FINAL REPORT

FOUNDATION INVESTIGATION AND DESIGN REPORT

Rosseau River Bridge Replacement, Highway 141, Rosseau, Ontario

Agreement No. 5013-E-0008
Assignment No. 11
GWP 5394-15-00
Geocres No. 31E-361

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1 FOUNDATION INVESTIGATION REPORT

1.1 Introduction

This foundation investigation report presents the results of a geotechnical investigation completed by **exp** Services Inc. for the rehabilitation/replacement of the Rosseau River Bridge located on Hwy 141, approximately 4.9 km east of the junction of Hwy 632, Rosseau, Ontario, the Ministry of Transportation (MTO) Northeastern Region. The work was undertaken under Agreement # 5013-E-0008, Assignment No. 11 (GWP 330-96-00). The terms of reference (TOR) were as presented in the MTO letter dated August 26, 2015.

Based on information included in the TOR it is understood that the existing Rossea River Bridge is a 13 m long single span bridge supported on shallow foundations. During the inspection of the bridge abutments in August 15 2015, it was noted that voids and scouring were present underneath the west abutment. Due to presence of these voids and scouring, the existing bridge was closed for traffic in September 2015, and both abutments of the existing bridge were initially considered to be rehabilitated. Consequently, the emergency temporary detour Acrow bridge was proposed and constructed north of existing bridge in September/October 2015. The geotechnical investigation for the temporary bridge (Phase I) was performed by **exp** on September 3 to10, 2015, and it included drilling of BH1, BH7, BH8 and BH9 shown on Drawing 1 in Appendix B. The memorandum with foundation recommendations for the detour Acrow bridge was issued on September 10 2015 (Appendix H). Following this investigation and construction of the temporary bridge, MTO decided to demolish the existing bridge and build the new bridge. Based on our correspondences with MTO and the preliminary GA drawing provided by MTO, it is understood that the new bridge will be a 28 m long single span bridge at the similar location as the existing bridge with the west abutment at Sta. 14+100 and the east abutment at Sta. 14+128. Therefore, the new bridge will be about 15 m longer than the existing structure, and the new abutments will be set back relatively to existing as shown on Drawing 1 in Appendix B. It is proposed that the alignment of the new bridge will be shifted either approximately 0.5 m to the south to allow use of 2.0 m shoulders on the bridge or approximately 1.0 m to the south to allow standard 2.5 m shoulders on the bridge. Both options of the new alignment allow for approximately 1.15 m clearance between the new bridge and temporary detour bridge. It is further understood that semi-integral and/or integral abutment options are considered for the new bridge with a grade raise of approximately 0.25 m. In addition, an approximately 40 m long retaining wall is proposed along the south side of the bridge approach embankment at the west side of the river. The geotechnical investigation for the new bridge (Phase 2) was performed on November 18 to 23, 2015, and it included drilling of BH2, BH3, BH4, BH5 and BH6 shown on Drawing 1 in Appendix B.

The purposed of this geotechnical investigation is to examine the existing soil conditions within the construction limits for the new bridge replacement. The site specific geotechnical investigation consisted of borings, soil sampling, borehole logging, and field and laboratory testing.

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

1.2 Site Description and Geological Setting

1.2.1 Site Description

The Rosseau River Bridge is located approximately 4.9 km east of the junction of Hwy 632, Rosseau, Ontario on Hwy 141. At the Rosseau River Bridge location, Hwy 141 is two-lane. The existing structure is single span steel girders concrete deck bridge, and is about 13.1 m in length and about 8.2 m wide double lanes. The approaches are about 10 m wide from shoulder to shoulder. The site plan and cross-section profiles for the Rosseau 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 Hwy 141 runs in a generally east to west direction, and Rosseau River flows from north to south towards the Rosseau Lake. At the time of investigation, September 2015, approximate river water elevation was 254.9 m and the elevation of top of the existing bridge deck was approximately 258.8 m.

The banks of the river in the vicinity of the bridge contained gravel, cobbles and boulders. Vegetation in the area consists of deciduous and coniferous trees and smaller low lying shrubs and grass. Bedrock outcrops were observed in the vicinity of site and riverbed. The drainage in the area consists of roadside ditches which drain into the Rosseau River. Selected photographs of the site are provided in Appendix A.

1.2.2 Geological Setting

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

In accordance with the Ministry of Northern Development and Mines Map 2544, Bedrock Geology of Ontario, Southern Sheet, the bedrock at the site consists of magmatic rocks and gneisses of undetermined protolith. Commonly layered biotite gneisses and migmatitites; locally includes quartzofeldspathic gneisses, orthogneisses, and paragnesisses.

1.3 Investigation Procedures

1.3.1 Site Investigation and Field Testing

The fieldwork for this project was carried out in two phases: Phase 1 - from September 3 to 10, 2015, and Phase 2 - from November 18 to 23, 2015. Prior to the field work commencement the clearances for existing utilities/services were provided by MTO. The investigation consisted of a total of 10 sampled boreholes (BH1, BH2, BH3, BH4, BH5, BH6, BH7, BH7A, BH8 and BH9). Boreholes BH1, BH7, BH7A, BH8 and BH9 were drilled during the geotechnical investigation for the detour Acrow bridge (Phase 1), while boreholes BH2, BH3, BH4, BH5 and BH6 were drilled during the geotechnical investigation for the new bridge (Phase 2).

Borehole BH1 was drilled through existing bridge deck close to the existing west abutment and was advanced to a depth of 20.5 m below the bridge deck. BH3, BH5 and BH6 were advanced at the abutment locations of new replacement bridge to depth between 7.3 m and 19.8 m below the existing road surface. BH7, BH7A and BH8 were advanced at the abutment locations of temporary modular bridge to depths of 5.8 m, 8.8m and 16.5 m respectively below ground surface. BH7 was terminated at a depth of 5.8 m due to spoon broke off hitting hard surface. However, BH7A was drilled adjacent to BH7, approximately 3 m east from BH7, to confirm the bedrock. BH9 was advanced at west approach of temporary detour to depth of 4.4 m below ground surface. BH2 and BH4 were advanced at the locations of retaining wall to depths of 10.8 m and 6.3 m respectively below the existing road surface. The locations of the boreholes are shown on Drawing 1 in Appendix B.

Phase 1 boreholes were advanced using a CME-75 truck mounted drill rig operated by Canadian Soil Drilling, while Phase 2 boreholes were advanced using a CME-55 truck mounted drill rig operated by Marathon Drilling Co. Ltd. Both drills were equipped with continuous flight hollow stem augers and standard soil/bedrock sampling equipment.

The borehole locations (referenced to the MTM NAD83 coordinate system) and their ground surface elevations were temporary surveyed by **exp** personnel using the Temporary Benchmark (TBM) on the nail in a temporary barrier (see Drawing 1 in Appendix B). Elevation of Temporary Benchmark (TBM) (Elev. 258.94 m) on the temporary barrier on the site (see Photograph 8, Appendix A) was provided by MTO Contract Administrator (CA).

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 HQ3 (Phase 1) and NQ (Phase 2) 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 tests were also performed, but all tested samples found non-plastic. 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 are presented graphically in Appendix D.

1.4 Subsurface Conditions

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

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

In general, the subsurface conditions along the new bridge and temporary bridge location consist of a layer of sand and gravel to sand fill underlain by native deposits of silty sand to sand layer followed by sand layer and bedrock. A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

1.4.1 Asphalt

Asphalt was encountered at the surface of all boreholes except boreholes drilled for temporary bridge investigation (BH7, BH8 and BH9). Thickness of the asphalt layer was between 80 mm to 90 mm. Asphalt thicknesses may further vary beyond the borehole location.

1.4.2 Fill: Sand and Gravel to Sand

Sand and gravel to sand fill was encountered below the asphalt in boreholes BH2, BH3, BH4, BH5 and BH6 and at the surface of boreholes BH7, BH8 and BH9. The thickness of this layer ranged from 1.4 m to 4.4 m extending from Elev. 258.9 m to Elev. 253.8 m. Borehole BH9 is terminated within this layer.

The composition of this fill layer is sand and gravel with occasional cobbles and boulders, and trace to little silt and clay size particles. The material is brown to grey in color, and moist. The SPT "N" values within this layer ranged from 2 blows per 300 mm penetration to 50 blows per 140 mm penetration, suggesting very loose to very dense compactness condition.

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

Moisture Content:

- 3.1% to 21.1%

Grain Size Distribution:

- 4% to 19 % gravel;
- 62% to 73% sand; and
- 12% to 34% 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 Cobbles and Boulders

Cobbles and boulders layer was encountered below the sand and gravel to sand fill in borehole BH6 and below silty sand to sand in BH7A. The thickness of this layer is approximately 1.4 m to 1.7 m extending from Elev. 255.7 m to Elev. 254.1 m.

The composition of this layer is cobbles and boulders with some silt and coarse gravel. The combination of Standard Penetration Tests and coring was attempted to obtain their samples. Based on the recovered cored samples, the boulder size is estimated to be up to 240 mm in diameter (see Photographs 4 and 6 in Appendix E).

1.4.4 Sand and Gravel

Native sand and gravel layer was encountered below the sand and gravel to sand fill in boreholes BH4, BH5 and below the cobbles and boulders layer in borehole BH6. The thickness of this layer ranged from 0.8 m to 2.3 m extending from Elev. 256.4 m to Elev. 251.8 m.

The composition of this layer is sand and gravel with trace to little silt and clay size particles. The material is brown in color, and moist to wet. The SPT “N” values within this layer ranged from 21 blows per 300 mm penetration to 50 blows per 80 mm penetration, suggesting compact to very dense compactness condition.

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

Moisture Content:

- 7.7% to 17.5%

Grain Size Distribution:

- 27% to 33 % gravel;
- 54% to 59% sand; and
- 13% to 14% 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.

1.4.5 Silty Sand to Sand

Native silty sand to sand layer was encountered below the sand and gravel to sand fill in boreholes BH2, BH7 and BH8, below the sand layer in borehole BH3, below the sand and gravel layer in BH4 and at the bottom of the river in BH1. The thickness of this layer ranged from 2.7 m to 7.7 m extending from Elev. 256.6 m to Elev. 246.9 m. Boreholes BH2 and BH7 are terminated within this layer.

The composition of this layer is sand trace to some silt, trace to some clay, trace gravel and occasional cobbles and boulders. The material is brown in color, and moist to wet. The SPT “N” values within this layer typically ranged from 4 to 77 blows per 300 mm penetration suggesting very loose to very dense compactness condition. Some SPT “N” value of 20 blows per 140 mm penetration (BH4) to 100 blows per 280 mm penetration (BH7) was encountered as well. It is suspected that the high SPT “N” values could be the influence of boulders or underlying bedrock.

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

Moisture Content:

- 8.3% to 30.4%

Grain Size Distribution:

- 0% to 12 % gravel;
- 24% to 80% sand;
- 13% to 66% silt and
- 6% to 37% 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 3a and 3b in Appendix D.

1.4.6 Sand

Native sand layer was encountered below the silty sand to sand layer in boreholes BH1, BH3 and BH8. In BH3 sand layer is also encountered below the sand and gravel to sand fill layer at depth 1.5 m below ground surface extending from Elev. 257.2 m to Elev. 255.6 m. The thickness lower sand layer ranged from 4.6 m to 6.2 m extending from Elev. 248.0 m to Elev. 241.1 m. Borehole BH8 is terminated within this layer.

The composition of this layer is mostly sand with trace to some silt, trace clay and trace gravel. The material is brown to grey in color, and moist to wet. The SPT “N” values within this layer typically ranged from 1 to 50 blows per 300 mm penetration suggesting very loose to very dense compactness condition. One SPT “N” value of 40 blows per 80 mm penetration was encountered at BH3 at a depth of 12.2 m below the ground surface.

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

Moisture Content:

- 9.3% to 21.2%

Grain Size Distribution:

- 1% to 10 % gravel;
- 54% to 88% sand;
- 2% to 43% 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 4 in Appendix D.

1.4.7 Bedrock

The presence of bedrock, at approximately between 5.8 m to 17.5 m below the existing road surface was recorded. The bedrock was inferred from auger/split spoon refusal in BH2 and BH7, or confirmed using coring of 0.5 m to 4.6 m long. The elevation of the inferred or actual bedrock surface below Hwy 141 ranges from Elev. 253.1 m to Elev. 241.3 m. The inferred or 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
BH1	17.5	241.3	Bedrock Cored
BH2	10.8	247.7	Inferred/ Spoon Refusal
BH3	15.3	243.4	Bedrock Cored
BH4	5.8	252.9	Bedrock Cored
BH5	6.1	252.8	Bedrock Cored
BH6	7.1	251.8	Bedrock Cored
BH7	5.8	253.1	Inferred/ Spoon Refusal
BH7A	4.6	254.3	Bedrock Cored

Based on the bedrock cores recovered, the bedrock consists of granite gneiss. In general, the bedrock samples are described as light grey, black and pink in colour and have a fine crystalline structure, slightly weathered. The Rock Quality Designation (RQD) measured on the core samples typically ranged from approximately 84% to 100%, indicating a rock mass of good to excellent quality.

1.5 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 1.2 and borehole logs. Water levels measured in open boreholes might not be stabilized due to a short term observation.

At the time of investigations, the water level measured at the river was approximately at Elev. 254.9 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 1.2 Groundwater levels recorded at the site

Borehole	Location Relative to Existing Bridge	Date of Drilling	Groundwater Level (Elevation, m)
BH1	Bridge deck (near west abutment)	09/03/2015	254.9
BH2	West Approach/ Retaining Wall	11/21/2015	254.7
BH3	West Abutment	11/20/2015	253.5
BH4	Retaining Wall	11/19/2015	254.5
BH5	East Abutment (EBL)	11/19/2015	255.2
BH6	East Abutment (WBL)	11/18/2015	255.2
BH7	East Abutment (Temporary Bridge)	09/10/2015	255.8
BH7A	East Abutment (Temporary Bridge)	09/11/2015	-
BH8	West Abutment (Temporary Bridge)	09/09/2015	253.7
BH9	West Approach	09/10/2015	255.8

2 DISCUSSIONS AND ENGINEERING RECOMMENATIONS

2.1 General

This section of the report provides geotechnical design recommendations for Rosseau River Bridge replacement on Hwy 141, located approximately 4.9 km east of the junction of Hwy 632. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the current investigation at the site and presented in **Part I-Foundation Investigation Report.** The interpretation and recommendations provided are intended solely to permit designers to assess foundation alternatives, and design the proposed 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)* (2006), the *provisions in the TOR* and good practice. It also provides discussion about the structure foundation type and stability analyses, as requested in the TOR.

2.2 Geotechnical Design Considerations for Structure Foundations

In general, the site is underlain by sand and gravel fill and cohesionless native deposits overlying bedrock. At the east side of the river the native deposits, consisting predominantly of layers of sand and gravel and cobbles and boulders extend to approximately between 6.1 m to 7.1 m below the existing ground surface. The compactness of these deposits varies from compact to very dense. The bedrock was encountered at between 6.1 m to 7.1 m depth on the east side of the river. At the west side of the river the native deposit consisting predominantly of layers of silty sand to sand extend to approximately between 5.8 m to 17.5 m below the existing ground surface. The compactness of these deposits varies from very loose to very dense. At the proposed location of the west abutment the bedrock was encountered at 15.3 m depth (BH3). The groundwater level encountered at the site was at an approximate elevation of 255 m (i.e. water level in the river).

2.2.1 Foundation Alternatives

Due to difference in stratigraphy at the west and east sides of the river including the depth to bedrock the proposed bridge can be supported either on integral abutments with deep foundations at the both sides or semi-integral abutments with deep foundations at the west side and semi-integral with shallow foundations on the east side. 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
End bearing steel H- pile driven to unyielding bedrock	▪ High geotechnical resistance available ▪ Negligible or minimum settlement ▪ Compatible for integral and semi-integral abutment	▪ May pose difficult driving condition through cobbles and boulder or possibility of piles "hanging up" on cobbles and boulders deposit ▪ May required pre-drilled to achieve sufficient fixity particularly on east abutment	▪ High	▪ Risk of pile tip damage, should adequately protected while driving through cobbles and boulders ▪ Variation in pile tip elevations	1
Spread footing supported on engineered fill over sand and gravel and/or cobbles and boulders	▪ Straightforward construction ▪ Less concrete for foundation ▪ Less expensive than other option	▪ Deeper excavation or below water excavation may required ▪ Dewatering system required ▪ Require granular materials	▪ Likely lowest cost	▪ Risk of unacceptable differential settlements if the entire foundation is not supported on the competent soil ▪ Higher scour risk	2
Spread footing supported on tremie concrete over bedrock	▪ Straightforward construction ▪ High geotechnical resistance available ▪ Reduce scour potential	▪ Deeper excavation or below water excavation may required ▪ Potential dewatering system required ▪ Require more concrete for uneven bedrock surface ▪ Limited supplier and availability	▪ Likely more expensive than option 2	▪ Risk concrete segregation if the proper pouring technique and concrete slump is not maintained	3

At the time or writing this report, it has not been determined what type of earth retaining structure will be employed, or if one is required at all. However, if used, it is recommended to use the shallow foundations to support the retaining wall.

2.2.2 Integral Abutments

Considering the site specific conditions, 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 will be installed through the upper loose to compact sandy deposits, and are expected to terminate on bedrock at the west side or socketed into bedrock at the east side. Based on the depth to bedrock encountered in the deep boreholes drilled at the locations of the proposed structure (BH1, BH2 and BH3 at the west side, and BH5 and BH6 at the east 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 2.1. It should be noted that the minimum length of the pile above the bedrock surface should be 5 m. It is anticipated that pile cap elevations would be below a frost depth of 1.8 m (approximate Elev. 257.0 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 2.2. These values represent the structural capacity of the steel member having a steel yield strength of 300 MPa, rather than a geotechnical limitation. It is anticipated that for H-piles driven and seated on the underlying unyielding bedrock, the geotechnical resistance at SLS for 25 mm of settlement will be greater than the factored axial resistance at ULS; as such, ULS conditions will govern for this foundation type.

Table 2.2. Factored geotechnical resistances for considered piles

<table>
<tr><th rowspan="2">Abutment</th><th rowspan="2">Pile Founding Stratum</th><th rowspan="2">Estimated Tip Elevation (m)</th><th rowspan="2">Approx. Design Pile Length (m)</th><th colspan="2">Factored Geotechnical Axial Resistance at ULS (kN/pile)</th><th colspan="2">Geotechnical Axial Resistance at SLS (kN/pile)</th></tr>
<tr><th>HP 310 x 79</th><th>HP 310 x 110</th><th>HP 310 x 79</th><th>HP 310 x 110</th></tr>
<tr><td>West</td><td>Bedrock</td><td>~243.4</td><td>13.0</td><td rowspan="2">1,450</td><td rowspan="2">2,000</td><td rowspan="2">NA</td><td rowspan="2">NA</td></tr>
<tr><td>East</td><td>~ 1.5 m Socketed into Bedrock</td><td>~252.3 (or ~250.8 in 1.5 m deep socket)</td><td>5.0*</td></tr>
</table>

Notes:
**has to be min 5 m*

NA-not applicable since for H-piles driven and seated on the underlying unyielding bedrock, the geotechnical resistance at SLS for 25 mm of settlement will be greater than the factored axial resistance at ULS and ULS conditions will govern

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 loose to compact silty sand 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 2.3. Subgrade reaction reduction factors for other pile spacing values may be interpolated for pile spacing in between those listed in this table.

Table 2.3. 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

Negative Skin Friction (Downdrag Loads) on Piles

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.

Pile Installation

Piles will be driven to bedrock and they should be installed in accordance with OPSS 903. The possibility of piles encountering cobbles and boulders in the soil layers at the west side of the river and presence of the cobblers and boulders layer encountered in BH6 on the east side 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.

For piles to be driven to bedrock, set criteria are dependent on the type of pile driving hammer and selected piles. The type of driving hammer depends on the Contractor. However, once the hammer and pile types are known, a wave equation (WEAP) analysis could be performed prior to driving piles in order to assess the driving stresses and the anticipated penetration resistance required to develop the required pile capacity. Therefore, a pile hammer should develop sufficient energy to efficiently drive the piles to the requisite driving resistance compatible with the design loads, yet limit the input energy so as not to overstress the pile during driving. The final driving resistance required to achieve the design load could be determined by the Pile Driving Analyzer. At least one CAPWAP (Case Pile Wave Analysis Program) Analysis should be performed on a selected records obtained in monitoring works of each pile sizes from each day of testing at each abutment locations. Dynamic testing (PDA testing) on a number of piles with the Pile Driving Analyser must be performed near the beginning of the pile driving phase of construction to confirm the pile capacities. Alternatively, static load tests can be performed, although these are typically much more difficult to set up and are more costly. The advantage of doing load test, either static or dynamic is that the higher usable capacity from a pile can be gotten.

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

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

2.2.3 Semi-integral Abutments

2.2.3.1 **East Abutment**

Based on geotechnical data encountered on the east side of the river a semi-integral abutment of the proposed bridge at that side of the river can be founded on shallow foundations set (i.e. spread footing) on a pad of granular engineered fill developed over the native compact to very dense sand and gravel, and/or very dense cobbles and boulders deposits, or on a pad of tremie concrete developed over bedrock. Spread footings which meet a requirement for an adequate protection against frost penetration in the project area of a minimum 1.8 m depth below the lowest surrounding area will be founded on native compact to very dense sand and gravel and/or cobbles and boulders. However, considering the presence of the cobbles and boulders layer and the possibility of an irregular or uneven surface, it is recommended that the footing be placed on an approximately 1.5 m thick granular engineered fill, set on the undisturbed native deposits of sand and gravel and/or cobbles and boulders. This granular pad will provide a smother foundation area for the spread footing to support the abutment. In addition less concrete will be used for the foundation as the footing will be shallower. The placement of engineered fill materials below the footings can minimize the risk of any differential settlements as well. Footings set at the higher level are, of course, subject of demonstration off satisfactory stability of the adjacent slopes. In that case high compressive strength polystyrene can be used to provide the required frost protection. It is recommended that the excavation for the footing on the granular pad should be above the groundwater level encountered at the site (i.e. above ~Elev. 255 m).

