



FINAL

**FOUNDATION INVESTIGATION AND DESIGN REPORT
POULIN CREEK CULVERT REPLACEMENT
HIGHWAY 129
SAULT STE MARIE DISTRICT
AGREEMENT NO.: 5016-E-0001
SITE NO.: 46-326
GEOCRES NO. 410-23
GWP: 5230-05-01**

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Table of Contents

1. INTRODUCTION	1
2. SITE DESCRIPTION	1
3. REGIONAL GEOLOGY.....	4
4. INVESTIGATION PROCEDURES AND LABORATORY TESTING.....	5
5. DESCRIPTION OF SUBSURFACE CONDITIONS	8
5.1 Asphalt.....	11
5.2 Topsoil.....	11
5.3 Embankment Fill	11
5.4 Organic Sand.....	12
5.5 Organic Silt	12
5.6 Peat	12
5.7 Silt	12
5.8 Gravelly Sand	13
5.9 Bedrock	13
5.10 Dynamic Cone Penetration Testing (DCPT).....	14
5.11 Groundwater	15
5.12 Chemical Test Results	15
6. MISCELLANEOUS	16
7. GENERAL.....	17
7.1 Replacement Structure	18
7.2 Foundation Design.....	18
7.2.1 Foundation Design (Concrete Box Culvert)	20
7.2.2 Foundation Design (Open Footing Culvert)	21
7.3 Lateral Earth Loads and Sliding Resistances	21
7.4 Staged Construction	22
7.5 Earth Excavation.....	24
7.6 Excavation Support and Roadway Protection	24
7.7 Bedding	26
7.8 Backfill and Cover	27
7.9 Channel Diversion.....	27
7.10 Erosion Control	27

7.11 Frost Protection	28
7.12 Embankment.....	29
7.13 Retaining Walls.....	29
7.14 Construction Concerns	30
7.15 Chemical Testing	31
7.16 Seismic Design Considerations.....	32
8. CLOSURE	33
9. REFERENCES	34
10. LIMITATIONS OF REPORT.....	35

Appendices

LIMITATIONS OF REPORT.....	'A'
DESCRIPTION OF TERMS	'B'
DRAWINGS	'C'
ENCLOSURES	'D'
NON-STANDARD SPECIAL PROVISIONS (NSSP)	'E'
CHEMICAL TEST RESULTS.....	'F'

List of Tables

Table 4-1: Summary of Boreholes.....	7
Table 5-1: Summary of Subsurface Strata Encountered	10
Table 5-2: Summary of Sieve analysis- Embankment Fill.....	11
Table 5-3: Summary of Sieve analysis- Organic Sand	12
Table 5-4: Summary of Sieve analysis- Silt	13
Table 5-5: Summary of Sieve analysis- Sand.....	13
Table 5-6: Bedrock Conditions and Parameters	14
Table 5-7: Groundwater depth	15
Table 5-8: Chemical Test Results – Soil sample	16
Table 7-1: Geotechnical resistances (Concrete Box Culvert) for final condition.....	20
Table 7-2: Geotechnical resistances (Concrete Box Culvert) during construction.....	20
Table 7-3: Geotechnical resistances and reactions for open footing culverts during construction with footing base at Elevation 437.6 m to 436.4 m	21

Table 7-4: Typical Soil Parameters for Earth Loads *	22
Table 7-5: Lateral Earth Pressure Coefficients	22
Table 7-6: Advantages and Disadvantages of Roadway Protection Methods	25
Table 7-7: Additional requirements for concrete subjected to Sulphate Attack	32
Table 8-1: Advantages and Disadvantages of the Proposed Culvert Options	33

List of Figures

Figure 1.1 View of the existing culvert at Highway 129 (looking East Inlet)	2
Figure 1.2 View of the existing culvert location at Highway 129 (looking West)	3

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PART 1: FACTUAL INFORMATION

1. INTRODUCTION

DST Consulting Engineers Inc. (DST) has been retained by the Ministry of Transportation (MTO), 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 12.1 km north of Highway 667 (Latitude 47.70097, Longitude -83.31546), Station 14+078 (Township of Daoust) in the District of Sault Ste Marie.

The site is located at the southwest head of a shallow valley about 5 km long and trending north east to south west. The creek flows from south west to north east within a meandering channel. The highway runs in a south-east direction. 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 supervisor during the fieldwork investigation.

The existing 21.0 m long corrugated steel ellipse culvert is approximately 3.5 m in diameter. The existing culverts (Figure 2.1 to 2.2) built in 1975 as stated in the Ontario Structure Inspection Manual (OSIM), and inspection by others indicates light to severe corrosion, sagging and lifting of the barrel in middle and on ends, perforations at east and west ends, localized areas throughout barrel, and possible settlements at mid-length.

The embankment height, as measured from the existing road level to the proposed culvert inlet, at the culvert location is approximately 5.3 m and the side slope of the embankment is approximately 3H: 1V.



Figure 2.1 View of the existing culvert at Highway 129 (looking East Inlet)



Figure 2.2 View of the existing culvert location at Highway 129 (looking West)

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 centre 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 are constructed very easily in eolian terrain, but the erosion potential is high once the protective vegetation cover has been removed. A high water table may limit excavation in places, and slumping and piping can occur. Driven well points will supply enough water for domestic use in eolian deposits below the water table. The pollution potential is 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. from December 16, 2014 to January 6 and 16, 2015. (Preliminary Foundation Investigation and Design Report for Poulin Creek Culvert Replacement, Highway 129, Township of Daoust, Algoma District, Ontario. Assignment No. 5013-E-0040, G.W.P. 5222-05-00, Site # 46-326/C, WP NO. 5230-05-01, GEOCREs No. 410-18, Peto MacCallum Ltd., (21 Sept 2016). Four boreholes (PCN-1, PCN-2, PCN-3 and PCN-4) were advanced to depths between 2.1 m to 7.2 m. DCPT testing was also undertaken at PCN-2 and PCN-3 from 5.2 m to 7.2 m and 5.2 m to 6.6 m respectively. This information was provided by MTO, and is included in this report as supplementary information. The borehole logs are included in Appendix D (Enclosures).

A site visit was undertaken by one of DST's professional engineers on July 27th and July 28th, 2016 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 between August 25th and August 30th, 2016 utilizing a CME 550 drill rig equipped for geotechnical drilling and operated by Landcore Drilling from Chelmsford, Ontario. A total of five boreholes (BH1, BH2, BH3, BH4, BH5) were advanced to depths ranging from 7.6 m to 15.4 m. The minimum number and depth of the boreholes were as specified by the Ministry of Transportation (MTO). Difficult drilling conditions were experienced at all borehole locations. Rockfill was confirmed at the base of the embankment fill at borehole BH3, and numerous cobbles and boulders were encountered in the underlying gravelly sand, which necessitated the use of wash boring techniques to advance the boreholes. At borehole BH3, the casing and drill rods were sheared off in cobbles and boulders at a depth of 7.6m, and three split spoons were rendered inoperative during SPT testing. Half of the lost casing was retrieved from the borehole, however the remaining 1.6 m length could not be recovered, and was left in the ground. The boreholes were partially backfilled with cement grout, prior to casing extraction. All of the boreholes experienced partial collapse after casing extraction, and prior to backfill with bentonite chips.

In the case of the two boreholes located off the embankment (BH1 and BH2), 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 the water level. The pipes were thereafter removed, before leaving the site.

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 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 detail of borehole locations and depths.

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. 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: Summary of Boreholes

Borehole ID	Station	Location	Ground Surface Elevation (m)	Depth (m)	Offset (m)	Completion Details
BH1	14+087	Outlet	440.6	10.7	15.6 Rt	Borehole backfilled with cement-grout from bottom of hole to caving depth followed by bentonite chips up to ground surface.
BH2	14+087	Inlet	441.2	11.3	13.2 Lt	Borehole backfilled with cement-grout from bottom of hole to caving depth followed by bentonite chips up to ground surface.
BH3	14+072	Roadway	444.1	7.6	1.8 Rt	Borehole backfilled with bentonite chips from bottom of the hole to road base then backfilled with granular material and asphalt cold patch to
BH4	14+084	Roadway	444.0	15.4	1.9 Lt	Borehole backfilled with cement grout from bottom of the hole to caving depth followed by bentonite chips to road base then backfilled with granular material and asphalt cold patch to surface.
BH5	14+073	Roadway	444.1	15.3	1.9 Lt	Borehole backfilled with cement grout from bottom of the hole to caving depth followed by bentonite chips to road base then backfilled with granular material and asphalt cold patch to surface.
PCN-1		Inlet	441.0	2.1	-	Borehole backfilled with bentonite/cement grout in accordance with the MTO guidelines and MOE Regulation 903 for borehole abandonment procedures.
PCN-2		Roadway	443.9	7.2	-	Borehole backfilled with bentonite/cement grout in accordance with the MTO guidelines and MOE Regulation 903 for borehole abandonment procedures.
PCN-3		Roadway	444.0	6.6	-	Borehole backfilled with bentonite/cement grout in accordance with the MTO guidelines and MOE Regulation 903 for borehole abandonment procedures.
PCN-4		Outlet	442.0	5.3	-	Borehole backfilled with bentonite/cement grout in accordance with the MTO guidelines and MOE Regulation 903 for borehole abandonment procedures.

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 thirty-eight (38) natural moisture content determinations, eleven (11) sieve analysis of aggregates (LS 602) sieve analysis tests and one (1) set of chemical tests has been carried out for this assignment. Laboratory test results are presented on the borehole 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 testing results where available.

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

- Asphalt at surface in boreholes BH3, BH4 and BH5; PCN-2 and PCN-3. Embankment fill at surface in Borehole BH1, peat at surface in PCN-4 and topsoil at surface in Borehole BH2.
- Embankment fill (Boreholes BH1, BH2, BH3, BH4 and BH5; PCN-2 and PCN-3) comprising of sand and gravel with trace silt to sandy silt. Occasional cobbles, boulders and rockfill were encountered in this layer.
- Underlain by a loose to compact organic sand and very loose to loose organic sandy silt to compact silt, and locally peat in Boreholes BH1, BH2 and PCN-4 respectively.
- Underlying the organic soil and embankment fill, there are discontinuous pockets of compact to dense silty sand to sandy silt. These are encountered at boreholes BH5, PCN-1 and PCN-4
- Underlain by a compact to dense gravelly sand layer in Boreholes BH1, BH2, BH3, BH4 and PCN-4. Numerous cobbles and boulders and possible rockfill (Borehole BH3) were encountered in this deposit.
- Granodiorite bedrock was confirmed in the deeper Boreholes BH1, BH2 and BH4.

Borehole BH3 was terminated early due to the loss of a damaged casing and core barrel at a depth of 7.6 m.

Attempts to undertake SPT testing within the embankment fill and underlying gravelly sand, were frequently obstructed by the presence of cobbles and boulders. Some tests at boreholes were unsuccessful due to bouncing of the SPT hammer on the rockfill, cobbles and boulders. Due to the difficult drilling conditions, the drilling method was changed to wash boring, to advance the borehole through the rockfill, cobbles and boulders.

