Foundation Investigation and Design Report

Highway 599 Trout Creek Culvert Replacement

Agreement No. 6019-E-0004
Assignment No. 1
GWP 6530-17-00
Geocres No. 52J-19
Latitude: 50.206730°; Longitude: -90.726231°

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Ministry of Transportation Ontario
Northwestern Region Geotechnical Section

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1 FOUNDATION INVESTIGATION REPORT

1.1 Introduction

This report presents the results of a geotechnical investigation completed by EXP Services Inc. for the replacement of the Trout Creek culvert on Highway 599, in Thunder Bay District. The work was undertaken under Assignment No. 6019-E-0004, Assignment No. 1. The terms of reference(TOR) were provided by MTO in an email dated September 17, 2020.

The purpose of the investigation was to permit detailed design for the replacement of the Trout Creek non-structural culvert to a structural culvert and provide construction staging recommendations. These recommendations include a roadway protection system along centreline and dewatering structures during culvert replacement at the site. 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 Trout Creek culvert is located on Highway 599 (Sta. 78+603; Latitude: 50.206730º; Longitude: -90.726231º), about 4.6 km south of Savant Lake CNR crossing within the District of Thunder Bay, Ontario. At the site, Highway 599 is a two lane roadway, with a speed limit of 80 km/h (unless otherwise posted) and is about 7.2 m wide from edge of pavement to edge of pavement, with 1.5 m and 1.0 m gravel shoulders on north and south sides, respectively. The elevation of highway pavement centerline at Sta. 78+603 is about 416.6 m. Based on documents provided by MTO, the roadway embankment above the creek bed is approximately 4.5 m high having the side slopes of approximately 1.1H:1V (outlet) to 1.3H:1V (inlet).

Based on the information in the TOR and AutoCAD drawing provided by MTO, the existing culvert is a 22.47 m long 1.5 m x 1.5 m wooden box culvert with two overflow CSP culverts on both sides. At the outlet side, the wooden box culvert was extended by an approximately 5.5 m long, 2 m diameter CSP pipe (Photo 5 in Appendix A), while at the inlet side the culvert starts with a concrete headwall (Photo 1 in Appendix A). The existing culvert alignment has a skew angle of 14 degree to the highway central line. Based on available information the obvert of the existing culvert is located at approximate elevation of Elev. 413.8 m at the inlet side and Elev. 413.2 m at the outlet side. Since the top elevation of the roadway is approximately at Elev. 416.6 m, the fill cover above the culvert crown is approximately 2.8 m thick. The existing overflow west and east culverts are approximately 23.7 m long CSP pipes having 1.22 m diameter. The obverts of the west and east CSPs were measured to be at an approximate Elev. 413.5 m and 413.7 m, respectively. Select photographs of the site and existing culverts are presented in Appendix A. The site plan and cross-section profiles for the proposed culvert alignment are shown on the drawings attached in Appendix B.

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

1.2.2 Geological Setting

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

1.3 Investigation Procedures

1.3.1 Site Investigation and Field Testing

The field investigation was performed between November 02, 2020 and November 06, 2020. The field program consisted of drilling three (3) sampled boreholes and nine (9) hand probe holes, numbered BH20-1 to BH20-3 and HP20-4 to HP20-11, respectively. Three (3) boreholes were located on the embankment to provide subsurface information for the culvert replacement and the temporary roadway protection system, while due to access restriction for the drill rig, nine (9) hand probe holes were drilled at the ends of the existing culvert (i.e. at toes of the embankment). BH20-1 to BH20-3 was advanced from the top of the embankment. HP20-4 to HP20-7 and HP20-6I were advanced at an accessible location near the inlet and HP20-8 to HP20-11 were advanced at an accessible location near the outlet. BH20-1 was drilled about 3 m east of the edge of the east outflow CSP culvert, BH20-2 was drilled about 3 m west of the edge of the west outflow CSP culvert and BH20-3 was drilled about 22 m east of the main culvert centreline. The locations of boreholes and hand probe holes drilled during this investigation are shown on Drawing 1 in Appendix B.

Three roadway boreholes drilled during this fieldwork were advanced using a rubber track mounted B54X drill rig equipped with solid stem augers, NQ core and standard soil sampling equipment, operated by a specialist drilling contractor, Maple Leaf Drilling Ltd., and all hand probe holes were advanced using a power hand auger with SSA. The roadway borehole BH20-1, BH20-2 and BH20-3 were advanced to depths of about 6.8 m, 10.4 m and 3.9 m below ground surface, respectively. The off-road probe holes (HP20-4 to PH20-11 and HP20-6I) were advanced to a depth of between 0.2 m and 0.8 m.

The borehole locations (referenced to the MTM NAD83 coordinate system) and their ground surface elevations were surveyed by EXP personnel using a GPS (Garmin 60 CSX) and a basic level and survey rod, respectively, having an accuracy of 2 m in the horizontal directions and 0.1 m in the vertical direction. A temporary benchmark (TBM) set on SSW rock bolt on the hydro pole, east of Trout Creek, north of Highway 599 was used. Based on survey data provided by MTO, the elevation of this benchmark (BM) was referenced to be Elev. 417.00 m. The BM location is shown on Drawing 1 in Appendix B.

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

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 Thunder Bay laboratory for additional visual, textual, olfactory examination and selective testing.

1.3.2 Laboratory Testing

All samples returned to the laboratory were subjected to visual examination and classification. The laboratory testing program included the determination of natural moisture content on all soil samples and particle size distribution for approximately 25% of the collected soil samples. Uniaxial compression tests were performed on selected rock cores from three boreholes. In addition, soil chemical package tests were performed on three (3) soil samples. All of the laboratory tests were carried out according to MTO and/or ASTM Standards as appropriate.

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

1.4 Subsurface Conditions

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

In general, the subsoil condition at the roadway consists of sand and gravel fill to gravelly sand below the asphalt treatment, followed by native sandy silt with gravel layer followed by silty sand with gravel till underlain by bedrock (sloping bedrock is observed at this site, 2.3 m to 3.7 m below ground surface to the east of culvert centerline while 7.0 m below ground surface to the west of centerline). At the inlet and outlet sides, the subsurface conditions consist of native silty sand/sandy gravel layers below topsoil or peat underlain by a layer of cobbles and boulders and/or bedrock.

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

1.4.1 Stratigraphy below Existing Embankment

1.4.1.1 Asphalt Treatment

Asphalt treatment, approximately 0.025 m (~1 inch) thick, was encountered at the surface of all boreholes BH20-1 to BH20-3.

1.4.1.2 Fill: Sand and Gravel

Sand and gravel fill was encountered below the surface treatment in boreholes advanced through the embankment, BH20-1 to BH20-3. The fill layer extended to depth between 0.8 m to 2.3 m below ground surface with elevation between Elev. 415.8 m to 413.8 m. The explored thickness of this layer was between 0.8 m to 2.3 m.

The composition of this fill material generally consisted of sand and gravel with occasional boulders and trace silt and clay. The fill was generally grey to brown in color, and moist. Boreholes BH20-1 and BH20-2 were augured while BH20-3 was drilled. The SPT “N” values obtained within this fill material recorded in the borehole BH20-3 ranged from 39 blows per 0.3 m to 50 per 0.125 m penetration, suggesting dense to very dense in relative density.

Laboratory testing performed on selected samples consisted of ten (10) moisture content tests and two (2) grain size distribution tests. The test results are as follows:

Moisture Content:

- 1% to 7%

Grain Size Distribution:

- 44% to 50% gravel;
- 42% to 43% sand; and
- 7% to 14% 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.

1.4.1.3 Fill: Gravelly Sand

Gravelly sand fill was encountered below the sand and gravel fill in boreholes advanced through the embankment, BH20-1 and BH20-2. The fill layer extended to depth between 3.7 m to 5.0 m below ground surface with elevation between Elev. 412.8 m to 411.5 m. The explored thickness of this layer was between 2.2 m to 4.2 m.

The composition of this fill material generally consisted of sand and gravel with occasional boulders and trace to some silt and clay. The fill was generally brown in color, and moist. The boreholes BH20-1 and BH20-2 were augured and at a depth of 4.4 m auger refusal in BH20-2 was noted. The SPT “N” values obtained at this depth within this fill material was about 100 blows per 0.3 m penetration, suggesting very dense in relative density.

Laboratory testing performed on selected samples consisted of eight (8) moisture content tests and two (2) grain size distribution tests. The test results are as follows:

Moisture Content:

- 2% to 9%

Grain Size Distribution:

- 22% to 24% gravel;
- 63% to 65% sand;
- 15% silt and clay in BH20-1;
- 10% silt in BH20-2; and
- 1% clay in BH20-2.

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 2 in Appendix D.

1.4.1.4 Sandy Silt with Gravel

Native sandy silt with gravel was encountered in borehole BH20-2, below the gravelly sand fill. The sandy silt with gravel layer extended to depth of about 6.1 m below ground surface with elevations about Elev. 410.4 m. The explored thickness of this layer was about 1.1 m. BH20-2 was cored due to auger refusal at a depth of 4.4 m and was switched back to drilling at a depth of 5.8 m.

The composition of this layer is sand, silt and gravel with trace clay. The material is grey in color, and wet. The SPT “N” values obtained within this layer is about 50 blows per 0.05 m penetration, suggesting very dense in relative density.

Laboratory testing performed on a selected sample consisted of two (2) moisture content tests and one (1) grain size distribution test. The test results are as follows:

Moisture Content:

- 9% to 11%

Grain Size Distribution:

- 23% gravel;
- 32% sand;
- 36% silt; and
- 9% 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 test is also provided on Figure 3 in Appendix D.

1.4.1.5 Till: Silty Sand with Gravel

Native silty sand with gravel till was encountered in borehole BH20-2, below the sandy silt with gravel layer. The silty sand with gravel till extended to depth of about 7.0 m below ground surface with elevation of about Elev. 409.6 m. The explored thickness of this layer is about 0.9 m.

The composition of this layer is silt, sand and gravel, occasional to some cobbles and boulders, trace clay. The material is grey in color and wet. The SPT "N" values within this layer ranged from 33 blows per 0.3 m to 50 blows per 0.15 m penetration, suggesting dense to very dense in relative density.

Laboratory testing performed on a selected sample consisted of two (2) moisture content tests and one (1) grain size distribution test. The test results are as follows:

Moisture Content:

- 1% to 10%

Grain Size Distribution:

- 21% gravel;
- 49% sand;
- 24% silt; and

- 6% 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 test is also provided on Figure 4 in Appendix D.

1.4.1.6 Bedrock

The presence of bedrock, at approximately between 2.3 m to 7.0 m below the existing ground surface was recorded. The bedrock was confirmed using coring of about 3 m long cores in all boreholes BH20-1 to BH20-3. The actual bedrock surface depth and elevation encountered at these borehole locations are listed in Table 1.1. Photographs of rock cores are included in Appendix E. It should be noted that sloping bedrock is observed at this site, as given in the table. The depth of bedrock is 2.3 m to 3.7 m east of culvert centerline while it is 7.0 m to the west of centerline of the Trout creek culvert.

Table 1.1. Depth and elevation of bedrock surface

Borehole	Depth Below Ground Surface (m)	Elevation (m)	Comments
BH20-1	3.7	412.8	Bedrock Cored
BH20-2	7.0	409.6	Bedrock Cored
BH20-3	2.3	413.8	Bedrock Cored

Based on the bedrock NQ cores (~ core diameter 47 mm) recovered, the bedrock at the site consists of meta-volcanic rock. In general, the rock samples are described as pink/white to grey/black in colour, fine to coarse grained, severely fractured to sound. The Rock Quality Designation (RQD) measured on the core samples typically ranged from approximately 37% to 79%, indicating a rock mass of poor to good quality, mostly fair quality. The total core recovery (TCR) ranges from 99% to 100%.

The uniaxial compression tests were performed on rock cores from three boreholes, BH20-1, BH20-2 and BH20-3, and the uniaxial compressive strength (UCS) was measured to be about 82 MPa, 83 MPa and 103 MPa, respectively, indicating strong to very strong (R4 to R5) rock, primarily strong (R4) according to CFEM. However, our experience suggests that the rock in this area could be very strong to extremely strong (i.e. UCS in the range of 150 to 250 MPa). The laboratory uniaxial compression tests results are presented on the borehole records in Appendix C as well as, in Appendix D.

1.4.2 Stratigraphy at Inlet and Outlet

As noted before, due to steep sides of embankment the access to the inlet and outlet of the Trout creek culvert was restricted and the drill rig was not able to be mobilized to drilled boreholes at those locations. Instead, the hand probe holes HP20-4 to HP20-7 and HP20-6I were extended at inlet, and hand probe holes HP20-8 to HP20-11 at the outlet. The hand probes were drilled from surface to a depth of 0.2 to 0.8 m upon refusal. At the inlet and outlet sides, bedrock outcrop and rock fill were observed at the toe of embankment, suggesting very shallow bedrock or buried rock fill in the vicinity of inlet and outlet.

1.4.2.1 Topsoil

Topsoil, approximately 0.1 m to 0.6 m thick, was encountered at the surface of boreholes at inlet HP20-4 to HP20-7 and HP20-6I, and at the surface of borehole at outlet HP20-11. Topsoil thicknesses may further vary beyond the borehole locations.

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

Laboratory testing performed on selected samples consisted of eight (8) moisture content test. The test result is as follows:

Moisture Content:

- 21% to 49%

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

1.4.2.2 Peat and Sand

Peat and sand, approximately 0.3 m thick, was encountered at the surface of probe holes at outlet HP20-8 to HP20-10. Peat and sand thicknesses may further vary beyond the borehole locations. Boreholes HP20-8 and HP20-9 was terminated in this layer due to auger refusal.

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

Laboratory testing performed on selected samples consisted of four (4) moisture content tests and two (2) grain size distribution tests. The test result is as follows:

Moisture Content:

- 44% to 94.5%

Grain Size Distribution:

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

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

1.4.2.3 Silty Sand / Silty Sand with Gravel

Native silty sand / silty sand with gravel was encountered in probe holes HP20-4, HP20-6, HP20-6I, HP20-7 and HP20-11 below topsoil and in HP20-10 below peat and sand. The silty sand / silty sand with gravel layer extended to depths ranging between 0.2 m to 0.8 m below ground surface with elevations ranging between Elev. 414.3 m to Elev. 412.6 m. The explored thickness of this layer was between 0.1 m to 0.7 m. Probe holes HP20-4, HP20-6, HP20-6I, HP20-7, HP20-10 and HP20-11 were terminated within this layer due to auger refusal.

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

Laboratory testing performed on a selected sample consisted of twelve (12) moisture content tests and six (6) grain size distribution tests. The test results are as follows:

Moisture Content:

- 9% to 24%

Grain Size Distribution:

- 3% to 25% gravel;
- 50% to 79% sand; and
- 15% to 33% silt and clay

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

1.4.2.4 Sandy Gravel

Native sandy gravel was encountered in probe hole HP20-5 below the topsoil. The sandy gravel layer extended to depth of about 0.6 m below ground surface with elevation of about Elev. 413.2 m. The explored thickness of this layer was 0.1 m. Probe hole BH20-5 was terminated within this layer.

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

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

Moisture Content:

- 19%

Grain Size Distribution:

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

The results of the moisture content and grain size distribution test are provided on the record of borehole sheets in Appendix C. The results of the grain size distribution test is also provided on Figure 7 in Appendix D.

1.4.2.5 Refusal

Drilling refusal was encountered at the depth of 0.2 m to 0.8 m below the existing ground at all locations of hand probe holes. The presence of bedrock and/or buried rock fill below the soils was suspected based on observed bedrock outcrops as well as boulders and cobbles sized rock pieces.

1.5 Groundwater and Surface Water Conditions

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

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

Groundwater levels would be expected to reflect levels in the adjacent open water and to fluctuate seasonally. Seasonal variations in the water table should be expected, with higher levels occurring during wetter periods of the year and lower levels during drier periods.

1.6 Chemical Analysis

Three soil samples were selected for chemical analysis and they were sent via courier, in a secure cooler under chain of custody, to BV Labs (formerly Maxxam Analytics Inc.), a CALA-certified and accredited laboratory in Mississauga, Ontario. The analytical laboratory results are presented in Appendix F, and are summarized in Table 1.2, below.

Table 1.2. Corrosivity chemical analysis

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

2 ENGINEERING DISCUSSION & RECOMMENDATIONS

2.1 General

This section of the report provides geotechnical design recommendations for replacement of the centerline culvert on Highway 599 (Sta. 78+603), about 4.6 km south of Savant Lake CNR crossing within the District of Thunder Bay, Ontario, the Ministry of Transportation (MTO) Northwestern Region. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the current investigation at the site performed by EXP. The compiled factual data is presented in **Part I-Foundation Investigation Report** of this report. The interpretation and recommendations provided are intended solely to permit designers, to assess foundation alternatives and design the new culvert and replacement. Comments on construction are only provided to highlight issues that could affect the design. Contractors bidding on the works should make their own assessments of the factual data and how it might affect construction means and methods, scheduling and the like.

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

At the time of preparation of this Foundation Investigation and Design Report, per the Terms of Reference (TOR) for this assignment, the type of new culvert and method of installation are not known. However, it is understood that the new culvert will be a structural culvert installed at the same location of the existing culvert having the same or similar elevations of the invert at the inlet and outlet sides (Elev. 412.3 m and Elev. 411.7 m, respectively). In addition, knowing that the existing non-structural culvert will be replaced with a structural culvert it is assumed that the new culvert will have a span of minimum 3 m. It is also assumed that the new embankment will be constructed with no grade change. According to the TOR, widening at the culvert location (top elevation of Elev. 416.6 m) should also be considered. Due to depth considerations, temporary protection systems and temporary cofferdams are assumed to be used for dewatering and water diversion during culvert replacement.

This part of the report addresses the geotechnical design of the foundation for the new culvert by providing geotechnical design parameters at the Ultimate Limit State (ULS) and Serviceability Limit States (SLS) as well as other geotechnical parameters that may be required in accordance with the latest edition of the *Canadian Highway Bridge Design Code (CHBDC) (CAN/CSA-S6-19)*, the *Canadian Foundation Engineering Manual (CFEM)* (2006), *MTO Gravity Pipe Design Guidelines* (May 2007) and generally accepted good practice. Pertinent construction issues from a geotechnical standpoint are examined in general accordance with the TOR provided to us in the MTO email dated September 17, 2020. The assessment involved review of options for replacement of the existing culvert along the proposed alignment. The protection of construction site by temporary protection systems and 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 599 is a two lane, west/east roadway (~ 7.2 m wide) with approximately 1 to 1.5 m wide shoulders at the culvert location. The highway crosses a 1.5 m x 1.5 m wooden box culvert with two overflow CSP culverts on both sides. Embankment fill above the culvert crown is approximately 2.8 m thick. The current elevation of the crest of roadway embankment is about Elev. 416.6 m and side slopes of between 1.1H:1V to 1.3H:1V.

b) Below the road surface which consist of 0.025 m (~1 inch) thick asphalt treatment, the highway embankment consists of compact to very dense sand and gravel fill (~0.8 to 2.3 m thick) to compact gravelly sand (~2.2 to 4.2 m thick) below the road surface, followed by very dense native sandy silt with gravel layer (~1.1 m thick) followed by silty sand with gravel till (~0.9 m thick) underlain by bedrock at about 2.3 m to 7 m (sloping bedrock) below the ground surface.

c) At the inlet, below the topsoil (~0.1 to 0.6 m thick), a layer of native silty sand (~0.7m thick) /sandy gravel layers (~0.2m thick) below the ground surface. At the outlet, below the peat and sand (~0.3 m thick) / topsoil (~0.3 m thick), a layer of silty sand with gravel (~0.2 to 0.4 m thick). It should be noted that the hand probes were terminated upon auger refusal and that the bedrock is shallow at the inlet and outlet. Bedrock outcrop was observed about 3 m and 1.5 m from the location of hand probe holes at the inlet side and outlet side, respectively. Some rock fill was present at the toes of the embankment.

d) The invert of the new culvert is proposed to be at same or similar elevation as the existing culvert Elev. 412.3 m and Elev. 411.7 m at the inlet and outlet, respectively. The foundation soil below the culvert is anticipated to be bedrock or native very dense sandy silt with gravel followed by dense to very dense silty sand with gravel till followed by bedrock. Rock fill was present below the culvert CSP extension at the outlet side.

e) At the time of investigation (November 2020), the top of the creek water was at Elev. 412.7 m and Elev. 410.9 m at the inlet and outlet side of the existing culvert, respectively. The groundwater table measured in the boreholes drilled from the roadway was observed to be at depths of about 3.1 m below ground surface, corresponding to Elev. 413.4 m. However, seasonal variations in the water table should be expected, with higher levels occurring during wetter periods of the year (such as spring thaw and late fall) and lower levels during drier periods. The water marks inside the culvert (see Photos 1, 2 and 5 in Appendix A) suggest that the level of water in the pipe could be up to the spring line of the pipe (~Elev. 413.1 m at inlet and ~Elev. 412.5 m at outlet).

2.3 Structure Foundations

Based on available information about the new culvert (i.e. a structural culvert with a spam more than 3 m) and stratigraphy at the site, the following options were considered as possible options for the culvert replacement and they are discussed below:

1. Pipe culvert (CSP, concrete or HDPE)
2. Precast rigid frame concrete box culvert
3. Cast-in-place rigid frame concrete box culvert
4. 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 dense sandy silt with gravel and bedrock is considered suitable for support of all replacement options. However, the choice of culvert type also depends on parameters such as the initial cost, maintenance costs, hydraulic performance, fish passage requirements, ease of construction, water and soil corrosiveness, salvageability and local availability of material and equipment.

It is noted that regardless of the option selected, the existing culvert is to be removed. This will require excavation down to the existing founding elevation for all options (~Elev. 412 m). This suggests the need for surface/groundwater control as discussed in Sections 2.8 and 2.12 below.

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 foundation soils can provide adequate support for all options listed in the table, the use of precast rigid frame concrete box culvert is ranked highest for the criteria evaluated.

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

Table 2.1 Evaluation of foundation alternatives

Options	Rank	Advantages	Disadvantages	Relative Costs	Risks/ Consequences
1. Pipe culvert	4	▪ Straightforward construction ▪ Reduced construction period, consequently, traffic management and water control period ▪ Reduced excavation depth	▪ Requires bedding material ▪ Limited design life ▪ Potential for corrosion if CSP is used	▪ Low to medium	▪ Possibility not to meet hydraulic and fish passage requirements ▪ Risk of structure segment loss due to corrosion if CSP is used
2. Precast rigid frame concrete box culvert	1	▪ Straightforward construction ▪ Reduced construction period; consequently, traffic management and water control period ▪ Reduced excavation depth ▪ Can be more readily installed during cold weather conditions	▪ If floor is thin and poorly reinforced, it may heave and cracks ▪ During high flows, the concrete floor can be undermined ▪ Susceptible to defects/leakage at joints ▪ Requires bedding material ▪ Possible sediments accumulation in the upstream of the culvert	▪ Low	▪ Risk of leaking from joints if not properly installed

Options	Rank	Advantages	Disadvantages	Relative Costs	Risks/ Consequences
3. Cast-in-place rigid frame concrete box culvert	2	▪ Suitable if site is not appropriate to heavy equipment for installation of precast sections ▪ Reduced excavation depth ▪ Culvert design can be customized in the field for high stress or load conditions or other site-specific requirements	▪ Slower construction process ▪ If floor is thin and poorly reinforced, it may heave and crack ▪ During high flows, the concrete floor can be undermined ▪ Requires concrete curing ▪ Possible sediments accumulation in the upstream of the culvert	▪ Likely more expensive than Option 2	▪ Risk of unacceptable differential settlements if the entire foundation is not supported on the competent soil ▪ Risk of disturbance of base during construction
4. Cast-in-place rigid frame open footing concrete culvert supported on shallow foundations	3	▪ Wider span may consider to provide a fish passage ▪ Less accumulation of sediments in the upstream of culvert	▪ Deeper excavation or below water excavation may required ▪ Dewatering system required	▪ Likely more expensive than Options 2 and 3	▪ Risk of delay in construction due to deeper excavation below water if proper dewatering is not maintained ▪ Higher scour risk

2.3.1 Shallow Foundations

2.3.1.1 Geotechnical Resistance

Based on the subsurface stratigraphy encountered at this site and the 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 (CAN/CSA-S6-19) and its commentary (Clause 6.10.3 and Clause 6.10.4). The geotechnical resistances provided in sections below were factored with typical consequence factor of 1.0 at ULS and SLS (Table 6.1 of CHBDC *CAN/CSA S6-19*), and factors of 0.5 at ULS and 0.8 at SLS for typical degree of understanding (Table 6.2 of the CHBDC *CAN/CSA S6-19*).

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 is planned and presence of shallow bedrock, 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.

Table 2.2 Recommended spread footing design parameters

Culvert Type	Reference BH	Founding Elevation (m)	Footing Size (m)	Founding Soil Type	Factored Geotechnical Resistance at ULS (kPa)	Geotechnical Reaction at SLS[2] (kPa)
Pipe culvert, precast or cast-in-place rigid frame concrete box culvert	BH20-2	~412	3.0	Very dense sandy silt with gravel	600	400
Cast-in-place rigid frame open footing concrete culvert		~409.6[1]	1.0	Bedrock	4000	N/A

Notes:
1. *Below the frost line*
2. *For maximum settlement of 25 mm*

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 (CAN/CSA S6-19), using the following parameters:

Table 2.3 Recommended parameters for calculation of unfactored horizontal resistance

Interface and Loading Conditions	Parameters
Between pre-cast concrete and Granular A	Coefficient of friction (tan δ)=0.50
Between cast-in-place concrete and Granular A	Coefficient of friction (tan δ)=0.55
Between Granular A and native sandy silt with gravel	Coefficient of friction (tan δ)=0.35

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

2.3.1.3 Frost Protection

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

Since the earth cover above the top of the proposed culvert is estimated to be about 1.5 m which is less than the frost penetration line of 2.6 m, frost tapers are required at this site. It should be installed in accordance with OPSD 803.031.

2.4 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:

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

where,

- P = earth pressure intensity at depth h, kPa
- K = earth pressure coefficient
- γ = unit weight of retained soil, kN/m^3
- 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 ϕ' (°)	Coefficient of Active Earth Pressure (K_a)	Coefficient of Passive Earth Pressure (K_p)	Coefficient of Earth Pressure At- Rest (K_o)	Unit Weight γ (kN/m^3)
Granular A	35	0.27	3.69	0.43	22
Granular B Type II	35	0.27	3.69	0.43	22
Sand and Gravel Fill (compact to very dense)	32	0.31	3.25	0.47	21
Gravelly Sand Fill (compact)	32	0.31	3.25	0.47	21
Sandy Silt with Gravel (very dense)	34	0.28	3.54	0.44	22
Silty Sand with Gravel Till (dense to very dense)	34	0.28	3.54	0.44	22

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 and Liquefaction Potential Considerations

Seismic characterization of the site must be compliant with the CHBDC (CAN/CSA S6-19). The potential for seismic loading must be considered for design in accordance with Section 4.4 of the CHDBC with respect to soil conditions encountered at the site. Table 4.1 in the 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 2 m thick very dense sandy silt with gravel and silty sand with gravel till underlain by bedrock. Based on these soil characteristics, the site class for this site is estimated conservatively to be Class “C” according to Table 4.1.