The other option of the shallow foundation is to excavate the native deposits to the bedrock surface and place the spread footing on the concrete pad raised above the groundwater level. This option requires more excavation below the groundwater level and placing of tremie concrete. More concrete will be required since the spread footing will be deeper

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 1.8 m below the lowest surrounding area), the following founding elevations of spread footings are recommended:

Table 2.4 Recommendations for footing depth

Material at Founding Level	**Foundation Elevation (m)**	**Foundation Depth Below Existing Grade**
Engineered Fill (1.5 m thick) over Native Compact to Very Dense Sand and Gravel and/or Cobbles and Boulders	257.1	min 1.8 m (+ 1.5 m excavation for engineered fill)
Tremie Concrete over Bedrock	255.1	3.9 m (+ 2.3 to 3.3 m excavation for tremie concrete pad)

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 levels given in Table 2.4, should be designed based on the factored resistances at ULS and geotechnical reactions at SLS for 25 mm of settlement given in Table 2.5 below. The footing width of 2 m to 3 m is assumed.

Table 2.5 Geotechnical resistance at ULS and geotechnical reaction at SLS for a 2 m to 3 m wide footing

Soil at Founding Level	Width of Footing (m)	Factored Geotechnical Resistance at ULS (kPa)	Geotechnical Reaction at SLS (kPa) (for 25 mm settlement)
Engineered Fill (1.5 m thick) over Native Compact to Very Dense Sand and Gravel and/or Cobbles and Boulders	2 to 3	525	350
Tremie Concrete over Bedrock	2 to 3	1000	1000*

** since for tremie concrete over 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*

Since the ULS resistance and the settlement depend on the footing size and depth of embedment, the geotechnical resistances given in Table 2.5 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 2.4 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 2.6. A factor of 0.8 should be applied in calculation of the horizontal resistance in accordance with CHBDC.

Table 2.6 Recommendations for coefficient of friction

Interface	Coefficient of Friction, tan δ*
Concrete and engineered fill	0.55
Concrete and tremie concrete	0.65

**- based on NAVFAC 1986, Table 1, pg. 7.2-63*

2.2.3.2 **West Abutment**

As mentioned above, semi-integral abutment with deep foundations can be used to support proposed bridge at west abutment. The semi- integral abutment with deep foundations can be designed based on geotechnical axial resistance provided in Section 2.2.2 above.

can be designed based on the earth pressure coefficients and soil parameters provided in Section 2.3.4, following.

2.2.4 Retaining Wall

Based on the results of the geotechnical investigation at the location of the proposed retaining wall, the fill material of the existing embankment was between 2.3 to 3.1 m thick. It is underlain by sand and gravel and silty sand to sand layers which are followed by bedrock. In BH4 (the west end of the proposed wall), the underlying bedrock was encountered at a depth of 5.8 m below the existing ground surface (Elev. 258.7 m), while in BH2 (~25 m east of BH4) the practical refusal was found at a depth of 10.8 m below the ground surface (Elev. 247.7 m).

Based on general arrangement sketch for proposed bridge provided by MTO, it is understood that, on south side of west approach embankment the retaining wall (STA 14+060 to STA 14+100) will used. However, at the time or writing this report, it has not been determined what type of earth retaining structure will be employed. It is recommended that this wall (if used) to be founded on shallow foundations (i.e. strip footing). Based on the results of the geotechnical investigation and a requirement for adequate protection against frost penetration in the project area (i.e. minimum 1.8 m), the founding elevation of the strip footing at Elev. 255.5 m is recommended. The strip footing placed on the properly prepared subgrade at the design level, should be designed based on the factored resistances at ULS of 200 kPa and geotechnical reactions at SLS of 125 kPa for 25 mm of settlement. The unfactored values of the coefficient of friction presented in Table 2.5 could be applied in the design of the footing for the retaining wall as well.

2.3 Frost Protection

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

2.4 Lateral Earth Pressure on Structures

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 2.7 can be used (assuming wall friction is neglected, the back wall is vertical and the ground surface is horizontal both on the retained side as well as in front of the toe):

Table 2.7 Material types and unfactored earth pressure properties under static conditions

Material	Unfactored Friction Angle ϕ' (^{o})	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
Granular B, Type II	32	0.31	3.25	0.47	21
Native Compact Silty Sand to Sand	32	0.31	3.25	0.47	20

2.5 Earthquake Considerations

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

Seismic characterization of the site must be compliant with CHBDC (CAN/CSA-S6-14). From the Natural Resources Canada website, 2015 NBCC seismic hazard values are obtained using the site location coordinates (45.2390 N, 79.5838 W) and the damped reference spectral accelerations for the project site are Sa(0.2)=0.054g, Sa(0.5)=0.039g, Sa(1.0)=0.023g, Sa(2.0)=0.011g and the reference peak ground acceleration (PGA) is 0.03g (g=acceleration due to gravity -9.81 m/s^2). These values are associated with an earthquake having 10 percent probability of exceedance in a 50-year period.

2.6 Stability and Settlement Analyses

2.6.1 Stability

To assess the global stability of the forward slopes of the east and west abutments and to check that a minimum Factor of Safety (FOS) of 1.5 for static conditions and 1.1 for seismic conditions will be achieved a series of slope stability analyses were performed based on the GA drawings provided by MTO. The static and seismic slope stability analyses were performed using the Morgenstern-Price method developed on the basis of limit equilibrium. The SLOPE/W computer program developed by GeoSlope International was employed for computation.

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

Tabulated below in Table 2.8 are the soil parameters used for the slope stability analyses. The soil parameters were generally estimated based on the results of field and laboratory investigation.

Table 2.8 Soil properties used in slope stability analyses

Material Type	Effective Stress Parameters		
	ϕ' (degrees)	**c' (kPa)**	**γ (kN/m³)**
Engineered fill – Granular A or Granular B Type II	32	0	21
Sand and Gravel (Compact to Very Dense)	35	0	21
Silty Sand to Sand (Compact)	32	0	20
Sand (Compact)	32	0	20.5
Cobbles and Boulders (Very Dense)	38	0	22
Rockfill	42	0	18

The results of the slope analyses for the east and west abutments are presented on Figures F1 to F6 in Appendix F.

As shown on the figures, the results of stability analyses suggest that the FOS greater than required (FOS 1.5 for static conditions and 1.1 for seismic conditions) can be obtained for the forward slopes of approximately 2H:1V at the east side and 15H:1V at the west side. Suitable erosion and scour protection measures should also be provided to the river banks adjacent to the bridge. 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.

2.6.2 Settlement

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

2.7 Construction Considerations

2.7.1 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. compact gravel and sand fill and loose to compact sandy silt) 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.

2.7.2 Temporary Shoring

Temporary excavation support systems, 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.

2.7.3 Dewatering

If the shallow foundation option with the granular pad is chosen, it is recommended that the bottom of excavation would be terminated no more than approximately 0.5 m above the groundwater to minimize disturbance and permit compaction of the exposed surface. It is anticipated that the amount of perched water within the upper granular fill materials at the abutment locations would be limited. For excavations through the soils at the abutment locations, groundwater control will likely be limited to diverting surface runoff and sump pumping. However, during the deeper excavation at the east side of the river to accommodate the concrete pad, if that option is chosen, a significant groundwater inflow could be expected. Therefore, the tremie concrete placement is recommended.

The design of unwatering systems for the excavations is responsibility of the Contractor who is expected to retain dewatering specialists for this task.

2.7.4 Foundation Base Preparation

As mentioned previously, the footing can be placed on a 1.5 m thick layer of engineered fill which should extend at least 1.0 meters beyond the outside edge of the founding level of the footing as shown in the attached drawing in Appendix G. Engineered fill should be placed in accordance with OPSS 501 and the attached drawing (Appendix G). The fill material should be placed in thin layers not exceeding approximately 300 mm when loose. Oversize particles larger than 120 mm should be discarded, and each fill layer should be uniformly compacted with heavy compactors, suitable for the type of fill used. The engineered fill below the footing should be compacted to 100% of its SPMDD.

Full-time geotechnical inspection and quality control (by means of frequent field density and laboratory testing) should be provided by the Geotechnical Engineer. Every lift should be evaluated by a sufficient number of tests to ensure that the level of compaction is constantly achieved and the compaction procedure is applied.

For the option with the concrete pad below the spread footing, the foundation base has to be cleaned as much as possible before the placing tremie concrete.

2.7.5 Abutment Stems Construction

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

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

3 CLOSURE

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

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

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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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: East side of existing bridge facing west

Photo 2: Looking south from west approach

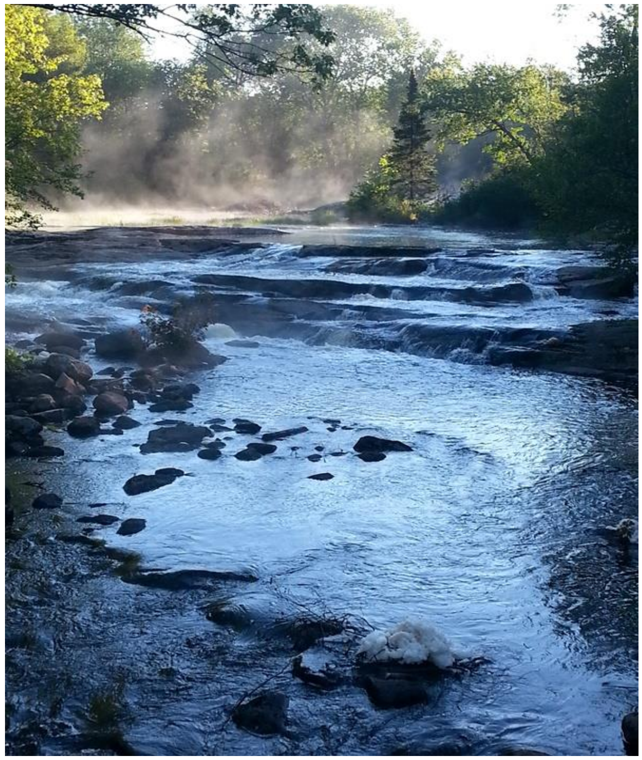


Photo 3: Looking north from existing bridge

Photo 4: Looking south from north side of existing bridge

Photo 5: Looking south from existing bridge

Photo 6: Temporary support under bridge

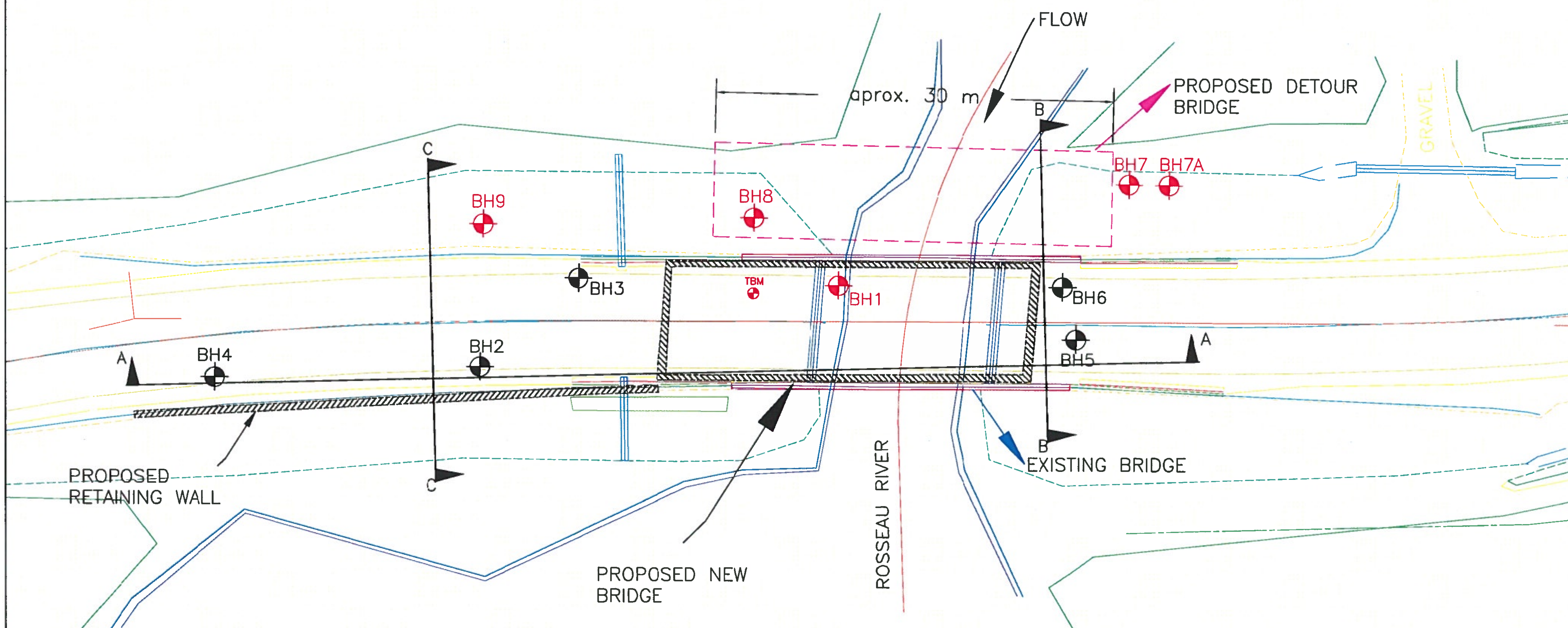
Photo 7: West approach of temporary detour looking east

Photo 8: Temporary benchmark on concrete barrier

Appendix B – Drawings

METRIC

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



Note:
The plan with proposed structures was provided by MTO.

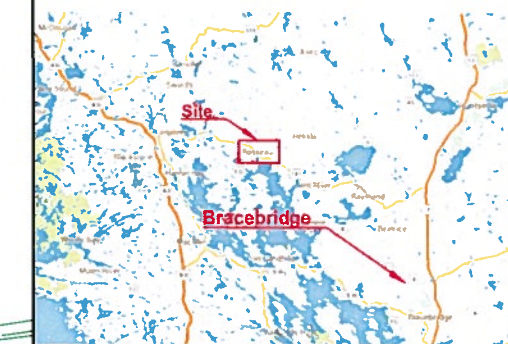
Agreement No. 5013-E-0008
Assignment No. 11
GWP No. 5394-15-00

ROSSEAU RIVER BRIDGE REPLACEMENT
(SITE NO. 42-013, HWY 141)
SITE PLAN/ BOREHOLE LOCATIONS

SHEET 1

exp Services Inc.

KEY PLAN



LEGEND

- Temporary Bench Mark (TBM)
- Location of Drilled Boreholes (Phase 1)
- Location of Drilled Boreholes (Phase 2)

BH No.	APPROX. ELEV. (m)	MTM CO-ORDINATES MTM ZONE 10	
		NORTH	EAST
TBM	258.9	5011014	298214
BH 1	258.8	5011010	298219
BH 2	258.5	5011025	298195
BH 3	258.7	5011024	298205
BH 4	258.7	5011038	298181
BH 5	258.9	5010998	298231
BH 6	258.9	5011000	298231
BH 7	258.9	5011000	298240
BH 7A	258.9	5011000	298243
BH 8	257.3	5011018	298218
BH 9	258.8	5011032	298203

NOTE

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

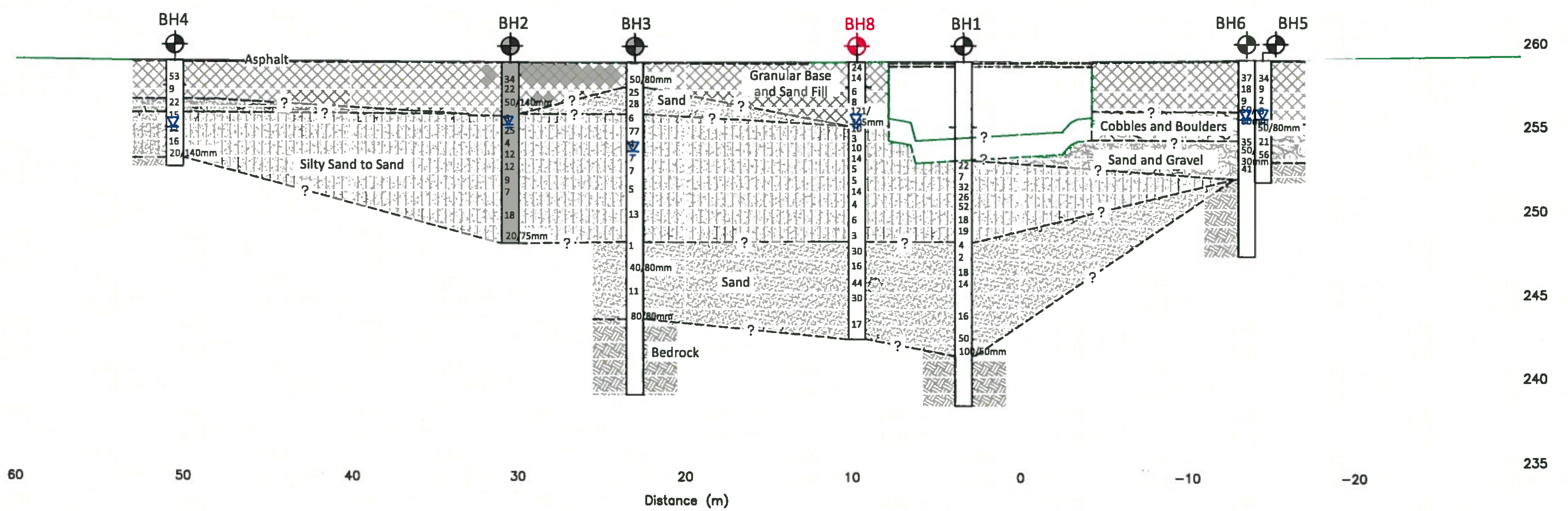
The complete foundation investigation and design report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in the report and related documents are specifically excluded in accordance with the conditions of Section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

DATE	BY	DESCRIPTION
2016.01.05	SM	SUBMISSION FOR MTO REVIEW
2015.12.11	SM	SUBMISSION FOR MTO REVIEW
2015.11.05	SM	SUBMISSION FOR MTO REVIEW
2015.09.10	SM	SUBMISSION FOR MTO REVIEW

GEOCRES NO. 31E-361
PROJECT NO. ADM-C0028245-M0

SUBM'D SM	CHECKED SM	DATE	2016.02.25
DRAWN SA	CHECKED SG	APPROVED	DWG. 01

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



SECTION A-A

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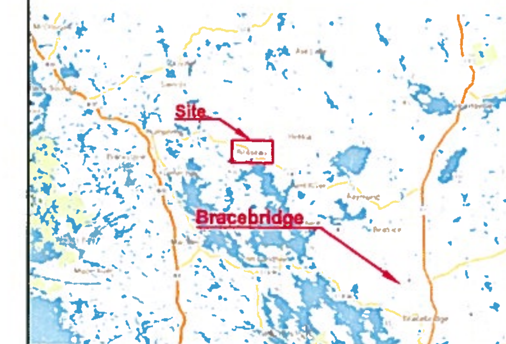
Agreement No. 5013-E-0008
Assignment No. 11
GWP No. 5394-15-00

ROSSEAU RIVER BRIDGE REPLACEMENT
(SITE NO. 42-013, HWY 141)
SOIL STRATA

SHEET
2

exp Services Inc.