A combined summary of the strata encountered during both field investigations is provided 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	444.1 – 440.0	0.1	N/A	N/A
	BH4	0.0 – 0.1	440.0 – 443.9	0.1		
	BH5	0.0 – 0.1	444.1 – 444.0	0.1		
	PCN-2	0.0-0.18	443.9 – 443.7	0.18		
	PCN-3	0.0 – 0.18	444.0 – 443.8	0.18		
Embankment Fill / Pavement Structure / Rockfill	BH1	0.0 – 1.5	440.6 – 439.1	1.5	6	Loose to very dense
	BH2	0.1 – 0.8	441.1 - 440.4	0.7	36	
	BH3	0.1 – 6.1	444.0 - 438.0	6.0	9-44	
	BH4	0.1 – 6.3	443.9 - 437.7	6.2	18-48	
	BH5	0.1 – 6.1	444.0 - 438.0	6.0	5-61	
	PCN-2	0.18 – 5.2	443.7 - 438.7	5.0	10-28	
	PCN-3	0.18 – 5.2	443.8 - 438.8	5.0	35	
Peat / Organic Soil	BH1	1.5 – 2.5	439.1 – 438.1	1.0	10	Very loose to loose
	BH2	0.8 – 3.2	440.4 – 438.0	2.4	2-5	
	PCN-4	0.5 – 2.1	441.5 – 439.9	1.6	2-3	
Silt with Sand	BH2	3.2 – 4.6	438.0 – 431.8	1.4	13 - 28	Compact to dense
	BH5	6.1 -12.3	438.0 – 431.8	6.2	32-40	
	PCN-4	2.1- 3.8	439.9 – 436.1	1.7	23-30	
Gravelly Sand with Silt	BH1	2.5 – 7.7	438.1 – 432.9	5.2	24 – 41	Compact to dense
	BH2	4.6 – 7.8	436.6 – 433.4	3.2	40 – 50	
	BH3	6.1 – 7.6	438.0 – 436.5	1.5	33	
	BH4	6.3 – 10.7	437.7 – 433.3	4.4	26 – 28	
	PCN-4	3.8 – 5.3	436.1 – 436.7	1.5	27 - 28	
Bedrock	BH1	7.7 – 10.7	432.9 – 429.9	3.0	N/A	Strong to very strong
	BH2	7.8 – 11.3	433.4 – 429.9	3.5		
	BH4	10.7 – 15.4	433.3 – 428.6	4.7		
	BH5	12.3 – 15.3	431.8 – 428.8	3.0		
Cobbles and Boulders	BH1	2.3 – 2.5	438.3 – 438.1	0.2	-	-
	BH1	5.0 -7.7	435.6 – 432.9	2.7	100+	*
	BH2	6.5 – 6.9	434.7 – 434.3	0.4	-	-
	BH3	0.8 – 1.5	443.3 – 442.6	0.7	-	-
	BH3	4.3 – 6.1	439.8 – 438.0	1.8	100+	*
	BH3	7.3 – 7.6	436.8 – 436.5	0.3	-	-
	BH4	1.0 – 3.4	443.0 – 440.6	2.4	100+	*
	BH4	4.3 – 5.2	439.7 – 438.8	0.9	-	-
	BH4	6.6 – 7.3	437.4 – 436.7	0.7	-	-
	BH4	8.5 – 8.8	435.5 – 435.2	0.3	-	-
	BH4	9.7 – 10.7	434.3 – 433.3	1.0	-	-
	BH5	2.3 – 3.4	441.8 – 440.7	1.1	-	-
	BH5	3.8 – 4.7	440.3 – 439.4	0.9	100+	-
	BH5	5.5 – 6.1	438.6 – 438.0	0.6	-	*
	BH5	7.0 – 7.3	437.1 – 436.8	0.3	-	-
	BH5	8.5 – 8.8	435.6 – 435.3	0.3	-	-
	BH5	9.1 – 9.7	435.0 – 434.4	0.6	100+	-
	BH5	10.7 – 11.7	433.4 - 432.4	1.0	100+	*
	PCN-1	2.1 – 2.4	438.9 – 438.6	0.3	-	*
	PCN-2	3.4 – 4.4	440.5 – 439.5	1.0	-	-

- 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
- The interpretation of cobbles and boulders is made in the field by drillers/supervisors observations, and a review of the SPT information where available. No direct observations have been made.

Details of the soil strata at the culvert location are presented on the borehole logs, and a brief description of each soil unit with laboratory test results is summarized below.

5.1 Asphalt

Asphalt was encountered at surface in Boreholes BH3, BH4, BH5, PCN-2 and PCN-3 with a thickness of 100 to 180 mm.

5.2 Topsoil

Topsoil was encountered at surface in Borehole 2 with a thickness of 100 mm.

5.3 Embankment Fill

Embankment fill consisting of brown, Sand and Gravel to gravelly Sand, with trace of silt was encountered at surface in borehole BH1, below topsoil in Borehole BH2 and below asphalt in boreholes BH3, BH4 and BH5 with thicknesses of 0.7 to 6.0 m. SPT 'N' values in the embankment fill layer vary from 5 to 61, indicating a loose to very dense condition. The natural moisture contents of samples tested ranged between 4 % and 22 %. The laboratory test results are summarized in Table 5-2.

The PML boreholes PCN-2 and PCN-3 confirmed the presence of Embankment fill comprising of pavement fill (sand and gravel) and rockfill and sandy gravel, under asphalt in boreholes PCN-2 and PCN-3 with a thickness of 5.2 m, and underlain by a thin layer of sandy silt to gravelly sand with organic at the base of the embankment.

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 42
Sand %	42 to 60
Fines %	9 to 58

5.4 Organic Sand

A dark grey to black, moist fine organic sand with silt and some gravel was encountered below the embankment fill layer in borehole BH1 with a thickness of 1.0 m. The SPT 'N' value for this layer is 10, indicating a loose to compact condition. The natural moisture content of one sample tested was 65 %. Wood fragments, cobbles and boulders were encountered within this layer. The laboratory test results are summarized in Table 5-3.

Table 5-3: Summary of Sieve analysis- Organic Sand

Laboratory Results – Sieve analysis	
Gravel %	20
Sand %	58
Fines %	22

5.5 Organic Silt

A black to grey, moist, fine organic sandy silt layer was encountered below the embankment fill layer in borehole 2 with a thickness of 2.4 m. This layer becomes sandy to silty and less organics with increasing in depth. The SPT 'N' values for this layer vary between 2 and 5, indicating a soft to firm consistency. The natural moisture content of samples tested ranged between 36 and 119 %. The laboratory test results indicated 10% organic matter in this layer.

5.6 Peat

A dark brown fibrous peat deposit was encountered only in borehole PCN-4 located near the outlet of the culvert at surface with a thickness of 1.6 m (Elev. 439.9 m). The natural moisture content determined from the representative sample by PML was about 70%.

5.7 Silt

A grey, moist silt, with sand to trace of sand, and some gravel to trace of gravel was encountered below the organic silt layer in borehole BH2 with a thickness of 1.4 m, and below the embankment fill layer in borehole BH5 with a thickness of 6.2 m. The silt at borehole BH2 is non-plastic, while the silt at borehole BH5 is described as being of low plasticity. SPT 'N' values for this layer vary from 13 to 40, indicating a compact to dense condition. The natural moisture contents of samples tested ranged between 8 % and 26 %. Numerous cobbles and boulders were encountered within this layer in Borehole 5. The laboratory test results are summarized in Table 5-4.

Table 5-4: Summary of Sieve analysis- Silt

Laboratory Results – Sieve analysis	
Gravel %	0 to 14
Sand %	3 to 23
Fines %	62 to 97

5.8 Gravelly Sand

A brown to grey, sand, gravelly to trace gravel, with silt to trace silt was encountered in boreholes BH1, BH2, BH3 and BH4 below organic sand and/or silt and fill layer. The thickness of this layer ranged from 3.2 m to 4.6 m in boreholes BH1, BH2, and BH4 and was undetermined in borehole BH3 due to termination within this layer. SPT 'N' values for this layer vary from 24 to 41, indicating a compact to dense condition. The natural moisture contents of samples tested ranged between 7 % and 21 %. Frequent cobbles and boulders were encountered within this layer. The laboratory test results are summarized in Table 5-5.

The PML borehole PCN-4 confirmed the presence of compact gravelly sand underlying the sandy silt layer with a thickness of 1.5 m.

Table 5-5: Summary of Sieve analysis- Sand

Laboratory Results – Sieve analysis	
Gravel %	15 to 37
Sand %	47 to 74
Fines %	9 to 29

5.9 Bedrock

Bedrock was encountered in four boreholes, BH1, BH2, BH4 and BH5 at depths of 7.7 m (Elev. 432.9 m), 7.8 m (Elev. 433.4 m), 10.7 m (Elev. 433.3 m) and 12.3 m (Elev. 431.8 m) respectively.

The bedrock comprises of Granodiorite which has been subjected to intense shearing, brecciation and local intrusion by fine grained dykes. The bedrock encountered in the boreholes may be generally described as pinkish grey, spotted black, white, pink and grey, coarse grained, strong to very strong, fresh unweathered, Granodiorite with closely to widely spaced, tight to partly open, rough planar, rough undulating and smooth planar, clean to surface stained, and manganese oxide and /or chlorite coated joints, dipping at 0°-10°, 10°-20°, 30°-40°, 50°-60°, and 70°-80°, and locally subvertical, locally highly fractured with >20 joints/m, and locally intruded by pink and greyish brown fine grained dykes. The rock core photos and point load test results can be seen

in Appendix D (Enclosures).

Total Core Recovery (TCR) in all boreholes BH1, BH2, BH4 and BH5 was good at 80% to 100%. Solid Core Recovery (SCR) varied from poor (32%) to excellent (100%). Rock Quality Designation (RQD) varied from very poor (12%) to excellent (93%). The mechanical indices are provided in Table 5.6 below, and on the borehole logs in Appendix D.

Point Load tests were carried out in accordance with Suggested Methods for Determining Point Load Strength, ISRM 1985. Three (3) point load tests were undertaken on selected core samples taken from boreholes BH1, BH2 and BH4, one from each borehole. A Point Load Strength conversion factor of 21 was used to convert the Point Load test results to unconfined compressive strength estimates.

The unconfined compressive strength of rock, estimated from the Point Load Tests, was between 250 MPa at borehole BH1, to 174 MPa at borehole BH4 indicating a Very Strong rock. A summary of the bedrock conditions encountered in the boreholes, and the parameters determined in the laboratory are provided in Table 5-6 below.

Table 5-6: Bedrock Conditions and Parameters

ID	Bedrock Depth (m)	Bedrock Elevation (m)	Core Length (m)	TCR (%)	SCR (%)	RQD (%)	Rock ¹ Quality	Point Load Index	UCS Strength (MPa)	Strength ² Classification
BH1	7.7	432.9	3.0	100-93	100-87	78-93	Good to Excellent	11.9	250	R5 Very Strong
BH2	7.8	433.4	3.5	100	93-74	93-32	Excellent to Poor	11.4	240	R5 Very Strong
BH4	10.7	433.3	4.7	100-80	60-32	57-12	Poor to Very Poor	8.3	174	R5 Very Strong
BH5	12.3	431.8	3.0	100-93	92-63	75-38	Good to Poor	-	-	-

¹After Table 3.10 CFEM 2006

²After Table 3.5 CFEM 2006

5.10 Dynamic Cone Penetration Testing (DCPT)

DCPTs were advanced during geotechnical investigation by PML at boreholes PCN-2 and PCN-3 at depths between 5.2 m and 7.2 m (Elev. 438.7 m and 436.7 m) and 5.2 m and 6.6 m (Elev. 438.8 and 437.4 m) respectively. It was described as probable sand with a compact to dense condition.

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.

5.11 Groundwater

Groundwater 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 (BH1 and BH2), and read at 24 hours after installation with final readings taken on completion of fieldwork on September 19, 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 five boreholes (see Table 5-7). The water level in the creek was measured at elevations 441.0 m and 440.2 m at the inlet and outlet locations respectively, at the time of the DST field investigation. The groundwater levels can be expected to vary both with the season, and with local precipitation events.

Table 5-7: Groundwater depth

Borehole And Location	Measured Water Levels (Depth/Elevation) m				
	After Drilling*		24 Hours Reading*		September 19, 2016*
BH1 (outlet)	Aug 26, 2016	0.3 (440.1)	Aug 27, 2016	0.5 (440.3)	0.4 (440.2)
BH2 (inlet)	Aug 25, 2016	0.9 (440.3)	Aug 26, 2016	1.0 (440.4)	0.9 (440.3)
BH3	Aug 27, 2016	5.0 (439.1)	-		-
BH4	Aug 30, 2016	3.7 (440.3)	-		-
BH5	Aug 28, 2016	6.3 (437.8)	-		-
PCN-1	Jan 16, 2015	0.5 (440.5)	-		-
PCN-2	Dec 16, 2014	4.8 (439.1)	-		-
PCN-4	Jan 16, 2015	0.5 (441.5)	-		-

Note: *0.3(440.1) – depth, m (elevation, m)

5.12 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 5-8 and discussed in Section 7.15. Copies of the Laboratory Certificate of Analyses are provided in Appendix 'F'.