From the Natural Resources Canada website, 2015 NBCC seismic hazard values are obtained using the site location coordinates (50.201°N, 90.726°W) and the defined damped reference spectral accelerations for the project site are included in Appendix I. As can see from the attached sheet, the PGA at this site for a reference Site Class C with a 2% probability of exceedance in 50 years (2475-year event) is 0.044 g.

Considering the shallow bedrock at the site, no liquefaction is expected due to the ground motion from an earthquake having 2% probability of exceedance in a 50-year period.

2.6 Construction Alternatives

Assuming that there is no local detour available to divert the traffic, the following 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. Temporary widening on one side of highway embankment to construct temporary detour at the site followed by staged open cut/unsupported excavation to expose and replace culvert
3. Staged construction using temporary modular bridge

All methods considered to 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/Risk	Ranking
OPTION 1.A Half-and-half Construction with Unsupported Cut Sides (Figure H1.A, Appendix H)	• Traffic flow maintained at the site during construction • Short mobilization time • Straight forward construction and construction procedures	• Traffic interruption • Roadway protection of up to 5 m high required to maintain one lane of traffic • Shallow bedrock • Require dewatering to provide safe temporary cut slopes • High cost of roadway protection system • Large amount of soil to be excavated • Need to temporarily control existing creek water • Risk of cost overrun and inability to finish job: moderate to high	Relatively less expensive and destructive than Option 2 since the extension of construction/excavation is less; Less expensive than Option 3, but only one lane of traffic can be maintained, existing culvert cannot be used to convey creek water and difficult installation of TPS due to shallow bedrock	3
OPTION 1.B Half-and- half Construction with Braced or Anchored Cut Sides (Figure H1.B, Appendix H)	• One or possibly two lanes of traffic flow maintained on existing road (e.g. steel decking, but costly) • Global stability of excavation enhanced by narrow geometry • Less traffic interruption than with unsupported cut sides approach • Temporary decking could be usable over braced cut to allow for excavation of both halves prior to diverting stream and backfilling • Cost savings due to limited excavation and backfill • Cost saving due to no need for temporary cut slopes and extensive dewatering	• Traffic interruption • Roadway protection of up to 5 m high required to maintain one lane of traffic if steel decking is not possible • High cost of roadway protection system and/or decking • Require side shoring and bracing • Bracing (e.g. struts) may interfere with excavation • Excavation of material and placement of bracing required in limited space • Need to decommission the shoring system • Need to temporarily control existing creek water	More expensive than other options due to high costs for shoring system and temporary decking (if feasible) to maintain continuous flow of traffic, however, global stability of excavation enhanced by shoring and no need for extensive dewatering	4

Installation Method	Advantages	Disadvantages	Relative Cost/Risk	Ranking
		• Risk of cost overrun and instability to finish job: low to moderate		
OPTION 2 Temporary widening of Highway 599 to construct local detour and open cut unsupported excavation – staged construction (see Figure H2 in Appendix H)	• Traffic flow maintained at the site during construction • Simple detour roads can be constructed • Existing culvert will completely remove and replaced with new culvert • No excavation support required	• Traffic interruption • Staged construction required- widening of one side of embankment to replace another half of culvert • Increased time for construction of detours and staging • Erosion control of temporary cuts required • Need to temporarily control existing creek water • Risk of cost overrun and inability to finish job: low to moderate	More expensive than Option 1A since requires extensive widening and construction of detours (i.e. ~ 50 m east and 50 m west)	2
OPTION 3 Staged Construction Using Temporary Modular Bridge (see Figure H3 in Appendix H)	• Traffic flow maintained at the site during construction without construction of local detour • No earth embankment fill material is required for building detours • No need for construction of the temporary support for excavation of the existing embankment which may not be possible due to shallow bedrock • No settlement since there is no new earth embankment fill • No need to temporary divert surface water flow since the existing overflow CSP culvert can be used to maintain the surface water flow • Removal of existing culverts	• Traffic interruption • Large amount of embankment fill to be excavated and replaced; Road has to be excavated longitudinally approximately 20 m at the top with forward slopes of 1.5H:1V • Additional cost for Temporary Modular Bridge and its foundations • Increased time for construction of staging • Erosion control of temporary cuts required • Risk of cost overrun and inability to finish job: low to moderate	More expensive than full road closer due to costs of Temporary Modular Bridge and foundation construction	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 3: Staged construction using temporary modular bridge (Figure H3 in Appendix H)
2. OPTION 2: Temporary widening with staged open cut unsupported excavation (Figure H2, Appendix H)
3. OPTION 1A: Half-and-half construction with unsupported cut sides (Figure H1.A, Appendix H)
4. 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 Half-and-Half Construction

The half-and-half construction method could be utilized to maintain the flow of the traffic on Highway 599 (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 5 m deep. Therefore, temporary shoring such as a soldier pile and lagging 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.PROV 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

2.6.1.1 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 2 soil above the groundwater table in conformance with the OHSA.

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.

2.6.1.2 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 in 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.

2.6.2 Widening with Staged Open Cut Unsupported Excavation

By widening of the existing embankment and formation of the detour at the site to maintain the local flow of traffic during the replacement (see Figure H2, Appendix H), allows for open cut, unsupported excavation to facilitate the replacement of the existing culvert. The major advantage is that neither excavation support nor roadway protection is required with this option. The major disadvantages of this option are traffic interruption, increased time for construction of detour and staging, and need for temporary construction of local unwatering and dewatering systems (i.e. cofferdams, and sumps and pumps, etc.) to prevent creek water and groundwater flow into the construction area.

Option 2 involves construction of temporary (or permanent) embankment widening at one side of the existing embankment. Compacted engineered fill for construction of the widening is recommended. Based on the results of slope stability analyses performed for this Option 2, it is recommended that the side slopes of the widening should not exceed 1.5H:1V to maintain the minimum global factor of safety of 1.3 for static condition, as shown on Figure G3 in Appendix G. However, the Contractor should perform their own stability analyses for temporary embankment widening as specified in the NSSP attached in Appendix K (NSSP for Slope Stability Analyses Required for Temporary Embankment Widening). In addition, prior to construction of widening, the slope sides of the existing embankment will need to be cleared and grubbed of any existing bushes and vegetation. All surficial topsoil (if exists), organics and softened or loosened soil should be stripped from below the proposed widening footprint. All subgrade soils should be proof-rolled prior to fill placement and embankment fill should be placed in accordance with OPSS.PROV 206 (dated November 2014). The widening portion of the embankment should be key into the existing slope as indicate in OPSD 208.010.

If the existing highway is widened for staged construction of new culvert, some settlement of the foundation soils might be expected due to an additional load. However, it is anticipated that that settlement will be less insignificant (i.e. less than 25 mm) considering the shallow bedrock, but monitoring of settlement using surface monitoring points installed on the top of temporary road is recommended.

2.6.3 Staged Construction Using Temporary Modular Bridge

Based on the above list of advantages and disadvantages, the staged construction using the Temporary Modular Bridge (TMB) is considered as the most viable method from practical, geotechnical and/or foundation perspectives. The major advantages of this cut and cover approach are (i) possibility to maintain traffic flow at the site during construction without construction of local detour, (ii) no earth embankment fill material is required for building detours, (iii) no need for construction of the temporary support for excavation of the existing embankment which may not be possible due to shallow bedrock, (iv) possibility to assess the foundation soil below the culvert location, and (v) to remove the existing culvert. On the other hand, the major disadvantages are (i) traffic interruption with one-way traffic, (ii) large excavation of the rockfill, and (iii) cost of the Temporary Modular Bridge.

Based on the results of slope stability analyses performed for this Option 3, it is recommended that the forward slopes of the bridge abutment should not exceed 2H:1V to maintain the minimum global factor of safety of 1.5 for static condition, as shown on Figure G4 in Appendix G.

Figure H.3 in Appendix H shows schematically the stages of the recommended cut and cover approach construction method - staged construction using the Temporary Modular Bridge. As can be seen, the TMB can be used for this project as the following staging:

STAGE 1:

(i) Close EBL with a traffic signal and shift the one-way traffic to the WBL
(ii) At the EBL area, excavate and place TBM footings
(iii) Launch TBM

STAGE 2:

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

STAGE 3:

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

STAGE 4:

(i) Switch traffic to the reconstructed side (WBL)
(ii) Remove the TMB
(iii) Backfill and reinstate roadway to final configuration

2.6.3.1 **Temporary Modular Bridge Foundations**

If a temporary modular bridge is considered to be built at one side of Highway 599, it is recommended to drill at least two additional boreholes at locations of proposed TMB abutments to facilitate design of TMB foundations. Data obtained in BH20-3 could be used for design of shallow temporary modular bridge locations if the bridge abutment will be located in the vicinity of that borehole. Therefore, for the preliminary purpose, based on the current information from BH20-3 it can be expected that the shallow spread footings (having minimum width of 2 m) on mass concrete placed on the dense to very dense gravelly sand fill can be recommended with the following geotechnical resistances:

- Factored Geotechnical Resistance at ULS of 300 kPa
- Geotechnical Reaction at SLS of 200 kPa

The geotechnical resistances provided above are given under the assumptions that the stratigraphy in additional boreholes and BH20-3 are similar, as well as for concentric vertical loading condition only. Where the load is not concentric vertical loading, load eccentricity and load inclination effects need to be considered.

2.6.3.2 **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 2.6 Recommended parameters for calculation of unfactored horizontal resistance

Interface and Loading Conditions	Parameters
Between cast-in-place concrete and Gravelly Sand Fill	Coefficient of friction (tan δ)=0.50

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

2.6.3.3 **Frost Protection**

The frost depth in the area of the temporary modular bridge is the same as that estimated in Section 2.3.1.3. A minimum 2.6 m of soil cover or equivalent frost protection should be provided using thermal insulation.

2.7 Temporary Roadway Protection

At this site, temporary roadway protection is anticipated to be a part of the half-and-half construction approach if that method is selected to maintain on-site traffic during the construction. It is recommended that roadway protection system should be design in accordance with OPSS.PROV 539. The complete design, construction, monitoring and removal of the installed protection system should be a responsibility of the Contractor. Due to 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.

Since the highway embankment at this site is underlain by shallow sloping bedrock, the shoring system such as soldier piles and timber lagging may be considered for design. The shoring system should be designed based on the earth pressures coefficients and soil parameters provided in Section 2.4. Considering the height of the roadway embankment and the depth of bedrock, a temporary shoring system with additional anchorage or tiebacks may be required for lateral resistance. Conventional practice is to incorporate either buried deadman anchors or soil grouted anchors. Alternatively, a system of rakers can be used for support. Deadman anchors can be also designed based on the earth pressure coefficients and soil parameters provided in Section 2.4. For this project, either continuous or individual concrete block anchors would likely be appropriate. The anchor resistance is provided by a combination of the dead weight and passive resistance. For the full passive resistance to be realized with no load transfer to the wall, the anchor needs to be fully beyond the active wedge acting on the wall. Pressure grouted soil anchors can be also designed in a preliminary fashion in accordance with Section 26 of the CFEM. Based on the generally compact to dense fill soils at this site, the estimated factored (0.4) ULS resistance of grouted anchors would be 90 kN/m length. Detailed design would be completed following the design of the wall and the loads have been established. Normally, such anchors are supplied and installed/tested by specialist vendors/contractors.

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

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

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

2.8 Site Dewatering - Cofferdams

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

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

Depending on the topography and overland flow drainage path, the existing creek should be diverted away from the construction site during the replacement of the existing culvert. Depending on the creek water level and surface water flow conditions at the time of construction, one of the existing overflow CSP culverts can be used to divert the creek water flow. Otherwise, a system of sumps and pumps might be required to divert the surface water up and over the existing embankment.

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 and gravelly sand fills may be classified as a Type 3 soil above the groundwater table in conformance with the OHSA. The native sandy silt with gravel and silty sand till soils above the groundwater table may be classified as a Type 2 soil and Type 3 soil below the groundwater table. It is expected that most of excavations will be above the groundwater levels except those at the invert level. 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 2 and 3 soils should not exceed 1H:1V in accordance with OHSA. There is a potential for sloughing to occur if the trench remains open for an extended period of time (i.e. > 24 hours) or during a rainfall event. In addition, some localized surficial sloughing may be experienced in areas of perched groundwater seepage (i.e. within the embankment fill).

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

2.10 Culvert Bedding

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

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

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

2.11 Culvert Backfill

The selection and placing of the backfill and cover should be in accordance with OPSS.PROV 902, OPSS.PROV 421, OPSS.PROV 422 and OPSDs 803.010 and 3101.150 for different culvert material. The backfill should consist of free-draining, non-frost susceptible granular materials 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 2.6 m from any finished road grade, approved compactable fill, such as select subgrade materials (OPSS.PROV 1010) or imported fill can be used.

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

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

Where less than frost depth (2.6 m) of earth cover is provided above the top of the culvert, a frost taper should be included as per OPSD 803.030, 803.031, and MTOD 803.021, whichever is applicable.

2.12 Groundwater Water Control

The groundwater level at the site was encountered around Elev. 413.2 m, while the excavation to the foundation level has to be carried out to Elev. 411.2 m. Therefore, the groundwater table could be above the bottom of excavation. The soils encountered on the site and within potential excavation depth consist of fill and native non-cohesive soils which the estimated range of hydraulic conductivity (k) could be around 10^{-3}-10^{-5} m/s.

Culvert construction, subgrade preparation and placement and compaction of granular bedding must be carried out in the dry; it is recommended that the groundwater be lowered to 0.5 m below the final culvert subgrade elevation. Furthermore, surface runoff will tend to seep into and accumulate into the excavations. The Contractor must control groundwater, perched groundwater and surface water flow at the site to permit the replacement of the culvert in a dry and stable excavation. Dewatering can be difficult to achieve for flow over and within the bedrock. However, a suitable sump and pump system behind the cofferdam will be required to remove any accumulation of water from the footing base prior to placing the culvert bedding.

As noted before in Section 2.8, creek channel should be diverted and cofferdam should be built prior to any excavation of the existing culvert. Dewatering requirements behind the cofferdams to keep the construction site dry will be impacted by water levels in the creek at the time of construction activities. Dewatering shall be carried out in accordance with OPSS.PROV 517, SP 517F01 (Construction Specification for Dewatering of Pipeline, Utility, and Associated Structure Excavation) and OPSS 518 (Construction 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.

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

Erosion and sediment control during culvert construction should be as per the MTO Drainage Manual, Volume 2. Silt fences and other sediment control measures should be included to protect the downstream environment from the construction activities. If 50,000 litres (L) or more of water per day was taken from the environment (including groundwater, lakes, rivers, ponds, etc.), a Permit to Take Water (PTTW) is generally required as per the Ontario Water Resources Act and Ontario Regulation 387/04. Further assessment of dewatering requirements and the need for the PTTW should be assessed in the hydrogeology report under a separate cover.

2.13 Embankment Reconstruction

The roadway embankment immediately adjacent to the culvert should be reconstructed in accordance with OPSS.PROV 206 using suitable earth borrow material as per OPSS.PROV 212 and/or OPSS.PROV 1010 Granular A or Granular B Type I. The existing embankment fill and the new fill along the existing roadway embankment slopes should be integrated in accordance 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 similar geometry as the previous embankment at the location of the culvert. The global stability and settlement of that embankment with 2H:1V side slopes was addressed in the sections below. If the embankment will be widen the additional fill will be placed along the widening side.

Prior to the placement of new fill for the embankment widening, the sites will need to be cleared and grabbed of the existing trees and bushes. All surficial topsoil, organic (i.e. muskeg/peat), loose, soft and/or deleterious materials should be stripped from bellow the proposed embankment widening areas. The exposed subgrade should be proofrolled under the direction of qualified geotechnical personnel.

2.13.1 Stability Analysis

Slope stability analyses were performed to assess the global stability of the final embankment configuration in order to check that a minimum Factor of Safety of 1.5 for static condition and 1.1 for seismic condition will be achieved. For temporary widening of embankment, a minimum Factor of Safety of 1.3 for static condition is required as mentioned above in Section 2.6.2. For Temporary Modular Bridge (TMB), a minimum Factor of Safety of 1.5 for static condition is required for forward slope as mentioned above in Section 2.6.3. The static and seismic slope stability analyses were performed using the Morgenstern-Price method developed on the basis of limit equilibrium. The SLOPE/W computer program developed by GeoSlope International (Geostudio 2018 version 9.0.2.15352) was employed for computation.

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

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

Tabulated below in Table 2.7 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 2.7 Soil properties used in slope stability analyses

Soil Type	Effective Stress Parameter	
	γ (kN/m^3)	ϕ' (degrees)
Granular A	22	35
Granular B Type II	22	35
Sand and Gravel Fill (compact to very dense)	20	32
Gravelly Sand Fill (compact)	20	32
Sandy Silt with Gravel (very dense)	21	34
Silty Sand with Gravel Till (dense to very dense)	21	34
Silty Sand with Gravel (compact to dense)	20	33
Sandy Gravel (compact)	20	32

The slope stability analyses were performed for existing embankment, embankment widening construction option on the north side, Temporary Modular Bridge (TMB) construction option and new embankment, approximately 4.5 m high. The results of these analyses are shown in Figures G1 to G8 in Appendix G and summarized in Table 2.8 below. Based on the results of slope stability analyses for existing embankment, it appears that the existing slopes (i.e. 1.1H:1V at the outlet side and 1.3H:1V at the inlet side) do not meet the required minimum factor of safety (i.e. FOS=1.5); therefore, construction of new embankment side slopes should be flattened. Based on the results of slope stability for the widening construction option, the widening embankment can safely be constructed with 1.5H:1V side slope, if that option is selected. If TMB construction option is selected, the 2H:1V excavation forward slope for the TMB appears to be safe. Further, it was calculated that the minimum factors of safety of critical slip surfaces for the new 4.5 m high embankment meet the design criteria for static and seismic conditions (i.e. FOS=1.5 for static and FOS=1.1 for seismic condition) if its side slopes are 2H:1V.

Table 2.8 Summary of results of slope stability analyses

Locations	Max Height (m)	Conditions	Min FOS
North side of existing embankment – Inlet (Side slopes 1.3H:1V)	~4.5	drained long-term conditions, static condition	1.1
South side of existing embankment – Outlet (Side slopes 1.1H:1V)	~4.5	drained long-term conditions, seismic condition	1.0
Temporary widening on North side of embankment – Inlet (Side slopes 1.5H:1V)	~4.5	drained long-term conditions, static condition	1.3
Temporary Modular Bridge on South side of embankment (forward slopes 2H:1V)	~4.5	drained long-term conditions, static condition	1.5
North side of embankment – Inlet (Side slopes 2H:1V)	~4.5	drained long-term conditions, static condition	1.5
		drained long-term conditions, seismic condition	1.4
South side of embankment – Outlet (Side slopes 2H:1V)	~4.5	drained long-term conditions, static condition	1.5
		drained long-term conditions, seismic condition	1.4

2.13.2 Embankment Settlement

Since the bedrock is shallow at this site and it is not planned to change the existing embankment grade, the settlement of foundation soils will be negligible under the existing embankment. However, fill for widening of the existing highway embankment, if any, might induce some settlement. However, noting no grade raise and presence of shallow bedrock and/or predominantly dense to very dense granular foundation soils, the resulting settlement is expecting to occur mainly during construction in the order of 25 mm or less. The post construction settlement of the widened embankment in these areas are expected to be minimum (i.e. less than 50 mm which is specified as a maximum limit during pavement design life in Table 1.3 - Post-Construction Settlement Criteria for Embankment Widening from Embankment Settlement Criteria for Design- July 2, 2010, MTO. It is also estimated that the differential settlement rate for both new widened embankment as well as the differential settlement rate between the existing and the new embankment will be less than 200:1 as per Table 1.3 mentioned above.

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 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. The proposed foundation design at this site for the centerline culvert replacement incorporates shallow foundations and requires such assessment and/or protection.

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 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.PROV 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 soils around and below the culvert bedding can migrate 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 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.

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 x 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 x (Liquid Limit – 20%).

- The clay seal is to be place 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. 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

Three (3) soil samples were selected for chemical analyses during this investigation. The testing was completed to determine the potential degradation of the concrete in the presence of soluble sulphates and the potential of corrosion of exposed steel used in foundations and buried infrastructure. The analyses results are summarized in Table 2.1 of this report.

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

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

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

3 CLOSURE

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

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

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

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

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

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

Ministry of Transportation, 2007. MTO Gravity Pipe Design Guidelines

Ontario Geological Survey, 1991. Map 2554: Quaternary Geology of Ontario, West-Central Sheet

Ontario Geological Survey, 1991. Map 2542: Bedrock Geology of Ontario, West-Central Sheet

ASTM International:

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

Ontario Provincial Standard Specifications (OPSS):

OPSS.PROV 206	Construction Specification for Grading
OPSS.PROV 401	Construction Specification for Trenching, Backfilling, And Compacting
OPSS.PROV 501	Construction Specification for Compacting
OPSS.PROV 517	Construction Specification for Dewatering
OPSS.PROV 539	Construction Specification for Temporary Protection Systems
OPSS.PROV 902	Construction Specification for Excavating and Backfilling – Structures
OPSS.PROV 1010	Material Specification for Aggregates - Base, Subbase, Select Subgrade, And Backfill Material
OPSS.PROV 421	Pipe Culvert Installation in Open Cut
OPSS.PROV 422	Precast Reinforced Concrete Box Culverts in Open Cut
OPSS 518	Control of Water from Dewatering Operations
OPSS.PROV 212	Earth Borrow
OPSS.PROV 802	Topsoil
OPSS.PROV 803	Vegetative Cover
OPSS.PROV 804	Temporary Erosion Control
OPSS.PROV 1205	Clay Seal

Special Provisions (SP):

SP 517F01	AMENDMENT TO OPSS 517; Construction Specification for Dewatering of Pipeline, Utility, and Associated Structure Excavation
NSSP FOUN0003	Dewatering Structure Excavation

Ontario Provincial Standard Drawings (OPSD)/Ministry of Transportation Ontario Drawing (MTOD):

OPSD 208.010	Benching of Earth Slope

OPSD 3090.100	Foundations, Frost Penetration Depths for Northern Ontario
OPSD 3101.150	Walls, Abutments, Backfill, Minimum Granular Requirement
OPSD 802.010	Flexible Pipe Embedment and Backfill, Earth Excavation
OPSD 803.010	Backfill and Cover for Concrete Culverts with Spans Less Than or Equal to 3.0 m
MTOD 803.021	Bedding and Backfill for Precast Concrete Box Culverts
OPSD 803.030	Frost Treatment, Pipe Culverts, Frost Penetration Line below Bedding Grade
OPSD 803.031	Frost Treatment, Pipe Culverts, Frost Penetration Line between Top of Pipe and Bedding Grade
OPSD 810.010	General Rip-Rap Layout for Sewer and Culvert Outlet

Ontario Water Resources Act:

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

Ontario Occupational Health and Safety Act (OHSA):

Ontario Regulation 213/91 Construction Projects

LIMITATIONS AND USE OF REPORT

BASIS OF REPORT

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

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

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

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

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

RELIANCE ON INFORMATION PROVIDED

The evaluation and conclusions contained in the Report are based on conditions in evidence at the time of site inspections and information provided to EXP by the Client and others. The Report has been prepared for the specific site, development, building, design or building assessment objectives and purpose as communicated by the Client. EXP has relied in good faith upon such representations, information and instructions and accepts no responsibility for any deficiency, misstatement or inaccuracy contained in the Report as a result of any misstatements, omissions, misrepresentation or fraudulent acts of persons providing information. Unless specifically stated otherwise, the applicability and reliability of the findings, recommendations, suggestions or opinions expressed in the Report are only valid to the extent that there has been no material alteration to or variation from any of the information provided to EXP.

