



## Foundation Investigation and Design Report

*Highway 638 Triple Cell CSP Culvert Replacement*

Assignment No. 5018-E-0012

Work Item No. 11 - Site 2

Latitude: 46.453707; Longitude: -83.900064

GWP 5138-19-00

Geocres No. 41J-131

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*Ministry of Transportation Ontario  
Northeastern Region Geotechnical Section*

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# 1 FOUNDATION INVESTIGATION REPORT

## 1.1 Introduction

This report presents the results of a geotechnical investigation completed by EXP Services Inc. (EXP) for the replacement of the triple cell CSP culvert on Highway 638. The triple cell CSP culvert is located at approximately 13.8 km east of Highway 17, Echo Bay in the Sault Ste. Marie Area (Latitude: 46.453707; Longitude: -83.900064). The work was undertaken under Assignment No. 5018-E-0012, Work Item No. 11 (Site 2). The terms of reference (TOR) were provided by MTO in an email dated March 8, 2021.

The purpose of the investigation is to evaluate the subsurface condition along the existing/new culvert, to permit detailed design for the culvert replacement using either open-cut replacement under a full highway closure or open cut-staged replacement with temporary protection system. The site specific geotechnical investigation consisted of a field investigation including visual inspections, drilling, soil sampling, and laboratory testing.

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

## 1.2 Site Description and Geological Setting

### 1.2.1 Site Description

The triple cell CSP culvert is located on Highway 638 (Sta. 10+860), about 13.8 km east of Highway 17 in Echo Bay Sault Ste. Marie Area, Ontario. At the site, Highway 638 is a two lane roadway, with a speed limit of 60 km/h (unless otherwise posted) and is about 7.6 m wide from edge of pavement to edge of pavement, with 0.5 m and 2.5 m gravel shoulders on north and south sides, respectively. The elevation of highway pavement centerline at the site is about 215.9 m. The roadway embankment above the existing ground is about 2 m high with side slopes of about 1.2H:1V.

Based on the information provided in the TOR and a sheet from Contract Drawing (WP No 7825-91-01), the existing triple cell culvert is between 15.2 m (East and Mid culverts) and 18.3 m long (West culvert) Corrugated Steel Pipe (CSP) culvert with a diameter of 1.2 m and with 1.1 to 1.3± m of cover. The invert of the culverts is at approximate Elev. 213.6 m. Select photographs of the site and existing culvert 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 in March 17, 2021 and during the investigation in April 2021. Highway 638 generally runs in an east to west direction and creek flows northwest to southeast beneath the highway. At the inlet site the creek flows almost parallel the road before enters in to the culvert. At the time of this investigation (April 15, 2021), the approximate top of water at the inlet and outlet was at about Elev. 213.8 m. Gabion wall retaining structures within the fill slope above the culverts were observed at the site at both the inlet and the outlet side (Photograph 1 and 2 in Appendix A). No riprap to protect against scour or erosion was observed on either side of the culvert. A hydro line approximately 5 m from the edge of pavement was observed at the outlet side, affecting the access to a rig at that side of the road. As given in TOR, it is understood that the creek typically experiences high levels throughout summer months and has been observed to overtop and washout onto the roadway at the culvert site. It can be seen on the site that the culvert was blocked with collapsed debris. Those obstructions contribute to upstream flooding during heavy rainfall events. On April 8 2021, during

the fieldwork at the site, flooding of inlet and outlet side was noticed as shown on Photographs 5 and 6, respectively. This flooding was caused by a beaver dam collapse approximately 1 km east of the site.

### 1.2.2 Geological Setting

According the Ministry of Northern Development and Mines, Map 2555 (Quaternary Geology of Ontario, East-Central Sheet, 1991) the surface conditions in the vicinity of the project area consists of bedrock: undifferentiated igneous and metamorphic rock, exposed at surface or covered by a discontinuous, thin layer of drift. According to Map 2543 (Bedrock Geology of Ontario, East-Central Sheet, 1991), the bedrock geology of the site is of Cobalt Gp: conglomerate, wacke, arkose, quartz arenite, argillite under Huronian Supergroup.

## 1.3 Previous Investigations

There are no available previous geotechnical reports at the site in the MTO GEOCRE library; the nearest available report on Highway 638 is about 8 km northeast from this site:

*Geocres No. 41J-62: "Foundation Investigation Report, Proposed Crossing at Leeburn Creek and Highway 638, Township of MacDonald, District of Algoma, District No. 18 (Sault Ste. Marie), Ontario" prepared by Dominion Soil Investigation Inc., August 1978.*

## 1.4 Investigation Procedures

### 1.4.1 Site Investigation and Field Testing

The field investigation was performed between April 8 and April 15, 2021. The field program consisted of drilling six (6) sampled boreholes, numbered BH21-2-1 to BH21-2-6. Two (2) boreholes were strategically located at each end of the existing culverts where cofferdams will be placed. Two (2) boreholes were located on the embankment to provide subsurface information for the culvert replacement and the temporary roadway protection. BH21-2-1 and BH21-2-2 were advanced at an accessible location near inlet. BH21-2-1 was drilled west of the culvert inlet from the top of the road, since the creek flows parallel to the road and there was no space to set a rig at the level of inlet. BH21-2-2 was drilled east of the culvert inlet, and the access to the lower level was provided by clearing trees. BH21-2-3 and BH21-2-4 were on-road boreholes advanced from the top of the embankment. BH21-2-3 was drilled about 2.5 m east of the edge of east culvert footprint, while BH21-2-4 was drilled about 1 m west of the west culvert footprint. BH21-2-5 and BH21-2-6 were drilled at the west and east sides of the outlet, respectively, at the accessible locations considering the presence of the hydro line and area conditions after the flooding. The locations of boreholes drilled during this investigation are shown on Drawing 1 in Appendix B.

All boreholes drilled during this fieldwork were advanced using a track mounted CME 55 drill rig equipped with hollow stem augers and standard soil sampling equipment, operated by a specialist drilling contractor, Landcore Drilling. Dynamic Cone Penetration Tests (DCPT) were also advanced besides BH21-2-1, BH21-2-4 and BH21-2-5. The on-road boreholes BH21-2-3 and BH21-2-4 were advanced to depths of about 12.8 m and 14.9 m below ground surface, respectively. The off-road boreholes (BH21-2-1, BH21-2-2, BH21-2-5 and BH21-2-6) were advanced to a depth of between 9.8 m and 15.2 m.

Traffic control required to close the driving lanes of Highway 638 during the drilling of on-road boreholes was provided by Beacon Lite Limited of Ottawa, Ontario.

The borehole locations (referenced to the MTM NAD83 coordinate system) and their ground surface elevations were surveyed by EXP personnel using a GPS (Magellan 316XS Hand Held) and a basic level and survey rod, respectively, having an accuracy of 2 m in the horizontal directions and 0.1 m in the vertical direction. A temporary benchmark (TBM) set on a nail on the hydro pole 114 located southeast of the existing culvert was used. Based on available survey data, the elevation of temporary benchmark (TBM) was estimated to be Elev. 216.74 m. The TBM location is shown on Drawing 1 in Appendix B.

For the drilling program, soil samples were obtained using a 51 mm outside diameter (O.D.) split-spoon sampler in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586) at intervals ranging from 0.75 m to 1.5 m in depth as shown on the attached borehole logs (Appendix C). The original field (uncorrected) SPT “N” values were recorded on the borehole logs as recommended in the Canadian Foundation Engineering Manual (CFEM, pg. 40) and used to provide an assessment of in-situ relative density of non-cohesive soils. Field vane testing was conducted in cohesive soil to measure the in-situ undrained shear strength of this soil. The standard MTO vane was used in accordance with ASTM D2573-08.

Upon completion of the boreholes, groundwater level measurements were carried out in boreholes in accordance with MTO guidelines. The recorded groundwater levels after completion of drilling boreholes were presented in the borehole 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).

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

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

Table 1.1. Summary of boreholes completed

Borehole No.	Location	Location (MTM NAD 83 Zone 13)		Ground Surface Elevation (m)	Borehole Depth (m)
		Northing	Easting		
21-2-1	Inlet, on-road	5146014.9	312463.3	215.9	15.24
21-2-2	Inlet, off-road	5146019.6	312475.1	214.2	9.75
21-2-3	On-road	5146012.9	312477.7	215.9	12.8
21-2-4	On-road	5146006.4	312472.5	215.9	14.94
21-2-5	Outlet, off-road	5145996.4	312477.8	214.6	9.75
21-2-6	Outlet, off-road	5146004.6	312484.2	215.9	12.8

### 1.4.2 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 samples and particle size distribution for approximately 25% of the collected soil samples. All of the laboratory tests were carried out according to MTO and/or ASTM Standards as appropriate.

## 1.5 Subsurface Conditions

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

A borehole location plan and cross section subsurface profiles are provided in Appendix B. It should be noted that the stratigraphic boundaries indicated on the borehole log and cross section stratigraphic profiles are inferred from semi-continuous sampling, observations of drilling progress and results of Standard Penetration Tests. These boundaries typically represent transitions from one soil type to another and should not be regarded as exact planes of geological change. Furthermore, subsurface conditions may vary between and beyond the borehole locations.

In general, the subsoil condition below the roadway consists of non-cohesive fill underlain by native silt to sandy silt layers followed by silty sand to sand layers. At the inlet side, the subsurface conditions consist of topsoil overlying native silty sand and silt layers. At the outlet side, topsoil and silt fill are underlain by layers of native silt and silty sand. Bedrock was not encountered at the locations of drilling.

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. The lab test results available at the time of writing this draft report are included, while the other results will be added in the next submission of this report.

### 1.5.1 Subsoils

#### 1.5.1.1 Topsoil / Surface Treatment

Topsoil, approximately 25 mm to 100 mm thick, was encountered at the surface of boreholes BH-21-2-2, BH-21-2-4, BH-21-2-5 and a 50mm thick surface treatment layer was encountered in BH-21-2-3.

#### 1.5.1.2 Fill: Sand / Silty Sand / Silt and Sand / Sand & Gravel

Sand/ Silty Sand/ Silt and Sand / Sand & Gravel fill was encountered at the surface in boreholes BH-21-2-1 and BH-21-2-6 and below the topsoil/surface treatment layer in boreholes BH-21-2-2, BH-21-2-3, BH-21-2-4 and BH-21-2-5. The fill layer extended to depths ranging from 2.3 m to 3.8 m below ground surface with elevations ranging from 212.1 m to 212.7 m. Therefore, the explored thicknesses of this layer ranged from 2.3 m 3.8 m.

The composition of this fill material generally consisted of sand with some silt, trace to some gravel with trace organics. The fill was generally grey to brown in color, and moist to wet. The SPT “N” values obtained within this fill material ranged from 0 to 69 blows per 0.3 m penetration, suggesting very loose to very dense in relative density.



Laboratory testing performed on selected samples up to date consisted of nineteen (19) moisture content tests and four (4) grain size distribution tests. The test results are as follows:

**Moisture Content:**

- 1% to 82%

**Grain Size Distribution:**

- 25% to 49% gravel;
- 50% to 68% sand; and
- 1% to 8% silt and clay

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

### **1.5.1.3 Silt**

Native silt was encountered below the fill and native sandy silt/silty sand layers in boreholes BH-21-2-1 to BH-21-2-6. The silt extended from depths ranging from 2.3 m to 4.6 m below ground surface (Elev. 211.3 m to 212.3 m) to depths ranging 9.8 m to 15.2 m (Elev. 204.5 m to 200.7 m). Boreholes BH-21-2-1, BH-21-2-2 and 21-2-3 were terminated within this layer. The explored thicknesses of these layers ranged from about 5.3 m to 9 m. Dynamic Cone Penetration Test (DCPT) was advanced beyond silt sampling in borehole BH-21-2-1.

The composition of this layer is silt with trace sand and trace to some clay. The material is grey in color, and moist to wet. The SPT “N” values obtained within this layer ranged from 0 blows to 17 per 0.3 m penetration, suggesting very loose to compact, but mostly loose in relative density. The results of DCPT showed that the density of this soil layer increases below Elev. 202.5 m.

Laboratory testing performed on selected samples up to date consisted of thirty (32) moisture content tests, and six (6) grain size distribution tests. Additional lab tests are ongoing and will be included in the next submission. The results of performed tests are as follows:

**Moisture Content:**

- 20% to 30%

**Grain Size Distribution:**

- 0% gravel;
- 3% to 9% sand;
- 85% to 89% silt; and
- 5% to 10% clay

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

#### **1.5.1.4 Sandy Silt/ Silty Sand**

Native sandy silt/ silty sand was encountered below the topsoil in borehole BH-21-2-2, below fill and native silt in boreholes 21-2-4, 21-2-5 and 21-2-6. The sandy silt/ silty sand layer extended to depths ranging from 0.1 m below the ground surface in borehole BH-21-2-2 (Elev. 214.1 m) and depths ranging from 2.3 m to 9.1 m below ground surface with a layer of silt encountered in between the remaining boreholes at elevations ranging from 206.8 m to 213.6 m. Boreholes BH-21-2-4, BH-21-2-5 and 21-2-6 were terminated within this layer at the depth between 9.8 m and 14.9 m (Elev. 204.9 m and 201.0 m). Dynamic Cone Penetration Test (DCPT) was advanced beyond silty sand sampling in boreholes BH-21-2-4 and from the ground surface in BH-21-2-5.

The composition of this layer is sand and silt with trace gravel and trace to some clay. The material is grey in color, and wet. The SPT “N” values obtained within this layer ranged from 0 blows to 18 per 0.3 m penetration, suggesting very loose to compact, but mostly loose in relative density. The results of DCPT in BH21-2-4 showed that the density of this soil layer increases below Elev. 203.5 m

Laboratory testing performed on selected samples up to date consisted of thirteen (13) moisture content tests, and one (1) grain size distribution test. Additional lab tests are ongoing and will be included in the next submission. The results of performed tests are as follows:

Moisture Content:

- 19% to 82%

Grain Size Distribution:

- 2% gravel;
- 35% sand;
- 50% silt; and
- 13% clay

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

#### **1.5.2 Groundwater and Surface Water Conditions**

The groundwater levels in boreholes were observed during and upon completion of their drilling during EXP’s investigation in April 2021. The groundwater levels encountered in the boreholes are shown on the borehole logs and presented below in Table 1.2.

Table 1.2. Groundwater levels

Borehole	Date Completed	Date Measured	Ground Surface Elevation <sup>1</sup>	Depth to Water <sup>2</sup>	Groundwater Elevation (m)
BH-21-2-1	4/12/2021	4/12/2021	215.9	Dry in open hole	
BH-21-2-2	4/13/2021	4/13/2021	214.2	0.6	213.6
BH-21-2-3	4/13/2021	4/13/2021	215.9	Dry in open hole	
BH-21-2-4	4/8/2021	4/8/2021	215.9	3.3	212.6
BH-21-2-5	4/15/2021	4/15/2021	214.6	0.9	213.7
BH-21-2-6	4/9/2021	4/9/2021	215.9	2.1	213.8

## Notes:

1. The ground surface elevations were referenced to a temporary benchmark (TBM), a nail on the hydro pole 114 located southeast of the existing culvert, with an elevation of 216.74 m.
2. Depths are relative to ground surface.

The measured elevations of the top of creek water at the existing triple cell CSP culvert location on April 15, 2021 were about Elev. 213.8 m at the outlet and inlet sides. As noted before, flooding on both sides of the culvert was observed on April 8, 2021. The water level was above the crown of the culvert (above Elev. 214.4 m). As informed, the flooding was caused by a beaver dam collapse approximately 1 km east of the site.

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.

### 1.5.3 Chemical Analysis

Two soil sample was selected for chemical analysis during the current investigations performed by EXP. The soil sample collected by EXP was tested at a CALA-certified and accredited laboratory. The results of the corrosion potential chemical analysis testing including sulfide, chloride, sulfate, pH, electrical conductivity, resistivity and redox potential are included in Appendix D and summarized in Table 1.3.

Table 1.3. Summary of chemical analysis results

Borehole ID	Sample	Depth (m)	Chloride (ppm)	Sulphate (ppm)	pH	Electrical Conductivity (mS/cm)	Resistivity (ohm-cm)	Redox Potential (mV)
21-2-3	SS3	1.5 – 2.1	47	7	7.54	0.122	8200	268 - 283
21-2-3	SS4	2.3 - 2.9	36	6	6.92	0.122	8200	242 - 248

## 2 ENGINEERING DISCUSSION & RECOMMENDATIONS

### 2.1 General

This section of the report provides geotechnical design recommendations for replacement of the triple cell CSP culvert on Highway 638 (Sta. 10+860), about 13.8 km East of Highway 17, Echo Bay in the Sault Ste. Marie Area, Algoma District, the Ministry of Transportation (MTO) Northeastern Region. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the current investigation at the site performed by EXP dated April 2021. 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 638 at Sta. 10+860 is a triple cell culvert. The cells are about 1.2 m diameter and 15.2 m (East and Mid culverts) and 18.3 m (West culvert) long Corrugated Steel Pipe (CSP) structures. The CSP culverts are positioned in a northwest to southeast direction having alignments at angles of approximately 34° (East and Mid culvert) and 46° (West culvert) to the highway central line. The top elevation of the roadway at the site is approximately Elev. 215.9 m, while the invert of the pipes was measured to be at approximate Elev. 213.6 m. The fill cover above the culvert crown is approximately 1.1 to 1.3± m thick. The embankment above the creek bed is approximately 2 m high having the side slopes of approximately 1.2H:1V. There are existing gabion wall retaining structures within the fill slope above the culverts at both the inlet and the outlet. The creek water at the culvert inlet and outlet was approximately at Elev. 213.8 m at the time of investigation. It is understood that the creek typically experiences high levels throughout summer months and has been observed to overtop and washout onto the roadway at the culvert site. It can be seen on the site that the culvert was blocked with collapsed debris. Those obstructions contribute to upstream flooding during heavy rainfall events.

As proposed in the TOR, the existing triple cell CSP culvert will be replaced with a similar CSP culvert or a concrete box culvert. It is understood that the new culvert will be installed at the same location of the existing culvert having the same or similar elevation of the invert at inlet and outlet sides (Elev. 213.7 m and 213.6 m, respectively). It is also proposed that the new embankment will be constructed with no grade change nor widening at the culvert location (top elevation of Elev. 215.9 m). It is understood that the open-cut replacement with full highway closure and the staged open-cut replacement with a properly designed temporary roadway protection system are options considered by MTO. MTO also requested to consider the design for a concrete box option with an assumed breadth of 3 m. Further, it is understood that the construction site has to be protected by a temporary dewatering system (i.e. cofferdam) designed by the Contractor.

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)*, *Guideline for MTO Foundation Engineering Services, Version 02 (October 2020)* and generally accepted good practice. Pertinent construction issues from a geotechnical standpoint are examined in general accordance with the Terms of Reference provided to us in the MTO email dated March 8, 2021. The assessment involved review of options for replacement of the existing culvert along the proposed alignment using a open-cut replacement method with full highway closure or staged open-cut replacement method with a properly designed temporary roadway protection system. The protection of construction site by cofferdams is also addressed.

