



FINAL REPORT

FOUNDATION INVESTIGATION AND DESIGN REPORT

**Five Mile Creek Culvert Replacement, Highway 129,
Township of Reaney, Algoma District, Ontario**

Agreement No. 5015-E-0007

Assignment No. 6

WO. 2017-11005

Geocres No. 41O-24

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Ontario Ministry of Transportation

Northeastern Region Geotechnical Section

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1 FOUNDATION INVESTIGATION REPORT

1.1 Introduction

This report presents the results of a geotechnical investigation completed by **exp** Services Inc. for the replacement of a Five Mile Creek Culvert on Highway 129, in Township of Reaney, Algoma District. The work was undertaken under Agreement No. 5015-E-0007, Assignment No. 6. The terms of reference (TOR) were as presented in MTO letter dated February 3, 2017.

The purpose of the investigation was to supplement the previous preliminary foundation investigation at the same location performed by Peto MacCallum Ltd. (PML) in September 2016 (Geocres No. 410-14), with intent to improve the understanding of the bedrock characteristics and to permit detailed design for the replacement of the culvert, including construction recommendations. The site specific geotechnical investigation consisted of a field investigation including visual inspections, drilling, soil sampling, and laboratory testing.

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

1.2 Site Description and Geological Setting

1.2.1 Site Description

The Five Mile Creek Culvert is located on Highway 129 (Sta. 10+000), about 86.1 km north of the junction of Hwy 129 and Hwy 556, in the Township of Reaney, Algoma District, south of Chapleau, Ontario. At the site, Hwy 129 is a two lane roadway, with a speed limit of 80 km/h and is about 7.5 m wide from edge of pavement to edge of pavement, with sand and gravel shoulders. Based on the GA drawing provided by MTO on April 13 2017, the roadway embankment above the creek bed is about 5 m high with side slopes of about 2.5H:1V.

Based on the information provided in the TOR, Preliminary Foundation Investigation Report (PML, 2016) and the GA drawing, the existing culvert is a 3.9 m span and 24 m long corrugated steel elliptical pipe structure with 1.2 m of cover height. The invert of the culvert is at approximate Elev. 448.8 m. Select photographs of the site and existing culvert are presented in Appendix A. The site plan and cross-section profiles for the proposed culvert alignment are shown on the drawing attached in Appendix B.

During the fieldwork on March 13 to 16, 2017, the general site conditions were assessed. Note that at the time of the investigation, the site was surfaced with snow which limited our observations. Hwy 129 generally runs in a north to south direction and Five Mile Creek flows east to west beneath the highway. At the time of this investigation, the creek was frozen and the approximate top of water / ice elevations at the inlet and outlet were about 451.79 m and 451.66 m, respectively. The elevation of highway pavement centerline at the culvert centerline is about 453.76 m. Based on observations at the site no riprap on either side of the creek, i.e. inlet and outlet of the existing culvert, to protect against scour or erosion. Since the sides of the embankment were covered with snow, any erosion and

instability was not able to be observed. However, it was reported in the PML report that a review of the Google Earth Map indicated that the toe of the embankment on both sides of the culvert was eroded and undermined.

As previously indicated, the site was surfaced with snow at the time of the investigation. At the culvert inlet and outlet, snow/ice was noted nearly to the top of the culvert, and in the area of the inlet and outlet, some shrubs were observed.

1.2.2 Geological Setting

According to the Ministry of Northern Development and Mines, Map 2555 (Quaternary Geology of Ontario, East-Central Sheet, 1991) the surface conditions in the vicinity of the project area consists of Glaciofluvial ice-contact deposits, which includes gravel, sand and minor till includes esker, kame, end moraine, ice-marginal delta and subaqueous fan deposits and according to Map 2543 (Bedrock Geology of Ontario, East-Central Sheet, 1991), the bedrock geology of the site is of granite and other intrusive rocks such as tonalite to granodiorite foliated to gneissic with minor supracrustal inclusions.

1.3 Investigation Procedures

1.3.1 Site Investigation and Field Testing

The field investigation was performed between March 14, 2017 and March 16, 2017. The field program consisted of drilling four (4) sampled boreholes, numbered BH1 to BH4. Three (3) boreholes were strategically located along the existing culvert alignment and one (1) borehole was located on the embankment to provide subsurface information for the temporary roadway protection. BH1 and BH2 were advanced from the top of the embankment. BH1 was advanced within the shoulder of southbound lane located about 12.5 m north of culvert centreline (Sta. 10+012.5), while BH2 was advanced within the travelled southbound lane located about 4 m north of culvert centreline (Sta. 10+004). BH3 and BH4 were advanced at accessible location near outlet and inlet, respectively. BH3 was advanced about 4.5 m south of the culvert centreline (Sta. 9+995.5) and 18 m west of the highway centreline. BH4 was about 6.2 m north of the culvert centreline (Sta. 10+006.2) and 17 m east of the highway centreline.

As indicated above, preliminary foundation investigation at this site was performed by PML in September 2016 and general sub surface investigation information was available in Geocres (Geocres No. 410-14, provided by MTO) that include four (4) boreholes numbered FM-1 to FM-4 to a maximum depth ranging from 6.4 m to 8.1 m (Elev. 444.9 m to Elev. 445.6 m). The locations of boreholes performed by **exp** and PML are shown on Drawing 1 in Appendix B.

All boreholes drilled during this fieldwork were advanced using a track mounted CME 55 drill rig equipped with hollow stem augers and standard soil sampling equipment, operated by a specialist drilling contractor, Landcore Drilling. The roadway boreholes (BH1 and BH2) were advanced to depths of about 15.9 m and 15.6 m below ground surface, respectively and the off-road boreholes (BH3 and BH4) were advanced to a depth of about 8.7 m and 8.8 m, respectively. The roadway boreholes were advanced below the target depth of 15 m below ground surface and off-road boreholes were terminated

above target depth of 10 m below ground surface due to practical refusal (i.e more than 3 consecutive 100 blows per 0.3 m penetration). Below the auger refusal level in BH1 (~8.8 m depth) and BH2 (~10.5 m depth) the drilling was advanced using a NW casing.

The borehole locations (referenced to the MTM NAD83 coordinate system) and their ground surface elevations were surveyed by **exp** personnel using a temporary benchmark (TBM) set on top of pavement at centerline of Hwy 129 and centerline of existing culvert alignment. The elevation of temporary benchmark (TBM) was assumed 453.76 m based on the drawing attached with the preliminary foundation investigation report (PML, 2016, provided by MTO). The TBM location is shown on Drawing 1 in Appendix B.

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

Upon completion of the boreholes, ground water level measurements were carried out in boreholes in accordance with MTO guidelines. The recorded ground water levels after completion of drilling boreholes were presented in the borehole log sheets in Appendix C. The boreholes were decommissioned by bentonite/cement mixtures in accordance with the Ministry of the Environment Regulation 903, as amended by Regulation 128/03 (the well regulation under the *Ontario Water Resources Act*).

The fieldwork was supervised by an **exp** geotechnical representative who directed the drilling and sampling operation, logged borehole data in accordance with MTO and/or ASTM Standards for Soils Classification, and retrieved soil samples for subsequent laboratory testing and identification.

All recovered soil samples were placed in labelled moisture-proof bags and returned to **exp**'s Thunder Bay laboratory for additional visual, textual, olfactory examination and selective testing.

1.3.2 Previous Investigation

The following previous/historical investigation report was provided by client

- Preliminary Foundation Investigation and Design Report for Five Mile Creek Culvert Replacement on Hwy 129, Township of Reaney, Algoma District; G.W.P. 5222-05-00; Agreement # 5013-E-0040; Site No. 46-006/C; WP 5227-05-01; Geocres No. 410-14; Peto MacCallum Ltd.; September 27, 2016.

Four borehole logs produced based on the investigation conducted by PML in September 2016 at location of this culvert (identified as FM-1 to FM-4) are attached in Appendix E of this report. The details of the borehole locations and elevations completed by PML at the site location are outlined in Table 1.1. The location details of each borehole should be considered an estimate only.

In addition, the Structural Design Report Hwy 129, Five Mile Creek Culvert Site No. 46-006C, G.W.P. 5222-05-00, W.P. 5227-05-01 prepared by Aecom Canada Ltd., dated August 2016, was provided by MTO.

Table 1.1. Summary of boreholes completed by PML

BH No.	Borehole Locations (Station and Offset from the centreline) ¹	Ground Elevation (m)	Borehole Depth (m)	Borehole Bottom Elevation (m)	Piezometer/ Monitoring Well
FM-1	14 m west of Hwy centreline at about STA 10+006 (outlet)	451.3	6.4	444.9	None
FM-2	2.0 m west of Hwy centreline at about STA 9+996	453.8	7.9	445.9	None
FM-3	2.0 m east of Hwy centreline at about STA 10+004	453.7	8.1	445.6	None
FM-4	12 m east of Hwy centreline at about STA 9+994 (inlet)	452.1	6.7	445.4	None

Note: ¹ Station and offset measurements are approximate.

1.3.3 Laboratory Testing

All samples returned to the laboratory were subjected to visual examination and classification. The laboratory testing program included the determination of natural moisture content on all samples and particle size distribution for approximately 25% of the collected soil samples. All of the laboratory tests were carried out according to MTO and/or ASTM Standards as appropriate.

1.4 Subsurface Conditions

The detailed subsurface conditions encountered in the boreholes advanced during this investigation are presented on the borehole log sheets in Appendix C. Laboratory test results of grain size analyses are provided in Appendix D. The “Explanation of Terms Used in Report” preceding the borehole logs in Appendix C forms an integral part of and should be read in conjunction with this report. Borehole logs prepared by PML during their investigation are attached in Appendix E.

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

The general stratigraphy encountered within the investigated depths of PML and current investigation are inline. In general, the subsoil condition at the site consist of silty sand fill overlying peat followed by silty sand to sandy silt and silty sand till. Bedrock was not encountered at the locations of drilling.

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

1.4.1 Pavement Structure

Asphalt layer, approximately 0.05 m to 0.23 m thick, was encountered at the surface of boreholes advanced on the paved area, i.e. BH2, FM-2 and FM-3. Pavement structure consists of varying proportions of sand and gravel (base). The pavement structure extended to depths ranging between 0.2 m to 0.8 m below ground surface with elevations ranging between Elev. 453.5 m to Elev. 453.0 m. The explored thickness of this granular base was between 0.2 m to 0.57 m. The moisture contents of granular base layer were between 4.6% to 8.6%.

1.4.2 Topsoil

Topsoil, approximately 0.2 m thick, was encountered at the surface of borehole FM-4 (inlet). Topsoil thicknesses may further vary beyond the borehole locations.

1.4.3 Fill: Silty Sand

Silty sand fill was encountered below the pavement structure in all boreholes advanced through road surface (BH1, BH2, FM-2 and FM-3) and below topsoil layer in the off-road borehole FM-4. The fill layer extended to depths ranging between 1.4 m to 5.2 m below ground surface with elevations ranging between Elev. 451.0 m to Elev. 448.5 m. The explored thickness of this layer was between 1.2 m to 4.9 m.

The composition of this fill material generally consisted of sand and silt with trace gravel, trace clay organics, and trace peat. The silty sand fill was generally brown in color, and moist to wet. Upper layer of fill materials was frozen. The SPT "N" values obtained within this fill material ranged from 3 to 10 blows per 0.3 m penetration, suggesting very loose to compact in relative density. One SPT "N" value of 53 blows per 0.3 m penetration was recorded in BH2 within the upper layer of this fill material, which could be due to influence of frozen upper layer.

Laboratory testing performed on selected samples consisted of sixteen (16) moisture content tests and three (3) grain size distribution tests. The test results are as follows:

Moisture Content: (Performed by **exp** and PML)

- 14.5% to 29.8%

Grain Size Distribution: (Performed by **exp** and PML)

- 0 % gravel;

- 70% to 80% sand;
- 19% to 29% silt; and
- 1% clay

The results of the moisture content and grain size distribution tests performed by **exp** are provided on the record of borehole sheets in Appendix C. The result of the grain size distribution test performed by **exp** is also provided on Figure 1 in Appendix D. The results of tests performed by PML are shown on the borehole logs attached in Appendix E.

1.4.4 Peat

A layer of peat was encountered below the embankment fill in BH1 and BH2 which were advanced through the road surface (Note: peat was not encountered in FM-2 and FM-3) as well as at the ground surface (BH3 and BH4), below topsoil (FM-4) and below the 0.5 m thick snow and ice layer (FM-1) at locations of off-road boreholes. The peat extended to depths ranging between 0.6 m to 3.8 m with elevations ranging between Elev. 451.2 m to Elev. 448.6 m. The explored thickness of this layer was between 0.3 m to 2.1 m.

The composition of this layer consisted of organic peat with some interbedded sand seams and some roots and rootlets. The peat was dark brown to brown in color, and wet. The SPT “N” values obtained within this layer ranged from 1 to 5 blows per 0.3 m penetration, suggesting very soft to firm in consistency.

Laboratory testing performed on selected samples consisted of nine (9) moisture content tests. The test results are as follows:

Moisture Content: (Performed by **exp**)

- 25.5% to 128.1%

The results of the moisture content tests performed by **exp** are provided on the record of borehole sheets in Appendix C. The results of tests performed by PML are shown on the borehole logs attached in Appendix E.

1.4.5 Sand

A native sand was encountered underlying the peat layer in BH2, BH3 and BH4. The sand layer extended to depths ranging between 1.2 m to 5.3 m below ground surface with elevations ranging between Elev. 450.6 m to Elev. 448.3 m. The explored thickness of this layer was between 0.5 m to 1.5 m.

The composition of this sand layer is mainly sand with few silt, trace peat and trace organics. The material is brown to grey in color and wet. The SPT “N” values obtained within this layer ranged from 3 to 14 blows per 0.3 penetration, suggesting very loose to compact in relative density.

Laboratory testing performed on selected samples consisted of five (5) moisture content tests and one (1) grain size distribution tests. The test results are as follows:

Moisture Content: (Performed by **exp**)

- 17.7% to 35.0%

Grain Size Distribution: (Performed by **exp**)

- 0 % gravel;
- 94% sand;
- 6% silt and clay

The results of the moisture content and grain size distribution tests performed by **exp** are provided on the record of borehole sheets in Appendix C. The result of the grain size distribution test performed by **exp** is also provided on Figure 4 in Appendix D.

1.4.6 Sandy Silt

Native sandy silt was encountered below the embankment fill in boreholes FM-2, FM-3 and FM-4, below peat in borehole FM-1, below sand in BH3 and sandwiched between silty sand layer in BH1 and BH4. The sandy silt layer was extended to depths ranging between 3.0 m to 6.7 m below ground surface with elevations ranging between Elev. 448.3 m to Elev. 444.6 m. The explored thickness of this layer was between 0.9 m to 2.4 m.

The composition of this layer is sand and silt with trace clay, trace gravel and occasional cobbles. The material is grey in color, and moist to wet. The SPT "N" values obtained within this layer ranged from 1 to 28 blows per 0.3 m penetration, suggesting very loose to compact in relative density.

Laboratory testing performed on a selected sample consisted of thirteen (13) moisture content tests, and seven (7) grain size distribution tests. The test results are as follows:

Moisture Content: (Performed by **exp** and PML)

- 11.4% to 26.4%

Grain Size Distribution: (Performed by **exp** and PML)

- 0% to 2% gravel
- 9% to 49% sand
- 50% to 88% silt; and
- 1% to 3% clay

The results of the moisture content and grain size distribution tests performed by **exp** are provided on the record of borehole sheets in Appendix C. The results of the grain size distribution tests performed

by **exp** are also provided on Figure 3 in Appendix D. The results of tests performed by PML are shown on the borehole logs attached in Appendix E.

1.4.7 Silty Sand

Native silty sand was encountered below the sandy silt layer in boreholes FM-1, FM-2, FM-3, FM-4 and BH3, below the sand layer in BH2 and below peat/sand interbedded with the sandy silt layer in BH1 and BH4. The silty sand layer extended to depths ranging between 2.4 m to 8.1 m below ground surface with elevations ranging between Elev. 449.4 m to Elev. 444.9 m. The lower silty sand layer in BH1 and BH4 extended to depths ranging between 4.7 m and 6.9 m with elevations ranging between Elev. 447.1 m to Elev. 446.7 m. The explored thickness of this layer was between 0.7 m to 3.4 m. Boreholes FM-1, FM-2, FM-3 and FM-4 were terminated within this layer.

The composition of this layer is silt and sand, trace to some gravel, trace clay and occasional clayey silt interbeds. The material is brown to grey in color and wet. The SPT "N" values within this layer ranged from 2 to 108 blows per 0.3 m penetration, suggesting very loose to very dense, but generally loose to compact in relative density.

Laboratory testing performed on a selected sample consisted of nineteen (19) moisture content tests, and nine (9) grain size distribution tests. The test results are as follows:

Moisture Content: (Performed by **exp** and PML)

- 7.8% to 25.8%

Grain Size Distribution: (Performed by **exp** and PML)

- 0% to 18% gravel
- 50% to 80% sand
- 12% to 34% silt; and
- 4% to 8% clay

The results of the moisture content and grain size distribution tests performed by **exp** are provided on the record of borehole sheets in Appendix C. The results of the grain size distribution tests performed by **exp** are also provided on Figure 2 in Appendix D. The results of tests performed by PML are shown on the borehole logs attached in Appendix E.

1.4.8 Silty Sand Till

Native silty sand till was encountered below the silty sand layer in BH1, BH2, BH3 and BH4. The silty sand layer extended to depths ranging between 8.7 m to 15.9 m below ground surface with elevations ranging between Elev. 443.1 to Elev. 437.7 m. The explored thickness of this layer was between 4.1 m to 9.0 m. Boreholes BH1, BH2, BH3 and BH4 were terminated within this layer.

The composition of this layer is sand and silt, trace gravel and trace to few clay. The material is grey in color, wet. The SPT “N” values within this layer ranged from 11 to 100 blows per 0.3 m penetration, suggesting compact to very dense, but generally very dense in relative density.

Laboratory testing performed on a selected samples consisted of thirty two(32) moisture content tests and three (3) grain size distribution tests. The test results are as follows:

Moisture Content: (Performed by **exp**)

- 6.2% to 12.6%

Grain Size Distribution: (Performed by **exp**)

- 3% to 4% gravel
- 52% to 54% sand
- 37% to 45% silt, and
- 5% to 7% clay

The results of the moisture content and grain size distribution tests performed by **exp** are provided on the record of borehole sheets in Appendix C. The results of the grain size distribution tests performed by **exp** are also provided on Figure 2 in Appendix D.

1.5 Groundwater Conditions

The groundwater levels in the boreholes were observed during and upon completion of their drilling. During **exp**'s investigation in March 2017, the groundwater levels below the existing grade of the road were measured at a depth of 2.5 m (corresponding Elev. 451.3 m) and 3.5 m (corresponding Elev. 450.1 m) in BH2 and BH1, respectively. In the PML report it was reported that the groundwater levels measured on December 2014/January 2015 were at a depth of 2.13 m to 5.94 m (corresponding Elev. 451.8 m to Elev. 447.9 m) below the existing grade of the road. In off-road boreholes BH3 (outlet) and BH4 (inlet) the groundwater levels in March 2017 were measured at the depths of 1.4 m (corresponding Elev. 450.4 m) and 0.6 m (corresponding Elev. 451.2 m), respectively. In December 2014/January 2015, the observed groundwater below the ground surface at the outlet (FM-1) and inlet (FM-4) were at a depth of 0.46 m (corresponding Elev. 450.8 m) and 1.4 m (corresponding Elev. 450.7 m), respectively.

Water in Mile Creek was frozen at the time of March 2017 investigation. The measured elevations of the top of ice at the existing culvert were Elev. 451.66 m at the outlet and Elev. 451.79 m at the inlet.