KEY PLAN



LEGEND

- ASPHALT
- SAND FILL AND GRANULAR BASE FILL
- SILTY SAND TO SAND
- BEDROCK
- CONCRETE
- SAND
- SAND AND GRAVEL
- COBBLES AND BOULDER

BH No.	APPROX. ELEV. (m)	MTM CO-ORDINATES MTM ZONE 10	
		NORTH	EAST
TBM	258.9	5011014	298214
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BH 5	258.9	5010998	298231
BH 6	258.9	5011000	298231
BH 7	258.9	5011000	298240
BH 7A	258.9	5011000	298243
BH 8	257.3	5011018	298218
BH 9	258.8	5011032	298203

NOTE

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

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

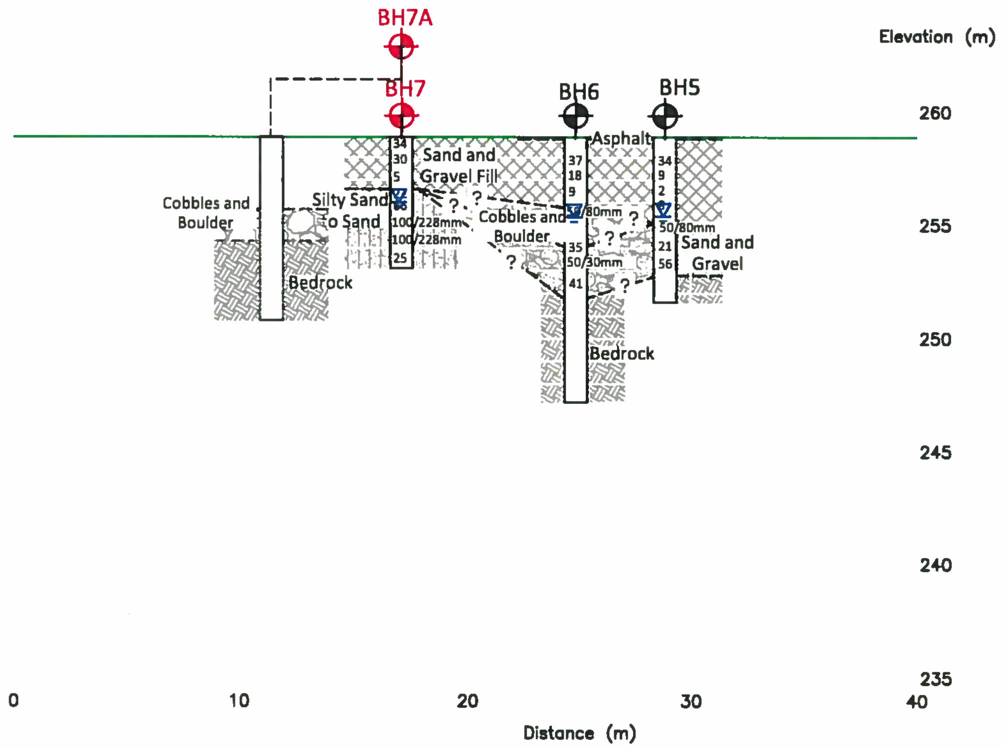
DATE	BY	DESCRIPTION
2016 01.05	SM	SUBMISSION FOR MTO REVIEW
2015.12.11	SM	SUBMISSION FOR MTO REVIEW
2015.11.05	SM	SUBMISSION FOR MTO REVIEW
2015.09.10	SM	SUBMISSION FOR MTO REVIEW

GEOCRES NO. 31E-361
PROJECT NO. ADM-00028245-M0

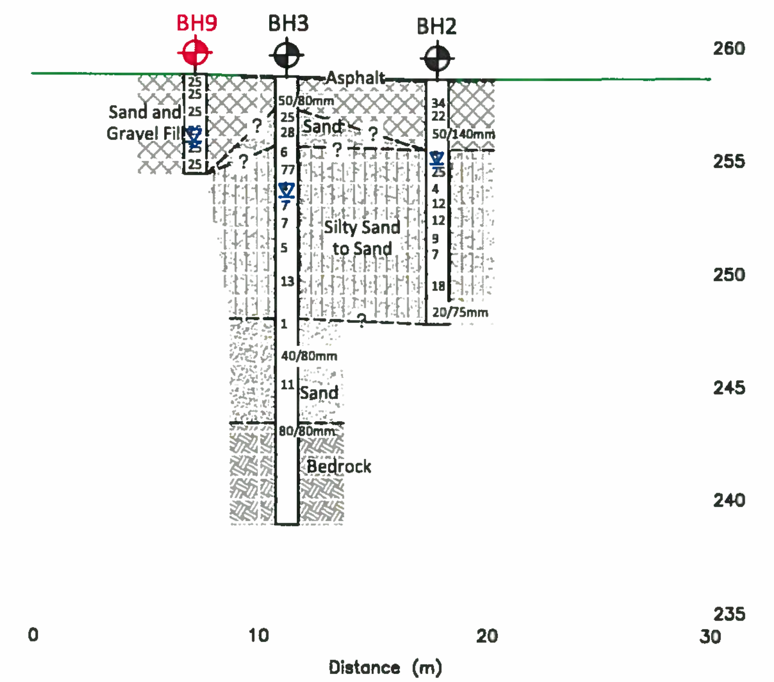
SUBM'D SM	CHECKED SM	DATE	2016.02.25
DRAWN SA	CHECKED SG	APPROVED	DWG. 02

METRIC

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



SECTION B-B



SECTION C-C

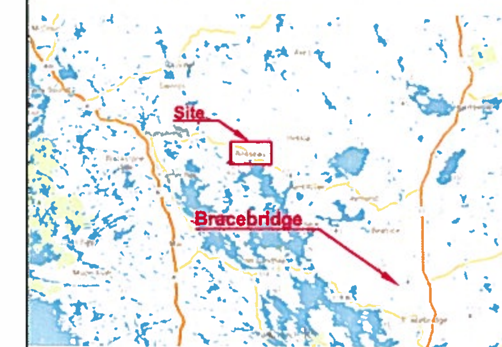
Agreement No. 5013-E-0008
Assignment No. 11
GWP No. 5394-15-00

ROSSEAU RIVER BRIDGE REPLACEMENT
(SITE NO. 42-013, HWY 141)
SOIL STRATA

SHEET
3

exp Services Inc.

KEY PLAN



LEGEND

- ASPHALT
- CONCRETE
- SAND FILL AND GRANULAR BASE FILL
- SAND
- SILTY SAND TO SAND
- SAND AND GRAVEL
- BEDROCK
- COBBLES AND BOULDER

BH No.	APPROX. ELEV. (m)	MTM CO-ORDINATES MTM ZONE 10	
		NORTH	EAST
TBM	258.9	5011014	298214
BH 1	258.8	5011010	298219
BH 2	258.5	5011025	298195
BH 3	258.7	5011024	298205
BH 4	258.7	5011038	298181
BH 5	258.9	5010998	298231
BH 6	258.9	5011000	298231
BH 7	258.9	5011000	298240
BH 7A	258.9	5011000	298243
BH 8	257.3	5011018	298218
BH 9	258.8	5011032	298203

NOTE

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

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

DATE	BY	DESCRIPTION
2016 01 05	SM	SUBMISSION FOR MTO REVIEW
2015.12.11	SM	SUBMISSION FOR MTO REVIEW
2015.11.05	SM	SUBMISSION FOR MTO REVIEW
2015.09.10	SM	SUBMISSION FOR MTO REVIEW

GEOCRES NO. 31E-361
PROJECT NO. ADM-00028245-M0

SUBM'D SM	CHECKED SM	DATE	2016.02.25
DRAWN SA	CHECKED SG	APPROVED	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		

0.002 0.006 0.02 0.06 0.2 0.6 2.0 6.0 20 60 200

EQUIVALENT GRAIN DIAMETER IN MILLIMETRES

CLAY (PLASTIC) TO	FINE	MEDIUM	CRS.	FINE	COARSE
SILT (NONPLASTIC)	SAND			GRAVEL	

UNIFIED SOIL CLASSIFICATION

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

Table a: Percent or Proportion of Soil, Pp

	Criteria
Trace	Particles are present but estimated to be less than 5%
Few	5≤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:

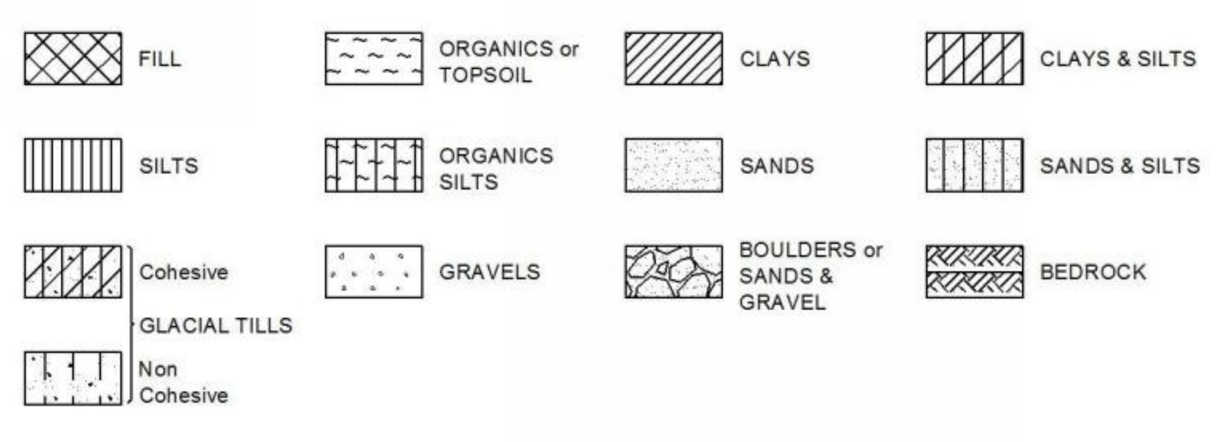
Table c: Consistency of Cohesive Soil

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 BH1

METRIC

W. P.	GWP 5394-15-00	LOCATION	Rosseau River Bridge, Rosseau, Ontario, MTM Z10, (N 5011010 E 298219)	ORIGINATED BY	CR
DIST	HWY 141	BOREHOLE TYPE	CME-75, Hollow Stem Augers/ Diamond Drill, Cased Hole	COMPILED BY	VP
DATUM	Geodetic	DATE	2015/09/03 - 2015/09/03	CHECKED BY	SM

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
258.8	**Bridge Deck**					
258.7 / 0.1	**ASPHALT:** 90mm					
258.5 / 0.3	**CONCRETE DECK**					
254.9 / 3.9	**RIVER WATER LEVEL**				∇	
252.9 / 5.9	**SILTY SAND TO SAND** some gravel, brown, very loose to very dense, wet	1	SS	22		
		2	SS	7		
	-fine to medium grained gravel, compact below 7.8 m depth	3	SS	32		12 63 (25)
		4	SS	26		
		5	SS	52		
		6	SS	18		7 80 (13)
		7	SS	19		
	-very loose below 20.5 m depth	8	SS	4		
247.5 / 11.3	**SAND** some silt, trace gravel, brown, very loose to very densewet	9	SS	2		10 88 (2)
		10	SS	18		
		11	SS	14		

Elevation scale: 258, 257, 256, 255, 254, 253, 252, 251, 250, 249, 248, 247, 246, 245, 244

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/m^3

Continued Next Page

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

RECORD OF BOREHOLE No BH1

METRIC

W. P.	GWP 5394-15-00	LOCATION	Rosseau River Bridge, Rosseau, Ontario, MTM Z10, (N 5011010 E 298219)	ORIGINATED BY	CR
DIST	HWY 141	BOREHOLE TYPE	CME-75, Hollow Stem Augers/ Diamond Drill, Cased Hole	COMPILED BY	VP
DATUM	Geodetic	DATE	2015/09/03 - 2015/09/03	CHECKED BY	SM

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SAND some silt, trace gravel, brown, very loose to very densewet *(continued)*		12	SS	16		243	○ ~20		3 54 (43)
	-some gravel, dense below 16.76 m depth		13	SS	50		242	○ ~17		
241.3			14	NR	100/ 50mm					
17.5	**BEDROCK** **HQ3 Coring** Lenght (m) RQD (%) Run1 1.6 88.0 Run2 1.4 97.0		15	HQ3			241 240			
			16	HQ3			239			
238.3 20.5	**END OF BOREHOLE at ~20.5 m depth**									

NOTES:
1. This drawing is to be read with the subject report and project numbers as presented above.
2. Interpretation assistance by exp is required before used by others.
3. Groundwater level at 3.9 m depth upon completion.

DNYAMIC 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

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

RECORD OF BOREHOLE No BH2

METRIC

W. P.	GWP 5394-15-00	LOCATION	Rosseau River Bridge, Rosseau, Ontario, MTM Z10, (N 5011025 E 298195)	ORIGINATED BY	CS
DIST	HWY 141	BOREHOLE TYPE	CME-55X, Hollow stem auger/ Diamond Drill, Cased Hole	COMPILED BY	VP
DATUM	Geodetic	DATE	2015/11/21 - 2015/11/23	CHECKED BY	SM

SOIL PROFILE

ELEV DEPTH	DESCRIPTION
258.5	**Ground Surface**
258.4 0.1	**ASPHALT:** 80mm
	FILL: SAND AND GRAVEL trace gravel, brown, dense, moist -boulder @ 0.9 m -compact below 1.5 m depth -very dense below 2.3 m depth
255.4 3.1	**SILTY SAND TO SAND** trace to some clay, brown, loose to compact, moist -some clay, trace gravel, compact, wet below 3.8 m depth -trace silt, compact to loose brown to grey, wet below 5.3 m depth -sandy silt, trace clay below 6.9 m depth -trace gravel below 9.1 m depth
247.7 10.8	**END OF BOREHOLE at ~ 10.8 m depth** Possible Bedrock

NOTES:
1. This drawing is to be read with the subject report and project numbers as presented above.
2. Interpretation assistance by exp is required before used by others.
3. Groundwater level at 3.8 m depth upon completion.
4. Borehole open upon completion.

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	34	
2	SS	22	
3	SS	50/ 140mm	
4	SS	6	
5	SS	25	8 72 (20)
6	SS	4	
7	SS	12	
8	SS	12	
9	SS	9	0 29 57 14
10	SS	7	0 65 (35)
11	SS	18	
12	SS	20/ 75mm	

GROUND WATER CONDITIONS: ▽ at approx. elev. 254.8

ELEVATION SCALE: 258, 257, 256, 255, 254, 253, 252, 251, 250, 249, 248

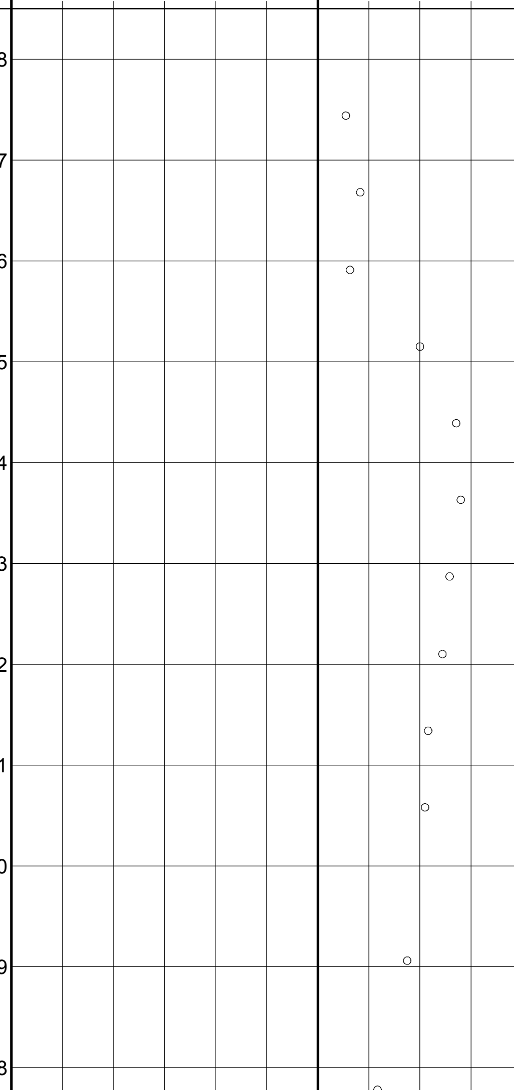
DNYAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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



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

RECORD OF BOREHOLE No BH3

METRIC

W. P.	GWP 5394-15-00	LOCATION	Rosseau River Bridge, Rosseau, Ontario, MTM Z10, (N 5011024 E 298205)
DIST	HWY 141	BOREHOLE TYPE	CME-55X, Hollow stem auger/ Diamond Drill, Cased Hole
DATUM	Geodetic	DATE	2015/11/20 - 2015/11/21
ORIGINATED BY	CS	COMPILED BY	VP
CHECKED BY	SM		

SOIL PROFILE · SAMPLES · GROUND WATER CONDITIONS · ELEVATION SCALE · DNYAMIC 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 · REMARKS & GRAIN SIZE DISTRIBUTION (%)

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
258.7	Ground Surface							
258.6 / 0.1	**ASPHALT:** 80mm							
	FILL: SAND AND GRAVEL trace gravel, brown, very dense, moist	1	SS	50/ 80mm				
257.2 / 1.5	**SAND** trace gravel, trace silt, brown/grey, compact, moist	2	SS	25	5	70	(25)	
		3	SS	28				
255.6 / 3.1	**SILTY SAND TO SAND** trace gravel, brown, loose, moist	4	SS	6				
	-trace gravel, very dense below 3.8 m depth	5	SS	77				
	-clayey silt, brown, soft below 4.6 m depth	6	SS	4	0	24	39	37
		7	SS	7				
		8	SS	7	0	34	(66)	
		9	SS	5				
	-compact below 9.1 m depth	10	SS	13				
248.0 / 10.7	**SAND** brown, very loose to very dense, wet	11	SS	1				
	-dense to compact, wet to moist below 12.2 m depth	12	SS	40/ 80mm				
		13	SS	11	1	79	(20)	

Elevation scale: 258, 257, 256, 255, 254, 253, 252, 251, 250, 249, 248, 247, 246, 245, 244

Continued Next Page

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

RECORD OF BOREHOLE No BH3

METRIC

W. P. GWP 5394-15-00	LOCATION Rosseau River Bridge, Rosseau, Ontario, MTM Z10, (N 5011024 E 298205)	ORIGINATED BY CS
DIST HWY 141	BOREHOLE TYPE CME-55X, Hollow stem auger/ Diamond Drill, Cased Hole	COMPILED BY VP
DATUM Geodetic	DATE 2015/11/20 - 2015/11/21	CHECKED BY SM

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE
243.4 / 15.3	-trace gravel below 15.2 m depth **BEDROCK** grey/black, pink granite **NQ Coring**	14	SS	50/ 80mm	243
	Lenght (m) RQD (%) Run1 1.3 84.0 Run2 1.8 94.0 Run3 1.3 96.0	15	NQ		242
		16	NQ		241
		17	NQ		240
238.9 / 19.8	**END OF BOREHOLE at ~ 19.8 m depth**				239

Column headings: SOIL PROFILE; SAMPLES; STRAT PLOT; GROUND WATER CONDITIONS; DNYAMIC 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; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

NOTES:
1. This drawing is to be read with the subject report and project numbers as presented above.
2. Interpretation assistance by exp is required before used by others.
3. Groundwater level at 5.2 m depth upon completion.

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

exp services Inc.
Brampton, Ontario

Foundation Design

RECORD OF BOREHOLE No BH4

METRIC

W. P.	GWP 5394-15-00	LOCATION	Rosseau River Bridge, Rosseau, Ontario, MTM Z10, (N 5011038 E 298181)	ORIGINATED BY	CS
DIST	HWY 141	BOREHOLE TYPE	CME-75, Hollow Stem Augers/ Diamond Drill, Cased Hole	COMPILED BY	VP
DATUM	Geodetic	DATE	2015/11/19 - 2015/11/19	CHECKED BY	SM

ELEV DEPTH	DESCRIPTION (SOIL PROFILE)	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	WATER CONTENT (%)	GR	SA	SI	CL
258.7	**Ground Surface**									
258.6 / 0.1	**ASPHALT:** 80mm									
	FILL: SAND AND GRAVEL very dense	1	SS	53		○				
	-Fill :Sand trace gravel, brown, loose, moist	2	SS	9		○	12	71	(17)	
256.4 / 2.3	**SAND AND GRAVEL** trace gravel, trace organics, brown, compact, moist	3	SS	22		○				
255.6 / 3.1	**SILTY SAND TO SAND** trace silt, trace peat, brown, compact, moist	4	SS	15		○	0	64	(36)	
	-some clay, brown, below 3.8 m depth	5	SS	11	▽	○				
		6	SS	16		○	0	36	(64)	
		7	SS	20/ 140mm		○				
252.9 / 5.8	**BEDROCK** grey/black, pink granite **NQ Coring**	8	NQ							
252.4 / 6.3	Run1 — Lenght (m) 0.5 — RQD (%) 100.0									
	END OF BOREHOLE at ~ 6.3 m depth									

Elevation scale: 258, 257, 256, 255, 254, 253

DNYAMIC 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

REMARKS & GRAIN SIZE DISTRIBUTION (%)

NOTES:
1. This drawing is to be read with the subject report and project numbers as presented above.
2. Interpretation assistance by exp is required before used by others.
3. Groundwater level at 4.2 m depth upon completion.

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

OPG_EXP RECORD OF BOREHOLE HWY 141.GPJ ONTARIO MOT.GDT 2/25/16

RECORD OF BOREHOLE No BH5

METRIC

W. P.	GWP 5394-15-00	LOCATION	Rosseau River Bridge, Rosseau, Ontario, MTM Z10, (N 5010998 E 298231)	ORIGINATED BY	CS
DIST	HWY 141	BOREHOLE TYPE	CME-55X, Hollow stem auger/ Diamond Drill, Cased Hole	COMPILED BY	VP
DATUM	Geodetic	DATE	2015/11/19 - 2015/11/19	CHECKED BY	SM

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
258.9	Ground Surface						
258.8 / 0.1	**ASPHALT:** 80mm						
	FILL: SAND AND GRAVEL TO SAND trace gravel, trace silt, brown, dense to loose, moist	1	SS	34			
		2	SS	9			18 70 (12)
		3	SS	2			
		4	SS	8	∇	40.4	
255.1 / 3.8	**SAND AND GRAVEL** trace silt, brown, compact to very dense, wet	5	SS	50/ 80mm			
		6	SS	21			
		7	SS	56			27 59 (14)
252.8 / 6.1	**BEDROCK** grey/black, pink granite **NQ Coring** Lenght (m) RQD (%) Run1 1.2 100.0	8	NQ				
251.6 / 7.3	**END OF BOREHOLE at ~ 7.3 m depth**						

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; ELEVATION SCALE (258, 257, 256, 255, 254, 253, 252); DNYAMIC 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/m^3

NOTES:
1. This drawing is to be read with the subject report and project numbers as presented above.
2. Interpretation assistance by exp is required before used by others.
3. Groundwater level at 3.7 m depth upon completion.

