



**FOUNDATION INVESTIGATION AND DESIGN REPORT
GRAYSTONE LAKE TIMBER CULVERT REPLACEMENT
HIGHWAY 599
THUNDER BAY DISTRICT
AGREEMENT NO.: 4014-E-0023
SITE NO.: 48W-189/C
GEOCRES NO. 52G-13
GWP: 6355-14-00**

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PART 1: FACTUAL INFORMATION

1. INTRODUCTION

DST Consulting Engineers Inc. (DST) has been retained by the prime consultant, Planmac Engineering Inc. (Planmac), to conduct a foundation investigation and provide a foundation design report for the proposed culvert replacement on Highway 599. This work was carried out under Agreement No.: 4014-E-0023. This report addresses the field investigation, laboratory test program, factual report on conditions (Part 1) and recommendations for design and construction for the proposed culvert replacement (Part 2).

This report was initially submitted to Planmac and MTO in March 2017. It has been re-issued, at the request of WSP on behalf of MTO, with revisions to accommodate its updated (90%) design, including references for dewatering, revisions to both its Drawings (Borehole Location Plan and Borehole Location Plan Soil Strata). WSP has requested the deletion of the following sections; precast concrete open footings, corrugated steel plate culverts, retaining walls and recommendations pertaining to advantages and disadvantages of precast concrete open footings and corrugated steel plate culverts.

2. SITE DESCRIPTION

The site is located on Highway 599, approximately 330 km east of Kenora (Latitude 49.9815, Longitude -91.063), Station 11+100, in unsurveyed territory, in the District of Thunder Bay.

Highway 599 follows a sweeping course on a narrow peninsula between Ten Mile Lake to the north, and Graystone Lake to the south. At the site location, the two lakes are separated by the width of the highway and joined by the twin culverts beneath. There is no distinct drainage channel or section of river/creek joining the two lakes. The twin culverts are skewed beneath the highway and aligned north-west to south-east. The topography surrounding the site is generally of low relief, with gently undulating ground, over shallow bedrock. The area is densely vegetated with

natural boreal forest.

The existing twin cell timber culvert structures at this location are 2.4 m in width, 1.7 m in height and 26.9 m in length, for each cell, with a thickness of soil cover of approximately 2.0 m. The existing culverts were built in 1899 as stated in the Ontario Structure Inspection Manual (OSIM), and inspection by others indicates discoloured timbers, especially the east end, crushed bottom timber at the North and Centre cells, separated wall timbers between stacked timber, and decay of timber at water level.

The embankment height at the culvert location is approximately 3.6 to 4.0 m with existing embankment side slopes of approximately 2H:1V. The condition of the existing twin culverts at the time of the inspection is shown in Figures 2-1 and 2-2 below.



Figure 2-1 Location of existing culvert at Highway 599 (Looking East)



Figure 2-2 Location of existing culvert at Highway 599 (Looking West)

3. REGIONAL GEOLOGY

Geological information is available from the published *Ontario Geological Survey Map #52GNW* by the *Ontario Ministry of Natural Resources* for the Press Lake area. The map indicates that the local area landform 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 is generally thin and discontinuous. Much of the glacial overburden consists of boulders, and sand-rich till that was transported only a short distance by the ice.

Rock knobs (RN) are the dominant bedrock landform in the Press Lake map-area. The bedrock is often covered by a thin, discontinuous mantle of till, making the complex terrain unit (RN(MG/R)) quite common. Other subordinate landforms associated with bedrock knobs include glaciofluvial outwash (GO) and glaciolacustrine plains (LP). Bedrock knobs occur as a subordinate land form in a unit dominated by bedrock ridges north of Watcomb and Whiterock Lakes in the northeastern part of the map-area.

As indicated on the Bedrock Geology of Ontario West Central Sheet Map 2542, the site is underlain by supracrustal rocks of Archaen age consisting of (5) mafic to intermediate

metavolcanic rocks, comprising variably of basaltic and andesitic flows, tuffs and breccias, chert, iron formation, minor metasedimentary and intrusive rocks, and related migmatite rocks.

4. INVESTIGATION PROCEDURES AND LABORATORY TESTING

Site work was carried out in two phases. In phase one, a total of two boreholes (BH2 and BH4) and four hand auger holes (HA1A, HA1B, HA1C and HA3) were advanced to depths ranging from 7.2 m to 10.0 m and 0.6 m to 1.5 m respectively on September 28th to September 30th, 2015 utilizing a CME 55 drill rig equipped for geotechnical drilling, and a hand auger, with site supervision provided by DST. The hand augered holes reached refusal at shallow depths on rockfill and/or presumed bedrock. The second phase involved the drilling of two supplementary boreholes (BH5 and BH6) to depths of 5.4 m and 5.1 m respectively. Bedrock was proven to a depth of 3m in each of the supplementary boreholes. The second phase of the field investigation was carried out on 8th June 2016 utilizing a CME 55 drill rig equipped for geotechnical drilling, operated by Paddock Drilling Ltd. of Winnipeg, and supervised by DST.

The borehole and hand auger hole locations and cross sections are shown on the Borehole Location Plan and Drawings 1 and 2 in Appendix C. Borehole 2 was positioned north of the existing culvert at Station 11+105, 3.6 m left of centreline, and advanced to a depth of 10.0 m below existing surface. Borehole 4 was positioned south of the existing culvert at Station 11+088, 4.0 m right of centreline, and advanced to a depth of 7.2 m below existing surface. Borehole 5 was positioned north of the existing culvert at Station 11+096, 17.1 m right of centreline, and advanced to a depth of 5.4 m below existing surface. Borehole 6 was positioned south of the existing culvert at Station 11+090, 11.3 m left of centreline, and advanced to a depth of 5.1 m below existing surface.

Hand auger 1A was located at the inlet of the existing culvert at Station 11+100, 13.8 m left of centreline, and advanced to a depth of 1.0 m below surface. Hand auger 1B was located at the inlet of the existing culvert at Station 11+097, 14.0 m left of centreline, and advanced to a depth of 1.3 m below surface. Hand auger 1C was located at the inlet of the existing culvert at Station 11+092, 14.3 m left of centreline, and advanced to a depth of 0.6 m below surface. Hand auger 3 was located at the outlet of the existing culvert at Station 11+101, 19.7 m right of centreline, and advanced to a depth of 1.5 m below surface.

The borehole locations are referenced to the MTO Station numbering system as indicated on the drawings provided by the Ministry. The ground surface elevation at the borehole locations were

surveyed by DST personnel and referenced to benchmark 417.654 m N and W in root F0 0.30Φ Spruce (N = 5538789.7 m, E = 228602.2 m) as indicated on the drawings provided by the Ministry. Table 4-1 summarizes the detail of borehole locations and depths.

All boreholes were backfilled with either bentonite chips or a cement bentonite grout as described in Ontario Regulation 903 and its amendments and presented in Table 4.1 below. Augured boreholes were decommissioned by backfilling to the bottom of the road base with bentonite chips. Boreholes advanced by wash boring and cored for bedrock were decommissioned by backfilling with cement bentonite grout to ground surface.

The fieldwork was supervised on a full-time basis by DST personnel who located the boreholes in the field, arranged for clearance of subsurface utilities, supervised the drilling and in-situ testing operations, retrieved samples, logged the boreholes, and supervised the backfilling of boreholes and reinstatement of drilling locations. Soil samples were obtained from the auger flights and from the split spoon sampler used for the Standard Penetration Test (SPT). The SPT testing was carried out in accordance with the procedures described in ASTM D1586. 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 geotechnical testing laboratory in Thunder Bay for further analyses.

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 testing included chemical tests, natural moisture contents, particle size analyses on selected soil samples, and point load index testing on selected sample of rock core. A total of twenty-nine (29) natural moisture contents and nine (9) particle size analyses, seven (7) point load tests and two (2) sets of chemical tests have been carried out. Laboratory test results are presented in the Boreholes Logs and graphical plots attached in Appendix D (Enclosures).

Table 4-1: Details of Borehole Locations

Borehole ID	Station	Elevation (m)*	Depth (m)	Offset (m)	Completion Details
HA 1A	11+100	414.3	1.0	13.8 Lt	Borehole backfilled with bentonite chips from bottom to ground surface.
HA 1B	11+097	414.7	1.3	14.0 Lt	Borehole backfilled with bentonite chips from bottom to ground surface.
HA 1C	11+092	415.1	0.6	14.3 Lt	Borehole backfilled with bentonite chips from bottom to ground surface.
BH2	11+105	417.3	10.0	3.6 Lt	Borehole backfilled with cement bentonite grout from the bottom to ground surface.
HA3	11+101	413.7	1.5	19.7 Rt	Borehole backfilled with bentonite chips from bottom to ground surface.
BH4	11+088	417.3	7.2	4.0 Rt	Borehole backfilled with cement bentonite grout from the bottom to ground surface.
BH5	11+096	413.4	5.4	17.1 Rt	Borehole backfilled with cement bentonite grout from the bottom to ground surface.
BH6	11+090	415.3	5.1	11.3 Lt	Borehole backfilled with cement bentonite grout from the bottom to ground surface.

5. DESCRIPTION OF SUBSURFACE CONDITIONS

The subsurface conditions are presented based on the information obtained during the site investigation and the subsequent laboratory testing program.

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

- Embankment fill comprising loose to compact sand with gravel to some gravel, some silt to trace of silt is encountered in all four boreholes, and three hand auger holes. Occasional cobbles and boulders were encountered within the embankment fill.
- Topsoil was encountered at surface in hand auger holes (1A, 1B, 1C, and 3) and Borehole 5.
- Cobbles and boulders were encountered underlying the embankment sand fill, in Boreholes 2 and 5.
- This is underlain by compact sand and gravel with trace of silt, and trace of organic material in Borehole 5. Trace of organics was encountered within the sand and gravel layer in Borehole 5.
- A thin layer of very dense sand till was encountered below the sand and gravel layer in Borehole 5.

- Meta-volcanic bedrock was encountered underlying the embankment fill in Boreholes 4 and 6, below the cobbles and boulders in Borehole 2, and below the sand till in Borehole 5.

The soil strata at the culvert location have been summarized in Tables 5-1 and 5-2 and details descriptions are provided below.

Attempts to undertake SPT testing in the layer of cobbles and boulders in boreholes 2 and 5 were unsuccessful due to the SPT rod bouncing on cobbles/boulders. As a result, the wash boring method was used to penetrate this layer.

In the embankment fill layer, SPT 'N'-values of more than 50 are indicative of embedded cobbles and boulders within this layer and are not representative of the relative density of this material.