STANDARD OF CARE

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

COMPLETE REPORT

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

USE OF REPORT

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

REPORT FORMAT

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

Appendix A –
Site Photographs

Photograph 1. Inlet side of the culvert, October 2020

Photograph 2. Outlet side of the culvert, October 2020

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

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

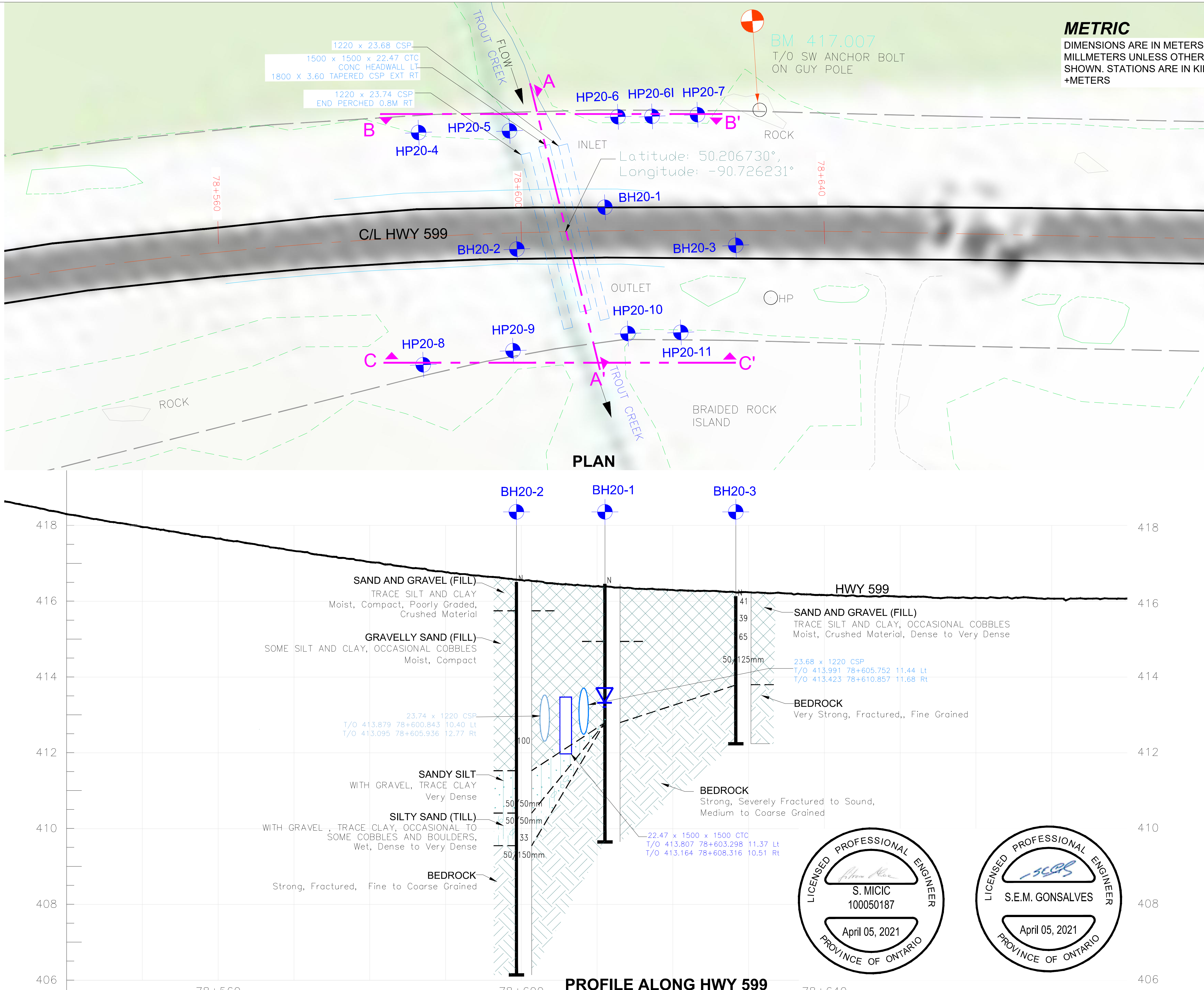
Photograph 5. Inside of the centreline culvert at outlet, October 2020

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

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

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

Appendix B – Drawings



Agreement No. 6019-E-0004/0005
GWP No. 6530-17-00
Assignment No. 1

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

SHEET
1

exp. **EXP Services Inc.**

KEY PLAN

LEGEND

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

SOIL STRATA SYMBOLS

TOPSOIL
SILTY SAND
SILTY SAND WITH GRAVEL
FILL
SANDY SILT
SILTY SAND WITH GRAVEL (TILL)
SANDY GRAVEL
PEAT AND SAND
BEDROCK

BH No.	ELEV.	MTM CO-ORDINATES (*ZONE ON-15*)	
		NORTHING	EASTING
BH20-01	416.46	5563540.7	252977.6
BH20-02	416.51	5563531.1	252968.2
BH20-03	416.13	5563542.0	252994.7
HP20-04	414.16	5563540.8	252950.6
HP20-05	413.89	5563545.4	252961.8
HP20-06	413.77	5563552.9	252974.1
HP20-06I	413.99	5563553.7	252978.1
HP20-07	414.47	5563556.5	252984.2
HP20-08	411.86	5563512.3	252962.4
HP20-09	411.60	5563518.9	252972.7
HP20-10	413.12	5563526.4	252986.0
HP20-11	414.96	5563529.1	252992.1

NOTES

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

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

SCALE: HOR 0 5 15 m
VERT 0 1 3 m

DATE	BY	DESCRIPTION
	SM	SUBMISSION FOR MTO REVIEW

GEOCRES NO. 52J-19
PROJECT NO. ADM-00262199-A0

SUBM'D SH	CHECKED SM	DATE April 05, 2021	
DRAWN SH	CHECKED SM	APPROVED SG	DWG. 1

Agreement No. 6019-E-0004/0005
GWP No. 6530-17-00
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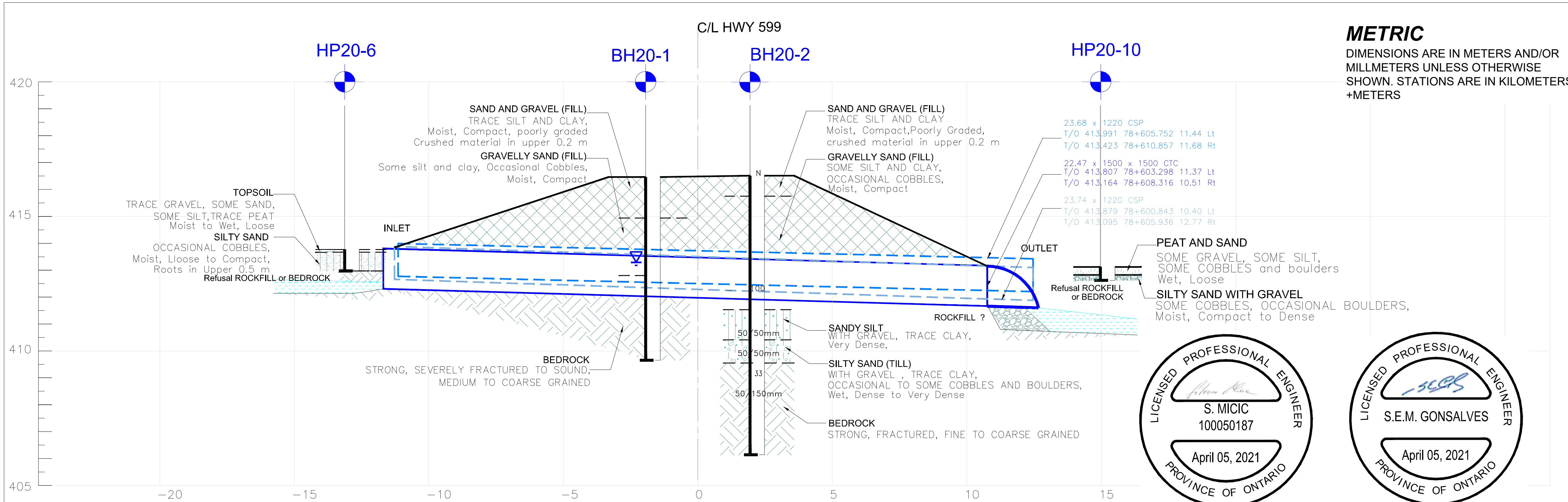
Latitude: 50.206730°, Longitude: -90.726231°
BOREHOLE LOCATION PLAN AND SOIL STRATA

SHEET 1

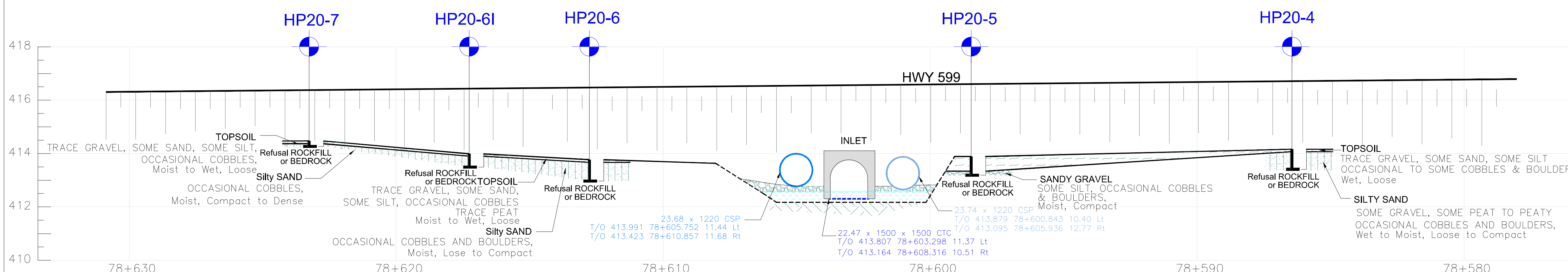
exp. **EXP Services Inc.**

METRIC

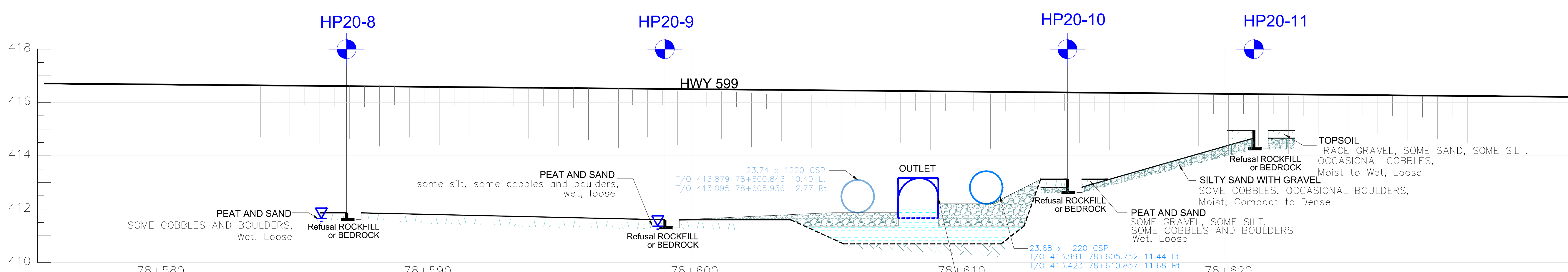
DIMENSIONS ARE IN METERS AND/OR MILLIMETERS UNLESS OTHERWISE SHOWN. STATIONS ARE IN KILOMETERS +METERS



SECTION A-A'



SECTION B-B'



SECTION C-C'

KEY PLAN

LEGEND

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

SOIL STRATA SYMBOLS

- TOPSOIL
- SILTY SAND
- SILTY SAND WITH GRAVEL
- FILL
- SANDY SILT
- SILTY SAND WITH GRAVEL (TILL)
- SANDY GRAVEL
- PEAT AND SAND
- BEDROCK

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SCALE: 0 1 5 m
SCALE

	SM	SUBMISSION FOR MTO REVIEW
DATE	BY	DESCRIPTION
		GEOCRES NO. 52J-19
		PROJECT NO. ADM-00262199-A0
SUBM'D SH	CHECKED SM	DATE April 05, 2021
DRAWN SH	CHECKED SM	APPROVED SG DWG. 1

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.

ISSMFE SOIL CLASSIFICATION

CLAY	SILT			SAND			GRAVEL			COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE		

0.002 0.006 0.02 0.06 0.2 0.6 2.0 6.0 20 60 200

EQUIVALENT GRAIN DIAMETER IN MILLIMETRES

CLAY (PLASTIC) TO	FINE	MEDIUM	CRS.	FINE	COARSE
SILT (NONPLASTIC)	SAND			GRAVEL	

UNIFIED 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:

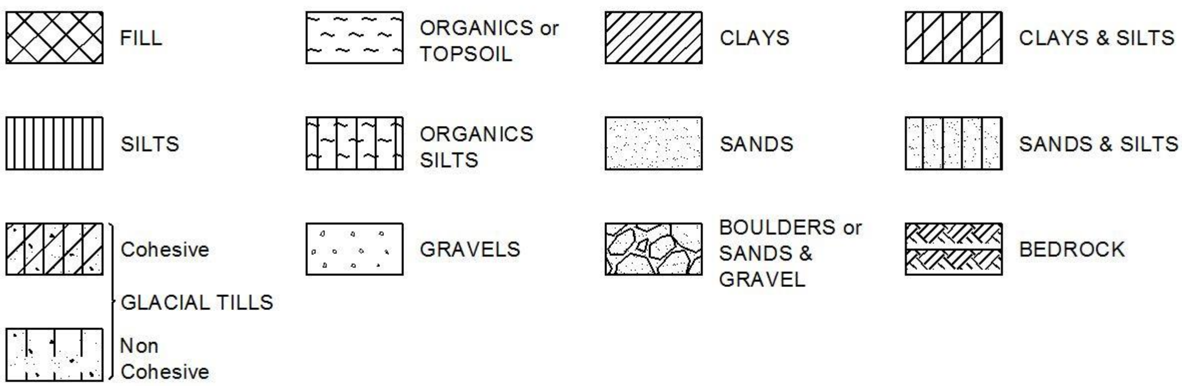
Table c: Consistency of Cohesive Soil

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

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
σ	kPa	Total normal stress
σ'	kPa	Effective normal stress
τ	kPa	Shear stress
$\sigma_1, \sigma_2, \sigma_3$	kPa	Principal stresses
ε	%	Linear strain
$\varepsilon_1, \varepsilon_2, \varepsilon_3$	%	Principal strains
E	kPa	Modulus of linear deformation
G	kPa	Modulus of shear deformation
μ	1	Coefficient of friction

MECHANICALL PROPERIES OF SOIL

m_v	kPa^{-1}	Coefficient of volume change
c_c	1	Compression index
c_s	1	Swelling index
c_r	1	Recompression index
c_v	m^2/s	Coefficient of consolidation
H	m	Drainage path
T_V	1	Time factor
U	%	Degree of consolidation
σ'_{v0}	kPa	Effective overburden pressure
σ'_P	kPa	Preconsolidation pressure
τ_f	kPa	Shear strength
c'	kPa	Effective cohesion intercept
ϕ'	$-°$	Effective angle of internal friction
c_u	kPa	Apparent cohesion intercept
ϕ_u	$-°$	Apparent angle of internal friction
τ_R	kPa	Residual shear strength
τ_r	kPa	Remoulded shear strength
S_t	1	Sensitivity = c_u/τ_r

PHYSICAL PROPERTIES OF SOIL

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

Explanatory Sheet to Rock Coring

Core Recovery: Core recovery is the total length of core pieces, irrespective of their individual lengths, obtained in a core run and expressed as a percentage of the length of that core run.

Rock Quality Designation (RQD): The total length of those pieces of sound core which are 10 cm (4 inches) or greater in length in a core run expressed as a percentage of the total length of that core run. Sound pieces of rack are those pieces separated by natural breaks and not machine breaks or subsequent artificial breaks.

0 25 percent	Very Poor Quality
25 – 40 percent	Poor Quality
40 - 75 percent	Fair Quality
75 – 90 percent	Good Quality
90 – 100 percent	Very Good Quality

Fracturing

Fu	Unfractured	No Fractures
Fvs	Very Slightly Fractured	Core length greater than 0.9 m (3 ft)
Fsl	Slightly Fractured	Core length from 0.3 to 0.9 m (1 to 3 ft)
Fm	Moderately Fractured	Core length from 0.1 to 0.3 m (4 in. to 1 ft)
Fi	Intensely Fractured	Core lengths from 0.25 to 0.1 m (1 in. to 4 in.)
Fvi	Very Intensely Fractured	Mostly chips and fragments

Hardness

H1	Extremely Hard	Cannot be scratched with a pocket knife or sharp pick. Can only be chipped with repeated heavy hammer blows
H2	Very Hard	Cannot be scratched with a pocket knife or sharp pick. Breaks with repeated heavy hammer blows
H3	Hard	Can be scratched with a pocket knife or sharp pick with difficulty (heavy pressure) Breaks with heavy hammer blows
H4	Moderately Hard	Can be scratched with a pocket knife or sharp pick with light or moderate pressure. Breaks with moderate hammer blows
H5	Moderately Soft	Can be grooved 1.6 mm (1/16 in) with a pocket knife or sharp pick with moderate or heavy pressure. Breaks with light hammer blows or manual pressure
H6	Soft	Can be grooved or gouged easily with a pocket knife or sharp pick with slight pressure, can be scratech with a finger nail. Breaks with light or moderate manual pressure

H7	Very Soft	Can readily be indented, grooved or gouged with a finger nail, or Carved with pocket knife. Breaks with light manual pressure

Strength (from ISRM) **Approx UCS**

Svh	Very High Strength	>200 MPa
Sh	High Strength	50 to 200 MPa
Sm	Medium Strength	15 to 50 MPa
Sl	Low Strength	4 to 15 MPa
Svl	Very Low Strength	1 to 4 MPa

Weathering

Wf	Fresh	no signs of discolouration or oxidation
Ws	Slightly Weathered	partial discolouration; fractures (joints) typically oxidized
Wm	Moderately Weathered	total discolouration
Wi	Intensely Weathered	total discolouration; typically friable & pitted
Wd	Decomposed	resembles soil; rock structure usually preserved

Discontinuity Description

Fracture Width (FW)

FWt	Tight	No visible separation
FWs	Slightly Open	FW< 0.8 mm (1/32 in.)
FWm	Moderately Open	0.8 mm (1/32 in.)≤FW<3.2 mm (1/8 in.)
FWo	Open	3.2 mm (1/8 in.) ≤FW<9.7 mm (3/8 in.)
FWmw	Moderatley Wide	9.7 mm(3/8 in.) ≤FW<25.4 mm (1 in.)
FWw	Wide	FW≥25.4 mm(1 in.)

Fracture Filling or Coating Thickness(FF)

FFc	Clean	No film coating
FFvt	Very Thin	FF< 0.8 mm (1/32 in.)
FFm	Moderately Thin	0.8 mm (1/32 in.)≤FF<3.2 mm (1/8 in.)
FFt	Thin	3.2 mm (1/8 in.) ≤FF<9.7 mm (3/8 in.)
FFmt	Moderately Thick	9.7 mm(3/8 in.) ≤FF<25.4 mm (1 in.)
FFw	Thick	FF≥25.4 mm(1 in.)

Roughness

Rst	Stepped	Near normal steps and ridges occur on the fracture surface
Rr	Rough	Large angular asperities can be seen

Rm	Moderately Rough	Asperities are cleanly visible and fracture surface feels abrasive
Rs	Slightly Rough	Small asperities on the fracture surface are visible and can be felt
Rsm	Smooth	No asperities, smooth to the touch

Bedding Spacing (Sb)

Bm	Massive	≤Sb > 3 m (10 ft)
Bvt	Very Thickly Bedded	0.9 m (3 ft) ≤ Sb ≤ 3 m (10 ft)
Bt	Thickly Bedded	0.3 m (1 ft) ≤ Sb ≤ 0.9 m (3 ft)
Bm	Moderately Bedded	0.1 m (4 in.) ≤ Sb ≤ 0.3 m (1 ft)
Bt	Thinly Bedded	25 mm (1 in.) ≤ Sb ≤ 0.1 m (4 in.)
Bvt	Very Thinly Bedded	6 mm (1/4 in.) ≤ Sb ≤ 25 mm (1 in.)
Bl	Laminated	SB ≤ 6 mm (1/4 in.)

Orientation

Of	Flat = 0 - 20º
Od	Dipping = 20 - 50º
Ov	Vertical = 50 - 90º

Surface Shape

Planar	Flat surface
Wavy	Undulating surface

Fracture Type:

B	Bedding
J	Fault
C	Joint
F	Foliation
S	Shear Plane
M	Mechanical Breaks

RECORD OF BOREHOLE No BH20-01

METRIC

W.P. GWP No. 6530-17-00 LOCATION Hwy 599 ~4.6 km South of Savant Lake CNR Crossing, MTM 15, N 5563540.7, E 252977.6 ORIGINATED BY EF

DIST Thunder Bay HWY 599 BOREHOLE TYPE B54X Rubber Track SSA / HQ COMPILED BY AM

DATUM Local DATE 2020.11.03 - 2020.11.03 LATITUDE 50.20676 LONGITUDE -90.726 CHECKED BY DG/SM

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
416.5	Asphalt treatment 1" thick					
0.0	**Sand and Gravel (FILL)** trace silt and clay, grey, moist, compact, poorly graded, crushed material in upper 0.2 m - becoming brown, occasional cobbles at about 0.2 m depth	S1	AS			
		S2	AS			
		S3	AS			50 43 (7)
		S4	AS			
414.9 1.5	**Gravelly Sand (FILL)** some silt and clay, occasional cobbles, brown, moist, compact	S5	AS			
		S6	AS			22 63 (15)
		S7	AS		∇	
412.8 3.7	**BEDROCK** strong, severely fractured to sound, white/pink to grey, medium to coarse grained	S8	CORE			UCS test at 3.8 m depth = 82 MPa Recovery=99%, RQD=79%
		S9	CORE			Recovery=100%, RQD=37%
		S10	CORE			Recovery=100%, RQD=66%
409.7 6.8	**End of Borehole** - groundwater was measured 3.1 m below ground surface					

Chart headings: SOIL PROFILE; SAMPLES; STRAT PLOT; ELEVATION SCALE (416, 415, 414, 413, 412, 411, 410); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × P. PENETROMETER 20 40 60 80 100; PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m^3

$+^3, \times^3$: Numbers refer to Sensitivity O$^{3\%}$ STRAIN AT FAILURE

EXP Services Inc.

Brampton, Ontario

RECORD OF BOREHOLE No BH20-02

METRIC

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

Column headings: SOIL PROFILE (ELEV / DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa (○ UNCONFINED + FIELD VANE; ● QUICK TRIAXIAL × P. PENETROMETER; 20, 40, 60, 80, 100); PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) (20, 40, 60); UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) (GR SA SI CL)

ELEV / DEPTH	DESCRIPTION
416.5	Asphalt treatment 1" thick
0.0	**Sand and Gravel (FILL)** trace silt and clay, grey, moist, compact, poorly graded, crushed material in upper 0.2 m
415.8 / 0.8	**Gravelly Sand (FILL)** some silt and clay, occasional cobbles, brown, moist, compact - refusal to SPT and auger at about 4.4 m depth, rock coring techniques initiated
411.5 / 5.0	**Sandy SILT with Gravel** trace clay, very dense, grey, wet
410.4 / 6.1	**Silty SAND with Gravel (TILL)** trace clay, , occasional to some cobbles and boulders, grey, wet, dense to very dense
409.6 / 7.0	**BEDROCK** strong, fractured, white and black, fine to coarse grained
406.1 / 10.4	**End of Borehole** - no obtainable groundwater level due to caved borehole

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
S1	AS		
S2	AS		
S3	AS		
S4	AS		24 65 10 1
S5	AS		
S6	AS		
S7	SS	100	
S8	CORE		
S9	SS	50/ 50mm	23 32 36 9
	SS	50/ 50mm	No recovery
S10	SS	33	21 49 24 6
S11	SS	50/ 150mm	
S12	CORE		UCS test at 7.1 m depth = 83 MPa Recovery=100%, RQD=61%
S13	CORE		Recovery=100%, RQD=66%
S14	CORE		Recovery=100%, RQD=77%

Elevation scale: 416, 415, 414, 413, 412, 411, 410, 409, 408, 407

$+^3, \times^3$: Numbers refer to Sensitivity ○ $^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No BH20-03

METRIC

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

ELEV DEPTH	DESCRIPTION (SOIL PROFILE)	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
416.1	Asphalt treatment 1" thick							
0.0	**Sand and Gravel (FILL)** trace silt and clay, occasional cobbles, grey to brown, moist, crushed material, dense to very dense		S1A	SS	41		416	
			S1B	SS	39			
			S2	SS	65		415	
			S3	SS	50/ 125mm			44 42 (14)
413.8							414	
2.3	**BEDROCK** very strong, fractured, green to blue, fine grained		S4	CORE			413	UCS test at 2.8 m depth = 103 MPa Recovery=99%, RQD=56%
	- becoming strong, white and black, medium to coarse grained at about 3.2 m depth							UCS test at 3.5 m depth = 95 MPa
412.2								
3.9	**End of Borehole**							
	- no obtainable groundwater level due to caved borehole							

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × P. PENETROMETER: 20 40 60 80 100

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L

WATER CONTENT (%): 20 40 60

UNIT WEIGHT γ kN/m³

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

RECORD OF BOREHOLE No HP20-04

METRIC

W.P. GWP No. 6530-17-00 | LOCATION **Hwy 599 ~4.6 km South of Savant Lake CNR Crossing, MTM 15, N 5563540.8, E 252950.6** | ORIGINATED BY EF

DIST Thunder Bay HWY 599 | BOREHOLE TYPE Power Hand Auger / SSA | COMPILED BY AM

DATUM Local | DATE 2020.11.05 - 2020.11.05 LATITUDE 50.20676 LONGITUDE -90.72638 | CHECKED BY DG/SM

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
414.2	Topsoil						
414.0 / 0.1	**TOPSOIL** trace gravel, some sand, some silt, occasional to some cobbles, occasional boulders, dark brown, wet, loose	S1	AS			414	
		S2	AS				29 48 (23)
		S3	AS				15 56 (30)
413.4 / 0.8	**Silty SAND** some gravel, occasional cobbles and boulders, some peat to peaty in upper 0.3 m dark brown, wet to moist, loose to compact	S4	AS				
	End of Borehole - refusal						
	- no groundwater encountered						
	- bedrock outcrop observed about 3 m from borehole						

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE; ● QUICK TRIAXIAL × P. PENETROMETER: 20 40 60 80 100

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%): 20 40 60

UNIT WEIGHT γ kN/m^3

$+^3, \times^3$: Numbers refer to Sensitivity   O $^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No HP20-05

METRIC

W.P. GWP No. 6530-17-00 LOCATION **Hwy 599 ~4.6 km South of Savant Lake CNR Crossing, MTM 15, N 5563545.4, E 252961.8** ORIGINATED BY EF

DIST Thunder Bay HWY 599 BOREHOLE TYPE Power Hand Auger / SSA COMPILED BY AM

DATUM Local DATE 2020.11.05 - 2020.11.05 LATITUDE 50.2068 LONGITUDE -90.72623 CHECKED BY DG/SM

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
413.9	Topsoil													
0.0	**TOPSOIL** trace gravel, some sand, some silt, occasional cobbles, dark brown, moist to wet, loose		S1	AS										
			S2	AS										
413.3			S3	AS										
413.2 / 0.6	**Sandy GRAVEL** some silt, occasional cobbles and boulders, light brown, moist, compact		S4	AS							46	39	16	0
0.7	**End of Borehole** - refusal													

- no groundwater encountered

- bedrock outcrop observed about 3 m from borehole

Legend: DCPT plot scale 20 40 60 80 100; SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × P. PENETROMETER; scale 20 40 60 80 100. PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) scale 20 40 60. REMARKS & GRAIN SIZE DISTRIBUTION (%).

