



FINAL REPORT

SUPPLEMENTARY FOUNDATION INVESTIGATION AND DESIGN REPORT

**Proposed Replacement of Culvert on Highway 118,
Township of Stanhope, Ontario**

**Agreement No. 5015-E-0007
Assignment No. 5
Geocres No. 31E-382**

Prepared for:

Ontario Ministry of Transportation
Provincial Highways Management
Northeastern Region
Geotechnical Section
447 McKeown Avenue
North Bay, ON P1B 9S9
Attn: J. P. Perron

Ontario Ministry of Transportation
Pavements and Foundations Section
Foundations Group
Building 'C', Room 223, 2/F,
145 Sir William Hearst Avenue,
Downsview, ON M3M 0B6
Attn: M.Kwak

exp Services Inc.
August 31, 2017

Ontario Ministry of Transportation

Northeastern Region Geotechnical Section

Foundation Investigation and Design Report

Agreement No. 5013-E-0007

Assignment No. 5

Geocres No. 31E-382

Type of Document:

Final

Project Name:

Supplementary Foundation Investigation and Design Report for Proposed Replacement of Culvert on Highway 118, Township of Stanhope

Project Number:

ADM-00233185-F0

Prepared By:

Nimesh Tamrakar, M.Eng., EIT.

Silvana Micic, Ph.D., P.Eng.

Reviewed By:

Tae Chul Kim, M.E.Sc., P.Eng.

Stan E. Gonsalves, M.Eng., P. Eng.

exp Services Inc.

56 Queen St, East, Suite 301

Brampton, ON L6V 4M8

Canada



Silvana Micic, Ph.D., P.Eng.
Senior Geotechnical Engineer



Stan E. Gonsalves, M.Eng., P.Eng.
Principal Engineer
Designated MTO Foundation Contact

Date Submitted:

August 31, 2017



Table of Contents

PART I FOUNDATION INVESTIGATION REPORT	1
1.1 Introduction	1
1.2 Site Description and Geological Setting	1
1.2.1 Site Description	1
1.2.2 Geological Setting	2
1.3 Investigation Procedures	2
1.3.1 Site Investigation and Field Testing	2
1.3.2 Previous Investigation	4
1.3.3 Laboratory Testing	5
1.4 Subsurface Conditions	5
1.4.1 Boreholes	6
1.4.1.1 Asphalt	6
1.4.1.2 Fill: Sandy Gravel to Gravelly Sand/Silty Sand	6
1.4.1.3 Fill: Various Sized Fragments of Blasted Rock in Soil Matrix	6
1.4.1.4 Rock Fill: Cobble and Boulder-Sized Blasted Rock	8
1.4.1.5 Gravelly Sand to Sand and Gravel	9
1.4.1.6 Peat	9
1.4.1.7 Sandy Silt	10
1.4.1.8 Cobbles and Boulders	10
1.4.1.9 Bedrock	11
1.4.2 Test Pits	11
1.4.2.1 Fill: Sandy Gravel to Gravelly Sand/Silty Sand	12
1.4.2.2 Fill: Various Sized Fragments of Blasted Rock in Soil Matrix	12
1.4.2.3 Rock Fill: Cobble and Boulder-Sized Blasted Rock	12
1.5 Ground Water Conditions	12
PART II DISCUSSIONS AND ENGINEERING RECOMMENDATIONS	13
2.1 General	13
2.2 Expected Ground Conditions	14
2.3 Culvert Installation Options	14
2.4 Culvert Installation by Trenchless (Tunneling) Method	19
2.4.1 Culvert Installation at a New Alignment	20
2.4.1.1 Pipe Ramming with Hammer Head to Break Rock	20
2.4.1.2 Jack and Boring Using Disc Cutting Small Boring Unit (SBU-M)	21

2.4.1.3	Micro-tunneling Method	21
2.4.1.4	Open Face Jacking Shield with Backhoe Digging Arm	22
2.4.2	Culvert Installation at an Existing Alignment	22
2.4.2.1	Pipe Eating (Swallowing)	22
2.4.2.2	Pipe Reaming	23
2.4.3	Considerations of Tunneling	23
2.4.3.1	Groundwater and Surface Water Control	23
2.4.3.2	Ground Grouting/Freezing	24
2.4.3.3	Ground Settlement	24
2.4.3.4	Excavation Pits	25
2.4.3.5	Backfilling in Pits	25
2.4.4	Monitoring and Contingency Plan	25
2.4.5	Protection Systems	27
2.5	Inlet and Outlet	27
2.5.1	Erosion Protection at Outlet	27
2.5.2	Stream Bed Rip-Rap	28
2.6	Slope Stability and Settlement Analyses	28
2.6.1	Stability	28
2.6.2	Settlement	28
PART III CLOSURE		29
PART IV LIMITATION AND USE OF REPORTS		30

Appendices

APPENDIX A: PHOTOGRAPHS

APPENDIX B: DRAWING

APPENDIX C: BOREHOLE LOGS

APPENDIX D: LABORATORY TEST RESULTS

APPENDIX E: BEDROCK CORE PHOTOGRAPHS

APPENDIX F: BOREHOLE LOGS AND LAB TEST RESULTS FROM THE 2016 INVESTIGATION

APPENDIX G: OPSDs

APPENDIX H: NON-STANDARD SPECIAL PROVISIONS

1 FOUNDATION INVESTIGATION REPORT

1.1 Introduction

This foundation investigation report presents the results of a geotechnical investigation completed by **exp** Services Inc. at a site proposed by the Ministry of Transportation of Ontario (MTO) for culvert replacement at Hwy 118 (i.e. Sta. 16+470), in the Township of Stanhope, Ontario, the Ministry of Transportation (MTO) Northeastern Region. The work was undertaken under Agreement # 5015-E-0007, Assignment No. 5. The terms of reference (TOR) were as presented in the MTO letter dated October 18, 2016.

The purpose of the investigation is to supplement a previous Foundation Investigation and Design Report at the same location performed by **exp** in 2016 (Geocres No. 31E-364, dated September 14 2016) with intent to improve understanding of the fill characteristics and to facilitate the selection of method for replacing the existing culvert. The site specific geotechnical investigation consisted of borings of vertical boreholes, excavation of test pits, soil sampling, borehole/test pit logging, and field and laboratory testing.

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

1.2 Site Description and Geological Setting

1.2.1 Site Description

The site for the proposed culvert replacement in by trenchless method as selected by a MTO representative is located at Hwy 118, approximately 1.3 km west of the junction of Hwy 118 and Hwy 35 in the Township of Stanhope. In the area of investigation, Hwy 118 is two-lane roadway with approximately 1.7 m wide partially paved shoulder. It is generally oriented in an east-west direction, but at site location it is oriented in a north-west direction with blind curve and sloping approximately 8% down towards westbound (north-west direction).

As noted in the TOR the existing structural plate corrugated steel pipe (SPCSP) culvert is situated beneath Hwy 118 at the site. The pipe is 50 m long having 1.2 m diameter. The existing culvert is intended to be replaced with a new culvert on a new alignment approximately 6 m offset to the north of the existing culvert. The invert of the new culvert will be approximately 0.7 m higher than existing and the length of the proposed culvert will be approximately 46 m. It is estimated that the highway embankment at the proposed culvert location varies from approximately 10.2 m high on inlet side to approximately 12.3 m high on outlet side having side slopes of approximately 1.5H:1V to 1.25H:1V on the inlet and outlet side, respectively. The site plan and cross-section profiles for the proposed culvert alignment are as shown on Drawings 1 and 2 in Appendix B. Photographs of the site/ existing culvert are presented in Appendix A.

During this site investigation carried out on December 2016 and May 2017, the general site conditions were assessed. At the site location water flows from east to west, towards Boshkung Lake, crossing Hwy

118 via a culvert. Note that at the time of site investigation on December, the site was covered with snow which limited our observations. However, during the site investigation on May, surficial flow of water through culvert was observed.

The vicinity of inlet and outlet of the culvert is heavily vegetated with trees. The slopes of the embankment were covered by rock fill. Bedrock outcrops were observed in the vicinity of site and the stream bed. The terrain generally slopes towards the lake (on the west side of the highway). At the site location, the bedrock slopes towards the inlet forming a valley, and further, also, slopes towards the lake (on west) at the outlet side. Selected photographs of the site are provided in Appendix A.

1.2.2 Geological Setting

In accordance with the Ministry of Northern Development and Mines Map 2556, Quaternary Geology of Ontario, Southern Sheet, the site is generally undifferentiated igneous and metamorphic rock, exposed at surface or covered by a discontinuous, thin layer of drift.

In accordance with the Ministry of Northern Development and Mines Map 2544, Bedrock Geology of Ontario, Southern Sheet, the bedrock at the site consists of tectonites, straight gneisses porphyroclastic gneisses, unsubdivided gneisses in major deformation zones, mylonites and protomylonites.

1.3 Investigation Procedures

1.3.1 Site Investigation and Field Testing

As indicated above, an initial foundation investigation at this site was performed by exp in March 2016 and the general sub surface investigation information was presented in the foundation investigation report (FIR) and foundation investigation and design report (FIDR) issued on September 2016 (Geocres No. 31E-364). The March 2016 investigation included drilling of four (4) boreholes numbered BH1 to BH4 to a maximum depth ranging from 1.8 m to 18.4 m (Elev. 332.2 m to Elev. 339.2 m). After this investigation, an additional field investigation was requested by MTO in order to obtain better understanding of the characteristics of the rock fill encountered at the site. Accordingly, additional filed investigation was performed on December 2 to 7, 2016 and May 3 to 11, 2017. The field program consisted of drilling two (2) sampled boreholes (BH101 and BH201) and excavation of four shallow test pits (TP1 to TP4). BH201 and test pits TP1 and TP2 were advanced in December 2016. However, due to bad weather and winter conditions, the completion of the field work was postponed for the spring time. The remaining filed work of drilling BH101 and excavation of test pits TP3 and TP4 was performed in May 2017.

The boreholes were strategically located along the proposed culvert alignment to provide subsurface information for the design of the proposed new culvert. BH101 and BH201 were advanced from the embankment crest within the travelled road. BH101 and BH201 were located along the proposed culvert centerline adjacent to the pavement edge line of EBL and WBL of the Hwy, respectively, and opposite to previously drilled boreholes BH1 and BH2. BH101 and BH201 were advanced to depths of 15.1 m and 16.3 m, respectively.

Four shallow test pits were excavated between the existing and proposed culverts on either side of embankment to provide better information about the rock fill used for the construction of the embankment. Drawing 1 in Appendix B shows the locations of these test pits. Two test pits (i.e. TP1 and TP2) were excavated at the top of the embankment slope, about 1.6 m and 1.3 m south and north from the edge of embankment crest, while the other two (i.e. TP3 and TP4) were excavated at the bottom of the embankment slope, just above the embankment toes. TP1 and TP2 were located about 2.5 m west of the centerline of existing culvert alignment on the south and north side slope, respectively. Whereas, TP3 and TP4 were located about 4 m and 4.8 m west of the centerline of existing culvert alignment on the south and north side slope, respectively. The size of the test pits TP1 and TP2 were about 6.5 m long x 1.2 m wide and about 0.4 m deep. TP3 was about 7.5 m long x 1.2 m wide and about 1 m deep, while TP4 was about 4.8 m long x 1.4 m wide, and also about 1 m deep.

Boreholes drilled from the embankment crest (BH101 and BH201) were advanced using a truck mounted CME-85 drill rig and CME-75 drill rig respectively. The drill rig was equipped with a hollow stem auger, tri-cone and standard soil/ rock sampling equipment operated by a specialist drilling contractor, Landcore Drilling and Marathon Drilling Company Ltd. A wash boring technique with casing in conjunction with core barrel was used to advance the boreholes through the embankment. When the cobbles and boulders were encountered, the core barrel was used to advance the borehole and obtain core samples. Between the obstructions, a combination of conventional SPT sampling and/or tri-coning was attempted. Considering the size of a SPT sampler (i.e. 35 mm inside diameter), only particles smaller than 35 mm in diameter was able to be collected in the sampler. The larger particles could be possibly pushed aside during the driving of the sampler. The test pits excavated from the embankment crest (TP1 and TP2) were excavated using a Kobelco-210 track mounted excavator and the test pits excavated from the toe of embankment (TP3 and TP4) were excavated using a CAT-311 track mounted excavator.

The borehole/test pit locations (referenced to the MTM NAD83 coordinate system) and their ground surface elevations were temporary surveyed by **exp** personnel using the Temporary Benchmark (TBM) provided (BM 828023 mark on rock, see photographs, in Appendix A) north of the site and west of the highway. The TBM elevation is assumed as Elev. 343.962 m. The location of the boreholes and test pits are shown on Drawing 1, in Appendix B.

During the drilling of the boreholes, a combination of Standard Penetration Tests (SPT) and coring was attempted to obtain the soil and rock samples. Soil samples were obtained using a 51 mm outside diameter (O.D.) split-spoon sampler in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586) at intervals ranging from 0.75 m to 1.5 m in depth as shown on the attached borehole logs (Appendix C). The original field (uncorrected) SPT "N" values were recorded on the borehole logs as recommended in the Canadian Foundation Engineering Manual (CFEM, pg. 40) and used to provide an assessment of in-situ consistency or relative density of non-cohesive soils. The SPT "N" values taken within the particles larger than diameter of split spoon sampler may not be reliable and collected samples are possibly not representative of the layer. When a hard stratum was reached (refusal of split spoon), sampling of hard material was performed by diamond core drilling using a 1.5 m long NQ double tube wireline core barrel.

Upon completion of the boreholes, ground water level measurements were carried out in the boreholes in accordance with the MTO guidelines. However, boreholes were advanced using a wash boring technique, so the stabilized ground water level could not be established by short term observations in boreholes. The drilled boreholes were decommissioned by bentonite/cement mixtures in accordance with the Ministry of the Environment Regulation 903, as amended by Regulation 128/03 (the well regulation under the Ontario Water Resources Act). The test pits were backfilled and compacted properly. It has been reinstated to the original conditions.

The fieldwork was supervised by members of **exp's** engineering who directed the drilling and sampling operations, logged borehole data in accordance with MTO and/or ASTM Standards for Soils Classification, and retrieved soil samples for subsequent laboratory testing and identification.

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

1.3.2 Previous Investigation

The following previous investigation report from the 2016 investigation is available on the MTO GEOCREs Library:

- Foundation Investigation and Design Report, Culvert Installation Hwy 118, Township of Stanhope; G.W.P. 5140-13-00; Agreement # 5015-E-0008; Geocres No. 31E-364; exp Services Inc.; September, 2016.

Four borehole logs produced based on the investigation conducted by exp in March 2016 at location of this culvert (identified as BH1 to BH4) are attached in Appendix F of this report. The details of the borehole locations and elevations completed at the site location at the 2016 investigation are outlined in Table 1.1. The location details of each borehole should be considered an estimate only.

Table 1.1. Summary of boreholes completed at the 2016 investigation

BH No.	Borehole Locations (Station and Offset from the centreline) ¹	Ground Elevation (m)	Borehole Depth (m)	Borehole Bottom Elevation (m)	Piezometer/ Monitoring Well
BH1	2 m west of the proposed culvert centreline adjacent to the pavement edge line of EBL	350.6	18.4	332.2	None
BH2	2 m east of the proposed culvert centreline adjacent to the pavement edge line of WBL	351.2	15.7	335.5	None
BH3	Toe of the embankment at inlet of proposed culvert	341.0	1.8	339.2	None

BH No.	Borehole Locations (Station and Offset from the centreline) ¹	Ground Elevation (m)	Borehole Depth (m)	Borehole Bottom Elevation (m)	Piezometer/ Monitoring Well
BH4	Toe of the embankment at outlet of proposed culvert	338.3	2.2	336.1	None

Note: ¹ Station and offset measurements are approximate.

1.3.3 Laboratory Testing

All soil and rock samples returned to the laboratory were subjected to visual examination and classification. The laboratory testing program included the determination of natural moisture content and particle size distribution from the selected soil samples. Since most of split spoon samples collected from boreholes within the rock fill layer were not able of adequate size for testing, particle size distribution tests were not possible to perform for many collected samples at this site. All the laboratory tests were carried out in accordance with MTO and/or ASTM Standards as appropriate. Since wash boring technique were used to advance boreholes and generally cohesionless material was encountered, it should be noted that the moisture content values obtained from laboratory tests may not be accurate representation of the soil moisture condition.

The laboratory test results are provided on the attached borehole log sheets in Appendix C. The results of the grain size analyses are presented graphically in Appendix D. The laboratory test results from are provided on the attached borehole log sheets in Appendix F.

1.4 Subsurface Conditions

The detailed subsurface conditions encountered in the boreholes advanced during this investigation are presented on the borehole log sheets in Appendix C. Laboratory test results are provided in Appendix D. The "Explanation of Terms Used in Report" preceding the borehole logs in Appendix C forms an integral part of and should be read in conjunction with this report.

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

The general stratigraphy encountered within the investigated depths of previous boreholes (exp 2016) and current investigation are inline. In general, the subsoil condition at the site consist of a layer of fill material composed of gravelly sand to various sized of fragments of blasted rock in a silty, sandy and gravelly soil matrix underlain by native peat, followed by a layer of gravelly sand and bedrock.

A detailed description of the subsurface conditions encountered in boreholes and test pits is discussed further in subsequent sections. It should be noted that the following sections are based on the geotechnical investigation conducted by **exp** for this assignment and previous assignment (exp 2016; Geocres 31E-364).

1.4.1 Boreholes

1.4.1.1 Asphalt

Asphalt was encountered at the surface of boreholes BH1, BH2, BH101 and BH201. Thickness of the asphalt layer was between 0.12 m and 0.15 m. Asphalt thicknesses may further vary beyond the borehole locations.

1.4.1.2 Fill: Sandy Gravel to Gravelly Sand/Silty Sand

Sandy gravel to gravelly sand/silty sand fill was encountered below the asphalt in all boreholes drilled at the top of the embankment (i.e. BH1, BH2, BH101 and BH201). The sandy gravel to gravelly sand fill extended to depths ranging between 1.5 m to 2.7 m below road surface with elevations ranging between Elev. 348.4 m and 349.7 m. The explored thickness of this layer was between 1.3 m to 2.6 m.

The composition of this fill layer is sand and gravel, with some cobbles and clay inclusions. The material is brown to grey in color, and moist. The SPT "N" values within this layer ranged from 13 to 77 blows per 300 mm penetration, suggesting compact to very dense, but generally compact, compactness condition.

Laboratory testing performed on selected samples consisted of moisture content and grain size distribution tests. The test results are as follows:

Moisture Content: (**exp 2016 and 2017**)

- 2.6% to 13.0%

Grain Size Distribution: (**exp 2016 and 2017**)

- 2% to 24 % gravel;
- 66% to 77% sand; and
- 8% to 25% silt and clay

The results of the moisture content and grain size distribution tests for this assignment are provided on the record of borehole sheets in Appendix C. The results of the grain size distribution tests for this assignment are also provided on Figure 1 in Appendix D. The borehole logs and graphical results from the 2016 investigation are provided in Appendix F.

1.4.1.3 Fill: Various Sized Fragments of Blasted Rock in Soil Matrix

A layer of fill consisting of various sized fragments of blasted rock within soil matrix was encountered below the sandy gravel to gravelly sand fill in all boreholes drilled at the top of the embankment (i.e. BH1, BH2, BH101 and BH201). This fill layer extended to depths ranging between 8.2 m to 12.2 m below road surface

with elevations ranging between Elev. 342.9 m and 339.0 m. The explored thickness of this layer was between 5.5 m to 10.7 m.

The composition of this fill layer is a silty, sandy and gravelly soil matrix with blasted rock fragments having various particle sizes from sand to boulder size. As explained in Section 1.3.1 the combination of SPT and coring was attempted to obtain samples from this fill layer. Where it was possible a standard split spoon sampler attempted to collect samples at this layer. However, in the majority cases adequate samples could not be recovered. The SPT “N” values measured during these tests ranged from 3 blows per 300 mm penetration to 60 blows per 50 mm penetration. However, only for information purpose, the obtained SPT “N” values within the soil matrix are recorded in the borehole logs suggesting very loose to very dense compactness condition. For the other part of this layer where the split spoon/auger refusal was encountered, coring was performed recovering cored rock samples together with infill soils as recorded on the borehole logs. Some voids were also identified during the coring and are recorded on the borehole logs.

Soils and rock fragments collected in the split spoon sampler and during coring were subjected to laboratory testing. In particular, laboratory testing performed on selected soil samples recovered in the split spoon sampler consisted of moisture content and grain size distribution tests. The test results are as follows:

Moisture Content: (exp 2016 and 2017)

- 3.6% to 11.9%

Grain Size Distribution: (exp 2016)

- 49% gravel;
- 43% sand; and
- 8% silt and clay

The results of the moisture content tests are provided on the record of borehole sheets in Appendix C. The borehole logs and graphical results from the 2016 investigation are provided in Appendix F.

