



FINAL REPORT

FOUNDATION INVESTIGATION AND DESIGN REPORT

**Flood Creek West Timber Culvert Replacement, Highway 11, 48W-261/C, District
of Rainy River**

Agreement No. 6014-E-0017

Assignment No. 6

GWP 6319-14-00

Geocres No. 52B-025

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Foundation Investigation and Design Report

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Foundation Investigation Report for Flood Creek West Timber Culvert Replacement
Highway 11, Site No. 48W-261/C, District of Rainy River

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Part I: FOUNDATION INVESTIGATION REPORT

1.1 Introduction

This foundation investigation report presents the results of a geotechnical investigation completed by **exp** Services Inc. for the replacement of Flood Creek West Timber Culvert, located on Highway 11, about 650 m west of the junction of Hwy 11 and Hwy 623, in the District of Rainy River, the Ministry of Transportation (MTO) Northwestern Region. The work was undertaken under Agreement # 6014-E-0017, Assignment No. 6 (GWP 6319-14-00). The terms of reference (TOR) were as presented in the MTO letter dated May 27, 2015.

Based on preliminary information provided, it is understood the existing culvert is a twin cell timber structure with a width of about 4.2 m (2.1 m for each cell of the twin culvert), length of about 22 m and a height of about 1.5 m. It is also understood that the existing culvert construction date was unknown, and is intended to be replaced with a new culvert along the same alignment.

The purpose of the investigation was to evaluate the subsurface conditions along the alignment, to permit detailed design for the culvert replacement. The site specific geotechnical investigation consisted of borings, soil sampling, borehole logging, and field 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

As shown on Drawing 1 (Appendix B), the Flood Creek West Timber Culvert is located on Highway 11, about 650 m west of the junction of Hwy 11 and Hwy 623, in the District of Rainy River, south of Sapawe, Ontario. At the site, Hwy 11 is a two lane roadway, with a speed limit of 90 km/h and is about 9.6 m wide from edge of pavement to edge of pavement (includes asphalt paved shoulders, about 1.9 m and 0.5 m wide on the north and south sides, respectively).

Based on a true north direction, Highway 11 runs generally in a northwest and southeast direction, and Flood Creek generally flows from the southwest to the northeast. However, for simplicity and for the purposes of this report a "project north" has been established and project north is oriented to the centerline alignment of the Flood Creek West Culvert (i.e. project north is in the same direction as true north's northeast direction). The orientation of project north is presented on Drawing 1 in Appendix B. Hereinafter, the directions indicated in this report are referenced to project north.

Based on field measurements, the roadway embankment is about 4.0 m and 3.1 m high on the south and north sides, respectively, with mild side slopes of about 2.5H:1V on the south side and 2.8H:1V on the north side.

During the fieldwork on June 27 and 28, 2015, the general site conditions were assessed. Hwy 11 runs in an east and west direction, and Flood Creek flows from south to north beneath the highway, ultimately towards Niobe Lake which is about 50 m east of the site. At the time of this investigation, the approximate creek elevations at the inlet and outlet were about 431.67 m and 431.60 m, respectively. The elevation of highway pavement centerline at the culvert centerline is about 434.8 m. Overhead wires were observed along the north and south side of the highway with some wires crossing the highway west of the culvert.

At the vicinity of the inlet and outlet of the culvert some tall grass was noted at both culvert ends. The surrounding area of the culvert also contained tall grass with mature trees further away from the culvert. The inlet and outlet appeared to be generally clear of debris and excess vegetation, and as such the flow does not appear to be restricted.

Select photographs are provided in Appendix A.

1.2.2 Geological Setting

According to the MNR Northern Ontario Engineering Geology Terrain Data Base Map, Ontario Geological Survey Map 5073, Scale 1:100,000, dated 1979, the underlying native soil at the site lies along the border of two landforms. The landform to the west consists of clay glaciolacustrine plain deposit with low local relief and wet surface conditions. The landform to the east consists of bedrock ridge subordinate bedrock knobs with moderate local relief including knobby, hummocky, ridged and dry surface conditions.

According the Ministry of Northern Development and Mines (MNDM) Bedrock Geology of Ontario, West-Central Sheet Map No. 2542, Scale 1:1,000,000, dated 1991, the bedrock geology of the site is of the Neo to Mesoproterozoic Era (2.5 to 3.4 Ga). Two formations border the site. To the north is supracrustal rock, generally consisting of metasedimentary rock. The metasedimentary rock include wacke, arkose, argillite, slate, marble, chert, iron formation, and minor metavolcanic rock complexes. To the south, intrusive rock consisting of massive to foliated, granodiorite to granite rock.

1.3 Investigation Procedures

1.3.1 Site Investigation and Field Testing

The field investigation was performed on June 27 and 28, 2015. The field program consisted of advancing four (4) sampled boreholes (BH201 to BH204). Two (2) boreholes were located within the highway, BH201, and BH202. BH201 was located about 5 m east of the culvert centerline and about 3.7 m south of the highway centerline. BH202 was located about 5 m west of the culvert centerline and about 1.8 m north of the highway centerline. An additional two (2) boreholes (BH203 and BH204) were advanced off of the highway. BH203 was located about 5.6 m east of the culvert centerline and about 15 m north of the highway centerline (outlet/downstream side). BH204 was located about 6.3 m west of the culvert centerline and about 15 m south of the highway centerline (inlet/upstream side). The borehole locations are shown on Drawing 1 in Appendix B.

Three (3) boreholes (BH201, BH202 and BH204) were advanced using a CME 850 track mounted drill rig and due to access issues, one borehole (BH203) was advanced using a hand probe / hand shovel. The

drill rig was equipped with hollow stem continuous flight augers and standard soil sampling equipment (includes 51 mm outside diameter split spoon samplers and *in situ* shear vane testing equipment). In addition, the CME 850 drill rig was equipped with rock coring equipment (HQ size). The roadway boreholes BH201 and BH202 were advanced to depths of about 9.6 m and 7.0 m below ground surface, respectively. The off-road boreholes, BH203 and BH204, were advanced to refusal, at depths of about 0.5 m and 1.2 m below ground surface, respectively. The off-road boreholes were terminated at the refusal depth.

At BH201 at about 4.9 m below ground surface, auger refusal was encountered and rock coring techniques were conducted to advance the borehole to determine the nature of refusal (i.e. cobbles and boulders). Rock coring techniques at BH201 were continued through the cobbles and boulders and into the bedrock. Rock coring techniques were initiated at about 4.3 m below ground surface at BH202 to advance the borehole into the bedrock. Rock core samples were collected at both borehole locations. No rock coring techniques were conducted at the remaining borehole locations.

The borehole locations were referenced to the MTM ON-16 NAD83 coordinate system and their ground surface elevations were surveyed by **exp** personnel. The ground surface elevations, including top of water in the creek, were referenced to a geodetic elevation located at the highway centerline at the culvert centerline. The elevation at the centerline is 434.83 m. The centerline elevation was tied into a round iron bar [RIB] in rock east of the site and north of the highway. The RIB elevation is 434.65 m. The location of the RIB is shown on Drawing 1, in Appendix B.

During the advancement of the three (3) boreholes with the drill rig (BH201, BH202, BH204), 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), and were generally performed at intervals of about 0.75 m. The original field (uncorrected) SPT “N” values were recorded on the borehole logs as recommended in the Canadian Foundation Engineering Manual and used to provide an assessment of *in-situ* compactness (cohesionless) or consistency (cohesive) soils.

Upon completion of the boreholes, groundwater level measurements were carried out in boreholes in accordance with the Ministry of Transportation guidelines. The measured groundwater levels after completion of drilling boreholes were recorded on borehole log sheets in Appendix C. The boreholes were backfilled with a mixture of bentonite and auger cuttings. The borehole decommissioning was in general 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 a member of **exp**'s engineering staff 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. All of the recovered soil samples were placed in labelled moisture-proof bags which, along with the rock cores, were brought to **exp**'s Thunder Bay laboratory for additional visual, textual and olfactory examination, and for subsequent examination by a geotechnical engineer and laboratory testing.

1.3.2 Laboratory Testing

All samples brought to the laboratory were subjected to visual examination and classification. The laboratory testing program included the determination of natural moisture content and particle size

distribution for approximately 25% of the collected soil samples. All of the laboratory tests were carried out in accordance with MTO and/or ASTM Standards, as appropriate, at the **exp** laboratory in Thunder Bay, Ontario.

The laboratory test results are provided on the attached borehole log sheets in Appendix C as well as graphically in Appendix D.

In addition, chemical testing of two select soil samples were conducted. The soil samples were sent via courier, in a secure cooler under chain of custody, to Maxxam Analytics Inc., a CALA-certified and accredited laboratory in Mississauga, Ontario. Details of the chemical testing are discussed below and the lab results are included in Appendix E.

1.4 Subsurface Conditions

The detailed subsurface conditions encountered in the boreholes advanced during this investigation are presented on the Borehole Records in Appendix C. Laboratory test results are provided in Appendix D. The "Explanation of Terms Used on Borehole Records" preceding the borehole logs in Appendix C forms an integral part of and should be read in conjunction with this report. In addition, photographs of the bedrock core obtained are included in Appendix C.

A borehole location plan and stratigraphic sections are provided in Appendix B. It should be noted that the stratigraphic boundaries indicated on the borehole log and stratigraphic sections 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 interpreted as exact planes of geological change. Furthermore, subsurface conditions may vary between and beyond the borehole locations.

In general, the subsurface conditions along the proposed culvert alignment consist of a layer of sand fill, overlying native sand, overlying cobbles and boulders and sand till, and overlying bedrock. A more detailed summary of the subsurface conditions encountered in the boreholes is provided in the following sections.

1.4.1 Well Graded Sand with Silt and Gravel Fill

Well graded sand with silt and gravel fill was encountered beneath the asphalt at BH201 and BH202. The asphalt thickness at BH201 and BH202 was about 90 mm. The sand fill was generally described as compact to very dense, brown, damp to moist, and containing trace asphalt pieces in the upper 0.25 m. Cobbles and boulders were encountered in BH201 from about 0.8 m to 1.5 m below ground surface during augering. The SPT "N" values of the fill ranged between 10 and 66 blows per 300 mm penetration, with an average "N" value of about 34. The fill extended to depths of about 4.3 m (430.7 m elevation) and 3.1 m (431.8 m elevation) below ground surface at BH201 and BH202, respectively.

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

Moisture content:

- 1.8% to 9.8%

Grain size distribution:

- 43% gravel;
- 46% sand; and
- 11% silt and clay sizes.

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 tests are also provided on Figure 1, in Appendix D.

1.4.2 Peat / Rootmat

Peat and rootmat was encountered surfacing BH203 and BH204, respectively. The peat / rootmat was generally described as soft to very soft, dark brown and wet. The peat contained trace gravel, trace sand and trace roots and rootlets. The rootmat contained some peat. The thickness of the peat was about 0.2 m and the thickness of the rootmat was about 0.1 m. The peat extended to 0.2 m (431.5 m elevation) below ground surface and the rootmat extended to about 0.1 m (431.8 m elevation) below ground surface.

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

Moisture content:

- 59.0% to 81.2%

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

1.4.3 Sand

Native sand was encountered underling the fill, peat and rootmat. The native sand ranged in composition from silty sand with gravel to well graded sand with silt and gravel. The sand was generally described as loose to very dense, brown to grey, wet, and containing trace peat and /or trace organics. Occasional to some cobbles and boulders were encountered at BH201 and BH204. The SPT "N" values ranged between 16 and 100 (i.e. SPT refusal) blows per 300 mm penetration, with an average "N" value of about 37. The native sand extended to depths ranging between about 0.5 m and about 4.9 m below ground surface, and elevations ranging between 430.1 m and 431.2 m.

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

Moisture content:

- 8.7% to 43.5%

Grain size distribution:

- 25% to 45% gravel;
- 43% to 62% sand; and
- 5% to 41% silt and clay sizes.

Total saturated unit weights have been calculated based on the moisture contents and are estimated to range from about 17.5 to 23.3 kN/m³.

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 are also provided on Figures 1 and 2 in Appendix D.

1.4.4 Silty Sand Till

At BH204, silty sand till was encountered underlying the poorly graded sand with silt and gravel. The till was described as very dense, grey, moist to wet and containing occasional cobbles. One SPT was conducted in the till. The SPT "N" value was 100 blows (i.e. SPT refusal) per 300 mm penetration. The till extended to a depth of about 1.2 m (430.7 m elevation) below ground surface.

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

Moisture content:

- 17.6%

Grain size distribution:

- 12% gravel;
- 49% sand; and
- 39% silt and clay sizes.

Total saturated unit weight has been calculated based on the moisture content and is estimated to be about 21.1 kN/m³.

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 are also provided on Figure 2 in Appendix D.

1.4.5 Cobbles and Boulders

Cobbles and boulders were encountered underlying the silty sand with gravel at BH201. No SPT sampling was conducted. The cobbles and boulders extended to about 6.6 m below ground surface (elevation 428.4 m).

No laboratory testing was performed on the cobbles and boulders.

1.4.6 Bedrock

Bedrock was encountered beneath the cobbles and boulders at BH201 and beneath the sand at BH102, at depths of about 6.6 m (428.4 m elevation) and 4.3 m (430.5 m elevation) below ground surface, respectively. The bedrock was generally described as a medium strong (25 MPa to 50 MPa compressive strength), fractured to very sound, greenish dark grey, and fine grained. The boreholes were extended by

rock coring between about 2.7 m and 3.0 m into bedrock, and to depths ranging between about 7.0 m and 9.6 m below ground surface. The boreholes were terminated at elevations ranging between about 425.4 m and 427.8 m. Photographs of the bedrock core samples are presented in Appendix C, after the Borehole Logs.

Gross recoveries ranged between about 99% and 100%. The Rock Quality Designation (RQD), which is a modified core recovery, ranged from 59% to 100% (fractured to very sound).

No laboratory testing was performed on the bedrock.

1.5 Groundwater and Surface Water Conditions

Information on groundwater levels at the site was obtained by measuring the water levels in the open boreholes after completion of drilling and probe. The groundwater levels encountered in the boreholes are shown on the borehole logs and presented below in Table 1.1.

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.

Table 1.1. Groundwater data

Borehole	Date Completed	Date Measured	Ground Surface Elevation ²	Depth to Water ³	Groundwater Elevation
BH201	Jun. 28/15	Jun. 28/15	434.99	3.40	431.59
BH202	Jun. 28/15	Jun. 28/15	434.82	3.30	431.52
BH203	Jun. 28/15	Jun. 28/15	431.65	0.15	431.50
BH204	Jun. 27/15	Jun. 27/15	431.89	0.25	431.64
Flood Creek WL Upstream (South) Side	--	Jun. 27/15			431.67 ⁴
Flood Creek WL Downstream (North) Side	--	Jun. 27/15	--	--	431.60 ⁴
Notes: 1) All units in metres. 2) Elevations surveyed are referenced to a geodetic elevation located at the highway centerline at the culvert centerline. The elevation at the centerline is 434.83 m. 3) Depths are relative to ground surface. 4) Indicates top of surface water elevation at Flood Creek.					

1.6 Chemical Analyses

Two soil samples were selected for chemical analyses and were sent via courier, in a secure cooler under chain of custody, to Maxxam Analytics Inc., a CALA-certified and accredited laboratory in Mississauga, Ontario. The analytical laboratory results are presented in Appendix E, and are summarized in Table 1.2, below.

Table 1.2. Corrosivity Chemical Analysis

Borehole	pH (unitless)	Chloride (ppm)	Soluble Sulphate (ppm)	Resistivity (ohm-cm)	Conductivity (μ S/cm)
BH201-S7A	5.49	170	<20	3,300	301
BH203-S3	5.43	320	<20	1,800	557

Part II: ENGINEERING DISCUSSIONS AND RECOMMENDATIONS

2.1 General

This section of the report provides geotechnical design recommendations for replacement of the existing Flood Creek West Timber Culvert, located on Highway 11, about 650 m west of the junction of Hwy 11 and Hwy 623, in the District of Rainy River, 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 and presented in **Part I-Foundation Investigation Report**. The interpretation and recommendations provided are intended solely to permit designers to assess foundation alternatives and design the new structure 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.

Based on information included in the TOR, it is understood that the existing twin cell timber culvert, each with a 2.1 m span, height of 1.5 m and length of about 22 m, will be replaced with a new culvert. It is understood that the existing culvert construction date was unknown, and inspection in 2013 indicated that some areas of timber walls were severely decayed, water dripping though soffit, timbers were slightly soft at water level, top timbers broken and rotating at inlet and outlet. The same inspection also noted medium erosion on north side embankment and along culvert walls on outlet side. It is also understood that the new culvert is proposed to be at the current alignment as well as that the road grade will be the same as that at the location of the existing culvert. The size and type of the new culvert is not defined at the time of writing this report. However for preliminary design purposes, the following options are being considered for the replacement: rigid frame box culvert (precast or cast-in place), rigid frame open footing culvert (precast or cast-in-place), or steel sheet pile abutments and precast concrete decking.

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-S6-14)*, 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 Terms of Reference from the MTO letter dated May 27, 2015. The assessment involved review of options for replacement of the existing culvert along the same alignment with a final selection to be made by the designer, based on the optimum solution.

2.2 Expected Ground Conditions

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

- a. Highway 11 at the site location is a two lane roadway, with a speed limit of 90 km/h and is about 9.6 m wide from edge of pavement to edge of pavement (includes asphalt paved shoulders, about

- 1.9 m and 0.5 m wide on the north and south sides, respectively). Based on field measurements, the highway embankment is about 4.0 m and 3.1 m high on the south and north sides, respectively, with mild side slopes of about 2.5H:1V on the south side and 2.8H:1V on the north side. The current elevation of the crest of the roadway is about 434.8 m.
- b. The highway embankment consists of well graded sand with silt and gravel fill (~3.1 m to 4.3 m thick on north and south respectively).
 - c. The embankment fill is underlain by native sand which ranged in composition from silty sand with gravel to well graded sand with silt and gravel to about 3.1 m to 4.3 m (~ 0.6 m to 1.2 m thick) below ground surface, followed by cobbles and boulders (~1.7 m thick) and bedrock on south side and followed by bedrock on north side. The bedrock was encountered approximately 4.3 m to 6.6 m below the ground surface (430.5 m to 428.4 m elevation) on south and north side respectively.
 - d. The foundation soil at the invert of the new culvert is anticipated to be sand with silt and gravel to sand fill located at elevations between 431.5 m and 431.3 m. Typical 'N' values ranged from 18 to 66.
 - e. The groundwater table in the embankment fill is expected to be at approximate elevation 431.6 m, or slightly higher. 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. Some groundwater mounding within the embankment and perched water would be anticipated.

