FINAL REPORT

FOUNDATION INVESTIGATION AND DESIGN REPORT

Caribus Lake Tributary Timber Culvert Replacement, Highway 11, Site No. 45-269/C, District of Rainy River

Agreement No. 6014-E-0017
Assignment No. 6
GWP 6320-14-00
Geocres No. 52B-024

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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 Caribus Lake Tributary Timber Culvert, located on Highway 11, about 3.1 km west of the junction of Hwy 11 and Hwy 11B, 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 6320-14-00). 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 20 m and a height of about 1.8 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 Caribus Lake Tributary Timber Culvert is located on Highway 11, about 3.1 km west of the junction of Hwy 11 and Hwy 11B, in the District of Rainy River, south of Atikokan, Ontario. At the site, Hwy 11 is a two lane roadway, with a speed limit of 90 km/h and is about 7.1 m wide from edge of pavement to edge of pavement, with sand and gravel shoulders about 2 m wide. Based on drawings provided, the roadway embankment is about 3.5 m high with side slopes of about 2H:1V.

During the fieldwork on June 19 to 21, and 26 to 28, 2015, the general site conditions were assessed. Hwy 11 runs in an east to west direction and Caribus Creek, flows from north to south beneath the highway, ultimately towards Steep Rock Lake which is about 5 km north of the site (note that Caribus Creek flows north to south beneath the highway, then west and then north). At the time of this investigation, the approximate creek elevations at the inlet and outlet were about 427.39 m and 427.38 m, respectively. The elevation of highway pavement centerline at the culvert centerline is about 430.5 m. Overhead wires were observed along the north side of the highway.

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. 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 consists of peat organic terrain with a subordinate landform consisting of bedrock plain; mainly low local relief, plain, wet 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 Mesoarchean Era (2.5 to 3.4 Ga), Supracrustal rocks, and generally consist of metasedimentary rocks. The metasedimentary rocks include wacke, arkose, argillite, slate, marble, chert, iron formation, and minor metavolcanic rock complexes.

1.3 Investigation Procedures

1.3.1 Site Investigation and Field Testing

The field investigation was performed on June 19 to 21, and 26 to 28, 2015. The field program consisted of drilling four (4) sampled boreholes (BH301 to BH304). Two (2) boreholes were located within the highway, BH301, and BH302. BH301 was located about 5 m west of the culvert centerline and about 3 m north of the highway centerline. BH302 was located about 5 m east of the culvert centerline and about 1.2 m south of the highway centerline. An additional two (2) boreholes (BH303 and BH304) were advanced off of the highway. BH303 was located about 5.5 m west of the culvert centerline and about 13 m north of the highway centerline (inlet/upstream side). BH304 was located about 3.2 m west of the culvert centerline and about 15 m south of the highway centerline (outlet/downstream side). The borehole locations are shown on Drawing 1 in Appendix B.

All the boreholes (BH301 to BH304) were advanced using a CME 850 track mounted drill rig. 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 BH301 and BH302 were advanced to depths of about 8.5 m, 8.3 m below ground surface, respectively. The off-road boreholes BH303 and BH304 were advanced to auger and SPT refusal, at depths of about 2.3 m and 3.5 m below ground surface, respectively. The off-road boreholes were terminated at the refusal depths.

At BH301, initial refusal to SPT was encountered at about 3.4 m depth; however, using augering techniques, the borehole was advanced beyond the SPT refusal. At BH302, SPT and auger refusal were encountered at about 2.7 m depth, and rock coring techniques were used to advance the borehole. Rock coring techniques at BH302 were continued through additional overburden soils and into the bedrock. At BH301, rock coring techniques were initiated at about 5.4 m depth 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 benchmark (BM) provided (regular iron bar [RIB] in rock) east of the site and south of the highway. The BM elevation is 431.009 m. The location of the BM is shown on Drawing 1, in Appendix B.

During the drilling of the boreholes (BH301 to BH304), 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. In addition, samples were collected from the auger flighting in the upper 0.3 m at BH301 and BH302.

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. Atterberg Limits tests were carried out on select cohesive 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 was 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 fill material composed of silty gravel with sand, and cobbles and boulders. In general, the fill was overlying peat, overlying clayey silt and overlying bedrock. A more detailed summary of the subsurface conditions encountered in the boreholes is provided in the following sections.

1.4.1 Silty Gravel with Sand Fill

Silty gravel with sand fill was encountered beneath the asphalt at BH301 and BH302. The asphalt thickness at BH301 and BH302 was about 75 mm and 60 mm, respectively. The silty gravel with sand fill was generally described as very dense to compact at depth, brown, damp to moist, containing occasional cobbles. Trace asphalt was noted in the upper 0.3 m at BH301. The SPT "N" values ranged between 13 and 100 (i.e. SPT refusal) blows per 300 mm penetration, with an average "N" value of about 47. The silty gravel with sand fill extended to depths ranging between about 2.3 m (428.2 m elevation) and 3.8 m (426.6 m elevation) below ground surface.

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

Moisture content:

- 4.0% to 12.4%

Grain size distribution:

- 38% to 45% gravel;
- 25% to 35% sand;
- 26% to <27% silt ; and
- 4% to <27% clay size.

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

1.4.2 Cobbles and Boulders Fill

Cobbles and boulders fill was encountered beneath the silty gravel with sand fill at BH302 and within the silty gravel with sand fill at BH301. The cobbles and boulders fill was generally described as compact to very dense, greenish grey, wet, weathered, fractured, and containing some sand and some silt. The SPT "N" values ranged between 10 and 100 (i.e. SPT refusal) blows per 300 mm penetration, with an average "N" value of about 46. The cobbles and boulders fill extended to depths of about 3.1 m (427.4 m elevation) and 3.8 m (426.7 m elevation) below ground surface.

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

Moisture content:

- 3.6% to 5.4%

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

1.4.3 Peat

Peat was generally encountered beneath the fill and surfacing the off-road boreholes. The peat was generally described as soft, dark brown, wet and containing trace roots to roolets. The SPT "N" values ranged between 0 (i.e. advanced by weight of hammer and rods alone) and 5 blows per 300 mm penetration, with an average "N" value of about 2. The peat thickness ranged between about 0.3 m and 2.1 m and extended to depths ranging between about 1.5 m and 4.1 m below ground surface. The peat extended to elevations ranging between about 426.2 m and 426.3 m.

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

Moisture content:

- 41.9% to 317.3%

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

1.4.4 Clayey Silt

Clayey silt was encountered underlying the peat. The clayey silt was generally described as firm to hard, brown to grey, moist to wet, and varved. Some gravel and some sand was encountered at depth at BH301. The SPT "N" values ranged between 3 and 22 blows per 300 mm penetration, with an average "N" value of about 9. Note that at each borehole where clayey silt was encountered (BH301, BH303 and BH304), SPT "N" values of 100 blows (i.e. SPT refusal) was encountered at the clayey silt termination depths and is not considered representative of the clayey silt. Two *in situ* field vane test were performed and the results at BH301 and BH304 were 116 kPa and >330 kPa, respectively. The clayey silt extended to depths ranging between about 2.3 m and about 5.4 m below ground surface, and elevations ranging between 424.4 m and 426.0 m.

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

Moisture content:

- 18.1% to 34.7%

Grain size distribution:

- 0% gravel;
- 2% to 3% sand;
- 70% to 75% silt; and
- 22% to 28% clay size.

Total saturated unit weights have been calculated based on the moisture contents and are estimated to range from about 18.4 to 21.0 kN/m^3. Two (2) Atterberg Limits tests were performed on representative samples of the clayey silt (BH301-S9B and BH304-S4). The results indicated that the soil is of low to medium plasticity. The data is shown on the plasticity chart, Figure 4. The liquid limit, plastic limit and plasticity index ranged between about 29 and 32, 19 and 20, and 9 and 13 respectively.

The results of the moisture content, grain size distribution and Atterberg Limits tests are provided on the record of borehole sheets in Appendix C. The results of the grain size distribution are also provided on Figure 3 in Appendix D, and Atterberg Limits tests are provided on Figure 4 in Appendix D.

1.4.5 **Silty Sand to Gravel and Cobbles**

At BH302 only, silty sand to gravel and cobbles was encountered beneath the fill. The silty sand was described as very dense, grey, and wet. One SPT sampling test was conducted the "N" value was 100 (i.e. SPT refusal) blows per 300 mm penetration. The silty sand extended to about 4.1 m below ground surface (elevation 426.4 m).

Gravel and cobbles were encountered underlying the silty sand at BH302. The gravel and cobbles were described as very dense and grey. No SPT sampling was conducted. The gravel and cobbles extended to about 5.3 m below ground surface (elevation 425.2 m).

Laboratory testing performed on selected samples consisted of moisture content, grain size distribution and Atterberg Limit tests. No laboratory testing was performed on the gravel and cobbles. The test results are as follows:

Moisture content:

- 12.7%

Grain size distribution:

- 0% gravel;
- 54% sand;
- 43% silt; and

- 3% clay size.

Total saturated unit weight has been calculated based on the moisture contents and is estimated to be about 22.3 kN/m^3. One (1) Atterberg Limits tests was performed on representative sample of the silty sand (BH302-S7), as some cohesive properties were noted. The results indicated that the soil is of low plasticity and the soil contained more cohesionless properties than cohesive properties. The data is shown on the plasticity chart, Figure 5. The liquid limit, plastic limit and plasticity results were 19, 12 and 7, respectively.

The results of the moisture content, grain size distribution and Atterberg Limits 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, and Atterberg Limits tests are provided on Figure 5 in Appendix D.

1.4.6 **Bedrock**

Bedrock was encountered underlying the clayey silt at BH301, and beneath the cobbles and boulders at BH302, at depths of about 5.4 m (425.0 m elevation) and 5.3 m (425.2 m elevation), respectively. The bedrock was generally described as a medium strong (25 MPa to 50 MPa compressive strength), fractured to very sound, green to grey, and fine grained. The boreholes were extended by rock coring about 3.0 m to 3.1 m into bedrock, and to depths ranging about 8.3 m and 8.5 m below ground surface. The boreholes were terminated at elevations ranging between about 422.0 m and 422.2 m. Photographs of the bedrock core samples are presented in Appendix C, after the Borehole Logs.

Gross recoveries ranged between about 93% and 100%. The Rock Quality Designation (RQD), which is a modified core recovery, ranged from 53% 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. 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
BH301	Jun. 20/15	Jun. 20/15	430.4	3.62	426.78
BH302	Jun. 21/15	Jun. 21/15	430.5	2.86	427.64
BH303	Jun. 27/15	Jun. 28/15	428.3	0.69	427.61

Borehole	Date Completed	Date Measured	Ground Surface Elevation[2]	Depth to Water[3]	Groundwater Elevation
BH304	Jun. 26/15	Jun. 27/15	427.8	0.25	427.55
Caribus Creek WL Upstream (North) Side	--	Jun. 27/15			427.39[4]
Caribus Creek WL Downstream (South) Side	--	Jun. 27/15	--	--	427.38[4]

Notes:
1) All units in metres.
2) Elevations surveyed are referenced to a geodetic benchmark (BM) provided (regular iron bar [RIB] in rock) east of the site and south of the highway. The BM elevation is 431.009 m.
3) Depths are relative to ground surface.
4) Indicates top of surface water elevation at Caribus Creek.

1.6 Chemical Analyses

Two soil sample 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

Sample	pH (unitless)	Chloride (ppm)	Soluble Sulphate (ppm)	Resistivity (ohm-cm)	Conductivity (µS/cm)
BH301-S9B/S10/S11	6.54	220	30	2,300	435
BH304-S3	6.72	36	<20	7,000	143

Note:
1) Due to insufficient sample volume, samples S9B, S10 and S11 from BH301 were combined for chemical analyses.

Part II: ENGINEERING DISCUSSIONS AND RECOMMENDATIONS

2.1 General

This section of the report provides geotechnical design recommendations for replacement of the existing Caribus Lake Tributary Timber Culvert, located on Highway 11, about 3.1 km west of the junction of Hwy 11 and Hwy 11B, 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 culvert and replacement. Comments on construction are only provided to highlight issues that could affect the design. Contractors bidding on the works should make their own assessments of the factual data and how it might affect construction means and methods, scheduling and the like.

Based on information included in the TOR, it is understood that 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 20 m and a height of about 1.8 m. It is also understood that the existing culvert construction date was unknown and inspected in July 2013. The inspection found that timbers in soffit and walls of the culvert barrel were medium decay as well as those at inlet and outlet. Weathering and shakes of the timbers at inlet and outlet ware also noted. The same inspection also noted that tops of mid wall and west wall lean 60 mm and 90 mm, respectively, west on the inlet 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), corrugated steel plate culvert, and steel pile and steel sheet pile abutment with 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. Hwy 11 is a two lane roadway, with a speed limit of 90 km/h and is about 7.1 m wide from edge of pavement to edge of pavement, with sand and gravel shoulders about 2 m wide. Based on drawings provided, the roadway embankment is about 3.5 m high with side slopes of about 2H:1V. The current elevation of the crest of the roadway is about 430.5 m.

b. The highway embankment consists of very dense to compact silty gravel with sand fill (~2.2 m thick) underlain by very dense to compact cobbles and boulders fill (~0.8 m to 1.5 m thick).
c. Toward the inlet (BH301), the embankment fill is underlain by soft peat to about 4.1 m (~0.3 m thick) below ground surface, followed by soft to very stiff clayey silt to about 5.4 m (~1.3 m thick) below ground surface and followed by bedrock at Elev. 425.0 m. In BH302 toward the outlet, the embankment fill is underlain by very dense silty sand to about 4.1 m (~0.3 m thick) below ground surface, followed by very dense gravel and cobbles to about 5.3 m (~1.2 m thick) below ground surface and followed by bedrock at Elev. 425.2 m.
d. At the inlet and outlet a surficial layer of soft peat (~2.1 m thick at inlet and 1.5 m thick at outlet) is underlain by soft to hard clayey silt.
e. The foundation soil at the invert of the new culvert is anticipated to be native very stiff to hard clayey silt between Elev. 425.7 m to Elev. 426.2 m. Typical 'N' values ranged from 22 to 100.
f. The groundwater table in the embankment fill is expected to be at approximate elevation 427.4 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 Structure Foundations

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

- Rigid frame box culvert,
- Rigid frame open footing culvert,
- Steel pile and steel sheet pile abutment with precast concrete decking

Based on the subsurface information obtained from the site investigation, the native very stiff to hard clayey silt between Elev. 425.7 m to Elev. 426.2 m is considered suitable for support of all replacement 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 relatively at shallow depth, steel pile and steel sheet pile abutment with precast concrete decking is not a feasible option.

The layer of soft peat encountered below the existing embankment should be excavated and removed to firm bearing of native clayey silt and grade restored with engineered fill. Since the depth of excavation to remove peat and/or other unstable soils could be excessive (approximately between 0.6 m and 1 m below the culvert invert), 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 for the new culvert. 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	▪ Straightforward construction ▪ Reduce construction period, consequently traffic management and water control period ▪ Reduce excavation depth	▪ Dewatering system required ▪ Require heavy lifting equipment	▪ Low to medium	▪ 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	▪ Wider span may consider to maintain existing channel ▪ High geotechnical resistance available ▪ Can incorporate dowels to enhance lateral resistance	▪ Deeper excavation or below water excavation may required ▪ Dewatering system required ▪ Possible uneven bedrock surface ▪ Require placement of lean concrete	▪ Likely more expensive than Option 1	▪ 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 all 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	~426.6 or below	4.0	Minimum 300 mm compacted granular material (Granular A or Granular B Type II) over native very stiff to hard clayey silt	600	400
Rigid frame open footing concrete culvert	~424.5* or bedrock level	1.0	Bedrock	900	600

Notes:
**Below the frost line, based on BH 304. Where the footing founded on bedrock requirement of full frost protection is not applicable. It is recommended to placed minimum 100 mm thick layer of compacted Granular A beneath footings.*
*** for maximum settlement of 25 mm*

It is presumed that underlying peat and any other soft or very loose materials are to be replaced with clean and compactable soil such as Granular A or Granular B Type II. Given that no 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 δ)=0.5
Between cast-in-place concrete and bedrock	Coefficient of friction (tan δ)=0.7

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 summarizes 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	• Existing 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	• 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 water flow • 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	• Traffic flow maintained at the site during construction • Simple detour roads can be constructed • Existing culvert will be completely remove and replaced with new culvert • No excavation support required • Install entire new culvert at once	• Traffic interruption • Construction of detour embankments required at each 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 lake water • Possible settlement due to new earth embankment fill • Risk of cost overrun and inability to finish job: low to moderate • Possible extra cost to purchase of private property	More expensive than full road closure due to high costs to build local detours	4

Installation Method	Advantages	Disadvantages	Relative Cost	Ranking
OPTION 3.A Half-and-half Construction with Unsupported Cut Sides	• Traffic flow maintained at the site during construction • Short mobilization time • Straight forward construction and construction procedures	• Traffic interruption • Roadway protection of up to 3.5 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 lake 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, but less traffic interruption than full road closer	1
OPTION 3.B Half-and- half Construction with Braced or Anchored Cut Sides	• 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	• Traffic interruption • Roadway protection of up to 3.5 m high required to maintain one lane of traffic if steel docking 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 lake 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 3.5 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 solder 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 form 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 with a 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 lake 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 (i.e. cofferdam) 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 form 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 cobbles and boulders layer and 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 40 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/m^3

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 ø'	Coefficient of Active Earth Pressure (K_a)	Coefficient of Passive Earth Pressure (K_p)	Coefficient of Earth Pressure at Rest (K_o)	Unit Weight γ kN/m^3)
Silty Gravel with Sand Fill	35°	0.27	3.67	0.43	20
Cobbles and Boulders Fill	40°	0.22	4.56	0.36	21
Clayey Silt	29°	0.35	2.88	0.52	19.5
Gravel and Cobbles	38°	0.24	4.20	0.38	20
Peat	17°	0.55	1.83	0.71	15

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 2.3 m 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 clayey silt. This material has high frost susceptibility based upon the MTO Frost Classification guideline of percent particles between 5 to 75 μm. 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

The selection and placing of the backfill should be in accordance with OPSD-803.010 for concrete culverts. The backfill should consist of free-draining, non-frost susceptible granular materials such as Granular A or Granular B Type II (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.

