



FINAL REPORT

FOUNDATION INVESTIGATION AND DESIGN REPORT **Trapp Creek Culvert (Site 46-329C), Hwy 101, Chapleau, Sault Ste. Marie Area**

Agreement No. 5013-E-0008

Assignment No. 8

WO 2015-11011

Geocres No. 410-13

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Foundation Investigation and Design Report

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Foundation Investigation and Design for Trapp Creek Culvert (Site 46-329C) Hwy 101, Chapleau, Sault Ste. Marie area

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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. for the detail design required at the Trapp Creek Culvert, located on Highway 101 near Chapleau in the Sault Ste. Marie Area. The site is approximately 140 m east of Highway 101/129 North Junction (LHRS 40430 o/s 15.22) at Sta. 10+138 IR No. 74A. the Ministry of Transportation (MTO) Northeastern Region. The work was undertaken under Agreement # 5013-E-0008, Assignment No. 8 (WO 2015-11011). The terms of reference (TOR) were as presented in the MTO letter dated April 2, 2015.

It has been reported by MTO that during construction, a settlement of $480 \pm \text{mm}$ was experienced on the north half of the culvert. The MTO Sault Ste. Marie Area Office has completed selective resurfacing to repair distortions within the fill embankment since the last rehabilitation in 2000. Prior to 2009, HMA patch was placed from approximately Sta. 10+150 to Sta. 10+210 in the WBL. Further selective resurfacing was completed in 2009 from Sta. 10+080 to Sta. 10+210 covering both EBL and WBL to address a longitudinal meandering crack spanning the length of embankment. In 2012, the Sault Ste. Marie Area office completed remedial work to repair a sinkhole that developed in the EBL within the east and west culvert haunches. As part of the 2012 repair, 30 bags of concrete were placed to fill voids around the culvert footing. Since 2012, settlements have redeveloped.

The purpose of this investigation is to evaluate/obtain the subsurface information at the culvert alignment and immediate approach embankment, verify the adequacy of the existing foundation at the culvert (in comparison to the assumed bearing capacity utilized for the initial culvert construction), investigate the cause of settlement and provide detailed recommendations with the appropriate alternatives to mitigate settlement and for the roadway distress treatment of Highway 101 from Sta. 10+080 to Sta. 10+210. 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 Trapp Creek Culvert site is located on Hwy 101 (Station 10+138 IR No. 74A) near Chapleau, approximately 140 m east of Hwy 101/129 North Junction. At this site Hwy 101 is two lanes, east/west roadway with a speed limit of 80 km/h and is about 7.3 m wide from edge of pavement to edge of pavement, with narrow sand and gravel shoulders and guardrails subsequently on both sides. It is estimated that the highway embankment at the investigated location is between 2.4 to 3 m high having side slopes of about 2.5H:1V. The location of the culvert and a cross section of the existing culvert alignment are shown on Drawing 1 in Appendix B.

According to TOR provided, the existing Super-Cor culvert was constructed on an 800mm deep x 1200 mm wide clear stone footing with a specified gradation of 12.5 mm to 100 mm, as part of contract 2000-0242. The steel footings were designed based on an assumed 200 kPa bearing capacity. Photographs of the site and inlet and outlet of the existing culvert are presented in Appendix A.

The surrounding terrain of the culvert location is relatively flat, with Hwy 101 sloping towards the culvert on east side of the culvert then flat towards the CPR track on the west side of the culvert. The area is densely covered with trees beyond the culvert flood plain. At the site location, creek water flows from south to north crossing Hwy 101 via culvert. During the field work, the flood plain of Trap Creek was full of water due to spring flood with water level as high as of Elevation 427.63 m. The water in the surrounding area was approximately 0.6 m to 1.24 m deep at the time; see photographs in Appendix A.

The general site conditions were assessed during the site reconnaissance in April, 2015. However, since the surrounding area of culvert was full of water, our observations were limited. It was observed that a portion of roadway along the culvert alignment and the WBL of Hwy 101 on west side of the culvert alignment was depressed forming a longitudinal crack approximately 1.7 m wide 20 m long from edge of pavement, along the embankment. However, longitudinal meandering cracks were also observed on east side and both (east/west) side of culvert alignment on WBL and EBL respectively but major depressions in the embankment were not observed in these areas. It was also observed that the surface of EBL roadway portion on east and west sides of the culvert alignment had been treated with asphalt. Based on this limited observation, it could be speculated that the roadway distress treatment was performed in the past. On the north slope of the embankment (i.e., outlet side), washed out surface fill material forming erosion channel and a pothole of depth about 0.3 m (measured in April 2015) was observed at the culvert location and at approximate Sta. 10+115. Selective photographs of the roadway distresses and slope distresses are presented in Appendix A.

At the time of site investigation, Trapp Creek was flowing freely and approximate elevations at the inlet and outlet were 427.74 m and 427.6 m respectively. The elevation of highway centerline pavement was 429.9 m.

1.2.2 Geological Setting

The Map 2543 (Bedrock Geology of Ontario, East-Central Sheet, 1991) of the Ministry of Northern Development and Mines, indicates that the bedrock formation of the project area is known to be in Neo to Mesoarchean Group, mainly of gneissic tonalite suite intrusive rocks comprised of minor supracrustal inclusions. The Map 2555 (Quaternary Geology of Ontario, East-Central Sheet, 1991) of the Ministry of Northern Development and Mines, indicates that the surface conditions in the vicinity of site consist of bedrock of undifferentiated igneous and metamorphic rock, exposed at surface of covered by discontinuous, thin layer of drift.

1.3 Investigation Procedures

1.3.1 Site Investigation and Field Testing

The field investigation was performed between April 27, 2015 and May 1, 2015. The field program consisted of drilling six (6) sampled boreholes (BH-1, BH-2, BH-3, BH-4, BH-5 and BH-6) and probing of surrounding and founding areas. The boreholes were strategically located along the existing culvert alignment and immediate approach embankments to provide subsurface information along the existing culvert and immediate approach embankments. BH-1, BH-2, BH-3 and BH-4 were advanced from the embankment crest and BH-5 and BH-6 were advanced (from the water using barge; see photographs 9 in Appendix A) at toe of embankment on outlet and inlet side of culvert, respectively. Among the boreholes drilled from embankment crest BH-1 and BH-4 were located at west/east approach embankment (approximately 25.75/25.9 m away from centerline of culvert alignment), respectively. Similarly, BH-2 and BH-3 were located about 9.75 m west/9.6 m east of centerline of culvert alignment within WBL/EBL, respectively. Two probe hole, PB1 (east of culvert alignment on WBL) and PB2 (west of culvert alignment on EBL) were advanced up to 7.62 m to confirm peat layer just beside the culvert foundation. Probe holes PB1 and PB2 were located about 5.5 m from the centerline of culvert alignment. The borehole locations are shown on Drawing 1 in Appendix B.

Boreholes drilled from the embankment crest (BH-1, BH-2, BH-3 and BH-4) were advanced using a truck mounted CME-55 drill rig, equipped with a hollow stem auger and standard soil sampling equipment operated by a specialist drilling contractor, Landcore Drilling Inc. Due to difficulties to access the inlet and outlet sides with the drill rig and the high water level in vicinity of culvert, boreholes at these locations (BH-5 and BH-6) were advanced using hand drilling/sampling equipment (a portable tripod with hammer) and barge operated also by Landcore Drilling Inc.

The boreholes drilled through the embankment BH-1, BH-2, BH-3 and BH-4 were advanced up to desired depths of 9.76 m, 20.43 m, 15.85 m and 9.76 m, respectively. BH-5 and BH-6 drilled through water using a barge at the toe of embankment were advanced to depths of 8.07 m and 11 m from water surface respectively.

The borehole locations (referenced to the MTM NAD83 coordinate system) and their ground surface elevations were surveyed by **exp** personnel. The ground surface elevations, including top of culvert and top of water, were referenced to a temporary bench mark assumed (top of Hwy 101 and CPR track crossing platform on south-east corner). Elevation of the bench mark (Elev. 430 m) was assumed based on the drawing sheet no. 29 (CONT NO. 200-0242, WP No. 467-00-01), provided by the MTO.

During the drilling of the boreholes, 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. One Shelby

tube samples was obtained in the peat layer. Since the conventional hammer of 63.5 kg was used for sampling done by a portable tripod, the corresponding blow counts were not factored.

Upon completion of the boreholes, ground water level measurements were carried out from the boreholes (BH-1 and BH-4) in accordance with the Ministry of Transportation guidelines. The measured ground water levels after completion of drilling boreholes were recorded on borehole log sheets in Appendix C. Since the wash boring technique was used to drilled boreholes (BH-2 and BH-3), the stabilized ground water level could not be established by short term observation in boreholes. BH-5 and BH-6 were drilled below the water level. The 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 staff who 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 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 for approximately 25% of the collected soil samples. Atterberg limits test were carried out for cohesive soils. Organic content test and consolidation test were carried out for peat soils. All of the laboratory tests were carried out in accordance with MTO and/or ASTM Standards as appropriate.

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

1.3.3 Previous Investigation

No foundation reports are available in the MTO GEOCRE library for this particular site. However, one foundation report from January 1966 related to the adjacent site on Hwy 129 and named "Soil Conditions and Foundations Proposed Nebskwashi River Bridge, Chapeau, Ontario" (WP # 102-65) is provided by MTO.

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 proposed culvert alignment consist of a layer of sand and gravel to gravelly sand fill underlain by native deposits of sand to silty sand layer followed by peat layer and sandy silt to sand layer. The subsurface conditions at the toe of the embankment (inlet and outlet) consist of native deposits of gravelly sand underlain by peat layer followed by silt to silty sand layer. A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

The results of the laboratory testing were plotted on the individual record of borehole sheets and also summarized on Figure 1 to 6 inclusive all of which are appended to this report.

The following are the detailed description of soil strata encountered.

1.4.1 Fill: Sand and Gravel to Gravelly Sand

Sand and gravel to gravelly sand layer was encountered at the road embankment below the 76 mm (BH-3) to 300 mm (BH-4) thick layer of asphalt in BH-1, BH-2, BH-3 and BH-4. The thickness of this layer ranged from 3.0 m to 3.3 m extending from Elevation 429.8 to Elevation 426.9 m.

The composition of this fill layer is sand and gravel with occasional cobbles, and trace to few silt and clay size particles. The material is brown to greyish brown in color, and moist to wet. The SPT “N” values within this layer typically ranged from 8 to 33 blows per 300 mm penetration, suggesting loose to compact relative density. One SPT “N” value of 60 blows per 50 mm penetration was encountered at BH-4.

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

Moisture Content:

- 1.7% to 16.2%

Grain Size Distribution:

- 21% to 62% gravel;
- 34% to 72% sand; and
- 4% to 7% 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.

1.4.2 Gravelly Sand

A layer of gravelly sand was encountered below the 0.6 m to 1.2 m high water table in BH-5 and BH-6. The thickness of this layer is approximately 0.8 m extending from Elevation 426.5 m to Elevation 425.8 m.

The composition of this layer is sand and gravel, trace silt and trace wood pieces. The material is brown to grey in color, and wet. The SPT "N" values within this layer ranged from 3 to 9 blows per 300 mm penetration, suggesting very loose to loose relative density.

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

Moisture Content:

- 11.8% to 18%

Grain Size Distribution:

- 31 % gravel,
- 68% sand, and
- 1% 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.

1.4.3 Sand to Silty Sand

A layer of sand to sandy silt was encountered below the sand and gravel to gravelly sand fill in BH-1, BH-2, BH-3 and BH-4. The thickness of this layer ranged from 2.0 m 3.3 m extending from Elevation 426.9 m to Elevation 423.3 m.

The composition of this layer is sand and silt, trace gravel and occasional cobbles. The material is brown to grey in color, and wet. The SPT "N" values within this layer ranged from 1 to 26 blows per 300 mm penetration, suggesting very loose to compact relative density.

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

Moisture content:

- 12.6% to 17.8%

Grain Size Distribution:

- 2% to 3% gravel,
- 68% to 90% sand, and
- 8% to 29% 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.

1.4.4 Peat

A layer of peat was encountered below the sand and silty sand layer in BH-1, BH-2, BH-3 and BH-4 and below gravelly sand layer in BH-5 and BH-6. The thickness of this layer ranged from 0.9 m to 2.0 m below the embankment extending from Elevation 424.7 m to Elevation 422.2 m and at inlet and outlet side the thickness of this layer ranged from 2.4 m to 2.7 m extending from Elevation 425.8 m to Elevation 423.0 m.

The peat consisted mostly of organics, some silt, and some sand, trace gravel and trace clay. The material is dark brown in color, and wet to saturate. The SPT “N” values within this layer measured below the embankment ranged from 4 to 7 blows per 300 mm penetration, suggesting very loose to loose relative density and the SPT “N” values within this layer measured at inlet and outlet side ranged from 1 to 3 blows per 300 mm penetration, suggesting very loose relative density.

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

Moisture Content:

- 55.2% to 381.5%

Grain Size Distribution:

- 0% to 20% gravel
- 32% to 62% sand
- 35% to 65% silt and
- 1% to 3% clay.

Organic Content Test:

BH No.	Sample No.	Moisture Content (%)	Organic content (%)
BH-1	SS-8	229.2	37.0
BH-3	SS-8	238.4	34.3
BH-4	TW-9	323.8	55.2
BH-5	SS-3	219.4	32.2
BH-6	SS-4	238.1	28.6

One Consolidation Test was performed on Shelby Tube sample (BH-4, TW-9) obtained underneath the embankment and the results are as follows:

- Moisture Content = 323.8 %
- Initial Void Ratio (e_0) = 5.318
- Pre-consolidation pressure (p'_c) = 115 kPa
- Recompression Index (C_r) = 0.459

- Compression Index (C_c) = 2.78

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 (Figure 4) and consolidation tests are also provided in Appendix D.

1.4.5 Clayey Silt

A layer of clayey silt was encountered below the peat layer in BH-3 and BH-4. The thickness of this layer ranged from 0.3 m to 0.6 m, extending from Elevation 422.9 m to Elevation 422.3 m.

The composition of this layer is silt and clay, trace peat and trace sand. The material is brown in color, and wet. The SPT "N" values within this layer ranged from 3 to 6 blows per 300 mm penetration, suggesting soft to firm in consistency.

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

Moisture content:

- 34.6% to 50.6%

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

1.4.6 Sandy Silt to Sand

A layer of sandy silt to sand was encountered below the peat layer in BH-1, BH-2, BH-5 and BH-6 and below the clayey silt layer in BH-3 and BH-4. The thickness of this layer ranged from 1.9 m to 12.8 m extending from Elevation 423.3 m to Elevation 409.4 m. All boreholes are terminated within this layer after reaching desire depth of investigation.

The composition of this layer is mainly sand and silt, trace peat, trace to some gravel, and trace clay and occasional cobbles. The material is brown to grey in color, and wet. The SPT "N" values within this layer ranged from 2 to 28 blows per 300 mm penetration, suggesting very loose to compact relative density.

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

Moisture content:

- 11.6% to 77.4%

Grain Size Distribution:

- 0% to 22% gravel,
- 10% to 97% sand, and
- 3% to 86% silt and
- 1% to 4% clay

Atterberg Limits:

- Selected samples (BH-2-SS9, BH-3-SS10 and BH-5-SS5) found non plastic

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 and Atterberg limit tests are also provided on Figure 5 and Figure 6 in Appendix D.

1.5 Ground Water Conditions

Information of groundwater levels at the site was obtained by measuring the water levels in the open boreholes (BH-1 and BH-4) after completion of drilling. The groundwater levels encountered in the boreholes are shown on the borehole logs. Since the wash boring method was used for drilling of BH-2 and BH-3, an accurate groundwater level at these holes was not able to be measured in the open holes at the time of drilling operations. BH-5 and BH-6 were drilled through the creek using barge and at the time of investigation (April 2015) the water level at BH-5 and BH-6 were 0.61 m and 1.24 m high at Elevation 427.2 m and 427.7 m respectively.

The water level measured at the time of investigation through the open boreholes (BH-1 and BH-4) was at Elevation 427.42 m and 427.36 m respectively. Water levels measured in open boreholes might not be stabilized due to short term observation. However, based on moisture content of the soil samples observed during drilling and measured subsequently in the lab, the inferred ground water level within the embankment was estimated to be at approximate Elevation of 427.7 m or slightly higher. This is in a good agreement with the water level observed in the culvert at the time of investigation which was at Elevation 427.74 m at the inlet and 427.60 m at the outlet.

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 ENGINEERING DISCUSSIONS AND RECOMMENATIONS

2.1 General

This section of the report provides geotechnical design recommendations for the possible remediation of the existing Trapp Creek Culvert situated beneath Hwy 101 and for the roadway distress treatment, approximately 140 m east of Hwy 101/129 North Junction (LHRS 40430 o/s 15.22) at Sta. 10+138 IR No. 74A, 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 foundation alternatives and design the proposed culvert and replacement. 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.

According to the contract drawings provided by MTO, the existing culvert below Hwy 101 is a Super-Cor culvert (MTO Contract Drawings 1 - 4, Contract 2000-0242, WP No. 467-00-01). The contract drawings do not explicitly show the dimensions of the culvert and an "as-built" condition drawing is not currently available to **exp.**, however, it was measured on the AutoCAD drawings (assuming that the drawings are in scale) that the culvert was designed with a span of 8.72 m and rise of 2.62 m. Based on Contract Drawing 4, an open footing of the culvert was placed on a 0.8 m thick layer of crushed rock (having a specific gradation of 12.5 mm to 100 mm) leveling pad, which is approximately 1.2 m wide and 23 m long (Contract Drawing 4, Contract 2000-0242). We understand that the steel footings were designed based on an assumed 200 kPa bearing capacity of the foundation soil, noting that this does not necessarily indicate that the design loads imposed this level of stress. No geotechnical data exists from this site at the time of initial construction.

We understand that the culvert was rehabilitated in 2000 and during the construction a settlement of $480 \pm$ mm was experienced on the north half of the culvert. It is reported in the TOR that the selective resurfacing to repair distortions within the fill embankment were completed by the MTO Sault Ste. Marie Area Office since the last rehabilitation in 2000. Prior to 2009, HMA patch was placed from approximately Sta. 10+150 to Sta. 10+210 in the WBL. Further selective resurfacing was completed in 2009 from Sta. 10+080 to Sta. 10+210 covering both EBL and WBL to address a longitudinal meandering crack spanning the length of embankment. In 2012, the Sault Ste. Marie Area office completed remedial work to repair a sinkhole that developed in the EBL within the east and west culvert haunches. As part of the 2012 repair, 30 bags of concrete were placed to fill voids around the culvert footing. Since 2012, settlements have redeveloped.

It is also reported by MTO that the water channel turns to the west at the outlet end of the culvert and goes back into the toe of existing slope, undermining the corduroy at the toe of slope.

Considering the discussion above, an investigation was commenced between April 27 and May 1, 2015 at the site to determine subsurface conditions below the existing culvert and highway

embankment with the aim of determining the causes of distress and to recommend the necessary remedial measures for the culvert and embankment. The investigation included: visual inspection at the culvert and immediate approaches by a geotechnical engineer, drilling four boreholes along the embankment within the distressed area (from Sta. 10+080 and Sta. 10+210) and two boreholes at the inlet and outlet of the culvert with soil sampling, *in-situ* testing and laboratory testing. The factual results of this investigation are presented in Part 1 "Foundation Investigation" of this report.

This foundation design report includes: (i) assessment of soil foundation conditions in the area of locations of the existing culvert and immediate approaches including the bearing capacity of the foundation soil and settlement and stability of the culvert and embankment, (ii) discussion about the possible causes of settlement based on current investigation findings, and (ii) recommendation for remedial measures. Preliminary examination of possible alternative solutions to the existing scheme is also presented for discussion and development of the remediation strategy.

2.2 Assessment of Existing Conditions

2.2.1 Foundation Soil

Based on the results of the current investigation it was found that the embankment of Hwy 101 at that location is approximately 3.0 m high, consisting of sand and gravel to gravelly sand fill. A layer of loose to compact sand to silty sand was found underneath the fill. The thickness of this layer varied from approximately 3.6 m close to the culvert (~10m from the culvert centre line) to approximately 2.1 m at the location approximately 25 m distanced longitudinally from the culvert centre line. Below that layer, peat was encountered in every borehole drilled. As shown on Drawing 2 and 3 (Appendix B), peat thickness of approximately 1.0 m was measured beneath the embankment adjacent to the culvert, while that thickness increased laterally and longitudinally from the culvert. Below the embankment, at the location approximately 25 m from the centre line of the culvert, the peat was found to be about 2.1 to 2.3 m thick. Beyond the embankment footing, at inlet and outlet locations, it was found that the peat layer was 2.4 m to 2.8 m thick.

