION Nestor Falls Bridge N 5 442 312.7 E 237 195.5 ORIGINATED BY GA
DIST Kenora HWY 71 BOREHOLE TYPE Wash Boring/NQ Coring COMPILED BY MC
DATUM Geodetic DATE 2022.11.11 - 2022.11.11 LATITUDE 49.115242 LONGITUDE -93.926151 CHECKED BY MEF

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				WATER CONTENT (%)				
332.1	GROUND SURFACE					▽									GR SA SI CL	
0.0	ASPHALT: (75mm)															
0.1																
331.8	CONCRETE: (200mm)															
0.3																
	SAND, trace to some silt, trace gravel, trace clay, occasional large gravel Dense to Compact Brown Wet (FILL)		1	SS	47											5 84 11 (SI+CL)
			2	SS	17											
			3	SS	27											8 79 12 1
327.6																
4.5	BEDROCK (GRANODIORITE) slightly weathered to fresh, very strong, white/grey with pink intrusions															
	Sub-horizontal fractures at 4.8m and 5.4m		1	RUN			327								RUN #1 TCR=100% SCR=93% RQD=79% UCS=196MPa (Avg Point Load) UCS=112MPa	
	Sub-horizontal fractures at 6.0m															
			2	RUN			326								RUN #2 TCR=100% SCR=100% RQD=75% UCS=180MPa (Avg Point Load)	
324.9																
7.2	END OF BOREHOLE AT 7.2m. BOREHOLE OPEN AND WATER LEVEL OBSERVED AT 3.3m UPON COMPLETION BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG TO 0.5m, THEN CONCRETE TO 0.2m, THEN ASPHALT SURFACE.						325									

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

RECORD OF BOREHOLE No 22-02

1 OF 2

METRIC

GWP# 6055-18-00 LOCATION Nestor Falls Bridge N 5 442 334.8 E 237 203.9 ORIGINATED BY GA
DIST Kenora HWY 71 BOREHOLE TYPE Wash Boring/NQ Coring COMPILED BY MC
DATUM Geodetic DATE 2022.09.11 - 2022.09.11 LATITUDE 49.115442 LONGITUDE -93.926041 CHECKED BY MEF

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											
332.0	GROUND SURFACE							20	40	60	80	100							
0.0	ASPHALT (75mm)																		
0.1																			
331.7	CONCRETE (200mm)																		
0.3																			
	SAND and GRAVEL, trace silt, occasional large gravel Compact to Dense Brown Wet (FILL)		1	SS	25														
			2	SS	30														
			3	SS	18														
329.8																			
2.2	SAND, trace to some gravel, trace to some silt, trace clay, occasional large gravel and cobbles Loose to Dense Brown Wet (FILL)		4	SS	4														
			5	SS	16														
			6	SS	44														
326.2																			
5.8	BEDROCK (GRANODIORITE) slightly weathered to fresh, very strong, white/grey with pink intrusions		1	RUN															
			2	RUN															
			3	RUN															
322.9																			
9.1	END OF BOREHOLE AT 9.1m BOREHOLE OPEN AND WATER LEVEL OBSERVED AT 3.4m UPON COMPLETION BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG TO 0.5m,																		