2.3 Culvert Foundations

For preliminary design purpose, several options are being considered for the replacement:

- Rigid frame box culvert,
- Rigid frame open footing culvert,

Based on the subsurface information obtained from the boreholes, the native sand with silt and gravel is considered suitable for support of rigid frame box culvert and rigid frame open footing culvert option. However, the choice of culvert type will also depend on parameters such as the initial cost, maintenance costs, hydraulic performance, ease of construction, salvageability and local availability of material and equipment. Since the cobbles and boulders were encountered below the invert level and bedrock was encountered at relatively shallow depth, steel sheet pile abutment with precast concrete decking may not be feasible options.

To prepare the culvert foundation subgrades, all exposed surfaces should be proof rolled to determine if any isolated loose and yielding areas are present. If excessively yielding areas are observed and cannot be stabilized in place by compaction, the affected soils should be excavated and removed to firm bearing and the grade restored with engineered fill. If the depth of excavation to remove unstable soils is significant, using a geotextile fabric, such as Terrafix 270R or equivalent, in conjunction with engineered fill can be considered to assist in providing a stable base. Based on previous experience, typically a minimum of 450 mm of a clear stone over geotextile fabric would establish a stable bearing surface. The fabric should be installed a manner to mitigate the migration of fines from adjacent material.

Table 2.1 Evaluation of foundation alternatives

Options	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Rigid frame box culvert	1	<ul style="list-style-type: none"> ▪ Straightforward construction ▪ Reduce construction period, consequently traffic management and water control period ▪ Reduce excavation depth 	<ul style="list-style-type: none"> ▪ Dewatering system required ▪ Require heavy lifting equipment 	<ul style="list-style-type: none"> ▪ Low to medium 	<ul style="list-style-type: none"> ▪ Risk of unacceptable differential settlements if the entire foundation is not supported on the competent soil ▪ Risk of leaking from joints if not properly installed
Rigid frame open footing concrete culvert	2	<ul style="list-style-type: none"> ▪ Wider span may consider to maintain existing channel ▪ High geotechnical resistance available ▪ Can incorporate dowels to enhance lateral resistance 	<ul style="list-style-type: none"> ▪ Deeper excavation or below water excavation may required ▪ Dewatering system required ▪ Possible uneven bedrock surface ▪ Require placement of lean concrete 	<ul style="list-style-type: none"> ▪ Likely more expensive than Option 1 	<ul style="list-style-type: none"> ▪ Risk of unacceptable differential settlements if the entire foundation is not supported on the competent soil ▪ Risk of delay in construction due to deeper excavation below water if proper dewatering is not maintain ▪ Higher scour risk

Table 2.1 compares the structure options from a foundations design and constructability perspective. Although the foundation soils are generally good and will provide adequate support for both options listed in the table, the use of rigid frame pre-cast box culvert is recommended.

2.3.1 Shallow Foundations

2.3.1.1 Geotechnical Resistance

Based on the subsurface stratigraphy encountered at this site and the assumed invert elevation of the new culvert, the recommended founding depths and geotechnical resistances for a structure founded on undisturbed competent natural soils are tabulated below.

Table 2.2 Recommended spread footing design parameters

Culvert Type	Founding Elevation (m)	Assumed Footing Size (m)	Founding Soil Type	Factored Geotechnical Resistance at ULS (kPa)	Geotechnical Reaction at SLS** (kPa)
Rigid frame box culvert	~431.3 or below	4.0	compact to dense sand with silt and gravel to very dense gravelly fill	400	250
Rigid frame open footing concrete culvert	~429.0* or below	1.0	Cobbles and boulders to bedrock	600	400

Notes:

*Elevation based on BH201 of the highway embankment. Where the footing founded on bedrock requirement of full frost protection is not applicable. It is recommended to place minimum 100 mm thick layer of compacted granular A beneath footings.

** for maximum settlement of 25 mm

It is presumed that any soft or very loose materials are to be replaced with clean and compactable soil such as Granular B Type II. Given that no significant grade raise is planned, the anticipated maximum total settlements for the new proposed culvert are not expected to exceed 25 mm for construction done in accordance with these design parameters and assuming good construction practice including sound base preparation.

Due to the variable nature of the bedrock surface at the new culvert alignment, it is possible that in some locations, continuous strip footing may transition from being founded partially on bedrock and partially on compacted granular material. The large difference in stiffness between these two foundation strata could result in undesirable differential settlements between foundation units and potentially cracking at the transition point of strip footings. A design detail will be required at these transition points to accommodate potential small differential settlements and ensure the performance of the footing and foundation wall. Use of mass concrete to raise site grades to the proposed founding level instead of the placement of compacted fill materials can minimize the risk of differential settlements. Non Standard Special Provision (NSSPs) should be included in contract documents to address some of the foundation/geotechnical issues that might be of concern during execution of the work (see draft NSSPs in Appendix I).

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

2.3.1.2 Resistance to Lateral Loads

Resistance to lateral forces/ sliding should be calculated in accordance with Section 6.9.1 of the CHBDC, using the following parameters:

Table 2.3 Recommended parameters for calculation of unfactored horizontal resistance

Interface and loading conditions	Parameters
Between Granular A pad and pre-cast concrete	Coefficient of friction ($\tan \delta$)=0.5
Between cast-in-place concrete and sand with silt subgrade	Coefficient of friction ($\tan \delta$)=0.4

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

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

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

2.3.1.3 Frost Protection

The frost depth in the area of the culvert is estimated to be approximately 2.3 m in accordance with OPSD 3090.100. During construction of any temporary and permanent support system using shallow foundations should be provided a minimum 2.3 m of soil cover or equivalent frost protection should be provided using thermal insulation.

2.4 Construction Alternatives

For the proposed culvert replacement the following methods were considered as possible alternatives for the new culvert installation at this site:

1. Full road closure followed by open cut/unsupported excavation to replace culvert
2. Construct temporary detour embankments at the site followed by open cut/unsupported excavation to expose and replace culvert
3. 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

All methods considered utilize a cut and cover approach for culvert replacement which allows complete removal of the existing culvert, but it requires disruption of traffic. In contrast, a trenchless approach for culvert replacement does not require disruption of traffic. However, considering the size and nature of the existing culvert and topography of the surrounding terrain, tunneling for trenchless replacement of this culvert was not considered as an applicable option. The other trenchless methods such as pipe bursting, pipe splitting, pipe swallowing and interior replacement methods were also not considered as applicable in this project, since the size of the host pipe classify this culvert as an unsuitable candidate for these techniques. For all approaches provision must be made to maintain surface water flow to the outlet.

The following Table 2.4 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 2.4 Construction alternatives for culvert replacement (see schematic sketches in Appendix H)

Installation Method	Advantages	Disadvantages	Relative Cost	Ranking
OPTION 1 Full Road Closure using Existing Roadways and Open Cut Unsupported Excavation	<ul style="list-style-type: none"> • Existing timber culvert will completely remove and replaced with new culvert • No construction of detour roads or roadway protection required • No excavation support required • Install entire new culvert at once • Straightforward construction • Short mobilization time • Low capital investment; cost saving in time and materials required for construction 	<ul style="list-style-type: none"> • Traffic interruption • Long detour around site using other existing roads required • Large amount of soil to be excavated • Existing fills and native soils require 2H:1V side slopes to maintain stability • Erosion control of temporary cuts required • Need to temporarily control existing creek water • Potential claims to compensate vehicle occupants and local business for delays or time lost due to detour routes • Risk of cost overrun and inability to finish job: low 	Relatively less expensive than other methods due to cost savings in time and materials required for construction, but potential claims to compensate vehicle occupants and local business for delays or time lost due to detour routes	3
OPTION 2 Temporary Local Detour and Open Cut Unsupported Excavation	<ul style="list-style-type: none"> • Traffic flow maintained at the site during construction • Simple detour roads can be constructed • Existing timber culvert will completely remove and replaced with new culvert • No excavation support required • Install entire new culvert at once 	<ul style="list-style-type: none"> • Traffic interruption • Construction of detour embankments required at north or south side of highway • Difficulties to construct detours due to inaccessible surrounding terrain • Increased time for construction of detour • Large amount of soil to be excavated • Erosion control of temporary cuts required • Need to temporarily control existing creek water • Possible settlement due to new earth embankment fill • Risk of cost overrun and inability to finish job: low to moderate 	More expensive than full road closure due to high costs to build local detours	4

Installation Method	Advantages	Disadvantages	Relative Cost	Ranking
		<ul style="list-style-type: none"> • Possible extra cost to purchase of private property 		
OPTION 3.A Half-and-half Construction with Unsupported Cut Sides	<ul style="list-style-type: none"> • Traffic flow maintained at the site during construction • Short mobilization time • Straight forward construction and construction procedures 	<ul style="list-style-type: none"> • Traffic interruption • Roadway protection of up to 4.0 m high required to maintain one lane of traffic • 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 instability to finish job: low to moderate 	Relatively more expensive than full road closure due to high costs of roadway protection system	1
OPTION 3.B Half-and- half Construction with Braced or Anchored Cut Sides	<ul style="list-style-type: none"> • 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 	<ul style="list-style-type: none"> • Traffic interruption • Roadway protection of up to 4.0 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 • Risk of cost overrun and instability to finish job: low to moderate 	More expensive than full road closure and other open cut sides approach due to high costs for shoring system and temporary decking (if feasible) to maintain continuous flow of traffic	2

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.A: Half-and-half construction with unsupported cut sides (Figure H3.A, Appendix H)
2. OPTION 3.B: Half-and-half construction with braced or anchored cut sides (Figure H3.B, Appendix H)
3. OPTION 1: Full road closure using existing roadways and open cut unsupported excavation (Figure H1, Appendix H)
4. OPTION 2: Temporary local detour and open cut unsupported excavation (Figure H2, Appendix H)

The following sections discuss these options in more details.

2.4.1 Half-and-Half Construction (Options 3)

If a long detour using existing roadways is not available and acceptable, the half-and-half construction method should be utilized (see Figures H.3.A and H.3.B, Appendix H). In that 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 that 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 required to remove half of the existing embankment would be up to 4.0 m deep. Therefore, temporary shoring such as a soldier pile and lagging system will be required as a roadway protection system to allow staging excavation/construction. It will be the Contractors responsibility to design a suitable temporary support system for the MTO review prior to installation. The Contractor is to follow OPSS 902, regarding excavations for structures, and OPSS.PROV 539, regarding temporary protection systems. Recommendations for a temporary roadway protection are given in Section 2.5. Using the half-and-half construction approach, several methods of culvert replacement were considered 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

Option 3.A could be more economical due to possible cost savings for reversible wall configuration, but it will be more disruptive to the highway embankment. Option 3.B will disrupt less of the embankment but would cost more, i.e. about 1.8 times of Option 3.A. Excavation and backfilling operations will also be more challenging with Option 3.B. Both options require decommissioning of shoring system upon completion of the work.

2.4.1.1 Option 3.A: 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 H3.A, Appendix H). The roadway protection can take the form of reversible shoring such as a soldier pile and lagging with rakers or anchors for horizontal support. Where the cut extends below prevailing groundwater a suitable control/system is required. 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 contractor.

Option 3.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 Options 3.B since it needs to excavate a large amount of soil.

2.4.1.2 Option 3.B: 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 H3.B., Appendix H). Excavation in this case would have to accommodate the necessary cross-bracing such as struts. With this option, consideration would have been given to how the new culvert sections will be installed given the relatively narrow work area and potential for obstructions from the lateral bracing using struts. Installation of tiebacks could be the solution. Temporary decking could possibly be used over the supported cut to allow for excavation of both halves prior to diverting stream and backfilling. However decking would be costly. As well as Option 3.A, decommissioning of the shoring system and temporary surface water flow control must be performed/developed by contractor.

Option 3.B will disrupt less of the embankment than Option 3.A but would cost more, i.e. about 1.8 times of Option 3.A, due to the cost of shoring system. Excavation and backfilling operations will also be more challenging with Option 3.B. Both options require decommissioning of shoring system upon completion of the work.

2.4.2 Detour Options (Options 1 and 2)

Both detour options, the option with full closure of Hwy 11 and long detours around the area using existing roadways (see Figure H1, Appendix H), and the option with the local detour embankment construction at the site to maintain the local flow of traffic during the replacement (see Figure H2, Appendix H), allow for open cut, unsupported excavation to facilitate the replacement of the existing culvert. A major benefit of these options is that the existing culvert will be completely removed and replaced new culvert. The other advantages are that neither excavation support nor roadway protection is required with these options. The major disadvantages of both options are traffic interruption, large amounts of excavated soils and need for temporary construction unwatering and dewatering systems (i.e. cofferdams, and sumps and pumps, etc.) to prevent existing creek water and groundwater flow into the construction area which is the responsibility of 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). All fills (i.e. sand with silt and gravel fill) may be classified as a Type 3 soil above the groundwater table in conformance with the OHSA. The native soils below the groundwater table may be classified as a Type 4 soil. It is expected that most of excavations will be above the groundwater levels except those at the invert level. To avoid disturbance of the founding subgrade and to allow placement of backfill in dry conditions, groundwater must be controlled to below the proposed invert excavation levels prior to digging to final levels. As mentioned before, the ingress of surface water must be controlled using a suitable system as well.

Temporary excavation side slopes for Type 3 soil should not exceed 1H:1V in accordance with OHSA, and 2H:1V is recommended for global stability of these deep cuts (i.e. to maintain a global factor of safety greater than 1.3) where excavation will be left open for some time. Temporary excavation side slopes for Type 4 soils should not exceed 3H:1V where applicable. There is a potential for sloughing to occur if the trench remains open for an extended period of time (i.e. > 24 hours) or during a rainfall event. In addition, some localized surficial sloughing may be experienced in areas of perched groundwater seepage (i.e. within the embankment fill).

The detour construction alternative would involve construction of a temporary on-site embankment at the one side of the existing embankment depending on the available space and suitable terrain. Compacted engineered fill for construction of the temporary detour road is recommended. Prior to construction of the temporary detour embankment, the site 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 temporary detour road embankment. 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).

2.5 Temporary Roadway Protection

Temporary roadway protection is anticipated to be a part of the half-and-half construction approach that will be required to maintain on-site traffic during the construction. It is recommended that roadway protection system be in accordance with OPSS.PROV 539. The lateral movement of the temporary shoring system should meet Performance Level 2 as specified in 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.

Due to the presence of bedrock at shallow depth, a simple cantilevered approach would not be possible at this site. Temporary shoring such as a soldier pile and lagging system with additional anchorage or tiebacks may be required for lateral resistance. Conventional practice is to incorporate either buried deadman anchors or grouted anchors.

Deadman anchors can be designed based on the earth pressure coefficients and soil parameters provided in Section 2.5.1, following. 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 designed in a preliminary fashion in accordance with Section 26 of the CFEM (2006). Based on the generally compact soils at this site, the estimated factored (0.4) ULS resistance of grouted anchors would be 35 kN/m length. Alternatively, for this site, rock anchors may be used to provide lateral stability. If considered rock anchors, pressure grouted rock anchors can be designed in a preliminary fashion in accordance with Table 26.7 of the CFEM (2006). The estimated factored (0.4) ULS resistance of grouted anchors would be 144 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.

2.5.1 Lateral Earth Pressure

Culvert walls at the outlet and inlet, and temporary shoring that may be required for excavation should be designed to resist lateral earth pressure. The expression for calculating lateral earth pressure is given by:

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

where P = earth pressure intensity at depth h , kPa

K = earth pressure coefficient

γ = unit weight of retained soil, kN/m³

q = surcharge near wall, kPa

h = depth to point of interest, m

H = total depth of excavation, m

The above expression does not take into account hydrostatic pressure, which must be included for the groundwater levels measured on the site. Table 2.5 lists earth pressure parameters for given materials.

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.

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)

Table 2.5 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_0)	Unit Weight γ kN/m ³
Sand with Silt and Gravel fill	35°	0.27	3.67	0.43	20
Silty Sand with Gravel	34°	0.28	3.55	0.44	20
Cobbles and boulders	40°	0.22	4.56	0.36	21

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.

2.6 Culvert Bedding

OPSDs 802.010, 802.031, 802.032, 803.010 and Figure C6.20 of (CHBDC) or OPSD 3101.150 which are included in Appendix G provide the bedding, embedment, cover and backfill standards for the different pipe material. According to these standards the culvert bedding should consist of Granular A or Granular B Type II (OPSS.PROV. 1010) with thickness of 300 mm beneath the culvert 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 to at least 95% of the Standard Proctor Maximum Dry Density (SPMDD) before a subsequent layer is placed in accordance with OPSS 514. Bedding material placed in the haunches must be compacted prior to continued placement of cover material. Bedding on each side of the pipe shall be completed simultaneously. At no time shall the levels on each side differ more than the 200 mm uncompacted layers.

Prior to placing any fill material, the exposed native subgrade should be inspected according to OPSS 902. A non-woven geotextile separator is to be placed between the approved subgrade and the

compacted fill to assist in material placement and maintain the integrity of the founding soil along the entire length of the culvert. The geotextile separator is to be a Class II non-woven material with an equivalent opening size of 75-150 μm .

For the site area, a frost penetration depth of approximately 2.3m can occur in open, unheated areas without snow cover. At the culvert inlet and outlet, and beneath the proposed culvert, mostly the native soils consist of sand with silt to silt. This material has high frost susceptibility based upon the MTO Frost Classification guideline of percent particles between 5 to 75 m. Therefore, non-frost susceptible materials such as sand and gravel need to be provided to the limit of frost penetration beneath the inlet and outlet of the culvert. However, considering that cold air blowing through the culvert during the winter season will freeze soil next to the culvert, a minimum 500 mm thick layer of non-susceptible material should be considered to be placed as a bedding along the entire culvert length.

2.7 Culvert Backfill

Backfill should be placed from the base of the culvert to the full height of the culvert and extend a minimum 1.3 m horizontal distance from the outside wall (**as per Figure C6.20a of the CHBDC**). This horizontal distance may be reduced by the use of suitable insulation (such as a heavy duty STYROFOAM). The insulation should be placed against the outside wall of the culvert from the base of the culvert to its total height. The material should be installed as per the manufacturer's instructions.

The backfill should consist of free-draining, non-frost susceptible granular materials such as Granular A or Granular B (OPSS.PROV 1010).

All granular backfill materials should be placed in thin lifts (i.e. not exceeding 300 mm before compaction) and each lift should be compacted to at least 95% of the material's SPMDD (Standard Proctor Maximum Dry Density). The final lift of embankment fill 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.

For fills immediately below any roadway, it is recommended that Granular A or B aggregates be used. Where necessary, proper tapering as per standards should be provided. Below a depth of about 1.5 m from any finished road grade, approved compactable fill, such as select subgrade materials (SSM) can be used.