At the time of the 2016 investigation, uniaxial compressive strength and abrasivity tests were also performed on selected boulder-sized rock fragment core samples recovered from BH1. The results of these tests are summarized in Table 1.2 below.

Table 1.2 Uniaxial compressive strength and CERCHAR abrasivity index of rock cores in the rock fill of various-sized blasted rock particles

Borehole	Ground Surface Elevation (m)	Rock Core Sample Depth (m)	Uniaxial Compressive Strength (MPa)	CERCHAR Abrasivity Index (CAI)
BH1	350.6	5.33 – 6.1	-	4.32
		7.01 – 7.62	94.1	-

The test results indicate that rock fragments are strong to very strong as per Table 3.5 of CFEM (2006) and highly abrasive based on modified classification of rock abrasiveness (Restner, 2007).

The results of uniaxial compression tests and CERCHAR abrasivity tests are also provided in Appendix F.

1.4.1.4 Rock Fill: Cobble and Boulder-Sized Blasted Rock

A layer of rock fill consisting of cobble and boulder-sized blasted rock fragments was encountered below the layer of fill of various sized fragments of blasted rock in the soil matrix found in BH1, BH101 and BH201. This rock fill layer extended to depths ranging between 12.7 m to 13.7 m below the road surface with elevations ranging between Elev. 336.9 m and 338.4 m. The explored thickness of this layer was between 1.9 m (BH101) and 5.5 m (BH201).

The composition of this rock fill layer is mainly of cobbles and boulders size grains. The coring was attempted to obtain their samples. As noted on the borehole logs, the size of cored rock samples was upto 1.0 m. Between the rock pieces some silty and sandy soil infills and/or voids were observed during the coring.

At the time of the 2016 investigation, uniaxial compressive strength and abrasivity tests were also performed on selected rock core samples from this rock fill recovered in BH1. The results of these tests are summarized in Table 1.3 below.

Table 1.3 Uniaxial compressive strength and CERCHAR abrasivity index of boulder cores in the rock fill of coble and boulder-sized blasted rock particles

Borehole	Ground Surface Elevation (m)	Rock Core Sample Depth (m)	Uniaxial Compressive Strength (MPa)	CERCHAR Abrasivity Index (CAI)
BH1	350.6	9.76 – 10.67	-	4.56
		11.28	100.8	-
		11.58 – 12.19	55.7	4.49

The test results indicate that boulder-sized rock pieces are strong to very strong as per Table 3.5 of CFEM (2006) and highly abrasive based on modified classification of rock abrasiveness (Restner, 2007).

The results of uniaxial compression tests and CERCHAR abrasivity tests are provided in Appendix F.

1.4.1.5 Gravelly Sand to Sand and Gravel

Native gravelly sand to sand and gravel layer was encountered below the rock fill layer in borehole BH1 and BH2 and below peat layer in BH101. The gravelly sand to sand and gravel extended to depths ranging between 12.7 m to 14.6 m below road surface with elevations ranging between Elev. 336.0 m to 338.5 m. The explored thickness of this layer was between 0.5 m to 0.9 m.

The composition of this layer is mostly sand and gravel, few silt and clay size particles, some peat and occasional cobbles and boulders. The material is grey to reddish brown, moist to wet. The SPT "N" values within this layer ranged from 30 blows per 300 mm penetration to 50 blows per 100 mm penetration suggesting compact to very dense compactness condition.

Laboratory testing performed on selected samples consisted of moisture content test and grain size distribution test and the test results are as follows:

Moisture content: **(exp 2016 and 2017)**

- 18.2% to 25.9%

Grain Size Distribution: **(exp 2016)**

- 27% to 48 % gravel;
- 45% to 65% sand;
- 7% to 8% silt and clay

The result of moisture content tests are provided on the record of borehole sheets in Appendix C. The borehole logs and graphical results obtained at the 2016 investigation are provided in Appendix F.

1.4.1.6 Peat

A peat layer was encountered the beneath sand and gravel layer in BH2, beneath the rock fill in BH101 and BH201, and at the ground surface in BH3 and BH4. The peat was described as very soft to stiff, brown to black, wet and containing trace sand, trace gravel and trace roots and rootlets. The peat layer extended to depths ranging between 13.3 m to 13.7 m below the road surface with elevation ranging between Elev. 337.3 m and 337.9 m in boreholes advanced from the roadway. In off-road boreholes advanced at inlet and outlet locations, the peat layer was at the ground surface and extended to depths ranging between 0.2 m and 0.6 m below the ground surface with elevations ranging between 338.2 m and 340.4 m. The explored thickness of this layer was between 0.2 m to 0.8 m.

Laboratory testing performed on selected peat samples consisted of moisture content tests. The test results are as follows:

Moisture content: **(exp 2016 and 2017)**

- 40.3% to 86.9%

The results of the moisture content tests are provided on the record of borehole sheets in Appendix C. The borehole logs from the 2016 investigation are provided in Appendix F.

1.4.1.7 Sandy Silt

A native sandy silt layer was encountered below the peat layer in BH2. The sandy silt layer extended to depth of about 14.0 m below the road surface with elevation about 337.2 m. The explored thickness of this layer was about 0.7 m.

The composition of the layer is mostly sand and silt, trace clay, trace gravel and trace organics. The material is grey in color, and wet. One SPT “N” value obtained within this layer was 50 blows per 100 mm penetration suggesting very dense compactness condition.

Laboratory testing performed on selected samples consisted of moisture content and grain size distribution tests. The test results are as follows:

Moisture Content: (**exp 2016**)

- 32.6%

Grain Size Distribution: (**exp 2016**)

- 6 % gravel;
- 37% sand;
- 53% silt and;
- 4% clay

The borehole logs and graphical results from the 2016 investigation are provided in Appendix F.

1.4.1.8 Cobbles and Boulders

Cobbles and boulders were encountered underlying a peat layer in BH3. The cobbles and boulders layer extended to depths about 1.5 m below ground surface with elevation about 339.5 m. The explored thickness of this layer was about 0.9 m.

The composition of this layer is mostly cobbles and boulders, trace some sand, trace silt and trace organics. One SPT “N” value obtained within this layer was 50 blows per 127 mm penetration suggesting very dense compactness condition. The recovered cored sample obtained within this layer is about 150 mm.

Laboratory testing performed on one collected sample consisted of moisture content test and the test result is as follows:

Moisture Content: (**exp 2016**)

- 16.7%

1.4.1.9 Bedrock

Bedrock was encountered underlying the peat/ sandy peat layer in BH1, BH201 and BH4, cobbles and boulders in BH3, gravelly sand in BH101 and beneath the sandy silt layer in BH2. The bedrock was encountered at depths ranging between about 0.2 m and 1.5 m below ground surface at the inlet and outlet, and about 13.5 m to 14.6 m below the existing road surface. The bedrock was confirmed by coring of 0.5 m to 3.9 m long rock cores. The elevation of the bedrock surface below Hwy 118 ranges from Elev. 336.0 m to Elev. 339.5 m. The bedrock surface depth and elevation encountered at the drilled borehole locations are listed in Table 1.4. Photographs of rock cores are included in Appendix E. All the boreholes are terminated within bedrock.

Table 1.4 Depth and elevation of bedrock surface

Borehole	Depth Below Ground Surface (m)	Elevation (m)	Comments
BH1	14.6	336.0	Bedrock Cored
BH2	14.0	337.2	Bedrock Cored
BH3	1.5	339.5	Bedrock Cored
BH4	0.2	338.2	Bedrock Cored
BH101	14.6	336.4	Bedrock Cored
BH201	13.5	337.6	Bedrock Cored

Based on the rock cores recovered, the bedrock consists of granitic gneiss. In general, the rock samples are described as grey, with pink and white striations have a fine crystalline structure, slightly weathered. The Rock Quality Designation (RQD) measured on the rock core samples typically ranged from approximately 21% to 100%, indicating a rock mass of very poor to excellent, but generally fair to excellent quality.

1.4.2 Test Pits

As noted before, four shallow test pits were excavated between the existing and proposed culverts on either side of embankment as shown on Drawing 1 in Appendix B. Test pits TP1 and TP2 were excavated at the top of the embankment slope, while test pits TP3 and TP4 were excavated above the toe of the embankment. TP1 and TP3 were excavated on the west (outlet) side, while TP2 and TP4 were excavated on the east (inlet) side. The size of the test pits TP1 and TP2 were about 6.5 m long x 1.2 m wide and about 0.4 m deep. TP3 was about 7.5 m long x 1.2 m wide and about 1 m deep, while TP4 was about 4.8 m long x 1.4 m wide, and also about 1 m deep.

Photographs taken during the excavation of the test pits (i.e. Photo15 to Photo 26) are included in Appendix A. The description of geotechnical findings during the excavation of the test pits is given in the attached records of test pit in Appendix C.

1.4.2.1 Fill: Sandy Gravel to Gravelly Sand/Silty Sand

Sandy gravel to gravelly sand/silty sand fill was encountered on the surface of test pits TP1 and TP2 as shown in Photos 15, 16 and 17, Appendix A. Thickness of sandy gravel to gravelly sand fill in test pits was about 0.2 m.

1.4.2.2 Fill: Various Sized Fragments of Blasted Rock in Soil Matrix

Fill consisting of blasted rock fragments in silty, sandy and gravelly soil matrix was encountered below sandy and gravelly fill in the test pits excavated along the embankment slope from the top of the embankment (TP 1 and TP2) as show in Photos 16 and 18, Appendix A. The size of rock fragments varied from gravel to bolder size. The rock fragments were round to angular.

1.4.2.3 Rock Fill: Cobble and Boulder-Sized Blasted Rock

Rock fill was encountered in the test pits excavated above toes of each side of the embankment slope (TP2 and TP4). These excavated test pits are shown through Photos 19 to 26 in Appendix A. The rock fill pieces had mostly cobble and boulder size and angular to round shape. Some rock fill pieces were 1.65 m in size as shown on Photo 24. An infill between rock grains was mostly silty, sandy, gravelly and organic soil, but voids were also observed. The shallow bedrock was encountered at the inlet side (Photo 26).

1.5 Ground Water Conditions

Since the wash boring method was used for drilling boreholes, accurate groundwater levels at these holes could not be measured in the open holes at the time of drilling operations.

At the time of previous investigation (exp 2016) surficial flow of water through the culvert was observed to be at approximate Elev. 341.1 m and 338.0 m at the inlet and outlet sides, respectively.

Seasonal variations in the water table should be expected, with higher levels occurring during wetter periods of the year and lower levels during drier periods.

2 DISCUSSIONS AND ENGINEERING RECOMMENATIONS

2.1 General

This section of the report provides geotechnical design recommendations for the proposed culvert installation by trenchless method at Hwy 118 (i.e. Sta. 16+470), in the Township of Stanhope, Ontario, the Ministry of Transportation (MTO) Northeastern Region. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the current investigation and previous investigation performed by **exp** in March 2016 (Geocres No. 31E-364) at the site and presented in **Part I- Foundation Investigation Report**. The interpretation and recommendations provided are intended solely to permit MTO to assess appropriate installation method and feasibility of installation of proposed culvert using a trenchless method. Comments on construction are only provided to highlight issues that could affect the design. Contractors bidding on the works should make their own assessments of the factual data and how it might affect construction means and methods, scheduling and the like.

We understand that the existing culvert below Hwy 118, which is 1200 mm diameter by 50 m long structural plate corrugated steel pipe (SPCSP), is intended to be replaced with a new culvert either on a new alignment approximately 6 m offset to the north of existing culvert (Option 1) or at the location of existing culvert using pipe swallowing techniques or like (Option 2) as shown on Drawing 1, Appendix B. In case of Option1, the invert of a new culvert will be approximately 0.7 m higher than existing and the length of the proposed culvert will be approximately 46 m. The location plan of the new proposed culvert was provided by MTO. However, the size and type of the new culvert is not defined at the time of writing this report.

This report addresses the geotechnical design of the foundation for the proposed culvert by providing geotechnical design parameters at the Ultimate Limit State (ULS) and Serviceability Limit States (SLS) as well as other geotechnical parameters that may be required in accordance with the latest edition of the *Canadian Highway Bridge Design Code (CHBDC)* (CAN/CSA-S6-14), the *Guideline for Professional Engineers Providing Geotechnical Engineering Service* (1992), the *Canadian Foundation Engineering Manual (CFEM)* (2006), MTO Gravity Pipe Design Guidelines (May 2007) and good practice. As requested in the TOR, this section also provides discussions on appropriate methods for culvert installation by trenchless method, including table with evaluated alternatives and cost estimates; ease of excavating of embankment, and any need for dewatering and roadway protection/shoring. Pertinent construction issues from a geotechnical standpoint are examined in general accordance with the Terms of Reference from MTO Letter dated October 18, 2016.

It should be noted herein that several tunneling engineers and contractors were contacted regarding the trenchless installation of a culvert in rock fill embankment, and all agree that it is a technically challenging crossing and requires a field trial of the selected method before it is adopted. Tunneling in these conditions is a risky undertaking with many potential issues during advance. It is recognised that the alternative

replacement strategy (i.e. open cut) is also unattractive in high embankment through rockfill, noting disruption and cost.

2.2 Expected Ground Conditions

According to the results of current foundation investigation, the following ground conditions along the proposed/existing culvert alignment are evident:

- a. The highway embankment consists of fill material composed of gravelly sand to rock fill of various sized fragments of blasted rock underlain by native peat, followed sandy/ cobbles and boulders and bedrock.
- b. The total thickness of the embankment fill along the proposed culvert alignment ranged from 12.2 m to 13.7 m at the investigated locations (BH1, BH2, BH101 and BH201).
- c. The proposed culvert inverts for Option 1 are assumed approximately at Elev. 341.8 m at the inlet and 338.7 m at the outlet. The possible excavation level, if a new alignment option is considered, will be through rock fill underlying thin layer of peat and followed by bedrock.
- d. Thickness of the peat layer underlying rock fill was between 0.6 m to 0.8 m.
- e. The water level observed in the culvert at the time of previous investigation was at Elevation 341.1 m at the inlet and Elevation 338.0 m at the outlet.
- f. Based on the measured elevation of the stream flowing through the existing culvert, the inferred groundwater level within the embankment was estimated to be at approximate Elev. of 341.1 m. Seasonal variations in the water table should be expected.
- g. The slopes of the embankment at inlet and outlet sides are covered by rock fill as show on the photos attached in Appendix A. No slope instability on either embankment slopes was observed.

The launching and receiving pits for the tunneling equipment are expected to be located at the outlet and inlet of the proposed culvert (or existing culvert) location, respectively. Access to launching and receiving pits could be difficult due to the high rock fill embankment (approximately 10.2 m to 12.3 m high on the inlet side and outlet side, respectively) and relatively steep side slopes (approximately 1.5H:1V to 1.25H:1V on inlet and outlet side, respectively). It should be noted that, if the replacement of culvert at the location of existing culvert option is selected, construction of the launching or receiving pit at the inlet side would be very difficult due to presence of bedrock outcrops (Photo 11 in Appendix A). It may need to blast large amount of bedrock to create space for the launching or receiving pit.

2.3 Culvert Installation Options

Only trenchless methods of culvert installation are discussed in this section since the open cut and cover method option has already been discussed in the previous foundation investigation and design report (exp 2016). For selection of appropriate construction methods for this culvert installation the following was considered: (i) whether a new alignment is proposed or not; (ii) soil conditions at zone of culvert installation; and (iii) diameter and length of the new culvert. Further, several items to keep in mind during the selection

were: (i) the trenchless (tunneling) approaches with a new culvert construction adjacent to the current alignment with the need to decommission the existing culvert including grouting and sealing ; (ii) installation of a replacement culvert at the location of the existing culvert using pipe swallowing/crushing trenchless (tunneling) techniques or the like; (iii) since the embankment consists of rock fill, appropriate equipment and construction method must be selected based on ability to accommodate these obstructions; and (iv) provision must be made to maintain surface water flow to the outlet.

It should be noted that the existing ground conditions (i.e. rock fill and mixed face) in the embankment will be a technically challenging crossing for using any trenchless method. Trenchless methods typically require a stable face, which is not provided by this poorly graded rock fill embankment with the presence of large voids and unstable rocks within the soil matrix. Therefore, trenchless methods work best when they are used in homogeneous materials. As indicated above, the highway embankment consists of fill material composed of gravelly sand to rock fill of various sized fragments of blasted rock. Some trenchless methods can deal with occasional boulders and rock; however, there is little experience in tunneling through mixed ground containing round to angular boulder sized blasted rock fragments (> 300 mm). Rock blasted material which is typically sharp and jagged is very difficult for any cutting tools used for boring.

Considering all above, the two installation options were considered as possible alternatives for the new culvert replacement method:

- Trenchless (tunneling) methods on a new alignment (i.e. pipe ramming with a hammer head to break rock; jack and boring using a disc cutting small boring unit (SBU-M); micro-tunneling; and open face jacking shield with backhoe digging arm (man-entry method); and
- Trenchless (tunneling) method on existing alignment (i.e. pipe eating/swallowing - modified micro-tunneling, pipe reaming- modified back reaming)

For trenchless replacement of the existing 1.2 m diameter culvert, pipe bursting methods were not considered as applicable in this project, since the nature of high rock fill embankment classify this culvert as an unsuitable candidate for these techniques. Based on OPSS 463 the existing pipe which is greater than 0.5 m - 0.9 m in diameter and deeper than 5.5 m is classified as extremely difficult to be replaced using pipe bursting techniques. The interior replacement method is another installation method without disrupting traffic, but considering the fact that the culvert capacity will be reduced, this method is also assessed as an unviable option.

Table 2.1 summarizes advantages, disadvantages and respective estimated cost of considered culvert installation methods as viable for this site.

Table 2.1 Installation methods for culvert replacement

Installation Method		Advantages	Disadvantages	Relative Cost	Ranking
<i>Trenchless Method on a new alignment</i>	Pipe ramming with a hammer head to break rock	<ul style="list-style-type: none"> • Better suited for penetrating through potential obstructions such as cobbles and boulder-sized rock pieces compared to pipe jacking methods • Fracturing of rock is possible with smaller drill hammer • Existing culvert could be used for surface water dewatering during construction • The method can also be used at existing pipe location with oversized casing and swallowing existing pipe • No traffic interruption and requirement for detour route 	<ul style="list-style-type: none"> • Boulder-sized rock pieces can stop penetration of casing requiring hand mining, which may not be feasible if the size of the pipe is not big enough • Vibration can cause settlement in the immediate vicinity of the installation causing deformation of the ground surface • Potential for soil heave due to blockage • Unable to correct for line and grade during installation • Ground pre-grouting or freezing might be required (contractor assessment will be required) • Containment and large voids could be issues for pre-grouting in rock fill • Vibration are issue • Requires decommissioning of old culvert, including its grouting and sealing 	Relatively less expensive than other tunneling method	1
<i>Trenchless Method on a new alignment</i>	Jack and boring using a disc cutting small boring unit (SBU-M)	<ul style="list-style-type: none"> • Combination of boring and cutting better suited for penetration through mixed ground condition including boulders • It can be used with any conventional auger boring machine • Can install casing diameter between 1.2 m to 2.0 m • Manned entry for laser guided steering is possible 	<ul style="list-style-type: none"> • Due to the presence of the bore, multiple large rocks could suddenly rearrange themselves within embankment leading to large surface settlement • Ground pre-grouting or freezing might be required to avoid collapse • Obstruction may be problematic if not grouted/freezeed properly and freezing strength may not be sustainable in open rock fragment 	Less expensive than micro tunneling but more expensive than pipe ramming	2

Installation Method		Advantages	Disadvantages	Relative Cost	Ranking
		<ul style="list-style-type: none"> No traffic interruption and requirement for detour route 	<ul style="list-style-type: none"> Containment and large voids could be issues for pre-grouting in rock fill Requires decommissioning of old culvert, including its grouting and sealing 		
Trenchless Method on a new alignment	Micro-tunneling	<ul style="list-style-type: none"> In general, handles wide variety of ground conditions Ability to control excavation face stability Less dewatering required Minimum surface disruption Accurate The existing culvert can be used to maintain the surface water flow during the construction No traffic interruption and requirement for detour route 	<ul style="list-style-type: none"> High construction cost Obstructions problematic Ground pre-grouting or freezing is required to enhance stability Containment and large voids could be issues for pre-grouting in rock fill No access during tunneling Excavation and shoring required to achieve starting grade, Requires large area for jacking shaft and support equipment Construction of launching/receiving pit on inlet side would be challenging due to limited space; bedrock blasting is required Dewatering possibly required at launching and receiving pits Requires decommissioning of old culvert, including its grouting and sealing Need to take precaution for tunneling, due to presence of peat layer underlying rock fill 	Probably less expensive than an open face digging shielded type TBM tunneling, if the equipment is readily available	4
	Open face jacking shield with backhoe digger arm (man-entry method)	<ul style="list-style-type: none"> Ability to access obstructions during tunneling Can handle boulders of size up to 33% of the casing diameter either by a backhoe digger arm or manually Possible experienced contractor in Ontario 	<ul style="list-style-type: none"> Requires a big pipe; not practical for small diameter pipe (min. 1.8 m diameter) Ground pre-grouting or freezing is required in rock fill to avoid collapse. Some risk and cost associated with these measures. 	Probably more expensive and risky than micro-tunneling	5