Table 5-8: Chemical Test Results – Soil sample

Sample ID	Sulphate (mg/kg)	Chloride (mg/kg)	pH	Conductivity (mS/cm)	Resistivity (ohm - cm)
BH4 @ 7.6 m depth	28	20.5	7.81	0.184	5450

6. MISCELLANEOUS

The Peto MacCallum Ltd. Fieldwork was undertaken during December 2014 to January 2015. DST site work was carried out on August 25th to August 30th, 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, 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.

**FOUNDATION INVESTIGATION AND DESIGN REPORT
POULIN CREEK CULVERT REPLACEMENT
HIGHWAY 129
SAULT STE MARIE DISTRICT
AGREEMENT NO.: 5016-E-0001
SITE NO.: 46-326
GEOCRES NO. 410-23
GWP: 5230-05-01**

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 5.3 m of granular embankment fill over compact to very dense sand with cobbles and boulders. At one location (BH5) this deposit is replaced by a dense silt, also with cobbles and boulders. These materials overly bedrock at about 11m below top of road. Peat and organic soil were found outside the embankment, extending to a level 5 to 6 m below top of roadway. The water table is near creek level.

The existing culvert will be replaced by a single box culvert expected to be 4.2 m in width and 30 m in length. For these conditions, and given that neither the embankment grade nor the culvert will be raised or widened beyond existing, a shallow foundation is considered suitable for this site. As the cross sectional area of the proposed culvert is larger than the existing culvert, the overall effect on the culvert foundation soils can be considered to be negligible due to the expected applied loads being smaller than the existing loads. 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 Box Culvert is the preferred option as a replacement structure. However, geotechnical recommendations for an open footing have also been provided in this report, as requested by the Ministry.

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

Top of Highway:	444.1
Creek level (at time of investigation):	441.0
New culvert invert:	438.6
Approx. base of 0.5 m thick culvert bedding:	437.8
Excavation level for 2.2 m frost penetration below concrete:	436.1
Deepest level of organic materials encountered in boreholes:	438.0

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 as the existing structure. 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 and these should be removed. Similarly, the inorganic materials at the excavation, which include wet sand and silt, will be very susceptible to disturbance by equipment. Construction in the dry will require dewatering of the soil. It is important to note that without adequate dewatering the foundation soils will be permanently rendered 'disturbed' to considerable depth from groundwater effects such as heaves and boils. The disturbed ground would be unsuitable for support of the structure and also 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 or Type II 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.

The soil underlying the site has zones which are considered frost susceptible and are within potential frost penetration depth below the culvert. The need to remove this material will depend on the structure's ability (both structurally and hydraulically) to accommodate potential differential frost heave (see Section 7.11). The need for subexcavation of frost susceptible materials can be deleted if (a) minimum creek flows are always enough to prevent freezing of the creek bed, or (b) the culvert can accommodate considerable seasonal differential heave (for example 200 or 300mm at one end) without adverse structural or hydraulic performance.

If sub-excavation is carried out in the dry (with adequate dewatering controls), any unsuitable material can be replaced with Granular A or Granular B Type I or Type II 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". Achieving the specified compaction over the saturated sands at this site is expected to be difficult.

A suitable alternative to the 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 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 engineered fill supporting foundations, such as bedding or fill replacing frost susceptible soil, 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 any additional width requirements for access, dewatering equipment and culvert sidefill.

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

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 also 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 and 7.2 below.

Our analyses assume that the entire culvert will be rigid enough to act as a stiff structure from end to end and side to side. The highest factored resistance at ULS (Table 7-1) applies to the final condition when all backfill has been replaced to the top of the road. A lower factored geotechnical resistance at ULS applies during construction, before backfill is placed (Table 7.2).

Table 7-1: Geotechnical resistances (Concrete Box Culvert) for final condition

Footing Elevation (m)	Footing Width (m)	Factored Geotechnical Resistance at ULS (kPa)	Factored Geotechnical Resistance at SLS (kPa)
438.6	B = 4.2	690	150

Table 7-2: Geotechnical resistances (Concrete Box Culvert) during construction.

Footing Elevation (m)	Footing Width (m)	Factored Geotechnical Resistance at ULS (kPa)	Factored Geotechnical Resistance at SLS (kPa)
438.6	B = 4.2	150	150

Groundwater levels were recorded in the boreholes at elevations between 437.8 m and 440.3 m during the field investigation, and with a creek level at 441.0 m. However, the actual groundwater table is expected to be close to or just above the prevailing creek level, which will fluctuate both with seasonal changes, local precipitation events. 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 geotechnical resistance was estimated for the ultimate limit state (ULS) and serviceability limit state (SLS) for a maximum settlement of 25 mm. The resistance at ULS was calculated by applying a load resistance factor of 0.5 according to the Canadian Highway Bridge Design Code (CHBDC) CAN/CSA-S6-14 section 6.6, and is shown in Table 7-2. The geotechnical resistance is estimated based on a strip footing 1 m wide with a length equal to 30 m situated at Elev. 437.6 m to 436.4 m) (see Table 7-3).

Table 7-3: Geotechnical resistances and reactions for open footing culverts during construction with footing base at Elevation 437.6 m to 436.4 m

Footing Width (m)	Soil Cover (m)	Founding Elevation (m)	Factored Geotechnical Resistance at ULS (kPa)	Factored Geotechnical Reaction at SLS (kPa)
B=1.0	1.0	437.6	125	125
	1.5	437.1	170	170
	2.2	436.4	230	230

*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.

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 7-4 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 7-4 and Table 7-5, 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 7-4: 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	35	22.3
Peat	12	28	18.5
Organic Sand	18	28	18.5
Organic Silt	18	28	18.5
Sand	20	33	21.8
Silt	19	33	21.8

*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 7-5: Lateral Earth Pressure Coefficients

Earth Pressure Coefficient	Equation*	Fill-Sand & Gravel*	Peat*	Organic Sand*	Organic Silt*	Sand*	Silt*
Active Earth Pressure (K _a)	$\left(\frac{1 - \sin\phi}{1 + \sin\phi}\right)$	0.27	0.36	0.36	0.36	0.30	0.30
Passive Earth Pressure (K _p)	$\left(\frac{1 + \sin\phi}{1 - \sin\phi}\right)$	3.65	2.8	2.8	2.8	3.35	3.35
At rest (K ₀)	$(1 - \sin\phi)$	0.43	0.53	0.53	0.53	0.46	0.46

* φ is an angle of internal friction

7.4 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.

Temporary road widening will involve fill placement either over the 2.4 m of peat and/or organic soil or replacing the layer with fill. In the former case (fill over peat or organic soil), 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 or organic soil). 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 case where organics are subexcavated, even a greater degree of settlement of the adjacent existing fill may occur. Regardless, all fill placed for a temporary widening should be removed prior to culvert placement on that side to avoid ongoing embankment settlement induced by the widening.

Use of a temporary soldier pile wall with lagging may be considered for all or part of the excavation sides. The presence of cobbles and boulders within the underlying natural soils at the site, is a constraint on effective installation of a vertical retaining wall system.

An excavation depth of up to approximately 6 m (Elev. 438 m) for the box culvert option may be required for the staged construction. The existing embankment slopes should be reinstated thereafter 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 at the anticipated depth 6.0 m (Elev. 438.0 m) were evaluated based on a fully dewatered slope and base. The results of the slope stability analyses indicate that the minimum factor of safety for a suitably dewatered excavation is 1.46 at 2H:1V slope and 1.15 at 1.5H:1V. 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, base disturbance from uncontrolled hydraulic pressures compaction difficulties, 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 variable fines content indicated by the laboratory analyses (and the corresponding hydraulic conductivity variability) for this site, the dewatering design will likely require well points with sand surround. The need for any pressure relief wells within the excavation footprint will also need to be considered in the design, particularly given the variable nature of the sand/silt deposits.

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 be 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

Peat and any other organic soils, 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".

Excavations for this project should be constructed in accordance with the requirements of the Occupational Health and Safety Act of Ontario (OHSA). According to O.Reg. 213/91, s.226, the soils in the area of interest classify as Type 3 (the granular fill above the water table) and Type 4 (the wet fill, native sand/silt and organic soils below the water table). These should be assessed and confirmed in the field as construction progresses, and slope design based on the soil with the highest number.

7.6 Excavation Support and Roadway Protection

An appropriate roadway protection system should be selected by the contractor and designed by the contractor's qualified engineer in accordance with OPS 539 "Construction Specification for Temporary Protection Systems". Potential roadway protection systems here include a steel sheet pile wall, a soldier pile wall with lagging, and a temporary dewatered excavation using side slopes of 2H:1V. A concrete or gabion block gravity wall would only be suitable for support of the upper part of the excavation above the level of dewatering or water table. The sheet pile wall and soldier

pile wall options may have some difficulty to extreme difficulty in achieving the required installation depths as a result of cobbles and boulders embedded in the subsurface soils. Driven soldier piles will have a better chance of penetrating such soils than augered soldier piles, which will in turn, have a better chance of success than sheet piles. The latter are not considered feasible here.

The advantages and disadvantages of using two temporary excavation support and roadway protection systems, namely soldier piles and cut slopes are shown in Table 7-6. The cut slope option is recommended.

Table 7-6: Advantages and Disadvantages of Roadway Protection Methods

Roadway Protection Option	Advantages	Disadvantages
Soldier Pile Retaining Walls with concrete or timber lagging	<ul style="list-style-type: none"> • Robust solution • Relatively impermeable if properly installed, and effectively grouted • Good erosion control capacity • Ease of culvert installation when working below the ground water table. • Can be designed with suitable factor of safety • Safer working area for construction • Small footprint 	<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 • Stronger bracing/anchors required with increasing excavation depth
Feasibility	<ul style="list-style-type: none"> • Driven heavy H-piles may be feasible. Groundwater control must be adequate 	
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 • Soils are susceptible to erosion • dewatering with sumps/trenches not feasible • Flatter slope required with increasing excavation depth. • Requires a larger excavation area • For construction in the wet, base preparation and geotextile placement are difficult
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 7-4 and 7-5, 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. The contractor should be alerted of the presence of cobbles and boulders within the sand and silt material below the embankment fill 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.

7.7 Bedding

Construction of a precast concrete box culvert and bedding should follow the provisions in OPSS 422 "Precast Concrete Box Culvert". The base of bedding materials will be in an excavation below the groundwater table. The bedding will be placed directly on adequately prepared fill materials placed as outlined in other sections herein. In addition, the bedding for the structure should be designed in accordance with Section 7.8 of the 2014 CHBDC and MTO D 803.021 "Bedding and Backfill for Precast Concrete Box Culvert".

The bedding should be a minimum of 500 mm thick and extend to a minimum horizontal distance beyond all sides of the culvert. equal to the bedding thickness The bedding material should consist of "Granular A or Granular B Type I or Type II" 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 I or Type II" should meet the requirements of OPSS.PROV 1010. The "Granular A or Granular B Type I or Type II" should be compacted to a minimum of 98 % of standard Proctor maximum dry density. The middle one-third of the culvert width of the top bedding layer should be loose for a depth of 75 mm.

A suitable alternative to conventional compacted granular bedding is 20 mm clear stone with a non-woven 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 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 culvert foundation and side support after construction.

7.8 Backfill and 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 channel. during construction, with a cofferdam at each end. Its design will need to incorporate the weak peat materials at either end of the new culvert. Both a low earth berm over the peat (with provisions for stability and settlement) and a sheet pile cut-off are expected to be feasible.

Depending on precipitation and the season, the amount of water flow through the creek may vary considerably. The contractor should be alerted to potentially high creek water levels and surface water runoff, 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 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”, OPSD 810.010 “General Rip-Rap Layout for Sewer and Culvert Outlets, OPSS 511 “Construction Specification for Rip-Rap, Rock Protection, and Granular Sheeting” and OPSS 1004 “Material Specifications for Aggregates – Miscellaneous”.