Where less than 1.3 m of earth cover is provided above the top of the culvert, a frost taper should be included as per OPSD 3101.150.

Backfilling behind any retaining (wing) walls should consist of granular materials in accordance with the MTO standards. Free draining backfill materials and perforated drains (as per Figure C6.20a of the CHBDC), suitably outleted etc. should be provided in order to prevent hydrostatic pressure build-up.

2.8 Groundwater and Surface Water Control

Cofferdams will be required at both upstream and downstream ends to envelop the construction site and keep it free of water during culvert installation. The investigation revealed that the subsurface conditions at the locations of cofferdams consist of a layer of peat/ rootmat (~0.2 m thick) underlain by loose to compact sand with silt and gravel and very dense silty sand till. The bedrock/refusal surface is encountered at 1.2 m depth at the inlet side and 0.5 m depth at the outlet side. Therefore, based on these geotechnical conditions (i.e. shallow bedrock), a rockfill cofferdam can be recommended to be used as a cofferdam at the site. 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. This cofferdam should be at least one meter above 100 year flood. 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. Cofferdams should be designed in accordance with OPSS 539 by a licensed Professional Engineer experienced in shoring design.

The soils encountered below the groundwater table and within potential excavation depths consist of native silty sand to sand with silt and cobbles and boulders. The materials are susceptible to disturbance from groundwater and mobilized equipment. The groundwater level needs to be controlled to at least 0.5 m below the excavation level to avoid disturbance, and any surface or groundwater seepage should be removed from the excavation prior to the culvert bedding material placement of granular backfill in the dry. In general, pumping using properly filtered sumps, and/or filtered drains placed along the base of the excavation should provide sufficient groundwater control during foundation works.

Provided that the existing culvert is to remain in use during construction of the new culvert, the majority of the upstream flow of the existing culvert can be diverted around the construction area. For the control of the water flow in the creek might require a cofferdam. If the existing culvert is to be removed prior to completion of the new culvert, a system of sumps and pumps will be required to divert the surface water up and over the existing embankment.

Dewatering requirements behind the cofferdams to keep the construction site dry will be impacted by water levels in the river at the time of construction activities. Dewatering shall be carried out in accordance with OPSS 517 and OPSS 518. It is responsibility of the Contractor to propose a suitable dewatering system based on the time of construction, water levels and river flow conditions for prior approval of the MTO. The method used should not undermine the existing road 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.

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

2.9 Embankment Design

2.9.1 Embankment Settlement

It is not planned to change the existing embankment grade at the culvert location. Therefore, there should be negligible additional settlements under the existing embankment because the soil under the existing embankment is non cohesive. However, a settlement of about 25 mm should be allowed for due to rebound during the construction.

2.9.2 Embankment Stability

A preliminary slope stability analysis was performed to assess the global stability of the existing embankment and to check that a minimum Factor of Safety of 1.3 will be achieved for the new embankment at the location of the proposed culvert. The static slope stability analyses were performed using the Morgenstern-Price method developed on the basis of limit equilibrium. The SLOPE/W computer program developed by GeoSlope International was employed for computation.

Stability assessments of existing slopes under static conditions were performed on the cross-section perpendicular to the highway at the proposed culvert location. The cross-section of the existing embankment with the approximate slope of 2H:1V to 1.5H:1V was established based on **exp's** survey data and the topographic plan provided by MTO. The stratigraphy and groundwater condition at the site were developed based on the results of the geotechnical investigation presented in Part I - Foundation Investigation Report.

Based on the borehole information, the subsoils encountered at the work area consist of embankment fill, underlain by native sand deposits. Therefore, an effective stress analysis for a long term stability assessment of the embankment slope was performed taking into consideration the subsoil conditions encountered beneath the existing embankment.

The SLOPE/W graphical printout, for analysis performed is included in Appendix F. Since the geometry and soil stratigraphy at the east and west side slopes are similar, the result of the slope analysis performed for the east side slope, is only presented.

Tabulated below in Table 2.5 are the soil parameters used for the slope stability analysis. The soil parameters were generally estimated based on the results of field and laboratory investigation.

Table 2.5 Soil properties used in slope stability analysis

	ϕ' (degrees)	c' (kPa)	γ' (kN/m ³)
Sand with Silt and Gravel Fill	35	0	20
Silty Sand with Gravel (Compact to Very Dense)	34	0	20
Cobbles and Boulders	40	0	21

The result of slope stability analysis for the 2.5H:1V south side slope of the existing embankment using drained (long term stability) soil parameters is presented graphically in Appendix F. As can be seen the calculated minimum factors of safety of critical slip surfaces meet the design criteria for static condition, indicating that the embankment constructed with 2.5H:1V is stable. However new embankment constructed with properly compacted granular material having side slopes 2H:1V and/or embankment constructed with rockfills having side slopes 1.25H:1V should be stable.

2.10 Inlet and Outlet

2.10.1 Erosion Protection at Outlet

Detailed design for Erosion protection should be carried out by the hydraulic engineer. However in general, Rip-rap protection should be provided where the culvert discharges into the open creek. 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 the OPSD 810.010, which is included in Appendix G of this report. Rip-rap placed at 1V:1H will be stable.

Where the embankment side slopes have been scarred and/or excavated (beyond rip-rap limit) to facilitate the existing culvert replacement, the scarred and/or reinstated embankment side slopes are to be vegetated with sodding, seeding or planting as necessary depending on the flow rate and volume. Should seeding be utilized, a 100 mm thick layer of topsoil should be placed along with a degradable erosion blanket to help minimize erosion until the vegetation begins to grow.

2.10.2 Stream Bed Rip-Rap

The stream bed rip-rap thickness is to be at least twice the median particle size, and/or 300 mm thick as a minimum as outlined by OPSD 810.010 included in Appendix G of this report.

2.10.3 Seepage Cut-off Requirements

The seepage cut-off requirements should be reviewed in the following context. The native sand with silt soil at the inlet, outlet side and below the culvert bedding has a high potential for migration with

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

2.10.3.1 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 (OPSD 802.095, Appendix F). 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 1×10^{-6} cm/s.

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

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

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

2.10.3.2 Cut-Off Trench

A cut-off trench 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.

2.11 Corrosion Protection

Two soil samples were selected for chemical analyses and were sent via courier, in a secure cooler under chain of custody, to Maxxam Analytics Inc., a CALA-certified and accredited laboratory in Mississauga, Ontario. The analytical laboratory results are presented in Appendix E, and are summarized in Table 1.2.

Similar to our experience with the soils in the Thunderbay area, the data in Table 1.2 indicates low to medium resistivity. Accordingly, buried metallic pipes and appurtenances would be susceptible to corrosion, unless protected; therefore, cathodic protection should be provided. The maximum chloride content reported is 320 ppm ($\mu\text{g/g}$) i.e. 0.032% which indicates a moderate potential for corrosion. In addition, the pH levels of the soils were 5.49 and 5.43 and indicate additional potential for corrosion.

The water soluble sulphate content of the soils tested is tested $<20\text{ppm}$ ($\mu\text{g/g}$), i.e. $<0.002\%$ and being less than 0.10%, does not require sulphate resistant cement. Normal Type 10 Portland cement can be used. These data also support our local experience.

2.12 Operational Constraints (OCs) and Non Standard Special Provisions (NSSPs)

In assembling contract documents, a number of OCs and NSSPs should be included to address some of the foundation/geotechnical issues that might be of concern during execution of the work. It is anticipated that the following list may apply based on current information:

- (1) NSSP for mass concrete on bedrock.
- (2) NSSP for sloping rock and cobble and rock piece obstructions.
- (3) NSSP for dowelling.
- (4) NSSP for condition surveys and monitoring during any blasting.

Appendix I presents draft of the suggested NSSPs.

These should be further assessed during planning and design development when actual approaches are more defined.

Part III: 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.

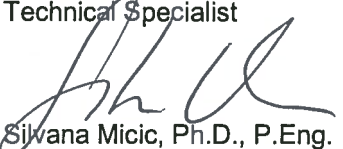
Contractors bidding on or undertaking any proposed work at this site should, relative to the subsurface conditions, decide on their own investigations, if deemed necessary, as well as their own interpretations of the factual results provided herein, so they may draw their own conclusions as to how the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

This Foundation Investigation and Design Report has been prepared by Ahileas Mitsopoulos, P.Eng., Nimesh Tamrakar, M.Eng, EIT., Demetri N. Georgiou, M.A.Sc. P.Eng., and Silvana Micic, Ph.D., P.Eng. It was reviewed by TaeChul Kim, P.Eng. and by Stan E. Gonsalves, M.Eng., P.Eng., Designated MTO Foundation Contact. The field investigation was supervised by Elwin Farkas.

Yours truly,

exp Services Inc.

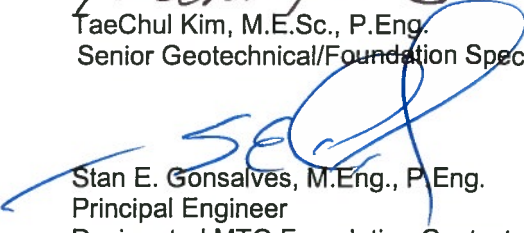

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Part IV: LIMITATIONS AND USE OF REPORT

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



Photo 1. Existing culvert inlet on south side of highway



Photo 2. Existing culvert outlet on north side of highway



Photo 3. Facing east on Highway 11 before the existing culvert



Photo 4. Facing west on Highway 11 before the existing culvert



Photo 5. Embankment slope on north side facing east



Photo 6. Embankment slope on south side facing west

Appendix B – Drawings



PROJECT NORTH



TRUE NORTH

Agreement No. 6014-E-0017
Assignment No. 6
GWP 6319-14-00

FLOOD CREEK WEST CULVERT
(Hwy 11, Rainy River District, Sapawee, ON)

DWG
1

PLAN

exp.

exp Services Inc.

KEY PLAN



LEGEND

- BH201 434.99 BOREHOLE LOCATION
GROUND SURFACE ELEVATION IN
METRES
- R.I.B.
434.65 ROUND IRON BAR LOCATION
GEODETIC ELEVATION IN METRES

BH No.	APPROX. ELEV. (m)	MTM COORDINATES	
		NORTH	EAST
BH201	434.99	5,400,586	427,349
BH202	434.82	5,400,594	427,346
BH203	431.65	5,400,591	427,361
BH204	431.89	5,400,588	427,330

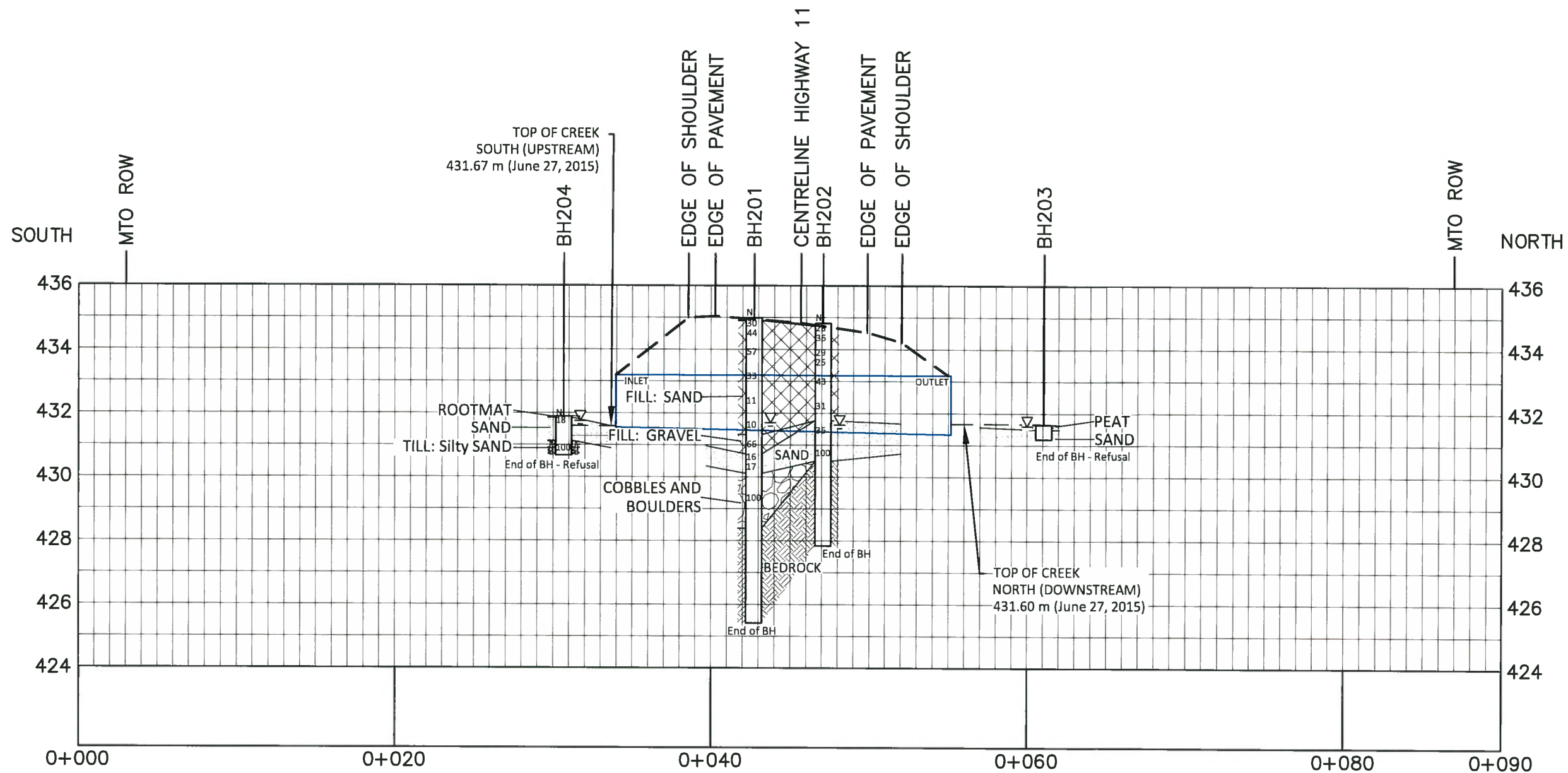
NOTES

- ALL DIMENSIONS ARE IN METRES.
- BASE MAP PROVIDED BY CLIENT.
- LOCATIONS OF FEATURES AND OBJECTS ARE BASED ON BOTH FIELD MEASUREMENTS AND CLIENT'S BASE MAP. WHERE CONFLICTS OCCUR FIELD MEASUREMENTS TAKE PRECEDENCE. BH LOCATIONS ARE BASED ON HAND HELD GPS UNIT COORDINATES
- COORDINATES PROJECTED TO MTM ON-16.
- THIS DRAWING IS FOR SUBSURFACE INFORMATION ONLY. THE PROPOSED STRUCTURE DETAILS/WORKS ARE SHOWN FOR ILLUSTRATION PURPOSES ONLY.
- CENTRELINE ELEVATION (434.83m) USED TO TIE IN ELEVATIONS OF R.I.B. AND OTHER SURVEYED POINTS.

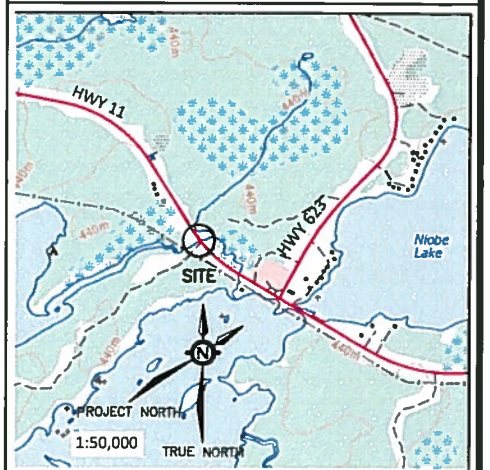
REVISIONS

DATE	BY	DESCRIPTION
GEOCRES No. 52B-025		Project No. ADM-00223648-E0
Date: December 8, 2015		Scale : 1:500
Drawn By: RM		Checked By: AM
		Checked By: DG





A - A'
PROFILE OF FLOOD CREEK WEST CULVERT



- N STANDARD PENETRATION TEST (BLOWS/0.3 m)
- MEASURED WATER LEVEL

BH No.	APPROX. ELEV. (m)	MTM COORDINATES	
		NORTH	EAST
BH201	434.99	5,400,586	427,349
BH202	434.82	5,400,594	427,346
BH203	431.65	5,400,591	427,361
BH204	431.89	5,400,588	427,330

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DATE	BY	DESCRIPTION

Appendix C – Borehole Logs and Bedrock Core Photos

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		
<div><div>0.002</div><div>0.006</div><div>0.02</div><div>0.06</div><div>0.2</div><div>0.6</div><div>2.0</div><div>6.0</div><div>20</div><div>60</div><div>200</div></div>											
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 Note 16 in ASTM D2488-09a:

Table a: Percent or Proportion of Soil, Pp

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

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

Table b: Apparent Density of Cohesionless Soil

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

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

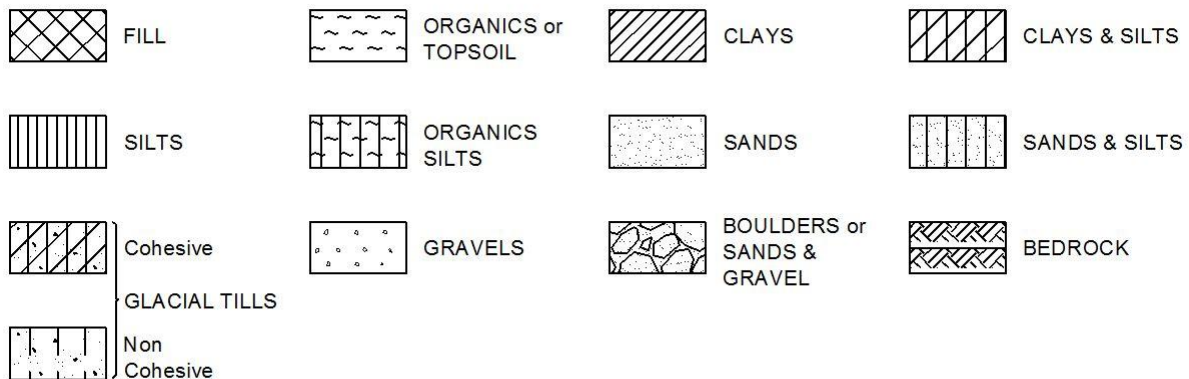
Table c: Consistency of Cohesive Soil

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

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

STRATA PLOT

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



WATER LEVEL MEASUREMENT



Open Borehole or Test Pit



Monitoring Well, Piezometer or Standpipe

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

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

STRESS AND STRAIN

u_w	kPa	Pore water pressure
r_u	1	Pore pressure ratio
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