Installation Method		Advantages	Disadvantages	Relative Cost	Ranking
		<ul style="list-style-type: none"> The existing culvert can be used to maintain the surface water flow during the construction No traffic interruption and requirement for detour route 	<ul style="list-style-type: none"> Excavation and shoring require to achieve starting grade Construction of launching/receiving pit on inlet side would be challenging due to limited space; bedrock blasting is required Requires decommissioning of old culvert, including its grouting and sealing Need to take precaution for tunneling, due to presence of peat layer underlying rock fill 		
<i>Trenchless Method on existing alignment</i>	<i>Pipe eating (modified micro-tunneling)</i>	<ul style="list-style-type: none"> No decommissioning of existing culvert; its grouting and sealing is not required Cutting head can cut the existing pipe and surrounding ground to the required diameter to allow installation of a new pipe No traffic interruption and requirement for detour route 	<ul style="list-style-type: none"> Construction of launching/receiving pit on inlet side would be challenging due to limited space; bedrock blasting is required Cutter head could be clogged due to presence of mix type of soil including cobbles and boulders The surface water flow during the construction has to be conveyed to some other route 	Probably less expensive than other tunneling, if the equipment is readily available	3
<i>Trenchless Method on existing alignment</i>	<i>Pipe reaming (modified back reaming)</i>	<ul style="list-style-type: none"> No decommissioning of existing culvert; its grouting and sealing is not required Reamer with cutting teeth can grind and crush the existing pipe No traffic interruption and requirement for detour route 	<ul style="list-style-type: none"> Construction of launching/receiving pit on inlet side would be challenging due to limited space; bedrock blasting is required The upsizing of new pipe size will be limited, as it is modified HDD method. The surface water flow during the construction has to be conveyed to some other route 	Probably less expensive than other tunneling, if the equipment is readily available	6

Based on the existing ground conditions and the list of advantages and disadvantages of all methods considered for this project including those in this and previous reports, cut and cover methods is considered as more viable methods from a geotechnical and/or foundation perspective if disruption of traffic at Hwy 118 is allowed. The major advantages of this approach are possibility to assess the foundation soil below the new culvert location and to remove the existing culvert. On the other hand, the major disadvantages are disruption of traffic and large excavation of rock fill. However, if the Regional Traffic Office requires replacement of the culvert without disrupting traffic, trenchless (tunneling) installation methods could be explored as options for installation of a new culvert. The major disadvantages of these trenchless installation methods in the existing ground condition are a lack of experience in culvert installation through a rock fill embankment (mixed ground condition), higher cost of installation than the cut and cover method and the need to decommission the existing culvert by its grouting and sealing. These approaches all entail risk of collapse. Among the tunneling methods discussed in Table 2.1, pipe ramming with a hammer head to break rock at the face is assessed as the most feasible method. This method can be used at a new or at the existing alignment with a swallowing technique. The jack and boring using a disc cutting small boring unit (SBU-M) method at a new alignment and the pipe eating/swallowing (modified micro-tunneling) method at an existing alignment are also possible alternative. Open face jacking shield with backhoe digger arm (man-entry method) and pipe reaming with modified back reaming are ranked as the least viable trenchless methods.

2.4 Culvert Installation by Trenchless (Tunneling) Method

As indicated above, the tunneling approach through rock fill consisting of various size of blasted rock fragments ranging from gravel to cobble/boulder size is risky and difficult. In this connection, the approach is only being considered if the culvert replacement is required in many applications where high rock fill embankments are present such as the northern Ontario and using open cut methods is not practical. Notwithstanding the precedence comments, for trenchless installation methods the procedures should conform to all relevant Ontario Provincial Standard Specifications (OPSS), Non-Standard Special Provisions (NSSP) such as Installation by Trenchless Method and all other industrial standards.

Therefore, based on the site conditions and characteristics of methods considered above, the following options for the culvert construction at the proposed new alignment or at the same alignment are discussed in the following sections:

- a) Culvert installation by tunneling methods on a new alignment:
 - Pipe ramming with a hammer head to break rock;
 - Jack and boring using a disc cutting Small Boring Unit (SBU-M);
 - Micro-tunneling; and
 - Open face jacking shield with backhoe digging arm
- b) Culvert installation by tunneling methods on existing alignment:
 - Pipe eating/swallowing (modified micro-tunneling); and
 - Pipe reaming (modified back reaming)

2.4.1 Culvert Installation at a New Alignment

According to OPSS 421, the minimum spacing allowed between new a culvert and an existing culvert is 600 mm (if pipe diameter <1.2 m) or a 0.5 times the pipe diameter (1.2 m < pipe diameter <2.4 m). Since the existing pipe is proposed to be abandoned, it is recommended that the new alignment has to be at least 3 pipe diameters offset to the north, relative to the existing culvert. The size and type of the new culvert is not defined at the time of writing this report. However, it is understood that, the proposed culvert will be installed 6 m offset to the north from the centerline of the existing culvert which accommodates pipe of up to 2 m in diameter. The existing culvert can be used to convey the surface water flow toward the outlet during the construction. However, after the construction of new culvert, the abandoned culvert must be properly decommissioned including its grouting and sealing.

2.4.1.1 Pipe Ramming with Hammer Head to Break Rock

Pipe ramming is a non-steerable system of advancing a pipe by driving an open-ended casing using a percussive hammer from a sending pit. Because of this force, only steel pipe may be used. A slurry of water and bentonite may be applied to the leading end and outer edge to help reduce friction. The leading edge of the casing can be protected with casing shoe while driving through rock fill. A smaller drill hammer can be used to fracture bigger rock that comes within tunnel alignment. In general, the spoils from the interior of the pipe can be removed as it is driven forward or after the pipe is in place. The major disadvantage of this method is associated with the difficulty to control line and grade. One of the main advantages of this method is its ability to break up cobbles and boulders that may be located along the tunnel alignment.

Based on the discussions with a local contractor, it may be possible to ram the open-ended pipe through the entire length of the installation using of hammer head to break the bigger boulders that may exist along the tunneling alignment. To use of a hammer head to break bigger boulders the soil within the casing may be removed partially or in whole which may create open-face conditions, and increased risk of ground loss into the casing. Therefore, it is recommended use of pre-grouting or freezing to avoid any kind of ground loss. However, grouting in rock fill would also be very challenging as it may have lot of voids between the big rocks. Lateral containment of grouting is also a challenge at this site. Likewise, ground freezing would not be option on this site since freezing works only in saturated soils which is not case at this site

This method is typically used to install liner pipes up to 1.5 m in diameter and drive lengths can be up to 60 m. Pipe diameters up to 3 m can also be achieved; however, installations of this size are not typical. Depending on the length of the installation and site condition, the soils inside the pipe can be removed either during or after the installation by manual excavation or auguring.

As mentioned above, pipe ramming is a non-steerable system, that means once the installation has begun, there is little control of the line and grade of the installation. Installation accuracy (vertical and horizontal) is usually about ± 1 percent of the length of the bore but the line and grade may deviate significantly on subsurface obstructions. The other disadvantage of this method is, vibration is created in the immediate vicinity of the installation due to pipe ramming operation, which may cause

deformation at the ground surface. During the pipe ramming operation, there is a risk of soil heave around the casing if the leading-edge is blocked due to obstructions.

2.4.1.2 Jack and Boring Using Disc Cutting Small Boring Unit (SBU-M)

The SBU-M is a motorized, manned-entry rock boring attachment to bore in a variety of rock types. It can be used with conventional auger boring machine from 1.2 m to 2.0 m or any standard pipe jacking unit. A SBU-M uses a hydraulic or electric motor to generate torque. The auger boring machine applies thrust to the steel casing. An invert auger enclosed in steel casing can be used to remove spoil. For mixed ground condition like at this site, the hard rock cutterhead features single disc cutters or two-row tungsten carbide insert cutters with carbide teeth can be used. In order to minimize the resistance along the pipe exterior, a bentonite grout lubricant can be injected behind the cutting face. Depending on the size of rock fragments and voids between the rock fill grains, pre-grouting or freezing might be required. Procedures that may be risky to implement successfully. If there is little to no fines/gravel between the big rock voids, grouting would be required in advance. A laser guided steering in SBU-M allow to check instantaneously the line and grade at any point during the bore.

A local contractor with this type of equipment might be available. However, during the boring a large rock or multiple large rocks could be suddenly rearranged within the embankment or at the tunnel face leading to large surface settlements or even a land slip if the rearranged section is close to either open end of the embankment.

2.4.1.3 Micro-tunneling Method

A micro-tunneling method is a non-entry, remotely controlled, guided 2-stage process, which provides continuous support to the excavation face. In this method, a Micro Tunneling Boring Machine (MTBM) is used for excavation, while a pipe is jacked into place behind the cutting head with hydraulics. The MTBM is equipped with a slurry spoil removal system to control the groundwater inflow and counterbalance the earth and hydrostatic pressure while tunneling through the mixed face conditions. The cutting tool and the drilling fluid must be able to handle the different materials including rock fill (rock fragments, cobbles and boulders) and the "mixed face" condition. In order to minimize the resistance along the pipe exterior, a bentonite grout lubricant can be injected behind the cutting face. Steel, concrete or fibreglass pipes can be installed with this method. Depending on the size of the rock fragments, voids between the rock fill grains and saturation of the infill, pre-grouting or freezing might be required (see Section 2.4.3.2). If there is little to no fines/gravel between the big rocks, grouting would be required in advance. Loss of slurry pressure would be a problem otherwise. If the voids in rock fill are well filled with gravel, the thick bentonite in the slurry water could be used to prevent loss of slurry pressure. In general, the major advantage of micro-tunneling method is that its performance is not affected by high groundwater levels, so the dewatering is not required. It is not a case at this site since the groundwater level was encountered high in the embankment. Major disadvantages of micro-tunneling for this project are considered to be the relatively high cost of mobilization and lack of locally skilled contractors for installation of culvert through the rock fill embankment. This option may become more attractive if potential bidders have available equipment in house and experience in such ground conditions. For excavation of the launching pit, a protection

system might be required to minimize possible negative impact on the stability of the existing embankment slope.

2.4.1.4 Open Face Jacking Shield with Backhoe Digging Arm

Open face jacking shield with backhoe digging arm is a man-entry tunneling method. The basic principle of this method is a rotating backhoe or digger arm with ripper-teeth at the end, which excavates only part of the face at one time. The shield provides unrestricted access to the tunnel heading and in the event, that large rock pieces are encountered they could be removed by a backhoe digger arm or manually. The tunnel support is by a jacking pipe or a rib and lagging system behind the shield. Forward movement of the shield is accomplished by a series of hydraulic jacks that thrust against the trailing pipe or last-installed steel rib. Some experience contractor might be available in Ontario.

The method is considered suitable for mixed ground tunneling, particularly in unsaturated soil conditions. It has superior capability in handling materials of different sizes at the face including boulders. However, the new pipe has to be big enough to accommodate this method operation. In addition, a sudden rush of boulders into the face would be safety concern for the miners. To prevent collapse, the rock fill could be pre-grouted which could be done through horizontal drilled holes (see Section 2.4.3.2). This procedure could be costly and risky.

2.4.2 Culvert Installation at an Existing Alignment

If the existing culvert alignment is considered for the installation of a new culvert, the culvert trenchless (tunneling) excavation will be carried out through the existing pipe (i.e. pilot hole) and surrounding backfill probably consisting of various size of blasted rock fragments. Since the tunnel excavation will be through rock fill, appropriate equipment and construction method must be selected based on ability to accommodate these obstructions. In this case, provision must be made to maintain surface water flow to the outlet.

2.4.2.1 Pipe Eating (Swallowing)

Pipe eating is a modified micro-tunneling system specially adapted for pipe replacement. The method is different than the pipe bursting operation. The crushed fragments of pipe mixed with soil are vacuumed out, as slurry, through the new pipe and out of the space. A new pipe, of the same or larger nominal size, is simultaneously installed by jacking it behind the microtunneling machine. The new pipe may follow the line of the old pipe on the entire length, or may cross the elevation of the old pipe on a limited segment only. The system is remotely controlled and guided with a surveyed laser line from the drive pit, and prepared to "eat" whatever is in the way, the old pipe or the ground only. The system has a cutting head and a shield section. The cutting head has cutting teeth and rollers that cut the pipe, and cutting arrangements close to the edge of the shield that cut the ground to the required diameter to take the new pipe. The cutting head is cone-shaped, which puts the material of the old pipe into tension and thus reduces the heavy wear of cutting teeth. The shield section carries the cutting head and its hydraulic motor system. The head and shield are launched from a drive pit,

where a thrust frame is located. It provides a thrust that is applied on the cutting head through the new pipe to push the head and shield forward through the ground.

As indicated above in micro-tunneling section, in order to minimize the resistance along the pipe exterior, a bentonite grout lubricant can be injected behind the cutting face. Depending on the size of the rock fragments and voids between the rock fill grains, pre-grouting or freezing might be required as discussed in Section 2.4.3.2 below. Primary support can be also provided by jacking a pipe from a jacking station behind the boring machine if pre-grouting is done. Pipes may be made of various materials (concrete, steel, fibreglass, etc.). Selected pipe must conform to OPS requirements for embankment depth of 12 m. The launching pit and jacking station should be constructed at the inlet side. A protection system might be required to minimize possible negative impact on the stability of the existing roadway.

2.4.2.2 Pipe Reaming

A pipe reaming operation is similar to the pipe eating technique in that it involves crushing the old pipe and mixing the pipe fragments and soil into a slurry, which is then pumped back to the surface for disposal. Pipe reaming is a modified back reaming method used in directional drilling, which is specially adapted for pipe replacement. First, the pilot drill string is inserted through the existing pipe. Next, a specially designed reaming tool is attached to the drill string and pulled back through the pipe, while simultaneously installing the new pipe. The reamer has cutting teeth, which grind and pulverize the existing pipe through a “cut and flow” process, rather than a compaction. The pipe fragments and the excess material from upsizing are carried with the drilling fluid to manholes or reception pits, and retrieved with a vacuum truck or slurry pump for disposal. The new pipe is typically a thermoplastic pipe suitable for installations using directional drilling equipment. In this case, the existing pipe could serve as a pilot hole.

To prevent collapse of rock fill during tunneling or to prevent loss of slurry pressure pre-grouting or freezing might be required as discussed in Section 2.4.3.2 below.

2.4.3 Considerations of Tunneling

2.4.3.1 Groundwater and Surface Water Control

As mentioned before, a small amount of groundwater seepage into the tunnel should be expected in the zone of tunneling suggesting a low complexity of dewatering. However, dewatering might be required in the launching pit prior to advancing the pipe to ensure dry working conditions and stabilize the excavation in that zone. The dewatering would need to be carried out to temporarily lower the groundwater level to at least 1 m below the base of the excavation. Dewatering shall be carried out in accordance with OPSS.PROV 517 and OPSS.PROV 518. It is responsibility of the Contractor to propose a suitable dewatering system based on the time of construction, water levels and flow conditions for prior approval of the MTO. The method used should not undermine the existing road embankment or adjacent side slopes. In this connection, the provision of toe protection at side slopes during drawdown may be required to minimize sloughing and undercutting during dewatering. Alternatively, and in accordance with SP 517F01, the dewatering systems may be completed by a

design Engineer and design-checking Engineer with a minimum of 5 year experience. For this application, this is considered a suitable approach but the owner should make final decision. Based on the estimated permeability of gravelly sand ($k \sim 5 \times 10^{-4}$ m/s), the preconstruction survey distance should be approximately 150 m (a recommended value for Table A in SP 517F01).

If the new culvert is installed at the new alignment, the existing culvert could be used to convey the surface water flow towards the outlet. However, if the existing alignment is used then the surface water flow has to be diverted from this site. The Contractor has to propose a suitable surface water collection system to allow safe and dry installation of the new culvert.

2.4.3.2 Ground Grouting/Freezing

To prevent collapse of rock fill during tunneling or to prevent loss of slurry pressure (in case of micro-tunneling) if there is little to no fines/ gravel between the big rock pieces, grouting would be required in advance. The grouting could be done through vertical and/or horizontal drilled holes, however this procedure could be costly. Alternatively, freezing of ground in the tunneling horizon can also be considered if constructed during winter period. However, ground freezing works only in saturated soils such as silty sand, which is not the case at this site. In both cases, specialized contractor for grouting/freezing have to be hired to address the ground stabilization.

Post tunneling grouting would also required as generally, there is a risk of over-excavation and the formation of voids around the liner pipe in any tunneling operation. To minimize ground surface settlement and to avoid unbalanced loads on the liner, grouting around the liner is generally recommended. The need for grouting around the liner pipe should be evaluated once tunneling is complete.

The amount of spoil removed during tunneling should be monitored to determine whether over-excavation is occurring. If there is suspicion that over-excavation has occurred, and/or if the settlement monitoring indicated that the ground surface has settled, then a plan should be in place for investigating of presence of gaps/voids in the soil above the pipe and for remediation measures such as filling the gaps/voids with grout. The contractor should develop a contingency plan incorporating appropriate soil volume monitoring to address loss of material from outside the pipe during the tunneling operation, as discussed in Section 2.4.4.

2.4.3.3 Ground Settlement

Settlement around the culvert is a result of ground loss or “immediate” settlement caused by tunneling. Presence of the peat layer underlying rock fill layer may also aggravate the settlement during tunneling. The immediate settlement is a direct result of the overcut and movement of ground at the heading during tunneling. The factors that influence the immediate settlement include the ground condition and the method of tunneling. The Contractor should keep the settlement under the MTO’s required limit of 10 mm. Technical specifications should ensure that:

- The use of over-cutters (excavating to a diameter greater than the pipe diameter) is kept under 10 mm;
- The overcut area is grouted in a timely manner (if a man-entry tunnel is constructed grout should be injected immediately after support is installed); and

- The program of instrumentation is carried out as per MTO guidelines (see Section 2.4.4).

Considering the fact that excavation will be in rock fill with significant size of voids within its matrix, it is anticipated that some soil stabilization measures such as grouting/ ground freezing (as mention above in Section 2.4.3.2) should be applied to arrest or reduce settlement.

2.4.3.4 Excavation Pits

The launching and receiving pits for the tunneling equipment are expected to be located at the inlet and outlet of the proposed culvert location, respectively. The bases of the pits are expected to be set at about 0.5 to 1 m depth from invert of the proposed culvert. Since, bedrock at inlet and outlet locations are at very shallow depth ranging between 0.2 m to 1.5 m below the existing grade, excavations for launching and receiving pits will be conducted through rock fill and/or bedrock. Some bedrock blasting will be required for excavation of launching and receiving pit. A Non-Standard Special Provision (NSSP) for the blasting of bedrock should be included in the contract documents and a sample has been provided in Appendix H of this report. In order to provide the required excavation geometry for the drilling (e.g. vertical front face for tunnel entry and a vertical rear face with a ballast system to act as a reaction force), the sides of the excavation will have to be shored. Ingress of groundwater and surface water has to be controlled as explained previously. Technical specifications must ensure that the Contractor submits a groundwater and surface water control plan describing the proposed method for control. In this site the existing culvert could be used to convey the creek water during the construction. As indicated above, it should be noted that, if the replacement of culvert at the location of existing culvert option is selected, construction of the launching or receiving pit at inlet side would be very difficult due to presence of bedrock outcrop. Blasting of some bedrock may be needed to create space for the launching or receiving pit.

2.4.3.5 Backfilling in Pits

It is anticipated that backfilling work will be required at the launching and receiving pits to return site condition to pre-construction grades. The following comments and recommendations are provided for backfilling such excavations.

All excavations should be backfilled with inorganic on-site soils placed in maximum 300 mm thick lifts and compacted to at least 95% of the Standard Proctor Maximum Dry Density (SPMDD). Any organic, excessively wet, compressible or otherwise deleterious materials should not be used for backfilling purposes. Any shortfall of suitable on-site excavated materials can be made up with imported and approved materials.

All backfill and compaction operations should be monitored by qualified geotechnical personnel to approve materials, to evaluate placement operations, and to verify that the specified degree of compaction is being achieved throughout the fill.

2.4.4 Monitoring and Contingency Plan

It is emphasized that the resulting performance of the installed culvert will largely be dependent upon construction procedures and techniques. However, regardless of the method of tunneling selected

for this project, it is recommended that the contractor develop a contingency plan incorporating an appropriate monitoring plan. The monitoring plan should be carried out in accordance with MTO CMO Guidelines for Tunnelling and Special Provision for Pipe Installation by Trenchless Method, and it should include at a minimum the following items:

- a) an "Alert" level(s) at which the plan would be implemented;
- b) a means to close the tunnel, and preferably to pressurize the pipe; and
- c) an emergency personnel/agency contact list.
- d) a clean communication strategy in the case of specific events/distress indicators.

