FINAL REPORT

FOUNDATION INVESTIGATION AND DESIGN REPORT

Mindemoya River Bridge Replacement, Hwy 551, Northeastern Region

Agreement No. 5015-E-0007
Assignment No. 2
GWP No. 5153-12-00
Geocres No. 41G-23

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August 15, 2016

Ontario Ministry of Transportation

Northeastern Region Geotechnical Section

Foundation Investigation and Design Report

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Type of Document:
FINAL

Project Name:
Foundation Investigation and Design for Mindemoya River Bridge Replacement, Hwy 551, Providence Bay, Ontario

Project Number:
ADM-00233185-B0

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August 15, 2016

Table of Contents

PART I FOUNDATION INVESTIGATION REPORT

1.1 Introduction 1
1.2 Site Description and Geological Setting 1
  1.2.1 Site Description 1
  1.2.2 Geological Setting 2
1.3 Investigation Procedures 2
  1.3.1 Site Investigation and Field Testing 2
  1.3.2 Laboratory Testing 3
1.4 Subsurface Conditions 3
  1.4.1 Topsoil 4
  1.4.2 Sand and Gravel 4
  1.4.3 Upper Silty Sand 5
  1.4.4 Sandy Silt 5
  1.4.5 Lower Silty Sand 6
  1.4.6 Bedrock 6
1.5 Chemical Analysis 7
1.6 Ground Water Conditions 7

PART II DISCUSSION AND ENGINEERING RECOMMENDATIONS

2.1 General 9
2.2 Geotechnical Design Considerations for Structure Foundations 9
  2.2.1 Foundation Alternatives 10
  2.2.2 Shallow Foundations 11
  2.2.2.1 Permanent Bridge 11
  2.2.2.2 Temporary Bridge 13
  2.2.3 Deep Foundations 15
  2.2.3.1 General 15
  2.2.3.2 Steel H-Piles or Steel Pipe (Tube) Pile 16
2.3 Approach Embankments 20
  2.3.1 General 20
  2.3.2 Stability Considerations 21
  2.3.2.1 Permanent Bridge 21
  2.3.2.2 Temporary Bridge 22
  2.3.3 Settlement Considerations 23
  2.3.4 Seismic and Liquefaction Potential Consideration 23
2.4 Other Considerations 23
  2.4.1 Lateral Earth Pressure on Structures 23

2.4.1.1 Static Earth Pressure 23
2.4.1.2 Seismic Earth Pressure 24
2.4.2 Earthquake Considerations 27
2.4.3 Corrosion Protection 27
2.4.4 Scour Protection 27
2.4.5 Excavation 28
2.4.6 Temporary Shoring System 28
2.4.7 Dewatering 29
2.4.8 Frost Protection 29
2.4.9 Vibration Monitoring During Pile Driving 29
2.4.10 Abutment Stems Construction 29
PART III CLOSURE 22
PART IV LIMITATION AND USE OF REPORTS 23

Appendices

APPENDIX A: PHOTOGRAPHS
APPENDIX B: DRAWING
Drawing 1 Minemoya River Bridge Replacement – Borehole Locations
Drawing 2 Minemoya River Bridge Replacement – Permanent Bridge – Soil Strata
Drawing 3 Minemoya River Bridge Replacement – Temporary Bridge – Soil Strata
APPENDIX C: BOREHOLE LOGS
APPENDIX D: LABORATORY TEST RESULTS
Figure 1 Grain Size Distribution – Gravelly Sand (SW)
Figure 2 Grain Size Distribution – Upper Silty Sand (SM-CL)
Figure 3 Grain Size Distribution – Sandy Silt (ML)
Figure 4 Grain Size Distribution – Lower Silty Sand (SW)
Figure 5 Plasticity Chart – Sandy Silt (ML-CL)
Chemical Results
APPENDIX E: BEDROCK CORE PHOTOGRAPHS
APPENDIX F: RESULTS OF STABILITY ANALYSES
Figure F1 Proposed Permanent Bridge South Abutment – Drained Static Condition
Figure F2 Proposed Permanent Bridge South Abutment – Drained Seismic Condition
Figure F3 Proposed Permanent Bridge South Embankment – Drained Static Condition
Figure F4 Proposed Permanent Bridge South Embankment – Drained Seismic Condition
Figure F5 Proposed Permanent Bridge North Embankment – Drained Static Condition
Figure F6 Proposed Permanent Bridge North Embankment – Drained Seismic Condition

APPENDIX G: FOUNDATION-RELATED SPECIFICAIONS

- **OPSD – 3101.150**
- **OPSD – 3121.150**
- **OPSD – 3190.100**
- **CHBDC CAN/CSA-S6-06 Figure C6.20**

APPENDIX H: NON-STANDARD SPECIAL PROVISIONS

APPENDIX I: PREVIOUS INVESTIGATION BOREHOLE LOGS

1 FOUNDATION INVESTIGATION REPORT

1.1 Introduction

This foundation investigation report presents the results of a geotechnical investigation completed by **exp** Services Inc. for the replacement of the Mindemoya River Bridge located on Hwy 551, approximately 1 km north of Government Road in the community of Providence Bay, Concession Road 12 with the Municipality of Central Manitoulin, Ontario, the Ministry of Transportation (MTO) Northeastern Region. The work was undertaken under Agreement No. 5015-E-0007, Assignment No. 2 (GWP No. 5153-12-00). The terms of reference (TOR) were as presented in the MTO letter dated May 16, 2016.

Based on information included in the TOR and the preliminary foundation investigation and design report for Highway 551 Mindemoya River Bridge Replacement recently completed by Stantec on August 2015, and the report (Geocres 41G-22) is available from GEOCRES, it is understood that the permanent structure will be single-span bridge placed at the same location as the current bridge. A single-span temporary bridge is proposed as a traffic detour which is to be located on the east of the existing bridge.

The purpose of this geotechnical investigation is to examine the existing soil conditions within the construction limits for this temporary bridge by drilling and sampling a limited number of boreholes. The site specific geotechnical investigation at temporary bridge location consisted of borings, soil sampling, borehole logging, and field and laboratory testing. Based on the interpretation of this site specific geotechnical information, recommendations for the geotechnical engineering aspects of design and construction of the temporary bridge were developed and are presented in this report. Similarly, this data and factual information from the referenced Stantec report of August 2015 were interpreted to provide geotechnical engineering recommendations for design of the permanent bridge replacement. It also includes the detailed design of permanent bridge replacement based on geotechnical investigation performed by Stantec on August 2015.

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

1.2 Site Description and Geological Setting

1.2.1 Site Description

The Mindemoya River Bridge is located approximately 1 km north of Government Road in the community of Providence Bay, Concession Road 12 with the Municipality of Central Manitoulin, Ontario on Highway 551. The existing structure is single span steel bridge with a wood bridge deck, and is about 12 m in length and about 8 m wide including two traffic lanes and respective shoulders. The existing structure was built around 1965 and is rated as being in fair condition.

The proposed temporary bridge for traffic detour is to be located approximately 4.5 m east of the existing bridge and the width is about 4.14 m. The site plan and cross-section profiles for the temporary bridge for the replacement of Mindemoya River Bridge are as shown on Drawings 1 to 3 in Appendix B. Photographs of the site/bridge are included in Appendix A.

At the site Highway 551 runs in a generally north to south direction, and Mindemoya River flows from east to west towards the Provindence Bay and Lake Huron at the southwestern boundary of the Municipality of Central Manitoulin. Highway 551 has a single 3.5 m wide lane of traffic in each direction with paved and gravel shoulder of varying widths. At the time of investigation, June 2016, approximate river water elevation was 176.9 m and the elevation of top of the existing bridge deck was approximately 178.8 m. Due to the difficulty in access to the proposed temporary detour bridge site, some tree removal was required. The Cranston's Tree Services removed the middle height tree and tree branches to create access to the temporary detour bridge site.

The banks of the river in the vicinity of the bridge is generally flat to rolling and the ground surface elevation generally decreases from the bridge toward the west. Vegetation in the area consists of deciduous and coniferous trees and smaller low-lying shrubs and grass. Selected photographs of the site are provided in Appendix A.

1.2.2 Geological Setting

The project site is located within the Canadian Shield and is characterized by rock tablelands tilted towards the southwest. Based on soil and bedrock rock mapping published by the Ontario Geological Survey, the subsurface conditions at the site consist of clay, silt, sand, gravel, and boulders underlain by Ordovician shale and limestone, and dolostone of the Amabel Formation.

1.3 Investigation Procedures

1.3.1 Site Investigation and Field Testing

The fieldwork for the temporary bridge investigation was carried out at June 20 to 22, 2016 and for permanent bridge the investigation was performed by Stantec on November 17 to 26, 2014 and the report (Geocres 41G-22) is available in GEOCRES. The geotechnical investigation conducted by Stantec consist of six sampled boreholes BH14-1, BH14-2, BH14-3, BH14-4, BH14-5 and BH14-6 and current investigation consists of two (2) sampled boreholes drilled at the locations close to the proposed north abutment and south abutment of the proposed temporary bridge. The locations of the boreholes are shown on Drawing 1 in Appendix B. The boreholes were strategically located to provide the appropriate subsurface information for the design and construction of the proposed temporary structure.

The boreholes (BH 16-1 and 16-2) were advanced using a CME-55 truck mounted drill rig operated by Landcore Drilling Co. Ltd. Both drills were equipped with continuous flight hollow stem augers and standard soil/bedrock sampling equipment. The hollow stem augers were used to advance Boreholes 16-1 and 16-2 to the depths of 3.0 m and 9.1 m, respectively and afterwards diamond drilling equipment and NW casing were used to advance to the end of boreholes. The two drilled Boreholes 16-1 and 16-2 were advanced to the depths 21.8 m and 21.4 m, respectively, up to 3 m into the bedrock. Drawing No. 1 to 3 in Appendix B shows the locations of boreholes and cross-sections of stratigraphy along the proposed permanent and temporary bridge.

The borehole locations and their ground surface elevations were temporary surveyed by **exp** personnel. The borehole locations (referenced to the MTM NAD83 coordinate system) were identified using Garmin Global Positioning System (GPS). The elevations of completed boreholes were surveyed by **exp** personnel relative to the Temporary Benchmark (TBM) set at the southeast corner of the existing bridge. The elevation of TBM (179.0 m) was assumed based on the Stantec's preliminary foundation investigation and design report provided in the TOR.

During the drilling of the boreholes, soil samples were obtained using a 51 mm outside diameter (O.D.) split-spoon sampler in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586) at intervals ranging from 0.75 m to 1.5 m in depth as shown on the attached borehole logs (Appendix C). The original field (uncorrected) SPT "N" values were recorded on the borehole logs as recommended in the Canadian Foundation Engineering Manual (CFEM, pg. 40) and used to provide an assessment of in-situ consistency or relative density of non-cohesive soils. When a hard stratum was reached sampling of hard material was performed by diamond core drilling, using a 1.5 m long NQ double tube wireline core barrel.

Upon completion of the boreholes, ground water level measurements were carried out from the boreholes in accordance with the MTO guidelines. The measured ground water levels after completion of drilling boreholes were recorded on borehole log sheets in Appendix C. The boreholes were decommissioned by bentonite/cement mixtures in accordance with the Ministry of the Environment Regulation 903, as amended by Regulation 128/03 (the well regulation under the *Ontario Water Resources Act)*.

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

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

1.3.2 Laboratory Testing

All samples returned to the laboratory were subjected to visual examination and classification. The laboratory testing program included the determination of natural moisture content and particle size distribution for approximately 25% of the collected soil samples. Atterberg limits, corrosivity and sulphides tests were also performed. All of the laboratory tests were carried out in accordance with MTO and/or ASTM Standards as appropriate.

The laboratory test results are provided on the attached borehole log sheets in Appendix C. The results of the grain size analyses and Atterberg limits tests are presented graphically in Appendix D. The chemical test results (corrosivity and sulphides) are also presented in Appendix D.

1.4 Subsurface Conditions

The detailed subsurface conditions encountered in the boreholes advanced during this investigation are presented on the borehole log sheets in Appendix C. The borehole logs and the Preliminary Foundation Investigation and Design Report by Stantec are presented in Appendix H and also included in Drawing 2 and 3 (Appendix B). Laboratory test results and Chemical test results are provided in Appendix D. The "Explanation of Terms Used in Report" preceding the borehole logs in Appendix C forms an integral part of and should be read in conjunction with this report.

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

The geological stratigraphy noted in the boreholes conducted by Stantec (BH14-1, BH14-2, BH14-3, BH14-4, BH14-5 and BH14-6) was in general agreement with the ground conditions encountered during **exp** investigations (BH16-1 and BH 16-2).

In general, the subsurface conditions along the proposed temporary bridge location consist of topsoil over gravelly sand underlain by silty sand over sandy silt over silty sand followed by gravelly sand layer overlying limestone bedrock. A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

A detailed description of the stratigraphy encountered is discussed further in subsequent sections. It should be noted that the following sections are based on the geotechnical investigation conducted by **exp** only.

1.4.1 Topsoil

Topsoil was encountered at the surface of both two boreholes. Thickness of the topsoil layer was about 100 mm. Topsoil thicknesses may further vary beyond the borehole locations.

1.4.2 Sand and Gravel

Sand and gravel layer was encountered below the topsoil only in Borehole 16-1. The thickness of this layer was about 1.4 m extending from Elev. 177.5 m to Elev. 176.1 m.

The composition of this layer is sand and gravel with trace silt and clay size particles. The material is brownish grey in color, and wet. The SPT "N" values within this layer is 23 blows per 300 mm penetration, suggesting compact compactness condition.

Laboratory testing performed on selected samples consisted of 2 moisture content and 1 grain size distribution tests. The test results are as follows:

Moisture Content:

- 8.1% to 17.8%

Grain Size Distribution:

- 35 % gravel;
- 56% sand; and
- 9% silt and clay

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

1.4.3 Upper Silty Sand

Silty sand layer was encountered below the sand and gravel layer in Borehole 16-1 and below topsoil in Borehole 16-2, respectively. The thickness of this layer is approximately 9.2 m to 12.1 m extending from Elev. 177.5 m to Elev. 165.4 m.

The composition of this layer is sand and silt with trace organics, occasional cobbles, trace gravel, and trace clay size particles. The material is blackish grey to grey in color, and wet. The SPT “N” values within this layer ranges from 0 to 34 blows per 300 mm penetration, suggesting very loose to dense compactness condition.

Laboratory testing performed on selected samples consisted of 25 moisture content and 9 grain size distribution tests. The test results are as follows:

Moisture Content:

- 16.4% to 23.3%

Grain Size Distribution:

- 0 % gravel;
- 47% to 92% sand; and
- 8% to 78% silt and clay

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

1.4.4 Sandy Silt

Sandy silt layer was encountered below the silty sand layer Boreholes 16-1 and 16-2. The thickness of this layer ranges from 3.7 m to 4.5 m extending from Elev. 166.9 m to Elev. 161.8 m.

The composition of this layer is sand and silt with trace organics, occasional cobbles, trace gravel and trace to some clay size particles. The material is blackish grey to grey in color, and wet. The SPT “N” values within this layer ranges from 22 to 35 blows per 300 mm penetration, suggesting compact to dense compactness condition.

Laboratory testing performed on selected samples consisted of 5 moisture content, 2 grain size distribution and 2 Atterberg Limits tests. The test results are as follows:

Moisture Content:

- 15.9% to 21.7%

Grain Size Distribution:

- 0 % gravel;

- 5% to 22% sand; and
- 78% to 95% (70% silt and 25% clay) silt and clay

Atterberg Limit:

- Liquid Limit: 21%
- Plastic Limit: 31%
- Plasticity Index: 10%

Selected sample Borehole 16-1 SS16 found non plastic.

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

1.4.5 Lower Silty Sand

Silty sand layer was encountered below the sandy silt layer Boreholes 16-1 and 16-2. The thickness of this layer ranged from 2.3 m to 3.5 m extending from Elev. 162.4 m to Elev. 158.9 m.

The composition of this layer is sand and silt, some gravel and trace clay size particles. The material is grey in color, and wet. The SPT “N” values within this layer typically ranges from 16 blows per 300 mm penetration to 50 blows per 125 mm penetration, suggesting compact to very dense compactness condition.

Laboratory testing performed on selected samples consisted of 3 moisture content and 1 grain size distribution tests. The test results are as follows:

Moisture Content:

- 7.4% to 20.9%

Grain Size Distribution:

- 0% gravel;
- 94% sand;
- 6% silt and clay

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

1.4.6 Bedrock

The presence of bedrock, at approximately between 18.2 m to 18.7 m below the existing road surface was recorded. The bedrock was confirmed using coring of about 3 m long cores in both boreholes. The elevation of the actual bedrock surface below the temporary bridge site ranges from Elev. 159.4 m to Elev. 158.9 m. The actual bedrock surface depth and elevation encountered at these borehole locations are listed in Table 1.1. Photographs of rock cores are included in Appendix E.

Table 1.1 Depth and elevation of bedrock or possible bedrock surface

Borehole	**Depth Below Ground Surface (m)**	**Elevation (m)**	**Comments**
16-1	18.7	158.9	Bedrock Cored
16-2	18.2	159.4	Bedrock Cored

Based on the bedrock cores recovered, the bedrock consists of limestone with dolomitic seams. In general, the bedrock samples are described as light grey in colour, smooth and fine grained, moderately to slightly weathered. The Rock Quality Designation (RQD) measured on the core samples typically ranged from approximately 57.5% to 88.3%, indicating a rock mass of fair to good quality. Based on these cores the fracture index is estimated to be 7 to 10. The uniaxial compression strength of the rock was not measured in this investigation.

1.5 Chemical Analysis

Two representative samples retrieved from the silty sand deposits in Boreholes 16-1 and 16-2 were tested for resistivity, pH, and water soluble sulphates and chloride concentrations. The results of this chemical analysis are provided in Table 1.2 and in Appendix D.

Table 1.2 Groundwater levels recorded at the site

Borehole and Sample No.	**Depth (m)**	**pH**	**Sulphate (µg/g)**	**Chloride (µg/g)**	**Resistivity (Ohm·m)**	**Conductivity (µmho/cm)**	**Oxidation-Reduction Potential (mV)**
16-1, SS5	3.05 to 3.66	7.65	32	<20	94	107	+158
16-2, SS3	1.37 to 2.13	7.72	28	<20	84	120	+162

1.6 Ground Water Conditions

Information regarding groundwater levels at the site was obtained by measuring the water levels in the open boreholes after completion of drilling. The groundwater levels measured in the boreholes are shown on Table 1.3 and borehole logs. Water levels measured in open boreholes might not be stabilized due to a short-term observation and using of wash boring technique to advance the boreholes.

At the time of investigations, the water level measured at the river was approximately at Elev. 176.9 m. 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 1.3 Groundwater levels recorded at the site

Borehole No.	Location Relative to Existing Bridge	Date of Drilling	Groundwater Level (Elevation, m)
16-1	South Approach/Abutment	June 21, 2016	175.8
16-2	North Approach/Abutment	June 20, 2016	175.8

2 DISCUSSIONS AND ENGINEERING RECOMMENATIONS

2.1 General

This section of the report provides geotechnical design recommendations for Mindemoya River Bridge replacement on Hwy 551, located approximately 1 km north of Government Road in the community of Providence Bay, Ontario, including construction of a new bridge at the same location of the existing bridge and a temporary detour bridge on the east side of the existing bridge. The recommendations are based on information included in the preliminary foundation investigation and design report for Highway 551 Mindemoya River Bridge Replacement recently completed by Stantec on August 2015, and interpretation of the factual data obtained from the boreholes advanced during the current investigation at the location of the proposed temporary bridge and presented in Part I- Foundation Investigation Report of this report. The Stantec report was provided by MTO. The interpretation and recommendations provided in this report are intended solely to permit designers to assess foundation alternatives, and design the proposed structures. 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.

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) (2014), the provisions in the TOR and good practice. It also provides discussion about the structure foundation type and stability analyses for both the permanent and temporary bridges, as requested in the TOR.

2.2 Geotechnical Design Considerations for Structure Foundations

In general, the Mindemoya River Main Bridge Replacement site is underlain by asphalt pavement over cohesionless fill materials and cohesionless native deposits overlying limestone bedrock. The native deposits, consisting predominantly of layers of silty sand and sandy silt extend to a depth of approximately between 16.9 m to 20.1 m below the existing ground surface. The compactness of these deposits generally varies from compact to very dense. The bedrock was encountered at between 16.9 m to 20.1 m depth. The groundwater level encountered at the site varied between approximate elevations of 175.2 m to 176.0 m.

The temporary detour bridge site is generally underlain by topsoil and cohesionless native deposits overlying bedrock. The native deposits, consisting predominantly of layers of silty sand and sandy silt extend to a depth of approximately between 18.2 m to 18.7 m below the existing ground surface.

The compactness of these deposits varies from very loose to very dense. The bedrock was encountered at between 18.2 m to 18.7 m depth. The groundwater level encountered at the temporary bridge site was at the depth of 1.8 m (Elev. 175.8 m). The water level of the Mindemoya River measured on June 22, 2016 was at an approximate elevation of 176.9 m.

2.2.1 Foundation Alternatives

Based on the sub-surface conditions encountered at the proposed permanent bridge and temporary detour bridge site, various shallow and deep foundation options have been considered. Table 2.1 shows the advantages and disadvantages of considered options.

Table 2.1 Evaluation of foundation alternatives

Options	Advantages	Disadvantages	Relative Costs	Risks/ Consequences	Rank
Spread footing supported on native soils	▪ Straightforward construction ▪ Limited excavation	▪ Sandy silt is easily disturbed - excavation and removal of unsuitable native soils may be required below the founding elevation ▪ Dewatering system is required ▪ Scour Protection may be required ▪ Not compatible for integral abutment	▪ Low to Medium	▪ Susceptible to differential settlements ▪ Excavation may be below groundwater and basal instability may be an issue ▪ Higher scour risk	1
End-bearing steel H-piles driven onto bedrock	▪ High geotechnical resistance available ▪ Negligible or minimum settlement ▪ Compatible for integral and semi-integral abutment	▪ High cost for mobilization for pile driving equipment ▪ Pile capacity may not be fully utilized	▪ Medium to high	▪ Risk of pile tip damage, should be adequately protected while driving through cobbles and boulders ▪ Variation in pile tip elevations	2
End-bearing steel tube	▪ High geotechnical resistance available	▪ Slightly greater risk than for steel H-pile foundations	▪ Medium to high	▪ Greater risk than steel H-piles option if obstructions	3

Options	Advantages	Disadvantages	Relative Costs	Risks/ Consequences	Rank
piles driven onto bedrock	▪ Negligible or minimum settlement	if obstructions (cobbles and/or boulders) are encountered during driving		(cobbles and/or boulders) are encountered during driving	
Caissons founded on bedrock	▪ Can transmit very large axial and lateral loads	▪ Not suitable for integral bridge abutment ▪ Generally, not suitable if bedrock is relatively deep ▪ Not compatible for integral abutment	▪ High	▪ Risk of cave-in, especially below groundwater table during drilling	4
Frictional piles in native soils	▪ No excavation required	▪ Noisy installation ▪ May cause ground heave	▪ Medium to high	▪ Larger settlement ▪ Piles may need to be driven to bedrock due to lower field capacities than estimated capacities	5

Based on comparison of the above foundation options, the preferred option from a geotechnical/foundations perspective is to support the abutments for the proposed permanent and temporary bridges with spread footings on native soils.

2.2.2 Shallow Foundations

2.2.2.1 Permanent Bridge

Based on geotechnical data encountered during the 2015 investigation (i.e. Stantec report), semi-integral abutments of the proposed permanent bridge can be founded on shallow foundations set (i.e. spread footing) on the native sandy silt to silty sand. Provision should be made for the construction of a levelling mat of lean concrete (i.e., mud slab) beneath the proposed founding level to fill in any voids that may be present on the surface of the native soils, and to provide an acceptable working surface. A Non-Standard Special Provision (NSSP) for the supply and installation of a working slab should be included in the contract documents and a sample has been provided in Appendix H of this report.

Spread footings which meet a requirement for an adequate protection against frost penetration in the project area of a minimum 1.6 m depth below the lowest surrounding area will be founded on the native sandy silt to silty sand.

Footing Elevation

Based on the results of the geotechnical investigation and a requirement for adequate protection against frost penetration in the project area (i.e. a minimum 1.6 m below the lowest surrounding area), the following founding elevations of spread footings are recommended:

Table 2.2 Recommendations for footing depth for the Proposed Permanent Bridge

Structure	Material at Founding Level	Foundation Elevation (m)	Foundation Depth Below Existing Grade
Permanent Bridge - Abutments	Native sandy silt to silty sand (Reference Boreholes 14-3 and 14-4)	176.0	2.9 m excavation for existing fill and loose to compact sand

Geotechnical Resistances

In the context of the CHBDC, a satisfactory foundation design would require, in terms of Limit States Design, the factored geotechnical resistance of its foundation to withstand and not exceed the imposed Ultimate Limit State loads - (ULS) Design Approach, and its ability to deform acceptably under the Service Limit State loads - (SLS) Design Approach. These associated loads are typically known as unfactored and factored loads, respectively.

Therefore, spread footings placed on the properly prepared subgrade at the design level given in Table 2.2, should be designed based on the factored resistances at ULS and geotechnical reactions at SLS for 25 mm of settlement given in Table 2.3 below. The footing width of 3 m is assumed.

Table 2.3 Geotechnical resistance at ULS and geotechnical reaction at SLS for a 3 m wide footing for the proposed permanent bridge

Structure	Soil at Founding Level	Width of Footing (m)	Factored Geotechnical Resistance at ULS (kPa)	Geotechnical Reaction at SLS (kPa) (for 25 mm settlement)
Permanent Bridge - Abutments	Native sandy silt to silty sand	3	300	200

Since the ULS resistance and the settlement depend on the footing size and depth of embedment, the geotechnical resistances given in Table 2.3 should be reviewed if the selected footing width or founding elevations differ from those given in the table. Similarly, if an inclined load is applied instead of a vertical load, which is used in these calculations, the values given in Table 2.2 has to be reviewed to take into account those inclinations.

Prior to placing footings, the exposed native subgrade should be inspected according with OPSS 902. A Qualified Geotechnical Engineer should check that the design foundation elevation is achieved and all unsuitable soils including fill, organics and those soils with the USCS classification of CH, OH, MH, OL or PT have been removed. It should be also checked that the entire footing is placed on the competent foundation soil.

