FINAL REPORT

FOUNDATION INVESTIGATION REPORT

Victoria Creek Culvert Replacement, Highway 672, Township of Arnold, Northeast of Kirkland Lake, Ontario

Agreement No. 5015-E-0007
Assignment No. 7
GWP 5027-17-00
Geocres No. 32D-22

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Foundation Investigation and Design Report

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Table of Contents

1 FOUNDATION INVESTIGATION REPORT 1

1.1 Introduction 1

1.2 Site Description and Geological Setting 1

1.2.1 Site Description 1

1.2.2 Geological Setting 2

1.3 Investigation Procedures 2

1.3.1 Site Investigation and Field Testing 2

1.3.2 Previous Investigation 4

1.3.3 Laboratory Testing 4

1.4 Subsurface Conditions 4

1.4.1 Proposed Culvert Replacement Location 5

1.4.2 Temporary Modular Bridge Location 10

1.5 Groundwater Conditions 11

2 ENGINEERING DISCUSSION & RECOMMENDATIONS 13

2.1 General 13

2.2 Expected Ground Conditions 14

2.3 Structure Foundations 15

2.3.1 Culvert Replacement 15

2.3.2 Temporary Modular Bridge 21

2.3.3 Temporary Cofferdam 23

2.4 Lateral Earth Pressure on Structures 26

2.5 Seismic and Liquefaction Potential Consideration 27

2.6 Construction Considerations 28

2.6.1 Excavations 28

2.6.2 Temporary Protection 28

2.6.3 Groundwater Control 29

2.7 Culvert Bedding 30

2.8 Culvert Backfill 31

2.9 Embankment Design 31

2.9.1 Stability Analysis 31

2.9.2 Embankment Settlement 32

2.9.3 Embankment Slope Stability and Settlement at Temporary Modular Bridge Location 33

2.10 Inlet and Outlet 33
2.10.1 Erosion Protection at Outlet 33
2.10.2 Stream Bed Rip-Rap 33
2.10.3 Seepage Cut-off Requirements 33
2.11 Obstructions 34
3 CLOSURE 35
4 LIMITATIONS AND USE OF REPORT 36

Appendices

Site Photographs ..... A
Drawings ..... B
Borehole Logs C
Laboratory Data ..... D
Rock Core Photographs ..... E
Golder's Technical Memorandum ..... F
Slope Stability Analysis ..... G
OPSDs ..... H
Non-Standard Special Provisions (NSSPs) I

1 FOUNDATION INVESTIGATION REPORT

1.1 Introduction

This report presents the results of a geotechnical investigation completed by **exp** Services Inc. for replacement of Victoria Creek Culvert on Highway 672, in Township of Arnold, northeast of Kirkland Lake, Ontario. The work was undertaken under Agreement No. 5015-E-0007, Assignment No. 7. The terms of reference (TOR) were as presented in Ministry of Transpiration Ontario (MTO) email dated May 23, 2017.

The purpose of the investigation is to determine the subsurface conditions along the existing culvert alignment to permit detailed design for its replacement including cofferdams, as well as at the location of a proposed Temporary Modular Bridge (TMB) which will serve as a detour during the culvert replacement. The site specific geotechnical investigation consisted of a field investigation including visual inspections, drilling at land and in water, soil sampling, 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 existing Victoria Creek culvert is located on Highway 672, at approximately Sta. 10+150 in the Township of Arnold, northeast of Kirkland Lake Ontario. Highway 672 is a two lane roadway, with a speed limit of 80 km/h and is about 6 m wide from edge of pavement with sand and gravel shoulders. The roadway embankment above the creek bed is about 5.5 m high with side slopes of about 1.7H:1V. Highway 672 runs in a generally north-south direction, and Victoria Creek flows from a west to east direction.

As reported by MTO, a washout has recently occurred at the project site and as a temporary and emergency measure, twin 2.4 m diameter temporary CSP pipes were installed and the embankment was restored by new earthfill. According to the field observation, the twin pipes were installed at about 8 m away from the existing pipe and at a much shallower depth with invert located at the obvert elevation of the existing pipe approximately, while the existing pipe remains in-place. The diameter of the existing CSP pipe is estimated to be approximately 2.4 m. A TMB which will serve as a detour during construction to allow traffic flow is proposed at the adjacent abandoned road located west of the existing Highway 672.

