



## **FINAL**

**FOUNDATION INVESTIGATION AND DESIGN REPORT  
BURYING CREEK CULVERT REPLACEMENT  
HIGHWAY 129  
SAULT STE MARIE DISTRICT  
AGREEMENT NO.: 5016-E-0001  
SITE NO.: 46-002  
GEOCRES NO. 410-21  
GWP: 5259-13-00**

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**PREPARED FOR:**

Ministry of Transportation  
North-eastern Region Office  
447 McKeown Avenue, 1<sup>st</sup> Floor  
North Bay, ON P1B 9S9

1 Copy - Ministry of Transportation, North Bay, ON  
1 Copy - DST Consulting Engineers

DST CONSULTING ENGINEERS INC.  
605 Hewitson Street, Thunder Bay, Ontario P7B 5V5  
Phone: 1-807-623-2929 Fax: 1-807-623-1792

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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 of the proposed culvert replacement (Part 2).

**2. SITE DESCRIPTION**

The site is located on Highway 129, approximately 21.5 km south of Highway 667 (latitude 47.43645, longitude -83.20496), Station 12+952 (Township of Langlois) in the District of Sault Ste Marie.

The highway runs in a north and south direction. The culverts are aligned across the highway at an angle of approximately 25°. The local topography is gentle and subdued, and the surrounding valley and slopes are densely vegetated with natural boreal forest cover. The site photographs presented in Figures 2.1 and 2.2 were taken by DST field staff during the fieldwork investigation.

The existing 22.0 m long corrugated steel ellipse culvert is approximately 3.6 m in diameter. The existing culverts (Figure 2.1 to 2.2) were built in 1982 as stated in the Ontario Structure Inspection Manual (OSIM), and inspection by others indicates light to moderate corrosion, cracking along south wall and north wall, some flattening of upper seam on north side, 70 mm average between crack ends, improper bolt layout at crack location, deflection of panels leaving gaps at some seam locations, minor build-up of debris in barrel at west and severe corrosion on some bolts below waterline.

The embankment height, as measured from the road level to the creek bed, at the culvert location is approximately 5 m and the side slopes of the embankment are approximately 3H: 1V.



Figure 2.1 View of the existing culvert at Highway 129 – BH1 (looking North-East near Outlet)





Figure 2.2 View of the existing culvert at Highway 129 BH2 (looking East near Inlet)

### **3. REGIONAL GEOLOGY**

Geological information is available from the published *Ontario Geological Survey Map #410SW* by the *Ontario Ministry of Natural Resources* for the Chapleau area. The map indicates that the local terrain unit is identified as Bedrock Knob (RN). Bedrock knob landscape is characterized by an irregular bedrock surface having complex multiple slopes of varying steepness. The cover of glacial deposits overlying the bedrock knobs is generally thin and discontinuous. Much of the glacial overburden consists of bouldery, sand-rich till that was transported only a short distance by the ice. Bedrock knob terrain is also indicated over much of the map area.

As indicated on the Bedrock Geology of Ontario West Central Sheet Map 2542, the site is underlain by metasedimentary rocks of Archaen age comprising wacke, arkose, argillite, slate, marble, chert, iron formation, and also minor metavolcanic rocks comprising conglomerate, arenite, paragneiss, and migmatites.

### **4. INVESTIGATION PROCEDURES AND LABORATORY TESTING**

A previous site investigation was carried out by Peto MacCallum Ltd. (PML) in December 2014 to January 2015. (Preliminary Foundation Investigation and Design Report for Burying Creek Culvert Replacement, Highway 129, Township of Langlois, Algoma District, Ontario. Assignment No. 5013-E-0040, G.W.P. 5222-05-00, Site # 46-002/C, WP NO. 5225-05-01, GEOCREs No. 41O-15, Peto MacCallum Ltd., (27 Sept 2016). Four boreholes (BC1, BC2, BC3 and BC4) were advanced to depths ranging from 4.8 m to 7.6 m. The boreholes were advanced using hollow stem augers, dynamic ram sounder and portable tripod with casing. This information was provided by MTO, as supplementary information. The borehole logs (BC1 to BC4) are included in Appendix D.

A site visit was undertaken by one of DST's professional engineers on July 27<sup>th</sup> and July 28<sup>th</sup>, 2016 to review the site conditions, including drill 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 interpreted stratigraphic section are shown on Drawings 1 and 2 in Appendix C.

DST site investigation fieldwork was carried out over three different phases (1) on August 24<sup>th</sup>, (2) on September 11, and (3) September 14 to 17<sup>th</sup>, 2016 utilizing both a CME 550 drill rig and a portable rig (near outlet only) equipped for geotechnical drilling and operated by Landcore Drilling from Chelmsford, Ontario. A total of five boreholes (BH1, BH2, BH3, BH4, and BH5) were



advanced to depths ranging from 7.8 m to 16.2 m. The minimum number and depth of the boreholes was specified by the Ministry of Transportation (MTO).

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 benchmark indicated on the drawings provided by the Ministry. Table 4-1 summarizes the detail of borehole locations and depths.

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

The fieldwork was supervised on a full-time basis by DST personnel who located the boreholes in the field, performed sampling, in-situ testing and logged the boreholes. Soil samples were obtained from the auger flights and from the split spoon sampler used in 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: Details of Borehole Location

Borehole ID	Station	Location	Elevation (m)	Depth (m)	Offset (m)	Completion Details
BH1	12+955	Outlet	441.7	7.8	11.6 Rt	Borehole backfilled with bentonite chips from caving depth to ground surface.
BH2	12+952	Inlet	442.0	10.8	11.5 Lt	Borehole backfilled with bentonite chips from caving depth to ground surface.
BH3	12+962	Roadway	443.8	16.2	2.0 Lt	Borehole backfilled with bentonite chips from caving depth up to road base then backfilled with granular material and asphalt cold patch to surface.
BH4	12+950	Roadway	444.1	15.3	2.0 Lt	Borehole backfilled with bentonite chips from caving depth to road base then backfilled with granular material and asphalt cold patch to surface.
BH5	12+958	Roadway	443.8	15.3	3.2 Rt	Borehole backfilled with bentonite chips from caving depth to road base then backfilled with granular material and asphalt cold patch to surface.
BC1	-	Inlet	442.5	5.5	-	Borehole backfilled with Bentonite/cement grout
BC2	-	Roadway	443.8	7.6	-	Borehole backfilled with Bentonite/cement grout
BC3	-	Roadway	444.2	4.8	-	Borehole backfilled with Bentonite/cement grout
BC4	-	Outlet	441.7	5.0	-	Borehole backfilled with Bentonite/cement grout

Classification and index tests were subsequently performed in the laboratory on samples collected from the boreholes to aid in the selection of engineering properties. Laboratory tests included natural moisture contents, particle size analyses, and chemical tests. A total of fifty-six

(56) natural moisture contents, sixteen (16) Sieve analysis LS 602 and one (1) set of chemical tests have been carried out for this assignment. Laboratory test results are presented in the Boreholes Logs and graphical plots are attached in Appendix D (Enclosures).

## **5. DESCRIPTION OF SUBSURFACE CONDITIONS**

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

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

- Asphalt at surface in Boreholes BH3, BH4, BH5, BC2, and BC3.
- Embankment Fill in Boreholes BH3, BH4, BH5, BC1, BC2, and BC3 comprising of sand with gravel and trace to some silt.
- Peat at surface in Borehole BH1 and BC4 and below the embankment fill in Boreholes BH3, BH4, BH5, BC1, BC2, and BC3.
- Underlain by a very loose to very dense Sand layer in all nine (9) boreholes. Cobbles and boulders were encountered within this layer.
- Underlain by Sand and Gravel, with trace to some silt in Boreholes BH1 and BH2 to the depth of termination, and in borehole BH3. Cobbles and boulders were encountered within this layer.
- Underlain by till comprising of sand and silt, with some gravel in Boreholes BH3, BH4 and BH5 to the depth of termination. Numerous cobbles and boulders were encountered within this layer.

Attempts to undertake SPT testing within the cobbles and boulders in all boreholes were unsuccessful due to bouncing of the SPT hammer on the cobbles and boulders. Due to difficult drilling conditions, the drilling method was changed to wash boring to advance the boreholes through these obstructions.

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	Present in Boreholes	Depth (from-to)	Elevation (from-to)	Thickness (m)	SPT N Value	Relative Density
Asphalt	BH3, BH4, BH5, BC2, BC3	0.0 – 0.23	443.8–444.2	0.1-0.23	N/A	N/A
Embankment Fill – Sand and Gravel	BH2, BH3, BH4, BH5, BC2, BC3	0.0 -3.3	444.0-440.4	1.6 – 3.2	2-29	Very loose to Compact
Peat	BH1, BH3, BH4, BH5, BC1, BC2, BC3, BC4	0 – 6.1	441.7–438.0	0.9 – 3.1	1– 5	very soft to firm
Sand	BH1, BH2, BH3, BH4, BH5, BC1, BC2, BC3, BC4	1.6 – 12.4	440.4–431.4	0.5 – 7.8	1 - 32	Very loose to dense
Sand & Gravel	BH1, BH2, BH3	4.6 – 12.4	437.4–431.2	3.2 - 6.2	11 – 23	compact
Till – Sand and Silt to sandy Silt	BH3, BH4, BH5	9.3 – 16.2	434.8–427.6	2.9 – 6.0	51 – 79	Very Dense
Cobbles and Boulders	BH1	7.6 – 7.8	434.1 – 433.9	0.2	100+	*
	BH2	8.1 – 8.4	433.9 – 433.6	0.3	N/A	-
	BH2	10.4 – 10.8	431.6 – 431.2	0.2	N/A	-
	BH3	7.0 – 7.6	436.8 – 436.2	0.6	N/A	-
	BH3	10.9 – 12.4	432.9 – 431.4	1.5	100+	*
	BH3	13.1 – 13.6	430.7 – 430.2	0.5	N/A	-
	BH3	15.8 – 16.2	428.0 – 427.6	0.4	N/A	-
	BH4	7.0 – 7.6	436.5 – 437.1	0.6	N/A	-
	BH4	8.4 – 9.1	435.0 – 435.7	0.7	N/A	-
	BH4	10.0 – 15.3	428.8 – 434.1	5.3	100+	*
	BH5	7.0 – 7.6	436.2 – 436.8	0.6	N/A	-
	BH5	8.5 – 9.1	434.7 – 435.3	0.6	N/A	-
	BH5	10.0 – 10.6	433.2 – 433.8	0.6	N/A	-
	BH5	12.0 – 12.4	431.4 – 431.8	0.4	N/A	-
	BC-1	5.2 – 5.5	437.0 – 437.3	0.3	30/5cm	*
	BC-2	7.6 – 7.9	436.2 – 435.9	0.3**	Ref***	*
	BC-3	4.5 – 4.8	349.4 – 439.7	0.3**	Ref***	-
	BC-4	4.5 – 5.0	437.2 – 437.6	0.5**	27	-

Notes: \* SPT 'N' values > 100 indicate bouncing of SPT hammer on cobbles/boulders, and not accurate representation

\*\* Thickness assumed \*\*\* refusal on cobbles/boulders

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

### 5.1 Asphalt

Asphalt was encountered at surface in BH3, BH4, BH5, BC2 and BC3 with a thickness of 100 to 230 mm.

