



## **FINAL REPORT**

### **FOUNDATION INVESTIGATION AND DESIGN REPORT** **Whitewood Creek Culvert Replacement, Highway 590, Site No. 48W-168/C,** **Township of Marks, District of Thunder Bay**

**Agreement No. 6014-E-0017**  
**Assignment No. 2**  
**GWP 6349-14-00**  
**Geocres No. 52A-213**

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# Ontario Ministry of Transportation

## Foundation Investigation and Design Report

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Foundation Investigation and Design Report for Whitewood Creek Culvert Replacement  
HWY 590, Site No. 48W-168/C, Township of Marks, District of Thunder Bay

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## **PART 1: FOUNDATION INVESTIGATION REPORT**

### **1.1 Introduction**

This foundation investigation report presents the results of a geotechnical investigation completed by **exp** Services Inc. for the replacement of Whitewood Creek Culvert on Highway 590, located approximately 14 km west of the junction of Highway 590 and Highway 11/17 at Whitewood Creek, in the Township of Marks, the Ministry of Transportation (MTO) Northwestern Region. The work was undertaken under Agreement # 6014-E-0017, Assignment No. 2 (GWP 6349-14-00). 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 corrugated steel pipe (CSP) culvert with a length of about 41.8 m and diameter of about 3.0 m. It is understood that the existing culvert was constructed/installed in 1977, and is intended to be replaced with a new culvert along the same alignment.

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

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

### **1.2 Site Description and Geological Setting**

#### **1.2.1 Site Description**

As shown on Drawing 1 (Appendix B), the Whitewood Creek Culvert replacement site is located on Hwy 590, approximately 14 km west of the junction of Hwy 590 and Hwy 11/17, in the Township of Marks. At the site, Hwy 590 is a two lane roadway, with a speed limit of 80 km/h and is about 6.5 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 7 m high with side slopes of about 2H:1V west of the roadway and 2.5H:1V east of the roadway.

During the fieldwork on March 5, 6, 11, 12 and 18, 2015, the general site conditions were assessed; however, the site was generally snow covered which limited visual observations. Hwy 590 runs in a north to south direction and Whitewood Creek, flows from northwest to southeast across the highway, and ultimately discharges towards Kakabeka Falls, which is about 12 km east of the site. At the time of this investigation, Whitewood Creek was frozen and the approximate creek elevations (top of ice) at the inlet and outlet were about 425.32 m and 424.99 m, respectively. The elevation of highway centerline pavement at the culvert centerline is about 431.91 m. Overhead wires were observed along the east side of the roadway.

The vicinity of the inlet and outlet of the culvert was snow covered but the visible vegetation was generally grass and small shrubs. The inlet and outlet appeared to be clear of debris, as such the flow does not appear to be impeded. However, immature trees/brush were present immediately in front (west) of the culvert inlet. No trees or shrubs were noted immediately in front of the culvert outlet.

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 Mesoproterozoic 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 porphyritic (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 March 5, 6, 11, 12 and 18, 2015. The field program consisted of drilling four (4) sampled boreholes (BH301 to BH304). Two (2) boreholes were located within the roadway, BH301 and BH302. BH301 was located about 3 m south of the culvert as near as possible to the edge of pavement in the southbound lane and BH302 was located about 4 m north of the culvert as near as was possible to the centerline of Hwy 590 within the northbound lane. An additional two (2) boreholes (BH303 and BH304) were advanced off the roadway, near the culvert inlet and outlet. BH303 was located about 5 m north of the culvert (inlet side) and BH304 was located about 5 m south of the culvert (outlet side). The borehole locations are shown on Drawing 1 in Appendix B.

The roadway boreholes (BH301 and BH302) and the off-road borehole at the outlet side of the culvert (BH304) were advanced using a CME 55 truck mounted drill rig and/or CME 850 track mounted drill rig. The remaining off-road borehole at the culvert inlet (BH303) was advanced using a CME 45 rubber track mounted drill rig. A T340XL crane was used to lower/lift the CME 45 drill rig at BH303. All drill rigs were equipped with hollow and solid stem continuous flight augers, and standard soil sampling equipment (includes 51 mm outside diameter split spoon samplers and *in situ* shear vane testing equipment). In addition, the CME 850 drill rig was equipped with rock coring equipment, NQ size. The roadway boreholes BH301 and BH302 were advanced to refusal depths of about 14.6 m and 13.4 m below ground surface, respectively, and the off-road boreholes BH303 and BH304, were advanced to refusal depths of about 7.8 m and 6.8 m below ground surface, respectively. Refusal was encountered at elevations ranging between about 417.3 m and 418.9 m.

At BH302 only, rock coring was conducted to an additional 5.7 m beyond auger/SPT refusal to about 19.4 m below asphalt surface (elevation 412.6 m) to determine the nature of refusal.

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 Pine tree north of the existing culvert). The elevation of the BM is 434.147 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. 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, and for subsequent examination by a geotechnical engineer and laboratory testing.

### 1.3.2 Laboratory Testing

All samples brought to the laboratory were subjected to visual examination and classification. The laboratory testing program included the determination of natural moisture content and particle size distribution for approximately 25% of the collected soil samples. Atterberg limits tests were carried out on select cohesive soil samples. All of the laboratory tests were carried out in accordance with MTO and/or ASTM Standards, as appropriate, at the **exp** laboratory in Thunder Bay, Ontario.

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

In addition, chemical testing of two select soil samples 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 sand, sandy silt, silty sand and well graded gravel. The fill layers are underlain by clayey silt and/or silty sand, which is overlying silty sand with gravel till, overlying cobbles and gravel, and overlying bedrock. 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 BH301 and BH302. The asphalt thickness was about 50 mm at both borehole locations. The fill generally comprised a conglomeration of gravel, sand, silt and clay layers with descriptions ranging between well graded gravel with sand to sandy silt with some clay. Where not frozen, the fill was generally described as compact to very dense, moist 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 5 and 110. The fill at BH301 and BH302 extended to about 6.9 m (425.0 m elevation) and 5.3 m (426.6 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:

- 2.6% to 21.2%

Grain size distribution:

- 5% to 48% gravel;
- 26% to 52% sand;



- <9% to 55% silt; and
- 6% to 20% 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, 2 and 3 in Appendix D.

#### 1.4.2 Topsoil

Topsoil was encountered surfacing BH303 and BH304. Where not frozen, the topsoil was generally described as very loose/soft at depth, brown, and moist to wet at depth. The SPT “N” values ranged between about 1 and 25 blows per 300 mm penetration. The topsoil extended to about 0.8 m below ground surface (425.8 m elevation) at BH303, and about 0.9 m below ground surface (424.8 m elevation) at BH304.

Silt and peat was encountered underlying the topsoil at BH304, and was described as soft, dark brown to black, and wet. The SPT “N” value was 2. The silt and peat extended to about 1.5 m depth (424.2 m elevation).

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

Moisture content:

- 28.5% to 46.5%

#### 1.4.3 Silty Sand with Gravel

A layer of silty sand with gravel was encountered underlying the fill at BH302. The native sand was generally described as compact, grey and moist. Trace organics were noted at about 6.1 m depth. The SPT “N” values ranged between about 12 and 24 blows per 300 mm penetration. The sand at BH302 extended to depth of about 7.6 m below ground surface (424.3 m elevation).

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

Moisture content:

- 14.5% to 27.1%

Grain size distribution:

- 32% gravel,
- 37% sand;
- 25% silt; and
- 6% clay size.

The results of the moisture content and grain size distribution are provided on the record of borehole sheets in Appendix C. The results of the grain size distribution are also provided on Figures 4 in Appendix D.

#### 1.4.4 Clayey Silt with Sand

Clayey silt with sand was encountered beneath the fill at BH301, beneath the silty sand with gravel at BH302, and beneath the topsoil/silt and peat at BH303 and BH304. The clayey silt with sand was generally described as firm to hard, grey to brown, and moist to wet. The SPT “N” values ranged between about 4 and 100 blows per 300 mm penetration. The clayey silt with sand extended to depths of about 3.1 m and 13.1 m below ground surface. The clayey silt with sand extended to elevations of about 418.9 m and 422.7 m.

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

Moisture content:

- 10.1% to 34.3%

Grain size distribution:

- <1% to 16% gravel,
- 3% to 36% sand,
- 33% to 73% silt, and
- 17% to 28% clay size.

Total saturated unit weights have been calculated based on the moisture contents and are estimated to range from about 18.5 to 22.9 kN/m<sup>3</sup>. Atterberg Limit testing was performed on three representative of the clayey silt with sand (BH301-S12, BH301-S14, BH302-S13, BH302-S15 and BH303-S5) and indicated that the soil is of low plasticity. The data is shown on the plasticity chart, Figure 7. The liquid limit, plastic limit and plasticity index ranged between about 24% to 29%, 15% to 19%, and 8% to 14%, 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 Figures 5 in Appendix D, and Atterberg Limit tests are provided on Figure 7 in Appendix D.

#### 1.4.5 Silty Sand with Gravel Till

Silty sand with gravel till was encountered underlying the lean clay to sandy lean clay. The till was generally described as dense to very dense, brown to grey, moist. The SPT “N” values of the till ranged between 20 and 100 (i.e. SPT refusal), per 300 mm penetration. The till extended to the depths ranging between about 6.8 m and 14.6 m below ground surface, and extended to elevations ranging between about 417.3 m and 418.9 m. BH301, BH303 and BH304 were terminated within the till.

Although no specifically identified in the boreholes, typically till by nature of their deposition likely contain cobbles and boulders and the presence of these materials should be expected.

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

Moisture content:

- 5.2% to 26.7%

Grain size distribution:

- 16% to 29% gravel,
- 36% to 40% sand,
- 29% to 37% silt, and
- 6% to 7% clay size

Total saturated unit weights have been calculated based on the moisture contents and are estimated to range from about 19.5 to 24.5 kN/m<sup>3</sup>. 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 6 in Appendix D.

#### 1.4.6 Cobbles and Gravel

Rock coring techniques were initiated at BH302 only to determine the nature of refusal. Cobbles and gravel were encountered underlying the till at BH302. The cobbles and gravel were described as very dense, grey, wet, containing some sand and some boulders. An SPT "N" value of 100 (i.e. SPT refusal), per 300 mm penetration occurred. The cobbles and gravel extended to about 16.6 m below ground surface (elevation 415.3 m).

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

Moisture content:

- 7.0%

#### 1.4.7 Bedrock

As noted above, rock coring was initiated within the cobbles and gravel strata, and rock coring techniques, NQ size, were continued at BH302, to about 2.8 m depth into bedrock. Bedrock was encountered at about 415.3 m elevation, and about 16.6 m below asphalt surface.

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

Gross recoveries ranged between about 99% and 100%. The Rock Quality Designation (RQD), which is a modified core recovery, ranged from 24% to 26% (very severely fractured to severely fractured).

## 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 <sup>2</sup>	Depth to Water <sup>3</sup>	Groundwater Elevation
BH301	Mar. 5/15	Mar. 11/15 <sup>4</sup>	431.84	6.42	425.42
BH302	Mar. 11/15	Mar. 11/15	431.97	6.10	425.87
BH303	Mar. 18/15	Mar. 18/15	426.57	1.52	425.05
BH304	Mar. 12/15	Mar. 12/15	425.70	5.26	420.44
Whitewood Creek WL Upstream (West) Side	--	Mar. 18/15	--	--	425.29 <sup>5</sup>
Whitewood Creek WL Downstream (East) Side	--	Mar. 5/15	--	----	424.96 <sup>5</sup>
Whitewood Creek WL Downstream (East) Side	--	Mar. 12/15	--	----	424.99 <sup>5</sup>
Notes: 1) All units in metres. 2) Elevations surveyed are referenced to a geodetic benchmark (BM) provided by the client (nail in Pine tree north of the existing culvert). The elevation of the BM is 434.147 m. 3) Depths are relative to ground surface. 4) Augers were left in at the borehole location until March 11, 2015. Between March 5 and 11, BH301 was covered with asphalt on top of a wooden board which was resting on the augers. 5) Indicates top of ice elevation at Whitewood 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)
BH302-S10	6.20	370	<20	1,500	0.69
BH303-S4	7.63	79	140	2,400	0.42

## **PART II: ENGINEERING DISCUSSIONS AND RECOMMENDATIONS**

### **2.1 General**

This section of the report provides geotechnical design recommendations for replacement of the existing of Whitewood Creek Culvert on Highway 590, located approximately 14 km west of the junction of Highway 590 and Highway 11/17 at Whitewood Creek, in the Township of Marks. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the current investigation at the site and presented in **Part I-Foundation Investigation Report**. The interpretation and recommendations provided are intended solely to permit designers to assess foundation alternatives and design the 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 corrugated steel pipe (CSP) culvert with a length of about 41.8 m and diameter of about 3.0 m. It is understood that the existing culvert was constructed/installed in 1977, and inspected in July 2013. The inspection remarked that the existing culvert was in in fair condition with mostly moderate deterioration. Severe corrosion with small perforations at WL near outlet for 5 m and small areas of deformations in soffit were observed. It is also understood that the existing culvert is intended to be replaced with a new culvert along the same 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: CSP circular culvert, 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.

This part of the report addresses the geotechnical design of the foundation for the new culvert by providing geotechnical design parameters at the Ultimate Limit State (ULS) and Serviceability Limit States (SLS) as well as other geotechnical parameters that may be required in accordance with the latest edition of the *Canadian Highway Bridge Design Code (CHBDC) (CAN/CSA-S6-14)*, the *Canadian Foundation Engineering Manual (CFEM) (2006)*, *MTO Gravity Pipe Design Guidelines (May 2007)* and generally accepted good practice. Pertinent construction issues from a geotechnical standpoint are examined in general accordance with the Terms of Reference from the MTO letter dated 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.

### **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 6.5 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 7 m high with side slopes of about 2H:1V west of the roadway and 2.5H:1V east of the roadway. The current elevation of the crest of the roadway is about 431.80 m.

- b. The highway embankment consists of layers of poorly graded sand with silt and gravel fill (~1.1 to 1.2 m thick) and sandy silt with clay fill (~2.7 m thick), underlain by sandy silt fill (~3.0 m thick) in BH301 and well graded gravel with sand fill (~1.5 thick) in BH302. These fill layers extend to about 6.9 m and 5.3 m below the road surface in BH301 and BH302, respectively.
- c. The embankment fill is underlain by native stiff to hard clayey silt with sand (~ 6.1 m thick) in BH301 and compact silty sand with gravel (~2.3 m thick) in BH302 underlain by hard to very stiff clayey silt with sand (~5.5 m). At both boreholes the clayey silt layers are underlain by very dense silty sand with gravel till followed by a layer of very dense cobbles and gravel. The bedrock was encountered at 16.6 m depth below the ground surface (~415.3 m).
- d. At the inlet, a surficial layer of topsoil (~0.8 m thick) is underlain by firm to hard clayey silt with sand (~5.0 m thick), and followed by compact to very dense silty sand with gravel till. At the outlet, a surficial layer of topsoil (~0.9 m thick) is underlain by a thin soft layer of silt and peat (~0.6 m thick), and followed by firm to hard clayey silt with sand (~1.6 m thick) and very dense silty sand with gravel till. The practical refusal at inlet and outlet sides was at about 7.8 to 6.8 m below ground surface, respectively.
- e. The foundation soil at the invert of the new culvert is anticipated to be native very stiff to hard clayey silt with sand at approximately Elev. 424 m. Typical 'N' values ranged from 26 to 100.
- f. At the time of investigation the water in the creek was frozen and approximate top of ice elevation was about 425.29 m at the inlet and 424.99 m at the outlet. The groundwater table in the embankment fill is expected to be at approximate elevation 425.00 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:

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

Based on the subsurface information obtained from the site investigation, the native very stiff to hard clayey silt with sand at approximately Elev. 424 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.

Table 2.1 compares the structure options from a foundations design and constructability perspective. Although the foundation soils are generally good and will provide adequate support for all options listed in the table, the use of closed bottom pre-cast concrete box culvert is recommended as the most practical and economical.

Table 2.1 Evaluation of culvert replacement alternatives

Options	Advantages	Disadvantages	Relative Costs	Risks/Consequences	Rank
Circular CSP culvert	<ul style="list-style-type: none"> <li>▪ Straightforward construction</li> <li>▪ Reduce construction period, consequently traffic management and water control period</li> <li>▪ Reduce excavation depth</li> </ul>	<ul style="list-style-type: none"> <li>▪ Dewatering system required</li> <li>▪ Corrosion potential due to pH levels and chloride content of soil</li> </ul>	<ul style="list-style-type: none"> <li>▪ Low to medium</li> </ul>	<ul style="list-style-type: none"> <li>▪ Risk of unacceptable differential settlements if the entire foundation is not supported on the competent soil</li> <li>▪ Risk of structure segment loss due to corrosion</li> </ul>	2
Closed bottom pre-cast concrete box culvert	<ul style="list-style-type: none"> <li>▪ Straightforward construction</li> <li>▪ Reduce construction period, consequently traffic management and water control period</li> <li>▪ Reduce excavation depth</li> </ul>	<ul style="list-style-type: none"> <li>▪ Dewatering system required</li> <li>▪ Require heavy lifting equipment</li> </ul>	<ul style="list-style-type: none"> <li>▪ Low to medium</li> </ul>	<ul style="list-style-type: none"> <li>▪ Risk of unacceptable differential settlements if the entire foundation is not supported on the competent soil</li> <li>▪ Risk of leaking from joints if not properly installed</li> </ul>	1
Open bottom concrete box, concrete arch, and CSP arch	<ul style="list-style-type: none"> <li>▪ Wider span may consider to maintain existing channel</li> <li>▪ High geotechnical resistance available</li> <li>▪ Can incorporate dowels to enhance lateral resistance</li> </ul>	<ul style="list-style-type: none"> <li>▪ Deeper excavation or below water excavation may required</li> <li>▪ Dewatering system required</li> <li>▪ Possible uneven bedrock surface</li> </ul>	<ul style="list-style-type: none"> <li>▪ Likely more expensive than other options</li> </ul>	<ul style="list-style-type: none"> <li>▪ Risk of unacceptable differential settlements if the entire foundation is not supported on the competent soil</li> <li>▪ Risk of delay in construction due to deeper excavation below water if proper dewatering is not maintain</li> <li>▪ Higher scour risk</li> </ul>	4



Steel sheet pile abutments with precast concrete deck	<ul style="list-style-type: none"> <li>▪ Environmentally friendly</li> <li>▪ Easy to construct</li> <li>▪ No need for dewatering and cofferdam</li> <li>▪ No need for detour</li> <li>▪ No need to redirect existing creek water</li> <li>▪ No need for decommissioning of shoring system</li> <li>▪ Cost effective</li> </ul>	<ul style="list-style-type: none"> <li>▪ Relatively new approach for MTO</li> <li>▪ Due to possible lateral movement need an anchor system, bracing or deadman</li> <li>▪ Durability issue with sheet pile walls</li> <li>▪ Some difficulty in excavating under concrete span</li> </ul>	<ul style="list-style-type: none"> <li>▪ Likely more expensive than Option 1 and 2</li> </ul>	<ul style="list-style-type: none"> <li>▪ Risk of cost overrun and inability to finish job: low to moderate</li> </ul>	3
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Areas of any loose and/or soft soils encountered below the existing culvert should be excavated and removed to firm bearing of native soils and 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 Shallow Foundation

### 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)
CSP culvert/Closed bottom pre-cast concrete box culvert	~424.0 or below	3.0	Minimum 300 mm compacted granular material (Granular A or Granular B Type II) over native very stiff to hard clayey silt	600	400
Open bottom concrete box, concrete arch, and CSP arch	~421.8*	1.0	Native hard clayey silt	400	250

Notes:

\*Below the frost line

\*\* for maximum settlement of 25 mm

It is assumed that any 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.

### 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 pre-cast concrete and Granular A pad	Coefficient of friction ( $\tan \delta$ )=0.5
Between cast-in-place concrete and clayey silt	Coefficient of friction ( $\tan \delta$ )=0.35

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 Deep Foundation

### 2.3.3.1 Steel Sheet Piles

Sheet piles can be used for retaining backfill soil during excavation, control of groundwater, as well as bearing elements to support culvert foundations for the option culvert replacement with steel sheet pile abutments and precast concrete decking. For design, a PZ-22 section can be considered.

#### Axial Resistance in Compression

Steel sheet piles driven to the very dense silty sand with gravel till can be designed using the factored (0.4) resistance values. The factored resistance values (per metre width of sheet pile) have been calculated as 125 kN for minimum 5.8 m embedment (tip elevation approx. 418.2 m). These values are based on a static analysis, considering skin friction and nominal end bearing resistance, using the effective stress  $\beta$

method. It is noted that, since the sheet piles will also be retaining the approach fills, only the embedded, outside portion of the sheet piles below the level of the creek bed is considered to contribute to axial resistance. The elastic compression at ULS should be less than 6 mm in all cases. Since there is no (or minimal) proposed grade raise, negative skin friction or drag loads are not a concern.