Continued Next Page

+³, ×³: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

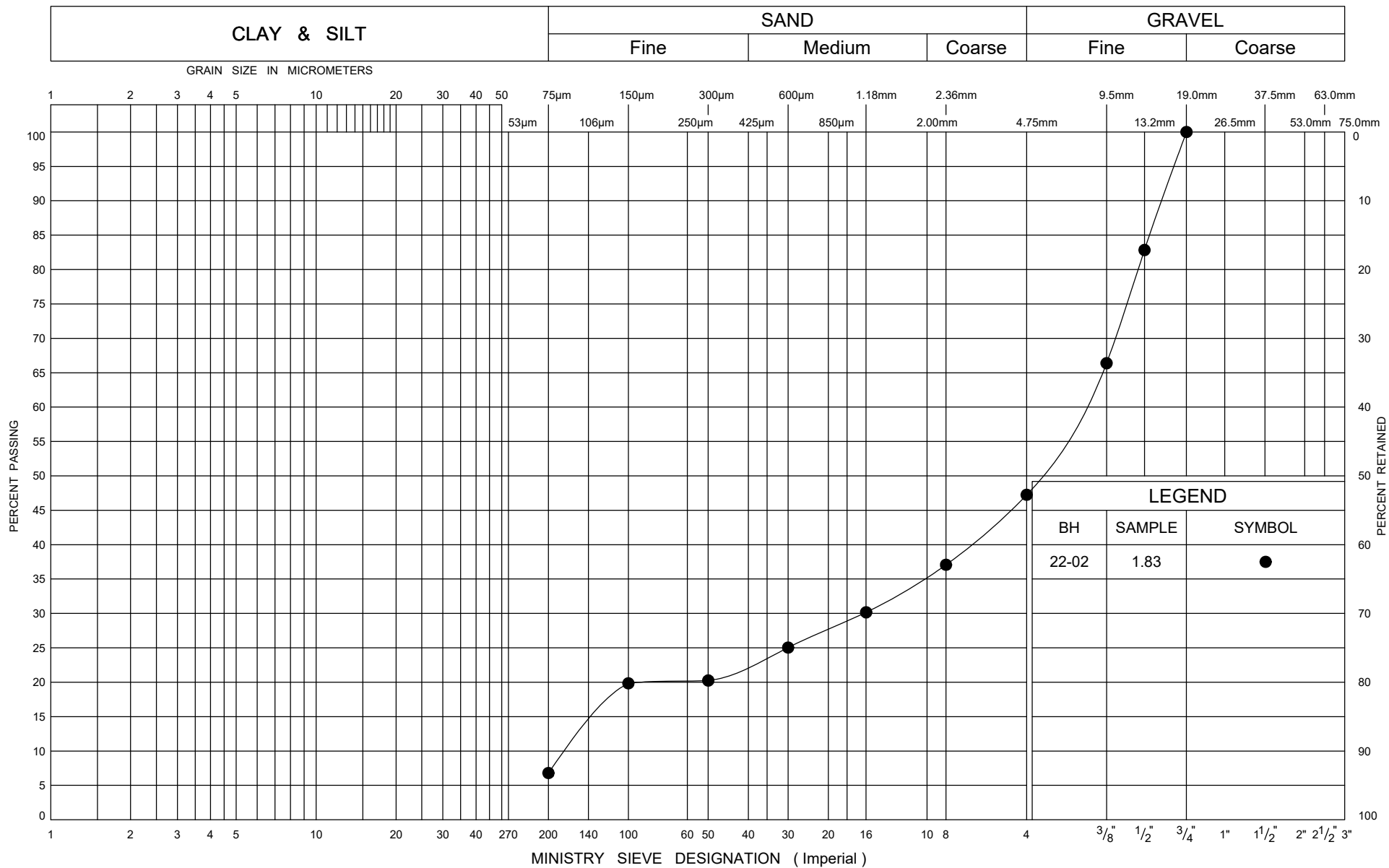
METRIC

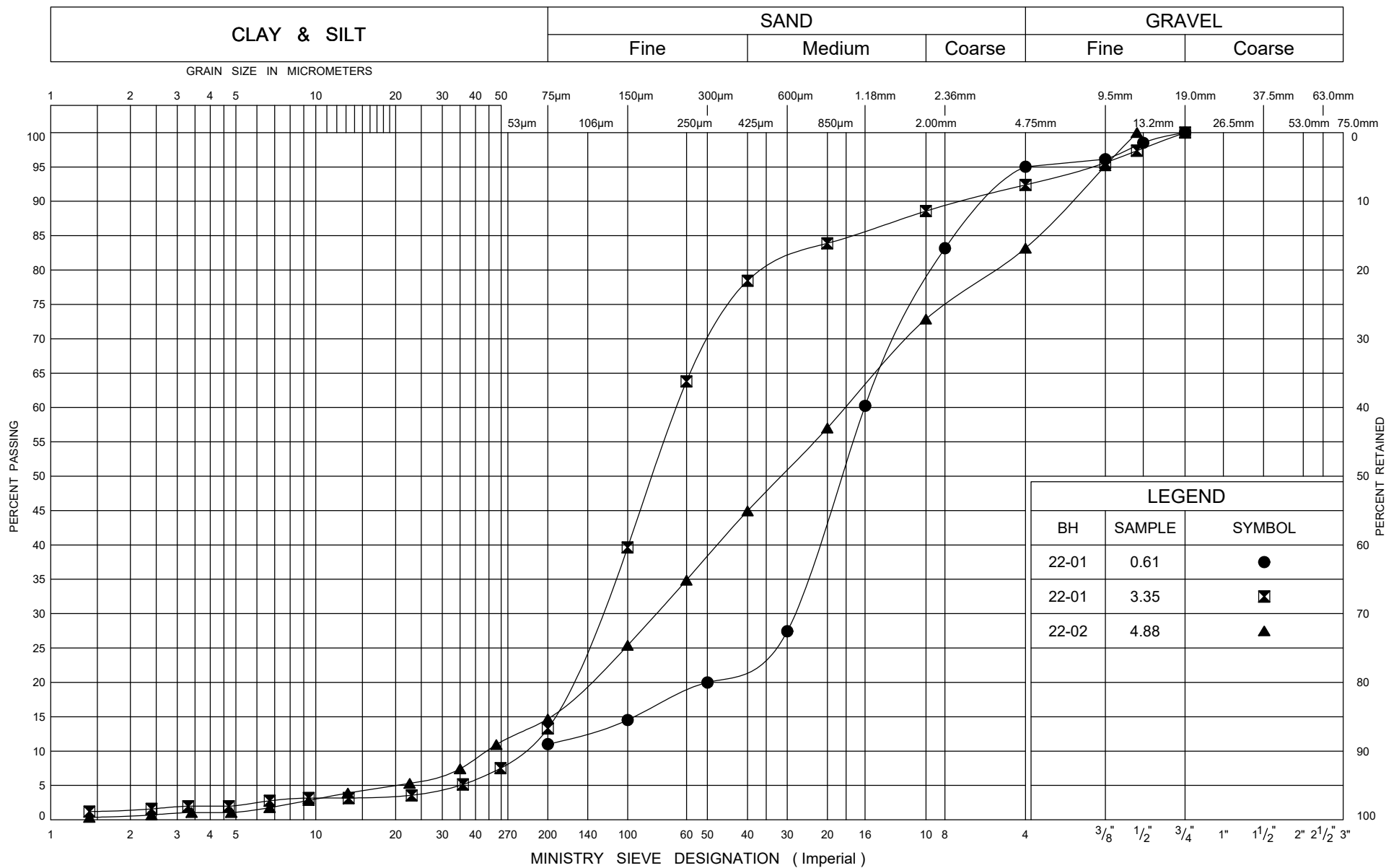
[illegible]