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

2 ENGINEERING DISCUSSION & RECOMMENDATIONS

2.1 General

This section of the report provides geotechnical design recommendations for replacement of the Five Mile Creek Culvert on Highway 129 (Sta. 10+000), about 86.1 km north of the junction of Hwy 129 and Hwy 556, in the Township of Reaney (south of Chapleau), Algoma District, the Ministry of Transportation (MTO) Northeastern Region. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the current investigation at the site performed by **exp** and previous preliminary investigation performed by Peto MecCallum Ltd. (PML) dated September 2016. The compiled factual data is presented in **Part I-Foundation Investigation Report** of this report. It is understood that a new replacement culvert is designed by Aecom Canada Ltd. (Aecom). The interpretation and recommendations provided are intended solely to permit designers, to assess foundation alternatives and design the new culvert and replacement. Comments on construction are only provided to highlight issues that could affect the design. Contractors bidding on the works should make their own assessments of the factual data and how it might affect construction means and methods, scheduling and the like.

Based on the TOR and PML's preliminary FIR provided by MTO, the existing culvert constructed in 1982 is a 3.9 m span and 24 m long corrugated steel ellipse pipe structure with a fill height of 1.2 m above the crown. Based on the GA drawing prepared by Aecom and dated June 2015 (attached in Appendix J), the invert of the existing culvert at the centerline of Hwy 129 is located at approximate elevation of Elev. 448.8 m and the embankment above the creek bed is approximately 5 m high. Based on observations at the site no riprap on either side of the creek, i.e. inlet and outlet of the existing culvert, to protect against scour or erosion. Since the sides of the embankment were covered with snow, any erosion and instability was not able to be observed during the **exp** investigation. However, it was reported in the PML report that a review of the Google Earth Map indicated that the toe of the embankment on both sides of the culvert was eroded and undermined. It is understood that the condition of the existing culvert was assessed to be poor with deficiency in several elements and deterioration of the coating protecting the culvert. Further, it is reported that culvert barrel is experiencing sagging in the mid-section and requires replacement.

Based on the updated GA drawing provided by MTO on April 13 2017 (Drawing 1 in Appendix B), the existing culvert would be replaced with a new 24 m long precast concrete box culvert with an opening size of 3.6 m in span, 3.3 m in rise and a wall thickness of 0.3 m. The proposed replacement culvert does not include headwalls or wing walls on the GA drawings provided to PML (GA drawing dated June 2015) and **exp** (GA drawing dated April 2017). The proposed invert of the box culvert slopes from about Elev. 448.53 m at the inlet to an elevation of Elev. 448.38 m at the outlet. According to drawings the founding level of the subgrade at the inlet and outlet is proposed to be at Elev. 447.93 m and Elev. 447.78 m, respectively. In the updated GA drawing, it is proposed that the new culvert will be constructed at Sta. 10+005.2 (5.2 m north from the central line of the existing culvert) with no grade change at the culvert location (top elevation of Elev. 453.76 m). Based on the GA drawing, it is

understood that the existing culvert will be used to convey the upstream flow of Five Mile creek during the construction. Since there is no local detour available to divert the traffic, it is understood that the staged open-cut replacement with a properly designed temporary roadway protection to maintain the traffic on one side of the highway is the preferred replacement option by MTO.

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 provided to us at February 3 2017 together with the MTO request email. The assessment involved review of options for replacement of the existing culvert along the proposed alignment using a staged open-cut replacement method.

2.2 Expected Ground Conditions

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

- a) Hwy 129 is a two lane, north/south roadway with approximately 2.0 m wide granular shoulders at the culvert location. The highway crosses a 3.9 m span corrugated steel ellipse pipe culvert with approximately 1.2 m of embankment fill above the culvert crown, and approximately 2.5H:1V sideslopes. As mentioned before, the current elevation of the crest of the roadway is about Elev. 453.76 m.
- b) Below the pavement structure, the highway embankment consists of silty sand fill (~1.2 m to 4.9 m thick). The embankment fill is underlain by layers of native soil: soft peat (~ 0.3 m to 0.5 m thick), loose to compact sandy silt (~ 1.5 m to 2.4 m thick), loose to compact silty sand (~0.8 to 1.6 m thick) followed by compact to very dense silty sand till to about 15.6 m depth from the ground surface. The bedrock was not encountered at the site
- c) At the outlet, a surficial layer of native soft to firm fibrous peat (~1.6 m to 1.8 m thick) is underlain by loose to compact sandy silt (~0.9 m to 2 m thick), very loose to loose silty sand (~0.8 m to 2.3 m thick) and dense to very dense silty sand till (~4.1 m thick) to about 8.7 m below the ground surface. At the inlet, a layer of native soft peat (~0.6m thick) is underlain by loose to compact sandy silt (~0.1.6 m to 2 m thick) and loose to compact silty sand (~0.7 m to 1.2 m thick) followed by dense to very dense silty sand till (~ 4.1 m thick) to about 8.8 m below the ground surface.
- d) The invert of the new culvert is proposed to at Elev. 448.53 m and Elev. 448.38 m at the inlet, and outlet, respectively. The foundation soil below these levels is anticipated to be native very loose silty sand/sandy silt (~0.5 m to 1.0 m thick) with N values of 3 to 4, underlain by compact to very dense silty sand till having N values mostly above 30.

- e) At the time of investigation, the top of the ice covering the creek water was at Elev. 451.79 m and Elev. 451.66 m at the inlet and outlet side of the existing culvert, respectively. The groundwater table measured in the boreholes drilled at the inlet and outlet was between Elev. 450.4 m and Elev. 451.2 m. The groundwater table in the embankment fill is expected to be around Elev. 451.8 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. It is noted that the GA drawings indicates an approximate creek water level of Elev. 449.8 m in September 2014 which is about 2 m lower than the highest ground water level observed during the March 2017 and December 2014/January 2015 investigations.

2.3 Structure Foundations

A precast concrete box culvert with an opening size of 3.6 m in span, 3.3 m in rise and a wall thickness of 0.3 m is a proposed option for the replacement of the existing culvert. Based on information provided, it is understood that only options of concrete culverts were recommended to be considered for preliminary design purpose, while the culverts using a corrugated steel plate arch or pipe culvert were not recommended. In particular, the following options were considered as possible options for the culvert replacement and they are discussed below:

- Precast rigid frame concrete box culvert,
- Cast-in-place rigid frame concrete box culvert
- Cast-in-place rigid frame open footing concrete culvert supported on shallow foundations

Based on the subsurface information obtained from the site investigations, the native silty sand/sandy silt is considered suitable for support of all replacement options. However, the choice of culvert type also depends on parameters such as the initial cost, maintenance costs, hydraulic performance, ease of construction, water and soil corrosiveness, salvageability and local availability of material and equipment.

It is noted that regardless of the option selected, the existing culvert is to be removed. This will require excavation down to the existing founding elevation for all options (~Elev. 448.66 m). This suggests the need for surface/groundwater control as discussed in Section 2.10 below.

Any loose and/or soft soils encountered below the existing embankment should be excavated and removed to firm bearing of native soils and the grade restored with engineered fill. If the depth of excavation to remove unstable soils is excessive, using a geotextile fabric, such as Terrafix 270R or equivalent, in conjunction with engineered fill can be considered to assist in providing a stable base for the new culvert. Based on previous experience, typically a minimum of 450 mm of a clear stone accordance with OPSS 1004 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.

Based on the subsoil condition, Table 2.1 below compares the possible structure options from a foundations design and constructability perspective with their advantages and disadvantages. Although the foundation soils can provide adequate support for all options listed in the table, the use of precast rigid frame concrete box culvert is ranked highest for the criteria evaluated.

Table 2.1 Evaluation of foundation alternatives

Options	Rank	Advantages	Disadvantages	Relative Costs	Risks/ Consequences
Precast rigid frame concrete box culvert	1	<ul style="list-style-type: none"> ▪ Straightforward construction ▪ Reduced construction period, consequently traffic management and water control period ▪ Reduced excavation depth ▪ Can be more readily installed during cold weather conditions 	<ul style="list-style-type: none"> ▪ If floor is thin and poorly reinforced, it may heave and crack ▪ During high flows, the concrete floor can be undermined ▪ Susceptible to defects/leakage at joints ▪ Requires bedding material ▪ Disturbance of natural streambed ▪ Possible sediments accumulation in the upstream of the culvert 	<ul style="list-style-type: none"> ▪ Low 	<ul style="list-style-type: none"> ▪ Risk of unacceptable differential settlements if the entire foundation is not supported on the competent soil ▪ Risk of leaking from joints if not properly installed
Cast-in-place rigid frame concrete box culvert	2	<ul style="list-style-type: none"> ▪ Suitable if site is not appropriate to heavy equipment for installation of precast sections ▪ Reduced excavation depth ▪ Culvert design can be customized in the field for high stress or load conditions or other site specific requirements 	<ul style="list-style-type: none"> ▪ Slower construction process ▪ If floor is thin and poorly reinforced, it may heave and crack ▪ During high flows, the concrete floor can be undermined ▪ Requires concrete curing ▪ Disturbance of natural streambed ▪ Possible sediments accumulation in the upstream of the culvert 	<ul style="list-style-type: none"> ▪ Likely more expensive than Option 1 	<ul style="list-style-type: none"> ▪ Risk of unacceptable differential settlements if the entire foundation is not supported on the competent soil ▪ Risk of disturbance of base during construction

Options	Rank	Advantages	Disadvantages	Relative Costs	Risks/ Consequences
Cast-in-place rigid frame open footing concrete culvert	3	<ul style="list-style-type: none"> Wider span may consider to maintain existing channel and so allows for natural streambed to remain intact Less accumulation of sediments in the upstream of culvert 	<ul style="list-style-type: none"> Deeper excavation or below water excavation may required Dewatering system required Require placement of lean concrete 	<ul style="list-style-type: none"> Likely more expensive than Option 1 	<ul style="list-style-type: none"> Risk of unacceptable differential settlements if the entire foundation is not supported on the competent soil Risk of delay in construction due to deeper excavation below water if proper dewatering is not maintained Higher scour risk

2.3.1 Shallow Foundations

2.3.1.1 Geotechnical Resistance

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

Table 2.2 Recommended spread footing design parameters

Culvert Type	Founding Elevation (m)	Footing Size (m)	Founding Soil Type	Factored Geotechnical Resistance at ULS (kPa)	Geotechnical Reaction at SLS ² (kPa)
Precast or cast-in-place rigid frame concrete box culvert	~447.8	4.2	Native very loose silty sand/sandy silt	225	145
	~447.8	4.2	~0.5 m thick lean concrete over native compact to dense silty sand till or minimum 0.5 m compacted granular material (Granular A or Granular B Type II) ³ over native compact to dense silty sand till	400	250
Cast-in-place rigid frame open footing concrete culvert	~445.5 ¹	1.0	Native dense to very dense silty sand till	550	350

Notes:

1. Below the frost line. Requires deeper groundwater control.
2. for maximum settlement of 25 mm
3. the granular material used for the granular pad shall be Granular 'A' or Granular B Type II conforming to OPSS 1010 and compacted to 98 % SPMDD

It is assumed that, if any, underlying organic soils and any other soft or very loose materials are to be replaced with clean and compactable soil such as Granular A or Granular B Type II. Given that no (or minimal) 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.

2.3.1.2 Resistance to Lateral Loads

Resistance to lateral forces/ sliding should be calculated in accordance with Section 6.10.5 of the CHBDC, using the following parameters:

Table 2.3 Recommended parameters for calculation of unfactored horizontal resistance

Interface and Loading Conditions	Parameters
Between Granular A and pre-cast concrete	Coefficient of friction ($\tan \delta$)=0.7
Between cast-in-place concrete and native silty sand	Coefficient of friction ($\tan \delta$)=0.55

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

2.3.1.3 Frost Protection

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

Since the earth cover above the top of the proposed culvert is estimated to be 1.8 m which is less than the frost penetration line of 2.3 m, frost tapers are required at this site. Frost tapers with granular backfill should be constructed in accordance with OPSD 3101.150.

2.4 Lateral Earth Pressure

Culvert walls and temporary shoring should be designed to resist lateral earth pressure. The expression for calculating lateral earth pressure is given by:

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

where

P = earth pressure intensity at depth h , kPa

K = earth pressure coefficient

γ = unit weight of retained soil, kN/m³

q = surcharge near wall, kPa

h = depth to point of interest, m

The above expression does not take into account hydrostatic pressure, which must be included for the groundwater levels measured on the site. Table 2.4 lists earth pressure parameters for given materials. These recommendations assume level backfill and ground surface behind the walls.

Table 2.4 Material types and earth pressure properties

Material	Unfactored Friction Angle ϕ'	Coefficient of Active Earth Pressure (K_a)	Coefficient of Passive Earth Pressure (K_p)	Coefficient of Earth Pressure At-Rest (K_0)	Unit Weight γ kN/m ³
Silty Sand Fill (very loose to loose)	28	0.36	2.77	0.53	20
Peat (very soft)	15	0.59	1.70	0.74	12
Silty Sand (loose to compact)	29	0.35	2.85	0.52	20
Sandy Silt (loose to compact)	27	0.37	2.66	0.55	19
Silty Sand Till (compact to very dense)	32	0.31	3.25	0.47	21
Granular A	35	0.27	3.69	0.43	22
Granular B Type II	35	0.27	3.69	0.43	22

The mobilization of full active or passive resistance requires a measurable and perhaps significant wall movement or rotation. Therefore, unless the structural element can tolerate these deflections, the at-rest earth pressure should be used in design. This would normally be the case for concrete box culverts.

The effect of compaction surcharge should be taken into account in the calculations of active and at-rest earth pressures. The lateral pressure due to compaction should be taken as at least 12 kPa at the surface, and its magnitude should be assumed to diminish linearly with depth to zero at the depth where the active (or at-rest) pressure is equal to 12 kPa. This pressure distribution should be added to the calculated active (or at-rest) pressure. Notwithstanding, lighter compaction equipment and smaller lifts should be used adjacent to culvert walls to prevent overstressing.

It is likely that bracing for the temporary support system will be required at a maximum interval of 5 m. For multiple support systems refer to *Canadian Foundation Engineering Manual* (CFEM) for apparent earth pressure distributions (CFEM, Section 26.10.3, Figure 26.8)

2.5 Seismic and Liquefaction Potential Consideration

Seismic characterization of the site must be compliant with the Canadian Highway Bridge Design Code CHBDC (CAN/CSA-S6-14). The potential for seismic loading must be considered for design in accordance with Section 4.4 of the CHBDC with respect to soil conditions encountered at the site. Table 4.1 in CHBDC (see Clause 4.4.3.2) shows site classification for seismic site response based on soil average properties in top 30 m. The borehole information shows the presence of native compact

to very dense soil, no bedrock encountered at investigated depth. Based on these soil characteristics, the site class for this site is estimated to be Class "D" according to Table 4.1.

From the Natural Resources Canada website, 2015 NBCC seismic hazard values are obtained using the site location coordinates (47.844486°N, 83.404366°W) and the damped reference spectral accelerations for the project site are $S_a(0.2)=0.028g$, $S_a(0.5)=0.022g$, $S_a(1.0)=0.013g$, $S_a(2.0)=0.0055g$ and the reference peak ground acceleration (PGA) is $0.015g$ (g =acceleration due to gravity -9.81 m/s^2). These values are associated with an earthquake having 10 percent probability of exceedance in a 50-year period.

Based on soils and groundwater condition encountered at the site, no liquefaction is expected due to the ground motion from an earthquake having 10% probability of exceedance in a 50-year period.

2.6 Construction Alternatives

Since there is no local detour available to divert the traffic, it is understood that staged open-cut construction is being preferred as a culvert replacement option at this site. Therefore, the following possible staged open-cut alternatives for the new culvert installation are listed below:

1. Half-and-half construction using roadway protection to allow excavation as maintaining signalized one lane of traffic on the existing embankment during construction. The following two options of excavation and replacement using the half-and-half approach were considered:
 - A. Construction using roadway protection and unsupported excavation of cut sides
 - B. Construction using roadway protection and braced cut sides
2. Construct temporary detour embankments at the site followed by open cut/unsupported excavation to expose and replace culvert.
3. Stage construction by grade lowering of the existing embankment using unsupported excavation to maintain signalized one-way traffic during construction

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. For all approaches provision must be made to maintain surface water flow to the outlet.

The following Table 2.5 summarize advantages and disadvantages of considered construction alternatives. The table also shows assessed risk/consequences and relative costs of the considered methods. Schematic diagrams of considered alternatives are attached in Appendix H.

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

Installation Method	Advantages	Disadvantages	Relative Cost	Ranking
<p>OPTION 1.A</p> <p>Half-and-half Construction with Unsupported Cut Sides (Figure H1.A, Appendix H)</p>	<ul style="list-style-type: none"> • Traffic flow maintained at the site during construction • Short mobilization time • Straight forward construction and construction procedures 	<ul style="list-style-type: none"> • Traffic interruption • Roadway protection of up to 7.0 m high required to maintain one lane of traffic • Require dewatering to provide safe temporary cut slopes • High cost of roadway protection system • Large amount of soil to be excavated • Need to temporarily control existing creek water • Risk of cost overrun and inability to finish job: low to moderate 	<p>Relatively more expensive than Option 2 due to high costs of roadway protection system, however, expensive dewatering is required for improvement of global stability of temporary cut slopes</p>	2
<p>OPTION 1.B</p> <p>Half-and- half Construction with Braced or Anchored Cut Sides (Figure H1.B, Appendix H)</p>	<ul style="list-style-type: none"> • One or possibly two lanes of traffic flow maintained on existing road (e.g. steel decking, but costly) • Global stability of excavation enhanced by narrow geometry • Less traffic interruption than with unsupported cut sides approach • Temporary decking could be usable over braced cut to allow for excavation of both halves prior to diverting stream and backfilling • Cost savings due to limited excavation and backfill • Cost saving due to no need for temporary cut slopes and extensive dewatering 	<ul style="list-style-type: none"> • Traffic interruption • Roadway protection of up to 7.0 m high required to maintain one lane of traffic if steel decking is not possible • High cost of roadway protection system and/or decking • Require side shoring and bracing • Bracing (e.g. struts) may interfere with excavation • Excavation of material and placement of bracing required in limited space • Need to decommission the shoring system • Need to temporarily control existing creek water • Risk of cost overrun and instability to finish job: low to moderate 	<p>More expensive than other open cut sides approach due to high costs for shoring system and temporary decking (if feasible) to maintain continuous flow of traffic, however, global stability of excavation enhanced by shoring and no need for extensive dewatering</p>	1

Installation Method	Advantages	Disadvantages	Relative Cost	Ranking
<p>OPTION 2</p> <p>Temporary Local Detour and Open Cut Unsupported Excavation</p> <p>(Figure H2, Appendix H)</p>	<ul style="list-style-type: none"> • Traffic flow maintained at the site during construction • Simple detour roads can be constructed • Existing culvert will completely remove and replaced with new culvert • No excavation support required 	<ul style="list-style-type: none"> • Construction of detour embankments required at one side of highway • Increased time for construction of detour • Large amount of soil to be excavated • Erosion control of temporary cuts required • Need to temporarily control existing creek water • Risk of cost overrun and inability to finish job: low to moderate • Possible extra cost to purchase of private property 	<p>Less expensive than Option 1A</p>	<p>4</p>
<p>OPTION 3</p> <p>Stage Construction by Grade Lowering</p> <p>(Figure H3, Appendix H)</p>	<ul style="list-style-type: none"> • Traffic flow maintained at the site during construction without construction of local detour • No earth embankment fill material is required for building detours • No settlement since there is no new earth embankment fill • No shoring scheme necessary 	<ul style="list-style-type: none"> • Traffic slowdown and congestion due to too many traffic lane changes • Large amount of embankment fills to be excavated and replaced; Road has to be excavated longitudinally (~120 m) to obtain 10H:1V slope • Need to temporarily control existing creek water • Additional cost for reconstruction of road • Increased time for construction of staging • Erosion control of temporary cuts required • Risk of cost overrun and inability to finish job: low to moderate 	<p>More expensive than other options due to large amount of embankment fill to be excavated and replaced</p>	<p>3</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.B: Half-and-half construction with braced or anchored cut sides (Figure H1.B, Appendix H)
2. OPTION 1.A: Half-and-half construction with unsupported cut sides (Figure H1.A, Appendix H)
3. OPTION 3: Stage construction by grade lowering (Figure H3, Appendix H)
4. OPTION 2: Temporary local detour and open cut unsupported excavation (Figure H2, Appendix H)

The following sections discuss these options in more details.