Assuming the founding levels are at the bottom of the existing embankment, the Factored Geotechnical Resistance at ULS and the Geotechnical Reaction at SLS are estimated based on the findings in this investigation and the values are given in the table below.

Table 1 Estimates of Factored Geotechnical Resistance at ULS and Geotechnical Reaction at SLS

Foundation Level	Foundation Soil	Geotechnical Resistance at ULS	Geotechnical Reaction at SLS (for total settlement of 25 mm)
Elev. 427 m	Compact Sand to Silty Sand over Very Loose Peat	150 kPa	75kPa

As mentioned in Section 2.1, the footings of the existing culvert were designed based on an assumed 200 kPa bearing capacity of the foundation soil. However, as noted in Table 2 the Factored Geotechnical Resistance at ULS and Geotechnical Reaction at SLS of the foundation soil

at Elev. 427 m are estimated to be less than that required in the design drawing (equivalent 200 kPa geotechnical reaction at SLS). In making this comment it should be noted that the actual loading condition may be less than the generic required bearing capacity stated on the drawing although this should be confirmed by the structural designer.

2.2.2 Super-Cor Box Culvert and Approach Embankment

At the time of this geotechnical investigation, the cover over the existing Super-Cor box culvert was measured as well as the span and rise of the culvert opening at the inlet and outlet sides. It was measured that the cover currently over the culvert was approximately 1.0 m at the inlet side and 1.2 m at the outlet side. At the inlet side, the span of the culvert at the footing level was measured to be 9.3 m and the rise from footing to the crown, 2.55 m. At the outlet side, the span of the culvert was also 9.3 m, but the rise was 2.18 m. As mentioned in Section 2.1, it appears that the Super-Cor box culvert was designed with a span of 8.72 m and rise of 2.62 m based on the contract drawings, although this would not be verified by “as-built” drawings. However, at the time of this geotechnical investigation measurements on site indicate that the dimensions were significantly different from original plans, likely reflecting some movement since construction of the culvert. Evidence of structural instability of the culvert was not observed during our site visit. However, inspection and assessment by a structural engineer is recommended to assess this condition.

Significant cracks in the WBL were observed with 40 to 60 mm deep depressions, approximately 1.7 m from edge of pavement along approximate STA 10+100 to 10+ 200. (Photographs 12, Appendix A). The cracks were approximately 5 to 10 mm wide. A longitudinal meandering crack on the east side of culvert alignment on WBL spanning approximately from STA 10+150 to 10 + 160 and on both sides (east/west) side of the culvert alignment on EBL was also observed (Photographs 13 and 14, Appendix A). However any bulging at the toe was difficult to observe due to high water level and vegetation.

It is reported by MTO that the water channel turns to the west at the outlet end of the culvert and goes back into the toe of existing slope undermining of the corduroy at the toe of slope. This erosion was not possible to be observed at the time of this geotechnical investigation, since the high water level precluded access to the toe.

2.2.3 Assessment of Settlement and Stability

In examining settlements under these conditions, a number of interpretations and judgments had to be used in the assessments due to the nature of the problem, the completeness and accuracy of historical information and nature of the subsurface conditions. Accordingly the findings should be considered in that context.

2.2.3.1 Settlement

Assuming that the initial thickness of the peat layer was 2.5 m, as encountered at the site outside of the embankment footprint (BH-5 and BH-6), the calculation of consolidation settlement due to adding a 3.0 m high embankment was carried out using a commercial software, Settle 3D by

Rocscience Inc. The objective of this analysis was to assess the settlement that had already occurred at the site based on soil properties encountered in this investigation, and the amount of remaining settlement that might be expected. Peat properties used in this analysis are presented in Table 2.

Table 2. Soil parameters used in settlement analysis

Peat (Normally Consolidated)	
Thickness of peat layer (m)	2.5
Unit weight, γ_{sat} (kN/m ³)	11
Void ratio e_0	5.318
Compression index, C_c	2.78
Recompression index, C_r	0.458
Coefficient of consolidation, C_v (cm ² /s)	2×10^{-5}

The value of the consolidation indices given in Table 2 were estimated based on the results of a consolidation test performed on a peat sample recovered in BH-4 (see Appendix D).. From these results the preconsolidation pressure (σ'_p) of 115 kPa on the peat layer was estimated.

The result of time-dependent consolidation settlement analysis is presented in the figure below (Figure 1). Also, results of 3 D settlement are included on Figure F1 in Appendix F. The results of this analysis suggest that consolidation settlement of about 570 mm would occur in the fifteen years after the culvert and approach embankment was rehabilitated (i.e. year 2000), which is generally consistent with what was observed on the site. In BH-4 (~ 25 m away from the culvert central line) it was found that the thickness of peat below the embankment at that location was reduced by approximately 400 to 700 mm relative to the peat thickness encountered outside of the embankment footprint. Similar interpreted compression of the peat layer was found in BH-1. In BH-2 and BH-3 (~ 10 m away from the culvert center line), that compression of the peat could be even greater, since the difference of thickness of the peat layer is in the range of 1200 to 1500 mm. This observation would infer a higher compression index for the peat layer i.e, about two times the laboratory value, a value consistent with the ranges in the literature.

According to the results of settlement analyses, it is estimated that an approximately 570 mm consolidation settlement occurred within the period of last 15 years, which is approximately 90% of estimated total settlement of approximately 640 mm which could be expected in a 35 years period under the existing loading condition. Therefore, it is estimated that the remaining settlement of the existing embankment at the location of BH-4 could be around 70 mm. Similarly, at the locations of BH-2 and BH-3 it is predicted the remaining settlement could be between 100 mm and 150 mm.

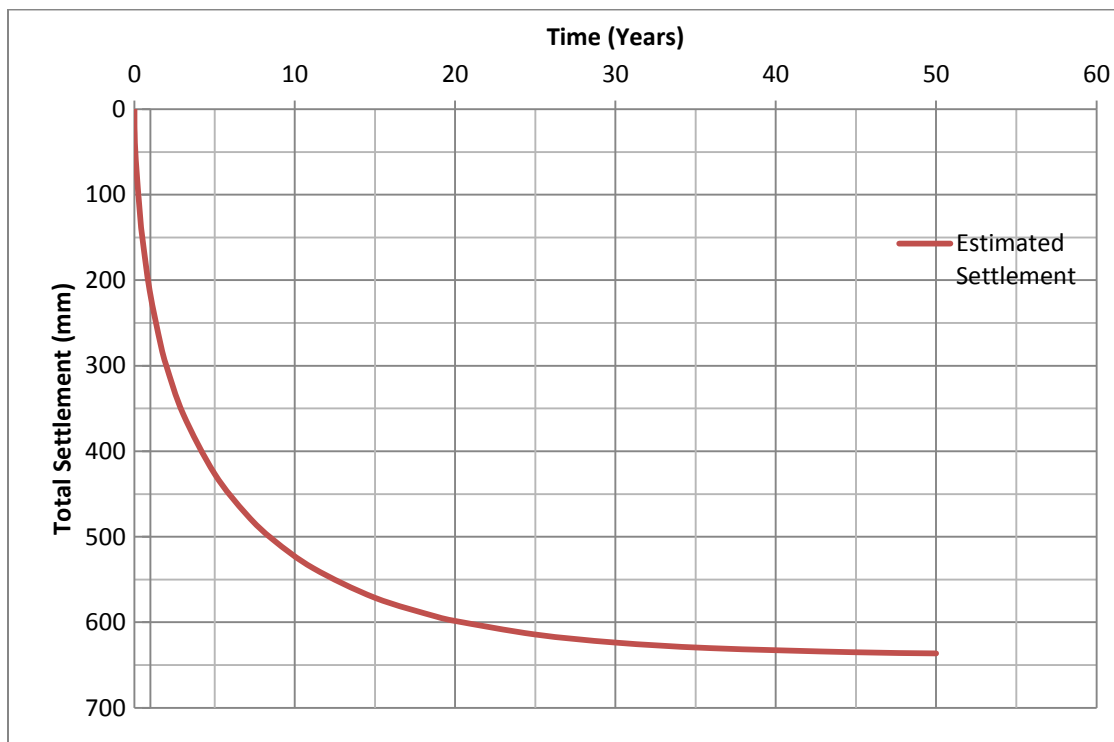


Figure 1. Result of the settlement analysis –estimated settlement of the peat layer under current condition at location of BH-4

2.2.3.2 Slope Stability

The global stability of the existing embankment at the culvert location was assessed using the conventional limit equilibrium method proposed by Morgenstern-Price and assuming a circular failure mode. The SLOPE/W computer program developed by GeoSlope International was employed for computation. Considering the time when the embankment was constructed, effective stress (long term) analyses were undertaken to assess the stability of the existing embankment. The stratigraphy at the site was developed primarily based on the results of the geotechnical investigation presented in Part 1 “Foundation Investigation Report”. The geometry of the existing structures was developed based on measurements on the site.

Figures in Appendix E show the results of global stability analyses at two locations: (i) at Section B-B shown on Drawing 2 in Appendix B and (ii) at Section D-D shown on Drawing 3 in Appendix B. As can be seen the minimum factor of safety (FOS) at both locations is around 1.0 (using conservative parameters), suggesting that the existing embankment slope is at the verge of instability. The cracks noticed in the pavement also can support this finding, noting also signs of erosion at the toe of the north embankment which are reported. Unfortunately, the area was flooded at the time of the investigation, so any detailed inspection of this erosion or bulging at the toe of the embankment was not possible. The meandering of the creek toward the west after its exit at the outlet also might contribute to the instability of the embankment at that location. In

assessing remediation strategies some toe protection suitably designed can be used to enhance stability and to mitigate against loss of fines (see Section 2.4)

2.3 Possible Causes of Distress

Two key issues likely precipitated the observed distresses at the site. The presence of the buried peat layer would explain the excessive settlement experienced. The extent of the layer of peat buried below the native superficial layer of sand to silt sand suggests that peat existed at the site before the culvert and embankment construction. Placing the load caused the compression of the peat layer and subsequent excessive settlement of the culvert and adjacent embankment. Further, in reviewing the foundations for the structure, it is apparent that the available bearing capacity at founding levels was significantly less than that required by the generic design. Section 2.2.1 provides the pertinent bearing values available based on current testing and assessment. These values do not satisfy the requirement of 200 kPa allowable stress stated on the design drawings.

The comparison of the opening size of the existing culvert with those provided on the contract drawings, also suggests that some changes might have occurred. These changes in the opening size infer that the footings of the culvert moved laterally, causing further instability and settlement of the highway surface. Without precise dimensions of “as-built” conditions, the actual structural distortion cannot be defined with certainty; nevertheless some movement is apparent based on interpretation and measurements using the available information.

In addition, and noting the “sinkhole” since filled, there is the possibility of loss of fines from coarse granular infiltration into the foundation pad or seepage at the toe.

2.4 Recommendations for Remedial Measures

The scope of work for this assignment was centered on the assessment of possible reasons for distress and geotechnical approach to rehabilitation of the existing culvert and approaching embankments. The following discussions are limited to geotechnical considerations; it is assumed that the final culvert assessment and rehabilitation scheme will be completed by MTO structural engineers or other qualified structural personnel designated by MTO, in accordance with applicable codes and standards related to this application. This includes structural evaluation of the existing structure if it is to remain. All requirements to satisfy hydraulics and erosion control requirements must be similarly addressed as well.

In cases that embankment is not completely reconstructed, adequate embankment toe treatment is recommended to improve embankment slope stability, mitigate further loss of fines and control erosion. The toe treatment could include installation of rockfill and/or sheetpiles at the toe in the critical areas. In particular, sheetpiles could be installed between Sta. 10+110 and Sta. 10+130 at the outlet side where the water channel goes along the embankment toe, while in the other areas of the critical zone between Sta. 10+080 and Sta. 10+210 rockfill could be placed at the toe. Further, it is recommended that the roadway distress treatment within the limits between Sta. 10+080 and

Sta. 10+210 includes full width excavation to a 300± mm depth, installation of a geogrid to prevent lateral support and reinstatement of the pavement matching to existing elevations.

2.4.1 Possible Remedial Approaches

Based on preceding assessments and discussion, three approaches can be considered for rehabilitation/reinstatement of the culvert/approaching embankment as outlined below. In Options 1 and 2 it is assumed that the existing structure can remain with required in-situ enhancements, if any, carried out in accordance with instructions from the structural designer.

- (i) Option 1: Allow culvert/embankment to continue to settle until 100% consolidation of the underlying peat layer is reached (estimated time: 35 years) and provide regular maintenance in the form of repair and resurfacing during that period. Provide erosion/filter protection at toe of embankment to mitigate any loss of fines and to enhance stability. Assess the existing culvert for structural integrity and repair/stabilize structure and/or foundations, if and where required.
- (ii) Option 2: Repair existing embankment by using lightweight fill (i.e. geofoam, cellular concrete or slag) to reduce the load and subsequently the remaining consolidation settlement of the underlying peat layer. Provide erosion/filter protection at toe of embankment to mitigate any loss of fines and enhance stability. Assess the existing culvert for structural integrity and repair/stabilize structure and/or foundations, if and where required.
- (iii) Option 3: Rebuild structures including culvert and embankment. Include foundations that conform to available bearing and assess any measures required to maintain settlements within OPS criteria for this type of roadway.

2.4.1.1 Option 1: Allow Culvert/Embankment Settlement

As mentioned above, the current culvert and approaching embankments have suffered distress. A structural assessment needs to be performed to make the final decision for the appropriateness of the culvert functionality in its current condition and how it might further be affected by any remaining settlements. This requires evaluation by the structural engineer. Assuming the structure is serviceable, one of remedial approaches could be to allow the culvert/embankment settlement to continue until 100% consolidation is reached. The predictions are that the remaining consolidation settlement could be in the range of 70 mm to 150 mm at and away from the culvert within the next 35 years. This approach will require constant instrumentation and monitoring and resurfacing of the highway periodically as recommended above. At least ten surface settlement pins should be installed on the critical pavement sections in the area (i.e. between Sta. 10+080 and Sta. 10+210), and the culvert should be periodically inspected for structural adequacy.

This approach will not improve the global stability of the existing embankment unless treatment of the toe is also included. The toe treatment could include installation of rockfill and/or sheetpiles at the toe in the critical area as mentioned in Section 2.4. The slope stability analyses for Section B-B

and Section D-D (see Drawings 2 and 3, respectively, in Appendix B) with sheetpiles and rockfill installed at the embankment toe, respectively, show that the minimum FOS could be increased by toe treatment as suggested in Figure E3 and E4 in Appendix E.

2.4.1.2 Option 2: Remedial Measures Using Lightweight Fill

The second option involves the reducing of the existing overburden pressure on the underlying peat by replacing approximately 1.0 m to 2.0 m of the existing earth embankment fill with lightweight fill between Sta. 10+080 and Sta. 10+210. Lightweight fills considered should be on MTO's Designated Sources for Material (DSM) list and conform to use and placement guidelines/directives. EPS-Expanded Polystyrene (Geofoam) with unit weight of approximately 1.0 kN/m^3 , cellular concrete with unit weight of approximately 5 kN/m^3 or slag with unit weight of approximately 13 kN/m^3 could be used as fill material.

The settlement analysis assuming that 1.0 m of the existing fill is replaced by 1.0 m of different lightweight fills was performed and results are plotted on the figure below (Figure 2).

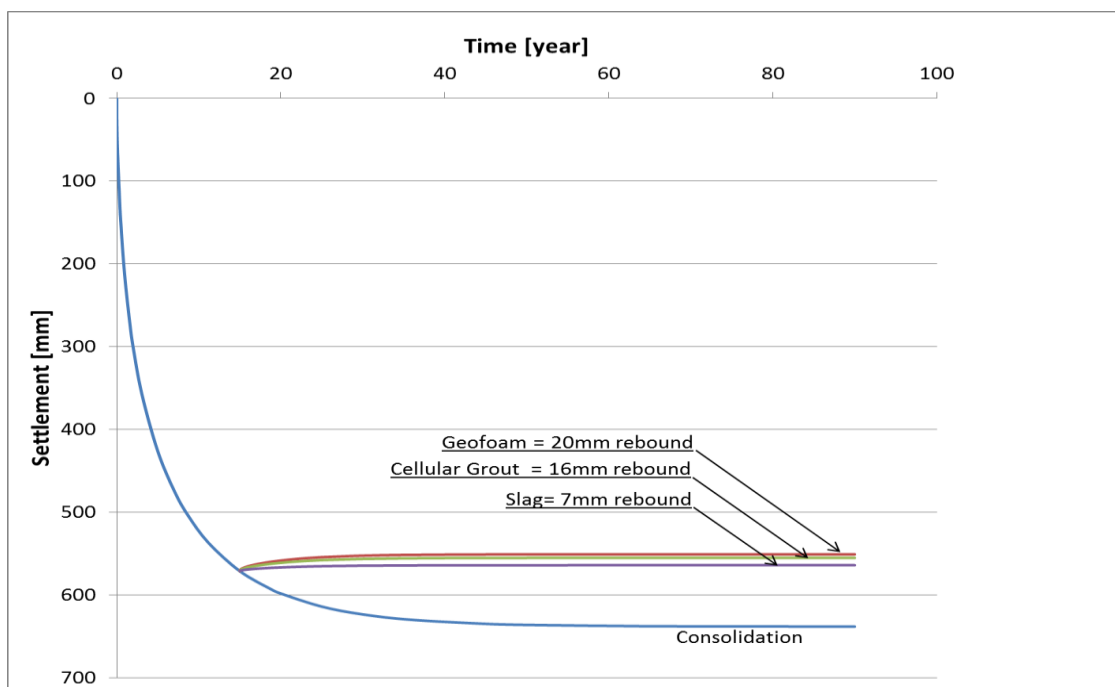


Figure 2. Result of the settlement analysis –estimated settlement of the peat layer after replacing 1.0 m of the existing embankment fill with lightweight fill at location of BH-4

As can be seen the calculated rebound due to reduction in the embankment load could be around 20 mm, 16 mm or 7 mm if the geofoam, cellular concrete or slag is used, respectively. Therefore,

these results suggest that replacement of the existing fill with the lightweight fill can cease the remaining consolidation settlement of the peat layer.

The performance of some lightweight materials such as geofoam and cellular concrete might be affected by higher elevated water levels (i.e. embankment being flooded). The geofoam/cellular concrete fill must be designed for an appropriate flood return period and concomitant flood elevation, which should be provided by a hydrologist. If the lightweight fill has to be placed below the flood elevation, a certain dead load has to be placed over the lightweight fill to counteract potential buoyancy. That load cannot be excessive because the large dead load is working against the reason that lightweight fill was considered necessary in the first place. Alternatively anchors could be considered. If the lightweight fill is above the flood water it is estimated that a minimum cover of 800 mm Granular B Type II will be required.

It is recommended that until remediation works have been completed, the settlement should be closely monitored. A minimum of five surface settlement pins should be installed on the critical pavement sections in the area.

Toe treatment to mitigate further loss of fines and to control erosion, and thus, improve the global stability of the slope, is also recommended. This is discussed in Section 2.4. The toe treatment could include installation of sheetpiles at the toe in the critical area (i.e between Sta. 10+110 to Sta. 10+130) at the outlet side where the water channel goes along the embankment toe to increase the resistance to movement and provide future erosion protection. In the other areas rockfill at the toe could be placed.

The slope stability analyses for Option 2 and toe improvement were performed for Section B-B and Section D-D (see Drawings 2 and 3, respectively, in Appendix B) and the results are presented in Appendix E. As can be seen on the figures the global stability of the embankment could be significantly improved by placing the sheetpiles at the embankment toe in Section B-B (Figure E5; min FOS=1.6) and/or rockfill in Section D-D at the embankment toe (Figure E6; min FOS=1.3).

If geofoam is used the NSSP "Expanded Polystyrene Embankment" attached in Appendix G should be applied.

2.4.1.3 Option 3: Rebuild Structures

Considering the subsurface conditions with a layer of compressible peat and unfavourable experience with the current structures including the culvert and embankment, an option to rebuild the structures on a new properly prepared subgrade could be also considered.

Since the design and construction of the new embankment fill is governed by the strength and consolidation properties of the compressible peat underlying the embankment, new embankments would have to be planned and constructed in stages. Use of prefabricated vertical drains and surcharge to reduce the time of settlement would have to be considered.