Appendix C

Laboratory Test Results





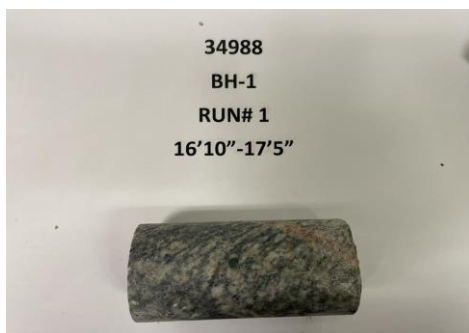
UNCONFINED COMPRESSION TEST REPORT

ASTM D7012-14

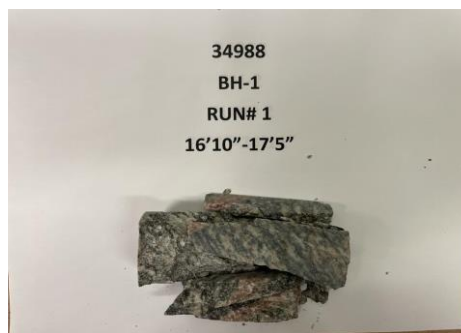
CLIENT:	HATCH	FILE NUMBER:	34988
PROJECT NAME:	Nestor Falls Bridge	REPORT DATE:	21-Jun-23
BOREHOLE No.:	22-01	TEST DATE:	13-Dec-22
SAMPLE No.:	Run 1		
SAMPLE DEPTH:	5.1 to 5.3 m		
DESCRIPTION:	Granodiorite		

Avg. Height (cm):	9.9	Weight (g):	472.2
Avg. Diameter (cm):	4.7	Wet Density (kg/m ³):	2,703
H. to Dia. Ratio**:	2.1:1	Dry Density (kg/m ³):	2,703
Cross Sectional Area (cm ²):	17.65	Moisture Content* (%):	N/A
Sample Volume (cm ³):	174.70		

ORIGINAL SPECIMEN



FRACTURED SPECIMEN



AVG. RATE OF STRAIN TO FAILURE:	0.250 MPa/s
MAXIMUM COMPRESSIVE LOAD:	197.7 kN
UNCONFINED COMPRESSIVE STRENGTH:	112.0 MPa

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

TEST DONE BY: AK
 REVIEWED BY:

34988 UCS BH 1 Run 1

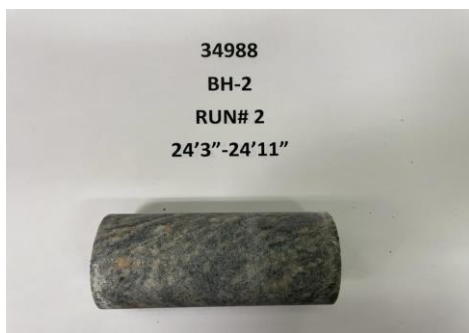
UNCONFINED COMPRESSION TEST REPORT

ASTM D7012-14

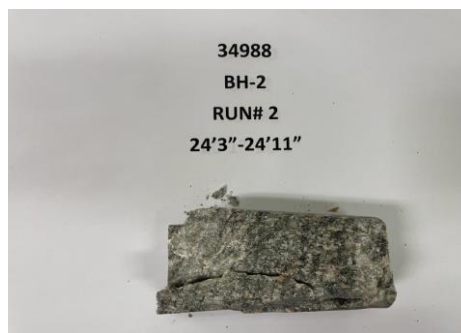
CLIENT:	HATCH	FILE NUMBER:	34988
PROJECT NAME:	Nestor Falls Bridge	REPORT DATE:	21-Jun-23
BOREHOLE No.:	22-02	TEST DATE:	13-Dec-22
SAMPLE No.:	Run 2		
SAMPLE DEPTH:	7.4 to 7.6 m		
DESCRIPTION:	Granodiorite		

Avg. Height (cm):	10.9	Weight (g):	521.9
Avg. Diameter (cm):	4.7	Wet Density (kg/m ³):	2,760
H. to Dia. Ratio**:	2.3:1	Dry Density (kg/m ³):	2,760
Cross Sectional Area (cm ²):	17.35	Moisture Content* (%):	N/A
Sample Volume (cm ³):	189.11		

ORIGINAL SPECIMEN



FRACTURED SPECIMEN



AVG. RATE OF STRAIN TO FAILURE:	0.250 MPa/s
MAXIMUM COMPRESSIVE LOAD:	217.8 kN
UNCONFINED COMPRESSIVE STRENGTH:	123.4 MPa

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

TEST DONE BY: AK
 REVIEWED BY:

34988 UCS BH 2 Run 2

Job No: 34988

Project Name: Nestor Falls Bridge

Core Size: NQ **BH No :** 22-01

Date Drilled: 09-Nov-22

Date Tested: 15-Nov-22

Tester: GA

Client: HATCH

[illegible]

POINT LOAD TEST SHEET

ASTM D5731-08

Job No: 34988

Project Name: Nestor Falls Bridge

Core Size:	NQ	BH No :	22-02
-------------------	----	----------------	-------

Date Drilled: 09-Nov-22

Date Tested: 15-Nov-22

Tester: GA

Client: HATCH

[illegible]



Photo C1: Borehole 22-01 Bedrock Core Sample (Runs 1 and 2)



Photo C2: Borehole 22-02 Bedrock Core Sample (Runs 1, 2 and 3)



FINAL REPORT

CA40190-NOV22 R1

34988, Nestor Falls, ON

Prepared for

Thurber Engineering Ltd.

First Page

CLIENT DETAILS

Client Thurber Engineering Ltd.

