



## Foundation Investigation and Design Report

*North Driftwood River Bridge Replacement, Highway 11*

Assignment No. 5018-E-0012

Work Item No. 7- Part A

GWP 5282-14-00

Geocres No. 42H-86

Latitude: 49.188917; Longitude: -81.438255

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**Prepared For:**

Ontario Ministry of Transportation  
Geotechnical Section, Northeastern Region  
447 McKeown Avenue, Suite 301  
North Bay, ON P1B 9S9  
Attn: Mark Winmill

**Prepared By:**

EXP Services Inc.  
56 Queen St, East, Suite 301  
Brampton, ON L6V 4M8  
Canada

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*Ministry of Transportation Ontario  
 Northeastern Region Geotechnical Section*

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# 1 FOUNDATION INVESTIGATION REPORT

## 1.1 Introduction

This report presents the results of a geotechnical investigation completed by EXP Services Inc. for the replacement of the existing North Driftwood River Bridge on Highway 11. North Driftwood River Bridge is located approximately 10.8 km west of the Highway 655 junction in the Cochrane Area (Latitude: 49.188917; Longitude: -81.438255). The work was undertaken under Assignment No. 5018-E-0012, Work Item No. 7. The terms of reference (TOR) were provided by MTO in an email dated June 11, 2020.

As noted in the TOR, the proposed replacement will be a single span bridge on a new alignment. The new alignment is located just to the south of the existing bridge. Photographs of the existing site/bridge are included in Appendix A.

This foundation investigation report has been prepared specifically and solely for the project described herein. It contains the factual results of the investigation and the laboratory testing completed for this project.

## 1.2 Site Description and Geological Setting

### 1.2.1 Site Description

The existing North Driftwood river bridge is located on Highway 11 (between Sta. 12+816 and Sta. 12+895), about 10.8 km west of the junction of Highway 655 in the Cochrane District, Ontario. The bridge is located on the boundary between the geographic townships of Calder (to the south) and Colquhoun (to the north). At the site, Highway 11 is two lane roadway, with a speed limit of 90 km/h (unless otherwise posted).

Based on the information provided by MTO, the existing bridge was constructed in 1943 and underwent rehabilitation activities in 1987 and 1988. The bridge consists of five (5) spans supported on cruciform sheet pile piers within the North Driftwood River. The bridge superstructure consists of a concrete T-beam deck with concrete barrier walls. The total span of the existing bridge is 79.25 m, consisting of five 15.85 m spans. Based on available information, we understand that the bridge abutments and piers were supported on 1.9 m diameter caissons formed from twelve (12) sections of driven sheet piles with bent flanges, filled with reinforced concrete. As reported, the original abutments failed due to excessive lateral earth pressure and in 1986/1987, they were reinforced with sixteen (16) HP 310 x 110 driven piles each in the original caissons and the pile cap reconstructed to re-incorporate the caissons/piles at the original design level. Select photographs of the site and existing bridge are presented in Appendix A. The site plan, cross-sections and profiles for the proposed alignment are shown on the drawings attached in Appendix B.

The general site conditions were assessed during the site reconnaissance in July 27, 2020. Highway 11 generally runs in east-west direction and the river flows south - north beneath the bridge. The elevations of highway pavement centerline at the west and east abutments are about 251.7 m and 250.8 m, respectively. The surrounding area south of the existing highway approach embankment is at about Elev. 247.5 m. As reported, the original ground surface at the location of the existing highway is at about Elev. 249 m, indicating the embankments are about 2 m to 3 m high. The existing embankment side slopes are estimated to be around 2H:1V. Based on observations on the site,

these embankment side slopes appear to be stable (i.e. no visible sign of slope instability and/or settlement of the roadway).

### 1.2.2 Geological Setting

According the Ministry of Northern Development and Mines, Map 2555 (Quaternary Geology of Ontario, East-Central Sheet, 1991) the surface conditions in the vicinity of the project area consists of till, and nearby organic terrain. According to Map 2543 (Bedrock Geology of Ontario, East-Central Sheet, 1991), the bedrock geology of the site is of massive to foliated granodiorite to granite bedrock.

### 1.3 Previous Investigations

The available report of the previous investigation in the MTO GEOCRE library is:

*Geocres No. 42H-80: "Preliminary Foundation Investigation and Design Report, North Driftwood River Bridge Replacement (Site 39E-0013/B0) Highway 11, Driftwood, Ontario" prepared by Golder Associates Ltd., April 10, 2019.*

The details of the boreholes completed by Golder Associate are outlined in Table 1.1 and the borehole locations are shown on Drawings in Appendix B. The borehole logs are included in Appendix H.

Table 1.1. Summary of boreholes completed by Golder Associates Ltd. in 2018

Borehole No.	Location (MTM NAD 83 Zone 12 )		Ground Surface Elevation (m)	Borehole Depth (m)	Bedrock Coring Length (m)
	Northing	Easting			
18-1	5450183.8	272857.1	248.7	12.0	3.4
18-2	5450198.9	272922.3	246.9	11.1	3.3

In addition, MTO provided the following document:

*Preliminary Design Report, Highway 11 Replacement of North Driftwood River Bridge, GWP 5282-14-00, prepared by D.M. Wills Associates Limited, dated November 2019.*

### 1.4 Investigation Procedures

#### 1.4.1 Site Investigation and Field Testing

The field investigation was performed between July 28 and August 27, 2020 for the North Driftwood River Bridge replacement by EXP. The field program consisted of drilling seven (7) sampled boreholes, numbered BH20-1 to BH20-7, and performance of several in-situ tests such as conventional standard penetration tests, packer tests and slug tests. As noted before, the previous geotechnical investigation conducted by Golder in 2018 consisted of two

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sampled boreholes BH18-1 and BH18-2 drilled in the vicinity of the existing west and east abutments, respectively. Since the new bridge is proposed to be built to the south of the existing bridge, the current investigation consisted of one (1) borehole strategically located at the proposed location of west abutment for the new bridge alongside BH18-1 (south of the existing bridge) and two (2) boreholes located at the proposed location of east abutment of the new bridge (south of the existing bridge). In addition, two (2) boreholes were located on each side of the river, between the new and existing bridges, to provide subsurface information for the temporary roadway protections. The locations of boreholes drilled during current and previous investigations are shown on Drawing 1 in Appendix B.

All off-road boreholes drilled during this fieldwork were advanced using a track mounted CME 55 drill rig equipped with hollow stem augers and standard soil sampling /bedrock sampling equipment and NW casing, operated by a specialist drilling contractor, Landcore Drilling. The off-road boreholes (BH20-1 to BH20-6) were advanced to a depth of between 9.8 m and 14.0 m. The roadway borehole BH20-7 was advanced using a truck mounted drill rig to a depth of about 8.5 m below ground surface.

The borehole locations (referenced to the MTM NAD83 coordinate system) and their ground surface elevations were surveyed by EXP personnel using a benchmark (BM) HCP 101 (19mm x 2.0m RIB) on top of the Round Iron Bar (RIB) located northwest of Highway 11. The benchmark (BM) was at Elev. 251.651 m. The BM location is shown on Drawing 1 in Appendix B.

For the drilling program, soil samples were obtained using a 51 mm outside diameter (O.D.) split-spoon sampler in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586) at intervals ranging from 0.75 m to 1.5 m in depth as shown on the attached borehole logs (Appendix C). The original field (uncorrected) SPT "N" values were recorded on the borehole logs as recommended in the Canadian Foundation Engineering Manual (CFEM, pg. 40) and used to provide an assessment of in-situ relative density of non-cohesive soils. In addition, one Shelby tube was driven into the organic silt layer encountered in BH20-1 to obtain undisturbed samples for consolidation tests; however, the Shelby tube was damaged by the underlying cobbles and boulders layer and the soil sample inside the tube was completely disturbed. Shelby tube samples were not attempted to be obtained from the other boreholes since the cohesive soil was not encountered in those boreholes. When a hard stratum was reached sampling of hard material was performed by diamond core drilling, using a 1.5 m long NQ double tube wireline core barrel. Core samples of the bedrock in BH20-1, BH20-4 and BH20-5 were obtained using a 1.5 m long NQ double tube wireline core barrel (core diameter ~47 mm).

Upon completion of the boreholes, groundwater level measurements were carried out in boreholes in accordance with MTO guidelines. A standpipe piezometer was installed in BH20-4 to permit monitoring of groundwater level at the east side of the river. The groundwater level at the west side was measured in the existing piezometer installed in BH18-1 by Golder. The recorded groundwater levels after completion of drilling boreholes and in the piezometer were presented in the borehole log sheets in Appendix C. All boreholes, including BH18-1 and BH20-4 with piezometers, were decommissioned by bentonite/cement mixtures in accordance with the Ministry of the Environment Regulation 903, as amended by Regulation 128/03 (the well regulation under the *Ontario Water Resources Act*).

Prior to their decommissioning, the in-situ permeability tests (slug tests) were conducted in BH20-4 and BH18-1 using the procedure described in ASTM D4044-15 (ASTM D4044-15 Standard Test Method -Field Procedure for Determining Hydraulic Properties of Aquifers). In addition, packer testing in the rock was conducted in BH20-4 between Elev. 235 m and 233.8 m (corresponding depth 12.8 m to 14 m) prior installation of the piezometer.

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The fieldwork was supervised by an EXP geotechnical representative who directed the drilling and sampling operation, logged borehole data in accordance with MTO and/or ASTM Standards for Soils Classification and retrieved soil samples for subsequent laboratory testing and identification.

All recovered soil samples were placed in labelled moisture-proof bags and returned to EXP's Sudbury and Brampton laboratory for additional visual, textual, olfactory examination and selective testing.

Table 1.2. Summary of boreholes completed by EXP in 2020

Borehole No.	Location (MTM NAD 83 Zone 12 )		Ground Surface Elevation (m)	Borehole Depth (m)	Bedrock Coring Length (m)
	Northing	Easting			
20-1	5450176.2	272855.4	247.5	12.8	3.3
20-2	5450175.2	272831.4	247.9	9.8	-
20-3	5450182.2	272829.4	251.8	9.8	-
20-4	5450173.2	272927.4	247.8	14.0	3.5
20-5	5450182.2	272921.4	247.5	12.0	3.6
20-6	5450173.2	272947.4	248.0	9.8	-
20-7	5450189.2	272948.4	251.0	8.5	-

#### 1.4.2 Laboratory Testing

All soil and rock samples returned to the laboratory were subjected to visual examination and classification. The laboratory testing program included the determination of natural moisture content on all soil samples and particle size distribution for approximately 25% of the collected soil samples. One soil sample was selected for chemical analysis and tested at a CALA-certified and accredited laboratory. Uniaxial compression tests were carried out on selected rock core samples. All of the laboratory tests were carried out according to MTO and/or ASTM Standards as appropriate.

### 1.5 Subsurface Conditions

The detailed subsurface conditions encountered in the boreholes advanced during this investigation are presented on the borehole log sheets in Appendix C. Laboratory test results of grain size analyses are provided in Appendix D. The "Explanation of Terms Used in Report" preceding the borehole logs in Appendix C forms an integral part of and should be read in conjunction with this report.

A borehole location plan and cross section subsurface profiles are provided in Appendix B. It should be noted that the stratigraphic boundaries indicated on the borehole log and cross section stratigraphic profiles are inferred from semi-continuous sampling, observations of drilling progress and results of Standard Penetration Tests. These

boundaries typically represent transitions from one soil type to another and should not be regarded as exact planes of geological change. Furthermore, subsurface conditions may vary between and beyond the borehole locations.

In general, the subsoil condition below the proposed bridge location on the west side of the river consists of topsoil/organic silt underlain by fill and peat/organic silt, followed by native cobbles and boulders/sandy gravel/sand layers overlaying igneous bedrock. On the east side of the river the subsoil stratigraphy below the proposed bridge consists of topsoil underlain by fill followed by native sand/silt/cobbles and boulders/ gravelly sand layers above the bedrock.

A detailed description of the subsurface conditions encountered is discussed further in subsequent sections. It should be noted that the following sections are based on the geotechnical investigation conducted by EXP.

### 1.5.1 Subsoils

#### 1.5.1.1 Topsoil

Topsoil, approximately 0.1 m thick, was encountered at the surface of boreholes BH20-1 and BH20-3 to BH20-6. Topsoil thicknesses may further vary beyond the borehole locations.

#### 1.5.1.2 Pavement Structure

Asphalt layer, approximately 0.2 m thick, was encountered at the surface of borehole BH20-7 advanced on the paved area. Pavement structure consists of varying proportions of sand and gravel (base). The pavement structure extended to depths of about 1.5 m below ground surface with elevations ranging between Elev. 251.0 m to Elev. 249.5 m. The explored thickness of this granular base was about 1.3 m.

Laboratory testing performed on a selected sample consisted of moisture content and grain size distribution tests. The test results are as follows:

Moisture Content:

- 3.7%

Grain Size Distribution:

- 41% gravel;
- 56% sand;
- 3% silt and clay.

The results of the moisture content and grain size distribution tests are included on the borehole logs. The result of the grain size distribution test is also provided on Figure 1 in Appendix D.

#### 1.5.1.3 Fill: Organic Silt/ Organic Silty Sand

Organic Silt/ Organic Silty Sand fill was encountered below the topsoil in boreholes BH20-1 and BH20-6. This organic fill layer extended to depth ranging between 0.6 m to 1.5 m below ground surface with elevation ranging between Elev. 246.9 m to Elev. 246.5 m. The explored thickness of this layer was between 0.5 m to 1.4 m.

The composition of this fill material generally consisted of organic silt with some sand, some gravel. The fill was generally dark brown to grey in color, and moist. The SPT “N” values obtained within this fill material ranged from 11 to 24 blows per 0.3 m penetration, suggesting compact in relative density.

Laboratory testing performed on selected samples consisted of moisture content tests. The test results are as follows:

Moisture Content:

- 3.6% to 18.0%

The results of the moisture content tests are provided on the record of borehole sheets in Appendix C.

#### 1.5.1.4 Fill: Clayey Silt/ Silt/ Sandy Silt to Silty Sand/ Sand

Clayey Silt/ Silt/ Sandy Silt to Silty Sand/ Sand fill was encountered below the topsoil in boreholes, BH20-3, BH20-4 and BH20-5 and below the pavement structure in borehole BH20-7. The fill layer extended to depth ranging between 1.5 m to 4.6 m below ground surface with elevation ranging between Elev. 250.3 m to Elev. 246.0 m. The explored thickness of this layer was between 1.4 m to 3.1 m.

The composition of this fill material generally consisted of sand, silt and clay with trace to some organics, some gravel, some wood. The fill was generally dark brown to brown in color, and moist. The SPT “N” values obtained within this fill material ranged from 2 to 10 blows per 0.3 m penetration, suggesting very loose to compact in relative density.

Laboratory testing performed on selected samples consisted of moisture content and grain size distribution tests. The test results are as follows:

Moisture Content:

- 14.3% to 23.0%

Grain Size Distribution:

- 0% gravel;
- 7% sand;
- 75% silt; and
- 18% clay.

The results of the moisture content and grain size distribution tests are provided on the record of borehole sheets in Appendix C. The results of the grain size distribution tests are also provided on Figure 2 in Appendix D.

#### 1.5.1.5 Peat

Peat was encountered below the organic silt fill in borehole BH20-1. This peat layer extended to depth of about 2.3 m below ground surface corresponding to elevation of about Elev. 245.2 m. The explored thickness of this layer was about 1.7 m.

The composition of this material generally consisted of organic non-cohesive material. This layer was generally dark brown to brown in color, and moist. The SPT “N” values obtained within this material ranged from 4 to 7 blows per 0.3 m penetration, suggesting very loose to loose in relative density.

Laboratory testing performed on selected samples consisted of moisture content tests. The test results are as follows:

Moisture Content:

- 88.4% to 110.1%

The results of the moisture content tests are provided on the record of borehole sheets in Appendix C.

#### 1.5.1.6 Organic Silt

A layer of organic silt was encountered below the peat deposit in borehole BH20-1 and at the ground surface in borehole BH20-2. This organic silt layer extended to depth ranging between 1.5 m to 3.5 m below ground surface with elevation ranging between Elev. 246.5 m to Elev. 244.0 m. The explored thickness of this layer was between 1.2 m to 1.5 m.

The composition of this material generally consisted of organic silt with some sand, some clay and some wood. This organic silt layer was generally dark brown to brown in color, and moist. The SPT “N” values obtained within this material ranged from 3 to 11 blows per 0.3 m penetration, suggesting very loose to compact in relative density.

Laboratory testing performed on selected samples consisted of moisture content and grain size distribution tests. The test results are as follows:

Moisture Content:

- 16.5% to 61.5%

Grain Size Distribution:

- 0% gravel;
- 11% sand;



- 76% silt; and
- 13% clay.

The results of the moisture content and grain size distribution tests are provided on the record of borehole sheets in Appendix C. The results of the grain size distribution tests are also provided on Figure 3 in Appendix D.

#### 1.5.1.7 Silt to Sand and Silt

Native silt to sand and silt was encountered in boreholes BH20-2 to BH20-7, below the native organic silt, sand and fill in boreholes BH20-2, BH20-3 and BH20-4 to BH20-7, respectively. The silt to sand and silt layer extended to depth ranging between 2.3 m to 7.6 m below ground surface with elevations ranging between Elev. 245.2 m to Elev. 241.9 m. The explored thickness of this layer was between 0.8 m to 5.3 m.

The composition of this layer is silt and sand with trace gravel, trace clay and trace wood. The material is brown and grey in color, and moist to wet. The SPT “N” values obtained within this layer is between 7 to 45 blows per 0.3 m penetration, suggesting loose to dense in relative density.

Laboratory testing performed on a selected sample consisted of moisture content and grain size distribution tests. The test results are as follows:

##### Moisture Content:

- 13.1% to 22.5%

##### Grain Size Distribution:

- 7% to 22% gravel;
- 6% to 13% sand;
- 62% to 80% silt; and
- 6% to 18% clay

The results of the moisture content and grain size distribution tests are provided on the record of borehole sheets in Appendix C. The results of the grain size distribution tests are also provided on Figure 4 in Appendix D.

#### 1.5.1.8 Cobbles and Boulders

Native layer of cobbles and boulders was encountered below the organic silt, sand and silt layers in boreholes BH20-1, BH20-4 and BH20-6, respectively. The cobbles and boulders deposit extended to depths ranging between 6.1 m to 9.2 m below ground surface with elevations ranging between Elev. 241.4 m to Elev. 238.8 m. The explored thickness of this layer was between 1.3 m to 2.6 m.

The composition of this layer is cobbles and boulders. The SPT “N” values obtained within this layer is between 100 to 135 blows per 0.3 m penetration. In BH20-1 this layer was attempted to core using NW casing, however, the sample recovery was zero.

Laboratory testing performed on a selected sample consisted of moisture content tests. The test results are as follows:

Moisture Content:

- 12.4% to 18.0%

The results of the moisture content tests are provided on the record of borehole sheets in Appendix C.

#### 1.5.1.9 Sand to Gravelly Sand/ Sandy Gravel

Native sand to gravelly sand/ sandy gravel layer was encountered in all the boreholes, BH20-1 to BH20-7, below the silt to sand and silt/ cobbles and boulders. The sand to gravelly sand/ sandy gravel layer extended to depths ranging between 8.5 m to 10.5 m below ground surface with elevations ranging between Elev. 242.5 m to Elev. 237.3 m. The explored thickness of this layer was between 0.6 m to 6.6 m. Boreholes BH20-2, BH20-3, BH20-6 and BH20-7 were terminated within this layer.

The composition of this layer is sand and gravel with cobbles and trace to some silt/clay. The material is brown to grey in color, and wet. The SPT “N” values obtained within this layer is between 15 to 120 blows per 0.3 m penetration, suggesting compact to very dense in relative density.

Laboratory testing performed on a selected sample consisted of moisture content and grain size distribution tests. The test results are as follows:

Moisture Content:

- 8.0% to 20.3%

Grain Size Distribution:

- 15% to 68% gravel;
- 24% to 80% sand;
- 5% to 25% silt and clay

The results of the moisture content and grain size distribution tests are provided on the record of borehole sheets in Appendix C. The results of the grain size distribution tests are also provided on Figure 5 in Appendix D.

#### 1.5.2 Bedrock

The presence of bedrock, at approximately between 8.5 m to 10.5 m below the existing ground surface was recorded. The bedrock was confirmed using coring of about 3 m long cores in boreholes BH20-1, BH20-4, BH20-5, BH18-1 and BH18-2. The actual bedrock surface depth and elevation encountered at these borehole locations are listed in Table 1.3. Photographs of rock cores are included in Appendix D.

Table 1.3. Depth and elevation of bedrock surface

Borehole	Depth Below Ground Surface (m)	Elevation (m)	Comments
BH20-1	9.5	238.0	Bedrock Cored
BH20-4	10.5	237.3	Bedrock Cored
BH20-5	8.5	239.0	Bedrock Cored
BH18-1	8.6	240.1	Bedrock Cored
BH18-2	7.8	239.1	Bedrock Cored

Based on the bedrock NQ cores (~ core diameter 47 mm) recovered, the bedrock at the site consists of meta-diorite with zones of granite. In general, the rock samples are described as grey to pink in colour, coarse grained, fresh to slightly weathered. The Rock Quality Designation (RQD) measured on the core samples typically ranged from approximately 68% to 100%, indicating a rock mass of fair to excellent quality with an exception of BH20-1 Run 2 core of about 0% indicating very poor quality. The total core recovery (TCR) ranges from about 14.3% to 100%, but mostly above 90%.

The uniaxial compression tests were performed on rock cores from BH20-1, BH20-4, BH18-1 and BH18-2 and the uniaxial compressive strength (UCS) was measured to be about 100.7 MPa, 85.3 MPa, 47 MPa and 39 MPa, respectively, indicating medium strong to very strong (R3 to R5) rock, according to CFEM. The laboratory uniaxial compression tests results are presented on the borehole records in Appendices C and H, as well as, in Appendix D.

## 1.6 Groundwater and Surface Water Conditions

The groundwater levels in the boreholes were observed during drilling, upon completion of drilling and in piezometers. Groundwater levels would be expected to reflect levels in the adjacent open water and to fluctuate seasonally. Seasonal variations in the water table should be expected, with higher levels occurring during wetter periods of the year and lower levels during drier periods.

A summary of the groundwater levels observed during and after the investigation are summarized in Table 1.4 and are also presented on the record of borehole sheets in Appendix C.

Table 1.4. Summary of observed groundwater levels

Borehole	Ground Surface Elevation (m)	Water level Depth/ Elevation (m)	Date
BH20-3	251.8	Dry	In open hole (Aug. 7, 2020)
BH20-4	247.8	2.55/245.25	In open hole (Jul. 28, 2020)
		2.6/245.20	In Piezometer (Aug. 4, 2020)
		2.3/245.5	In Piezometer (Aug 27, 2020)

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BH20-7	251.0	5.4/245.60	In open hole (Aug. 18, 2020)
BH18-1	248.7	0.8/247.9	In Piezometer (Oct. 31, 2018)
		1.0/247.7	In Piezometer (Nov. 29, 2018)
		1.38/247.3	In Piezometer (Aug. 27, 2020)

In addition, the groundwater level in the existing piezometer installed in Golder's BH18-1 was measured on August 27, 2020 by EXP and recorded level was at the depth of 1.38 m below the ground surface, corresponding to Elev. 247.3 m. In October 2018 and November 2018, the depths of groundwater in this piezometer were recorded by Golder to be 0.8 m and 1.0 m below the ground surface (Elev. 248.7 m), respectively, corresponding to Elev. 247.9 m and Elev. 247.7 m.

On August 7, 2020, the water level of North Driftwood River was measured to be at Elev. 245.93 m. The water level of the river was about Elev. 245.2 m on November 1982, at about Elev. 245.5 in November 2018 and about Elev. 244.1 m in January 2019, as measured by others.

## 1.7 Hydraulic Properties

### 1.7.1 Hydraulic Properties of Overburden Soil (Slug Test Results)

The in-situ permeability tests, referred to as slug tests, were also performed on August 27, 2020 in the piezometers installed in BH18-1 and BH20-4. The screen of these piezometers was installed in the native sand/sand and gravel/sandy silt layers in BH18-1 and in the sand layer in BH20-4. The slug test was performed to evaluate the transmissivity and hydraulic conductivity of these layers.

The slug test method is based on causing a sudden change in head in a control well followed by making a rapid series of water-level measurements to assess the rate of water-level recovery within that control well (return to the initial static water level). The head change is induced either by adding or removing a measured quantity of water in the well rapidly, usually by injection and/or removal of a mechanical "slug". The slug test can thus be either rising-head or falling-head test.

The raw data obtained at the site for rising head and falling head slug tests were analyzed using the standard Hvorslev method (Hvorslev, 1951). The summary of results is shown in Table 1.5. The complete data and results are presented in Appendix D.

Table 1.5. Summary of slug tests results

Piezometers	Location	Depth of Well (m)	Depth of Water (m)	Elevation of Water (m)	Hydraulic Conductivity k (m/s)	Type of Soil
BH18-1	West side	8.6	1.38	247.32 <sup>(1)</sup>	1.69x10 <sup>-5</sup>	Sand/Sand and Gravel/Sandy Silt

BH20-4	East side	6.2	2.33	245.47	1.11x10 <sup>-5</sup>	Sand
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Note:

1. Based on Golder's BH18-1 log

### 1.7.2 Rock Mass Secondary Hydraulic Conductivity (Packer Test Results)

A packer test was conducted in the rock of BH20-4 before the piezometer was installed. The test was conducted at the depth between 12.8 m and 14 m below the ground surface. The hydraulic conductivity of the rock at that depth was calculated from packer test results using the formulae given in the United States Bureau of Reclamation Earth Manual (1963):

$$k = \frac{Q}{2\pi LH_t} \ln\left(\frac{L}{r}\right)$$

where k=hydraulic conductivity,  
 Q=constant rate of flow into the borehole,  
 Ht=hydrostatic pressure head;  
 L=test length, and  
 r=radius of borehole.

The results of packer tests are presented in Appendix D. The measured value of rock mass secondary hydraulic conductivity at depth between 12.8 m and 14 m below the ground surface was approximately 2.1x10<sup>-7</sup> m/s.

## 1.8 Chemical Analysis

One soil sample was selected for chemical analysis during the current investigations performed by EXP. The soil sample collected by EXP was tested at a CALA-certified and accredited laboratory. The results of the corrosion potential chemical analysis testing including sulfide, chloride, sulfate, pH, electrical conductivity, resistivity and redox potential are summarized in Table 1.6.

Table 1.6. Summary of chemical analysis results

Borehole ID	Sample	Depth (m)	Chloride (ppm)	Sulphate (ppm)	pH	Electrical Conductivity (mS/cm)	Resistivity (ohm-cm)	Redox Potential (mV)
BH20-3	SS4	2.3 – 2.9	186	13	8.92	0.509	1960	56

## 2 ENGINEERING DISCUSSION & RECOMMENDATIONS

### 2.1 General

This section of the report provides geotechnical design recommendations for North Driftwood River Bridge replacement on Highway 11, located about 10.8 km west of the Highway 655 junction the Cochrane District, Ontario, the Ministry of Transportation (MTO) Northeastern Region. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the current investigation at the site performed by EXP dated August 2020. The compiled factual data is presented in **Part I-Foundation Investigation Report** of this report. The interpretation and recommendations provided are intended solely to permit designers, to assess foundation alternatives and design the new bridge and its replacement. Comments on construction are only provided to highlight issues that could affect the design. Contractors bidding on the works should make their own assessments of the factual data and how it might affect construction means and methods, scheduling and the like.

Based on the information provided by MTO, the existing five (5) span bridge was constructed in 1943. The total span of the existing bridge is 79.25 m, consisting of five 15.85 m spans. The bridge superstructure consists of a concrete T-beam deck (~ 10 m wide) with concrete barrier walls. The bridge was supported on cruciform sheet pile piers within the North Driftwood River. Based on available information, we understood that the bridge abutments and piers were supported on 1.9 m diameter caissons formed from twelve (12) sections of driven sheet piles with bent flanges, filled with reinforced concrete. As reported, the original abutments failed due to excessive lateral earth pressure and in 1986/1987, they were reinforced with sixteen (16) HP 310 x 110 driven piles each in the original caissons and the pile cap reconstructed to re-incorporate the caissons/piles at the original design level.

As noted in the TOR, the proposed replacement of the existing bridge will be a single span bridge on a new alignment located just to the south of the existing bridge. According to the preliminary GA drawing provided by MTO and attached in Appendix B, the new bridge is proposed to be an approximately 62 m long single span bridge with abutments supported on the caissons with steel liners. The proposed elevation of the top of the west and east abutments are Elev. 252.9 m and 252.2 m, respectively, resulting in a grade raise of about 5 m with respect to the ground surface at the toe of the existing approach embankments. Additionally, it appears that the top of the new abutments will be approximately 1.2 m to 1.4 m higher than those of the existing bridge, resulting in higher approach embankments for the new bridge. It is assumed that the existing bridge will be used to maintain the traffic flow on Highway 11 during the construction of the new bridge.

During the site investigation, the water level of North Driftwood River was measured at Elev. 245.9 m in Aug. 7, 2020. Seasonal variations in the water level should be expected, with higher levels occurring during wetter periods of the year and lower levels during drier periods. Considering the permeable nature of the surrounding soils, it is reasonable to expect that the groundwater level at the site is not going to be lower than the river water. Therefore, based on the measured water level in the river and observed groundwater in the piezometers it is anticipated that the groundwater level would be about Elev. 248 m on the west side and Elev. 246 m on the east side.

