



FINAL
FOUNDATION INVESTIGATION AND DESIGN REPORT
TRAP CREEK CULVERT REPLACEMENT
HIGHWAY 129
SAULT STE MARIE DISTRICT
AGREEMENT NO.: 5016-E-0001
SITE NO.: 46-333
GEOCRES NO. 410-19
GWP: 5231-05-01

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PART 1: FACTUAL INFORMATION

1. INTRODUCTION

DST Consulting Engineers Inc. (DST) has been retained by the Ministry of Transportation (the Ministry), to conduct a foundation investigation and provide a foundation design report for the proposed culvert replacement on Highway 129. This work was carried out under Agreement No.: 5016-E-0001, Pavement Engineering, Foundation Engineering and Engineering Materials Testing and Evaluation. This report addresses the field investigation, laboratory test program, factual report on conditions (Part 1) and recommendations for design and construction for the proposed culvert replacement (Part 2).

2. SITE DESCRIPTION

The site is located on Highway 129, approximately 0.05 km South of the South Junction of Highway 101 (latitude 47.74871, longitude -83.38767), LHRS 62920, offset 1.080, Station 11+775 (Township of Chappise) in the District of Sault Ste Marie.

The highway runs in a north-west and south-east direction. The culverts are aligned across the highway at an angle of approximately 45°. A secondary road, Sheppard Morse Road, crosses Highway 129 directly over the twin culverts location. The local topography is gentle and subdued, and the surrounding valley and slopes are densely vegetated with natural boreal forest cover. The site photographs presented in Figures 2.1 and 2.2 were taken by DST field staff during the fieldwork investigation.

It is understood that each of the existing 50.0 m long corrugated steel twin culvert pipes is approximately 2.7 m in diameter. The existing culverts (Figure 2.1 to 2.2) built in 1981 as stated in the Ontario Structure Inspection Manual (OSIM), and inspection by others indicates light corrosion, minor sagging of the barrel under the roadway of both barrels, a missing bolt in the North barrel at the East and West ends, deformation of the North barrel at outlet, improperly installed bolts, improper bolt layout, and active leakage at bolts.

The embankment height at the culvert location is approximately 2.3 m and the side slopes of the embankment are approximately 2H: 1V.

Two sand and gravel pits are indicated in close proximity to the site (Ontario West Central Sheet Map 2542), and these can be seen on aerial photo images approximately 350m east and 500m north respectively.



Figure 2.1 View of the existing culverts at Highway 129 (looking West Outlet)



Figure 2.2 View of the existing culverts at Highway 129 (looking East Inlet)

3. REGIONAL GEOLOGY

Geological information is available from the published *Ontario Geological Survey Map #410NW* by the *Ontario Ministry of Natural Resources* for the Chapleau area. The map indicates that the local terrain unit is identified as Eolian deposits Sand Dunes (sED/pOT)

Eolian deposits (ED) of fine sand (s) occur extensively in the central portion of the map-area. The wind-blown deposits are derived mainly from glaciofluvial sand and occur as U-shaped dunes, small hills, and ridges. The deposits generally form a discontinuous blanket over other terrain units, most often outwash (GO), but also hummocky moraine (MH) and organic deposits (OT). Local relief in dune fields ranges from 2 to 5 m, which is also the approximate thickness of these deposits. The largest dune field straddles the Woman River in the center of the map-area. Here, U-shaped dunes are all stabilized by vegetation and most are oriented in a southeasterly direction, indicating a prevailing northwest wind at the time of dune formation (assuming a parabolic origin).

Extensive dunes also occur to the south of the Sultan Scarp in Wakami Township. Blanket sand is up to 1m thick and occurs as a discontinuous mantle in dune fields and on parts of adjacent terrain units. Much of the eolian sand shown on the outwash deposit south of the Sultan Scarp is blanket sand.

Drainage in the eolian unit varies from good to mixed (D-M) with a high water table (h) in places, especially between some dunes. Access roads in eolian terrain are typically constructed very easily, but the erosion potential is high once the protective vegetation cover has been removed. A high water table may typically limit excavation in places, and slumping and piping can occur. Driven well points usually supply enough water for domestic use in eolian deposits below the water table. The pollution potential is considered high.

As indicated on the Bedrock Geology of Ontario West Central Sheet Map 2542, the site is underlain by (11) intrusive rocks of Neo to MesoArchaen age comprising of Gneissic tonalite suite and consisting of Tonalite to granodiorite, foliated to gneissic with minor supracrustal inclusions. Intrusive dykes of Palaeo-Proterozoic age, described as Matachewan mafic diabase dykes, are indicated to cross-cut the region in a north north west to south south east trend.

4. INVESTIGATION PROCEDURES AND LABORATORY TESTING

A previous site investigation was carried out by Peto MacCallum Ltd. (PML) during December 2014 to January 2015. (Preliminary Foundation Investigation and Design Report for Trap Creek Culvert Replacement, Highway 129, Township of Chappise, Algoma District, Ontario. Assignment No. 5013-E-0040, G.W.P. 5222-05-00, Site # 46-333/C, WP NO. 5231-05-01, GEOCREs No. 410-17, Peto MacCallum Ltd., (05 October 2016). Four boreholes (TC1, TC2, TC3, and TC4) were advanced to depths ranging from 6.7 m to 11.3 m. DCPT testing was also undertaken at TC2 from 11.3m to 18.3m depth. This information was provided by MTO, and is included in this report as supplementary information. The borehole logs (TC1 to TC4) are included in Appendix D Enclosures.

A site visit was undertaken by one of DST's professional engineers on July 27th and July 28th to review site conditions including access to inlet and outlet locations and to approximately locate the boreholes. The borehole locations were later approved by MTO's foundation engineer. The borehole locations and stratigraphic sections are shown on Drawings 1 and 2 in Appendix C.

DST site investigation fieldwork was carried out in two phases, (1) on August 23rd and (2) on September 18th and 19th, 2016 utilizing a CME 550 drill rig equipped for geotechnical drilling and operated by Landcore Drilling from Chelmsford, Ontario. A total of five boreholes were advanced to depths ranging from 11.3 m to 26.7 m. The minimum number and depth of the boreholes was specified by the Ministry of Transportation (MTO).

In the case of the two boreholes located off the embankment, a 25 mm slotted pipe was temporarily installed in each borehole to a 1.5 m depth in order to maintain an open hole for the purpose of measuring a water level. The pipes were thereafter removed, within 24 hours.

The borehole locations are referenced to the MTO Station numbering system as indicated on the drawings provided by the Ministry. The ground surface elevations at the borehole locations were surveyed by Delta Survey Inc. from Thunder Bay, Ontario and referenced to a benchmark indicated on the drawings provided by the Ministry. Table 4-1 summarizes the details of borehole locations and depths.

All boreholes were abandoned using a suitable abandonment barrier as described in Ontario Regulation 903 and its amendments. Augured boreholes were decommissioned by backfilling the hole upon removal of casing/augers, either to the bottom of the road base or ground surface with bentonite chips. Boreholes advanced by wash boring were decommissioned by backfilling with bentonite – cement grout to the bottom of road base. From the bottom of the road base, granular materials were replaced to the bottom of the asphalt and the asphalt was sealed with a cold patch.

The fieldwork was supervised on a full-time basis by DST personnel who located the boreholes in the field, performed sampling, in-situ testing and logged the boreholes. Soil samples were obtained from the auger flights and from the split spoon sampler used for the standard penetration test (SPT). The SPT involves driving a 51 mm diameter thick-walled sampler into the soil under the energy of a 63.5 kg weight falling through 760 mm. The number of blows required to drive the sampler 300 mm is known as the standard penetration blow count (N) which provides an indication of the condition or consistency of the soil. A Dynamic Cone Penetration Test (DCPT) was also conducted below 20 m depth in Boreholes 3, 4 and 5. The purpose of the test was to investigate the competency of the soil below the termination depth of the borehole. The test is performed by dropping a hammer from a certain fall height, measuring the penetration depth (of a 51mm diameter cone tip) per blow, with termination when blows exceeded 100 blows per 0.3 m. The soil samples collected during drilling were identified in the field, placed in labelled containers and

transported to DST's laboratory in Thunder Bay for further analyses.

Table 4-1: Details of Borehole Location

Borehole ID	Station	Location	Ground Surface Elevation (m)	Depth (m)	Offset (m)	Completion Details
BH1	11+803	Outlet	439.4	11.3	13.2 Lt	Borehole backfilled with bentonite chips from caving depth up to ground surface.
BH2	11+764	Inlet	439.1	14.3	18.3 Rt	Borehole backfilled with bentonite chips from caving depth up to ground surface.
BH3	11+766	Roadway	442.1	20.4 24.7 *	1.2 Rt	Borehole backfilled with bentonite chips from caving depth up to road base then backfilled with granular material and asphalt cold patch to surface.
BH4	11+790	Roadway	442.2	20.4 26.7 *	1.2 Lt	Borehole backfilled with bentonite chips from caving depth up to road base then backfilled with granular material and asphalt cold patch to surface.
BH5	11+790	Shoulder	442.2	20.4 26.7 *	4.4 Rt	Borehole backfilled with bentonite chips from caving depth up to road base then backfilled with granular material and asphalt cold patch to surface.
TC1	-	Inlet	439.1	6.7	-	Borehole backfilled with Bentonite/cement grout
TC2	-	Roadway	442.2	11.3 18.3*	-	Borehole backfilled with Bentonite/cement grout
TC3	-	Roadway	442.1	11.3	-	Borehole backfilled with Bentonite/cement grout
TC4	-	Outlet	438.8	6.7	-	Borehole backfilled with Bentonite/cement grout

*includes DCPT depth

Classification and index tests were subsequently performed in the laboratory on samples collected from the DST boreholes to aid in the selection of engineering properties. Laboratory tests included natural moisture contents, particle size analyses, and chemical tests. A total of eighty-two (82) natural moisture contents, one (1) particle size analysis LS 702, twenty-one (21) sieve analysis LS 602 and one (1) set of chemical tests has been carried out for this assignment. Laboratory test results are presented in the Boreholes Logs and graphical plots are attached in Appendix D (Enclosures).

5. DESCRIPTION OF SUBSURFACE CONDITIONS

The subsurface conditions presented below are based on the information obtained during the DST and PML field investigations, visual and field descriptions of soil and rock, and supplemented by laboratory test results where available.

The generalized ground profile through the existing embankment and outside the embankment footprint based on the conditions encountered in the boreholes consists of the following:

- Asphalt at surface in Boreholes 3, 4 and 5, and TC2 and TC3
- Embankment fill (Boreholes 3, 4 and 5, and TC2 and TC3) comprising of sand and gravel to trace gravel with trace to some silt, with occasional cobbles and boulders.
- Peat at surface in Boreholes 1, 2, TC1, and TC4, and below the Embankment fill in boreholes TC2 and TC3.
- Organic sand underlying the peat at borehole TC2.
- Underlain by a loose to compact sandy silt layer in Borehole 2.
- Underlain by very loose to compact sand and silt layer in all seven (7) boreholes and to the depth of termination in Boreholes 2, 3, 5, TC2 and TC3.
- Underlain by a very loose to compact silt layer in Boreholes 1 and 4 to the depth of termination.

For a more detailed description of encountered materials, please refer to the respective borehole logs presented in Appendix D Enclosures. A brief description of each soil unit with associated laboratory test results is summarized in Table 5-1 below.

Table 5-1: Summary of Subsurface Strata Encountered

Soil Type Description	Encountered in Boreholes	Depth (from-to)	Elevation (from-to)	Thickness (m)	SPT N Value	Relative Density
Asphalt	BH3	0.0 – 0.1	442.1 – 442.0	0.1	N/A	N/A
	BH4	0.0 – 0.1	442.2 – 442.1	0.1	N/A	N/A
	BH5	0.0 – 0.1	442.2 – 442.1	0.1	N/A	N/A
	TC-2	0.0-0.18	442.1 – 441.9	0.18	N/A	N/A
	TC-3	0.0 – 0.24	441.9 – 441.7	0.24	N/A	N/A
Embankment Fill / Pavement Structure / Rockfill	BH3	0.1 – 4.6	442.0 – 437.5	4.5	7-100	Very loose to dense
	BH4	0.1 – 3.3	442.1 – 438.9	3.2	3-100	
	BH5	0.1 – 3.8	442.1 – 438.4	3.7	14-28	
	*TC-2	0.18 – 4.4	441.9 – 437.7	4.2	N/A	
	TC-3	0.24 – 4.6	441.7 – 437.3	4.4	WH-27	
Peat / Organic Soil	BH1	0.0 – 2.4	439.4 – 437.0	2.4	2	Very loose to firm
	BH2	0.0 – 2.4	439.1 – 436.7	2.4	WH	
	BH3	6.1 – 6.6	436.0 – 435.5	0.5	7	
	BH4	4.3 – 4.8	437.9 – 437.7	0.5	(11) **	
	BH5	4.2 – 4.8	438.0 – 437.4	0.6	(9) **	
	TC-1	0.0 – 2.1	439.1 – 437.0	2.1	1-2	
	TC-2	4.4 – 5.9	437.7 – 436.2	1.5	5-6	
	TC-3	4.6 – 6.9	437.3 – 435.0	2.3	3-6	
Sand and Silt	TC-4	0.0 – 2.7	438.8 – 436.1	2.7	1-3	Very loose to Compact
	BH1	2.4 – 9.1	437.0 – 430.3	6.7	1 - 7	
	BH2	4.6 – 14.3	435.0 – 424.8	10.2	2-12	
	BH3	4.6 – 20.4	437.5 – 421.7	15.8	4-13	
	BH4	3.3 – 9.1	438.9 – 433.1	5.8	7-14	
	BH5	3.8 – 20.4	438.4 – 421.8	16.6	4-22	
	TC-1	2.1 – 6.7	437.0 – 432.4	4.6	4-10	
	TC-2	5.9 – 11.3	436.2 – 430.8	5.4	1-6	
Silt	TC-3	6.9 – 11.3	435.0 – 430.6	4.2	1-8	Very loose to compact
	TC-4	2.7 – 6.7	436.1 – 432.1	4.0	6-12	
	BH1	9.1 – 11.3	430.3 – 428.1	2.2	2 – 14	
	BH2	2.4 – 4.1	436.7 – 435.0	1.7	7 – 10	
	BH4	9.1 – 20.4	433.1 – 421.8	11.3	5 – 18	

- SPT 'N' values > 100 indicate bouncing of SPT hammer on cobbles/boulders, and are not an accurate representation of the relative density of the strata
- *Rockfill encountered within the Embankment fill at borehole TC-2 between elevations 441.5 m to 439.0 m.
- ** SPT 'N' numbers not applicable – due to penetration into underlying sand and silt layer.

5.1 Asphalt

Asphalt was encountered at surface in Boreholes 3, 4, 5, TC2 and TC3 with a thickness of 100 to 240 mm.

5.2 Embankment Fill

Embankment fill consisting of brown, sand, some to trace gravel, some silt, was encountered below the asphalt layer in Boreholes 3, 4 and 5 with thicknesses of 3.0 to 3.4 m. SPT 'N' values for the embankment fill layer vary from 14 to 72, indicating a compact to very dense condition. The natural moisture contents of samples tested range between 5 and 21 %. The laboratory test results are summarized in Table 5-2.

The PML boreholes TC2 and TC3 confirmed the presence of Embankment fill comprising of cobbles to depths of 2.3m and 3.1m respectively, underlain by very loose to compact silty Sand Fill to depths of 4.4m and 4.6m respectively.

In the embankment fill, high SPT 'N'-values of more than 100 are likely indicative of embedded cobbles/boulders and therefore, may not be representative of the relative density of these materials.

Table 5-2: Summary of Sieve analysis- Embankment Fill

Laboratory Results – Sieve analysis	
Gravel %	0 to 14
Sand %	70 to 84
Fines %	12 to 17

5.3 Peat

A black, fibrous to amorphous peat was encountered at all borehole locations. Surface peat was encountered in Boreholes 1 and 2 outside the embankment with depths of 2.3 m at both locations, and at boreholes TC1 and TC4 with depths of between 2.1 to 2.7m. SPT 'N' values in the surface peat layer vary from 0 to 3, indicating a very soft to soft consistency. The natural moisture contents of samples tested range between 97 and 320 %.