### Pile Installation

Contractor's means and methods are their own responsibility. The following comments are provided for information. Piles should be installed in accordance with OPSS 903. The possibility of piles encountering potential cobbles and boulders in the till layers should be anticipated. Therefore, panel installation combined with staggered driving is recommended. This method improves verticality and alignment, reducing risk of driving problems and jumped interlocks. Driving in stages guides each pile as it is driven between neighbouring pairs. Toe reinforcement can be used to increase the strength of the toe of the sheet pile and help maintain the shape where difficult driving conditions are encountered. However, the reinforced toe does not provide increased protection to the interlock. Therefore, it is recommended as rule of thumb that no sheet pile be driven more than one-third its length before adjacent sheet piling is driven. Rigid guides and frames are essential to maintain horizontal and vertical alignment during driving, prevent piles from leaning or twisting and assist in driving when obstructions or hard ground are encountered. Care must be taken to avoid overdriving and damaging the pile tip (i.e., the structural capacity of the piles should not be exceeded).

In addition, all piles should be visually monitored by experienced personnel during installation to check for plumbness, set, internal damage, jump out of interlock etc. All damaged piles or jump out of interlock piles should be rejected and re-driven.

### Lateral Resistance

For relatively short (typically less than 3 m to 4 m) abutments, a cantilever sheet pile design using the earth pressure coefficients and soil parameters provided in Section 2.3.4, following. Note that if this design is implemented, the precast concrete deck will likely be designed to be installed such that lateral support is provided at the top of the sheet piles.

Depending on the abutment height and steel sheet section used, additional anchorage or tiebacks may be required. Conventional practice is to incorporate either buried deadman anchors or grouted soil anchors.

Deadman anchors can be designed based on the earth pressure coefficients and soil parameters provided in Section 2.3.4, following. For this project, either continuous or individual concrete block anchors would likely be appropriate. The anchor resistance is provided by a combination of the dead weight and passive resistance. For the full passive resistance to be realized with no load transfer to the wall, the anchor needs to be fully beyond the active wedge acting on the wall.

Pressure grouted soil anchors can be designed in a preliminary fashion in accordance with Section 26 of the CFEM (2006). Based on the generally compact to dense silt and sandy soils at this site, the estimated factored (0.4) ULS resistance of grouted anchors would be 40 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.

### 2.3.4 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:

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

$P$  = earth pressure intensity at depth  $h$ , kPa

$K$  = earth pressure coefficient

$\gamma$  = unit weight of retained soil, kN/m<sup>3</sup>

$q$  = surcharge near wall, kPa

$h$  = depth to point of interest, m

$H$  = 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 $\phi'$	Coefficient of Active Earth Pressure ( $K_a$ )	Coefficient of Passive Earth Pressure ( $K_p$ )	Coefficient of Earth Pressure at Rest ( $K_0$ )	Unit Weight $\gamma$ kN/m <sup>3</sup>
Sand to Silty Sand Fill	32°	0.31	3.25	0.47	21
Sandy Silt with some Clay Fill	28°	0.36	2.77	0.53	20
Sandy Silt Fill	30°	0.33	3.0	0.5	20
Gravel Fill	35°	0.27	3.69	0.43	22
Clayey Silt with Sand	30°	0.33	3.0	0.5	19
Silty Sand with Gravel	35°	0.27	3.69	0.43	21
Silt and Peat	20°	0.49	2.04	0.66	17
Silty Sand with Gravel Till	35°	0.27	3.69	0.43	21
Cobbles and Gravel	38°	0.24	4.26	0.38	20

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
- C. Construction using roadway protection and a steel sheet pile abutments with precast concrete deck system

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

Installation Method	Advantages	Disadvantages	Relative Cost	Ranking
<p>OPTION 1</p> <p>Full Road Closure using Existing Roadways and Open Cut Unsupported Excavation</p>	<ul style="list-style-type: none"> <li>Existing culvert will be completely remove and replaced with new culvert</li> <li>No construction of detour roads or roadway protection required</li> <li>No excavation support required</li> <li>Install entire new culvert at once</li> <li>Straightforward construction</li> <li>Short mobilization time</li> <li>Low capital investment; cost saving in time and materials required for construction</li> </ul>	<ul style="list-style-type: none"> <li>Traffic interruption</li> <li>Long detour around site using other existing roads required</li> <li>Large amount of soil to be excavated</li> <li>Existing fills and native soils require 2H:1V side slopes to maintain stability</li> <li>Erosion control of temporary cuts required</li> <li>Need to temporarily control existing water flow</li> <li>Potential claims to compensate vehicle occupants and local business for delays or time lost due to detour routes</li> <li>Risk of cost overrun and inability to finish job: low</li> </ul>	<p>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</p>	2
<p>OPTION 2</p> <p>Temporary Local Detour and Open Cut Unsupported Excavation</p>	<ul style="list-style-type: none"> <li>Traffic flow maintained at the site during construction</li> <li>Simple detour roads can be constructed</li> <li>Existing culvert will be completely remove and replaced with new culvert</li> <li>No excavation support required</li> <li>Install entire new culvert at once</li> </ul>	<ul style="list-style-type: none"> <li>Traffic interruption</li> <li>Construction of detour embankments required at each side of highway</li> <li>Difficulties to construct detours due to inaccessible surrounding terrain</li> <li>Increased time for construction of detour</li> <li>Large amount of soil to be excavated</li> <li>Erosion control of temporary cuts required</li> <li>Need to temporarily control existing creek water flow</li> <li>Possible settlement due to new earth embankment fill</li> <li>Risk of cost overrun and inability to finish job: low to moderate</li> <li>Possible extra cost to purchase of private</li> </ul>	<p>More expensive than full road closure due to high costs to build local detours</p>	5

Installation Method	Advantages	Disadvantages	Relative Cost	Ranking
		property		
OPTION 3.A  Half-and-half Construction with Unsupported Cut Sides	<ul style="list-style-type: none"> <li>• Traffic flow maintained at the site during construction</li> <li>• Short mobilization time</li> <li>• Straight forward construction and construction procedures</li> </ul>	<ul style="list-style-type: none"> <li>• Traffic interruption</li> <li>• Roadway protection of up to 6.0 m high required to maintain one lane of traffic</li> <li>• High cost of roadway protection system</li> <li>• Large amount of soil to be excavated</li> <li>• Need to temporarily control existing creek water</li> <li>• Risk of cost overrun and instability to finish job: low to moderate</li> </ul>	Relatively more expensive than full road closure due to high costs of roadway protection system	3
OPTION 3.B  Half-and- half Construction with Braced or Anchored Cut Sides	<ul style="list-style-type: none"> <li>• One or possibly two lanes of traffic flow maintained on existing road (e.g. steel decking, but costly)</li> <li>• Global stability of excavation enhanced by narrow geometry</li> <li>• Less traffic interruption than with unsupported cut sides approach</li> <li>• Temporary decking could be usable over braced cut to allow for excavation of both halves prior to diverting stream and backfilling</li> <li>• Cost savings due to limited excavation and backfill</li> </ul>	<ul style="list-style-type: none"> <li>• Traffic interruption</li> <li>• Roadway protection of up to 6.0 m high required to maintain one lane of traffic if steel docking is not possible</li> <li>• High cost of roadway protection system and/or decking</li> <li>• Require side shoring and bracing</li> <li>• Bracing (e.g. struts) may interfere with excavation</li> <li>• Excavation of material and placement of bracing required in limited space</li> <li>• Need to decommission the shoring system</li> <li>• Need to temporarily control existing creek water</li> <li>• Risk of cost overrun and instability to finish job: low to moderate</li> </ul>	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	4



Installation Method	Advantages	Disadvantages	Relative Cost	Ranking
<p>OPTION 3.C</p> <p>Half-and-half Construction with Installation of Steel Sheet Pile Abutments with Precast Concrete Deck</p>	<ul style="list-style-type: none"> <li>• Environmentally friendly</li> <li>• Easy to construct</li> <li>• No need for dewatering and cofferdam</li> <li>• No need for detour</li> <li>• No need to redirect existing creek water</li> <li>• No need for decommissioning of shoring system</li> <li>• Cost effective</li> <li>•</li> </ul>	<ul style="list-style-type: none"> <li>• Traffic interruption</li> <li>• Roadway protection required to maintain one lane of traffic</li> <li>• High cost of roadway protection system</li> <li>• Relatively new approach for MTO</li> <li>• Due to possible lateral movement need an anchor system, bracing or deadman</li> <li>• Durability issue with sheet pile walls</li> <li>• Some difficulty in excavating under concrete span</li> <li>• Difficulties for sheet pile driving due to presence of cobbles and boulders</li> <li>• Risk of cost overrun and inability to finish job: low to moderate</li> </ul>	<p>Relatively more expensive than full road closer due to high costs of shoring abutments, but more practical</p>	<p>1</p>

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: Half-and-half construction with the steel sheet pile abutments and precast concrete deck (Figure H3.C, Appendix H)
2. Full road closure using existing roadways and open cut unsupported excavation (Figure H1, Appendix H)
3. OPTION 3.A: Half-and-half construction with unsupported cut sides (Figure H3.A, Appendix H)
4. OPTION 3.B: Half-and-half construction with braced or anchored cut sides (Figure H3.B, Appendix H)
5. 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. silty clayey sand and sandy silty clay 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 from below the proposed temporary detour road embankment. All subgrade soils should be proof-rolled prior to fill placement and embankment fill should be placed in accordance with OPSS. PROV 206 (dated November 2014).

#### 2.4.1.2 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
- C. Construction using roadway protection and a steel sheet pile abutments with precast concrete deck system

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 soldier pile and lagging with rakers or anchors for horizontal support. Where the cut extends below prevailing groundwater a suitable control/system is required. Once one lane is completed the supports can be reversed and the other lane constructed in similar fashion. The shoring system would likely be decommissioned in place. Temporary surface water flow control must be developed by contractor.

Option 3.A could be more economical due to possible cost savings for reversible wall configuration, but it will be more disruptive to the highway embankment than Options 3.B since it needs to excavate a large amount of soil.

#### 2.4.1.2.2 Option 3.B: Half-and-Half Construction with Braced or Anchored Cut Sides

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

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

#### 2.4.1.2.3 Option 3.C: Half-end-Half Construction using Steel Sheet Pile Abutments with Precast Concrete Deck

This option provides shoring system consisting of sheet piles perpendicular to the highway, which will serve the dual purpose of retaining backfill soil during excavation and being bearing elements to support culvert foundations after excavation (see Figure H3.C, Appendix H). As shown on Figure H3.C, the sheet piles will be installed perpendicularly in the half of the embankment at both sides of the existing culvert after installation of the roadway protection system for Stage 1 construction. Next the fill will be excavated to the designed elevation of the deck and its precast panels will be

installed over the existing culverts. Then the fill below the deck panels will be excavated within construction limits for Stage 1 allowing the existing culverts to be removed. The excavation above the deck will be backfilled with a free-draining granular material up to the highway grade. The same processes will be repeated in Stage 2 construction, on the other side of the roadway protection. The contractor should be responsible for the complete design, construction and monitoring of the described system. It is their responsibility to provide the work and design that should accommodate all relevant conditions including local and global stability for all stages of installation, including any necessary groundwater or surface water controls.

A major benefit of this method is that sheet piles will be permanently installed creating abutments for the precast concrete deck, so decommissioning of the shoring system is eliminated. Further, this method allows the use of the existing culvert to convey the cedar creek water flow below Hwy 590 even during the construction work. However, due to the depth of the fill, some lateral movement of the abutments might be possible and installation of anchor systems, bracing or deadman might be required.

#### **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 relatively shallow depth and a potential insufficient embedment depth, a simple cantilevered approach might not be possible at this site. Temporary shoring such as a soldier pile and lagging system with additional anchorage or tiebacks may be required for lateral resistance. Conventional practice is to incorporate either buried deadman anchors or grouted anchors.

Deadman anchors can be designed based on the earth pressure coefficients and soil parameters provided in Section 2.5.1, following. For this project, either continuous or individual concrete block anchors would likely be appropriate. The anchor resistance is provided by a combination of the dead weight and passive resistance. For the full passive resistance to be realized with no load transfer to the wall, the anchor needs to be fully beyond the active wedge acting on the wall.

Pressure grouted soil anchors can be designed in a preliminary fashion in accordance with Section 26 of the CFEM (2006). Based on the generally compact to dense sandy soils at this site, the

estimated factored (0.4) ULS resistance of grouted anchors would be 40 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  $\mu\text{m}$ .

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 clayey silt. This material has high frost susceptibility based upon the MTO Frost Classification guideline of percent particles between 5 to 75  $\mu\text{m}$ . Therefore, non-frost susceptible materials such as sand and gravel 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 firm to hard clayey silt with sand (~3.5 m thick) underlain by the layers of very stiff to stiff clayey silt (~1.5 m thick), and compact to very dense silty sand with gravel till (~2.0 m thick). The practical refusal is encountered at Elev. 418.8 m (~7.8 m below the ground surface). At the outlet side, the subsurface condition at the location of cofferdam consists of a thin layer of soft silt and peat (~0.6 m thick) underlain by a layer of firm to hard clayey silt with sand (~1.6 m thick) and a ~3.7 m thick layer of silty sand with gravel till. The practical refusal at this location is at Elev. 418.8 m (~7.8 m below the ground surface). Therefore, based on these geotechnical conditions, a sheet pile or rockfill cofferdam can be recommended to be used as a cofferdam at the site. The earth pressure coefficients and soil parameters provided in Section 2.3.4 for the sheet pile cofferdam design. If the rockfill cofferdam is chosen, 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 water level in the creek 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 sandy silt and silty sand fills and native silty sand and clayey silt. 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. For deeper excavations relative to the groundwater level, more positive dewatering/ groundwater control in the form of steel pile confinement or well point systems would be required. This would for instance apply to deeper excavations for open bottom culvert footings.

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 7 m high with side slopes of about 2H:1V west of the roadway and 2.5H:1V east of the roadway.

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 and clayey silt deposits. Therefore, an effective stress analysis for a long term stability assessment of the embankment slope was performed taking into consideration the subsoil conditions encountered beneath the existing embankment.

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

Tabulated below in Table 2.6 are the soil parameters used for the slope stability analysis. The soil parameters were generally estimated based on the results of field and laboratory investigation.

*Table 2.6 Soil properties used in slope stability analysis*

Material	$\phi'$ (degrees)	$c'$ (kPa)	$\gamma'$ (kN/m <sup>3</sup> )
Engineered Fill	32	0	21
Clayey Silt (Very Stiff to Hard)	30	0	19
Silty Sand with Gravel (Compact)	35	0	21
Silt and Peat (Soft)	20	0	17
Silty Sand with Gravel Till (Compact to Very Dense)	35	0	21
Cobbles and Gravel (Very Dense)	38	0	20

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 Inlet and Outlet**

### **2.9.1 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.

### **2.9.2 Stream Bed Rip-Rap/ Scour Protection**

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. Footings for open bottom culverts, if this option is selected, must be provided with suitable scour protection as confirmed by the project hydrologist.

### **2.9.3 Seepage Cut-off Requirements**

The seepage cut-off requirements should be reviewed in the following context. The native clayey 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.

### 2.9.3.1 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 shall be natural clay, clay mixture (1 part Bentonite powder and 3.5 parts Granular "A") or a geosynthetic clay liner (GCL). The coefficient of permeability shall not exceed  $1 \times 10^{-6}$  cm/s.

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

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

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

### 2.9.3.2 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.

## 2.10 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. The maximum chloride content is 370 ppm ( $\mu\text{g/g}$ ) i.e. 0.037% which exceeds 0.025% and indicates a low to moderate potential for additional corrosion. . The soil pH was about 6.9 (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 maximum water soluble sulphate content of the soils tested is 140 ppm ( $\mu\text{g/g}$ ), i.e. 0.014% and being less than 0.10%, does not require sulphate resistant cement. These data also support our local experience.

April 15, 2016

### PART III: Closure

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

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

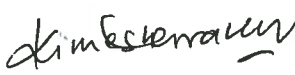
A subsurface investigation is a limited sampling of a site; the subsurface conditions have been established only at the test hole locations. Should conditions at the site be encountered which differ from those reported at the test locations, we require that we be notified immediately in order to assess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

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

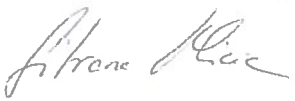
This Foundation Investigation and Design Report has been prepared by Ahileas Mitsopoulos, P.Eng., Nimesh Tamrakar, M.Eng, EIT., Demetri N. Georgiou, M.A.Sc. P.Eng., and Silvana Micic, Ph.D., P.Eng. It was reviewed by TaeChul Kim, P.Eng. and by Stan E. Gonsalves, M.Eng., P.Eng., Designated MTO Foundation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

Yours truly,

**exp Services Inc.**

  
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Stan E. Gonsalves, M.Eng., P.Eng.  
Principal Engineer  
Designated MTO Foundation Contact

Encl.



## **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 of existing culvert at west side of Highway 590



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



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



Photo 4. Facing south on Hwy 590 before the existing culvert



April 15, 2016

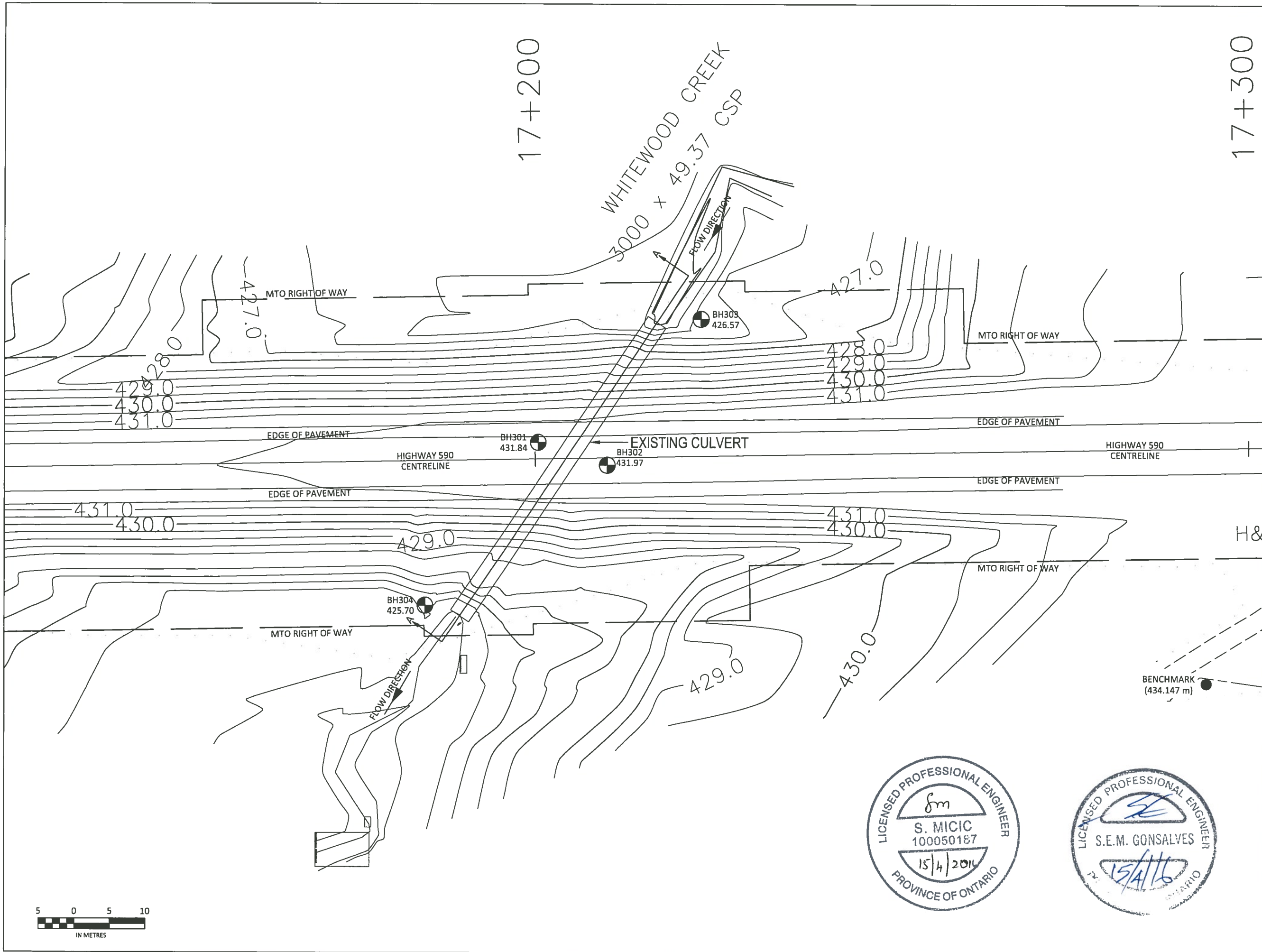


Photo 5. Embankment slope on east side facing north



Photo 6. Embankment slope on west side facing south

## **Appendix B – Drawings**



Agreement No. 6014-E-0017  
Assignment No. 2  
GWP 6349-14-00

WHITEWOOD CREEK CULVERT  
(Highway 590, Marks Township)  
PLAN

DWG  
1

\*exp.

exp Services Inc.