Resistance of Footing to Lateral Loads

Resistance to lateral forces/sliding resistance between the subgrade and concrete should be calculated in accordance with Section 6.7.5 of the CHBDC. The unfactored values of the coefficient of friction, tan δ, between the base of cast-in-place concrete footing and the granular subgrade soils below the frost level are presented in Table 2.4. A factor of 0.8 should be applied in calculation of the horizontal resistance in accordance with CHBDC.

Table 2.4 Recommendations for coefficient of friction

Interface	Coefficient of Friction, tan δ
Granular A and cast-in-place concrete	0.70
Sandy silt to silty sand and cast-in-place concrete	0.58

2.2.2.2 Temporary Bridge

Based on geotechnical data encountered in the current investigation, semi-integral abutments of the proposed temporary detour bridge can be founded on the native silty sand. As mentioned in Section 2.2.2.1 provision should be made for the construction of a levelling mat of lean concrete (i.e., mud slab) beneath the proposed founding level to fill in any voids that may be present on the surface of the native soils, and to provide an acceptable working surface. A Non-Standard Special Provision (NSSP) for the supply and installation of a working slab is provided in Appendix H of this report.

Spread footings which meet a requirement for an adequate protection against frost penetration in the project area of a minimum 1.6 m depth below the lowest surrounding area will be founded on the native silty sand.

Footing Elevation

Based on the results of the geotechnical investigation and a requirement for adequate protection against frost penetration in the project area (i.e. a minimum 1.6 m below the lowest surrounding area), the following founding elevations of spread footings are recommended:

Table 2.5 Recommendations for footing depth for the temporary bridge

Structure	Material at Founding Level	Foundation Elevation (m)	Foundation Depth Below Existing Grade
Temporary Bridge – South Abutment	Native gravelly sand	176.5/ 177.0*	1.1/0.6 m excavation for topsoil and gravelly sand
Temporary Bridge – North Abutment	Native silty sand		1.1/0.6 m excavation for topsoil and silty sand

*Note: * - If the proposed temporary bridge will be removed before the winter time, the minimum 1.6 m frost depth is not necessary to consider, higher founding elevation of spread footings are recommended.*

<u>Geotechnical Resistances</u>

In the context of the CHBDC, spread footings placed on the properly prepared subgrade at the design levels given in Table 2.5, should be designed based on the factored resistances at ULS and geotechnical reactions at SLS for 25 mm of settlement given in Table 2.6 below. The footing width of 3 m is assumed.

Table 2.6 Geotechnical resistance at ULS and geotechnical reaction at SLS for a 3 m wide footing for temporary bridge

Structure	Soil at Founding Level	Width of Footing (m)	Factored Geotechnical Resistance at ULS (kPa)	Geotechnical Reaction at SLS (kPa) (for 25 mm settlement)
Temporary Bridge – South Abutment	Native silty sand/ gravelly sand	3	300	200
Temporary Bridge – North Abutment	Native silty sand		225	150

Since the ULS resistance and the settlement depend on the footing size and depth of embedment, the geotechnical resistances given in Table 2.6 should be reviewed if the selected footing width or founding elevations differ from those given in the table. Similarly, if an inclined load is applied instead of a vertical load, which is used in these calculations, the values given in Table 2.5 has to be reviewed to take into account those inclinations.

Prior to placing footings, the exposed native subgrade should be inspected according with OPSS 902. A Qualified Geotechnical Engineer should check that the design foundation elevation is achieved and all unsuitable soils including fill, organics and those soils with the USCS classification of CH, OH, MH, OL or PT have been removed. It should be also checked that the entire footing is placed on the competent foundation soil.

Resistance of Footing to Lateral Loads

Resistance to lateral forces/sliding resistance between the subgrade and concrete should be calculated in accordance with Section 6.7.5 of the CHBDC. The unfactored values of the coefficient of friction, tan δ, between the base of cast-in-place concrete footing and the granular subgrade soils below the frost level are presented in Table 2.7. A factor of 0.8 should be applied in calculation of the horizontal resistance in accordance with CHBDC.

Table 2.7 Recommendations for coefficient of friction

Interface	**Coefficient of Friction, tan δ**
Granular A and cast-in-place concrete	0.70
Gravelly sand and cast-in-place concrete	0.70
Silty sand and cast-in-place concrete	0.55

2.2.3 Deep Foundations

2.2.3.1 General

Based on the subsurface conditions, deep foundation options have also been considered for the proposed permanent and temporary bridges. A summary of the advantages and disadvantages associated with each deep foundation option is provided below, and a comparison of the alternative foundation options based on advantages, disadvantages, relative costs, risks and consequences is provided in Table 2.1 in Section 2.2.1 of this report.

Driven steel H-piles

Steel H-pile foundations would be suitable for the construction of integral abutment. Steel H-piles driven through the sandy silt to silty sand soil to refusal on the underlying bedrock are feasible for support of the proposed permanent and temporary bridges. Piles driven to bedrock will provide high geotechnical resistances and minimize foundation settlement.

Driven steel pipe (tube) piles

Closed-ended steel pipe (tube) piles could also be considered as a deep foundation option to support the abutments of the proposed permanent and temporary bridges. Similar to steel H-piles, tube (pipe) piles will provide high geotechnical resistance and minimize foundation settlement. It is considered that pipe piles will have a higher risk than H-piles for ‘hanging up’ or being deflected away from their vertical or battered orientation if cobbles and/or boulders are encountered. However, since no significant cobbles and boulders were encountered at this site during the investigations, closed-ended steel tube piles could also be considered.

Drilled concrete caissons

Caissons founded on the bedrock to support the abutments are also feasible for this site. However, the use of drilled caissons is not considered to be practical or cost effective due to the anticipated difficulties and mitigation measures required for installation of drilled caissons at this site. Temporary or permanent liners would be required to mitigate the potential risks of ground loss from the water bearing cohesionless sand and silt layers during construction. Since the strength and sloping nature of the rock surface, establishing a seal between the liner and the bedrock could be problematic.

Based on the above considerations, the preferred option from a geotechnical/foundations perspective is to support the abutments for the proposed permanent and temporary bridges on driven steel H-piles to found the structure on the bedrock.

2.2.3.2 Steel H-Piles or Steel Pipe (Tube) Pile

2.2.3.2.1 Permanent Bridge

Considering the site specific conditions for the proposed permanent bridge, steel H-piles (HP 310 x 79 or HP 310 x 110) can be used to support a bridge designed with integral abutments. The piles for integral abutments should be in one row. The piles should be driven through 600 mm diameter and 3 m deep CSP pipe filled with uniformly graded sand (similar to Ottawa Sand). According to Table 1 in MTO Integral Abutment Bridges, the gradation for the uniformly graded sand shall be as follows:

Backfill to Internal Abutment – Augured Hole

MTO Sieve Designation	Percentage Passing Mass
2 mm (#10)	100%
600 µm (#30)	80% - 100%
425 µm (#40)	40% - 80%
250 µm (#60)	5% - 25%
150 µm (#100)	0% - 6%

Commercially available materials which meet the above gradation may be considered. The depth of such holes below the abutment shall be at least 3.0 m. Commercially available granular material which can be used for backfilling the annular space between the CSP pipe and the pile instead of Ottawa Sand.

Alternatively, closed-ended pipe (tube) piles (232 mm × 9.5 mm or 355 mm × 11 mm) are also considered and the factored geotechnical resistances are shown in Table 2.8.

The piles will be installed through the upper loose to very dense sandy deposits, and are expected to terminate on bedrock surface. Based on the depth to bedrock encountered in the deep boreholes drilled at the locations of the proposed permanent structure (Boreholes 14-1 and 14-3 at the south side, and Boreholes 14-2 and 14-4 at the north side, Boreholes 14-1 and 14-2 are approximate 10 m away from the proposed structure), 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 are estimated and given in Table 2.8. Based on GA drawings provided by MTO, the pile cap elevation would be at Elev. 175.2 m, which is below a frost depth of 1.6 m (approximate Elev. 177.4 m).

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 2.8. It is anticipated that for H-piles or pipe 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; as such, ULS conditions will govern for this foundation type.

Table 2.8. Factored geotechnical resistances for considered piles for the permanent bridge

Structure Element	Pile Founding Stratum	Estimated Tip Elevation (m)	Approx. Design Pile Length (m)	Factored Geotechnical Axial Resistance at ULS (kN/pile)				Geotechnical Axial Resistance at SLS (kN/pile)			
				HP 310 x 79	HP 310 × 110	323mm × 9.5 mm	355 mm × 11 mm	HP 310 x 79	HP 310 × 110	323mm × 9.5 mm	355 mm × 11 mm
South Abutment	Bedrock	~158.8	16.6	1,450	2,000	1,600	2,000	N/A			
North Abutment	Bedrock	~159.9	15.3	1,450	2,000	1,600	2,000				

Note: N/A-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.

2.2.3.2.2 Temporary Bridge

Based on the borehole information obtained at the site of the proposed temporary detour bridge, steel H-piles (HP 310 x 79 or HP 310 x 110) or closed-ended pipe (tube) piles (232 mm ×9.5 mm or 355 mm × 11 mm) are recommended for support this temporary detour bridge in this section if the shallow foundation is not sufficient.

The piles will be installed through the upper very loose to very dense sandy deposits, and are expected to terminate on bedrock surface. Based on the depth to bedrock encountered in the deep boreholes drilled at the location of the proposed temporary detour structure (Borehole 16-1 at the south side, and Borehole 16-2 at the north side), 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 are estimated and given in Table 2.10. It is anticipated that pile cap elevations would be below a frost depth of 1.6 m (approximate Elev. 176.5 m). Since no GA drawing for the proposed temporary bridge has been provided, the pile cap elevation is assumed at 174.3 m (lower than pile cap elevation of the proposed permanent bridge 0.86 m).

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 2.9. It is anticipated that for H-piles or pipe 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; as such, ULS conditions will govern for this foundation type.

Table 2.9. Factored geotechnical resistances for considered piles for the temporary bridge

Structure Element	**Pile Founding Stratum**	**Estimated Tip Elevation (m)**	**Approx. Design Pile Length (m)**	**Factored Geotechnical Axial Resistance at ULS (kN/pile)**				**Geotechnical Axial Resistance at SLS (kN/pile)**			
				HP 310 × 79	**HP 310 × 110**	**323mm × 9.5 mm**	**355 mm × 11 mm**	**HP 310 × 79**	**HP 310 × 110**	**323mm × 9.5 mm**	**355 mm × 11 mm**
South Abutment	Bedrock	~158.9	15.5	1,450	2,000	1,600	2,000	N/A			
North Abutment	Bedrock	~159.4	15.0	1,450	2,000	1,600	2,000				

Note: N/A-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.

Driven frictional steel piles also can be considered to support the proposed temporary bridge. Such piles, driven into the underlying very loose to compact silty sand over compact sandy silt can be designed using the factored (0.4) resistance values in the following Table 2.10. These values result from a static analysis based on skin friction with a nominal end bearing resistance, and using the effective stress β method. The elastic compression at ULS should be less than 10 mm in all cases. Since there is a minimal proposed grade raise and no significant soft cohesive materials, negative skin friction or drag loads are not a concern.

Table 2.10. Factored geotechnical resistances for frictional piles for the proposed temporary bridge

Pipe Size or HP Section	**Factored ULS (kN) for Embedment (below pile cap)**		
	10 m	**12 m**	**15 m**
HP310 × 110	170	200	260
HP310 × 79	120	150	180
323 mm × 9.5 mm	240	350	560
355 mm × 11 mm	270	390	620

Since the capacities of frictional steel pile are low, the frictional steel piles are not recommended at this site.

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:

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

where,

k_s=coefficient of horizontal subgrade reactions (MPa/m)
d=pile diameter (m)
n_h=constant of horizontal subgrade reaction (MPa/m)
z=depth below ground surface (m)

The recommended value of n_h is 5 MPa/m for very loose to very dense cohesionless soils encountered at this site.

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

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

Table 2.11. 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

<u>Negative Skin Friction (Downdrag Loads) on Piles</u>

Since there is no significant raise of the approach embankment and the foundation soil is cohesionless, the negative skin friction (or downdrag load) will not need to be taken into consideration during design of the piles supporting the integral abutment.

<u>Pile Installation</u>

Piles will be driven to bedrock and the installation procedure could be followed as specified in NSSP attached in Appendix H. In addition, the possibility of piles encountering cobbles and boulders in the soil layers in Borehole 16-2 at the north side of the river should be considered. Care must be taken to avoid overdriving and damaging the pile tip (i.e., the structural capacity of the piles should not be exceeded). Therefore, to minimize the risk of significant pile toe damage, a rock driving shoe is recommended.

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 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.

2.3 Approach Embankments

2.3.1 General

Based on the information provided to **exp** by the client, the proposed grade for the new permanent bridge and the proposed temporary bridge will be raised by approximately 0.5 m (Elev.179.0 m) and 1.0 m (Elev. 178.1 m) above existing grade, respectively. The proposed 2H:1V slopes of the embankment should be provided with adequate erosion protection against surface water runoff.

2.3.2 Stability Considerations

To assess the static and seismic slope stability of the forward slopes of the abutments and embankments of the proposed permanent bridge and temporary bridge, the SLOPE/W computer program developed by GeoSlope International Ltd. was employed for computation. Factors of safety were calculated using the Morgenstern-Price method for critical failure surfaces. The required minimum Factor of Safety (FOS) of 1.5 and 1.3 were adopted as the design criteria for abutments and embankments in static conditions, respectively. The minimum factor of safety of 1.1 was adopted for seismic conditions.

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 bridges.

In addition, a traffic surcharge pressure of 12 kPa was adopted in the slope stability assessments for the abutments and approach embankment.

2.3.2.1 Permanent Bridge

Tabulated below in Table 2.12 are the soil parameters used for the slope stability analyses for the permanent bridge. The soil parameters were generally estimated based on the results of field and laboratory investigation.

Table 2.12 Soil properties used in slope stability analyses for the proposed permanent bridge

Material Type	**Effective Stress Parameters**		
	ϕ' (degrees)	**c' (kPa)**	**γ (kN/m^3)**
Engineered fill	32	0	21
Sandy Silt to Silty Sand	32	0	21

Table 2.13 Summary of results of slope stability analyses for the proposed permanent bridge

Locations	**Max Height (m)**	**Conditions**	**Min FOS**
South Abutment (Global stability)	~3.5	Drained long-term conditions, static condition	1.6
		Drained long-term conditions, seismic condition	1.5
South Approach Embankment (Side slopes 2H:1V)	~2.0	Drained long-term conditions, static condition	1.4
		Drained long-term conditions, seismic condition	1.3

The results of the slope stability analyses for the proposed permanent bridge using shallow foundation option are presented on Figures F1 to F4 in Appendix F and summarized in Table 2.13.

As can be seen the calculated minimum factors of safety of critical slip surfaces meet the design criteria for static and seismic conditions given above. Therefore, based on these results, the proposed abutment/embankments can safely be constructed with 2H:1V side slopes.

2.3.2.2 Temporary Bridge

Tabulated below in Table 2.14 are the soil parameters used for the slope stability analyses for the embankment of the proposed temporary bridge. The soil parameters were generally estimated based on the results of field and laboratory investigation.

Table 2.14 Soil properties used in slope stability analyses for the proposed temporary bridge

Material Type	Effective Stress Parameters		
	ϕ' (degrees)	**c' (kPa)**	**γ (kN/m^3)**
Engineered fill	32	0	21
Silty Sand	29	0	20
Sandy Silt	31	0	21
Silty Sand	32	0	21

Table 2.15 Summary of results of slope stability analyses for the proposed temporary bridge

Locations	Max Height (m)	Conditions	Min FOS
North Approach Embankment (Side slopes 2H:1V)	~1.0	Drained long-term conditions, static condition	1.3
		Drained long-term conditions, seismic condition	1.2

The results of the slope stability analyses for the proposed temporary bridge using shallow foundation option are presented on Figures F5 to F6 in Appendix F and summarized in Table 2.15. As can be seen the calculated minimum factors of safety of critical slip surfaces meet the design criteria for static and seismic conditions given above. Therefore, based on these results, the proposed embankments can safely be constructed with 2H:1V side slopes.

Minimum 2H:1V forward slope with scour and erosion protection should be safe for the temporary bridge. However, global stability shall be checked again when GA drawing is available.

Suitable erosion and scour protection measures should also be provided to the river banks adjacent to the bridges. The requirement for design of erosion/scour protection should be determined by the hydraulic design engineer. Such measures may include appropriate sized rip-rap underlain by suitable granular filter or schemes involving sheeting. This should be reviewed by environmental and hydraulic specialists. The slope stability analyses presented were performed assuming that both protections are appropriately designed using some proper filter system between large rocks and original ground by a hydraulic engineer.

2.3.3 Settlement Considerations

For both the proposed permanent and temporary bridges, since the approach embankments are not going to be raised significantly, settlement of the structure should not exceed 25 mm for footings designs in accordance with this indicated in this report. If high capacities are required for the temporary bridge, consideration can be given to according for higher settlement and jacking provision.

2.3.4 Seismic and Liquefaction Potential Consideration

Seismic characterization of the site should be compliant with Canadian Highway Bridge Design Code (CHBDC, CAN/CSA-S6-14). Table 4.1 in CHBDC (see Clause 4.4.3.2) shows site classification for seismic site response based on soil average properties in top 30 m. At the site, the subsoil generally consists of sandy silt to silty sand. The groundwater level is at about 1.0 m to 2.7 m depth below existing grade. The reported N-values for the soil below the founding level ranged from 0 to over 98 blows for 300 mm of penetration, with an average value being greater than 15 blows and lower than 50 blows per 300 mm of penetration within the drilled depth. Based on these soil characteristics, the site class for this site is estimated to be Class “D” according to Table 4.1. However, these parameters should be reviewed by the Structural Engineer.

According to the observed soil properties, the subsoil in some areas (i.e. BH 16-2) could potentially be susceptible to liquefaction. Accordingly, liquefaction analyses have been performed using the Seed’s approach, which is recommended by the CFEM (4th Edition 2006; Chapter 6, pg.101). This approach defines a factor of safety against liquefaction as the ratio of the induced cyclic stress ratio over the cyclic resistance ratio. The calculated factor of safety for the site subsoil in vicinity of BH 16-2 is generally more than 1.0. As a result, liquefaction is not likely to occur in the upper soils at the project site for the earthquake having 2% probability of exceedance in a 50-year period.

2.4 Other Considerations

2.4.1 Lateral Earth Pressure on Structures

2.4.1.1 Static Earth Pressure

The abutment stems, retaining wall and temporary roadway protection, if any, 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 CHBDC.

The expression for calculating lateral earth pressure is given by:

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

where

P = earth pressure intensity at depth h, kPa

K = earth pressure coefficient
γ = unit weight of retained soil, kN/m^3
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 2.16 can be used:

Table 2.16. 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)**
Granular A	35	0.27	3.69	0.43	22.8
Granular B	32	0.31	3.25	0.47	21.2
Native Sandy Silt to Silty Sand	32	0.31	3.25	0.47	21
Native Silty Sand	30	0.33	3.00	0.50	20

2.4.1.2 Seismic Earth Pressure

2.4.1.2.1 Yielding Walls

Seismic loading should be taken into account in the design in accordance with Section 4.6.5 of the CHBDC. These estimates are based on the Mononobe-Okabe (M-O) pseudo-static method of analysis. The M-O method produces seismic loads that are more critical than the static loads that act prior to an earthquake.

In accordance with Section 4.4.3.2 and 4.4.6 of the CHBDC and NBC (2010), the PGA of 0.040g for the Site Class ‘D’, earthquake having a 2% probability of exceedance in 50 years (0.000404 per annum) can be used in the calculation of the seismic active pressure coefficient. The minimum peak vertical acceleration is taken as two/thirds of the peak horizontal acceleration in accordance with Section 4.4.3.6 of the CHBDC.

It should be noted that in the computation of seismic earth pressure coefficients, the wall back-face geometry, backfill slope and wall friction effects need to be addressed.

For dry cohesionless backfill, the total active and passive thrusts can be expressed using the following equations:

$P_{AE} = 1/2 K_{AE}\, \gamma\, H^2(1-k_v)$

$P_{PE} = 1/2 K_{PE}\, \gamma\, H^2(1-k_v)$

Where

K_{AE} = active earth pressure coefficient (combined static and seismic, equation 6.26 in CFEM)

K_{PE} = passive earth pressure coefficient (combined static and seismic, equation 6.33 in CFEM)

H = height of wall

k_h = horizontal acceleration coefficient

k_v = vertical acceleration coefficient

γ = total unit weight

The following design parameter were used to develop the recommended K_{AE} and K_{PE} values.

Zonal Acceleration Ratio, A or PGA	0.040
Horizontal Acceleration Coefficient, k_h	0.020
Vertical Acceleration Coefficient, k_v	0.133
Horizontal back slope to wall	0˚
Vertical back of wall	0˚
The angle of friction of wall-backfill interface	0˚

For design purposes, the following unfactored seismic lateral earth pressure parameters can be used:

Table 2.17. Material types and earth pressure properties under seismic conditions for yielding walls

Material	**Unfactored Friction Angle** **ϕ’ (º)**	**Coefficient of Seismic Earth Pressure - Active** **(K_{AE})**	**Coefficient of Seismic Earth Pressure - Passive** **(K_{PE})**	**Unit Weight** **γ kN/m^3)**
Compacted Granular A or Granular B Type II	35	0.35	4.45 (ULS)	22.8
Compacted Granular B Type I	32	0.38	3.79 (ULS)	22

2.4.1.2.2 Non-yielding Walls

When the wall movements are insufficient to mobilize the shear strength of the backfill soil, the following question can be used to calculate additional seismic earth pressure.

$$\Delta P_e = \alpha_k \, . \, K_h \, \gamma \, h^2$$

Where

α_k = 1.02 for rigid base

α_k = 1.17 for non-rigid base

K_h = ß (PGA)

ß = ½ for essential structure

ß = 1/3 for other structure

Thrust Point =0.45 H

For retaining walls component subject to passive resistance that is less than 1.5 m in height, the passive resisting force shall be computed using static passive forces. Static passive forces for wall heights or foundation thickness less than 1.5 m shall be used because it is anticipated that the inertial effects from earthquake loadings will be small.

For design purposes, the following unfactored seismic lateral earth pressure parameters can be used:

Table 2.18. Material types and earth pressure properties under seismic conditions for non-yielding walls

Material	**Unfactored Friction Angle** **ϕ’ (º)**	**Coefficient of Seismic Earth Pressure - Active** **(K_{AE})**	**Coefficient of Seismic Earth Pressure - Passive** **(K_{PE})**	**Unit Weight** **γ kN/m^3)**
Compacted Granular A or Granular B Type II	35	0.37	4.35 (ULS)	22.8
Compacted Granular B Type I	32	0.46	4.20 (ULS)	22

2.4.2 Earthquake Considerations

Seismic loading may result in increased lateral pressure acting on the abutment stems. Seismic characterization of the site must be compliant with CHBDC (CAN/CSA-S6-14). From the Natural Resources Canada website, 2015 NBCC seismic hazard values are obtained using the site location coordinates (45.6768 N, 82.2704 W) and the damped reference spectral accelerations for the project site are Sa(0.2)=0.071g, Sa(0.5)=0.055g, Sa(1.0)=0.036g, Sa(2.0)=0.019g and the reference peak ground acceleration (PGA) is 0.040g (g=acceleration due to gravity -9.81 m/s^2). These values are associated with an earthquake having 2 percent probability of exceedance in a 50-year period.

2.4.3 Corrosion Protection

Two soil samples were submitted to Maxxam Analytics Inc., a CALA-certified and accredited laboratory in Mississauga, Ontario, for analyses of pH, water soluble sulphate, chloride concentrations, resistivity, conductivity and oxidation-reduction potential. The testing was completed to determine the potential for degradation of the concrete in the presence of soluble sulphate and the potential for corrosion of exposed steel used in foundations and buried infrastructure. The analysis results are summarized in Table 1.2.

The data in Table 1.2 indicates low to medium resistivity. Accordingly, buried metallic pipes and appurtenances would be susceptible to corrosion, unless protected; therefore, cathodic protection should be provided. The chloride content is <20 ppm µg/g) i.e. <0.002% which indicates a low potential for additional corrosion. The soil pH was about 7.7 (average) which is within what is considered the normal range for soil pH of 5.0 to 9.0. Therefore, the pH levels of the tested soils do not indicate a highly corrosive environment. The test results in Table 1.2 may be used to aid in the selection of coatings and corrosion protection systems for buried steel objects.

The water soluble sulphate content of the soils tested ranged from 28 to 32 ppm (µg/g), which is less than 0.10%, does not require sulphate resistant cement. Normal Type 10 Portland cement can be used. These data also support our local experience.

2.4.4 Scour Protection

The scour design is the responsibility of a qualified hydraulic engineer. 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.

Structures which contain spread footings founded on highly erodible/scourable soils (sand, silt, or fine gravel) are very vulnerable to failure caused by scour and undermining and should not be used without appropriate protection. Spread footings can be protected against structural undermining by locating the foundations at an appropriate depth by providing scour protection blankets or by using sheet piling. Sheet piling used for this purpose should have sufficient stiffness and strength to maintain the bearing capacity of the soil within and on the outside of the sheet piling. Spread footings close to creeks, channels or rivers are very likely to be exposed to stream flow. Bank protection is therefore required and must be remain effective for the design life of the bridge.

Geotechnical soil parameters necessary for the scour analyses are: 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). These parameters are determined based on the soils encountered at the site, and are presented on the borehole logs attached in Appendix C and the graphs included in Appendix D. All tested soils were classified using the Unified Soil Classification System which can be used for evaluation of erosion rates.

2.4.5 Excavation

All excavations must be carried out in accordance with the latest edition of the Ontario Occupational Health and Safety (OHSA) and good construction practice. The native soils which should be excavated for construction of the abutments (i.e. loose to dense silty sand fill, compact gravelly sand and very loose to compact sandy silt to silty sand) 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.

2.4.6 Temporary Shoring System

According to the separation distances between the new structures, temporary shoring system may be required to permit the proposed staged construction of the temporary bridge and the permanent bridge. This support system, if any, should be designed and constructed in accordance with OPSS.PROV 539. The lateral movement of the temporary shoring system should meet Performance Level 2 as specified in OPSS.PROV 539. For design parameters, please refer to Table 2.17.