Table 5-1: Summary of soil strata at the culvert inlet and outlet location (HA1A, HA1B, HA1C, HA3, BH5 and BH6)

Layer	Depth (m)	Elevation (m)	Comments
Topsoil	0.0 to 0.2	414.3 to 414.1	HA 1A
	0.0 to 0.2	414.7 to 414.5	HA 1B
	0.0 to 0.2	415.1 to 414.9	HA 1C
	0.0 to 0.2	413.7 to 413.5	HA3
	0.0 to 0.1	413.4 to 413.3	BH5
Fill – Sand	0.2 to 1.0	414.1 to 413.3	HA 1A
	0.2 to 1.3	414.5 to 413.4	HA 1B
	0.2 to 0.6	414.9 to 414.5	HA 1C
	0.2 to 1.5	413.5 to 412.2	HA3
	0.1 to 0.8	413.3 to 412.6	BH5
	0.0 to 2.3	415.3 to 413.0	BH6
Cobbles and Boulders	0.8 to 1.5	412.6 to 411.9	BH5
Sand and Gravel	1.5 to 2.3	411.9 to 411.1	BH5
Till	2.3 to 2.4	411.1 to 411.0	BH5
Bedrock	2.4 to 5.4	411.0 to 408.0	BH5
	2.3 to 5.1	413.0 to 410.2	BH6

Table 5-2: Summary of soil strata at the embankment location (BH2 and BH4)

Layer	Depth (m)	Elevation (m)	Comments
Fill – Sand	0.0 to 3.6	417.3 to 413.7	BH2
	0.0 to 4.0	417.3 to 413.3	BH4
Cobbles and Boulders	3.6 to 6.9	413.7 to 410.4	BH2
Bedrock	6.9 to 10.0	410.4 to 407.3	BH2
	4.0 to 7.2	413.3 to 410.1	BH4

5.1 Topsoil

Topsoil was encountered at surface in Hand augers 1A, 1B, 1C, 3 and Borehole 5 at depths from 0.0 to 0.2 m (Elev. 414.3 to 414.1 m, Elev. 414.7 to 414.5 m, Elev. 415.1 to 414.9 m, Elev. 413.7 to 413.5 m and Elev. 413.4 to 413.3 m).

5.2 Fill – Sand

Sand fill with various portions of gravel and silty to trace silt was encountered below the topsoil in Boreholes 1A, 1B, 1C, 3, at surface in Boreholes 2, 4 and 6 and below the topsoil in Borehole 5 at depths from 0.2 to 1.0 m (Elev. 414.1 to 413.3 m), 0.2 to 1.3 m (Elev. 414.5 to 413.4 m), 0.2 to 0.6 m (Elev. 414.9 to 414.5 m), 0.2 to 1.5 m (Elev. 413.5 to 412.2 m), 0.0 to 3.6 m (Elev. 417.3 to 413.7 m), 0.0 to 4.0 m (Elev. 417.3 to 413.3 m), 0.0 to 2.3 m (Elev. 415.3 to 413.0 m) and 0.1 to 0.8 (Elev. 413.3 to 412.8 m) with thicknesses of 0.8 m, 1.1 m, 0.4 m, 1.3 m, 3.6 m, 4.0 m, 2.3 m and 0.7 m respectively.

SPT 'N' values vary from 2 to 30, indicating a very loose to compact condition. The higher SPT 'N' values (>50) within the layer likely indicate the presence of occasional cobbles and boulders within the material. The natural moisture contents of the sand fill material vary from 2 % to 24 %. The laboratory test results are summarized in Table 5-3.

Table 5-3: Summary of Particle size distribution - Fill – Sand

Laboratory Results – Particle size distribution	
Gravel %	12 to 28
Sand %	55 to 75
Fines %	8 to 33

5.3 Cobbles and Boulders

Inferred cobbles and boulders were encountered in Boreholes 2 and 5 at depths from 3.6 m to 6.9m

(Elev. 413.7 to 410.4 m) and 0.8 to 1.5 m (Elev. 412.6 to 411.9 m) with thicknesses of 3.3 m and 0.7 m respectively. A soil matrix for this deposit is a possibility, whether permeable sand and/or gravel, or less permeable soil.

5.4 Sand and Gravel

Sand and gravel was encountered below the cobbles and boulders in Borehole 5 at a depth of 1.5 to 2.3 m (Elev. 411.9 to 411.1 m), with a thickness of 0.8 m.

A SPT 'N' value of 27 was recorded, indicating a compact condition. Trace of organics was also encountered within the sand and gravel layer. The natural moisture content of the sand and gravel layer was 15 %.

5.5 Till

Sand with gravel and silt was encountered below the sand and gravel layer in Borehole 5 at depths of 2.3 to 2.4 m (Elev. 411.1 to 411.0 m) with a thickness of 0.1 m.

The natural moisture content of the material was 12 %. The laboratory test results are summarized in Table 5-4.

Table 5-4: Summary of Particle size distribution - Till – Sand and Gravel

Laboratory Results – Particle size distribution	
Gravel %	26
Sand %	50
Fines %	24

5.6 Bedrock

Bedrock was encountered in Boreholes 2, 4, 5, and 6 at depths of 6.9 m (Elev. 410.4 m), 4.0 m (Elev. 413.3 m), 2.4 m (Elev.411.0 m) and 2.3 m (Elev. 413.0 m) respectively. Therefore, the bedrock surface level is expected to vary considerably and unpredictably throughout the site,

The bedrock comprises of a mix of volcanic, volcanoclastic and igneous rocks that have been subjected to varying degrees of metamorphism to form stronger, tougher rocks that are more resistant to weathering. The rocks encountered during drilling at the site may be generally

described as Medium grey, very strong, fresh, fine grained, Meta-Volcanic, with closely to moderately spaced, very tight to partly open, smooth planar, clean to surface stained and quartz coated, joints, dipping at 20°-30°, and 60°-70°. A strongly developed Eutaxitic texture is present dipping at 70°-80° with flattened, elongate fragments of rock between 1-50mm in thickness and 5-20mm long. Local pyrite and chalcopyrite mineralization is present. The rock core photos and point load test results can be seen in Appendix D (Enclosures).

In Borehole 2, Total Core Recovery (TCR) and Solid Core Recovery (SCR) was 100%. Rock Quality Designation (RQD) was found to range from 74 % to 100 % indicating a Good to Excellent Rock. In Borehole 4, Total Core Recovery (TCR) and Solid Core Recovery (SCR) was found between 97 % and 100 %. Rock Quality Designation (RQD) was found to range from 87 % to 100 % indicating a Good to Excellent Rock. In Borehole 5, Total Core Recovery (TCR) and Solid Core Recovery (SCR) was found to range from 96 % to 100%. Rock Quality Designation (RQD) was found to range from 81 % to 100 % indicating a Good to Excellent Rock. In Borehole 6, Total Core Recovery (TCR) and Solid Core Recovery (SCR) was found to range from 85 % to 100%. Rock Quality Designation (RQD) was found to range from 62 % to 78 % indicating a Fair to Good Rock.

The estimates for unconfined compressive strength of the intact rock, as derived from the point load test results, vary between 162 and 249 MPa at Borehole 2, 168 and 186 MPa at Borehole 4, 231 MPa in Borehole 5 and 248 to 265 MPa in Borehole 6, indicating a very strong to extremely strong rock.

A summary of the bedrock conditions encountered in the boreholes, and the parameters determined in the laboratory are provided in Table 5.5 below.

Table 5-5: Bedrock Conditions and Parameters

ID	Bedrock Depth (m)	Bedrock Elevation	Core Length (m)	TCR (%)	SCR (%)	RQD (%)	Rock ¹ Quality	Point Load Index	UCS Strength (MPa)	Strength ² Classification
BH2	6.9	410.4	3.1	100	100	74-100	Fair to Excellent	7.7 to 11.8	162-249	R5 Very Strong
BH4	4.0	413.3	3.2	97-100	97-100	87-100	Good to Excellent	8.0 to 8.9	168-186	R5 Very Strong
BH5	2.4	411.0	3.0	96-100	96-100	81-100	Good to Excellent	11	231	R5 Very Strong
BH6	2.3	413.0	2.8	85-100	85-100	62-78	Fair to Good	11.8 to 12.6	248-265	R5/R6 Very Strong to Extremely Strong

¹After Table 3.10 CFEM 2006

²After Table 3.5 CFEM 2006

5.7 Groundwater

Groundwater levels in the boreholes where seepage was noted were measured upon completion of borehole drilling and prior to backfilling of the borehole. This information is included on the borehole logs in Appendix D.

At the time of the field investigation, groundwater was observed in Hand Auger 1A, 3 and in Boreholes 4, 5 and 6 on completion of drilling (see Table 5-6). The water level of the lakes was recorded during the first and second phase of the field investigation at elevations 413.7 m and 413.9 m respectively. The interpreted groundwater table at the site is close to or slightly above the lake level.

Given the permeable nature of the stratigraphy at this site, groundwater levels can be expected to vary as the lake water level varies with seasonal and local precipitation events.

Table 5-6: Groundwater Level Depths Measured in Boreholes

Borehole	Groundwater Depth (m)	Groundwater Elev. (m)
HA 1A	1.0	413.3
BH2	3.9	413.4
HA3	0.4	413.3
BH4	3.8	413.5
BH5	0.5	412.9
BH6	2.1	413.2

5.8 Chemical Test Results

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

The results are presented below in Table 5-7 and 5-8 and discussed in Section 7.16. Copies of the Laboratory Certificate of Analyses are provided in Appendix 'F'.

Table 5-7: Chemical Test Results – Soil Sample

Sample ID	Moisture (%)	Sulphate (mg/kg)	Chloride (mg/kg)	pH	Conductivity (umhos/cm)	Resistivity (ohm - cm)
BH2 @ 3.6 m depth	13.2	139	<20	6.04	148	6760

Table 5-8: Chemical Test Results – Water Sample

Sample ID	Sulphate (mg/L)	Chloride (mg/L)	pH	Conductivity (umhos/cm)	Resistivity (ohm - cm)
Lake sample	1.21	0.13	7.14	31.1	32155

6. MISCELLANEOUS

Site investigation fieldwork was carried out from September 25th to September 28th, 2015 (Phase 1) and June 8th, 2016 (Phase 2) utilizing a CME 55 drill rig supervised by DST personnel. Soil samples retrieved 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., and approved by Mike Fabius, Senior Geotechnical Engineer, P. Eng.

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PART 2: ENGINEERING DISCUSSIONS AND RECOMMENDATIONS

7. GENERAL

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

As discussed in Part I of the report, the generalized ground profile through the existing embankment and outside the embankment footprint is based on the conditions encountered in the boreholes, and hand auger holes and consists of the following:

- Embankment fill comprising loose to compact sand with gravel to some gravel, some silt to trace silt is encountered in all four boreholes and the four hand auger holes. Occasional cobbles and boulders was encountered within the embankment fill.
- Topsoil was encountered at surface in hand auger holes (1A, 1B, 1C, and 3) and Borehole 5.
- Underlying the embankment sand fill, cobbles and boulders were encountered in Boreholes 2 and 5.
- Which is underlain by compact sand and gravel with trace silt and trace of organics in Borehole 5.
- Very dense sand till was encountered below the sand and gravel layer in Borehole 5.
- Bedrock was encountered underlying the embankment fill in Boreholes 4 and 6, below the cobbles and boulders in Borehole 2, and below the sand till in Borehole 5.

As the proposed culvert is not expected to be permanently heavily loaded, a shallow foundation is considered suitable for this site. As the cross-sectional area of the proposed culvert is larger than the existing culvert, the overall effect on the culvert foundation soils can be considered negligible.

DST further understands that an open cut excavation will be carried out and a temporary bridge installed during replacement of the structure. At the time of this revision WSP has provided the 90% detailed design information. The final construction drawings should be reviewed by a geotechnical engineer to confirm the design satisfies the geotechnical recommendations.

DST also understood that there will be no embankment grade raises or widening as part of the culvert replacement work.

7.1 Replacement Structure

It is understood that a box culvert (4.8 m x 2.3 m) will replace the existing culvert structure with no anticipated grade raises.

The design of the replacement structure must be in accordance with the Canadian Highway Bridge Design Code CAN/CSA-S6-06 (CHBDC, 2006) and all relevant Ministry of Transportation specifications and guidelines.

