FINAL REPORT

FOUNDATION INVESTIGATION AND DESIGN REPORT

Cedar Creek Culvert No. 1 Replacement, Highway 590, Site No. 48W-154/C, Township of O'Connor, District of Thunder Bay

Agreement No. 6014-E-0017
Assignment No. 2
GWP 6347-14-00
Geocres No. 52A-214

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Foundation Investigation and Design Report for Cedar Creek Culvert No. 1 Replacement
HWY 590, Site No. 48W-154/C, Township of O'Connor, District of Thunder Bay

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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 rehabilitation/replacement of Cedar Creek Culvert No. 1 on Highway 590, located approximately 8 km west of the junction of Highway 590 and Highway 11/17 at Cedar Creek, in the Township of O'Connor, the Ministry of Transportation (MTO) Northwestern Region. The work was undertaken under Agreement # 6014-E-0017, Assignment No. 2 (GWP 6347-14-00). The terms of reference (TOR) were as presented in the MTO letter dated January 23, 2015.

Based on preliminary information provided, it is understood the existing culvert is a rigid frame open footing/retaining wall concrete structure with a span of 7.2 m, depth of 1.8 m and a length of about 24.5 m. It is understood that the existing culvert was constructed at an unknown date, 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, including possible cofferdams on the inlet and outlet sides of the culvert. 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 Cedar Creek Culvert No. 1 replacement site is located on Hwy 590, approximately 8 km west of the junction of Hwy 590 and Hwy 11/17, in the Township of O'Connor. At the site, Hwy 590 is a two lane roadway, with a speed limit of 80 km/h and is about 7 m wide from edge of pavement to edge of pavement, with narrow sand and gravel shoulders. Based on drawings provided, the roadway embankment is about 6 m high with side slopes of about 2H:1V.

During the fieldwork on February 26 to 28, as well as March 20 and 23, 2015, the general site conditions were assessed; however, the site was generally snow covered which limited observations possible. Hwy 590 runs in an east to west direction and Cedar Creek, flows from north to south towards Kakabeka Falls, which is about 10 km east of the site. At the time of this investigation, Cedar Creek was frozen and the approximate creek elevations (top of ice) at the inlet and outlet were about 370.63 m and 369.50 m, respectively. The elevation of highway centerline pavement is about 374.90 m. Overhead wires were observed along the south side of the roadway.

The vicinity of the inlet and outlet of the culvert was snow covered but the visible vegetation was generally reeds and grasses. The inlet and outlet appeared to be 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 5047. Scale 1:100,000, dated 1979, the underlying native soil at the sites consists of silt till ground moraine with mainly low local relief, undulating to rolling and dry surface conditions.

The Precambrian Geology Compilation Series, Map 2664, Thunder Bay Sheet, indicates that the bedrock geology of the site is of the Neo to Mesoarchean Era (2.5 to 3.4 Ga), and generally consist of granite-granodiorite. The granite-granodiorite is generally expected to be of a massive to foliated texture; locally porphorytic (phenocrysts include quartz, feldspar, biotite and amphibole minerals) and containing quartz diorite and diorite in some plutons or plutons complexes,

1.3 Investigation Procedures

1.3.1 Site Investigation and Field Testing

The field investigation was performed on February 26 to 28, as well as March 20 and 23 2015. The field program consisted of drilling five (5) sampled boreholes (BH101 to BH106, excluding BH103). Two (2) boreholes were located within the roadway, BH101 and BH102. BH101 was located about 3 m west of the culvert as near as possible to the edge of pavement (about 0.9 m) in the westbound lane and BH102 was located about 3 m east of the culvert as near as was possible to the centerline of Hwy 590 (about 1.2 m) within the eastbound lane. An additional three (3) boreholes (BH104, BH105 and BH106) were advanced off the roadway, for the culvert and potential cofferdam locations. BH104 and BH106 were located southeast of the culvert (outlet side) about 11 m and 20 m, respectively. BH105 was located about 2 m north of the existing culvert (inlet side). BH103, which was intended to be advanced for the purposes of a cofferdam at the inlet side, was not advanced as the distance between the culvert inlet and the MTO right of way is less than about 1.2 m, and a second borehole in close proximity to BH105 was considered redundant. The borehole locations are shown on Drawing 1 in Appendix B.

The roadway boreholes (BH101 and BH102) were advanced using a CME 850 track mounted drill rig, and the off-road boreholes (BH104 to BH106) were advanced using a CME 45 rubber track mounted drill rig. Both drill rigs were equipped with hollow and solid stem continuous flight augers, standard soil sampling equipment (includes 51 mm outside diameter split spoon samplers and *in situ* shear vane testing equipment) and rock coring equipment, NQ size. The roadway boreholes BH101 and BH102 were advanced to auger refusal depths of about 8.0 m and 6.6 m, respectively, and then rock coring was conducted to depths of about 13.4 m and 14.3 m, respectively. The off-road boreholes BH104, BH105, and BH106 were advanced to auger and SPT refusal, at depths of about 4.0 m, 4.0 m and 3.9 m, respectively. The off-road boreholes were terminated at the refusal depths.

The borehole locations were referenced to the MTM ON-15 NAD83 coordinate system and their ground surface elevations were surveyed by **exp** personnel. The ground surface elevations, including top of culvert and top of water/ice at the upgradient and downgradient sides of the highway, were referenced to a geodetic benchmark (BM) provided by the client (nail in tree southwest of the existing culvert). The elevation of the BM is 375.219 m, and location of the BM is shown on Drawing 1, in Appendix B.

During the drilling of the boreholes, 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 select boreholes, when refusal was encountered, sampling of the refusing stratum was performed by diamond core drilling using a 1.5 m long NQ double tube wireline core barrel.

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 and cold patch was used to repair the asphalt surface damaged by the augers. 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 for subsequent laboratory testing and identification.

All of the recovered soil samples were placed in labelled moisture-proof bags and rock cores were brought to **exp**'s Thunder Bay laboratory for additional visual, textual and olfactory examination.

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, ON.

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

1.4 Subsurface Conditions

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

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

In general, the subsurface conditions along the proposed culvert alignment consist of layers of fill material composed of poorly graded gravel with silt and sand, and silty sand to sandy silt with some clay. The fill layers are underlain by native deposits of peat and silt, or sand, and underlain by cobbles and boulders to depths of between about 10 and 11 m below the pavement surface where bedrock was encountered. A more detailed summary of the subsurface conditions encountered in the boreholes is provided in the following sections.

1.4.1 **Fill**

Fill material was encountered beneath the asphalt at BH101 and BH102. The asphalt thickness was about 180 mm at both borehole locations. The fill generally comprised a conglomeration of gravel, sand, silt and clay layers with descriptions ranging between poorly graded gravel with silt and sand to snady silt with clay. The fill was generally described as frozen (in the upper zones), becoming loose to very dense at depth, moist to wet and brown. SPT sampling was not conducted in the upper 1.5 m of fill due to the frozen soil; at these depths samples were collected from the augers. SPT sampling was conducted semi continuously from about 1.5 m below ground surface with "N" values ranging between 4 and 78. The fill at BH101 and BH102 extended to about 6.9 m (367.9 m elevation) and 6.1 m (368.9 m elevation) below ground surface, respectively.

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

Moisture content:

- 1.8% to 31.7%

Grain size distribution:

- 4% to 73% gravel;
- 21% to 55% sand;
- 6% to 47% silt; and
- 15% to 19% clay size.

The results of the 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 Figures 1 and 2 in Appendix D.

1.4.2 **Topsoil / Silt and Peat**

The topsoil was encountered surfacing BH104, BH105 and BH106. The silt and peat was encountered underlying the fill at BH102. The topsoil was generally described as frozen to soft, brown, wet, containing silt, roots and rootlets. The silt and peat was described as very dense/hard, dark brown to grey, wet and containing trace sand and trace gravel. The SPT "N" values ranged between about 3 and 23 blows per 300 mm penetration in the top soil to 100 blows per 300 mm (i.e. SPT refusal) in the silt and peat. The topsoil / silt and peat extended to about 0.1 m below ground surface (369.5 m elevation) at BH104, about 1.1 m below ground surface (368.6 m elevation) at BH105, about 0.8 m below ground surface (368.8 m elevation) at BH106 and about 6.6 m below asphalt surface (368.4 m elevation) and was about 500 mm in thickness at BH102,.

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

Moisture content:

- 66.5% to 213.9%

1.4.3 **Silty Sand**

Native silty sand was encountered underlying the fill at BH101 and the topsoil at BH106. This layer at BH101 contains some clay, while at BH106 some peat. The native silty sand was generally described as very loose to dense, grey to brown and wet to moist. The SPT "N" values ranged between about 2 and 100 blows per 300 mm penetration. The silty sand extended to depths ranging between about 1.8 m and 8.0 m below ground surface, and elevations ranging between 366.7 m and 367.7 m.

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

Moisture content:

- 8.9% to 69.6%

Grain size distribution:

- 7% to 32% gravel,
- 49% to 65% sand; and
- 19% to 38% silt and clay size.

The results of the 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 Figures 3 in Appendix D.

1.4.4 **Silty Sand with Gravel**

Silty sand with gravel was encountered underlying the silty sand at BH101 and BH106 and, topsoil at BH104 and BH105. The layer was generally described as loose to very dense, grey to brown and wet. Some peat/clay was often encountered within this layer. The SPT "N" values of the till ranged between 2 and 100 (i.e. SPT refusal), per 300 mm penetration. The silty sand with gravel extended to the termination depths of BH104, BH105 and BH106 at about 4.0 m (365.6 m elevation), 4.0 m below (365.7 m elevation) and 3.9 m (365.7 m elevation) below ground surface, respectively.

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

Moisture content:

- 7.3% to 14.2%

Grain size distribution:

- 40% gravel,
- 39% sand, and
- 21% silt and clay size

Total saturated unit weights have been calculated based on the moisture contents and are estimated to range from about 22.0 to 24.0 kN/m^3. The results of the moisture content and grain size distribution tests are provided on the record of borehole sheet in Appendix C. The result of the grain size distribution tests also provided on Figure 4 in Appendix D.

1.4.5 **Clay with Silt**

A layer of clay with silt was encountered at BH105 sandwiched between two layers of silty sand with gravel, between the depth of 2.3 and 3.8 m (between Elev. 367.4 m and 365.9 m). The clay was generally described as firm to stiff, brown and moist. The SPT "N" values ranged between about 7 and 10 blows per 300 mm penetration.

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

Moisture content:

- 24.5% to 27.6%

Grain size distribution:

- 0% gravel,
- 16% sand,
- 36% silt, and
- 48% clay size.

Total saturated unit weights have been calculated based on the moisture contents and are estimated to range from about 19.5 to 20.0 kN/m^3. An Atterberg limit test was performed on a representative of the lean clay with sand (BH105-S5) and indicated that the clay is of medium plasticity. The data is shown on the plasticity chart, Figure 6. The liquid limit, plastic limit and plasticity index was about 44%, 20% and 24%, respectively.

The results of the moisture content, grain size distribution and Atterberg limit tests are provided on the record of borehole sheets in Appendix C. The results of the grain size distribution are also provided on Figure 5 in Appendix D, and Atterberg limit tests are provided on Figure 6 in Appendix D.

1.4.6 **Cobbles and Boulders**

Cobbles and boulders were encountered underlying the sand at BH101 and underlying the silt and peat at BH102. The cobbles and boulders were generally described as very dense, and in BH102 some sand, silt and gravel were encountered at about 7.3 m depth. The SPT "N" value of 100 (i.e. SPT refusal), per 300 mm penetration occurred in each of the two SPTs. The cobbles and boulders at BH101 and BH102 extended to depths of 10.3 m (364.5 m elevation) and 10.9 m (364.0 m elevation) below asphalt surface, respectively.

Laboratory testing performed on a select sample to determine moisture content. The test result is as follows:

Moisture content:

- 8.8%

1.4.1 **Bedrock**

As noted above rock coring was initiated within the cobbles and boulders strata, and rock coring techniques, NQ size, were continued at BH101 and BH102, to about 3.1 m and 3.4 m depths into bedrock, respectively. The bedrock elevations were about 364.5 m and 364.0 m, using geodetic benchmark.

The bedrock was generally described as a medium strong (25 MPa to 50 MPa compressive strength), very severely fractured to sound, white and pink, fine to coarse grained granite. Photographs of the bedrock core samples are presented in Appendix C, after the Borehole Logs.

Gross recoveries were about 100%. The Rock Quality Designation (RQD), which is a modified core recovery, ranged from 0% to 82% (very severely fractured to sound).

1.5 Groundwater and Surface Water Conditions

Information of 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
BH101	Feb. 28/15	Feb. 28/15	374.74	6.07	368.67
BH102	Feb. 27/15	Feb. 27/15	374.96	6.21	368.75
BH104	Mar. 23/15	Mar. 23/15	369.60	1.07	368.53
BH105	Mar. 20/15	Mar. 20/15	369.67	0.30	369.37
BH106	Mar. 23/15	Mar. 23/15	369.55	0.91	368.64
Cedar Creek WL Upstream (North) Side	--	Mar. 23/15	--	--	370.63[4]
Cedar Creek WL Downstream (South) Side	--	Mar. 23/15	--	----	369.50[4]

Notes:
1) All units in metres.
2) Elevations surveyed are referenced to a geodetic benchmark (BM) provided by the client (nail in tree southwest of the existing culvert). The elevation of the BM is 375.219 m,
3) Depths are relative to ground surface.
4) Indicates top of ice elevation at Cedar Creek.

1.6 Chemical Analysis of Soil

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

Table 1.2. Corrosivity Chemical Analysis

Borehole	pH (unitless)	Chloride (ppm)	Soluble Sulphate (ppm)	Resistivity (ohm-cm)	Conductivity (mS/cm)
BH101–S12	6.97	<20	<20	4,300	0.23
BH105–S4	7.79	<20	<20	4,100	0.24

PART II: ENGINEERING DISCUSSIONS AND RECOMMENDATIONS

2.1 General

This section of the report provides geotechnical design recommendations for rehabilitation/replacement of the existing Cedar Creek Culvert No. 1 situated beneath Hwy 590, located approximately 8.0 km west of the junction of Highway 590 and Highway 11/17 at Cedar Creek, in the Township of O'Connor. 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,** and the existing geotechnical data presented in the reports provided by MTO (see Section 1.3.3). The interpretation and recommendations provided are intended solely to permit designers to assess foundation alternatives and design the proposed 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 rigid frame open footing/retaining wall concrete structure with a span of 7.2 m, depth of 1.8 m and a length of about 24.5 m, and provides flow from north to south under Hwy 590. It is understood that the existing culvert was constructed at an unknown date, and inspected in June 2013. The inspection remarked that the existing culvert was in fair condition with significant deterioration of the retaining walls. Some vertical cracks in the both walls of the barrel were observed. It is also understood that the existing culvert will be ether rehabilitated (new wingwalls and headwalls and repairs at the construction joints) or replaced with a new culvert. In the case of replacement, 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 firmly defined at the time of writing this report. However for preliminary design purposes, the following options are being considered for the replacement: close bottom pre-cast concrete box culvert, open bottom concrete box culvert, open bottom concrete arch, open bottom corrugated steel plate arch culvert, and steel sheet pile abutment with precast concrete decking. For the case of rehabilitation of the existing culvert, it is understood that new wingwalls and headwalls will be constructed likely using RSS walls.

This part of the report addresses the geotechnical design of the foundation for the new culvert and roadway protection system 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 January 23, 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. In addition recommendations on roadway protection for full depth excavation for replacement of wingwalls and headwalls and for full depth repairs at the construction joints of the existing culvert are included.

2.2 Expected Ground Conditions

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

a. Hwy 590 is a two lane roadway, with a speed limit of 80 km/h and is about 7.0 m wide from edge of pavement to edge of pavement, with narrow sand and gravel shoulders. Based on drawings provided, the roadway embankment is about 6.0 m high with side slopes of about 2H:1V and gabion retaining walls at the culvert outlet . The current elevation of the crest of the roadway is about 374.90 m.
b. The highway embankment consists of layers of poorly graded gravel with silt and sand fill (~1.0 to 1.2 m thick), and silty sand to sandy silt fill (~4.1 m to 5.9 m thick) and sandy silt with gravel fill (~0.8 m). These fill layers extend to about 6.1 m to 6.9 m below the road surface.
c. The embankment fill is underlain by native dense silty sand with some clay, very dense silty sand with gravel and very dense/hard silt and peat to about 6.6 m to 8.7 m below ground surface followed by a layer of cobbles and boulders to about 10.3 m to 10.9 m below ground surface where the bedrock was encountered (~between Elev. 364.5 m and 364.0 m).
d. At the inlet, a surficial layer of silt and topsoil (~1.1 m thick) is underlain by very loose to compact silty sand with gravel (~1.2 m thick), and followed by stiff to firm clay with silt (~1.5 m thick) and very dense silty sand with gravel again. At the outlet, a surficial layer of topsoil (~0.1 to 0.8 m thick) is underlain by loose to very dense silty sand with gravel (~3.9 m thick) and very loose to loose silty sand with some peat (~1.0 thick). Thee practical refusal at inlet and outlet sides was at about 4.0 m below ground surface.
e. The foundation soil at the invert of the new culvert is anticipated to be native dense silty sand or very dense cobbles and boulders at approximately Elev. 368.0 m. Typical 'N' values ranged from 35 to 100.
f. At the time of investigation the water in the creek was frozen and approximate top of ice elevation was about 370.63 m at the inlet and 369.50 at the outlet. The groundwater table in the embankment fill is expected to be at approximate elevation 368.7 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

2.3.1 Structure/Foundation Options

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

- Closed bottom pre-cast concrete box culvert,
- Open bottom concrete box culvert,
- Open bottom concrete arch culvert,
- Open bottom corrugated steel plate (CSP) arch culvert, and
- Steel sheet pile abutment with precast concrete decking

Based on the subsurface information obtained from the site investigation, the native dense silty sand or very dense cobbles and boulders at approximately Elev. 368.0 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 bedrock was encountered relatively at shallow depth, steel sheet pile abutment with precast concrete decking is not a feasible option.

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

Table 2.1 Evaluation of culvert replacement alternatives

Options	Advantages	Disadvantages	Relative Costs	Risks/ Consequences	Rank
Closed bottom pre-cast concrete box culvert	▪ 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	1
Open bottom concrete box, concrete arch, and CSP arch	▪ 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	2

Areas spots of any loose and/or soft soils encountered below the existing culvert should be excavated and removed to firm bearing of native soils and the grade restored with engineered fill. If the depth of excavation to remove unstable soils is excessive, 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.

2.3.2 Foundation Recommendations

2.3.2.1 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

Founding Element	Founding Elevation (m)	Assumed Footing Size (m)	Founding Soil Type	Factored Geotechnical Resistance at ULS (kPa)	Geotechnical Reaction at SLS** (kPa)
Closed bottom pre-cast concrete box culvert	~368.0 or below	6.0 to 7.0	Minimum 300 mm compacted granular material (Granular A or Granular B Type II) over native dense silty sand or very dense cobbles and boulders	600	400
Open bottom concrete box, concrete arch, and CSP arch	~365.8* or bedrock level	1.0	Cobbles and boulders	900	500
Wingwalls and headwalls (RSS walls)	~368.5 or below	1.0	Native compact silty sand	300	200

Notes:
**Below the frost line, based on BH 101. Where the footing founded on bedrock requirement of full frost protection is not applicable. It is recommended to place minimum 100 mm thick layer of compacted Granular A beneath footings.*
*** for maximum settlement of 25 mm*

It is assumed that underlying peat/organic material 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.

The foundation layer consisting of cobbles and boulders could be uneven within the foundation footprint creating irregular bearing and possible higher differential settlement. To avoid any problems the entire area has to be thoroughly checked by a geotechnical engineer and a thin layer of lean concrete can be placed prior to constructing the final foundation. 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).

2.3.2.2 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 lean concrete on cobbles and boulders and cast-in-place concrete	Coefficient of friction (tan δ)=0.65

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

2.3.2.3 Frost Protection

The frost depth in the area of the culvert is estimated to be approximately 2.2 m in accordance with OPSD 3090.100. During construction of any temporary and permanent support system using shallow foundations should be provided a minimum 2.2 m of soil cover or equivalent frost protection should be provided using thermal insulation.

2.3.3 Lateral Earth Pressure

Culvert wingwalls and headwalls 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 2.4 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 2.4 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)
Gravel with Silt and Sand Fill	33°	0.29	3.39	0.46	21
Silty Sand to Sandy Silt Fill	30°	0.33	3.00	0.50	20
Silt and Peat	20°	0.49	2.04	0.66	17
Silty Sand with Gravel	32°	0.31	3.25	0.47	20
Silty Sand	30°	0.33	3.00	0.50	20
Clay with Silt	26°	0.39	2.56	0.56	19

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.

2.4 Construction Considerations

2.4.1 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 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 2.3 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 2.3 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 be 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	1
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 creek water flow • 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 6.0 m high required to maintain one lane of traffic • High cost of roadway protection system • Large amount of soil to be excavated • Need to temporarily control existing creek water • Risk of cost overrun and instability to finish job: low to moderate	Relatively more expensive than full road closure due to high costs of roadway protection system	2
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 6.0 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 creek water • Risk of cost overrun and instability to finish job: low to moderate	More expensive than full road closure and other open cut sides approach due to high costs for shoring system and temporary decking (if feasible) to maintain continuous flow of traffic	3

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 1: Full road closure using existing roadways and open cut unsupported excavation (Figure H1, Appendix H).

