



FINAL REPORT

FOUNDATION INVESTIGATION AND DESIGN REPORT Culvert Installation, Highway 118, Township of Stanhope, Ontario

**Agreement No. 5013-E-0008
Assignment No. 13
GWP 5140-13-00
Geocres No. 31E-364**

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Foundation Investigation and Design Report

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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 # 5013-E-0008, Assignment No. 13. The terms of reference (TOR) were as presented in the MTO letter dated January 28, 2016.

The purpose of the investigation is to determine the subsurface conditions along the proposed culvert replacement alignment and to provide recommendations on the feasibility of and method for replacing the culvert. The site specific geotechnical investigation consisted of borings, soil sampling, borehole 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 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 and 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 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 the site reconnaissance on March 15, 2016, the general site conditions were assessed. At the site location water flows from east to west, towards Boshkung Lake, crossing Hwy 118 via culvert. At the time of investigation, surficial flow of water through culvert was observed to be approximately at Elevation 341.1

m to 338.0 m at the inlet and outlet sides, respectively. The elevation of highway pavement at proposed culvert alignment is 350.9 m.

The vicinity of inlet and outlet of the culvert is heavily vegetated with trees. The slopes of the embankment were covered by rock fill of boulders (see Photos 6, 7, 8 and 10, in Appendix A). Bedrock outcrops were observed in the vicinity of site and stream bed. The terrain, in general, is the sloping bedrock towards lake (on west side of Hwy). At site location, the bedrock sloping towards the inlet forming valley and further slope towards lake (on west) at 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

The field investigation was performed on March 14 to 19, 2016. The field program consisted of drilling four (4) sampled boreholes (BH1 to BH4). The boreholes were strategically located along the proposed culvert alignment to provide subsurface information for the design of the proposed new culvert. BH1 and BH2 were advanced from the embankment crest within the travelled road, and these boreholes (BH1 and BH2) were located about 2 m north and south of the proposed culvert centerline adjacent to the pavement edge line of EBL and WBL of the Hwy, respectively. BH3 and BH4 were advanced off the road at inlet and outlet locations of the proposed culvert, respectively.

Boreholes drilled from the embankment crest (BH1 and BH2) were advanced using a truck mounted CME-75 drill rig. The drill rig was equipped with a hollow stem auger, tri-cone and standard soil/ rock sampling equipment operated by a specialist drilling contractor, 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), it is anticipated that only particles smaller than 35 mm in diameter will be able to be collected in the sampler. The larger particles could be possibly pushed aside during the driving of the sampler.

Due to the high rock fill embankment (approximately 10.2 m to 12.3 m high) with relatively steep side slopes of 1.5H:1V to 1.25H:1V on inlet and outlet side, respectively, and due to the presence of boulders on side slopes it was difficult to access the inlet and outlet sides with the drill rig. So, the boreholes at these locations (BH3 and BH4) were advanced using portable hydraulic drilling equipment, Husqvarna DS

800, and hand sampling with a portable hammer operated also by Marathon Drilling Company Ltd. The borehole locations are shown on Drawing No. 1 in Appendix B.

The boreholes drilled through the embankment BH1 and BH2 were advanced up to depths between 18.4 m and 15.7 m until auger/split spoon refusal or cored up to 3.9 m and 1.7 m, respectively. BH3 and BH4 located at the toes of embankment were advanced to depths between 2.8 m and 2.2 m until split spoon refusal or cored up to 2.2 m and 2.0 m, respectively.

The borehole 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 343.962 m. The location of the boreholes and the TBM are shown on Drawing 1, in Appendix B.

During the drilling of the boreholes, combination of Standard Penetration Tests (SPT) and coring was attempted to obtain the soil and rock samples. A 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. However, in the case of sampling done by the manually lifting portable hammer (31.7 kg, half weight of conventional hammer weight) at BH3 and BH4, the corresponding blow counts were factored by 0.5. 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/auger sampling of hard material was performed by diamond core drilling, using a 1.5 m long NQ double tube wireline core barrel and 0.6 m long BQ core barrel for boreholes drilled through embankment crest and boreholes drilled at embankment toes location, respectively).

Upon completion of the boreholes, ground water level measurements were carried out from the boreholes in accordance with the MTO guidelines. However, boreholes were advanced using wash boring technique, so the stabilized ground water level could not be established by short term observation 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 fieldwork was supervised by members of **exp's** engineering directed the drilling and sampling operation, 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 returned to **exp's** Brampton laboratory for additional visual, textual, olfactory examination and selective testing.

1.3.2 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 the split spoon samples collected from boreholes within the rock fill layer could not recovered adequate samples for testing; the particle size distribution tests for usually required 25% of collected samples was not possible to achieve for this site. In addition, as requested by MTO, the laboratory testing program included uniaxial compression strength tests and abrasivity tests on selected boulder core samples collected within the rock fill. The boulder core samples for testing were selected based on available cores within the proposed elevation of the culvert. All of the laboratory tests were carried out in accordance with MTO and/or ASTM Standards as appropriate. In particular, the abrasivity test was performed using the CERCHAR method (ASTM D7625-10). Since the wash boring technique is 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 representative 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.

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 location plan and stratigraphic section are provided in Appendix B. It should be noted that the stratigraphic boundaries indicated on the borehole 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 locations.

In general, the subsurface conditions along the new culvert location consist of a layer of fill material composed of gravelly sand to blast rockfill/ boulder underlain by native peat, followed by sandy silt/ cobbles and boulder and bedrock. A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

1.4.1 Asphalt

Asphalt was encountered at the surface of boreholes BH1 and BH2. Thickness of the asphalt layer was about 150 mm. Asphalt thicknesses may further vary beyond the borehole location.

1.4.2 Fill: Sandy Gravel to Gravelly Sand

Sandy gravel to gravelly sand fill was encountered below the asphalt in boreholes BH1 and BH2. The sandy gravel to gravelly sand fill extended to depths ranging between 1.5 m to 2.1 m below road surface with elevations ranging between 348.5 m and 349.7 m. The explored thickness of this layer was between 1.3 m to 1.9 m.

The composition of this fill layer is sand and gravel, and trace to little silt and clay size particles. The material is brown to grey in color, and moist. The SPT “N” values within this layer ranged from 16 to 39 blows per 300 mm penetration, suggesting compact to 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:

- 3.5% to 13.0%

Grain Size Distribution:

- 8% to 24 % gravel;
- 68% to 77% sand; and
- 8% to 15% silt and clay

The results of the moisture content and grain size distribution tests are provided on the record of borehole sheets in Appendix C. The results of the grain size distribution tests are also provided on Figure 1 in Appendix D.

1.4.3 Rock Fill: Variety-Sized Fragments of Blasted Rock

A rock fill composed of fragments of blasted rock was encountered below the sandy gravel to gravelly sand fill in boreholes BH1 and BH2. In general, the layer was encountered with matrix of blasted rock fragments having the variety of particle size, from sand to boulder size (i.e from 2 mm to greater than 300 mm). The rock fill layer extended to depths ranging between 8.2 m to 12.2 m below road surface with elevations ranging between 342.4 m and 339.0 m. The explored thickness of this layer was between 6.1 m to 10.7 m.

The composition of this fill layer is matrix of blasted rock fragments with some sand, some gravel and some cobbles and boulders size particles. As explained in Section 1.3.1 the combination of SPT and coring was attempted to obtain the samples from this fill layer. Where it was possible the split spoon tests attempted to collect samples at this layer. However, in majority cases the adequate samples were not able to be recovered, and whatever collected was possibly not representative of this layer. The materials which were successfully collected in the split spoon were pink and grey in color. The SPT “N” values measured during these tests ranged from 3 to 34 blows per 300 mm penetration. Though, relative density does not apply to rock fill, for information purpose the obtained SPT “N” values within this part of the layer matrix suggesting very loose to compact compactness condition. For the other part of this layer matrix

where the split spoon/auger refusal was encountered, the coring was performed recovering the cored samples of upto 150 mm in size.

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

Moisture Content:

- 4.5% to 11.9%

Grain Size Distribution:

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

The results of the moisture content and grain size distribution tests are provided on the record of borehole sheets in Appendix C. The results of the grain size distribution tests are also provided on Figure 2 in Appendix D.

Uniaxial compressive strength and abrasivity tests were performed on selected boulder core samples from BH1. The results of these tests are summarized in Table 1.1 below.

Table 1.1 Uniaxial compressive strength and CERCHAR abrasivity index of boulder cores in the rock fill of variety-sized blasted rock particles

Borehole	Ground Surface Elevation (m)	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 boulders 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 D.

1.4.4 Rock Fill: Cobble and Boulder-Sized Blasted Rock

A rock fill consisting of cobble and boulder-sized blasted rock grains was encountered below the layer of rock fill of variety-sized fragments of blasted rock in BH1. This rock fill layer extended to depth about 13.7 m below road surface with elevation about 336.9 m. The explored thickness of this layer was about 5.5 m.

The composition of this fill layer is mainly of cobbles and boulders size grains. The coring was attempted to obtain their samples. The recovered cored samples size varied from 40 mm to 200 mm.

Uniaxial compressive strength and abrasivity tests were performed on selected boulder core samples from this layer in BH1. The results of these tests are summarized in Table 1.2 below.

Table 1.2 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)	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 boulders 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 D.

1.4.5 Gravelly Sand to Sand and Gravel

Native gravelly sand to sand and gravel layer was encountered below the rock fill layer in borehole BH 1 and BH2. 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 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 77 blows per 150 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 result is as follows:

Moisture content:

- 18.2%

Grain Size Distribution:

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

The results of the moisture content and grain size distribution tests are provided on the record of borehole sheets in Appendix C. The results of the grain size distribution tests are also provided on Figure 3 in Appendix D.

1.4.6 Peat

A peat layer was encountered beneath sand and gravel layer in BH2 and at 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 depth 13.3 m below road surface with elevation 337.9 m in boreholes advanced from roadway and in off-road boreholes advance at inlet and outlet locations, the peat layer was at ground surface which was extended to depths ranging between 0.2 m to 0.4 m below ground surface with elevations ranging between 338.2m and 340.4 m. The explored thickness of this layer was between 0.2 m to 0.6 m.

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

Moisture content:

- 40.3% to 86.9%

The results of the moisture content tests are provided on the record of borehole sheets in Appendix C.

1.4.7 Sandy Silt

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

The composition of this 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:

- 32.6%

Grain Size Distribution:

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

The results of the moisture content and grain size distribution tests are provided on the record of borehole sheets in Appendix C. The results of the grain size distribution tests are also provided on Figure 4 in Appendix D.

1.4.8 Cobbles and Boulders

Cobbles and boulders were encountered underlying peat layer in BH3. The cobbles and boulders layer extended to depths about 0.8 m below ground surface with elevation about 340.2 m. The explored thickness of this layer was about 0.2 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:

- 16.7%

1.4.9 Bedrock

Bedrock was encountered underlying the peat/ sandy peat layer in BH1 and BH4, underlying cobbles and boulders in BH3 and beneath the sandy silt layer in BH 2. The bedrock was encountered at depths ranging between about 0.2 m to 0.8 m below ground surface at the inlet and outlet, and about 14.0 m to 14.6 m below the existing road surface. The bedrock was confirmed by coring of 1.7 m to 3.9 m long rock cores in the drilled boreholes. The elevation of the bedrock surface below Hwy 118 ranges from Elev. 336.0 m to Elev. 340.2 m. The bedrock surface depth and elevation encountered at the drilled borehole locations are listed in Table 1.2. Photographs of rock cores are included in Appendix E. All the boreholes are terminated within bedrock.

Table 1.2 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	0.8	340.2	Bedrock Cored
BH4	0.2	338.2	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 51% to 100%, indicating a rock mass of fair to excellent quality.

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 investigation surficial flow of water through the culvert was observed to be at approximate Elevation 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 at the site and presented in **Part I-Foundation Investigation Report**. The interpretation and recommendations provided are intended solely to permit designers to assess appropriate method and feasibility of installation of proposed culvert by 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 on a new alignment approximately 6 m offset to the north of existing culvert as shown on Drawing 1, Appendix B. 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. 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 RFQ, this section also provides discussion about the suitability of trenchless methods (i.e. jack and bore, pipe jacking using a tunnel boring machine, pipe ramming, micro-tunneling, utility tunneling, pipe bursting, pipe swallowing, in-line replacement, etc.) of culvert installation at the specific site. Pertinent construction issues for both methods from a geotechnical standpoint are examined in general accordance with the Terms of Reference from MTO Letter dated January 28, 2016.