MECHANICAL PROPERTIES 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'	$-\circ$	Effective angle of internal friction
c_u	kPa	Apparent cohesion intercept
ϕ_u	$-\circ$	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

RECORD OF BOREHOLE No BH201

1 OF 1

METRIC

W.P. GWP No. 6319-14-00 LOCATION Flood Creek West (Site No. 48W-261C) MTM ON-16 5,400,586N 427,349E ORIGINATED BY EF
 DIST 61 HWY Hwy 11 BOREHOLE TYPE CME 850 Track Carrier / HSA / HQ COMPILED BY RM
 DATUM Geodetic DATE 6.28.15 - 6.28.15 CHECKED BY DG/AM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				
435.0	Asphalt							20	40	60	80	100					
430.0	ASPHALT - about 90 mm		S1A	SS	30												
0.1	Well Graded SAND with Silt and Gravel (FILL) - very dense to compact, brown, damp to moist, trace asphalt pieces in upper 0.25 m		S1B	SS	44												
	- cobbles and boulders noted during augering from about 0.8 m to 1.5 m depth		S2	SS	57		434										
			S3	SS	33		433										43 46 (11)
			S4	SS	11		432										
	- becoming compact to loose, moist to wet at about 3.0 m depth		S5	SS	10												
431.3	Poorly Graded GRAVEL with Silt and Sand (FILL) - very dense, brown, wet		S6	SS	66		431										
430.7			S7A	SS	16												
4.3	Silty SAND with Gravel - compact to dense, brown to grey, wet, trace peat in upper 4.5 m		S7B	SS	17												25 62 (13)
	- becoming grey, occasional to some cobbles at about 4.5 m depth						430										
430.1	COBBLES AND BOULDERS		S8	CORE													
			S9	SS	100		429										No Recovery
			S10	CORE													
			S11	CORE													
428.4	BEDROCK - medium strong, greenish dark grey, very sound, fine grained		S12	CORE			428										Recovery=100%, RQD=100%
6.6	- becoming fractured at about 7.1 m depth		S13	CORE			427										Recovery=100%, RQD=59%
			S14	CORE			426										Recovery=100%, RQD=61%
425.4	End of Borehole																
9.6																	

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

RECORD OF BOREHOLE No BH202

1 OF 1

METRIC

W.P. GWP No. 6319-14-00 LOCATION Flood Creek West (Site No. 48W-261C) MTM ON-16 5,400,594N 427,346E ORIGINATED BY EF
DIST 61 HWY Hwy 11 BOREHOLE TYPE CME 850 Track Carrier / HSA / HQ COMPILED BY RM
DATUM Geodetic DATE 6.28.15 - 6.28.15 CHECKED BY DG/AM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE								
434.8	Asphalt						20	40	60	80	100					
430.0	ASPHALT - about 90 mm		S1A	SS	28											
0.1	Well Graded SAND with Silt and Gravel (FILL) - dense to very dense, brown, damp to moist, trace asphalt pieces in upper 0.25 m		S1B	SS	36											
			S2A	SS	29											
			S2B	SS	25											
			S3	SS	43											
			S4	SS	31											
431.8																
3.1	Silty SAND with Gravel - dense, grey, wet		S5	SS	36											
431.0																
3.8	Well Graded SAND with Silt and Gravel - very dense, grey, wet		S6	SS	100											
430.5																
4.3	BEDROCK - medium strong, greenish dark grey, very sound to sound, fine grained		S7	CORE												
			S8	CORE												
427.8																
7.0	End of Borehole															

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

RECORD OF BOREHOLE No BH203

1 OF 1

METRIC

W.P. GWP No. 6319-14-00 LOCATION Flood Creek West (Site No. 48W-261C) MTM ON-16 5,400,591N 427,361E ORIGINATED BY EF
 DIST 61 HWY Hwy 11 BOREHOLE TYPE Hand Probe / Hand Shovel COMPILED BY RM
 DATUM Geodetic DATE 6.28.15 - 6.28.15 CHECKED BY DG/AM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
431.7	Peat		S1	AUGER													
430.8	PEAT - very soft, dark brown, wet, trace gravel, trace sand, trace roots and rootlets		S2	AUGER													
0.2																	
431.2																	
0.5	Poorly Graded SAND with Silt and Gravel - loose, brown, wet, some peat, trace roots and rootlets		S3	AUGER													
	End of Borehole - refusal																

ONL MOT F-15122-BG - ADM-00223648-E0 - MTO 6 - FLOOD CREEK WEST.GPJ ONL MOT.GDT 10/21/15

RECORD OF BOREHOLE No BH204

1 OF 1

METRIC

W.P. GWP No. 6319-14-00 LOCATION Flood Creek West (Site No. 48W-261C) MTM ON-16 5,400,588N 427,330E ORIGINATED BY EF
 DIST 61 HWY Hwy 11 BOREHOLE TYPE CME 850 Track Carrier / HSA COMPILED BY RM
 DATUM Geodetic DATE 6.27.15 - 6.27.15 CHECKED BY DG/AM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE								WATER CONTENT (%)	
431.9	Rootmat		S1A	SS			431										
430.9 0.1	ROOTMAT - soft, dark brown, wet, some peat		S1B	SS	18												
	Poorly Graded SAND with Silt and Gravel - compact, brown to grey, moist to wet, occasional cobbles and boulders, trace organics in upper 0.6 m		S1C	SS													
431.1 0.8	Silty SAND (TILL) - very dense, grey, moist to wet, occasional cobbles		S2	SS	100												12 49 (39)
430.7																	
1.2	End of Borehole - refusal to SPT and auger																

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

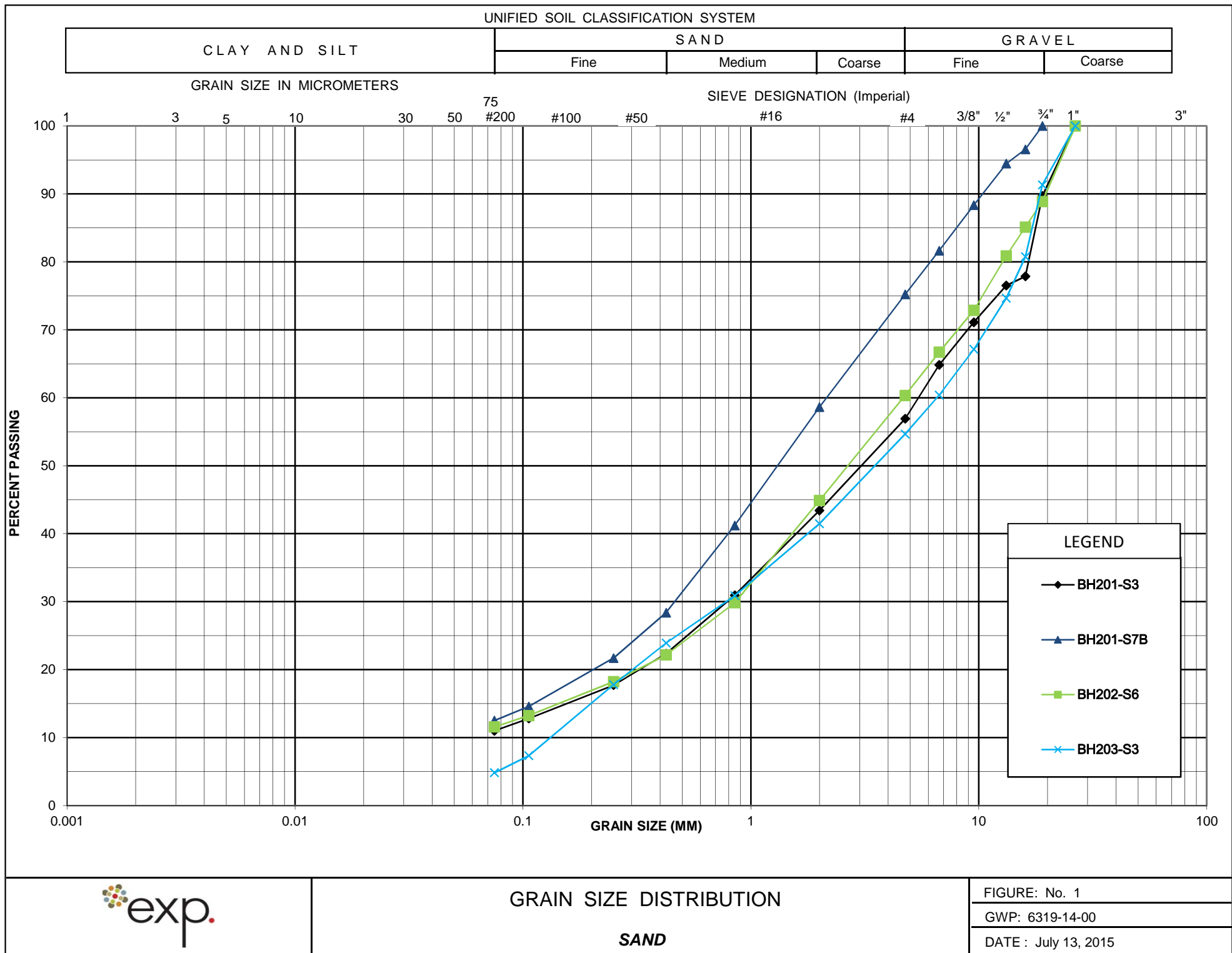


BH201 - Bedrock Core Samples with Depths and Elevations



BH202 - Bedrock Core Samples with Depths and Elevations

Appendix D – Laboratory Data

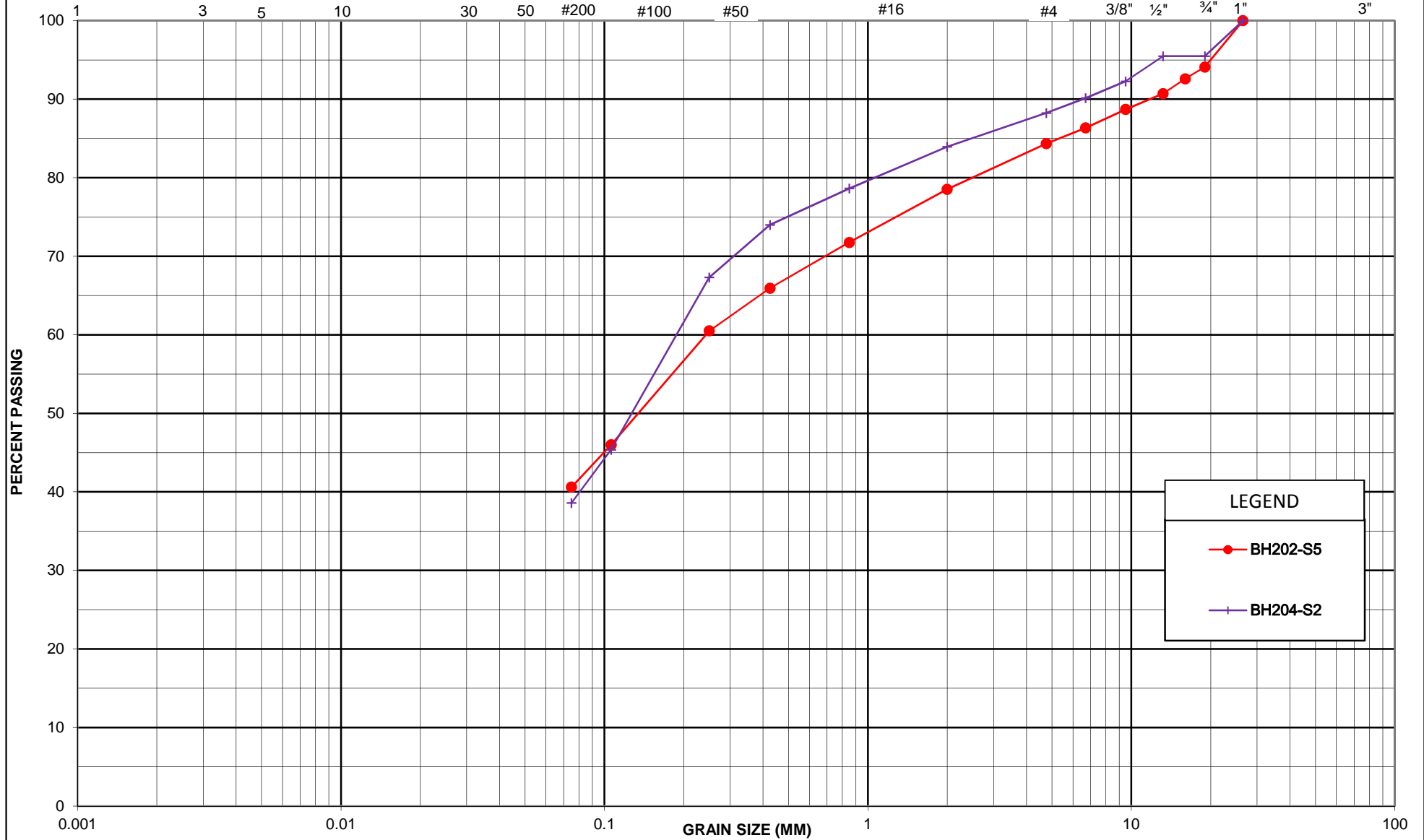


UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY AND SILT	SAND			GRAVEL	
	Fine	Medium	Coarse	Fine	Coarse

GRAIN SIZE IN MICROMETERS

SIEVE DESIGNATION (Imperial)



GRAIN SIZE DISTRIBUTION

SILTY SAND

FIGURE: No. 2

GWP: 6319-14-00

DATE : August 4, 2015

Appendix E – Chemical Analyses

Your Project #: ADM-00223648-E0
Site Location: MTO ASSIGNMENT #6 - HWYS 11 & 502
Your C.O.C. #: na

Attention: Ahileas Mitsopoulos/Michael S

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

Report Date: 2015/07/09
Report #: R3568313
Version: 1 - Final

CERTIFICATE OF ANALYSIS

MAXXAM JOB #: B5C9097

Received: 2015/07/03, 10:55

Sample Matrix: Soil
Samples Received: 10

Analyses	Quantity	Date Extracted	Date Analyzed	Laboratory Method	Reference
Chloride (20:1 extract)	10	N/A	2015/07/09	CAM SOP-00463	EPA 325.2 m
Conductivity	10	N/A	2015/07/08	CAM SOP-00414	OMOE E3138 v2 m
pH CaCl2 EXTRACT	10	2015/07/08	2015/07/08	CAM SOP-00413	EPA 9045 D m
Resistivity of Soil	5	2015/07/03	2015/07/08	CAM SOP-00414	SM 22 2510 m
Resistivity of Soil	5	2015/07/03	2015/07/09	CAM SOP-00414	SM 22 2510 m
Sulphate (20:1 Extract)	10	N/A	2015/07/09	CAM SOP-00464	EPA 375.4 m

Remarks:

Maxxam Analytics has performed all analytical testing herein in accordance with ISO 17025 and the Protocol for Analytical Methods Used in the Assessment of Properties under Part XV.1 of the Environmental Protection Act. All methodologies comply with this document and are validated for use in the laboratory. The methods and techniques employed in this analysis conform to the performance criteria (detection limits, accuracy and precision) as outlined in the Protocol for Analytical Methods Used in the Assessment of Properties under Part XV.1 of the Environmental Protection Act.

The CWS PHC methods employed by Maxxam conform to all prescribed elements of the reference method and performance based elements have been validated. All modifications have been validated and proven equivalent following the 'Alberta Environment Draft Addenda to the CWS-PHC, Appendix 6, Validation of Alternate Methods'. Documentation is available upon request. Maxxam has made the following improvements to the CWS-PHC reference benchmark method: (i) Headspace for F1; and, (ii) Mechanical extraction for F2-F4. Note: F4G cannot be added to the C6 to C50 hydrocarbons. The extraction date for samples field preserved with methanol for F1 and Volatile Organic Compounds is considered to be the date sampled.

Maxxam Analytics is accredited for all specific parameters as required by Ontario Regulation 153/04. Maxxam Analytics is limited in liability to the actual cost of analysis unless otherwise agreed in writing. There is no other warranty expressed or implied. Samples will be retained at Maxxam Analytics for three weeks from receipt of data or as per contract.

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

* RPDs calculated using raw data. The rounding of final results may result in the apparent difference.

Your Project #: ADM-00223648-E0
Site Location: MTO ASSIGNMENT #6 - HWYS 11 & 502
Your C.O.C. #: na

Attention:Ahileas Mitsopoulos/Michael S

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

Report Date: 2015/07/09
Report #: R3568313
Version: 1 - Final

CERTIFICATE OF ANALYSIS

MAXXAM JOB #: B5C9097
Received: 2015/07/03, 10:55

Encryption Key

Please direct all questions regarding this Certificate of Analysis to your Project Manager.
Hina Siddiqui, Project Manager –Environmental Customer Service
Email: HSiddiqui@maxxam.ca
Phone# (905) 817-5700

=====

Maxxam has procedures in place to guard against improper use of the electronic signature and have the required "signatories", as per section 5.10.2 of ISO/IEC 17025:2005(E), signing the reports. For Service Group specific validation please refer to the Validation Signature Page.