2. OPTION 3.A: Half-and-half construction with unsupported cut sides (Figure H3.A, Appendix H)

3. OPTION 3.B: Half-and-half construction with braced or anchored cut sides (Figure H3.B, 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.

2.4.1.1 **Detour Options (Options 1 and 2)**

Both detour options, the option with full closure of Hwy 590 and long detours around the area using existing roadways (see Figure H1, Appendix H), and the option with the local detour embankment construction at the site to maintain the local flow of traffic during the replacement (see Figure H2, Appendix H), allow for open cut, unsupported excavation to facilitate the replacement of the existing culvert. A major benefit of these options is that the existing culvert will be completely removed and replaced new culvert. The other advantages are that neither excavation support nor roadway protection is required with these options. The major disadvantages of both options are traffic interruption, large amounts of excavated soils and need for temporary construction unwatering and dewatering systems (i.e. cofferdams, and sumps and pumps, etc.) to prevent existing creek water and groundwater flow into the construction area which is the responsibility of the contractor.

All excavations at this site must be conducted in accordance with the Occupational Health and Safety Act (OHSA) and Regulations for Construction (O. Reg. 213/91). All fills (i.e. gravel with sand and silt and silty sand to sandy silty fills) may be classified as a Type 3 soil above the groundwater table in conformance with the OHSA. The native soils below the groundwater table may be classified as a Type 4 soil. It is expected that most of excavations will be above the groundwater levels except those at the invert level. To avoid disturbance of the founding subgrade and to allow placement of backfill in dry conditions, groundwater must be controlled to below the proposed invert excavation levels prior to digging to final levels. As mentioned before, the ingress of surface water must be controlled using a suitable system as well.

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

The detour construction alternative would involve construction of a temporary on-site embankment at the one side of the existing embankment depending on the available space and suitable terrain. Compacted engineered fill for construction of the temporary detour road is recommended. Prior to construction of the temporary detour embankment, the site will need to be cleared and grubbed of any existing bushes and vegetation. All surficial topsoil (if exists), organics and softened or loosened soil should be stripped 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).

2.4.1.2 **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 6.0 m deep. Therefore, temporary shoring such as a soldier pile and lagging system will be required as a roadway protection system to allow staging excavation/construction. It will be the Contractors responsibility to design a suitable temporary support system for the MTO review prior to installation. The Contractor is to follow OPSS 902, regarding excavations for structures, and OPSS.PROV 539, regarding temporary protection systems. Recommendations for a temporary roadway protection are given in Section 2.5.Using the half-and-half construction approach, several methods of culvert replacement were considered as discussed below:

A. Construction using roadway protection and unsupported excavation of cut sides

B. Construction using roadway protection and braced or anchored cut sides

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

2.4.1.2.1 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.

2.4.1.2.2 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 Temporary Roadway Protection

Temporary roadway protection is anticipated to be required to facilitate the full depth excavation for replacement of wingwalls and headwalls and for full depth repairs at the construction joints during the culvert rehabilitation. 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 cobles and boulders 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 2.3.3. 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 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 2.2 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 silty sands. 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 clean 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.

2.6 Culvert Backfill

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

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

All granular backfill materials should be placed in thin lifts (i.e. not exceeding 300 mm before compaction) and each lift should be compacted to at least 95% of the material's SPMDD (Standard Proctor Maximum Dry Density). The final lift of embankment fill prior to placing pavement sub-base should be compacted to 100 % SPMDD. The Granular A base and Granular B sub-base courses (for pavement) should be compacted to 100% of the material's SPMDD.

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

For fills immediately below any roadway, it is recommended that Granular A or B aggregates be used. Where necessary, proper frost tapering, in accordance with the OPSD 803.030 and 803.031 should be provided. Below a depth of about 1.5 m from any finished road grade, approved compactable fill, such as select subgrade materials (SSM) can be used.

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

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

2.7 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 condition at the location of cofferdam at the inlet side consists of a layer of very loose to compact silty sand with gravel (~1.2 m thick) underlain by the layers of firm to stiff clay with silt (~1.5 m thick), and very dense silty sand with gravel (~0.2 m thick). The practical refusal is encountered at Elev. 365.7 m (~4.0 m below the ground surface). At the outlet side, the subsurface condition at the location of cofferdam consists of very loose to loose silty sand with gravel (~3.9 m thick) and/or very loose to loose silty sand with some peat underlain be a layer of loose to very dense silty sand with gravel (~2.1 m thick). The practical refusal at this location is at Elev. 365.6 m (~4.0 m below the ground surface). Therefore, based on these geotechnical conditions (i.e. shallow practical refusal and possible 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 extend at least one meter above the creek water level at the time of construction. 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 silty sand fill and native silty sands. The native materials are susceptible to disturbance from groundwater and mobilized equipment. The groundwater level needs to be controlled to at least 0.5 m below the excavation level to avoid disturbance, and any surface or groundwater seepage should be removed from the excavation prior to the culvert bedding material placement of granular backfill in the dry. In general, pumping using properly filtered sumps, and/or filtered drains placed along the base of the excavation should provide sufficient groundwater control during foundation works.

Dewatering requirements behind the cofferdams to keep the construction site dry will be impacted by water levels in the creek at the time of construction activities. Dewatering shall be carried out in accordance with OPSS 517 and 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.

2.8 Embankment Design

2.8.1 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 mostly non cohesive. However, a settlement of about 25 mm should be allowed for due to rebound during the construction.

2.8.2 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 was established based on **exp**'s survey data and the topographic plan provided by MTO. Based on drawings provided, the roadway embankment is about 6.0 m high with side slopes of about 2H:1V.

The stratigraphy and groundwater condition at the site were developed based on the results of the geotechnical investigation presented in Part I - Foundation Investigation Report.

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

The SLOPE/W graphical printout, for analysis performed is included in Appendix F. Since the geometry and soil stratigraphy at the 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 2.5 are the soil parameters used for the slope stability analysis. The soil parameters were generally estimated based on the results of field and laboratory investigation.

Table 2.5 Soil properties used in slope stability analysis

Material	ϕ' (degrees)	c' (kPa)	γ' (kN/m^3)
Engineered Fill	32	0	21
Topsoil/Silt	17	0	18
Silt and Peat (Soft)	20	0	17
Silty Sand with Gravel (Very Loose to Compact)	32	0	20
Silty Sand (Dense)	30	0	19.5
Clay with Silt (Firm to Stiff)	26	0	19
Silty Sand with Gravel (Very Dense)	35	0	21
Cobbles and Boulders (Very Dense)	38	0	22

The results of slope stability analyses are shown on Figures F1 and F2 in Appendix F. As can be see, the minimum factor of safety of 1.3 could be achieved if the new granular embankment having slopes of 2H:1V is properly constructed using standard construction practice.

2.9 Retaining Walls

It is proposed that wingwalls and headwalls of the culvert will be constructed as RSS wall systems. In RSS wall systems the earth behind the wall is stabilized and included in the mechanism for resisting the lateral loads of the native ground. Typically, mechanical stabilization of wall backfill is achieved by the following:

- Placing and compacting a layer of earth backfill (typically 0.3 m to 0.6 m in thickness);
- Laying steel straps, steel wire grids, or plastic grids (geogrids) on the surface of the backfill layer as reinforcing elements;
- Attaching a structural face (typically consisting of interlocking concrete blocks or panels) to the reinforcing elements;
- Placing and compacting additional backfill on top of the reinforcing elements; and
- Repeating the above sequence until a structural face is provided to the required height, with the mass of earth stabilized with internal "reinforcement" behind the face.

Many systems are available for constructing mechanically stabilized earth wall system on the market (Terrafix Geosynthetics, Reinforcement Earth Company, Lock + Load, etc.). These systems are often patented with respect to the method of earth reinforcement, attachment of the reinforcement to the structural face, and/or the structural face finish and interlocking mechanisms.

RSS walls offer the advantage that they are relatively inexpensive and rapid to construct and, depending on the wall facing units, can be more tolerant of differential settlements than cast-in-place concrete walls. Since RSS systems relay on both the backfill soil and reinforcing elements for support, excavation into reinforced zone behind the wall must be restricted, providing a constraint on future infrastructure construction behind such walls.

The global stability of a typical RSS retaining wall which would be used in this project was analyzed using Slope/W. The soil conditions and the geometry of assumed retaining RSS walls used in the analyses are shown on Figures F3 and F4 in Appendix F. It has been assumed that the width of granular backfill will be approximately 0.8 of the wall height. Figures F3 and F4 also shows the results of the analyses. It can be seen that a factor of safety of 1.3 against global stability can be achieved, which is considered acceptable under static loading conditions.

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 submitted to Maxxam Analytics Inc., a CALA-certified and accredited laboratory in Mississauga, Ontario, for analyses of pH, water soluble sulphate and chloride concentrations, and resistivity. The testing was completed to determine the potential for degradation of the concrete in the presence of soluble sulphate and the potential for corrosion of exposed steel used in foundations and buried infrastructure. The analysis results are summarized in Table 1.2.

Similar to our experience with the soils in the Thunder Bay 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 chloride content is <20 ppm (μg/g) i.e. <0.002% which indicates a low potential for additional corrosion. The soil pH was about 7.4 (average) which is within what is considered the normal range for soil pH of 5.0 to 9.0. Therefore, the pH levels of the tested soils do not indicate a highly corrosive environment. The test results in Table 1.2 may be used to aid in the selection of coatings and corrosion protection systems for buried steel objects.

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

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,

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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. Inlet side of existing culvert north side of Highway 590

Photo 2. Outlet of existing culvert at south side of Highway 590

Photo 3. Facing west on Hwy 590 before the existing culvert

Photo 4. Facing east on Hwy 590 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. 2
GWP 6347-14-00

CEDAR CREEK CULVERT #1

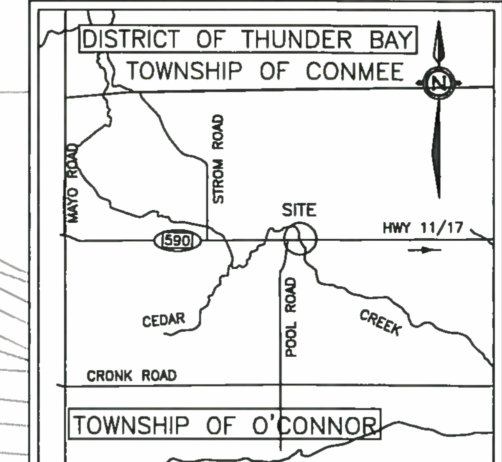
(Highway 590, O'Connor Township)

PLAN

DWG 1

exp Services Inc.

KEY PLAN



LEGEND

BH101 374.74 BOREHOLE LOCATION GROUND SURFACE ELEVATION IN METRES

BH No.	APPROX. ELEV. (m)	MTM COORDINATES	
		NORTH	EAST
BH101	374.74	5,362,184	324,731
BH102	374.96	5,362,180	324,747
BH104	369.60	5,362,163	324,752
BH105	369.67	5,362,196	324,740
BH106	369.55	5,362,159	324,759

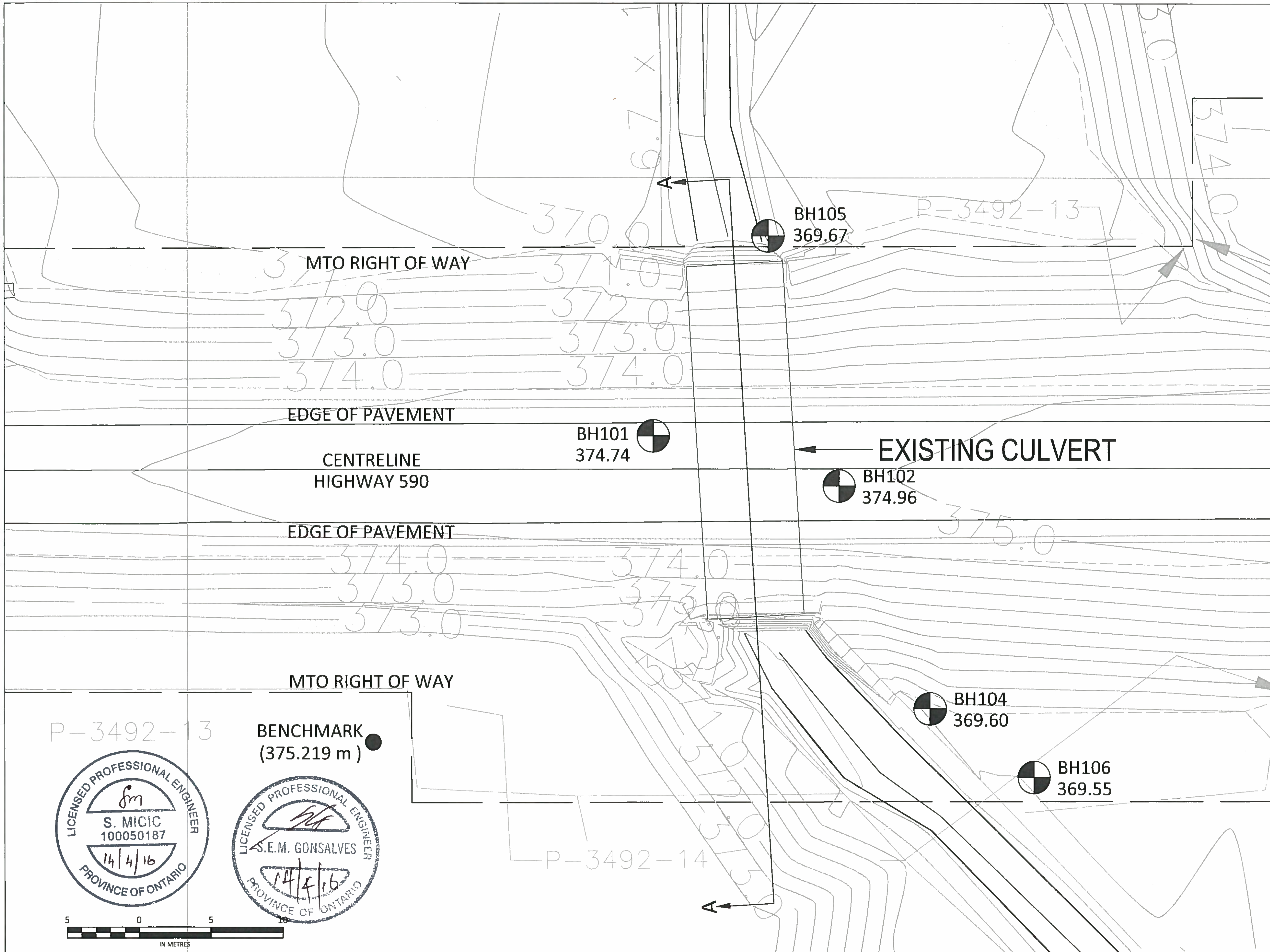
NOTES

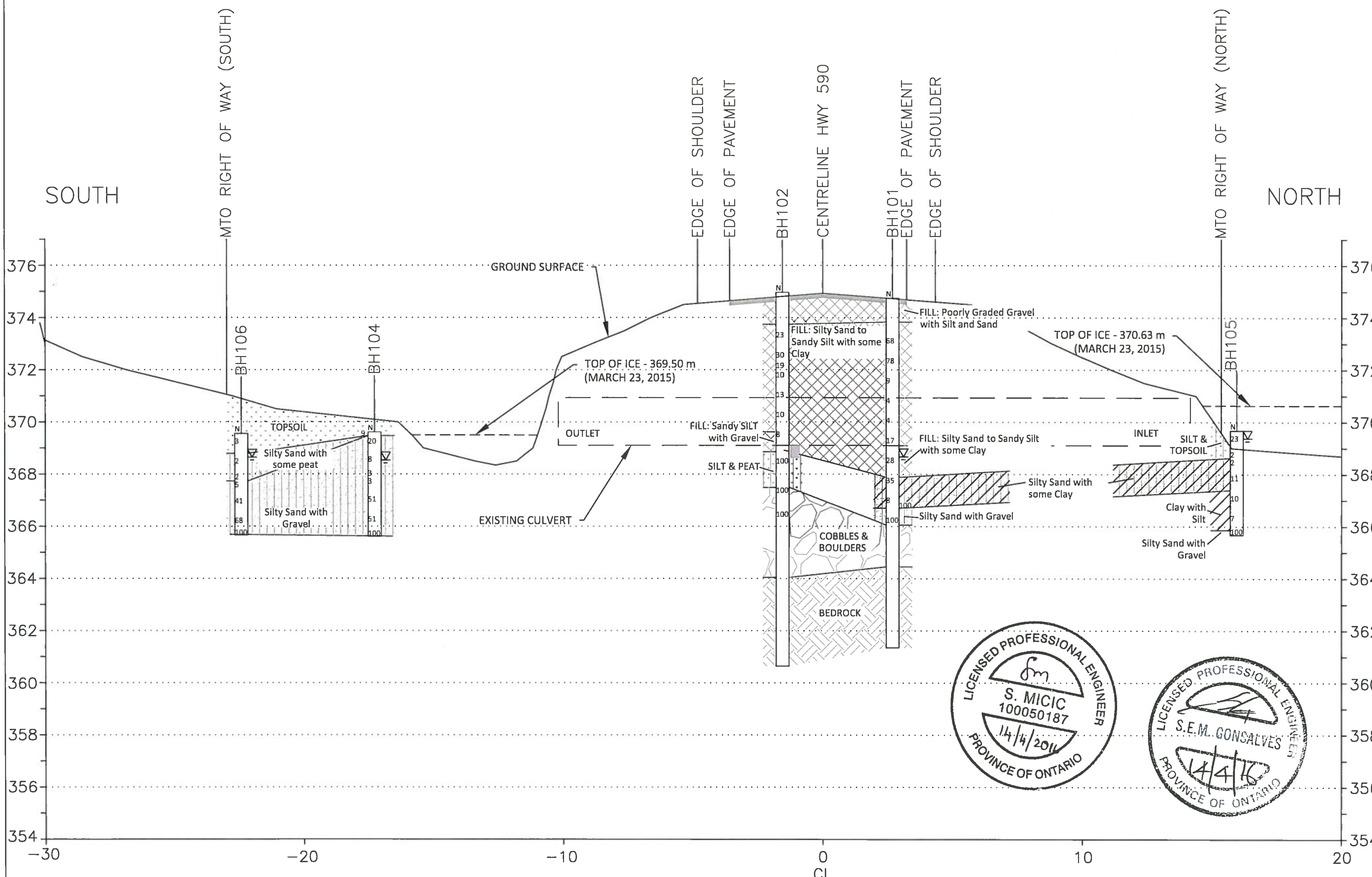
1. ALL DIMENSIONS ARE IN METRES.
2. BASE MAP PROVIDED BY CLIENT.
3. BOREHOLE LOCATIONS ARE BASED ON FIELD MEASUREMENTS FROM EXISTING CULVERT AND/OR PROJECTED MTM COORDINATES FOR ZONE ON-15 AS PER PROVIDED FIGURE.
4. THIS DRAWING IS FOR SUBSURFACE INFORMATION ONLY. THE PROPOSED STRUCTURE DETAILS/WORKS ARE SHOWN FOR ILLUSTRATION PURPOSES ONLY.

REVISIONS

DATE	BY	DESCRIPTION

GEOCRES No. 52A-214	Project No. ADM-00223648-B0
Date: April 10, 2015	Scale : 1:250
Drawn By: RM	Checked By: AM
	Checked By: DG





A–A

PROFILE OF CEDAR CREEK CULVERT #1

3 0 3 6

IN METRES

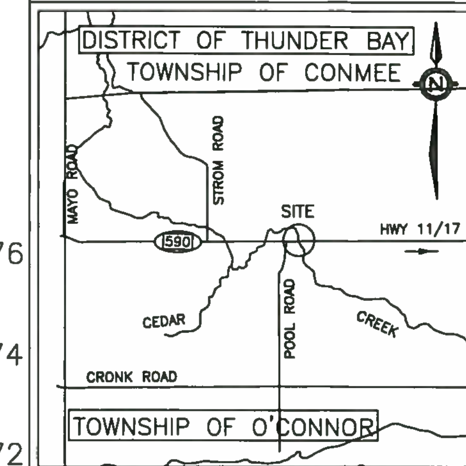
Agreement No. 6014-E-0017
Assignment No. 2
GWP 6347-14-00

CEDAR CREEK CULVERT #1
(Highway 590, O'Connor Township)
CROSS SECTION A-A

DWG 2

exp. **exp** Services Inc.