Proper frost treatment is required in accordance with OPSD-803.030 or 803.031, whichever is applicable.

Backfilling behind any retaining (wing) walls, should consist of granular materials in accordance with the MTO standards. Free draining backfill materials, weepholes or perforated drains, etc. should be provided in order to prevent hydrostatic pressure build-up.

2.8 Surface Water and Groundwater 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 soft peat (~2.1 m thick at inlet and 1.5 m thick at outlet) underlain by soft to hard clayey silt. The bedrock surface is encountered at 2.3 m depth at the inlet side and 3.5 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 clayey silt and silty sand to gravel and cobbles. The clayey silt is 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.

Dewatering requirements behind the cofferdams to keep the construction site dry will be impacted by water levels in the lake 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 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 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 clayey silt and silty sand to gravel and cobbles 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 north and south side slopes are similar, the result of the slope analysis performed for the east side slope, is only presented.

Tabulated below in Table 2.6 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 2.6 Soil properties used in slope stability analysis

Material	ϕ' (degrees)	c' (kPa)	γ' (kN/m^3)
Silty Gravel with Sand Fill	35	0	20
Cobbles and Boulders Fill	40	0	21
Clayey Silt (Very Stiff)	29	0	19.5
Peat (Soft)	17	0	15
Engineered Fill	32	0	21

The results of slope stability analyses are attached as Appendix F. The results of stability analyses on the existing embankment shown on Figure F1 suggest that the existing embankment could be on the verge of stability due to presence of the soft peat layer below the embankment. To achieve the factor of safety greater than 1.3 for the global stability of the new embankment with 2H:1V slopes, the soft peat layer has to be excavated and replaced by engineered fill, as confirmed by the analyses which results are shown in Figure F2.

2.10 Inlet and Outlet

2.10.1 Erosion Protection at Outlet

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

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 G). OPSS. PROV 1205 specifies that material used for clay seals shell be natural clay, clay mixture (1 part Bentonite powder and 3.5 parts Granular “A”) or a geosynthetic clay liner (GCL). The coefficient of permeability shall not exceed 1 x 10^{-6} cm/s.

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

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

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

2.10.3.2 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 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 220 ppm (µg/g) i.e. 0.022% which indicates a low potential for corrosion.

The water soluble sulphate content of the soils tested is tested ranges between <20 and 30 ppm (µg/g), i.e. about 0.003% 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, MASc. 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 north side of highway

Photo 2. Existing culvert outlet on south side of highway

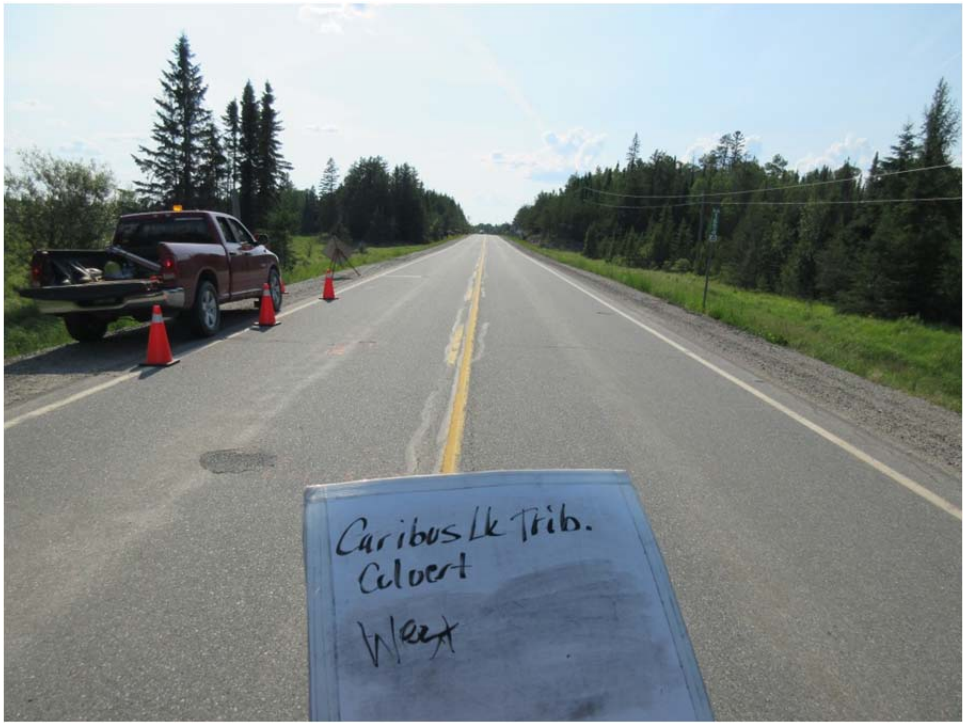


Photo 3. Facing west on Highway 11 before the existing culvert

Photo 4. Facing east 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

Agreement No. 6014-E-0017
Assignment No. 6
GWP 6320-14-00

CARIBUS LAKE CULVERT

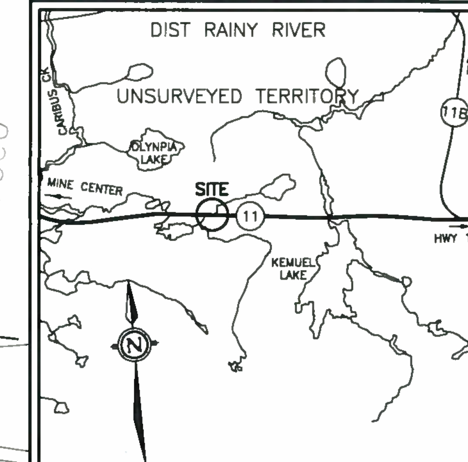
(Hwy 11, Rainy River District, Atikokan, ON)

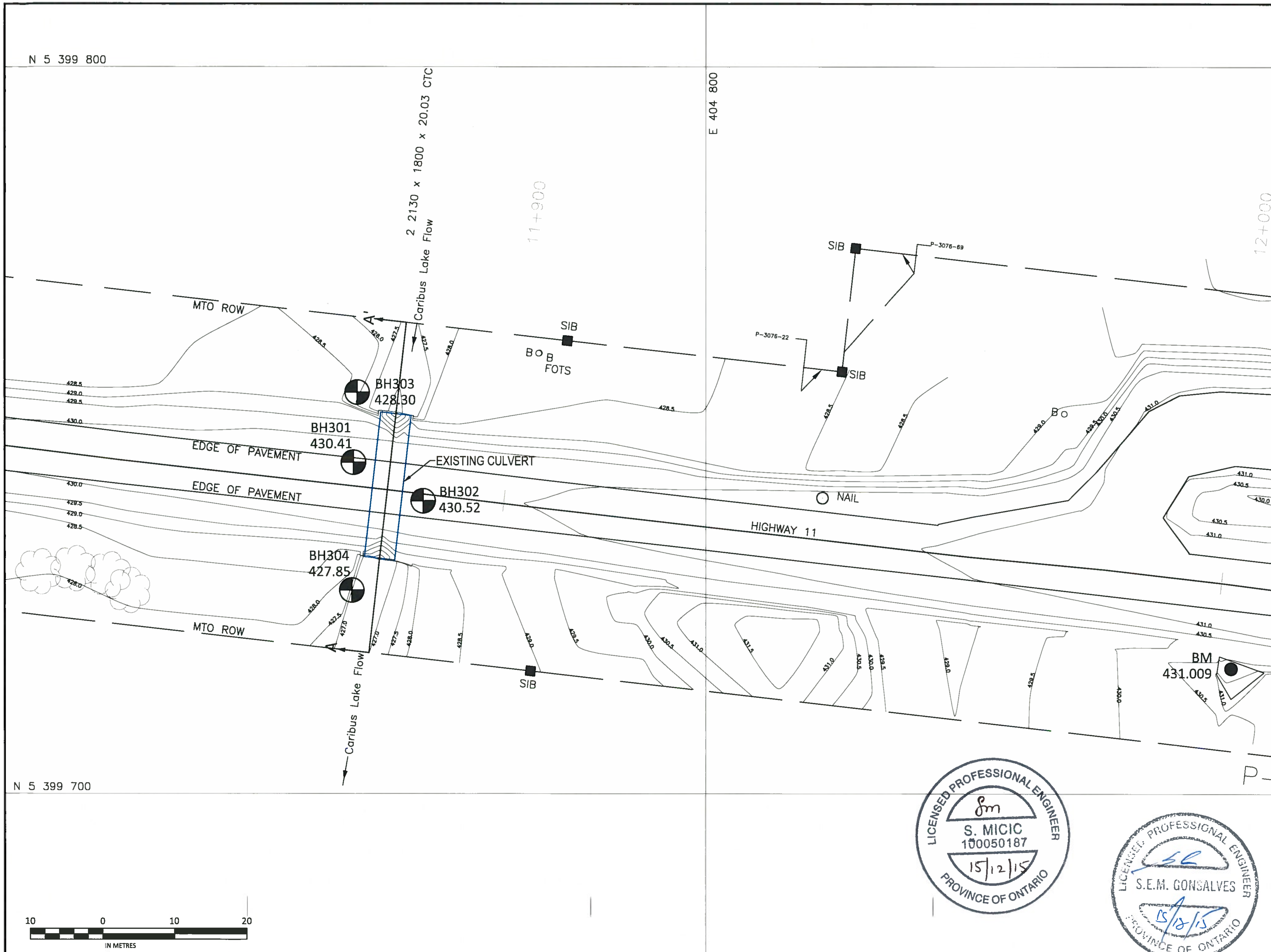
PLAN

DWG 1

exp. **exp** Services Inc.

KEY PLAN





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LEGEND

(borehole symbol)	BH301 430.41	BOREHOLE LOCATION GROUND SURFACE ELEVATION IN METRES
(benchmark symbol)	BM 431.009	BENCHMARK LOCATION GEODETIC ELEVATION IN METRES

BH No.	APPROX. ELEV. (m)	MTM COORDINATES NORTH	MTM COORDINATES EAST
BH301	430.41	5,399,746	404,751
BH302	430.52	5,399,740	404,761
BH303	428.30	5,399,755	404,752
BH304	427.85	5,399,728	404,751

NOTES

1. ALL DIMENSIONS ARE IN METRES.
2. BASE MAP PROVIDED BY CLIENT.
3. MTM COORDINATES BASE ON MTM ZONE ON-16 PROJECTION, AS PER PROVIDED FIGURE.
4. THIS DRAWING IS FOR SUBSURFACE INFORMATION ONLY. THE PROPOSED STRUCTURE DETAILS/WORKS ARE SHOWN FOR ILLUSTRATION PURPOSES ONLY.

REVISIONS

DATE	BY	DESCRIPTION

GEOCRES No. 52B-024	Project No. ADM-00223648-E0
Date: December 8, 2015	Scale : 1:500
Drawn By: RM	Checked By: AM
	Checked By: DG

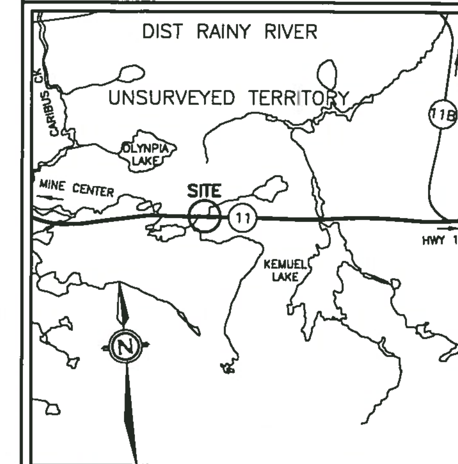
Agreement No. 6014-E-0017
Assignment No. 6
GWP 6320-14-00

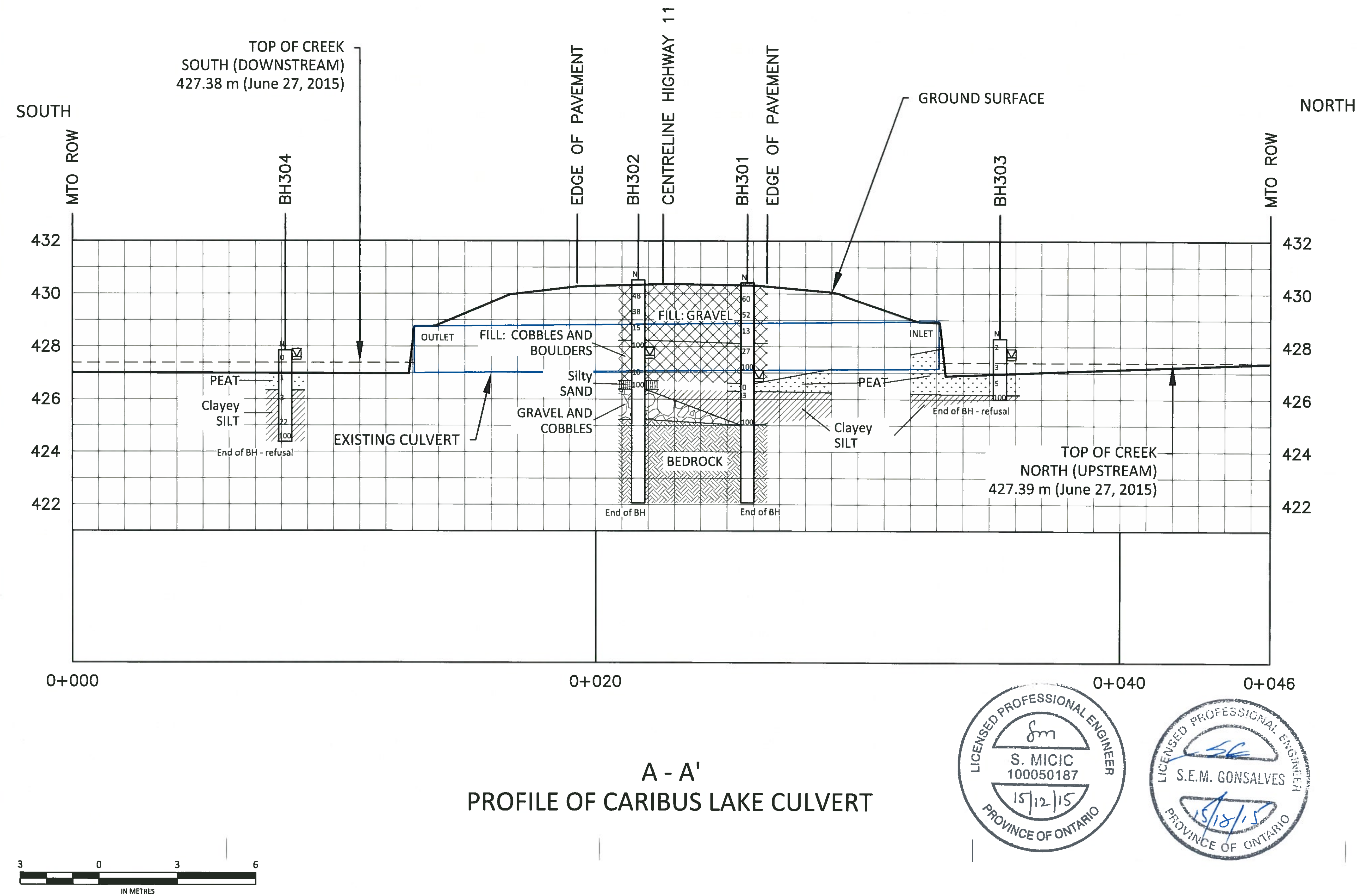
CARIBUS LAKE CULVERT
(Hwy 11, Rainy River District, Atikokan, ON)
SECTION A-A'

DWG 2

exp. **exp** Services Inc.