The other option is to excavate the peat layer and replace it with less compressible material. This option will require temporary closure of the road and massive earth works. Significant problems could be encountered during the excavation. A suitably designed dewatering scheme and the cofferdams will be required. Given these issues, this option is not considered practical or economically viable.

2.4.1.4 Review of Recommended Options

Advantages and disadvantages of three approach options discussed in preceding sections are summarized in Table 3. Considering all advantages and disadvantages discussed above, Option 2, replacement of the existing fill with the lightweight fill together with protection of the toe of the slope, providing that the lightweight fill is appropriately designed, is assessed as the most practical and economical solution for this site. Options 1 and 2 require an evaluation of the existing structure by a structural engineer prior to further actions. Option 3 is assessed as the least practical and feasible, and therefore, it is not elaborated in detail in this report.

Table 3. Possible remedial approaches

Remedial Approach	Advantages	Disadvantages	Ranking
Option 1*: Allow culvert/embankment settlement and stabilize embankment toe	<ul style="list-style-type: none"> o Use existing culvert and embankment o The least expensive o Assumes existing culvert remains 	<ul style="list-style-type: none"> o Settlement continues o Requires confirmation of structural adequacy of culvert o Requires resurfacing periodically o Requires constant monitoring o Requires monitoring 	2
Option 2*: Remedial measures using lightweight fill and stabilize embankment toe	<ul style="list-style-type: none"> o Use existing culvert and embankment o Reduce settlement o Assumes existing culvert remains o Less expensive than Option 3 	<ul style="list-style-type: none"> o Requires lightweight fill o Requires confirmation of structural adequacy of culvert o Buoyancy issue if geofoam or cellular concrete are used o Requires monitoring o Dewatering might be required 	1
Option 3: Rebuild structures and control embankment settlement	<ul style="list-style-type: none"> o Improved subgrade o structures more tolerable to settlement o More stable structures o Possibility of use of deep foundation 	<ul style="list-style-type: none"> o Massive earth work o Requires construction of cofferdam and dewatering o The most expensive o Waste of all existing structures o Requires the new bridge and embankment 	3

*-Requires evaluation of the existing structure by a structural engineer prior to further actions

The final approach is contingent on whether the existing structure is sufficiently integral to remain or can be salvaged/repared in place. This decision must be sanctioned by the structural engineer. Under this circumstance Option 2 is judged to be the preferred approach. On the other hand, if it is determined that the structure must be replaced, consideration of alternative structures set on deep foundations or scheme such as sheetpile abutment with pre cast decks can also be considered. To

support this, more detailed foundation assessment possibly including additional field investigation may be required.

2.5 Construction Considerations

Based on the above assessment Option 2 is considered most feasible subject to confirmation of the structural adequacy of the existing culvert. Some preliminary comments of construction issues for Option 2 follow.

2.5.1 Construction Alternatives

For the replacement of the existing fill with lightweight fill the following methods can be considered as possible alternatives for fill installation at this site:

1. Full closure of the road followed by open cut excavation to replace the embankment fills. For the traffic an available local road could be considered for use as a temporary detour.
2. Constructing a local temporary detour followed by open cut unsupported excavation the existing culvert could be used for maintenance and diversion of surface water flow during the construction. This method could be very expensive due to high costs to build the detour road.
3. Staged construction with shoring system (i.e. half-and-half construction) along the centerline of the road with an one-way signaled traffic. The construction sequences for this method may include: (i) one lane of the highway is utilized while the other lane of the highway embankment is excavated; (ii) rebuilt the embankment with the lightweight fill to grade in this side; and (iii) the traffic could be moved onto the new fill and the process is repeated to complete the construction. In this case approach to ensure integrity of geofoam fill must be developed, including anchoring for buoyancy if needed.

2.5.2 Temporary Shoring

Temporary excavation support systems, if any, should be designed and constructed in accordance with OPSS 539. The lateral movement of the temporary shoring system should meet Performance Level 2 as specified in OPSS 539.

2.5.3 Dewatering

Temporary dewatering is the responsibility of the contractor. However, for general guidelines covering the probable scenarios, the following information is provided.

It is anticipated that sheet piled cofferdams will be used at the culvert locations, and the sheetpiles will act as a form of groundwater control. The depth of sheetpile penetration should be such that the risk of piping adjacent to the sheetpiles be minimized. Pumping inside the cofferdams will required to maintain a dry base to facilitate foundation construction.

Filtered sumps must be designed properly so that construction drainage water containing eroded soils and fines does not flow into the creek.

It is not expected a temporary MOE permit to take water (PTTW) will be required for this site as the groundwater pumping rate shall not exceed 50,000 liters/day during the construction period. This is contingent on effective diversion of water flow in the stream or cutoff flow via a cofferdam constructed upstream of the works area.

As indicated, the design of dewatering systems for the excavations is responsibility of the Contractor who is expected to retain dewatering specialists for this task and submit plans for prior approval by MTO.

2.5.4 Instrumentation and Monitoring

In the case of Option 2 Settlements should be monitored during and after construction to ensure compliance with MTO guidelines and the contract requirements. Surface survey pins should be installed immediately after completion of embankment within the critical area (between Sta. 10+080 and Sta. 10+210). The survey pins can be in the form of a steel bar with a round top installed into the ground in the middle of the highway and/or edges of the road. An average of at least two readings should be taken to establish the initial conditions. A minimum of one (1) set of reading should be taken daily during construction and work stoppages. The monitoring should be extended weekly after the construction completion for at least 10 weeks to demonstrate that future settlement trended to acceptable conditions.

2.5.5 Erosion Protection

Erosion/scour protection should be provided at the culvert inlet and outlet (including the side slopes). The erosion/scour protection should be designed by a specialist River Engineer/Scientist (as erosion and scour largely depend on the velocity of water in the watercourse and its regime), who is familiar with the findings of this report.

2.5.6 Frost Protection

A frost penetration depth of approximately 2.4 m can occur in open unheated areas of Chapleau without snow cover. At the culvert inlet and outlet, and beneath the proposed culvert, the native soils consist of a sand to silty sand material. The silty sand material has a high frost susceptibility based upon the MTO Frost Classification guideline of percent particles between 5 and 75 μm . Therefore, non-frost susceptible materials such as sand and gravel (Granular "A") and/or soil cover/polystyrene insulation need to be provided to provide the necessary frost protection over the footing areas.

August 10, 2015

3 CLOSURE

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

This Foundation Investigation and Design Report has been prepared by Silvana Micic, Ph.D., P.Eng. and Nimesh Tamrakar, M.Eng. and reviewed by TaeChul Kim, M.E.Sc., P.Eng. and Stan E. Gonsalves, M.Eng., P.Eng., Designated MTO Foundation Contact. The field investigation was conducted by Nimesh Tamrakar, M.Eng.

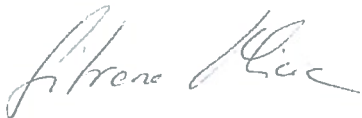
We trust that these comments provide you with sufficient information to proceed with design. Should you have any questions, please do not hesitate to contact this office.

Yours truly,

exp Services Inc.



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Encl.





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

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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. Facing west on Hwy 101 from the culvert alignment



Photo 2. Facing east on Hwy 101 from the culvert alignment



Photo 3. Facing south on inlet side of existing culvert



Photo 4. Facing north on outlet side of existing culvert



Photo 5. Outlet side of existing culvert facing south



Photo 6. Inlet side of existing culvert facing north



Photo 7. Embankment slope on south side (inlet) facing east



Photo 8. Embankment slope on north side (outlet) facing east



Photo 9. Barge with Portable Tripod



Photo 10. Washed away embankment slope on outlet side facing west



Photo 11. Washed away portion of embankment slope on outlet side



Photo 12. WBL roadway portion depressed on north-west half of embankment



Photo 13. Typical crack on roadway along the WBL facing west



Photo 14. Typical crack on roadway along the EBL facing west

Appendix B – Drawings

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

Agreement No. 5013-E-0008
Assignment No. 8
WO 2015-11011

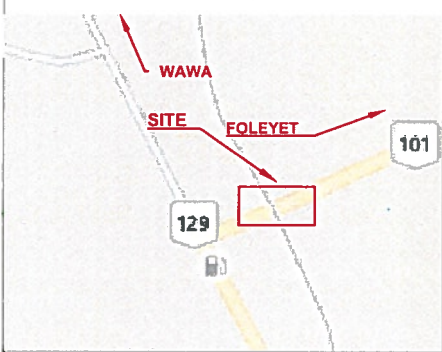


TRAPP CREEK CULVERT REPLACEMENT
(HWY 101, Township of Chapleau)
SITE PLAN/ BOREHOLE LOCATIONS

SHEET
1

exp. Services Inc.

KEY PLAN



LEGEND

- Approximate Borehole Locations
- TBM (Temporary Bench Mark)
- Approximate Probe Borehole Locations

BH No.	APPROX. ELEV.	MTM CO-ORDINATES	
		NORTH	EAST
TBM	430.0	5297258	350412
BH 1	429.7	5297258	350442
BH 2	429.8	5297280	350468
BH 3	429.9	5297290	350482
BH 4	430.1	5297303	350494
BH 5	426.5	5297298	350452
BH 6	426.7	5297276	350474
PB 1	429.8	5297303	350476
PB 2	429.8	5297258	350460

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.

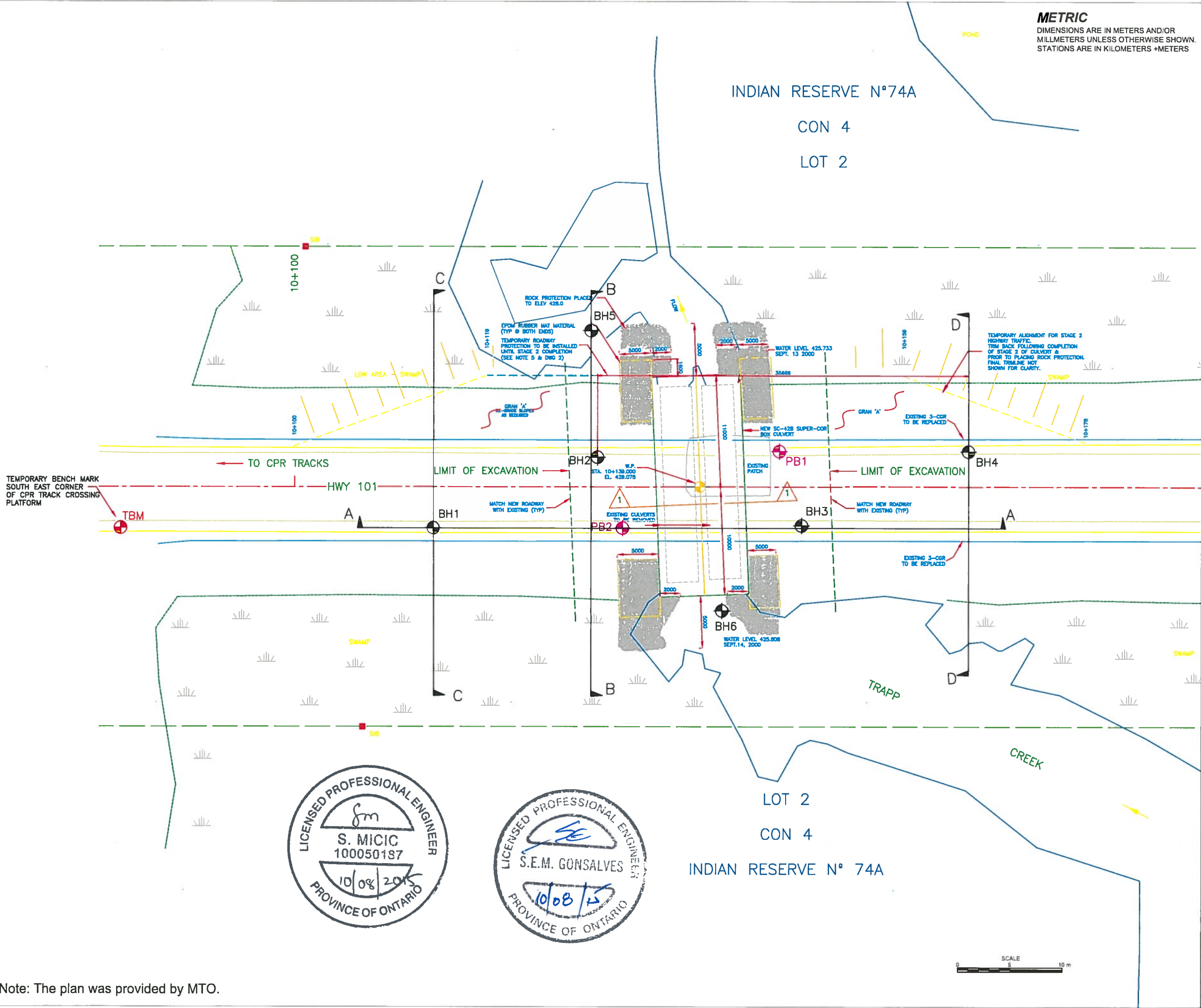
The complete foundation investigation and design report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in the report and related documents are specifically excluded in accordance with the conditions of Section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

DATE	BY	DESCRIPTION
2015 07 17	SM	FINAL SUBMISSION
2015 05 29	SM	SUBMISSION FOR MTO REVIEW
GEOCRES NO. 410-13		
PROJECT NO. ADM-00028245-J0		
SUBMD SM	CHECKED SM	DATE 2015 07 17
DRAWN NT	CHECKED SG	APPROVED DWG. 01

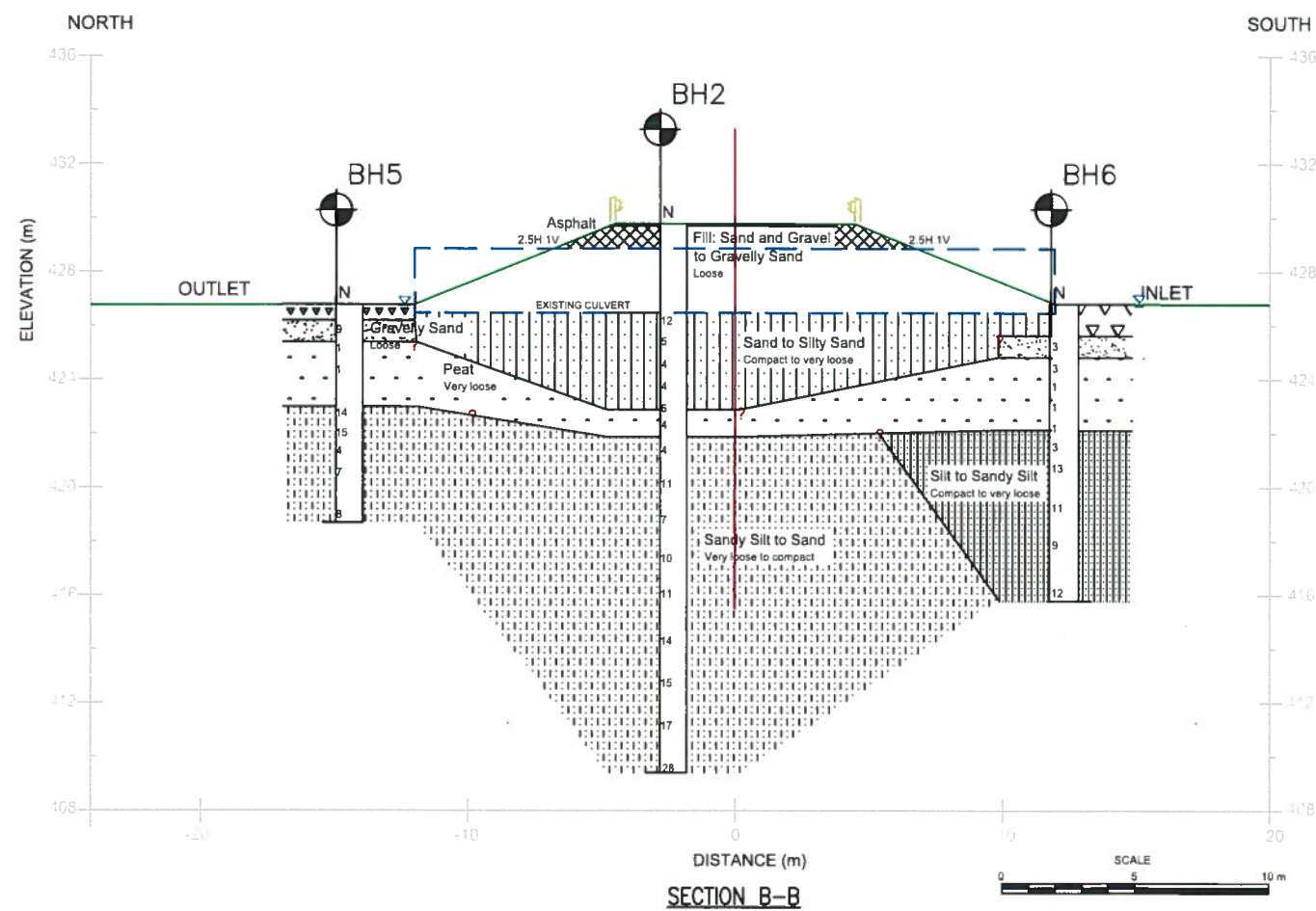
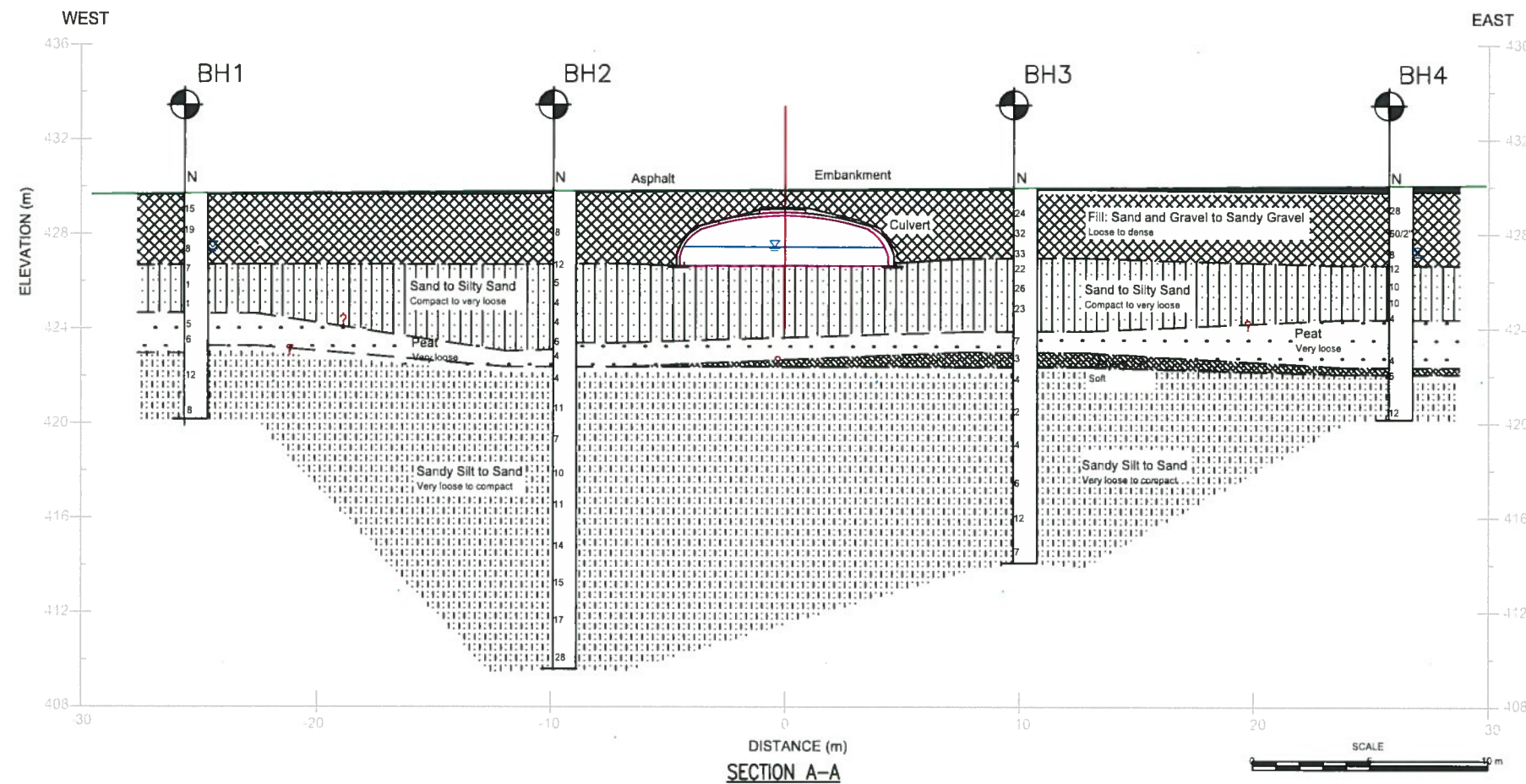
INDIAN RESERVE N°74A

CON 4

LOT 2



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. 8
WO 2015-11011



TRAPP CREEK CULVERT REPLACEMENT
(HWY 101, Township of Chapleau)

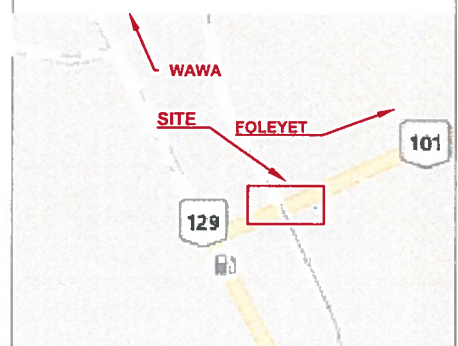
SOIL STRATA

SHEET
2



exp Services Inc.