2.2 Expected Ground Conditions

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

- a. The highway embankment consists of fill material composed of gravelly sand to rock fill of variety-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 and BH2).
- c. The proposed culvert inverts are assumed approximately at elevation 341.8 m at the inlet and 338.7 m at the outlet. Excavation for the trenchless method of culvert installation, if selected, will be through rock fill underlying thin layer of peat and followed by bedrock.
- d. Thickness of the peat layer underlying rock fill in BH2 was 0.6 m.
- e. The groundwater levels in the open boreholes were not recorded due to the wash boring technique was used to advance boreholes. However, based on the measured water level in the stream flowing through the existing culvert, the inferred groundwater level within the embankment was estimated to be at approximate Elevation of 341.1 m. Seasonal variations in the water table should be expected.
- f. The water level observed in the culvert at the time of investigation was at Elevation 341.1 m at the inlet and Elevation 338.0 m at the outlet.
- g. The slopes of the embankment at the inlet and outlet sides are covered by rock fill. No slope instability on either embankment slopes was observed.

It is understood that it is intention to install the new culvert by a trenchless method at the proposed location which is approximately 6 m offset to the north of the existing culvert. However, for report completeness a non-trenchless method for culvert installation such as a open cut and cover method is also discussed as a possible construction option.

It is assumed that for the open cut and cover method the new culvert will be replaced at the same location and founding levels of the existing culvert (i.e. invert elevations: 341.1 m at the inlet and 338.0 m at the outlet). Since, no borehole was drilled at the existing culvert location, the comments and recommendations for the open cut and cover method are based on information gathered in the boreholes drilled at the proposed culvert alignment location (i.e. approximately 6 m offset to the north of the existing culvert).

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. 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).

2.3 Culvert Installation Options

For selection of appropriate construction methods for this culvert installation it was considered the following: (i) whether disruption of the traffic is acceptable or not; (ii) whether a new alignment is proposed or not; (iii) soil conditions at zone of culvert installation; and (iv) diameter and length of the new culvert. Further, several items to keep in mind during the selection were: (i) only approach that would allow removal of the existing culvert is a cut and cover method; (ii) the trenchless (tunneling) approaches involve construction adjacent to the current alignment with the need to decommission the existing culvert including

grouting and sealing; (iii) since the embankment consist 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.

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

- Traditional cut and cover method (using existing local roadways as temporary detours around the area, depending on availability); and
- Trenchless (tunnelling) methods (i.e. micro-tunnelling and TBM tunnelling)

For cut and cover replacement of this culvert, a half and half construction approach using a temporary shoring (soldier pile and lagging) is likely not possible due to the high rock fill embankment and sloping highway. In addition, due to the very limited space available on west and east sides of the highway, the construction of a temporary detour at the site is likely not possible. Therefore, these two construction options as the cut and cover replacement method were not considered for this culvert. Use of existing local roadways as temporary detours around the site is recommended.

For trenchless replacement of this culvert, pipe bursting, pipe splitting and pipe swallowing methods were not considered as applicable in this project, since the nature of rock fill embankment classify this culvert as an unsuitable candidate for these 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 assessed as an unviable option as well. The installation of the culvert using tunneling methods which use an auger for excavation such as jack and bore and pipe ramming methods were also assessed as impractical in rock fill. The reason is that auger would struggle in this material with possible misalignment in the coarse media. In addition, these methods are not very good control in mixed face. For the same reasons a horizontal directional drilling (HDD) method was eliminated as a viable 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>Cut and Cover Method</i>	<i>Using local temporary detour and open cut unsupported excavation</i>	<ul style="list-style-type: none"> • Assessment of the foundation soil • The existing culvert can be used to maintain the surface water flow during the construction • Removal of existing culvert • Existing embankment fill can be removed and replaced with free draining granular material • Adaptable to changing ground conditions 	<ul style="list-style-type: none"> • Traffic interruption • Large amount of rock fill to be excavated • Rock fall hazard for unsupported open cut excavation • May possess difficulty in excavating rock fill embankment • Risk of cost overrun and inability to finish job: low to moderate 	Less expensive than trenchless method	1 (if traffic interruption is acceptable)
<i>Trenchless Method</i>	<i>Micro-tunneling (Non-entry Method)</i>	<ul style="list-style-type: none"> • No traffic interruption and requirement for detour route • 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 	<ul style="list-style-type: none"> • High construction cost • Obstruction problematic; pre-grouting might be required • No access during tunneling • Excavation and shoring required to achieve starting grade, as well as to minimize possible impact on the global stability of the embankment • Requires large area for jacking shaft and support equipment • Dewatering possibly required at launching and receiving pits • Requires decommissioning of old culvert, including grouting and sealing • Risk of cost overrun and instability to finish job: moderate to high 	Probably less expensive than TBM tunneling, if the equipment is readily available	3

Installation Method		Advantages	Disadvantages	Relative Cost	Ranking
			<ul style="list-style-type: none"> • Need to take precaution for tunnelling, due to presence of peat layer underlying rock fill • 		
<i>Trenchless Method</i>	<i>An open face digger shielded type TBM Tunneling (Man-entry Method)</i>	<ul style="list-style-type: none"> • No traffic interruption and requirement for detour route • Safe to use in mixed ground condition • Ability to access obstructions during tunneling • Can handle small boulders of size up to 33% of the casing diameter • The existing culvert can be used to maintain the surface water flow during the construction • Possibly more contractors willing to undertake 	<ul style="list-style-type: none"> • High capital investment • Pre-grouting is required in rock fill to avoid collapse • Excavation and shoring require to achieve starting grade, as well as to minimize possible impact on the global stability of the embankment • Not practical for small diameter pipe (min. 1.8 m diameter) • Requires decommissioning of old culvert, including grouting and sealing • Risk of cost overrun and instability to finish job: moderate to high • Need to take precaution for tunnelling, due to presence of peat layer underlying rock fill 	Probably more expensive than micro-tunneling	2

Based on the above list of advantages and disadvantages, cut and cover methods might be 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 the rock fill. However, if the Regional Traffic Office requires replacement of the culvert without disrupting traffic, then trenchless (tunneling) installation methods listed in Table 2.1 are more viable. The major disadvantages of these trenchless installation methods are higher cost of installation than the cut and cover method and the need to decommission the existing culvert by grouting and sealing. Between two tunnelling methods discussed in the table, an open face digger shielded type TBM tunneling (Man-entry Method) is assessed as the more practical. Micro-tunnelling is ranked as the less viable trenchless method since the nature of embankment fill may create significant obstruction.

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

- Culvert installation by the cut and cover method: open cuts using local temporary detours
- Culvert installation by tunnelling methods: micro-tunnelling and TBM tunnelling

2.4 Culvert Installation by Cut and Cover Method

Considering the very limited space available on west or east sides of Hwy 118, construction of a temporary detour at the site followed by open cut unsupported excavation appears to be not possible. However, local existing roadways may be used for temporary detours (depending on availability). With this approach, grouting and sealing of the existing culvert (i.e. decommissioning) will be eliminated since this option will allow removal of the existing pipe. It will also allow for the assessment of the foundation soils below the proposed location, and peat or soft materials encountered can be removed. The existing culvert could be used for maintenance and diversion of surface water flow during the construction. However, as mentioned before, the temporary support for excavation of the existing embankment may not be possible due to cobble and boulder-sized rock fill. On the other hand, the unsupported open cut excavation may be concern for the nature of high rock fill embankment and possibility of rock tumbles.

2.4.1 Temporary Excavations

All excavations at this site must be conducted in accordance with the Occupational Health and Safety Act (OHSA) and Regulations for Construction (O.Reg 213/91). The embankment fill soils (i.e. gravelly sand to loosely placed rock fill of fragments of blasted rock/ rock fill of cobbles and boulders) may be classified as a Type 1 soil above the groundwater table and Type 3 soil below the groundwater table, in conformance with the OHSA. Excavations are mostly expected to be above the groundwater level measured in the boreholes on the embankment crest during this investigation. To avoid disturbance

of the founding subgrade and to allow placement of backfill in dry conditions, groundwater must be controlled to below the proposed excavation levels prior to digging to final levels.

Temporary excavations side slopes for Type 3 soil should not exceed 1H:1V in accordance with OHSA. But considering the nature of rock fill may possess rock fall hazard it is recommended to protect the construction site from rock fall hazard either by excavating in wedges or by installing temporary rock fall protection system like draped mesh along with silt protection mesh to protect free moving of fines. Depending on topography and overland flow drainage path beyond the crest of the proposed cut slope, a drainage ditch may be required near the crest to divert water away from the cut slope to prevent wash out and erosion. For the anticipated roadway cut, a 2 m wide bench should be incorporated into the slopes at the approximate mid-height of the slope and extend the full length of cut. Although the slope is considered to be stable without the bench, incorporation of the bench will allow for control of erosion and sloughing of surficial soil.

2.4.2 Maximum Fill Height

Pipe selections for new culverts must conform to maximum height restrictions as outlined in OPSD for Roads and Public Works (e.g. OPSDs, 805.010 for CSP, 805.020 for CSP-Arched, and 807.010 for Reinforced Concrete Pipe-Confined Trench, 807.030 for Reinforced Concrete Pipe).

2.4.3 Culvert Bedding

OPSDs 802.010, 802.031, and 802.032, which are included in Appendix F, provide the bedding, embedment, cover and backfill standards for the different pipe materials. According to these standards the culvert bedding should consist of Granular "A" (OPSS 1010) with thickness of 300 mm or alternatively a 100 mm thick concrete working slab with 75 mm of bedding materials beneath the culvert and extend a minimum of 500 mm horizontally on either side of the culvert edge. The bedding material should be placed in layers not exceeding 200 mm in thickness, loose measurement, and compacted to at least 95% of the Standard Proctor Maximum Dry Density (SPMDD) before a subsequent layer is placed in accordance with OPSS 514. Bedding material placed in the haunches must be compacted prior to continued placement of cover material. Bedding on each side of the pipe shall be completed simultaneously. At no time shall the levels on each side differ more than the 200 mm uncompacted layers.

Prior to placing any fill materials, the exposed native subgrade should be inspected according to OPSS 902. A non-woven geotextile separator is to be placed between the approved subgrade and the compacted fill to assist in material placement and to maintain the integrity of the founding soil along the entire length of the culvert. The geotextile separator is to be a Class II non-woven material with an equivalent opening size of 75-150 µm.

2.4.4 Culvert Backfill

The culvert backfill should consist of Granular "B", Type I or Granular "A" (OPSS 1010) placed in layers not exceeding 300 mm in thickness for the full width of the trench and each layer shall be compacted to 95% of the Standard Proctor Maximum Dry Density before placing a subsequent layer,

according to OPSS 514.

The culvert should be encased with a minimum of 300 mm of compacted material. Typical backfill diagrams are presented in Appendix F, OPSD 802.010, 802.031 and 802.032. The minimum height of fill cover above the crown of the pipe before power operated tractors or rolling equipment shall be 900 mm, unless otherwise noted by the structural engineer.

2.4.5 Dewatering

Provided that the existing SPCSP culvert is to remain in use during construction of the new culvert, the majority of the upstream flow of the existing culvert can be diverted around the construction area. It is the responsibility of the Contractor to propose a suitable dewatering system based on the time of construction and groundwater levels and surface water flow conditions for prior approval of the MTO. The method used should not undermine the existing road.

Erosion and sediment control during culvert construction should be as per the MTC Drainage Manual, Volume 2. Silt fences and other sediment control measures should be included to protect the downstream environment from the construction activities.

2.5 Culvert Installation by Trenchless (Tunneling) Method

Tunneling is considered a viable installation method for culvert replacement along the proposed culvert alignment. However, noting that any tunneling approach through rock fill consisting of variety size of blasted rock fragments including cobble and boulder size is risky and difficult. In this connection the approach is only being considered due to the many applications where high rock fills are present and replacement using open cut methods may be impractical. 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 industrial standards. According to OPSS 421, the minimum spacing allowed between new culvert and existing culvert is 600 mm (if pipe diameter <1.2 m) or a 0.5 times of 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 diameter 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. The existing abandoned culvert must be properly decommissioned including grouting and sealing.