KEY PLAN

LEGEND

BH301 BOREHOLE LOCATION  
431.84 GROUND SURFACE ELEVATION IN METRES

BH No.	APPROX. ELEV. (m)	MTM COORDINATES	
		NORTH	EAST
BH301	431.84	5,359,723	320,570
BH302	431.97	5,359,736	320,573
BH303	426.57	5,359,749	320,557
BH304	425.70	5,359,711	320,594

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

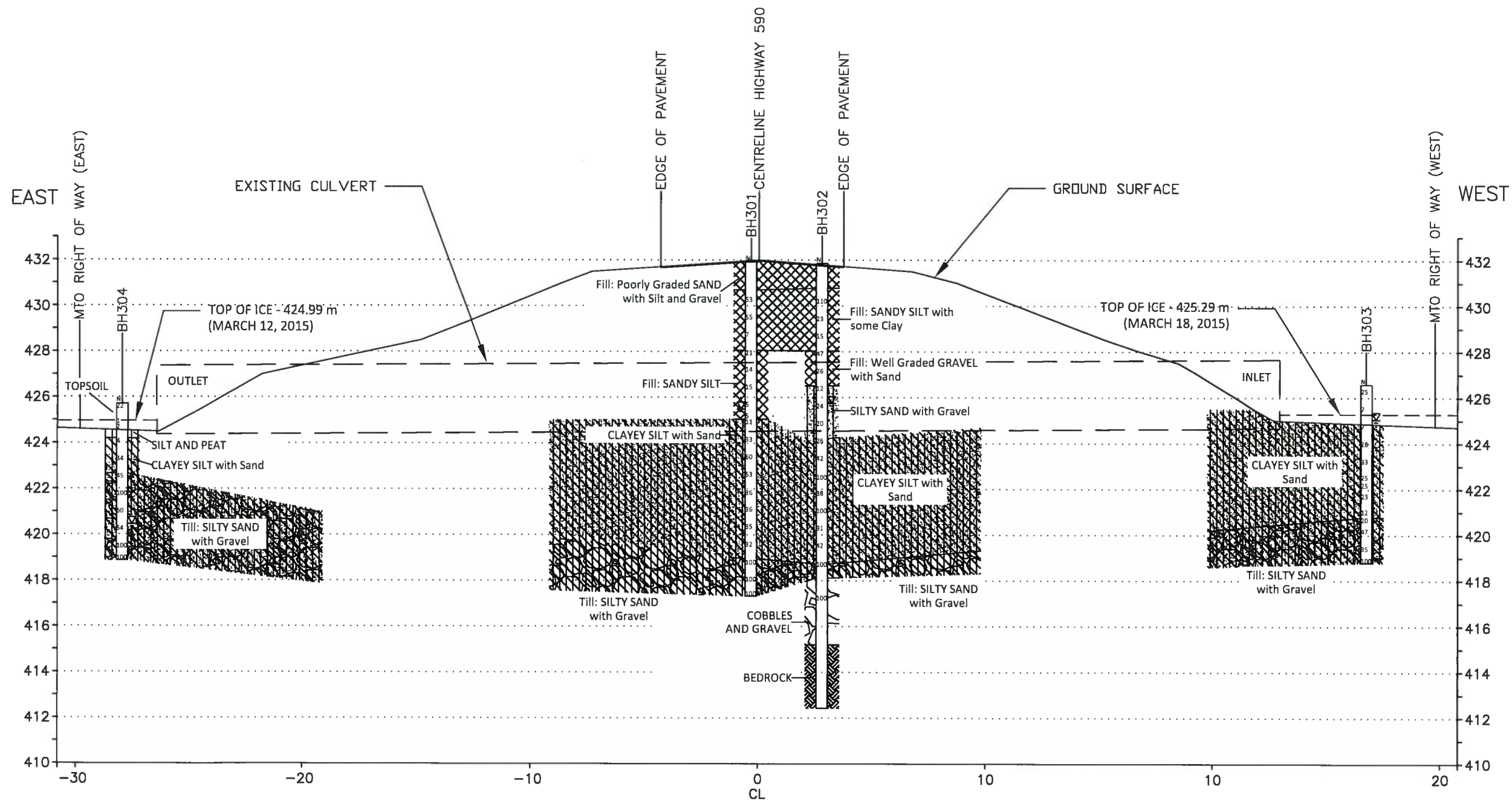
DATE	BY	DESCRIPTION

GEOCRES No. 52A-213  
Date: April 24, 2015  
Drawn By: RM

Project No. ADM-00223648-B0  
Scale : 1:500  
Checked By: AM  
Checked By: DG







A-A  
PROFILE OF WHITEWOOD CREEK CULVERT



Agreement No. 6014-E-0017  
Assignment No. 2  
GWP 6349-14-00

WHITEWOOD CREEK CULVERT  
(Highway 590, Marks Township)  
CROSS SECTION

exp. exp Services Inc.

KEY PLAN

LEGEND

SOIL STRATA SYMBOLS

BH No.	APPROX. ELEV. (m)	MTM COORDINATES	
		NORTH	EAST
BH301	431.84	5,359,723	320,570
BH302	431.97	5,359,736	320,573
BH303	426.57	5,359,749	320,557
BH304	425.70	5,359,711	320,594

NOTES

REVISIONS

DATE	BY	DESCRIPTION

GEOCRES No. 52A-213  
Date: May 15, 2015  
Drawn By: RM  
Checked By: DG

Project No. ADM-00223648-B0  
Horizontal Scale : 1:200  
Vertical Scale : 1:200  
Checked By: AM

## **Appendix C – Borehole Logs and Bedrock Core Photos**

# Explanation of Terms Used on Borehole Records

## SOIL DESCRIPTION

Terminology describing common soil genesis:

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

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

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

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

Terminology describing soil structure:

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

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

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

*Fissured:* material breaks along plane of fracture.

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

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

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



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

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

*Homogeneous:* same color and appearance throughout.

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

*Uniformly Graded:* predominantly on grain size.

All soil sample descriptions included in this report follow generally the ASTM D2487-11 Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System) with some modification to reflect current MTO practices. The system divides soils into three major categories: (1) coarse grained, (2) fine-grained, and (3) highly organic. The soil is then subdivided based on either gradation or plasticity characteristics. The system provides a group symbol (e.g. SM) and group name (e.g. silty sand) for identification. The classification excludes particles larger than 76 mm. Please note that, with the exception of those samples where a grain size analysis has been made, all samples are classified visually in accordance with ASTM D2488-09a Standard Practice for Description and Identification of Soils (Visual-Manual Procedure). Visual classification is not sufficiently accurate to provide exact grain sizing or precise differentiation between size classification systems. Others may use different classification systems; one such system is the ISSMFE Soil Classification.

ISSMFE SOIL CLASSIFICATION											
CLAY	SILT			SAND			GRAVEL			COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE		
<div><div>0.002</div><div>0.006</div><div>0.02</div><div>0.06</div><div>0.2</div><div>0.6</div><div>2.0</div><div>6.0</div><div>20</div><div>60</div><div>200</div></div>											
EQUIVALENT GRAIN DIAMETER IN MILLIMETRES											
CLAY (PLASTIC) TO				FINE		MEDIUM		CRS.		FINE COARSE	
SILT (NONPLASTIC)				SAND				GRAVEL			
UNIFIED SOIL CLASSIFICATION											

Terminology describing materials outside the USCS, (e.g. particles larger than 76 mm, visible organic matter, construction debris) is based upon the proportion of these materials present and as described below in accordance with Note 16 in ASTM D2488-09a:

Table a: Percent or Proportion of Soil, Pp

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

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

Table b: Apparent Density of Cohesionless Soil

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

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

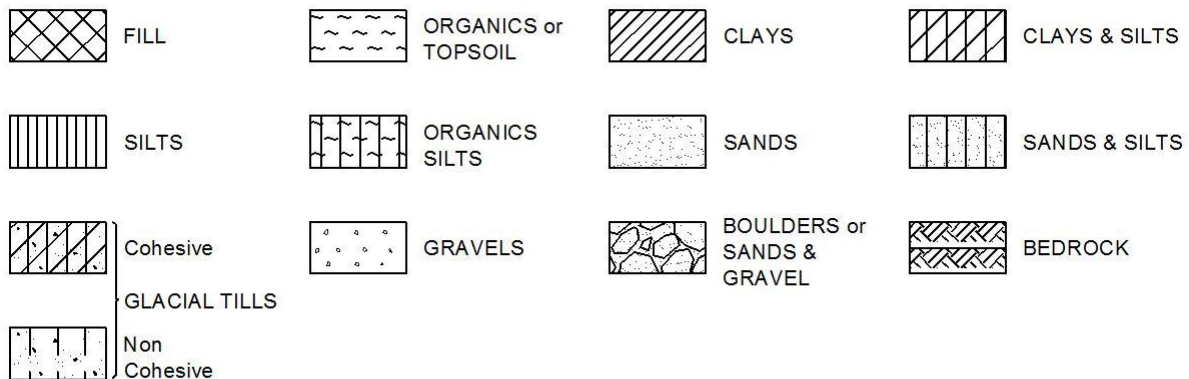
Table c: Consistency of Cohesive Soil

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

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

## STRATA PLOT

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



## WATER LEVEL MEASUREMENT



Open Borehole or Test Pit



Monitoring Well, Piezometer or Standpipe

## ABBREVIATIONS AND SYMBOLS

### FIELD SAMPLING

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

### STRESS AND STRAIN

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

### MECHANICAL PROPERTIES OF SOIL

$m_v$	$\text{kPa}^{-1}$	Coefficient of volume change
$c_c$	1	Compression index
$c_s$	1	Swelling index
$c_r$	1	Recompression index
$c_v$	$\text{m}^2/\text{s}$	Coefficient of consolidation
H	m	Drainage path
$T_v$	1	Time factor
U	%	Degree of consolidation
$\sigma'_{v0}$	kPa	Effective overburden pressure
$\sigma'_p$	kPa	Preconsolidation pressure
$\tau_f$	kPa	Shear strength
$c'$	kPa	Effective cohesion intercept
$\phi'$	$-\circ$	Effective angle of internal friction
$c_u$	kPa	Apparent cohesion intercept
$\phi_u$	$-\circ$	Apparent angle of internal friction
$\tau_R$	kPa	Residual shear strength
$\tau_r$	kPa	Remoulded shear strength
$S_t$	1	Sensitivity = $c_u/\tau_r$

### PHYSICAL PROPERTIES OF SOIL

$P_s$	$\text{kg}/\text{m}^3$	Density of solid particles
$\gamma_s$	$\text{kN}/\text{m}^3$	Unit weight of solid particles
$\rho_w$	$\text{kg}/\text{m}^3$	Density of water
$\gamma_w$	$\text{kN}/\text{m}^3$	Unit weight of water
$\rho$	$\text{kg}/\text{m}^3$	Density of soil
$\gamma$	$\text{kN}/\text{m}^3$	Unit weight of soil
$\rho_d$	$\text{kg}/\text{m}^3$	Density of dry soil
$\gamma_d$	$\text{kN}/\text{m}^3$	Unit weight of dry soil
$\rho_{sat}$	$\text{kg}/\text{m}^3$	Density of saturated soil
$\gamma_{sat}$	$\text{kN}/\text{m}^3$	Unit weight of saturated soil
$\rho'$	$\text{kg}/\text{m}^3$	Density of submerged soil
$\gamma'$	$\text{kN}/\text{m}^3$	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	$\text{m}^3/\text{s}$	Rate of discharge
v	m/s	Discharge velocity
i	1	Hydraulic gradient
k	m/s	Hydraulic conductivity
j	$\text{kN}/\text{m}^3$	Seepage force

Brampton, Ontario

## RECORD OF BOREHOLE No BH301

1 OF 1

METRIC

W. P. GWP No. 6349-14-00 LOCATION Whitewood Creek Culvert (Site No. 48W-168/C) MTM ON-15 320,570E 5,359,723N ORIGINATED BY EF  
 DIST Thunder Bay, Hwy 590 BOREHOLE TYPE CME 55 Truck Mount / HSA COMPILED BY RM  
 DATUM Geodetic DATE 2015/03/05 - 2015/03/05 CHECKED BY DG/AM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100					
431.8	Asphalt																
430.8	ASPHALT - about 50 mm		S1	AUGER													
	Poorly Graded SAND with Silt and Gravel (SP-SM) Fill - frozen, brown		S2	AUGER													
430.6			S3	AUGER			431						○				
1.2	SANDY SILT with some Clay (ML) Fill - very dense, frozen, brown, trace gravel		S4	AUGER													
			S5	SS	53		430							○			5 26 49 20
			S6	SS	55												
			S7	SS	7		429										
428.0														○			
3.9	SANDY SILT (ML) Fill - compact, brown, moist, trace to little gravel, trace to few clay		S8	SS	21		428										
			S9	SS	14		427							○			11 28 55 6
			S10	SS	15									○			
							426										
	- becoming moist to wet at about 6.1 m depth		S11A	SS	5									○			
425.0	- trace organics at about 6.6 m depth		S11B	SS	5												
6.9	CLAYEY SILT with Sand (CL) - stiff to hard, brown to grey, moist, some gravel		S12	SS	11		425								—○—		16 36 33 15
			S13	SS	33		424							○			
			S14	SS	60		423							—			0 17 66 17
			S15	SS	53									○			
			S16	SS	36		422								○		
			S17	SS	36		421								○		0 3 71 26
			S18	SS	35		420							○			
			S19	SS	32		419								○		
418.9	Silty SAND with Gravel (SM) Till - very dense, grey, moist		S20	SS	100									○			
13.0			S21	SS	100		418								○		
417.3																	
14.6	END OF BOREHOLE - refusal to SPT and auger		S22	SS	100									○			
	NOTES: 1. This drawing is to be read with the subject report and project numbers as presented above. 2. Groundwater level at 6.4 m depth upon completion of borehole.																

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

OPG\_EXP RECORD OF BOREHOLE F-15103-CG - ADM-00223648-B0 - MTO 2 - WHITEWOOD CREEK CULVERT.GPJ ONTARIO MOT.GDT 16/4/14

Brampton, Ontario

## RECORD OF BOREHOLE No BH302

1 OF 2

METRIC

W. P. GWP No. 6349-14-00 LOCATION Whitewood Creek Culvert (Site No. 48W-168/C) MTM ON-15 320,573E 5,359,736N ORIGINATED BY EF  
 DIST Thunder Bay, Hwy 590 BOREHOLE TYPE CME 55 Truck Mount & CME 850 Track Mount / HSA / NQ COMPILED BY RM  
 DATUM Geodetic DATE 2015/03/06 - 2015/03/11 CHECKED BY DG/AM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100					
432.0	Asphalt																
430.9	ASPHALT - about 50 mm		S1	AUGER													
	Poorly Graded SAND with Silt and Gravel (SP-SM) Fill - frozen, brown		S2	AUGER													38 52 (10)
430.9							431										
1.1	SILTY SAND with Gravel (SM) Fill - compact to very dense, frozen, brown, some clay		S3	AUGER													
	- becoming very stiff to stiff, moist at about 2.3 m depth		S4	SS	110		430										
			S5	SS	23												
			S6	SS	15		429										
428.2																	17 41 33 9
3.8	Well graded GRAVEL with Sand (GW) Fill - dense to compact, brown, moist		S7	SS	47		428										
			S8	SS	26		427										48 43 (9)
426.6																	
5.3	Silty SAND with Gravel (SM) - compact, grey, moist		S9	SS	12		426										
	- trace organics at about 6.1 m depth		S10	SS	24												32 37 25 6
			S11	SS	20		425										
424.3																	
7.6	CLAYEY SILT with Sand (CL) - very stiff to hard, grey, moist to wet, trace gravel		S12	SS	26		424										
			S13	SS	42		423										1 8 70 21
			S14	SS	100												
			S15	SS	38		422										0 1 73 26
			S16	SS	100		421										
			S17	SS	91		420										
			S18	SS	42												
418.9			S19A	SS	49		419										
13.1	Silty SAND with Gravel (SM) Till - very dense, brown, moist		S19B	SS	100												
418.2	- refusal to SPT at about 13.4 m depth																
13.7	- rock coring initiated at 13.7 m depth		S20	CORE			418										
	COBBLES AND GRAVEL - very dense, grey, wet, some sand, occasional boulders		S21	SS	100		417										
			S22	CORE													
			S23	CORE			416										
415.3																	

Continued Next Page

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

OPG\_EXP RECORD OF BOREHOLE F-15103-CG - ADM-00223648-B0 - MTO 2 - WHITEWOOD CREEK CULVERT.GPJ ONTARIO MOT.GDT 16/4/14

Brampton, Ontario

## RECORD OF BOREHOLE No BH302

2 OF 2

METRIC

W. P. GWP No. 6349-14-00 LOCATION Whitewood Creek Culvert (Site No. 48W-168/C) MTM ON-15 320,573E 5,359,736N ORIGINATED BY EF  
 DIST Thunder Bay, Hwy 590 BOREHOLE TYPE CME 55 Truck Mount & CME 850 Track Mount / HSA / NQ COMPILED BY RM  
 DATUM Geodetic DATE 2015/03/06 - 2015/03/11 CHECKED BY DG/AM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100					
16.6	<b>BEDROCK</b> - medium strong, very severely to severely fractured, white and pink, medium to coarse grained, granite ( <i>continued</i> )		S24	CORE			415										Recovery=99%, RQD=24%
							414										
			S25	CORE			413										
412.6 19.4	<b>END OF BOREHOLE</b>  <b>NOTES:</b> 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.																

OPG\_EXP RECORD OF BOREHOLE F-15103-CG - ADM-00223648-B0 - MTO 2 - WHITEWOOD CREEK CULVERT.GPJ ONTARIO MOT.GDT 16/4/14






Brampton, Ontario

## RECORD OF BOREHOLE No BH303

1 OF 1

METRIC

W. P. GWP No. 6349-14-00 LOCATION Whitewood Creek Culvert (Site No. 48W-168/C) MTM ON-15 320,557E 5,359,749N ORIGINATED BY EF  
 DIST Thunder Bay, Hwy 590 BOREHOLE TYPE CME 45 Yanmar Track Mount / HSA COMPILED BY RM  
 DATUM Geodetic DATE 2015/03/18 - 2015/03/18 CHECKED BY DG/AM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)					
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH: Cu, KPa									WATER CONTENT (%)			GR	SA	SI
								○ UNCONFINED	+	FIELD VANE	×	QUICK TRIAXIAL	LAB VANE									
426.6	Topsoil		S1	SS	25		426															
425.8	CLAYEY SILT with Sand(CL) - firm to hard, brown, moist, some organics in upper 1.5 m		S2	SS	7		425															
0.8			S3	SS	7		424															
			S4	SS	28		423															
			S5	SS	33		422															
			S6A	SS	25		421															
			S6B	SS	25		420															
			S7	SS	23		419															
420.8	- becoming clayey silt very stiff to stiff, grey, moist		S8A	SS	12																	
5.8			S8B	SS	20																	
			S9	SS	47																	
			S10	SS	35																	
418.8	Silty SAND with Gravel (SM) Till - compact to very dense, brown, moist		S11	SS	100																	
7.8	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 1.5 m depth upon completion of borehole.																					