KEY PLAN





A - A'
PROFILE OF CARIBUS LAKE CULVERT

LEGEND

N STANDARD PENETRATION TEST (BLOWS/0.3 m)

MEASURED WATER LEVEL

BH No.	APPROX. ELEV. (m)	MTM COORDINATES NORTH	MTM COORDINATES EAST
BH301	430.41	5,399,746	404,751
BH302	430.52	5,399,740	404,761
BH303	428.30	5,399,755	404,752
BH304	427.85	5,399,728	404,751

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REVISIONS

DATE	BY	DESCRIPTION

GEOCRES No. 52B-024	Project No. ADM-00223648-E0
Date: December 8, 2015	Horizontal Scale : 1:150
Drawn By: RM	Vertical Scale : 1:150
Checked By: AM	Checked By: DG

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		

0.002 0.006 0.02 0.06 0.2 0.6 2.0 6.0 20 60 200

EQUIVALENT GRAIN DIAMETER IN MILLIMETRES

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

UNIFIED SOIL CLASSIFICATION

Terminology describing materials outside the USCS, (e.g. particles larger than 76 mm, visible organic matter, construction debris) is based upon the proportion of these materials present and as described below in accordance with 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≤Pp≤10%
Little	15≤Pp≤25%
Some	30≤Pp≤45%
Mostly	50≤Pp≤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≤N<10
Compact	10≤N<30
Dense	30≤N<50
Very Dense	50≤N

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

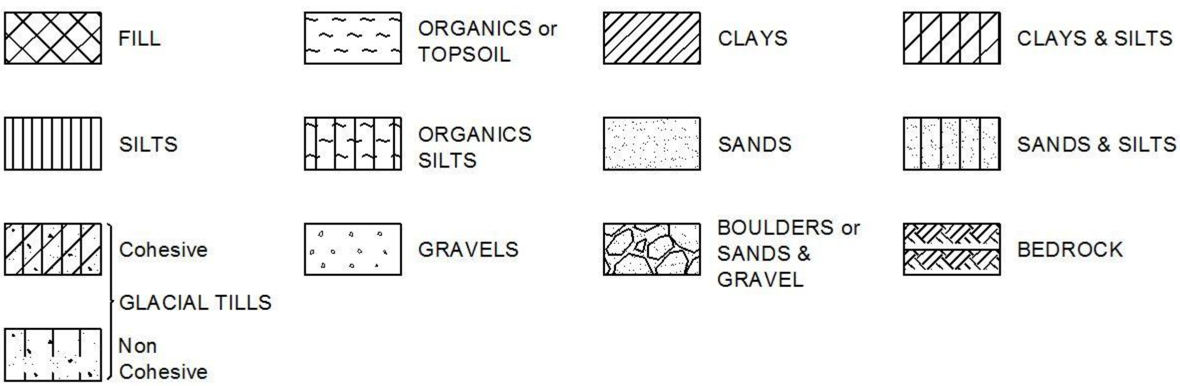
Table c: Consistency of Cohesive Soil

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

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

STRATA PLOT

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



WATER LEVEL MEASUREMENT

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

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

STRESS AND STRAIN

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

MECHANICALL PROPERIES OF SOIL

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

PHYSICAL PROPERTIES OF SOIL

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

RECORD OF BOREHOLE No BH301

1 OF 1 **METRIC**

W.P. GWP No. 6320-14-00 LOCATION **Caribus Lake Culvert (Site No. 45-269/C) MTM ON-16 5,399,746N 404,751E** ORIGINATED BY EF

DIST 61 HWY Hwy 11 BOREHOLE TYPE CME 850 Track Carrier / HSA / HQ COMPILED BY RM

DATUM **Geodetic** DATE 6.20.15 - 6.20.15 CHECKED BY AM/DG

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
430.4	Asphalt					
430.0 0.1	**ASPHALT** - about 75 mm	S1	AUGER			
	Silty GRAVEL with Sand (FILL) - very dense to compact, brown, damp to moist, occasional cobbles, trace asphalt in upper 0.3 m	S2	AUGER			
		S3	SS	60		
		S4	SS	52		
	- becoming moist to wet at about 1.5 m depth	S5	SS	13		45 25 26 4
428.1 2.3	**COBBLES AND BOULDERS (FILL)** - compact, greenish grey, wet, weathered, fractured	S6	SS	27		
427.4 3.1	**Silty GRAVEL with Sand (FILL)** - very dense, brown, wet, occasional cobbles	S7	SS	100		
	- cobbles and boulders at 3.4 m depth	S8	AUGER		∇	
426.6 3.8	**PEAT** - soft, dark brown, wet, some silt to silty	S9A	SS	0		
426.3 4.1	**Clayey SILT** - firm to very stiff, grey, wet, varved	S9B	SS	3		0 3 75 22
		S10	VANE			
425.0 5.4	- some gravel, some sand at about 5.3 m depth	S11	SS	100		
	BEDROCK - medium strong, green to grey, fractured, fine grained	S12	CORE			Recovery=93%, RQD=53%
	- becoming very sound at about 6.9 m depth	S13	CORE			Recovery=100%, RQD=100%
422.0 8.5	**End of Borehole**					

Plot headings: DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³

ELEVATION SCALE: 430, 429, 428, 427, 426, 425, 424, 423, 422

145.02

2

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

RECORD OF BOREHOLE No BH302

METRIC

W.P. GWP No. 6320-14-00	LOCATION Caribus Lake Culvert (Site No. 45-269/C) MTM ON-16 5,399,740N 404,761E	ORIGINATED BY EF
DIST 61 HWY Hwy 11	BOREHOLE TYPE CME 850 Track Carrier / HSA / HQ	COMPILED BY RM
DATUM Geodetic	DATE 6.19.15 - 6.21.15	CHECKED BY AM/DG

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
430.5	Asphalt						
430.4 / 0.1	**ASPHALT** - about 60 mm						
	Silty GRAVEL with Sand (FILL) - dense to compact, brown, damp to moist, occasional cobbles	S1	AUGER				
		S2	SS	48			
		S3	SS	38		o	38 35 (27)
		S4	SS	15		o	
428.2 / 2.3	**COBBLES AND BOULDERS (FILL)** - very dense to compact, greenish grey, wet, some sand, some silt, weathered, fractured - refusal to auger at about 2.7 m depth	S5	SS	100		o	
		S6	CORE		∇		
			SS	10			No recovery
426.7 / 3.8	**Silty SAND** - very dense, grey, wet	S7	SS	100		o	0 54 43 3
426.4 / 4.1	**GRAVEL and COBBLES** - very dense, grey	S8	CORE				
425.2 / 5.3	**BEDROCK** - medium strong, green to grey, fractured, fine grained	S9	CORE				Recovery=100%, RQD=60%
		S10	CORE				Recovery=100%, RQD=57%
422.2 / 8.3	**End of Borehole**						

Elevation scale: 430, 429, 428, 427, 426, 425, 424, 423

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100
SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 20 40 60 80 100
PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60
UNIT WEIGHT γ kN/m³

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

RECORD OF BOREHOLE No BH303

METRIC

W.P.	GWP No. 6320-14-00	LOCATION	Caribus Lake Culvert (Site No. 45-269/C) MTM ON-16 5,399,755N 404,752E	ORIGINATED BY	EF
DIST	61 HWY Hwy 11	BOREHOLE TYPE	CME 850 Track Carrier / HSA	COMPILED BY	RM
DATUM	Geodetic	DATE	6.27.15 - 6.28.15	CHECKED BY	AM/DG

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)
428.3	Peat						
0.0	**PEAT** - soft, dark brown, wet, trace gravel, trace sand, trace roots and rootlets	S1	SS	2		428	~41
					▽		
		S2	SS	3			~50
						427	
		S3	SS	5			~63
426.2 / 2.1	**Clayey SILT** - very stiff, brown to grey, moist to wet	S4	SS	100			~23
426.0 / 2.3	**End of Borehole** - refusal to SPT and auger					426	

Column headings: SOIL PROFILE; SAMPLES; STRAT PLOT; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

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

RECORD OF BOREHOLE No BH304

METRIC

W.P. GWP No. 6320-14-00	LOCATION Caribus Lake Culvert (Site No. 45-269/C) MTM ON-16 5,399,728N 404,751E		ORIGINATED BY EF
DIST 61	HWY Hwy 11	BOREHOLE TYPE CME 850 Track Carrier / HSA	COMPILED BY RM
DATUM Geodetic	DATE 6.26.15 - 6.27.15		CHECKED BY AM/DG

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
427.8	Peat								
0.0	**PEAT** - soft, dark brown, wet, trace roots and rootlets	S1	SS	0	▽		307.8		
		S2	SS	1		427	317.3		
426.3 / 1.5	**Clayey SILT** - firm to hard, grey, wet	S3	SS	3		426			
			VANE						Field Vane > 330 kPa
		S4	SS	22		425			0 2 70 28
		S5	SS	100					
424.4 / 3.5	**End of Borehole** - refusal to SPT and auger								

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

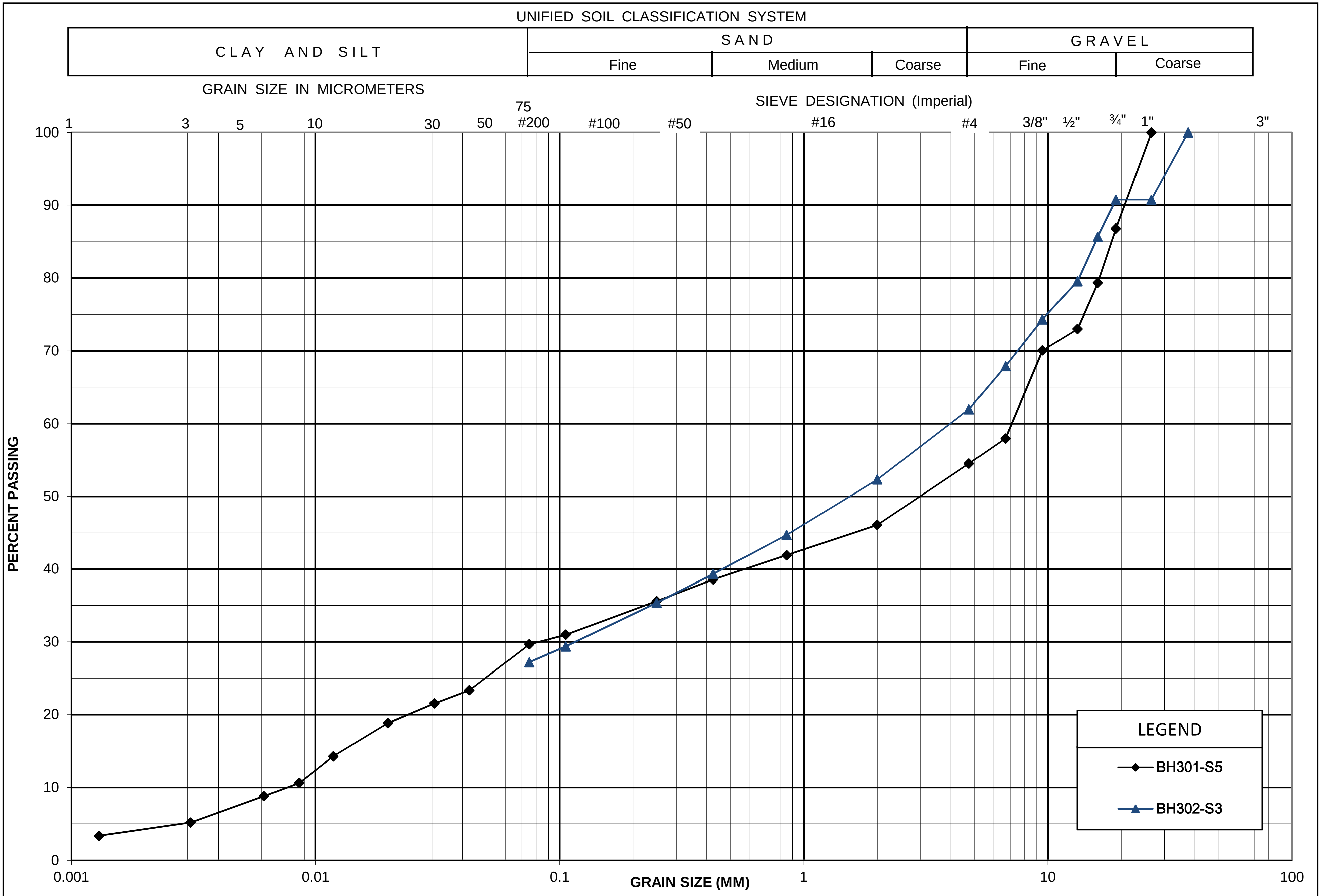
PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L

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

BH301 - Bedrock Core Samples with Depths and Elevations

BH302 - Bedrock Core Samples with Depths and Elevations

Appendix D – Laboratory Data



exp.