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

RECORD OF BOREHOLE No BH6

METRIC

W. P.	GWP 5394-15-00	LOCATION	Rosseau River Bridge, Rosseau, Ontario, MTM Z10, (N 5011000 E 298231)	ORIGINATED BY	CS
DIST	HWY 141	BOREHOLE TYPE	CME-55X, Hollow stem auger/ Diamond Drill, Cased Hole	COMPILED BY	VP
DATUM	Geodetic	DATE	2015/11/18 - 2015/11/18	CHECKED BY	SM

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
258.9	**Ground Surface**						
258.8 0.1	**ASPHALT:** 80mm						
	FILL: SAND AND GRAVEL TO SAND brown, compact to loose, moist	1	SS	37		258	15 69 (16)
		2	SS	18		257	
	-silty sand, trace gravel, trace organics, brown, loose, moist at 2.3 m depth	3	SS	9		256	4 62 (34)
255.8 3.1	**COBBLES AND BOULDERS** some gravel, some silt, very dense, moist -black with pink/white granite and sample rock (112mm) recorded below 3.3 m depth -more bolders below 4.0 m depth	4	SS	50/ 30mm	▽	255	
254.1 4.8	**SAND AND GRAVEL** trace silt, brown, dense to very dense, wet	5	SS	35		254	
		6	SS	50/ 80mm			33 54 (13)
		7	SS	41		253, 252	
251.8 7.1	**BEDROCK** grey/black, pink granite **NQ Coring**	8	NQ				
	Lenght (m) / RQD (%): Run1 0.7 / 100.0; Run2 1.5 / 100.0; Run3 1.5 / 98.0; Run4 0.9 / 100.0	9	NQ			251, 250	
		10	NQ			249	
		11	NQ			248	
247.2 11.7	**END OF BOREHOLE at ~ 11.7 m depth**						

Water content plotted (approx. %): Sample 1 ≈ 7; Sample 2 ≈ 15; Sample 3 ≈ 19; Sample 4 ≈ 24; Sample 5 ≈ 10; Sample 6 ≈ 10; Sample 7 ≈ 13.

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/m^3

NOTES:
1. This drawing is to be read with the subject report and project numbers as presented above.
2. Interpretation assistance by exp is required before used by others.
3. Groundwater level at 3.7 m depth upon completion.

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

RECORD OF BOREHOLE No BH7

METRIC

W. P. GWP 5394-15-00
LOCATION Rosseau River Bridge, Rosseau, Ontario, MTM Z10, (N 5011000 E 298240)
ORIGINATED BY CS
DIST HWY 141
BOREHOLE TYPE CME-75, Hollow Stem Augers
COMPILED BY VP
DATUM Geodetic
DATE 2015/09/10 - 2015/09/10
CHECKED BY SM

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
258.9	Ground Surface					
	FILL: SAND AND GRAVEL some cobbles, occassional boulder, grey loose to Dense	1	SS	34		
		2	SS	30		
		3	SS	5		
256.6 / 2.3	**SILTY SAND TO SAND** trace to some gravel, trace clay, trace coobles and boulders, trace organics, trace rootlets, brown, moist to very moist, compact to very dense - Boulder @ 3.05 m - becoming clayey	4	SS	22		
		5	SS	66	∇	
		6	SS	100/ 228mm		9 63 22 6
	- Bedrock @ 4.6 m on adjacent borehole BH-7A	7	SS	100/ 280mm		10 71 (19)
	-Boulder @ 5.2 m	8	SS	67		
253.1 / 5.8	- Spoon broke off					

Elevation scale: 258, 257, 256, 255, 254

DNYAMIC 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

END OF BOREHOLE
Possible Bedrock, Spoon broke off

NOTES:
1. This drawing is to be read with the subject report and project numbers as presented above.
2. Interpretation assistance by exp is required before used by others.
3. Groundwater level at 3.1 m depth upon completion.

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

RECORD OF BOREHOLE No BH7A

METRIC

W. P. GWP 5394-15-00	LOCATION Rosseau River Bridge, Rosseau, Ontario, MTM Z10, (N 5011000 E298243)	ORIGINATED BY CS
DIST HWY 141	BOREHOLE TYPE CME-75, Hollow Stem Augers/ Diamond Drill, Cased Hole	COMPILED BY NT
DATUM Geodetic	DATE 2015/09/11 - 2015/09/11	CHECKED BY SM

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: Cu, KPa (○ UNCONFINED, + FIELD VANE, × QUICK TRIAXIAL, LAB VANE)	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
258.9	**Ground Surface**										
	-Refer BH-7 for soil description, Auger flight upto boulder surface						258, 257, 256				
255.7 / 3.2	**COBBLES AND BOULDERS** some sand, some gravel, dark grey with pink/white granite boulder and sample rock (242 mm) recorded -becoming more cobbles and sand and gravel @ 3.4 m		1	HQ3			255				
254.3 / 4.6	**BEDROCK** pink and grey granite **HQ3 Coring**		2	HQ3			254				
	Lenght (m) RQD (%) Run1 0.4 90 Run2 1.5 100 Run3 1.5 100		3	HQ3			253				
			4	HQ3			252, 251				
250.8 / 8.1	**END OF BOREHOLE**										

NOTES:
1. This drawing is to be read with the subject report and project numbers as presented above.
2. Interpretation assistance by exp is required before used by others
3. No Groundwater was measured

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

RECORD OF BOREHOLE No BH8

METRIC

W. P. GWP 5394-15-00	LOCATION Rosseau River Bridge, Rosseau, Ontario, MTM Z10, (N 5011018 E 298218)	ORIGINATED BY CS
DIST HWY 141	BOREHOLE TYPE CME-75, Hollow Stem Augers/ Diamond Drill, Cased Hole	COMPILED BY VP
DATUM Geodetic	DATE 2015/09/09 - 2015/09/09	CHECKED BY SM

SOIL PROFILE | SAMPLES | GROUND WATER CONDITIONS | ELEVATION SCALE | DNYAMIC 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 | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
257.6	**Ground Surface**							
	NEW FILL: SAND AND GRAVEL some cobbles, brown, compact	1	SS	24				
	-grey to brown	2	SS	14				
	OLD FILL: SAND medium to fine grained sand, some bolders, trace gravel, trace silt, trace organics, brown, loose to compact, moist	3	SS	6				
		4	SS	8	4	73	(23)	
	- dense below 3.1 m depth	5	SS	121/ 305mm				
253.8 / 3.8	**SITLY SAND TO SAND** fine grained sand, trace gravel, trace silt, trace organics, brown, very loose to very dense, wet	6	SS	10				
		7	SS	3	0	51	(49)	
		8	SS	10				
	-seam coarse grained sand below 6.1 m depth	9	SS	14				
		10	SS	5	1	62	(37)	
		11	SS	5				
		12	SS	14				
		13	SS	4				
		14	SS	6				
246.9 / 10.7	**SAND** trace to some silt, trace clay, brown, very loose to compact, wet	15	SS	3	2	84	(14)	
		16	SS	30				
		17	SS	16				
		18	SS	44				
		19	SS	30	5	74	(21)	

Elevation scale: 257, 256, 255, 254, 253, 252, 251, 250, 249, 248, 247, 246, 245, 244, 243

OPG_EXP RECORD OF BOREHOLE HWY 141.GPJ ONTARIO MOT.GDT 2/25/16

Continued Next Page

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

exp services Inc.

Brampton, Ontario

Foundation Design

RECORD OF BOREHOLE No BH8

2 OF 2 **METRIC**

W. P. GWP 5394-15-00	LOCATION Rosseau River Bridge, Rosseau, Ontario, MTM Z10, (N 5011018 E 298218)	ORIGINATED BY CS
DIST HWY 141	BOREHOLE TYPE CME-75, Hollow Stem Augers/ Diamond Drill, Cased Hole	COMPILED BY VP
DATUM Geodetic	DATE 2015/09/09 - 2015/09/09	CHECKED BY SM

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: 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	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SAND trace to some silt, trace clay, brown, very loose to compact, wet *(continued)* compact below 15.24 m depth		20	SS	17		242		○		
241.1 / 16.5	**END OF BOREHOLE at ~ 16.5 m depth**										

NOTES:
1. This drawing is to be read with the subject report and project numbers as presented above.
2. Interpretation assistance by exp is required before used by others.
3. Groundwater level at 4.0 m depth upon completion.

OPG_EXP RECORD OF BOREHOLE HWY 141.GPJ ONTARIO MOT.GDT 2/25/16

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

RECORD OF BOREHOLE No BH9

METRIC

W. P. GWP 5394-15-00 LOCATION Rosseau River Bridge, Rosseau, Ontario, MTM Z10, (N 5011032 E 298203) ORIGINATED BY CS

DIST HWY 141 BOREHOLE TYPE CME-75, Hollow Stem Augers COMPILED BY VP

DATUM Geodetic DATE 2015/09/10 - 2015/09/10 CHECKED BY SM

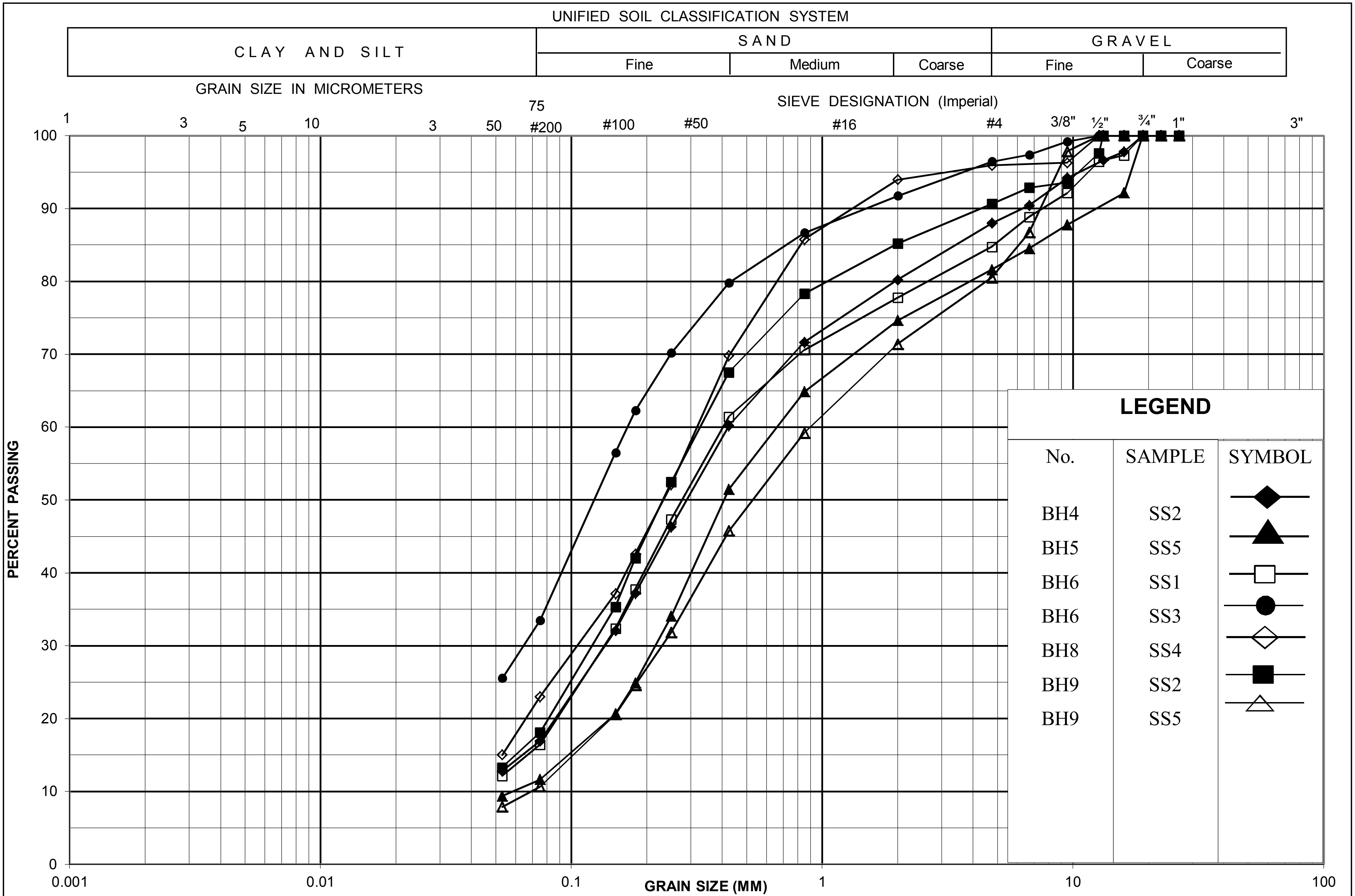
ELEV / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
258.8	**Ground Surface**						
	FILL: SAND AND GRAVEL grey, compact, moist	1	SS	23			
	-medium to fine grained sand, some gravel, brown, loose, moist below 0.8 m depth	2	SS	8		258	9 73 (18)
		3	SS	5		257	
	-seam topsoil, trace rootlets 2.3 m depth	4	SS	6		256	
	-brown, compact, wet below 3.1 m depth	5	SS	29	▽		19 70 (11)
		6	SS	19		255	
254.4 / 4.4	**END OF BOREHOLE at ~ 4.4 m depth**						

NOTES:
1. This drawing is to be read with the subject report and project numbers as presented above.
2. Interpretation assistance by exp is required before used by others.
3. Groundwater level at 3.1 m depth upon completion.

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

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

Appendix D – Laboratory Test Results

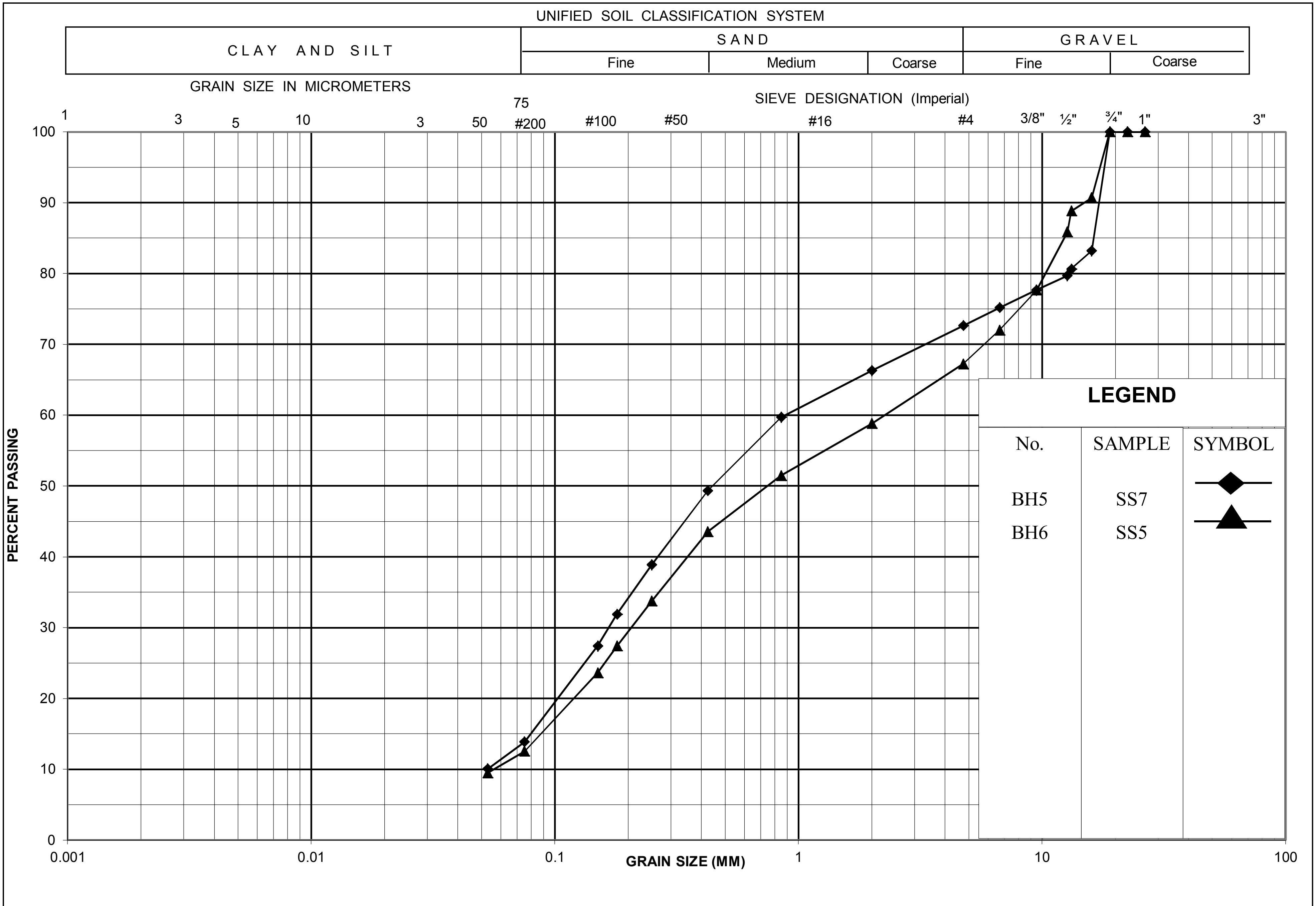


exp.

GRAIN SIZE DISTRIBUTION

FILL: SAND AND GRAVEL TO SAND

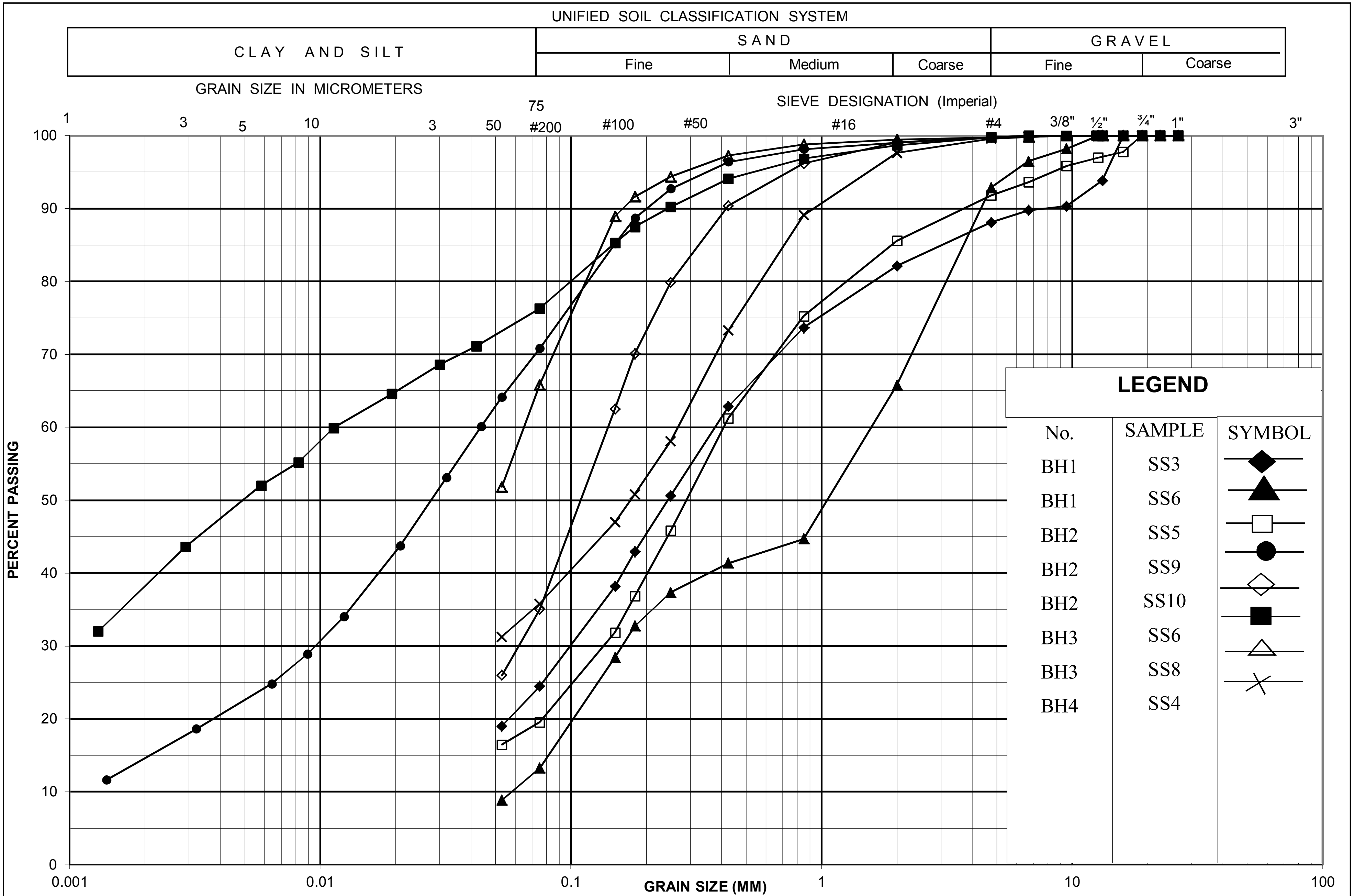
FIGURE No. 1
GWP : 5394-15-00
DATE: 1/5/2016