It is projected that the culvert trenchless (tunneling) excavation will be carried out through rock fill consisting of variety size of blasted rock fragments, assuming that the approximate elevation of the new culvert invert is between 341.8 m at the inlet and 338.7 m at the outlet. Based on the measurements during this investigation, the inferred ground water level within the embankment was estimated to be at approximate Elevation of 341.1 m or slightly above, which appears to be below the tunnel invert. However, seasonal variations in the groundwater table should be expected. 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 addition, it is recommended to temporary excavation of surficial rock fill to precede the start of tunneling. Provision must be made to maintain surface water flow to the outlet.

Two methods, a non-entry method such as micro-tunneling and a man-entry method using an open face digger shield type TBM are discussed in Table 2.1 and the subsequent paragraphs.

2.5.1 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 and voids between the rock fill grains, pre-grouting might be required. If there is little to no fines/gravel between the big rock voids, 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-tunnelling method is that its performance is not affected by high groundwater levels, so the dewatering is not required. Major disadvantages of micro-tunneling for this project are considered to be the relatively high cost of mobilization and lack of locally skilled contractors. This option may become more attractive if potential bidders have available equipment in house. 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.

2.5.2 TBM Tunneling Method

TBM tunneling is a man-entry tunneling method and encompasses the use of a tunnel boring machine (TBM). This method utilizes laser-guided targeting that achieves a very accurate line and grade to the pipe being installed. A shielded TBM equipped with cutters for hard rock (i.e. a carbide tooth type) should be used. If the rock is exceptionally hard a beam machine may be an option. To control ground movement behind the TBM a primary liner must be installed. TBM can employ single pass or two pass system. In the two pass system the temporary liner can be ribs and lagging with the permanent liner cast-in-place afterwards. The primary liner can be provided by steel, cast iron or precast concrete liner plates. To prevent collapse, the rock fill must be pre-grouted which could be done through horizontal drilled holes. This procedure could be costly. 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 as noted in Section 2.4.4. 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. TBM tunneling might be the more expensive method for the installation of the proposed culvert considering the short length of the tunnel. However, cost might be reduced if, and where, existing Contractor's suitable TBM is available. Tunneling using a TBM is assessed as the best viable tunnel excavation method in rock fill.

2.5.3 Considerations of Tunneling

2.5.3.1 Groundwater Control

As mentioned before, a small amount of groundwater seepage into the tunnel should be expected in the zone of tunneling. However, the 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 requirements will be governed by the time of the year when the construction is performed. It is the responsibility of the Contractor to propose a suitable dewatering system based on the time of construction and groundwater levels. The method used should not undermine the existing highway. Dewatering shall conform to OPSS 517.

2.5.3.2 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 2.5.4).

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 should be applied to arrest or reduce settlement.

2.5.3.3 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. Excavations for launching and receiving pits will be conducted through rock fill. 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.

2.5.3.4 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.

2.5.4 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. This plan 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.

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 (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 2.4 m in this area.

A reading schedule should be as follows:

- A minimum one set of readings prior to construction as a baseline reading.
- A minimum three sets of readings during construction provided the movements are within

the anticipated limits. Otherwise, the reading frequency may have to be increased.

- A minimum of two sets of readings on a weekly basis after completion of the work.

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.

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.

2.5.5 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 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.

2.6 Inlet and Outlet

2.6.1 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 F of this report. Rip-rap placed at 1V:1H will be stable.

2.6.2 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 F of this report.

2.6.3 Frost Protection

According to Ontario Provincial Standard Drawing (OPSD – 3090.101), the frost depth in the subject site is about 1.8 m. Consequently, any footing exposed to seasonal freezing conditions should be protected from frost action by at least 1.8 m of soil cover or equivalent.

2.7 Slope Stability and Settlement Analyses

2.7.1 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. If the cut and cover method is selected for the culvert replacement the slope, the new embankment properly constructed as described in Section 2.4.4 should be stable with the slopes of 1.5H:1V (i.e. FOS >1.3), as shown in Figures 1 and 2 in Appendix G. The SLOPE/W computer program developed by GeoSlope International was employed for computation of slope stability. The slope stability analyses were performed using the Morgenstern-Price method developed on the basis of limit equilibrium.

2.7.2 Settlement

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

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.


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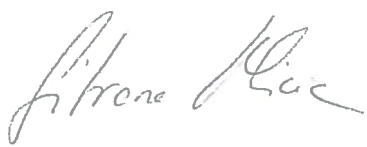
We trust that these comments provide you with sufficient information to for your present requirements. Should you have any questions, please do not hesitate to contact this office

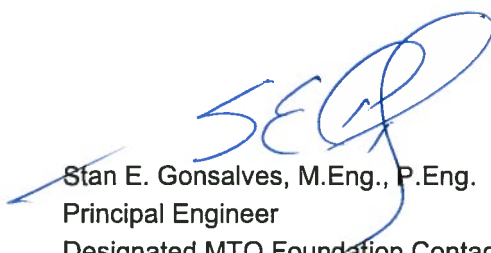
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4 LIMITATIONS AND USE OF REPORT

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.

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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 (upward) from existing culvert location



Photo 2: Hwy 118 looking south (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: East side embankment slope looking from existing culvert inlet



Photo 6: Looking north from BH3, proposed culvert inlet location



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: Deterioration of culvert at inlet



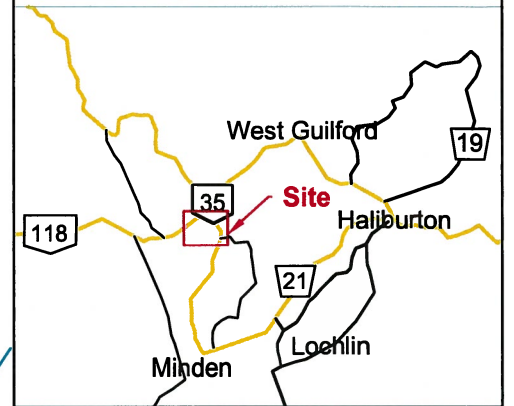
Photo 12: TBM on rock outcrop north-west of proposed culvert



Appendix B – Drawings

DIMENSIONS ARE IN METERS AND/OR
MILLIMETERS UNLESS OTHERWISE SHOWN.
STATIONS ARE IN KILOMETERS + METERS

1

KEY PLAN



 BH 1 Borehole location
 TBM Temporary Benchmark Location

BH No.	APPROX. ELEV.	MTM CO-ORDINATES (ZONE 12)	
		NORTH	EAST
TBM	343.69	4990034	366863
BH 1	350.8	4989983	366930
BH 2	351.2	4989983	366938
BH 3	341.0	4998997	366954
BH 4	338.3	4998969	366912

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.

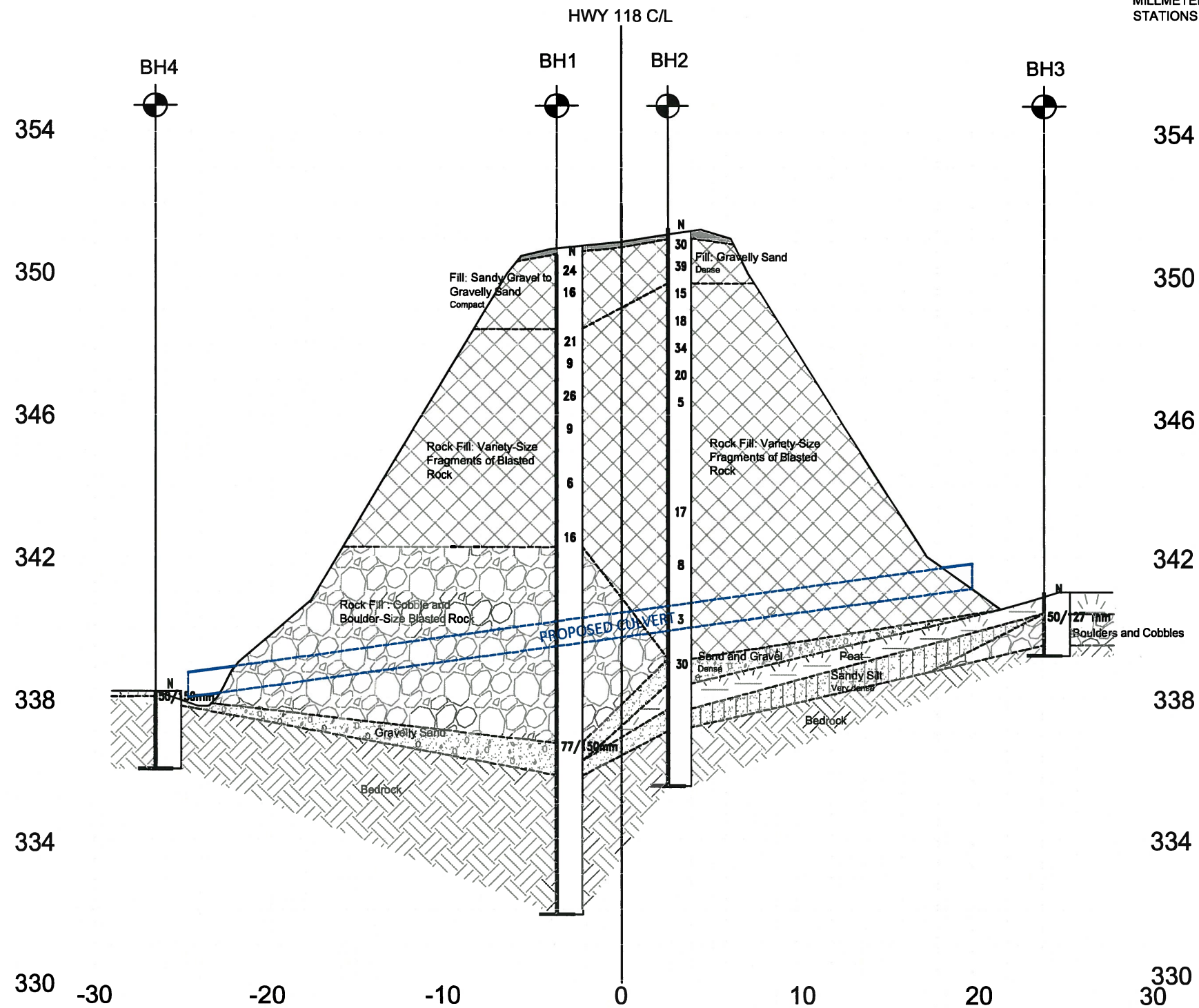
The complete foundation investigation and design report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in the report and related documents are specifically excluded in accordance with the conditions of Section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

2016.07.08	SM	FINAL SUBMISSION	
2016.05.31	SM	SUBMISSION FOR MTO REVIEW	
DATE	BY	DESCRIPTION	
		GEOCRES NO. 31E-364	
SCALE	2.5:1	PROJECT NO. ADM-00282450-P0	
SUBM'D SM	CHECKED SM	DATE	2016.07.08
DRAWN SA	CHECKED SG	APPROVED SG	DWG. 1

Note: The plan was provided by MTO.



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



AGREEMENT No. 5013-E-0008
ASSIGNMENT No. 13
GWP 5140-13-00

PROPOSED CULVERT INSTALLATION BY TRENCHLESS METHOD
(HWY 118, TOWNSHIP OF STANHOPE)
SOIL STRATIGRAPHIC SECTION

exp Services Inc.

KEY PLAN

LEGEND

SOIL STRATA SYMBOLS

BH No.	APPROX. ELEV.	MTM CO-ORDINATES (ZONE 12)	
		NORTH	EAST
TBM	343.69	4990034	366863
BH 1	350.6	4989963	366930
BH 2	351.2	4989983	366938
BH 3	341.0	4989997	366954
BH 4	338.3	4989969	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.