Settlements should be monitored during construction to ensure compliance with MTO guidelines and the contract requirements. The instrumentation program should adequately verify effects of tunneling on the overlying highway and obtain advance warning of ground movements. The scope and layout of settlement instruments should be in general accordance with the MTO guidelines presented in Appendix: Settlement Monitoring Guideline – Tunneling. This should include a series of surface monitoring points placed at a maximum spacing of 5 m along the entire length of the proposed culvert. All monitoring points located in the unpaved portion of the right-of-way are to be founded below the frost penetration depth, which is typically 1.8 m in this area.

According to MTO Special Provision for Pipe Installation by Trenchless Method, a reading schedule should be as follows:

- Three consecutive readings at least one week prior to construction as a baseline reading.
- Once per shift during tunneling operation periods provided the movements are within the anticipated limits. Otherwise, the reading frequency may have to be increased.
- Weekly readings after completion of the work for one month, or until such time at which all parties agree that further movement has stopped.

Instrumentation plans should be finalized once the Contractor is selected and when his construction methods are known.

As mentioned, control of ground settlement on this project depends on the behaviour of rock fill at the tunnel face and on the tunneling methodology employed by the Contractor. Therefore, it is recommended that a geotechnical engineer be present during active excavation to verify that the ground conditions are consistent with those encountered in the investigation boreholes. Furthermore, it is recommended that the volume of the material removed from the tunnel be monitored and continuously compared to the rate of tunnel advance. This will provide an indication if any over-excavation is taking place. A NSSP for the monitoring of tunneling activities by a geotechnical engineer should be included in the contract documents and a sample has been provided in Appendix H of this report.

The criteria for evaluation of settlement should be based on the following action levels:

1. *Review Level:* If a maximum value of 10 mm relative to the baseline readings is reached, the

method, rate or sequence of construction, or ground stabilization measures shall be reviewed or modified to mitigate further ground displacements.

2. **Alert Level:** If a maximum of 15 mm relative to the baseline readings is reached, the Contractor shall be required to cease construction operation or to execute pre-planned measures to secure the site to mitigate further unacceptable settlement and to assure safety of public.

A field monitoring program for the tunnel also should be proposed to monitor the tunnel excavation stability. The program of instrumentation in the tunnel could include surveying targets/prisms. Usually these targets are installed at the crown, spring line and invert levels to obtain the radial convergence measurements, if necessary, and along the tunnel. A special laser survey equipment could be used to measure the distances.

2.4.5 Protection Systems

Depending on the tunneling method chosen for this project and the excavations that will be required to implement them, protection system(s) may be required for the existing roadway. The need for these systems will depend on the proposed geometry of the required excavations and their proximity to the existing highway structure. If required, protection systems (design, materials, construction, maintenance, monitoring and removal) will be required to meet the specifications set out in OPSS 539 and Special Provision No. 105S19. The lateral movement of the temporary shoring system should meet Performance Level 2 as specified in OPSS.PROV 539.

The protection system (shoring) should be designed using the state-of-the-practice information presented in the fourth edition of the Canadian Foundation Engineering Manual (CFEM). Geotechnical parameters that are considered to be appropriate are as follows:

Earth Pressure Coefficient $K_a = 0.22$ where small movements permissible ($\phi=40$ deg)

Rock fill Unit Weight $\gamma = 18 \text{ KN/m}^3$

It should be recognized that the final shoring design will be prepared by the shoring contractor. It is not possible to comment further on specific design details until this design is completed.

2.5 Inlet and Outlet

2.5.1 Erosion Protection at Outlet

Final requirements for and design of erosion protection measures for the inlet and outlet of culverts should be assessed by the hydraulics engineer. The following comments are provided for general guidance. The rip-rap should extend approximately 5 m beyond the ends of the culvert and line the embankment slope to the spring line of the culvert. The size of the rip-rap is a function of the surface water hydrology. As a rule of thumb the thickness of the rip-rap should be a minimum of twice the median particle size, and 300 mm thick as a minimum. The rip-rap configuration at the downstream bed should generally follow the OPSD 810.010, which is included in Appendix G of this report. Rip-

rap placed at 1V:1H will be stable. The final design for erosion protection should be developed/approved by the hydraulic engineer.

2.5.2 Stream Bed Rip-Rap

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

2.6 Slope Stability and Settlement Analyses

2.6.1 Stability

As mentioned before no stability issues was observed during field investigation. Therefore, the existing rock fill embankment with the approximate side slopes of 1.5H:1V to 1.25H:1V on inlet and outlet sides, respectively, are expected to be stable if the tunneling installation method is applied.

2.6.2 Settlement

Since the approach embankments are not going to be raised significantly no significant settlement of the structures is anticipated at the site.

August 31, 2017

3 CLOSURE


The comments given in this report are intended only for the guidance of design engineers. The number of boreholes required to determine the localized underground conditions between boreholes affecting construction costs, techniques, sequencing, equipment, scheduling, etc. could be greater than has been carried out for design purposes. Contractors bidding on or undertaking the works should, in this light, decide on their own investigations, as well as their own interpretations of the factual borehole results, so that they may draw their own conclusions as to how the subsurface conditions may affect them.

This Foundation Investigation Report has been prepared by Mr. Nimesh Tamrakar, M.Eng, EIT. and Dr. S. Micic, Ph.D., P. Eng. and reviewed by Mr. T.C. Kim, M.E.Sc., P.Eng. and Mr. S.E. Gonsalves, M.Eng., P.Eng. designated MTO foundation contact. The field investigation was conducted by Mr. Devendra Panchal.


We trust that these comments provide you with sufficient information to for your present requirements. Should you have any questions, please do not hesitate to contact this office


Yours truly,

exp Services Inc.


for Nimesh Tamrakar, M.Eng., EIT.
Technical Specialist


TaeChul Kim, M.E.Sc., P.Eng.
Senior Geotechnical/Foundation Specialist


Silvana Micic, Ph.D., P.Eng.
Senior Geotechnical Engineer
Project Manager


Stan E. Gonsalves, M.Eng., P.Eng.
Principal Engineer
Designated MTO Foundation Contact

Encl.



4 LIMITATIONS AND USE OF REPORT

BASIS OF REPORT

This report ("Report") is based on site conditions known or inferred by the geotechnical investigation undertaken as of the date of the Report. Should changes occur which potentially impact the geotechnical condition of the site, or if construction is implemented more than one year following the date of the Report, the recommendations of exp may require re-evaluation.

The Report is provided solely for the guidance of design engineers and on the assumption that the design will be in accordance with applicable codes and standards. Any changes in the design features which potentially impact the geotechnical analyses or issues concerning the geotechnical aspects of applicable codes and standards will necessitate a review of the design by exp. Additional field work and reporting may also be required.

Where applicable, recommended field services are the minimum necessary to ascertain that construction is being carried out in general conformity with building code guidelines, generally accepted practices and exp's recommendations. Any reduction in the level of services recommended will result in exp providing qualified opinions regarding the adequacy of the work. exp can assist design professionals or contractors retained by the Client to review applicable plans, drawings, and specifications as they relate to the Report or to conduct field reviews during construction.

Contractors contemplating work on the site are responsible for conducting an independent investigation and interpretation of the borehole results contained in the Report. The number of boreholes necessary to determine the localized underground conditions as they impact construction costs, techniques, sequencing, equipment and scheduling may be greater than those carried out for the purpose of the Report.

Classification and identification of soils, rocks, geological units, contaminant materials, building envelopment assessments, and engineering estimates are based on investigations performed in accordance with the standard of care set out below and require the exercise of judgment. As a result, even comprehensive sampling and testing programs implemented with the appropriate equipment by experienced personnel may fail to locate some conditions. All investigations or building envelope descriptions involve an inherent risk that some conditions will not be detected. All documents or records summarizing investigations are based on assumptions of what exists between the actual points sampled. Actual conditions may vary significantly between the points investigated. Some conditions are subject to change over time. The Report presents the conditions at the sampled points at the time of sampling. Where special concerns exist, or the Client has special considerations or requirements, these should be disclosed to exp to allow for additional or special investigations to be undertaken not otherwise within the scope of investigation conducted for the purpose of the Report.

RELIANCE ON INFORMATION PROVIDED

The evaluation and conclusions contained in the Report are based on conditions in evidence at the time of site inspections and information provided to exp by the Client and others. The Report has been

prepared for the specific site, development, building, design or building assessment objectives and purpose as communicated by the Client. exp has relied in good faith upon such representations, information and instructions and accepts no responsibility for any deficiency, misstatement or inaccuracy contained in the Report as a result of any misstatements, omissions, misrepresentation or fraudulent acts of persons providing information. Unless specifically stated otherwise, the applicability and reliability of the findings, recommendations, suggestions or opinions expressed in the Report are only valid to the extent that there has been no material alteration to or variation from any of the information provided to exp.

STANDARD OF CARE

The Report has been prepared in a manner consistent with the degree of care and skill exercised by engineering consultants currently practicing under similar circumstances and locale. No other warranty, expressed or implied, is made. Unless specifically stated otherwise, the Report does not contain environmental consulting advice.

COMPLETE REPORT

All documents, records, data and files, whether electronic or otherwise, generated as part of this assignment form part of the Report. This material includes, but is not limited to, the terms of reference given to exp by its client ("Client"), communications between exp and the Client, other reports, proposals or documents prepared by exp for the Client in connection with the site described in the Report. In order to properly understand the suggestions, recommendations and opinions expressed in the Report, reference must be made to the Report in its entirety. exp is not responsible for use by any party of portions of the Report.

USE OF REPORT

The information and opinions expressed in the Report, or any document forming part of the Report, are for the sole benefit of the Client. No other party may use or rely upon the Report in whole or in part without the written consent of exp. Any use of the Report, or any portion of the Report, by a third party are the sole responsibility of such third party. exp is not responsible for damages suffered by any third party resulting from unauthorised use of the Report.

REPORT FORMAT

Where exp has submitted both electronic file and a hard copy of the Report, or any document forming part of the Report, only the signed and sealed hard copy shall be the original documents for record and working purposes. In the event of a dispute or discrepancy, the hard copy shall govern. Electronic files transmitted by exp have utilize specific software and hardware systems. exp makes no representation about the compatibility of these files with the Client's current or future software and hardware systems. Regardless of format, the documents described herein are exp's instruments of professional service and shall not be altered without the written consent of exp.

Appendix A – Photographs



Photo 1: Hwy 118 looking south-east (upward) from existing culvert location



Photo 2: Hwy 118 looking north-west (downward) from existing culvert location



Photo 3: Looking east (inlet side) at existing culvert location



Photo 4: Looking west (outlet side) at existing culvert location



Photo 5: Looking South-East from north of existing culvert inlet



Photo 6: Looking west from inlet side of existing culvert



Photo 7: Looking east from outlet of existing embankment



Photo 8: Looking north and BH4 location from existing culvert outlet



Photo 9: East side (Inlet) slope at proposed culvert location. Looking west from BH3 location



Photo 10: West side (Outlet) slope at proposed culvert location. Looking east from BH4 location



Photo 11: Inlet side, bedrock outcrops



Photo 12: Outlet side, bedrock outcrops



Photo 13: Deterioration of culvert at inlet



Photo 14: TBM on rock outcrop north-west of proposed culvert

TEST PITS



Photo 15: Test pit TP1 at the west embankment slope facing outlet side



Photo 16: Test pit TP1 at the west embankment slope, various sized rock fragments in soil matrix



Photo 17: Test pit TP2 at the east embankment slope facing west, inlet side



Photo 18: Test pit TP2 at the east embankment slope, various sized rock fragments in soil matrix



Photo 19: Test pit TP3 at the toe of west embankment slope facing west, outlet side



Photo 20: Test pit TP3 at the toe of west embankment slope, beginning of excavation



Photo 21: Test pit TP3 at the toe of the west embankment side, cobble and boulder sized rock fill



Photo 22: Test pit TP4 at the toe of the east embankment side, inlet side



Photo 23: Test pit TP4 at the toe of the east embankment side, beginning of excavation



Photo 24: Test pit TP4 at the toe of the east embankment side, cobbles and boulder sized rock fill

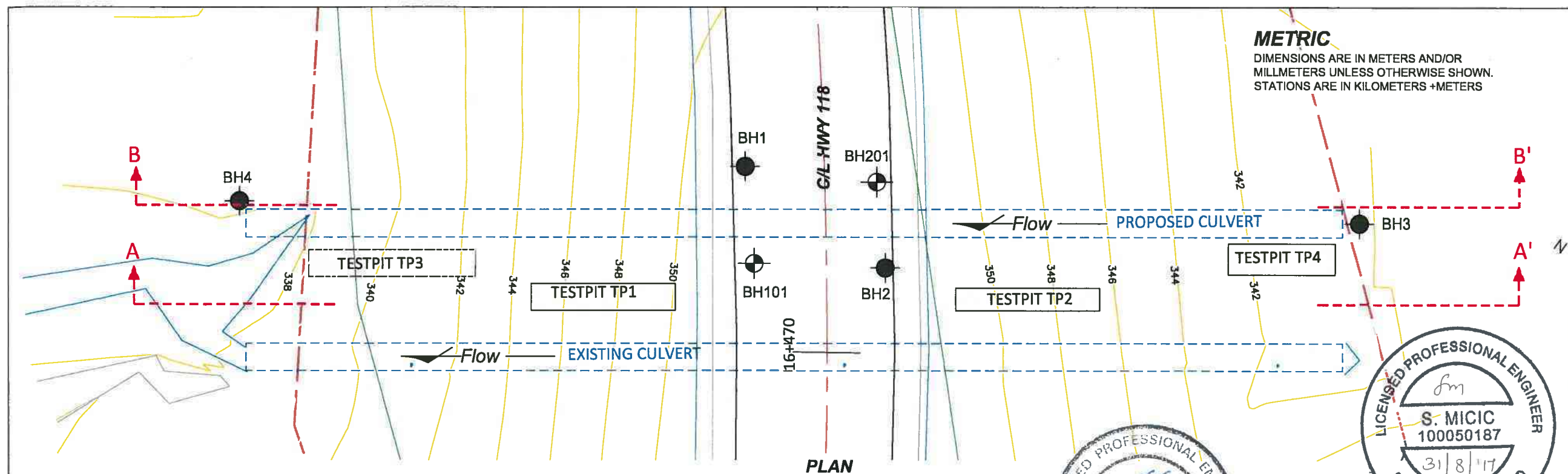


Photo 25: Test pit TP4 at the toe of the east embankment side, boulder sized rock fill

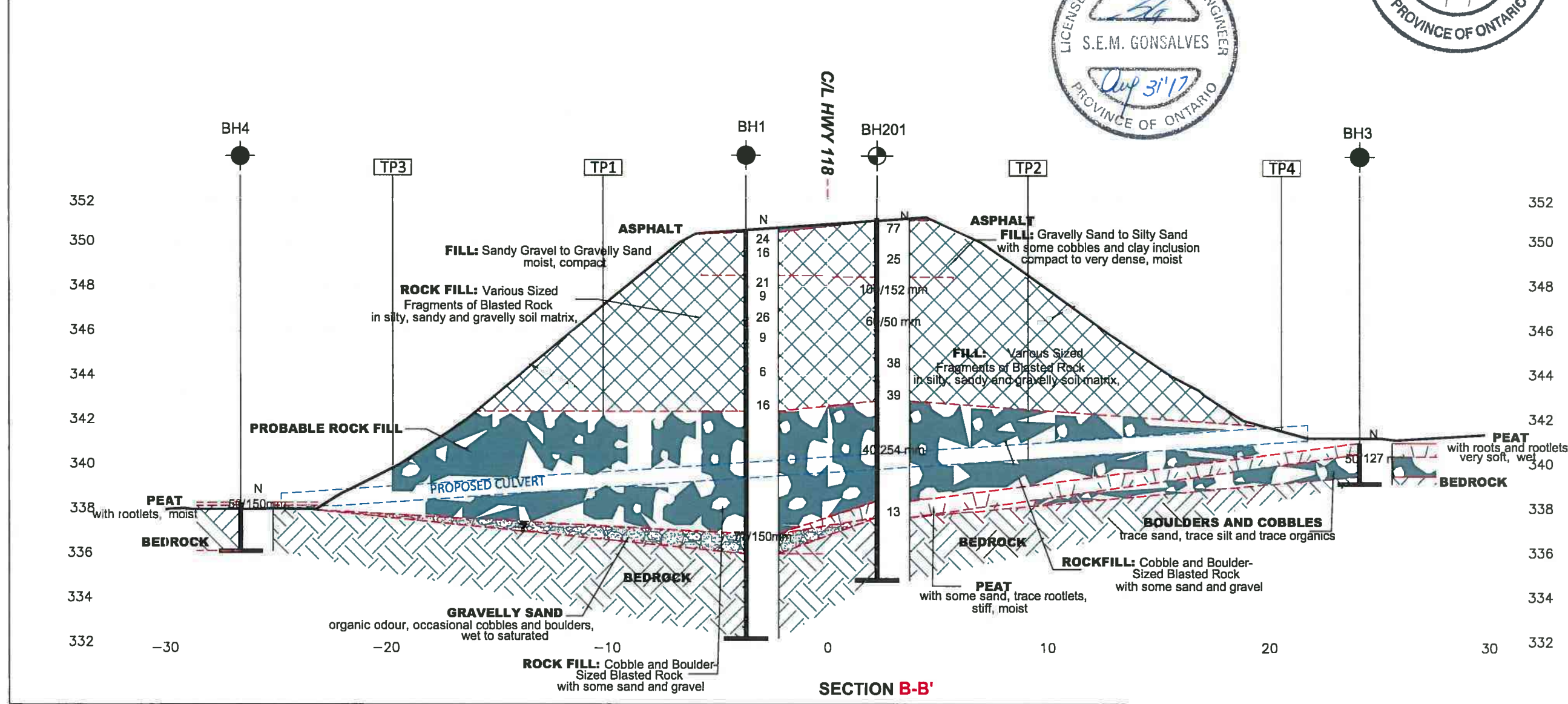


Photo 26: Test pit TP4 at the toe of the east embankment side, bedrock elevation

Appendix B – Drawings



PLAN



SECTION B-B'

NER Agreement No. 5015-E-0007
Assignment No. 5
G.W.P. 5140-13-00

PROPOSED REPLACEMENT OF CULVERT ON HWY 118,
TOWNSHIP OF STANHOPE
BOREHOLE LOCATION PLAN AND SOIL STRATA

SHEET

exp

exp Services Inc.

KEY PLAN

SITE

LEGEND

New Borehole by EXP (2017)

Previous Borehole by EXP (2016)

Test Pit (2017)

N Standard Penetration Test (Blows/0.3 m)

SOIL STRATA SYMBOLS

ASPHALT

BEDROCK

SANDY SILT

FILL

COBBLES & BOULDERS

PEAT

GRAVELLY SAND

BH No.	APPROX. ELEV.	MTM CO-ORDINATES	
		NORTHING	EASTING
BH 101	350.7	4989980	366933
BH 201	351.1	4989986	366935
BH 1	350.6	4989983	366930
BH 2	351.2	4989983	366938
BH 3	341.0	4989997	366954
BH 4	338.3	4989969	366912
TP 1	-	4989974	366928
TP 2	-	4989985	366944
TP 3	-	4989997	366954
TP 4	-	4989993	366912

NOTE

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

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

SCALE

0 5 m

DATE	BY	DESCRIPTION
24/08/2017	-	SUBMISSION FOR MTO REVIEW
		GEOCRES NO. 31E 382
		PROJECT NO. ADM-00233185-F0
SUBMD SM	CHECKED SM	DATE 24/08/2017
DRAWN SH	CHECKED SG	APPROVED SG DWG 2

Appendix C – Boreholes Logs

Explanation of Terms Used on Borehole Records

SOIL DESCRIPTION

Terminology describing common soil genesis:

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

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

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

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

Terminology describing soil structure:

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

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

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

Fissured: material breaks along plane of fracture.

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

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

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

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

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

Homogeneous: same color and appearance throughout.

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

Uniformly Graded: predominantly on grain size.