2.6.1 Half-and-Half Construction (Options 1)

The half-and-half construction method could be utilized to maintain the flow of the traffic on Hwy 129 (see Figures H1.A, and H1.B, Appendix H). In this method, one lane of the existing highway will be used to maintain the local traffic while the other half of the existing highway will be excavated and the half of the existing culvert will be exposed. Then the excavated portion of the existing culvert will be removed and replaced with a new culvert, followed by rebuilding of that half of the embankment to grade. Upon completion of the new embankment, the traffic will be moved onto the new fill and the process will be repeated to complete the construction and culvert replacement.

The temporary excavation required to remove half of the existing embankment would be up to 7.0 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 Contractor 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 2.7. Using the half-and-half construction approach, two methods of culvert replacement were considered for this site suitable as discussed below:

- A. Construction using roadway protection and unsupported excavation of cut sides
- B. Construction using roadway protection and braced or anchored cut sides

2.6.1.1 Option 1.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 H1.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 the Contractor.

Option 1.A could be more economical due to possible cost savings for reversible wall configuration, but it will be more disruptive to the highway embankment than Option 1.B since it needs to excavate a large amount of soil (i.e. the temporary cut slopes of 3H:1V below the groundwater level will be required to provide the safe stable condition), or extensive dewatering is required.

2.6.1.2 Option 1.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 H1.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 1.A, decommissioning of the shoring system and temporary surface water flow control must be performed/developed by contractor.

Option 1.B will disrupt less of the embankment than Option 1.A but it might cost more due to the cost of an additional shoring system. However, the global stability of excavation will be enhanced with that shoring system. Both options require decommissioning of shoring system upon completion of the work.

2.6.2 Temporary Local Detour (Options 2)

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. The advantages are that neither excavation support nor roadway protection is required with this option. The major disadvantages 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.

The local 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. 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).

2.6.3 Stage Construction by Grade Lowering (Option 3)

The stage construction by grade lowering is schematically presented in Figure H3, Appendix H. As shown, the method includes several stages of excavation of the existing road maintaining the constant

one-way traffic flow. This method does not require the road protection system or detours, but it is very disruptive to the highway, since approximately 120 m of the road has to be excavated longitudinally to obtain a maximum slope of 10H:1V.

2.7 Temporary Roadway Protection

Temporary roadway protection is anticipated to be a part of the half-and-half construction approach that will be required to maintain on-site traffic during the construction. It is recommended that roadway protection system should be design in accordance with OPSS.PROV 539. The lateral movement of the temporary shoring system should meet Performance Level 2 as specified. The complete design, construction, monitoring and removal of the installed protection system should be a responsibility of the Contractor. Due to nature of this application it is expected that much of temporary shoring will be decommissioned in place noting the high cost for removal. Decommissioning must be consistent with good practice to avoid interference with highway systems and utilities, if any. The protection system should be designed to provide protection for excavations as required by the OHSA, at locations specified in the contract, and at any locations where the stability, safety or function of an existing structure and/or utility may be impaired by construction work.

Since very dense silty sand till layer ($N \geq 100$) exists within shallow depths. At this site, the shoring system such as soldier piles and timber lagging may be considered for design. It should be designed based on the earth pressures coefficients and soil parameters provided in Section 2.4. The actual depth of embedment should be determined by balancing moments about the pile tip. However, considering the height of the roadway embankment temporary shoring system with additional anchorage or tiebacks may be required for lateral resistance. Conventional practice is to incorporate either buried deadman anchors or soil grouted anchors. Alternatively, a system of rakers can be used for support.

Deadman anchors can be designed based on the earth pressure coefficients and soil parameters provided in Section 2.4. For this project, either continuous or individual concrete block anchors would likely be appropriate. The anchor resistance is provided by a combination of the dead weight and passive resistance. For the full passive resistance to be realized with no load transfer to the wall, the anchor needs to be fully beyond the active wedge acting on the wall. Pressure grouted soil anchors can be also designed in a preliminary fashion in accordance with Section 26 of the CFEM (2006). Based on the generally compact soils at this site, the estimated factored (0.4) ULS resistance of grouted anchors would be 40 kN/m length. 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.

For design of the timber lagging, earth pressures can be reduced by 25 percent to account for soil arching effects. This is provided that the center-to-center spacing of the soldier piles does not exceed 2.5 m. Excavation can proceed following installation of the soldier piles. The unshored height of the excavation should not exceed 1.2 m at any given time. No excavation height should remain unshored for more than 24 hours.

Cobbles were encountered in some boreholes during the investigations. Therefore, a Non-Standard Special Provision (NSSP) to alert the contractor about the presence of cobbles and/or boulders is included in Appendix I.

As mentioned above, 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.

2.8 Excavation

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). Silty sand fill and native silt sand and sandy silt soils may be classified as a Type 3 soil above the groundwater table in conformance with the OHSA. The 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, while 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).

2.9 Culvert Bedding

MTOD 803.021, OPSD 3101.150 and Figure C6.20 of (CHBDC) which are included in Appendix G provide the bedding, embedment, cover and backfill standards for the concrete box culverts. According to these standards the culvert bedding should consist of Granular A (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 accordance with OPSS.PROV 501 before a subsequent layer is placed in accordance with OPSS. PROV 401.

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

For the site area, a frost penetration depth of approximately 2.3 m can occur in open, unheated areas without snow cover. At the culvert inlet and outlet, and beneath the proposed culvert, mostly the native soils consist of silty sand and sandy silt. This material has low to medium frost susceptibility based upon the MTO Frost Classification guideline of percent particles between 5 to 75 μm . Therefore, non-frost susceptible materials such as sand and gravel might be considered 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.

2.10 Culvert Backfill

The selection and placing of the backfill and cover should be in accordance with OPSS 902, OPSS 422 and OPSD 3101.150 for concrete box culverts. The backfill should consist of free-draining, non-frost susceptible granular materials confirming to OPSS.PROV 1010.

For fills immediately below any roadway, it is recommended that Granular A or B materials be used. Where necessary, proper tapering as per standards should be provided. Below a depth of about 2.3 m from any finished road grade, approved compactable fill, such as select subgrade materials (OPSS.PROV 1010) or imported fill can be used.

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

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

Where less than frost depth (2.3 m) of earth cover is provided above the top of the culvert, a frost taper should be included as per OPSD 803.030, 803.031, MTOD 803.021, whichever is applicable. Since, earth cover above the top of the culvert is less than 2.3 m so frost taper is required at this site with accordance to 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.

2.11 Groundwater and Surface Water Control

The groundwater level at the site was encountered between Elev. 451.8 m and Elev. 447.9 m, while the excavation to the foundation level has to be carried out to Elev. 447.8 m. Therefore, it is possible that the groundwater table is about 4 m above the bottom of excavation. Considering that the soils encountered below the groundwater table and within potential excavation depths consist of native silty sand and sandy silt, it is assessed that these soils are susceptible to disturbance from groundwater and mobilized equipment. Therefore, 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, where the excavation base is within 0.5 m of the prevailing groundwater level at the time of construction, it is anticipated that control of seepage can be accomplished by using properly filtered sumps. However, given the conditions at the site, it is expected that positive dewatering systems will be required to control the groundwater seepage.

Based on the GA drawing, it is understood that the existing culvert will be used to convey the upstream flow of Five Mile creek during the construction. In addition, for the control of the water flow in the creek and protection of the construction area a cofferdam might be required. A cofferdam consisting of sand bags and clay puddle may be used. Dewatering requirements behind the cofferdam to keep the construction site dry will be impacted by water levels in the stream at the time of construction activities. However, dewatering may be carried out from the sumps located along the periphery of the cofferdam.

Dewatering shall be carried out in accordance with OPSS.PROV 517 and OPSS.PROV 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. Alternatively, and in accordance with SP 5017F01 (approval pending), the dewatering systems may be completed by a design Engineer and design-checking Engineer with a minimum of 5 year experience. For this application, this is considered a suitable approach but the owner should make final decision. Based on the estimated permeability of silty sand ($k \sim 5 \times 10^{-5}$ m/s), the preconstruction survey distance should be approximately 100 m.

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.

Dewatering may require water taking permits (i.e. Permit To Take Water PTTW), if the volume of pumped water exceeds 50,000 L/day. The rate and volume required for dewatering will be dependent on construction methods and staging chosen by the Contractor.

2.12 Embankment Design

2.12.1 Stability Analysis

A preliminary slope stability analysis was performed to assess the global stability of the final embankment configuration and to check that a minimum Factor of Safety of 1.3 will be achieved. 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.

The cross-section and the approximate slopes were developed based on the drawing provided by MTO. The stratigraphy and groundwater condition at the site were developed based on the results of the geotechnical investigation presented in Part I - Foundation Investigation Report.

Based on the borehole information, the subsoils encountered at the work area consist of embankment fill, underlain by silty sand and sandy silt deposits. Therefore, an effective stress analysis for a long term assessment of the embankment slope was performed taking into consideration the subsoil conditions encountered beneath the existing embankment. The analyses assume that all topsoil and peat encountered in boreholes will be removed prior to construction.

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

Soil Type	Effective Stress Parameters		
	ϕ' (degrees)	c' (kPa)	γ' (kN/m ³)
Silty Sand Fill (very loose to loose)	28	0	20
Peat (very soft)	15	0	12
Silty Sand (loose to compact)	29	0	20
Sandy Silt (loose to compact)	27	0	19
Silty Sand Till (compact to very dense)	32	0	21

The results of slope stability analyses suggest that a minimum factor of safety above 1.3 could be obtained if the embankment with 2.5H:1V slopes is constructed after culvert replacement with the open

cut method and any peat layer within its footprint is eliminated prior the embankment construction.

2.12.2 Embankment Settlement

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

2.13 Inlet and Outlet

2.13.1 Erosion Protection at Outlet

The detailed design for erosion protection should be carried out by a 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.

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.

2.13.2 Stream Bed Rip-Rap

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

2.13.3 Seepage Cut-off Requirements

The seepage cut-off requirements should be reviewed in the following context. The native silty soils at the inlet side, 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.

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

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

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

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

2.13.3.2 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.

2.14 Obstructions

Very dense silty sand till layer was noted to be underneath silty sand layer at the most boreholes. Cobbles and boulders occur in this deposits. These potential obstructions may impact excavations and/or element of temporary protection systems. A non-standard special provision is provided in Appendix I which may form the basis for advise to the Contractor on this issue.

3 CLOSURE

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

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

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


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

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

exp Services Inc.



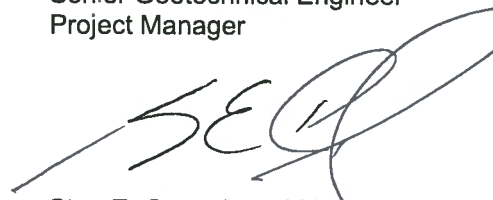
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4 LIMITATIONS AND USE OF REPORT

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. Drilling of BH1 at Hwy 129, facing North



Photo 2. Drilling of BH1. Inlet of the existing culvert, facing West



Photo 3. Drilling of BH2 at Hwy 129, facing South



Photo 4. Drilling of BH2. Outlet of the existing culvert, facing East

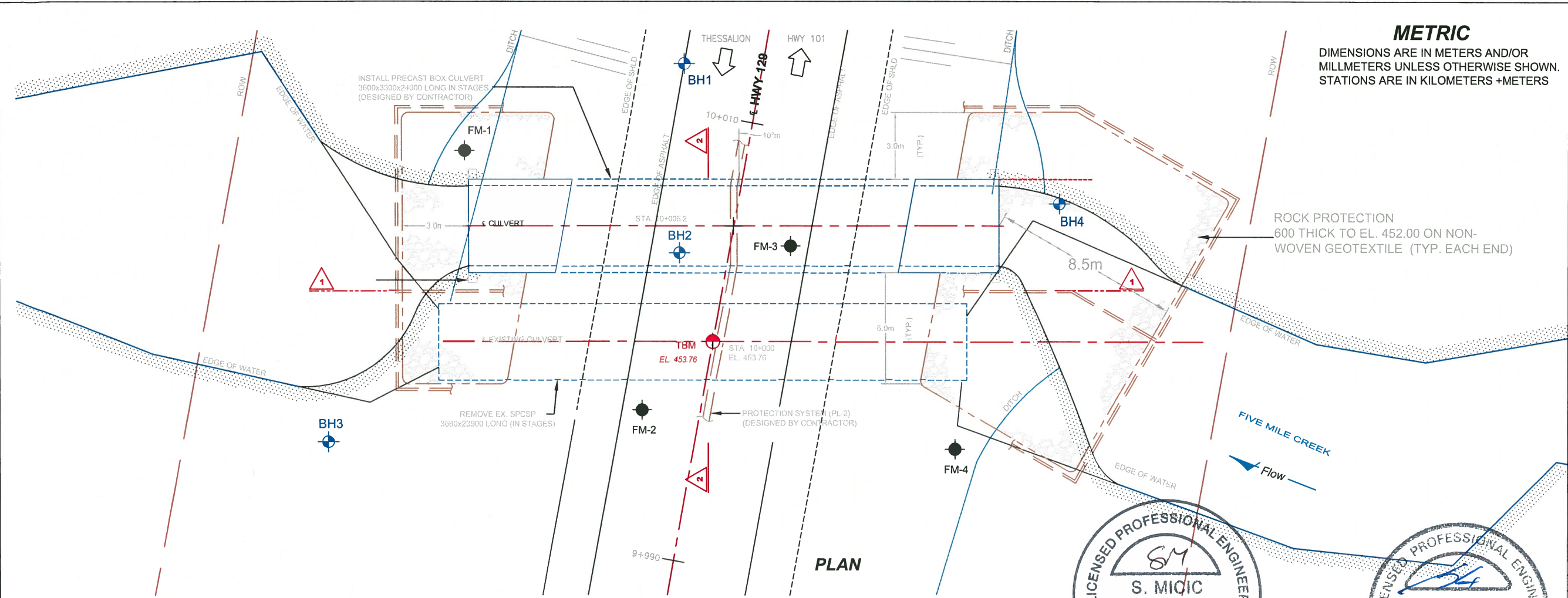


Photo 5. Drilling of BH3, facing Southeast. Outlet of the existing culvert



Photo 6. Drilling of BH4, facing Northwest. Inlet of the existing culvert

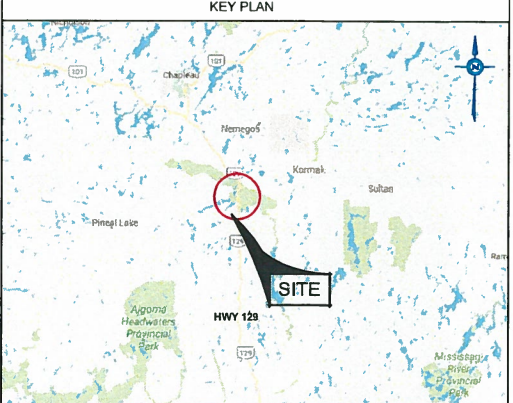
Appendix B – Drawings



Agreement No. 3015-E-0007
Assignment No. 6
WO - 2017-1105

**PROPOSED REPLACEMENT OF
FIVE MILE CREEK CULVERT ON HWY 129,
TOWNSHIP OF REANEY, ALGOMA DISTRICT, ON
BOREHOLE LOCATION PLAN AND SOIL STRATA**

exp Services Inc.



- LEGEND**
- New Borehole by exp (March, 2017)
 - Previous Borehole by Peto (September, 2016)
 - N Standard Penetration Test (Blows/0.3 m)
 - Water Level Upon Completion of Drilling
 - Temporary Bench Mark (EL. 453.76 m)

SOIL STRATA SYMBOLS

ASPHALT	PEAT
FILL	SANDY SILT/ SILTY SAND
TOPSOIL	SILTY SAND (TILL)
SAND	


BH No.	APPROX. ELEV.	MTM CO-ORDINATES	
		NORTH	EAST
BH1	453.6	527009	364145
BH2	453.7	5270086	364149
BH3	451.8	5270076	364138
BH4	451.8	5270091	364164
FM-1	451.3	5270083	364136
FM-2	453.8	5270077	364149
FM-3	453.7	5270087	364153
FM-4	452.1	5270080	364163

NOTE

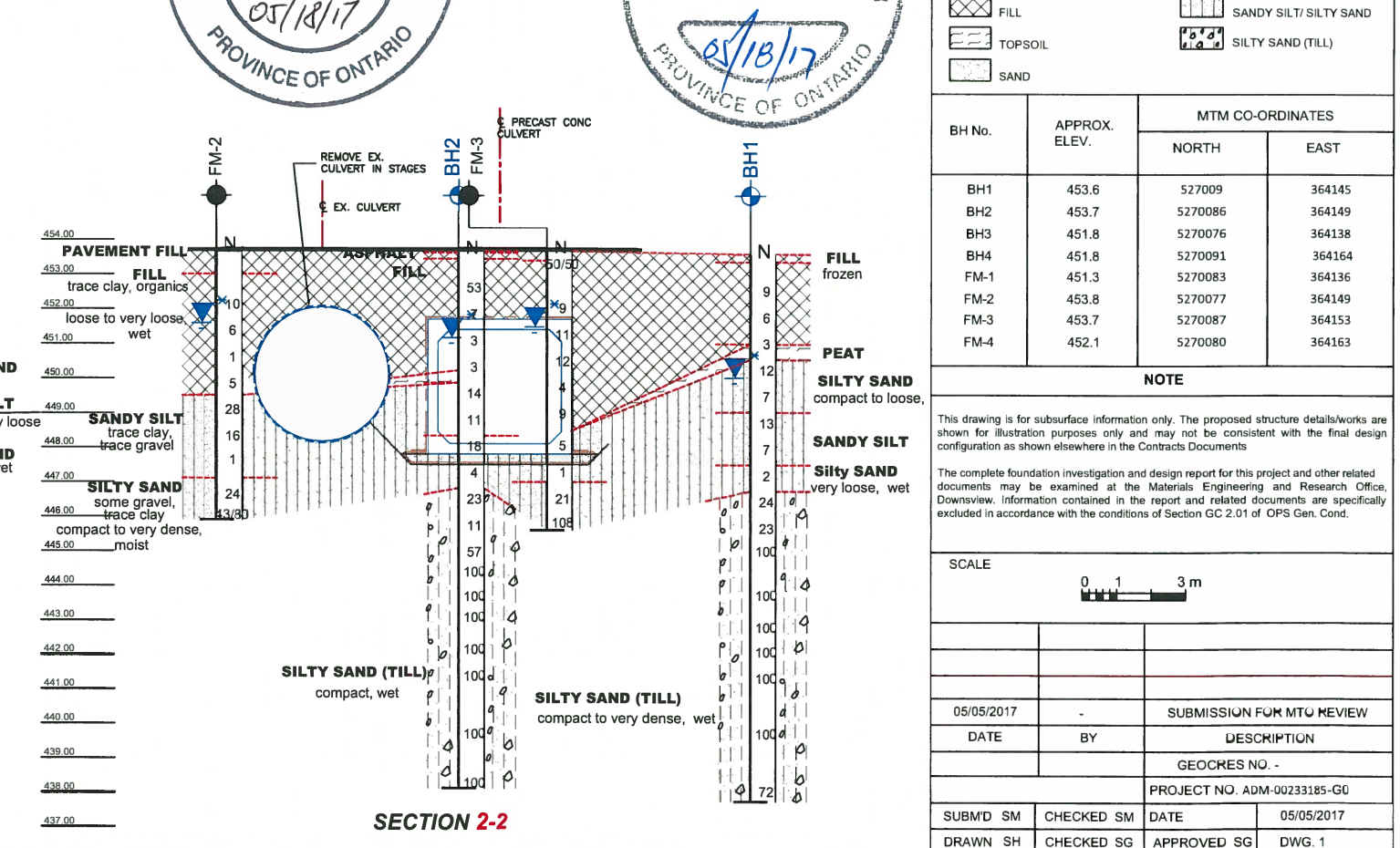
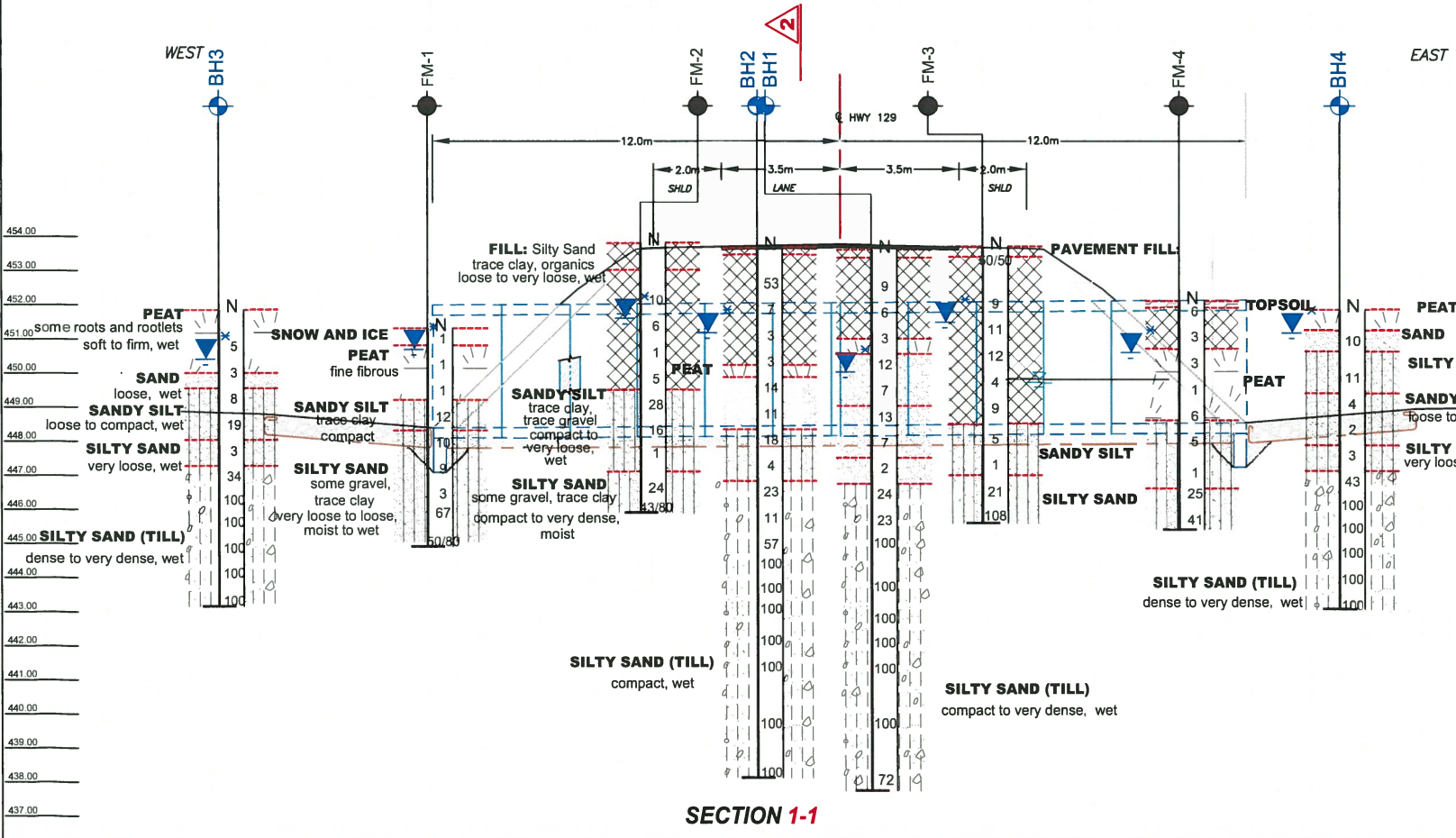
This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