DATE	BY	DESCRIPTION
2016.07.08	SM	FINAL SUBMISSION
2016.05.31	SM	SUBMISSION FOR MTO REVIEW
GEOCRES NO. 31E-364		
PROJECT NO. ADM-00282450-P0		
SUBMD SM	CHECKED SM	DATE 2016.07.08
DRAWN SA	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:

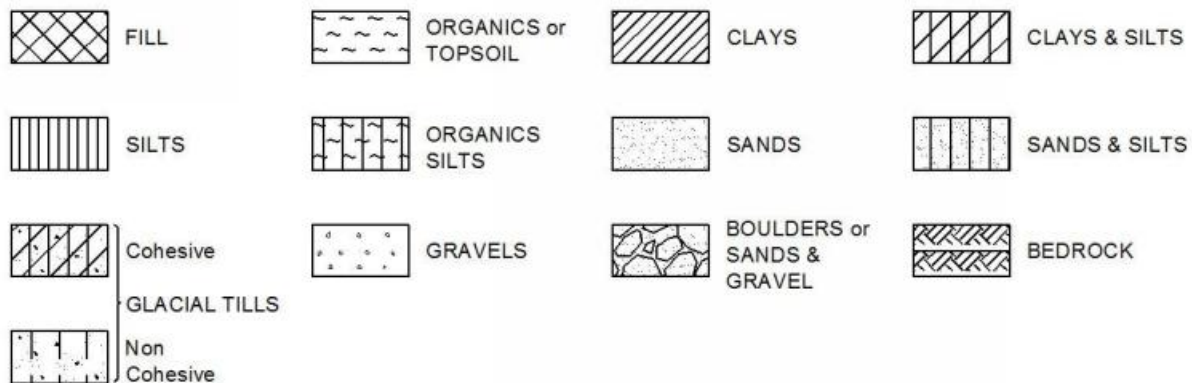
Table c: Consistency of Cohesive Soil

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 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	60	80	100	10	20	30										
336.9	ROCK FILL: COBBLE AND BOULDER-SIZE BLASTED ROCK pink and grey NQ CORING (continued)		13	NQ																			
	-soft layer @ 13.1 m																						
13.7	GRAVELLY SAND organic odour, occasional cobbles and boulders, brown, wet to saturated		14	SS	77/ 150mm								○				27	65 (8)					
336.0																							
14.6	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																			
				16	NQ																		
			17	NQ																			
332.2	END OF BOREHOLE																						
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.																						

+ 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

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			SHEAR STRENGTH: Cu, KPa									WATER CONTENT (%)			GR	SA	SI	CL
								○ UNCONFINED + FIELD VANE															
								× QUICK TRIAXIAL LAB VANE															
						20	40	60	80	100	10	20	30										
339.0																							
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															40.3							
337.9																							
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																			
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	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																
1.5	BEDROCK grey and white granite		4	NQ													
339.2	NQ CORING		5	NQ													
	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

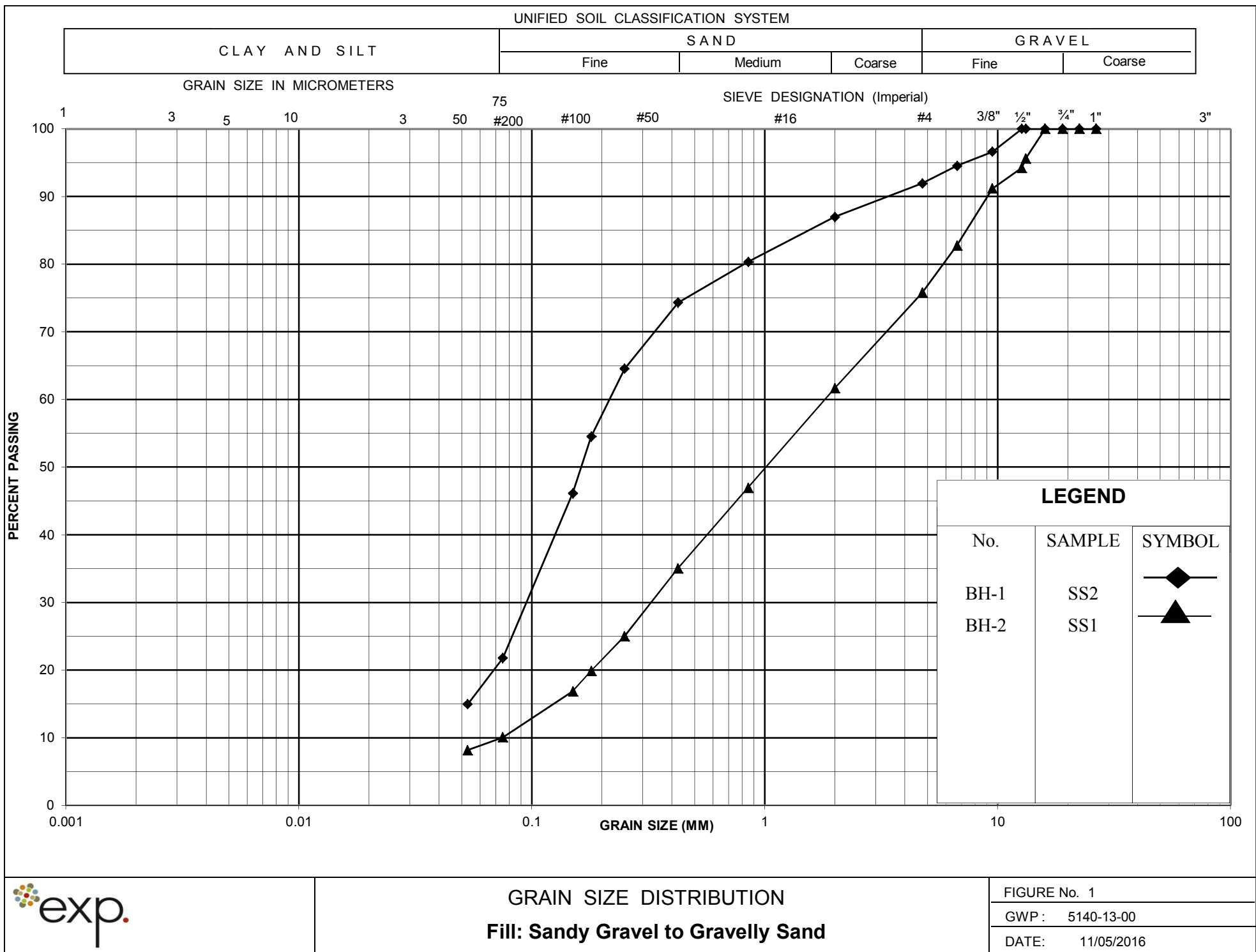
W. P. GWP 5140-13-00 LOCATION MTM ZONE10 N4998969 E366912 ORIGINATED BY CS
 DIST Hwy 118, Township of Stanhope BOREHOLE TYPE HUSQVARNA DS 800 Portable Hydraulic Drill/ NW/ BQ COMPILED BY JH
 DATUM BM Elev. 343.96 m DATE 2016/03/19 - 2016/03/19 CHECKED BY SM

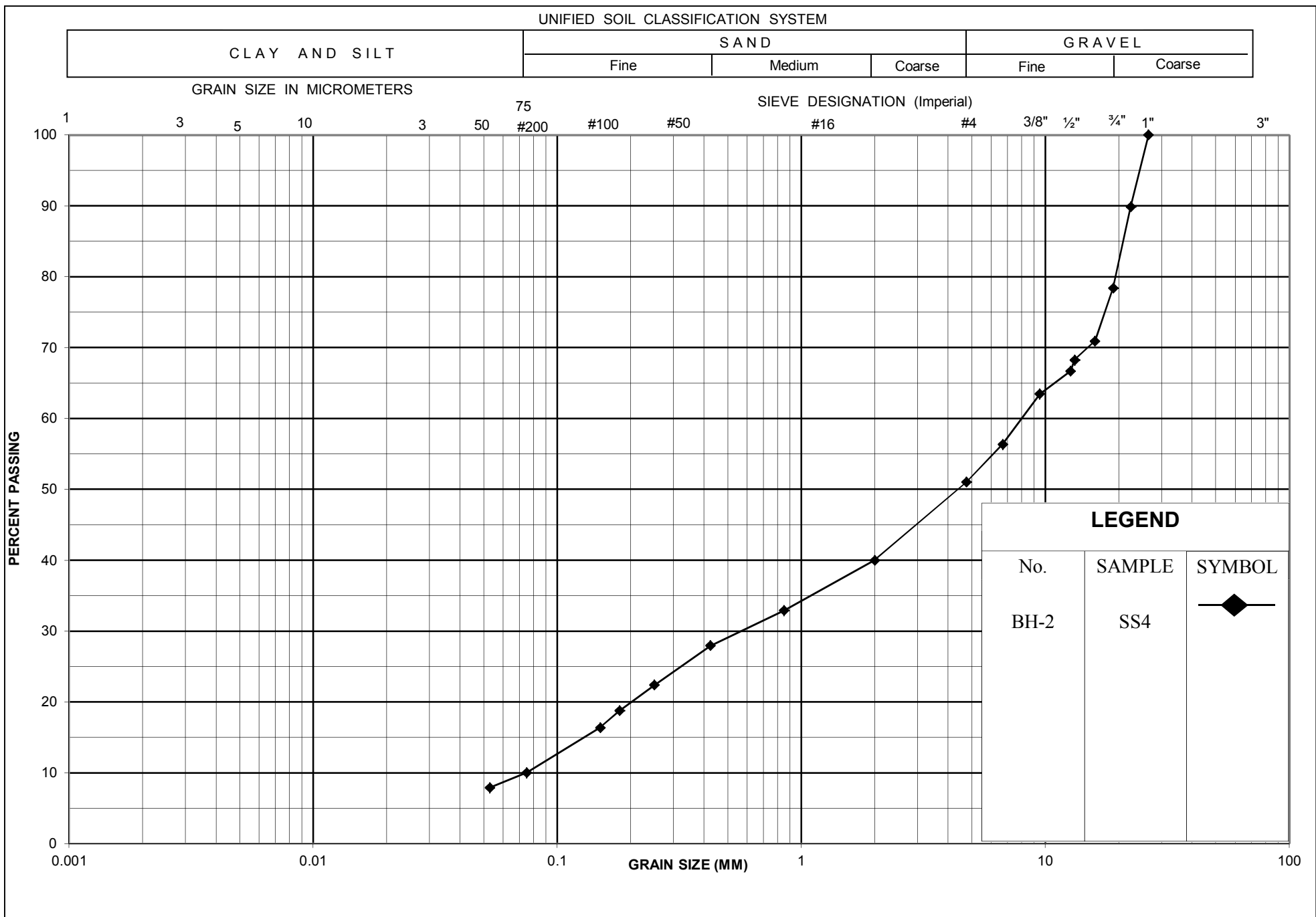
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			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, ×³: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