$+^3, \times^3$: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No HP20-06

METRIC

W.P. GWP No. 6530-17-00 LOCATION Hwy 599 ~4.6 km South of Savant Lake CNR Crossing, MTM 15, N 5563552.9, E 252974.1 ORIGINATED BY EF

DIST Thunder Bay HWY 599 BOREHOLE TYPE Power Hand Auger / SSA COMPILED BY AM

DATUM Local DATE 2020.11.04 - 2020.11.04 LATITUDE 50.20687 LONGITUDE -90.72605 CHECKED BY DG/SM

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
413.8	Topsoil										
413.7 / 0.1	**TOPSOIL** trace gravel, some sand, some silt, brown, moist to wet, loose		S1	AS							
	Silty SAND occasional cobbles, brown, moist, loose to compact, roots in upper 0.5 m - trace peat at about 0.3 m depth		S2	AS							
			S3	AS							
			S4	AS			413				3 64 (33)
413.0 / 0.8	**End of Borehole** - refusal										
	- no groundwater encountered										
	- bedrock outcrop observed about 3 m from borehole										

Legend: DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × P. PENETROMETER 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60

$+^3, \times^3$: Numbers refer to Sensitivity $\circ^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No HP20-06I

METRIC

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

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × P. PENETROMETER) (20 40 60 80 100)	PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) (20 40 60)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
414.0	Topsoil										
413.9 / 0.1	**TOPSOIL** trace gravel, some sand, some silt, occasional cobbles, brown, moist to wet, loose		S1	AS							
			S2	AS							
413.5 / 0.5	**Silty SAND** occasional cobbles and boulders, brown, moist, compact		S3	AS							
	End of Borehole refusal										
	- no groundwater encountered										
	- bedrock outcrop observed about 3 m from borehole										

$+^3, \times^3$: Numbers refer to Sensitivity　　$\circ^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No HP20-07

METRIC

W.P. GWP No. 6530-17-00 LOCATION Hwy 599 ~4.6 km South of Savant Lake CNR Crossing, MTM 15, N 5563556.5, E 252984.2 ORIGINATED BY EF

DIST Thunder Bay HWY 599 BOREHOLE TYPE Power Hand Auger / SSA COMPILED BY AM

DATUM Local DATE 2020.11.04 - 2020.11.04 LATITUDE 50.2069 LONGITUDE -90.72591 CHECKED BY DG/SM

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × P. PENETROMETER (20 40 60 80 100)	PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) (20 40 60)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
414.5	Topsoil										
414.4 0.1	**TOPSOIL** trace gravel, some sand, some silt, occasional cobbles, brown, moist to wet, loose		S1	AS							
414.3 0.2	**Silty SAND** occasional cobbles, brown, moist, compact to dense		S2	AS							6 79 (15)
	End of Borehole refusal										

- no groundwater encountered

- bedrock outcrop observed about 3 m from borehole

$+^3, \times^3$: Numbers refer to Sensitivity	O $^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No HP20-08

METRIC

W.P. GWP No. 6530-17-00 LOCATION **Hwy 599 ~4.6 km South of Savant Lake CNR Crossing, MTM 15, N 5563512.3, E 252962.4** ORIGINATED BY EF

DIST Thunder Bay HWY 599 BOREHOLE TYPE Power Hand Auger / SSA COMPILED BY AM

DATUM Local DATE 2020.11.02 - 2020.11.02 LATITUDE 50.2065 LONGITUDE -90.72621 CHECKED BY DG/SM

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × P. PENETROMETER (20 40 60 80 100)	PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) (20 40 60)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
411.9	Peat										
0.0	**PEAT AND SAND** some cobbles and boulders, dark brown, wet, loose		S1	AS					94.5		
411.6			S2	AS		∇			81.2		6 57 (37)
0.3	**End of Borehole** refusal										

- three additional hand probes / digging were conducted in about a 2.5 m radius, all with similar findings

- groundwater was measured 0.2 m below ground surface

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

RECORD OF BOREHOLE No HP20-09

METRIC

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

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × P. PENETROMETER) 20 40 60 80 100	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
411.6	Peat										
0.0	**PEAT AND SAND** some silt, some cobbles and boulders, dark brown, wet, loose		S1	AS					o		
411.3			S2	AS		∇			o		3 54 (43)
0.3	**End of Borehole** refusal										
	- five additional hand probes / digging were conducted in about a 3.0 m radius, all with similar findings										
	- groundwater was measured 0.2 m below ground surface										

$+^3, \times^3$: Numbers refer to Sensitivity $O^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No HP20-10

METRIC

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

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
413.1	Peat										
0.0 / 412.8	**PEAT AND SAND** some gravel, some silt, some cobbles and boulders, dark brown to black, wet, loose		S1	AS			413		o (~45)		
0.3 / 412.6	**Silty SAND with Gravel** some cobbles, occasional boulders, brown, moist, compact to dense		S2	AS					o (~15)		21 57 (22)
0.5	**End of Borehole** refusal										
	- no groundwater encountered										
	- bedrock outcrop observed about 1.5 m from borehole										

Legend: DCPT plot scale 20 40 60 80 100; O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × P. PENETROMETER; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60

$+^3, \times^3$: Numbers refer to Sensitivity $O^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No HP20-11

METRIC

W.P. GWP No. 6530-17-00 LOCATION Hwy 599 ~4.6 km South of Savant Lake CNR Crossing, MTM 15, N 5563529.1, E 252992.1 ORIGINATED BY EF

DIST Thunder Bay HWY 599 BOREHOLE TYPE Power Hand Auger / SSA COMPILED BY AM

DATUM Local DATE 2020.11.02 - 2020.11.02 LATITUDE 50.20665 LONGITUDE -90.7258 CHECKED BY DG/SM

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
415.0	Topsoil											
0.0 / 414.7 / 0.3	**TOPSOIL** trace gravel, some sand, some silt, occasional cobbles, dark brown, moist to wet, loose	S1	AS									
	Silty SAND with Gravel some cobbles, occasional boulders, brown, moist, compact to dense	S2	AS									
414.3 / 0.7		S3	AS						25	50	(25)	
	End of Borehole refusal											
	- no groundwater encountered											
	- bedrock outcrop observed about 1.5 m from borehole											

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

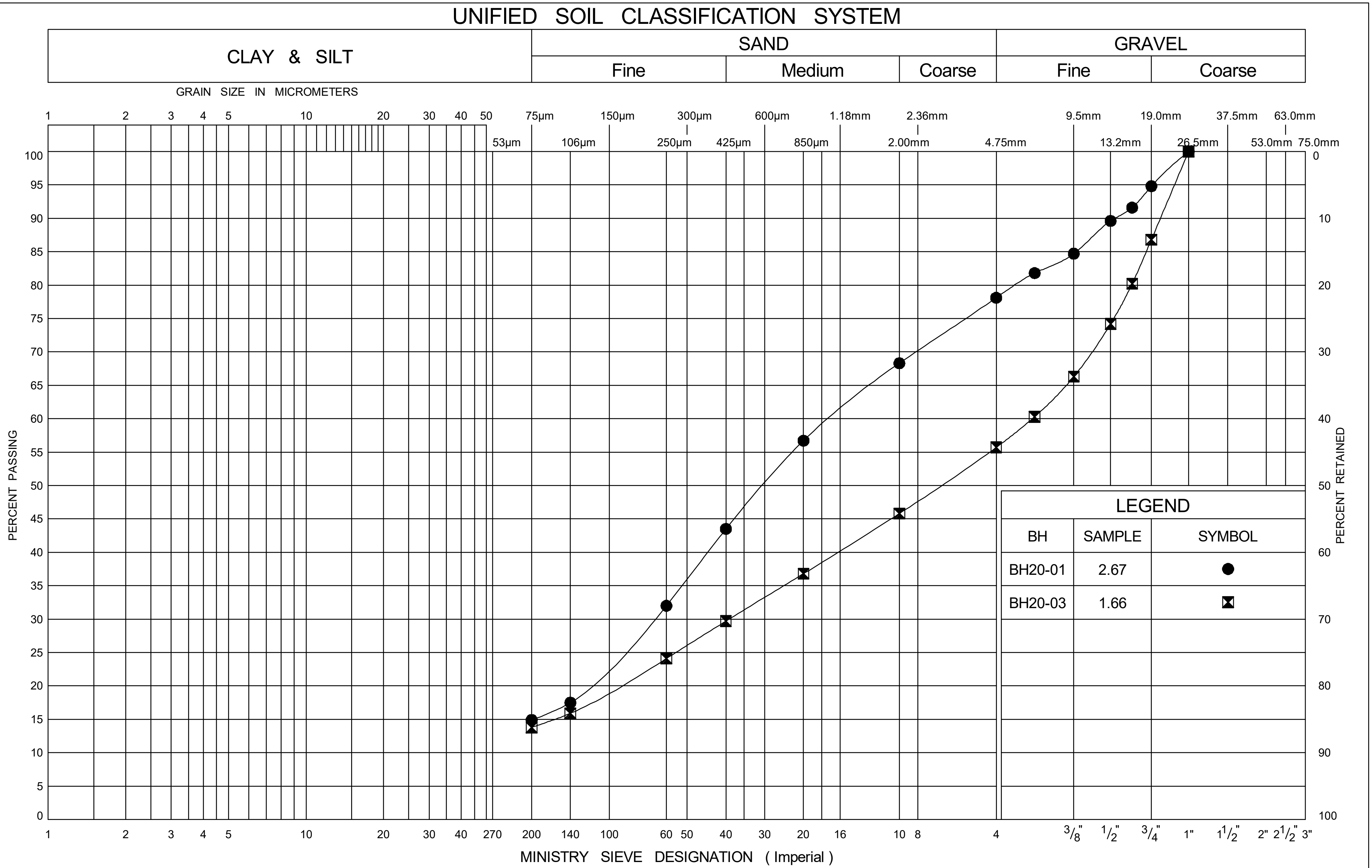
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × P. PENETROMETER: 20 40 60 80 100

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%): 20 40 60

REMARKS & GRAIN SIZE DISTRIBUTION (%)

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

Appendix D –
Laboratory Data



ONTARIO MOT GRAIN SIZE F-20166-AG - ADM-00262199-A0 - MTO 1 - TROUT CREEK CULVERT, SAVANT LAKE.GPJ ONTARIO MOT.GDT 3/1/21

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Ontario

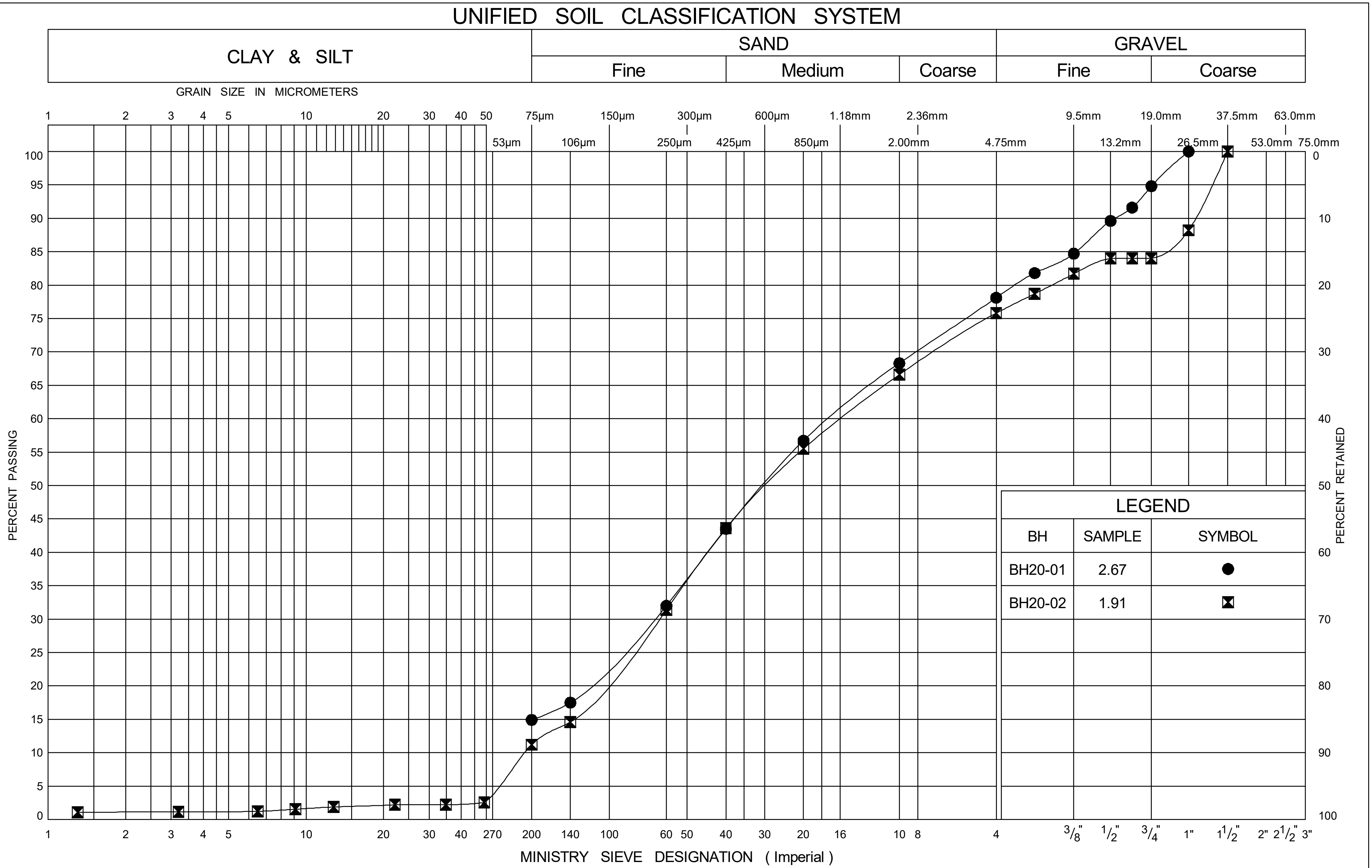
GRAIN SIZE DISTRIBUTION

Fill: Sand and Gravel

FIG No 1

W P GWP No. 6530-17-00

6019-E-0004, Assignment 1



Ministry of Transportation
Ontario

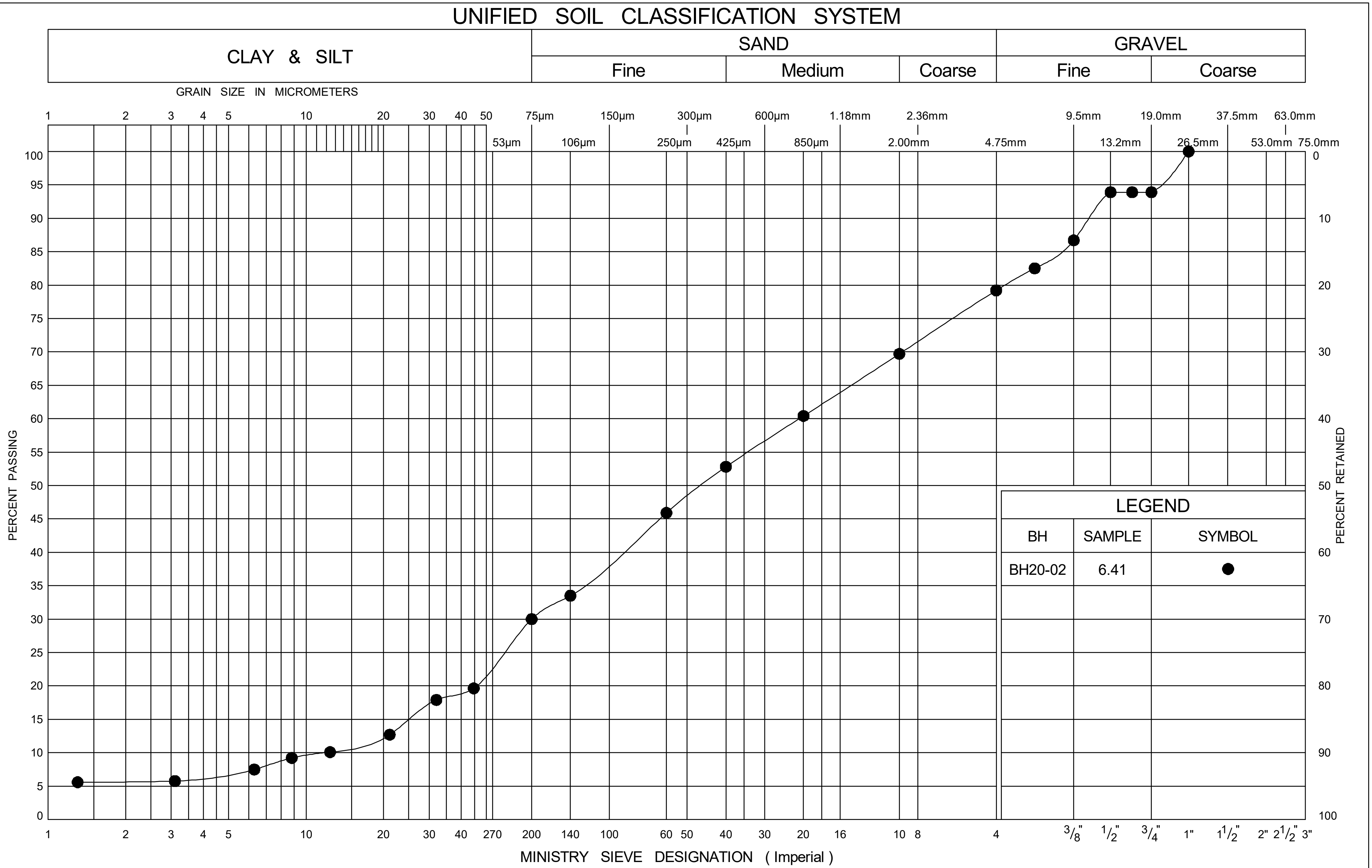
GRAIN SIZE DISTRIBUTION

Fill: Gravelly Sand

FIG No 2

W P GWP No. 6530-17-00

6019-E-0004, Assignment 1



Ministry of Transportation

Ontario

GRAIN SIZE DISTRIBUTION

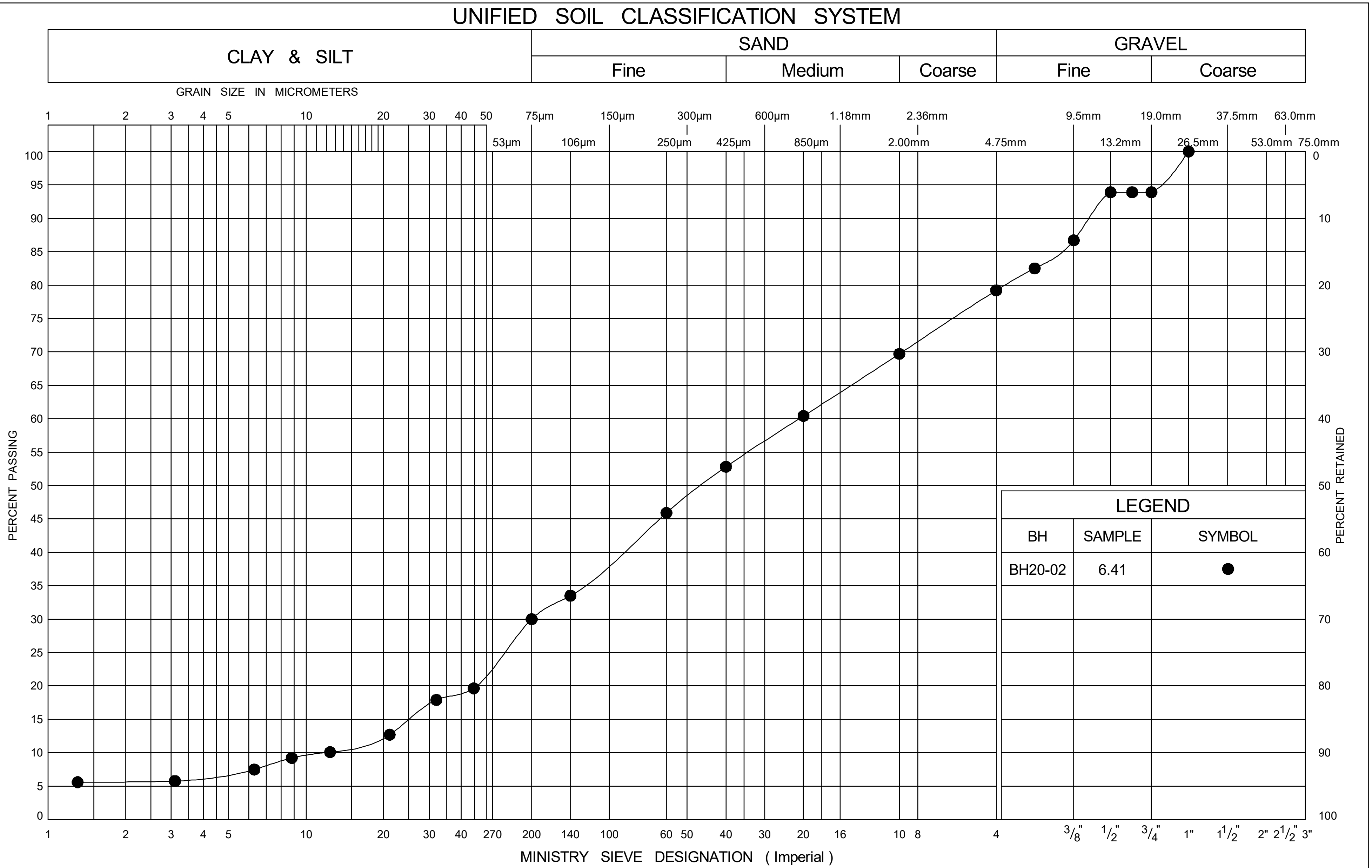
Sandy Silt with Gravel

FIG No 3

W P GWP No. 6530-17-00

6019-E-0004, Assignment 1

ONTARIO MOT GRAIN SIZE F-20166-AG - ADM-00262199-A0 - MTO 1 - TROUT CREEK CULVERT, SAVANT LAKE.GPJ ONTARIO MOT.GDT 3/1/21



Ministry of Transportation

Ontario

GRAIN SIZE DISTRIBUTION

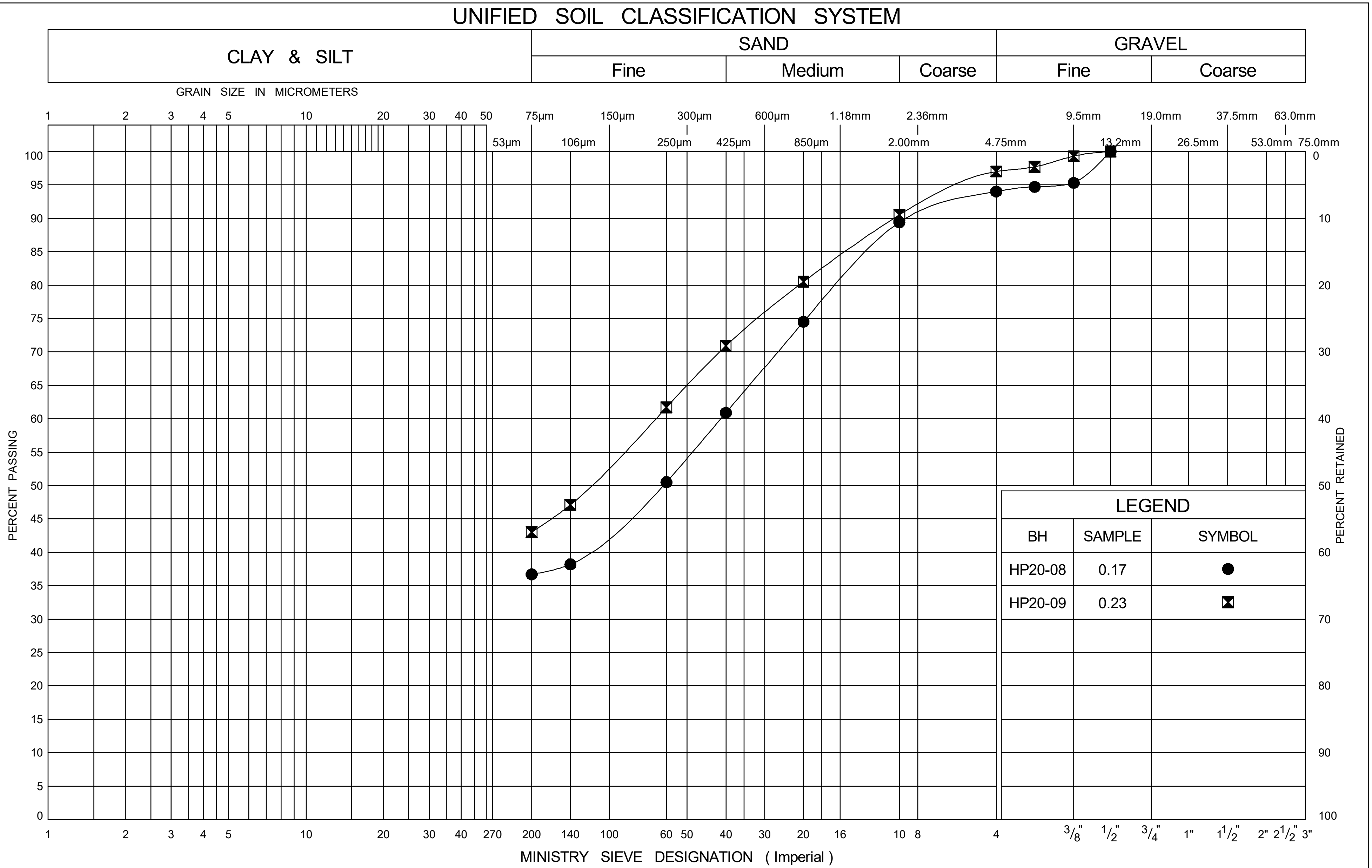
Till: Silty Sand with Gravel

FIG No 4

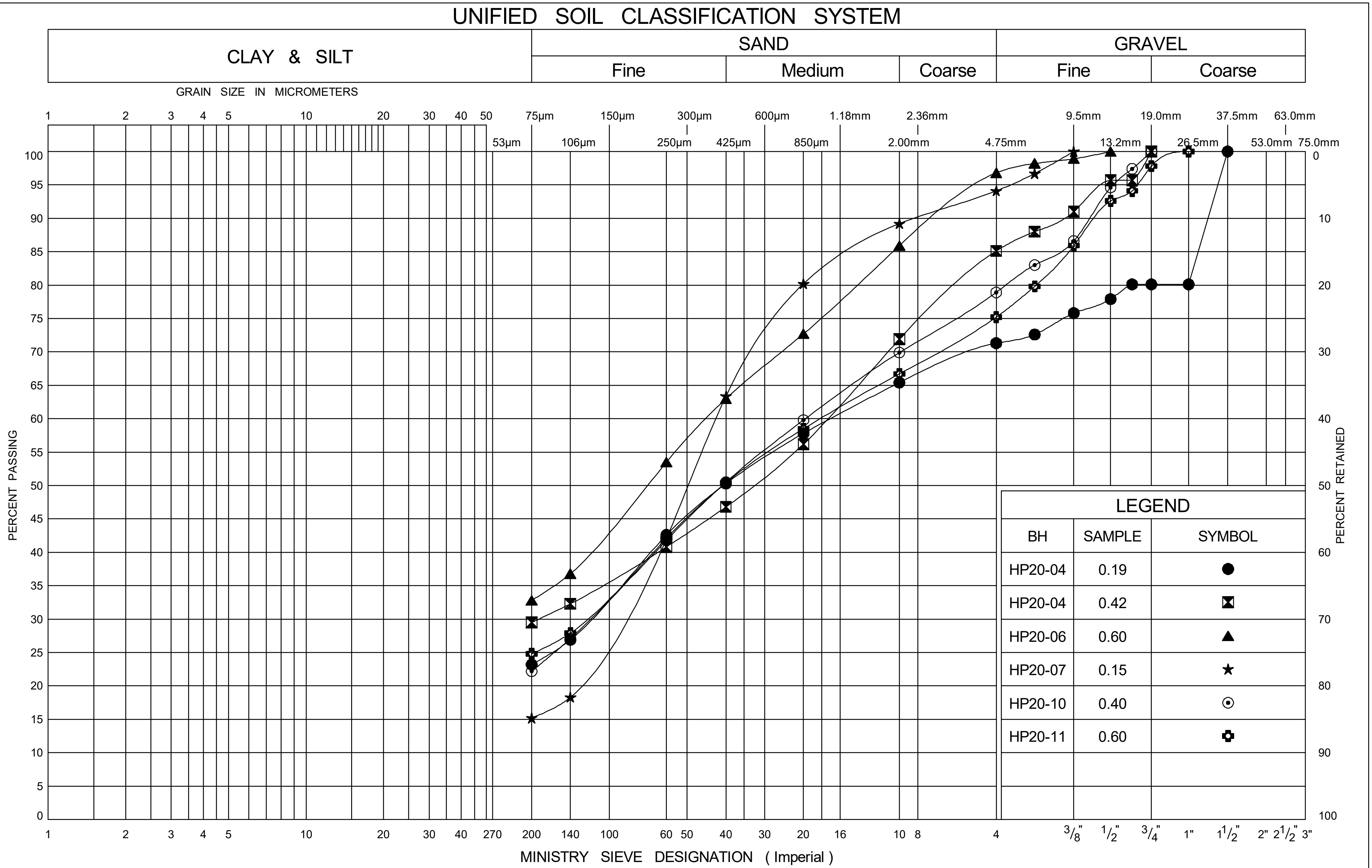
W P GWP No. 6530-17-00

6019-E-0004, Assignment 1

ONTARIO MOT GRAIN SIZE F-20166-AG - ADM-00262199-A0 - MTO 1 - TROUT CREEK CULVERT, SAVANT LAKE.GPJ ONTARIO MOT.GDT 3/1/21



Ministry of Transportation Ontario	**GRAIN SIZE DISTRIBUTION**	FIG No 5
	Peat and Sand	W P GWP No. 6530-17-00
		6019-E-0004, Assignment 1