7.2 Foundation Design

The replacement culvert foundations will be installed as deep as the depth of 4.8 m (Elev. 412.5 m) for the proposed box culvert. A 0.3 m thick bedding will be placed below the culvert foundation elevation. Therefore, the founding level will either be on the cobble layer or on the bedrock formation, or partly on both, and the proposed foundation materials will be adequate in terms of bearing capacity (i.e. no bearing capacity issue).

Additional excavation in the cobbles/bedrock will be undertaken, in order to place the granular bedding. All foundation preparation work 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 7.8 Bedding.

The depth to top of bedrock at the borehole locations range from 2.3 m to 6.9 m (Elev. 413.0 to 410.4 m). Therefore, the bedrock surface level is expected to vary considerably and unpredictably throughout the site, and bedrock material may be encountered above foundation level at some or

all of the culvert location. Allowance should be made for local variations in bedrock levels which will likely require rock excavation for footing construction. It is possible that bedrock blasting and levelling may be required for the box culvert base surface preparation. Excavation in the very strong to extremely strong Metavolcanic bedrock would require either pre-drilling and blasting, and/or hoe ramming depending on the required extent of rock removal. All blasting work would be carried out according to OPSS.PROV 120 (Explosives) and OPSS.PROV 202 (Rock Removal – Manual or Blasting). Presplitting, use of airleg jackhammers, or silent blasting using expansion agents are other means of controlled excavation in rock, whilst preserving the local integrity of the rock mass.

The geotechnical resistance for engineered fill was estimated for the Ultimate Limit State (ULS). The ULS can be used as Serviceability Limit State or SLS for the foundation since the foundation will be placed on a competent granular material (cobbles) or directly on competent bedrock. The resistance at ULS was calculated by applying a load resistance factor of 0.5 in accordance with the Bridge Design Code (CHBDC) CAN/CSA-S6-06 Section 6.6 and is shown in Table 7-1.

Table 7-1: Geotechnical resistances and reactions

Bearing Layer	Footing Size	Ultimate bearing capacity (kPa)	Resistance at ULS (kPa)	Resistance at SLS (kPa)
Cobble/Engineered Fill (Granular B)	B= 4.8 m	825	410	410

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 7.8 Bedding.

Should the precast box culvert founding level be on the bedrock, a bedding layer with a minimum thickness of 75 mm is required between the foundation and bedrock.

Groundwater was encountered at elevations approximately between 412.9 m (BH 5) and 413.5m (BH4) during the fieldwork investigation. However, groundwater levels will fluctuate over time, both with seasonal changes and after local precipitation events, and with the lake levels. The proposed founding depth of the foundation will fall below the groundwater table; therefore, soil dewatering work will be required for the foundation preparation.

Temporary bridge foundations should be designed by a suitably qualified professional engineer and take into account the soil/groundwater conditions as well as slope stability of embankments

and temporary excavations.

7.3 Lateral and Sliding Resistances

The analysis of horizontal and vertical effects of earth loads on the culvert can be performed considering the soil parameters provided in Table 7-2 and as described in Section 7.6.3.1 in the Canadian Highway Bridge Design Code. Temporary bracing and shoring may be designed using the lateral earth pressures given in Table 7-3 and, however the designer/contractor should verify the appropriate soil parameters for the designs of 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 7-2: Typical Soil Parameters for Earth Loads*

Soil type	Unit weight (kN/m ³)	Internal drained friction angle* (Degree)	Interface friction angle** δ (Degree)
Fill Sand	21	38	25
Cobble/Boulder	19	38	25

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

**Interface between soil and concrete

Table 7-3: Lateral Earth Pressure Coefficients

Soil type	Active Earth Pressure (K_a)	Passive Earth Pressure, (K_p)	Earth Pressure at Rest, (K_0)
Equation *	$\left(\frac{1 - \sin\phi}{1 + \sin\phi}\right)$	$\left(\frac{1 + \sin\phi}{1 - \sin\phi}\right)$	$(1 - \sin\phi)$
Fill Sand	0.23	4.26	0.38
Cobble/Boulder	0.23	4.26	0.38

* ϕ angle of internal friction

**The earth pressure coefficients provided here are for the normally consolidated soils condition considering fully mobilized condition

7.4 Staged Construction

It is understood a five-stage construction method as per WSP's 90% construction drawings (*Graystone Lake Culvert Staging Typical Sections November 27, 2018*) will be implemented to complete the culvert replacement as summarized below.

- Stage 1 - Construct temporary road widening on west side and divert traffic to the east side. Construct rock embankment widening for the detour. Install temporary modular bridge, place granular base and pave detour.
- Stage 2A – Install temporary roadway protection system. Install temporary dewatering and control flow system and divert flow through existing timber culvert south cell. Divert traffic to the west side. Excavate and install temporary diversion pipe and channel.
- Stage 2B – Relocate/install dewatering and flow control system and divert flow through temporary diversion pipe and channel. Remove existing timber culvert. Install new precast concrete box culvert and backfill. Place streambed material inside box culvert and within watercourse at new culvert inlet and outlet.
- Stage 2C – Relocate/install temporary dewatering and flow control system and divert flow through new culvert. Remove temporary diversion pipe and backfill diversion channel. Reinstatement embankment slopes and place rock protection. Complete backfill, reinstate roadway to final, install steel beam guide rails on east side and pave.
- Stage 3 – Relocate temporary concrete barrier, energy attenuators, install signage and pavement marking. Shift traffic to east side, remove temporary modular bridge, temporary footings, detour asphalt and re-grade granular shoulder. Backfill remaining roadway to final, reinstate embankment slopes and place rock protection. Install steel beam and guide rails. Pave and provide final pavement marking and signage. Remove temporary concrete barriers and traffic control.

To maintain traffic lanes during construction, an appropriate roadway protection system should be selected and designed by the contractor in accordance with the recommendations in this report and OPSS 539 "Construction Specification for Temporary Protection Systems".

Based on the presence of boulders and cobbles, the installation of a shoring system will require specialized equipment to drill through the fill and native soils. Piles should be seated in the bedrock. The proposed shoring system may require predrilling in those locations with a high concentration of boulders and cobbles, and tiebacks at different elevations in the excavation. Groundwater lowering is expected to be problematic due to the permeable granular nature of the

excavation soils as well as the shallow/uneven soil/bedrock interface and will require careful consideration prior to construction. The advantages and disadvantages of selected vertical shoring methods are discussed in further detail in section 8, Table 8-1 at the end of this report.

An experienced contractor should be consulted during the design process to confirm the suitability of the vertical shoring method for the subsurface conditions encountered during drilling, specifically the ability to advance piles through boulders, and achieve an effective groundwater cut-off at the uneven bedrock surface. Also, the contractor selected for the work should be experienced and prepared to handle unexpected and/or difficult soil/rock/groundwater conditions.

7.5 Temporary Road Widening

As discussed in Section 7.4 the five stages (1, 2A, 2B, 2C, and 3), construction will incorporate a temporary road widening. The stability of the temporary side slopes was evaluated using the computer program SLOPE/W developed by GeoStudio International Ltd. Based on the Morgenstern and Price method of slices, the static stability was assessed by calculating the factors of safety (FOS) along possible planes of failure in the proposed embankment fill and through the native soil. Two critical cross sections were selected for analyses based on the maximum slope height (4.6 m and 4.0 m at Sta. 11+090 and Sta. 11+110 respectively). The groundwater table is modelled below the base of the free-draining temporary fill slope (drained analysis).

Slope stability analyses indicate that the minimum factors of safety for embankment heights of 4.6 and 4.0 m with side slopes of 2H:1V exceed 1.3. This exceeds the MTO minimum requirement of FOS of 1.3 for a temporary slope condition.

The embankment backfill for the temporary lane should be constructed in accordance with OPSS 206 (Embankment Construction) and OPSD 208 (Benching of Earth slopes).

7.6 Excavation

Excavation will be required adjacent to the existing and replacement structure and will also require installation of a temporary diversion pipe and channel. An open excavation method can readily accommodate removal of large boulders, if encountered during excavation. As a minimum, the procedures should be in accordance with OPSS 902 "Construction Specifications for Excavating and Backfilling-Structures". Where temporary protection systems are required they shall be constructed in accordance with OPSS.PROV 539 "Construction Specification for Temporary Protection Systems" and Section 7.7 "Roadway Protection".

The side slopes of the excavations at the maximum anticipated depth (4.8 m) were evaluated based on the method of slices and the Morgenstern and Price method using the computer program SLOPE/W developed by GeoStudio International Ltd. The soil was assumed to be dewatered to a level at or below the excavation level, under the entire excavation footprint. The static stability was assessed by calculating the factors of safety (FOS) along possible planes of failure in the existing embankment fill and through the native soil.

Road widening with side slopes of between 1.5H:1.0V and 2H:1V or a vertical shoring method using a retaining wall can be used. The 1.5:1 slope requires a 1.0 m thick slope cover of high strength material such as well graded rockfill (OPSS 1004.05.05) whereas 2:1 slopes do not. Therefore, this arrangement is a considered to be a feasible option. Another alternative for minimizing the footprint of construction stages is to temporarily reduce the roadway level. Use of a temporary concrete block barrier wall or cantilever concrete wall or soldier pile wall with lagging will be required if the vertical shoring method is selected.

An excavation depth of up to approximately 4.8 m (Elev. 412.5 m) may be required for the staged construction. The stability of the excavation side slopes will be highly dependent on the contractor's methodology and ability to effectively dewater the excavation (See NSSP 3). The embankment slopes at the inlet and outlet should be 2H: 1V. The final embankment slopes should be reinstated as presented in Section 7.13, Embankment Slopes.

If organic materials are encountered during excavation, the excavations to remove these organics and wood should be completed in accordance with OPSS.PROV 209 "Construction Specification for Embankments Over Swamps and Compressible Soils".

For open excavations below the groundwater table, soil dewatering will be necessary to lower the

groundwater table and maintain stable excavation side slopes. It is recommended that side slopes no steeper than 2.0H: 1V are maintained, assuming the groundwater table directly below the entire excavation is kept below the deepest part of the base level of the excavation. The width of the excavation must be sufficient to accommodate the design culvert, any frost protection measures, and at least 1.5 m beyond the culvert perimeter for placement of the structural backfill zone, or as otherwise required by the culvert design.

Excavation in the very strong to extremely strong Metavolcanic bedrock would require either pre-drilling or blasting, and/or hoe ramming depending on the required extent of rock removal. All blasting work would be carried out according to OPSS.PROV 120 (Explosives) and OPSS.PROV 202 (Rock Removal – Manual or Blasting). Presplitting, use of airleg jackhammers, or silent blasting using expansion agents are other means of controlled excavation in rock, whilst preserving the local integrity of the rock mass. The contractor should be alerted of the shallow sloping bedrock, for example through a non-standard special provision (NSSP 2).

It is likely that the natural bedrock surface may be sloped and/or undulating rather than flat-lying when exposed during excavation. Therefore, provisions may need to be made for excavation of a bench or step in the bedrock surface, to allow bedding placement and compaction to proceed on a flat surface. Provision should also be made for addressing the presence of natural cobbles/boulders within the founding materials.

7.7 Roadway Protection

Roadway protection 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 above the water table and soils previously dewatered in the area of interest are classified as Type 2 and Type 3. Type 2 soils generally are very stiff, dense and can be penetrated with moderate difficulty by a small sharp object, low to medium natural moisture content and a medium degree of internal strength and has damp appearance after it is excavated. Type 3 soils generally are stiff to firm and compact to loose or are previously excavated soil, exhibit signs of surface cracking, exhibit signs of seepage, if it is dry, may run easily into a conical pile and have a low degree of internal strength. In accordance with O. Reg. 213/91, s.227 (3), if an excavation contains more than one type of soil, the soil shall be classified with the highest number as described in section 226. These should be assessed and confirmed in the field as construction progresses.