To prevent undermining of the bedding, 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 full width and 1.0 m below the culvert as a minimum.

The temporary erosion and sedimentation measures installed and maintained during the construction of the culvert should meet the requirements of 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 about 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 heaving).

Frost protection within the 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 both frost susceptible and non-frost susceptible soils. During winter season, ice may form inside the culvert and a low flow rate may assist ice formation to the culvert invert. Frost may then extend into the soils below the culverts, generating ice lenses, possibly as deep as 2.25 m below any concrete exposed to the elements. 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 caused by a given soil 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 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. 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 2.25 m below concrete and replacement with non-frost-susceptible material, 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 an equivalent material 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 2.44 m beyond all sides of the buried structure.

If sub-excavation for frost effects is carried out in the dry (with adequate dewatering controls), the existing soil can be replaced with Granular B Type 1 material compacted to 95% of standard Proctor maximum dry density. If the excavation is in the wet (water is maintained at or above the adjacent groundwater table) then the material should be clear stone surrounded by geotextile, without the need for compaction.

7.12 Embankment

The embankment slopes should be reinstated with a slope not steeper than 2H: 1V if being constructed with compacted granular materials. 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 the embankment footprint at the ends of the culvert. The foundation parameters provided above are applicable for the retaining walls 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.

Stiff braced retaining walls may be designed using an “at rest” earth pressure coefficient (K_0) and soil unit weights as listed in Tables 7-4 and 7-5. The lateral soil pressure distribution (σ_h) may be assumed to increase linearly with depth according to:

$$\sigma_h = K_0 (\sigma_z + q)$$

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 7-4)
 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 unbalanced hydrostatic (water) conditions adjacent to the below-grade walls. Backfill should, therefore, consist of clean, free-draining granular material. 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 I or Type II materials. This controlled compacted zone should extend a distance laterally at least equal to twice the depth 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 involve removal of cover/embankment materials, staged removal of the existing culvert, temporary roadway protection, diversion of the creek, subsurface dewatering, excavation, base preparation, placement of foundation fill and bedding, culvert placement, backfilling and reinstatement of the embankment fill and removal of any temporary widening.

Particularly challenging issues for this site are the frequent occurrence of cobbles and boulders within the soil profile, groundwater control, excavation support for road protection, and suitably replacing any subexcavated materials (such as peat) below the foundations.

To confirm subsurface conditions are as anticipated, a geotechnical engineer should inspect the excavation base as well as the foundation and surrounding soils before installation of fill, bedding and other backfills.

7.15 Chemical Testing

The results of the analytical testing conducted on a soil samples at the site location have been presented in Table 5-8 and also included in Appendix F. The suite of 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 chemical test results for the soil sample were compared with applicable Canadian Standards Association (CSA) standards, as shown in Table 7-7 below. The chemical sulphate content analyses for a representative soil sample tested indicate a sulphate concentration of 28 mg/kg (0.0028 %) in soil. The results were compared with Canadian Standards Association (CSA) Standard A23.1 for sulphate attack potential on concrete structures and indicate a “negligible” risk for sulphate attack on concrete material. Accordingly, conventional GU or MS Portland cement may be used in the construction of the proposed concrete elements.

The pH values for the soil and water samples were reported as 7.27, 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. These results were 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 present negligible concrete corrosion potential. Resistivity and conductivity was found to be 5450 ohm-cm and 0.184 mS / cm respectively for the samples analysed from Borehole BH4 at depth of 7.6 m.

Table 7-7: 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 the granular soils was estimated using N_{60} values interpreted from the in-situ SPT data applying Seed and Idriss (1971). The analysis confirmed that liquefaction is not a concern at this site, with a factor of safety against liquefaction of greater than 1.9.

The site was also classified for seismic response 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 (< 760 m/s), and an average standard penetration resistance ($N_{60} > 50$), the project site is classified as "Class C" (Table 4.1.8.4 A, NBC 2015). For seismic design purposes, the site coefficients of Section 4.3.4.3.3 of the 2014 Canadian Highway Bridge Code should be used.

8. CLOSURE

The detailed design of this project should be reviewed with respect to the applicability of the subsurface information and design recommendations presented herein.

Table 8-1 below provides a comparison of alternate culvert options, and summarizes the advantages and disadvantages of each.

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 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	increase construction time. Requires foundation excavation and preparation. Requires roadway protection system
Risk/Consequences	In general terms low risk option (except for shoring)	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

Building Research Establishment (BRE) Digest 363 (SD1 - 2005), UK *“Concrete in aggressive ground”*

Canadian Foundation Engineering Manual. 2006. Fourth Edition, Canadian Geotechnical Society.
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C. (1959)

Municipal and Provincial Common, Volume 1 - General and Construction Specifications, *“Ontario Provincial Standard for Roads and Public Works”* Spec No. OPSS 422, 511, 517, 518, 805, 902.

Municipal and Provincial Common, Volume 2 - Material Specifications, *“Ontario Provincial Standard for Roads and Public Works”* Spec No. OPSS 1860.

Municipal and Provincial Common, Volume 3 - Drawings for Roads, Barriers, Drainage, Sanitary Sewers, Water mains and Structures, *“Ontario Provincial Standard for Roads and Public Works”* Spec No. OPSD 203.040, 803.010, 803.030, 803.031, 810.010, 810.020, 3090.100.

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 Poulin Creek Culvert Replacement, Highway 129, Township of Daoust, Algoma District, Ontario. Assignment No. 5013-E-0040, G.W.P. 5222-05-00, Site # 46-326/C, WP NO. 5230-05-01, GEOCREC No. 410-18, Peto MacCallum Ltd., (21 Sept 2016)

Provincial-Orientated, Volume 5 - MTO General Conditions of Contract and General and Construction Specifications, *“Ontario Provincial Standard for Roads and Public Works”* Spec No. OPSS.PROV 209, 501, 510, 539.

Provincial-Orientated, Volume 6 - Material Specifications, *“Ontario Provincial Standard for Roads and Public Works”* Spec No. OPSS.PROV 1004, 1010.

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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:

Reviewed by:

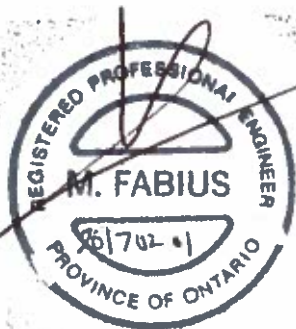


Selorm Danku P. Eng.
Geotechnical Engineer

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, Senior Principal

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

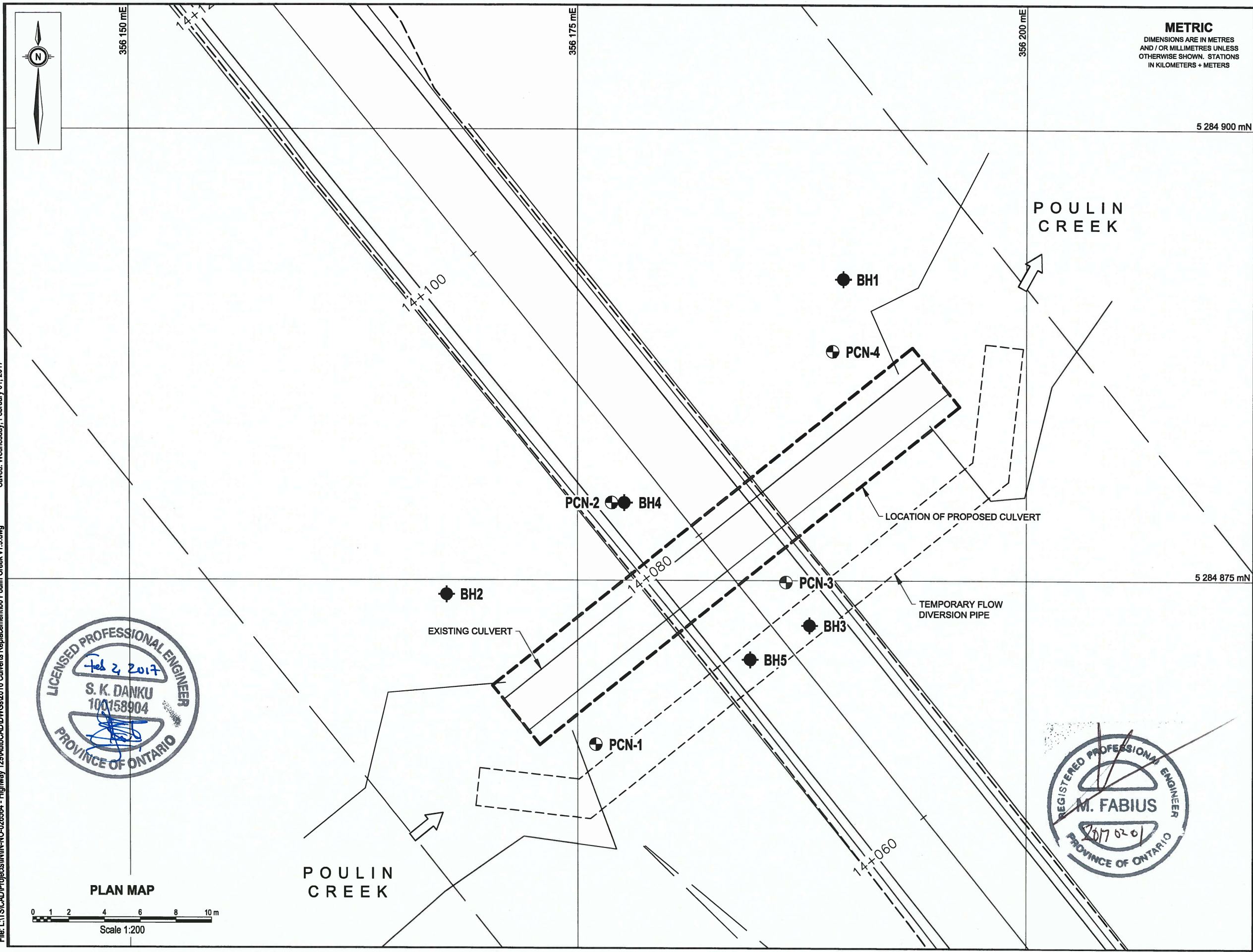
SS	SPLIT SPOON SAMPLE	WS	WASH SAMPLE
TW	THIN WALL SHELBY TUBE SAMPLE	AS	AUGER (GRAB) SAMPLE
PH	SAMPLER ADVANCED BY HYDRAULIC PRESSURE	TP	THIN WALL PISTON SAMPLE
WH	SAMPLER ADVANCED BY SELF STATIC WEIGHT	PM	SAMPLER ADVANCED BY MANUAL PRESSURE
SC	SOIL CORE	RC	ROCK CORE
	WATER LEVEL	$SENSITIVITY = \frac{UNDISTURBED\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

*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

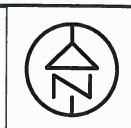
DRAWINGS

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

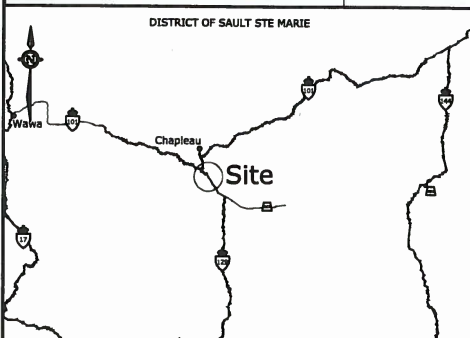
AG NO 5016-E-0001
WP NO 5230-05-01
SITE NO 46-326
GEOCRES NO 410-23



**CULVERT REPLACEMENT
POULIN CREEK CULVERT**
STA 14+049 TO STA 14+118
Survey _____ Revised _____

**SHEET
1**

BOREHOLE LOCATIONS



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

LEGEND

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

No.	Elev. (m)	MTM Zone 16		Survey	
		North (m)	East (m)	Station	Offset
BH1	440.60	5284891.7	356189.8	14+087	15.6 m Rt
BH2	441.20	5284874.2	356167.7	14+087	13.2 m Lt
BH3	444.10	5284872.5	356187.9	14+072	1.8 m Rt
BH4	444.00	5284879.3	356177.6	14+084	1.9 m Lt
BH5	444.10	5284870.6	356184.6	14+073	1.9 m Lt
PCN-1	444.0	5284865.9	356176.0	-	-
PCN-2	443.9	5284879.3	356176.9	-	-
PCN-3	441.9	5284874.9	356186.6	-	-
PCN-4	442.0	5284887.7	356189.2	-	-

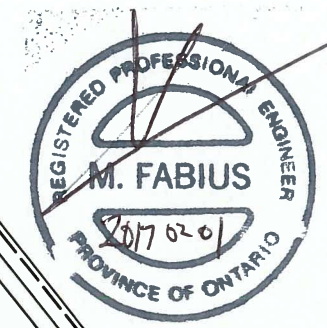
REV	DATE	ISSUE	DRAWN BY	CHECKED	APPROVAL
A	06-Nov-16	DRAFT	RW	SA	MK
B	29-Nov-16	DRAFT	RW	SA / POS	MK
C	17-Jan-17	DRAFT	RW	SA / POS	MF

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.