Ministry of Transportation
Ontario

GRAIN SIZE DISTRIBUTION

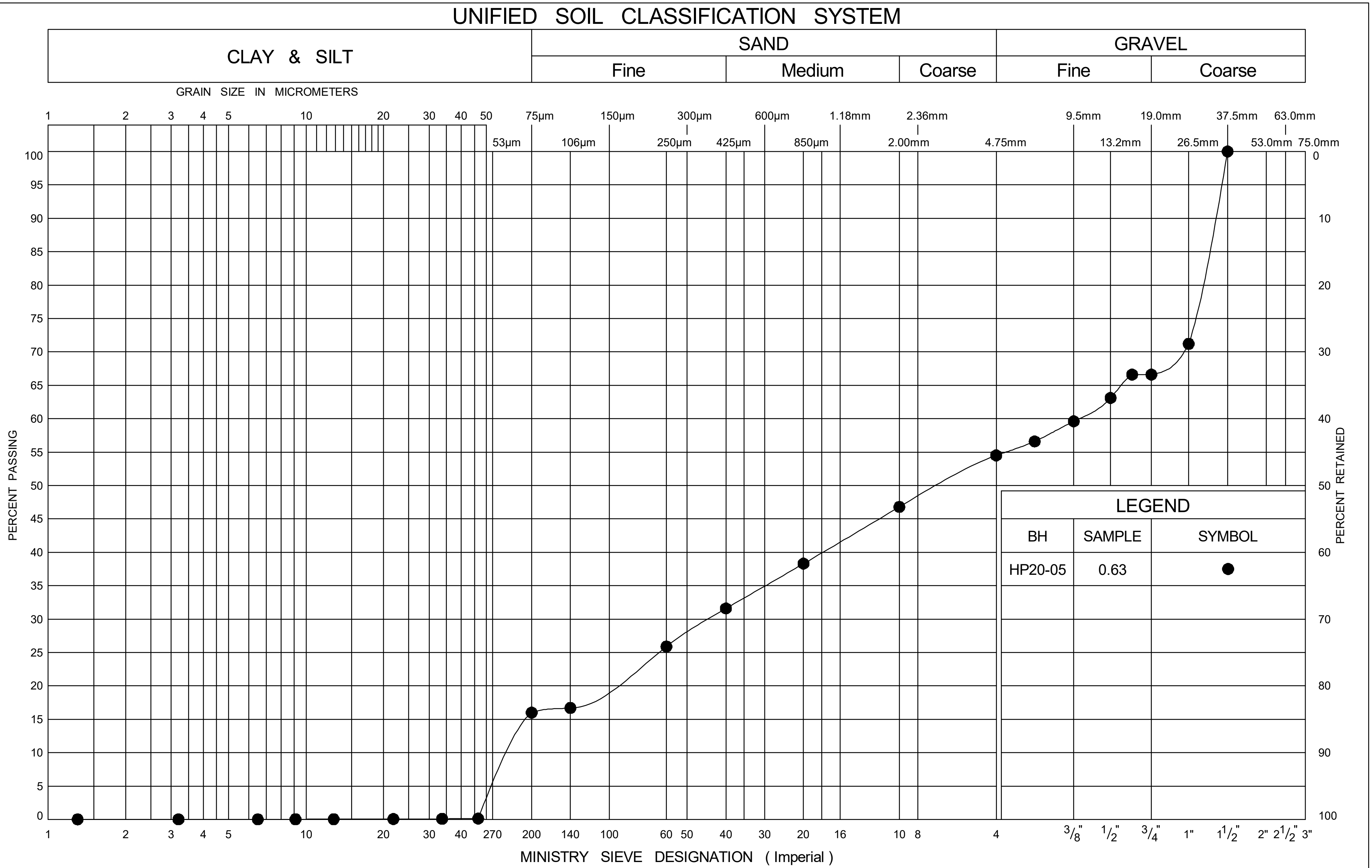
Silty Sand / Silty Sand with Gravel

FIG No 6

W P GWP No. 6530-17-00

6019-E-0004, Assignment 1

ONTARIO MOT GRAIN SIZE F-20166-AG - ADM-00262199-A0 - MTO 1 - TROUT CREEK CULVERT, SAVANT LAKE.GPJ ONTARIO MOT.GDT 3/1/21



ONTARIO MOT GRAIN SIZE F-20166-AG - ADM-00262199-A0 - MTO 1 - TROUT CREEK CULVERT, SAVANT LAKE.GPJ ONTARIO MOT.GDT 3/1/21

Ministry of Transportation
Ontario

GRAIN SIZE DISTRIBUTION

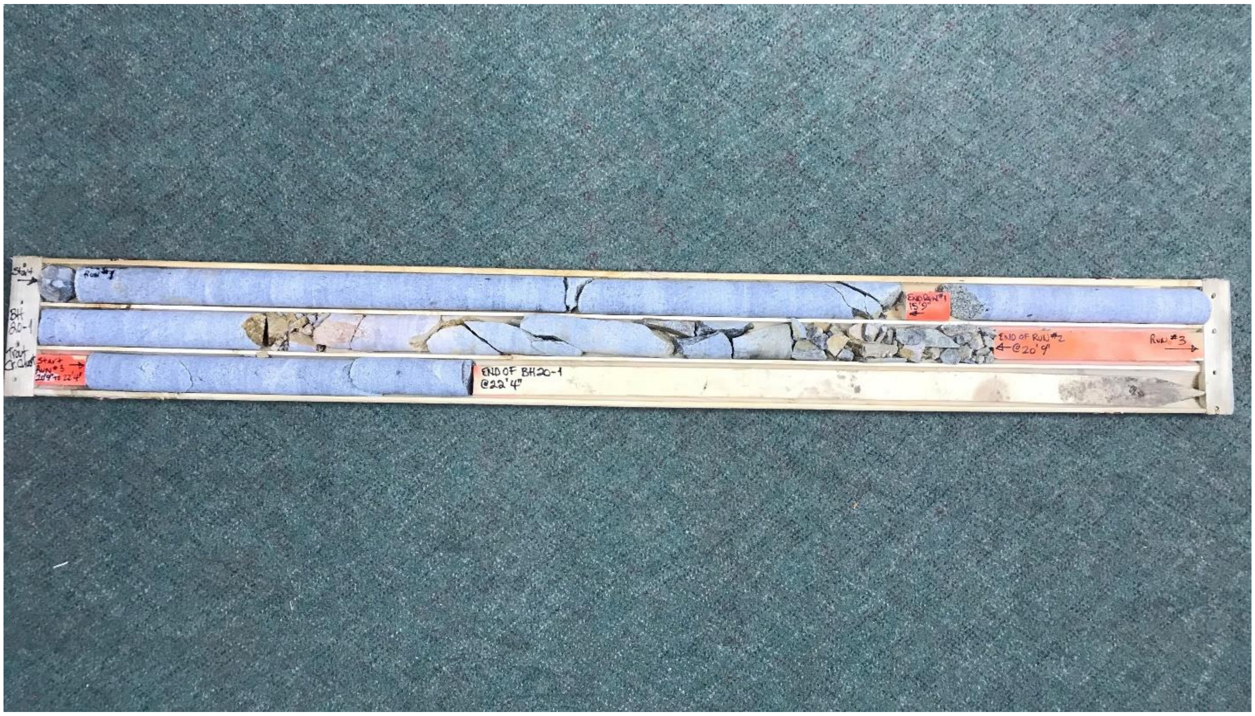
Sandy Gravel

FIG No 7

W P GWP No. 6530-17-00

6019-E-0004, Assignment 1

Appendix E –
Bedrock Core Photographs



Photograph E1. Bedrock core samples, borehole BH20-1, November 2020

Photograph E2. Bedrock core samples, borehole BH20-2, November 2020

Appendix F –
Chemical Analysis

BUREAU VERITAS

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

Attention: Ahileas Mitsopoulos

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

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

CERTIFICATE OF ANALYSIS

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

Sample Matrix: Soil
Samples Received: 3

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

Remarks:

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

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

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

Solid sample results, except biota, are based on dry weight unless otherwise indicated. Organic analyses are not recovery corrected except for isotope dilution methods.
Results relate to samples tested. When sampling is not conducted by BV Labs, results relate to the supplied samples tested.
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Reference Method suffix "m" indicates test methods incorporate validated modifications from specific reference methods to improve performance.

Bureau Veritas Laboratories 6740 Campobello Road, Mississauga, Ontario, L5N 2L8 Tel: (905) 817-5700 Toll-Free: 800-563-6266 Fax: (905) 817-5777 www.bvlabs.com

Microbiology testing is conducted at 6660 Campobello Rd. Chemistry testing is conducted at 6740 Campobello Rd.

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

Attention: Ahileas Mitsopoulos

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

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

CERTIFICATE OF ANALYSIS

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

Encryption Key

AUTHORIZED REPORT
RAPPORT AUTORISÉ

Bureau Veritas Laboratories
24 Nov 2020 17:18:27

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

==

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BV Labs Job #: C0U9063
Report Date: 2020/11/24

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

RESULTS OF ANALYSES OF SOIL

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

RDL = Reportable Detection Limit
QC Batch = Quality Control Batch
Lab-Dup = Laboratory Initiated Duplicate

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Microbiology testing is conducted at 6660 Campobello Rd. Chemistry testing is conducted at 6740 Campobello Rd.

BUREAU VERITAS

BV Labs Job #: C0U9063
Report Date: 2020/11/24

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

TEST SUMMARY

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

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

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

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

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

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

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

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

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

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

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

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

BUREAU VERITAS

BV Labs Job #: C0U9063
Report Date: 2020/11/24

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

GENERAL COMMENTS

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

Package 1	3.7°C

Results relate only to the items tested.

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Microbiology testing is conducted at 6660 Campobello Rd. Chemistry testing is conducted at 6740 Campobello Rd.

QUALITY ASSURANCE REPORT

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

			Matrix Spike		SPIKED BLANK		Method Blank		RPD	
QC Batch	Parameter	Date	% Recovery	QC Limits	% Recovery	QC Limits	Value	UNITS	Value (%)	QC Limits
7069471	Soluble (20:1) Sulphate (SO4)	2020/11/23	NC	70 - 130	104	70 - 130	<20	ug/g	NC	35
7069474	Soluble (20:1) Chloride (Cl-)	2020/11/23	NC	70 - 130	107	70 - 130	<20	ug/g	3.0	35
7072893	Available (CaCl2) pH	2020/11/24			100	97 - 103			1.7	N/A
7072990	Conductivity	2020/11/24			103	90 - 110	<0.002	mS/cm	1.3	10

N/A = Not Applicable

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

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

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

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

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

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

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Microbiology testing is conducted at 6660 Campobello Rd. Chemistry testing is conducted at 6740 Campobello Rd.

BUREAU VERITAS

BV Labs Job #: C0U9063
Report Date: 2020/11/24

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

VALIDATION SIGNATURE PAGE

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

Anastassia Hamanov, Scientific Specialist

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

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

CHAIN OF CUSTODY RECORD

Invoice Information
Company Name: EXP Services Inc.
Contact Name: Ahileas Mitsopoulos
Address: 1142 Rolland St.
Thunder Bay, ON
Phone: 807-623-9495 Fax:
Email: ahileas.mitsopoulos@exp.com

Report Information (if differs from invoice)
Company Name:
Contact Name:
Address:
Phone: Fax:
Email:

Project Information (where applicable)
Quotation #: B45998
P.O. #/ AFE#:
Project #: ADM-00262199-A0
Site Location: Trout Creek Culvert, Hwy 599
Site #: MTO 1
Site Location Province: Ontario
Sampled By: EF

Turnaround Time (TAT) Required
[X] Regular TAT (5-7 days) Most analyses
PLEASE PROVIDE ADVANCE NOTICE FOR RUSH PROJECTS
Rush TAT (Surcharges will be applied)
[ ] 1 Day [ ] 2 Days [ ] 3-4 Days
Date Required:
Rush Confirmation #:

MOE REGULATED DRINKING WATER OR WATER INTENDED FOR HUMAN CONSUMPTION MUST BE SUBMITTED ON THE BUREAU VERITAS LABORATORIES' DRINKING WATER CHAIN OF CUSTODY

Regulation 153
[ ] Table 1 [ ] Res/Park [ ] Med/ Fine
[ ] Table 2 [ ] Ind/Comm [ ] Coarse
[ ] Table 3 [ ] Agri/ Other
[ ] Table ___
FOR RSC (PLEASE CIRCLE) Y / N

Other Regulations
[ ] CCME [ ] Sanitary Sewer Bylaw
[ ] MISA [ ] Storm Sewer Bylaw
[ ] PWQO Region
[ ] Other (Specify)
[ ] REG 558 (MIN. 3 DAY TAT REQUIRED)
[ ] REG 406 Table

Include Criteria on Certificate of Analysis: Y / N

SAMPLES MUST BE KEPT COOL (< 10 °C) FROM TIME OF SAMPLING UNTIL DELIVERY TO BUREAU VERITAS

Analysis Requested

	SAMPLE IDENTIFICATION	DATE SAMPLED (YYYY/MM/DD)	TIME SAMPLED (HH:MM)	MATRIX	# OF CONTAINERS SUBMITTED	FIELD FILTERED (CIRCLE) Metals / Hg / CrVI	BTEX/ PHC F1	PHCs F2 - F4	VOCs	REG 153 METALS & INORGANICS	REG 153 ICPMS METALS	REG 153 METALS (Hg, Cr VI, ICPMS Metals, HWS - B)	pH	Conductivity	Resistivity	Water Soluble Sulphate	Chloride				HOLD- DO NOT ANALYZE
1	BH20-01 S7	2020/11/03	12:00 PM	Soil	1								x	x	x	x	x				
2	HP20-05 S4	2020/11/05	10:00 AM	Soil	1								x	x	x	x	x				
3	HP20-11 S2	2020/11/02	3:00 PM	Soil	1								x	x	x	x	x				
4																					
5																					
6																					
7																					
8																					
9																					
10																					

LABORATORY USE ONLY

CUSTODY SEAL Y / N Present	Intact	COOLER TEMPERATURES
N	N/A	5/3/3
		4,4,4

COOLING MEDIA PRESENT: Y / N
COMMENTS

Rec'd In Thunder Bay

RELINQUISHED BY: (Signature/Print)	DATE: (YYYY/MM/DD)	TIME: (HH:MM)	RECEIVED BY: (Signature/Print)	DATE: (YYYY/MM/DD)	TIME: (HH:MM)
	2020/11/19	1:30 PM	James Klapperich	2020/11/19	13:50
			C Yewpan Tchiz	2020/11/20	09:17

19-Nov-20 13:50
Julie Clement
C0U9063
GID ENV-021

Appendix G –
Slope Stability Analyses

6019-E-0004 - Assignment No.1
Trout Creek Culvert Replacement
North Embankment Slope (Inlet)
Drained Static Condition

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

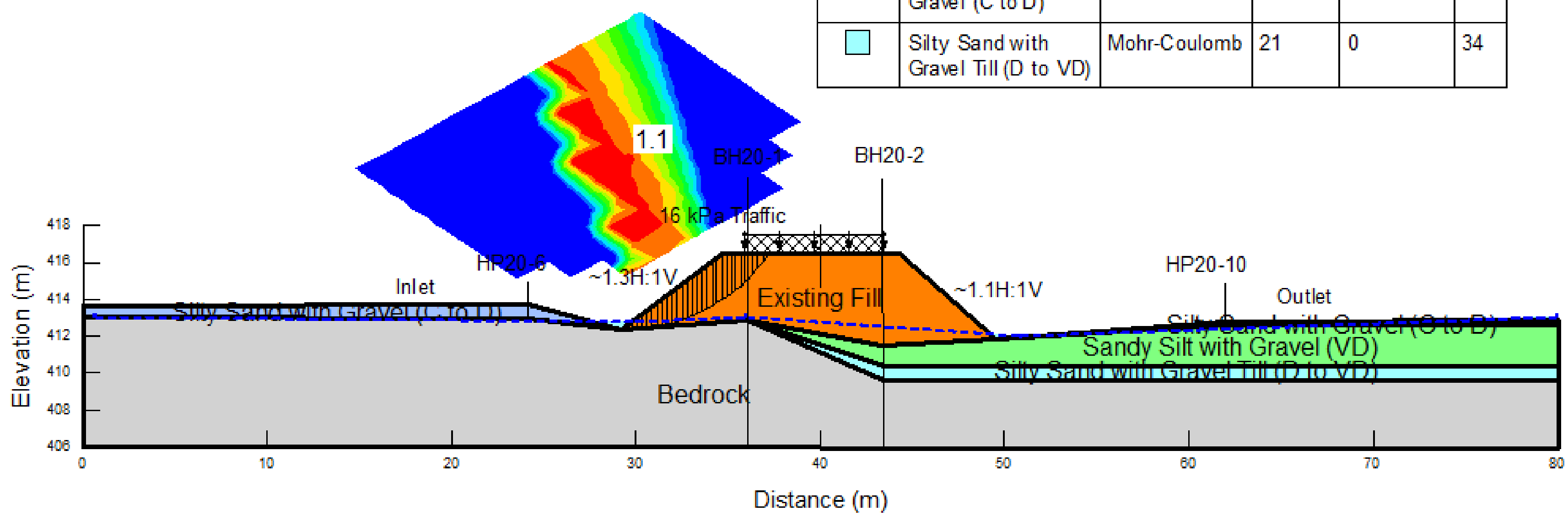


Figure G1: Slope stability analysis for existing north side of embankment (1.3H:1V) – drained static condition

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

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

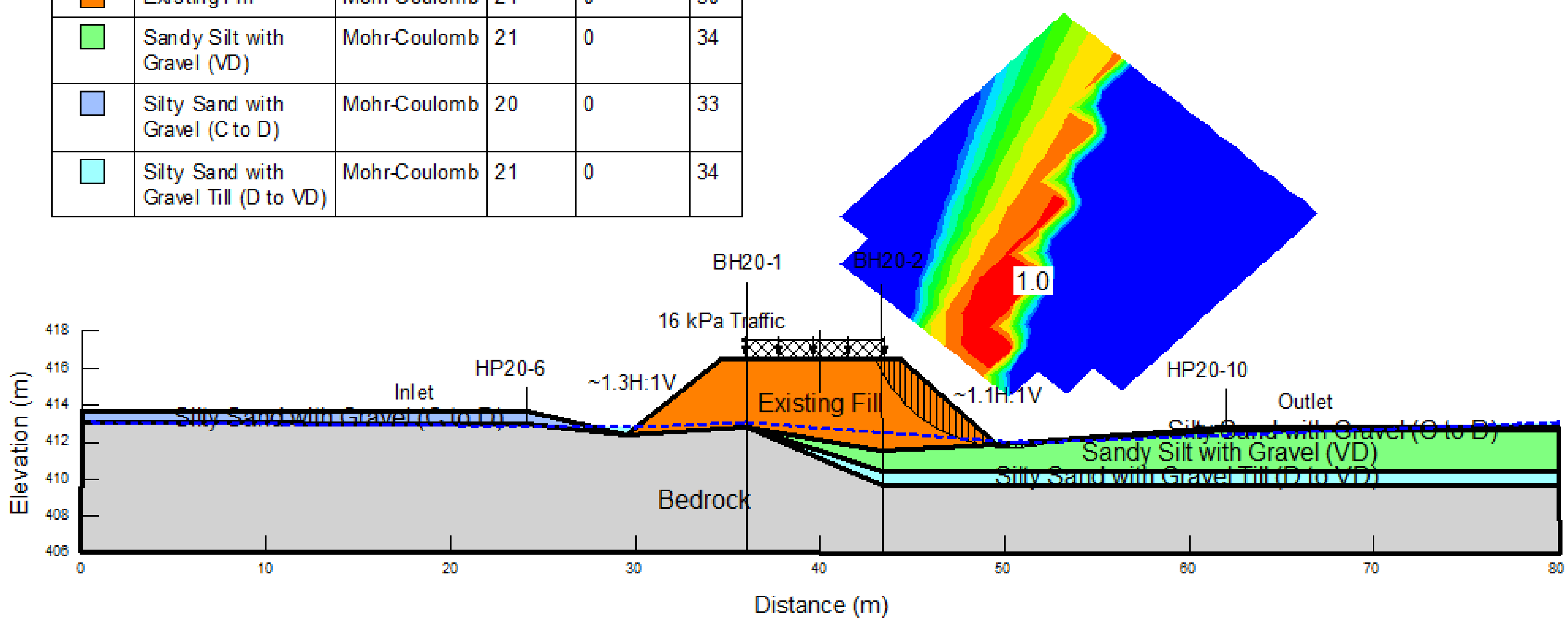


Figure G2: Slope stability analysis for existing south side of embankment (1.1H:1V) – drained static condition

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

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

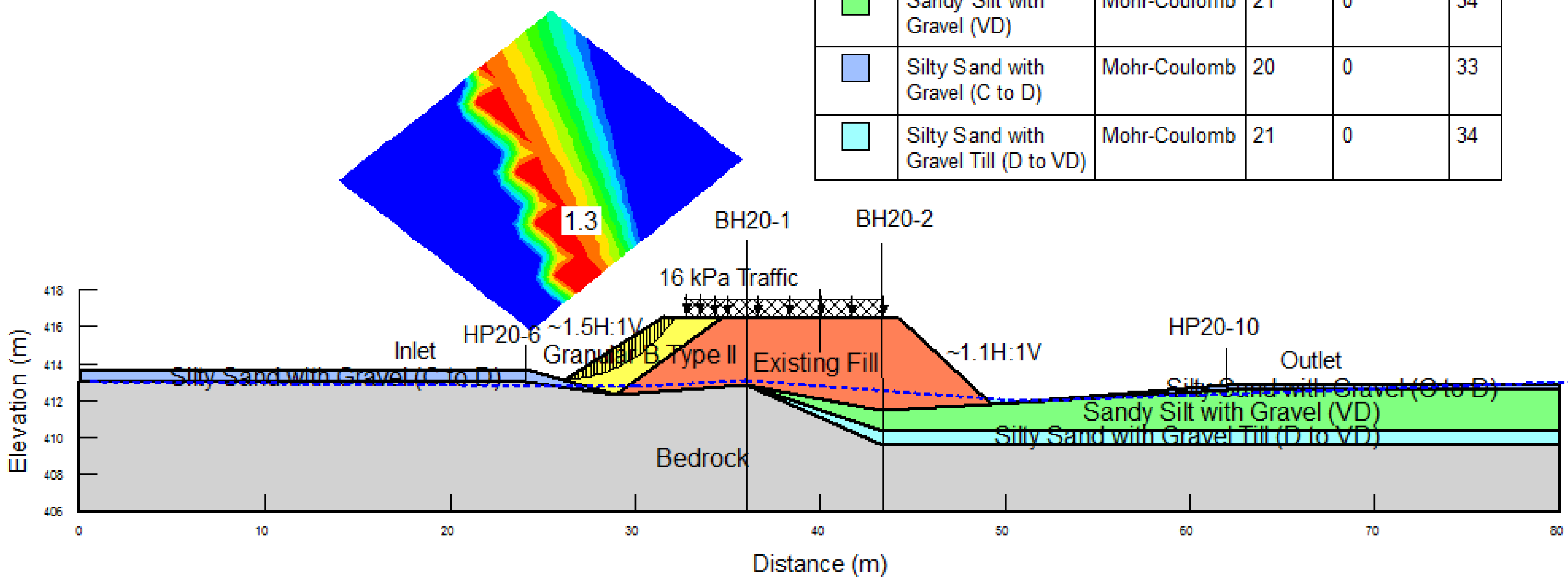


Figure G3: Slope stability analysis for temporary widening north side of embankment (1.5H:1V) – drained static condition

6019-E-0004 - Assignment No.1
Trout Creek Culvert Replacement
Temporary Modular Bridge
Drained Static Condition

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
	Abutment	Mohr-Coulomb	23	100	45
	Bedrock	Bedrock (Impenetrable)			
	Existing Fill	Mohr-Coulomb	21	0	30
	Granular B Type II	Mohr-Coulomb	22	0	35
	Sandy Silt with Gravel (VD)	Mohr-Coulomb	21	0	34
	Silty Sand with Gravel Till (D to VD)	Mohr-Coulomb	21	0	34

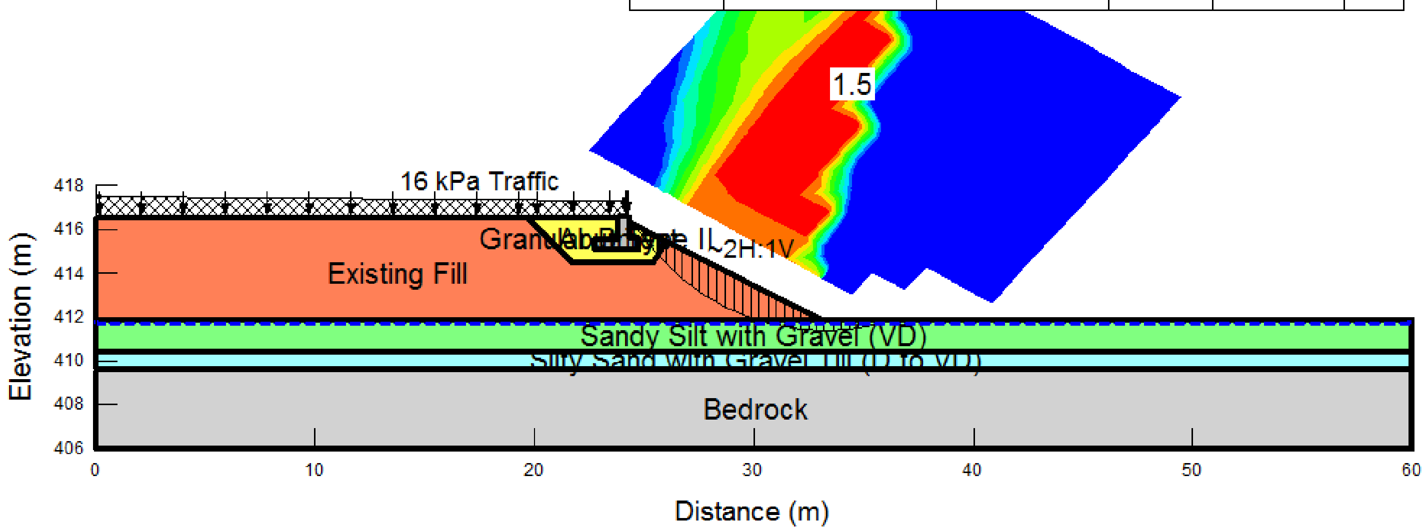


Figure G4: Slope stability analysis for Temporary Modular Bridge south side of embankment (forward slope 2H:1V) – drained static condition

exp.