OPG\_EXP RECORD OF BOREHOLE F-15103-CG - ADM-00223648-B0 - MTO 2 - WHITEWOOD CREEK CULVERT.GPJ ONTARIO MOT.GDT 16/4/14

Brampton, Ontario

## RECORD OF BOREHOLE No BH304

1 OF 1

METRIC

W. P. GWP No. 6349-14-00 LOCATION Whitewood Creek Culvert (Site No. 48W-168/C) MTM ON-15 320,594E 5,359,711N ORIGINATED BY EF  
 DIST Thunder Bay, Hwy 590 BOREHOLE TYPE CME 850 Track Mount / HSA COMPILED BY RM  
 DATUM Geodetic DATE 2015/03/12 - 2015/03/12 CHECKED BY DG/AM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>P</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT  γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH: Cu, KPa										WATER CONTENT (%)		
								○ UNCONFINED		+ FIELD VANE								× QUICK TRIAXIAL		
425.7	Topsoil		S1	SS	22															
424.8	- becoming very loose/soft, moist to wet at about 0.8 m depth		S2A	SS	1															
0.9			S2B	SS	2															
424.2	SILT AND PEAT - soft, dark brown to black, wet																			
1.5	CLAYEY SILT with Sand (CL) - firm to hard, grey to brown, moist to wet		S3	SS	4															
			S4	SS	34															
422.7																				
3.1	Silty SAND with Gravel (SM) Till - very dense, brown to grey, moist		S5A	SS																
			S5B	SS	45															
			S5C	SS																
			S6	SS	100															
			S7	SS	60															
		S8	SS	54																
		S9	SS	100																
418.9																				
6.8	END OF BOREHOLE - refusal to SPT and auger		S10	SS	100															
<div>NOTES:</div> <div>1. This drawing is to be read with the subject report and project numbers as presented above.</div> <div>2. Groundwater level at 5.3 m depth upon completion of borehole.</div>																				

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

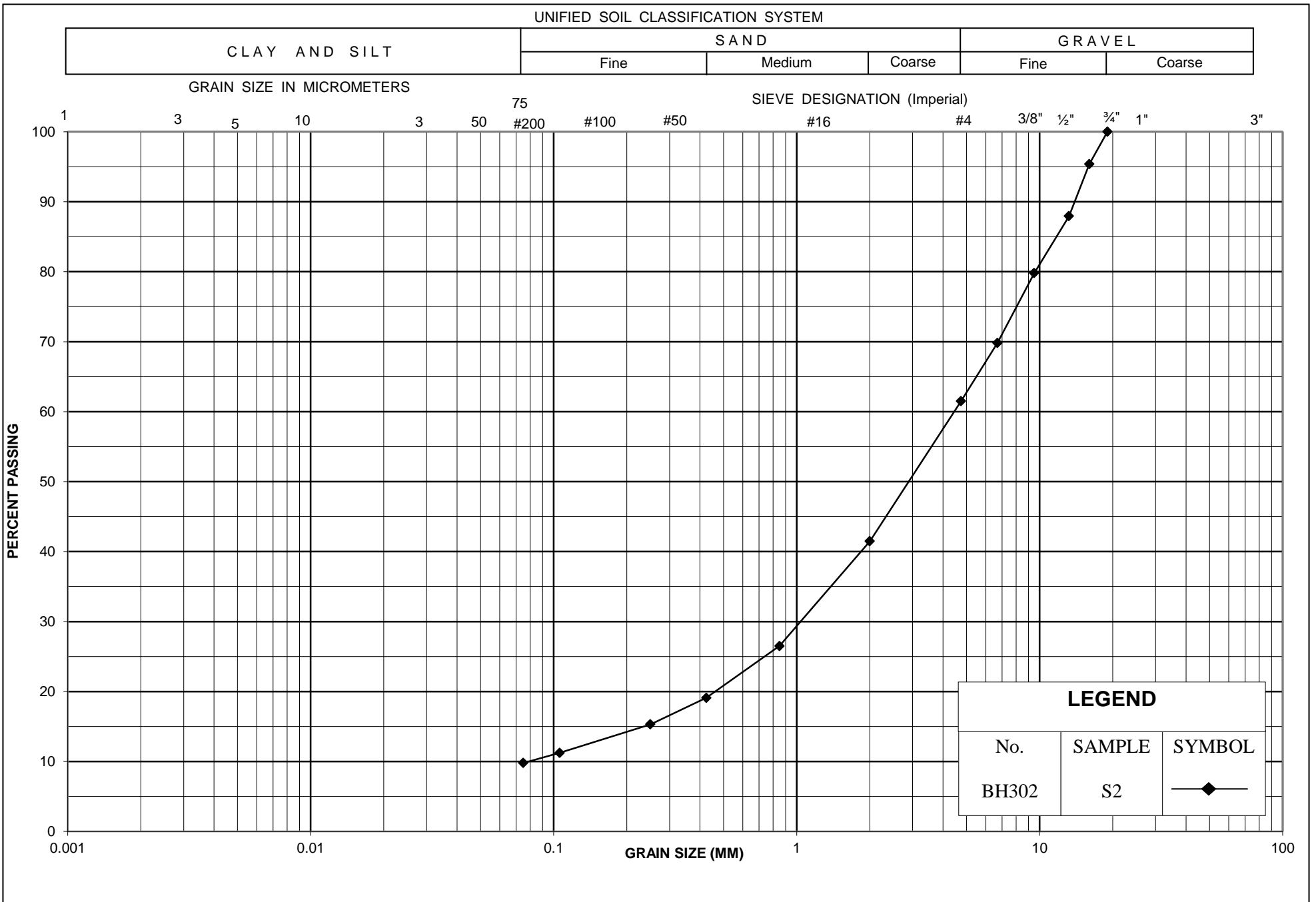
OPG\_EXP RECORD OF BOREHOLE F-15103-CG - ADM-00223648-B0 - MTO 2 - WHITEWOOD CREEK CULVERT.GPJ ONTARIO MOT.GDT 16/4/14

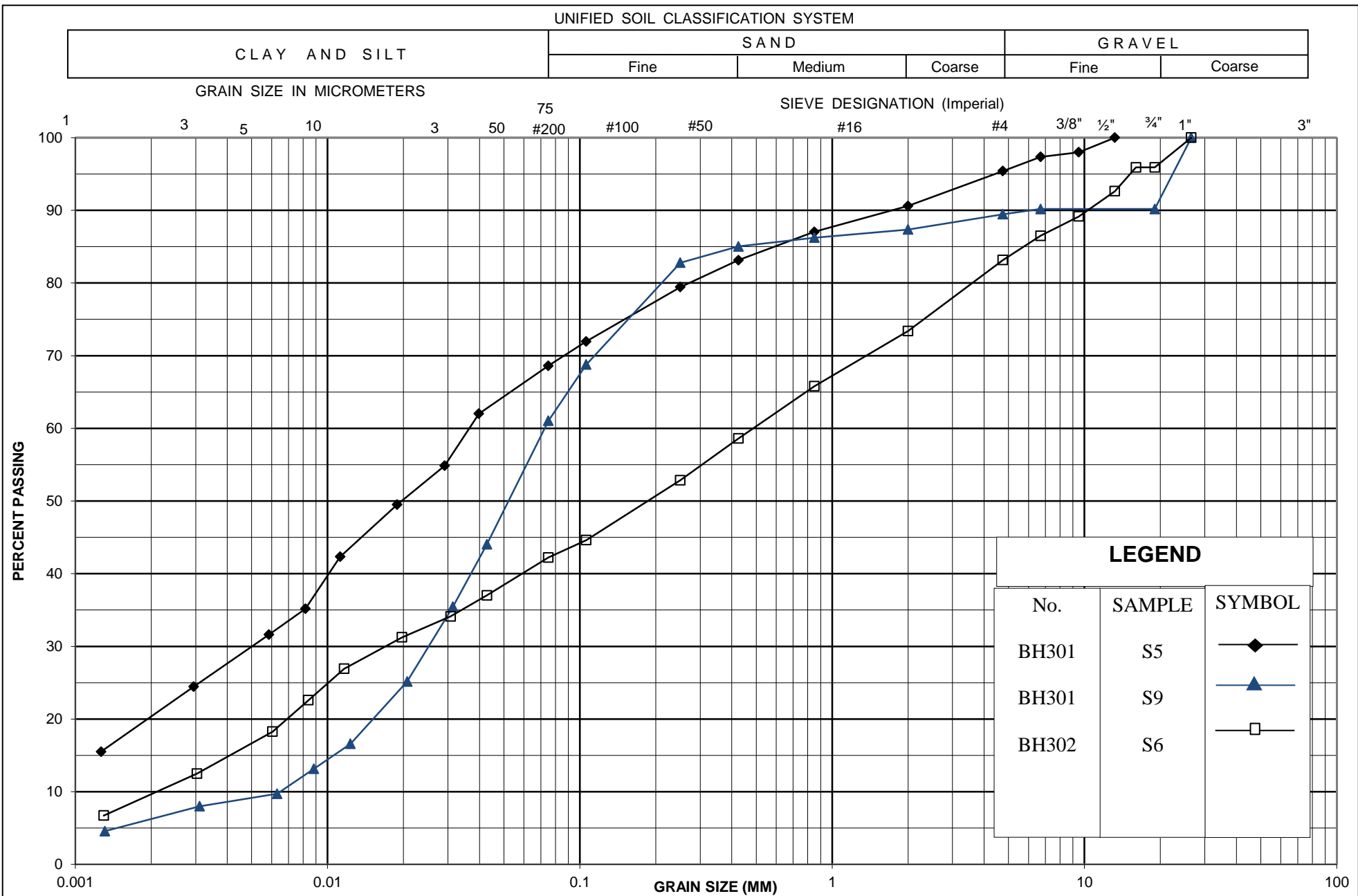


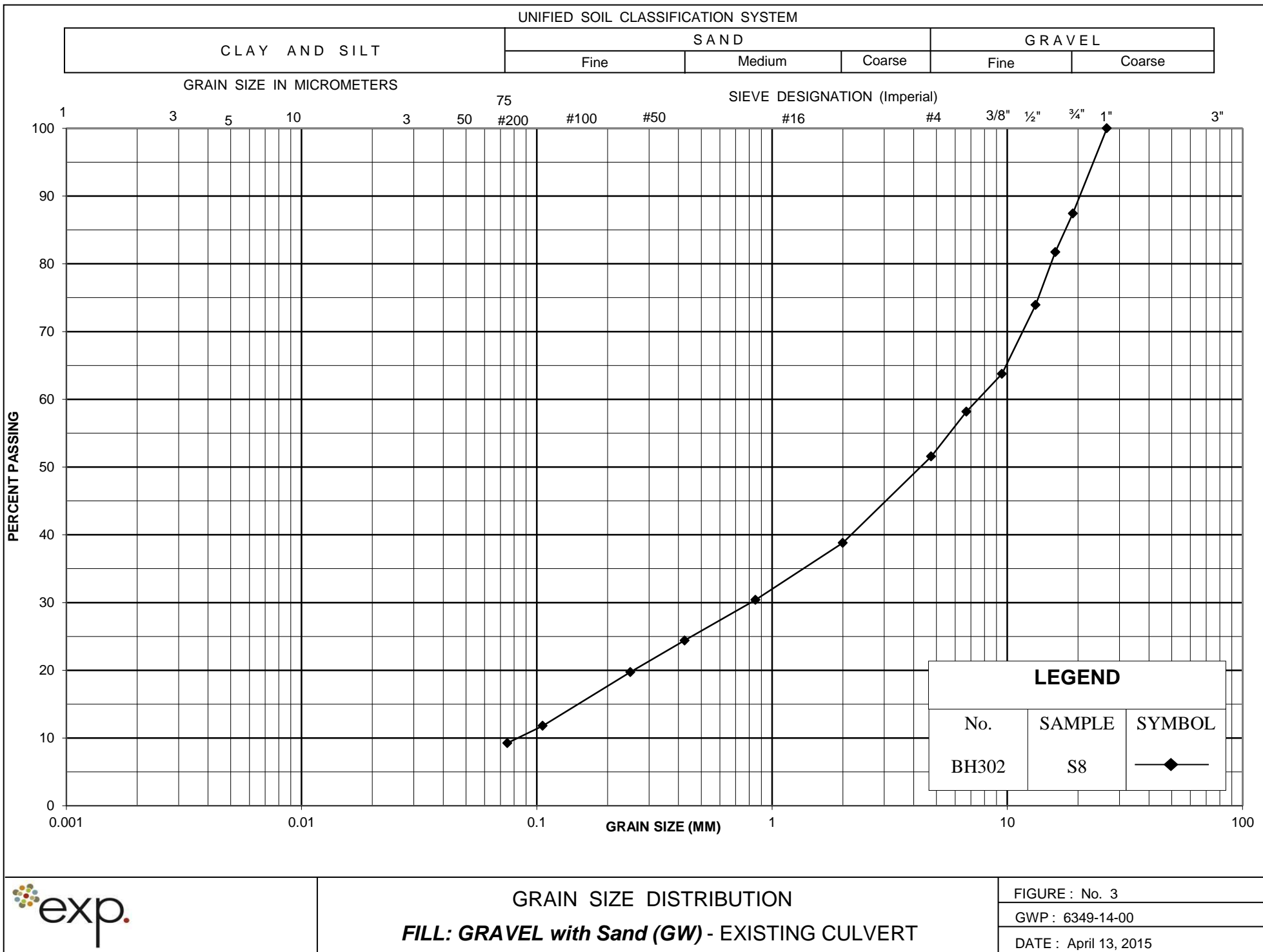


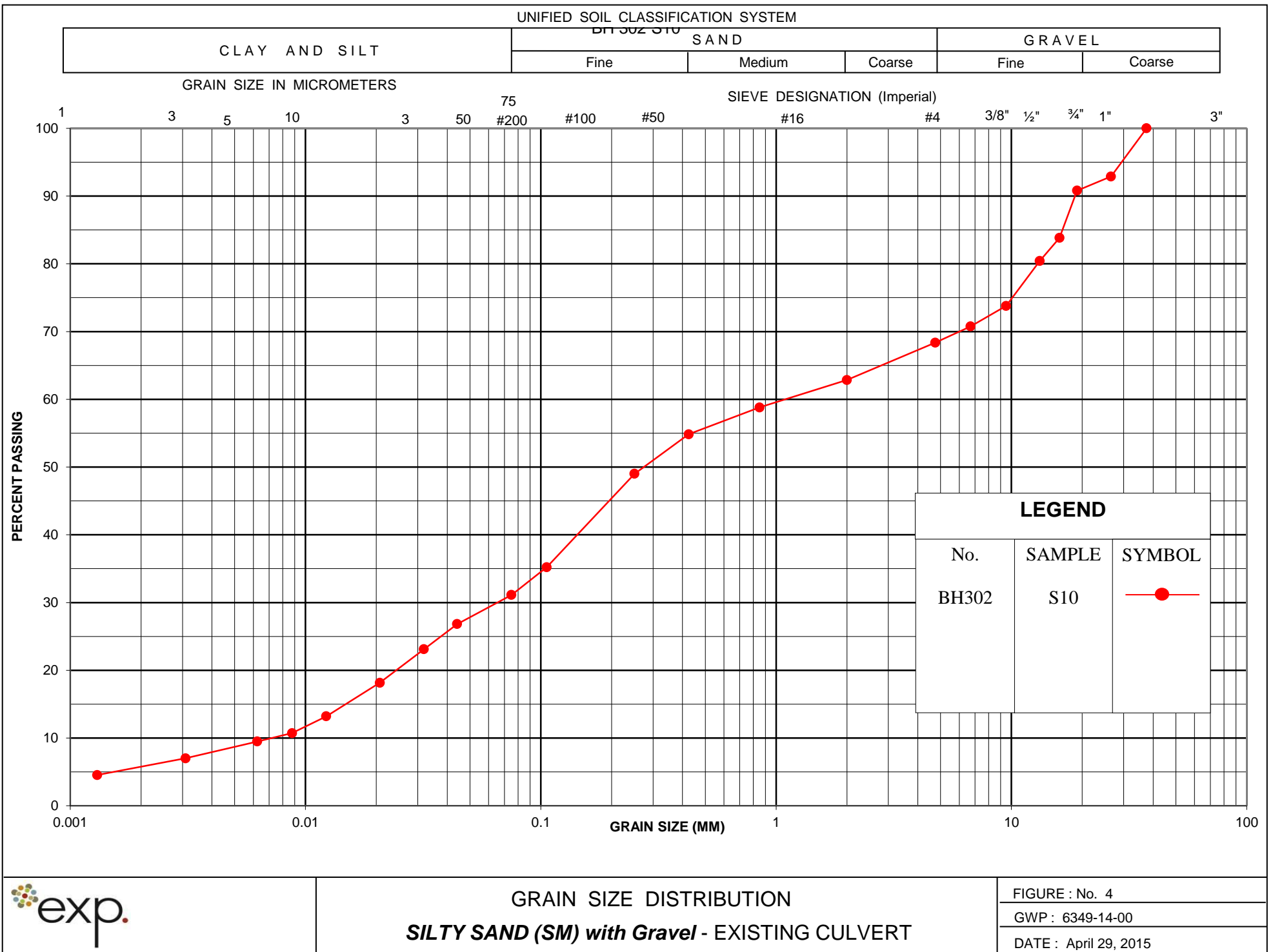
BH302 - Bedrock Core Samples with Depths and Elevations