Address 103, 2010 Winston Park Drive
Oakville, ON
L6H 5R7, Canada

Contact Madisan Chiarotto

Telephone 647-548-8390

Facsimile

Email mchiarotto@thurber.ca

Project 34988, Nestor Falls, ON

Order Number

Samples Soil (2)

LABORATORY DETAILS

Project Specialist Maarit Wolfe, Hon.B.Sc

Laboratory SGS Canada Inc.

Address 185 Concession St., Lakefield ON, K0L 2H0

Telephone 705-652-2000

Facsimile 705-652-6365

Email Maarit.Wolfe@sgs.com

SGS Reference CA40190-NOV22

Received 11/14/2022

Approved 12/05/2022

Report Number CA40190-NOV22 R1

Date Reported 12/05/2022

COMMENTS

Temperature of Sample upon Receipt: 4 degrees C

Cooling Agent Present: Yes

Custody Seal Present: Yes

Chain of Custody Number: n/a

Corrosivity Index is based on the American Water Works Corrosivity Scale according to AWWA C-105. An index greater than 10 indicates the soil matrix may be corrosive to cast iron alloys.

SIGNATORIES

Maarit Wolfe, Hon.B.Sc





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FINAL REPORT

CA40190-NOV22 R1

Client: Thurber Engineering Ltd.

Project: 34988, Nestor Falls, ON

Project Manager: Madisan Chiarotto

Samplers: George Azzopardi

MATRIX: SOIL

Sample Number	5	6
Sample Name	22-01 CORR	22-02 CORR
Sample Matrix	Soil	Soil
Sample Date	11/11/2022	09/11/2022

Parameter	Units	RL		Result	Result
Corrosivity Index					
Corrosivity Index	none	1		4	14
Soil Redox Potential	mV	no		131	213
Sulphide (Na2CO3)	%	0.04		< 0.04	< 0.04
pH	pH Units	0.05		10.1	11.1
Resistivity (calculated)	ohms.cm	-9999		3400	471
General Chemistry					
Conductivity	uS/cm	2		294	2120
Metals and Inorganics					
Moisture Content	%	0.1		7.8	11.5
Sulphate	µg/g	0.4		52	100
Other (ORP)					
Chloride	µg/g	0.4		84	1400



FINAL REPORT

CA40190-NOV22 R1

QC SUMMARY

Anions by IC
Method: EPA300/MA300-Ions1.3 | Internal ref.: ME-CA-IENVIIC-LAK-AN-001

Parameter	QC batch Reference	Units	RL	Method Blank	Duplicate		LCS/Spike Blank			Matrix Spike / Ref.		
					RPD	AC (%)	Spike Recovery (%)	Recovery Limits (%)		Spike Recovery (%)	Recovery Limits (%)	
								Low	High		Low	High
Chloride	DIO0438-NOV22	µg/g	0.4	<0.4	4	35	99	80	120	80	75	125
Sulphate	DIO0438-NOV22	µg/g	0.4	<0.4	0	35	96	80	120	107	75	125

Carbon/Sulphur
Method: ASTM E1915-07A | Internal ref.: ME-CA-IENVIARD-LAK-AN-020

Parameter	QC batch Reference	Units	RL	Method Blank	Duplicate		LCS/Spike Blank			Matrix Spike / Ref.		
					RPD	AC (%)	Spike Recovery (%)	Recovery Limits (%)		Spike Recovery (%)	Recovery Limits (%)	
								Low	High		Low	High
Sulphide (Na2CO3)	ECS0066-NOV22	%	0.04	< 0.04	ND	20	102	80	120			

Conductivity
Method: SM 2510 | Internal ref.: ME-CA-IENVIEWL-LAK-AN-006

Parameter	QC batch Reference	Units	RL	Method Blank	Duplicate		LCS/Spike Blank			Matrix Spike / Ref.		
					RPD	AC (%)	Spike Recovery (%)	Recovery Limits (%)		Spike Recovery (%)	Recovery Limits (%)	
								Low	High		Low	High
Conductivity	EWL0343-NOV22	uS/cm	2	< 2	3	20	101	90	110	NA		



FINAL REPORT

CA40190-NOV22 R1

QC SUMMARY

pH
Method: SM 4500 | Internal ref.: ME-CA-IENVIEWL-LAK-AN-001

Parameter	QC batch Reference	Units	RL	Method Blank	Duplicate		LCS/Spike Blank			Matrix Spike / Ref.		
					RPD	AC (%)	Spike Recovery (%)	Recovery Limits (%)		Spike Recovery (%)	Recovery Limits (%)	
								Low	High		Low	High
pH	EWL0343-NOV22	pH Units	0.05	NA	0		101			NA		

Method Blank: a blank matrix that is carried through the entire analytical procedure. Used to assess laboratory contamination.

Duplicate: Paired analysis of a separate portion of the same sample that is carried through the entire analytical procedure. Used to evaluate measurement precision.

LCS/Spike Blank: Laboratory control sample or spike blank refer to a blank matrix to which a known amount of analyte has been added. Used to evaluate analyte recovery and laboratory accuracy without sample matrix effects.

Matrix Spike: A sample to which a known amount of the analyte of interest has been added. Used to evaluate laboratory accuracy with sample matrix effects.

Reference Material: a material or substance matrix matched to the samples that contains a known amount of the analyte of interest. A reference material may be used in place of a matrix spike.

RL: Reporting limit

RPD: Relative percent difference

AC: Acceptance criteria

Multielement Scan Qualifier: as the number of analytes in a scan increases, so does the chance of a limit exceedance by random chance as opposed to a real method problem. Thus, in multielement scans, for the LCS and matrix spike, up to 10% of the analytes may exceed the quoted limits by up to 10% absolute and the spike is considered acceptable.