KEY PLAN



LEGEND

N — STANDARD PENETRATION TEST (BLOWS/0.3 m)

∇ — MEASURED WATER LEVEL

SOIL STRATA SYMBOLS

FILL; SILT & PEAT; SILTY SAND with Clay; COBBLES & BOULDERS; BEDROCK

BH No.	APPROX. ELEV. (m)	MTM COORDINATES NORTH	EAST
BH101	374.74	5,362,184	324,731
BH102	374.96	5,362,180	324,747
BH104	369.60	5,362,163	324,752
BH105	369.67	5,362,196	324,740
BH106	369.55	5,362,159	324,759

NOTES

1. ALL DIMENSIONS ARE IN METRES.
2. BASE MAP PROVIDED BY CLIENT.
3. MTM COORDINATES BASE ON MTM ZONE ON-15 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. 52A-214	Project No. ADM-00223648-B0
Date: May 15, 2015	Horzontal 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:

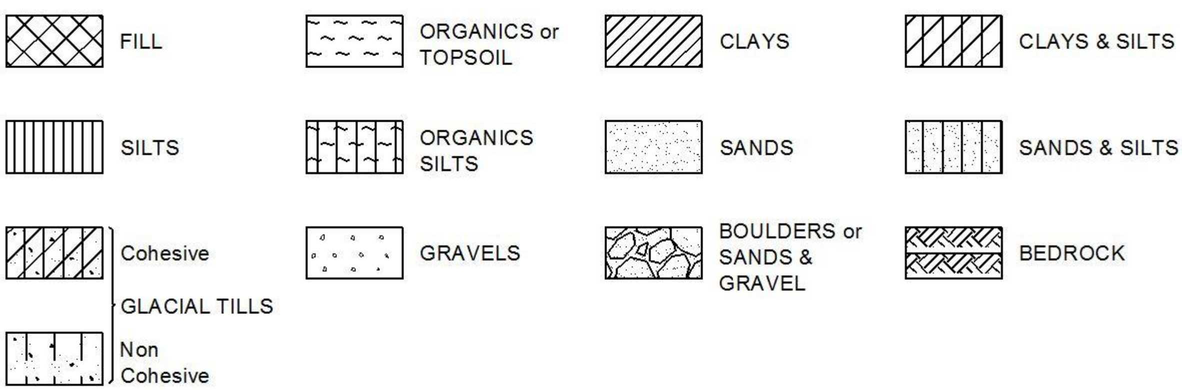
Table c: Consistency of Cohesive Soil

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 BH101

METRIC

W. P.	GWP No. 6347-14-00	LOCATION	Cedar Creek Culvert #1 (Site No. 48W-154/C) MTM ON-15 324,730 E 5,362,184 N	ORIGINATED BY	EF
DIST	Thunder Bay, Hwy 590	BOREHOLE TYPE	CME 850 Track Carrier / HSA / NQ	COMPILED BY	AM
DATUM	Geodetic	DATE	2015/02/26 - 2015/02/28	CHECKED BY	DG

Column headings: SOIL PROFILE (ELEV / DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE; DNYAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH: Cu, 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 (%) (10, 20, 30); UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) (GR SA SI CL)

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
374.7	Asphalt				
374.6 / 0.2	**ASPHALT** - about 180 mm **Poorly Graded GRAVEL with Silt and Sand (GP-GM) Fill** - frozen, brown	S1	AUGER		
		S2	AUGER		
		S3	AUGER		
373.8 / 1.0	**SILTY SAND TO SANDY SILT with some Clay (SM-ML) Fill** - very loose to very dense, frozen, brown, trace to little gravel	S4	AUGER		
		S5	SS	68	12 55 (33)
	- trace asphalt pieces at 2.3 m depth	S6	SS	78	
		S7	SS	9	
	- becoming very loose to loose, moist at about 3.8 m depth	S8	SS	4	4 34 47 15
	- becoming grey, moist to wet, trace to some wood pieces at about 4.6 m depth	S9	SS	4	
		S10	SS	17	
	- grey, moist to wet, some wood pieces	S11	SS	28	12 54 (34)
367.9 / 6.9	**SILTY SAND with some Clay (SM)** - dense, grey to brown, wet to moist	S12	SS	35	11 51 (38)
		S13A	SS	8	
366.7 / 8.0	- refusal to SPT at about 8.0 m depth, rock coring initiated at about 8.0 m depth	S13B	SS	100	
			CORE		
366.1 / 8.7	**Silty SAND with Gravel (SM)** - very dense, light brown, wet	S14	SS	100	32 49 (19)
	COBBLES AND BOULDERS	S15	CORE		
364.5 / 10.3	**BEDROCK** - medium strong, very severely fractured, white and pink, fine to coarse grained, granite				Recovery=100%, RQD=0%
	- becoming fractured to sound at about 10.5 m depth	S16	CORE		Recovery=100%, RQD=75%
	- becoming fractured at about 12.0 m depth	S17	CORE		Recovery=100%, RQD=43%
361.4 / 13.4	**END OF BOREHOLE**				

Elevation scale: 374, 373, 372, 371, 370, 369, 368, 367, 366, 365, 364, 363, 362

NOTES:
1. This drawing is to be read with the subject report and project numbers as presented above.
2. Groundwater level at 6.1 m depth upon completion of borehole.

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

RECORD OF BOREHOLE No BH102

METRIC

W. P. GWP No. 6347-14-00	LOCATION Cedar Creek Culvert #1 (Site No. 48W-154/C) MTM ON-15 324,747 E 5,362,180 N	ORIGINATED BY EF
DIST Thunder Bay, Hwy 590	BOREHOLE TYPE CME 850 Track Carrier / HSA / NQ	COMPILED BY AM
DATUM Geodetic	DATE 2015/02/27 - 2015/02/27	CHECKED BY DG

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
375.0	Asphalt
374.8 / 0.2	**ASPHALT** - about 180 mm **Poorly Graded GRAVEL with Silt and Sand (GP-GM) Fill** - frozen, brown, occasional cobbles
373.7 / 1.2	**SILTY SAND TO SANDY SILT with some Clay (SM-ML) Fill** - compact, frozen, brown, few to little gravel
369.6 / 5.3	**SANDY SILT with Gravel (ML) Fill** - compact, moist to wet, brown to grey, trace organics
368.9 / 6.1	**SILT AND PEAT** - very dense/hard, dark brown to grey, wet, trace sand, trace gravel - refusal to auger and SPT at about 6.6 m depth, rock coring was initiated at about 6.6 m depth
368.4 / 6.6	**COBBLES AND BOULDERS** - very dense - some sand, some silt, trace gravel, light brown, wet at about 7.5 m depth
364.0 / 10.9	**BEDROCK** - medium strong, fractured, white and pink, fine to coarse grained, granite - becoming sound at about 11.4 m depth - becoming fractured at about 12.9 m depth
360.6 / 14.3	**END OF BOREHOLE**

SAMPLES

NUMBER	TYPE	"N" VALUES	NATURAL MOISTURE CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
S1	AUGER			
S2	AUGER			73 21 (6)
S3	AUGER			
S4	AUGER			
S5	SS	23		10 38 36 16
S6A	SS	30		
S6B	SS	19		
S7	SS	10		7 30 44 19
S8	SS	13		
S9	SS	10		
S10	SS	8		23 25 44 8
S11	SS	100	59.7	
S12	SS	100		
S13	SS	100		
S14	CORE			
S15	CORE			
S16	CORE			Recovery=100%, RQD=59%
S17	CORE			Recovery=100%, RQD=82%
S18	CORE			Recovery=100%, RQD=56%

Ground water conditions: groundwater level symbol at about elevation 368.8.

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE (374 to 361); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH: Cu, 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 (%) 10 20 30; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

NOTES:
1. This drawing is to be read with the subject report and project numbers as presented above.
2. Groundwater level at 6.2 m depth upon completion of borehole.

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

RECORD OF BOREHOLE No BH104

METRIC

W. P.	GWP No. 6347-14-00	LOCATION	Cedar Creek Culvert #1 (Site No. 48W-154/C) MTM ON-15 324,752 E 5,362,163 N
DIST	Thunder Bay, Hwy 590	BOREHOLE TYPE	CME 45 Yanmar Track Mount / HSA
DATUM	Geodetic	DATE	2015/03/23 - 2015/03/23

ORIGINATED BY EF
COMPILED BY RM
CHECKED BY AM/DG

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	GR	SA	SI	CL
369.6	Topsoil										
369.5 0.1	**TOPSOIL** - frozen, brown, trace sand, some silt	S1A	SS	9			78.4				
	SILTY SAND with Gravel (SM) - frozen, brown, some roots & rootlets	S1B	SS	20		369					
	- becoming loose, wet, some peat at about 0.8 m depth	S2	SS	8	∇		69.6	8	63	(29)	
	- becoming grey, moist clayey silt	S3A	SS	3		368					
		S3B	SS	3							
	-becoming sand and gravel	S4	SS	51		367		40	39	(21)	
		S5	SS	51		366					
365.6		S6	SS	100							
4.0	**END OF BOREHOLE** - refusal to SPT and auger										

NOTES:
1. This drawing is to be read with the subject report and project numbers as presented above.
2. Groundwater level at 1.1 m depth upon completion of borehole.

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; DNYAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH: Cu, 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 (%) 10 20 30; UNIT WEIGHT γ kN/m^3; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

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

RECORD OF BOREHOLE No BH105

METRIC

W. P.	GWP No. 6347-14-00	LOCATION	**Cedar Creek Culvert #1 (Site No. 48W-154/C) MTM ON-15 324,740 E 5,362,196 N**	ORIGINATED BY	EF
DIST	Thunder Bay, Hwy 590	BOREHOLE TYPE	CME 45 Yanmar Track Mount / HSA	COMPILED BY	RM
DATUM	**Geodetic**	DATE	2015/03/20 - 2015/03/20	CHECKED BY	AM/DG

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20, 40, 60, 80, 100
SHEAR STRENGTH: Cu, 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 (%): 10, 20, 30
UNIT WEIGHT γ kN/m³
REMARKS & GRAIN SIZE DISTRIBUTION (%): GR SA SI CL

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
369.7	Topsoil
	SILT & TOPSOIL - frozen, brown, some roots & rootlets, trace wood pieces - becoming soft, wet at about 0.8 m depth
368.6 / 1.1	**SILTY SAND with Gravel (SM)** - very loose to compact, grey, wet, trace peat in upper 1.5 m
367.4 / 2.3	**CLAY with Silt (CL)** - firm to stiff, brown, moist, some sand
365.9 / 3.8	**SILTY SAND with Gravel (SM)** - very dense, light brown, wet, some clay
365.7 / 4.0	**END OF BOREHOLE** - refusal to SPT and auger

SAMPLES

NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	Water content plot label	GR	SA	SI	CL
S1	SS	23	∇	213.9				
S2A	SS	2		101				
S2B	SS	2						
S3	SS	11			20	40	29	11
S4	SS	10						
S5	SS	7		44.2	0	16	36	48
S6	SS	100						

ELEVATION SCALE: 369, 368, 367, 366

NOTES:
1. This drawing is to be read with the subject report and project numbers as presented above.
2. Groundwater level at 0.3 m depth upon completion of borehole.

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

exp services Inc.
Brampton, Ontario

Foundation Design

RECORD OF BOREHOLE No BH106

METRIC

W. P. GWP No. 6347-14-00 | LOCATION **Cedar Creek Culvert #1 (Site No. 48W-154/C) MTM ON-15 324,759 E 5,362,159 N** | ORIGINATED BY EF

DIST Thunder Bay, Hwy 590 | BOREHOLE TYPE CME 45 Yanmar Track Mount / HSA | COMPILED BY RM

DATUM **Geodetic** | DATE 2015/03/23 - 2015/03/23 | CHECKED BY AM/DG

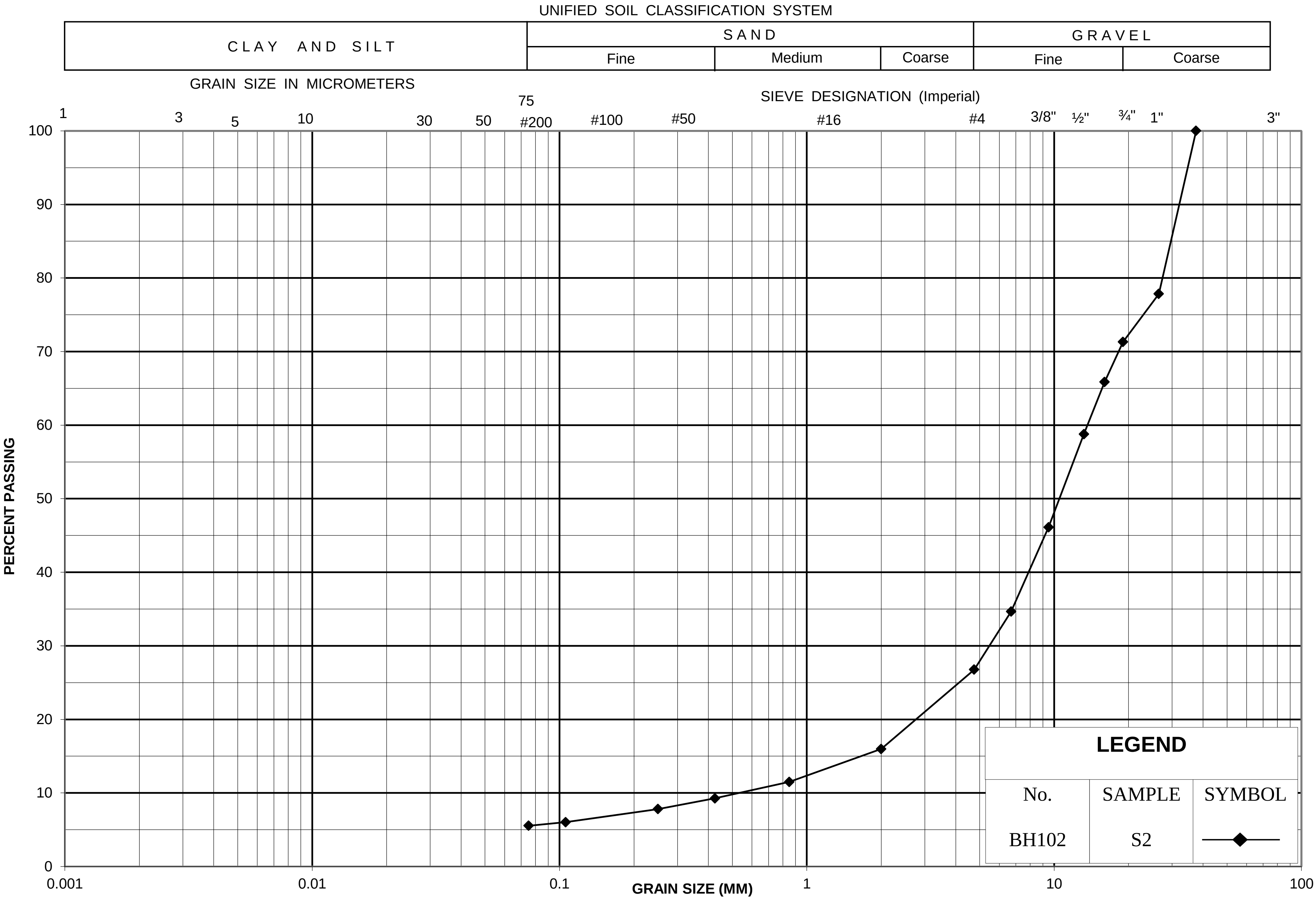
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
369.6	Topsoil											
	TOPSOIL - soft, brown, moist, some silt	S1	SS	3		369	66.5					
368.8 0.8	**SILTY SAND with some Peat (SM)** - very loose to loose, brown, wet, trace wood pieces	S2	SS	2	∇		50		7	65	(28)	
		S3A	SS	2		368	51.2					
367.7 1.8	**SILTY SAND with Gravel (SM)** - loose to very dense, brown, wet, some clay	S3B	SS	5								
		S4	SS	41		367						
		S5	SS	68		366						
365.7 3.9	**END OF BOREHOLE** - refusal to SPT and auger	S6	SS	100								

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; DNYAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH: Cu, 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 (%) 10 20 30; REMARKS & GRAIN SIZE DISTRIBUTION (%)

NOTES:
1. This drawing is to be read with the subject report and project numbers as presented above.
2. Groundwater level at 0.9 m depth upon completion of borehole.

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

Appendix D – Laboratory Data



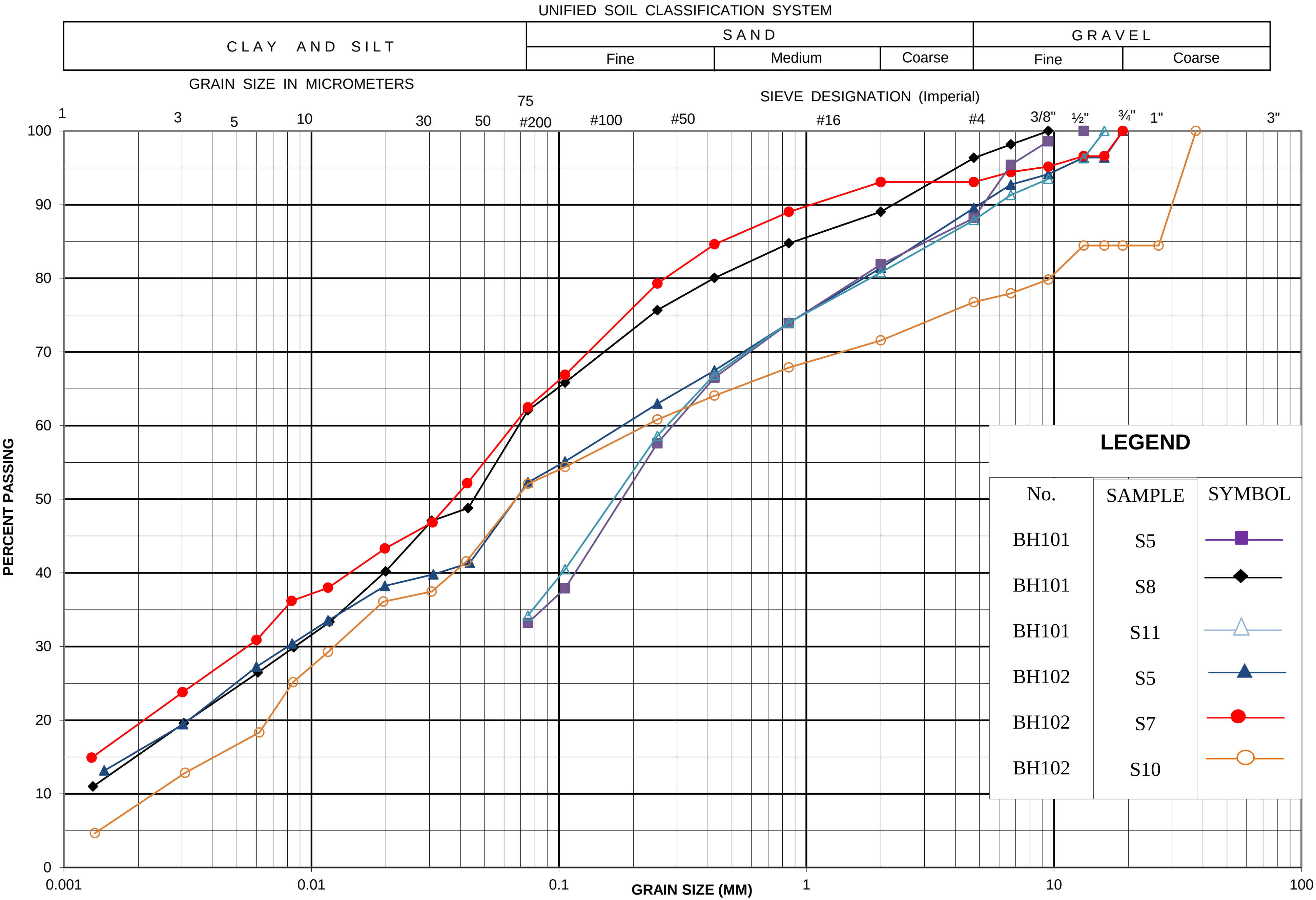
exp.

GRAIN SIZE DISTRIBUTION

Poorly Graded GRAVEL with Silt and Sand (GP-GM)

EXISTING CULVERT

FIGURE : No. 1	
GWP : 6347-14-00	
DATE : April 9, 2015	



No.	SAMPLE	SYMBOL
BH101	S5	
BH101	S8	
BH101	S11	
BH102	S5	
BH102	S7	
BH102	S10	

exp.