### 5.2 Embankment Fill

Embankment fill, comprising of brown, fine to coarse, Sand and Gravel to Sand with trace of gravel, angular to subangular and trace to some silt was encountered at surface in BH2 and below the asphalt layer in boreholes BH3, BH4, BH5, BC1, BC2, and BC3 with a thickness of 1.6 to 3.3 m. SPT 'N' values for the embankment fill layer vary from 2 to 29, indicating a very loose to compact condition. The natural moisture contents of samples tested range between 7 % and 26 %. The laboratory test results are summarized in Table 5-1.

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

Laboratory Results – Sieve analysis	
Gravel %	8 to 33
Sand %	46 to 79
Fines %	9 to 21

### 5.3 Peat

A black, amorphous peat with trace of sand and trace of silt was encountered at surface in BH1 and BC4 outside the embankment, and below the embankment fill in BH3, BH4, BH5, BC1, BC2, and BC3 with a thickness of 0.9 m to 3.1 m. Samples at BC2 and BC3 suggest that the interface between the peat and underlying sands may be gradual rather than distinct, in the form of sand and peat seams. SPT 'N' values for this deposit vary from 1 to 5, indicating a very soft to firm condition. The natural moisture contents of samples tested range between 33 % and 327 %.

### 5.4 Sand

A grey, fine to coarse Sand, subangular, with gravel to trace of gravel and trace of silt to some silt was encountered below the fill and peat in all boreholes with a thicknesses of 0.5 m to 7.8 m. The SPT 'N' values for this layer vary between 1 and 30 indicating a very loose to dense condition. The natural moisture contents of samples tested range between 10 % and 33 %. The silt/clay

fraction as measured in samples of the sand layer varies from 2 to 23%. The laboratory test results are summarized in Table 5-2.

Table 5-3: Summary of Sieve analysis- Sand

Laboratory Results – Sieve analysis	
Gravel %	0 to 28
Sand %	53 to 98
Fines %	0 to 16

## 5.5 Sand and Gravel

A grey, fine to coarse Sand and Gravel to gravelly Sand, angular with trace to some silt was encountered in boreholes BH1, BH2 and BH3. The thicknesses of this layer was not proven at all locations, and varies from > 3.2 m at BH1, to 4.8 m at BH3, to > 6.2 m at BH2. Boreholes BH1 and BH2 were terminated within this layer. SPT 'N' values for this layer vary from 11 to 32, indicating a compact to dense condition. The natural moisture contents of samples tested range between 9 % and 17 %. Cobbles and boulders were encountered within the sand and gravel layer. The laboratory test results are summarized in Table 5-3.

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

Laboratory Results – Sieve analysis	
Gravel %	34 to 41
Sand %	48 to 51
Fines %	10 to 17

## 5.6 Till

A till described as grey, fine to coarse, Sand with Silt to sandy Silt, subangular, with some gravel to trace of gravel was encountered in boreholes BH3, BH4 and BH5 below the gravel/sand layer. The thickness of this layer was not proven, as the boreholes were terminated within this layer (practical refusal). SPT 'N' values for this layer vary from 51 to 79, indicating a very dense condition. The natural moisture contents of samples tested range between 10 % and 12%. Cobbles and boulders were encountered within this layer. The laboratory test results are summarized in Table 5-4.



Table 5-5: Summary of Sieve analysis - Till

Laboratory Results – Sieve analysis	
Gravel %	9 to 17
Sand %	39 to 55
Fines %	28 to 46

## 5.7 Groundwater

Groundwater levels in the boreholes, were measured within 30 minutes of completion of borehole drilling and prior to backfilling of the borehole. In addition, shallow temporary standpipes were installed at the inlet and outlet boreholes, in order to keep the boreholes open after drilling, and to facilitate later monitoring of groundwater levels. The standpipes were read at 24 hours after installation, and then removed. This information is included on the Borehole Logs in Appendix D.

At the time of the field investigations, groundwater was observed in all nine boreholes (see Table 5-5). The water level in the creek was at elevation 441.6 m at both inlet and outlet at the time of field investigation. The reported groundwater levels during the investigation conducted by PML have also been included in Table 5-6 below. The groundwater levels can be expected to vary with the season and after local precipitation events.

Table 5-6: Groundwater Monitoring

Borehole and Location	Measured Water Levels (Depth/Elevation) m				
	After Drilling		24 Hours Reading		September 18, 2016
BH1 (outlet)	Sep 11, 2016	0.2 (441.5)	Sep 12, 2016	0.0 (441.7)	0.0 (441.7)
BH2 (inlet)	Aug 24, 2016	0.8 (441.2)	Aug 25, 2016	0.4 (441.6)	0.2 (441.8)
BH3	Sep 16, 2016	2.1 (441.7)	(Roadway)		(Roadway)
BH4	Sep 15, 2016	2.2 (441.9)	(Roadway)		(Roadway)
BH5	Sep 17, 2016	2.2 (441.6)	(Roadway)		(Roadway)
Creek (Inlet)	Sep 20, 2016	441.6	N/A		N/A
Creek (Outlet)	Sep 20, 2016	441.6	N/A		N/A
BC1 (Inlet)	Jan 15, 2015	0.6 (441.9)	N/A		N/A
BC2	Dec 12, 2014	2.0 (441.8)	(Roadway)		(Roadway)
BC3	Dec 12, 2014	2.1 (442.1)	(Roadway)		(Roadway)
BC4 (Outlet)	Jan 15, 2015	0.6 (441.1)	N/A		N/A

## 5.8 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 5-6 and discussed in Section 7.15. Copies of the Laboratory Certificate of Analyses are provided in Appendix 'F'.

Table 5-7: Chemical Test Results – Soil sample

Sample ID	Sulphate (mg/kg)	Chloride (µg/g)	pH	Conductivity (mS/cm)	Resistivity (ohm - cm)
BH4 (4.6 m depth)	288	63.7	7.27	0.464	2160

## 6. MISCELLANEOUS

The Peto MacCallum Ltd. (PML) fieldwork was undertaken December 2014 to January 2015. DST site investigation fieldwork was carried out in different phases between August 24<sup>th</sup> and September 17<sup>th</sup>, 2016 utilizing both a CME 550 drill rig and a portable rig (near outlet only) 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.

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**PART 2: ENGINEERING DISCUSSIONS AND RECOMMENDATIONS**

**7. GENERAL**

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

As discussed in Part I of the report, the generalized stratigraphy at the culvert location, based on the conditions encountered in the boreholes, consists of a low granular fill embankment over discontinuous up to 6m below road surface which is underlain by a deposit of very loose to compact sand which is further underlain by compact to very dense sand with varying amounts of gravel, and finally by a very dense sand till to the maximum 15 m borehole depth. Of note are the high silt content variations in the sand layer below the peat (indicating both highly frost susceptible zones and highly permeable zones) and the significant cobble and boulder content in the sand/gravel layer below the sand.

The existing culvert will be replaced by a single box culvert expected to be 4.8 m wide. 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. 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. A comparison of the advantages and disadvantages of each proposed culvert option 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 a Box Culvert is the preferred option for the replacement structure. The culvert will be approximately 4.8 m wide. Geotechnical recommendations for an open footing culvert 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): .....	441.6
New culvert invert: .....	439.0
Approx. base of 0.5 m thick culvert bedding: .....	438.2
Excavation level for 2.2 m frost penetration below invert: .....	436.7
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 understood that the culvert will be located at approximately the same vertical and horizontal alignment of the existing structure. The design parameters provided in Tables 7-1 to 7-5 below are recommended for design.

It is expected that the new culvert excavation to the base of bedding elevation may encounter organic materials at the excavation base, which will need to be removed. Fill from the previous culvert construction may also be found at this level, and will need to be inspected by a geotechnical engineer for suitability as foundation support. The inorganic materials at the base of this excavation are very loose and loose wet sands, are permeable and very susceptible to disturbance by equipment. These materials are below the water table, and without adequate groundwater control prior to excavation will furthermore be permanently rendered 'disturbed' to considerable depth from groundwater effects such as heaves and boils. The materials described above, when disturbed, will be unsuitable for support of the structure and very difficult to remove and replace.

Where unsuitable soils are encountered the foundation soils should be removed to undisturbed and clean native soils and replaced with Granular A or Granular B Type I material meeting

OPSS.PROV 1010 specifications and compacted to a minimum of 95 % of Standard Proctor Maximum Dry Density (SPMDD) to the foundation (or bedding) grade. Before backfilling or bedding is placed, the foundation soils should be inspected by a geotechnical engineer to confirm conditions are as described herein.

The sand underlying the site has zones which are considered frost susceptible and are within the potential frost penetration depth below the culvert. The need to remove this silt 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) with respect to both structural and 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 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. If sub-excavation for frost effects is carried out in the wet (water is maintained at or above adjacent groundwater table), all foundation preparation should be completed in accordance with OPSS 421 "Construction Specification for Pipe Culvert Installation in Open Cut", any specifications provided in the contract documents and as indicated in Section 7.7 Bedding.

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 all fill supporting foundations should not be less than a distance (from the side of the foundation) equal to the depth of fill below the footing. This applies for the full depth of the fill. For example, a 3 m wide culvert underlain by 2 m of engineered fill would require an excavation base 7 m wide plus 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 somewhat.

### 7.2.1 Foundation Design (Concrete Box Culvert)

The geotechnical resistance was estimated for the ultimate limit state (ULS) as well as the serviceability limit state (SLS) for a maximum settlement of 25 mm. The factored resistances at ULS were calculated by applying 1 uncertainty factors in accordance with the Bridge Design Code (CHBDC) CAN/CSA-S6-14 section 6.6, and are shown in Table 7-1.