This report addresses the geotechnical design of the foundation for the proposed bridge structure by providing geotechnical design parameters at the Ultimate Limit State (ULS) and Serviceability Limit States (SLS) as well as other geotechnical parameters that may be required in accordance with the latest edition of the Canadian Highway Bridge Design Code (CHBDC) (CAN/CSA-S6-19), the Guideline for Professional Engineers Providing Geotechnical Engineering Service (1992), the Canadian Foundation Engineering Manual (CFEM) (2006), the provisions in the TOR and good

practice. The proposed structure and its foundation system are interpreted to be classified as having a “typical” consequence level associated with exceeding limits states design. Given the level of foundation investigation completed, the level of confidence for design is interpreted to be “typical” degree of site and prediction model understanding. Table 6.1 and 6.2 of the CHBDC, CAN/CSA-S6-19, 2019 have been used in the design to establish the appropriate consequence factor and geotechnical resistance factors.

The report provides discussion about the structure foundation type as well as other geotechnical and construction considerations such as assessment of slope stability and settlement of approach embankments, lateral earth pressure on structure, site preparation, excavation, dewatering, frost and scour protections.

## 2.2 Structure Foundations

### 2.2.1 Foundation Alternatives

Based on the sub-surface conditions encountered at the proposed bridge site, various shallow and deep foundation options have been considered. Table 2.1 shows the advantages and disadvantages of the considered options.

Table 2.1. Evaluation of foundation alternatives

Options	Advantages	Disadvantages	Relative Costs	Risks/ Consequences
<b>Shallow Foundation</b>  Spread footing supported on native soils (compact to very dense sand to gravelly sand deposit) or on a granular pad	<ul style="list-style-type: none"> <li>▪ Straightforward construction</li> <li>▪ Permits semi-integral abutment</li> </ul>	<ul style="list-style-type: none"> <li>▪ Removal of unsuitable native soils will be required below the founding elevation</li> <li>▪ Cofferdam and dewatering system are required</li> <li>▪ Scour protection is required</li> <li>▪ Not compatible for integral abutment</li> </ul>	<ul style="list-style-type: none"> <li>▪ Medium to high</li> </ul>	<ul style="list-style-type: none"> <li>▪ Susceptible to differential settlements</li> <li>▪ Excavation below groundwater and basal instability/foundation soils disturbance may be an issue</li> <li>▪ Higher scour risk</li> </ul>
<b>Deep Foundation</b>  Steel H-piles driven onto bedrock	<ul style="list-style-type: none"> <li>▪ High geotechnical resistance available</li> <li>▪ Negligible or minimum settlement</li> <li>▪ Compatible for integral and semi-integral abutment</li> <li>▪ Less excavation since the pile caps could be higher than shallow foundations</li> </ul>	<ul style="list-style-type: none"> <li>▪ High cost for mobilization of pile driving equipment</li> <li>▪ Possible obstruction due to cobbles and/or boulders and potential “hung up”</li> </ul>	<ul style="list-style-type: none"> <li>▪ Medium to high</li> </ul>	<ul style="list-style-type: none"> <li>▪ Risk of pile tip damage, should be adequately protected while driving through cobbles and boulders</li> <li>▪ Variation in pile tip elevations</li> <li>▪ Sloping bedrock</li> </ul>

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Options	Advantages	Disadvantages	Relative Costs	Risks/ Consequences
Steel tube piles driven onto bedrock	<ul style="list-style-type: none"> <li>High geotechnical resistance available</li> <li>Negligible or minimum settlement</li> <li>Less excavation since the pile caps could be higher than shallow foundations</li> </ul>	<ul style="list-style-type: none"> <li>Greater risk of encountering obstruction and “hung up”</li> </ul>	<ul style="list-style-type: none"> <li>Medium to high</li> </ul>	<ul style="list-style-type: none"> <li>Risk of pile tip damage</li> <li>Sloping bedrock</li> </ul>
Caissons socketed into bedrock	<ul style="list-style-type: none"> <li>Can transmit very large axial and lateral loads</li> <li>Negligible or minimum settlement</li> <li>Requires fewer elements than driven piles</li> </ul>	<ul style="list-style-type: none"> <li>Temporary or permanent liner is required</li> <li>Tremie concrete required</li> <li>Not suitable for integral bridge abutment</li> </ul>	<ul style="list-style-type: none"> <li>Medium to high</li> </ul>	<ul style="list-style-type: none"> <li>Risk of cave-in for unsupported excavation, especially below groundwater table</li> <li>Difficulty to penetrate through cobbles and boulder layer</li> <li>Sloping bedrock</li> <li>Due to sloping bedrock sealing below the bedrock surface could be difficult (cleaning of the bottom is difficult)</li> </ul>

Based on comparison of the above foundation options, the preferred option from a geotechnical/foundations perspective is to support the abutments for the proposed bridge with deep foundation steel H-piles or caissons.

## 2.2.2 Shallow Foundations

Shallow foundations (i.e. spread footing) can be founded on the native compact to very dense sand to gravelly sand or on an engineered granular pad. Any existing fill, organic silts and loose soils have to be removed below the footprint of the foundation/granular pad, including the layers of peat and organic silt encountered at the locations of BH20-1 and BH18-1 (west abutment).

### 2.2.2.1 Footing Elevation

Based on the results of the geotechnical investigation and a requirement for adequate protection against frost penetration in the project area (i.e. a minimum 2.6 m below the lowest surrounding area, see Section 2.6.4 ), the following founding elevations of spread footings are recommended:



Table 2.2. Recommendations for footing depth for the proposed new bridge

Structure Unit	Material at Founding Level	Foundation Elevation <sup>1</sup> (m)	Foundation Depth Below Existing Grade
West Abutment	Min 2 m thick engineered granular pad over compact gravelly sand/cobbles and boulders	244.0	2.6 m (3.5 m to 4.7 m excavation of existing fill, peat and loose organic silt)
East Abutment	Compact to very dense sand/dense to very dense gravelly sand	245.0	2.6 m (2.5 m excavation of existing fill, peat and loose organic silt)

Note:

1. Below the frost depth of 2.6 m

## 2.2.2.2 Geotechnical Resistances

Spread footings placed on the properly prepared subgrade at the design level given in Table 2.2, should be designed based on the factored geotechnical resistances at ULS and factored serviceability geotechnical resistances (for 25 mm of settlement) given in Table 2.3 below. The footing width of 4.5 m is assumed.

Table 2.3. Factored geotechnical resistance at ULS and factored serviceability geotechnical resistance for a 4.5 m wide footing for the proposed bridge

Structure Unit	Soil at Founding Level	Width of Footing (m)	Factored Geotechnical Resistance at ULS (kPa)	Factored Serviceability Geotechnical Resistance (for 25 mm settlement) (kPa)
<b>West Abutment</b>	Engineered granular pad over compact gravelly sand/cobbles and boulders	4.5	525	320
<b>East Abutment</b>	Compact to very dense sand/dense to very dense gravelly sand	4.5	525	320

Since the ULS resistance and the settlement depend on the footing size and depth of embedment, the geotechnical resistances given in Table 2.3 should be reviewed if the selected footing width or founding elevations differ from those given in the table. Similarly, if an inclined load is applied instead of a vertical load, which is used in these calculations, the values given in Table 2.3 must be reviewed to take into account those inclinations.

Prior to placing footings, the exposed native subgrade should be inspected according with OPSS 902. A Qualified Geotechnical Engineer should check that the design foundation elevation is achieved and all unsuitable soils including fill, organics and those soils with the USCS classification of CH, OH, MH, OL or PT have been removed. It should be also checked that the entire footing is placed on the competent foundation soil. Dewatering will be required during construction of spread footings, as discussed in Section 2.6.3.

### 2.2.2.3 Resistance of Footing to Lateral Loads

Resistance to lateral forces/sliding resistance between the subgrade and concrete should be calculated in accordance with Section 6.10.5 of the CHBDC/CSA S6-19. The unfactored values of the coefficient of friction,  $\tan \delta$ , between the base of cast-in-place concrete footing and the granular subgrade soils below the frost level are presented in Table 2.4. A factor of 0.8 should be applied in calculation of the horizontal resistance in accordance with the CHBDC.

Table 2.4. Recommendations for coefficient of friction

Interface	Coefficient of Friction, $\tan \delta$
Granular pad and cast-in-place concrete	0.65
Native sand and gravel deposits and cast-in-place concrete	0.50

## 2.2.3 Deep Foundations

### 2.2.3.1 General

Based on the subsurface conditions, deep foundation options have been considered as favorable options for the proposed new bridge. A summary of the advantages and disadvantages associated with each deep foundation option is provided below, and a comparison of the alternative foundation options based on advantages, disadvantages, relative costs, risks and consequences is provided in Table 2.1 in Section 2.2.1 of this report.

**Driven Steel H-piles:** Steel H-pile foundations would be suitable for the construction of integral abutment. Steel H-piles driven through the overburden to refusal on the underlying bedrock are feasible for support of the proposed bridge. However, driving these piles at this site could be difficult due to presence of the layer with cobbles and boulders encountered in boreholes (BH20-1, BH20-4, BH20-6 and BH18-2), therefore care (i.e. pile flange reinforcement or be fitted with a driving shoe) has to be taken during installation of these piles. Piles driven to bedrock will provide high geotechnical resistances and minimize foundation settlement. Steel H-piles can be readily lengthened or cut to size, and can be relatively roughly handled during delivery with little hazard of damage. These piles have minimal disturbance to neighbouring piles or structures.

**Driven Steel Pipe (Tube) Piles:** Closed-ended steel pipe (tube) piles could also be considered as a deep foundation option to support the abutments of the proposed bridge. Similar to steel H-piles, tube (pipe) piles will provide high geotechnical resistance and minimize foundation settlement. However, considering the presence of the layer with cobbles and/or boulders encountered at this site, pipe piles will have a higher risk than H-piles for 'hanging up' or being deflected away from their vertical or battered orientation.

**Drilled Concrete Caissons:** Caissons socketed into the sound bedrock to support the abutments are also feasible for this site. Temporary or permanent liners would be required to mitigate the potential risks of ground loss from the water bearing cohesionless sand and gravel layers during construction. Since the strength and sloping nature of the rock surface, establishing a seal between the liner and the bedrock could be problematic. Tremie concrete would likely be required.

### 2.2.3.2 Steel H-Piles or Steel Pipe (Tube) Pile

Considering the site specific conditions for the proposed bridge, driven steel H-piles (HP 310 x 79 and HP 310 x 110) and closed-end concrete fill 323 mm OD x 9.5 mm wall thickness and 355 mm OD x 9.5 mm wall thickness steel H-piles (HP 310 x 79 or HP 310 x 110) could be used to support the bridge abutments.

#### 2.2.3.2.1 Geotechnical Axial Resistances of Piles

The factored geotechnical axial resistances at ULS and factored serviceability geotechnical axial reactions (at SLS) for 25 mm of displacement for the recommended driven piles are presented in Table 2.5. It is anticipated that for H-piles or pipe piles driven and seated on the underlying unyielding bedrock, the geotechnical resistance at SLS for 25 mm of settlement will be greater than the factored axial resistance at ULS; as such, ULS conditions will govern for this foundation type.

Table 2.5. Factored geotechnical resistances for considered piles for the permanent bridge

Structure Element	Pile Founding Stratum	Estimated Tip Elevation (m)	Approx. Design Pile Length <sup>1</sup> (m)	Factored Geotechnical Axial Resistance at ULS (kN/pile)				Factored Serviceability Geotechnical Axial Resistance (kN/pile)			
				HP 310 x 79	HP 310 x 110	323 mm x 9.5 mm	355 mm x 11 mm	HP 310 x 79	HP 310 x 110	323 mm x 9.5 mm	355 mm x 11 mm
West Abutment	Bedrock	~238 (south) to ~240.1 (north)	7.5 (south) to 5.8 (north)	1,600	2,000	1,600	2,000	N/A			
East Abutment	Bedrock	~239 (north) to ~237 (south)	6.5 (north) to 8.5 (south)	1,600	2,000	1,600	2,000				

*Note: N/A-not applicable since for H-piles driven and seated on the underlying unyielding bedrock, the geotechnical resistance at SLS for 25 mm of settlement will be greater than the factored axial resistance at ULS and ULS conditions will govern; (1) below the pile caps provided with a minimum 2.6 m of the soil cover for frost protection*

As can be seen in the table above, steel pipe piles can provide similar axial resistances; however, these piles are less suitable for integral abutments and are more likely to 'hang-up' during driving at levels above the desired penetration.

If integral abutments are adopted, 600 mm CSP filled with loose uniform sand in a predrilled oversized hole will be required to reduce resistance to lateral movements and reduce stress on piles. The annular space between the pre-augured oversized hole and the pile should be filled with uniformly graded sand (i.e. Ottawa type sand)

#### 2.2.3.2.2 Negative Skin Friction (Downdrag Loads) on Steel H-Piles

Since the foundation soil is cohesionless, the negative skin friction (or downdrag load) will not need to be taken into consideration during design of the H-piles.

#### 2.2.3.2.3 Steel H-Pile Installation

Steel H-piles will be driven to bedrock through overburden. Presence of the cobbles and boulders layer within overburden must be considered for the proper pile installation. In view of this, the piles should be fitted with a driving shoe section (Titus pile point due to sloping bedrock, APF Hard Bite bearing points or similar) offering some protection against buckling at the toe. The piles should be stiffened as per OPSD 3000.100, Type II to minimize damage to the piles in anticipation of heavy driving conditions. Care must be taken to avoid overdriving and damaging the pile tip (i.e., the structural capacity of the piles should not be exceeded).

In addition, all piles should be visually monitored by experienced personnel during installation to check for plumbness, set, internal damage, etc. All damaged piles should be rejected, or if the damage is considered to be minor, the pile can be tested to determine the available pile capacity.

Piles in groups should be spaced no closer than 3 pile diameters. All piles in a group should be checked for heaving during the driving of the adjacent piles.

### 2.2.3.3 Caissons

#### 2.2.3.3.1 General

Caissons socketed into the sound bedrock could be used to support the bridge abutments. Caisson construction can be classified into three broad categories which are: (1) the dry method, (2) the casing method, and (3) the wet method. At this site, using casing and the wet method of construction, sometimes referred to as the slurry-displacement method, would be appropriate.

#### 2.2.3.3.2 Geotechnical Axial Resistance for Caissons

The resistance values and base elevations in Table 2.7 are recommended for caissons of 0.9 m and 1.2 m diameter socketed into bedrock. Table below provides the factored geotechnical axial resistances for these caissons socketed

a nominal length of minimum 1.0 m into bedrock. The given values for caissons include the shaft resistance of the bedrock socket and its end bearing.

Table 2.7. Caisson geotechnical resistances

Foundation Unit	Relevant Borehole	Founding Stratum	Recommended Highest Elevation for Top of Rock Socket (m)	Approx. Caisson Length (m) <sup>1</sup>	0.9 m Dia. Caisson		1.2 m Dia. Caisson	
					Factored ULS (KN)	Factored SLS (KN) <sup>2</sup>	Factored ULS (KN)	Factored SLS (KN) <sup>2</sup>
West Abutment	BH20-1 & BH18-1	Bedrock	238 <sup>(4)</sup>	7.5	9,000	N/A <sup>(3)</sup>	12,000	N/A <sup>(3)</sup>
East Abutment	BH20-4 & BH20-5	Bedrock	239 <sup>(4)</sup>	6.5	9,000	N/A <sup>(3)</sup>	12,000	N/A <sup>(3)</sup>

Notes:

1. Below frost line.
2. For 25mm total settlement.
3. N/A-not applicable since for H-piles driven and seated on the underlying unyielding bedrock, the geotechnical resistance at SLS for 25 mm of settlement will be greater than the factored axial resistance at ULS and ULS conditions will govern the design
4. Drilling through the cobbles and boulders layer

#### 2.2.3.3.3 Negative Skin Friction (Downdrag Loads) on Caissons

Since the foundation soil is cohesionless, the negative skin friction (or downdrag load) will not need to be taken into consideration during design of the pails, as noted in Section 2.2.3.2.2.

#### 2.2.3.3.4 Caissons Installation

The socketed caisson should be installed in accordance with the latest version of OPSS 903. The rock surface at the base of each casing should be thoroughly cleaned of any cuttings and loose soils, loose rock, and debris prior to concreting.

Temporary or permanent liners may be required at this site to support the overburden during construction. Groundwater seepage should be expected into the caissons given the relatively high groundwater level and the high permeability of surrounding soil. Techniques such as tremie concreting and maintaining a head of concrete within the liner may be required to ensure satisfactory caisson construction. Since the layer of cobbles and boulders above the bedrock is present at the site, design and construction of the caissons should take into account the presence of this layer as well as socketing in strong rock.

To determine the structural quality and integrity, non-destructive evaluation tests which may include (a) downhole tests conducted in access tubes, including crosshole acoustic tests and backscatter gamma ray (gamma-gamma) test, (b) sonic echo tests, or (c) TIP-Thermal Integrity Profile and PIT-Pile Integrity Testing can be used.

The recommended resistances are contingent on satisfactory cleaning of bases for which the measures have been described below. It is recognized that under these conditions there may be cases where the base condition may be difficult to verify.

Human entry into the caissons to hand clean the caisson bases is not generally allowed in the Province of Ontario. One or more of the following inspection procedures may be used to inspect the cleanliness of the caisson excavation bottom prior to placement of the reinforcement steel and concrete.

**Steel Probe:** Lower the steel probe to the bottom of the caisson excavation to check that cleaning has been satisfactorily completed. A steel probe at 0.6 m long with a flat tip on the sounding end and a weight of approximately four kilograms (#32 rebar) is suspended from the opposite end with a non-stretch cable.

**Shaft Inspection Device (SID):** The engineer may use the SID to take sediment measurements and observe the bottom conditions of the caisson excavation at a minimum of five locations selected by the engineer. The SID is a remotely operated camera capable of observing bottom conditions and measuring sediment underwater and slurry. Each SID inspection (including all 5 locations) takes approximately 1 hour after the equipment has been set up.

**Shaft Quantitative Inspection Device (SQUID):** The SQUID system is a new technology for quantitatively assessing the quality of the bottom surface of a bored pile or drilled shaft. It measures both the thickness of soft material or debris that may be covering the bearing strata, providing a force and displacement in numerical and graphical form. The SQUID test consists of mounting the device on a Kelly bar or winch system and lowering it into a drilled hole. Once the SQUID is located at the bottom of the hole, the buoyant weight of the Kelly bar will transfer sufficient force for the probes to penetrate the debris and bearing layers, and for the displacement plates to retract. Accurate, real time force vs displacement measurements are plotted and displayed digitally in the SQUID tablet.

#### 2.2.3.4 Resistance of Piles to Lateral Loads

The resistance to the lateral load will have to be derived from the soil in front of the vertical piles/caissons. The resistance to lateral load in front of a vertical pile/caissons may be calculated using subgrade reaction theory (Broms' Method) where the coefficient of lateral subgrade reaction,  $K_{py}$  (kPa/m), is based on the following equations (Terzaghi, 1955 and Davisson, 1970):

For cohesionless soils:

$$K_h = n_h(z/d)$$

For cohesive soils:

$$K_h = 67C_u/d$$

where,

$K_h$ =coefficient of horizontal subgrade reactions (kPa/m)  
 $d$ =pile diameter/ width (m)  
 $n_h$ =constant of horizontal subgrade reaction (kPa/m)  
 $C_u$ =undrained shear strength (kPa)

$z$ =depth below ground surface (m)

The following Table 2.8 presents the estimated soil properties and their coefficients of subgrade reaction on the location of west and east abutments. The data presented in the table can be used for lateral load analyses using the L-pile software.

The notations (other than those explained above) used in the table are defined below:

$N_{SPT}$	Standard Penetration Test, N-value
$\gamma$	bulk unit weight ( $kN/m^3$ )
$\phi$	internal friction angle (deg)
$\delta$	friction angle between steel pile and soils (deg)
$\epsilon_{50}$	strain corresponding to 50% of the maximum principal stress difference
$K_p$	coefficient of passive earth pressure

Group action for lateral loading should be considered by Reese method using reduction factors on the single pile capacity depending on the geometry of the pile layout.

The reduction factors are as follows:

1. Reduction factors for the piles in a row

$$e = 1 \text{ for } s/b \geq 3.75$$

$$e = 0.64 (s/b)^{0.34} \text{ for } 1 \leq s/b < 3.75$$

2. Reduction factors for leading piles in a line

$$e = 1 \text{ for } s/b \geq 4.0$$

$$e = 0.7 (s/b)^{0.26} \text{ for } 1 \leq s/b < 4.0$$

3. Reduction factors for trailing piles in a line

$$e = 1 \text{ for } s/b \geq 7.0$$

$$e = 0.48 (s/b)^{0.38} \text{ for } 1 \leq s/b < 7.0$$

Table 2.8. Parameters for lateral load analyses

Strata	Elevation (m)	$N_{SPT}$	$\gamma$ ( $kN/m^3$ )	$\phi$ ( $^\circ$ )	$c_u$ (kPa)	$\delta$ ( $^\circ$ )	$K_{py}$ ( $MN/m^3$ )		$\epsilon_{50}$	$n_h$ ( $MN/m^3$ )	$K_p$
							Static	Cyclic			
Engineered Fill	-	-	21.0	32	-	14	10	10		6.6	3.0
West Abutment (BH20-1)											
Cobbles and Boulders (very dense)	244.0 – 242.3	>100	20	35	-	20	40	40	-	14.5	3.69
Gravelly Sand	242.3 –	56 -	22	32	-	20	34	34	-	7.5	3.25

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Strata	Elevation (m)	N <sub>SPT</sub>	$\gamma$ (kN/m <sup>3</sup> )	$\phi$ (°)	c <sub>u</sub> (kPa)	$\delta$ (°)	K <sub>py</sub> (MN/m <sup>3</sup> )		$\epsilon_{50}$	n <sub>h</sub> (MN/m <sup>3</sup> )	K <sub>p</sub>
							Static	Cyclic			
(very dense)	238.0	>100									
East Abutment (BH20-4 & BH20-5)											
Silty Sand to Gravel Fill (loose)	247.5 – 246.0	3 - 9	20	30	-	14	5	5	-	1.5	3.0
Silt to Silty Sand (compact)	246.0 – 245.2	7 - 15	20	32	-	20	10	10	-	6.6	3.25
Sand to Gravelly Sand (dense to very dense)	245.2 – 237.3	15 - 100	22	35	-	20	34	34	-	7.5	3.69

Lateral loading could be resisted fully or partially by use of battered piles. The piles could be installed at a batter of up to 4 vertical to 1 horizontal by simply tilting the pile-driver leads.

### 2.2.3.5 Frost Protection

The pile caps should be provided with the minimum soil cover for frost protection as mention in Section 2.6.4 following below.

## 2.3 Approach Embankments

### 2.3.1 General

Based on the provided GA drawing, the proposed grade for the west and east abutments of the new bridge will be raised by approximately 1.2 m (Elev. 252.9 m) and 1.4 m (Elev. 252.2 m) above the existing bridge, respectively. This results in construction of approach embankments of up to 5.4 m and 4.7 m height with respect to original ground level at the west and east abutment locations, respectively. The GA drawing also suggests that the approach embankments could be designed to be constructed of rockfill with 1.5H:1V side slopes. Providing that specifications of OPSS 1010 are met, fill for construction of new embankment could also consist of Selected Subgrade Materials (SSM) or Granular B Type I or Type II with the side slopes not steeper than 2H:1V. However, the following stability and settlement analyses were performed assuming that the rockfill embankment will be constructed at the site.

### 2.3.2 Stability Considerations

To assess the static and seismic slope stability of the forward slopes of abutments and embankments of the proposed new bridge, the SLOPE/W computer program developed by GeoSlope International Ltd. was employed for computation. Factors of safety were calculated using the Morgenstern-Price method for critical failure surfaces. The required minimum Factor of Safety (FOS) of 1.5 and 1.3 were adopted as the design criteria for abutments and embankments in static conditions, respectively. The minimum factor of safety of 1.1 was adopted for seismic conditions.

The geometry of abutments and embankments at the site used in the analyses was adopted from the provided preliminary GA drawing (i.e. embankment - rockfill with 1.5H:1V slopes), while the soil stratigraphy was defined based on the data obtained during this investigation. Given the subsurface conditions at the site (i.e. non-cohesive soils), effective stress analyses for a long term stability assessment were performed taking into consideration the



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subsoil conditions encountered directly beneath and adjacent the proposed bridge. Peat and organic soils under the abutment and embankment were assumed to be removed completely.

In addition, a traffic surcharge pressure of 12 kPa was adopted in the slope stability assessments for the abutments and approach embankment.

Tabulated below in Table 2.9 are the soil parameters used for the slope stability analyses for the proposed bridge. The soil parameters were generally estimated based on the results of field and laboratory investigation.

Table 2.9. Soil properties used in slope stability analyses

Material Type	Effective Stress Parameters		
	$\phi'$ (degrees)	$c'$ (kPa)	$\gamma$ (kN/m <sup>3</sup> )
Rockfill	42	0	18
Organic Silt (Loose to Compact)	22	0	18
Silt (Loose to Dense)	30	0	20
Sand (Dense to Very Dense)	32	0	22
Sandy Gravel (Dense to Compact)	31	0	21
Peat	18	0	12
Organic Silt (Loose)	18	0	17
Cobbles and Boulder	35	0	20
Gravelly Sand (Very Dense)	32	0	22

Table 2.10. Summary of results of slope stability analyses

Locations	Max Height (m)	Conditions	Min FOS
West Abutment (Global stability)	~5.4	Drained long-term conditions, static condition	2.9
		Drained long-term conditions, seismic condition	2.0
West Approach Embankment (Side slopes 1.5H:1V)	~5.4	Drained long-term conditions, static condition	1.8
		Drained long-term conditions, seismic condition	1.4

The results of the slope stability analyses for the proposed bridge supported on piles and 5.4 m high west rockfill embankment having 1.5H:1V slopes, as shown on the preliminary GA drawing, are presented on Figures E1 to E4 in Appendix E and summarized in Table 2.10. As can be seen the calculated minimum factors of safety of critical slip surfaces meet the design criteria for static and seismic conditions given above. Therefore, based on these results, the proposed abutment/rockfill embankments can safely be constructed with 1.5H:1V side slopes. The stability of rockfill embankments constructed with the slope of 1.25H:1V, as recommended OPSD 202.010, was also analyzed and results showed that the rockfill embankment with 1.25H:1V slopes can be safely constructed on this site as well.

### 2.3.3 Settlement Considerations

#### 2.3.3.1 Settlement of Foundation Soils

A computer program, Settle3D (Rocscience) was employed for settlement calculation of foundation soils due to construction of new embankment sections at the site. Settle3D is a 3-dimensional program for the analysis of immediate and consolidation settlement under foundations, embankments and surface loads. The program combines the simplicity of one-dimensional analysis with the power and visualization capabilities of more sophisticated three-dimensional programs.

The magnitudes of total settlement for the approach embankments has been assessed based on Standard Penetration Test (SPT) results and elastic modulus which is correlated from the available data adopted in the settlement analyses for the west and east approach embankments is summarized in Table 2.11.

Table 2.11. Soil strength parameters for settlement assessment

Soil Layers	Unit Weight (kN/m <sup>3</sup> )	E (MPa)	Compression Index (Cc)	Recompression Index (Cr)	Void Ratio (e)	Preconsolidation Pressure (p' <sub>c</sub> ) (kPa)
West Approach Embankment						
Rockfill	18	90	-	-	-	-
Silt (Compact)	20	25	-	-	-	-
Sandy Gravel (Dense to Compact)	20	55	-	-	-	-
Sand (Very Dense)	22	65	-	-	-	-
East Approach Embankment						
Rockfill	18	90	-	-	-	-
Clayey Silt Fill	17	-	0.18	0.02	0.55	180
Silt (Compact)	20	25	-	-	-	-
Gravelly Sand to Sand (Dense to Very Dense)	21	65	-	-	-	-

The summary of results of settlement analyses for the 5.4 m high west and 4.7 m high east approach embankments is given in Table 2.12. The Settle 3D results of these cases can be seen Appendix F.

Table 2.12. Results of settlement analyses

Locations	Abutment Height (m)	Assumed Embankment Width (m)	Calculated Immediate Settlement (mm)	Calculated Consolidation Settlement (mm)	Calculated Total Settlement (mm)
West Abutment	5.4	10	23.9	0	23.9
East Abutment	4.7	10	14.1	17.4	31.5

The estimated total settlement under the embankments could be about 24 mm and 32 mm at the west approach embankment and east approach embankment, respectively. Almost all of this settlement is expected to occur during and immediately following construction of approach embankments.

Based on the post- construction settlement criteria for embankment widening stipulated in MTO's "Embankment Settlement Criteria for Design", the maximum settlement limits during pavement design life of the widened embankment are 50 mm of the total settlement and 200:1 of the differential settlement rate. The differential settlement rate is applicable to both new widened embankment and, also, the differential settlement rate between the existing and the new embankment. The settlement across the widened embankment shall transition uniformly from the widening point (existing highway embankment rounding) to the new embankment rounding such that surface drainage is not impeded. The maximum settlement at structure/ embankment interface during pavement design life should be 25 mm for distance of 0 - 20 m from transition point. Considering estimated settlement amounts, it is expected that these post-construction settlement criteria will be generally met. However, to mitigate the risk further and ensure the long term performance, it is recommended to preload the embankment for 3 months.