## 2.2 Expected Ground Conditions

The following ground conditions along the proposed culvert alignment are evident from the current investigation:

- a) Highway 638 is a two lane, east/west roadway (~ 7.6 m wide) with approximately 0.5 m and 2.5 m wide gravel shoulders at the north and south side of the culvert location. The highway crosses three 1.2 m diameter CSP structures with approximately 1.1 m to 1.3 m of embankment fill above the crowns. The current elevation of the crest of roadway embankment is about Elev. 215.9 m and side slopes of 1.2H:1V with gabion wall retaining structures within the slope above the pipes at both sides.
- b) Below the pavement structure, the highway embankment consists of gravelly sand fill (~1.5 to 2.1 m thick) followed by silty sand fill (~ 1.5 m thick). The embankment fill is underlain by layers of native soils: loose to very loose silt (~5.3 m to 9 m thick), loose sandy silt (~1.6 m thick) and loose silty sand (~4.1 m thick). Below Elev. 203.5 m the silty sand layer becomes dense. The bedrock was not encountered at the site.
- c) At the inlet, below the topsoil (~0.1 m thick), a layer of native very loose silty sand (~2.2m thick) is underlain by very loose to loose silt (~ 7.5 m thick) to about 9.8 m below the ground surface. At the outlet, below the topsoil (~0.05 m thick), a layer of very loose to loose silt fill (~2.3 m thick) is underlain by native compact to very loose sandy silt to silt layer (~ 6.9 m thick) followed by loose silty sand to sand layer (~2.0 m thick) to about 12.8 m below the ground surface (~ Elev. 203.1 m).
- d) The invert of the new culvert(s) is proposed to be at Elev. 213.7 m and Elev. 213.6 m at the inlet and outlet, respectively. The foundation soil below the culvert is anticipated to be gravelly sand fill/engineered fill followed by native silt/sandy silt/silty sand.
- e) At the time of investigation (April 15, 2021), the top of the creek water was at Elev. 213.8 m . The groundwater table measured in the boreholes drilled at the site was between Elev. 212.6 m and 213.8 m. However, the creek typically experiences high levels throughout summer months and has been observed to overtop and washout onto the roadway at the culvert site (~Elev. 215.9 m).

## 2.3 Structure Foundations

Based on the TOR, it is understood that MTO prefers to consider the following options for the new culvert, (i) corrugated steel pipe (CSP) culvert with a hydraulically appropriate opening size and (ii) concrete box culvert with an assumed breadth of 3 m. Therefore, these options were considered as possible options for the culvert replacement at this site and they are discussed below:

- Corrugated Steel Pipe (CSP) culvert,
- Precast rigid frame concrete box culvert,
- Cast-in-place rigid frame concrete box culvert, and
- Cast-in-place rigid frame open footing concrete culvert supported on shallow foundations.

Based on the subsurface information obtained from the site investigations, the native very loose to compact, mostly very loose to loose silt/sandy silt is not considered suitable for support the culvert; therefore, the native soil below the new culvert needs to be replaced with a 0.5 m thick granular engineered fill.

It is noted that regardless of the option selected, the existing triple cell culvert is to be removed. This will require excavation down to the existing founding elevation for all options (~Elev. 213.0 m). This suggests the need for surface/groundwater control as discussed in Section 2.8 below.

In general, any loose and/or soft soils encountered below the existing embankment should be excavated and replaced with engineered fill. If the depth of excavation to remove unstable soils is excessive, using a geotextile fabric in accordance with OPSS.PROV 1860, Class II Non-Woven (OPSS 1860 II-N), in conjunction with engineered fill can be considered to assist in providing a stable base for the new culvert. Based on previous experience, typically it 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; D~3m) and extend a minimum of 500 mm horizontally on either side of the culvert edge. In this case a 500 mm layer of Granular A or Granular B Type II is recommended. The fabric should be installed a manner to mitigate the migration of fines from adjacent material.

Based on the subsoil condition, Table 2.1 below compares the possible structure options from a foundations design and constructability perspective with their advantages and disadvantages. Although the improved foundation soils can provide adequate support for all options listed in the table, the use of CSP culvert is ranked highest for the criteria evaluated. Therefore, the CSP culvert is recommended as the preferred option from a geotechnical engineering perspective.

Table 2.1 Evaluation of foundation alternatives

Options	Rank	Advantages	Disadvantages	Relative Costs	Risks/ Consequences
Corrugated Steel Pipe (CSP) culvert	1	<ul style="list-style-type: none"> <li>▪ Straightforward construction</li> <li>▪ Reduced construction period, consequently, traffic management and water control period</li> <li>▪ Reduced excavation depth</li> </ul>	<ul style="list-style-type: none"> <li>▪ Requires bedding material</li> <li>▪ Limited design life</li> <li>▪ Potential for corrosion</li> </ul>	Low to medium	<ul style="list-style-type: none"> <li>▪ Risk of unacceptable differential settlements if the entire foundation is not supported on the competent soil</li> <li>▪ Risk of structure segment loss due to corrosion</li> </ul>
Precast rigid frame 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>▪ Reduced excavation depth</li> <li>▪ Can be more readily installed during cold weather conditions</li> </ul>	<ul style="list-style-type: none"> <li>▪ If floor is thin and poorly reinforced, it may heave and cracks</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>▪ Disturbance of natural streambed</li> </ul>	Low	<ul style="list-style-type: none"> <li>▪ Risk of unacceptable differential settlements if the entire foundation is not supported on the competent soil</li> <li>▪ Risk of leaking from joints if not properly installed</li> </ul>

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Options	Rank	Advantages	Disadvantages	Relative Costs	Risks/ Consequences
			<ul style="list-style-type: none"> <li>Possible sediments accumulation in the upstream of the culvert</li> </ul>		
Cast-in-place rigid frame concrete box culvert	3	<ul style="list-style-type: none"> <li>Suitable if site is not appropriate to heavy equipment for installation of precast sections</li> <li>Reduced excavation depth</li> <li>Culvert design can be customized in the field for high stress or load conditions or other site specific requirements</li> </ul>	<ul style="list-style-type: none"> <li>Slower construction process</li> <li>If floor is thin and poorly reinforced, it may heave and crack</li> <li>During high flows, the concrete floor can be undermined</li> <li>Requires concrete curing</li> <li>Disturbance of natural streambed</li> <li>Possible sediments accumulation in the upstream of the culvert</li> <li>Extensive Dewatering is required</li> </ul>	Likely more expensive than Option 2 due to need for extensive dewatering	<ul style="list-style-type: none"> <li>Risk of unacceptable differential settlements if the entire foundation is not supported on the competent soil</li> <li>Risk of disturbance of base during construction</li> </ul>
Cast-in-place rigid frame open footing concrete culvert	4	<ul style="list-style-type: none"> <li>Wider span may consider to maintain existing channel and so allows for natural streambed to remain intact</li> <li>Less accumulation of sediments in the upstream of culvert</li> </ul>	<ul style="list-style-type: none"> <li>Deeper excavation or below water excavation required</li> <li>Dewatering system required</li> <li>Require placement of lean concrete</li> </ul>	Likely more expensive than other options due to need for extensive dewatering	<ul style="list-style-type: none"> <li>Risk of unacceptable differential settlements if the entire footing is not supported on the competent soil</li> <li>Risk of delay in construction due to deeper excavation below water if proper dewatering is not maintained</li> <li>Higher scour risk</li> </ul>

### 2.3.1 Shallow Foundations

#### 2.3.1.1 Geotechnical Resistance

Based on the subsurface stratigraphy encountered at this site and the proposed invert elevation 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 and its commentary. The geotechnical resistances provided in sections below were factored with typical consequence factor of 1.0 at ULS and SLS; typical degree of understanding (factor of 0.5 at ULS and factor of 0.8 at SLS) in accordance with Table 6.1 and 6.2 of the CHBDC S6-19.

Table 2.2 Recommended spread footing design parameters

Culvert Type	Founding Elevation (m)	Footing Size (m)	Founding Soil Type	Factored Geotechnical Resistance at ULS (kPa)	Geotechnical Reaction at SLS <sup>2</sup> (kPa)
CSP culvert, precast or cast-in-place rigid frame concrete box culvert	~213.6	3.0	~ 0.5 m thick Granular 'A' or 'B' Type II <sup>(3)</sup> Pad underlain by loose silt/ sandy silt/silty sand	225	150
Cast-in-place rigid frame open footing concrete culvert	~211.8 <sup>(1)</sup>	1.0	~ 0.5 m thick Granular 'A' or 'B' Type II <sup>(3)</sup> Pad underlain by loose silt/ sandy silt/silty sand	225	150

Notes:

1. Below the frost line. Requires deeper groundwater control
2. For maximum settlement of 25 mm
3. The granular material used for the granular pad shall be Granular 'A' or Granular B Type II conforming to OPSS.PROV 1010 and compacted to 98 % SPMDD

It is assumed that, if any, underlying 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. Given that no grade raise and/or widening is planned, the anticipated maximum total settlements for the new proposed culvert 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.

#### 2.3.1.2 Resistance to Lateral Loads

Resistance to lateral forces/ sliding should be calculated in accordance with Section 6.10.5 of the CHBDC, using the following parameters:



Table 2.3 Recommended parameters for calculation of unfactored horizontal resistance

Interface and Loading Conditions	Parameters
Between Granular A and pre-cast concrete	Coefficient of friction ( $\tan \delta$ )=0.55
Between Granular A and cast-in-place concrete	Coefficient of friction ( $\tan \delta$ )=0.6

The listed values are unfactored; in accordance with the CHBDC, a factor of 0.8 is to be applied in calculating the horizontal resistance.

### 2.3.1.3 Frost Protection

The frost depth in the area of the culvert is estimated to be approximately 1.8 m in accordance with OPSD 3090.100. A minimum 1.8 m of soil cover or equivalent frost protection should be provided using thermal insulation only to the rigid frame open footing culvert option. For the box culvert and CSP culvert the frost protection is not required.

## 2.4 Lateral Earth Pressure

Culvert walls and temporary shoring 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 2.4 lists earth pressure parameters for given materials. These recommendations assume level backfill and ground surface behind the walls.

Table 2.4 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

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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> )
Sand and Gravel Fill	31	0.32	3.16	0.48	21
Gravelly Sand Fill	30	0.33	3.00	0.5	20
Silt and Sand Fill	28	0.36	2.77	0.53	19
Silty Sand Fill	29	0.34	2.90	0.51	19
Silt Fill	24	0.42	2.37	0.59	18
Silt (very loose to compact)	24	0.42	2.37	0.59	18
Sandy Silt (very loose to compact)	25	0.40	2.49	0.57	19
Silty Sand (very loose to compact)	26	0.39	2.57	0.56	20

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.

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.

It is likely that bracing for the temporary support system will be required at a maximum interval of 5 m. For multiple support systems refer to Canadian Foundation Engineering Manual (CFEM) for apparent earth pressure distributions (CFEM, Section 26.10.3, Figure 26.8)

## 2.5 Seismic Potential Consideration

Seismic characterization of the site must be compliant with the Canadian Highway Bridge Design Code (CHBDC - CAN/CSA-S6-19). The potential for seismic loading must be considered for design in accordance with Section 4.4 of the CHBDC with respect to soil conditions encountered at the site. Table 4.1 in CHBDC (see Clause 4.4.3.2) shows site classification for seismic site response based on soil average properties in top 30 m. The borehole information shows the presence of native very loose to compact cohesionless soil; no bedrock encountered at investigated depth. Based on these soil characteristics, the site class for this site is estimated to be Class "E" according to Table 4.1.

From the Natural Resources Canada website, 2015 NBCC seismic hazard values are obtained using the site location coordinates (46.453707°N, 83.900064°W) and the damped reference spectral accelerations for the project site are  $S_a(0.2)=0.064g$ ,  $S_a(0.5)=0.046g$ ,  $S_a(1.0)=0.029g$ ,  $S_a(2.0)=0.015g$  and the reference peak ground acceleration (PGA) is  $0.037g$  ( $g$  =acceleration due to gravity  $-9.81 \text{ m/s}^2$ ). These values are associated with an earthquake having 2 percent probability of exceedance in a 50-year period for Site Class C as shown on the GSC seismic hazard calculation data sheet for this site attached in Appendix G.

The site coefficients used to determine the design spectral acceleration and displacement values are a function of the Site Class and the reference peak ground acceleration ( $PGA_{ref}$ ). For Seismic Site Class E, a  $PGA_{ref}$  with a 2% probability of exceedance in 50 years of  $0.8 * 0.037g$  and a  $F(PGA)$  of 1.81 as per Table 4.8 of the CHBDC (S6-19) can be considered in seismic analyses.

## 2.6 Construction Alternatives

The following construction methods were considered as possible construction alternatives for the culvert replacement at this site:

1. Half-and-half construction using roadway protection to allow excavation as maintaining signalized one-lane of traffic on the existing embankment during construction. The following two options of excavation and replacement using the half-and-half approach were considered:
  - A. Construction using roadway protection and unsupported excavation of cut sides
  - B. Construction using roadway protection and braced cut sides
2. Open-cut replacement under a full highway closure

Both methods considered utilize a cut and cover approach for culvert replacement which allows complete removal of the existing culvert, but it requires disruption of traffic.

The following Table 2.5 summarize advantages and disadvantages of considered construction alternatives. The table also shows assessed risk/consequences and relative costs of the considered methods. Schematic diagrams of considered alternatives are attached in Appendix H.

Table 2.5 Construction alternatives for culvert replacement (see schematic sketches in Appendix H)

Installation Method	Advantages	Disadvantages	Relative Cost	Ranking*
<b>OPTION 1.A</b> Half-and-half Construction with Unsupported Cut Sides (Figure H1.A, Appendix H)	<ul style="list-style-type: none"> <li>Traffic flow maintained at the site during construction</li> <li>Short mobilization time</li> <li>Straight forward construction and construction procedures</li> </ul>	<ul style="list-style-type: none"> <li>Traffic interruption</li> <li>Roadway protection of up to 3 m high required to maintain one lane of traffic</li> <li>Require dewatering to provide safe temporary cut slopes</li> <li>High cost of roadway protection system</li> <li>Large amount of soil to be excavated</li> <li>Need to temporarily control existing creek water</li> <li>Risk of cost overrun and inability to finish job: low to moderate</li> </ul>	More expensive than Option 2; Relatively less expensive and destructive than Option 1.B since the extension of construction/excavation is less	2
<b>OPTION 1.B</b> Half-and- half Construction with Braced or Anchored Cut Sides (Figure H1.B, Appendix H)	<ul style="list-style-type: none"> <li>One or possibly two lanes of traffic flow maintained on existing road (e.g. steel decking, but costly)</li> <li>Global stability of excavation enhanced by narrow geometry</li> <li>Less traffic interruption than with unsupported cut sides approach</li> <li>Temporary decking could be usable over braced cut to allow for excavation of both halves prior to diverting stream and backfilling</li> <li>Cost savings due to limited excavation and backfill</li> </ul>	<ul style="list-style-type: none"> <li>Traffic interruption</li> <li>Roadway protection of up to 3 m high required to maintain one lane of traffic if steel decking is not possible</li> <li>High cost of roadway protection system and/or decking</li> <li>Require side shoring and bracing</li> <li>Bracing (e.g. struts) may interfere with excavation</li> <li>Excavation of material and placement of bracing required in limited space</li> <li>Need to decommission the shoring system</li> <li>Need to temporarily control existing creek water</li> </ul>	More expensive than Option 2 and Option 1.A due to high costs for shoring system and temporary decking (if feasible) to maintain continuous flow of traffic	3

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Installation Method	Advantages	Disadvantages	Relative Cost	Ranking*
	<ul style="list-style-type: none"> <li>Cost saving due to no need for temporary cut slopes and extensive dewatering</li> </ul>	<ul style="list-style-type: none"> <li>Risk of cost overrun and instability to finish job: low to moderate</li> </ul>		
<b>OPTION 2</b> Open Cut Unsupported Excavation under a Full Highway Closure	<ul style="list-style-type: none"> <li>Assessment of the foundation soil</li> <li>Removal of existing CSP culvert</li> <li>Existing embankment fill can be removed and replaced with free draining granular material</li> <li>Adaptable to changing ground conditions</li> </ul>	<ul style="list-style-type: none"> <li>Traffic interruption</li> <li>Large amount of soil to be excavated</li> <li>Risk of cost overrun and inability to finish job: low</li> </ul>	Less expensive than Option 1	1

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

1. OPTION 2: Open-cut replacement under a full highway closure
2. OPTION 1A: Half-and-half construction with unsupported cut sides (Figure H1.A, Appendix H)
3. OPTION 1B: Half-and-half construction with braced or anchored cut sides (Figure H1.B, Appendix H)

The following sections discuss these options in more details.

#### **2.6.1 Option 2: Open-cut Replacement under a Full Highway Closure**

From geotechnical and economical perspectives open cut unsupported excavation appears to be one of the most viable culvert replacement method if the local detour is available and a full highway closer is possible. With this approach, the existing pipes will be removed and the existing foundation soil conditions will be assessed. The existing embankment fill will be excavated in open cut with the slopes of 1H:1V and replaced with free draining granular material. One of the existing culverts could be used for maintenance and diversion of surface water flow during the construction.

#### **2.6.2 Option 1: Half-and-Half Construction**

The half-and-half construction method could be utilized to maintain the flow of the traffic on Highway 638 (see Figures H1.A and H1.B, Appendix H). In this method, one lane of the existing highway will be used to maintain the local traffic while the other half of the existing highway will be excavated and the half of the existing culvert will be exposed. Then the excavated portion of the existing culvert will be removed and replaced with a new culvert, followed by rebuilding of that half of the embankment to grade. Upon completion of the new embankment, the traffic will be moved onto the new fill and the process will be repeated to complete the construction and culvert replacement.

The temporary excavation at the site required to remove half of the existing embankment would be up to 3 m deep. Therefore, temporary shoring such as a sheet pile system will be required as a roadway protection system to allow staging excavation/construction. It will be the Contractor responsibility to design a suitable temporary support system for the MTO review prior to installation. The Contractor is to follow OPSS 902, regarding excavations for structures, and OPSS.PROV 539, regarding temporary protection systems. Recommendations for a temporary roadway protection are given in Section 2.7. Using the half-and-half construction approach, two methods of culvert replacement were considered for this site suitable as discussed below:

- A. Construction using roadway protection and unsupported excavation of cut sides
- B. Construction using roadway protection and braced or anchored cut sides

### **2.6.2.1 Option 1A: Half-and-Half Construction with Unsupported Cut Sides**

This method provides roadway protection parallel to the highway between two lanes, and allows to divert traffic to the one side and undertake open cut with sloping sides at the other side (see Figure H1.A, Appendix H). The roadway protection can take the form of reversible shoring such as a sheet pile system for horizontal support. Once one-lane is completed the supports can be reversed and the other lane constructed in similar fashion. The shoring system would likely be decommissioned in place. Temporary surface water flow control must be developed by the Contractor.

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). The fill which will be excavated at the site during the half-and-half construction with unsupported cut slopes may be classified as a Type 3 soil above the groundwater table in conformance with the OHSA. In accordance with OHSA temporary excavation side slopes for Type 3 soil should not exceed 1H:1V.

Between these two options, Option 1.A could be more economical due to possible cost savings for reversible wall configuration, but it will be more disruptive to the highway embankment than Option 1.B since it needs to excavate a large amount of soil.

### **2.6.2.2 Option 1B: Half-and-Half Construction with Braced or Anchored Cut Sides**

This method provides braced or anchored cut shoring system perpendicular to the highway for face protection and to allow culvert construction (see Figure H1.B, Appendix H). Excavation in this case would have to accommodate the necessary cross-bracing such as struts which in the relatively narrow working area could create difficulties for installation of the new culvert. Installation of tiebacks could be the alternative solution. Temporary decking could possibly be used over the supported cut to allow for excavation of both halves prior backfilling. However, decking would be costly. As well as Option 1.A, temporary surface water flow control must be performed/developed by the Contractor.

Option 1.B will disrupt less of the embankment than Option 1.A, but it might cost more due to the cost of an additional shoring system. However, the global stability of excavation will be enhanced with that shoring system. Both options require decommissioning of shoring system upon completion of the work.