During the investigations, it was observed that earthfill used in the restored embankment in the vicinity of the existing culvert was predominantly sandy soils with some large rock and boulders at the bottom. The top of the road was unpaved having the elevation of about 313.4 m. Water seepages were noticed at the embankment toe at the outlet side, north and south of the existing culvert. Some submerged wood logs were noticed at inlet side of the existing culvert. Bedrock outcrops were observed in the vicinity of the existing culvert. At the proposed location of the TMB, existing concrete abutments of the old bridge were found. The approaching abandon road was about one lane wide having shoulders grown in with vegetation. Its pavement was in rough shape having nnumerous cracks. At the inlet side, the creek water forms a body of water (i.e. pool) encompassed with the old road embankment at the west and Highway 672 embankment at the east side. At the outlet, a flowing stream was observed. In June 2017, the inlet of the existing culvert was completely submerged.

Selected photographs of the site and existing culvert are presented in Appendix A. The site plan and cross-section profiles for the proposed culvert and TMB alignments are shown on the drawing attached in Appendix B.

1.2.2 Geological Setting

According the Ministry of Northern Development and Mines, Map 2555 (Quaternary Geology of Ontario, East-Central Sheet, 1991) the surface conditions in the vicinity of the project area consists of undifferentiated igneous and metamorphic rock exposed at surface to glaciolacustrine deposits which includes sand, gravelly sand and gravel. Glaciofluvial ice-contacts deposits, which includes gravel, sand and minor till includes esker, kame, end moraine, ice-marginal delta and subaqueous fan deposits and according to Map 2543 (Bedrock Geology of Ontario, East-Central Sheet, 1991), the bedrock geology of the site is of mafic to intermediate metavolcanics rocks of basaltic and andesitic flows, tuffs and breccias, chert, iron formation, minor metasedimentary and intrusive rocks.

1.3 Investigation Procedures

1.3.1 Site Investigation and Field Testing

The field investigation was performed in two phases, Phase I between June 13 and 16, 2017 and Phase II between July 5 and 13, 2017. Phase I field investigation consisted of drilling four (4) sampled boreholes (numbered BH-2, BH-3, BH-7 and BH-8) through road surface and Phase II field investigation consisted of drilling four (4) sampled off-road boreholes (numbered BH-1, BH-4, BH-5 and BH-6) in water using a barge. Among the boreholes drilled through the road surface, two boreholes (BH-2 and BH-3) were advanced at the location of existing culvert on Highway 672, while other two boreholes (BH-7 and BH-8) were advanced at the location of TMB on the detour route. Among the in-water boreholes, two boreholes (BH-1 and BH-5) were advanced on the outlet side and two boreholes (BH-4 and BH-6) were advanced on the inlet side of the existing culvert. The locations of boreholes are shown on Drawing 1 attached in Appendix B.

All culvert boreholes (BH-1, BH-2, BH-3, BH-4, BH-5 and BH-6) were strategically located along the existing culvert alignment to provide subsurface information for the replacement of existing culvert and construction of cofferdams. BH-2 and BH-3 were located on the south side of the existing culvert within NBL and SBL, respectively. BH-3 was intended to be drilled north of the existing culvert, however, since the inlet side of the culvert was submerged at the time of drilling of this land borehole it was difficult to estimate its proposed location relative to the culvert. BH-4 and BH-5 were located near the inlet and outlet of the existing culvert, respectively. BH-1 and BH-6 were located between the temporary twin culverts and the existing culvert at the outlet and inlet side, respectively, assuming that cofferdams will be located there. Due to the presence of submerged wood logs near the inlet of existing culvert, BH-4 and BH-6 could not be drilled closer to the existing culvert, while BH-1 could not be drilled closer to the existing culvert due to the shallow water depth for a barge setup. BH-7 and BH-8 were strategically located at the proposed TMB to provide subsurface information along the TMB alignment. BH-7 was advanced on the approximate location of the north abutment, while BH-8 was advanced on the approximate south abutment location.

At the site location, Golder Associates Ltd. (Golder) also performed a preliminary foundation investigation, dated May 25, 2017, to support MTO on design of the culvert replacement. Golder's field investigation included advancing three rock probes (P1, P2 and P4) to depths ranging from 12.5 m to 17.4 m below the existing ground surface. The results of this preliminary foundation investigation were provided by MTO along with the TOR. The locations of probeholes performed by Golder are also shown on drawing in Appendix B.

