



FINAL REPORT

FOUNDATION INVESTIGATION AND DESIGN REPORT

**Murky Creek Bridge Replacement, Highway 584, Site No. 48E-004, District of
Thunder Bay, Ontario**

**Agreement No. 6014-E-0017
Assignment No. 10 & 11
GWP 6054-08-00
Geocres No. 42L-002**

Prepared for:

Ontario Ministry of Transportation
Northwestern Region Geotechnical Section
615 James Street South
Thunder Bay, ON P7E 6P6
Attn: Mike Satten

Ontario Ministry of Transportation
Pavements and Foundations Section
Materials Engineering and Research Office (MERO)
Building 'C', Room 223
1201 Wilson Avenue
Downsview, ON M3M 1J8
Attn: K.Ahmad

exp Services Inc.
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Foundation Investigation Report

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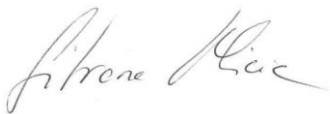
Ahileas Mitsopoulos, P.Eng.
Nimesh Tamrakar, M.Eng, EIT.
Demetri N. Georgiou, M.ASc. P.Eng.
Silvana Micic, Ph.D., P.Eng.

Reviewed By:


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

exp Services Inc.

56 Queen St, East, Suite 301
Brampton, ON L6V 4M8
Canada



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



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

Date Submitted:

December 12, 2016

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1 FOUNDATION INVESTIGATION REPORT

1.1 Introduction

This foundation investigation report presents the results of two geotechnical investigations (combined as one) completed by **exp** Services Inc. for the replacement of the Murky Creek Bridge, located on Highway 584, about 44 km north of Geraldton, Ontario, in the District of Thunder Bay, the Ministry of Transportation (MTO) Northwestern Region. The work was undertaken under Agreement # 6014-E-0017, Assignment Nos. 10 and 11 (GWP 6054-08-00). The terms of reference (TOR) for Assignment 10 were as presented in the MTO letter dated February 16, 2016, requesting that a minimum of two boreholes be conducted at each proposed abutment and a minimum of one borehole be conducted at each approach. The additional/expanded scope for Assignment 11 was presented in an MTO letter dated March 14, 2016, for a minimum of one borehole at each proposed pier location, due to a change in proposed design from a single span bridge to a three span bridge.

Based on the information provided and our observations, the existing bridge consists of steel girders with a concrete deck resting on wooden piers. The bridge has five (5) spans of a total length of about 55 m and a width of about 9.2 m. It is understood that the existing bridge was constructed in 1972. The bridge was initially intended to be replaced with a new three (3) span bridge with span lengths of 10.5 m at the ends and 21.5 m in the center but according to the new GA drawing provided on September 22, 2016, it is understood that the existing bridge is proposed to be replaced with a single span bridge with span length of 36.5 m and a width of about 10.5 m. The new bridge will be constructed of precast concrete box girder located along the same alignment and staged construction will be carried out to replace the existing bridge. It is also understood that a grade raise of about 0.4 m with respect to original road surface is currently proposed at this site.

The purpose of the Assignment 10 investigation was to evaluate the subsurface conditions at the locations of the proposed abutments as well as the approaches of the new bridge, while the purpose of the Assignment 11 investigation was to investigate the conditions at the two proposed pier locations. Both assignments were undertaken to permit detailed design for the bridge replacement. 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

As shown on Drawing 1 (Appendix B), the Murky Creek Bridge is located on Highway 584, about 44 km north of the Geraldton, in the District of Thunder Bay, Ontario. At the site, Hwy 584 is a two lane roadway, with a speed limit of 80 km/h and is about 9.2 m wide from edge to edge of the bridge, with steel guard rails along the bridge and extending to the approaches on both the north and south sides.

Based on drawings provided, the roadway embankment is about 3.0 m high with slopes on the east and west sides of the roadway of about 3H:1V. The ground at the north and south abutments slopes down towards the creek at about 2H:1V and 3H:1V, respectively.

During the fieldwork on March 14 to 20, 2016, the general site conditions were assessed. Hwy 584 runs in a generally north and south direction and the water flows from northwest to southeast beneath the highway. At the time of this investigation, the approximate creek elevation (top of ice elevation) was about 321.04 m. Steel guard rails were present on both sides of the roadway along the bridge and the approaches.

The upstream and downstream sides of the bridge appeared to be free of vegetation and other obstructions apart from the ice. The shoreline in the vicinity of the bridge, both upstream and downstream was snow covered; as such only trees and larger bushes were observed, both of which were present only sporadically along the shoreline and more prevalent inland from the shoreline.

Select photographs of the site are provided in Appendix A.

1.2.2 Geological Setting

According to the MNR Northern Ontario Engineering Geology Terrain Data Base Map, Ontario Geological Survey Map 5125, Scale 1:100,000, dated 1981, the underlying native soil at the site predominantly consists of bedrock knob, with subordinate landforms of sand glacial outwash plains or valley trains, peat organic terrain, or till ground moraine. The local relief is mainly low, undulating to rolling or knobby, with dry surface conditions in the bedrock knob and mixed in the subordinate landforms.

According to the Ministry of Northern Development and Mines (MNDM) Bedrock Geology of Ontario, West-Central Sheet Map No. 2542, Scale 1:1,000,000, dated 1991, the site lies near the border of two geologic formations of intrusive rock from the Neo to Mesoarchean Era (2.5 to 3.4 Ga). To the north is the foliated tonalite suite, consisting of tonalite to granodiorite, and to the south is granodiorite to granite. Both formations are massive to foliated in texture.

1.3 Investigation Procedures

1.3.1 Site Investigation and Field Testing

The field investigation was performed on March 14 to 20, 2016 based on the initially proposed three (3) span bridge replacement design. Since, due to the change in GA drawing (with a single span bridge) boreholes drilled at the piers locations may not be applicable for design purpose. However, these boreholes are also considered to describe the subsurface conditions encountered in this site in the section below.

The field program consisted of drilling eight (8) sampled boreholes (BH101 to BH108) within the traveled road way including six (6) through the bridge deck and into the ground below. Four (4) boreholes were located within proposed new bridge abutment locations. BH101 and BH104 were advanced at the proposed north abutment, and BH102 and BH103 at the proposed south abutment.

Two (2) boreholes (BH107 and BH108) were advanced at the proposed north and south pier locations (based on initial three span bridge design), respectively. BH105 and BH106 were located within the roadway at the south and north approaches, respectively. The borehole locations are shown on Drawing 1 in Appendix B.

All the boreholes (BH101 to BH108) were advanced using a CME 55 rubber track mounted drill rig. The drill rig was equipped with hollow and solid stem continuous flight augers and standard soil sampling equipment (includes 51 mm outside diameter split spoon samplers and *in situ* shear vane testing equipment). In addition, the CME 55 drill rig was equipped with rock coring equipment (NQ size). The boreholes BH101, BH102, BH103, BH104, BH107 and BH108 were advanced through the bridge deck after pre-coring through the concrete bridge deck using a dedicated concrete core drill equipped with a 10 inch (250 mm) concrete core bit, prior to advancing the augers or casing with the CME 55 drill rig.

At BH101, BH102 and BH104, rock coring techniques were initiated to advance the borehole beyond refusal at depths of about 1.4 m, 8.2 and 1.6 m, respectively. At BH103, rock coring techniques were conducted from ground surface in an attempt to expedite the drilling, rather than using hollow stem augers. BH107 and BH108 were also advanced using rock coring techniques from the surface of the boreholes, as these holes were advanced through ice and into, at the time, an unknown depth of flowing water (i.e. using casing was more stable/controlled to prevent loss of equipment). The same rock coring techniques were used to core and sample the bedrock at BH101, BH102, BH104, BH107 and BH108. No rock coring techniques were conducted at BH105 and BH106 (the approach boreholes).

The borehole locations were referenced to the MTM ON-14 NAD83 coordinate system and their ground surface elevations were surveyed by **exp** personnel. The ground surface elevations, including top of water at the creek (top of ice), were referenced to a geodetic benchmark (BM) provided (nail in tree root) south of the site and west of the highway. The BM elevation is 322.805 m. The location of the boreholes and the BM is shown on Drawing 1, in Appendix B.

During the drilling of the boreholes (BH101 to BH108), 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), and were generally performed at intervals of about 0.75 m. The original field (uncorrected) SPT "N" values were recorded on the borehole logs as recommended in the Canadian Foundation Engineering Manual and used to provide an assessment of *in-situ* compactness (cohesionless) or consistency (cohesive) soils. In addition, samples were collected directly from the field shear vane at BH105 (i.e. where sample was available).

Upon completion of the boreholes, groundwater level measurements were carried out in boreholes in accordance with the Ministry of Transportation guidelines. The measured groundwater levels after completion of drilling boreholes were recorded on borehole log sheets in Appendix C. The boreholes were backfilled with a mixture of bentonite and auger cuttings. The borehole decommissioning was in general 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 concrete core holes advanced through the bridge deck were repaired after drilling was completed. At the abutment boreholes (BH101 – BH104), i.e. where the underside of the bridge deck was readily accessible, plywood forms were placed and quick setting high strength concrete was poured to repair the bridge deck with about 50 mm to 100 mm of cold patch asphalt surfacing the repaired holes. The forms were left in place on the underside of the bridge deck. At the holes (BH107 and BH108), i.e. where the underside of the bridge deck was not readily access about 50 mm of additional asphalt was removed from the perimeter of the concrete core holes and a plywood sheet was placed directly onto the concrete deck. On top of the plywood sheet 100 mm of quick setting high strength concrete was poured and about 50 mm to 100 mm of cold patch asphalt was placed surfacing the hole,

The fieldwork was supervised by a member 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. All of the recovered soil samples were placed in labelled moisture-proof bags which, along with the rock cores, were brought to **exp's** Thunder Bay laboratory for additional visual, textual and olfactory examination, and for subsequent examination by a geotechnical engineer and laboratory testing.

1.3.2 Laboratory Testing

All samples brought 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 tests were carried out on select cohesive soil samples. All of the laboratory tests were carried out in accordance with MTO and/or ASTM Standards, as appropriate, at the **exp** laboratory in Thunder Bay, Ontario.

The laboratory test results are provided on the attached borehole log sheets in Appendix C as well as graphically in Appendix D.

In addition, chemical testing of two select soil samples were conducted. The soil samples was sent via courier, in a secure cooler under chain of custody, to Maxxam Analytics Inc., a CALA-certified and accredited laboratory in Mississauga, Ontario. Details of the chemical testing are discussed below and the lab results are included in Appendix E.

1.3.3 Previous Investigations

The following previous/historical investigation report was provided by the client.

1. Foundation Investigation Report for Proposed Crossing at Murky Creek and Hwy# 584 Unsurveyed Territory, District No. 19, Thunder Bay; W.O. 70-11058; W.P. 29-68-02; Geocres No. 42L-001; Department of Highways Ontario; November 1970.

1.4 Subsurface Conditions

The detailed subsurface conditions encountered in the boreholes advanced during this investigation are presented on the Borehole Records in Appendix C. Laboratory test results are provided in Appendix D. The "Explanation of Terms Used on Borehole Records" preceding the borehole logs in Appendix C

forms an integral part of and should be read in conjunction with this report. In addition, photographs of the bedrock core obtained are included in Appendix C.

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

At the proposed abutments and existing approaches, the subsurface conditions generally consist of a layer of fill material composed of sand, gravel, cobbles and boulders, overlying native sand or clayey silt to silt, overlying cobbles and boulders, and overlying bedrock. The subsurface conditions at the BH107 and BH108 lie within the stream bed, and generally consist of native sand and gravel overlying the silty sand to silt, overlying the cobbles and boulders, and overlying bedrock. A more detailed summary of the subsurface conditions encountered in the boreholes is provided in the following sections.

The ground surface at the proposed abutments (BH101, BH102, BH103 and BH104) was encountered at depths ranging between about 2.1 m and 2.6 m below the top of the bridge deck. The top of ice/water at the bore holes (BH107 and BH108) was about 3.4 m below the top of the bridge deck at both boreholes, and the ground surface was encountered at about 7.1 m and 6.1 m below the top of the bridge deck at BH107 and BH108, respectively. At all boreholes advanced through the bridge deck, the thickness of the asphalt and concrete were about 90 mm and 250 mm, respectively.

1.4.1 Sand with Gravel with Sand Fill

Well graded sand with gravel fill was generally encountered surfacing the boreholes and beneath the asphalt at BH105 and BH106. The asphalt thickness was about 200 mm and 230 mm at BH105 and BH106, respectively. The fill was generally described as frozen in the upper zones and brown, becoming loose to compact, and damp to wet. Sand blow up of about 150 mm was noted at 1.7 m depth below ground surface at BH102. The SPT "N" values ranged between 4 and 48 blows per 300 mm penetration, with an average "N" value of about 17. The sand fill extended to depths ranging between about 0.8 m and 3.8 m below ground surface, with elevations ranging between about 319.9 m and 321.4 m.

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

Moisture content:

- 2.3% to 13.0%

Grain size distribution:

- 26% to 85% gravel;
- 15% to 69% sand; and

- 0% to 5% silt and clay size.

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 Cobbles and Boulders Fill

Cobbles and boulders fill was encountered underlying the sand fill at BH101 and BH104. The cobbles and boulders fill extended to depths of 4.7 m (317.4 m elevation), and 4.6 m (317.6 m elevation) below ground surface at BH101 and BH104, respectively.

1.4.3 Peat

Peat was encountered underlying the sand fill at BH106. The peat was described as soft, dark brown, and wet. One SPT was conducted within the peat layer and the “N” value was 6 blows per 300 mm penetration. The peat layer was about 0.4 m in thickness and extended to a depth of about 3.8 m (320.9 m elevation) below ground surface.

Laboratory testing performed on the peat sample consisted of moisture content. The test results are as follows:

Moisture content:

- 52.5%

1.4.4 Poorly Graded Sand to Silty Sand

Poorly graded sand to silty sand was generally encountered underlying the fill. The poorly graded sand was also described as either with silt or with gravel. The poorly graded sand to silty sand was generally described as very loose to compact, brown to grey, and wet. At BH102, occasional cobbles and boulders were noted in the upper 2.4 m. In addition, at BH102, sand blow up of about 300 mm and 900 mm at about 2.4 m and 5.5 m below ground surface, respectively. The SPT “N” values ranged between 0 (i.e. advance by the weight of the rods and hammer) and 26 blows per 300 mm penetration, with an average “N” value of about 10. The poorly graded sand to silty sand extended to depths ranging between about 3.4 m and 8.2 m below ground surface, with elevations ranging between about 313.4 m and 319.4 m.

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

Moisture content:

- 7.9% to 38.0%

Grain size distribution:

- 0% to 43% gravel;
- 38% to 99% sand; and
- 1% to 31% silt and clay size.

Total saturated unit weights have been calculated based on the moisture contents and are estimated to range from about 18.0 to 23.6 kN/m³.

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 Figures 2 and 3, in Appendix D.

1.4.5 Clayey Silt

Clayey silt was encountered within the poorly graded sand to silty sand at BH102, BH103 and below silt with sand layer at BH105. The clayey silt was generally described as soft to stiff, grey, and wet. Blow up of about 600 mm was noted at 5.5 m below ground surface at BH102. The SPT “N” values ranged between 0 (i.e. advanced by weight of hammer and rods) and 11 blows per 300 mm penetration; an SPT “N” value of 100 (i.e. SPT refusal) was noted in BH105, however, this is likely refusal on bedrock or cobbles and boulders. Three (3) *in situ* field vane tests were performed and the results ranged between about 17 kPa and 83 kPa. The clayey silt extended to depths ranging between about 4.9 m and about 7.9 m below ground surface, and elevations ranging between about 316.4 m and 317.4 m. The clayey silt was present at the termination depth of BH105 at about 7.9 m (316.4 m elevation) below ground surface.

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

Moisture content:

- 21.6% to 50.8%

Grain size distribution:

- 0% gravel;
- 0% to 4% sand;
- 40% to 66% silt; and
- 31% to 56% clay sizes.

Total saturated unit weights have been calculated based on the moisture contents and are estimated to range from about 16.8 to 20.3 kN/m³. Three (3) Atterberg Limits tests were performed on

representative samples of the silty clay to clayey silt (BH102-S6B, BH103-S6 and BH105-S10). The results indicated that the soil is of low to medium plasticity. The data is shown on the plasticity chart, Figure 5. The liquid limit, plastic limit and plasticity index ranged between about 28 and 35, 15 and 16, and 13 and 19, respectively.

The results of the moisture content, grain size distribution, *in-situ* field vanes and Atterberg Limits tests are provided on the record of borehole sheets in Appendix C. The results of the grain size distribution are also provided on Figure 4 in Appendix D, and Atterberg Limits tests are provided on Figure 5 in Appendix D.

1.4.6 Silt

Silt was generally encountered underlying the poorly graded sand to silty sand. The silt was generally described as loose to very dense, grey, and wet. Silt and/or sand blow up of about 1,200 mm was noted at 7.0 m below ground surface at BH102. The SPT "N" values ranged between 7 and 100 (i.e. SPT refusal) blows per 300 mm penetration, with an average "N" value of about 28. Four (4) *in situ* field vane tests were performed, as some cohesive properties were noted, yielding results ranging between about 25 kPa and <122 kPa. The silt extended to depths ranging between about 5.0 m and about 8.8 m below ground surface, and elevations ranging between 312.5 m and 318.1 m.

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

Moisture content:

- 20.8% to 26.2%

Grain size distribution:

- 0% gravel;
- 0% to 36% sand;
- 57% to 91% silt; and
- 7% to 14% clay sizes.

Total saturated unit weights have been calculated based on the moisture contents and are estimated to range from about 19.6 to 20.5 kN/m³. Three (3) Atterberg Limits tests were performed on representative samples of the silt (BH106-S6, BH107-S4 and BH108-S6). The results indicated that the soil contains little cohesive properties. The data is shown on the plasticity chart, Figure 5. The liquid limit, plastic limit and plasticity index ranged between about 18 and 23, 16 and 17, and 3 and 6, respectively.

The results of the moisture content, grain size distribution and Atterberg Limits tests are provided on the record of borehole sheets in Appendix C. The results of the grain size distribution are provided on Figure 4 in Appendix D, and Atterberg Limits tests are provided on Figure 5 in Appendix D.

1.4.7 Cobbles and Boulders

Cobbles and boulders were encountered underlying the silt at BH101, BH102, BH103 and BH108, as well as beneath the poorly graded sand at BH104. The cobbles and boulders extended to depths ranging between about 7.3 m and 12.5 m below ground surface, and elevations ranging between about 309.8 m and 312.3 m. Cobbles and boulders were present at the termination depth of BH103 at about 11.9 m (310.4 m elevation) below ground surface.

1.4.8 Bedrock

Bedrock was encountered underlying the cobbles and boulders at BH101, BH102, BH104 and BH108, and beneath the silt at BH107. The bedrock was encountered at depths ranging between about 5.0 m and 12.5 m below ground surface. The bedrock was encountered at elevations ranging between 309.8 m and 312.5 m. The bedrock was generally described as medium strong (25 MPa to 50 MPa compressive strength), white, black and red to pink in colour, severely fractured to very sound and medium grained, granite. The boreholes were extended by rock coring to depths ranging between about 1.4 m to 3.4 m into bedrock, and to depths ranging between about 8.1 m and 15.5 m below ground surface. The boreholes were terminated at elevations ranging between about 306.8 m and 310.9 m. Photographs of the bedrock core samples are presented in Appendix C, after the Borehole Logs.

Gross recoveries ranged between about 80% and 100%. The Rock Quality Designation (RQD), which is a modified core recovery, ranged between 35% and 100% (severely fractured to very sound).

1.5 Groundwater and Surface Water Conditions

Information on groundwater levels at the site was obtained by measuring the water levels in the open boreholes after completion of drilling. The groundwater levels encountered in the boreholes are shown on the borehole logs and presented below in Table 1.1.

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.

Table 1.1. Groundwater data

Borehole	Date Completed	Date Measured	Ground Surface Elevation ²	Depth to Water ³	Groundwater Elevation
BH101	Mar. 16/16	Mar. 16/16	322.01	0.97	321.04
BH102	Mar. 18/16	Mar. 18/16	322.29	0.91	321.38
BH103	Mar. 19/16	Mar. 19/16	322.30	0.69	321.61
BH104	Mar. 16/16	Mar. 16/16	322.20	0.57	321.63
BH105 ⁴	Mar. 16/16	--	324.73	--	--

Borehole	Date Completed	Date Measured	Ground Surface Elevation ²	Depth to Water ³	Groundwater Elevation
BH106	Mar. 14/16	Mar. 14/16	324.31	4.40	319.91
BH107	Mar. 17/16	Mar. 17/16	317.49	-3.66	321.15
BH108	Mar. 20/16	Mar. 20/16	318.34	-2.74	321.08
Murky Creek (top of ice)	--	Mar. 18/16	--	--	321.04 ⁵

Notes:

- 1) All units in metres.
- 2) Elevations surveyed are referenced to a geodetic benchmark (BM) provided (nail in tree) south of the site and west of the highway. The BM elevation is 322.805 m.
- 3) Depths are relative to ground surface. Negative value indicates water/ice encountered above ground surface.
- 4) BH105 collapsed at time of auger removal, depth to groundwater could not be measured.
- 5) Indicates top of ice elevation at approximately the centre of bridge.