Since temporary roadway protection is required during the structure replacement, installation of a temporary concrete blocks barrier wall or cantilever concrete wall may be considered to ensure the stability of the bank and is a feasible option. Appropriate roadway protection system shall be selected by the contractor and designed by the contractor's qualified engineer. Temporary road way protection systems shall be constructed with the OPSS 539 "Construction Specification for Temporary Protection Systems". Potential roadway protection systems include a concrete block barrier wall or cantilever concrete wall and using temporary cut side slopes of 2.0H:1.0V with suitable soil dewatering. The advantages and disadvantages of using different road way protection systems are presented in Table 8-1.

The design of roadway protection may be performed using the typical soil parameters given in Table 7-2 and Table 7-3, however the designer/contractor should verify the appropriate soil parameters for the designs. As cobbles and boulders were encountered at borehole locations, the contractor should be prepared to handle this with the selection of adequate driving or vibratory equipment as well as steel thickness, for example through a non-standard special provision (NSSP 1, Appendix E). The construction methodology must be in accordance with all applicable standards and regulations related to the method proposed. The contractor's method and equipment must be suitable for the site conditions and materials used. This soil investigation does show evidence of shallow bedrock, cobbles and boulders along the soil profile, this may create a construction constrain to install any driven retaining wall.

The culvert foundation level preparation will require bedrock removal. Prepared bedrock surface should be protected from entering unsuitable material during the construction. In the event that the bedrock surface is contaminated with soil/dust, cleaning by water pressure jet will be required before concrete casting on the bedrock foundation.

7.8 Bedding

Any foundation soils and bedrock that could be disturbed shall be protected from construction activities by restricting access to culvert foundation area. The bottom of the excavation on which the culvert or granular pad is to rest shall not be disturbed. The bedding placement should commence immediately after the final removal of material to the foundation level has been completed. Construction of precast concrete box culverts is OPSS 422 "Precast Concrete Box Culvert".

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 culvert foundation will consist of both bedrock and cobbles. Therefore, the bedding shall be a minimum of 150 mm thick and extend to a minimum width (half of the width of culvert) beyond all sides of the culvert. The bedding material should consist of "Granular A or Granular B Type II" as per Soil Group I in accordance with Table 7.4 of the Canadian Highway Bridge Design Code. The "Granular A or Granular B Type II" shall be in accordance to OPSS.PROV 1010. The "Granular A or Granular B Type II" should be placed and compacted to 100 % 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, shall be loosely placed. Where cobbles and boulders are encountered without a soil matrix, a separation by non-woven geotextile shall be placed over the cobbles and boulders prior to placement of the "Granular A or Granular B Type II" bedding material. The placement of the geotextile shall be in accordance with OPSS 1860.

If construction is performed without soil dewatering, bedding material should consist of 19 mm Type I or II clear stone as defined in OPSS.PROV 1004.05.02. In that case a non-woven geotextile (OPSS 1860.07.05.01 Class II) with the filtration opening size (FOS) less than 135 µm will be required around the stone for separation. No compaction is required of the clear stone.

7.9 Sidefill and Overfill

The material used for culvert sidefill 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 shall be deposited uniformly on each side of the structure in order to prevent lateral displacement. The minimum width of the sidefill should be at least 1.0 m on each side of the culvert for constructability purposes. The sidefill should consist of "Granular A or Granular B Type II" and compacted to 95% of standard Proctor maximum dry

density.

Overfill should consist of “Granular A or Granular B Type II” and should be compacted to not greater than the compaction or equivalent stiffness of soils in the sidefill zone and bedding. Should open cobbles and boulders without a soil matrix be encountered, then the backfill materials should be separated from the adjacent cobbles/boulders with a non-woven Class II geotextile, with a filtration opening size of between 50 and 100 µm, specified in OPSS 1860 “Material Specifications for Geotextiles”.

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. Non-woven geotextile shall be installed to cover all exterior joints of the culvert, including the top slab. 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”.

7.10 Flow Control and Dewatering

The culvert is to be replaced by diverting the lake channel temporarily through a temporary CSP culvert. It is important to ensure that a flood in the channel does not cause damage to the partly constructed permanent works or to the temporary works.

Groundwater was encountered during the field investigation and groundwater levels are reported in Section 5.6 of the report. Based on the groundwater elevations encountered during the field investigation, dewatering work will be required for maintaining a dry excavation during the proposed culvert replacement work. The proposed excavation depth of approx. 4.8 m (412.5 m elevation) will be below the groundwater levels which were observed between elevations 412.9 m and 413.5 m and are likely to be at or above lake level at the time of construction.

In order to prevent back up of water from upstream and downstream, a dyke made of sand bags has sometimes been used as a hydraulic barrier. A suitable sump and pump system, possibly supported by an efficient well-point system, will be required to dewater and stabilize the excavation during installation. A well designed well-point system with a suitable diameter of well point at an appropriate spacing will perform better for working under dry condition and to prevent

disturbance of the excavation base through sand boiling and hydraulic heave. However, a wellpoint system can still have difficulty in cutting off water flow over and through the bedrock. It should be noted that depending on the season, and the depth of excavation, the amount of water flow through the culvert may vary. The contractor should be prepared to tackle this situation. The contractor should be alerted of the high-water table and surface water, for example through a non-standard special provision (NSSP 3).

A continuous dewatering operation must be provided to keep the culvert excavation stable and free of water. The excavation must be monitored daily throughout the duration of excavation until the completion of backfilling to confirm this. The dewatering system must be maintained, and the surrounding area monitored for impacts to items such as, but not limited to, settlement and groundwater usage. Dewatering can be difficult to achieve for flow over and within the bedrock. The control of water from the dewatering operation should be accordance with OPSS 517 "Construction Specification for Dewatering of Pipeline, Utility, and Associated Structure Excavation", and corresponding amendment SSP 517F01. In addition, the dewatering system will require preconstruction surveys and an experienced design engineer's approval.

7.11 Erosion Control

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

To prevent erosion of the surrounding soils at the inlet, rip-rap treatment shall 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 Sheetting”.

The outlet shall be rip-rapped to prevent erosion of the surrounding soils in accordance with OPSD 810.010 “General Rip-Rap Layout for Sewer and Culvert Outlets” and OPSS 511 “Construction Specification for Rip-Rap, Rock Protection, and Granular Sheetting”.

To prevent undermining of the bedding, cutoff walls should be installed at the culvert ends. Cutoff walls should be designed based on velocity of the water flow and the type of soil underneath. Cutoff walls should be extended beyond the culvert width and 1.0 m below the bedding layer as a minimum.

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

7.12 Frost Protection

In accordance with OPSD 3090.100 “Foundation Frost Depths for Northern Ontario”, the frost penetration at this location for sandy soil is about 3.0 m. The frost susceptible soils shall not be used adjacent to the culvert wall within the depth of frost penetration from the road surface.

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 3.0 m. Only the thin layer of till overlying the bedrock as encountered at one borehole can be considered frost susceptible. However, the majority of the site is underlain by cobble and granular material evaluated as non-frost susceptible soil. Therefore, frost protection measures for the culvert foundation may not be required for the site. However, if frost susceptible soil (>10% of 0.02mm particle) is encountered during culvert bedding preparation, the soil must be removed and replaced with Granular B, Type II.

7.13 Embankment Slopes

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

7.14 Construction Concerns

The main construction issues which 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, excavation below the water table, shallow bedrock conditions, excavation dewatering and reinstatement of the embankment fill. These items are important for the successful installation of the new culvert.

As discussed above, the contractor should be aware of potential subsurface conditions at each phase of the culvert replacement work, including the cobbles and boulders that were encountered at borehole locations, an effective soil/bedrock dewatering method, as well as the relatively shallow bedrock condition and for proper shoring design and installation.

A Geotechnical Engineer should inspect the condition of the foundation and surrounding soils before installation of bedding and other backfills and ensure the width of trench and trench wall slopes are suitable and ensure compliance with material placements and compaction methods.

7.15 Chemical Testing

The results of the analytical test conducted on water and soil samples at the site location have been presented in Table 5-7 and 5-8 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 and water samples were compared with applicable Canadian Standards Association (CSA) standards as shown in Table 7-4 below.

The chemical sulphate content analyses for representative soil and water sample tested indicate a sulphate concentration of 139 mg/kg or (0.0139 %) and 1.21 mg/L (0.000121%) in soil and water respectively. 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 value for the soil samples was reported to be 6.04 and 7.14, indicating a durable condition against corrosion. These results were evaluated using Table C1 of Building Research Establishment (BRE) Digest 363 (SD1 - 2005). The pH is greater than 5.5 indicating the concrete will not be exposed to attack from acids. The chloride content of the selected soil sample was also compared with the threshold level and present negligible concrete corrosion potential. Soil resistivity and conductivity was found to be 6760 ohm-cm and 148 umhos / cm respectively for the sample analysed from borehole BH2.

Table 7-4: Additional requirements for concrete subjected to Sulphate Attack

Class of Exposer	Degree of Exposer	Water soluble Sulphate in soil sample (%)	Cementing Material to be used
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

8. CLOSURE

This report has been prepared in advance of final design or construction details. The subsurface information and design recommendations presented above must be reviewed at the time of final design to determine if additional geotechnical investigation is required.

Table 8-1 summarizes the advantages and disadvantages of the use of concrete block or cantilever concrete walls and 1.5H: 1V side-slopes for roadway protection. Since it is a temporary roadway protection and factor of safety of 1.3 can be achieved, 1.5H: 1V side-slopes (which must include 1 m thickness of well graded rockfill) without surface water presence is considered to be a suitable option; however, design of roadway protection is the responsibility of the contractor as per the contract drawings.

Table 8-1: Advantages and Disadvantages of Roadway Protection Methods

Roadway Protection Option	Advantages	Disadvantages
Concrete block barrier wall or cantilever concrete wall	<ul style="list-style-type: none"> • Could be fabricated. • Reasonable cost of installation. • Increased erosion control capacity. • Can design with suitable factor of safety. 	<ul style="list-style-type: none"> • Requires concrete blocks/cantilever wall material transportation. • Requires maintaining a dewatered excavation, which can be challenging.
Soldier Pile Retaining Wall with concrete or wood lagging	<ul style="list-style-type: none"> • Robust solution • Relatively impermeable if properly installed, and effectively grouted. • Increased erosion control capacity. • Ease of culvert installation when working below ground water table. • Safer working area for construction 	<ul style="list-style-type: none"> • Difficult driving through boulders. • Strong to very strong bedrock. • Pre-drilling of rock sockets. • High installation cost. • Specialized construction equipment and design required. • Seepage cut-off at bedrock surface will be difficult – specialised grouting. • Increased construction time.
Temporary Cut Slope (sideslopes of 1.5H:1V)	<ul style="list-style-type: none"> • Does not require specialized equipment above water table. • Relatively short construction time. • Low construction cost. • Ease of backfill and reinstatement. • Design with suitable factor of safety. 	<ul style="list-style-type: none"> • Permeable soils and possibly bedrock will need extensive dewatering. Dewatering can be challenging. • Increased potential for surface and internal soil erosion due to drawdown requirements • Low factor of safety with increasing excavation depth. • Temporary surface protection of slope materials may be required • 1.5H:1V slope requires 1 m thickness of well-graded rockfill.