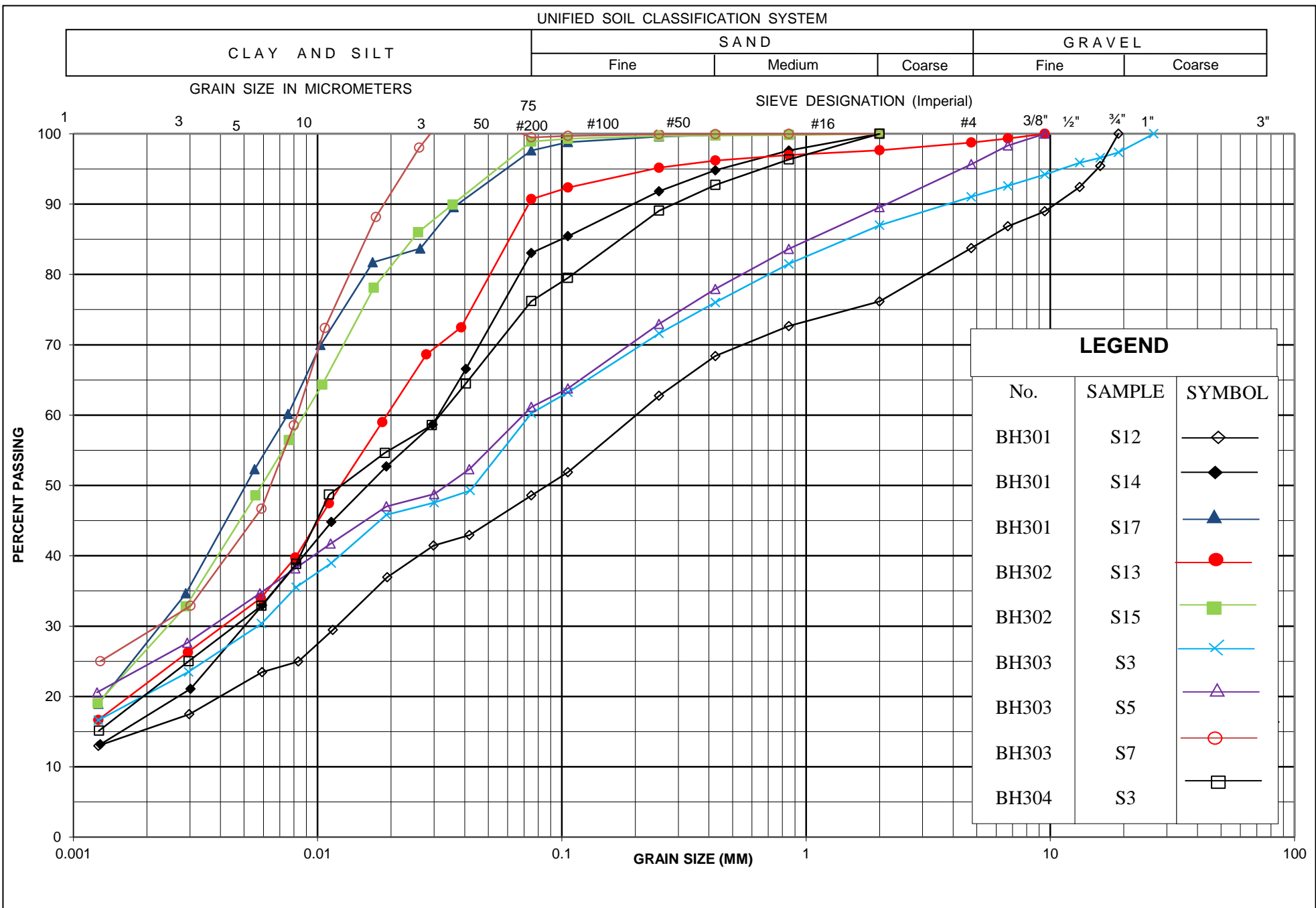
## **Appendix D – Laboratory Data**

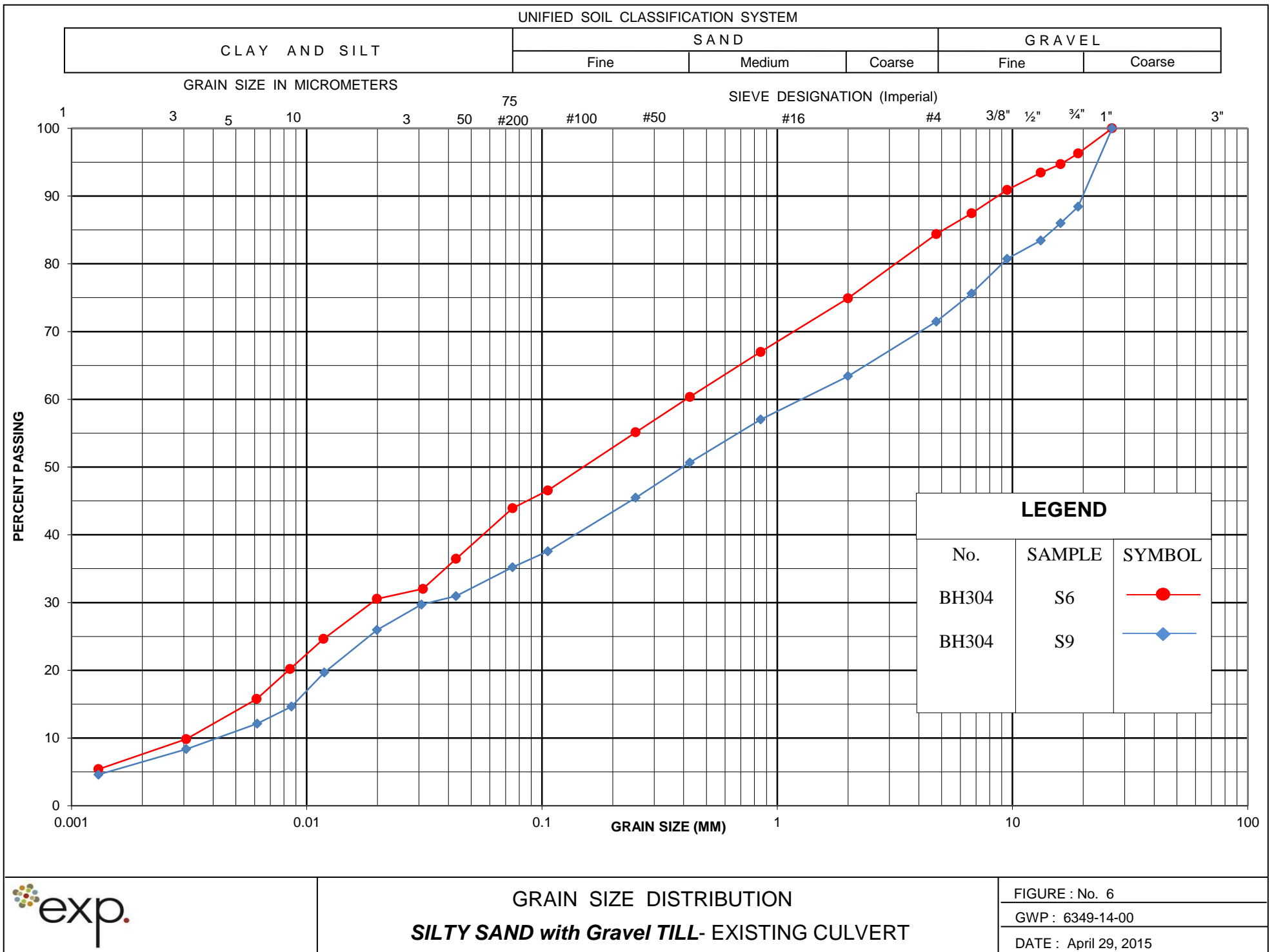












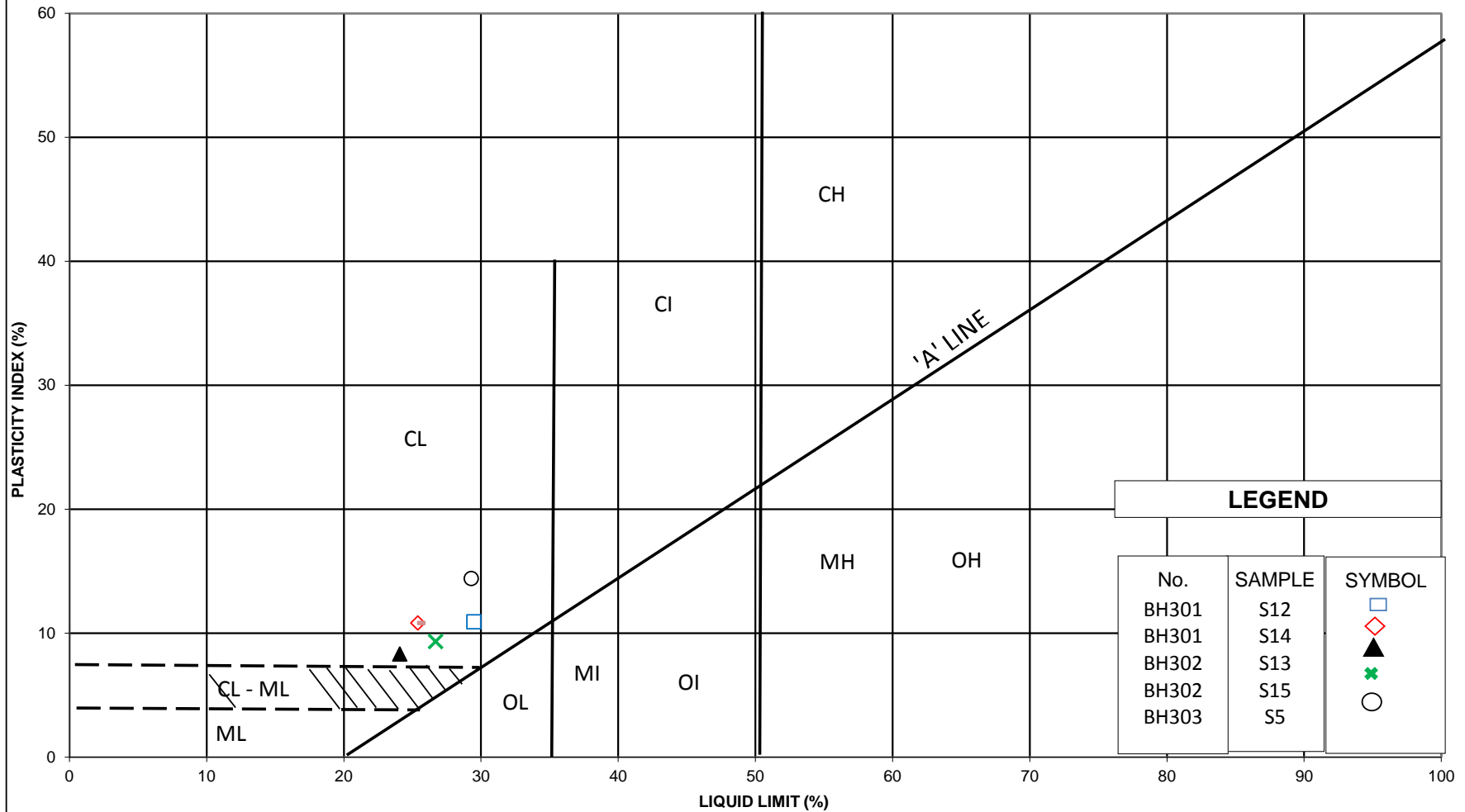
GRAIN SIZE DISTRIBUTION  
***SILTY SAND with Gravel TILL- EXISTING CULVERT***

FIGURE : No. 6  
GWP : 6349-14-00  
DATE : April 29, 2015





Whitewood Creek Culvert (Site No. 48W-168/C)  
GWP No. 6346-14-00, Highway 590, Marks Township, Ontario



## **Appendix E – Chemical Analyses**

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		
	<b>Units</b>	<b>B101-S12</b>	<b>BH105-S4</b>	<b>BH202-S10/S11</b>	<b>BH203-S3</b>	<b>BH302-S10</b>	<b>BH302-S10 Lab-Dup</b>	<b>RDL</b>	<b>QC Batch</b>

#### 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		
	<b>Units</b>	<b>BH303-S4</b>	<b>BH402-S14</b>	<b>BH403-S3</b>	<b>BH403-S3 Lab-Dup</b>	<b>RDL</b>	<b>QC Batch</b>

#### 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 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:** 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 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 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



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

exp Services Inc  
Client Project #: ADM-00223648-B0  
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.**

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

## QUALITY ASSURANCE REPORT

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

QC Batch	Parameter	Date	Matrix Spike		Spiked Blank		Method Blank		RPD	
			% Recovery	QC Limits	% Recovery	QC Limits	Value	Units	Value (%)	QC Limits
3965076	Available (CaCl <sub>2</sub> ) 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 (SO <sub>4</sub> )	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 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.



6740 Campobello Road, Mississauga, Ontario L5N 2L8 www.maxxam.ca  
Phone: 905-817-5700 Fax: 905-817-5779 Toll Free: 800-563-6266

# CHAIN OF CUSTODY RECORD

Page 1 of 1

INVOICE INFORMATION		REPORT INFORMATION (if differs from invoice)		PROJECT INFORMATION		TURNAROUND TIME (TAT) REQUIRED	
Company Name: exp Services Inc.		Company Name:		Quotation #:		<input checked="" type="checkbox"/> 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 Thunder Bay, ON P7B 5M4		Address:		Project #: ADM-00223648-B0		Rush TAT (Applicable Surcharge)	
Phone: 807.623.9495 Fax: 807.623.8070		Phone: Fax:		Site Location: Hwy 590, Kakabeka, Ontario		<input type="checkbox"/> 1 Day (100%)	
Email: michael.suslyk@exp.com, ahileas.mitsopoulos@exp.com		Email:		Site #:		<input type="checkbox"/> 2 Days (50%)	
				Sampled By: Elwin Farkas		<input type="checkbox"/> 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				ANALYSIS REQUESTED		Rush Confirmation #:	
REGULATION 153 (2011)		OTHER REGULATIONS		FIELD FILTERED (PLEASE CIRCLE) Metals / Hg / Cu / V pH Water Soluble Sulphate Resistivity Conductivity Chloride		Date Required:	
<input type="checkbox"/> Table 1 <input type="checkbox"/> Res/Park <input type="checkbox"/> Med/Fine <input type="checkbox"/> Table 2 <input type="checkbox"/> Ind/Comm <input type="checkbox"/> Coarse <input type="checkbox"/> Table 3 <input type="checkbox"/> Agri/Other <input type="checkbox"/> Table _____		<input type="checkbox"/> CCME <input type="checkbox"/> Sanitary Sewer Bylaw <input type="checkbox"/> MISA <input type="checkbox"/> Storm Sewer Bylaw <input type="checkbox"/> PWQO Municipality: _____ <input type="checkbox"/> Other (Specify): _____ <input type="checkbox"/> REG 558 (MINIMUM 3 DAY TAT REQUIRED)				LABORATORY USE ONLY	
FOR RSC (PLEASE CIRCLE) Yes / <input checked="" type="checkbox"/> No						CUSTODY SEAL (Y/N) Present <input checked="" type="checkbox"/> Intact <input checked="" type="checkbox"/>	
Include Criteria on Certificate of Analysis (Y/N)? <input checked="" type="checkbox"/> Y						Temperature (°C) on Receipt 4/5/5	
SAMPLES MUST BE KEPT COOL ( < 10 °C ) FROM TIME OF SAMPLING UNTIL DELIVERY TO MAXXAM						COOLING MEDIA PRESENT ( Y / N ) Y	
SAMPLE IDENTIFICATION		DATE SAMPLED	TIME SAMPLED	MATRIX	# OF CONT.	COMMENTS / TAT COMMENTS	
1	B101-S12	Feb. 26/15	3:00	Soil	1		
2	BH105-S4	Mar. 20/15	4:30	Soil	1		
3	BH202-S10/S11	Mar. 04/15	11:15	Soil	1		
4	BH203-S3	Mar. 19/15	10:20	Soil	1		
5	BH302-S10	Mar. 11/15	2:40	Soil	1		
6	BH303-S4	Mar. 18/15	11:25	Soil	1		
7	BH402-S14	Mar. 07/15	5:10	Soil	1		
8	BH403-S3	Mar. 17/15	10:00	Soil	1		
9							
10							
RELINQUISHED BY: (Signature/Print)		DATE: (YYYY/MM/DD)	TIME:	RECEIVED BY: (Signature/Print)		DATE: (YYYY/MM/DD)	TIME:
 Michael Suslyk		26-Mar-15	1:30	 Hina Siddiqui		2015/03/27	10:00

Maxxam Analytics International Corporation o/a Maxxam Analytics

27-Mar-15 10:00  
Hina Siddiqui  
B553991

HP6 ENV-789

## **Appendix F – Slope Stability Analysis**

Whitewood Creek Culvert  
 West side of Embankment (Inlet)  
 Drained Condition

Name: Engineered Fill Model: Mohr-Coulomb Unit Weight: 21 kN/m<sup>3</sup> Cohesion: 0 kPa  $\Phi$ : 32°  
 Name: Silt and Peat (Soft) Model: Mohr-Coulomb Unit Weight: 17 kN/m<sup>3</sup> Cohesion: 0 kPa  $\Phi$ : 20°  
 Name: Silty Sand with Gravel (Compact) Model: Mohr-Coulomb Unit Weight: 21 kN/m<sup>3</sup> Cohesion: 0 kPa  $\Phi$ : 35°  
 Name: Clayey Silt with Sand (Very Stiff to Hard) Model: Mohr-Coulomb Unit Weight: 19 kN/m<sup>3</sup> Cohesion: 0 kPa  $\Phi$ : 30°  
 Name: Silty Sand with Gravel Till (Compact to Very Dense) Model: Mohr-Coulomb Unit Weight: 21 kN/m<sup>3</sup> Cohesion: 0 kPa  $\Phi$ : 35°  
 Name: Cobbles and Gravel (Very Dense) Model: Mohr-Coulomb Unit Weight: 20 kN/m<sup>3</sup> Cohesion: 0 kPa  $\Phi$ : 38°  
 Name: Bedrock Model: Bedrock (Impenetrable)

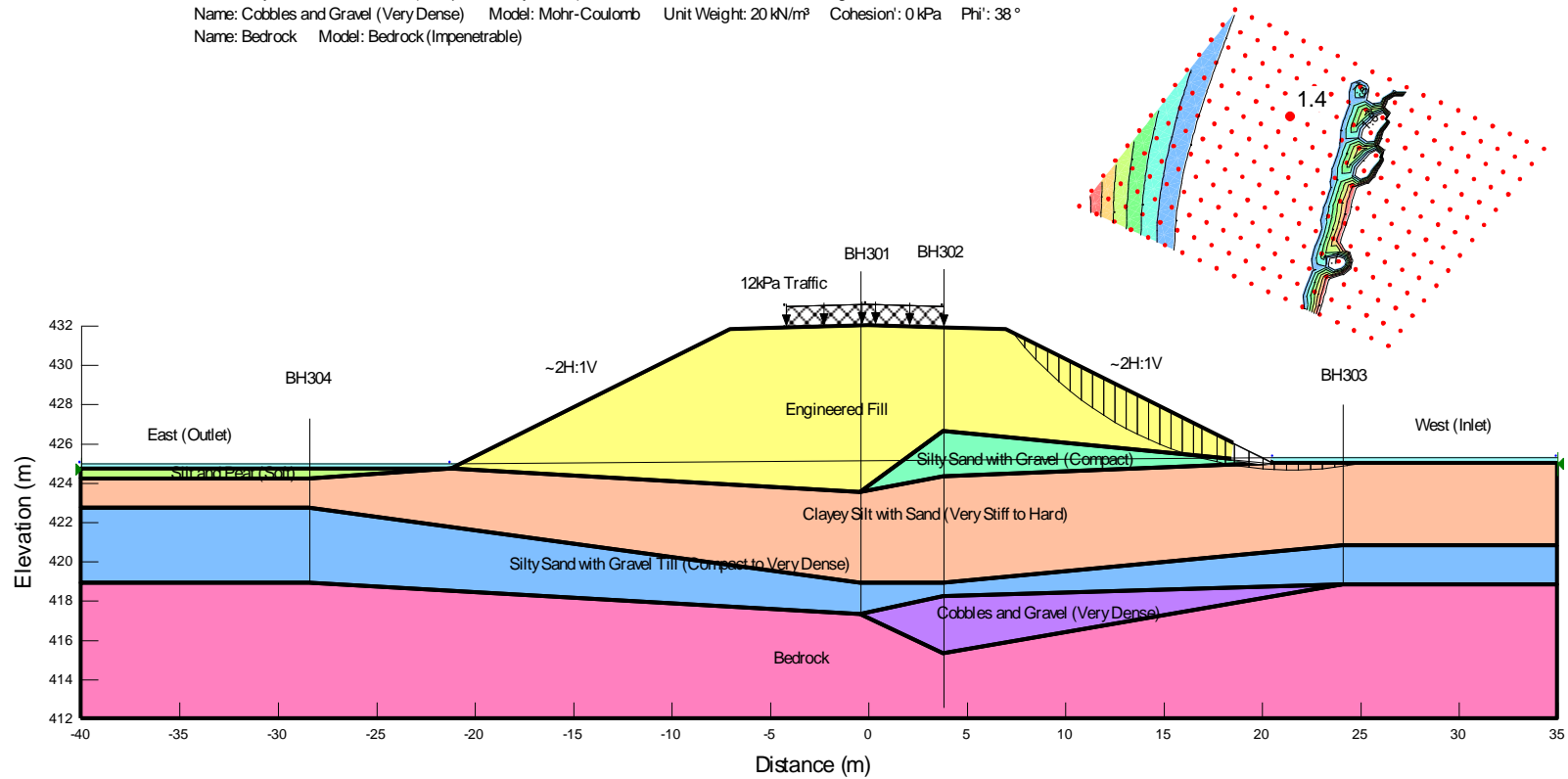
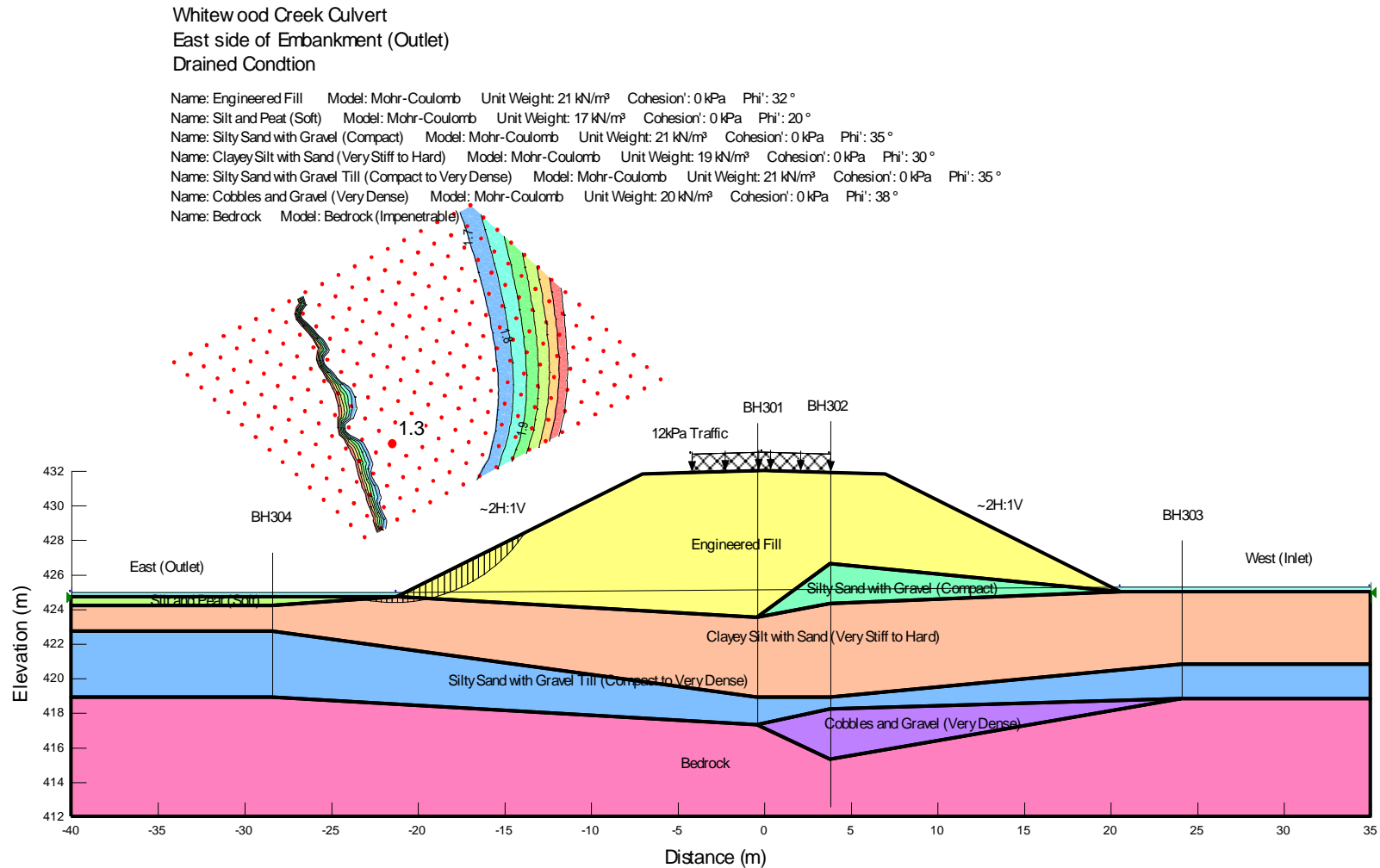
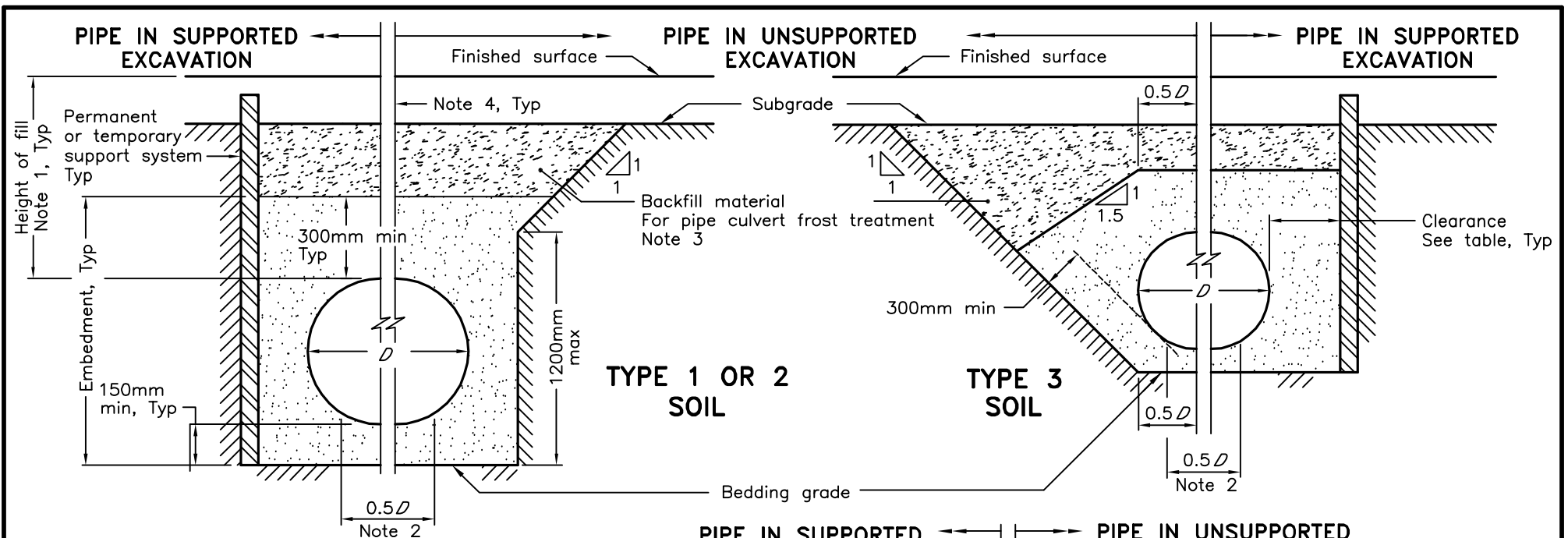