### 2.3.3.2 Settlement of Embankment Fill

The fill is also expected to experience some settlement. It is estimated that the embankment itself will compress by about 0.5 to 1.0 percent of the embankment height under its self-weight, depending on material type and assuming placement as per MTO practices. More granular material fills would compress less and over a shorter time period, typically within the period of embankment construction. If non- granular earth fills are used, some additional settlement over time might be exhibited. To minimize the post construction settlement, the non-granular earth fill materials should be compacted to 98% Standard Proctor Maximum Dry Density. Some differential settlements can be expected at the structure/embankment interface, but these movements should be able to be accommodated during the construction process ranging from 2 to 4 months depending on the nature of embankment fill employed. As stated above, where the granular fill is used, the required delay will be less.

## 2.4 Seismic Potential Consideration

Seismic characterization of the site should be compliant with the Canadian Highway Bridge Design Code (CHBDC, CAN/CSA-S6-19). Table 4.1 in CHBDC (see Clause 4.4.3.2) shows site classification for seismic site response based on average soil properties in the top 30 m.

At this site, the subsoil generally consists of fill underlain by silt/sand/gravelly sand/cobbles and boulders layers underlain by bedrock. The groundwater level varies with the river water level (Elev. 245.6 m in Aug 2020, corresponding depth approximately 2.6 m below the ground surface). The top of bedrock is located between Elev. 237.3 m and 240.1 m, corresponding to depths between 8.6 m and 10.5 m. Based on soil characteristics, the site class for this site is estimated to be Class “C” according to Table 4.1 of the CHBDC .

From the Natural Resources Canada website, 2015 NBCC seismic hazard values are obtained using the site location coordinates and the damped reference spectral accelerations for the project site are shown in Table 2.13 below:

Table 2.13. Seismic design values for footings

Probability of Exceedance in 50 Years (Return Period)	Sa(0.2) (g)	Sa(0.5) (g)	Sa(1.0) (g)	Sa(2.0) (g)	PGA (g)
(Latitude: 49.188917; Longitude: -81.438255)					
2%	0.209	0.101	0.049	0.022	0.136
5%	0.104	0.055	0.028	0.013	0.066
10%	0.056	0.033	0.017	0.007	0.033

These values are associated with an earthquake having a 2 percent, 5 percent, and 10 percent probability of exceedance in a 50-year period.

## 2.5 Lateral Earth Pressure on Structures – Abutments and Wing Walls

### 2.5.1 Static Earth Pressure

The following recommendations are provided concerning the design of the abutments and wing walls, if any, in accordance with the CHBDC (2019). It should be noted that these design recommendations and parameters assume level backfill and ground surface behind the walls. Where there is sloping ground behind the wall, the coefficient of lateral earth pressure must be adjusted to account for the slope. The lateral pressures acting on the abutment stems will depend on the type and method of placement of the backfill materials, on the nature of the soil behind the backfill, on the magnitude of surcharge including construction loadings, on the freedom of lateral movement of the structure, and on the drainage conditions behind the walls.

A compaction surcharge equal to at least 12 kPa should be included in the lateral earth pressures for the structural design in accordance with CHBDC Figure 6.6.

The mobilization of full active or passive resistance requires a measurable and perhaps significant wall movement or rotation. For active earth pressure, a rotation of 0.002 about the base of vertical walls (horizontal displacement divided by wall height) or translation of 0.001 times wall height or combination of these is required.

If the wall support allows lateral and/or rotational yielding (unrestrained structure, such as typically the case for retaining walls), active earth pressures may be used in the geotechnical design of the structure. The granular fill should be placed in a wedged shaped zone (with a width equal to frost depth at the ground level in front of the wall) against a cut slope which begins at the footing level and extends upwards at a maximum inclination of 1 horizontal to 1 vertical (Case (b) from commentary on CHBDC Figure C6.31).

For walls backfilled using free draining granular materials in accordance with Case (b), the parameters (unfactored) given in Table 2.14 may be assumed.

Table 2.14. Material types and unfactored earth pressure properties under static conditions

Material	Unfactored Friction Angle $\phi' (^\circ)$	Coefficient of Active Earth Pressure ( $K_a$ )	Coefficient of Passive Earth Pressure ( $K_p$ )	Coefficient of Earth Pressure at Rest ( $K_o$ )	Unit Weight $\gamma$ (kN/m <sup>3</sup> )
Compacted Granular A or Granular B Type II	35	0.27	3.69	0.43	22
Compacted Granular B Type I	32	0.31	3.25	0.47	21
Existing granular fill	32	0.31	3.25	0.47	21

Note:

Assumes long term conditions. In short term conditions  $K_a = K_p = 1$

The coefficients of lateral earth pressure above are provided for level backfill behind the wall (perpendicular to the wall face plane) and should be adjusted in the case of sloping backfill. For a 2 horizontal to 1 vertical (2H:1V) slope, the active earth pressure coefficients provided above should be adjusted by a factor of 1.5. The given values of active earth pressure coefficients depend on angles of friction and inclination. For preliminary design purposes, the adjustment for slopes between horizontal and 2H:1V may be linearly proportioned, however, some modification of the design pressures may be required depending on the backfill type and geometry. The coefficient of at-rest earth pressure for sloping granular backfill can be calculated using the equation:

$$K_o = (1 - \sin\phi')(1 + \sin\beta)$$

Where  $\beta$  is angle of sloping backfill above the horizontal.

## 2.5.2 Lateral Earth Pressure for Seismic Design

### 2.5.2.1 Walls Capable of Moving 25 to 50mm

Seismic loading should be taken into account in the design of the bridge in accordance with Section 4.6.5 of the CHBDC. These estimates are based on the Mononobe-Okabe (M-O) pseudo-static method of analysis. The M-O method produces seismic loads that are more critical than the static loads that act prior to an earthquake. In accordance with Section 4.6.5 and C.4.6.5 of the CHBDC and its Commentary, respectively, for structures which allow lateral yielding, a design seismic coefficient of  $1/2 K_h$  in the M-O analysis would be adequate for pseudo-static design of walls, provided that the wall is able to move a distance equal to  $250 \times K_h$  (mm); where  $K_h$  is the site-adjusted horizontal peak acceleration coefficient at the base of the wall in units of "g". The effect of vertical accelerations should be ignored when calculating the seismic lateral earth pressures.

When calculating seismic lateral earth pressures on walls that are capable of moving 25 to 50 mm using the M-O formulation, the seismic horizontal acceleration coefficient ( $k_h$ ) should be taken as half of  $k_{h0}$ , where,  $k_{h0}$  is the seismic horizontal acceleration coefficient that corresponds to zero wall movement. The site-adjusted PGA estimated at ground surface is given as  $F(\text{PGA}) \bullet \text{PGA}$ , where,  $F(\text{PGA})$  is the PGA-based amplification factor that corresponds to the applicable Site Class as defined in Table 4.8 of the Code.

The earthquake-induced dynamic pressure distribution, which is to be added to the static earth pressure distribution, is a linear distribution with maximum pressure at the top of the wall and minimum pressure at the its toe. The total pressure distribution including static and seismic may be determined per Section C4.6.5 of the Commentary to CHBDC.

In accordance with Section 4.4.3.2 of the CHBDC, a PGA of 0.136 g for the Site Class 'C' and earthquake having a 2% probability of exceedance in 50 years (0.0021 per annum) can be used in the calculation of the seismic active pressure coefficient.

It should be noted that in the computation of seismic earth pressure coefficients, the wall back-face geometry, backfill slope and wall friction effects need to be addressed.

For design purposes, the following unfactored seismic lateral earth pressure parameters can be used (assuming wall friction is neglected, the back wall is vertical, and the ground surface is horizontal both on the retained side as well as in front of the toe):

Table 2.15. Material types and earth pressure properties under seismic conditions

Material	Unfactored Friction Angle $\phi'$ (o)	Coefficient of Seismic Earth Pressure – Active (K <sub>ae</sub> )	Coefficient of Seismic Earth Pressure – Passive (K <sub>pe</sub> )	Unit Weight $\gamma$ (kN/m <sup>3</sup> )
Compacted Granular A or Granular B Type II	35	0.33	6.2 (ULS)	22
Compacted Granular B Type I	32	0.37	5.3 (ULS)	21

### 2.5.2.2 Non-yielding Walls

For walls that are restrained against lateral movement, the seismic lateral earth pressures should be obtained using the M-O formulation and using a seismic horizontal acceleration coefficient ( $k_h$ ) equal to  $k_{h0}$ , where,  $k_{h0}$  is the seismic horizontal acceleration coefficient that corresponds to zero wall movement. It is the site-adjusted PGA estimated at the ground surface, given as  $F(\text{PGA}) \cdot \text{PGA}$ . The acceleration coefficient determined at the original ground surface should be considered to be the acceleration coefficient acting at the wall base. The seismic vertical acceleration coefficient ( $k_v$ ) should be ignored.

## 2.6 Construction Considerations

### 2.6.1 Excavation

All excavations must be carried out in accordance with the latest edition of the Ontario Occupational Health and Safety (OHSA) and good construction practice. The existing fill and native soils which should be excavated for construction of the abutments (i.e. very loose to loose peat, loose organic silt, loose to compact sand and silt, dense to very dense gravelly sand to sand) are considered as Type 3 soils above the groundwater table and Type 4 soils below the groundwater table. Temporary excavations (i.e. those that are open only for a short period) above the groundwater table may be made with side slopes not steeper than about 1H:1V, while the temporary slopes below the groundwater table have to be formed at 3H:1V unless a suitable dewatering system is installed to lower the water level below the base of the excavation.

### 2.6.2 Temporary Shoring and Protection Systems

Temporary excavation support systems are likely required to facilitate the construction of new abutments and wing walls, if any, in order to maintain traffic on the existing bridge, limit environmental impact on the surroundings, and to reduce the quantity of sub-excavation required for the project. The temporary support systems should be designed and constructed by a Contractor in accordance with OPSS.PROV 539 as amended by SP105S09 and using the factual data presented in Section 1 of this report. The lateral movement of the temporary shoring system should meet Performance Level 2 as specified in OPSS.PROV 539, provided that the existing, adjacent structures can tolerate this magnitude of deformation or re-routed away from excavation influence zone.

To safely support the excavation walls and minimize the impact to existing structures, temporary shoring consisting

of driven steel sheet piling or soldier H-piles with lagging should be practical options at this location. It is considered that a sheet pile with a sufficiently robust cross section could be driven through the existing fill and natural deposits at these sites. Difficulties with installation may occur where cobbles and boulders are encountered in the soil, requiring their removal before further driving. Alternatively, an H-pile with a lagging wall can be used as a vertical temporary shoring system. The H-piles are installed, and lagging is inserted between installed H-piles during excavation. Space between the excavation and lagging must be suitably backfilled and drained. Lagging wall material can be selected as wood (timber), steel or concrete. For the relatively shallow depth of excavation anticipated, cantilevered systems may be adequate. However, depending on the actual excavation depth, embedment depth and shoring system used, additional anchorage or tiebacks may be required.

The sloping bedrock encountered at the site should be considered during design and construction of protection systems. The Contract Drawings should reflect these constraints.

### 2.6.3 Dewatering

Dewatering shall be carried out in accordance with OPSS.PROV 517 and SP517F01. It is the responsibility of the Contractor to propose a suitable dewatering system based on the time of construction, water levels and flow conditions in the river. In accordance with SP 517F01, the dewatering systems may be completed by a design Engineer and design-checking Engineer with a minimum of 5 years of experience. Surface water should be directed away from the excavation area(s) at all times.

It is expected that the groundwater level at the site will be at the similar level as the river water level (i.e. estimated to oscillate between Elev. 244 m and 247 m). The soils encountered within potential excavation depths consist of sand/gravelly sand soils which hydraulic conductivity has been measured to be between  $1.11 \times 10^{-5}$  m/s and  $1.69 \times 10^{-5}$  m/s. Temporary excavations for the construction footings is recommended to be up to Elev. 244 m, which could be up to 3 m below the groundwater level, depending on the water level in the river. In this case, given the conditions at this site, it is anticipated that active dewatering will be required at the site to allow for construction in dry.

Temporary cofferdams will likely be required at both abutments to envelop the construction site and control river and ground water influx during foundation construction. Considering the ground condition at the site encountered in the boreholes drilled close to the river, suitably designed steel sheetpile or rockfill dam with a impervious water barrier, might be considered. Given the groundwater conditions and soils present, piping of the soil is anticipated to be a potential issue at the site. Piping should be controlled by lowering the water table outside the cofferdam or driving the sheeting to sufficient depth to mitigate against piping. In case of rockfill cofferdam, piping can be control by installing clay cutoff trench, slurry trench or impervious blanket at upstream of cofferdam.

Design and construction specification for the chosen temporary cofferdam system should be prepared in accordance with OPSS 539 (Construction Specification for Temporary Protection Systems) and designed to accommodate a Performance Level 2. The proposed sheetpile cofferdam should be at least one meter above the designed HWL defined by the Hydraulics Engineer. The cofferdam should be designed based on the geotechnical data given in Section 1. It must be designed to withstand the anticipated design loads and to be watertight as practically possible.

It is expected that cobbles and boulders will be encountered in the deposits, therefore care must to be taken during installation of sheet piles. A Non-Standard Special Provision (NSSP) should be included in the Contract to alert the contractor the cobbles and boulders may present within the river deposit in the selection of the appropriate



equipment and procedures for piling. As noted, before, the sloping bedrock should be considered during design and construction of cofferdam as well as for other protection systems.

If drilled piles are adopted, balancing the groundwater pressures during construction by utilizing a head of the water or bentonite drilling slurry inside the temporary liner will be required.

#### 2.6.4 Frost Protection

Ontario Provincial Standard Drawing (OPSD) 3090.100 indicates that the frost penetration for the Cochrane area is 2.6 m. Therefore, all foundation elements should be provided with a minimum of 2.6 m of earth cover or equivalent approved insulation for frost protection. Equivalent protection could be provided by using polystyrene as suggested by the "Canadian Foundation Engineering Manual 2006, Section 13.5.2. page 196". It is usually accepted that 25 mm of polystyrene provides a protection which is equivalent to 600 mm of soil.

#### 2.6.5 Subgrade Preparation and Embankment Construction

Prior to construction of the new approach embankments it is recommended that any loosened/softened fill and topsoil/organic soils be removed from the footprint of the approach embankments and replaced with clean and compactible soils with minimum 98% of Standard Proctor Maximum Dry Density (SPMDD) to avoid any fill post-construction settlement. Providing that specifications of OPSS 1010 are met, fill for construction of new embankment could consist of Selected Subgrade Materials (SSM) or Granular B Type I or Type II with the side slopes not steeper than 2H:1V. As suggested in the preliminary GA the embankment fill could also consist of rockfill with side slopes not steeper than 1.25H:1V. Construction should be in accordance with OPSS 206. For rockfill embankment the layers should not exceed 1.5 m thickness prior to compaction. Material in each layer should be fully compacted prior to the succeeding layer is placed. Each rockfill layer should be compacted with a tractor bulldozer with a minimum number of complete passes of 6 and the maximum passes of 8. A complete pass should be defined as 100% coverage of layer surface. Before placing any granular fill over the rockfill, proper chinking should be applied. Alternatively, a suitably robust geotextile can be placed for separation purposes.

The final lift of fill prior to placement of the roadway granular subbase and base courses should be compacted to 100% of SPMDD. At the locations where the new embankment will be close to the existing embankment (i.e. widening) the embankment fill should be placed in accordance with OPSD 208.010 and OPSS 206.

Quality assurance should be provided as per MTO standard 501.08 (OPSS 501). Inspection and field density should be carried out by qualified personnel during placement operations to ensure that appropriate materials are used and that adequate levels of compaction have been achieved.

If the earth fill is used for embankment, the slopes of the new embankment should be provided with adequate erosion protection against surface water runoff. To reduce surface erosion on the embankment side slopes, prompt seed and cover (OPSS 804) or sodding (OPSS 803) should be carried out as soon as possible after construction of the embankment.

Considering the findings at the site, the anticipated stripping/excavation depths/elevations at the borehole locations are as follows:

Table 2.17. Recommended stripping/excavation depths/elevations at borehole locations

Borehole No.	Existing Ground Elevation at Borehole Location (m)	Recommended Stripping (Excavation) Depth/ Elevation (m)
<b>West Side</b>		
BH20-1	247.5	3.6/244.0
BH20-2	247.9	1.5/246.4
BH20-3	251.8	0.1/251.7
BH18-1	248.7	4.6/244.1
<b>East Side</b>		
BH20-4	247.8	0.8/247.0
BH20-5	247.5	0.5/247.0
BH20-6	248.0	0.1/249.9

After stripping/excavation, the exposed subgrade should be inspected, approved and properly compacted (i.e. proof rolled) from the surface, using a heavy compactor. The groundwater table should be lowered to at least 0.5 m in below the subgrade level, before any proof rolling and the application of significant compaction effort.

According to the current and previous investigations, layers of peat and organic silt was encountered at the west side of the river in BH20-1, BH20-2 and BH18-1. The thickness of these organic deposits is 1.4 m in BH20-2 (at the ground surface), 3.0 m in BH20-1 (below of 0.6 m thick fill layer) and 3.1 m in BH18-1 (below 1.5 m thick fill layer). These deposits were not encountered in BH20-3. It is recommended that these organic deposits be excavated in accordance with OPSD 203.030, attached in Appendix J. The following procedure is recommended:

1. The top layer of fill and underlying organic deposits shall be excavated along the whole footprint of the new embankment in accordance with OPSD 202.030.
2. The excavation shall be backfilled by granular material, such as Granular A or Granular B Type II, or rockfill as soon as possible.
3. The removal of organic deposits within the proposed footprint and backfilling of the excavation shall be carried out simultaneously in accordance with OPSS 209.
4. These simultaneous excavation and backfilling operations shall be done in sections parallel to the existing highway not longer than 3 m.
5. Temporary excavation side slopes through organic deposits shall be in accordance to OPSD 203.030 and the latest edition of Ontario Occupational Health and Safety Act and Regulations for Construction Projects. The

existing fill and native soils above the groundwater level would be classified as Type 3 soil, while below the groundwater they would be classified as Type 4 soil.

6. The construction of embankment using granular material or rockfill should be in accordance with OPSS 206 as noted before.

### 2.6.6 Corrosion Protection

One (1) soil sample from BH20-3 was selected for chemical analysis during the current investigation performed by EXP. The testing was completed to determine the potential degradation of the concrete in the presence of soluble sulphates and the potential of corrosion of exposed steel used in foundations and buried infrastructure. The analyses results are summarized in Table 1.6.

The pH, resistivity and chloride concentration provide an indication of the degree of corrosiveness of the sub-surface environment. In general, the soil pH value at the site measured 8.92 is slightly higher than the limit what is considered the normal range of soil pH of 7.5 to 8.5. The chemical data indicates very low (<2000 ohm-cm) resistivity of the tested soil, which suggests severe potential for corrosion of buried metallic elements. Therefore, some level of corrosion protection for buried metallic elements is required, depending upon the material type. However, coating of steel H Piles is not done in general practice. It is up to the designer to determine the requirements of appropriate protective coating measures to ensure that all aspects of CHBDC 2019, Section 2 "Durability" requirements are followed. The test results provided in Table 1.6 may be used to aid in the selection of coatings and corrosion protection systems for buried steel objects. Based on the results of sample tested, and given that the structure is located adjacent to the roadway and will be exposed to de-icing salt, consideration should be given by the designer to designing for a « C » type of exposure class as defined by CSA A23.1 Table 1.

The maximum water-soluble sulphate content of the soil tested is 13 ppm ( $\mu\text{g/g}$ ), i.e. 0.0013%, and being less than 0.10%, does not require sulphate resistant cement.

### 2.6.7 Scour Protection

The scour design is the responsibility of a qualified hydraulic engineer. Foundation recommendations outlined in this report assume that proper scour protection is used.

Structures closed to the river which contain spread footings founded on highly erodible/scourable soils (sand, silt, or fine gravel) are very vulnerable to failure caused by scour and undermining by water flow, and should not be used without appropriate protection. Spread footings can be protected against structural undermining by locating the foundations at an appropriate depth by providing scour protection blankets or by using sheet piling. Sheet piling used for this purpose should have sufficient stiffness and strength to maintain the bearing capacity of the soil within and on the outside of the sheet piling. Scour protection must be maintained effective for the design life of the bridge.

Geotechnical soil parameters necessary for the scour analyses are: SPT N-value, in-situ moisture content, percent passing the No. 200 sieve (% 200), and mean grain size diameter ( $d_{50}$ ). These parameters are determined based on the soils encountered at the site, and are presented on the borehole logs attached in Appendix C and the graphs included in Appendix D. All tested soils were classified using the Unified Soil Classification System which can be used for evaluation of erosion rates.

### 2.6.8 Winter Condition

In the event of construction during freezing temperatures, the foundation stratum should be protected from freezing by the use of loose straw, tarpaulins, propane heaters or other suitable means. In this regard, the base of the excavation should be insulated from sub-zero temperatures immediately upon exposure and until such time the footings are protected with sufficient soil cover to prevent freezing at the foundation level.

### 2.6.9 Obstructions

Cobbles and boulders were noted to be contained in overburden, therefore care (i.e. pile flange reinforcement or be fitted with a driving shoe) has to be taken during installation of piles. These potential obstructions may also impact excavations and/or elements of temporary protection systems. It is recommended that a NSSP be included in the Contract Documents to warn the Contractor of the presence of cobbles and/or boulders within the overburden soils.

## 2.7 Monitoring Program

Monitoring of the effect of the construction of the new structure on the existing structure, must be carried on. Provided that the unwatering/dewatering and shoring are carried out in accordance with specifications and good practice significant impact on the existing amenities are not anticipated. However, monitoring of movements of the existing structures and shoring systems during construction of the new bridge is recommended. Therefore, for this site the following elements of monitoring are anticipated.

### 2.7.1 Movements of Existing Bridge

Survey points and electro level/tiltmeters should be used to monitor movements of the existing bridge. The monitoring plan should include following:

- Install survey points along existing bridge (min 6 m c/c) and the existing adjacent abutment and bridge deck (min 5m c/c).
- Install electro level/tiltmeter sensors attached to the existing north and south abutments in addition to survey points.
- Conduct pre-construction condition survey of structure prior to construction, including abutments and pavement structures. Record condition and location of any cracks. Crack gauges may be required.
- Monitoring frequency will be:
  - Preconstruction: Minimum 3 baseline readings, one month prior to construction
  - During construction: Daily during excavation and construction of the bridge.
  - Post construction: Daily until the new bridge is put in service.
- The criteria for evaluation of settlement shall be based on the following action levels:

Structure Limits:

1. Review Level: If a maximum value of 5 mm relative to the baseline readings is reached, the method and rate or sequence of construction shall be reviewed or modified to mitigate further ground displacements.
2. Alert Level: If a maximum of 10 mm relative to the baseline readings is reached, the Contractor shall be required to cease construction operation or to execute pre-planned measures to secure the site to mitigate further unacceptable settlement and to assure safety of public.

**Pavement Surface Limits:**

1. Review Level: If a maximum deformation of 300 horizontal:1 vertical relative to the baseline readings is reached, the method and rate or sequence of construction shall be reviewed or modified to mitigate further ground displacements.
2. Alert Level: If a maximum deformation of 150 horizontal : 1 vertical relative to the baseline readings is reached, the Contractor shall be required to cease construction operation or to execute pre-planned measures to secure the site to mitigate further unacceptable settlement and to assure safety of public.

### **2.7.2 Movements of Temporary Protection Systems**

The minimum requirements for monitoring of temporary protection system should include the survey measurements of scaled targets attached to the shoring wall at the elevations specified. The scaled targets should be placed at a maximum spacing of 6 m with targets placed at the extreme ends and the targets distributed between the outer limits. The survey targets shall be monitored for horizontal displacement from the vertical at the frequency specified. The limit for horizontal deformation is 0.1% of the excavated height or a maximum horizontal displacement is 25 mm (as per OPSS.PROV 539 Performance Level 2).

**Shoring limits shall follow OPSS.PROV 539, Performance Level 2:**

1. Review Level: If a maximum value of 15 mm relative to the baseline readings is reached, the method and rate or sequence of construction shall be reviewed or modified to mitigate further ground displacements.
2. Alert Level: If a maximum of 25 mm relative to the baseline readings is reached, the Contractor shall be required to cease construction operation or to execute pre-planned measures to secure the site to mitigate further unacceptable settlement and to assure safety of public.

### 3 CLOSURE

The recommendations made in this report are in accordance with our present understanding of the project and are provided solely for the team responsible for the design of the works described herein.


We recommend that we be retained to review our recommendations as the design nears completion to ensure that the final design is in agreement with the assumptions on which our recommendations are based and that our recommendations have been interpreted as intended. If not accorded this review, EXP will assume no responsibility for the interpretation and use of the recommendations in this report.

A subsurface investigation is a limited sampling of a site; the subsurface conditions have been established only at the test hole locations. Should conditions at the site be encountered which differ from those reported at the test locations, we require that we be notified immediately in order to assess this additional information and our recommendations, as appropriate. It may then be necessary to perform additional investigation and analysis.

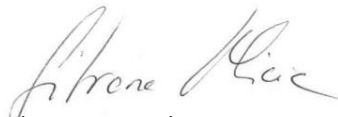
Contractors bidding on or undertaking any proposed work at this site should, relative to the subsurface conditions, decide on their own investigations, if deemed necessary, as well as their own interpretations of the factual results provided herein, so they may draw their own conclusions as to how the subsurface conditions may affect them.

This Foundation Investigation and Design Report has been prepared by Sugitha Anandakumar, M.Eng., P.Eng., and Silvana Micic, Ph.D., P.Eng. It was reviewed by TaeChul Kim, M.E.Sc., P.Eng. and by Stan E. Gonsalves, M.Eng., P.Eng., Designated MTO Foundation Contact. The field investigation was supervised by Shane Tobias.

#### EXP Services Inc.



Sugitha Anandakumar, M.Eng., P.Eng.  
Geotechnical Engineer



Silvana Micic, Ph.D., P.Eng.  
Senior Geotechnical Engineer  
Project Manager



TaeChul Kim, M.E.Sc., P.Eng.  
Senior Geotechnical/Foundation Specialist



Stan E. Gonsalves, M.Eng., P.Eng.  
Principal Engineer  
Designated MTO Foundation Contact



## 4 REFERENCES

Bureau of Reclamation, 1963. Earth Manual, 1st Edition, p 783.

Canadian Geotechnical Society, 2006. Canadian Foundation Engineering Manual, 4th Edition. The Canadian Geotechnical Society, BiTech Publisher Ltd., British Columbia.

Canadian Standards Association (CSA), 2019. Canadian Highway Bridge Design Code and Commentary on CAN/CSA-S6-19. CSA Special Publication.

Davisson, M. T., 1970. Lateral Load Capacity of Piles. Highway Research Record, No. 333, 104-112

Hvorslev, M.J., 1951. Time Lag and Soil Permeability in Ground-Water Observations, Bull. No. 36, Waterways Exper. Sta. Corps of Engrs, U.S. Army, Vicksburg, Mississippi, p 1-50.

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Ministry of Northern Development and Mines Map 2543. Bedrock Geology of Ontario, East-Central Sheet, 1991

MTO Structural Office, 1996. Integral Abutment Manual. Ronen House

Terzaghi, K., 1955. Evaluation of Coefficients of Subgrade Reaction. Geotechnique, Vol. 5, No. 4, 297-326.

### **ASTM International:**

ASTM D1586 Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils

ASTM D4044 - 15 Standard Test Method for (Field Procedure) for Instantaneous Change in Head (Slug) Tests for Determining Hydraulic Properties of Aquifers

### **Ontario Provincial Standard Specifications (OPSS):**

OPSS 539 Construction Specification for Temporary Protection Systems

OPSS 206 Construction Specification for Grading

OPSS 1010 Material Specification for Aggregates - Base, Subbase, Select Subgrade, And Backfill Material

OPSS 212 Construction Specification for Earth Borrow

OPSS 501 Construction Specification for Compacting

OPSS 517 Construction Specification for Dewatering

OPSS 902 Construction Specification for Excavating and Backfilling – Structures

### **Ontario Provincial Standard Drawings (OPSD):**

OPSD 3090.100 Foundation Frost Depths for Northern Ontario

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Date: January 4, 2021*

OPSD 3121.150	Walls, Retaining, Backfill, Minimum Granular Requirement
OPSD 3101.150	Walls, Abutment, Backfill, Minimum Granular Requirement
OPSD 3190.100	Walls, Retaining and Abutment Wall Drain
OPSD 3000.100	Foundation Piles, Steel H-Pile Driving Shoe
OPSD 208.010	Benching of Earth Slopes
OPSD 203.010	Embankments over Swamp, New Construction
OPSD 203.030	Embankments over Swamp, Existing Slope Maintained
OPSD 202.010	Slope Flattening Using Surplus Excavated Material on Earth and Rock Embankment

**Special Provisions (SP):**

SP 105S22	AMENDMENT TO OPSS 501
SP 517F01	AMENDMENT TO OPSS 517
SP 105F09	AMENDMENT TO OPSS 539

**Ontario Water Resources Act:**

R.R.O 1990, Regulation 903 Wells, under Ontario Water Resources Act, R.S.O. 1990, c. O.40

**Ontario Occupational Health and Safety Act (OHSA):**

Ontario Regulation 213/91 Construction Projects



## 5 LIMITATIONS AND USE OF REPORT

### **BASIS OF REPORT**

This report (“Report”) is based on site conditions known or inferred by the geotechnical investigation undertaken as of the date of the Report. Should changes occur which potentially impact the geotechnical condition of the site, or if construction is implemented more than one year following the date of the Report, the recommendations of EXP may require re-evaluation.