9. REFERENCES

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10. LIMITATIONS OF REPORT

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

For DST CONSULTING ENGINEERS INC.

Prepared by:



Selorm Danku, P.Eng.
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Approved by:



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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



TEN MILE LAKE

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



AG No 4014-E-0023
WP No 6355-14-00
SITE No 48W-0189/C0
GEOCRES No 52G-13

CULVERT REPLACEMENT
GRAYSTONE LAKE

STA 11+048 TO STA 11+148

Survey 14-07 Revised
BOREHOLE LOCATION PLAN

SHEET

1



FLOW

HA1A

BH2

HA1B

HA1C

BH6

PROPOSED CULVERT (OPTION 1)

EXISTING CULVERT

11+120

11+100

HA3

BH4

BH5

413.5

FLOW

413.0

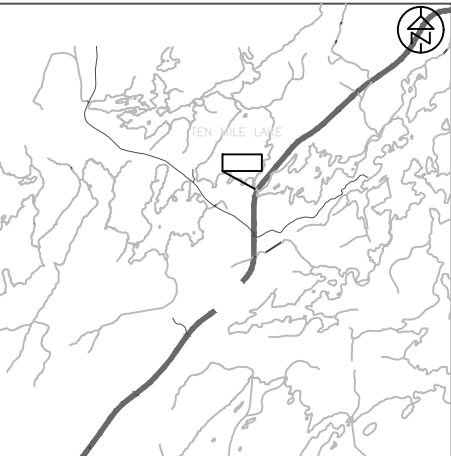
11+080

11+060

PLAN MAP

Scale 1:250

0 5m 10m



KEY PLAN

LEGEND



Borehole



Hand Auger Hole

No.	Elev. (m)	MTM Zone 15		Survey	
		North (m)	East (m)	Station	Offset
HA1A	414.3	5538767	228549	11+100	13.8 m LT
HA1B	414.7	5538765	228547	11+097	14.0 m LT
HA1C	415.1	5538760	228545	11+092	14.3 m LT
BH2	417.3	5538767	228560	11+105	3.6 m LT
HA3	413.7	5538751	228579	11+101	19.7 m RT
BH4	417.3	5538749	228556	11+088	0.7 m RT
BH5	413.4	5538748	228574	11+096	17.1 m RT
BH6	415.3	5538757	228547	11+090	11.3 m LT

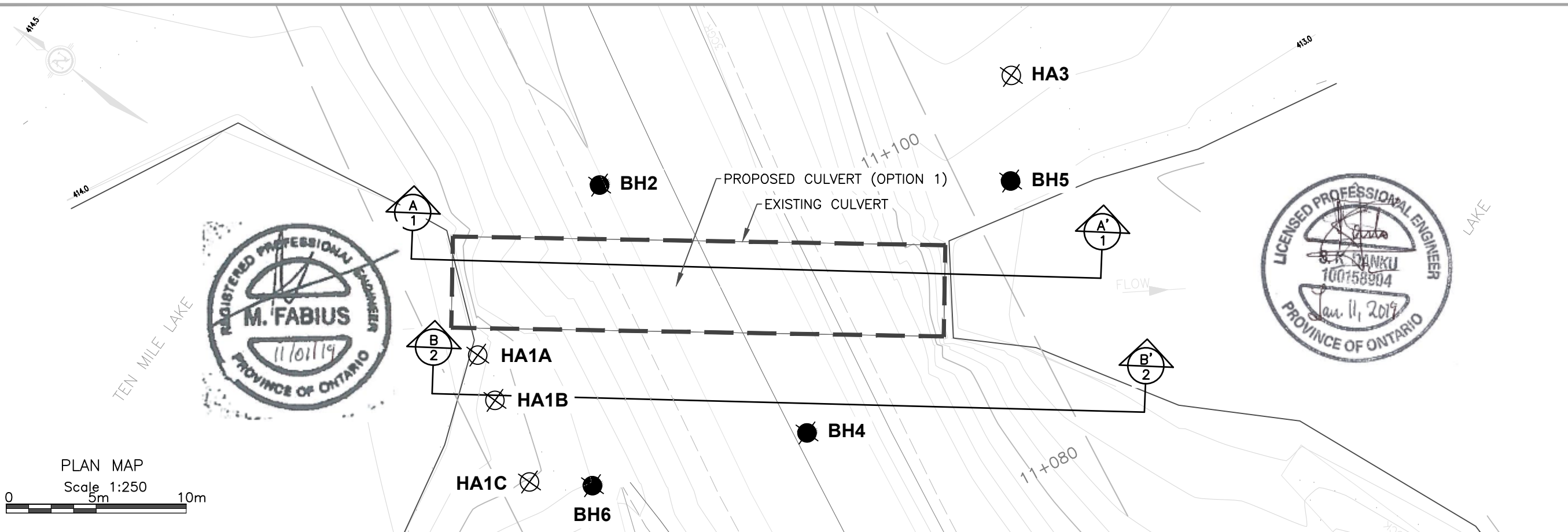
REV	DATE	ISSUE	DRAWN BY	CHECKED	APPROVAL
1	15-Jun-16	DRAFT	EM	PDS	MK
2	10-Dec-16	DRAFT	RW	PDS	MK
3	25-Oct-18	DRAFT	CS	SD	BV

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



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DRAWING 1



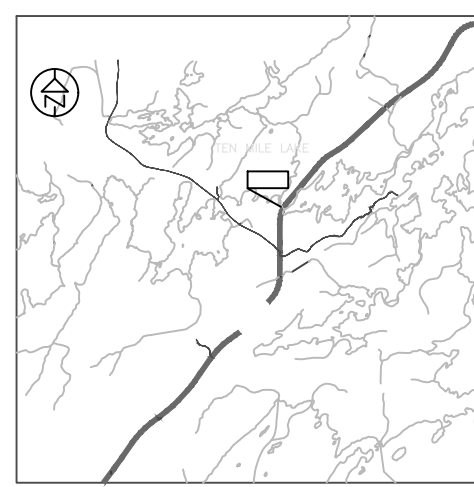
AG No 4014-E-0023
WP No 6355-14-00
SITE No 48C-0189/C0
GEOCRES No 52G-13

CULVERT REPLACEMENT
GRAYSTONE LAKE
STA 11+048 TO STA 11+148

SHEET
2

Survey 14-07 Revised

BOREHOLE LOCATION PLAN
SOIL STRATA



KEY PLAN

LEGEND

Borehole

Hand Auger Hole

SPT (BLOWS/300 mm)

Water level at completion of drilling

Fill

Organics

Topsoil

Till

Bedrock

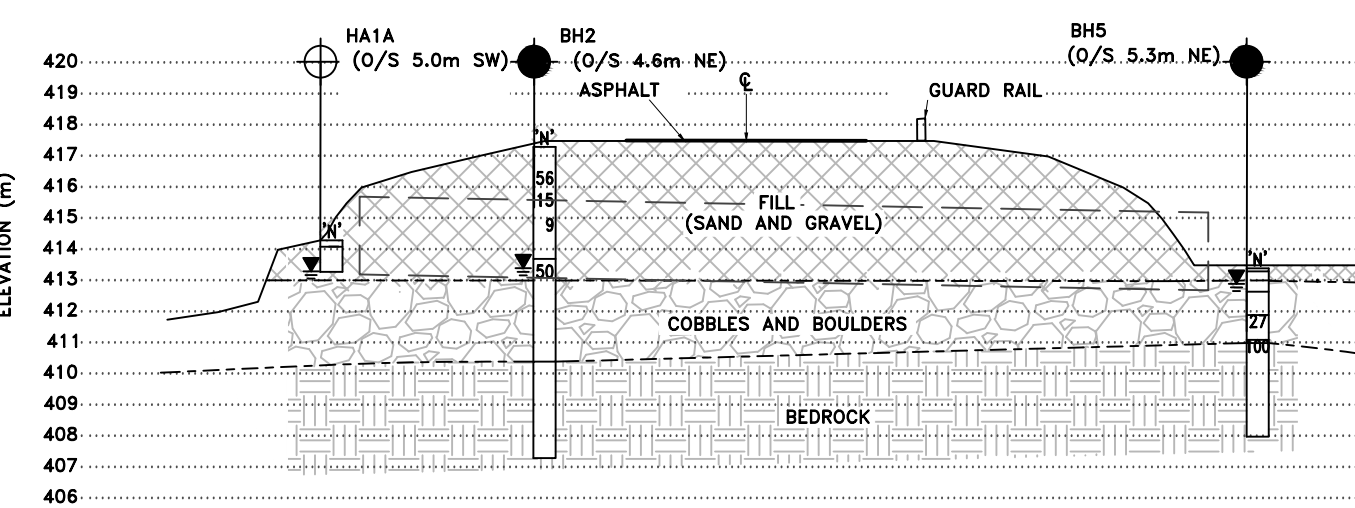
Sand

Silt

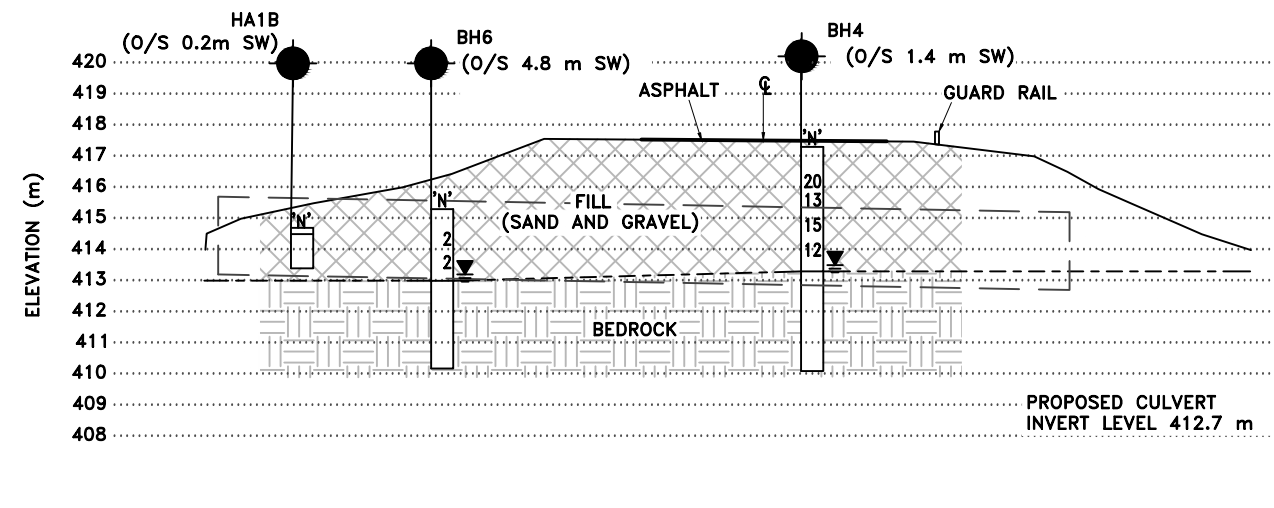
Clay

Sand & Gravel

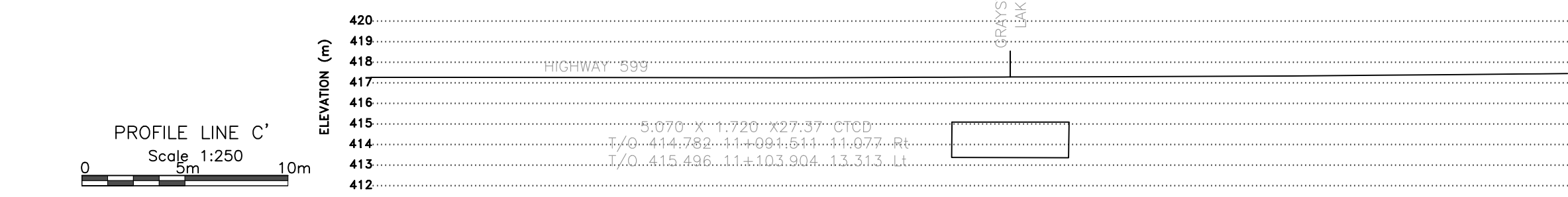
Boulders



CROSS SECTION A-A'
Scale 1:250



CROSS SECTION B-B'
Scale 1:250



PROFILE LINE C-C'
Scale 1:250

No.	Elev. (m)	MTM Zone 15		Survey	
		North (m)	East (m)	Station	Offset
HA1A	414.3	5538767	228549	11+100	13.8 m LT
HA1B	414.7	5538765	228547	11+097	14.0 m LT
HA1C	415.1	5538760	228545	11+092	14.3 m LT
BH2	417.3	5538767	228560	11+105	3.6 m LT
HA3	413.7	5538751	228579	11+101	19.7 m RT
BH4	417.3	5538749	228556	11+088	0.7 m RT
BH5	413.4	5538748	228574	11+096	17.1 m RT
BH6	415.3	5538757	228547	11+090	11.3 m LT