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

SCALE



05/05/2017	-	SUBMISSION FOR MTO REVIEW	
DATE	BY	DESCRIPTION	
		GEOCRES NO. -	
		PROJECT NO. ADM-00233185-G0	
SUBM'D SM	CHECKED SM	DATE	05/05/2017
DRAWN SH	CHECKED SG	APPROVED SG	DWG. 1



Appendix C – Borehole Logs

Explanation of Terms Used on Borehole Records

SOIL DESCRIPTION

Terminology describing common soil genesis:

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

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

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

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

Terminology describing soil structure:

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

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

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

Fissured: material breaks along plane of fracture.

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

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

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

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

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

Homogeneous: same color and appearance throughout.

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

Uniformly Graded: predominantly on grain size.

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

ISSMFE SOIL CLASSIFICATION											
CLAY	SILT			SAND			GRAVEL			COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE		
<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:

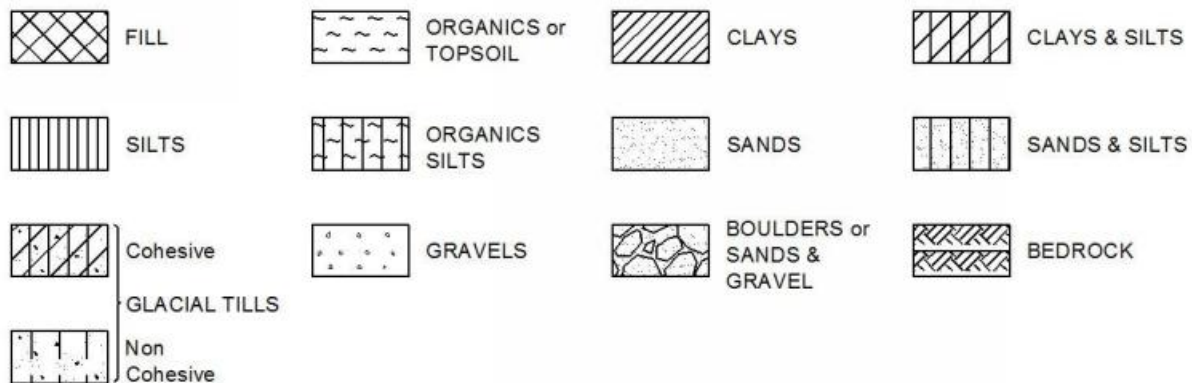
Table c: Consistency of Cohesive Soil

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



ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

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

STRESS AND STRAIN

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

MECHANICAL PROPERTIES OF SOIL

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

PHYSICAL PROPERTIES OF SOIL

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

Brampton, Ontario

RECORD OF BOREHOLE No BH1

1 OF 2

METRIC

W. P. GWP No. 5222-05-00 LOCATION Five Mile Creek, Chapleau, Highway 129 MTM ON-13 5,270,094N 364,145E ORIGINATED BY EF
 DIST Algoma HWY Hwy 129 BOREHOLE TYPE CME 55 Track Carrier / HSA / HQ COMPILED BY JZ/AM
 DATUM Geodetic DATE 2017/03/14 - 2017/03/14 CHECKED BY SM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100					
453.6	Sand and Gravel																
453.4	SAND and GRAVEL (Fill) - frozen, brown		S1	AUGER													
0.2	Silty SAND (Fill) - frozen, brown		S2	AUGER													
	- becoming loose, moist at about 0.8 depth		S3	SS	9												
	- becoming wet, at about 1.5 m depth		S4	SS	6												
	- becoming very loose, trace peat at about 2.3 m depth		S5	SS	3												
451.0	PEAT - soft, brown, wet, some interbedded sand seams																
2.6																	
450.6	Silty SAND - compact to loose, grey, wet, fine grained, some rootlets in upper 3.8 m		S6	SS	12												
3.1																	
	- occasional about 3 to 6 mm interbedded clayey silt from 3.8 m depth to 4.6 m depth		S7	SS	7												
	- about 300 mm of sand blow-up inside augers upon SPT removal, at about 3.8 m depth																
449.0	Sandy SILT - compact to loose, grey, wet		S8	SS	13												
4.6																	
	- occasional pebbles at about 5.3 m depth		S9	SS	7												
447.5	Silty SAND - very loose, grey, wet, occasional pebbles		S10	SS	2												
6.1																	
446.7	Silty SAND (Till) - compact to very dense, grey, wet		S11	SS	24												
6.9																	
	- becoming moist at about 8.4 m depth		S12	SS	23												
	- refusal to auger at about 8.8 m depth																
	- NW casing advaced from 8.8 m to 15.3 m depth		S13	SS	100												
			S14	SS	100												

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

EXP RECORD OF BOREHOLE F-17105-AG - ADM-00233185-G0 - MTO 6 - FIVE MILE CREEK - HWY 129 GPJ ONTARIO MOT.GDT 4/13/17

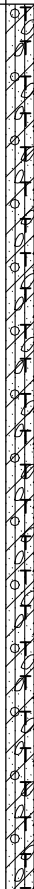
Brampton, Ontario

RECORD OF BOREHOLE No BH1

2 OF 2

METRIC

W. P. GWP No. 5222-05-00 LOCATION Five Mile Creek, Chapleau, Highway 129 MTM ON-13 5,270,094N 364,145E ORIGINATED BY EF
 DIST Algoma HWY Hwy 129 BOREHOLE TYPE CME 55 Track Carrier / HSA / HQ COMPILED BY JZ/AM
 DATUM Geodetic DATE 2017/03/14 - 2017/03/14 CHECKED BY SM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH: Cu, KPa ○ UNCONFINED + FIELD VANE × QUICK TRIAXIAL LAB VANE									
								20	40	60	80	100	10	20	30		
	Silty SAND (Till) - compact to very dense, grey, wet (<i>continued</i>) Silty SAND (Till) - continued																
			S15	SS	100		443								○		
			S16	SS	100		442								○		
			S17	SS	100		441								○		
							440							○			
			S18	SS	100												
							439										
			S19	SS	72		438							○			
437.7 15.9	End of Borehole																

EXP RECORD OF BOREHOLE F-17105-AG - ADM-00233185-G0 - MTO 6 - FIVE MILE CREEK - HWY 129 GPJ ONTARIO MOT.GDT 4/13/17

1 OF 2

METRIC

ORIGINATED BY EF

COMPILED BY JZ/AM

CHECKED BY SM

EXP RECORD OF BOREHOLE F-17105-AG - ADM-00233185-G0 - MTO 6 - FIVE MILE CREEK - HWY 129.GPJ ONTARIO MOT.GDT 4/13/17

Continued Next Page

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

Brampton, Ontario

RECORD OF BOREHOLE No BH2

2 OF 2

METRIC

W. P. GWP No. 5222-05-00 LOCATION Five Mile Creek, Chapleau, Highway 129 MTM ON-13 5,270,086N 364,149E ORIGINATED BY EF
 DIST Algoma HWY Hwy 129 BOREHOLE TYPE CME 55 Track Carrier / HSA / HQ COMPILED BY JZ/AM
 DATUM Geodetic DATE 2017/03/15 - 2017/03/15 CHECKED BY SM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)							
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH: Cu, KPa ○ UNCONFINED + FIELD VANE × QUICK TRIAXIAL LAB VANE									WATER CONTENT (%)			GR	SA	SI	CL	
								20	40	60	80	100	20	40	60		80	100	10	20	30			
	Silty SAND (Till) - compact, grey, wet <i>(continued)</i> Silty SAND (Till) - continued - refusal to auger at about 10.5 m depth - NW casing advaced from 10.5 m to 15.3 m depth - becoming moist at about 12.2 m depth		S15	SS	100																			
			S16	SS	100																			
			S17	SS	100																			
			S18	SS	100																			
			S19	SS	100																			
			S20	SS	100																			
438.1 15.6	End of Borehole																							

EXP RECORD OF BOREHOLE F-17105-AG - ADM-00233185-G0 - MTO 6 - FIVE MILE CREEK - HWY 129 GPJ ONTARIO MOT.GDT 4/13/17

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

Brampton, Ontario

RECORD OF BOREHOLE No BH3

1 OF 1

METRIC

W. P. GWP No. 5222-05-00 LOCATION Five Mile Creek, Chapleau, Highway 129 MTM ON-13 5,270,076N 364,138E ORIGINATED BY EF
 DIST Algoma HWY Hwy 129 BOREHOLE TYPE CME 55 Track Carrier / HSA COMPILED BY JZ/AM
 DATUM Geodetic DATE 2017/03/15 - 2017/03/16 CHECKED BY SM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100					
451.8	Peat																
	PEAT - soft to firm, brown, wet, some roots and rootlets		S1	AUGER												53.2	
	- some interbedded sand seams at about 0.8 m depth		S2	SS	5												
450.0																	
1.8	SAND - loose, grey, wet		S3	SS	3												
449.5																	
2.3	Sandy SILT - loose to compact, grey, wet		S4	SS	8												
	- possible cobbles noted during augering at about 2.8 m depth																
	- about 610 mm of sand blow-up inside augers upon SPT removal, at about 3.0 m depth		S5	SS	19												
448.0																	
3.8	Silty SAND - very loose, grey, wet		S6	SS	3												
447.3																	
4.6	Silty SAND (Till) - dense to very dense, grey, wet		S7	SS	34												
			S8	SS	100												
	- some cobbles noted during augering at about 5.8 m depth		S9	SS	100												
			S10	SS	100												
			S11	SS	100												
443.1			S12	SS	100												
8.7	End of Borehole																

EXP RECORD OF BOREHOLE F-17105-AG - ADM-00233185-G0 - MTO 6 - FIVE MILE CREEK - HWY 129 GPJ ONTARIO MOT GDT 4/13/17

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

Brampton, Ontario

RECORD OF BOREHOLE No BH4

1 OF 1

METRIC

W. P. GWP No. 5222-05-00 LOCATION Five Mile Creek, Chapleau, Highway 129 MTM ON-13 5,270,091N 364,164E ORIGINATED BY EF
 DIST Algoma HWY Hwy 129 BOREHOLE TYPE CME 55 Track Carrier / HSA COMPILED BY JZ/AM
 DATUM Geodetic DATE 2017/03/16 - 2017/03/16 CHECKED BY SM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)												
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH: Cu, KPa ○ UNCONFINED + FIELD VANE × QUICK TRIAXIAL LAB VANE					W _P	W	W _L		GR	SA	SI	CL									
451.8	Peat						451																						
	PEAT - soft, brown, wet, some sand to sandy		S1	AUGER																									
451.2								451																					
0.6	SAND - loose, brown, wet, trace peat, trace organics		S2	SS	10																								
450.6									450																				
1.2	Silty SAND - loose to compact, grey, wet		S3	SS	11																								
										449																			
449.4	Sandy SILT - loose to very loose, grey, wet		S4	SS	4																								
2.4											448																		
			S5	SS	2																								
447.9												447																	
4.0	Silty SAND - very loose, grey, wet - about 1.2 m of sand blow-up inside augers upon SPT removal, at about 4.0 m depth		S6	SS	3																								
447.1						446																							
4.7	Silty SAND (Till) - dense to very dense, grey, wet		S7	SS	43																								
							445																						
	- becoming moist at about 5.5 m depth		S8	SS	100																								
								444																					
			S9	SS	100																								
									443																				
			S10	SS	100																								
										442																			
			S11	SS	100																								
											441																		
			S12	SS	100																								
443.1						440																							
8.8	End of Borehole											439																	
							438																						
													437																
								436																					
														435															
									434																				
															433														
										432																			
																431													
											430																		
																	429												
						428																							

EXP RECORD OF BOREHOLE F-17105-AG - ADM-00233185-G0 - MTO 6 - FIVE MILE CREEK - HWY 129 GPJ ONTARIO MOT.GDT 4/13/17

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

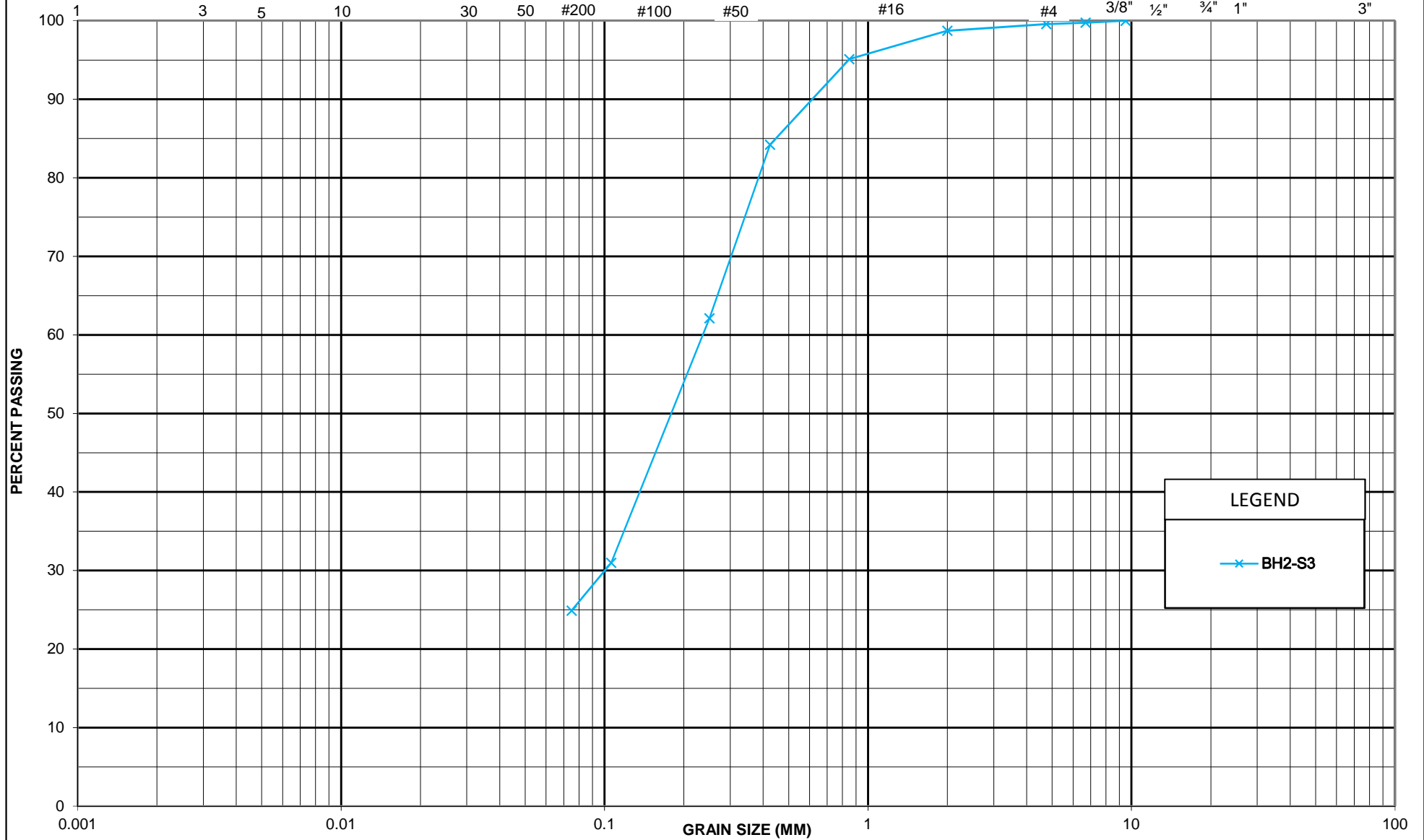
Appendix D – Laboratory Data

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY AND SILT	SAND			GRAVEL	
	Fine	Medium	Coarse	Fine	Coarse

GRAIN SIZE IN MICROMETERS

SIEVE DESIGNATION (Imperial)