The Report is provided solely for the guidance of design engineers and on the assumption that the design will be in accordance with applicable codes and standards. Any changes in the design features which potentially impact the geotechnical analyses or issues concerning the geotechnical aspects of applicable codes and standards will necessitate a review of the design by EXP. Additional field work and reporting may also be required.

Where applicable, recommended field services are the minimum necessary to ascertain that construction is being carried out in general conformity with building code guidelines, generally accepted practices and EXP’s recommendations. Any reduction in the level of services recommended will result in EXP providing qualified opinions regarding the adequacy of the work. EXP can assist design professionals or contractors retained by the Client to review applicable plans, drawings, and specifications as they relate to the Report or to conduct field reviews during construction.

Contractors contemplating work on the site are responsible for conducting an independent investigation and interpretation of the borehole results contained in the Report. The number of boreholes necessary to determine the localized underground conditions as they impact construction costs, techniques, sequencing, equipment and scheduling may be greater than those carried out for the purpose of the Report.

Classification and identification of soils, rocks, geological units, contaminant materials, building envelopment assessments, and engineering estimates are based on investigations performed in accordance with the standard of care set out below and require the exercise of judgment. As a result, even comprehensive sampling and testing programs implemented with the appropriate equipment by experienced personnel may fail to locate some conditions. All investigations or building envelope descriptions involve an inherent risk that some conditions will not be detected. All documents or records summarizing investigations are based on assumptions of what exists between the actual points sampled. Actual conditions may vary significantly between the points investigated. Some conditions are subject to change over time. The Report presents the conditions at the sampled points at the time of sampling. Where special concerns exist, or the Client has special considerations or requirements, these should be disclosed to EXP to allow for additional or special investigations to be undertaken not otherwise within the scope of investigation conducted for the purpose of the Report.

### **RELIANCE ON INFORMATION PROVIDED**

The evaluation and conclusions contained in the Report are based on conditions in evidence at the time of site inspections and information provided to EXP by the Client and others. The Report has been prepared for the specific site, development, building, design or building assessment objectives and purpose as communicated by the Client. EXP has relied in good faith upon such representations, information and instructions and accepts no responsibility

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Assignment No. 5018-E-0012; Work Item No. 7  
Date: January 4, 2021*

for any deficiency, misstatement or inaccuracy contained in the Report as a result of any misstatements, omissions, misrepresentation or fraudulent acts of persons providing information. Unless specifically stated otherwise, the applicability and reliability of the findings, recommendations, suggestions or opinions expressed in the Report are only valid to the extent that there has been no material alteration to or variation from any of the information provided to EXP.

## **STANDARD OF CARE**

The Report has been prepared in a manner consistent with the degree of care and skill exercised by engineering consultants currently practicing under similar circumstances and locale. No other warranty, expressed or implied, is made. Unless specifically stated otherwise, the Report does not contain environmental consulting advice.

## **COMPLETE REPORT**

All documents, records, data and files, whether electronic or otherwise, generated as part of this assignment form part of the Report. This material includes, but is not limited to, the terms of reference given to EXP by its client ("Client"), communications between EXP and the Client, other reports, proposals or documents prepared by EXP for the Client in connection with the site described in the Report. In order to properly understand the suggestions, recommendations and opinions expressed in the Report, reference must be made to the Report in its entirety. EXP is not responsible for use by any party of portions of the Report.

## **USE OF REPORT**

The information and opinions expressed in the Report, or any document forming part of the Report, are for the sole benefit of the Client. No other party may use or rely upon the Report in whole or in part without the written consent of EXP. Any use of the Report, or any portion of the Report, by a third party are the sole responsibility of such third party. EXP is not responsible for damages suffered by any third party resulting from unauthorised use of the Report.

## **REPORT FORMAT**

Where EXP has submitted both electronic file and a hard copy of the Report, or any document forming part of the Report, only the signed and sealed hard copy shall be the original documents for record and working purposes. In the event of a dispute or discrepancy, the hard copy shall govern. Electronic files transmitted by EXP have utilized specific software and hardware systems. EXP makes no representation about the compatibility of these files with the Client's current or future software and hardware systems. Regardless of format, the documents described herein are EXP's instruments of professional service and shall not be altered without the written consent of EXP.

## Appendix A – Site Photographs



Photograph A1. West-south side of the North Driftwood River Bridge



Photograph A2. West-north side of the North Driftwood River Bridge showing the BM





Photograph A3. West-south side of the North Driftwood River Bridge, showing drilling BH20-1



Photograph A4. West-south side of the North Driftwood River Bridge, showing drilling BH20-2





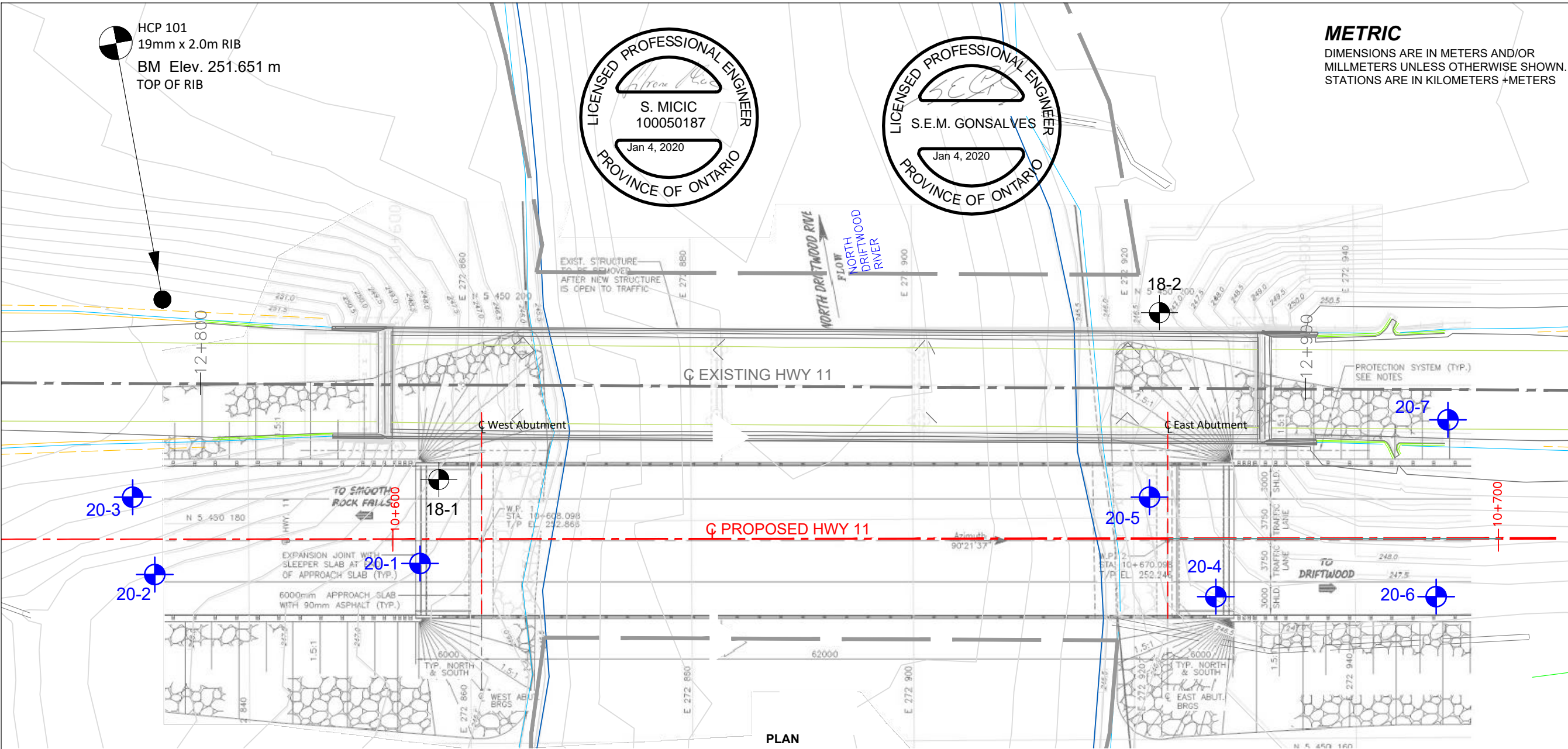
Photograph A5. West-south side of the North Driftwood River Bridge, showing drilling BH20-3



Photograph A6. East-south side of the North Driftwood River Bridge, showing drilling of BH20-5

## Appendix B – Drawings





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GWP -

HWY 11, NORTH DRIFTWOOD RIVERWOOD  
BRIDGE REPLACEMENT

BOREHOLE LOCATION PLAN AND SOIL STRATA

exp. EXP Services Inc.

KEY PLAN

LEGEND

- Borehole Location (EXP 2020)
- Existing Borehole Location (GOLDER 2018)
- Bench Mark Location
- N Standard Penetration Test (Blows/0.3 m)
- Groundwater level measured in open hole
- Water level measured in Piezometer

SOIL STRATA SYMBOLS

FILL	SAND	GRAVELLY SAND/ SAND & GRAVEL
ASPHALT	SAND & SILT	COBBLES & BOULDERS
TOPSOIL	ORGANIC SILT	BEDROCK
SILT	PEAT	

BH No.	ELEV.	COORDINATES (NAD83 MTM Zone 12)	
		NORTHING	EASTING
20-1	247.5	5450176.2	272855.4
20-2	247.9	5450175.2	272831.4
20-3	251.8	5450182.2	272829.4
20-4	247.8	5450173.2	272927.4
20-5	247.5	5450182.2	272921.4
20-6	248.0	5450173.2	272947.3
20-7	251.0	5450189.2	272948.4
18-1	248.0	5450188.2	272945.4
18-2	248.0	5450189.2	272948.4

SCALE

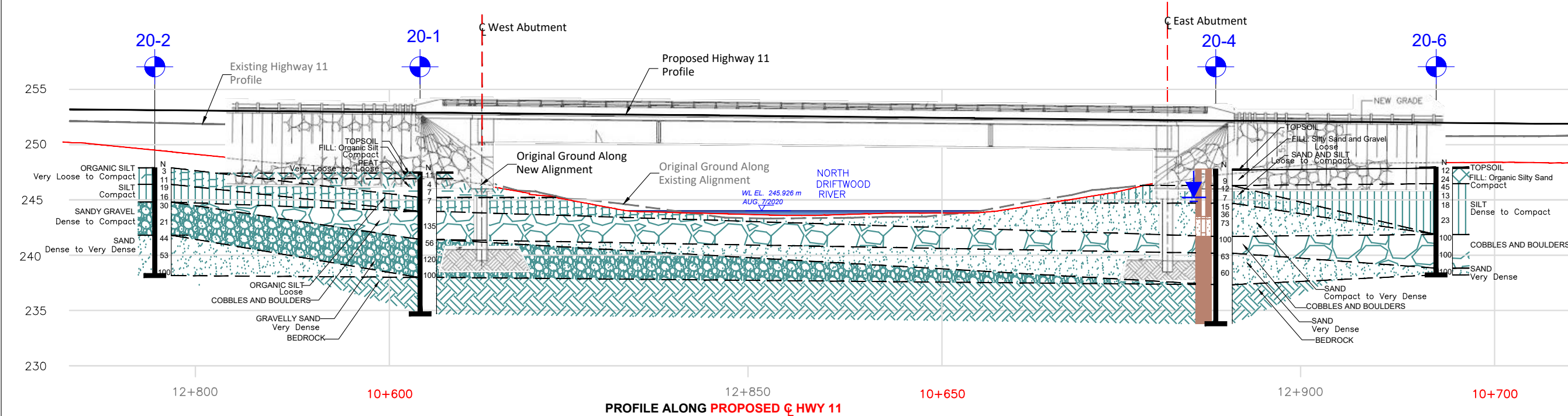
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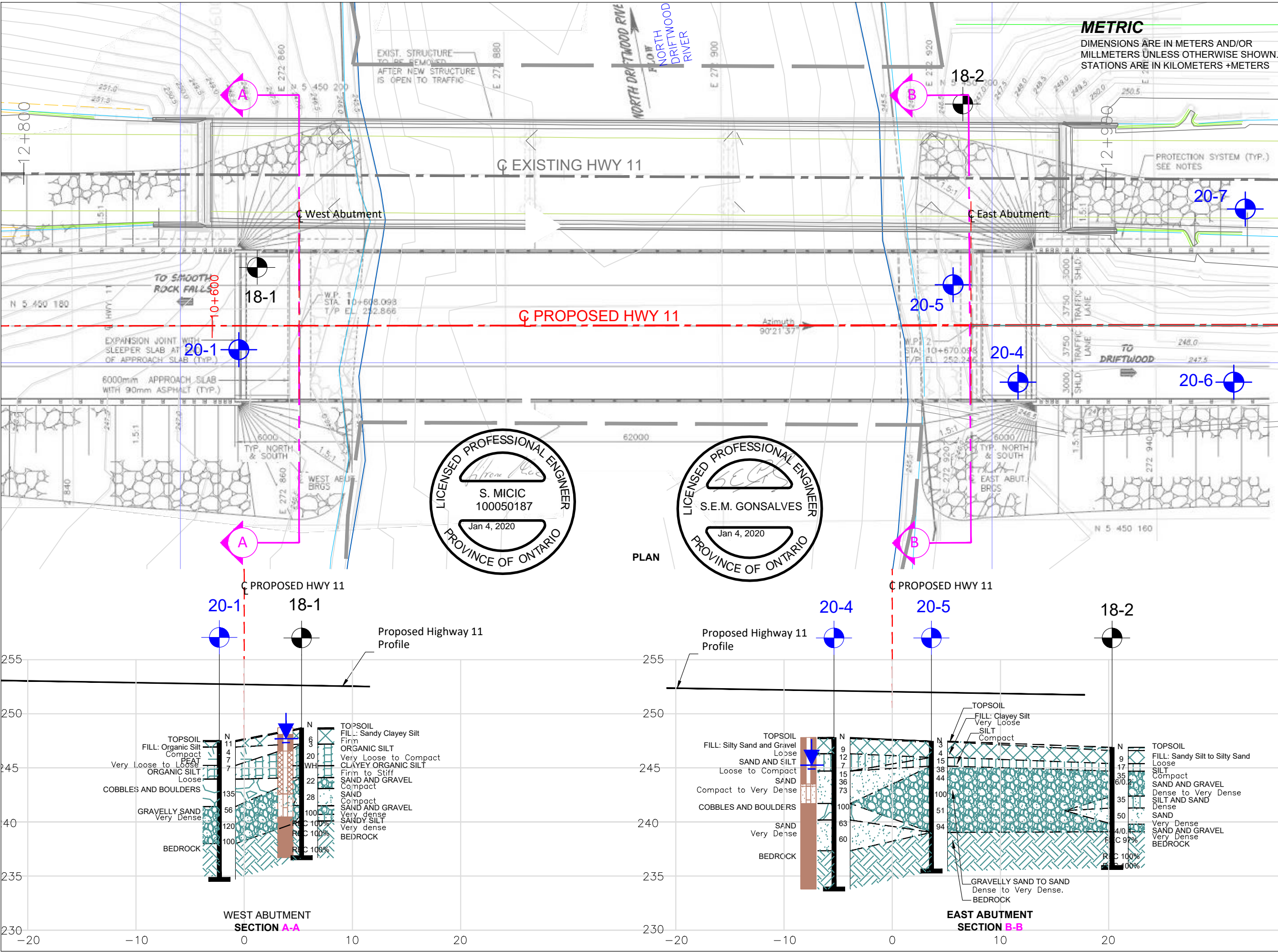
This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

The complete foundation investigation and design report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in the report and related documents are specifically excluded in accordance with the conditions of Section GC 2.01 of OPS Gen. Cond.

DATE	SM	SUBMISSION FOR MTO REVIEW	
DATE	BY	DESCRIPTION	
		GEOCRETS NO. 31M-1216	
		PROJECT NO. ADM-00257843-G0	
SUBM'D SA	CHECKED SM	DATE	2020-09-29
DRAWN SH	CHECKED TC	APPROVED SG	DWG. 1







Agreement No. 5018-E-0012  
Assignment No. 7  
GWP -

HWY 11, NORTH DRIFTWOOD RIVERWOOD  
BRIDGE REPLACEMENT

SHEET  
1

exp.

EXP Services Inc.

KEY PLAN

LEGEND

Borehole Location (EXP 2020)

Existing Borehole Location (GOLDER 2018)

Standard Penetration Test (Blows/0.3 m)

Groundwater level measured in open hole

Water level measured in Piezometer

SOIL STRATA SYMBOLS

FILL

ASPHALT

TOPSOIL

SILT

SAND

SAND & SILT

ORGANIC SILT

PEAT

GRAVELLY SAND/  
SAND & GRAVEL

COBBLES &  
BOULDERS

BEDROCK

BH No.	ELEV.	COORDINATES (NAD83 MTM Zone 12)	
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18-2	248.0	5450189.2	272948.4

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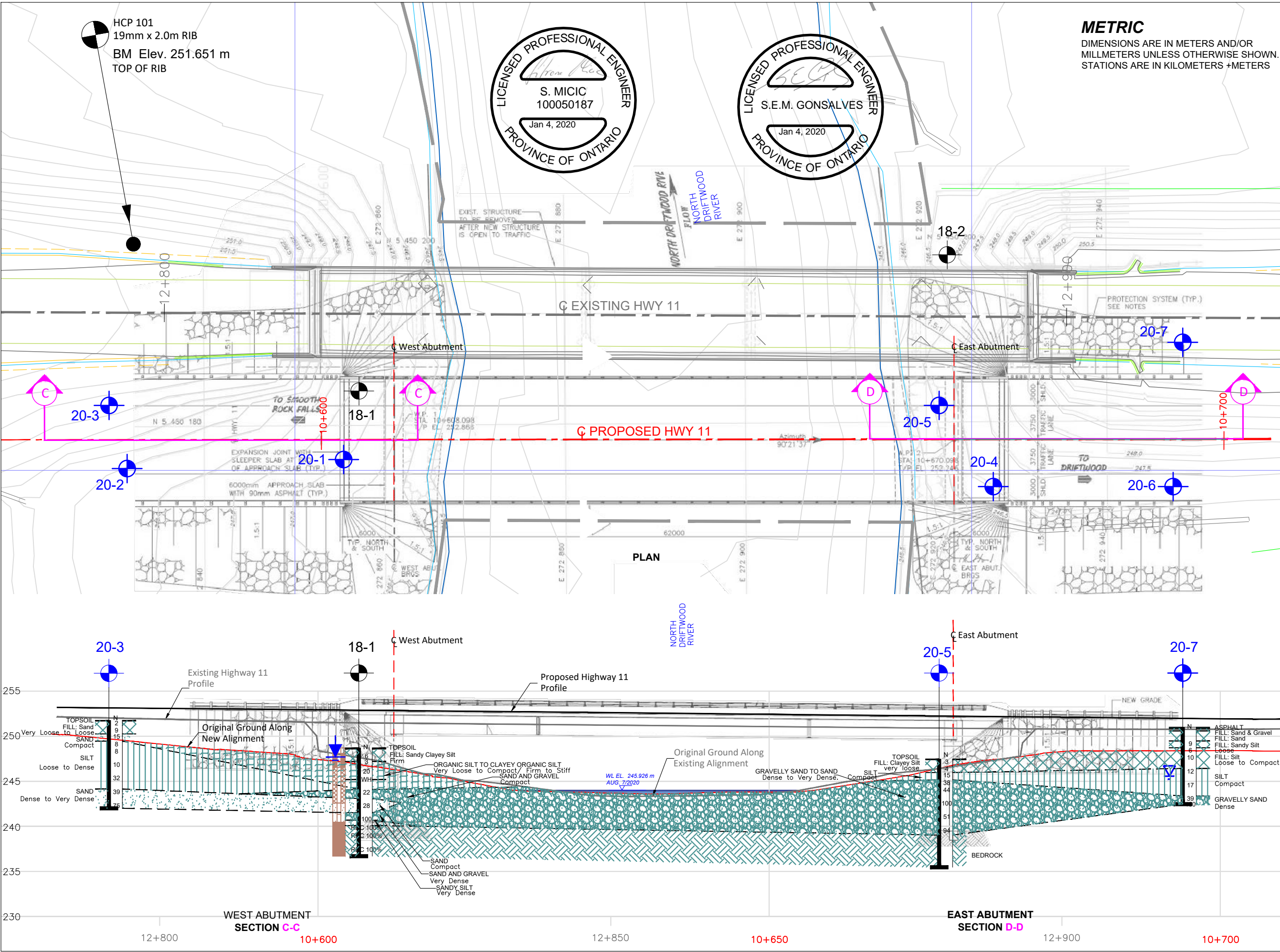
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SUBMISSION FOR MTO REVIEW			
DATE	DESCRIPTION		
	GEOCRES NO. 31M-1216		
	PROJECT NO. ADM-00257843-G0		
SUBM'D SA	CHECKED SM	DATE	2020-09-28
DRAWN SH	CHECKED TC	APPROVED SG	DWG. 2





Agreement No. 5018-E-0012  
Assignment No. 7  
GWP -




HWY 11, NORTH DIFTWOOD RIVERWOOD  
BRIDGE REPLACEMENT

BOREHOLE LOCATION PLAN AND SOIL STRATA

exp.

EXP Services Inc.

KEY PLAN



LEGEND

- Borehole Location (EXP 2020)
- Existing Borehole Location (Golder)
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0 5 10 m

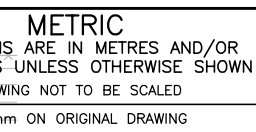
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DATE	BY	DESCRIPTION	
		GEOCRETS NO. 31M-1216	
		PROJECT NO. ADM-00257843-G0	
SUBM'D SA	CHECKED SM	DATE	2020-09-28
DRAWN SH	CHECKED TC	APPROVED SG	DWG. 3



## Appendix C – Borehole Logs



# Explanation of Terms Used on Borehole Records

## SOIL DESCRIPTION

Terminology describing common soil genesis:

*Topsoil:* mixture of soil and humus capable of supporting good vegetative growth.

*Peat:* fibrous fragments of visible and invisible decayed organic matter.

*Fill:* where fill is designated on the borehole log it is defined as indicated by the sample recovered during the boring process. The reader is cautioned that fills are heterogeneous in nature and variable in density or degree of compaction. The borehole description may therefore not be applicable as a general description of site fill materials. All fills should be expected to contain obstruction such as wood, large concrete pieces or subsurface basements, floors, tanks, etc.; none of these may have been encountered in the boreholes. Since boreholes cannot accurately define the contents of the fill, test pits are recommended to provide supplementary information. Despite the use of test pits, the heterogeneous nature of fill will leave some ambiguity as to the exact composition of the fill. Most fills contain pockets, seams, or layers of organically contaminated soil. This organic material can result in the generation of methane gas and/or significant ongoing and future settlements. Fill at this site may have been monitored for the presence of methane gas and, if so, the results are given on the borehole logs. The monitoring process does not indicate the volume of gas that can be potentially generated nor does it pinpoint the source of the gas. These readings are to advise of the presence of gas only, and a detailed study is recommended for sites where any explosive gas/methane is detected. Some fill material may be contaminated by toxic/hazardous waste that renders it unacceptable for deposition in any but designated land fill sites; unless specifically stated the fill on this site has not been tested for contaminants that may be considered toxic or hazardous. This testing and a potential hazard study can be undertaken if requested. In most residential/commercial areas undergoing reconstruction, buried oil tanks are common and are generally not detected in a conventional geotechnical site investigation.

*Till:* the term till on the borehole logs indicates that the material originates from a geological process associated with glaciation. Because of this geological process the till must be considered heterogeneous in composition and as such may contain pockets and/or seams of material such as sand, gravel, silt or clay. Till often contains cobbles (60 to 200 mm) or boulders (over 200 mm). Contractors may therefore encounter cobbles and boulders during excavation, even if they are not indicated by the borings. It should be appreciated that normal sampling equipment cannot differentiate the size or type of any obstruction. Because of the horizontal and vertical variability of till, the sample description may be applicable to a very limited zone; caution is therefore essential when dealing with sensitive excavations or dewatering programs in till materials.

Terminology describing soil structure:

*Desiccated:* having visible signs of weathering by oxidization of clay minerals, shrinkage cracks, etc.

*Stratified:* alternating layers of varying material or color with the layers greater than 6 mm thick.

*Laminated:* alternating layers of varying material or color with the layers less than 6 mm thick.

*Fissured:* material breaks along plane of fracture.

*Varved:* composed of regular alternating layers of silt and clay.

*Slickensided:* fracture planes appear polished or glossy, sometimes striated.

*Blocky:* cohesive soil that can be broken down into small angular lumps which resist further breakdown.

*Lensed:* inclusion of small pockets of different soil, such as small lenses of sand scattered through a mass of clay; not thickness.

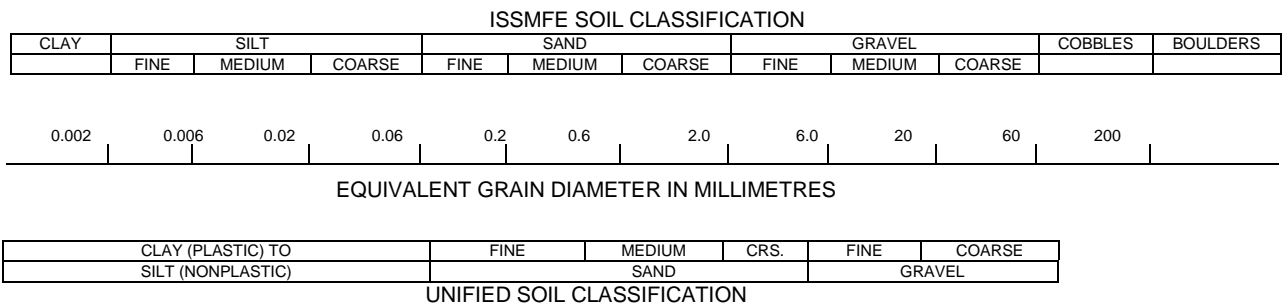
*Seam:* a thin, confined layer of soil having different particle size, texture, or color from materials above and below.

*Homogeneous:* same color and appearance throughout.

*Well Graded:* having wide range in grain sized and substantial amounts of all predominantly on grain size.

*Uniformly Graded:* predominantly on grain size.

All soil sample descriptions included in this report follow the ASTM D2487-11 Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System). The system divides soils into three major categories: (1) coarse grained, (2) fine-grained, and (3) highly organic. The soil is then subdivided based on either gradation or plasticity characteristics. The system provides a group symbol (e.g. SM) and group name (e.g. silty sand) for identification. The classification excludes particles larger than 76 mm. Please note that, with the exception of those samples where a grain size analysis has been made, all samples are classified visually in accordance with ASTM D2488-09a Standard Practice for Description and Identification of Soils (Visual-Manual Procedure). Visual classification is not sufficiently accurate to provide exact grain sizing or precise differentiation between size classification systems. Others may use different classification systems; one such system is the ISSMFE Soil Classification.



Terminology describing materials outside the USCS, (e.g. particles larger than 76 mm, visible organic matter, construction debris) is based upon the proportion of these materials present and as described below in accordance with Note 16 in ASTM D2488-09a:

Table a: Percent or Proportion of Soil, Pp

	Criteria
Trace	Particles are present but estimated to be less than 5%
Few	$5 \leq Pp \leq 10\%$
Little	$15 \leq Pp \leq 25\%$
Some	$30 \leq Pp \leq 45\%$
Mostly	$50 \leq Pp \leq 100\%$

The standard terminology to describe cohesionless soils includes the compactness as determined by the Standard Penetration Test 'N' value:

Table b: Apparent Density of Cohesionless Soil

	'N' Value (blows/0.3 m)
Very Loose	$N < 5$
Loose	$5 \leq N < 10$
Compact	$10 \leq N < 30$
Dense	$30 \leq N < 50$
Very Dense	$50 \leq N$

The standard terminology to describe cohesive soils includes consistency, which is based on undrained shear strength as measured by insitu vane tests, penetrometer tests, unconfined compression tests or similar field and laboratory analysis, Standard Penetration Test 'N' values can also be used to provide an approximate indication of the consistency and shear strength of fine grained, cohesive soils:

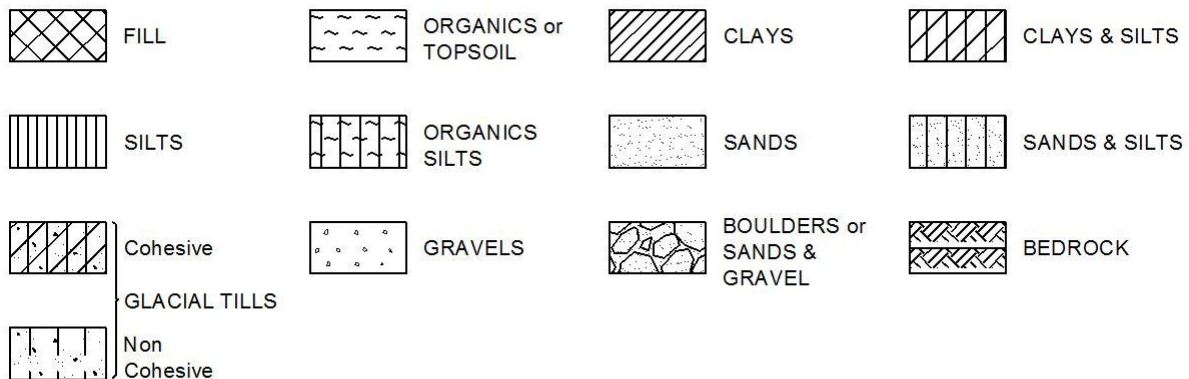
Table c: Consistency of Cohesive Soil

Consistency	Vane Shear Measurement (kPa)	'N' Value
Very Soft	<12.5	<2
Soft	12.5-25	2-4
Firm	25-50	4-8
Stiff	50-100	8-15
Very Stiff	100-200	15-30
Hard	>200	>30

Note: 'N' Value - The Standard Penetration Test records the number of blows of a 140 pound (64kg) hammer falling 30 inches (760mm), required to drive a 2 inch (50.8mm) O.D. split spoon sampler 1 foot (305mm). For split spoon samples where full penetration is not achieved, the number of blows is reported over the sampler penetration in meters (e.g. 50/0.15).