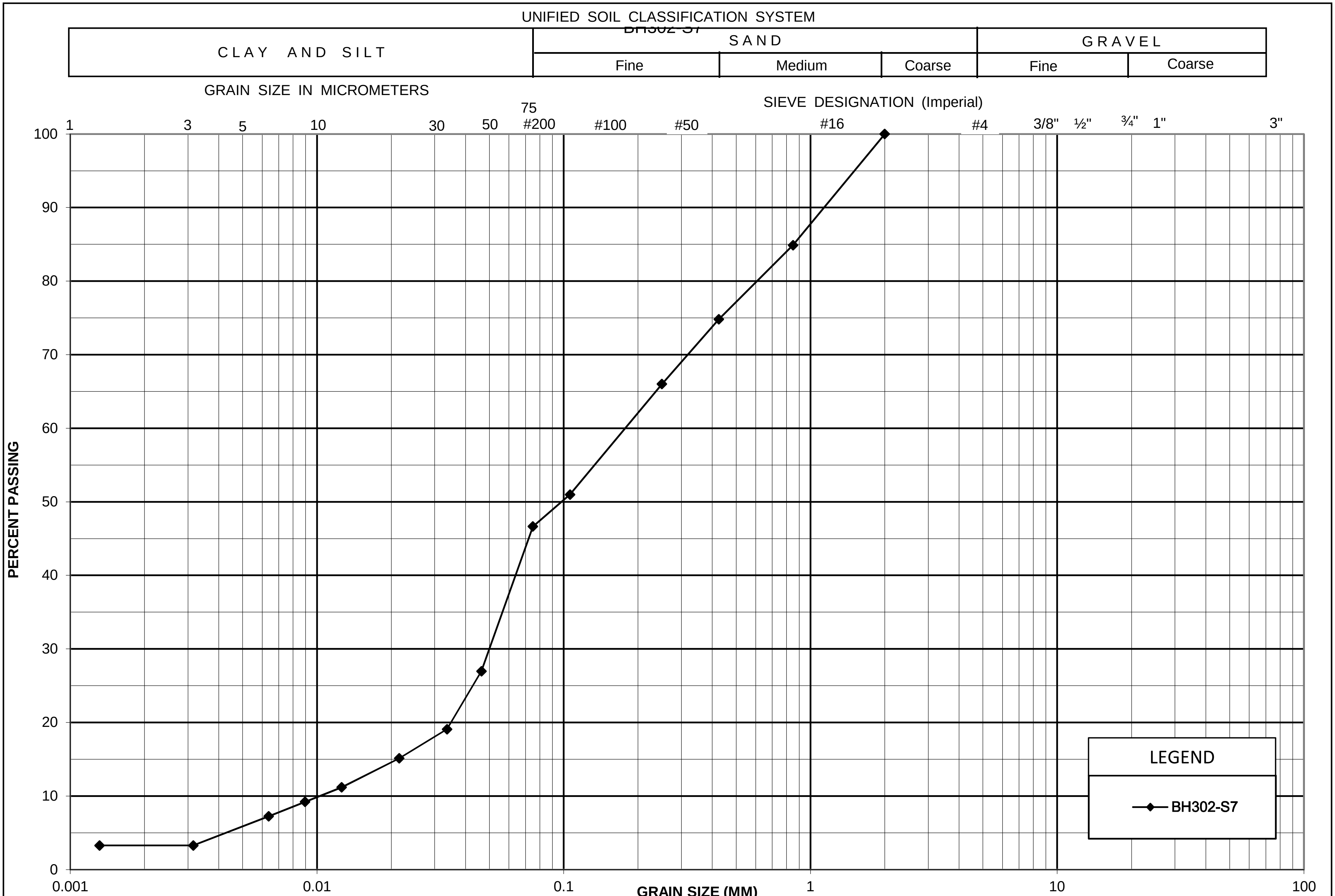
GRAIN SIZE DISTRIBUTION

Silty GRAVEL with Sand

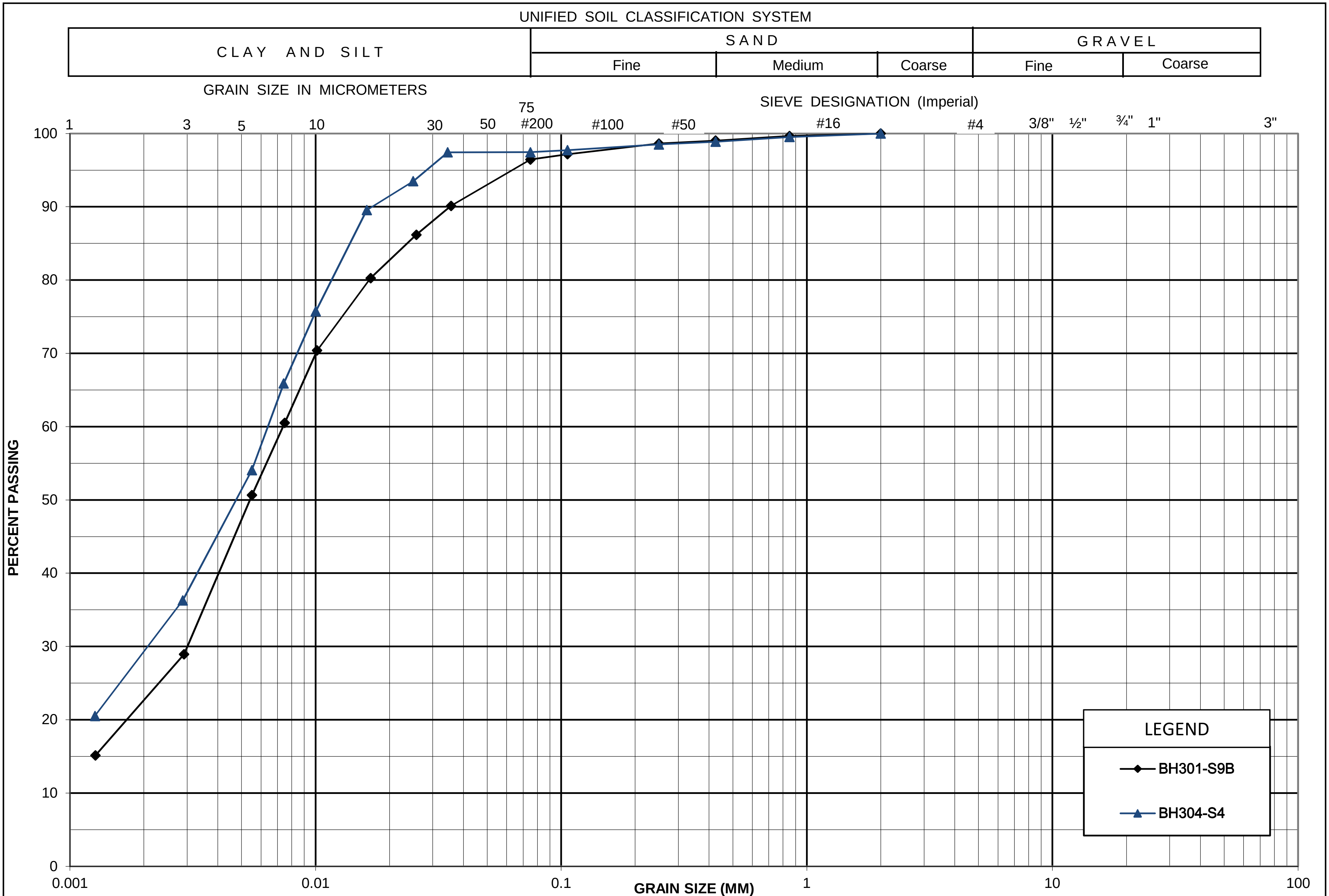
FIGURE: No. 1

GWP: 6320-14-00

DATE : July 16, 2015



exp.	GRAIN SIZE DISTRIBUTION ***Silty SAND***	FIGURE: No. 2 GWP: 6320-14-00 DATE : July 16, 2015



GRAIN SIZE DISTRIBUTION

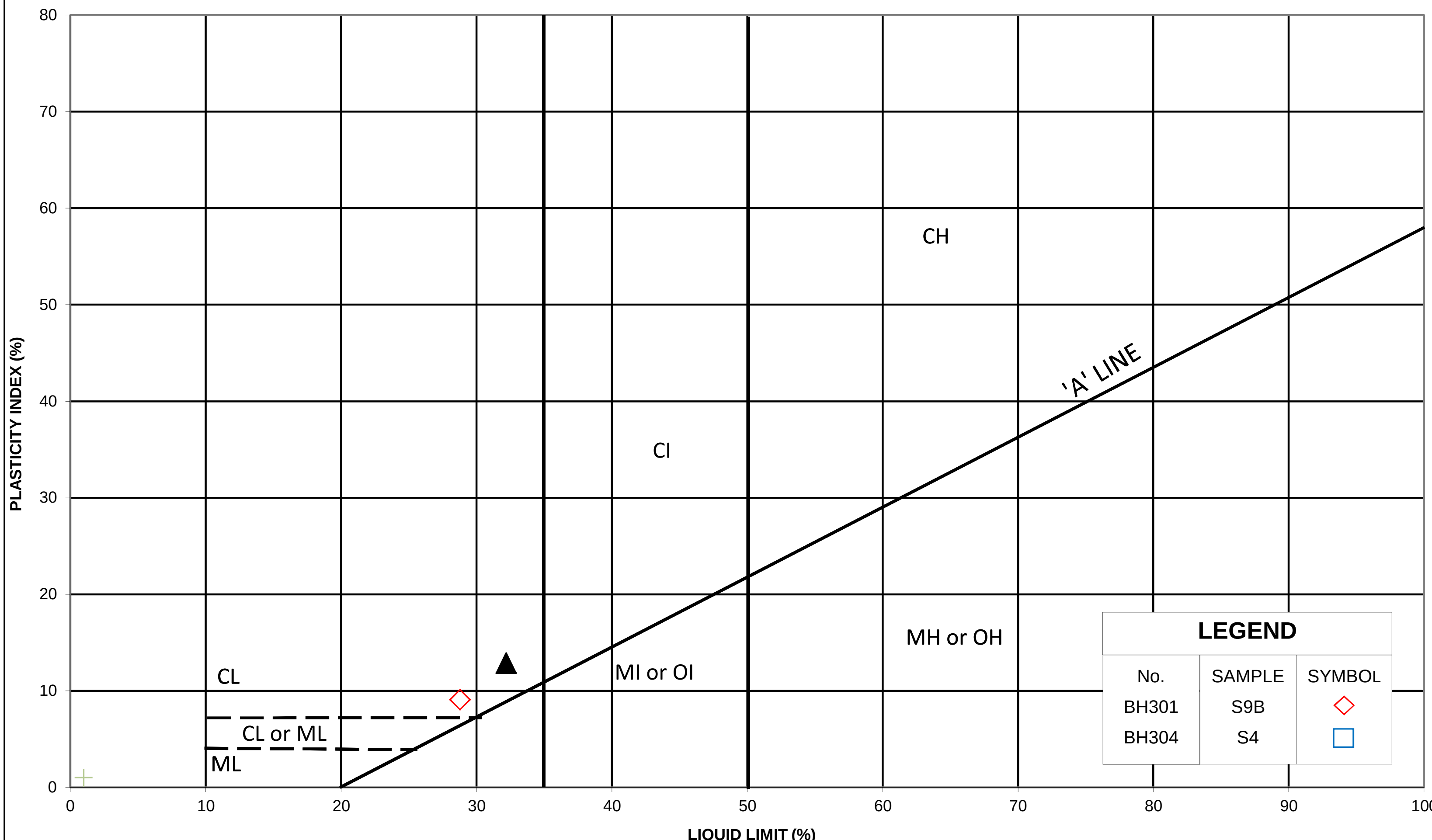
Clayey SILT

FIGURE: No. 3

GWP: 6320-14-00

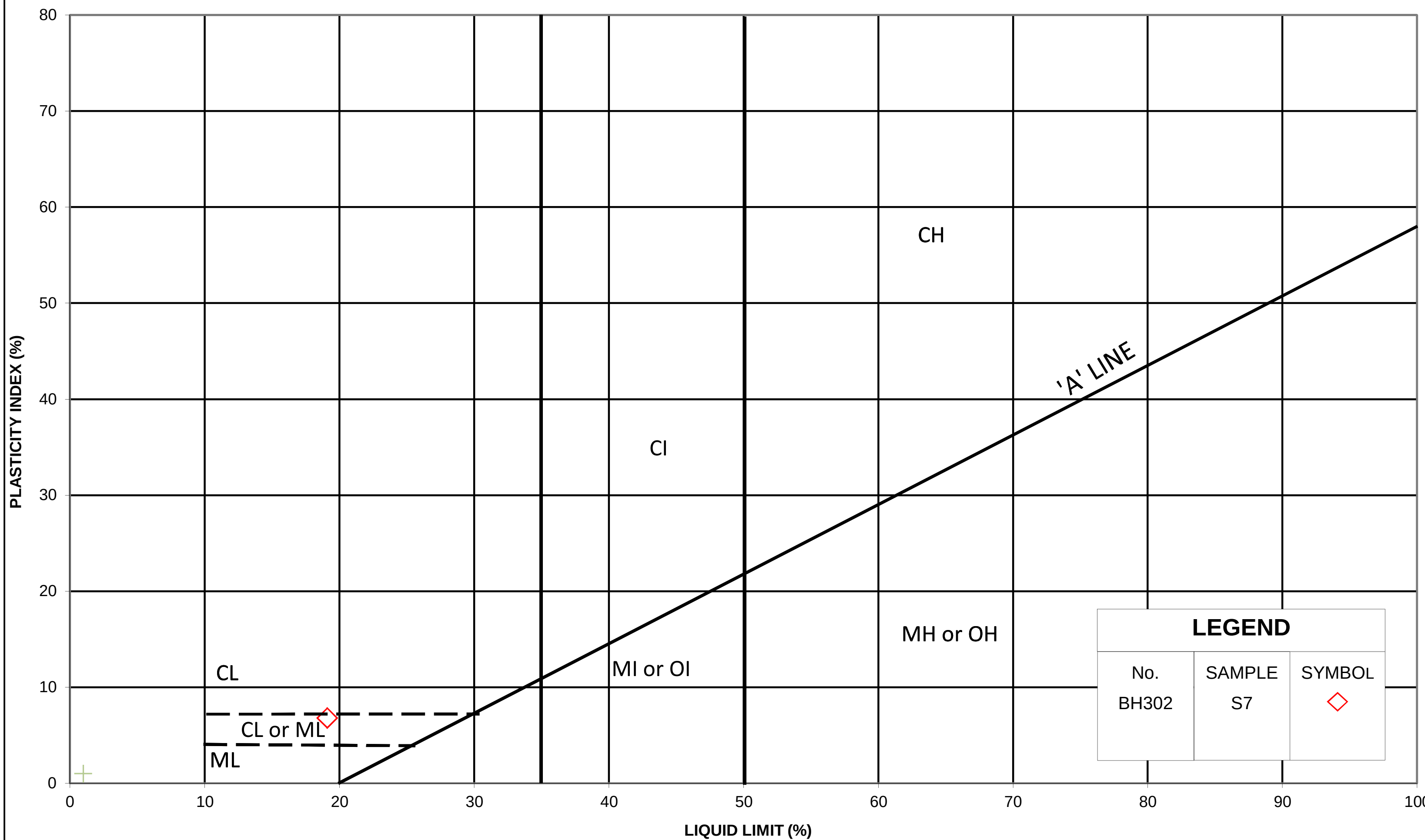
DATE : July 16, 2015

Caribus Lake Culvert (Site No. 45-259/C)
GWP No. 6320-14-00, Highway 11, Atikokan, Ontario



exp.	PLASTICITY CHART ***Clayey SILT***	FIGURE No. 4 ADM-00223648-E0 July 22, 2015

Caribus Lake Culvert (Site No. 45-259/C)
GWP No. 6320-14-00, Highway 11, Atikokan, Ontario



exp.

PLASTICITY CHART

Silty SAND

FIGURE No. 5

ADM-00223648-E0

July 22, 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.

Maxxam Analytics International Corporation o/a Maxxam Analytics 6740 Campobello Road, Mississauga, Ontario, L5N 2L8 Tel: (905) 817-5700 Toll-Free: 800-563-6266 Fax: (905) 817-5777 www.maxxam.ca

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

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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

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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

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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 Analytics International Corporation o/a Maxxam Analytics 6740 Campobello Road, Mississauga, Ontario, L5N 2L8 Tel: (905) 817-5700 Toll-Free: 800-563-6266 Fax: (905) 817-5777 www.maxxam.ca

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

			Matrix Spike		Spiked Blank		Method Blank		RPD	
QC Batch	Parameter	Date	% 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).

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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).

CHARTERED Eva Pranjic CHEMIST

Ewa Pranjic, 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.

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IMMEDIATE TEST

CHAIN OF CUSTODY RECORD

Page 1 of 1

INVOICE INFORMATION		REPORT INFORMATION (if differs from invoice)		PROJECT INFORMATION		TURNAROUND TIME (TAT) REQUIRED
Company Name:	exp Services Inc.	Company Name:		Quotation #:		[x] Regular TAT (5-7 days)
Contact Name:	Michael Suslyk, Ahileas Mitsopoulos	Contact Name:		P.O. #:		PLEASE PROVIDE ADVANCE NOTICE FOR RUSH PROJECTS
Address:	1142 Roland Street	Address:		Project #:	ADM-00223648-E0	Rush TAT (Applicable Surcharge)
	Thunder Bay, ON P7B 5M4			Site Location:	MTO Assignment # 6 - Hwys 11 & 502	[ ] 1 Day (100%)
Phone:	807.623.9495 Fax: 807.623.8070	Phone:	Fax:	Site #:		[ ] 2 Days (50%)
Email:	michael.suslyk@exp.com, ahileas.mitsopoulos@exp.com	Email:		Sampled By:	Elwin Farkas	[ ] 3-4 Days (25%)

MOE REGULATED DRINKING WATER OR WATER INTENDED FOR HUMAN CONSUMPTION MUST BE SUBMITTED ON THE MAXXAM DRINKING WATER CHAIN OF CUSTODY

REGULATION 153 (2011)
[ ] Table 1 [ ] Res/Park [ ] Med/Fine
[ ] Table 2 [ ] Ind/Comm [ ] Coarse
[ ] Table 3 * [ ] Agri/Other
[ ] Table ___
FOR RSC (PLEASE CIRCLE) Yes / No

OTHER REGULATIONS
[ ] CCME [ ] Sanitary Sewer Bylaw
[ ] MISA [ ] Storm Sewer Bylaw
[ ] PWQO Municipality:
[ ] Other (Specify):
[ ] REG 558 (MINIMUM 3 DAY TAT REQUIRED)

Include Criteria on Certificate of Analysis (Y/N)? Y

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

ANALYSIS REQUESTED

	SAMPLE IDENTIFICATION	DATE SAMPLED	TIME SAMPLED	MATRIX	# OF CONT.	FIELD FILTERED (PLEASE CIRCLE) Metals / Hg / CrVI	pH	Water Soluble Sulphate	Resistivity	Conductivity	Chloride
1	BH101-S7	19-Jun-15	2:10	Soil	2		x	x	x	x	x
2	BH104-S3B/S4/S5	27-Jun-15	12:15	Soil	2		x	x	x	x	x
3	BH201-S7A	28-Jun-15	10:20	Soil	2		x	x	x	x	x
4	BH2013-S3	28/06/2015	5:00	Soil	2		x	x	x	x	x
5	BH301-S9B/S10/S11	20/06/2015	7:25	Soil	2		x	x	x	x	x
6	BH304-S3	26/06/2015	6:20	Soil	2		x	x	x	x	x
7	BH403-S3	26/06/2015	4:15	Soil	2		x	x	x	x	x
8	BH404-S5B	25/06/2015	3:30	Soil	2		x	x	x	x	x
9	BH503-S4	25/06/2015	10:30	Soil	2		x	x	x	x	x
10	BH504-S1B	25/06/2015	2:10	Soil	2		x	x	x	x	x

Rush Confirmation #:

Date Required:

LABORATORY USE ONLY

CUSTODY SEAL (Y/N)		Temperature (°C) on Receipt
Present	N	2, 3, 3
Intact	N	
COOLING MEDIA PRESENT (Y / N)	Y	

COMMENTS / TAT COMMENTS

RELINQUISHED BY: (Signature/Print)	DATE: (YYYY/MM/DD)	TIME:	RECEIVED BY: (Signature/Print)	DATE: (YYYY/MM/DD)	TIME:
Michael Suslyk	02-Jul-15	13:00	Alex Vandafi ALEX VANDAFI	2015/07/03	10:55