Figure F1: West side of embankment (inlet) – drained static condition



## **Appendix G – OPSDs**



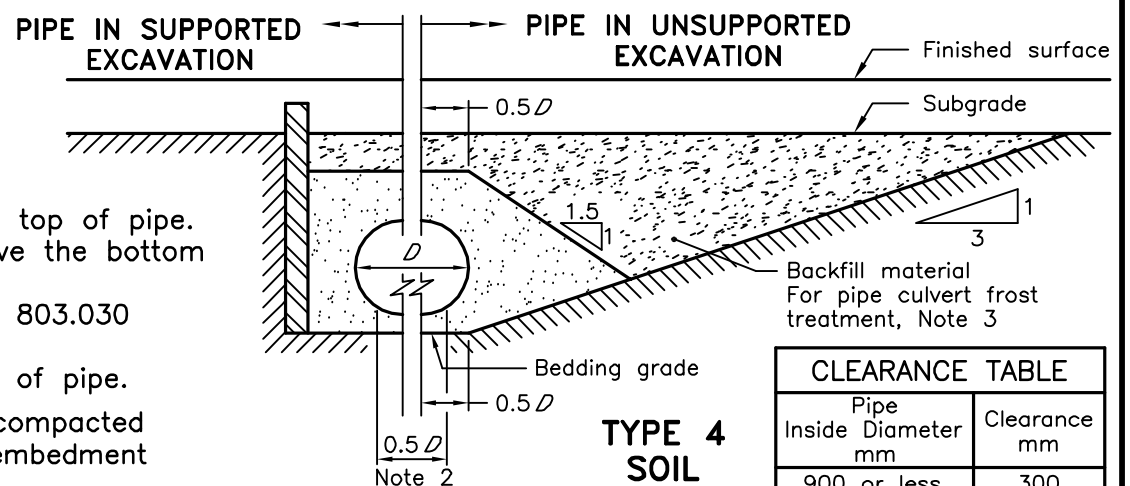


# LEGEND:

$D$  - 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

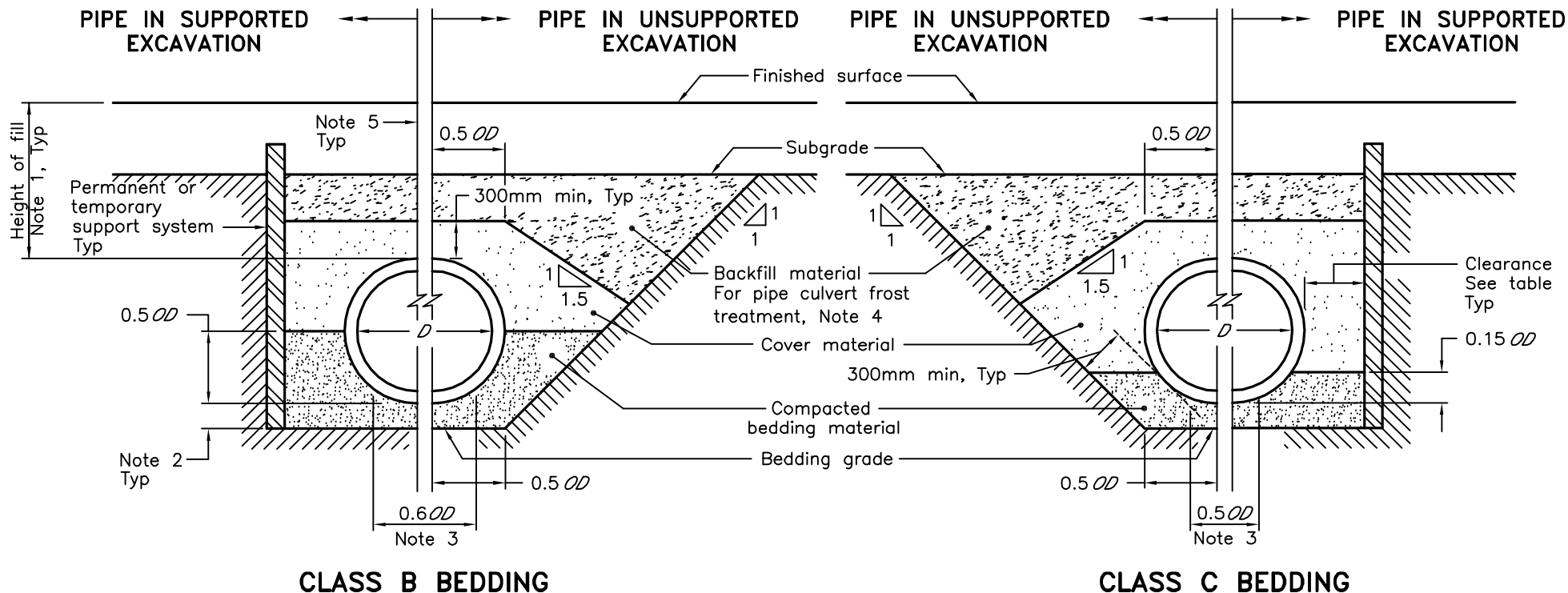
Nov 2010

Rev 2

FLEXIBLE PIPE  
EMBEDMENT AND BACKFILL  
EARTH EXCAVATION

OPSD 802.010





# 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  $0.15D$ . 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

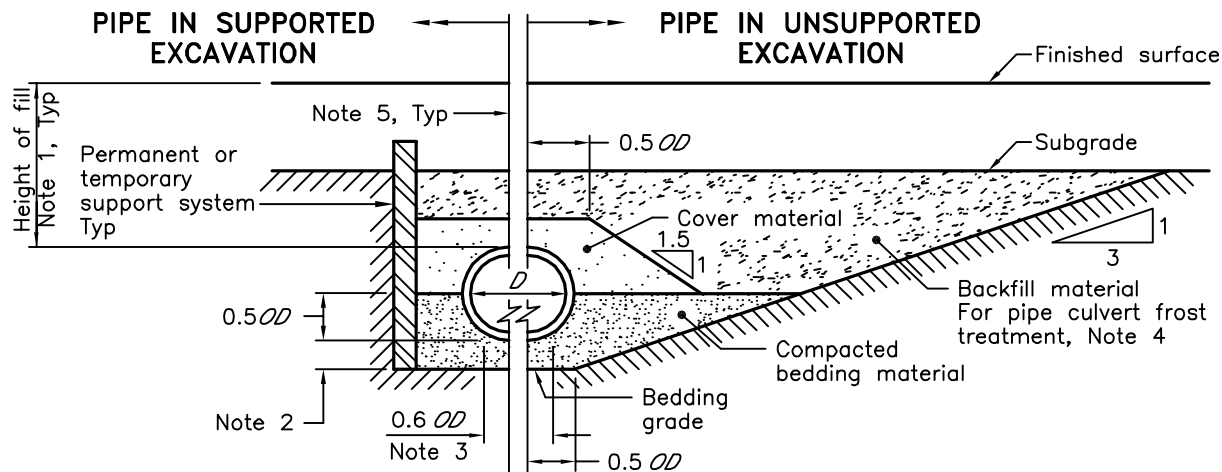
Nov 2010

Rev 2

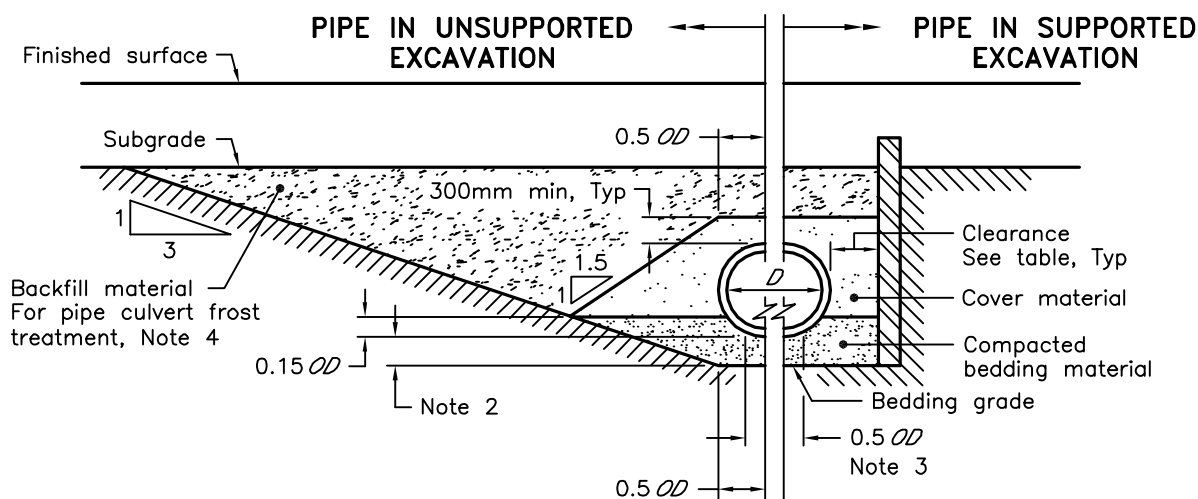
**RIGID PIPE BEDDING,  
COVER, AND BACKFILL  
TYPE 3 SOIL – EARTH EXCAVATION**

**OPSD 802.031**





### CLASS B BEDDING



### CLASS C BEDDING

#### 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  $0.15D$ .  
 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.

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

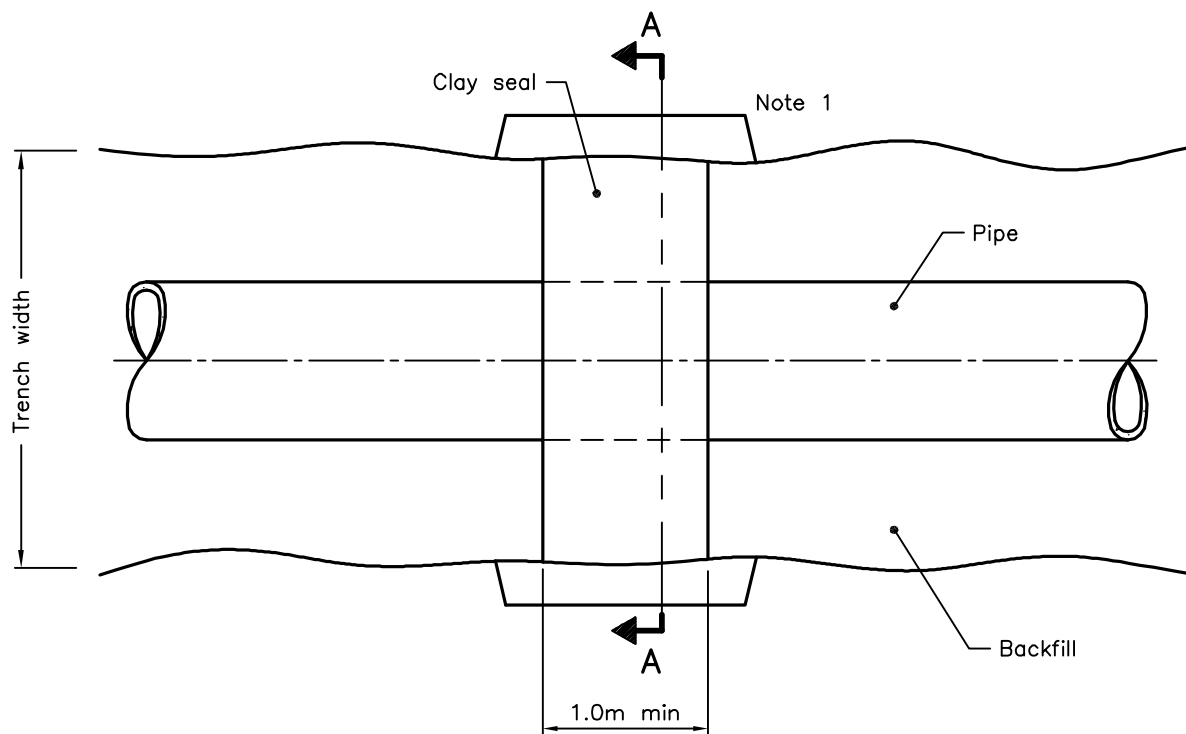
ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2010 Rev 2

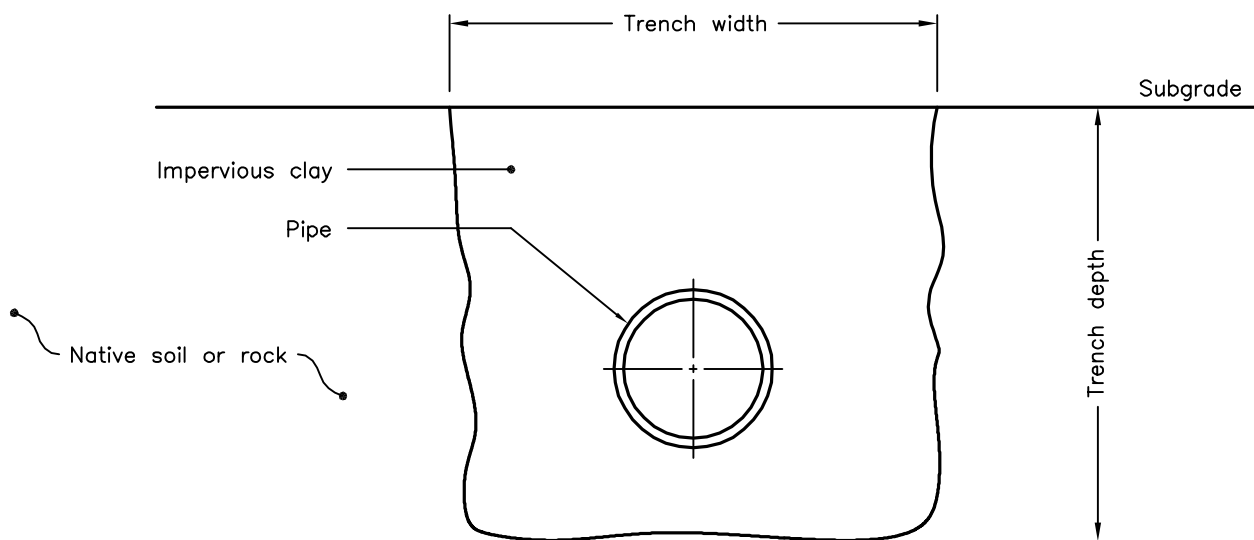
**RIGID PIPE BEDDING,  
 COVER, AND BACKFILL  
 TYPE 4 SOIL – EARTH EXCAVATION**

**OPSD 802.032**





PLAN



SECTION A-A

NOTES:

1. Key into undisturbed trench soil.

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

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

C All dimensions are in metres unless otherwise shown.

ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2011

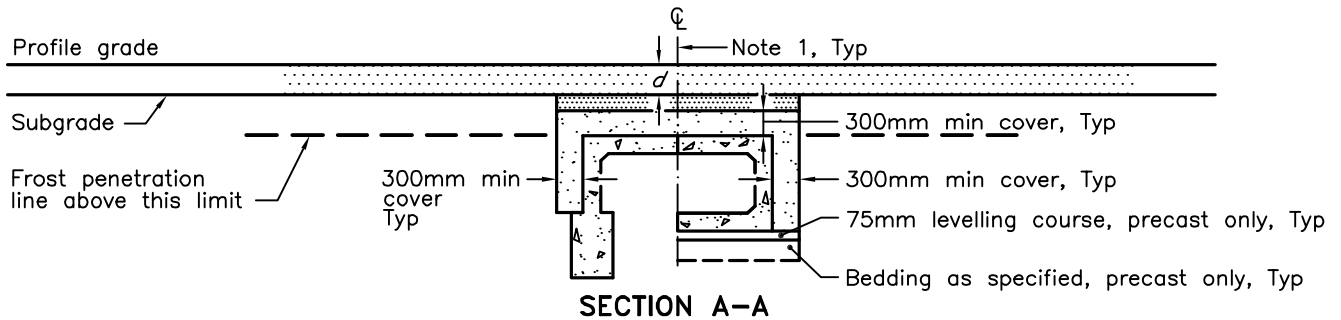
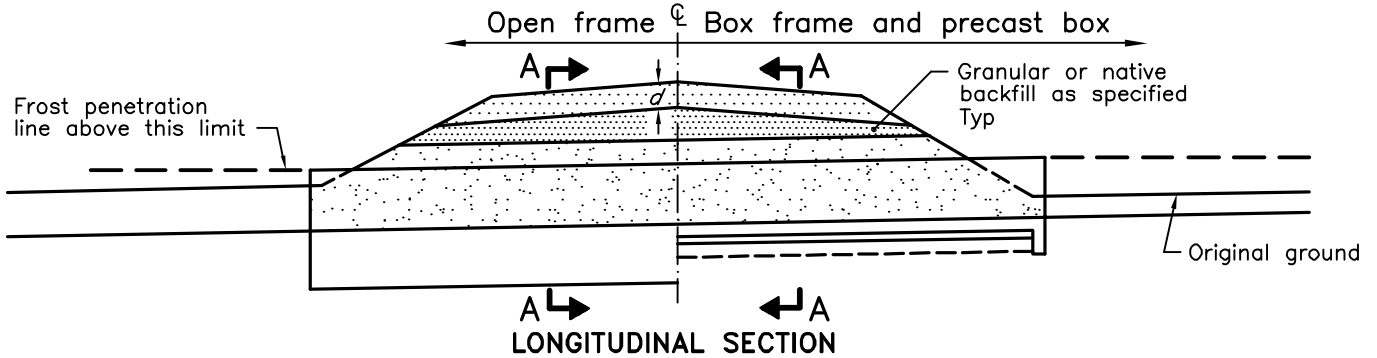
Rev 1