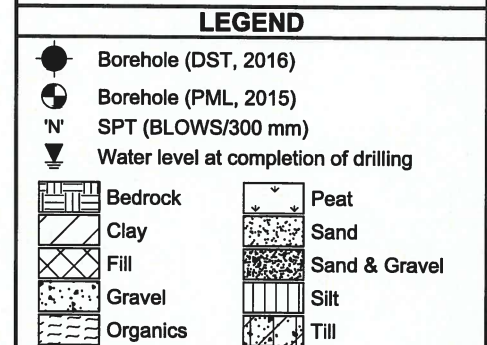
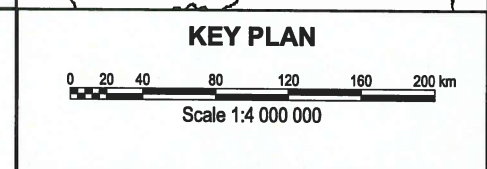
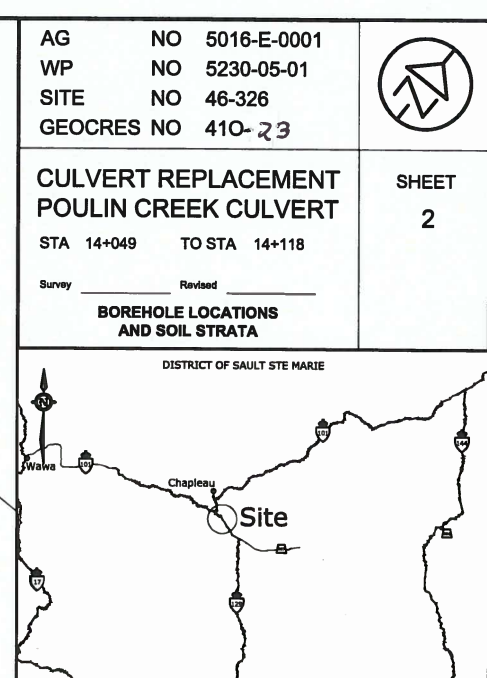
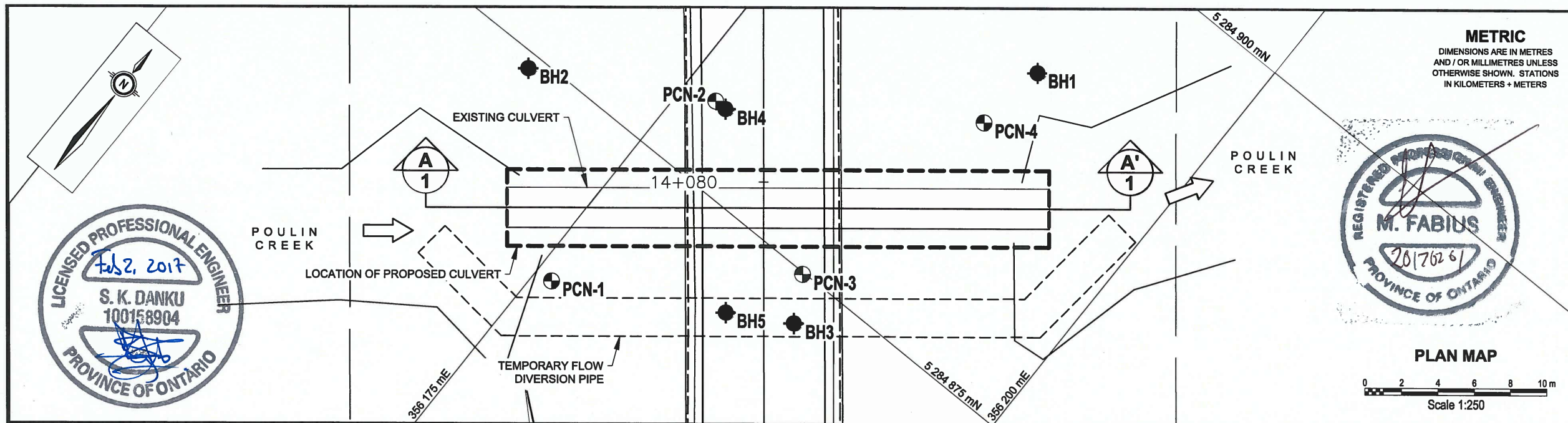


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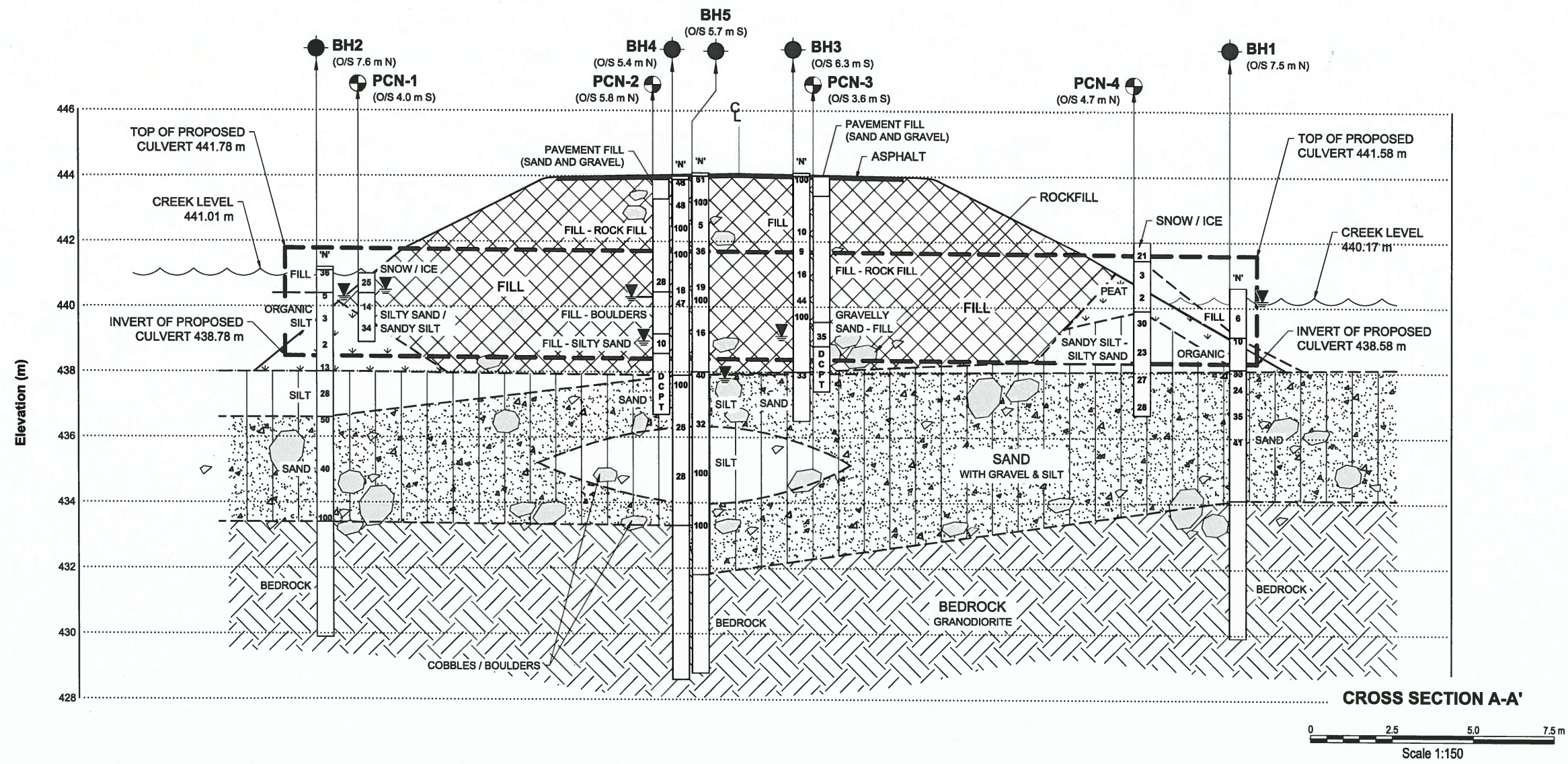
No.	Elev. (m)	MTM Zone 16		Survey	
		North (m)	East (m)	Station	Offset
BH1	440.60	5284891.7	356189.8	14+087	15.6 m Rt
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BH3	444.10	5284872.5	356187.9	14+072	1.8 m Rt
BH4	444.00	5284879.3	356177.6	14+084	1.9 m Lt
BH5	444.10	5284870.6	356184.6	14+073	1.9 m Lt
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PCN-2	443.9	5284879.3	356176.9	-	-
PCN-3	441.9	5284874.9	356186.6	-	-
PCN-4	442.0	5284887.7	356189.2	-	-

REV	DATE	ISSUE	DRAWN BY	CHECKED	APPROVAL
A	06-Nov-16	DRAFT	RW	SA	MK
B	29-Nov-16	DRAFT	RW	SA / POS	MK
C	17-Jan-17	DRAFT	RW	SA / 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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Appendix D
ENCLOSURES

METRIC

[illegible]

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

RECORD OF BOREHOLE No BH2

1 OF 1

METRIC

G.W.P. 5230-05-01 LOCATION POULIN CREEK ORIGINATED BY KN
 DIST S.S. MARIE HWY 129 BOREHOLE TYPE WASHBORE COMPILED BY SA
 DATUM GEODETIC DATE 2016 08 25 - 2016 08 25 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 (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				
								WATER CONTENT (%)				
441.2	GROUND SURFACE											
441.1	TOPSOIL		SS1	SS	36							
440.4	FILL - GRAVEL, some SAND dense, moist, angular to subangular grey, fine to coarse											
0.8	ORGANIC SILT, some to trace CLAY, trace to some SAND soft, becoming soft to firm from 3.0 m to 3.2 m, moist, non-plastic black to grey		SS2	SS	5							
			SS3	SS	3						112	
			SS4	SS	2						119	
438.0			SS5	SS	13							
3.2	SILT, trace SAND compact, moist, non-plastic grey		SS6	SS	28							
436.6	SAND, trace to some GRAVEL, trace to some SILT dense, moist, subangular brown to grey, fine to coarse		SS7	SS	50							
4.6			SS8	SS	40							
	cobbles and boulders from 6.5 m to 6.9 m											
433.4			SS9	SS	100+							
7.8	BEDROCK		RC1	RC								
	Pinkish grey spotted black, white, pink and grey, fresh, unweathered, strong to very strong, coarse grained, GRANODIORITE with closely to widely spaced, tight to partly open, rough planar to smooth planar, rough undulating, clean to surface stained, and manganese oxide and/or chlorite coated joints, dipping at 0°-10°, 10°-20°, 30°-40° and 50°-60° with occasional brecciation and quartz infill		RC2	RC								
			RC3	RC								
429.9												
11.3	END OF BOREHOLE at 11.3 m											