3-Jul-15 10:55
Hina Siddiqui
B5C9097
FW ENV-417

Maxxam Analytics International Corporation o/a Maxxam Analytics

Appendix F – Slope Stability Analysis

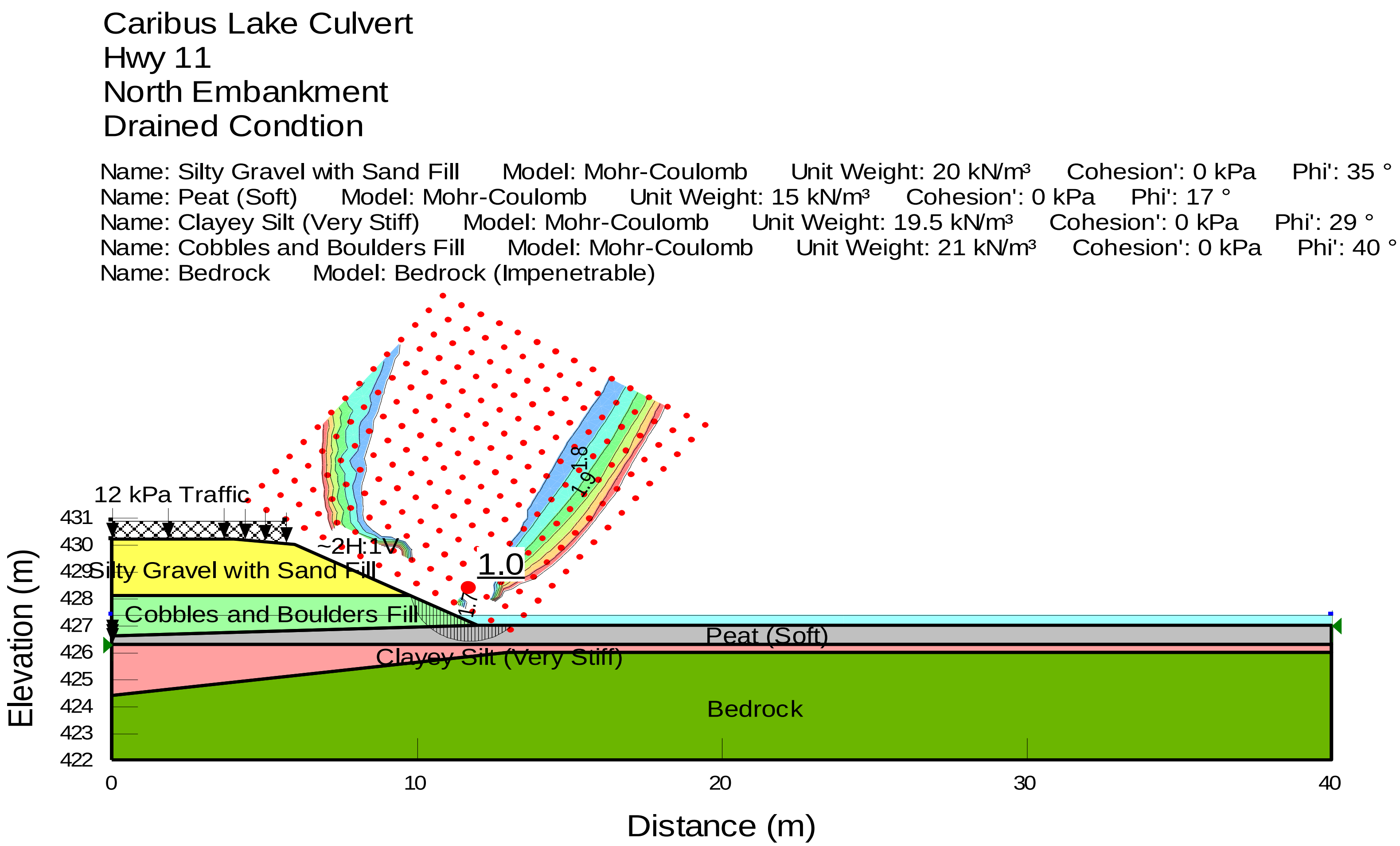


Figure F1: North embankment (inlet) – drained static condition with peat below embankment

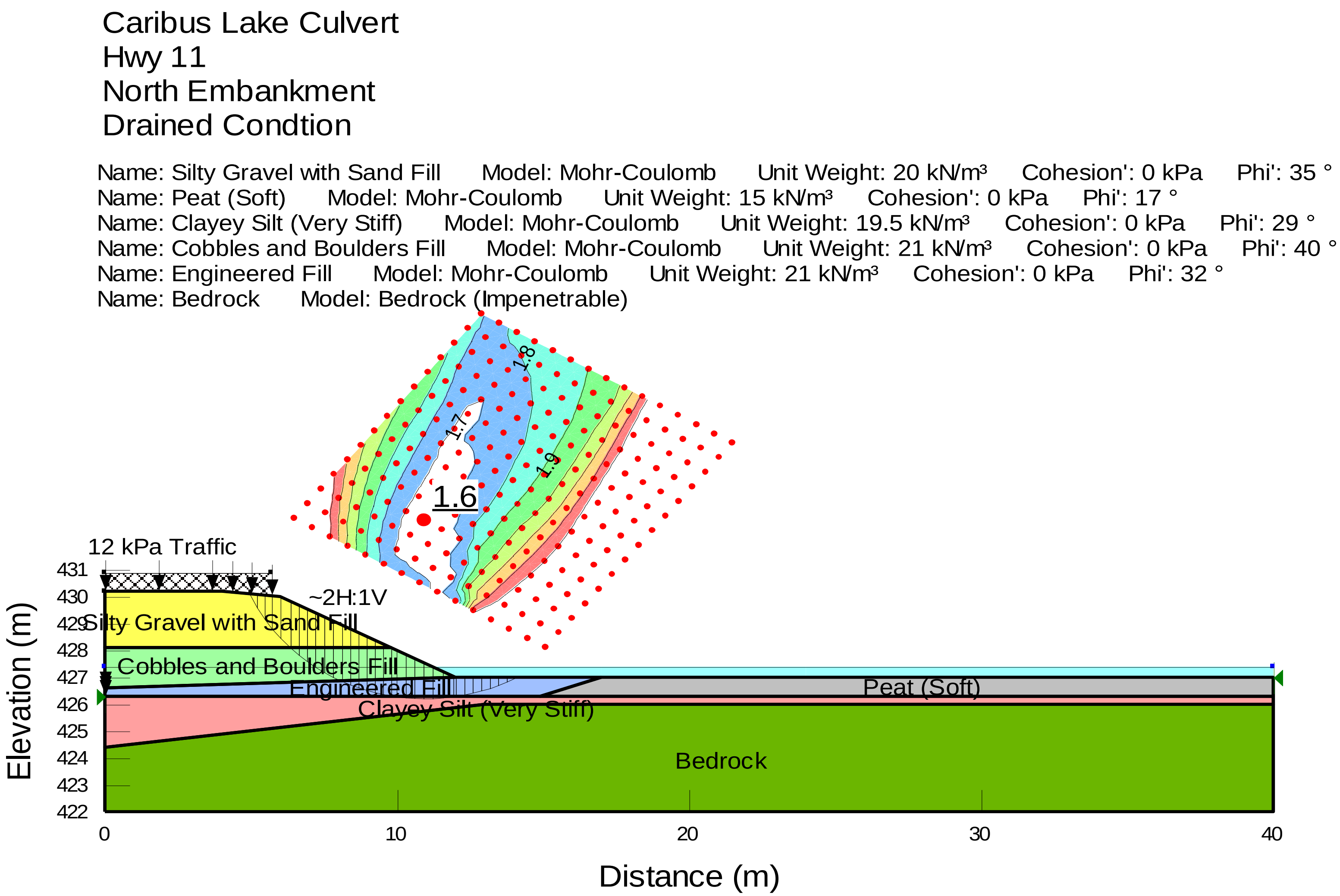
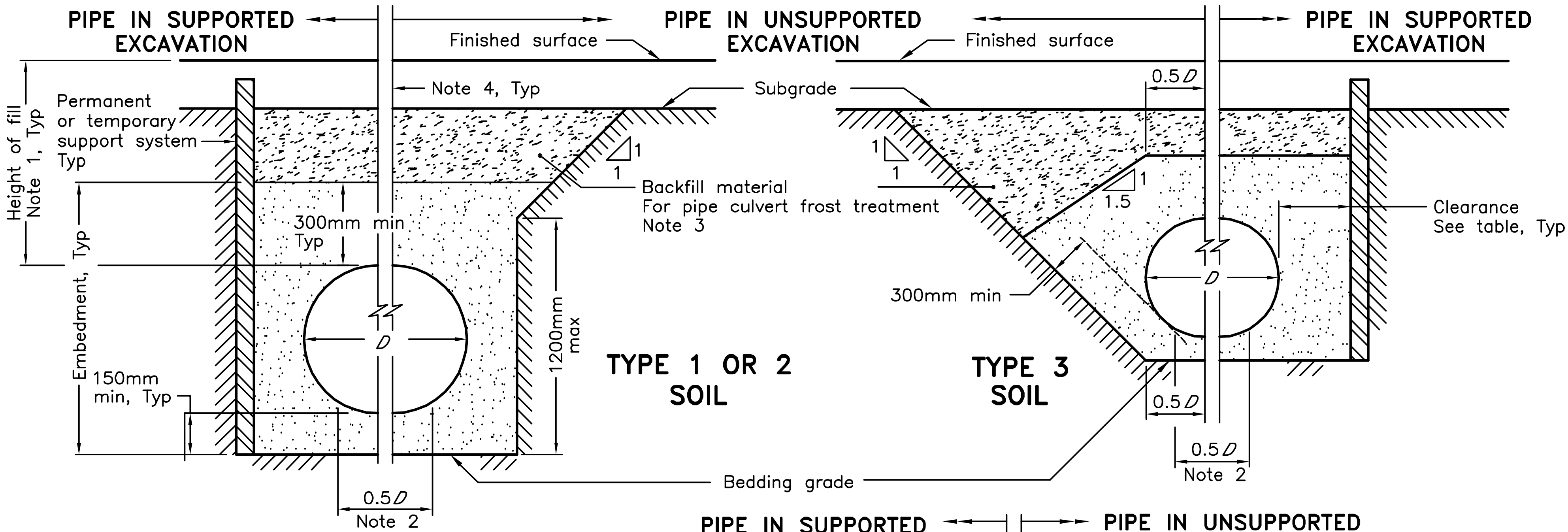
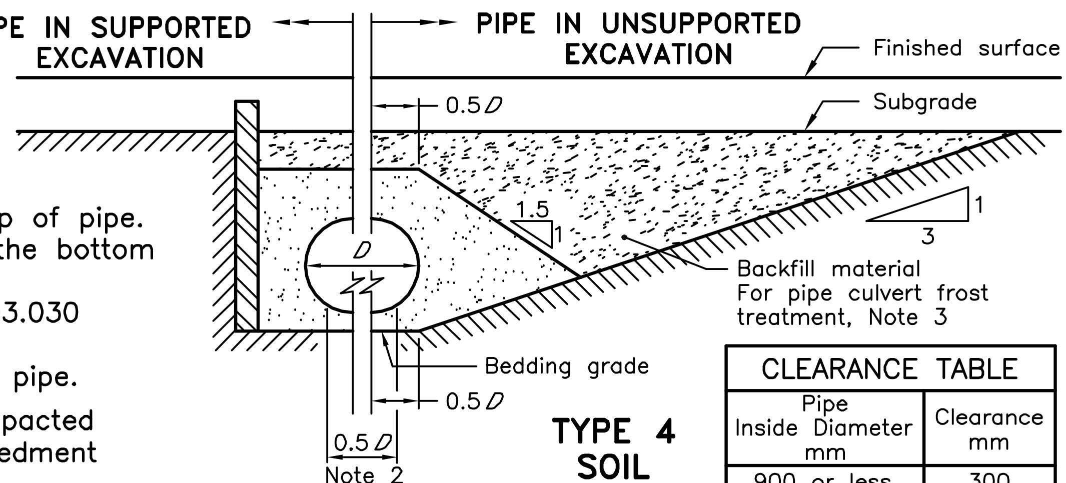


Figure F2: North embankment (inlet) – drained static condition after replacement of peat with engineered fill below embankment

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

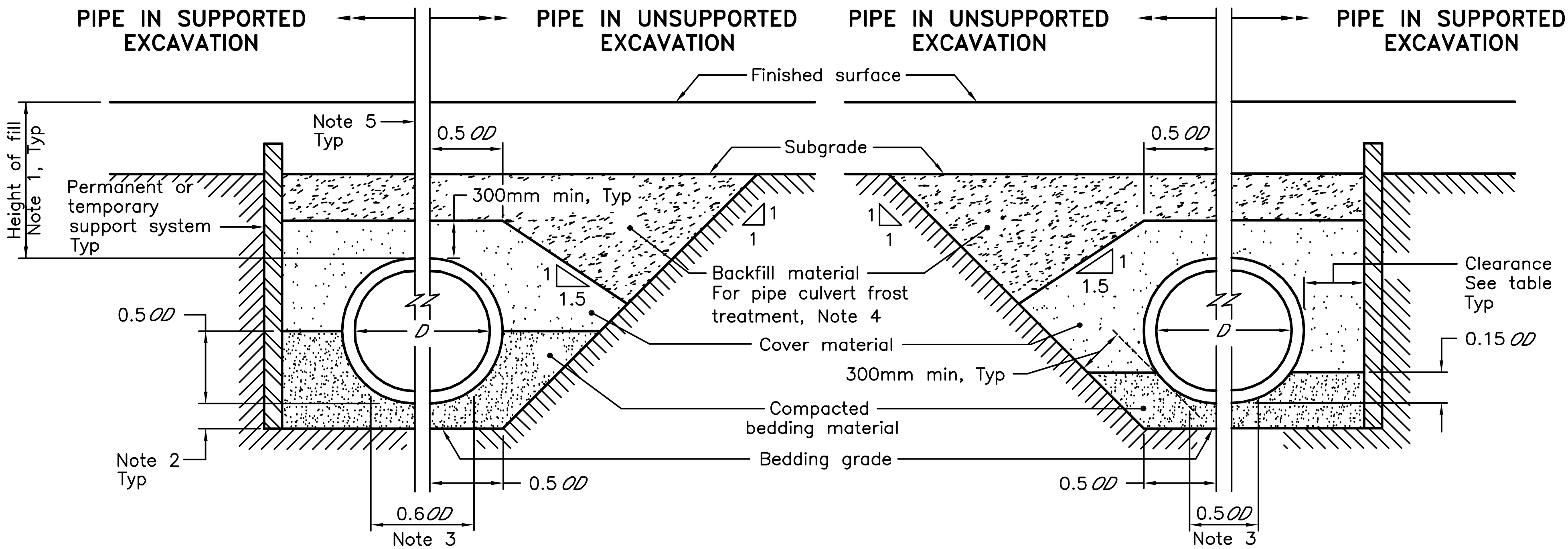
Pipe Inside Diameter mm	Clearance mm
900 or less	300
Over 900	500

ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2010 | Rev 2

FLEXIBLE PIPE EMBEDMENT AND BACKFILL EARTH EXCAVATION

OPSD 802.010



CLASS B BEDDING **CLASS C BEDDING**

NOTES:

1 Height of fill is measured from the finished surface to top of pipe.
2 The minimum bedding depth below the pipe shall be 0.15*D*. In no case shall this dimension be less than 150mm or greater than 300mm.
3 The pipe bed shall be compacted and shaped to receive the bottom of the pipe.
4 Pipe culvert frost treatment shall be according to OPSD 803.030 and 803.031.
5 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.