Duplicate Qualifier: for duplicates as the measured result approaches the RL, the uncertainty associated with the value increases dramatically, thus duplicate acceptance limits apply only where the average of the two duplicates is greater than five times the RL.

Matrix Spike Qualifier: for matrix spikes, as the concentration of the native analyte increases, the uncertainty of the matrix spike recovery increases. Thus, the matrix spike acceptance limits apply only when the concentration of the matrix spike is greater than or equal to the concentration of the native analyte.

LEGEND

FOOTNOTES

NSS Insufficient sample for analysis.

RL Reporting Limit.

↑ Reporting limit raised.

↓ Reporting limit lowered.

NA The sample was not analysed for this analyte

ND Non Detect

Results relate only to the sample tested.

Data reported represent the sample as submitted to SGS. Solid samples expressed on a dry weight basis.

"Temperature Upon Receipt" is representative of the whole shipment and may not reflect the temperature of individual samples.

Analysis conducted on samples submitted pursuant to or as part of Reg. 153/04, are in accordance to the "Protocol for Analytical Methods Used in the Assessment of Properties under Part XV.1 of the Environmental Protection Act and Excess Soil Quality" published by the Ministry and dated March 9, 2004 as amended.

SGS provides criteria information (such as regulatory or guideline limits and summary of limit exceedances) as a service. Every attempt is made to ensure the criteria information in this report is accurate and current, however, it is not guaranteed. Comparison to the most current criteria is the responsibility of the client and SGS assumes no responsibility for the accuracy of the criteria levels indicated.

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This report supersedes all previous versions.

-- End of Analytical Report --



Environment, Health & Safety

Request for Laboratory Services and CHAIN OF CUSTODY

- Lakefield: 185 Concession St., Lakefield, ON K0L 2H0 Phone: 705-652-2000 Fax: 705-652-6365 Web: www.sgs.com/environment
- London: 657 Consortium Court, London, ON, N6E 2S8 Phone: 519-672-4500 Toll Free: 877-848-8060 Fax: 519-672-0361

No:

Page 1 of 1

Laboratory Information Section - Lab use only

Received By: Nicole Brizant
Received Date (mm/dd/yy): Nov 14/22
Received Time: 17:30

Received By (signature): [Signature]Custody Seal Present: ☒Custody Seal Intact: ☒Cooling Agent Present: ☒Temperature Upon Receipt (°C): 4.44

LAB LIMS #:

CA-40190-NOV22

REPORT INFORMATION		INVOICE INFORMATION		PROJECT INFORMATION																			
Company: <u>Thurber Engineering Ltd.</u>	<input checked="" type="checkbox"/> (same as Report Information)	Quotation #: _____	P.O. #: _____																				
Contact: <u>Madisan Chiarotto</u>	Company: _____	Project #: <u>34988</u>	Site Location/ID: <u>Nestor Falls ON</u>																				
Address: <u>103-2010 Winston Park Drive</u> <u>Oakville, Ontario</u>	Contact: _____	TURNAROUND TIME (TAT) REQUIRED																					
Phone: <u>647-548-8390</u>	Address: _____	<input checked="" type="checkbox"/> Regular TAT (5-7 days) TAT's are quoted in business days (exclude statutory holidays & weekends). Samples received after 6pm or on weekends: TAT begins next business day																					
Email: <u>mchiarotto@thurber.ca</u>	Phone: _____	RUSH TAT (Additional Charges May Apply): <input type="checkbox"/> 1 Day <input type="checkbox"/> 2 Days <input type="checkbox"/> 3 Days <input type="checkbox"/> 4 Days																					
Email: _____	Email: _____	PLEASE CONFIRM RUSH FEASIBILITY WITH SGS REPRESENTATIVE PRIOR TO SUBMISSION																					
REGULATIONS		NOTE: DRINKING (POTABLE) WATER SAMPLES FOR HUMAN CONSUMPTION MUST BE SUBMITTED WITH SGS DRINKING WATER CHAIN OF CUSTODY																					
Regulation 153/04: <input type="checkbox"/> Table 1 <input type="checkbox"/> R/P/I <input type="checkbox"/> Table 2 <input type="checkbox"/> W/C/C <input type="checkbox"/> Table 3 <input type="checkbox"/> A/O <input type="checkbox"/> Table _____	Soil Texture: <input type="checkbox"/> Coarse <input type="checkbox"/> Medium <input type="checkbox"/> Fine	Other Regulations: <input type="checkbox"/> Reg 347/558 (3 Day min TAT) <input type="checkbox"/> PWQO <input type="checkbox"/> MMR <input type="checkbox"/> CCME <input checked="" type="checkbox"/> Other: <input type="checkbox"/> MISA <input type="checkbox"/> O.Reg 406 Table 1	Sewer By-Law: <input type="checkbox"/> Sanitary <input type="checkbox"/> Storm Municipality: _____																				
RECORD OF SITE CONDITION (RSC) <input type="checkbox"/> YES <input checked="" type="checkbox"/> NO		ANALYSIS REQUESTED																					
SAMPLE IDENTIFICATION	DATE SAMPLED	TIME SAMPLED	# OF BOTTLES	MATRIX	Field Filtered (Y/N)	Metals & Inorganics	PAH <input type="checkbox"/> ABN <input type="checkbox"/> SVOC (all) <input type="checkbox"/>	PCB Total <input type="checkbox"/> Aroclor <input type="checkbox"/>	PHC F1-F4 <input type="checkbox"/> VOC <input type="checkbox"/>	BTEX <input type="checkbox"/> BTEX/F1 <input type="checkbox"/> F2-F4 <input type="checkbox"/>	VOC <input type="checkbox"/> BTEX <input type="checkbox"/> THM <input type="checkbox"/>	Pesticides OC <input type="checkbox"/> OP <input type="checkbox"/>	TCLP M&I <input type="checkbox"/> VOC <input type="checkbox"/> PCB <input type="checkbox"/>	B(a)P <input type="checkbox"/> ABN <input type="checkbox"/> Ignit. <input type="checkbox"/>	Water Pkg Gen. <input type="checkbox"/> Ext. <input type="checkbox"/>	Sewer Use: <input type="checkbox"/>	SVOCs (all) <input type="checkbox"/>	OC Pesticides <input type="checkbox"/>	PA Herbicides <input type="checkbox"/>	SAR/EC <input type="checkbox"/>	Corrosivity Package <input type="checkbox"/>	COMMENTS:	
1 22-01 CORR	<u>11/11/22</u>	AM	1	SOIL																			
2 22-02 CORR	11/9/22	AM	1	SOIL																			
3				SOIL																			
4				SOIL																			
5				SOIL																			
6				SOIL																			
7				SOIL																			
8				SOIL																			
9				SOIL																			
10				SOIL																			
11				SOIL																			
12				SOIL																			
Observations/Comments/Special Instructions																							
Sampled By (NAME): <u>George Azzopardi</u>		Signature: <u>[Signature]</u>		Date: _____ (mm/dd/yy)		Pink Copy - Client																	
Relinquished by (NAME): <u>Madisan Chiarotto</u>		Signature: <u>[Signature]</u>		Date: <u>11/14/22</u> (mm/dd/yy)		Yellow & White Copy - SGS																	