Roadway boreholes drilled during Phase I of this fieldwork were advanced using a track mounted CME 55 drill rig equipped with hollow stem augers and standard soil sampling equipment, operated by a specialist drilling contractor, Marathon Drilling Co. Ltd. In-water boreholes in Phase II were advanced using a barge mounted Dierich D-25 drill rig with hollow stem augers and standard soil sampling equipment, operated by a specialist drilling contractor, Landcore Drilling Inc. Due to the difficulties to access inlet and outlet sides and high water level in the vicinity of culvert, the drill rig and the barge were lifted and placed in the water using a crane (90-ton link belt), as shown on the attached photos in Appendix A.

Roadway boreholes drilled at the existing culvert location (BH-2 and BH-3) were advanced to depths ranging between 14.5 m and 14.6 m below ground surface, while the boreholes drilled at the TMB location (BH-7 and BH-8) were advanced to depths ranging between 5.3 m to 6.8 m below ground surface. In-water boreholes (BH-1, BH-4, BH-5 and BH-6) were advanced to depths ranging between 9.7 m and 15.4 m below the water level in the creek at the time of investigation. Except BH-4, all the boreholes drilled at this site were cored approximately 3 m in to the bedrock. BH-4 was terminated at a desired depth of 15.4 m.

The borehole locations (referenced to the MTM NAD83 coordinate system) and their ground surface elevations were surveyed by **exp** personnel. The ground surface elevations, including top of culvert and top of water at the location of existing culvert, were referenced to a geodetic benchmark (HCP 101) located on north-east side of the existing culvert; while the ground surface elevations at the location of proposed TMB location, were referenced to a geodetic benchmark (HCP 175) located on the south-west side of the proposed south abutment of TMB. The elevation of the BMs for HCP 101 and HCP 175 are 313.147 m and 311.228 m, respectively. The benchmarks locations are shown on Drawing 1 in Appendix B.

During the drilling of all 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 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 HQ/NQ double tube wireline core barrel.

Upon completion of the boreholes, ground water level measurements were carried out in boreholes in accordance with MTO guidelines. The recorded ground water levels after completion of drilling boreholes were presented in the borehole log sheets in Appendix C. The roadway 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 the **exp** geotechnical representative who directed the drilling and sampling operation, logged borehole data in accordance with MTO and/or ASTM Standards for Soils Classification, and retrieved soil samples for subsequent laboratory testing and identification.

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

1.3.2 Previous Investigation

The following previous/historical investigation report was provided by MTO:

- Technical Memorandum for Field Investigation for Highway 672 Centerline Culvert at Approximately STA 10+150, Township of Arnold; Assignment #10; Agreement # 5013-E-0012; Golder Associates Ltd.; May 25, 2017.

The technical memorandum and probehole logs prepared by Golder are attached in Appendix F of this report.

1.3.3 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 on all samples and particle size distribution for approximately 25% of the collected soil samples. All of the laboratory tests were carried out according to 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 tests 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 of grain size analyses 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. Probehole logs prepared by Golder after their investigation are attached in Appendix F.

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

1.4.1 Proposed Culvert Replacement Location

The general stratigraphy encountered within the investigated depths of Golder and current investigation are inline. In general, the subsurface conditions along the propose culvert replacement site consists of a layer of sand and gravel fill underlain by a layer of cobbles and boulders fill followed by native deposit of silty sand with gravel and bedrock. Organic silt with sand/sandy silt deposit overlying the bedrock was encountered at the inlet and outlet locations including the potential locations of cofferdams at those ends. A detailed description of the subsurface conditions encountered along the culvert and cofferdams locations is discussed further in subsequent sections. It should be noted that the following sections are based on the geotechnical investigation conducted by **exp**. Since, Golder conducted only probehole (no sampling and laboratory testing), the logs prepared by Golder are not considered here to describe subsurface conditions encountered at site.

1.4.1.1 Fill: Sand and Gravel

Sand and gravel fill was encountered at the surface of roadway BH-2 and BH-3. The fill layer extended to depths ranging between 3.1 m to 4.6 m below ground surface with elevations ranging between 310.7 m and 309.1 m. The explored thickness of this layer was between 3.1 m and 4.6 m.

The composition of this fill layer was generally sand and gravel with some cobbles, trace silt and clay. The material is brown in color, and moist. The SPT ‘N’ values obtained within this layer ranged from 9 to 38 blows per 0.3 m penetration, suggesting loose to dense material, but generally compact to dense in relative density.