1.6 Chemical Analyses

Two soil samples were selected for chemical analyses and were sent via courier, in a secure cooler under chain of custody, to Maxxam Analytics Inc., a CALA-certified and accredited laboratory in Mississauga, Ontario. The analytical laboratory results are presented in Appendix E, and are summarized in Table 1.2, below.

Table 1.2. Corrosivity chemical analysis

Sample Identification	pH (unitless)	Soluble Chloride (ppm)	Soluble Sulphate (ppm)	Resistivity (ohm-cm)	Conductivity (μS/cm)
BH101-S5 Sand with Silt	8.05	<20	30	11,000	89
BH104-S2 Sand with Gravel Fill	7.63	240	<20	1,700	575

2 DISCUSSIONS AND ENGINEERING RECOMMENATIONS

2.1 General

This section of the report provides geotechnical design recommendations for Murky Creek Bridge replacement on Highway 584, located about 44 km north of Geraldton, Ontario, in the District of Thunder Bay, the Ministry of Transportation (MTO) Northwestern 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 structure. 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.

Based on information included in the TOR and our observations, the existing bridge consists of steel girders with a concrete deck resting on wooden piers (Photographs No. 1 in Appendix A). The bridge has five (5) spans of a total length of about 55 m and a total width of about 9.2 m. It is understood that the existing bridge was constructed in 1972. It is also understood that the bridge is supported by timber piles driven to refusal on bedrock.

Initially the Murky Creek Bridge was proposed to be replaced with a three (3) span structure with spans of 10.5 m, 21.5 m and 10.5 m (south, middle and north). Current, new design proposed a single span structure with span of 36.5 m with a precast concrete box girder bridge for replacement of the Murky Creek Bridge. It is understood that the bridge will be replaced on the same alignment with abutments set inside the existing abutment as noted on Drawing 1 in Appendix B and staged construction will be carried out to replace the existing bridge. A grade raise of about 0.4 m with respect to original road surface is proposed at the site. A RSS wall is proposed along the total width of roadway and on sides measuring about 10.6 m long (sides of abutment) and approximately 6.0 m and 6.2 m high behind the south and north abutment, respectively.

During site investigation, the water level in the creek was measured at elevation 321.04 m. Seasonal variations in the water level should be expected, with higher levels occurring during wetter periods of the year and lower levels during drier periods.

This report addresses the geotechnical design of the foundation for the proposed bridge structure by providing geotechnical design parameters at the Ultimate Limit State (ULS) and Serviceability Limit States (SLS) as well as other geotechnical parameters that may be required in accordance with the latest edition of the *Canadian Highway Bridge Design Code (CHBDC)* (CAN/CSA-S6-14), the *Guideline for Professional Engineers Providing Geotechnical Engineering Service* (1992), the *Canadian Foundation Engineering Manual (CFEM)* (2006), the *provisions in the TOR* and good practice. It also provides discussion about the structure foundation type and stability analyses, as requested in the TOR.

2.2 Geotechnical Design Considerations for Structure Foundations

In general, the subsurface conditions at proposed abutment locations consist of a layer of fill material composed of sand, gravel, cobbles and boulders overlying native sand or silty sand to silt, followed by a layer of cobbles and boulders, and overlying bedrock. The subsurface conditions at the BH107 and BH108 lie within the stream bed, and generally consist of native sand and gravel overlying the silty sand to silt, overlying cobbles and boulders, and overlying bedrock. The native deposits, consisting predominantly of sand and silt extend approximately to 8.0 m to 9.0 m below the existing ground surface. The compactness of the deposit varies from very loose to compact. The thickness of cobbles and boulders layer underlying sand and silt layer was found to be 1.6 m to 2.3 m at south abutment side and 3.1 m to 4.3 m at north abutment side. The bedrock was encountered at approximate depth of 9.7 m below ground surface on the north side of the creek and at approximate depth of 12.5 m below ground surface on the south side of the creek. The ground water level encountered at the site was at an approximate elevation of 321.04 m (i.e. the water level in the creek).

2.2.1 Foundation Alternatives

Given the relatively loose soil encountered under the fill, we recommend that the bridge foundation elements be founded on deep foundation such as steel H-piles driven to bedrock. The abutments can be designed either as integral or semi-integral supported on deep foundations. Caissons socketed into the bedrock are also feasible at this site, but not economical. Shallow spread footings are not recommended due to presence of relatively loose cohesionless deposits encountered below fill and the high creek water level. Due to this condition, it may not be feasible to properly dewater the excavation for construction of spread footing in dry condition. A comparison of the technical advantages and disadvantages of the alternative foundation schemes is presented in Table 2.1.

Table 2.1 Evaluation of foundation alternatives

Options	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
1. End bearing steel H- pile driven to unyielding bedrock	1	<ul style="list-style-type: none"> ▪ High geotechnical resistance available ▪ Negligible or minimum settlement ▪ Compatible for integral and semi-integral abutment 	<ul style="list-style-type: none"> ▪ May pose difficult driving condition through cobbles and boulder or possibility of piles “hanging up” on cobbles and boulders deposit ▪ Pile tip reinforcement is required to drive piles through cobbles and boulders ▪ Cofferdam construction may be required for pile cap construction adjacent to creek or in the creek 	<ul style="list-style-type: none"> ▪ Lower relative cost compared with Option 2 	<ul style="list-style-type: none"> ▪ Risk of pile tip damage, should adequately protected while driving through cobbles and boulders ▪ May not achieve minimum pile length, may need socketing pile into bedrock
2. Caissons socketed into the bedrock	2	<ul style="list-style-type: none"> ▪ High geotechnical resistance available ▪ Reduce number of deep elements compared to steel-H-piles ▪ Possible elimination of pile cap 	<ul style="list-style-type: none"> ▪ May be difficult to install caissons through cobbles and boulders ▪ Temporary liners would be required for groundwater control and support through the granular overburden ▪ Concrete for caissons would have to be placed by tremie methods below the water level ▪ No suitable for integral abutments 	<ul style="list-style-type: none"> ▪ More expensive than Option 1 	<ul style="list-style-type: none"> ▪ Potential installation difficulties through cobbles and boulders

2.2.2 Shallow Foundations

Based on the findings during the subsurface investigation presented in this report, spread footings are not recommended to support the proposed bridge structure since competent native soils are not encountered within reasonable depth. In addition, their construction would require cofferdam enclosures to enable foundation excavation extending below the creek water level. Considering these conditions, the spread footing option was not discussed further.

RSS walls are proposed to be used as false abutments and wing walls for this bridge. Based on the GA drawing RSS wall founding elevation is proposed approximately at Elevation 319.4 m, which is below the creek water level (Elev. 321.04 m). Therefore, in-order to construct strip footings for RSS walls cofferdam enclosures would be required to enable foundation excavation below the creek water level. Provided that the footings are constructed in dry condition using suitably designed cofferdams and/or appropriate dewatering, the geotechnical resistances for strip footings founded on granular levelling pad given in Table 2.2 below may be used.

Table 2.2. Recommended spread footing design parameters

Foundation Unit	Relevant Boreholes	Founding Elevation ¹ (m)	Assumed Footing Size (m)	Founding Soil Type	Factored Geotechnical Resistance at ULS (kPa)	Geotechnical Reaction at SLS ² (kPa)
South Abutment	BH102 BH103	~ 319.4 or above (Varies)	>1	Minimum 300 mm compacted granular material (Granular A or Granular B Type II) over Native compact sand underlain by very soft to stiff clayey silt	185	125
North Abutment	BH101 BH104	~ 319.4 or above (Varies)	>1	Minimum 300 mm compacted granular material (Granular A or Granular B Type II) over existing cobbles and boulders fill	225	150

Notes:

(1) below frost line of 2.6 m

(2) for maximum settlement of 25 mm

It is presumed that any soft or very loose materials encountered are to be replaced with clean and compactible soil such as Granular A or Granular B type II. Given that no significant grade raise is planned, the anticipated maximum total settlements for the walls are not expected to exceed 25 mm for construction done in accordance with these design parameters and assuming good construction

practice including sound base preparation.

Frost Protection and Embedment

The RSS retaining wall foundations consisting of strip footings should be provided with adequate embedment and frost protection comprised of 2.6 metres of soil cover or thermal equivalent. The RSS walls founded on levelling pads should also have sufficient embedment. The embedment depth, defined as the distance from the top of the levelling pad to the top of the adjoining finished grade at the toe of the wall should be the maximum of:

- 0.5 metres;
- The minimum depth required for overall stability;
- $H/20$ – if the area in front of the wall is horizontal; or
- $H/7$ – if the area in front of the wall slopes at 2 horizontal to 1 vertical.

where H is the total wall height (*MSEW and reinforced soil slope, design and construction guidelines, FHWA, 2001*).

In general, the embankment slope in front of the wall will be sloped at 2 horizontal to 1 vertical. To improve resistance to general bearing failure and to provide access for future maintenance and repairs, each RSS retaining wall should be provided with a bench in front of the wall with a minimum width of 1.0 metre measured from the face of the wall.

2.2.2.1 Resistance to Lateral Loads

Resistance to lateral forces/ sliding should be calculated in accordance with Section 6.9 of the CHBDC/CSA S6-06, using the following parameters:

Table 2.3. Recommended parameters for calculation of unfactored horizontal resistance

Interface Conditions	Parameter
Between RSS wall levelling pad and compact sand	Coefficient of friction ($\tan \delta$)=0.55
Between RSS wall levelling pad and cobbles and boulders	Coefficient of friction ($\tan \delta$)=0.6

The listed values are unfactored; in accordance with the CHBDC, a factor of 0.8 is to be applied in calculating the horizontal resistance.

2.2.3 Deep Foundations

2.2.3.1 Driven Steel Piles

Considering the site specific conditions, steel H-piles (HP 310 x 79 or HP 310 x 110) driven to bedrock can be used to support a bridge designed with integral or semi- integral abutments.

The piles will be installed through the very loose to compact sandy deposits and cobbles and boulders layers, and are expected to terminate on bedrock. It is anticipated that pile cap elevations would be below a frost depth of 2.6 m. Based on the depth to bedrock encountered in boreholes drilled adjacent to proposed abutment locations, it appears that the termination depths for the piles could be variable. However, for design purpose, the tip elevations for the piles discussed in this report estimated are given in Table 2.4.

Geotechnical Axial Resistances of Piles

The factored geotechnical axial resistances at ULS and geotechnical axial reactions at SLS for 25 mm of displacement for the recommended driven piles are presented in Table 2.4. These values represent the structural limitation for the pile rather than a geotechnical limitation. It is anticipated that for H-piles driven and seated on the underlying unyielding bedrock, the geotechnical resistance at SLS for 25 mm of settlement will be much greater than the factored axial resistance at ULS; as such, ULS conditions will govern for this foundation type and the SLS value would not apply.

Table 2.4. Factored geotechnical resistances for considered piles

Foundation Unit	Pile Founding Stratum	Estimated Tip Elevation (m)	Approx. Design Pile Length below underside of pile cap ^{1,2} (m)	Factored Geotechnical Axial Resistance at ULS ⁴ (kN/pile)		Geotechnical Axial Resistance at SLS (kN/pile) ³	
				HP 310 x 79	HP 310 x 110	HP 310 x 79	HP 310 x 110
South Abutment	Bedrock	~309.8	11.2	1,450	2,000	NA	NA
North Abutment	Bedrock	~311.7 to 312.3	8.7 to 9.3				

Notes:

- (1) based on GA drawing elevation underside of pile cap is about 321.0 m
- (2) pile cap below frost depth of 2.6 m
- (3) NA-not applicable since for H-piles driven and seated on the underlying unyielding bedrock, the geotechnical resistance at SLS for 25 mm of settlement will be greater than the factored axial resistance at ULS and ULS conditions will govern the design
- (4) In this case factored geotechnical resistances at ULS available for pile founded on unyielding bedrock is the same as the factored structural axial resistance for a pile founded on unyielding bedrock.

Resistance of Piles to Lateral Loads

For vertical piles, the resistance to lateral loading has to be derived from the soil in front of the piles. That resistance may be estimated using Subgrade Reaction Theory (with deformations less than 5% of pile diameter) in which the coefficient of horizontal subgrade reactions k_s is based on the following equations:

For cohesionless soils:

$$k_s = n_h(z/d)$$

For cohesive soils:

$$K_s = 67C_u/d$$

where,

k_s =coefficient of horizontal subgrade reactions (MPa/m)

d =pile diameter (m)

n_h =constant of horizontal subgrade reaction (MPa/m)

C_u =undrained shear strength (kPa)

z =depth below ground surface (m)

The lateral resistance of the piles should be developed primarily from the passive resistance of the soil. The values of n_h to be used to calculate the coefficient of horizontal subgrade reaction k_s which can be utilized in the structural analysis for the piles at this site are given in table below.

Table 2.5. Parameters for lateral load analyses

Structure Element	Strata	Elevation (m)	C_u (kPa)	n_h (MN/m ³)
North Abutment	Engineered Fill	-	-	6.6
	Loose to very dense sand with gravel, cobbles and boulders	320.7 -317.4	-	10
	Very loose to compact sand with silt to silt	317.6 – 314.0	-	1.3
South Abutment	Engineered Fill	-	-	6.6
	Compact to very dense sand with gravel	322.2 -319.9	-	10
	Very loose to compact sand	320.6 – 318.0	-	1.5
	Very Soft to stiff clayey silt	318.8 – 316.8	20	-
	Loose to very dense silty sand to silt, cobbles and boulders	317.4 – 309.8		10.0

Lateral loading could be resisted fully or partially by use of battered piles. The piles could be installed at a batter of up to 4 vertical to 1 horizontal by simply tilting the pile-driver leads.

Group action for lateral loading should be considered where the pile spacing in the direction of the loading is less than eight pile diameters. Group action can be evaluated by reducing the coefficient of lateral subgrade reaction in the direction of loading by a reduction factor R , as indicated in Table

2.6. Subgrade reaction reduction factors for other pile spacing values may be interpolated for pile spacing in between those listed in this table.

Table 2.6. Lateral load capacity reduction factor for pile group

Pile Spacing in Direction of Loading D=Pile Diameter/Width	Subgrade Reaction Reduction Factor R
8d	1
6d	0.7
4d	0.4
3d	0.25

Frost Protection

The pile caps should be provided with the minimum soil cover for frost protection as mention in Section 2.2.4 following below.

Negative Skin Friction (Downdrag Loads) on Piles

Since the approach embankments are going to be raised only about 0.4 m and the foundation soils are cohesionless (non-compressible), the negative skin friction (or downdrag load) will not need to be taken into consideration during design of the piles supporting the integral abutment.

Pile Installation

Piles will be driven to bedrock and the installation procedure could be followed as specified in NSSP attached in Appendix J. Presence of the cobbles and boulders layer encountered in boreholes must be considered for the proper pile installation. In view of this, the piles should be fitted with a driving shoe section (Titus 'H' Bearing Pile point, APF Hard Bite bearing points or similar) offering some protection against buckling at the toe as the piles are driven through the cobbles and boulders or the piles should be stiffened as per OPSD 3000.100, Type I to minimize damage to the piles in anticipation of heavy driving conditions. Care must be taken to avoid overdriving and damaging the pile tip (i.e., the structural capacity of the piles should not be exceeded).

In addition, all piles should be visually monitored by experienced personnel during installation to check for plumbness, set, internal damage, etc. All damaged piles should be rejected, or if the damage is considered to be minor, the pile can be dynamically tested to determine the available pile capacity.

Piles in groups should be spaced no closer than 3 pile diameters. All piles in a group should be checked for heaving during the driving of the adjacent piles.

2.2.3.2 Caissons

Alternatively, the proposed structure may also be supported on caissons socketed into the bedrock. However, this scheme is not suitable for integral abutment design. For completeness the design data for caissons are included. The high axial capacity of caissons would result in fewer units being required to support the bridge abutments than that required for the H –piles, as well as the possible elimination of pile cap. There will, however, be difficulty in socketing the large diameter caissons within sloping bedrock and achieving an adequate seal. Temporary liners and tremie concrete will be required to install caissons at this site.

Table 2.7 below provides the factored geotechnical axial resistances for different caisson diameters socketed a minimum of 2 m in to the bedrock. The given values for caissons were results mainly from the shaft resistance of the bedrock socket. The end-bearing will be neglected due to the difficulties in cleaning and inspecting the base of sockets.

Table 2.7. Caisson's geotechnical resistance

Location	Relevant Borehole	Foundation Elevation (m)	1.2 m Diameter Caisson		1.5 m Diameter Caisson	
			Factored ULS (kN)	SLS (kN)	Factored ULS (kN)	SLS (kN)
North Abutment	BH101 and BH104	309.7	6,500	N/A	8,000	N/A
South Abutment	BH102 and BH103	307.8		N/A		N/A

NA-not applicable since for caissons socketed into the bedrock, the geotechnical resistance at SLS for 25 mm of settlement will be greater than the factored axial resistance at ULS and ULS conditions will govern

To verify the soundness/structural integrity of the caissons, one of the following non-destructive evaluation tests may be performed:

- Cross-hole acoustic testing and backscatter gamma ray (gamma-gamma) tests through access tubes installed within the caissons during the placement of the concrete; or
- Sonic echo tests. The advantage of these tests is that they do not require preparation during construction of the caissons. The disadvantage is that these tests do not identify all imperfections in a caisson, but provides information about continuity, defects, such as cracks, necking, soil incursions, changes in cross section and approximate pile lengths, unless the pile is very long or the skin friction is too high.

Static load tests to confirm the bearing capacity of the caissons may also be completed as described in ASTM D1143-81 (compression test quick method) and ASTM D3966-90 (Lateral Test) or as per designer's specification.

Giving the uncertainties associated with cleaning and inspection of the caisson base, this foundation type is not the preferred option. As indicated, it is not suited for integral abutment designs.

2.2.4 Frost Protection

According to Ontario Provincial Standard Drawing (OPSD – 3090.100), the frost depth in the subject site is about 2.6 m. Consequently, all pile caps or footings exposed to seasonal freezing conditions must be protected from frost action by at least 2.6 m of soil cover or equivalent insulation.

2.2.5 Temporary Excavation Protection System

Temporary excavation protection system construction is required to facilitate the staged construction. It may also be required for the construction of RSS wall. Given the proximity of the creek and the depth of excavation required for the RSS wall foundation and abutment pile caps, consideration should be given to the use of temporary shoring such as sheet-pile-cut-off wall or other cofferdam type construction. Above the water level, other types of temporary shoring could be considered including braced excavations such as soldier pile and lagging or tie-back walls. If sheet-pile-cut-off wall was chosen, it should be of sufficient robust cross section to be driven through the cobbles and boulders contained in the existing embankment fill and immediate above the bedrock.

Design and construction specification for the roadway protection system should be prepared in accordance with OPSS 539. Piling should be in accordance with OPSS 903. Cantilevered walls should be designed for the earth pressures shown in Section 2.4 and earth pressure diagram shown in CFEM Figure 26.3.

2.2.6 Lateral Earth Pressure on Structures

The abutment stems, and temporary shoring that may be required for excavation should be designed to resist lateral earth pressure. Where the abutment stems can be drained effectively to eliminate hydrostatic pressure on the walls, earth pressures equation can be simplified in accordance with the the Canadian Highway Bridge Design Code (CHBDC).

The expression for calculating lateral earth pressure is given by:

$$P = K(\gamma h + q) \text{ for non-braced cut, or } K(0.65\gamma H + q) \text{ for braced support}$$

where P = earth pressure intensity at depth h , kPa

K = earth pressure coefficient

γ = unit weight of retained soil, kN/m³

q = surcharge near wall, kPa

h = depth to point of interest, m

H = depth of excavation (m)

The mobilization of full active or passive resistance requires a measurable and perhaps significant wall movement or rotation. Therefore, unless the structural element can tolerate these deflections, the at-rest earth pressure should be used in design.

The effect of compaction surcharge should be taken into account in the calculations of active and at-rest earth pressures. The lateral pressure due to compaction should be taken as at least 12 kPa at the surface, and its magnitude should be assumed to diminish linearly with depth to zero at the depth where the active (or at rest) pressure is equal to 12 kPa. This pressure distribution should be added to the calculated active (or at rest) pressure. Notwithstanding, lighter compaction equipment and smaller lifts should be used adjacent to walls to prevent overstressing.

For design purposes, the unfactored static earth pressure parameters given in Table 2.9 can be used (assuming wall friction is neglected, the back wall is vertical and the ground surface is horizontal both on the retained side as well as in front of the toe):

Table 2.9 Material types and unfactored earth pressure properties under static conditions

Material	Unfactored Friction Angle ϕ' (°)	Coefficient of Active Earth Pressure (K_a)	Coefficient of Passive Earth Pressure (K_p)	Coefficient of Earth Pressure at Rest (K_o)	Unit Weight γ kN/m ³
Compacted Granular A	35	0.27	3.69	0.43	22
Compacted Granular B, Type II	32	0.31	3.25	0.47	21
Sand with gravel fill	32	0.31	3.25	0.47	20
Cobbles and boulders fill	36	0.26	3.87	0.41	20
Native loose to compact sand to silty sand	30	0.33	3.0	0.5	19
Soft to stiff clayey silt	27	0.38	2.64	0.55	19
Compact to very dense silt	30	0.33	3.0	0.5	20

2.2.7 Earthquake Considerations

Seismic loading may result in increased lateral pressure acting on the abutment stem and retaining walls. The walls should be designed to withstand the combined lateral loading for the appropriate static pressure conditions given above, plus the earthquake-induced dynamic earth pressure.