REV	DATE	ISSUE	DRAWN BY	CHECKED	APPROVAL
1	15-Jun-16	DRAFT	EM	POS	MK
2	7-Nov-16	DRAFT	RW	SD	MK
3	10-Dec-16	DRAFT	RW	POS	MK
4	3-Mar-17	DRAFT	RW	TL	MF
5	25-Oct-18	DRAFT	CS	SD	MF

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

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Appendix D
ENCLOSURES

RECORD OF BOREHOLE No HA1A

1 OF 1

METRIC

W.P. GWP 6355-14-00 LOCATION GRAYSTONE LAKE CULVERT ORIGINATED BY CH
 DIST HWY 599 BOREHOLE TYPE HAND AUGER COMPILED BY SA
 DATUM Geodetic DATE 2029 09 15 CHECKED BY BV

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				
							20	40	60	80	100	W _p	W	W _L			
414.3	GROUND SURFACE																
414.1	TOPSOIL		HA1	AS		414										18 73 (9) Water level at 1.0 m, taken after drilling AUGER REFUSAL	
0.2	FILL - SAND, SOME GRAVEL, TRACE SILT BROWN		HA2	AS													
413.3			HA3	AS													
1.0	END OF BOREHOLE AT 1.0 m																

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

ENCLOSURE 1

RECORD OF BOREHOLE No HA1B

1 OF 1

METRIC

W.P. GWP 6355-14-00 LOCATION GRAYSTONE LAKE CULVERT ORIGINATED BY CH
 DIST HWY 599 BOREHOLE TYPE HAND AUGER COMPILED BY SA
 DATUM Geodetic DATE 2029 09 15 CHECKED BY BV

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE LIQUID CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				
							20	40	60	80	100	W _p	W	W _L			
414.7	GROUND SURFACE																
414.5	TOPSOIL		HA1	AS		414						•					
0.2	FILL - SAND, WITH GRAVEL, TRACE SILT BROWN		HA2	AS									•				
			HA3	AS									•				
			HA4	AS									•				
413.4	END OF BOREHOLE AT 1.3 m															28 64 (8) AUGER REFUSAL	

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



ENCLOSURE 2

RECORD OF BOREHOLE No HA1C

1 OF 1

METRIC

W.P. GWP 6355-14-00 LOCATION GRAYSTONE LAKE CULVERT ORIGINATED BY CH
 DIST HWY 599 BOREHOLE TYPE HAND AUGER COMPILED BY SA
 DATUM Geodetic DATE 2029 09 15 CHECKED BY BV

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
415.1	GROUND SURFACE													
414.9	TOPSOIL - OGRANIC SAND		HA1	AS			415							19 65 (16)
0.2	FILL - SAND, SOME GRAVEL, SOME SILT		HA2	AS										
414.5														
0.6	END OF BOREHOLE AT 0.6 m													AUGER REFUSAL

ON_MOT-HIGH VANES GS-TB-020407 - GREYSTONE.GPJ DST_MIN.GDT 8/7/16

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

ENCLOSURE 3

RECORD OF BOREHOLE No BH2

1 OF 1

METRIC

W.P. GWP 6355-14-00 LOCATION GRAYSTONE LAKE CULVERT ORIGINATED BY CH
 DIST HWY 599 BOREHOLE TYPE HOLLOW STEM AUGER - 80 mm ID/ NQ CORING COMPILED BY SA
 DATUM Geodetic DATE 2028 09 15 CHECKED BY BV

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE □ QUICK TRIAXIAL × LAB VANE							
417.3	GROUND SURFACE						20	40	60	80	100				
	FILL - SAND, WITH SILT, SOME GRAVEL LOOSE TO COMPACT OCCASIONAL BOULDERS		AS1	AS											
			AS2	AS	56+										
			SS1	SS	15										
			SS2	SS	9										
			AS3	AS											
413.7 3.6	COBBLES AND BOULDERS														
			SS3	SS	50+										
410.4 6.9	BEDROCK														
	MEDIUM GREY LOCALLY STRIPED PALE GREY, VERY STRONG, FRESH, FINE GRAINED META-VOLCANIC BRECCIA WITH VERY CLOSELY TO MODERATELY SPACED, VERY TIGHT TO PARTLY OPEN, SMOOTH PLANAR, CLEAN TO SURFACE STAINED AND SILT COATED, JOINTS, DIPPING AT 0°-10°, 20°-30, 60°-70°, AND 70°-80°. LOCAL PYRITE AND CHALCOPYRITE MINERALIZATION.		RC1	RC	TCR 100% SCR 100%										

ON MOT-HIGH VANES GS-TB-020407 - GREYSTONE.GPJ DST_MIN.GDT 8/7/16

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

ENCLOSURE 4

RECORD OF BOREHOLE No HA3

1 OF 1

METRIC

W.P. GWP 6355-14-00 LOCATION GRAYSTONE LAKE CULVERT ORIGINATED BY CH
 DIST HWY 599 BOREHOLE TYPE HAND AUGER COMPILED BY SA
 DATUM Geodetic DATE 2029 09 15 CHECKED BY BV

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
413.7	GROUND SURFACE													
413.5	TOPSOIL - ORGANIC		HA1	AS										
0.2	FILL - SAND, SILTY, SOME GRAVEL DENSE TO COMPACT GREY		SS1	SS	30		413							
	OCCASIONAL BOULDERS													
412.2			SS2	SS	110									12 55 (33)
1.5	END OF BOREHOLE AT 1.5 m													AUGER REFUSAL

ON_MOT-HIGH VANES GS-TB-020407 - GREYSTONE.GPJ DST_MIN.GDT 8/7/16

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

ENCLOSURE 5

RECORD OF BOREHOLE No BH4

1 OF 1

METRIC

W.P. GWP 6355-14-00 LOCATION GRAYSTONE LAKE CULVERT ORIGINATED BY CH
 DIST HWY 599 BOREHOLE TYPE HOLLOW STEM AUGER - 80 mm ID/ NQ CORING COMPILED BY SA
 DATUM Geodetic DATE 1930 09 15 CHECKED BY BV

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
								20	40	60	80	100		
417.3	GROUND SURFACE													
	FILL - SAND, WITH GRAVEL, TRACE SILT COMPACT BROWN		AS1	AS										
			SS1	SS	20									15 76 (10)
			SS2	SS	13									
			SS3	SS	15									23 69 (8)
			SS4	SS	12									
413.3 4.0	BEDROCK													Water level at 3.8 m, taken after drilling
	DARK GREENISH GREY, VERY STRONG, FRESH, FINE GRAINED, META-VOLCANICS WITH CLOSELY TO MODERATELY SPACED, VERY TIGHT TO TIGHT, SMOOTH PLANAR TO SMOOTH STEPPED, CLEAN TO SURFACE STAINED AND OCCASIONAL QUARTZ-INFILLED, JOINTS, DIPPING AT 10°-20°, AND 30°-40. LOCAL PYRITE AND CHALCOPYRITE MINERALIZATION.		RC1	RC	TCR 97% SCR 97%									RQD = 87%
			RC2	RC	TCR 100% SCR 100%									RQD = 100%
410.1 7.2	END OF BOREHOLE AT 7.2 m													

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

RECORD OF BOREHOLE No BH5

1 OF 1

METRIC

W.P. GWP 6355-14-00 LOCATION GRAYSTONE LAKE CULVERT ORIGINATED BY AC
 DIST HWY 599 BOREHOLE TYPE SOLID STEM AUGER - 80 mm ID/ WASH BORING/ NQ CORING COMPILED BY EM
 DATUM Geodetic DATE 2016 08 06 CHECKED BY SD

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
413.4	GROUND SURFACE													
413.3 0.1	TOPSOIL		AS1	AS			413							Water level at 0.5 m, taken after drilling
412.7 0.8	SAND COARSE GRAINED, WITH GRAVEL TRACE ROOTLETS, BROWN/GREY OCCASIONAL COBBLES COBBLES AND BOULDERS		AS2	RC			412							
411.9 1.5	SAND AND GRAVEL TRACE, TRACE ORGANICS, BLACK TO GREY		SS3	SS	27		411							26 50 (24)
411.1 2.4	TILL SAND SOME GRAVEL, SOME SILT BEDROCK		SS4	SS	100+		410							RQD = 100%
			RC1	RC	TCR 100% SCR 100%		409							RQD = 81%
			RC2	RC	TCR 100% SCR 100%									
			RC3	RC	TCR 100% SCR 100%									
408.0 5.4	END OF BOREHOLE AT 5.4 m						408							

ON_MOT-HIGH VANES GS-TB-020407 - GREYSTONE.GPJ DST_MIN.GDT 8/7/16

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

ENCLOSURE 7

RECORD OF BOREHOLE No BH6

1 OF 1

METRIC

W.P. GWP 6355-14-00 LOCATION GRAYSTONE LAKE CULVERT ORIGINATED BY AC
DIST HWY 599 BOREHOLE TYPE HOLLOW STEM AUGER - 80 mm ID/ WASH BORING/ NQ CORING COMPILED BY EM
DATUM Geodetic DATE 2016 08 06 CHECKED BY SD