Maxxam Job #: B5C9097
Report Date: 2015/07/09

exp Services Inc
Client Project #: ADM-00223648-E0
Site Location: MTO ASSIGNMENT #6 - HWYS 11 & 502

RESULTS OF ANALYSES OF SOIL

Maxxam ID		AOD715	AOD716	AOD716	AOD717	AOD718		
Sampling Date		2015/06/19 14:10	2015/06/27 12:15	2015/06/27 12:15	2015/06/28 10:20	2015/06/28 17:00		
COC Number		na	na	na	na	na		
	Units	BH101-S7	BH104-S3B/S4/S5	BH104-S3B/S4/S5 Lab-Dup	BH201-S7A	BH203-S3	RDL	QC Batch

Calculated Parameters								
Resistivity	ohm-cm	1300	2500		3300	1800		4091370
Inorganics								
Soluble (20:1) Chloride (Cl)	ug/g	790	190	200	170	320	20	4094438
Conductivity	umho/cm	773	395	399	301	557	2	4096183
Available (CaCl2) pH	pH	6.34	6.65		5.49	5.43	N/A	4094481
Soluble (20:1) Sulphate (SO4)	ug/g	270	25	24	<20	<20	20	4094443
RDL = Reportable Detection Limit QC Batch = Quality Control Batch Lab-Dup = Laboratory Initiated Duplicate N/A = Not Applicable								

Maxxam ID		AOD719	AOD720	AOD721	AOD722	AOD723	AOD724		
Sampling Date		2015/06/20 07:25	2015/06/26 06:20	2015/06/26 16:15	2015/06/25 15:30	2015/06/25 10:30	2015/06/25 14:10		
COC Number		na	na	na	na	na	na		
	Units	BH301-S9B/S10/S11	BH304-S3	BH403-S3	BH404-S5B	BH503-S4	BH504-S1B	RDL	QC Batch

Calculated Parameters									
Resistivity	ohm-cm	2300	7000	4800	8400	5300	1500		4091370
Inorganics									
Soluble (20:1) Chloride (Cl)	ug/g	220	36	81	<20	89	370	20	4094438
Conductivity	umho/cm	435	143	209	119	190	646	2	4096183
Available (CaCl2) pH	pH	6.54	6.72	6.59	6.72	5.89	4.90	N/A	4094481
Soluble (20:1) Sulphate (SO4)	ug/g	30	<20	<20	27	<20	<20	20	4094443
RDL = Reportable Detection Limit QC Batch = Quality Control Batch N/A = Not Applicable									

Maxxam Job #: B5C9097
Report Date: 2015/07/09

exp Services Inc
Client Project #: ADM-00223648-E0
Site Location: MTO ASSIGNMENT #6 - HWYS 11 & 502

TEST SUMMARY

Maxxam ID: AOD715
Sample ID: BH101-S7
Matrix: Soil

Collected: 2015/06/19
Shipped:
Received: 2015/07/03

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	4094438	N/A	2015/07/09	Deonarine Ramnarine
Conductivity	AT	4096183	N/A	2015/07/08	Lemeneh Addis
pH CaCl2 EXTRACT	AT	4094481	2015/07/08	2015/07/08	Surinder Rai
Resistivity of Soil		4091370	2015/07/08	2015/07/08	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	4094443	N/A	2015/07/09	Deonarine Ramnarine

Maxxam ID: AOD716
Sample ID: BH104-S3B/S4/S5
Matrix: Soil

Collected: 2015/06/27
Shipped:
Received: 2015/07/03

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	4094438	N/A	2015/07/09	Deonarine Ramnarine
Conductivity	AT	4096183	N/A	2015/07/08	Lemeneh Addis
pH CaCl2 EXTRACT	AT	4094481	2015/07/08	2015/07/08	Surinder Rai
Resistivity of Soil		4091370	2015/07/08	2015/07/08	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	4094443	N/A	2015/07/09	Deonarine Ramnarine

Maxxam ID: AOD716 Dup
Sample ID: BH104-S3B/S4/S5
Matrix: Soil

Collected: 2015/06/27
Shipped:
Received: 2015/07/03

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	4094438	N/A	2015/07/09	Deonarine Ramnarine
Conductivity	AT	4096183	N/A	2015/07/08	Lemeneh Addis
Sulphate (20:1 Extract)	KONE/EC	4094443	N/A	2015/07/09	Deonarine Ramnarine

Maxxam ID: AOD717
Sample ID: BH201-S7A
Matrix: Soil

Collected: 2015/06/28
Shipped:
Received: 2015/07/03

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	4094438	N/A	2015/07/09	Deonarine Ramnarine
Conductivity	AT	4096183	N/A	2015/07/08	Lemeneh Addis
pH CaCl2 EXTRACT	AT	4094481	2015/07/08	2015/07/08	Surinder Rai
Resistivity of Soil		4091370	2015/07/08	2015/07/08	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	4094443	N/A	2015/07/09	Deonarine Ramnarine

Maxxam ID: AOD718
Sample ID: BH203-S3
Matrix: Soil

Collected: 2015/06/28
Shipped:
Received: 2015/07/03

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	4094438	N/A	2015/07/09	Deonarine Ramnarine
Conductivity	AT	4096183	N/A	2015/07/08	Lemeneh Addis
pH CaCl2 EXTRACT	AT	4094481	2015/07/08	2015/07/08	Surinder Rai
Resistivity of Soil		4091370	2015/07/08	2015/07/08	Automated Statchk

Maxxam Job #: B5C9097
Report Date: 2015/07/09

exp Services Inc
Client Project #: ADM-00223648-E0
Site Location: MTO ASSIGNMENT #6 - HWYS 11 & 502

TEST SUMMARY

Maxxam ID: AOD718
Sample ID: BH203-S3
Matrix: Soil

Collected: 2015/06/28
Shipped:
Received: 2015/07/03

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Sulphate (20:1 Extract)	KONE/EC	4094443	N/A	2015/07/09	Deonarine Ramnarine

Maxxam ID: AOD719
Sample ID: BH301-S9B/S10/S11
Matrix: Soil

Collected: 2015/06/20
Shipped:
Received: 2015/07/03

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	4094438	N/A	2015/07/09	Deonarine Ramnarine
Conductivity	AT	4096183	N/A	2015/07/08	Lemeneh Addis
pH CaCl2 EXTRACT	AT	4094481	2015/07/08	2015/07/08	Surinder Rai
Resistivity of Soil		4091370	2015/07/08	2015/07/08	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	4094443	N/A	2015/07/09	Deonarine Ramnarine

Maxxam ID: AOD720
Sample ID: BH304-S3
Matrix: Soil

Collected: 2015/06/26
Shipped:
Received: 2015/07/03

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	4094438	N/A	2015/07/09	Deonarine Ramnarine
Conductivity	AT	4096183	N/A	2015/07/08	Lemeneh Addis
pH CaCl2 EXTRACT	AT	4094481	2015/07/08	2015/07/08	Surinder Rai
Resistivity of Soil		4091370	2015/07/09	2015/07/09	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	4094443	N/A	2015/07/09	Deonarine Ramnarine

Maxxam ID: AOD721
Sample ID: BH403-S3
Matrix: Soil

Collected: 2015/06/26
Shipped:
Received: 2015/07/03

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	4094438	N/A	2015/07/09	Deonarine Ramnarine
Conductivity	AT	4096183	N/A	2015/07/08	Lemeneh Addis
pH CaCl2 EXTRACT	AT	4094481	2015/07/08	2015/07/08	Surinder Rai
Resistivity of Soil		4091370	2015/07/09	2015/07/09	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	4094443	N/A	2015/07/09	Deonarine Ramnarine

Maxxam ID: AOD722
Sample ID: BH404-S5B
Matrix: Soil

Collected: 2015/06/25
Shipped:
Received: 2015/07/03

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	4094438	N/A	2015/07/09	Deonarine Ramnarine
Conductivity	AT	4096183	N/A	2015/07/08	Lemeneh Addis
pH CaCl2 EXTRACT	AT	4094481	2015/07/08	2015/07/08	Surinder Rai
Resistivity of Soil		4091370	2015/07/09	2015/07/09	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	4094443	N/A	2015/07/09	Deonarine Ramnarine

Maxxam Job #: B5C9097
Report Date: 2015/07/09

exp Services Inc
Client Project #: ADM-00223648-E0
Site Location: MTO ASSIGNMENT #6 - HWYS 11 & 502

TEST SUMMARY

Maxxam ID: AOD723
Sample ID: BH503-S4
Matrix: Soil

Collected: 2015/06/25
Shipped:
Received: 2015/07/03

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	4094438	N/A	2015/07/09	Deonarine Ramnarine
Conductivity	AT	4096183	N/A	2015/07/08	Lemeneh Addis
pH CaCl2 EXTRACT	AT	4094481	2015/07/08	2015/07/08	Surinder Rai
Resistivity of Soil		4091370	2015/07/09	2015/07/09	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	4094443	N/A	2015/07/09	Deonarine Ramnarine

Maxxam ID: AOD724
Sample ID: BH504-S1B
Matrix: Soil

Collected: 2015/06/25
Shipped:
Received: 2015/07/03

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	4094438	N/A	2015/07/09	Deonarine Ramnarine
Conductivity	AT	4096183	N/A	2015/07/08	Lemeneh Addis
pH CaCl2 EXTRACT	AT	4094481	2015/07/08	2015/07/08	Surinder Rai
Resistivity of Soil		4091370	2015/07/09	2015/07/09	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	4094443	N/A	2015/07/09	Deonarine Ramnarine

Maxxam Job #: B5C9097
Report Date: 2015/07/09

exp Services Inc
Client Project #: ADM-00223648-E0
Site Location: MTO ASSIGNMENT #6 - HWYS 11 & 502

GENERAL COMMENTS

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

Package 1	2.7°C
-----------	-------

Results relate only to the items tested.

Maxxam Job #: B5C9097
Report Date: 2015/07/09

QUALITY ASSURANCE REPORT

exp Services Inc
Client Project #: ADM-00223648-E0
Site Location: MTO ASSIGNMENT #6 - HWYS 11 & 502

QC Batch	Parameter	Date	Matrix Spike		Spiked Blank		Method Blank		RPD	
			% Recovery	QC Limits	% Recovery	QC Limits	Value	Units	Value (%)	QC Limits
4094438	Soluble (20:1) Chloride (Cl)	2015/07/09	NC	70 - 130	107	70 - 130	<20	ug/g	6.5	35
4094443	Soluble (20:1) Sulphate (SO4)	2015/07/09	NC	70 - 130	109	70 - 130	<20	ug/g	NC	35
4094481	Available (CaCl2) pH	2015/07/08			100	97 - 103			0.51	N/A
4096183	Conductivity	2015/07/08			102	90 - 110	<2	umho/cm	1.0	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 spiked amount was too small to permit a reliable recovery calculation (matrix spike concentration was less than 2x that of the native sample concentration).

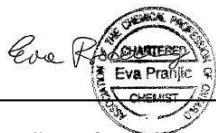
NC (Duplicate RPD): The duplicate RPD was not calculated. The concentration in the sample and/or duplicate was too low to permit a reliable RPD calculation (one or both samples < 5x RDL).

Maxxam Job #: B5C9097
Report Date: 2015/07/09

exp Services Inc
Client Project #: ADM-00223648-E0
Site Location: MTO ASSIGNMENT #6 - HWYS 11 & 502

VALIDATION SIGNATURE PAGE

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



Ewa Pranjić, M.Sc., C.Chem, Scientific Specialist

Maxxam has procedures in place to guard against improper use of the electronic signature and have the required "signatories", as per section 5.10.2 of ISO/IEC 17025:2005(E), signing the reports. For Service Group specific validation please refer to the Validation Signature Page.

Maxxam Analytics International Corporation o/a Maxxam Analytics

Appendix F – Slope Stability Analysis

Flood Creek West Culvert
 Hwy 11
 South Embankment
 Drained Conditions

Name: Sand with Silt and Gravel Fill
 Model: Mohr-Coulomb
 Unit Weight: 20 kN/m³
 Cohesion': 0 kPa
 Phi': 35 °

Name: Silty Sand with Gravel (Compact to Very Dense)
 Model: Mohr-Coulomb
 Unit Weight: 20 kN/m³
 Cohesion': 0 kPa
 Phi': 34 °

Name: Silty Sand with Gravel (Very Dense)
 Model: Mohr-Coulomb
 Unit Weight: 20 kN/m³
 Cohesion': 0 kPa
 Phi': 34 °

Name: Cobbles and Boulders
 Model: Mohr-Coulomb
 Unit Weight: 21 kN/m³
 Cohesion': 0 kPa
 Phi': 40 °

Name: Bedrock
 Model: Bedrock (Impenetrable)

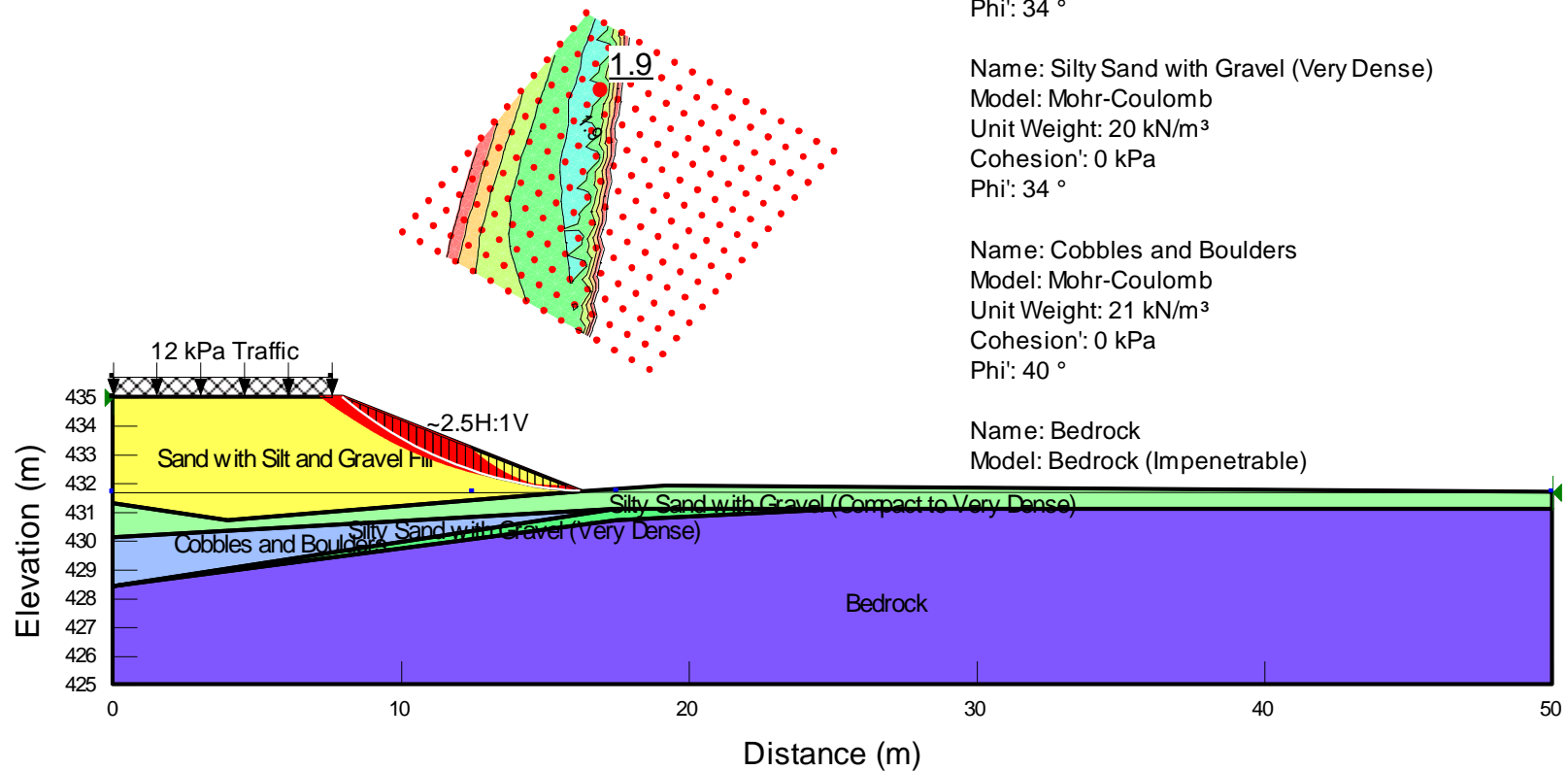
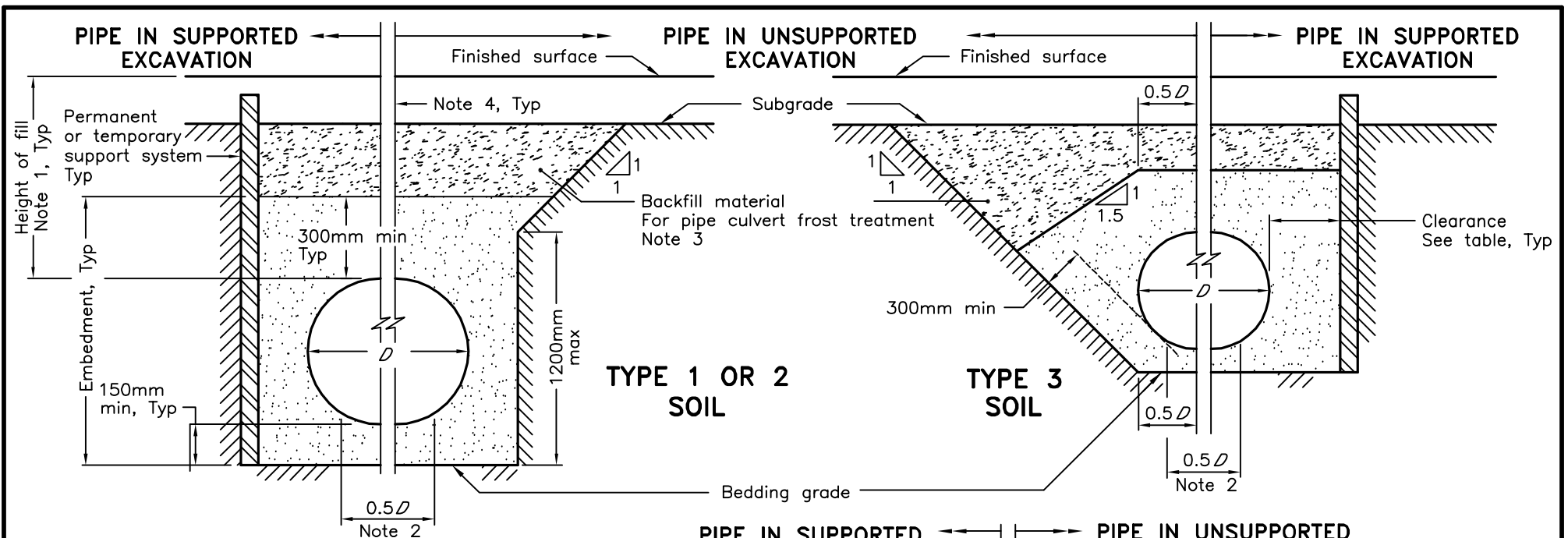


Figure F1: South Embankment – drained static Condition

Appendix G – OPSDs

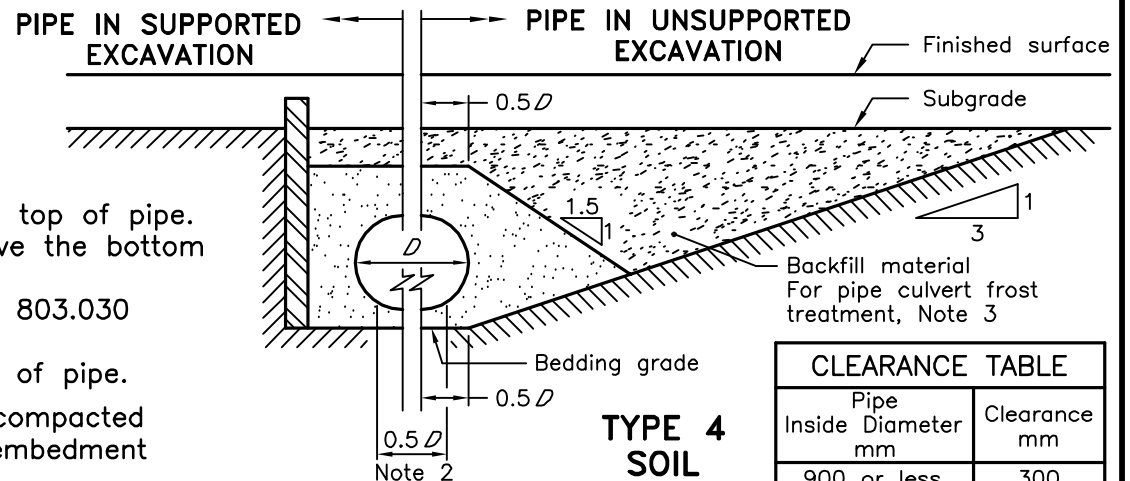


LEGEND:

