



## Updated Foundation Investigation and Design Report

*Highway 599 Trout Creek Culvert Replacement*

Agreement No. 6019-E-0004

Assignment No. 8

GWP 6530-17-00

Geocres No. 52J-20

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*Ministry of Transportation Ontario  
Northwestern Region Geotechnical Section*

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# 1 UPDATED FOUNDATION INVESTIGATION REPORT

## 1.1 Introduction

This report presents the results of the geotechnical investigation completed by EXP Services Inc. for the replacement of the Trout Creek Culvert on Highway 599, in the Thunder Bay District. An initial investigation was conducted in the fall 2020<sup>1</sup> under Agreement No. 6019-E-0004, Assignment No. 1 and the follow-up, additional investigation was conducted in the fall 2021 under Agreement No. 6019-E-0004, Assignment No. 8. The terms of reference (TOR) for Assignment No. 1 were provided by the MTO via email, dated September 17, 2020 (i.e., initial investigation). For the follow-up investigation, following a meeting on November 5, 2021 involving the MTO and EXP, a TOR for Assignment No. 8 was sent via email on November 9, 2021.

The purpose of the initial investigation (i.e. Assignment No. 1) was to evaluate the subsurface condition along the existing/new culvert, to permit detailed design for the replacement of the Trout Creek non-structural culvert to a structural culvert and to provide construction staging recommendations. As per the TOR, the purposes of the follow-up investigation (i.e. Assignment No. 8) are to better establish the bedrock profile along the proposed culvert and to include recommendations for the foundation of the temporary modular bridge and any other geotechnical and hydrogeological concerns that are involved with the use of a temporary modular bridge for staging purposes of this project. The site specific geotechnical investigations consisted of field investigations including visual inspections, drilling, soil sampling, and laboratory testing.

This foundation investigation report discusses the findings of both the initial and follow-up investigations to produce one combined report. This 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 Trout Creek culvert is located on Highway 599 (Sta. 78+603; Latitude: 50.206730°; Longitude: -90.726231°), about 4.6 km south of Savant Lake CNR crossing within the District of Thunder Bay, Ontario. Although Highway 599 is generally a northbound-southbound highway it is oriented east-west at the site location, thus it will be described as oriented east-west in this project. At the site, Highway 599 is a two lane roadway, with a speed limit of 80 km/h (unless otherwise posted) and is about 7.2 m wide from edge of pavement to edge of pavement, with 1.5 m and 1.0 m gravel shoulders on north and south sides, respectively. The elevation of highway pavement centerline at Sta. 78+603 is about 416.6 m. Based on documents provided by MTO, the roadway embankment above the creek bed is approximately 4.5 m high with side slopes of approximately 1.1H:1V (outlet) to 1.3H:1V (inlet).

Based on the information in the TORs and AutoCAD drawing provided by MTO, the existing culvert is a 22.47 m long 1.5 m x 1.5 m wooden box culvert with two overflow CSP culverts on both sides. At the outlet side, the wooden box culvert was extended with a 2 m diameter CSP pipe, approximately 5.5 m in length (Photo 5 in Appendix A), while at the inlet side the culvert starts with a concrete headwall (Photo 1 in Appendix A). The existing culvert alignment has a skew angle of 14 degrees to the highway central line. Based on available information the obvert of the existing culvert is located at approximate elevation of Elev. 413.8 m at the inlet side and Elev. 413.2 m at the outlet side.

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<sup>1</sup> Foundation Investigation Design Report "Highway 599 Trout Creek Culvert Replacement" prepared by EXP Services Inc., dated April 5, 2021. (Geocres No. 52J-19)

Since the top elevation of the roadway is approximately at Elev. 416.6 m, the fill cover above the culvert crown is approximately 3.4 m thick. The existing overflow west and east culverts are approximately 23.7 m long CSP pipes with a 1.22 m diameter. The obverts of the west and east CSPs were measured to be at an approximate Elev. 413.5 m and 413.7 m, respectively. Selected photographs of the site and existing culverts are presented in Appendix A. The site plan and cross-section profiles for the proposed culvert alignment are shown on the drawings attached in Appendix B.

The general site conditions were assessed during the site reconnaissance visit on October 8, 2020 as well as during the initial and follow-up field investigations on November 2-6, 2020 and November 23-28, 2021, respectively. Since Highway 599 runs in an east to west direction at this location, the creek flows north to south beneath the highway. Based on observations at the site, it appears that to the north of the culvert inlet, the Trout Creek flows through a rocky canyon (Photos 3, 7 & 9 in Appendix A), while on the other side of the culvert (outlet), the creek flows into a larger water body which is a part of Sturgeon Lake located south of Highway 599 (Photos 4 & 10 in Appendix A). Rapids and relatively steep gradient of rocky creek bed were observed north of the inlet (Photos 7 & 9 in Appendix A). However, a small pool of water was observed in front of the culvert inlet as it was partially blocked by branches and broken CSP pipes (Photo 1 in Appendix A). South of the outlet, the creek becomes calm upon entering the lake as shown on Photos 2, 4 and 10 in Appendix A. At the time of the 2020 investigation, the approximate top of water elevations at the inlet and outlet were about 412.7 m and 410.9 m, respectively. The water depth in the pool formed in front of the inlet was measured to be approximately 0.4 m to 0.75 m above the rocky bottom. The measured water depth in the creek beyond the culvert outlet was around 0.7 m. Based on observations at the site, riprap (rock fill) was present on the outlet and inlet sides of the existing culvert, to protect against scour or erosion (Photos 1 and 2 in Appendix A). The roadway elevation generally increases towards the west direction. The terrain at the site is covered by bushes and trees. Bedrock outcrops were observed in the vicinity on both sides of the roadway. Some surface erosion and instability of the existing embankment was observed at the inlet side.

### 1.2.2 Geological Setting

According the Ministry of Northern Development and Mines, Map 2554 (Quaternary Geology of Ontario, West-Central Sheet, 1991) the surface conditions in the vicinity of the project area consists of bedrock, undifferentiated igneous and metamorphic rock, exposed at surface or rock covered by a discontinuous, thin layer of drift and according to Map 2542 (Bedrock Geology of Ontario, West-Central Sheet, 1991), the bedrock geology of the site is of foliated tonalite suite: tonalite to granodiorite – foliated to massive.

## 1.3 Investigation Procedures

### 1.3.1 Site Investigation and Field Testing

The initial and follow-up field investigations were performed between November 2 and 6, 2020 and between November 23 and 28, 2021, respectively.

The initial field program consisted of drilling three (3) sampled boreholes and nine (9) hand probe holes, numbered BH20-1 to BH20-3 and HP20-4 to HP20-11, respectively. The three (3) boreholes were located on the embankment to provide subsurface information for the culvert replacement and the temporary roadway protection system, while due to access restriction for the drill rig, nine (9) hand probe holes were drilled at the ends of the existing culvert (i.e. at toes of the embankment). BH20-1 to BH20-3 were advanced from the top of the embankment. HP20-4 to HP20-7 and HP20-6I were advanced at an accessible location near the inlet and HP20-8 to HP20-11 were advanced at an accessible location near the outlet. BH20-1 was drilled about 3 m east of the edge of the east outflow CSP

culvert, BH20-2 was drilled about 3.5 m west of the edge of the west outflow CSP culvert and BH20-3 was drilled about 22 m east of the main culvert centreline.

During the follow-up field program four (4) boreholes and one (1) test pit, numbered BH21-1 to BH21-4 and TP21-1, respectively, were advanced. The four (4) boreholes were advanced at the top of the embankment to provide subsurface information for the temporary modular bridge and any other geotechnical and hydrogeological concerns that are involved with the use of a temporary modular bridge for staging purposes of this project. Boreholes BH21-1 to BH21-4 were advanced near the shoulder of the northbound lane. The lone test pit was advanced about 7 m west of the existing culvert outlet and about 12 m from the edge of pavement (i.e., at the toe of the embankment). BH21-1 and BH21-4 were drilled about 22.4 m and 4.5 m west of the main culvert centreline, respectively, and BH21-2 and BH21-3 were drilled about 17.5 m and 4.5 m east of the main culvert centreline, respectively.

Table 1.1. Summary of boreholes completed by EXP

Borehole/Hand Probe/Test pit No.	Location	Coordinates (MTM NAD 83 Zone 15)		Ground Surface Elevation Elev. (m)	Borehole Depth (m)
		Northing	Easting		
November 2020					
BH20-1	Top of Roadway ~3 m east of the east outflow CSP culvert	5563540.7	252977.6	416.5	6.8
BH20-2	Top of Roadway ~3.5 m west of the west outflow CSP culvert	5563531.1	252968.2	416.5	10.4
BH20-3	Top of Roadway ~22 m east of the main culvert C/L	5563542.0	252994.7	416.1	3.9
HP20-4	~13 m west of culvert inlet	5563540.8	252950.6	414.2	0.8
HP20-5	Culvert inlet	5563545.4	252961.8	413.9	0.7
HP20-6	~8 m east of the culvert inlet	5563552.9	252974.1	413.8	0.8
HP20-6l	~13 m east of the culvert inlet	5563553.7	252978.1	413.9	0.5
HP20-7	~20 m east of the culvert inlet	5563556.5	252984.2	414.5	0.2
HP20-8	~20 m west of the culvert outlet	5563512.3	252962.4	411.9	0.3
HP20-9	~7 m west of the culvert outlet	5563518.9	252972.7	411.6	0.3
HP20-10	Culvert outlet	5563526.4	252986.0	413.1	0.5

Borehole/Hand Probe/Test pit No.	Location	Coordinates (MTM NAD 83 Zone 15)		Ground Surface Elevation Elev. (m)	Borehole Depth (m)
		Northing	Easting		
HP20-11	~10 m east of the culvert outlet	5563529.1	252992.1	414.9	0.7
<b>November 2021</b>					
BH21-1	Top of roadway ~22.4 m west of the main culvert C/L	5563525.7	252953.9	416.9	12.3
BH21-2	Top of roadway ~17.5 m east of the main culvert C/L	5563540.2	252990.6	416.3	5.3
BH21-3	Top of roadway ~4.5 m east of the main culvert C/L	5563535.7	252978.4	416.4	7.4
BH21-4	Top of roadway ~4.5 m west of the main culvert C/L	5563532.0	252970.2	416.5	9.1
TP21-1	~7 m west of the culvert outlet	5563516.9	252973.6	411.6	3.5

Note:

- (1) Attempted BH/split spoons with portable rig due to drill rig access issues. Could not penetrate rock fill at the surface.

The three (3) roadway boreholes drilled during the initial fieldwork were advanced using a rubber track mounted B54X drill rig equipped with solid stem augers, NQ core and standard soil sampling equipment, operated by a specialist drilling contractor, Maple Leaf Drilling Ltd., and all hand probe holes were advanced using a power hand auger with SSA. The roadway boreholes BH20-1, BH20-2 and BH20-3 were advanced to depths of about 6.8 m, 10.4 m and 3.9 m below ground surface, respectively. The off-road probe holes (HP20-4 to HP20-11 and HP20-6I) were advanced to a depth of between 0.2 m and 0.8 m.

During the follow-up investigation, BH21-1 was advanced using a rubber track mounted CME 55 drill rig with NW core. Due to drill rig issues, the remaining three (3) boreholes (BH21-2, BH21-3, BH21-4) were advanced using a rubber track mounted CME 750 drill rig with HW core. Both drill rigs were equipped with solid stem augers and standard soil sampling equipment, and were operated by RPM Drilling. The boreholes were advanced to depths ranging between 5.3 m and 12.3 m below ground surface. The off-road test pit was advanced to a refusal depth of about 3.5 m below ground surface using a track mounted Link-Belt 235 Excavator, operated by Perron Contracting.

The borehole locations (referenced to the MTM NAD83 coordinate system) and their ground surface elevations were surveyed by EXP personnel using a GPS (Garmin 60 CSX) and a basic level and survey rod, respectively, having an accuracy of  $\pm 2$  m in the horizontal directions and 0.1 m in the vertical direction. The relative distances between the location of boreholes and geographical and structural features on the site were also measured by a field measuring tape. The locations of boreholes, hand probe holes and test pit advanced during the investigations are shown on Drawing 1 in Appendix B. Note that the elevations for the boreholes and test pit advanced during the follow-up investigation are interpolated from the topographic survey provided by MTO.

For both drilling programs, 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.

Upon completion of the boreholes and test pit, groundwater level measurements were carried out in accordance with MTO guidelines. The recorded groundwater levels after completion of drilling boreholes and dug test pits were presented in the borehole / test pit log sheets in Appendix C. The boreholes 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*).

All fieldwork was supervised by an EXP geotechnical representative who directed the drilling, excavating and sampling operation, logged borehole / test pit 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 Thunder Bay laboratory for additional visual, textual, olfactory examination and selective testing.

### **1.3.2 Laboratory Testing**

All 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, particle size distribution and Atterberg Limit testing for approximately 30% of the collected soil samples. Uniaxial compression tests were performed on selected rock cores from six (6) boreholes. In addition, soil chemical package tests were performed on three (3) soil samples. All laboratory tests were carried out according to MTO and/or ASTM Standards as appropriate.

The results of laboratory tests on soil and rock samples are provided on the attached borehole / test pit log sheets in Appendix C. The results of the grain size analyses and Atterberg Limits are presented graphically in Appendix D. The soil chemical test results are presented in Appendix F.

## **1.4 Subsurface Conditions**

The detailed subsurface conditions encountered in the boreholes and test pit advanced during the investigations are presented on the borehole and test pit log sheets in Appendix C. Laboratory test results of grain size analyses tests and Atterberg Limits are provided in Appendix D. The “Explanation of Terms Used in Report” preceding the borehole / test pit 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 the drawings attached 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 at the roadway consisted of sand and gravel fill to rock fill below the asphalt treatment, followed by native gravelly to silty sand / sandy silt layers followed by silty sand with gravel / silt till underlain by bedrock (sloping bedrock is observed at this site, 2.1 m to 4.6 m below ground surface to the east of culvert centerline while 6.8 m to > 12.3 m below ground surface to the west of centerline). In addition, a peat with silt and sand layer was encountered beneath the fill material at BH21-4 and a native gravel layer was encountered beneath the native gravelly sand layer in BH21-1.

At the inlet and outlet sides, the subsurface conditions consist of native silty sand/sandy gravel layers below topsoil or peat underlain by a layer of native silty sand with gravel, cobbles and boulders and/or 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.

#### **1.4.1 Stratigraphy below Existing Embankment**

##### **1.4.1.1 Asphalt Treatment**

Asphalt treatment, approximately 0.025 m (~1 inch) to 0.05 m (~2 inch) thick, was generally encountered at the surface of boreholes BH20-1 to BH20-3 and BH21-2 to BH21-4.

##### **1.4.1.2 Fill: Sand and Gravel / Gravel**

Sand and gravel fill was encountered below the surface treatment in boreholes advanced through the embankment, BH20-1 to BH20-3 and BH21-2 to BH21-4 as well as at the surface at BH21-1. The depths and elevations encountered at these borehole locations are listed in Table 1.2.

The fill layer extended to depths between 0.2 m to 4.6 m below ground surface with an elevation between Elev. 416.2 m to 412.0 m. The explored thickness of this layer ranged from 0.2 m to 4.6 m.

The composition of this fill material generally consisted of sand and gravel with occasional cobbles and/or boulders and trace to some silt and trace clay. The fill was generally grey to brown in color, and moist. Samples of the fill material were generally collected from the auger flights. The SPT "N" values obtained within this fill material recorded in the boreholes BH20-3, BH21-1, BH21-2 and BH21-4 ranged from 5 to 65 blows per 0.3 m and 50 blows per 0.1 m penetration (likely affected by cobbles/boulders), suggesting that this layer was loose to very dense in relative density.

Table 1.2. Summary of Fill: Sand and Gravel / Gravel

Borehole / Hand Probe / Test Pit No.	Elevation (m)		Layer Surface Depth <sup>1</sup> (m)	Layer Thickness (m)
	Top	Bottom		
November 2020				
BH20-1	416.5	414.9	0	1.5
BH20-2	416.5	415.8	0	0.8
BH20-3	416.1	413.8	0	2.3
November 2021				
BH21-1	416.9	414.6	0	3.5
BH21-2	416.3	414.5	0	1.8
BH21-3	416.4	416.2	0	0.2
BH21-4	416.5	411.9	0	4.6

Notes:

1. Depths are relative to ground surface.

Laboratory testing performed on selected samples consisted of thirty-two (32) moisture content tests and seven (7) grain size distribution tests. The test results are as follows:

Moisture Content:

- 1.5% to 10.6%

Grain Size Distribution:

- 15% to 50% gravel;
- 42% to 59% sand; and
- 7% to 27% silt and clay

The results of the moisture content and grain size distribution tests performed by EXP are provided on the record of borehole sheets in Appendix C. The results of the grain size distribution tests performed by EXP are also provided on Figure 1 in Appendix D.

#### 1.4.1.3 Fill: Gravelly Sand

Gravelly sand fill was encountered below the sand and gravel/gravel fill in boreholes advanced through the embankment, BH20-1, BH20-2, and BH21-3. The depths and elevations encountered at these borehole locations are listed in Table 1.3.



Table 1.3. Summary of Fill: Gravelly Sand

Borehole / Hand Probe / Test Pit No.	Elevation (m)		Layer Surface Depth <sup>1</sup> (m)	Layer Thickness (m)
	Top	Bottom		
November 2020				
BH20-1	414.9	412.8	1.5	2.2
BH20-2	415.8	411.5	0.8	4.2
November 2021				
BH21-3	416.2	414.1	0.2	2.1

Notes:

1. Depths are relative to ground surface.

This fill layer extended to depths between 2.3 m to 5.0 m below ground surface with elevations between Elev. 414.1 m and 411.5 m. The explored thickness of this layer ranged from 2.1 m to 4.2 m.

The composition of this fill material generally consisted of sand and gravel with occasional boulders and trace to some silt and clay. The fill was generally brown in color, and moist. Samples of the fill material were generally collected from the auger flights, however, refusal was encountered in BH20-1 at about 4.4 m depth below ground surface within this layer. The SPT “N” values obtained at this depth within this fill material ranged from about 38 blows to 100 blows per 0.3 m penetration, suggesting that this layer dense to very dense relative density.

Laboratory testing performed on selected samples consisted of eleven (11) moisture content tests and three (3) grain size distribution tests. The test results are as follows:

Moisture Content:

- 2.8% to 25.4%

Grain Size Distribution:

- 22% to 34% gravel;
- 56% to 65% sand;
- 10% to 15% silt and clay in BH20-1 and BH21-3;
- 10% silt in BH20-2; and
- 1% clay in BH20-2

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

#### 1.4.1.4 Rock Fill

A layer of rock fill was encountered beneath the sand and gravel/gravel fill layer in borehole BH21-1 and gravelly sand fill layer in BH21-3. The depths and elevations encountered at these borehole locations are listed in Table 1.4.

Table 1.4. Summary of Rock Fill

Borehole / Hand Probe / Test Pit No.	Elevation (m)		Layer Surface Depth <sup>1</sup> (m)	Layer Thickness (m)
	Top	Bottom		
November 2021				
BH21-1	413.5	412.3	3.5	1.1
BH21-3	414.1	411.8	2.3	2.3

Notes:

1. Depths are relative to ground surface.

The rock fill layer extended to a depth of about 4.6 m below ground surface with elevations of Elev. 412.3 m and 411.8 m at BH21-1 and BH21-3, respectively. Rock coring techniques were generally conducted; however, one SPT was conducted, and the “N” value obtained at this depth was 16 blows per 0.3 m penetration, suggesting that this layer was compact in relative density. The explored thickness of the rock fill ranged from 1.1 m to 2.3 m.

#### 1.4.1.5 Peat with Silt and Sand

Peat with silt and sand was encountered in borehole BH21-4, below the sand and gravel fill material. The depth and elevation encountered at the borehole location is listed in Table 1.5.

Table 1.5. Summary of Peat with Silt and Sand

Borehole / Hand Probe / Test Pit No.	Elevation (m)		Layer Surface Depth <sup>1</sup> (m)	Layer Thickness (m)
	Top	Bottom		
	November 2021			
BH21-4	411.9	411.2	4.6	0.7

Note:

1. Depths are relative to ground surface.

The peat with silt and sand layer extended to a depth of about 5.3 m below ground surface with an elevation of about Elev. 411.2 m. The explored thickness of this layer was about 0.7 m.

The composition of this layer was peat, silt and sand with trace gravel. The material was dark brown in color, and moist to wet. The SPT “N” value obtained within this layer is about 44 blows per 0.3 m penetration, suggesting that this layer was dense in relative density.

Laboratory testing performed on a selected sample consisted of one (1) moisture content test. The test result is as follows:

Moisture Content:

- 38.8%

The result of the moisture content test performed by EXP is provided on the record of borehole sheets in Appendix C.

#### 1.4.1.6 Silty Sand to Sandy Silt

Native silty sand to sandy silt was encountered in boreholes BH20-2 and BH21-2, below the sand and gravel fill and in BH21-1, below the rock fill. The depths and elevations encountered at these borehole locations are listed in Table 1.6.

Table 1.6. Summary of Silty Sand to Sandy Silt

Borehole / Hand Probe / Test Pit No.	Elevation (m)		Layer Surface Depth <sup>1</sup> (m)	Layer Thickness (m)
	Top	Bottom		
November 2020				
BH20-2	411.5	410.4	5.0	1.1
November 2021				
BH21-1	412.3	411.6	4.6	0.7
BH21-2	414.5	414.2	1.8	0.3

Notes:

1. Depths are relative to ground surface.

The silty sand to sandy silt layer extended to depths ranging from 2.1 m to 6.1 m below ground surface with elevations between about Elev. 414.2 m and 410.4 m. The explored thickness of this layer ranged from 0.3 m to 1.1 m. BH20-2 and BH21-1 were cored due to auger refusal at a depth of 4.4 m and 3.5 m, respectively, and was switched back to drilling at a depth of 5.8 m and 4.6 m, respectively.

The composition of this layer was sand, silt and gravel with trace clay. The material was grey to light brown in color, and moist to wet. The SPT “N” values obtained within this layer was between 20 blows to 34 blows per 0.3 m and 50 blows per 0.05 m penetration (likely due to cobble/boulder), suggesting that this layer was compact to very dense in relative density.

Laboratory testing performed on a selected sample consisted of four (4) moisture content tests and three (3) grain size distribution tests. The test results are as follows:

**Moisture Content:**

- 9.3% to 16.5%

**Grain Size Distribution:**

- 4% to 23% gravel;
- 31% to 53% sand;
- 8% to 9% silt and clay in BH21-2;
- 36% to 57% silt in BH20-2 and BH21-1; and
- 8% to 9% clay in BH20-2 and BH21-1

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

#### 1.4.1.7 Gravelly Sand

Native gravelly sand was encountered in BH21-1 below the native sandy silt layer. The depth and elevation encountered at this borehole location is listed in Table 1.7.

Table 1.7. Summary of Gravelly Sand

Borehole / Hand Probe / Test Pit No.	Elevation (m)		Layer Surface Depth <sup>1</sup> (m)	Layer Thickness (m)
	Top	Bottom		
November 2021				
BH21-1	411.6	406.1	5.3	5.5

Note:

1. Depths are relative to ground surface.

The gravelly sand layer extended to a depth of 10.8 m below ground surface with an elevation of about Elev. 406.1 m. The explored thickness of this layer was 5.5 m.

The composition of this layer was sand and gravel with trace to some silt and clay. The material was light brown in color, and wet. The SPT “N” values obtained within this layer was between about 35 to 71 blows per 0.3 m penetration, suggesting that this layer was dense to very dense in relative density.

Laboratory testing performed on a selected sample consisted of six (6) moisture content tests and four (4) grain size distribution tests. The test results are as follows:

**Moisture Content:**

- 6.8% to 13.9%

**Grain Size Distribution:**

- 25% to 44% gravel;
- 44% to 62% sand; and
- 11% to 18% silt and clay

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

#### 1.4.1.8 Gravel

Native gravel was encountered in borehole BH21-1, below the gravelly sand layer. The depth and elevation encountered at this borehole location is listed in Table 1.8.

Table 1.8. Summary of Gravel

Borehole / Hand Probe / Test Pit No.	Elevation (m)		Layer Surface Depth <sup>1</sup> (m)	Layer Thickness (m)
	Top	Bottom		
November 2021				
BH21-1	406.1	404.6	10.8	1.5

Note:

1. Depths are relative to ground surface.

The gravel extended to depth of about 12.3 m below ground surface with elevation of about Elev. 404.6 m. The explored thickness of this layer was about 1.5 m.

The composition of this layer was gravel, with trace sand and trace silt. The material was reddish brown in color and wet. Rock coring techniques were generally conducted; however, one SPT (1) was conducted and the “N” value

obtained at this depth was 50 blows per 0.075 m penetration, suggesting that this layer was very dense in relative density.

Laboratory testing performed on a selected sample consisted of one (1) moisture content test. The test result is as follows:

Moisture Content:

- 7.2%

The result of the moisture content test performed by EXP is provided on the record of borehole sheets in Appendix C.

#### 1.4.1.9 Till: Silty Sand with Gravel to Silt

Silty sand with gravel to silt till was encountered in borehole BH20-2, below the sandy silt layer, and in borehole BH21-4, below the peat with silt and sand layer. The depths and elevations encountered at these borehole locations are listed in Table 1.9.

Table 1.9. Summary of Silty Sand to Sandy Silt

Borehole / Hand Probe / Test Pit No.	Elevation (m)		Layer Surface Depth <sup>1</sup> (m)	Layer Thickness (m)
	Top	Bottom		
November 2020				
BH20-2	410.4	409.6	6.1	0.9
November 2021				
BH21-4	411.2	409.7	5.3	1.5

Note:

1. Depths are relative to ground surface.

The till layer extended to depths ranging from 5.3 m to 6.1 m below ground surface with elevations between about Elev. 409.7 m and Elev. 409.6 m. The explored thickness of this layer ranged from 0.9 m to 1.5 m.

The composition of this layer was silt, sand, and gravel with occasional to some cobbles and boulders and trace clay. The material was grey in color and wet. The SPT “N” values within this layer ranged from 33 blows per 0.3 m to 50 blows per 0.125 m penetration, suggesting that this layer was dense to very dense in relative density. It is noted that rock coring techniques were initiated at BH21-4 at about 5.6 m and that a boulder was observed from 5.6 m to 6.3 m below ground surface. Below the boulder, a rock void within the silt till was observed to about 6.8 m below ground surface.

Laboratory testing performed on a selected sample consisted of three (3) moisture content tests and one (1) grain size distribution test. The test results are as follows:

#### Moisture Content:

- 2.5% to 15.1%

#### Grain Size Distribution:

- 21% gravel;
- 49% sand;
- 24% silt; and
- 6% clay.

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

#### 1.4.1.10 Bedrock

The presence of bedrock was encountered at depths ranging from 2.1 m to 7.0 m below the existing ground surface. The bedrock was confirmed by coring about 1.6 m to 3.4 m in the stratum in boreholes BH20-1 to BH20-3 and BH21-2 to BH21-4. Bedrock was not encountered to a depth of 12.3 m below ground surface in BH21-1 (about 22 m west of culvert center line). It should be noted that sloping bedrock was observed at the site, decreasing in elevation from east to west, as shown in Table 1.10 and Drawing 1 in Appendix B. On the east side of the culvert center line, bedrock was encountered at 2.1 m to 3.7 m below ground surface, while bedrock was encountered at 6.8 m to 7.0 m below ground surface on the west side. In addition, the bedrock elevation is about 1 m higher on the north side of the embankment (Elev. 412.8 m) compared to the south side of the embankment (Elev. 411.8 m), as shown on Drawing 2 in Appendix B. The actual bedrock surface depth and elevation encountered at these borehole locations are listed below in Table 1.10. Photographs of rock cores are included in Appendix E.

Table 1.10. Depth and elevation of encountered bedrock surface

Borehole / Hand Probe / Test Pit No.	Location	Depth Below Ground Surface (m)	Elevation (m)	Uniaxial Compressive Strength – UCS (MPa)
BH20-1	East of culvert center line, north embankment	3.7	412.8	82
BH20-2	West of culvert center line, south embankment	7.0	409.6	83
BH20-3	East of culvert center line, south embankment	2.3	413.8	103 <sup>1</sup> 95 <sup>2</sup>
BH21-1	West of culvert center line, south embankment	>12.3	<404.6	N/A

Borehole / Hand Probe / Test Pit No.	Location	Depth Below Ground Surface (m)	Elevation (m)	Uniaxial Compressive Strength – UCS (MPa)
BH21-2	East of culvert center line, south embankment	2.1	414.2	175
BH21-3	East of culvert center line, south embankment	4.6	411.8	91
BH21-4	West of culvert center line, south embankment	6.8	409.7	134

Notes:

1. UCS test at 2.8 m depth
2. UCS test at 3.5 m depth

Based on the bedrock NQ and HQ cores (~ core diameter 47 mm and 64 mm, respectively) recovered, the bedrock at the site consisted of meta-volcanic rock. In general, the rock samples are described as pink/white to grey/black and/or green in colour, fine to coarse grained, severely fractured to very sound. The Rock Quality Designation (RQD) measured on the core samples typically ranged from approximately 30% to 93%, indicating a rock mass of poor to excellent quality, mostly fair quality. The total core recovery (TCR) ranged from 99% to 100%. The top 0.5 m of the bedrock is estimated to be weathered.

The uniaxial compressive strength (UCS) was measured to be about 82 MPa to 175 MPa, indicating strong to very strong (R4 to R5) rock, primarily strong (R4) according to CFEM. However, our experience suggests that the rock in this area could be very strong to extremely strong (i.e. UCS in the range of 150 to 250 MPa). The laboratory uniaxial compression tests results are presented on the borehole records in Appendix C.

#### 1.4.2 Stratigraphy at Inlet and Outlet

As previously indicated, due to steep side slopes of the embankment, access to the inlet and outlet of the Trout Creek Culvert was restricted and not accessible by a drill rig. Instead, during the initial investigation, hand probe holes, HP20-4 to HP20-7 and HP20-6I, were advanced at the inlet areas, and hand probe holes, HP20-8 to HP20-11, were advanced at the outlet areas. Furthermore, due to limited information at the outlet area, one test pit (TP21-1) was advanced at the outlet area using a tracked excavator during the follow-up investigation. The hand probes were drilled to refusal depths ranging between about 0.2 and 0.8 m below ground surface, and the test pit was advanced to a refusal depth of about 3.5 m. At the inlet and outlet sides, bedrock outcrop and rock fill were observed at the toe of embankment, suggesting very shallow bedrock or buried rock fill in the vicinity of inlet and outlet.

##### 1.4.2.1 Topsoil

Topsoil, approximately 0.1 m to 0.6 m thick, was encountered at the surface of probe holes at inlet HP20-4 to HP20-7 and HP20-6I, at the surface of probe hole at outlet HP20-11, and at the surface of test pit TP21-1 (outlet area). Topsoil thicknesses may further vary beyond these locations. The depths and elevations encountered at these locations are listed in Table 1.11.



Table 1.11. Summary of Topsoil

Borehole / Hand Probe / Test Pit No.	Elevation (m)		Layer Surface Depth <sup>1</sup> (m)	Layer Thickness (m)
	Top	Bottom		
November 2020				
HP20-4	414.2	414.1	0	0.1
HP20-5	413.9	413.3	0	0.6
HP20-6	413.8	413.7	0	0.1
HP20-6I	413.9	413.8	0	0.1
HP20-7	414.5	414.4	0	0.1
HP20-11	414.9	414.6	0	0.3
November 2021				
TP21-1	411.6	411.4	0	0.2

Note:

1. Depths are relative to ground surface.

The composition of this layer consisted of occasional boulders, occasional to some cobbles, trace gravel, some sand, some silt. The topsoil was brown to dark brown in color, moist to wet and loose.

Laboratory testing performed on selected samples consisted of nine (9) moisture content test. The test result is as follows:

Moisture Content:

- 22.1% to 129.4%

The results of the moisture content tests performed by EXP are provided on the record of borehole / test pit sheets in Appendix C.

#### 1.4.2.2 Peat and Sand

A peat and sand layer, approximately 0.3 m to 1.0 m thick, was encountered at the surface of outlet probe holes, HP20-8 to HP20-10, and beneath the topsoil at TP21-1. Peat and sand thicknesses may further vary beyond the probe hole and test pit locations. Probe holes, HP20-8 and HP20-9 were terminated in this layer due to auger refusal. The depths and elevations encountered at these locations are listed in Table 1.12.

Table 1.12. Summary of Peat and Sand

Borehole / Hand Probe / Test Pit No.	Elevation (m)		Layer Surface Depth <sup>1</sup> (m)	Layer Thickness (m)
	Top	Bottom		
November 2020				
HP20-8	411.9	411.6	0	0.3
HP20-9	411.6	411.3	0	0.3
HP20-10	413.1	412.8	0	0.3
November 2021				
TP21-1	411.4	410.4	0.2	1.0

Note:

1. Depths are relative to ground surface.

The composition of this layer generally consisted of peat, sand and silt, some cobbles and boulders, and some gravel. The peat and sand material was dark brown to black in color, wet and loose.

Laboratory testing performed on selected samples consisted of seven (7) moisture content tests and three (3) grain size distribution tests. The test results are as follows:

Moisture Content:

- 34.3% to 94.5%

Grain Size Distribution:

- 3% to 6% gravel;
- 54% to 57% sand;
- 37% to 43% silt and clay

The results of the moisture content and grain size distribution tests performed by EXP are provided on the record of borehole/test pit sheets in Appendix C. The results of the grain size distribution tests performed by EXP are also provided on Figure 6 in Appendix D.

#### 1.4.2.3 Silt to Clayey Silt

Native silt to clayey silt was encountered underlying the peat and sand at test pit, TP21-1. The depth and elevation encountered at this location is listed in Table 1.13.

Table 1.13. Summary of Silt to Clayey Silt

Borehole / Hand Probe / Test Pit No.	Elevation (m)		Layer Surface Depth <sup>1</sup> (m)	Layer Thickness (m)
	Top	Bottom		
November 2021				
TP21-1	410.4	409.0	1.2	1.4

Note:

1. Depths are relative to ground surface.

The silt to clayey silt layer extended to a depth of about 2.6 m below ground surface (Elev. 409.0 m). The explored thickness of this layer was about 1.4 m.

The composition of this layer is silt, some clay to clayey soils, trace to some sand, and occasional cobbles. The material is light grey to grey in color, and wet.

Laboratory testing performed on selected samples consisted of four (4) moisture content tests, three (3) grain size distribution tests, and two (2) Atterberg limits tests. The test results are as follows:

Moisture Content:

- 13.2% to 22.4%

Grain Size Distribution:

- 0% to 1% gravel;
- 2% to 12% sand
- 77% to 82% silt; and
- 11% to 16% clay

Atterberg Limits:

- 19.6% to 23.3% liquid limit;
- 15.7% to 16.1% plastic limit; and
- 3.6% to 7.6% plasticity index

The results of the moisture content grain size distribution, and Atterberg limits tests performed by EXP are provided on the record of borehole/test pit sheets in Appendix C. The results of the grain size distribution tests performed by EXP are also provided on Figure 7 in Appendix D, and the Atterberg limits tests results are presented graphically on the Plasticity Chart on Figure 10, in Appendix D.

#### 1.4.2.4 Silty Sand / Silty Sand with Gravel

Native silty sand / silty sand with gravel was encountered in probe holes HP20-4, HP20-6, HP20-6I, HP20-7 and HP20-11 below topsoil and in HP20-10 below peat and sand, as well as below the silt to clayey silt at test pit TP21-1. The depths and elevations encountered at these locations are listed in Table 1.14.

Table 1.14. Summary of Silty Sand / Silty Sand with Gravel

Borehole / Hand Probe / Test Pit No.	Elevation (m)		Layer Surface Depth <sup>1</sup> (m)	Layer Thickness (m)
	Top	Bottom		
November 2020				
HP20-4	414.1	413.4	0.1	0.7
HP20-6	413.7	413.0	0.1	0.7
HP20-6I	413.8	413.4	0.1	0.4
HP20-7	414.4	414.3	0.1	0.1
HP20-10	412.8	412.6	0.3	0.2
HP20-11	414.6	414.2	0.3	0.4
November 2021				
TP21-1	409.0	408.1	2.6	0.9

Note:

1. Depths are relative to ground surface.

The silty sand/silty sand with gravel layer extended to depths ranging between 0.2 m to 3.5 m below ground surface with elevations ranging between Elev. 414.3 m to Elev. 408.1 m. The explored thickness of this layer was between 0.1 m to 0.9 m. Probe holes HP20-4, HP20-6, HP20-6I, HP20-7, HP20-10 and HP20-11 as well as test pit TP21-1 were terminated within this layer due to auger / excavator refusal.

The composition of this layer was sand, silt and gravel with occasional cobbles and boulders, trace to some peat and trace roots. The material is dark brown to brown in color, wet to moist, and loose to dense but mostly compact in in compactness condition.

Laboratory testing performed on a selected sample consisted of thirteen (13) moisture content tests and six (6) grain size distribution tests. The test results are as follows:

#### Moisture Content:

- 8.5% to 30.4%

#### Grain Size Distribution:

- 3% to 29% gravel;
- 48% to 79% sand; and
- 15% to 33% silt and clay

The results of the moisture content and grain size distribution tests performed by EXP are provided on the record of borehole / test pit sheets in Appendix C. The results of the grain size distribution tests performed by EXP are also provided on Figure 8 in Appendix D.

#### 1.4.2.5 Sandy Gravel

Native sandy gravel was encountered in probe hole HP20-5 below the topsoil. The depth and elevation encountered at this location is listed in Table 1.15.

Table 1.15. Summary of Sandy Gravel

Borehole / Hand Probe / Test Pit No.	Elevation (m)		Layer Surface Depth <sup>1</sup> (m)	Layer Thickness (m)
	Top	Bottom		
November 2020				
HP20-5	413.3	413.2	0.6	0.1

#### Notes:

1. Depths are relative to ground surface.

The sandy gravel layer extended to depth of about 0.6 m below ground surface with elevation of about Elev. 413.2 m. The explored thickness of this layer was 0.1 m. Probe hole BH20-5 was terminated within this layer.

The composition of this layer is sand and gravel with occasional cobbles and boulders and some silt. The material is light brown in color, moist and compact in compactness condition.

Laboratory testing performed on a selected sample consisted of one (1) moisture content tests and one (1) grain size distribution test. The test results are as follows:

#### Moisture Content:

- 18.7%

#### Grain Size Distribution:

- 45% gravel;
- 39% sand;
- 16% silt; and
- 0% clay

The results of the moisture content and grain size distribution tests performed by EXP are provided on the record of borehole sheets in Appendix C. The results of the grain size distribution test performed by EXP are also provided on Figure 9 in Appendix D.

#### **1.4.2.6 Refusal**

Probe hole drilling refusal was encountered at depths ranging between 0.2 m and 0.8 m below the existing ground surface (Elev. 414.3 m to 411.3 m). At test pit, TP21-1, refusal was encountered at 3.5 m depth below the existing ground surface (Elev. 408.1 m); note that the nature of refusal (e.g., boulder or bedrock) could not be determined as groundwater was present within the test pit obstructing observations. The presence of bedrock and/or buried rock fill below the soils was suspected based on observed bedrock outcrops as well as boulders and cobbles sized rock pieces.

### **1.5 Groundwater and Surface Water Conditions**

The groundwater levels in the boreholes were observed during and upon completion of their drilling. During EXP's investigation in November 2020, the groundwater levels in the inlet holes were dry. In the outlet holes, groundwater was observed to be at depths of about 0.2 m below ground surface, corresponding to Elev. 411.7 m and Elev. 411.4 m in HP20-8 and HP20-9 respectively while other holes were dry. In borehole BH20-1 drilled from the road, the groundwater was observed to be at depths of about 3.1 m below ground surface, corresponding to Elev. 413.4 m.

The measured elevations of the top of water at the inlet and outlet of the existing culvert and overflow culverts in November 2020 were Elev. 412.7 m and Elev. 410.9 m, respectively. As noted in Section 1.2.1, the water depth in the pool formed in front of the inlet was measured to be approximately 0.4 m to 0.75 m above the rocky bottom, while the measured water depth in the creek beyond the culvert outlet was around 0.7 m.

During EXP's follow-up investigation in November 2021, the groundwater level at the outlet location (TP21-1) was observed to be at a depth of about 0.6 m below ground surface, corresponding to Elev. 411.0 m. In boreholes, BH21-1, BH21-3 and BH21-4, which were drilled through the embankment, groundwater levels were measured the day following the borehole completion to allow for water levels to stabilize (note that water was introduced into the boreholes for rock coring purposes). The groundwater levels at BH21-1, BH21-3 and BH21-4 were all at about 4.4 m below ground surface, and at elevations ranging between about 412.0 m and 412.5 m. No groundwater measurement was obtained at BH21-2, as water levels were not representative before the borehole was backfilled (again water was introduced in the borehole for rock coring purposes). The groundwater levels encountered in the boreholes are shown on the borehole logs and is summarized below in Table 1.16.

Table 1.16. Groundwater levels encountered

Borehole / Hand Probe / Test Pit No.	Date Measured	Ground Surface Elevation (m)	Groundwater Depth <sup>1</sup> /Elevation (m)
<b>November 2020</b>			
BH20-1	November 3, 2020	416.5	3.1/413.4
HP20-8	November 2, 2020	411.9	0.2/411.7
HP20-9	November 2, 2020	411.6	0.2/411.4
<b>November 2021</b>			
BH21-1	November 25, 2021	416.9	4.4/412.5
BH21-3	November 27, 2021	416.4	4.4/412.0
BH21-4	November 27, 2021	416.5	4.4/412.1
TP21-1	November 26, 2021	411.6	0.6/411.0

Note:

1. Depths are relative to ground surface.

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.

## 1.6 Chemical Analysis

Three soil samples were selected for chemical analyses and they were sent via courier, in a secure cooler under chain of custody, to Bureau Veritas Laboratories (formerly Maxam Analytics Inc.), a CALA-certified and accredited laboratory in Mississauga, Ontario. The analytical laboratory results are presented in Appendix F, and are summarized in Table 1.17, below.

Table 1.17. Corrosivity chemical analysis

Sample Identification	pH (unitless)	Soluble Chloride (ppm)	Soluble Sulphate (ppm)	Resistivity (ohm-cm)	Conductivity (mS/cm)
BH20-1 S7	7.66	23	<20	7,900	0.130
HP20-5 S4	4.76	<20	<20	34,000	0.029
HP20-11 S2	6.13	30	<20	13,000	0.080

## 2 ENGINEERING DISCUSSION & RECOMMENDATIONS

### 2.1 General

This section of the report provides geotechnical design recommendations for replacement of the centerline culvert on Highway 599 (Sta. 78+603; Latitude: 50.206730°; Longitude: -90.726231°), about 4.6 km south of Savant Lake CNR crossing within the District of Thunder Bay, Ontario, the Ministry of Transportation (MTO) Northwestern Region. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the initial and followed-up investigations at the site performed by EXP in November 2020 and November 2021, respectively. 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 culvert and 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.

The existing culvert which conveys the creek water below Highway 599 at Sta. 78+603 is a non-structural culvert that has been identified as needing to be replaced with a structural culvert to meet the hydraulic and fish passage requirements. The existing culvert is a 22.47 m long 1.5 m x 1.5 m wooden box culvert with two overflow CSP culverts on both sides. At the outlet side, the wooden box culvert was extended by an approximately 5.5 m long, 2 m diameter CSP pipe (Photos 2 and 5 in Appendix A), while at the inlet side the culvert starts with a concrete headwall (Photos 1 and 3 in Appendix A). The existing culvert alignment has a skew angle of 14 degree to the highway central line. Based on available information the obvert of the existing wooden box culvert is located at approximate elevation of Elev. 413.8 m at the inlet side and Elev. 413.2 m at the outlet side. Since the top elevation of the roadway is approximately at Elev. 416.6 m, the fill cover above the culvert crown is approximately 3.4 m thick. The existing overflow culverts are approximately 23.7 m long CSP pipes having 1.22 m diameter. The obverts of the west and east CSPs were estimated to be at an approximate Elev. 413.5 m and 413.7 m, respectively. The embankment above the creek bed is approximately 4.5 m high with side slopes of approximately 1.1 H:1V (outlet) to 1.3H:1V (inlet).

Based on information provided by MTO on a meeting on November 5, 2021 and following that meeting including the provided preliminary GA drawings attached in Appendix L, we understand that three options for replacement of the culvert at the same alignment of the existing culvert are considered:

- 1) SPCSP open footing culvert (having a span of 9 m and rise of 3.1 m)
- 2) Twin Corrugated Steel Pipes (CSP) culvert (each diameter: ~3 m)
- 3) Precast concrete box structural culvert (dimensions: 5.4 m x 3.0 m)

Based on the preliminary GA drawings it appears that the new culvert invert for all options will be lower than that of the existing culvert (~Elev. 412 m at the CL of the roadway), probably due to need to provide an adequate hydraulic and fish passage. It is understood that the new embankment at the culvert location will be constructed with no grade change. Widening at the culvert location requested to be considered as per the TOR from 2020 (6019-E-0004, Assignment #1). The use of a temporary modular bridge for staging purposes of this project as well as temporary protection systems and temporary cofferdams used for dewatering and water diversion during culvert replacement were requested to be considered as per the TORs from 2020 (6019-E-0004, Assignment #1) and 2021 (6019-E-0004, Assignment #8).



This part of the report addresses the geotechnical design of the foundation for the new culvert 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 *Canadian Foundation Engineering Manual (CFEM) (2006)*, *MTO Gravity Pipe Design Guidelines (May 2007)* and generally accepted good practice. Pertinent construction issues from a geotechnical standpoint are examined in general accordance with the TORs provided to us in the MTO emails dated September 17, 2020 and November 9, 2021. The assessment involved review of proposed options for replacement of the existing culvert along the same alignment. The use of a temporary modular bridge for staging purposes and protection of construction site by temporary protection systems, if necessary, and cofferdams are also addressed. The construction option with widening of the existing road on the east side was addressed in the FIDR of Assignment #1 (Geocres No. 52J-19), while the construction option with widening on the south side was addressed in this report.

## 2.2 Expected Ground Conditions

The following ground conditions along the existing/new alignment are evident from the investigations carried out at the site:

- a) Highway 599 is a two lane, west/east roadway (~ 7.2 m wide) with approximately 1 to 1.5 m wide shoulders at the culvert location. The highway crosses a 1.5 m x 1.5 m wooden box culvert with two overflow CSP culverts on both sides. Embankment fill above the culvert crown is approximately 3.4 m thick. The current elevation of the crest of roadway embankment is about Elev. 416.6 m and side slopes of between 1.1H:1V to 1.3H:1V. Some surface erosion and instability of the existing embankment was observed at the inlet side.
- b) Below the road surface which consist of 0.025 m (~1 inch) thick asphalt treatment, the highway embankment consists of layers of fill: compact to very dense sand and gravel fill (~0.2 to 4.6 m thick), compact gravelly sand fill (~2.1 to 4.2 m thick) and rock fill (~1.1 to 2.3 m thick). At the west side of the existing culvert, the fill is followed by dense to very dense native sandy silt layer (~0.7 to 1.1 m thick) underlain by a layer of dense to very dense gravelly sand (~5.5 m thick) followed by very dense gravel (~1.5 m thick) in BH21-1 and dense to very dense silty sand/silt till (~0.9 to 1.5 m thick) in BH20-2 and BH21-4. The overburden soil layers are underlain by bedrock at about 6.8 m to 12.3 m (sloping bedrock towards the west) below the ground surface at that side of the culvert. At the east side of the existing culvert, the fill is generally underlain by bedrock at about 2.1 m to 4.6 m (sloping bedrock towards the west) below the ground surface.
- c) At the inlet, below the topsoil (~0.1 to 0.6 m thick), a layer of native silty sand (~0.7 m thick) /sandy gravel layers (~0.2 m thick) below the ground surface. At the outlet, below the topsoil and peat and sand (~1.2 m thick), a layer of silt to clayey silt (~1.4 m thick) followed by silty sand with gravel (~0.9 m thick) was encountered in TP21-1. It should be noted that bedrock outcrops were observed at the inlet side (east and west of the existing culvert) and outlet side (east of the existing culvert). Rock fill was present at the toes of the embankment and along the creek channel at outlet side.
- d) The foundation soil below the culvert is anticipated to be bedrock at the east side or native very dense sandy silt with gravel followed by dense to very dense silty sand with gravel/silt till at the west side followed by bedrock. Rock fill was present below the culvert CSP extension at the outlet side. A 0.7 m thick layer of peat was encountered below the embankment fill in BH21-4.
- e) At the time of investigations (November 2020 and November 2021), the top of the creek water was at Elev. 412.7 m and Elev. 410.9 m at the inlet and outlet side of the existing culvert, respectively. The highest

groundwater table measured in the boreholes drilled from the roadway was observed to be at depths of about 3.1 m below ground surface, corresponding to Elev. 413.4 m (November 3, 2020). However, seasonal variations in the water table should be expected, with higher levels occurring during wetter periods of the year (such as spring thaw and late fall) and lower levels during drier periods. The water marks inside the culvert (see Photos 1, 2 and 5 in Appendix A) suggest that the level of water in the pipe could be up to the spring line of the pipe (~Elev. 413.1 m at inlet and ~Elev. 412.5 m at outlet). The GA drawings note that a 25-year design storm water level is at Elev. 413.85 m.