Laboratory testing performed on selected samples consisted of eight (8) moisture content tests and three (3) grain size distribution tests. The test results are as follows:

Moisture Content:

- 1.8% to 11%

Grain Size Distribution:

- 26% to 54% gravel;
- 42% to 55% sand;
- 24% silt;

- 1% clay; and
- 4% to 5% 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 result of the grain size distribution tests are also provided on Figure 1 in Appendix D.

1.4.1.2 Fill: Cobbles and Boulders

Cobbles and boulders fill was encountered below the sand and gravel fill layer of BH-2 and BH-3. The fill layer extended to depths ranging between 4.6 m and 6.1 m below ground surface with elevations ranging between 307.6 m and 309.1 m. The explored thickness of this layer was about 1.5 m.

The composition of this layer was generally cobbles and boulders with some sand and some gravel. The SPT ‘N’ values obtained within this layer were well above 100 blows per 0.3 m penetration.

Laboratory testing performed on selected samples consisted of two (2) moisture content tests. The test results are as follows:

Moisture Content:

- 2.4% to 8.8%

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

1.4.1.3 Sand and Gravel

A native sand and gravel layer was encountered below the water in BH-5. The sand and gravel layer extended to depth about 3.1 m below the water surface with elevation about 306.5 m. The explored thickness of this layer was about 2.0 m.

The composition of this layer was sand and gravel, some silt and occasional boulder. The material is brown to grey in color, and wet. The SPT ‘N’ values obtained within this layer ranged from weight hammer (WH) to 16 blows per 0.3 m penetration, suggesting very loose to very dense in relative density. One SPT ‘N’ value of 74 blows per 0.3 m penetration was also recorded within this layer, which could be influence of a boulder encountered.

Laboratory testing performed on selected samples consisted of two (2) moisture content tests. The test results are as follows:

Moisture Content:

- 5.2% to 6.9%

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

1.4.1.4 Silty Sand/Sandy Silt

A native silty sand layer was encountered below the water in BH-1 and BH-4 and below organic silt with sand layer in BH-5 and BH-6. The silty sand/ sandy silt layer extended to depths ranging between 3.2 m and 7.2 m below the water surface with elevations ranging between 302.3 m and 306.3 m. The explored thickness of this layer was between 0.5 m and 4.4 m.

The composition of this layer was generally sand and silt, some gravel, trace to some clay and occasional boulder. The material is brown to grey in color, and wet. The SPT 'N' values obtained within this layer ranged from 2 to 32 blows per 0.3 m penetration, suggesting very loose to dense, but generally loose to compact in relative density.

Laboratory testing performed on selected samples consisted of eleven (11) moisture content and three (3) grain size distribution tests. The test results are as follows:

Moisture Content:

- 11.2% to 30%

Grain Size Distribution:

- 0% to 10% gravel;
- 31% to 73% sand;
- 27% to 51% silt;
- 0% to 10% clay

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

1.4.1.5 Organic Silt with Sand/ Organic Sandy Silt

A native organic silt with sand/organic sandy silt layer was encountered below the silty sand layer in BH-1 and BH-4, below the sand and gravel layer in BH-5 and below the water in BH-6. The organic silt with sand/organic sandy silt layer extended to depths ranging between 4.6 m and 7.6 m below the water surface with elevations ranging between 302.2 m and 304.9 m. The explored thickness of this layer was between 1.5 m and 4.9 m.

The composition of this layer was silt and sand with organics and occasional wood. The material is dark brown in color, and wet. The SPT 'N' values obtained within this layer ranged from weight hammer (WH) to 32 blows per 0.3 m penetration, suggesting very loose in relative density.

Laboratory testing performed on selected samples consisted of ten (10) moisture content and two (2) grain size distribution tests. The test results are as follows:

Moisture Content:

- 40.4% to 112.8%

Grain Size Distribution:

- 0% gravel;
- 5% to 6% sand;
- 89% to 92% silt;
- 3% to 5% clay

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

1.4.1.6 Silty Sand with Gravel/Silty Sand with Frequent Boulders

Native silty sand with gravel to silty sand with frequent boulders layer was encountered below the organic silty with sand layer in BH-1 and BH-4 and below the cobbles and boulders fill layer in BH-2 and BH-3. The silty sand with gravel/ silty sand with frequent boulders layer extended to depths ranging between 7.5 m and 11.6 m below the water surface in BH-1, and BH-4 with elevations ranging between 302.0 m and 298.2 m; and extended to depths ranging between 11.4 m and 11.6 below ground surface in BH-2 and BH-3 with elevations ranging between 302.3 m and 302.1 m. The explored thickness of this layer was between 2.6 m and 7.0 m.