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

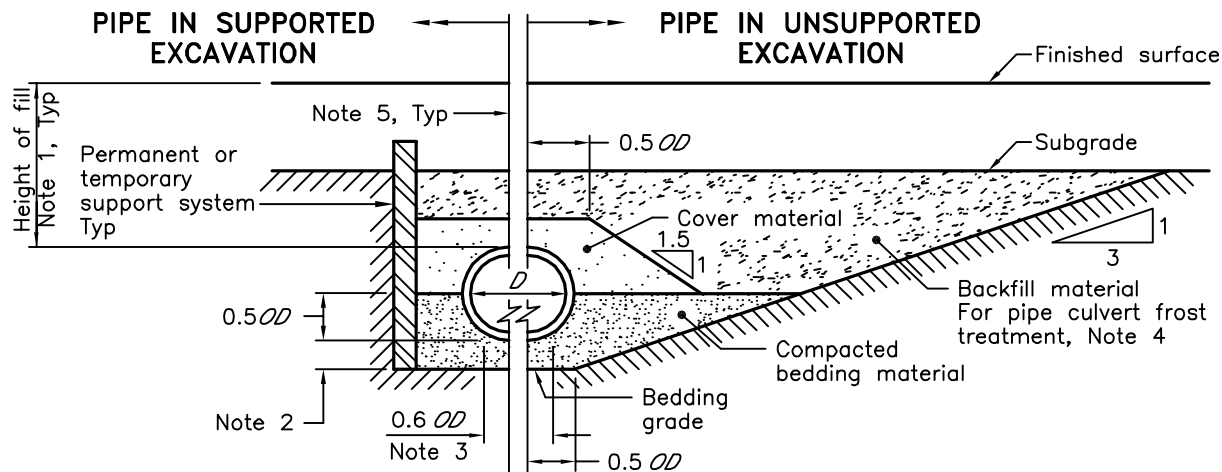
Nov 2010

Rev 2

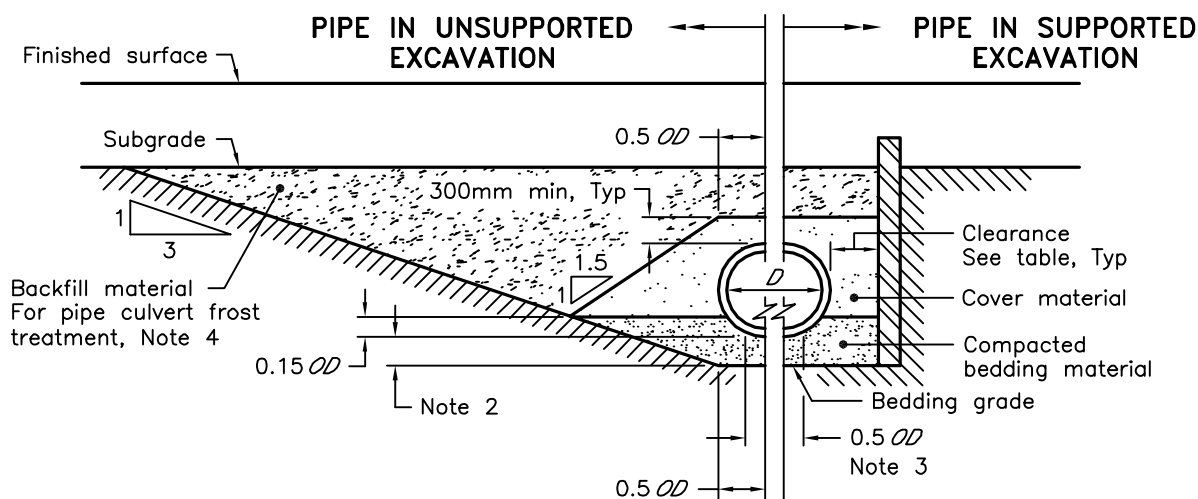
FLEXIBLE PIPE
EMBEDMENT AND BACKFILL
EARTH EXCAVATION

OPSD 802.010





CLASS B BEDDING



CLASS C BEDDING

LEGEND:

D – Inside diameter
 OD – Outside diameter

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

NOTES:

- Height of fill is measured from the finished surface to top of pipe.
 - The minimum bedding depth below the pipe shall be $0.15D$.
In no case shall this dimension be less than 150mm or greater than 300mm.
 - The pipe bed shall be compacted and shaped to receive the bottom of the pipe.
 - Pipe culvert frost treatment shall be according to OPSD 803.030 and 803.031.
 - Condition of excavation is symmetrical about centreline of pipe.
- A Soil types as defined in the Occupational Health and Safety Act and Regulations for Construction Projects.
- B All dimensions are in metres unless otherwise shown.

ONTARIO PROVINCIAL STANDARD DRAWING

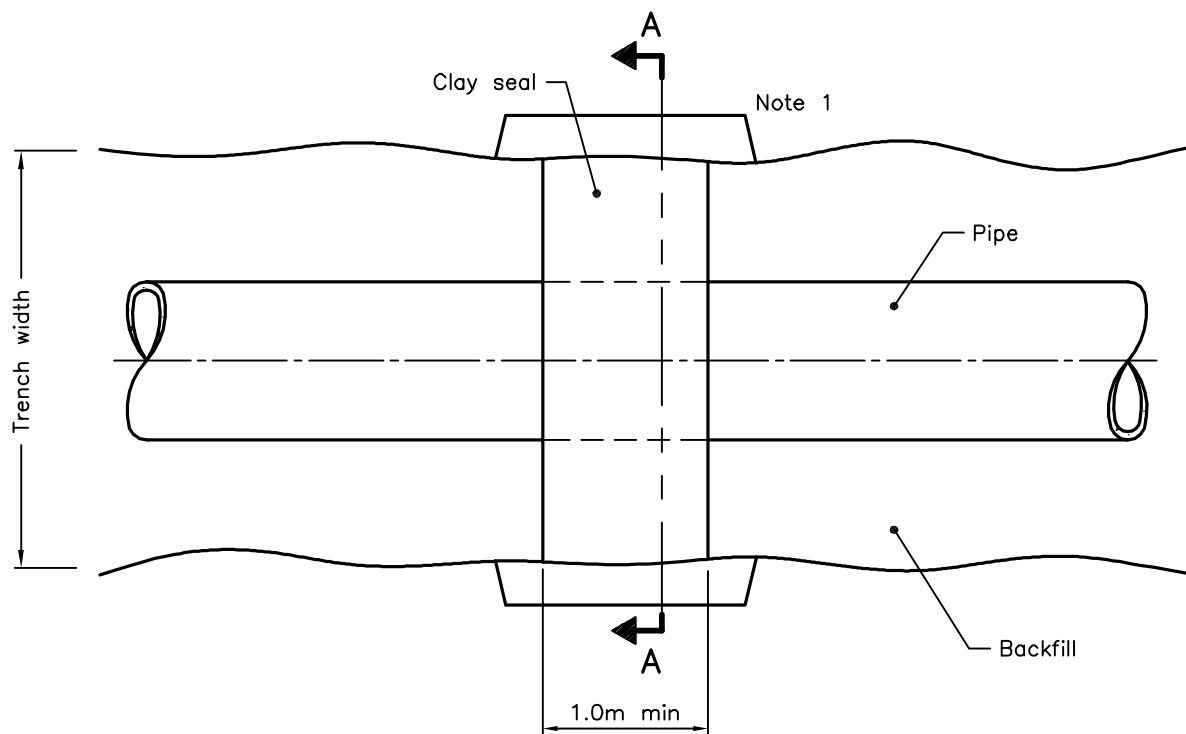
Nov 2010

Rev 2

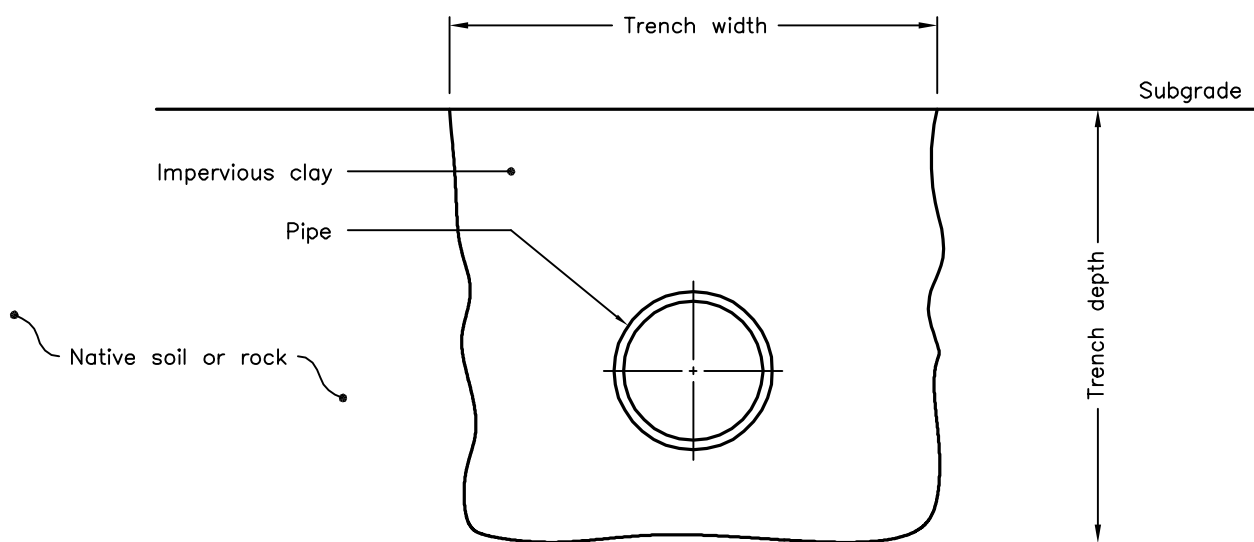
**RIGID PIPE BEDDING,
 COVER, AND BACKFILL
 TYPE 4 SOIL – EARTH EXCAVATION**

OPSD 802.032





PLAN



SECTION A-A

NOTES:

1. Key into undisturbed trench soil.

A Clay seal shall extend from bottom of trench excavation to the subgrade.

B Clay seal shall be located so that no pipe joints are within the clay seal material.

C All dimensions are in metres unless otherwise shown.

ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2011

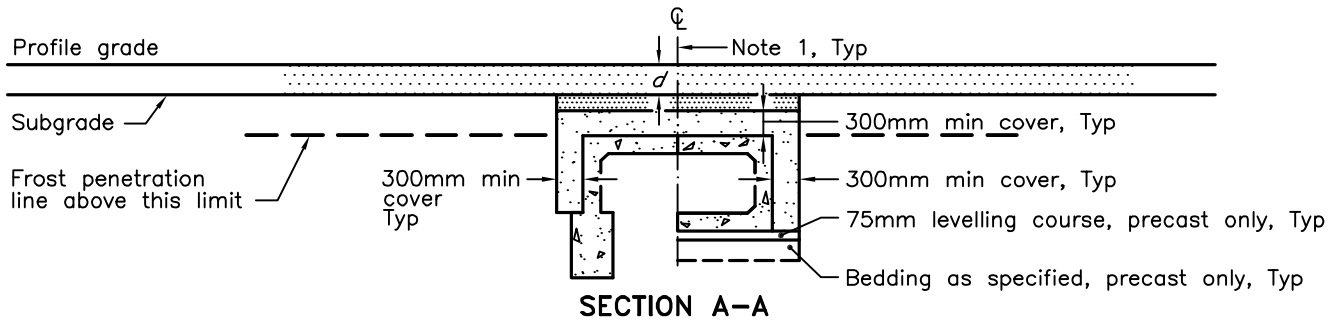
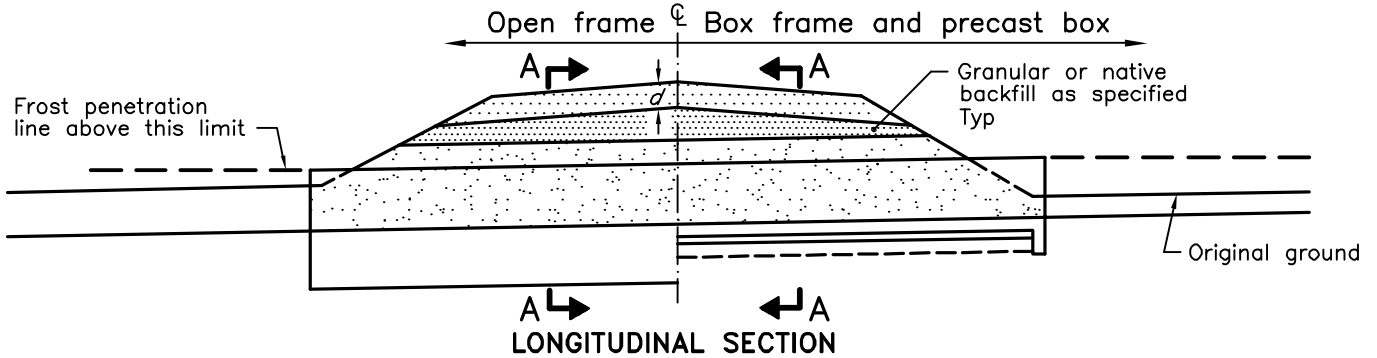
Rev 1

CLAY SEAL FOR PIPE TRENCHES

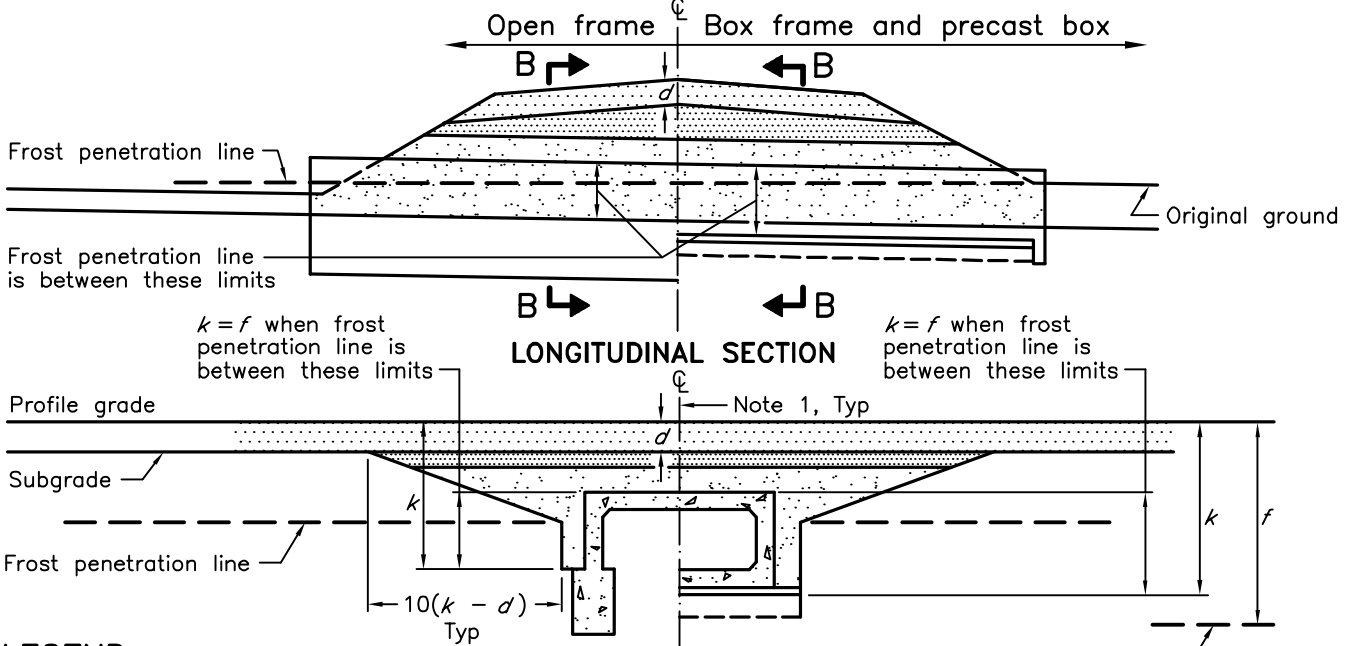
OPSD 802.095



FROST PENETRATION LINE AT OR ABOVE TOP OF CULVERT



FROST PENETRATION LINE BELOW TOP OF CULVERT



LEGEND:

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

NOTES:

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

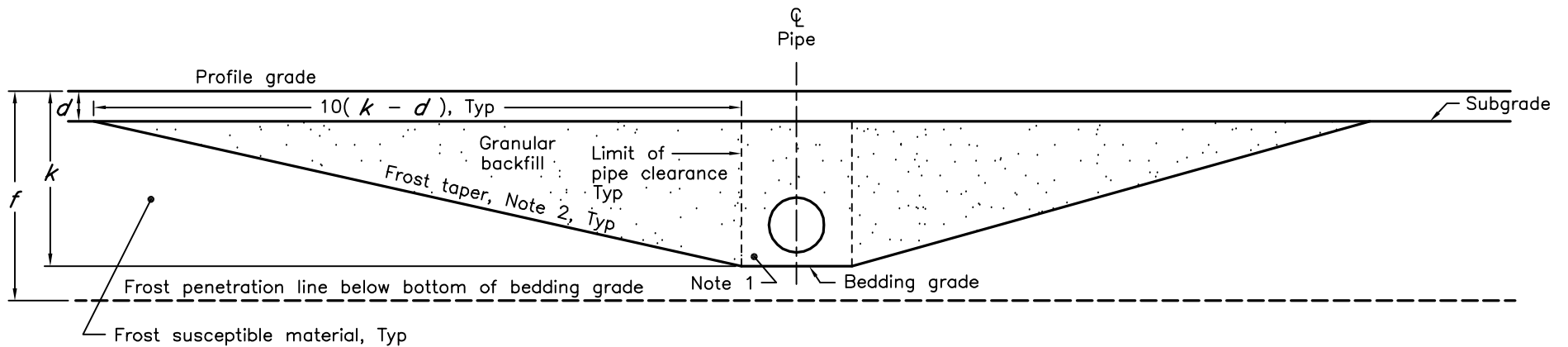
ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2010 Rev 2

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

OPSD 803.010





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 Frost tapers shall start at bedding grade.