## 2.3 Structure Alternatives

Based on the TOR from 2021, the following structure options were proposed for replacement of the existing culvert:

1. SPCSP open footing culvert (having a span of 9 m and rise of 3.1 m)
2. Twin Corrugated Steel Pipes (CSP) culvert (each diameter: ~3 m)
3. Precast concrete box structural culver (dimensions: 5.4 m x 3.6 m)

As specified in the TOR and preliminary GA drawings provided, the existing culvert is proposed to be replaced by a staged construction method using a Temporary Modular Bridge (TMB), therefore only this construction method will be addressed in-detail in this report. The other construction methods such as temporary widening with staged open cut unsupported excavation, half-and-half construction with unsupported cut sides and half-and-half construction with braced or anchored cut sides have been addressed in the FIDR of Assignment #1 (Geocres No. 52J-19).

In addition to these three options proposed in the TOR, the option with a permanent replacement with a Modular Bridge is also considered in the report.

An option with a sheet pile wall supporting precast concrete slabs is not considered at this site due to the presence of shallow and variable bedrock along the length of the culvert and not meeting embedment requirements.

It is noted that regardless of the option selected, the existing culvert is to be removed utilizing the open cut approach. This will require excavation down to the existing culvert founding elevation (~Elev. 412 m), and even deeper to accommodate proposed invert levels for the new culvert (i.e., between Elev. 411.3 m and Elev. 410.5 m, depending on the structure option). This implies that the significant amount of the embankment fill has to be excavated longitudinally approximately 30-35 m at the top with forward slopes of at least 2H:1V. In addition, based on the stratigraphy encountered at the site rock excavation/blasting will be required to accommodate those proposed invert levels. This deep excavation might require extensive surface/groundwater control during construction.

Based on the subsurface information obtained from the site investigations, the native very dense silty sand with gravel/silt till and/or bedrock is considered suitable for support of all replacement options. However, the choice of structure type also depends on parameters such as the initial cost, maintenance costs, hydraulic performance, fish passage requirements, ease of construction, water and soil corrosiveness, salvageability and local availability of material and equipment.

Table 2.1 summarizes advantages and disadvantages of the considered replacement alternatives. The table also shows assessed risk/consequences and relative costs of the considered options.

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Table 2.1 Evaluation of replacement alternatives for the existing culvert

Options	Rank	Advantages	Disadvantages	Relative Costs	Risks/ Consequences
SPCSP Open Footing Culvert	2	<ul style="list-style-type: none"> <li>Wider span provides an adequate fish passage</li> <li>Less accumulation of sediments in the upstream of culvert</li> </ul>	<ul style="list-style-type: none"> <li>Longer construction period</li> <li>Different foundation medium due to sloping bedrock (i.e., bedrock on the east/north side and native very dense sandy silt with gravel till at the west/south side)</li> <li>Rock excavation/blasting might be required</li> <li>Dewatering system required for construction in the dry</li> <li>Mass concrete might be required</li> <li>Rock dowels might be required</li> </ul>	<ul style="list-style-type: none"> <li>Likely more expensive than option with permanent bridge and other culvert options due to construction of footings</li> </ul>	<ul style="list-style-type: none"> <li>Risk of delay in construction due to deeper excavation below water if proper dewatering is not maintained for construction in the dry; alternatively, construction in the wet is possible</li> <li>Sloping bedrock; requires installation of rock dowels</li> <li>Potential differential settlement due to different foundation medium</li> <li>Higher scour risk</li> </ul>
Twin Corrugated Steel Pipes (CSP)	4	<ul style="list-style-type: none"> <li>Straightforward construction</li> <li>Reduced construction period, consequently, traffic management and water control period</li> </ul>	<ul style="list-style-type: none"> <li>Rock excavation/blasting might be required</li> <li>Deeper excavation than the existing culvert to meet requirements for hydraulic and fish passage</li> <li>Requires bedding material</li> <li>Dewatering system required for construction in the dry</li> </ul>	<ul style="list-style-type: none"> <li>Likely more expensive than option with permanent bridge due to cost of new culvert, TMB removal and backfill; but</li> </ul>	<ul style="list-style-type: none"> <li>Risk of delay in construction due to deeper excavation below water if proper dewatering is not maintained for construction in the dry; alternatively,</li> </ul>

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Options	Rank	Advantages	Disadvantages	Relative Costs	Risks/ Consequences
			<ul style="list-style-type: none"> <li>▪ Limited design life</li> <li>▪ Potential for corrosion</li> </ul>	likely less expensive than SPCSP option	construction in the wet is possible <ul style="list-style-type: none"> <li>▪ Sloping bedrock; rock excavation/blasting might be required</li> <li>▪ Risk of structure segment loss due to corrosion</li> </ul>
Precast Concrete Box Culvert	2	<ul style="list-style-type: none"> <li>▪ Straightforward construction</li> <li>▪ Reduced construction period; consequently, traffic management and water control period</li> <li>▪ Can be more readily installed during cold weather conditions</li> </ul>	<ul style="list-style-type: none"> <li>▪ Rock excavation/blasting might be required</li> <li>▪ Deeper excavation than the existing culvert to meet requirements for hydraulic and fish passage</li> <li>▪ Dewatering system required for construction in the dry</li> <li>▪ During high flows, the concrete floor can be undermined</li> <li>▪ Susceptible to defects/leakage at joints</li> <li>▪ Requires bedding material</li> <li>▪ Possible sediments accumulation in the upstream of the culvert</li> </ul>	<ul style="list-style-type: none"> <li>▪ Likely more expensive than option with permanent bridge due to cost of new culvert, TMB removal and backfilled; but likely less expensive than other culvert options</li> </ul>	<ul style="list-style-type: none"> <li>▪ Risk of delay in construction due to deeper excavation below water if proper dewatering is not maintained for construction in the dry; alternatively, construction in the wet is possible</li> <li>▪ Risk of leaking from joints if not properly installed</li> <li>▪ Sloping bedrock; rock excavation/blasting might be required</li> </ul>
Permanent Replacement with a	1	<ul style="list-style-type: none"> <li>▪ Larger opening to provide requirements for hydraulic and</li> </ul>	<ul style="list-style-type: none"> <li>▪ Large amount of embankment fill to be excavated</li> </ul>	<ul style="list-style-type: none"> <li>▪ Likely less expensive than other culvert</li> </ul>	<ul style="list-style-type: none"> <li>▪ Low</li> </ul>

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Options	Rank	Advantages	Disadvantages	Relative Costs	Risks/ Consequences
Modular Bridge		<ul style="list-style-type: none"><li>fish passage; therefore no need for deeper excavation</li><li>No need to construct new culvert</li><li>No need to remove modular bridge after construction</li></ul>	<ul style="list-style-type: none"><li>Additional cost for the MB and its foundations</li><li>Erosion control of temporary cuts required</li></ul>	options since there is no need for new culvert, TMB removal and backfilling	

Based on the above list of advantages and disadvantages of possible replacement options, we recommend the following ranking of the considered options:

1. Permanent replacement with a Modular Bridge
2. SPCSP open footing culvert or precast concrete box structural culvert
4. Twin Corrugated Steel Pipes (CSP) culvert

## 2.4 Foundation Design Recommendations

The foundation recommendations for the proposed structures in this project were developed based on subsurface conditions encountered in the geotechnical soil borings performed for this study. Considering the findings, shallow foundations founded on the native very dense silty sand with gravel/silt till and/or the intact bedrock are recommended as the most preferable alternative from geotechnical/foundation perspectives.

Deep foundations are not necessary or practical at this site based on the subsurface conditions encountered (i.e., the varying bedrock surface at the site; shallow bedrock at the east and north sides and deep bedrock at the west/south).

It is recommended that at location where the foundation will be placed on the rock, all overburden soils be stripped to expose the bedrock within the structure footprint. Since the bedrock surface in the area of the structure footprint is variable, engineered fill or mass concrete can be used to level the grade. The fill and/or mass concrete can be placed above the bedrock surface following removal of the existing overburden soils.

Due to the variable nature of the bedrock surface at the proposed structures it is possible that in some locations, continuous strip footing/culvert may transition from being founded partially on bedrock and partially on soil/engineered fill. The large difference in stiffness between these two foundation strata could result in undesirable differential settlements between foundation units and potentially cracking at the transition point of strip footings/culvert. A design detail will be required at these transition points to accommodate potential small differential settlements and ensure the performance of the footing and foundation wall. Use of mass concrete to raise site grades to the proposed founding level instead of the placement of engineered fill materials can minimize the risk of differential settlements. An example of Non-Standard Special Provision (NSSP) for mass concrete on bedrock is attached in Appendix K.

All loose, shattered and/or fractured rock within the foundation footprint should be removed and scaled prior to placement of mass concrete or concrete foundations in accordance with OPSS.PROV 902 and Special Provision SP902S01.

### 2.4.1 Shallow Foundations for Culverts, Temporary Modular Bridge and Permanent Modular Bridge

#### 2.4.1.1 Foundation Geotechnical Resistances

##### 2.4.1.1.1 Culverts

Based on the subsurface stratigraphy encountered at this site and the proposed invert elevations of the new culvert, the following Table 2.2 summarizes the recommended resistances at founding elevations for different types of culverts. The geotechnical resistances provided are for vertical loading condition only; load eccentricity and load

inclination effects should be addressed in accordance with the CHBDC (CAN/CSA-S6-19) and its commentary (Clause 6.10.3 and Clause 6.10.4). The geotechnical resistances provided in sections below were factored with typical consequence factor of 1.0 at ULS and SLS (Table 6.1 of CHBDC CAN/CSA S6-19), and factors of 0.5 at ULS and 0.8 at SLS for typical degree of understanding (Table 6.2 of the CHBDC CAN/CSA S6-19).

For the culvert cases recommended to be founded on the soil, it is assumed that underlying peat, organic soils and any other soft or very loose materials are to be replaced with clean and compactable soil such as Granular A or Granular B Type II. For all culvert replacement cases considered, some dewatering will be required for construction in the dry, as addressed in Sections 2.6.6 and 2.6.7. However, as noted in Section 2.6.7 some kind of work in the wet could be implemented by backfilling excavations with smaller size of rock fill, crushed stone or Granular B Type II until the excavation is out of the wet.

Given that no grade raise is planned and presence of shallow bedrock, the anticipated maximum total settlements for the new proposed culvert founded on soils are not expected to exceed 25 mm for construction done in accordance with these design parameters and assuming good construction practice including sound base preparation. For the culvert cases recommended to be founded on bedrock and/or mass concrete it is anticipated that the geotechnical resistance at SLS for 25 mm of settlement will be greater than the geotechnical resistance at ULS.

Table 2.2 Recommended spread footing design parameters

Culvert Type	Location Relative to CL of Culvert	Reference BH	Founding Elevation <sup>3</sup> (m)	Footing Size <sup>3</sup> (m)	Founding Soil Type	Factored Geotechnical Resistance at ULS (kPa)	Factored Geotechnical Resistance at SLS <sup>2</sup> (kPa)
Precast Concrete Box Structural Culvert	West	BH20-2 & BH21-4	~410.0	6	Bedrock <sup>4</sup> (along ~ 1/3 from the inlet)	650	350
					Very Dense Silty Sand/Silt Till (along rest of culvert)		
	East	BH20-1 & BH21-3	~410.0	6	Bedrock <sup>4</sup>		
Twin CSP Culverts	West	BH20-2 & BH21-4	~410.5	-	Bedrock <sup>4</sup> (along ~ 1/3 from the inlet)	650	350
					Very Dense Silty Sand/Silt Till (along rest of culvert)		

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	East	BH20-1 & BH21-3	~410.5	-	Bedrock <sup>4</sup>		
SPCSP Open Footing Culvert	West	BH20-2 & BH21-4	OPTION 1 <sup>5</sup> 411.3 <sup>5</sup>	1.5	Bedrock <sup>4</sup> (along ~ 1/3 from the inlet)	560	300
					Very Dense Silty Sand/Silt Till (along rest of culvert)		
			OPTION 2 <sup>1</sup> 409.7 <sup>1</sup>	1.5	Bedrock <sup>4</sup> Along (~ 2/3 from the inlet)	750	400
					Very Dense Silty Sand/Silt Till (along rest of culvert)		
			OPTION 3 411.8	1.5	Bedrock <sup>4</sup> (along ~ 1/3 from the inlet)	2,000	N/A
					Mass Concrete over Bedrock <sup>6</sup> (along rest of culvert)		
	East	BH20-1	412.0	1.5	Bedrock (along ~1/3 from the inlet)	2,000	N/A
		BH21-3			Mass Concrete over Bedrock <sup>6</sup> (along rest of culvert)		

Notes:

1. Below the frost line (i.e., 2.6 m below the ground surface in the creek channel)
  2. For maximum settlement of 25 mm
  3. Based on the GA drawings attached in Appendix L
  4. Might need rock excavation/blasting and/or engineered fill or mass concrete to level the grade
  5. Might need frost protection, depending on the designed bottom of the creek channel
  6. If recommended foundation elevations are above the bedrock, the soil should be excavated up to the bedrock and then replaced with mass concrete up to recommended footing elevations.
- N/A – A Geotechnical Resistance at SLS for 25 mm of settlement will be greater than the Geotechnical Resistance at ULS, since the bedrock/mass concrete is considered to be a relatively unyielding material.

Since the ULS resistance and the settlement depend on the footing size and depth of embedment, the geotechnical resistances given in Table 2.2 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 2.2 has to be reviewed to take into account those inclinations.

For the precast box culvert option, a gabion toe walls at the culvert inlet are being considered to shorten the culvert and avoid conflict with the overhead powerline utilities during installation. Based on the conditions at the inlet the retaining wall base will likely be located on the bedrock. Therefore, for design of this gabion wall, the recommended factored geotechnical resistances at ULS and SLS for 25 mm of settlement are 650 kPa and 350 kPa, respectively. If the gabion wall base is founded on the existing fill material, the recommended factored geotechnical resistances at ULS and SLS for 25 mm of settlement are 270 kPa and 150 kPa, respectively. The gabion walls should be designed in accordance with OPSS.PROV 512 and OPSS.PROV 1430. Suitably designed geotextile, supplied and placed in accordance with the referenced standards, should be provided to prevent migration of fines.

#### 2.4.1.1.2 Temporary Modular Bridge

The design of the temporary modular bridge (TMB) is the responsibility of the contractor. The contractor must retain a Professional Engineer, experienced in bridge design, to design the temporary modular bridge.

For the temporary modular bridge, spread footings are considered a suitable foundation option. For the east abutment of TMB, data obtained in BH20-3 and BH21-2 could be used for the design of a shallow TMB foundation, while for the west abutment the data from BH21-1 could be representative for the design. Based on the information from these boreholes it can be expected that the shallow spread footings (having a minimum width of 2 m) placed below the frost depth can be recommended with the following geotechnical resistances:

Table 2.3 Recommended spread footing design parameters for TMB

Abutment of TMB	Reference BH	Founding Elevation <sup>1</sup> (m)	Founding Soil Type	Factored Geotechnical Resistance at ULS (kPa)	Factored Geotechnical Resistance at SLS <sup>2</sup> (kPa)
West	BH21-1	~414.3 <sup>4</sup>	Rock fill <sup>3</sup> over native dense sandy silt and very dense gravelly sand	750	400
East	BH20-3 & BH21-2	~413.8 to 414.2	Bedrock	4,000	N/A

**Notes:**

1. Based on the frost line of 2.6 m below the ground surface
  2. For maximum settlement of 25 mm
  3. Might need engineered fill or mass concrete to level the grade
  4. Excavation will be up to Elev. 414.0 m and then 0.3 m backfill with Granular A or Granular B Type II
- N/A - A Geotechnical Resistance at SLS for 25 mm of settlement will be greater than the Geotechnical Resistance at ULS, since the bedrock is considered to be a relatively unyielding material.

The capacities provided above include a resistance factor of 0.5 and 0.8 for ULS and SLS conditions, respectively, as per Table 6.2 of the CHBDC (static analysis – typical understanding). The geotechnical resistances presented are given for concentric vertical loading conditions only. Where the load is not concentric vertical loading, load eccentricity and load inclination effects need to be considered. The front edge of the footing should be set back a minimum of 2 m from the crest of at least 2H:1V forward and side slopes that are present at the time of construction. To level the grade of the rock fill layer engineered fill or mass concrete can be used.

#### 2.4.1.1.3 Permanent Modular Bridge

Spread footings for the permanent Modular Bridge can be founded on an engineered granular pad over native compact sandy silt and very dense gravelly sand at the west side and bedrock on the east side. It is recommended that the existing layer of rock fill at the west side be excavated and replaced with the engineered granular pad. The engineered pad should consist of a Granular A material constructed on competent soils that do not contain deleterious materials, have not been disturbed by construction activities and that are not frost susceptible.

Spread footings placed on the properly prepared subgrade at the design level given in Table 2.4, should be designed based on the factored geotechnical resistances at ULS and factored serviceability geotechnical resistances (for 25 mm of settlement) given in the table. The footing width of 2 m is considered. The capacities provided below include a resistance factor of 0.5 and 0.8 for ULS and SLS conditions, respectively, as per Table 6.2 of the CHBDC (static analysis – typical understanding).

Table 2.4 Recommended spread footing design parameters for the permanent Modular Bridge

Abutment	Reference BH	Foundation Elevation <sup>1</sup> (m)	Soil at Founding Level	Factored Geotechnical Resistance at ULS (kPa)	Factored Serviceability Geotechnical Resistance (for 25 mm settlement) (kPa)
West	BH21-1	~414.3	2 m thick engineered granular pad <sup>2</sup> over native dense sandy silt and very dense gravelly sand	750	400
East	BH20-3 & BH21-2	~413.8 to 414.2	Bedrock	4,000	N/A

**Notes:**

1. Below the frost depth of about 2.6 m
  2. It is recommended that the layer of rock fill be excavated and replaced with Granular A or Granular B Type II
- N/A - A Geotechnical Resistance at SLS for 25 mm of settlement will be greater than the Geotechnical Resistance at ULS, since the bedrock is considered to be a relatively unyielding material.

Since the ULS resistance and the settlement depend on the footing size and depth of embedment, the geotechnical resistances given in Table 2.4 should be reviewed if the selected footing width or founding elevations differ from those given in the table. If an inclined load is applied instead of a vertical load, which is used in these calculations,

the values given in Table 2.4 must be reviewed to address those inclinations in accordance with CHBDC, CAN/CSA-S6-19.

Prior to construction of footings, the subgrade should be inspected according with OPSS.PROV 902. A Qualified Geotechnical Engineer should check that the design foundation elevation is achieved and all unsuitable soils 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 properly compacted Granular A pad. Dewatering will not be required during construction of spread footings since excavations are above the groundwater level of Elev. 412.5 m.

#### 2.4.1.2 Resistance to Lateral Loads

Resistance to lateral forces/ sliding should be calculated in accordance with Section 6.10.5 of the CHBDC (CAN/CSA S6-19), using the following parameters:

Table 2.5 Recommended parameters for calculation of unfactored horizontal resistance

Interface and Loading Conditions	Parameters
Between pre-cast concrete and Granular A	Coefficient of friction ( $\tan \delta$ )=0.50
Between cast-in-place concrete and Granular A	Coefficient of friction ( $\tan \delta$ )=0.55
Between cast-in-place concrete and bedrock	Coefficient of friction ( $\tan \delta$ )=0.70
Between pre-cast concrete and bedrock	Coefficient of friction ( $\tan \delta$ )=0.60
Between pre-cast concrete and mass concrete	Coefficient of friction ( $\tan \delta$ )=0.65

The listed values are unfactored; in accordance with the CHBDC (CAN/CSA S6-19), a factor of 0.8 is to be applied in calculating the horizontal resistance.

For footings supported on the bedrock, the sliding resistance can be supplemented by dowelling into the bedrock as per OPSS.PROV 904. The horizontal resistance of the dowels is dependent on the strength of the bedrock, grout and steel. The dowels should have a minimum embedded length within the unfractured bedrock of 2 m. The structural strength of the dowel and compressive strength of grout should be designed in the same way as a dowel embedded into the concrete, assuming that the unconfined compressive strength of the grout will be similar to that of the concrete. If dowels are included in the design, a Non-Standard Special Provision (NSSP) should be included to address dowelling materials, installation and testing. An example of such NSSP is included in Appendix K.

For uplift resistance from the dowels, an ULS design value of 700 kPa may be assumed for the grout-to rock bond strength, based on applying a factor of 0.4 to the ultimate bond strength estimated to be about 1,700 kPa. It is expected that ULS conditions will govern for the installation of dowels, since the geotechnical resistance at SLS assuming displacement of 25 mm is greater. The upper 0.5 m of the bond should be ignored in the calculation of required bond length since that zone of the rock is weathered or disturbed. The final bond strength for the rock-grout interface should be verified in the field by pull-out testing.

### 2.4.1.3 Frost Protection

The frost depth in the area of the culvert is estimated to be approximately 2.6 m in accordance with OPSD 3090.100. A minimum 2.6 m of soil cover or equivalent frost protection using thermal insulation should be provided only to the SPCSP open footing culvert option as well as for footings for bridges. For the box culvert the frost protection is not required.

If the earth cover above the top of the proposed culvert is less than the frost penetration line of 2.6 m, frost tapers are required at this site. It should be installed in accordance with OPSD 803.031.

### 2.4.1.4 Lateral Earth Pressure

Culvert walls, gabion walls and temporary shoring, if any, should be designed to resist lateral earth pressure. The expression for calculating lateral earth pressure is given by:

$$P = K(\gamma h + q) \text{ for non-braced cut, or } K(0.65\gamma h + q) \text{ for braced cut}$$

where,

$P$  = earth pressure intensity at depth  $h$ , kPa

$K$  = earth pressure coefficient

$\gamma$  = unit weight of retained soil, kN/m<sup>3</sup>

$q$  = surcharge near wall, kPa

$h$  = depth to point of interest, m

The above expression does not take into account hydrostatic pressure, which must be included for the groundwater levels measured on the site. Table 2.6 lists earth pressure parameters for given materials. These recommendations assume level backfill and ground surface behind the walls.

Table 2.6 Material types and earth pressure properties

Material	Unfactored Friction Angle $\phi'$ (°)	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> )
Granular A	35	0.27	3.69	0.43	22
Granular B Type II	35	0.27	3.69	0.43	22
Sand and Gravel Fill (compact to very dense)	32	0.31	3.25	0.47	21
Gravelly Sand Fill (compact)	30	0.33	3.00	0.5	21
Peat with Silt and Sand	20	0.49	2.04	0.66	17

Material	Unfactored Friction Angle $\phi'$ (°)	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> )
Silt to Clayey Silt	29	0.35	2.88	0.52	20
Sandy Silt with Gravel (very dense)	34	0.28	3.54	0.44	21
Silty Sand with Gravel Till (dense to very dense)	34	0.28	3.54	0.44	21
Silt Till (very dense)	34	0.28	3.54	0.44	21

The mobilization of full active or passive resistance requires a measurable and perhaps significant wall movement or rotation. Therefore, unless the structural element can tolerate these deflections, the at-rest earth pressure should be used in design. This would normally be the case for concrete box culverts and bridge abutments.

The effect of compaction surcharge should be taken into account in the calculations of active and at-rest earth pressures. The lateral pressure due to compaction should be taken as at least 12 kPa at the surface, and its magnitude should be assumed to diminish linearly with depth to zero at the depth where the active (or at-rest) pressure is equal to 12 kPa. This pressure distribution should be added to the calculated active (or at-rest) pressure. Notwithstanding, lighter compaction equipment and smaller lifts should be used adjacent to culvert walls to prevent overstressing.

## 2.5 Structure Stability Considerations

### 2.5.1 Embankment

#### 2.5.1.1 Stability Analyses of Side Slopes

The roadway embankment immediately adjacent to the culvert should be reconstructed in accordance with OPSS.PROV 206 using suitable earth borrow material as per OPSS.PROV 212 and/or OPSS.PROV 1010 Granular A or Granular B Type I. The existing embankment fill and the new fill along the existing roadway embankment slopes should be integrated in accordance with OPSD 208.010. The final embankment side slopes should be protected against erosion by surface water runoff as soon as practical after completion of slope grading using a combination of materials in accordance with OPSS.PROV 802, OPSS.PROV 803 and/or OPSS.PROV 804.

The global stability and settlement of that embankment with 2H:1V side slopes was addressed in the sections below. If the embankment will be widened the additional fill will be placed along the north or south widening side. Prior to the placement of new fill for the embankment widening, the sites will need to be cleared and grabbed of the existing trees and bushes. All surficial topsoil, organic (i.e., muskeg/peat), loose, soft and/or deleterious materials should be stripped from bellow the proposed embankment widening areas. The exposed subgrade should be proof rolled under the direction of qualified geotechnical personnel.

Slope stability analyses were performed to assess the global stability of the final embankment configuration as well as the existing embankment in order to check if a minimum Factor of Safety of 1.5 for static condition and 1.1 for seismic condition is achieved. For temporary widening of embankment, a minimum Factor of Safety of 1.3 for static condition is required, while for permanent widening of embankment a minimum Factor of Safety of 1.5 is required.

The static and seismic slope stability analyses were performed using the Morgenstern-Price method developed on the basis of limit equilibrium. The SLOPE/W computer program developed by GeoSlope International (GeoStudio 2018 version 9.0.2.15352) was employed for computation.

The cross-section and geometry of the existing and new embankment were developed based on EXP's observations and measurements on the site and the drawing provided by MTO, respectively. Since the new embankment at the site is recommended to be constructed with granular material, its slope should not be steeper than 2H:1V. The stratigraphy and groundwater condition at the site were developed based on the results of the geotechnical investigation presented in Part I - Foundation Investigation Report.

Based on the borehole information, the subsoils encountered at the work area consist of embankment fill, underlain by native sandy silt with gravel/silty sand till deposits and/or bedrock. Therefore, effective stress analysis for a long term assessment of the slopes was performed taking into consideration the subsoil conditions encountered at the site. The analyses assume that all topsoil and peat encountered in boreholes will be removed prior to construction. The SLOPE/W graphical printout, for analyses performed is included in Appendix G.

Table 2.7 summarizes the soil parameters used for the slope stability analyses. The soil parameters were generally estimated based on the results of field and laboratory investigation.

Table 2.7 Soil properties used in slope stability analyses

Soil Type	Effective Stress Parameter	
	$\gamma'$ (kN/m <sup>3</sup> )	$\phi'$ (degrees)
Granular A	22	35
Granular B Type II	22	35
Existing Embankment Fill (generally compact to dense)	21	30
Sandy Silt with Gravel (very dense)	21	34
Silty Sand with Gravel Till (dense to very dense)	21	34
Silty Sand with Gravel (compact to dense)	20	33
Peat with Silt and Sand	17	20
Silt to Clayey Silt	20	29
Silty Sand with Gravel	20	32

As noted above, the slope stability analyses were performed for existing embankment, embankment widening construction options on the north and south sides and new embankment, approximately 6.5 m high. The results of these analyses are shown in Figures G1 to G10 in Appendix G and summarized in Table 2.8 below.

Based on the results of slope stability analyses for existing embankment, it appears that the existing slopes (i.e., 1.1H:1V at the outlet side and 1.3H:1V at the inlet side) do not meet the required minimum factor of safety (i.e. FOS=1.5); therefore, construction of new embankment side slopes should be flattened. Based on the results of slope stability for the temporary widening construction, the widening embankment can safely be constructed with 1.5H:1V side slope, if that option is selected. As mentioned above in this section, if permanent widening of embankment is considered, the embankment slope should not be steeper than 2H:1V. Further, it was calculated that the minimum factors of safety of critical slip surfaces for the new 6.5 m high embankment meet the design criteria for static and seismic conditions (i.e., FOS=1.5 for static and FOS=1.1 for seismic condition) if its side slopes are 2H:1V.

Table 2.8 Summary of results of slope stability analyses

Locations	Max Height (m)	Conditions	Min FOS
North side of existing embankment – Inlet (Side slopes 1.3H:1V)	~4.2	drained long-term conditions, static condition	1.1 (Figure G1)
South side of existing embankment – Outlet (Side slopes 1.1H:1V)	~4.5	drained long-term conditions, static condition	1.0 (Figure G2)
Temporary widening on North side of embankment – Inlet (Side slopes 1.5H:1V)	~4.2	drained long-term conditions, static condition	1.3 (Figure G3)
Permanent widening on North side of embankment – Inlet (Side slopes 2H:1V)	~4.2	drained long-term conditions, static condition	1.5 (Figure G4)
Temporary widening on South side of embankment – Outlet (Side slopes 1.5H:1V)	~4.5	drained long-term conditions, static condition	1.3 (Figure G5)
Permanent widening on South side of embankment – Outlet (Side slopes 2H:1V)	~4.5	drained long-term conditions, static condition	1.5 (Figure G6)
North side of new embankment – Inlet (Side slopes 2H:1V)	~6.5	drained long-term conditions, static condition	1.5 (Figure G7)
		drained long-term conditions, seismic condition	1.4 (Figure G8)
South side of new embankment – Outlet (Side slopes 2H:1V)	~6.5	drained long-term conditions, static condition	1.5 (Figure G9)
		drained long-term conditions, seismic condition	1.4 (Figure G10)

### 2.5.1.2 Settlement

Since the bedrock is shallow at this site and it is not planned to change the existing embankment grade, the settlement of foundation soils will be negligible under the existing embankment. However, the fill for widening of the existing highway embankment, if any, might induce some settlement. Noting no grade raise and presence of shallow bedrock and/or predominantly dense to very dense granular foundation soils, the resulting settlement is expecting to occur mainly during construction in the order of 25 mm or less.

## 2.5.2 Temporary Modular Bridge

### 2.5.2.1 Stability Analyses of Forward Slopes

As noted before, the design of the temporary modular bridge (TMB) is the responsibility of the Contractor. However, preliminary slope stability analyses were performed in this report to assess the global stability of the TMB forward slopes for the staged construction. The results of stability analyses for the forward abutment slopes are provided below.

The SLOPE/W computer program was also employed for computation. The minimum Factor of Safety (FOS) of 1.3 is required as the design criteria for the TMB abutment forward slopes.

The geometry of the embankment at this site used in the analyses was adopted from the provided AutoCAD drawing, 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 were performed. In addition, a traffic surcharge pressure of 16 kPa was adopted in the slope stability assessments.

Table 2.9 shows the soil parameters used for the slope stability analyses for the proposed TMB. 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 for the TMB

Soil Type	Effective Stress Parameter	
	$\gamma$ (kN/m <sup>3</sup> )	$\phi'$ (degrees)
Granular B Type II	22	35
Existing Embankment Fill (generally compact to dense)	21	30
Sandy Silt with Gravel (very dense)	21	34
Silty Sand with Gravel to silt Till (dense to very dense)	21	34
Bedrock	Impenetrable	

Table 2.10 Summary of results of forward slope stability analyses for the TMB

Locations	Max Height (m)	Conditions	Min FOS
TMB with 2H:1V Forward Slopes	~6.5 m	Drained long-term conditions, static condition	1.3 (Figure G11)



The result of the slope stability analysis for the TMB forward slopes is presented in Figure G11 in Appendix G and summarized in Table 2.10. Based on this result it appears that the east and west abutments of the TMB can safely be constructed with 2H:1V forward slopes.

### 2.5.2.2 Settlement

If there is no change in the existing embankment grade nor widening at the temporary modular bridge location, the settlement under the temporary modular bridge should be negligible considering the subgrade conditions at the bridge foundations.

### 2.5.3 Modular Bridge

#### 2.5.3.1 Stability Analyses

To assess the static and seismic stability of the forward slopes of the permanent Modular Bridge abutments, the slope stability analyses using the SLOPE/W computer program were also performed. Factors of safety were calculated using the Morgenstern-Price method for critical failure surfaces. The required minimum Factor of Safety (FOS) of 1.5 was adopted as the design criteria for abutment forward and side slopes in static conditions and 1.1 for seismic conditions.

The geometry of the Modular Bridge abutments at the site used in the analyses was assumed based on available information, while the soil stratigraphy was defined based on the data obtained during this investigation. Effective stress analyses for long-term stability assessments were performed taking into consideration the subsoil conditions encountered directly beneath and adjacent the proposed bridge (i.e., non-cohesive soils). Any existing fill, peat or organic soils under the abutment and embankment were assumed to be removed completely. In addition, a traffic surcharge pressure of 16 kPa was adopted in the slope stability assessments.

Tabulated below in Table 2.11 are the soil parameters used for the slope stability analyses for the proposed Modular Bridge. Again, the soil parameters were generally estimated based on the results of field and laboratory investigation.

Table 2.11 Soil properties used in slope stability analyses for the Modular Bridge

Soil Type	Effective Stress Parameter	
	$\gamma'$ (kN/m <sup>3</sup> )	$\phi'$ (degrees)
Granular Pad	22	35
Granular B Type II	22	35
Silty Sand with Gravel to Silt Till (dense to very dense)	21	34
Topsoil	10	12
Sandy Gravel (compact)	20	32
Peat with Silt and Sand	17	20
Silty Sand with Gravel	20	32

Soil Type	Effective Stress Parameter	
	$\gamma'$ (kN/m <sup>3</sup> )	$\phi'$ (degrees)
Gravelly Sand (dense to very dense)	21	34
Bedrock	Impenetrable	

Table 2.12 Summary of results of forward slope stability analyses

Locations	Max Height (m)	Conditions	Min FOS
Permanent Modular Bridge with 2H:1V Fwd. Slopes – Abutment on Gran. Pad (Failure Behind Abutment)	~6.5	Drained long-term conditions, static condition	2.4 (Figure G12)
		Drained long-term conditions, seismic condition	2.1 (Figure G13)
Permanent Modular Bridge with 2H:1V Fwd. Slopes – Abutment on Gran. Pad (Failure In Front of Abutment)		Drained long-term conditions, static condition	1.6 (Figure G14)
		Drained long-term conditions, seismic condition	1.4 (Figure G15)
Permanent Modular Bridge with 2H:1V Side Slopes -Inlet		Drained long-term conditions, static condition	1.8 (Figure G16)
		Drained long-term conditions, seismic condition	1.6 (Figure G17)
Permanent Modular Bridge with 2H:1V Side Slopes -Outlet		Drained long-term conditions, static condition	1.5 (Figure G18)
		Drained long-term conditions, seismic condition	1.3 (Figure G19)

The results of the slope stability analyses are presented in Figures G12 to G19 (Appendix G) for the permanent Modular Bridge with abutments on granular pads, and forward and side slopes of 2H:1V are summarized in Table 2.12. 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 east and west abutments of the permanent Modular Bridge option can safely be constructed with 2H:1V forward slopes with proper rock protection of the sides to mitigate scour.

### 2.5.3.2 Settlement

It is predicted that the settlement of foundation soils will be negligible under the new bridge since the bedrock at the site is shallow and it is not planned to change the existing embankment grade at the location of the modular bridge. However, fill for widening or flattening of the existing highway embankment, if any, might induce some settlement. Noting no grade raise and presence of shallow bedrock and/or predominantly dense to very dense granular foundation soils, the resulting settlement is expecting to occur mainly during construction in the order of 25 mm or less.

## 2.6 Construction Considerations

### 2.6.1 Staged Construction Using Temporary Modular Bridge

Staged construction using a Temporary Modular Bridge (TMB) is considered as a viable construction option at this site. The major advantage of this cut and cover approach comparing to the full roadway closure is a possibility to maintain traffic flow at the site during construction. On the other hand, the major disadvantages are (i) cost of the TMB, and (ii) cost of that fill placement and removal afterwards.

Based on the results of slope stability analyses, it is recommended that the forward slopes of the bridge abutment should not exceed 2H:1V to maintain the minimum global factor of safety of 1.3 for static condition, as shown on Figure G9 in Appendix G.

Figure H.1 in Appendix H shows schematically the stages of the recommended cut and cover approach construction method - staged construction using the TMB. As can be seen, the TMB can be used for this project as the following staging:

#### STAGE 1:

- (i) Close SBL with a traffic signal and shift the one-way traffic to the NBL
- (ii) At the SBL area, place TBM footings. The footings have to be placed minimum two (2) m away from the forward slope.
- (iii) Launch TBM

#### STAGE 2:

- (i) Redirect traffic to the TMB (signalized one-way traffic)
- (ii) Install dewatering system by building cofferdam upstream and maintain creek flow through the temporary flow passage system with culvert in place.
- (iii) Excavate embankment beneath the TMB and the other side of the embankment with forward slopes of 2H:1V

#### STAGE 3:

- (i) Keep one-way traffic on the TMB
- (ii) Remove the existing culvert and construct the new culvert
- (iii) Backfill the reverse sequences
- (iv) Divert flow to new culvert
- (v) Remove temporary flow passage system and backfill. Remove dewatering system and place rip rap

#### STAGE 4:

- (i) Switch traffic to the reconstructed side (NBL)
- (ii) Remove the TMB
- (iii) Reinstatement roadway to final configuration

### 2.6.2 Excavation

As noted before, all excavations at this site must be conducted in accordance with the *Occupational Health and Safety Act* (OHSA) and *Regulations for Construction* (O. Reg. 213/91). Sand and gravel and gravelly sand fills may be

classified as a Type 3 soil above the groundwater table in conformance with the OHSA. The native sandy silt with gravel and silty sand till soils above the groundwater table may be classified as a Type 3 soil and Type 4 soil below the groundwater table. To allow placement of backfill in dry conditions, groundwater must be controlled to below the proposed invert excavation levels prior to digging to final levels. The ingress of surface water must be controlled using a suitable system as well, as described in Section 2.6.6

Temporary excavation side slopes for Type 3 soils above and below the groundwater table should not exceed 1H:1V and 3H:1V, respectively, in accordance with OHSA. There is a potential for sloughing to occur if the trench remains open for an extended period of time (i.e., > 24 hours) or during a rainfall event. In addition, some localized surficial sloughing may be experienced in areas of perched groundwater seepage (i.e., within the embankment fill).

Considering the bedrock depth at the site and proposed invert levels of the new culvert, bedrock excavation is anticipated. However, due to sloping bedrock at the site from the east toward the west and from the north toward the south, the rock excavation for structures will be uneven along the structures. More rock excavation should be anticipated at the east and north sides (i.e. relative to the existing culvert). At the east side it is expected that the rock excavation will be along the whole culvert/footing length, while at the west side it is estimated that approximately 1/3 to 2/3 length of the structure will be within the bedrock while the rest will be within the soil (i.e., depending on the foundation level). The rock excavation can be carried out using line drilling and pre-shearing approaches to minimize overbreak and rock shatter. Pre-condition surveys should be done, and monitoring carried out during any blasting. A Non-Standard Special Provision (NSSP) should be included with the Contract documents to cover this aspect. An example for NSSP for pre-condition survey and monitoring during blasting is included in Appendix K.

It is noticed that the ground surface at the toes of embankment is currently covered with rip-rap and other cobble and boulder sized rock. Cobbles and boulders were also encountered in the fill and native soils. Therefore, in addition to the uneven sloping bedrock, the Contractor shall be alerted to presence of cobbles, boulders and rock fragments in the fill and native soils, since consideration of presence of these obstructions must be made in the selection of appropriate equipment and procedures for excavation for culvert replacement and installation of any cofferdams/protection systems that may be required. It is recommended that a NSSP be included in the Contract Documents to warn the contractor of these site characteristics as well. An example for NSSP for obstructions is included in Appendix K.

### 2.6.3 Culvert Bedding

OPSDs 802.010, 803.010, 3101.150, MTOD 803.021 and Figure C6.20 of (CHBDC) which are included in Appendix J provide the bedding, embedment, cover and backfill standards for the different culvert material. According to these standards the culvert bedding for this culvert should consist of Granular A or Granular B Type II (OPSS.PROV 1010) with minimum thickness of 450 mm beneath the culvert (i.e., 0.15 D; assuming D~3m) and extend a minimum of 500 mm horizontally on either side of the culvert edge. The bedding material should be placed in layers not exceeding 200 mm in thickness, loose measurement, and compacted accordance with OPSS.PROV 501 before a subsequent layer is placed in accordance with OPSS. PROV 401.

Prior to placing any fill material, the exposed native subgrade should be inspected according to OPSS.PROV 902. A non-woven geotextile separator is to be placed between the approved subgrade and the compacted fill to assist in material placement and maintain the integrity of the founding soil along the entire length of the culvert. The geotextile separator is to be a Class II non-woven material with an equivalent opening size of 75-150 µm.

The culvert foundation level preparation might require bedrock removal. Prepared bedrock surface should be protected from entering unsuitable material during the construction. In the event that the bedrock surface is contaminated with soil/dust, cleaning by water pressure jet will be required before concrete casting on the bedrock foundation.

#### **2.6.4 Culvert Backfill**

The selection and placing of the backfill and cover should be in accordance with OPSS.PROV 902, OPSS.PROV 421, OPSS.PROV 422 and OPSDs 803.010 and 3101.150 for different culvert material. The backfill should consist of free-draining, non-frost susceptible granular materials conforming to OPSS.PROV 1010.

For fills immediately below any roadway, it is recommended that Granular A or B materials be used. As noted below, proper tapering as per standards should be provided. Below a depth of about 2.6 m from any finished road grade, approved compactable fill, such as select subgrade materials (OPSS.PROV 1010) or imported fill can be used.

All granular backfill materials should be placed in thin lifts (i.e., not exceeding 300 mm before compaction) and each lift should be compacted accordance with OPSS.PROV 501. The final lift of embankment fills prior to placing pavement sub-base should be compacted to 100 % SPMDD. The Granular A base and Granular B sub-base courses (for pavement) should be compacted to 100% of the material's SPMDD.

The use of heavy compaction equipment should be avoided immediately adjacent and above the culvert, as per MTO practice. The minimum height of fill cover above the crown of the culvert before power operated tractors or rolling equipment shall be 900 mm, unless otherwise noted by the Structural Engineer. During backfill placement, the height of the backfill should be maintained at approximately same level on both sides of the structure, to avoid lateral displacement of the structure.

Where less than frost depth (2.6 m) of earth cover is provided above the top of the culvert, a frost taper should be included as per OPSD 803.030, 803.031, and MTOD 803.021, whichever is applicable. Considering the elevation of the proposed box culvert shown in the provided GA drawings attached in Appendix L, the top of the 5.4 m (width) by 3.0 m (height) box culvert will be approximately 2.7 m below the surface of the roadway. Therefore, the frost line will be above the top of the proposed culvert.

#### **2.6.5 Temporary Protection System**

A temporary protection system should be designed to provide protection for excavations as required by the OHSA, at locations specified in the contract, and at any locations where the stability, safety or function of an existing structure and/or utility may be impaired by construction work. If a temporary protection system is required for any excavation and construction at this site it is recommended that it should be design in accordance with OPSS.PROV 539. The complete design, construction, monitoring and removal of the installed protection system should be a responsibility of the Contractor. Due to the nature of this application it is expected that much of temporary shoring will be decommissioned in place noting the high cost for removal. Decommissioning must be consistent with good practice to avoid interference with highway systems and utilities, if any.

Since the highway embankment at this site is underlain by shallow sloping bedrock, the shoring system such as soldier piles and timber lagging may be considered for design. The shoring system should be designed based on the earth pressures coefficients and soil parameters provided in Section 2.4.1.4. Considering the height of the roadway

embankment and the depth of bedrock, a temporary shoring system with additional anchorage or tiebacks may be required for lateral resistance. Conventional practice is to incorporate either buried deadman anchors or soil grouted anchors. Alternatively, a system of rakers can be used for support. Deadman anchors can be also designed based on the earth pressure coefficients and soil parameters provided in Section 2.4.1.4. For this project, either continuous or individual concrete block anchors would likely be appropriate. The anchor resistance is provided by a combination of the dead weight and passive resistance. For the full passive resistance to be realized with no load transfer to the wall, the anchor needs to be fully beyond the active wedge acting on the wall. Pressure grouted soil anchors can be also designed in a preliminary fashion in accordance with Section 26 of the CFEM. Based on the generally compact to dense fill soils at this site, the estimated factored (0.4) ULS resistance of grouted anchors would be 90 kN/m length. Detailed design would be completed following the design of the wall and the loads have been established. Normally, such anchors are supplied and installed/tested by specialist vendors/contractors.

Cobbles and boulders were noted to be contained within the fill at the site; therefore, special care has to be taken during installation of sheet piles, if any. In addition, the uneven bedrock surface encountered at the site has to be considered during the pile installation.

The protection system should be designed for the Performance Level 2 (for small, less important sections). The minimum requirements for monitoring should include the survey measurements of 6 m apart scaled targets attached to the shoring wall at the elevations specified. If movement approaches the allowable limit of 25 mm (Performance level 2), suitable measures should be taken to ensure stability of the protection system and to ensure that the movement does not exceed the performance level specified.

After construction of the new culvert/bridge, the protection system could be removed. In that case the details of the procedures associated with the removal of the protection system indicating method, sequence of work, and removal limits are required from the Contractor as per OPSS.PROV 539. However, if protection system is decided to be left in place the top should be removed to at least 1.2 m below the finished grade or ground level or at least 0.6 m below the streambed. All disturbed areas should be restored to an equivalent or better condition than existed prior to the commencement of construction. It is recommended that an NSSP for removal of protection system be included in the Contract Documents. An example of that NSSP is provided in Appendix K.

#### **2.6.6 Site Dewatering - Cofferdams**

At the time of site reconnaissance on October 8, 2020, the water level in the creek was encountered at Elev. 412.7 m at the inlet side (corresponding measured depth between 0.4 m and 0.75 m to the rocky creek bed) and 410.9 m at the outlet side (corresponding measured depth of 0.7 m). Therefore, temporary cofferdams will be required at both upstream and downstream ends to envelop the construction site and keep it free of water during replacement of the existing culvert. Considering the subsurface stratigraphy with shallow bedrock, a rock fill or sand bags and clay puddle dam could be considered as types of cofferdam suitable at this site. The cofferdam will have to be constructed to follow the topography at each end of the existing culvert. If the rock fill cofferdam is selected, the size of material suitable for use depends on the erosion potential, stream flow velocity, etc. The rock fill cofferdam should be designed with a more impervious water barrier at the outside face to create a more watertight enclosure. Schemes involving 2-inch minus crusher run with finer facing material upstream have been successfully used in similar settings. Any required permitting must be determined.

The design of these cofferdams, which are temporary retaining structures is the responsibility of the Contractor. The cofferdam must be designed to withstand the anticipated design loads and to be watertight as practically possible. The Contractor is also responsible for cofferdam's materials, construction, monitoring and removal.