## STRATA PLOT

Strata plots symbolize the soil or bedrock description. They are combinations of the following basic symbols:



## WATER LEVEL MEASUREMENT



Open Borehole or Test Pit



Monitoring Well, Piezometer or Standpipe

## ABBREVIATIONS AND SYMBOLS

### FIELD SAMPLING

SS	Split spoon sample (obtained from the Standard Penetration Test)
WS	Wash sample
BS	Bulk sample
TW	Thin wall sample or Shelby tube
PS	Piston sample
AS	Auger sample
VT	Vane test
GS	Grab sample
HQ, NQ, etc.	Rock core samples obtained with the use of standard size diamond drilling bits

### STRESS AND STRAIN

$u_w$	kPa	Pore water pressure
$r_u$	1	Pore pressure ratio
$\sigma$	kPa	Total normal stress
$\sigma'$	kPa	Effective normal stress
$\tau$	kPa	Shear stress
$\sigma_1, \sigma_2, \sigma_3$	kPa	Principal stresses
$\varepsilon$	%	Linear strain
$\varepsilon_1, \varepsilon_2, \varepsilon_3$	%	Principal strains
E	kPa	Modulus of linear deformation
G	kPa	Modulus of shear deformation
$\mu$	1	Coefficient of friction

### MECHANICAL PROPERTIES OF SOIL

$m_v$	$\text{kPa}^{-1}$	Coefficient of volume change
$c_c$	1	Compression index
$c_s$	1	Swelling index
$c_r$	1	Recompression index
$c_v$	$\text{m}^2/\text{s}$	Coefficient of consolidation
H	m	Drainage path
$T_v$	1	Time factor
U	%	Degree of consolidation
$\sigma'_{v0}$	kPa	Effective overburden pressure
$\sigma'_p$	kPa	Preconsolidation pressure
$\tau_f$	kPa	Shear strength
$c'$	kPa	Effective cohesion intercept
$\phi'$	$^\circ$	Effective angle of internal friction
$c_u$	kPa	Apparent cohesion intercept
$\phi_u$	$^\circ$	Apparent angle of internal friction
$\tau_R$	kPa	Residual shear strength
$\tau_r$	kPa	Remoulded shear strength
$S_t$	1	Sensitivity = $c_u/\tau_r$

### PHYSICAL PROPERTIES OF SOIL

$P_s$	$\text{kg}/\text{m}^3$	Density of solid particles
$\gamma_s$	$\text{kN}/\text{m}^3$	Unit weight of solid particles
$\rho_w$	$\text{kg}/\text{m}^3$	Density of water
$\gamma_w$	$\text{kN}/\text{m}^3$	Unit weight of water
$\rho$	$\text{kg}/\text{m}^3$	Density of soil
$\gamma$	$\text{kN}/\text{m}^3$	Unit weight of soil
$\rho_d$	$\text{kg}/\text{m}^3$	Density of dry soil
$\gamma_d$	$\text{kN}/\text{m}^3$	Unit weight of dry soil
$\rho_{sat}$	$\text{kg}/\text{m}^3$	Density of saturated soil
$\gamma_{sat}$	$\text{kN}/\text{m}^3$	Unit weight of saturated soil
$\rho'$	$\text{kg}/\text{m}^3$	Density of submerged soil
$\gamma'$	$\text{kN}/\text{m}^3$	Unit weight of submerged soil
$e$	1, %	Void ratio
$n$	1, %	Porosity
$w$	1, %	Water content
$S_r$	%	Degree of saturation
$W_L$	%	Liquid limit
$W_P$	%	Plastic limit
$W_s$	%	Shrinkage limit
$I_p$	%	Plasticity index = $(W_L - W_P)$
$I_L$	%	Liquidity index = $(W - W_P)/I_p$
$I_C$	%	Consistency index = $(W_L - W)/I_p$
$e_{max}$	1, %	Void ratio in loosest state
$e_{min}$	1, %	Void ratio in densest state
$I_D$	1	Density index = $(e_{max} - e)/(e_{max} - e_{min})$
D	mm	Grain diameter
$D_n$	mm	N percent - diameter
$C_u$	1	Uniformity coefficient
h	m	Hydraulic head or potential
q	$\text{m}^3/\text{s}$	Rate of discharge
v	m/s	Discharge velocity
i	1	Hydraulic gradient
k	m/s	Hydraulic conductivity
j	$\text{kN}/\text{m}^3$	Seepage force



Brampton, Ontario

## 1 OF 2

METRIC

W.P.	Agreement No. 5018-E-0012, WO No. 7		LOCATION	North Driftwood River Bridge, N5450176.2, E272855.4 NAD83 MTM Zone 12		ORIGINATED BY	ST	
DIST	NE	HWY 11	BOREHOLE TYPE	Continuous Flight HSA, NW Casing, and NQ Core Barrel		COMPILED BY	PH	
DATUM	Geodetic		DATE	2020.08.05 - 2020.08.06	LATITUDE	49.18884	LONGITUDE	-81.4383
						CHECKED BY	IM	

SOIL PROFILE					
ELEV DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES	GROUND WATER CONDITIONS	DYNAMIC CONE PENETRATION RESISTANCE PLOT
			NUMBER	TYPE	"N" VALUES
247.5					
0.1	TOP SOIL (~100 mm thick)	[Symbol]	SS1	SS	11
246.9	FILL, organic silt, some sand, some gravel, dark brown to brown, moist, compact	[Symbol]			
0.6	PEAT, dark brown to brown, moist, very loose to loose	[Symbol]	SS2	SS	4
		[Symbol]			
		[Symbol]	SS3	SS	7
		[Symbol]			
245.2					
2.3	ORGANIC SILT, some sand, some clay, some wood, dark brown, moist, loose	[Symbol]	SS4	SS	7
		[Symbol]			
		[Symbol]	ST5	TW	
244.0					
3.6	COBBLES AND BOULDERS, cored using NW casing, no recovery.	[Symbol]			
	No sample recovery. Split spoon likely pushing on cobbles/boulders.	[Symbol]	SS6	SS	135
	COBBLES ~50 to 150 mm in diameter.	[Symbol]	CORE		
241.4					
6.1	GRAVELLY SAND, trace silt/clay, grey, wet, very dense	[Symbol]	SS7	SS	56
		[Symbol]			
		[Symbol]	SS8	SS	120
		[Symbol]			
		[Symbol]	SS9	SS	100
238.0					
9.5	BEDROCK meta-diorite, grey, fresh to slightly weathered	[Symbol]	Run 1	CORE	
		[Symbol]	Run 2	CORE	

UNCONFINED + FIELD VANE

QUICK TRIAXIAL & P. PENETROMETER

SHEAR STRENGTH kPa

PLASTIC LIMIT W<sub>P</sub>

NATURAL MOISTURE CONTENT W

LIQUID LIMIT W<sub>L</sub>

UNIT WEIGHT γ

REMARKS &  
GRAIN SIZE DISTRIBUTION (%)

GR SA SI CL

20 40 60 80 100

20 40 60

kN/m³

0 11 76 13

(6)

UCS: 100.7 MPa

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      ○<sup>3%</sup> STRAIN AT FAILURE

ONTARIO MTO ADM-00257843-G0 - DRIFTWOOD RIVER BRIDGE.GPJ ONTARIO MTO.GDT 9/30/20

METRIC

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      ○<sup>3%</sup> STRAIN AT FAILURE

Brampton, Ontario

# RECORD OF BOREHOLE No 20-2

1 OF 1

METRIC

W.P. Agreement No. 5018-E-0012, WO No. 7 LOCATION North Driftwood River Bridge, N5450175.2, E272831.4 NAD83 MTM Zone 12 ORIGINATED BY ST  
 DIST NE HWY 11 BOREHOLE TYPE Continuous Flight HSA and NW Casing COMPILED BY PH  
 DATUM Geodetic DATE 2020.08.06 - 2020.08.06 LATITUDE 49.18883 LONGITUDE -81.4386 CHECKED BY IM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									WATER CONTENT (%)		
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL P. PENETROMETER											
247.9							20	40	60	80	100								
0.0	ORGANIC SILT, some sand, brown to dark brown, moist, very loose to compact		SS1	SS	3														
	some gravel, compact below ~0.8 m depth.		SS2	SS	11														
246.4																			
1.5	SILT, trace gravel, trace sand, trace clay, brown, moist, compact		SS3	SS	19														
			SS4	SS	16											8 6 80 6			
244.8																			
3.1	SANDY GRAVEL, with cobbles (~50 to 150 mm diameter), trace silt/ clay, brown, wet, dense to compact		SS5	SS	30											68 24 (8)			
	Cored with NW casing.																		
			SS6	SS	21														
241.8																			
6.1	SAND, with cobbles (~75 to 150 mm diameter), some gravel, some silt/ clay, brown, wet, dense to very dense		SS7	SS	44											18 70 (12)			
			SS8	SS	53														
			SS9	SS	100														
238.2																			
9.8	END OF BOREHOLE																		
	Groundwater level could not be obtained upon completion due to water pumped into borehole as a result of coring.																		

+ 3, X 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

ONTARIO MTO ADM-00257843-G0 - DRIFTWOOD RIVER BRIDGE.GPJ ONTARIO MTO.GDT 9/30/20

Brampton, Ontario

# RECORD OF BOREHOLE No 20-3

1 OF 1

METRIC

W.P. Agreement No. 5018-E-0012, WO No. 7 LOCATION North Driftwood River Bridge, N5450182.2, E272829.4 NAD83 MTM Zone 12 ORIGINATED BY ST  
 DIST NE HWY 11 BOREHOLE TYPE Continuous Flight HSA COMPILED BY PH  
 DATUM Geodetic DATE 2020.08.07 - 2020.08.07 LATITUDE 49.18889 LONGITUDE -81.4386 CHECKED BY IM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT  γ  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL P. PENETROMETER				W <sub>P</sub>	W	W <sub>L</sub>		GR	SA	SI	CL
251.8								20	40	60	80	100							
251.8	TOPSOIL (~50 mm thick)		SS1	SS	2														
251.8	FILL, sand, trace silt, brown, moist, very loose to loose																		
	trace gravel below ~0.8 m depth.		SS2	SS	9		251												
250.3																			
1.5	SAND, trace gravel, trace silt, brown, moist, compact		SS3	SS	15		250												
249.5																			
2.3	SILT, trace gravel, some sand, some to trace clay, trace organics, brown, moist to wet, loose to dense		SS4	SS	8		249												
	trace gravel below ~3.1 m depth.		SS5	SS	8		248											7 13 62 18	
	moist to wet, compact below ~4.6 m depth.		SS6	SS	10		247												
							246												
	becomes sandy with gravel, dense below ~6.1 m		SS7	SS	32		245											22 39 37 2	
244.2																			
7.6	SAND, some gravel, trace silt/ clay, brown, wet, dense to very dense		SS8	SS	39		244											15 80 (5)	
							243												
			SS9	SS	75														
242.0																			
9.8	END OF BOREHOLE																		
	Groundwater Level: Dry (Upon Completion)																		
	Cave to ~4.3 m depth.																		

+ 3, X 3: Numbers refer to Sensitivity O 3% STRAIN AT FAILURE

ONTARIO MTO ADM-00257843-G0 - DRIFTWOOD RIVER BRIDGE.GPJ ONTARIO MTO.GDT 9/30/20

Brampton, Ontario

# RECORD OF BOREHOLE No 20-4

1 OF 2

METRIC

W.P. Agreement No. 5018-E-0012, WO No. 7 LOCATION North Driftwood River Bridge, N5450173.2, E272927.4 NAD83 MTM Zone 12 ORIGINATED BY ST  
 DIST NE HWY 11 BOREHOLE TYPE Continuous Flight HSA, NW Casing, and NQ Core Barrel COMPILED BY PH  
 DATUM Geodetic DATE 2020.07.28 - 2020.07.28 LATITUDE 49.18882 LONGITUDE -81.4373 CHECKED BY IM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT   NATURAL MOISTURE CONTENT   LIQUID LIMIT			UNIT WEIGHT  $\gamma$  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)				
247.8								20	40	60	80	100		
247.0	TOPSOIL (~100 mm thick)		AG1	AS										
0.1	FILL, silty sand, some gravel, trace to some organics, with cobbles, dark brown, moist, loose													
			SS2	SS	9		247							
246.3														
1.5	SAND AND SILT, trace gravel, trace clay, trace wood, brown, moist to wet, loose to compact		SS3	SS	12		246							2   42   48   8
	brown and grey, wet, loose below ~2.3 m depth.		SS4	SS	7		245							
244.7														
3.1	SAND, some gravel, trace silt/ clay, brown, wet, compact to very dense		SS5	SS	15		244							17   78   (5)
			SS6	SS	36									
			SS7	SS	73		243							
							242							
241.7														
6.1	COBBLES AND BOULDERS, up to 200 mm diameter, cored using NW casing.		SS8	SS	100		241							
240.2														
7.6	SAND, some gravel, some silt/ clay, brown to grey, wet, very dense		SS9	SS	63		240							19   56   (25)
							239							
			SS10	SS	60		238							
237.3														
10.5	BEDROCK meta-diorite, grey, fresh to slightly weathered		Run	CORE			237							

Continued Next Page

+ 3, X 3: Numbers refer to Sensitivity O 3% STRAIN AT FAILURE

ONTARIO MTO ADM-00257843-G0 - DRIFTWOOD RIVER BRIDGE.GPJ ONTARIO MTO.GDT 9/30/20

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      ○<sup>3%</sup> STRAIN AT FAILURE

Brampton, Ontario

# RECORD OF BOREHOLE No 20-5

1 OF 2

METRIC

W.P. Agreement No. 5018-E-0012, WO No. 7 LOCATION North Driftwood River Bridge, N5450182.2, E272921.4 NAD83 MTM Zone 12 ORIGINATED BY ST  
 DIST NE HWY 11 BOREHOLE TYPE Continuous Flight HSA, NW Casing, and NQ Core Barrel COMPILED BY PH  
 DATUM Geodetic DATE 2020.08.04 - 2020.08.04 LATITUDE 49.1889 LONGITUDE -81.4374 CHECKED BY IM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL P. PENETROMETER							
247.5 247.0 0.1	TOPSOIL (~75 mm thick) FILL, clayey silt, some sand, some gravel, trace organics, brown, moist, very loose		SS1	SS	3		247								
			SS2	SS	4										
246.0 1.5	SILT, trace to some sand, brown, moist, compact		SS3	SS	15		246								
245.2 2.3	GRAVELLY SAND TO SAND with cobbles, some silt and clay, grey, wet, dense to very dense.		SS4	SS	38		245								
			SS5	SS	44		244								
	50 to 100 mm diameter cobbles below ~4.0 m depth. Cored using NW casing														
	150 mm diameter cobbles below ~4.9 m depth.		SS6	SS	100		243								
							242								
			SS7	SS	51		241								
	some gravel below ~7.6 m depth		SS8	SS	94		240								
239.0 8.4	BEDROCK meta-diorite, grey to pink, fresh to slightly weathered						239								
	Run 1: Start/End: 8.4 to 9.6 m Recovery: 100% RQD: 86.4% Water Colour: Grey		Run 1	CORE											
	Run 2: Start/End: 9.6 to 11.0 m Recovery: 92.7% RQD: 92.7% Water Colour: Grey to Pink						238								
	Run 3: Start/End: 11.0 to 12.0 m Recovery: 100% RQD: 100% Water Colour: Pink		Run 2	CORE			237								

Continued Next Page

+ 3, X 3: Numbers refer to Sensitivity O 3% STRAIN AT FAILURE

ONTARIO MTO ADM-00257843-GO - DRIFTWOOD RIVER BRIDGE.GPJ ONTARIO MTO.GDT 9/30/20


Brampton, Ontario

# RECORD OF BOREHOLE No 20-5

2 OF 2

METRIC

W.P. Agreement No. 5018-E-0012, WO No. 7 LOCATION North Driftwood River Bridge, N5450182.2, E272921.4 NAD83 MTM Zone 12 ORIGINATED BY ST  
 DIST NE HWY 11 BOREHOLE TYPE Continuous Flight HSA, NW Casing, and NQ Core Barrel COMPILED BY PH  
 DATUM Geodetic DATE 2020.08.04 - 2020.08.04 LATITUDE 49.1889 LONGITUDE -81.4374 CHECKED BY IM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL & P. PENETROMETER								WATER CONTENT (%)									
							20	40	60	80	100		20	40	60		
235.5			Run 3	CORE			236										
12.0	END OF BOREHOLE  Groundwater level could not be obtained upon completion due to water pumped into borehole as a result of coring.																



Brampton, Ontario

## RECORD OF BOREHOLE No 20-6

1 OF 1

METRIC

W.P. Agreement No. 5018-E-0012, WO No. 7 LOCATION North Driftwood River Bridge, N5450188.2, E272945.4 NAD83 MTM Zone 12 ORIGINATED BY ST  
 DIST NE HWY 11 BOREHOLE TYPE Continuous Flight HSA and NW Casing COMPILED BY PH  
 DATUM Geodetic DATE 2020.08.04 - 2020.08.04 LATITUDE 49.18882 LONGITUDE -81.437 CHECKED BY IM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W <sub>P</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL P. PENETROMETER									
248.0								20	40	60	80	100					
248.0 0.1	<b>TOPSOIL</b> (~100 mm thick) <b>FILL</b> , organic silty sand, some gravel, dark brown to grey, moist, compact		AG1	AS	12												
			SS2	SS	24		247										
246.5																	
1.5	<b>SILT</b> , trace sand, trace clay, brown, moist to wet, dense to compact		SS3	SS	45		246										
			SS4	SS	13		245										
			SS5	SS	18		244										
			SS6	SS	23		243										
							242										
241.9																	
6.1	<b>COBBLES AND BOULDERS</b> , 75 to 300 mm diameter, cored using NW casing.		SS7	SS	100		241										
	No sample recovery.		SS8	SS	100		240										
							239										
238.8																	
9.1	<b>SAND</b> , some silt, some gravel, grey, wet, very dense		SS9	SS	100												
238.2																	
9.8	END OF BOREHOLE  Groundwater level could not be obtained upon completion due to water pumped into borehole as a result of coring.																

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

ONTARIO MTO ADM-00257843-GO - DRIFTWOOD RIVER BRIDGE.GPJ ONTARIO MTO.GDT 9/30/20

Brampton, Ontario

## RECORD OF BOREHOLE No 20-7

1 OF 1

METRIC

W.P. Agreement No. 5018-E-0012, WO No. 7 LOCATION North Driftwood River Bridge, N5450189.2, E272948.4 NAD83 MTM Zone 12 ORIGINATED BY ST  
 DIST NE HWY 11 BOREHOLE TYPE Continuous Flight HSA COMPILED BY PH  
 DATUM Geodetic DATE 2020.08.18 - 2020.08.18 LATITUDE 49.18896 LONGITUDE -81.437 CHECKED BY IM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT   NATURAL MOISTURE CONTENT   LIQUID LIMIT			UNIT WEIGHT  <b>γ</b>  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)						
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20   40   60   80   100	20   40   60	W <sub>P</sub> W   W <sub>L</sub>	GR	SA		SI	CL					
251.0						▽														
250.9	ASPHALT (~150 mm thick)																			
0.2	FILL, sand and gravel, trace silt/ clay, brown, moist																			
250.6																				
0.4	FILL, sand, some gravel, trace silt, with cobbles, brown, moist		AG1	AS												41	56	(3)		
249.5																				
1.5	FILL, sandy silt, trace to some clay, brown, moist, loose		SS2	SS	9															
248.7																				
2.3	FILL, silt, trace sand, some clay, some wood, brown, moist, loose to compact		SS3	SS	6												0	7	75	18
246.4																				
4.6	SILT, trace sand, trace clay, brown, moist to wet, compact		SS5	SS	12															

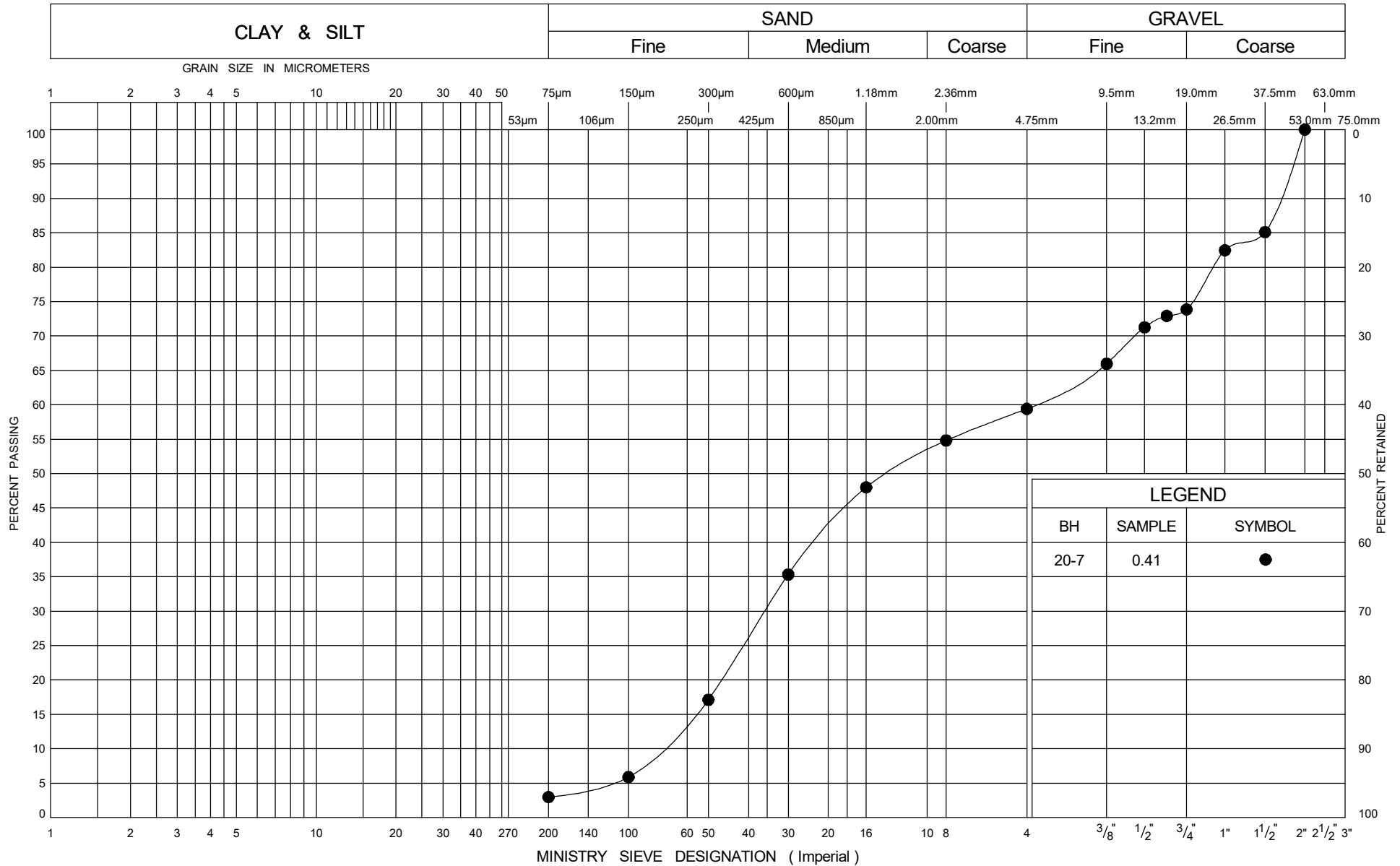
+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

ONTARIO MTO ADM-00257843-G0 - DRIFTWOOD RIVER BRIDGE.GPJ ONTARIO MTO.GDT 9/30/20

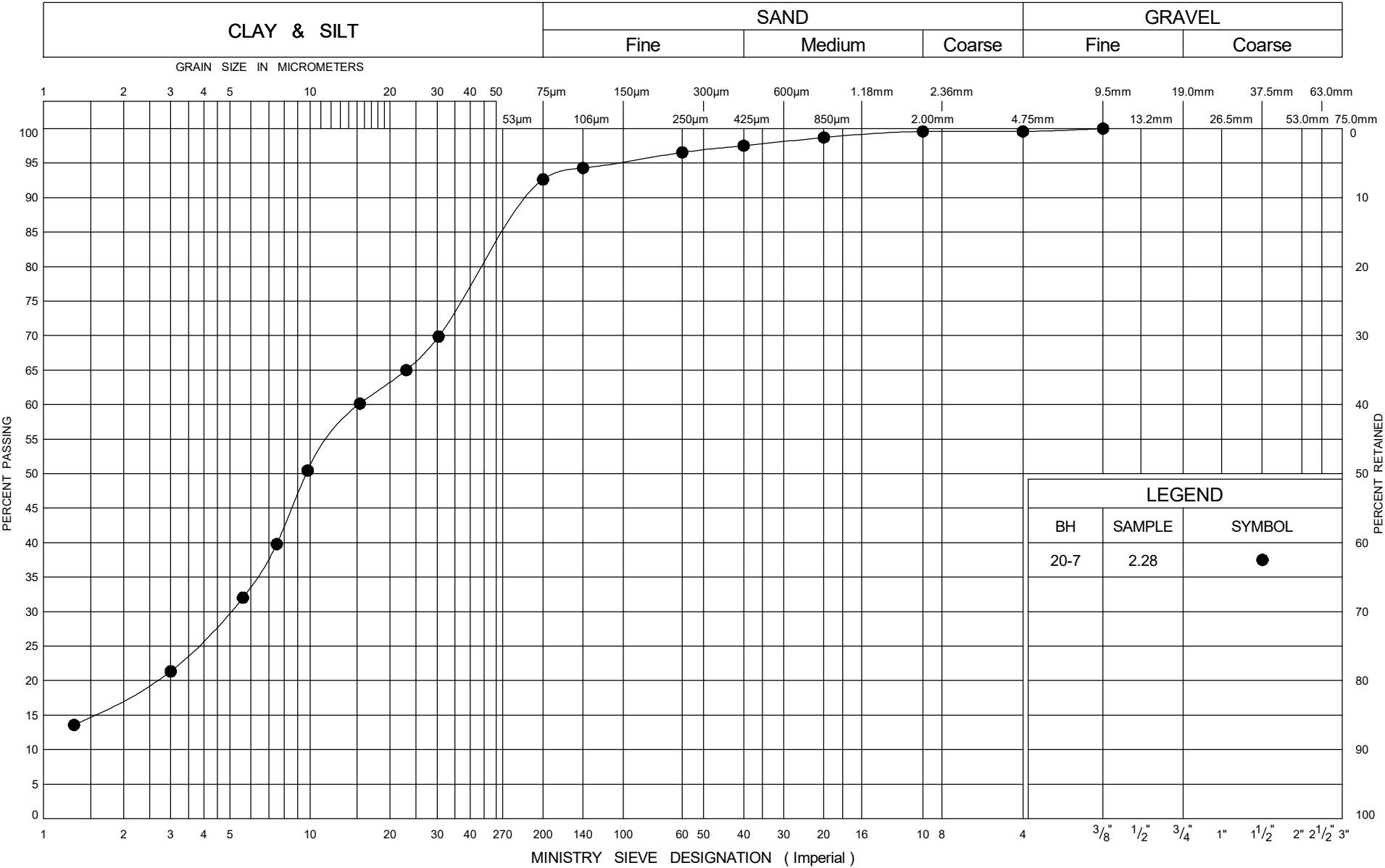
Appendix D –  
Laboratory/In-situ Testing Data and Bedrock Core Photographs