LEGEND

—x— BH2-S3



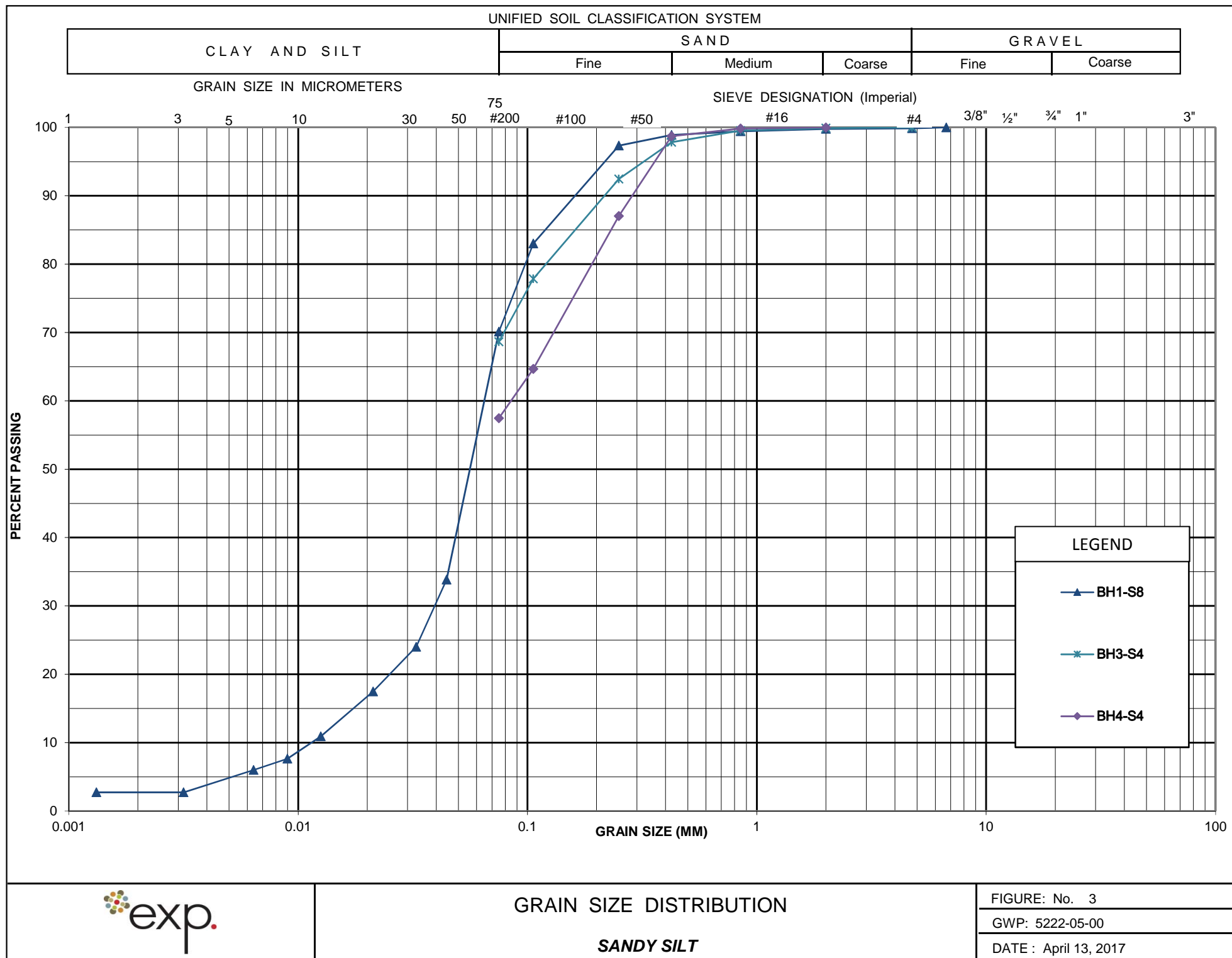
GRAIN SIZE DISTRIBUTION

SILTY SAND FILL

FIGURE: No. 1

GWP: 5222-05-00

DATE : April 13, 2017

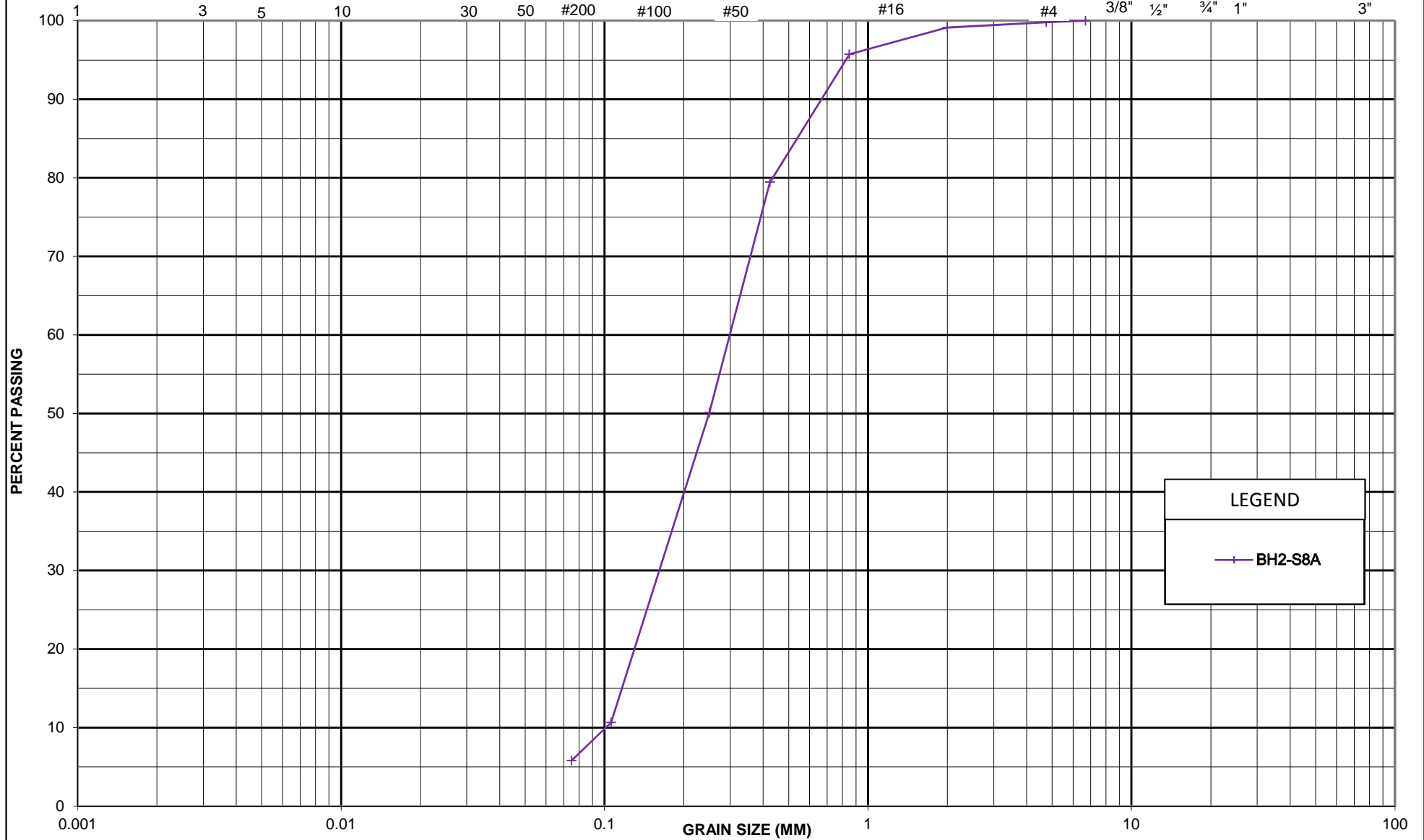


UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY AND SILT	SAND			GRAVEL	
	Fine	Medium	Coarse	Fine	Coarse

GRAIN SIZE IN MICROMETERS

SIEVE DESIGNATION (Imperial)



GRAIN SIZE DISTRIBUTION

Poorly Graded SAND with Silt

FIGURE: No. 4

GWP: 5222-05-00

DATE : April 13, 2017

Appendix E – Borehole Logs from PML Report

EXPLANATION OF TERMS USED IN REPORT

N VALUE: THE STANDARD PENETRATION TEST (SPT) N VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O.D. SPLIT BARREL SAMPLER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5kg, FALLING FREELY A DISTANCE OF 0.76m. FOR PENETRATIONS OF LESS THAN 0.3m N VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N VALUE IS DENOTED THUS \bar{N} .

DYNAMIC CONE PENETRATION TEST: CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60° CONE ANGLE) DRIVEN BY 475 J IMPACT ENERGY ON 'A' SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

COMPOSITION: SECONDARY SOIL COMPONENTS ARE DESCRIBED ON THE BASIS OF PERCENTAGE BY MASS OF THE WHOLE SAMPLE AS FOLLOWS:

PERCENT BY MASS	0 - 10	10 - 20	20 - 30	30 - 40	> 40
	TRACE	SOME	WITH	ADJECTIVE (SILTY)	AND (AND SILT)

CONSISTENCY: COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH (c_u) AS FOLLOWS:

c_u (kPa)	0 - 12	12 - 25	25 - 50	50 - 100	100 - 200	> 200
	VERY SOFT	SOFT	FIRM	STIFF	VERY STIFF	HARD

DENSENESS: COHESIONLESS SOILS ARE DESCRIBED ON THE BASIS OF DENSENESS AS INDICATED BY SPT N VALUES AS FOLLOWS:

N (BLOWS/0.3m)	0 - 5	5 - 10	10 - 30	30 - 50	> 50
	VERY LOOSE	LOOSE	COMPACT	DENSE	VERY DENSE

ROCKS ARE DESCRIBED BY THEIR COMPOSITION AND STRUCTURAL FEATURES AND / OR STRENGTH.

RECOVERY: SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH OF THE CORING RUN.

MODIFIED RECOVERY: SUM OF THOSE INTACT CORE PIECES, 100mm+ IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (RQD), FOR MODIFIED RECOVERY, IS:

RQD (%)	0 - 25	25 - 50	50 - 75	75 - 90	90 - 100
	VERY POOR	POOR	FAIR	GOOD	EXCELLENT

JOINTING AND BEDDING:

SPACING	50mm	50 - 300mm	0.3m - 1m	1m - 3m	> 3m
JOINTING	VERY CLOSE	CLOSE	MOD. CLOSE	WIDE	VERY WIDE
BEDDING	VERY THIN	THIN	MEDIUM	THICK	VERY THICK

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

S S SPLIT SPOON	T P THINWALL PISTON
W S WASH SAMPLE	O S OSTERBERG SAMPLE
S T SLOTTED TUBE SAMPLE	R C ROCK CORE
B S BLOCK SAMPLE	P H T W ADVANCED HYDRAULICALLY
C S CHUNK SAMPLE	P M T W ADVANCED MANUALLY
T W THINWALL OPEN	F S FOIL SAMPLE
F V FIELD VANE	

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
u	1	PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ϵ	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

MECHANICAL PROPERTIES OF SOIL

m_v	kPa ⁻¹	COEFFICIENT OF VOLUME CHANGE
C_c	1	COMPRESSION INDEX
C_s	1	SWELLING INDEX
C_α	1	RATE OF SECONDARY CONSOLIDATION
c_v	m ² /s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{v0}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	-°	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_i	1	SENSITIVITY = $\frac{c_u}{\tau_r}$

PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m ³	DENSITY OF SOLID PARTICLES	n	1, %	POROSITY	e_{max}	1, %	VOID RATIO IN LOOSEST STATE
γ_s	kN/m ³	UNIT WEIGHT OF SOLID PARTICLES	w	1, %	WATER CONTENT	e_{min}	1, %	VOID RATIO IN DENSEST STATE
ρ_w	kg/m ³	DENSITY OF WATER	S_r	%	DEGREE OF SATURATION	I_D	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
γ_w	kN/m ³	UNIT WEIGHT OF WATER	w_L	%	LIQUID LIMIT	D	mm	GRAIN DIAMETER
ρ	kg/m ³	DENSITY OF SOIL	w_p	%	PLASTIC LIMIT	D_n	mm	n PERCENT - DIAMETER
γ	kN/m ³	UNIT WEIGHT OF SOIL	w_s	%	SHRINKAGE LIMIT	C_u	1	UNIFORMITY COEFFICIENT
ρ_d	kg/m ³	DENSITY OF DRY SOIL	I_p	%	PLASTICITY INDEX = $w_L - w_p$	h	m	HYDRAULIC HEAD OR POTENTIAL
γ_d	kN/m ³	UNIT WEIGHT OF DRY SOIL	I_L	1	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$	q	m ³ /s	RATE OF DISCHARGE
ρ_{sat}	kg/m ³	DENSITY OF SATURATED SOIL	I_C	1	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$	v	m/s	DISCHARGE VELOCITY
γ_{sat}	kN/m ³	UNIT WEIGHT OF SATURATED SOIL	DTPL		DRIER THAN PLASTIC LIMIT	i	1	HYDRAULIC GRADIENT
ρ'	kg/m ³	DENSITY OF SUBMERGED SOIL	APL		ABOUT PLASTIC LIMIT	k	m/s	HYDRAULIC CONDUCTIVITY
γ'	kN/m ³	UNIT WEIGHT OF SUBMERGED SOIL	WTP		WETTER THAN PLASTIC LIMIT	j	kN/m ³	SEEPAGE FORCE
e	1, %	VOID RATIO						

RECORD OF BOREHOLE No FM-1

1 of 1

METRIC

G.W.P. 5222-05-00 LOCATION Five Mile Creek Coords: 5 270 083.0 N; 364 136.2 E ORIGINATED BY F.P.
DIST Algoma HWY 129 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY A.K.
DATUM Geodetic DATE January 09, 2015 CHECKED BY M.V.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									WATER CONTENT (%)			GR
451.3	Ground Surface							20	40	60	80	100								
0.0	Snow and ice		1	SS	1	▽* ▼*	451													
450.8	Peat, fine fibrous						450													
0.5	Dark Brown		2	SS	1															
			3	SS	1															
449.2	Sandy silt, trace clay						449													
2.1	Compact Grey Wet		4	SS	12														0 49 50 1	
448.3	Silty sand some gravel, trace clay						448													
3.0	Very loose Grey Moist to loose to wet		5	SS	10															
			6	SS	9		447											16 40 40 4		
			7	SS	3															
	Very dense					446														
			8	SS	67													11 51 31 7		
444.9	End of borehole		9	SS	50/8cm	445														
6.4	End of borehole																			
	Sample 9: Sampler bouncing																			
	* 2015 01 09																			
	▽ Water level observed during drilling																			
	▼ Water level measured after drilling																			
	NOTE: Borehole caved in at 2.7m																			

RECORD OF BOREHOLE No FM-2

1 of 1

METRIC

G.W.P. 5222-05-00 LOCATION Five Mile Creek Coords: 5 270 077.0 N; 364 148.7 E ORIGINATED BY F.P.

DIST Algoma HWY 129 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY A.K.

DATUM Geodetic DATE December 15, 2014 CHECKED BY M.V.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								○ UNCONFINED		+ FIELD VANE		× LAB VANE								
453.8	Ground Surface						20	40	60	80	100									
0.0	230mm asphalt over sand and gravel		1	AS																
453.0	(PAVEMENT FILL)																			
0.8	Silty sand, trace clay organics					453														
	Loose to Grey Wet very loose		2	SS	10	452														
	(FILL)																			
			3	SS	6	451										0 80 19 1				
			4	SS	1															
						450														
449.5			5	SS	5															
4.3	Sandy silt trace clay, trace gravel																			
	Compact to Grey Wet very loose		6	SS	28	449														
			7	SS	16	448										1 13 83 3				
			8	SS	1															
447.1						447														
6.7	Silty sand some gravel, trace clay																			
	Compact to Grey Moist very dense		9	SS	24											18 44 30 8				
445.9			10	SS	43/8cm	446														
7.9	End of borehole																			
	Sample 10: Sampler bouncing																			
	* 2014 12 15																			
	▽ Water level observed during drilling																			
	▼ Water level measure after drilling																			

RECORD OF BOREHOLE No FM-3

1 of 1

METRIC

G.W.P. 5222-05-00 LOCATION Five Mile Creek Coords: 5 270 087.3 N; 364 152.5 E ORIGINATED BY F.P.
DIST Algoma HWY 129 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY A.K.
DATUM Geodetic DATE December 15, 2014 CHECKED BY M.V.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								○ UNCONFINED + FIELD VANE										○		
								● QUICK TRIAXIAL × LAB VANE												
453.7	Ground Surface						20	40	60	80	100									
0.0	200mm asphalt over																			
453.4	sand and gravel		1	AS	50/5cm															
0.3	(PAVEMENT FILL)																			
	Silty sand, trace clay																			
	organics																			
	Loose to Brown Wet		2	AS	9															
	very dense to grey																			
			3	SS	11															
		4	SS	12																
		5	SS	4																
	(FILL)	6	SS	9																
448.5	Sandy silt																			
5.2	trace clay, trace gravel		7	SS	5															
	Loose to Grey Moist																			
	very loose																			
			8	SS	1															
447.0	Silty sand																			
6.7	some gravel, trace clay																			
	Compact to Grey Moist		9	SS	21															
	very dense																			
		10	SS	108																
445.6	End of borehole																			
8.1	Sample 1: Sampler bouncing																			
	* 2014 12 15																			
	Water level observed during drilling																			
	Water level measured after drilling																			

RECORD OF BOREHOLE No FM-4

1 of 1

METRIC

G.W.P. 5222-05-00 LOCATION Five Mile Creek Coords: 5 270 079.6 N; 364 162.5 E ORIGINATED BY F.P.
DIST Algoma HWY 129 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY A.K.
DATUM Geodetic DATE January 08, 2015 CHECKED BY M.V.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								20 40 60 80 100										20 40 60		
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE												
452.1	Ground Surface					▽*	452													
451.9	Topsoil																			
0.2	Silty sand, trace gravel organics		1	SS	6															
	Very loose Brown Wet to loose (FILL)		2	SS	3															
450.7	Peat, amorphous sand seams																			
1.4	Dark brown		3	SS	3															
			4	SS	1															
448.6	Sandy silt trace clay, trace gravel		5	SS	6															
3.5	Loose to Grey Moist very loose to wet		6	SS	5															
			7	SS	1															
446.6	Silty sand some gravel, trace clay		8	SS	25															
5.5	Compact Grey Moist to dense																			
445.4	End of borehole		9	SS	41															
6.7																				
													</							

* 2015 01 08

▽* Water level observed during drilling

▽ Water level measured after drilling

NOTE: Borehole caved in at 2.7m

Appendix F – Slope Stability Analyses

Five Mile Creek Culvert Replacement on Hwy 129
Chapleau
West side of Embankment (Outlet)
Drained Static Condition

Name: Peat (Very Soft)	Model: Mohr-Coulomb	Unit Weight: 12 kN/m ³	Cohesion: 2 kPa	Phi: 15 °
Name: Silty Sand (Loose to Compact)	Model: Mohr-Coulomb	Unit Weight: 20 kN/m ³	Cohesion: 0 kPa	Phi: 29 °
Name: Sandy Silt (Loose to Compact)	Model: Mohr-Coulomb	Unit Weight: 19 kN/m ³	Cohesion: 0 kPa	Phi: 27 °
Name: Silty Sand Till (Compact to Very Dense)	Model: Mohr-Coulomb	Unit Weight: 21 kN/m ³	Cohesion: 0 kPa	Phi: 32 °
Name: Engineered Fill	Model: Mohr-Coulomb	Unit Weight: 21 kN/m ³	Cohesion: 0 kPa	Phi: 32 °