KEY PLAN



LEGEND

- Approximate Borehole Locations
- Standard Penetration Test (Blows/0.3 m)
- Water Level

SOIL STRATA SYMBOLS

- FILL
- SAND TO SILTY SAND
- PEAT
- SANDY SILT TO SAND
- WATER
- CLAYEY SILT
- GRAVELLY SAND
- SILT TO SANDY SILT
- ASPHALT

BH No.	APPROX. ELEV.	MTM CO-ORDINATES	
		NORTH	EAST
TBM	430.00	5297258	350412
BH 1	429.71	5297258	350442
BH 2	429.80	5297280	350468
BH 3	429.88	5297290	350482
BH 4	430.13	5297303	350494
BH 5	427.15	5297298	350452
BH 6	428.07	5297276	350474

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 Contract 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. Downstream information contained in the report and related documents are specifically excluded in accordance with the conditions of Section GC 2.01 of DPS Gen Cond.

2015.07.17	SM	FINAL SUBMISSION			
2015.05.29	SM	SUBMISSION FOR MTO REVIEW			
DATE	BY	DESCRIPTION			
		GEOCRES NO. 410-13			
		PROJECT NO. ADM-00028245-J0			
SUBM'D	SM	CHECKED	SM	DATE	2015.07.17
DRAWN	SA	CHECKED	SG	APPROVED	DWG. 02

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. 8
WO 2015-11011

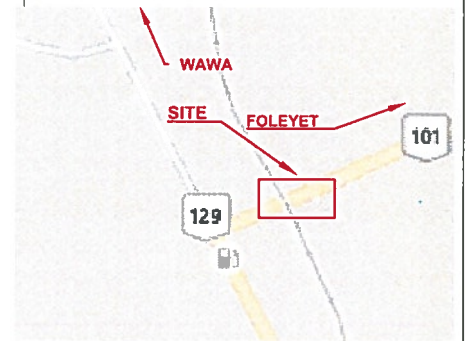


TRAPP CREEK CULVERT REPLACEMENT
(HWY 101, Township of Chapleau)
SOIL STRATA

SHEET
3

exp. Services Inc.

KEY PLAN



LEGEND

- Approximate Borehole Locations
- Standard Penetration Test (Blows/0.3 m)
- Water Level

SOIL STRATA SYMBOLS

- FILL
- CLAYEY SILT
- SAND TO SILTY SAND
- GRAVELLY SAND
- PEAT
- SILT TO SANDY SILT
- SANDY SILT TO SAND
- ASPHALT
- WATER

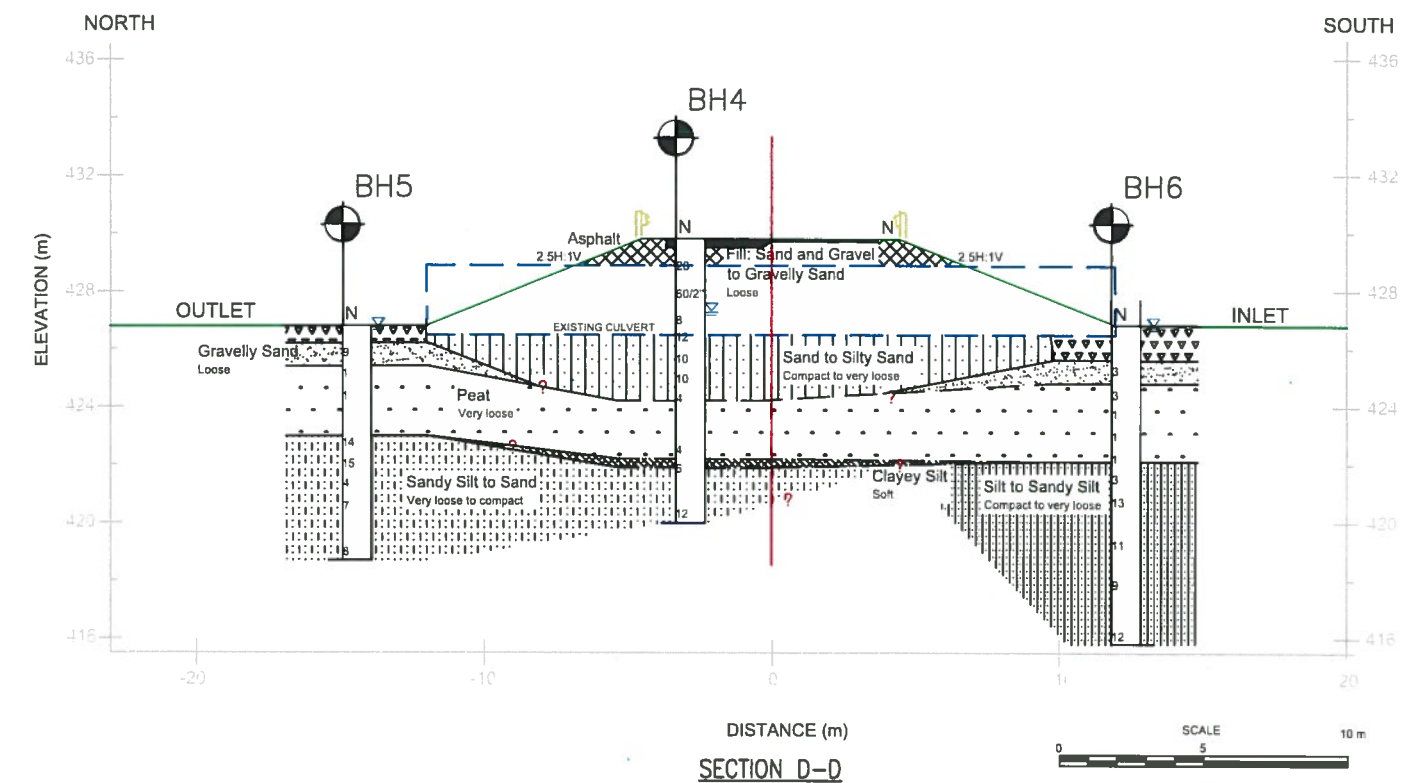
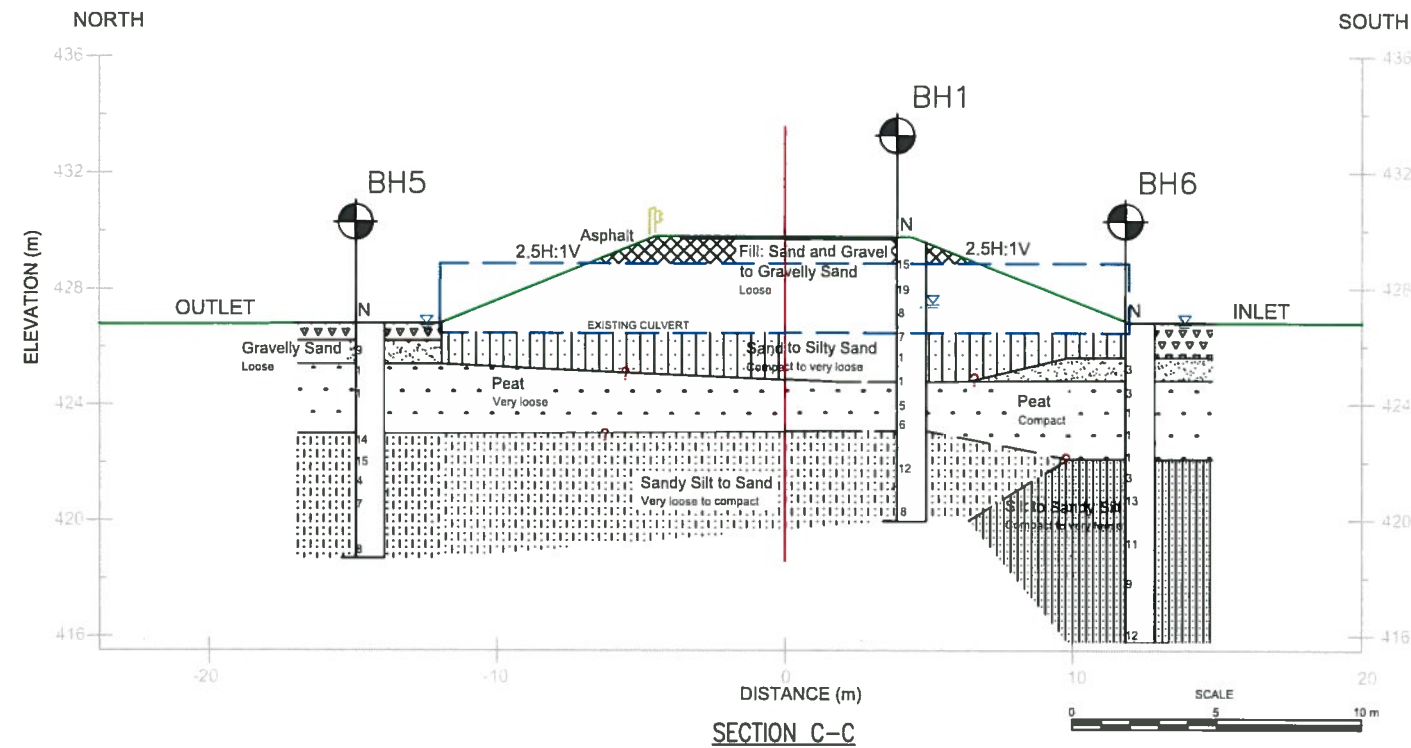
BH No.	APPROX. ELEV.	MTM CO-ORDINATES	
		NORTH	EAST
TBM	430.00	5297258	350412
BH 1	429.71	5297258	350442
BH 2	428.80	5297280	350468
BH 3	429.88	5297290	350482
BH 4	430.13	5297303	350494
BH 5	427.15	5297298	350452
BH 6	428.07	5297276	350474

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 Contract 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. Downview 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.

2015.07.17	SM	FINAL SUBMISSION
2015.05.29	SM	SUBMISSION FOR MTO REVIEW
DATE	BY	DESCRIPTION
GEOCRE NO. 410-13		
PROJECT NO. ADM-00028245-J0		
SUBM'D SM	CHECKED SM	DATE 2015.07.17
DRAWN SA	CHECKED SG	APPROVED DWG. 03



Note: The plan was provided by MTO.



Appendix C – Borehole 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.

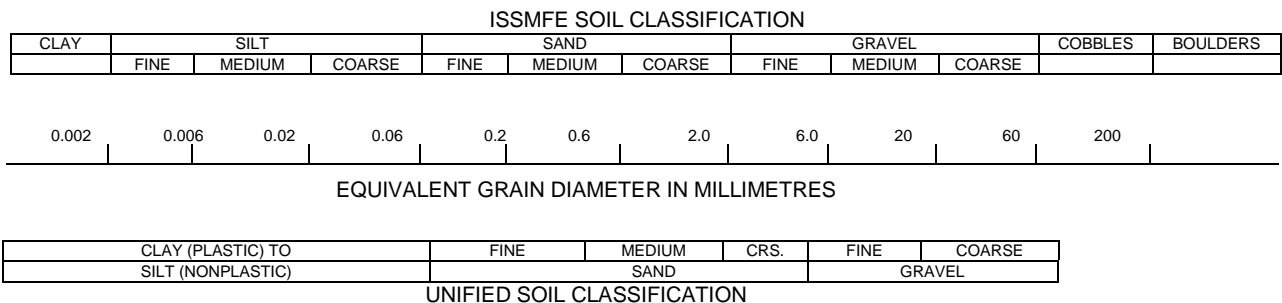
Seam: a thin, confined layer of soil having different particle size, texture, or color from materials above and below.

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 the ASTM D2487-11 Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System). 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.



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:

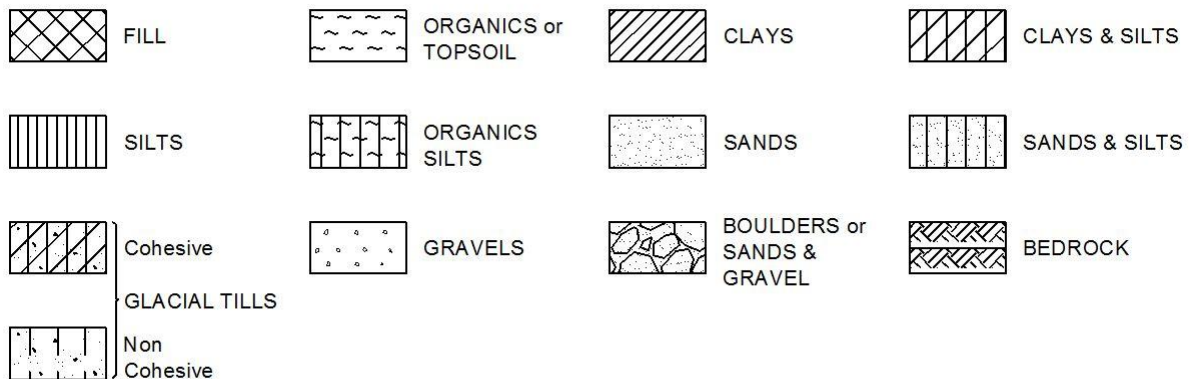
Table c: Consistency of Cohesive Soil

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



Open Borehole or Test Pit



Monitoring Well, Piezometer or Standpipe

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 BH-1

1 OF 1

METRIC

W. P. WO 2015-11011 LOCATION Chapleau, Sault Ste. Marie are, ON, MTM Z13 350477.15E 5297280.17N ORIGINATED BY NT
 DIST HWY 101 BOREHOLE TYPE CME-55X, Hollow stem auger, not cased COMPILED BY NT
 DATUM BM ELEV. 430.5 m MTM Z13 350411.87E 5297280.17E DATE 2015/04/28 - 2015/04/28 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					
429.7	Ground Surface																
429.6	ASPHALT 85 mm thickness		1	AS													
	FILL: SAND AND GRAVEL TO GRAVELLY SAND(GW-SW) trace silt, occasional cobbles, brown to greyish brown, moist to wet, compact to loose		2	SS	15												
			3	SS	19												
	- Becoming wet, loose		4	SS	8												
426.7	SAND TO SILTY SAND(SW-SM) trace gravel, trace clay, greyish brown to grey, wet, dilatant, loose to very loose		5	SS	7												
3.0			6	SS	1												
			7	SS	1												
424.7	PEAT silty sand, with roots and rootlets, trace clay, dark brown, wet, very loose to compact		8	SS	5												
5.0			9	SS	6												
423.0	SANDY SILT(SM) Greyish brown to grey, wet, dilatant, tr gravel, tr clay, loose to very loose		10	SS	12												
6.7			11	SS	8												
420.0	END OF BOREHOLE																
9.8	NOTES: 1. This drawing is to be read with the subject report and project numbers as presented above. 2. Interpretation assistance by exp is required before used by others. 3. Groundwater level was measured in open hole.																

OPG_EXP RECORD OF BOREHOLE 5013-E-0008 ASSIG. 8- BH LOGS.GPJ ONTARIO MOT.GDT 6/10/15

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

Brampton, Ontario

RECORD OF BOREHOLE No BH-2

1 OF 2

METRIC

W. P. WO 2015-11011 LOCATION Chapleau, Sault Ste. Marie are, ON, MTM Z13 E N ORIGINATED BY NT
 DIST HWY 101 BOREHOLE TYPE CME-55X, Diamond drill, Cased hole COMPILED BY NT
 DATUM BM ELEV. 430.5 m MTM Z13 350411.87E 52972 DATE 2015/04/28 - 2015/04/28 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 (%)		
								○ UNCONFINED	+ FIELD VANE	×	QUICK TRIAXIAL	LAB VANE								
							20	40	60	80	100		10	20	30		GR SA SI CL			
429.8	Ground Surface																			
428.7	ASPHALT 85 mm thickness		1	AS									○							
	FILL: SAND AND GRAVEL TO GRAVELLY SAND(GW-SW) trace silt, occasional cobbles, brown , moist , loose		2	SS	8								○				22 71 (7)			
426.8	- Wash bore from 3.05 m																			
3.0	SAND TO SILTY SAND(SW-SM) trace gravel, occasional cobbles, greyish brown to grey, wet, compact to very loose		3	SS	12								○							
			4	SS	5								○				4 87 (9)			
			5	SS	4								○							
			6	SS	4								○							
			7	SS	6								○							
423.3																69.2				
6.6	PEAT sandy silt,with roots and rootlets, trace clay, dark brown, wet, very loose		8	SS	4											89.2				
422.2																				
7.6	SANDY SILT TO SAND(SM-SW) some silt, trace peat, trace clay, dark brown to grey, wet, very loose to compact		9	SS	4											77.4				
																	0 31 68 1 Non Plastic			
			10	SS	11								○							
			11	SS	7								○				0 51 47 2			
			12	SS	10								○							
			13	SS	11								○							
			14	SS	14								○							
	- becoming sand, some silt, brownish grey, wet compact																			
			15	SS	15								○				1 84 (15)			
			16	SS	17								○							

Continued Next Page

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

OPG_EXP RECORD OF BOREHOLE 5013-E-0008 ASSIG. 8- BH LOGS.GPJ ONTARIO MOT.GDT 6/10/15

Brampton, Ontario

RECORD OF BOREHOLE No BH-2

2 OF 2

METRIC

W. P. WO 2015-11011 LOCATION Chapleau, Sault Ste. Marie are, ON, MTM Z13 E N ORIGINATED BY NT
 DIST HWY 101 BOREHOLE TYPE CME-55X, Diamond drill, Cased hole COMPILED BY NT
 DATUM BM ELEV. 430.5 m MTM Z13 350411.87E 52972 DATE 2015/04/28 - 2015/04/28 CHECKED BY SM

SOIL PROFILE		SAMPLES				GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100	W _p	W	W _L		
NOTES: 1. This drawing is to be read with the subject report and project numbers as presented above. 2. Interpretation assistance by exp is required before used by others. 3. No groundwater level was measured since washboring technique was used to advance borehole																	

OPG_EXP RECORD OF BOREHOLE 5013-E-0008 ASSIG. 8- BH LOGS.GPJ ONTARIO MOT.GDT 6/10/15

Brampton, Ontario

RECORD OF BOREHOLE No BH-3

1 OF 1

METRIC

W. P. WO 2015-11011 LOCATION Chapeau, Sault Ste. Marie are, ON, MTM Z13 E N ORIGINATED BY NT
 DIST HWY 101 BOREHOLE TYPE CME-55X, Diamond drill, Cased hole COMPILED BY NT
 DATUM BM ELEV. 430.5 m MTM Z13 350411.87E 52972 DATE 2015/04/27 - 2015/04/27 CHECKED BY SM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH: Cu, KPa									WATER CONTENT (%)		
								○ UNCONFINED		+ FIELD VANE									
								× QUICK TRIAXIAL		LAB VANE									
429.9	Ground Surface						20	40	60	80	100								
429.8	ASPHALT 76 mm thickness		1	AS												44 50 (6)			
	FILL: SAND AND GRAVEL TO SANDY GRAVEL(GW) trace silt, occasional cobbles, brown , moist to wet, compact to dense		2	SS	24														
			3	SS	32														
	- Wash bore from 2.28 m		4	SS	33														
426.8																62 34 (4)			
3.0	SAND TO SILTY SAND(SW-SM) trace gravel, occasional cobbles, greyish brown to grey, wet, compact		5	SS	22														
			6	SS	26														
			7	SS	23														
423.8																			
6.1	PEAT sandy silt, with roots and rootlets, trace clay, dark brown, wet, compact		8	SS	7											0 32 66 2 Organic Content = 34 %			
422.9																			
7.0	CLAYEY SILT(ML) trace peat, , trace sand, brown, wet, soft		9	SS	3														
422.3																			
7.6	SANDY SILT TO SAND(SM-SW) some silt, trace gravel, trace clay, greyish brown to grey, wet, very loose to compact		10	SS	4											0 43 56 1 Non Plastic			
			11	SS	2														
			12	SS	4														
			13	SS	6											0 97 (3)			
			14	SS	12														
	- becoming gravelly sand		15	SS	7											21 78 (1)			
414.0																			
15.9	END OF BOREHOLE																		
	NOTES: 1. This drawing is to be read with the subject report and project numbers as presented above. 2. Interpretation assistance by exp is required before used by others. 3. No groundwater level was measured since washboring technique was used to advance borehole																		

OPG_EXP RECORD OF BOREHOLE 5013-E-0008 ASSIG. 8- BH LOGS.GPJ ONTARIO MOT.GDT 6/10/15

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

Brampton, Ontario

RECORD OF BOREHOLE No BH-4

1 OF 1

METRIC

W. P. WO 2015-11011 LOCATION Chapleau, Sault Ste. Marie are, ON, MTM Z13 E N ORIGINATED BY NT
 DIST HWY 101 BOREHOLE TYPE CME-55X, Hollow stem auger, not cased COMPILED BY NT
 DATUM BM ELEV. 430.5 m MTM Z13 350411.87E 52972 DATE 2015/04/29 - 2015/04/29 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					
430.1	Ground Surface																
429.8	ASPHALT 300 mm thickness						430										
0.3	FILL: SAND AND GRAVEL TO GRAVELLY SAND(GW-SW) asphalt mix, trace silt, trace cobbles, black/brown to brown, moist to wet, loose to compact - Asphalt mix up to 1.55m - hit cobbles/ boulder, grab auger sample		1	AS			429										36 58 (6)
			2	SS	28		428										21 72 (7)
			3	SS	60/2"		427										
			4	SS	8		426										
426.9	SAND TO SILTY SAND(SW-SM) trace gravel, grey, wet, compact		5	SS	12		425										2 90 (8)
3.3			6	SS	10		424										
			7	SS	10		423										
424.5	PEAT sandy silt, with roots and rootlets, trace clay, dark brown, wet, very loose		8	SS	4		422										
5.6			9	TW			421										
			10	SS	4												
422.5	CLAYEY SILT(ML) trace peat, , trace sand, brown, wet, soft		11	SS	6												
422.0	SANDY SILT TO SAND(SM-SW) trace gravel, trace clay, grey, wet, loose to compact																
7.9																	
420.4	- becoming brown gravelly sand		12	SS	12												22 74 (4)
9.8	END OF BOREHOLE																
	NOTES: 1. This drawing is to be read with the subject report and project numbers as presented above. 2. Interpretation assistance by exp is required before used by others. 3. Groundwater level was measured in open hole.																