Depending on the topography and overland flow drainage path, the existing creek should be diverted away from the construction site during the replacement of the existing culvert. Depending on the creek water level and surface water flow conditions at the time of construction, one of the existing overflow CSP culverts might be used to divert the creek water flow. Otherwise, a temporary diversion pipe for creek flow should be installed within the construction area and utilized during construction. The pipe should be removed upon completion of construction.

### 2.6.7 Groundwater Control

The groundwater level at the site was encountered around Elev. 413.4 m, while the excavation to the foundation level has to be carried out up to Elev. 409.4 m. Therefore, the groundwater table could be significantly above the bottom of excavation. The soils encountered on the site and within potential excavation depth consist of fill and native non-cohesive soils which the estimated range of hydraulic conductivity (k) could be around  $10^{-3}$ - $10^{-5}$  m/s.

Culvert construction, subgrade preparation and placement and compaction of granular bedding must be carried out in the dry; it is recommended that the groundwater be lowered to 0.5 m below the final culvert subgrade elevation. Furthermore, surface runoff will tend to seep into and accumulate into the excavations. The Contractor must control groundwater, perched groundwater and surface water flow at the site to permit the replacement of the culvert in the dry and stable excavation. However, some kind of work in the wet could be implemented by backfilling excavations with smaller size of rock fill, crushed stone or Granular B Type II until the excavation is out of the wet. Dewatering can be difficult to achieve for flow over and within the bedrock. However, a suitable sump and pump system behind the cofferdam will be required to remove any accumulation of water from the footing base prior to placing the culvert bedding.

As noted before in Section 2.6.6, creek channel should be diverted and cofferdam should be built prior to any excavation of the existing culvert. Dewatering requirements behind the cofferdams to keep the construction site dry will be impacted by water levels in the creek at the time of construction activities. Dewatering shall be carried out in accordance with OPSS.PROV 517, SP 517F01 (Construction Specification for Dewatering of Pipeline, Utility, and Associated Structure Excavation). It is responsibility of the Contractor to propose a suitable dewatering system based on the time of construction, water levels and creek flow conditions for prior approval of the MTO. The method used should not undermine the existing culvert, highway embankment or adjacent side slopes. In this connection the provision of toe protection at side slopes during drawdown may be required to minimize sloughing and undercutting during dewatering.

The Contract Documents must alert the Contractor to this responsibility and to design the system in accordance with NSSP FOUN0003 which amends OPSS.PROV 902. A preconstruction survey is not recommended, thus Designer Fill-In \*\* in this NSSP should be "NA" (Appendix K).

Erosion and sediment control during culvert construction should be as per the MTO Drainage Manual, Volume 2. Silt fences and other sediment control measures should be included to protect the downstream environment from the construction activities.



The groundwater infiltration volumes depend on the soil hydraulic conductivity and size and depth of excavations. It has been noted above that the soils encountered on the site and within potential excavation depth consist of fill and native non-cohesive soils which the estimated range of hydraulic conductivity ( $k$ ) could be around  $10^{-3}$ - $10^{-5}$  m/s. For the preliminary purposes, it was estimated that the groundwater infiltration volumes in the excavation of 33 m x 5.4 m x 4.0 m (length x width x depth below GWL) which size was assessed based on the provided GA drawings, should be less than 50,000 L/day. However, once the size and depth of excavation are known the groundwater infiltration could be estimated by specialists experienced in this field.

If 50,000 litres (L) or more of water per day was taken from the environment (including groundwater, lakes, rivers, ponds, etc.), a Permit to Take Water (PTTW) is generally required as per the Ontario Water Resources Act and Ontario Regulation 387/04. Water takings in excess of 50,000 L/day are regulated by the Ministry of the Environment, Conservation and Parks (MECP). Certain takings of groundwater for construction dewatering purposes with a combined total less than 400,000 L/day qualify for self-registration on the MECP's Environmental Activity and Sector Registry (EASR). Registry on the EASR replaces the need to obtain a PTTW for water taking less than 400,000 L/day and a Section 53 approval for discharge of water to the environment. A "Water Taking Plan" and a "Discharge Plan" are required by the MECP if water is taken in accordance with an EASR. In all cases, discharge under the EASR must be in accordance with a Discharge Plan. A Category 3 PTTW would be required for water takings in excess of 400,000 L/day. It is recommended that the need for the EASR or PTTW permits should be assessed in the hydrogeology report under a separate cover.

## **2.6.8 Scour/Erosion Protection**

Scour/erosion protection should be provided at the culvert inlet and outlet (including the side slopes) and/or at the Modular Bridge forward slopes. The erosion/scour protection should be designed by a specialist Hydraulic Engineer (as erosion and scour largely depend on the velocity of water in the watercourse and its regime) who is familiar with the findings of this report. Geotechnical soil parameters necessary for the scour analyses are: SPT N-value, in-situ moisture content, percent passing the No. 200 sieve (%200), mean grain size diameter ( $D_{50}$ ), liquid limit (LL), plastic limit (PL), and plasticity index (PI). These parameters can be determined based on the soils encountered at the site during the investigation. The following are some general suggestions, considering that the boreholes indicate that the main soil type consists of silt and sand.

### **2.6.8.1 Culvert**

Rip-rap protection should be provided where the culvert discharges into the open creek and where the open creek enters the culvert. The design should be finalized by the Hydraulics Engineer. For preliminary guidance, the rip-rap should extend approximately 5 m beyond the ends of the culvert and line the embankment slope to the spring line of the culvert. Such protection may involve 0.5 m thick rock (OPSS.PROV 511) extending from 1 m above the high water level to the toe of the slope and into the stream bed within the plan limits of the culvert. The rip-rap configuration at the creek bed should generally follow OPSD 810.010. The slope of the riprap shall follow the embankment fill slope.

The erosion protection should consider the possible installation of seepage protection measures at both upstream and downstream ends. For culverts, the following are typical options for seepage cutoff approaches: a typical clay seal, steel or wooden sheet pile cutoff at the upstream end of culvert, a cutoff wall incorporated in the apron slab (if one is used) of the culvert, and a cut-off trench constructed with geotextile and rock fill at the upstream end of



the culvert barrel to terminate below the granular bedding of the culvert. The seepage protection is addressed in the following section.

A clay seal should be placed at the inlet of the proposed culvert, to prevent the migration of material along the face of the culvert, the formation of flow paths, and any potential internal erosion within the road embankment. The installation procedures and the material used for the clay seal should conform to all the requirements stipulated in OPSS.PROV 1205.

#### 2.6.8.1.1 Seepage Cut-off Requirements

The seepage cut-off requirements should be reviewed in the following context. The native silty soils at the inlet side and outlet side and below the culvert bedding has a high potential for migration with high seepage gradients. For the culvert replacement and new culvert installation, it is prudent to examine possible methods to avoid piping of material resulting from seepage along the culvert. For culverts the following are typical methods: (i) clay seal, (ii) steel or wooden sheet pile cutoff at the upstream end of culvert, (iii) cut-off wall incorporated in the apron slab (if one is used) of the culvert, (iv) cut-off trench constructed with geotextile, and (v) rock fill at the upstream end of the culvert barrel to terminate below the granular bedding of the culvert. Only the clay seal and cut-off trench will be addressed since the sheet pile cut-off will require the understanding of the hydraulics of the stream.

#### 2.6.8.1.2 Clay Seal

Where readily available a clay seal should be placed at the inlet of the proposed culvert, to prevent the migration of material along the face of the culvert, the formation of flow paths, and any potential internal erosion within the road embankment. OPSS.PROV 1205 specifies that material used for clay seals shall be natural clay, clay mixture (1 part Bentonite powder and 3.5 parts Granular "A") or a Geosynthetic Clay Liner (GCL). The coefficient of permeability shall not exceed  $1 \times 10^{-6}$  cm/s.

The following outlines the installation procedures and minimum material requirement of the clay seal:

- The clay seal should be placed along the sides and top of the culvert a minimum of 1.0 m along the side of the culvert and extending out laterally 1.0 m from the culvert.
- The clay seal should be placed from the top of the culvert footings and extend along the side and the top of the culvert. The clay must not be placed below the culvert.
- The clay should have a Liquid Limit greater than 40% and a Plasticity Index greater than  $0.73 \times (\text{Liquid Limit} - 20\%)$ .
- The clay seal is to be placed in maximum 150 mm thick lifts and compacted to 95% SPMDD within 2% of the optimum moisture content.

If a GCL is used as a clay seal its material specifications containing the physical, mechanical and hydraulic properties shall be obtained from the manufacture. It is estimated that an approximately 12 mm thick GCL should be installed a minimum 1.0 m along the side of the culvert.

#### 2.6.8.1.3 Cut-Off Trench/Wall

A cut-off trench/wall can be used at both the upstream and downstream ends of the culvert and can be incorporated when the rip-rap apron at both ends of the culvert are being installed (see Figure H2 in Appendix H). In general, a

trench is dug across the stream alignment to well beyond the walls of the culvert and a geomembrane liner is laid on the side of the trench keyed into the culvert at the top and on the base of the trench. The trench is then backfilled with graded rip-rap.

### **2.6.8.2 Modular Bridge**

Structures close to the creek 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 for the bridge can be protected against structural undermining by locating the foundations at an appropriate depth by providing scour protection blankets. Scour protection must be remained effective for the design life of the bridge.

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

Erosion protection should be provided on creek banks and adjacent to abutments and fore slopes. Such protection may involve 0.5 m thick rock (OPSS.PROV 511) extending from the high water level to the toe of the slope and into the stream bed within the plan limits of the bridge. The erosion protection should be installed a minimum 1.0 m above the design high water.

## **2.7 Corrosion Potential and Cement Type**

Three (3) soil samples were selected for chemical analyses during this investigation. 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 1.17 of this report.

The pH, resistivity and chloride concentration provide an indication of the degree of corrosiveness of the sub-surface environment. The soil pH values measured at the site were ranged between 4.78 to 7.66 which is slightly beyond the lower limit of what is considered the normal range of soil pH of 5.5 to 8.5. The chemical data indicates very high ( $R > 6000$  ohm-cm) resistivity of the tested soil, which suggests very low potential for corrosion of buried metallic elements as per Table 3.2 of the MTO Gravity Pipe Design Guideline. The measured chloride content is between  $< 20$  ppm ( $\mu\text{g/g}$ ) and  $30$  ppm ( $\mu\text{g/g}$ ) (i.e.,  $< 0.002\%$  and  $0.003\%$ ) which indicates also a low potential for additional corrosion.

These chemical test results may be used to aid in the selection of coatings and corrosion protection systems for buried steel culvert, if selected. If the concrete culvert is selected, consideration should be given by the designer to designing for a « C » type of exposure class of concrete as defined by CSA A23.1:19 Table 1, since the culvert will be exposed to de-icing salt.

The maximum water soluble sulphate content of the soils tested is  $< 20$  ppm ( $\mu\text{g/g}$ ), i.e.  $< 0.002\%$  and being less than  $0.10\%$  (as per CSA A23.1:19, Table 3) does not require sulphate resistant cement. The data supports our local experience.

### 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

This Foundation Investigation and Design Report has been prepared by Ahileas Mitsopoulos, 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 Elwin Farkas.

#### EXP Services Inc.



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Designated MTO Foundation Contact



## REFERENCES

- 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, Highway Research Board, Washington D.C., pp 104-112.
- Ministry of Northern Development and Mines, Ontario Geological Survey, 1991. Map 2554: Quaternary Geology of Ontario, West-Central Sheet
- Ministry of Northern Development and Mines, Ontario Geological Survey, 1991. Map 2542: Bedrock Geology of Ontario, West-Central Sheet
- Ministry of Transportation Ontario, 2005. MTO Gravity Pipe Design Guidelines. Highway Design Office, St. Catharines, Ontario.
- Ontario Ministry of Transportation, 2019. Guideline for Foundation Engineering Services, Version 1.0.

### **ASTM International:**

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

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

OPSS.PROV 206    Construction Specification for Grading

OPSS.PROV 212    Construction Specification for Earth Borrow

OPSS.PROV 401    Construction Specification for Trenching, Backfilling and Compacting

OPSS.PROV 421    Construction Specification for Pipe Culvert Installation in Open Cut

OPSS    422        Construction Specification for Precast Reinforced Concrete Box Culverts in Open Cut

OPSS.PROV 501    Construction Specification for Compacting

OPSS.PROV 511    Rip Rap, Rock Protection and Granular Sheetting

OPSS.PROV 539    Construction Specification for Temporary Protection Systems

OPSS.PROV 802    Construction Specification for Topsoil

OPSS.PROV 803    Construction Specification for Vegetative Cover

OPSS.PROV 804    Construction Specification for Seed and Cover

OPSS.PROV 902    Construction Specification for Excavating and Backfilling – Structures

OPSS.PROV 904    Construction Specification for Concrete Structures

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

OPSS.PROV 1205 Material Specification for Clay Seal

OPSS.PROV 1440 Material Specification for Steel Reinforcement for Concrete

OPSS.PROV 1860 Material Specification for Geotextiles

OPSS.PROV 512 Construction Specification for Installation of Gabions

OPSS.PROV 1430 Material Specification for Gabion Baskets and Mats

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

OPSD 208.010 Benching of Earth Slopes

OPSD 802.010 Flexible Pipe Embedment and Backfill Earth Excavation

OPSD 803.010 Backfill and Cover for Concrete Culverts with Span Less Than or Equal to 3.0 m

MTOD 803.021 Bedding and Backfill for Precast Concrete Box Culverts

OPSD 803.031 Frost Treatment – Pipe Culverts Frost Penetration Line Between Top of Pipe and Bedding Grade

OPSD 810.010 Rip-Rap Treatment for Sewer and Culvert Outlets

OPSD 3090.100 Foundation Frost Depths for Northern Ontario

OPSD 3101.150 Walls, Abutment, Backfill, Minimum Granular Requirement

**Special Provisions (SP):**

NSSP FOUN0003 Amendment to OPSS.PROV 902

**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

## 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

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 1. Inlet side of the culvert, October 2020



Photograph 2. Outlet side of the culvert, October 2020





Photograph 3. Inlet side of the culvert (facing south), November 2020



Photograph 4. Outlet side of the culvert (facing south), November 2020





Photograph 5. Inside of the centreline culvert at outlet, October 2020



Photograph 6. Drilling borehole BH20-1 on the Highway 599 embankment (facing east), November 2020





Photograph 7. Drilling hand probe hole HP20-4, HP20-5, at the inlet side, November 2020



Photograph 8. Drilling hand probe hole HP20-6I, at the inlet side, November 2020





Photograph 9. Inlet side of culvert (facing north), November 2021



Photograph 10. Outlet side of culvert (facing south), November 2021



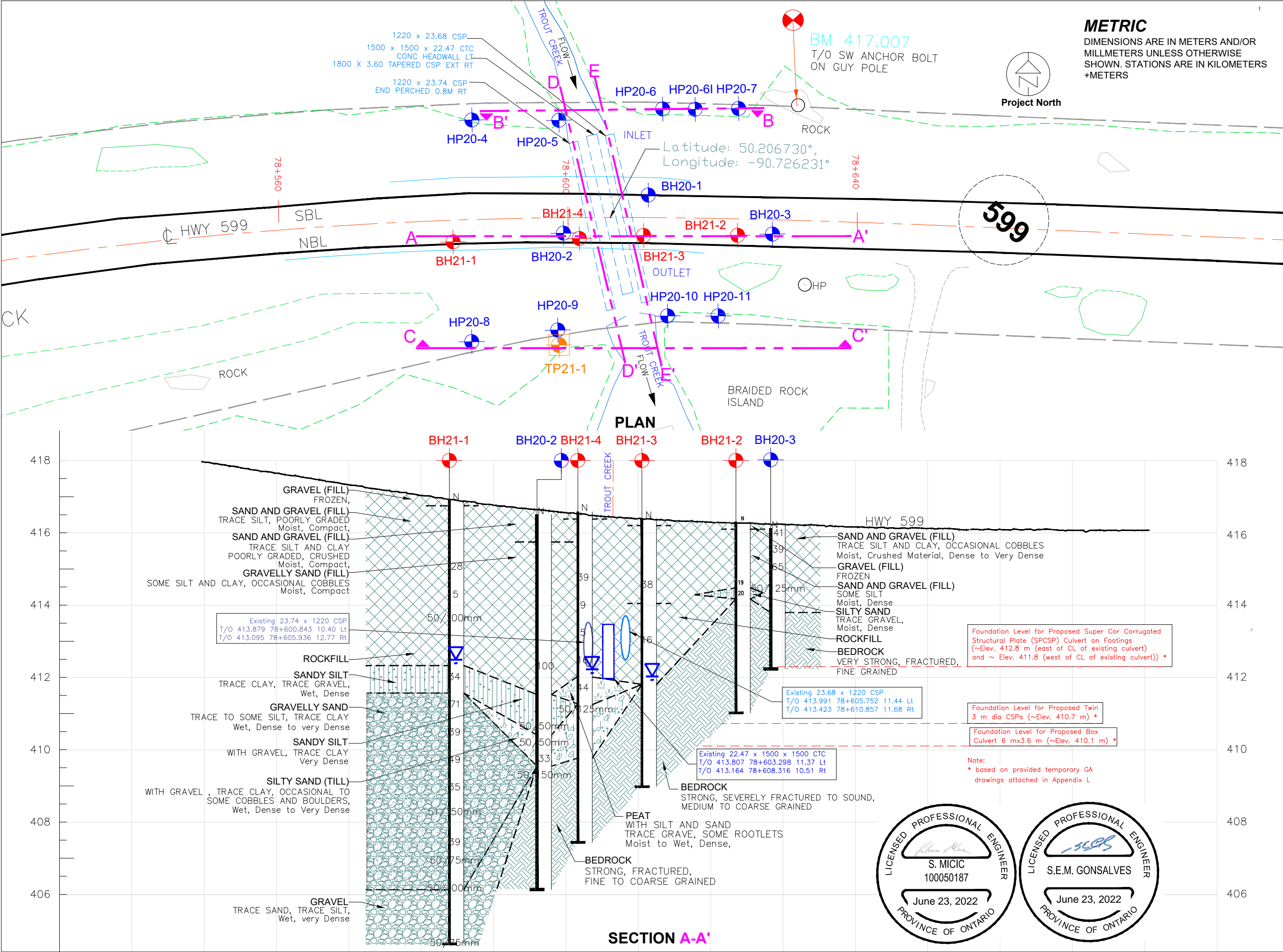


Photograph 11. View of north embankment (facing east), November 2021



Photograph 12. View of south embankment (facing east), November 2021. Test pit TP21-1 advanced on the south side of Highway 599 and west of culverts.

## Appendix B – Drawings



Agreement No. 6019-E-0004/0005  
GWP No. 6530-17-00  
Assignment No. 8

SHEET  
1

Trout Creek Non-Structural Culvert to be Replaced with  
a Structural Culvert- Highway 599  
Northwestern Region, Thunder Bay, ON  
Latitude: 50.206730°, Longitude: -90.726231°  
BOREHOLE LOCATION PLAN AND SOIL STRATA

exp. EXP Services Inc.

KEY PLAN

LEGEND

- Borehole Location (2021)
- Borehole Location (2020)
- Test Pit Location (2021)
- Bench Mark Location (Elev. 417.007m)
- Standard Penetration Test (Blows/0.3 m)
- Groundwater level measured in open hole

SOIL STRATA SYMBOLS


BH No.	ELEV.	MTM CO-ORDINATES (ZONE ON-15)	
		NORTHING	EASTING
BH20-01	416.5	5563540.7	252977.6
BH20-02	416.5	5563531.1	252968.2
BH20-03	416.1	5563542.0	252994.7
HP20-04	414.2	5563540.8	252950.6
HP20-05	413.9	5563545.4	252961.8
HP20-06	413.8	5563552.9	252974.1
HP20-06I	413.9	5563553.7	252978.1
HP20-07	414.5	5563556.5	252984.2
HP20-08	411.9	5563512.3	252962.4
HP20-09	411.6	5563518.9	252972.7
HP20-10	413.1	5563526.4	252986.0
HP20-11	414.9	5563529.1	252992.1
BH21-1	416.9	5563524.6	252954.9
BH21-2	416.3	5563540.2	252990.6
BH21-3	416.4	5563535.7	252978.4
BH21-4	416.5	5563532.0	252970.2
TP21-1	411.6	5563516.9	252973.6

NOTES

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.

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DATE	SM	BY	DESCRIPTION
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			PROJECT NO. ADM-00262199-H0
SUBM'D SH	CHECKED SM	DATE	APRIL 27, 2022
DRAWN SH	CHECKED SM	APPROVED SG	DWG. 1

LICENSED PROFESSIONAL ENGINEER  
S. MICIC  
100050187  
June 23, 2022  
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER  
S.E.M. GONSALVES  
June 23, 2022  
PROVINCE OF ONTARIO



**METRIC**  
DIMENSIONS ARE IN METERS AND/OR  
MILLIMETERS UNLESS OTHERWISE  
SHOWN. STATIONS ARE IN KILOMETERS  
+METERS

Agreement No. 6019-E-0004/0005  
GWP No. 6530-17-00  
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Northwestern Region, Thunder Bay, ON  
*Latitude: 50.206730°, Longitude: -90.726231°*

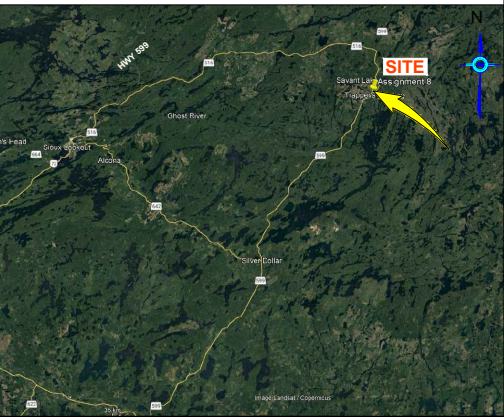
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BOREHOLE LOCATION PLAN AND SOIL STRATA



EXP Services Inc.

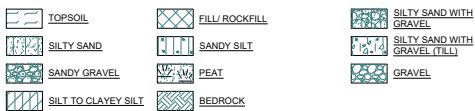
KEY PLAN



LEGEND

- Borehole Location (2021)
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- Test Pit Location (2021)
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NOTES

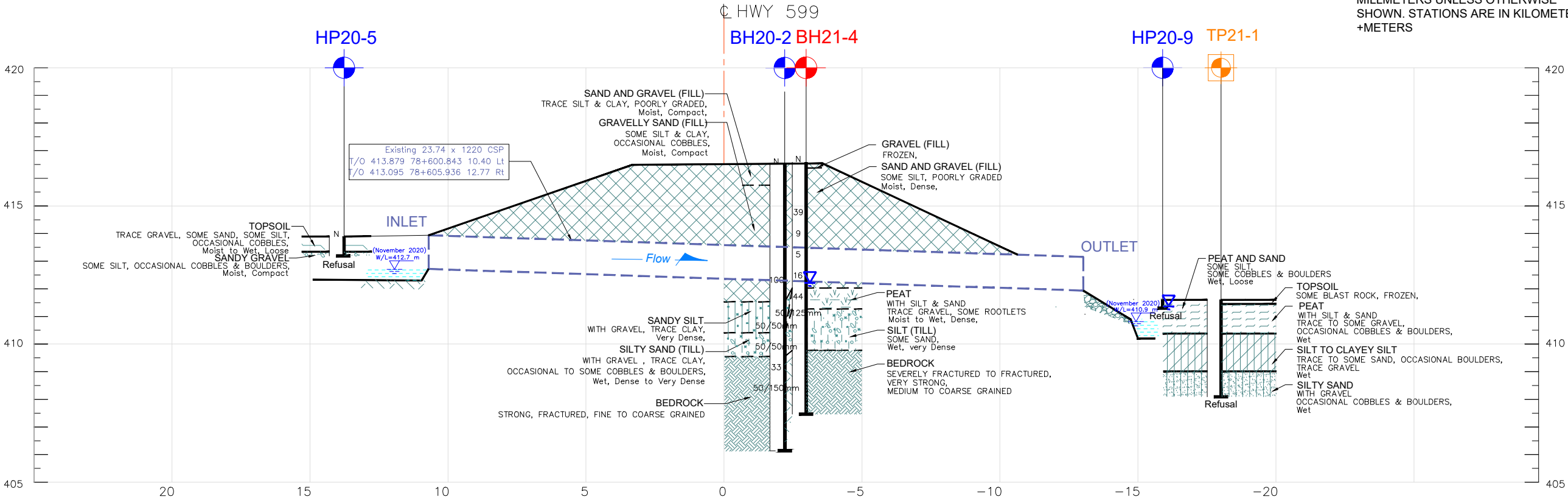
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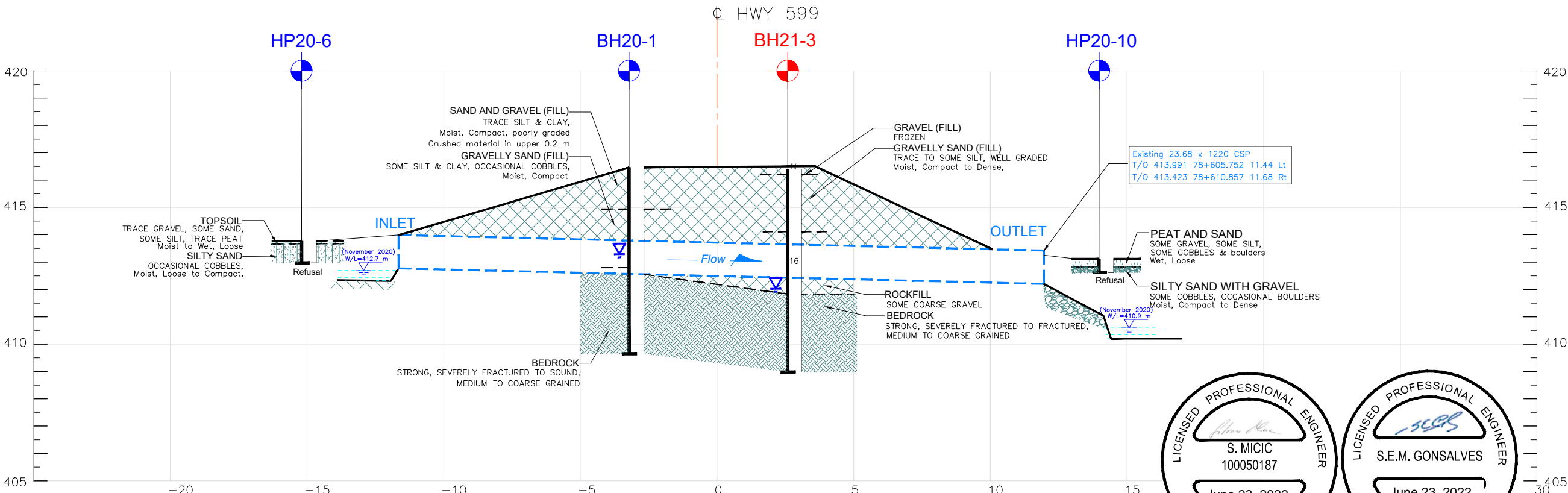
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	BY	DESCRIPTION	
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		PROJECT NO.	ADM-00262199-H0
SUBM'D SH	CHECKED SM	DATE	APRIL 27, 2022
DRAWN SH	CHECKED SM	APPROVED SG	DWG. 2



SECTION D-D'



SECTION E-E'





## METRIC

DIMENSIONS ARE IN METERS AND/OR  
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+METERS

Agreement No. 6019-E-0004/0005  
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SHEET  
3



EXP Services Inc.

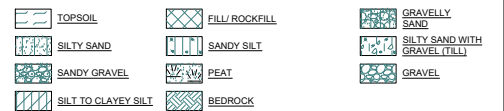
### KEY PLAN



### LEGEND

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- Borehole Location (2020)
- Test Pit Location (2021)
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HP20-08	411.9	5563512.3	252962.4
HP20-09	411.6	5563518.9	252972.7
HP20-10	413.1	5563526.4	252986.0
HP20-11	414.9	5563529.1	252992.1
BH21-1	416.9	5563524.6	252954.9
BH21-2	416.3	5563540.2	252990.6
BH21-3	416.4	5563535.7	252978.4
BH21-4	416.5	5563532.0	252970.2
TP21-1	411.6	5563516.9	252973.6

### NOTES

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.

SCALE:

DATE	SM	BY	DESCRIPTION
			GEOCRES NO. 52J-20
			PROJECT NO. ADM-00262199-H0
SUBM'D SH	CHECKED SM	DATE	APRIL 27, 2022
DRAWN SH	CHECKED SM	APPROVED SG	DWG. 3

## SECTION B-B'

## SECTION C-C'

## Appendix C – Borehole and Test Pit 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.

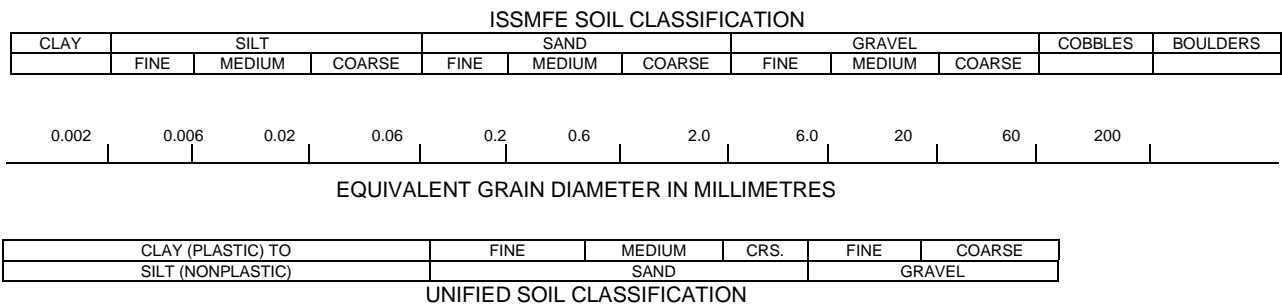
*Seam:* a thin, confined layer of soil having different particle size, texture, or color from materials above and below.

*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 generally the ASTM D2487-11 Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System) with some modification to reflect current MTO practices. 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 Canadian Foundation Engineering Manual (CFEM):

Table a: Percent or Proportion of Soil

Term	Description	Criteria
"trace"	trace gravel, trace sand, etc.	1% - 10%
"some"	some gravel, some sand, etc.	10% - 20%
Adjective	gravelly, sandy, silty and clayey	20% - 35%
"and"	and gravel, and sand, etc.	>35%
Noun	gravel, sand, silt, clay	>35% and main fraction

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≤N<10
Compact	10≤N<30
Dense	30≤N<50
Very Dense	50≤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:

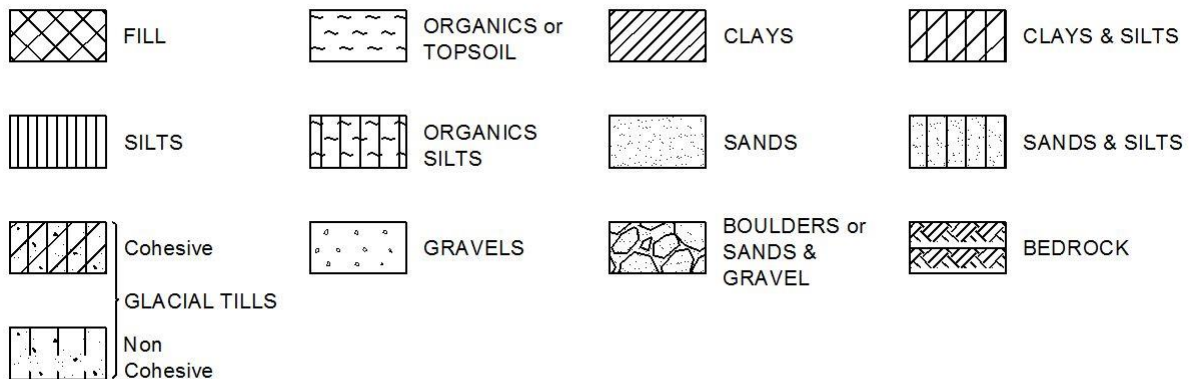
Table c: Consistency of Cohesive Soil

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

$m_v$	kPa <sup>-1</sup>	Coefficient of volume change
$c_c$	1	Compression index
$c_s$	1	Swelling index
$c_r$	1	Recompression index
$c_v$	m <sup>2</sup> /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
$\phi'$	—°	Effective angle of internal friction
$c_u$	kPa	Apparent cohesion intercept
$\phi_u$	—°	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

$P_s$	kg/m <sup>3</sup>	Density of solid particles
$\gamma_s$	kN/m <sup>3</sup>	Unit weight of solid particles
$\rho_w$	kg/m <sup>3</sup>	Density of water
$\gamma_w$	kN/m <sup>3</sup>	Unit weight of water
$\rho$	kg/m <sup>3</sup>	Density of soil
$\gamma$	kN/m <sup>3</sup>	Unit weight of soil
$\rho_d$	kg/m <sup>3</sup>	Density of dry soil
$\gamma_d$	kN/m <sup>3</sup>	Unit weight of dry soil
$\rho_{sat}$	kg/m <sup>3</sup>	Density of saturated soil
$\gamma_{sat}$	kN/m <sup>3</sup>	Unit weight of saturated soil
$\rho'$	kg/m <sup>3</sup>	Density of submerged soil
$\gamma'$	kN/m <sup>3</sup>	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	m <sup>3</sup> /s	Rate of discharge
v	m/s	Discharge velocity
i	1	Hydraulic gradient
k	m/s	Hydraulic conductivity
j	kN/m <sup>3</sup>	Seepage force

# RECORD OF BOREHOLE No BH21-1

1 OF 1

**METRIC**

W.P.	<u>GWP No. 6530-17-00</u>	LOCATION	<u>Hwy 599 ~ 4.6 km South of Savant Lake CNR Crossing MTM ON-15 252954.9E 5563524.6N</u>			ORIGINATED BY	<u>EF</u>		
DIST	<u>Thunder Bay</u>	HWY	<u>599</u>	BOREHOLE TYPE	<u>CME 55 Rubber Track / SSA / NW</u>			COMPILED BY	<u>KP</u>
DATUM	<u>Local</u>	DATE	<u>2021.11.23 - 2021.11.24</u>	LATITUDE	<u>50.20661</u>	LONGITUDE	<u>-90.72632</u>	CHECKED BY	<u>AM/SM</u>

[illegible]

ONTARIO MTO ADM-00262199-H0 - MTO 1&8 - HWY 599 - EL.GPJ ONTARIO MTO.GDT 2/1/22

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



Brampton, Ontario

## RECORD OF BOREHOLE No BH21-2

1 OF 1

METRIC

W.P. GWP No. 6530-17-00 LOCATION Hwy 599 ~ 4.6 km South of Savant Lake CNR Crossing MTM ON-15 252990.6E 5563540.2N ORIGINATED BY EF  
 DIST Thunder Bay HWY 599 BOREHOLE TYPE CME 750 Rubber Track / SSA / HW COMPILED BY KP  
 DATUM Local DATE 2021.11.28 - 2021.11.28 LATITUDE 50.20675 LONGITUDE -90.72582 CHECKED BY AM/SM

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									
416.3	Asphalt 2" thick							20	40	60	80	100					
416.0	<b>GRAVEL (FILL)</b> - frozen, grey		S1	AS			416										
0.2	<b>Sand and Gravel (FILL)</b> - some silt, brown, moist, dense		S2	AS													
			S3	AS				415									39 50 (11)
414.5			S4A	SS	19												
1.8	<b>Silty SAND</b> - trace gravel, light brown to grey, moist, dense		S4B	SS	20												9 53 (38)
414.2	- refusal to SPT and auger at about 2.1 m depth, rock coring techniques initiated		S5	CORE				414									UCS test at 2.4 m depth = 175 MPa Recovery=100% RQD=93%
2.1	<b>BEDROCK</b> - very strong, very sound, black/white to grey, medium to coarse grained - becoming severely fractured, green/white to grey, fine to coarse grained at about 3.1 m depth		S6	CORE				413									Recovery=99% RQD=38%
	- becoming sound, black/white to greenish grey at about 4.6 m depth		S7	CORE			412										Recovery=100% RQD=86%
411.0	<b>End of Borehole</b>																
5.3	- no groundwater measurements were recorded as the introduced water for rock coring purposes had not stabilized before backfilling																

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

Brampton, Ontario

# RECORD OF BOREHOLE No BH21-3

1 OF 1

METRIC

W.P. GWP No. 6530-17-00 LOCATION Hwy 599 ~ 4.6 km South of Savant Lake CNR Crossing MTM ON-15 252978.4E 5563535.7N ORIGINATED BY EF  
 DIST Thunder Bay HWY 599 BOREHOLE TYPE CME 750 Rubber Track / SSA / HW COMPILED BY KP  
 DATUM Local DATE 2021.11.27 - 2021.11.27 LATITUDE 50.20671 LONGITUDE -90.72599 CHECKED BY AM/SM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT  γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																														
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100			W <sub>P</sub>	W	W <sub>L</sub>																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																											
416.4	Asphalt 2" thick		S1	AS		▽	416																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																					

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

ONTARIO MTO ADM-00262199-H0 - MTO 1&8 - HWY 599 - EL.GPJ ONTARIO MTO.GDT 2/1/22

Brampton, Ontario

## RECORD OF BOREHOLE No BH21-4

1 OF 1

METRIC

W.P. GWP No. 6530-17-00 LOCATION Hwy 599 ~ 4.6 km South of Savant Lake CNR Crossing MTM ON-15 252970.2E 5563532N ORIGINATED BY EF  
 DIST Thunder Bay HWY 599 BOREHOLE TYPE CME 750 Rubber Track / SSA / HW COMPILED BY KP  
 DATUM Local DATE 2021.11.25 - 2021.11.26 LATITUDE 50.20668 LONGITUDE -90.72611 CHECKED BY AM/SM

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								
								20	40	60	80	100				
416.5	Asphalt 2" thick		S1	AS												
416.0	Gravel (FILL) - frozen, grey		S2	AS												
0.2	Sand and Gravel (FILL) - some silt, brown, moist, dense, poorly graded		S3	AS												
			S4	AS												
			S5	SS	39											38 50 (12)
			S6	SS	9											
			S7	SS	5											
			S8	SS	16											40 47 (13)
411.9	PEAT with Silt and Sand - trace gravel, dark brown, moist to wet, dense, some rootlets		S9	SS	44											
411.2																
5.3	SILT (TILL) - some sand, grey, wet, very dense, cobbles and boulders - refusal to SPT and auger at about 5.6 m depth, rock coring techniques initiated - void/soil noted from about 6.3 m to 6.8 m depth		S10	SS	50/ 125mm											
			S11	CORE												
409.7			S12	CORE												Recovery=99% RQD=76% UCS test at 7.0 m depth = 134 MPa
6.8	BEDROCK - very strong, severely fractured to fractured, black/white to greenish grey, some pink, medium to coarse grained		S13	CORE												Recovery=100% RQD=73%
407.4																
9.1	End of Borehole  - groundwater was measured 4.4 m below ground surface															

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

ONTARIO MTO ADM-00262199-H0 - MTO 1&amp;8 - HWY 599 - EL.GPJ ONTARIO.MTO.GDT 2/1/22

Brampton, Ontario

## RECORD OF BOREHOLE No TP21-1

1 OF 1

METRIC

W.P. GWP No. 6530-17-00 LOCATION Hwy 599 ~ 4.6 km South of Savant Lake CNR Crossing MTM ON-15 252973.6E 5563516.9N ORIGINATED BY EF  
 DIST Thunder Bay HWY 599 BOREHOLE TYPE Link-Belt 235 Track Excavator COMPILED BY KP  
 DATUM Local DATE 2021.11.26 - 2021.11.26 LATITUDE 50.20654 LONGITUDE -90.72606 CHECKED BY AM/SM

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								WATER CONTENT (%)		
411.6	Topsoil		S1	GRAB		▽	411								GR SA SI CL			
410.8	<b>TOPSOIL</b> - some blast rock, frozen, dark brown		S2	GRAB														
	<b>PEAT with Silt and Sand</b> - trace to some gravel, occasional cobbles and boulders, dark brown, wet		S3	GRAB														
410.4	<b>SILT to Clayey SILT</b> - trace to some sand, occasional boulders, light grey to grey, wet  - trace gravel at about 2.3 m depth	S4	GRAB		410												0 12 77 11	
		S5	GRAB															
		S6	GRAB															
		S7	GRAB															
409.0		S8	GRAB		409												0 2 82 16 1 3 82 14	
	<b>Silty SAND with Gravel</b> - occasional cobbles and boulders, dark brown, wet																	
408.1	<b>End of Test Pit</b> - refusal  - groundwater was measured 0.6 m below ground surface																	
3.5																		

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





Brampton, Ontario

## RECORD OF BOREHOLE No BH20-1

1 OF 1

METRIC

W.P. GWP No. 6530-17-00 LOCATION Hwy 599 ~4.6 km South of Savant Lake CNR Crossing MTM ON-15 252977.6E 5563540.7N ORIGINATED BY EF  
 DIST Thunder Bay HWY 599 BOREHOLE TYPE B54X Rubber Track SSA / HQ COMPILED BY AM  
 DATUM Local DATE 2020.11.03 - 2020.11.03 LATITUDE 50.20676 LONGITUDE -90.726 CHECKED BY DG/SM

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			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL P. PENETROMETER				W <sub>P</sub>	W	W <sub>L</sub>		GR	SA	SI	CL		
416.5	Asphalt treatment 1" thick		S1	AS			416														
0.0	<b>Sand and Gravel (FILL)</b> trace silt and clay, grey, moist, compact, poorly graded, crushed material in upper 0.2 m - becoming brown, occasional cobbles at about 0.2 m depth		S2	AS																	
			S3	AS																	
			S4	AS																	
414.9			S5	AS					415												
1.5	<b>Gravelly Sand (FILL)</b> some silt and clay, occasional cobbles, brown, moist, compact		S6	AS					414												
			S7	AS					413												
412.8	<b>BEDROCK</b> strong, severely fractured to sound, white/pink to grey, medium to coarse grained		S8	CORE					412												
3.7			S9	CORE					411												
			S10	CORE					410												
409.7																					
6.8	<b>End of Borehole</b>  - groundwater was measured 3.1 m below ground surface																				

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

ONTARIO MTO ADM-00262199-H0 - MTO 1&amp;8 - HWY 599 - EL.GPJ ONTARIO MTO.GDT 2/1/22

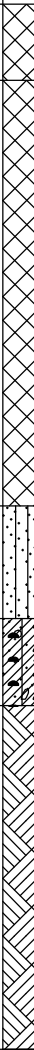
Brampton, Ontario

## RECORD OF BOREHOLE No BH20-2

1 OF 1

METRIC

W.P. GWP No. 6530-17-00 LOCATION Hwy 599 ~4.6 km South of Savant Lake CNR Crossing MTM ON-15 252968.2E 5563531.1N ORIGINATED BY EF  
 DIST Thunder Bay HWY 599 BOREHOLE TYPE B54X Rubber Track SSA / HSA / HQ COMPILED BY AM  
 DATUM Local DATE 2020.11.03 - 2020.11.06 LATITUDE 50.20667 LONGITUDE -90.72614 CHECKED BY DG/SM

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  <b>γ</b>  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								WATER CONTENT (%)				GR	SA	SI	CL
416.5	Asphalt treatment 1" thick		S1	AS		416										24	65	10	1				
0.0	<b>Sand and Gravel (FILL)</b> trace silt and clay, grey, moist, compact, poorly graded, crushed material in upper 0.2 m		S2	AS																			
415.8	<b>Gravelly Sand (FILL)</b> some silt and clay, occasional cobbles, brown, moist, compact		S3	AS			415																
0.8			S4	AS																			
			S5	AS																			
			S6	AS																			
			S7	SS	100																		
	- refusal to SPT and auger at about 4.4 m depth, rock coring techniques initiated				414																		
411.5	<b>Sandy SILT with Gravel</b> trace clay, very dense, grey, wet	S8	CORE																				
5.0		S9	SS	50/50mm				411															
410.4		SS	50/50mm																				
6.1	<b>Silty SAND with Gravel (TILL)</b> trace clay, , occasional to some cobbles and boulders, grey, wet, dense to very dense	S10	SS	50/50mm/33					410														
409.6		S11	SS	50/150mm																			
7.0	<b>BEDROCK</b> strong, fractured, white and black, fine to coarse grained	S12	CORE							409													
		S13	CORE																				
		S14	CORE			408																	
							407																
</																							

ONTARIO MTO ADM-00262199-H0 - MTO 1&amp;8 - HWY 599 - EL GPJ ONTARIO MTO GDT 2/1/22

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



Brampton, Ontario

# RECORD OF BOREHOLE No BH20-3

1 OF 1

METRIC

W.P. GWP No. 6530-17-00 LOCATION Hwy 599 ~4.6 km South of Savant Lake CNR Crossing MTM ON-15 252994.7E 5563542.0N ORIGINATED BY EF  
 DIST Thunder Bay HWY 599 BOREHOLE TYPE B54X Rubber Track SSA / HSA / HQ COMPILED BY AM  
 DATUM Local DATE 2020.11.05 - 2020.11.06 LATITUDE 50.20677 LONGITUDE -90.72576 CHECKED BY DG/SM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)						
416.1	Asphalt treatment 1" thick		S1A	SS	41		416						kN/m³	GR SA SI CL		
0.0	<b>Sand and Gravel (FILL)</b> trace silt and clay, occasional cobbles, grey to brown, moist, crushed material, dense to very dense		S1B	SS	39											
			S2	SS	65											
			S3	SS	50/125mm											
413.8	<b>BEDROCK</b> very strong, fractured, green to blue, fine grained		S4	CORE			413							UCS test at 2.8 m depth = 103 MPa Recovery=99%, RQD=56% UCS test at 3.5 m depth = 95 MPa		
2.3	- becoming strong, white and black, medium to coarse grained at about 3.2 m depth															
412.2	<b>End of Borehole</b>															
3.9	- no obtainable groundwater level due to caved borehole															

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

Brampton, Ontario

# RECORD OF BOREHOLE No HP20-4

1 OF 1

METRIC

W.P. GWP No. 6530-17-00 LOCATION Hwy 599 ~4.6 km South of Savant Lake CNR Crossing MTM ON-15 252950.6E 5563540.8N ORIGINATED BY EF  
 DIST Thunder Bay HWY 599 BOREHOLE TYPE Power Hand Auger / SSA COMPILED BY AM  
 DATUM Local DATE 2020.11.05 - 2020.11.05 LATITUDE 50.20676 LONGITUDE -90.72638 CHECKED BY DG/SM

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							
414.2	Topsoil		S1	AS		414	20	40	60	80	100	20	40	60	GR SA SI CL
414.0	<b>TOPSOIL</b> trace gravel, some sand, some silt, occasional to some cobbles, occasional boulders, dark brown, wet, loose		S2	AS											
			S3	AS											
413.4			S4	AS											
0.8	<b>Silty SAND</b> some gravel, occasional cobbles and boulders, some peat to peaty in upper 0.3 m dark brown, wet to moist, loose to compact <b>End of Borehole</b> - refusal - no groundwater encountered - bedrock outcrop observed about 3 m from borehole														



Brampton, Ontario

# RECORD OF BOREHOLE No HP20-5

1 OF 1

METRIC

W.P. GWP No. 6530-17-00 LOCATION Hwy 599 ~4.6 km South of Savant Lake CNR Crossing MTM ON-15 252961.8E 5563545.4N ORIGINATED BY EF  
 DIST Thunder Bay HWY 599 BOREHOLE TYPE Power Hand Auger / SSA COMPILED BY AM  
 DATUM Local DATE 2020.11.05 - 2020.11.05 LATITUDE 50.2068 LONGITUDE -90.72623 CHECKED BY DG/SM