Seismic characterization of the site must be compliant with CHBDC (CAN/CSA-S6-14). The potential for seismic loading must be considered for design of abutment in accordance with Section 4.4 of the CHBDC. With respect to soil conditions encountered at the site, the borehole information shows the presence of approximately 5 m to 14 m thick layers of overburden soils (i.e. sand with gravel fill, very loose to compact sand to sandy silt and very dense layer of cobbles and boulders) overlying bedrock, such condition fall into the category defined in Section 4.4.3.2 which is a Site Class B.

From the NBCC seismic calculation, the damped reference spectral accelerations for the project site are $S_a(0.2)=0.035g$, $S_a(0.5)=0.022g$, $S_a(1.0)=0.011g$, $S_a(2.0)=0.003g$ and the reference peak ground acceleration (PGA) is $0.011g$ (g =acceleration due to gravity -9.81 m/s^2). These values are associated with an earthquake having 10 percent probability of exceedance in a 50-year period.

2.2.8 Stability and Settlement Analyses

2.2.8.1 Stability

To assess the global stability of the forward slopes of the north and south abutments and to check that a minimum Factor of Safety (FOS) of 1.5 for static conditions and 1.1 for seismic conditions will be achieved a series of slope stability analyses were performed based on the existing bridge drawing provided by MTO. The seismic condition was analyzed for the completeness of the assessment. In addition, global stability of the RSS wall which would be used in this project was also analyzed. The static and seismic slope stability analyses were performed using the Morgenstern-Price method developed on the basis of limit equilibrium. The SLOPE/W computer program developed by GeoSlope International was employed for computation.

Given the above, effective stress analyses for a long term stability assessment were performed taking into consideration the subsoil conditions encountered directly beneath and adjacent the proposed bridge.

Tabulated below in Table 2.10 are the soil parameters used for the slope stability analyses. The soil parameters were generally estimated based on the results of field and laboratory investigation.

Table 2.10 Soil properties used in slope stability analyses

Material Type	Effective Stress Parameters		
	ϕ' (degrees)	c' (kPa)	γ (kN/m ³)
Engineered fill – Granular A or Granular B Type II	32	0	21
Sand with gravel fill	32	0	20
Cobbles and boulder fill	36	0	20
Loose gravel	32	0	20
Loose to compact sand to silty sand	30	0	19
Soft to stiff clayey silt	27	0	19
Compact to very dense silt	30	0	20
Very dense cobbles and boulders	38	0	20

The results of the slope analyses for the south and north abutments are presented on Figures F1 to F8 in Appendix F.

As shown on the figures, the results of stability analyses suggest that the FOS greater than required (FOS 1.5 for static conditions and 1.1 for seismic conditions for abutment and FOS 1.3 for static conditions for RSS wall) can be obtained for the forward slopes of approximately 2H:1V at the south side and the north side, assuming that the adequate erosion protection is applied at the forward slopes.

2.2.8.2 Settlement

Considering that the approach embankments are going to be raised about 0.4 m no significant settlement of mostly cohesionless foundation soils is anticipated at the site. However for the sake of completeness, the total settlement is predicted using a computer program, Settle3D (Rocscience). Settle3D is a 3-dimensional program for the analysis of vertical consolidation and settlement under foundations, embankments and surface loads. The program combines the simplicity of one-dimensional analysis with the power and visualization capabilities of more sophisticated three-dimensional programs.

The magnitudes of total settlement for the north and south embankments have been assessed based on Standard Penetration Test (SPT) results with in-situ field vane shear testing, and site-specific correlations between consolidation parameters and the natural moisture content of samples

obtained. Soil compressibility parameters adopted in the settlement analyses for the north and south approach embankment are summarised in Table 2.11.

The value of the consolidation indices given in the tables were estimated based on the empirical relationship between the compressive index and moisture content of the soil, as well as based on available background data in the general area, bearing in mind site specific variations.

Following equation is used to estimate compressive index (C_c):

$$C_c = 0.01 (W_n - 5)$$

Based on the results of moisture content tests, natural moisture contents of the clayey silt soil was between 20% and 40%. Therefore, the estimated the estimated compression index using the empirical equation is between 0.15 and 0.35. The preconsolidation pressure (p'_c) of 100 kPa on south abutment and 180 kPa on north abutment t was estimated based on available lab data on the similar soil in the area.

Table 2.11. Soil parameters used in settlement analyses

Soil Layers	Unit Weight (kN/m ³)	E (MPa)	Compression Index (C_c)	Recompression Index (C_r)	Void Ratio (e)	Preconsolidation Pressure (p'_c) (kPa)
Engineered Fill	21	30	-	-	-	-
North Approach Embankment						
Sand and gravel, cobbles and boulders fill compact to very dense	20	50	-	-	-	-
Loose to compact sand to silt	19	20				
Very soft to firm clayey silt	18	-	0.3	0.03	0.94	100
Cobbles and boulders	20	500	-	-	-	-
South Approach Embankment						
Sand and gravel fill compact to very dense	20	50	-	-	-	-
Very loose to compact sand	19	15				
Very soft to stiff clayey silt	18	-	0.3	0.03	0.94	150
Loose to Compact silty sand to silt	19	25	-	-	-	-
Cobbles and boulders	20	500	-	-	-	-

The summary of results of settlement analyses for the approached embankments is given in Table

2.12. The Settle 3D results of these cases can be seen Appendix G. Based on MTO's "Embankment Settlement Criteria for Design", the maximum settlement limits during pavement design life are 50 mm and 100 mm for non-compressible and compressible soils, respectively. However, the maximum settlement at structure/embankment interface during pavement design life should be 25 mm for distance of 0 - 20 m from transition point.

Table 2.12. Summary of results of settlement analyses

Locations	Grade Raise (m)*	Foundation Soil Type	Calculated Total Settlement (mm)	Calculated Immediate Settlement (mm)
North Approach	3.0	See Table 2.8	25	17
South Approach	3.5	See Table 2.8	30	9

Notes:

*The grade raise estimated based on the existing ground level at proposed abutment location and proposed grade raise of about 0.4 m of approach embankment.

The settlement analyses suggested that the total settlement could be in the order of 25 to 30 mm, at the locations of north and south approach embankments, respectively. It is expected that post construction settlement will be within allowable limit at the structure/embankment interface. Settlement of embankment fill itself should also expect depending on the material and placement methods.

2.2.9 Construction Considerations

2.2.9.1 Excavation

All excavations must be carried out in accordance with the latest edition of the Ontario Occupational Health and Safety (OHS) and good construction practice. The fill and native soils which may need excavation for construction are considered as Type 3 soils above the groundwater table and Type 4 soils below the groundwater table. Temporary excavations (i.e. those that are open only for a short period) above the groundwater table may be made with side slopes not steeper than about 1H:1V, while the temporary slopes below the groundwater table have to be formed at 3H:1V unless a suitable dewatering system is installed to lower the water level below the base of the excavation. There is a potential for sloughing to occur if the cut remains open for an extended period of time (i.e. > 24 hours) or during a rainfall event. In addition, some localized surficial sloughing may be experienced in areas of perched groundwater seepage (i.e. within the embankment fill).

2.2.9.2 Dewatering

Since the groundwater level encountered at higher level in the boreholes at abutment locations the perched water within the embankment fill is anticipated during the excavation works for abutments.

For construction of pile caps at abutment locations, groundwater level needs to be controlled to 0.5 m below the excavation level to avoid disturbance, and any surface or groundwater seepage should be removed from the excavation prior to any bedding material being placed. In general, pumping using properly filtered sumps, and/or filtered drains placed along the base of the excavation should provide sufficient groundwater control during foundation works. However, dewatering schemes are the responsibility of the contractor and qualified designer should be used to develop system to achieve the needed control.

In order to construct a shallow foundation for RSS wall, cofferdams will be required to envelop the construction site and keep it free of water during pile cap construction. The design of these cofferdams, which are temporary retaining structures, is the responsibility of the Contractor. The cofferdam must be designed to withstand the anticipated design loads and to be watertight as practically possible. The Contractor is also responsible for cofferdam's materials, construction, monitoring and removal. Cofferdams should be designed in accordance with OPSS 539 by a licensed Professional Engineer experienced in shoring design.

Dewatering requirements behind the cofferdams to keep the construction site dry will be impacted by water levels in the creek at the time of construction activities. Dewatering shall be carried out in accordance with OPSS 517 and OPSS 518. It is responsibility of the Contractor to propose a suitable dewatering system based on the time of construction, water levels and flow conditions for prior approval of the MTO.

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

2.2.9.3 Obstructions

As part of the design and construction of the new foundations, careful consideration should be given to the location of the existing foundations (timber piles) relative to the new construction, temporary shoring and cofferdams. Specially, the designer should check that the new piles (batter and orientation) and temporary shoring don not interfere with the existing piles. This should be checked to the full extent of the pile/shoring length.

Cobbles and boulders was encountered within the existing fill at the north abutment location and overlying the bedrock at both north and south abutment locations. Therefore, a Non-Standard Special Provision (NSSP) to alert the contractor about the presence of cobbles and boulders in the embankment fill and overlying bedrock should be considered. Suggestions for the NSSP are included in Appendix j.

2.2.10 Scour Protection

The scour design including mitigation measures is the responsibility of a qualified hydraulic engineer. Geotechnical soil parameters pertinent to scour analyses are the following: SPT N-value, insitu moisture content, percent passing the No. 200 sieve (% 200), mean grain size diameter (D₅₀), liquid limit (LL), plastic limit (PL), and plasticity index (PI). The parameters for this site can be found / interpreted on the borehole logs and on the graphs attached in Appendix D. All tested soils were classified using the Unified Soil Classification System which can be used for evaluation of erosion rates. Pertinent geotechnical parameters to support this design have been provided in this report as noted above. Foundation recommendations outlined in this report assumes that proper scour protection is used. The following additional information is provided for general guidance.

2.2.10.1 Bank Slopes

The design embankment slope should not be steeper than 2H: 1V. Further limits on side slope steepness may be imposed by slope instability, groundwater flows, or rapid water level recession and piping failure, all of which should be carefully considered in slope design.

Rock riprap revetments are normally continued to the top of the bank or to design water level, plus a freeboard, if the bank is not over topped. Freeboard is added to account for wave, runoff, super-elevation, profile irregularities, floating debris, ice and surface waves.

2.2.10.2 Toe

Toe scour, along revetments, is thought to be the most common cause of failure. The following are commonly used to prevent undermining, as described below:

The slope is excavated and covered with rock riprap to below expected scour levels. This method is most permanent, but it may be uneconomical if the lower limit is deeply buried. Extensive disturbance of the stream bed is often strongly opposed by the environmental agencies.

A flexible "Launching apron" is laid horizontally on the bed at the foot of the revetment with a height of about 1.5 times the predicted revetment thickness. The intention is that when scour occurs, the apron will settle and cover the side of the scour hole on a natural slope.

A rock-filled toe trench or toe berm is constructed at the foot of the slope. This is a variant of the launching apron since the rock in the trench launches as scour develops. This method requires encroachment into the creek channel; however, a toe trench can be re-buried beneath native stream bed materials.

A sheet pile cutoff wall is installed from the toe of the revetment down to an in-erodible material or to below the expected scour depth.

2.2.11 Abutment Stems Construction

The following recommendations are made concerning the abutment stems in accordance with the CHBDC:

- Select free-draining granular fill meeting the specifications of OPSS Granular 'A' or Granular 'B' Type II but with less than 5 percent passing the No. 200 sieve should be used as backfill behind the wall. This fill should be compacted in accordance with OPSS 501.
- Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the granular backfill requirements with respect to subdrains and frost tapers should be in accordance with OPSD 3101.150, 3190.100, and 3121.150. The outlets for these subdrains should not be subject to freezing or flooding.
- Care must be taken during the compaction operation not to overstress the wall. Heavy construction equipment should be maintained at a distance of 1.0 meter away from walls where the backfill soils are being placed. Hand-operated compaction equipment should be used to compact backfill soils within a 1.0 meter zone adjacent to the walls. Other surcharge should be accounted for in the design, as required.
- The granular fill may be placed in a zone with width equal to 1.8 m behind the back of the abutment stem (Case (a) on Figure C6.20 of the Commentary to the CHBDC) with a frost taper should be included as per OPSD 3101.150 or within the wedge shaped zone defined by a line drawn at 1.5H:1.0V extending up and back from the rear face of the footing (Case (b) on Figure C6.20 of Commentary to the CHBDC). As an alternative OPSD 3101.150 standard drawing can be used.

2.3 Corrosion Protection

Two soil samples were selected for chemical analyses and were sent via courier, in a secure cooler under chain of custody, to Maxxam Analytics Inc., a CALA-certified and accredited laboratory in Mississauga, Ontario. The analytical laboratory results are summarized in section 1.6 of this report and detailed results are included in Appendix E.

Similar to our experience with the soils in the area, the chemical data indicates low to high resistivity, which indicates a high to low potential for corrosion of buried metallic elements, particularly pipes and appurtenances. This is in agreement with our experience in the area. The maximum chloride content reported is 240 ppm ($\mu\text{g/g}$) i.e. 0.024% which indicates a low potential for additional corrosion.

The maximum water soluble sulphate content of the soils tested is 30 ppm ($\mu\text{g/g}$), i.e. <0.003% and being less than 0.10%, does not indicate the potential to corrode normal Portland cement concrete. These data also support our local experience.

December 12, 2016

3 CLOSURE

The recommendations made in this report are in accordance with our present understanding of the project and are provided solely for the team responsible for the design of the works described herein.

We recommend that we be retained to review our recommendations as the design nears completion to ensure that the final design is in agreement with the assumptions on which our recommendations are based and that our recommendations have been interpreted as intended. If not accorded this review, **exp** will assume no responsibility for the interpretation and use of the recommendations in this report.

A subsurface investigation is a limited sampling of a site; the subsurface conditions have been established only at the test hole locations. Should conditions at the site be encountered which differ from those reported at the test locations, we require that we be notified immediately in order to assess this additional information and our recommendations, as appropriate. It may then be necessary to perform additional investigation and analysis.

Contractors bidding on or undertaking any proposed work at this site should, relative to the subsurface conditions, decide on their own investigations, if deemed necessary, as well as their own interpretations of the factual results provided herein, so 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 Ahileas Mitsopoulos, P.Eng., Nimesh Tamrakar, M.Eng, EIT., Demetri N. Georgiou, MASc. P.Eng., and Silvana Micic, Ph.D., P.Eng. It was reviewed by TaeChul Kim, M.E.Sc., P.Eng. and by Stan E. Gonsalves, M.Eng., P.Eng., Designated MTO Foundation Contact. The field investigation was supervised by Elwin Farkas.

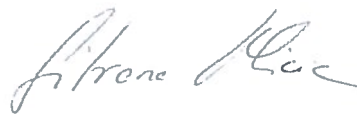
Yours truly,

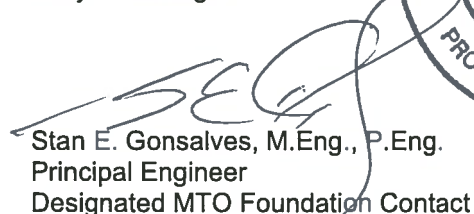
exp Services Inc.


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


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

Encl.


Silvana Micic, PhD., P.Eng.
Senior Geotechnical
Project Manager


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



4 LIMITATIONS AND USE OF REPORT

BASIS OF REPORT

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

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

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

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

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

RELIANCE ON INFORMATION PROVIDED

The evaluation and conclusions contained in the Report are based on conditions in evidence at the time of site inspections and information provided to exp by the Client and others. The Report has been prepared for the specific site, development, building, design or building assessment objectives

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

STANDARD OF CARE

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

COMPLETE REPORT

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

USE OF REPORT

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

REPORT FORMAT

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

Appendix A – Site Photographs



Photo 1. Looking north at downstream side of existing bridge, east side of highway



Photo 2. Looking north at upstream side of existing bridge, west side of highway



Photo 3. Facing north on Highway 584 from existing bridge



Photo 4. Facing south on Highway 584 from existing bridge



Photo 5. Embankment slope on east side facing south



Photo 6. Embankment slope on west side facing south



Photo 7. Existing north abutment slope facing northeast



Photo 8. Existing south abutment slope facing southeast

Appendix B – Drawings

Appendix C – Borehole Logs and Bedrock Core Photos

Explanation of Terms Used on Borehole Records

SOIL DESCRIPTION

Terminology describing common soil genesis:

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

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

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

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

Terminology describing soil structure:

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

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

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

Fissured: material breaks along plane of fracture.

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

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

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

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

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

Homogeneous: same color and appearance throughout.

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

Uniformly Graded: predominantly on grain size.

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

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

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

Table a: Percent or Proportion of Soil, Pp

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

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

Table b: Apparent Density of Cohesionless Soil

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

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

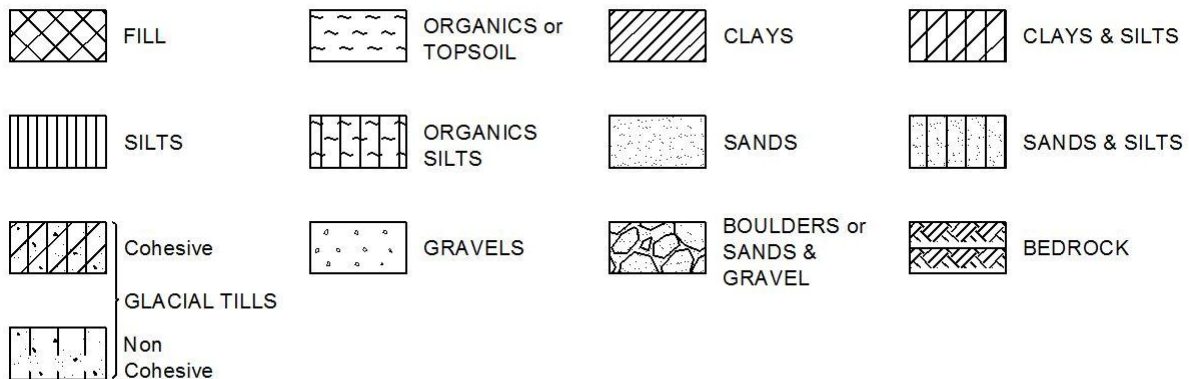
Table c: Consistency of Cohesive Soil

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

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

STRATA PLOT

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



WATER LEVEL MEASUREMENT



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'	—°	Effective angle of internal friction
c_u	kPa	Apparent cohesion intercept
ϕ_u	—°	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

RECORD OF BOREHOLE No BH101

1 OF 1

METRIC

W.P. GWP No. 6054-08-00 LOCATION Murky Creek Bridge (Site No. 48E-004) MTM ON-14 5,544,584N 320,774E ORIGINATED BY RM
DIST 61 HWY 584 BOREHOLE TYPE CME 55 Track Carrier / HSA / NW/NQ COMPILED BY AM
DATUM Geodetic DATE 3.16.16 - 3.16.16 CHECKED BY DG

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 (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE									
322.0	Fill						20	40	60	80	100	20	40	60		GR SA SI CL	
0.0	Well Graded SAND with Gravel (FILL) - loose, brown, moist		S1	SS	8											85 15 (0)	
321.3	Well Graded GRAVEL with Sand (FILL) - loose, brown, wet		S2	SS	5												
0.8																	
320.7	- refusal to auger at 1.4 m depth continued with NW/NQ casing and coring COBBLES & BOULDERS (FILL)		S3	CORE													
1.4	- washboring techniques initiated at about 2.9 m depth																
317.4	Poorly Graded SAND with Silt - loose to compact, grey, wet		S4	SS	10											0 94 (6)	
4.7			S5	SS	18												
314.6	SILT - loose, grey, wet		S6A	SS	3											Recovery=98% RQD=60%	
7.4			S6B	SS	4												
314.0	- refusal to SPT at about 8.0 m depth COBBLES AND BOULDERS			SS	100												
8.1			S7	CORE													
312.3	- refusal to SPT at about 9.6 m depth BEDROCK - medium strong, white, black, red to pink, fractured, medium grained, granite			SS	100											312	
9.7			S8	CORE													
310.9	End of Borehole															311	
11.1	Borehole advanced from top of bridge deck (324.6 m elevation), about 2.6 m above ground surface. Asphalt and concrete thickness were about 90 mm and 250 mm, respectively.																