ONL MOT-HIGH VANES POULIN CREEK GPJ DATA TEMPLATE.GDT 1/12/16

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

ENCLOSURE 2

RECORD OF BOREHOLE No BH3

1 OF 1

METRIC

G.W.P. 5230-05-01 LOCATION POULIN CREEK ORIGINATED BY KN
 DIST S.S. MARIE HWY 129 BOREHOLE TYPE WASHBORE COMPILED BY SA
 DATUM GEODETIC DATE 2016 08 26 - 2016 08 27 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									
								20	40	60	80						100
						○ UNCONFINED + FIELD VANE □ QUICK TRIAXIAL × LAB VANE											
444.1	GROUND SURFACE																
444.0	ASPHALT		SS1	SS	100+											31 60 (9)	
0.1	FILL - SAND, GRAVELLY, trace SILT compact to dense, moist, subangular brown, fine to coarse cobbles and boulders from 0.8 m to 1.5 m															100+ blows for 0.2 m penetration	
				SS	10											No sample recovery	
			SS2	SS	9												
			SS3	SS	16												
			SS4	SS	44												
			SS5	SS	100+											100+ blows for 0.2 m penetration	
	rockfill from 4.3 m to 6.1 m															Water level measured after 30 minutes of the end of drilling	
438.0																	
6.1	SAND, some GRAVEL, trace SILT compact, moist, subangular grey, medium to coarse		SS6	SS	33											17 74 (9)	
	cobbles and boulders from 7.3 m to 7.6 m																
436.5																	
7.6	END OF BOREHOLE at 7.6 m															Casing and core barrel broke at 7.6 m	

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

ENCLOSURE 3

RECORD OF BOREHOLE No BH4

1 OF 1

METRIC

G.W.P. 5230-05-01 LOCATION POULIN CREEK ORIGINATED BY KN
 DIST S.S. MARIE HWY 129 BOREHOLE TYPE WASHBORE COMPILED BY SA
 DATUM GEODETIC DATE 2016 08 29 - 2016 08 30 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 (%) GR SA SI CL		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
								20	40	60			80	100
								20 40 60 80 100					20 40 60 80 100	
444.0	GROUND SURFACE													
443.9	ASPHALT		SS1	SS	48									
443.8	FILL - SAND and GRAVEL, trace SILT dense to compact, dry to moist, angular brown to grey, fine to coarse		SS2	SS	48									
				SS	100+									
	cobbles and boulders from 1.0 m to 3.4 m			SS	100+									
			SS3	SS	18									
			SS4	SS	47									
	cobbles and boulders from 4.3 m to 5.2 m													
437.7			SS5	SS	100+									
6.3	SAND, GRAVELLY to some GRAVEL, trace to some SILT compact, wet, angular to subangular grey, fine to coarse cobbles and boulders from 6.6 m to 7.3 m		SS6	SS	26									
	cobbles and boulders from 8.5 m to 8.8 m		SS7	SS	28									
	cobbles and boulders from 9.7 m to 10.7 m													
433.3														
10.7	BEDROCK		RC1	RC										
	Pinkish grey spotted black, white, pink and grey, fresh, unweathered, strong to very strong, coarse grained, GRANODIORITE with closely to widely spaced, tight to partly open, rough planar to smooth planar, rough undulating, clean to surface stained, and manganese oxide and/or chlorite coated joints, dipping at 0°-10°, 10°-20°, 30°-40°, 50°-60° and subvertical with occasional brecciation and quartz infill, locally highly fractured, >20/m		RC2	RC										
			RC3	RC										
428.6														
15.4	END OF BOREHOLE at 15.4 m													

ONL MOT-HIGH VANES POULIN CREEK GPJ DATA TEMPLATE.GDT 1/12/16

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

ENCLOSURE 4

RECORD OF BOREHOLE No BH5

1 OF 1

METRIC

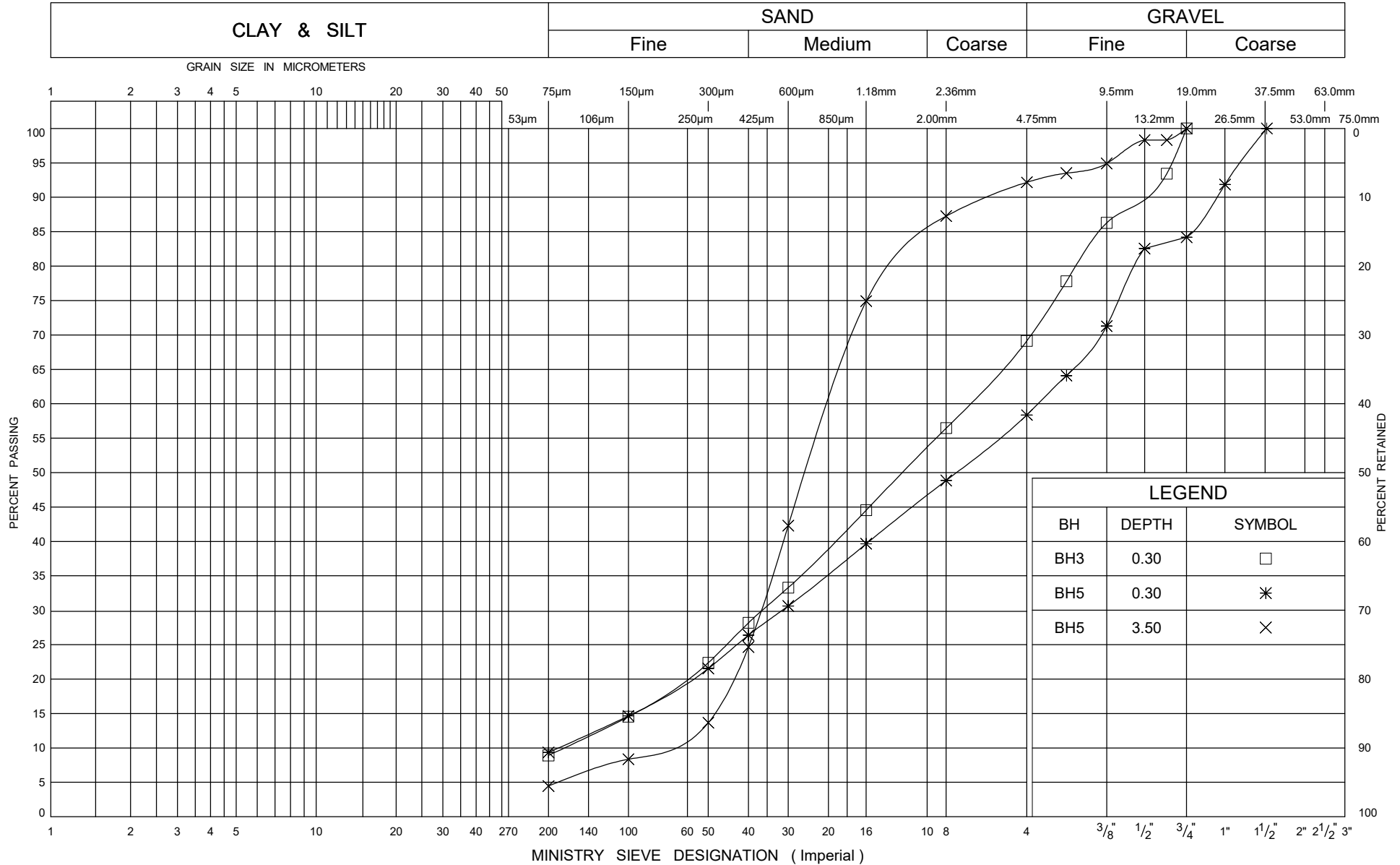
G.W.P. 5230-05-01 LOCATION POULIN CREEK ORIGINATED BY KN
 DIST S.S. MARIE HWY 129 BOREHOLE TYPE WASHBORE COMPILED BY SA
 DATUM GEODETIC DATE 2016 08 27 - 2016 08 28 CHECKED BY MK/POS

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				
								WATER CONTENT (%)				
444.1	GROUND SURFACE											
444.0	ASPHALT											
0.1	FILL - SAND and GRAVEL to trace GRAVEL, trace SILT very dense to compact, moist, angular grey, fine to coarse		SS1	SS	61							42 49 (9)
			SS2	SS	100+							100+ blows for 0.2 m penetration
			SS3	SS	5							
				SS	36							No sample recovery
	cobbles and boulders from 2.3 m to 3.4 m											
			SS4	SS	19							8 88 (4)
			SS5	SS	100+							100+ blows for 0.1 m penetration
	cobbles and boulders from 3.8 m to 4.7 m											
			SS6	SS	16							
	cobbles and boulders from 5.5 m to 6.1 m											
438.0												
6.1	SILT, with to some SAND, some to trace GRAVEL dense, moist, low plasticity grey		SS7	SS	40							15 23 (62)
	cobbles and boulders from 7.0 m to 7.3 m											Water level measured after 30 minutes of the end of drilling
			SS8	SS	32							
	cobbles and boulders from 8.5 m to 8.8 m											
	cobbles and boulders from 9.1 m to 9.7 m		SS9	SS	100+							100+ blows for 0.2 m penetration
			SS10	SS	100+							100+ blows for 0.2 m penetration
	cobbles and boulders from 10.7 m to 11.7 m											
431.8												
12.3	BEDROCK											
	Pinkish grey spotted black, white, pink and grey, fresh, unweathered, strong to very strong, coarse grained, GRANODIORITE with closely to widely spaced, tight to partly open, rough planar to smooth planar, rough undulating, clean to surface stained, and manganese oxide and/or chlorite coated joints, dipping at 0°-10°, 10°-20°, 30°-40° and 50°-60° with occasional brecciation and quartz infill, locally with pink and greyish-brown fine grained intrusive dykes		RC1	RC								TCR=100% SCR=63% RQD=38%
			RC2	RC								TCR=93% SCR=92% RQD=75%
428.8												
15.3	END OF BOREHOLE at 15.3 m											