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

ISSMFE SOIL CLASSIFICATION											
CLAY	SILT			SAND			GRAVEL			COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE		
<div><div>0.002</div><div>0.006</div><div>0.02</div><div>0.06</div><div>0.2</div><div>0.6</div><div>2.0</div><div>6.0</div><div>20</div><div>60</div><div>200</div></div>											
EQUIVALENT GRAIN DIAMETER IN MILLIMETRES											
CLAY (PLASTIC) TO				FINE		MEDIUM		CRS.		FINE COARSE	
SILT (NONPLASTIC)				SAND				GRAVEL			
UNIFIED SOIL CLASSIFICATION											

Terminology describing materials outside the USCS, (e.g. particles larger than 76 mm, visible organic matter, construction debris) is based upon the proportion of these materials present and as described below in accordance with Note 16 in ASTM D2488-09a:

Table a: Percent or Proportion of Soil, Pp

	Criteria
Trace	Particles are present but estimated to be less than 5%
Few	$5 \leq Pp \leq 10\%$
Little	$15 \leq Pp \leq 25\%$
Some	$30 \leq Pp \leq 45\%$
Mostly	$50 \leq Pp \leq 100\%$

The standard terminology to describe cohesionless soils includes the compactness as determined by the Standard Penetration Test 'N' value:

Table b: Apparent Density of Cohesionless Soil

	'N' Value (blows/0.3 m)
Very Loose	$N < 5$
Loose	$5 \leq N < 10$
Compact	$10 \leq N < 30$
Dense	$30 \leq N < 50$
Very Dense	$50 \leq N$

The standard terminology to describe cohesive soils includes consistency, which is based on undrained shear strength as measured by insitu vane tests, penetrometer tests, unconfined compression tests or similar field and laboratory analysis, Standard Penetration Test 'N' values can also be used to provide an approximate indication of the consistency and shear strength of fine grained, cohesive soils:

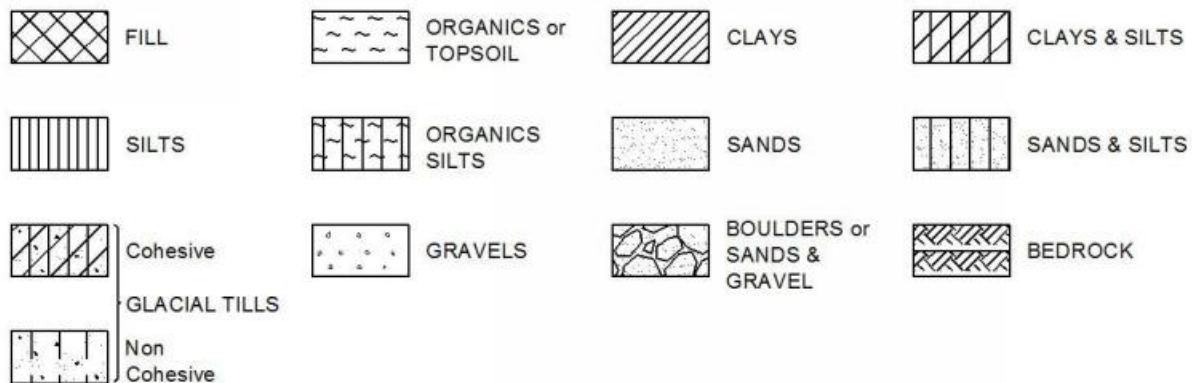
Table c: Consistency of Cohesive Soil

Consistency	Vane Shear Measurement (kPa)	'N' Value
Very Soft	<12.5	<2
Soft	12.5-25	2-4
Firm	25-50	4-8
Stiff	50-100	8-15
Very Stiff	100-200	15-30
Hard	>200	>30

Note: 'N' Value - The Standard Penetration Test records the number of blows of a 140 pound (64kg) hammer falling 30 inches (760mm), required to drive a 2 inch (50.8mm) O.D. split spoon sampler 1 foot (305mm). For split spoon samples where full penetration is not achieved, the number of blows is reported over the sampler penetration in meters (e.g. 50/0.15).

STRATA PLOT

Strata plots symbolize the soil or bedrock description. They are combinations of the following basic symbols:



WATER LEVEL MEASUREMENT



ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

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

STRESS AND STRAIN

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

MECHANICAL PROPERTIES OF SOIL

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

PHYSICAL PROPERTIES OF SOIL

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

Brampton, Ontario

RECORD OF BOREHOLE No BH101

1 OF 2

METRIC

W.P. 5140-13-00 LOCATION Highway 118, MTM-10, 5461909.5N, 259501.0E ORIGINATED BY DP
 DIST Haliburton County HWY 118 TEST PIT TYPE CME 85/SSA/NW/NQ COMPILED BY NT
 DATUM BM Elev. 343.96 m DATE 2017.05.03 - 2017.05.04 LATITUDE 49.2935 LONGITUDE -81.3728 CHECKED BY SM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)						
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									WATER CONTENT (%)			GR	SA	SI	CL
								20	40	60	80	100											
351.0	Road Surface																						
350.9	ASPHALT (~ 115 mm thick)																						
0.1	FILL: GRAVELLY SAND TO SILTY SAND with some cobbles, brown, moist, compact .		1	AS														2	73 (25)				
			2	SS	13		350																
	- Switch to casing/boring @ 1.5 m																						
							349																
348.7																							
2.3	FILL: VARIOUS SIZED FRAGMENTS OF BLASTED ROCK in silty, sandy and gravelly soil matrix		3	NQ																			
	Run 1 (3 NQ) - 2.28 m to 2.6 m																						
	Hard @2.28 m L = 0.1 m																						
	Soft @2.39 m L = 0.050 m		4	SS	22		348																
	Hard @2.44 m L = 0.050 m																						
	Soft @2.49 m L = 0.1 m																						
	Run 2 (5 NQ) - 3.05 m to 4.57 m																						
	Hard @3.05 m L = 0.74 m																						
	Soft @3.78 m L = 0.48 m																						
	Hard @4.27 m L = 0.178 m																						
	Soft @4.45 m L = 0.127 m		5	NQ			347																
	-recovery only 0.025 m thick sample		6	SS	7		346																
	Run 3 (7 NQ) - 5.03 m to 6.05 m																						
	Hard @5.03 m L = 0.76 m																						
	Soft @5.79 m L = 0.254 m																						
			7	NQ			345																
	-recovery only 0.076 m thick sample		8	SS	17		344																
	Run 4 (9 NQ) - 6.66 m to 7.55 m																						
	Void @6.66 m L = 0.51 m																						
	Hard @7.16 m L = 0.076 m																						
	Soft @7.24 m L = 0.1 m																						
	Hard @7.34 m L = 0.076 m																						
	Soft @7.42 m L = 0.050 m		9	NQ			343																
	Hard @7.47 m L = 0.076 m																						
	Run 5 (10 NQ) - 7.55 m to 8.92 m																						
	Hard @7.55 m L = 0.152 m																						
	Void @7.7 m L = 0.84 m (Soft driven)																						
	Hard @8.54 m L = 0.051 m																						
	Soft @8.59 m L = 0.20 m																						
	Hard @8.79 m L = 0.127 m																						

Continued Next Page

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

ONTARIO MTO ADM-00233185.F0 - HWY 118.GPJ ONTARIO MTO.GDT 7/27/17

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

Brampton, Ontario

RECORD OF BOREHOLE No BH201

1 OF 3

METRIC

W.P. 5140-13-00 LOCATION Highway 118, MTM-10, 5461901.9N, 259513.9E ORIGINATED BY DP
 DIST Haliburton County HWY 118 TEST PIT TYPE CME 75/HSA/NW/NQ COMPILED BY NT
 DATUM BM Elev. 343.96 m DATE 2016.12.06 - 2016.12.07 LATITUDE 49.29344 LONGITUDE -81.62263 CHECKED BY SM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)		
								○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE × LAB VANE									
351.1	Road Surface																	
351.0	ASPHALT (~ 140 mm thick)																	
0.1	FILL: GRAVELLY SAND TO SILTY SAND with some cobbles and clay inclusion, brown, moist, compact to very dense.		1	SS	77													
	- Switch to casing/boring @ 1.5 m		2	SS	25										21 66 (13)			
348.4																		
2.7	FILL: VARIOUS SIZED FRAGMENTS OF BLASTED ROCK in silty, sandy and gravelly soil matrix Run 1 (3 NQ) - 2.67 m to 3.05 m Hard @2.67 m L = 0.076 m Soft @2.74 m L = 0.127 m Hard @2.87 m L = 0.178 m Run 2 (5 NQ) - 3.35 m to 4.57 m Soft @3.35 m L = 0.53 m Hard @3.89 m L = 0.076 m Soft @3.96 m L = 0.254 m Hard @4.22 m L = 0.05 m Soft @4.27 m L = 0.127 m Hard @4.39 m L = 0.178 m		3	NQ														
			4	SS	100/152 mm													
			5	NQ														
			6	SS	60/50 mm													
	-recovery only 0.05 m thick sample Run 3 (7 NQ) - 4.72 m to 6.25 m Hard @4.72 m L = 0.381 m Soft @5.1 m L = 0.584 m Hard @5.69 m L = 0.152 m Soft @5.84 m L = 0.254 m Hard @6.1 m L = 0.152 m		7	NQ														
			8	SS	38													
	-recovery only 0.076 m thick sample																	
	Run 4 (9 NQ) - 6.94m to 7.62 m Soft @6.94 m L = 0.076 m Hard @7.01 m L = 0.127 m Soft @7.14 m L = 0.076 m Hard @7.21 m L = 0.279 m Soft @7.49 m L = 0.127 m		9	NQ														
	-recovery only 0.05 m thick sample		10	SS	39													

Continued Next Page

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

ONTARIO MTO ADM-00233185.F0 - HWY 118.GPJ ONTARIO MTO.GDT 7/27/17


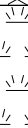

Brampton, Ontario

RECORD OF BOREHOLE No BH201

2 OF 3

METRIC

W.P. 5140-13-00 LOCATION Highway 118, MTM-10, 5461901.9N, 259513.9E ORIGINATED BY DP
 DIST Haliburton County HWY 118 TEST PIT TYPE CME 75/HSA/NW/NQ COMPILED BY NT
 DATUM BM Elev. 343.96 m DATE 2016.12.06 - 2016.12.07 LATITUDE 49.29344 LONGITUDE -81.62263 CHECKED BY SM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL					
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)				
								○ UNCONFINED		+ FIELD VANE						● QUICK TRIAXIAL		× LAB VANE		
						20	40	60	80	100	20	40	60							
348.2	ROCKFILL: COBBLE AND BOULDER-SIZED BLASTED ROCK with some sand and gravel Run 5 (11 NQ) - 8.23 m to 9.27 m Soft @8.23 m L = 0.152 m Hard @8.38 m L = 0.356m Soft @8.74 m L = 0.178 m Hard @8.91 m L = 0.356 m		11	NQ																
	Run 6 (12 NQ)- 9.27 m to 10.16 m Hard @9.27 m L = 0.533 m Void @9.8 m L = 0.356 m (dropped suddenly)		12	NQ																
	----- -recovery only 0.025 m thick sample		13	SS	40/ 254 mm															
	Run 7 (14 NQ) - 10.67 m to 11.58 m Hard @10.67 m L = 0.432m Soft @11.1 m L = 0.152 m Hard @11.25 m L = 0.178m Soft @11.38 m L = 0.178 m Hard @11.51 m L = 0.076 m		14	NQ																
	Run 8 (15 NQ)- 11.58 m to 12.73 m Soft @11.58 m L = 0.152 m Hard @11.74 m L = 1.0 m Soft @12.74 m L = 0.228 m (Probably Peat Layer)		15	NQ																
338.4	Peat , with some sand, trace rootlets, brown/black, moist, stiff.		16	SS	13								83.7							
337.6	BEDROCK black, grey granite with pink NQ CORING Length (m) RQD(%) Run 1 1.07 21.4 Run 2 1.5 35.6		17	NQ																
			18	NQ																
334.8	END OF BOREHOLE																			
16.3																				

Continued Next Page

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

ONTARIO MTO ADM-00233185.F0 - HWY 118.GPJ ONTARIO MTO.GDT 7/27/17

Brampton, Ontario

RECORD OF BOREHOLE No BH201

3 OF 3

METRIC

W.P. 5140-13-00 LOCATION Highway 118, MTM-10, 5461901.9N, 259513.9E ORIGINATED BY DP
 DIST Haliburton County HWY 118 TEST PIT TYPE CME 75/HSA/NW/NQ COMPILED BY NT
 DATUM BM Elev. 343.96 m DATE 2016.12.06 - 2016.12.07 LATITUDE 49.29344 LONGITUDE -81.62263 CHECKED BY SM

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) 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			
	Borehole terminated @ ~16.28 m NOTES: 1. This drawing is to be read with the subject report and project numbers as presented above. 2. Since washboring technique was used to advance borehole groundwater level in open hole was not measured															

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

Brampton, Ontario

1 OF 1

METRIC

W. P.	5140-13-00	LOCATION	Hwy 118, Township of stanhope	ORIGINATED BY	DP
DIST	Haliburton County	BOREHOLE TYPE	Kobelco-210 excavator	COMPILED BY	NT
DATUM		DATE	2017/11/05 - 2017/11/05	CHECKED BY	SM

[illegible]

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

MTO TEST PIT TEMPLATE TEST PITS.GPJ ONTARIO MOT.GDT 7/28/17

Brampton, Ontario

RECORD OF TEST PIT No TP2

1 OF 1

METRIC

W. P. 5140-13-00 LOCATION Hwy 118, Township of stanhope ORIGINATED BY DP
 DIST Haliburton County BOREHOLE TYPE Kobelco-210 excavator COMPILED BY NT
 DATUM DATE 2017/11/05 - 2017/11/05 CHECKED BY SM

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	20	40	60	80	100	W _p	W		
0.0	FILL: SAND AND GRAVEL 150 mm thickness															
0.2	FILL: VARIOUS SIZED FRAGMENTS OF BLASTED ROCK in silty, sandy and gravelly soil matrix															
0.4	END OF TEST PIT NOTES: 1. No groundwater level or cave were observed in this test pit. 2. Test Pit was terminated in boulder layer.															

MTO TEST PIT TEMPLATE TEST PITS.GPJ ONTARIO MOT.GDT 7/28/17

Brampton, Ontario

RECORD OF TEST PIT No TP3

1 OF 1

METRIC

W. P. 5140-13-00 LOCATION Hwy 118, Township of stanhope ORIGINATED BY DP
 DIST Haliburton County BOREHOLE TYPE CAT-311 excavator COMPILED BY NT
 DATUM DATE 2017/11/05 - 2017/11/05 CHECKED BY SM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100	W _p	W	W _L		
0.0	Embankment Slope surface/Elevation varies ROCKFILL: COBBLE AND BOULDER-SIZED BLASTED ROCK with some silty sand and gravel																
1.0	END OF TEST PIT NOTES: 1. No groundwater level or cave were observed in this test pit.																

MTO TEST PIT TEMPLATE TEST PITS.GPJ ONTARIO MOT.GDT 7/28/17

Brampton, Ontario

RECORD OF TEST PIT No TP4

1 OF 1

METRIC

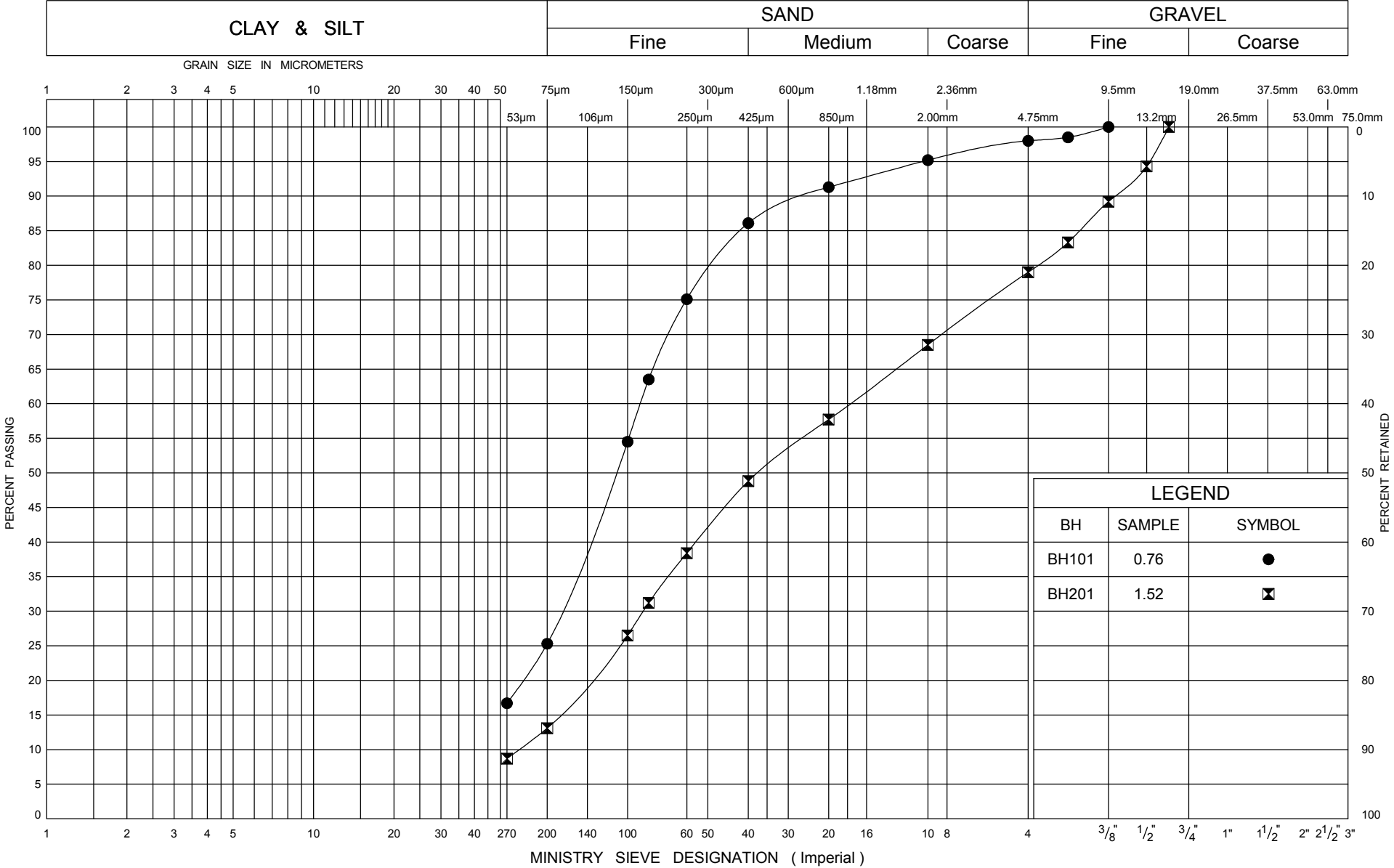
W. P. 5140-13-00 LOCATION Hwy 118, Township of stanhope ORIGINATED BY DP
 DIST Haliburton County BOREHOLE TYPE CAT-311 excavator COMPILED BY NT
 DATUM DATE 2017/11/05 - 2017/11/05 CHECKED BY SM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH: Cu, KPa ○ UNCONFINED + FIELD VANE × QUICK TRIAXIAL LAB VANE					W _p	W	W _L		
0.0	Embankment Slope surface/Elevation varies ROCKFILL: COBBLE AND BOULDER-SIZED BLASTED ROCK with some silty sand and gravel																
1.0	END OF TEST PIT NOTES: 1. No groundwater level or cave were observed in this test pit.																