Revision #: 1.1

Date of Issue: 04 April, 2018



FINAL REPORT

CA40186-NOV22 R

Nestor Falls Bridge, Nestor Falls ON

Prepared for

Thurber Engineering Ltd.

First Page

CLIENT DETAILS

Client Thurber Engineering Ltd.

Address 103, 2010 Winston Park Drive
Oakville, ON
L6H 5R7, Canada

Contact Madisan Chiarotto

Telephone 647-548-8390

Facsimile

Email mchiarotto@thurber.ca

Project Nestor Falls Bridge, Nestor Falls ON

Order Number

Samples Solution (1)

LABORATORY DETAILS

Project Specialist Jill Campbell, B.Sc.,GISAS

Laboratory SGS Canada Inc.

Address 185 Concession St., Lakefield ON, K0L 2H0

Telephone 2165

Facsimile 705-652-6365

Email jill.campbell@sgs.com

SGS Reference CA40186-NOV22

Received 11/14/2022

Approved 12/14/2022

Report Number CA40186-NOV22 R

Date Reported 12/14/2022

COMMENTS

Temperature of Sample upon Receipt: 6 degrees C

Cooling Agent Present: Yes

Custody Seal Present: Yes

SIGNATORIES

Jill Campbell, B.Sc.,GISAS





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FINAL REPORT

CA40186-NOV22 R

Client: Thurber Engineering Ltd.

Project: Nestor Falls Bridge, Nestor Falls ON

Project Manager: Madisan Chiarotto

Samplers: George Azzopardi

MATRIX: WATER

Sample Number 6
Sample Name Nestor Falls
Sample Matrix Solution
Sample Date 11/11/2022

Parameter	Units	RL	Result
General Chemistry			
Conductivity	uS/cm	2	80
Redox Potential	mV	no	224
Sulphide	µg/L	6	< 6
Metals and Inorganics			
Sulphate	mg/L	0.04	1.3
Other (ORP)			
pH	No unit	0.05	7.31
Chloride	mg/L	0.04	3.0



FINAL REPORT

CA40186-NOV22 R

QC SUMMARY

Anions by IC
Method: EPA300/MA300-Ions1.3 | Internal ref.: ME-CA-IENVIIC-LAK-AN-001

Parameter	QC batch Reference	Units	RL	Method Blank	Duplicate		LCS/Spike Blank			Matrix Spike / Ref.		
					RPD	AC (%)	Spike Recovery (%)	Recovery Limits (%)		Spike Recovery (%)	Recovery Limits (%)	
								Low	High		Low	High
Chloride	DIO0118-DEC22	mg/L	0.04	<0.04	ND	20	101	90	110	98	75	125
Sulphate	DIO0118-DEC22	mg/L	0.04	<0.04	0	20	99	90	110	113	75	125

Conductivity
Method: SM 2510 | Internal ref.: ME-CA-IENVIEWL-LAK-AN-006

Parameter	QC batch Reference	Units	RL	Method Blank	Duplicate		LCS/Spike Blank			Matrix Spike / Ref.		
					RPD	AC (%)	Spike Recovery (%)	Recovery Limits (%)		Spike Recovery (%)	Recovery Limits (%)	
								Low	High		Low	High
Conductivity	EWL0352-NOV22	uS/cm	2	< 2	0	20	100	90	110	NA		

pH
Method: SM 4500 | Internal ref.: ME-CA-IENVIEWL-LAK-AN-006

Parameter	QC batch Reference	Units	RL	Method Blank	Duplicate		LCS/Spike Blank			Matrix Spike / Ref.		
					RPD	AC (%)	Spike Recovery (%)	Recovery Limits (%)		Spike Recovery (%)	Recovery Limits (%)	
								Low	High		Low	High
pH	EWL0352-NOV22	No unit	0.05	NA	0		100			NA		