The ULS geotechnical resistance was estimated assuming a rigid strip footing consisting of a width equal to the width of the new culvert (4.8 m). The culvert must be installed on top of bedding material placed on undisturbed native soils (sand), or in the case of additional sub-excavation, on compacted fill. The box culvert will be installed with proposed invert levels at 4.7 m and 5.1 m depth (Elev.439.37 m and 438.97 m inlet and outlet respectively). The calculated factored geotechnical resistances are presented in Table 7-1 below.

Deep peat was encountered below the proposed footing elevation at borehole BH4 location only. Any and all organic soil encountered at founding level will require local subexcavation and replacement with granular fill as previously described above. Preparation of an undisturbed base will be difficult given the loose and saturated nature of the soils

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

Table 7-1: Geotechnical resistances for Concrete Box Culvert

Construction Phase	Footing size	Founding Elevation	Factored Geotechnical Resistance at ULS (kPa)	Factored Geotechnical Resistance at SLS (kPa)
Post Construction	4.8 m	438.7	695	80
During Construction	4.8 m	438.7	170	80

Groundwater was encountered at elevations close to the prevailing creek level. The groundwater level is expected to fluctuate both with seasonal changes, local precipitation events and in response to rising water levels in the creek during the spring thaw. The proposed founding depth of the foundation will be below the measured groundwater level. The expected foundation soils



are cohesionless 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 factored resistance at ULS was calculated by applying uncertainty factors according to the Bridge Design Code (CHBDC) CAN/CSA-S6-14 section 6. 6. The geotechnical resistance is estimated based on a strip footing 1 m wide with a length equal to 24 m situated at depths between 6.1 m and 7.3 m (Elev. 438.0 m to 436.8 m) below the existing road elevation (see Table 7-2).

Table 7-2: Geotechnical resistances for open footing culverts with footing base at Elevation 438.0 m to 436.8 m

Footing Width (m)	Soil Cover over Footing base** (m)	Founding Elevation (m)	Factored Geotechnical Resistance at ULS (kPa)	Factored Geotechnical Resistance at SLS (kPa)
B = 1.0	1.0	438.0	125	125
	1.5	437.5	170	170
	2.2	436.8	230	230

\*\*Assumes creek bed cannot be eroded below this level

Groundwater was encountered at elevations close to the prevailing creek level. The groundwater level is expected to fluctuate both with seasonal changes, local precipitation events as well as with changes in the creek level. The proposed founding depth of the foundation will be below the groundwater elevation. The expected foundation soils are cohesionless and therefore dewatering work will be required for the foundation preparation.

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

### 7.3 Lateral Earth Pressures and Sliding Resistances

The analysis of horizontal and vertical effects of earth loads on the culvert can be undertaken with the use of the soil parameters provided in Table 7-3 and as described in Section 7.6.3.1 in the 2014 Canadian Highway Bridge Design Code. Temporary bracing and shoring may be designed using the typical soil parameters given in Table 7-3 and Table 7-4, however the

designer/contractor should verify the appropriateness of soil parameters for the design of any specific bracing and shoring system.

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

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

Soil type	Unit weight (kN/m <sup>3</sup> )	Internal drained friction angle (Degree)	Interface friction angle** δ (Degree)
Fill-Sand	20	33	22
Compressed Peat	12	28	18.5
Sand	20	33	22

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, 4<sup>th</sup> Edition) and our previous experience with similar materials.

\*\*interface between soil and concrete.

Table 7-4: Lateral Earth Pressure Coefficients

Earth Pressure Coefficient	Equation*	Fill-Sand*	Peat*	Sand*	Till – Sand & Silt*
Active Earth Pressure ( $K_a$ )	$\left(\frac{1 - \sin\phi}{1 + \sin\phi}\right)$	0.30	0.36	0.30	0.27
Passive Earth Pressure ( $K_p$ )	$\left(\frac{1 + \sin\phi}{1 - \sin\phi}\right)$	3.35	2.8	3.35	3.65
At rest ( $K_0$ )	$(1 - \sin\phi)$	0.46	0.53	0.46	0.43

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

Use of a temporary sheet pile wall or soldier pile wall with lagging may be considered for all or part of the excavation sides. 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. The soil below this site is extremely susceptible to consolidation as a result of nearby vibration such as that from a vibratory sheet pile driver/extractor. If vibratory equipment is used, control vibration to ensure no deformation of the culvert occurs as a result. Monitor and document culvert movements accurately during vibration to confirm satisfactory culvert performance.

The contractor should be alerted to the presence of soils which are susceptible to consolidation under the effects of vibration, for example through a non-standard special provision (NSSP 3).

An excavation depth (for bedding) of up to approximately 5.7 m (Elev. 438.4 m) for the box culvert option will be required for the staged construction. The existing embankment slopes should be reinstated as presented in Section 7.12 Embankment Slopes. 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 5.7 m (Elev. 438.4 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 1.3 at a 2H:1V slope. Thus, a temporary open excavation with 2H:1V sides with effective dewatering is a feasible construction method.

Excavation below the water table in the wet (water in the excavation maintained at or above creek level) can also be considered. This would require clear stone (for example 20mm size) and a geotextile surround. Whereas this method considerably reduces the risk of, for example, (a) base disturbance from uncontrolled hydraulic pressures, and (b) disturbance from compaction, it also requires a very rigorous method of preparation of the underwater excavation base (no disturbed materials allowed), underwater placement of a geotextile with pre-sewn seams, and underwater preparation of the clear stone upper surface to receive the foundations.

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, impending base heave, nearby ground settlement and any nearby 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 and completed in accordance with OPSS.PROV 209 "Construction Specification for Embankments Over Swamps and Compressible Soils". The

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

Excavations for this project should be constructed in accordance with the requirements of the Occupational Health and Safety Act of Ontario (OHSA), O.Reg. 213/91. According to O.Reg. 213/91, s.226, the soils in the area of interest classify as Type 3 and Type 4 if located above and below the water table respectively. In accordance with O. Reg. 213/91, s.227 (3), if an excavation contains more than one type of soil, the soil should be classified with the highest number as described in section 226. These should be assessed and confirmed in the field as construction progresses.

The dominant soil type to be exposed in the excavation (granular fill and loose sands) are expected to be Type 3 above the water table and Type 4 below the water table. Peat is also expected to be locally present within the excavation area, and is Type 4.

## **7.6 Roadway Protection**

Since temporary roadway protection is required during the structure replacement, an appropriate roadway protection system should be selected by the contractor and designed by the contractor's qualified engineer. Temporary roadway protection systems should be constructed in accordance with OPS 539 "Construction Specification for Temporary Protection Systems". Potential roadway protection systems include a steel sheet pile wall, a soldier pile wall with lagging, and a temporary dewatered excavation using side slopes of 2H:1V. The sheet pile wall and soldier pile wall options may have some difficulty to extreme difficulty in achieving the required installation depths, based on the cobbles and boulders encountered during driving. The advantages and disadvantages of using different road way protection systems are shown in Table 7-5.

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

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

The design of roadway protection between the excavation and the open lane may be performed using the typical soil parameters given in Tables 7-3 and 7-4, however the designer/contractor



should verify the appropriate soil parameters for the designs. The construction methodology should be in accordance with all applicable standards and regulations. The contractor's method and equipment should be suitable for the site conditions and materials used. This soil investigation encountered cobbles and boulders at several locations below approximately a 7 m depth below road surface. Furthermore, the presence of organic materials requiring subexcavation need to be addressed by the contractor. The contractor should be alerted of the presence of these through a non-standard special provision (NSSP 1).

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

## **7.7 Bedding**

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. Construction of a precast concrete box culvert and its bedding should follow the provisions in OPSS 422 "Precast Concrete Box Culvert". In addition, the bedding for the structure should be designed in accordance with Section 7.8 of the CHBDC and MTOD 803.021 "Bedding and Backfill for Precast Concrete Box Culvert".

The bedding should be a minimum of 500 mm thick. The bedding material should consist of "Granular A or Granular B Type I" as per Soil Group I in accordance with Table 7.4 of the Canadian Highway Bridge Design Code. The "Granular A or Granular B Type I" should be in accordance to OPSS.PROV 1010. The "Granular A or Granular B Type I" should be placed in layers not exceeding 200 mm in thickness, loose measurement, and each layer compacted to a minimum of 98 % of standard Proctor maximum dry density. The middle one-third of the culvert width of the top bedding layer, having minimum thickness of 75 mm, should be loosely placed.

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

## **7.8 Backfill and Cover**

The material used for culvert sidefill (backfill) should not contain debris, organic matter, frozen materials, or large stones of a diameter greater than one-half the thickness of the compacted layers being placed or 100 mm, whichever is smaller. Soils should be placed uniformly on each side of the structure in order to minimize lateral displacement. The sidefill should consist of Granular A or Granular B Type I" and compacted to 95% of standard Proctor maximum dry density.

Overfill above the culvert and below the pavement structure should consist of "Granular A or Granular B Type I" and should be compacted to not greater than the compaction or equivalent stiffness of soils in the sidefill zone and bedding.

Prior to placement of backfill the contractor shall ensure that the joints of the proposed culvert are effectively covered to prevent influx of material from the backfill through the joints with a 600 mm (minimum) wide coverage strip. A non-woven geotextile shall be installed to cover all exterior joints of the culvert, including the top slab. The geotextile shall be free of folds, tears, and wrinkles. The geotextile and the seam requirements at the joints shall be in accordance with OPSS 1860. In addition joint sealing compounds or preformed gaskets for sealing joints between box culvert units, shall be applied in accordance with the manufacturer's recommendations as per OPSS 422 "Construction Specification for Precast Reinforced Concrete Box Culverts in Open Cut".

## **7.9 Channel Diversion**

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

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

### **7.10 Erosion Control**

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

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

The outlet should be rip-rapped to prevent erosion of the surrounding soils in accordance with OPSD 810.010 "General Rip-Rap Layout for Sewer and Culvert Outlets" and OPSS 511 "Construction Specification for Rip-Rap, Rock Protection, and Granular Sheeting". Rip-Rap material should comply with OPSS 1004.

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

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

### **7.11 Frost Protection**

In accordance with OPSD 3090.100 "Foundation Frost Depths for Northern Ontario", the frost penetration at this location is about 2.25 m. Frost susceptible soils should not be used adjacent to the culvert wall. The soils under the culvert are assessed as being highly frost susceptible (capable of forming thick ice lenses with the associated pressures and heave).