MTO TEST PIT TEMPLATE TEST PITS.GPJ ONTARIO MOT.GDT 7/28/17

Appendix D – Laboratory Test Results

UNIFIED SOIL CLASSIFICATION SYSTEM



LEGEND		
BH	SAMPLE	SYMBOL
BH101	0.76	●
BH201	1.52	⊠



GRAIN SIZE DISTRIBUTION

FIG No 1
W P5140-13-00
Supplementary Foundation Investigat

Appendix E – Rockfill and Bedrock Core Photographs

Project NO: ADM 0002331875-F0
BH NO: 201
Run NO: 1,2,3,4 & 5 (**Rockfill**)
Sample Depth: 2.67 m to 9.27 m
Elevation: 348.4 m to 341.83 m
Date: December 6 & 7, 2016



Photo 1. Rockfill Core Sample for BH201 from Elevation 348.4 m to 341.83 m

Project NO: ADM 0002331875-F0
BH NO: 201
Run NO: 6,7 & 8 (**Rockfill**)
Sample Depth: 9.27 m to 12.73
Elevation: 341.83 m to 338.4 m
Date: December 6 & 7, 2016



Photo 2. Rockfill Core Sample for BH201 from Elevation 341.83 m to 338.4 m

Project NO: ADM 0002331875-F0
BH NO: 201
Run NO: 1 & 2 (**Bedrock**)
Sample Depth: 13.5 m to 16.3 m
Elevation: 337.6 m to 334.8 m
Date: December 7, 2016



Photo 3. Bedrock Core Sample for BH201 from Elevation 337.6 m to 334.8 m

Project NO: ADM 0002331875-F0
BH NO: 201
Run NO: 1 & 2 (**Bedrock**)
Sample Depth: 13.5 m to 16.3 m
Elevation: 337.6 m to 334.8 m
Date: December 7, 2016

Appendix F – Borehole Logs and Laboratory Test Results from the 2016 Investigation

Brampton, Ontario

RECORD OF BOREHOLE No BH1

1 OF 2

METRIC

W. P. GWP 5140-13-00 LOCATION MTM ZONE10 N4989983 E366930 ORIGINATED BY CS
 DIST Hwy 118, Township of Stanhope BOREHOLE TYPE CME-75, Hollow Stem Auger/ NW/ NQ COMPILED BY JH
 DATUM BM Elev. 343.96 m DATE 2016/03/15 - 2016/03/17 CHECKED BY SM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100					
350.6	Ground Surface																
350.5	ASPHALT 150 mm thickness																
0.2	FILL: SANDY GRAVEL TO GRAVELLY SAND brown, grey and black, moist, compact		1	SS	24		350										
	-becoming few gravel, brown		2	SS	16												
							349										
348.5	ROCK FILL: VARIETY-SIZED FRAGMENTS OF BLASTED ROCK with some sand, some gravel, pink and grey, moist, loose to compact		3	SS	21		348										
2.1			4	SS	9												
			5	SS	26		347										
			6	SS	9		346										
	-boulder cored sample @ 5.3 m, run length = 0.76 m, granite core (~150 mm)		7	SS	6		345										
							344										
	-boulder cored sample @ 7 m, run length = 0.61 m		8	SS	16		343										
342.4	ROCK FILL: COBBLE AND BOULDER-SIZE BLASTED ROCK pink and grey		9	NQ			342										
8.2	NQ CORING Core sample Length (m) Run 1 @ 8.2 m 0.3 Run 2 @ 9.15 m 0.61 Run 3 @ 9.76 m 0.91 Run 4 @ 11.58 m 0.61 Run 5 @ 12.2 m 0.91 -soft layer @ 9.6 m		10	NQ			341										
			11	NQ			340										
			12	NQ			339										

Continued Next Page

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

EXP RECORD OF BOREHOLE 5013-E-0008 ASSIG. 13 BH LOGS, UPDATED - FINAL.GPJ ONTARIO MOT.GDT 8/5/16

Brampton, Ontario

RECORD OF BOREHOLE No BH1

2 OF 2

METRIC

W. P. GWP 5140-13-00 LOCATION MTM ZONE10 N4989983 E366930 ORIGINATED BY CS
 DIST Hwy 118, Township of Stanhope BOREHOLE TYPE CME-75, Hollow Stem Auger/ NW/ NQ COMPILED BY JH
 DATUM BM Elev. 343.96 m DATE 2016/03/15 - 2016/03/17 CHECKED BY SM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)						
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH: Cu, KPa									WATER CONTENT (%)			GR	SA	SI	CL
								○ UNCONFINED	+	FIELD VANE	×	QUICK TRIAXIAL					LAB VANE	20	40				
336.9	ROCK FILL: COBBLE AND BOULDER-SIZE BLASTED ROCK pink and grey NQ CORING (continued) -soft layer @ 13.1 m		13	NQ			338																
13.7	GRAVELLY SAND organic odour, occasional cobbles and boulders, brown, wet to saturated		14	SS	77/ 150mm		337							○				27	65 (8)				
336.0	BEDROCK pink streaks on grey and white seams of pyrite and mica, granite NQ CORING Length (m) RQD(%) Run 6 1.15 Run 7 1.32 51 Run 8 1.32 67		15	NQ			336																
14.6			16	NQ			335																
			17	NQ			334																
332.2	END OF BOREHOLE						333																
18.4	NOTES: 1. This drawing is to be read with the subject report and project numbers as presented above. 2. No groundwater level was measured. Washboring technique was used to drill borehole. 3. The reltve density does not apply to rockfill, however, for information purpose the relative density is provided (which is possibly not representative of the layer) based on obtained SPT "N" values wherever possible.																						

EXP RECORD OF BOREHOLE 5013-E-0008 ASSIG. 13 BH LOGS_UPDATED - FINAL.GPJ ONTARIO MOT.GDT 8/5/16

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

Brampton, Ontario

RECORD OF BOREHOLE No BH2

1 OF 2

METRIC

W. P. GWP 5140-13-00 LOCATION MTM ZONE10 N4989983 E366938 ORIGINATED BY CS
 DIST Hwy 118, Township of Stanhope BOREHOLE TYPE CME-75, Hollow Stem Auger/ NW/ NQ COMPILED BY JH
 DATUM BM Elev. 343.96 m DATE 2016/03/18 - 2016/03/19 CHECKED BY SM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100					
351.2	Ground Surface																
351.1	ASPHALT 150 mm thickness																
0.2	FILL: GRAVELLY SAND grey seams, fine to medium sand, brown, moist, dense		1	SS	30		351										24 68 (8)
			2	SS	39		350										
349.7	FILL: VARIETY-SIZED FRAGMENTS OF BLASTED ROCK with some sand and gravel, grey with pink seams, very loose to dense		3	SS	15		349										
1.5			4	SS	18		348										49 43 (8)
			5	SS	34		347										
	-boulder @ 3.66 m, no sample taken		6	SS	20		346										
			7	SS	5		345										
							344										
			8	SS	17		343										
							342										
			9	SS	8		341										
							340										
	-boulder @ 6.1 m, tricone up to 7.6 m, no sample taken		10	SS	3												

Continued Next Page

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

EXP RECORD OF BOREHOLE 5013-E-0008 ASSIG. 13 BH LOGS_UPDATED - FINAL.GPJ ONTARIO MOT.GDT 8/5/16





Brampton, Ontario

RECORD OF BOREHOLE No BH2

2 OF 2

METRIC

W. P. GWP 5140-13-00 LOCATION MTM ZONE10 N4989983 E366938 ORIGINATED BY CS
 DIST Hwy 118, Township of Stanhope BOREHOLE TYPE CME-75, Hollow Stem Auger/ NW/ NQ COMPILED BY JH
 DATUM BM Elev. 343.96 m DATE 2016/03/18 - 2016/03/19 CHECKED BY SM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100					
339.0							339										
12.2	SAND AND GRAVEL coarse angular and subangular sand and gravel, reddish brown, dense		11	SS	30												48 45 (7)
338.5																	
12.7	PEAT saturated, wood fragments, strong earthy odour, trace sand																
337.9							338										
13.3	SANDY SILT fine, trace organics (rootlets), few gravel, trace clay, some pyrite flecks, very dense																
337.2			12	SS	50/100 mm												6 37 53 4
14.0	BEDROCK black and grey granite with white and pink seams and striations, occasional pyrite and mica seams NQ CORING Length (m) RQD(%) Run1 1.65 83.2		13	NQ			337										
							336										
335.5																	
15.7	END OF BOREHOLE NOTES: 1. This drawing is to be read with the subject report and project numbers as presented above. 2. No groundwater level was measured. Washboring technique was used to drill borehole. 3. The relative density does not apply to rockfill, however, for information purpose the relative density is provided (which is possibly not representative of the layer) based on obtained SPT "N" values wherever possible.																

EXP RECORD OF BOREHOLE 5013-E-0008 ASSIG. 13 BH LOGS_UPDATED - FINAL.GPJ ONTARIO MOT.GDT 8/5/16

Brampton, Ontario

RECORD OF BOREHOLE No BH3

1 OF 1

METRIC

W. P. GWP 5140-13-00 LOCATION MTM ZONE10 N4998997 E366954 ORIGINATED BY CS
 DIST Hwy 118, Township of Stanhope BOREHOLE TYPE HUSQVARNA DS 800 Portable Hydraulic Drill/ NW/ BQ COMPILED BY JH
 DATUM BM Elev. 343.96 m DATE 2016/03/18 - 2016/03/18 CHECKED BY SM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100					
341.0	Ground Surface																
	PEAT with roots and rootlets, black, wet, very soft																
340.4																	
0.6	BOULDERS AND COBBLES trace sand, trace silt and trace organics		1	SS	50/127 mm												
	NQ CORING		2	NQ													
	Length (m)																
339.5	Run 1 0.15		3	NQ													
	Run 2 0.46																
339.5	BEDROCK grey and white granite		4	NQ													
1.5	NQ CORING		5	NQ													
339.2	Length (m)																
1.8	Run 3 0.20																
	Run 4 0.13																
	END OF BOREHOLE																
	NOTES: 1. This drawing is to be read with the subject report and project numbers as presented above. 2. No groundwater level was measured. Washboring technique was used to drill borehole.																

EXP RECORD OF BOREHOLE 5013-E-0008 ASSIG. 13 BH LOGS_UPDATED - FINAL.GPJ ONTARIO MOT.GDT 8/5/16

Brampton, Ontario

RECORD OF BOREHOLE No BH4

1 OF 1

METRIC

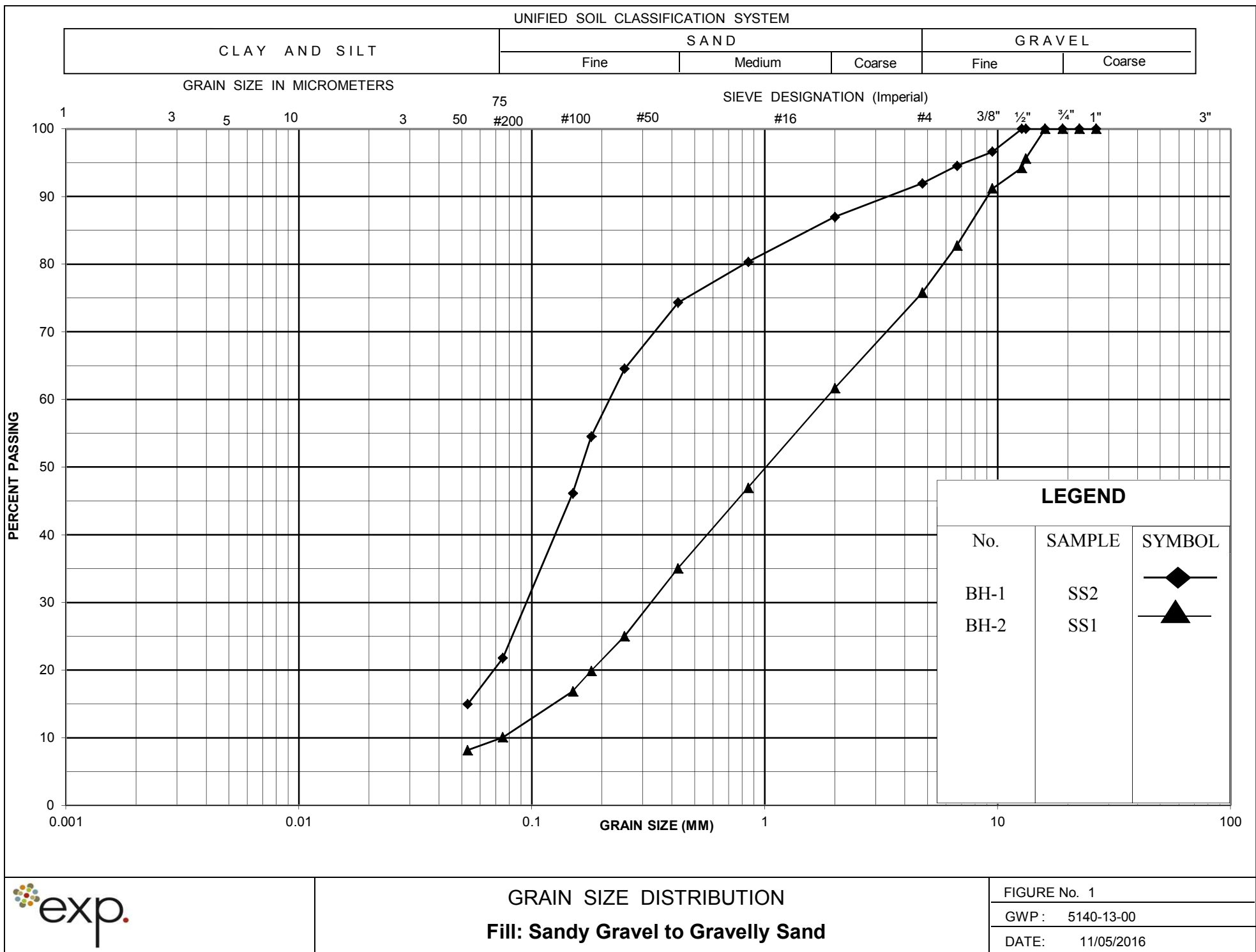
W. P. GWP 5140-13-00 LOCATION MTM ZONE10 N4998969 E366912 ORIGINATED BY CS
 DIST Hwy 118, Township of Stanhope BOREHOLE TYPE HUSQVARNA DS 800 Portable Hydraulic Drill/ NW/ BQ COMPILED BY JH
 DATUM BM Elev. 343.96 m DATE 2016/03/19 - 2016/03/19 CHECKED BY SM

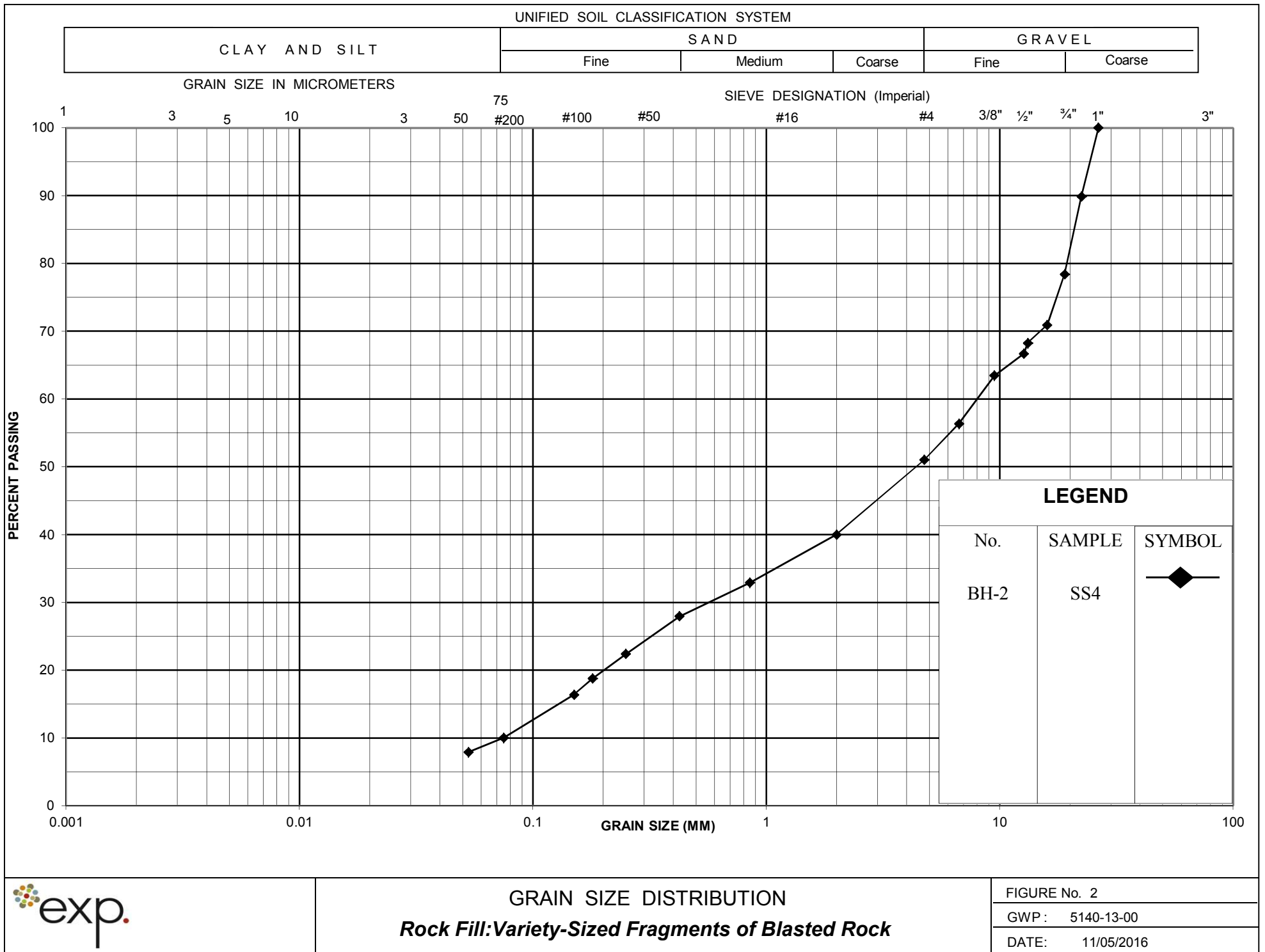
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100					
338.3	Ground Surface																
338.2	PEAT with rootlets, black, moist		1	SS	50/		338										
0.2	BEDROCK grey and white granite		2	NQ	150mm												
	NQ CORING																
	Length (m) RQD(%)																
	Run 1 0.26 60.0		3	NQ													
	Run 2 0.45 77.7																
	Run 3 0.41 62.5		4	NQ													
	Run 4 0.51 70.0																
	Run 5 0.40 100.0		5	NQ			337										
336.1			6	NQ													
2.2	END OF BOREHOLE																
	NOTES: 1. This drawing is to be read with the subject report and project numbers as presented above. 2. No groundwater level was measured. Washboring technique was used to drill borehole.																

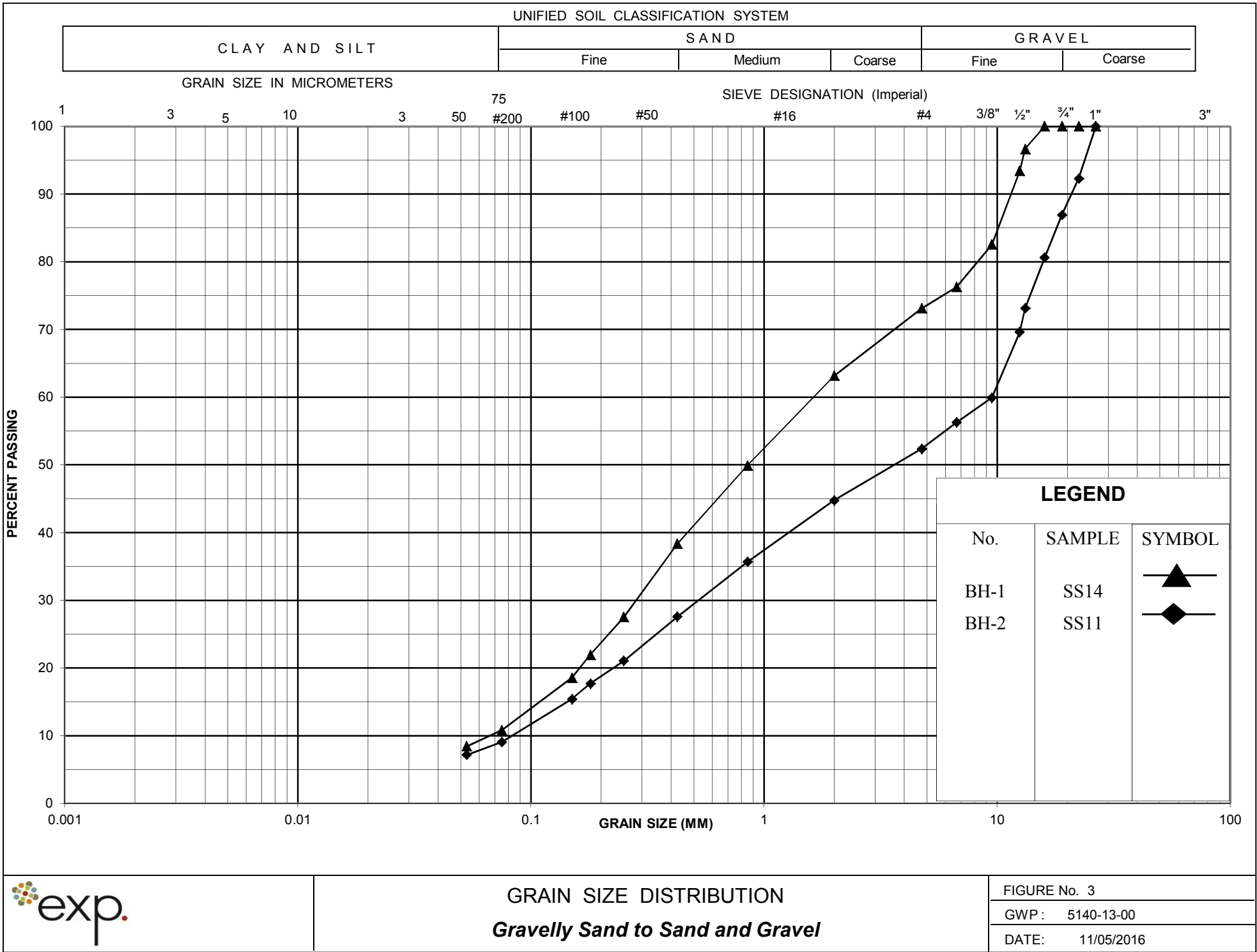
EXP RECORD OF BOREHOLE 5013-E-0008 ASSIG. 13 BH LOGS_UPDATED - FINAL.GPJ ONTARIO MOT.GDT 8/5/16