GRAIN SIZE DISTRIBUTION

SAND AND GRAVEL

FIGURE No. 2	
GWP :	5394-15-00
DATE:	1/5/2016



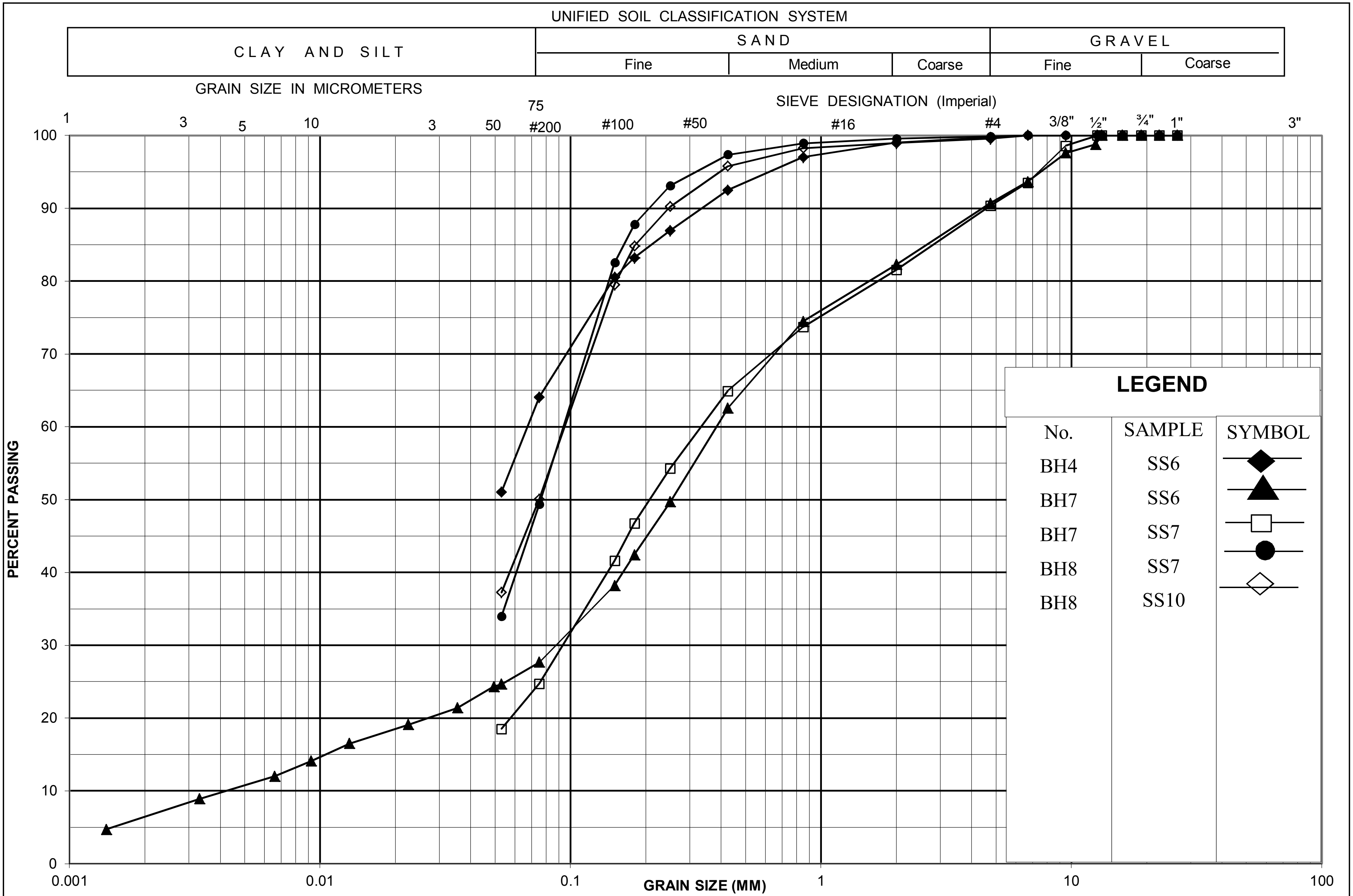
GRAIN SIZE DISTRIBUTION

SILTY SAND TO SAND

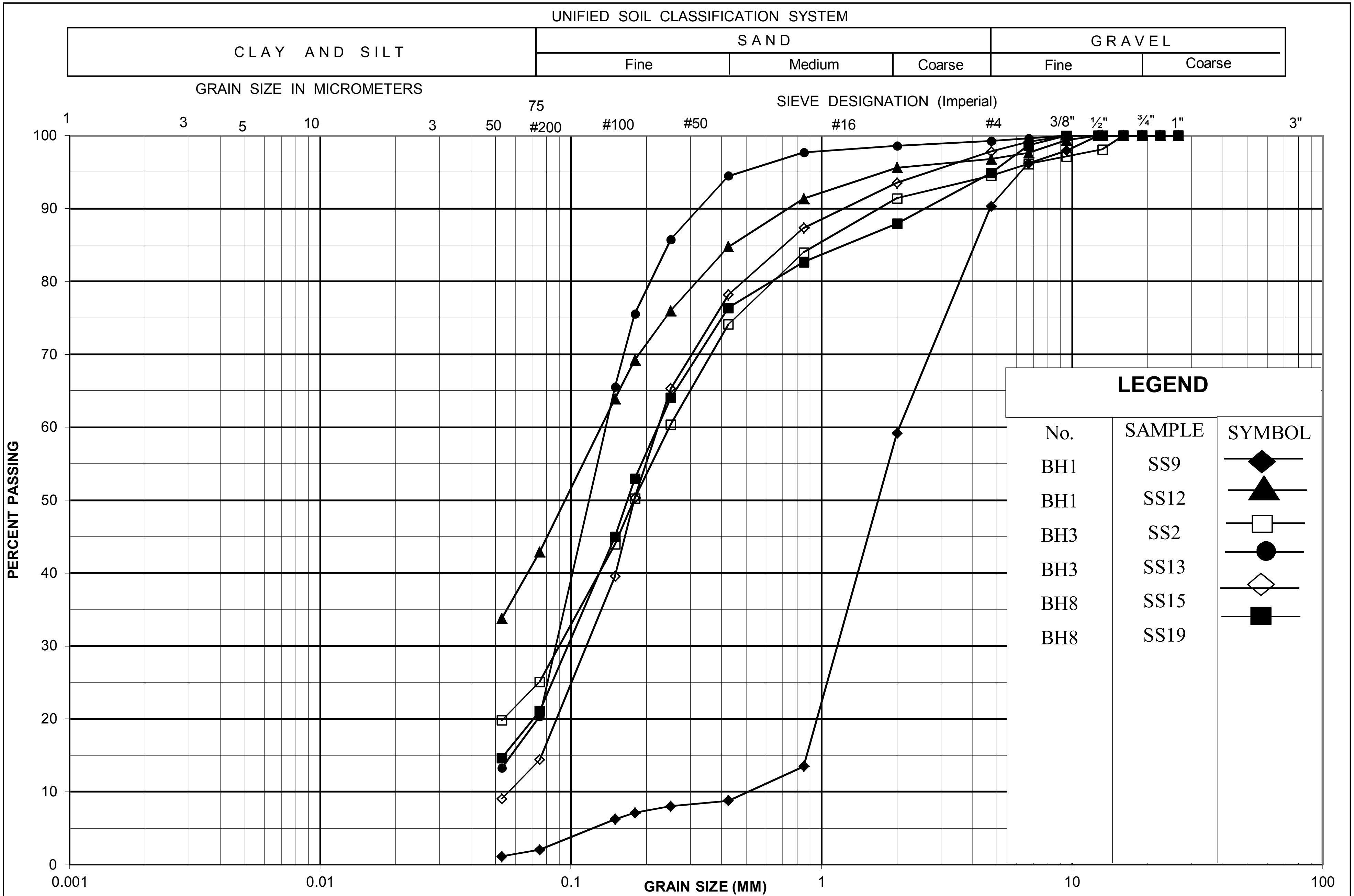
FIGURE No. 3a

GWP : 5394-15-00

DATE: 1/5/2016



exp.	GRAIN SIZE DISTRIBUTION ***SILTY SAND TO SAND***	FIGURE No. 3b GWP : 5394-15-00 DATE: 1/5/2016



exp.

GRAIN SIZE DISTRIBUTION

SAND

FIGURE No. 4

GWP : 5394-15-00

DATE: 1/5/2016

Appendix E – Bedrock Core Photographs

Project NO: ADM 00028245-M0
BH NO: 1
Run NO: 1 & 2
Sample Depth: 17.5 m to 20.5 m
Elevation: 241.3 m to 238.3 m
RQD: 88% to 97%
Date: September 03, 2015

Photo 1. Core Sample for BH1 from Elevation 241.3 m to 238.3 m

Project NO: ADM 00028245-M0
BH NO: 3
Run NO: 1, 2 & 3
Sample Depth: 15.3 m to 19.8 m
Elevation: 243.4 m to 238.9 m
RQD: 84% to 96%
Date: November 11, 2015

Photo 2. Core Sample for BH3 from Elevation 243.4 m to 238.9 m

Project NO: ADM 00028245-M0
BH NO: 4 and 5
Run NO: 1
Sample Depth: 5.8 m to 6.3m (BH4) 6.1 m to 7.3 m (BH5)
Elevation: 252.9 m to 252.4 m (BH4) 252.8 m to 251.6 m (BH5)
RQD: 100%
Date: November 19, 2015

Photo 3. Core Samples for BH4 and BH5

Project NO: ADM 00028245-M0
BH NO: 6
Cobbles and Boulders
Sample Depth: 4.0 m to 4.6m, 5.8 m to 6.2 m and 6.9 m to 7.1 m
Bedrock
Run NO: 1, 2 & 3 Sample Depth: 7.1 m to 10.0 m
Elevation: 251.8 m to 248.9 m RQD: 98% to 100%
Date: November 18, 2015

Photo 4. Core Samples BH6 Cobbles and Boulders from Elevation 255.8 m to 251.8 m and Bedorck from Elevation 251.8 m to 248.9 m

Project NO: ADM 00028245-M0
BH NO: 6
Run NO: 3 & 4
Sample Depth: 10.0 m to 11.7 m
Elevation: 248.9 m to 247.2 m
RQD: 98% to 100%
Date: November 18, 2015

Photo 5. Core Sample for BH6 from Elevation 248.9 m to 247.2 m

Project NO: ADM 00028245-M0
BH NO: 7A
Cobbles and Boulders
Sample Depth: 3.2 m to 3.5 m, 3.5 m to 4.6 m
Bedrock
Run NO: 1 Sample Depth: 4.6 m to 5.0 m
Elevation: 254.3 m to 253.9 m RQD: 90%
Date: September 11, 2015

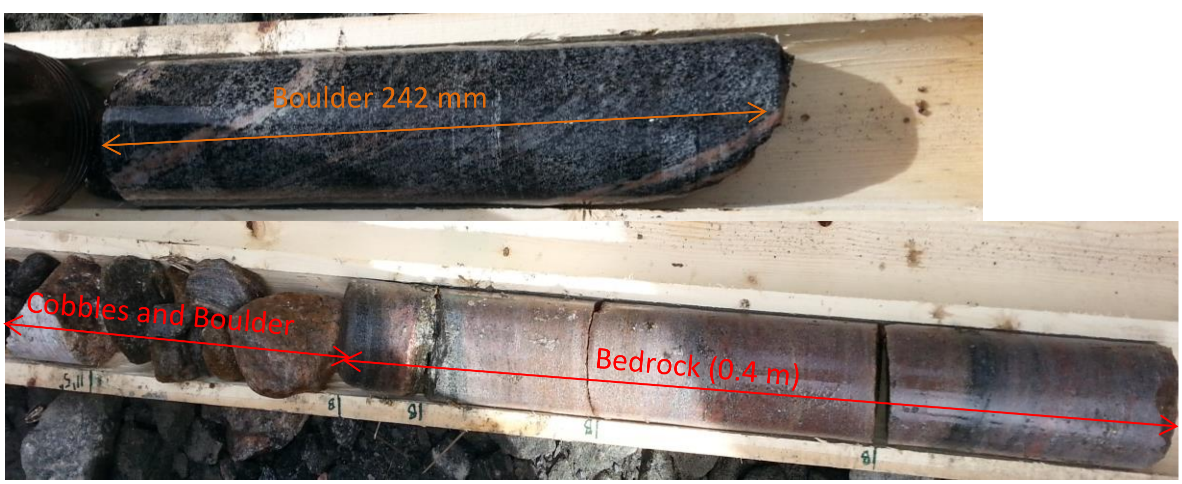


Photo 6. Core Sample for BH7A from Elevation 255.7 m to 253.9 m

Appendix F –
Results of Slope Stability Analyses

Rosseau River Bridge Replacement
East Approach Abutment
Drained Static Condition

Name: Concrete Model: Mohr-Coulomb Unit Weight: 24 kN/m³ Cohesion': 100 kPa Phi': 45 °
Name: Engineered Fill Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion': 0 kPa Phi': 32 °
Name: Rockfill Model: Mohr-Coulomb Unit Weight: 18 kN/m³ Cohesion': 0 kPa Phi': 42 °
Name: Cobbles and Boulder (Very Dense) Model: Mohr-Coulomb Unit Weight: 22 kN/m³ Cohesion': 0 kPa Phi': 38 °
Name: Silty Sand to Sand (Compact) Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion': 0 kPa Phi': 32 °
Name: Sand and Gravel (Compact to Very Dense) Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion': 0 kPa Phi': 35 °
Name: Sand (compact) Model: Mohr-Coulomb Unit Weight: 20.5 kN/m³ Cohesion': 0 kPa Phi': 32 °
Name: Bedrock Model: Bedrock (Impenetrable)
Name: Granular A Model: Mohr-Coulomb Unit Weight: 22 kN/m³ Cohesion': 0 kPa Phi': 35 °

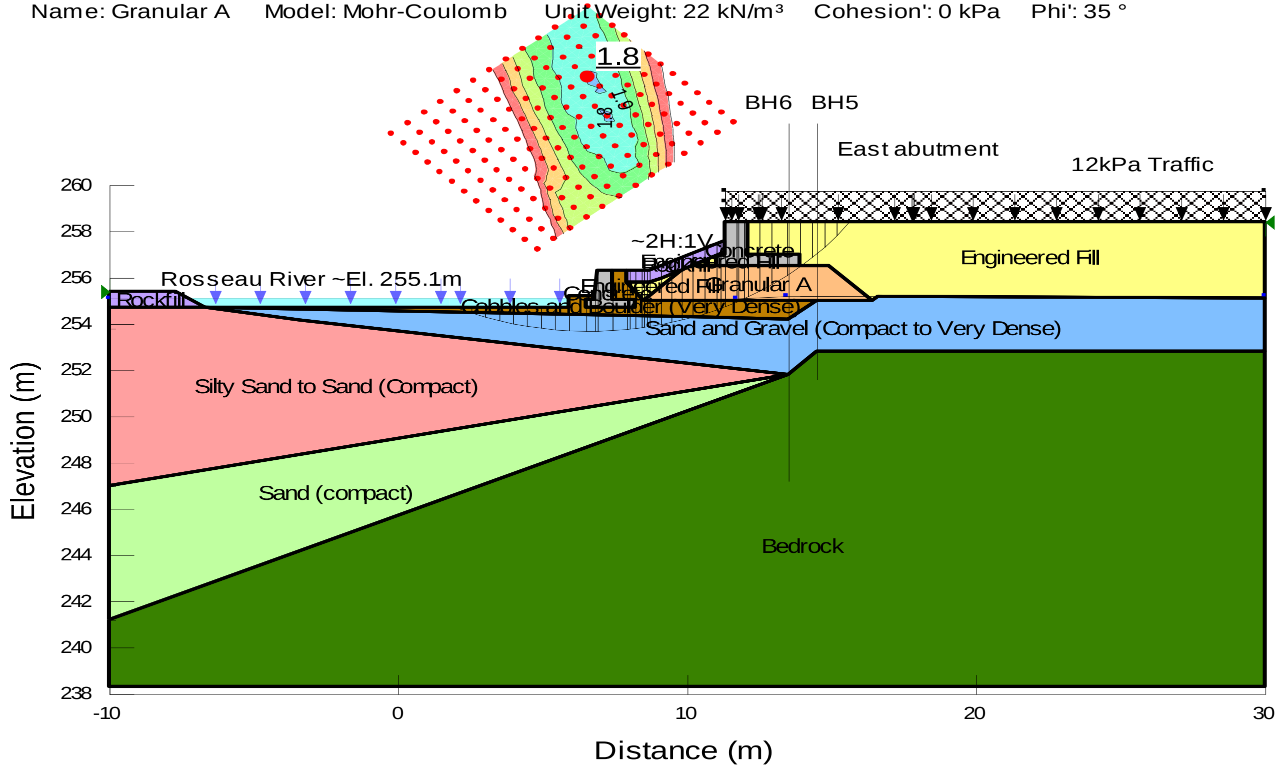


Figure F1: East abutment - drained static condition with granular pad option

Rosseau River Bridge Replacement
East Approach Abutment
Drained Seismic Condition

Name: Concrete Model: Mohr-Coulomb Unit Weight: 24 kN/m³ Cohesion': 100 kPa Phi': 45 °
Name: Engineered Fill Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion': 0 kPa Phi': 32 °
Name: Rockfill Model: Mohr-Coulomb Unit Weight: 18 kN/m³ Cohesion': 0 kPa Phi': 42 °
Name: Cobbles and Boulder (Very Dense) Model: Mohr-Coulomb Unit Weight: 22 kN/m³ Cohesion': 0 kPa Phi': 38 °
Name: Silty Sand to Sand (Compact) Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion': 0 kPa Phi': 32 °
Name: Sand and Gravel (Compact to Very Dense) Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion': 0 kPa Phi': 35 °
Name: Sand (compact) Model: Mohr-Coulomb Unit Weight: 20.5 kN/m³ Cohesion': 0 kPa Phi': 32 °
Name: Bedrock Model: Bedrock (Impenetrable)
Name: Granular A Model: Mohr-Coulomb Unit Weight: 22 kN/m³ Cohesion': 0 kPa Phi': 35 °

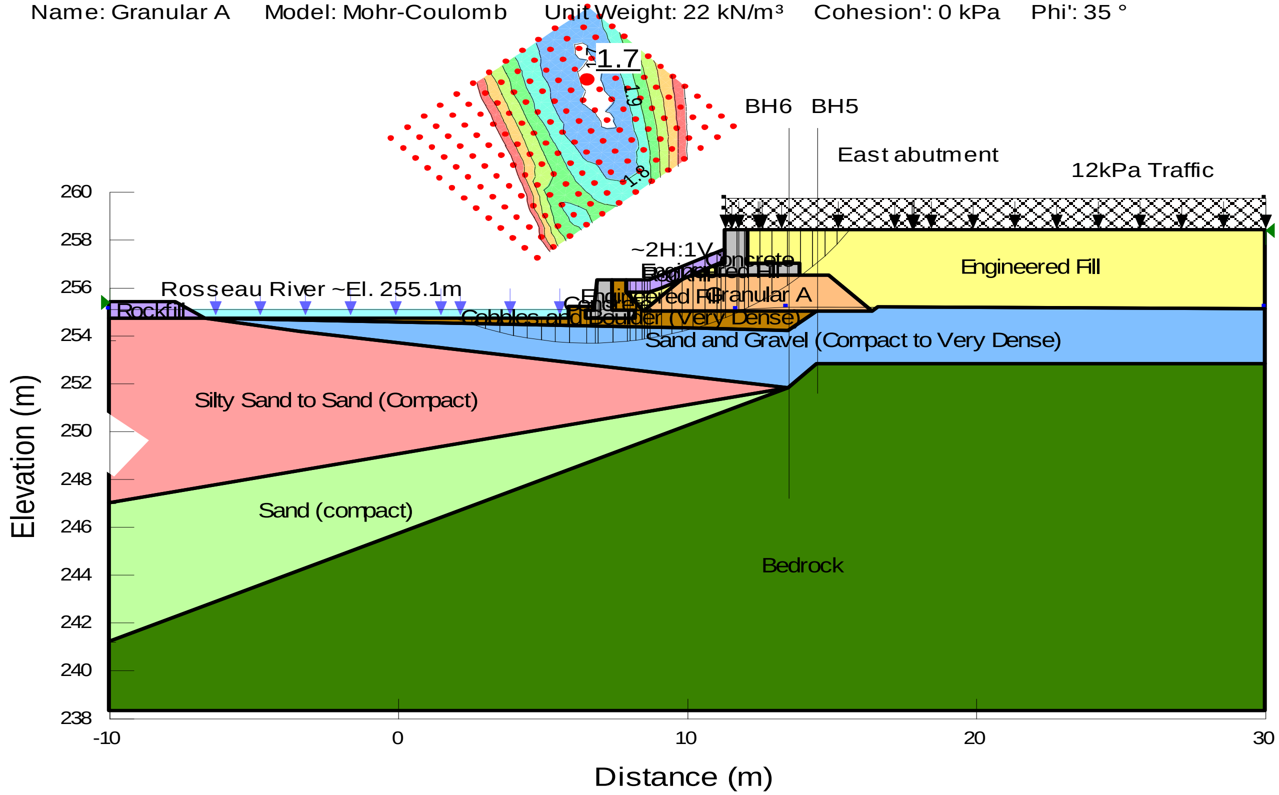


Figure F2: East abutment - drained seismic condition with granular pad option

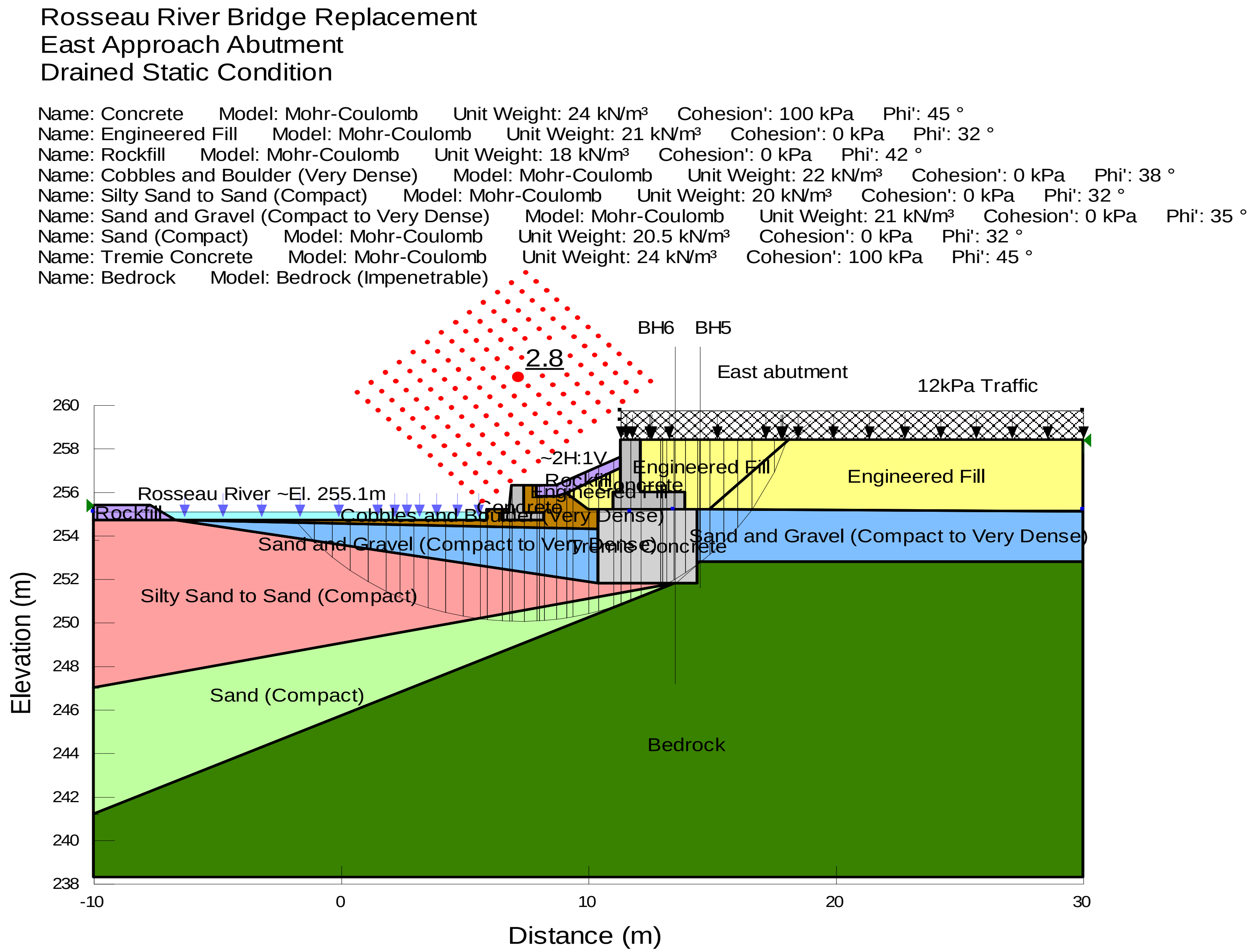


Figure F3: East abutment - drained static condition with tremie concrete pad option