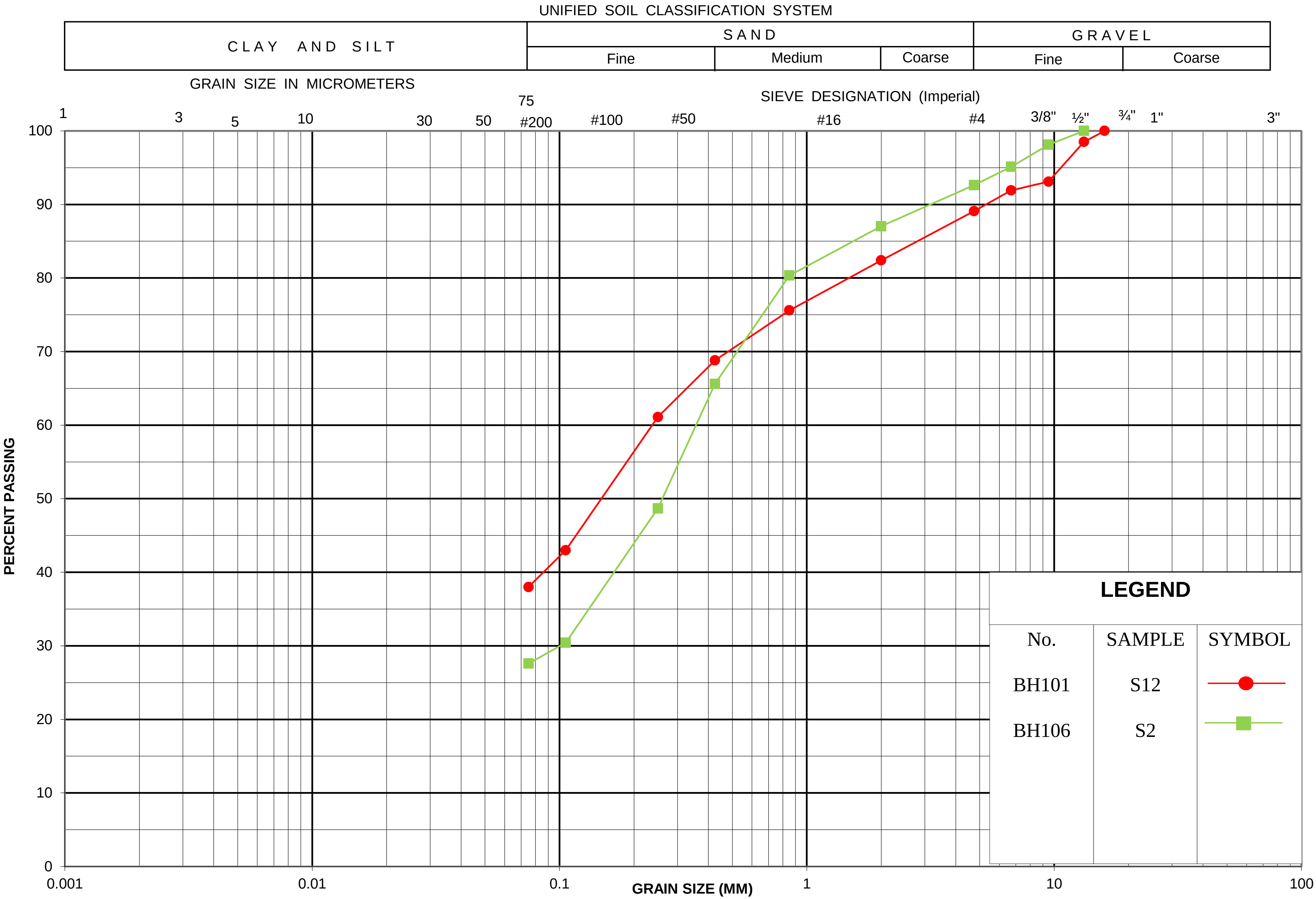
GRAIN SIZE DISTRIBUTION

Silty Sand to Sandy Silt : Fill - EXISTING CULVERT

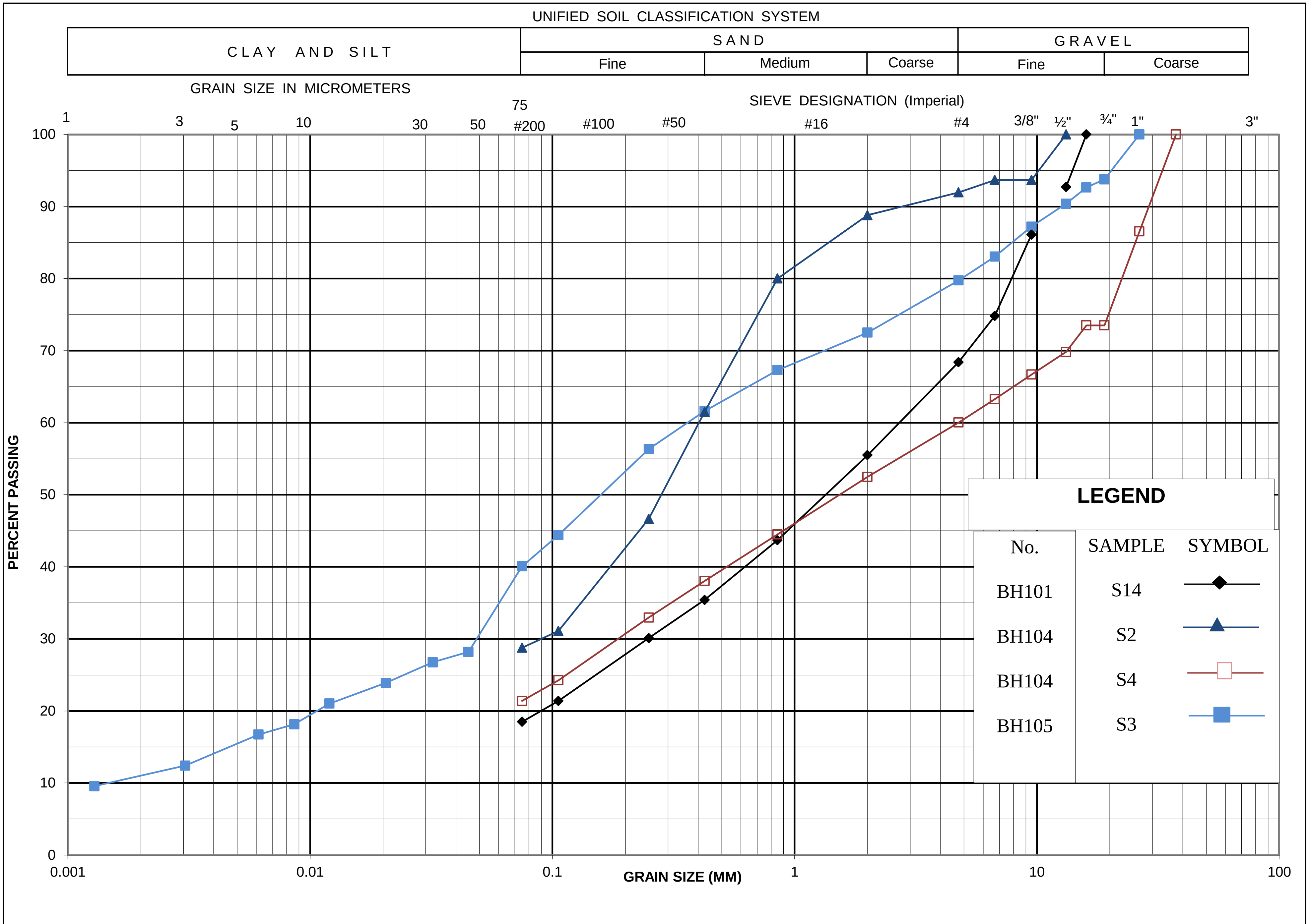
FIGURE : No. 2

GWP : 6347-14-00

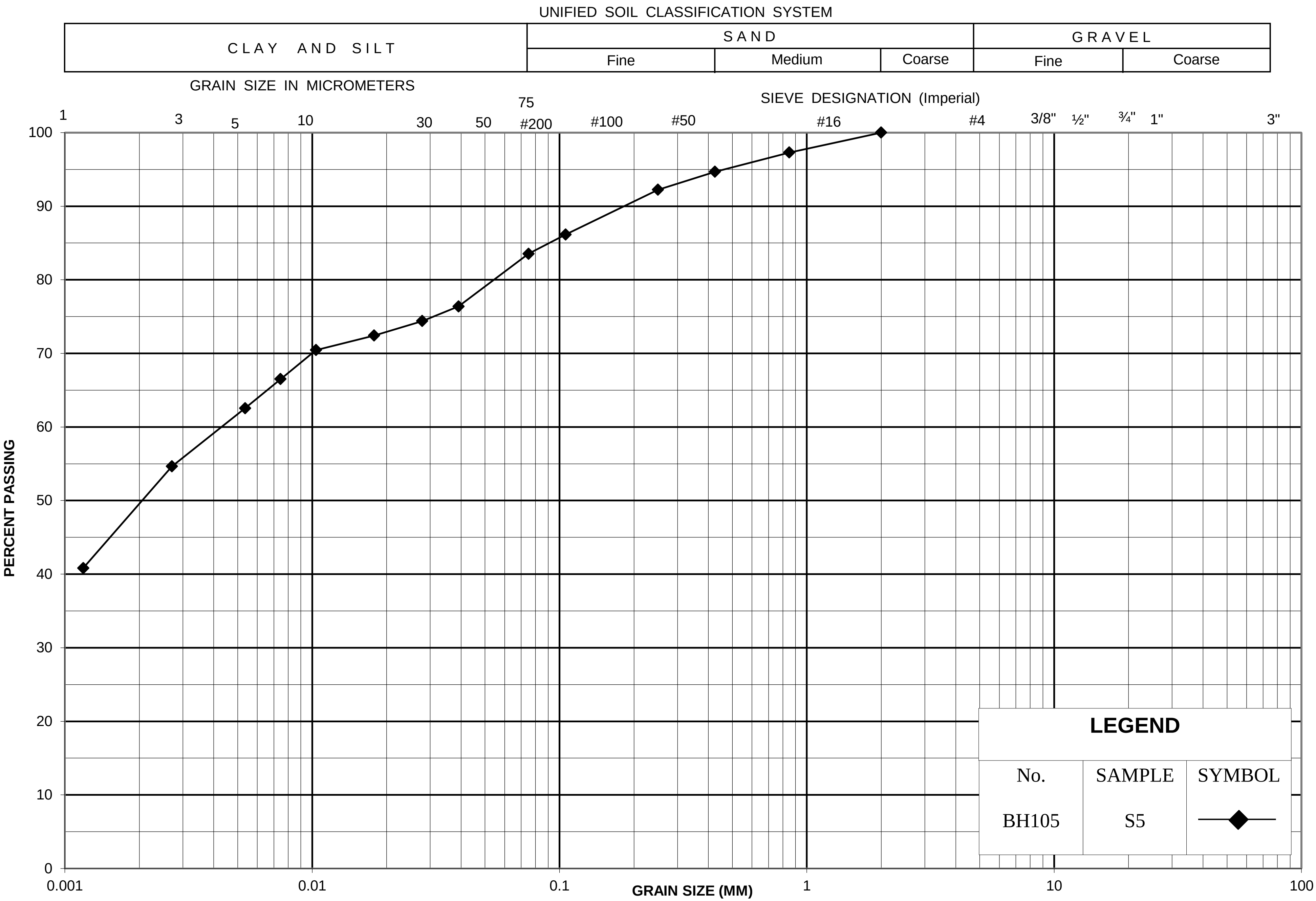
DATE : April 9, 2015



exp.	GRAIN SIZE DISTRIBUTION	FIGURE : No. 3
	Silty Sand (SM) - EXISTING CULVERT	GWP : 6347-14-00
		DATE : April 7, 2015

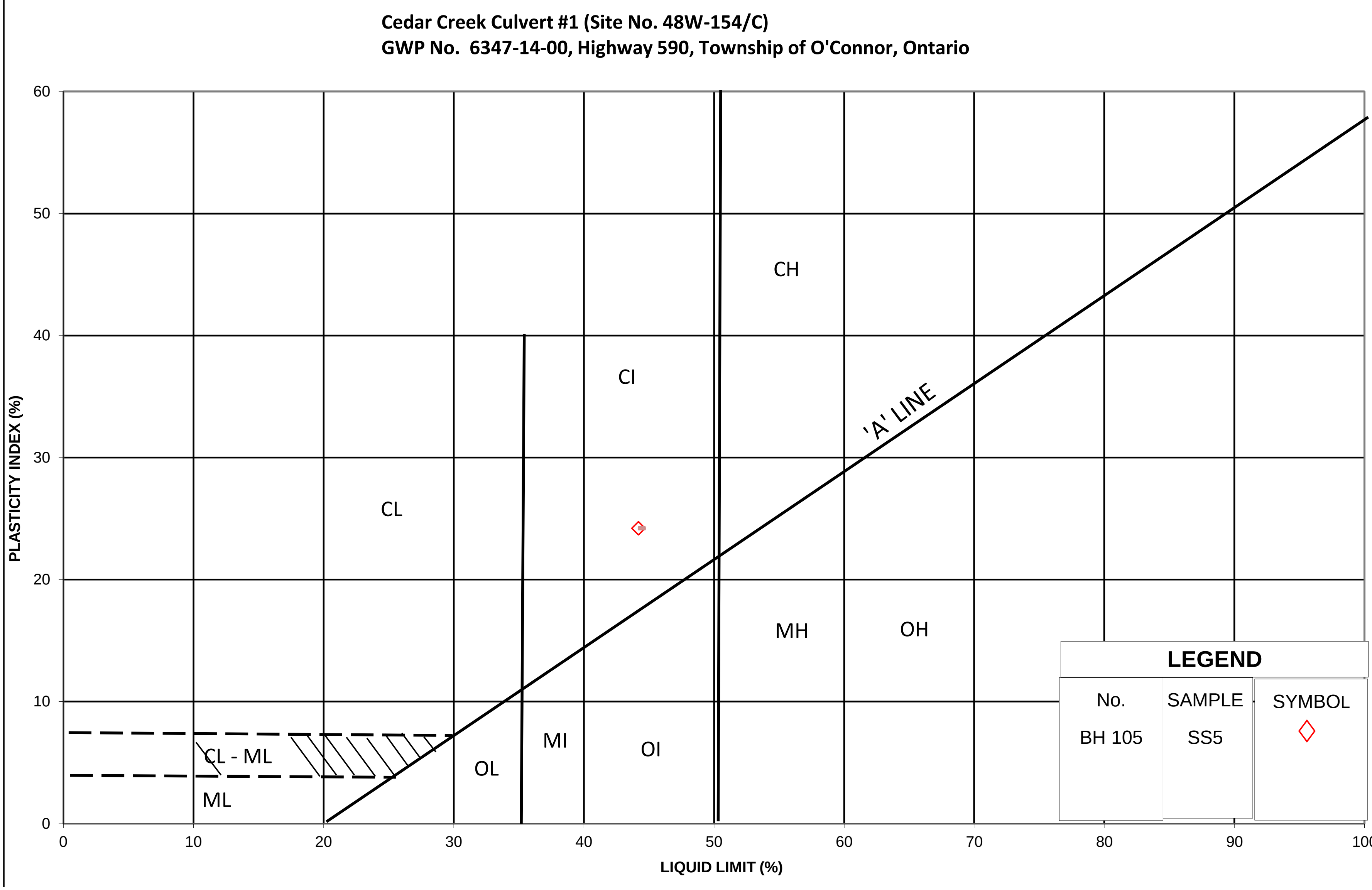


exp.	GRAIN SIZE DISTRIBUTION ***Silty SAND with Gravel (SM)*** - EXISTING CULVERT	FIGURE : No. 4 GWP : 6347-14-00 DATE : April 9, 2015



exp.	GRAIN SIZE DISTRIBUTION	FIGURE : No. 5
	CLAY with Silt (CL) - EXISTING CULVERT	GWP : 6347-14-00
		DATE : April 9, 2015

Plasticity Chart



Appendix E –
Chemical Analyses

Maxxam
A Bureau Veritas Group Company

Success Through Science®

Your Project #: ADM-00223648-B0
Site Location: HWY 590, KAKABEKA, ONTARIO
Your C.O.C. #: na

Attention:Ahileas Mitsopoulos

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

Report Date: 2015/04/01
Report #: R3378881
Version: 1 - Final

CERTIFICATE OF ANALYSIS

MAXXAM JOB #: B553991
Received: 2015/03/27, 10:00

Sample Matrix: Soil
Samples Received: 8

Analyses	Quantity	Date Extracted	Date Analyzed	Laboratory Method	Reference
Chloride (20:1 extract)	8	N/A	2015/04/01	CAM SOP-00463	EPA 325.2 m
Conductivity	8	N/A	2015/04/01	CAM SOP-00414	OMOE E3138 v2 m
pH CaCl2 EXTRACT	8	2015/03/31	2015/03/31	CAM SOP-00413	EPA 9045 D m
Resistivity of Soil	8	2015/03/27	2015/04/01	CAM SOP-00414	SM 22 2510 m
Sulphate (20:1 Extract)	8	N/A	2015/04/01	CAM SOP-00464	EPA 375.4 m

Remarks:

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

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

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

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

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

Your Project #: ADM-00223648-B0
Site Location: HWY 590, KAKABEKA, ONTARIO
Your C.O.C. #: na

Attention:Ahileas Mitsopoulos

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

Report Date: 2015/04/01
Report #: R3378881
Version: 1 - Final

CERTIFICATE OF ANALYSIS

MAXXAM JOB #: B553991
Received: 2015/03/27, 10:00

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 #: B553991
Report Date: 2015/04/01

exp Services Inc
Client Project #: ADM-00223648-B0
Site Location: HWY 590, KAKABEKA, ONTARIO

RESULTS OF ANALYSES OF SOIL

Maxxam ID		AAG172	AAG173	AAG174	AAG175	AAG176	AAG176		
Sampling Date		2015/02/26 15:00	2015/03/20 16:30	2015/03/04 11:15	2015/03/19 10:20	2015/03/11 14:40	2015/03/11 14:40		
COC Number		na	na	na	na	na	na		
	Units	**B101-S12**	**BH105-S4**	**BH202-S10/S11**	**BH203-S3**	**BH302-S10**	**BH302-S10 Lab-Dup**	**RDL**	**QC Batch**
Calculated Parameters									
Resistivity	ohm-cm	4300	4100	2400	5200	1500			3963203
Inorganics									
Soluble (20:1) Chloride (Cl)	ug/g	<20	<20	57	<20	370		20	3966279
Conductivity	mS/cm	0.23	0.24	0.42	0.19	0.69	0.69	0.002	3967584
Available (CaCl2) pH	pH	6.97	7.79	7.82	7.95	6.20		N/A	3965076
Soluble (20:1) Sulphate (SO4)	ug/g	42	36	240	50	<20		20	3966281

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

Maxxam ID		AAG177	AAG178	AAG179	AAG179		
Sampling Date		2015/03/18 11:25	2015/03/07 17:10	2015/03/17 10:00	2015/03/17 10:00		
COC Number		na	na	na	na		
	Units	**BH303-S4**	**BH402-S14**	**BH403-S3**	**BH403-S3 Lab-Dup**	**RDL**	**QC Batch**
Calculated Parameters							
Resistivity	ohm-cm	2400	3000	3100			3963203
Inorganics							
Soluble (20:1) Chloride (Cl)	ug/g	79	<20	<20	<20	20	3966279
Conductivity	mS/cm	0.42	0.33	0.32		0.002	3967584
Available (CaCl2) pH	pH	7.63	7.92	7.76	7.85	N/A	3965076
Soluble (20:1) Sulphate (SO4)	ug/g	140	190	170	150	20	3966281

RDL = Reportable Detection Limit
QC Batch = Quality Control Batch
Lab-Dup = Laboratory Initiated Duplicate
N/A = Not Applicable

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

TEST SUMMARY

Maxxam ID: AAG172
Sample ID: B101-S12
Matrix: Soil

Collected: 2015/02/26
Shipped:
Received: 2015/03/27

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	3966279	N/A	2015/04/01	Deonarine Ramnarine
Conductivity	AT	3967584	N/A	2015/04/01	Lemeneh Addis
pH CaCl2 EXTRACT	AT	3965076	2015/03/31	2015/03/31	Neil Dassanayake
Resistivity of Soil		3963203	2015/04/01	2015/04/01	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	3966281	N/A	2015/04/01	Deonarine Ramnarine

Maxxam ID: AAG173
Sample ID: BH105-S4
Matrix: Soil

Collected: 2015/03/20
Shipped:
Received: 2015/03/27

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	3966279	N/A	2015/04/01	Deonarine Ramnarine
Conductivity	AT	3967584	N/A	2015/04/01	Lemeneh Addis
pH CaCl2 EXTRACT	AT	3965076	2015/03/31	2015/03/31	Neil Dassanayake
Resistivity of Soil		3963203	2015/04/01	2015/04/01	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	3966281	N/A	2015/04/01	Deonarine Ramnarine

Maxxam ID: AAG174
Sample ID: BH202-S10/S11
Matrix: Soil

Collected: 2015/03/04
Shipped:
Received: 2015/03/27

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	3966279	N/A	2015/04/01	Deonarine Ramnarine
Conductivity	AT	3967584	N/A	2015/04/01	Lemeneh Addis
pH CaCl2 EXTRACT	AT	3965076	2015/03/31	2015/03/31	Neil Dassanayake
Resistivity of Soil		3963203	2015/04/01	2015/04/01	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	3966281	N/A	2015/04/01	Deonarine Ramnarine

Maxxam ID: AAG175
Sample ID: BH203-S3
Matrix: Soil

Collected: 2015/03/19
Shipped:
Received: 2015/03/27

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	3966279	N/A	2015/04/01	Deonarine Ramnarine
Conductivity	AT	3967584	N/A	2015/04/01	Lemeneh Addis
pH CaCl2 EXTRACT	AT	3965076	2015/03/31	2015/03/31	Neil Dassanayake
Resistivity of Soil		3963203	2015/04/01	2015/04/01	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	3966281	N/A	2015/04/01	Deonarine Ramnarine