CLAY SEAL FOR PIPE TRENCHES

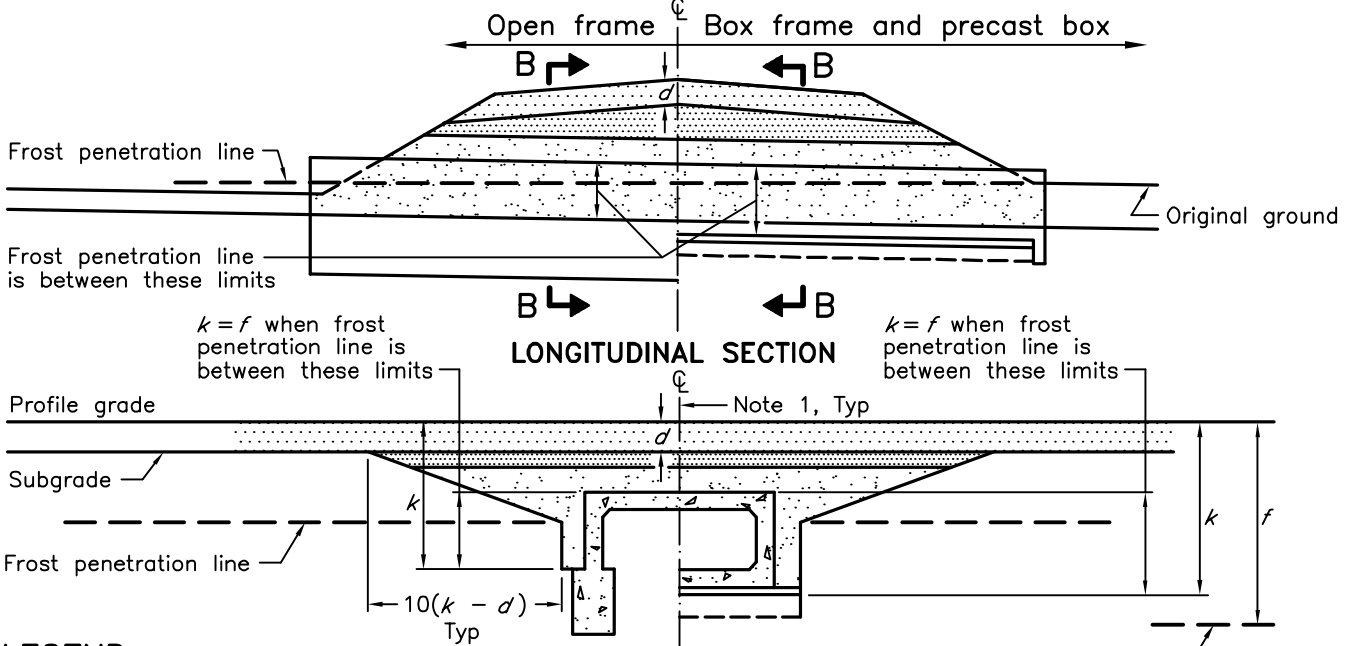
OPSD 802.095



## FROST PENETRATION LINE AT OR ABOVE TOP OF CULVERT



## FROST PENETRATION LINE BELOW TOP OF CULVERT



### LEGEND:

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

### NOTES:

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

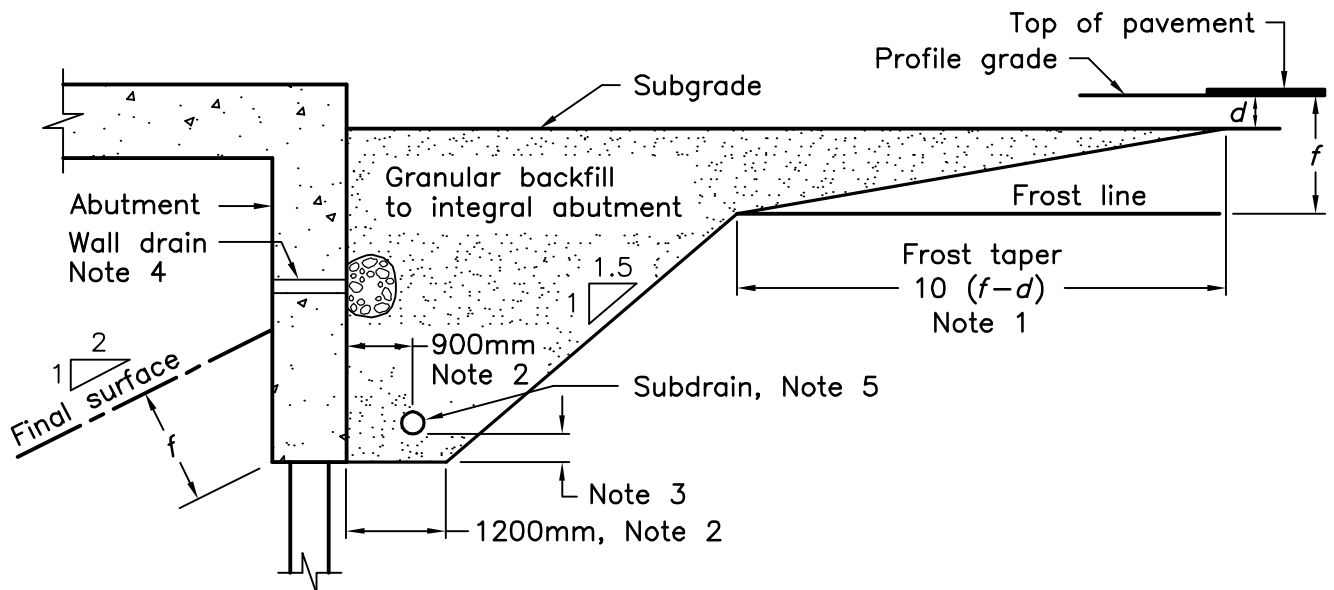
ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2010    Rev    2

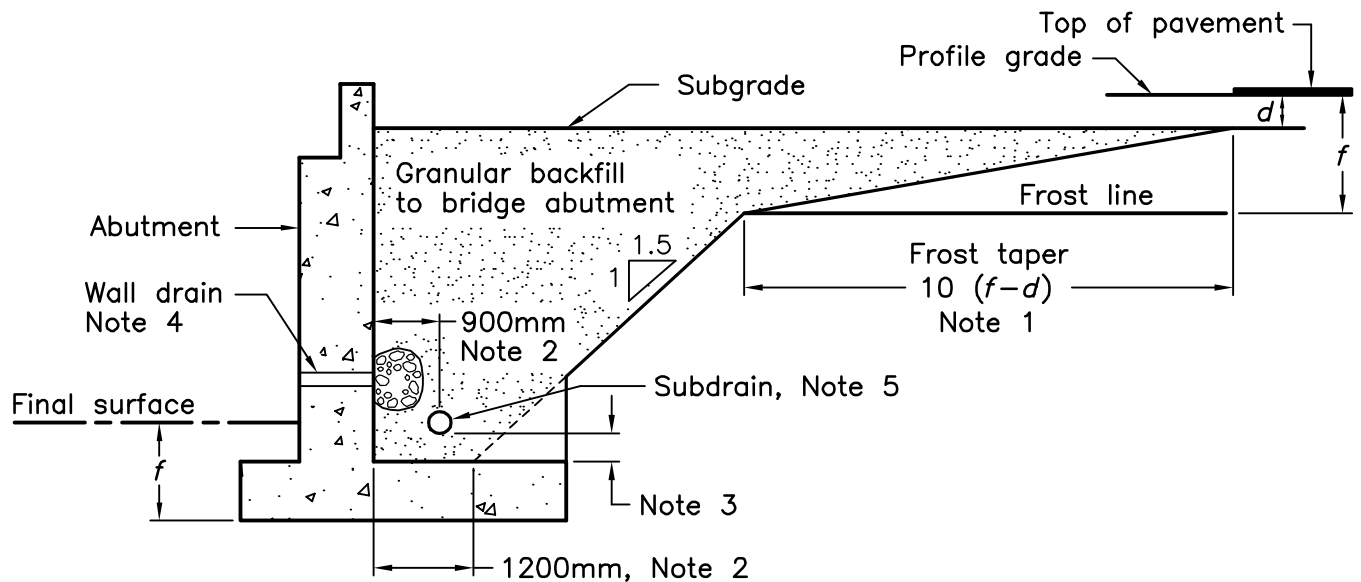
**BACKFILL AND COVER FOR  
CONCRETE CULVERTS WITH SPANS  
LESS THAN OR EQUAL TO 3.0M**

**OPSD 803.010**





### 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

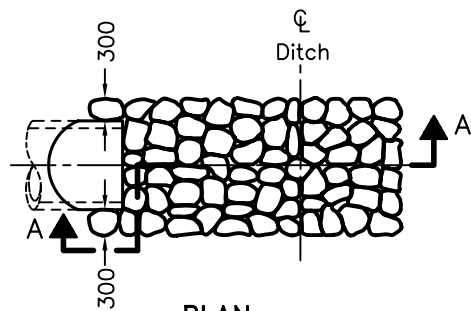
Nov 2010

Rev 1

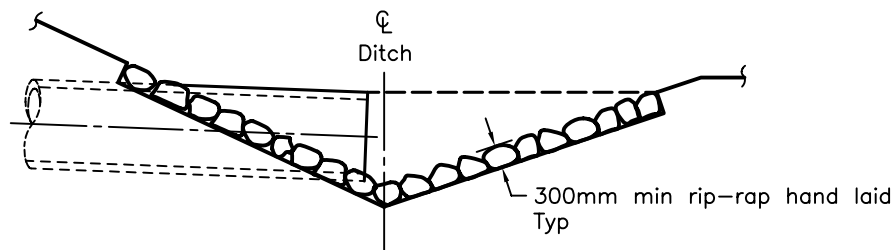


**WALLS**  
**ABUTMENT, BACKFILL**  
**MINIMUM GRANULAR REQUIREMENT**

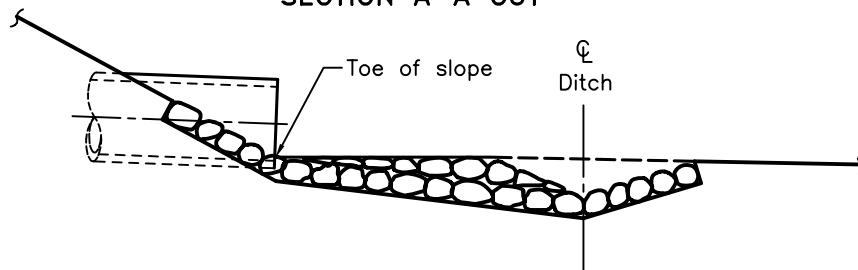
**OPSD 3101.150**



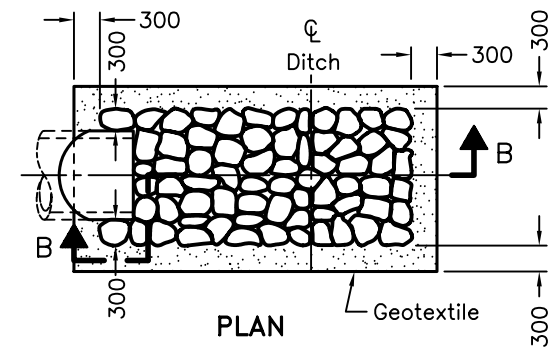
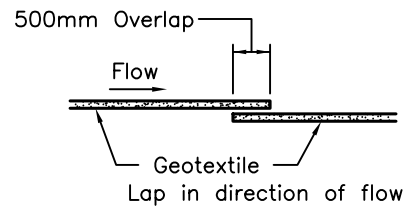
PLAN  
CUT OR FILL



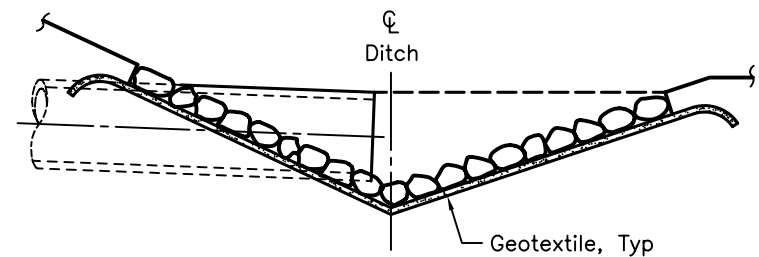
SECTION A-A CUT



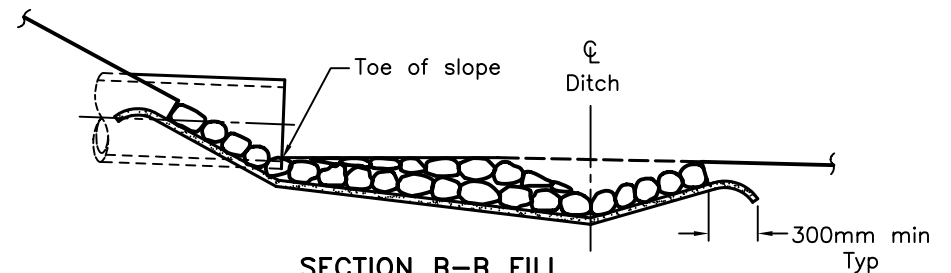
SECTION A-A FILL  
TYPE A – WITHOUT GEOTEXTILE



PLAN  
CUT OR FILL



SECTION B-B CUT



SECTION B-B FILL  
TYPE B – WITH GEOTEXTILE

NOTES:

A All dimensions are in millimetres unless otherwise shown.

ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2001

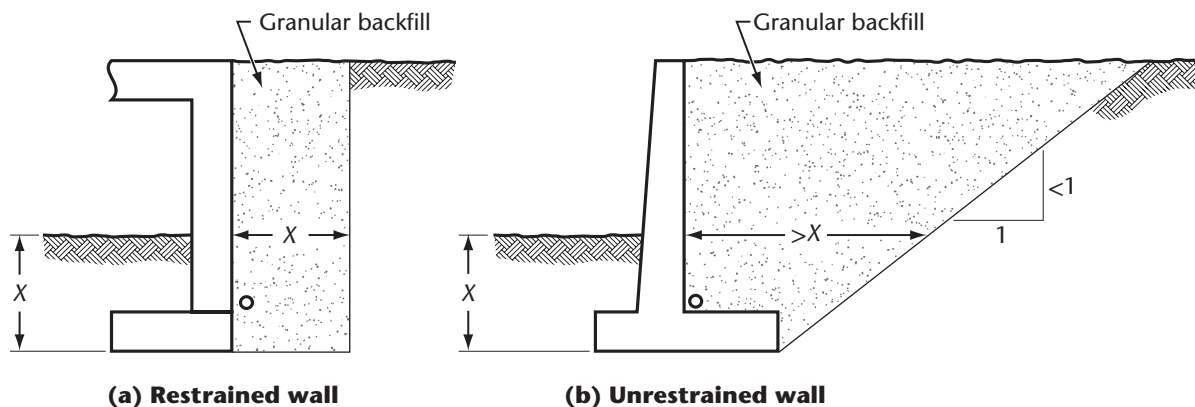
Rev 0

RIP-RAP TREATMENT  
FOR SEWER AND CULVERT OUTLETS



OPSD – 810.010

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.

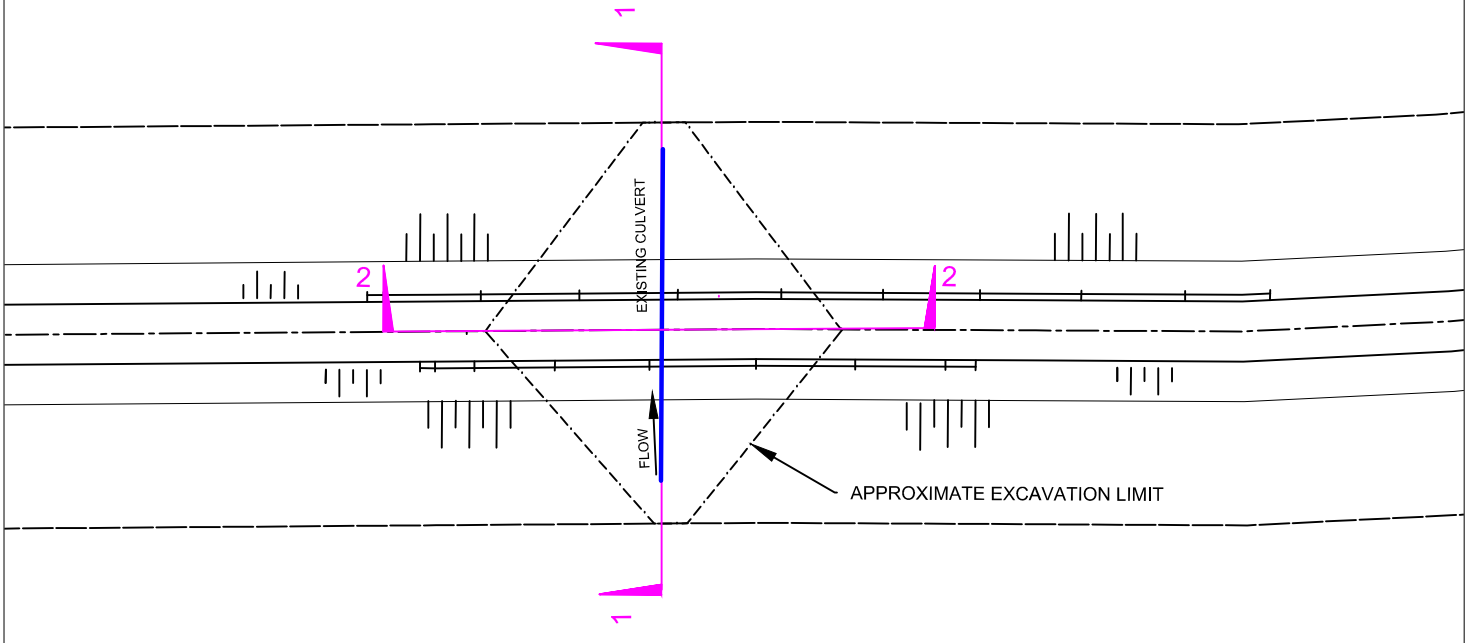
The supporting material will generally be more robust than what is assumed by the Geotechnical Engineer for factored conditions in design. Hence, following installation of the backfill, movement sufficient to cause active condition will generally not have taken place. Horizontal or rotational movement 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.



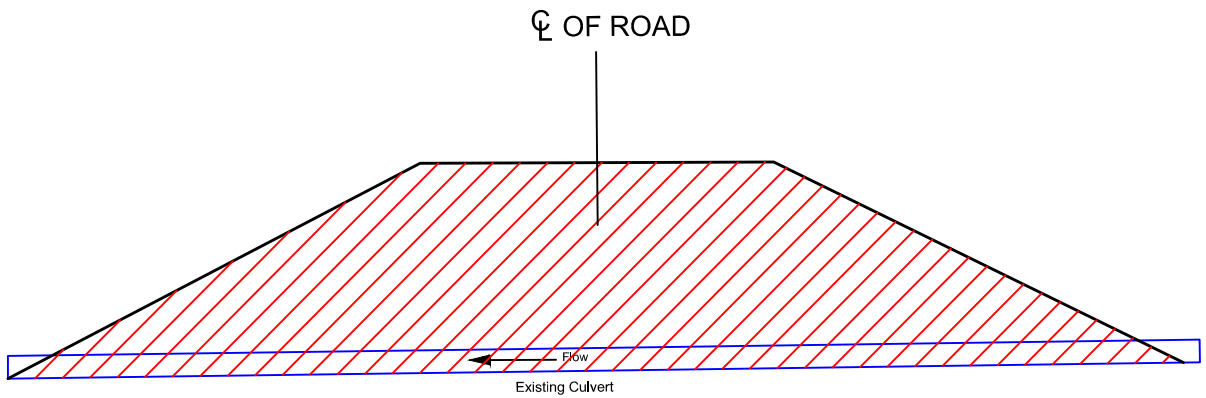
## **Appendix H – Schematic Sketches for Construction Alternatives**

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

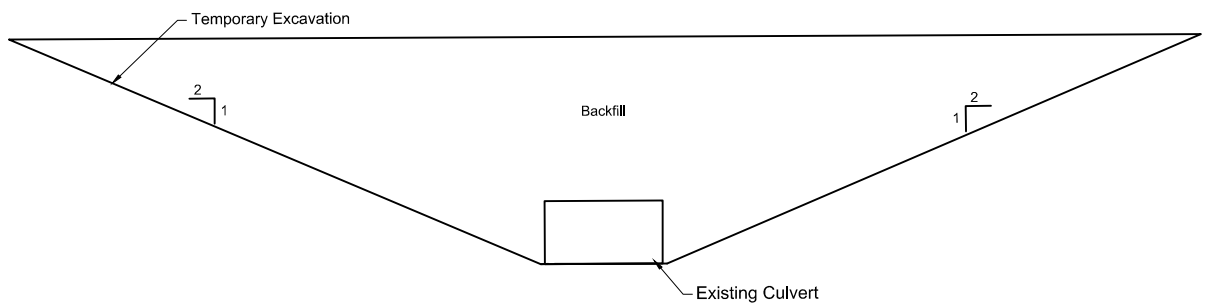
**SCHEMATIC DIAGRAMS (NTS)**



**PLAN**



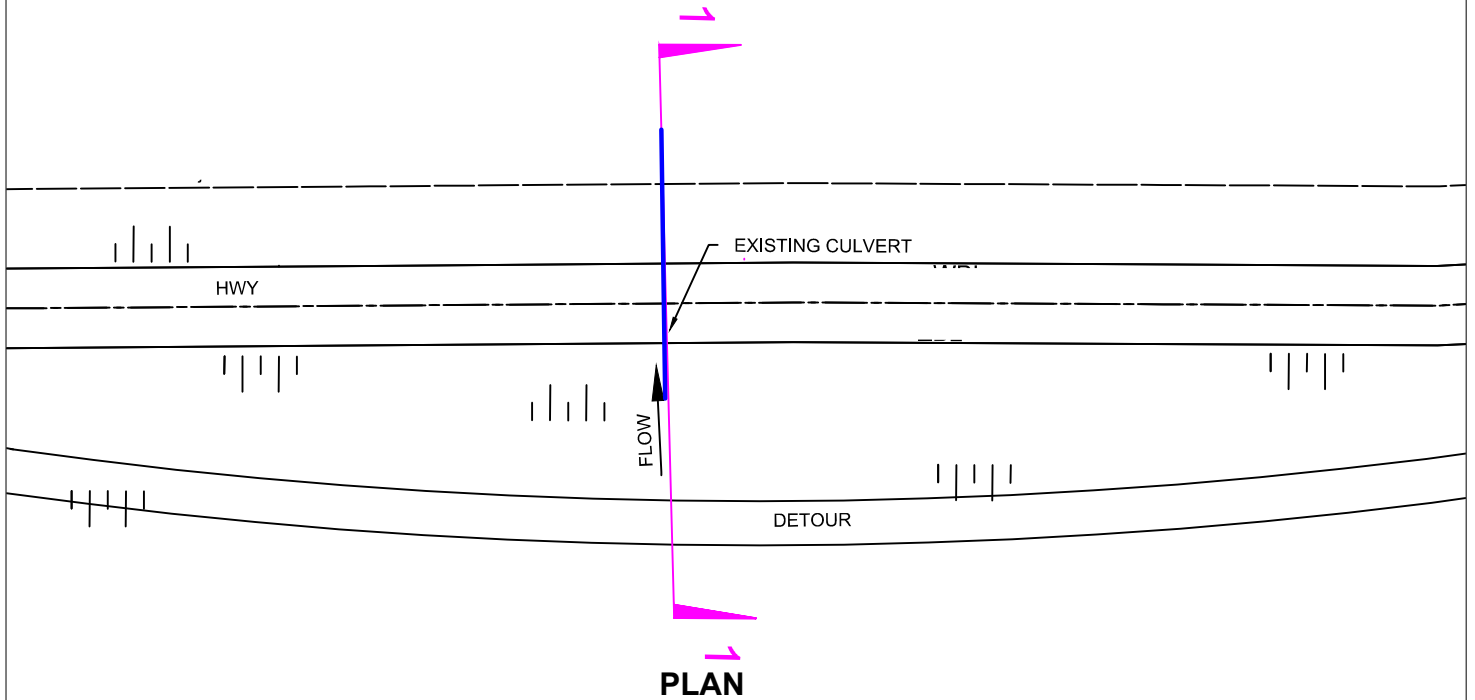
**SECTION 1-1**



**SECTION 2-2**

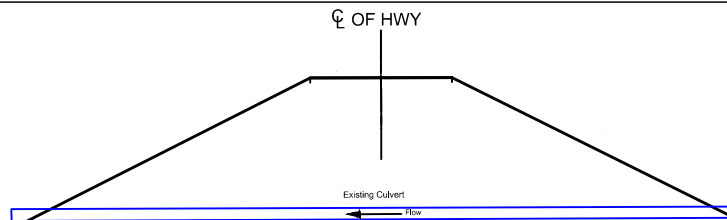
# FIGURE H.2: TEMPORARY LOCAL DETOUR AND OPEN CUT UNSUPPORTED EXCAVATION (OPTION 2)