Appendix D – Laboratory Test Results



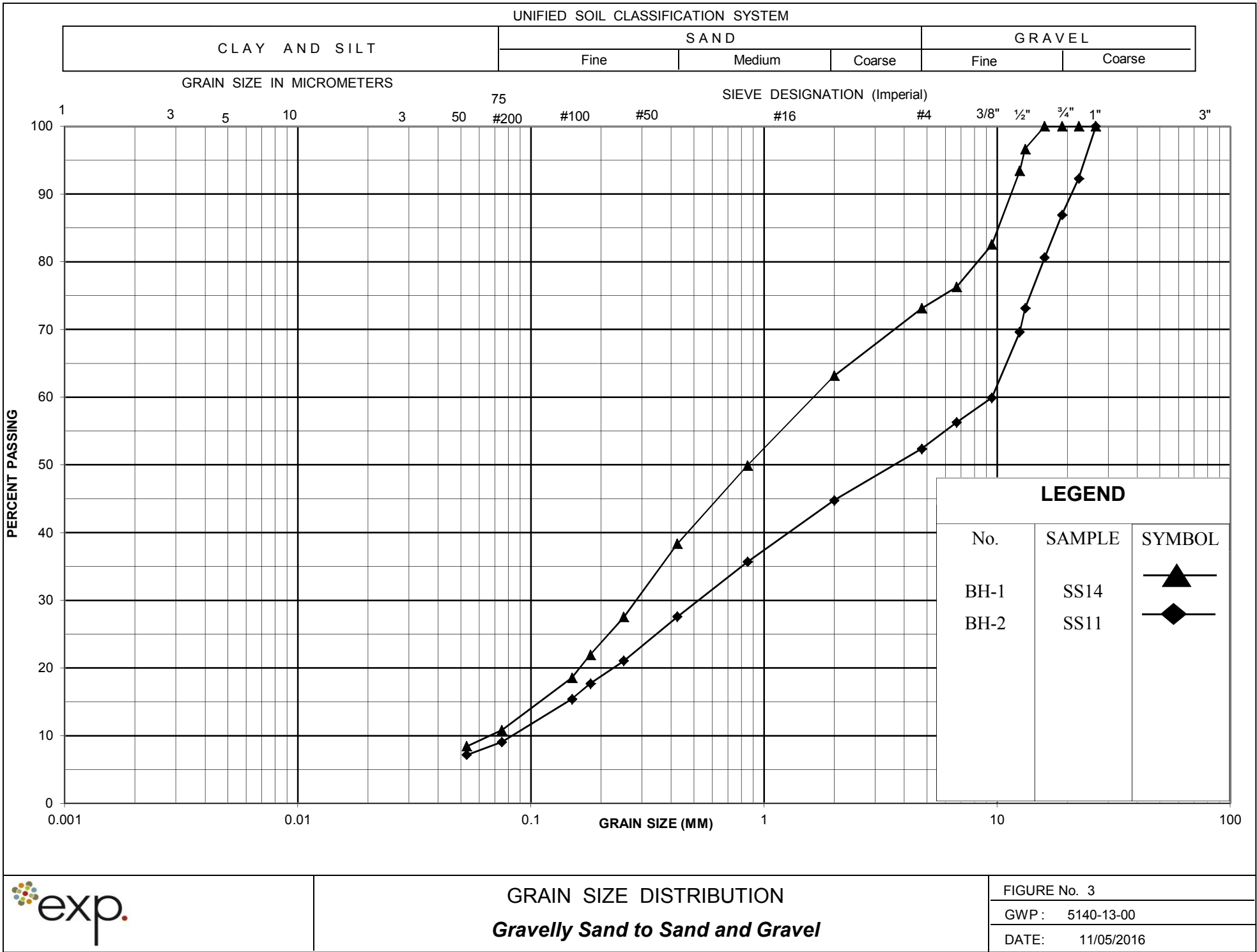


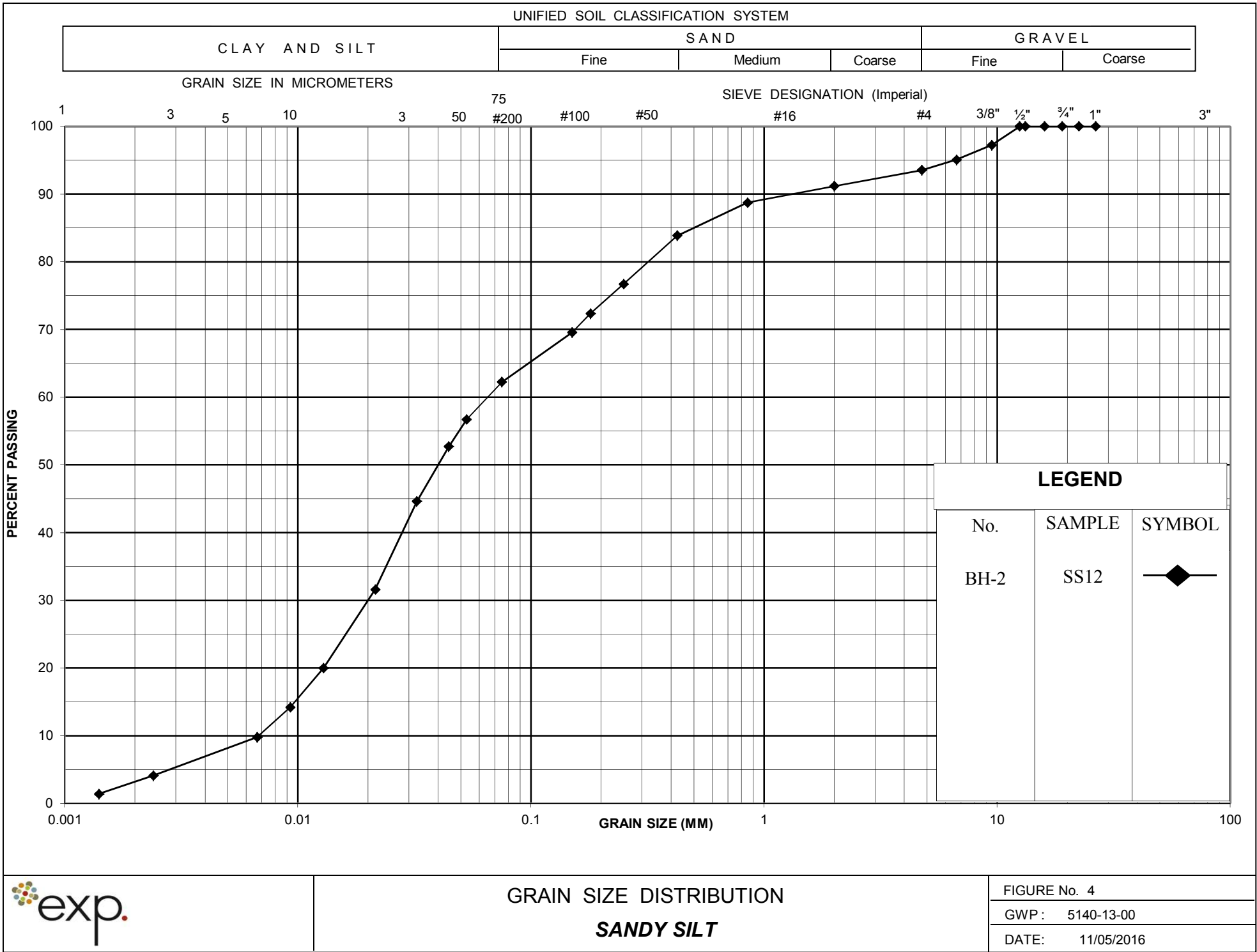
GRAIN SIZE DISTRIBUTION
Rock Fill: Variety-Sized Fragments of Blasted Rock

FIGURE No. 2

GWP : 5140-13-00

DATE: 11/05/2016







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.

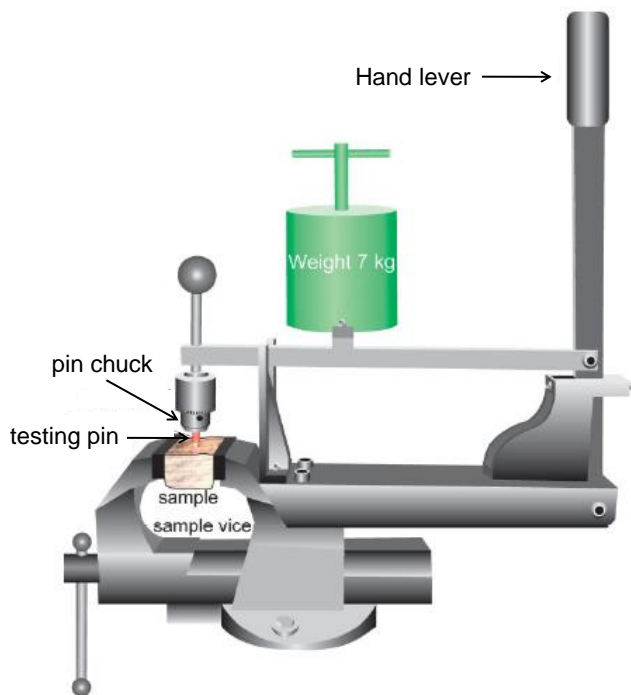


Figure 1 - CERCHAR test apparatus (after Plinninger and Restner, 2008)



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.

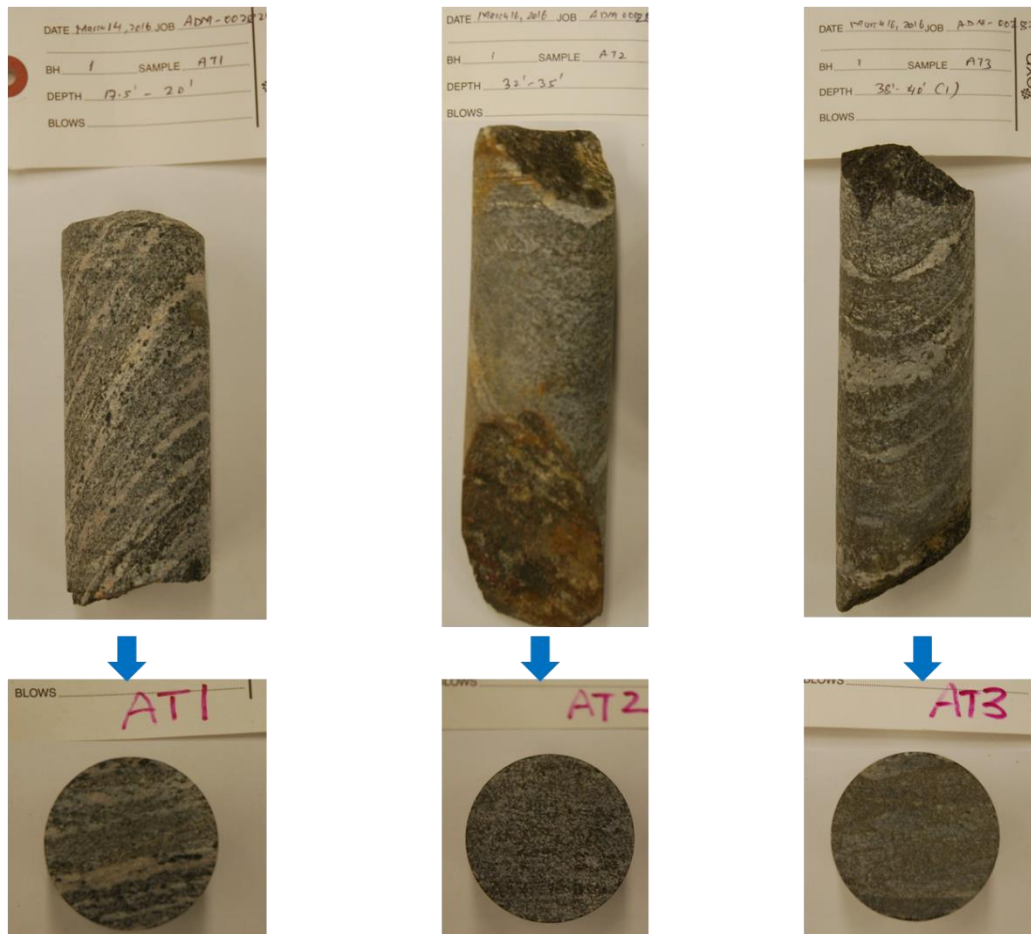


Figure 2- The supplied core and three test specimens after cutting



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](#).

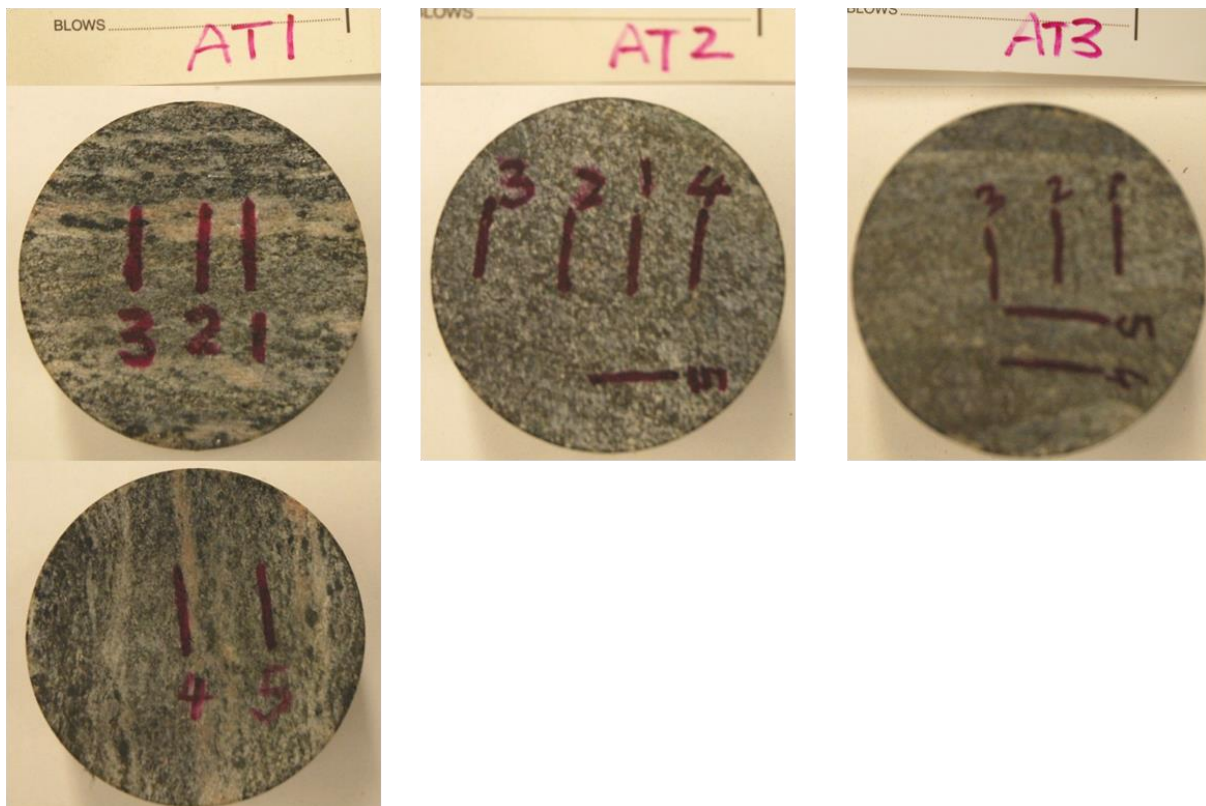


Figure 3-Specimens AT1, AT2, and AT3 after testing (scratches have been highlighted by marker pen)

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</i> _s	<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.