The composition of this layer was generally silty sand and gravel with some cobbles and boulders and trace to some clay. The material is grey in color, and wet. The SPT ‘N’ values obtained within this layer ranged from 3 to 46 blows per 0.3 m penetration, suggesting very loose to dense, but generally very loose to compact in relative density. One SPT ‘N’ value of above 100 blow per 0.2 m penetration was also recorded within this layer, which could be influence of boulders encountered.

Laboratory testing performed on selected samples consisted of twelve (12) moisture content tests and five (5) grain size distribution tests. The test results are as follows:

Moisture Content:

- 0.8% to 33%

Grain Size Distribution:

- 1% to 23% gravel;
- 42% to 91% sand;
- 8% to 56%silt;
- 0% to 16% clay

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

1.4.1.7 Cobbles and Boulders

Cobbles and boulders were encountered underlying the silty sand with frequent boulders layer in BH-4. The cobbles and boulders layer extended to depths about 15.4 m below the water surface with elevation about 294.4 m. The explored thickness of this layer was about 3.8 m. BH-4 was terminated within this layer.

The composition of this layer is mostly cobbles and boulders, some sand and some gravel. The SPT "N" value obtained within this layer was 50 blows per 127 mm penetration suggesting very dense compactness condition. The recovered cored sample obtained within this layer is about 150 mm.

Laboratory testing performed on one collected sample consisted of moisture content test and the test result is as follows:

Moisture Content:

- 0.8% to 33%

1.4.1.8 Bedrock

Bedrock was encountered underlying the silty sand with gravel/ silty sand with frequent boulders layer in all boreholes except in BH-4, which was terminated within the cobbles and boulders layer to its desired depth. The bedrock was encountered at depths ranging between about 6.6 m and 7.5 m below the water surface at the inlet and outlet, and about 11.4 m to 11.6 m below the existing road surface. The bedrock was confirmed by coring of 3.0 m to 3.2 m long rock cores. The elevation of the actual bedrock surface at proposed culvert replacement location ranges from 302.0 m to 303.2 m. Boreholes (BH-1, BH-2, BH-3, BH-5 and BH-6) were terminated within the bedrock level. The bedrock surface depth and elevation encountered at the drilled borehole locations are listed in Table 1.1. Photographs of rock cores are included in Appendix E.

Golder's investigation with rock probeholes showed that the inferred bedrock surface could be approximately between Elev. 305.8 m and 300.8 m at the locations of drilling (Appendix F).

Table 1.1 Depth and elevation of bedrock surface

Borehole	Depth Below Ground Surface (m)	Elevation (m)	Comments
BH-1	7.5	302.0	Bedrock Cored
BH-2	11.4	302.3	Bedrock Cored
BH-3	11.6	302.1	Bedrock Cored
BH-5	7.2	302.3	Bedrock Cored
BH-6	6.6	303.2	Bedrock Cored

Based on the rock cores recovered, the bedrock consists of mafic metavolcanics rock. In general, the rock samples are described as dark grey, with white striations have a fine crystalline structure, slightly weathered, very strong. The Rock Quality Designation (RQD) measured on the rock core samples ranged from approximately 40% to 96.9%, indicating a rock mass of poor to excellent, but generally good to excellent quality.

1.4.2 Temporary Modular Bridge Location

In general, the subsurface conditions along the proposed temporary modular bridge site consist of a layer of sand and gravel fill underlain by bedrock. A detailed description of subsurface conditions encountered is discussed further in subsequent sections.

1.4.2.1 Fill: Sand and Gravel

Sand and gravel fill was encountered at the surface of BH-7 and BH-8 at the proposed TMB site. The fill layer extended to depth ranging between 2.3 m and 3.7 m below ground surface with elevations ranging between 307.7 m and 309.0 m. The explored thickness of this layer was between 2.3 m and 3.7 m.

The composition of this fill layer was generally sand and gravel with some silt, some cobbles, trace organics and trace wood. The material is brown in color, and moist. The SPT 'N' values within this layer ranged from 6 to 31 blows per 0.3 m penetration, suggesting loose to dense but generally compact in relative density.