## **2.7 Temporary Roadway Protection**

Temporary roadway protection is anticipated to be a part of the half-and-half construction approach that will be required to maintain on-site traffic during the construction. Roadway protection systems shall 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 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. The 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.

Based on the geotechnical conditions at the site, the shoring system such as steel sheet piling can be considered for design. It should be designed based on the earth pressures coefficients and soil parameters provided in Section 2.4. If a cantilever system is used, an embedded depth of sheet piles should be determined by balancing moments about the pile tip and it could be approximately 2.0 to 2.5 times of its exposed height. Alternatively, a system of rakers can be used for support.

Cobbles and/or boulders were not noted to be contained within the native soil deposits at the site; however, the presence of cobbles was observed within gravelly sand fill in BH21-2-1 and BH21-2-3. Therefore, care has to be taken during installation of sheet piles.

The protection system shall 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, 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.

## 2.8 Site Dewatering

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. A system of sumps and pumps might be required to divert the surface water up and over the existing embankment.

### 2.8.1 Cofferdams

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 triple cell CSP culvert. Two types of cofferdams, i) sheet pile wall and ii) rock fill/earth dam, could be considered.

Based on the geotechnical conditions, suitably designed steel sheet pile walls can be used as cofferdams at this site. Sheet piles perpendicular to the highway at least 3 m into the embankment slopes should be considered to prevent water getting in through the sides. If a cantilever system is used, an embedded depth of sheet piles can be approximately 2.0 to 2.5 times of its exposed height. The proposed sheet pile wall should be at least one meter above the designed HWL defined by the Hydraulic Engineer. The required minimum section modulus and embedment pile length should be designed based on the recommended design parameters. Cobbles and/or boulders were not noted to be contained within the native soil deposits at the site; however, their presence is possible and care has to be taken during installation of sheet piles.

Alternatively, a rock fill/earth cofferdam can be used. This cofferdam will have to be constructed to the same topographic constraints as the sheet pile cofferdam, i.e. at each end of the existing culvert. The size of material suitable for use depends on the erosion potential, stream flow velocity, etc. The rockfill/earth 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.

As mentioned, which cofferdam system is best suited depends on many technical and economic factors. The advantages and disadvantages of both cofferdam systems are summarized in Table 2.6.

Table 2.6 Evaluation of foundation alternatives

Options	Rank	Advantages	Disadvantages	Relative Costs	Risks/ Consequences
Steel sheet piles	1	<ul style="list-style-type: none"> <li>▪ Provides more watertight base</li> <li>▪ Structural elements and seals easier to positively construct</li> <li>▪ Increased safety with appropriate design</li> <li>▪ Easily removed</li> <li>▪ Less seepage</li> <li>▪ Reusable</li> </ul>	<ul style="list-style-type: none"> <li>▪ More costly</li> <li>▪ More likely time consuming for installation</li> <li>▪ May present issues for seepage and/or piping</li> <li>▪ Larger machines required</li> <li>▪ May require bracing</li> <li>▪ May face difficulty driving through soil deposits if cobbles and/or boulders are present</li> <li>▪ May require strengthening toe of sheet pile</li> </ul>	Medium to High	<ul style="list-style-type: none"> <li>▪ May take longer to install</li> <li>▪ Environmental permits</li> </ul>
Rock fill/ Earth	2	<ul style="list-style-type: none"> <li>▪ Less costly</li> <li>▪ Relatively less time consuming for installation</li> <li>▪ Native material can be usable</li> <li>▪ Not affected by presence of cobbles and/or boulders</li> </ul>	<ul style="list-style-type: none"> <li>▪ Require more space for installation</li> <li>▪ Less safe</li> <li>▪ Subjected to wave erosion</li> <li>▪ Less watertight</li> <li>▪ Prone to land shifts, slides and collapse</li> <li>▪ More likely time consuming to remove</li> </ul>	Low to Medium	<ul style="list-style-type: none"> <li>▪ Less stable and safe. May generate 'mud waves'</li> <li>▪ May take longer to remove</li> <li>▪ May require to install clay cut off</li> <li>▪ More dewatering</li> <li>▪ Environmental permits</li> </ul>

Given the soil conditions, topography of the surrounding terrain and available space, the use of a suitably designed steel sheet pile system of sufficiently robust cross-section is recommended for the inlet and outlet of new culvert. 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.

Design and construction specification for the chosen temporary cofferdam system should be prepared in accordance with OPSS 539 (Construction Specification for Temporary Protection Systems) by the Contractor. Pilling should be in accordance with OPSS 903. Cantilevered walls should be designed for the earth pressures shown in Section 2.8.2 and earth pressure diagram shown in the CFEM Figure 26.3. As mentioned before, besides design and construction of the temporary cofferdam system, the Contractor is also responsible for its materials, maintenance, monitoring and removal. The temporary cofferdam shall be fully removed, unless it is specified in the Contract Documents that the cofferdam system may be partially left in place. The method and sequence of removal shall be so that there shall be no damage to the new work, existing work, and facility being protected.

### 2.8.2 Soil Parameters for Cofferdam Design

Temporary cofferdam system, which is temporary retaining structure, should properly designed by a Professional Engineer, experienced in this type of work and employed by the Contractor.

For unbraced design, the triangular pressure relationship outlined below is applicable, as follows:

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

where,

P = earth pressure intensity at depth h, kPa

K = earth pressure coefficient

$\gamma$  = unit weight of retained soil/ water, 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 within the existing ground and within the depth of the structure, and for water in the creek.

For design purposes, the following parameters given in Table 2.7 can be assumed after installation of retaining system.

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Table 2.7 Soil parameters and lateral earth pressure coefficient information required for temporary cofferdam design

Unit	Relevant Boreholes	Approx. Elevation (m)	Materials	Unit Weight $\gamma$ (kN/m <sup>3</sup> )	GWL/ Creek Water Elevation (m)	Angle of Friction $\phi'$ (°)	Effective Stress Properties		
							Coefficient of Lateral Earth Pressure		
Inlet	BH21-2-2	214.1 – 211.9	Silty Sand Very loose	19	213.8	25	0.40	2.49	0.57
		211.9 – 204.5	Silt Very loose to loose	18		24	0.42	2.37	0.59
Outlet	BH21-2-5	214.6 – 212.3	Fill, Silt Very loose to loose	18		24	0.42	2.37	0.59
		212.3 – 207.0	Silt Very loose to loose	18		24	0.42	2.37	0.59
		207.0 – 204.9	Silty Sand Compact to very loose	20		26	0.39	2.57	0.56

**Notes:**

*K<sub>a</sub> = active earth pressure coefficient*

*K<sub>o</sub> = coefficient of earth pressure at rest*

*K<sub>p</sub> = passive earth pressure coefficient*

### 2.8.3 Piping

Given the groundwater conditions and soils present (pervious materials silty sand/sandy silt/silt), piping of the soil is anticipated to be a potential issue at the site due to an unbalanced hydrostatic head causing large upward seepage pressures in the soil at the bottom of the inside cofferdam. Piping should be controlled by lowering the water table outside the cofferdam or driving the sheeting to sufficient depth to mitigate against piping. In case of rock fill/earth cofferdam, piping can be control by installing clay cutoff trench, slurry trench or impervious blanket at upstream of cofferdam.

## 2.9 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 fill and gravelly sand fill as well as very loose native silty sand may be classified as a Type 3 soil above the groundwater table in conformance with the OHSA. The soils below the groundwater table may be classified as a Type 4 soil. It is expected that most of excavations will be above the groundwater levels except those at the invert level. To avoid disturbance of the founding subgrade and 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 2.8.

Temporary excavation side slopes for Type 3 soil should not exceed 1H:1V in accordance with OHSA, while temporary excavation side slopes for Type 4 soils should not exceed 3H:1V where applicable. 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).

## 2.10 Culvert Bedding

OPSDs 802.010, 803.010 and MTOD 803.021 which are included in Appendix F provide the bedding, embedment, cover and backfill standards for the different culvert material. According to these standards the culvert bedding for the 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; D~3m) and extend a minimum of 500 mm horizontally on either side of the culvert edge. As noted in Section 2.3, a 500 mm layer of Granular A or Granular B Type II is recommended at this site to provide a stable base for the new culvert. 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. Based on the existing conditions at the site, Granular B Type II is preferred material for culvert bedding.

Prior to placing any fill material, the exposed native subgrade should be inspected according to OPSS 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.

For the site area, a frost penetration depth of approximately 1.8 m can occur in open, unheated areas without snow cover. At the culvert inlet and outlet, and beneath the proposed culvert, mostly the silt and sand fill and native silty and sandy soils. This material has low to medium frost susceptibility based upon the MTO Frost Classification

guideline of percent particles between 5 to 75  $\mu\text{m}$ . Therefore, non-frost susceptible materials such as sand and gravel might be considered to be provided to the limit of frost penetration beneath the inlet and outlet of the culvert. However, considering that cold air blowing through the culvert during the winter season will freeze soil next to the culvert, a minimum 500 mm thick layer of non-susceptible material should be considered to be placed as a bedding along the entire culvert length.

## **2.11 Culvert Backfill**

The selection and placing of the backfill and cover should be in accordance with OPSS 902, OPSS.PROV 421, OPSS 422 and OPSDs 803.010 and 3101.150 for different culvert material. The backfill should consist of free-draining, non-frost susceptible granular materials confirming 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 1.8 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 (1.8 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. If the frost taper exists at the site it will be reinstalled within the zone of excavation with accordance to OPSD 803.031.

## **2.12 Groundwater Water Control**

The groundwater level at the site was encountered between Elev. 213.8 m and Elev. 212.6 m at the time of investigation, while the excavation to the foundation level has to be carried out to Elev. 213.1 m. Therefore, the groundwater table is above the bottom of excavation at the time of investigation. The soils encountered on the site and within potential excavation depth consist of silty sand/gravelly sand/silt fill or native silty sand/sandy silt/silt. Grain size distribution curves are presented in Appendix D. The estimated range of hydraulic conductivity (k) of these materials is  $10^{-2}$ - $10^{-5}$  m/s.

Considering that the soils within potential excavation depths consist of silty and sandy native/fill soils, it is assessed that these soils are susceptible to disturbance from groundwater and mobilized equipment. Therefore, the groundwater level needs to be controlled to at least 0.5 m below the excavation level to avoid disturbance, and any surface or groundwater seepage should be removed from the excavation prior to the culvert bedding material placement of granular backfill in the dry. In general, where the excavation base is within 0.5 m of the prevailing

groundwater level at the time of construction, it is anticipated that control of seepage can be accomplished by using properly filtered sumps.

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 517 and SP 517F01 (Construction Specification for Dewatering of Pipeline, Utility, and Associated Structure Excavation) and OPSS 518 (Construction Specification for Control of Water from Dewatering Operations). 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.

Dewatering may require water taking permits (i.e. Permit to Take Water PTTW). A PTTW is required for any water taking if the volume exceeds 50,000 L/day. The rate and volume required for dewatering will be dependent on construction methods and staging chosen by the Contractor.

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.

## **2.13 Embankment Reconstruction**

The roadway embankment immediately adjacent to the culvert should be reconstructed in accordance with OPSS 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 to 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.

It is anticipated that the reconstructed embankment will have the the side slopes not steeper than 2H:1V at the location of the culverts. The global stability and settlement of that embankment with existing 1.2H:1V side slopes are also addressed in the sections below.

### **2.13.1 Stability Analysis**

Preliminary slope stability analyses were performed to assess the global stability existing and final embankment configurations in order to check if a minimum Factor of Safety of 1.3 is achieved. The static 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 was employed for computation.

The cross-section and the approximate slopes were developed based on the drawing provided by MTO. 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 non-cohesive fill and native soils. Therefore, effective stress analysis of the slopes were 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 analysis performed is included in Appendix E.

Tabulated below in Table 2.8 are 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 2.8 Soil properties used in slope stability analyses

Soil Type	$\gamma'$ (kN/m <sup>3</sup> )	$\phi'$ (degrees)
Engineered Fill	21	32
Sand and Gravel Fill	21	31
Gravelly Sand Fill	20	30
Silt and Sand Fill	19	28
Silty Sand Fill	19	29
Silt Fill	18	24
Silt (very loose to compact)	18	24
Sandy Silt (very loose to compact)	19	25
Silty Sand (very loose to compact)	20	26

The slope stability analyses were performed for: (i) the existing embankment and (ii) new reconstructed embankment. The results of these analyses for the existing embankment slopes are shown in Figure E1 in Appendix E. As shown, a minimum factor of safety for the existing embankment slopes of 1.2H:1V is around 1.1. For the new reconstructed embankment, the results of these analyses are shown in Figure E2 attached in Appendix E. These results suggest that the new reconstructed embankment having the side slopes of 2H:1V is stable for the soil conditions at the site with a minimum Factor of Safety 1.3.

#### 2.13.2 Embankment Settlement

It is not planned to change the existing embankment grade nor widening at the culvert location. Therefore, there should be negligible additional settlements under the existing embankment. However, a settlement of about 25 mm should be allowed for due to rebound during the construction.

### 2.14 Scour/Erosion Protection

Scour/erosion protection should be provided at the culvert inlet and outlet (including the side 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. The following are some general suggestions, considering that the boreholes indicate that the main soil type consists of silt and sand.

The need for and nature of scour and erosion protection systems must be assessed and where required, must be designed, implemented and remain effective for the design life of the culvert. The potential for scour below foundations must be incorporated into the design.

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. The size of the rip-rap is a function of the creek's hydrology. As a rule of thumb, the thickness of the rip-rap should be a minimum of twice the median particle size, and 300 mm thick as a minimum. 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 which for the subsoil materials should be no steeper than 2H:1V for stability reasons.

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, a cut-off trench constructed with geotextile and rockfill 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 2.15.

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 highway embankment. The installation procedures and the material used for the clay seal should conform to all the requirements stipulated in OPSS 1205, as detailed in Section 2.15.1.

The scour design, nature and extent of the required protection is the responsibility of a qualified Hydraulic Design Engineer experienced in this field. 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.

## **2.15 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) rockfill 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. Piping related to temporary cofferdam is discussed in Section 2.8.1.

### **2.15.1 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



highway 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 the 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.

### 2.15.2 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 H3 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.

## 2.16 Corrosion Potential and Cement Type

Two (2) 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 culvert. The analyses results are summarized in Table 1.2 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 6.92 to 7.54 which is within 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 36 ppm ( $\mu\text{g/g}$ ) and 47 ppm ( $\mu\text{g/g}$ ) 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.

## **2.17 Obstructions during Installation of Temporary Protection Systems**

Cobbles and/or boulders were not noted to be contained within the native soil deposits at the site during site investigation; however, their presence was noted within gravelly sand fill in BH21-2-1 and BH21-2-3. Therefore, care has to be taken since the presence of these obstructions may affect the excavation for culvert replacement and installation of protection system elements including the temporary roadway protection system and temporary dewatering/unwatering systems. It is recommended that a NSSP be included in the Contract Documents to warn the Contractor of the presence of cobbles and/or boulders within the overburden soils. An example of NSSP for obstructions is provided in Appendix I.

### 3 CLOSURE

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

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

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

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

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

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Ministry of Northern Development and Mines, Map 2555. Quaternary Geology of Ontario, East-Central Sheet, 1991

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

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

### **ASTM International:**

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

ASTM D2573-08 Standard Test Method for Field Vane Shear Test in Cohesive Soil

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

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

OPSS 539 Construction Specification for Temporary Protection Systems

OPSS 206 Construction Specification for Grading

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

OPSS 212 Construction Specification for Earth Borrow

OPSS 501 Construction Specification for Compacting

OPSS 517 Construction Specification for Dewatering

OPSS 902 Construction Specification for Excavating and Backfilling – Structures

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

OPSD 3090.100 Foundation Frost Depths for Northern Ontario

OPSD 3101.150 Walls, Abutment, Backfill, Minimum Granular Requirement

OPSD 802.010 Flexible Pipe Embedment and Backfill Earth Excavation

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- 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 Benching of Earth Slopes
- OPSD 810.010 Rip-Rap Treatment for Sewer and Culvert Outlets

**Special Provisions (SP):**

SP 517F01 AMENDMENT TO OPSS 517

**Ontario Water Resources Act:**

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

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

Ontario Regulation 213/91 Construction Projects

## 5 LIMITATIONS AND USE OF REPORT

### BASIS OF REPORT

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

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

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

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

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

### RELIANCE ON INFORMATION PROVIDED

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

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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**

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## Appendix A – Site Photographs





Photograph 1. Inlet side of the culvert (facing southeast), April 2021



Photograph 2. Outlet side of the culvert (facing west), April 15, 2021





Photograph 3. BH21-2-2 at the inlet side of the culvert (facing north), April 13, 2021



Photograph 4. Overhead wire at the outlet side of the culvert (facing west), March 2021





Photograph 5. Flooding at the inlet side of the culvert (facing northwest), April 8, 2021

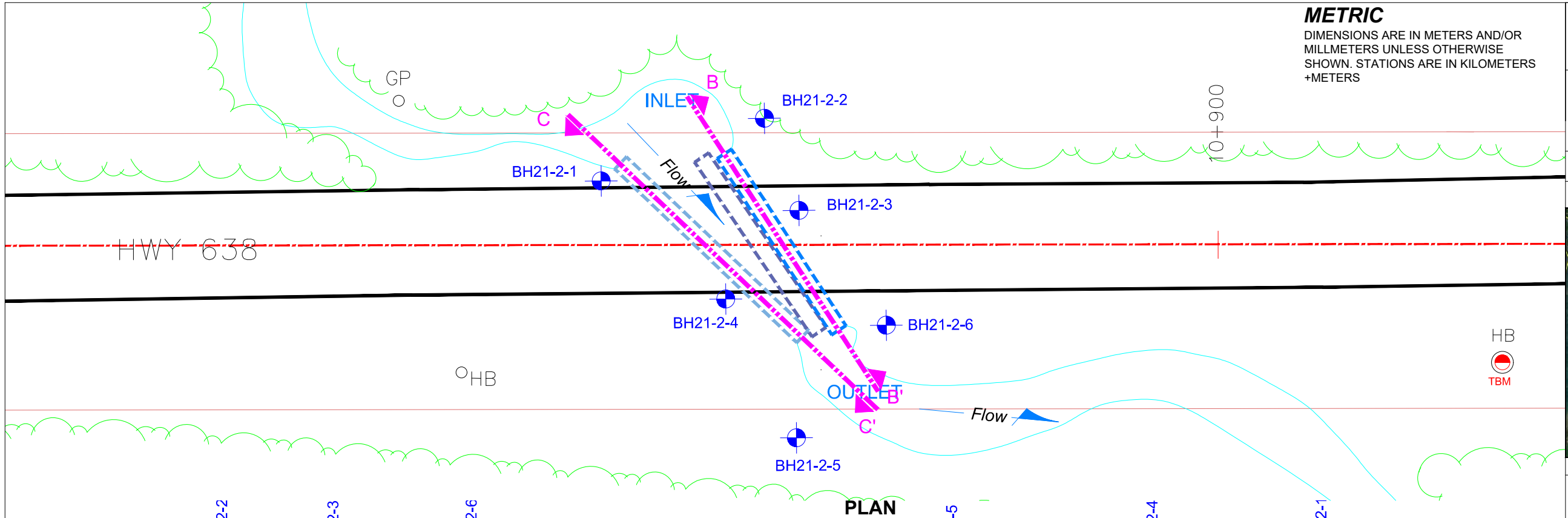


Photograph 6. Flooding at the outlet side of the culvert (facing south), April 8, 2021