6019-E-0004 - Assignment No.1
Trout Creek Culvert Replacement
North Embankment Slope (Inlet)
Drained Static Condition

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

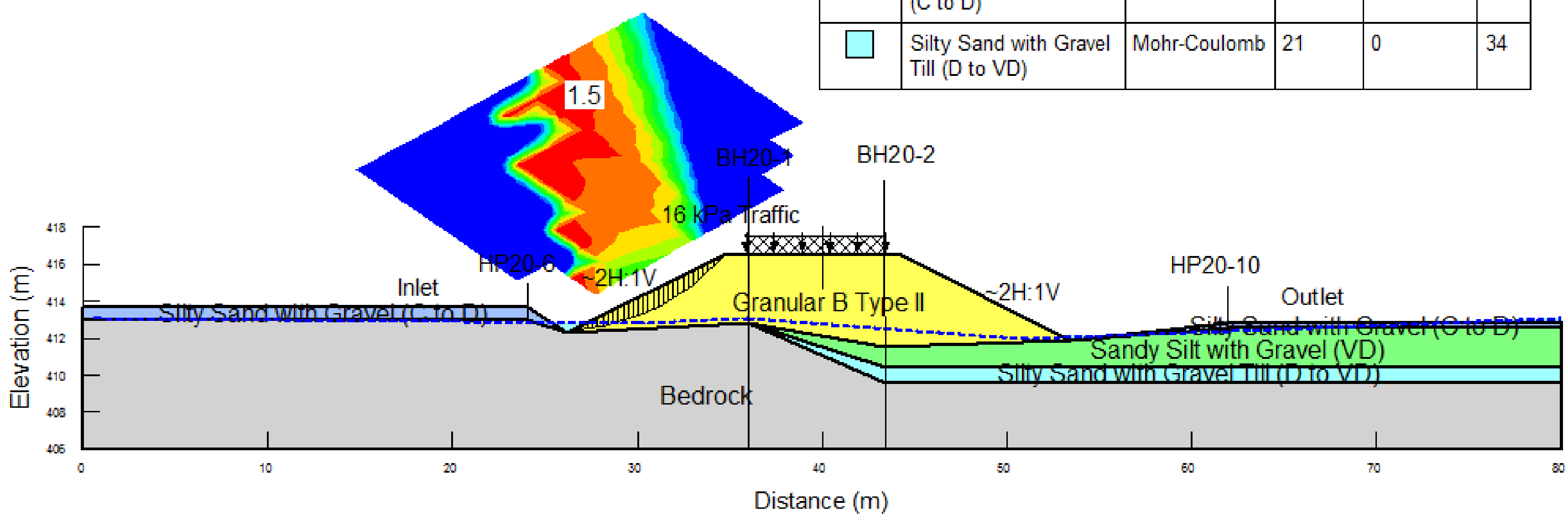


Figure G5: Slope stability analysis for north side of embankment (2H:1V) – drained static condition

6019-E-0004 - Assignment No.1
Trout Creek Culvert Replacement
North Embankment Slope (Inlet)
Drained Seismic Condition

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

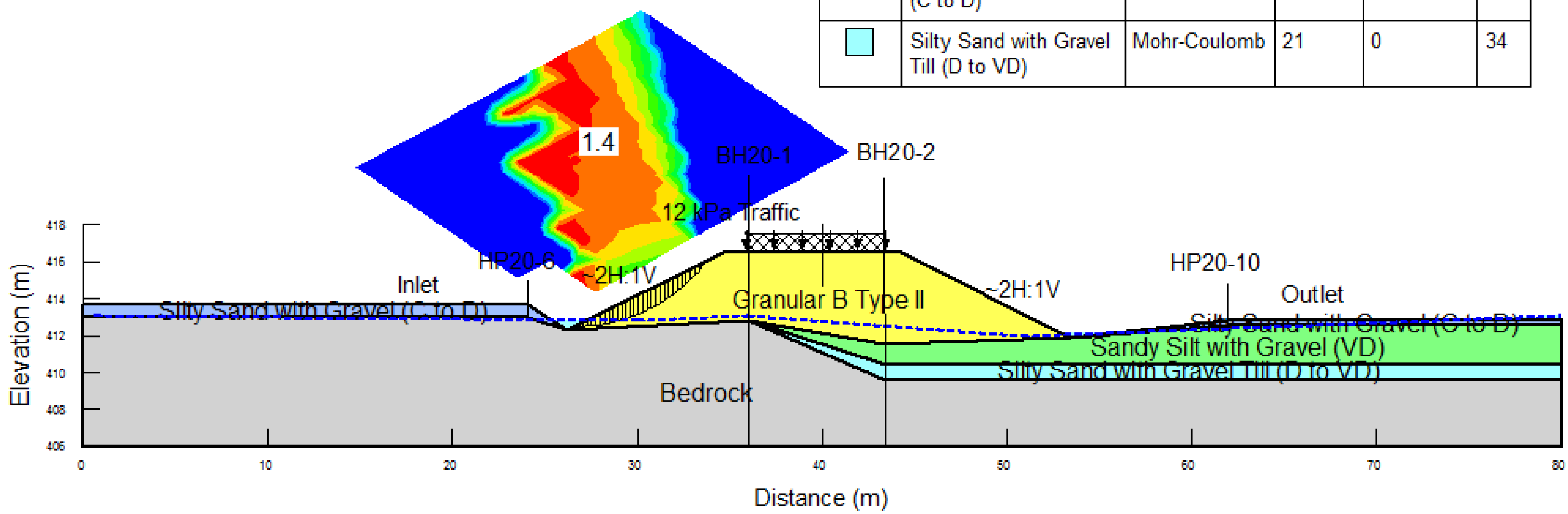


Figure G6: Slope stability analysis for north side of embankment (2H:1V) – drained seismic condition

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

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

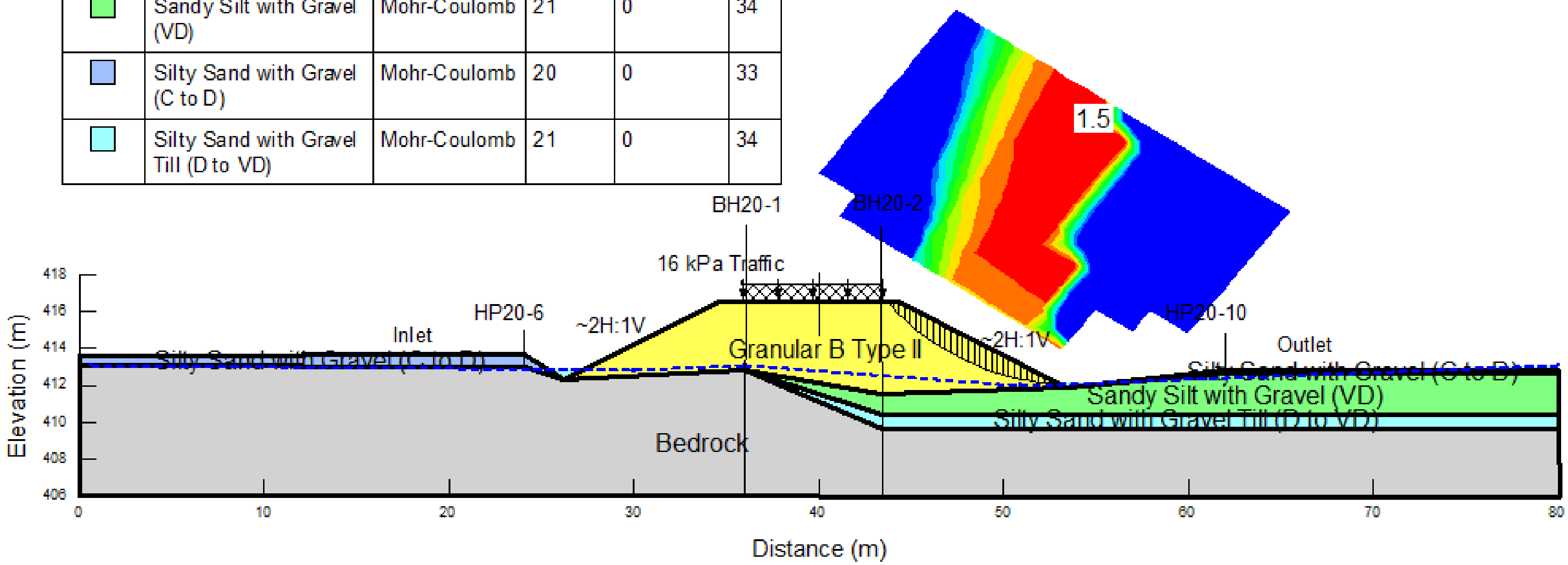


Figure G7: Slope stability analysis for south side of embankment (2H:1V) – drained static condition

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

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

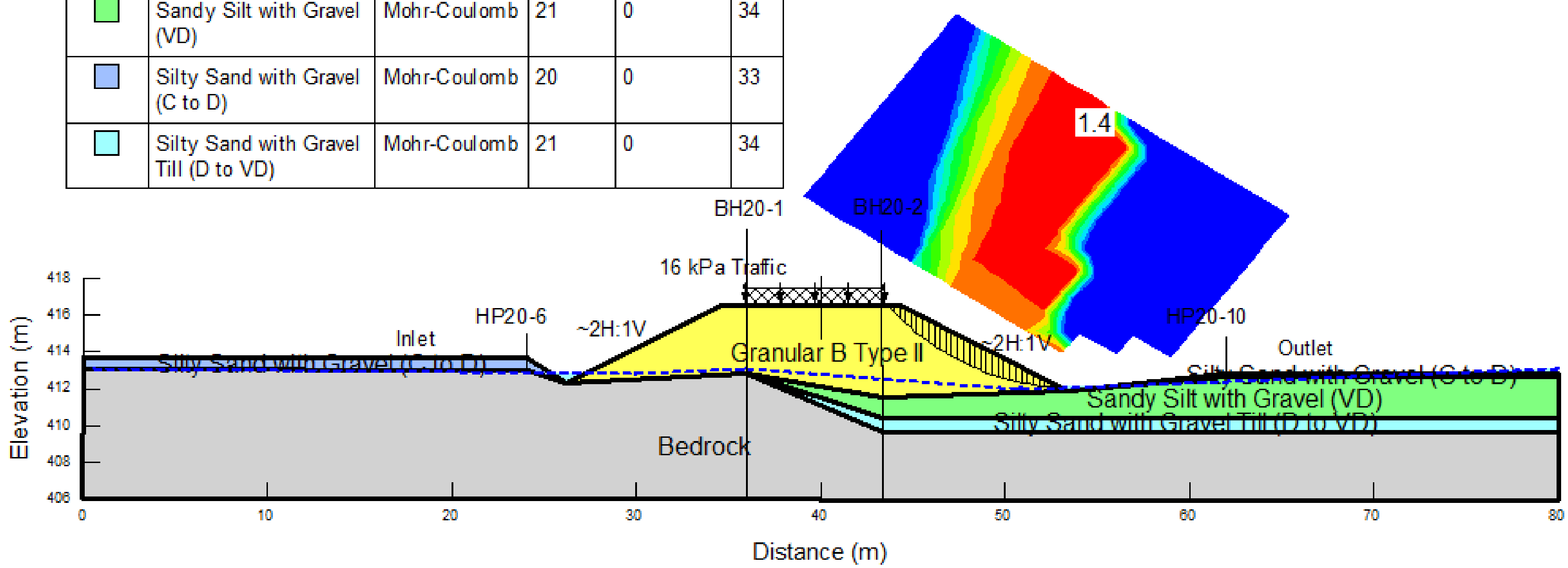
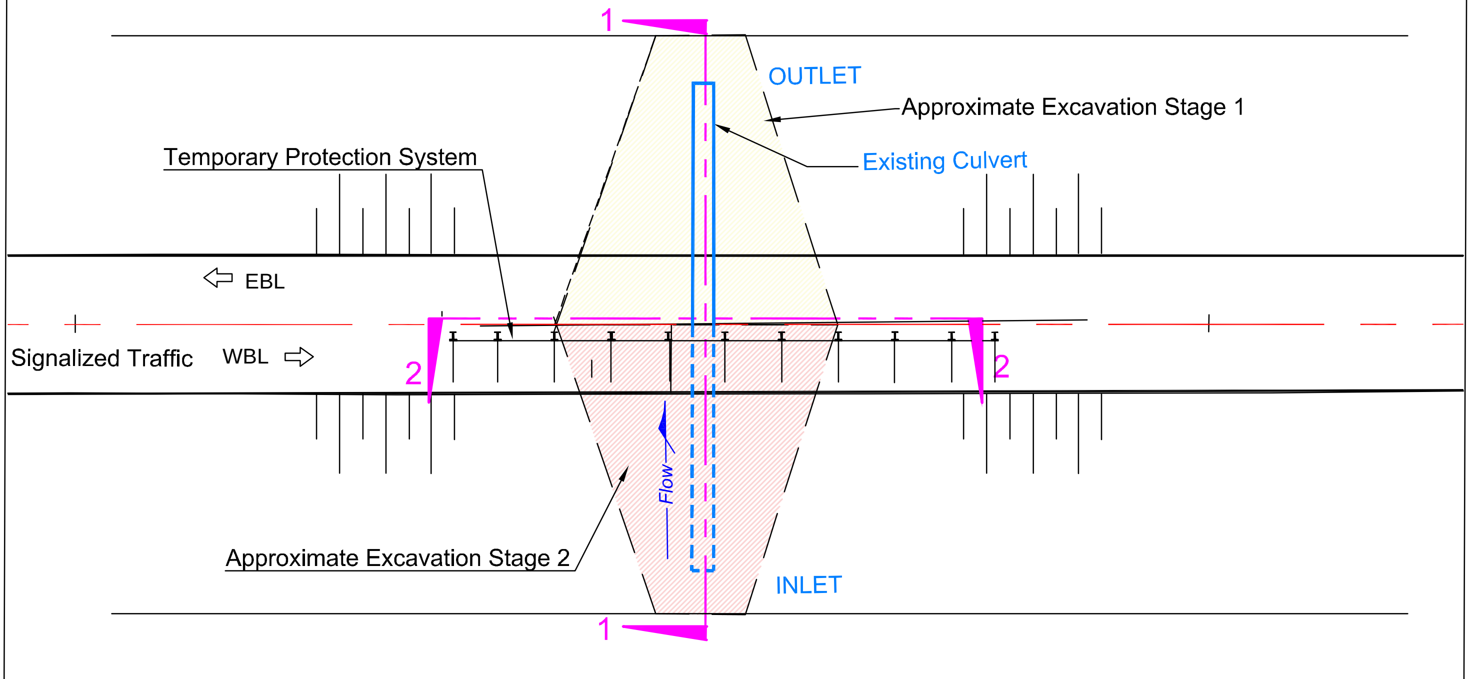


Figure G8: Slope stability analysis for south side of embankment (2H:1V) – drained seismic condition

Appendix H –
Schematic Sketches

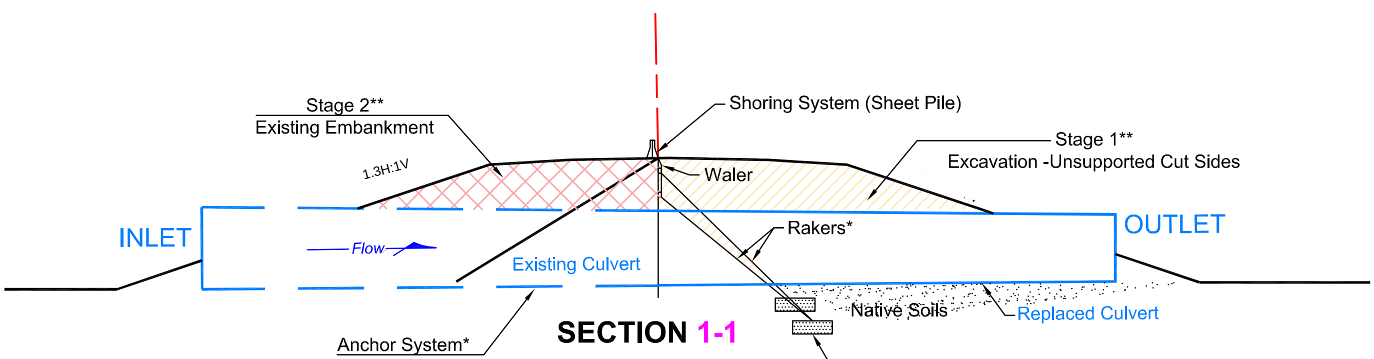
FIGURE H1A: HALF AND HALF CONSTRUCTION WITH UNSUPPORTED CUT SIDES

SCHEMATIC DIAGRAMS (NST)



PLAN

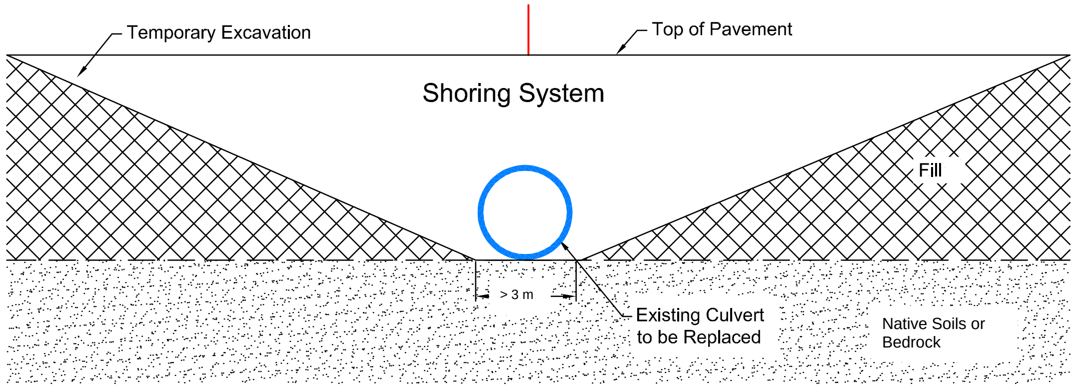
Half and Half Construction, Shoring System with either Cut or Anchor System - Unsupported Cut



SECTION 1-1

NOTES:

* Rakers or Anchor System
** Stage 2 Following Stage 1 in Opposite Way

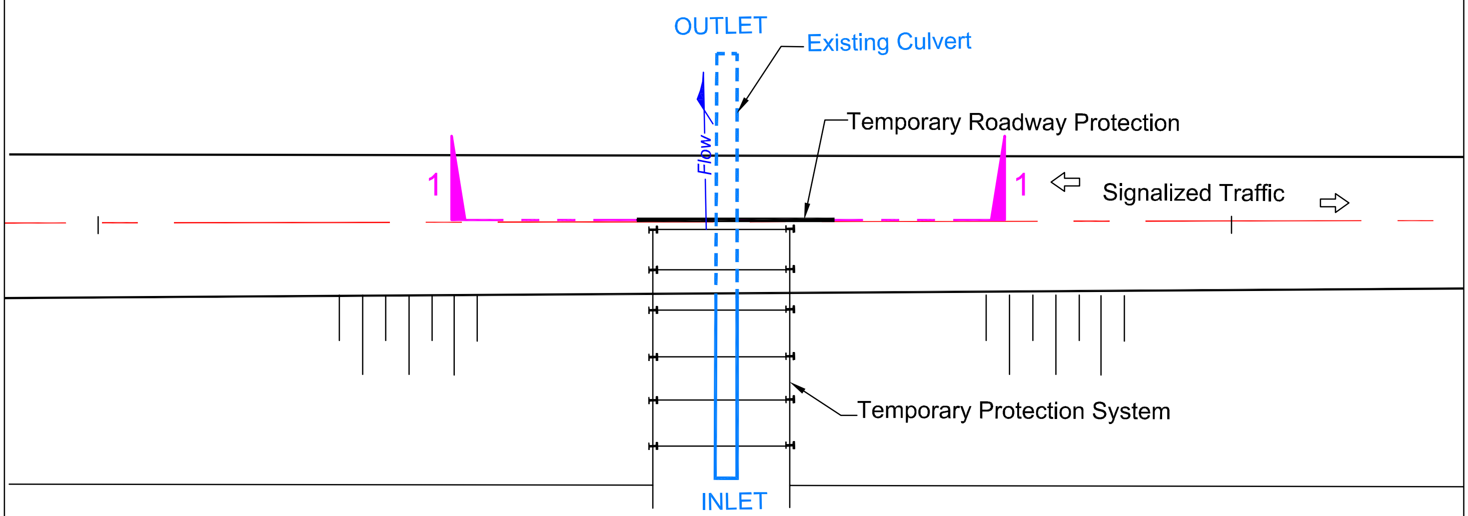


SECTION 2-2

FIGURE H1B: HALF AND HALF CONSTRUCTION WITH BRACED CUT SIDES OR ANCHOR SYSTEM

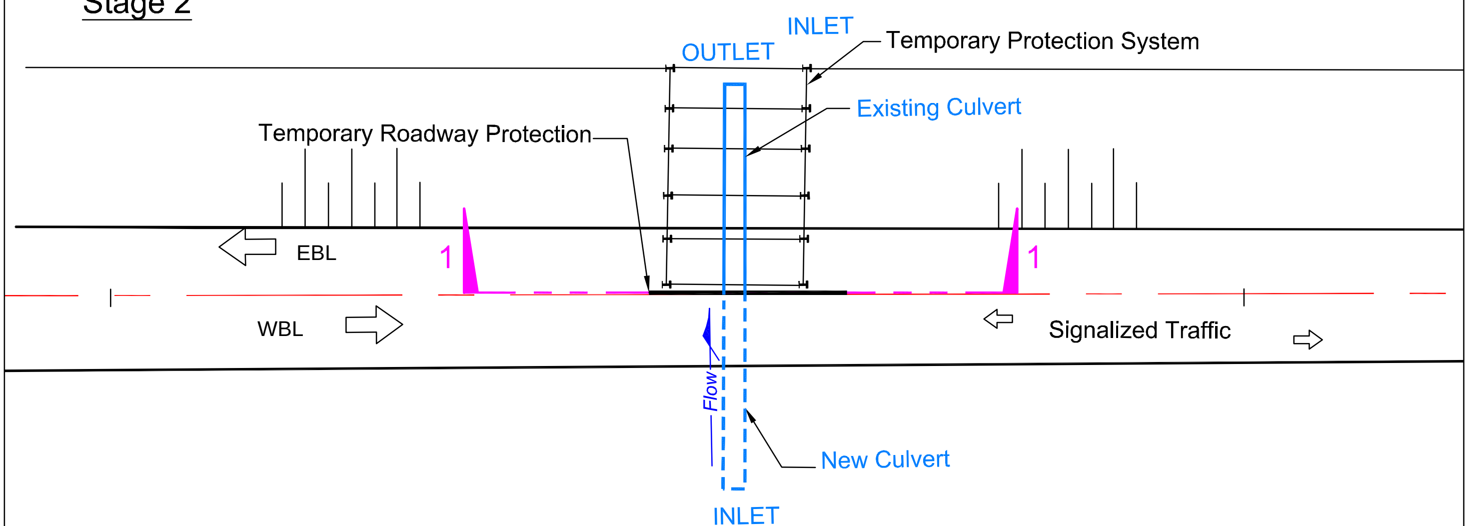
SCHEMATIC DIAGRAMS (NST)

Stage 1



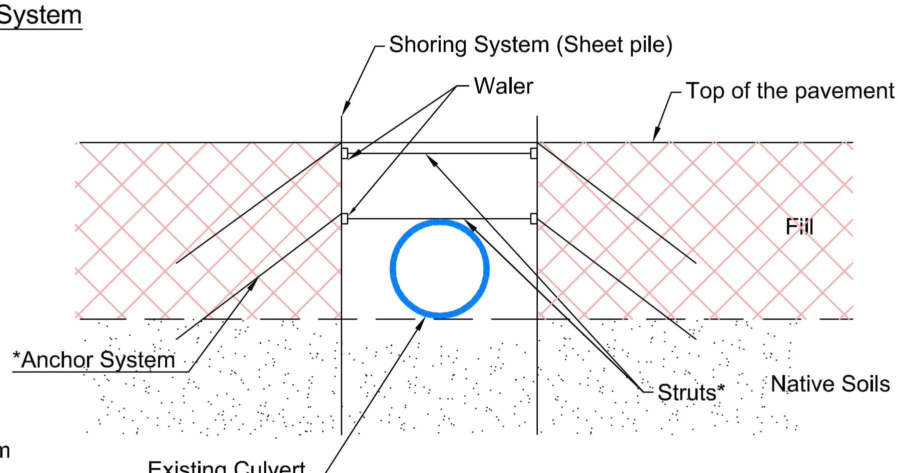
PLAN

Stage 2



PLAN

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

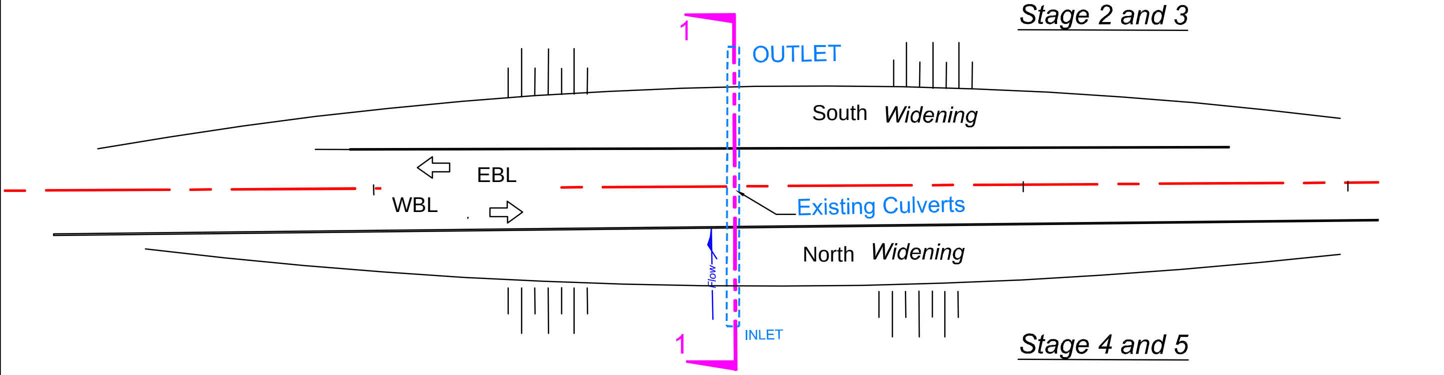


NOTE:
* Struts or Anchor System

SECTION 1-1

FIGURE H2: TEMPORARY WIDENING WITH STAGED OPEN CUT UNSUPPORTED EXCAVATION

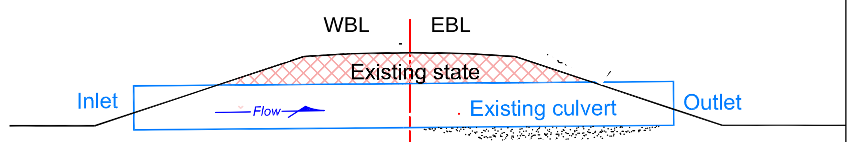
SCHEMATIC DIAGRAMS (NTS)



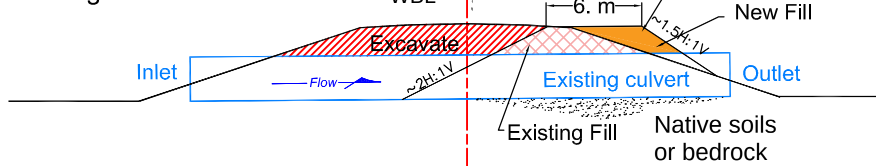
PLAN

RECOMMENDED STAGES

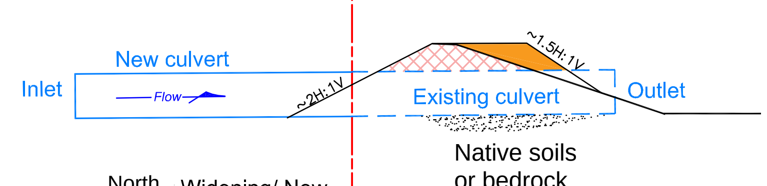
Stage 1 - Current condition;
Two-way traffic on existing road



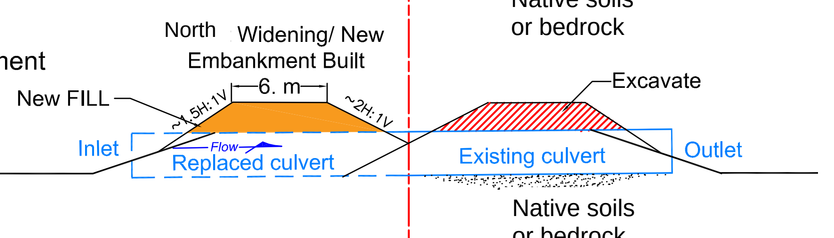
Stage 2 - Built temporary (SOUTH) widening detour
Excavation of embankment at east detour side
One-way traffic shifted to (SOUTH) widening