FINAL REPORT

CA40186-NOV22 R

QC SUMMARY

Redox Potential
Method: SM 2580 I

Parameter	QC batch Reference	Units	RL	Method Blank	Duplicate		LCS/Spike Blank			Matrix Spike / Ref.		
					RPD	AC (%)	Spike Recovery (%)	Recovery Limits (%)		Spike Recovery (%)	Recovery Limits (%)	
								Low	High		Low	High
Redox Potential	EWL0330-NOV22	mV	no	NA	0	20	103	80	120	NA		

Sulphide by SFA
Method: SM 4500 I Internal ref.: ME-CA-IENVISFA-LAK-AN-008

Parameter	QC batch Reference	Units	RL	Method Blank	Duplicate		LCS/Spike Blank			Matrix Spike / Ref.		
					RPD	AC (%)	Spike Recovery (%)	Recovery Limits (%)		Spike Recovery (%)	Recovery Limits (%)	
								Low	High		Low	High
Sulphide	SKA0182-NOV22	ug/L	6	<0.006	ND	20	106	80	120	NA	75	125



FINAL REPORT

CA40186-NOV22 R

QC SUMMARY

Method Blank: a blank matrix that is carried through the entire analytical procedure. Used to assess laboratory contamination.

Duplicate: Paired analysis of a separate portion of the same sample that is carried through the entire analytical procedure. Used to evaluate measurement precision.

LCS/Spike Blank: Laboratory control sample or spike blank refer to a blank matrix to which a known amount of analyte has been added. Used to evaluate analyte recovery and laboratory accuracy without sample matrix effects.

Matrix Spike: A sample to which a known amount of the analyte of interest has been added. Used to evaluate laboratory accuracy with sample matrix effects.

Reference Material: a material or substance matrix matched to the samples that contains a known amount of the analyte of interest. A reference material may be used in place of a matrix spike.

RL: Reporting limit

RPD: Relative percent difference

AC: Acceptance criteria

Multielement Scan Qualifier: as the number of analytes in a scan increases, so does the chance of a limit exceedance by random chance as opposed to a real method problem. Thus, in multielement scans, for the LCS and matrix spike, up to 10% of the analytes may exceed the quoted limits by up to 10% absolute and the spike is considered acceptable.

Duplicate Qualifier: for duplicates as the measured result approaches the RL, the uncertainty associated with the value increases dramatically, thus duplicate acceptance limits apply only where the average of the two duplicates is greater than five times the RL.

Matrix Spike Qualifier: for matrix spikes, as the concentration of the native analyte increases, the uncertainty of the matrix spike recovery increases. Thus, the matrix spike acceptance limits apply only when the concentration of the matrix spike is greater than or equal to the concentration of the native analyte.

LEGEND

FOOTNOTES

NSS Insufficient sample for analysis.

RL Reporting Limit.

↑ Reporting limit raised.

↓ Reporting limit lowered.

NA The sample was not analysed for this analyte

ND Non Detect

Results relate only to the sample tested.

Data reported represent the sample as submitted to SGS. Solid samples expressed on a dry weight basis.

"Temperature Upon Receipt" is representative of the whole shipment and may not reflect the temperature of individual samples.

Analysis conducted on samples submitted pursuant to or as part of Reg. 153/04, are in accordance to the "Protocol for Analytical Methods Used in the Assessment of Properties under Part XV.1 of the Environmental Protection Act and Excess Soil Quality" published by the Ministry and dated March 9, 2004 as amended.

SGS provides criteria information (such as regulatory or guideline limits and summary of limit exceedances) as a service. Every attempt is made to ensure the criteria information in this report is accurate and current, however, it is not guaranteed. Comparison to the most current criteria is the responsibility of the client and SGS assumes no responsibility for the accuracy of the criteria levels indicated.

SGS Canada Inc. statement of conformity decision rule does not consider uncertainty when analytical results are compared to a specified standard or regulation.

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The Client's attention is drawn to the limitation of liability, indemnification and jurisdiction issues defined therein. Any other holder of this document is advised that information contained hereon reflects the Company's findings at the time of its intervention only and within the limits of Client's instructions, if any. The Company's sole responsibility is to its Client and this document does not exonerate parties to a transaction from exercising all their rights and obligations under the transaction documents. Reproduction of this analytical report in full or in part is prohibited.

This report supersedes all previous versions.

-- End of Analytical Report --



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- London: 657 Consortium Court, London, ON, N6E 2S8 Phone: 519-672-4500 Toll Free: 877-848-8060 Fax: 519-672-0361

Request for Laboratory Services and CHAIN OF CUSTODY

No:

Page 1 of 1

Laboratory Information Section - Lab use only

Received By: Nicole Bryant
Received Date (mm/dd/yy): Nov 14/22
Received Time: 12:30

Received By (signature): [Signature]
Custody Seal Present: ☒
Custody Seal Intact: ☒