ONL_MOT_F-16103-AG - ADM-00223648-J0 - MTO 10 & 11 - MURKY CREEK BRIDGE.GPJ ON_MOT_GDT_4/6/16

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

RECORD OF BOREHOLE No BH102

1 OF 2

METRIC

W.P. GWP No. 6054-08-00 LOCATION Murky Creek Bridge (Site No. 48E-004) MTM ON-14 5,544,554N 320,774E ORIGINATED BY RM
 DIST 61 HWY 584 BOREHOLE TYPE CME 55 Track Carrier / HSA / NW/NQ COMPILED BY AM
 DATUM Geodetic DATE 3.17.16 - 3.18.16 CHECKED BY DG

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 (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)				
322.3	Fill						20 40 60 80 100						GR SA SI CL	
0.0	Well Graded SAND with Gravel (FILL) - dense, brown, damp		S1	SS	48									
	- frozen from about 0.9 m depth to about 1.7 m depth		S2	SS	100									
320.6														
1.7	- about 150 mm blow up at about 1.7 m depth		S3	SS	11									
	Poorly Graded SAND with Gravel - compact, brown, wet, occasional cobbles and boulders in upper 2.4 m		S4	SS	15								23 72 (5)	
	- about 300 mm blow up at about 2.4 m depth		S5	SS	19									
	- becoming very loose at about 4.0 m depth		S6A	SS	0									
318.0			S6B	SS	0									
4.3	Clayey SILT - stiff, grey, wet		S7	VANE									0 4 40 56	
316.8														
5.5	- about 600 mm of blow up at about 5.5 m depth		S8	SS	26									
	Silty SAND - compact, grey, wet													
316.0														
6.3	- about 900 mm of blow up at about 6.2 m depth		S9	SS	13								0 36 57 7	
	Sandy SILT - compact, grey, wet													
	- about 1,200 mm of blow up at about 7.0 m depth		S10A	SS	4									
			S10B	SS	6									
	- becoming very dense at about 7.8 m depth													
314.1			S11	SS	100									
8.2	COBBLES AND BOULDERS													
			S12	CORE										
			S13	CORE										
			S14	CORE										
309.8														
12.5	BEDROCK - medium strong, white, black, red to pink, fractured to very sound, medium grained, granite		S15	CORE										Recovery=100% RQD=66%
														Recovery=100% RQD=100%

Continued Next Page

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

METRIC



RECORD OF BOREHOLE No BH103

1 OF 1

METRIC

W.P. GWP No. 6054-08-00 LOCATION Murky Creek Bridge (Site No. 48E-004) MTM ON-14 5,544,554N 320,787E ORIGINATED BY RM
 DIST 61 HWY 584 BOREHOLE TYPE CME 55 Track Carrier / NW/NQ COMPILED BY AM
 DATUM Geodetic DATE 3.18.16 - 3.19.16 CHECKED BY DG

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 (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				
								20 40 60 80 100					W _p W W _L				
						○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE											
322.3	Fill																
0.0	Well Graded SAND with Gravel (FILL) - frozen, brown		S1	SS	100		322										
321.4	Well Graded GRAVEL with Sand (FILL) - frozen, brown		S2	SS	100		321										
320.6	Well Graded SAND with Gravel (FILL) - loose, brown, wet		S3	SS	9		320										
319.9	Poorly Graded SAND with Silt - very loose to loose, brown, wet		S4	SS	0		319										
318.8	- becoming Poorly Graded SAND with Gravel at about 3.2 m depth		S5A	SS	8												
3.5	Clayey SILT - soft to very soft, grey, wet		S5B	SS	3												
			S6	SS	0		318										
317.4	Poorly Graded SAND with Silt - compact, grey, wet		S7	VANE													
4.9			S8	SS	20		317										
			S9	SS	13		316										
	- becoming loose at about 7.0 m depth		S10	SS	7	315											
314.5																	
7.8	SILT - compact to very dense, grey, wet		S11	SS	16	314											
			S12	SS	100												
313.5	COBBLES & BOULDERS		S13	CORE		313											
8.8			SS		100	312											
	- refusal to SPT at about 10.3 m depth		S14	CORE		311											
310.4	End of Borehole - refusal to SPT		SS		100												
11.9	Borehole advanced from top of bridge deck (324.4 m elevation), about 2.1 m above ground surface. Asphalt and concrete thickness were about 90 mm and 250 mm, respectively.																

ONL MOT F-16103-AG - ADM-00223648-J0 - MTO 10 & 11 - MURKY CREEK BRIDGE GPJ ON_MOT_GDT 4/6/16

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

RECORD OF BOREHOLE No BH104

1 OF 1

METRIC

W.P. GWP No. 6054-08-00 LOCATION Murky Creek Bridge (Site No. 48E-004) MTM ON-14 5,544,592N 320,773E ORIGINATED BY RM
DIST 61 HWY 584 BOREHOLE TYPE CME 55 Track Carrier / HSA / NW/NQ COMPILED BY AM
DATUM Geodetic DATE 3.14.16 - 3.16.16 CHECKED BY DG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)					
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					W _p W W _L			GR	SA	SI	CL	
								○ UNCONFINED + FIELD VANE					WATER CONTENT (%)							
								● QUICK TRIAXIAL × LAB VANE												
322.2	Fill						20	40	60	80	100	20	40	60						
0.0	Well Graded SAND with Gravel (FILL) - compact, brown, moist - frozen from about 0.6 m depth to about 1.6 m depth - refusal to SPT and auger (HSA) at about 1.3 m depth continued with NW/NQ casing and coring COBBLES & BOULDERS (FILL) - washboring techniques initiated at about 3.1 m depth		S1	SS	17		322													
			S2	SS	100		321													
					SS		100	320												
320.6									319											
1.6			S3	CORE			318													
							317											No Recovery		
							316									3	89	(8)		
							315											No Recovery		
			S6	SS	18		314													
314.0							313													
8.2	COBBLES & BOULDERS		S7	CORE			312													
							311											Recovery=100% RQD=35%		
								310										Recovery=95% RQD=80%		
			S8	CORE			309													
311.7																				
10.5	BEDROCK - medium strong, white, black, red to pink, severely fractured to sound, medium grained, granite		S9	CORE																
308.8																				
13.4	End of Borehole																			
	Borehole advanced from top of bridge deck (324.6 m elevation), about 2.4 m above ground surface. Asphalt and concrete thickness were about 90 mm and 250 mm, respectively.																			

ONL MOT F-16103-AG - ADM-00223648-J0 - MTO 10 & 11 - MURKY CREEK BRIDGE.GPJ ON MOT.GDT 4/6/16

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

RECORD OF BOREHOLE No BH105

1 OF 1

METRIC

W.P. GWP No. 6054-08-00 LOCATION Murky Creek Bridge (Site No. 48E-004) MTM ON-14 5,544,607N 320,767E ORIGINATED BY RM
DIST 61 HWY 584 BOREHOLE TYPE CME 55 Track Carrier / SSA / HSA COMPILED BY AM
DATUM Geodetic DATE 3.14.16 - 3.14.16 CHECKED BY DG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE									
324.3	Asphalt						20	40	60	80	100						
320.0	ASPHALT - about 200 mm		S1	SS	100												No Recovery. Sample collected from augers.
0.2	Well Graded SAND with Gravel (FILL) - frozen, brown		S2	SS	100												No Recovery. Sample collected from augers.
	- becoming dense, moist at about 1.5 m depth		S3	SS	38												
	- becoming compact at about 2.3 m depth		S4	SS	18												
	- becoming wet at about 3.0 m depth		S5	SS	13												
320.5																	
3.8	Silty GRAVEL with Sand - loose, grey, wet		S6	SS	8												43 38 (19)
319.4			S7A	SS	6												
4.9	SILT with Sand - loose, grey, wet		S7B	SS	9												
318.7			S8A	SS	6												0 28 63 9
5.6	Clayey SILT - firm to soft, grey, wet		S8B	SS	5												
			S9	VANE													
			S10	SS	0												0 3 66 31
			S11	VANE													
	- refusal to auger at about 7.3 m depth		S12	SS	100												
316.4																	
7.9	End of Borehole - refusal to SPT																

ONL_MOT_F-16103-AG - ADM-00223648-J0 - MTO 10 & 11 - MURKY CREEK BRIDGE.GPJ ON_MOT_GDT 4/6/16

RECORD OF BOREHOLE No BH106

1 OF 1

METRIC

W.P. GWP No. 6054-08-00 LOCATION Murky Creek Bridge (Site No. 48E-004) MTM ON-14 5,544,540N 320,798E ORIGINATED BY RM
DIST 61 HWY 584 BOREHOLE TYPE CME 55 Track Carrier / SSA / HSA COMPILED BY AM
DATUM Geodetic DATE 3.14.16 - 3.14.16 CHECKED BY DG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)								
324.7	Asphalt							20	40	60	80	100						
324.9	ASPHALT - about 230 mm		S1	SS	100													
0.2	Well Graded SAND with Gravel (FILL) - frozen, brown		S2	SS	100													
	- becoming compact, moist at about 1.5 m depth		S3	SS	12													
	- becoming moist to wet at about 2.3 m depth		S4	SS	12													
321.4			S5A	SS	4													
3.4	PEAT - soft, dark brown, wet		S5B	SS	2													
320.9	SILT - compact to loose, grey, wet		S6	SS	20													
3.8			S7	VANE														
			S8	SS	8													
			S9	VANE														
318.3			S10A	SS	0													
318.4	SILT with Sand - loose to very dense, grey, wet		S10B	SS	100													
6.6	End of Borehole - refusal to auger and SPT																	

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

RECORD OF BOREHOLE No BH107

1 OF 1

METRIC

W.P. GWP No. 6054-08-00 LOCATION Murky Creek Bridge (Site No. 48E-004) MTM ON-14 5,544,582N 320,778E ORIGINATED BY RM
 DIST 61 HWY 584 BOREHOLE TYPE CME 55 Track Carrier / NW/NQ COMPILED BY AM
 DATUM Geodetic DATE 3.17.16 - 3.17.16 CHECKED BY DG

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 (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa			WATER CONTENT (%)				
317.5	Sand		S1A	SS	3	317								0 99 (1) Casing continuously moved during SPT	
0.0 317.2	Poorly Graded SAND - very loose, brown to grey, wet		S1B	SS	2										
0.3	Poorly Graded GRAVEL with Sand - very loose, brown, wet														
316.0						316									
1.5	Poorly Graded SAND - loose to compact, brown, wet, medium grained		S2	SS	-										
314.7						315									
2.8	Silty SAND - loose to compact, brown, wet		S3	SS	10										
313.4						314									
4.1	SILT - compact, grey, wet		S4	SS	18	313									0 0 91 9
312.5			S5	VANE											
5.0	BEDROCK - medium strong, white, black, red to pink, severely fractured to fractured, medium grained, granite		S6	CORE		312									Recovery=97% RQD=60%
			S7	CORE		311									
309.4						310									
8.1	End of Borehole														
	Borehole advanced from top of bridge deck (324.6 m elevation), about 3.4 m above top of ice/water and about 7.1 above ground surface. Asphalt and concrete thickness were about 90 mm and 250 mm, respectively.														

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

RECORD OF BOREHOLE No BH108

1 OF 1

METRIC

W.P. GWP No. 6054-08-00 LOCATION Murky Creek Bridge (Site No. 48E-004) MTM ON-14 5,544,561N 320,784E ORIGINATED BY RM
 DIST 61 HWY 584 BOREHOLE TYPE CME 55 Track Carrier / NW/NQ COMPILED BY AM
 DATUM Geodetic DATE 3.19.16 - 3.20.16 CHECKED BY DG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
								○ UNCONFINED	+	FIELD VANE							
318.3	Sand						● QUICK TRIAXIAL	×	LAB VANE	WATER CONTENT (%)							
0.0	Poorly Graded SAND with Gravel - very loose, brown, wet		S1	SS	3		20	40	60	80	100	20	40	60			
			S2	SS	1												
316.8																	
1.5	Silty SAND - very loose to loose, brown, wet		S3	SS	1							○					
			S4	SS	0								○				
315.0	- becoming grey at about 3.0 m depth		S5A	SS	5								○			0 69 18 13	
3.4	SILT - loose to compact, grey, wet		S5B	SS	4								○				
			S6	SS	16								H○			0 0 90 10	
313.3	- becoming very dense at about 4.6 m depth		S7	SS	100								○				
5.0	COBBLES & BOULDERS - some gravel, some sand																
			S8	CORE													
			S9	CORE													
311.0																	
7.3	BEDROCK - medium strong, white, black, red to pink, severely fractured to sound, medium grained, granite		S10	CORE												Recovery=92% RQD=42%	
			S11	CORE												Recovery=80% RQD=73%	
			S12	CORE												Recovery=98 % RQD=80%	
307.7																	
10.7	End of Borehole Borehole advanced from top of bridge deck (324.4 m elevation), about 3.4 m above top of ice/water and about 6.1 above ground surface. Asphalt and concrete thickness were about 90 mm and 250 mm, respectively.																

ONL_MOT_F-16103-AG - ADM-00223648-J0 - MTO 10 & 11 - MURKY CREEK BRIDGE.GPJ ON_MOT_GDT 4/6/16



BH101 - Bedrock Core Samples with Depths and Elevations



BH102 - Bedrock Core Samples with Depths and Elevations



BH104 - Bedrock Core Samples with Depths and Elevations

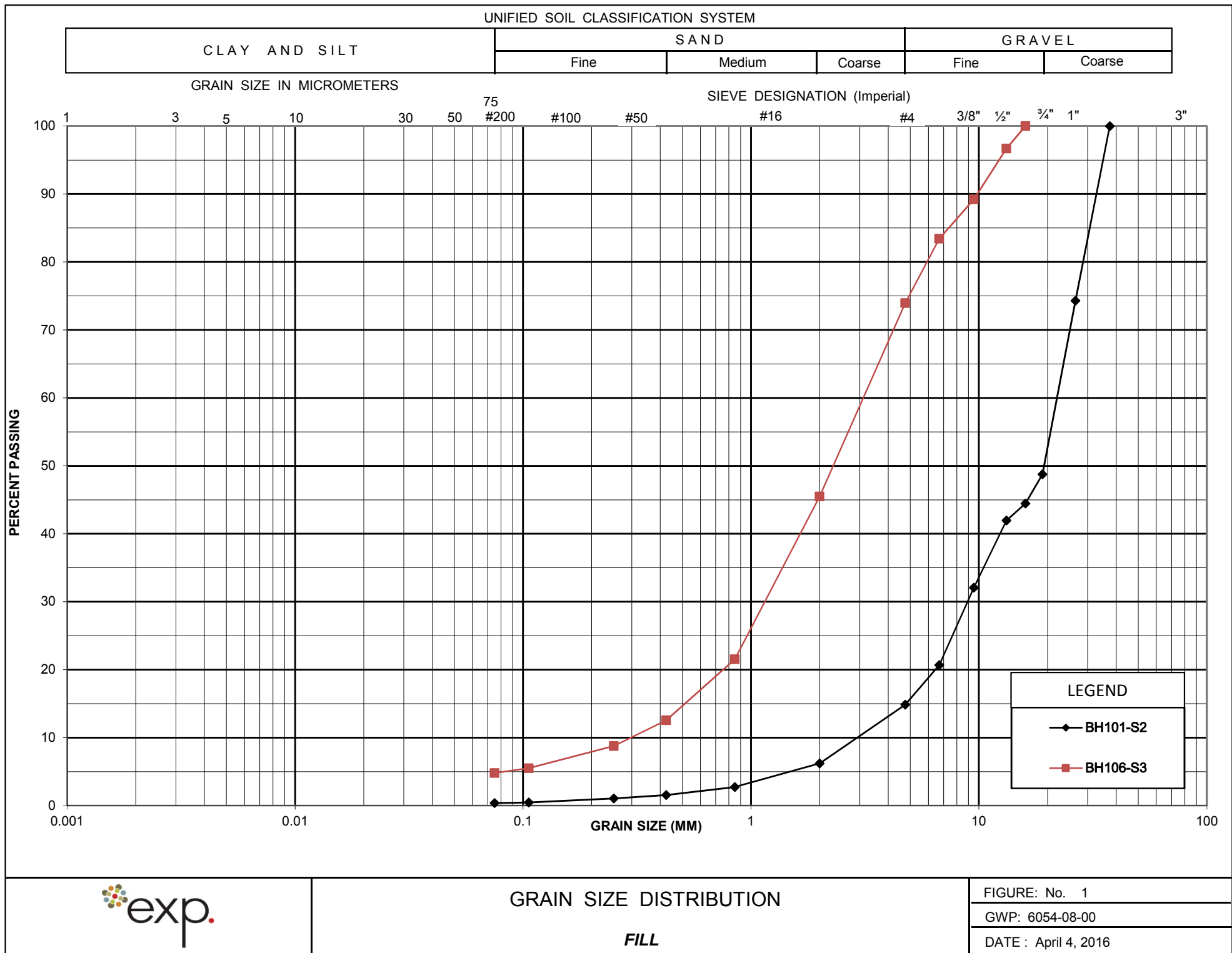


BH107 - Bedrock Core Samples with Depths and Elevations



BH108 - Bedrock Core Samples with Depths and Elevations

Appendix D – Laboratory Data



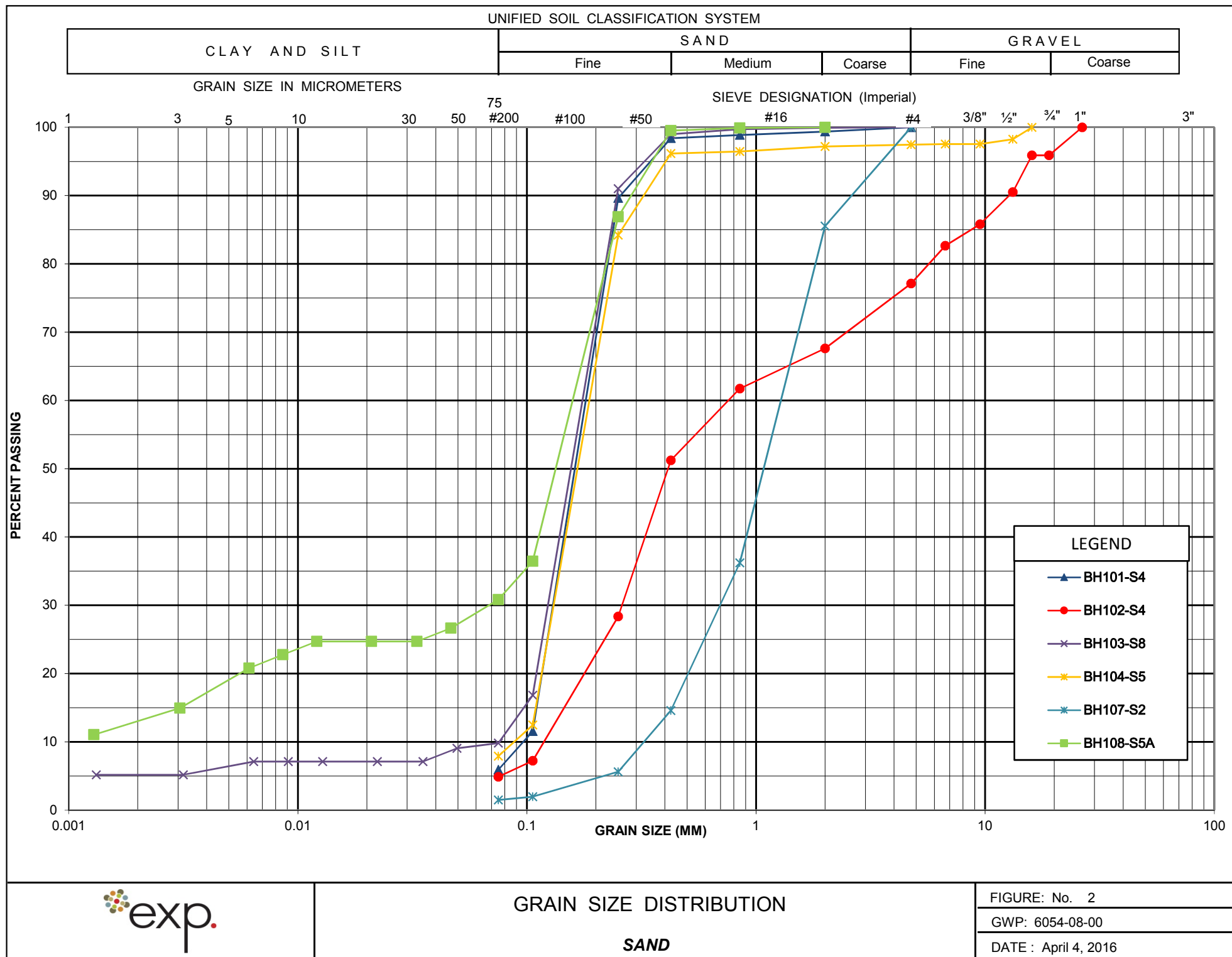
GRAIN SIZE DISTRIBUTION

FILL

FIGURE: No. 1

GWP: 6054-08-00

DATE : April 4, 2016

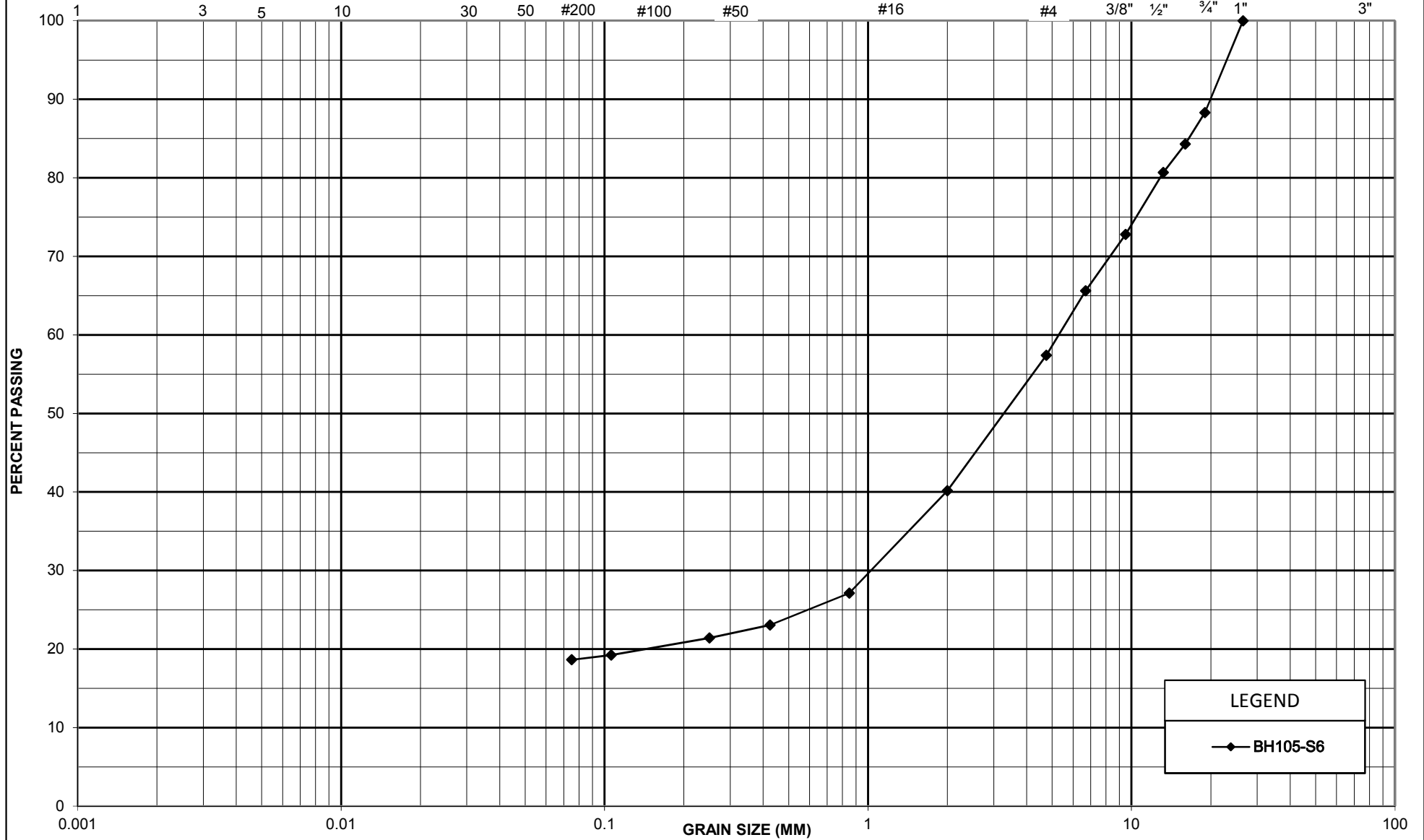


UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY AND SILT	SAND			GRAVEL	
	Fine	Medium	Coarse	Fine	Coarse