Maxxam ID: AAG176
Sample ID: BH302-S10
Matrix: Soil

Collected: 2015/03/11
Shipped:
Received: 2015/03/27

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	3966279	N/A	2015/04/01	Deonarine Ramnarine
Conductivity	AT	3967584	N/A	2015/04/01	Lemeneh Addis

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
A Bureau Veritas Group Company

Maxxam Job #: B553991
Report Date: 2015/04/01

exp Services Inc
Client Project #: ADM-00223648-B0
Site Location: HWY 590, KAKABEKA, ONTARIO

TEST SUMMARY

Maxxam ID: AAG176
Sample ID: BH302-S10
Matrix: Soil

Collected: 2015/03/11
Shipped:
Received: 2015/03/27

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
pH CaCl2 EXTRACT	AT	3965076	2015/03/31	2015/03/31	Neil Dassanayake
Resistivity of Soil		3963203	2015/04/01	2015/04/01	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	3966281	N/A	2015/04/01	Deonarine Ramnarine

Maxxam ID: AAG176 Dup
Sample ID: BH302-S10
Matrix: Soil

Collected: 2015/03/11
Shipped:
Received: 2015/03/27

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Conductivity	AT	3967584	N/A	2015/04/01	Lemeneh Addis

Maxxam ID: AAG177
Sample ID: BH303-S4
Matrix: Soil

Collected: 2015/03/18
Shipped:
Received: 2015/03/27

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	3966279	N/A	2015/04/01	Deonarine Ramnarine
Conductivity	AT	3967584	N/A	2015/04/01	Lemeneh Addis
pH CaCl2 EXTRACT	AT	3965076	2015/03/31	2015/03/31	Neil Dassanayake
Resistivity of Soil		3963203	2015/04/01	2015/04/01	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	3966281	N/A	2015/04/01	Deonarine Ramnarine

Maxxam ID: AAG178
Sample ID: BH402-S14
Matrix: Soil

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

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	3966279	N/A	2015/04/01	Deonarine Ramnarine
Conductivity	AT	3967584	N/A	2015/04/01	Lemeneh Addis
pH CaCl2 EXTRACT	AT	3965076	2015/03/31	2015/03/31	Neil Dassanayake
Resistivity of Soil		3963203	2015/04/01	2015/04/01	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	3966281	N/A	2015/04/01	Deonarine Ramnarine

Maxxam ID: AAG179
Sample ID: BH403-S3
Matrix: Soil

Collected: 2015/03/17
Shipped:
Received: 2015/03/27

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	3966279	N/A	2015/04/01	Deonarine Ramnarine
Conductivity	AT	3967584	N/A	2015/04/01	Lemeneh Addis
pH CaCl2 EXTRACT	AT	3965076	2015/03/31	2015/03/31	Neil Dassanayake
Resistivity of Soil		3963203	2015/04/01	2015/04/01	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	3966281	N/A	2015/04/01	Deonarine Ramnarine

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 #: B553991
Report Date: 2015/04/01

exp Services Inc
Client Project #: ADM-00223648-B0
Site Location: HWY 590, KAKABEKA, ONTARIO

TEST SUMMARY

Maxxam ID: AAG179 Dup
Sample ID: BH403-S3
Matrix: Soil

Collected: 2015/03/17
Shipped:
Received: 2015/03/27

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	3966279	N/A	2015/04/01	Deonarine Ramnarine
pH CaCl2 EXTRACT	AT	3965076	2015/03/31	2015/03/31	Neil Dassanayake
Sulphate (20:1 Extract)	KONE/EC	3966281	N/A	2015/04/01	Deonarine Ramnarine

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Site Location: HWY 590, KAKABEKA, ONTARIO

GENERAL COMMENTS

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

Package 1	4.7°C

Sample AAG172-01 : CONDUCT-SB/PHCACL-S: Sample extracted/analysed past holding time.

Results relate only to the items tested.

QUALITY ASSURANCE REPORT

			Matrix Spike		Spiked Blank		Method Blank		RPD	
QC Batch	Parameter	Date	% Recovery	QC Limits	% Recovery	QC Limits	Value	Units	Value (%)	QC Limits
3965076	Available (CaCl2) pH	2015/03/31			100	97 - 103			1.2	N/A
3966279	Soluble (20:1) Chloride (Cl)	2015/04/01	NC	70 - 130	99	70 - 130	<20	ug/g	NC	35
3966281	Soluble (20:1) Sulphate (SO4)	2015/04/01	NC	70 - 130	100	70 - 130	<20	ug/g	9.0	35
3967584	Conductivity	2015/04/01			99	90 - 110	<0.002	mS/cm	0.44	10

N/A = Not Applicable

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

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

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

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

NC (Matrix Spike): The recovery in the matrix spike was not calculated. The relative difference between the concentration in the parent sample and the spiked amount was too small to permit a reliable recovery calculation (matrix spike concentration was less than 2x that of the native sample concentration).

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

Maxxam Job #: B553991
Report Date: 2015/04/01

exp Services Inc
Client Project #: ADM-00223648-B0
Site Location: HWY 590, KAKABEKA, ONTARIO

VALIDATION SIGNATURE PAGE

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

Cristina Carriere

Cristina Carriere, Scientific Services

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

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Phone: 905-817-5700 Fax: 905-817-5779 Toll Free: 800-563-6266

CHAIN OF CUSTODY RECORD

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-B0	Rush TAT (Applicable Surcharge)
Thunder Bay, ON P7B 5M4		Site Location: Hwy 590, Kakabeka, Ontario	[ ] 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)	OTHER REGULATIONS
[ ] Table 1 [ ] Res/Park [ ] Med/Fine	[ ] CCME [ ] Sanitary Sewer Bylaw
[ ] Table 2 [ ] Ind/Comm [ ] Coarse	[ ] MISA [ ] Storm Sewer Bylaw
[ ] Table 3 [ ] Agri/Other	[ ] PWQO Municipality:
[ ] Table ___	[ ] Other (Specify):
FOR RSC (PLEASE CIRCLE) Yes / No	[ ] 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

Rush Confirmation #:

Date Required:

LABORATORY USE ONLY

CUSTODY SEAL (Y/N)		Temperature (°C) on Receipt
Present	N	4/5/5
Intact	N	
COOLING MEDIA PRESENT (Y / N)	Y	

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								COMMENTS / TAT COMMENTS
1	B101-S12	Feb. 26/15	3:00	Soil	1		x	x	x	x	x								
2	BH105-S4	Mar. 20/15	4:30	Soil	1		x	x	x	x	x								
3	BH202-S10/S11	Mar. 04/15	11:15	Soil	1		x	x	x	x	x								
4	BH203-S3	Mar. 19/15	10:20	Soil	1		x	x	x	x	x								
5	BH302-S10	Mar. 11/15	2:40	Soil	1		x	x	x	x	x								
6	BH303-S4	Mar. 18/15	11:25	Soil	1		x	x	x	x	x								
7	BH402-S14	Mar. 07/15	5:10	Soil	1		x	x	x	x	x								
8	BH403-S3	Mar. 17/15	10:00	Soil	1		x	x	x	x	x								
9																			
10																			

RELINQUISHED BY: (Signature/Print)	DATE: (YYYY/MM/DD)	TIME:	RECEIVED BY: (Signature/Print)	DATE: (YYYY/MM/DD)	TIME:	# JARS USED AND NOT SUBMITTED
Michael Suslyk	26-Mar-15	1:30	for MUSRAT MAZ	2015/03/27	10:00	

27-Mar-15 10:00
Hina Siddiqui
B553991
HP6 ENV-789

Maxxam Analytics International Corporation o/a Maxxam Analytics

Appendix F –
Slope Stability Analysis

Cedar Creek Culvert #1
North side of Embankment (Inlet)
Drained Condtion

Name: Engineered Fill Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion': 0 kPa Phi': 32 °
Name: Topsoil/Silt Model: Mohr-Coulomb Unit Weight: 18 kN/m³ Cohesion': 0 kPa Phi': 17 °
Name: Silty Sand with Peat (Very Loose to Loose) Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion': 0 kPa Phi': 18 °
Name: Cobbles and Boulders (Very Dense) Model: Mohr-Coulomb Unit Weight: 22 kN/m³ Cohesion': 0 kPa Phi': 38 °
Name: Silt and Peat (Soft) Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion': 0 kPa Phi': 17 °
Name: Silty Sand (Dense) Model: Mohr-Coulomb Unit Weight: 19.5 kN/m³ Cohesion': 0 kPa Phi': 30 °
Name: Silty Sand with Gravel (Very Loose to Compact) Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion': 0 kPa Phi': 32 °
Name: Silty Sand with Gravel (Very Dense) Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion': 0 kPa Phi': 35 °
Name: Clay with Silt (Firm to Stiff) Model: Mohr-Coulomb Unit Weight: 19 kN/m³ Cohesion': 0 kPa Phi': 26 °
Name: Bedrock Model: Bedrock (Impenetrable)

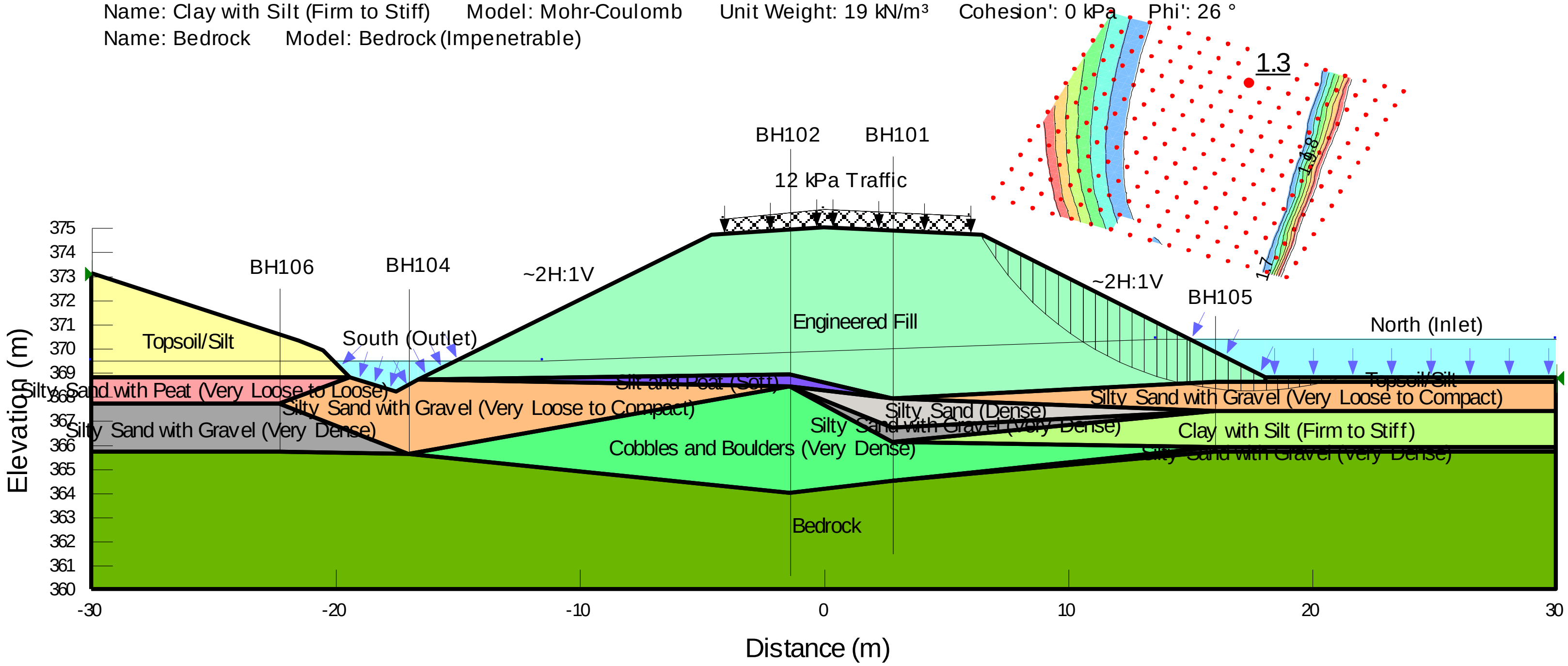


Figure F1: North side of embankment (inlet) – drained static condition

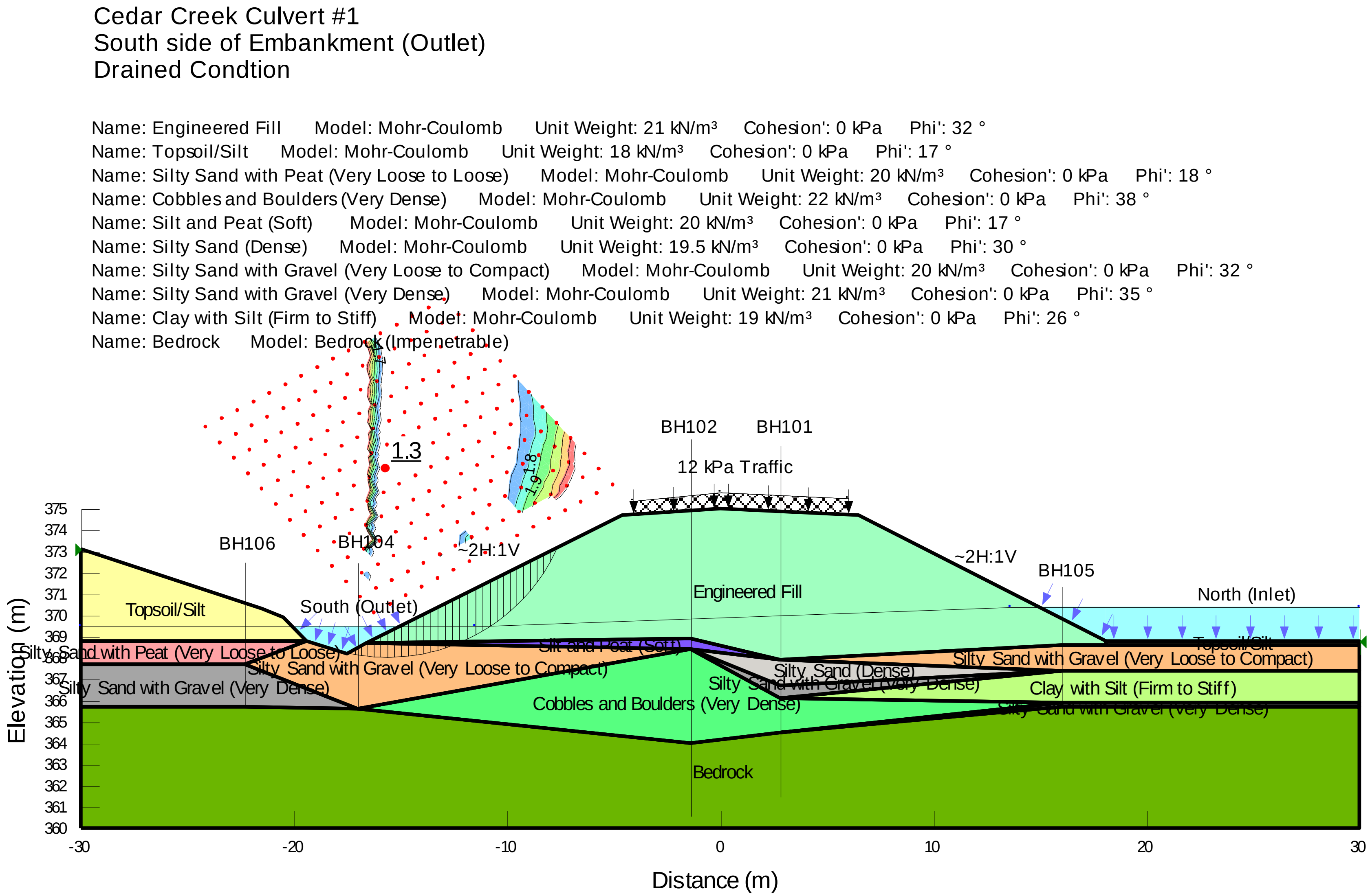


Figure F2: South side of embankment (outlet) – drained static condition

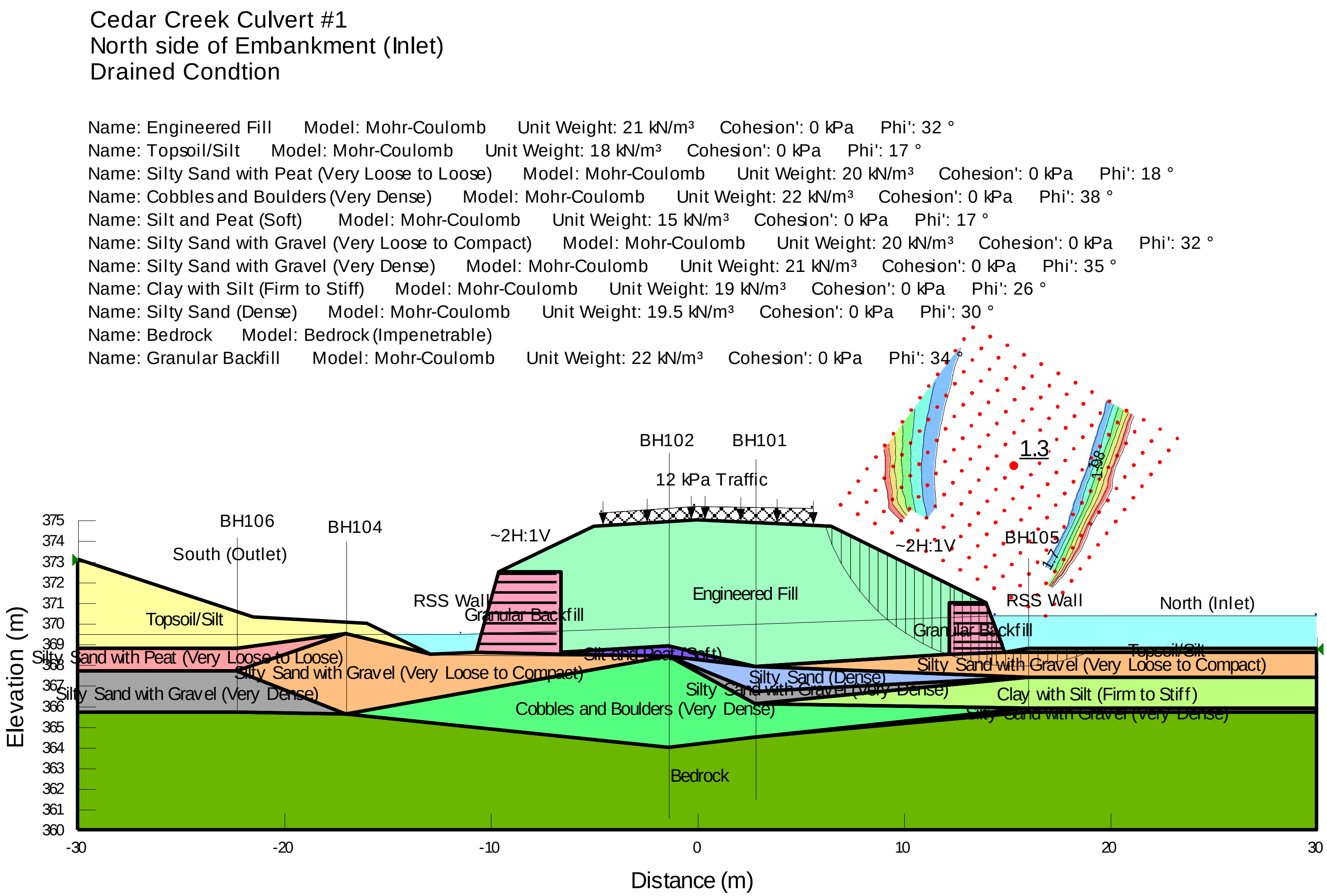


Figure F3: North side of embankment (inlet) with RSS wall – drained static condition

Cedar Creek Culvert #1
South side of Embankment (Outlet)
Drained Condtion

Name: Engineered Fill Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion': 0 kPa Phi': 32 °
Name: Topsoil/Silt Model: Mohr-Coulomb Unit Weight: 18 kN/m³ Cohesion': 0 kPa Phi': 17 °
Name: Silty Sand with Peat (Very Loose to Loose) Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion': 0 kPa Phi': 18 °
Name: Cobbles and Boulders (Very Dense) Model: Mohr-Coulomb Unit Weight: 22 kN/m³ Cohesion': 0 kPa Phi': 38 °
Name: Silt and Peat (Soft) Model: Mohr-Coulomb Unit Weight: 15 kN/m³ Cohesion': 0 kPa Phi': 17 °
Name: Silty Sand with Gravel (Very Loose to Compact) Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion': 0 kPa Phi': 32 °
Name: Silty Sand with Gravel (Very Dense) Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion': 0 kPa Phi': 35 °
Name: Clay with Silt (Firm to Stiff) Model: Mohr-Coulomb Unit Weight: 19 kN/m³ Cohesion': 0 kPa Phi': 26 °
Name: Silty Sand (Dense) Model: Mohr-Coulomb Unit Weight: 19.5 kN/m³ Cohesion': 0 kPa Phi': 30 °
Name: Bedrock Model: Bedrock (Impenetrable)
Name: Granular Backfill Model: Mohr-Coulomb Unit Weight: 22 kN/m³ Cohesion': 0 kPa Phi': 34 °