Rosseau River Bridge Replacement
East Approach Abutment
Drained Seismic Condition

Name: Concrete Model: Mohr-Coulomb Unit Weight: 24 kN/m³ Cohesion': 100 kPa Phi': 45 °
Name: Engineered Fill Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion': 0 kPa Phi': 32 °
Name: Rockfill Model: Mohr-Coulomb Unit Weight: 18 kN/m³ Cohesion': 0 kPa Phi': 42 °
Name: Cobbles and Boulder (Very Dense) Model: Mohr-Coulomb Unit Weight: 22 kN/m³ Cohesion': 0 kPa Phi': 38 °
Name: Silty Sand to Sand (Compact) Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion': 0 kPa Phi': 32 °
Name: Sand and Gravel (Compact to Very Dense) Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion': 0 kPa Phi': 35 °
Name: Sand (Compact) Model: Mohr-Coulomb Unit Weight: 20.5 kN/m³ Cohesion': 0 kPa Phi': 32 °
Name: Tremie Concrete Model: Mohr-Coulomb Unit Weight: 24 kN/m³ Cohesion': 100 kPa Phi': 45 °
Name: Bedrock Model: Bedrock (Impenetrable)

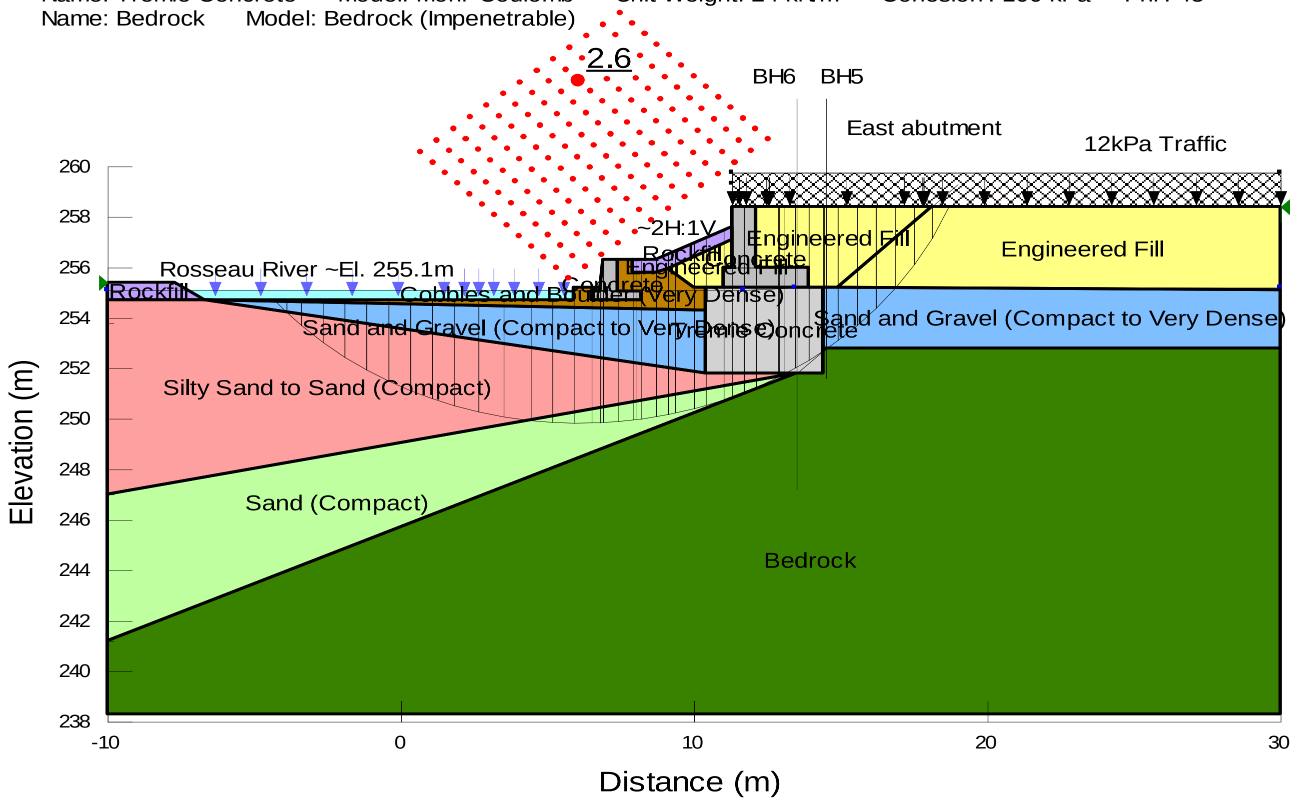


Figure F4: East abutment - drained seismic condition with tremie concrete pad option

Rosseau River Bridge Replacement
West Approach Abutment
Drained Static Condition

Name: Concrete Model: Mohr-Coulomb Unit Weight: 24 kN/m³ Cohesion': 100 kPa Phi': 45 °
Name: Engineered Fill Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion': 0 kPa Phi': 32 °
Name: Sand (Compact) Model: Mohr-Coulomb Unit Weight: 20.5 kN/m³ Cohesion': 0 kPa Phi': 32 °
Name: Cobbles and Boulder (Very Dense) Model: Mohr-Coulomb Unit Weight: 22 kN/m³ Cohesion': 0 kPa Phi': 38 °
Name: Silty Sand to Sand (Compact) Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion': 0 kPa Phi': 32 °
Name: Sand and Gravel (Compact to Very Dense) Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion': 0 kPa Phi': 35 °
Name: Rockfill Model: Mohr-Coulomb Unit Weight: 18 kN/m³ Cohesion': 0 kPa Phi': 42 °
Name: Bedrock Model: Bedrock (Impenetrable)

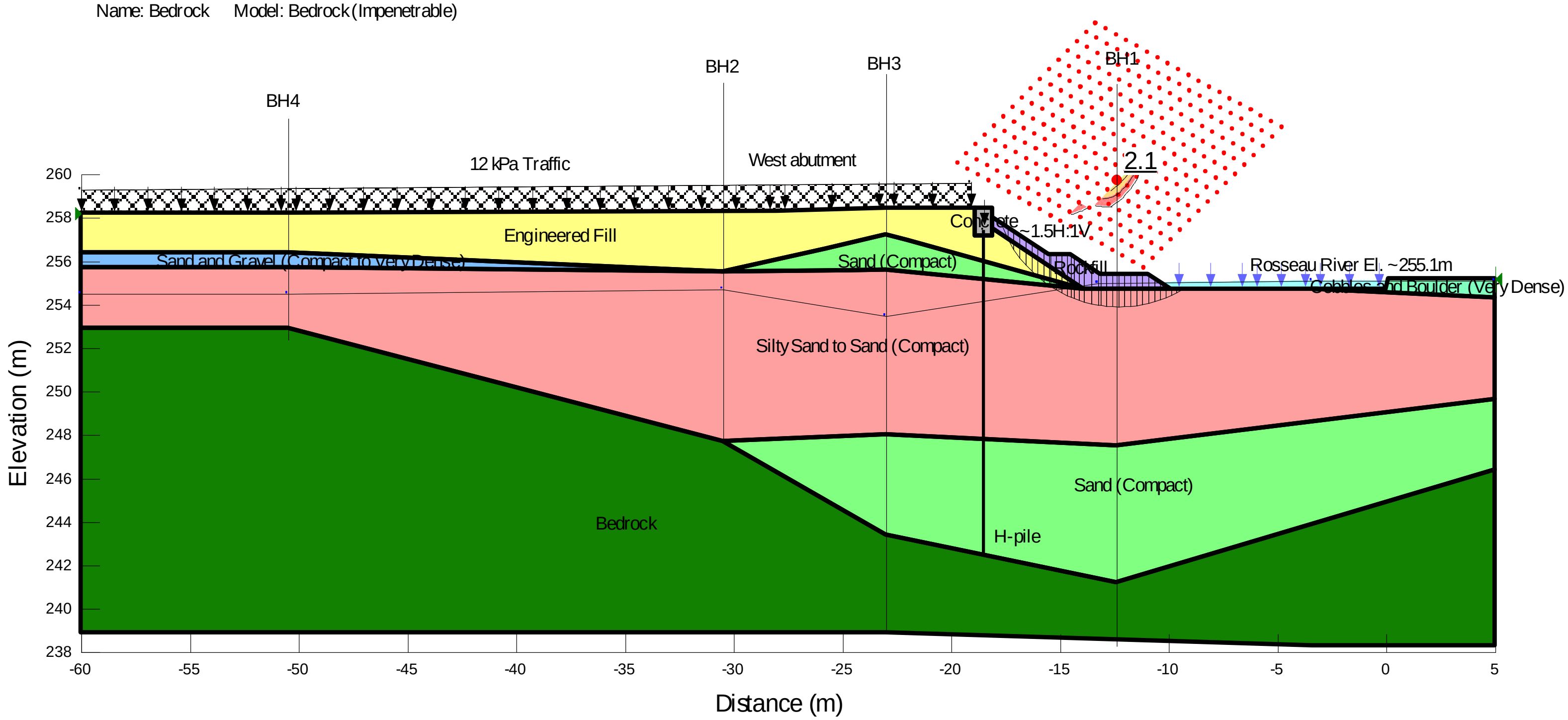


Figure F5: West abutment - drained static condition

Rosseau River Bridge Replacement
West Approach Abutment
Drained Seismic Condition

Name: Concrete Model: Mohr-Coulomb Unit Weight: 24 kN/m³ Cohesion': 100 kPa Phi': 45 °
Name: Engineered Fill Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion': 0 kPa Phi': 32 °
Name: Sand (Compact) Model: Mohr-Coulomb Unit Weight: 20.5 kN/m³ Cohesion': 0 kPa Phi': 32 °
Name: Cobbles and Boulder (Very Dense) Model: Mohr-Coulomb Unit Weight: 22 kN/m³ Cohesion': 0 kPa Phi': 38 °
Name: Silty Sand to Sand (Compact) Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion': 0 kPa Phi': 32 °
Name: Sand and Gravel (Compact to Very Dense) Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion': 0 kPa Phi': 35 °
Name: Rockfill Model: Mohr-Coulomb Unit Weight: 18 kN/m³ Cohesion': 0 kPa Phi': 42 °
Name: Bedrock Model: Bedrock (Impenetrable)

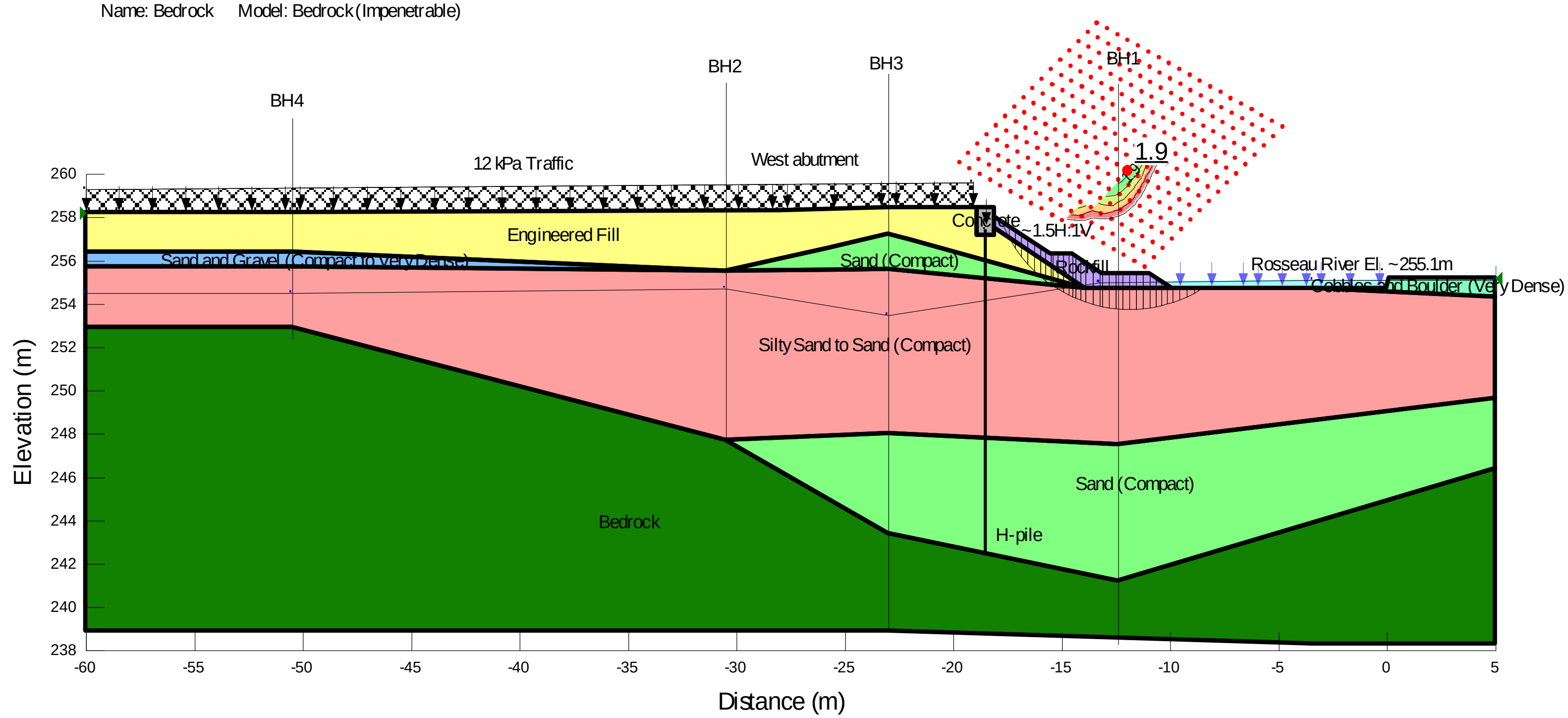
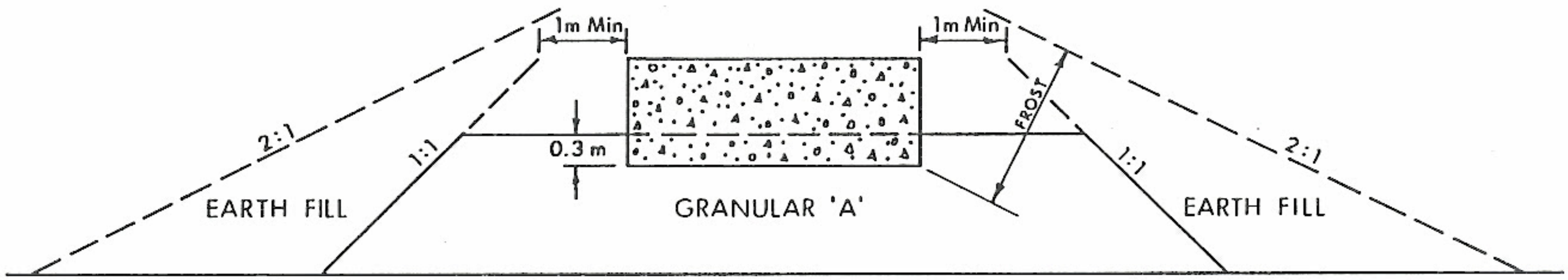


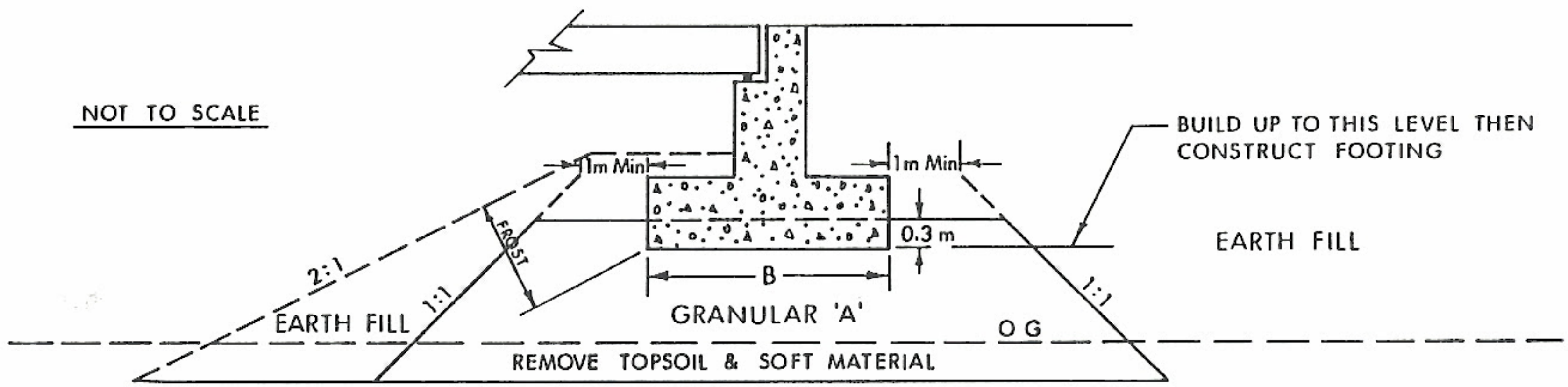
Figure F6: West abutment - drained seismic condition

Appendix G – OPSDs



X SECTION

NOT TO SCALE



LONGITUDINAL SECTION

NOTES:

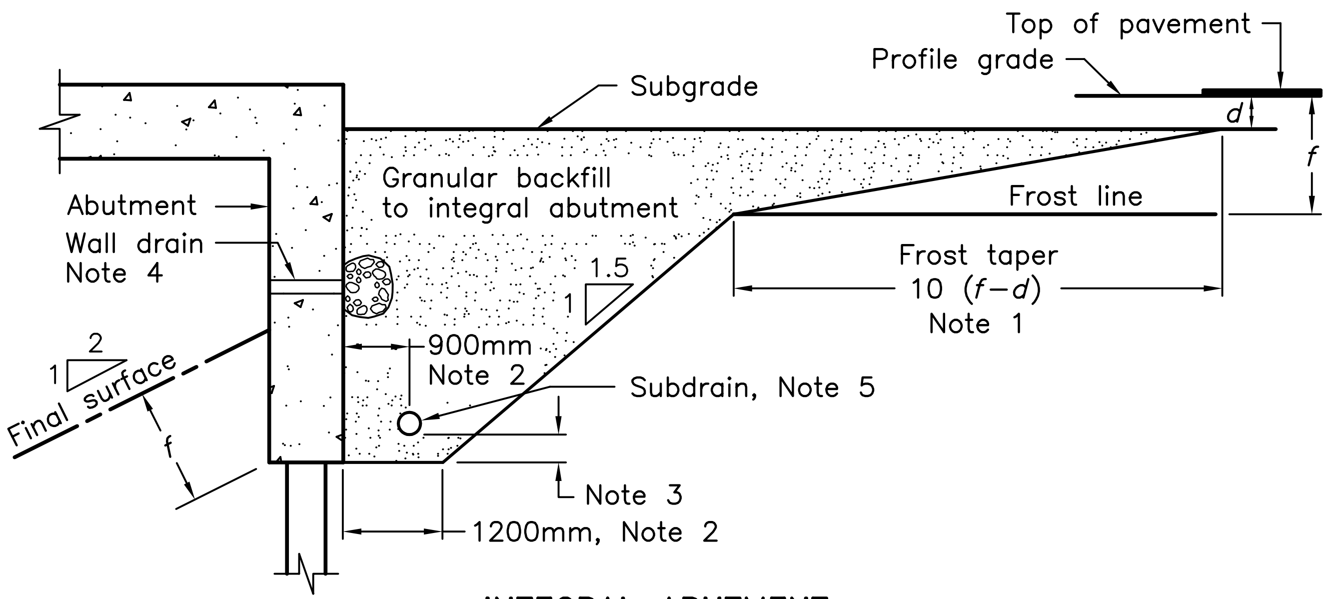
1 - REMOVE TOPSOIL &/OR SOFT SUBSOIL UNDER AREA OF COMPACTED GRANULAR 'A' & EARTH FILL.