GRAIN SIZE IN MICROMETERS

SIEVE DESIGNATION (Imperial)



LEGEND
—◆— BH105-S6



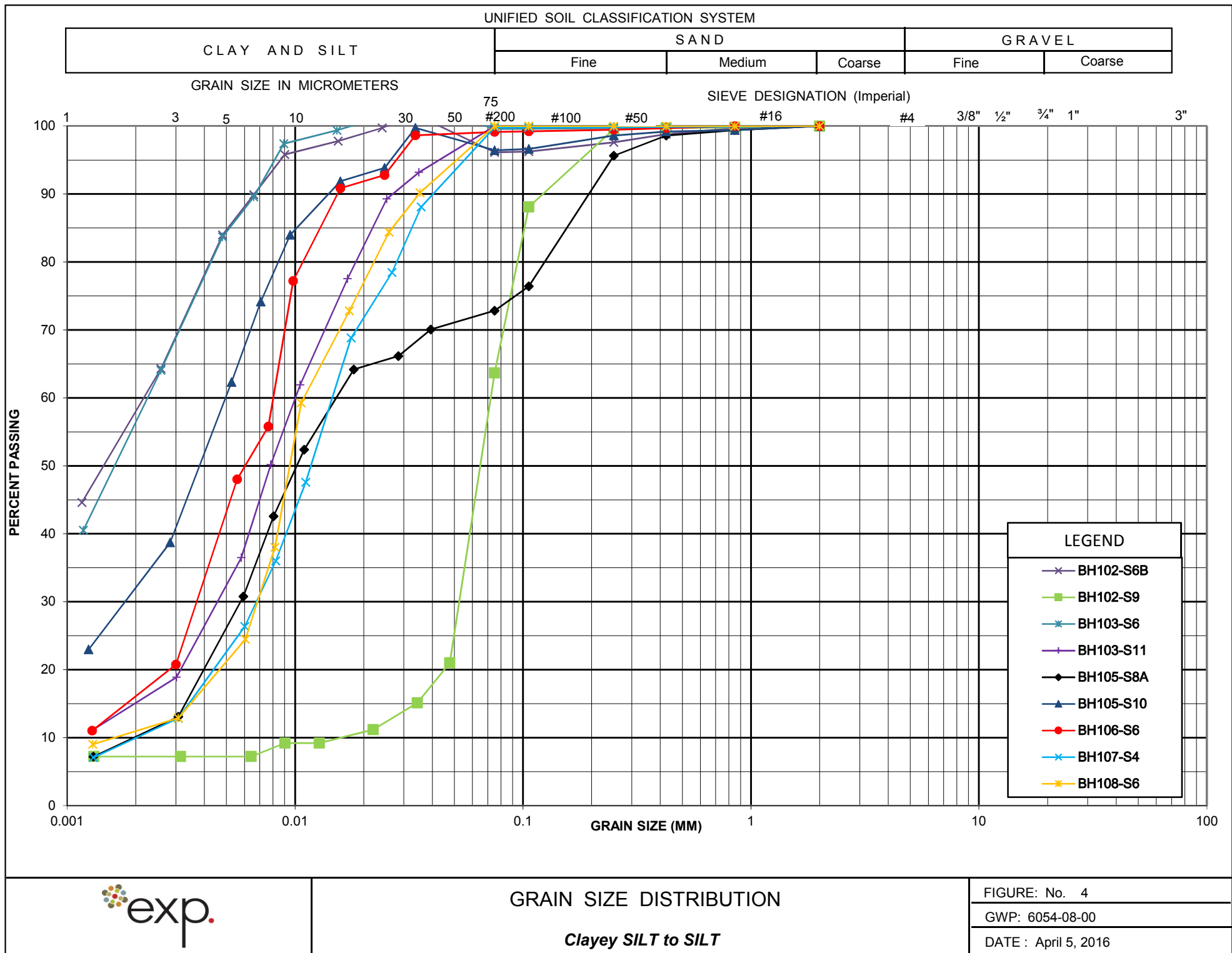
GRAIN SIZE DISTRIBUTION

Silty GRAVEL with Sand

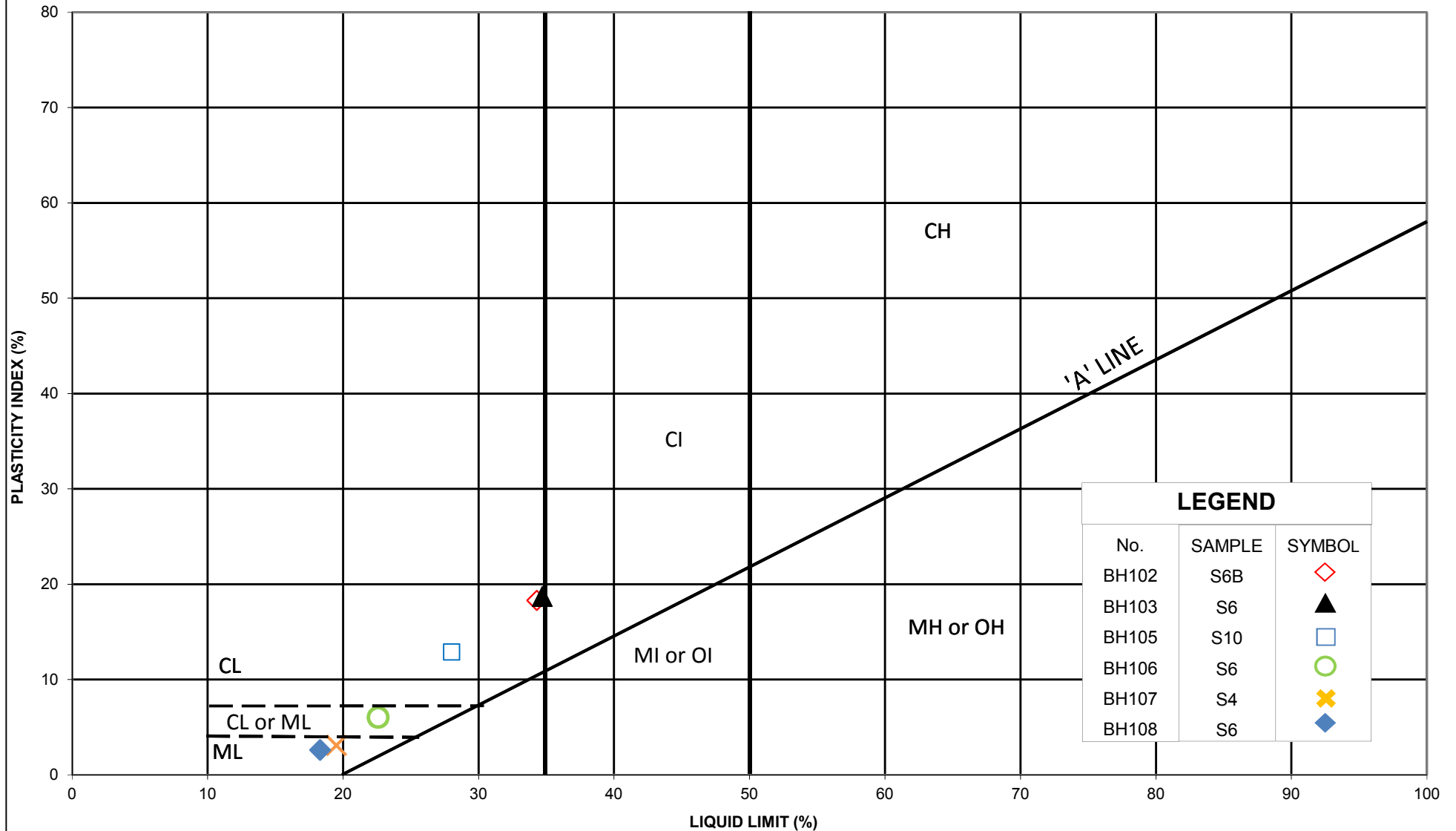
FIGURE: No. 3

GWP: 6054-08-00

DATE : April 5, 2016



Murky Creek Bridge Replacement (Site No. 48E-004)
GWP No. 6054-08-00, Highway 584, District of Thunder Bay, Ontario



Appendix E – Chemical Analyses

Your Project #: ADM-00223648-J0

Site Location: HWY 584

Your C.O.C. #: 81617

Attention: Ahileas Mitsopoulos/Michael S

exp Services Inc
Thunder Bay Branch
1142 Roland St
Thunder Bay, ON
P7B 5M4

Report Date: 2016/04/01

Report #: R3948724

Version: 1 - Final

CERTIFICATE OF ANALYSIS

MAXXAM JOB #: B660807

Received: 2016/03/29, 09:02

Sample Matrix: Soil
Samples Received: 2

Analyses	Date		Date Analyzed	Laboratory Method	Reference
	Quantity	Extracted			
Chloride (20:1 extract)	2	N/A	2016/04/01	CAM SOP-00463	EPA 325.2 m
Conductivity	2	N/A	2016/04/01	CAM SOP-00414	OMOE E3138 v2 m
pH CaCl2 EXTRACT	2	2016/03/31	2016/03/31	CAM SOP-00413	EPA 9045 D m
Resistivity of Soil	2	2016/03/29	2016/04/01	CAM SOP-00414	SM 22 2510 m
Sulphate (20:1 Extract)	2	N/A	2016/04/01	CAM SOP-00464	EPA 375.4 m

Remarks:

Maxxam Analytics has performed all analytical testing herein in accordance with ISO 17025 and the Protocol for Analytical Methods Used in the Assessment of Properties under Part XV.1 of the Environmental Protection Act. All methodologies comply with this document and are validated for use in the laboratory. The methods and techniques employed in this analysis conform to the performance criteria (detection limits, accuracy and precision) as outlined in the Protocol for Analytical Methods Used in the Assessment of Properties under Part XV.1 of the Environmental Protection Act.

Maxxam Analytics is accredited for all specific parameters as required by Ontario Regulation 153/04. Maxxam Analytics is limited in liability to the actual cost of analysis unless otherwise agreed in writing. There is no other warranty expressed or implied. Samples will be retained at Maxxam Analytics for three weeks from receipt of data or as per contract.

Reference Method suffix "m" indicates test methods incorporate validated modifications from specific reference methods to improve performance.

* RPDs calculated using raw data. The rounding of final results may result in the apparent difference.

Encryption Key

Please direct all questions regarding this Certificate of Analysis to your Project Manager.

Hina Siddiqui, Project Manager –Environmental Customer Service

Email: HSiddiqui@maxxam.ca

Phone# (905) 817-5700

=====

Maxxam has procedures in place to guard against improper use of the electronic signature and have the required "signatories", as per section 5.10.2 of ISO/IEC 17025:2005(E), signing the reports. For Service Group specific validation please refer to the Validation Signature Page.

Maxxam Job #: B660807
Report Date: 2016/04/01

exp Services Inc
Client Project #: ADM-00223648-J0
Site Location: HWY 584
Sampler Initials: RM

RESULTS OF ANALYSES OF SOIL

Maxxam ID		CCA483	CCA483	CCA484		
Sampling Date		2016/03/16 12:00	2016/03/16 12:00	2016/03/17 10:00		
COC Number		81617	81617	81617		
	UNITS	BH101-S5	BH101-S5 Lab-Dup	BH102-S6B	RDL	QC Batch
Calculated Parameters						
Resistivity	ohm-cm	11000		1700		4434942
Inorganics						
Soluble (20:1) Chloride (Cl)	ug/g	<20		240	20	4439872
Conductivity	umho/cm	89	89	575	2	4440067
Available (CaCl2) pH	pH	8.05		7.63		4437390
Soluble (20:1) Sulphate (SO4)	ug/g	30	22	<20	20	4439873
RDL = Reportable Detection Limit						
QC Batch = Quality Control Batch						
Lab-Dup = Laboratory Initiated Duplicate						

Maxxam Job #: B660807
Report Date: 2016/04/01

exp Services Inc
Client Project #: ADM-00223648-J0
Site Location: HWY 584
Sampler Initials: RM

TEST SUMMARY

Maxxam ID: CCA483
Sample ID: BH101-S5
Matrix: Soil

Collected: 2016/03/16
Shipped:
Received: 2016/03/29

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	4439872	N/A	2016/04/01	Deonarine Ramnarine
Conductivity	AT	4440067	N/A	2016/04/01	Lemeneh Addis
pH CaCl2 EXTRACT	AT	4437390	2016/03/31	2016/03/31	Neil Dassanayake
Resistivity of Soil		4434942	2016/04/01	2016/04/01	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	4439873	N/A	2016/04/01	Deonarine Ramnarine

Maxxam ID: CCA483 Dup
Sample ID: BH101-S5
Matrix: Soil

Collected: 2016/03/16
Shipped:
Received: 2016/03/29

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Conductivity	AT	4440067	N/A	2016/04/01	Lemeneh Addis
Sulphate (20:1 Extract)	KONE/EC	4439873	N/A	2016/04/01	Deonarine Ramnarine

Maxxam ID: CCA484
Sample ID: BH102-S6B
Matrix: Soil

Collected: 2016/03/17
Shipped:
Received: 2016/03/29

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	4439872	N/A	2016/04/01	Deonarine Ramnarine
Conductivity	AT	4440067	N/A	2016/04/01	Lemeneh Addis
pH CaCl2 EXTRACT	AT	4437390	2016/03/31	2016/03/31	Neil Dassanayake
Resistivity of Soil		4434942	2016/04/01	2016/04/01	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	4439873	N/A	2016/04/01	Deonarine Ramnarine

Maxxam Job #: B660807
Report Date: 2016/04/01

exp Services Inc
Client Project #: ADM-00223648-J0
Site Location: HWY 584
Sampler Initials: RM

GENERAL COMMENTS

Each temperature is the average of up to three cooler temperatures taken at receipt

Package 1	2.7°C
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Results relate only to the items tested.

QUALITY ASSURANCE REPORT

exp Services Inc
Client Project #: ADM-00223648-J0
Site Location: HWY 584
Sampler Initials: RM

QC Batch	Parameter	Date	Matrix Spike		SPIKED BLANK		Method Blank		RPD	
			% Recovery	QC Limits	% Recovery	QC Limits	Value	UNITS	Value (%)	QC Limits
4437390	Available (CaCl ₂) pH	2016/03/31			98	97 - 103			0.36	N/A
4439872	Soluble (20:1) Chloride (Cl)	2016/04/01	NC	70 - 130	108	70 - 130	<20	ug/g	1.1	35
4439873	Soluble (20:1) Sulphate (SO ₄)	2016/04/01	NC	70 - 130	104	70 - 130	<20	ug/g	NC	35
4440067	Conductivity	2016/04/01			99	90 - 110	<2	umho/cm	0.56	10

N/A = Not Applicable

Duplicate: Paired analysis of a separate portion of the same sample. Used to evaluate the variance in the measurement.

Matrix Spike: A sample to which a known amount of the analyte of interest has been added. Used to evaluate sample matrix interference.

Spiked Blank: A blank matrix sample to which a known amount of the analyte, usually from a second source, has been added. Used to evaluate method accuracy.

Method Blank: A blank matrix containing all reagents used in the analytical procedure. Used to identify laboratory contamination.

NC (Matrix Spike): The recovery in the matrix spike was not calculated. The relative difference between the concentration in the parent sample and the spiked amount was too small to permit a reliable recovery calculation (matrix spike concentration was less than 2x that of the native sample concentration).

NC (Duplicate RPD): The duplicate RPD was not calculated. The concentration in the sample and/or duplicate was too low to permit a reliable RPD calculation (one or both samples < 5x RDL).

VALIDATION SIGNATURE PAGE

The analytical data and all QC contained in this report were reviewed and validated by the following individual(s).

Cristina Carriere

Cristina Carriere, Scientific Services

Maxxam has procedures in place to guard against improper use of the electronic signature and have the required "signatories", as per section 5.10.2 of ISO/IEC 17025:2005(E), signing the reports. For Service Group specific validation please refer to the Validation Signature Page.



6740 Campobello Road, Mississauga, ON L5N 2L8
Phone: 905-817-5700 Fax: 905-817-5779 Toll Free: (800) 563-6266

CHAIN OF CUSTODY RECORD

81617

Page 1 of 1

INVOICE INFORMATION		REPORT INFORMATION (if differs from invoice)		PROJECT INFORMATION		MAXXAM JOB NUMBER	
Company Name:	exp Services Inc	Company Name:		Quotation #:		CHAIN OF CUSTODY # 00	
Contact Name:	Robert Moen, Anilous Mitgale	Contact Name:		P.O. #:			
Address:	1142 Roland Street Thunder Bay ON P7B 5M4	Address:		Project #:	ADM-08223648-JO		
Phone:	807 623 9445	Phone:		Site Location:	Hwy 584		
Fax:	807 623 8070	Fax:		Site #:			
Email:	robert.moen@exp.com	Email:		Sampled By:	Robert Moen		

Note: For MOE Regulated Drinking Water samples, please use the Drinking Water CofC.

Regulation 153 (2011)				Other Regulations				ANALYSIS REQUESTED (Please be specific)												TURNAROUND TIME (TAT) REQUIRED									
Table 1	Res/Park	Med/Env	CCME	Table 2	Ind/Comm	Coarse	Reg. 558	Table 3	Agri/Other	For RSC	MISA	Municipality:	Table 4	Yes	PWQO	Other (specify):	PLEASE PROVIDE ADVANCE NOTICE FOR RUSH PROJECTS.												
																	Regular (Standard) TAT: <input checked="" type="checkbox"/> (5-7 working days for most tests)												
																	Rush TAT: ***Samples must be received by 3pm to guarantee your TAT***												
																	Rush Confirmation #: PN <input type="checkbox"/> 1 day <input type="checkbox"/> 2 days <input type="checkbox"/> 3 days												
																	Date Req'd: _____												
Include Criteria on Certificate of Analysis (Y/N)?																		TATs for certain tests are > 5 days. Please contact your Project Manager for details.											
SAMPLES MUST BE KEPT COOL (<10°C) FROM TIME OF SAMPLING UNTIL DELIVERY TO MAXXAM.																		# of Cont. COMMENTS / TAT COMMENTS											
Sample Identification																													
Date Sampled																													
Time Sampled																													
Matrix (GW, SW, Soil, etc.)																													
MOE Regulated Drinking Water? (Y/N)																													
Metals Field Filtered? (Y/N)																													
pH																													
Water Soluble Sulphate																													
Resistivity																													
Conductivity																													
Chloride																													
1 BH101 - SS																		2											
2 BH102 - SGB																		2											
3																													
4																													
5																													
6																													
7																													
8																													
9																													
10																													
*RELINQUISHED BY (Signature/Print)																		#JARS USED AND NOT SUBMITTED											
Date (YYYY/MM/DD)																		Laboratory Use Only											
Time:																		Custody Seal Yes No											
RECEIVED BY: (Signature/Print)																		Present Intact											
Date (YYYY/MM/DD)																		Temperature (°C) on Receipt											
Time:																		1/4/13											

*MANDATORY SECTIONS IN GREY MUST BE FILLED OUT. AN INCOMPLETE CHAIN OF CUSTODY MAY RESULT IN ANALYTICAL TAT DELAYS.