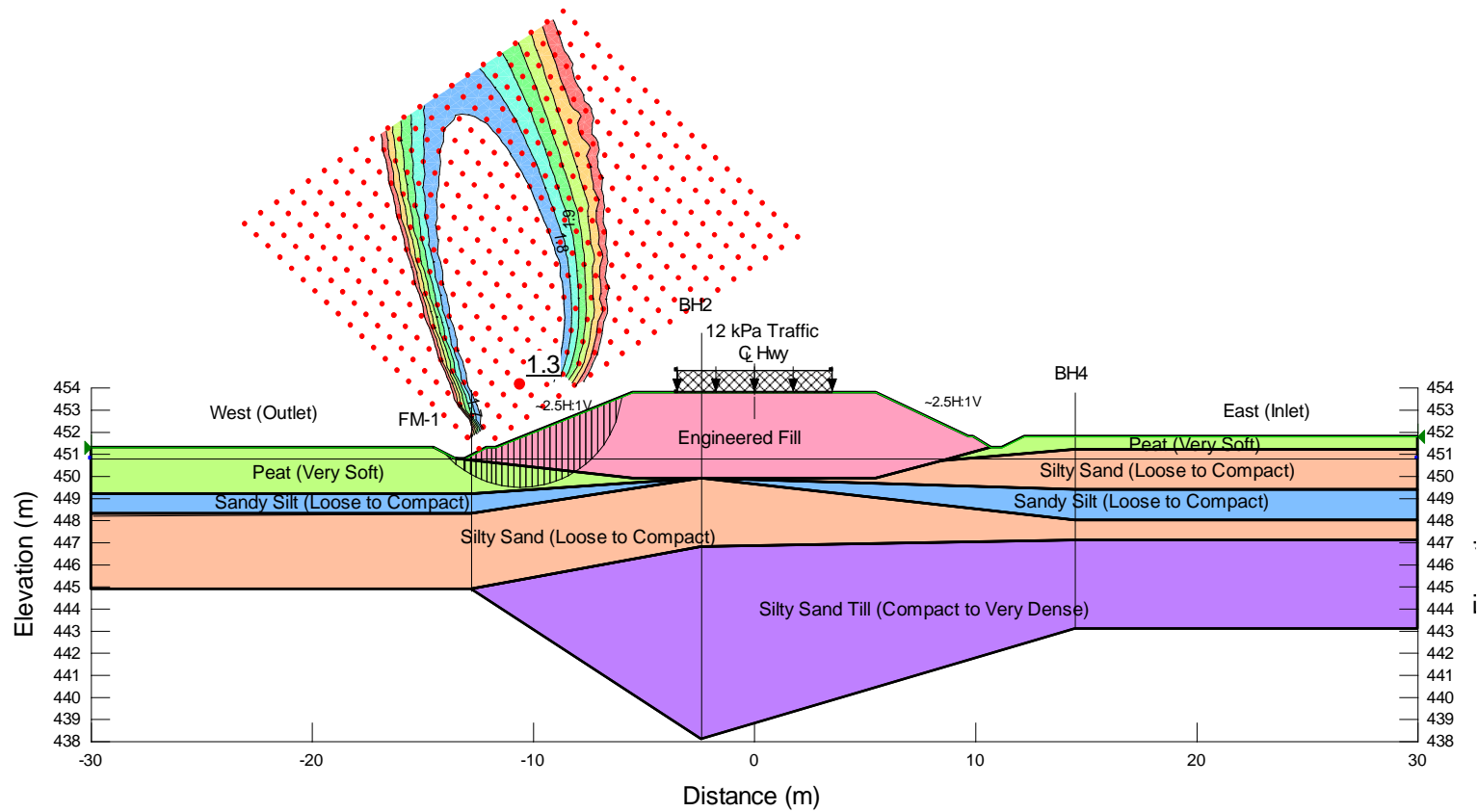
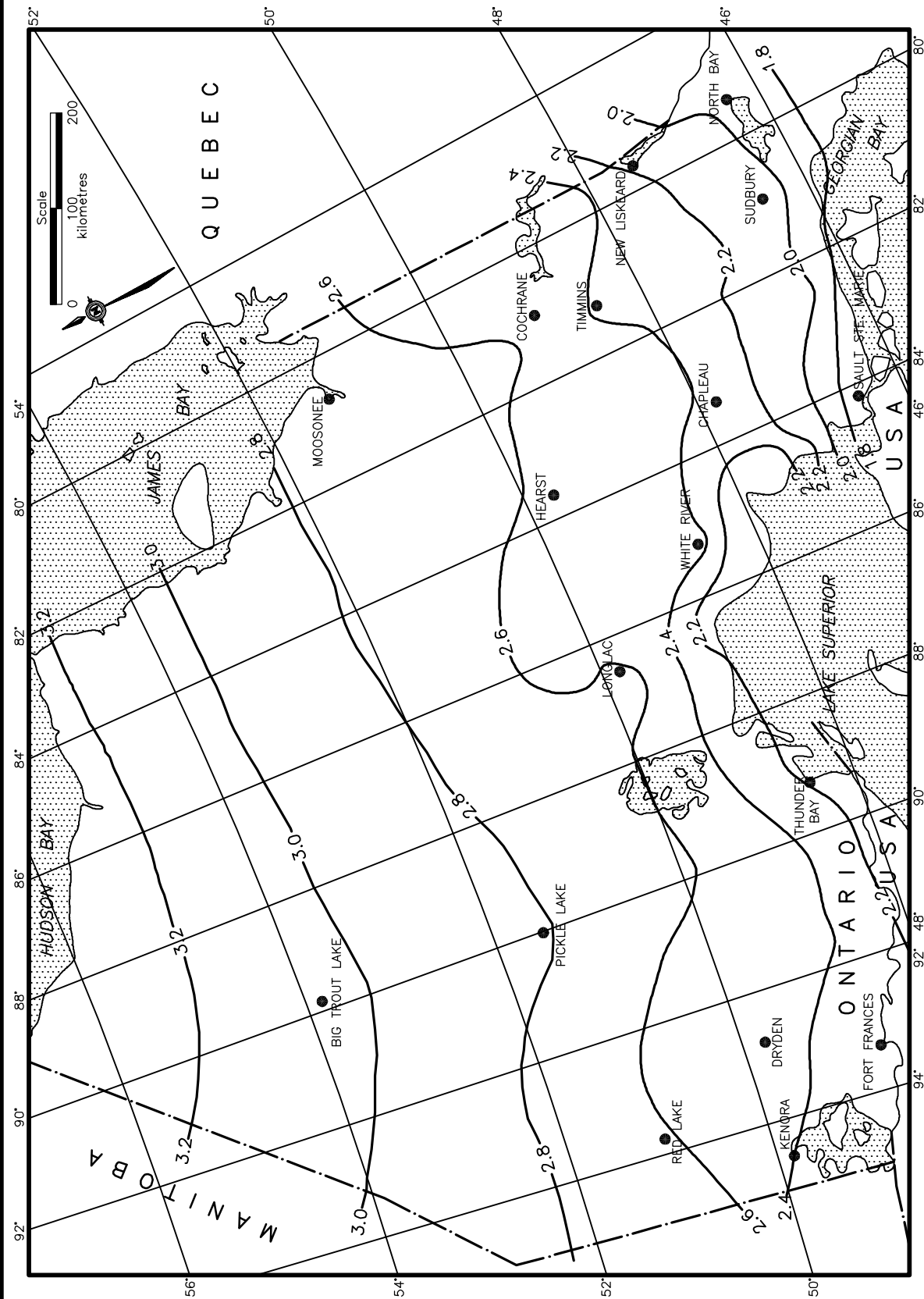


Figure 1: Slope stability analysis – Drained static conditions for west side of embankment

Appendix G – Ontario Provincial Standard Drawings



NOTES:

- A These values are approximate and should only be used where the recommendations of a geotechnical engineer are not available.
- B This information is based on the Ministry of Transportation and Communications Research Publication RR225 "Aspects of Prolonged Exposure of Pavements to Sub-Zero Temperatures" dated December 1981.
- C Values between contours should be interpolated. If interpolation is not possible, use the adjacent contour with the greater depth.
- D Frost penetration depths are in metres.

ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2010

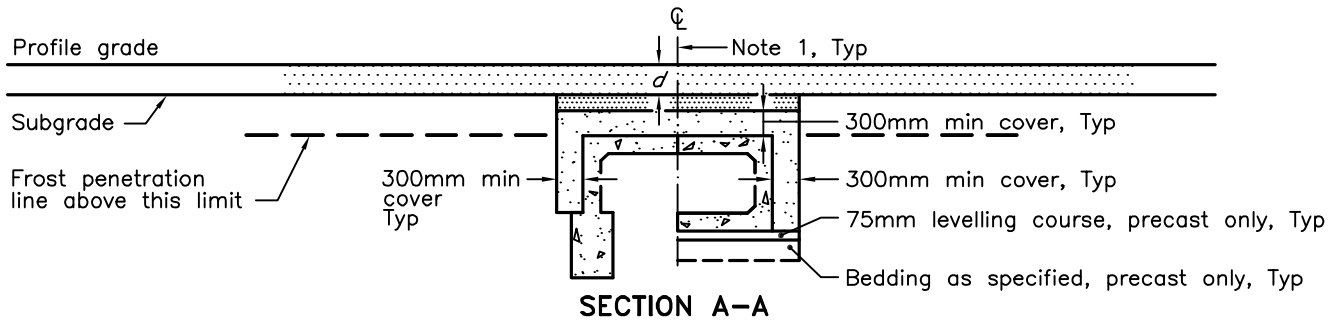
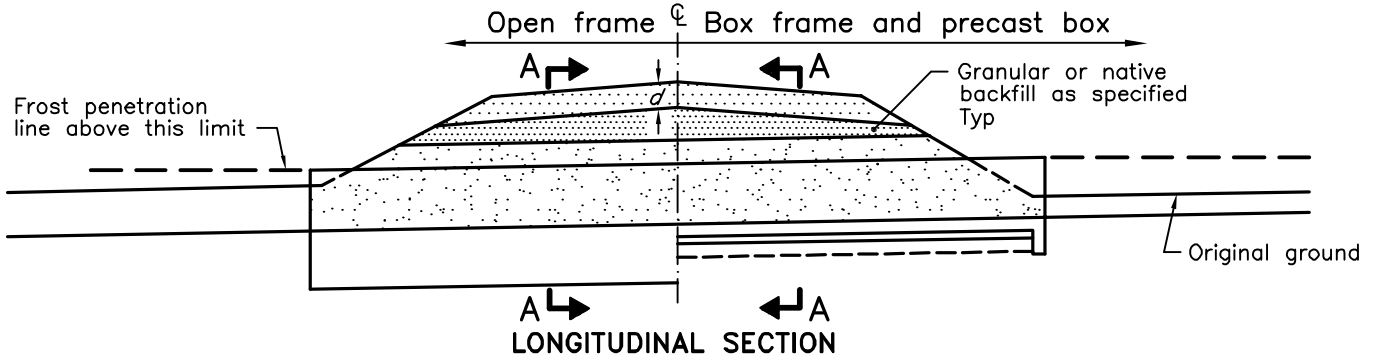
Rev 1

FOUNDATION FROST PENETRATION DEPTHS FOR NORTHERN ONTARIO

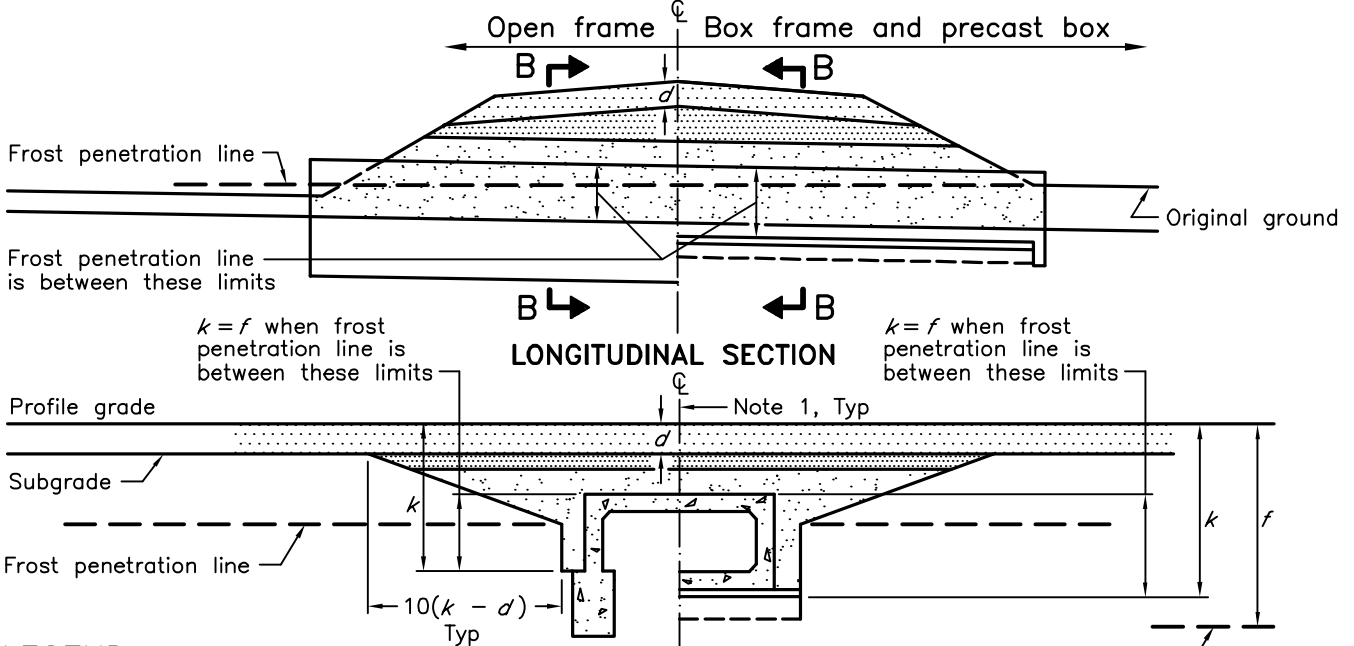
OPSD 3090.100



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.

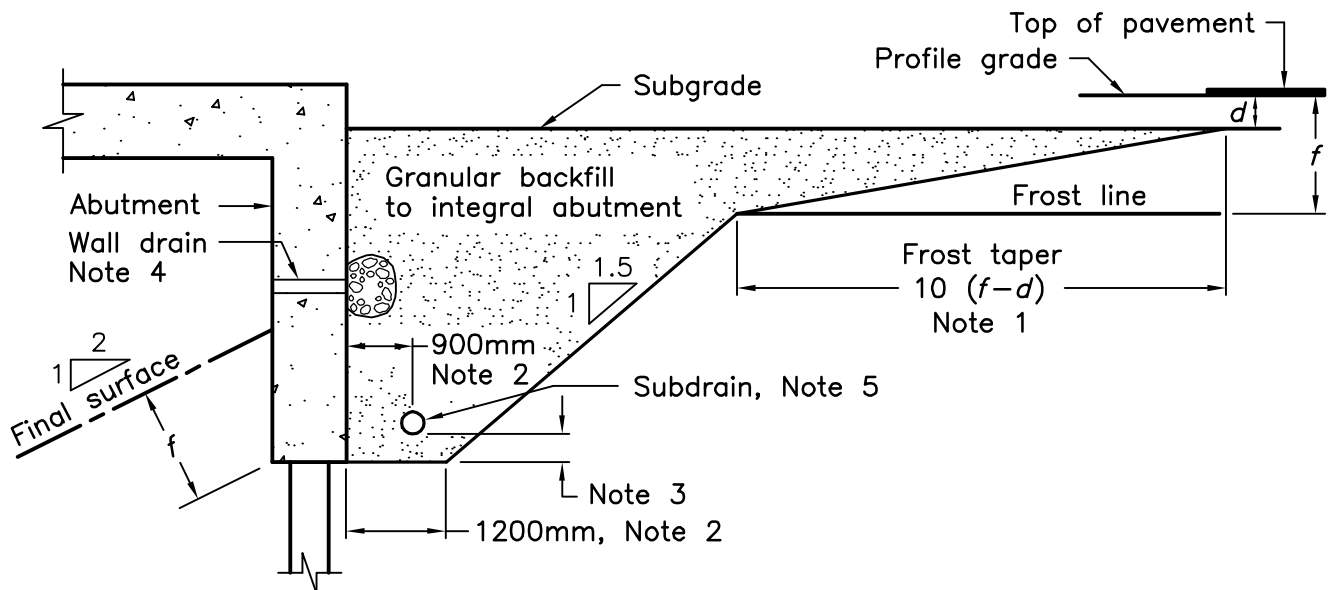
ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2010 Rev 2

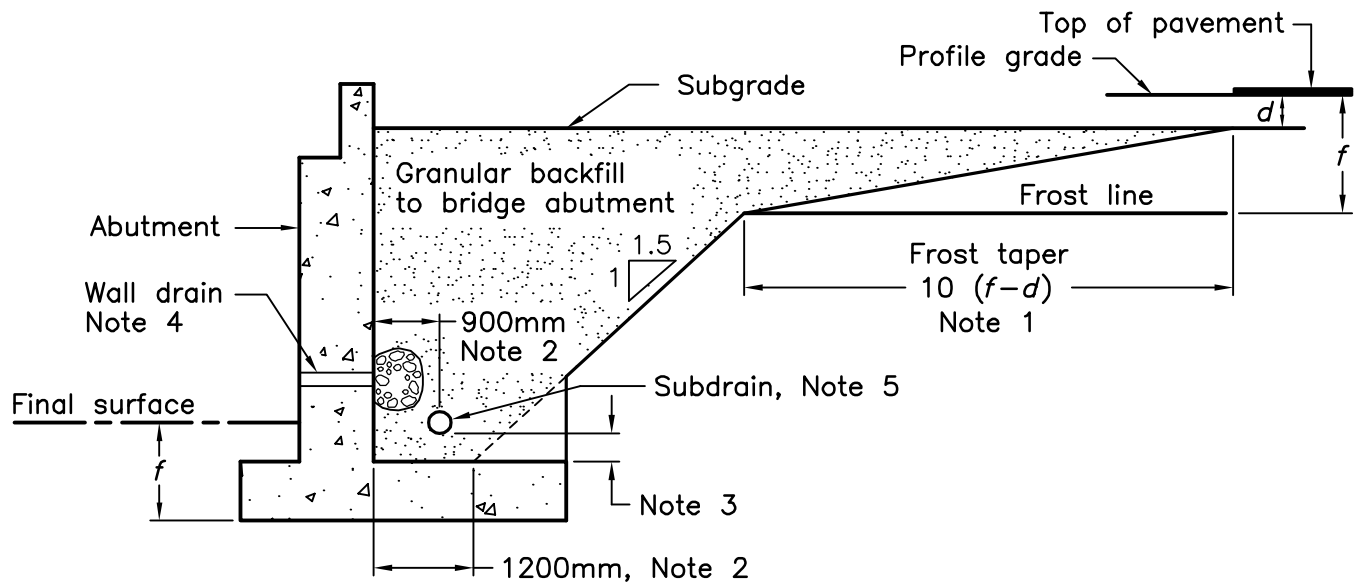
**BACKFILL AND COVER FOR
CONCRETE CULVERTS WITH SPANS
LESS THAN OR EQUAL TO 3.0M**

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

- A drainage system behind a retaining structure should ensure that a groundwater table does not exist above the footing level. Preferably, the ground water level is controlled by the use of free-draining granular backfill and a collection system such as weep holes or perforated drains at the footing level. These weep holes and drains should be inspected and maintained to ensure that they do not become blocked. If free-draining, granular backfill is not employed, the permeability of the backfill and the hydrostatic head will control the extent to which the groundwater table can be depressed locally by seepage towards a footing drain. In practice, design for frost protection is best done using free-draining backfill.

The design should also consider the risk of unusually large inflows of water creating a temporary hydrostatic head of water behind the wall. An example is the overtopping of a retaining wall, adjoining a large body of water, by storm waves. Measures such as the use of quarried rock backfill, design for full hydrostatic pressure, or provision of a sloped impermeable surface layer should be considered.

Measurements have shown that earth pressures can vary seasonally, but the effects have normally been neglected in design, except for winter frost pressures. These latter can be very large if the backfill is frost susceptible and for this reason free-draining granular backfill is recommended.
- Figure C6.20 shows examples of minimum backfill requirements.

The distance, x , should be equal to or greater than the estimated vertical frost penetration. This distance may be reduced if the wall abuts a vertical face of bedrock that is not susceptible to frost. The frost penetration may be reduced by the use of suitable insulation, in which case a thermal analysis should be performed by a Geotechnical Engineer.

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

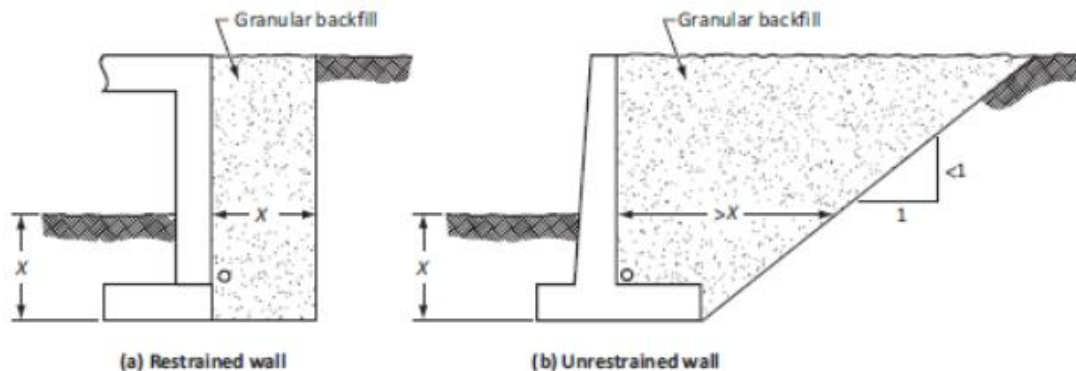
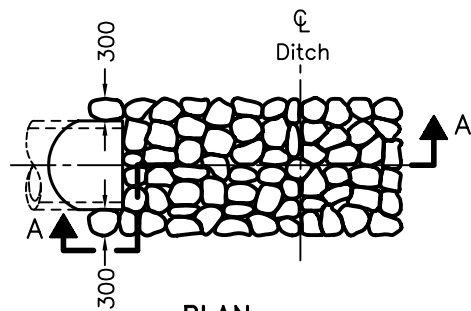


Figure C6.20
Backfill for frost protection
 (See Clause C6.12.1.)