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. During winter season, ice may form

inside the culvert and a low flow rate may assist the ice formation. It is expected that ice may extend to the culvert invert and frost could therefore extend into the soils below the culverts, possibly as deep as 2.25 m below concrete exposed to the elements. The frost heave may generate additional stresses on the culvert foundation and walls, and is likely to be non-uniform along the culvert length.

Frost heaving impacts for 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.

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, the sand underlying the culvert has both frost susceptible and non-frost susceptible zones.

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 the face 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 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 embankment footprint at the ends of the culvert. The foundation parameters provided above are applicable for the retaining wall foundation design, provided that the founding level for the base of the wall does not vary significantly from the founding level for the base of the culvert.

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

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

where

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

The above assumes horizontal ground beyond the wall and no hydrostatic (water) conditions adjacent to the below-grade walls. Backfill should, therefore, consist of clean, free-draining granular material. 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 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 materials. This controlled compacted zone should extend a distance laterally at least equal to twice the height of the foundation element. Passive pressures under these conditions can be calculated using a coefficient of passive pressure ( $K_p$ ). Significant deformation of the soil is needed before passive pressure is fully mobilized.

#### **7.14 Construction Concerns**

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

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

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

#### **7.15 Chemical Testing**

The results of the analytical testing conducted on a soil sample at the site location have been presented in Table 5-7 and also included in Appendix F. The suite of the parameters tested is intended to allow the design engineer to assess the requirements for the appropriate type of cement to be used in construction and the need for corrosion protection of steel reinforcing elements.

The analytical results of the soil samples were compared with applicable Canadian Standards Association (CSA) standards as shown in Table 7-6 below. The chemical sulphate content



analyses for the representative soil samples tested indicate a sulphate concentration of 63.7 ug/g (0.00637 %) in soil. The results were compared with Canadian Standards Association (CSA) Standards A23.1 for sulphate attack potential on concrete structures and possess a “negligible” risk for sulphate attack on concrete material and accordingly, conventional GU or MS Portland cement may be used in the construction of the proposed concrete elements. 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 2160 ohm-cm and 0.464 mS/cm respectively for the samples analysed from BH4 at the depth of 4.6 m.

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

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

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

## 7.16 Seismic Design Considerations

The design peak horizontal acceleration was estimated as 0.041 g using the 2005 National Building Code Seismic Hazard maps. Assessment of the liquefaction potential for granular soils was estimated using  $N_{60}$  values interpreted from the in-situ SPT data applying Seed and Idriss (1971). Liquefaction was evaluated and it was confirmed that liquefaction is not a concern at the site with a factor of safety against liquefaction of greater than 1.0.

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

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

## 8. CLOSURE

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: Comparison of Proposed Culvert Options

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

## 9. REFERENCES

- Building Research Establishment (BRE) Digest 363 (SD1 - 2005), UK *“Concrete in aggressive ground”*
- Canadian Foundation Engineering Manual. 2006. Fourth Edition, Canadian Geotechnical Society.
- Canadian Highway Bridge Design Code. 2006, CAN/CSA-S6-14, A National Standard of Canada, Canadian standards Association.
- Discussion on Proc. Paper 1732 (Wu, 1958), Proc. ASCE, Vol. 85, No. SM3 (67-79). Kenney, T. 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 Burying Creek Culvert Replacement, Highway 129, Township of Langlois, Algoma District, Ontario. Assignment No. 5013-E-0040, G.W.P. 5222-05-00, Site # 46-002/C, WP NO. 5225-05-01, GEOCRETS No. 410-15, Peto MacCallum Ltd., (27 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.
- Soil Mechanics in Engineering Practice. Third Edition. Terzaghi, Karl; Peck, Ralph B.; and Mesri, Gholamreza
- 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

**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**

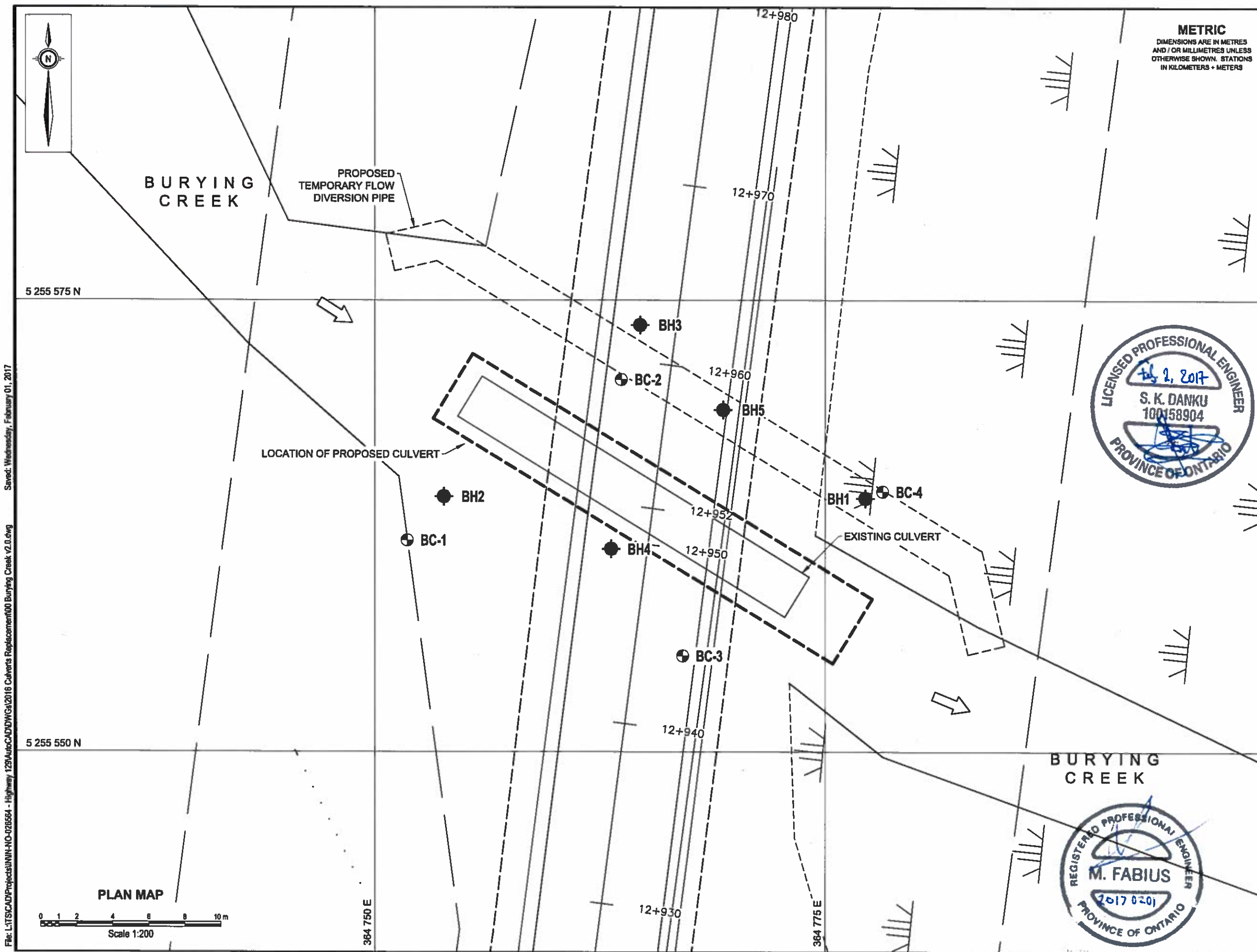
SS	SPLIT SPOON SAMPLE	WS	WASH SAMPLE
TW	THIN WALL SHELBY TUBE SAMPLE	AS	AUGER (GRAB) SAMPLE
PH	SAMPLER ADVANCED BY HYDRAULIC PRESSURE	TP	THIN WALL PISTON SAMPLE
WH	SAMPLER ADVANCED BY SELF STATIC WEIGHT	PM	SAMPLER ADVANCED BY MANUAL PRESSURE
SC	SOIL CORE	RC	ROCK CORE
	WATER LEVEL	$SENSITIVITY = \frac{UNDISTURBED\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

\*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**

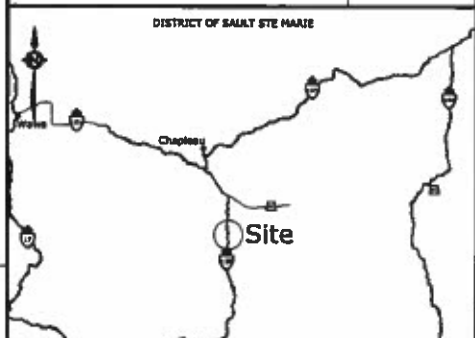


**METRIC**  
DIMENSIONS ARE IN METRES  
AND / OR MILLIMETRES UNLESS  
OTHERWISE SHOWN. STATIONS  
IN KILOMETERS + METERS

AG	NO	5016-E-0001
WP	NO	5259-13-00
SITE	NO	48-002
GEOCRES	NO	410-21



<b>CULVERT REPLACEMENT BURYING CREEK CULVERT</b>		<b>SHEET</b> 1
STA 12+927	TO STA 12+980	
Survey	Revised	
<b>BOREHOLE LOCATIONS</b>		



**KEY PLAN**  
Scale 1:4 000 000



LEGEND	
	Borehole (DST, 2016)
	Borehole (PML, 2014 - 2015)
	Flow direction

No.	Elev. (m)	MTM Zone 18		Survey	
		North (m)	East (m)	Station	Offset
BH1	441.70	5255563.5	384775.9	12+955	11.6 m Rt
BH2	442.00	5255563.6	384752.5	12+952	11.5 m Lt
BH3	443.80	5255573.1	384763.4	12+962	2.0 m Lt
BH4	444.10	5255580.7	384781.8	12+950	2.0 m Lt
BH5	443.80	5255568.4	384768.0	12+958	3.2 m Rt
BC-1	442.5	5255561.7	384751.8	-	-
BC-2	443.8	5255570.6	384763.7	-	-
BC-3	444.2	5255555.3	384787.1	-	-
BC-4	441.7	5255564.4	384778.2	-	-

REV	DATE	ISSUE	DRAWN BY	CHECKED	APPROVAL
A	6-Nov-16	DRAFT	RW	SA	MK
B	28-Nov-16	DRAFT	RW	SA / POS	MK
C	16-Jan-17	DRAFT	RW	SA	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.