The selection of design of the protection system will be the responsibility of the Contractor. However, the following recommendations about costing and assessment of temporary excavation and protection system options for this site are provided.

The occasional cobbles present within silty sand at the north approach embankment area may pose obstructions to the installation of temporary shoring system. If enough space is available, sufficient separation between the structures could be provided to allow for open cut excavations.

Based on the subsurface soil and groundwater conditions, a solider pile and timber lagging system may be required for the temporary excavation support at this site. The soldier piles should be socketed to sufficient depth in order to provide the necessary passive resistance for the retained sol height. It is considered that pre-drilling and the use of temporary liners would be required for the soldier pile installation. Due to the displacement of occasional cobbles, the necessary measures may be implemented to mitigate the potential for ground movements behind the support system. Seepage control or include mitigation measures for loss of soil particles through the lagging boards may also be required.

Lateral support to the soldier piles could be provided in the form of rakers or temporary anchors.

2.4.7 Dewatering

If the shallow foundation option is chosen, it is usually recommended that the bottom of excavation would be terminated no lower than approximately 0.5 m above the groundwater to minimize disturbance and permit compaction of the exposed surface. However, since the groundwater level is very high at this site, if that option is chosen, a significant groundwater inflow could be expected. Therefore, the dewatering system is recommended.

Surface runoff should be diverted from excavations.

The design of dewatering systems for the excavations is responsibility of the Contractor who is expected to retain dewatering specialists for this task (OPSS 518, November 2011).

2.4.8 Frost Protection

According to Ontario Provincial Standard Drawing (OPSD – 3090.101), the frost depth in the subject site is about 1.6 m. Consequently, all footings exposed to seasonal freezing conditions should be protected from frost action by at least 1.6 m of soil cover or equivalent insulation.

2.4.9 Vibration Monitoring During Pile Driving

The proposed bridge construction involves construction of a temporary detour bridge while the existing main bridge remains in service, and then construction of the replacement of the main bridge following completion of the temporary detour bridge. If is recommended that vibration levels within tolerable ranges for the portions of the bridge in service at the time, or for any temporary modular structure if used at the site.

A maximum peak particle velocity of 50 mm/sec is recommended at the existing abutments (OPSS.PROV 120, November 2014). The piles furthest from the existing structure should be driven first, in order to check the vibration level at the existing structure and, if necessary, alter the installation procedures for the remaining piles.

2.4.10 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’ 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 1.0meter 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.

3 CLOSURE

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

This Foundation Investigation and Design Report has been prepared by Ms. Jia He, M.Eng., EIT. and Mr. Nimesh Tamrakar, M.Eng,, EIT. and reviewed by Mr. T.C. Kim, M.E.Sc., P.Eng. and Mr. S.E. Gonsalves, M.Eng., P.Eng. designated MTO foundation contact. The field investigation was conducted by Mr. Nimesh Tamrakar, M.Eng., EIT.

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

Yours truly,

exp Services Inc.

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Technical Specialist

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

Encl.

4 LIMITATIONS AND USE OF REPORT

BASIS OF REPORT

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

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

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

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

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

RELIANCE ON INFORMATION PROVIDED

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

STANDARD OF CARE

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

COMPLETE REPORT

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

USE OF REPORT

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

REPORT FORMAT

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

Appendix A – Photographs

Photo 1: Looking east on the west side of Mindemoya River Bridge

Photo 2: Looking south on the west side of Mindemoya River Bridge

Photo 3: Looking north on the east side of Mindemoya River Bridge

Photo 4: Looking north on Hwy 551

Photo 5: Looking south on Hwy 551

Photo 6: Looking east on the existing bridge

Photo 7: Looking west on the existing bridge

Appendix B – Drawings

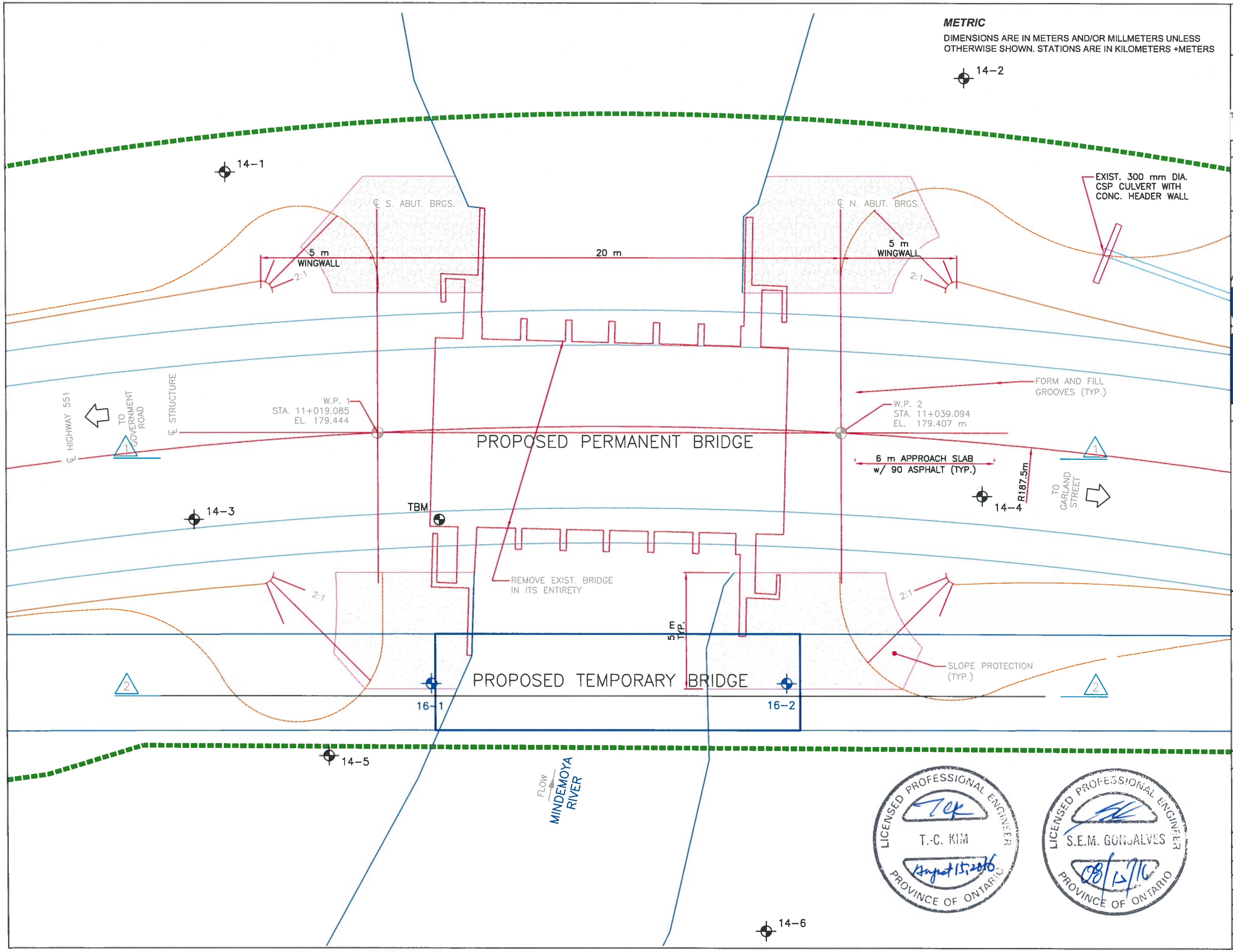
AGREEMENT NO. 5015-E-0007
ASSIGNMENT NO. 2
GWP NO. 5153-12-00

MINEMOYA RIVER BRIDGE REPLACEMENT
(HWY 551, PROVIDENCE BAY, ONTARIO)
BORE HOLE LOCATIONS

SHEET
1

exp Services Inc.

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



LICENSED PROFESSIONAL ENGINEER
T.-C. KIM
August 15, 2016
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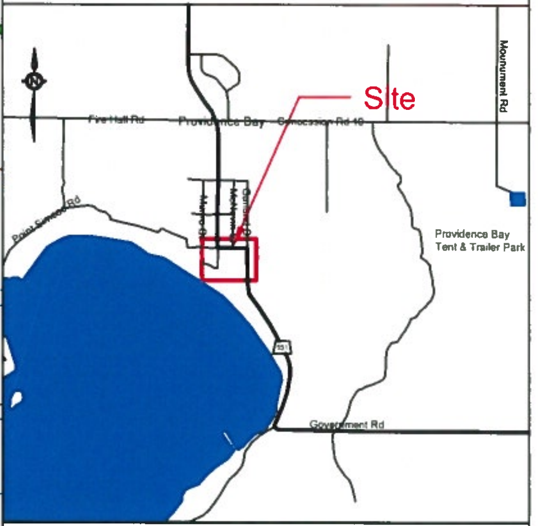
LICENSED PROFESSIONAL ENGINEER
S.E.M. GONSALVES
08/15/16
PROVINCE OF ONTARIO

KEY PLAN



LEGEND

⊕	Approx.Previous Investigated Borehole Locations
⊕	Approx. Current Investigated Borehole Locations
◐	TBM (Temporary Bench Mark, Elevation 179.0 m)

BH No.	APPROX. ELEV.	MTM CO-ORDINATES	
		NORTH	EAST
16-1	177.6	5058574	322993
16-2	177.6	5058569	322982
14-1	178.5	5058548	322974
14-2	177.0	5058578	322961
14-3	178.9	5058551	322989
14-4	179.2	5058584	322978
14-5	177.4	5058560	322997
14-6	176.9	5058580	322999

NOTE

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

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

DATE	BY	DESCRIPTION
15/08/2016	TK	FINAL SUBMISSION
15/07/2016	TK	SUBMISSION FOR REVIEW

		GEOCRES NO 41G-23	
SCALE	1:125	PROJECT NO. ALL-00233185-A0	
SUBM'D SM	CHECKED TK	DATE 2016 07 15	SITE No.
DRAWN JH	CHECKED SG	APPROVED SG	DWG 01

AGREEMENT NO. 5015-E-0007
ASSIGNMENT NO. 2
GWP NO. 5153-12-00

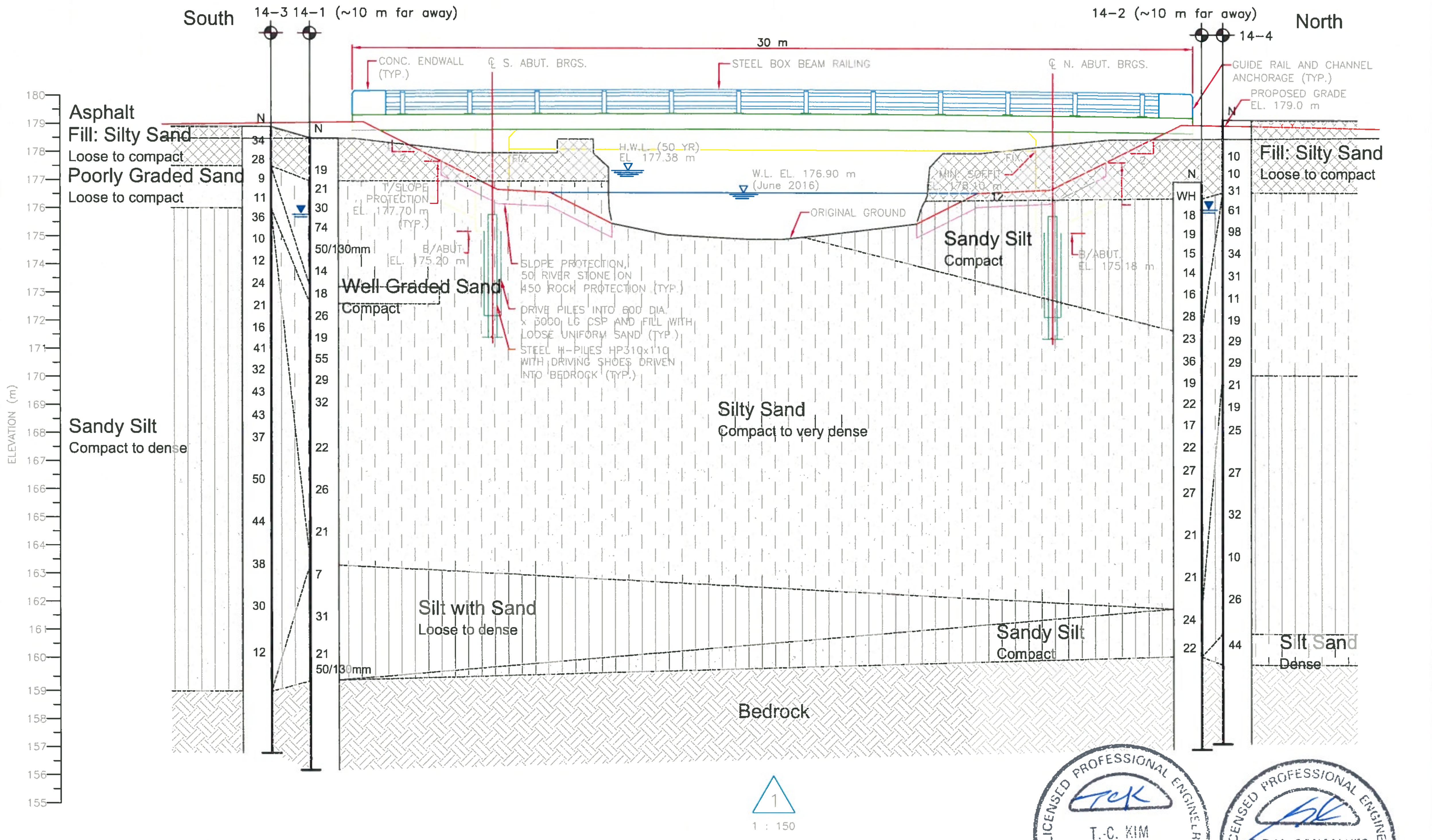
MINEMOYA RIVER BRIDGE REPLACEMENT
(HWY 551, PROVIDENCE BAY, ONTARIO)

PERMANENT BRIDGE - SOIL STRATA

SHEET
1

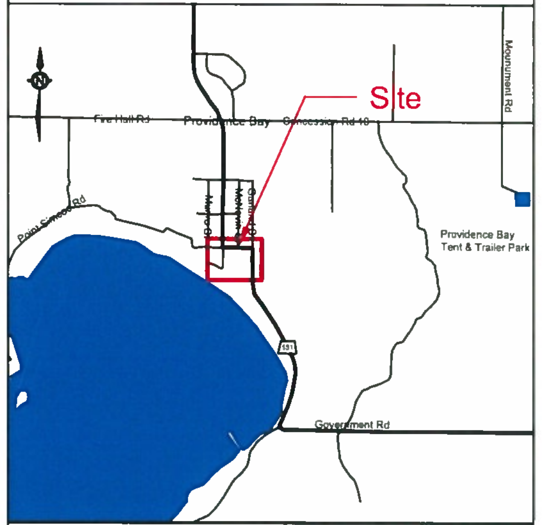
exp Services Inc.

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



PROFESSIONAL ENGINEER
T.C. KIM
August 15, 2016
PROFESSIONAL ENGINEER
S.E.M. GONSALVES
08/15/16

KEY PLAN



LEGEND

⊕	Approx.Previous Investigated Borehole Locations
⊕	Approx. Current Investigated Borehole Locations
N	Standard Penetration Test (Blows/0.3 m)
▼	Groundwater Level Measured in Piezometer

SOIL STRATA SYMBOLS

▨	FILL	▥	SANDY SILT
☐	SAND	▥	SILTY SAND
▥	SILT WITH SAND	▨	BEDROCK

BH No.	APPROX. ELEV.	MTM CO-ORDINATES	
		NORTH	EAST
14-1	178.5	5058548	322974
14-2	177.0	5058578	322961
14-3	178.9	5058551	322989
14-4	179.2	5058584	322978

NOTE

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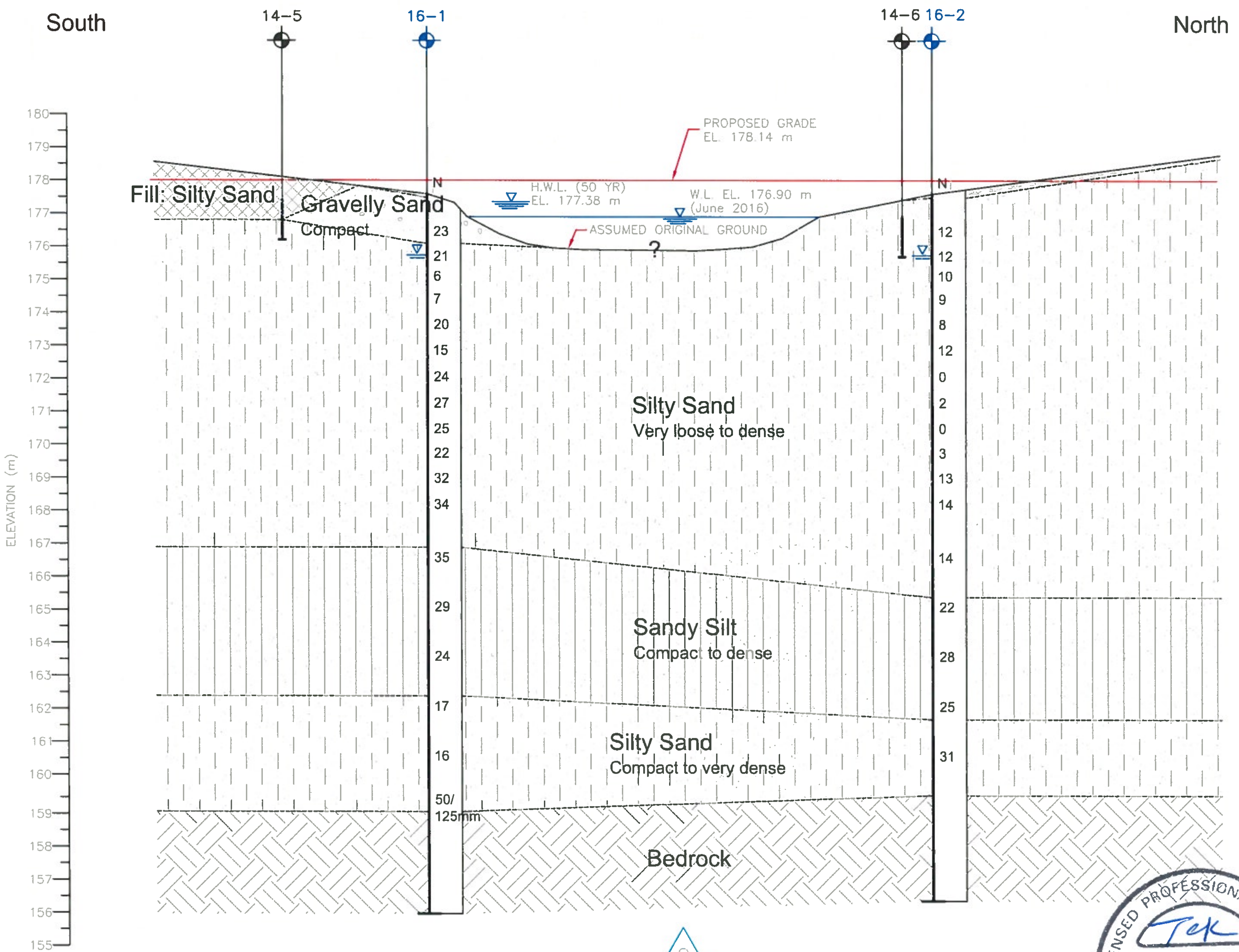
15/08/2016	TK	FINAL SUBMISSION
15/07/2016	TK	SUBMISSION FOR REVIEW
DATE	BY	DESCRIPTION

		GEOCRES NO. 41G-23	
SCALE	1:150	PROJECT NO. ALL-00233185-A0	
SUBM'D TK	CHECKED TK	DATE 2016 08 15	SITE No.
DRAWN JH	CHECKED SG	APPROVED SG	DWG. 02

METRIC

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



2

1 : 150

T.-C. KIM

August 15, 2016

S.E.M. GONSALVES

08/15/16

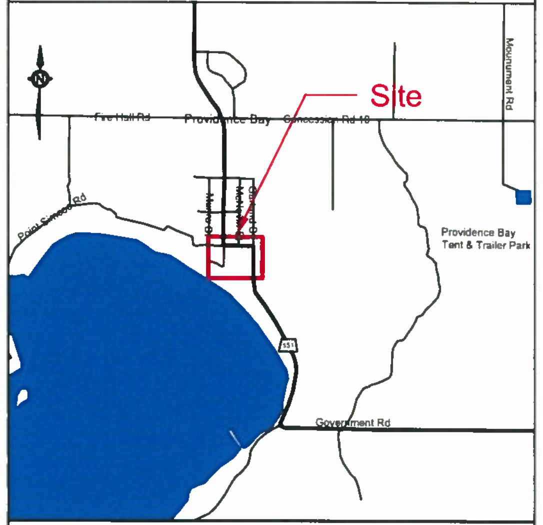
AGREEMENT NO. 5015-E-0007
ASSIGNMENT NO. 2
GWP NO. 5153-12-00

MINEMOYA RIVER BRIDGE REPLACEMENT
(HWY 551, PROVIDENCE BAY, ONTARIO)
TEMPORARY BRIDGE - SOIL STRATA

SHEET
1

exp Services Inc.

KEY PLAN



LEGEND

⊕	Approx.Previous Investigated Borehole Locations
⊕	Approx. Current Investigated Borehole Locations
N	Standard Penetration Test (Blows/0.3 m)
▽	Groundwater Level Measured in the Open Hole

SOIL STRATA SYMBOLS

▨	FILL	▨	TOPSOIL
▨	SILTY SAND	▨	SANDY SILT
☐	GRAVELLY SAND	▨	BEDROCK

BH No.	APPROX. ELEV.	MTM CO-ORDINATES	
		NORTH	EAST
16-1	177.6	5058574	322993
16-2	177.6	5058569	322982
14-5	177.4	5058560	322997
14-6	176.9	5058580	322999

NOTE

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

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DATE	BY	DESCRIPTION
15/08/2016	TK	FINAL SUBMISSION
15/07/2016	TK	SUBMISSION FOR REVIEW

		GEOCRES NO. 41G-23	
SCALE	1:150	PROJECT NO. ALL-00233185-A0	
SUBM'D TK	CHECKED TK	DATE 2016.07.15	SITE No.
DRAWN JH	CHECKED SG	APPROVED SG	DWG. 03

Appendix C – Boreholes Logs

Explanation of Terms Used on Borehole Records

SOIL DESCRIPTION

Terminology describing common soil genesis:

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

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

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

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

Terminology describing soil structure:

Desiccated:	having visible signs of weathering by oxidization of clay minerals, shrinkage cracks, etc.
Stratified:	alternating layers of varying material or color with the layers greater than 6 mm thick.
Laminated:	alternating layers of varying material or color with the layers less than 6 mm thick.
Fissured:	material breaks along plane of fracture.
Varved:	composed of regular alternating layers of silt and clay.
Slickensided:	fracture planes appear polished or glossy, sometimes striated.
Blocky:	cohesive soil that can be broken down into small angular lumps which resist further breakdown.

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

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

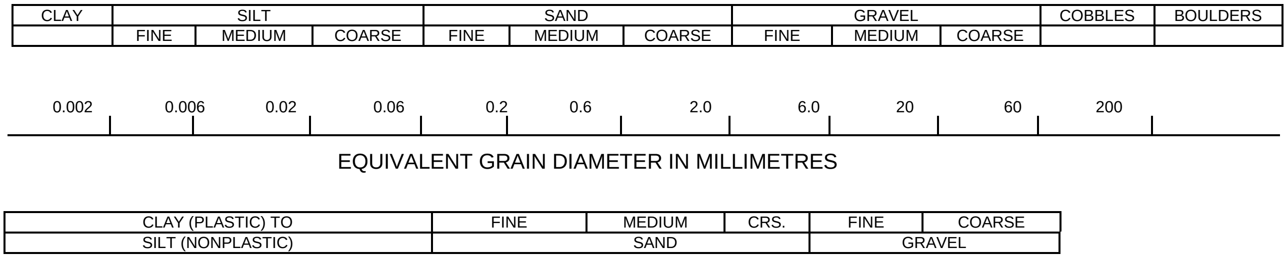
Homogeneous: same color and appearance throughout.

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

Uniformly Graded: predominantly on grain size.