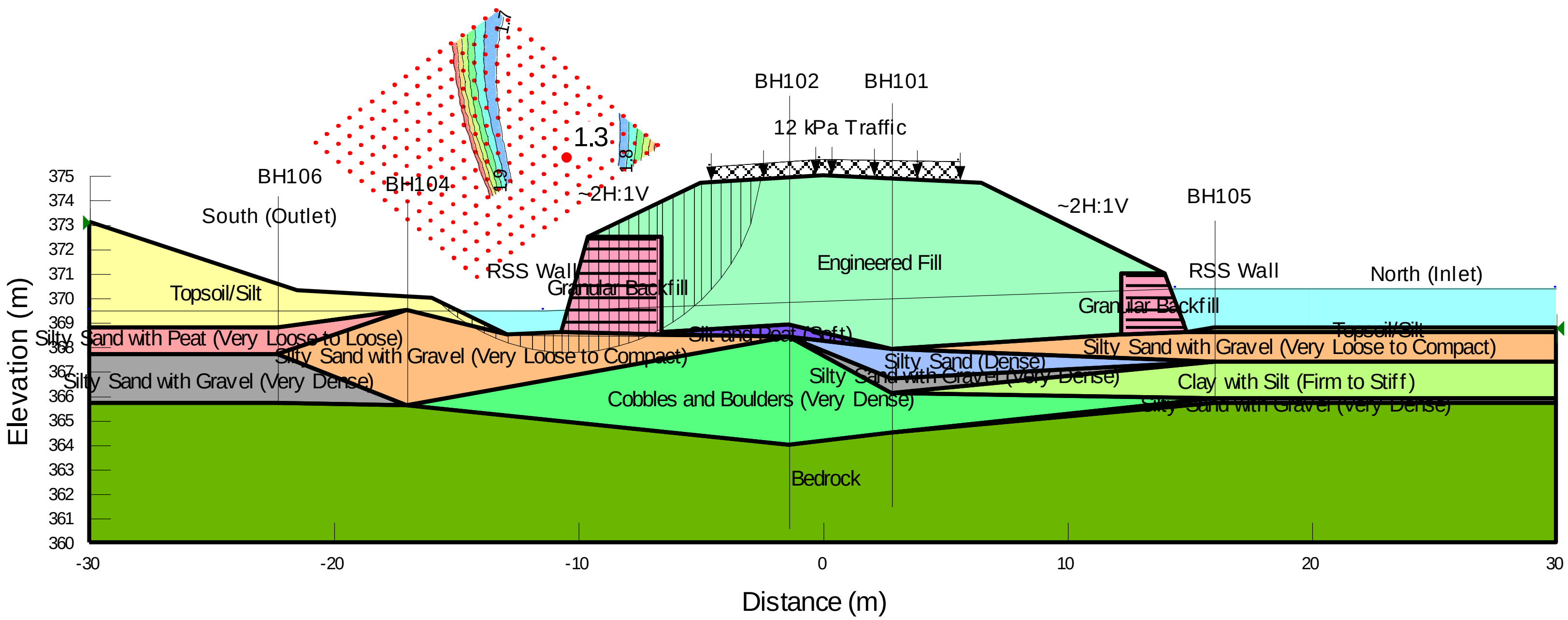
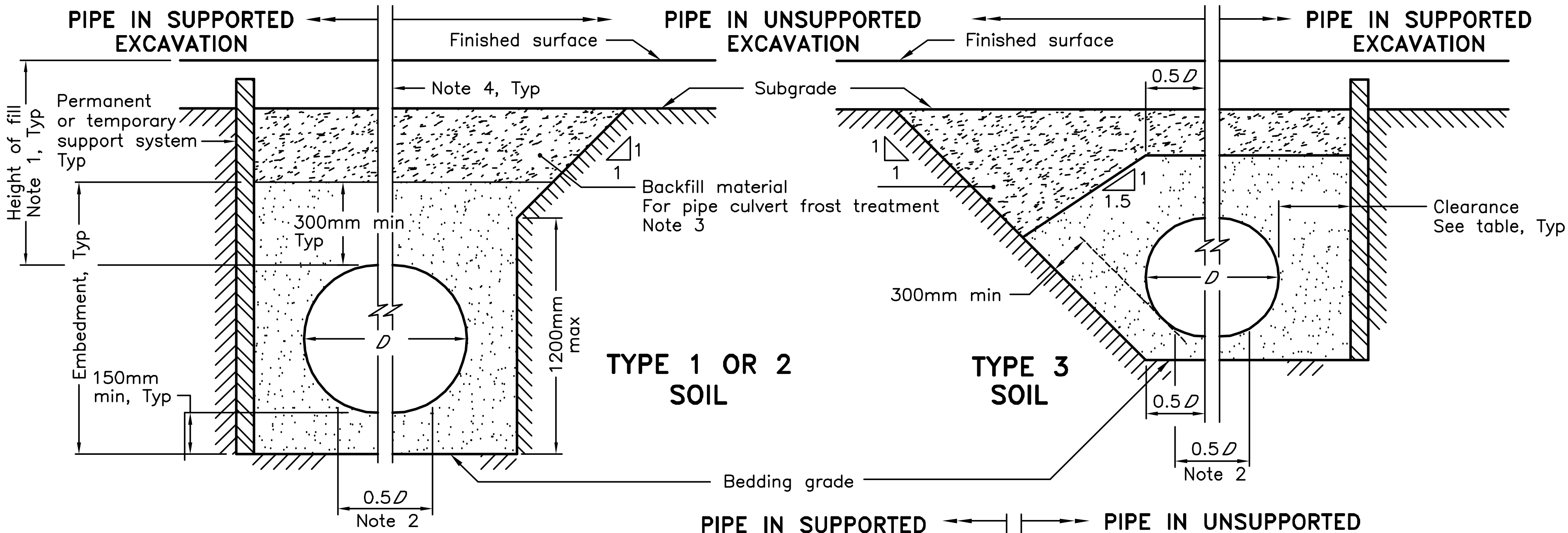
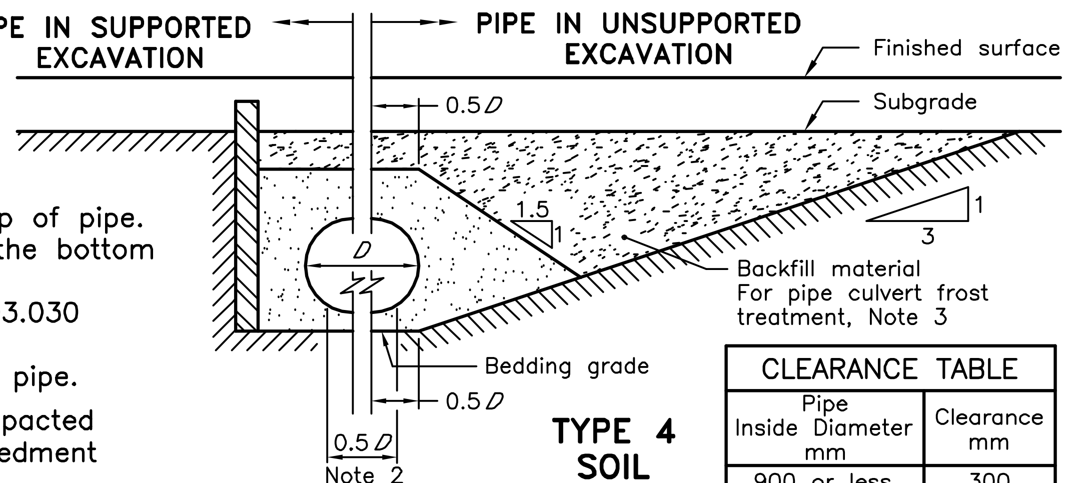


Figure F4: South side of embankment (outlet) with RSS wall – drained static condition

Appendix G – OPSDs





LEGEND:

D – Inside diameter

NOTES:

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

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

CLEARANCE TABLE

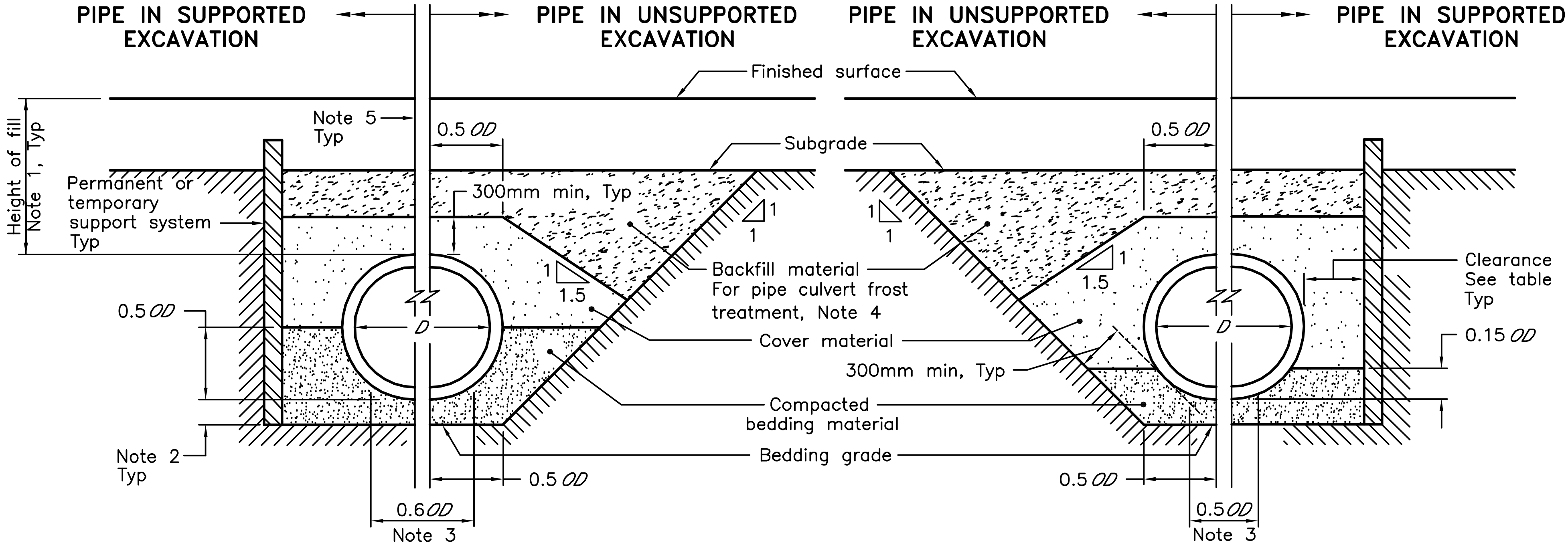
Pipe Inside Diameter mm	Clearance mm
900 or less	300
Over 900	500

ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2010 | Rev 2

FLEXIBLE PIPE
EMBEDMENT AND BACKFILL
EARTH EXCAVATION

OPSD 802.010



CLASS B BEDDING

CLASS C BEDDING

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

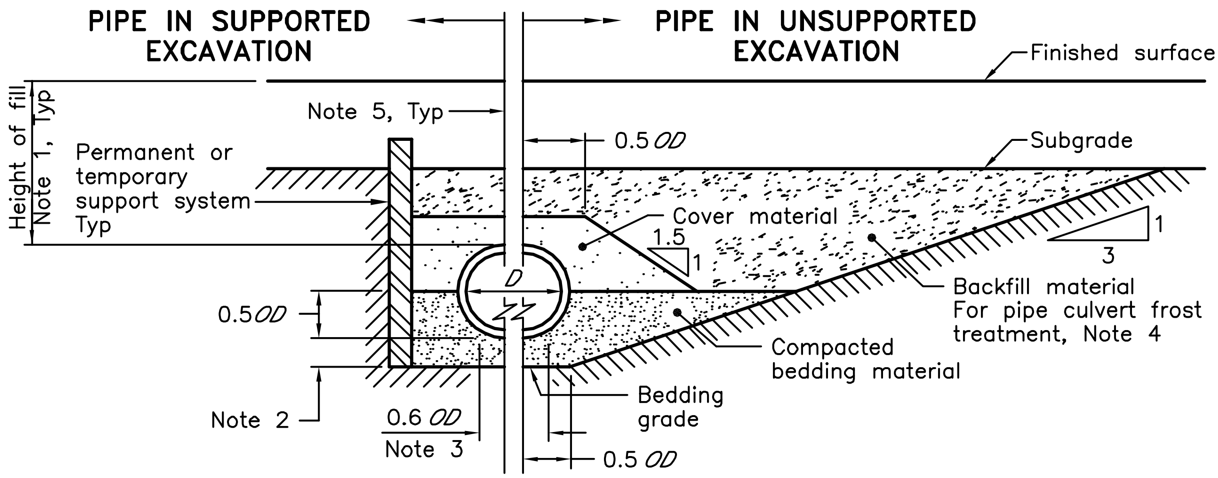
Pipe Inside Diameter mm	Clearance mm
900 or less	300
Over 900	500

ONTARIO PROVINCIAL STANDARD DRAWING

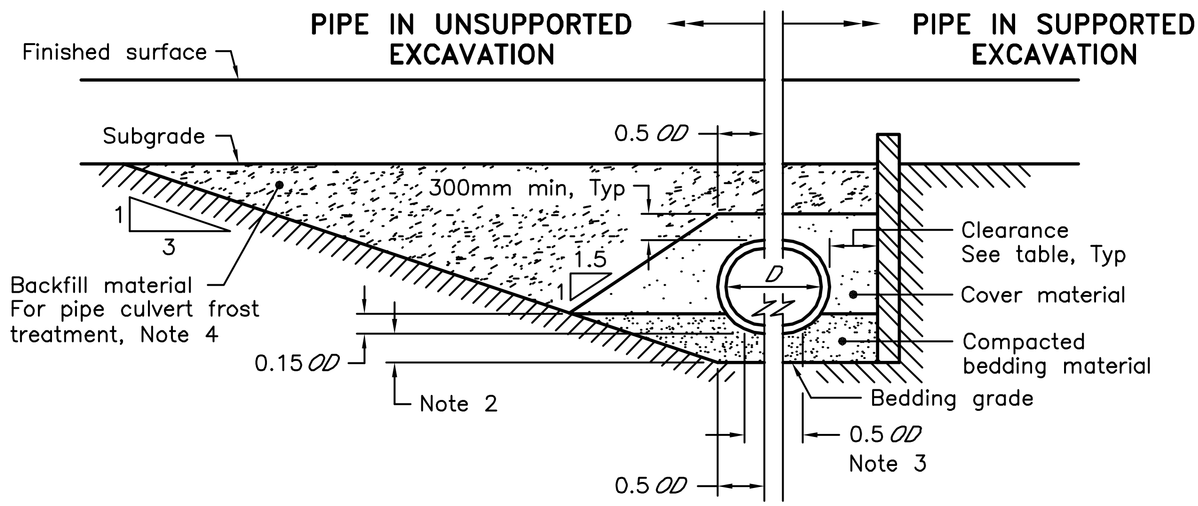
Nov 2010 Rev 2

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

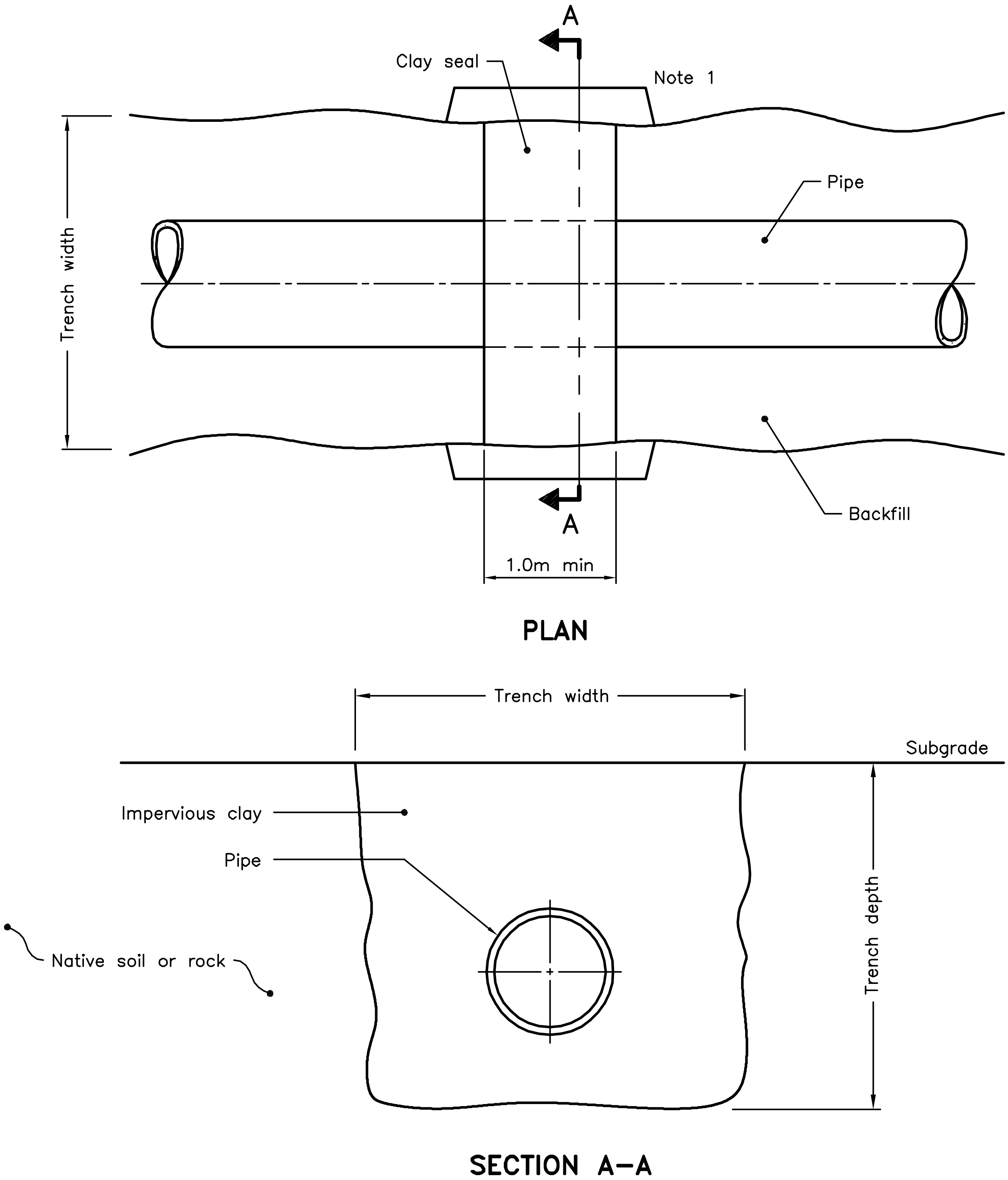
LEGEND:

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**	



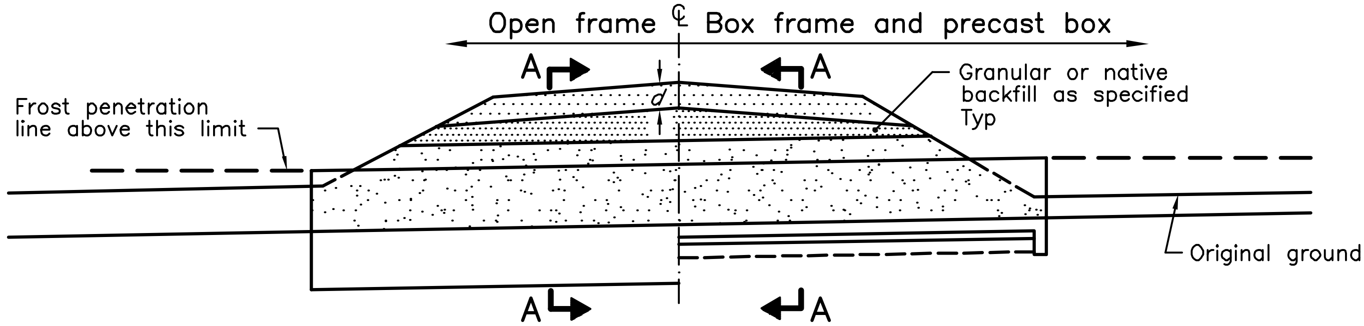
NOTES:

1. Key into undisturbed trench soil.

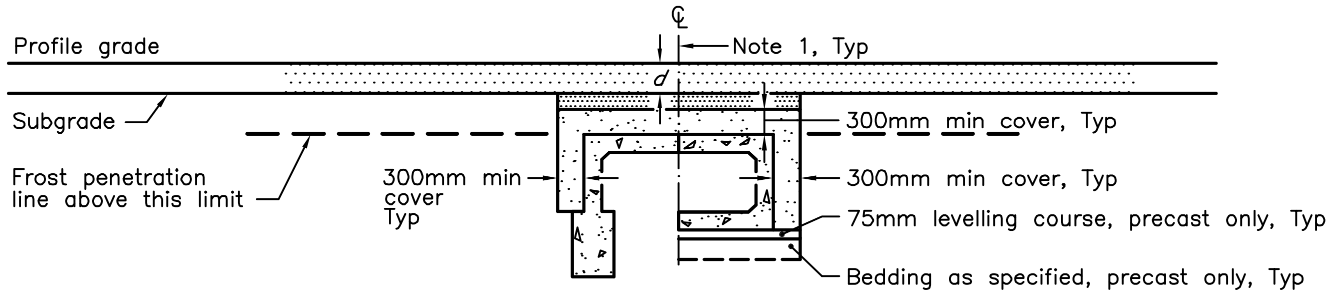
A Clay seal shall extend from bottom of trench excavation to the subgrade.
B Clay seal shall be located so that no pipe joints are within the clay seal material.
C All dimensions are in metres unless otherwise shown.