## Laboratory Testing Data

# UNIFIED SOIL CLASSIFICATION SYSTEM



UNIFIED SOIL CLASSIFICATION SYSTEM

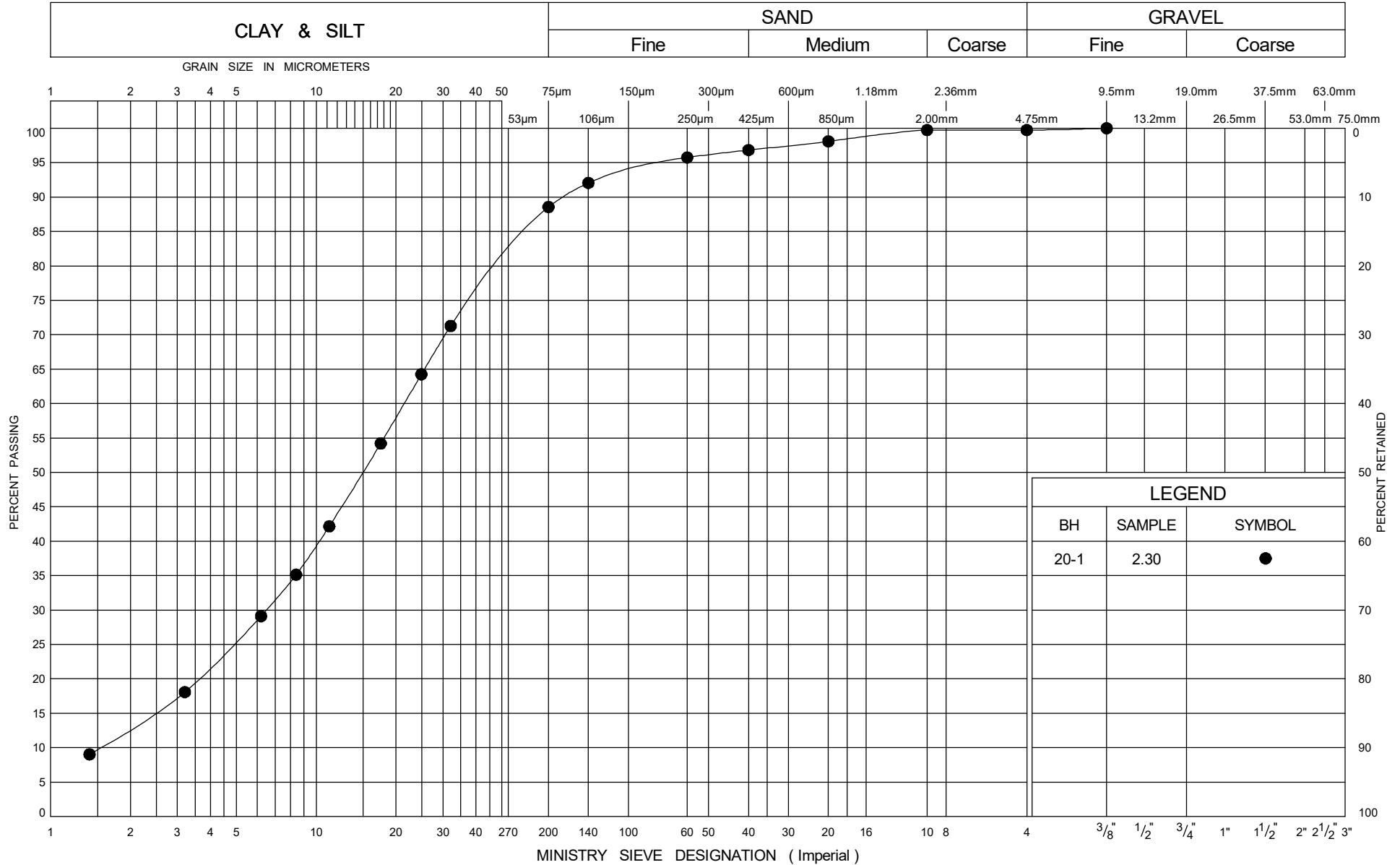


GRAIN SIZE DISTRIBUTION

Fill: Silt

FIG No 2  
W P Agreement No. 5018-E-0012  
Driftwood River Brige

# UNIFIED SOIL CLASSIFICATION SYSTEM



Ministry of  
Transportation

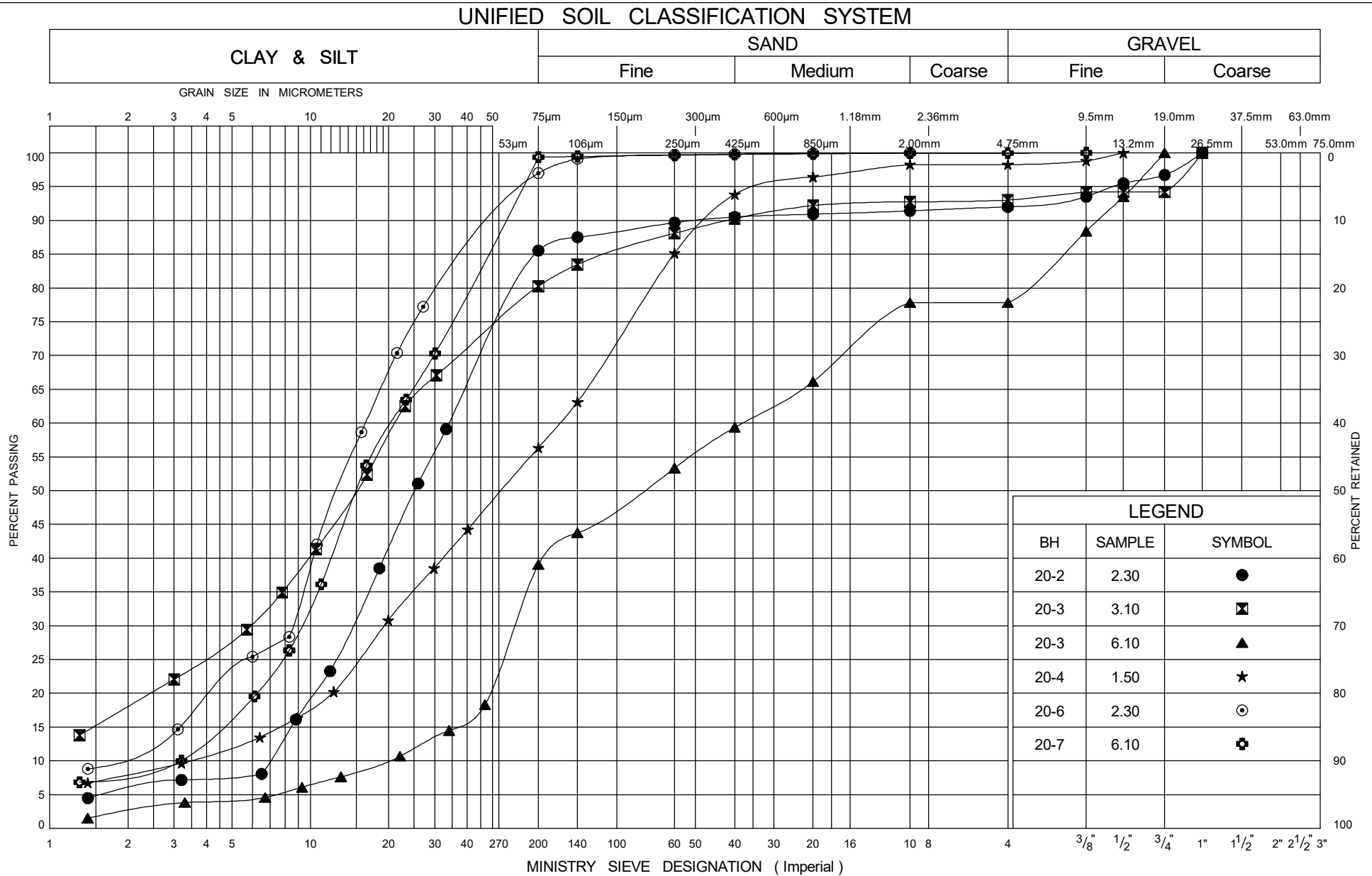
## GRAIN SIZE DISTRIBUTION

Organic Silt

FIG No 3

W P Agreement No. 5018-E-0012

Driftwood River Brige



Ministry of  
Transportation

## GRAIN SIZE DISTRIBUTION

Silt to Sand and Silt

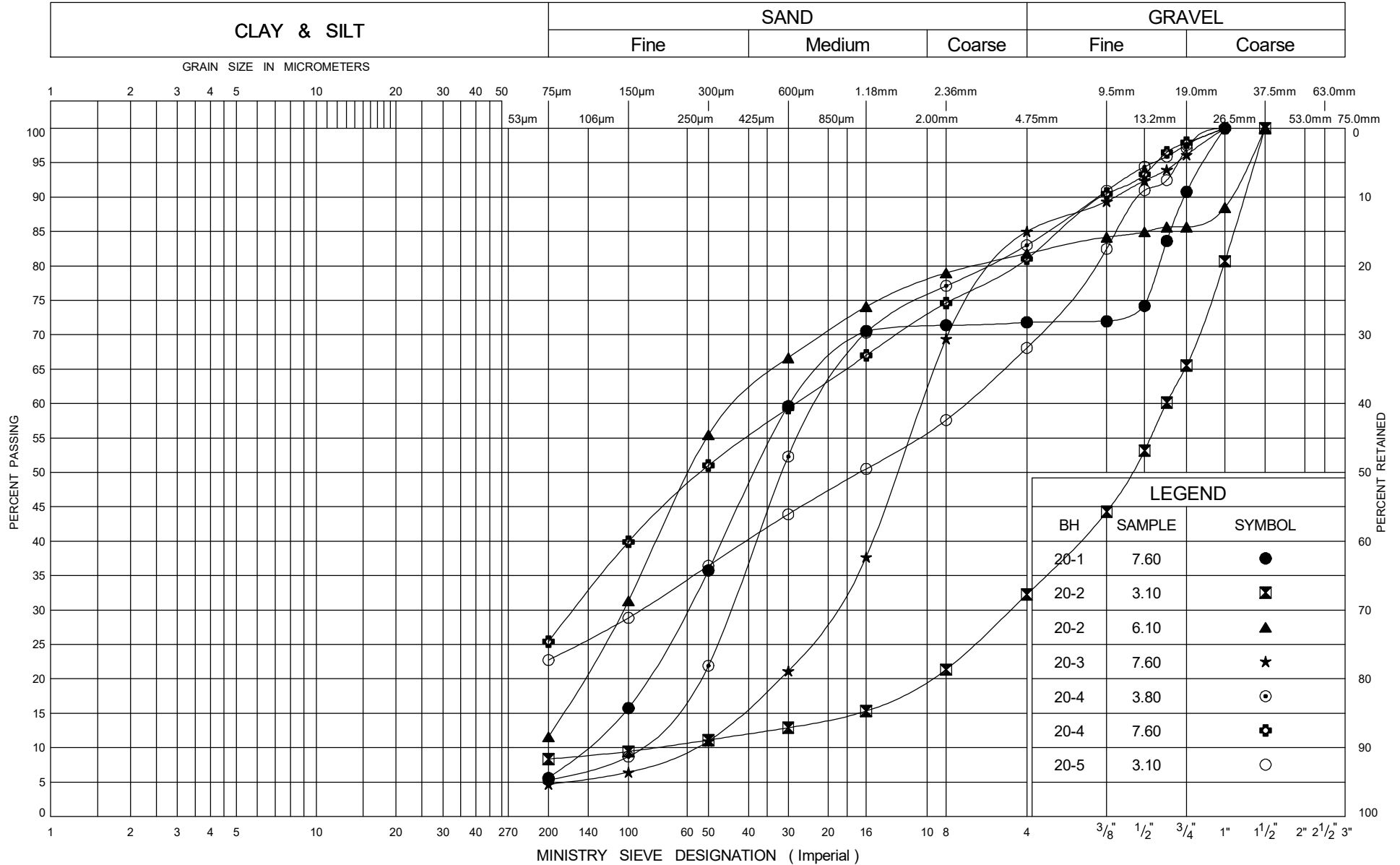
FIG No 4

W P Agreement No. 5018-E-0012

Driftwood River Brige



# UNIFIED SOIL CLASSIFICATION SYSTEM



Ministry of  
Transportation

GRAIN SIZE DISTRIBUTION  
Sand to Gravelly Sand to Sandy Gravel

FIG No 5

W P Agreement No. 5018-E-0012

Driftwood River Brige



**EXP Services Inc.**  
885 Regent Street  
Sudbury, Ontario  
P3E 5M4  
Telephone: (705) 674-9681  
Facsimile: (705) 674-8271

**SUMMARY OF ROCK CORE TEST DATA**  
**ASTM D7012 - 14 (Method C)**

**CLIENT:** Ministry of Transportation Ontario  
**JOB NUMBER:** ADM-00257843-GO  
**JOB NAME:** 5018-E-0012 Assignment #7 - Highway 11 a

**DATE:** September 8, 2020

LAB No.	20874	20875
CORE LOCATION	20-1	20-4
DEPTH	34' 9"-35' 1 1/2"	41' 2 3/4"- 41'-7 1/4"
DATE TESTED	8-Sep-20	8-Sep-20
LENGTH (mm)	109.0	107.0
DIAMETER (mm)	47.5	47.5
DENSITY (kg/m <sup>3</sup> )	2599	2706
COMPRESSIVE STRENGTH (MPa)	<b>100.7</b>	<b>85.3</b>
TYPE OF FRACTURE	SHEAR	SHEAR
CONDITION AT TIME OF TESTING	DRY	DRY

**COMMENTS:**

**DISTRIBUTION:**

**CLIENT NAME: EXP. SERVICES INC.  
885 REGENT ST  
SUDBURY, ON P3E5M4  
(705) 674-9681**

**ATTENTION TO: Ian MacMillan**

**PROJECT: 20U644156**

**AGAT WORK ORDER: 20T645717**

**SOLID ANALYSIS REVIEWED BY: Sherin Moussa, Senior Technician**

**DATE REPORTED: Sep 04, 2020**

**PAGES (INCLUDING COVER): 5**

Should you require any information regarding this analysis please contact your client services representative at (905) 501-9998

**\*NOTES**



**AGAT** Laboratories

## Certificate of Analysis

AGAT WORK ORDER: 20T645717

PROJECT: 20U644156

5623 McADAM ROAD  
MISSISSAUGA, ONTARIO  
CANADA L4Z 1N9  
TEL (905)501-9998  
FAX (905)501-0589  
<http://www.agatlabs.com>

CLIENT NAME: EXP. SERVICES INC.

ATTENTION TO: Ian MacMillan

### (201-042) Sulfide

DATE SAMPLED: Sep 01, 2020

DATE RECEIVED: Sep 02, 2020

DATE REPORTED: Sep 04, 2020

SAMPLE TYPE: Other

Analyte: Sulfide

Unit: %

Sample ID (AGAT ID) RDL: 0.05

BH 20-3 SS4-1400119 (1410806) <0.05

BH 20-3 SS4-1400119-DUP (1410807) <0.05

Comments: RDL - Reported Detection Limit

Analysis performed at AGAT 5623 McAdam Rd., Mississauga, ON (unless marked by \*)

**Certified By:**

*Sherin Hoossaf*



**AGAT** Laboratories

**Quality Assurance - Replicate**

**AGAT WORK ORDER: 20T645717**

**PROJECT: 20U644156**

5623 McADAM ROAD  
MISSISSAUGA, ONTARIO  
CANADA L4Z 1N9  
TEL (905)501-9998  
FAX (905)501-0589  
<http://www.agatlabs.com>

**CLIENT NAME: EXP. SERVICES INC.**

**ATTENTION TO: Ian MacMillan**

**(201-042) Sulfide**

Parameter	REPLICATE #1				REPLICATE #2											
	Sample ID	Original	Replicate	RPD	Sample ID	Original	Replicate	RPD								
S	1410806	0.010	0.011	9.5%	1410807	0.010	0.010	0.0%								
Sulfate	1410806	< 0.01	< 0.01	0.0%	1410807	< 0.01	< 0.01	0.0%								
Sulfide	1410806	< 0.05	< 0.05	0.0%	1410807	< 0.05	< 0.05	0.0%								



**AGAT** Laboratories

**Quality Assurance - Certified Reference materials**

**AGAT WORK ORDER: 20T645717**

**PROJECT: 20U644156**

5623 McADAM ROAD  
MISSISSAUGA, ONTARIO  
CANADA L4Z 1N9  
TEL (905)501-9998  
FAX (905)501-0589  
<http://www.agatlabs.com>

**CLIENT NAME: EXP. SERVICES INC.**

**ATTENTION TO: Ian MacMillan**

**(201-042) Sulfide**

Parameter	CRM #1				CRM #2											
	Expect	Actual	Recovery	Limits	Expect	Actual	Recovery	Limits								
S	0.80	0.80	100%	90% - 110%	0.80	0.80	100%	90% - 110%								
Sulfate	0.01	0.01	100%	90% - 110%	0.01	0.01	100%	90% - 110%								
Sulfide	0.80	0.79	98%	90% - 110%	0.80	0.79	98%	90% - 110%								

## Method Summary

CLIENT NAME: EXP. SERVICES INC.

AGAT WORK ORDER: 20T645717

PROJECT: 20U644156

ATTENTION TO: Ian MacMillan

SAMPLING SITE:

SAMPLED BY:

PARAMETER	AGAT S.O.P	LITERATURE REFERENCE	ANALYTICAL TECHNIQUE
Solid Analysis			
Sulfide	MIN-200-12037		LECO



**CLIENT NAME: EXP. SERVICES INC.**  
**885 REGENT ST**  
**SUDBURY, ON P3E5M4**  
**(705) 674-9681**

**ATTENTION TO: Ian MacMillan**

**PROJECT: ADM-00257843-G0**

**AGAT WORK ORDER: 20U644156**

**SOIL ANALYSIS REVIEWED BY: Nivine Basily, Inorganics Report Writer**

**DATE REPORTED: Sep 04, 2020**

**PAGES (INCLUDING COVER): 5**

**VERSION\*: 1**

Should you require any information regarding this analysis please contact your client services representative at (905) 712-5100

**\*Notes**

**Disclaimer:**

- All work conducted herein has been done using accepted standard protocols, and generally accepted practices and methods. AGAT test methods may incorporate modifications from the specified reference methods to improve performance.
- All samples will be disposed of within 30 days following analysis, unless expressly agreed otherwise in writing. Please contact your Client Project Manager if you require additional sample storage time.
- AGAT's liability in connection with any delay, performance or non-performance of these services is only to the Client and does not extend to any other third party. Unless expressly agreed otherwise in writing, AGAT's liability is limited to the actual cost of the specific analysis or analyses included in the services.
- This Certificate shall not be reproduced except in full, without the written approval of the laboratory.
- The test results reported herewith relate only to the samples as received by the laboratory.
- Application of guidelines is provided "as is" without warranty of any kind, either expressed or implied, including, but not limited to, warranties of merchantability, fitness for a particular purpose, or non-infringement. AGAT assumes no responsibility for any errors or omissions in the guidelines contained in this document.
- All reportable information as specified by ISO/IEC 17025:2017 is available from AGAT Laboratories upon request.



# AGAT Laboratories

## Certificate of Analysis

AGAT WORK ORDER: 20U644156

PROJECT: ADM-00257843-G0

5835 COOPERS AVENUE  
MISSISSAUGA, ONTARIO  
CANADA L4Z 1Y2  
TEL (905)712-5100  
FAX (905)712-5122  
<http://www.agatlabs.com>

CLIENT NAME: EXP. SERVICES INC.

SAMPLING SITE:

ATTENTION TO: Ian MacMillan

SAMPLED BY:

### Corrosivity Package

DATE RECEIVED: 2020-08-28

DATE REPORTED: 2020-09-04

		SAMPLE DESCRIPTION: BH 20-3 SS4	
		SAMPLE TYPE: Solid	
		DATE SAMPLED: 2020-08-07	
Parameter	Unit	G / S	RDL
			1400119
Chloride (2:1)	µg/g	2	186
Sulphate (2:1)	µg/g	2	13
pH (2:1)	pH Units	NA	8.92
Electrical Conductivity (2:1)	mS/cm	0.005	0.509
Resistivity (2:1) (Calculated)	ohm.cm	1	1960
Redox Potential 1	mV	NA	56
Redox Potential 2	mV	NA	61
Redox Potential 3	mV	NA	50

**Comments:** RDL - Reported Detection Limit; G / S - Guideline / Standard

**1400119** EC, pH, Chloride and Sulphate were determined on the extract obtained from the 2:1 leaching procedure (2 parts DI water: 1 part soil). Resistivity is a calculated parameter. Redox potential measured on as received sample. Due to the potential for rapid change in sample equilibrium chemistry with exposure to oxidative/reduction conditions laboratory results may differ from field measured results. Redox potential measurement in soil is quite variable and non reproducible due in part, to the general heterogeneity of a given soil. It is also related to the introduction of increased oxygen into the sample after extraction. The interpretation of soil redox potential should be considered in terms of its general range rather than as an absolute measurement.

Analysis performed at AGAT Toronto (unless marked by \*)

**Certified By:**



*Nivine Basly*

## Quality Assurance

CLIENT NAME: EXP. SERVICES INC.

PROJECT: ADM-00257843-G0

SAMPLING SITE:

AGAT WORK ORDER: 20U644156

ATTENTION TO: Ian MacMillan

SAMPLED BY:

### Soil Analysis

RPT Date: Sep 04, 2020

			DUPLICATE				REFERENCE MATERIAL			METHOD BLANK SPIKE			MATRIX SPIKE		
PARAMETER	Batch	Sample Id	Dup #1	Dup #2	RPD	Method Blank	Measured Value	Acceptable Limits		Recovery	Acceptable Limits		Recovery	Acceptable Limits	
								Lower	Upper		Lower	Upper		Lower	Upper

#### Corrosivity Package

Chloride (2:1)	1400119	1400119	186	186	0.0%	< 2	92%	70%	130%	96%	80%	120%	101%	70%	130%
Sulphate (2:1)	1400119	1400119	13	13	0.0%	< 2	101%	70%	130%	103%	80%	120%	109%	70%	130%
pH (2:1)	1400119	1400119	8.92	8.93	0.1%	NA	97%	90%	110%						
Electrical Conductivity (2:1)	1400119	1400119	0.509	0.511	0.4%	< 0.005	101%	80%	120%						
Redox Potential 1		1					100%	90%	110%						

Comments: NA signifies Not Applicable.

pH duplicates QA acceptance criteria was met relative as stated in Table 5-15 of Analytical Protocol document.

### Certified By:


*Nivine Basily*

## Method Summary

CLIENT NAME: EXP. SERVICES INC.

PROJECT: ADM-00257843-G0

SAMPLING SITE:

AGAT WORK ORDER: 20U644156

ATTENTION TO: Ian MacMillan

SAMPLED BY:

PARAMETER	AGAT S.O.P	LITERATURE REFERENCE	ANALYTICAL TECHNIQUE
<b>Soil Analysis</b>			
Chloride (2:1)	INOR-93-6004	modified from SM 4110 B	ION CHROMATOGRAPH
Sulphate (2:1)	INOR-93-6004	modified from SM 4110 B	ION CHROMATOGRAPH
pH (2:1)	INOR 93-6031	MSA part 3 & SM 4500-H+ B	PH METER
Electrical Conductivity (2:1)	INOR-93-6036	modified from MSA PART 3, CH 14 and SM 2510 B	EC METER
Resistivity (2:1) (Calculated)	INOR-93-6036	McKeague 4.12, SM 2510 B, SSA #5 Part 3	CALCULATION
Redox Potential 1	INOR-93-6066	modified G200-09, SM 2580 B	REDOX POTENTIAL ELECTRODE
Redox Potential 2	INOR-93-6066	modified G200-09, SM 2580 B	REDOX POTENTIAL ELECTRODE
Redox Potential 3	INOR-93-6066	modified G200-09, SM 2580 B	REDOX POTENTIAL ELECTRODE





**EXP Services Inc.**  
885 Regent Street  
Sudbury, Ontario  
P3E 5M4  
Telephone: (705) 674-9681  
Facsimile: (705) 674-8271

**SUMMARY OF ROCK CORE TEST DATA**  
**ASTM D7012 - 14 (Method C)**

**CLIENT:** Ministry of Transportation Ontario  
**JOB NUMBER:** ADM-00257843-GO  
**JOB NAME:** 5018-E-0012 Assignment #7 - Highway 11 a

**DATE:** September 8, 2020

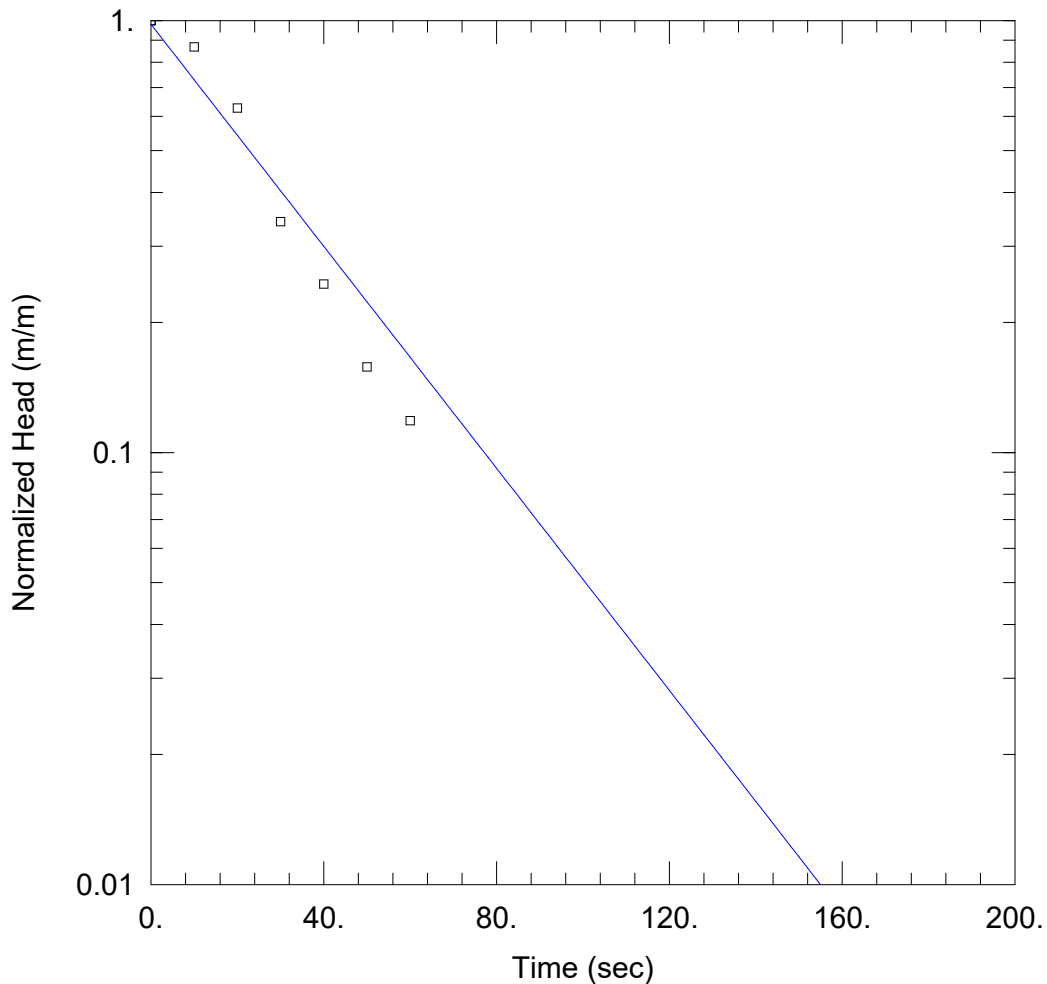
LAB No.	20874	20875
CORE LOCATION	20-1	20-4
DEPTH	34' 9"-35' 1 1/2"	41' 2 3/4"- 41'-7 1/4"
DATE TESTED	8-Sep-20	8-Sep-20
LENGTH (mm)	109.0	107.0
DIAMETER (mm)	47.5	47.5
DENSITY (kg/m <sup>3</sup> )	2599	2706
COMPRESSIVE STRENGTH (MPa)	<b>100.7</b>	<b>85.3</b>
TYPE OF FRACTURE	SHEAR	SHEAR
CONDITION AT TIME OF TESTING	DRY	DRY

**COMMENTS:**

**DISTRIBUTION:**

In-situ Testing Data





### SWRT - MW-18-1- FALLING HEAD

Data Set: C:\...\MW-18-1 - Falling.aqt

Date: 09/22/20

Time: 14:09:35

### PROJECT INFORMATION

Company: Exp Services Inc.

Client: Great Gulf Enterprise

Project: BRM-00257843-G0

Location: HWY 11 Bridge Widening

Test Well: MW-204

Test Date: August 27, 2020

### AQUIFER DATA

Saturated Thickness: 7.252 m

Anisotropy Ratio ( $K_z/K_r$ ): 1.