OPG_EXP RECORD OF BOREHOLE 5013-E-0008 ASSIG. 8- BH LOGS.GPJ ONTARIO MOT.GDT 6/10/15

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

Brampton, Ontario

RECORD OF BOREHOLE No BH-5

1 OF 1

METRIC

W. P. WO 2015-11011 LOCATION Chapleau, Sault Ste. Marie are, ON, MTM Z13 E N ORIGINATED BY NT
 DIST HWY 101 BOREHOLE TYPE Portable Tripod, Wash boring, Cased hole COMPILED BY NT
 DATUM BM ELEV. 430.5 m MTM Z13 350411.87E 52972 DATE 2015/04/30 - 2015/04/30 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					
427.2	Water Surface																
	Water	△					427										
426.5																	
0.6	GRAVELLY SAND (SW) trace wood pieces, grey, wet, loose	○	1	SS	9		426							○			
425.8																	
1.4	PEAT sand and sit, some gravel, trace clay, with roots and rootlets, brown, saturated to wet, very loose	∩	2	SS	1		425									381.5	
		∩	3	SS	1		424									141.2	
	- Attempted Shelby, no recovery, grab split spoon sample	∩	4	GS			423									269.2	
423.3																	
3.8	SANDY SILT (ML) trace clay, grey, wet, dilatant, compact to very loose		5	SS	14		422							○			
			6	SS	15		421							○			
	- becoming silt, dilatant		7	SS	4		420							○			
			8	SS	7												
419.1			9	SS	8									○			
8.1	END OF BOREHOLE																
	NOTES: 1. This drawing is to be read with the subject report and project numbers as presented above. 2. Interpretation assistance by exp is required before used by others. 3. Groundwater level was measured in open hole.																

OPG_EXP RECORD OF BOREHOLE 5013-E-0008 ASSIG. 8- BH LOGS.GPJ ONTARIO MOT.GDT 6/10/15

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

Brampton, Ontario

RECORD OF BOREHOLE No BH-6

1 OF 1

METRIC

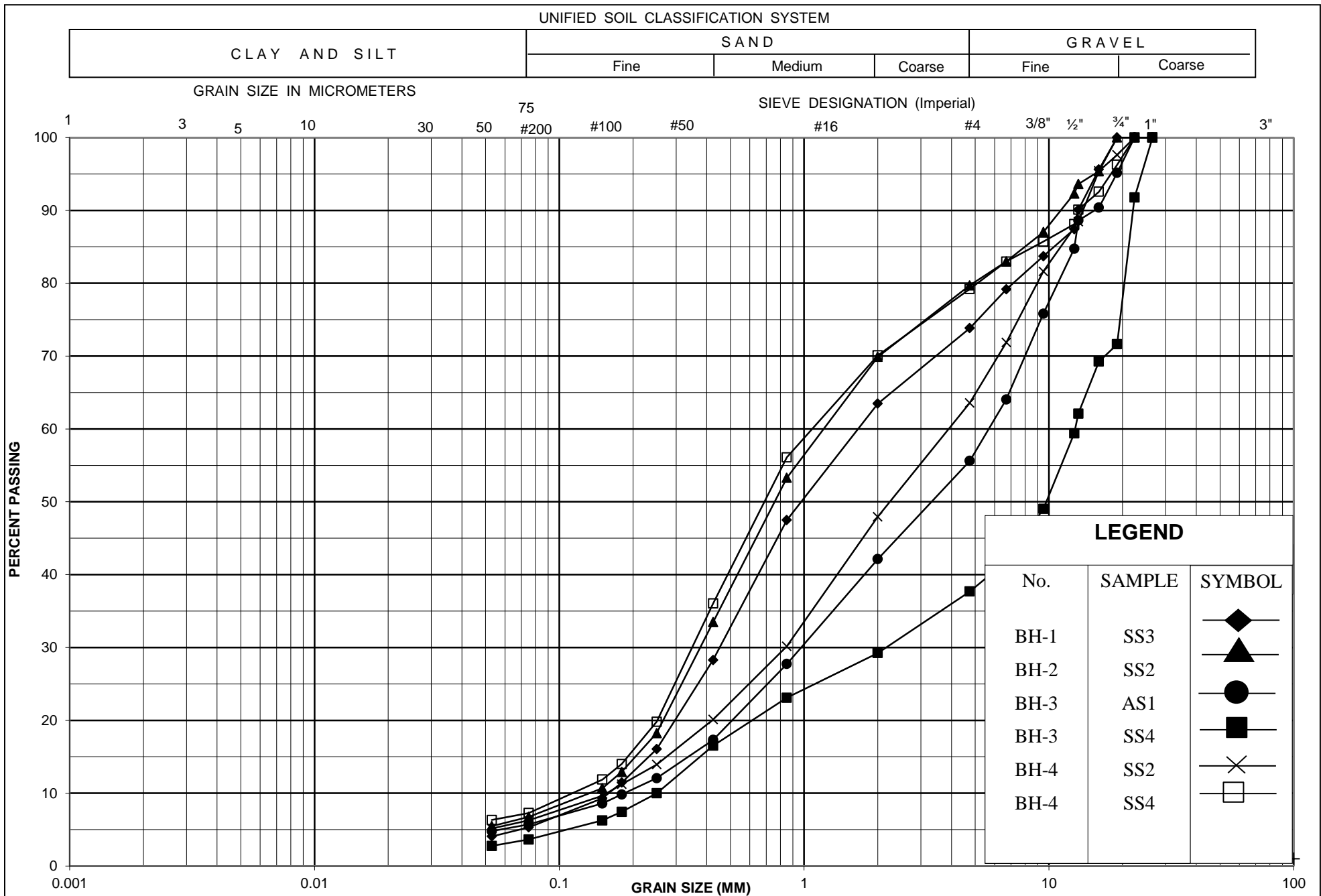
W. P. WO 2015-11011 LOCATION Chapleau, Sault Ste. Marie are, ON, MTM Z13 E N ORIGINATED BY NT
 DIST HWY 101 BOREHOLE TYPE Portable Tripod, Wash boring, Cased hole COMPILED BY NT
 DATUM BM ELEV. 430.5 m MTM Z13 350411.87E 52972 DATE 2015/04/30 - 2015/04/30 CHECKED BY SM

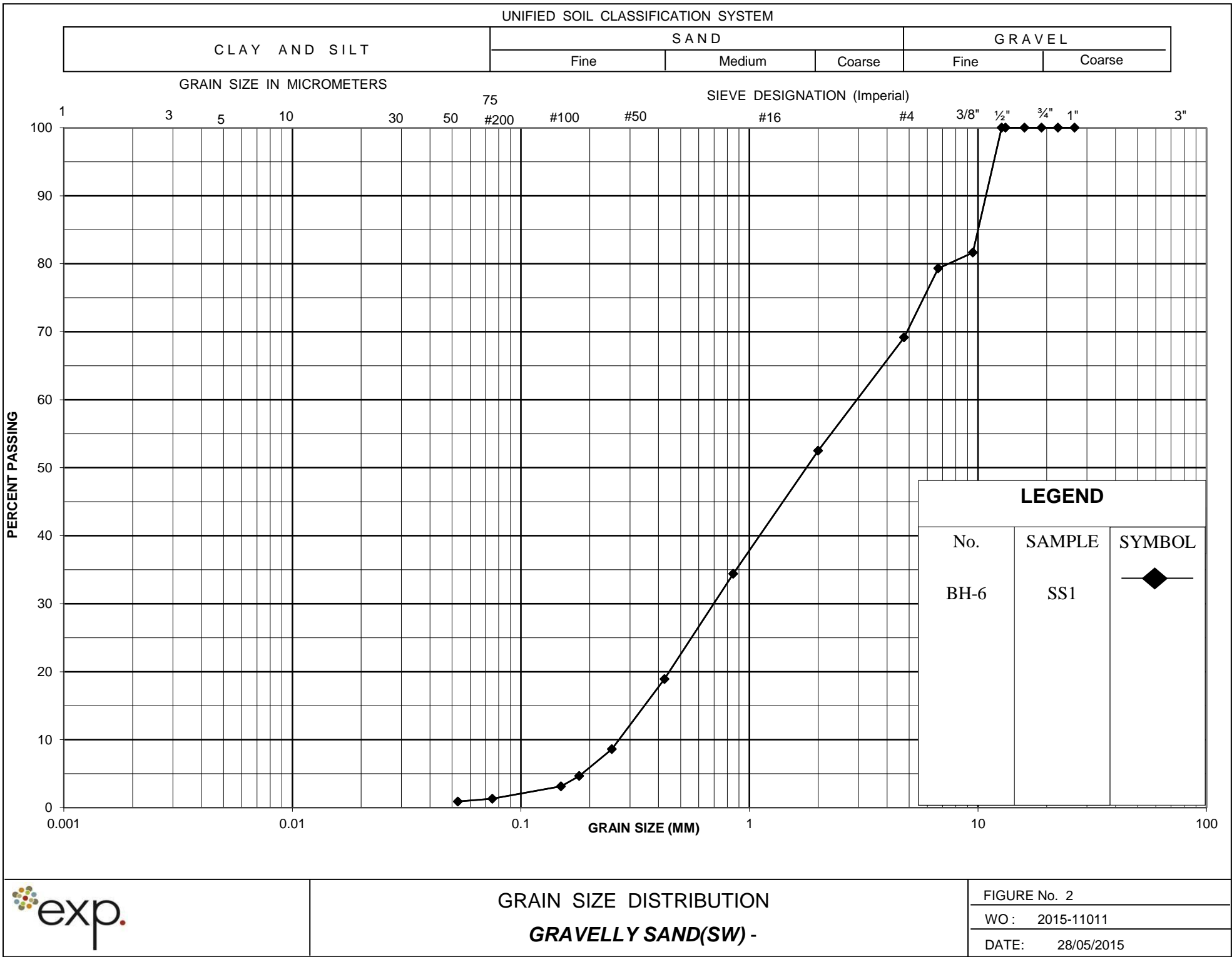
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100					
427.7	Water Surface	△				▽											
	Water	△					427										
426.5		△															
1.2	GRAVELLY SAND (SW) brown/grey, wet, loose	○	1	SS	3		426							○			31 68 (1)
425.8		○															
2.0	PEAT sand and sit, trace clay, with roots and rootlets, dark brown, wet, very loose	∩ ∩ ∩	2	SS	3		425									119.2	
		∩ ∩ ∩	3	SS	1											55.2	
		∩ ∩ ∩	4	SS	1		424									174.7	
		∩ ∩ ∩	5	SS	1											175.2	
423.0		∩ ∩ ∩					423									42.1	
4.7	SILT TO SILTY SAND (ML) trace clay, grey, wet, dilatant, compact to very loose	∩ ∩ ∩	6	SS	3												
		∩ ∩ ∩	7	SS	13		422							○			
		∩ ∩ ∩															
		∩ ∩ ∩	8	SS	11		421										
	- becoming brownish grey, silty sand	∩ ∩ ∩					420							○			
		∩ ∩ ∩															
		∩ ∩ ∩	9	SS	9		419							○			
		∩ ∩ ∩					418										
		∩ ∩ ∩															
416.8		∩ ∩ ∩	10	SS	12		417							○			
11.0	END OF BOREHOLE																
	NOTES: 1. This drawing is to be read with the subject report and project numbers as presented above. 2. Interpretation assistance by exp is required before used by others. 3. Water level above the ground surface																

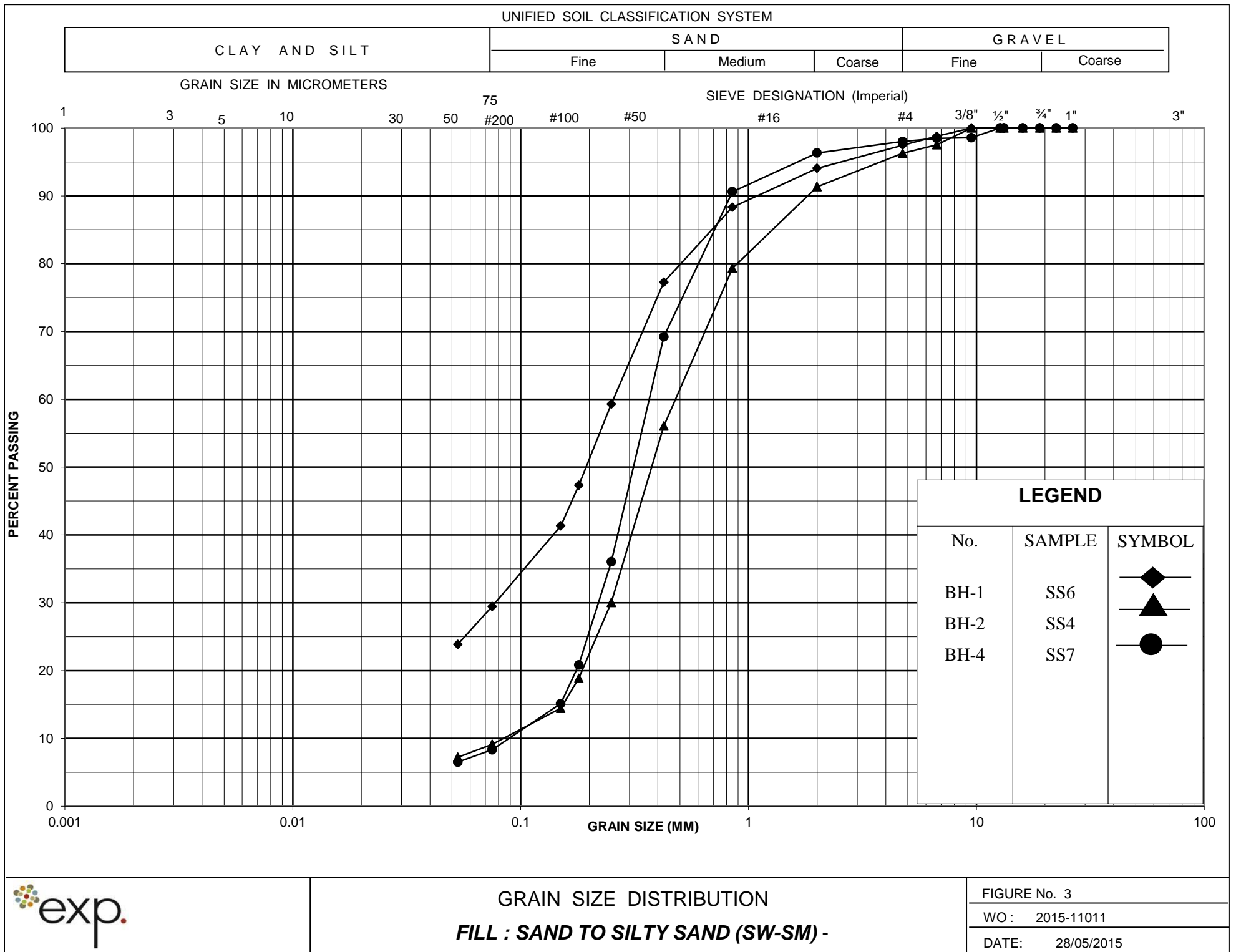
+ 3, × 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

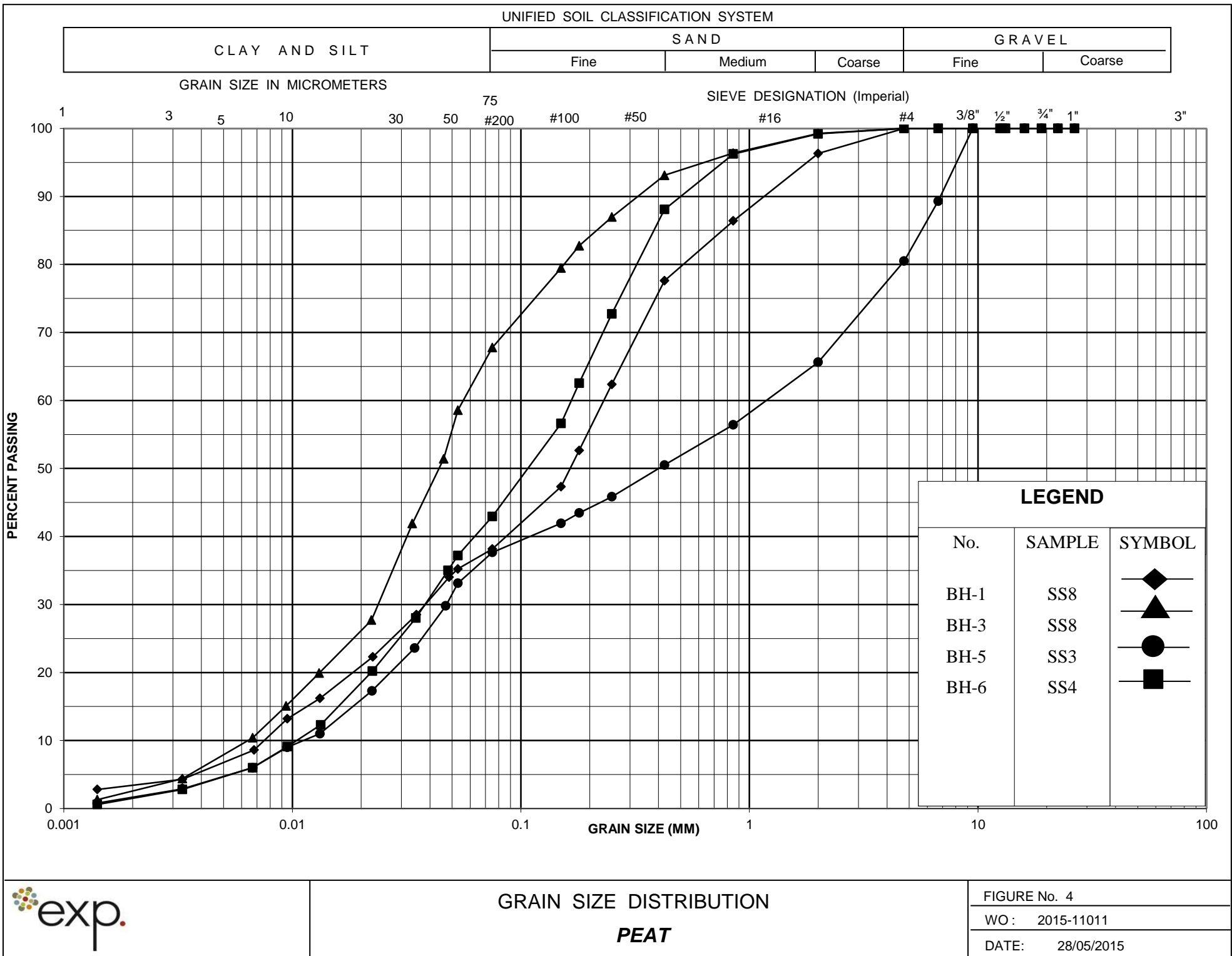
OPG_EXP RECORD OF BOREHOLE 5013-E-0008 ASSIG. 8- BH LOGS.GPJ ONTARIO MOT.GDT 6/10/15