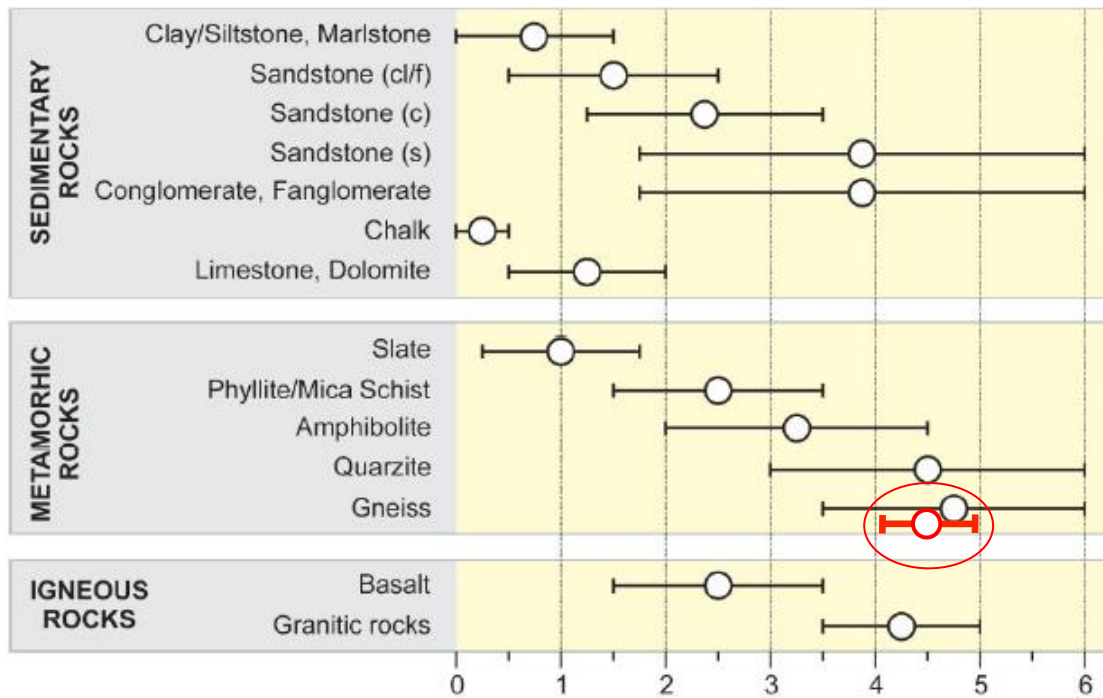


Figure 4- Comparison of test results (red) with compilation of typical CAI values (Plinninger and Restner, 2008)

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.

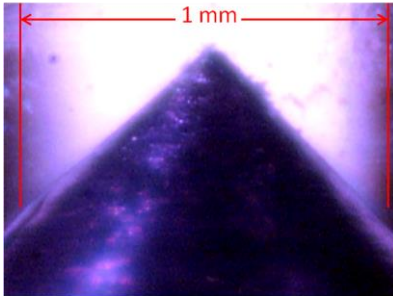
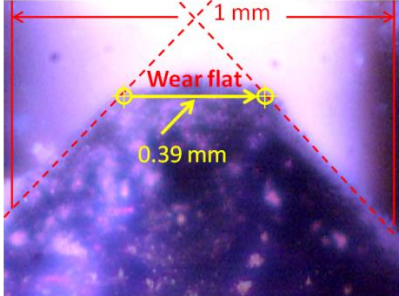
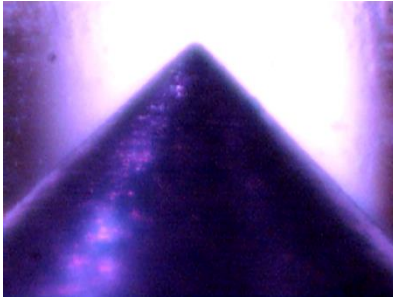
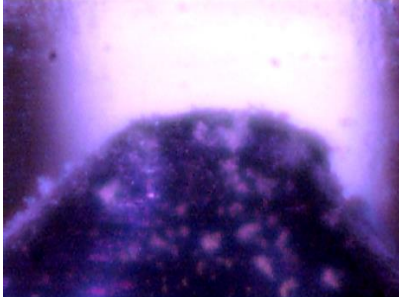
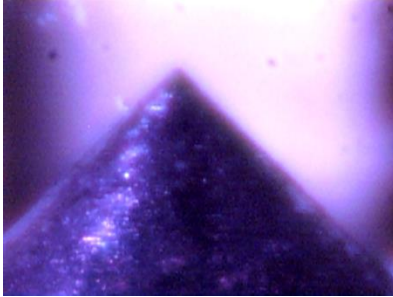
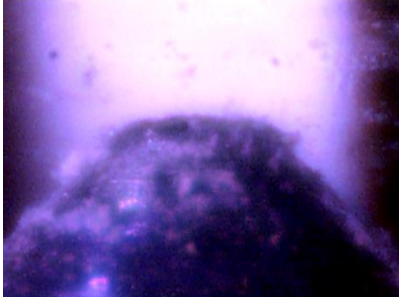
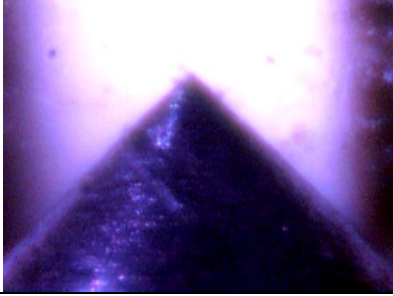
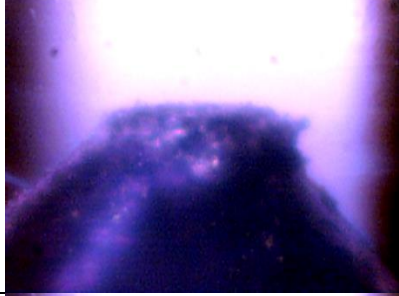
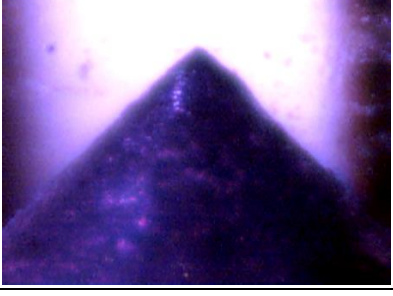
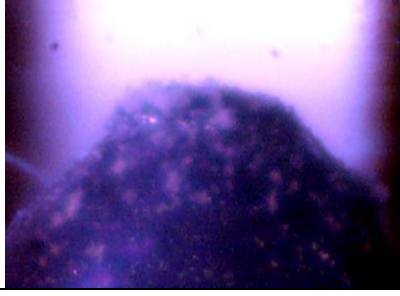
Plinninger, R., Kasling, H., Thuro, K. and Spaun, G., 2003. Technical note – Testing conditions and geomechanical properties influencing the CERCHAR abrasiveness index (CAI) value. *International Journal of Rock Mechanics & Mining Sciences*, **40**(2): 259-263.