### WELL DATA (MW-18-1)

Initial Displacement: 0.228 m

Static Water Column Height: 7.252 m

Total Well Penetration Depth: 7.252 m

Screen Length: 3. m

Casing Radius: 0.0254 m

Well Radius: 0.0254 m

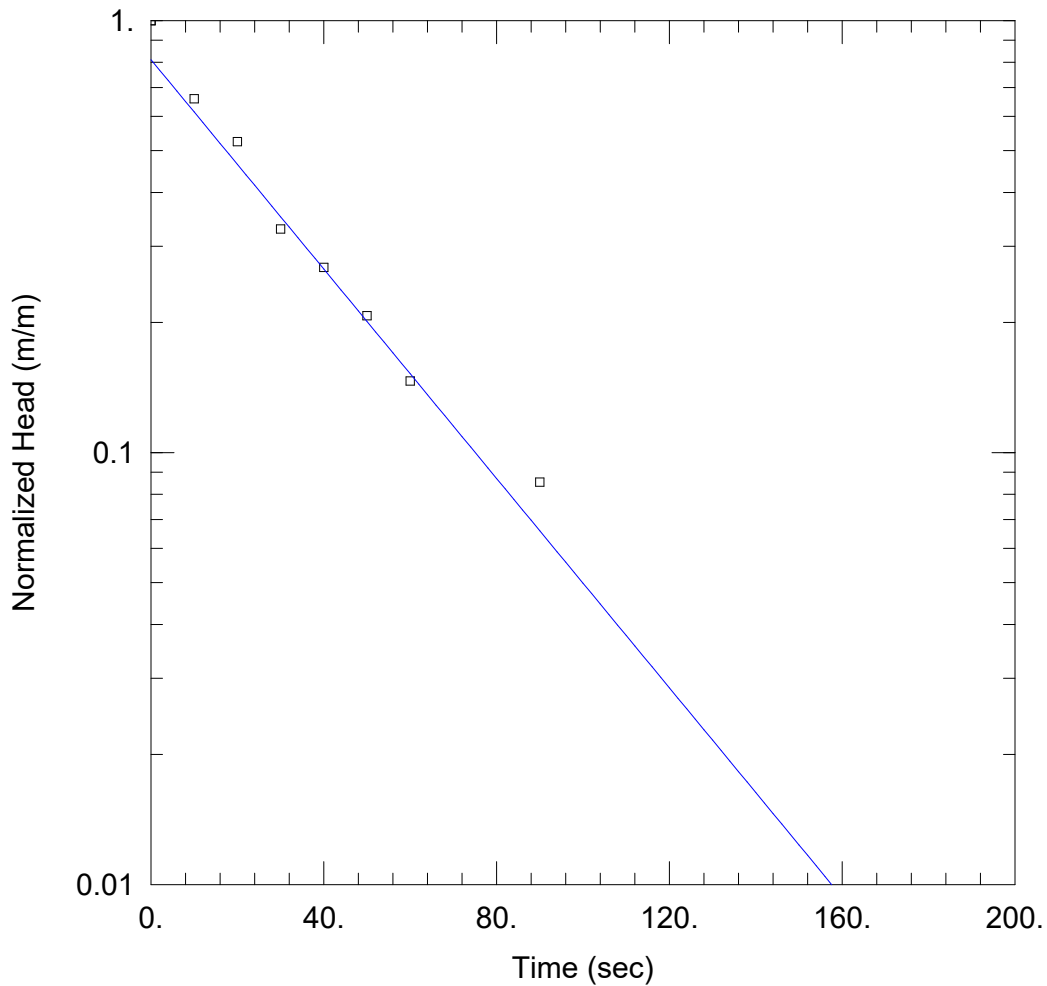
### SOLUTION

Aquifer Model: Unconfined

Solution Method: Hvorslev

$K = 1.738E-5$  m/sec

$y_0 = 0.2234$  m



#### SWRT - MW-18-1- RISING HEAD

Data Set: C:\...\MW-18-1 - Rising.aqt  
Date: 09/29/20

Time: 08:57:09

#### PROJECT INFORMATION

Company: Exp Services Inc.  
Client: Great Gulf Enterprise  
Project: BRM-00257843-G0  
Location: HWY 11 Bridge Widening  
Test Well: MW-204  
Test Date: August 27, 2020

#### AQUIFER DATA

Saturated Thickness: 7.252 m

Anisotropy Ratio (Kz/Kr): 1.

#### WELL DATA (MW-18-1)

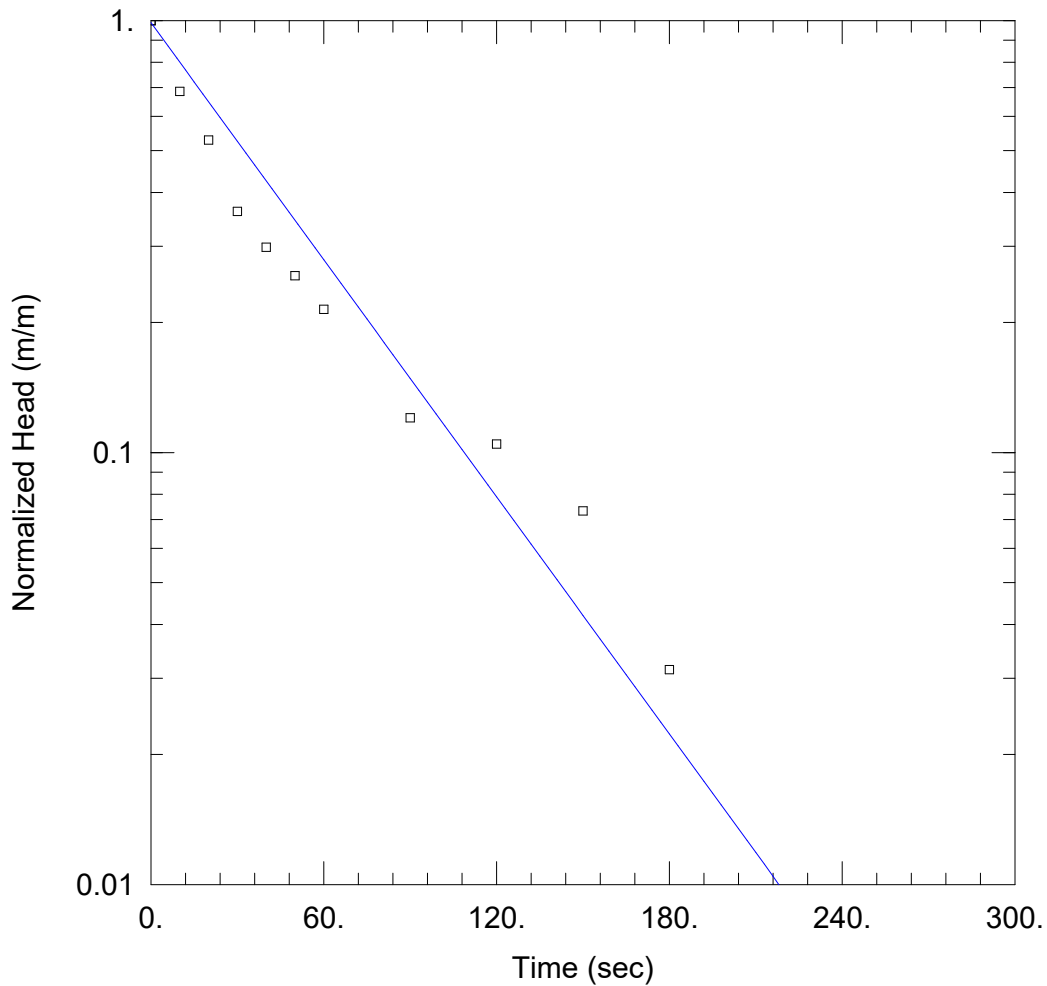
Initial Displacement: 0.082 m  
Total Well Penetration Depth: 7.252 m  
Casing Radius: 0.0254 m

Static Water Column Height: 7.252 m  
Screen Length: 3. m  
Well Radius: 0.0254 m

#### SOLUTION

Aquifer Model: Unconfined  
K = 1.64E-5 m/sec

Solution Method: Hvorslev  
y0 = 0.06655 m



#### SWRT - MW-20-4 - FALLING HEAD

Data Set: C:\...\MW-20-4 - Falling.aqt

Date: 09/17/20

Time: 16:10:51

#### PROJECT INFORMATION

Company: Exp Services Inc.

Client: Great Gulf Enterprise

Project: BRM-00257843-G0

Location: HWY 11 Bridge Widening

Test Well: MW-204

Test Date: August 27, 2020

#### AQUIFER DATA

Saturated Thickness: 3.854 m

Anisotropy Ratio ( $K_z/K_r$ ): 1.

#### WELL DATA (MW-20-4)

Initial Displacement: 0.191 m

Static Water Column Height: 3.854 m

Total Well Penetration Depth: 3.854 m

Screen Length: 3. m

Casing Radius: 0.0254 m

Well Radius: 0.0254 m

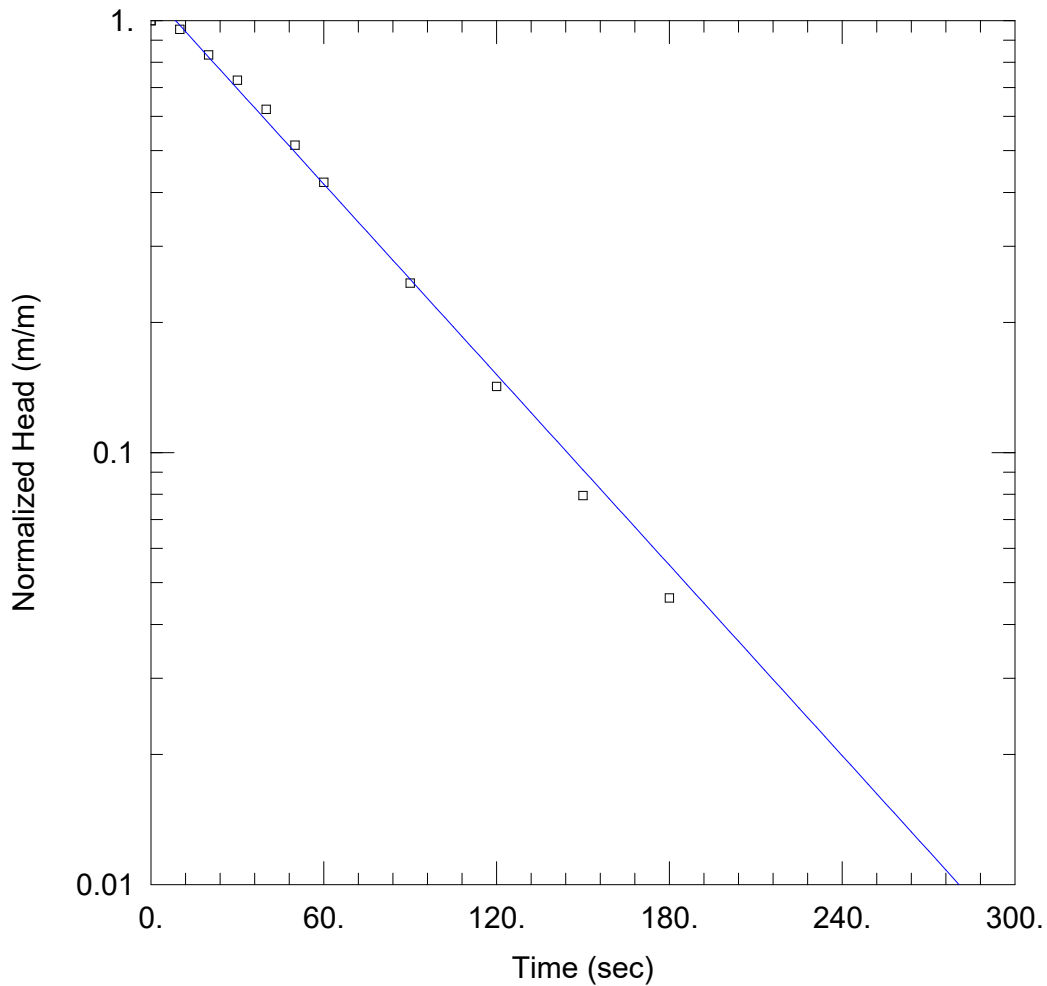
#### SOLUTION

Aquifer Model: Unconfined

Solution Method: Hvorslev

$K = 1.238E-5$  m/sec

$y_0 = 0.1888$  m



#### SWRT - MW-20-4 - RISING HEAD

Data Set: C:\...\MW-20-4 - Rising.aqt

Date: 09/17/20

Time: 16:40:04

#### PROJECT INFORMATION

Company: Exp Services Inc.

Client: Great Gulf Enterprise

Project: BRM-00257843-G0

Location: HWY 11 Bridge Widening

Test Well: MW-204

Test Date: August 27, 2020

#### AQUIFER DATA

Saturated Thickness: 3.854 m

Anisotropy Ratio ( $K_z/K_r$ ): 1.

#### WELL DATA (MW-20-4)

Initial Displacement: 0.239 m

Static Water Column Height: 3.854 m

Total Well Penetration Depth: 3.854 m

Screen Length: 3. m

Casing Radius: 0.0254 m

Well Radius: 0.0254 m

#### SOLUTION

Aquifer Model: Unconfined

Solution Method: Hvorslev

$K = 9.946E-6$  m/sec

$y_0 = 0.276$  m

BOREHOLE 20-4 (12.8 to 14.0 m)									
T min	dt s	10 psi		20 psi		30 psi		40 psi	
		V m³	Q L/s	V m³	Q L/s	V m³	Q L/s	V m³	Q L/s
0	0	3.60		3.60		3.60		3.62	
1	60	3.60	0.0E+00	3.60	0.0E+00	3.60	0.0E+00	3.62	0.0E+00
2	60	3.60	0.0E+00	3.60	0.0E+00	3.60	0.0E+00	3.62	1.7E-02
3	60	3.60	0.0E+00	3.60	0.0E+00	3.60	0.0E+00	3.62	0.0E+00
5	120	3.60	0.0E+00	3.60	0.0E+00	3.60	0.0E+00	3.62	8.3E-03
10	300	3.60	0.0E+00	3.60	0.0E+00	3.60	0.0E+00	3.62	6.7E-03
Q average			0.0E+00		0.0E+00		0.0E+00		6.3E-03

K geometric mean	PSI	Q L/s	Pg m	Pi m	T m²/s	K m/s	m/s
	10	0.0E+00	7.03	9.61	0.0E+00	0E+00	
	20	0.0E+00	14.06	16.64	0.0E+00	0E+00	
	30	0.0E+00	21.09	23.67	0.0E+00	0E+00	
	40	6.3E-03	28.12	30.70	2.5E-07	2E-07	
2.1E-07							

R	10.00	m	radius of influence		m	
rb	0.005	m	radius of borehole		12.80	Depth to bottom of top packer
hg	0.12	m	height of the gauge above ground level		14.00	Depth to top of bottom packer
hs	2.46	m	depth to pre-test water level		1.20	Length of test section
hf	0.00	m	friction losses			

P		psi	pressure	
T		min	time	
dt		min or s	elapsed time	
Pg		m	gauge pressure	
Pi		m	net injection pressure	
V		ft³ or m³	volume	
Q		ft³/min or L/s	flow rate	
T		m²/s	transmissivity	
K		m/s	hydraulic conductivity	

Thiem Equation

$$T = \frac{Q \cdot \ln(\frac{R}{r_b})}{2\pi P_i}$$

$$K = \frac{T}{Lenght\ of\ Test\ Interval}$$

## Bedrock Core Photographs



Photograph D1. Rock cores from BH20-1



Photograph D2. Rock cores from BH20-4



Photograph D3. Rock cores from BH20-5



## Appendix E – Slope Stability Analyses

5018-E-0012 - Work Item No. 7  
 North Driftwood River Bridge  
 West Abutment  
 Drained Static Condition

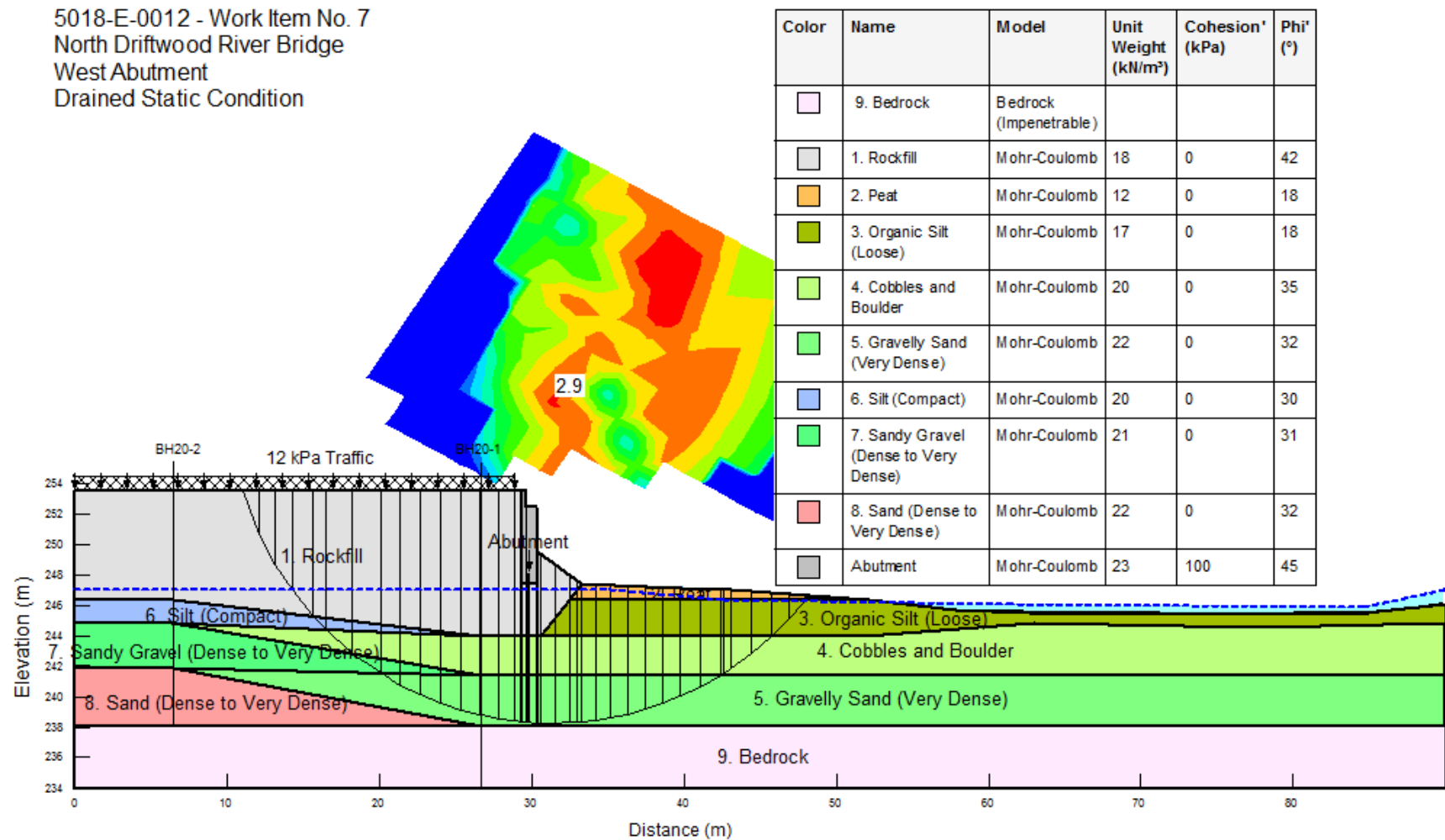


Figure E1. Slope stability analysis for west abutment – drained static condition with H-piles

5018-E-0012 - Work Item No. 7  
 North Driftwood River Bridge  
 West Abutment  
 Drained Seismic Condition

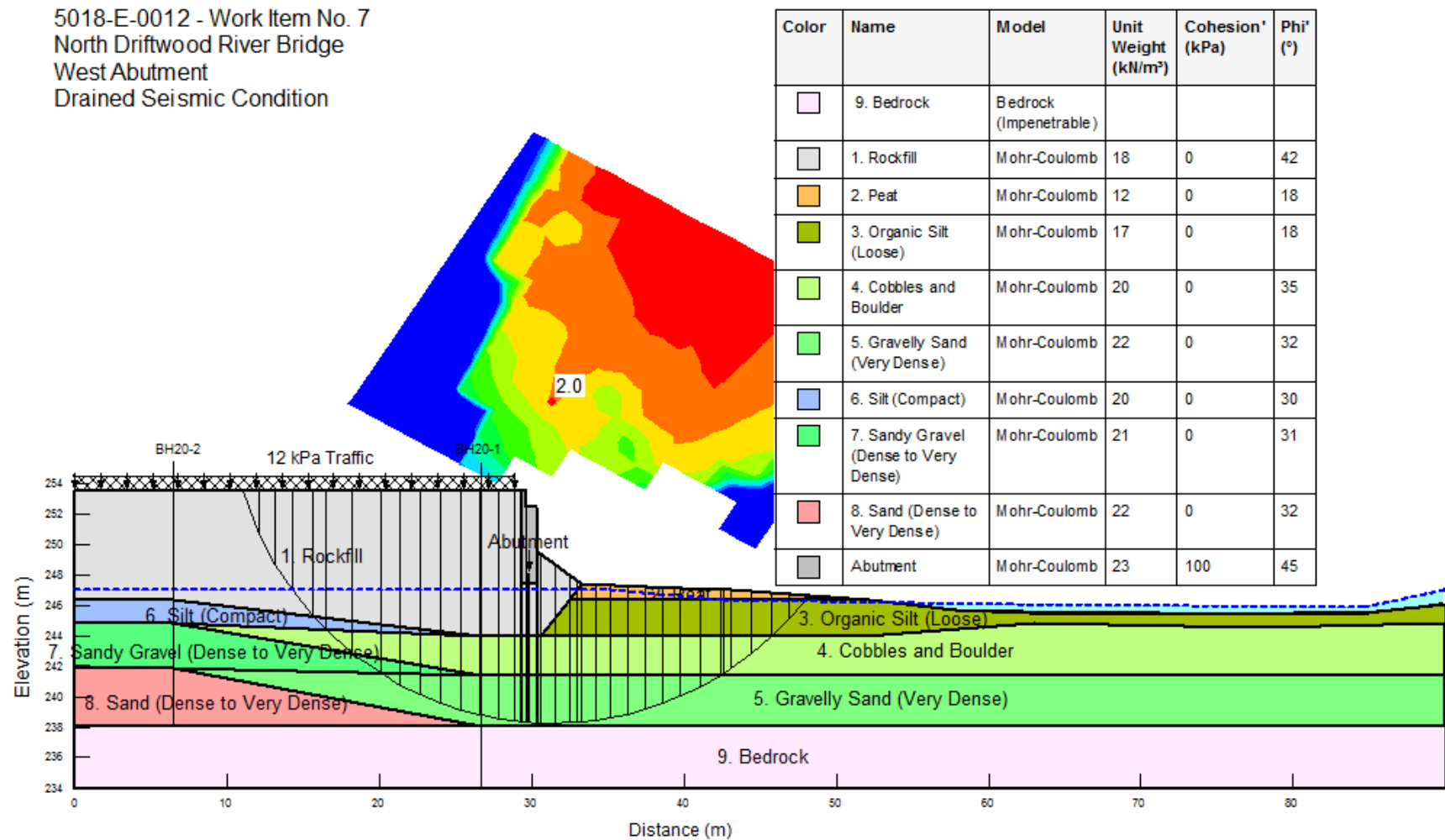


Figure E2. Slope stability analysis for west abutment – drained seismic condition with H-piles

5018-E-0012 - Work Item No. 7  
 North Driftwood River Bridge  
 West Embankment Slope  
 Drained Static Condition

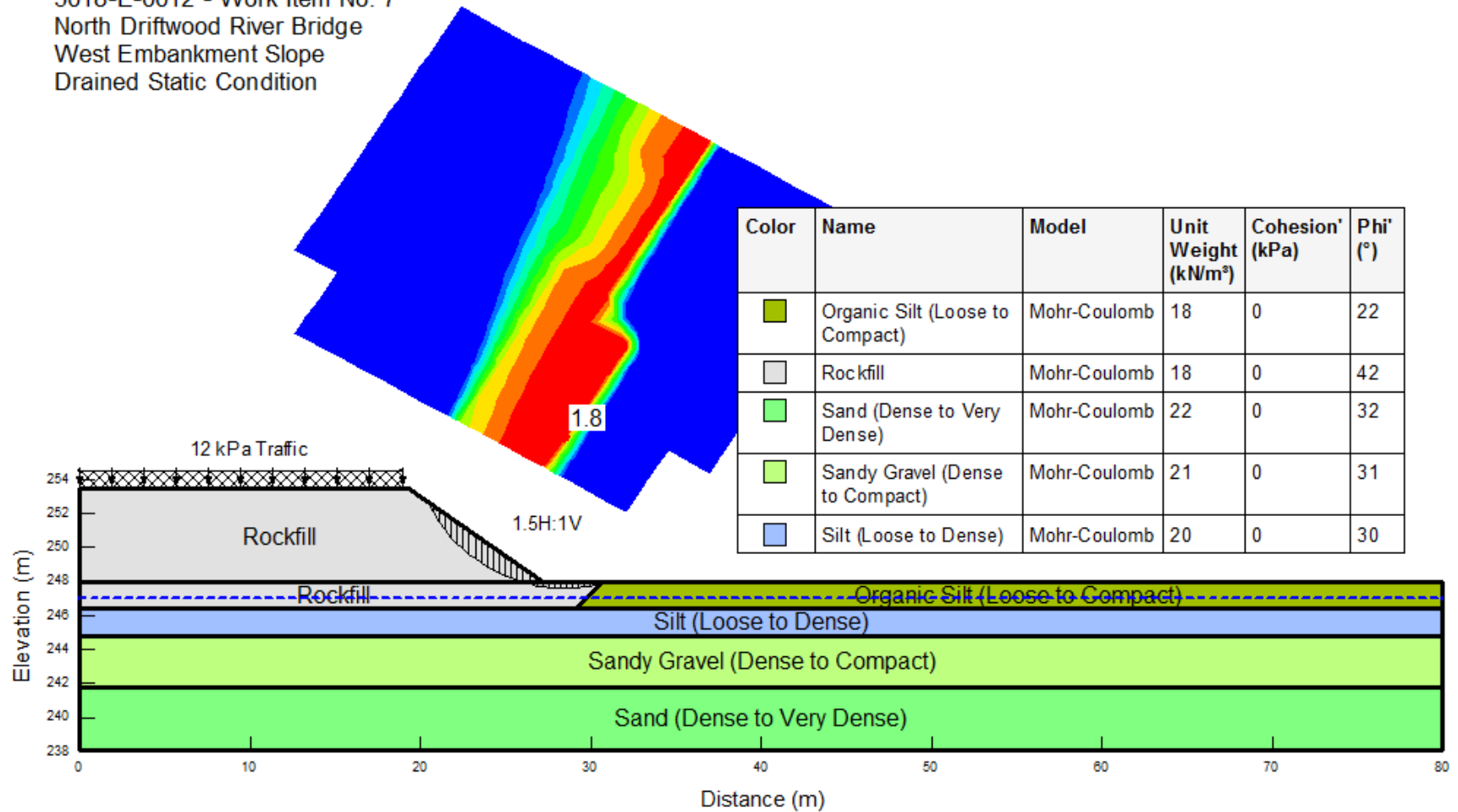


Figure E3. Slope stability analysis for west approach embankment – drained static condition

5018-E-0012 - Work Item No. 7  
 North Driftwood River Bridge  
 West Embankment Slope  
 Drained Seismic Condition

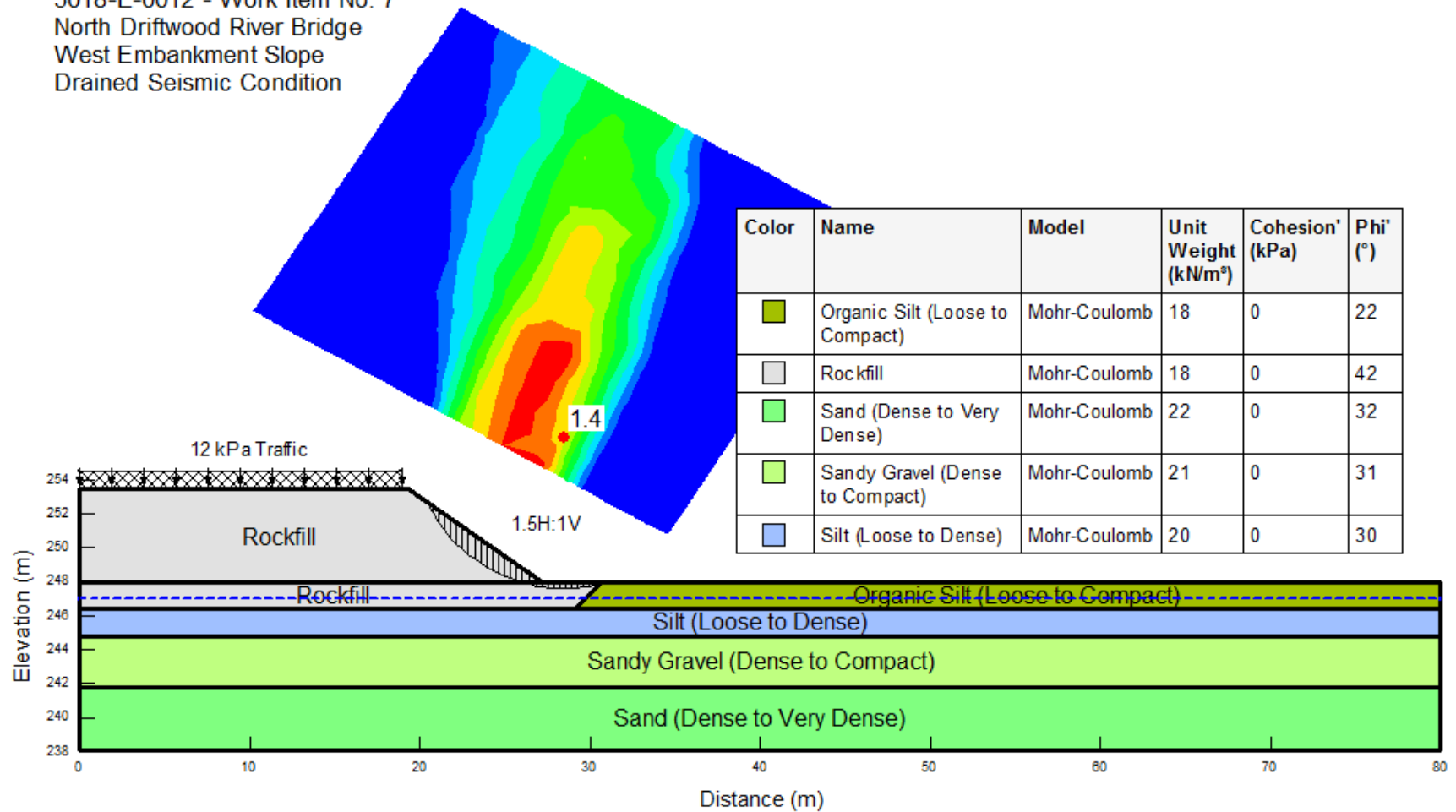


Figure E4. Slope stability analysis for west approach embankment – drained seismic condition