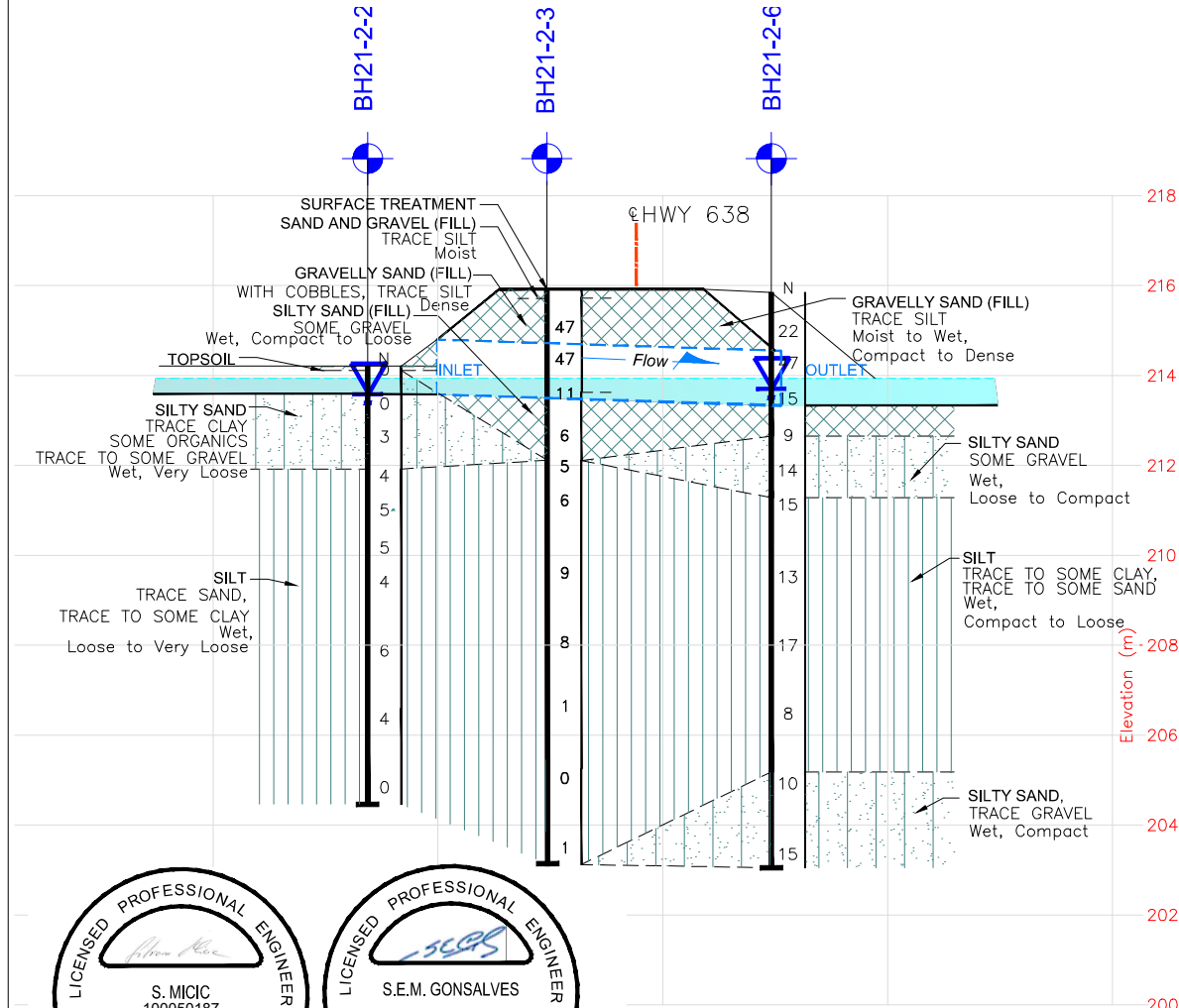
## Appendix B – Drawings



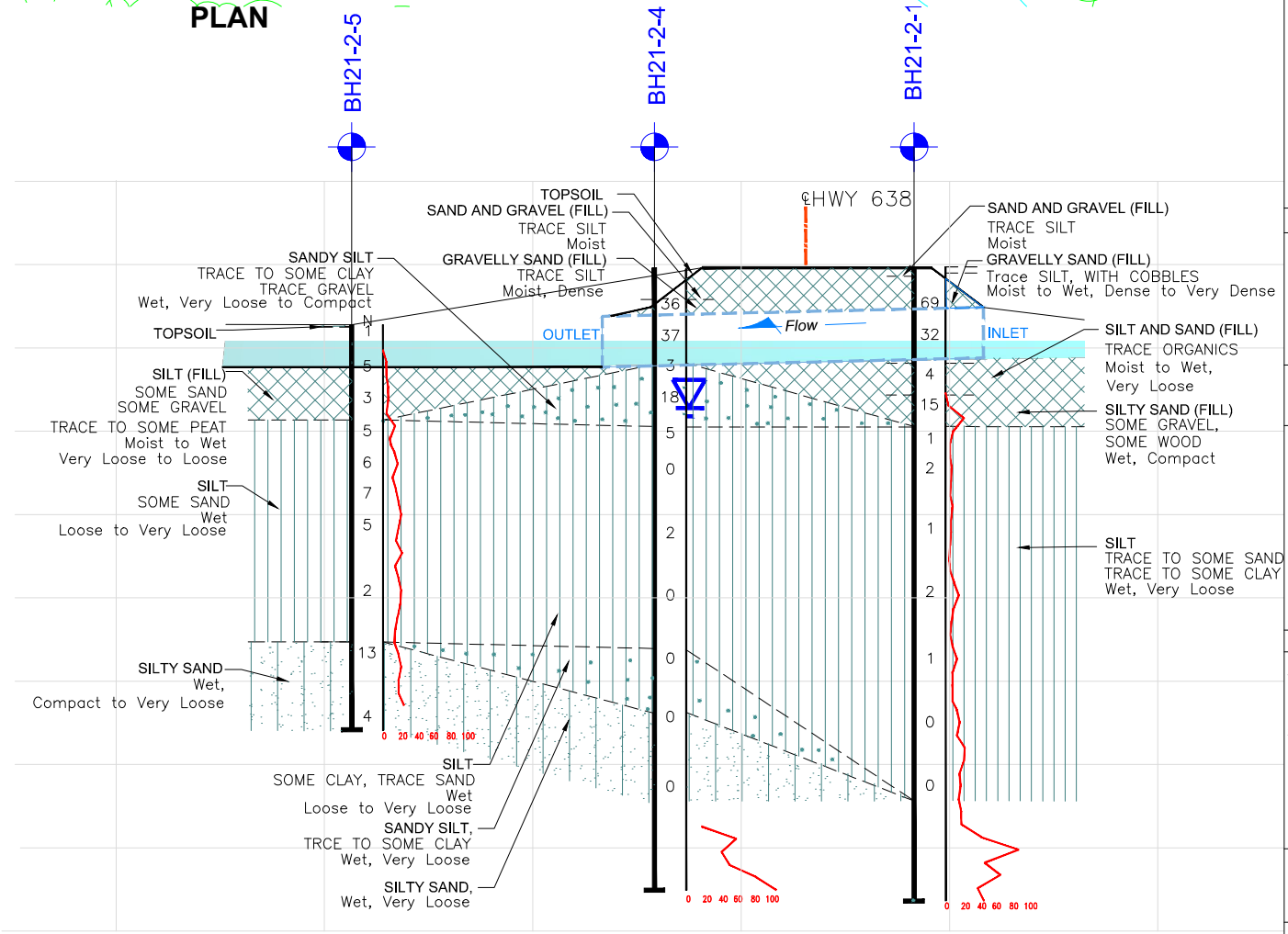




PLAN



SECTION B-B'



SECTION C-C'

**METRIC**

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

Agreement No. 5018-E-0012  
GWP No. 5138-19-00  
Assignment No. 11 (SITE 2)

Triple Cell CSP Culverts Replacement  
Highway 638, Sault Ste. Marie Area  
Echo Bay, Ontario  
(Longitude: 46.453707; Latitude: -83.900064)

BOREHOLE LOCATION PLAN AND SOIL STRATA

EXP Services Inc.

KEY PLAN

LEGEND

Borehole Location

Standard Penetration Test (Blows/0.3 m)

Groundwater Level Measured in open hole

Temporary BenchMark Location (Nail on HP) (El. 216.74 m)

SOIL STRATA SYMBOLS

TOPSOIL

FILL

SURFACE TREATMENT

SAND

SILT

SILTY SAND

SANDY SILT

BH No.	ELEV. (m)	MTM CO-ORDINATES NAD 83 (ZONE ON-13)	
		NORTHING	EASTING
BH21-2-1	215.9	5146014.9	312463.3
BH21-2-2	214.2	5146019.6	312475.1
BH21-2-3	215.9	5146012.9	312477.7
BH21-2-4	215.9	5146006.4	312472.5
BH21-2-5	214.6	5145996.4	312477.8
BH21-2-6	215.9	5146004.6	312484.2

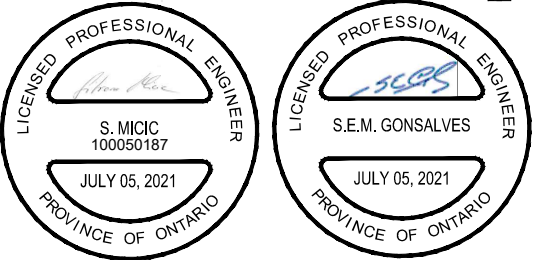
NOTES

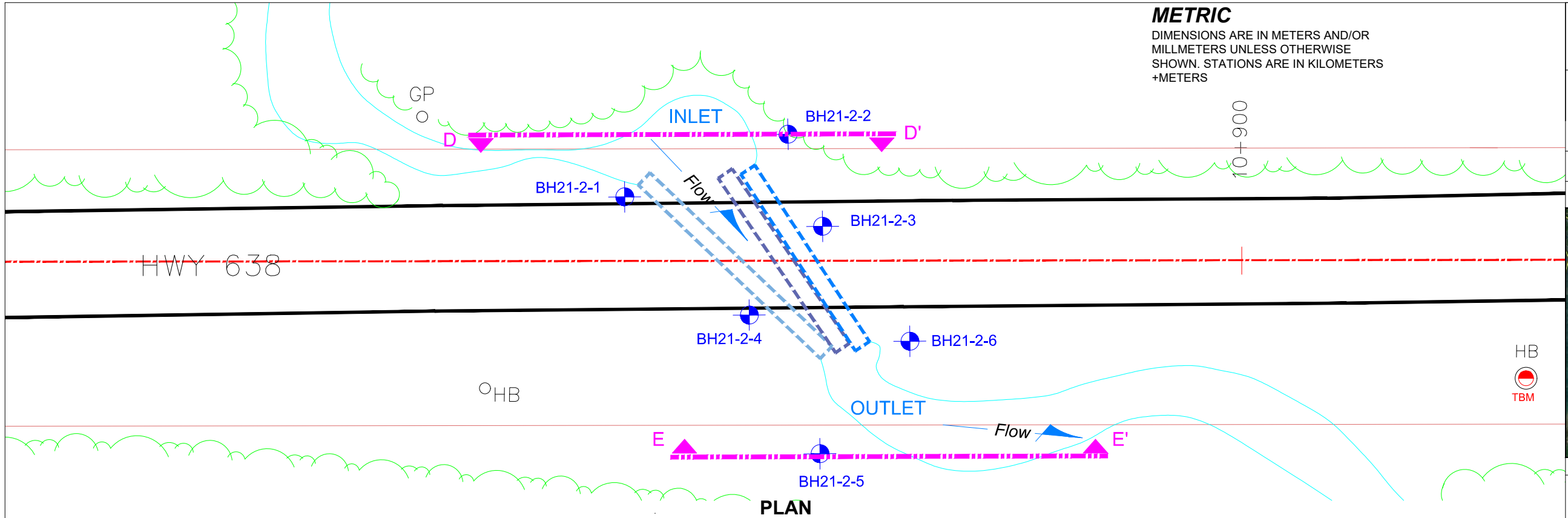
1-This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents

2- 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: HORIZ 0 5 10 m  
VERT 0 2.5 5 m

SM	SUBMISSION FOR MTO REVIEW		
BY	DESCRIPTION		
	GEOCRES NO. 41J-131		
	PROJECT NO. ADM-00257843-K0		
SUBM'D SH	CHECKED SM	DATE	July 5, 2021
DRAWN SH	CHECKED TC	APPROVED SG	DWG. 2





Agreement No. 5018-E-0012  
GWP No. 5138-19-00  
Assignment No. 11 (SITE 2)

Triple Cell CSP Culverts Replacement  
Highway 638, Sault Ste. Marie Area  
Echo Bay, Ontario  
(Longitude: 46.453707; Latitude: -83.900064)

BOREHOLE LOCATION PLAN AND SOIL STRATA

EXP Services Inc.

KEY PLAN

LEGEND

Borehole Location

Standard Penetration Test (Blows/0.3 m)

Groundwater Level Measured in open hole

Temporary BenchMark Location (Nail on HP) (El. 216.74 m)

SOIL STRATA SYMBOLS

TOPSOIL

SAND

SANDY SILT

FILL

SILT

SILT AND SAND (FILL)

SURFACE TREATMENT

SILTY SAND

BH No.	ELEV. (m)	MTM CO-ORDINATES (ZONE ON-13)	
		NORTHING	EASTING
BH21-2-1	215.9	5146014.9	312463.3
BH21-2-2	214.2	5146019.6	312475.1
BH21-2-3	215.9	5146012.9	312477.7
BH21-2-4	215.9	5146006.4	312472.5
BH21-2-5	214.6	5145996.4	312477.8
BH21-2-6	215.9	5146004.6	312484.2

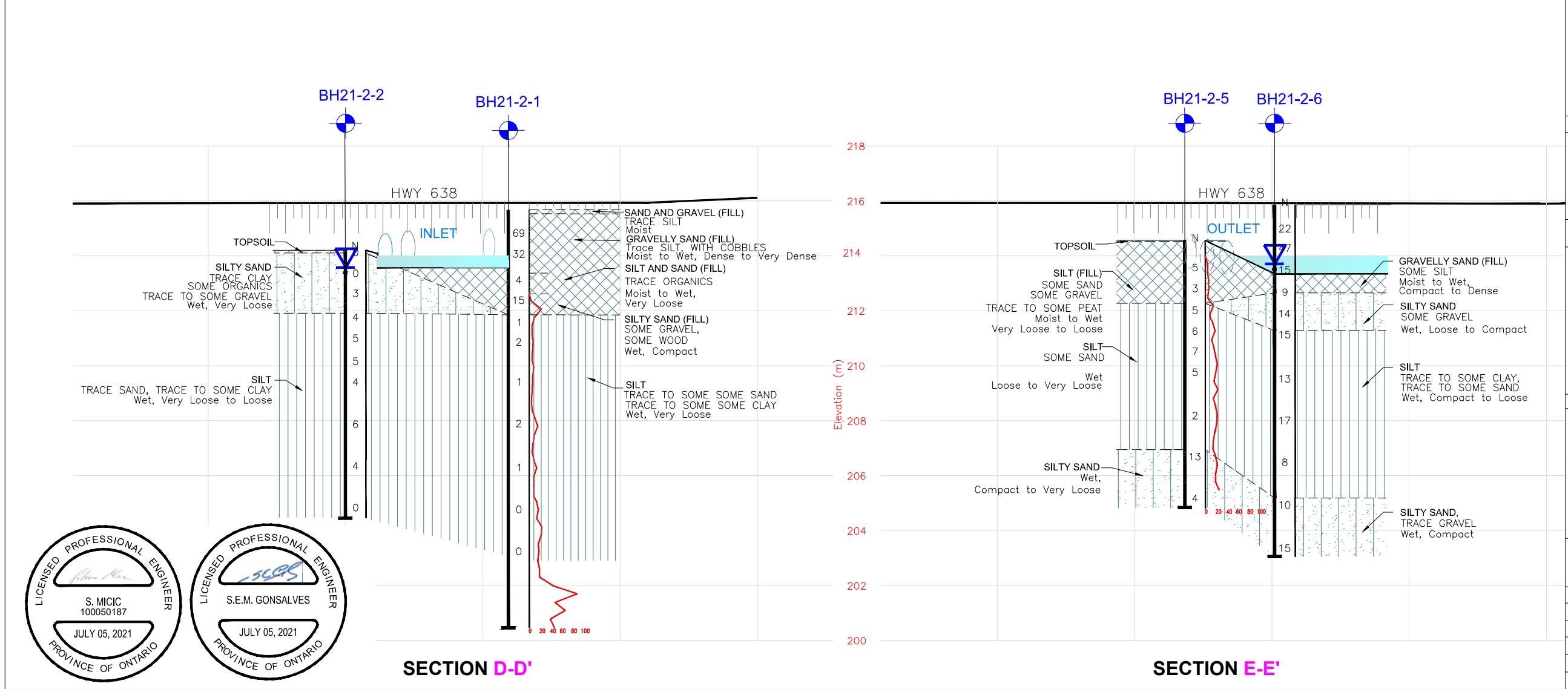
NOTES

1-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

2- 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: HORIZ 0 5 10 m  
VERT 0 2.5 5 m

SM	SUBMISSION FOR MTO REVIEW		
BY	DESCRIPTION		
	GEOCRES NO. 411-131		
	PROJECT NO. ADM-00257843-K0		
SUBM'D SH	CHECKED SM	DATE	July 5, 2021
DRAWN SH	CHECKED TC	APPROVED SG	DWG. 3



## Appendix C – Borehole Logs



# Explanation of Terms Used on Borehole Records

## SOIL DESCRIPTION

Terminology describing common soil genesis:

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

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

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

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

Terminology describing soil structure:

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

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

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

*Fissured:* material breaks along plane of fracture.

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

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

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

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

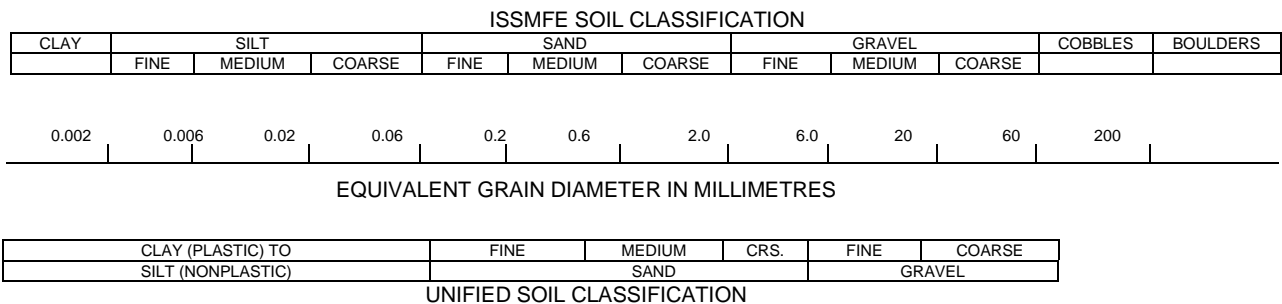
*Seam:* a thin, confined layer of soil having different particle size, texture, or color from materials above and below.

*Homogeneous:* same color and appearance throughout.

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

*Uniformly Graded:* predominantly on grain size.