2 - PLACE GRANULAR 'A' & EARTH FILL TO BOTTOM OF FOOTING LEVEL, COMPACTED ACCORDING TO CURRENT M T O STANDARDS.

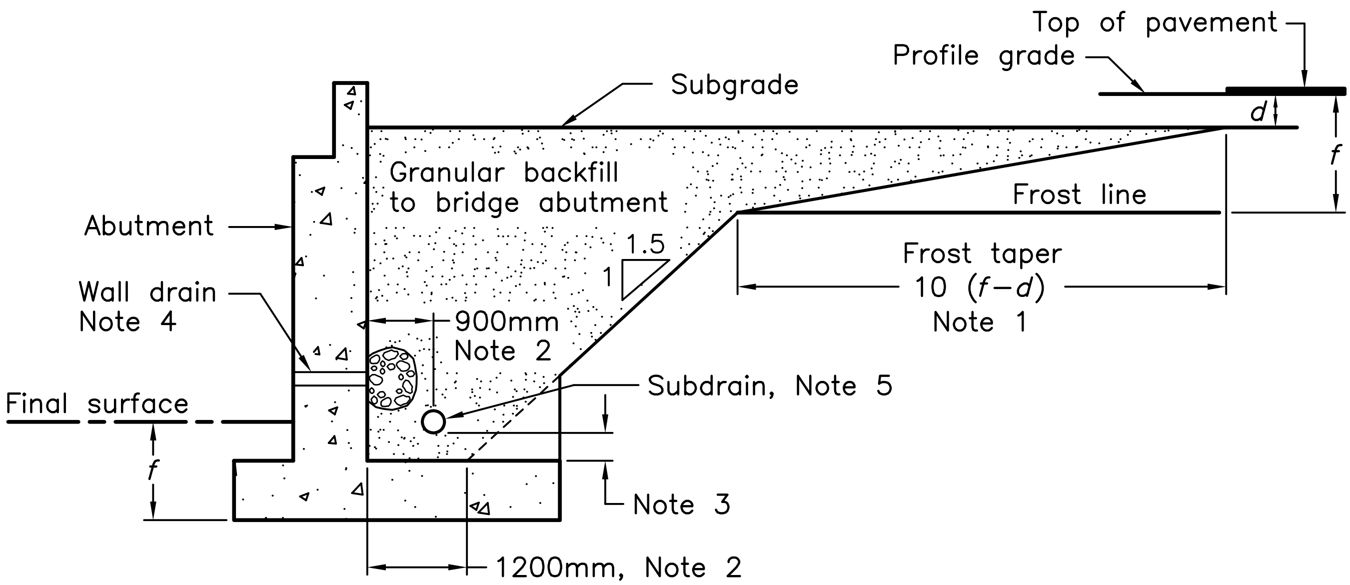
3 - CONSTRUCT CONCRETE FOOTING.

4 - PLACE REMAINDER OF GRANULAR 'A' & EARTH FILL AS REQUIRED.

Ministry of Transportation Ontario	ABUTMENT ON COMPACTED FILL SHOWING GRANULAR 'A' CORE	FIG No W P



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.

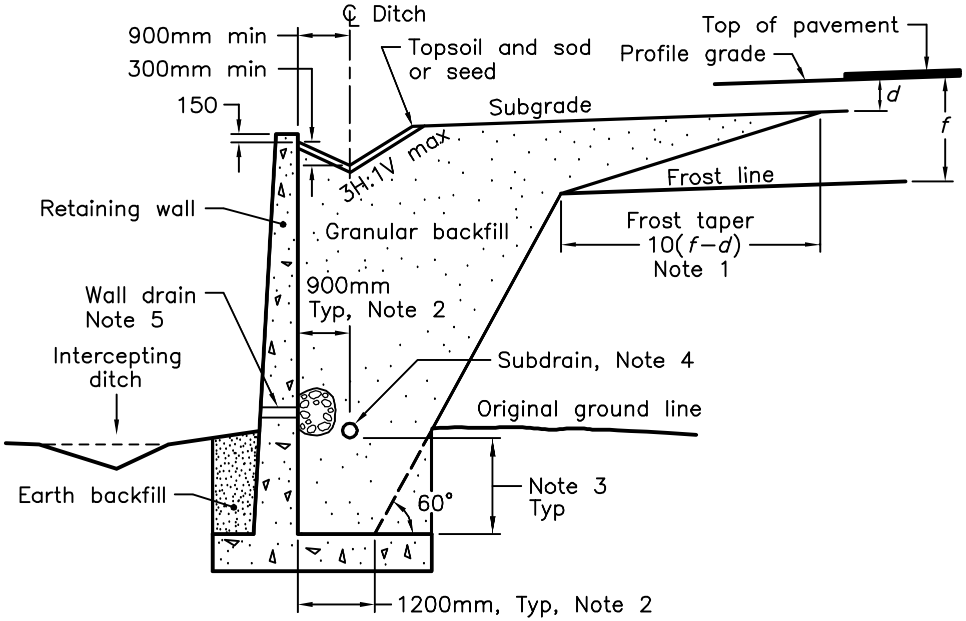
A Lateral limits of granular backfill to bridge abutment to be inside face to inside face of retaining wall or wingwall. Frost taper shall extend the full width of the fill unless interrupted by the retaining wall or wingwall.

B Sections shown are parallel to centreline of roadway.

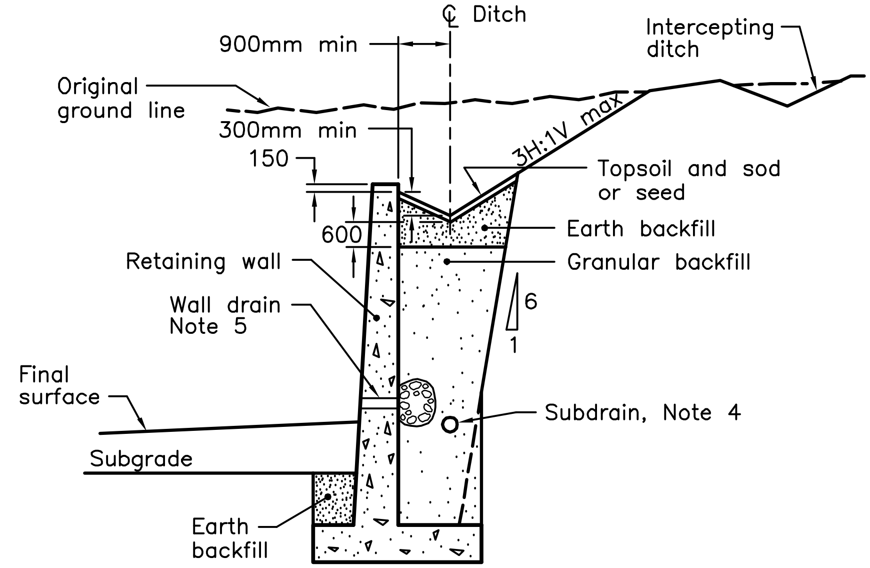
C Subdrain to be installed with a 2% gradient behind wall.

D All dimensions are in millimetres unless otherwise shown.

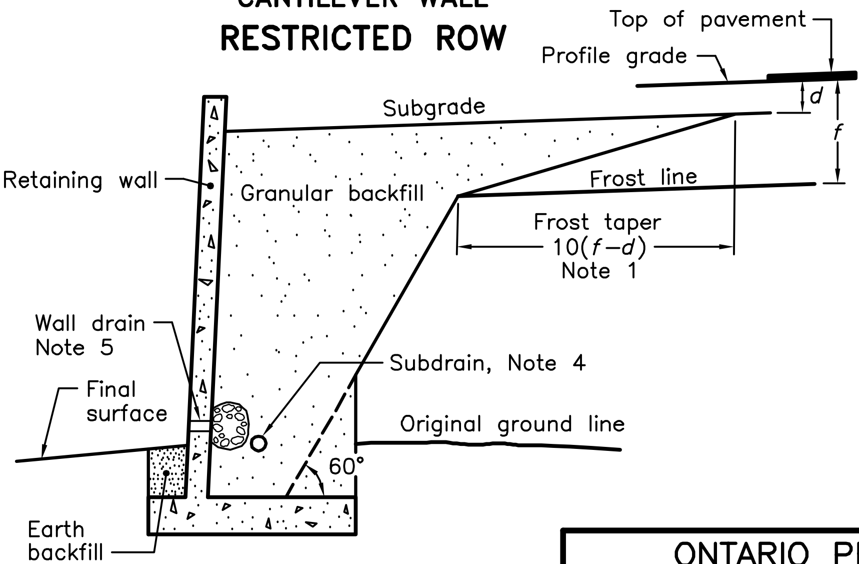
ONTARIO PROVINCIAL STANDARD DRAWING	Nov 2005	Rev 0
WALLS ABUTMENT, BACKFILL MINIMUM GRANULAR REQUIREMENT	OPSD – 3101.150	



CANTILEVER WALL
RESTRICTED ROW



CANTILEVER WALL
CUT TREATMENT

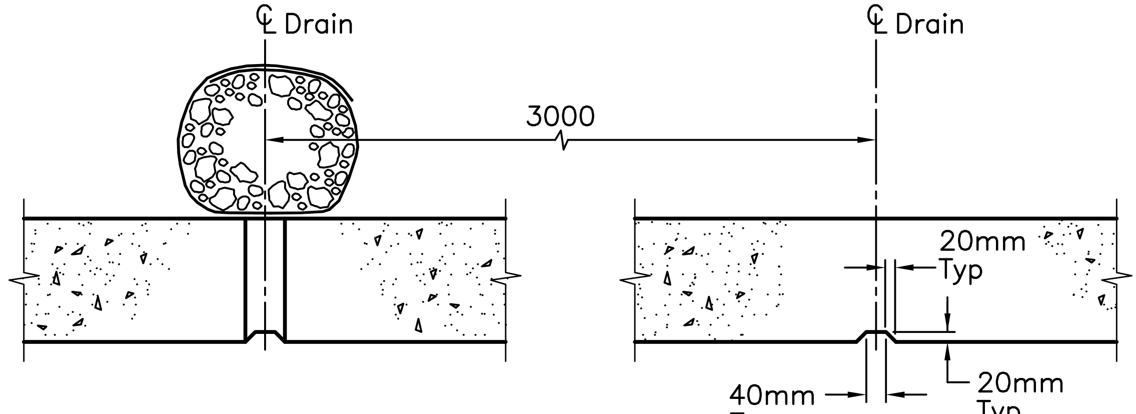


COUNTERFORT AND
CANTILEVER WALL
FILL TREATMENT
SHOULDER OVER RETAINING WALL BACKFILL

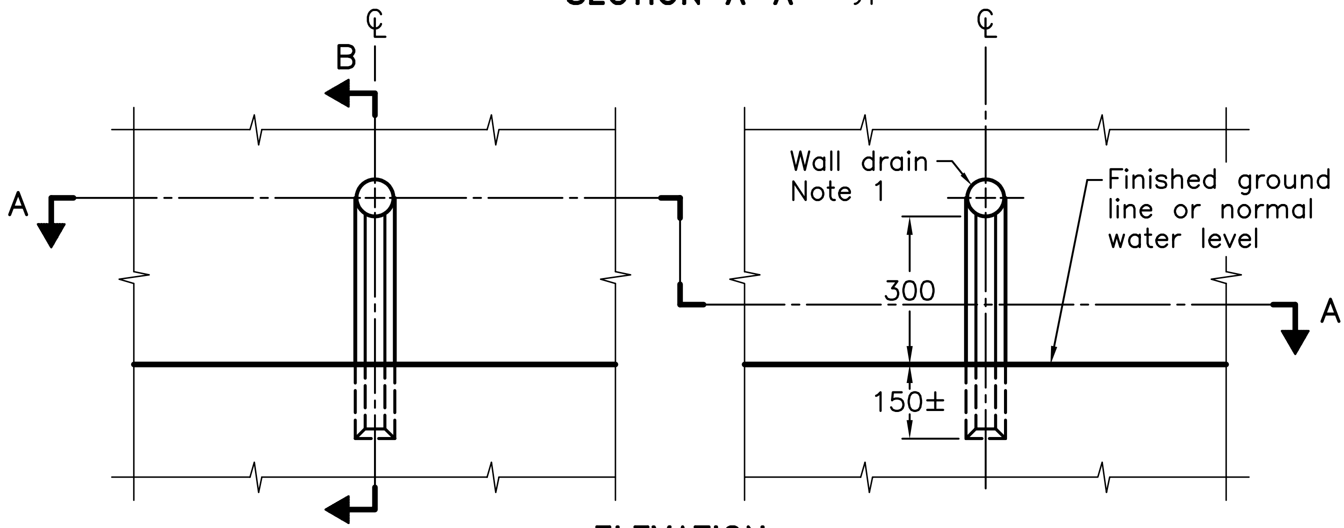
NOTES:

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.

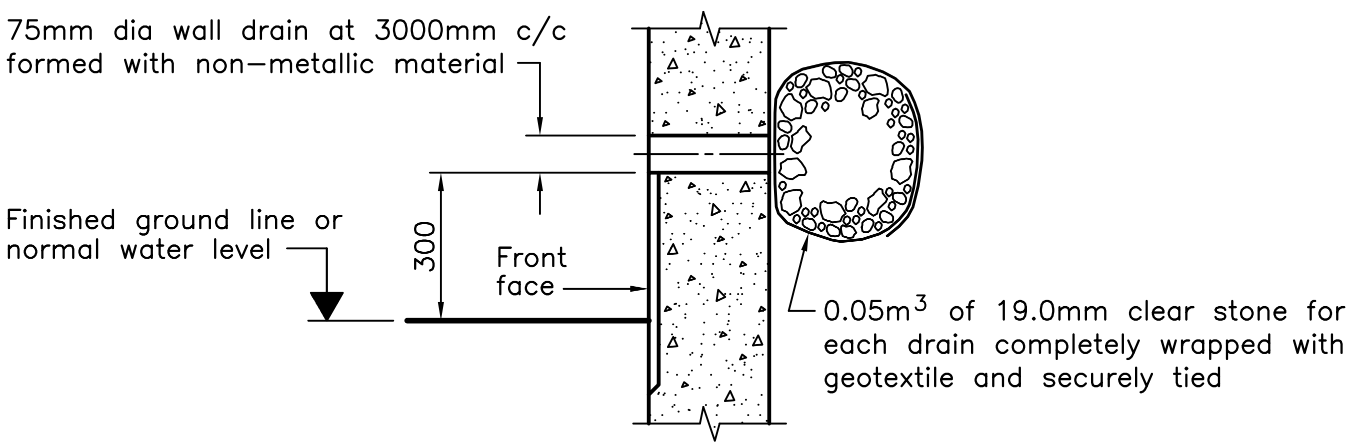
ONTARIO PROVINCIAL STANDARD DRAWING	Nov 2010	Rev 1
WALLS RETAINING, BACKFILL MINIMUM GRANULAR REQUIREMENT	OPSD 3121.150	



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.

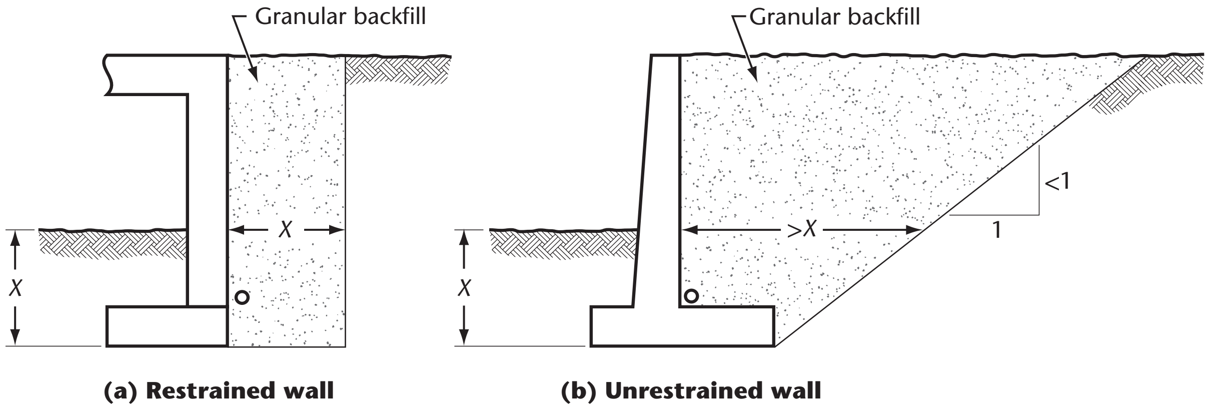


Figure C6.20
Backfill for frost protection
(See Clause C6.9.1.)

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 –
Memorandum-Foundation Recommendations Proposed Detour Acrow Bridge

MEMORANDUM

Date: September 10, 2015

To: Olta Kociu, P.Eng.
Foundation Engineer
Ministry of Transportation
Pavements and Foundations Section
1201 Wilson Avenue, 2nd Floor
Room 232, Building C
Downsview, Ontario M3M 1J8

Cc: Jean-Pierre Paron, P.Eng.
Project Manger
Ministry of Transportation
and
Ken Ahmed, P.Eng.
Senior Foundation Engineer
Ministry of Transportation

From: Silvana Micic, Ph.D., P.Eng.
Senior Geotechnical Engineer
exp Services Inc.

Cc: Stan Gonsalves, M.Eng., P.Eng
Designated MTO Contact
exp Services Inc.

Subject: Foundation Recommendations
Proposed Detour Acrow Bridge at the Rosseau River on Hwy 141
Rosseau, Ontario
GWP 330-96-00, Site No. 42-013
Agreement No. 5013-E-0008, Assignment No. 11

Introduction

The proposed fieldwork for the foundation investigation for the above noted project with drilling of two (2) foundation boreholes and one(1) pavement hole was commenced on September 9, 2015 after the approach embankment was built. However, only one foundation borehole (BH

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8) was completed until today (September 10, 2015). Due to the urgency of this project as per your request, we are herewith submitting our advanced recommendations in a memo format for Detour Acrow Bridge based on the data found in BH 8. This memorandum provides summary of subsurface conditions encountered in BH 8 and recommendations which will permit your office to proceed with design of the above structure.

The complete foundation investigation and design report will be forwarded to your office at a later date upon the completion of fieldwork, laboratory testing and drafting.

Site Description

The Rosseau River Bridge is located on Hwy 141, approximately 4.9 km east of the junction of Hwy 632. The Rosseau River Bridge is currently under rehabilitation and as part of the rehabilitation scope of the work; both east and west abutments were to be rehabilitated. During the inspection of the abutments, it was noted that voids and scouring were present at the west abutment. A detailed inspection was conducted at both abutments. The inspection concluded that voids and scouring are present at the west abutment and the east abutment is in sound conditions. Due to presence of voids and scouring underneath the west abutment, the existing bridge is closed for traffic. Emergency temporary detour Acrow bridge will be constructed north of existing bridge.

The existing structure is single span steel girders concrete deck bridge, and is about 13.1 m in length and about 8.2 m wide double lanes. The approaches are about 10 m wide from shoulder to shoulder. At the time of the fieldwork, the water level at Portage Creek was about 3.9 m below the top of the bridge deck.

During the fieldwork on September 3 to10, 2015 for the existing and temporary bridges, the general site conditions were assessed. Hwy 141 runs in a generally east to west direction, and Rosseau River flows from north to south at the site, towards the Rosseau Lake. The banks of the river in the vicinity of the Bridge contained gravel, cobbles and boulders, and vegetation including grass, trees and shrubs were noted at the banks further away from the bridge.

Subsurface Conditions

As encountered in BH 8, the subsurface soils on the site of the detour bridge consisted of new well graded sand and gravel fill recently placed for the temporary bridge approach. The new fill was generally described as compact, grey to brown, and containing some cobbles. The new fill extended to depth of 1.5 m below the ground surface. The old fill from the existing bridge approach embankment was encountered beneath of the new fill. The old fill consisted of medium fine sand. In general, it was described as loose, brown, moist, and containing trace silt, gravel and organics. It also contained occasional boulders. The old fill extended to the depth between about 1.5 m and 3.8 m below the ground surface. The last 0.5 m of this fill

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was a very dense gravelly layer. The fill layers were underlain by native sand soils. This native soil layer was generally described as very loose to compact, grey, wet, containing some gravel and extending to depth between about 3.8 m and 16.4 m below the ground surface. At the depth of about 13 m the layer became dense.