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT $\gamma$	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			SHEAR STRENGTH kPa					W <sub>p</sub>	W	W <sub>L</sub>		
413.9	Topsoil		S1	AS			20	40	60	80	100					
0.0	TOPSOIL trace gravel, some sand, some silt, occasional cobbles, dark brown, moist to wet, loose		S2	AS												
413.3			S3	AS												
410.8	Sandy GRAVEL some silt, occasional cobbles and boulders, light brown, moist, compact		S4	AS												
0.7	End of Borehole - refusal															
	- no groundwater encountered															
	- bedrock outcrop observed about 3 m from borehole															

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

Brampton, Ontario

## RECORD OF BOREHOLE No HP20-6

1 OF 1

METRIC

W.P. GWP No. 6530-17-00 LOCATION Hwy 599 ~4.6 km South of Savant Lake CNR Crossing MTM ON-15 252974.1E 5563552.9N ORIGINATED BY EF  
 DIST Thunder Bay HWY 599 BOREHOLE TYPE Power Hand Auger / SSA COMPILED BY AM  
 DATUM Local DATE 2020.11.04 - 2020.11.04 LATITUDE 50.20687 LONGITUDE -90.72605 CHECKED BY DG/SM

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									
413.8	Topsoil		S1	AS		413	20	40	60	80	100	20	40	60		GR SA SI CL	
413.7	<b>TOPSOIL</b> trace gravel, some sand, some silt, brown, moist to wet, loose		S2	AS													
			S3	AS													
413.0	<b>Silty SAND</b> occasional cobbles, brown, moist, loose to compact, roots in upper 0.5 m - trace peat at about 0.3 m depth <b>End of Borehole</b> - refusal - no groundwater encountered		S4	AS													3 64 (33)
0.8	<b>End of Borehole</b> - refusal - no groundwater encountered  - bedrock outcrop observed about 3 m from borehole																

ONTARIO MTO ADM-00262199-H0 - MTO 1&amp;8 - HWY 599 - EL.GPJ ONTARIO MTO.GDT 2/1/22

Brampton, Ontario

# RECORD OF BOREHOLE No HP20-6I

1 OF 1

METRIC

W.P. GWP No. 6530-17-00 LOCATION Hwy 599 ~4.6 km South of Savant Lake CNR Crossing MTM ON-15 252978.1E 5563553.7N ORIGINATED BY EF  
 DIST Thunder Bay HWY 599 BOREHOLE TYPE Power Hand Auger / SSA COMPILED BY AM  
 DATUM Local DATE 2020.11.05 - 2020.11.05 LATITUDE 50.20687 LONGITUDE -90.726 CHECKED BY DG/SM

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									
413.9	Topsoil							20	40	60	80	100					
413.9	<b>TOPSOIL</b> trace gravel, some sand, some silt, occasional cobbles, brown, moist to wet, loose		S1	AS													
413.4			S2	AS													
413.4			S3	AS													
0.5	<b>Silty SAND</b> occasional cobbles and boulders, brown, moist, compact																
	<b>End of Borehole</b> refusal																
	- no groundwater encountered																
	- bedrock outcrop observed about 3 m from borehole																

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

Brampton, Ontario

# RECORD OF BOREHOLE No HP20-7

1 OF 1

METRIC

W.P. GWP No. 6530-17-00 LOCATION Hwy 599 ~4.6 km South of Savant Lake CNR Crossing MTM ON-15 252984.2E 5563556.5N ORIGINATED BY EF  
 DIST Thunder Bay HWY 599 BOREHOLE TYPE Power Hand Auger / SSA COMPILED BY AM  
 DATUM Local DATE 2020.11.04 - 2020.11.04 LATITUDE 50.2069 LONGITUDE -90.72591 CHECKED BY DG/SM

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									
414.5	Topsoil		S1	AS												GR SA SI CL	
414.4	<b>TOPSOIL</b> trace gravel, some sand, some silt, occasional cobbles, brown, moist to wet, loose		S2	AS												6 79 (15)	
414.3	<b>Silty SAND</b> occasional cobbles, brown, moist, compact to dense																
414.2	<b>End of Borehole</b> refusal																
	- no groundwater encountered																
	- bedrock outcrop observed about 3 m from borehole																
													</				

ONTARIO MTO ADM-00262199-H0 - MTO 1&8 - HWY 599 - EL.GPJ ONTARIO MTO.GDT 2/1/22

Brampton, Ontario

## RECORD OF BOREHOLE No HP20-8

1 OF 1

METRIC

W.P. GWP No. 6530-17-00 LOCATION Hwy 599 ~4.6 km South of Savant Lake CNR Crossing MTM ON-15 252962.4E 5563512.3N ORIGINATED BY EF  
 DIST Thunder Bay HWY 599 BOREHOLE TYPE Power Hand Auger / SSA COMPILED BY AM  
 DATUM Local DATE 2020.11.02 - 2020.11.02 LATITUDE 50.2065 LONGITUDE -90.72621 CHECKED BY DG/SM

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 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL P. PENETROMETER			WATER CONTENT (%) q <sub>d</sub> f <sub>sc</sub>						
411.9	Peat		S1	AS										kN/m <sup>3</sup>	GR SA SI CL
411.8	<b>PEAT AND SAND</b> some cobbles and boulders, dark brown, wet, loose		S2	AS											
0.3	<b>End of Borehole</b> refusal														6 57 (37)
	- three additional hand probes / digging were conducted in about a 2.5 m radius, all with similar findings														
	- groundwater was measured 0.2 m below ground surface														

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



Brampton, Ontario

# RECORD OF BOREHOLE No HP20-9

1 OF 1

METRIC

W.P. GWP No. 6530-17-00 LOCATION Hwy 599 ~4.6 km South of Savant Lake CNR Crossing MTM ON-15 252972.7E 5563518.9N ORIGINATED BY EF  
 DIST Thunder Bay HWY 599 BOREHOLE TYPE Power Hand Auger / SSA COMPILED BY AM  
 DATUM Local DATE 2020.11.02 - 2020.11.02 LATITUDE 50.20656 LONGITUDE -90.72607 CHECKED BY DG/SM

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 (%)																		
411.6	Peat		S1	AS																									
0.0	<b>PEAT AND SAND</b> some silt, some cobbles and boulders, dark brown, wet, loose		S2	AS																									
411.3	<b>End of Borehole</b> refusal																												
0.3	<div>- five additional hand probes / digging were conducted in about a 3.0 m radius, all with similar findings</div> <div>- groundwater was measured 0.2 m below ground surface</div>																												

Brampton, Ontario

## RECORD OF BOREHOLE No HP20-10

1 OF 1

METRIC

W.P. GWP No. 6530-17-00 LOCATION Hwy 599 ~4.6 km South of Savant Lake CNR Crossing MTM ON-15 252986.0E 5563526.4N ORIGINATED BY EF  
 DIST Thunder Bay HWY 599 BOREHOLE TYPE Power Hand Auger / SSA COMPILED BY AM  
 DATUM Local DATE 2020.11.02 - 2020.11.02 LATITUDE 50.20663 LONGITUDE -90.72588 CHECKED BY DG/SM

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 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL P. PENETROMETER				WATER CONTENT (%)							GR	SA	SI	CL
413.1	Peat		S1	AS			413													
0.0	<b>PEAT AND SAND</b> some gravel, some silt, some cobbles and boulders, dark brown to black, wet, loose  <b>Silty SAND with Gravel</b> some cobbles, occasional boulders, brown, moist, compact to dense <b>End of Borehole</b> refusal  - no groundwater encountered   <																			

ONTARIO MTO ADM-00262199-H0 - MTO 1&amp;8 - HWY 599 - EL.GPJ ONTARIO MTO.GDT 2/1/22

Brampton, Ontario

# RECORD OF BOREHOLE No HP20-11

1 OF 1

METRIC

W.P. GWP No. 6530-17-00 LOCATION Hwy 599 ~4.6 km South of Savant Lake CNR Crossing MTM ON-15 252992.1E 5563529.1N ORIGINATED BY EF  
 DIST Thunder Bay HWY 599 BOREHOLE TYPE Power Hand Auger / SSA COMPILED BY AM  
 DATUM Local DATE 2020.11.02 - 2020.11.02 LATITUDE 50.20665 LONGITUDE -90.7258 CHECKED BY DG/SM

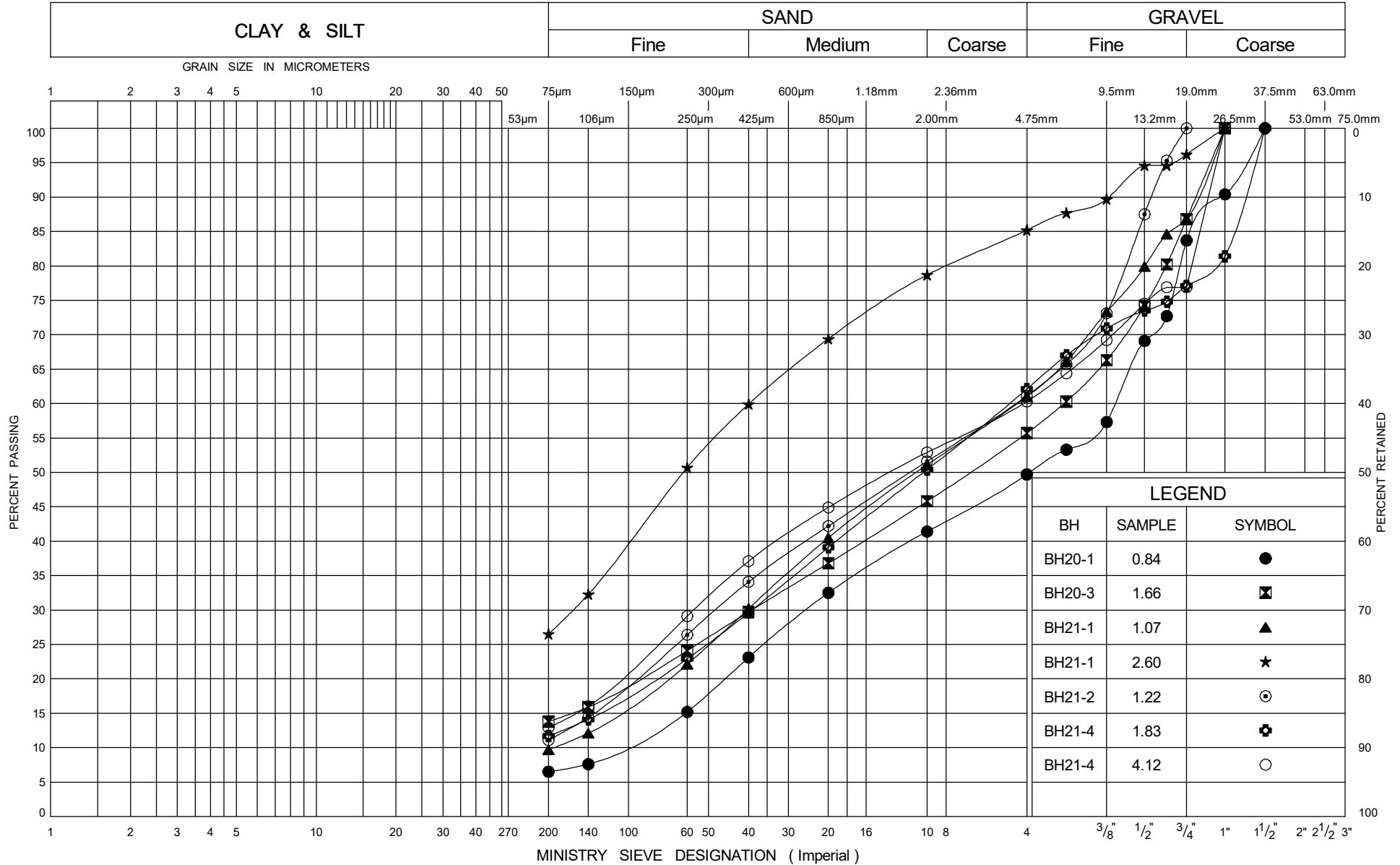
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										
414.9	Topsoil		S1	AS														
0.0	<b>TOPSOIL</b> trace gravel, some sand, some silt, occasional cobbles, dark brown, moist to wet, loose		S2	AS														
414.6			S3	AS														
0.3	<b>Silty SAND with Gravel</b> some cobbles, occasional boulders, brown, moist, compact to dense																	
414.2																		
0.7	<b>End of Borehole</b> refusal																	
	- no groundwater encountered																	
	- bedrock outcrop observed about 1.5 m from borehole																	

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

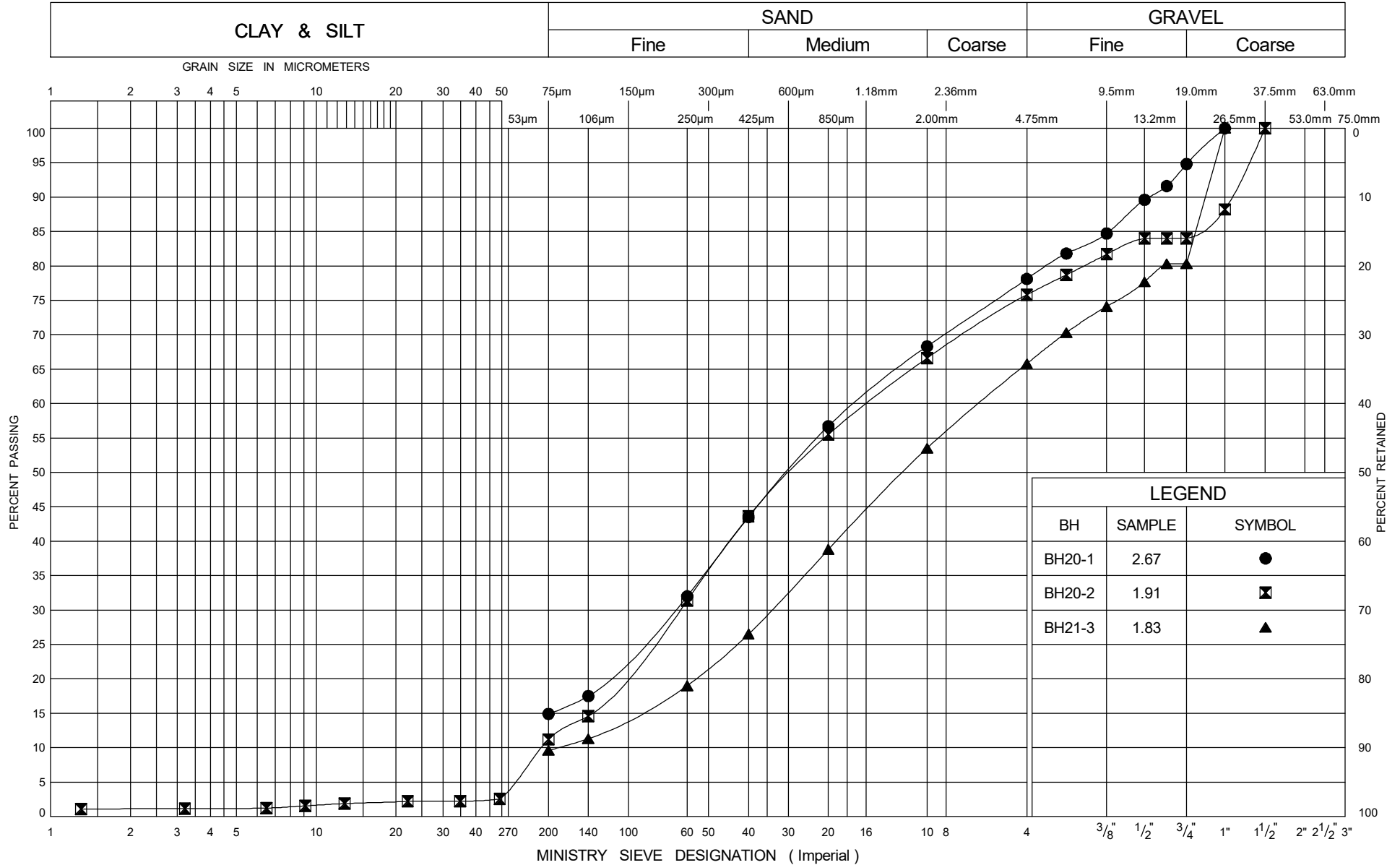


## Appendix D – Laboratory Data

# UNIFIED SOIL CLASSIFICATION SYSTEM



# UNIFIED SOIL CLASSIFICATION SYSTEM



## GRAIN SIZE DISTRIBUTION

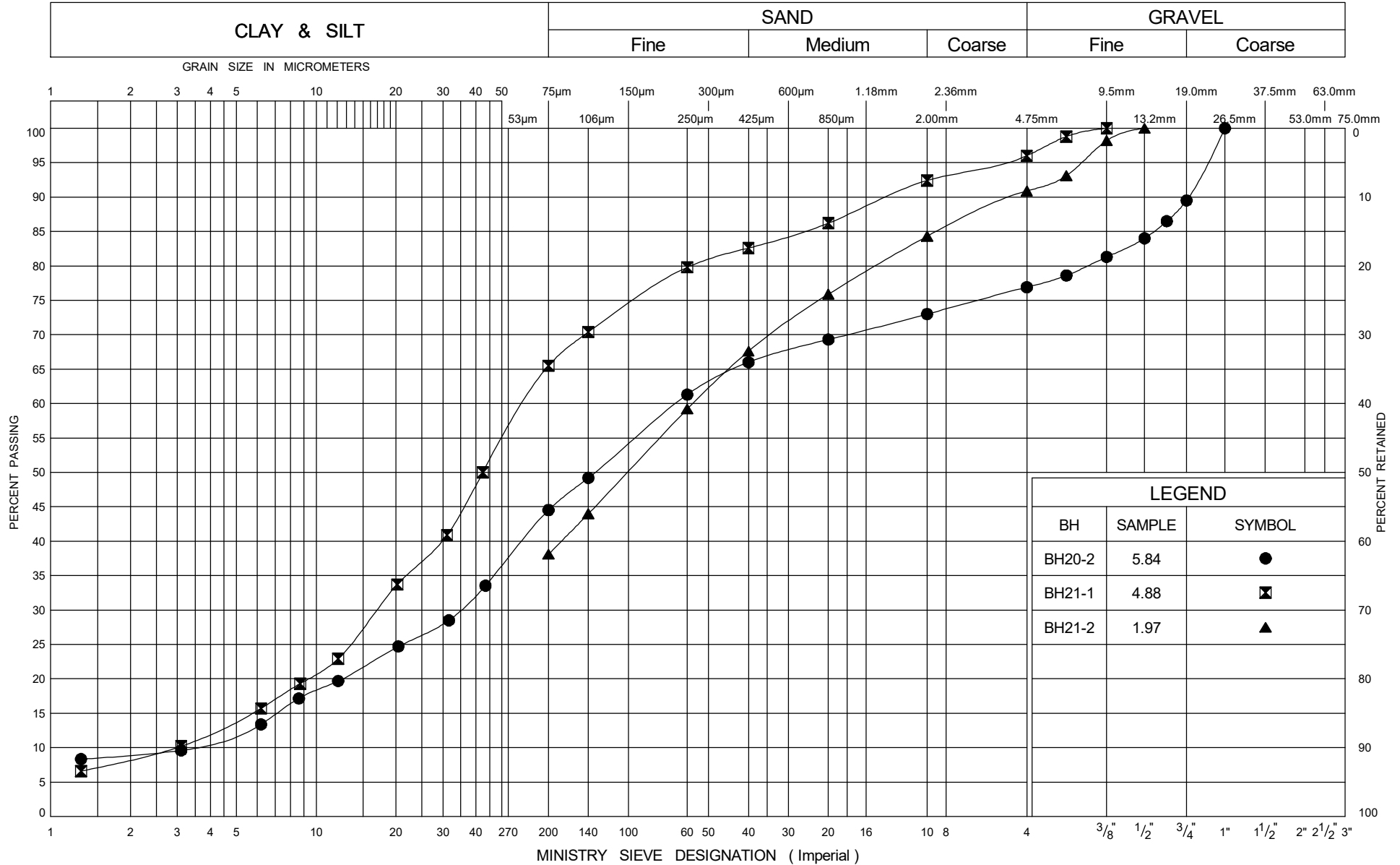
Fill: Gravelly Sand

FIG No 2

W P GWP No. 6530-17-00

6019-E-0004, Assignment 8

# UNIFIED SOIL CLASSIFICATION SYSTEM



Ministry of  
Transportation

## GRAIN SIZE DISTRIBUTION

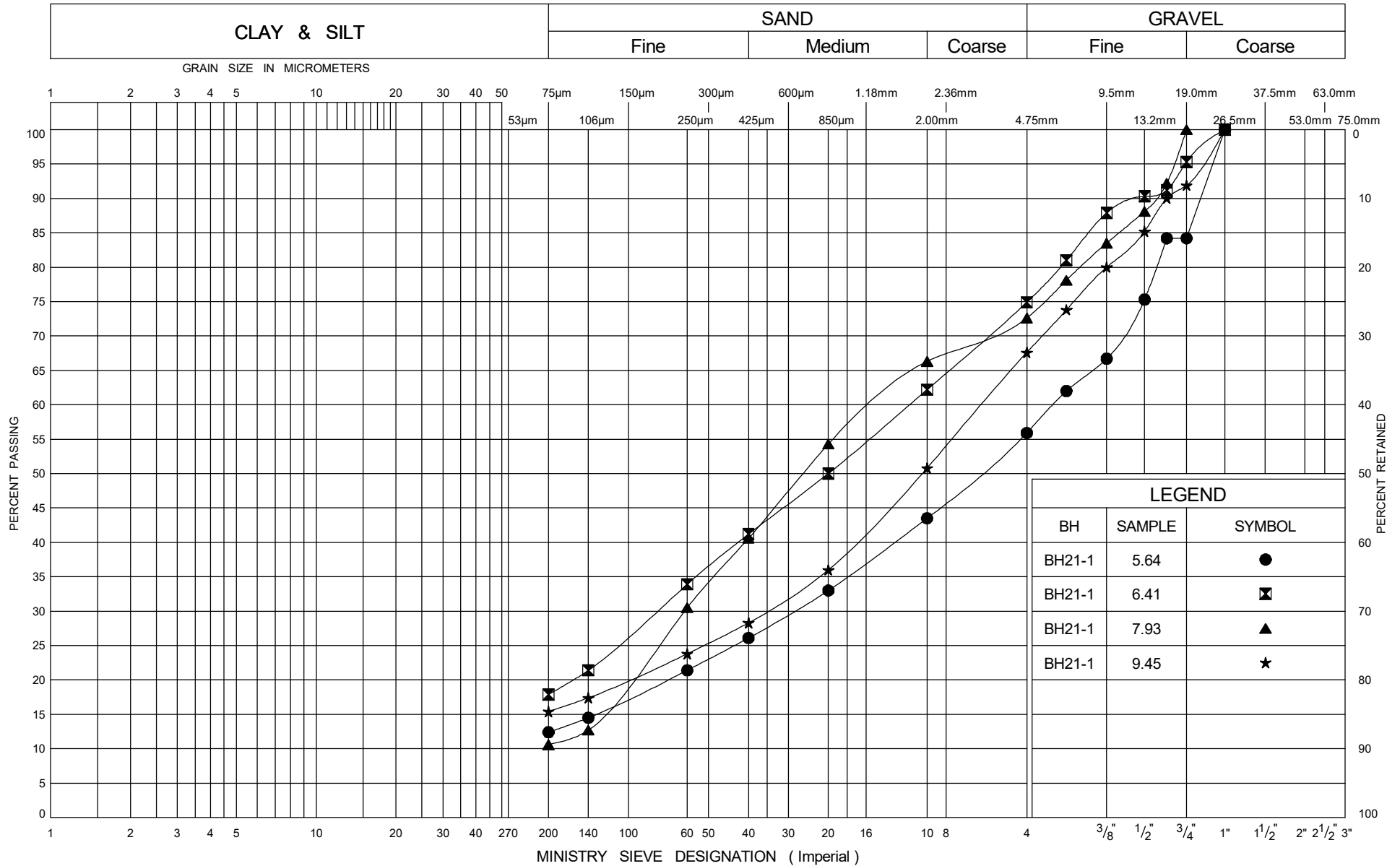
Silty Sand to Sandy Silt

FIG No 3

W P GWP No. 6530-17-00

6019-E-0004, Assignment 8

# UNIFIED SOIL CLASSIFICATION SYSTEM



## GRAIN SIZE DISTRIBUTION

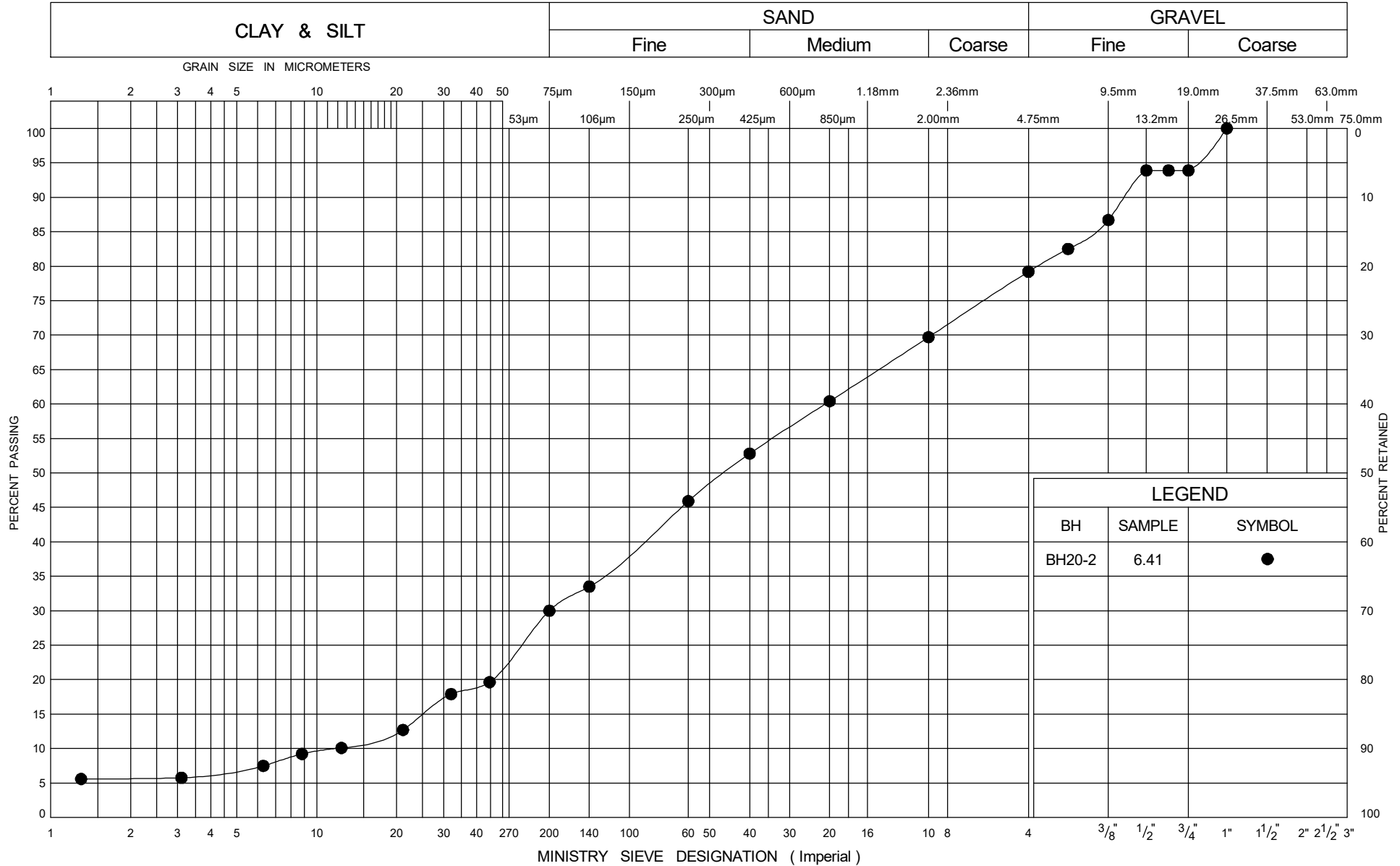
Gravelly Sand

FIG No 4

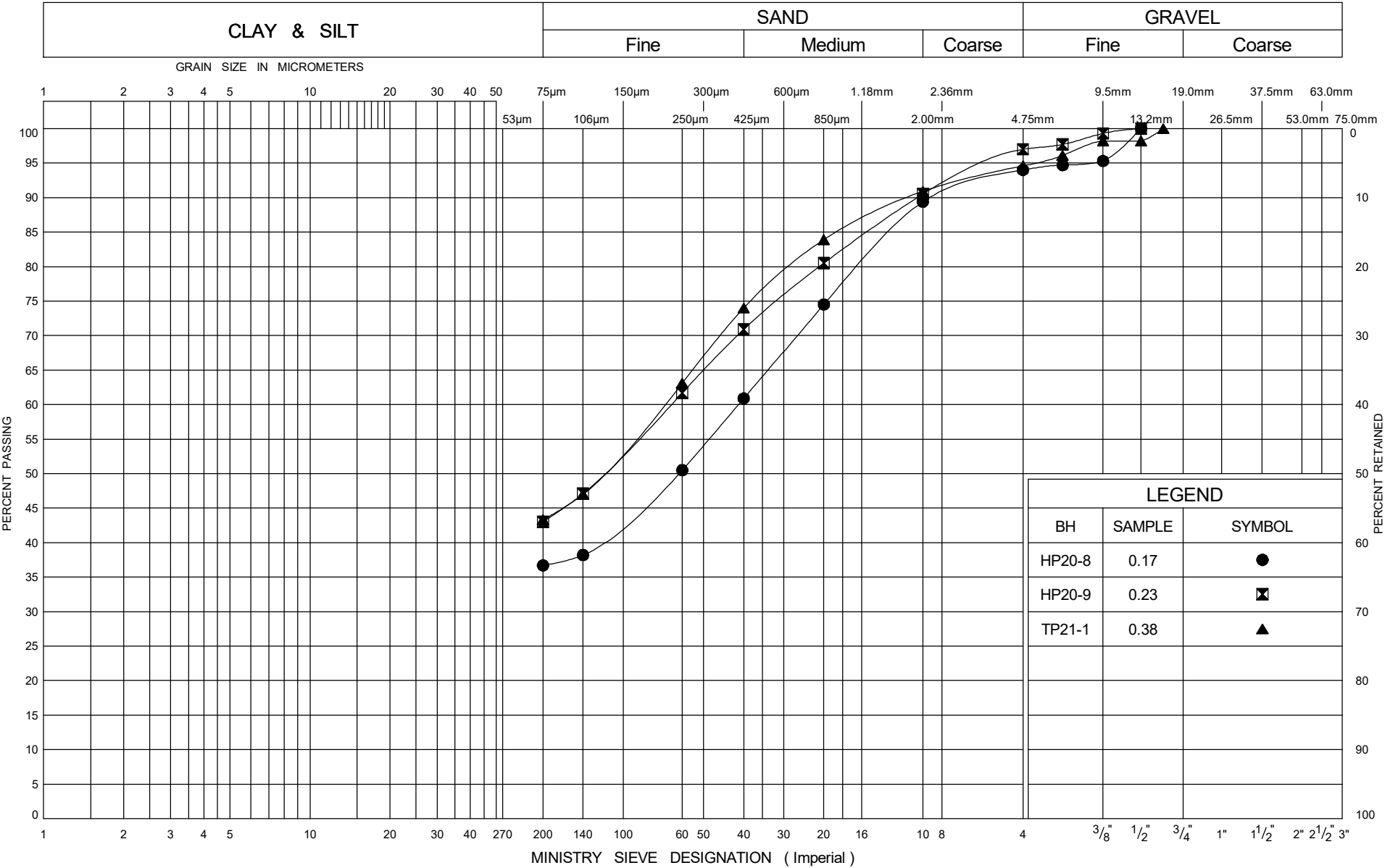
W P GWP No. 6530-17-00

6019-E-0004, Assignment 8

# UNIFIED SOIL CLASSIFICATION SYSTEM



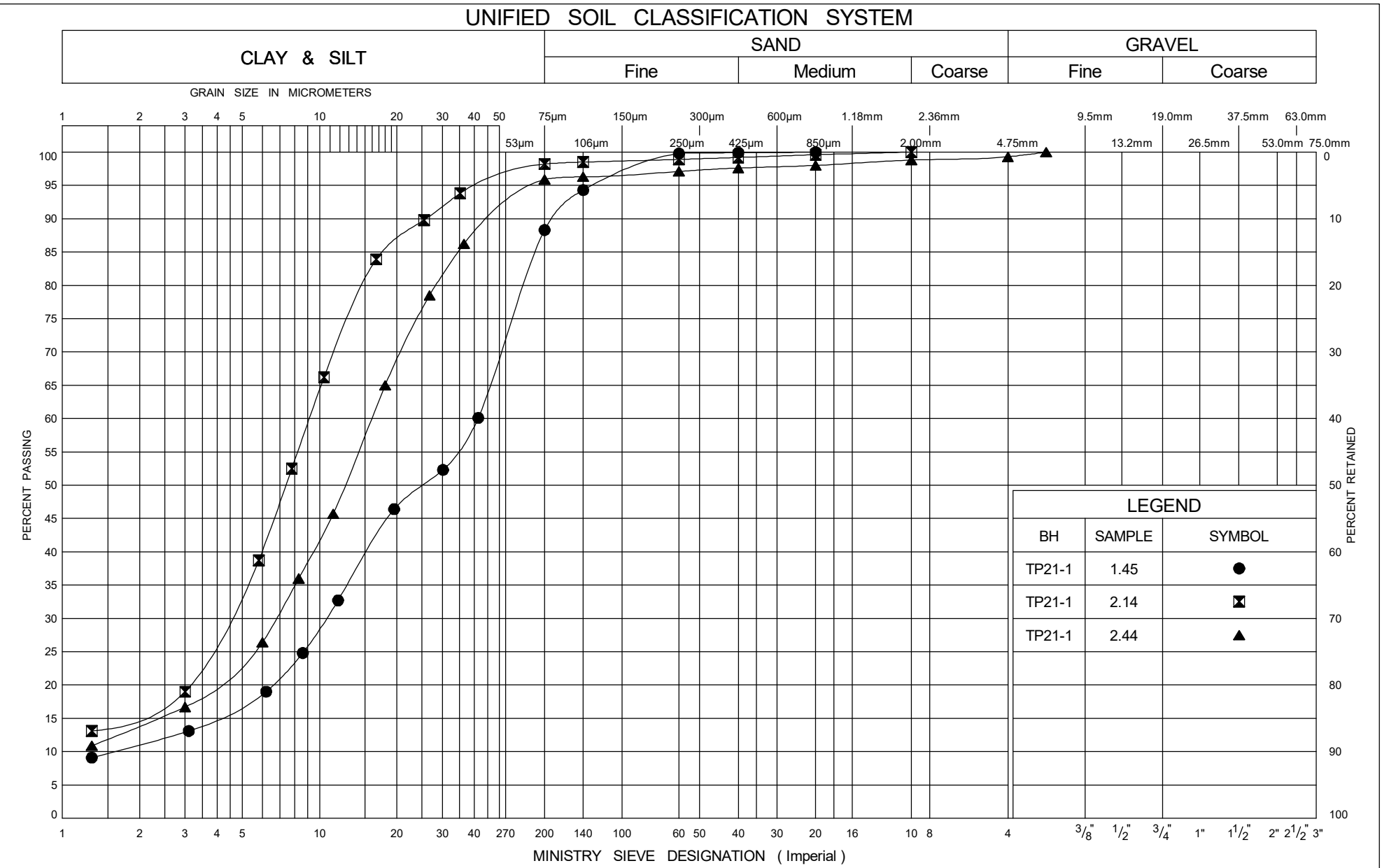
UNIFIED SOIL CLASSIFICATION SYSTEM



GRAIN SIZE DISTRIBUTION

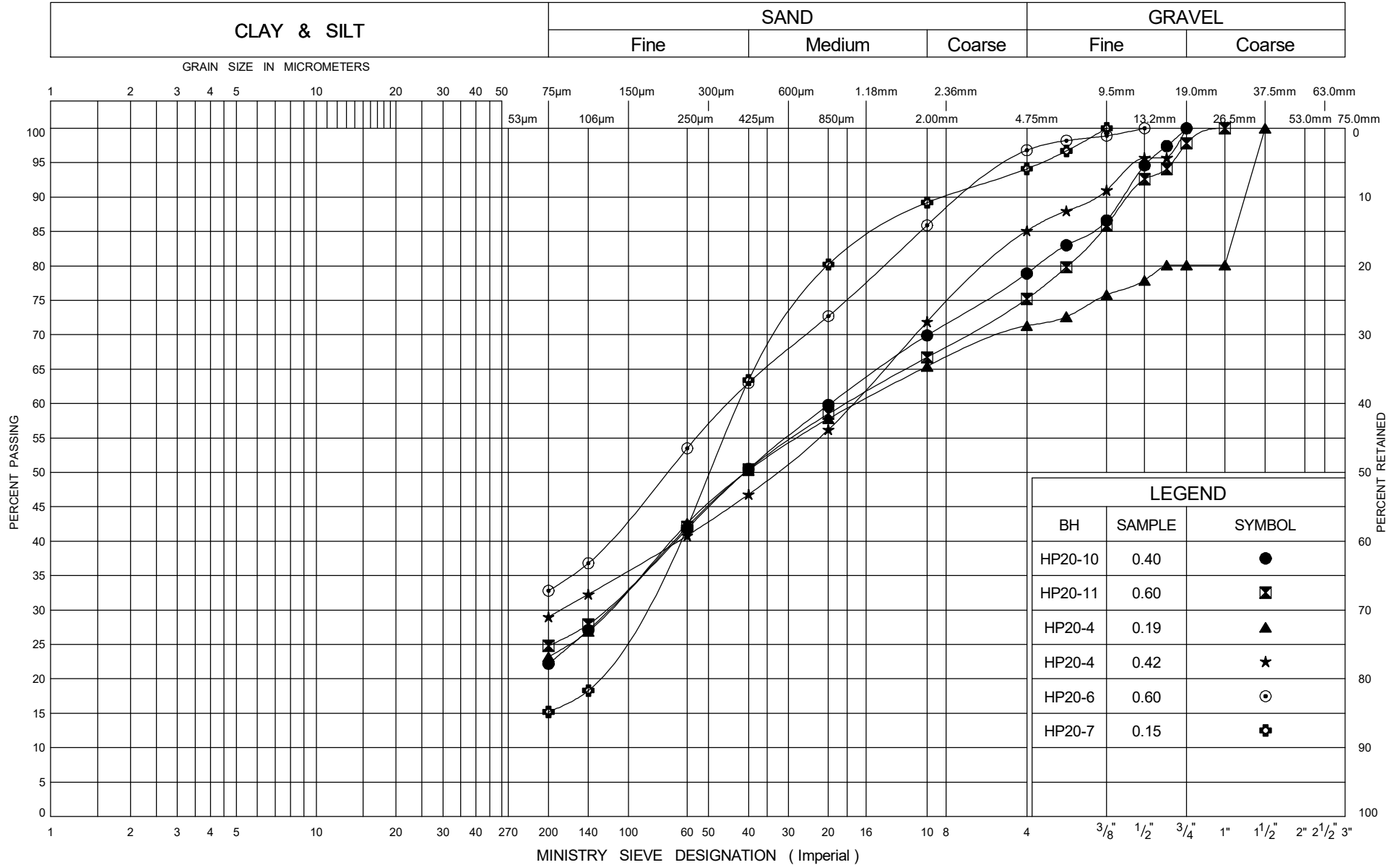
Peat and Sand

FIG No 6  
W P GWP No. 6530-17-00  
6019-E-0004, Assignment 8

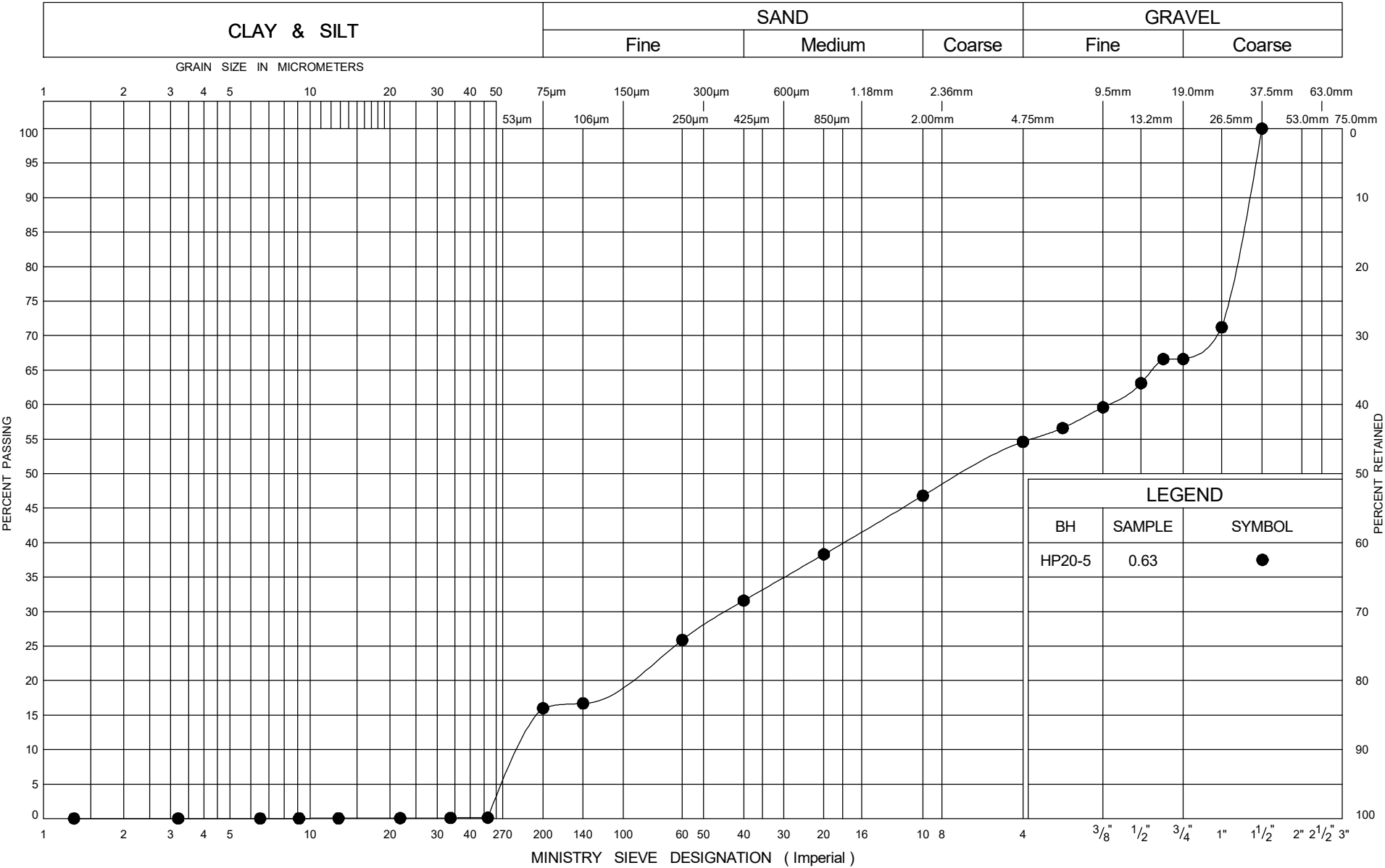




# UNIFIED SOIL CLASSIFICATION SYSTEM



UNIFIED SOIL CLASSIFICATION SYSTEM

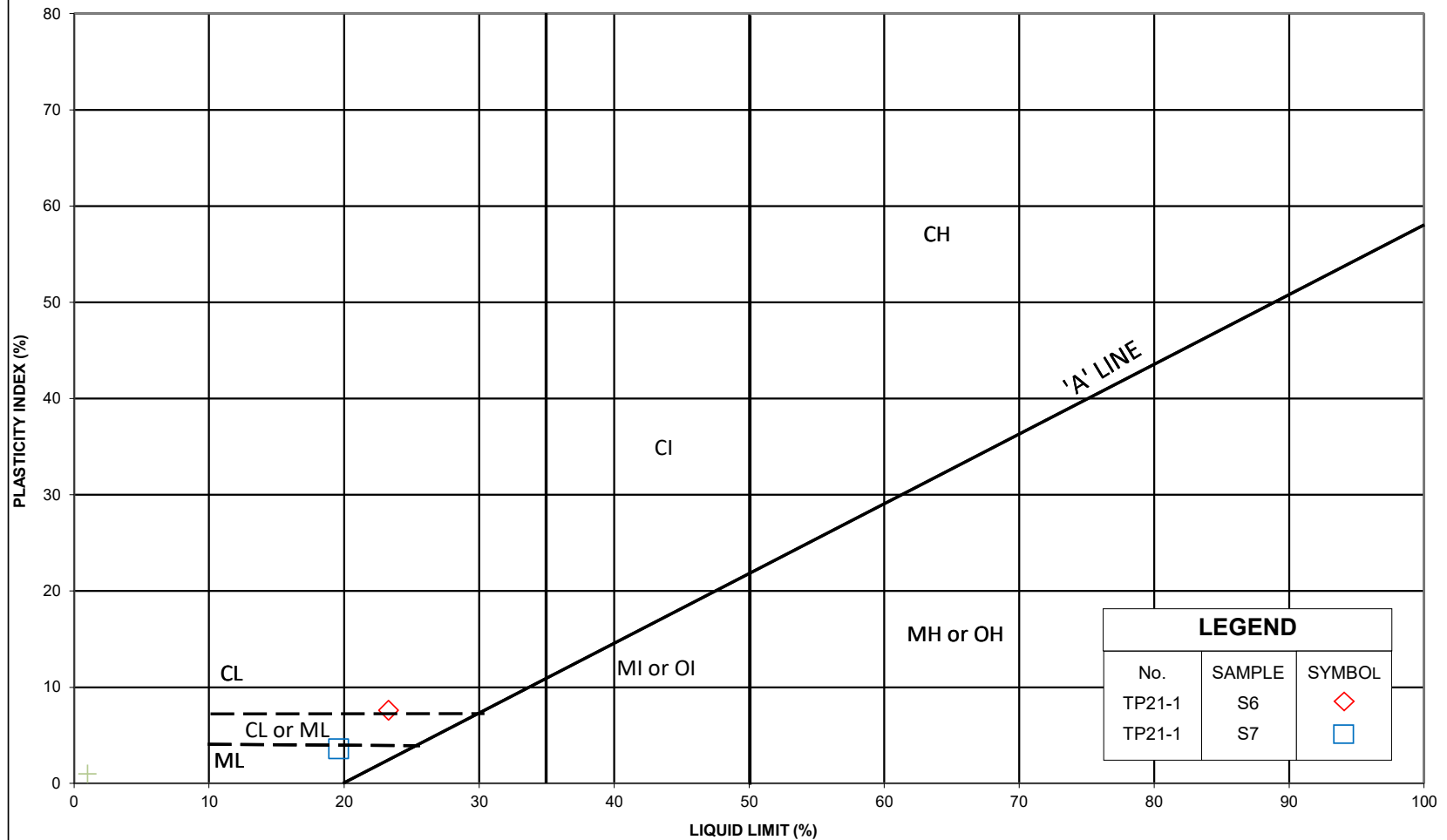


GRAIN SIZE DISTRIBUTION

Sandy Gravel

FIG No 9  
W P GWP No. 6530-17-00  
6019-E-0004, Assignment 8

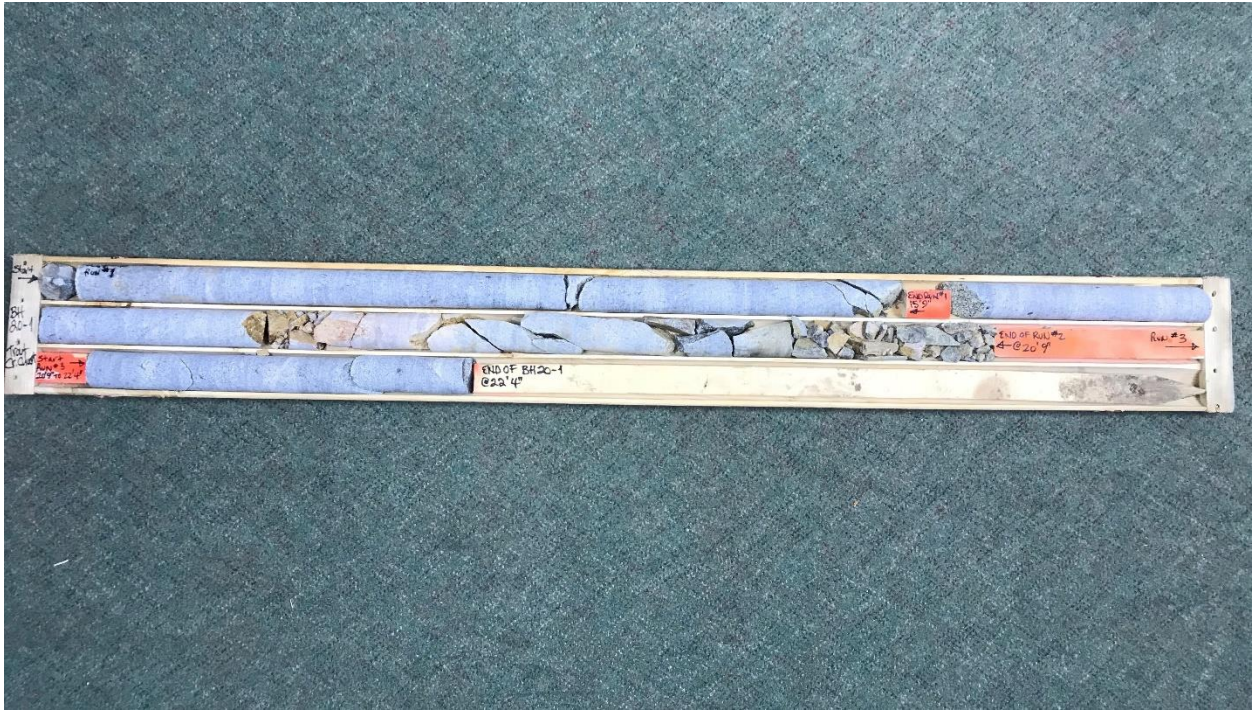
**Trout Creek Culvert Replacement**  
**GWP No. 6530-17-00, Highway 599, Savant Lake, Ontario**



**PLASTICITY CHART**  
**SILT to Clayey SILT**

FIGURE No. 10  
 ADM-00262199-H0  
 January 20, 2022

## Appendix E – Bedrock Core Photographs



Photograph E1. Bedrock Core Samples, Borehole **BH20-1**, November 2020



Photograph E2. Bedrock Core Samples, Borehole **BH20-2**, November 2020

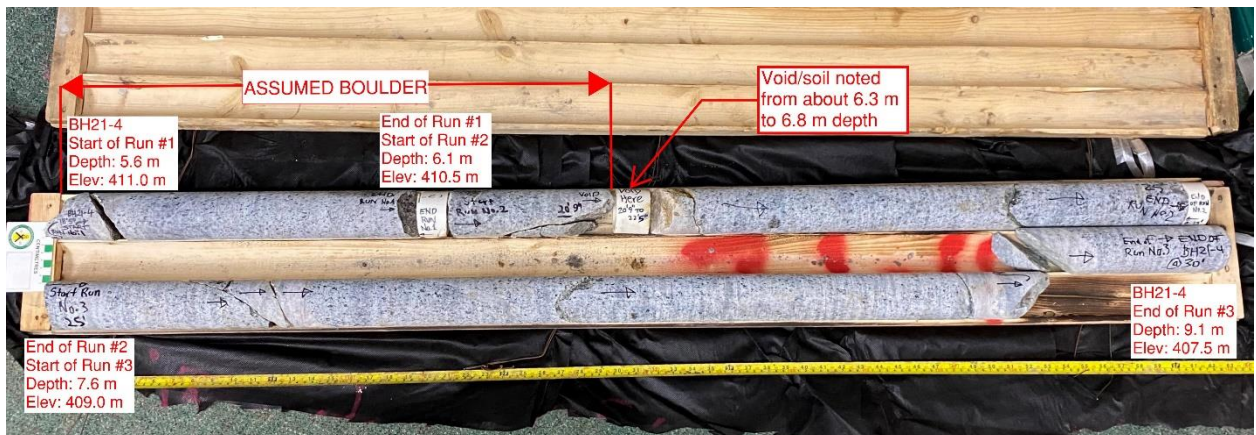




Photograph E3. Bedrock Core Samples, Borehole **BH21-2**, November 2021



Photograph E4. Bedrock Core Samples, Borehole **BH21-3**, November 2021



Photograph E5. Assumed Boulder (5.6 m to 6.8 m depth) and Bedrock Core Samples (6.8 m to 9.1 m), Borehole **BH21-4**, November 2021

## Appendix F – Chemical Analysis





Your Project #: ADM-00262199-A0  
Site Location: TROUT CREEK CULVERT, HWY 599  
Your C.O.C. #: n/a

**Attention: Ahileas Mitsopoulos**

exp Services Inc  
Thunder Bay Branch  
1142 Roland St  
Thunder Bay, ON  
CANADA P7B 5M4

**Report Date: 2020/11/24**  
Report #: R6423596  
Version: 1 - Final

**CERTIFICATE OF ANALYSIS**

**BV LABS JOB #: C0U9063**

**Received: 2020/11/19, 13:50**

Sample Matrix: Soil  
# Samples Received: 3

Analyses	Quantity	Date	Date	Laboratory Method	Analytical Method
		Extracted	Analyzed		
Chloride (20:1 extract)	3	2020/11/21	2020/11/23	CAM SOP-00463	SM 23 4500-Cl E m
Conductivity	3	2020/11/24	2020/11/24	CAM SOP-00414	OMOE E3530 v1 m
pH CaCl2 EXTRACT	3	2020/11/24	2020/11/24	CAM SOP-00413	EPA 9045 D m
Resistivity of Soil	3	2020/11/20	2020/11/24	CAM SOP-00414	SM 23 2510 m
Sulphate (20:1 Extract)	3	2020/11/21	2020/11/23	CAM SOP-00464	EPA 375.4 m

**Remarks:**

Bureau Veritas Laboratories are accredited to ISO/IEC 17025 for specific parameters on scopes of accreditation. Unless otherwise noted, procedures used by BV Labs are based upon recognized Provincial, Federal or US method compendia such as CCME, MELCC, EPA, APHA.