Organic peat materials were also encountered at depth below the fill materials, likely originally from the same stratigraphic unit as the peat. These were noted as pockets or zones within the underlying native soils in Boreholes 3, 4 and 5 at depths between 4.2 and 6.6 m (Elev. 438.0 m to 435.5 m) below the asphalt. Furthermore, these are noted as discrete peat layers in Boreholes TC2 and TC3 to depths of up to 6.6m below asphalt (Elev. 437.7 m to 435.0 m). Peat thickness at depth varies from 0.5 m to 2.3 m, with SPT 'N' values of 3 to 7 indicating a soft to firm consistency. The higher recorded SPT values at BH4 (11) and BH5 (9) penetrated the underlying silt and sand layer and are thus not representative. A black organic sand, with wood fragments

and rootlets is encountered at this level in borehole 5 at a depth of 4.2 m to 4.8 m (Elev. 438.0 m to 437.4 m).

5.4 Silt

A grey, non-plastic, sandy silt was encountered below the peat layer in Borehole 2 with a thicknesses of 1.7 m. SPT 'N' values for this layer vary from 7 to 10, indicating a loose to compact condition. The natural moisture contents of samples tested range between 17 and 57 %, (the latter indicating significant organic content). The laboratory test results are summarized in Table 5-3.

Table 5-3: Summary of Sieve analysis- Silt

Laboratory Results – Sieve analysis	
Gravel %	0
Sand %	14
Fines %	86

5.5 Sand and Silt

A brown to grey, non-plastic, sand and silt was encountered in all boreholes. The thickness of this layer was not proven as the boreholes were terminated within this layer. The Sand and Silt deposit was encountered at depths of between 2.4 m to 20.4 m (Elev. 438.9 m to 421.7 m), with a thickness of between 4.0 m and 15.8 m. The SPT 'N' values for this layer vary from 1 to 22, indicating a very loose to compact condition. The natural moisture contents of samples tested range between 16 and 85 %. The laboratory test results are summarized in Table 5-4.

Table 5-4: Summary of Sieve analysis- Sand and Silt

Laboratory Results – Sieve analysis	
Gravel %	0 to 28
Sand %	3 to 86
Fines %	13 to 87

5.6 Silt

A grey, silt, with some to trace of sand layer was encountered below the sand and silt layer in Boreholes 1 and 4 up to the depth of termination. As a result, the full depth of the silt deposit was not proven. The silt was encountered at depths of between 9.1 m to 20.4 m (Elev. 433.1 m to 421.8 m), with an encountered thickness of between 2.2 m to 11.3 m. The silt at BH1 is non-plastic, while the silt at BH4 is of low to intermediate plasticity. SPT 'N' values for this layer vary

from 2 to 18, indicating a very loose to compact condition. The natural moisture contents of samples tested range between 18 and 29 %. The laboratory test results are summarized in Table 5-5.

Table 5-5: Summary of Sieve analysis- Silt

Laboratory Results – Sieve analysis	
Gravel %	0
Sand %	3 to 20
Fines %	80 to 97

5.7 Dynamic Cone Penetration Testing (DCPT)

DCPTs were advanced in Boreholes 3, 4 and 5 at depths between 20.4 and 24.7 m (Elev. 421.7 and 417.4 m), 20.4 and 26.7 m (Elev. 421.8 and 415.5 m) and 20.4 and 26.7 m (Elev. 421.8 and 415.5 m) respectively. The DCPT values obtained range from 7 to 100+, 14 to 100+ and 9 to 100+ blows per 0.3 m penetration respectively. A DCPT was also advanced from 11.3m to 18.3m depth at borehole TC2.

It is important to note that the soil penetrated by the DCPT was not sampled, and furthermore friction of soil on the rods make interpretation of the soil condition at the tip difficult. At best, it can be concluded that bedrock surface is deeper than the depth of the DCPT.

5.8 Groundwater

Water levels in the boreholes where seepage was noted were measured upon completion of borehole drilling and prior to backfilling of the borehole. In addition, temporary standpipes were installed at the inlet and outlet boreholes (BH 1 and 2), and read at 24 hours after installation, with final readings taken on completion of fieldwork on 19 September 2016 and then removed. This information is included on the Borehole Logs in Appendix D.

At the time of the field investigation groundwater was observed in all seven boreholes (see Table 5-6). The water level of the creek was at elevation 438.5 m at both inlet and outlet at the time of the DST field investigation. The groundwater levels at the site are expected to be close to or slightly above creek level. These can be expected to vary with the season and with local precipitation events.

Table 5-6: Groundwater depth

Borehole And Location	Measured Water Levels (Depth/Elevation) m				
	After Drilling		24 Hours Reading		September 19, 2016
BH1 (outlet)	Aug 23, 2016	0.9 (438.6)	Aug 24, 2016	0.4 (439.1)	0.5 (439.0)
BH2 (inlet)	Aug 23, 2016	0.9 (438.2)	Aug 24, 2016	0.3 (438.8)	0.3 (438.8)
BH3	Sep 18, 2016	2.8 (439.7)	(Roadway)		(Roadway)
BH4	Sep 19, 2016	2.8 (439.3)	(Roadway)		(Roadway)
BH5	Sep 18, 2016	2.5 (439.4)	(Roadway)		(Roadway)
TC1	Dec 17, 2015	0.8 (438.3)	-		-
TC2	Jan 17, 2015	3.1 (438.9)	-		-
TC3	Dec 17, 2014	2.0 (440.1)	-		-
TC4	Jan 17, 2015	0.8 (438.0)	-		-

5.9 Chemical Test Results

A selected soil sample was submitted to ALS Laboratories Thunder Bay for chemical analyses (pH, sulphate, conductivity, resistivity and Chloride) to assess the potential for corrosion and sulphate attack on buried structures.

The results are presented below in Table 5-7 and discussed in Section 7.15. Copies of the Laboratory Certificate of Analyses are provided in Appendix 'F'.

Table 5-7: Chemical Test Results – Soil sample

Sample ID	Sulphate (mg/kg)	Chloride (µg/g)	pH	Conductivity (mS/cm)	Resistivity (ohm - cm)
BH3 (6.1 m depth)	312	<5.0	7.39	0.410	2440

6. MISCELLANEOUS

The Peto MacCallum Ltd. fieldwork was undertaken during December 2014 to January 2015. DST site investigation fieldwork was carried out over two phases in August 23rd 2016 and on September 18th and 19th, 2016 utilizing a CME 550 drill rig equipped for geotechnical drilling and operated by Landcore Drilling from Chelmsford, Ontario. Fieldwork was supervised on a full time basis by Syed Ahmed, Engineer-in-Training, who located the boreholes in the field, performed sampling, in-situ testing and logged the boreholes. Soil samples collected during drilling were identified in the field, placed in labelled containers and transported to DST's laboratory in Thunder Bay for further analysis. Interpretation of the data and preparation of the report was completed by Selorm Danku, Geotechnical Engineer P. Eng., reviewed by Paul O'Sullivan, Regional Manager, P. Eng., and approved by Mike Fabius, Senior Principal, P. Eng. and a designated principal contact for MTO projects.

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PART 2: ENGINEERING DISCUSSIONS AND RECOMMENDATIONS

7. GENERAL

This section presents an interpretation of the geotechnical data presented in the factual report, and provides geotechnical design recommendations and construction concerns for the proposed culvert replacement.

As discussed in Part I of the report, the generalized stratigraphy at the culvert location, based on the conditions encountered in the boreholes, consists of a 3 m high granular fill embankment over a deep deposit of loose sands and silts. The fill has partially displaced and/or replaced a 2.3 m thick peat deposit, leaving discontinuous organics to depths up to 6.9 m below the top of embankment. The underlying soils, likely of aeolian or outwash origin, are typically normally consolidated (suitable for supporting only light loads), stratified (affecting groundwater flow), highly erodible and highly susceptible to disturbance where below the groundwater table.

The existing twin culverts will be replaced by a single box culvert expected to be 3.5 m wide. For these conditions, and given that the neither the embankment grade nor the culvert will be raised or widened beyond existing, a shallow foundation is considered suitable for this site. Should a culvert with an open base be considered, the effects of the foundations on the soil will likely be more significant.

It is understood that an open cut excavation is proposed to replace the structure. Advantages and disadvantages of the culvert options from the foundation evaluation perspective are tabulated in Table 8-1. Preliminary General Arrangement (GA) drawings were available at the time of the report preparation and the following recommendations are based on the information in the GA drawings. Final construction drawings should be reviewed by DST to confirm the design satisfies the geotechnical recommendations.

7.1 Replacement Structure

It is understood that although a Box Culvert is the preferred option as a replacement structure, geotechnical recommendations for an open footing culvert have also been included, as requested by the Ministry.

The following is a summary of the proposed and potential construction levels:

Top of Highway:	442.2
Creek level (at time of investigation):	438.5
New culvert invert:	437.5
Approx. base level of 0.5 m thick culvert bedding:.....	436.8
Excavation level for 2.25 m frost penetration below invert:	435.5
Deepest level of organic materials encountered in boreholes:.....	435.0

The design of the replacement structure should be in accordance with the Canadian Highway Bridge Design Code CAN/CSA-S6-14 (CHBDC, 2014) and all relevant Ministry of Transportation specifications and guidelines.

7.2 Foundation Design

It is anticipated that the culvert will be located at approximately the same vertical and horizontal alignment of one of the existing structures. The design parameters provided below are recommended for design.

It is expected that the new culvert excavation will encounter organic materials to some degree at the excavation base. Organic peat materials were encountered below the fill materials, likely originally from the same stratigraphic unit as the peat. An assessment of the borehole data indicates that the fill materials may have been placed through displacement methods of construction rather than removal. Therefore, organic materials may be expected to be found mixed in heterogeneously with the native materials and fill materials to depths up to 6.9 m below the asphalt.

Although these organic materials will have been compressed over many years by the weight of the embankment, they are easily disturbed by excavation activity and thereby rendered very compressible. Similarly, the inorganic materials at the excavation, which include very loose and loose wet sands and silts, are very susceptible to disturbance by equipment. These materials are below the water table, and without adequate dewatering prior to excavation will furthermore be

permanently rendered 'disturbed' to considerable depth from groundwater effects such as heaves and boils. The materials described above, when disturbed, will be unsuitable for support of the structure and very difficult to remove and replace.

Where unsuitable soils are encountered the foundation soils should be removed to undisturbed and clean native soils and replaced with Granular A or Granular B Type I material meeting OPSS.PROV 1010 specifications and compacted to a minimum of 95 % of Standard Proctor Maximum Dry Density (SPMDD) to the foundation (or bedding) grade. Before backfilling or bedding is placed, the foundation soils should be inspected by a geotechnical engineer to confirm conditions are as described herein

If sub-excavation is carried out in the dry (with adequate dewatering controls), the material can be replaced with Granular A or Granular B Type I material meeting OPSS.PROV 1010 specifications and compacted to a minimum of 95 % of standard Proctor maximum dry density in accordance with OPSS.PROV 501 "Construction Specification for Compacting". If sub-excavation for frost effects is carried out in the wet (water is maintained at or above adjacent groundwater table), all foundation preparation should be completed in accordance with OPSS 421 "Construction Specification for Pipe Culvert Installation in Open Cut", any specifications provided in the contract documents and as indicated in Section 7.7 Bedding. Achieving the specified compaction over the saturated sands and silts at this site is expected to be difficult.

A suitable alternative to compacted fill as noted above is 20 mm clear stone with a geotextile (OPSS1004.05.02, Class II) wrap. No compaction is then required. In this case complete encasement in geotextile is crucial given that the native sands and silts are extremely erodible even from groundwater seepage. Any openings through the geotextile or its seams can result in ground loss into the stone with subsequent loss of foundation support after construction.

The lateral extent of all fill supporting foundations should not be less than a distance (from the side of the foundation) equal to the depth of fill below the footing. This applies for the full depth of the fill. For example, a 3 m wide culvert underlain by 2 m of engineered fill would require an excavation base 7 m wide plus additional width requirements for access, dewatering equipment and culvert sidefill.

Backfill placement into excavations should commence immediately after the bottom of an excavation is adequately completed. Reducing open excavation time through the use of small stages will reduce the disturbance risk somewhat.

7.2.1 Foundation Design (Concrete Box Culvert)

The geotechnical resistance was estimated for the ultimate limit state (ULS) and serviceability limit state (SLS). The factored resistance at ULS was calculated by applying a load resistance factor of 0.5 to the ultimate bearing capacity, in accordance with the Bridge Design Code (CHBDC) CAN/CSA-S6-14 Section 6.6, and are shown in Table 7-1. The resistance for the SLS was calculated in accordance with CHBDC requirements, for a settlement of 25 mm.

The calculated factored geotechnical resistance at ULS and geotechnical reaction at SLS are presented in Tables 7-1.

Our analyses assume that the box culvert will be rigid end to end and side to side. Factored resistances at ULS are provided for both the post construction (fully backfilled) case and during construction (before backfilling) case. The culvert must be installed on top of bedding material placed on undisturbed native soils, or in the case of additional excavation for controlling frost effects, on compacted fill.

Where the design excavation grade is at the base of culvert bedding all organic materials exposed should be removed and replaced with compacted fill as previously described, particularly to avoid uneven support of the culvert. Note that organic materials were encountered as deep as 6.9 m below road surface (Elevation 435.0 m).

Table 7-1: Geotechnical resistances for Concrete Box Culvert

Footing Size	*Deepest level of subexcavation (m)	Culvert Base	Founding Elevation	Factored Geotechnical Resistance at ULS (kPa)	Factored Geotechnical Resistance at SLS (kPa)
Post Construction	436.8 to 435.0 (Borehole TC3)	3.5 m	437.2	565	85
During Construction		3.5 m	437.2	130	85

*Peat encountered at TC3 from 437.3 m to 435.0 m

Groundwater was encountered at elevations approximately between 438.8 m and 439.7m (DST), and 439.8 m and 438.0 m (PML), respectively during the field investigations, with a creek level at 438.5 m (DST). However, the groundwater conditions are expected to fluctuate both with seasonal changes, local precipitation events as well as the creek level. The proposed founding level of the foundation will be below the recorded groundwater table in cohesionless soil and, therefore dewatering work will be required for the foundation preparation.

7.2.2 Foundation Design (Open Footing Culvert)

The factored geotechnical resistance was calculated for the ultimate limit state (ULS) and serviceability limit state (SLS) for a maximum settlement of 25 mm. The calculations are based on a strip footing 1 m wide with a length equal to 40 m, situated at depths between 5.7 m to 7.2 m (Elev. 436.5 m to 435.0 m) below the existing road elevation (see Table 7-2).

Table 7-2: Geotechnical resistances for open footing culverts with footing base at Elevation 436.5 m to 435.3 m

Footing Width	Soil Cover over Footing base**, m	Founding Level (Depth) m	Founding Level (Elevation) m	Factored Resistance at ULS (kPa)	Factored Resistance at SLS (kPa)
1.0 m	1.0	5.7	436.5	110	75
	1.5	6.2	436.0	150	75
	2.2	6.9	435.3	205	75

** assumes creek bed cannot be eroded below this level

The proposed founding level of the foundation will be below the recorded groundwater table in cohesionless soil and, therefore dewatering work will be required for the foundation preparation.

Any and all organic soil encountered at founding level will require subexcavation and replacement with fill as previously described. Preparation of an undisturbed base will be difficult given the loose and saturated nature of the soils.

7.3 Lateral Earth Loads and Sliding Resistances

The analysis of horizontal and vertical effects of earth loads on the culvert can be performed considering the soil parameters provided in Table 7-3 and as described in Section 7.6.3.1 in the 2014 Canadian Highway Bridge Design Code. Temporary bracing and shoring may be designed using the typical soil parameters given in Table 7-3 and Table 7-4, but the designer/contractor should verify the appropriate soil parameters for the design of a specific bracing and shoring system.

It is recommended that all excavations be either adequately sloped or securely shored and braced to prevent earth caving and to provide a safe and stable work area. The design should incorporate the effects of hydrostatic pressure, traffic surcharge and retained sloping earth conditions in the bracing design.

Table 7-3: Typical Soil Parameters for Earth Loads *

Soil type	Unit weight (kN/m ³)	Internal drained friction angle (Degree)	Interface friction angle** δ (Degree)
Fill-Sand & gravel	20	33.5	22.3
Peat	12	28	18.5
Organic Sand	18	28	18.5
Silt and Sand	19	30	20

*Recommended parameters have been estimated based on visual observation of the soil conditions, results of measured field testing, laboratory testing, correlation with published information (Terzaghi, Peck, and Mesri, Third Edition; Kenney, 1959; CFEM, 4th Edition) and our previous experience with similar materials.