ONL MOT-HIGH VANES POULIN CREEK GPJ DATA TEMPLATE.GDT 1/12/16

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

UNIFIED SOIL CLASSIFICATION SYSTEM



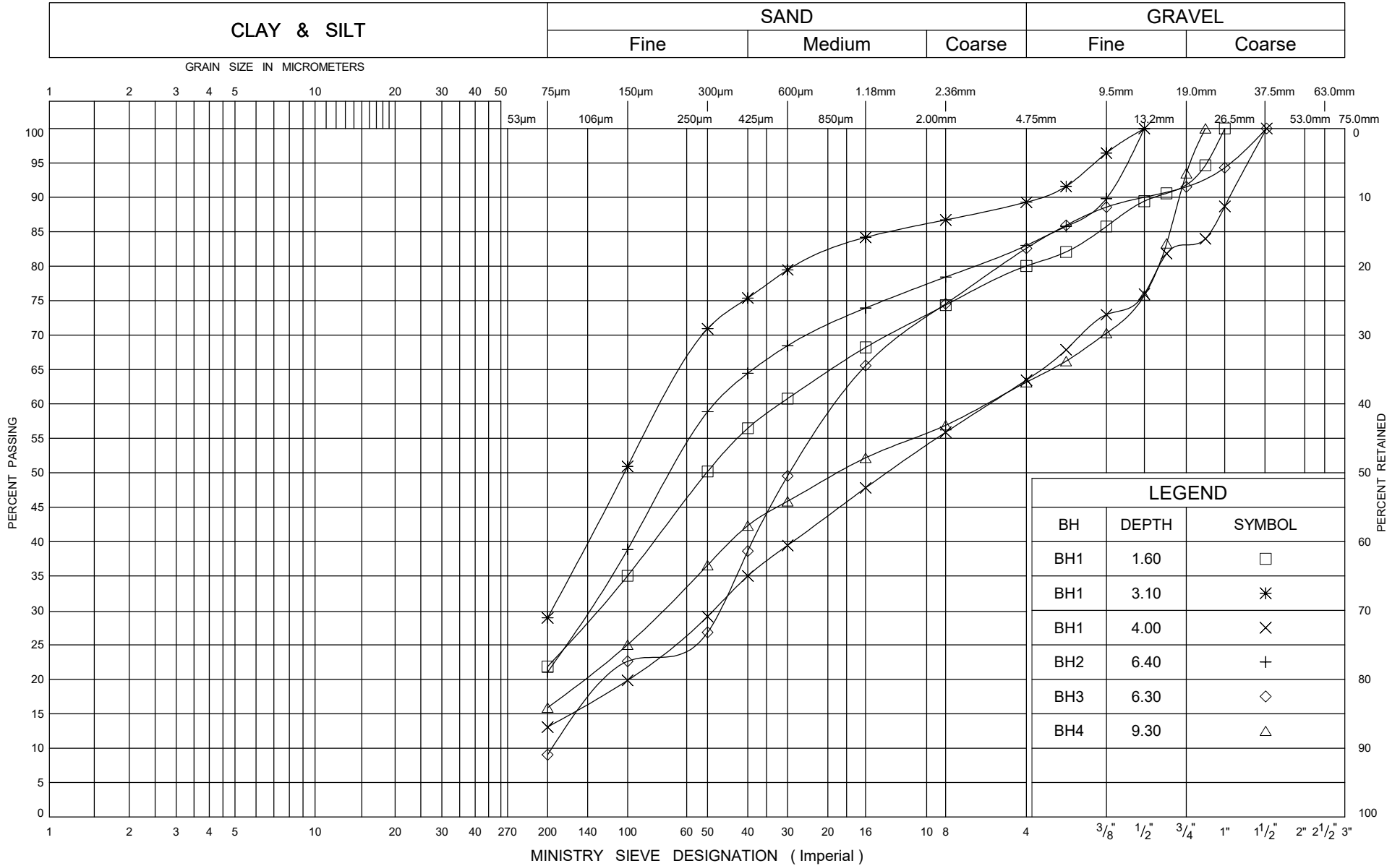
GRAIN SIZE DISTRIBUTION
FILL

ENCLOSURE 6

G.W.P. # 5230-05-01

POULIN CREEK

UNIFIED SOIL CLASSIFICATION SYSTEM



GRAIN SIZE DISTRIBUTION
SAND

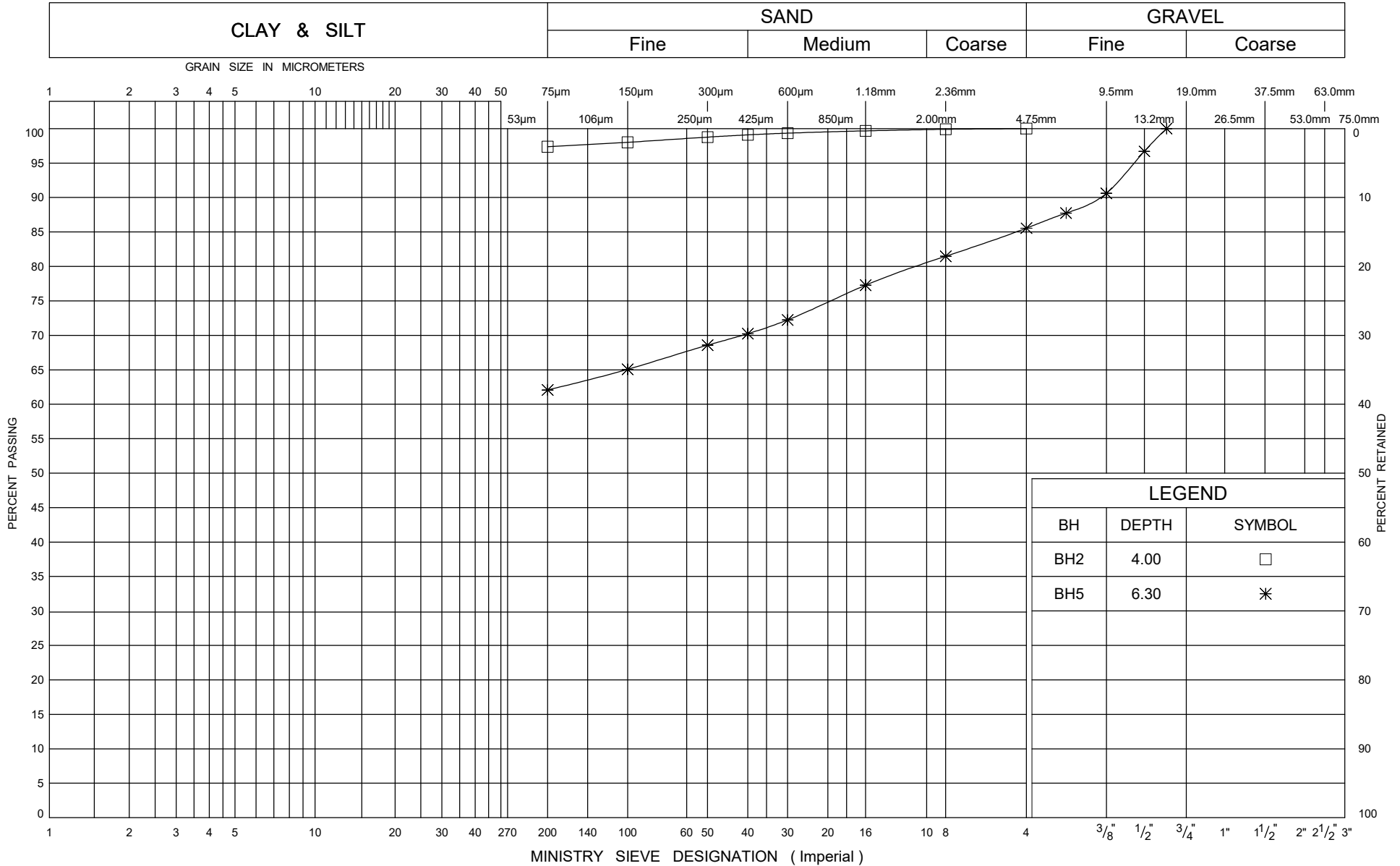
ENCLOSURE 7

G.W.P. # 5230-05-01

POULIN CREEK



UNIFIED SOIL CLASSIFICATION SYSTEM



GRAIN SIZE DISTRIBUTION
SILT

ENCLOSURE 8

G.W.P. # 5230-05-01

POULIN CREEK

RECORD OF BOREHOLE No PCN-1

1 of 1

METRIC

G.W.P. 5222-05-00 LOCATION Poulin Creek Coords: 5 285 664.0 N; 326 267.0 E ORIGINATED BY F.P.
DIST Algoma HWY 129 BOREHOLE TYPE Tripod and Casing COMPILED BY M.Kh.
DATUM Geodetic DATE January 16, 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 (%)			GR	SA	SI	CL
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL	× LAB VANE	20					40	60	80				
441.0	Ground Surface																						
0.0	Snow and ice		1	SS	25	▽* ▼*	440																
440.5	Sandy silt to silty sand trace clay																						
0.5	Compact Grey Wet to dense		2	SS	14																		
438.9			3	SS	34		439																
2.1	End of borehole																						
	Casing refusal on probable boulders																						

RECORD OF BOREHOLE No PCN-2

1 of 1

METRIC

G.W.P. 5222-05-00

LOCATION

Poulin Creek

Coords: 5 284 879.3 N; 356 176.9 E

ORIGINATED BY F.P.

DIST Algoma HWY 129

BOREHOLE

TYPE C.F.S.S.A. + Casing and Dynamic Cone Penetration Test

COMPILED BY M.Kh.

DATUM Geodetic

DATE _____

December 16, 2014

CHECKED BY M.V.

[illegible]

RECORD OF BOREHOLE No PCN-3

1 of 1

METRIC

G.W.P. 5222-05-00

LOCATION

Poulin Creek

Coords: 5 284 874.9 N; 356 186.6 E

ORIGINATED BY F.P.

DIST Alqoma HWY 129

BOREHOLE

TYPE C.F.S.S.A. + Casing + Dynamic Cone Penetration Test

COMPILED BY M.Kh.

DATUM Geodetic

DATE _____

January 06, 2015

CHECKED BY M.V.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS *	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
444.0	Ground Surface																			
0.0	180mm asphalt over sand and gravel (PAVEMENT FILL)																			
443.4																				
0.6	Rockfill consisting of 100 mm to 150 mm particles (ROCKFILL)																			

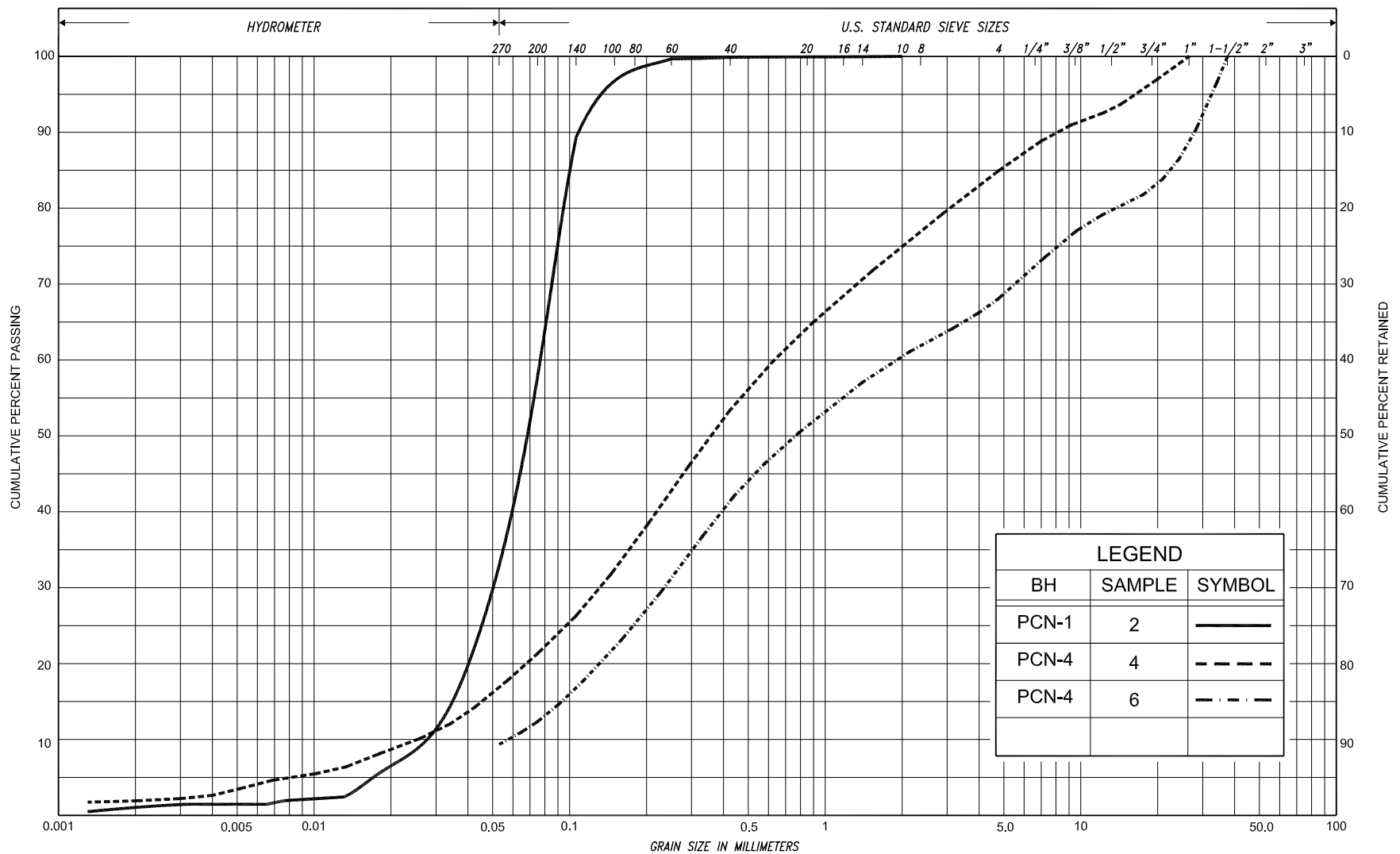
RECORD OF BOREHOLE No PCN-4

1 of 1

METRIC

G.W.P.	5222-05-00	LOCATION	<div> <div>Foulin Creek</div> <div>Coords: 5 285 684.0 N; 326 280.0 E</div> </div>	ORIGINATED BY	F.P.
DIST	Algoma	HWY	129	BOREHOLE TYPE	Tripod and Casing
DATUM	Geodetic	DATE	January 16, 2015	CHECKED BY	M.V.
COMPILED BY M.Kh.					