All work recorded herein has been done in accordance with procedures and practices ordinarily exercised by professionals in BV Labs profession using accepted testing methodologies, quality assurance and quality control procedures (except where otherwise agreed by the client and BV Labs in writing). All data is in statistical control and has met quality control and method performance criteria unless otherwise noted. All method blanks are reported; unless indicated otherwise, associated sample data are not blank corrected. Where applicable, unless otherwise noted, Measurement Uncertainty has not been accounted for when stating conformity to the referenced standard.

BV Labs liability is limited to the actual cost of the requested analyses, unless otherwise agreed in writing. There is no other warranty expressed or implied. BV Labs has been retained to provide analysis of samples provided by the Client using the testing methodology referenced in this report. Interpretation and use of test results are the sole responsibility of the Client and are not within the scope of services provided by BV Labs, unless otherwise agreed in writing. BV Labs is not responsible for the accuracy or any data impacts, that result from the information provided by the customer or their agent.

Solid sample results, except biota, are based on dry weight unless otherwise indicated. Organic analyses are not recovery corrected except for isotope dilution methods.

Results relate to samples tested. When sampling is not conducted by BV Labs, results relate to the supplied samples tested.

This Certificate shall not be reproduced except in full, without the written approval of the laboratory.

Reference Method suffix "m" indicates test methods incorporate validated modifications from specific reference methods to improve performance.



Your Project #: ADM-00262199-A0  
Site Location: TROUT CREEK CULVERT, HWY 599  
Your C.O.C. #: n/a

**Attention: Ahileas Mitsopoulos**

exp Services Inc  
Thunder Bay Branch  
1142 Roland St  
Thunder Bay, ON  
CANADA P7B 5M4

**Report Date: 2020/11/24**  
Report #: R6423596  
Version: 1 - Final

**CERTIFICATE OF ANALYSIS**

**BV LABS JOB #: C0U9063**  
**Received: 2020/11/19, 13:50**

Encryption Key

Please direct all questions regarding this Certificate of Analysis to your Project Manager.  
Julie Clement, Technical Account Manager  
Email: Julie.CLEMENT@bvlabs.com  
Phone# (613)868-6079

=====

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BUREAU  
VERITAS

BV Labs Job #: COU9063  
Report Date: 2020/11/24

exp Services Inc  
Client Project #: ADM-00262199-A0  
Site Location: TROUT CREEK CULVERT, HWY 599  
Sampler Initials: EF

### RESULTS OF ANALYSES OF SOIL

BV Labs ID		OFG477	OFG478	OFG479			OFG479	
Sampling Date		2020/11/03 12:00	2020/11/05 10:00	2020/11/02 15:00			2020/11/02 15:00	
COC Number		n/a	n/a	n/a			n/a	
	<b>UNITS</b>	<b>BH20-01 S7</b>	<b>HP20-05 S4</b>	<b>HP20-11 S2</b>	<b>RDL</b>	<b>QC Batch</b>	<b>HP20-11 S2 Lab-Dup</b>	<b>QC Batch</b>
<b>Calculated Parameters</b>								
Resistivity	ohm-cm	7900	34000	13000		7067730		
<b>Inorganics</b>								
Soluble (20:1) Chloride (Cl-)	ug/g	23	<20	30	20	7069474		
Conductivity	mS/cm	0.13	0.029	0.080	0.002	7072990		
Available (CaCl2) pH	pH	7.66	4.76	6.13		7072893	6.03	7072893
Soluble (20:1) Sulphate (SO4)	ug/g	<20	<20	<20	20	7069471		
RDL = Reportable Detection Limit QC Batch = Quality Control Batch Lab-Dup = Laboratory Initiated Duplicate								



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BV Labs Job #: COU9063  
Report Date: 2020/11/24

exp Services Inc  
Client Project #: ADM-00262199-A0  
Site Location: TROUT CREEK CULVERT, HWY 599  
Sampler Initials: EF

## TEST SUMMARY

**BV Labs ID:** OFG477  
**Sample ID:** BH20-01 S7  
**Matrix:** Soil

**Collected:** 2020/11/03  
**Shipped:**  
**Received:** 2020/11/19

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	7069474	2020/11/21	2020/11/23	Deonarine Ramnarine
Conductivity	AT	7072990	2020/11/24	2020/11/24	Neil Dassanayake
pH CaCl2 EXTRACT	AT	7072893	2020/11/24	2020/11/24	Neil Dassanayake
Resistivity of Soil		7067730	2020/11/24	2020/11/24	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	7069471	2020/11/21	2020/11/23	Deonarine Ramnarine

**BV Labs ID:** OFG478  
**Sample ID:** HP20-05 S4  
**Matrix:** Soil

**Collected:** 2020/11/05  
**Shipped:**  
**Received:** 2020/11/19

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	7069474	2020/11/21	2020/11/23	Deonarine Ramnarine
Conductivity	AT	7072990	2020/11/24	2020/11/24	Neil Dassanayake
pH CaCl2 EXTRACT	AT	7072893	2020/11/24	2020/11/24	Neil Dassanayake
Resistivity of Soil		7067730	2020/11/24	2020/11/24	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	7069471	2020/11/21	2020/11/23	Deonarine Ramnarine

**BV Labs ID:** OFG479  
**Sample ID:** HP20-11 S2  
**Matrix:** Soil

**Collected:** 2020/11/02  
**Shipped:**  
**Received:** 2020/11/19

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	7069474	2020/11/21	2020/11/23	Deonarine Ramnarine
Conductivity	AT	7072990	2020/11/24	2020/11/24	Neil Dassanayake
pH CaCl2 EXTRACT	AT	7072893	2020/11/24	2020/11/24	Neil Dassanayake
Resistivity of Soil		7067730	2020/11/24	2020/11/24	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	7069471	2020/11/21	2020/11/23	Deonarine Ramnarine

**BV Labs ID:** OFG479 Dup  
**Sample ID:** HP20-11 S2  
**Matrix:** Soil

**Collected:** 2020/11/02  
**Shipped:**  
**Received:** 2020/11/19

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
pH CaCl2 EXTRACT	AT	7072893	2020/11/24	2020/11/24	Neil Dassanayake



### GENERAL COMMENTS

Each temperature is the average of up to three cooler temperatures taken at receipt

Package 1	3.7°C
-----------	-------

Results relate only to the items tested.



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VERITAS

BV Labs Job #: C0U9063

Report Date: 2020/11/24

## QUALITY ASSURANCE REPORT

exp Services Inc

Client Project #: ADM-00262199-A0

Site Location: TROUT CREEK CULVERT, HWY 599

Sampler Initials: EF

QC Batch	Parameter	Date	Matrix Spike		SPIKED BLANK		Method Blank		RPD	
			% Recovery	QC Limits	% Recovery	QC Limits	Value	UNITS	Value (%)	QC Limits
7069471	Soluble (20:1) Sulphate (SO <sub>4</sub> )	2020/11/23	NC	70 - 130	104	70 - 130	<20	ug/g	NC	35
7069474	Soluble (20:1) Chloride (Cl <sup>-</sup> )	2020/11/23	NC	70 - 130	107	70 - 130	<20	ug/g	3.0	35
7072893	Available (CaCl <sub>2</sub> ) pH	2020/11/24			100	97 - 103			1.7	N/A
7072990	Conductivity	2020/11/24			103	90 - 110	<0.002	mS/cm	1.3	10

N/A = Not Applicable

Duplicate: Paired analysis of a separate portion of the same sample. Used to evaluate the variance in the measurement.

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

Spiked Blank: A blank matrix sample to which a known amount of the analyte, usually from a second source, has been added. Used to evaluate method accuracy.

Method Blank: A blank matrix containing all reagents used in the analytical procedure. Used to identify laboratory contamination.

NC (Matrix Spike): The recovery in the matrix spike was not calculated. The relative difference between the concentration in the parent sample and the spike amount was too small to permit a reliable recovery calculation (matrix spike concentration was less than the native sample concentration)

NC (Duplicate RPD): The duplicate RPD was not calculated. The concentration in the sample and/or duplicate was too low to permit a reliable RPD calculation (absolute difference ≤ 2x RDL).



BUREAU  
VERITAS

BV Labs Job #: COU9063

Report Date: 2020/11/24

exp Services Inc

Client Project #: ADM-00262199-A0

Site Location: TROUT CREEK CULVERT, HWY 599

Sampler Initials: EF

### VALIDATION SIGNATURE PAGE

The analytical data and all QC contained in this report were reviewed and validated by the following individual(s).

Anastassia Hamanov, Scientific Specialist

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BV Labs has procedures in place to guard against improper use of the electronic signature and have the required "signatories", as per ISO/IEC 17025, signing the reports. For Service Group specific validation please refer to the Validation Signature Page.





6740 Campbell Road, Mississauga, Ontario L5N 2L8  
Phone: 905-817-5700 Fax: 905-817-5779 Toll Free: 800-563-6266  
CAM FCD-01191/6

# CHAIN OF CUSTODY RECORD

Page 1 of 1

Invoice Information		Report Information (if differs from invoice)		Project Information (where applicable)		Turnaround Time (TAT) Required	
Company Name:	EXP Services Inc.	Company Name:		Quotation #:	B45998	<input checked="" type="checkbox"/> Regular TAT (5-7 days) Most analyses	
Contact Name:	Ahileas Mitsopoulos	Contact Name:		P.O. #/ AFE#:		PLEASE PROVIDE ADVANCE NOTICE FOR RUSH PROJECTS	
Address:	1142 Rolland St. Thunder Bay, ON	Address:		Project #:	ADM-00262199-A0	Rush TAT (Surcharges will be applied)	
Phone:	807-623-9495	Phone:		Site Location:	Trout Creek Culvert, Hwy 599	<input type="checkbox"/> 1 Day	<input type="checkbox"/> 2 Days <input type="checkbox"/> 3-4 Days
Fax:		Fax:		Site #:	MT01	Date Required:	
Email:	ahileas.mitsopoulos@exp.com	Email:		Site Location Province:	Ontario	Rush Confirmation #:	
MOE REGULATED DRINKING WATER OR WATER INTENDED FOR HUMAN CONSUMPTION MUST BE SUBMITTED ON THE BUREAU VERITAS LABORATORIES' DRINKING WATER CHAIN OF CUSTODY				Sampled By:	EF		
Regulation 153		Other Regulations		Analysis Requested		LABORATORY USE ONLY	
<input type="checkbox"/> Table 1 <input type="checkbox"/> Table 2 <input type="checkbox"/> Table 3 <input type="checkbox"/> Table	<input type="checkbox"/> Res/Park <input type="checkbox"/> Ind/Comm <input type="checkbox"/> Agri/ Other	<input type="checkbox"/> CCME <input type="checkbox"/> MISA <input type="checkbox"/> PWQO <input type="checkbox"/> Other (Specify) <input type="checkbox"/> REG 558 (MIN. 3 DAY TAT REQUIRED) <input type="checkbox"/> REG 406 Table	<input type="checkbox"/> Sanitary Sewer Bylaw <input type="checkbox"/> Storm Sewer Bylaw Region	<input type="checkbox"/> FIELD FILTERED (CIRCLE) Metals / Pb / Cu / Ni <input type="checkbox"/> BTEX / PHC F1 <input type="checkbox"/> PHCs F2 - F4 <input type="checkbox"/> VOCs <input type="checkbox"/> REG 153 METALS & INORGANICS <input type="checkbox"/> REG 153 IC/PWS METALS <input type="checkbox"/> REG 153 METALS (Pb, Cu, V, IC/PWS Metals, HWS - B) <input type="checkbox"/> pH <input type="checkbox"/> Conductivity <input type="checkbox"/> Reactivity <input type="checkbox"/> Water Soluble Sulphate <input type="checkbox"/> Chloride	<input type="checkbox"/> HOLD - DO NOT ANALYZE	CUSTODY SEAL Y / N Present Intact N N/A 5/3/3 4.4.4 COOLING MEDIA PRESENT: <input checked="" type="checkbox"/> Y / N COMMENTS	
Include Criteria on Certificate of Analysis: Y / N		SAMPLES MUST BE KEPT COOL (< 10 °C) FROM TIME OF SAMPLING UNTIL DELIVERY TO BUREAU VERITAS					
SAMPLE IDENTIFICATION	DATE SAMPLED (YYYY/MM/DD)	TIME SAMPLED (HH:MM)	MATRIX	# OF CONTAINERS SUBMITTED			
1 BH20-01 S7	2020/11/03	12:00 PM	Soil	1			
2 HP20-05 S4	2020/11/05	10:00 AM	Soil	1			
3 HP20-11 S2	2020/11/02	3:00 PM	Soil	1			
4							
5							
6							
7							
8							
9							
10							
RELINQUISHED BY: (Signature/Print)		DATE: (YYYY/MM/DD)	TIME: (HH:MM)	RECEIVED BY: (Signature/Print)		DATE: (YYYY/MM/DD)	TIME: (HH:MM)
		2020/11/19	1:30 PM	James Klapperich		2020/11/19	13:50
				GUYAN Idriz		2020/11/20	09:17

Rec'd In Thunder Bay

19-Nov-20 13:50  
Julie Clement  
COU9063  
GID ENV-021

## Appendix G – Slope Stability Analyses

6019-E-0004 - Assignment No.8  
 Trout Creek Culvert Replacement  
 North Embankment Slope (Inlet)  
 Drained Static Condition

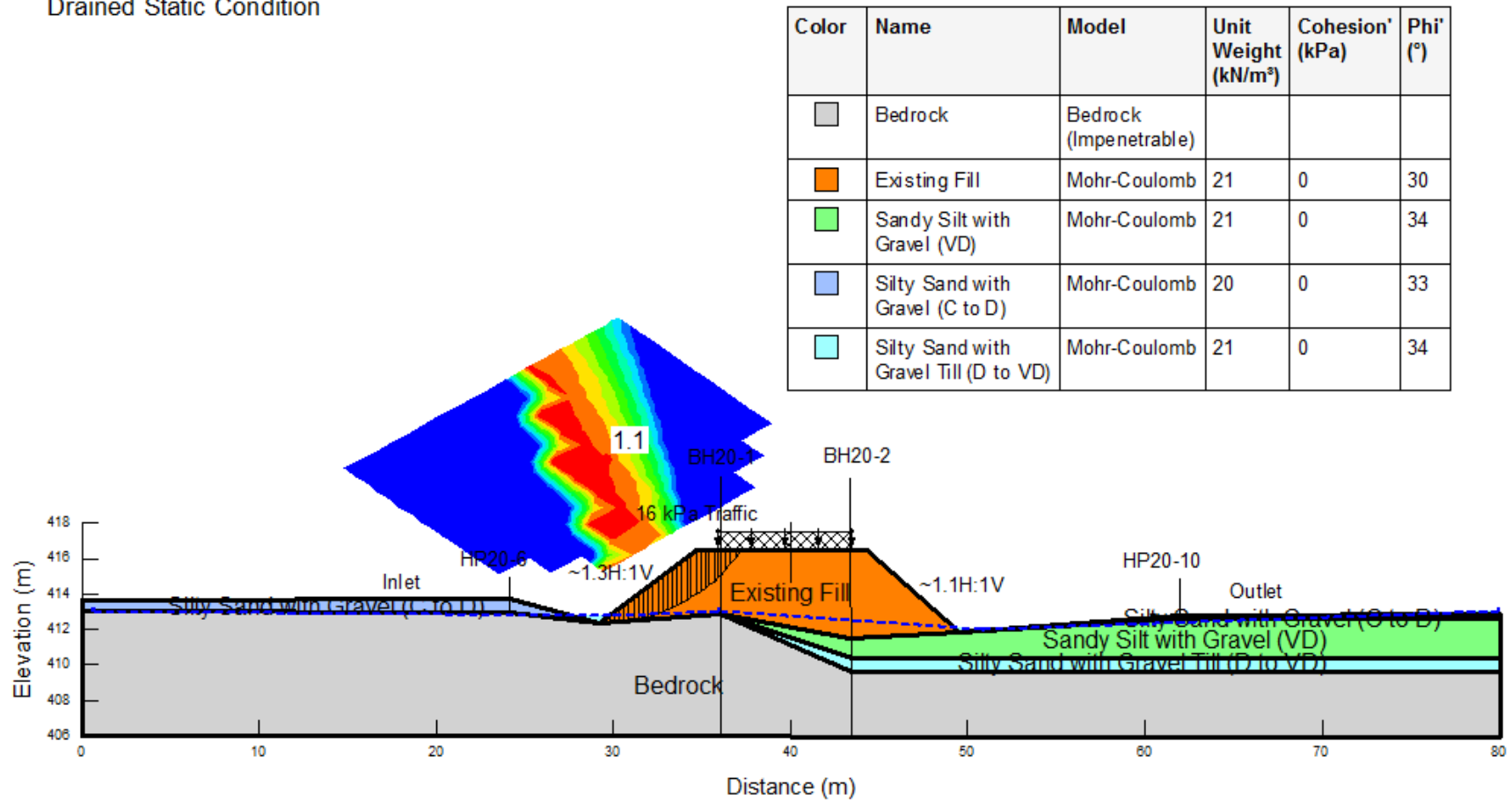







Figure G1: Slope stability analysis for existing north side of embankment (1.3H:1V) – drained static condition

6019-E-0004 - Assignment No.8  
 Trout Creek Culvert Replacement  
 South Embankment Slope (Outlet)  
 Drained Static Condition

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
	Bedrock	Bedrock (Impenetrable)			
	Existing Fill	Mohr-Coulomb	21	0	30
	Sandy Silt with Gravel (VD)	Mohr-Coulomb	21	0	34
	Silty Sand with Gravel (C to D)	Mohr-Coulomb	20	0	33
	Silty Sand with Gravel Till (D to VD)	Mohr-Coulomb	21	0	34

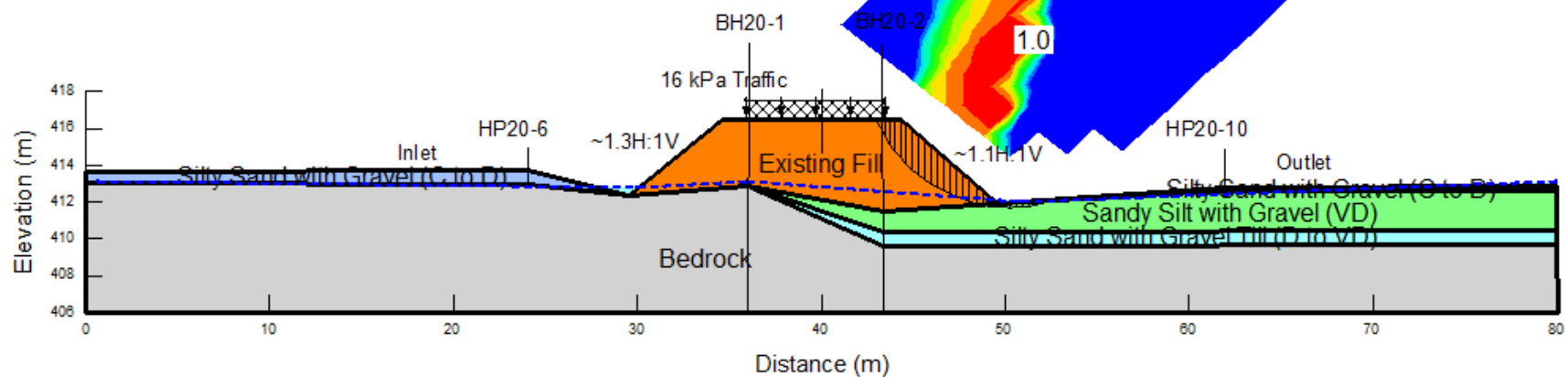


Figure G2: Slope stability analysis for existing south side of embankment (1.1H:1V) – drained static condition

6019-E-0004 - Assignment No.8  
 Trout Creek Culvert Replacement  
 North Embankment Slope (Inlet) - Widening  
 Drained Static Condition

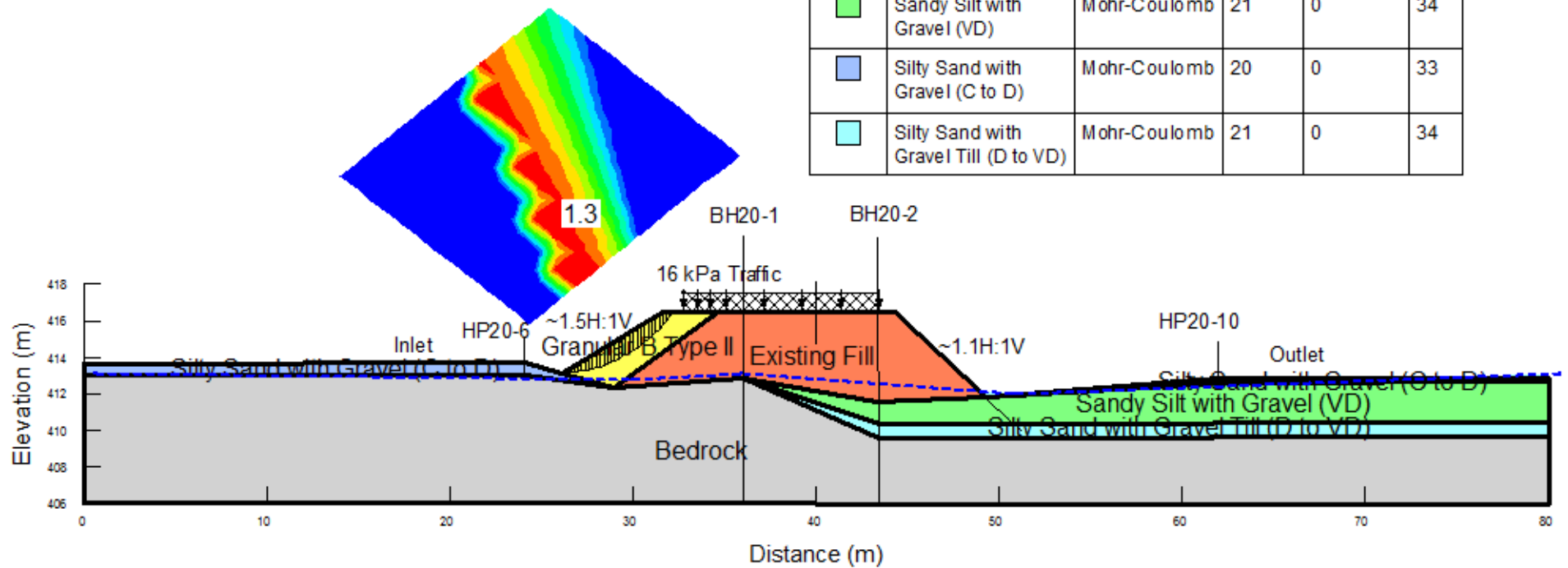








Figure G3: Slope stability analysis for temporary widening north side of embankment (1.5H:1V) – drained static condition

6019-E-0004 - Assignment No.8  
Trout Creek Culvert Replacement  
North Embankment Slope (Inlet) - Permanent Widening  
Drained Static Condition

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
	Bedrock	Bedrock (Impenetrable)			
	Existing Fill	Mohr-Coulomb	21	0	30
	Granular B Type II	Mohr-Coulomb	22	0	35
	Sandy Silt with Gravel (VD)	Mohr-Coulomb	21	0	34
	Silty Sand with Gravel (C to D)	Mohr-Coulomb	20	0	33
	Silty Sand with Gravel Till (D to VD)	Mohr-Coulomb	21	0	34

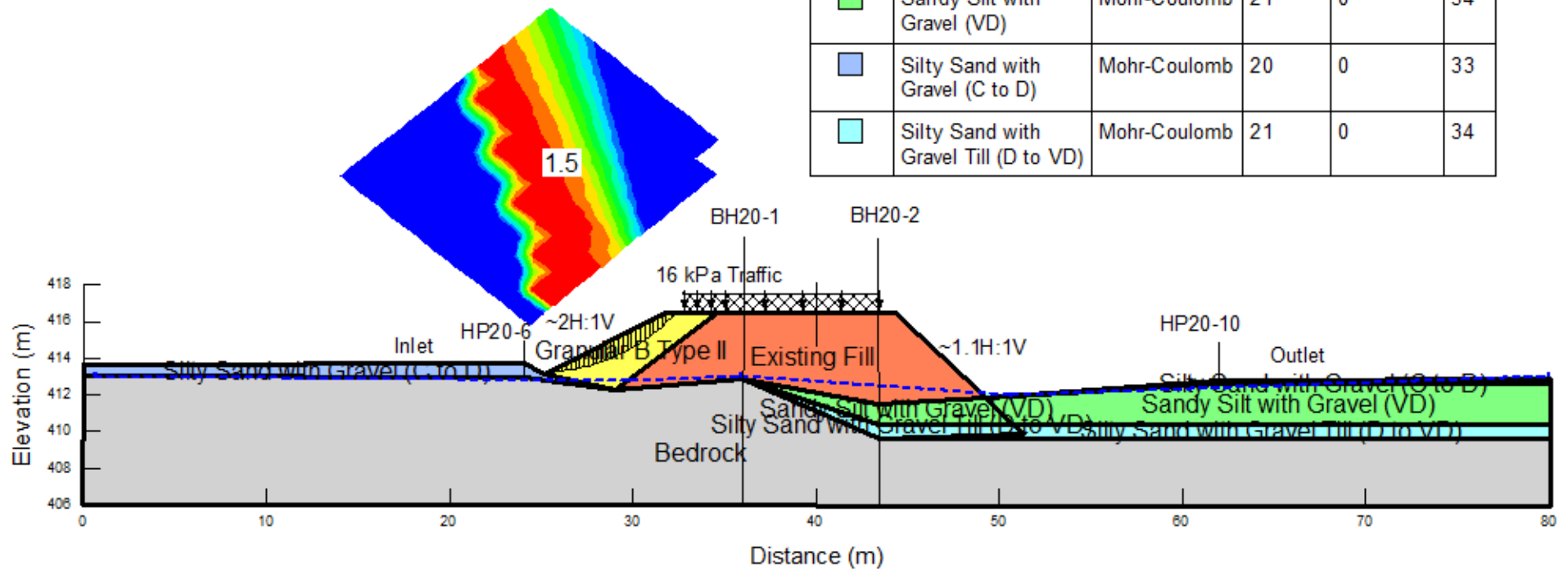








Figure G4: Slope stability analysis for permanent widening north side of embankment (2H:1V) – drained static condition

6019-E-0004 - Assignment No.1  
 Trout Creek Culvert Replacement  
 South Embankment Slope (Outlet) - Widening  
 Drained Static Condition

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
	Bedrock	Bedrock (Impenetrable)			
	Existing Fill	Mohr-Coulomb	21	0	30
	Granular B Type I	Mohr-Coulomb	22	0	35
	Sandy Silt with Gravel (VD)	Mohr-Coulomb	21	0	34
	Silty Sand with Gravel (C to D)	Mohr-Coulomb	20	0	33
	Silty Sand with Gravel Till (D to VD)	Mohr-Coulomb	21	0	34

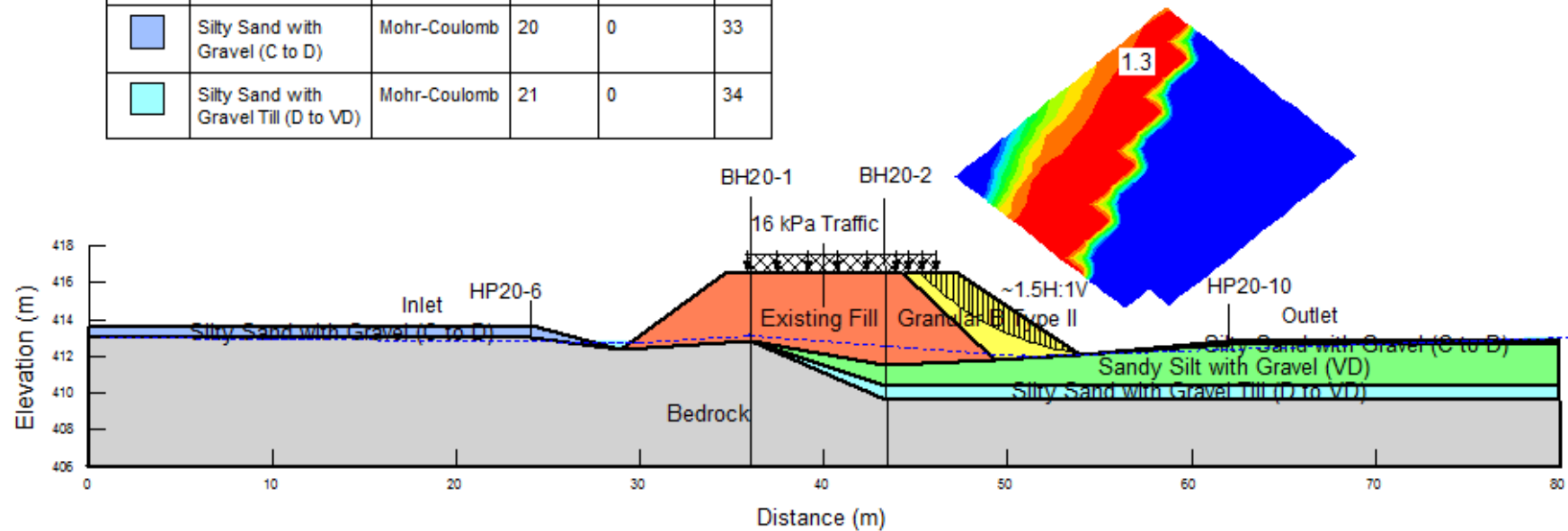








Figure G5: Slope stability analysis for temporary widening south side of embankment (1.5H:1V) – drained static condition



6019-E-0004 - Assignment No.1  
 Trout Creek Culvert Replacement  
 South Embankment Slope (Outlet) - Permanent Widening  
 Drained Static Condition

Color	Name	Model	Unit Weight (kN/m <sup>3</sup> )	Cohesion' (kPa)	Phi' (°)
	Bedrock	Bedrock (Impenetrable)			
	Existing Fill	Mohr-Coulomb	21	0	30
	Granular B Type II	Mohr-Coulomb	22	0	35
	Sandy Silt with Gravel (VD)	Mohr-Coulomb	21	0	34
	Silty Sand with Gravel (C to D)	Mohr-Coulomb	20	0	33
	Silty Sand with Gravel Till (D to VD)	Mohr-Coulomb	21	0	34

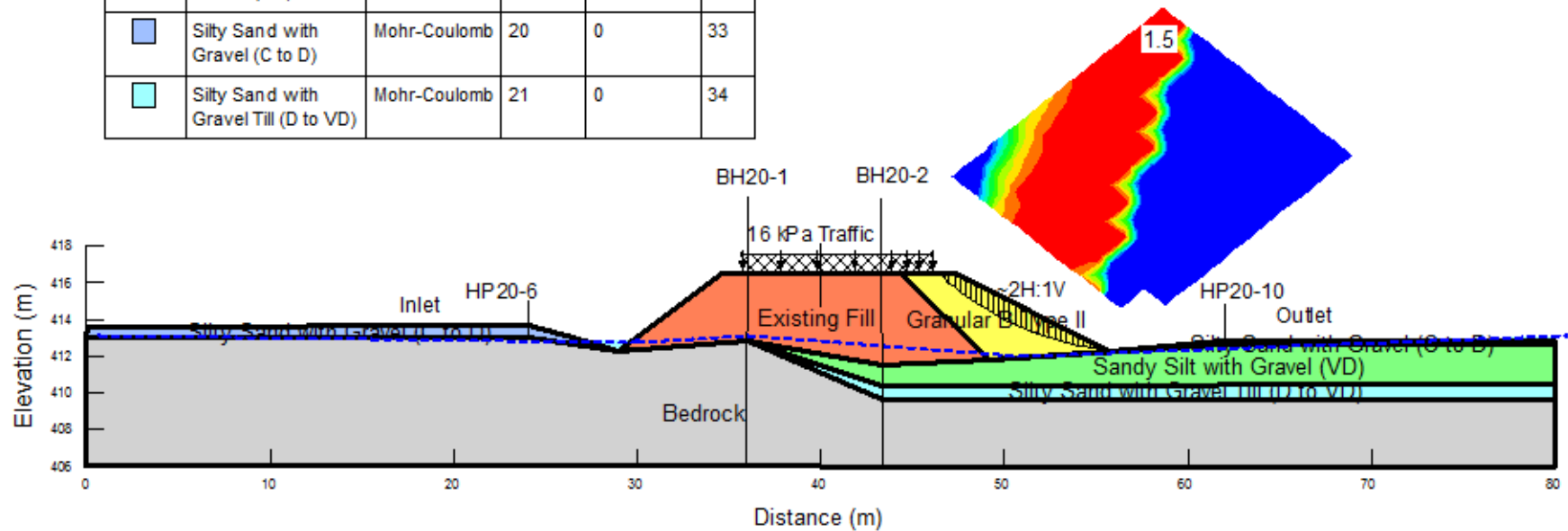








Figure G6: Slope stability analysis for permanent widening south side of embankment (2H:1V) – drained static condition

6019-E-0004 - Assignment No.8  
 Trout Creek Culvert Replacement  
 North Side - New Embankment Slope (Inlet)  
 Drained Static Condition

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
	Bedrock	Bedrock (Impenetrable)			
	Granular B Type II	Mohr-Coulomb	22	0	35
	Peat with Silt and Sand	Mohr-Coulomb	12	0	18
	Silt to Clayey Silt	Mohr-Coulomb	20	0	29
	Silty Sand with Gravel	Mohr-Coulomb	20	0	32
	Silty Sand with Gravel (C to D)	Mohr-Coulomb	20	0	33

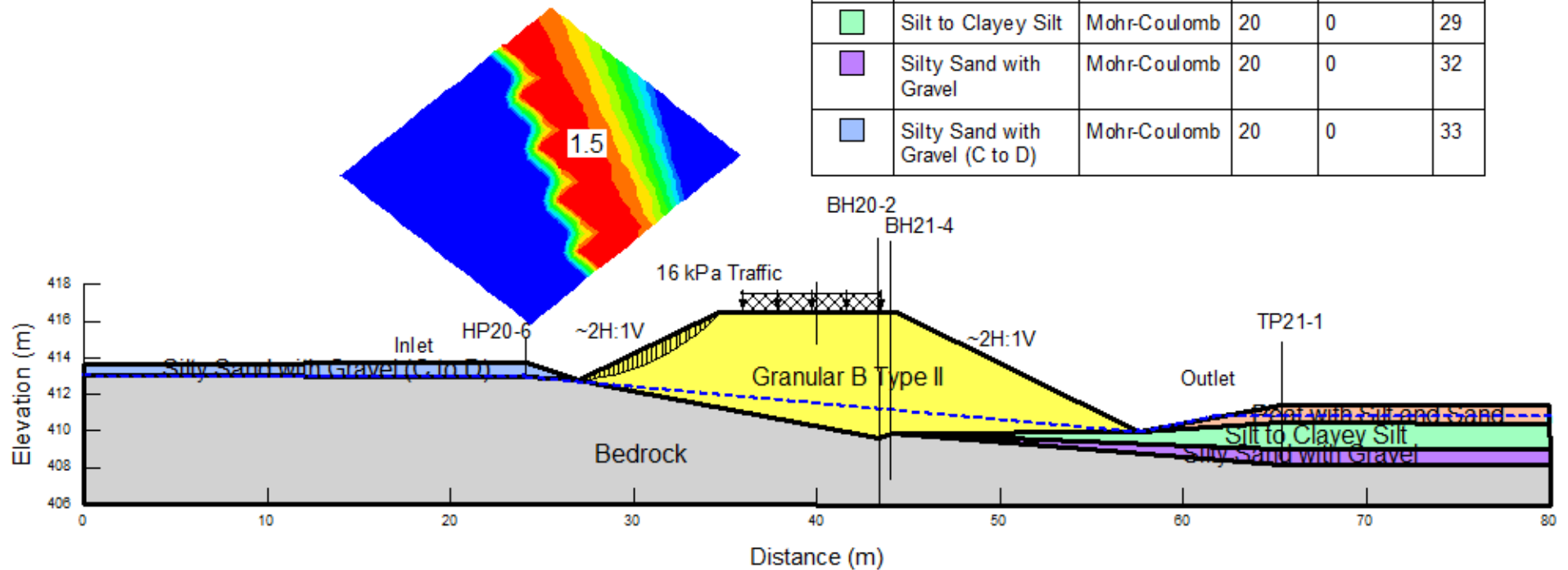








Figure G7: Slope stability analysis for north side of new embankment (side slopes 2H:1V) – drained static condition

6019-E-0004 - Assignment No.8  
 Trout Creek Culvert Replacement  
 North Side - New Embankment Slope (Inlet)  
 Drained Seismic Condition

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
	Bedrock	Bedrock (Impenetrable)			
	Granular B Type II	Mohr-Coulomb	22	0	35
	Peat with Silt and Sand	Mohr-Coulomb	12	0	18
	Silt to Clayey Silt	Mohr-Coulomb	20	0	29
	Silty Sand with Gravel	Mohr-Coulomb	20	0	32
	Silty Sand with Gravel (C to D)	Mohr-Coulomb	20	0	33

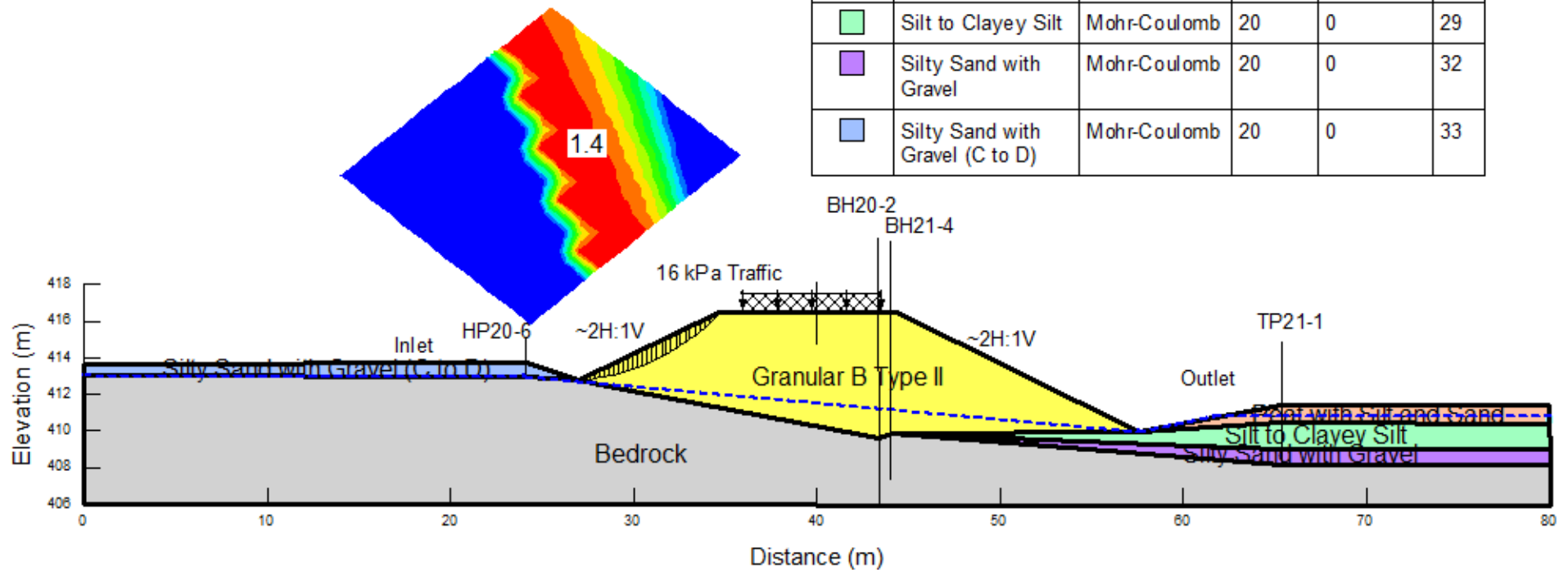








Figure G8: Slope stability analysis for north side of new embankment (side slopes 2H:1V) – drained seismic condition

6019-E-0004 - Assignment No.8  
 Trout Creek Culvert Replacement  
 South Side - New Embankment Slope (Outlet)  
 Drained Static Condition

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
	Bedrock	Bedrock (Impenetrable)			
	Granular B Type II	Mohr-Coulomb	22	0	35
	Peat with Silt and Sand	Mohr-Coulomb	12	0	18
	Silt to Clayey Silt	Mohr-Coulomb	20	0	29
	Silty Sand with Gravel	Mohr-Coulomb	20	0	32
	Silty Sand with Gravel (C to D)	Mohr-Coulomb	20	0	33

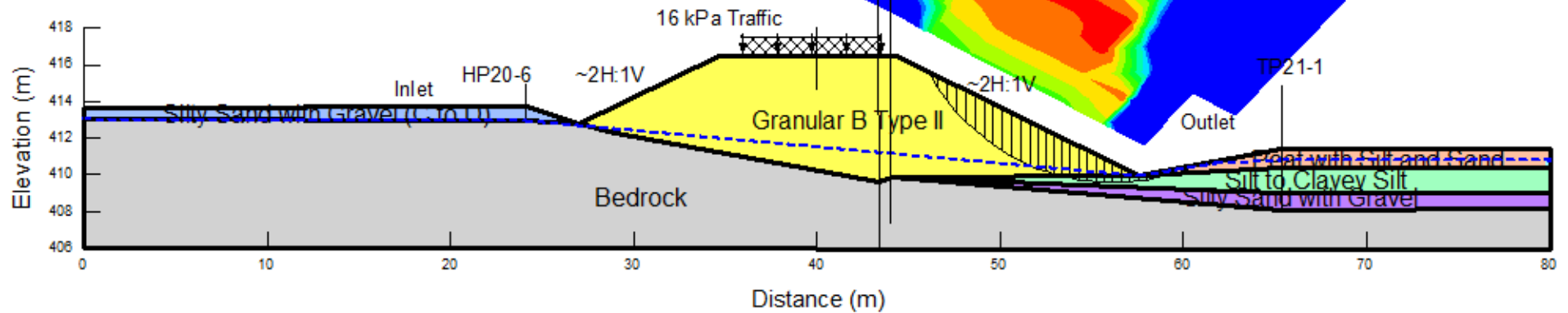








Figure G9: Slope stability analysis for south side of new embankment (side slopes 2H:1V) – drained static condition