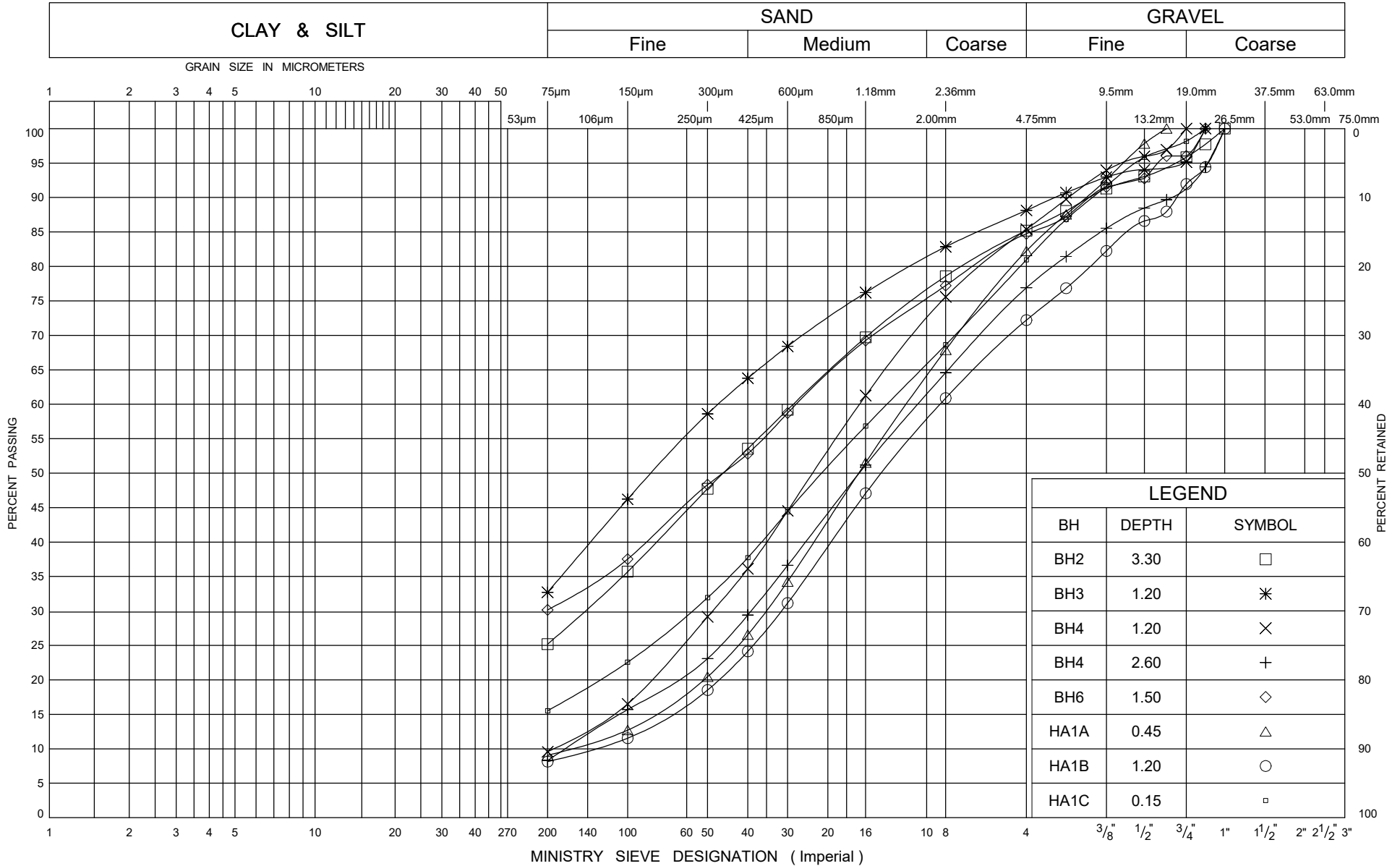
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
415.3	GROUND SURFACE													
	FILL SAND WITH SILT, SOME GRAVEL BROWN		AS1	AS			415							
	TRACE ORGANICS		AS2	AS	2		414							
			SS3	SS	2									15 55 (30)
413.0	BEDROCK						413							Water level at 2.1 m, taken after drilling
2.3	MEDIUM GREY TO DARK BLuish-GREY, VERY STRONG TO EXTREMELY STRONG, FRESH, FINE GRAINED META-VOLCANICS WITH CLOSELY TO MODERATELY SPACED, VERY TIGHT TO TIGHT, SMOOTH PLANAR TO SMOOTH UNDULATING, CLEAN TO SURFACE STAINED AND QUARTZ COATED, JOINTS, DIPPING AT 20°-30°, 30°-40°, 60°-70°, AND 70°-80°. EUTAXITIC TEXTURE IS PRESENT DIPPING AT 70°-80°. WITH FLATTENED, ELONGATE FRAGMENTS OF ROCK BETWEEN 1-15MM IN THICKNESS AND 5-60MM LONG. LOCAL PYRITE AND CHALCOPYRITE MINERALIZATION.		RC1	RC	TCR 98% SCR 98%		412							RQD = 78%
			RC2	RC	TCR 100% SCR 100%		411							RQD = 62%
410.2	END OF BOREHOLE AT 5.1 m													
5.1														

ON_MOT-HIGH VANES GS-TB-020407 - GREYSTONE.GPJ DST_MIN.GDT 8/7/16

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

ENCLOSURE 8

UNIFIED SOIL CLASSIFICATION SYSTEM



Ministry of
Transportation
Ontario

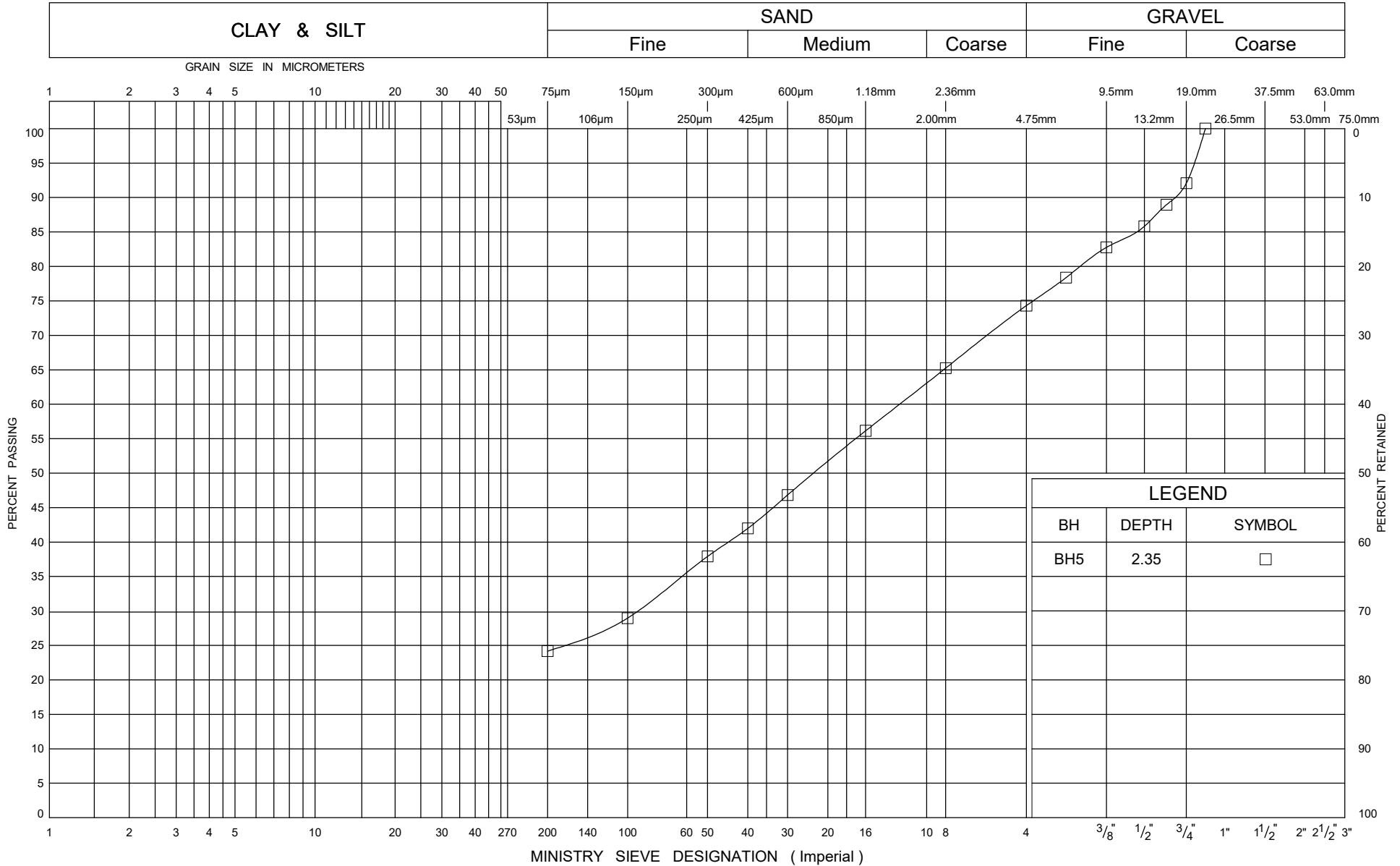
GRAIN SIZE DISTRIBUTION
FILL - SAND

ENCLOSURE 9

GWP 6355-14-00

GRAYSTONE LAKE

UNIFIED SOIL CLASSIFICATION SYSTEM




Ministry of
Transportation
Ontario

GRAIN SIZE DISTRIBUTION TILL


ENCLOSURE 10

W P GWP 6355-14-00


GRAYSTONE LAKE

	Project	15 Culverts Replacement AG # 4014-E-0023	Borehole Number	BH2		
	Project Number	GS-TB-020407	Box Number	1	of	1
	Client	Planmac Engineering	Depth (m)	6.9	to	10.0




	Project	15 Culverts Replacement AG # 4014-E-0023	Borehole Number	BH4		
	Project Number	GS-TB-020407	Box Number	1	of	1
	Client	Planmac Engineering	Depth (m)	4.0	to	7.2



	Project	15 Culverts Replacement AG # 4014-E-0023	Borehole Number	BH5		
	Project Number	GS-TB-020407	Box Number	1	of	1
	Client	Planmac Engineering	Depth (m)	2.4	to	5.4



	Project	15 Culverts Replacement AG # 4014-E-0023	Borehole Number	BH6		
	Project Number	GS-TB-020407	Box Number	1	of	1
	Client	Planmac Engineering	Depth (m)	2.3	to	5.1



POINT LOAD TEST RESULTS (diametric and axial)

PROJECT: Graystone Lake Culvert

JOB NO.: GS-TB-020407

This spreadsheet is based on information from 'Suggested Method for Determining Point Load Strength', *International Society for Rock Mechanics Commission on Testing Methods*, 1985.

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

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

Borehole #	Test No.	Depth (m)	Test Type	*Valid or Invalid	W(mm) (enter for axial only)	D(mm)	Load P P(lbf)	Load P (kN)	I _s = P/D _e ² (MPa)	F	I _{s(50)} (MPa)	Uniaxial Compressive Strength (MPa)
Pinder												
BH2	PL#1	7.00	DIAM	V		36.3	2640		8.91	0.87	7.72	162
	PL#2	9.30	DIAM	V		36.3	4050		13.67	0.87	11.84	249
BH4	PL#3	4.30	DIAM	V		36.3	2740		9.25	0.87	8.01	168
	PL#4	5.60	DIAM	V		36.3	3030		10.23	0.87	8.86	186
BH5	PL#5	3.50	DIAM	V		63.3	8910		9.89	1.11	11.00	231
BH6	PL#6	2.70	DIAM	V		63.3	10210		11.33	1.11	12.60	265
	PL#7	4.90	DIAM	V		63.3	9580		10.64	1.11	11.83	248
										Min.	7.7	162
										Avg.	10.3	216
										Max.	12.6	265

Appendix E

**NON-STANDARD SPECIAL
PROVISION**

COBBLES IN THE FILL STRATUM - Item No. 1

Non-Standard Special Provision

This special provision covers the cobbles and boulders in subsurface stratum.