COC-1004 (06/12) - ENV. ENG

Maxxam Analytics International Corporation o/a Maxxam Analytics

White: Maxxam

Yellow: Mail

Pink: Client

Appendix F – Results of Slope Stability Analyses

Murky Creek Bridge Replacement
South Approach Abutment
Drained Static Condition

Name: Concrete Model: Mohr-Coulomb Unit Weight: 24 kN/m³ Cohesion: 100 kPa Phi: 45 °
 Name: Engineered Fill Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 32 °
 Name: Sand (Loose to Compact) Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 32 °
 Name: Cobbles and Boulder Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 36 °
 Name: Silty Sand (Loose to Compact) Model: Mohr-Coulomb Unit Weight: 19 kN/m³ Cohesion: 0 kPa Phi: 30 °
 Name: Gravel (Very Loose to Loose) Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 32 °
 Name: Silt (Compact to Very Dense) Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 30 °
 Name: Sand and Gravel Fill Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 32 °
 Name: Clayey Silt (Soft to Stiff) Model: Mohr-Coulomb Unit Weight: 19 kN/m³ Cohesion: 0 kPa Phi: 27 °
 Name: Bedrock Model: Bedrock (Impenetrable)
 Name: Cobbles and Boulder Fill Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 36 °

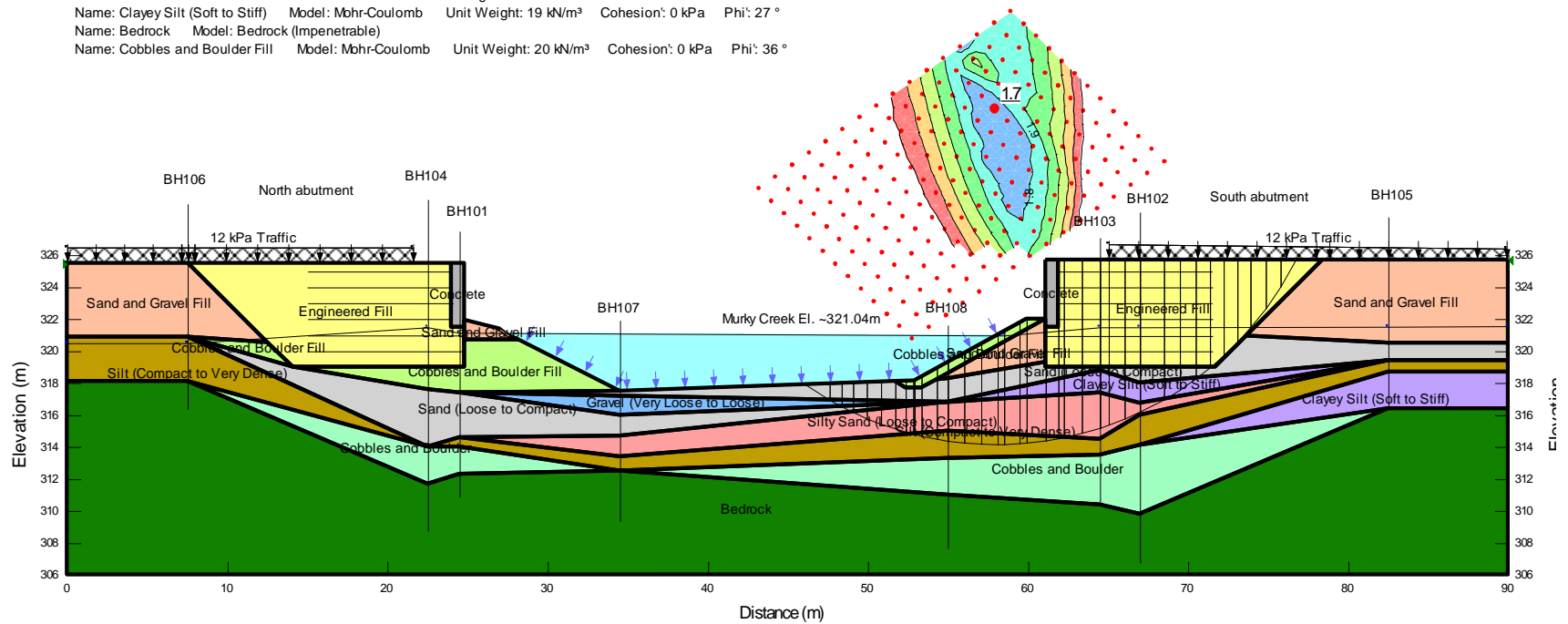


Figure 1: South approach abutment - drained static condition

Murky Creek Bridge Replacement
South Approach Abutment
Drained Seismic Condition

Name: Concrete Model: Mohr-Coulomb Unit Weight: 24 kN/m³ Cohesion: 100 kPa Phi: 45 °
 Name: Engineered Fill Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 32 °
 Name: Sand (Loose to Compact) Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 32 °
 Name: Cobbles and Boulder Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 36 °
 Name: Silty Sand (Loose to Compact) Model: Mohr-Coulomb Unit Weight: 19 kN/m³ Cohesion: 0 kPa Phi: 30 °
 Name: Gravel (Very Loose to Loose) Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 32 °
 Name: Silt (Compact to Very Dense) Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 30 °
 Name: Sand and Gravel Fill Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 32 °
 Name: Clayey Silt (Soft to Stiff) Model: Mohr-Coulomb Unit Weight: 19 kN/m³ Cohesion: 0 kPa Phi: 27 °
 Name: Bedrock Model: Bedrock (Impenetrable)
 Name: Cobbles and Boulder Fill Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 36 °

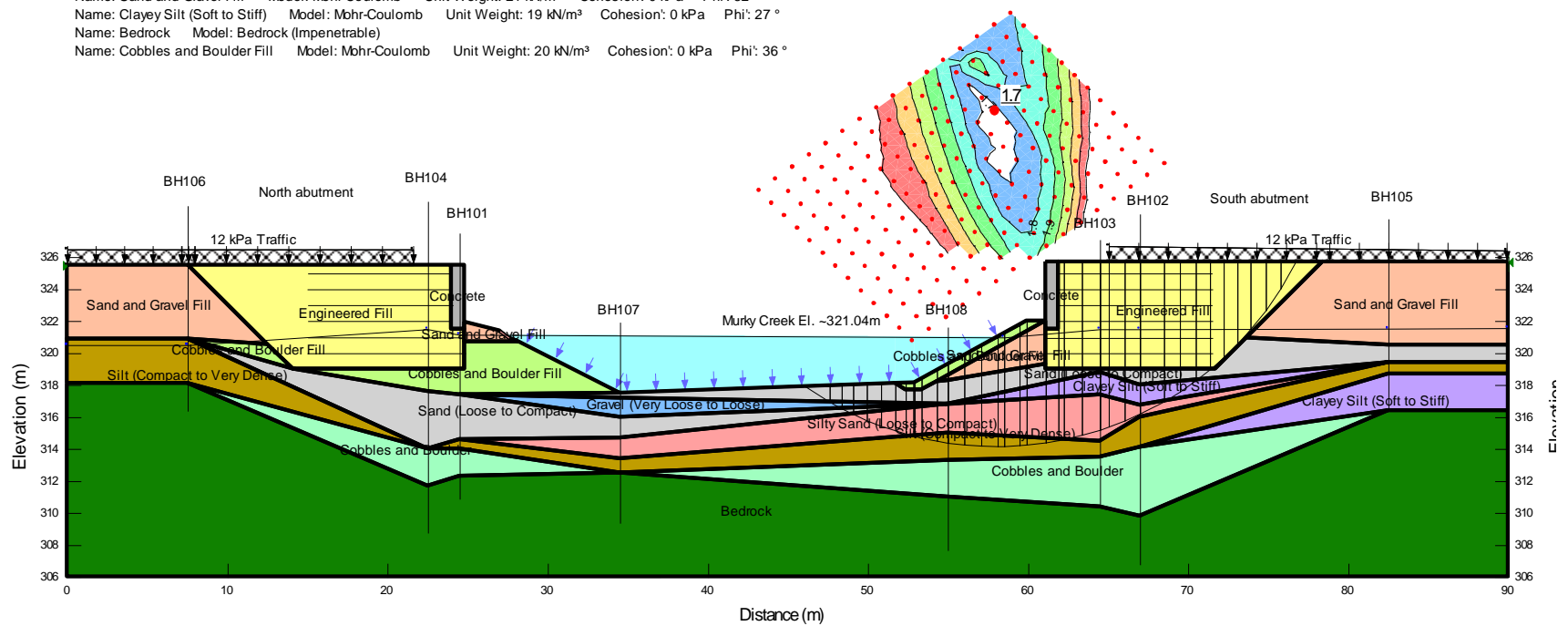


Figure 2: South approach abutment - drained seismic condition

Murky Creek Bridge Replacement RSS Wall behind South Abutment Drained Static Condition

Name: RSS Wall Model: Mohr-Coulomb Unit Weight: 24 kN/m³ Cohesion': 100 kPa Phi': 45 °
 Name: Engineered Fill Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion': 0 kPa Phi': 32 °
 Name: Sand (Loose to Compact) Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion': 0 kPa Phi': 32 °
 Name: Cobbles and Boulder Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion': 0 kPa Phi': 36 °
 Name: Silty Sand (Loose to Compact) Model: Mohr-Coulomb Unit Weight: 19 kN/m³ Cohesion': 0 kPa Phi': 30 °
 Name: Silt (Compact to Very Dense) Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion': 0 kPa Phi': 30 °
 Name: Clayey Silt (Soft to Stiff) Model: Mohr-Coulomb Unit Weight: 19 kN/m³ Cohesion': 0 kPa Phi': 27 °
 Name: Bedrock Model: Bedrock (Impenetrable)

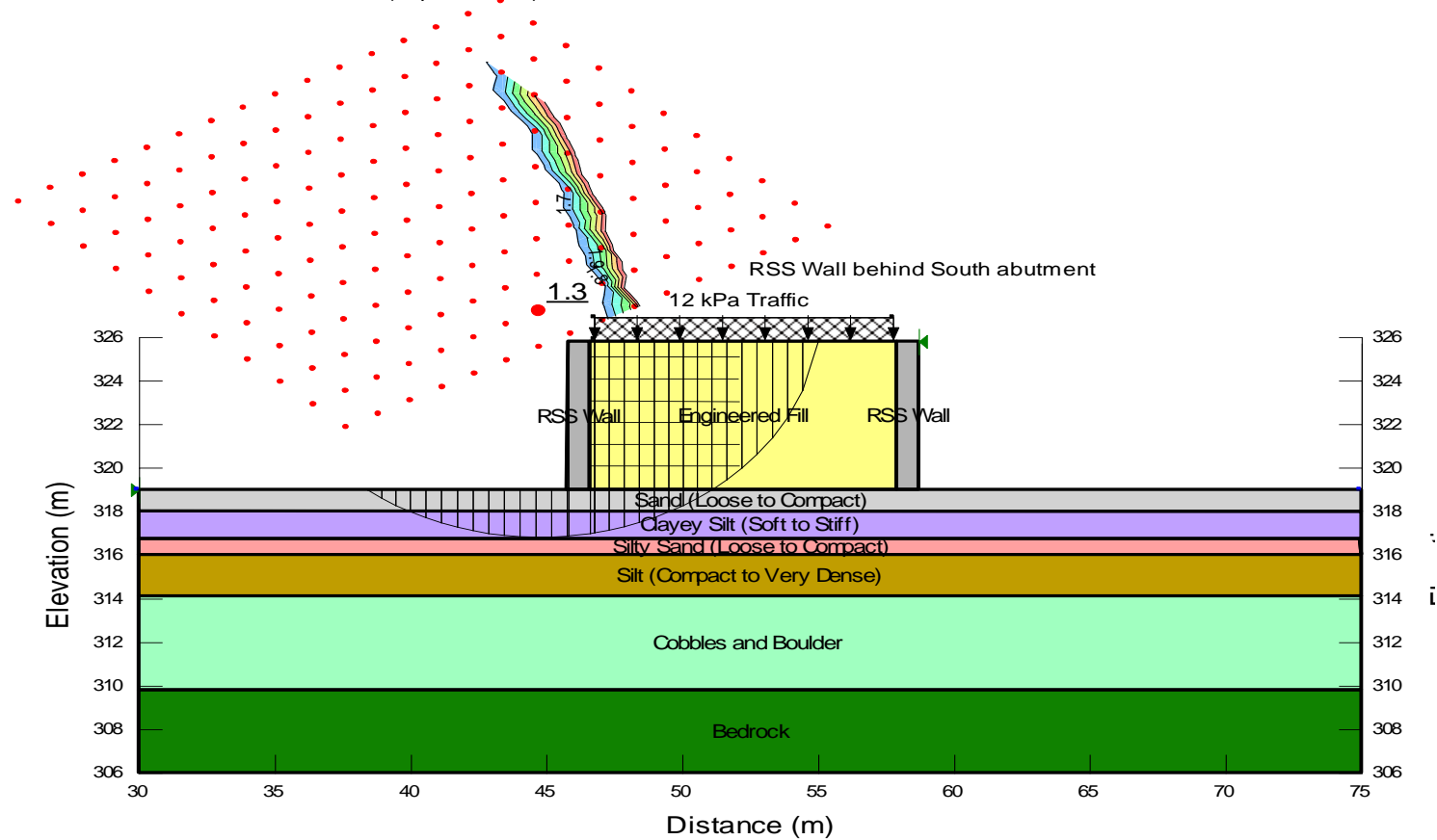


Figure 3: RSS wall behind South abutment - drained static condition

Murky Creek Bridge Replacement RSS Wall behind South Abutment Drained Seismic Condition

Name: RSS Wall Model: Mohr-Coulomb Unit Weight: 24 kN/m³ Cohesion': 100 kPa Phi': 45 °
 Name: Engineered Fill Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion': 0 kPa Phi': 32 °
 Name: Sand (Loose to Compact) Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion': 0 kPa Phi': 32 °
 Name: Cobbles and Boulder Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion': 0 kPa Phi': 36 °
 Name: Silty Sand (Loose to Compact) Model: Mohr-Coulomb Unit Weight: 19 kN/m³ Cohesion': 0 kPa Phi': 30 °
 Name: Silt (Compact to Very Dense) Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion': 0 kPa Phi': 30 °
 Name: Clayey Silt (Soft to Stiff) Model: Mohr-Coulomb Unit Weight: 19 kN/m³ Cohesion': 0 kPa Phi': 27 °
 Name: Bedrock Model: Bedrock (Impenetrable)

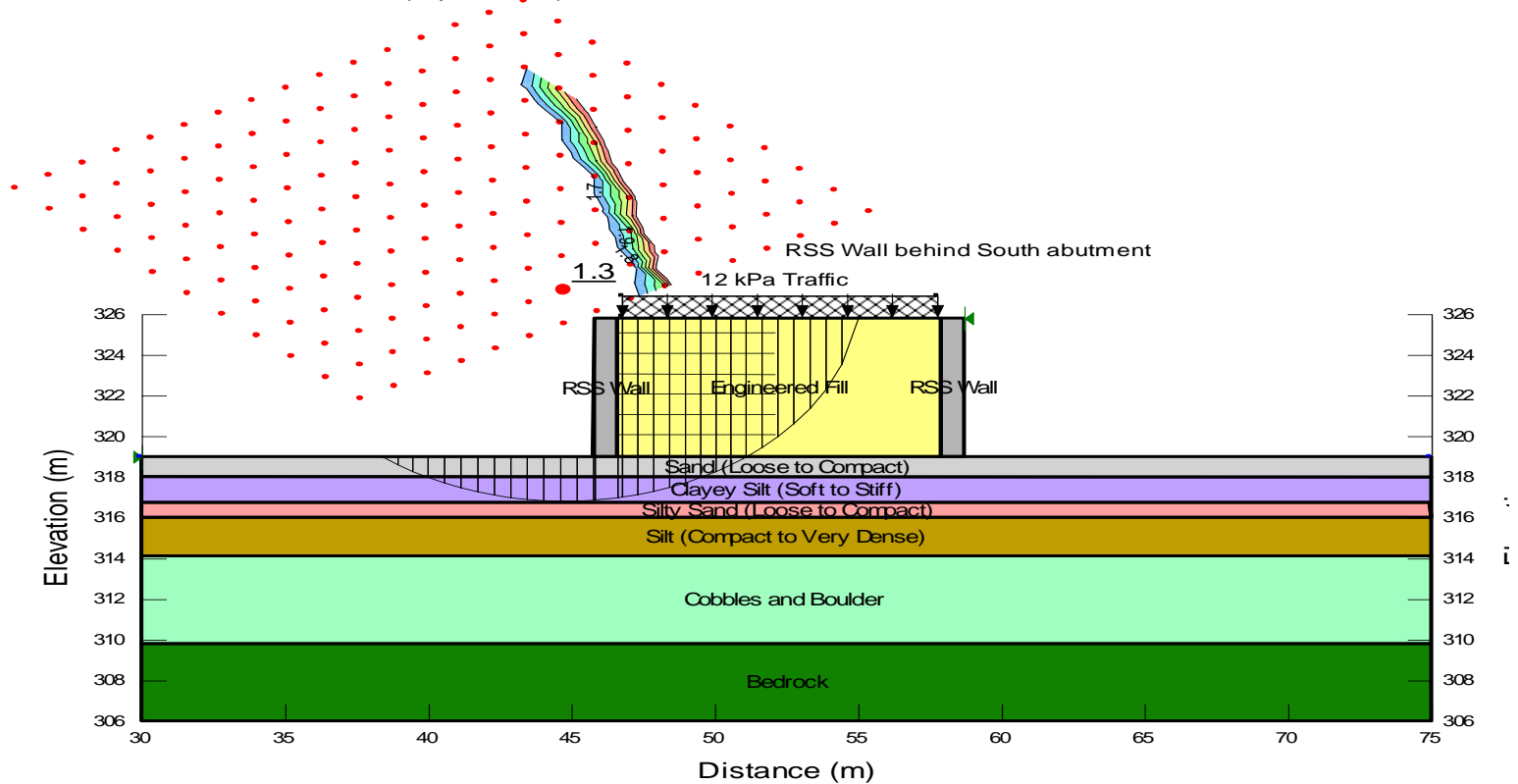


Figure 4: RSS wall behind South abutment - drained seismic condition

Murky Creek Bridge Replacement
North Approach Abutment
Drained Static Condition

Name: Concrete Model: Mohr-Coulomb Unit Weight: 24 kN/m³ Cohesion: 100 kPa Phi: 45 °
 Name: Engineered Fill Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 32 °
 Name: Sand (Loose to Compact) Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 32 °
 Name: Cobbles and Boulder Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 36 °
 Name: Silty Sand (Loose to Compact) Model: Mohr-Coulomb Unit Weight: 19 kN/m³ Cohesion: 0 kPa Phi: 30 °
 Name: Gravel (Very Loose to Loose) Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 32 °
 Name: Silt (Compact to Very Dense) Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 30 °
 Name: Sand and Gravel Fill Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 32 °
 Name: Clayey Silt (Soft to Stiff) Model: Mohr-Coulomb Unit Weight: 19 kN/m³ Cohesion: 0 kPa Phi: 27 °
 Name: Bedrock Model: Bedrock (Impenetrable)
 Name: Cobbles and Boulder Fill Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 36 °