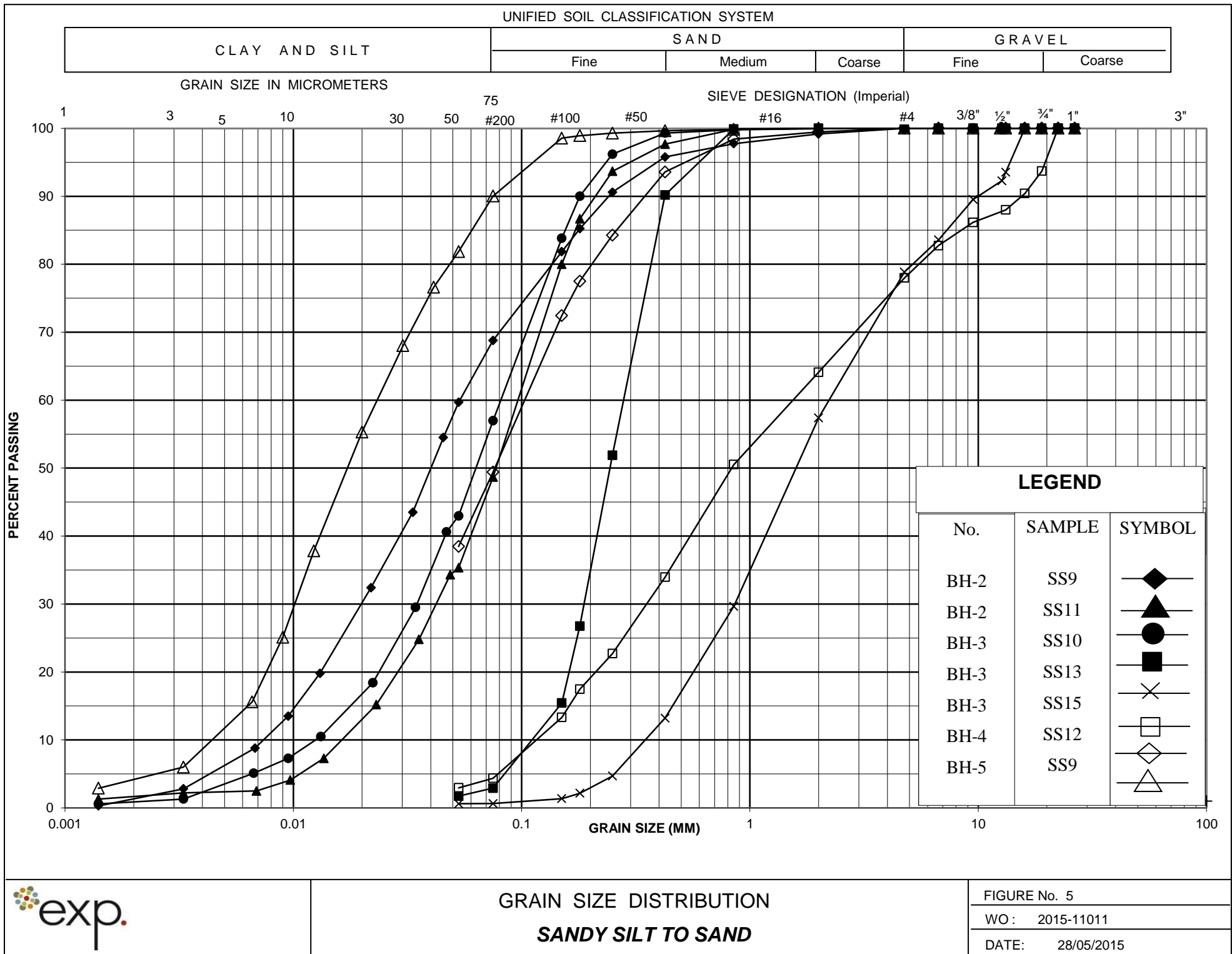
Appendix D – Laboratory Data



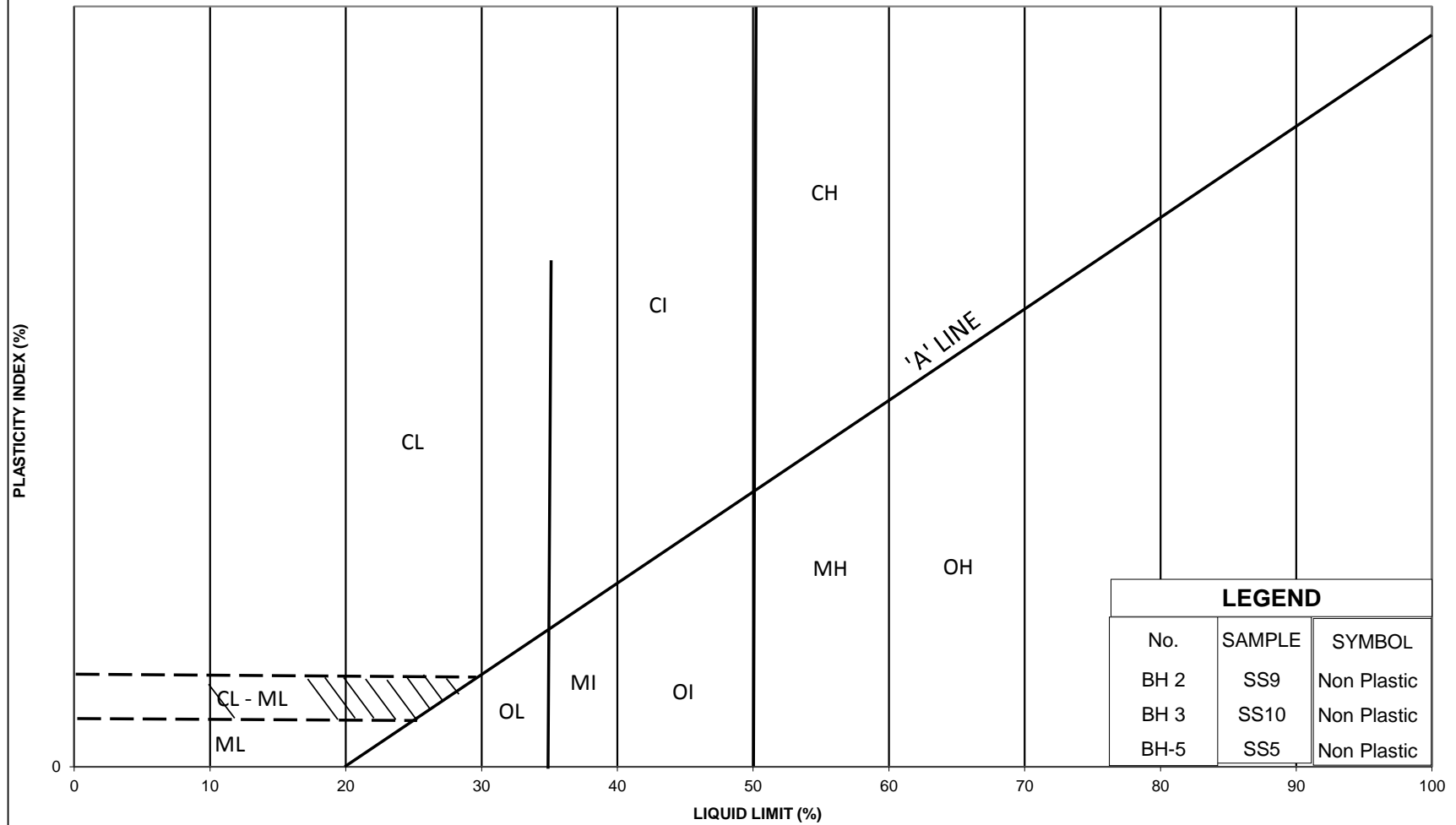








Trapp Creek Culvert, Hwy 101, Chapleau





exp Services Inc.

1595 Clark Boulevard
Brampton, ON
L6T 4V1
Tel.: 905-793-9800
Fax: 905-793-0641

**Consolidation Test
Summary Data Sheet
(ASTM: D 2435-96)**

Project No.: adm-00028245-j0 adm-100Project Name: TRAPP CREEK CULVERTBorehole No. BH 4

Client Job No.: _____

Sample No. TW-9

Sample Location: _____

Depth: 6.1 - 6.9mSample Description: Amorphous Black Peat

Water Content Determination	Before Test		After Test
	Specimen	Trimming	Specimen
Wt. of wet sample + Ring (tare) - g	146.38	68.79	126.31
Wt. of dry sample + Ring (tare) - g	96.17	17.94	96.17
Wt. of water (W_w) - g	50.21	50.85	30.14
Wt. of Ring - g	78.02	1.95	78.02
Wt. of dry soil (W_s) - g	18.15	15.99	18.15
Water Content (W) - %	276.6	318.0	166.1
Average (W) - %	297.3		166.1

Apparatus:

Machine No.	1
Cell No.	1
Ring No.	1
Diameter of Ring (in) :	2.5
Height of Ring - H_1 (in):	0.75
Area of Ring (in^2) :	4.9087

Load Factor:

1.55
500

 lb. on Hanger
lb/ft² on Sample

Test Data

Initial Dial Reading (in) :	0.4585
Final Dial Reading (in) :	0.1579
Difference (in) :	0.3006
Machine Correction 0 to 0 (in) :	0.0056
Change in Ht., specimen, ΔH (in) :	0.2950
Final Ht. of specimen, $H_2 = H_1 - \Delta H$:	0.4550

Spec. Gr. of Solids (G) :	(estimated)	1.9
Spec. Gr. of Solids (G) :	(determined)	
Initial Height of Specimen, H_1 (in):		0.7500

Calculations	Before Test	After Test
Height of Specimen, H_1, H_2 (in):	0.7500	0.4550
Ht of Solids, H_s (in):	0.1187	0.1187
Ht. of Voids, H_v (in):	0.6313	0.3363
Ht. of Water, H_w (in):	0.6240	0.3745
Saturation, S_r (%):	98.8	100.0
Void ratio (e):	5.318	2.833

Comments:

Reported By: Willie RodychDate Reported: 20/05/2015



exp Services Inc.

1595 Clark Boulevard

Brampton, ON

L6T 4V1

Tel.: 905-793-9800

Fax: 905-793-0641

**Consolidation Test
Determination of Void Ratio
(ASTM: D 2435-96)**

Project No. adm-00028245-j0 adm-100
 Project Name Foundation Engineering
 Client Job No.:
 Sample No. BH 4 TW-9 6.1 - 6.9m
 Sample Location

Height of Solids (in):	0.119
Initial Height of Voids (in):	0.631
Initial Void Ratio (e_0):	5.318
Initial Dial Reading:	0.459

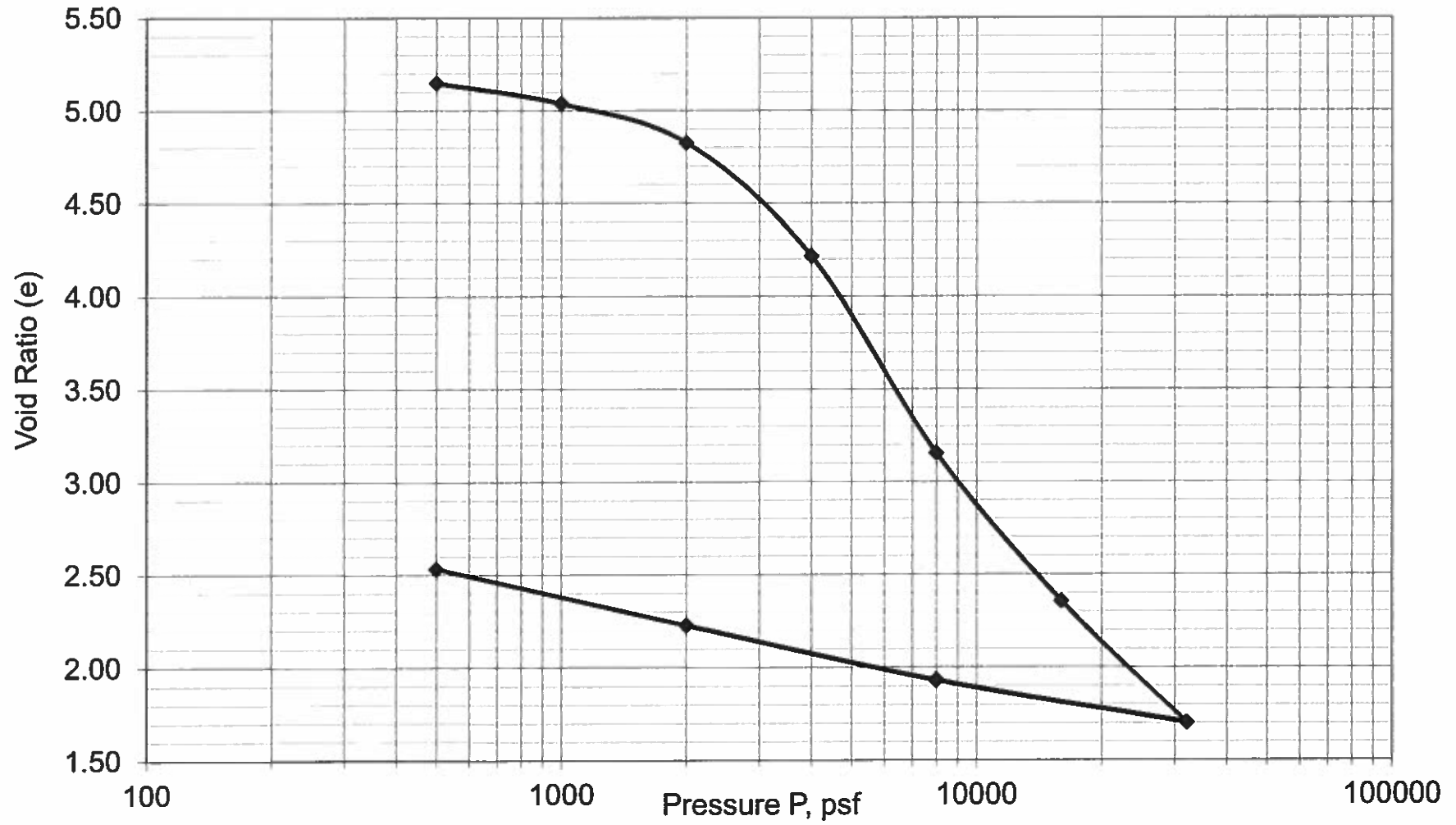
Load No.	Hanger Load (lbs.)	Pressure on sample (lb/ft ²)	Final Dial Reading	Decrease in Height of Voids (in)	Machine Deflection (in)	Net Decrease in Height of Voids (in)	Height of Voids (in)	Void Ratio (e)
1	1.55	500	0.4371	0.0214	0.0013	0.0201	0.6112	5.149
2	3.1	1000	0.4225	0.0360	0.0026	0.0334	0.5979	5.037
3	6.2	2000	0.3957	0.0628	0.0043	0.0585	0.5728	4.825
4	12.4	4000	0.3211	0.1374	0.0064	0.1310	0.5003	4.214
5	24.8	8000	0.1928	0.2657	0.0087	0.2570	0.3743	3.153
6	49.6	16000	0.0953	0.3632	0.0115	0.3517	0.2796	2.355
7	99.2	32000	0.0146	0.4439	0.0146	0.4293	0.2020	1.702
8	24.8	8000	0.0449	0.4136	0.0115	0.4021	0.2292	1.931
9	6.2	2000	0.0821	0.3764	0.0094	0.3670	0.2643	2.226
10	1.55	500	0.1200	0.3385	0.0078	0.3307	0.3006	2.532
11								
12								
13								
14								
15								

Tested By: Willie Rodych
 Date: 20/05/2015

Graph - e vs log P

Sample Test No.: 225601-3

Page 3 of 5





exp Services Inc.

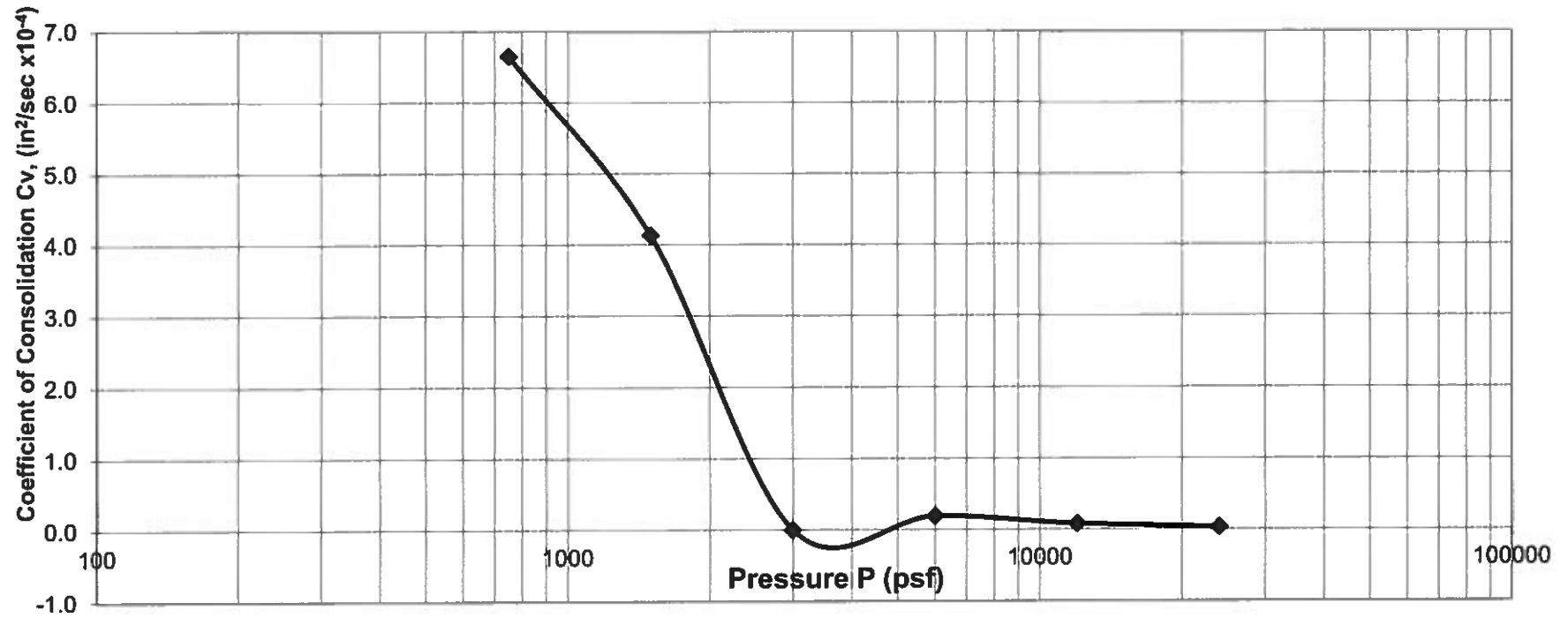
**1595 Clark Boulevard
Brampton, ON
L6T 4V1
Tel.: 905-793-9800
Fax: 905-793-0641**

**Consolidation Test
Coefficient of Consolidation
(ASTM: D 2435-96)**

Project No.:	adm-00028245-j0 adm-100
Project Name:	Foundation Engineering
Client Job No.:	
Site Location:	
Sample No.	BH 4 TW-9 6.1 - 6.9m