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

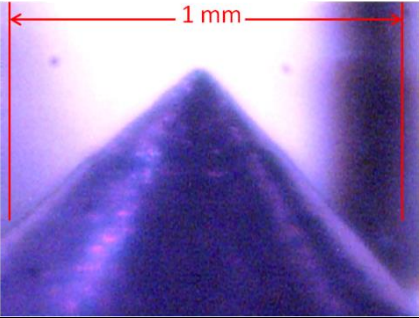
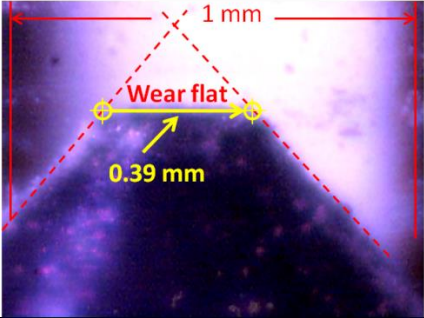
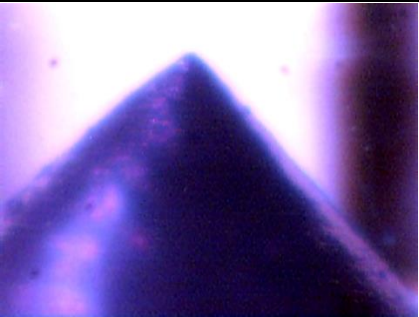
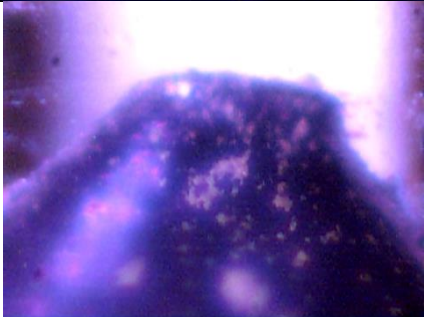
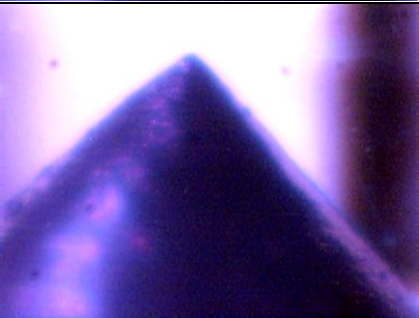
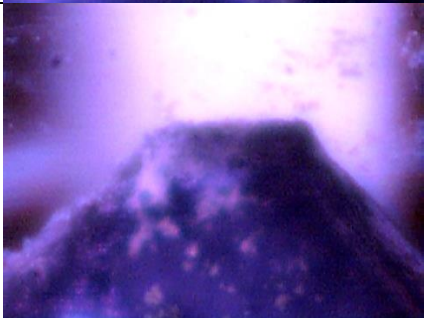
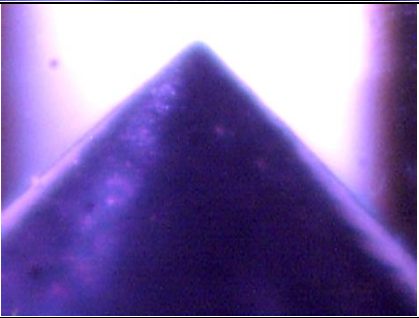
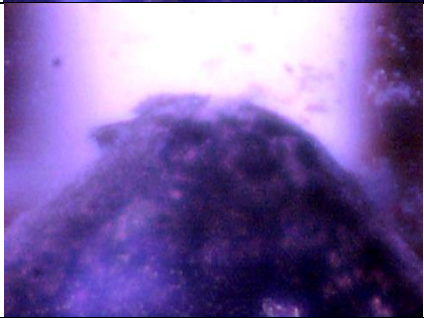
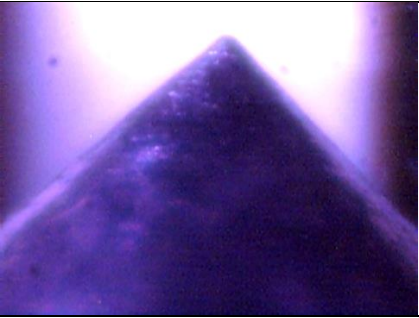
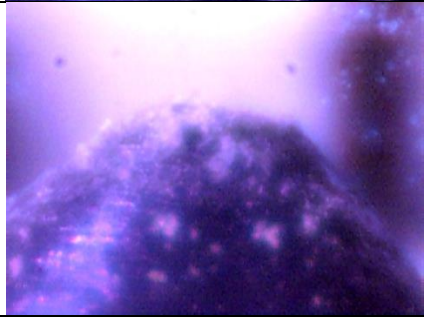
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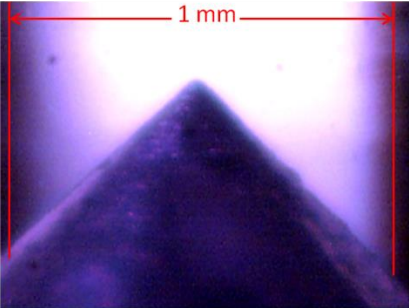
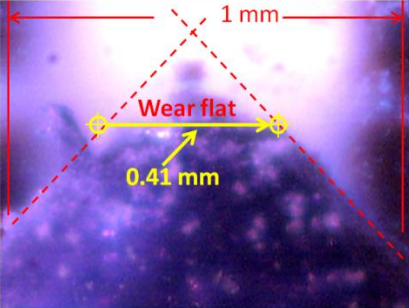
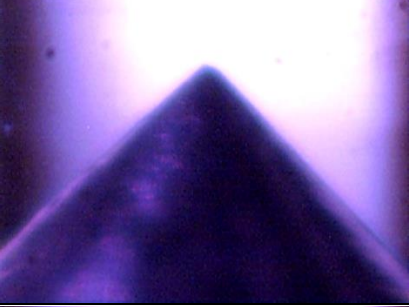
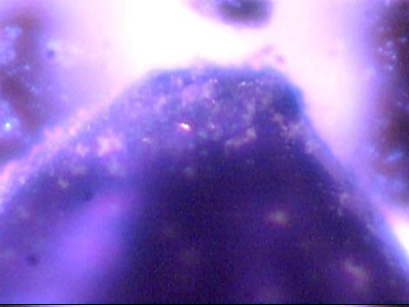
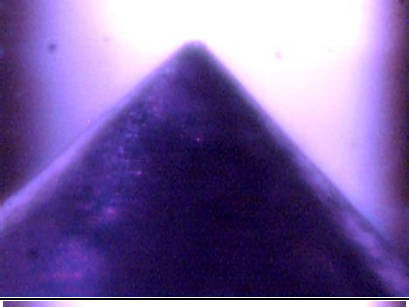
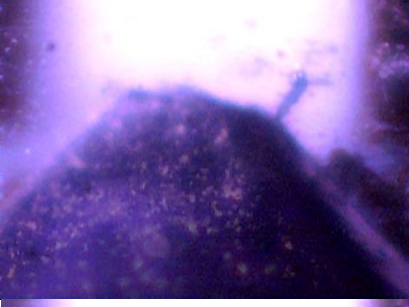
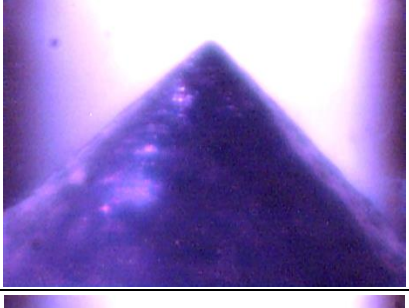
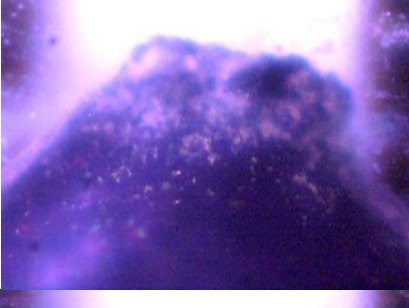
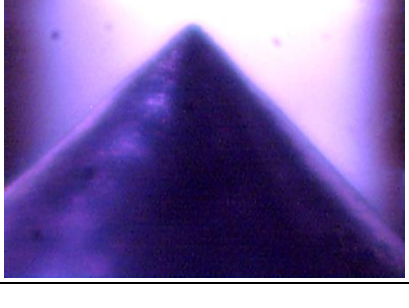
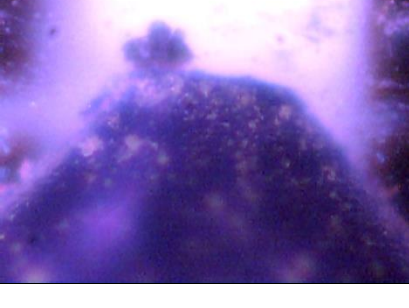
Appendix : Photographs of pins before and after testing

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 E – Bedrock Core Photographs

Project NO: ADM 00028245-P0
BH NO: 1
Run NO: 1
Sample Depth: 14.6 m to 15.75 m
Elevation: 336.0 m to 334.85 m
Date: March 17, 2016



Photo 1. Core Sample for BH1 from Elevation 336.0 m to 334.85 m

Project NO: ADM 00028245-P0
BH NO: 1
Run NO: 2 & 3
Sample Depth: 15.75 m to 18.4 m
Elevation: 334.85 m to 332.0 m
RQD: 51% to 67%
Date: March 17, 2016



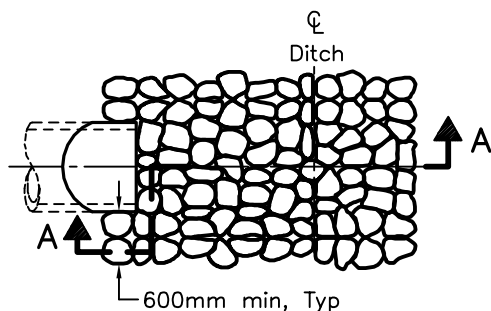
Photo 2. Core Sample for BH1 from Elevation 334.85 m to 332.0 m

Project NO: ADM 00028245-P0
BH NO: 2
Run NO: 1
Sample Depth: 14 m to 15.7m
Elevation: 337.2 m to 335.5 m
RQD: 83.2%
Date: March 19, 2016

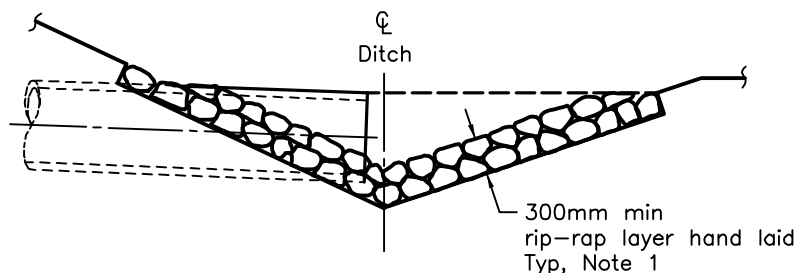


Photo 3. Core Sample for BH2 from Elevation 337.2 m to 335.5 m

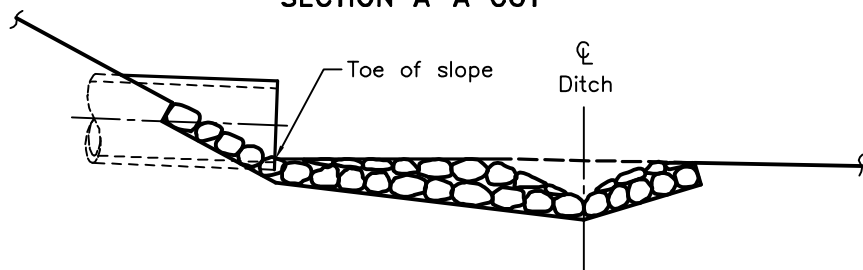
Appendix F – OPSDs



PLAN
CUT OR FILL

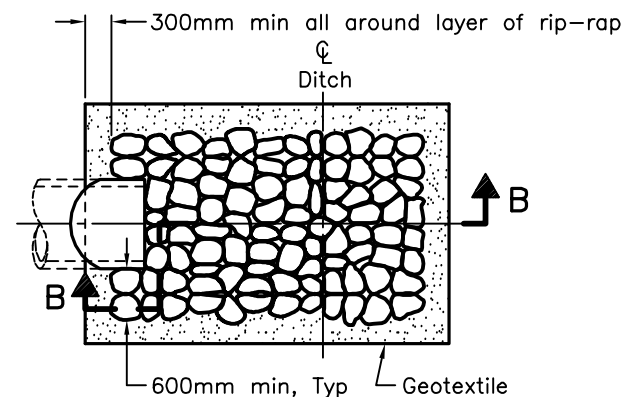
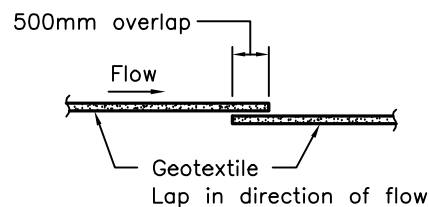


SECTION A-A CUT

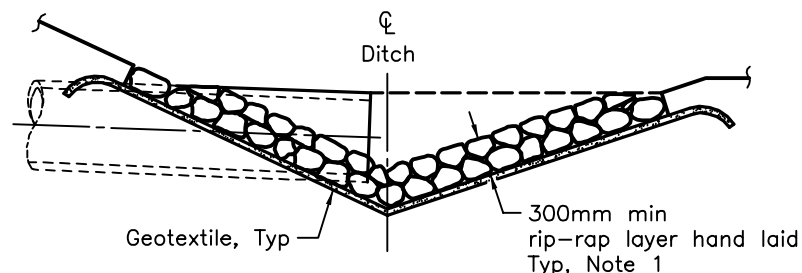


SECTION A-A FILL

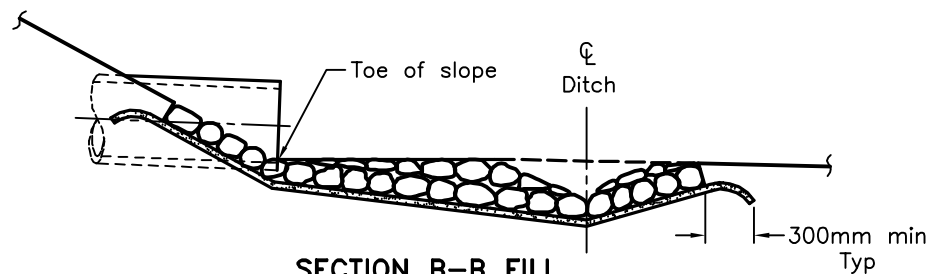
TYPE A – WITHOUT GEOTEXTILE



PLAN
CUT OR FILL



SECTION B-B CUT



SECTION B-B FILL

TYPE B – WITH GEOTEXTILE

NOTES:

1 The thickness of the rip-rap layer shall be at least 1.5 times the rip-rap mean diameter.

A All dimensions are in millimetres unless otherwise shown.