**interface between soil and concrete.

Table 7-4: Lateral Earth Pressure Coefficients

Earth Pressure Coefficient	Equation*	Fill-Sand & Gravel*	Peat*	Organic Sand*	Silt-Sand*
Active Earth Pressure (K _a)	$\left(\frac{1 - \sin\phi}{1 + \sin\phi}\right)$	0.29	0.36	0.36	0.33
Passive Earth Pressure(K _p)	$\left(\frac{1 + \sin\phi}{1 - \sin\phi}\right)$	3.44	2.8	2.8	3.00
At rest (K _o)	$(1 - \sin\phi)$	0.45	0.53	0.53	0.50

* Φ is an angle of internal friction

**The earth pressure coefficient provided here assumes a fully mobilized condition, and for horizontal ground surface only.

7.4 Excavation Stability and Staged Construction

A four stage construction method is typically considered to complete the culvert replacement.

- Stage 1 - divert traffic to side A and construct temporary road widening on side B.
- Stage 2 - divert traffic to temporary lane on side B, excavate existing culvert on side A, construct new culvert, backfill and reinstate road.
- Stage 3 - construct temporary road widening on side A, divert traffic to temporary lane on side A, excavate existing culvert on side B, construct new culvert, backfill and reinstate road.
- Stage 4 – remove all temporary works measures, and road widening fill materials and fully restore shoulders, barriers and road conditions.

Widening the road temporarily will involve fill placement either over the 2.4 m of peat or replacing the peat with fill. In the former case (fill over peat), significant consolidation of the new fill will result as well as a lesser degree of settlement of the adjacent existing fill (assuming fill placement is slow enough to avoid failure, i.e. displacement of the peat). Given organic materials below the existing fill, such settlement may continue for some time (months or more) thereafter while the temporary fill is in place. In the latter case (peat subexcavated), a greater degree of settlement of the adjacent existing fill can be expected. Regardless, all fill placed for a temporary widening should be removed prior to culvert placement on that side to avoid ongoing foundation settlement induced by the widening.

Use of a temporary sheet pile wall or soldier pile wall with lagging may be considered for all or part of the excavation sides. Given the nature of the soils at this site, and in order to prevent consolidation of the materials supporting the culvert, vibration should not be used for pile removal.

The existing embankment slopes should be reinstated as presented in Section 7.12 Embankment Slopes.

An experienced contractor should be consulted during the design process to confirm the suitability of the vertical shoring method for the subsurface conditions encountered at this site. Also, the contractor selected for the work should be experienced and prepared to handle difficult soil conditions.

The side slopes of the excavations were evaluated by calculating the factors of safety (FOS) along possible planes of failure in the existing embankment fill and through the native soil.

The results of the slope stability analyses indicate that the minimum factor of safety for a suitably dewatered excavation with a peat layer included below the fill is 1.32 for a temporary 2H:1V side slope and 1.12 for 1.5H:1V side slope. Thus, a temporary open excavation with 2H:1V sides with effective dewatering is a feasible construction method.

Excavation below the water table in the wet (water in the excavation maintained at or above creek level) can also be considered. This would require clear stone (for example 20mm size) and a geotextile surround. Whereas this method considerably reduces the risk of, for example, (a) base disturbance from uncontrolled hydraulic pressures, (b) settlement of nearby installed culvert during dewatering, and (c) disturbance from compaction, it also requires a very rigorous method of preparation of the underwater excavation base (no disturbed materials allowed), underwater

placement of a geotextile with pre-sewn seams, and preparation of the clear stone upper surface to receive the foundations.

Given the fine nature of the sand and silt, and its stratified nature, it is likely that effective dewatering design will require closely spaced vacuum well points rather than pumped sumps or deep wells. The need for any pressure relief wells within the excavation footprint will also need to be considered in the design, particularly given the stratified nature of the deposits.

A potential detrimental effect during dewatering is the consolidation of the deep deposits of sands and silts below the area dewatered. This can potentially be several centimetres, and its time versus settlement relationship is not easily predicted given any stratification effects within the deposit. Any temporary dewatering design should address this with vertical monitoring of any structure in and adjacent the excavation during any dewatering, together with a plan to deal with movements in excess of tolerable limits. The smaller the excavation area at any one time, and the quicker each section is completed to above the original water table, the less the settlement effects.

A continuous dewatering operation should be provided to keep the excavation stable and free of water. The excavation should be monitored daily throughout the duration of excavation until the completion of backfilling to confirm this. The dewatering system should be maintained and the surrounding area monitored for impacts to items such as, but not limited to, settlement and groundwater usage. The control of water from the dewatering operation should in accordance with OPSS 518 "Construction Specification for Control of Water from Dewatering Operations".

The contractor should be alerted of the relatively high water table and difficult soil conditions as well as the presence of surface water, for example through a non-standard special provision (NSSP 2).

7.5 Earth Excavation

Earth excavation will be required adjacent to the existing and replacement structure and will require temporary surface water ditch diversion and temporary support for traffic. This method can more readily accommodate excavation of large boulders (compared to a vertical structure such as sheet piling), if encountered during excavation. As a minimum, the procedures should be in accordance with OPSS 902 "Construction Specifications for Excavating and Backfilling-Structures". Where temporary protection systems are required they should be constructed in accordance with OPSS.PROV 539 "Construction Specification for Temporary Protection

Systems” and Section 7.6 “Roadway Protection”.

Peat and any other organic soils, or wood/tree remnants wherever encountered, should be excavated and replaced with Granular A or Granular B Type I or Type II and completed in accordance with OPSS.PROV 209 “Construction Specification for Embankments Over Swamps and Compressible Soils”.

The stability of the excavation side slopes will be highly dependent on the contractor’s methodology and ability to effectively dewater the excavation.

Excavations for this project should be constructed in accordance with the requirements of the Occupational Health and Safety Act of Ontario (OHSA), O.Reg. 213/91. According to O.Reg. 213/91, s.226, the soils in the area of interest classify as Type 3 and Type 4 if located above and below the water table respectively. These should be assessed and confirmed in the field as construction progresses, and slope design based on the soil with the highest number. The dominant soil type to be exposed in the excavation (granular fill and loose sands and silts) is expected to be Type 3 above the prevailing creek level and Type 4 below the creek level.

7.6 Excavation Support and Roadway Protection

Since temporary roadway protection is required during the structure replacement, installation of a sheet pile wall or soldier pile wall with lagging and dewatering may be considered to ensure the stability of the excavation. An appropriate roadway protection system should be selected by the contractor and designed by the contractor’s qualified engineer. Temporary road way protection systems should be constructed in accordance with OPS 539 “Construction Specification for Temporary Protection Systems”.

Potential methods for stabilizing excavations and roadway protection include a steel sheet pile wall, soldier pile wall with lagging, and a temporary dewatered excavation using side slopes of 2H:1V or flatter. Table 7-5 below summarizes the advantages and disadvantages of the use of sheet piles, soldier piles, and a sloped excavation. For this site, a sheet pile wall is the most appropriate option. However, it is noted that design of roadway protection is the responsibility of the contractor as per the contract drawings.

Table 7-5: Advantages and Disadvantages of Roadway Protection Methods

Roadway Protection Option	Advantages	Disadvantages
Steel Sheet Pile Wall	<ul style="list-style-type: none"> • Relatively non permeable. • Ease of culvert installation when working below the ground water table. • Can be designed with suitable factor of safety • excellent groundwater control • small footprint 	<ul style="list-style-type: none"> • Difficult driving through cobbles, boulders • High installation cost, • Specialized construction equipment and design is required. • Lateral support with bracing or anchors is required if full height
Feasibility	<ul style="list-style-type: none"> • Recommended option 	
Soldier Pile Retaining Walls with concrete or wood lagging	<ul style="list-style-type: none"> • Robust solution • Relatively impermeable if properly installed, and effectively grouted • Ease of culvert installation when working below the ground water table. • Can be designed with suitable factor of safety • Safer working area for construction 	<ul style="list-style-type: none"> • Difficult driving through cobbles, boulders. • Poor seepage cut-off below excavation, • High installation cost. • Special construction equipment and design is required. • Lateral support with bracing or anchors is required if full height • Control of flow of fine sands and silts through lagging • relief wells in excavation depth may be needed. • Dewatering system required
Feasibility	<ul style="list-style-type: none"> • Alternate option to sheet piling 	
Temporary Cut Slope (2H:1V) Recommended option	<ul style="list-style-type: none"> • Does not require specialized equipment other than dewatering installation. • Relatively short construction time. • Low construction cost. • Ease of construction. • No dewatering if construction is in-the-wet. • Adequate Factor of Safety (1.3) is achievable with suitable dewatering system 	<ul style="list-style-type: none"> • Requires large construction area • Dewatering likely requires well point system with severe consequences if unsuccessful • For construction in the wet, base preparation is difficult • Soils are highly susceptible to erosion and disturbance • dewatering with sumps/trenches not feasible
Feasibility	<ul style="list-style-type: none"> • Feasible if sufficient platform/ road area is available 	

The design of roadway protection may be performed using the typical soil parameters given in Tables 7-3 and 7-4, however the designer/contractor should verify the appropriate soil parameters for the designs. The construction methodology should be in accordance with all applicable standards and regulations. The contractor's method and equipment should be suitable for the site conditions and materials used. This soil investigation does not show evidence of buried obstructions which prevented advancing augers or casing during this investigation. However, occasional cobbles/boulders and rockfill was encountered during drilling observations within the fill material in Boreholes 3, 4 and TC-2. The contractor should be alerted of the presence of these through a non-standard special provision (NSSP 1).

In accordance with OPSS 539.04 the Contractor's selected roadway protection system shall comply with Performance Level 2 as stated.

7.7 Bedding

The base of bedding materials will be in an excavation below the groundwater table placed over undisturbed native soil or on engineered fill. Construction of a precast concrete box culvert and its bedding should follow the provisions in OPSS 422 "Precast Concrete Box Culvert". In addition, the bedding for the structure should be designed in accordance with Section 7.8 of the CHBDC and MTOD 803.021 "Bedding and Backfill for Precast Concrete Box Culvert".

The bedding should be a minimum of 500 mm thick.) beyond all sides of the culvert. The bedding material should consist of "Granular A or Granular B Type I" as per Soil Group I in accordance with Table 7.4 of the Canadian Highway Bridge Design Code. The "Granular A or Granular B Type II" should be in accordance to OPSS.PROV 1010. The "Granular A or Granular B Type I" should be placed in layers not exceeding 200 mm in thickness, loose measurement, and each layer compacted to a minimum of 100 % of standard Proctor maximum dry density.

A suitable alternative to conventional compacted granular bedding is 20 mm clear stone with a geotextile (OPSS 1004.05.02, Class II) wrap. No compaction is then required. In this case complete encasement in geotextile is crucial given that the native sands and silts are extremely erodible even from groundwater seepage. Any openings through the geotextile or its seams can result in ground loss into the stone with subsequent loss of foundation and culvert support after construction.

7.8 Backfill & Cover

The material used for culvert sidefill (backfill) should be raised uniformly on each side of the structure in order to minimize lateral displacement. The sidefill and cover should consist of Granular A or Granular B Type I or Type II" and compacted to not less than 95% of standard Proctor maximum dry density.

All concrete joints should be covered with a 600 mm wide strip of geotextile (OPSS 1860, non woven Type II) to prevent influx of backfill materials through the joints with a 600 mm (minimum) wide coverage strip.

7.9 Channel Diversion

The excavation for the new culvert will incorporate a temporary creek diversion using one of the existing culvert channels. In order to prevent surface water from entering the construction excavation, a cofferdam will be required at each end. Its design will need to incorporate the weak peat materials at either end of the new culvert. Either a low earth berm suitably designed and constructed over the peat (including provisions for stability and settlement) or a sheet pile cut-off is expected to be feasible.

It should be noted that depending on precipitation and the season, the amount of water flow through the creek may vary considerably. The contractor should be prepared to tackle this situation. The contractor should be alerted to high creek water levels and surface water, for example through a non-standard special provision (NSSP 2).

7.10 Erosion Control

Erosion control is essential at the inlet and outlet for the successful performance of a culvert. Generally, rip-rap is used to avoid the erosion at the inlet and outlet of the culvert. The rip-rap slows down the flow close to the channel bed and prevents culvert failure by undermining. The native fine sand and silt soils at this site are highly erosive.

To prevent erosion of the surrounding soils at the inlet, rip-rap treatment should be applied in accordance with OPSD 810.020 "General Rip-Rap Layout for Ditch Inlets" and OPSS 511 "Construction Specification for Rip-Rap, Rock Protection, and Granular Sheeting".

The outlet should be rip-rapped to prevent erosion of the surrounding soils in accordance with OPSD 810.010 "General Rip-Rap Layout for Sewer and Culvert Outlets" and OPSS 511

“Construction Specification for Rip-Rap, Rock Protection, and Granular Sheetting”. Rip-Rap material should comply with OPSS 1004.

To prevent undermining of the bedding, cut-off walls should be installed at both sides of the culvert ends. Cut-off walls should be designed based on the velocity of the water flow, and the type of soil underneath. Cut-off walls should be extended for the whole width and 1.0 m below the culvert as a minimum.

The temporary erosion and sedimentation measures during the construction of culvert should be controlled as described in OPSS 805 “Construction Specification for Temporary Erosion and Sedimentation Control Measures”.

7.11 Frost Protection

In accordance with OPSD 3090.100 “Foundation Frost Depths for Northern Ontario”, the frost penetration at this location is approximately 2.25 m. The soils under the culvert are assessed as being frost susceptible (capable of forming thick ice lenses with the associated pressures and heave).

Frost protection within the culvert backfill and embankment should be in accordance with OPSD 803.030 and 803.031 “Frost Treatment - Pipe Culverts, Frost Penetration Line Below Bedding Grade” and “Frost Treatment - Pipe Culverts, Frost Penetration Line Between Top of Pipe and Bedding Grade”

Frost effects below the culvert should also be considered, given that soils below culvert level include frost susceptible soils. During winter season, ice may form inside the culvert and a very low flow rate may assist ice formation to the culvert invert. Frost may then extend below any concrete exposed to freezing temperatures into the soils below the culvert, generating ice lenses possibly to the depth of frost penetration noted above. The frost heaving would generate additional stresses on the culvert foundation, and is likely to be non-uniform along the culvert length.

Frost heaving impacts depend to a large part on the stiffness of the structure and its susceptibility to differential heave. A narrow footing for an open base culvert will typically be more susceptible to differential heave damage than a wide concrete box culvert foundation. Furthermore, a precast sectional concrete box will typically be more susceptible to heave damage than a cast in place culvert. Heave is generally more severe near the inlet and outlet, where constraining overburden

pressure is least and where there is a higher degree of exposure. Frost susceptible soils generally cause considerably more heave than non-frost susceptible soils (for example 300 mm vs 50 mm, in saturated soil).

Three solutions are available to control heave: (a) a buried insulation layer to prevent frost penetration below the culvert, (b) removal of all frost susceptible material to the full frost penetration depth below concrete and replacement with engineered fill and (c) partial excavation/replacement of frost susceptible material (reduces differential effects rather than total effects, a similar approach to pavement design). Note also that the bedding already provides some replacement of any frost-susceptible soil. The method selected will depend on the structural ability of the culvert to withstand seasonal differential heave, and on how critical its hydraulic design is with respect to differential movement.

At this site, frost protection is not required if winter flows are adequate to prevent frost penetration below invert or if the culvert can accommodate differential movements, both structurally and hydraulically.

Acceptable insulation to prevent frost penetration would be 125 mm Dow Styrofoam Highload 40 Insulation or equivalent with a compressive strength of approximately 275 kPa or greater. For this region with a freezing index greater than 1500 Celsius Degree-Days, it is recommended that the insulation be placed beneath the structure and extend 2.4 m beyond all sides of the buried structure.

7.12 Embankment Slopes

The embankment slopes should be reinstated with a slope not steeper than 2H: 1V if being constructed with compacted granular materials. The slopes should be reinstated with a grade not steeper than 1.5H: 1V if being constructed with rock fill. The minimum thickness of rock fill (measured perpendicular to slope) should be greater than 2 m to achieve an adequate FOS for the reinstated rock fill embankment.

Without any grade raise and with adequate installation methods that avoid soil disturbance, post construction settlement of the embankment surface is expected to be limited to less than 25mm. To achieve this given the difficult soil and groundwater conditions will be difficult but feasible.

7.13 Retaining Walls

Retaining walls may be required at the inlet or outlet of the new culvert to limit embankment footprint at the ends of the culvert. The foundation parameters provided above are applicable for the retaining wall foundation design, provided that the founding level for the base of the wall does not vary significantly from the founding level for the base of the culvert.