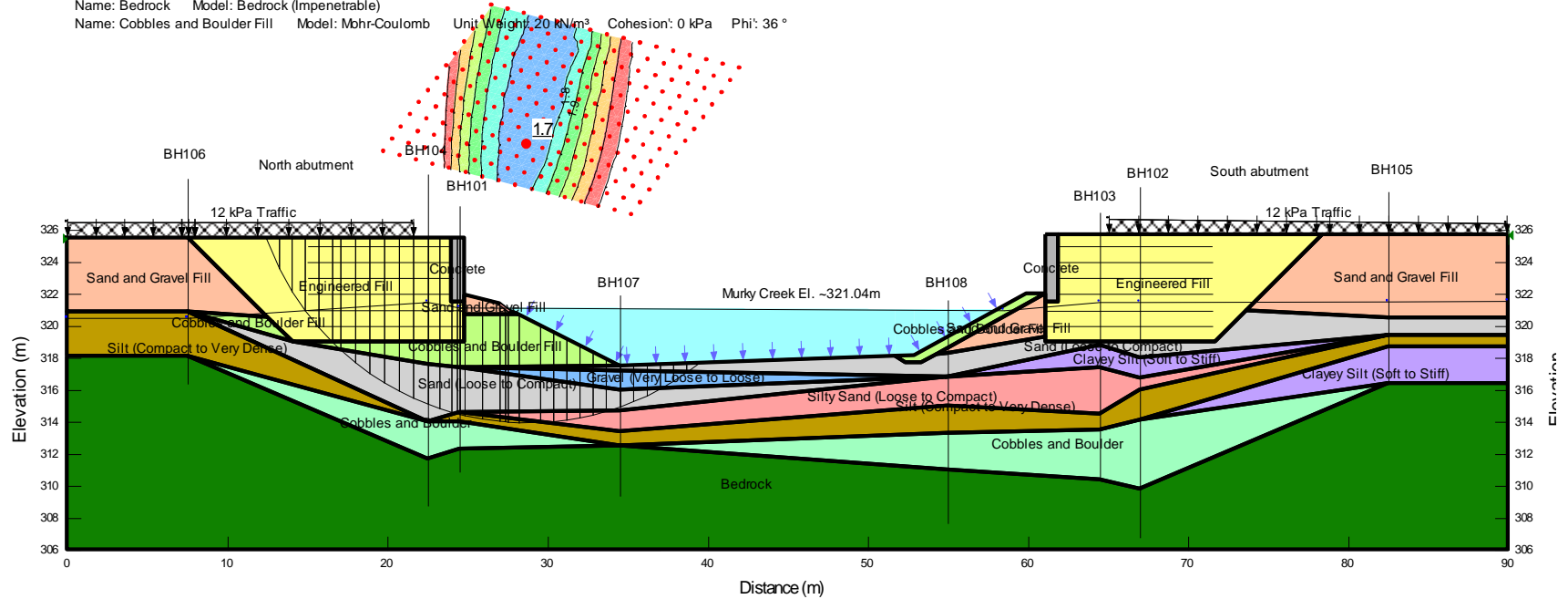


Figure 5: North approach abutment - drained static condition

Murky Creek Bridge Replacement
North Approach Abutment
Drained Seismic Condition

Name: Concrete Model: Mohr-Coulomb Unit Weight: 24 kN/m³ Cohesion': 100 kPa Phi': 45 °
 Name: Engineered Fill Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion': 0 kPa Phi': 32 °
 Name: Sand (Loose to Compact) Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion': 0 kPa Phi': 32 °
 Name: Cobbles and Boulder Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion': 0 kPa Phi': 36 °
 Name: Silty Sand (Loose to Compact) Model: Mohr-Coulomb Unit Weight: 19 kN/m³ Cohesion': 0 kPa Phi': 30 °
 Name: Gravel (Very Loose to Loose) Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion': 0 kPa Phi': 32 °
 Name: Silt (Compact to Very Dense) Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion': 0 kPa Phi': 30 °
 Name: Sand and Gravel Fill Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion': 0 kPa Phi': 32 °
 Name: Clayey Silt (Soft to Stiff) Model: Mohr-Coulomb Unit Weight: 19 kN/m³ Cohesion': 0 kPa Phi': 27 °
 Name: Bedrock Model: Bedrock (Impenetrable)
 Name: Cobbles and Boulder Fill Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion': 0 kPa Phi': 36 °

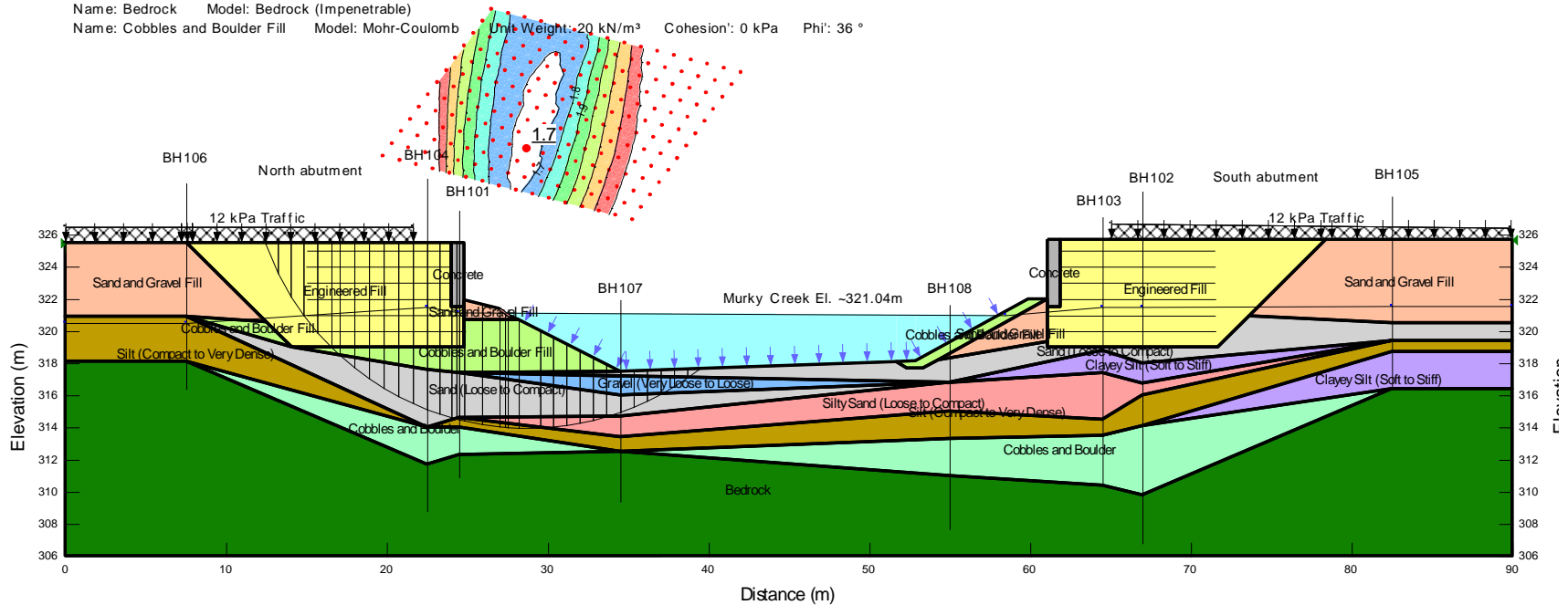


Figure 6: North approach abutment - drained seismic condition

Murky Creek Bridge Replacement
RSS Wall behind North Abutment
Drained Static Condition

Name: RSS Wall Model: Mohr-Coulomb Unit Weight: 24 kN/m³ Cohesion': 100 kPa Phi': 45 °
Name: Engineered Fill Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion': 0 kPa Phi': 32 °
Name: Sand (Loose to Compact) Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion': 0 kPa Phi': 32 °
Name: Cobbles and Boulder Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion': 0 kPa Phi': 36 °
Name: Bedrock Model: Bedrock (Impenetrable)
Name: Cobbles and Boulder Fill Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion': 0 kPa Phi': 36 °

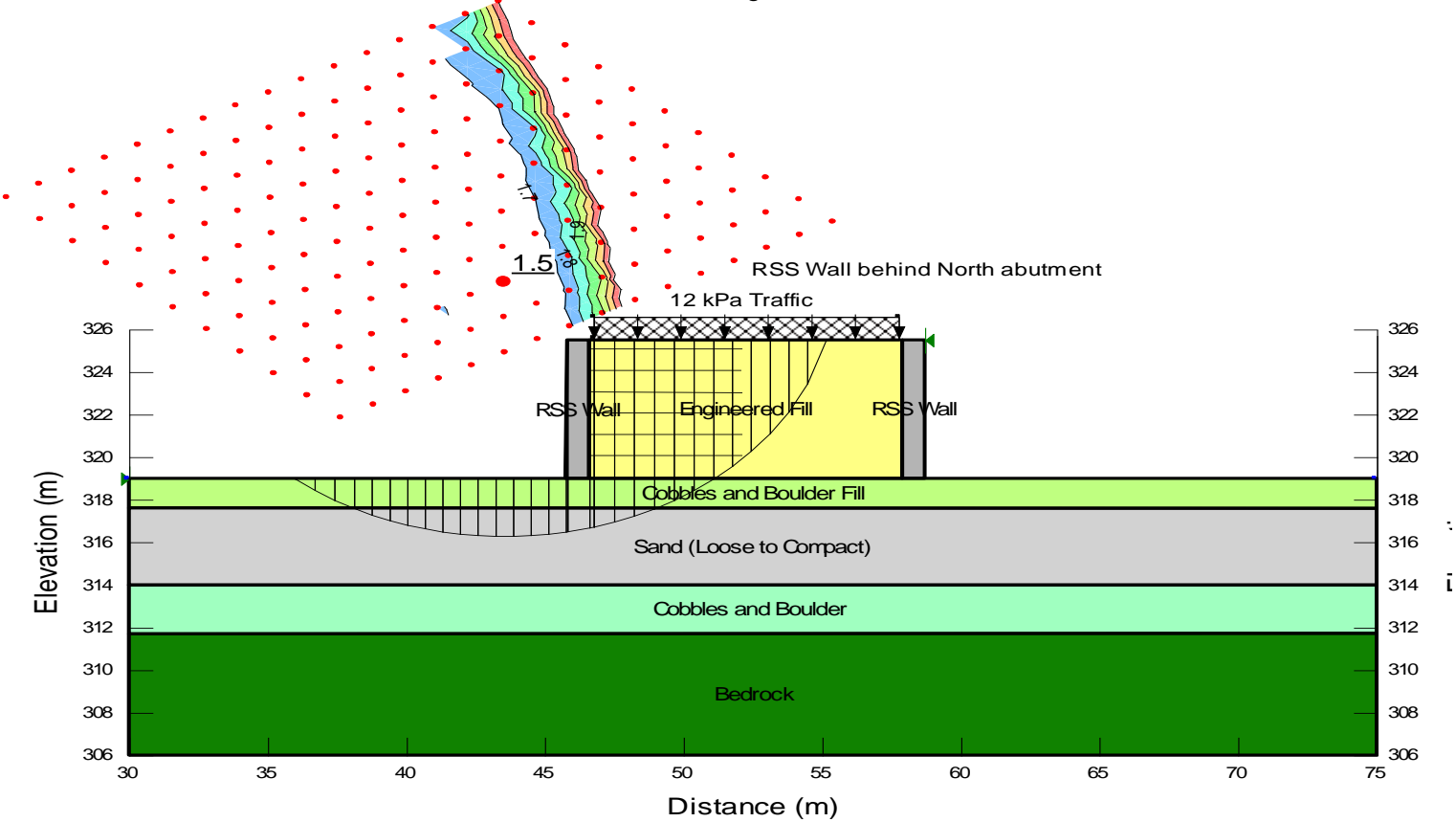


Figure 7: RSS wall behind North abutment - drained static condition

Murky Creek Bridge Replacement
RSS Wall behind North Abutment
Drained Seismic Condition

Name: RSS Wall Model: Mohr-Coulomb Unit Weight: 24 kN/m³ Cohesion': 100 kPa Phi': 45 °
Name: Engineered Fill Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion': 0 kPa Phi': 32 °
Name: Sand (Loose to Compact) Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion': 0 kPa Phi': 32 °
Name: Cobbles and Boulder Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion': 0 kPa Phi': 36 °
Name: Bedrock Model: Bedrock (Impenetrable)
Name: Cobbles and Boulder Fill Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion': 0 kPa Phi': 36 °

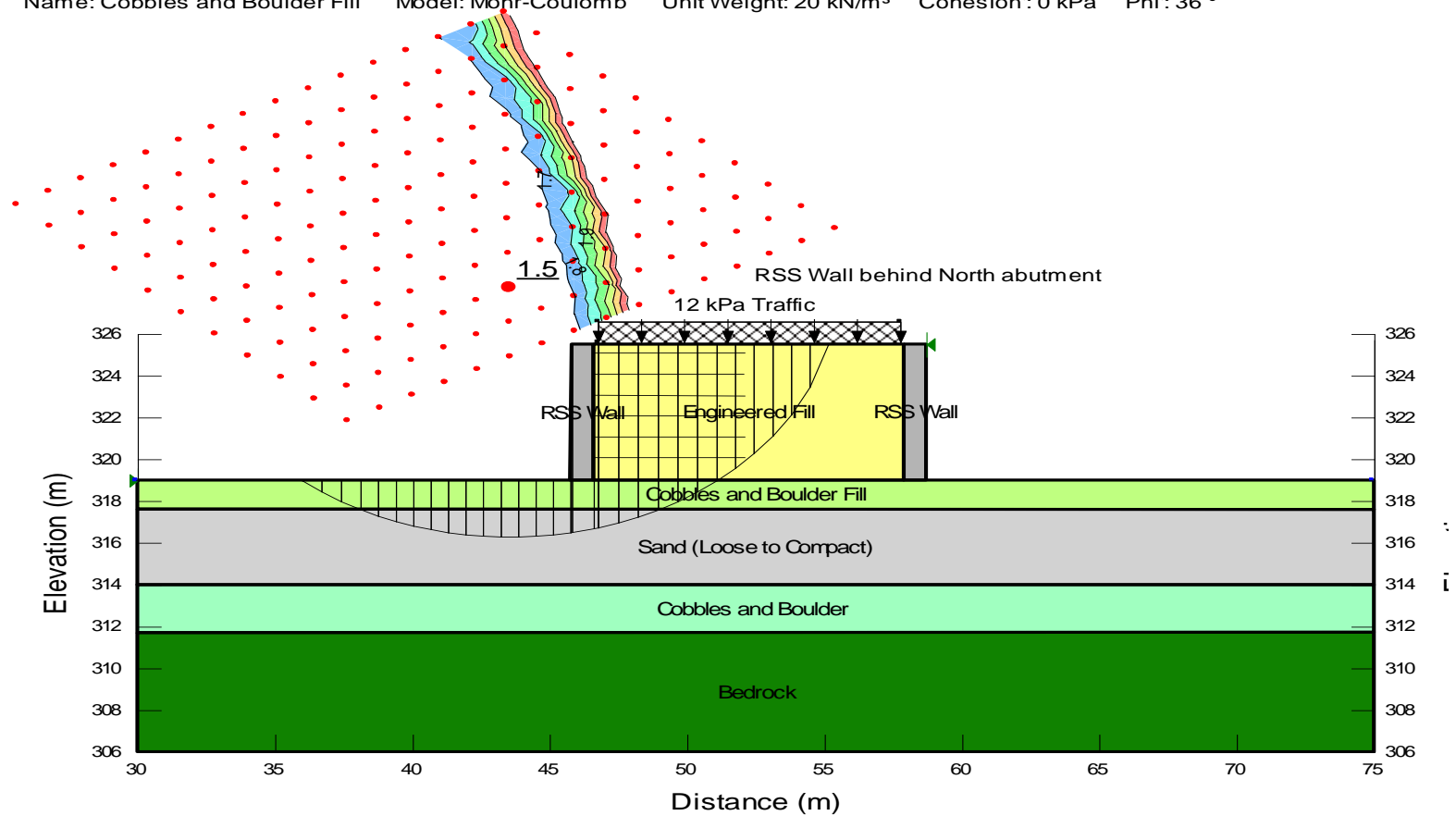
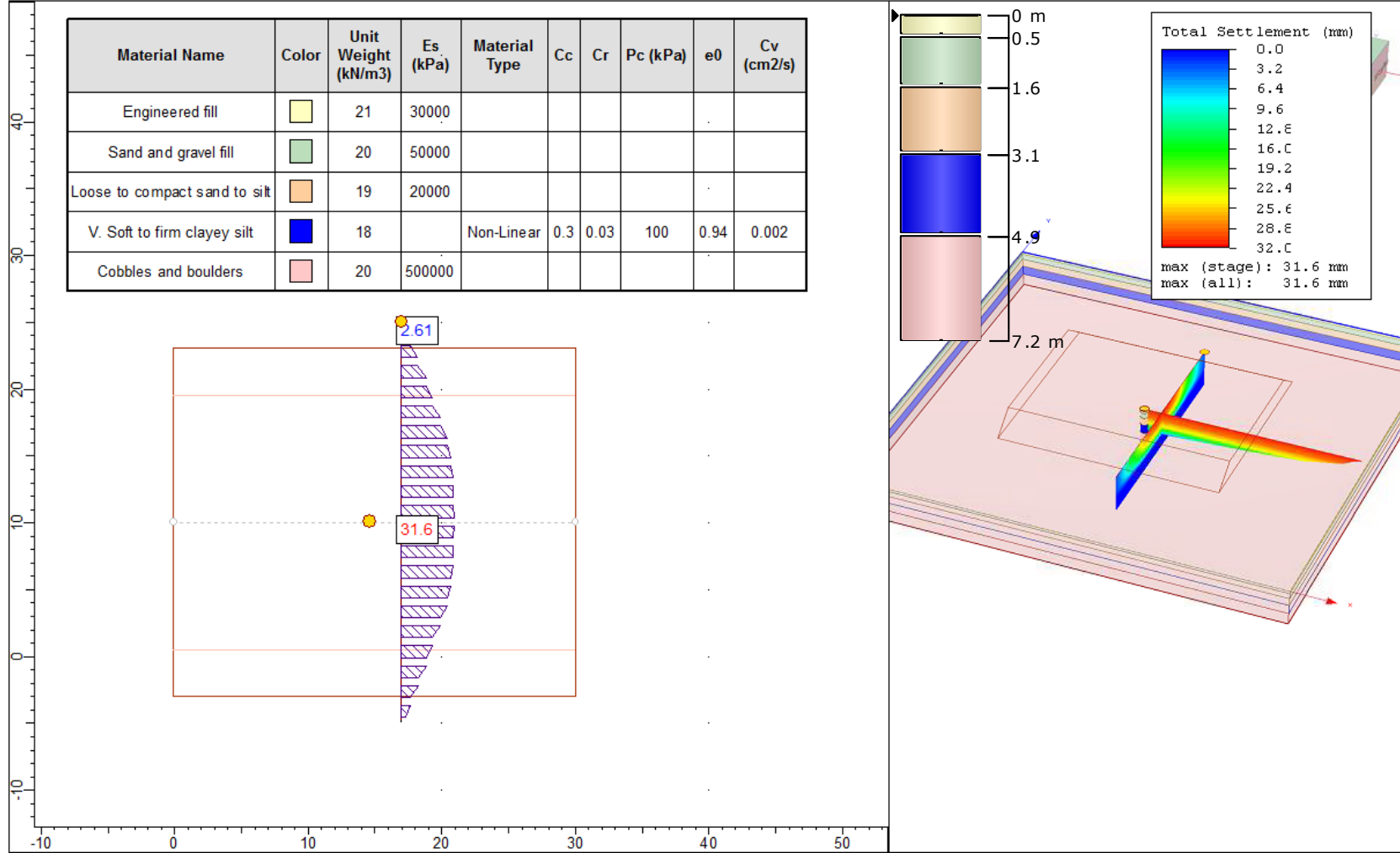


Figure 8: RSS wall behind North abutment - drained seismic condition

Appendix G – Results of Settlement Analyses



Project: Murky Creek Bridge Replacement, Hwy 584, District of Thunder Bay

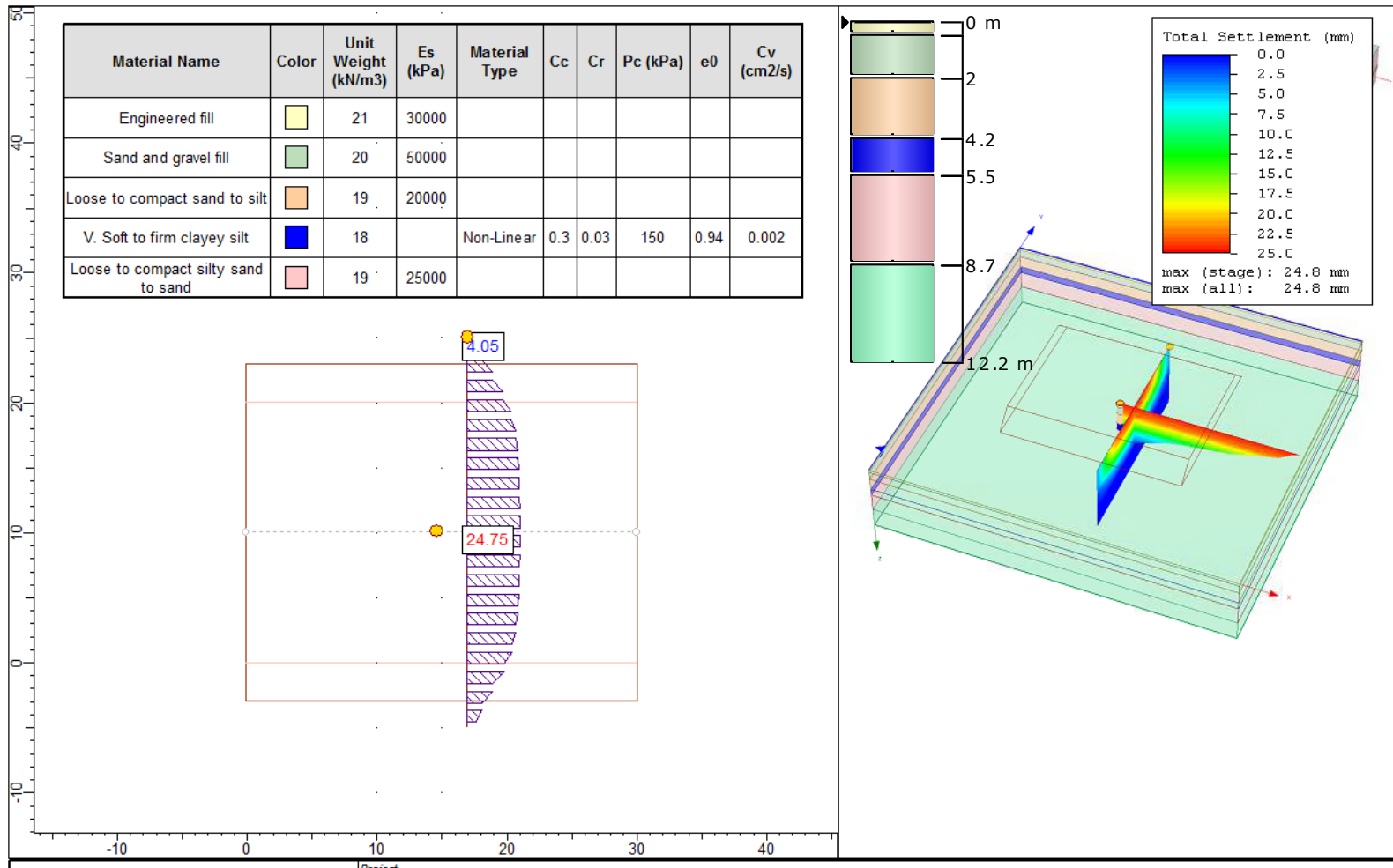
Analysis Description: South Approach Embankment – Total Settlement

Figure No: G-1

Company: exp Services Inc.