LEGEND:

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

ONTARIO PROVINCIAL STANDARD DRAWING

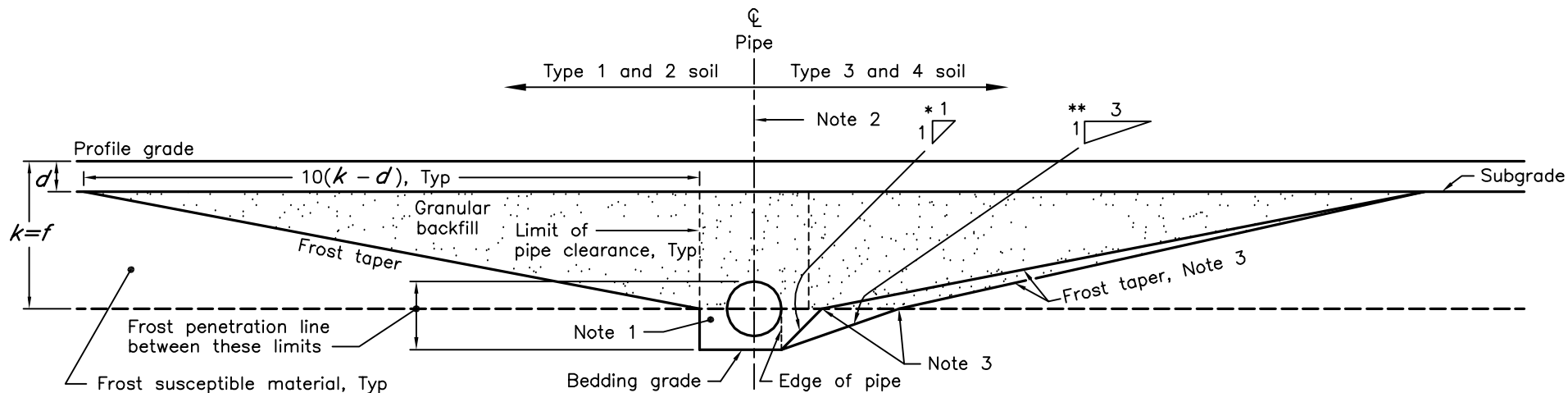
Nov 2010

Rev 2

FROST TREATMENT – PIPE CULVERTS
FROST PENETRATION LINE BELOW
BEDDING GRADE

OPSD 803.030





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

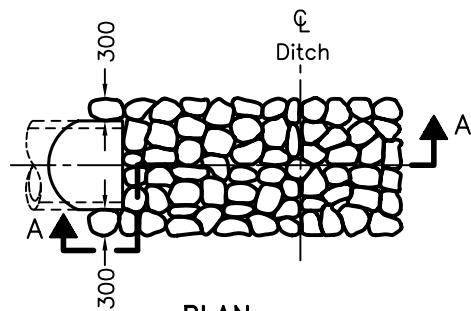
ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2010 Rev 3

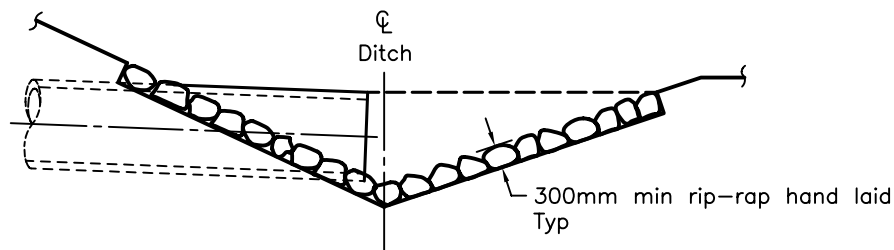
FROST TREATMENT – PIPE CULVERTS
FROST PENETRATION LINE BETWEEN
TOP OF PIPE AND BEDDING GRADE

OPSD 803.031

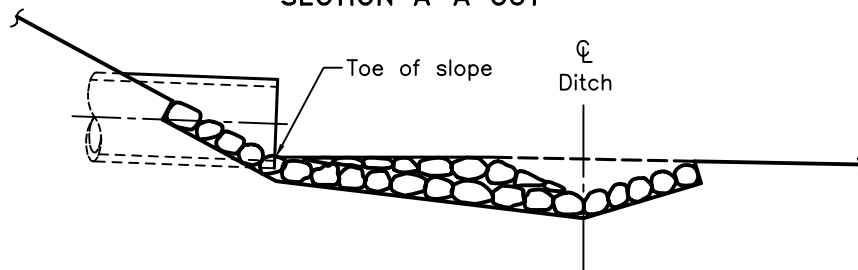




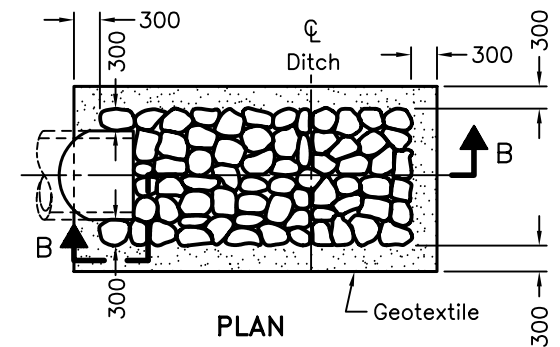
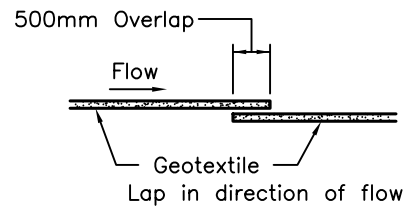
PLAN
CUT OR FILL



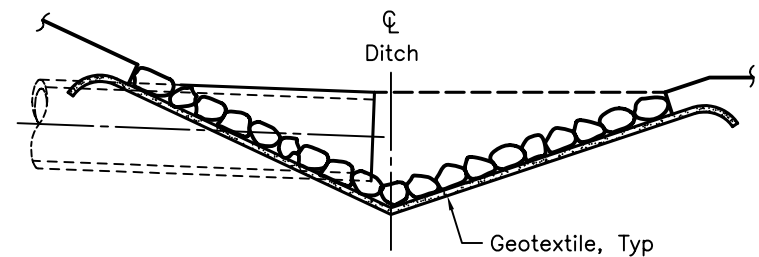
SECTION A-A CUT



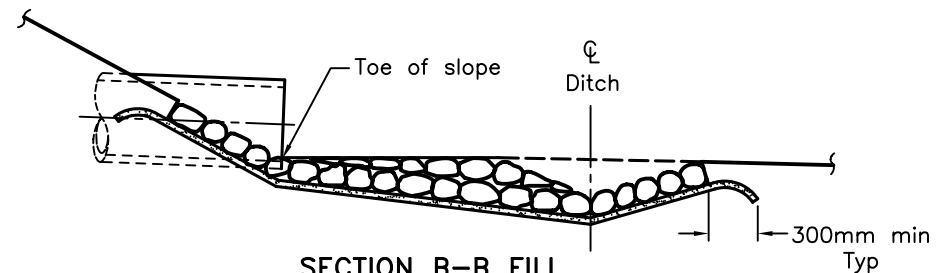
SECTION A-A FILL
TYPE A – WITHOUT GEOTEXTILE



PLAN
CUT OR FILL



SECTION B-B CUT



SECTION B-B FILL
TYPE B – WITH GEOTEXTILE

NOTES:

A All dimensions are in millimetres unless otherwise shown.

ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2001

Rev 0

RIP-RAP TREATMENT
FOR SEWER AND CULVERT OUTLETS



OPSD – 810.010

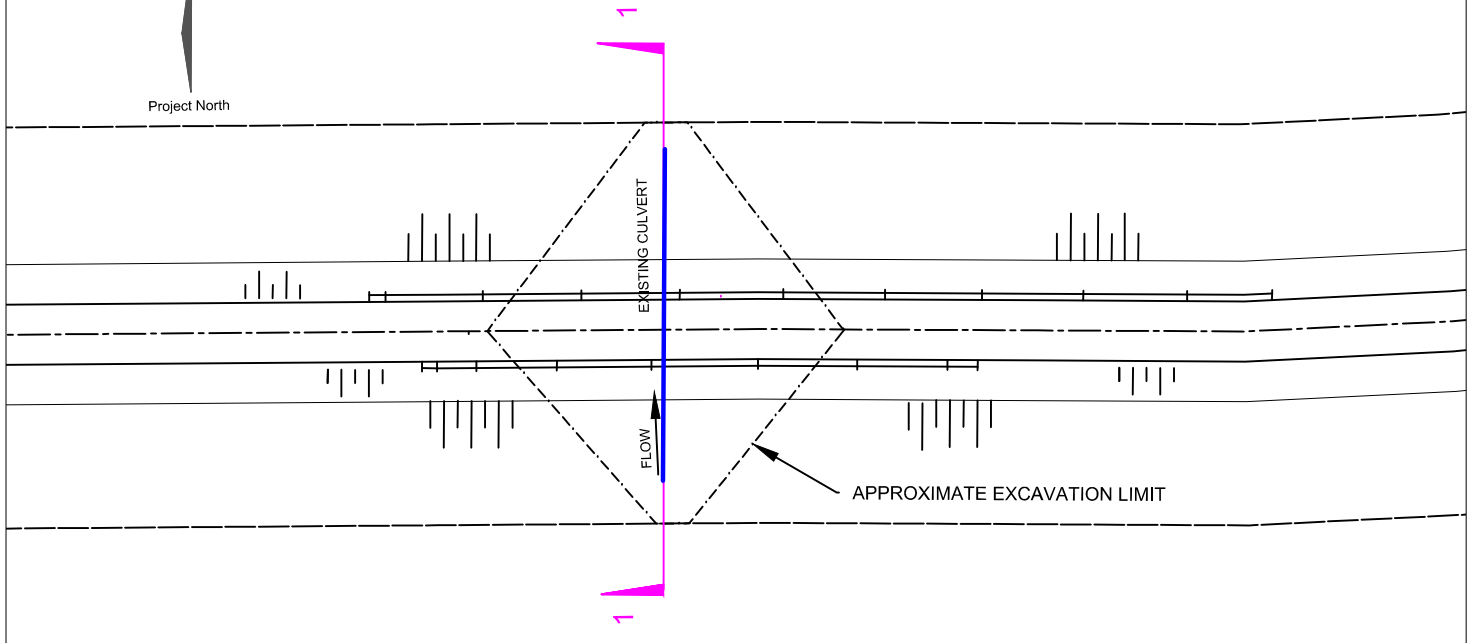
Appendix H – Schematic Sketches for Construction Alternatives

FIGURE H.1: FULL ROAD CLOSURE USING EXISTING ROADWAYS AND OPEN CUT UNSUPPORTED EXCAVATION OPTION1

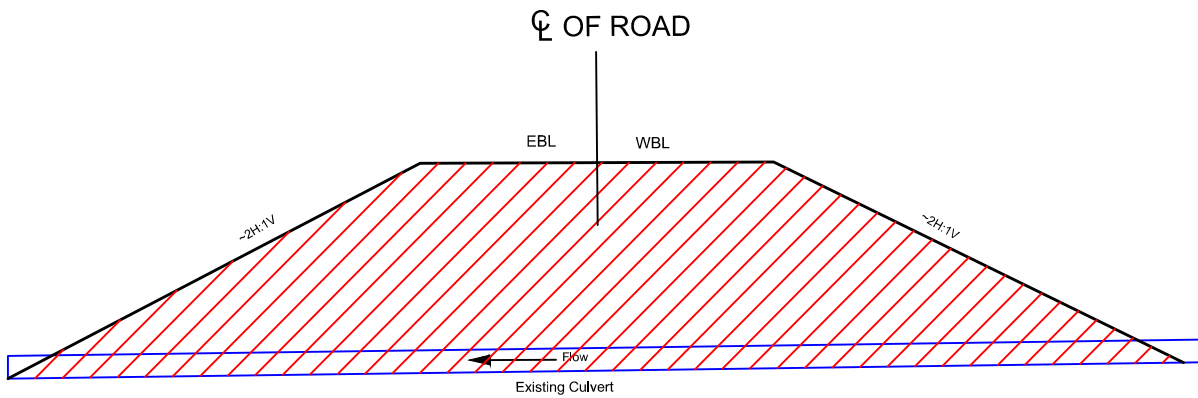
SCHEMATIC DIAGRAMS (NTS)



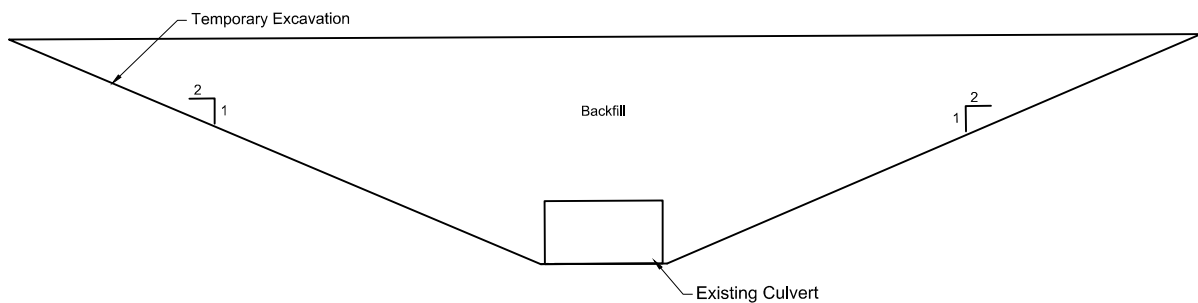
Project North



PLAN



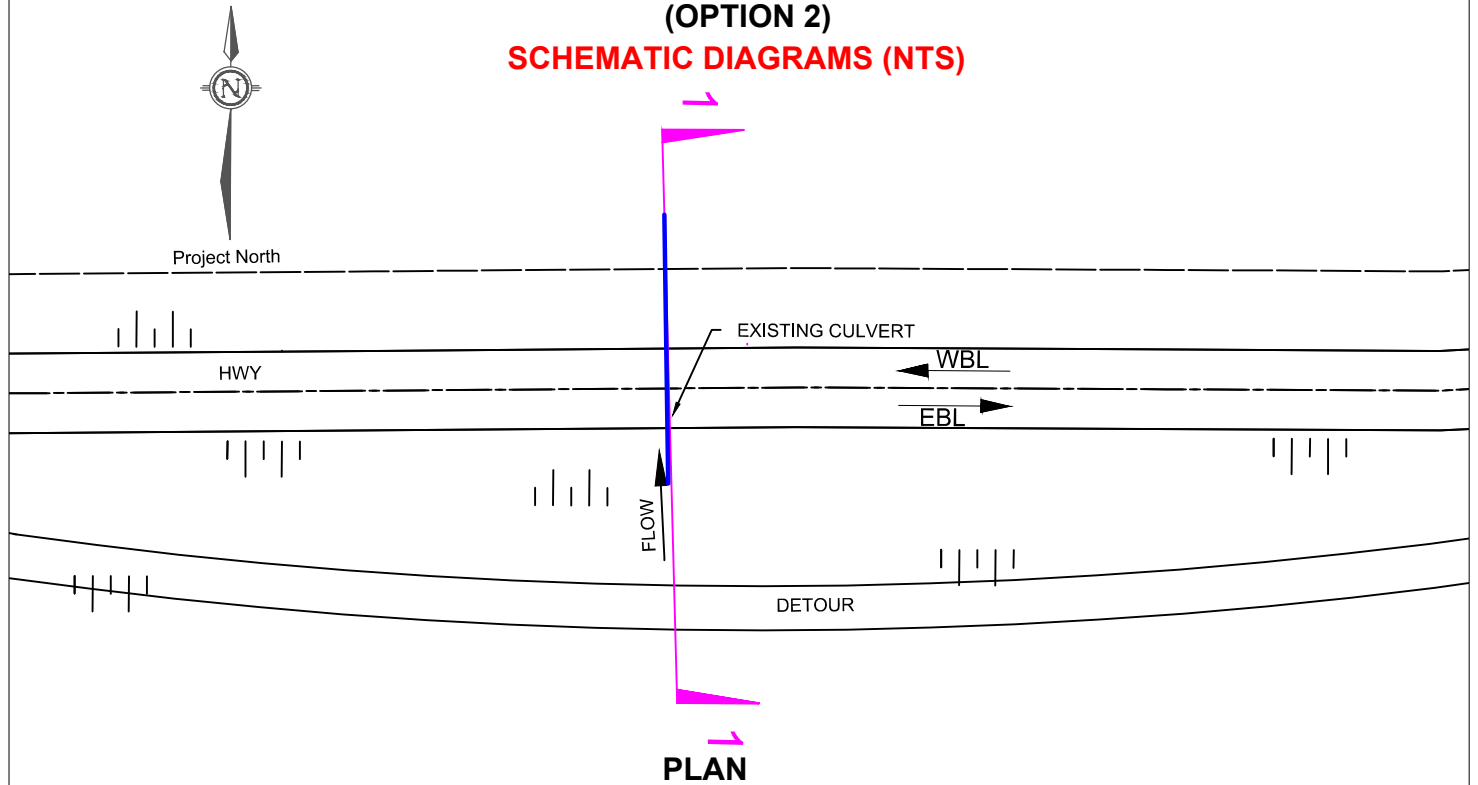
SECTION 1-1



SECTION 2-2

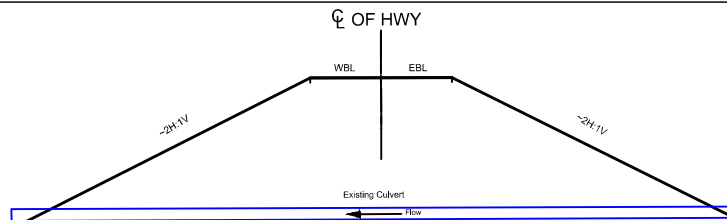
FIGURE H.2: TEMPORARY LOCAL DETOUR AND OPEN CUT UNSUPPORTED EXCAVATION (OPTION 2)

SCHEMATIC DIAGRAMS (NTS)