The Contractor is advised of the following foundation conditions:

Occasional cobbles were identified within the fill within the advanced borehole locations. The contractor shall aware of potential for encountering cobbles or boulders at the site during excavation or installation of temporary roadway protection.

BEDROCK - Item No. 2

Non-Standard Special Provision

This special provision covers the presence of bedrock at the culvert replacement location.

The Contractor is advised of the following foundation conditions:

Shallow sloping bedrock is to be expected at the culvert replacement location. Excavation in the strong to extremely strong bedrock would require either pre-drilling or blasting, and/or hoe ramming depending on the required extent of rock removal. The contractor shall be aware of encountering bedrock at the site during excavation or installation of temporary roadway protection.

WATER LEVEL FLACTUATION - Item No. 3

Non-Standard Special Provision

This special provision covers the fluctuating water level.

It should be noted that depending on the season, depth of excavation and amount of water level through the creek may vary. The contractor should be prepared to tackle this situation. The contractor should be noted of the high water table and surface water fluctuation for dewatering operation.

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-DEC-15
Report Date: 09-DEC-15 14:53 (MT)
Version: FINAL

Client Phone: 807-345-3620

Certificate of Analysis

Lab Work Order #: L1708981
Project P.O. #: NOT SUBMITTED
Job Reference:
C of C Numbers:
Legal Site Desc:

Rikki Thomson
Account Manager

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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

Sample Details/Parameters		Result	Qualifier*	D.L.	Units	Extracted	Analyzed	Batch
L1708981-1	GRAYSTONE							
Sampled By:	Client on 01-DEC-15 @ 00:01							
Matrix:	Soil							
Physical Tests								
Conductivity		148		4.0	umhos/cm	07-DEC-15	07-DEC-15	R3325981
% Moisture		13.2		0.10	%	04-DEC-15	05-DEC-15	R3325347
pH		6.04		0.10	pH units		07-DEC-15	R3326423
Resistivity		6760		100	ohm cm	07-DEC-15	07-DEC-15	R3325976
Leachable Anions & Nutrients								
Chloride		<20		20	mg/kg	04-DEC-15	08-DEC-15	R3327616
Anions and Nutrients								
Sulphate		139		20	mg/kg	04-DEC-15	08-DEC-15	R3327616

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

Reference Information

Test Method References:

ALS Test Code	Matrix	Test Description	Method Reference**
CL-WT	Soil	Chloride in Soil	EPA 300.0
EC-WT	Soil	Conductivity (EC)	EPA 9050A
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.			
MOISTURE-WT	Soil	% Moisture	Gravimetric: Oven Dried
PH-WT	Soil	pH	MOEE E3137A
Soil samples are mixed in the deionized water and the supernatant is analyzed directly by the pH meter.			
RESISTIVITY-WT	Soil	Resistivity	MOECC E3138
Resistivity on a soil is a 2:1 extraction of DI water to soil. Sample is tumbled for 30 min. Conductivity of the extraction is taken and the inverse is calculated for 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 wwwt - 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: L1708981

Report Date: 09-DEC-15

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-WT	Soil							
Batch	R3327616							
WG2226771-3 CRM		AN-CRM-WT						
Chloride			102.9		%		70-130	08-DEC-15
WG2226771-2 LCS								
Chloride			95.9		%		70-130	08-DEC-15
WG2226771-1 MB								
Chloride			<20		mg/kg		20	08-DEC-15
EC-WT	Soil							
Batch	R3325981							
WG2227671-1 MB								
Conductivity			<4.0		umhos/cm		4	07-DEC-15
MOISTURE-WT	Soil							
Batch	R3325347							
WG2226652-2 LCS								
% Moisture			95.5		%		90-110	05-DEC-15
WG2226652-1 MB								
% Moisture			<0.10		%		0.1	05-DEC-15
PH-WT	Soil							
Batch	R3326423							
WG2228014-1 LCS								
pH			6.99		pH units		6.7-7.3	07-DEC-15
SO4-WT	Soil							
Batch	R3327616							
WG2226771-3 CRM		AN-CRM-WT						
Sulphate			110.5		%		60-140	08-DEC-15
WG2226771-2 LCS								
Sulphate			96.2		%		70-130	08-DEC-15
WG2226771-1 MB								
Sulphate			<20		mg/kg		20	08-DEC-15

Quality Control Report

Workorder: L1708981

Report Date: 09-DEC-15

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

Hold Time Exceedances:

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

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

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

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



DST Thunder Bay
ATTN: Bernie Villegas
DST Consulting Engineers Inc.
605 Hewitson street
Thunder Bay ON P7B 5V5

Date Received: 19-OCT-15
Report Date: 21-OCT-15 10:30 (MT)
Version: FINAL

Client Phone: 807-626-1310

Certificate of Analysis

Lab Work Order #: L1689521
Project P.O. #: NOT SUBMITTED
Job Reference: GS-TB-020407
C of C Numbers:
Legal Site Desc:

Rikki Thomson
Account Manager

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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
L1689521-1 LOW RIVER - 48W-6C Sampled By: CLIENT on 15-OCT-15 @ 14:05 Matrix: WATER Physical Tests Conductivity (EC) pH Anions and Nutrients Chloride (Cl) Sulfate (SO4)								
		46.6		3.0	uS/cm		19-OCT-15	R3292437
		7.37		0.10	pH		19-OCT-15	R3292437
		0.47		0.10	mg/L		20-OCT-15	R3293233
		0.77		0.30	mg/L		20-OCT-15	R3293233
L1689521-2 WIGGLE CREEK -48W-312C Sampled By: CLIENT on 15-OCT-15 @ 15:30 Matrix: WATER Physical Tests Conductivity (EC) pH Anions and Nutrients Chloride (Cl) Sulfate (SO4)								
		42.0		3.0	uS/cm		19-OCT-15	R3292437
		7.11		0.10	pH		19-OCT-15	R3292437
		0.91		0.10	mg/L		20-OCT-15	R3293233
		1.13		0.30	mg/L		20-OCT-15	R3293233
L1689521-3 SAVANT LAKE - 48W-313C Sampled By: CLIENT on 15-OCT-15 @ 16:10 Matrix: WATER Physical Tests Conductivity (EC) pH Anions and Nutrients Chloride (Cl) Sulfate (SO4)								
		52.3		3.0	uS/cm		19-OCT-15	R3292437
		6.98		0.10	pH		19-OCT-15	R3292437
		2.25		0.10	mg/L		20-OCT-15	R3293233
		0.51		0.30	mg/L		20-OCT-15	R3293233
L1689521-4 GRAYSTONE LAKE -48W-189C Sampled By: CLIENT on 15-OCT-15 @ 17:13 Matrix: WATER Physical Tests Conductivity (EC) pH Anions and Nutrients Chloride (Cl) Sulfate (SO4)								
		31.1		3.0	uS/cm		19-OCT-15	R3292437
		7.14		0.10	pH		19-OCT-15	R3292437
		0.13		0.10	mg/L		20-OCT-15	R3293233
		1.21		0.30	mg/L		20-OCT-15	R3293233
L1689521-5 ASINN CREEK-48W-314C Sampled By: CLIENT on 15-OCT-15 @ 10:05 Matrix: WATER Physical Tests Conductivity (EC) pH Anions and Nutrients Chloride (Cl) Sulfate (SO4)								
		65.6		3.0	uS/cm		19-OCT-15	R3292437
		7.22		0.10	pH		19-OCT-15	R3292437
		2.76		0.10	mg/L		20-OCT-15	R3293233
		1.43		0.30	mg/L		20-OCT-15	R3293233

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

QC Samples with Qualifiers & Comments:

QC Type Description	Parameter	Qualifier	Applies to Sample Number(s)
Method Blank	Chloride (Cl)	B	L1689521-1, -2, -3, -4, -5
Matrix Spike	Chloride (Cl)	MS-B	L1689521-1, -2, -3, -4, -5

Sample Parameter Qualifier key listed:

Qualifier	Description
B	Method Blank exceeds ALS DQO. All associated sample results are at least 5 times greater than blank levels and are considered reliable.
MS-B	Matrix Spike recovery could not be accurately calculated due to high analyte background in sample.

Test Method References:

ALS Test Code	Matrix	Test Description	Method Reference**
CL-L-IC-N-TB	Water	Chloride in Water by IC (Low Level)	EPA 300.1 (mod)
Inorganic anions are analyzed by Ion Chromatography with conductivity and/or UV detection.			
EC-TITR-TB	Water	Conductivity	APHA 2510 B
This analysis is carried out using procedures adapted from APHA Method 2510 "Conductivity". Conductivity is determined using a conductivity electrode.			
PH-TITR-TB	Water	pH	APHA 4500-H
This analysis is carried out using procedures adapted from APHA Method 4500-H "pH Value". The pH is determined in the laboratory using a pH electrode			
SO4-IC-N-TB	Water	Sulfate in Water by IC	EPA 300.1 (mod)
Inorganic anions are analyzed by Ion Chromatography with conductivity and/or UV detection.			

** 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
TB	ALS ENVIRONMENTAL - THUNDER BAY, 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: L1689521

Report Date: 21-OCT-15

Page 1 of 3

Client: DST Thunder Bay
DST Consulting Engineers Inc. 605 Hewitson street
Thunder Bay ON P7B 5V5

Contact: Bernie Villegas

Test	Matrix	Reference	Result	Qualifier	Units	RPD	Limit	Analyzed
CL-L-IC-N-TB		Water						
Batch	R3293233							
WG2196265-6	LCS							
Chloride (Cl)			99.7		%		90-110	20-OCT-15
WG2196265-5	MB							
Chloride (Cl)			<0.10		mg/L		0.1	20-OCT-15
EC-TITR-TB		Water						
Batch	R3292437							
WG2195669-2	LCS							
Conductivity (EC)			97.8		%		90-110	19-OCT-15
WG2195669-1	MB							
Conductivity (EC)			<3.0		uS/cm		3	19-OCT-15
PH-TITR-TB		Water						
Batch	R3292437							
WG2195669-2	LCS							
pH			6.02		pH		5.9-6.1	19-OCT-15
SO4-IC-N-TB		Water						
Batch	R3293233							
WG2196265-6	LCS							
Sulfate (SO4)			98.7		%		90-110	20-OCT-15
WG2196265-5	MB							
Sulfate (SO4)			<0.30		mg/L		0.3	20-OCT-15

Quality Control Report

Workorder: L1689521

Report Date: 21-OCT-15

Page 2 of 3

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
B	Method Blank exceeds ALS DQO. All associated sample results are at least 5 times greater than blank levels and are considered reliable.

Quality Control Report

Workorder: L1689521

Report Date: 21-OCT-15

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Hold Time Exceedances:

ALS Product Description	Sample ID	Sampling Date	Date Processed	Rec. HT	Actual HT	Units	Qualifier
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Legend & Qualifier Definitions:

EHTR-FM: Exceeded ALS recommended hold time prior to sample receipt. Field Measurement recommended.
EHTR: Exceeded ALS recommended hold time prior to sample receipt.
EHTL: Exceeded ALS recommended hold time prior to analysis. Sample was received less than 24 hours prior to expiry.
EHT: Exceeded ALS recommended hold time prior to analysis.
Rec. HT: ALS recommended hold time (see units).

Notes*:

Where actual sampling date is not provided to ALS, the date (& time) of receipt is used for calculation purposes.
Where actual sampling time is not provided to ALS, the earlier of 12 noon on the sampling date or the time (& date) of receipt is used for calculation purposes. Samples for L1689521 were received on 19-OCT-15 09:25.

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

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

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