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

ISSMFE SOIL CLASSIFICATION



CLAY	SILT			SAND			GRAVEL			COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE		

CLAY (PLASTIC) TO SILT (NONPLASTIC)	FINE	MEDIUM	CRS.	FINE	COARSE
	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≤Pp≤10%
Little	15≤Pp≤25%
Some	30≤Pp≤45%
Mostly	50≤Pp≤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≤N<10
Compact	10≤N<30
Dense	30≤N<50
Very Dense	50≤N

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

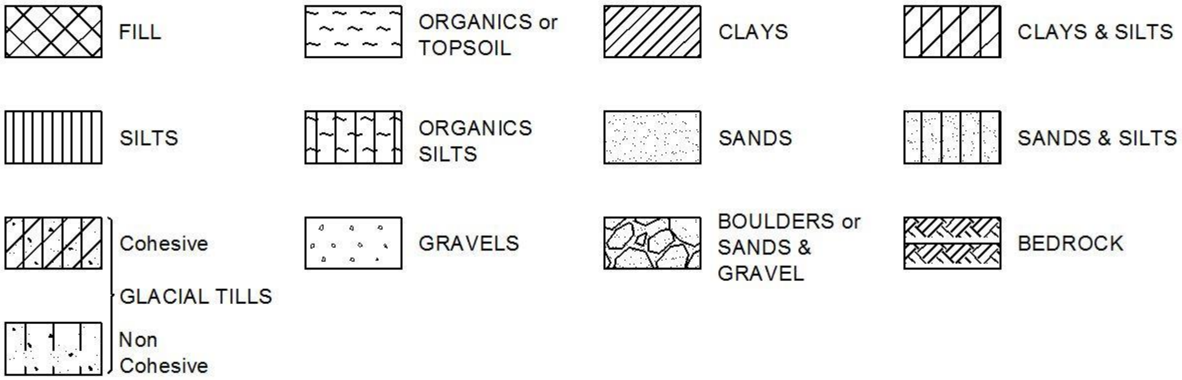
Table c: Consistency of Cohesive Soil

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

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

STRATA PLOT

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



WATER LEVEL MEASUREMENT

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

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

STRESS AND STRAIN

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

MECHANICALL PROPERIES 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

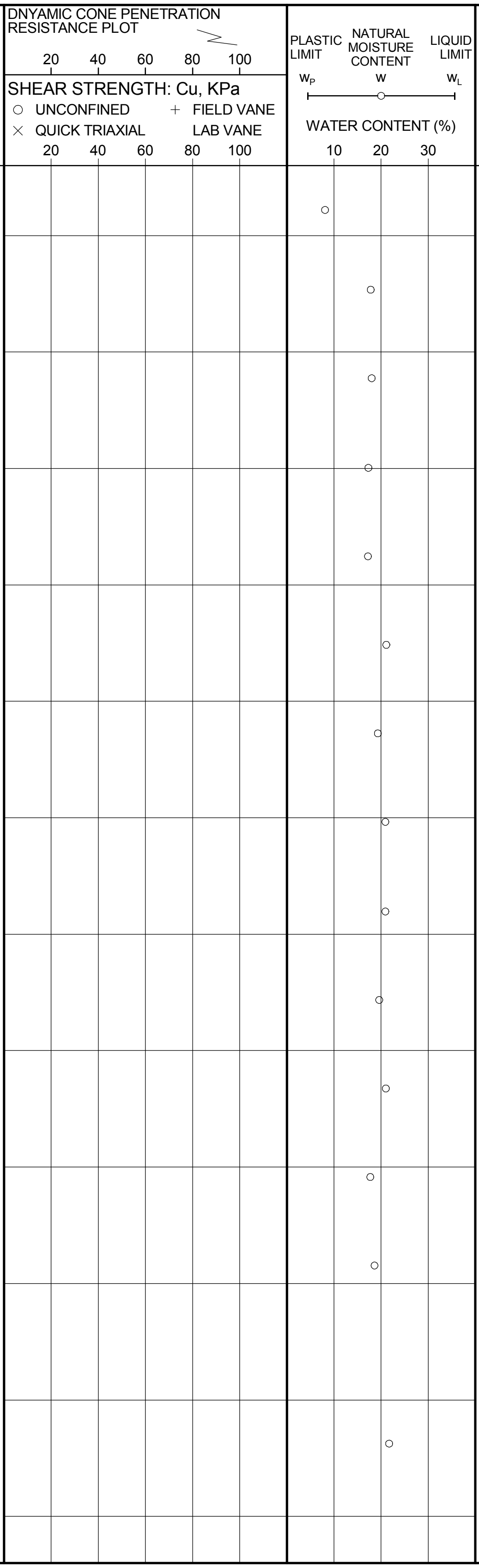
PHYSICAL PROPERTIES OF SOIL

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 BH 16-1 1 OF 2 METRIC

W. P. ADM-00233185-B0
LOCATION Highway 551, Providence Bay, Ontario, MTM Z11 (N5058574 E322993)
ORIGINATED BY N.T.
DIST Manitoulin
BOREHOLE TYPE CME 55 Track, Hollow stem auger/ Diamond Drill, Cased Hole
COMPILED BY S.A.
DATUM Geodetic
DATE 2016/06/21 - 2016/06/22
CHECKED BY S.M.

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES			GR SA SI CL
177.6							
177.5 0.1	**TOPSOIL** 100 mm thickness, grass and roots, some silt, blackish brown, moist **GRAVELLY SAND (SW)** trace silt, trace clay, brownish grey, wet, compact	1	AUGER			177	
		2	SS	23			wet spoon 35 56 (9)
176.1 1.5	**SILTY SAND (SM)** trace organics, some gravel, trace clay, grey, wet, loose to dense	3	SS	21	∇	176	
	-sand blow up, dilatant	4	SS	6		175	0 65 (34)
	-clayey silt seam	5	SS	7		174	
	-becoming trace clayey silt seam	6	SS	20			0 83 14 3
		7	SS	15		173	
		8	SS	24		172	
		9	SS	27		171	0 83 (17)
		10	SS	25		170	
		11	SS	22			
	-dense below 8.4 m	12	SS	32		169	0 62 (38)
		13	SS	34		168	
						167	
166.9 10.7	**SANDY SILT (ML)** trace to some clay, grey, wet, compact to dense	14	SS	35			
						166	



UNIT WEIGHT γ kN/m³

Continued Next Page

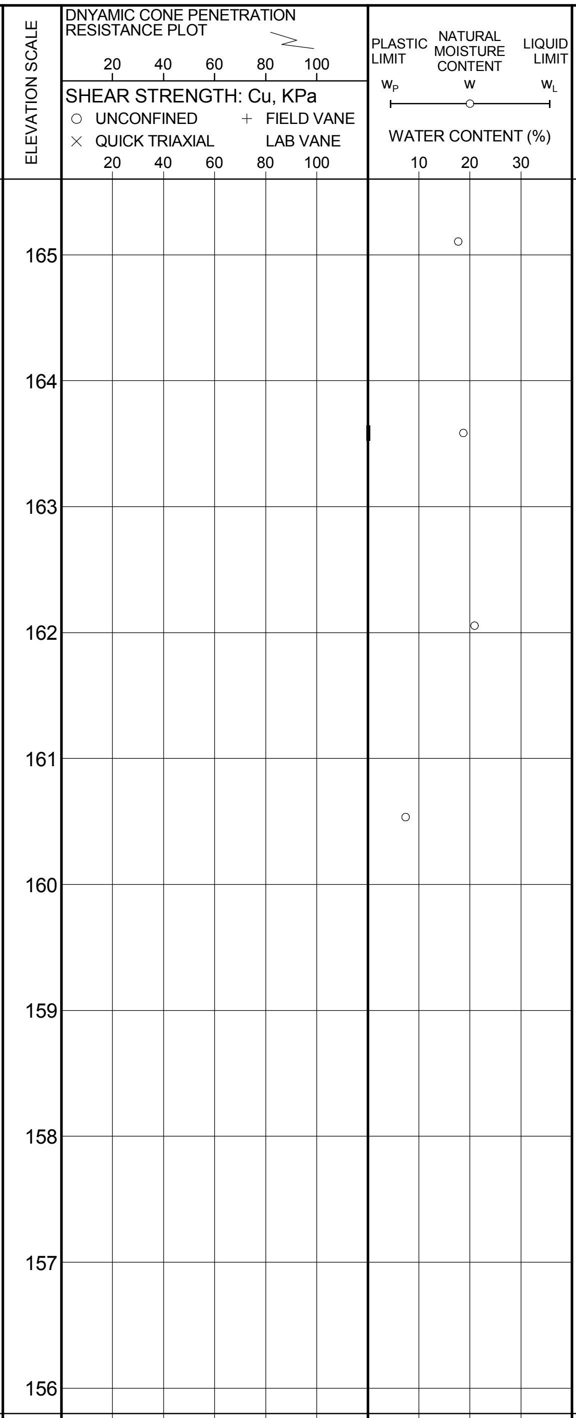
+ 3, × 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No BH 16-1

2 OF 2 **METRIC**

W. P. ADM-00233185-B0
LOCATION Highway 551, Providence Bay, Ontario, MTM Z11 (N5058574 E322993)
ORIGINATED BY N.T.
DIST Manitoulin
BOREHOLE TYPE CME 55 Track, Hollow stem auger/ Diamond Drill, Cased Hole
COMPILED BY S.A.
DATUM Geodetic
DATE 2016/06/21 - 2016/06/22
CHECKED BY S.M.

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SANDY SILT (ML) trace to some clay, grey, wet, compact to dense *(continued)*	15	SS	29			
	-150 mm clayey silt interbedded between sandy silt and 100 mm sand at bottom of spoon	16	SS	24			0 22 (78) non-plastic
162.4 15.2	**SILTY SAND (SM)** trace clay, grey, wet, compact to very dense	17	SS	17			0 94 (6)
	-becoming more gravel	18	SS	16			
		19	SS	50/ 125mm			
158.9 18.7	**BEDROCK** smooth, fine grained, light grey limestone with dolomatic seams, fair to good quality, moderately to slightly weathered, intensively to moderately fractured, close to very close joint opening **NQ Coring** Lenght (m) RQD (%) Run 1 1.6 57.5 Run 2 1.5 77.5	20	NQ				
		21	NQ				
155.8 21.8	**END OF BOREHOLE**						



NOTES:
1. This drawing is to be read with the subject report and project numbers as presented above.
2. Hole caved at 1.83 m depth upon completion.
3. Groundwater level at 1.83 m depth upon completion.

$+^3, \times^3$: Numbers refer to Sensitivity ○ $^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No BH 16-2

1 OF 2 **METRIC**

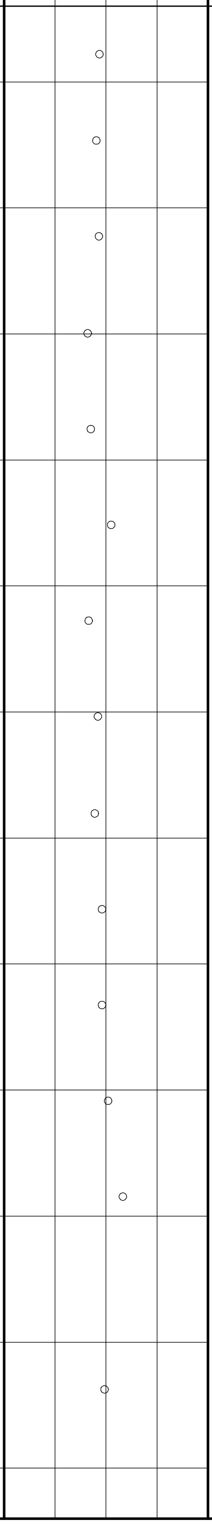
W. P. ADM-00233185-B0
LOCATION Highway 551, Providence Bay, Ontario, MTM Z11 (N5058569 E322982)
ORIGINATED BY N.T.
DIST Manitoulin
BOREHOLE TYPE CME 55 Track, Hollow stem auger/ Diamond Drill, Cased Hole
COMPILED BY S.A.
DATUM Geodetic
DATE 2016/06/20 - 2016/06/21
CHECKED BY S.M.

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100
SHEAR STRENGTH: Cu, KPa
○ UNCONFINED + FIELD VANE
× QUICK TRIAXIAL LAB VANE
20 40 60 80 100

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L
WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ kN/m³

ELEV DEPTH	DESCRIPTION (SOIL PROFILE)	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	GR	SA	SI	CL
177.6										
177.5 0.1	**TOPSOIL** 100 mm thickness, grass and roots, trace rootlets, trace gravel, fine sand, some silt, blackish brown, wet	1	AUGER				0	83	(17)	
	SILTY SAND (SM-CL) trace organics, occasional cobbles, trace gravel, trace clay, blackish grey to grey, wet, very loose to compact					177				
		2	SS	12						
						176				
		3	SS	12	∇					
		4	SS	10		175	0	47	51	2
		5	SS	9						
						174				
		6	SS	8			0	74	(26)	
	-becoming trace clayey silt seam					173				
		7	SS	12						
	-very loose below 5.3 m									
		8	SS	0		172				
		9	SS	2						
						171				
		10	SS	0			0	80	(20)	
						170				
		11	SS	3						
	-compact below 8.4 m									
		12	SS	13		169				
		13	SS	14			0	92	(8)	
						168				
						167				
		14	SS	14						
						166				



Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity
$\circ^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No BH 16-2

2 OF 2

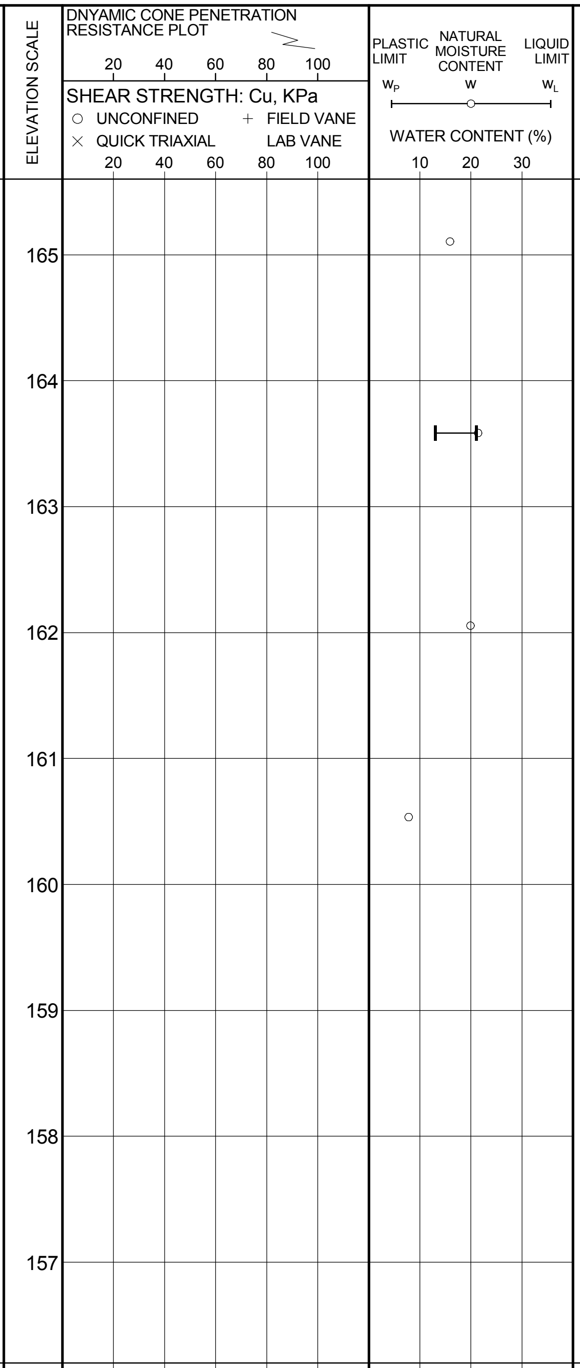
METRIC

W. P. ADM-00233185-B0
LOCATION Highway 551, Providence Bay, Ontario, MTM Z11 (N5058569 E322982)
ORIGINATED BY N.T.
DIST Manitoulin
BOREHOLE TYPE CME 55 Track, Hollow stem auger/ Diamond Drill, Cased Hole
COMPILED BY S.A.
DATUM Geodetic
DATE 2016/06/20 - 2016/06/21
CHECKED BY S.M.

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
165.4								
12.2	**SANDY SILT (ML-CL)** trace organics, occasional cobbles, trace gravel, trace clay, blackish grey to grey, wet, compact to dense	15	SS	22				
	- 150 mm sandy silt interbedded between 300 mm and 150 mm clayey silt	16	SS	28	0	5	70	25
		17	SS	25				
161.8 15.9	**SILTY SAND (SM)** some gravel, trace clay, grey, wet, dense	18	SS	31				
159.4 18.2	**BEDROCK** smooth, fine grained, light grey limestone with dolomatic seams, fair to good quality, moderately to slightly weathered, intensively to moderately fractured, close to very close joint spacing (refer to Explanation Sheet to Core Log) **NQ Coring**	19	NQ					
		20	NQ					
156.2 21.4	**END OF BOREHOLE**							

	Lenght (m)	RQD (%)
Run 1	1.6	84.6
Run 2	1.6	88.3

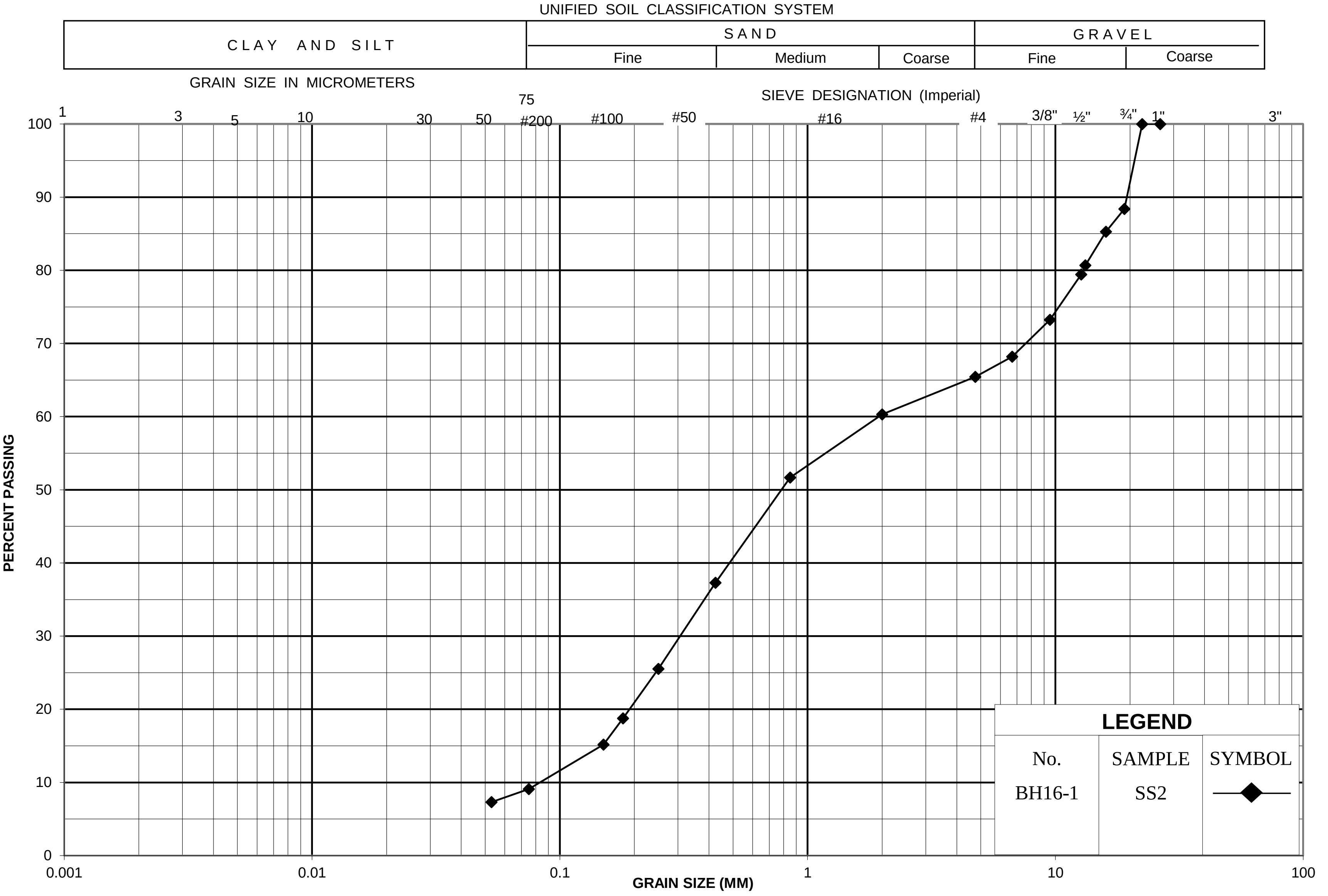
NOTES:
1. This drawing is to be read with the subject report and project numbers as presented above.
2. Hole caved at 1.83 m depth upon completion.
3. Groundwater level at 1.83 m depth upon completion.



UNIT WEIGHT γ kN/m^3

$+^3, \times^3$: Numbers refer to Sensitivity ○ $^{3\%}$ STRAIN AT FAILURE

Appendix D –
Laboratory Test Results



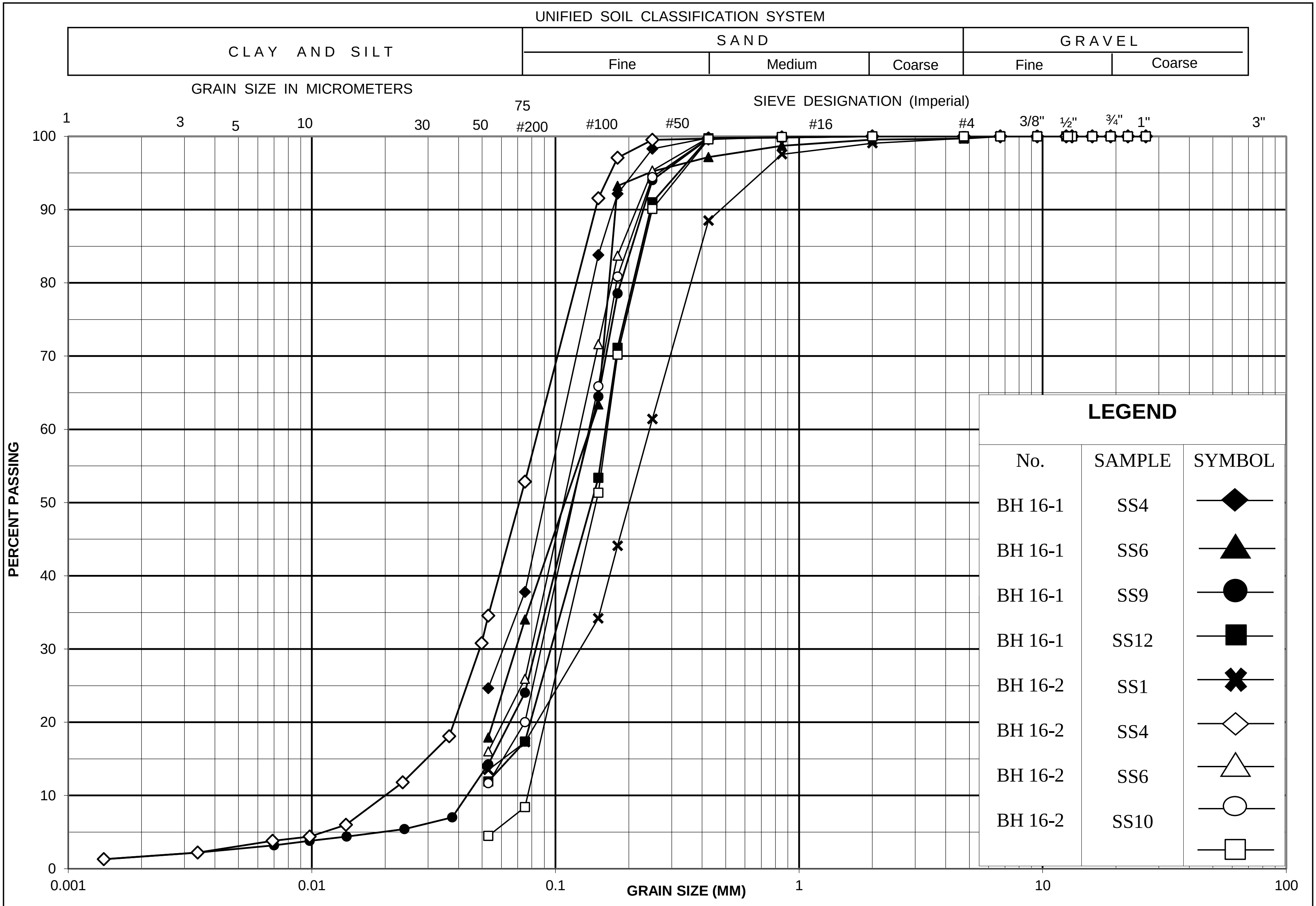
GRAIN SIZE DISTRIBUTION

Gravelly Sand (SW)

FIGURE 1

GWP NO. 5153-12-00

DATE July 2016



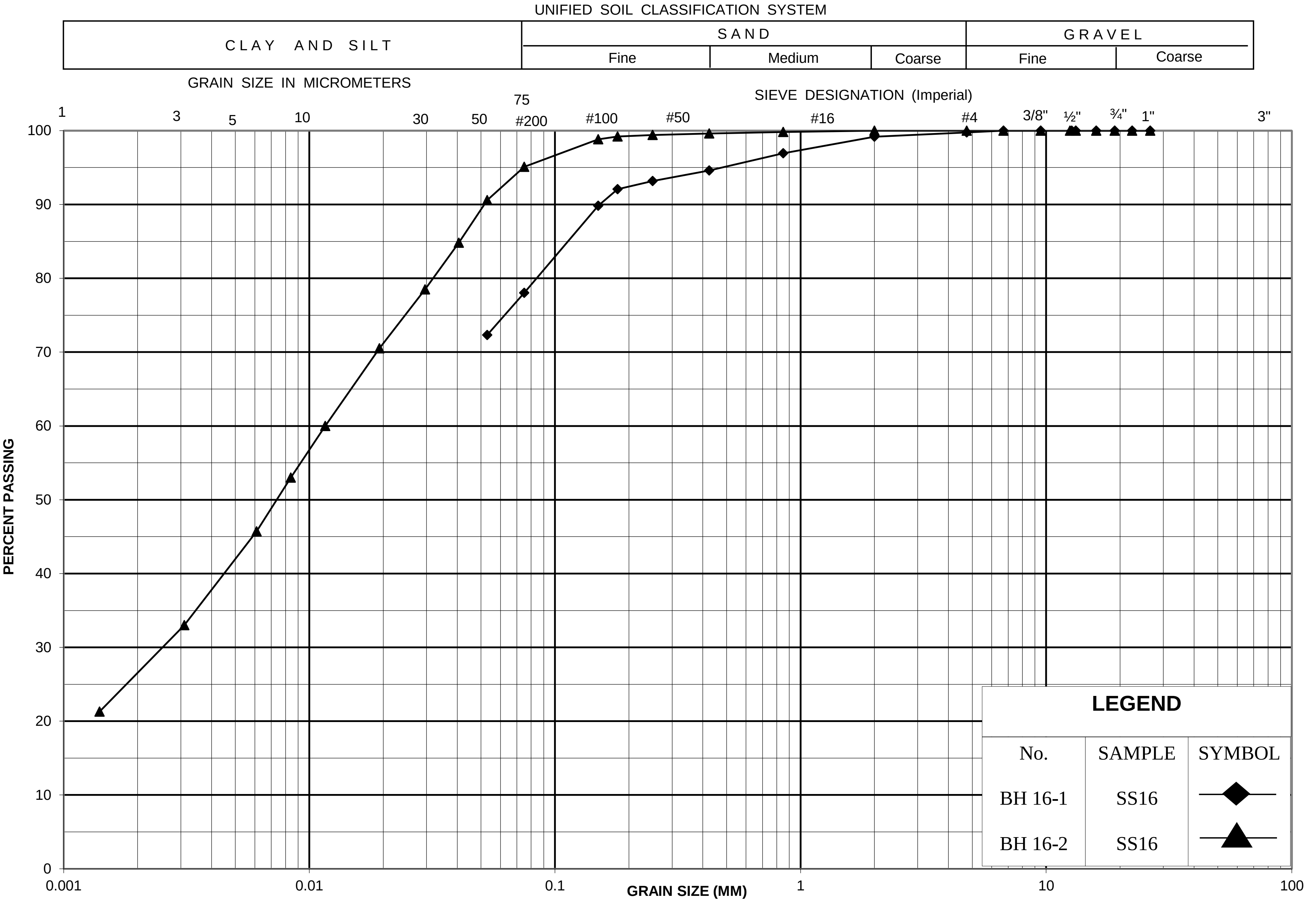
GRAIN SIZE DISTRIBUTION

Upper Silty Sand (SM-CL)

FIGURE 2

GWP NO. 5153-12-00

DATE July 2016



exp.