6019-E-0004 - Assignment No.8  
 Trout Creek Culvert Replacement  
 South Side - New Embankment Slope (Outlet)  
 Drained Static Condition

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
	Bedrock	Bedrock (Impenetrable)			
	Granular B Type II	Mohr-Coulomb	22	0	35
	Peat with Silt and Sand	Mohr-Coulomb	12	0	18
	Silt to Clayey Silt	Mohr-Coulomb	20	0	29
	Silty Sand with Gravel	Mohr-Coulomb	20	0	32
	Silty Sand with Gravel (C to D)	Mohr-Coulomb	20	0	33

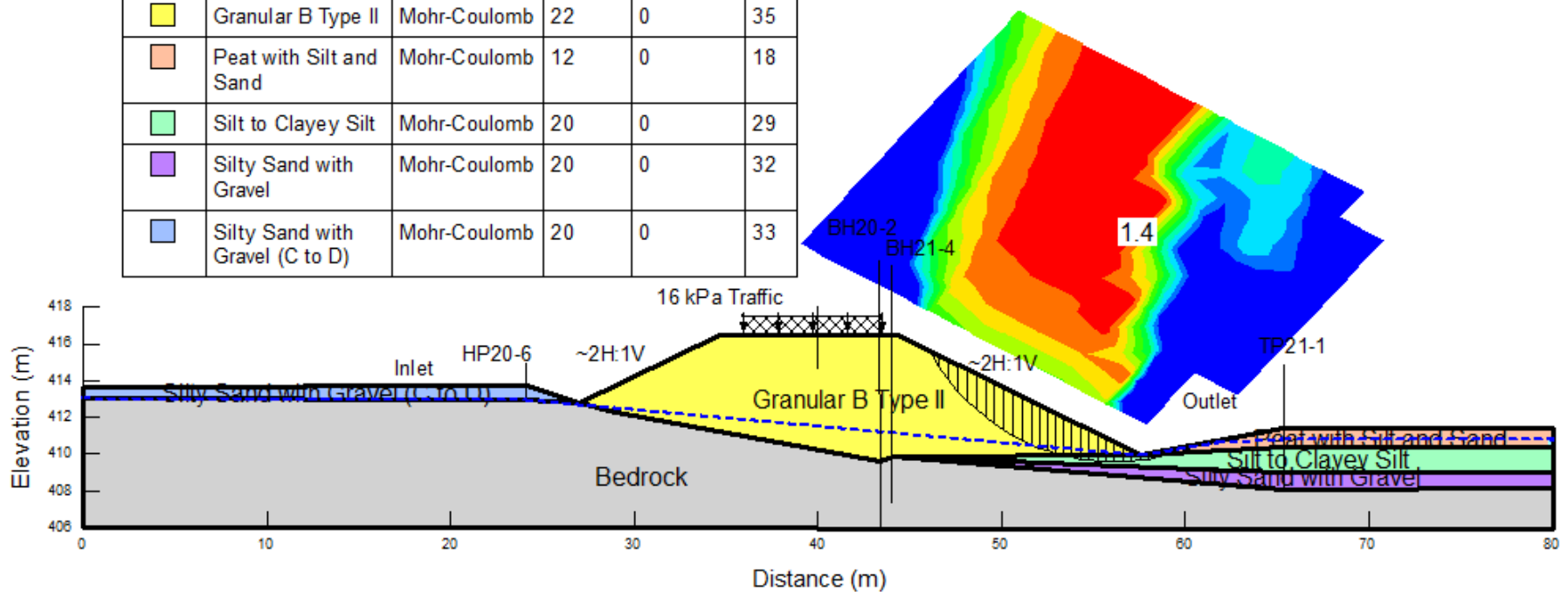








Figure G10: Slope stability analysis for south side of new embankment (side slopes 2H:1V) – drained seismic condition

6019-E-0004 - Assignment No.8  
 Trout Creek Culvert Replacement\*  
 Temporary Modular Bridge  
 Drained Static Condition

Color	Name	Model	Unit Weight (kN/m <sup>3</sup> )	Cohesion' (kPa)	Phi' (°)
	Abutment	Mohr-Coulomb	23	100	45
	Bedrock	Bedrock (Impenetrable)			
	Existing Fill	Mohr-Coulomb	21	0	30
	Granular B Type II	Mohr-Coulomb	22	0	35
	Sandy Silt with Gravel (VD)	Mohr-Coulomb	21	0	34
	Silty Sand with Gravel to Silt Till (D to VD)	Mohr-Coulomb	21	0	34

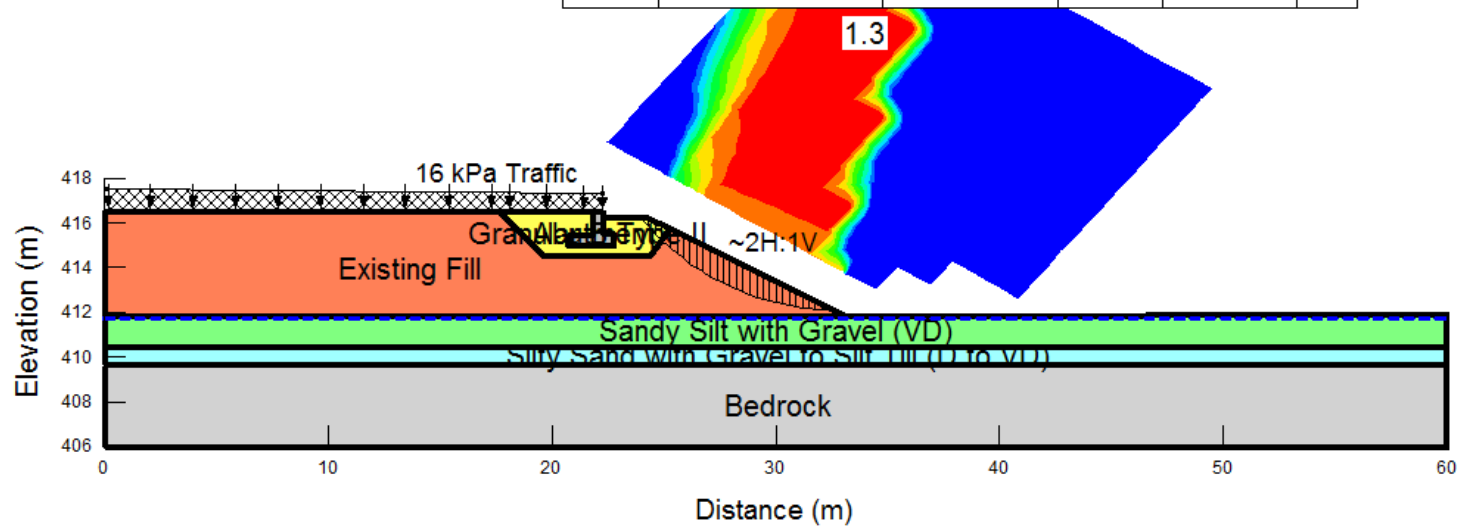







Figure G11: Slope stability analysis for Temporary Modular Bridge (forward slope 2H:1V) – drained static condition

6019-E-0004 - Assignment No.8  
 Trout Creek Culvert Replacement  
 Permanent Modular Bridge  
 Failure Behind Abutment  
 Drained Static Condition

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
	Abutment	Mohr-Coulomb	23	100	45
	Bedrock	Bedrock (Impenetrable)			
	Granular B Type II	Mohr-Coulomb	22	0	35
	Granular pad	Mohr-Coulomb	22	0	35
	Gravelly Sand (D to VD)	Mohr-Coulomb	21	0	34

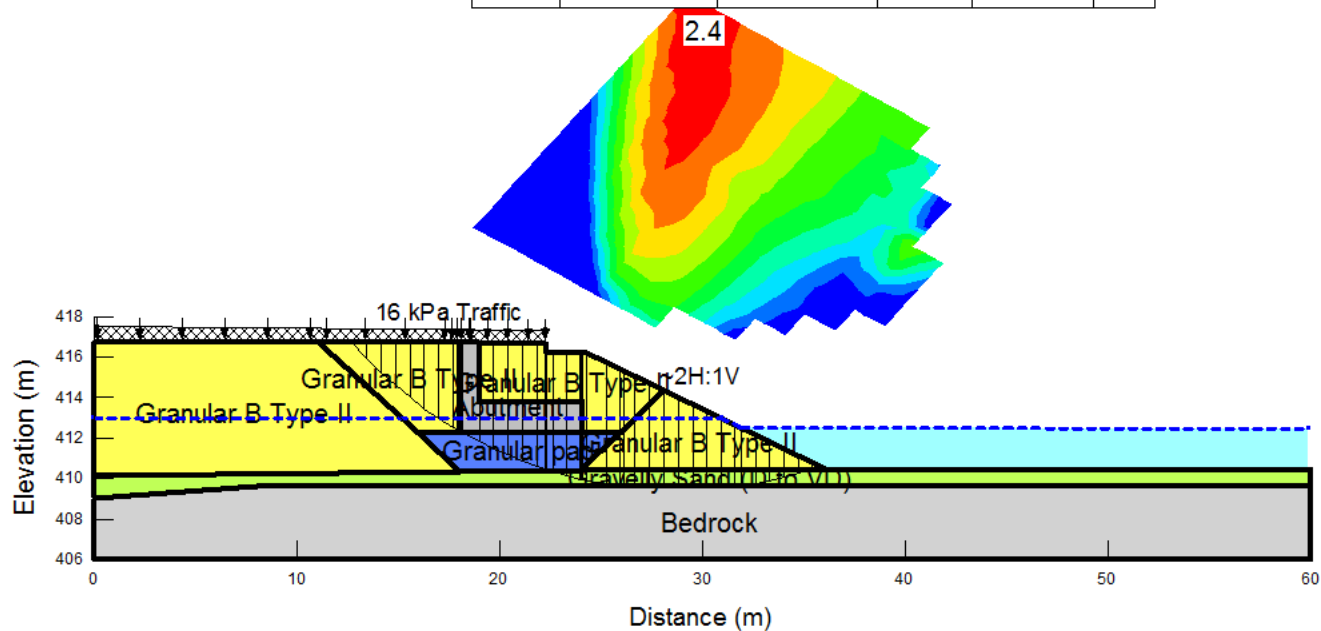







Figure G12: Slope stability analysis for Permanent Modular Bridge (forward slope 2H:1V) with abutment on granular pad (failure behind abutment) – drained static condition



6019-E-0004 - Assignment No.8  
 Trout Creek Culvert Replacement  
 Permanent Modular Bridge  
 Failure Behind Abutment  
 Drained Seismic Condition

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
	Abutment	Mohr-Coulomb	23	100	45
	Bedrock	Bedrock (Impenetrable)			
	Granular B Type II	Mohr-Coulomb	22	0	35
	Granular pad	Mohr-Coulomb	22	0	35
	Gravelly Sand (D to VD)	Mohr-Coulomb	21	0	34

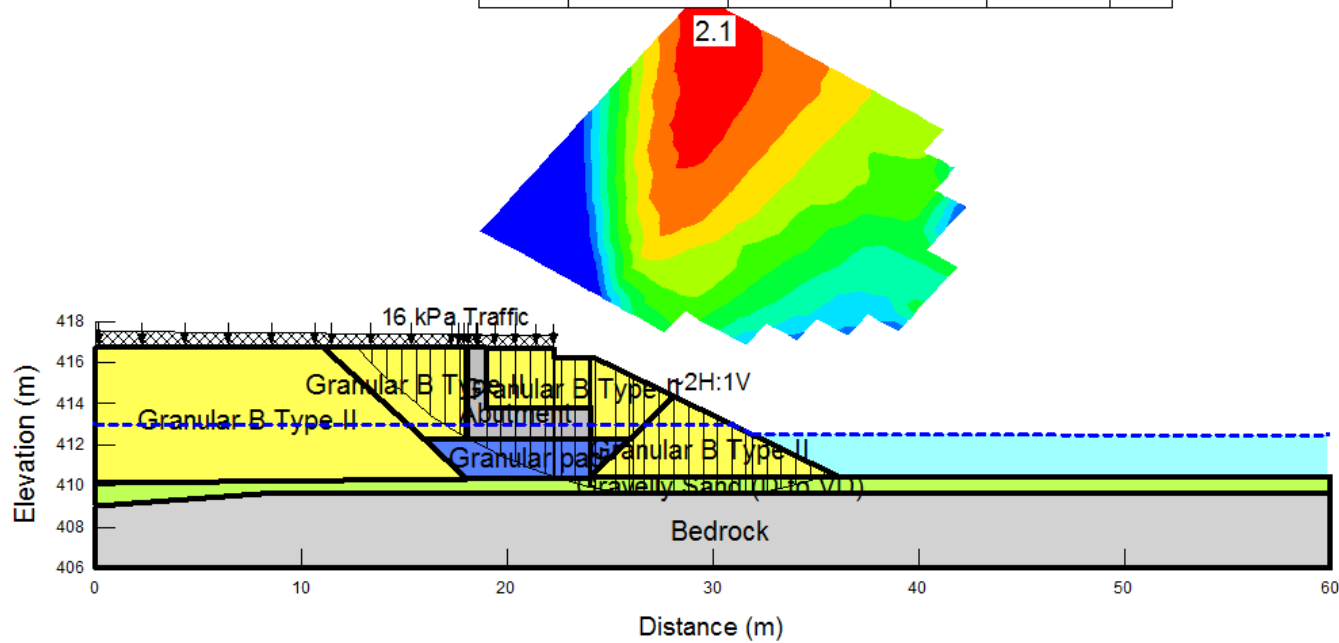







Figure G13: Slope stability analysis for Permanent Modular Bridge (forward slope 2H:1V) with abutment on granular pad (failure behind abutment) – drained seismic condition

6019-E-0004 - Assignment No.8  
 Trout Creek Culvert Replacement  
 Permanent Modular Bridge  
 Failure in Front of Abutment  
 Drained Static Condition

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
	Abutment	Mohr-Coulomb	23	100	45
	Bedrock	Bedrock (Impenetrable)			
	Granular B Type II	Mohr-Coulomb	22	0	35
	Granular pad	Mohr-Coulomb	22	0	35
	Gravelly Sand (D to VD)	Mohr-Coulomb	21	0	34

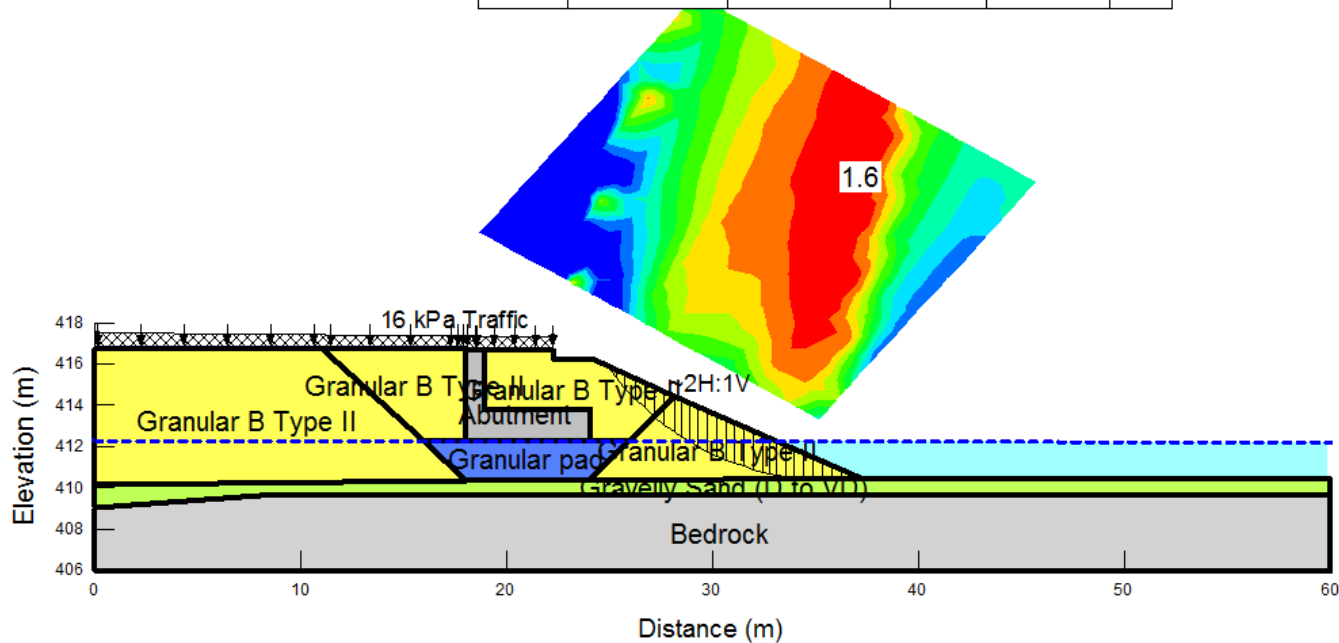







Figure G14: Slope stability analysis for Permanent Modular Bridge (forward slope 2H:1V) with abutment on granular pad (failure in front of abutment) – drained static condition

6019-E-0004 - Assignment No.8  
 Trout Creek Culvert Replacement  
 Permanent Modular Bridge  
 Failure in Front of Abutment  
 Drained Seismic Condition

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
	Abutment	Mohr-Coulomb	23	100	45
	Bedrock	Bedrock (Impenetrable)			
	Granular B Type II	Mohr-Coulomb	22	0	35
	Granular pad	Mohr-Coulomb	22	0	35
	Gravelly Sand (D to VD)	Mohr-Coulomb	21	0	34

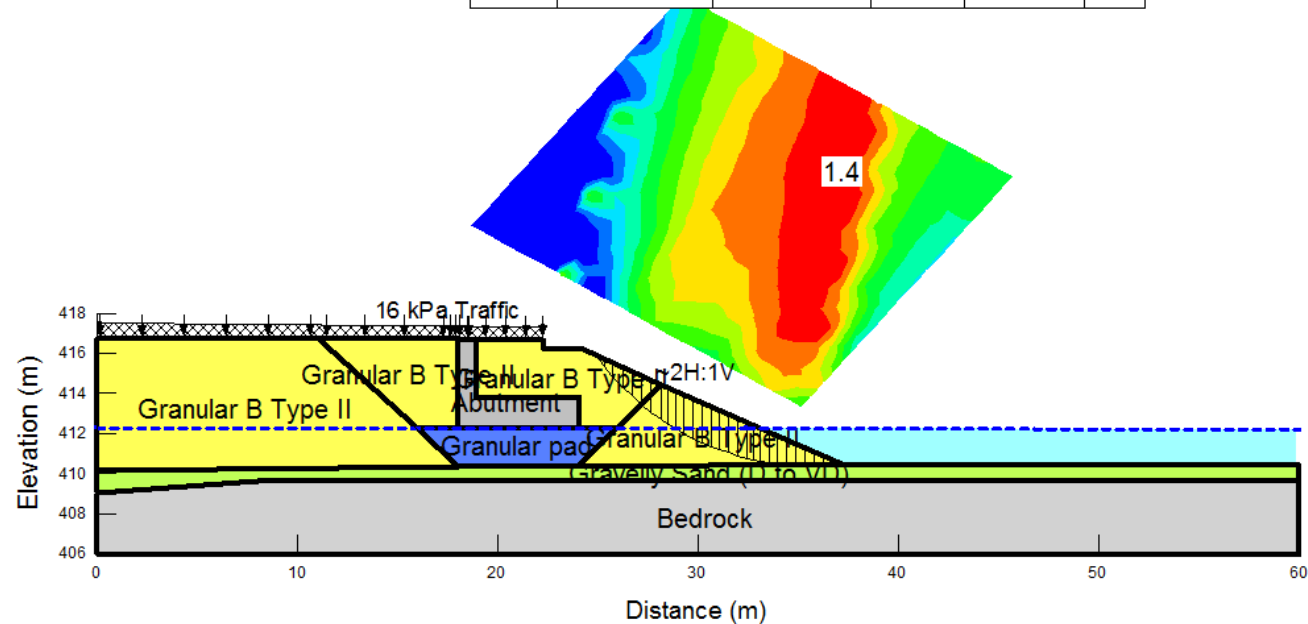


Figure G15: Slope stability analysis for Permanent Modular Bridge (forward slope 2H:1V) with abutment on granular pad (failure in front of abutment) – drained seismic condition

6019-E-0004 - Assignment No.8  
 Trout Creek Culvert Replacement  
 Permanent Modular Bridge  
 Side slopes at Inlet  
 Drained Static Condition

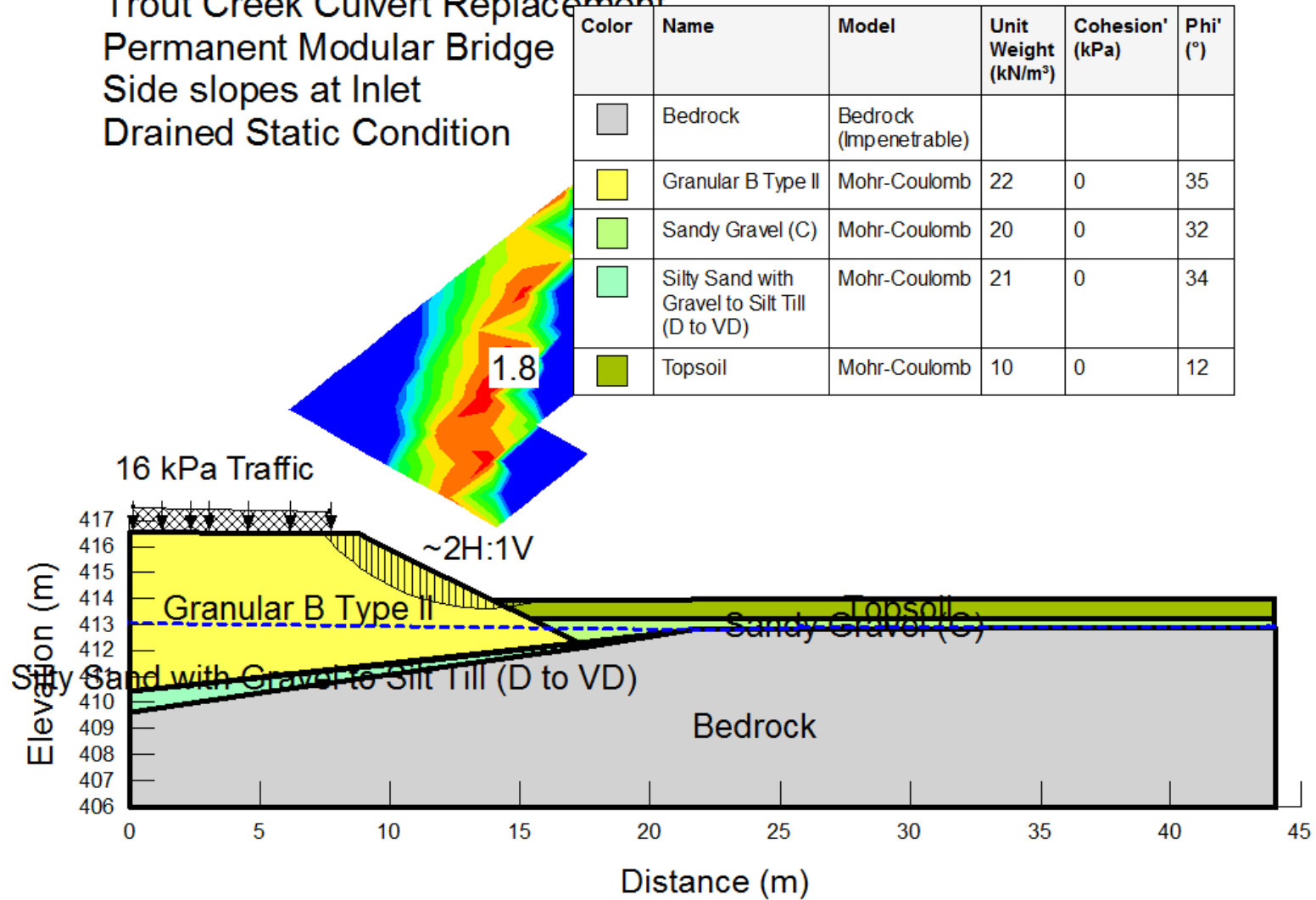


Figure G16: Slope stability analysis for Permanent Modular Bridge (side slope 2H:1V) at Inlet – drained static condition

6019-E-0004 - Assignment No.8  
 Trout Creek Culvert Replacement  
 Permanent Modular Bridge  
 Side slopes at Inlet  
 Drained Seismic Condition

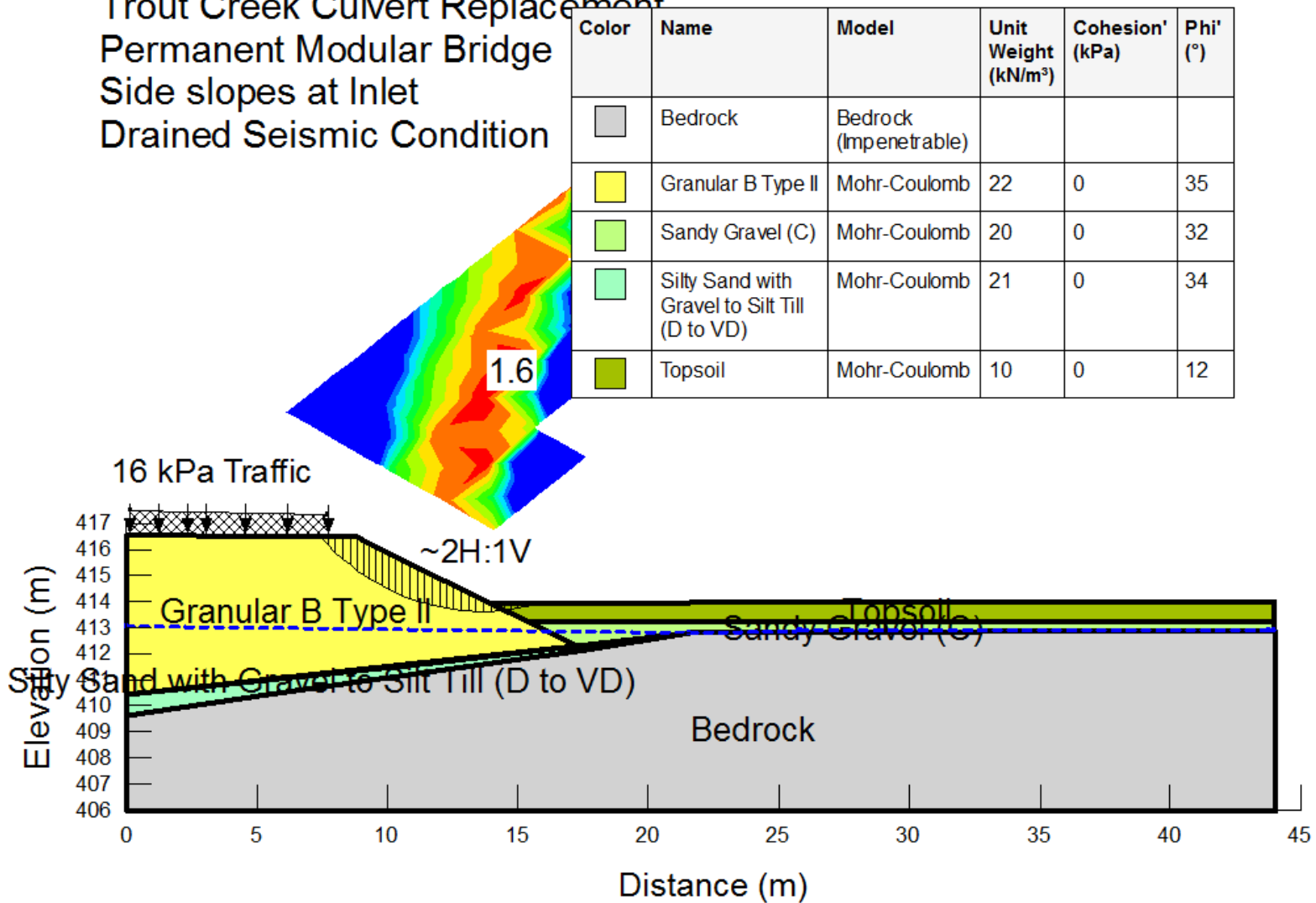


Figure G17: Slope stability analysis for Permanent Modular Bridge (side slope 2H:1V) at Inlet – drained seismic condition

6019-E-0004 - Assignment No.8  
 Trout Creek Culvert Replacement  
 Permanent Modular Bridge  
 Side slopes at Outlet  
 Drained Static Condition

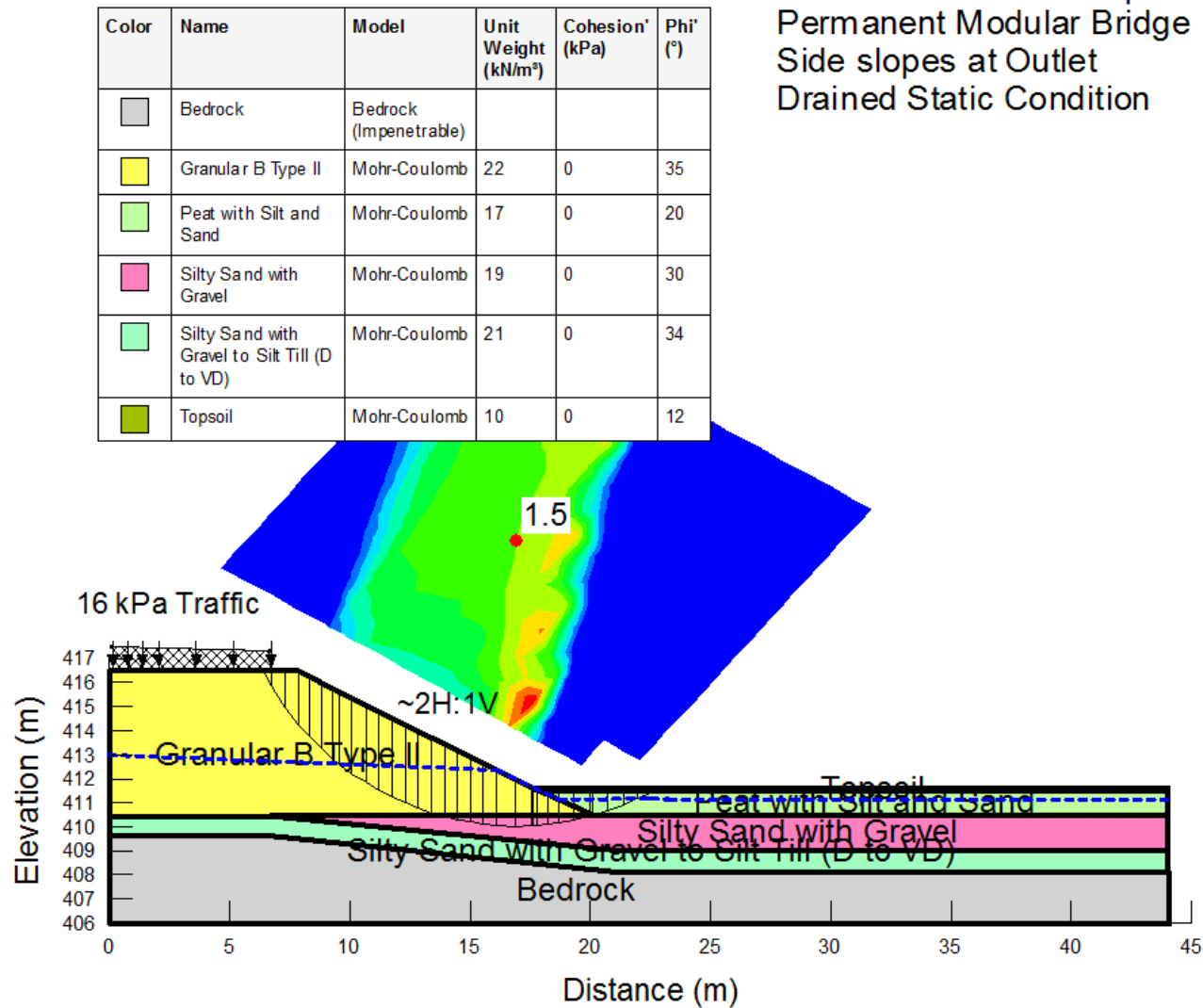


Figure G18: Slope stability analysis for Permanent Modular Bridge (side slope 2H:1V) at Outlet – drained static condition

6019-E-0004 - Assignment No.8  
 Trout Creek Culvert Replacement  
 Permanent Modular Bridge  
 Side slopes at Outlet  
 Drained Seismic Condition

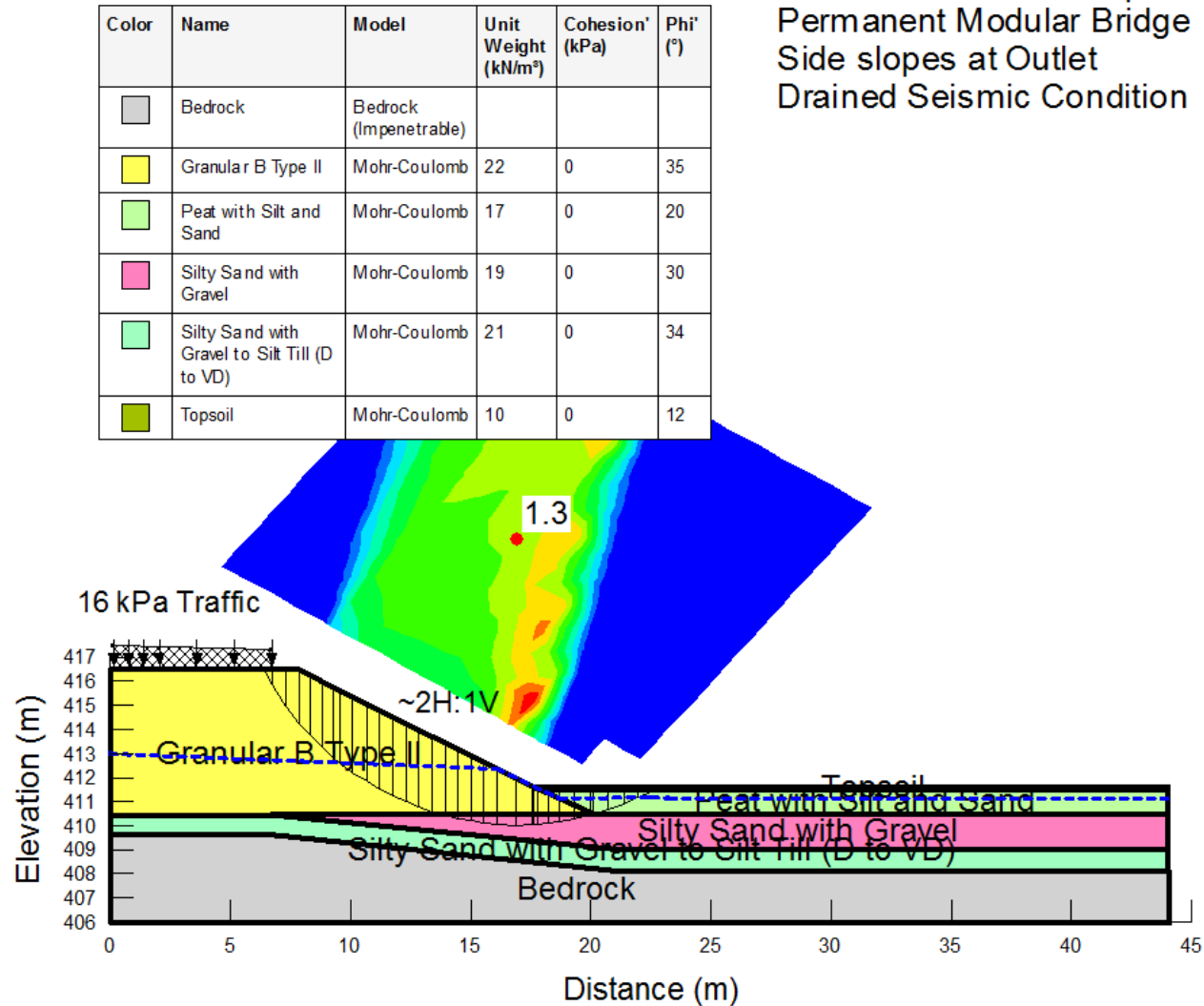
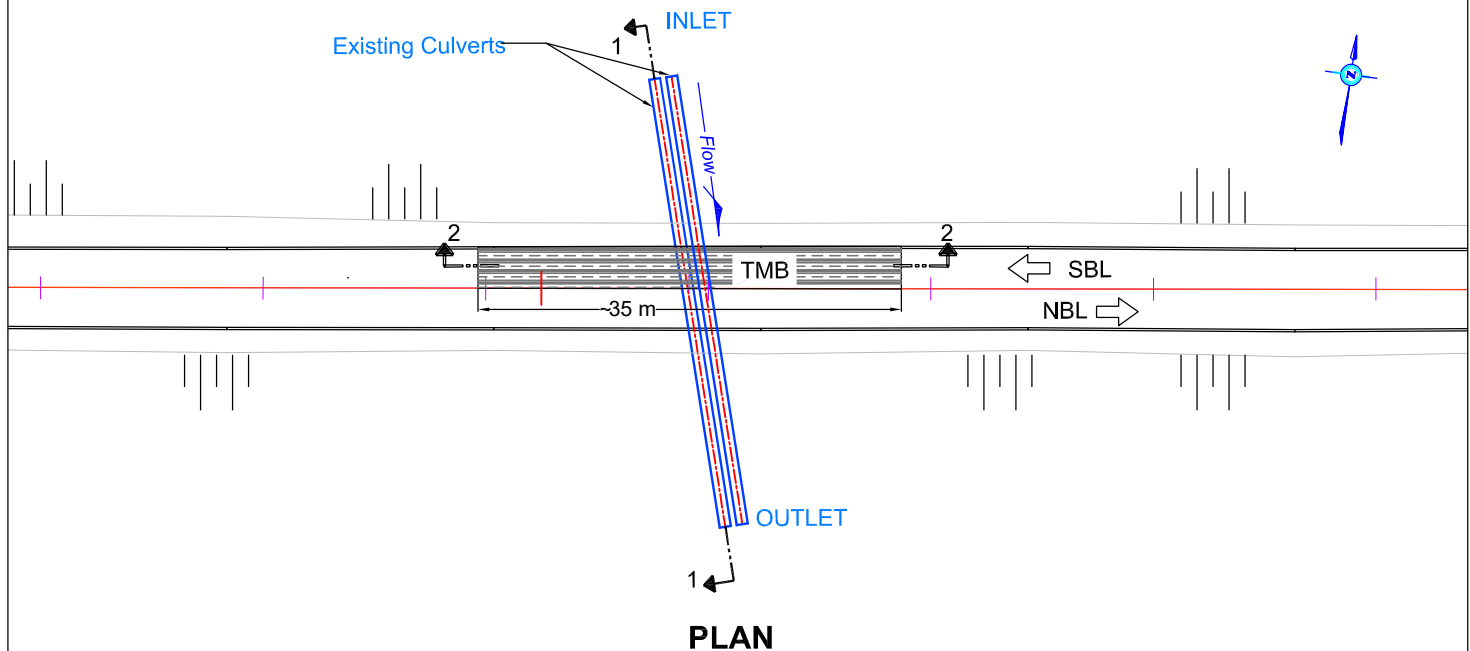


Figure G19: Slope stability analysis for Permanent Modular Bridge (side slope 2H:1V) at Outlet – drained seismic condition



## Appendix H – Schematic Sketches

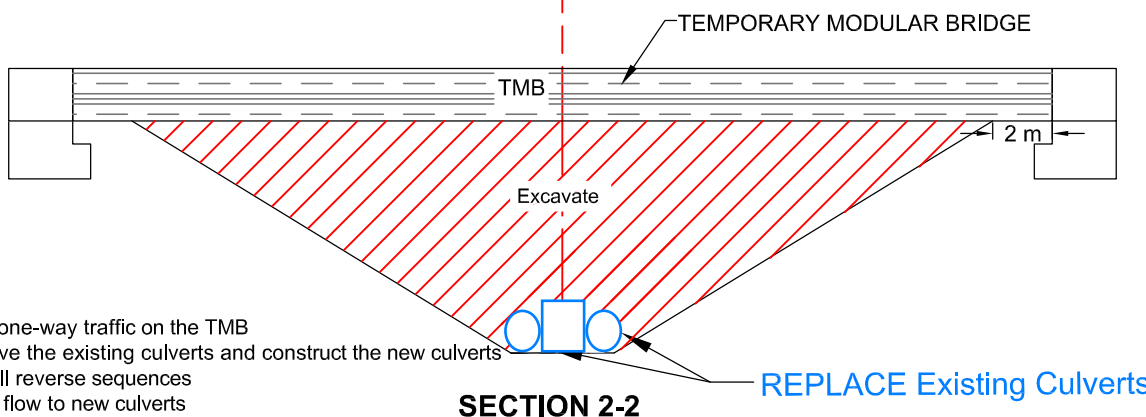
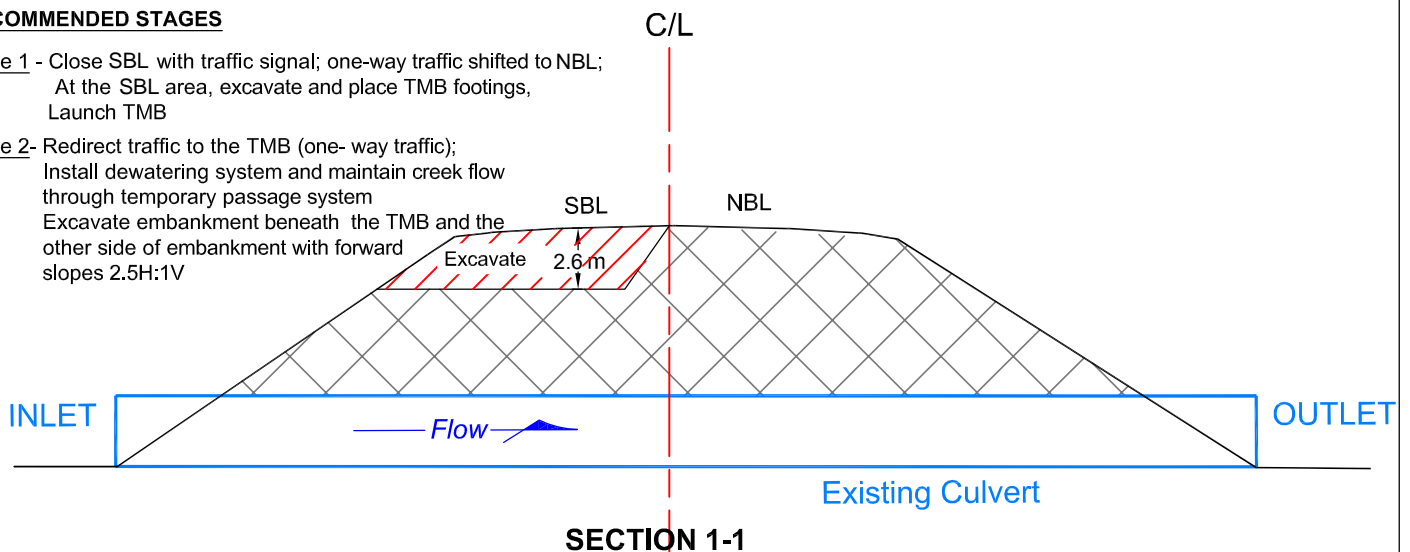
**FIGURE H1: STAGED CONSTRUCTION WITH TEMPORARY MODULAR BRIDGE**  
**SCHEMATIC DIAGRAMS (NST)**



**RECOMMENDED STAGES**

**Stage 1** - Close SBL with traffic signal; one-way traffic shifted to NBL;  
 At the SBL area, excavate and place TMB footings,  
 Launch TMB

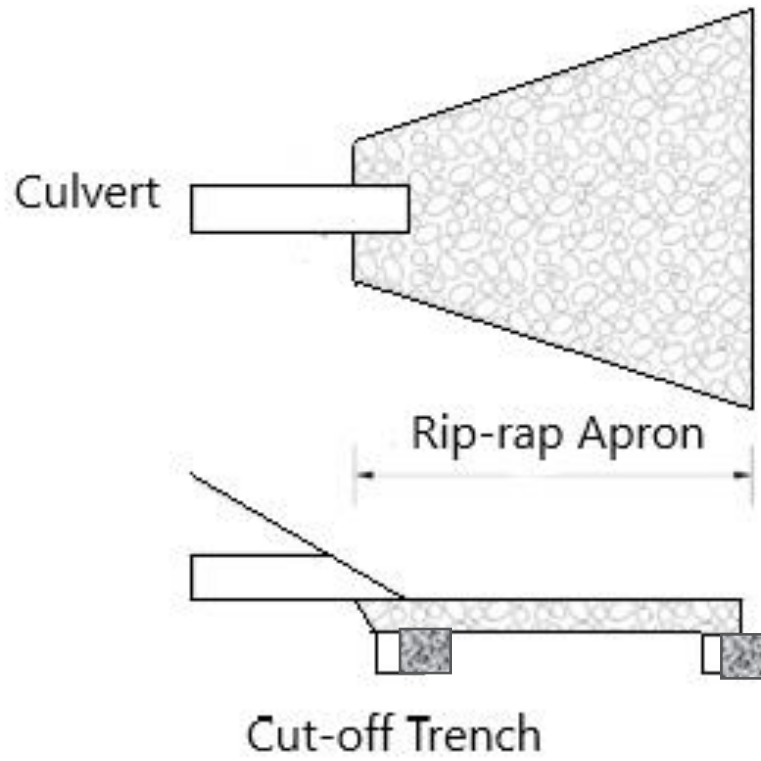
**Stage 2** - Redirect traffic to the TMB (one-way traffic);  
 Install dewatering system and maintain creek flow  
 through temporary passage system  
 Excavate embankment beneath the TMB and the  
 other side of embankment with forward  
 slopes 2.5H:1V



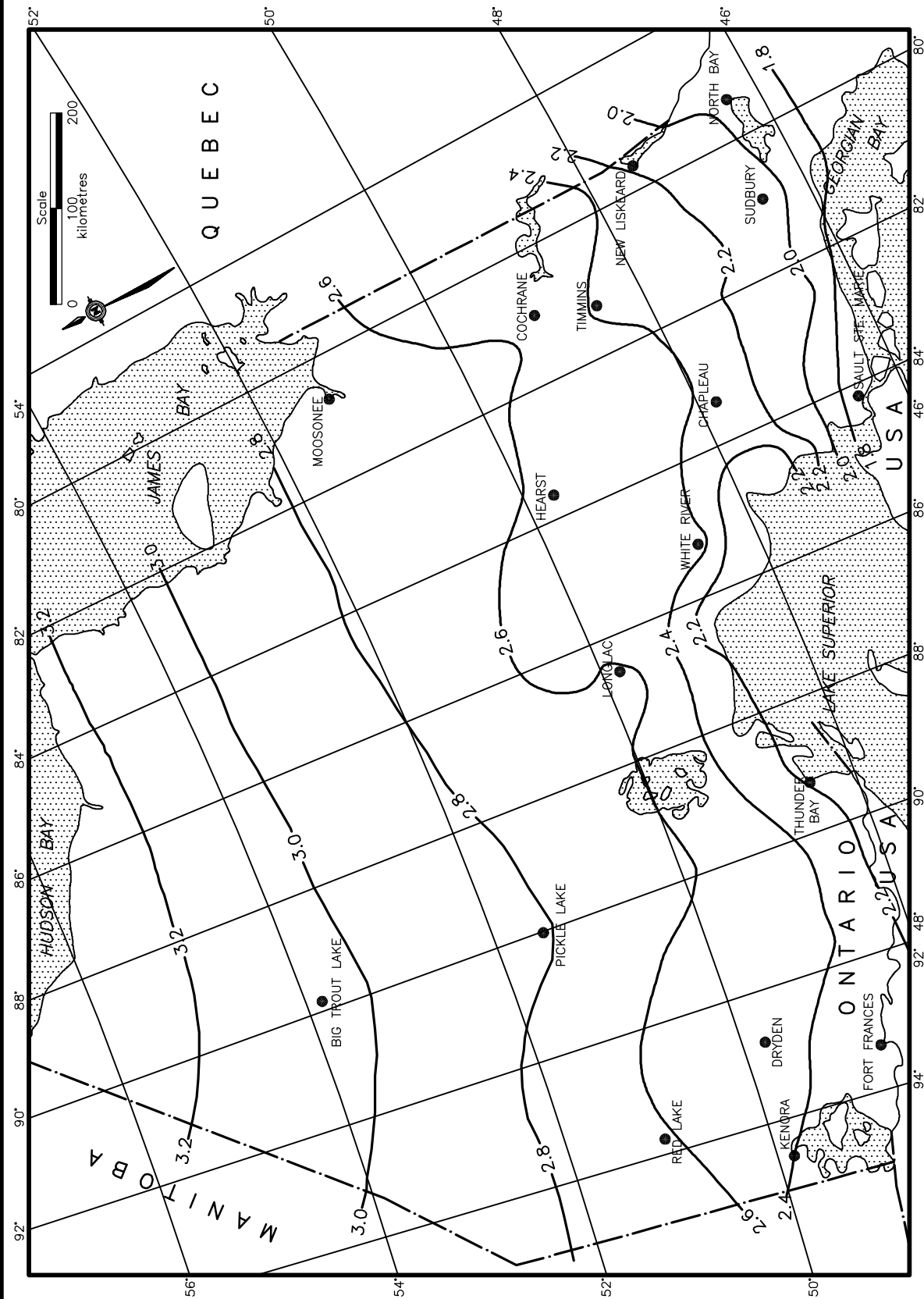
**Stage 3** - Keep one-way traffic on the TMB  
 Remove the existing culverts and construct the new culverts  
 Backfill reverse sequences  
 Divert flow to new culverts  
 Remove temporary flow passage system and backfill  
 Remove dewatering system and place rip rap

**Stage 4** - Switch traffic to the reconstructed side (NBL);  
 Remove the TMB;  
 Backfill and reinstate roadway to final configuration.