## SCHEMATIC DIAGRAMS (NTS)

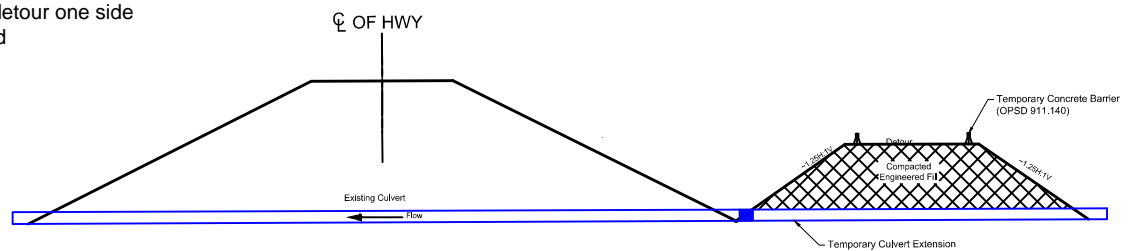


### RECOMMENDED STAGES

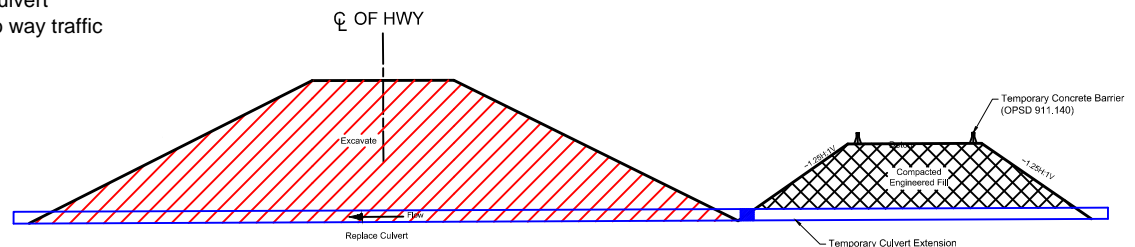
1.0 Stage 1 - Current condition



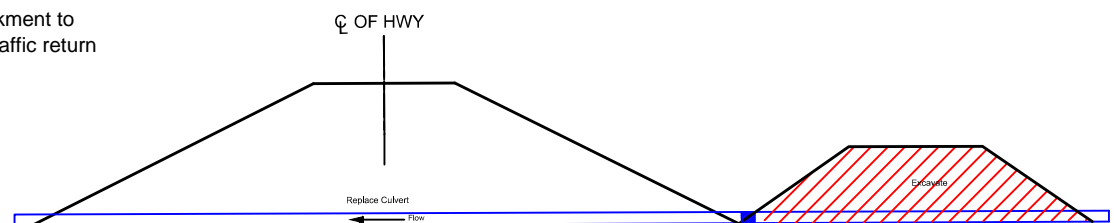
2.0 Stage 2 - Build temporary detour one side  
Two-way traffic on existing road



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



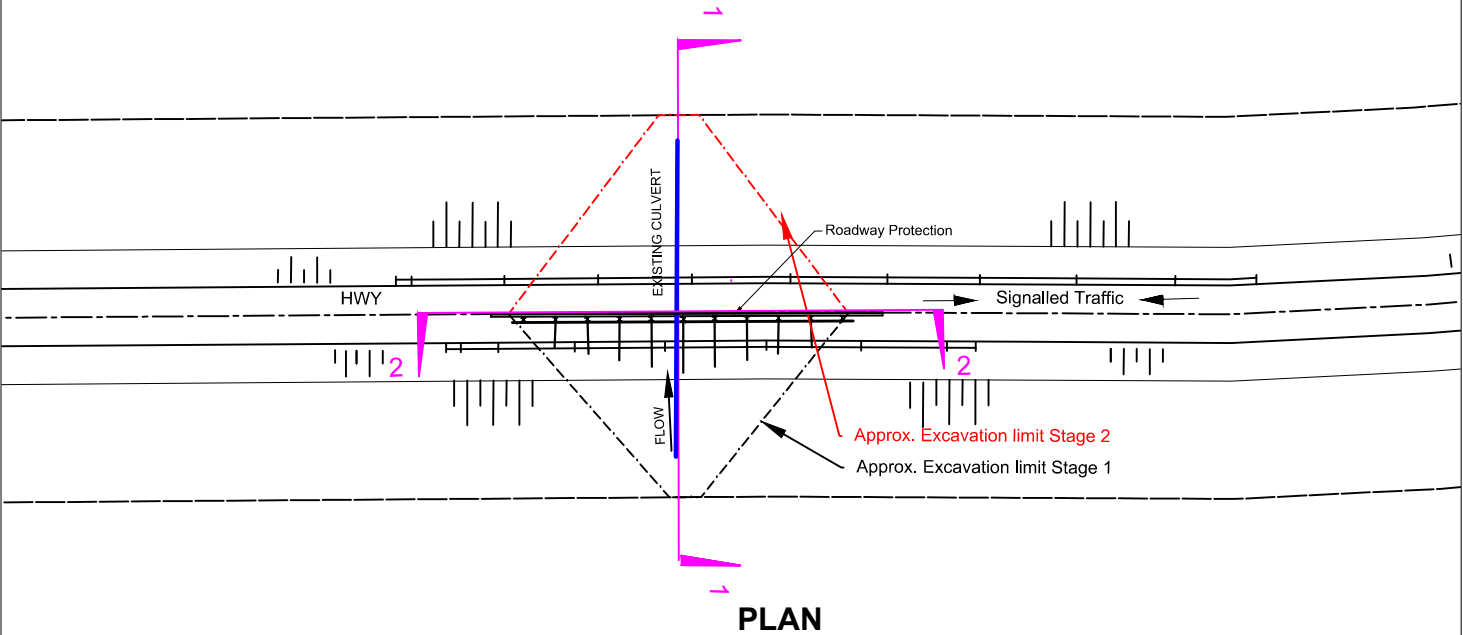
4.0 Stage 4 - Build the embankment to existing alignment; Two-way traffic return



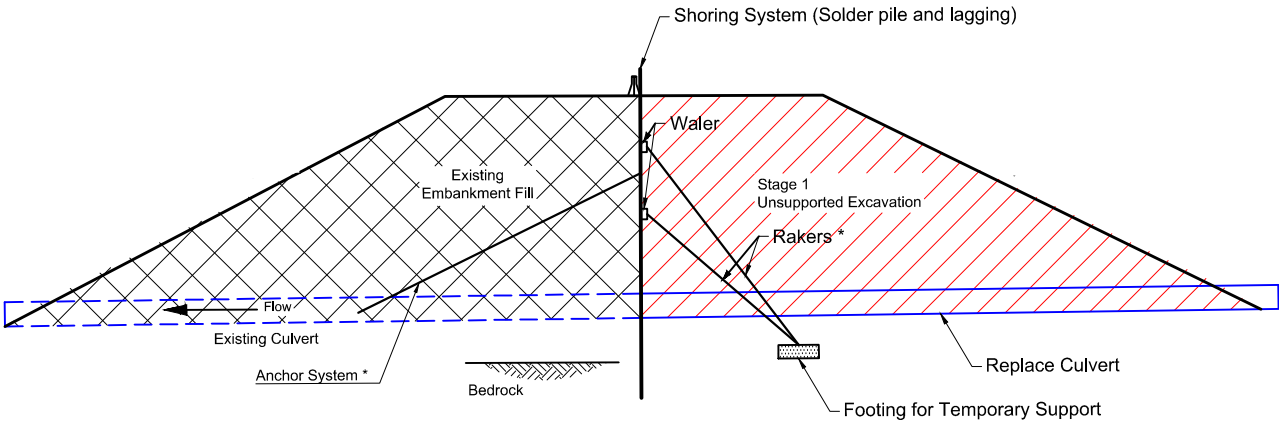
### SECTION 1-1

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

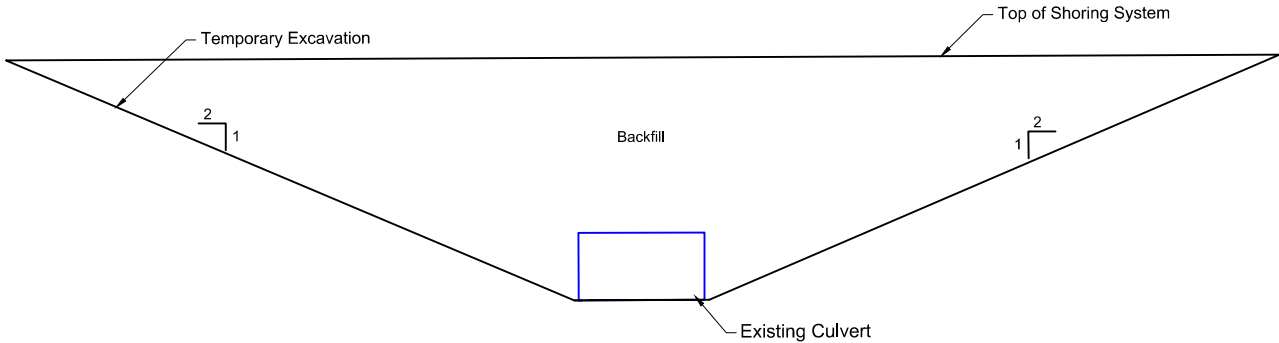
SCHEMATIC DIAGRAMS (NTS)



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

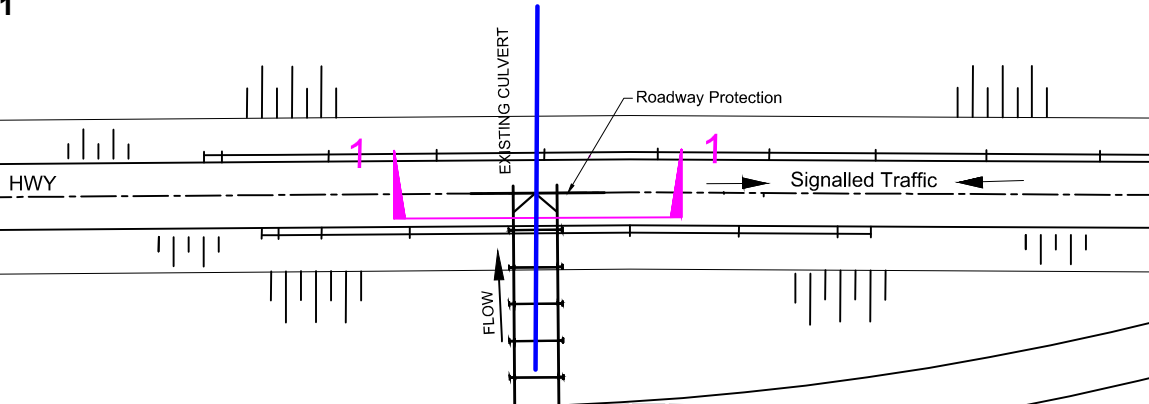


\* Rakers or Anchor System

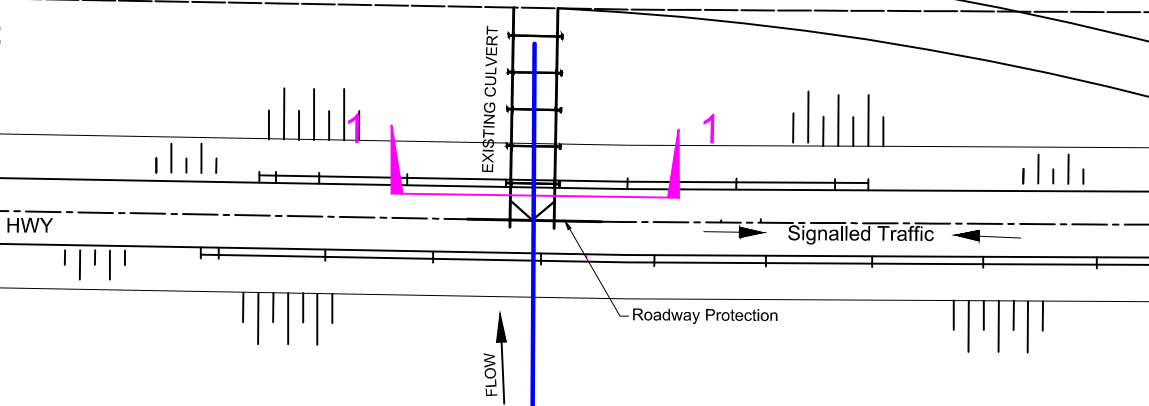


**FIGURE H.3.B: HALF AND HALF CONSTRUCTION WITH BRACED CUT SIDES  
OR ANCHOR SYSTEM OPTION 3.B  
SCHEMATIC DIAGRAMS (NTS)**

**Stage 1**

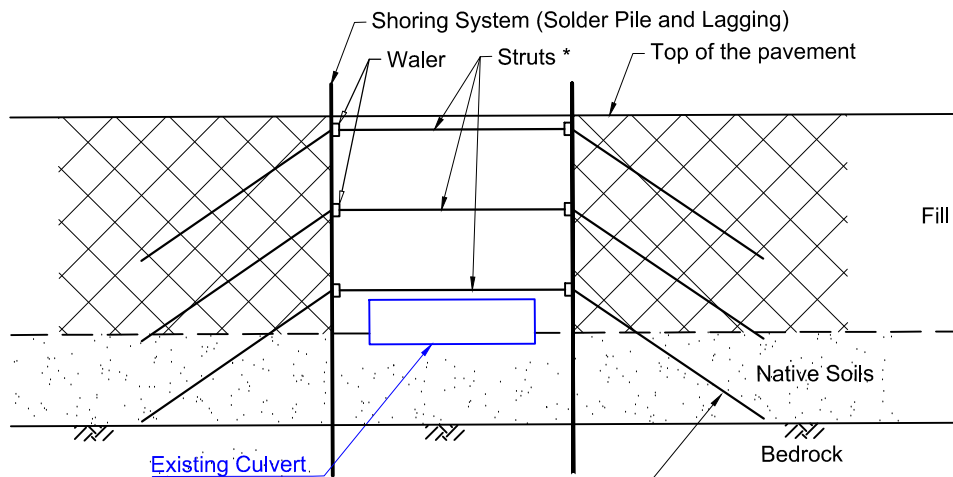


**Stage 2**



**PLAN**

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



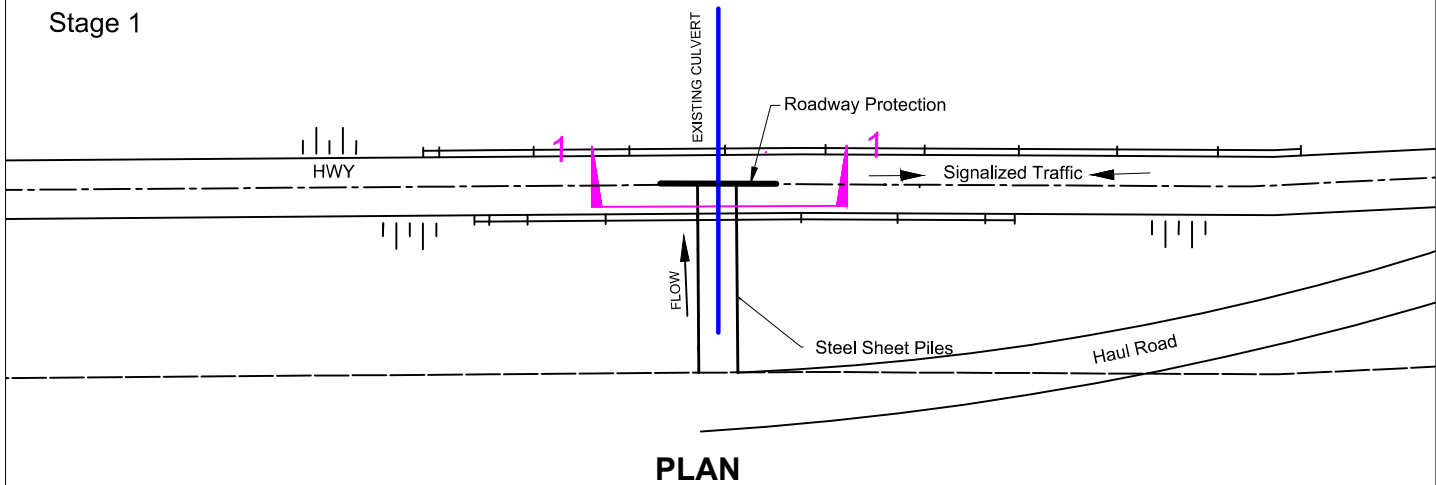
\* Struts or Anchor System

**SECTION 1-1**

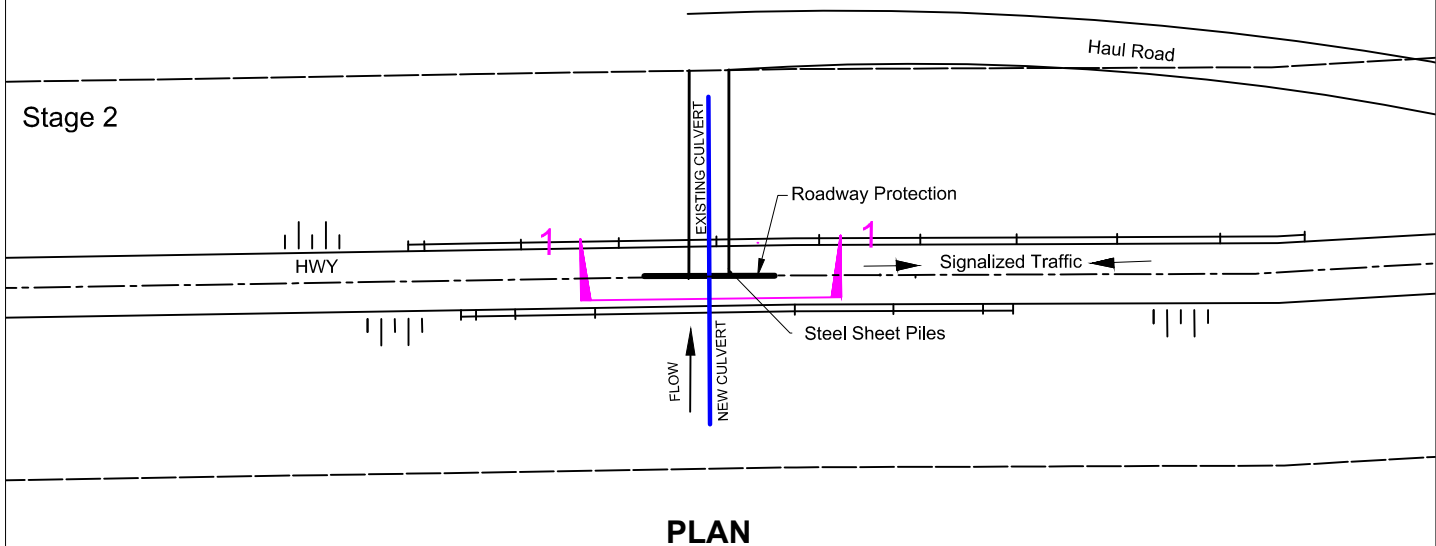
**FIGURE H.3.C: HALF AND HALF CONSTRUCTION WITH INSTALLATION OF STEEL SHEET PILE ABUTMENTS WITH PRECAST CONCRETE DECK (OPTION 3.C)**

**SCHEMATIC DIAGRAMS (NTS)**

Stage 1



Stage 2



**Half and Half Construction, Steel Sheet Pile Abutments with Precast Concrete Deck**