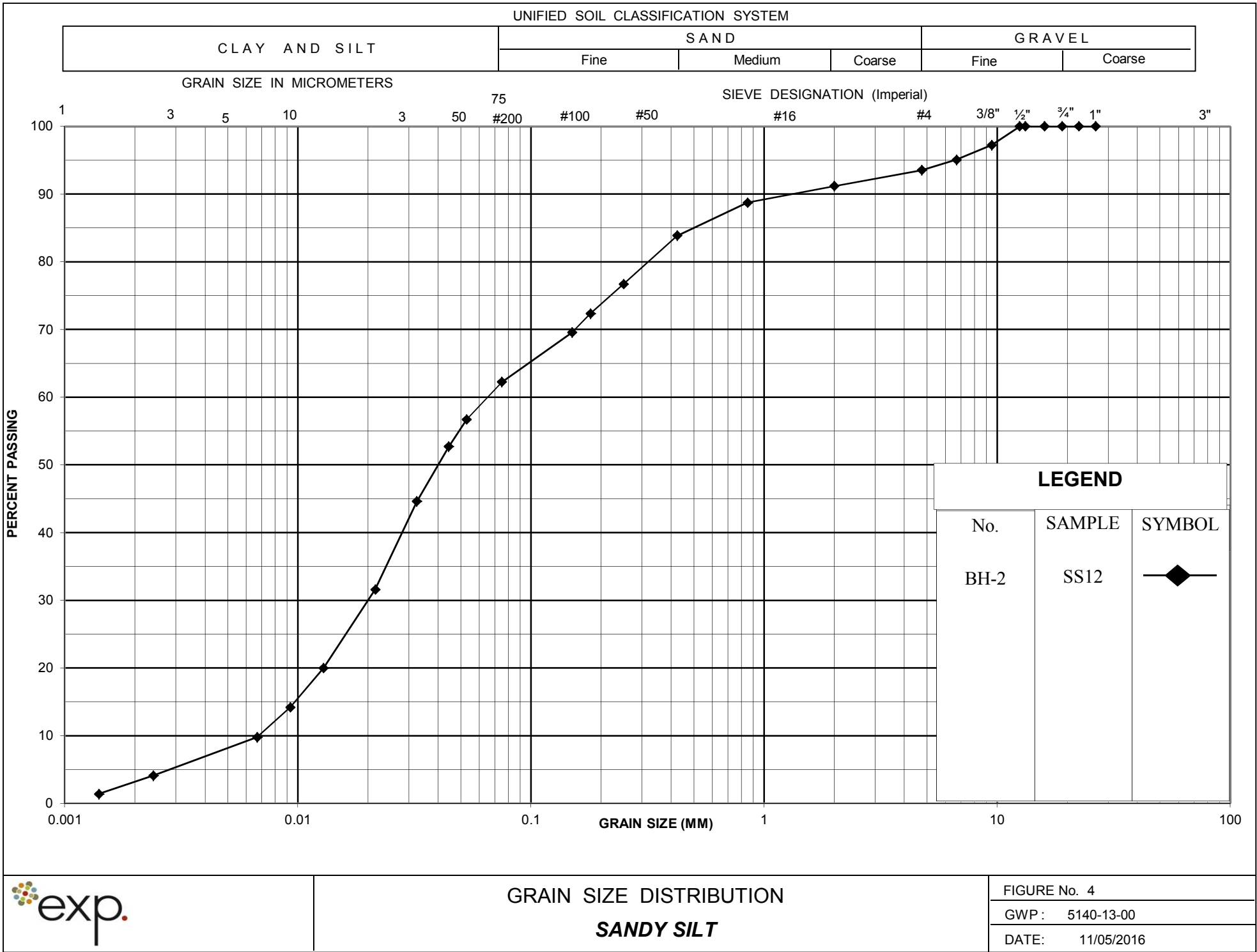
+ 3, X 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

Previous Laboratory Test Results
exp 2016











exp Services Inc.
1595 Clark Boulevard
Brampton, Ontario, L6T 4V1
Tel: (905) 793-9800
Fax: (905) 793-0641
www.exp.com

Uniaxial Compressive Strength of Rock Cores

Project No.: ADM-00282450-P0 ADM-100

Project Name: 5013-E0008 Assignment #13

Date: April 19, 2016

Sample No.	BH1 UCT1	BH1 UCT2	BH1 UCT3
Location	23' – 25'	37'	38' – 40'
Date Received	April 19, 2016		
Date Tested	April 19, 2016		
Height – [mm]	86.0	115.0	117.0
Average Diameter – [mm]	47.0	47.0	47.0
Area [mm ²]	1734.9	1734.9	1734.9
L/D Ratio	1.83*	2.45	2.49
Failure Load [kN]	163.22	174.89	96.68
Compressive Strength – [MPa]	94.1	100.8	55.7
Remarks			

ASTM D4543, ASTM D2938

L/D Ratio: 2.0- 2.5

Minimum Diameter: 47.0 mm

*L/D was less than the minimum requirements of 2.0

Testing Laboratory Representative Signature
Ammanuel Yousif, C.E.T.

April 19, 2016

Date



CERCHAR Abrasivity test of three rock cores

Final Report

May 4, 2016

Prepared by: Xin Wang, Research Engineer/Post-Doctoral Fellow
Geomechanics Research Centre (GRC), MIRARCO

Reviewed by: Sean Maloney, VP Operations, GRC Director
Geomechanics Research Centre (GRC), MIRARCO

Prepared for: Silvana Micic (PhD, PEng), Senior Geotechnical Engineer
exp Services Inc.



1 Introduction

MIRARCO's Geomechanics Research Centre was contracted by Silvana Micic of exp Services Inc. to undertake a series of abrasivity tests on three select gneiss samples designated AT1, AT2, and AT3. These were collected from depths of 17.5'-20', 32'-35', and 38'-40' respectively and delivered to MIRARCO for testing. The abrasivity tests were conducted in accordance with ASTM Standard D7265-10 "Standard Test Method for Laboratory Determination of Abrasiveness of Rock Using the CERCHAR Method".

2 Background

Rock abrasivity is a characteristic of significance in estimating wear on mechanical excavation equipment such as core bits and disc cutters. While a number of tests have been proposed, the most widely accepted remains the CERCHAR scratch test (West 1989; Plinninger et al., 2003). In this test, a conical steel point of cone angle 90° is slowly drawn 10 mm across the rock surface under a normal, static force of 70 N. A drawing of the test device is presented in Figure 1. The abrasivity is then determined by the wear flat of the steel cone; units of measurement correspond to the diameter of the wear flat in tenths of a millimeter (e.g., a 0.3 mm diameter wear flat yields a measurement of 3). It is generally recommended that more than one measurement be made and the CERCHAR Abrasivity Index, *CAI*, be taken as the mean value.

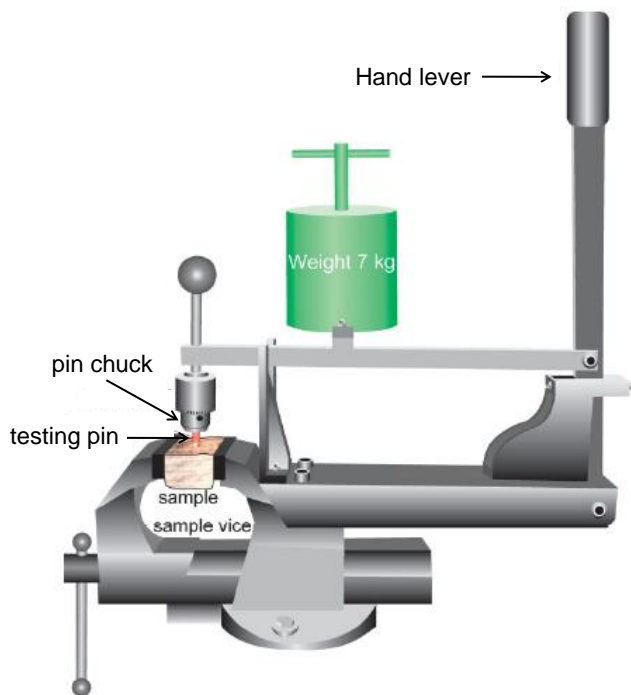


Figure 1 - CERCHAR test apparatus (after Plinninger and Restner, 2008)



Studies conducted by [Plinninger et al. \(2003\)](#) on the influence of surface conditions showed that CAI values obtained from 'rough' surfaces were about 0.5 higher than those from smooth surfaces. The authors recommended that for rock samples that have unsuitable sample surfaces after breaking, a diamond saw be used for surface formatting. The test result can be corrected according to the following equation:

$$CAI = 0.99CAI_s + 0.48$$

where CAI_s represents the index obtained from the smooth surface.

3 Methodology

3.1 Specimen preparation

All the testing processes were undertaken sequentially on each individual specimen to minimize the time of exposure prior to testing. In order to obtain a suitable fresh surface for testing, the samples were wet cut using a diamond saw in MIRARCO'S Laboratory. As shown in [Figure 2](#), three specimens (marked as AT1, AT2, and AT3) were prepared from the supplied core.

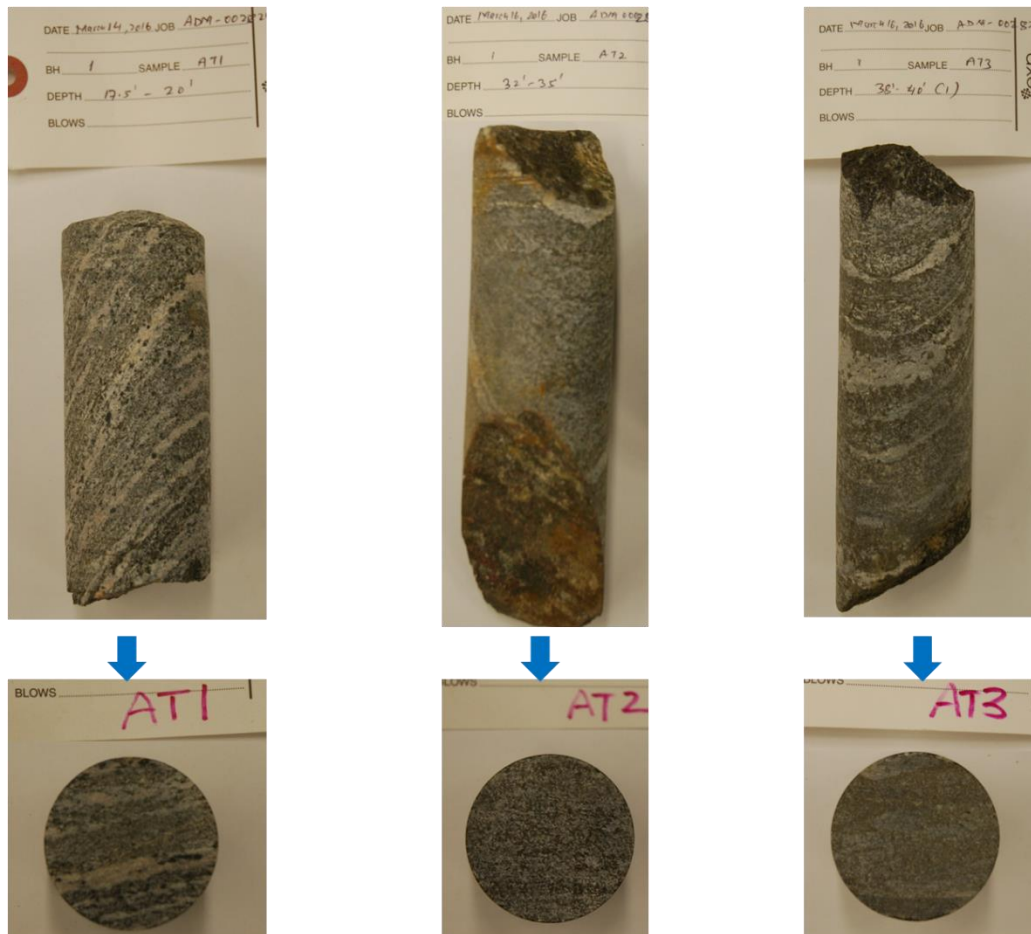


Figure 2- The supplied core and three test specimens after cutting



3.2 Testing

According to the ASTM Standard D7265-10, each specimen was clamped in position with the test surface horizontal. A new pin was placed in the chuck and carefully brought to bear against the surface under the prescribed load of 70N. Because there is a pronounced fabric existing in the specimens, most of the scratches were made with the fabric oriented perpendicular to draw direction. The pin was then drawn across the surface for a distance of 10 mm. It was then removed for inspection. Then, the specimen was repositioned and the test was repeated two more times for each specimen (the pins were examined after each test). If the first three scratches on a sample yielded relatively consistent results, the following two scratches were performed perpendicular to the first three to investigate the possibility of a directional variability in the abrasiveness. Photographs of the three specimens following testing are presented in [Figure 3](#).

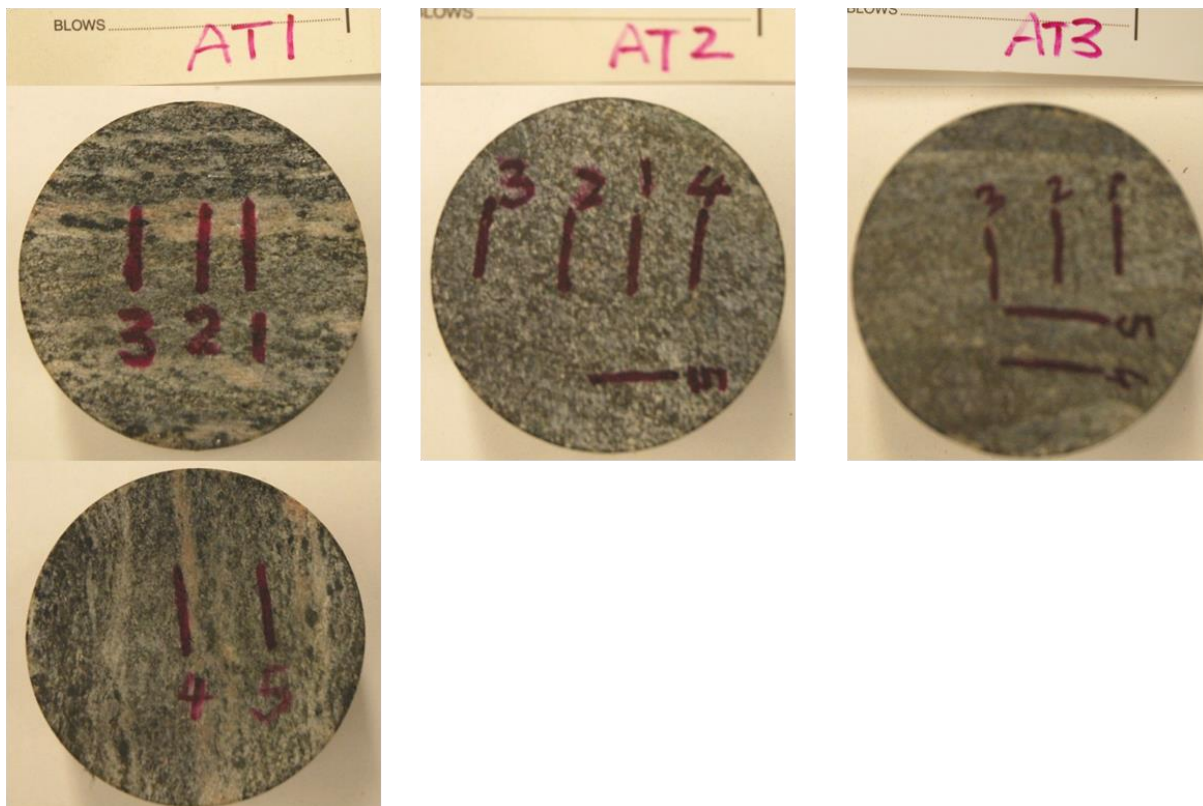


Figure 3-Specimens AT1, AT2, and AT3 after testing (scratches have been highlighted by marker pen)

The wear flat of each pin is measured before and after testing using a Wild M38 binocular microscope with a measuring ocular at 40X magnification. All pins and the test surface were also photographed through a 6.0X magnification microscope for archival purposes before and after testing (see [Appendix](#)).

4 Results

The abrasivity values determined from the testing of the three specimens are shown in [Table 1](#). Individual CAI values ranged from 4.22 to 4.34 for specimen AT1; a quite consistent result. For



specimens AT2 and AT3, the *CAI* values were only slightly more variable, ranging from 4.22 to 4.83 and from 4.10 to 4.84, respectively. The average *CAI* values for specimens AT1, AT2 and AT3 are 4.32, 4.56 and 4.49, respectively. The overall average *CAI* value of the three specimens is 4.46.

Table 1: Abrasivity test results

Sample ID	Trial#	Wear Flat (mm)	<i>CAI_s</i>	<i>CAI</i>
AT1	1	0.39	3.90	4.34
	2	0.39	3.90	4.34
	3	0.38	3.80	4.22
	4	0.39	3.90	4.34
	5	0.39	3.90	4.34
	Mean.		3.88	4.32
	Standard Error.		0.055	0.054
AT2	1	0.39	3.90	4.34
	2	0.38	3.80	4.22
	3	0.44	4.40	4.83
	4	0.41	4.14	4.58
	5	0.44	4.41	4.83
	Mean.		4.12	4.56
	Standard Error.		0.278	0.275
AT3	1	0.41	4.15	4.58
	2	0.37	3.66	4.10
	3	0.39	3.90	4.34
	4	0.44	4.41	4.84
	5	0.41	4.14	4.58
	Mean.		4.05	4.49
	Standard Error.		0.281	0.278
Average <i>CAI</i>	Mean	4.46	Standard Error	0.124

The results obtained are consistent with published data for similar rock types as demonstrated in [Figure 4](#). According to the criteria established by [CERCHAR \(1986\)](#) (see [Table 2](#)), the three specimens would be classified as extremely abrasive. Using the more recent classification proposed by [Restner \(2007\)](#) (see [Table 3](#)) the rock would be considered as highly abrasive for the average *CAI* value (4.46) whereas Specimen AT2 itself would be classified as extremely abrasive.

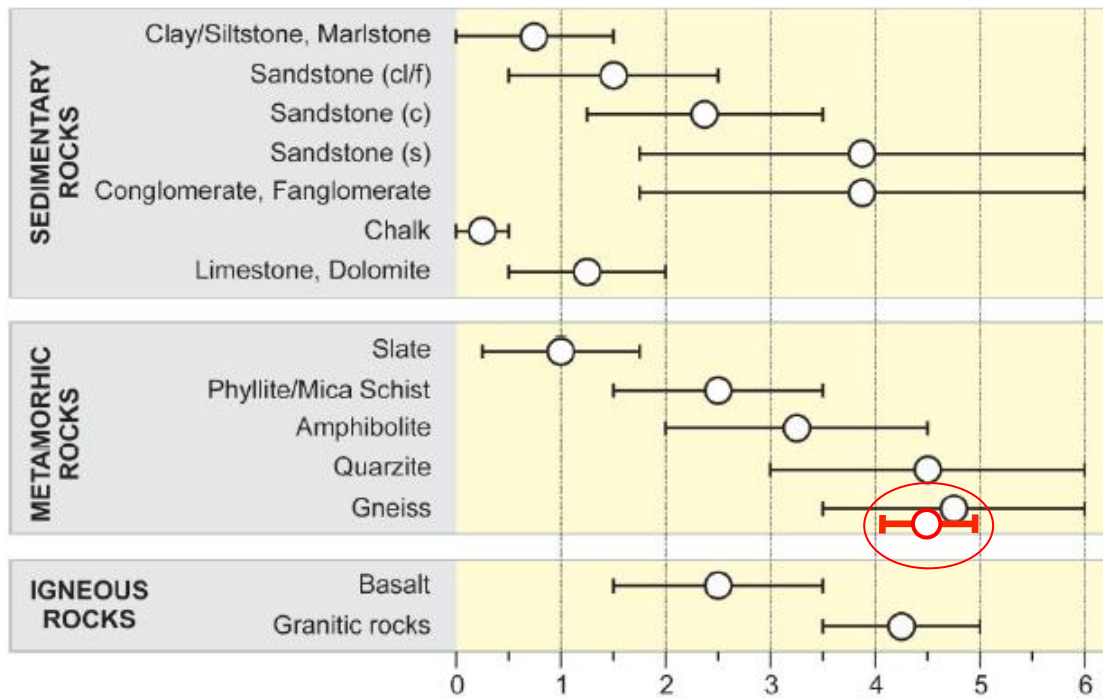


Figure 4- Comparison of test results (red) with compilation of typical CAI values (Plinninger and Restner, 2008)

Table 2: Classification of rock abrasiveness (CERCHAR, 1986)

CAI	Classification
0.3 – 0.5	not very abrasive
0.5 – 1.0	slightly abrasive
1.0 – 2.0	medium abrasive
2.0 – 4.0	very abrasive
4.0 – 6.0	extremely abrasive

Table 3: Modified classification of rock abrasiveness (Restner, 2007)

CAI	Classification
< 0.5	not abrasive
0.5 - 1.0	little abrasive
1.0 – 1.3	moderately abrasive
1.3 – 1.8	considerably abrasive
1.8 – 2.3	abrasive
2.3 – 3.0	very abrasive
3.0 – 4.5	highly abrasive
> 4.5	extremely abrasive



More recently, [Alber \(2008\)](#) showed that the CERCHAR Abrasivity Index is stress-dependent, i.e., the more the rock is confined the higher the CAI will be. This suggests that in situ CAI values are higher than unconfined lab values and correspondingly, more wear can be expected in the field. Note that this is particularly applicable to TBMs where various cutters are acting in different stress environments.