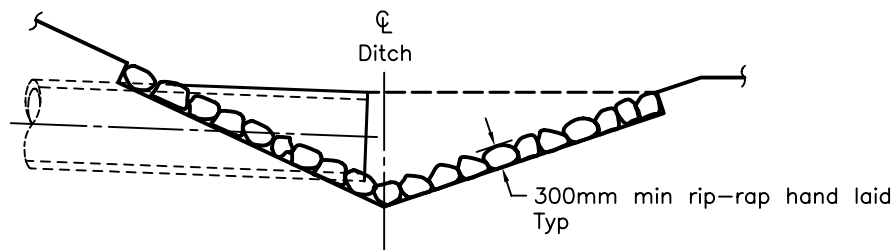
C6.12.2 Lateral ground pressures

C6.12.2.1 General

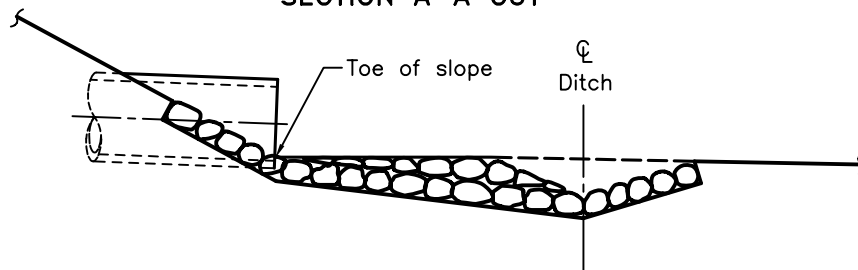
Earth pressure acting on a structure depends on the relative movement of the structure, the backfill, the type of soil adjacent to the backfill, and the soil below the footing or supporting piles. Appropriate geotechnical parameters should be chosen for the calculation of lateral pressures based on recognized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



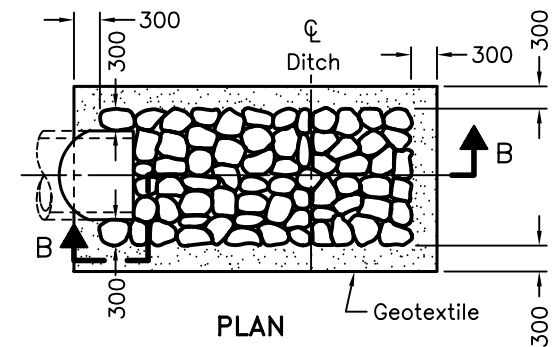
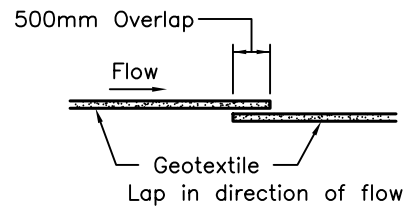
PLAN
CUT OR FILL



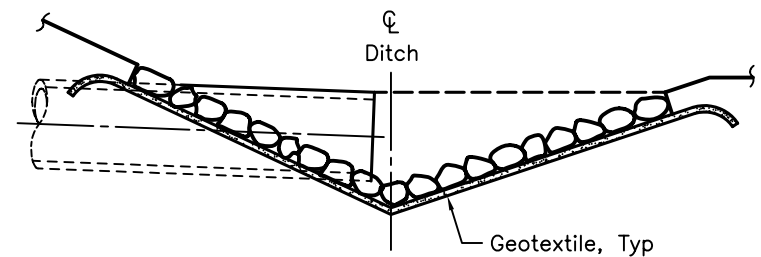
SECTION A-A CUT



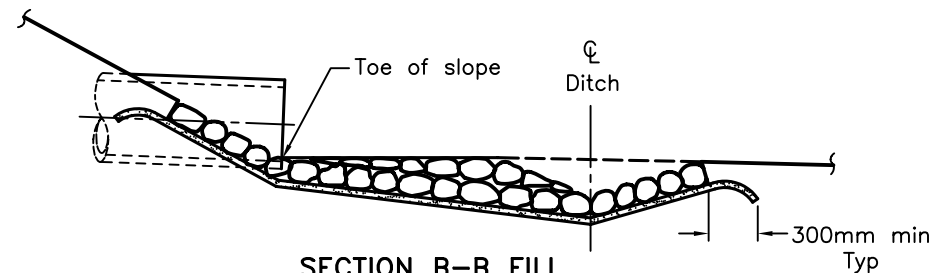
SECTION A-A FILL
TYPE A – WITHOUT GEOTEXTILE



PLAN
CUT OR FILL



SECTION B-B CUT



SECTION B-B FILL
TYPE B – WITH GEOTEXTILE

NOTES:

A All dimensions are in millimetres unless otherwise shown.

ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2001

Rev 0

RIP-RAP TREATMENT
FOR SEWER AND CULVERT OUTLETS

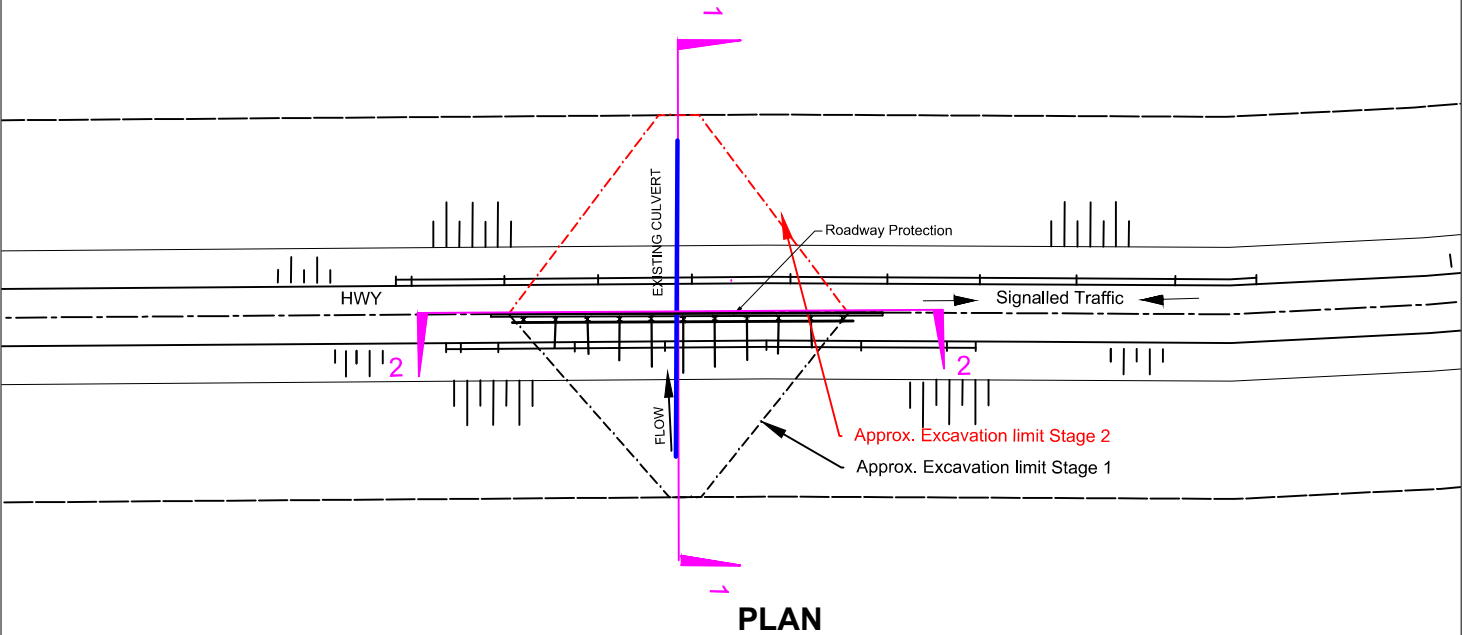


OPSD – 810.010

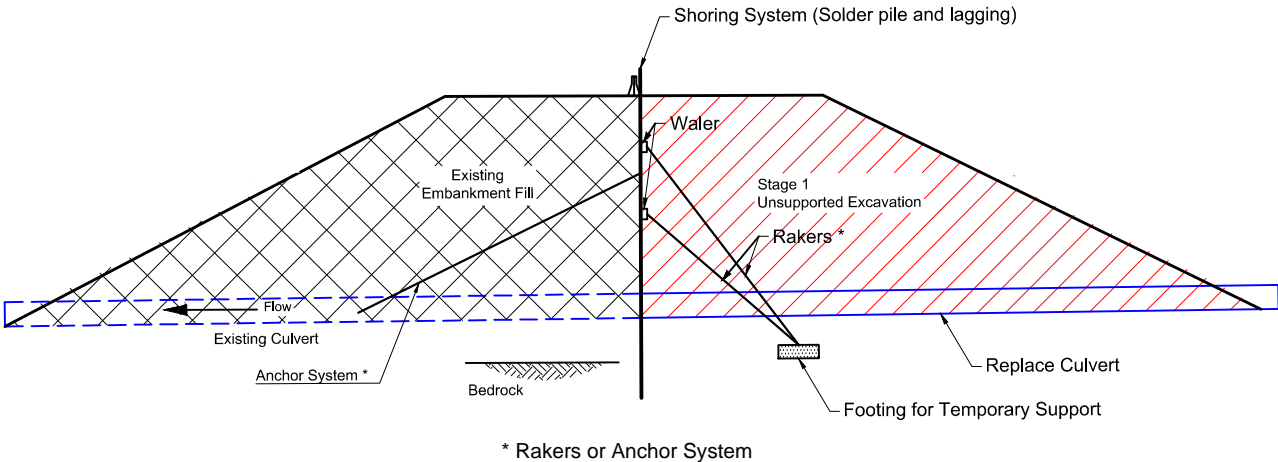
Appendix H – Schematic Sketches for Construction Staging

FIGURE H.1.A: HALF AND HALF CONSTRUCTION WITH UNSUPPORTED CUT SIDES

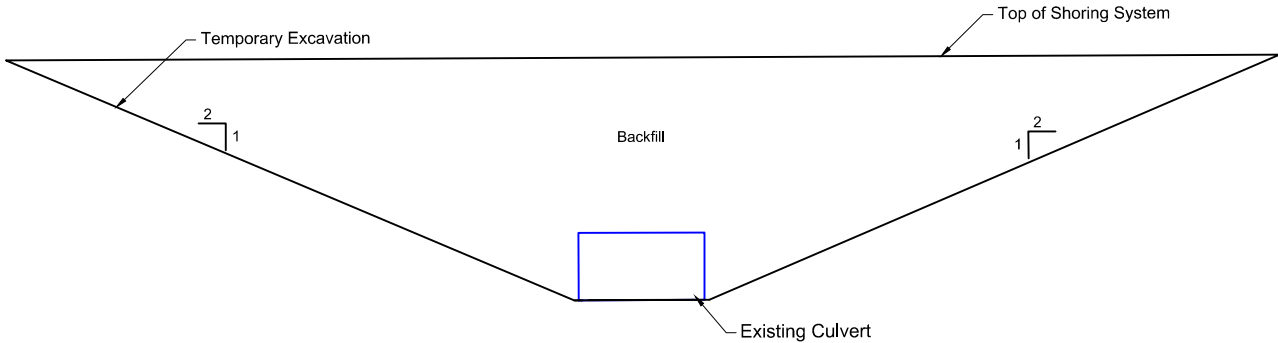
SCHEMATIC DIAGRAMS (NTS)



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

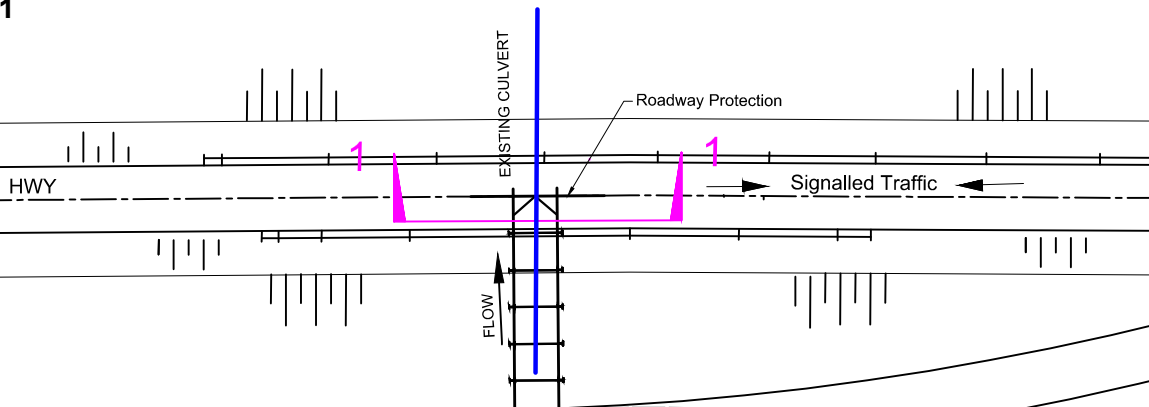


* Rakers or Anchor System

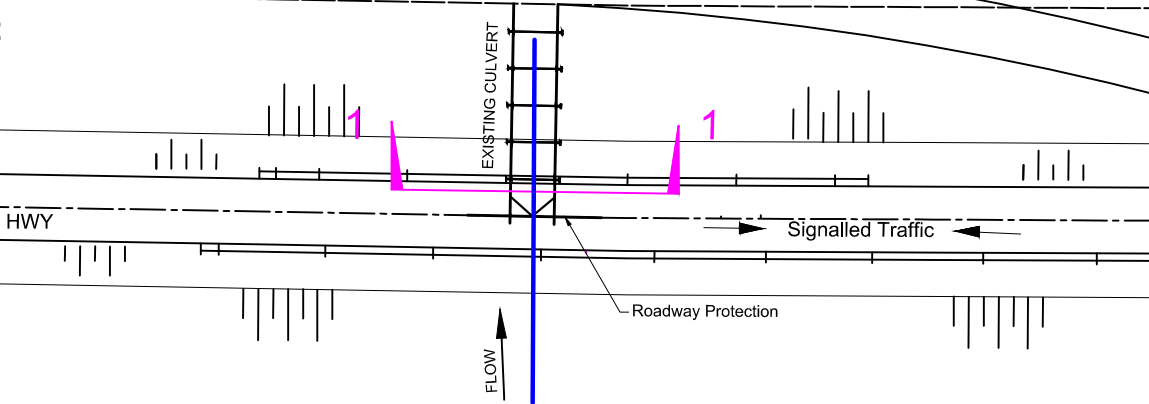


**FIGURE H.1.B: HALF AND HALF CONSTRUCTION WITH BRACED CUT SIDES
OR ANCHOR SYSTEM OPTION 3.B
SCHEMATIC DIAGRAMS (NTS)**

Stage 1

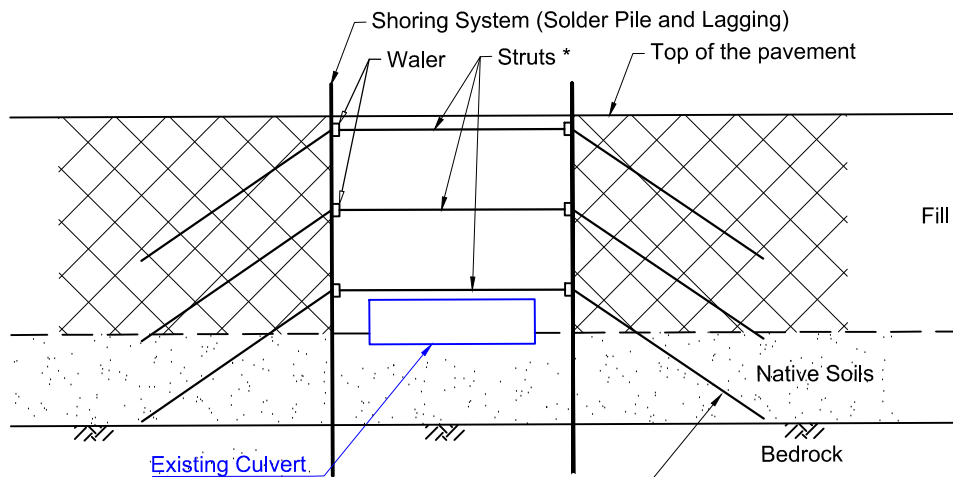


Stage 2



PLAN

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

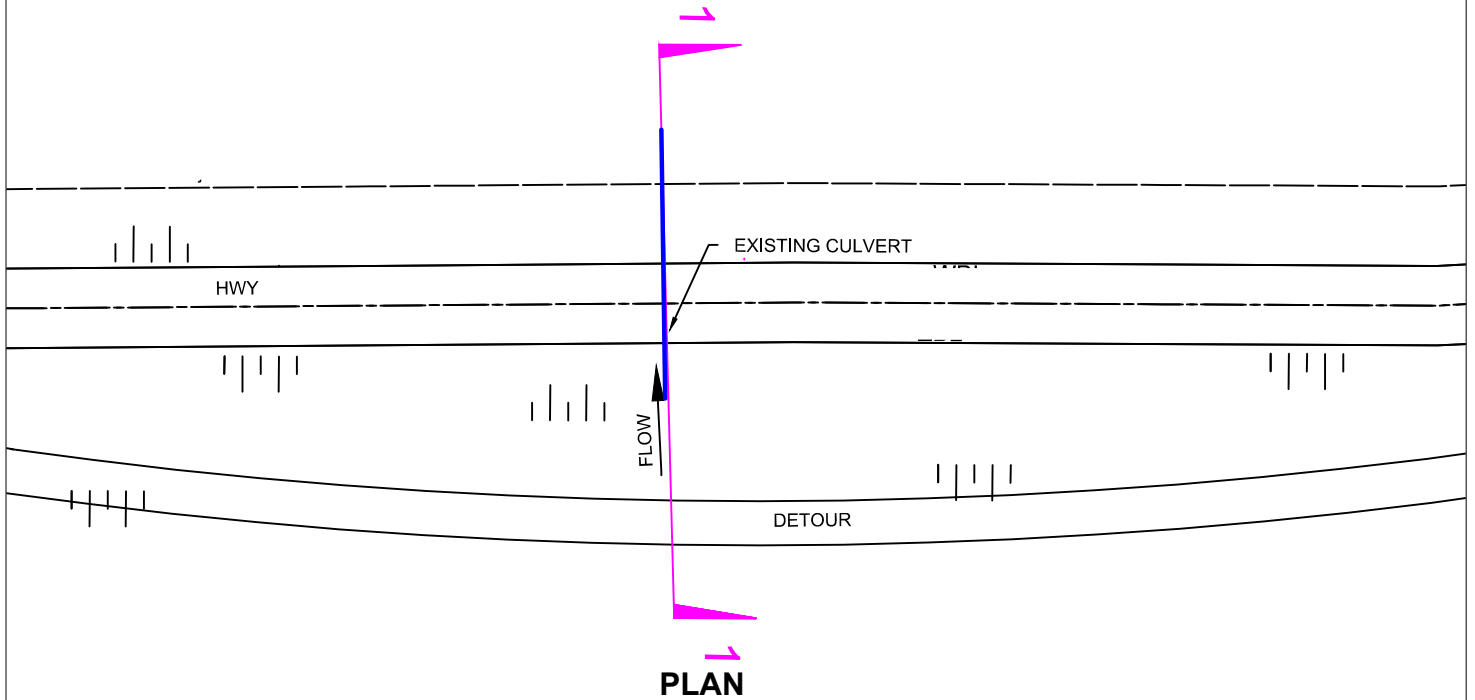


* Struts or Anchor System

SECTION 1-1

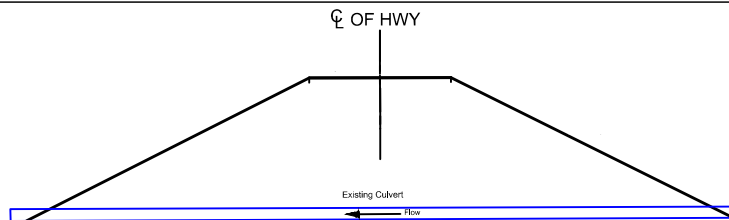
FIGURE H.2: TEMPORARY LOCAL DETOUR AND OPEN CUT UNSUPPORTED EXCAVATION (OPTION 2)