Laboratory testing performed on selected samples consisted of eight (8) moisture content tests and two (2) grain size distribution tests. The test results are as follows:

Moisture Content:

- 4.1% to 26.7%

Grain Size Distribution:

- 43% to 53% gravel;
- 15% to 54% sand;
- 32% silt;
- 0% clay; and
- 3% 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 result of the grain size distribution tests are also provided on Figure 5 in Appendix D.

1.4.2.2 Bedrock

Bedrock was encountered underlying the sand and gravel fill in all boreholes drilled at the TMB site location. The bedrock was encountered at depths ranging between about 2.3 m and 3.7 m below the ground surface. The bedrock was confirmed by coring of 3.0 m to 3.1 m long rock cores. The elevation of the actual bedrock surface ranges from 307.9 m to Elev. 309.2 m. All boreholes were terminated within the bedrock level. The bedrock surface depth and elevation encountered at the drilled borehole locations are listed in Table 1.2. Photographs of rock cores are included in Appendix E.

Table 1.2 Depth and elevation of bedrock surface

Borehole	Depth Below Ground Surface (m)	Elevation (m)	Comments
BH-7	3.7	307.9	Bedrock Cored
BH-8	2.3	309.2	Bedrock Cored

Based on the rock cores recovered, the bedrock consists of mafic metavolcanics rock. In general, the rock samples are described as dark grey, with white striations have a fine crystalline structure, slightly weathered, very strong. The Rock Quality Designation (RQD) measured on the rock core samples ranged from approximately 66.7% to 7.9%, indicating a rock mass of fair to excellent, but generally good to excellent quality.

1.5 Groundwater Conditions

Information regarding groundwater levels at the site was obtained by measuring water levels in open boreholes of land boreholes (BH-2, BH-3 and BH-8) after completion of drilling. The groundwater levels measured in the boreholes are shown on borehole logs. Water levels measured in open boreholes might not be stabilized due to a short-term observation and using of a wash boring technique to advance the boreholes. Since boreholes BH-1, BH-4, BH-5 and BH-6 were drilled in water, the water level in the pool/creek was measured as a relevant water level for these boreholes.

The groundwater levels measured in open boreholes upon completion of drilling were recorded at 4.1 m (BH-2) and 4.0 m (BH-3) below the ground surface corresponding to Elev. 309.6 m and 309.7 m, respectively at the proposed culvert replacement site location, and about 1.8 m below ground surface (Elev. 309.7 m) at the proposed TMB site location.

In June 2017, the inlet of the existing culvert was completely submerged and it was estimated that the water level was about Elev. 310.3 m. In July 2017, the water level at the inlet side was about Elev. 309.8 m. At the outlet, the water level of the flowing stream was measured approximately at elevation 309.5 m in June and July 2017.

Groundwater levels would be expected to reflect levels in the adjacent open water body/stream and to fluctuate seasonally. 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 as noted during this investigation.

2 ENGINEERING DISCUSSION & RECOMMENDATIONS

2.1 General

This section of the report provides geotechnical design recommendations for replacement of Victoria Creek Culvert on Highway 672, north of the Highway 66 junction in Township of Arnold, Northeast of Kirkland Lake, Ontario, Ministry of Transportation (MTO) Northeastern Region. The recommendations are based on interpretation of the factual data obtained from the boreholes/rock probes advanced during the current investigation at the site performed by **exp** and previous preliminary investigation performed by Golder Associates Ltd. dated May 25, 2017. The compiled factual data is presented in **Part I-Foundation Investigation Report** of this report. The interpretation and recommendations provided are intended solely to permit designers, to assess foundation alternatives and design new structure at the site. 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.

It is understood that a washout has recently occurred at the project site and as a temporary and emergency measure, twin 2.4 m diameter temporary CSP pipes were installed. In addition, earthfill consisting of sandy soils along with some large rock and boulders at the bottom were placed to restore the highway embankment. The twin pipes were installed at about 8 m south from the existing pipe and at a much shallower depth with invert located at the obvert elevation of the existing pipe approximately, while the existing pipe remains in-place. The diameter of the existing CSP pipe is estimated to be approximately 2.4 m, and its alignment is skewed to the highway central line (~67°). Based on the contract drawings provided by MTO, it is understood that the existing culvert will be replaced with a 5.0 m wide and 2.8 m high precast concrete box culvert, located at the location of the existing pipe, but not skewed. It is proposed that the temporary twin pipes will be functioning as a bypass channel during the replacement and removed after the new structure is in place. The construction site has to be protected by a temporary dewatering system (i.e. cofferdam) designed by Contractor. No grade raise is anticipated. A temporary modular bridge (TMB) at the adjacent abandoned road which will serve as a detour during the construction is proposed. Foundation design recommendations are required for the new culvert, TMB and cofferdams.