## Appendix F – Settlement Analyses

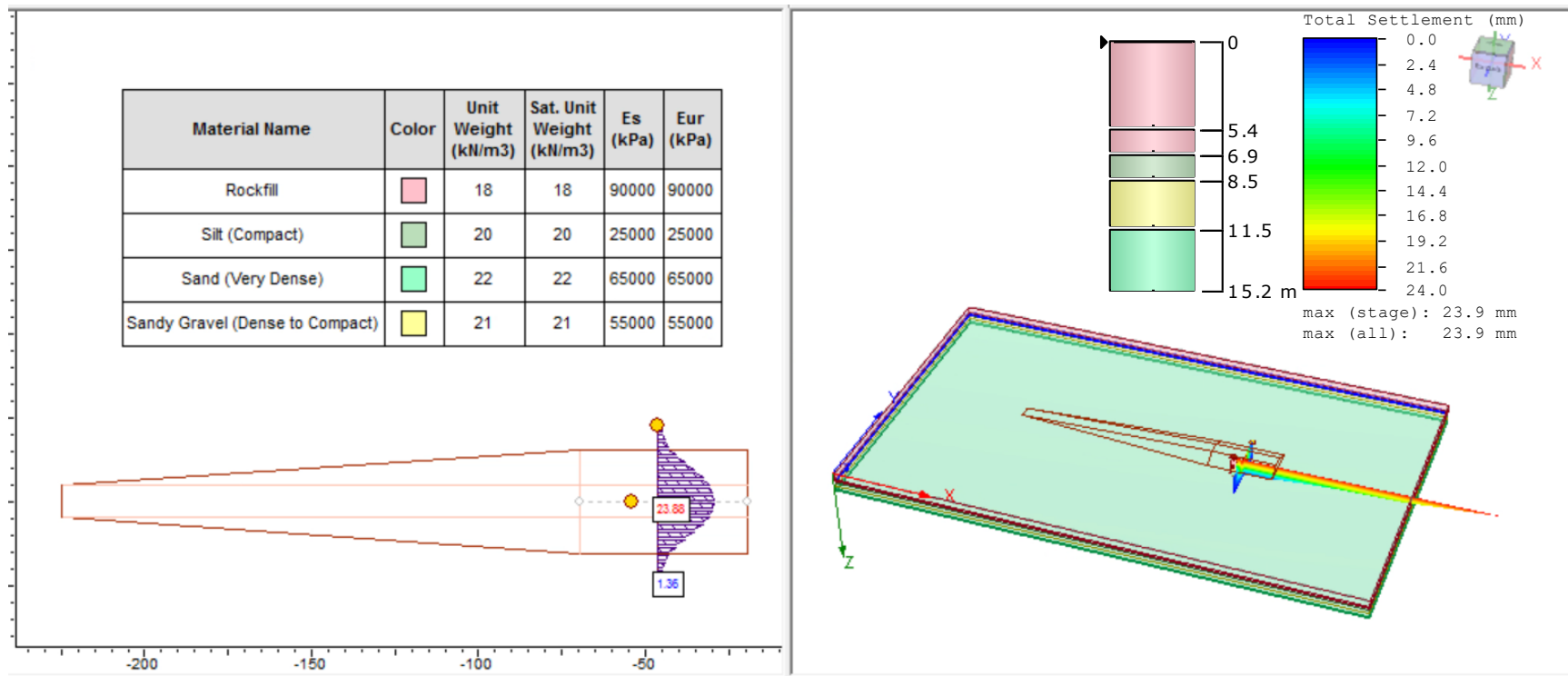


Figure F1: West Embankment – Total Settlement



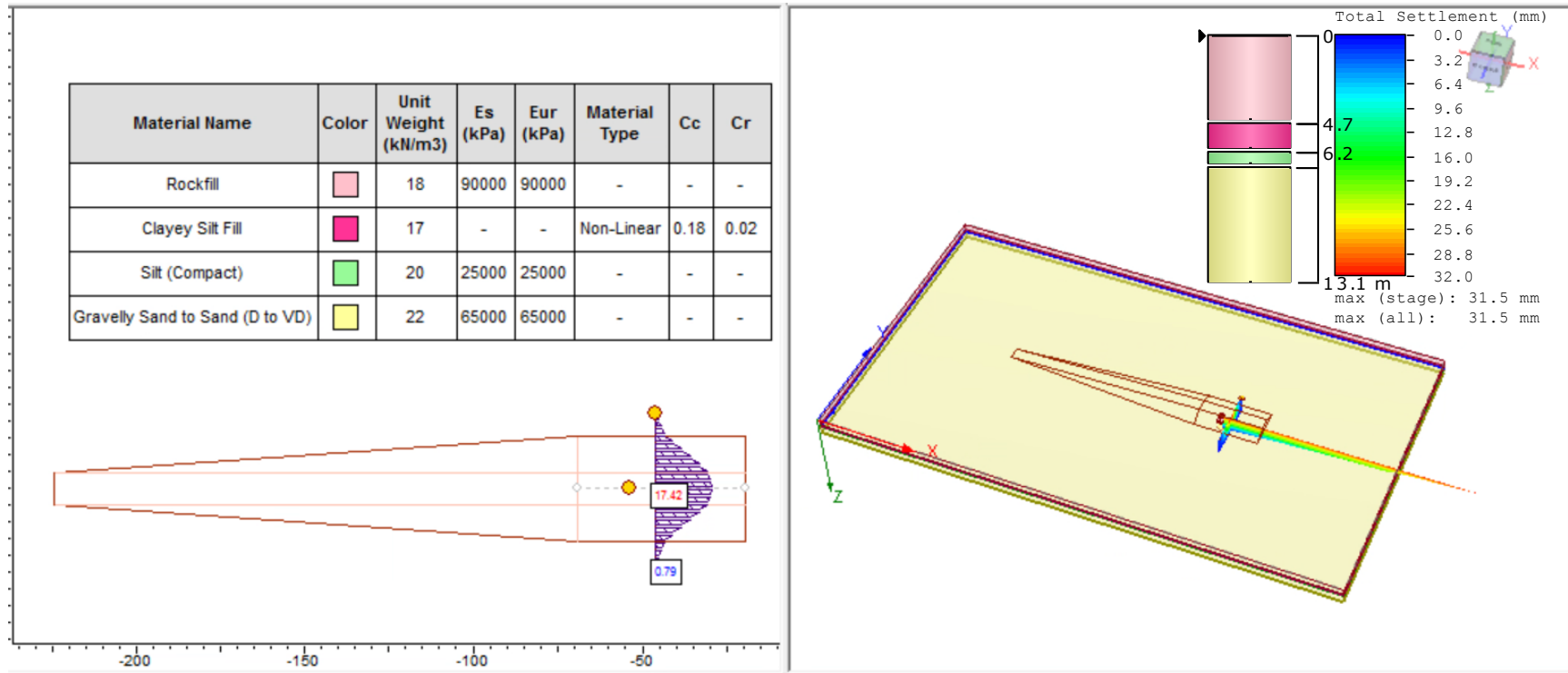


Figure F2: East Embankment – Total Settlement

## Appendix G – 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: 49.189N 81.438W

2020-09-29 13:09 UT

Requested by: exp

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)	0.215	0.099	0.046	0.007
Sa (0.1)	0.257	0.125	0.063	0.012
Sa (0.2)	0.209	0.104	0.056	0.013
Sa (0.3)	0.153	0.078	0.044	0.012
Sa (0.5)	0.101	0.055	0.033	0.009
Sa (1.0)	0.049	0.028	0.017	0.004
Sa (2.0)	0.022	0.013	0.007	0.002
Sa (5.0)	0.005	0.003	0.002	0.000
Sa (10.0)	0.002	0.001	0.001	0.000
PGA (g)	0.136	0.066	0.033	0.006
PGV (m/s)	0.078	0.041	0.023	0.005

**Notes:** Spectral ( $S_a(T)$ , where  $T$  is the period in seconds) and peak ground acceleration (PGA) values are given in units of  $g$  ( $9.81 \text{ m/s}^2$ ). Peak ground velocity is given in  $\text{m/s}$ . Values are for "firm ground" (NBCC2015 Site Class C, average shear wave velocity  $450 \text{ m/s}$ ). 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](http://www.EarthquakesCanada.ca) and [www.nationalcodes.ca](http://www.nationalcodes.ca) for more information



Natural Resources  
Canada

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Canada

## Appendix H – Previous Investigation Borehole Logs

# LIST OF ABBREVIATIONS

The abbreviations commonly employed on Records of Boreholes, on figures and in the text of the report are as follows:

I. SAMPLE TYPE	III. SOIL DESCRIPTION
AS Auger sample	(a) Non-Cohesive (Cohesionless) Soils
BS Block sample	Compactness N
CS Chunk sample	Condition Blows/300 mm or Blows/ft
DS Denison type sample	Very loose 0 to 4
FS Foil sample	Loose 4 to 10
RC Rock core	Compact 10 to 30
SC Soil core	Dense 30 to 50
SS Split-spoon	Very dense over 50
ST Slotted tube	
TO Thin-walled, open	
TP Thin-walled, piston	
WS Wash sample	
II. PENETRATION RESISTANCE	(b) Cohesive Soils
Standard Penetration Resistance (SPT), N:	Consistency
The number of blows by a 63.5 kg. (140 lb.) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) drive open sampler for a distance of 300 mm (12 in.)	
Dynamic Cone Penetration Resistance; N <sub>a</sub> :	
The number of blows by a 63.5 kg (140 lb.) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60° cone attached to "A" size drill rods for a distance of 300 mm (12 in.).	
PH: Sampler advanced by hydraulic pressure	
PM: Sampler advanced by manual pressure	
WH: Sampler advanced by static weight of hammer	
WR: Sampler advanced by weight of sampler and rod	
Piezo-Cone Penetration Test (CPT)	
A electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm <sup>2</sup> pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (Q <sub>t</sub> ), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.	
V. MINOR SOIL CONSTITUENTS	IV. SOIL TESTS
Per cent by Weight	w water content
0 to 5	w <sub>p</sub> plastic limit
5 to 12	w <sub>l</sub> liquid limit
12 to 20	C consolidation (oedometer) test
20 to 30	CHEM chemical analysis (refer to text)
over 30	CID consolidated isotropically drained triaxial test <sup>1</sup>
Modifier	CIU consolidated isotropically undrained triaxial test with porewater pressure measurement <sup>1</sup>
Trace	D <sub>R</sub> relative density (specific gravity, G <sub>s</sub> )
Trace to Some (or Little)	DS direct shear test
Some	M sieve analysis for particle size
(ey) or (y)	MH combined sieve and hydrometer (H) analysis
And (non-cohesive (cohesionless)) or	MPC Modified Proctor compaction test
With (cohesive)	SPC Standard Proctor compaction test
	OC organic content test
	SO <sub>4</sub> concentration of water-soluble sulphates
	UC unconfined compression test
	UU unconsolidated undrained triaxial test
	V field vane (LV-laboratory vane test)
	γ unit weight
	Note: 1 Tests which are anisotropically consolidated prior to shear are shown as CAD, CAU.
</	

LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERINGS STATE

**Fresh:** no visible sign of weathering

**Faintly weathered:** weathering limited to the surface of major discontinuities.

**Slightly weathered:** penetrative weathering developed on open discontinuity surfaces but only slight weathering of rock material.

**Moderately weathered:** weathering extends throughout the rock mass but the rock material is not friable.

**Highly weathered:** weathering extends throughout rock mass and the rock material is partly friable.

**Completely weathered:** rock is wholly decomposed and in a friable condition but the rock and structure are preserved.

BEDDING THICKNESS

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

JOINT OR FOLIATION SPACING

Description	Spacing
Very wide	Greater than 3 m
Wide	1 m to 3 m
Moderately close	0.3 m to 1 m
Close	50 mm to 300 mm
Very close	Less than 50 mm

GRAIN SIZE

Term	Size*
Very Coarse Grained	Greater than 60 mm
Coarse Grained	2 mm to 60 mm
Medium Grained	60 microns to 2 mm
Fine Grained	2 microns to 60 microns
Very Fine Grained	Less than 2 microns

Note: \* Grains greater than 60 microns diameter are visible to the naked eye.

CORE CONDITION

**Total Core Recovery (TCR)**

The percentage of solid drill core recovered regardless of quality or length, measured relative to the length of the total core run.

**Solid Core Recovery (SCR)**

The percentage of solid drill core, regardless of length, recovered at full diameter, measured relative to the length of the total core run.

**Rock Quality Designation (RQD)**

The percentage of solid drill core, greater than 100 mm length, recovered at full diameter, measured relative to the length of the total core run. RQD varied from 0% for completely broken core to 100% for core in solid sticks.

DISCONTINUITY DATA

**Fracture Index**

A count of the number of discontinuities (physical separations) in the rock core, including both naturally occurring fractures and mechanically induced breaks caused by drilling.

**Dip with Respect to Core Axis**

The angle of the discontinuity relative to the axis (length) of the core. In a vertical borehole a discontinuity with a 90° angle is horizontal.

**Description and Notes**

An abbreviation description of the discontinuities, whether naturally occurring separations such as fractures, bedding planes and foliation planes or mechanically induced features caused by drilling such as ground or shattered core and mechanically separated bedding or foliation surfaces. Additional information concerning the nature of fracture surfaces and infillings are also noted.

Abbreviations

JN	Joint	PL	Planar
FLT	Fault	CU	Curved
SH	Shear	UN	Undulating
VN	Vein	IR	Irregular
FR	Fracture	K	Slickensided
SY	Stylolite	PO	Polished
BD	Bedding	SM	Smooth
FO	Foliation	SR	Slightly Rough
CO	Contact	RO	Rough
AXJ	Axial Joint	VR	Very Rough
KV	Karstic Void		
MB	Mechanical Break		

PROJECT 18104224 / WO#4										RECORD OF BOREHOLE No 18-1										1 OF 2 METRIC									
G.W.P. 5282-14-00					LOCATION N 5450183.8; E 272857.1 NAD83 MTM ZONE 12 (LAT. 49.188917; LONG. -81.438254)					ORIGINATED BY MA/SK																			
DIST NE HWY 11					BOREHOLE TYPE Wash Boring; NW Casing; NQ Rock Coring					COMPILED BY SB/AMP																			
DATUM Geodetic					DATE October 31, 2018					CHECKED BY JMAC																			
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	WATER CONTENT (%)	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)													
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								GR	SA	SI	CL										
								○ UNCONFINED + FIELD VANE																					
								● QUICK TRIAXIAL × REMOULDED																					
248.7	GROUND SURFACE					20	40	60	80	100																			
0.0	Sandy TOPSOIL (125 mm)																												
0.1	Sandy clayey silt, some gravel (FILL) Firm Brown Moist																												
247.2			1	SS	6																								
1.5	ORGANIC SILT, some gravel, some sand, trace to some clay, trace rootlets, trace wood and shell fragments Very loose to compact Dark brown to grey Moist to wet		2	SS	3																								
			3	SS	20																								
			4A	SS	WH																								
245.2			4B																										
3.5	Clayey ORGANIC SILT Firm to stiff Grey Wet																												
244.1																													
4.6	SAND and GRAVEL, some fines Compact Brown Wet		5	SS	22																								
243.1																													
5.6	SAND, some gravel, trace to some silt Compact Brown Wet		6	SS	28																								
241.5																													
7.2	SAND and GRAVEL, trace to some silt Very dense Brown Wet		7A	SS	100																								
240.7			7B																										
8.0	Sandy SILT, some gravel Very dense Brown Wet																												
240.1																													
8.6	META-DIORITE (BEDROCK)		1	RC	REC 100%																								
	Bedrock cored from a depth of 8.6 m to 12.0 m																												
	For bedrock coring details, refer to Record of Drillhole No. 18-1		2	RC	REC 100%																								
			3	RC	REC 100%																								

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity  
○ 3% STRAIN AT FAILURE

PROJECT		RECORD OF BOREHOLE										2 OF 2		METRIC				
18104224 / WO#4																		
G.W.P. 5282-14-00		LOCATION N 5450183.8; E 272857.1 NAD83 MTM ZONE 12 (LAT. 49.188917; LONG. -81.438254)										ORIGINATED BY		MA/SK				
DIST NE HWY 11		BOREHOLE TYPE Wash Boring; NW Casing; NQ Rock Coring										COMPILED BY		SB/AMP				
DATUM Geodetic		DATE October 31, 2018										CHECKED BY		JMAC				
SOIL PROFILE			SAMPLES				GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	SHEAR STRENGTH kPa					WATER CONTENT (%)							
12.0	END OF BOREHOLE																	
NOTES:																		
1. Groundwater measured in standpipe piezometer as follows:																		
Date Depth(m) Elev. (m)																		
31-Oct-18 0.8 247.9																		
29-Nov-18 1.0 247.7																		

+ 3, × 3: Numbers refer to Sensitivity  
○ 3% STRAIN AT FAILURE

SUD-MTO-RCK S:\CLIENTS\MTO\HWY11\02\_DATA\GINT\18104224-RUSSEL AND WABBLER CULVERTS\18104224.GPJ GAL-MISS.GDT 3-7-19 TR

+3, ×3: Numbers refer to Sensitivity      ○ 3% STRAIN AT FAILURE

PROJECT: 18104224 / WO#4

LOCATION: N 5450198.9; E 272922.3

NAD83 MTM ZONE 12 (LAT. 49.189056; LONG. -81.437362)

INCLINATION: -90°      AZIMUTH: ---

RECORD OF DRILLHOLE: 18-2

DRILLING DATE: October 30, 2018

DRILL RIG: CME 55 Coring, Wash Bore

DRILLING CONTRACTOR: Landcore Drilling

SHEET 1 OF 1

DATUM: Geodetic

DEPTH SCALE METRES

DRILLING RECORD

DESCRIPTION

SYMBOLIC LOG

ELEV. DEPTH (m)

RUN No.

COLOUR

% RETURN

FLUSH

JN - Joint

FLT - Fault

SHR - Shear

VN - Vein

CJ - Conjugate

BD - Bedding

FO - Foliation

CO - Contact

OR - Orthogonal

CL - Cleavage

PL - Planar

CU - Curved

UN - Undulating

ST - Stepped

IR - Irregular

PO - Polished

K - Slickensided

SM - Smooth

Ro - Rough

MB - Mechanical Break

BR - Broken Rock

NOTE: For additional abbreviations refer to list of abbreviations & symbols.

RECOVERY

TOTAL CORE %

SOLID CORE %

R.O.D. %

FRACT. INDEX

B Angle

DIP W/RT CORE AXIS

DISCONTINUITY DATA

TYPE AND SURFACE DESCRIPTION

Jr

Ja

Jh

ROCK STRENGTH INDEX

R<sub>1</sub>

R<sub>2</sub>

R<sub>3</sub>

R<sub>4</sub>

R<sub>5</sub>

WEATHERING INDEX

W1

W2

W3

W4

Q AVG.

8

9

10

11

12

13

14

15

16

17

18

19

Refer to Previous Page

Fresh, slightly foliated, grey, coarse grained, medium strong META-DIORITE

END OF DRILLHOLE

239.1

7.8

1

90

2

90

3

90

235.8

11.1

•

•

•

•

•

JN,UN,RO

JN,UN,RO

JN,UN,RO

JN,UN,RO

JN,UN,RO

3 1

3 1

3 1

3 1

3 1

3 1

3 1

3 2

3 1

UCS=39 MPa

DEPTH SCALE

1 : 60

GOLDER

LOGGED: MA/SK

CHECKED: JMAC

April 10, 2019

18104224-WO#4

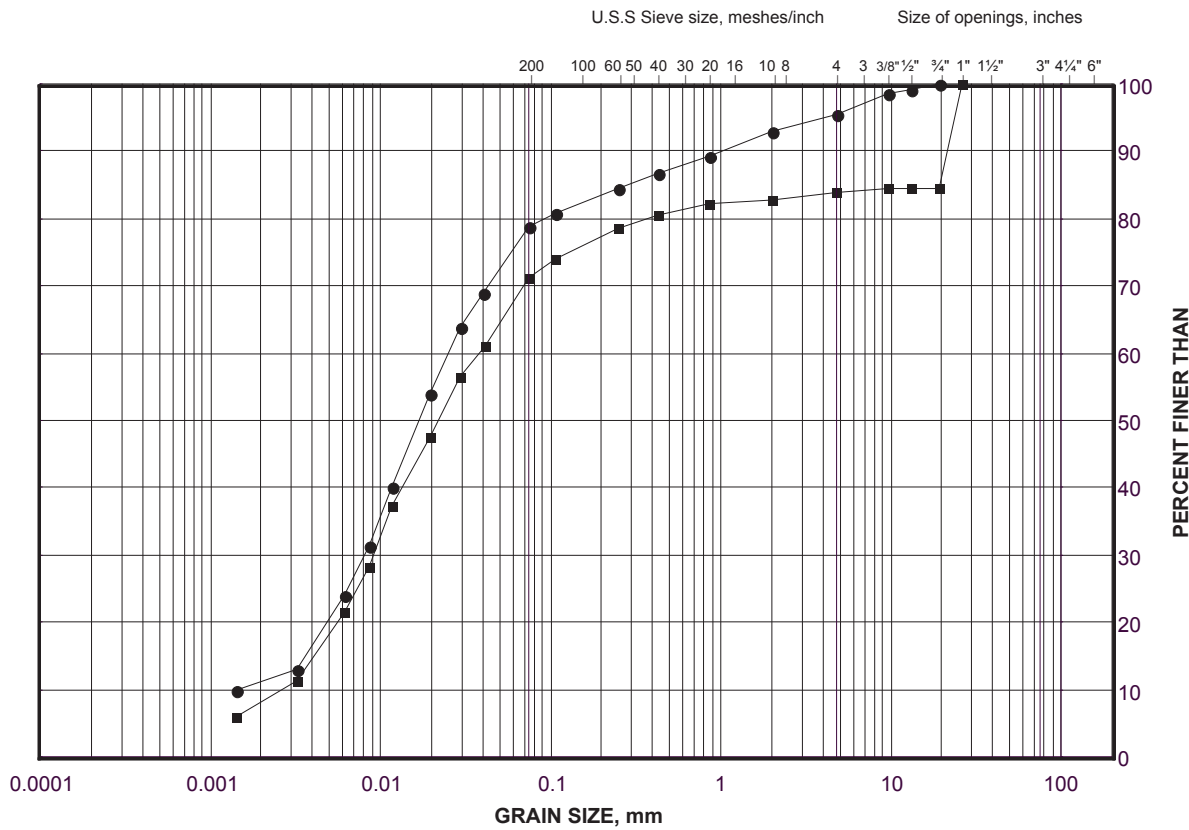
## APPENDIX B

## Laboratory Test Results and Bedrock Core Photographs



GRAIN SIZE DISTRIBUTION  
ORGANIC SILT / SILT

FIGURE B-1



SILT AND CLAY SIZES	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	18-2	2	245.1
■	18-1	3	246.1

Project Number: 18104224 (WO#4)

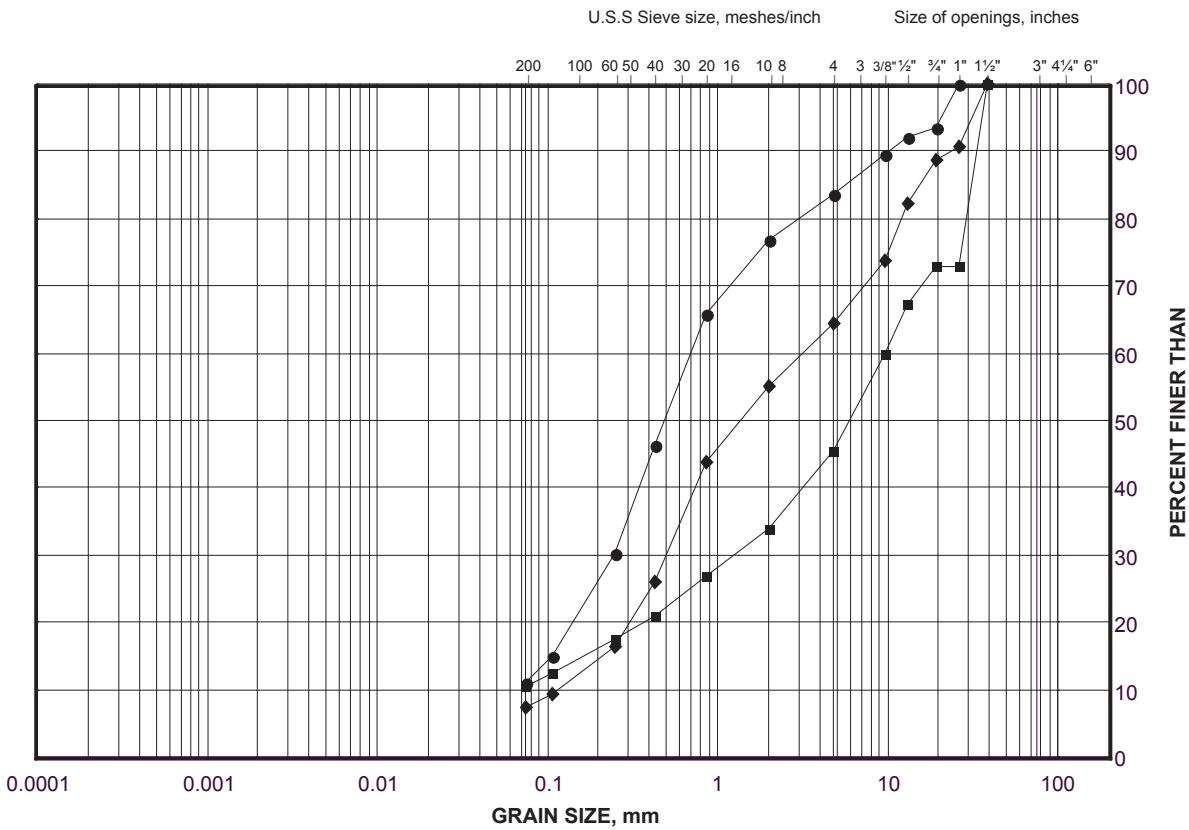
Checked By: AMP

Golder Associates

Date: 13-Feb-19

GRAIN SIZE DISTRIBUTION  
SAND to SAND and GRAVEL

FIGURE B-2



SILT AND CLAY SIZES	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	18-1	6	242.3
■	18-2	7	239.2
◆	18-1	7A	240.9

Project Number: 18104224 (WO#4)

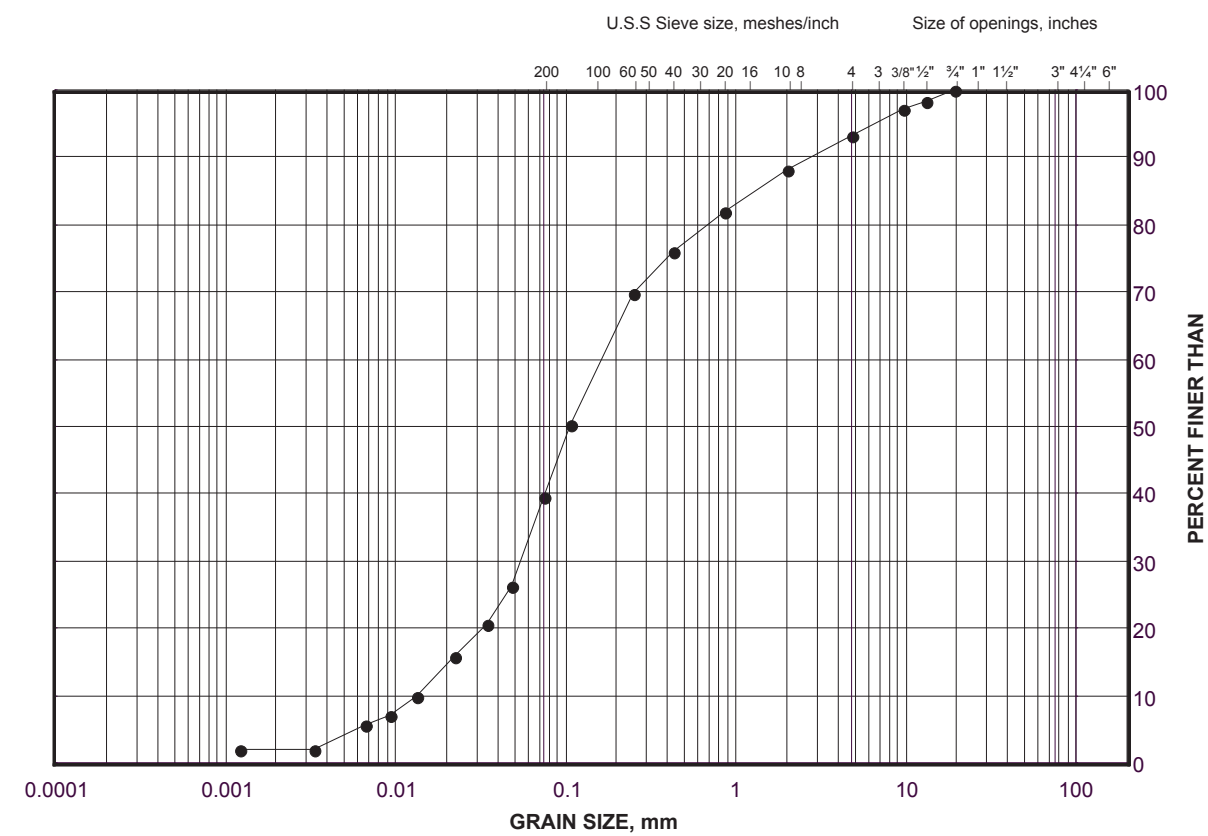
Checked By: AMP

Golder Associates

Date: 28-Feb-19

GRAIN SIZE DISTRIBUTION  
SILT and SAND

FIGURE B-3



SILT AND CLAY SIZES	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE
FINE GRAINED	SAND SIZE			GRAVEL SIZE		SIZE

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
•	18-2	5	242.1