The very stiff braced retaining walls may be designed using an "at rest" earth pressure coefficient (K_0) and soil unit weights as listed in Tables 7-3 and 7-4. The lateral soil pressure distribution (σ_h) may be assumed to increase linearly with depth according to:

$$\sigma_h = K_0 (\gamma z + q)$$

where

- σ_h = lateral earth pressure (kPa)
- K_0 = coefficient of lateral earth pressure at rest (use 0.45 for compacted backfill)
- γ = unit weight of soil (use the unit weights in Table 7-3)
- z = depth below grade (m)
- q = surcharge loading (kPa) (A minimum nominal pressure of 10 kPa is recommended)

The above assumes horizontal ground beyond the wall and no hydrostatic (water) conditions adjacent to the below-grade walls. Backfill should, therefore, consist of clean, free-draining granular material or clayey backfill with appropriate drainage considerations. Care should be taken to ensure that the backfill immediately adjacent to the walls is not over compacted, as this could result in excessively high earth pressures against the wall and possible cracking of the wall. As such, the use of large compaction devices (such as ride-on rollers) should be avoided close to the wall. Alternatively, the wall should be designed for compaction induced stresses, as described in the 2006 Canadian Foundation Manual (2006 CFEM).

Free standing, unbraced retaining walls supporting horizontal ground surfaces can be designed, assuming a coefficient of active earth pressure for backfill, K_a . If sloping ground exists beyond the wall, K_a will increase depending on the inclination of the ground. Soil unit weights can be assumed as described above.

Passive pressures may be mobilized against buried foundation elements provided that the foundation is in intimate contact with compacted backfill or has been poured against undisturbed native soil. In the case of compacted fill, the material should consist of compacted, well-graded, Granular A or Granular B Type II materials. This controlled compacted zone should extend a

distance laterally at least equal to twice the height of the foundation element. Passive pressures under these conditions can be calculated using a coefficient of passive pressure (K_p). Significant deformation of the soil is needed before passive pressure is fully mobilized.

7.14 Construction Concerns

The main construction issues that need to be addressed for this site are removal of cover/embankment materials, staged removal of the existing culvert, provisions required for temporary roadway protection, diversion of the channel, effective dewatering, excavation, base preparation, placement of foundation fill and bedding, culvert placement/backfilling, and reinstatement of the embankment fill.

Particularly challenging issues for this site are groundwater control, achieving an undisturbed excavation base, and suitably replacing any subexcavated materials (such as peat) below the foundations. If the base is disturbed, for example through soil heaving or boiling from inadequate dewatering, it may not be feasible to recover adequate soil support conditions. Consequences would likely be severe, requiring either a piled foundation or a significant change in culvert location.

As discussed above, the contractor should be aware of potential subsurface conditions at each phase of the culvert replacement work.

A Geotechnical Engineer should inspect the condition of the excavation base, the foundation and surrounding soils before installation of fill, bedding and other backfills.

7.15 Chemical Testing

The results of the analytical test conducted on water and soil samples at the site location have been presented in Table 5-6 and also included in Appendix F. The suite of the parameters tested is intended to allow the design engineer to assess the requirements for the appropriate type of cement to be used in construction, and the need for corrosion protection of steel reinforcing elements.

The analytical results of the soil samples were compared with applicable Canadian Standards Association (CSA) standards as shown in Table 7-5 below. The chemical sulphate content analyses for the representative soil samples tested indicate a sulphate concentration of 312 mg/kg (0.0312 %) in soil. The results were compared with Canadian Standards Association (CSA) Standards A23.1 for sulphate attack potential on concrete structures and possess a “negligible”

risk for sulphate attack on concrete material and accordingly, conventional GU or MS Portland cement may be used in the construction of the proposed concrete elements.

These results were further evaluated using Table C1 of Building Research Establishment (BRE) Digest 363 (SD1 - 2005). The chloride content of the selected soil sample was also compared with the threshold level and presents a negligible potential for concrete corrosion. Resistivity and conductivity was found to be 2440 ohm-cm and 0.410 mS/ cm respectively for the samples analysed from BH3. The pH values for the soil and water samples were reported as 7.39, which is within the normal range of pH values for soil of 5.5 to 9.0, and indicates a neutral pH condition against corrosion. The pH levels in the soil do not indicate a highly corrosive environment. The test results provided in Table 5-6 may be used to aid in the selection of coating and corrosion protection systems for buried steel objects.

It should be noted that chemical tests on de-icing salts was not conducted on soil samples collected during the site investigations. However, if this site has issues with de-icing salts, protective measures should be taken in accordance with Section 8.11.2.2 Table 8.5 of the CHBDC 2014.

Table 7-6: Additional requirements for concrete subjected to Sulphate Attack

Class of Exposure	Degree of Exposure	Water soluble Sulphate in soil sample (%)	Recommended Cement Grade
S-1	Very Severe	> 2.0	HS or HSb
S-2	Severe	0.20 – 2.0	HS or HSb
S-3	Moderate	0.10 – 0.20	MS, MSb, LH, HS, or HSb

* Information from Table 3 of CSA Standards A23.1-04

7.16 Seismic Design Considerations

The design peak horizontal acceleration was estimated as 0.041 g using the 2005 National Building Code Seismic Hazard maps. Assessment of the liquefaction potential for granular soils was estimated using N_{60} values interpreted from the in-situ SPT data applying Seed and Idriss (1971). Liquefaction was evaluated and it was confirmed that liquefaction is not a concern at the site with a factor of safety against liquefaction of greater than 1.4.

An assessment was completed using the soil information gathered at the site and applying the methodology described in section 4.8.4.4 and the User's Guide – NBC 2015 Structural Comments (part 4 Division B), Commentary J of the 2015 National Building Code.

Based on the estimated average shear wave velocity for the soil (< 180 m/s), and an average standard penetration resistance ($N_{60} < 15$), the project site is classified as site "Class E" (Table 4.1.8.4 A, NBC 2015). For seismic design purposes, the site coefficients in accordance with section 4.3.4.3.3 of the 2014 Canadian Highway Bridge Code should be used.

8. CLOSURE

This report was issued before any final design or construction details have been prepared or issued. The subsurface information and design recommendations presented above should be reviewed at the time of detailed design.

Table 8-1 below summarizes the advantages and disadvantages of the use of the proposed culvert options.

Table 8-1: Advantages and Disadvantages of the Proposed Culvert Options

Option	Option 1: Precast Concrete Box Culvert	Option 2: Open Footing Culvert
Feasibility	Feasible – Preferred Option	Feasible (however needs to be evaluated with the design load)
Relative Cost	Low to Moderate	Moderate
Advantages	Bearing capacity is not a concern Robust pre-cast construction Can withstand more differential settlement Ease of construction/installation Low maintenance cost	Natural streambed maintained Lower excavation cost Ease of installation Use of pre-cast members Low maintenance cost
Disadvantages	Roadway protection system Natural streambed disturbed Higher excavation cost	Increased construction time. Requires foundation excavation and preparation. Requires roadway protection system
Risk/Consequences	In general terms low risk option (except for shoring and dewatering)	In general terms low risk option (except for shoring and dewatering) Greater potential for differential settlement effects, and more susceptible to frost action
Recommended	First	Second

9. REFERENCES

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Northern Ontario Engineering Geology Terrain (NOEGTS) 042 Frazer Lake Area (1981)

Ontario Geological Survey 1991, Bedrock Geology of Ontario – West Central Sheet, Map 2542
Scale 1: 1,000,000

(Preliminary Foundation Investigation and Design Report for Trap Creek Culvert Replacement, Highway 129, Township of Chappise, Algoma District, Ontario. Assignment No. 5013-E-0040, G.W.P. 5222-05-00, Site # 46-333/C, WP NO. 5231-05-01, GEOCREC No. 410-17, Peto MacCallum Ltd., (05 October 2016).

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The Surveys and Design Office, Highway Engineering Division, Ministry of Transportation, 1990, Pavement Design and Rehabilitation Manual.

10. LIMITATIONS OF REPORT

A description of limitations which are inherent in carrying out site investigation studies is given in Appendix 'A', and this forms an integral part of this report.

For DST CONSULTING ENGINEERS INC.

Prepared by:



Selorm Danku P. Eng.
Geotechnical Engineer

Reviewed by:

A handwritten signature in blue ink, appearing to read "P. O'Sullivan".

Paul O'Sullivan,
BEng (Hons), Nat Cert., Nat. Dip., P.Eng
Regional Manager, Infrastructure

Approved by:



Mike Fabius, P.Eng.
Senior Geotechnical Engineer

APPENDIX 'A'
LIMITATIONS OF REPORT

LIMITATIONS OF REPORT

GEOTECHNICAL STUDIES

The data, conclusions and recommendations which are presented in this report, and the quality thereof, are based on a scope of work authorized by the Client. Note that no scope of work, no matter how exhaustive, can identify all conditions below ground. Subsurface and groundwater conditions between and beyond the testholes may differ from those encountered at the specific locations tested, and conditions may become apparent during construction which were not detected and could not be anticipated at the time of the site investigation. Conditions can also change with time. It is recommended practice that DST Consulting Engineers be retained during construction to confirm that the subsurface conditions throughout the site do not deviate materially from those encountered in the testholes. The benchmark and elevations used in this report are primarily to establish relative elevation differences between the testhole locations and should not be used for other purposes, such as grading, excavation, planning, development, etc.

The design recommendations given in this report are applicable only to the project described in the text and then only if constructed substantially in accordance with details stated in this report. Since all details of the design may not be known, we recommend that we be retained during the final stage to verify that the design is consistent with our recommendations, and that assumptions made in our analysis are valid.

Unless otherwise noted, the information contained herein in no way reflects on environmental aspects of either the site or the subsurface conditions.

The comments given in this report on potential construction problems and possible methods are intended only for the guidance of the designer. The number of testholes may not be sufficient to determine all the factors that may affect construction methods and costs, e.g. the thickness of surficial topsoil or fill layers may vary markedly and unpredictably. The contractors bidding on this project or undertaking the construction should, therefore, make their own interpretation of the factual information presented and draw their own conclusion as to how the subsurface conditions may affect their work.

Any results from an analytical laboratory or other subcontractor reported herein have been carried out by others, and DST Consulting Engineers Inc. cannot warranty their accuracy. Similarly, DST cannot warranty the accuracy of information supplied by the client.

Appendix B

DESCRIPTION OF TERMS

EXPLANATION OF TERMS USED IN REPORT

SPT 'N' VALUE: THE STANDARD PENETRATION TEST (SPT) N VALUE OF THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51 mm O.D. SPLIT BARREL SAMPLES TO PENETRATE 0.3 m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5 kg, FALLING FREELY A DISTANCE OF 0.76 m. FOR PENETRATION OF LESS THAN 0.3 m 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 (DCPT): CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51 mm 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 0.3 m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS

TEXTURAL CLASSIFICATION OF SOILS

BOULDERS	COBBLES	GRAVEL	SAND	SILT	CLAY
GREATER THAN 200 mm	75 TO 200 mm	4.75 TO 75 mm	0.075 TO 4.75 mm	0.002 TO 0.075 mm	LESS THAN 0.002 mm

COARSE GRAIN SOIL DESCRIPTION (50% GREATER THAN 0.075 mm)

TERMINOLOGY	TRACE OR OCCASIONAL	SOME	WITH	ADJECTIVE (e.g. SILTY OR SANDY)	AND (e.g. SAND AND SILT)
	LESS THAN 10%	10 TO 20%	20 TO 30%	30 TO 40%	40 TO 60%

CONSISTENCY*: COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH (C_u) AND SPT 'N' VALUES AS FOLLOWS

C_u (kPa)	0 – 12	12 – 25	25 – 50	50 – 100	100 – 200	> 200
N (BLOWS / 0.3 m)	<2	2 - 4	4 - 8	8 - 15	15 - 30	>30
	VERY SOFT	SOFT	FIRM	STIFF	VERY STIFF	HARD

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

N (BLOWS / 0.3 m)	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, 100 mm+ IN LENGTH EXPRESSED AS A PERCENTAGE OF THE LENGTH OF THE CORING RUN.

THE **ROCK QUALITY DESIGNATION (R.Q.D)** FOR MODIFIED RECOVERY IS:

R.Q.D (%)	0 – 25	25 – 50	50 – 75	75 – 90	90 – 100
	VERY POOR	POOR	FAIR	GOOD	EXCELLENT

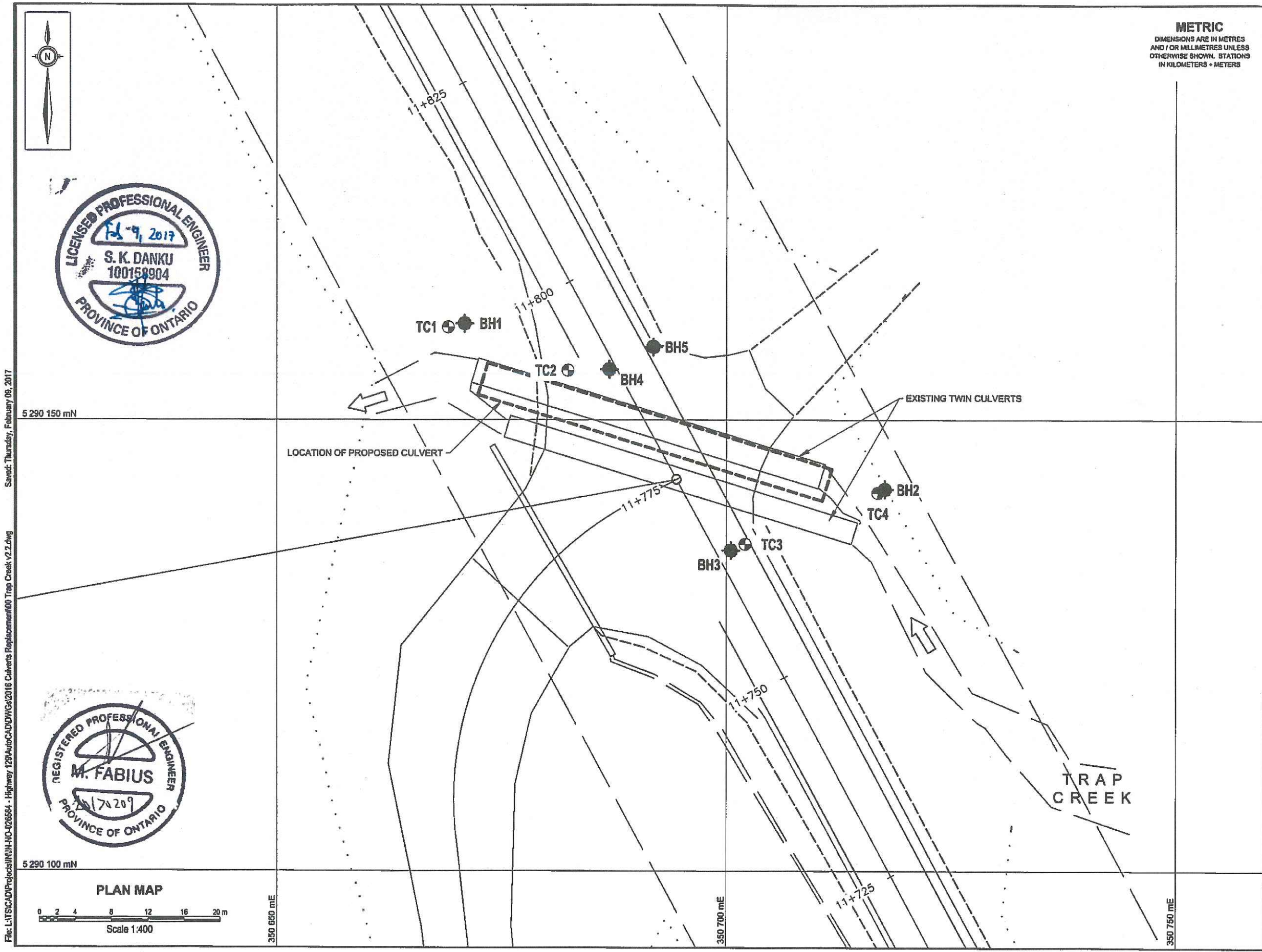
LEGEND OF RECORDS FOR BOREHOLES: SYMBOLS AND ABBREVIATIONS FOR SAMPLE TYPE

SS	SPLIT SPOON SAMPLE	WS	WASH SAMPLE
TW	THIN WALL SHELBY TUBE SAMPLE	AS	AUGER (GRAB) SAMPLE
PH	SAMPLER ADVANCED BY HYDRAULIC PRESSURE	TP	THIN WALL PISTON SAMPLE
WH	SAMPLER ADVANCED BY SELF STATIC WEIGHT	PM	SAMPLER ADVANCED BY MANUAL PRESSURE
SC	SOIL CORE	RC	ROCK CORE
	WATER LEVEL	$SENSITIVITY = \frac{\text{UNDISTURBED SHEAR STRENGTH}}{\text{REMOLDED SHEAR STRENGTH}}$	

*HIERARCHY OF SOIL STRENGTH PREDICTION: 1) LABORATORY TRIAXIAL TESTING. 2) FIELD INSITU VANE TESTING.
3) LABORATORY VANE TESTING. 4) SPT VALUES. 5) POCKET PENETROMETER.