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

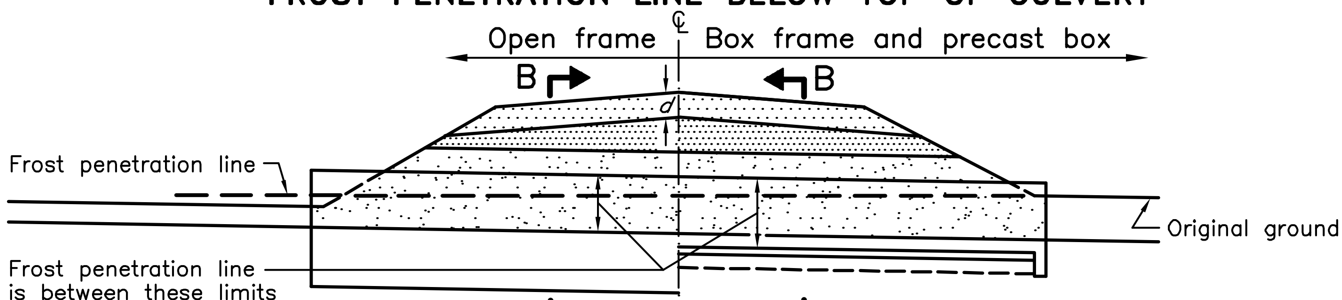


LONGITUDINAL SECTION

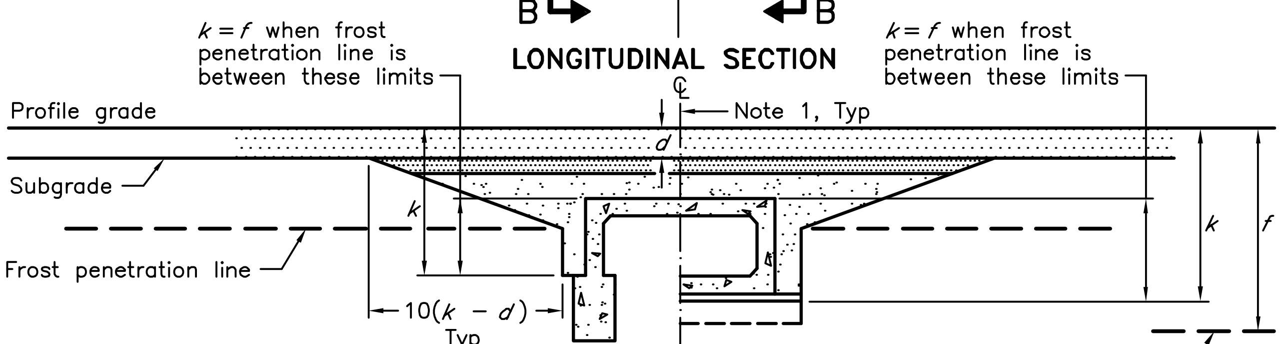


SECTION A–A

FROST PENETRATION LINE BELOW TOP OF CULVERT



LONGITUDINAL SECTION



SECTION B–B

LEGEND:

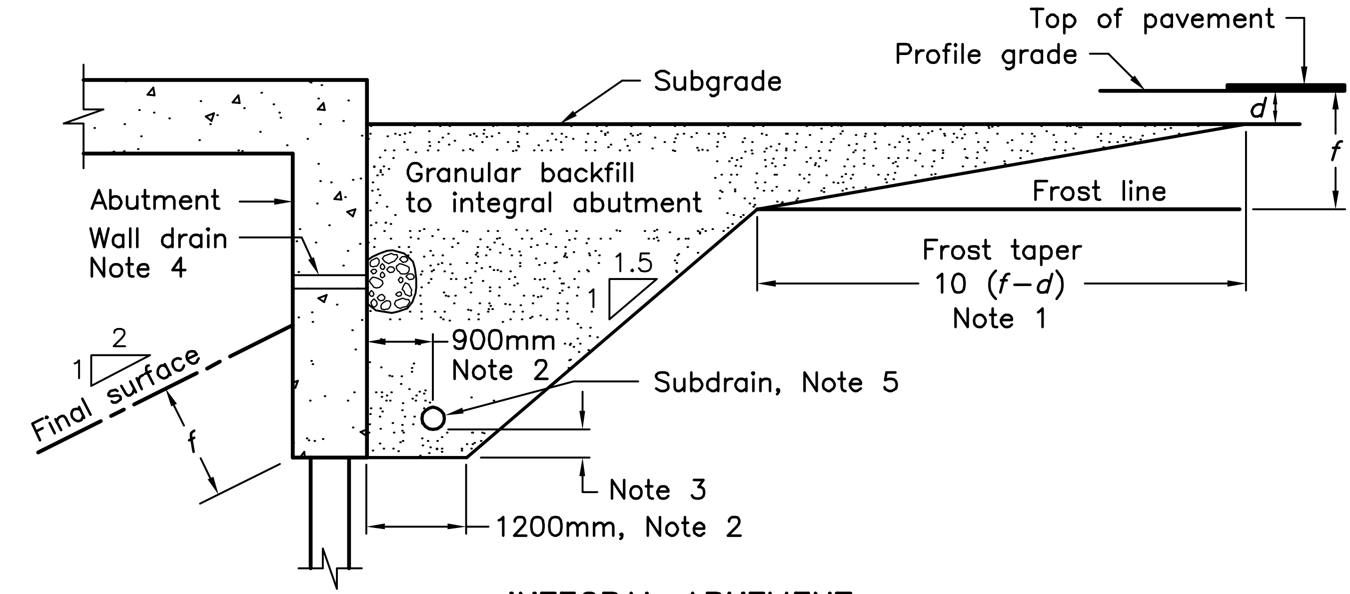
d = depth of roadbed granular
k = depth of frost treatment below profile grade
f = depth of frost penetration below profile grade

NOTES:

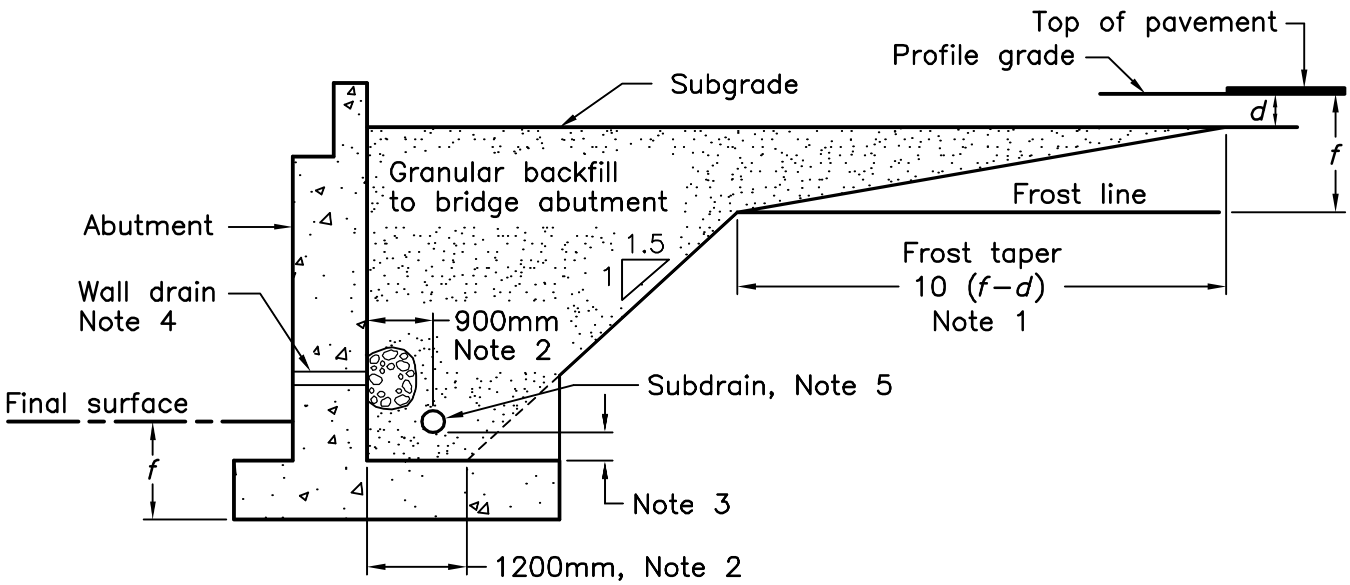
1 Condition of frost treatment symmetrical about centreline of culvert.

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	



INTEGRAL ABUTMENT



ABUTMENT

NOTES:

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

2 Dimensions perpendicular to back face of abutment.

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

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

5 150mm dia perforated pipe subdrain wrapped with geotextile.

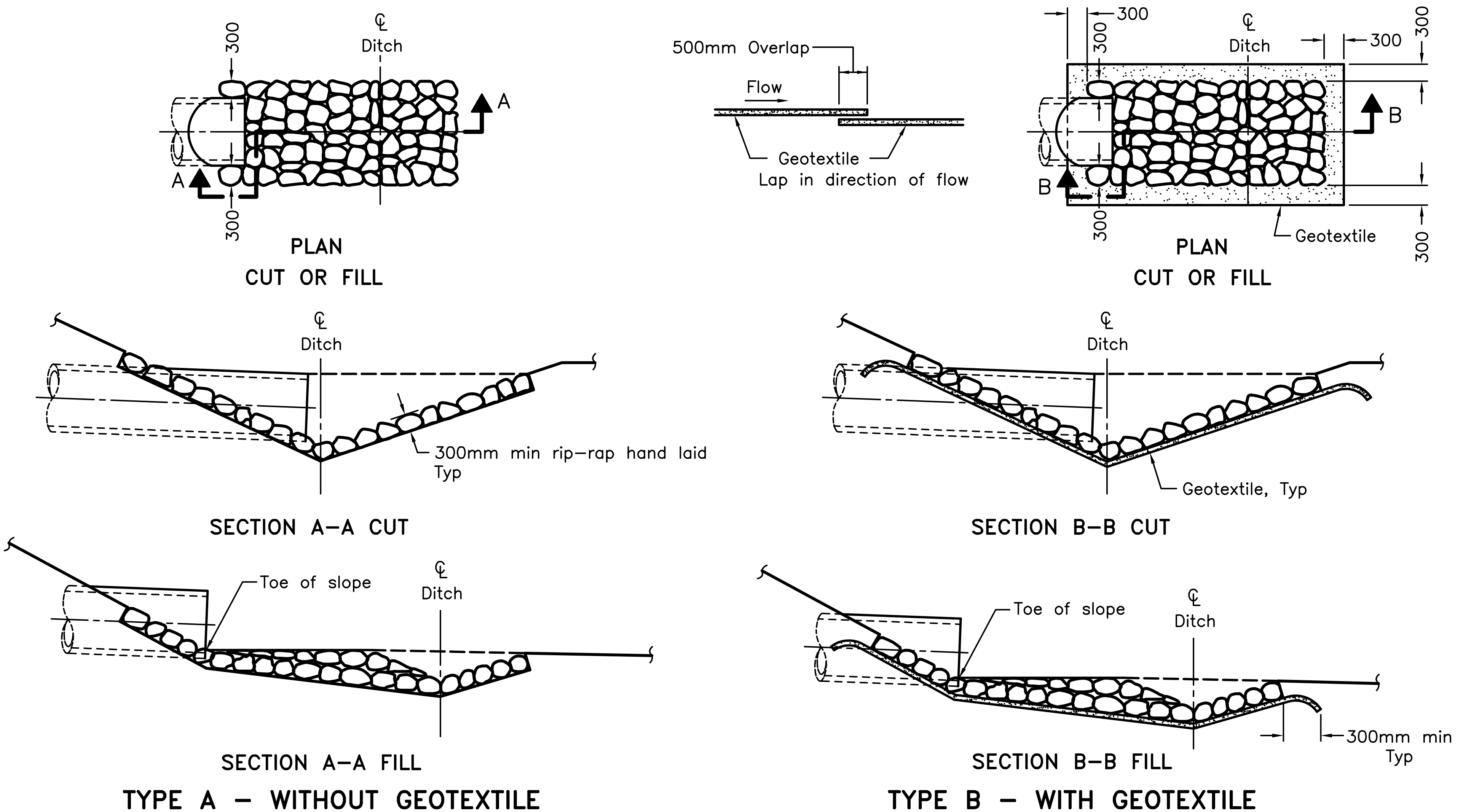
A Lateral limits of granular backfill to bridge abutment to be inside face to inside face of retaining wall or wingwall. Frost taper shall extend the full width of the backfill unless interrupted by the retaining wall or wingwall.

B Sections shown are parallel to centreline of roadway.

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

D All dimensions are in millimetres unless otherwise shown.

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



NOTES:

A 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	

ONTARIO STANDARDS · MUNICIPAL – PROVINCIAL · O P S

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

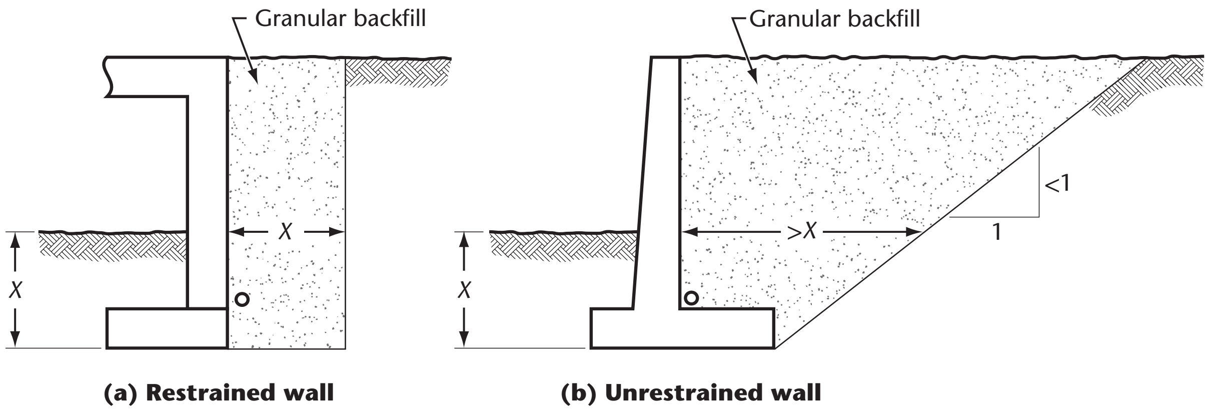


Figure C6.20
Backfill for frost protection
(See Clause C6.9.1.)

C6.9.2 Lateral pressures

C6.9.2.1 General

Earth pressure acting on a structure depends on the relative movement of the structure, the backfill, the type of soil adjacent to the backfill, and the soil below the footing or supporting piles. Appropriate geotechnical parameters should be chosen for the calculation of lateral pressures based on recognized geotechnical theories as specified in Clause 6.9.2.2 for the backfill behind the wall. Geotechnical parameters frequently used in allowable stress design methods are applicable in limit states design pressure calculation. Where the possibility exists, hydrostatic pressure needs to be considered, e.g., in situations where walls are partially submerged or where non-free-draining backfill is used.

Clause 6.9.2.1 includes the specification of four lateral pressure conditions for design. The first two cases apply to unrestrained structures, with Item (a) applying to the sizing of the base or pile arrangement with respect to external stability, and Item (b) to the sizing of the structural sections with respect to internal stability. Such sections could be of structural concrete, structural steel, or a proprietary product.

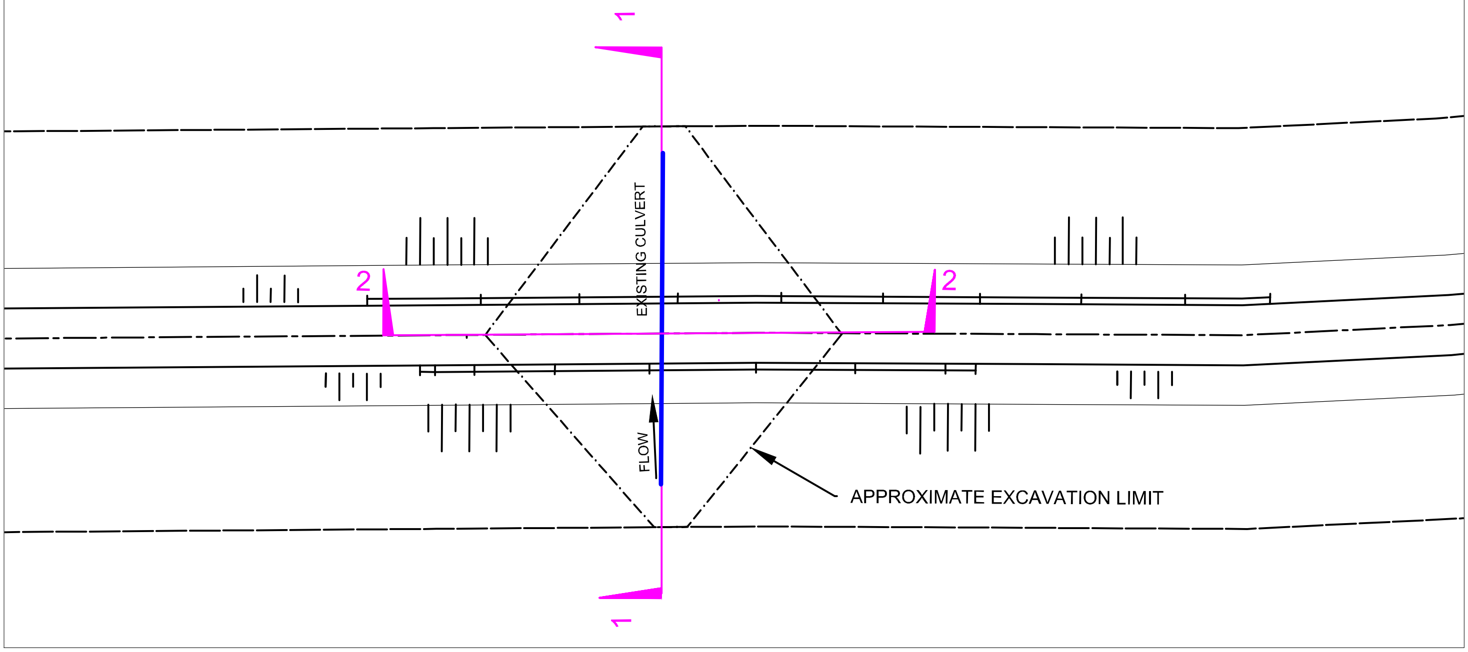
An unrestrained structure is one in which active pressure is mobilized in the backfill due to movement in the supporting structure. This movement corresponds to a rotation of approximately 0.002 about the base of a vertical wall, a horizontal translation of 0.001 times the height of the wall, or a combination of these movements. The lateral pressure applied to the wall for the condition described is an active pressure.

The supporting material will generally be more robust than what is assumed by the Geotechnical Engineer for factored conditions in design. Hence, following installation of the backfill, movement sufficient to cause active condition will generally not have taken place. Horizontal or rotational movement of the base will occur during the installation of each lift of the backfill. Wall deflection during each application and compaction of the backfill will add to the existing deformations. For such a post placement of the fill condition, Item (b) applies, the forces acting on the retaining structure being a function of the compacting equipment and the flexural stiffness of the wall. The residual horizontal pressures due to compaction are largest at the top of the wall, and this is reflected in Clause 6.9.3.