LEGEND:

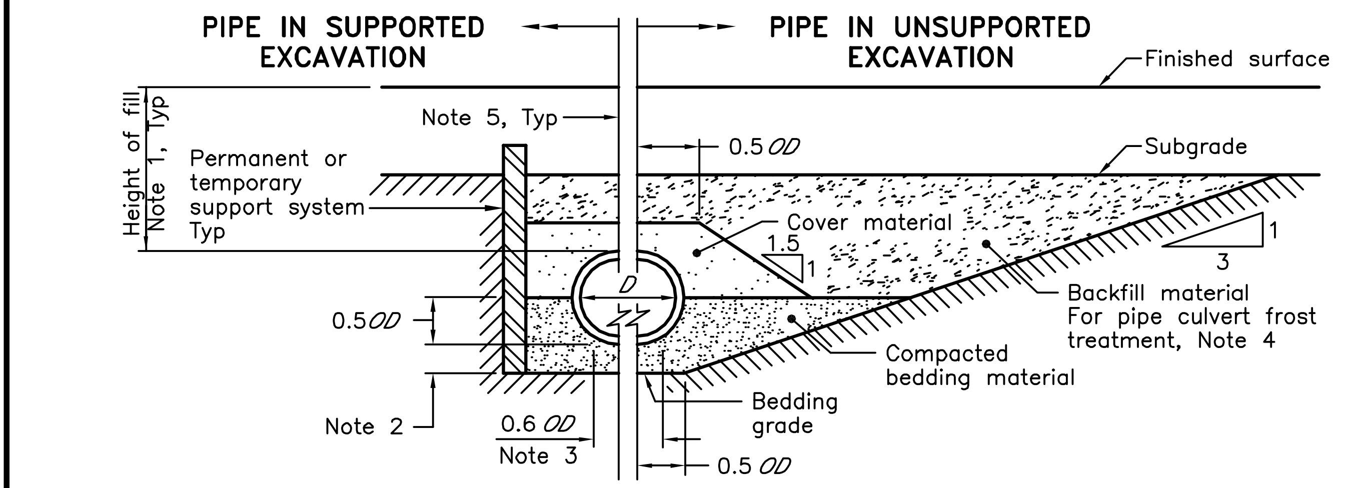
D – Inside diameter
OD – Outside diameter

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

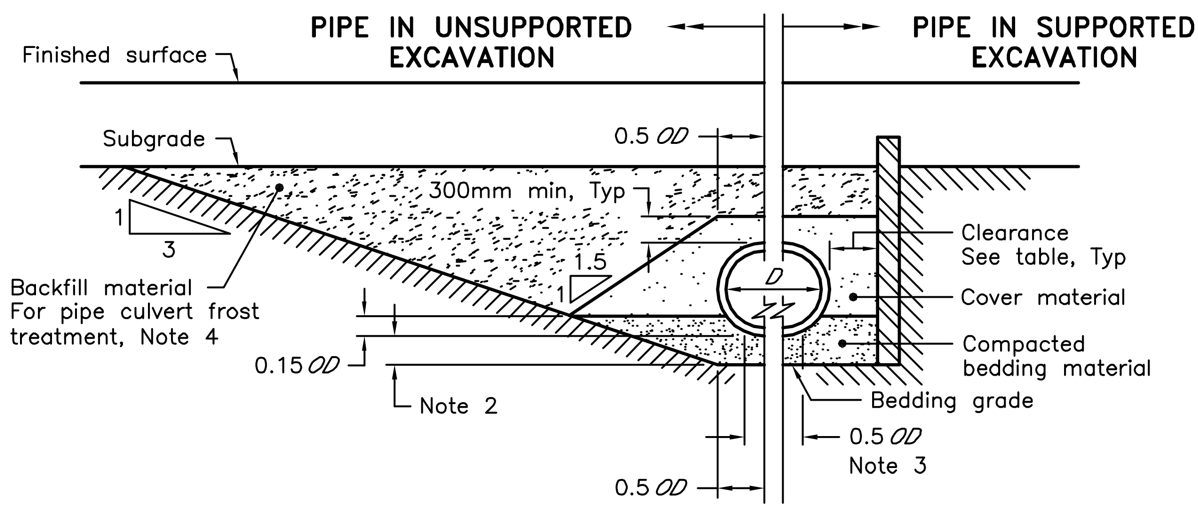
ONTARIO PROVINCIAL STANDARD DRAWING | Nov 2010 | Rev 2

RIGID PIPE BEDDING, COVER, AND BACKFILL TYPE 3 SOIL – EARTH EXCAVATION

OPSD 802.031



CLASS B BEDDING



CLASS C BEDDING

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

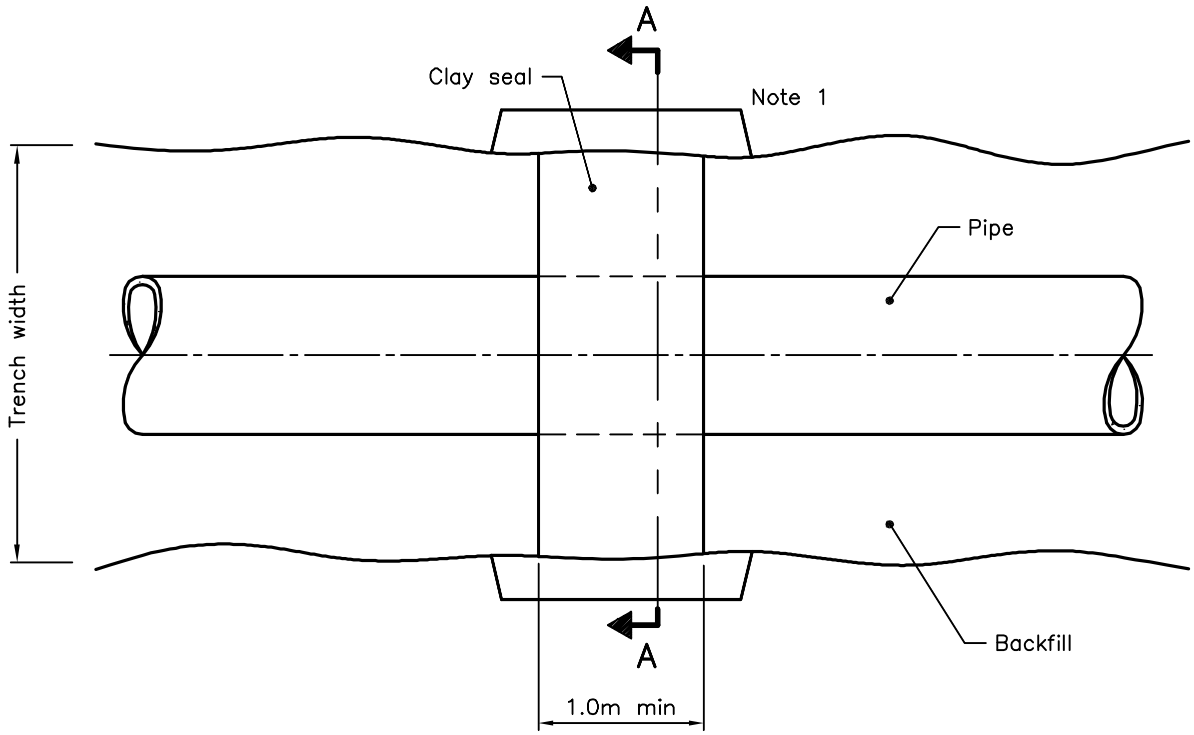
LEGEND:

D – Inside diameter
OD – Outside diameter

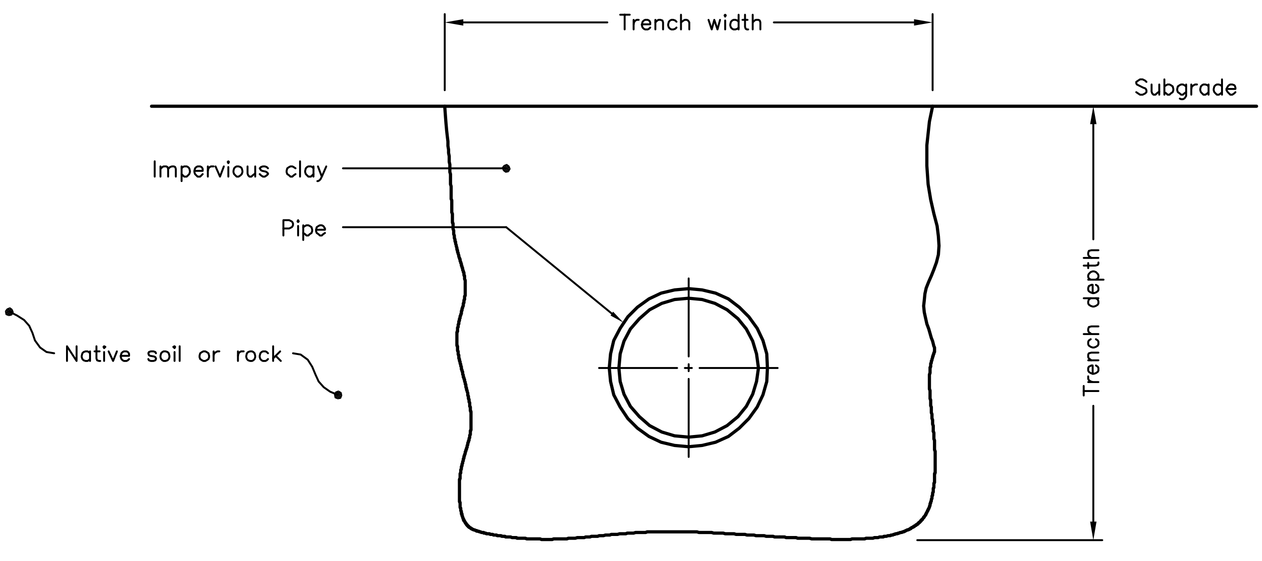
NOTES:

1 Height of fill is measured from the finished surface to top of pipe.
2 The minimum bedding depth below the pipe shall be 0.15*D*. In no case shall this dimension be less than 150mm or greater than 300mm.
3 The pipe bed shall be compacted and shaped to receive the bottom of the pipe.
4 Pipe culvert frost treatment shall be according to OPSD 803.030 and 803.031.
5 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

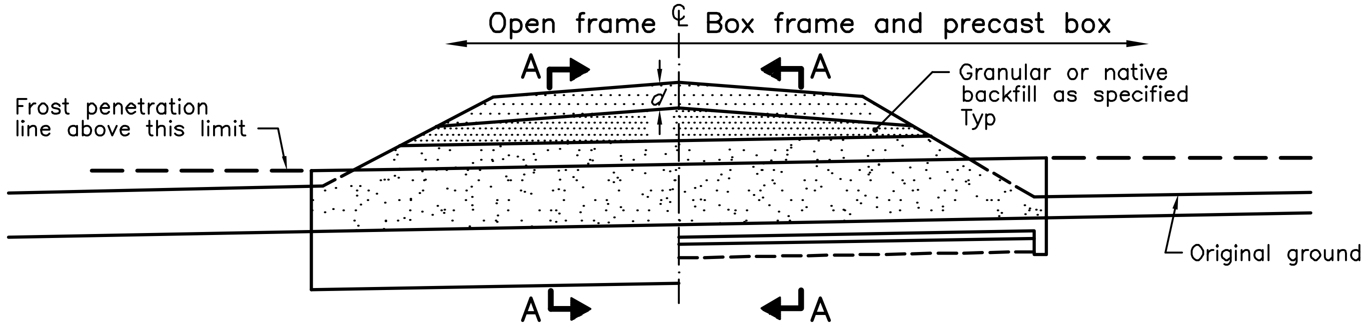
NOTES:

1. Key into undisturbed trench soil.

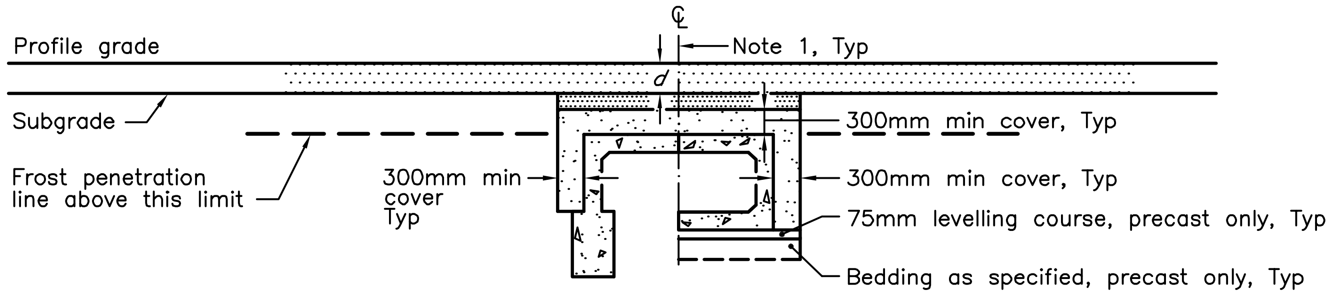
A Clay seal shall extend from bottom of trench excavation to the subgrade.
B Clay seal shall be located so that no pipe joints are within the clay seal material.
C All dimensions are in metres unless otherwise shown.

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

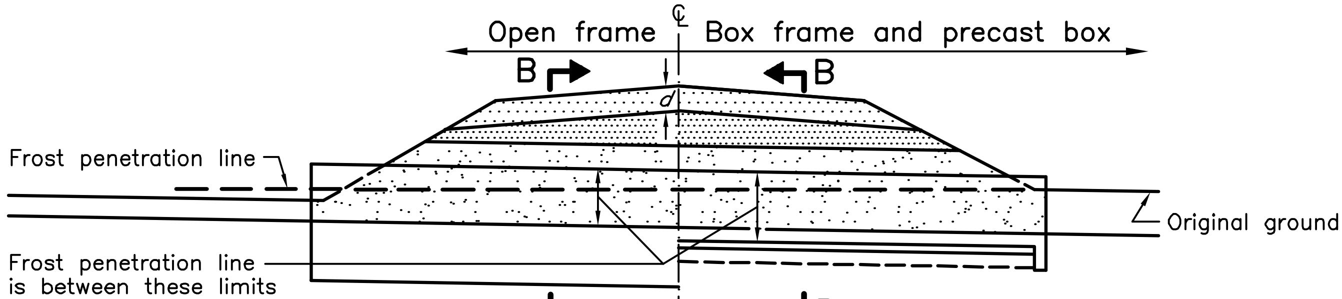


LONGITUDINAL SECTION

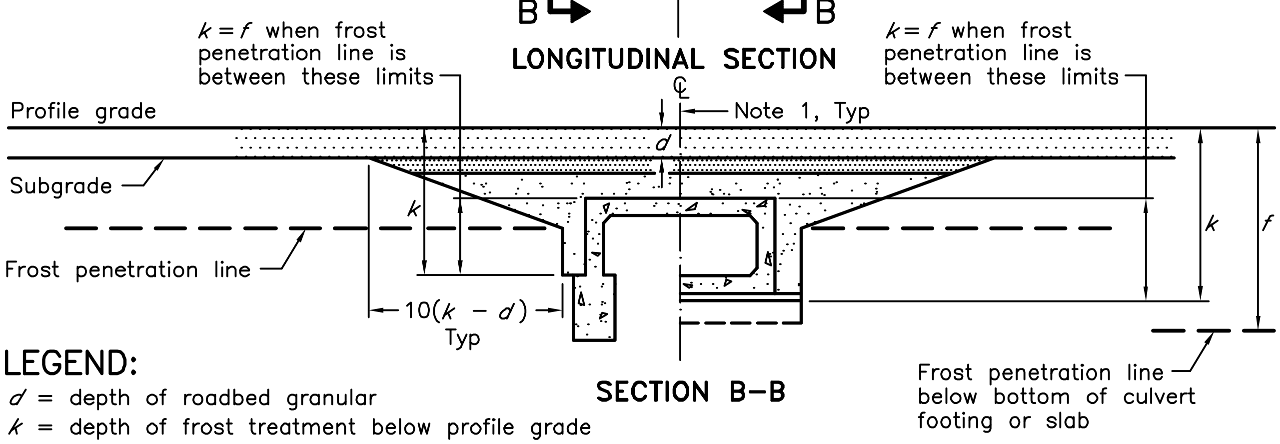


SECTION A–A

FROST PENETRATION LINE BELOW TOP OF CULVERT



LONGITUDINAL SECTION



SECTION B–B

LEGEND:

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

NOTES:

1 Condition of frost treatment symmetrical about centreline of culvert.

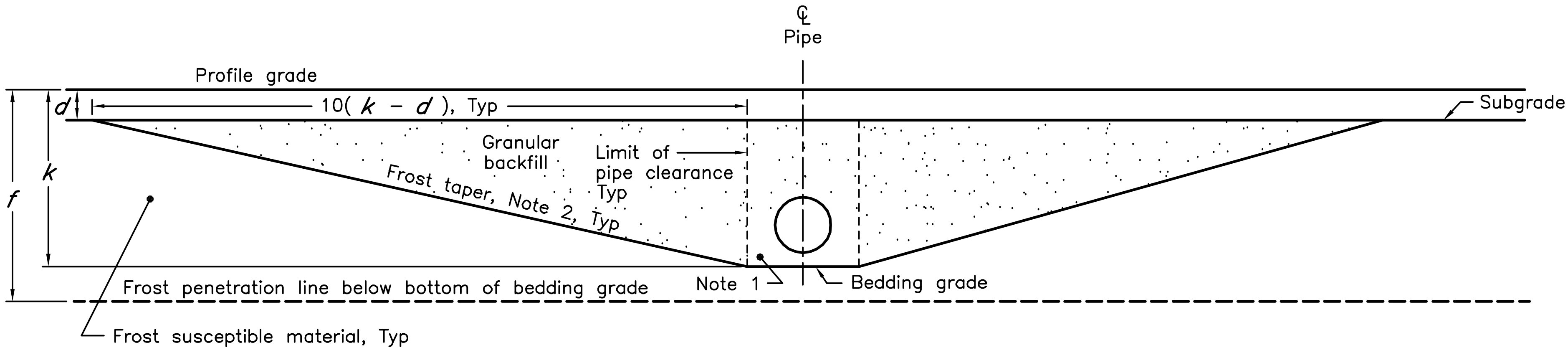
A Bedding, levelling, and cover material shall be granular as specified.

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

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

D All dimensions are in millimetres unless otherwise shown.