Initial Height of Sample (in):	0.7500
Initial Dial Reading:	0.4585

[illegible]

Graph - C_v vs $\log P$ 

Appendix E – Results of Slope Stability Analyses

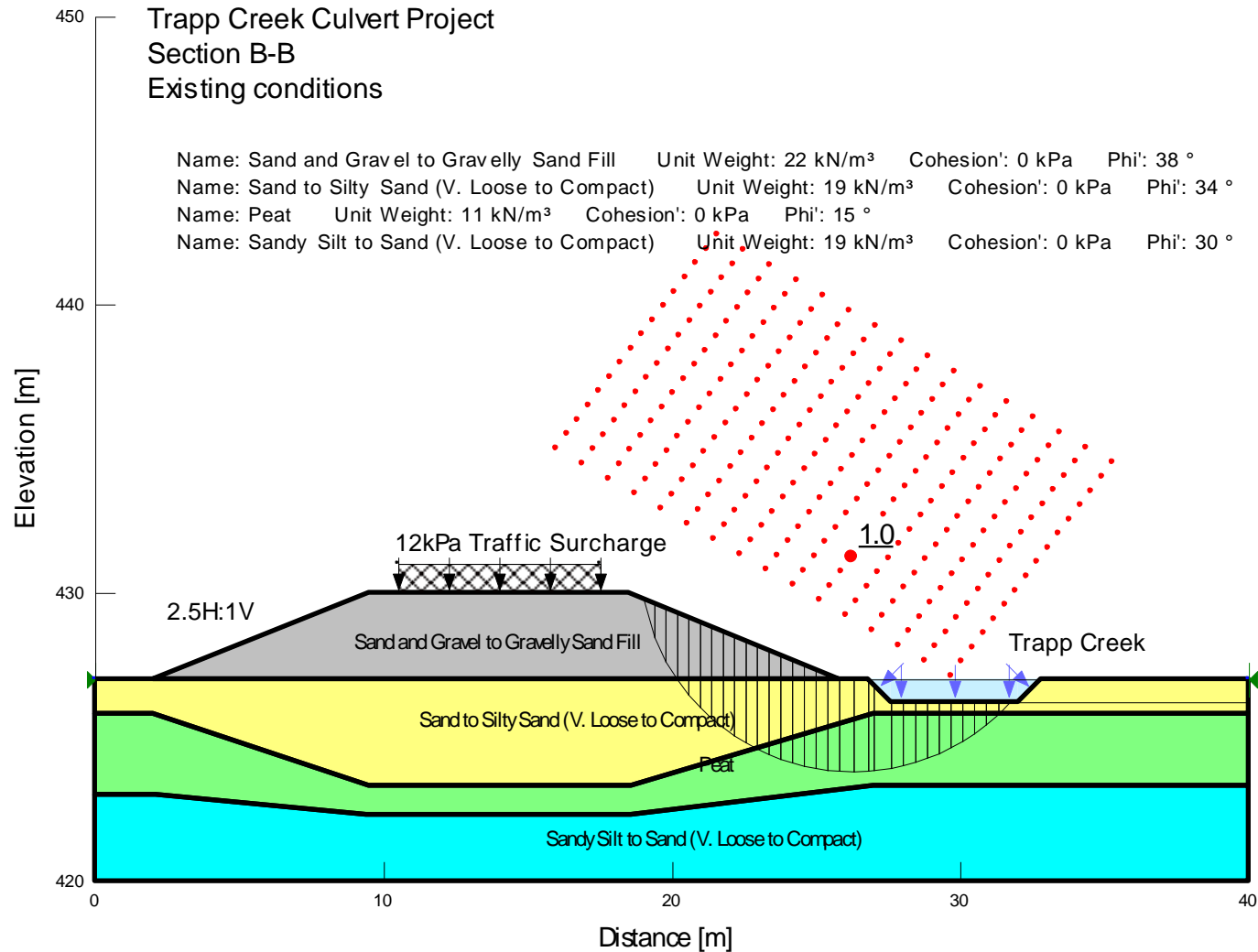


Figure E1. Results of slope stability analyses, existing embankment at Section B-B

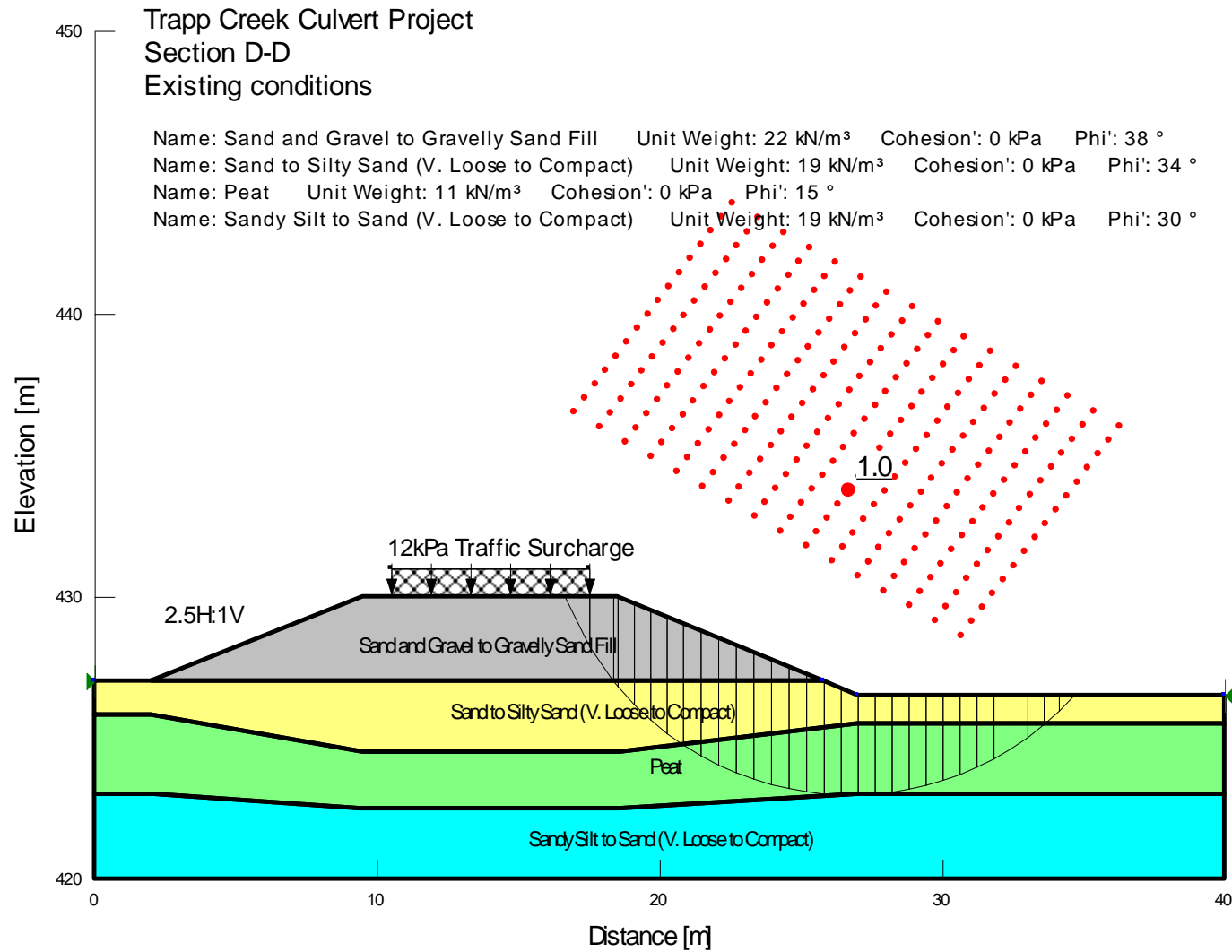


Figure E2. Results of slope stability analyses, existing embankment at Section D-D

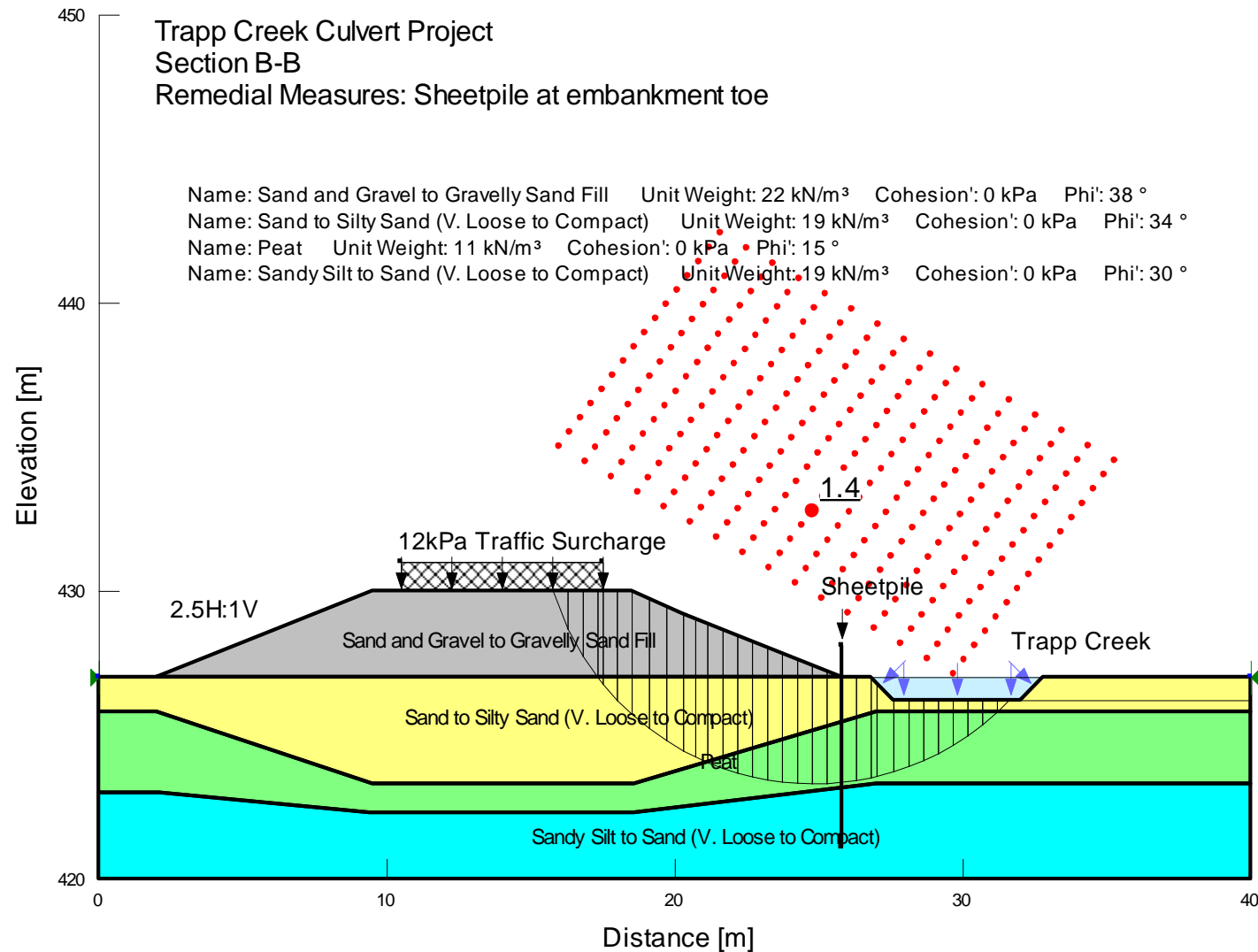


Figure E3. Results of slope stability analyses at Section B-B; Remedial measure – sheetpiles at embankment toe

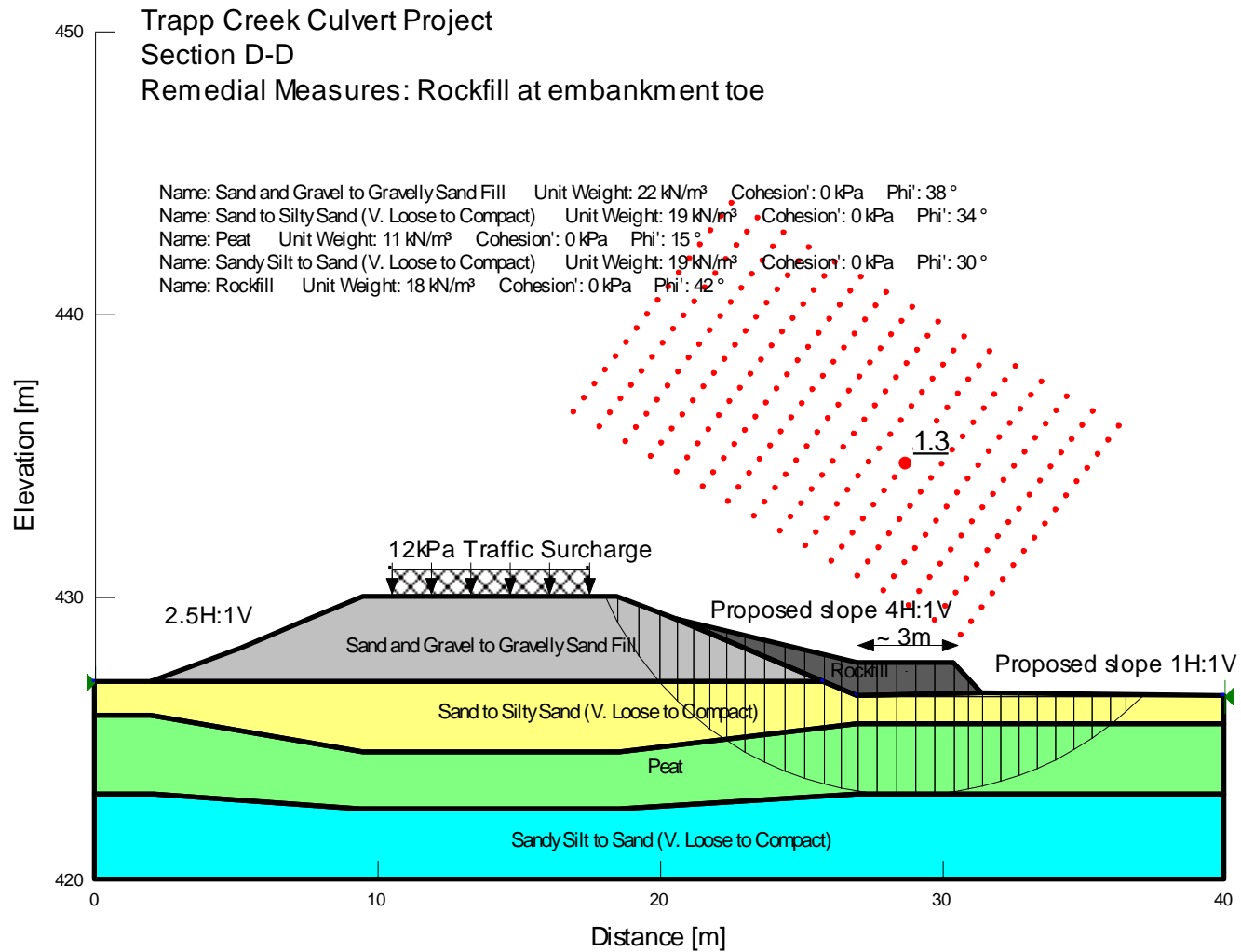


Figure E4. Results of slope stability analyses at Section D-D; Remedial measure – rockfill at embankment toe

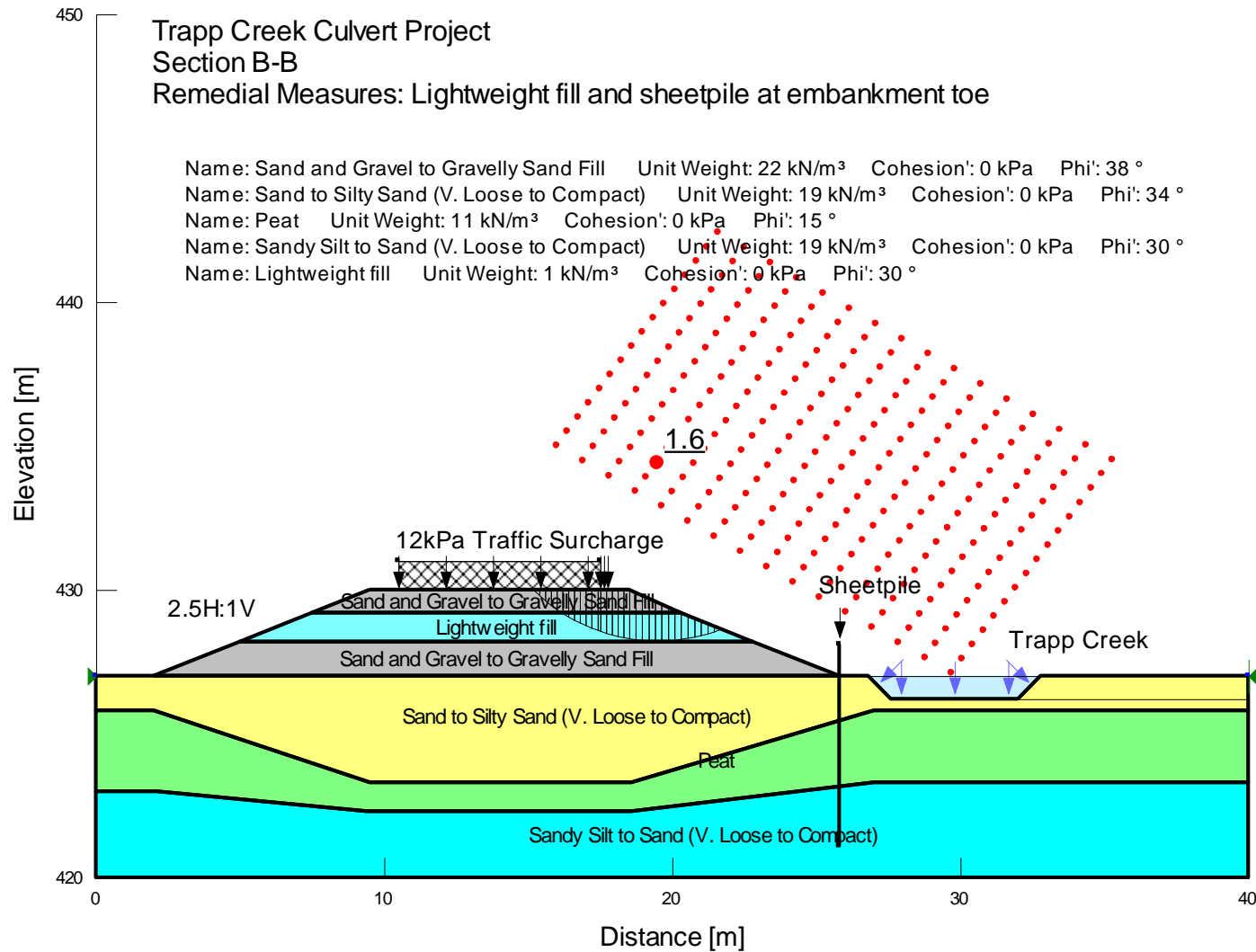


Figure E5. Results of slope stability analyses at Section B-B; Remedial measures – lightweight fill and sheetpiles at embankment toe