Appendix C

DRAWINGS



METRIC
DIMENSIONS ARE IN METRES
AND / OR MILLIMETRES UNLESS
OTHERWISE SHOWN. STATIONS
IN KILOMETERS + METERS

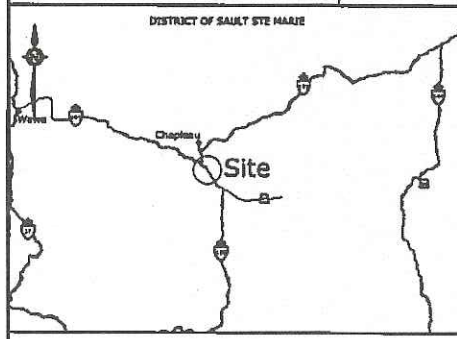
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WP NO 5231-05-01
SITE NO 46-333
GEOCRES NO 410-19



**CULVERT REPLACEMENT
TRAP CREEK CULVERT**
STA 11+716 TO STA 11+835

**SHEET
1**

Survey _____ Revised _____
BOREHOLE LOCATIONS



KEY PLAN
0 20 40 80 120 160 200 km
Scale 1:4 000 000

LEGEND

- Borehole (DST, 2016)
- ⊕ Borehole (PMC, 2015)
- ➔ Flow direction

No.	Elev. (m)	MTM Zone 18		Survey	
		North (m)	East (m)	Station	Offset
BH1	439.40	5290160.7	350670.9	11+803	13.2 m Lt
BH2	439.10	5290142.3	350717.6	11+764	18.3 m Rt
BH3	442.10	5290135.5	350700.5	11+766	1.2 m Rt
BH4	442.20	5290155.6	350687.0	11+790	1.2 m Lt
BH5	442.20	5290158.1	350691.9	11+790	4.4 m Rt

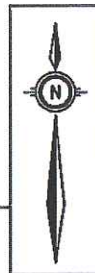
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A	6-Nov-16	DRAFT	RW	SA	MK
B	29-Nov-16	DRAFT	RW	SA / POS	MK
C	08-Feb-17	FINAL	RW	POS	MF

NOTE:
The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed by interpolation and may not represent actual conditions.

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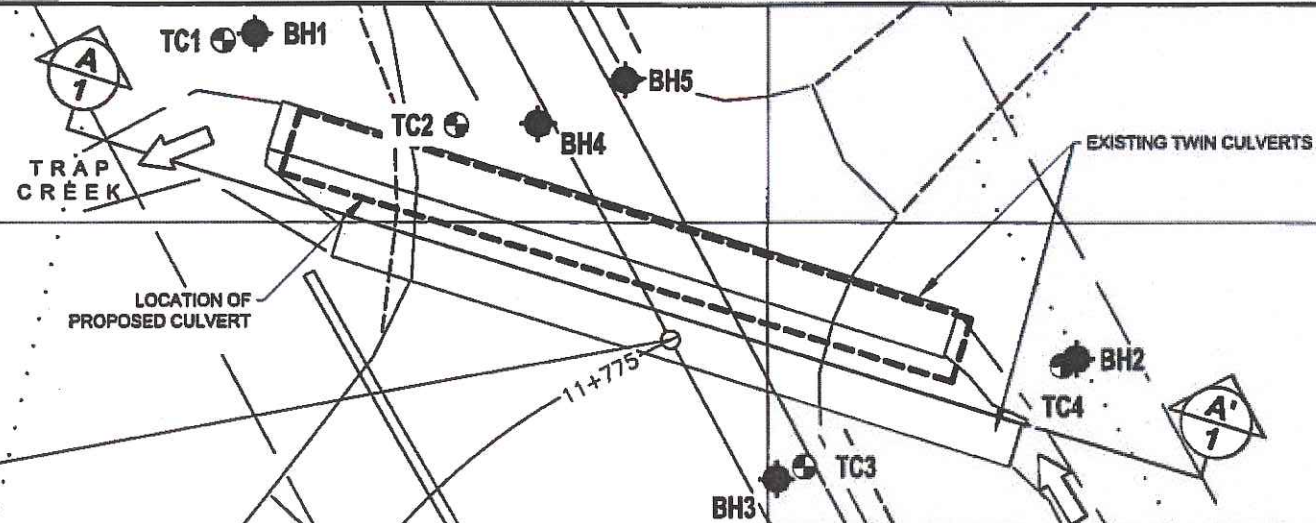
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Saved: Thursday, February 08, 2017



PLAN MAP



Scale 1:400



METRIC
DIMENSIONS ARE IN METRES
AND / OR MILLIMETRES UNLESS
OTHERWISE SHOWN. STATIONS
IN KILOMETERS + METERS

AG NO 5016-E-0001
WP NO 5231-05-01
SITE NO 46-333
GEOCRES NO 410-19

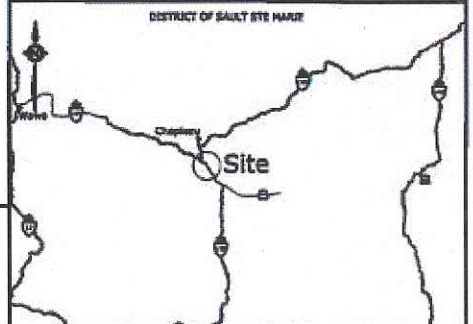


**CULVERT REPLACEMENT
TRAP CREEK CULVERT**

STA 11+716 TO STA 11+835

Survey Borehole Locations
AND SOIL STRATA

SHEET
2

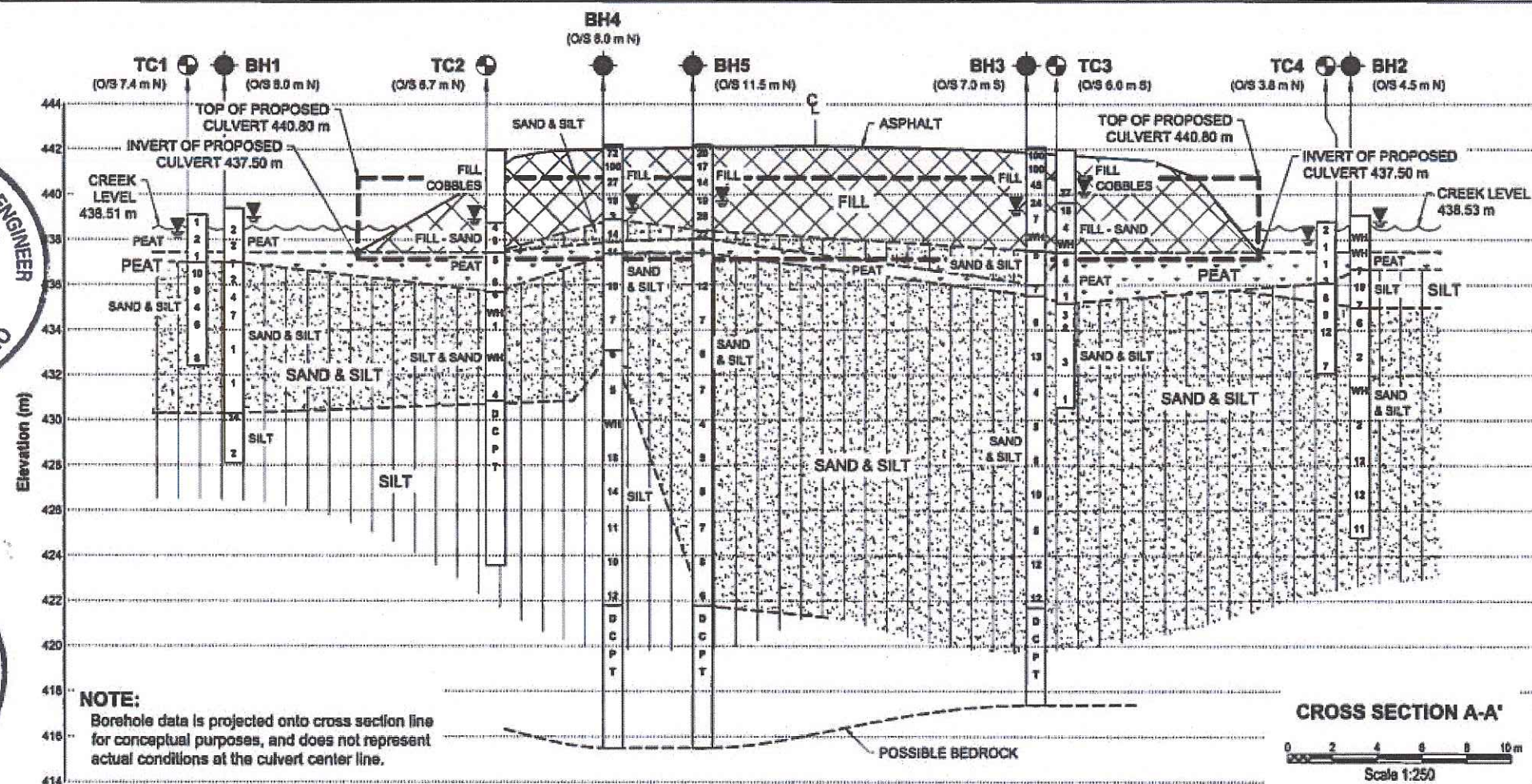


KEY PLAN

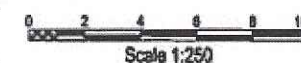


LEGEND

- Borehole (DST, 2016)
- Borehole (PMC, 2015)
- Flow direction
- SPT (BLOWS/300 mm)
- Water level at completion of drilling
- Bedrock
- Clay
- Fill
- Gravel
- Organics
- Peat
- Sand
- Sand & Gravel
- Sand & Silt
- Silt
- Fill

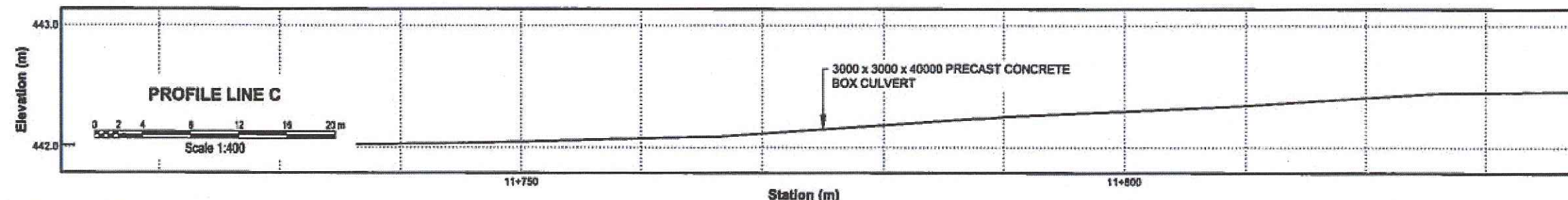


CROSS SECTION A-A'



Scale 1:250

NOTE:
Borehole data is projected onto cross section line
for conceptual purposes, and does not represent
actual conditions at the culvert center line.



PROFILE LINE C



Scale 1:400



No.	Elev. (m)	MTM Zone 18		Survey	
		North (m)	East (m)	Station	Offset
BH1	439.40	5290160.7	350670.9	11+803	13.2 m Lt
BH2	439.10	5290142.3	350717.6	11+784	18.3 m Rt
BH3	442.10	5290135.5	350700.5	11+788	1.2 m Rt
BH4	442.20	5290155.6	350687.0	11+790	1.2 m Lt
BH5	442.20	5290158.1	350691.9	11+790	4.4 m Rt
TC1	439.10	5290160.3	350669.1		
TC2	442.10	5290155.5	350682.4		
TC3	441.80	5290138.2	350702.1		
TC4	438.90	5290141.9	350716.8		

REV	DATE	ISSUE	DRAWN BY	CHECKED	APPROVAL
A	6-Nov-16	DRAFT	RW	SA	MK
B	28-Nov-16	DRAFT	RW	SA / POS	MK
C	08-Feb-17	FINAL	RW	POS	MF

NOTE:
The boundaries between soil strata have been established only at
borehole locations. Between boreholes the boundaries are assumed
by interpolation and may not represent actual conditions.

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DRAWING 2

Appendix D
ENCLOSURES

RECORD OF BOREHOLE No BH1

1 OF 1

METRIC

G.W.P. 5231-05-01 LOCATION TRAP CREEK ORIGINATED BY KN
 DIST S.S. MARIE HWY 129 BOREHOLE TYPE HOLLOW STEM AUGER COMPILED BY SA
 DATUM GEODETIC DATE 2016 08 23 - 2016 08 23 CHECKED BY MK/POS

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			20 40 60 80 100	20 40 60 80 100	W _p W W _L					
								SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE □ QUICK TRIAXIAL x LAB VANE			WATER CONTENT (%)				
439.4	GROUND SURFACE						20 40 60 80 100	20 40 60 80 100					GR SA SI CL		
437.0 2.4	PEAT-amorphous very soft to soft, moist black roots, wood, bad odour from 0.0 m to 2.4 m		AS1	AS			439							Water level measured on September 19, 2016	
			SS2	SS	2										
			SS3	SS	2								97		
													182		
			SS4	SS	7			437							
	SAND and SILT very loose to loose, moist to wet, non-plastic, subrounded grey, fine		SS5	SS	2								0 48 (52)		
SS6			SS	4		436									
SS7			SS	7		435									
			SS8	SS	1								0 49 (51)		
430.3 9.1	SILT, trace SAND compact to very loose, wet, non-plastic grey, fine		SS10	SS	14								0 8 (92)		
			SS11	SS	2										
428.1 11.3	END OF BOREHOLE at 11.3 m														

ON_MOT-HIGH VANES TRAP CREEK.GPJ DATA TEMPLATE.GDT 1/12/16

RECORD OF BOREHOLE No BH2

1 OF 1

METRIC

G.W.P. 5231-05-01 LOCATION TRAP CREEK ORIGINATED BY KN
 DIST S.S. MARIE HWY 129 BOREHOLE TYPE HOLLOW STEM AUGER COMPILED BY SA
 DATUM GEODETIC DATE 2016 08 23 - 2016 08 23 CHECKED BY MK/POS

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				
								○ UNCONFINED + FIELD VANE □ QUICK TRIAXIAL x LAB VANE	20 40 60 80 100	20 40 60 80 100		
439.1	GROUND SURFACE											
	PEAT-amorphous very soft, moist black		AS1	AS								
			SS2	WH								
	bad odour from 0.0 m to 2.4 m		SS3	WH								
436.7												
2.4	SILT, some SAND loose to compact, moist to wet, non-plastic grey		SS4	SS	7							
			SS5	SS	10							
435.0												
4.1	SAND and SILT very loose to compact, wet, non-plastic, subrounded grey, fine to medium		SS6	SS	7							
			SS7	SS	6							
			SS8	SS	2							
			SS9	WH								
			SS10	SS	2							
			SS11	SS	12							
			SS12	SS	12							
			SS13	SS	11							
424.8												
14.3	END OF BOREHOLE at 14.3 m											

ON_MOT-HIGH VANES TRAP CREEK.GPJ DATA TEMPLATE.GDT 1/12/16

+ 3, X 3: Numbers refer to Sensitivity O 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No BH3

2 OF 2

METRIC

G.W.P. 5231-05-01 LOCATION TRAP CREEK ORIGINATED BY KN
DIST S.S. MARIE HWY 129 BOREHOLE TYPE WASHBORE COMPILED BY SA
DATUM GEODETIC DATE 2016 09 18 - 2016 09 19 CHECKED BY MK/POS

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 (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	20 40 60 80 100	20 40 60 80 100	W _p	W		
	GROUND SURFACE												
			SS13	SS		10							
			SS14	SS		5							
			SS15	SS		12							
			SS16	SS		12							
421.7 20.4	DCPT started at 20.4 m												
	very loose to loose from 20.4 m to 22.4 m												
	loose to compact from 22.4 m to 24.7 m												
417.4 24.7	PROBABLE BEDROCK END OF BOREHOLE at 24.7 m												100+ blows for less than 0.3 m penetration