ONTARIO PROVINCIAL STANDARD DRAWING	Nov 2010	Rev 2
BACKFILL AND COVER FOR CONCRETE CULVERTS WITH SPANS LESS THAN OR EQUAL TO 3.0M		OPSD 803.010



FROST TREATMENT – RIGID AND FLEXIBLE PIPE

NOTES:

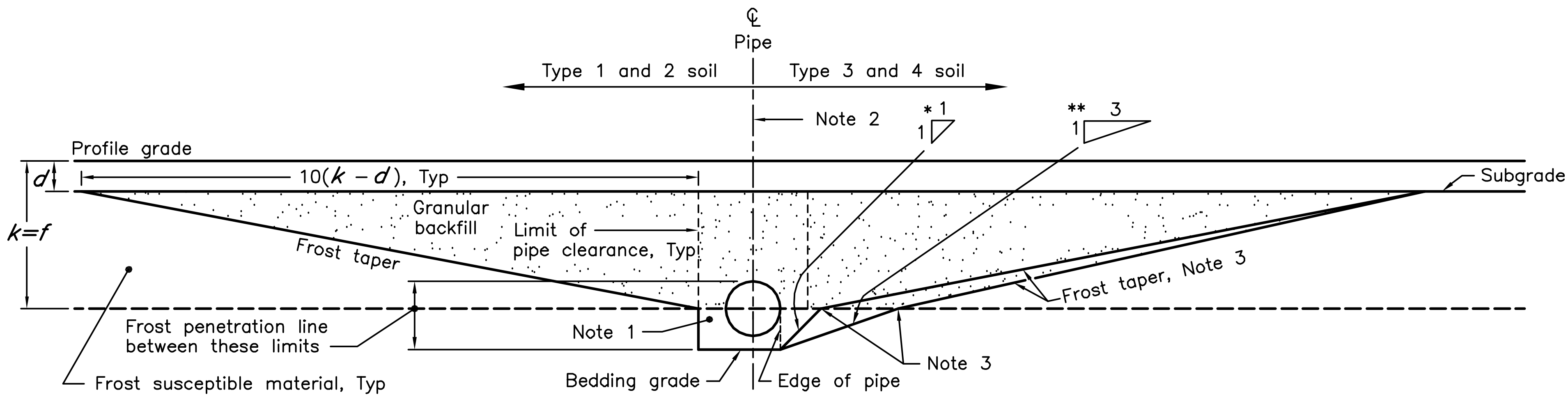
1 Pipe embedment or bedding, cover, and backfill shall be according to:
   a) Flexible – OPSD 802.010, 802.013, 802.014, 802.020, 802.023, and 802.024
   b) Rigid – OPSD 802.030, 802.031, 802.032, 802.033, 802.034, 802.050, 802.051, 802.052, 802.053, and 802.054.

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.

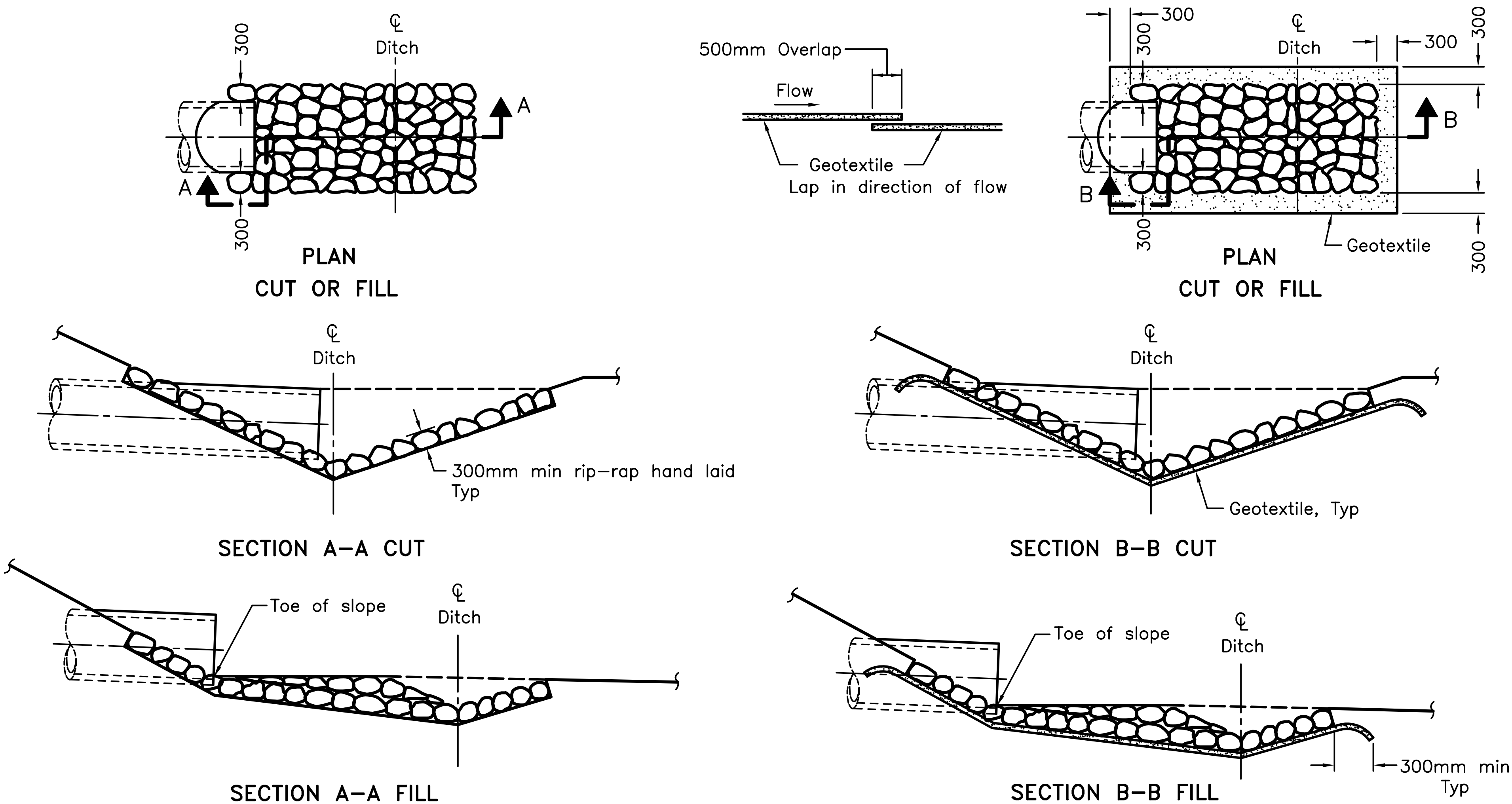
3 Frost tapers shall start at the intersection of the 1H:1V or 3H:1V slope and the frost penetration line.

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	



TYPE A – WITHOUT GEOTEXTILE

TYPE B – WITH GEOTEXTILE

NOTES:

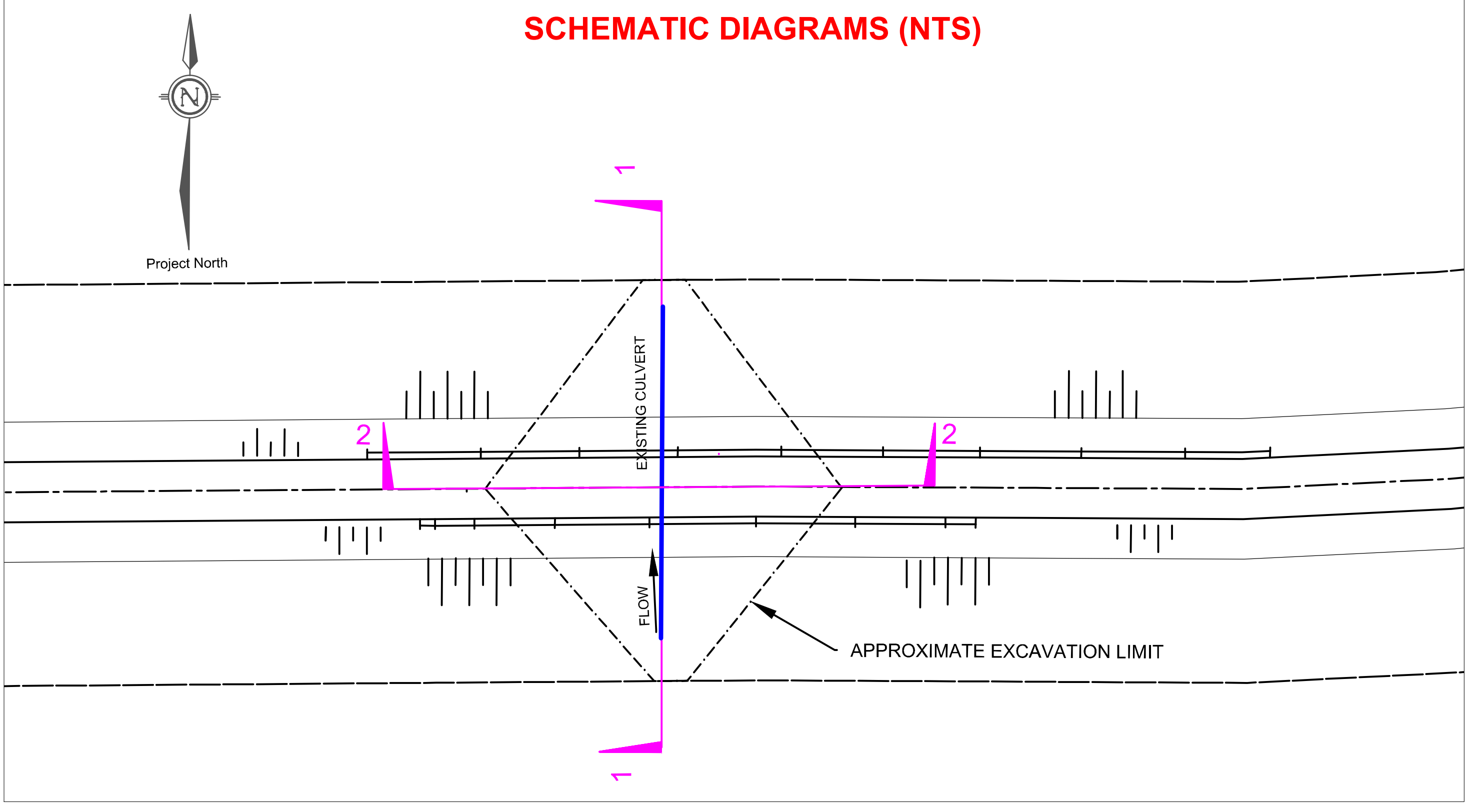
A All dimensions are in millimetres unless otherwise shown.

ONTARIO PROVINCIAL STANDARD DRAWING	Nov 2001	Rev 0
RIP–RAP TREATMENT FOR SEWER AND CULVERT OUTLETS		
	OPSD – 810.010	

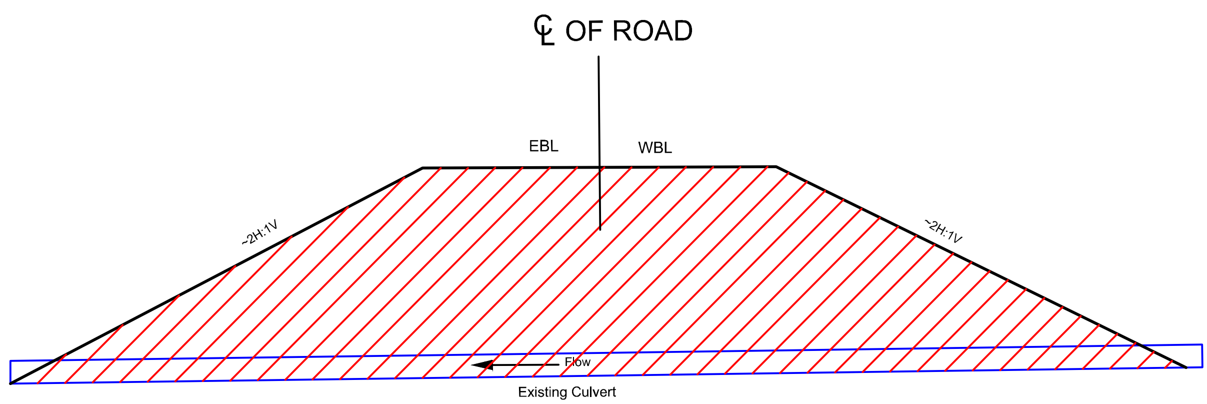
Appendix H –
Schematic Sketches for Construction Alternatives

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

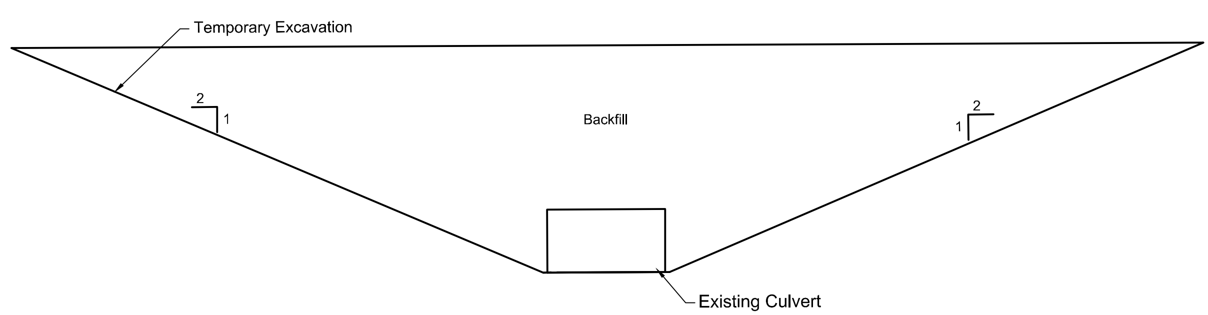
SCHEMATIC DIAGRAMS (NTS)



PLAN



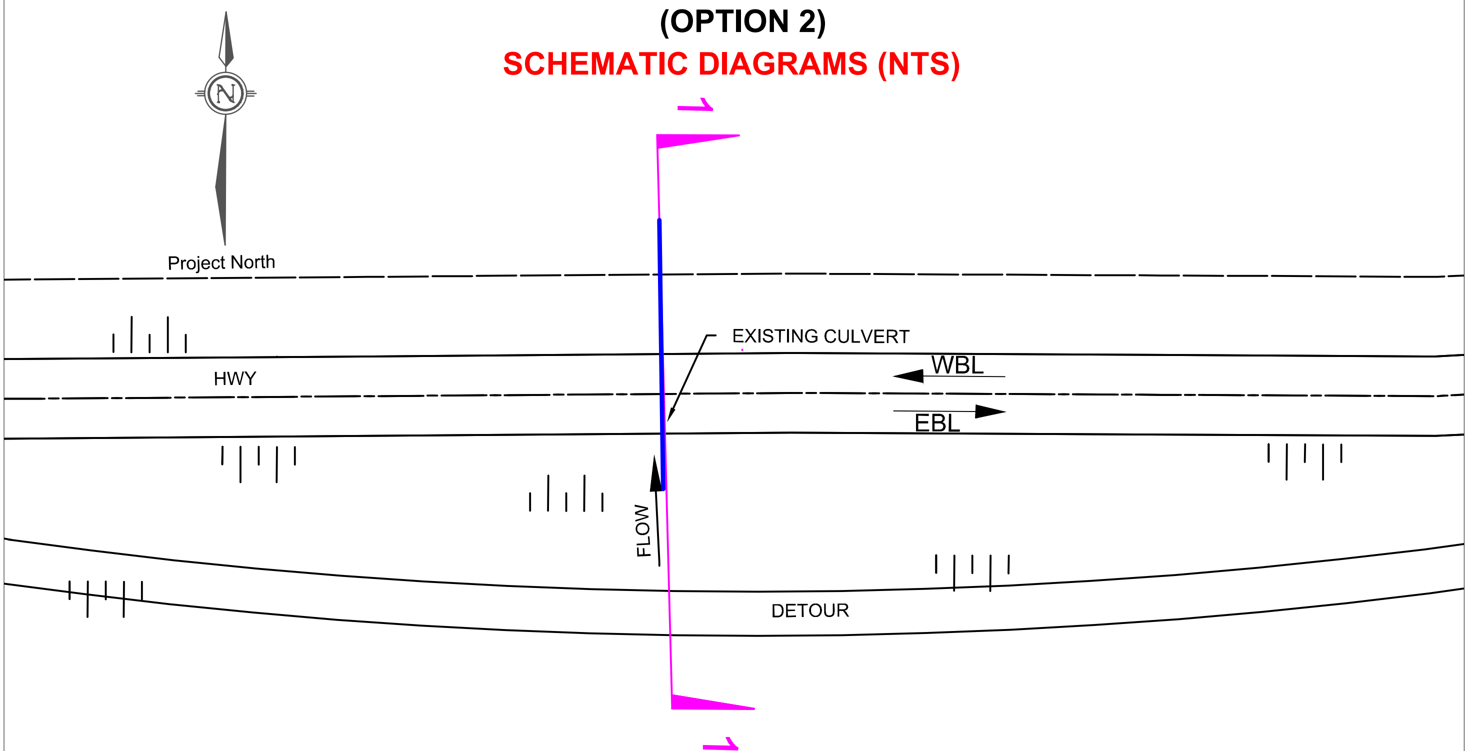
SECTION 1-1



SECTION 2-2

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

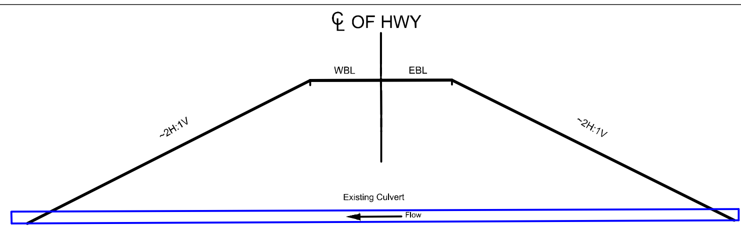
SCHEMATIC DIAGRAMS (NTS)