Cooling Agent Present: ☒
Temperature Upon Receipt (°C): 6.66

LAB LIMS #: CA-40186-NOV 22

REPORT INFORMATION	INVOICE INFORMATION	PROJECT INFORMATION
Company: <u>Thurber Engineering Ltd.</u> Contact: <u>Madisan Chiarotto</u> Address: <u>103-2010 Winston Park Drive</u> <u>Oakville, Ontario</u> Phone: <u>647-548-8390</u> Email: <u>mchiarotto@thurber.ca</u> Email:	<input checked="" type="checkbox"/> (same as Report Information) Company: _____ Contact: _____ Address: _____ Phone: _____ Email: _____	Quotation #: _____ P.O. #: _____ Project #: <u>34988</u> Site Location/ID: <u>Nestor Falls Bridge, Nestor Falls ON</u> TURNAROUND TIME (TAT) REQUIRED <input checked="" type="checkbox"/> Regular TAT (5-7days) TAT's are quoted in business days (exclude statutory holidays & weekends). Samples received after 6pm or on weekends: TAT begins next business day RUSH TAT (Additional Charges May Apply): <input type="checkbox"/> 1 Day <input type="checkbox"/> 2 Days <input type="checkbox"/> 3 Days <input type="checkbox"/> 4 Days PLEASE CONFIRM RUSH FEASIBILITY WITH SGS REPRESENTATIVE PRIOR TO SUBMISSION Specify Due Date: _____ Rush Confirmation ID: _____

REGULATIONS						NOTE: DRINKING (POTABLE) WATER SAMPLES FOR HUMAN CONSUMPTION MUST BE SUBMITTED WITH SGS DRINKING WATER CHAIN OF CUSTODY																	
Regulation 153/04:			Other Regulations:			Sewer By-Law:			ANALYSIS REQUESTED														
<input type="checkbox"/> Table 1 <input type="checkbox"/> R/P/I <input type="checkbox"/> Table 2 <input type="checkbox"/> I/C/C <input type="checkbox"/> Table 3 <input type="checkbox"/> A/O <input type="checkbox"/> Table _____	Soil Texture: <input type="checkbox"/> Coarse <input type="checkbox"/> Medium <input type="checkbox"/> Fine		<input type="checkbox"/> Reg 347/558 (3 Day min TAT) <input type="checkbox"/> PWQO <input type="checkbox"/> MMR <input type="checkbox"/> CCME <input checked="" type="checkbox"/> Other: _____ <input type="checkbox"/> MISA <input type="checkbox"/> O.Reg 406 Table 1	<input type="checkbox"/> Sanitary <input type="checkbox"/> Storm Municipality: _____		Field Filtered (Y/N)	Metals & Inorganics	PAH <input type="checkbox"/> ABN <input type="checkbox"/> SVOC (all) <input type="checkbox"/>	PCB Total <input type="checkbox"/> Aroclor <input type="checkbox"/>	PHC F1-F4 <input type="checkbox"/> VOC <input type="checkbox"/>	BTEX <input type="checkbox"/> BTEX/F1 <input type="checkbox"/> F2-F4 <input type="checkbox"/>	VOC <input type="checkbox"/> BTEX <input type="checkbox"/> THM <input type="checkbox"/>	Pesticides OC <input type="checkbox"/> OP <input type="checkbox"/>	TCLP M&I <input type="checkbox"/> VOC <input type="checkbox"/> PCB <input type="checkbox"/>	B(a)p <input type="checkbox"/> ABN <input type="checkbox"/> Ignit. <input type="checkbox"/>	Water Pkg Gen. <input type="checkbox"/> Ext. <input type="checkbox"/>	Sewer Use: <input type="checkbox"/>	SVOCs (all) <input type="checkbox"/>	OC Pesticides <input type="checkbox"/>	PA Herbicides <input type="checkbox"/>	SAR/EC <input type="checkbox"/>	Corrosivity <input checked="" type="checkbox"/>	COMMENTS:
RECORD OF SITE CONDITION (RSC) <input type="checkbox"/> YES <input checked="" type="checkbox"/> NO																							
SAMPLE IDENTIFICATION		DATE SAMPLED	TIME SAMPLED	# OF BOTTLES	MATRIX																		
1	Nestor Falls	11/11/22	PM	5	WATER																		
2																							
3																							
4																							
5																							
6																							
7																							
8																							
9																							
10																							
11																							
12																							

Observations/Comments/Special Instructions

Sampled By (NAME): <u>George Azzopardi</u>	Signature: <u>[Signature]</u>	Date: _____ (mm/dd/yy)	Pink Copy - Client
Relinquished by (NAME): <u>Madisan Chiarotto</u>	Signature: <u>[Signature]</u>	Date: <u>11/14/22</u> (mm/dd/yy)	Yellow & White Copy - SGS



Appendix D

Site Photographs



Photo D1: Looking north along east side of Nestor Falls bridge and Highway 71 (Nov. 2022)



Photo D2: Looking south along east side of Nestor Falls bridge and Highway 71 (Nov. 2022)



Photo D3: Looking north along west side of Nestor Falls bridge near spillway structure (Nov. 2022)



Photo D4: Looking east towards west side of Nestor Falls bridge, spillway and falls (Nov. 2022)



Photo D5: Looking north along Highway 71 towards bridge and north approach (Nov. 2022)



Photo D6: Looking south along Highway 71 towards bridge and south approach (Nov. 2022)



Appendix E

List of Referenced OPSS and OPSD Documents



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

OPSS.PROV 206	Construction Specification for Grading
OPSS.PROV 501	Construction Specification for Compacting
OPSS.PROV 539	Construction Specification for Temporary Protection Systems
OPSS.PROV 804	Construction Specification for Seed and Cover
OPSS.PROV 903	Construction Specification for Deep Foundations
OPSS.PROV 1010	Material Specification for Aggregates Base, Subbase, Select Subgrade, and Backfill Material
OPSD 3090.100	Foundation Frost Penetration Depths for Northern Ontario
OPSD 3101.150	Walls, Abutment, Backfill Minimum Granular Requirement