**DST**  
consulting engineers

DST Consulting Engineers Inc.  
605 Hewitson Street  
Thunder Bay, ON P7B 5V5  
Ph: (807) 623-2929  
F: (807) 623-1792  
Email: thunderbay@dstgroup.com



**Appendix D**  
**ENCLOSURES**



# RECORD OF BOREHOLE No BH1

1 OF 1

**METRIC**

G.W.P. 5259-13-00 LOCATION BURYING CREEK ORIGINATED BY KN  
 DIST S.S. MARIE HWY 129 BOREHOLE TYPE PORTABLE - TRIPOD COMPILED BY SA  
 DATUM GEODETIC DATE 2016 09 11 - 2016 09 11 CHECKED BY MK/POS

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									WATER CONTENT (%)		
								20	40	60	80						100	20	40
441.7	GROUND SURFACE																		
0.0	Black PEAT - amorphous, trace sand, trace silt, very soft to firm		SS1	SS	1													Water level measured on September 19, 2016; from 25 mm opening 1.5 m long standpipe, installed with bentonite chips up to existing surface	
			SS2	SS	5														
			SS3	SS	1														
439.4																			
2.3	Grey fine to coarse SAND, some to trace silt, very loose to compact, angular			SS	1													No sample recovery	
			SS4	WH														0 96 (4)	
			SS5	SS	12														
437.1																			
4.6	Grey fine to coarse GRAVELLY SAND, some to trace silt, compact, angular		SS6	SS	19														
			SS7	SS	32													34 50 (16)	
433.9				SS	100+														
7.8	PROBABLE BOULDERS																	Refusal at 7.8 m; 100+ blows for 0.2 m penetration; No sample recovery	
	END OF BOREHOLE at 7.8 m																		

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

ENCLOSURE 1

ONL\_MOT-HIGH VANES BURYING CREEK.GPJ DATA TEMPLATE.GDT 18/1/17

# RECORD OF BOREHOLE No BH2

1 OF 1

**METRIC**

G.W.P. 5259-13-00 LOCATION BURYING CREEK ORIGINATED BY KN  
 DIST S.S. MARIE HWY 129 BOREHOLE TYPE HOLLOW STEM AUGER/WASHBORE COMPILED BY SA  
 DATUM GEODETIC DATE 2016 08 24 - 2016 08 24 CHECKED BY MK/POS

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT  w <sub>p</sub>	NATURAL MOISTURE CONTENT  w	LIQUID LIMIT  w <sub>L</sub>	UNIT WEIGHT  γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								20	40	60	80	100						20	40	60
442.0	GROUND SURFACE		AS1	AS																
0.0	FILL - Brown fine to coarse SAND and GRAVEL, trace to some silt, trace clay, very loose, angular		SS2	SS	2												Water level measured on September 19, 2016; from 25 mm opening 1.5 m long standpipe, installed with bentonite chips up to existing surface			
440.4			SS3	SS	2												0 83 (17)			
1.6	Grey fine to coarse SAND, some to trace silt, very loose to loose, angular		SS4	SS	1															
			SS5	SS	1															
			SS6	SS	7															
437.4			SS7	SS	22															
4.6	Grey fine to coarse SAND and GRAVEL some to trace silt, compact, angular		SS8	SS	11												41 48 (11)			
			SS9	SS	18												Starts washboring at 6.8 m to 10.8 m			
	cobbles and boulders from 8.1 m to 8.4 m		SS10	SS	20												39 51 (10)			
	cobbles and boulders from 10.4 m to 10.8 m			SS	100+												No sample recovery; 100+ blows for 0.2 m penetration			
431.2	END OF BOREHOLE at 10.8 m																			
10.8																				

ONL\_MOT-HIGH VANES BURYING CREEK.GPJ DATA TEMPLATE.GDT 18/1/17

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

ENCLOSURE 2

# RECORD OF BOREHOLE No BH3

1 OF 1

METRIC

G.W.P. 5259-13-00 LOCATION BURYING CREEK ORIGINATED BY KN  
 DIST S.S. MARIE HWY 129 BOREHOLE TYPE WASHBORE COMPILED BY SA  
 DATUM GEODETIC DATE 2016 09 16 - 2016 09 16 CHECKED BY MK/POS

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				
								20 40 60 80 100				
								20 40 60 80 100				
443.8	GROUND SURFACE											
443.7 0.1	ASPHALT  FILL - Brown fine to coarse SAND, some to trace gravel, trace silt, compact, subangular		SS1	SS	29							
			SS2	SS	14							
			SS3	SS	11							
441.3	Black PEAT - amorphous, trace sand, trace silt, soft to firm		SS4	SS	4							
2.5			SS5	SS	5							
440.0	Grey fine to coarse SAND with to some gravel, some to trace silt, very loose to compact, subangular		SS6	SS	3							
3.8			SS7	SS	3							
			SS8	SS	7							
	cobbles and boulders from 7.0 m to 7.6 m		SS9	SS	23							
			SS10	SS	20							
	cobbles and boulders from 10.9 m to 12.4 m			SS	100+							
431.4	TILL - Grey fine to coarse SAND with silt, some gravel, very dense, subangular		SS11	SS	79							
12.4	cobbles and boulders from 13.1 m to 13.6 m											
				SS	100+							
				SS	100+							
	cobbles and boulders from 15.8 m to 16.2 m											
427.6	END OF BOREHOLE at 16.2 m											
16.2												

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

ENCLOSURE 3

ONL MOT-HIGH VANES BURYING CREEK.GPJ DATA TEMPLATE.GDT 18/1/17

# RECORD OF BOREHOLE No BH4

1 OF 1

METRIC

G.W.P. 5259-13-00 LOCATION BURYING CREEK ORIGINATED BY KN  
 DIST S.S. MARIE HWY 129 BOREHOLE TYPE WASHBORE COMPILED BY SA  
 DATUM GEODETIC DATE 2016 09 14 - 2016 09 15 CHECKED BY MK/POS

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																	
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ONL MOT-HIGH VANES BURYING CREEK.GPJ DATA TEMPLATE.GDT 18/1/17

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

ENCLOSURE 4



# RECORD OF BOREHOLE No BH5

1 OF 1

METRIC

G.W.P. 5259-13-00 LOCATION BURYING CREEK ORIGINATED BY KN  
 DIST S.S. MARIE HWY 129 BOREHOLE TYPE WASHBORE COMPILED BY SA  
 DATUM GEODETIC DATE 2016 09 17 - 2016 09 17 CHECKED BY MK/POS

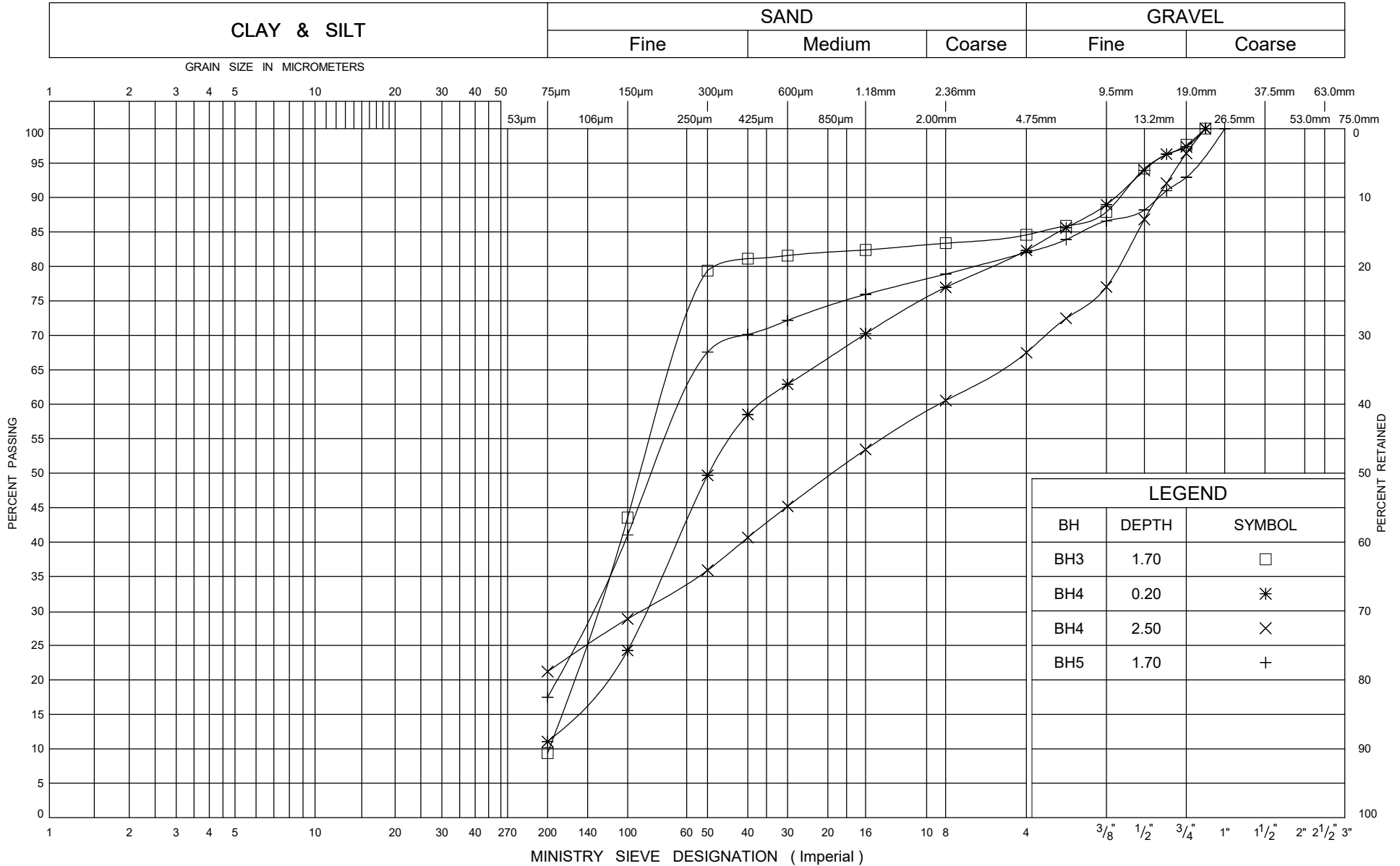
ONL\_MOT-HIGH VANES BURYING CREEK.GPJ DATA TEMPLATE.GDT 18/1/17