FIGURE H2: SCHEMATIC SKETCH OF EROSION PROTECTION AT OUTLET



## Appendix I – OPSDs



# NOTES:

- A These values are approximate and should only be used where the recommendations of a geotechnical engineer are not available.
- B This information is based on the Ministry of Transportation and Communications Research Publication RR225 "Aspects of Prolonged Exposure of Pavements to Sub-Zero Temperatures" dated December 1981.
- C Values between contours should be interpolated. If interpolation is not possible, use the adjacent contour with the greater depth.
- D Frost penetration depths are in metres.

ONTARIO PROVINCIAL STANDARD DRAWING

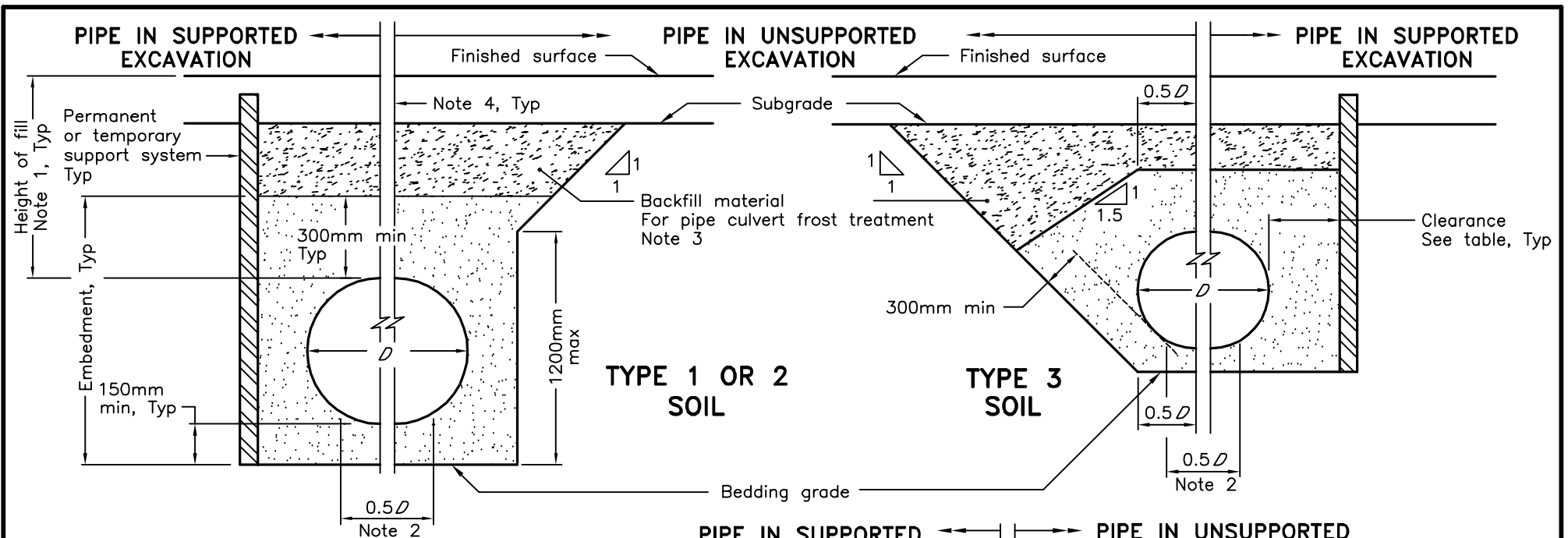
Nov 2010

Rev 1

## FOUNDATION FROST PENETRATION DEPTHS FOR NORTHERN ONTARIO

OPSD 3090.100



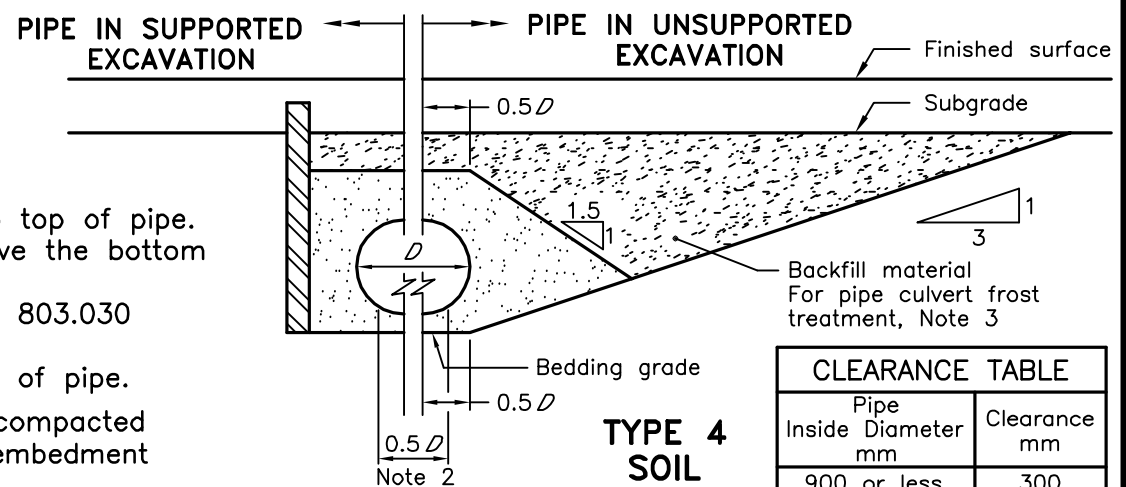


#### LEGEND:

$D$  - Inside diameter

#### NOTES:

- 1 Height of fill is measured from the finished surface to top of pipe.
  - 2 The pipe bed shall be compacted and shaped to receive the bottom of the pipe.
  - 3 Pipe culvert frost treatment shall be according to OPSD 803.030 and 803.031.
  - 4 Condition of excavation is symmetrical about centreline of pipe.
- A Granular material placed in the haunch area shall be compacted prior to placing and compacting the remainder of the embedment material.
- B Soil types as defined in the Occupational Health and Safety Act and Regulations for Construction Projects.
- C All dimensions are in metres unless otherwise shown.



CLEARANCE TABLE	
Pipe Inside Diameter mm	Clearance mm
900 or less	300
Over 900	500

ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2014

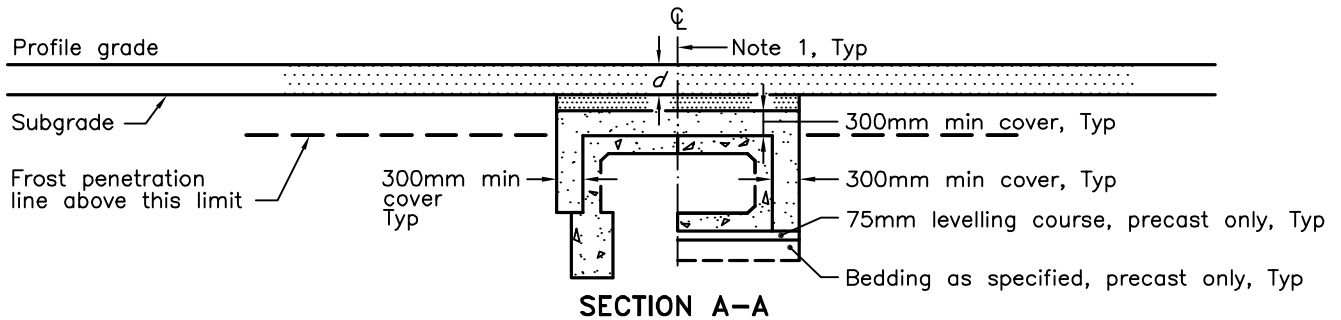
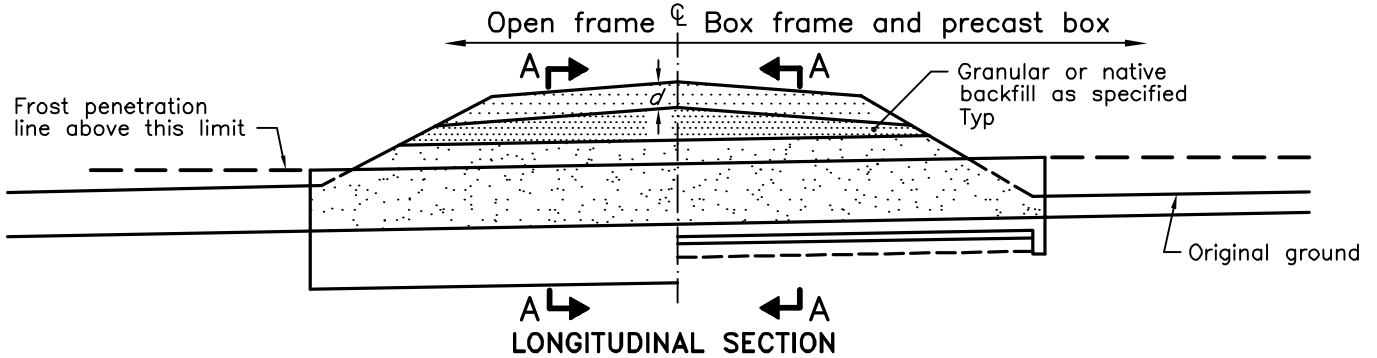
Rev 3

FLEXIBLE PIPE  
EMBEDMENT AND BACKFILL  
EARTH EXCAVATION

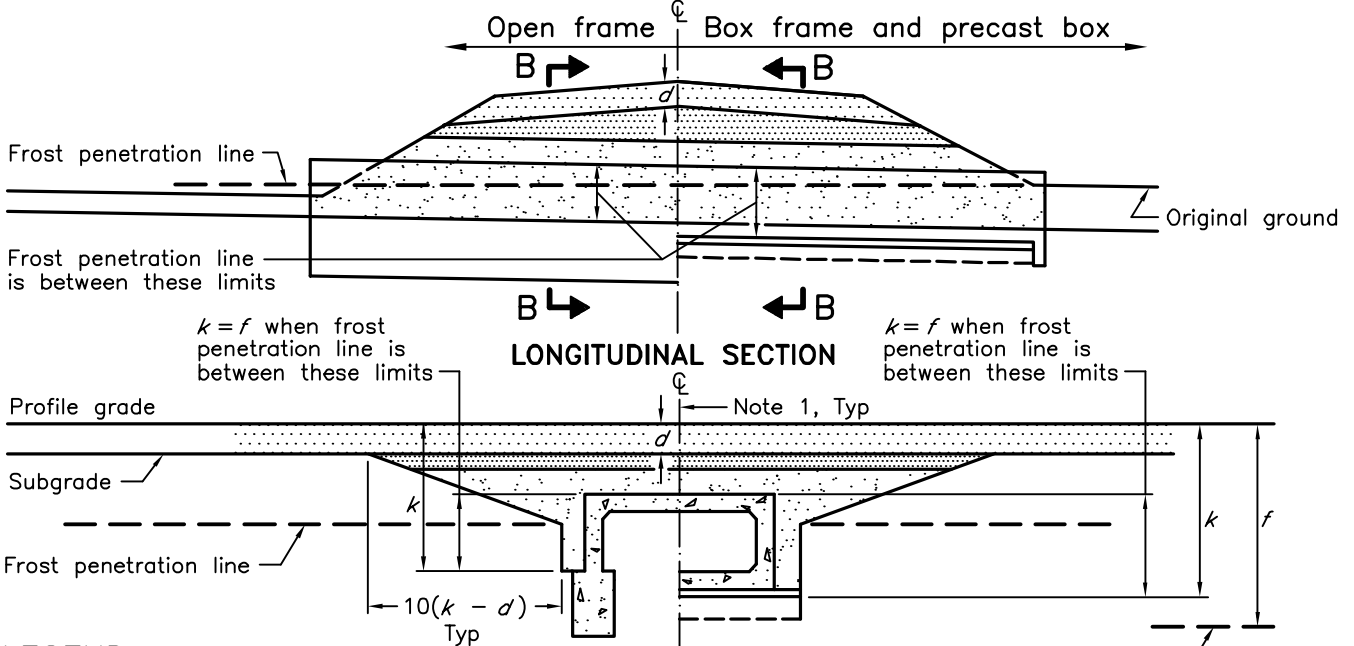
OPSD 802.010



## FROST PENETRATION LINE AT OR ABOVE TOP OF CULVERT



## FROST PENETRATION LINE BELOW TOP OF CULVERT



### LEGEND:

- $d$  = depth of roadbed granular
- $k$  = depth of frost treatment below profile grade
- $f$  = depth of frost penetration below profile grade

### NOTES:

- 1 Condition of frost treatment symmetrical about centreline of culvert.
- A Bedding, levelling, and cover material shall be granular as specified.
- B The depth of roadbed granular shall be 600mm minimum.
- C The maximum depth of frost treatment shall be bottom of box frame or top of footing.
- D All dimensions are in millimetres unless otherwise shown.

ONTARIO PROVINCIAL STANDARD DRAWING

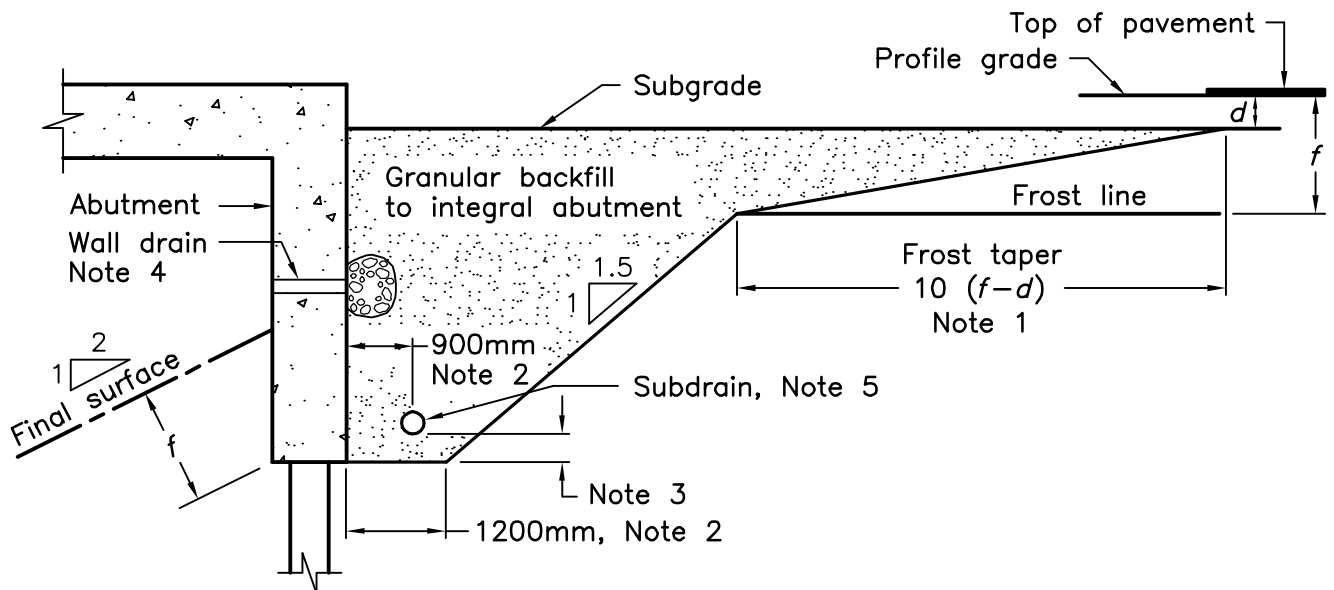
Nov 2010    Rev    2

**BACKFILL AND COVER FOR  
CONCRETE CULVERTS WITH SPANS  
LESS THAN OR EQUAL TO 3.0M**

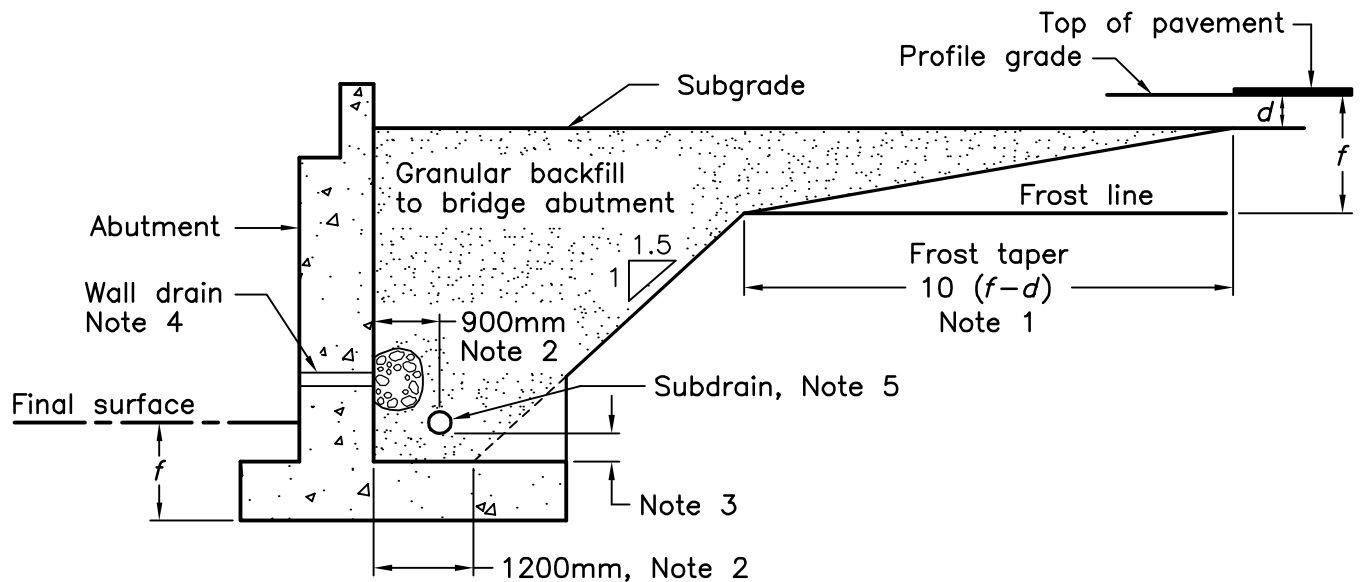
**OPSD 803.010**







### INTEGRAL ABUTMENT



### ABUTMENT

#### NOTES:

- 1  $d$  = depth of combined base and subbase courses  
 $f$  = frost penetration depth as specified
- 2 Dimensions perpendicular to back face of abutment.
- 3 Height to be consistent with positive drainage of subdrain as specified.
- 4 Where specified, wall drains shall be installed according to OPSD 3190.100.
- 5 150mm dia perforated pipe subdrain wrapped with geotextile.
- A Lateral limits of granular backfill to bridge abutment to be inside face to inside face of retaining wall or wingwall. Frost taper shall extend the full width of the backfill unless interrupted by the retaining wall or wingwall.
- B Sections shown are parallel to centreline of roadway.
- C Subdrain shall be installed with a 2% gradient behind wall.
- D All dimensions are in millimetres unless otherwise shown.

ONTARIO PROVINCIAL STANDARD DRAWING

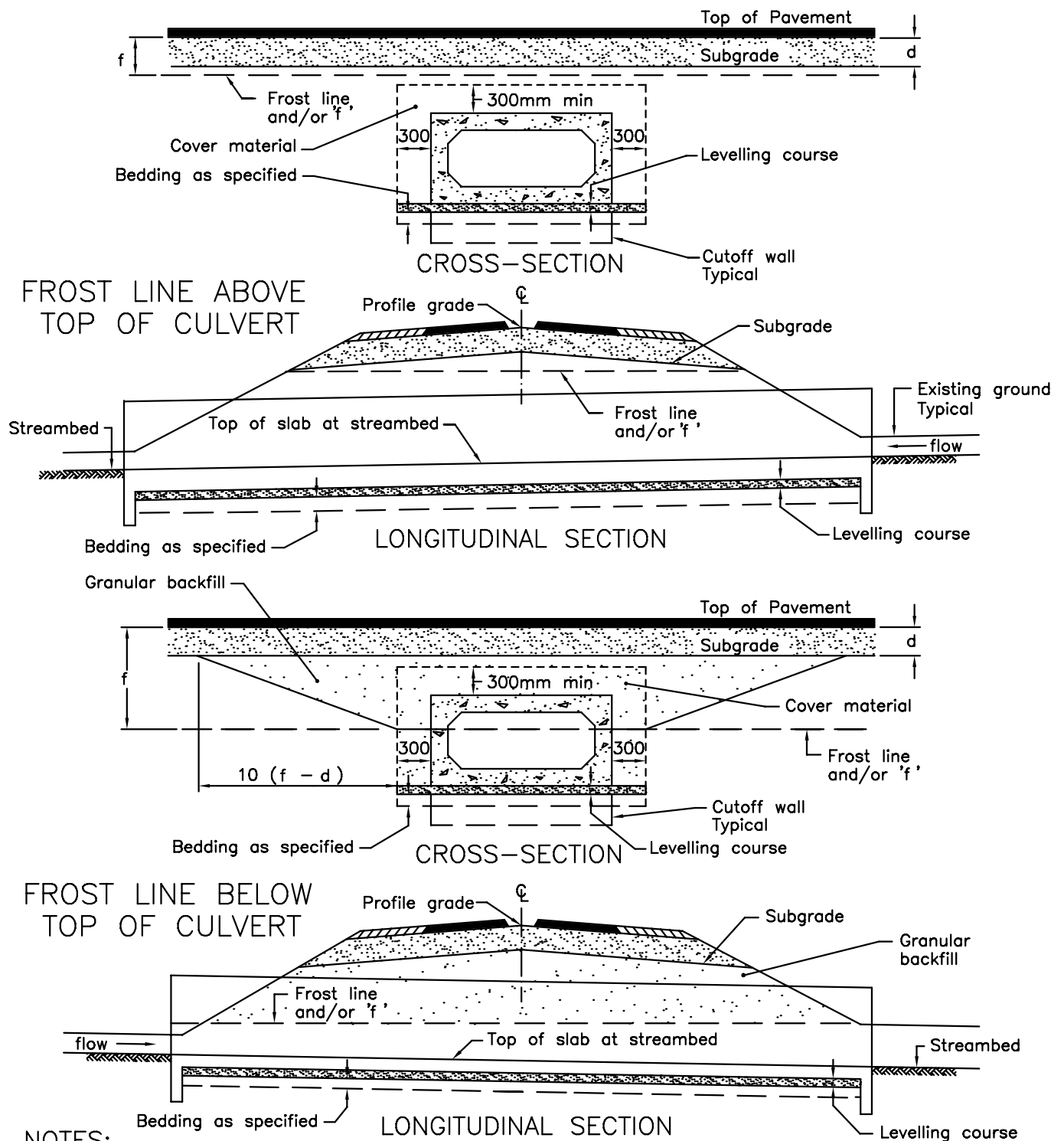
Nov 2010

Rev 1



**WALLS**  
**ABUTMENT, BACKFILL**  
**MINIMUM GRANULAR REQUIREMENT**

**OPSD 3101.150**



NOTES:

- A Bedding, levelling, cover and backfill material to be granular as specified.
- B Where frost line is below bottom of levelling course, frost tapers start at the bottom of levelling course.
- C All dimensions are in millimetres unless otherwise shown.

LEGEND:

d = Denotes depth of granular (roadbed)  
 f = Depth of frost treatment=\_\_\_\_  
 (measured from profile grade)

MINISTRY OF TRANSPORTATION ONTARIO DRAWING

Date | 1994 05 25 | Rev |

**BEDDING AND BACKFILL**  
 FOR PRECAST CONCRETE BOX CULVERTS

Issue Date  
 WP  
 Issued by

MTOD - 803.021

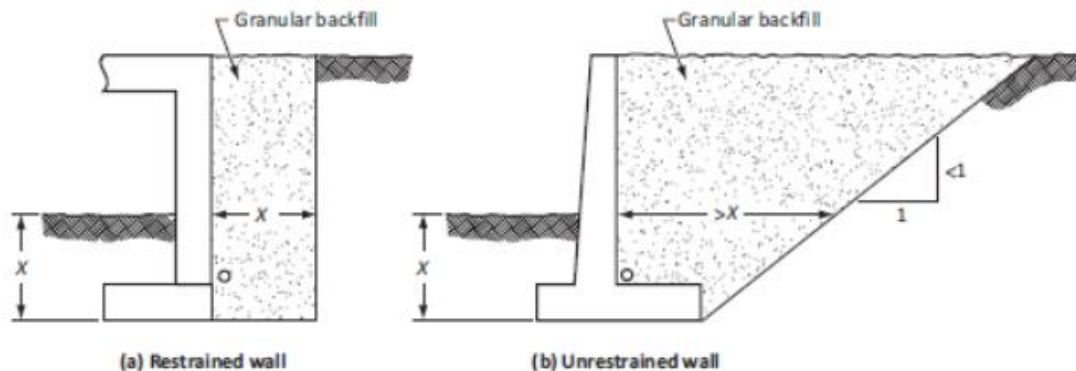
- A drainage system behind a retaining structure should ensure that a groundwater table does not exist above the footing level. Preferably, the ground water level is controlled by the use of free-draining granular backfill and a collection system such as weep holes or perforated drains at the footing level. These weep holes and drains should be inspected and maintained to ensure that they do not become blocked. If free-draining, granular backfill is not employed, the permeability of the backfill and the hydrostatic head will control the extent to which the groundwater table can be depressed locally by seepage towards a footing drain. In practice, design for frost protection is best done using free-draining backfill.

The design should also consider the risk of unusually large inflows of water creating a temporary hydrostatic head of water behind the wall. An example is the overtopping of a retaining wall, adjoining a large body of water, by storm waves. Measures such as the use of quarried rock backfill, design for full hydrostatic pressure, or provision of a sloped impermeable surface layer should be considered.

Measurements have shown that earth pressures can vary seasonally, but the effects have normally been neglected in design, except for winter frost pressures. These latter can be very large if the backfill is frost susceptible and for this reason free-draining granular backfill is recommended.
- Figure C6.20 shows examples of minimum backfill requirements.

The distance,  $x$ , should be equal to or greater than the estimated vertical frost penetration. This distance may be reduced if the wall abuts a vertical face of bedrock that is not susceptible to frost. The frost penetration may be reduced by the use of suitable insulation, in which case a thermal analysis should be performed by a Geotechnical Engineer.

If rock fill is used as a backfill material, consideration should be given to the possible deterioration of the rockfill with time, which could result in the reduction or even the total loss of free-draining properties and, hence, increased frost susceptibility.

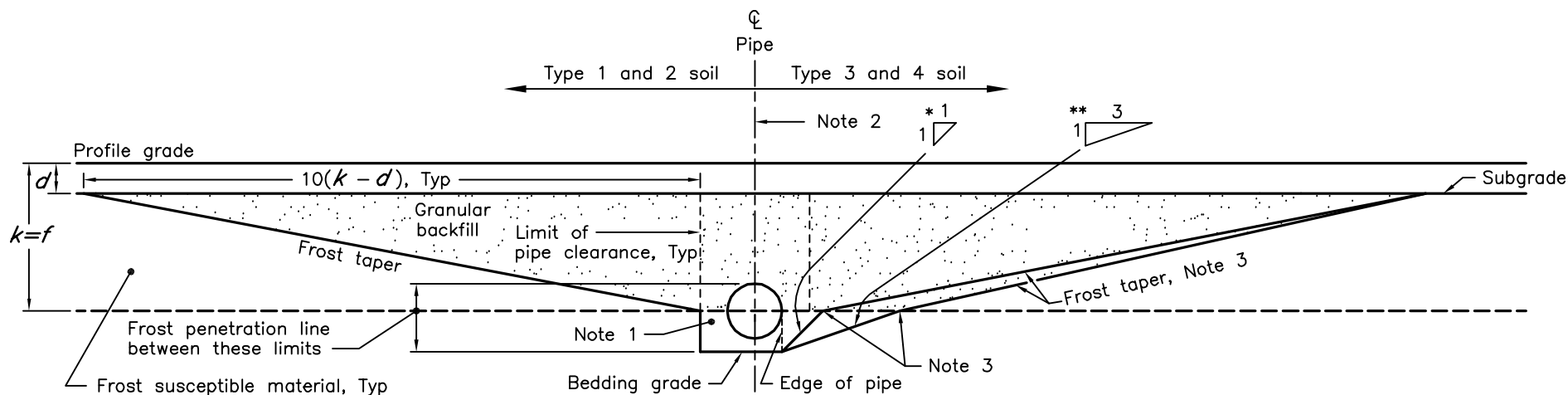


**Figure C6.20**  
**Backfill for frost protection**  
 (See Clause C6.12.1.)

## C6.12.2 Lateral ground pressures

### C6.12.2.1 General

Earth pressure acting on a structure depends on the relative movement of the structure, the backfill, the type of soil adjacent to the backfill, and the soil below the footing or supporting piles. Appropriate geotechnical parameters should be chosen for the calculation of lateral pressures based on recognized geotechnical theories as specified in Clause 6.12.2.2 for the backfill behind the wall. Geotechnical parameters frequently used in allowable stress design methods are applicable in limit states design pressure calculation. Where the possibility exists, hydrostatic pressure needs to be considered, e.g., in situations where walls are partially submerged or where non-free-draining backfill is used.



## FROST TREATMENT RIGID AND FLEXIBLE PIPE

### NOTES:

- 1 Pipe embedment or bedding, cover, and backfill shall be according to:
  - a) Flexible OPSD 802.010, 802.013, 802.014, 802.020, 802.023, and 802.024.
  - b) Rigid – OPSD 802.030, 802.031, 802.032, 802.033, 802.034, 802.050, 802.051, 802.052, 802.053, and 802.054.
- 2 Condition of frost treatment symmetrical about centreline of pipe.
- 3 Frost tapers shall start at the intersection of the 1H:1V or 3H:1V slope and the frost penetration line.
- A Soil types as defined in the Occupational Health and Safety Act and Regulations for Construction Projects.

### LEGEND:

- $d$  – depth of roadbed granular  
 $k$  – depth of frost treatment below profile grade  
 $f$  – depth of frost penetration below profile grade  
 $*$  – Type 3 soil  
 $**$  – Type 4 soil

ONTARIO PROVINCIAL STANDARD DRAWING

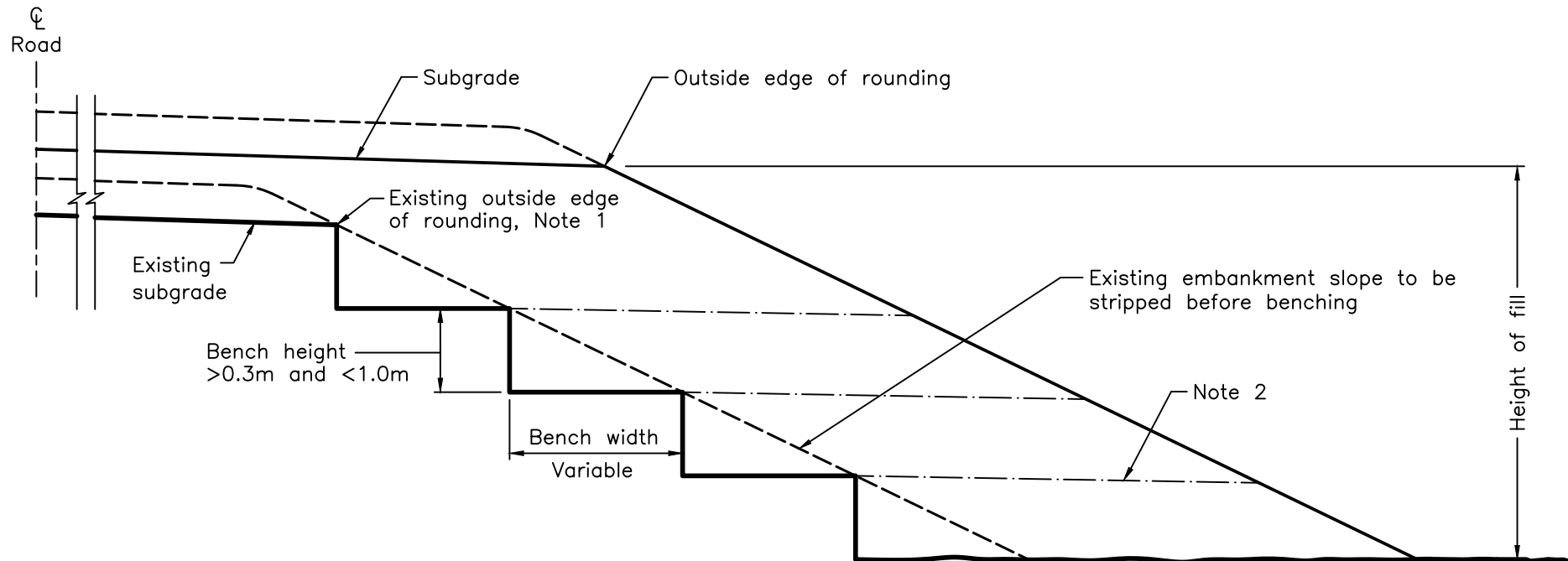
FROST TREATMENT – PIPE CULVERTS  
FROST PENETRATION LINE BETWEEN  
TOP OF PIPE AND BEDDING GRADE

Nov 2015

Rev 4



OPSD 803.031



**NOTES:**

- 1 When the subgrade is below the existing outside edge of rounding, benching shall be carried out below the point where the subgrade intersects the existing slope.
  - 2 Benches shall be excavated one level at a time and the fill placed and compacted before the next bench is excavated.
- A Benching is not required on existing slopes flatter than 3H:1V.

ONTARIO PROVINCIAL STANDARD DRAWING

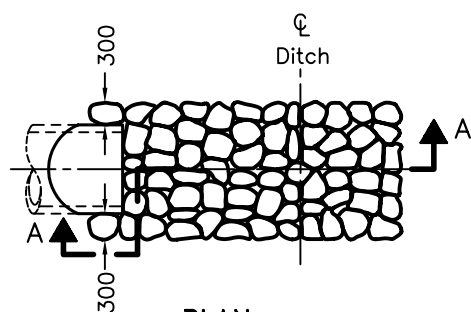
Apr 2019

Rev 4

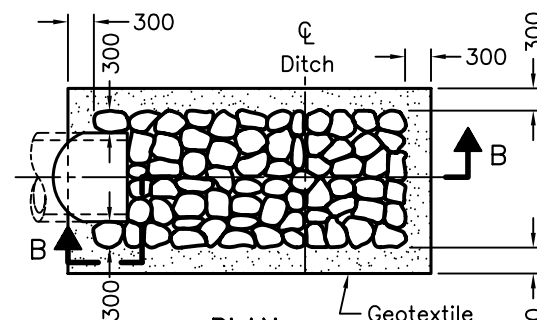
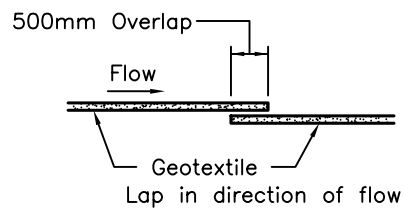
**BENCHING OF EARTH SLOPES**

**OPSD 208.010**

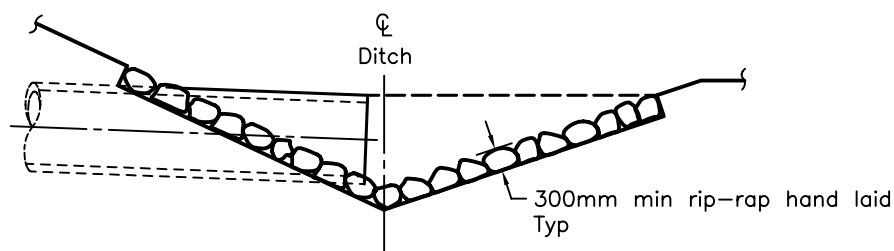




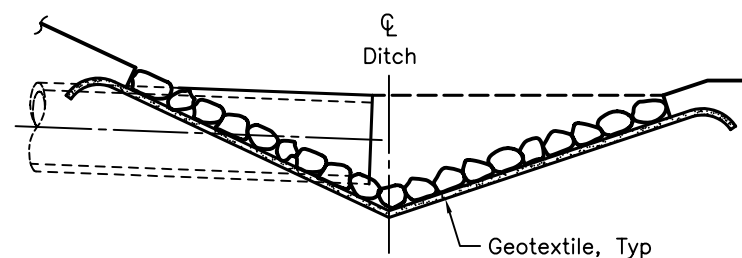
PLAN  
CUT OR FILL



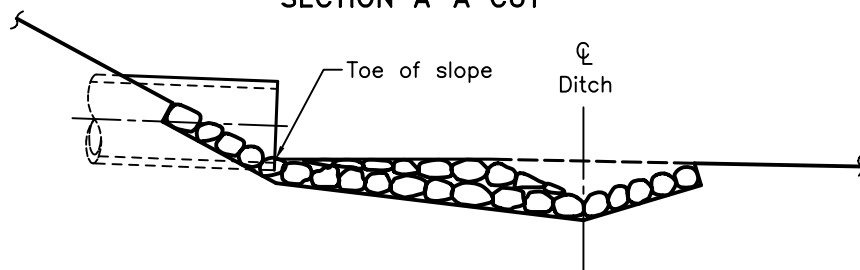
PLAN  
CUT OR FILL



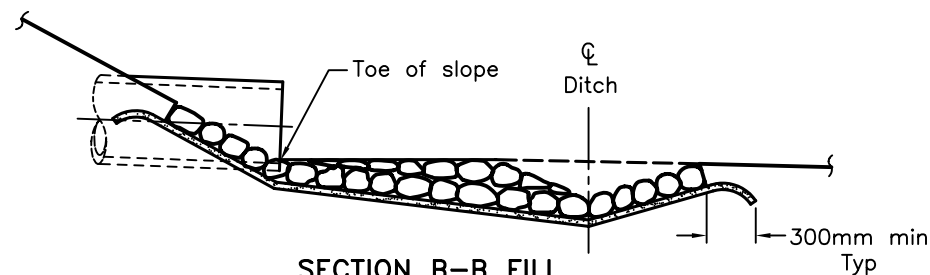
SECTION A-A CUT



SECTION B-B CUT



SECTION A-A FILL  
TYPE A – WITHOUT GEOTEXTILE



SECTION B-B FILL  
TYPE B – WITH GEOTEXTILE

NOTES:

A All dimensions are in millimetres unless otherwise shown.

ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2001

Rev 0

RIP-RAP TREATMENT  
FOR SEWER AND CULVERT OUTLETS



OPSD – 810.010

## Appendix J – 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: 50.201N 90.726W

2021-02-22 20:53 UT

Probability of exceedance per annum	0.000404	0.001	0.0021	0.01
Probability of exceedance in 50 years	2 %	5 %	10 %	40 %
Sa (0.05)	0.060	0.029	0.015	0.002
Sa (0.1)	0.081	0.042	0.023	0.004
Sa (0.2)	0.072	0.039	0.022	0.005
Sa (0.3)	0.057	0.032	0.019	0.004
Sa (0.5)	0.040	0.023	0.014	0.003
Sa (1.0)	0.020	0.011	0.006	0.001
Sa (2.0)	0.008	0.004	0.002	0.001
Sa (5.0)	0.002	0.001	0.001	0.000
Sa (10.0)	0.001	0.001	0.000	0.000
PGA (g)	0.044	0.022	0.012	0.002
PGV (m/s)	0.028	0.015	0.008	0.001

**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

Ressources naturelles  
Canada

Canada



## Appendix K – NSSPs

## **OBSTRUCTIONS – LAYER OF COBBLES AND BOULDERS AND SLOPING STRONG BEDROCK**

---

### **Non Standard Special Provision**

---

#### **Scope of Work**

The Contractor shall be alerted to the presence of a layer of cobbles and boulders in the fill and overburden deposits as encountered in various boreholes advanced at the site. The Contractor also should be aware that there is sloping bedrock in the area and fractured bedrock was encountered within the upper 0.5 m of the bedrock surface. Therefore, appropriate equipment and procedures will be required for excavation of overburden and bedrock. Since such obstructions may impede excavation progress, the Contractor shall be prepared the plan how to penetrate/remove these obstructions to achieve the design depths.

Due to sloping bedrock at the site from the east toward the west and from the north toward the south, the rock excavation for structure will be uneven along the structure. More rock excavation should be anticipated at the east and north sides (i.e. relative to the existing culvert).

#### **Basis of Payment**

Payment at the lump sum contract price for this tender item shall be full compensation for all labour, equipment and materials for completion of the work.

## **MASS CONCRETE ON BEDROCK**

---

Non Standard Special Provision

---

### **Scope of Work**

The scope of work for the above noted tender item includes the mass concrete under the strip footings for the SPCSP open footing culvert and modular bridge.

The Contractor should be aware that there is sloping bedrock in the area and fractured bedrock was encountered within the upper 0.5 m of the bedrock surface. Mass concrete volumes will vary depending on the variable intact bedrock surface.

### **Construction**

Concrete shall be of the same strength as the footing concrete and placed in accordance with OPSS.PROV 904 "Construction Specification for Concrete Structures".

### **Basis of Payment**

Payment at the contract price for the above noted tender item includes full compensation for all labour, equipment and materials to do the required work.

## **DOWELLS INTO ROCK**

---

### **Non Standard Special Provision**

---

#### **Scope of Work**

Work under this item is for the placement and field testing of dowels into rock at the foundations for spread footings founded on bedrock.

#### **Materials and Installation**

Dowels into rock shall be constructed in accordance with OPSS.MUNI 904 "Construction Specification For Concrete Structures". All reinforcing steel supplied shall be in accordance with OPSS.PROV 1440 "Material Specification For Steel Reinforcement For Concrete" (dowel bars conforming to CSA Standard CSAG30.18, Grade 400 W).

Where dowels are to be placed in rock, holes shall be drilled to the required depth and size. Hole diameter shall be two times the nominal diameter of the dowel. Each hole shall be cleaned out, grouted and the dowel set in place. Grout shall be of the same strength as the footing concrete (or at least 25 MPa at 28 days).

If the hole contains water, the contractor shall remove the water otherwise a tremie procedure shall be used to completely fill the hole with grout. The dowel shall be installed into the hole after the grout has been placed and while it is still fresh.

Dowels shall be capable of sustaining the pull test loads specified in OPSS.MUNI 904 (Table 1) without displacement for a time period of not less than 1 minute.

#### **Dowel Testing**

The Contractor shall carry out pull testing of dowels in the trial installations and during production. Pull test loads shall be according to OPSS.MUNI 904 (Table 1).

#### **Basis of Payment**

Payment at the contract price for this tender item shall be full compensation for all labour, equipment and materials for completion of the work.

## **PRE-CONDITION SURVEYS AND MONITORING DURING ANY BLASTING**

---

### **Non Standard Special Provision**

---

#### **Scope of Work**

If any blasting is required, the Blast Contractor must be fully qualified and experienced. The Blast Contractor shall outline the procedure and extent of the pre-blast survey. The blast methodology, including drill hole patterns, hole size and depths, size of blast, explosive and initiation product details, as well as all blast control procedures shall be required. Blast control procedures would include details on controlling flyrock, temporary road closures, blast signaling and site clearing procedures. Details on instrumentation, number and location of monitoring sites, blast recording and reporting procedures, and procedures to be followed in the event of excessive vibration readings are required as well. OPSS 120 will be used to provide a limit of peak particle velocity (PPV).

Instrumentation or monitoring ground and air vibration effects from the blasting could be set up in accordance with the International Society of Explosives Engineering field practice guidelines (2020).

#### **Basis of Payment**

Payment at the contract price for this tender item shall be full compensation for all labour, equipment and materials for completion of the required work.

## **REMOVAL OF PROTECTION SYSTEM**

---

### **Non Standard Special Provision**

---

#### **Scope of Work**

If protection systems are specified for removal or the Contractor elects to remove, the method and sequence of removal; should be such that there will be no damage to the new work, existing work, and facility being protected.

If protection systems are left in place, the top should be removed to at least 1.2 m below the finished grade or ground level or at least 0.6 m below the streambed.

All disturbance areas have to be restored to an equivalent or better condition than existing prior to the commencement of construction.

#### **Basis of Payment**

Payment at the lump sum contract price for this tender item shall be full compensation for all labour, equipment and materials for completion of the work.

## **NSSP FOR SLOPE STABILITY ANALYSES REQUIRED FOR TEMPORARY EMBANKMENT WIDENING**

### **Scope of Work**

The Contractor shall perform their own slope stability analyses with widened embankment if during construction temporary widening is used.

### **Basis of Payment**

Payment at the lump sum contract price for this tender item shall be full compensation for labours for completion of the work.

## **DEWATERING STRUCTURE EXCAVATIONS - Item No.**

---

Special Provision

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### **Amendment to OPSS 902, November 2010**

#### **902.02 REFERENCES**

Section 902.02 of OPSS 902 is amended by the addition of the following:

#### **Ontario Provincial Standard Specifications, Construction**

OPSS 805 Temporary Erosion and Sediment Control Measures

#### **902.03 DEFINITIONS**

Section 903.03 of OPSS 902 is amended by the addition of the following:

**Automatic Transfer Switch** means an electrical device that transfers power supply to a backup power source when there is an outage of the primary power source.

**Cofferdam** means as defined in OPSS 539.

**Cut-Off Wall** means a below grade wall that restricts groundwater flow and/or supports excavations, typically using soil-bentonite or cement-bentonite.

**Design Storm Return Period** means the average number of years based upon probability, between the occurrences of a storm event of a certain severity or greater.