SCHEMATIC DIAGRAMS (NTS)

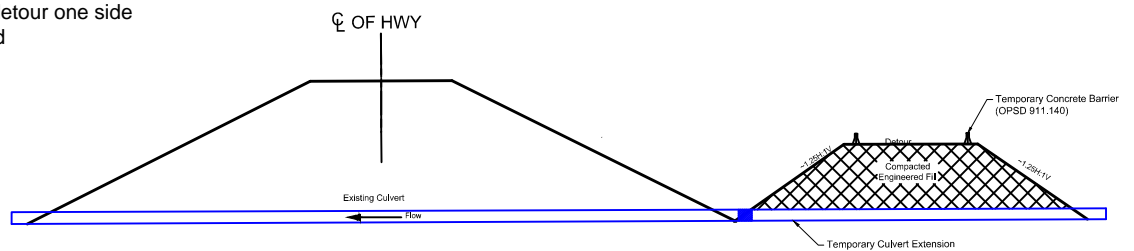


RECOMMENDED STAGES

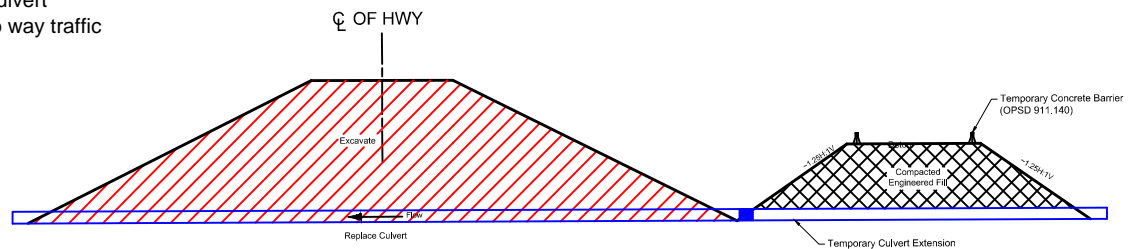
1.0 Stage 1 - Current condition



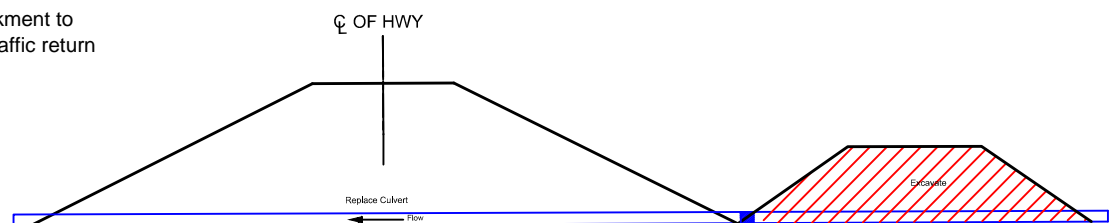
2.0 Stage 2 - Build temporary detour one side
Two-way traffic on existing road



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

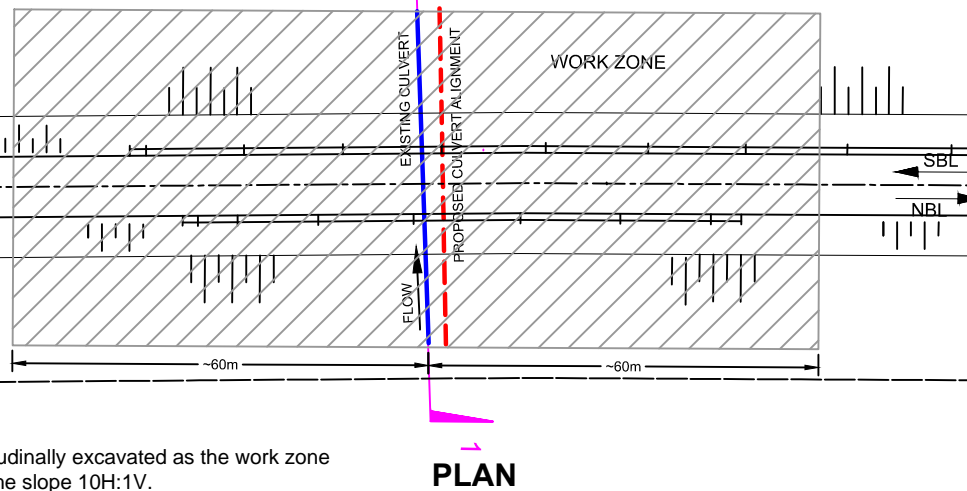


4.0 Stage 4 - Build the embankment to existing alignment; Two-way traffic return



SECTION 1-1

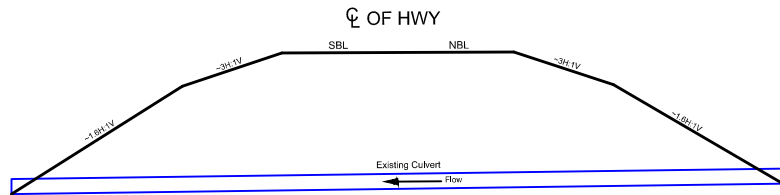
**FIGURE H.3: STAGE CONSTRUCTION BY GRADE LOWERING
SCHEMATIC DIAGRAMS (NST)**



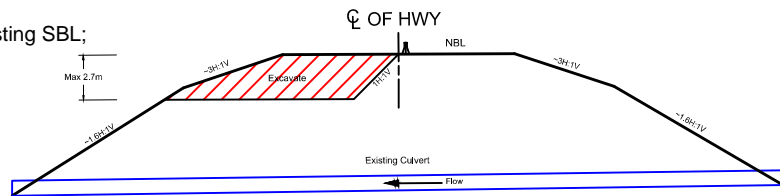
Note:
Highway should be longitudinally excavated as the work zone shows in the plan using the slope 10H:1V.

RECOMMENDED STAGES

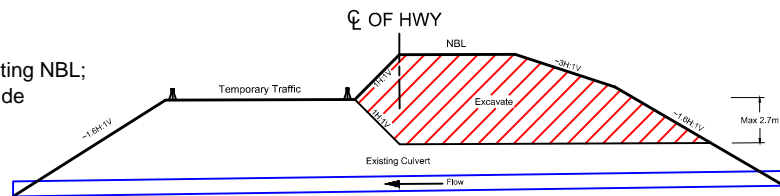
1.0 Stage 1 - Current condition



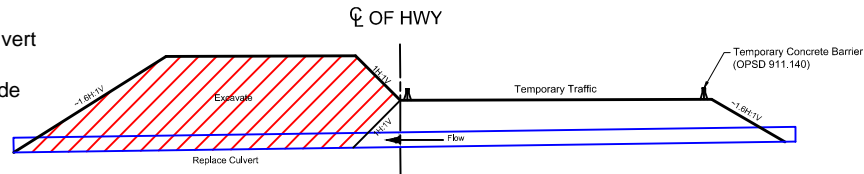
2.0 Stage 2 - Excavation on existing SBL;
One-way traffic on existing road



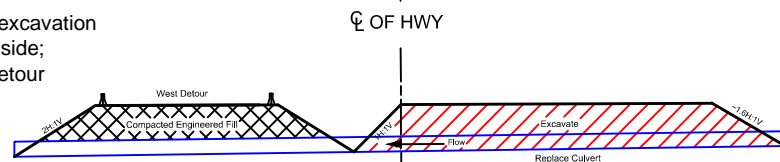
3.0 Stage 3 - Excavation on existing NBL;
One-way traffic shifted to west side



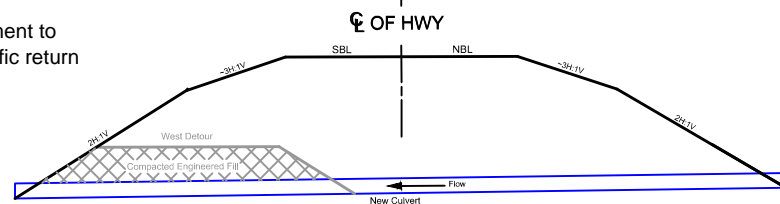
4.0 Stage 4 - Excavation and culvert construction on west side;
One-way traffic shifted to east side



5.0 Stage 5 - Build west detour, excavation and culvert construction on east side;
One-way traffic shifted to west detour



6.0 Stage 6 - Build the embankment to existing alignment; Two-way traffic return



SECTION 1-1

Appendix I – Non - Standard Special Provision (NSSP)

NSSP FOR COBBLES AND/OR BOULDERS OBSTRUCTIONS

Scope of Work

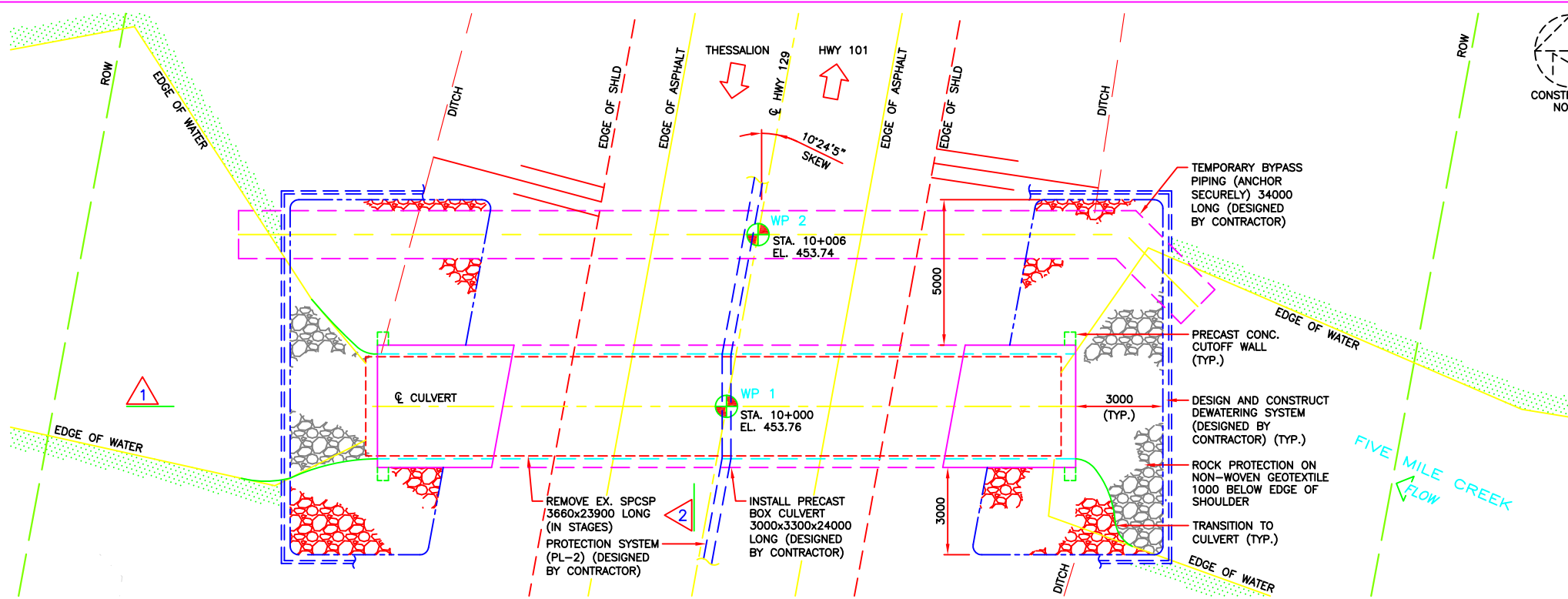
The Contractor should be aware that the cobbles and/or boulders may be encountered during the installation of shoring elements and during excavation of the embankment fill. Appropriate equipment and procedures will be required to penetrate/remove cobbles and/or boulders that may encountered during installation of shoring and excavation.

Basis of Payment

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

Appendix J – Five Mile Creek Culvert General Arrangement

DRAWING FILE: C:\Users\User\Documents\2016 DWGS\1470330\Aug. 14, 2015 CAD QA\60333079-P30
DATE: 2/9/2016 2:19:50 PM

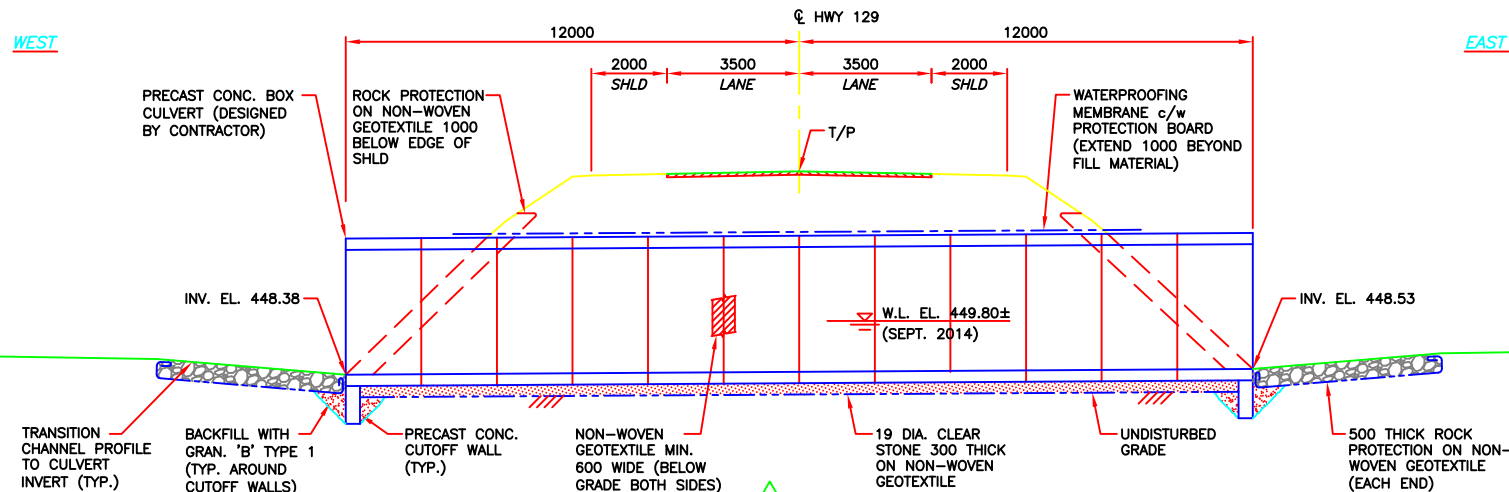


PLAN
1 : 200

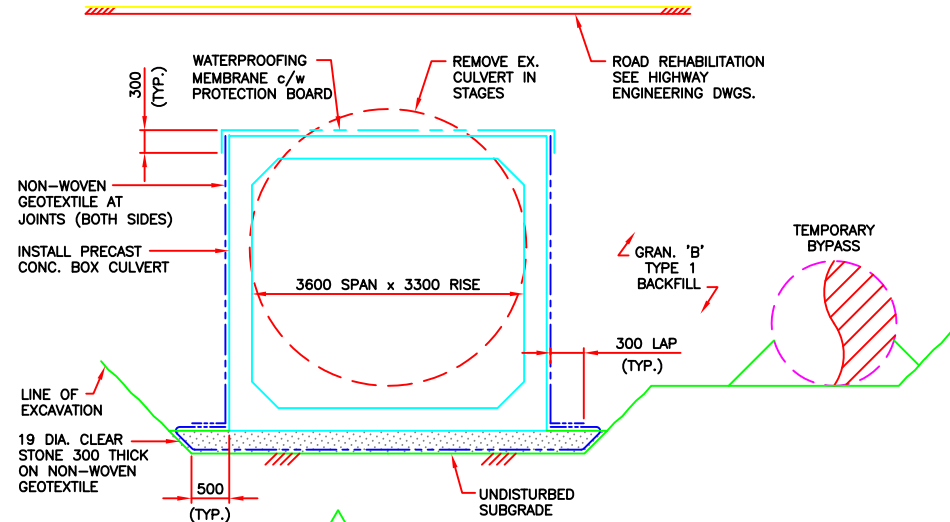
WEST

EAST

EL. 454.00
EL. 453.00
EL. 452.00
EL. 451.00
EL. 450.00
EL. 449.00
EL. 448.00



SECTION 1
1 : 200



SECTION 2
1 : 100



METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

HWY 129
CONT No
WP No 5227-05-01



FIVE MILE CREEK CULVERT
STA. 10+000, REANEY TWP.
GENERAL ARRANGEMENT

SHEET

AECOM

GENERAL NOTES :

1. CLASS OF CONCRETE : PRECAST 40 MPa
2. CLEAR COVER TO REINFORCING STEEL : PRECAST 50 ± 10
3. REINFORCING STEEL :
 1. REINFORCING STEEL SHALL BE GRADE 400W UNLESS OTHERWISE SPECIFIED.
 2. UNLESS SHOWN OTHERWISE, TENSION LAP SPLICES FOR REINFORCING STEEL SHALL BE CLASS B.
 3. BAR HOOKS SHALL HAVE STANDARD HOOK DIMENSIONS USING MINIMUM BEND DIAMETERS, WHILE STIRRUPS AND TIES SHALL HAVE MINIMUM HOOK DIMENSIONS. ALL HOOKS SHALL BE IN ACCORDANCE WITH THE STRUCTURAL STANDARD DRAWING SS12-1, UNLESS INDICATED OTHERWISE.
4. GEOTEXTILE :
 1. NON-WOVEN, CLASS II, FOS 50 TO 100µm.

CONSTRUCTION NOTES :

1. THE CONTRACTOR SHALL VERIFY ALL DIMENSIONS OF THE EXISTING WORK AND ALL DETAILS ON SITE AND REPORT ANY DISCREPANCIES TO THE CONTRACT ADMINISTRATOR BEFORE PROCEEDING WITH THE WORK.
2. THE CONTRACTOR SHALL CARRY OUT SITE SURVEYS TO DETERMINE THE EXISTING ELEVATIONS OF ASPHALT PRIOR TO REMOVALS.
3. BACKFILL SHALL BE PLACED SIMULTANEOUSLY BEHIND BOTH CULVERT WALLS, KEEPING THE HEIGHT OF BACKFILL APPROXIMATELY THE SAME, AT NO TIME SHALL THE DIFFERENCE IN BACKFILL HEIGHTS BE GREATER THEN 200mm.
4. ALL SITE ACCESS TO COMPLETE THE WORK IS THE RESPONSIBILITY OF THE CONTRACTOR.

APPLICABLE STANDARD DRAWINGS :

OPSD 3941.200 FIGURES IN CONCRETE, SITE NUMBER, AND DATE, LAYOUT

LIST OF ABBREVIATIONS :

CL	CENTRELINE	SBGR	STEEL BEAM GUIDE RAIL
CONC.	CONCRETE		
c/w	COMPLETE WITH	SPCSP	STRUCTURAL PLATE CORRUGATED STEEL PIPE
DIA.	DIAMETER		
DWG.	DRAWING		
EL.	ELEVATION (METRES)	STA	STATION
EX.	EXISTING	TYP.	TYPICAL
MIN.	MINIMUM	T/P	TOP OF PAVEMENT
ROW	RIGHT OF WAY	W.L.	WATER LEVEL
SHLD	SHOULDER		

DRAWING NOT TO BE SCALED
50 mm ON ORIGINAL DRAWING

REVISIONS	DATE	BY	DESCRIPTION
DESIGN	J.P.	CHK G.M.	CODE CHBDC 2006 LOAD CL.-625-ONT
DRAWN	T.G.	CHK J.P.	SITE 46-006/C STRUCT SCHEME DWG P1