All soil sample descriptions included in this report follow 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:

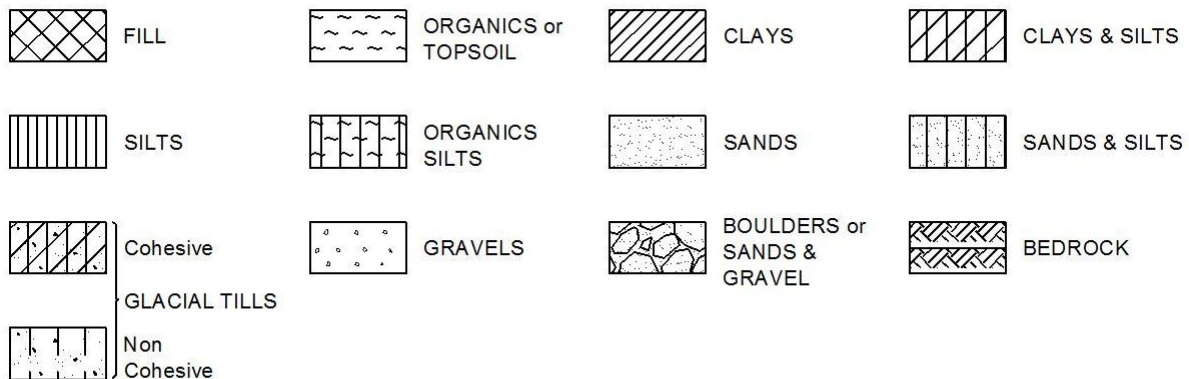
Table c: Consistency of Cohesive Soil

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

Brampton, Ontario

## RECORD OF BOREHOLE No BH-21-2-1

1 OF 2

METRIC

W.P. Agreement No. 5018-E-0012, WO No. 11 LOCATION Triple cell CSP culvert, 312464E, 5146008N NAD83 MTM Zone 13 ORIGINATED BY ST  
 DIST Algoma HWY 638 BOREHOLE TYPE Continuous Flight HSA, NW Casing, NQ Core Barrel COMPILED BY KR  
 DATUM Local (Non-Geodetic) DATE 2021.04.12 - 2021.04.12 LATITUDE 46.453707 LONGITUDE -83.90024 CHECKED BY IM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT  γ  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)		
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL P. PENETROMETER							W <sub>P</sub>	W	W <sub>L</sub>
						20	40	60	80	100	20	40	60				
215.9																	
216.0	FILL, sand and gravel, trace silt, brown, moist		AG1	AUGER													
0.2	FILL, gravelly sand, trace silt, with cobbles, brown, moist, very dense to dense																
			SS2	SS	69												
	dense, moist to wet below ~ 1.5 m depth		SS3	SS	32												
213.6																	
2.3	FILL, silt and sand, trace organics, grey, moist to wet, very loose		SS4	SS	4												
212.9																	
3.1	FILL, silty sand, some gravel, some wood, grey, wet, compact		SS5	SS	15												
212.1																	
3.8	SILT, trace to some sand, trace to some clay, grey, wet, very loose		SS6	SS	1												
			SS7	SS	2												
			SS8	SS	1												
			SS9	SS	2												
			SS10	SS	1												

Continued Next Page

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

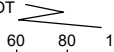
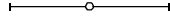
ONTARIO MTO ADM-00257843-K0 - WO 11 HIGHWAY 638 - SITE 2 - ELEVATIONS ADJUSTED.GPJ ONTARIO MTO.GDT 6/3/21

Brampton, Ontario

## 2 OF 2

METRIC

W.P.	Agreement No. 5018-E-0012, WO No. 11		LOCATION	Triple cell CSP culvert, 312464E, 5146008N NAD83 MTM Zone 13			ORIGINATED BY	ST
DIST	Algoma	HWY 638	BOREHOLE TYPE	Continuous Flight HSA, NW Casing, NQ Core Barrel			COMPILED BY	KR
DATUM	Local (Non-Geodetic)		DATE	2021.04.12 - 2021.04.12	LATITUDE	46.453707	LONGITUDE	-83.90024
			CHECKED BY	IM				

SOIL PROFILE					
ELEV DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES NUMBER TYPE "N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE
					DYNAMIC CONE PENETRATION RESISTANCE PLOT  SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL P. PENETROMETER WATER CONTENT (%)  PLASTIC LIMIT Wp NATURAL MOISTURE CONTENT W LIQUID LIMIT Wl UNIT WEIGHT γ kN/m³
203.1 12.8	SILT, trace to some sand, trace to some clay, grey, wet, very loose (continued)  Sampling Terminated at ~ 12.7 m depth		SS11 SS 0		204 203 202 201
200.7 15.2	BOREHOLE TERMINATED AT ~ 15.2 m DEPTH  Dynamic Cone Penetration Test (DCPT) advanced immediately adjacent to borehole.  Groundwater Level: Dry (Upon Completion)				

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

ONTARIO MTO ADM-00257843-K0 - WO 11 HIGHWAY 638 - SITE 2 - ELEVATIONS ADJUSTED.GPJ ONTARIO MTO.GDT 6/3/21

Brampton, Ontario

## RECORD OF BOREHOLE No BH-21-2-2

1 OF 1

METRIC

W.P. Agreement No. 5018-E-0012, WO No. 11 LOCATION Triple cell CSP culvert, 312475E, 5146015N, NAD83 MTM Zone 13 ORIGINATED BY ST  
 DIST Algoma HWY 638 BOREHOLE TYPE Continuous Flight HSA, NW Casing, NQ Core Barrel COMPILED BY KR  
 DATUM Local (Non-Geodetic) DATE 2021.04.13 - 2021.04.13 LATITUDE 46.453765 LONGITUDE -83.900087 CHECKED BY IM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W <sub>P</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)		
								○ UNCONFINED + FIELD VANE										
								● QUICK TRIAXIAL P. PENETROMETER										
20	40	60	80	100	20	40	60	82.22										
214.2							214											
214.0	TOPSOIL (~ 100 mm thick)		AG1	AUGER	0													
0.1	SILTY SAND, trace clay, grey, wet, very loose																	
	some organics, grey and brown below ~ 2.3 m depth		SS2	SS	0		213											
	trace to some gravel below ~ 1.5 m depth		SS3	SS	3								82.22					
211.9							212											
2.3	SILT, trace sand, trace to some clay, grey, wet, very loose to loose		SS4	SS	4													
			SS5	SS	5		211											
			SS6	SS	5		210											
			SS7	SS	4		209								0 3 87 10			
			SS8	SS	6		208											
			SS9	SS	4		207								0 6 87 7			
			SS10	SS	0		206											
204.5							205											
9.8	BOREHOLE TERMINATED AT ~ 9.8 m DEPTH																	
	Groundwater Level: 0.6 m (April 13, 2021)																	

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

ONTARIO MTO ADM-00257843-K0 - WO 11 HIGHWAY 638 - SITE 2 - ELEVATIONS ADJUSTED.GPJ ONTARIO MTO.GDT 6/3/21

METRIC

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



# RECORD OF BOREHOLE No BH-21-2-3

2 OF 2

METRIC

W.P. Agreement No. 5018-E-0012, WO No. 11 LOCATION Triple cell CSP culvert, 312476E, 5146009N, NAD83 MTM Zone 13 ORIGINATED BY ST

DIST Algoma HWY 638 BOREHOLE TYPE Continuous Flight HSA, NW Casing, NQ Core Barrel COMPILED BY KR

DATUM Local (Non-Geodetic) DATE 2021.04.13 - 2021.04.13 LATITUDE 46.453711 LONGITUDE -83.900084 CHECKED BY IM

[illegible]

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

ONTARIO MTO ADM-00257843-K0 - WO 11 HIGHWAY 638 - SITE 2 - ELEVATIONS ADJUSTED .GPJ ONTARIO MTO.GDT 6/3/21

Brampton, Ontario

## RECORD OF BOREHOLE No BH-21-2-4

1 OF 2

METRIC

W.P. Agreement No. 5018-E-0012, WO No. 11 LOCATION Triple cell CSP culvert, 312473E, 5146007N, NAD83 MTM Zone 13 ORIGINATED BY ST  
 DIST Algoma HWY 638 BOREHOLE TYPE Continuous Flight HSA, NW Casing, NQ Core Barrel COMPILED BY KR  
 DATUM Local (Non-Geodetic) DATE 2021.04.08 - 2021.04.08 LATITUDE 46.453692 LONGITUDE -83.900122 CHECKED BY IM

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
			NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
215.9	<b>TOPSOIL</b> (~25 mm thick)													
215.0	<b>FILL</b> , sand and gravel, trace silt, black, moist		AG1	AUGER										49 50 (1)
215.1	<b>FILL</b> , gravelly sand, trace silt, brown, moist, dense		SS2	SS	36		215							
0.8	moist to wet below ~ 1.5 m depth		SS3	SS	37		214							
213.6	<b>SANDY SILT</b> , trace to some clay, trace gravel, grey, wet, very loose to compact		SS4	SS	3		213							2 35 50 13
2.3	trace to some gravel, compact below ~ 3.1 m depth		SS5	SS	18		212							
212.1	<b>SILT</b> , some clay, trace sand, grey, wet, loose to very loose		SS6	SS	5		211							0 4 89 7
3.8			SS7	SS	0		210							
	some sand below ~ 6.1 m depth		SS8	SS	2		209							
							208							
	trace to some clay below ~ 7.62 m depth		SS9	SS	0		207							
206.8	<b>SANDY SILT</b> , trace to some clay, grey, wet, very loose		SS10	SS	0		206							
9.1							205							
205.2	<b>SILTY SAND</b> , grey, wet, very loose													
10.7														

Continued Next Page

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

ONTARIO MTO ADM-00257843-K0 - WO 11 HIGHWAY 638 - SITE 2 - ELEVATIONS ADJUSTED.GPJ ONTARIO MTO.GDT 6/3/21

## 2 OF 2

METRIC

W.P.	Agreement No. 5018-E-0012, WO No. 11		LOCATION	Triple cell CSP culvert, 312473E, 5146007N, NAD83 MTM Zone 13			ORIGINATED BY	ST
DIST	Algoma	HWY 638	BOREHOLE TYPE	Continuous Flight HSA, NW Casing, NQ Core Barrel			COMPILED BY	KR
DATUM	Local (Non-Geodetic)		DATE	2021.04.08 - 2021.04.08	LATITUDE	46.453692	LONGITUDE	-83.900122
			CHECKED BY	IM				

[illegible]

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

ONTARIO MTO ADM-00257843-K0 - WO 11 HIGHWAY 638 - SITE 2 - ELEVATIONS ADJUSTED.GPJ ONTARIO MTO.GDT 6/3/21

Brampton, Ontario

# RECORD OF BOREHOLE No BH-21-2-5

1 OF 1

METRIC

W.P. Agreement No. 5018-E-0012, WO No. 11 LOCATION Triple cell CSP culvert, 312475E, 5146000N, NAD83 MTM Zone 13 ORIGINATED BY ST  
 DIST Algoma HWY 638 BOREHOLE TYPE Continuous Flight HSA, NW Casing, NQ Core Barrel COMPILED BY KR  
 DATUM Local (Non-Geodetic) DATE 2021.04.13 - 2021.04.15 LATITUDE 46.45363 LONGITUDE -83.900093 CHECKED BY IM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT  γ  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)			
								○ UNCONFINED + FIELD VANE							○			
								● QUICK TRIAXIAL							●			
214.6							20	40	60	80	100	20	40	60	GR	SA	SI	CL
214.4	TOPSOIL (~ 50 mm thick)		SS1	SS	1													
	FILL, silt, some sand, some gravel, brown, moist to wet, very loose to loose																	
	some peat below ~ 0.8 m depth		SS2	SS	5													
	trace to some peat below ~ 1.5 m depth		SS3	SS	3													
212.3																		
2.3	SILT, some sand, brown, wet, loose to very loose		SS4	SS	5													
			SS5	SS	6													
			SS6	SS	7													
			SS7	SS	5													
	very loose below ~ 6.1 m depth		SS8	SS	2													
207.0																		
7.6	SILTY SAND, brown, wet, compact to very loose		SS9	SS	13													
	0.5 m diameter boulder at ~ 8.2 m depth																	
	very loose below ~ 9.1 m depth																	
			SS10	SS	4													
204.9																		
9.8	BOREHOLE TERMINATED AT ~ 9.8 m DEPTH																	
	Dynamic Cone Penetration Test (DCPT) advanced immediately adjacent to borehole																	
	Groundwater Level: 0.88 m (April 15, 2021)																	

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

ONTARIO MTO ADM-00257843-K0 - WO 11 HIGHWAY 638 - SITE 2 - ELEVATIONS ADJUSTED.GPJ ONTARIO MTO.GDT 6/3/21



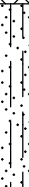


Brampton, Ontario

## RECORD OF BOREHOLE No BH-21-2-6

1 OF 2

METRIC

W.P. Agreement No. 5018-E-0012, WO No. 11 LOCATION Triple cell CSP culvert, 312482E, 5146006N, NAD83 MTM Zone 13 ORIGINATED BY ST  
 DIST Algoma HWY 638 BOREHOLE TYPE Continuous Flight HSA, NW Casing, NQ Core Barrel COMPILED BY KR  
 DATUM Local (Non-Geodetic) DATE 2021.04.09 - 2021.04.09 LATITUDE 46.453686 LONGITUDE -83.900005 CHECKED BY IM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT   NATURAL MOISTURE CONTENT   LIQUID LIMIT			UNIT WEIGHT  γ  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		W <sub>p</sub>	W	W <sub>L</sub>						
								○ UNCONFINED + FIELD VANE										
								● QUICK TRIAXIAL P. PENETROMETER										
		20   40   60   80   100		WATER CONTENT (%)														
215.9 0.0	FILL, gravelly sand, trace silt, brown, moist, compact to dense   moist to wet below ~ 2.3 m depth		AG1	AUGER											GR   SA   SI   CL			
					SS2		SS	22										
					SS3		SS	47										
			SS4	SS	15													
212.7 3.2	SILTY SAND, some gravel, grey, wet, loose to compact		SS5	SS	9													
			SS6	SS	14													
211.3 4.6	SILT, some clay, trace sand, grey, wet, compact to loose   trace to some clay below ~ 7.6 m depth   some sand, trace clay, loose below ~ 9.1 m depth		SS7	SS	15													
					SS8	SS	13											
			SS9	SS	17													
			S10	SS	8													
205.2 10.7	SILTY SAND, grey, wet, compact																	

Continued Next Page

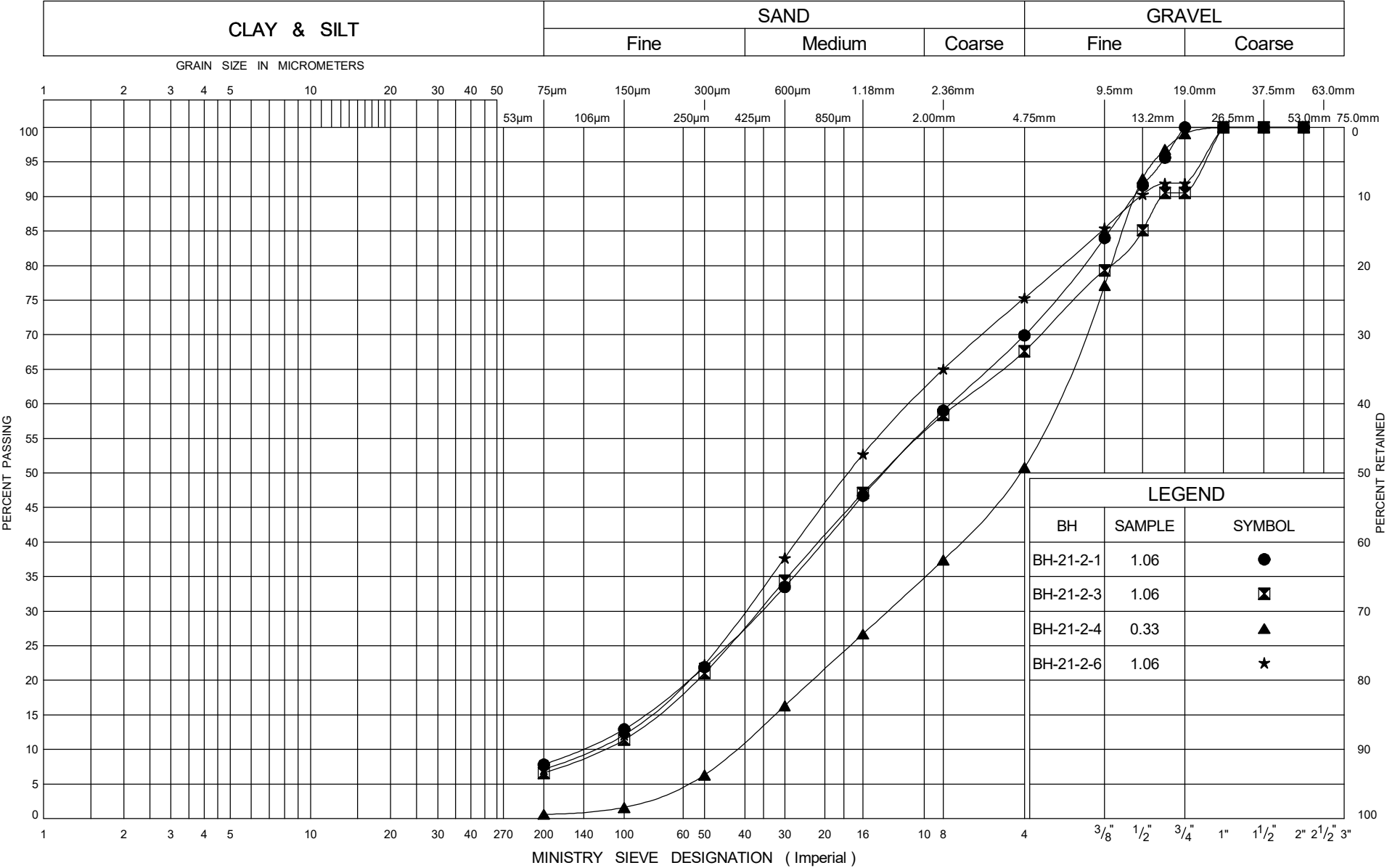
+ 3, X 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