Date: April 22, 2016

File Name: Settlement Analysis – Murky Creek Bridge



Project: Murky Creek Bridge Replacement, Hwy 584, District of Thunder Bay

Analysis Description: North Approach Embankment – Total Settlement

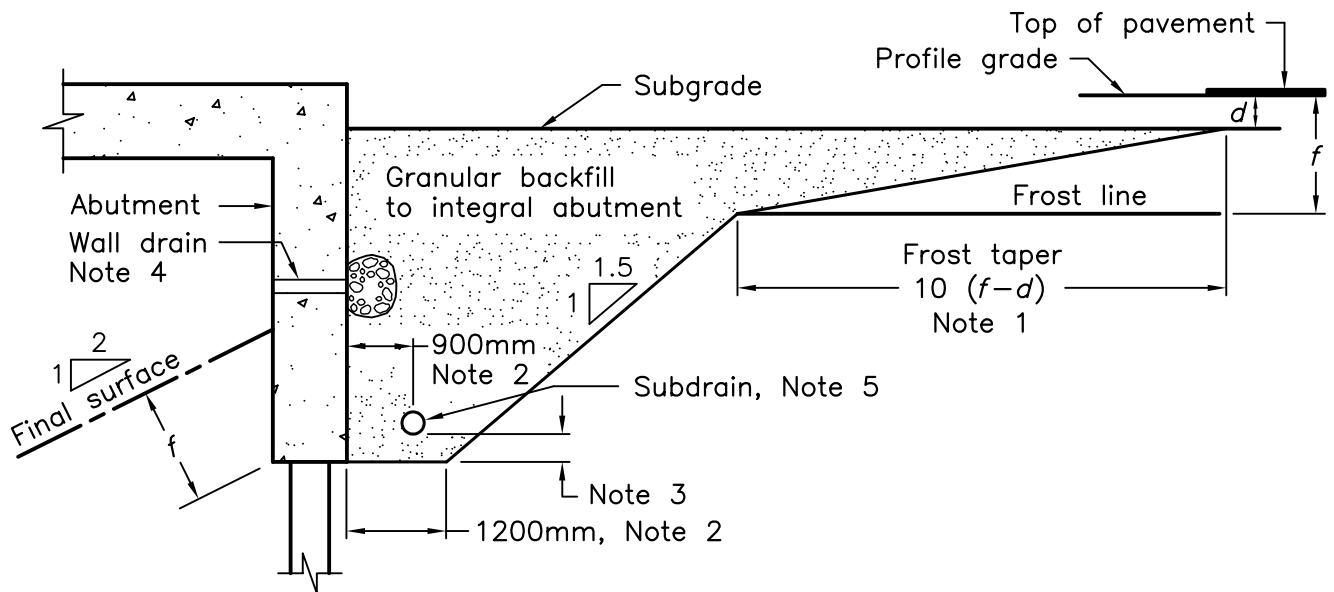
Figure No: G-2

Company: exp Services Inc.

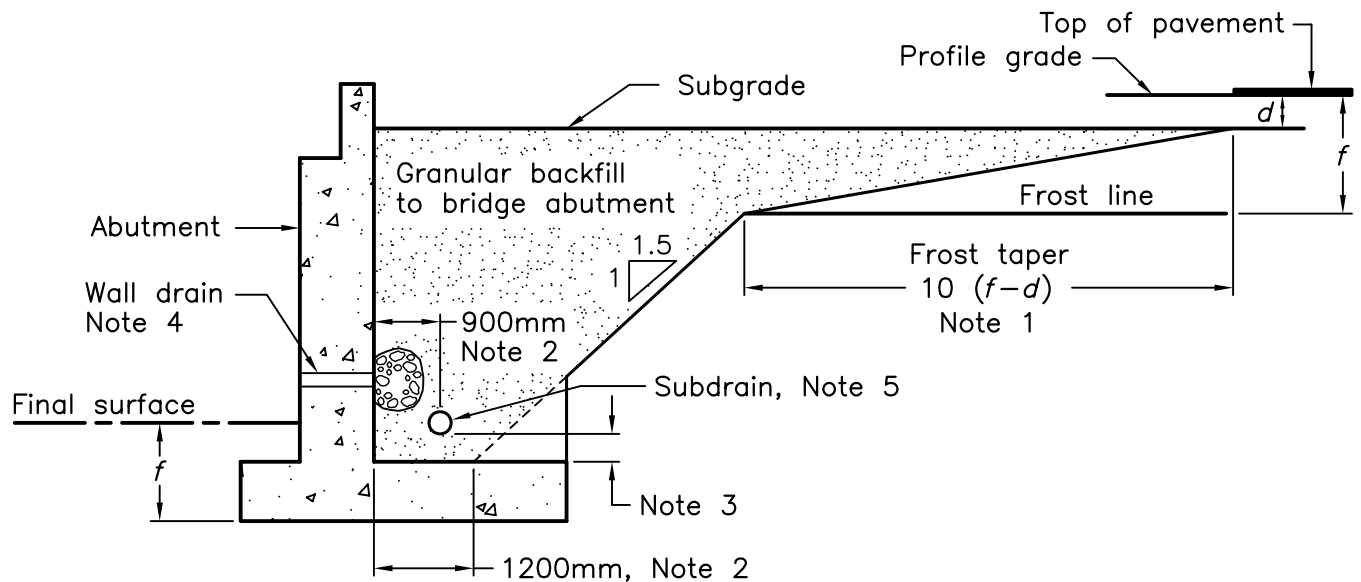
Date: April 22, 2016

File Name: Settlement Analysis – Murky Creek Bridge

Appendix H – OPSDs



INTEGRAL ABUTMENT



ABUTMENT

NOTES:

- 1 d = depth of combined base and subbase courses.
 f = roadbed depth of frost penetration as specified.
- 2 Dimensions perpendicular to back face of abutment.
- 3 Height to be consistent with positive drainage of subdrain as specified.
- 4 Where specified, wall drains shall be installed according to OPSD-3190.100.
- 5 150mm dia perforated pipe subdrain wrapped with geotextile.
- A Lateral limits of granular backfill to bridge abutment to be inside face to inside face of retaining wall or wingwall. Frost taper shall extend the full width of the fill unless interrupted by the retaining wall or wingwall.
- B Sections shown are parallel to centreline of roadway.
- C Subdrain to be installed with a 2% gradient behind wall.
- D All dimensions are in millimetres unless otherwise shown.

ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2005

Rev 0

WALLS

ABUTMENT, BACKFILL

MINIMUM GRANULAR REQUIREMENT



OPSD - 3101.150



- 1 d = depth of combined base and subbase courses.
 f = frost penetration depth as specified.
 - 2 Dimensions perpendicular to back face of retaining wall.
 - 3 Height to be consistent with positive drainage of subdrain as specified.
 - 4 150mm dia perforated pipe subdrain wrapped with geotextile.
Provision shall be made to carry pipe through counterfort wall.
 - 5 Where specified, wall drains shall be installed as per OPSD 3190.100.
- A All dimensions are in millimetres unless otherwise shown.



ONTARIO PROVINCIAL STANDARD DRAWING

WALLS

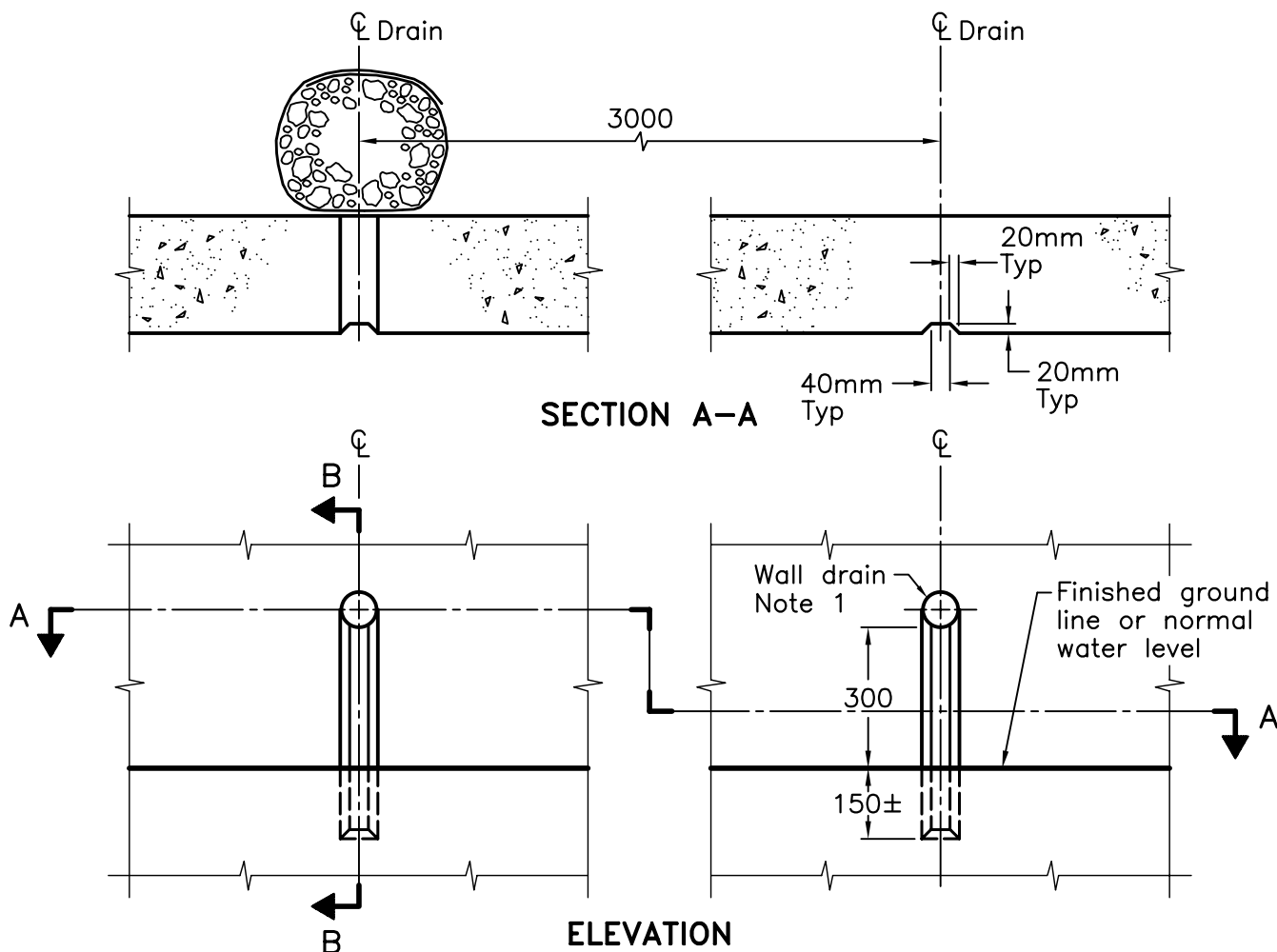
RETAINING, BACKFILL

MINIMUM GRANULAR REQUIREMENT

Nov 2010	Rev	1
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OPSD 3121.150

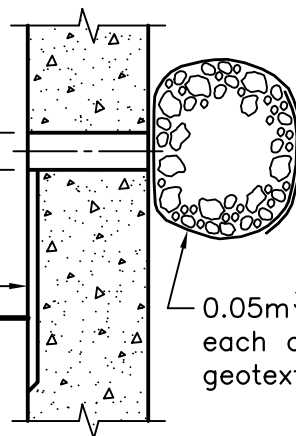


75mm dia wall drain at 3000mm c/c
formed with non-metallic material

Finished ground line or
normal water level

300

Front
face



0.05m³ of 19.0mm clear stone for
each drain completely wrapped with
geotextile and securely tied

SECTION B-B

NOTES:

1 Bottom half of drain to be contoured to shape of vertical groove after removal of formwork.

A Minimum cover to reinforcing bars shall be measured from the base of the groove.

B All dimensions are in millimetres unless otherwise shown.

ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2005

Rev 0

**WALLS
RETAINING AND ABUTMENT
WALL DRAIN**

OPSD – 3190.100



If rock fill is used as a backfill material, consideration should be given to the possible deterioration of the rockfill with time, which could result in the reduction or even the total loss of free-draining properties and, hence, increased frost susceptibility.

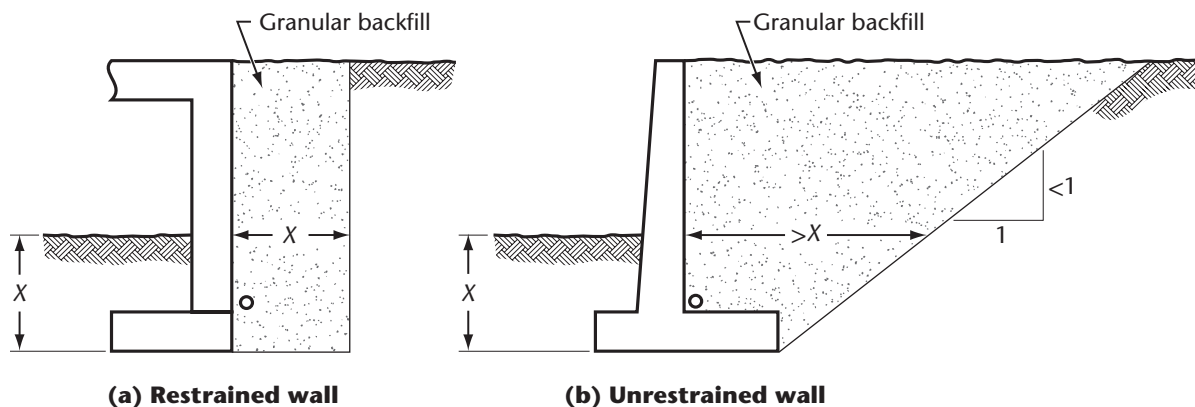


Figure C6.20
Backfill for frost protection
(See Clause C6.9.1.)

C6.9.2 Lateral pressures

C6.9.2.1 General

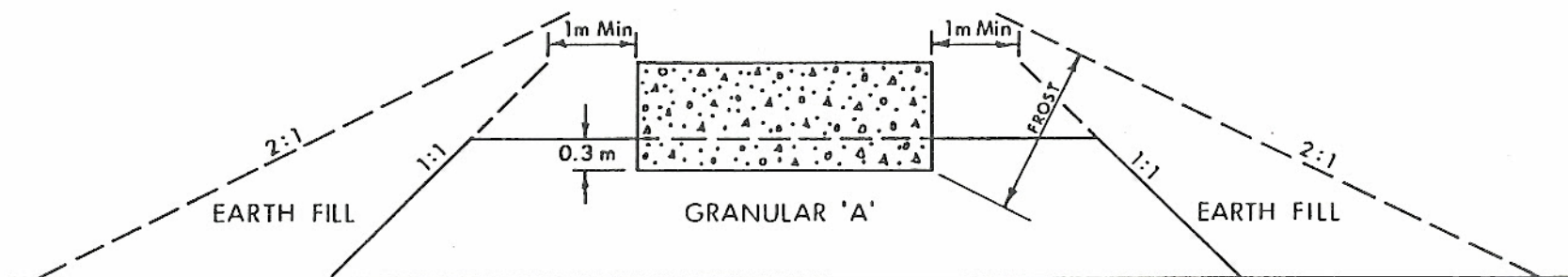
Earth pressure acting on a structure depends on the relative movement of the structure, the backfill, the type of soil adjacent to the backfill, and the soil below the footing or supporting piles. Appropriate geotechnical parameters should be chosen for the calculation of lateral pressures based on recognized geotechnical theories as specified in Clause 6.9.2.2 for the backfill behind the wall. Geotechnical parameters frequently used in allowable stress design methods are applicable in limit states design pressure calculation. Where the possibility exists, hydrostatic pressure needs to be considered, e.g., in situations where walls are partially submerged or where non-free-draining backfill is used.

Clause 6.9.2.1 includes the specification of four lateral pressure conditions for design. The first two cases apply to unrestrained structures, with Item (a) applying to the sizing of the base or pile arrangement with respect to external stability, and Item (b) to the sizing of the structural sections with respect to internal stability. Such sections could be of structural concrete, structural steel, or a proprietary product.

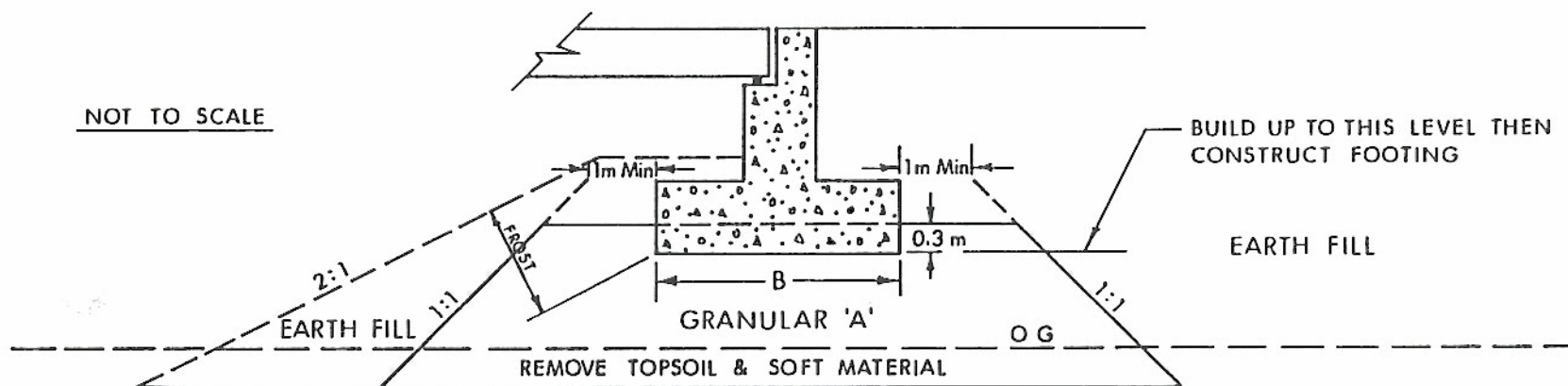
An unrestrained structure is one in which active pressure is mobilized in the backfill due to movement in the supporting structure. This movement corresponds to a rotation of approximately 0.002 about the base of a vertical wall, a horizontal translation of 0.001 times the height of the wall, or a combination of these movements. The lateral pressure applied to the wall for the condition described is an active pressure.

The supporting material will generally be more robust than what is assumed by the Geotechnical Engineer for factored conditions in design. Hence, following installation of the backfill, movement sufficient to cause active condition will generally not have taken place. Horizontal or rotational movement of the base will occur during the installation of each lift of the backfill. Wall deflection during each application and compaction of the backfill will add to the existing deformations. For such a post placement of the fill condition, Item (b) applies, the forces acting on the retaining structure being a function of the compacting equipment and the flexural stiffness of the wall. The residual horizontal pressures due to compaction are largest at the top of the wall, and this is reflected in Clause 6.9.3.

Appendix I – Granular Pad Construction



X SECTION



LONGITUDINAL SECTION

NOTES:

- 1 - REMOVE TOPSOIL &/OR SOFT SUBSOIL UNDER AREA OF COMPACTED GRANULAR 'A' & EARTH FILL.
- 2 - PLACE GRANULAR 'A' & EARTH FILL TO BOTTOM OF FOOTING LEVEL, COMPACTED ACCORDING TO CURRENT M T O STANDARDS.
- 3 - CONSTRUCT CONCRETE FOOTING.
- 4 - PLACE REMAINDER OF GRANULAR 'A' & EARTH FILL AS REQUIRED.



Ministry of
Transportation

ABUTMENT ON COMPACTED FILL
SHOWING GRANULAR 'A' CORE

FIG No

W P

Appendix J – Non Standard Special Provisions (NSSP)

NSSP FOR COBBLES AND/ BOULDERs OBSTRUCTIONS

Scope of Work

The Contractor should be aware that cobbles and boulders layers were encountered overlying the bedrock at the boreholes advanced at the site. It is also encountered in embankment fill at the boreholes advanced at north abutment location. Consideration of the presence of these obstructions must be made in the selection of appropriate equipment and procedures for piling for deep foundations/or for temporary shoring through these materials.

Basis of Payment

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

NSSP FOR SETTING ROCK POINTS INTO BEDROCK

Scope of Work

This Non-Standard Special Provision (NSSP) applies to driving piles fitted with rock points into bedrock.

Procedure

Since the piles will be founded on bedrock, the following procedure can be followed:

1. Drive the pile to bedrock;
2. Drive full energy (60 kJ) 10 blows for the penetration of less than 12 mm;
3. Reduce the hammer energy to 25% of the maximum value and strike the pile 10 times;
4. Increase the hammer energy by 50% of the maximum value and strike the pile 10 times;
5. Increase the hammer energy to 100% of the maximum value and strike the pile 20 times.

END OF SECTION