In BH 8 the refusal to farther penetration of the drilling equipment was encountered at the depth of 16.4 m below the ground surface. This level is in a good agreement with the level of bedrock encountered in the adjacent borehole BH 1 (approximately 17.4 m below the existing bridge deck) drilled at the location of the west abutment of the existing Rosseau River Bridge. Bedrock was proven in BH 1 by coring to 3 m. It was granite gneiss bedrock. Therefore, in BH 8 the bedrock was assumed at refusal to farther penetration and the borehole was terminated.

The groundwater level measured in BH 8 upon completion was 3.9 m below the ground surface. At the time of BH 1 drilling, the river level was approximately 3.9 m below the existing bridge deck. 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.

The site sketch showing the location of boreholes and preliminary borehole logs are attached to this memo.

Recommendations

Based on our correspondence with MTO the existing Rosseau River bridge will be replaced with a new bridge which will be at the current bridge alignment as well as that the road grade will be the same as that at the location of the existing bridge. It is also understood that the approximately 30 m span detour Acrow bridge will be placed north of the existing bridge to facilitate traffic temporarily during the construction of the new bridge. The Acrow bridge will have the facility to permit post construction adjustment by jacking to permit vertical realignment during the period of use.

Recommendations pertaining to the foundations of the temporary bridge and related earth work are summarized in the following sections. The geotechnical parameters provided in the memorandum are recommended in accordance with the *Canadian Highway Bridge Design Code (CHBDC)* (CAN/CSA-S6-14).

Structure Foundations

Considering subsurface conditions encountered in the geotechnical soil boring performed for this project and the type of temporary structure proposed, shallow foundations (i.e spread footings) founded on the pad of granular engineered fill developed over native sand are recommended as the most preferable alternative from geotechnical/foundation perspectives. It is recommended that the excavation for the footing should be approximately 0.5 m above the groundwater level encountered at the site. It is also recommended that the footing is

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placed on an approximately 1.5 m thick Granular 'A' core (see the attached drawing). This granular pad will provide a more competent foundation area for the spread footing to support the abutments.

Therefore, 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 1.8 m below the lowest surrounding area), the following founding levels of spread footings are recommended:

Table 1. Recommendations for footing depth

Soil at Founding Level	Abutment	Foundation Depth Below Existing Grade
Engineered Fill over Native Compact Sand	West	min 1.8 m (+1.5 m excavation for engineered fill)
	East	

Shallow Foundations

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 levels given in Table 1, should be designed based on the factored resistances at ULS and geotechnical reactions at SLS given in Table 2 below. The footing width of 2 m is assumed.

Table 2. Geotechnical resistance at ULS and geotechnical reaction at SLS for a 2 m wide footing

Soil at Founding Level	Width of Footing (m)	Factored Geotechnical Resistance at ULS (kPa)	Geotechnical Reaction at SLS (kPa)
Engineered Fill over Native Compact Sand	2	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 should be reviewed if the selected footing width or founding elevations differ from those given in the table. Similarly, if an

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inclined load is applied instead of a vertical load, which is used in these calculations, the values given in Table 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 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 3. A factor of 0.8 should be applied in calculation of the horizontal resistance in accordance with CHBDC.

Table 3. Recommendations for coefficient of friction

Interface	**Coefficient of Friction, tan δ***
Concrete and engineered fill	0.55

**- based on NAVFAC 1986, Table 1, pg. 7.2-63*

Frost Protection

The frost depth in the area of the bridge is estimated to be approximately 1.8 m in accordance with OPSD 3090.100. During construction of any temporary and permanent support system using shallow foundations should be provided a minimum 1.8 m of soil cover or equivalent frost protection should be provided using thermal insulation.

Lateral Earth Pressures

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

The expression for calculating lateral earth pressure is given by:

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

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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 4 can be used (assuming wall friction is neglected, the back wall is vertical and the ground surface is horizontal both on the retained side as well as in front of the toe):

Table 4 Material types and unfactored earth pressure properties under static conditions

Material	**Unfactored Friction Angle ϕ' (^{o})**	**Coefficient of Active Earth Pressure (K_a)**	**Coefficient of Passive Earth Pressure (K_p)**	**Coefficient of Earth Pressure at Rest (K_o)**	**Unit Weight γ kN/m^3)**
Compacted Granular A	35	0.27	3.69	0.43	22
Compacted Granular B	32	0.31	3.25	0.47	21

Construction Considerations

Excavations

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. compact gravel and sand fill and loose to compact sandy silt) are considered as Type 3 soils above the groundwater table

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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.

Embankment Slopes

No stability problems are anticipated for temporary embankment's forward and side slopes constructed to a 2H:1V geometry. However, the slope surface should be protected from erosion of the sand with silt and gravel fill by a thin layer of topsoil as per current MTO standards. Suitable erosion and scour protection measures should also be provided to the river banks adjacent to the bridge. 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.

Temporary Shoring

Temporary excavation support systems, 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.

Dewatering

It is recommended that the bottom of excavation would be terminated no more than approximately 0.5 m above the groundwater to minimize disturbance and permit compaction of the exposed surface. It is anticipated that the amount of perched water within the upper granular fill materials at the abutment locations would be limited. For excavations through the soils at the abutment locations, groundwater control will likely be limited to diverting surface runoff and sump pumping.
The design of unwatering systems for the excavations is responsibility of the Contractor who is expected to retain dewatering specialists for this task.

Foundation Base Preparation

As mentioned previously, the footing can be placed on a 1.5 m thick layer of engineered fill as shown in the attached drawing. The footing should be set at least 2.0 m behind a line drawn up at 2H:1V from the base of the slope. If this cannot be met, the designer should consider lowering the grade or a suitably designed sheetpile protection system should be installed to meet the requirements.

Engineered fill should be placed in accordance with OPSS 501 and the attached drawing. The fill material should be placed in thin layers not exceeding approximately 300 mm when loose. Oversize particles larger than 120 mm should be discarded, and each fill layer should be uniformly compacted with heavy compactors, suitable for the type of fill used. The engineered fill below the footing should be compacted to 100% of its SPMDD.

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Full-time geotechnical inspection and quality control (by means of frequent field density and laboratory testing) should be provided by the Geotechnical Engineer. Every lift should be evaluated by a sufficient number of tests to ensure that the level of compaction is constantly achieved and the compaction procedure is applied.

We trust the above meets with your present requirements. If you have any question, please contact us.

Sincerely,

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

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

Stan E. Gonsalves, M.Eng., P.Eng.
Executive Vice- President
Designated MTO Contact

Attach: Site sketch
Borehole log
Drawing of abutment in excavated areas

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DRAFT

METRIC

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

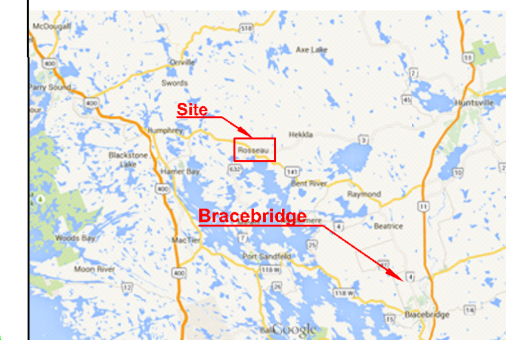
Agreement No. 5013-E-0008
Assignment No. 11
WO

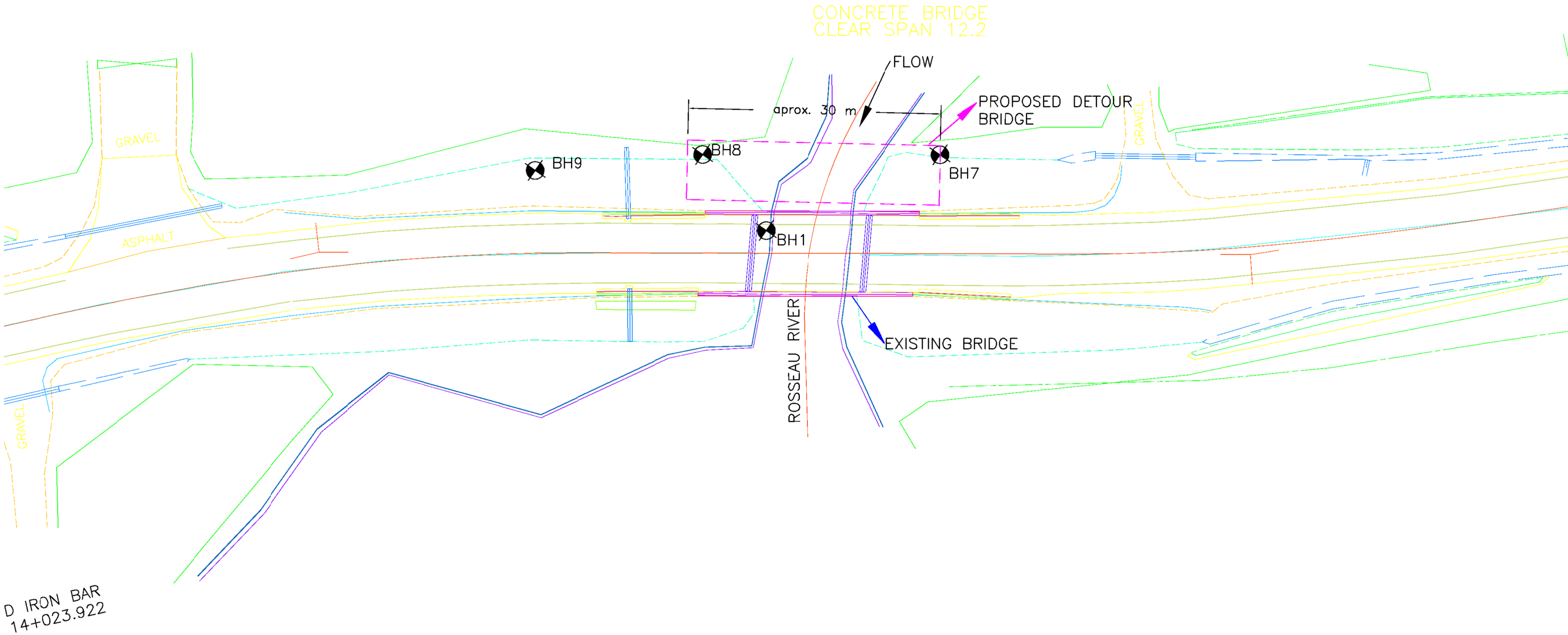
ROSSEAU RIVER BRIDGE REPLACEMENT
(SITE NO. 42-013, HWY 141)
SITE PLAN/ BOREHOLE LOCATIONS

SHEET 1

exp Services Inc.

KEY PLAN





LEGEND

Approximate Borehoe Locations

BH No.	APPROX. ELEV.	MTM CO-ORDINATES	
		NORTH	EAST
BH 1	TBP	TBP	TBP
BH 7	TBP	TBP	TBP
BH 8	TBP	TBP	TBP
BH 9	TBP	TBP	TBP

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
2015.09.10	SM	SUBMISSION FOR MTO REVIEW

GEOCRES NO.
PROJECT NO. ADM-00028245-M0

SUBM'D SM	CHECKED SM	DATE	2015.09.10
DRAWN SA	CHECKED SG	APPROVED	DWG. 01

Note: The plan was provided by MTO.

exp services Inc.
Brampton, Ontario

Foundation Design

RECORD OF BOREHOLE No BH-7

METRIC

W. P. GWP 330-96-00 | LOCATION Rosseau River Bridge | ORIGINATED BY NT

DIST HWY 141 | BOREHOLE TYPE CME-55X, Hollow stem auger | COMPILED BY NT

DATUM Geodetic | DATE 2015/09/10 - 2015/09/10 | CHECKED BY SM

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	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	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Ground Surface										
	FILL: SAND AND GRAVEL some cobbles, occassional boulder, grey loose to Dense		1	SS	34						
			2	SS	30						
			3	SS	5						
2.3	**SAND** some silt, some gravel, some clay, trace coobles and boulders, trace organics, trace rootlets, brown, moist to very moist, compact to very dense		4	SS	22						
	- Boulder @ 3.05 m					∇					
	- becoming clayey		5	SS	66						
			6	SS	100/ 228mm						
	- Bedrock @ 4.6 m on adjacent borehole BH-7A		7	SS	100/ 280mm						
	-Boulder @ 5.2 m										
			8	SS	67						
	- Spoon broke off										
5.8	**END OF BOREHOLE** **Possible Bedrock, Spoon broke off** Bedrock encountered @ 4.6m on adjacent borehole BH-7A										

NOTES:
1. This drawing is to be read with the subject report and project numbers as presented above.
2. Interpretation assistance by exp is required before used by others.
3. Groundwater level was measured in open hole.

DRAFT

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

OPG_EXP RECORD OF BOREHOLE 5013-E-0008 ASSIG.11- BH LOGS.GPJ ONTARIO MOT.GDT 9/15/15

RECORD OF BOREHOLE No BH8

METRIC

W. P. ADM-00282450-M0 | LOCATION Roseau River Bridge | ORIGINATED BY CS

DIST HWY14 | BOREHOLE TYPE CME-75, Hollow Stem Augers | COMPILED BY VP

DATUM GEODETIC | DATE 2015/09/09 - 2015/09/09 | CHECKED BY

ELEV / DEPTH	DESCRIPTION (SOIL PROFILE)	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	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	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Ground Surface										
	NEW FILL: SAND AND GRAVEL some cobbles, brown, compact		1	SS	24						
	-grey to brown		2	SS	14						
1.5	**OLD FILL: SAND** medium to fine grained sand, some bolders, trace gravel, trace silt, trace organics, brown, loose to compact, moist		3	SS	6						
			4	SS	8						
	- dense below 3.05 m depth		5	SS	121/ 305mm						
3.8	**SAND** fine grained sand, trace gravel, trace silt, trace organics, brown, compact, wet		6	SS	10	▽					
	-very loose below 4.6 m depth		7	SS	3						
	-grey compact below 5.3 m depth		8	SS	10						
	-seam coarse grained sand below 6.1 m depth		9	SS	14						
	fine to medium grained sand, loose below 6.9 m depth		10	SS	5						
	-some medium grained sand, trace gravel below 7.6 m depth		11	SS	5						
	-medium to fine grained sand, trace gravel, grey, compact, wet below 8.4 m depth		12	SS	14						
	-brownish grey, very loose below 9.1 m depth		13	SS	4						
	-loose below 9.9 m depth		14	SS	6						
	- medium to fine grained sand, trace gravel, very loose below 10.7 m depth		15	SS	3						
	-dense below 11.4 m depth		16	SS	30						
	- trace gravel , grey, compact, wet below 12.2		17	SS	16						
	- medium to fine grained sand, some coarse grained sand, trace gravel, dense below 12.9 m depth		18	SS	44						
	-dense below 13.7 m depth		19	SS	30						

Continued Next Page

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

DRAFT

RECORD OF BOREHOLE No BH8

METRIC

W. P.	ADM-00282450-M0	LOCATION	Roseau River Bridge	ORIGINATED BY	CS
DIST	HWN/W/1141	BOREHOLE TYPE	CME-75, Hollow Stem Augers	COMPILED BY	VP
DATUM	GEODETIC	DATE	2015/09/09 - 2015/09/09	CHECKED BY	

ELEV DEPTH	DESCRIPTION (SOIL PROFILE)	STRAT PLOT	NUMBER (SAMPLES)	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	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	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SAND fine grained sand, trace gravel, trace silt, trace organics, brown, compact, wet *(continued)* compact below 15.24 m depth		20	SS	17						
16.5	**ASSUMED BEDROCK at ~16.46 m DEPTH BOREHOLE TERMINATED** **NOTES:** 1. This drawing is to be read with the subject report and project numbers as presented above. 2. Interpretation assistance by exp is required before used by others.										

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

exp. exp services Inc.
Brampton, Ontario

Foundation Design

RECORD OF BOREHOLE No BH9

 METRIC

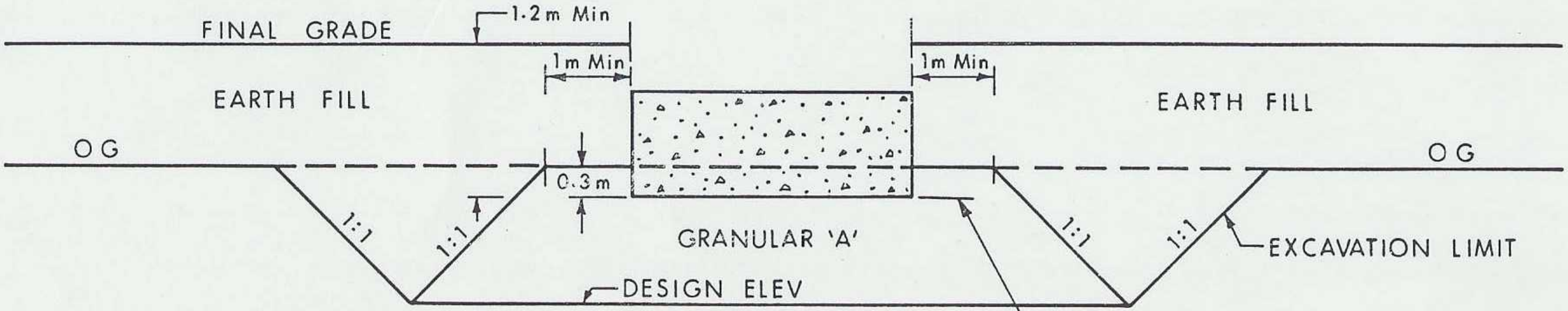
W. P.	ADM-00282450-M0	LOCATION	Roseau River Bridge	ORIGINATED BY	CS
DIST	HWNW41141	BOREHOLE TYPE	CME-75, Hollow Stem Augers	COMPILED BY	VP
DATUM	GEODETIC	DATE	2015/09/10 - 2015/09/10	CHECKED BY	

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: Cu, 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 (%) GR SA SI CL
	Ground Surface										
	FILL: SAND AND GRAVEL grey, compact, moist		1	SS	23						
	-medium to fine grained sand, some gravel, brown, loose, moist below 0.8 m depth		2	SS	8						
			3	SS	5						
	-seam topsoil, trace rootlets 2.3 m depth		4	SS	6						
	-brown, compact, wet below 3.1 m depth		5	SS	29	∇					
			6	SS	19						
4.4	**END OF BOREHOLE at ~ 4.42 m depth**										

NOTES:
1. This drawing is to be read with the subject report and project numbers as presented above.
2. Interpretation assistance by exp is required before used by others.

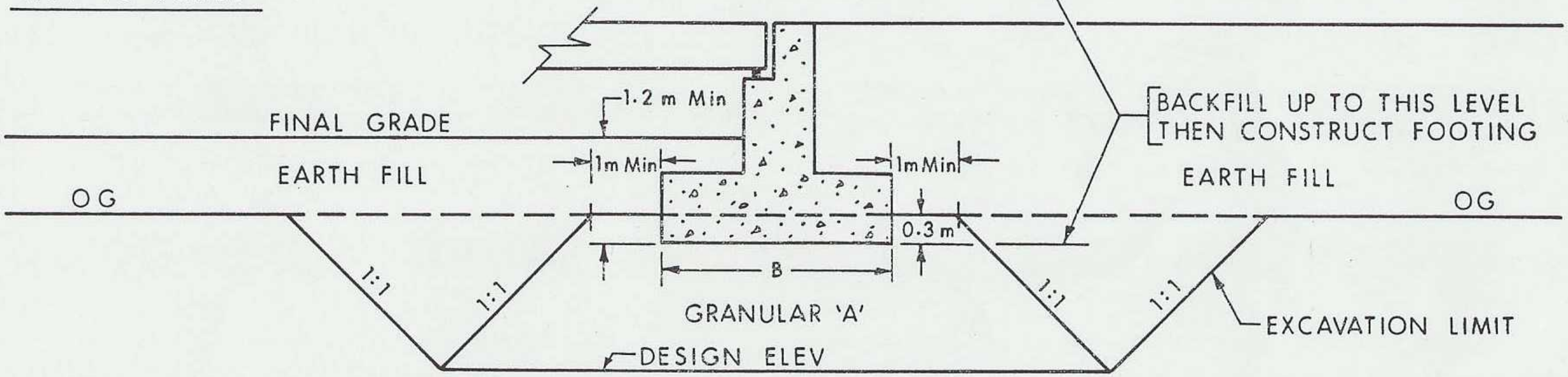
$+^3, \times^3$: Numbers refer to Sensitivity $\circ^{3\%}$ STRAIN AT FAILURE

OPG_EXP RECORD OF BOREHOLE SILVANA-HWY 141.GPJ ONTARIO MOT.GDT 9/15/15



X - SECTION

NOT TO SCALE



LONGITUDINAL SECTION

NOTES:

1- EXCAVATE TO DESIGN ELEVATION UNDER AREA OF COMPACTED GRANULAR 'A'.

2- BACKFILL GRANULAR 'A' & EARTH FILL TO BOTTOM OF FOOTING LEVEL, COMPACTED ACCORDING TO CURRENT MTO STANDARDS.

3- CONSTRUCT CONCRETE FOOTING.

4- PLACE REMAINDER OF GRANULAR 'A' & EARTH FILL AS REQUIRED.

Ministry of Transportation Ontario	ABUTMENT IN EXCAVATED AREA SHOWING GRANULAR 'A' CORE	FIG No W P