ONTARIO MTO ADM-00257843-K0 - WO 11 HIGHWAY 638 - SITE 2 - ELEVATIONS ADJUSTED.GPJ ONTARIO MTO.GDT 6/3/21

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

## Appendix D – Laboratory Data

UNIFIED SOIL CLASSIFICATION SYSTEM



Ministry of  
Transportation

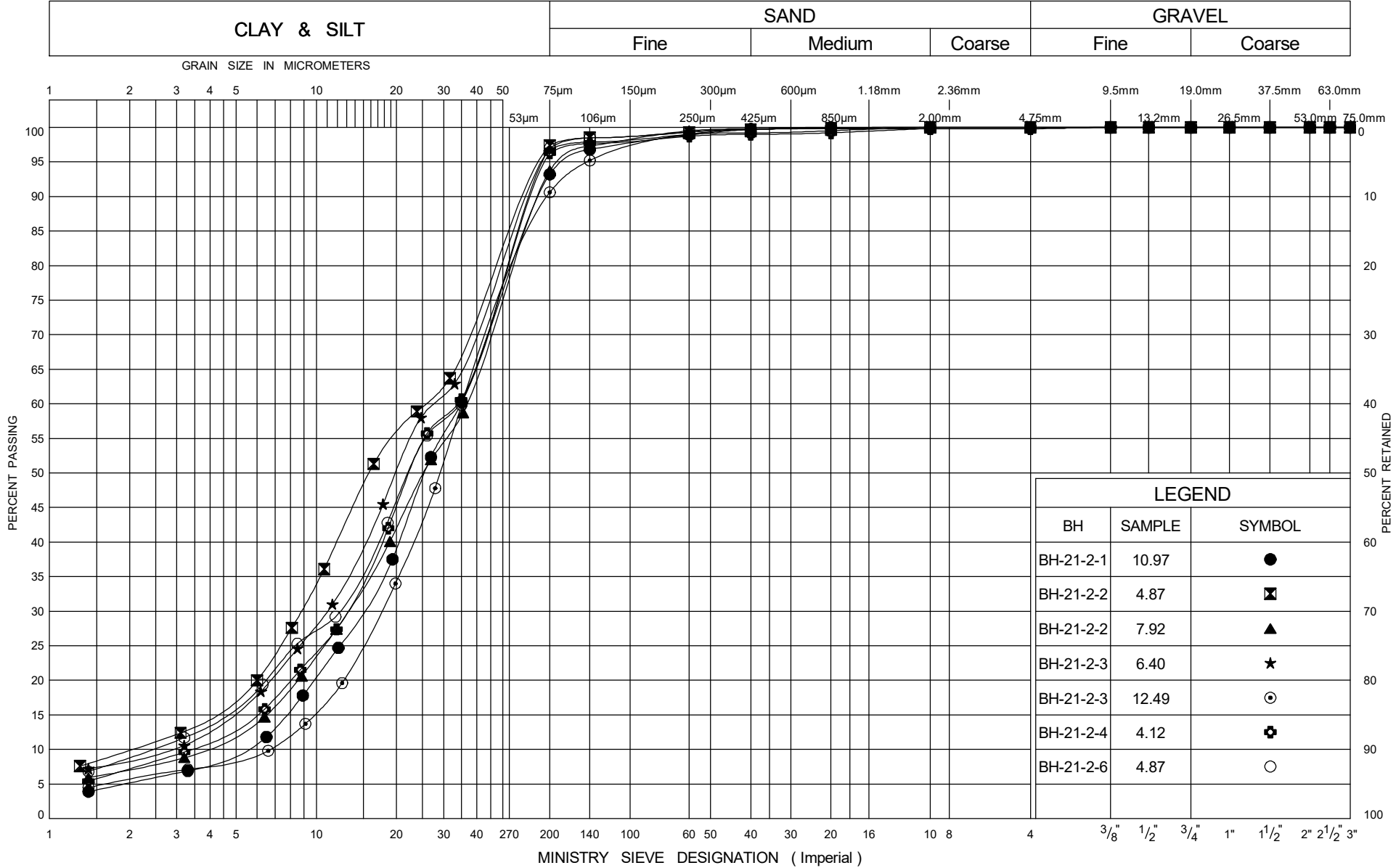
GRAIN SIZE DISTRIBUTION

FILL: NON-COHESIVE FILL

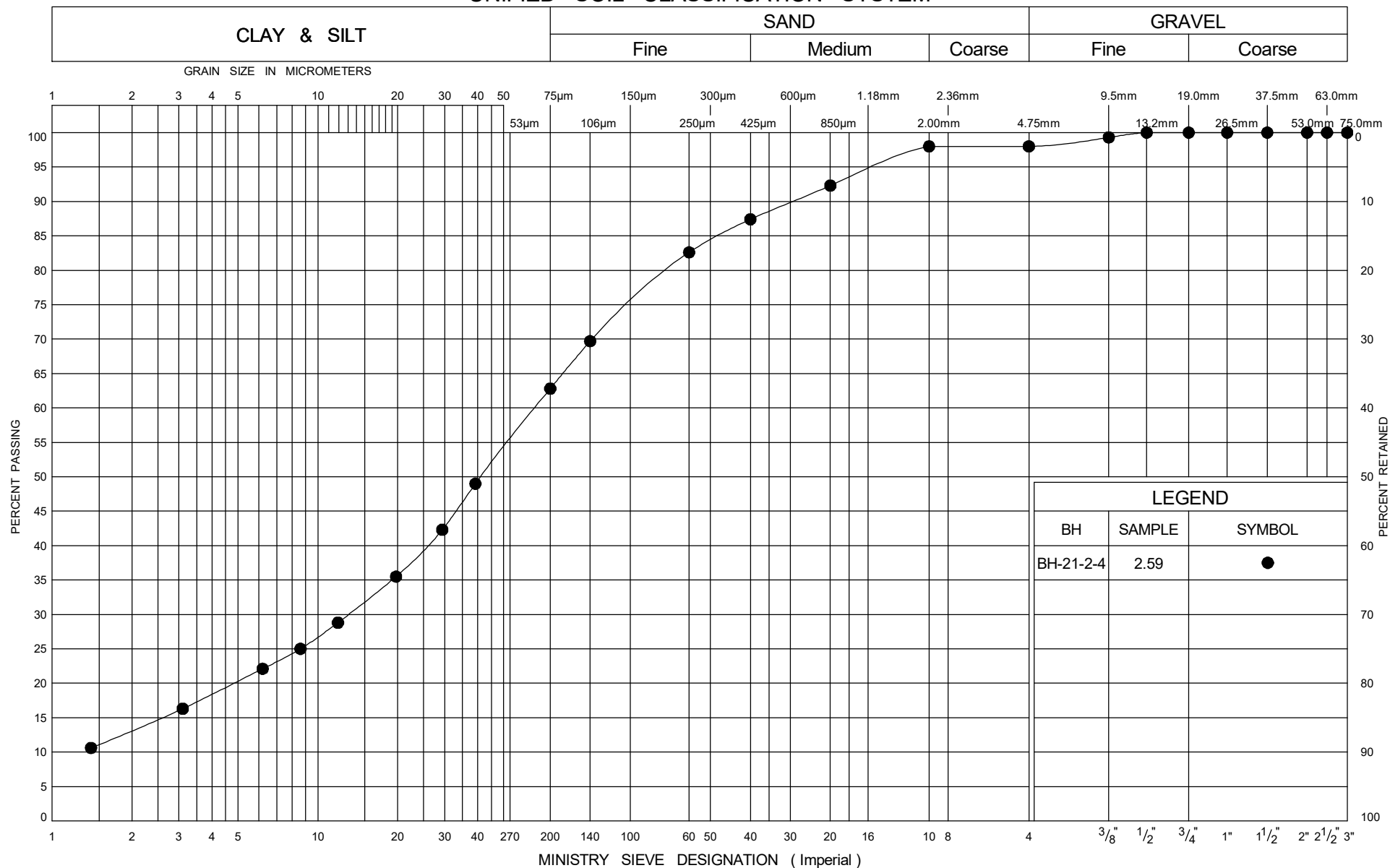
FIG No 1



UNIFIED SOIL CLASSIFICATION SYSTEM



## UNIFIED SOIL CLASSIFICATION SYSTEM

Ministry of  
Transportation

## GRAIN SIZE DISTRIBUTION

SANDY SILT

FIG No 3

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

**ATTENTION TO: Ian MacMillan**

**PROJECT: ADM-000257843-KO**

**AGAT WORK ORDER: 21U736481**

**SOIL ANALYSIS REVIEWED BY: Amanjot Bhela, Inorganic Lab Manager**

**DATE REPORTED: Apr 30, 2021**

**PAGES (INCLUDING COVER): 5**

**VERSION\*: 2**

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

**\*Notes**

VERSION 2: Version 2 supersedes work order 21U736481, Version 1, issued April 28, 2021. Including Sulphide in Soil.

**Disclaimer:**

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



# AGAT Laboratories

## Certificate of Analysis

AGAT WORK ORDER: 21U736481

PROJECT: ADM-000257843-KO

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

CLIENT NAME: EXP. SERVICES INC.

SAMPLING SITE:

ATTENTION TO: Ian MacMillan

SAMPLED BY:PL

### Corrosivity Package

DATE RECEIVED: 2021-04-21

DATE REPORTED: 2021-04-30

		21-1-3, SS4, 7		21-1-4, SS5, 10 -		21-2-3, SS4, 7		21-2-3, SS3, 5	
SAMPLE DESCRIPTION:		1/2 - 9 1/2'		12'		1/2 - 9 1/2'		-7'	
SAMPLE TYPE:		Soil		Soil		Soil		Soil	
DATE SAMPLED:		2021-04-21		2021-04-21		2021-04-21		2021-04-21	
Parameter	Unit	G / S	RDL	2378994	2379005	2379006	2379007		
Chloride (2:1)	µg/g		2	79	42	36	47		
Sulphate (2:1)	µg/g		2	4	5	38	7		
pH (2:1)	pH Units		NA	7.84	8.00	6.92	7.54		
Electrical Conductivity (2:1)	mS/cm		0.005	0.164	0.136	0.122	0.122		
Resistivity (2:1) (Calculated)	ohm.cm		1	6100	7350	8200	8200		
Redox Potential 1	mV		NA	275	131	242	268		
Redox Potential 2	mV		NA	224	142	248	278		
Redox Potential 3	mV		NA	234	142	244	283		

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

**2378994-2379007** EC, pH, Chloride and Sulphate were determined on the extract obtained from the 2:1 leaching procedure (2 parts DI water: 1 part soil). Resistivity is a calculated parameter.

Redox potential measured on as received sample. Due to the potential for rapid change in sample equilibrium chemistry with exposure to oxidative/reduction conditions laboratory results may differ from field measured results.

Redox potential measurement in soil is quite variable and non reproducible due in part, to the general heterogeneity of a given soil. It is also related to the introduction of increased oxygen into the sample after extraction. The interpretation of soil redox potential should be considered in terms of its general range rather than as an absolute measurement.

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

**Certified By:**



## Quality Assurance

CLIENT NAME: EXP. SERVICES INC.

PROJECT: ADM-000257843-KO

SAMPLING SITE:

AGAT WORK ORDER: 21U736481

ATTENTION TO: Ian MacMillan

SAMPLED BY: PL

### Soil Analysis

RPT Date: Apr 30, 2021			DUPLICATE			Method Blank	REFERENCE MATERIAL		METHOD BLANK SPIKE		MATRIX SPIKE	
PARAMETER	Batch	Sample Id	Dup #1	Dup #2	RPD		Measured Value	Acceptable Limits	Recovery	Acceptable Limits	Recovery	Acceptable Limits
								Lower		Upper		Lower

#### Corrosivity Package

Chloride (2:1)	2378994	2378994	79	79	0.0%	< 2	93%	70%	130%	108%	80%	120%	109%	70%	130%
Sulphate (2:1)	2378994	2378994	4	4	NA	< 2	100%	70%	130%	105%	80%	120%	107%	70%	130%
pH (2:1)	2378994	2378994	7.84	7.68	2.1%	NA	100%	90%	110%						
Electrical Conductivity (2:1)	2378994	2378994	0.164	0.160	2.5%	< 0.005	98%	80%	120%	NA			NA		
Redox Potential 1		1					100%	90%	110%						

Comments: NA signifies Not Applicable.

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

Duplicate NA: results are under 5X the RDL and will not be calculated.

### Certified By:




## Appendix E – Slope Stability Analyses

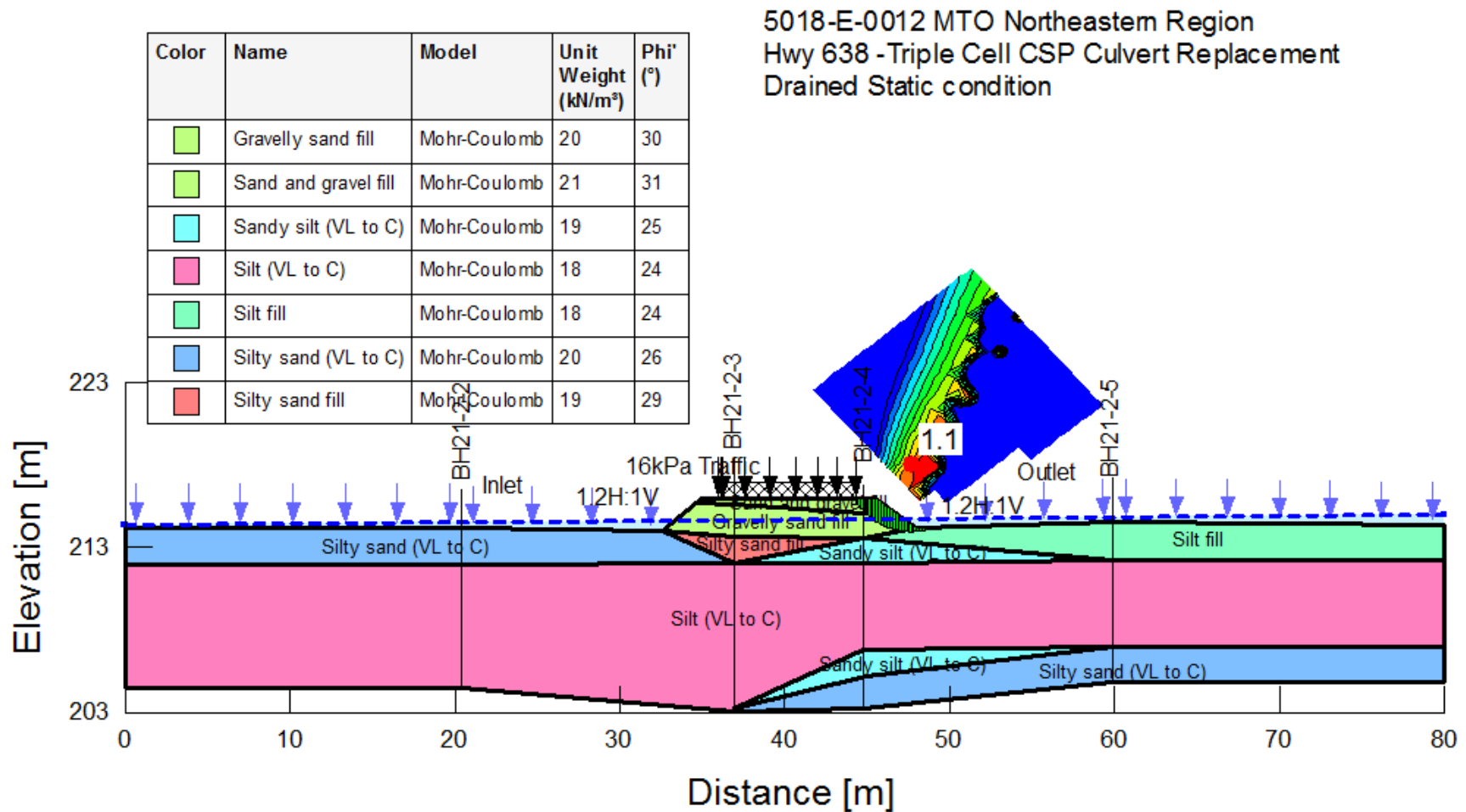


Figure 1: Slope stability analysis for Triple Cell CSP culvert at Hwy 638 – drained static condition with existing embankment

5018-E-0012 MTO Northeastern Region  
Hwy 638 -Triple Cell CSP Culvert Replacement  
Drained Static condition

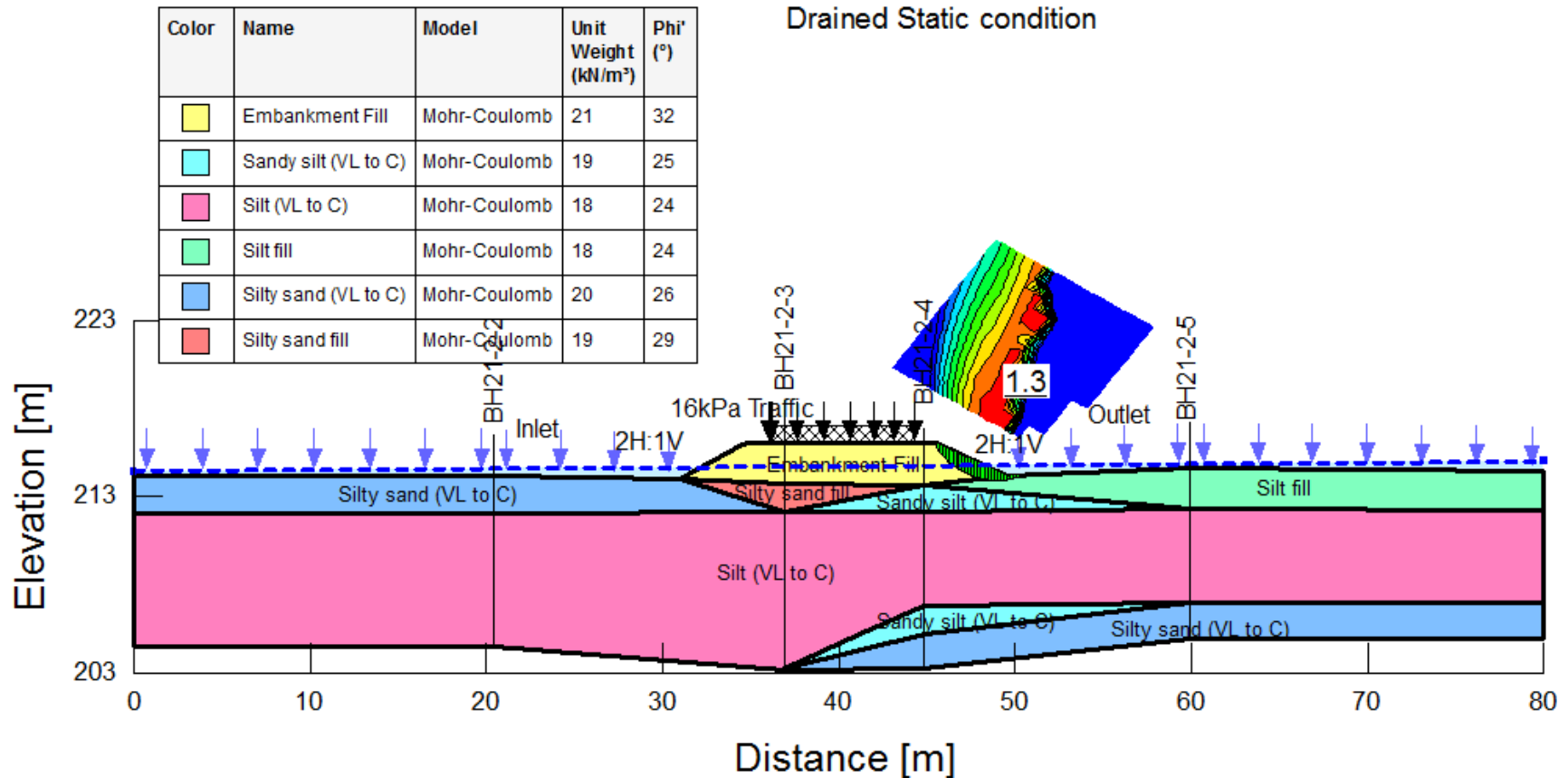
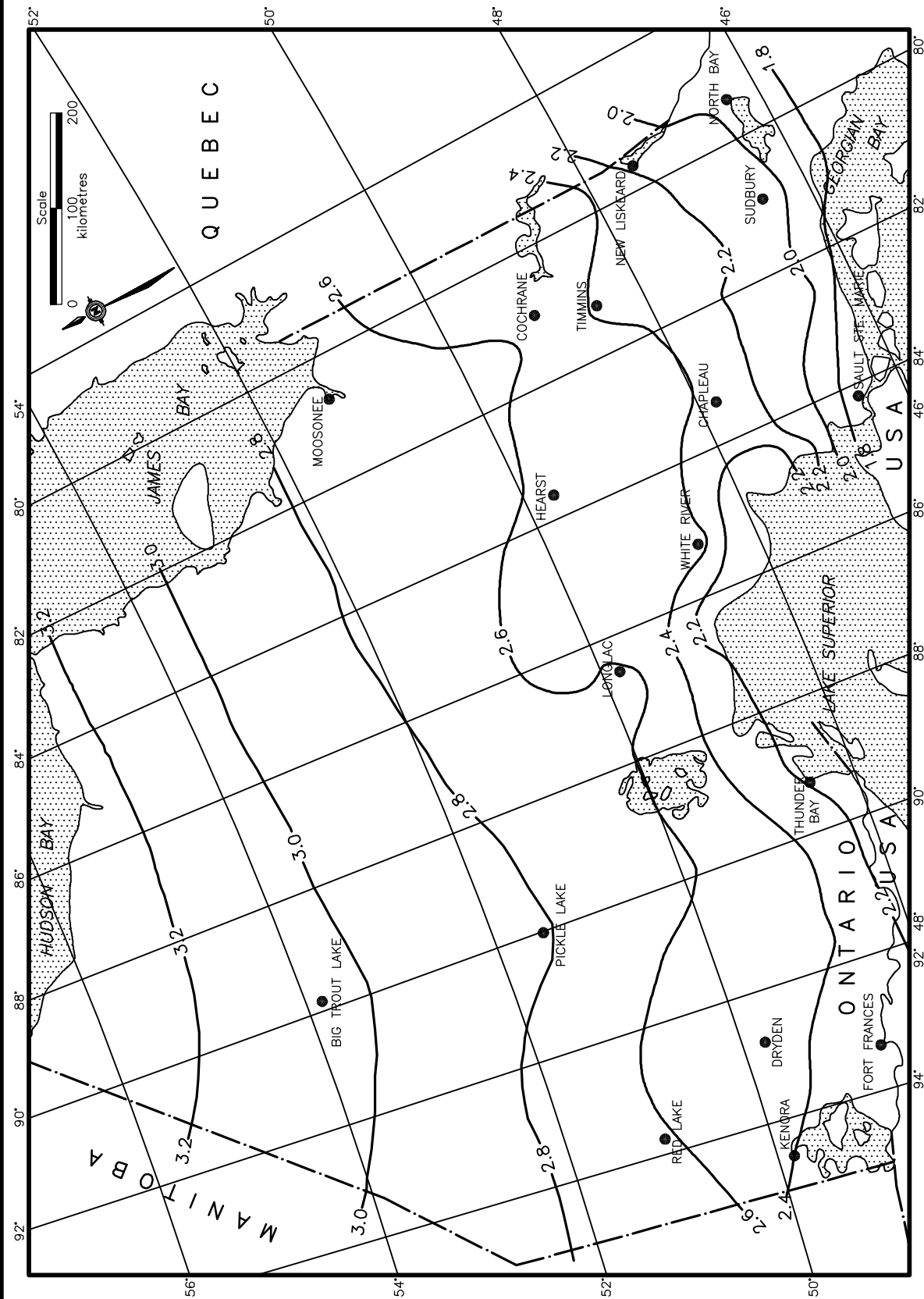