ON_MOT-HIGH-VANES TRAP CREEK.GPJ DATA TEMPLATE.GDT 1/12/16

+³, X³: Numbers refer to Sensitivity O³% STRAIN AT FAILURE

RECORD OF BOREHOLE No BH4

1 OF 2

METRIC

G.W.P. 5231-05-01 LOCATION TRAP CREEK ORIGINATED BY KN
 DIST S.S. MARIE HWY 129 BOREHOLE TYPE WASHBORE COMPILED BY SA
 DATUM GEODETIC DATE 2016 09 19 - 2016 09 19 CHECKED BY MK/POS

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	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100					
442.2	GROUND SURFACE												
442.1	ASPHALT												
	FILL - SAND and GRAVEL to trace GRAVEL, trace to some SILT very dense to compact, dry to moist, subangular to subrounded brown, fine to coarse trace ASPHALT at 0.4 m depth	SS1	SS	72		442							100+ blows for less than 0.1 m penetration
		SS2	SS	100+		441							13 70 (17)
		SS3	SS	27		440							
		SS4	SS	19		439							Water level measured after 30 minutes of the end of drilling
438.9		SS5	SS	3		438							
3.3	SAND and SILT to some SILT, trace GRAVEL very loose to compact, wet, subangular, low plasticity grey, fine to medium	SS6	SS	14		437							
	wood, roots, bad odour from 4.6 m to 5.0 m	SS7	SS	11		436							2 86 (12)
		SS8	SS	10		435							
		SS9	SS	7		434							0 57 (43)
433.1						433							
9.1	SILT, some to trace SAND, trace CLAY very loose to compact, moist to wet, low plasticity grey	SS10	SS	6		432							0 20 74 6
		SS11	SS	5		431							
		SS12	WH			430							0 3 (97)
		SS13	SS	18		429							
						428							

ON_MOT-HIGH VANES TRAP CREEK.GPJ DATA TEMPLATE.GDT 1/12/16

Continued Next Page

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

RECORD OF BOREHOLE No BH4

2 OF 2

METRIC

G.W.P. 5231-05-01 LOCATION TRAP CREEK ORIGINATED BY KN
 DIST S.S. MARIE HWY 129 BOREHOLE TYPE WASHBORE COMPILED BY SA
 DATUM GEODETIC DATE 2016 09 19 - 2016 09 19 CHECKED BY MK/POS

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)		
								○ UNCONFINED	+ FIELD VANE	□ QUICK TRIAXIAL						× LAB VANE		
GROUND SURFACE							20 40 60 80 100											
421.8 20.4	DCPT started at 20.4 m		SS14	SS	14		427											
							426											
			SS15	SS	11		425											
							424											
			SS16	SS	10		423											
							422											
415.5 26.7	very loose to loose from 20.4 m to 22.1 m loose to compact from 22.1 m to 25.1 m compact to dense from 25.1 m to 26.7 m						422											
							421											
							420											
							419											
							418											
							417											
							416											
415.5 26.7	PROBABLE BEDROCK END OF BOREHOLE at 26.7 m														100+ blows for less than 0.3 m penetration			

ON_MOT-HIGH VANES TRAP CREEK.GPJ DATA TEMPLATE.GDT 1/12/16

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

1 OF 2

METRIC

G.W.P.	5231-05-01	LOCATION	TRAP CREEK	ORIGINATED BY	KN
DIST	S.S. MARIE	HWY	129	BOREHOLE TYPE	WASHBORE
DATUM	GEODETIC	DATE	2016 09 18 - 2016 09 18	CHECKED BY	MK/POS

[illegible]

Continued Next Page

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

ON MOT-HIGH VANES TRAP CREEK.GPJ DATA TEMPLATE.GDT 11/2/16

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT

SAND

Fine

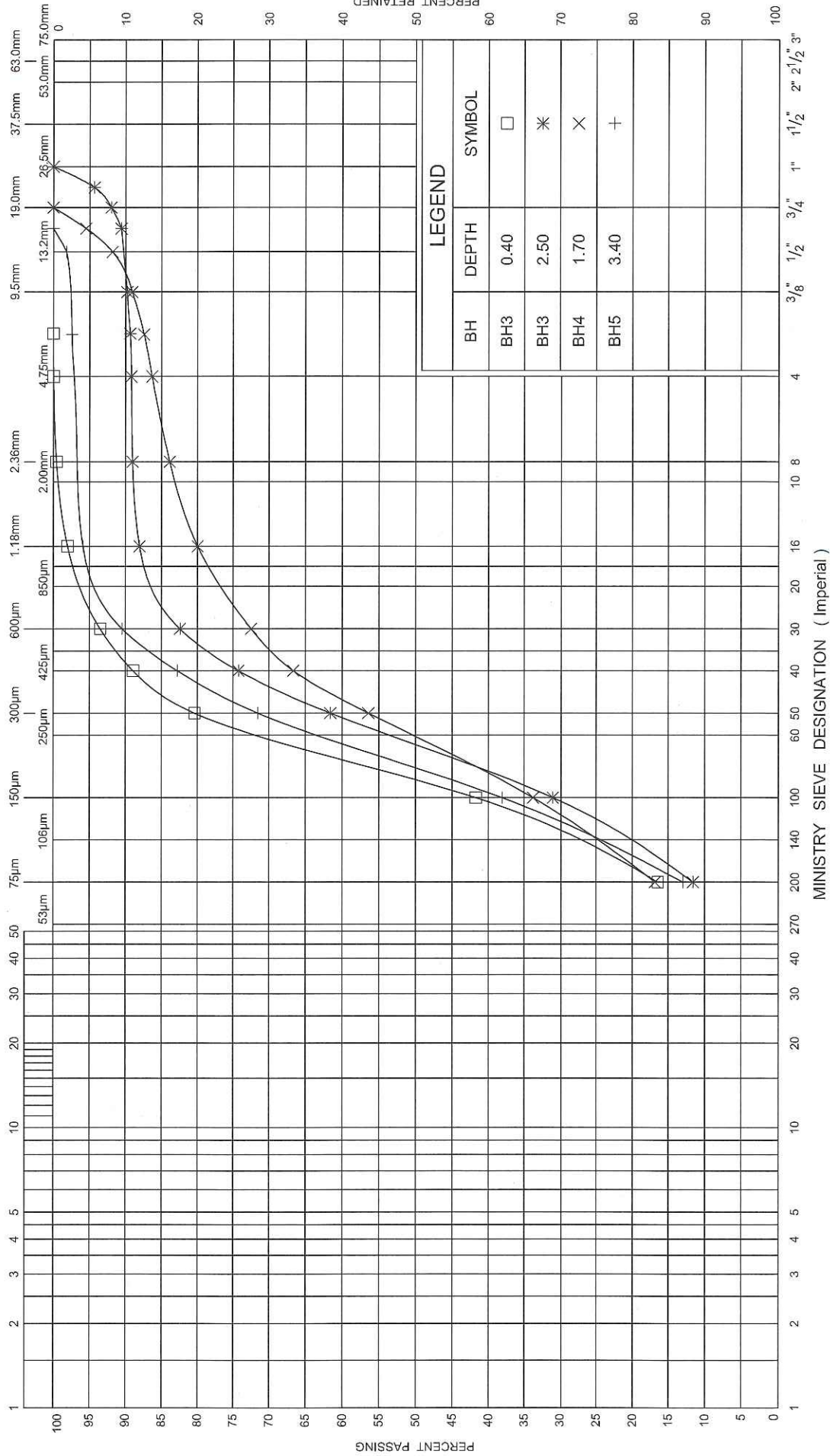
Coarse

GRAVEL

Fine

Coarse

GRAIN SIZE IN MICROMETERS



GRAIN SIZE DISTRIBUTION

FILL

ENCLOSURE 9

G.W.P. # 5231-05-01

TRAP CREEK



UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT

SAND

GRAVEL

Fine

Coarse

Fine

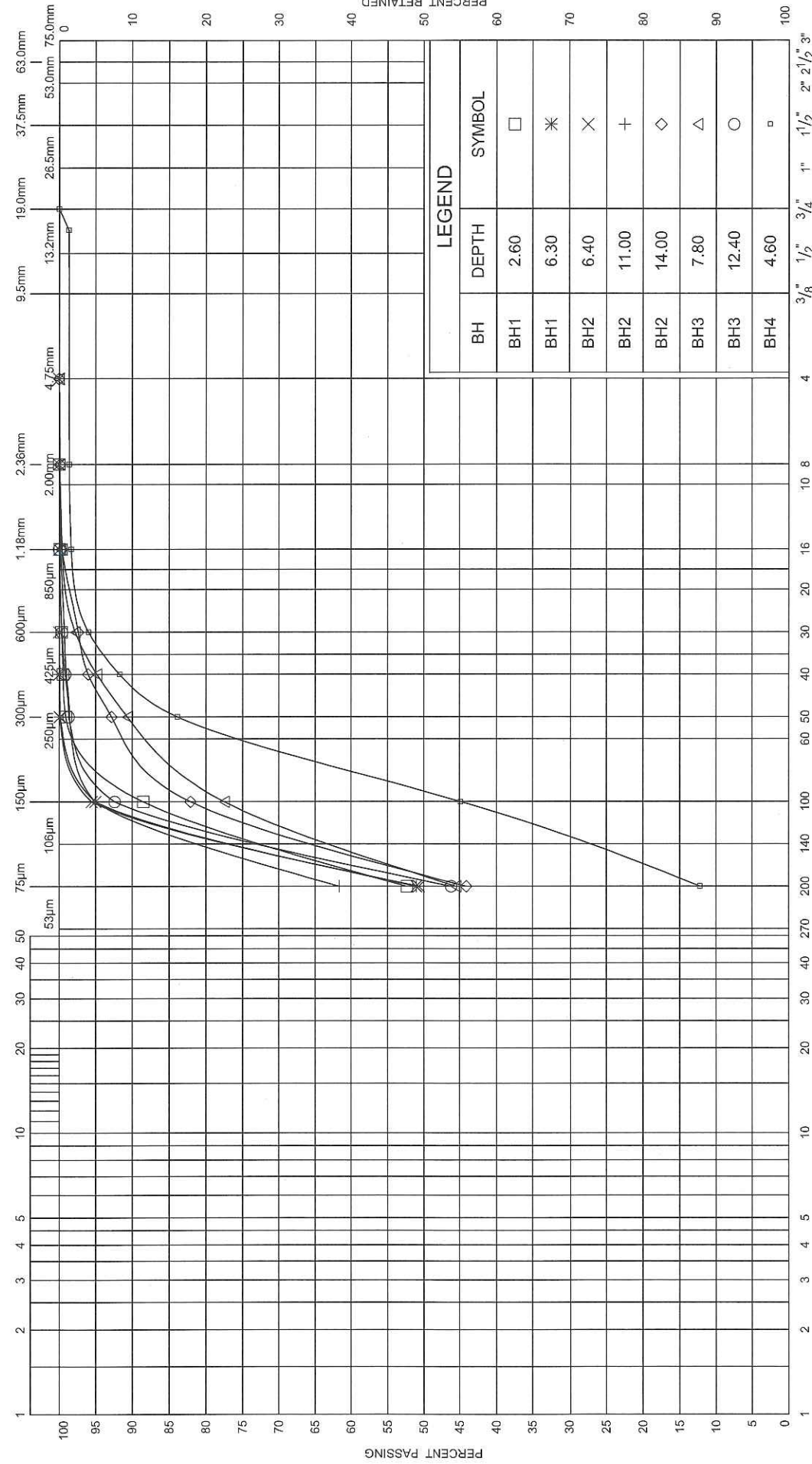
Medium

Coarse

Fine

Coarse

GRAIN SIZE IN MICROMETERS



MINISTRY SIEVE DESIGNATION (Imperial)



GRAIN SIZE DISTRIBUTION

SAND AND SILT

ENCLOSURE 10

G.W.P. # 5231-05-01

TRAP CREEK

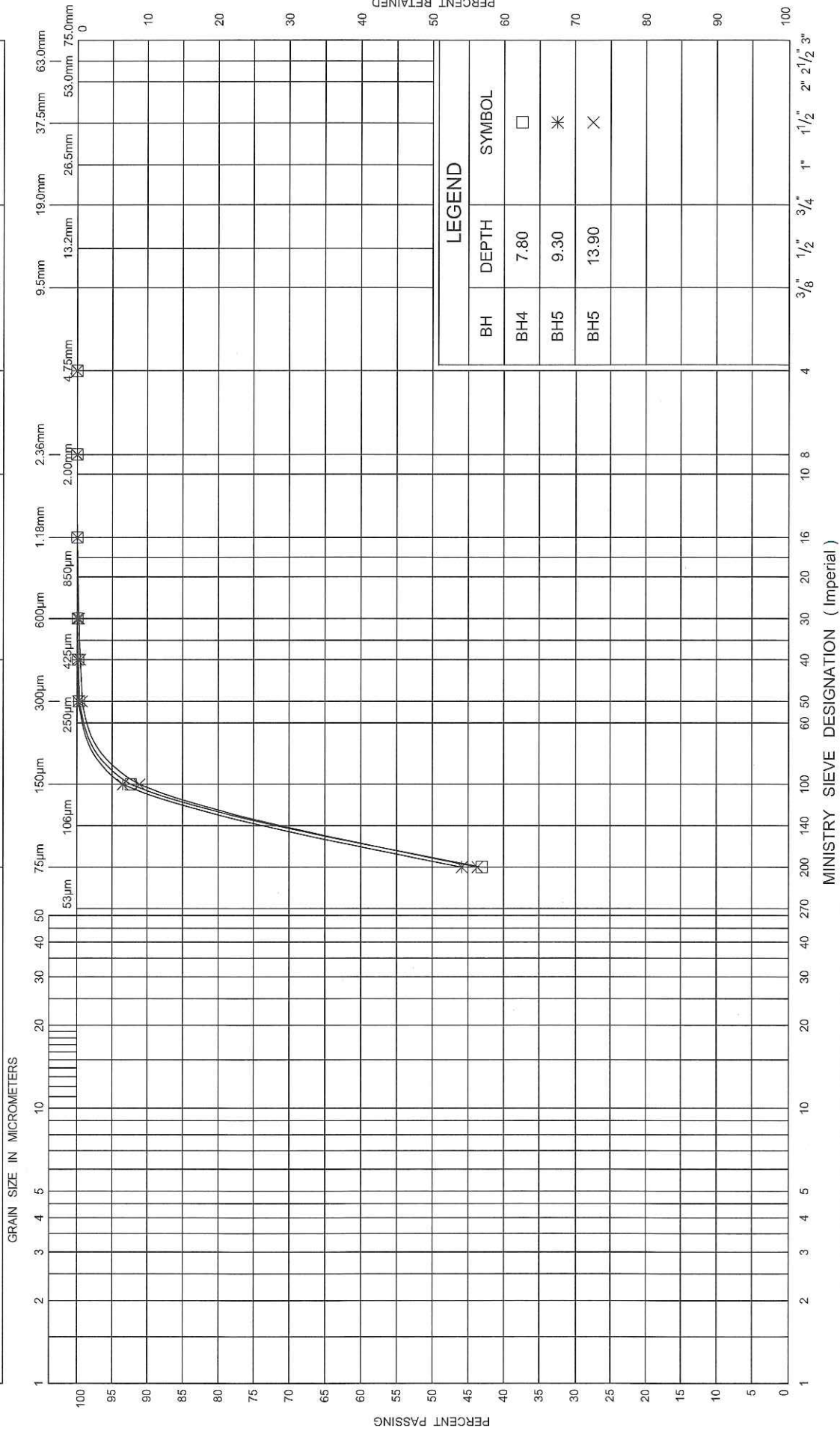
UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT

SAND

GRAVEL

CLAY & SILT		SAND		GRAVEL	
		Fine	Medium	Fine	Coarse



GRAIN SIZE DISTRIBUTION SAND AND SILT

ENCLOSURE 11

G.W.P. # 5231-05-01

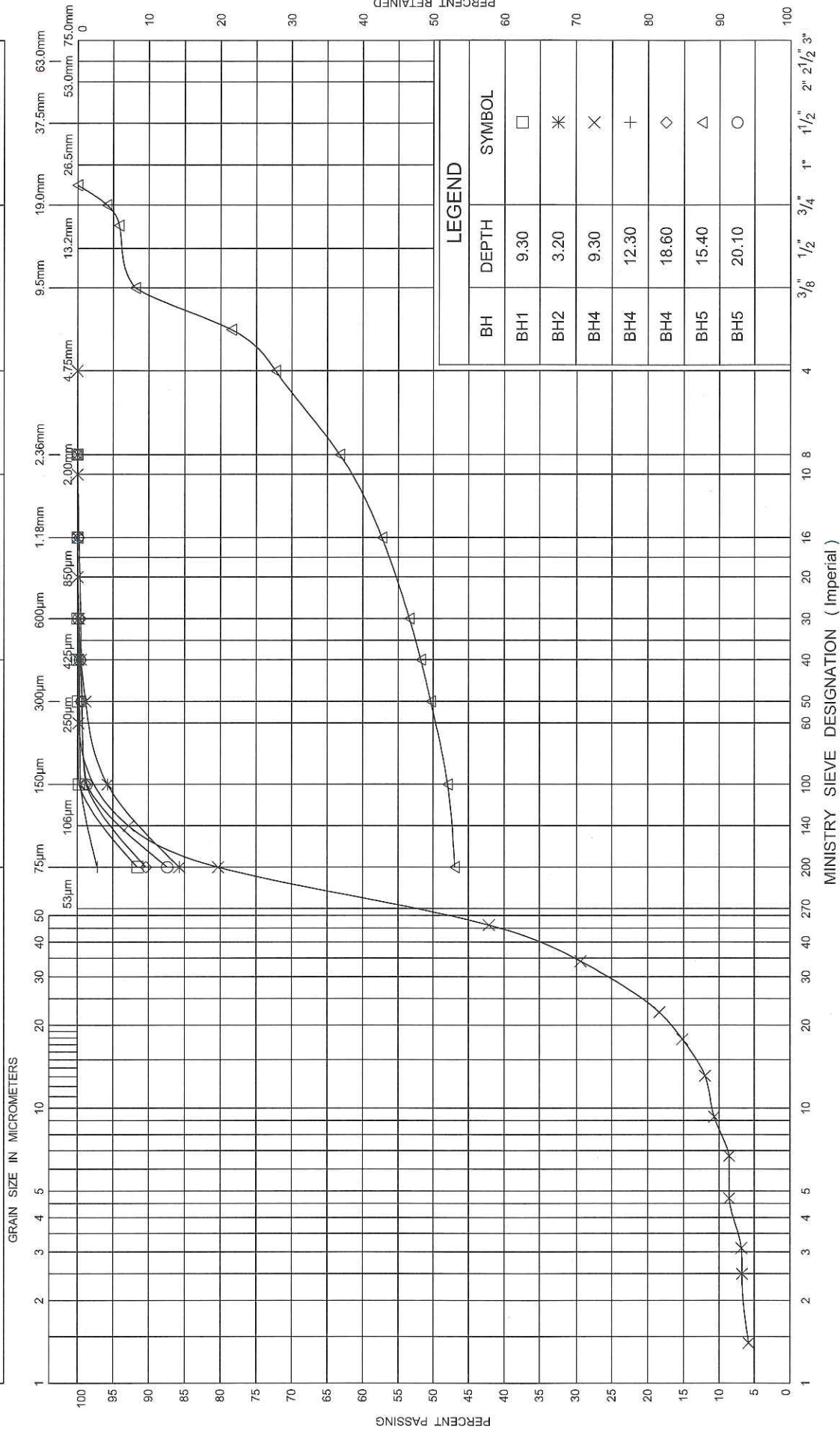
TRAP CREEK



UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT

SAND		GRAVEL	
Fine	Coarse	Fine	Coarse



GRAIN SIZE DISTRIBUTION

SILT

ENCLOSURE 12

G.W.P. # 5231-05-01

TRAP CREEK



RECORD OF BOREHOLE No TC-1

1 of 1

METRIC

G.W.P.	5222-05-00	LOCATION	Trap Creek Coords: 5 290 160.3 N; 350 669.1 E	ORIGINATED BY	F.P.
DIST	Algoma	HWY	129	BOREHOLE TYPE	Tripod + Casing
DATUM	Geodetic	DATE	January 17, 2015	COMPILED BY	M.Kh.
				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 (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE									
439.1	Ground Surface						20	40	60	80	100						
0.0	Peat, fine fibrous Dark brown		1	SS	1		439										
	amorphous		2	SS	2		438										
			3	SS	1										144		
437.0	Sandy silt to silty sand trace clay		4	SS	10		437									0 37 61 2	
2.1	Loose Grey Wet		5	SS	9		436										
			6	SS	4		435										
			7	SS	6		434									0 46 52 2	
			8	SS	8		433										
432.4	End of borehole																
6.7																	
<div>* 2015 01 17</div> <div> Water level observed during drilling</div> <div> Water level measured on completion</div> <div>Borehole caved in at 1.5m</div>																	

RECORD OF BOREHOLE No TC-2

1 of 2

METRIC

G.W.P. 5222-05-00 LOCATION Trap Creek Coords: 5 290 155.5 N; 350 682.4 E ORIGINATED BY F.P.
DIST Algoma HWY 129 BOREHOLE TYPE C.F.H.S.A. and Dynamic Cone Penetration Test COMPILED BY M.Kh.
DATUM Geodetic DATE January 17, 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 (%) 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										
442.1	Ground Surface						20 40 60 80 100				20 40 60							
0.0	180mm asphalt over sand and gravel (PAVEMENT FILL)		1	AS	-		442											
441.5	cobbles		2	AS	-													
0.6	(ROCKFILL)																	
439.0	Sand to silty sand		3	SS	4		439							0 91 (9)				
3.1	Very Loose Wet		4	SS	3		438											
437.7	Peat, amorphous		5	SS	5		437											
4.4	Dark brown		6	SS	6		436											
436.2	Silty sand to sandy silt trace clay		7	SS	6		435											
5.9	Very loose Grey Wet to loose		8	SS	WH**		434							0 47 52 1				
			9	SS	1		433											
			10	SS	WH		432											
			11	SS	4		431							0 49 49 2				
430.8	End of borehole						430											
11.3	Switch to dynamic cone penetration at 11.3m						429											
	Probable silty sand						428											
	Loose to dense																	

Cont'd

RECORD OF BOREHOLE No TC-2

2 of 2

METRIC

G.W.P. 5222-05-00 LOCATION Trap Creek Coords: 5 290 155.5 N; 350 682.4 E ORIGINATED BY F.P.
 DIST Algoma HWY 129 BOREHOLE TYPE C.F.H.S.A. and Dynamic Cone Penetration Test COMPILED BY M.Kh.
 DATUM Geodetic DATE January 17, 2015 CHECKED BY M.V.

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 (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		W _p W W _L				
427.1								20 40 60 80 100						
15.0	Probable silty sand Loose to dense (Cont'd.)						427	○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE						
							426							
							425							
423.8							424							
18.3	End of dynamic cone penetration test							120/15cm						
	<div>* 2014 12 17</div> <div>∇ Water level observed during drilling</div> <div>\blacktriangledown Water level measured on completion</div> <div>WH** denotes penetration due to weight of hammer and rods</div> <div>C.F.H.S.A. denotes Continuous Flight Hollow Stem Augers</div>													

RECORD OF BOREHOLE No TC-3

1 of 1

METRIC

G.W.P. 5222-05-00 LOCATION Trap Creek Coords: 5 290 136.2 N; 350 702.1 E ORIGINATED BY F.P.
DIST Algoma HWY 129 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY M.Kh.
DATUM Geodetic DATE December 17, 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	REMARKS & GRAIN SIZE DISTRIBUTION (%)					
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)				
								● QUICK TRIAXIAL × LAB VANE										○ UNCONFINED + FIELD VANE				
441.9	Ground Surface							20	40	60	80	100	20	40	60	kN/m ³	GR SA SI CL					
0.0	240mm Asphalt over sand and gravel (PAVEMENT FILL)		1	AS																		
441.1			2	AS																		
0.8	cobbles		3	SS	27																	
			4	SS	18																	
	Sand to silty sand occasional cobbles		5	SS	4																	
	Compact to loose (FILL) Moist to wet		6	SS	WH**												0 90 (10)					
437.3	Peat, amorphous sand and silt seams Dark brown		7	SS	6																	
4.6			8	SS	4																	
			9	SS	3																	
435.0	Sandy silt to silty sand trace clay		10	SS	3																	
6.9	Very loose Grey Wet to loose		11	SS	8												0 69 30 1					
			12	SS	3																	
			13	SS	1																	
430.6	End of borehole																					
11.3																						
<div>* 2014 12 17</div> <div> Water level observed during drilling</div> <div> Water level measured on completion</div> <div>WH** denotes penetration due to weight of hammer and rods</div>																						

* 2014 12 17

▽ Water level observed during drilling

▼ Water level measured on completion














WH** denotes penetration due to weight of hammer and rods

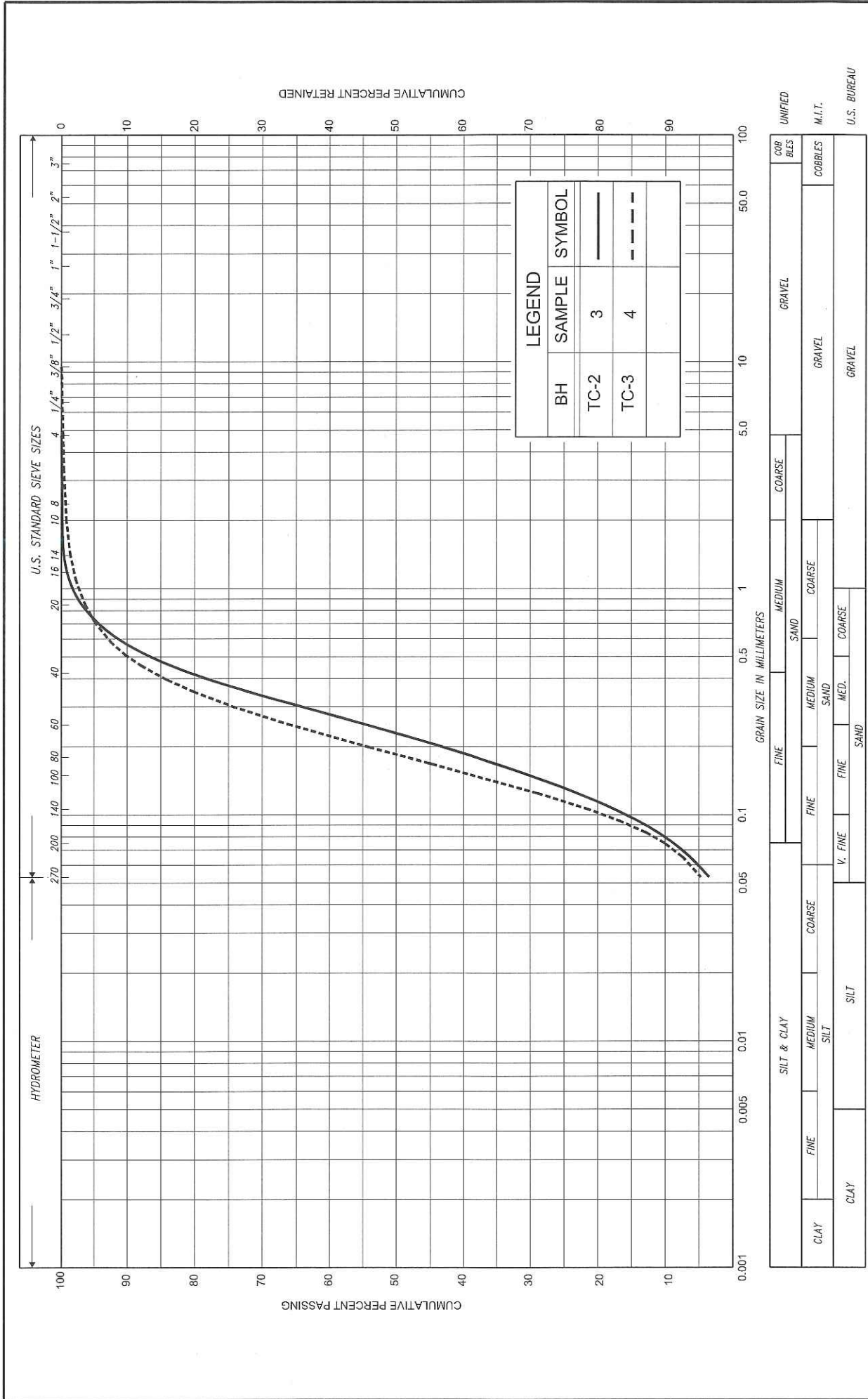
RECORD OF BOREHOLE No TC-4

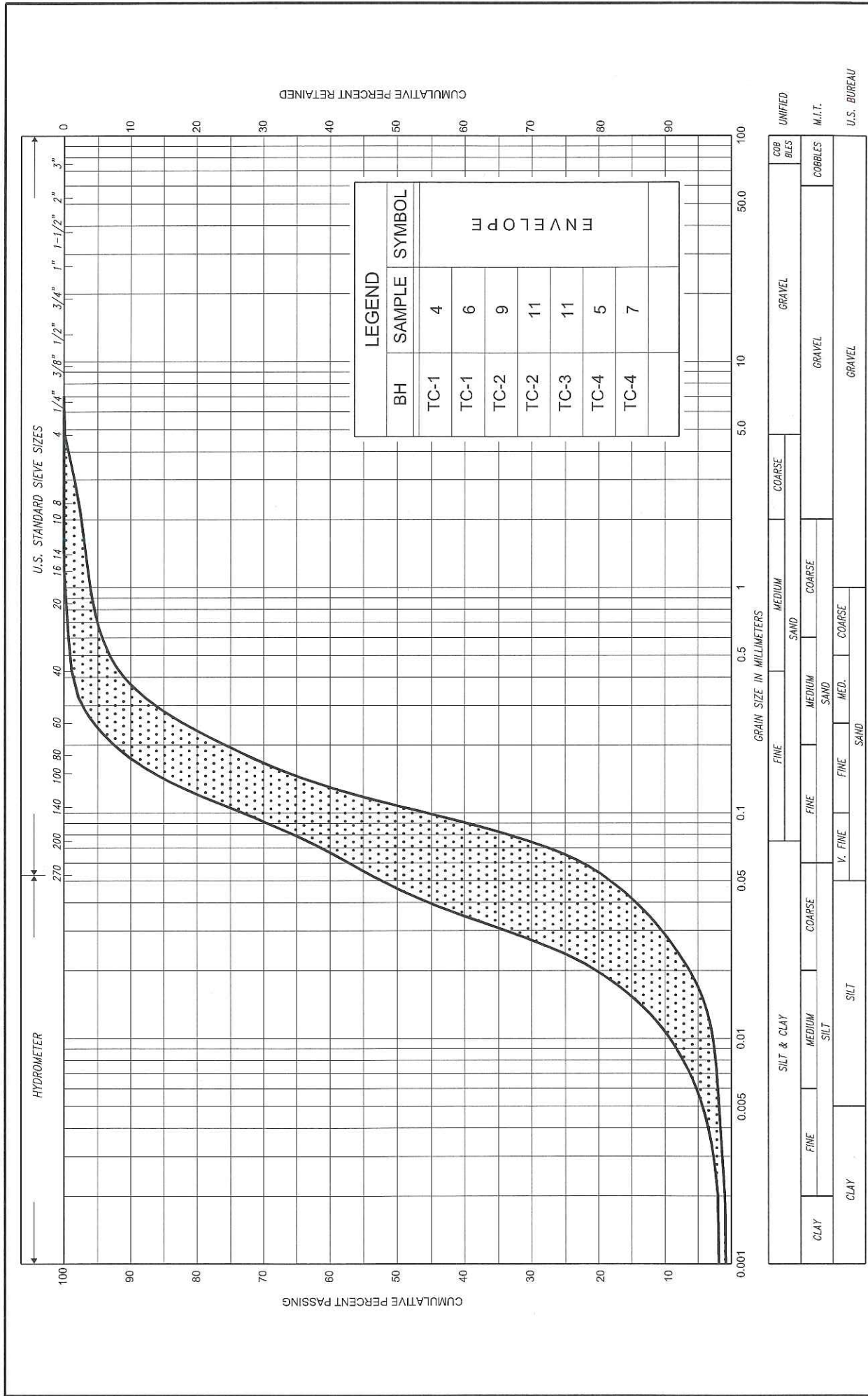
1 of 1

METRIC

G.W.P. 5222-05-00 LOCATION Trap Creek Coords: 5 290 141.9 N; 350 716.8 E ORIGINATED BY F.P.
DIST Algoma HWY 129 BOREHOLE TYPE Tripod + Casing COMPILED BY M.Kh.
DATUM Geodetic DATE January 17, 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	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								○ UNCONFINED		+ FIELD VANE								● QUICK TRIAXIAL		× LAB VANE
438.8	Ground Surface						20	40	60	80	100	20	40	60	kn/m ³	GR SA SI CL				
0.0	Peat, fine fibrous Dark brown		1	SS	2															
	amorphous		2	SS	1															
			3	SS	1															
436.1			4	SS	3															
2.7	Silty sand to sandy silt trace clay																			
	Loose to Grey Wet compact		5	SS	6															
			6	SS	9															
			7	SS	12															
432.1																				
6.7	End of borehole		8	SS	7															
<div>* 2015 01 17</div> <div> Water level observed during drilling</div> <div> Water level measured on completion</div> <div>NOTE: Borehole caved in at 1.5m</div>																				