This part of the report addresses the geotechnical design of the foundations for the new culvert and TMB 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 *Canadian Foundation Engineering Manual (CFEM)* (2006), *MTO Gravity Pipe Design Guidelines* (May 2007) and generally accepted good practice. Pertinent construction issues from a geotechnical standpoint are examined in general accordance with the Terms of Reference provided to us at May 23, 2017 together with the MTO request email. The assessment involved review of options for replacement of the existing culvert at its location using an open-cut replacement method as well as construction of TMB at the old adjacent road as a detour. The protection of construction site by cofferdams is also addressed.

2.2 Expected Ground Conditions

The following ground conditions along the proposed culvert alignment are evident from the current investigation:

a) Highway 672 is a two lane, north/south unpaved roadway at the site location. The highway crosses a 2.4 m span corrugated steel pipe culvert with approximately 3.7 m of embankment fill above the culvert crown, and approximately 1.7H:1V sideslopes after earthfill was placed to restore the highway embankment. The elevation of the crest of the roadway is about Elev. 313.7 m. As mentioned before, twin 2.4 m diameter temporary CSP pipes were recently installed approximately 8 m south of the existing culvert

b) The highway embankment consists of sand and gravel fill (~3.1 m to 4.6 m thick) underlain by cobbles and boulders fill (~1.5 m thick). The embankment fill is underlain by loose to compact silty sand with gravel (~5.3 m to 7 m thick) to about 11.5 m depth from the ground surface where bedrock was encountered (~Elev. 302.2 m).

c) At the outlet side, below approximately 0.9 m to 1.1 m deep water, a surficial layer of native very loose to compact silty sand to sand and gravel (~2 m to 2.3 m thick) is underlain by very loose organic silt with sand (~1.5 m to 1.7 m thick) and loose sandy layer (~2.6 m thick) to about 7.2 m to 7.5 m below the water surface. The soil deposits are underlain by bedrock at approximate Elev. 302.0 m to 302.3 m.

d) At the inlet side, below approximately 1.2 m to 1.7 m deep water, a layer of native very loose to compact silty sand (~4.4 m thick) underlain by layers of soil deposits: very loose organic sandy silt (~1.5 m thick), very loose to dense silty sand with frequent boulders (~4 m thick), and a layer of cobbles and boulders (~3.8 m thick within the depth of exploration) to about 15.4 m (Elev. 294.4 m) below the water surface. The bedrock underlying the site was not encountered at the inlet location of the culvert. However, it is encountered at approximate Elev. 303.2 m in the borehole drilled south from the inlet.

e) Based on information provided by MTO, the invert of the new culvert is proposed to be at Elev. 306.95 m at the inlet side, and assuming some slope of 1% of the pipe it is estimated that the invert at the outlet will be at Elev. 306.6 m. The foundation soil below these levels is anticipated to be mostly native loose to compact silty sand with gravel with N values of 6 to 19, underlain by bedrock. However, at the outlet side a very loose organic silt with sand layer is present and must be excavated and replaced with clean and compactable soil such as Granular A or Granular B Type II as discussed below.

f) The top of encountered bedrock slopes down toward the north from Elev. 302 m at the location of temporary culvert to more than Elev. 294.4 m at the location of existing culvert (west side of the embankment).

g) At location of TMB, a surficial layer of sand and gravel fill (~2.3 m to 3.7 m thick) is underlain by bedrock.

h) At potential location of a cofferdam at the inlet side, deposits of silty sand and organic silt with sand (~5.4 m to 9.9 m thick) are underlain by the layer of cobbles and boulders and/or bedrock (~between Elev. 302.9 m and Elev. 297.9 m). At the outlet side, deposits of silty sand, organic silt with sand, and silty sand with frequent boulders (~6.1 m to 6.6 m thick) are underlain by bedrock at Elev. 302.3 m.

i) In June 2017, the water level in the pool at the inlet side was higher than the crown of the existing culvert (~Elev. 310.0 m), so the culvert was completely submerged. The water level was approximately at Elev. 310.3 m. In July 2017, the water in the pool was relatively lower having the measured elevation of 309.8 m. At the outlet side the water level of the flowing creek was about 306.5 m in June and July 2017. The difference in the water tables at the inlet and outlet side of the highway embankment creates a hydraulic gradient, so the seepage at the toe of the embankment at the outlet side is evident. The groundwater table in the embankment fill reflects the water levels on the both sides of the embankment. Therefore, seasonal variations in the water table should be expected, with higher levels occurring during wetter periods of the year (such as spring thaw and late fall) and lower levels during drier periods. In addition, seepage through the embankment body could be expected if the water level at the inlet side is higher than that at the outlet side. However, with a proper design of the new culvert and drainage of the upstream side, it is expected that the hydraulic gradient seepage through the embankment will be minimized.