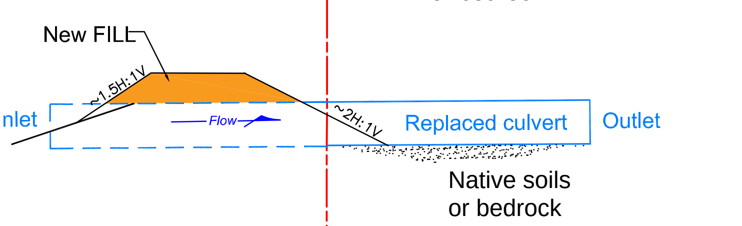
Stage 3 - One- way traffic on (SOUTH) widening detour
Installation of new culvert



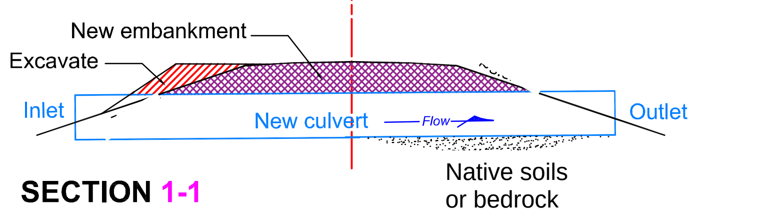
Stage 4 - Built new (NORTH) widening detour
Excavation on(SOUTH) widening detour
One-way traffic shifted to new embankment



Stage 5 - One- way traffic on new embankment
Complete culvert replacement



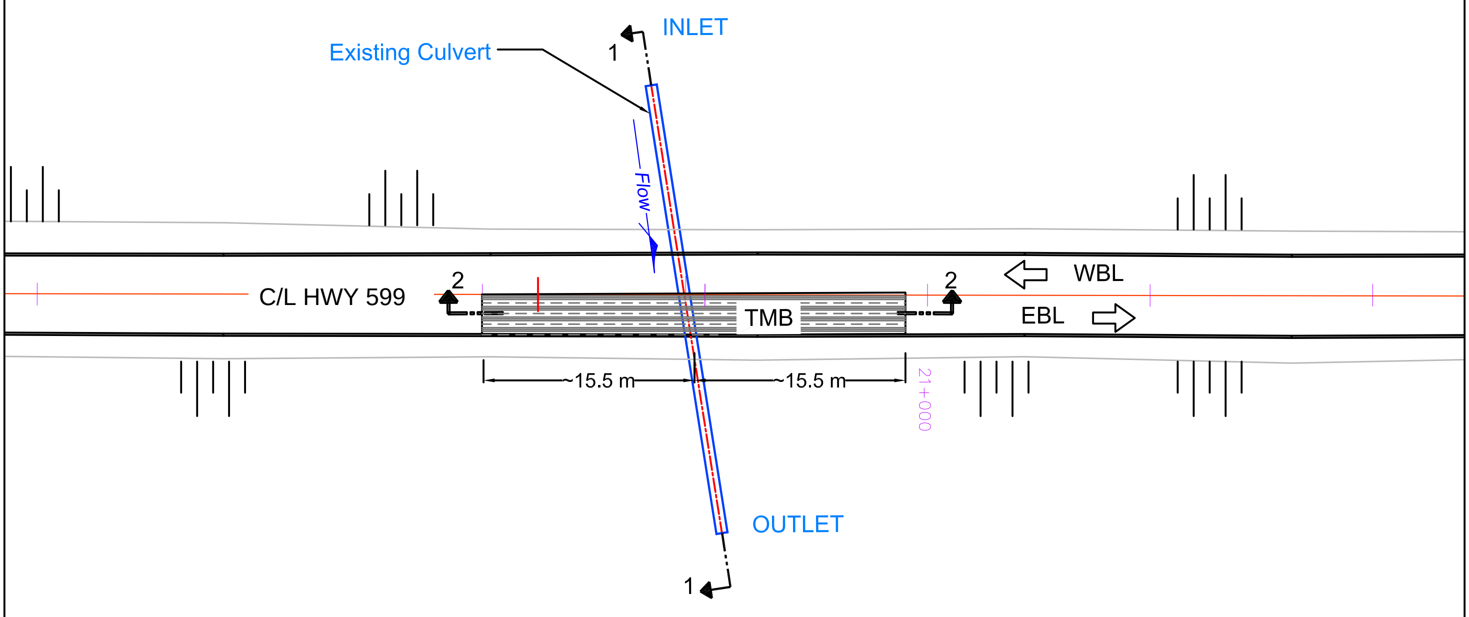
Stage 6 - Built new embankment to original state
Switching traffic between WBL and EBL
during full Height embankment construction
Two-way traffic return



SECTION 1-1

FIGURE H3: STAGED CONSTRUCTION WITH TEMPORARY MODULAR BRIDGE

SCHEMATIC DIAGRAMS (NST)



PLAN

RECOMMENDED STAGES

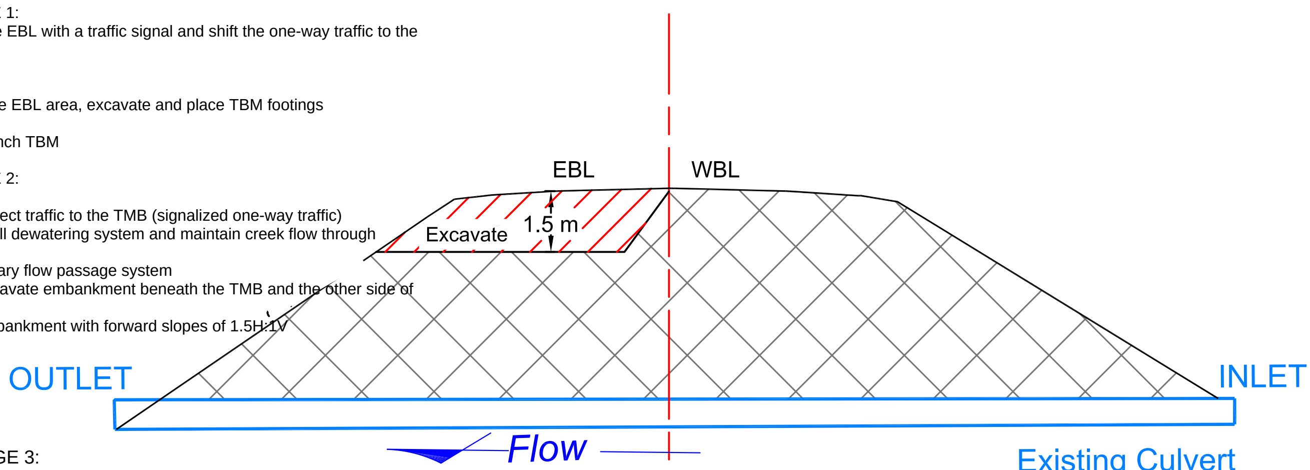
STAGE 1:
(i)Close EBL with a traffic signal and shift the one-way traffic to the WBL

(ii)At the EBL area, excavate and place TBM footings

(iii)Launch TBM

STAGE 2:

(i)Redirect traffic to the TMB (signalized one-way traffic)
(ii)Install dewatering system and maintain creek flow through temporary flow passage system
(iii) Excavate embankment beneath the TMB and the other side of the embankment with forward slopes of 1.5H:1V



SECTION 1-1

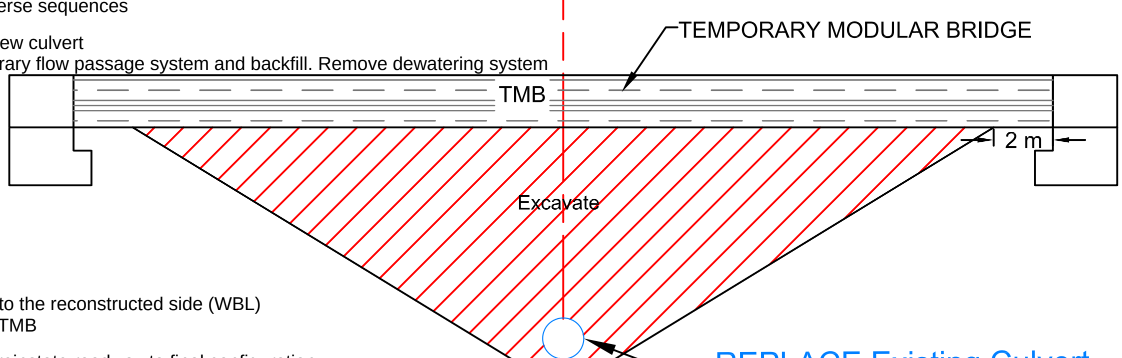
STAGE 3:

(i)Keep one-way traffic on the TMB

(ii)Remove the existing culvert and construct the new culvert

(iii)Backfill the reverse sequences

(iv)Divert flow to new culvert
(v)Remove temporary flow passage system and backfill. Remove dewatering system and place rip rap



STAGE 4:
(i)Switch traffic to the reconstructed side (WBL)
(ii)Remove the TMB

(iii)Backfill and reinstate roadway to final configuration

SECTION 2-2

APPENDIX I –
Seismic Hazard Values

2015 National Building Code Seismic Hazard Calculation

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

Site: 50.201N 90.726W

2021-02-22 20:53 UT

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

Notes: Spectral (Sa(T), where T is the period in seconds) and peak ground acceleration (PGA) values are given in units of g (9.81 m/s^2). Peak ground velocity is given in m/s. Values are for "firm ground" (NBCC2015 Site Class C, average shear wave velocity 450 m/s). NBCC2015 and CSAS6-14 values are highlighted in yellow. Three additional periods are provided - their use is discussed in the NBCC2015 Commentary. Only 2 significant figures are to be used. **These values have been interpolated from a 10-km-spaced grid of points. Depending on the gradient of the nearby points, values at this location calculated directly from the hazard program may vary. More than 95 percent of interpolated values are within 2 percent of the directly calculated values.**

References

National Building Code of Canada 2015 NRCC no. 56190; Appendix C: Table C-3, Seismic Design Data for Selected Locations in Canada

Structural Commentaries (User's Guide - NBC 2015: Part 4 of Division B)
Commentary J: Design for Seismic Effects

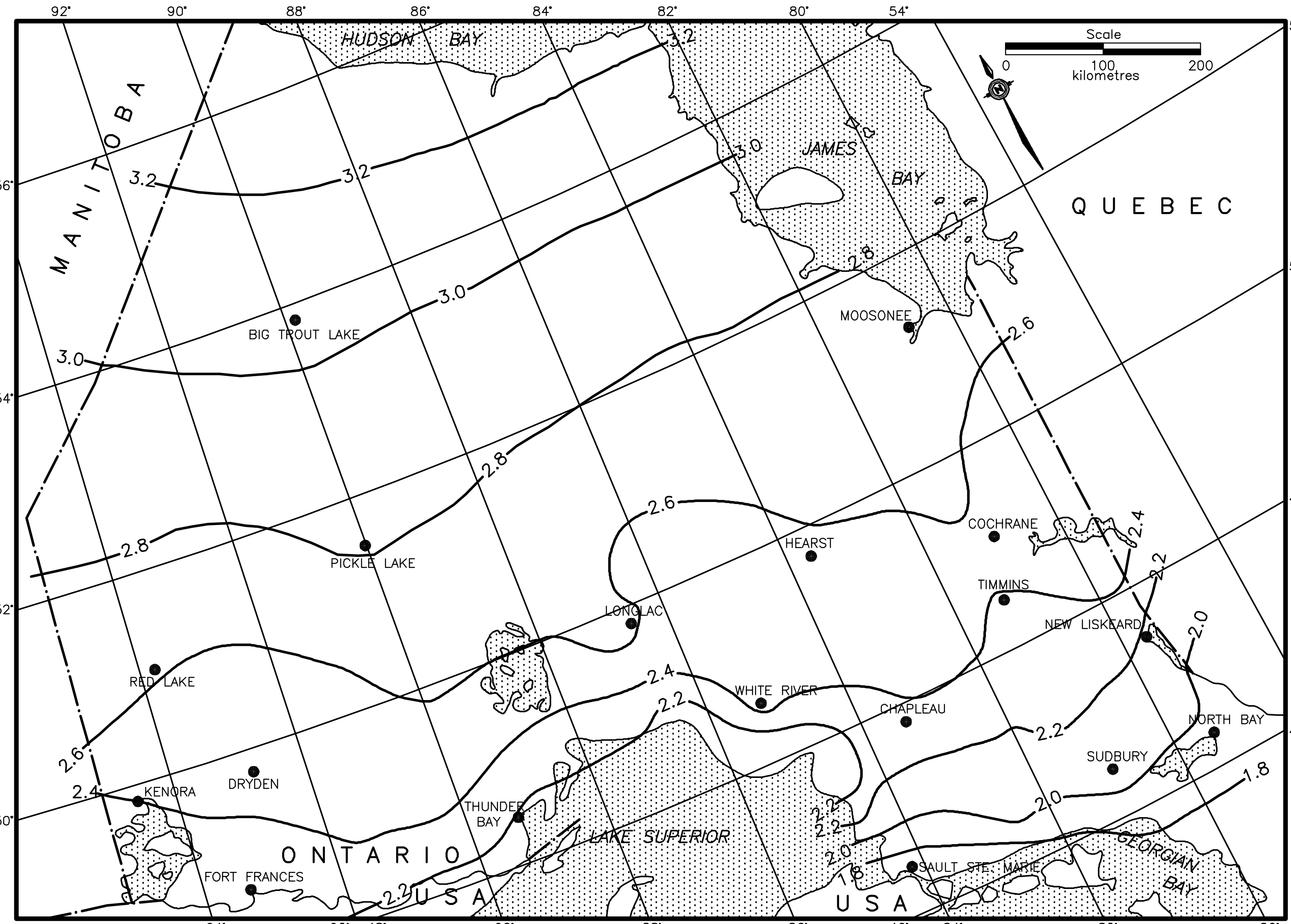
Geological Survey of Canada Open File 7893 Fifth Generation Seismic Hazard Model for Canada: Grid values of mean hazard to be used with the 2015 National Building Code of Canada

See the websites www.EarthquakesCanada.ca and www.nationalcodes.ca for more information

Natural Resources Canada Ressources naturelles Canada

Canada

APPENDIX J –
OPSD's



NOTES:

A These values are approximate and should only be used where the recommendations of a geotechnical engineer are not available.

B This information is based on the Ministry of Transportation and Communications Research Publication RR225 "Aspects of Prolonged Exposure of Pavements to Sub-Zero Temperatures" dated December 1981.

C Values between contours should be interpolated. If interpolation is not possible, use the adjacent contour with the greater depth.

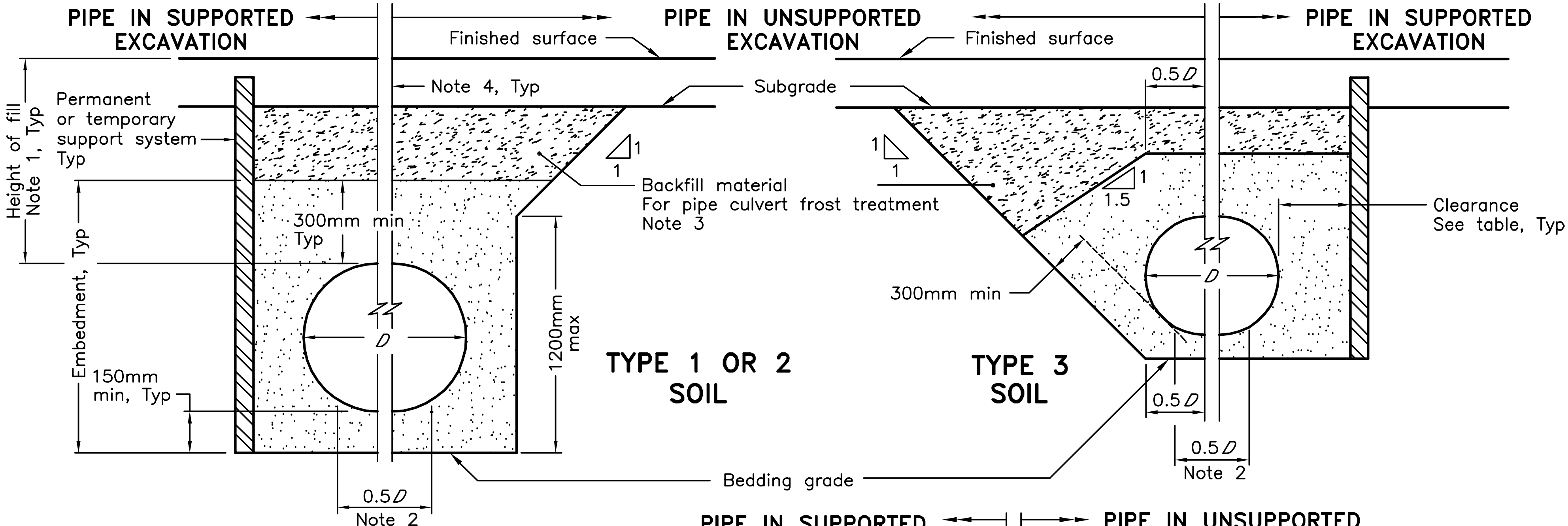
D Frost penetration depths are in metres.

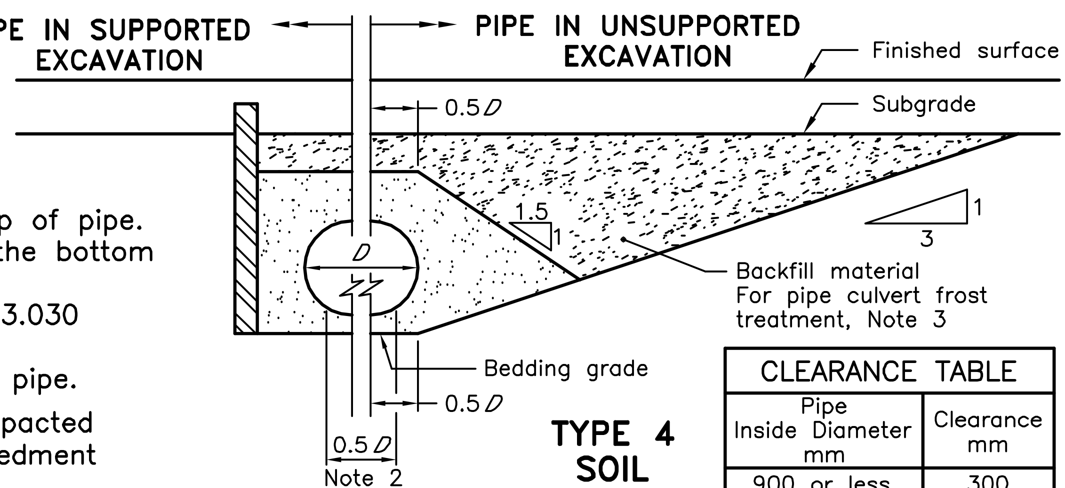
ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2010 | Rev 1

FOUNDATION FROST PENETRATION DEPTHS FOR NORTHERN ONTARIO

OPSD 3090.100





LEGEND:

D – Inside diameter

NOTES:

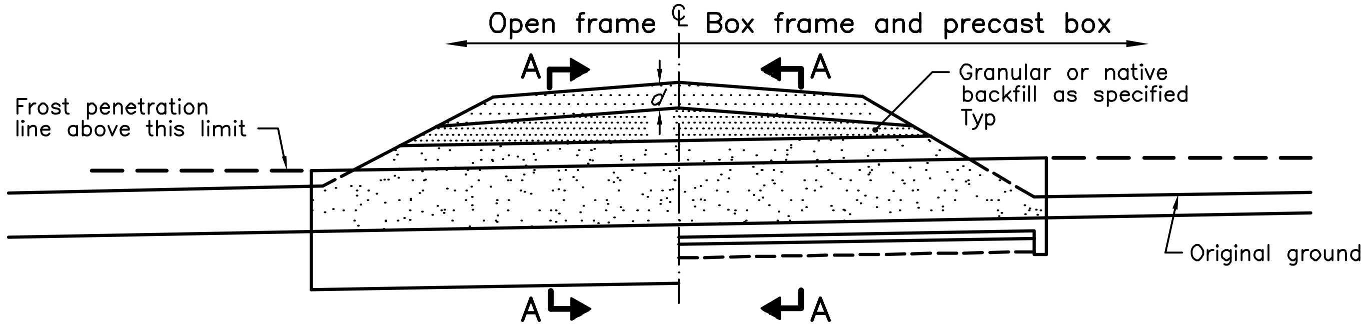
1 Height of fill is measured from the finished surface to top of pipe.
2 The pipe bed shall be compacted and shaped to receive the bottom of the pipe.
3 Pipe culvert frost treatment shall be according to OPSD 803.030 and 803.031.
4 Condition of excavation is symmetrical about centreline of pipe.

A Granular material placed in the haunch area shall be compacted prior to placing and compacting the remainder of the embedment material.
B Soil types as defined in the Occupational Health and Safety Act and Regulations for Construction Projects.
C All dimensions are in metres unless otherwise shown.

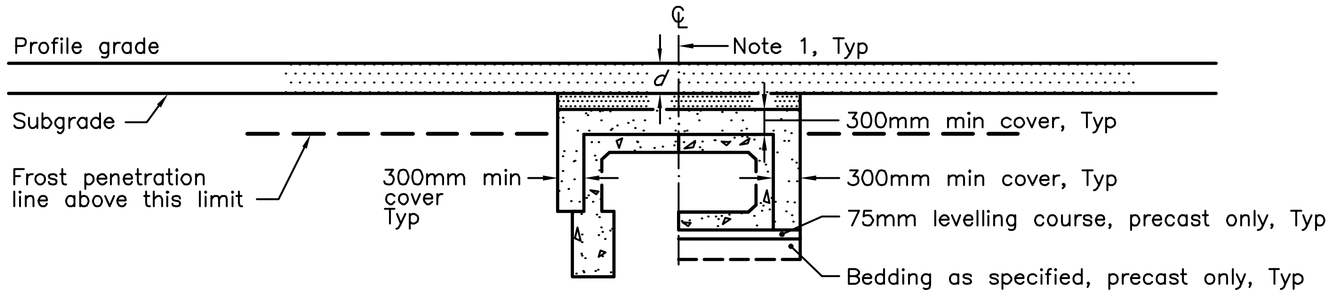
CLEARANCE TABLE	
Pipe Inside Diameter mm	Clearance mm
900 or less	300
Over 900	500

ONTARIO PROVINCIAL STANDARD DRAWING	Nov 2014	Rev 3
FLEXIBLE PIPE EMBEDMENT AND BACKFILL EARTH EXCAVATION		
	OPSD 802.010	

FROST PENETRATION LINE AT OR ABOVE TOP OF CULVERT

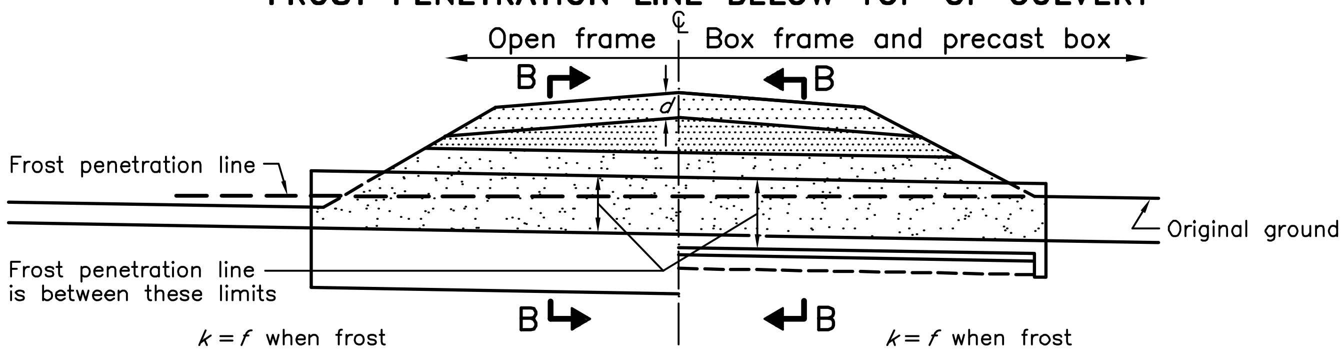


LONGITUDINAL SECTION

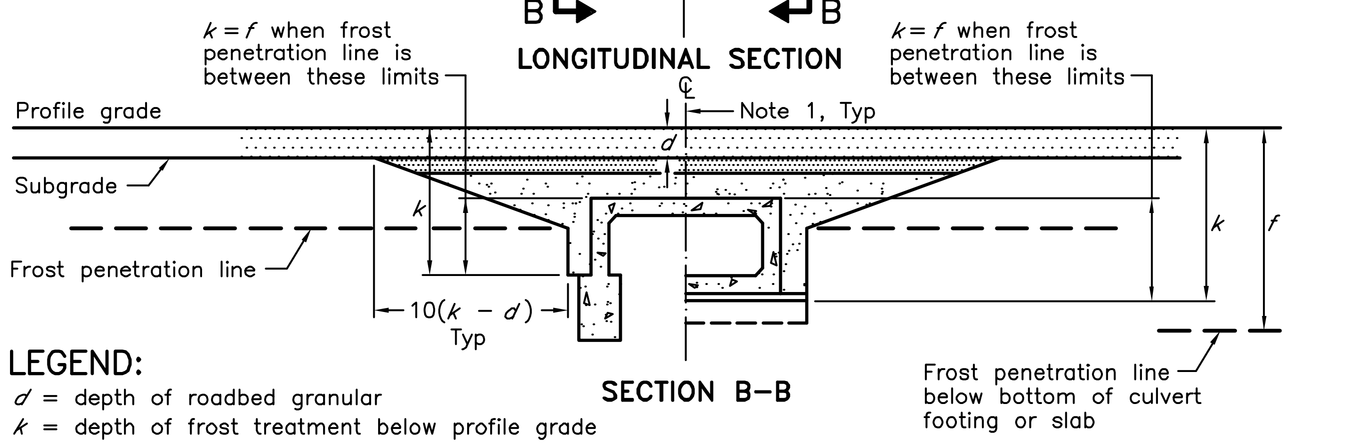


SECTION A–A

FROST PENETRATION LINE BELOW TOP OF CULVERT



LONGITUDINAL SECTION



SECTION B–B

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.