ONTARIO PROVINCIAL STANDARD DRAWING

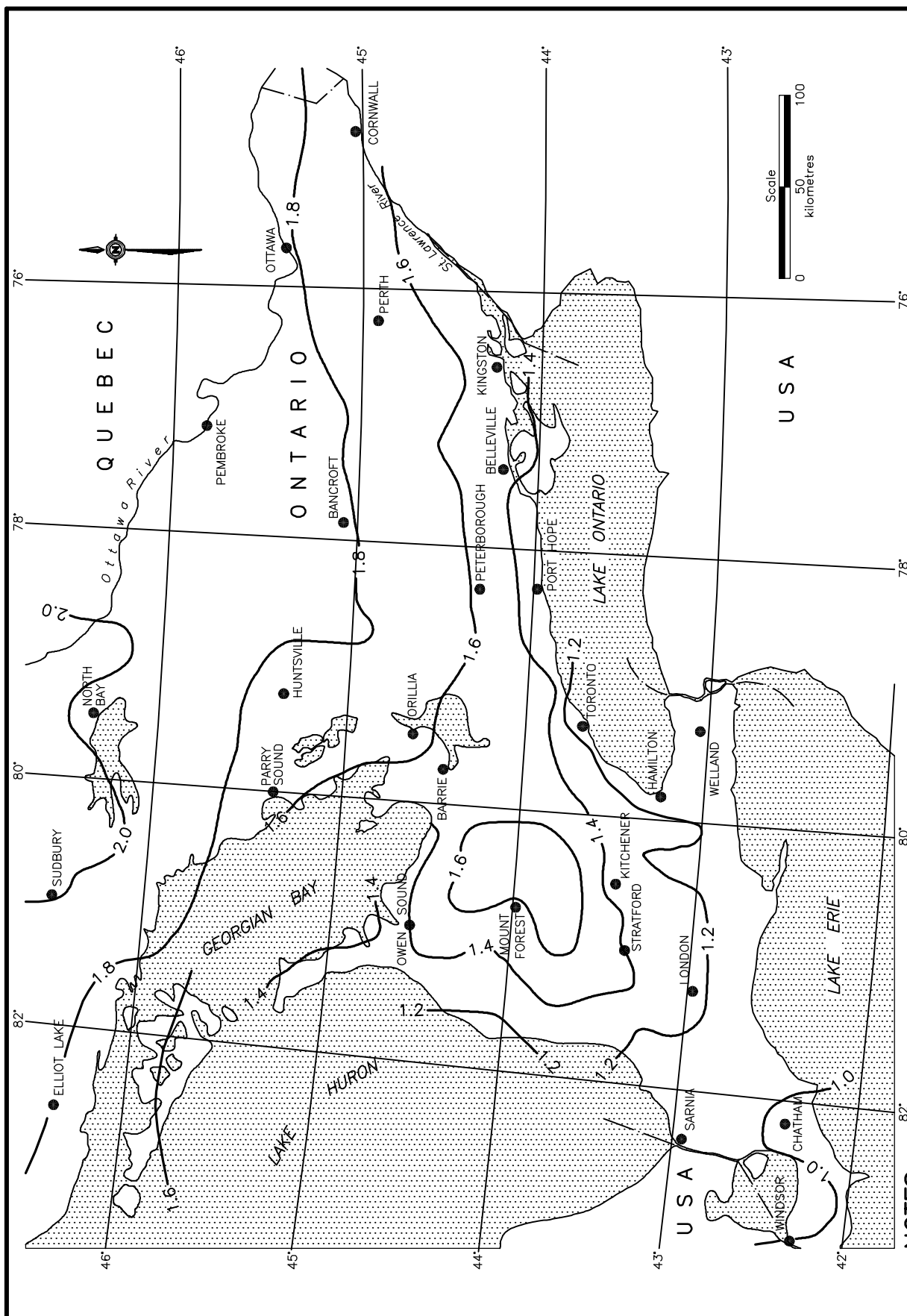
Nov 2013

Rev 2

GENERAL RIP-RAP LAYOUT
FOR SEWER AND CULVERT OUTLETS

OPSD 810.010





NOTES:

- A These values are approximate and should only be used where the recommendations of a geotechnical engineer are not available.
- B This information is based on the Ministry of Transportation and Communications Research Publication RR225 "Aspects of Prolonged Exposure of Pavements to Sub-Zero Temperatures" dated December 1981.
- C Values between contours should be interpolated. If interpolation is not possible, use the adjacent contour with the greater depth.
- D Frost penetration depths are in metres.

ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2010

Rev 1

**FOUNDATION
FROST PENETRATION DEPTHS
FOR SOUTHERN ONTARIO**

OPSD 3090.101



Appendix G – Results of Stability Analyses

Culvert Replacement by Cut and Cover Method
Hwy 118, Township of Stanhope
Embankment Slope Stability Analysis
Drained Static Condition

Name: Engineered Fill (Granular B Type II) Model: Mohr-Coulomb Unit Weight: 22 kN/m³ Cohesion': 0 kPa Phi': 35 °
Name: Gravelly Sand (Very Dense) Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion': 0 kPa Phi': 36 °
Name: Rockfill: Cobbles and Boulders Model: Mohr-Coulomb Unit Weight: 18 kN/m³ Cohesion': 0 kPa Phi': 40 °
Name: Bedrock Model: Bedrock (Impenetrable)

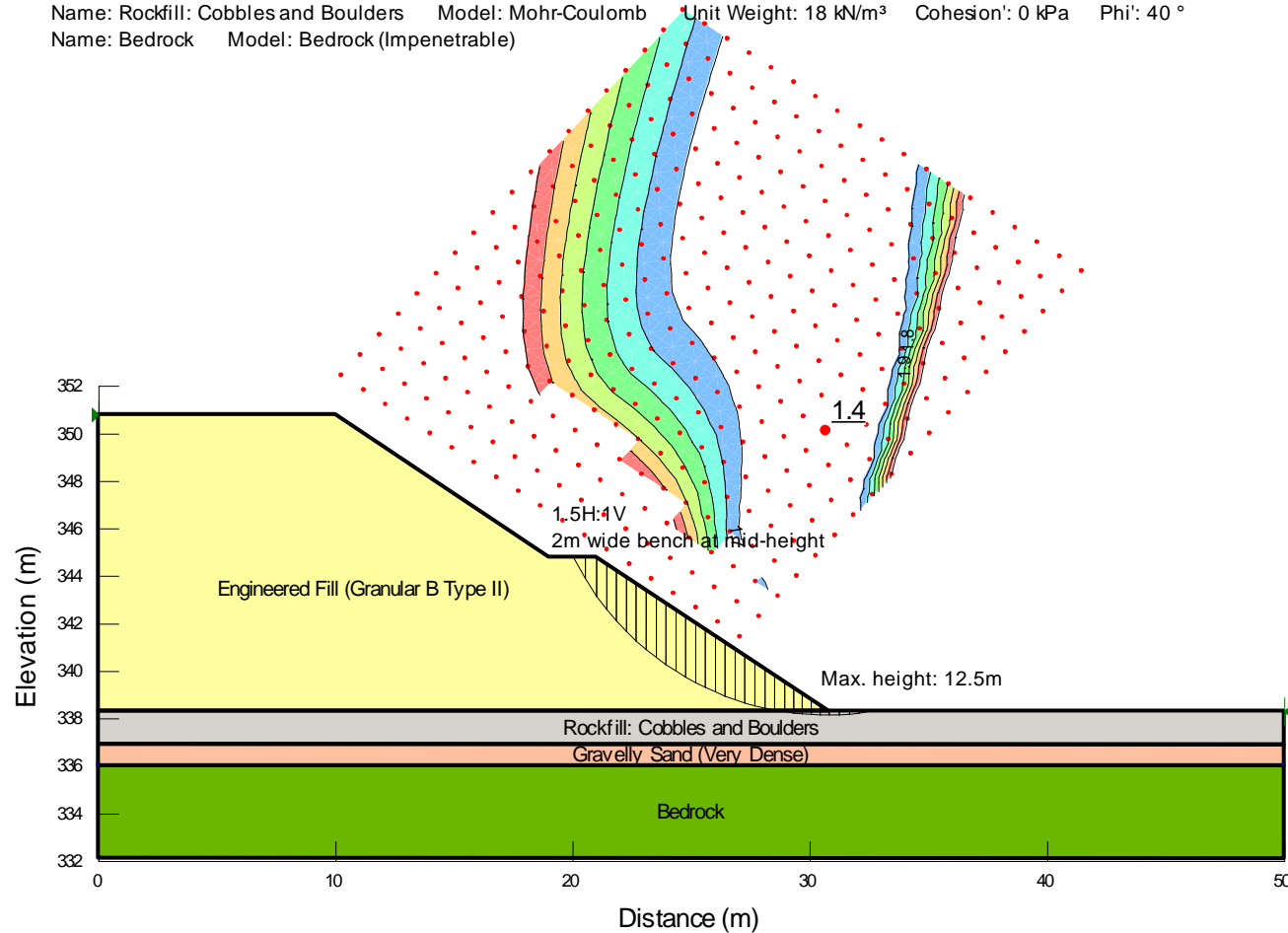


Figure 1: Slope stability analysis for new embankment constructed by cut and cover replacement method - drained static condition

Culvert Replacement by Cut and Cover Method
Hwy 118, Township of Stanhope
Embankment Slope Stability Analysis
Drained Seismic Condition

Name: Engineered Fill (Granular B Type II) Model: Mohr-Coulomb Unit Weight: 22 kN/m³ Cohesion': 0 kPa Phi': 35 °
Name: Gravelly Sand (Very Dense) Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion': 0 kPa Phi': 36 °
Name: Rockfill: Cobbles and Boulders Model: Mohr-Coulomb Unit Weight: 18 kN/m³ Cohesion': 0 kPa Phi': 40 °
Name: Bedrock Model: Bedrock (Impenetrable)

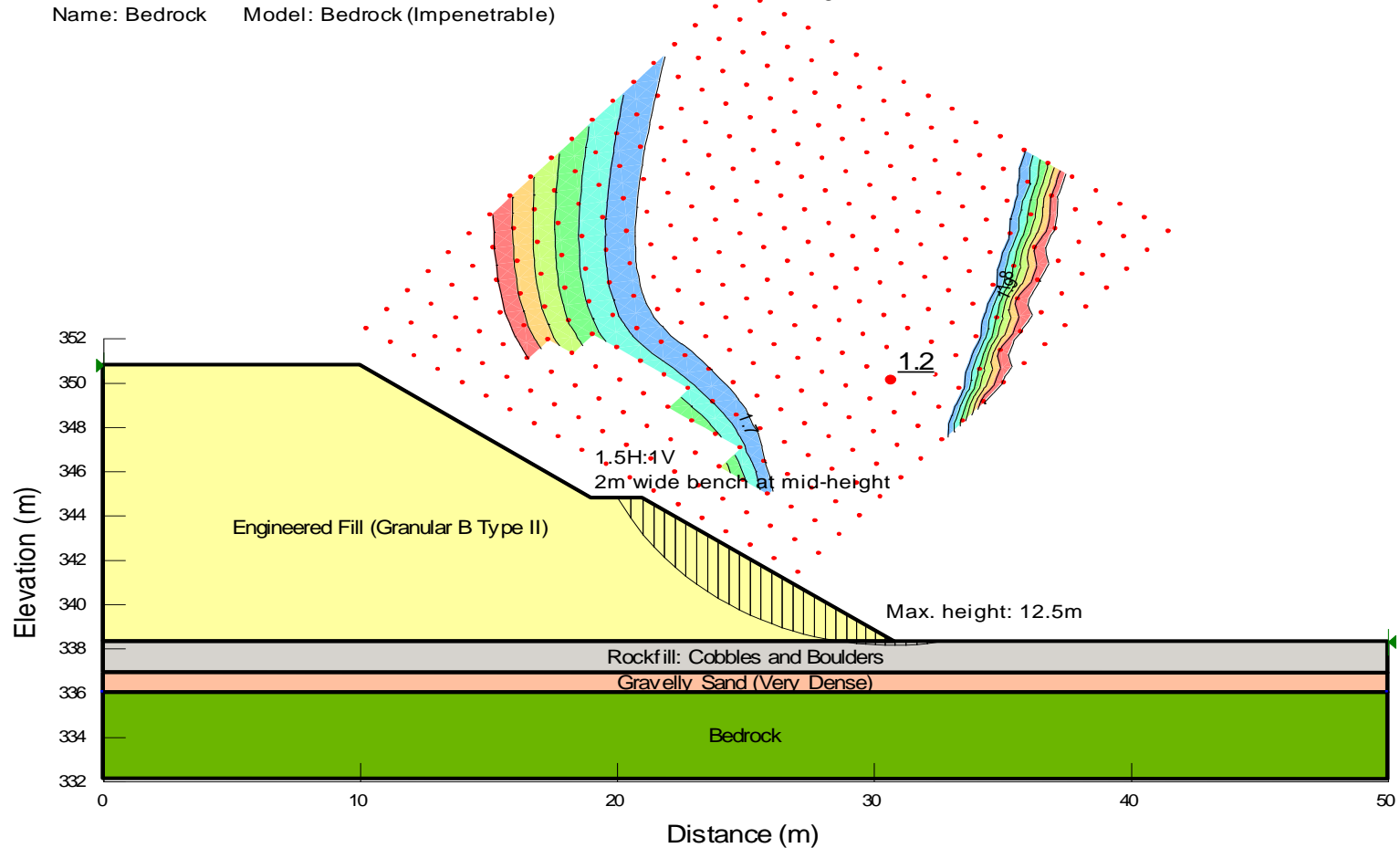


Figure 2: Slope stability analysis for new embankment constructed by cut and cover replacement method - drained seismic condition