2015 National Building Code Seismic Hazard Calculation

INFORMATION: Eastern Canada English (613) 995-5548 français (613) 995-0600 Facsimile (613) 992-8836
Western Canada English (250) 363-6500 Facsimile (250) 363-6565

December 22, 2016

Site: 47.7487 N, 83.3876 W User File Reference: Trap Creek Culvert - Highway 129, Ontario

Requested by: , DST Consulting Engineers Inc,

National Building Code ground motions: 2% probability of exceedance in 50 years (0.000404 per annum)

Sa(0.05)	Sa(0.1)	Sa(0.2)	Sa(0.3)	Sa(0.5)	Sa(1.0)	Sa(2.0)	Sa(5.0)	Sa(10.0)	PGA (g)	PGV (m/s)
0.052	0.073	0.070	0.060	0.050	0.031	0.016	0.0037	0.0016	0.041	0.039

Notes. Spectral (Sa(T), where T is the period in seconds) and peak ground acceleration (PGA) values are given in units of g (9.81 m/s²). Peak ground velocity is given in m/s. Values are for "firm ground" (NBCC 2015 Site Class C, average shear wave velocity 450 m/s). NBCC2015 and CSAS6-14 values are specified in **bold font**. Three additional periods are provided - their use is discussed in the NBCC2015 Commentary. Only 2 significant figures are to be used. *These values have been interpolated from a 10-km-spaced grid of points. Depending on the gradient of the nearby points, values at this location calculated directly from the hazard program may vary. More than 95 percent of interpolated values are within 2 percent of the directly calculated values.*

Ground motions for other probabilities:

Probability of exceedance per annum	0.010	0.0021	0.001
Probability of exceedance in 50 years	40%	10%	5%
Sa(0.05)	0.0049	0.017	0.028
Sa(0.1)	0.0082	0.026	0.041
Sa(0.2)	0.0100	0.028	0.043
Sa(0.3)	0.0090	0.026	0.039
Sa(0.5)	0.0069	0.022	0.033
Sa(1.0)	0.0035	0.013	0.020
Sa(2.0)	0.0013	0.0056	0.0096
Sa(5.0)	0.0004	0.0012	0.0021
Sa(10.0)	0.0003	0.0007	0.0010
PGA	0.0046	0.015	0.024
PGV	0.0039	0.015	0.024

References

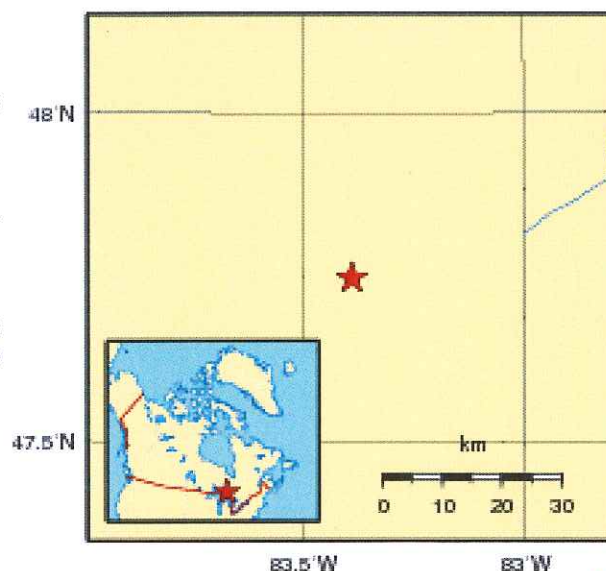
National Building Code of Canada 2015 NRCC no. 58190;
Appendix C: Table C-3, Seismic Design Data for Selected Locations in Canada

User's Guide - NBC 2015, Structural Commentaries NRCC no. xxxxxx (in preparation)
Commentary J: Design for Seismic Effects

Geological Survey of Canada Open File 7893 Fifth Generation Seismic Hazard Model for Canada: Grid values of mean hazard to be used with the 2015 National Building Code of Canada

See the websites www.EarthquakesCanada.ca and www.nationalbuildingcode.ca for more information

Aussi disponible en français



Natural Resources
Canada

Ressources naturelles
Canada



Appendix E

**NON-STANDARD SPECIAL
PROVISION**

COBBLES AND BOULDERS - Item No. 1

Non-Standard Special Provision 1

This special provision covers the presence of rockfill, cobbles and boulders materials in subsurface strata. The Contractor is advised of the following foundation conditions:

Cobbles and boulders were identified within the embankment fill at borehole locations. The contractor should be aware of the potential for encountering cobbles and boulders at the site during excavation and installation of temporary roadway protection as well as dewatering systems.

GROUNDWATER- Item No. 2

Non-Standard Special Provision 2

This special provision covers surface and groundwater dewatering at the site location.

Depending on the season and precipitation events, the amount of water flow through the creek may vary. The contractor should be prepared for potentially high flows through the diversion system. Flooding of the excavation could have significant impacts on the integrity of soils supporting the foundations. It is furthermore noted that the native soils include loose fine sand and silts, which are considered particularly susceptible to erosion by flowing surface water.

Cohesionless soils below the groundwater table will be subjected to unbalanced hydrostatic conditions. Potential effects of inadequate control include disturbance of soils supporting culvert foundations (for example through hydraulic base heave, excavation slope instability, internal soil erosion (piping), boils, quick conditions, base instability) and deep soil consolidation causing uneven settlement of any parts of the culvert already installed. The contractor shall provide effective dewatering systems which are adequate to lower the groundwater level, allow excavation and construction, and maintain subgrade integrity at all times.”

Appendix F

**CHEMICAL TEST
RESULTS**



DST Thunder Bay
ATTN: Selorm Danku
DST Consulting Engineers Inc.
1120 Premier Way , Suite 200
Thunder Bay ON P7B 0A3

Date Received: 01-NOV-16
Report Date: 11-NOV-16 13:47 (MT)
Version: FINAL

Client Phone: 807-345-3620

Certificate of Analysis

Lab Work Order #: L1851709

Project P.O. #: NOT SUBMITTED

Job Reference:

C of C Numbers:

Legal Site Desc:

Christine Paradis
Project Manager

[This report shall not be reproduced except in full without the written authority of the Laboratory.]

ADDRESS: 1081 Barton Street, Thunder Bay, ON P7B 5N3 Canada | Phone: +1 807 623 6463 | Fax: +1 807 623 7598
ALS CANADA LTD Part of the ALS Group A Campbell Brothers Limited Company

ALS ENVIRONMENTAL ANALYTICAL REPORT

Sample Details/Parameters	Result	Qualifier*	D.L.	Units	Extracted	Analyzed	Batch
L1851709-1 TOE CREEK Sampled By: Client on 01-NOV-16 @ 00:01 Matrix: Soil							
Physical Tests							
Conductivity	0.0589		0.0040	mS/cm		09-NOV-16	R3591555
% Moisture	16.1		0.10	%	05-NOV-16	06-NOV-16	R3589113
pH	7.78		0.10	pH units		08-NOV-16	R3590446
Resistivity	17000		1.0	ohm*cm		09-NOV-16	
Leachable Anions & Nutrients							
Chloride	<5.0		5.0	ug/g	09-NOV-16	09-NOV-16	R3592422
Anions and Nutrients							
Sulphate	<20		20	mg/kg	04-NOV-16	07-NOV-16	R3590601
L1851709-2 TRAP CREEK Sampled By: Client on 01-NOV-16 @ 00:01 Matrix: Soil							
Physical Tests							
Conductivity	0.410		0.0040	mS/cm		09-NOV-16	R3591555
% Moisture	18.5		0.10	%	05-NOV-16	06-NOV-16	R3589113
pH	7.39		0.10	pH units		08-NOV-16	R3590446
Resistivity	2440		1.0	ohm*cm		09-NOV-16	
Leachable Anions & Nutrients							
Chloride	<5.0		5.0	ug/g	09-NOV-16	09-NOV-16	R3592422
Anions and Nutrients							
Sulphate	312		20	mg/kg	04-NOV-16	07-NOV-16	R3590601
L1851709-3 BURYING CREEK Sampled By: Client on 01-NOV-16 @ 00:01 Matrix: Soil							
Physical Tests							
Conductivity	0.464		0.0040	mS/cm		09-NOV-16	R3591555
% Moisture	50.7		0.10	%	05-NOV-16	06-NOV-16	R3589113
pH	7.27		0.10	pH units		08-NOV-16	R3590446
Resistivity	2160		1.0	ohm*cm		09-NOV-16	
Leachable Anions & Nutrients							
Chloride	63.7		5.0	ug/g	09-NOV-16	09-NOV-16	R3592422
Anions and Nutrients							
Sulphate	288		20	mg/kg	04-NOV-16	07-NOV-16	R3590601
L1851709-4 POULIN CREEK Sampled By: Client on 01-NOV-16 @ 00:01 Matrix: Soil							
Physical Tests							
Conductivity	0.184		0.0040	mS/cm		09-NOV-16	R3591555
% Moisture	19.2		0.10	%	05-NOV-16	06-NOV-16	R3589113
pH	7.81		0.10	pH units		08-NOV-16	R3590446
Resistivity	5450		1.0	ohm*cm		09-NOV-16	
Leachable Anions & Nutrients							
Chloride	20.5		5.0	ug/g	09-NOV-16	09-NOV-16	R3592422
Anions and Nutrients							
Sulphate	28		20	mg/kg	04-NOV-16	07-NOV-16	R3590601

Refer to Referenced Information for Qualifiers (if any) and Methodology.

Reference Information

Test Method References:

ALS Test Code	Matrix	Test Description	Method Reference**
CL-R511-WT	Soil	Chloride-O.Reg 153/04 (July 2011)	EPA 300.0
5 grams of dried soil is mixed with 10 grams of distilled water for a minimum of 30 minutes. The extract is filtered and analyzed by ion chromatography.			
Analysis conducted in accordance with the Protocol for Analytical Methods Used in the Assessment of Properties under Part XV.1 of the Environmental Protection Act (July 1, 2011).			
EC-WT	Soil	Conductivity (EC)	MOEE E3138
A representative subsample is tumbled with de-ionized (DI) water. The ratio of water to soil is 2:1 v/w. After tumbling the sample is then analyzed by a conductivity meter.			
Analysis conducted in accordance with the Protocol for Analytical Methods Used in the Assessment of Properties under Part XV.1 of the Environmental Protection Act (July 1, 2011).			
MOISTURE-WT	Soil	% Moisture	Gravimetric: Oven Dried
PH-WT	Soil	pH	MOEE E3137A
A minimum 10g portion of the sample is extracted with 20mL of 0.01M calcium chloride solution by shaking for at least 30 minutes. The aqueous layer is separated from the soil and then analyzed using a pH meter and electrode.			
Analysis conducted in accordance with the Protocol for Analytical Methods Used in the Assessment of Properties under Part XV.1 of the Environmental Protection Act (July 1, 2011).			
RESISTIVITY-CALC-WT	Soil	Resistivity Calculation	APHA 2510 B
Resistivity are calculated based on the conductivity using APHA 2510B where Conductivity is the inverse of Resistivity.			
RESISTIVITY-CALC-WT	Soil	Resistivity Calculation	MOECC E3138
Resistivity are calculated based on the conductivity using APHA 2510B where Conductivity is the inverse of Resistivity.			
SO4-WT	Soil	Sulphate	EPA 300.0

* ALS test methods may incorporate modifications from specified reference methods to improve performance.

The last two letters of the above test code(s) indicate the laboratory that performed analytical analysis for that test. Refer to the list below:

Laboratory Definition Code	Laboratory Location
WT	ALS ENVIRONMENTAL - WATERLOO, ONTARIO, CANADA

Chain of Custody Numbers:

GLOSSARY OF REPORT TERMS

Surrogates are compounds that are similar in behaviour to target analyte(s), but that do not normally occur in environmental samples. For applicable tests, surrogates are added to samples prior to analysis as a check on recovery. In reports that display the D.L. column, laboratory objectives for surrogates are listed there.

mg/kg - milligrams per kilogram based on dry weight of sample

mg/kg wwt - milligrams per kilogram based on wet weight of sample

mg/kg lwt - milligrams per kilogram based on lipid weight of sample

mg/L - unit of concentration based on volume, parts per million.

< - Less than.

D.L. - The reporting limit.

N/A - Result not available. Refer to qualifier code and definition for explanation.

Test results reported relate only to the samples as received by the laboratory.

UNLESS OTHERWISE STATED, ALL SAMPLES WERE RECEIVED IN ACCEPTABLE CONDITION.

Analytical results in unsigned test reports with the DRAFT watermark are subject to change, pending final QC review.

Quality Control Report

Workorder: L1851709

Report Date: 11-NOV-16

Page 1 of 2

Client: DST Thunder Bay
DST Consulting Engineers Inc. 1120 Premier Way , Suite 200
Thunder Bay ON P7B 0A3

Contact: Selorm Danku

Test	Matrix	Reference	Result	Qualifier	Units	RPD	Limit	Analyzed
CL-R511-WT	Soil							
Batch	R3592422							
WG2429128-4 CRM		AN-CRM-WT						
Chloride			88.8		%		70-130	09-NOV-16
WG2429128-3 LCS								
Chloride			99.5		%		80-120	09-NOV-16
WG2429128-1 MB								
Chloride			<5.0		ug/g		5	09-NOV-16
EC-WT	Soil							
Batch	R3591555							
WG2429395-1 LCS								
Conductivity			98.9		%		90-110	09-NOV-16
WG2429109-1 MB								
Conductivity			<0.0040		mS/cm		0.044	09-NOV-16
MOISTURE-WT	Soil							
Batch	R3589113							
WG2427118-2 LCS								
% Moisture			100.2		%		90-110	06-NOV-16
WG2427118-1 MB								
% Moisture			<0.10		%		0.1	06-NOV-16
PH-WT	Soil							
Batch	R3590446							
WG2427820-1 DUP		L1851709-1						
pH		7.78	7.70	J	pH units	0.08	0.3	08-NOV-16
WG2428294-1 LCS								
pH			7.00		pH units		6.7-7.3	08-NOV-16
SO4-WT	Soil							
Batch	R3590601							
WG2426086-3 CRM		AN-CRM-WT						
Sulphate			110.3		%		60-140	07-NOV-16
WG2426086-2 LCS								
Sulphate			99.5		%		80-120	07-NOV-16
WG2426086-1 MB								
Sulphate			<20		mg/kg		20	07-NOV-16

Quality Control Report

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Legend:

Limit	ALS Control Limit (Data Quality Objectives)
DUP	Duplicate
RPD	Relative Percent Difference
N/A	Not Available
LCS	Laboratory Control Sample
SRM	Standard Reference Material
MS	Matrix Spike
MSD	Matrix Spike Duplicate
ADE	Average Desorption Efficiency
MB	Method Blank
IRM	Internal Reference Material
CRM	Certified Reference Material
CCV	Continuing Calibration Verification
CVS	Calibration Verification Standard
LCSD	Laboratory Control Sample Duplicate

Sample Parameter Qualifier Definitions:

Qualifier	Description
J	Duplicate results and limits are expressed in terms of absolute difference.

Hold Time Exceedances:

All test results reported with this submission were conducted within ALS recommended hold times.

ALS recommended hold times may vary by province. They are assigned to meet known provincial and/or federal government requirements. In the absence of regulatory hold times, ALS establishes recommendations based on guidelines published by the US EPA, APHA Standard Methods, or Environment Canada (where available). For more information, please contact ALS.

The ALS Quality Control Report is provided to ALS clients upon request. ALS includes comprehensive QC checks with every analysis to ensure our high standards of quality are met. Each QC result has a known or expected target value, which is compared against pre-determined data quality objectives to provide confidence in the accuracy of associated test results.

Please note that this report may contain QC results from anonymous Sample Duplicates and Matrix Spikes that do not originate from this Work Order.