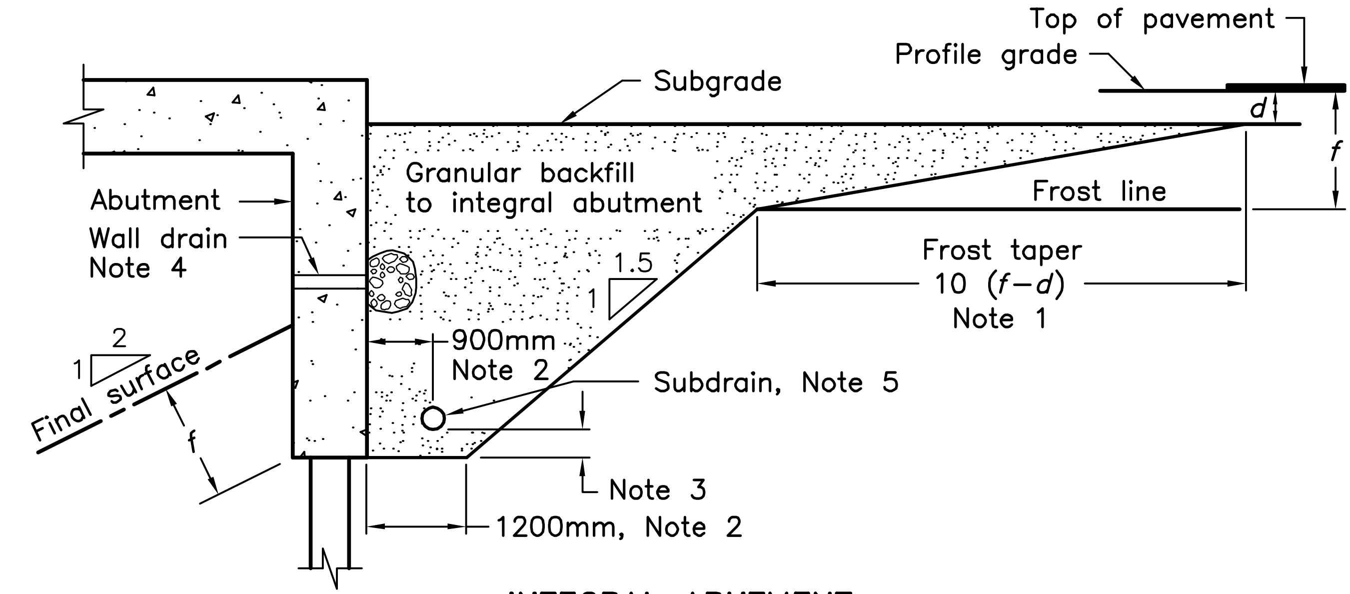
B The depth of roadbed granular shall be 600mm minimum.

C The maximum depth of frost treatment shall be bottom of box frame or top of footing.

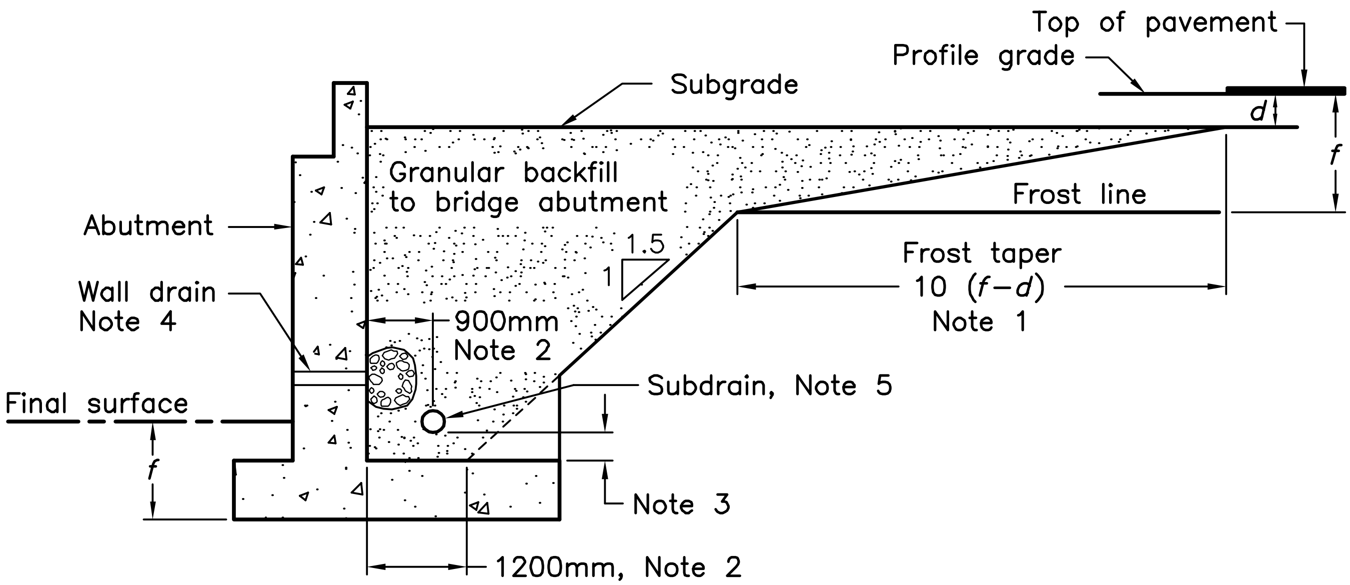
D All dimensions are in millimetres unless otherwise shown.

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	

ONTARIO STANDARDS · MUNICIPAL – PROVINCIAL · OPS



INTEGRAL ABUTMENT



ABUTMENT

NOTES:

1 d = depth of combined base and subbase courses
  f = frost penetration depth as specified

2 Dimensions perpendicular to back face of abutment.

3 Height to be consistent with positive drainage of subdrain as specified.

4 Where specified, wall drains shall be installed according to OPSD 3190.100.

5 150mm dia perforated pipe subdrain wrapped with geotextile.

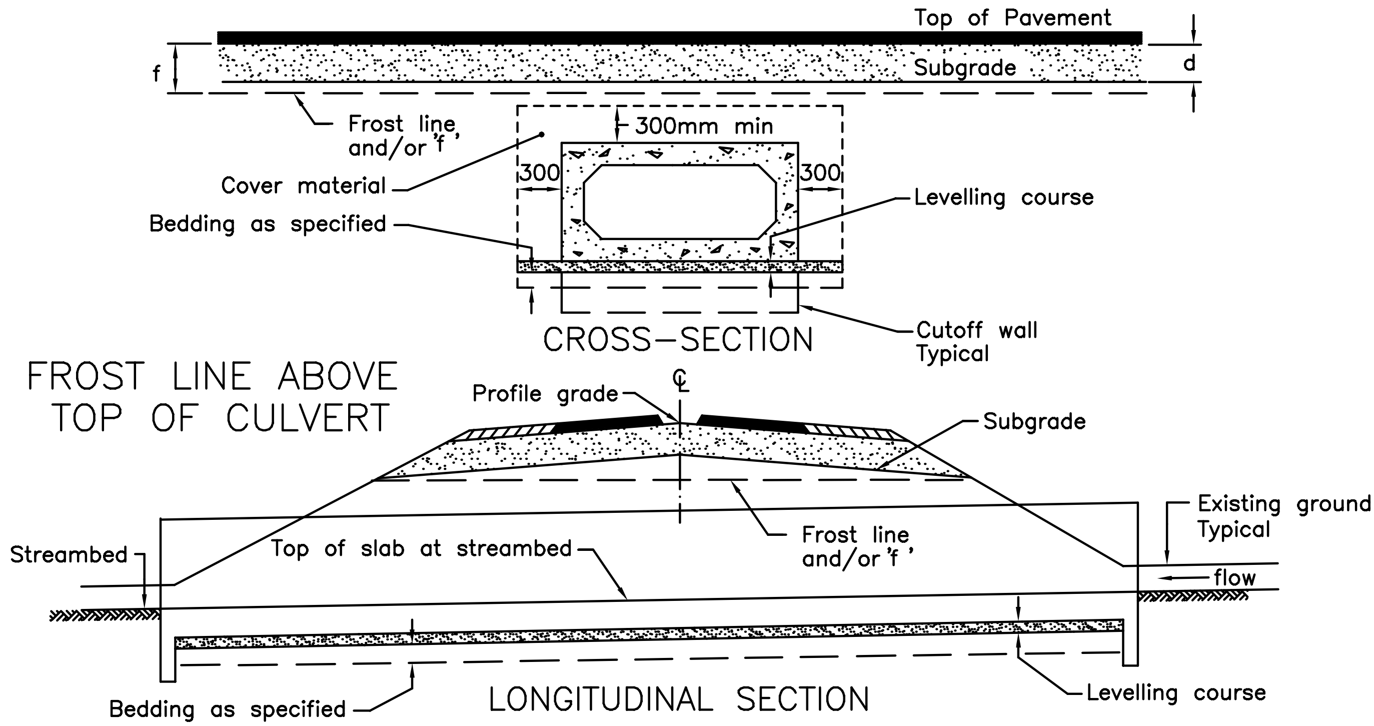
A Lateral limits of granular backfill to bridge abutment to be inside face to inside face of retaining wall or wingwall. Frost taper shall extend the full width of the backfill unless interrupted by the retaining wall or wingwall.

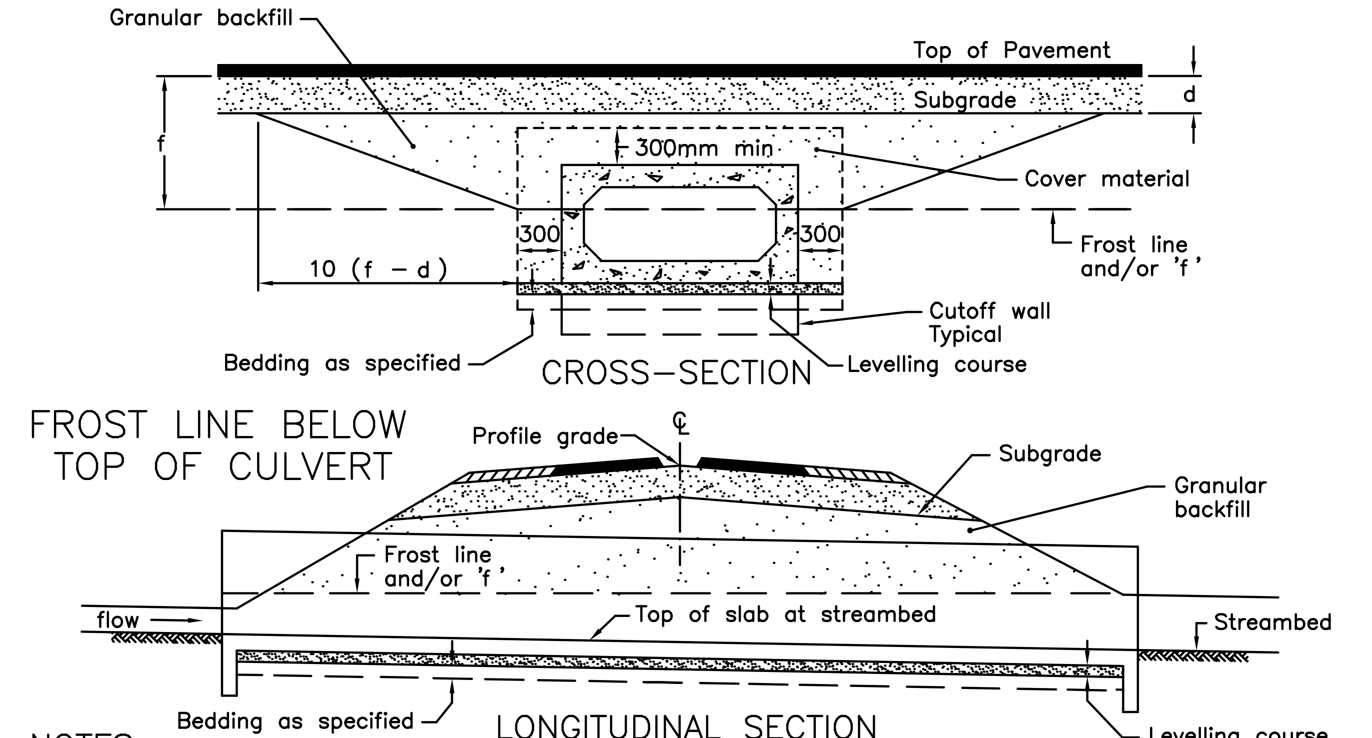
B Sections shown are parallel to centreline of roadway.

C Subdrain shall be installed with a 2% gradient behind wall.

D All dimensions are in millimetres unless otherwise shown.

ONTARIO PROVINCIAL STANDARD DRAWING	Nov 2010	Rev 1
WALLS ABUTMENT, BACKFILL MINIMUM GRANULAR REQUIREMENT		
	OPSD 3101.150	





NOTES:

A Bedding, levelling, cover and backfill material to be granular as specified.

B Where frost line is below bottom of levelling course, frost tapers start at the bottom of levelling course.

C All dimensions are in millimetres unless otherwise shown.

LEGEND:

d = Denotes depth of granular (roadbed)

f = Depth of frost treatment=____ (measured from profile grade)

MINISTRY OF TRANSPORTATION ONTARIO DRAWING	Date	1994 05 25	Rev
BEDDING AND BACKFILL FOR PRECAST CONCRETE BOX CULVERTS	Issue Date WP Issued by		
	MTOD − 803.021		

- A drainage system behind a retaining structure should ensure that a groundwater table does not exist above the footing level. Preferably, the ground water level is controlled by the use of free-draining granular backfill and a collection system such as weep holes or perforated drains at the footing level. These weep holes and drains should be inspected and maintained to ensure that they do not become blocked. If free-draining, granular backfill is not employed, the permeability of the backfill and the hydrostatic head will control the extent to which the groundwater table can be depressed locally by seepage towards a footing drain. In practice, design for frost protection is best done using free-draining backfill.
  The design should also consider the risk of unusually large inflows of water creating a temporary hydrostatic head of water behind the wall. An example is the overtopping of a retaining wall, adjoining a large body of water, by storm waves. Measures such as the use of quarried rock backfill, design for full hydrostatic pressure, or provision of a sloped impermeable surface layer should be considered.
  Measurements have shown that earth pressures can vary seasonally, but the effects have normally been neglected in design, except for winter frost pressures. These latter can be very large if the backfill is frost susceptible and for this reason free-draining granular backfill is recommended.
- Figure C6.20 shows examples of minimum backfill requirements.
  The distance, *x*, should be equal to or greater than the estimated vertical frost penetration. This distance may be reduced if the wall abuts a vertical face of bedrock that is not susceptible to frost. The frost penetration may be reduced by the use of suitable insulation, in which case a thermal analysis should be performed by a Geotechnical Engineer.
  If rock fill is used as a backfill material, consideration should be given to the possible deterioration of the rockfill with time, which could result in the reduction or even the total loss of free-draining properties and, hence, increased frost susceptibility.

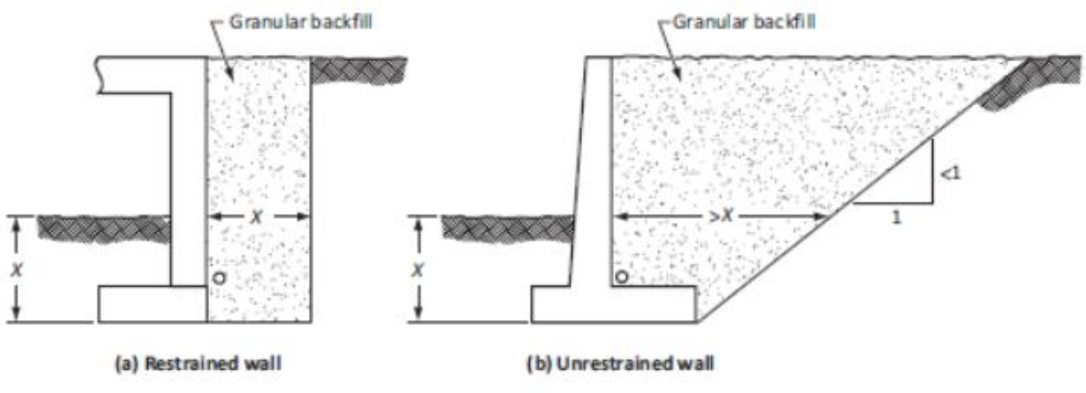
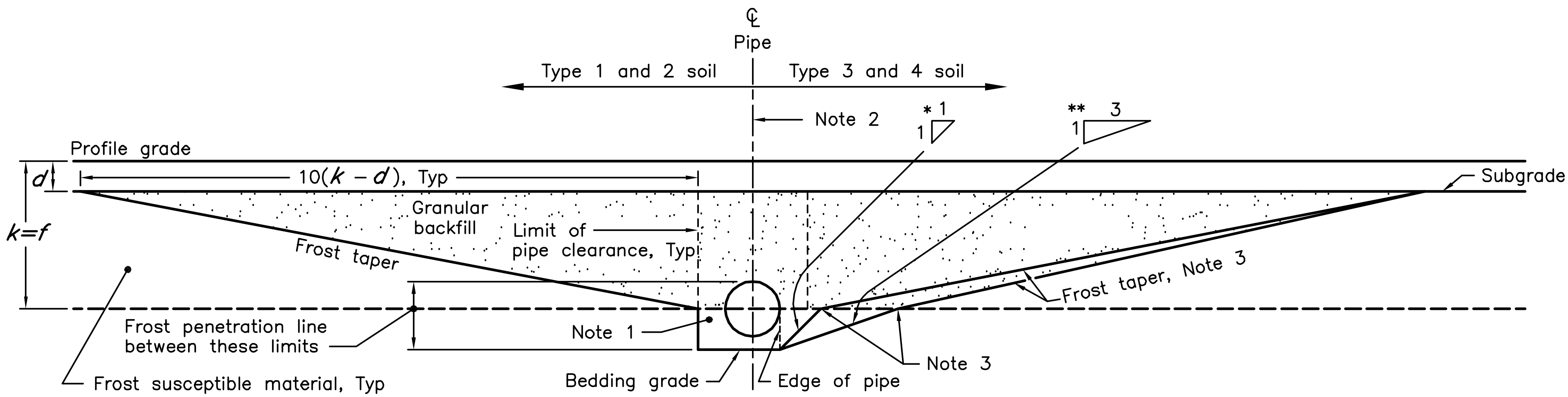


Figure C6.20
Backfill for frost protection
(See Clause C6.12.1.)

C6.12.2 Lateral ground pressures

C6.12.2.1 General

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



FROST TREATMENT
RIGID AND FLEXIBLE PIPE

NOTES:

1 Pipe embedment or bedding, cover, and backfill shall be according to:
   a) Flexible OPSD 802.010, 802.013, 802.014, 802.020, 802.023, and 802.024.
   b) Rigid – OPSD 802.030, 802.031, 802.032, 802.033, 802.034, 802.050, 802.051, 802.052, 802.053, and 802.054.

2 Condition of frost treatment symmetrical about centreline of pipe.

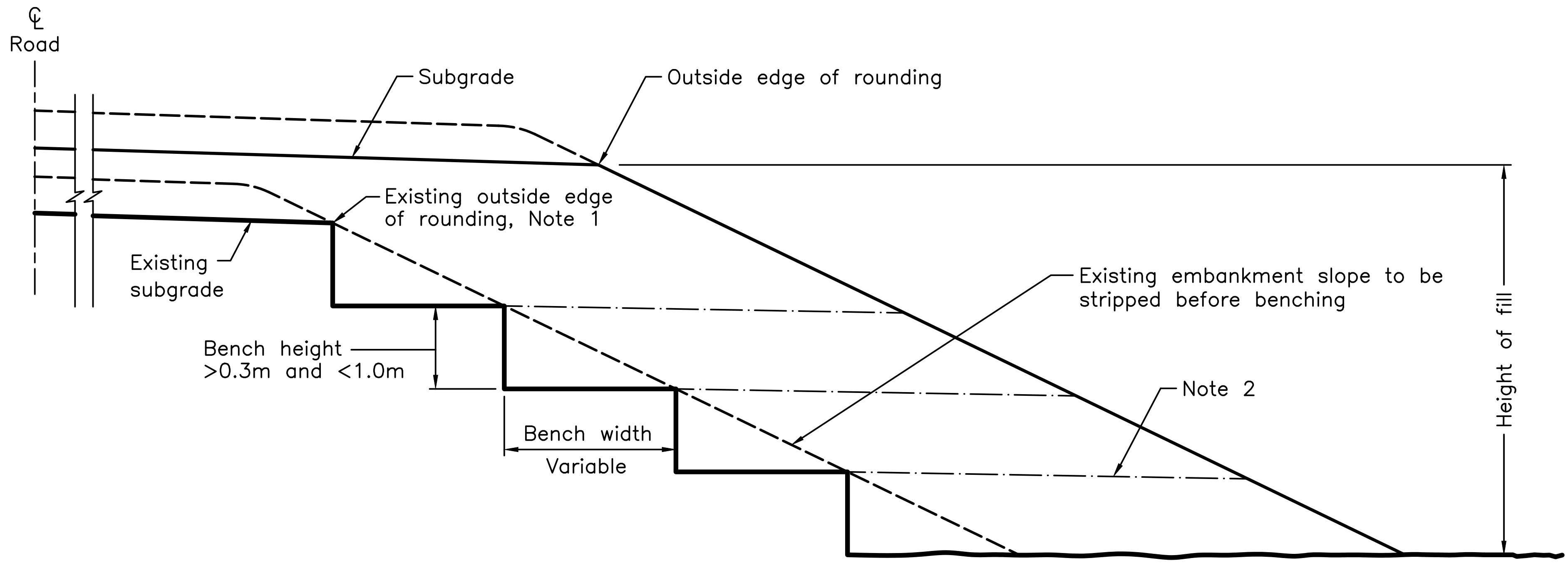
3 Frost tapers shall start at the intersection of the 1H:1V or 3H:1V slope and the frost penetration line.

A Soil types as defined in the Occupational Health and Safety Act and Regulations for Construction Projects.

LEGEND:

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

ONTARIO PROVINCIAL STANDARD DRAWING	Nov 2015	Rev	4
FROST TREATMENT – PIPE CULVERTS FROST PENETRATION LINE BETWEEN TOP OF PIPE AND BEDDING GRADE	OPSD 803.031		



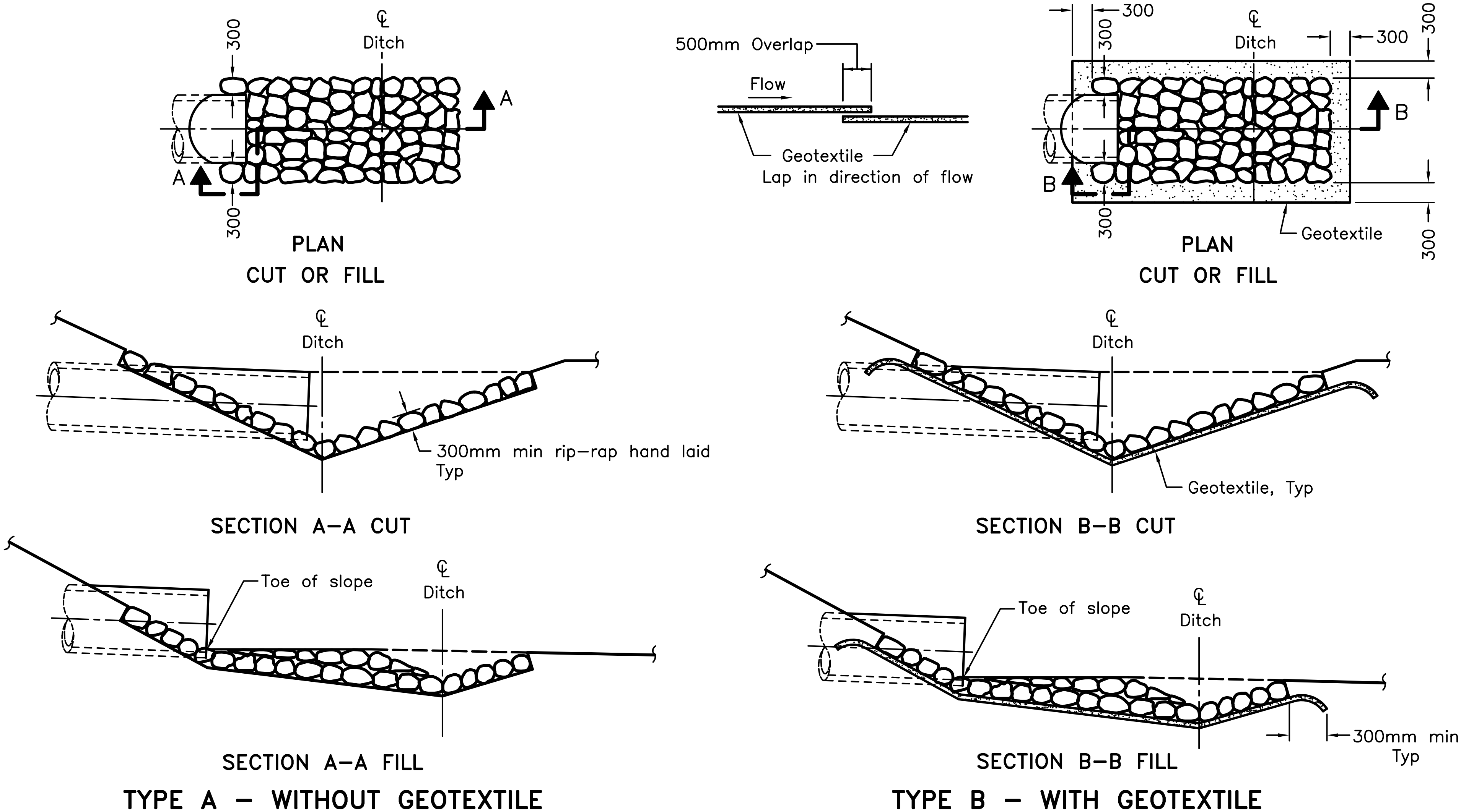
NOTES:

1 When the subgrade is below the existing outside edge of rounding, benching shall be carried out below the point where the subgrade intersects the existing slope.

2 Benches shall be excavated one level at a time and the fill placed and compacted before the next bench is excavated.

A Benching is not required on existing slopes flatter than 3H:1V.

ONTARIO PROVINCIAL STANDARD DRAWING	Apr 2019	Rev 4
BENCHING OF EARTH SLOPES		
	OPSD 208.010	



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 K –
Non-Standard Special Provisions (NSSP’s)

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 as well as shallow bedrock as encountered in various boreholes advanced at the site. Therefore, appropriate equipment and procedures will be required for excavation of fill/rock and installation of cofferdams/protection systems, if any.

Basis of Payment

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

NSSP FOR REMOVAL OF PROTECTION SYSTEM

Scope of Work

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

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

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

Basis of Payment

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

NSSP FOR SLOPE STABILITY ANALYSES REQUIRED FOR TEMPORARY EMBANKMENT WIDENING

Scope of Work

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

Basis of Payment

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

Special Provision

Amendment to OPSS 902, November 2010

902.02 REFERENCES

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

Ontario Provincial Standard Specifications, Construction

OPSS 805 Temporary Erosion and Sediment Control Measures

902.03 DEFINITIONS

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

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

Cofferdam means as defined in OPSS 539.

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

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

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

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

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

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

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

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

Unwatering means the removal of ponded or flowing surface water.

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

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

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

902.04 DESIGN AND SUBMISSION REQUIREMENTS

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

902.04.01 Design Requirements

902.04.01.01 Dewatering

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

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

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

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

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

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

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

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

902.04.02 Submission Requirements

902.04.02.01 Working Drawings

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

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

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

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

a) Plans, Elevations, and Details

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

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

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

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

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

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

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

902.04.02.02 Preconstruction Survey

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

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

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

902.04.02.03 Milestone Inspections

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

a) Dewatering of excavation for structure.

b) Completion of excavation for foundation.

c) Excavation for backfill and frost tapers.

d) Backfilling.

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

902.07 CONSTRUCTION

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

902.07.04 Dewatering Structure Excavation

902.07.04.01 General

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

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

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

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

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

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

Unwatering shall be carried out as necessary.

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

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

902.07.04.02 Discharge of Water

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

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

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

902.07.04.03 Monitoring

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

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

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

902.07.04.04 System Amendments

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

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

902.07.04.05 Removal

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

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

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

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

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

NOTES TO DESIGNER:

<u>Designer Fill-Ins</u>

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

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

N/A

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

CUSTODIAN: Tony Sangiuliano, MERO - Foundation Group.