Figure 2: Slope stability analysis for Triple Cell CSP culvert at Hwy 638 – drained static condition after culvert replacement



## Appendix F – 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

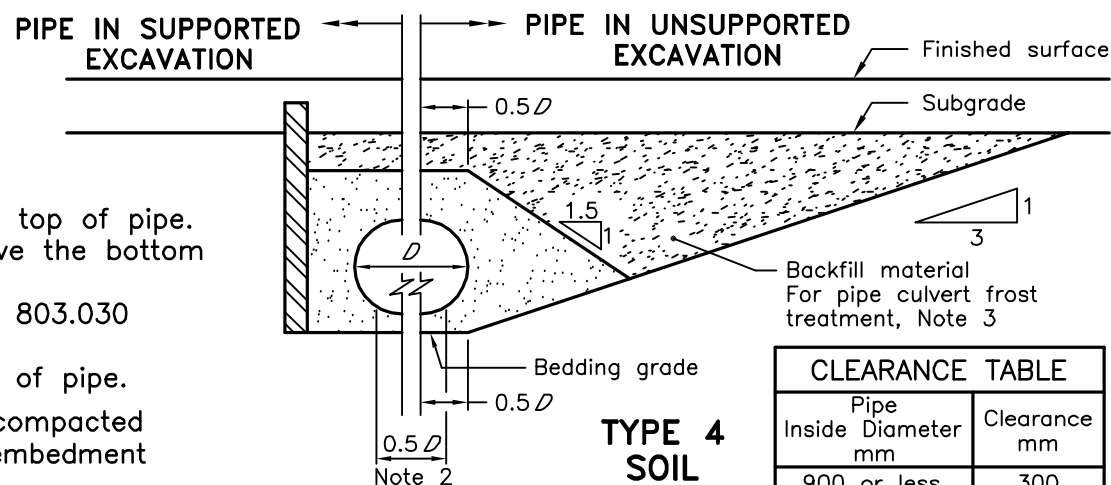
Nov 2010

Rev 1

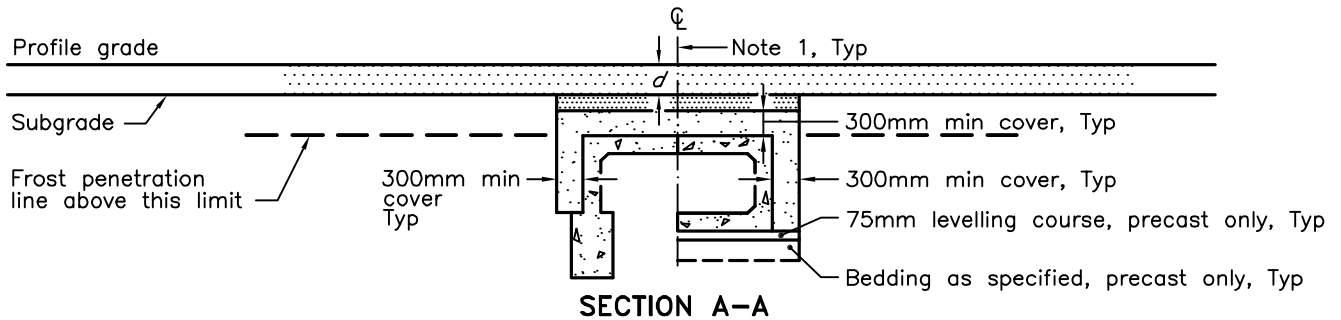
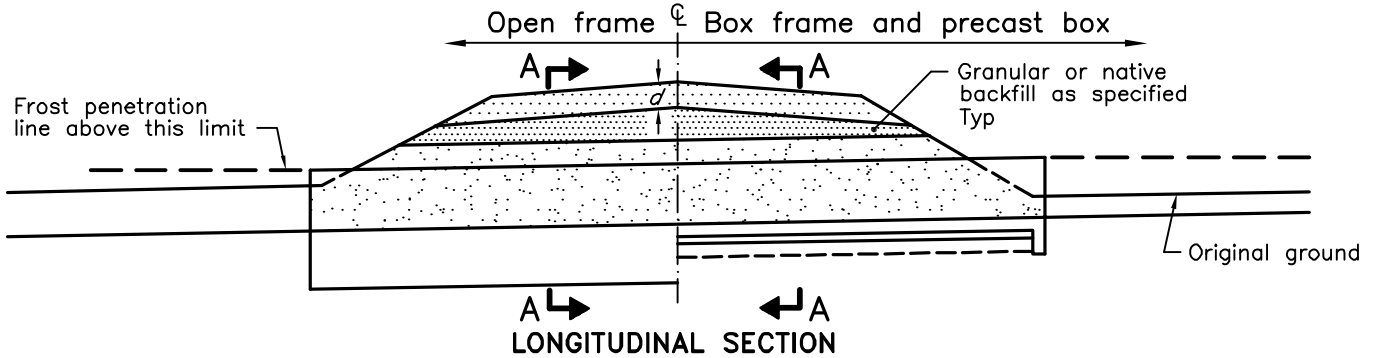
## FOUNDATION FROST PENETRATION DEPTHS FOR NORTHERN ONTARIO

OPSD 3090.100

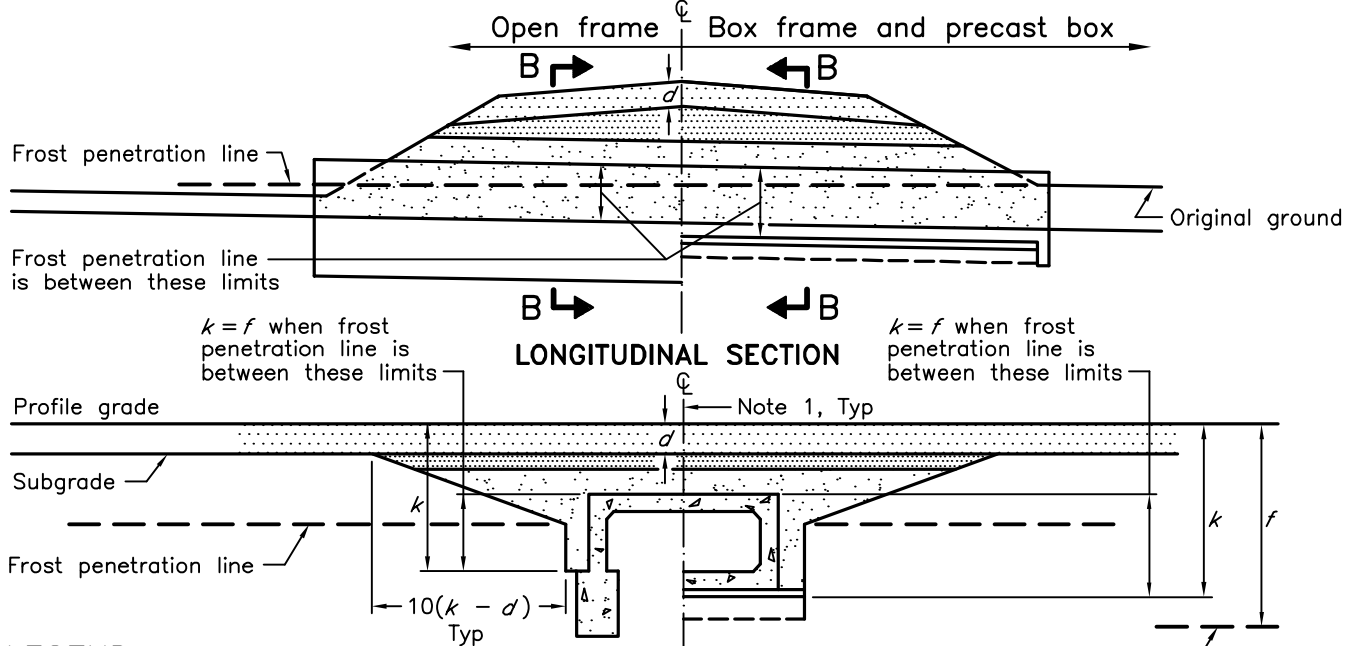




## FROST PENETRATION LINE AT OR ABOVE TOP OF CULVERT



## 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.

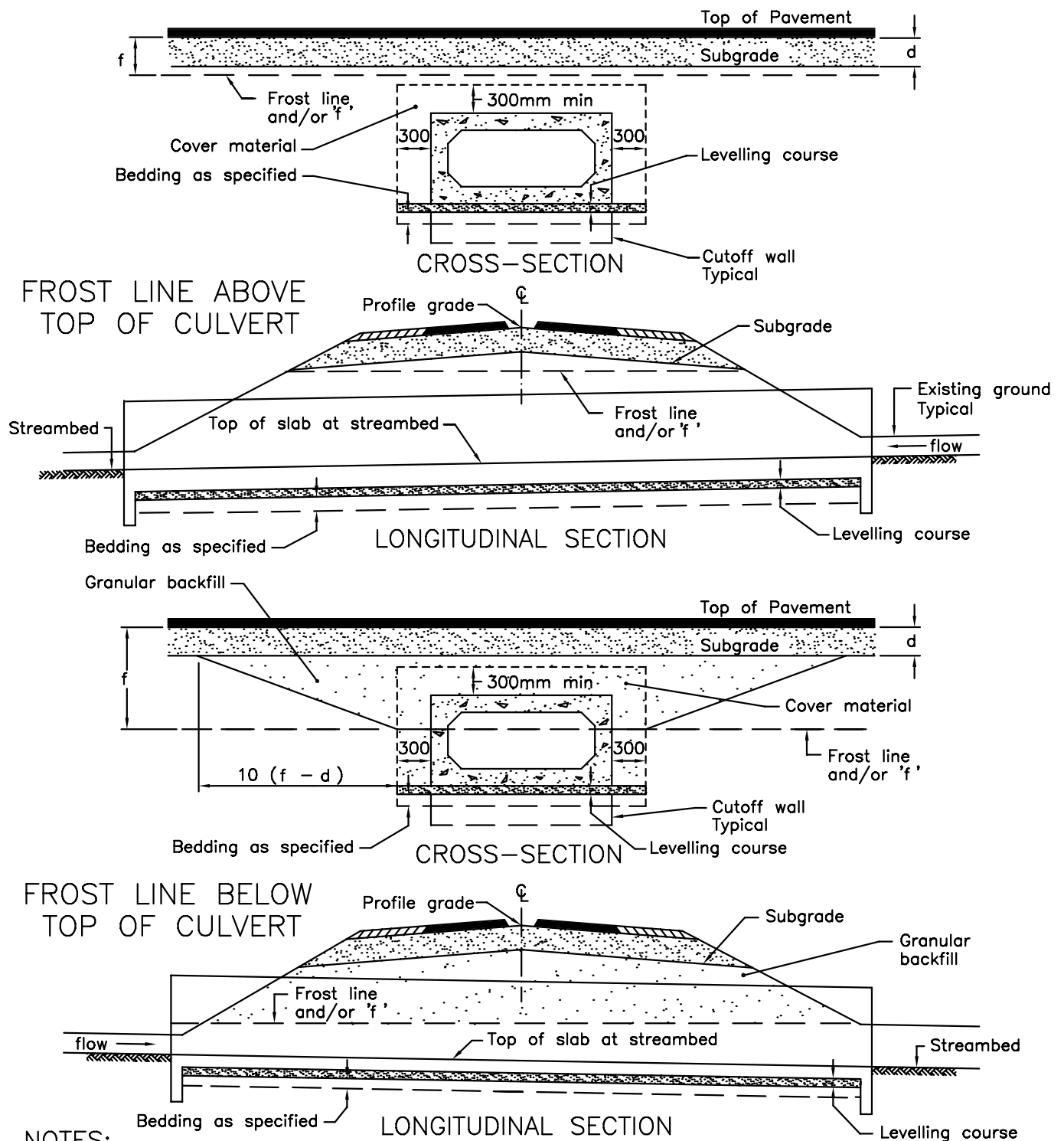
ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2010    Rev    2

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

**OPSD 803.010**





#### NOTES:

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

#### LEGEND:

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

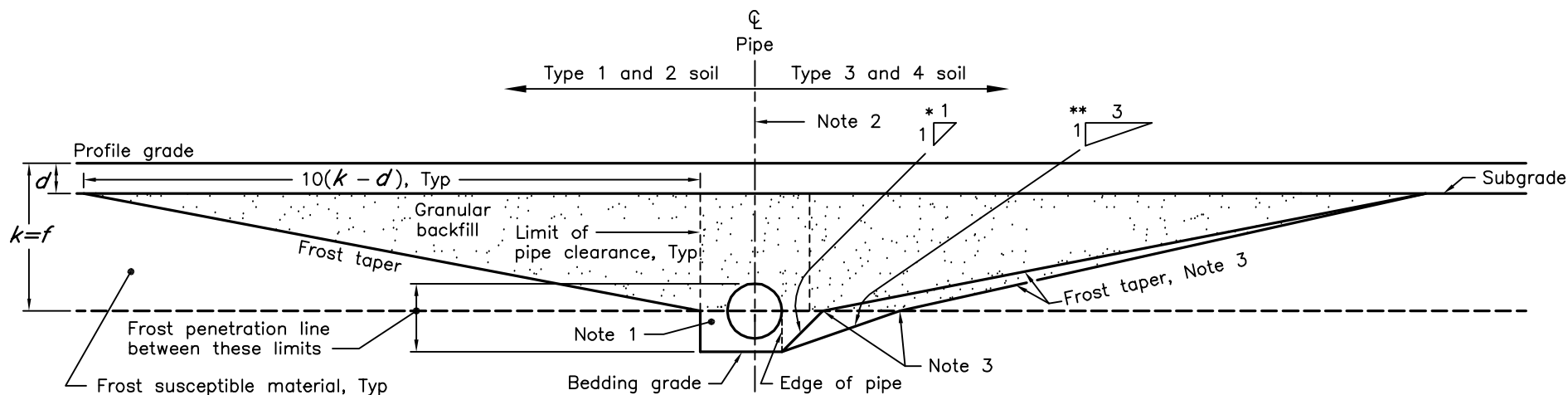
MINISTRY OF TRANSPORTATION ONTARIO DRAWING

Date | 1994 05 25 | Rev |

## BEDDING AND BACKFILL FOR PRECAST CONCRETE BOX CULVERTS

Issue Date  
WP  
Issued by

MTOD - 803.021



## 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

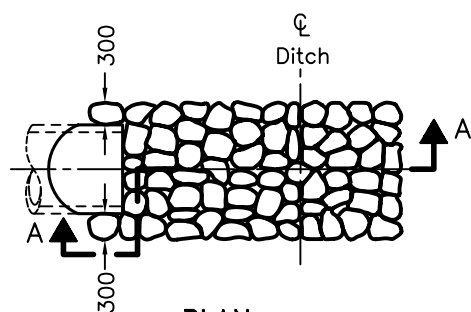
Nov 2015

Rev 4

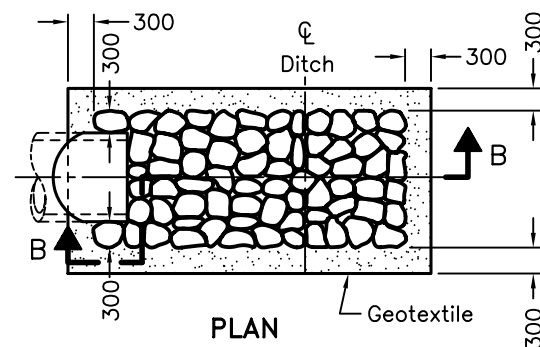
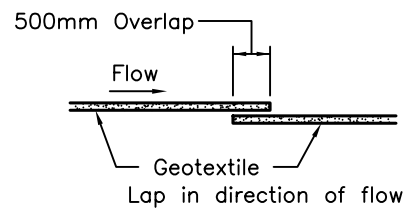