References

Alber, M., 2008. Stress dependency of the CERCHAR abrasivity index (CAI) and its effects on wear of selected rock cutting tools. *Tunneling & Underground Space Technology*, **23**: 351-359.

CERCHAR – Centre d'Études et Recherches de Charbonnage de France, 1986. The CERCHAR Abrasiveness Index. Verneuil, 12p.

Plinninger, R. and Restner, U. 2008. Abrasiveness testing, Quo Vadis? – A commented overview of abrasiveness testing methods. *Geomechanik and Tunnelbau*, Heft 1, 61-70.

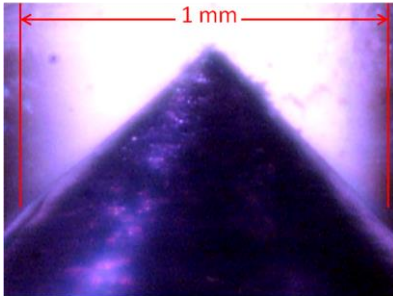
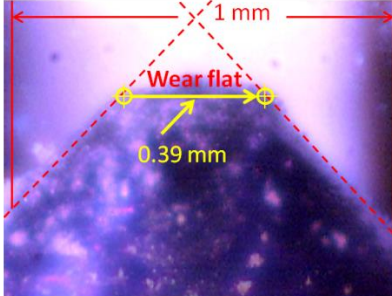
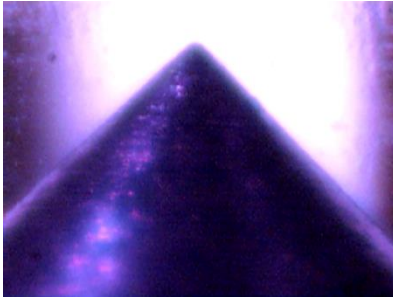
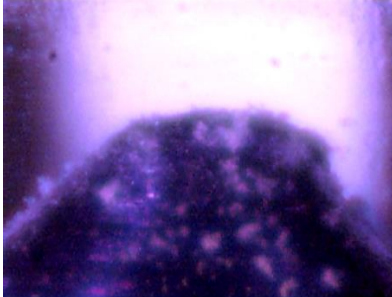
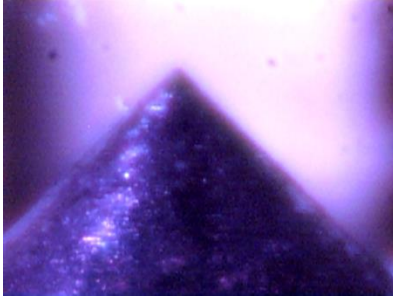
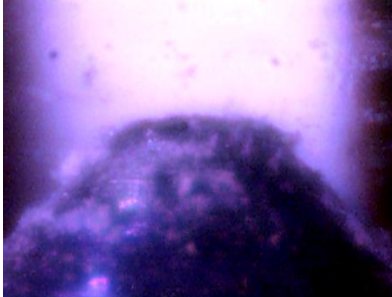
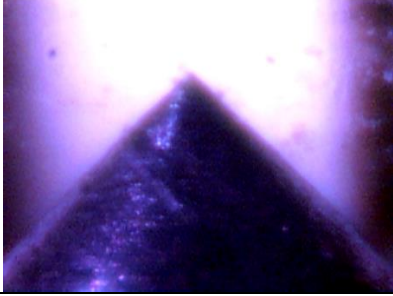
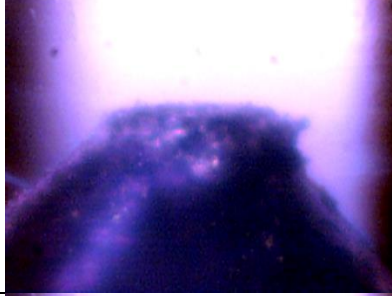
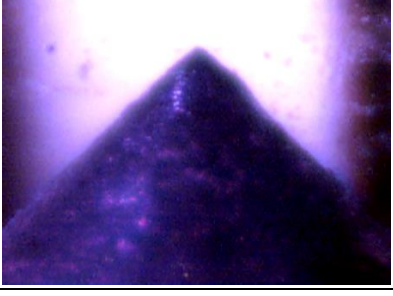
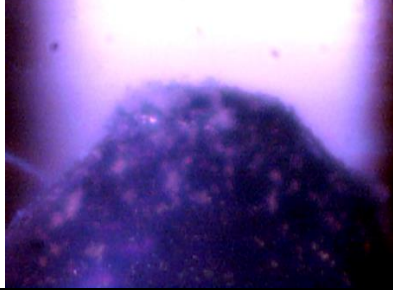
Plinninger, R., Kasling, H., Thuro, K. and Spaun, G., 2003. Technical note – Testing conditions and geomechanical properties influencing the CERCHAR abrasiveness index (CAI) value. *International Journal of Rock Mechanics & Mining Sciences*, **40**(2): 259-263.

Rester, U., 2007. Sandvik Mining and Construction's rock testing standards, Sandvik Mining and Construction GmbH., Department of Geotechnical Consulting & Engineering, Zeltweg.

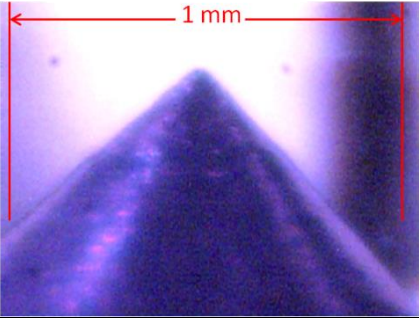
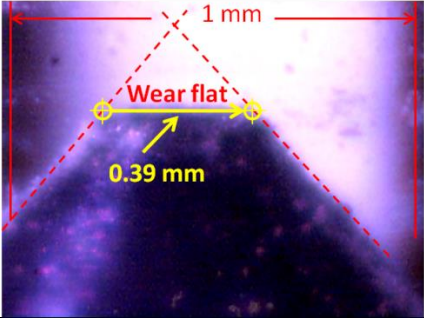
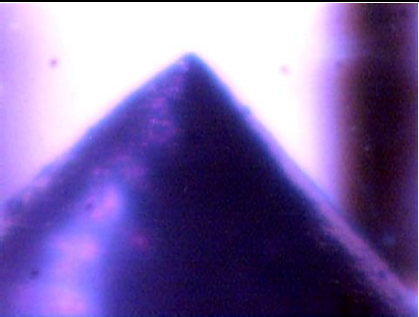
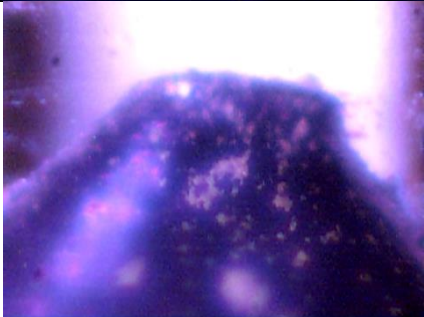
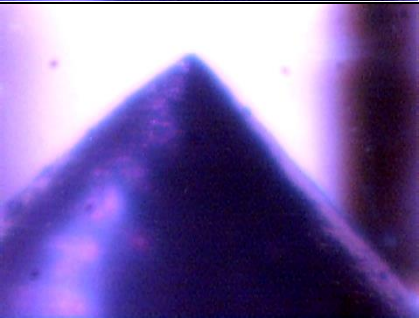
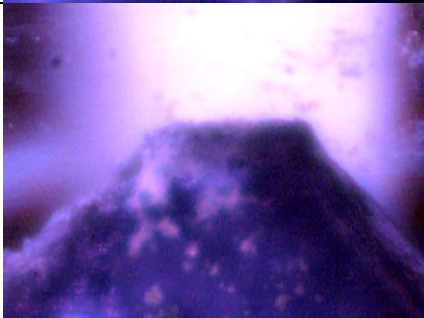
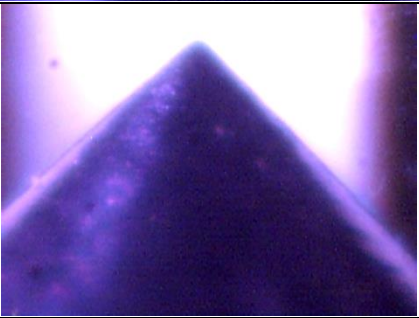
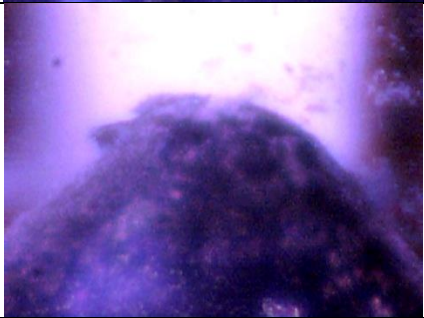
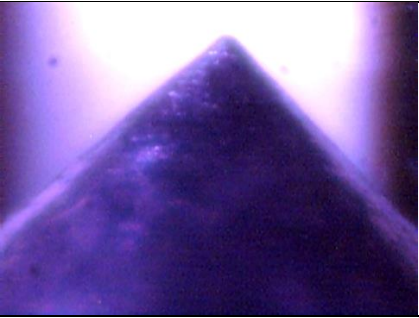
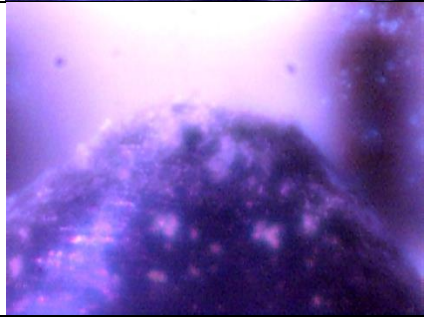
West, G., 1989. Technical note - Rock abrasiveness testing for tunneling. *International Journal of Rock Mechanics, Mining Sciences & Geomechanics Abstracts*, **26**(2): 151-160.



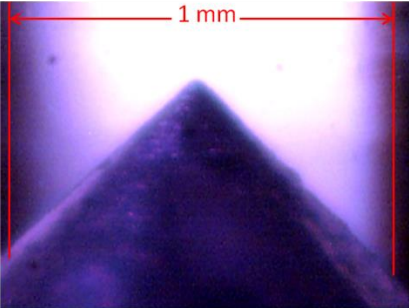
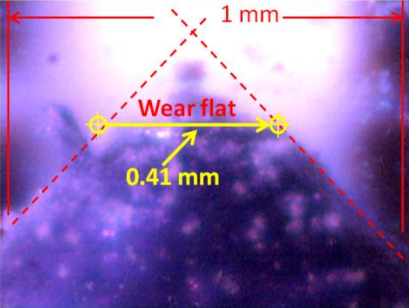
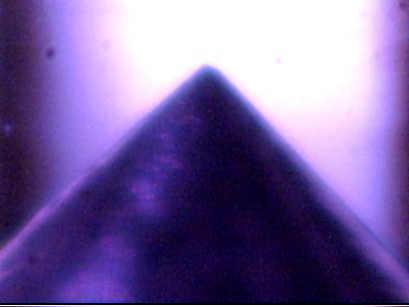
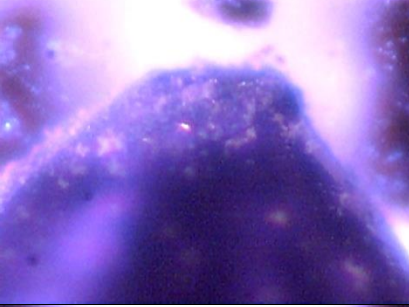
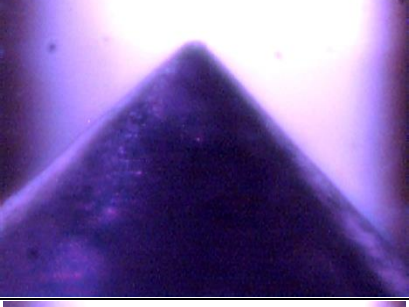
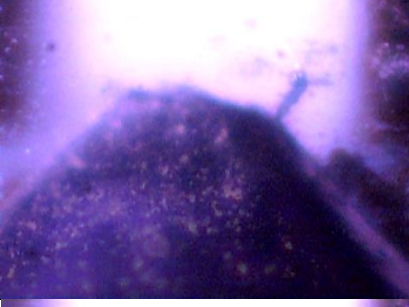
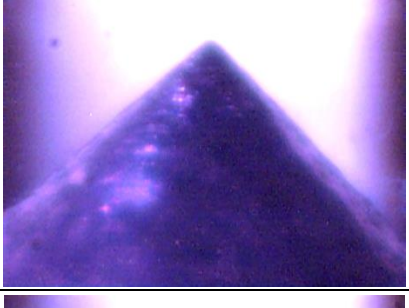
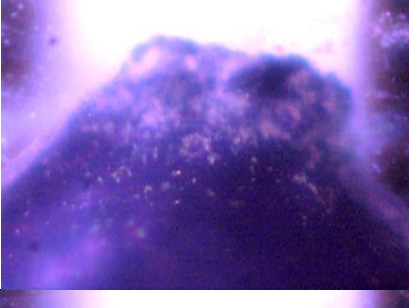
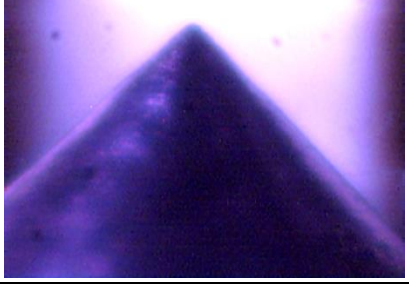
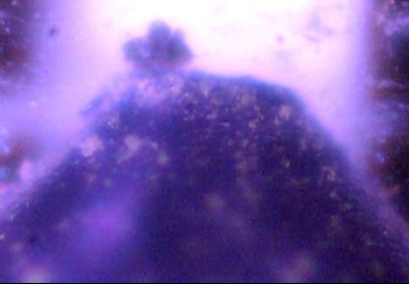
Appendix : Photographs of pins before and after testing

Specimen AT1		
Trail#	Before Test	After Test
AT1 - 1		
AT1 - 2		
AT1 - 3		
AT1 - 4		
AT1 - 5		

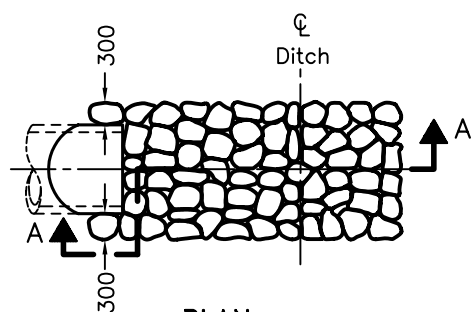


Specimen AT2		
Trail#	Before Test	After Test
AT2 - 1		
AT2 - 2		
AT2 - 3		
AT2 - 4		
AT2 - 5		

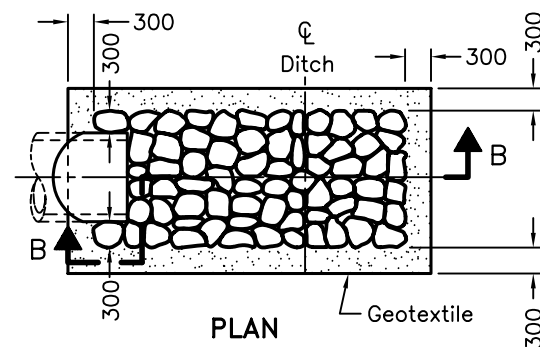
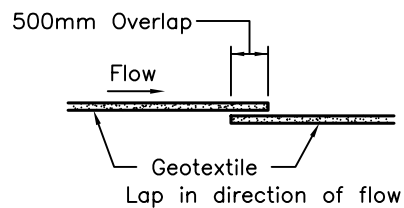


Specimen AT3		
Trail#	Before Test	After Test
AT3 - 1		
AT3 - 2		
AT3 - 3		
AT3 - 4		
AT3 - 5		

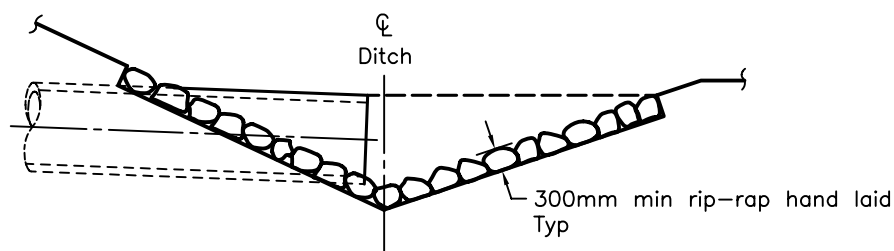
Appendix G – OPSDs



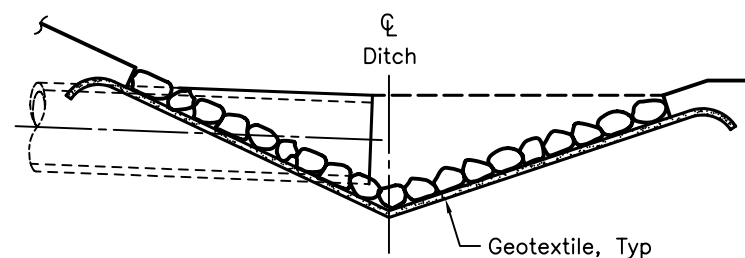
PLAN
CUT OR FILL



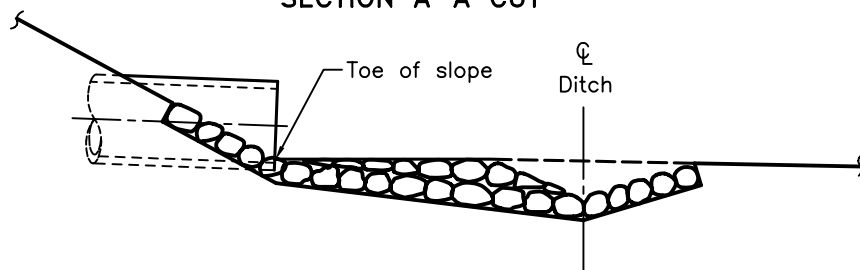
PLAN
CUT OR FILL



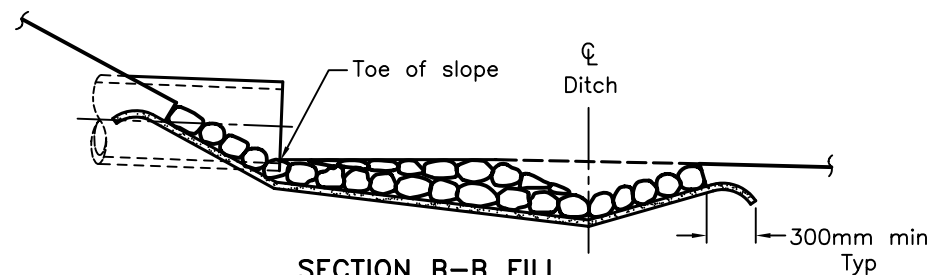
SECTION A-A CUT



SECTION B-B CUT



SECTION A-A FILL
TYPE A – WITHOUT GEOTEXTILE



SECTION B-B FILL
TYPE B – WITH GEOTEXTILE

NOTES:

A All dimensions are in millimetres unless otherwise shown.

ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2001

Rev 0

RIP-RAP TREATMENT
FOR SEWER AND CULVERT OUTLETS



OPSD – 810.010

Appendix H– NSSPs

NSSP FOR CONDITION SURVEYS AND MONITORING DURING ANY BLASTING

Scope of Work

If any blasting is required, the Blast Contractor must be fully qualified and experienced. The Blast Contractor shall outline the procedure and extent of the pre-blast survey. The blast methodology, including drill hole patterns, hole size and depths, size of blast, explosive and initiation product details, as well as all blast control procedures shall be required. Blast control procedures would include details on controlling flyrock, temporary road closures, blast signaling and site clearing procedures. Details on instrumentation, number and location of monitoring sites, blast recording and reporting procedures, and procedures to be followed in the event of excessive vibration readings are required as well.

Instrumentation or monitoring ground and air vibration effects from the blasting should be set up in accordance with the International Society of Explosives Engineering field practice guidelines (1999).

Basis of Payment

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

NSSP FOR MONITORING DURING ACTIVE TUNNEL EXCAVATION

Scope of Work

A geotechnical engineer shall be present during active excavation of the tunnel to verify that the ground conditions are consistent with those encountered in the investigation boreholes. The volume of the material removed from the tunnel shall be monitored and continuously compared to the rate of tunnel advance.

The geotechnical engineer shall be a firm registered in RAQS for tunneling under medium complexity.

Basis of Payment

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