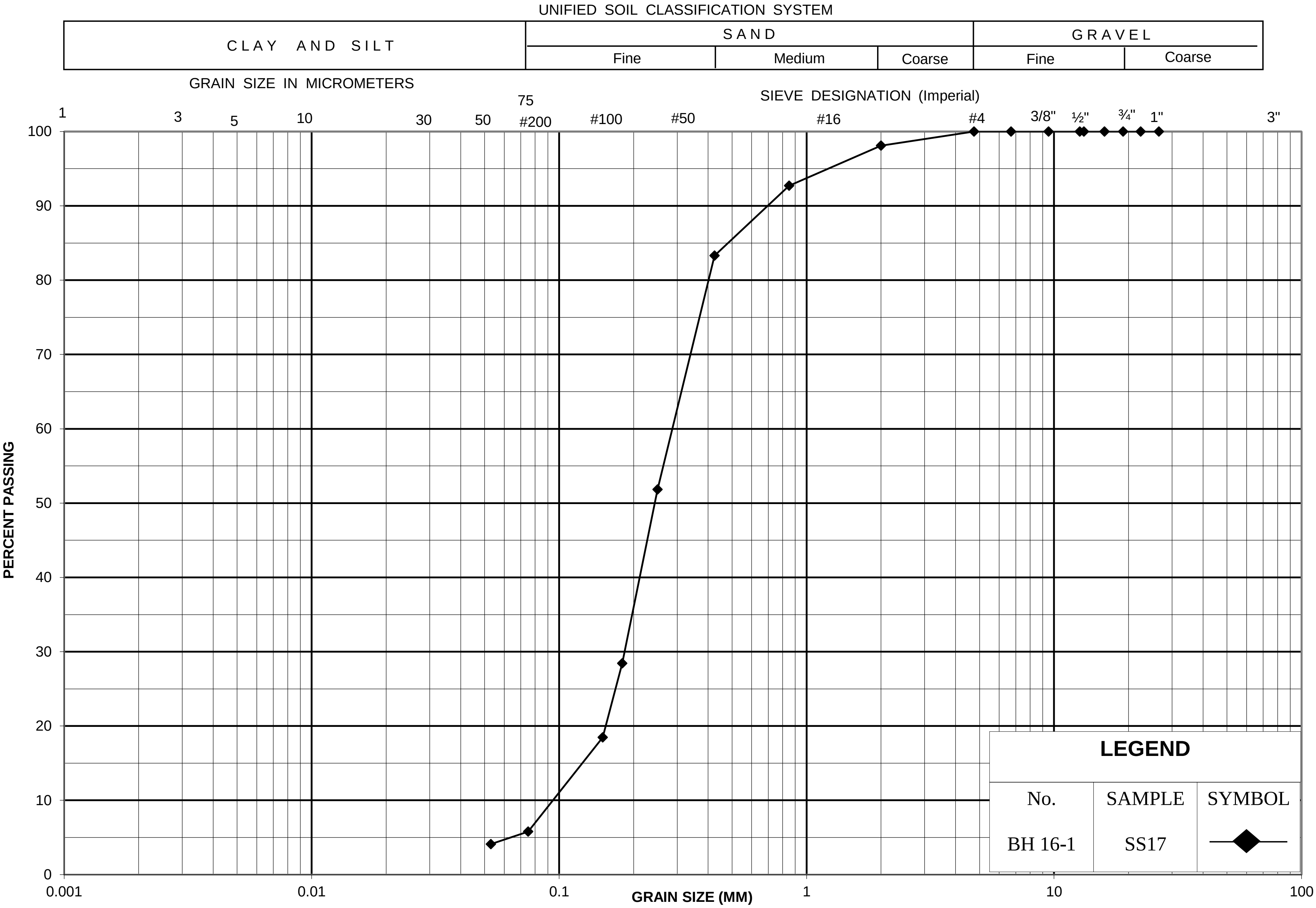
GRAIN SIZE DISTRIBUTION

Sandy Silt (ML)

FIGURE 3

GWP NO. 5153-12-00

DATE July 2016



exp.

GRAIN SIZE DISTRIBUTION

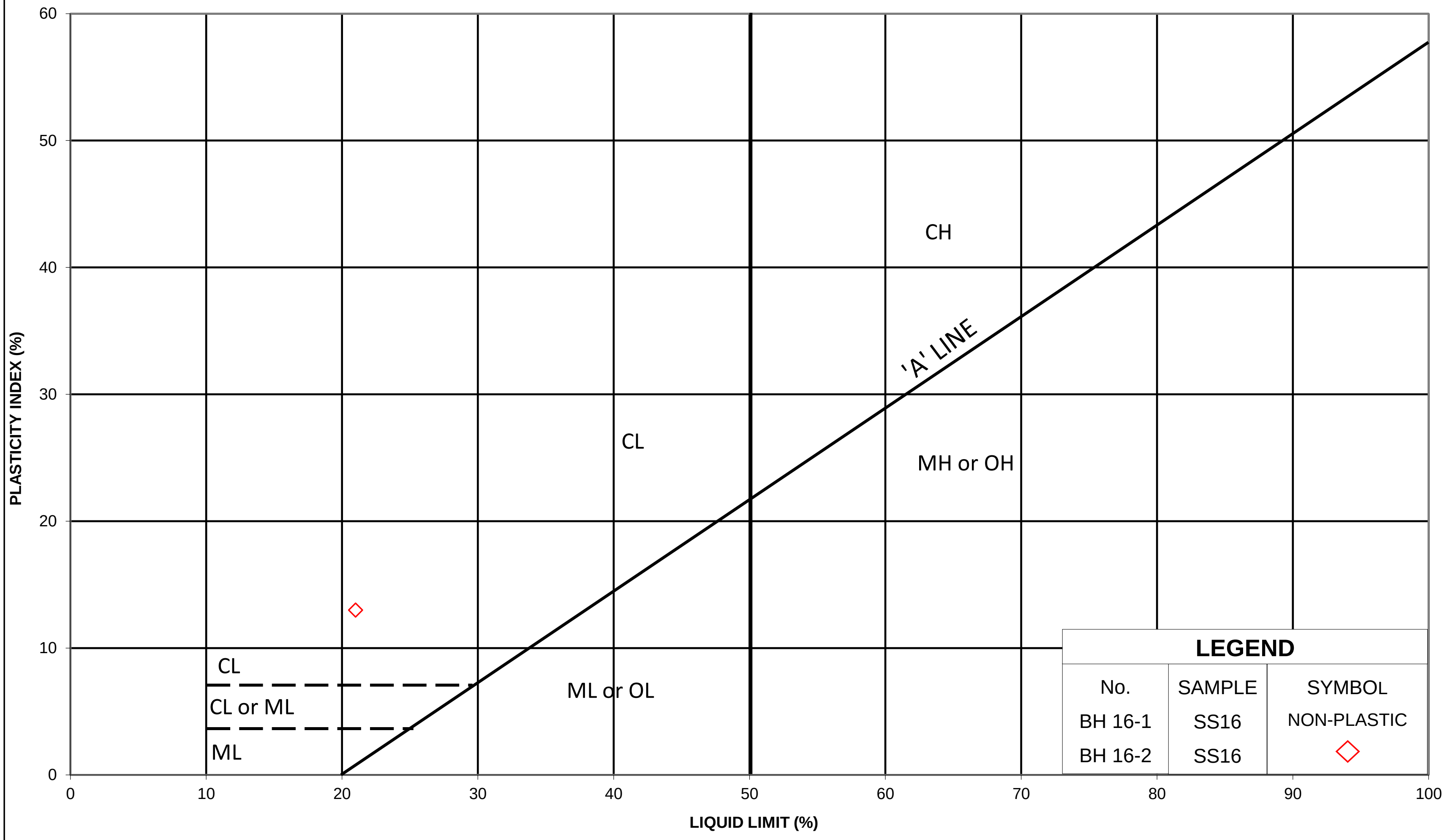
Lower Silty Sand (SM)

FIGURE 4

GWP NO. 5153-12-00

DATE July 2016

Mindemoya River Bridge No. 4, Hwy 551



PLASTICITY CHART

SANDY SILT (ML-CL)

FIGURE 5

GWP NO. 5153-12-00

DATE July 2016

Maxxam
A Bureau Veritas Group Company

Your P.O. #: BRM-GEO
Your Project #: ADM-00233185-B0
Site Location: HWY 551, PROVIDENCE BAY
Your C.O.C. #: 65436

Attention:Nimesh Tamrakar

exp Services Inc
1595 Clark Blvd
Brampton, ON
L6T 4V1

Report Date: 2016/07/07
Report #: R4055446
Version: 1 - Final

CERTIFICATE OF ANALYSIS

MAXXAM JOB #: B6C9783
Received: 2016/06/23, 12:35

Sample Matrix: Soil
Samples Received: 2

Analyses	Quantity	Date Extracted	Date Analyzed	Laboratory Method	Reference
Chloride (20:1 extract)	1	N/A	2016/06/27	CAM SOP-00463	EPA 325.2 m
Chloride (20:1 extract)	1	N/A	2016/06/28	CAM SOP-00463	EPA 325.2 m
Conductivity	2	N/A	2016/06/27	CAM SOP-00414	OMOE E3138 v2 m
pH CaCl2 EXTRACT	2	2016/06/27	2016/06/27	CAM SOP-00413	EPA 9045 D m
Resistivity of Soil	2	2016/06/23	2016/06/27	CAM SOP-00414	SM 22 2510 m
Sulphate (20:1 Extract)	2	N/A	2016/06/27	CAM SOP-00464	EPA 375.4 m
Oxidation-Reduction Potential (1, 2)	2	2016/06/24	2016/07/07	SLA SOP-00101	In house

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.

(1) This test was performed by Maxxam Sladeview Petrochemical
(2) Oxidation-Reduction Potential (ORP) values are determined using a Ag/AgCl reference electrode.

Encryption Key

Please direct all questions regarding this Certificate of Analysis to your Project Manager.
Sara Singh, B.Sc, Senior Project Manager
Email: sarasingh@maxxam.ca
Phone# (905)817-5730
==
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.

Total Cover Pages : 1

SOIL CORROSIVITY PACKAGE (SOIL)

Maxxam ID		CPA280	CPA280		CPA281	CPA281		
Sampling Date		2016/06/21 11:30	2016/06/21 11:30		2016/06/20 11:00	2016/06/20 11:00		
COC Number		65436	65436		65436	65436		
	UNITS	**BH16-1 (SS5)**	**BH16-1 (SS5) Lab-Dup**	**QC Batch**	**BH16-2 (SS3)**	**BH16-2 (SS3) Lab-Dup**	**RDL**	**QC Batch**
Calculated Parameters								
Resistivity	ohm-cm	9400		4552877	8400			4552877
Inorganics								
Soluble (20:1) Chloride (Cl)	ug/g	<20		4555735	<20	<20	20	4557233
Conductivity	umho/cm	107		4555834	120	118	2	4555834
Available (CaCl2) pH	pH	7.65		4556067	7.72			4556067
Soluble (20:1) Sulphate (SO4)	ug/g	29	32	4555745	28		20	4555745
Subcontracted Analysis								
Oxidation-Reduction Potential	mV	+158		4554472	+162			4554472

RDL = Reportable Detection Limit
QC Batch = Quality Control Batch
Lab-Dup = Laboratory Initiated Duplicate

TEST SUMMARY

Maxxam ID: CPA280
Sample ID: BH16-1 (SS5)
Matrix: Soil

Collected: 2016/06/21
Shipped:
Received: 2016/06/23

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	4555735	N/A	2016/06/27	Deonarine Ramnarine
Conductivity	AT	4555834	N/A	2016/06/27	Neil Dassanayake
pH CaCl2 EXTRACT	AT	4556067	2016/06/27	2016/06/27	Neil Dassanayake
Resistivity of Soil		4552877	2016/06/27	2016/06/27	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	4555745	N/A	2016/06/27	Alina Dobreanu
Oxidation-Reduction Potential	PH	4554472	2016/06/24	2016/07/07	Grace Sison

Maxxam ID: CPA280 Dup
Sample ID: BH16-1 (SS5)
Matrix: Soil

Collected: 2016/06/21
Shipped:
Received: 2016/06/23

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Sulphate (20:1 Extract)	KONE/EC	4555745	N/A	2016/06/27	Alina Dobreanu

Maxxam ID: CPA281
Sample ID: BH16-2 (SS3)
Matrix: Soil

Collected: 2016/06/20
Shipped:
Received: 2016/06/23

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	4557233	N/A	2016/06/28	Deonarine Ramnarine
Conductivity	AT	4555834	N/A	2016/06/27	Neil Dassanayake
pH CaCl2 EXTRACT	AT	4556067	2016/06/27	2016/06/27	Neil Dassanayake
Resistivity of Soil		4552877	2016/06/27	2016/06/27	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	4555745	N/A	2016/06/27	Alina Dobreanu
Oxidation-Reduction Potential	PH	4554472	2016/06/24	2016/07/07	Grace Sison

Maxxam ID: CPA281 Dup
Sample ID: BH16-2 (SS3)
Matrix: Soil

Collected: 2016/06/20
Shipped:
Received: 2016/06/23

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	4557233	N/A	2016/06/28	Deonarine Ramnarine
Conductivity	AT	4555834	N/A	2016/06/27	Neil Dassanayake

GENERAL COMMENTS

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

Package 1	5.0°C

Results relate only to the items tested.

QUALITY ASSURANCE REPORT

			Matrix Spike		SPIKED BLANK		Method Blank		RPD		QC Standard	
QC Batch	Parameter	Date	% Recovery	QC Limits	% Recovery	QC Limits	Value	UNITS	Value (%)	QC Limits	% Recovery	QC Limits
4554472	Oxidation-Reduction Potential	2016/06/28					+140	mV			+247	238 - 248
4555735	Soluble (20:1) Chloride (Cl)	2016/06/27	NC	70 - 130	105	70 - 130	<20	ug/g	NC	35		
4555745	Soluble (20:1) Sulphate (SO4)	2016/06/27	NC	70 - 130	100	70 - 130	<20	ug/g	NC	35		
4555834	Conductivity	2016/06/27			101	90 - 110	<2	umho/cm	0.94	10		
4556067	Available (CaCl2) pH	2016/06/27			98	97 - 103			0.34	N/A		
4557233	Soluble (20:1) Chloride (Cl)	2016/06/28	109	70 - 130	101	70 - 130	<20	ug/g	NC	35		

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.

QC Standard: A sample of known concentration prepared by an external agency under stringent conditions. Used as an independent check of method accuracy.

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).

FUNDAMENTAL LABORATORY ACCEPTANCE GUIDELINE

Invoice To:
exp Services Inc
ATTN: Central Services
1595 Clark Blvd
Brampton, ON
L6T 4V1
Client Contact:
Nimesh Tamrakar

Maxxam Job #:	B6C9783
Date Received:	2016/06/23
Your C.O.C. #:	65436
Your Project #:	ADM-00233185-B0
Your P.O. #:	BRM-GEO
Maxxam Project Manager:	Sara Singh
Quote #:	B46066

No discrepancies noted.

Report Comments

Received Date:	**2016/06/23**	**Time:**	**12:35**	**By:**	
Inspected Date:		**Time:**		**By:**	
FLAG Created Date:		**Time:**		**By:**	

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

ASSOCIATION OF THE CHEMICAL PROFESSION OF ONTARIO
CHARTERED
Grace M. Sison
CHEMIST

Grace Sison, B.Sc., C.Chem, Senior Project Manager - Petroleum Division

---

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.

Your Project #: MB6C9783
Site Location: AD19-00233185-B0
Your C.O.C. #: 08423476

Attention:SUB CONTRACTOR

MAXXAM ANALYTICS
CAMPOBELLO
6740 CAMPOBELLO ROAD
MISSISSAUGA, ON
CANADA L5N 2L8

Report Date: 2016/06/29
Report #: R2208460
Version: 1 - Final

CERTIFICATE OF ANALYSIS

MAXXAM JOB #: B651722
Received: 2016/06/27, 10:20

Sample Matrix: Soil
Samples Received: 2

Analyses	Quantity	Date Extracted	Date Analyzed	Laboratory Method	Analytical Method
Moisture	2	2016/06/27	2016/06/28	BBY8SOP-00017	BC MOE Lab Manual
Sulfide (AVS) (soil)	2	2016/06/27	2016/06/29	BBY6SOP-00006	SM 22 4500 S2- D m

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.
Gail Pedersen, Project Manager –Environmental Customer Service
Email: gpedersen@maxxam.ca
Phone# (604) 734 7276
==
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Total Cover Pages : 1

RESULTS OF CHEMICAL ANALYSES OF SOIL

Maxxam ID		OX5026	OX5026	OX5027		
Sampling Date		2016/06/21 11:30	2016/06/21 11:30	2016/06/20 11:00		
COC Number		08423476	08423476	08423476		
	UNITS	**BH16-1 (SS5) (CPA280-02)**	**BH16-1 (SS5) (CPA280-02) Lab-Dup**	**BH16-2 (SS3) (CPA281-02)**	**RDL**	**QC Batch**
MISCELLANEOUS						
Sulphide	ug/g	0.76 (1)	0.87	1.20	0.50	8312329

RDL = Reportable Detection Limit

Lab-Dup = Laboratory Initiated Duplicate

(1) Matrix spike exceeds acceptance limits due to matrix interference. Re-analysis yields similar results.

PHYSICAL TESTING (SOIL)

Maxxam ID		OX5026	OX5027		
Sampling Date		2016/06/21 11:30	2016/06/20 11:00		
COC Number		08423476	08423476		
	UNITS	**BH16-1 (SS5) (CPA280-02)**	**BH16-2 (SS3) (CPA281-02)**	**RDL**	**QC Batch**
Physical Properties					
Moisture	%	14	14	0.30	8311638
RDL = Reportable Detection Limit					

TEST SUMMARY

Maxxam ID: OX5026
Sample ID: BH16-1 (SS5) (CPA280-02)
Matrix: Soil

Collected: 2016/06/21
Shipped:
Received: 2016/06/27

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Moisture	BAL/BAL	8311638	2016/06/27	2016/06/28	Lolita Obusan
Sulfide (AVS) (soil)	SPEC/COL	8312329	2016/06/27	2016/06/29	Prabhleen Sodhi

Maxxam ID: OX5026 Dup
Sample ID: BH16-1 (SS5) (CPA280-02)
Matrix: Soil

Collected: 2016/06/21
Shipped:
Received: 2016/06/27

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Sulfide (AVS) (soil)	SPEC/COL	8312329	2016/06/27	2016/06/29	Prabhleen Sodhi

Maxxam ID: OX5027
Sample ID: BH16-2 (SS3) (CPA281-02)
Matrix: Soil

Collected: 2016/06/20
Shipped:
Received: 2016/06/27

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Moisture	BAL/BAL	8311638	2016/06/27	2016/06/28	Lolita Obusan
Sulfide (AVS) (soil)	SPEC/COL	8312329	2016/06/27	2016/06/29	Prabhleen Sodhi

GENERAL COMMENTS

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

Package 1	18.3°C

Results relate only to the items tested.

QUALITY ASSURANCE REPORT

			Matrix Spike		Spiked Blank		Method Blank		RPD	
QC Batch	Parameter	Date	% Recovery	QC Limits	% Recovery	QC Limits	Value	UNITS	Value (%)	QC Limits
8311638	Moisture	2016/06/28					<0.30	%	3.1	20
8312329	Sulphide	2016/06/29	4.6 (1)	75 - 125	90	75 - 125	<0.50	ug/g	NC	30

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 (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).

(1) Recovery or RPD for this parameter is outside control limits. The overall quality control for this analysis meets acceptability criteria.

VALIDATION SIGNATURE PAGE

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

Rob Reinert, B.Sc., Scientific Specialist

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.

Appendix E –
Bedrock Core Photographs

Project NO: ADM-00233185-B0
BH NO: 16-1
Run NO: 1&2
Sample Depth: 18.7 m to 21.8 m
Elevation: 158.9 m to 155.8 m
RQD: 57.5% to 77.5%
Date: June 21, 2016

Photo 1. Core Sample for BH 16-1 from Elevation 158.9 m to 155.8 m

Project NO: ADM-00233185-B0
BH NO: 16-2
Run NO: 1&2
Sample Depth: 18.2 m to 21.4 m
Elevation: 159.4 m to 156.2 m
RQD: 84.6% to 88.3%
Date: June 21, 2016

Photo 2. Core Sample for BH 16-2 from Elevation 159.4 m to 156.2 m

Appendix F –
Results of Slope Stability Analyses

Mindemoya River Bridge Replacement
Proposed Permanent Bridge - South 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: Sandy Silt to Silty Sand (Compact to Very Dense) Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion': 0 kPa Phi':
Name: Bedrock Model: Bedrock (Impenetrable)

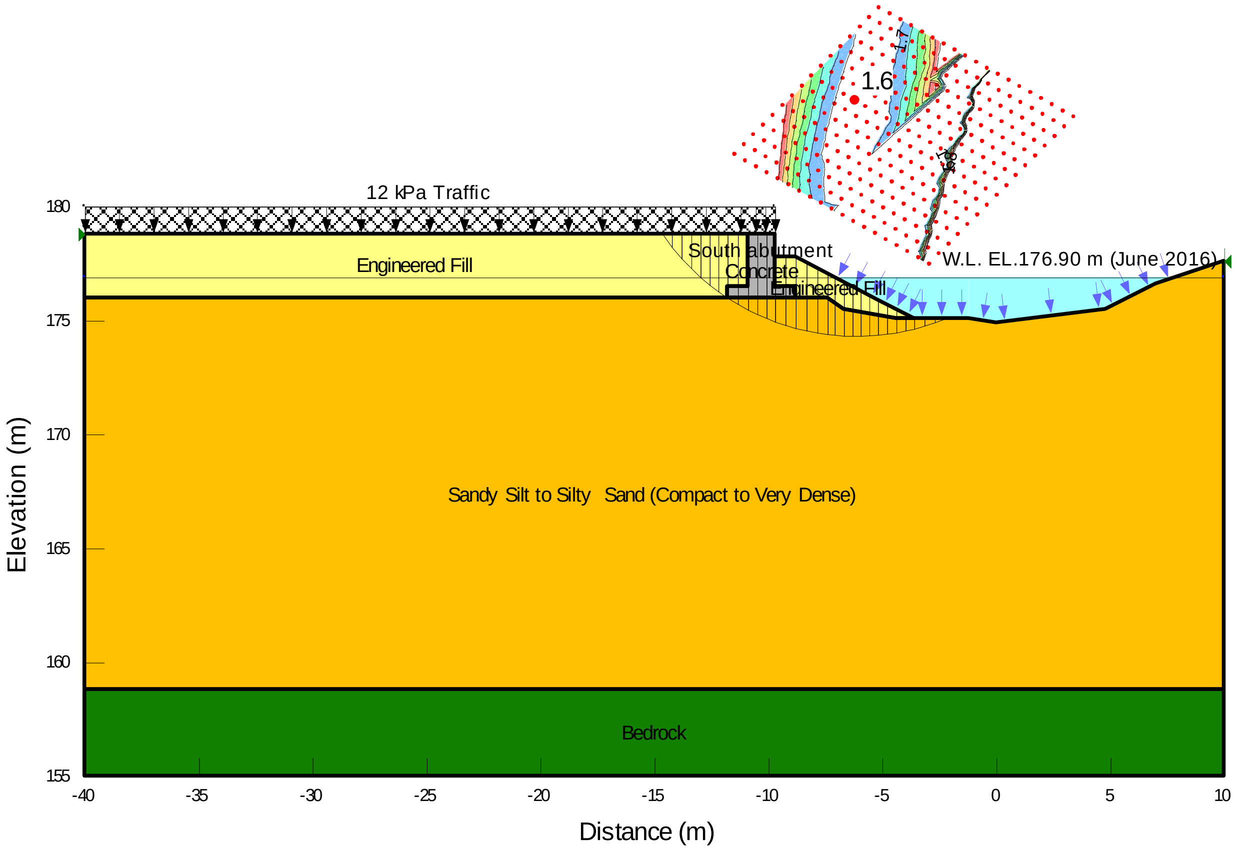


Figure F1: Proposed Permanent Bridge South abutment - Drained Static Condition

Mindemoya River Bridge Replacement
Proposed Permanent Bridge - South 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: Sandy Silt to Silty Sand (Compact to Very Dense) Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion': 0 kPa Phi': 32 °
Name: Bedrock Model: Bedrock (Impenetrable)