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
								UNCONFINED		FIELD VANE				
								QUICK TRIAXIAL	LAB VANE	WATER CONTENT (%)				
443.8	GROUND SURFACE					20	40	60	80	100	W <sub>p</sub>	W	W <sub>L</sub>	
443.7 0.1	ASPHALT FILL - Brown fine to coarse SAND with to some gravel, trace to some silt, compact, angular to rounded		SS1	SS	23									
			SS2	SS	12									
			SS3	SS	13									18 65 (17)
			SS4	SS	4									Water level measured at open hole after 30 minutes of the end of drilling
440.5			SS5	SS	5									
3.3	Black PEAT - amorphous, trace sand, trace silt, firm		SS6	SS	5									
439.2														
4.6	Grey fine to coarse SAND, some to trace gravel, some to trace silt, compact, subangular		SS7	SS	4									
			SS8	SS	100+									100+ blows for 0.2 m penetration
	cobbles and boulders from 7.0 m to 7.6 m													
			SS9	SS	17									
	cobbles and boulders from 8.5 m to 9.1 m													
			SS10	SS	24									15 70 (15)
	cobbles and boulders from 10.0 m to 10.6 m													
			SS11	SS	100+									100+ blows for less than 0.3 m penetration
	cobbles and boulders from 12.0 m to 12.4 m													
431.4			SS12	SS	100+									9 45 (46)
12.4	TILL - Grey fine to coarse SAND and SILT, some to trace gravel, very dense, subangular													100+ blows for less than 0.3 m penetration
			SS13	SS	100+									100+ blows for 0.2 m penetration
428.5				SS	100+									
15.3	END OF BOREHOLE at 15.3 m													No sample recovery; 100+ blows for 0.1 m penetration