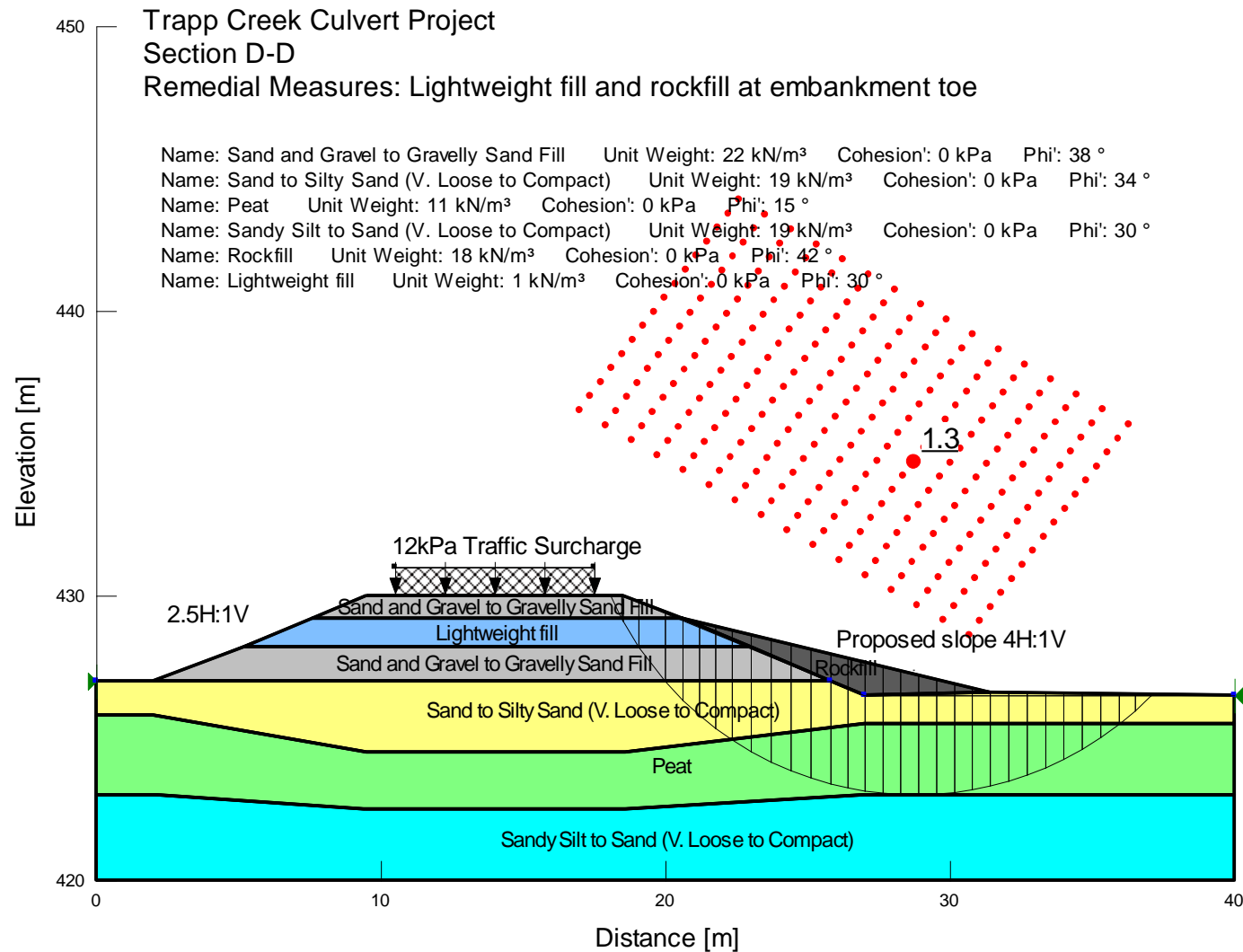


Figure E3. Results of slope stability analyses at Section D-D; Remedial measures – lightweight fill and rockfill at embankment toe

Appendix F – Results of Settlement Analyses

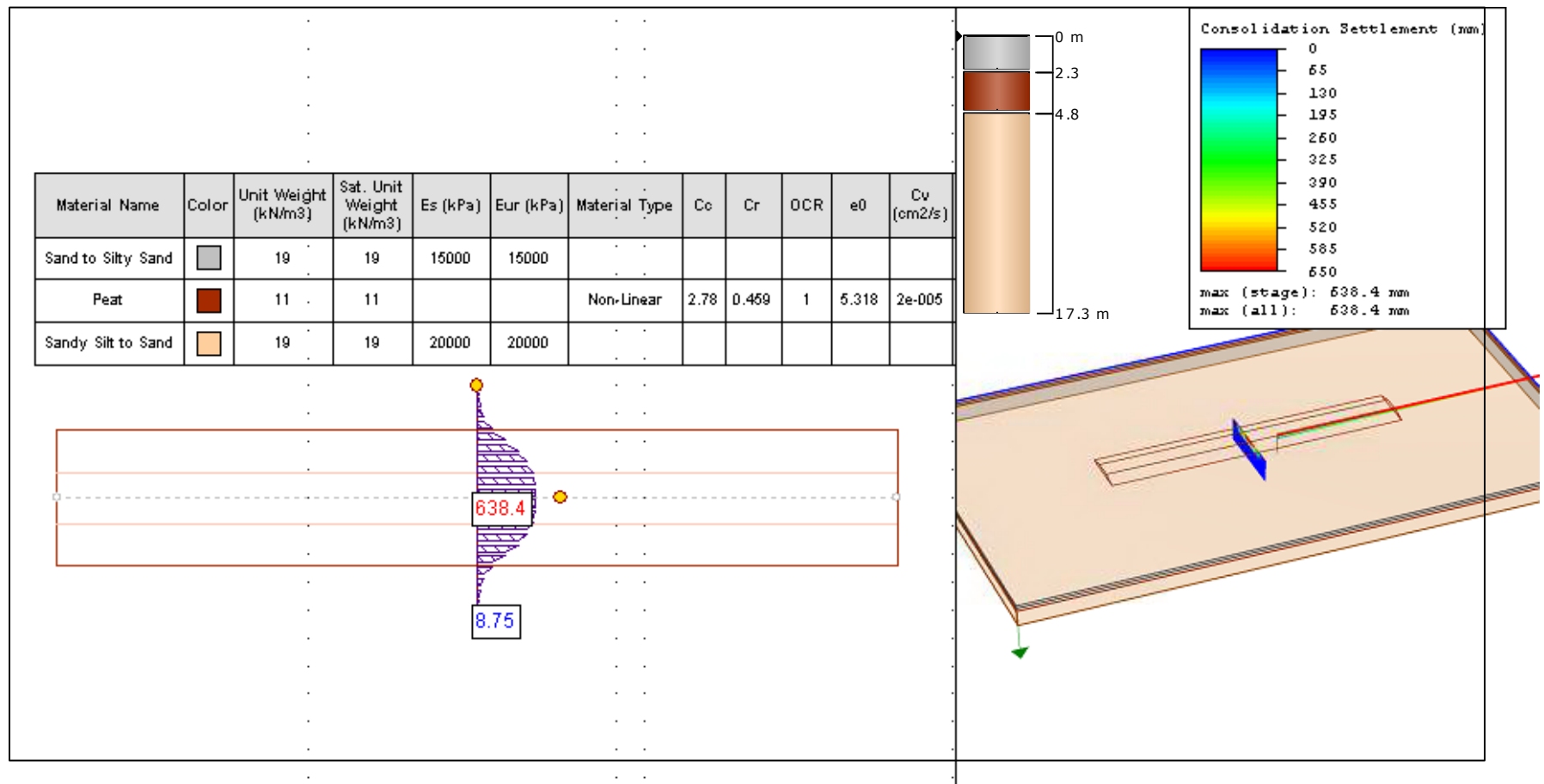


Figure F1. Settlement analysis at BH-4 location

Appendix G – NSSP – Expanded Polystyrene Embankment

EXPANDED POLYSTYRENE EMBANKMENT

Non-Standard Special Provision

REQUIREMENTS FOR EXPANDED POLYSTYRENE EMBANKMENT FILL

1.0 SCOPE

This special provision covers the requirements for the supply and construction of the rigid expanded polystyrene embankment fill and associated works as shown on the contract drawings.

2.0 REFERENCES

This special provision refers to the following standards, specifications or publications.

2.1 National Standards of Canada

CAN/CGSB - 51.20 M87

2.2 ASTM

ASTM D1621 Test Method for Compressive Properties of Rigid Cellular Plastics

ASTM C203 Test Method for Breaking Load and Flexural Properties of Block Type Thermal Insulation

ASTM C177 Test Method for Steady State Heat Flux Measurements and Thermal Transmission Properties by Means of the Heat Flow Apparatus

ASTM D2842 Test Method for Water Absorption by Rigid Cellular Plastics

ASTM D2863 Test Method for Measuring the Minimum Oxygen Content

ASTM D2126 Test Method for Response of Rigid Cellular Plastics to Thermal and Humid Aging

2.3 OPSS - Ontario Provincial Standard Specification

OPSS 212 Borrow

OPSS 501 Compaction

OPSS 517 Dewatering

OPSS 1010 Aggregates – Granular A, B, M, and Selected Subgrade Material

OPSS 1605 Expanded Extruded Polystyrene Pavement Insulation

OPSS 1860 Geotextiles

3.0 SUBSURFACE CONDITIONS

The subsurface conditions at the site are described in the Foundation Investigation Report for this Contract.

4.0 DEFINITIONS

For the purpose of this special provision, the following definitions apply:

Rigid Expanded Polystyrene: Moulded rigid blocks produced by a process of pre-expansion, aging and forming of petroleum based raw material.

Rigid Extruded Expanded Polystyrene: Rigid boards made by extrusion of expanded polystyrene beads.

Production Lot: The quantity of rigid polystyrene blocks produced in a continuous period of manufacturing the same grade and thickness of product within the same production day.

Quality Verification Engineer: Quality Verification Engineer means an Engineer with a minimum of five (5) years experience related to the design and/or construction of expanded polystyrene systems of similar scope to that in the Contract, or alternatively has demonstrated expertise by providing satisfactory quality verification services for the work at a minimum of two (2) projects of similar scope to the Contract. The Quality Verification Engineer shall be retained by the Contractor to ensure conformance with the contract documents and issue of certificate(s) of conformance.

5.0 QUALIFICATION

The Contractor shall have on site at the commencement of the work, a representative of the supplier of the rigid expanded polystyrene to advise on recommended construction procedure.

The Contractor shall maintain liaison with the supplier throughout the construction of the embankment for advice and guidance as required. Periodic site visits by the supplier should be coordinated as required.

6.0 SUBMISSION AND DESIGN REQUIREMENTS

6.1 Submission of Shop Drawings

At least three weeks before the commencement of work, the Contractor shall submit to the Contract Administrator six copies of the shop drawings and method statement signed and sealed by the Quality Verification Engineer that provides full details of materials and construction procedure.

6.2 Delivery, Storage, Handling, and Protection

The Contractor shall submit the method of delivery, storage, handling and protection from damage by weather, traffic, construction staging and other causes as per the rigid expanded polystyrene manufacturers requirement.

6.3 Construction

The contractor shall submit full details of the following.

- a) The method of foundation excavation and preparation.
- b) Construction of levelling pad.
- c) The method of placement of expanded polystyrene blocks including temporary ballasting and protection of blocks during installation. The shop drawings shall indicate laying pattern and block dimensions on a layer-by-layer basis.
- d) The method and limits of placement of polyethylene sheeting.
- e) The method of placement of 125 mm reinforced concrete base pad (or equivalent).
- f) The method of placement of subbase material.
- g) The method of placement of side slope cover.

6.4 Quality Verification Engineer

- (1) The Contractor shall submit details of the sequence and method of installation to the Quality Verification Engineer for review. The submittals shall satisfy the specifications and at a minimum include a detailed description of proposed installation procedures. The details shall be submitted at least three weeks prior to the installation of the rigid expanded polystyrene embankments the Contractor shall also submit to the Contract Administrator, for information purposes, details of the sequence and method of installation. The submittals shall satisfy the specifications and at a minimum contain the above information as provided to the Contractor's Quality Verification Engineer.
- (2) The Contractor shall submit to the Contract Administrator a Certificate of Conformance sealed and signed by the Quality Verification Engineer a minimum of one week prior to commencement of work under this item. The Certificate shall state that the installation procedures are in conformance with the requirements and specifications of the contract documents. Quality test certificates for each production lot supplied, showing compliance with all requirements of this special provision shall be obtained by the Contractor and submitted to the Contract Administrator prior to installation. Upon completion of the Expanded Polystyrene Embankment the Contractor shall submit to the Contract Administrator a Certificate of Conformance sealed and signed by the Quality Verification Engineer stating that the Expanded Polystyrene Embankment has been constructed in conformance with the installation procedures and specifications of the contract documents.

7.0 MATERIALS

7.1 Granular Levelling Pad

The levelling pad shall consist of a Granular "A" material with gradation and physical requirements as specified in OPSS 1010.

7.2 Rigid Expanded Polystyrene

7.2.1 General

7.2.1.1 The Contractor shall submit:

1. A general statement as to the type, composition, and method of production of the material.
2. The manufacturer's name, address, phone number, identification of a contact person and description of experience background in the manufacturing of the rigid expanded polystyrene.
3. Certification of compliance of physical and mechanical properties.
4. An identification of a laboratory accredited by the Standards Council of Canada to conduct the testing of the physical and mechanical properties of the rigid expanded polystyrene.
5. The physical and mechanical properties of the rigid expanded polystyrene including:
 1. Geometry
 2. Nominal Density
 3. Compressive Strength
 4. Flexural Strength
 5. Thermal Resistance
 6. Dimensional Stability
 7. Flammability
 8. Water Absorption
6. Aging and durability characteristics of the polystyrene including the chemical, biological and ultra-violet degradation resistance of the rigid polystyrene.
7. A sample of the expanded polystyrene material to the Quality Verification Engineer for review.
8. To the Contract Administrator a Certificate of Conformance sealed and signed by the Quality Verification Engineer a minimum of one week prior to commencement of work under this item. The Certificate shall state that the expanded polystyrene material is in conformance with the requirements and specifications of the contract documents.

7.2.1.2 Production Lots

Each block of the same production lot shall be stamped with the same production code showing plant identification, type and date of production. The polystyrene shall be free from defects affecting serviceability.

7.2.2 Detail Requirements

Requirements shall be as shown in Table 1 and as described below.

Table 1 – Material Properties

PROPERTY	UNIT	REQUIREMENTS	TEST PROCEDURE
Geometry - Linear - Flatness - Squareness - Thickness	mm	1200 x 600 x 300 with tolerances $\pm 1\%$ 10 mm in 3 m $\pm 0.5\%$ -3, +5	
Compressive Strength	kPa (min)	110	ASTM D1621 (Procedure A)
Flexural Strength	kPa (min)	240	ASTM C203
Dimensional Stability	% linear change (max)	1.5	ASTM D2126
Thermal Resistance	m ² .°C/W (min for 25 mm thickness)	0.7	ASTM C177 or C518
Flammability	Limiting Oxygen Index (min)	24	ASTM D2863
Water Absorption	% by Volume (max)	4	ASTM D2842

7.2.2.1 Geometry

The expanded polystyrene shall be supplied in the form of rectangular parallel blocks of minimum acceptable dimensions of 1200 mm x 600 mm x 300 mm.

The maximum deviation from the specified linear dimensions shall be $\pm 1\%$. The flatness of the block faces shall be within ± 10 mm of a line formed by a 3 m straight edge.

The maximum difference in corner-to-corner dimensions (squareness) shall be 0.5%. The thickness shall be within -3 to $+5$ mm.

7.2.2.2 Compressive Strength

The minimum compressive strength, measured in accordance with ASTM D1621, Procedure A, shall be 110 kPa at a strain of not more than 5%. The maximum permissible permanent stress level should not exceed 30% of the compressive strength of the material at 5% strain.

7.2.2.3 Flexural Strength

The minimum flexural strength of the polystyrene shall be 240 kPa. The flexural strength shall be determined in accordance to ASTM C203, method 1, Procedure B.2.7.4 Dimensional Stability.

7.2.2.4 Dimensional Stability

Dimensional Stability shall be determined in accordance with ASTM D2126, Procedure G. A tolerance of 1.5% shall be satisfied.

7.2.2.5 Thermal Resistance

The thermal resistance shall be $0.7 \text{ m}^2 \cdot ^\circ\text{C}/\text{W}$ for a 25 mm thickness using the following equation and using the average value from three specimens:

$$R_{25\text{mm}} = \frac{R_{\text{measured}}}{\text{thickness (mm)}} \times 25$$

The thermal resistance shall be measured in accordance with ASTM C177 or C518.

7.2.2.6 Flammability

The expanded polystyrene shall be classified as to surface burning characteristics in accordance with CAN/ULC - 51022 having a flame spread rating less than 500. The expanded polystyrene shall have a minimum limiting oxygen index measured in accordance with ASTM D2863

7.2.2.7 Water Absorption

The water absorption as measured by ASTM D2842 shall be limited to 4% by volume.

7.2.2.8 Chemical Resistance

The expanded polystyrene shall be resistant to common inorganic acids and alkalies. A table identifying the chemical resistance as either resistant limited or not resistant shall be submitted.

7.2.2.9 Biological Resistance

The expanded polystyrene shall be resistant to biological degradation caused by organisms or enzymes.

7.2.2.10 Environmental

The expanded polystyrene shall be inert, non-nutritive and highly stable and shall not produce undesirable gases or leachate.

8.0 DELIVERY, STORAGE AND HANDLING

The product shall be suitably marked to identify its type, number and the manufacturer's name or trademark.

The Contractor shall protect the expanded polystyrene from exposure to sunlight to avoid ultraviolet degradation as per manufacturer's recommendation.

Protection of materials and works from damage by weather, traffic, construction staging, fire or vandalism and other causes shall be the responsibility of the Contractor.

9.0 CONSTRUCTION

9.1 Foundation Excavation

Foundation excavation shall be carried out to the design elevations shown on the drawings. Any softened, loosened or deleterious materials at the foundation footing elevation shall be subexcavated and replaced with Granular 'A' or Granular 'B' material.

9.2 Leveling Pad

Place, level and compact a layer of Granular 'A' or Granular 'B' material in accordance with OPSS 501 to within ± 30 mm of the design elevation. The leveling pad shall not deviate by more than 10 mm at any place on a 3 m straight edge over the limits of the bottom course of blocks. The leveling pad shall not be placed on frozen ground.

9.3 Installation of Blocks

- (1) The individually marked blocks shall be placed on the prepared leveling pad. The top surface of the first layer of blocks is to be set plane and level. Local trimming of the blocks may be necessary.
- (2) Subsequent successive layers shall be oriented with the long axis of blocks positioned at 90° to the previous layer in order to avoid continuous joints. Block joints shall be offset and staggered between layers.
- (3) A continuous check shall be kept to ensure the evenness of the blocks is satisfactory in each layer. Blocks shall be laid with joints with maximum opening of 10 mm between blocks. Differences in heights between adjacent blocks in the same layer should not exceed 5 mm.
- (4) Sloping end adjustments at the abutments shall be accomplished by leveling terraces in the subsoil in accordance with the block thickness.
- (5) Temporary ballast shall be provided as necessary to prevent movement of expanded polystyrene both in storage and as placed due to windy conditions. Timber fasteners or equivalent shall be used as necessary.
- (6) The expanded polystyrene embankment shall be protected from accidental ignition due to welding, smoking, grinding or cutting tools, etc. The Contractor shall take all necessary precautions to prevent ignition of the expanded polystyrene.
- (7) The expanded polystyrene shall be protected from organic solvents and other aggressive, harmful chemicals during construction. The proposed method of protection during construction shall be submitted to the Contractor's Quality Verification Engineer for review and to the Contract Administrator for information purposes.
- (8) Exposed blocks shall be covered immediately to avoid possible burrowing by animals.
- (9) Individually marked blocks shall be fabricated and placed to ensure the top surface matches the elevation and crossfall shown on the drawings.
- (10) The top surface and side surfaces of the expanded polystyrene shall be covered with 0.6 mil polyethylene sheeting extending onto adjacent work at the longitudinal ends of the

embankment. All joints shall be lapped a minimum of 300 mm to provide a fully sealed enclosure.

(11) The contractor shall install the concrete base pad as detailed elsewhere in the contract.

(12) The side slope of the rigid expanded polystyrene embankment shall be covered with Lightweight fill and waste material as detailed elsewhere in this contract.

10.0 EQUIPMENT

All cutting of polystyrene materials shall be by electric equipment or by hand.

Heavy equipment shall be limited in weight and size and restricted in operation to avoid damaging the expanded polystyrene as per the manufacturer's requirement.

11.0 QUALITY ASSURANCE

General

The Contract Administrator may undertake an independent testing program of the expanded polystyrene. Sampling and testing will be carried out in conformance with the relevant test procedure. The physical and thermal property testing identified in Table 1 will be conducted. A recognized testing laboratory accredited by the Standards Council of Canada shall conduct the testing.

Sampling Frequency

Sufficient sample material shall be obtained from blocks randomly selected by the Contract Administrator from each production lot as soon as the material arrives on site. As a minimum, three blocks shall be tested.

Acceptance/Rejection

Failure of any one of the sample blocks to comply with any requirements of this special provision shall be cause for rejection of the production lot from which it was taken. Replacement of the blocks shall be at the Contractor's expense.

12.0 MEASUREMENT FOR PAYMENT

Actual Measurement

Measurement will be by volume in cubic metres measured in its original position and based on cross-sections.

13.0 PAYMENT

Basis of Payment

The Concrete Base pad and granular leveling pad shall be paid for with the appropriate tender items as detailed elsewhere in the contract.

Payment at the contract price for the above tender item shall be full compensation for all labour, materials and equipment to do the work as described above and no extra payments will be made.

WARRANT: Always with this tender item.