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		
442.0	Ground Surface																			
0.0	Snow and ice		1	SS	21	▽*	▼*													
441.5	Peat, fine fibrous																			
0.5	Dark brown		2	SS	3															
			3	SS	2										○					
439.9	Sandy silt to silty sand trace clay																			
2.1	Compact Grey Moist		4	SS	30							○				15 64 19 2				
			5	SS	23															
	mixed sand and gravel		6	SS	27							○				32 56 12 0				
			7	SS	28															
436.7	End of borehole																			
5.3																				
						</														



SILT & CLAY					FINE		MEDIUM		COARSE	GRAVEL			COBBLES	UNIFIED		
CLAY	FINE		MEDIUM		COARSE	FINE		MEDIUM		COARSE	GRAVEL			COBBLES	M.I.T.	
	SILT					SAND										
CLAY		SILT			V. FINE	FINE	MED.	COARSE		GRAVEL						U.S. BUREAU
					SAND											



GRAIN SIZE DISTRIBUTION

SANDY SILT TO SILTY SAND, trace to some gravel, trace clay

FIG No. PCN-GS-1

HWY: 129

G.W.P. No. 5222-05-00

POINT LOAD TEST RESULTS (diametric and axial)

PROJECT: Poulin Creek

JOB NO.: IN-NO-026564


This spreadsheet is based on information from 'Suggested Method for Determining Point Load Strength',

International Society for Rock Mechanics Commission on Testing Methods, 1985.


* Valid or Invalid based on description of break according to Fig 4 from 'Suggested Method for Determining Point Load Strength'

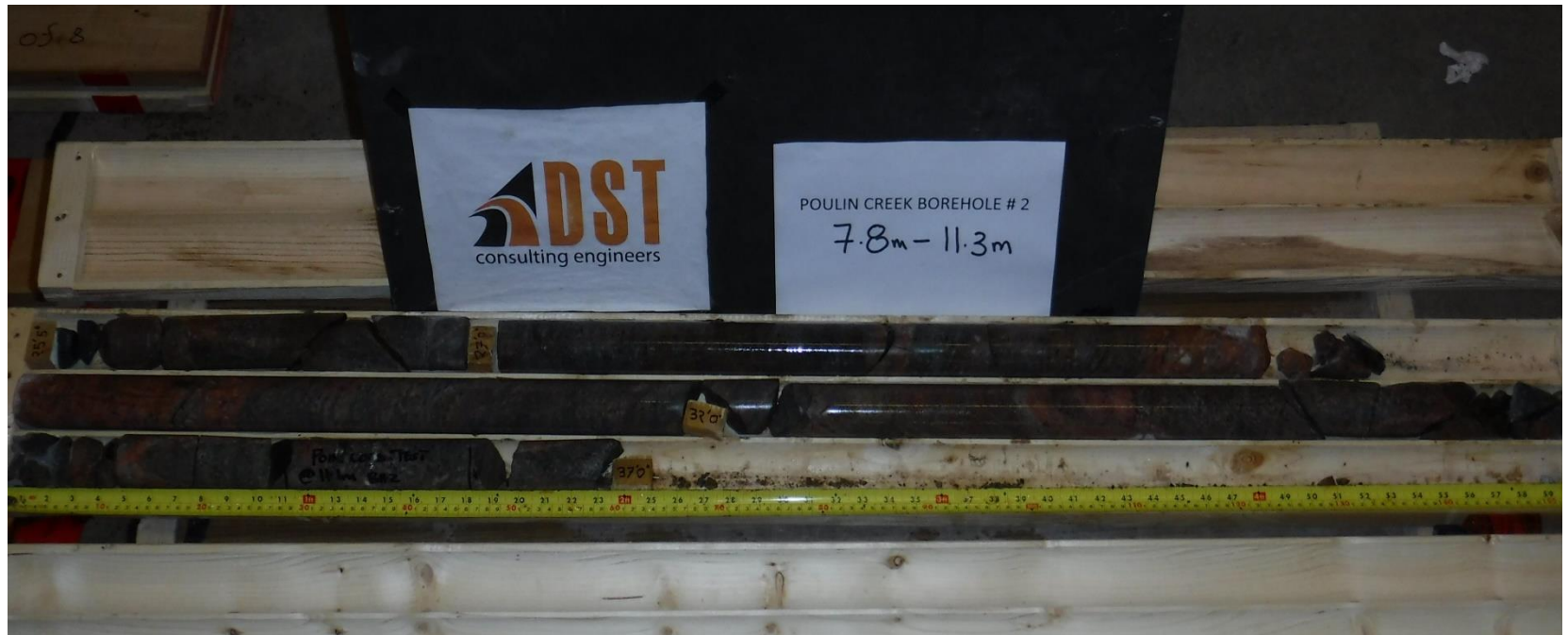
I_s = uncorrected point load strength	$D_e^2 = D^2$ for diametral tests	F = size correction factor
P = load	$D_e^2 = 4A/\pi$ for axial tests	$F = (D_e/50)^{0.45}$ or Fig. 7 from 'Suggested Method for Determining Point Load Strength'
D_e = equivalent core diameter	where A= WD	F = SQRT($D_e/50$) for tests near the standard (50 mm) size
	$I_s = P/D_e^2$	Size Correction $I_{s(50)} = F \times I_s$
Uniaxial Compressive Strength = $C_o = 21 \times I_s$ (50)		
21 is from: "Using the Point Load Test to Determine the Uniaxial Compressive Strength of Coal Measure Rock", Peng SS, Mark C, eds. Proceedings of the 19th International Conference on Ground Control in Mining. Morgantown, WV: West Virginia University.		


Borehole #	Test No.	Depth (m)	Test Type	*Valid or Invalid	W(mm) (enter for axial only)	D(mm)	Load P P(lbf)	Load P (kN)	$I_s = P/D_e^2$ (MPa)	F	$I_{s(50)}$ (MPa)	Uniaxial Compressive Strength (MPa)
BH1	PL#1	8.90	DIAM	V		47.4	6170		12.22	0.98	11.93	250
BH2	PL#1	11.10	DIAM	V		47.4	5920		11.72	0.98	11.44	240
BH4	PL#1	11.40	DIAM	V		47.4	4280		8.47	0.98	8.27	174
										Min.	8.3	174
										Avg.	10.5	221
										Max.	11.9	250

	Project	Hwy 129 - 4 Culverts Replacement AG # 5016-E-0001	Borehole Number	BH1		
	Project Number	IN-NO-026564	Box Number	1	of	1
	Client	MTO	Depth (m)	7.7	to	10.7




	Project	Hwy 129 - 4 Culverts Replacement AG # 5016-E-0001	Borehole Number	BH2		
	Project Number	IN-NO-026564	Box Number	1	of	1
	Client	MTO	Depth (m)	7.8	to	11.3




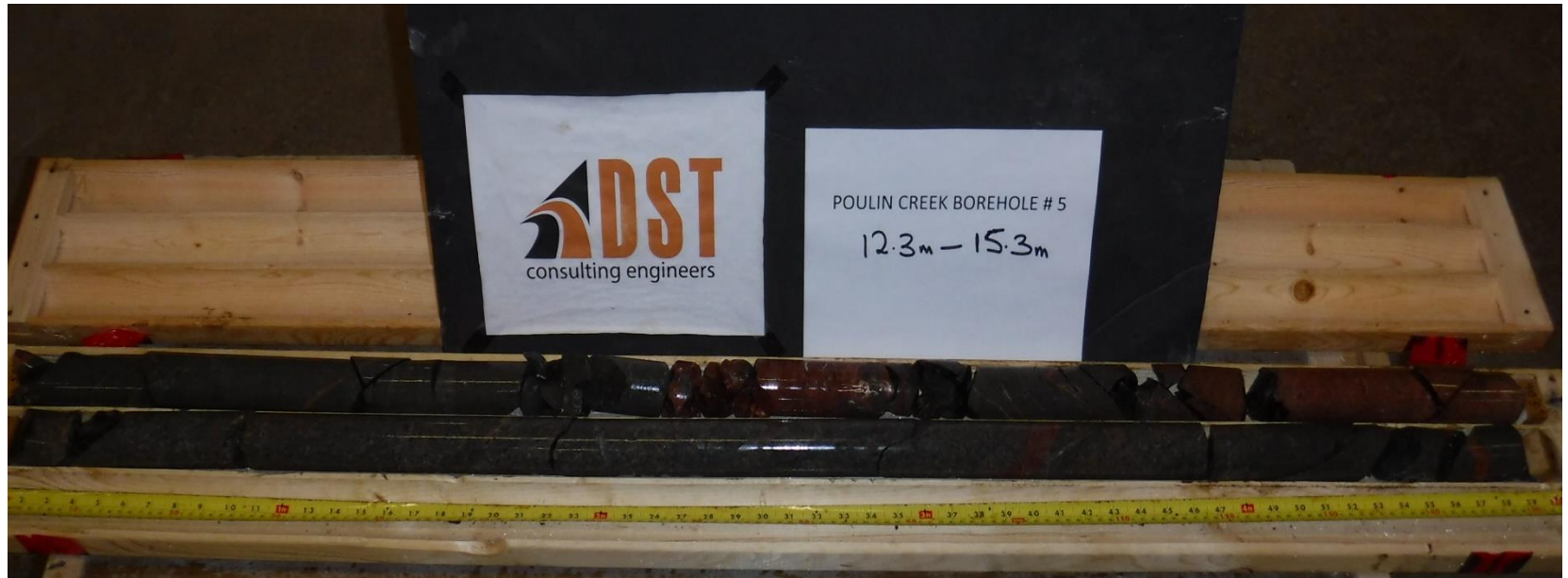
	Project	Hwy 129 - 4 Culverts Replacement AG # 5016-E-0001	Borehole Number	BH4		
	Project Number	IN-NO-026564	Box Number	1	of	2
	Client	MTO	Depth (m)	10.7	to	13.9



	Project	Hwy 129 - 4 Culverts Replacement AG # 5016-E-0001	Borehole Number	BH4		
	Project Number	IN-NO-026564	Box Number	2	of	2
	Client	MTO	Depth (m)	13.9	to	15.4



	Project	Hwy 129 - 4 Culverts Replacement AG # 5016-E-0001	Borehole Number	BH5		
	Project Number	IN-NO-026564	Box Number	1	of	1
	Client	MTO	Depth (m)	12.3	to	15.3



Appendix E

**NON-STANDARD SPECIAL
PROVISION**

COBBLES, BOULDERS and ROCKFILL - Item No. 1

Non-Standard Special Provision

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

Cobbles, boulders and rockfill were identified within the subsurface soil layers at the borehole locations. The contractor should be aware of the potential for encountering cobbles or boulders or rockfill at the site during excavation and for installation of temporary roadway and coffer dam protection system, as well as dewatering systems.

GROUNDWATER- Item No. 2

Non-Standard Special Provision 2

This special provision covers the presence of HIGH GROUNDWATER LEVELS in the subsurface stratum.

The Contractor is advised of the following foundation conditions:

High groundwater and/or surface water levels are expected within the embankment fill at the culvert location, and in the underlying soils. The contractor should anticipate the possibility of encountering groundwater and/or surface water at the site during excavation, dewatering, bedding or installation of temporary roadway and coffer dam protection system.

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.

* 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 ww - 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
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

Workorder: L1851709

Report Date: 11-NOV-16

Page 2 of 2

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.