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

ENCLOSURE 5

# UNIFIED SOIL CLASSIFICATION SYSTEM



GRAIN SIZE DISTRIBUTION  
FILL

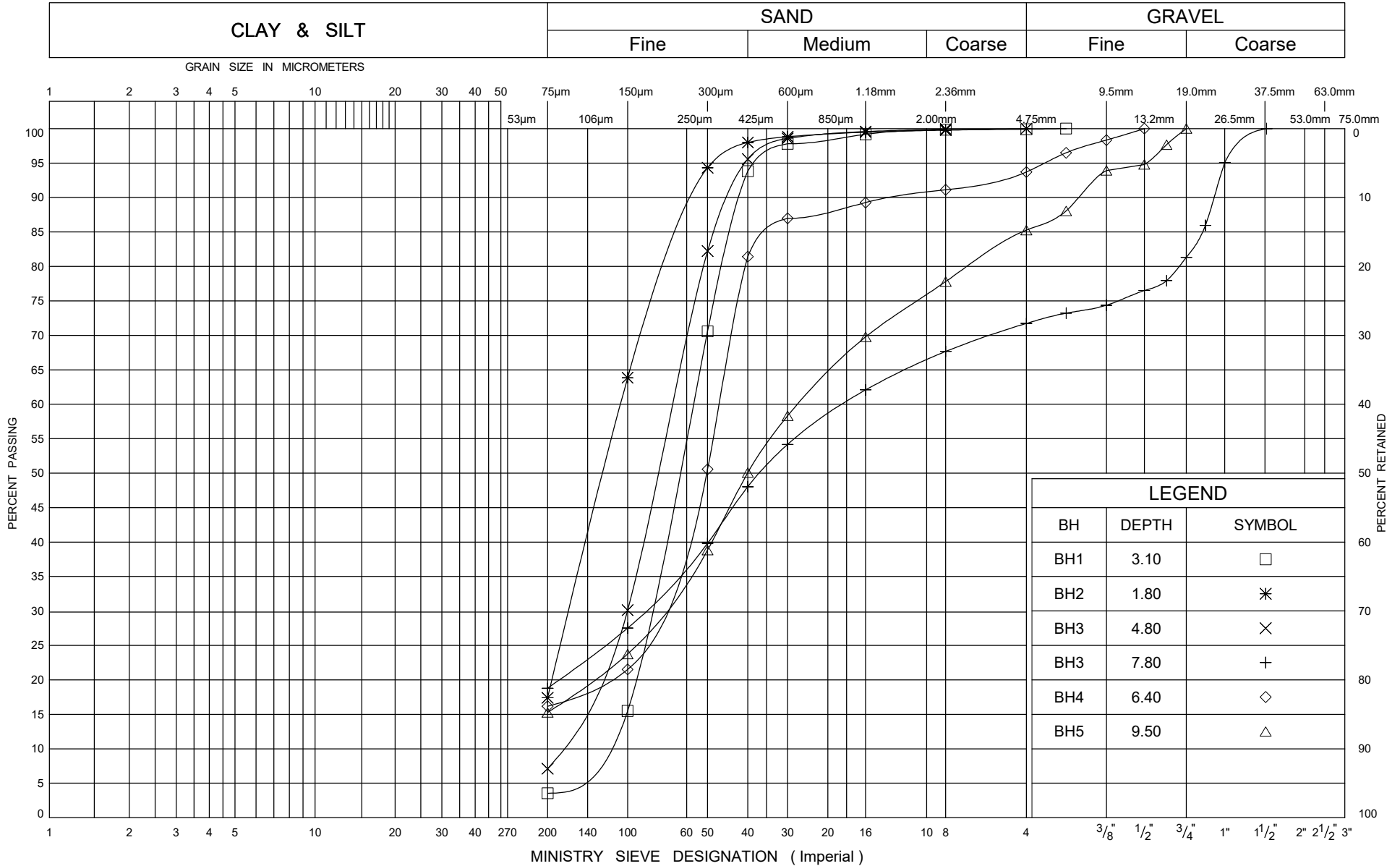
ENCLOSURE 6

G.W.P. # 5259-13-00

BURYING CREEK



# UNIFIED SOIL CLASSIFICATION SYSTEM



GRAIN SIZE DISTRIBUTION  
SAND

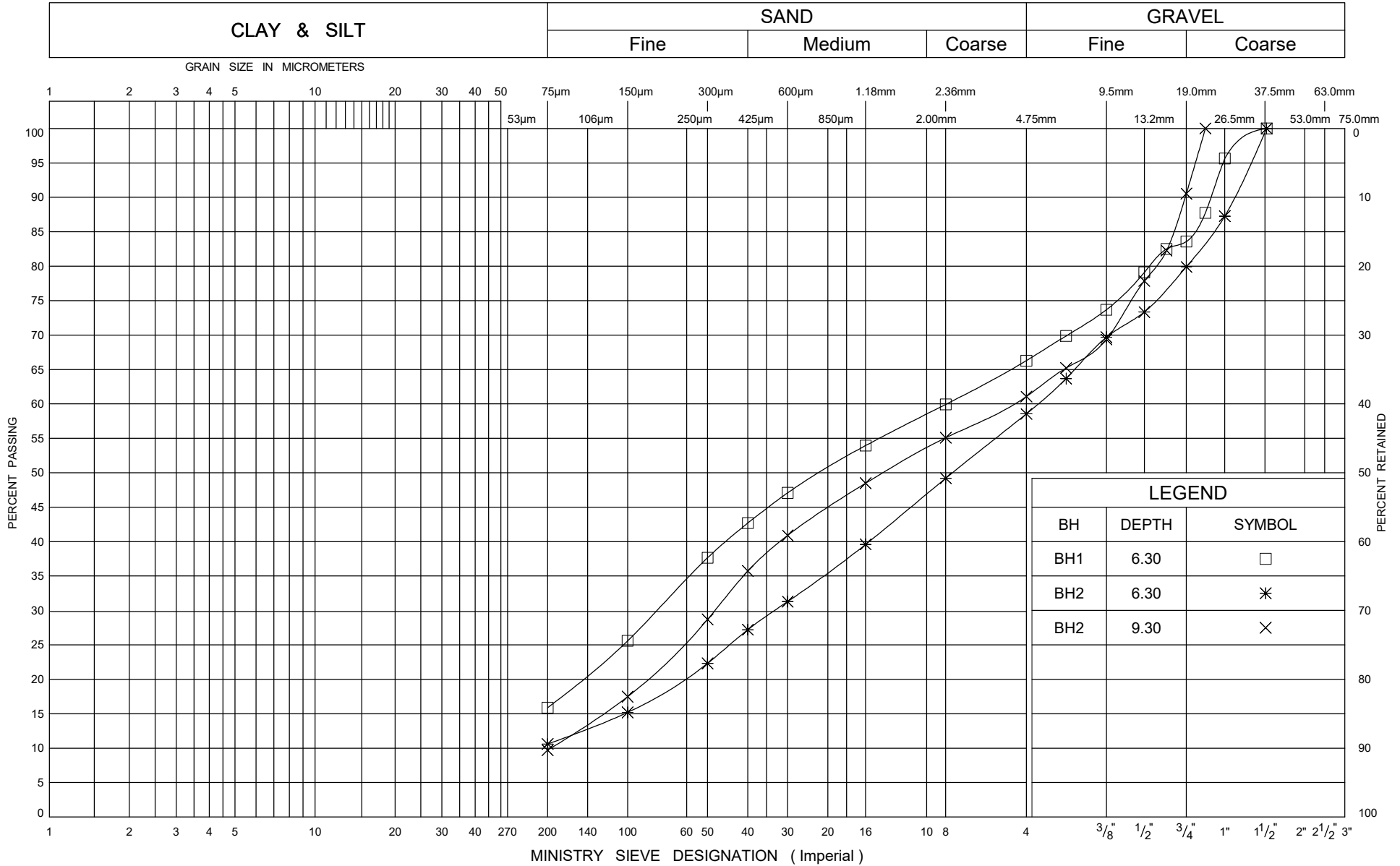
ENCLOSURE 7

G.W.P. # 5259-13-00

BURYING CREEK



# UNIFIED SOIL CLASSIFICATION SYSTEM



## GRAIN SIZE DISTRIBUTION SAND AND GRAVEL

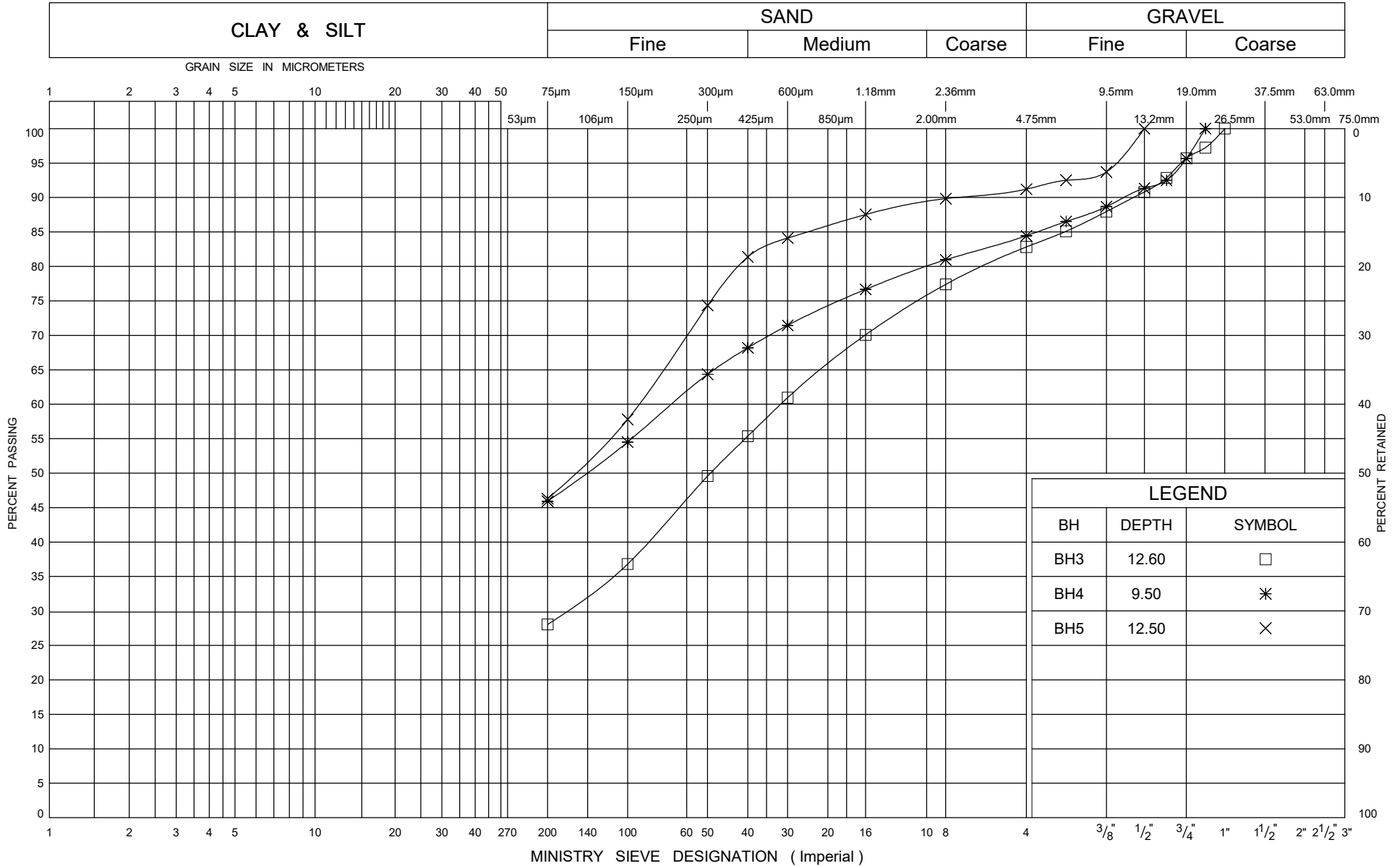
ENCLOSURE 8

G.W.P. # 5259-13-00

BURYING CREEK



# UNIFIED SOIL CLASSIFICATION SYSTEM



## GRAIN SIZE DISTRIBUTION TILL

ENCLOSURE 9

G.W.P. # 5259-13-00

BURYING CREEK



## RECORD OF BOREHOLE No BC-1

1 of 1

METRIC

G.W.P. 5222-05-00

LOCATION

Burying Creek

Coords: 5 255 561.7 N; 364 751.8 E

ORIGINATED BY F.P.

DIST Algoma HWY 129

BOREHOLE TYPE Tripod + Casing

COMPILED BY A.K.

DATUM Geodetic

DATE January 15, 2015

CHECKED BY M.V.

[illegible]

**RECORD OF BOREHOLE No BC-2**

1 of 1

**METRIC**

G.W.P. 5222-05-00 LOCATION Burying Creek Coords: 5 255 570.6 N; 364 763.7 E ORIGINATED BY F.P.  
DIST Algoma HWY 129 BOREHOLE TYPE DYNAMIC RAM SOUNDER COMPILED BY A.K.  
DATUM Geodetic DATE December 12, 2014 CHECKED BY M.V.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT  γ  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								20 40 60 80 100										20 40 60		
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE												
443.8	Ground Surface																			
0.0	230mm asphalt over gravelly sand (PAVEMENT FILL)		1	SS	60															
443.2	Silty sand, trace gravel						443													
0.6	Compact																			
	(FILL)																			
	Sand, organics		2	SS	10		442									8 79 12 1				
441.5	Loose Grey Wet																			
2.3	Peat, fine fibrous		3	SS	4															
	Dark brown						441													
	Wet																			
			4	SS	5															
	Sand seams						440													
	Grey		5	SS	4															
439.3	Sand, trace silt																			
4.5	Loose to Grey Wet		6	SS	1		439													
	very loose																			
			7	SS	5		438									0 96 (4)				
			8	SS	7															
	trace gravel						437													
	occasional cobbles		9	SS	8															
436.2	End of borehole																			
7.6	Refusal on probable bedrock																			
		</																		

**RECORD OF BOREHOLE No BC-3**

1 of 1

**METRIC**

G.W.P. 5222-05-00 LOCATION Burying Creek Coords: 5 255 555.3 N; 364 767.1 E ORIGINATED BY F.P.  
DIST Algoma HWY 129 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY A.K.  
DATUM Geodetic DATE December 12, 2014 CHECKED BY M.V.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT  γ  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
								○ UNCONFINED      + FIELD VANE									
								● QUICK TRIAXIAL      × LAB VANE									
							WATER CONTENT (%)										
444.2	Ground Surface							20	40	60	80	100					
0.0	230mm asphalt over gravelly sand (PAVEMENT FILL)		1	SS	78		444										
443.6	Silty sand some gravel, trace clay																
0.6	Very loose to Loose (FILL)		2	SS	7		443										
	organics						442										
	Wet		3	SS	1												
441.3	Peat, fine fibrous wood chips																
2.9	Dark brown		4	SS	2		441										
			5	SS	4												
439.9	Sand, trace silt, cobbles						440										
4.3	Very dense Grey Wet		6	SS	30/3cm												
439.4	End of borehole																
4.8	Sampler 6: Sampler bouncing																



## RECORD OF BOREHOLE No BC-4

1 of 1

## METRIC

G.W.P. 5222-05-00

LOCATION

Burving Creek

Coords: 5 255 564.4 N: 364 778.2 E

ORIGINATED BY F.P.

DIST Algoma HWY 129

BOREHOLE TYPE Tripod + Casing

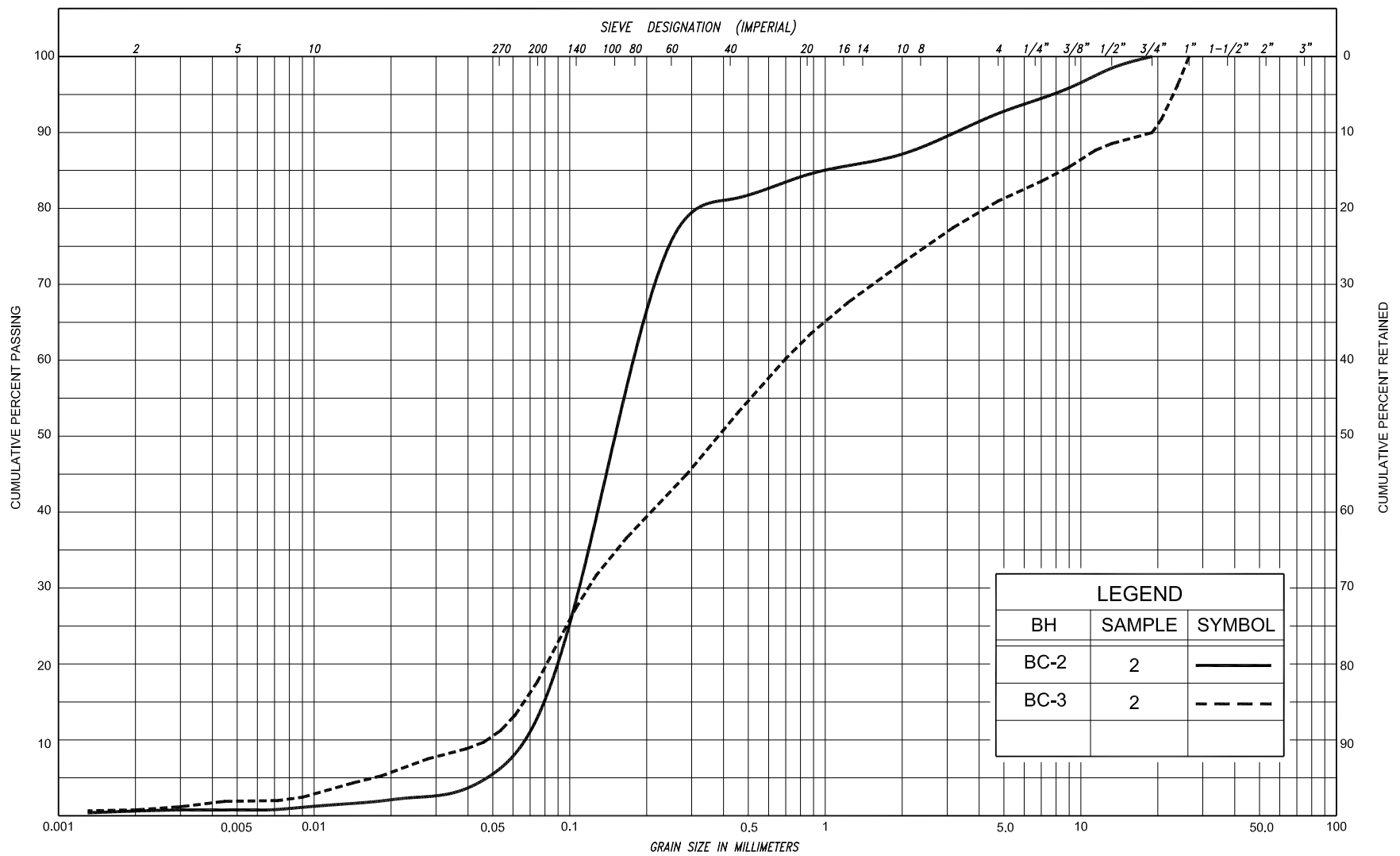
COMPILED BY A.K.