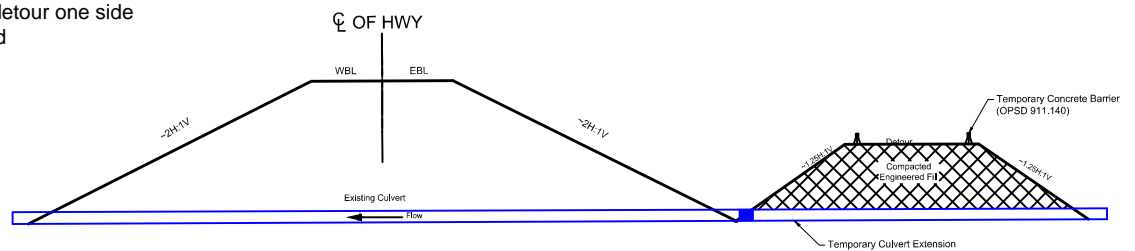


RECOMMENDED STAGES

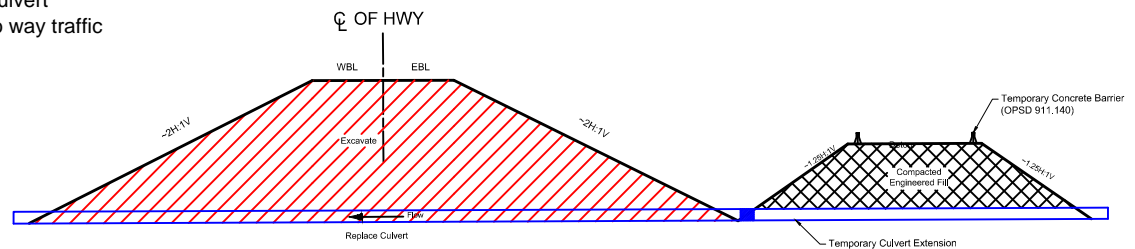
1.0 Stage 1 - Current condition



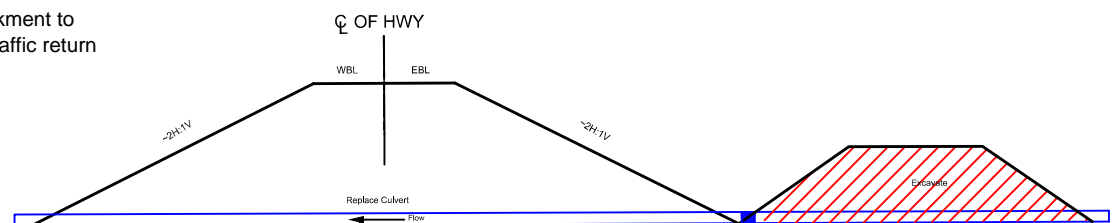
2.0 Stage 2 - Build temporary detour one side
Two-way traffic on existing road



3.0 Stage 3 - Excavation and culvert construction on other side; Two way traffic shifted to detour



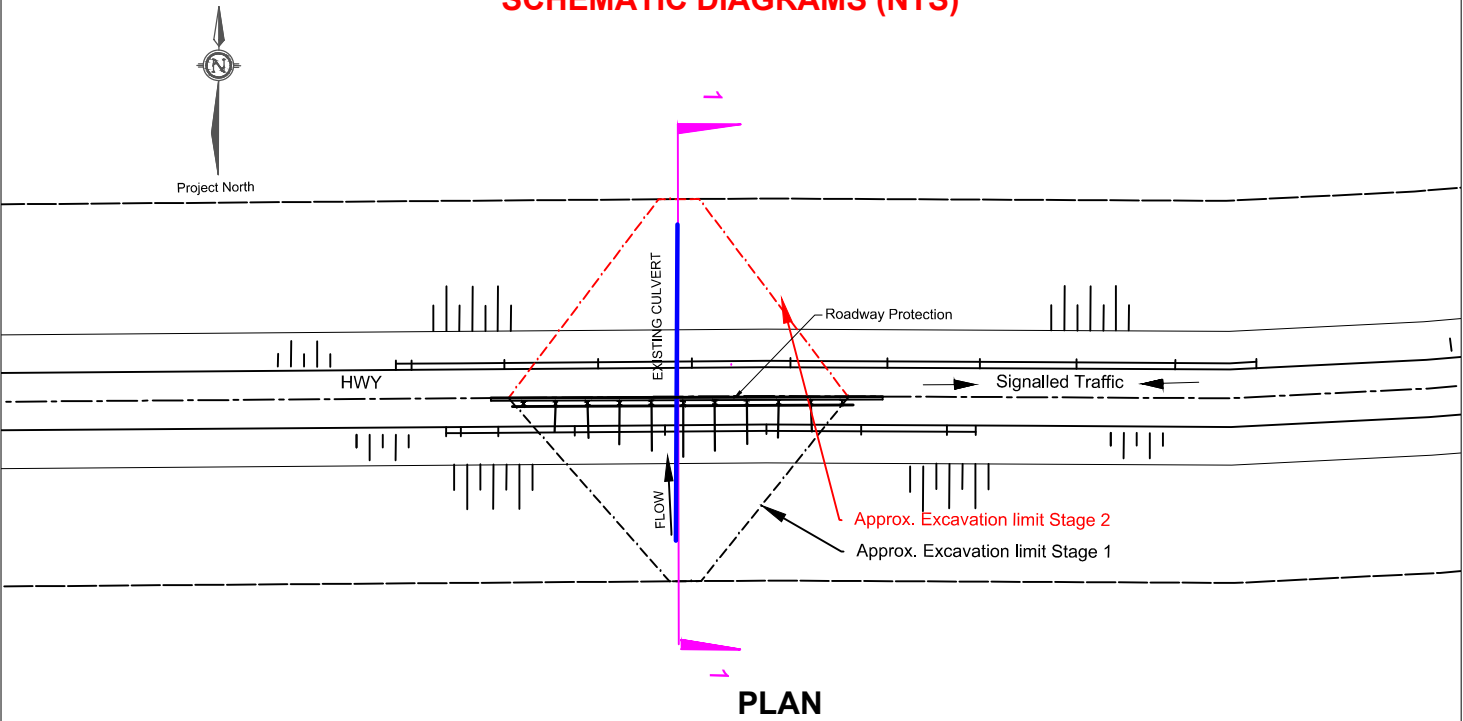
4.0 Stage 4 - Build the embankment to existing alignment; Two-way traffic return



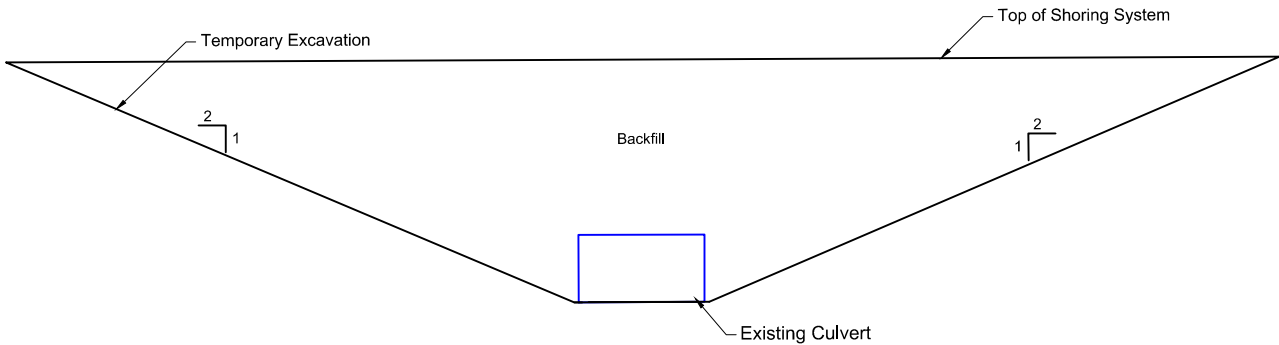
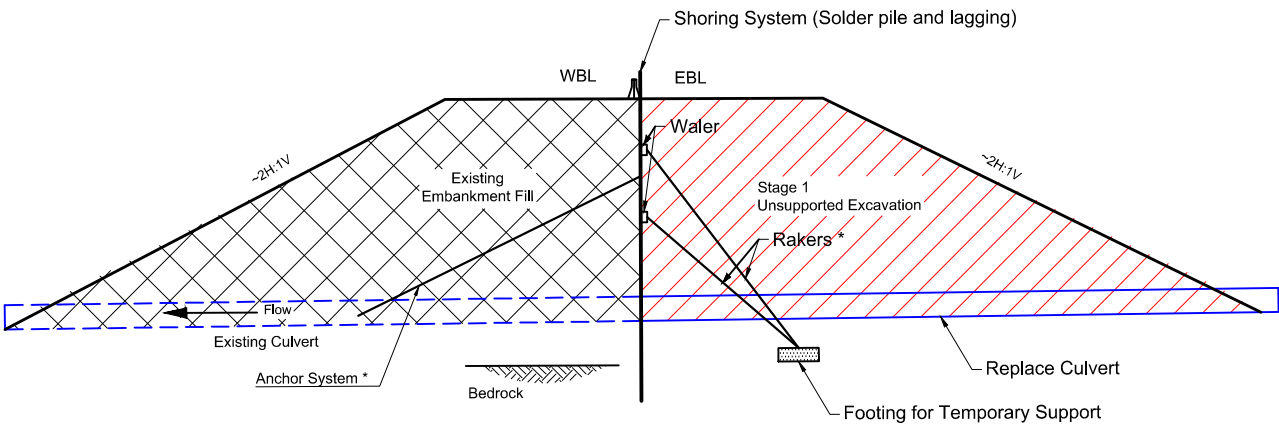
SECTION 1-1

FIGURE H.3.A: HALF AND HALF CONSTRUCTION WITH UNSUPPORTED CUT SIDES

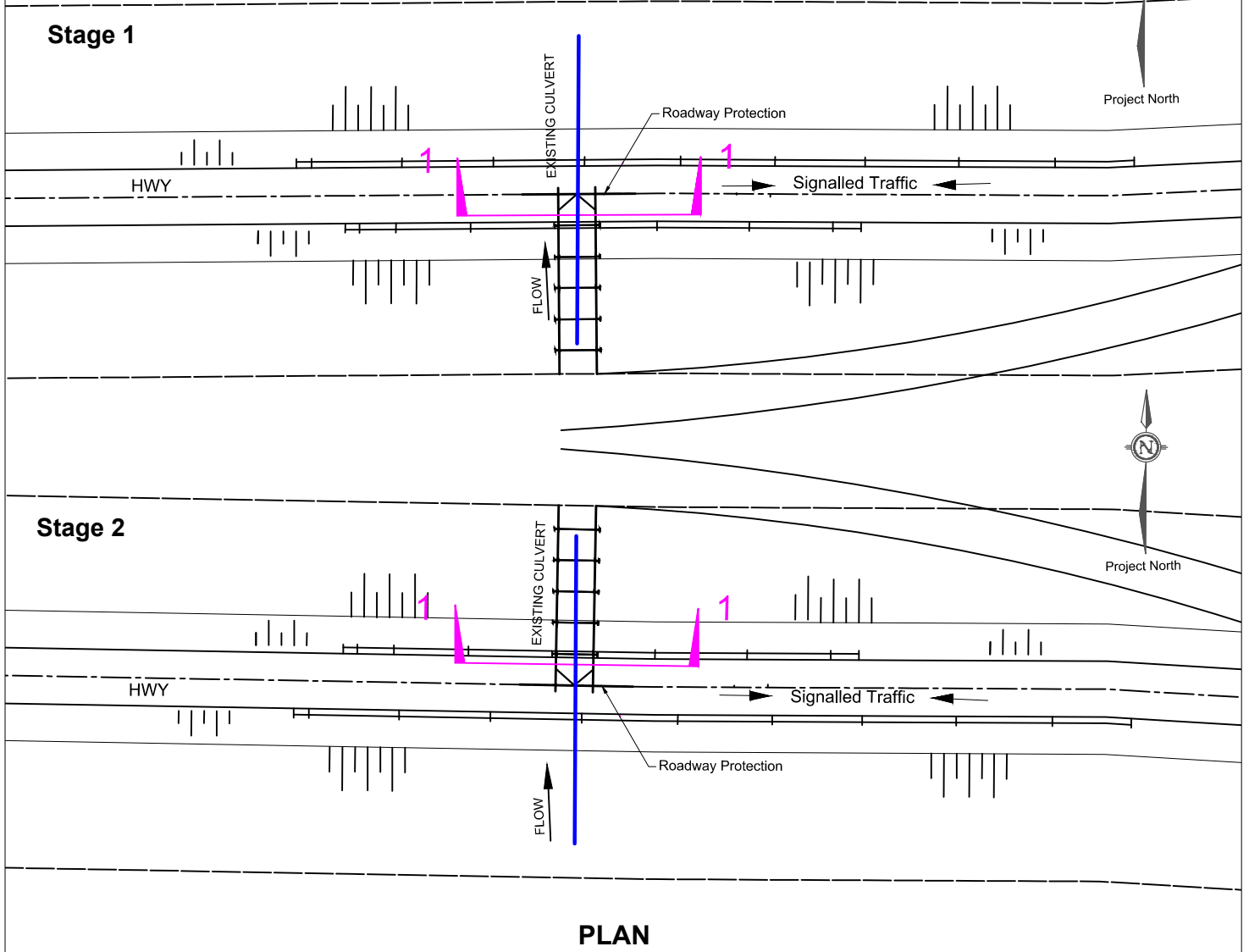
SCHEMATIC DIAGRAMS (NTS)



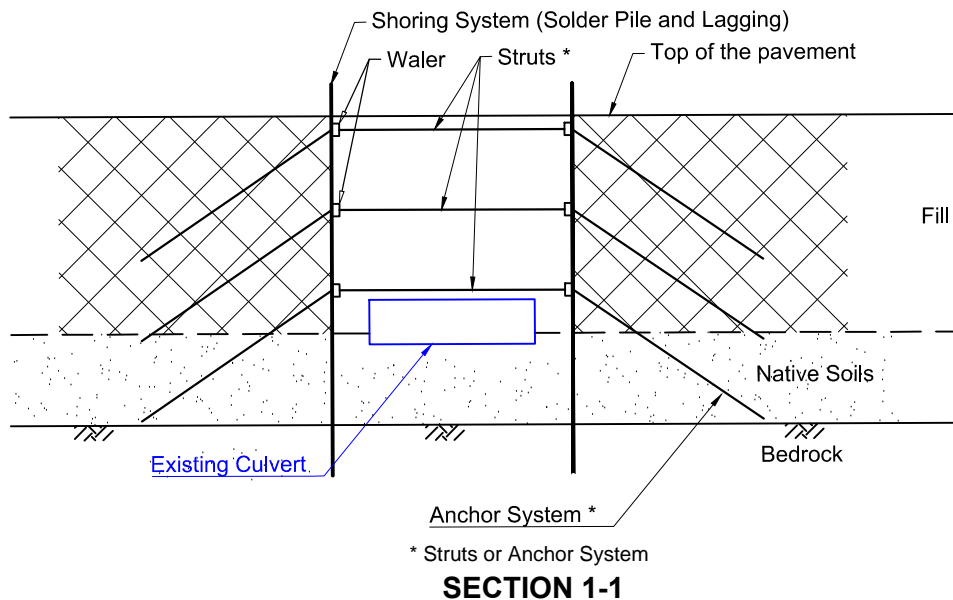
Half and Half Construction, Shoring system with either rakers or anchor system - Unsupported Excavation



**FIGURE H.3.B: HALF AND HALF CONSTRUCTION WITH BRACED CUT SIDES
OR ANCHOR SYSTEM OPTION 3.B
SCHEMATIC DIAGRAMS (NTS)**



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



Appendix I – Operational Constrains and Non Standard Special Provisions

NSSP FOR MASS CONCRETE ON BEDROCK

Scope of Work

The scope of work for the above noted tender item includes the mass concrete under the strip footings for the Flood Creek East Timber Culvert Replacement.

The Contractor should be aware that there is sloping bedrock in the area and fractured bedrock was encountered within the upper 1 m of the bedrock surface. Mass concrete volumes will vary depending on the variable intact bedrock surface.

Construction

Concrete shall be of the same strength as the footing concrete and placed in accordance with OPSS.PROV 904 "CONSTRUCTION SPECIFICATION FOR CONCRETE STRUCTURES".

Basis of Payment

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

NSSP FOR SLOPING ROCK AND COBBLES AND ROCK PIECE OBSTRUCTIONS

Scope of Work

The Contractor should be aware that there is sloping bedrock in the area and fractured bedrock was encountered within the upper 1 m of the bedrock surface. The overburden soils at the site consist of gravelly sand and silty sand fill materials which may contain cobbles and rock fragments especially near the bedrock interface.

Appropriate equipment and procedures will be required to penetrate/remove cobbles and fractured bedrock that are encountered during excavation.

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 DOWELLING

Scope of Work

Work under this item is for the placement and field testing of dowels into rock.

Materials and Installation

Dowels into rock shall be constructed in accordance with OPSS.PROV 904 "CONSTRUCTION SPECIFICATION FOR CONCRETE STRUCTURES". All reinforcing steel supplied shall be in accordance with OPSS.PROV 1440 "MATERIAL SPECIFICATION FOR STEEL REINFORCEMENT FOR CONCRETE" (dowel bars conforming to CSA Standard CSAG30.18, Grade 400).

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

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

Dowel Testing

All proposed testing procedures shall be in general conformance with ASTM D 3689-90 and ASTM D 114381 (Re-approved 1994). Field testing must be carried out in the presence of, and the results reviewed and approved by, the Contract Administrator.

Performance Tests

Performance tests shall be carried out on two rock dowels to confirm that the design load of the rock can be achieved. Performance test shall be by axial tensioning using a hydraulic jack with a capacity of at least 1.5 times the ultimate strength of the dowels.

Rock dowels shall be loaded and unloaded in 3 cycles and measurements of the dowel displacement shall be carried out at each load step in accordance with the following schedule:

Cycle-Step	1-1	1-2	1-3	2-1	2-2	2-3	2-4	3-1	3-2
% of Design Load	50	75	25	50	75	100	25	50	75

Cycle-Step	3-3	3-4	3-5
% of Design Load	100	110	25

Displacement measurements shall be carried out at each load step using displacement gauges with precision of 0.0025 cm. Measurements shall be referenced to an independent fixed referenced pint.

Rock dowels which do not meet the acceptance criteria shall be replaced at the Contractor's expense and re-tested. If a rock dowel fails, three additional rock dowels shall be tested at or near the same footing location as directed by the Contract Administrator.

Acceptance criteria for the rock dowels will be in accordance with the Post-testing Institute (1985) as follows:

The dowels are acceptable if the total elastic movement is greater than 80% of the theoretical elastic elongation of the free stressing and is less than the theoretical elongation of the free stressing length plus 50% of the bond length.

Basis of Payment

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

NSSP FOR CONDITION SURVEYS AND MONITORING DURING ANY BLASTING

Scope of Work

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

Instrumentation or monitoring ground and air vibration effects from the blasting should be set up in accordance with the International Society of Explosives Engineering field practice guidelines (1999).

Basis of Payment

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