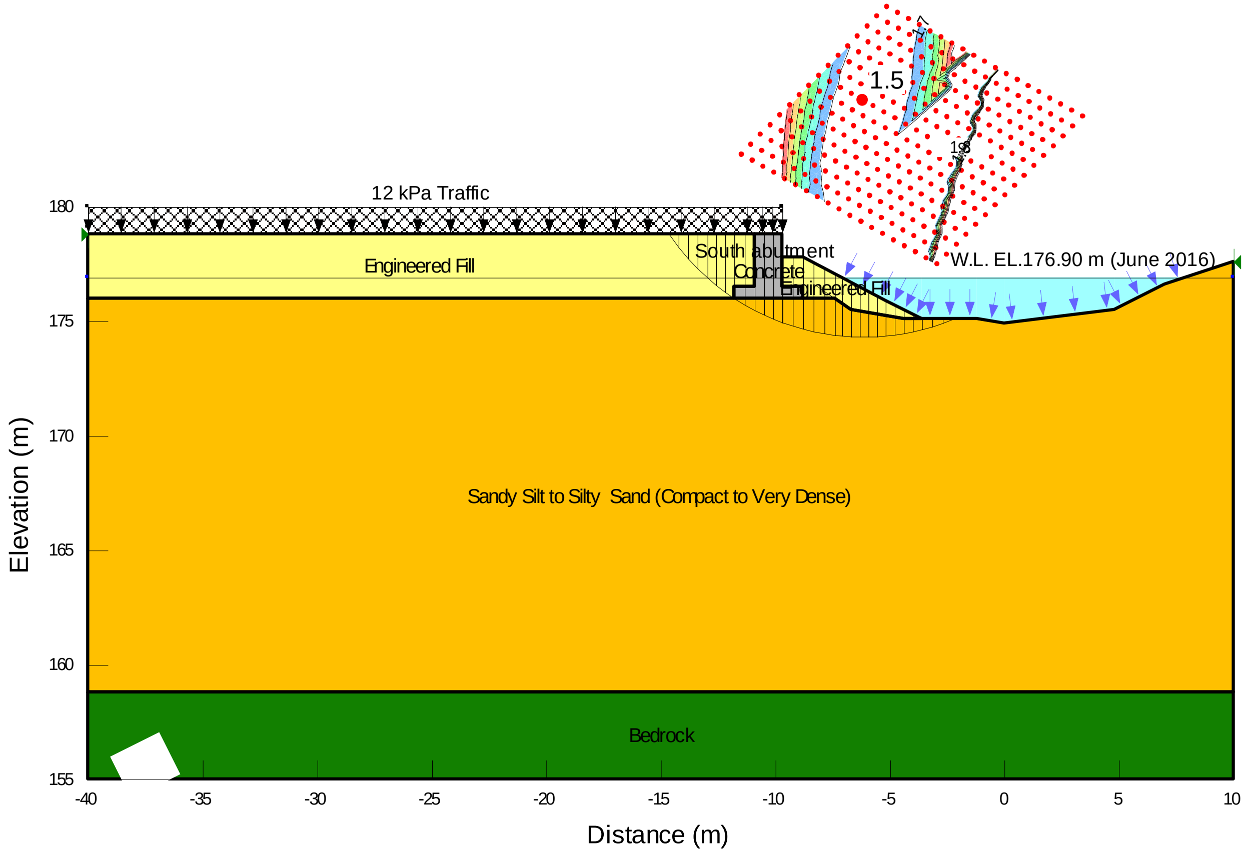


Figure F2: Proposed Permanent Bridge South Abutment - Drained Seismic Condition

Mindemoya River Bridge Replacement
The Proposed Permanent Bridge
South Embankment
Drained Static Conditions

Name: Engineered Fill Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion': 0 kPa Phi': 32 °
Name: Bedrock Model: Bedrock (Impenetrable)
Name: Sandy Silt to Silty Sand (Compact to very dense) Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion': 0 kPa Phi': 32 °

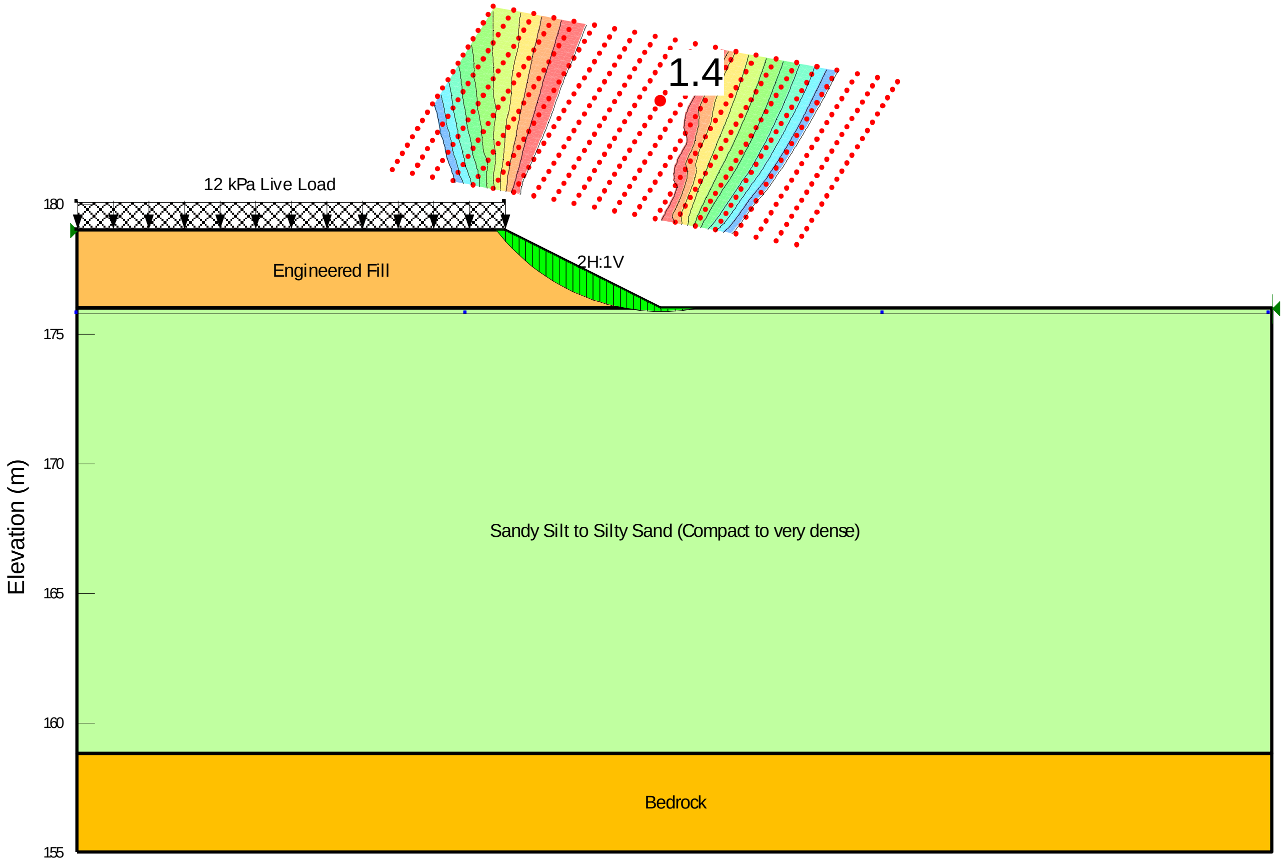


Figure F3: Proposed Permanent Bridge South Embankment - Drained Static Condition

Mindemoya River Bridge Replacement
The Proposed Permanent Bridge
South Embankment
Drained Seismic Conditions

Name: Engineered Fill Model: Mohr-Coulomb Unit Weight: 21 kN/m^3 Cohesion': 0 kPa Phi': 32 °
Name: Bedrock Model: Bedrock (Impenetrable)
Name: Sandy Silt to Silty Sand (Compact to very dense) Model: Mohr-Coulomb Unit Weight: 21 kN/m^3 Cohesion': 0 kPa Phi': 32 °

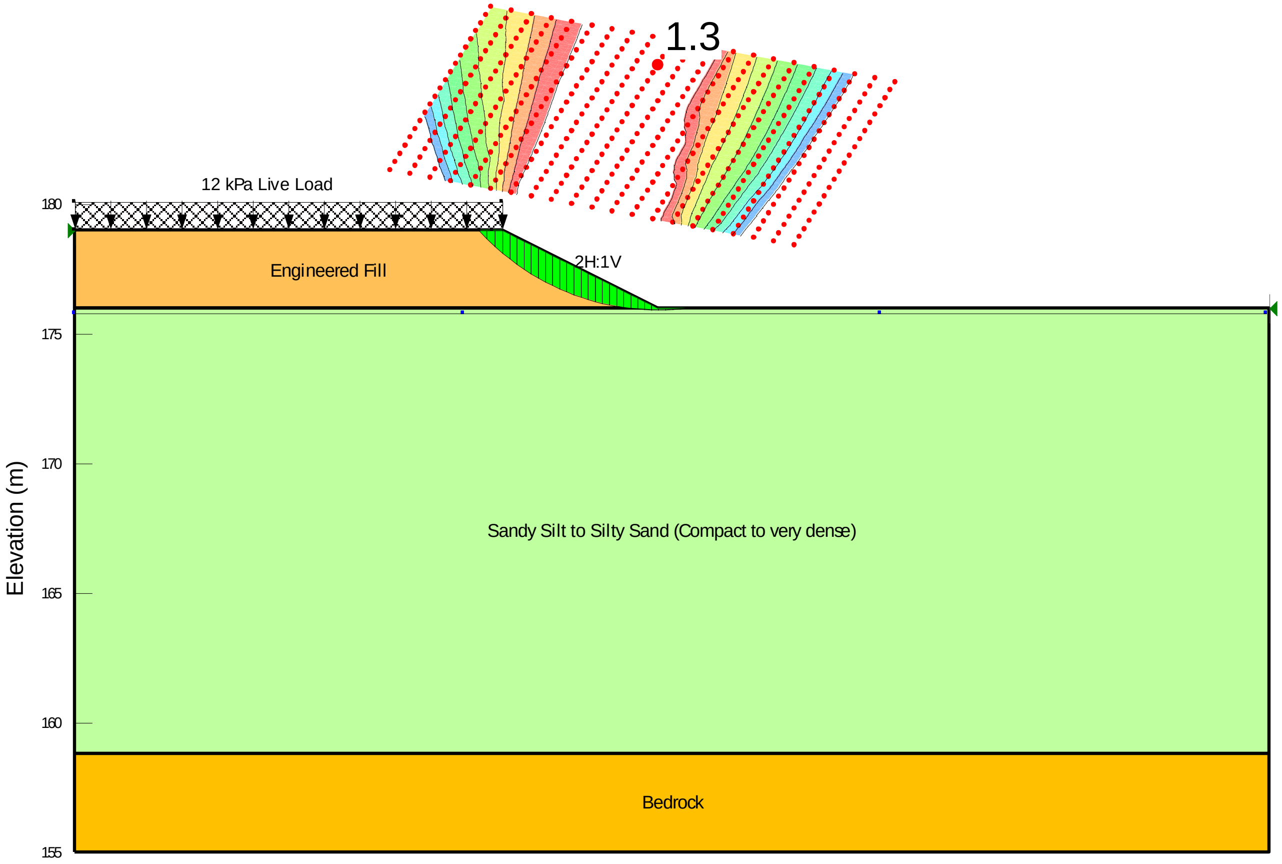


Figure F4: Proposed Permanent Bridge South Embankment - Drained Seismic Condition

Mindemoya River Bridge Replacement
The Proposed Temporary Bridge
North Embankment
Drained Static Conditions

Name: Engineered Fill Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion': 0 kPa Phi': 32 °
Name: Bedrock Model: Bedrock (Impenetrable)
Name: Silty Sand (Very loose to loose) Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion': 0 kPa Phi': 29 °
Name: Sandy Silt (Compact to dense) Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion': 0 kPa Phi': 31 °
Name: Silty Sand (Compact to very dense) Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion': 0 kPa Phi': 32 °

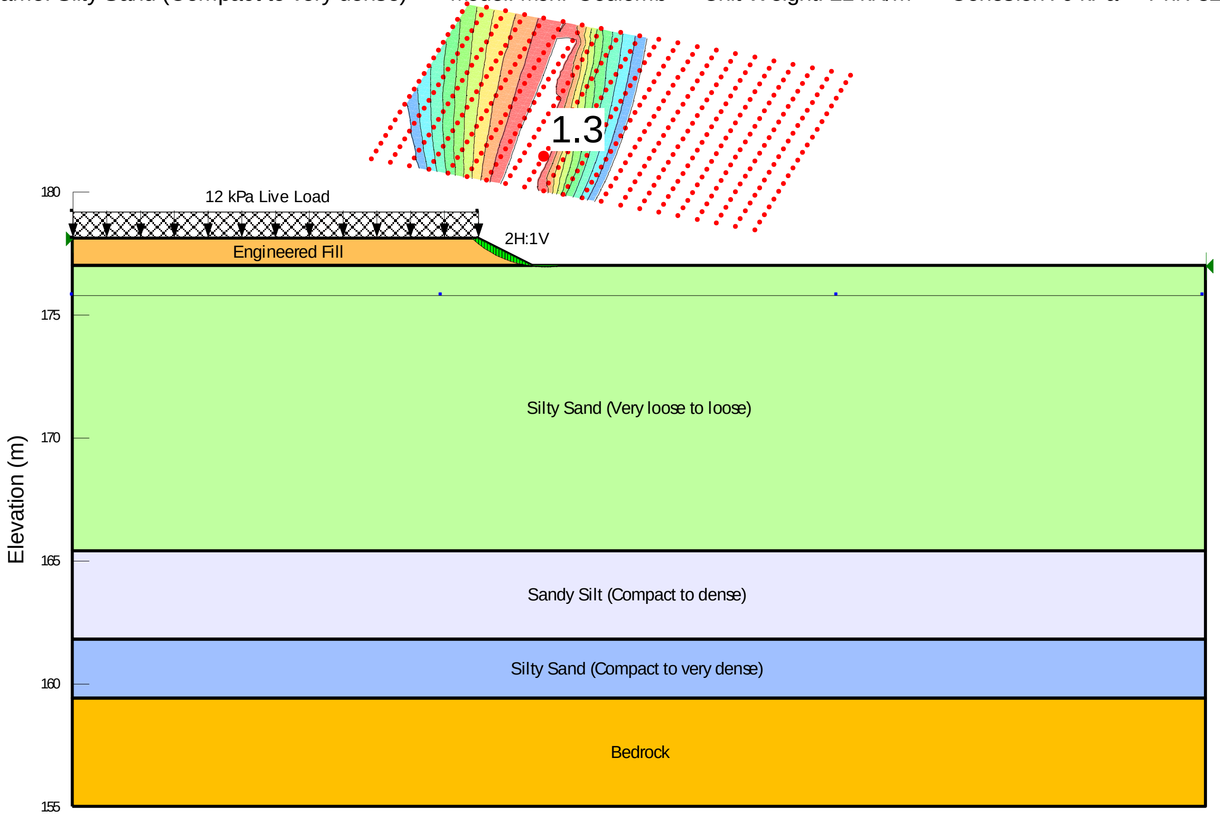


Figure F5: Proposed Temporary Bridge North Embankment - Drained Static Condition

Mindemoya River Bridge Replacement
The Proposed Temporary Bridge
North Embankment
Drained Seismic Conditions

Name: Engineered Fill Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion': 0 kPa Phi': 32 °
Name: Bedrock Model: Bedrock (Impenetrable)
Name: Silty Sand (Very loose to loose) Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion': 0 kPa Phi': 29 °
Name: Sandy Silt (Compact to dense) Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion': 0 kPa Phi': 31 °
Name: Silty Sand (Compact to very dense) Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion': 0 kPa Phi': 32 °

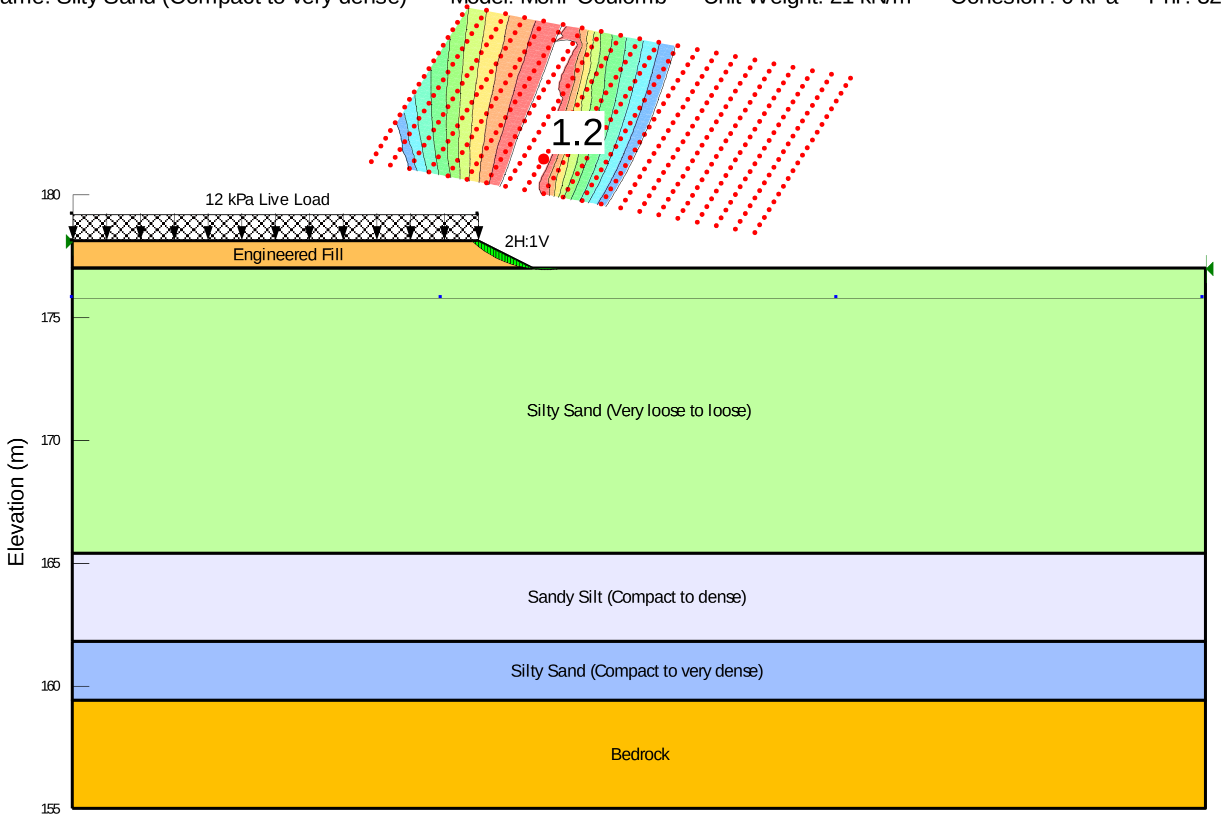
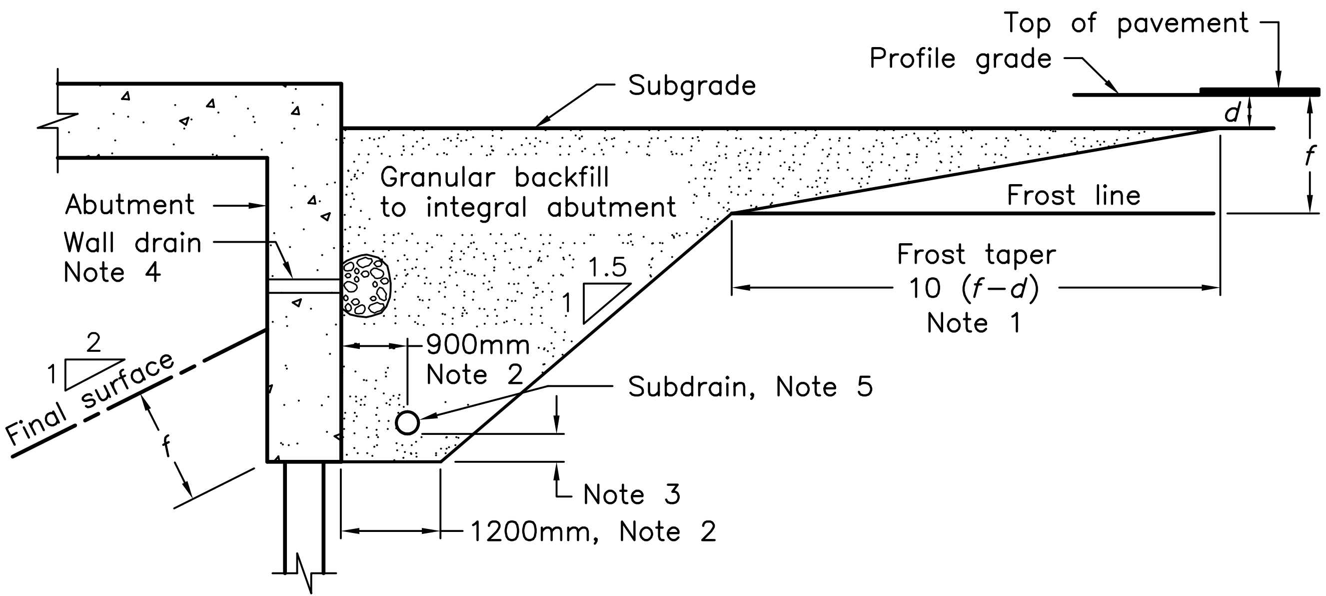
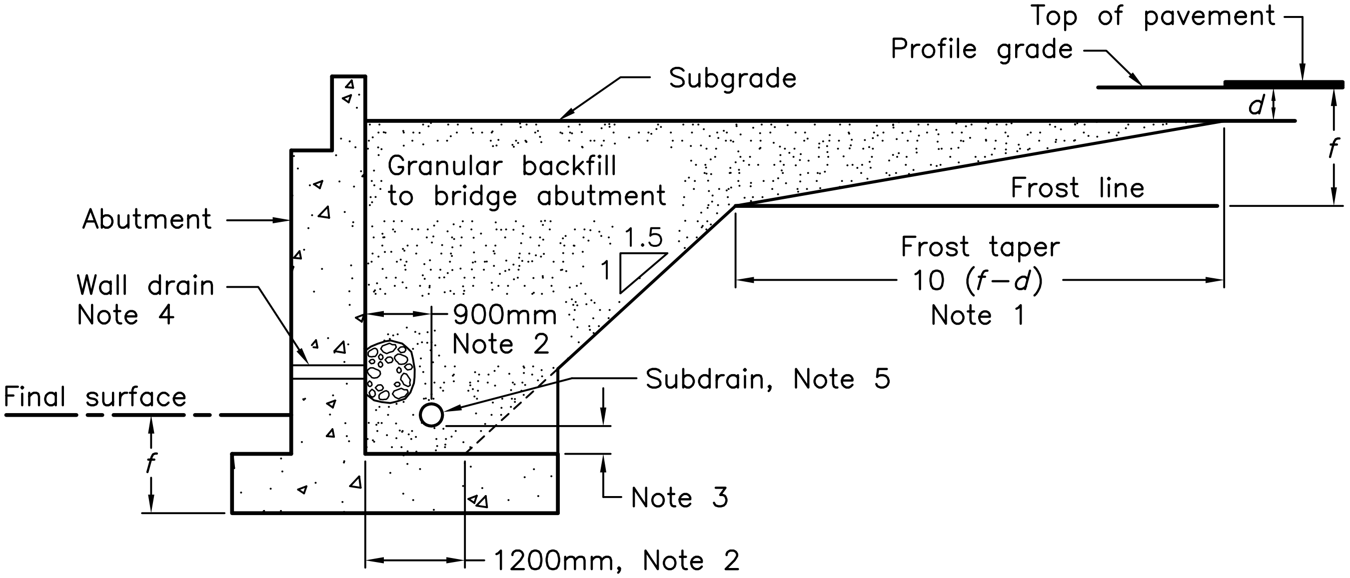


Figure F6: Proposed Temporary Bridge North Embankment - Drained Seismic Condition

Appendix G –
Foundation-related Specifications



INTEGRAL ABUTMENT



ABUTMENT

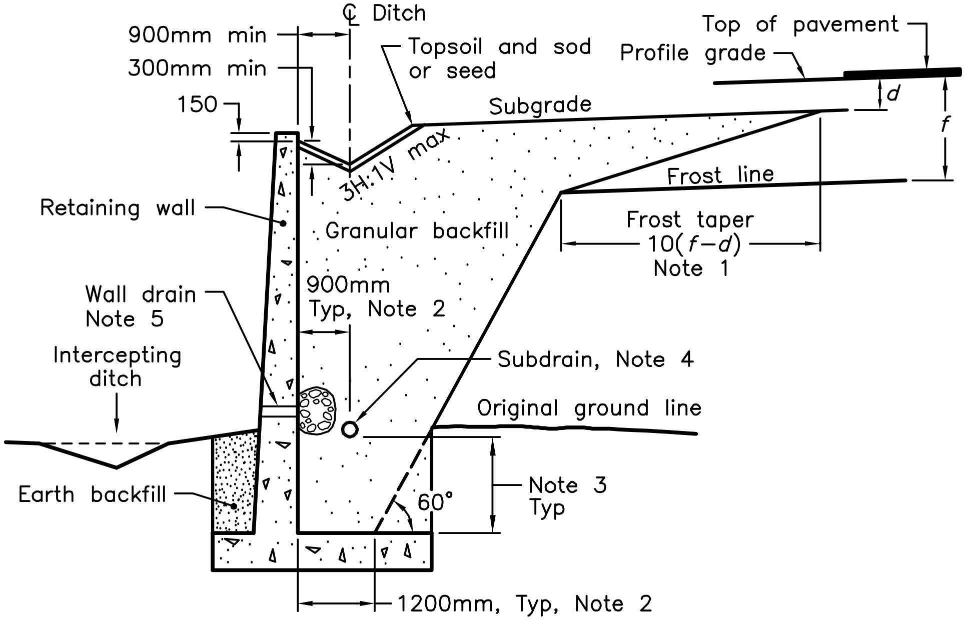
NOTES:

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

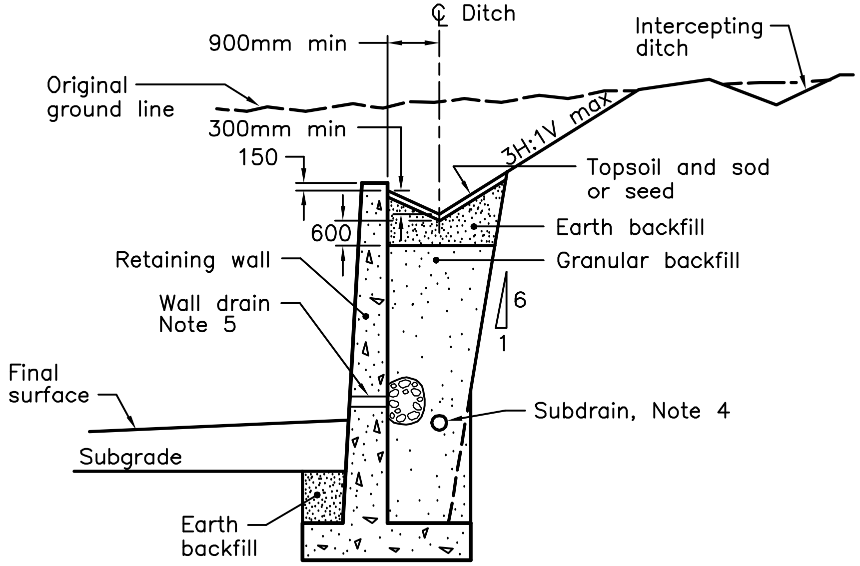
WALLS
ABUTMENT, BACKFILL
MINIMUM GRANULAR REQUIREMENT

OPSD − 3101.150



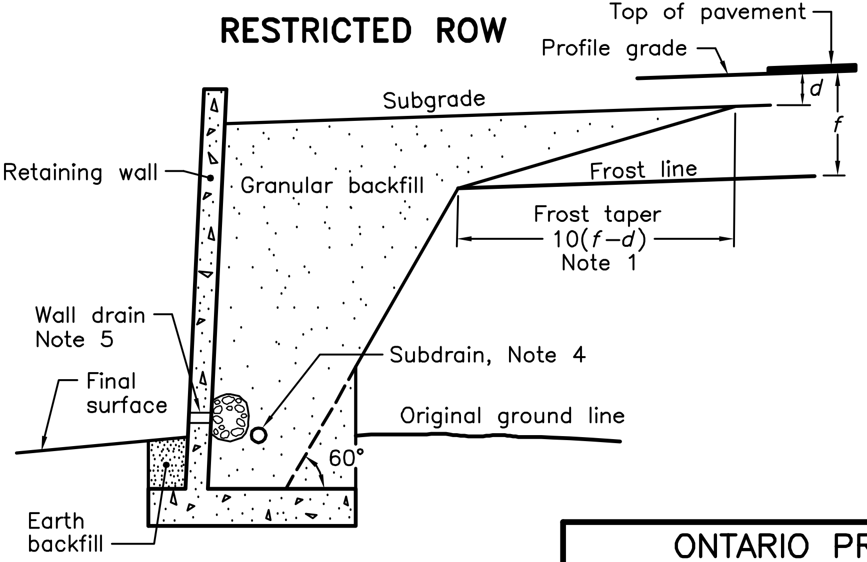
CANTILEVER WALL

RESTRICTED ROW



CANTILEVER WALL

CUT TREATMENT



COUNTERFORT AND CANTILEVER WALL

FILL TREATMENT

SHOULDER OVER RETAINING WALL BACKFILL

NOTES:

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.