DATUM Geodetic

DATE January 15, 2015

CHECKED BY M.V.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  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												
								20 40 60 80 100										20 40 60		
								20 40 60 80 100										20 40 60		
441.7	Ground Surface					▽* ▼*	441													
0.0	Peat, fine fibrous		1	SS	1															
	Dark brown																			
440.6		2	SS	4										327						
1.1	Silty sand to sand organics, layers of amorphous peat							440												
	Very loose Grey/ Wet to compact dark brown	3	SS	1																
		4	SS	1				439												
		5	SS	2																
		6	SS	4		438														
		7	SS	27		437														
436.7	End of borehole																			
5.0																				

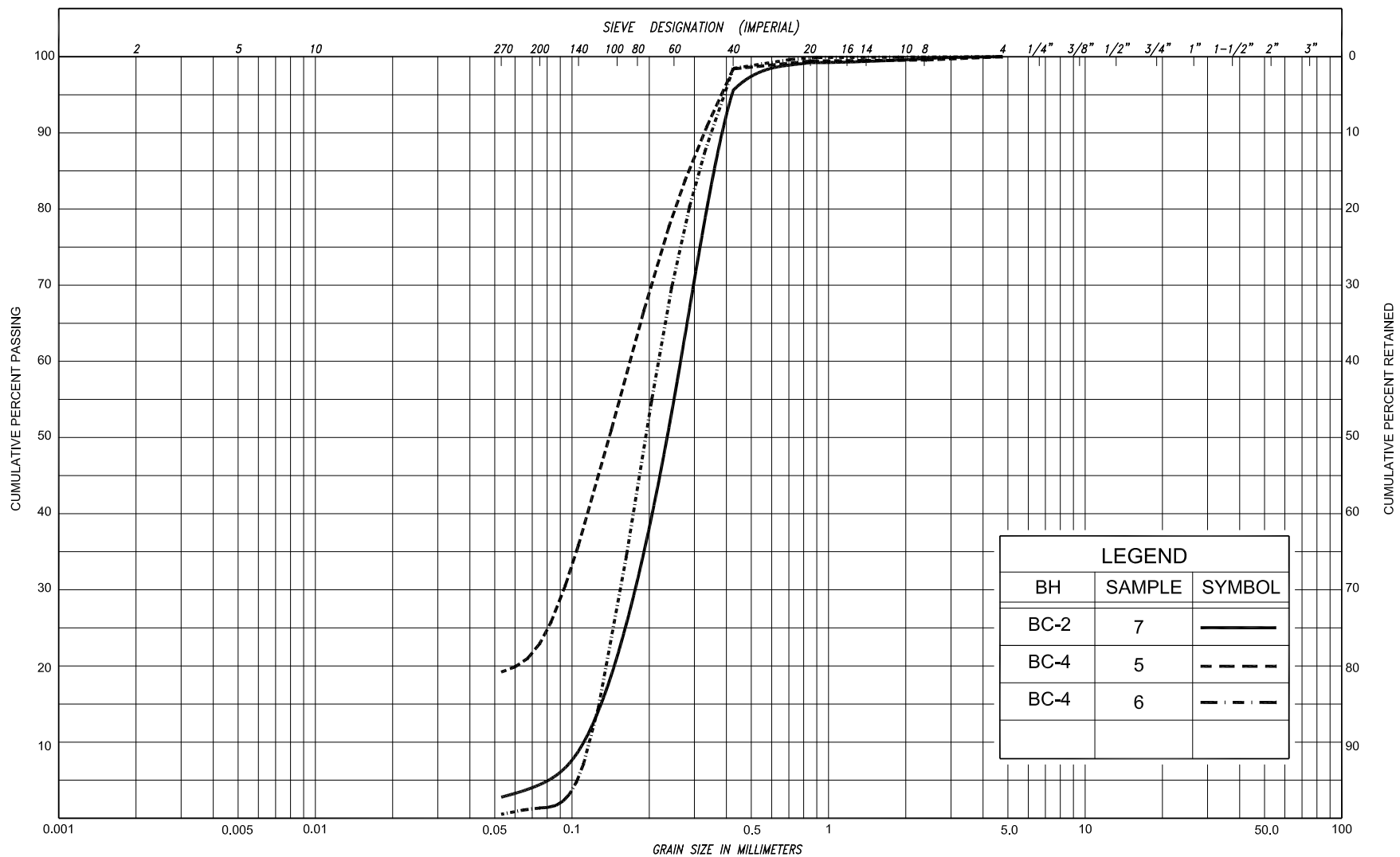


SILT & CLAY					FINE		MEDIUM		COARSE	GRAVEL			COBBLES	UNIFIED		
					SAND											
CLAY	FINE		MEDIUM		COARSE	FINE		MEDIUM		COARSE		GRAVEL			COBBLES	M.I.T.
	SILT					SAND										
CLAY		SILT			V. FINE	FINE	MED.	COARSE		GRAVEL					U.S. BUREAU	
					SAND											



**GRAIN SIZE DISTRIBUTION**  
 SILTY SAND, trace to some gravel, trace clay  
 (FILL)

FIG No. BC-GS-1  
 HWY: 129  
 G.W.P. No. 5222-05-00



SILT & CLAY				FINE		MEDIUM		COARSE	GRAVEL			COBBLES	UNIFIED		
				SAND											
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

SAND, trace to some silt

FIG No. BC-GS-2

HWY: 129

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



**Table 1: Summary of Geotechnical Parameters for Slope Stability Analysis**

Material Number	1	2	3	4
Soil Description	Granular Fill	Sand	Peat	Till
Soil Parameters <sup>†</sup>				
Unit Weight (kN/m <sup>3</sup> )	20	20	12	21
Cohesion, C (kPa)	0	0	3	0
Friction Angle, phi (Deg.)	33	33	28	35
Soil Sequence	1	3	2	4

<sup>†</sup> Derivation of Parameters - 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, 4<sup>th</sup> Edition) and our previous experience with similar materials.

\* with Sequence of Modelled Soil Stratigraphy from ground level down (1,2,3 etc.)

**Table 2: Summary of Results of Slope Stability Analysis**

Site ID	Temporary Slope (H:V)	Analyses Min. Achieved FOS	Analysis Method	Modelled Groundwater Level	Failure Mode	MTO Min. Required FOS	Meets Minimum stability requirements
Burying Creek	2:1	1.41	Morgenstern-Price	Base of excavation	Shallow circular	1.3	yes
	1.5:1	1.23	Morgenstern-Price	Base of excavation	Shallow circular	1.3	No

# 2015 National Building Code Seismic Hazard Calculation

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

January 12, 2017

Site: 47.4364 N, 83.2049 W User File Reference: Burying Creek

Requested by: , DST Consulting Engineering

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

Sa(0.05)	Sa(0.1)	<b>Sa(0.2)</b>	Sa(0.3)	<b>Sa(0.5)</b>	<b>Sa(1.0)</b>	<b>Sa(2.0)</b>	<b>Sa(5.0)</b>	<b>Sa(10.0)</b>	<b>PGA (g)</b>	<b>PGV (m/s)</b>
0.052	0.073	<b>0.071</b>	0.061	<b>0.051</b>	<b>0.032</b>	<b>0.016</b>	<b>0.0039</b>	<b>0.0017</b>	<b>0.041</b>	<b>0.040</b>

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

Ground motions for other probabilities:

Probability of exceedance per annum	0.010	0.0021	0.001
Probability of exceedance in 50 years	40%	10%	5%
Sa(0.05)	0.0050	0.017	0.028
Sa(0.1)	0.0085	0.026	0.042
Sa(0.2)	0.010	0.029	0.044
Sa(0.3)	0.0093	0.027	0.040
Sa(0.5)	0.0071	0.023	0.034
Sa(1.0)	0.0036	0.013	0.020
Sa(2.0)	0.0014	0.0058	0.010
Sa(5.0)	0.0004	0.0013	0.0022
Sa(10.0)	0.0003	0.0007	0.0010
PGA	0.0047	0.015	0.024
PGV	0.0040	0.015	0.024

## References

**National Building Code of Canada 2015 NRCC no. 56190;**  
**Appendix C:** Table C-3, Seismic Design Data for Selected Locations in Canada

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

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

See the websites [www.EarthquakesCanada.ca](http://www.EarthquakesCanada.ca) and [www.nationalcodes.ca](http://www.nationalcodes.ca) for more information

Aussi disponible en français



Natural Resources  
Canada

Ressources naturelles  
Canada



**Appendix E**

**NON-STANDARD SPECIAL  
PROVISION**

## **COBBLES AND BOULDERS - Item No. 1**

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### **Non-Standard Special Provision**

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

Cobbles and boulders were identified within the subsurface soil layers within the borehole locations. The contractor should be aware of the potential for encountering frequent cobbles or boulders at the site during excavation or installation of temporary roadway protection.

## **GROUNDWATER- Item No. 2**

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### **Non-Standard Special Provision 2**

---

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

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

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



### USE OF VIBRATORY EQUIPMENT FOR PILING - Item No. 3

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#### Non-Standard Special Provision

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This special provision covers the use of vibratory equipment for installation or extraction of piles. The Contractor is advised of the following foundation conditions:

The soil below this site is extremely susceptible to consolidation as a result of nearby vibration such as that from a vibratory sheet pile driver/extractor. If vibratory equipment is used, control vibration to ensure no deformation of the culvert occurs as a result. Monitor and document culvert movements accurately during vibration to confirm satisfactory culvert performance.

**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								
<b>Physical Tests</b>								
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	
<b>Leachable Anions &amp; Nutrients</b>								
Chloride		<5.0		5.0	ug/g	09-NOV-16	09-NOV-16	R3592422
<b>Anions and Nutrients</b>								
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								
<b>Physical Tests</b>								
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	
<b>Leachable Anions &amp; Nutrients</b>								
Chloride		<5.0		5.0	ug/g	09-NOV-16	09-NOV-16	R3592422
<b>Anions and Nutrients</b>								
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								
<b>Physical Tests</b>								
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	
<b>Leachable Anions &amp; Nutrients</b>								
Chloride		63.7		5.0	ug/g	09-NOV-16	09-NOV-16	R3592422
<b>Anions and Nutrients</b>								
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								
<b>Physical Tests</b>								
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	
<b>Leachable Anions &amp; Nutrients</b>								
Chloride		20.5		5.0	ug/g	09-NOV-16	09-NOV-16	R3592422
<b>Anions and Nutrients</b>								
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.

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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.