**Dewatering System** means the components required to control water to permit construction work to proceed under specified conditions, and may include a groundwater control system, impermeable barriers, pumps, and/or equipment to carry out unwatering.

**Groundwater Control System** means sump pumps, oversized excavations with perimeter ditches, deep wells or well points or other systems used to lower the groundwater table.

**Plug** means an impervious, natural, or constructed drainage work that blocks water.

**Sediment** means soil particles detached from an earth surface by erosion.

**Sediment Control Measure** means a measure to remove sediment from water prior to discharge to the natural environment and sewer systems.

**Temporary Flow Control** means temporary flow control devices, channels, pipes, and other materials used to convey or divert water past an area under construction.

**Unwatering** means the removal of ponded or flowing surface water.

**Vegetated Discharge Area** means a sloped, open area of land with existing vegetation suitable to prevent erosion.



**Waterbody** means as any permanent or intermittent, natural or constructed body of water including lakes, ponds, wetlands and watercourses, but does not include sewage works as defined in the Ontario Water Resources Act.

**Watercourse** means a stream, creek, river, or channel including ditches, in which the flow of water is permanent, intermittent, or temporary.

## **902.04 DESIGN AND SUBMISSION REQUIREMENTS**

Subsections 902.04.01 and 902.04.02 of OPSS 902 are deleted in their entirety and replaced with the following:

### **902.04.01 Design Requirements**

#### **902.04.01.01 Dewatering**

A dewatering system shall be designed to control water and the flow of water into the excavation, prevent disturbance of the foundation, permit the placing of concrete in the dry, and complete the excavating and backfilling for structures work. The design of the system shall be sufficient to permit the work to be carried out as specified in the Contract Documents.

The design shall meet the requirements of the Contract Documents, and where a waterbody is present, shall include channel and inlet and outlet protection measures as required to protect the environment in the event of system failure or the design flow rate being exceeded.

The design shall not include the use of embankments and/or structures in public use, either existing or to be constructed as part of the Work, to control or stop water flow, unless approved by the Contract Administrator.

The design shall not result in displacement or damage to property, buildings, structures, utilities and other facilities adjacent to the Working Area, including from drawdown related settlement or other groundwater related effects.

The system shall be designed to prevent soil loss or erosion where water is removed, pumped, or discharged. The system shall be designed to prevent basal heave or instability.

Where the system involves the taking of water from a waterbody, the design shall maintain the flow of water and the natural functions of the waterbody upstream and downstream of the work area, and shall not interfere with other uses of the water.

When the system includes temporary flow control, the temporary flow control shall be designed, as a minimum, for a [\* Designer Fill-In, See Notes to Designer] year design storm return period, and groundwater discharge. A longer return period shall be used when determined appropriate for the work.

Temporary flow control shall include provision for fish passage during low flows.

## **902.04.02**

## **Submission Requirements**

### **902.04.02.01**

### **Working Drawings**

Three (3) sets of Working Drawings for the dewatering system shall be submitted to the Contract Administrator at least 7 Days prior to commencement of the dewatering system installation, for information purposes only. Prior to submission of Working Drawings, the seals and signatures of a design Engineer and a design-checking Engineer shall be affixed on the Working Drawings verifying that the drawings are consistent with the Contract Documents.

One person shall not perform both the design Engineer and design-checking Engineer roles for a system.

Where multi-discipline engineering work is depicted on the same Working Drawing and the design or design-checking Engineer or both are unable to seal and sign the Working Drawing for all aspects of the work, the drawing shall be sealed and signed by as many additional design and design-checking Engineers as necessary.

The following information and details shall be shown on the Working Drawings, where applicable:

a) Plans, Elevations, and Details

- i. Type of system(s).
- ii. Design calculations demonstrating adequacy of the system and equipment.
- iii. Design flow rate(s).
- iv. Plan location, description, and dimensions of system components, including dams, cofferdams, cut-off walls, temporary channels, pipes, culverts, sewers, groundwater control systems employing wells and/or well points, sedimentation basins, tanks, pumps, power supply, and standby equipment.
- v. Method of management of pumped water and plan location of all dewatering discharge points.
- vi. Profile drawings shall extend through and immediately beyond the limits of the system.
- vii. Water elevations upstream and downstream of the system at design flow rate.
- viii. Dam height or crest elevation, cofferdam depth and tip elevation, cutoff wall depth or base elevation, pipe invert elevations, depths of wells and wellpoints, pump intake elevation, and sedimentation basin depth or base elevation.
- ix. Plan location, elevation, and dimensions of environmental protection measures.
- x. Pipe type, size, and length, pump capacity, and tank capacity.
- xi. Material and construction standards to be used for the work.
- xii. Method for establishing and monitoring construction site groundwater levels.
- xiii. Criteria and method of removal of the system.

b) Procedures for the system construction, operation, and maintenance, including daily start-up sequence where applicable, and operation shut down.

c) Procedures for the removal of the system, including the removal sequence, and well decommissioning.

d) Stand-by power or pumping system requirements and the use of automatic transfer switching, when required to protect the environment and the Work.

e) A copy of the Permit to Take Water issued by the Ministry of the Environment and Climate Change or confirmation of registration of water taking for construction dewatering, if a permit or registration is required by provincial regulation.

f) When applicable, a copy of the water taking report and discharge plan required by provincial regulation.

- g) A copy of any necessary permits for the discharge of water to a sanitary sewer, or stormwater sewer system, stormwater pond, or other facility.

#### **902.04.02.02                      Preconstruction Survey**

When a groundwater control system by wells or a well point system will be used, a condition survey of property and structures that may be affected by the work shall be carried out. The condition survey shall include the location and condition of adjacent properties, buildings, underground structures, water wells, Utilities, and structures, within a distance of [\*\* ] **N/A** See Notes to Designer] metres from the groundwater control system. In addition, all water wells used as a supply of drinking water and located within this distance shall be tested for compliance with Ontario Drinking Water Quality Standards.

Water wells within the preconstruction survey distance can be located using the website <https://www.ontario.ca/environment-and-energy/map-well-records> or its successor site.

Copies of the condition survey and water quality test results shall be submitted to the Contract Administrator prior to the operation of the groundwater control system.

#### **902.04.02.03                      Milestone Inspections**

The Quality Verification Engineer shall witness the following Interim Inspections of the work:

- a) Dewatering of excavation for structure.
- b) Completion of excavation for foundation.
- c) Excavation for backfill and frost tapers.
- d) Backfilling.

A copy of the written permission to proceed shall be submitted to the Contract Administrator prior to commencement of the successive operation.

#### **902.07                                      CONSTRUCTION**

Subsection 902.07.04 of OPSS 902 is deleted in its entirety and replaced with the following:

##### **902.07.04                                      Dewatering Structure Excavation**

##### **902.07.04.01                                      General**

The dewatering systems shall be constructed and operated according to the Working Drawings.

Activation of temporary flow control, if applicable, shall be as specified in the Contract Documents.

The dewatering system shall be continuously operational to control buoyancy forces until such forces can be resisted by backfill and structure self-weight, to keep excavations stable, to avoid erosion impacts from the release of accumulated water, and to keep the work area in the condition required to complete the associated work as specified in the Contract Documents.

When temporary flow control is to remain operational through a seasonal shutdown period, the Contractor shall be responsible for any maintenance or repair costs due to the temporary flow control during the seasonal shutdown period.

Temporary erosion and sediment control measures, including to control the discharge of water, shall be according to OPSS 805. Measures not specified in OPSS 805 shall be according to the Working Drawings. Temporary erosion and sediment control measures and cover material to protect exposed soils, as required by the Working Drawings, shall be installed as soon as is practical.

Stranded fish shall be managed as specified in the Contract Documents.

Unwatering shall be carried out as necessary.

Water suspected of being contaminated as indicated by visual or olfactory observations shall be reported to the Contract Administrator.

Dewatering and temporary flow control shall be discontinued in a manner that does not disturb any structure, pipeline, or flow channel. Operation of the dewatering system shall be shut down according to the procedures specified in the Working Drawings, where applicable.

#### **902.07.04.02 Discharge of Water**

Water from dewatering and unwatering operations shall be directed to a sediment control measure and/or a vegetated discharge area 30 m away from waterbodies or as far away as practicable from the top of the bank of any waterbody, prior to discharge to the natural environment.

Equipment and materials shall not be used or stored in vegetated discharge areas.

The discharge of water to the natural environment shall not be directed across pavements, sidewalks, curb and gutter or similar hard surfaces except through appurtenances as specified in the Contract Documents.

#### **902.07.04.03 Monitoring**

The Contract Administrator shall be notified of any complaints and any action taken or proposed to be taken in response to complaints.

Daily external visual monitoring of the surrounding area and property and structures on the preconstruction survey, if applicable, for impacts such as settlement and erosion shall be completed. Any observed impacts shall be immediately reported to the Contract Administrator. When public safety, the environment, or property is impacted or potentially impacted, the design Engineer shall, without delay, make a full assessment and direct changes to the system to eliminate impacts or potential impacts. Any changes shall be documented according to the System Amendments subsection.

When a groundwater control system is observed to negatively impact water supplies obtained from any adequate sources that were in use prior to groundwater control system operation, then water shall be supplied to the affected water users. The water shall be equivalent in quantity and quality to the normal water takings of the users. Supply shall continue until the negative impacts on the water supplies are removed, or until Contract Completion, whichever occurs first.

#### **902.07.04.04                      System Amendments**

When displacement or damage to embankments and/or structures, or property adjacent to the Working Area, occurs due to the operation of the system, or soil loss or erosion occurs where water is removed, pumped, or discharged, the dewatering system or temporary flow control shall be amended to stop the displacement, damage, soil loss, or erosion.

Amendments shall be submitted to the Contract Administrator within two Business Days of the system being amended, on revised Working Drawings bearing the seal and signature of the design Engineer and design-checking Engineer.

#### **902.07.04.05                      Removal**

Dewatering system and temporary flow control components shall be removed when no longer required. Removal of system components shall be according to the procedures specified on the Working Drawings, where applicable, and as specified in the Contract Documents.

Deactivation of temporary flow control shall be as specified in the Contract Documents.

Removal of temporary drainage work shall be according to OPSS 510.

Environmental protection measures and cut-off walls shall be removed, unless approved otherwise by the Contract Administrator.

Sedimentation basins and other excavations shall be backfilled with the original soil excavated, unless approved otherwise by the Contract Administrator. All disturbed areas shall be restored to an equivalent or better condition than existed prior to the commencement of construction.

#### **NOTES TO DESIGNER:**

##### **Designer Fill-Ins**

\* Fill in the design storm return period according to MTO Drainage Design Standard TW-1.

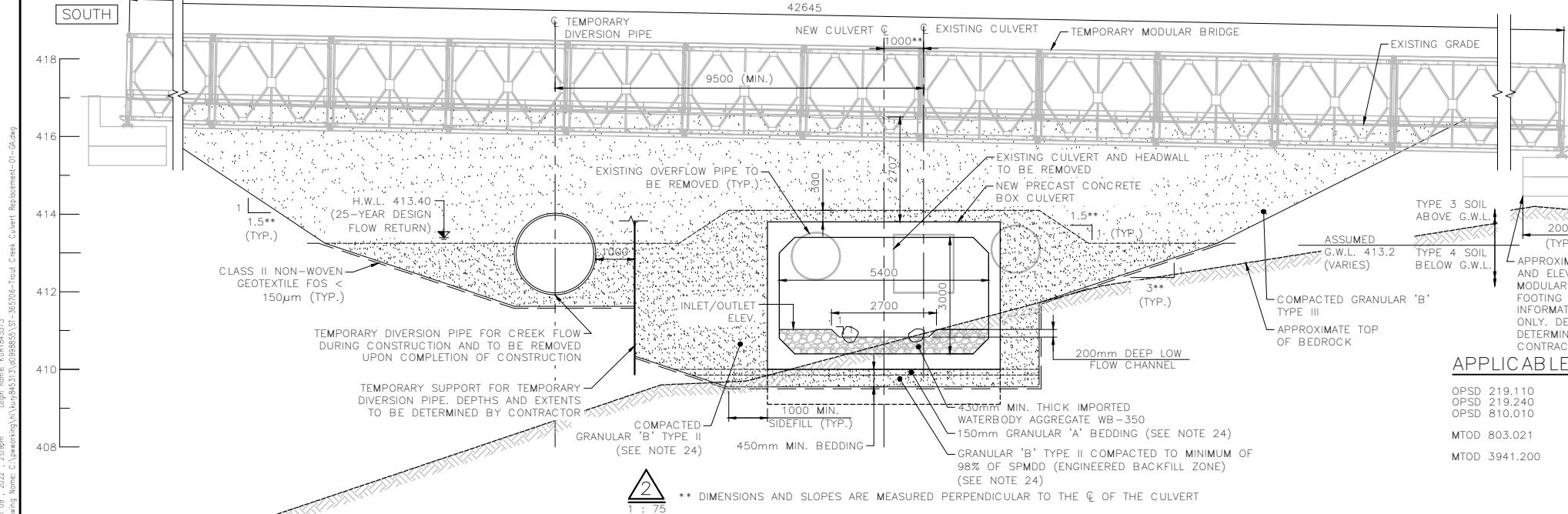
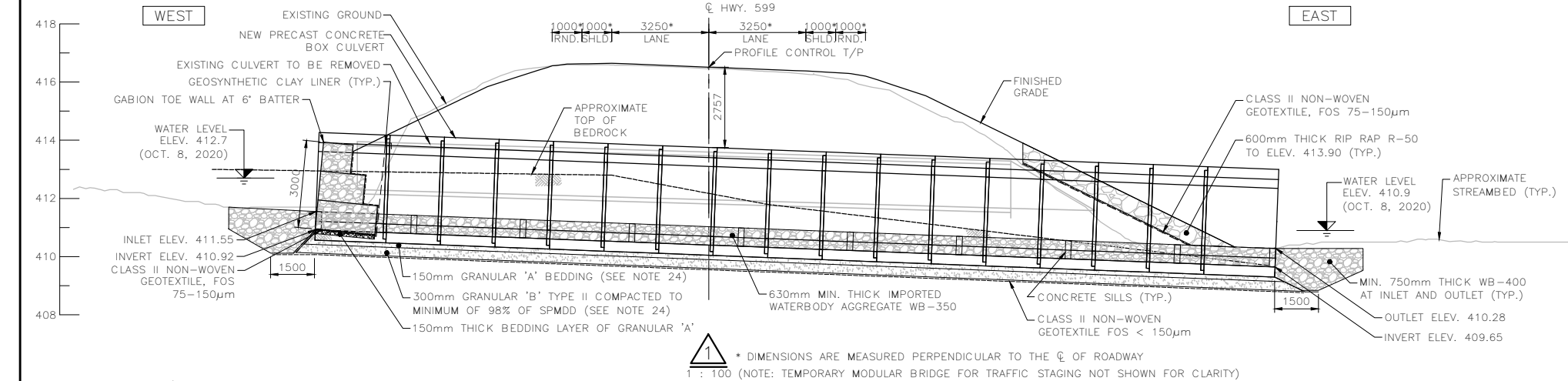
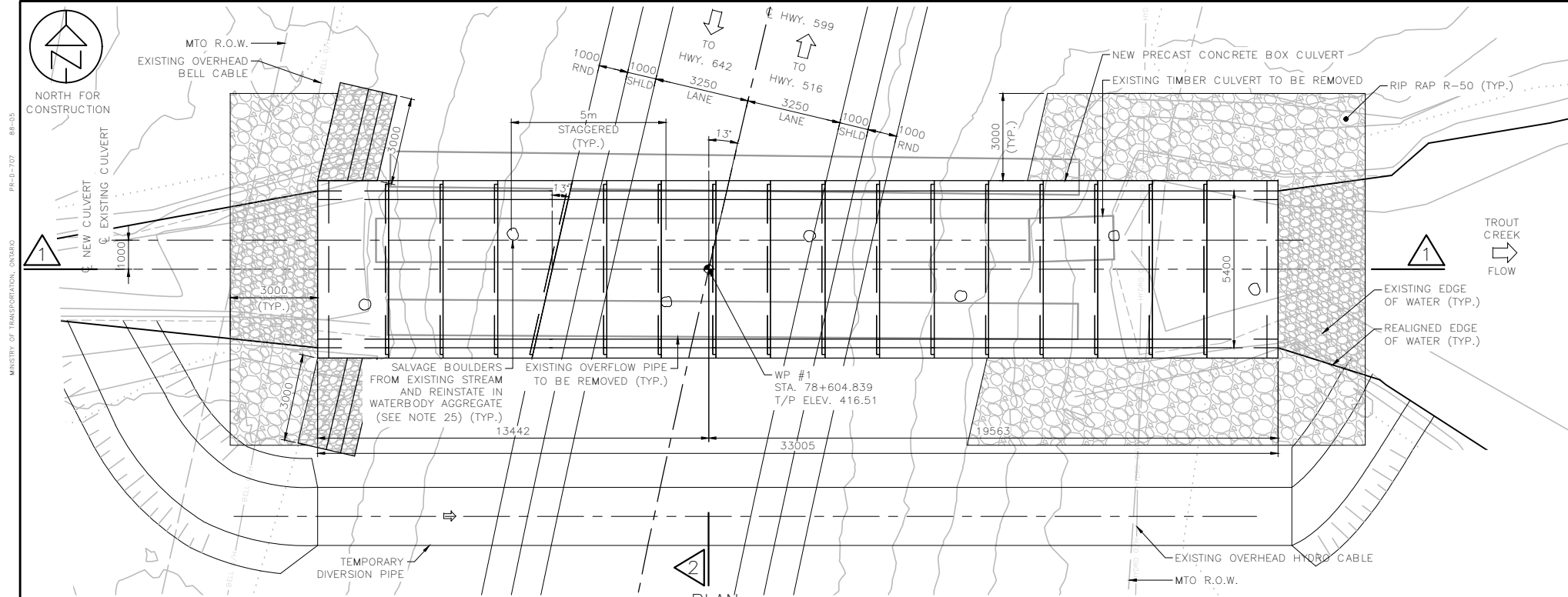
\*\* Fill in the preconstruction survey distance as recommended by the foundation engineer.

**N/A**

**WARRANT:** Include with this item **only** on the recommendation of a foundation engineer.

**CUSTODIAN:** Tony Sangiuliano, MERO - Foundation Group.

APPENDIX L  
Preliminary GA Drawings



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

CONT No. XXXX-XX-XX	SHEET X
WP No. 6229-20-01	
TROUT CREEK CULVERT REPLACEMENT OPTION 1-PRECAST CONCRETE BOX GENERAL ARRANGEMENT	
HATCH	

- GENERAL NOTES
- LIFTING ANCHORS TO BE DESIGNED AND PLACED BY PRECAST MANUFACTURER.
  - FACTORED GEOTECHNICAL AXIAL RESISTANCE AT ULS: 650kPa GEOTECHNICAL RESISTANCE AT SLS FOR 25mm OF SETTLEMENT: 350kPa
  - PRECAST CULVERT MANUFACTURER TO DESIGN CULVERT IN ACCORDANCE WITH CHBDC CAN/CSA S16-19.

CLASS OF CONCRETE  
ALL PRECAST CONCRETE.....45 MPa

CLEAR COVER TO REINFORCEMENT  
CLEAR COVER TO REINFORCING STEEL FOR PRECAST BOTTOM OF BOTTOM SLAB 55 ± 10mm. REMAINDER 50 ± 10mm UNLESS OTHERWISE NOTED.

- REINFORCEMENT
- REINFORCING STEEL TO BE GRADE 400W, UNLESS OTHERWISE SPECIFIED.
  - LAPS NOT INDICATED ON DRAWING SHALL BE CLASS B.
  - STAINLESS STEEL REINFORCING BARS SHALL BE TYPE 316LN OR DULEX 2205 AND HAVE A MINIMUM YIELD STRENGTH OF 500MPa. BAR MARKS WITH PREFIX "S" DENOTES STAINLESS STEEL BARS.
  - BAR HOOKS, WHERE REQUIRED, SHALL BE MINIMUM LENGTH AND STIRRUPS SHALL HAVE MINIMUM HOOKS AS PER MANUFACTURER'S GUIDELINES UNLESS INDICATED OTHERWISE.

- CONSTRUCTION NOTES
- THE CONTRACTOR IS ADVISED NOT TO RELY ON THE WATER LEVEL SHOWN ON DRAWINGS. THE WATER LEVEL IS SUBJECT TO VARIATIONS.
  - INSTALLATION OF PRECAST CONCRETE SEGMENTS SHALL BE CARRIED OUT IN STRICT ACCORDANCE WITH THE MANUFACTURER'S INSTRUCTIONS.
  - THE CONTRACTOR SHALL VERIFY ALL DIMENSIONS OF THE PROPOSED WORK AND ALL DETAILS ON SITE AND REPORT ANY DISCREPANCIES TO THE CONTRACT ADMINISTRATOR BEFORE PROCEEDING WITH THE WORK.
  - CONTRACTOR SHOULD BE AWARE OF THE PRESENCE OF COBBLES AND BOULDERS OR OTHER BURIED OBSTRUCTIONS THAT MAY BE ENCOUNTERED DURING EXCAVATION OR INSTALLATION OF TEMPORARY ROADWAY PROTECTION SYSTEMS AND BE ADVISED TO REFER TO BOREHOLE LOGS.
  - CONTRACTOR IS RESPONSIBLE FOR STABILITY OF BOTH EXISTING AND NEW STRUCTURES AT ALL TIMES THROUGHOUT CONSTRUCTION INCLUDING EXCAVATION, BACKFILL, REMOVALS, INSTALLATIONS, ETC. CONTRACTOR TO DESIGN AND PROVIDE ANY TEMPORARY SUPPORT SYSTEMS FOR EXISTING AND NEW STRUCTURES AT VARIOUS STAGES OF CONSTRUCTION AS REQUIRED TO SUIT THEIR METHOD OF CONSTRUCTION.
  - CONTRACTOR IS FULLY RESPONSIBLE FOR THE DESIGN, CONSTRUCTION METHODS AND PERFORMANCE OF THE TEMPORARY SLOPES, PROTECTION SYSTEM AND ASSOCIATED WORKS.
  - SURFACE WATER CONTROL MEASURES MAY BE REQUIRED. A SUITABLE DEWATERING SCHEME SHALL BE USED. SUBGRADE PREPARATION AND COMPACTION OF BEDDING AND GRANULAR FILL MUST BE CARRIED OUT IN THE DRY.
  - CONTRACTOR IS RESPONSIBLE FOR TEMPORARY DIVERSION OF THE FLOW AROUND THE SITE DURING CONSTRUCTION AS PER THE CONTRACT DOCUMENTS. CONTRACTOR TO DETERMINE THE SIZE OF TEMPORARY DIVERSION PIPES BASED ON MINIMUM 2-YEAR STORM EVENT OF 5.24m<sup>3</sup>/s.
  - CONTRACTOR IS FULLY RESPONSIBLE FOR THE DESIGN OF THE DEWATERING SCHEME TO ENSURE THE SITE IS DEWATERED EFFECTIVELY DURING CONSTRUCTION. THE LOCATION AND LENGTH OF DEWATERING EQUIPMENT IS THE RESPONSIBILITY OF THE CONTRACTOR.
  - DEWATERING TRAP SHOWN FOR INFORMATION PURPOSES ONLY. SHAPE, LOCATION AND ALL DETAILS SUCH AS SIZE, SILT FENCE, ROCK PAD, OUTLETS, ETC. TO BE ADJUSTED BY THE CONTRACTOR AT THE TIME OF CONSTRUCTION TO SUIT EXISTING SITE AND FLOWS.
  - THE DEWATERING SYSTEM MUST BE EFFECTIVE TO LOWER THE GROUNDWATER TABLE AT A MINIMUM OF 0.5m BELOW THE FINAL SUBGRADE LEVEL.
  - CULVERT SUBGRADE TO BE INSPECTED AND APPROVED BY CA FOLLOWING SUB-EXCAVATION TO ENSURE THAT ALL OBSTACLES, LOOSE SOIL, PEAT, AND OTHER UNSUITABLE MATERIALS HAVE BEEN REMOVED AND BACKFILLED WITH COMPACTED GRANULAR MATERIAL IN THE DRY.
  - SUBGRADE SHALL BE PROTECTED AGAINST FREEZING AT ALL TIMES UNTIL COMPLETION OF BACKFILLING. BEDDING MATERIAL SHALL NOT BE PLACED ON A DISTURBED OR FROZEN EARTH GRADE.
  - THE SURFACE TO SUPPORT THE PRECAST UNITS SHALL BE PREPARED IN ACCORDANCE WITH THE CULVERT DESIGN AND SUPPLIER REQUIREMENTS.
  - NO HIGHWAY TRAFFIC ALLOWED OVER CULVERT UNTIL THE MINIMUM DESIGN COVER IS ACHIEVED. COMPLY WITH MANUFACTURER'S SHOP DRAWINGS FOR OTHER RESTRICTIONS WITH REGARDS TO EQUIPMENT DURING CONSTRUCTION LOADING.
  - BACKFILL AND COMPACTION SHALL BE AS PER MANUFACTURER'S INSTRUCTIONS AND AS PER OPSS PROV 501. WHICHEVER IS MORE STRINGENT.
  - BACKFILL SHALL BE PLACED AND COMPACTED SIMULTANEOUSLY BEHIND BOTH SIDES OF THE CULVERT WITH LIFT HEIGHTS NOT EXCEEDING 200mm AND KEEPING THE HEIGHT OF THE BACKFILL APPROXIMATELY THE SAME. AT NO TIME SHALL THE DIFFERENCE IN BACKFILL ELEVATION BE GREATER THAN 500mm AS PER THE CONTRACT DOCUMENTS.
  - ALL DISTURBED EARTH SLOPES SHALL BE REINSTATED TO THEIR ORIGINAL GROUND CONTOUR UPON COMPLETION OF THE WORK (EXCEPT WHERE OTHERWISE NOTED) AND BE TREATED WITH 50mm TOPSOIL IN ACCORDANCE WITH OPSS 802, AND EROSION CONTROL BLANKET, AND SEED, IN ACCORDANCE WITH OPSS 804.
  - ALL DISTURBED CREEK BANKS TO BE REINSTATED WITH MINIMUM 300mm THICK RIP RAP.
  - FOR AREAS OF IMPORTED STREAMBED MATERIAL GRANULAR 'A' SHALL BE WASHED INTO THE VOIDS. GRANULAR 'A' MATERIAL SHALL CONFORM TO THE REQUIREMENTS OF OPSS PROV 1010. CONTRACTOR TO ENSURE THAT VOIDS WITHIN THE ENTIRE DEPTH OF IMPORTED STREAMBED MATERIAL ARE FILLED WITH GRANULAR 'A'.
  - EXCAVATION AND BACKFILLING FOR DIVERSION PIPE INSTALLATION AND REMOVAL SHALL BE COMPLETED IN THE DRY WITH FULL DEWATERING.
  - BACKFILL MATERIAL OUTSIDE THE ENGINEERED FILL ZONE SHALL BE PLACED IN LAYERS NOT EXCEEDING 200mm IN THICKNESS AND EACH LAYER SHALL BE COMPACTED TO 95% SPMD BEFORE A SUBSEQUENT LAYER IS PLACED, ACCORDING TO OPSS 501, UNLESS NOTED OTHERWISE.
  - THE CONTRACTOR SHALL ENSURE THAT THE STREAMBED MATERIAL AT THE INLET AND OUTLET PROVIDES A SMOOTH TRANSITION TO THE EXISTING STREAMBED.
  - IF FULL DEWATERING IS NOT POSSIBLE AND CULVERT MUST BE INSTALLED IN THE WET, CLEAR STONE SHALL BE USED AS BEDDING/BACKFILL BELOW WATER LEVEL. BEDDING/BACKFILL AS NOTED IN SECTION 2 SHALL BE PROVIDED WHEN WORK OCCURS IN THE DRY.
  - THE CONTRACTOR SHALL SALVAGE STREAMBED MATERIAL AND BOULDER (LARGEST DIMENSION OF APPROXIMATELY 300mm) FOR PLACEMENT WITHIN CULVERT AS DETAILED.

APPLICABLE STANDARD DRAWINGS

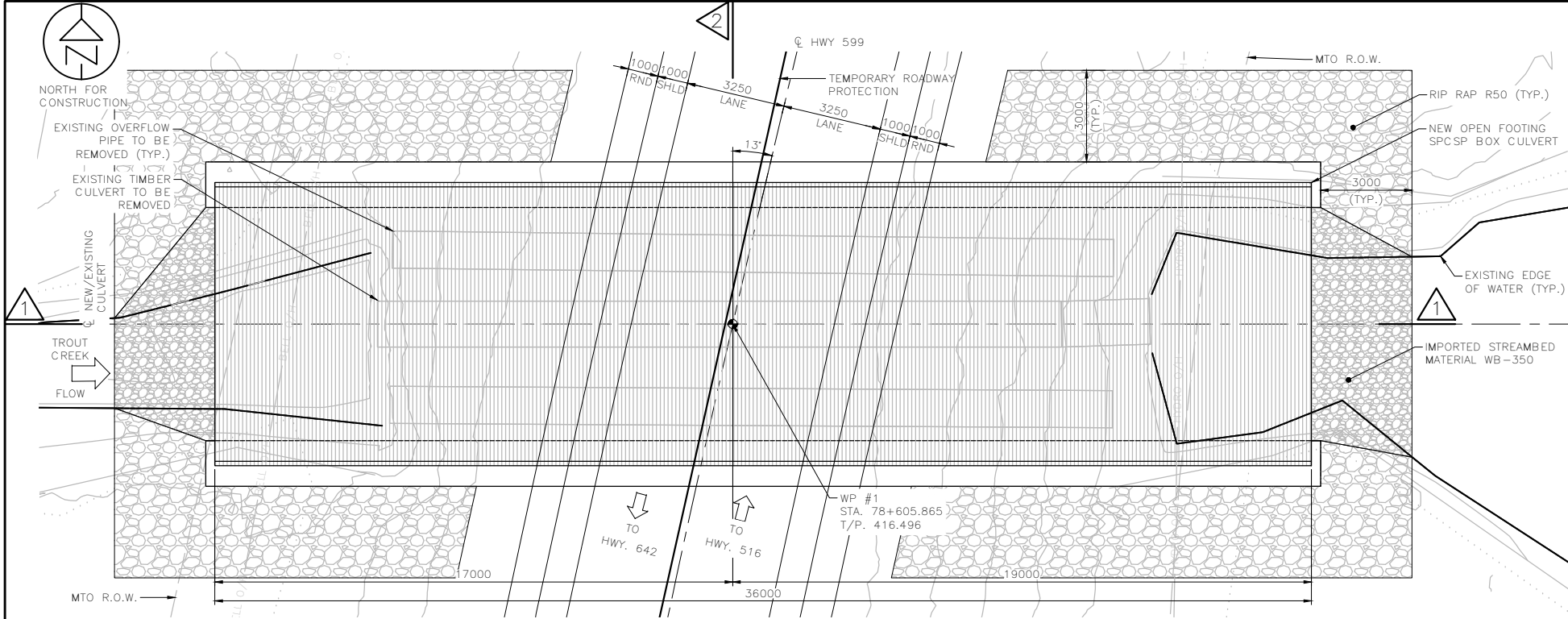
OPSD 219.110	LIGHT DUTY SILT FENCE BARRIER
OPSD 219.240	SEDIMENT TRAP FOR DEWATERING
OPSD 810.010	GENERAL RIP-RAP LAYOUT FOR SEWER AND CULVERT OUTLETS
MTOD 803.021	BEDDING AND BACKFILL FOR PRECAST CONCRETE BOX CULVERTS
MTOD 3941.200	FIGURES IN CONCRETE SITE NUMBER AND DATE LAYOUT

DRAWING NOT TO BE SCALED  
100mm ON ORIGINAL DRAWING

REVISIONS	DATE	REV.	DESCRIPTION
DESIGN	JG	CHK	SA CODE/CAN/CSA S6-19 LOAD CL-625-01
DRAWN	CR	CHK	CP SITE 48W-0322/C/O
			DATE JUNE 2022
			DWG XX

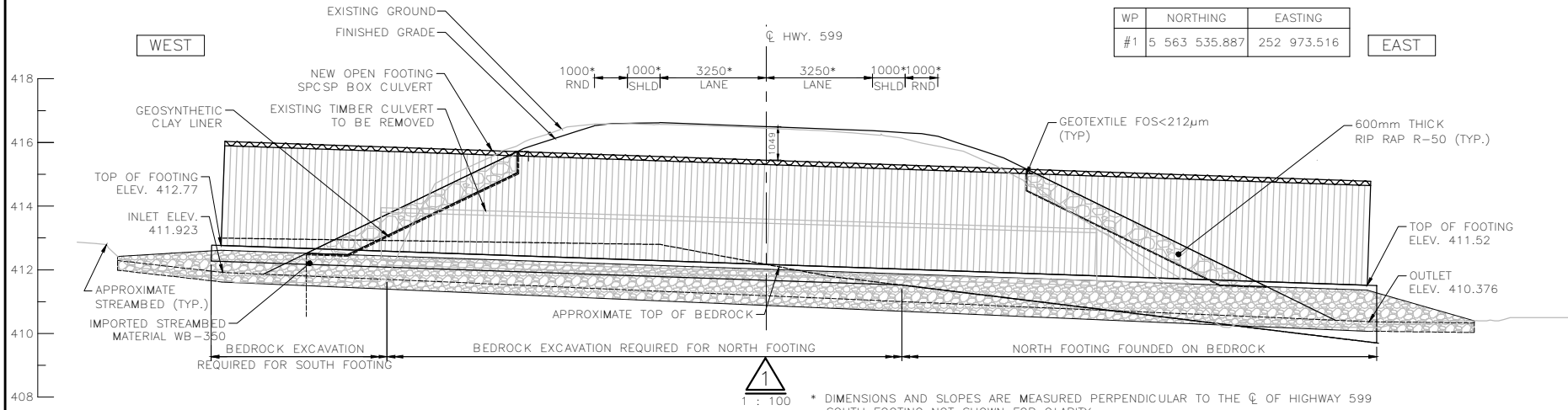


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MINISTRY OF TRANSPORTATION, ONTARIO  
May 09 - 2022 - 12:42pm Login Name: BENB6603  
Drawing Name: C:\drawing\h\hens565603\0106\077\H55706 - CA - Open Footing.dwg



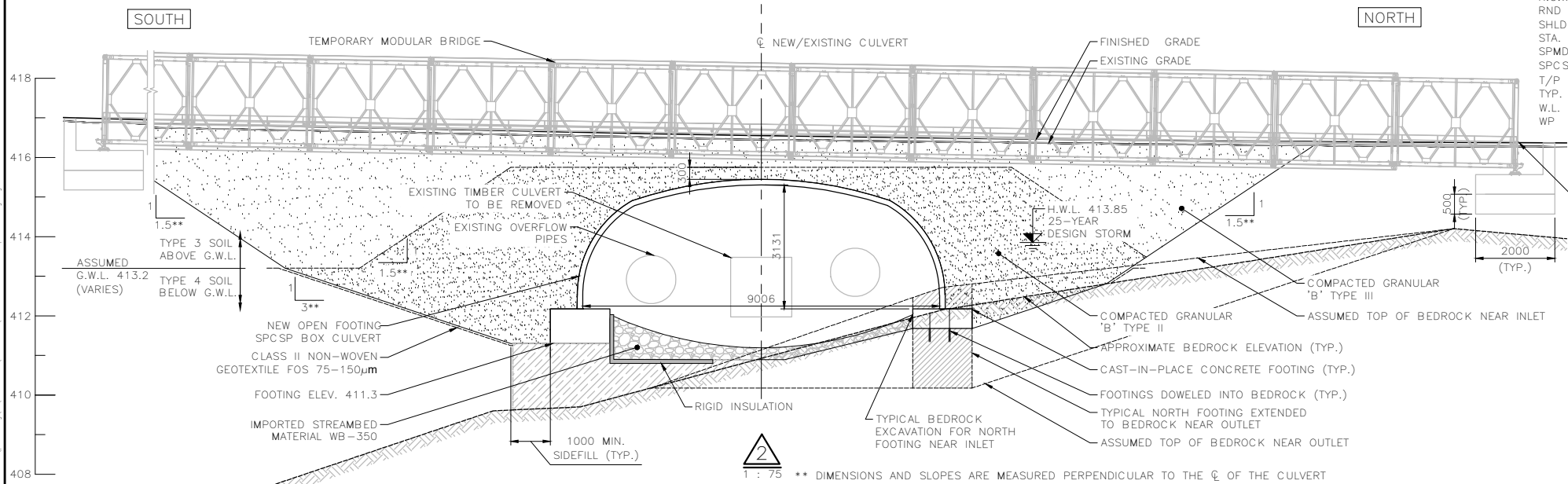
PLAN  
1 : 100

WP	NORTHING	EASTING
#1	5 563 535.887	252 973.516



1  
1 : 100

\* DIMENSIONS AND SLOPES ARE MEASURED PERPENDICULAR TO THE CL OF HIGHWAY 599  
SOUTH FOOTING NOT SHOWN FOR CLARITY.



2  
1 : 75

\*\* DIMENSIONS AND SLOPES ARE MEASURED PERPENDICULAR TO THE CL OF THE CULVERT

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

#### GENERAL NOTES:

- THE CULVERT SHALL BE A STRUCTURAL PLATE CORRUGATED STEEL BOX WITH 7016mm SPAN AND 2363mm RISE (MIN. WALL THICKNESS=4.76mm CORRUGATION PROFILE IS 381mm X 140mm), OR APPROVED EQUIVALENT. APPROVED ALTERNATE SIZE MAY BE 7000mm SPAN AND 2400mm RISE (MIN. WALL THICKNESS = 7.0mm CORRUGATION PROFILE IS 400mm X 150mm). CONTRACTOR IS RESPONSIBLE TO DESIGN, SUPPLY, ASSEMBLE AND ERECT THE NEW CULVERT. CULVERT DESIGN SHALL BE IN ACCORDANCE WITH CHBDC S6-14, LIVE LOAD SHALL BE CL-625-ONT.
- FACTORED GEOTECHNICAL AXIAL RESISTANCE AT ULS 2000kPa FOR FOOTING FOUNDED ON BEDROCK.
- FOR MODULAR BRIDGE FOOTING, THE FACTORED GEOTECHNICAL AXIAL RESISTANCE OF A MINIMUM 1.5m WIDE FOOTING FOUNDED ON A MINIMUM 0.5m THICK COMPACTED GRANULAR PAD IS 150kPa AT ULS AND 100kPa AT SLS WITH UP TO 25mm OF SETTLEMENT.
- THE ELEVATIONS OF BEDROCK SHOWN ARE APPROXIMATE AND MAY VARY. FOR ADDITIONAL DETAILS OF THE SUBSURFACE CONDITIONS, REFER TO DWGS 2 AND 3.
- DIMENSIONS AND DETAILS SHOWN ON DRAWINGS ARE BASED ON A 7016mm SPAN AND 2363mm RISE SPCSP BOX WITH A CORRUGATION PROFILE OF 381mm X 140mm. CULVERT SPAN, RISE, AND LENGTH MAY VARY BASED ON ACTUAL DIMENSION OF SPCSP MANUFACTURER. DETAILS OF CULVERT CONNECTION TO FOOTING MAY VARY BASED ON CULVERT SUPPLIED AND SHALL BE VERIFIED PRIOR TO COMMENCEMENT OF THE CONSTRUCTION.
- PERMANENT SPCSP METAL BOX CULVERT SHALL BE POLYMER LAMINATE COATED AND SHALL BE DESIGNED FOR A 75 YEAR SERVICE LIFE. WATER RESISTIVITY AT THIS SITE IS MEASURED TO BE 11900 Ohm-cm. REFER TO FOUNDATION INVESTIGATION REPORT FOR MORE INFORMATION ON CHEMICAL ANALYSES.

#### CLASS OF CONCRETE:

CAST-IN-PLACE CONCRETE .....35MPa

#### REINFORCING STEEL:

- REINFORCING STEEL SHALL BE GRADE 400W UNLESS OTHERWISE SPECIFIED.
- LAPS NOT INDICATED ON DRAWING SHALL BE CLASS B.
- STAINLESS STEEL SHALL BE TYPE 316LM OR DUPLEX 2205 AND HAVE MINIMUM YIELD STRENGTH OF 500MPa. BAR MARKS WITH PREFIX 'S' DENOTE STAINLESS STEEL BARS.

#### CLEAR COVER TO REINFORCEMENT:

FOOTING.....100 ± 25mm

#### ABBREVIATIONS

BH	BOREHOLE
CL	CENTRELINE
ELEV.	ELEVATION
G.W.L.	GROUND WATER LEVEL
H.W.L.	HIGH WATER LEVEL
HWY	HIGHWAY
INV.	INVERT
MAX	MAXIMUM
MIN	MINIMUM
MTO	MINISTRY OF TRANSPORTATION OF ONTARIO
R.O.W.	RIGHT OF WAY
RND	ROUNDING
SHLD	SHOULDER
STA.	STATION
SPMDD	STANDARD PROCTOR MAXIMUM DRY DENSITY
SPCSP	STRUCTURAL PLATE CORRUGATED STEEL PIPE
T/P	TOP OF PAVEMENT
TYP.	TYPICAL
W.L.	WATER LEVEL
WP	WORKING POINT

CONT No. XXXX-XX-XX  
WP No. XXXX-XX-XX

TROUT CREEK CULVERT REPLACEMENT  
OPTION 3-OPEN FOOTING SPCSP ARCH  
GENERAL ARRANGEMENT

SHEET  
XX

# HATCH

#### CONSTRUCTION NOTES

- THE CONTRACTOR IS ADVISED NOT TO RELY ON THE WATER LEVEL SHOWN ON DRAWINGS. THE WATER LEVEL IS SUBJECT TO VARIATIONS.
- CULVERT ASSEMBLY AND BACKFILLING OPERATIONS SHALL BE CARRIED OUT IN ACCORDANCE WITH THE SUPPLIER'S RECOMMENDATIONS.
- THE CONTRACTOR SHALL VERIFY ALL DIMENSIONS OF THE PROPOSED WORK AND ALL DETAILS ON SITE AND REPORT ANY DISCREPANCIES TO THE CONTRACT ADMINISTRATOR BEFORE PROCEEDING WITH THE WORK.
- CONTRACTOR IS RESPONSIBLE FOR STABILITY OF BOTH EXISTING AND NEW STRUCTURES AT ALL TIMES THROUGHOUT CONSTRUCTION INCLUDING EXCAVATION, BACKFILL, REMOVALS, INSTALLATIONS, ETC. CONTRACTOR TO DESIGN AND PROVIDE ANY TEMPORARY SUPPORT SYSTEMS FOR EXISTING AND NEW STRUCTURES AT VARIOUS STAGES OF CONSTRUCTION AS REQUIRED TO SUIT THEIR METHOD OF CONSTRUCTION.
- CONTRACTOR IS FULLY RESPONSIBLE FOR THE DESIGN, CONSTRUCTION METHODS AND PERFORMANCE OF THE TEMPORARY SLOPES, PROTECTION SYSTEM AND ASSOCIATED WORKS.
- SURFACE WATER CONTROL MEASURES SHALL BE REQUIRED. A SUITABLE DEWATERING SCHEME SHALL BE USED. SUBGRADE PREPARATION AND COMPACTION OF BEDDING AND GRANULAR FILL MUST BE CARRIED OUT IN THE DRY.
- CONTRACTOR IS RESPONSIBLE FOR TEMPORARY DIVERSION OF THE FLOW AROUND THE SITE DURING CONSTRUCTION AS PER THE CONTRACT DOCUMENTS. ALL DETAILS OF THE TEMPORARY DIVERSION PIPE, INCLUDING EXCAVATION LIMITS, ARE SHOWN FOR INFORMATION PURPOSES ONLY.
- CONTRACTOR HAS THE OPTION TO ADJUST SPACING BETWEEN NEW CULVERT AND DIVERSION PIPE TO SUIT THEIR CONSTRUCTION METHOD.
- THE LOCATION AND LENGTH OF DEWATERING EQUIPMENT IS THE RESPONSIBILITY OF THE CONTRACTOR.
- SUBGRADE SHALL BE PROTECTED AGAINST FREEZING AT ALL TIMES UNTIL COMPLETION OF BACKFILLING. BEDDING MATERIAL AND FOOTING SHALL NOT BE PLACED ON A DISTURBED OR FROZEN EARTH GRADE.
- NO HIGHWAY TRAFFIC SHALL BE ALLOWED OVER CULVERT UNTIL THE MINIMUM DESIGN COVER IS ACHIEVED. CONTRACTOR SHALL COMPLY WITH MANUFACTURER'S SHOP DRAWINGS FOR OTHER RESTRICTIONS WITH REGARDS TO EQUIPMENT DURING CONSTRUCTION LOADING.
- BACKFILL AND COMPACTION SHALL BE AS PER MANUFACTURER'S INSTRUCTIONS AND AS PER OPSS PROV 501. WHICHEVER IS MORE STRINGENT.
- BACKFILL SHALL BE PLACED SIMULTANEOUSLY BEHIND BOTH SIDES OF CULVERT WITH LIFT HEIGHTS NOT EXCEEDING 200mm AND KEEPING THE HEIGHT OF THE BACKFILL APPROXIMATELY THE SAME. AT NO TIME SHALL THE DIFFERENCE IN ELEVATION BE GREATER THAN 200mm.
- ALL DISTURBED EARTH SLOPES SHALL BE REINSTATED TO THEIR ORIGINAL GROUND CONTOUR UPON COMPLETION OF THE WORK (EXCEPT WHERE OTHERWISE NOTED) AND BE TREATED WITH TOPSOIL, EROSION CONTROL BLANKET, AND SEED, IN ACCORDANCE WITH OPSS 802, OPSS 804 AND OPSS PROV 182.
- ALL DISTURBED CREEK BANKS TO BE REINSTATED WITH MINIMUM 300mm THICK RIP RAP.
- FOR AREAS OF IMPORTED STREAMBED MATERIAL AND ROCK PROTECTION, GRANULAR 'A' SHALL BE WASHED INTO THE VOIDS. GRANULAR 'A' MATERIAL SHALL CONFORM TO THE REQUIREMENTS OF OPSS 1010. CONTRACTOR TO ENSURE THAT VOIDS WITHIN THE ENTIRE DEPTH OF IMPORTED STREAMBED MATERIAL AND ROCK PROTECTION ARE FILLED WITH GRANULAR 'A'.
- THE CONTRACTOR SHALL CONSULT THE TEMPORARY MODULAR BRIDGE MANUFACTURER AND ENSURE THAT THE DEFLECTION OF THE TEMPORARY MODULAR BRIDGE SOFFIT UNDER OPERATION (CONSIDERING ALL DEAD LOADS AND LIVE LOADS) SHALL NOT CONFLICT WITH THE NEW SPCSP BOX CULVERT. THE CONTRACTOR SHALL ADJUST THE PROFILE AS REQUIRED, AS PER THE REQUIREMENTS DETAILED ELSEWHERE IN THE CONTRACT DOCUMENTS.
- THE CONTRACTOR HAS THE OPTION TO SALVAGE STREAMBED MATERIAL AS NEEDED.
- THE CONTRACTOR SHALL CONSULT THE TEMPORARY MODULAR BRIDGE MANUFACTURER AND ENSURE THAT THE DEFLECTION OF THE TEMPORARY MODULAR BRIDGE SOFFIT UNDER OPERATION (CONSIDERING ALL DEAD LOADS AND LIVE LOADS) SHALL NOT CONFLICT WITH THE NEW SPCSP ARCH CULVERT. THE CONTRACTOR SHALL ADJUST THE PROFILE AS REQUIRED, AS PER THE REQUIREMENTS DETAILED ELSEWHERE IN THE CONTRACT DOCUMENTS.

REVISIONS	DATE	REV.	DESCRIPTION
DESIGN	XX	CHK	XX
DRAWN	XX	CHK	XX

DRAWING NOT TO BE SCALED  
100mm ON ORIGINAL DRAWING