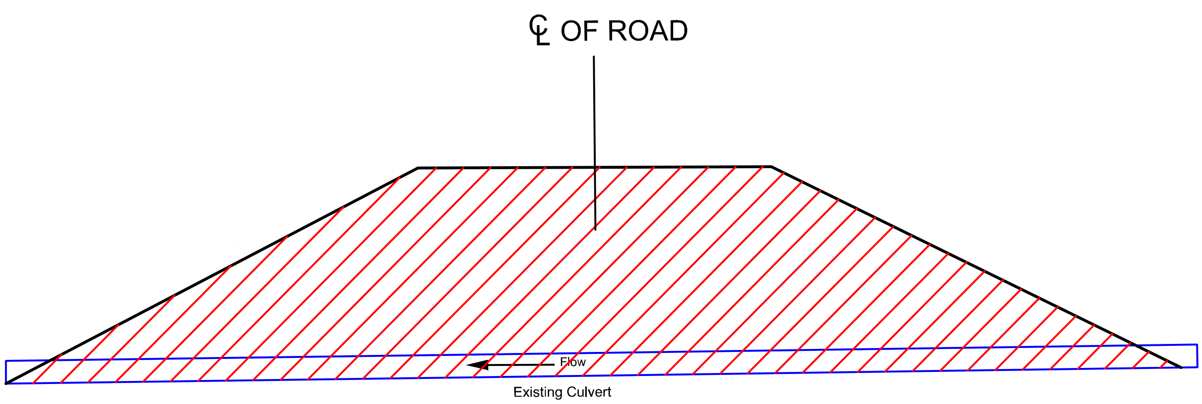
Appendix H –
Schematic Sketches for Construction Alternatives

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

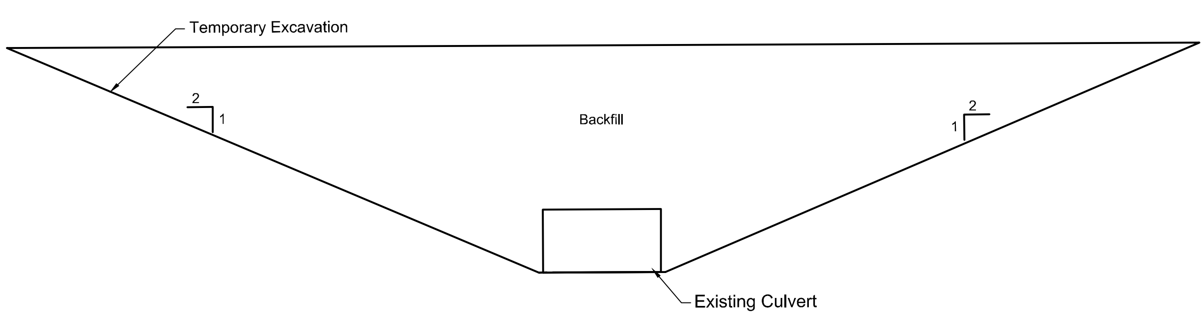
SCHEMATIC DIAGRAMS (NTS)



PLAN



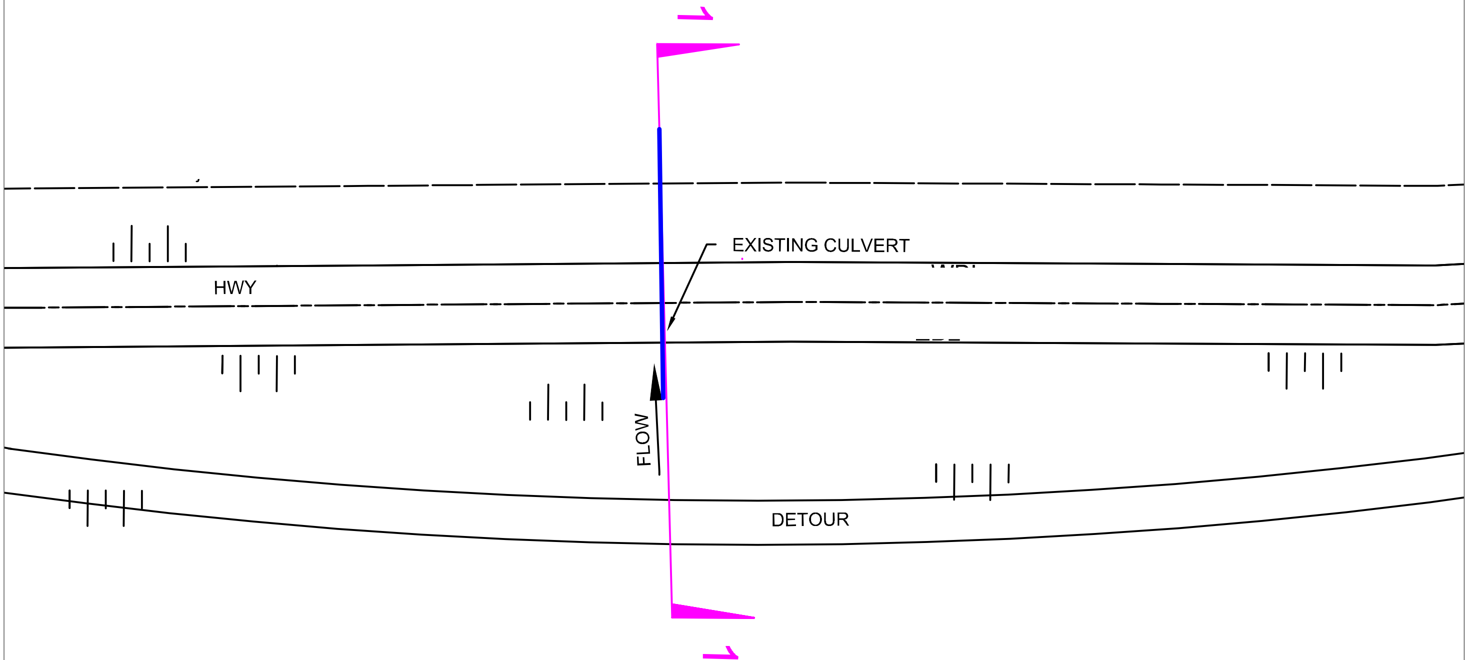
SECTION 1-1



SECTION 2-2

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

SCHEMATIC DIAGRAMS (NTS)



PLAN

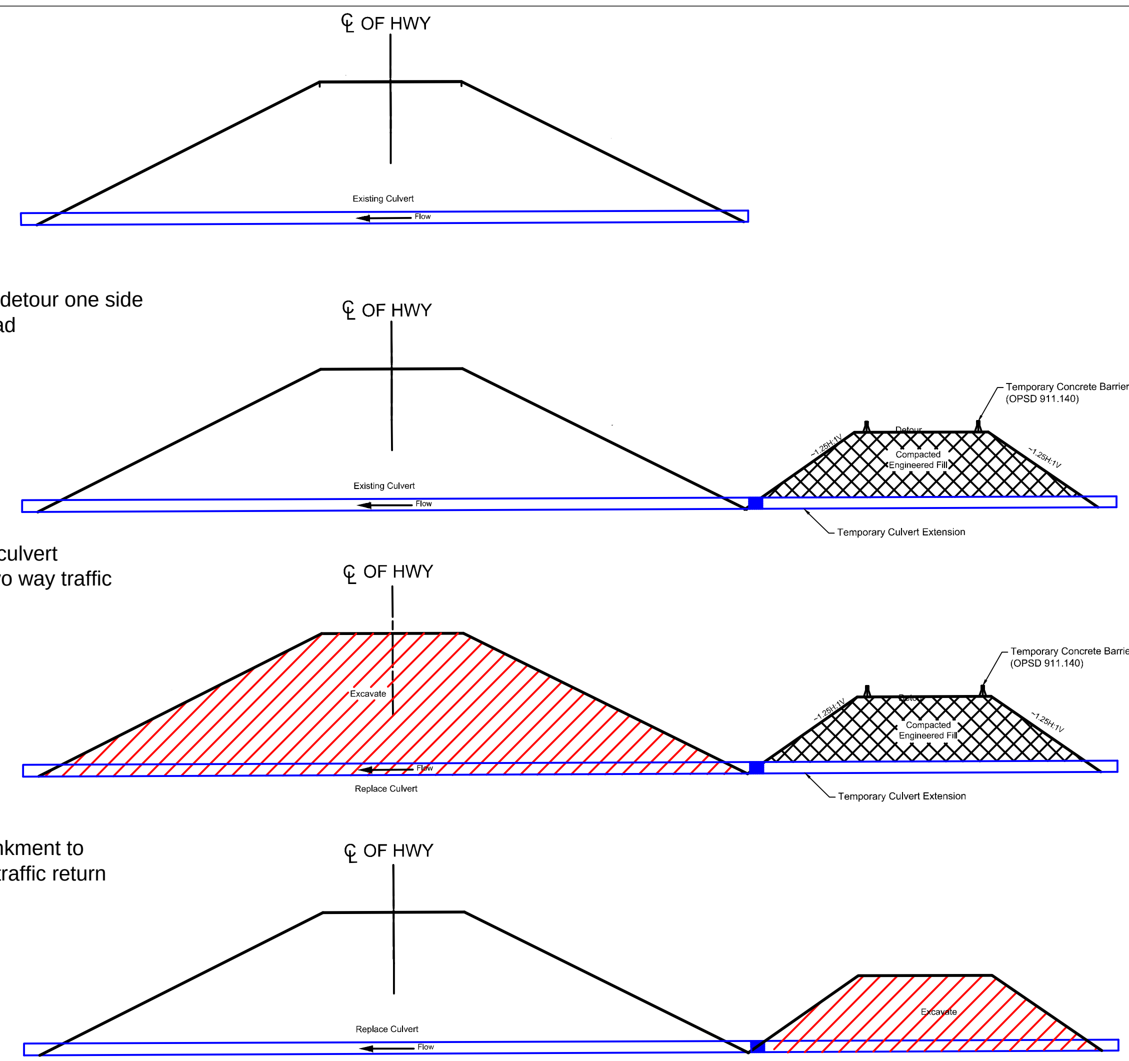
RECOMMENDED STAGES

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

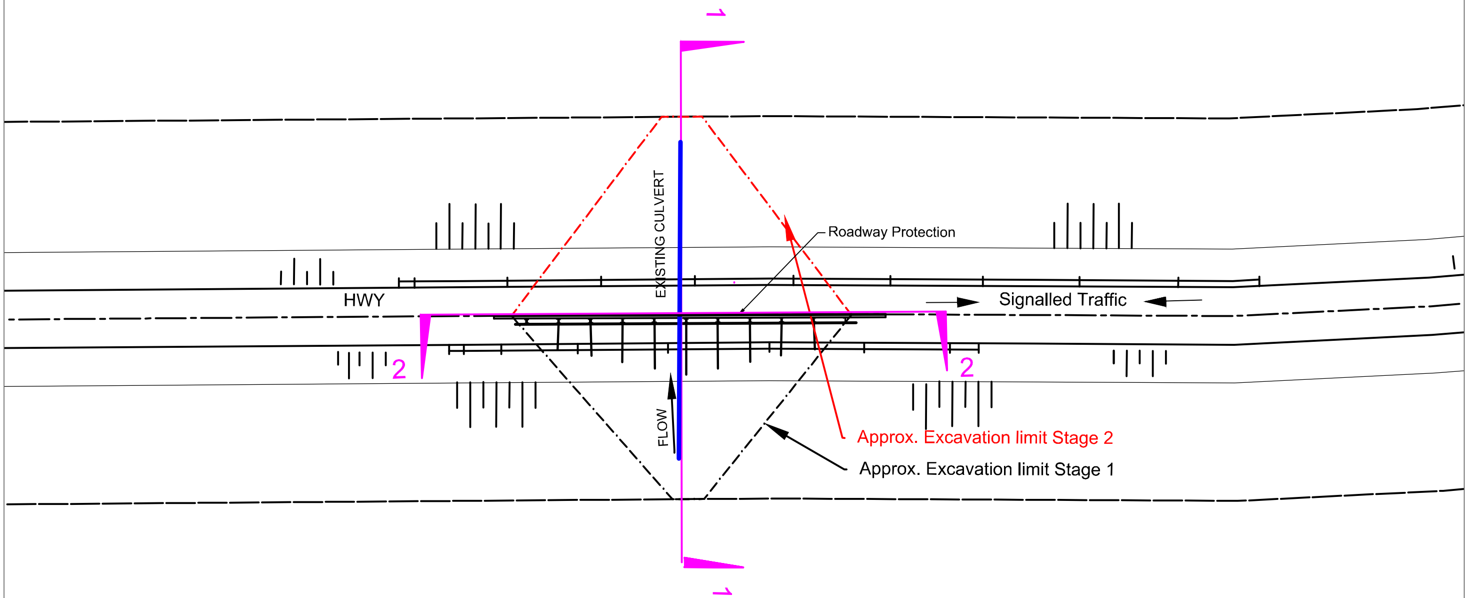
4.0 Stage 4 - Build the embankment to existing alignment; Two-way traffic return



SECTION 1-1

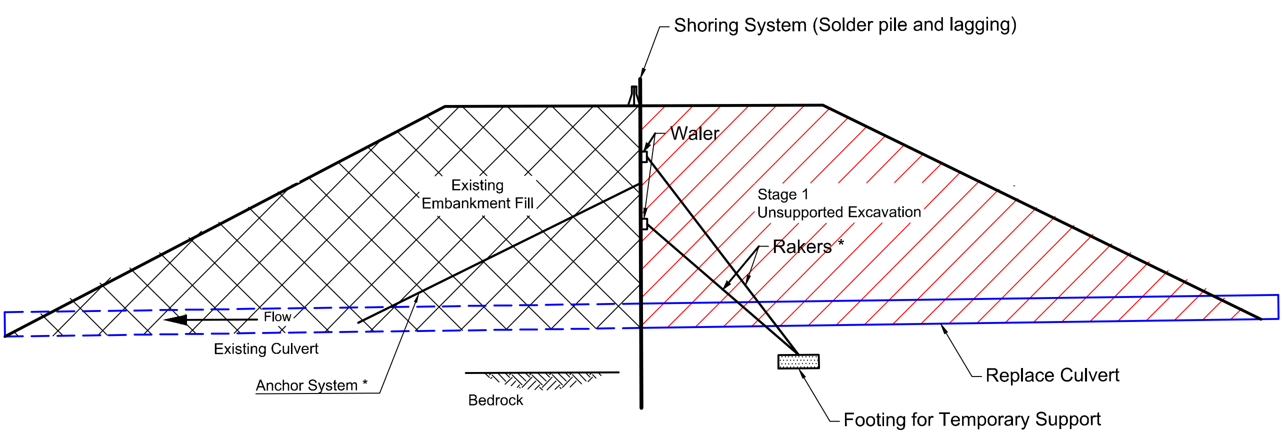
FIGURE H.3.A: HALF AND HALF CONSTRUCTION WITH UNSUPPORTED CUT SIDES

SCHEMATIC DIAGRAMS (NTS)



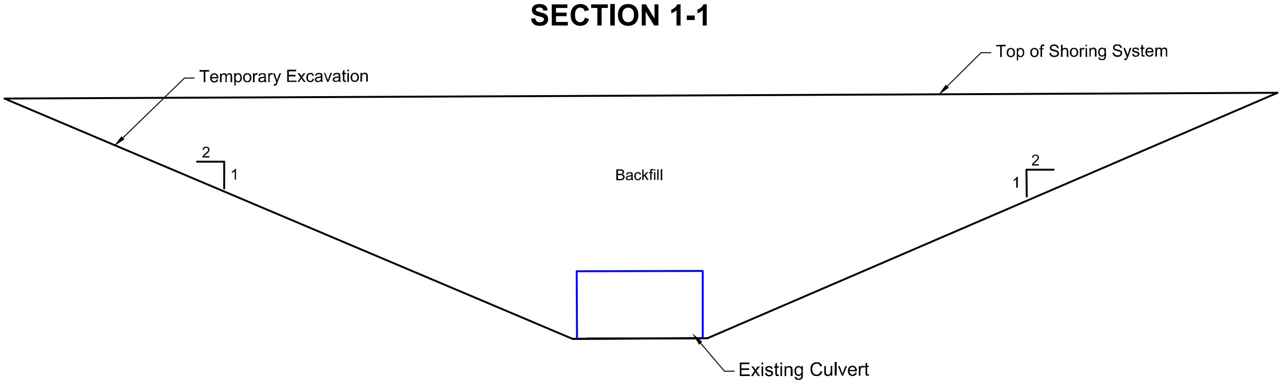
PLAN

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



* Rakers or Anchor System

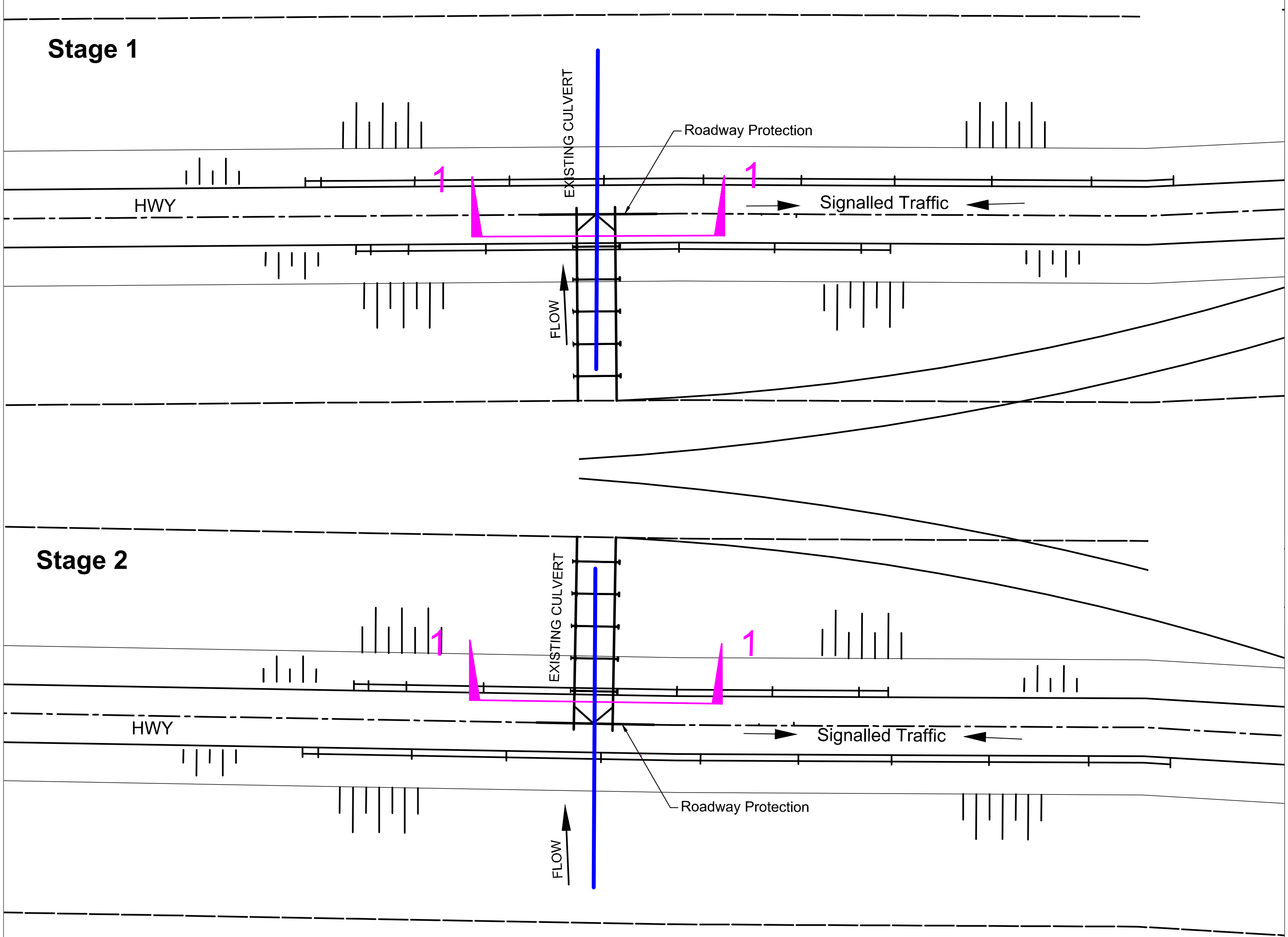
SECTION 1-1



SECTION 2-2

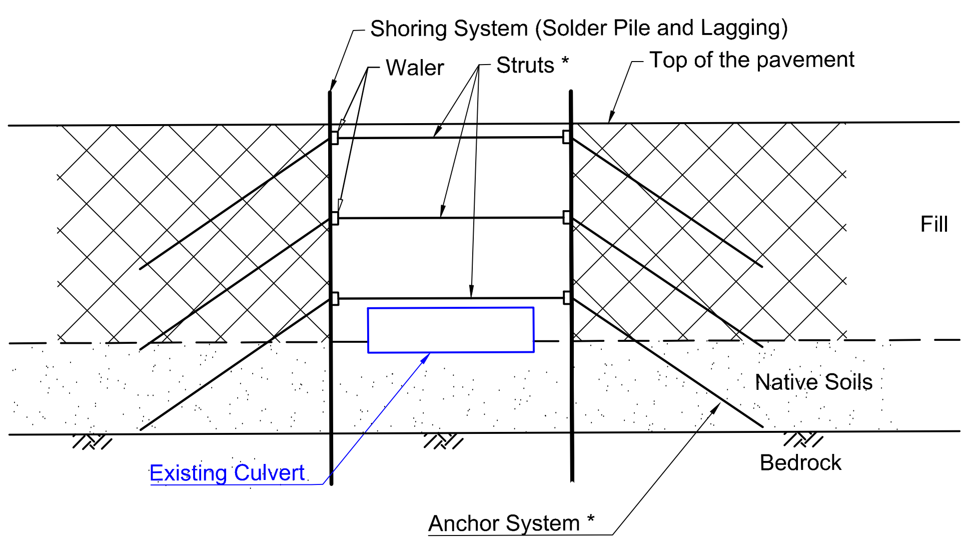
FIGURE H.3.B: HALF AND HALF CONSTRUCTION WITH BRACED CUT SIDES OR ANCHOR SYSTEM OPTION 3.B

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 COBLES AND BOULDERS LAYER/BEDROCK

Scope of Work

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

The Contractor should be aware that the layer of cobles and boulders was encountered above the sloping bedrock. Mass concrete volumes will vary depending on the variable cobbles and boulders layer/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 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.