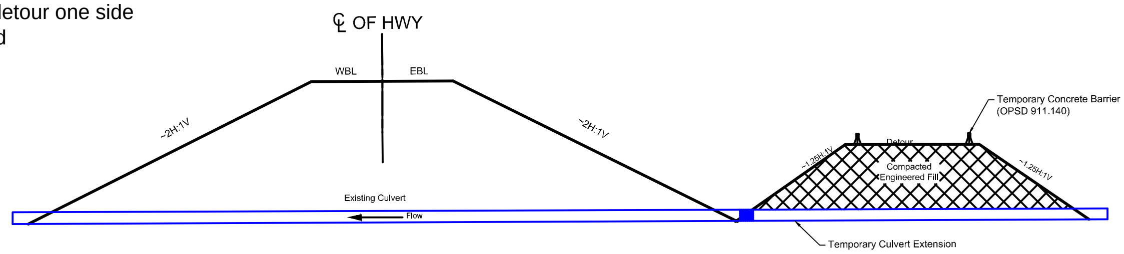
PLAN

RECOMMENDED STAGES

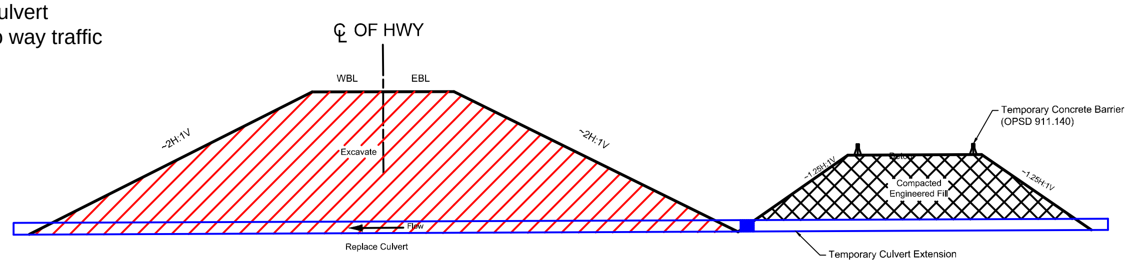
1.0 Stage 1 - Current condition



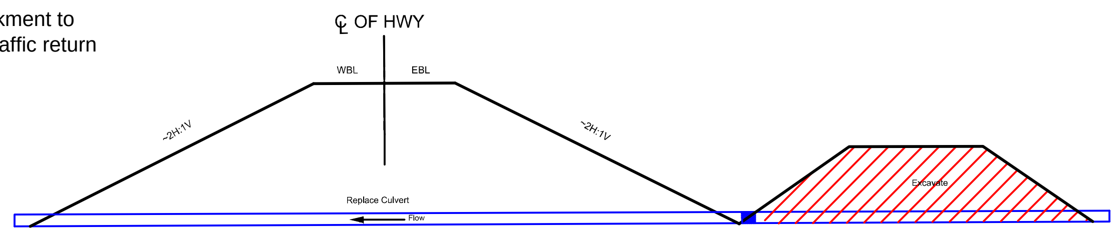
2.0 Stage 2 - Build temporary detour one side
Two-way traffic on existing road



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



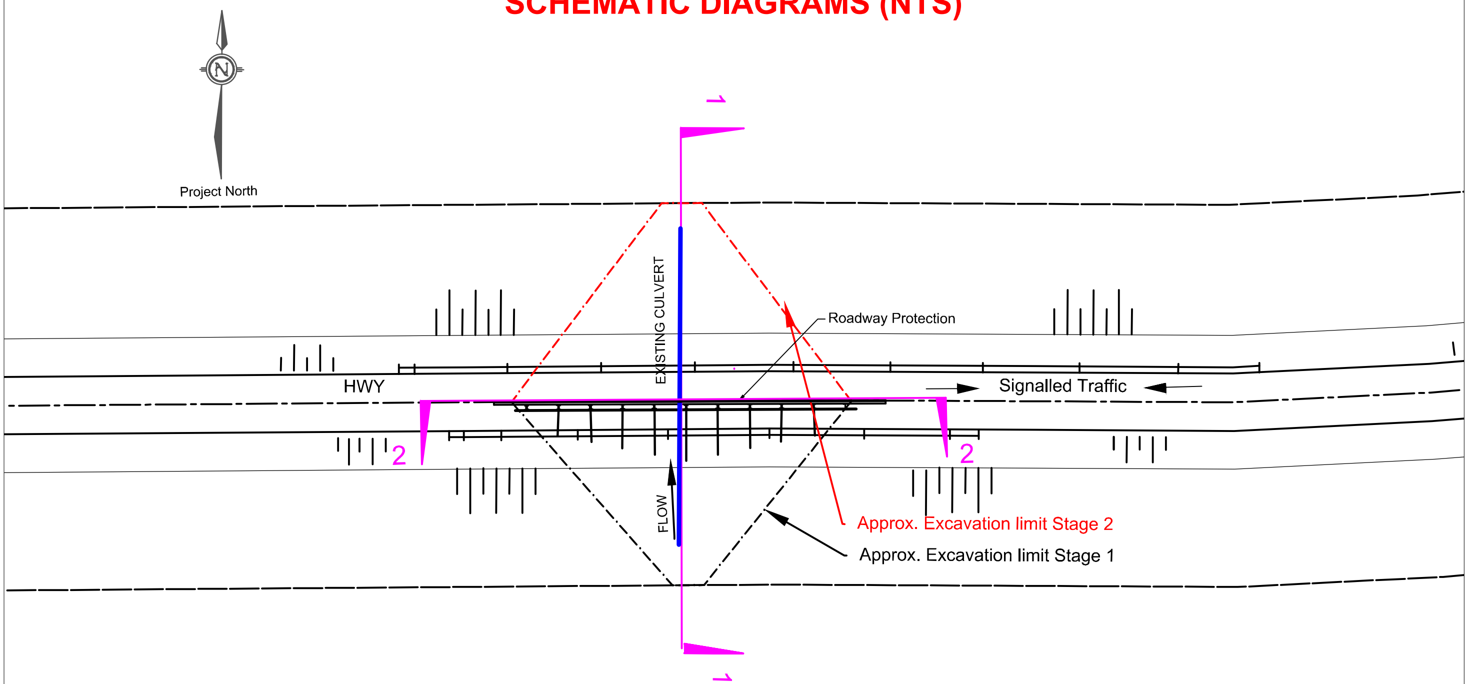
4.0 Stage 4 - Build the embankment to existing alignment; Two-way traffic return



SECTION 1-1

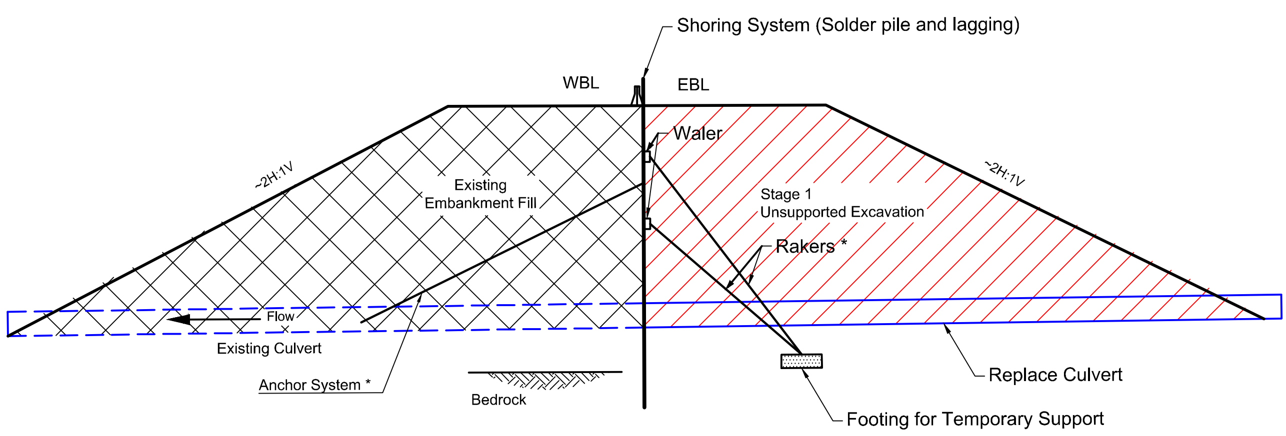
FIGURE H.3.A: HALF AND HALF CONSTRUCTION WITH UNSUPPORTED CUT SIDES

SCHEMATIC DIAGRAMS (NTS)



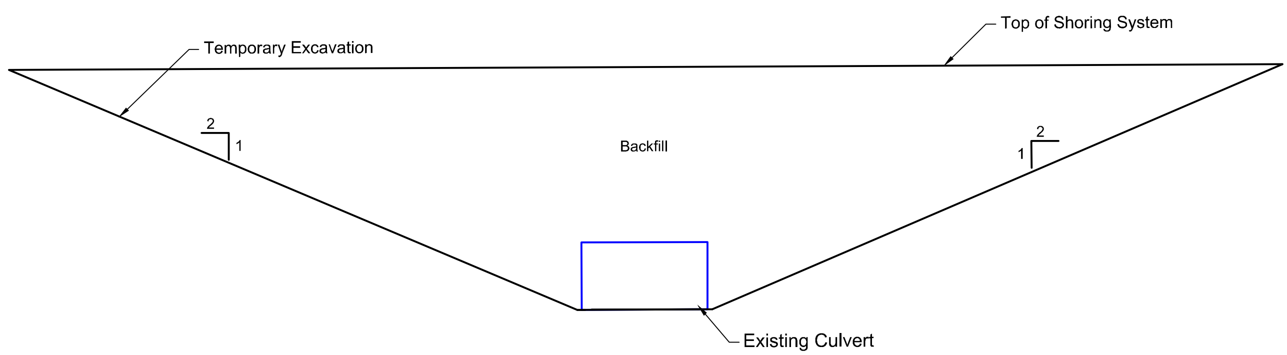
PLAN

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



* Rakers or Anchor System

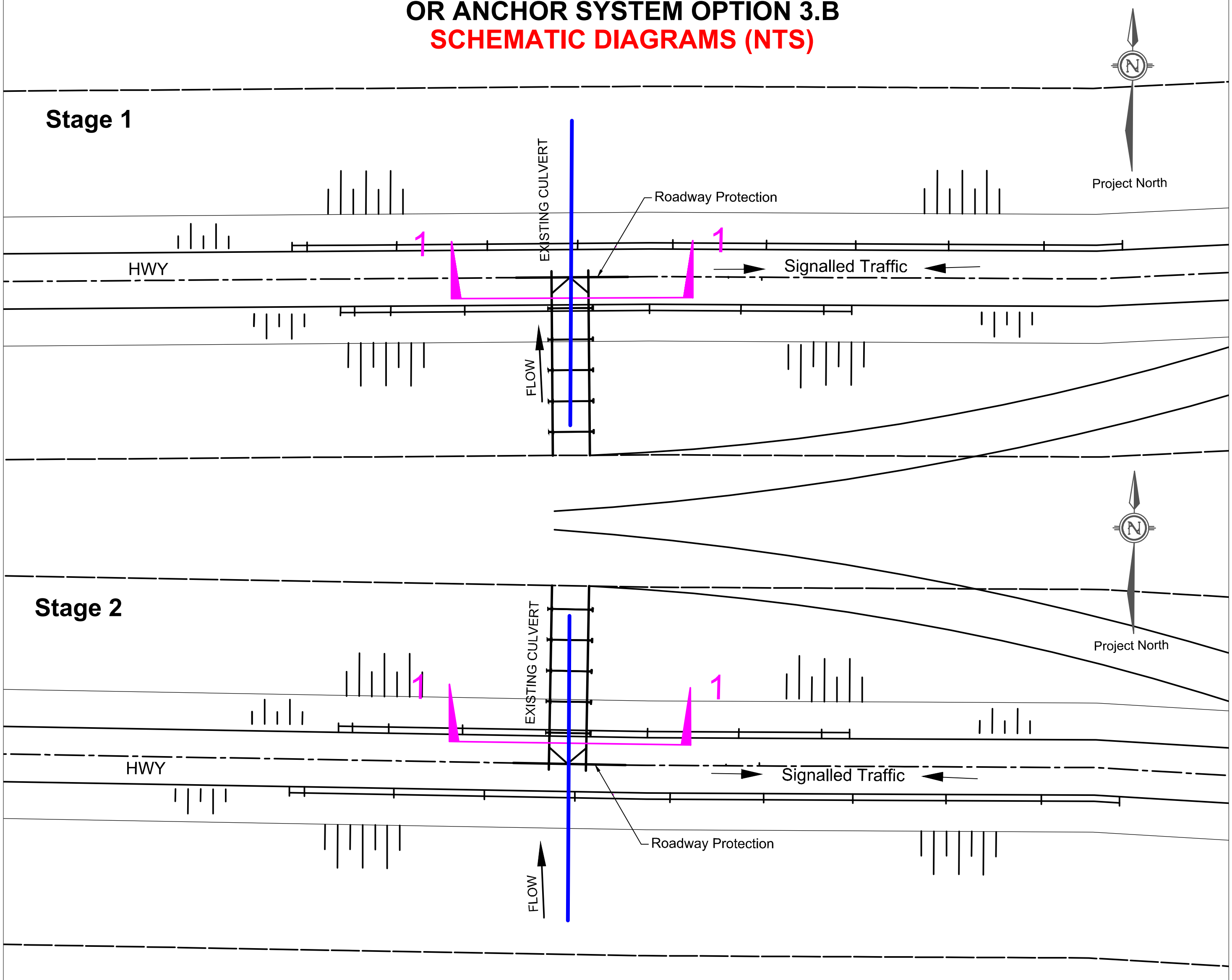
SECTION 1-1



SECTION 2-2

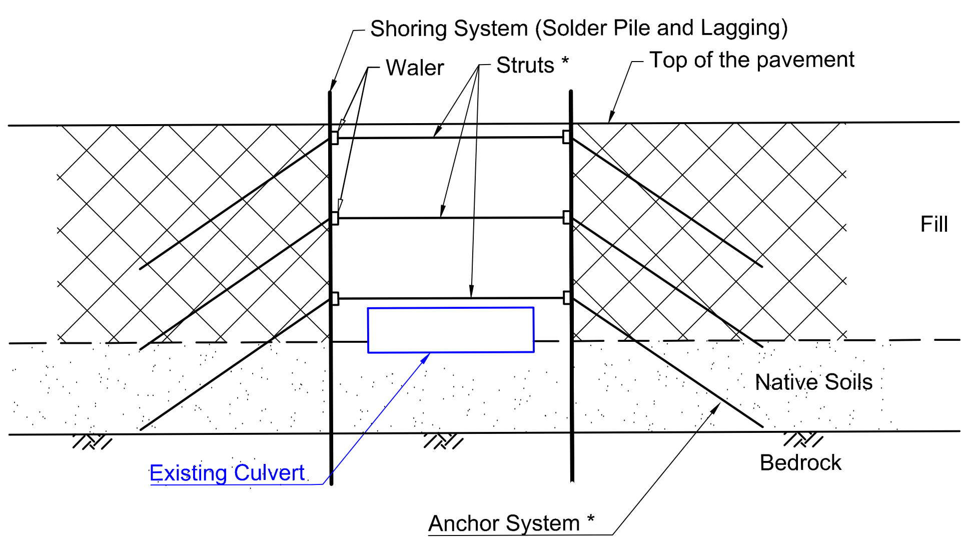
FIGURE H.3.B: HALF AND HALF CONSTRUCTION WITH BRACED CUT SIDES OR ANCHOR SYSTEM OPTION 3.B

SCHEMATIC DIAGRAMS (NTS)



PLAN

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



* Struts or Anchor System

SECTION 1-1

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 SLOPINIG 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 shell 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.