OPSD 803.031

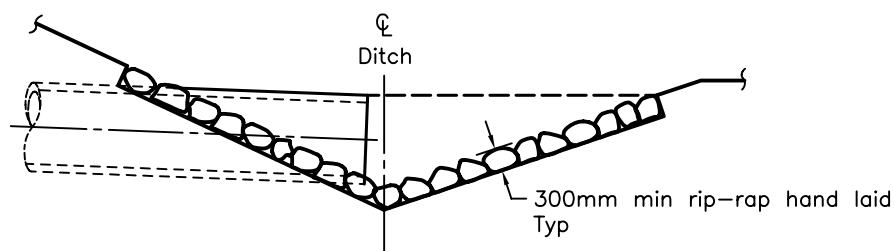




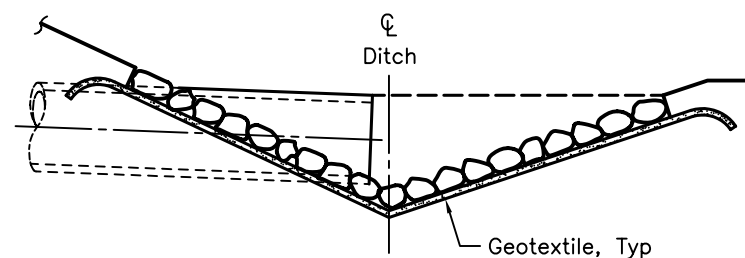
PLAN  
CUT OR FILL



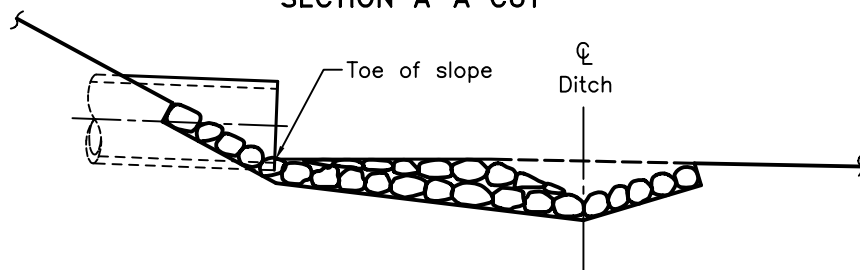
PLAN  
CUT OR FILL



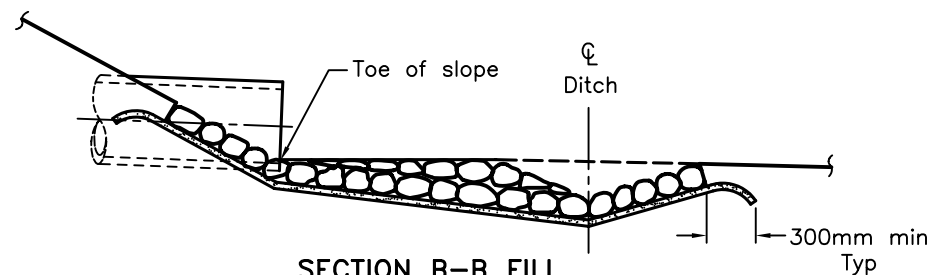
SECTION A-A CUT



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 G – Seismic Hazard Calculation

# 2015 National Building Code Seismic Hazard Calculation

INFORMATION: Eastern Canada English (613) 995-5548 français (613) 995-0600 Facsimile (613) 992-8836  
Western Canada English (250) 363-6500 Facsimile (250) 363-6565

Site: 46.454N 83.900W

2021-06-03 10:48 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)	0.047	0.024	0.015	0.004
Sa (0.1)	0.066	0.036	0.023	0.007
Sa (0.2)	0.064	0.039	0.025	0.009
Sa (0.3)	0.055	0.035	0.024	0.008
Sa (0.5)	0.046	0.031	0.021	0.006
Sa (1.0)	0.029	0.019	0.012	0.003
Sa (2.0)	0.015	0.009	0.005	0.001
Sa (5.0)	0.004	0.002	0.001	0.000
Sa (10.0)	0.002	0.001	0.001	0.000
PGA (g)	0.037	0.021	0.013	0.004
PGV (m/s)	0.036	0.022	0.014	0.004

**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

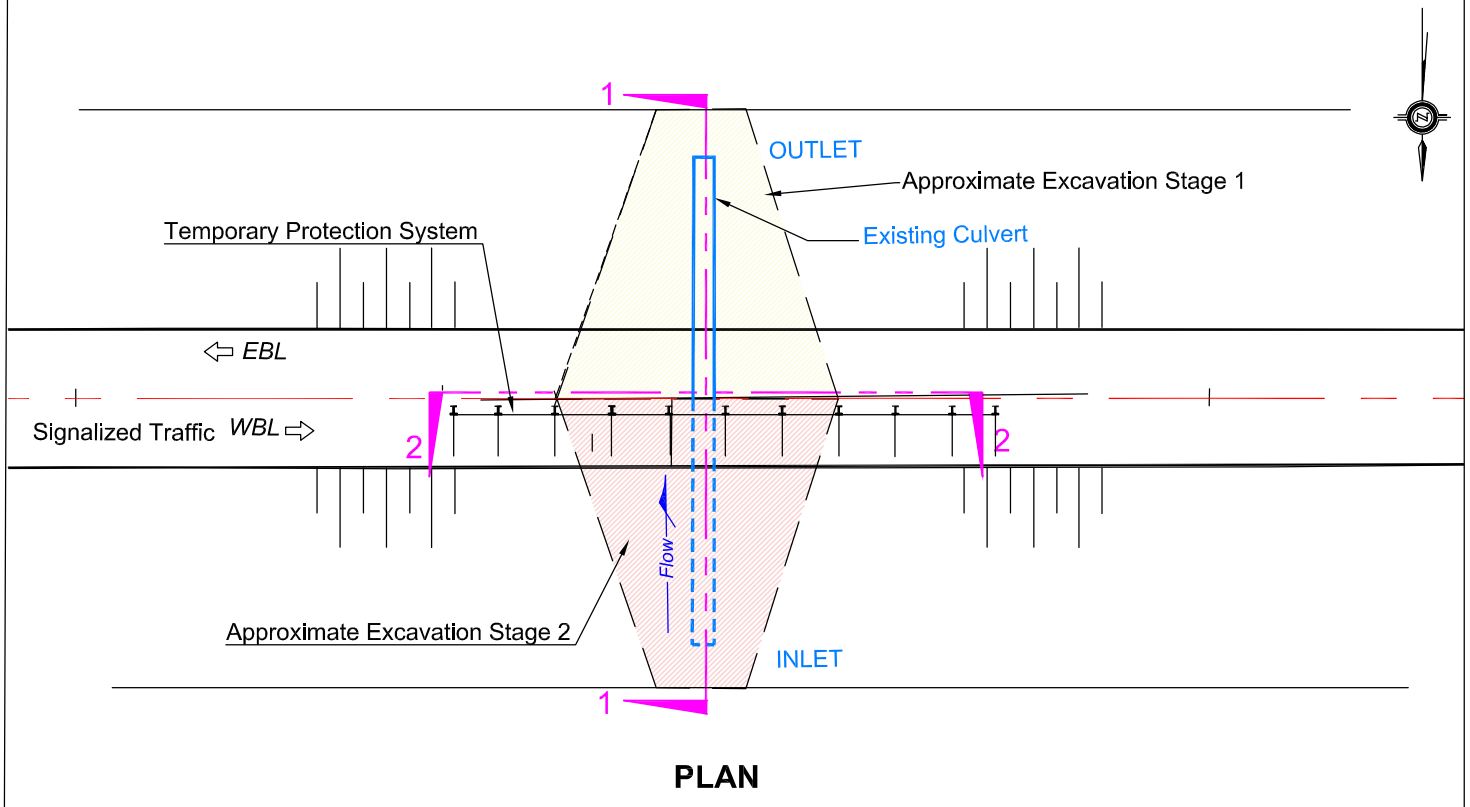
Ressources naturelles  
Canada

Canada

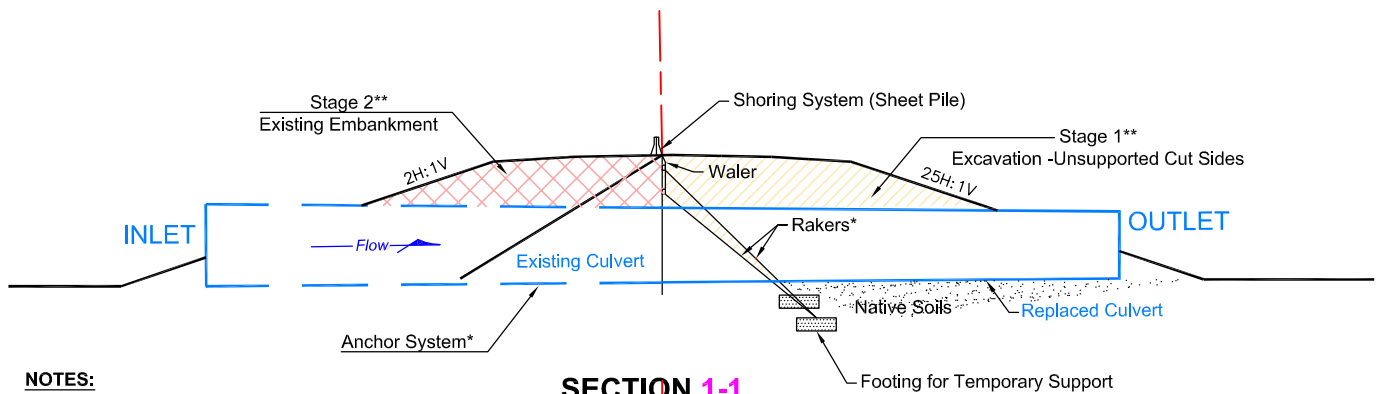
## Appendix H – Schematic Sketches

FIGURE H1A: HALF AND HALF CONSTRUCTION WITH UNSUPPORTED CUT SIDES

**SCHEMATIC DIAGRAMS (NST)**



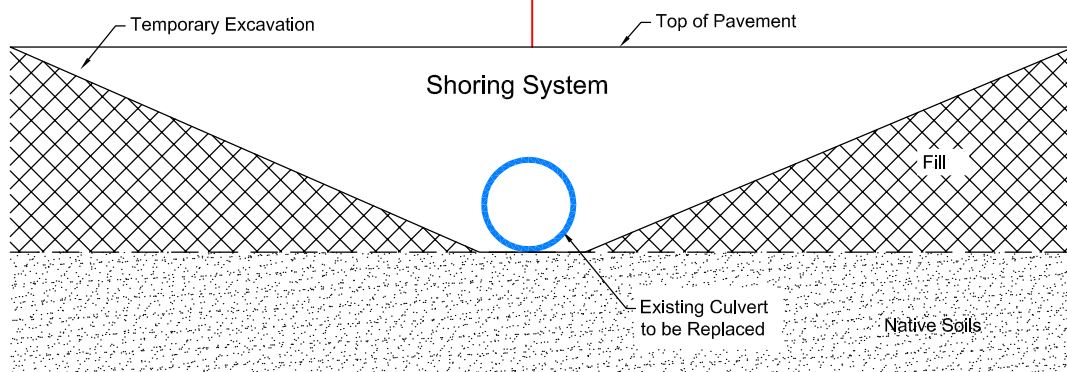
Half and Half Construction, Shoring System with either Cut or Anchor System - Unsupported Cut



**NOTES:**

\* Rakers or Anchor System

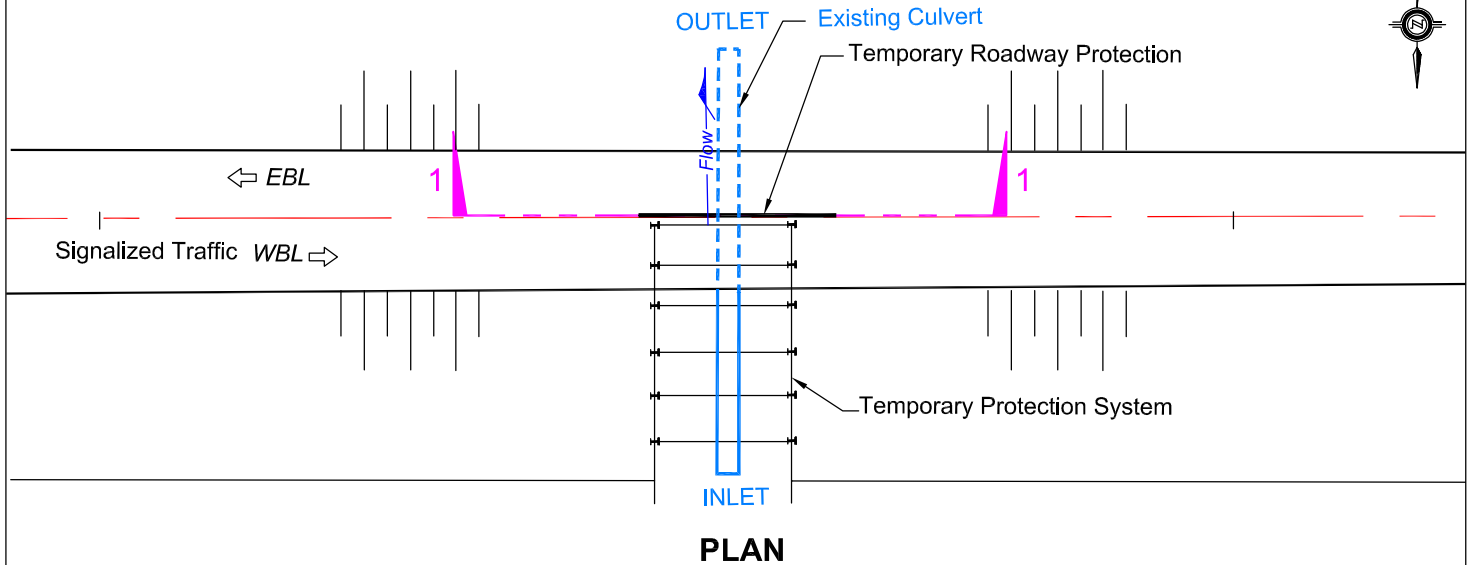
\*\* Stage 2 Following Stage 1 in Opposite Way



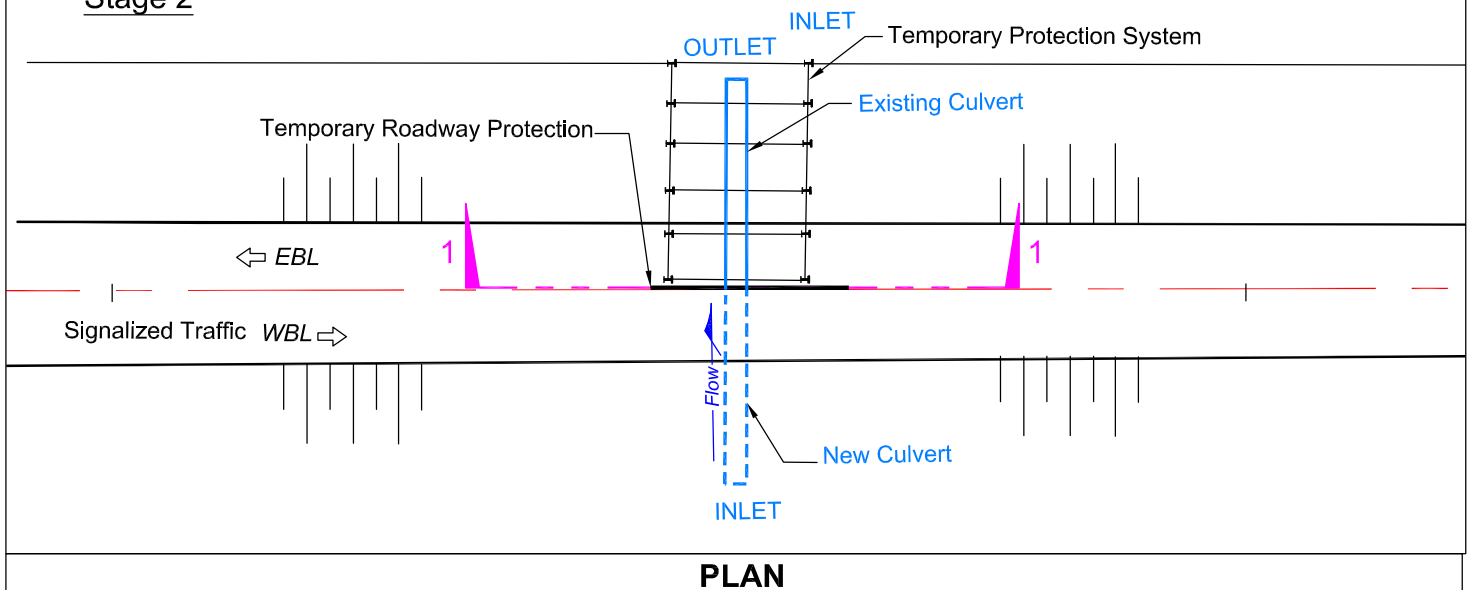
**FIGURE H1B: HALF AND HALF CONSTRUCTION WITH BRACED CUT SIDES OR ANCHOR SYSTEM**  
**SCHEMATIC DIAGRAMS (NST)**



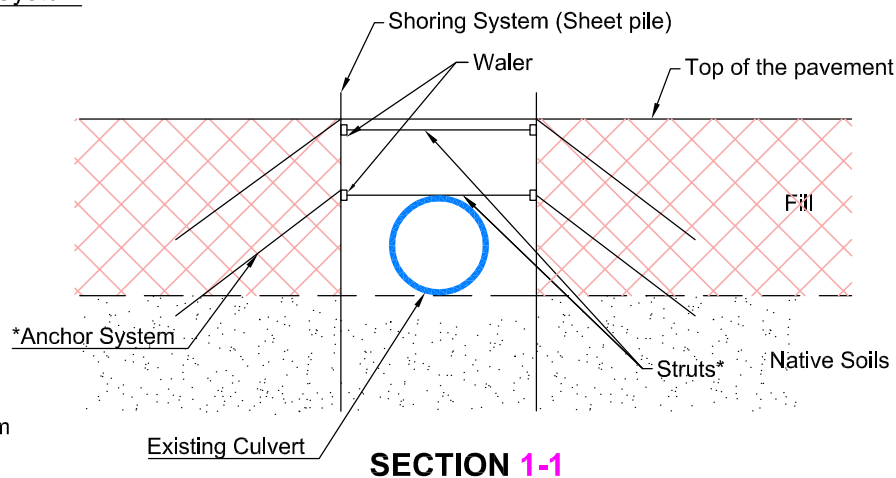
Stage 1



Stage 2



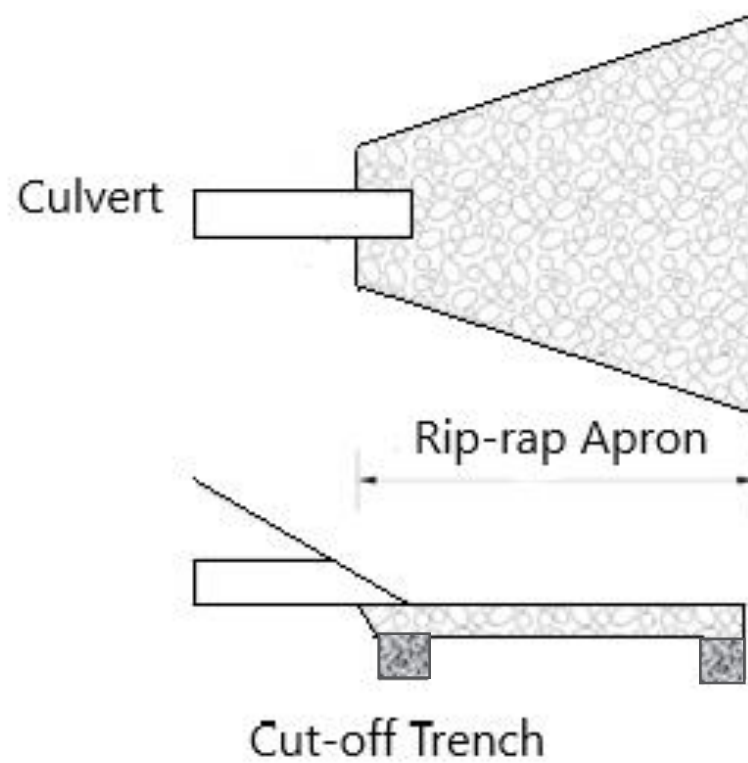
Half and half Construction, Shoring System- Braced Cut Struts or Anchor System



NOTE:

\* Struts or Anchor System

FIGURE H3: SCHEMATIC SKETCH OF EROSION PROTECTION AT OUTLET



## Appendix I – NSSP for Obstructions

## **NSSP FOR OBSTRUCTIONS**

### **Scope of Work**

The Contractor shall be alerted to the potential presence of cobbles and boulders in the fill and native soils encountered in few boreholes advanced at the site. Therefore, appropriate equipment and procedures will be required for open cut excavation and installation of roadway protection systems and temporary dewatering/unwatering systems.

### **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.