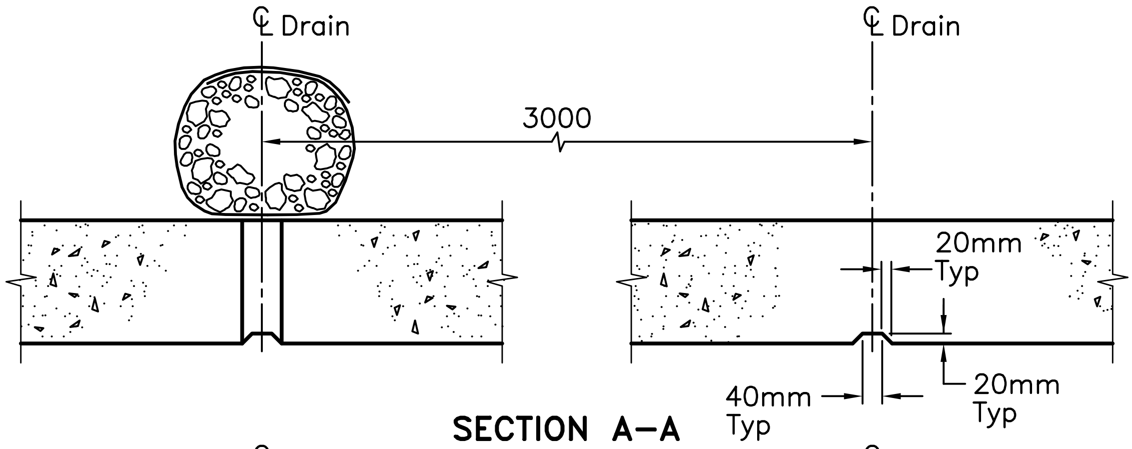
A All dimensions are in millimetres unless otherwise shown.

ONTARIO PROVINCIAL STANDARD DRAWING

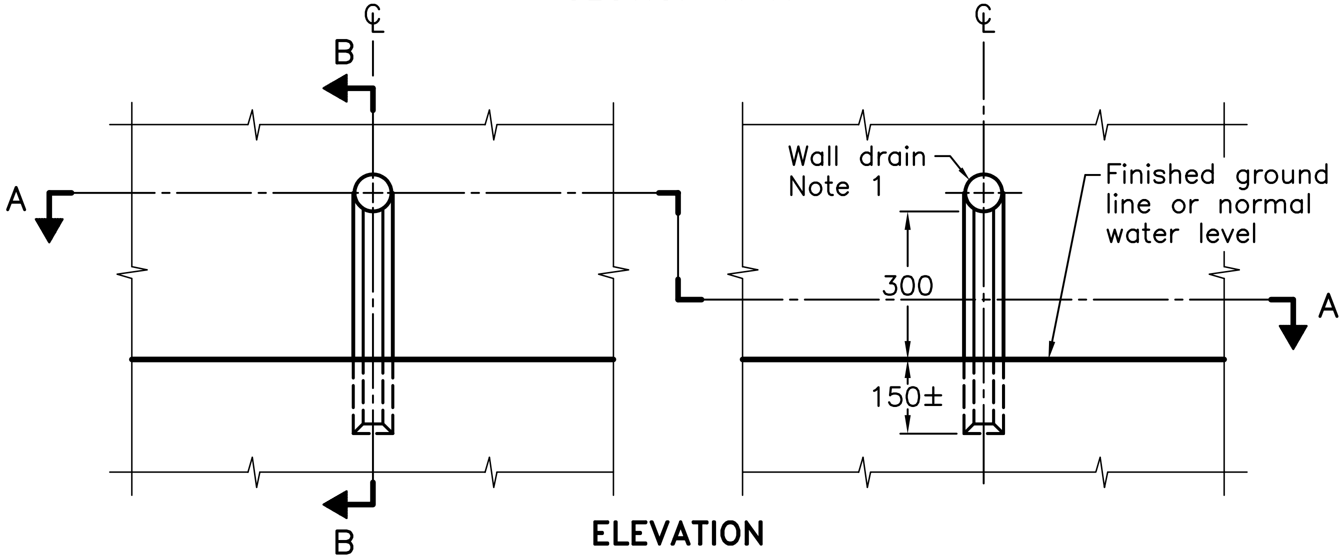
Nov 2010 | Rev 1

WALLS
RETAINING, BACKFILL
MINIMUM GRANULAR REQUIREMENT

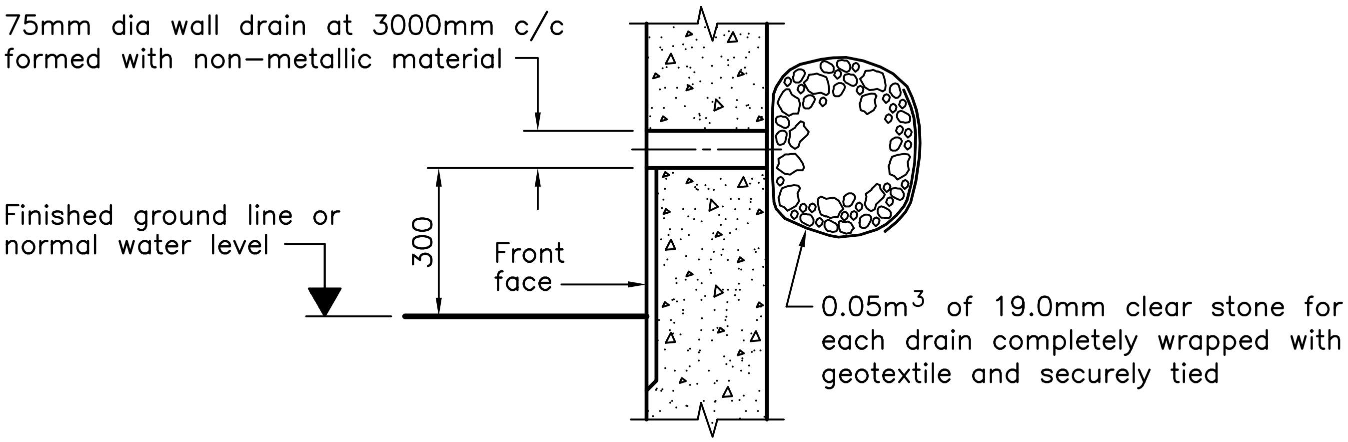
OPSD 3121.150



SECTION A–A



ELEVATION



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.

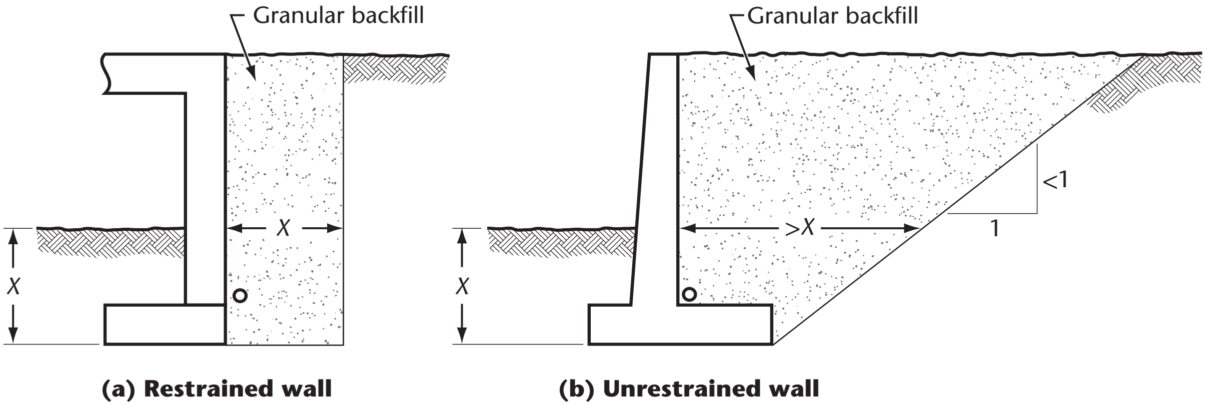
B All dimensions are in millimetres unless otherwise shown.

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.



(a) Restrained wall **(b) Unrestrained wall**

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 H –
Non-standard Special Provisions

NSSP FOR WORKING SLAB

Scope of Work

This Non-Standard Special Provision (NSSP) covers the requirements for the supply and placement of a concrete working slab on top of approved subgrade under structure foundation.

References

This NSSP refers to the following standards, specifications or publications:

Ontario Provincial Standard Specifications, Construction

OPSS 902	Excavating and Backfilling – Structures

Materials

Concrete for working slabs shall have a minimum 28-day strength of 20 MPa. The concrete curing requirements of OPSS.PROV 904 shall not apply.

Construction

Excavation for the working slab shall be according to OPSS 902.

Protection of Founding Soil

Within four hours flowing inspection and approval of the prepared subgrade, a working slab with a minimum thickness of 100 mm shall be placed on the foundation subgrade as specified in the Contract Documents.

Dewatering

Dewatering shall be carried out in accordance with OPSS 902.

Basis of Payment

Payment at the Contract price for the above tender item shall be full compensation for all labour. Equipment and Material to do the work.

END OF SECTION

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

Appendix I –
Previous Investigation Borehole Logs

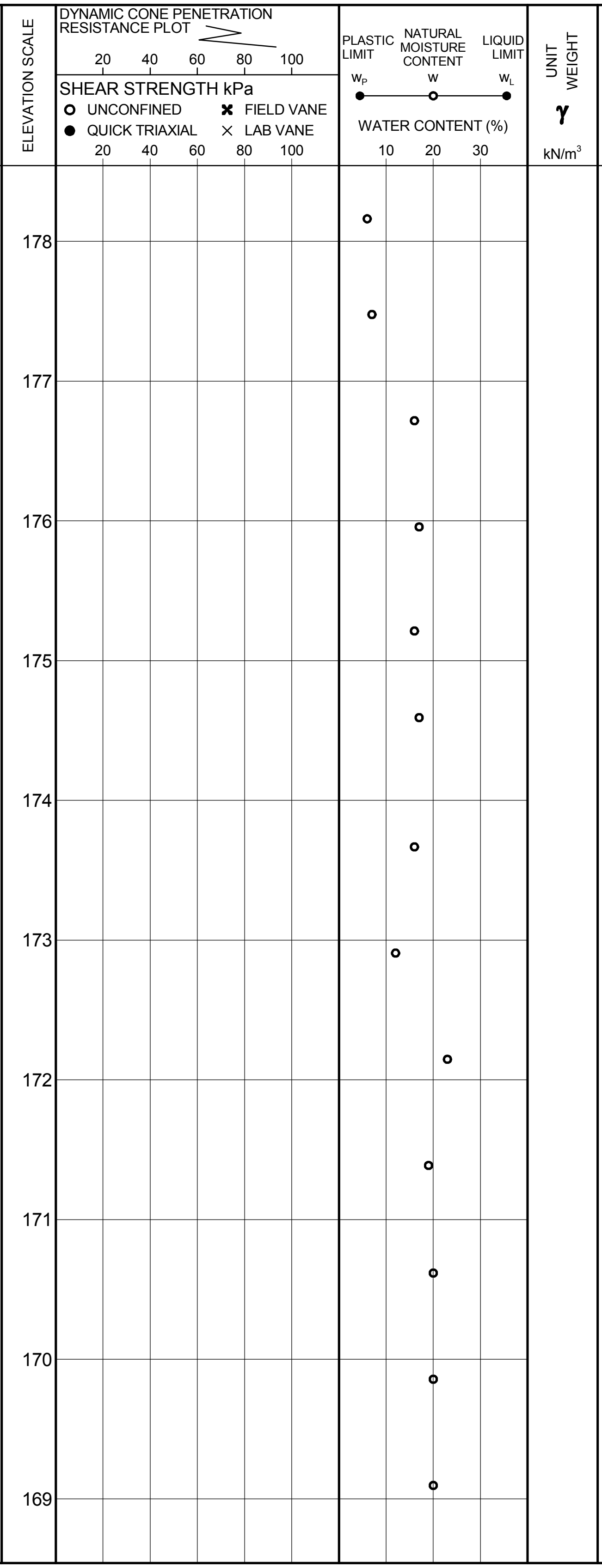
Stantec

RECORD OF BOREHOLE No BH14-1

1 OF 3 **METRIC**

W.P.	5153-12-00	LOCATION	Mindemoya River Bridge, Central Manitoulin, ON N: 5 058 548 E: 322 974	ORIGINATED BY	AN
DIST	Manitoulin HWY 551	BOREHOLE TYPE	Casing - Split spoon Sampler	COMPILED BY	AN
DATUM	Geodetic	DATE	2014 11 18 - 2014 11 20	CHECKED BY	CM

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES		GR	SA	SI	CL
178.5									
0.0	Fill: loose to compact brown poorly graded sand (SP) -moist to wet -some organics	1	GS	-					
		2	SS	19		3	93	(4)	
177.0 1.5	SILTY SAND (SM) Compact to very dense grey, wet	3	SS	21					
		4	SS	30	▼				
		5	SS	74					
		6	SS	50/ 130mm					
		7	SS	14					
173.2 5.3	Well-graded SAND (SW-SM) wih silt and gravel Compact Grey, wet	8	SS	18		18	76	(6)	
172.6 5.9	SILTY SAND (SM) Compact to very dense Grey, wet	9	SS	26					
		10	SS	19					
		11	SS	55		3	84	12	1
		12	SS	29					
		13	SS	32					



Continued Next Page

✖^{3}, ×^{3}: Numbers refer to Sensitivity

O$^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No BH14-1

2 OF 3 **METRIC**

W.P. 5153-12-00
LOCATION Mindemoya River Bridge, Central Manitoulin, ON N: 5 058 548 E: 322 974
ORIGINATED BY AN
DIST Manitoulin HWY 551
BOREHOLE TYPE Casing - Split spoon Sampler
COMPILED BY AN
DATUM Geodetic
DATE 2014 11 18 - 2014 11 20
CHECKED BY CM

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	GR	SA	SI	CL
	SILTY SAND (SM) Compact to very dense Grey, wet *(continued)*								
		14	SS	22					
		15	SS	26					
		16	SS	21					
163.3 15.2	SILT with sand (ML) Loose to dense Grey, wet								
		17	SS	7	163	0	15	70	1
		18	SS	31					
		19	SS	21					
		20	SS	50/130mm					
159.3 19.3	Limestone BEDROCK with shaly partings and dolomite seams - good quality rock - light grey - moderately to slighty weathered	21	NQ	-		UCS = 237 MPa TCR = 98% RQD = 85%			
158.5									

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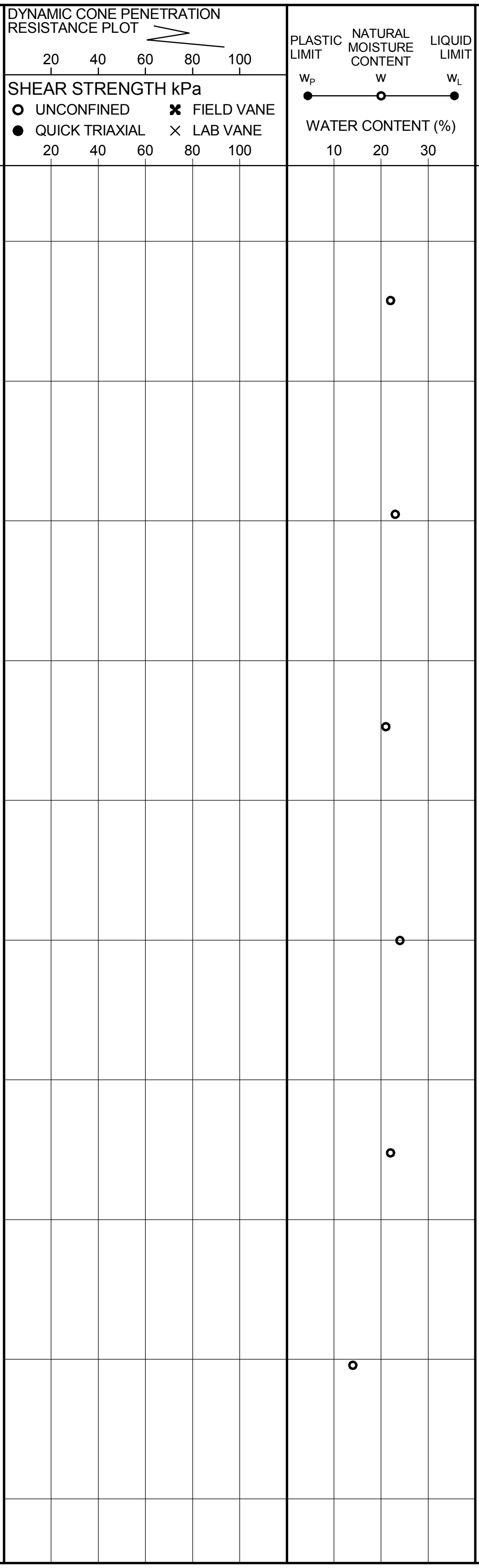


UNIT WEIGHT γ kN/m³

✖ 3, × 3: Numbers refer to Sensitivity O 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No BH14-1

METRIC

W.P. 5153-12-00
LOCATION Mindemoya River Bridge, Central Manitoulin, ON N: 5 058 548 E: 322 974
ORIGINATED BY AN
DIST Manitoulin HWY 551
BOREHOLE TYPE Casing - Split spoon Sampler
COMPILED BY AN
DATUM Geodetic
DATE 2014 11 18 - 2014 11 20
CHECKED BY CM

SOIL PROFILE			SAMPLES					DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED ✖ FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE				
20.0	- close to very close joint spacing (Refer to Field Bedrock Core Log)										
	Light grey shaly limestone with dolomite seams BEDROCK - good quality rock - moderately to slighty weathered		22	NQ	-		158				TCR = 89% RQD = 73%
			23	NQ	-		157				UCS = 117 MPa TCR = 98% RQD = 75%
156.0 22.5	End of Borehole -Vibrating Wire Piezometer tip installed at 5.48 m below ground surface										

✖^{3}, ×^{3}: Numbers refer to Sensitivity ○$^{3\%}$ STRAIN AT FAILURE

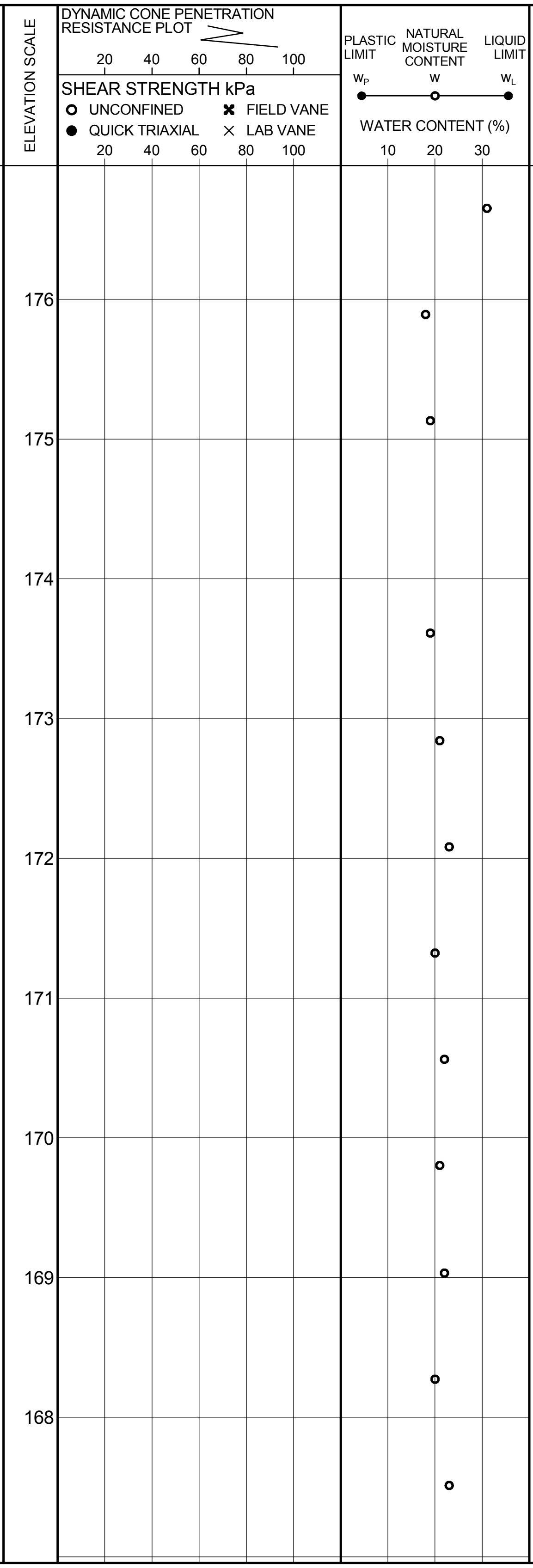
Stantec

RECORD OF BOREHOLE No BH14-2

1 OF 3 **METRIC**

W.P. 5153-12-00
LOCATION Mindemoya River Bridge, Central Manitoulin, ON N: 5 058 578 E: 322 961
ORIGINATED BY AN
DIST Manitoulin HWY 551
BOREHOLE TYPE Casing - Split spoon Sampler
COMPILED BY AN
DATUM Geodetic
DATE 2014 11 20 - 2014 11 20
CHECKED BY CM

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES		GR	SA	SI	CL
177.0									
0.0 176.3	Fill: very loose brown poorly graded sand (SP) -moist -some organics	1	SS	WH					
0.6	SANDY SILT (ML) Compact Grey, wet	2	SS	18	▼				
		3	SS	19		2	34	(64)	
		4	SS	15					
		5	SS	14					
		6	SS	16					
171.6		7	SS	28					
5.3	SILTY SAND (SM) Compact to dense Grey,wet	8	SS	23		1	64	33	2
		9	SS	36					
		10	SS	19					
		11	SS	22					
		12	SS	17					
167.0		13	SS	22					



UNIT WEIGHT γ kN/m³

Continued Next Page

✖³, ×³: Numbers refer to Sensitivity

O³% STRAIN AT FAILURE

Stantec

RECORD OF BOREHOLE No BH14-2

2 OF 3 **METRIC**

W.P. 5153-12-00	LOCATION Mindemoya River Bridge, Central Manitoulin, ON N: 5 058 578 E: 322 961	ORIGINATED BY AN
DIST Manitoulin HWY 551	BOREHOLE TYPE Casing - Split spoon Sampler	COMPILED BY AN
DATUM Geodetic	DATE 2014 11 20 - 2014 11 20	CHECKED BY CM

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES		
10.0	SILTY SAND (SM) Compact to dense Grey, wet	14	SS	27		
		15	SS	27		
		16	SS	21		
		17	SS	21		
161.7 15.2	SANDYSILT (ML) Compact Grey, wet	18	SS	24		5 45 41 9
		19	SS	22		
160.1 16.9	Limestone BEDROCK with shaly partings and dolomite seams - poor to fair quality rock - light grey - moderately to slighty weathered - close to very close joint spacing (Refer to Field Bedrock Core Log)	20	NQ	-		UCS = 146 MPa TCR = 99% RQD = 55%
		21	NQ	-		UCS = 202 MPa TCR = 71% RQD = 47%
157.0						

ELEVATION SCALE

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH kPa

O UNCONFINED ✖ FIELD VANE

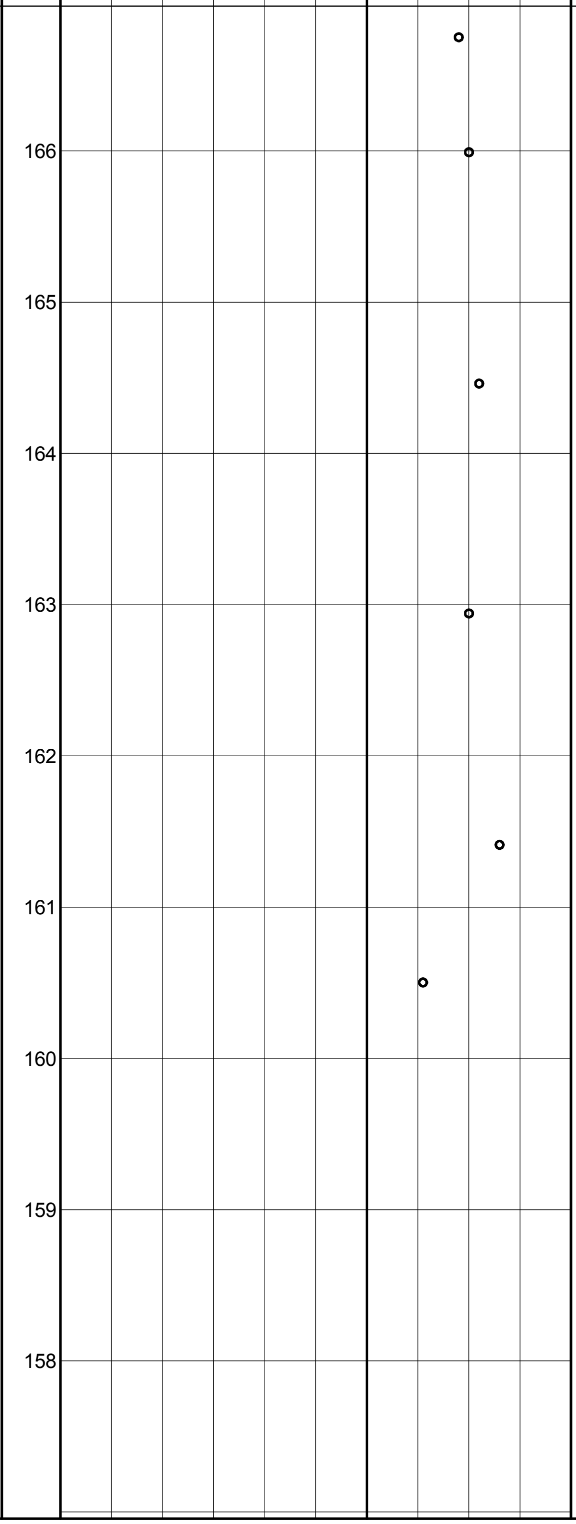
● QUICK TRIAXIAL × LAB VANE

20 40 60 80 100

PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L

WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ kN/m^3



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✖^{3}, ×^{3}: Numbers refer to Sensitivity O$^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No BH14-2

3 OF 3

METRIC

W.P.	5153-12-00	LOCATION	Mindemoya River Bridge, Central Manitoulin, ON N: 5 058 578 E: 322 961
DIST	Manitoulin HWY 551	BOREHOLE TYPE	Casing - Split spoon Sampler
DATUM	Geodetic	DATE	2014 11 20 - 2014 11 20

ORIGINATED BY AN
COMPILED BY AN
CHECKED BY CM

SOIL PROFILE			SAMPLES					DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa O UNCONFINED ✖ FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE				GR SA SI CL
20.0 156.8 20.2	Light grey shaly limestone BEDROCK with dolomite seams - poor to fair quality rock - moderately to slighty weathered End of Borehole -Vibrating Wire Piezometer tip installed at 1.65 m below ground surface										

✖ ^{3}, × ^{3}: Numbers refer to Sensitivity

O $^{3\%}$ STRAIN AT FAILURE

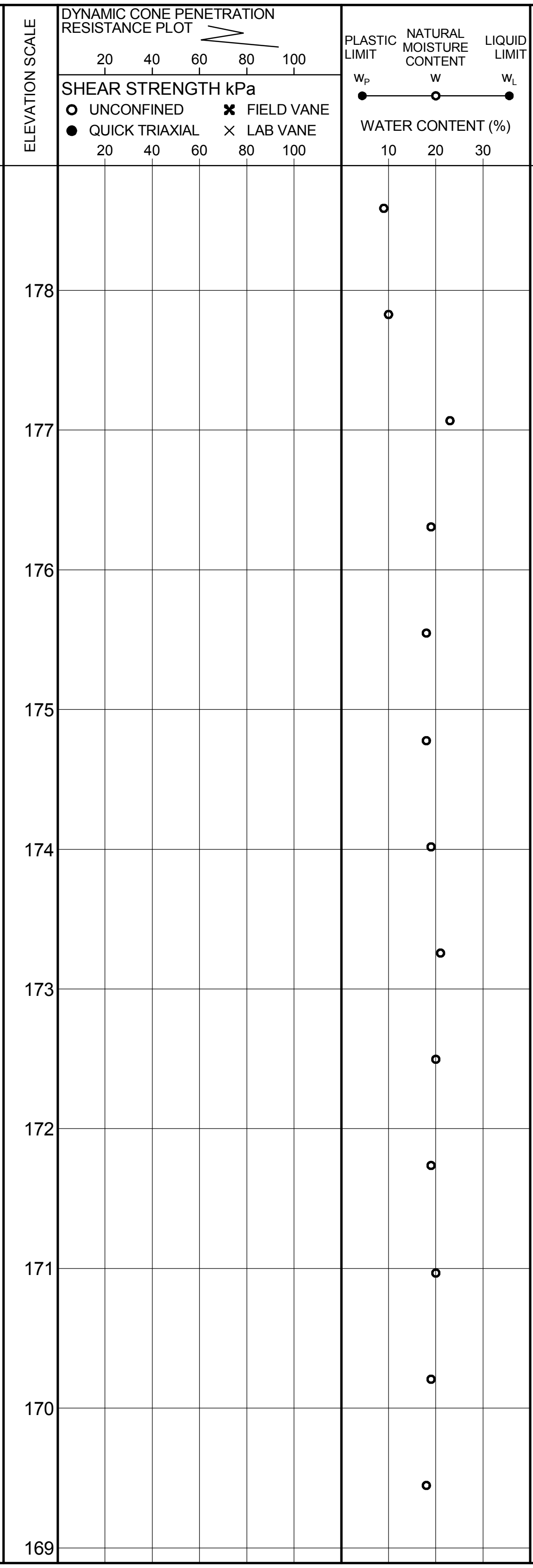
RECORD OF BOREHOLE No BH14-3

1 OF 3

METRIC

W.P. 5153-12-00
LOCATION Mindemoya River Bridge, Central Manitoulin, ON N: 5 058 551 E: 322 989
ORIGINATED BY AN
DIST Manitoulin HWY 551
BOREHOLE TYPE Casing - Split spoon Sampler
COMPILED BY AN
DATUM Geodetic
DATE 2014 11 25 - 2014 11 26
CHECKED BY CM

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
178.9 0.0	20 mm Asphalt Fill: dense to compact brown silty sand (SM) with gravel -moist	1	SS	34						
		2	SS	28						
177.5 1.4	Poorly graded SAND (SP) Loose to compact Brown, wet	3	SS	9			4	91	4	1
		4	SS	11						
176.0 2.9	SANDY SILT (ML) Dense to compact Grey, wet	5	SS	36						
		6	SS	10			5	38	55	2
		7	SS	12						
		8	SS	24						
		9	SS	21						
		10	SS	16						
		11	SS	41						
		12	SS	32						
		13	SS	43						
168.9										



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✖³, ×³: Numbers refer to Sensitivity

O³% STRAIN AT FAILURE

Stantec

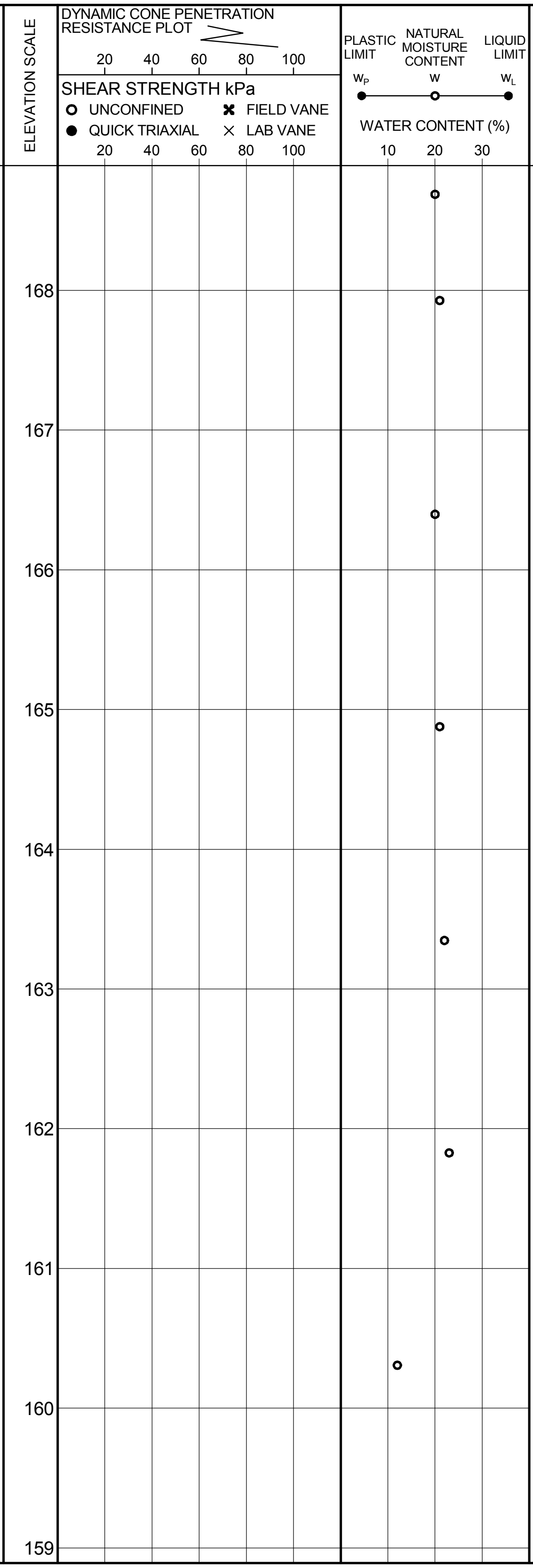
RECORD OF BOREHOLE No BH14-3

2 OF 3

METRIC

W.P. 5153-12-00
LOCATION Mindemoya River Bridge, Central Manitoulin, ON N: 5 058 551 E: 322 989
ORIGINATED BY AN
DIST Manitoulin HWY 551
BOREHOLE TYPE Casing - Split spoon Sampler
COMPILED BY AN
DATUM Geodetic
DATE 2014 11 25 - 2014 11 26
CHECKED BY CM

SOIL PROFILE		SAMPLES			REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
10.0	SANDY SILT (ML) Dense to compact Grey, wet	14	SS	43				
		15	SS	37				
		16	SS	50				
		17	SS	44				
		18	SS	38	0	32	57	11
		19	SS	30				
		20	SS	12				
158.9								



UNIT WEIGHT γ kN/m³

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✖3, ×3: Numbers refer to Sensitivity

○3% STRAIN AT FAILURE

STN13-ONTARIO MTO STANTEC 165000912 - HWY 551, MINDEMOYA BRIDGE - MTO.GPJ ONTARIO MOT.GDT 5/6/15

Stantec

RECORD OF BOREHOLE No BH14-3

3 OF 3

METRIC

W.P. 5153-12-00
LOCATION Mindemoya River Bridge, Central Manitoulin, ON N: 5 058 551 E: 322 989
ORIGINATED BY AN
DIST Manitoulin HWY 551
BOREHOLE TYPE Casing - Split spoon Sampler
COMPILED BY AN
DATUM Geodetic
DATE 2014 11 25 - 2014 11 26
CHECKED BY CM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED ✖ FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m^3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
158.8 20.0	SANDY SILT (ML) Dense to compact Grey, wet										
20.1	Limestone BEDROCK with shaly partings and dolomite seams - fair quality rock - Grey - moderately to slighty weathered - Close to very close joint spacing (Refer to Field Bedrock Core Log)		21	NQ	-		158				UCS = 142 MPa TCR = 98% RQD = 64%
156.6			22	NQ	-		157				TCR = 73% RQD = 66% UCS = 124 MPa
22.3	End of Borehole										

✖ ^{3}, × ^{3}: Numbers refer to Sensitivity

○ $^{3\%}$ STRAIN AT FAILURE

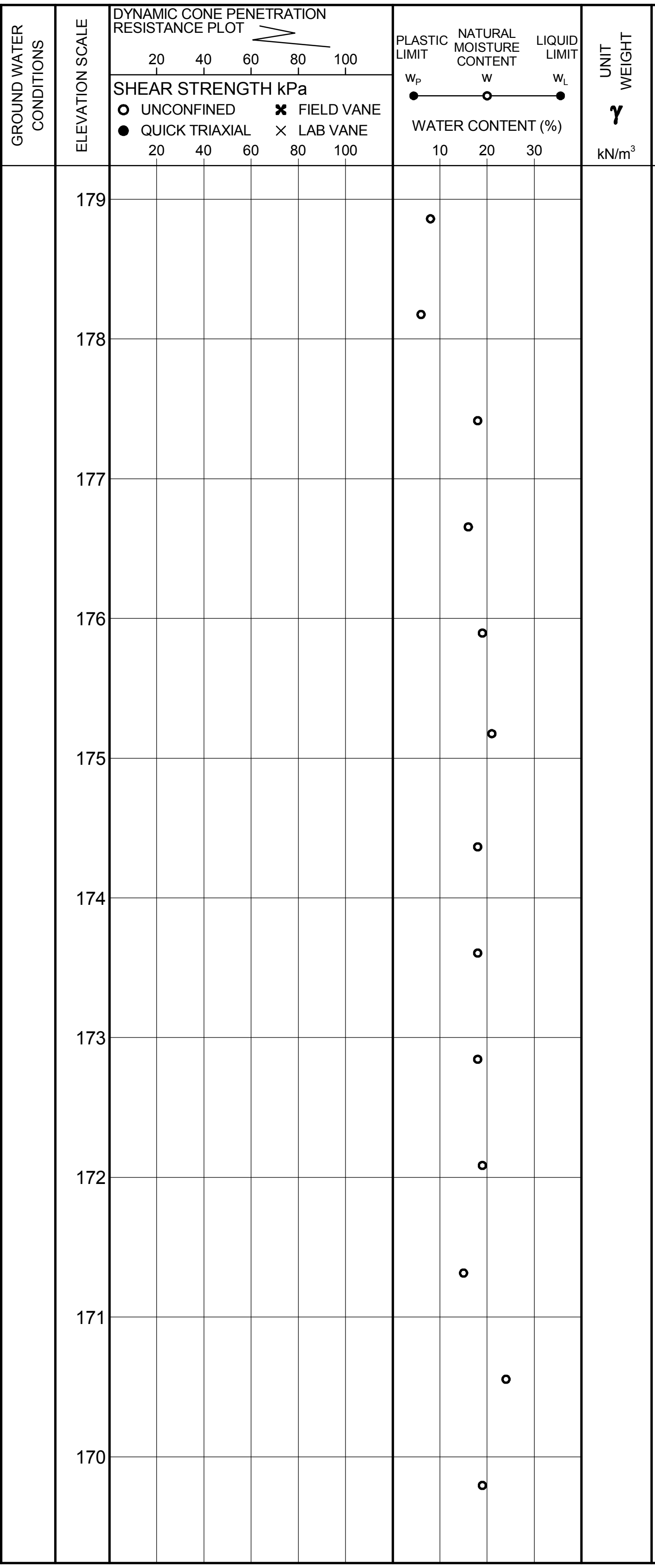
Stantec

RECORD OF BOREHOLE No BH14-4

1 OF 3 **METRIC**

W.P. 5153-12-00
LOCATION Mindemoya River Bridge, Central Manitoulin, ON N: 5 058 584 E: 322 978
ORIGINATED BY AN
DIST Manitoulin HWY 551
BOREHOLE TYPE Hollow Stem Augurs - Split spoon Sampler
COMPILED BY AN
DATUM Geodetic
DATE 2014 11 17 - 2014 11 25
CHECKED BY CM

SOIL PROFILE		SAMPLES			REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
179.2								
179.2 0.0	40 mm Asphalt							
	FILL: loose to compact brown silty sand (SM) with gravel -moist	1	GS	-				
		2	SS	10	23	58	(19)	
		3	SS	10				
176.7 2.6	SILTY SAND (SM) Very dense to compact Grey, wet	4	SS	31				
		5	SS	61				
		6	SS	98				
		7	SS	34	0	62	36	2
		8	SS	31				
		9	SS	11				
		10	SS	19				
		11	SS	29				
		12	SS	29				
170.1 9.1	SANDY SILT (ML) Compact to dense Grey, wet	13	SS	21	0	45	53	2
169.2								



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✖3, ×3: Numbers refer to Sensitivity

O 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No BH14-4

2 OF 3 **METRIC**

W.P. 5153-12-00

LOCATION Mindemoya River Bridge, Central Manitoulin, ON N: 5 058 584 E: 322 978

ORIGINATED BY AN

DIST Manitoulin HWY 551

BOREHOLE TYPE Hollow Stem Augurs - Split spoon Sampler

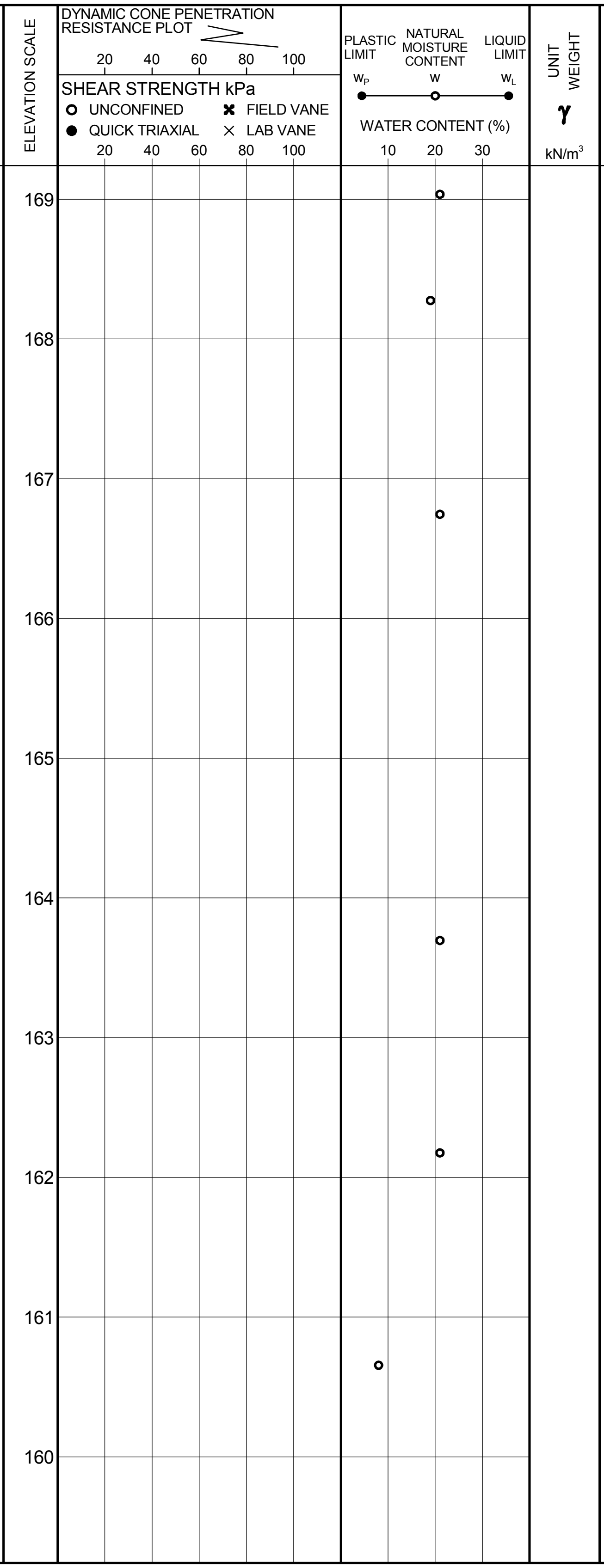
COMPILED BY AN

DATUM Geodetic

DATE 2014 11 17 - 2014 11 25

CHECKED BY CM

ELEV DEPTH	SOIL PROFILE: DESCRIPTION	SAMPLES: NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
10.0	SANDY SILT (ML) Compact to dense Grey, wet	14	SS	19				
		15	SS	25				
		16	SS	27				
		17	SS	32	0	45	53	2
		18	SS	10				
		19	SS	26	1	43	42	2
161.0 18.3	SILTY SAND (SM) with gravel Dense Grey, wet	20	SS	44	23	48	(29)	
159.9 19.4								



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✖ 3, × 3: Numbers refer to Sensitivity

O 3% STRAIN AT FAILURE

STN13-ONTARIO MTO STANTEC 165000912 - HWY 551, MINDEMOYA BRIDGE - MTO.GPJ ONTARIO MOT.GDT 5/6/15

RECORD OF BOREHOLE No BH14-4

3 OF 3 **METRIC**

W.P. 5153-12-00	LOCATION Mindemoya River Bridge, Central Manitoulin, ON	N: 5 058 584 E: 322 978	ORIGINATED BY AN
DIST Manitoulin HWY 551	BOREHOLE TYPE Hollow Stem Augurs - Split spoon Sampler		COMPILED BY AN
DATUM Geodetic	DATE 2014 11 17 - 2014 11 25		CHECKED BY CM

SOIL PROFILE			SAMPLES								
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED ✖ FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Limestone BEDROCK with shaly partings and dolomite seams - fair to good quality rock - light grey - moderately to slighty weathered - close to very close joint spacing (Refer to Field Bedrock Core Log) *(continued)*		21	NQ	-		159				TCR = 98% RQD = 68% UCS = 147 MPa
			22	NQ	-		158				TCR = 100% RQD = 79% UCS = 194 MPa
157.0 22.2	End of Borehole										

✖ ^{3}, × ^{3}: Numbers refer to Sensitivity

○ $^{3\%}$ STRAIN AT FAILURE

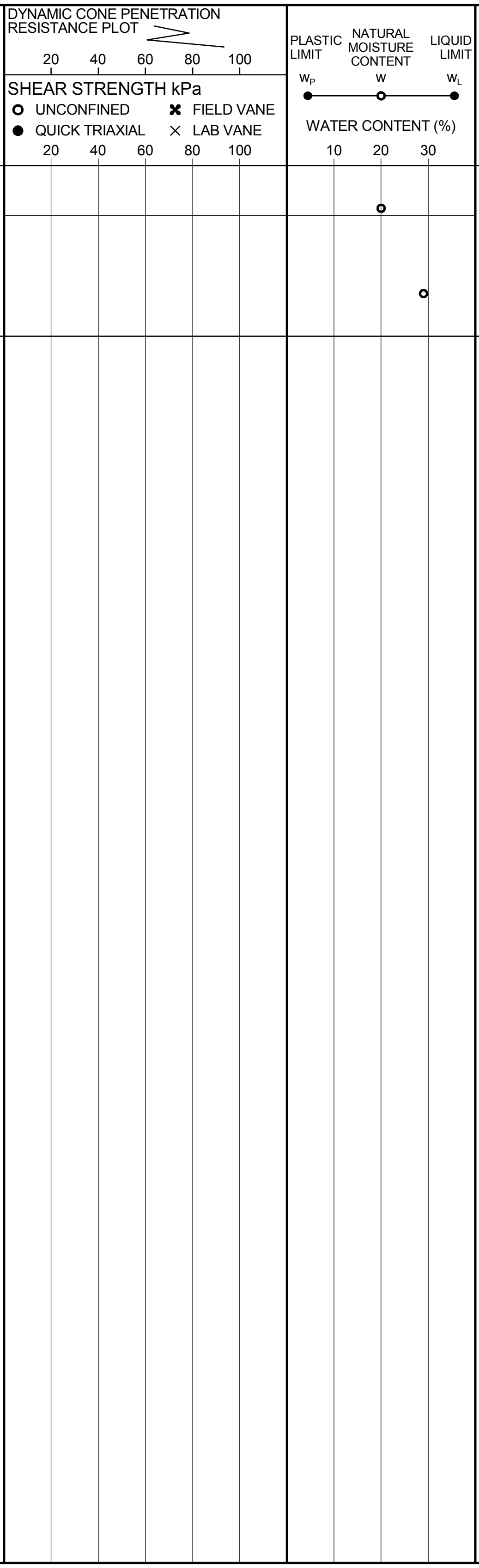
Stantec

RECORD OF BOREHOLE No BH14-5

1 OF 1 **METRIC**

W.P. 5153-12-00	LOCATION Mindemoya River Bridge, Central Manitoulin, ON	N: 5 058 560 E: 322 997	ORIGINATED BY AN
DIST Manitoulin HWY 551	BOREHOLE TYPE Split spoon Sampler		COMPILED BY AN
DATUM Geodetic	DATE 2014 11 26 - 2014 11 26		CHECKED BY CM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES				GR	SA	SI	CL
177.4												
0.0	FILL: loose brown silty sand (SM) with gravel -moist	▨	1	SS	-		177					
176.7 0.6	SILTY SAND (SM) Compact Grey, wet	⁘	2	SS	-				1	59	(40)	
176.1 1.2	End of Borehole											



✖³, ×³: Numbers refer to Sensitivity

O 3% STRAIN AT FAILURE

Stantec

RECORD OF BOREHOLE No BH14-6

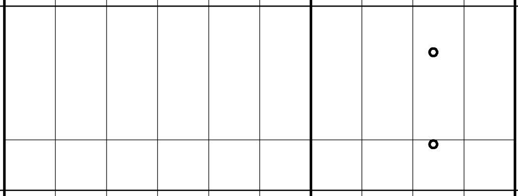
1 OF 1 **METRIC**

W.P. 5153-12-00 LOCATION Mindemoya River Bridge, Central Manitoulin, ON N: 5 058 580 E: 322 999 ORIGINATED BY AN

DIST Manitoulin HWY 551 BOREHOLE TYPE Split spoon Sampler COMPILED BY AN

DATUM Geodetic DATE 2014 11 26 - 2014 11 26 CHECKED BY CM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa O UNCONFINED ✖ FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						GR	SA	SI	CL
176.9														
0.0	SILTY SAND (SM) Loose to compact Grey, moist to wet -some organics		1	SS	-				o					
			2	SS	-		176		o		0	86	(14)	
175.7														
1.2	End of Borehole													



✖ ³, × ³: Numbers refer to Sensitivity

O ³% STRAIN AT FAILURE