2.3 Structure Foundations

2.3.1 Culvert Replacement

Based on the TOR, it is understood that the following options were considered for the Victoria Creek culvert replacement:

- box culvert on shallow foundation,
- open footing culvert supported on shallow foundation or deep foundations,
- corrugated steel pipe culvert supported on shallow foundations,
- steel sheet pile abutments with precast concrete decking, and
- short-span bridge.

Based on the subsurface information obtained on the site, the native compact sand with gravel is considered suitable foundation soil for the support of all replacement options considered, assuming that underlying organic soils and any other soft or very loose materials are to be replaced with clean and compactable soils. However, the option with steel sheet pile abutments and precast concrete decking is assessed as unsuitable at this location due to difficulties to drive the sheet piles through the embankment fill with the layer of cobbles and boulders. Considering the presence of uneven bedrock surface and deep layer of cobbles and boulders at the inlet side the open footing culvert supported on shallow foundation is more preferable option than the deep foundation solution. Beside the geotechnical criteria, the choice of culvert type also depends on parameters such as the initial cost, maintenance costs, hydraulic performance, ease of construction, water and soil corrosiveness, salvageability and local availability of material and equipment.

As noted, any loose and/or soft soils encountered below the existing embankment should be excavated and removed to firm bearing of native soils and the grade restored with engineered fill. If the depth of excavation to remove unstable soils is excessive, using a geotextile fabric, such as Terrafix 270R or equivalent, in conjunction with engineered fill can be considered to assist in providing a stable base for the new culvert. Based on previous experience, typically a minimum of 450 mm of a clear stone accordance with OPSS.PROV 1004 over geotextile fabric would establish a stable bearing surface. The fabric should be installed in a manner to mitigate the migration of fines from adjacent material.

Based on subsoil conditions encountered at the site, Table 2.1 below compares the possible structure options from a foundation design and constructability perspective with their advantages and disadvantages. Considering the subsurface soil condition in the embankment and below the embankment, the use of precast rigid frame concrete box culvert is ranked highest for the criteria evaluated.

Table 2.1 Evaluation of foundation alternatives

Options	Rank	Advantages	Disadvantages	Relative Costs	Risks/ Consequences
Precast rigid frame concrete box culvert	1	▪ Straightforward construction ▪ Reduced construction period, consequently traffic management and water control period ▪ Reduced excavation depth ▪ Can be more readily installed during cold weather conditions	▪ If floor is thin and poorly reinforced, it may heave and crack ▪ During high flows, the concrete floor can be undermined ▪ Susceptible to defects/leakage at joints ▪ Requires bedding material ▪ Disturbance of natural streambed ▪ Possible sediments accumulation in the upstream of the culvert	▪ Low	▪ Risk of unacceptable differential settlements if the entire foundation is not supported on the competent soil ▪ Risk of leaking from joints if not properly installed
Cast-in-place rigid frame open footing concrete culvert	3	▪ Wider span may consider to maintain existing channel and so allows for natural streambed to remain intact ▪ Less accumulation of sediments in the upstream of culvert	▪ Deeper excavation or below water excavation may required ▪ Dewatering system required ▪ Require placement of lean concrete ▪ Risky due to sloping bedrock	▪ Likely more expensiv e than Option 1	▪ Risk of unacceptable differential settlements if the entire foundation is not supported on the competent soil ▪ Risk of sloping bedrock ▪ Risk of delay in construction due to deeper excavation

Options	Rank	Advantages	Disadvantages	Relative Costs	Risks/ Consequences
					below water if proper dewatering is not maintained ▪ Higher scour risk
Corrugated steel pipe culvert	2	▪ Straightforward construction ▪ Reduce construction period, consequently traffic management and water control period ▪ Reduce excavation depth	▪ Require bedding material ▪ Limited design life ▪ Potential for Corrosion ▪ Disturbance of natural streambed ▪ Possible sediments accumulation in the upstream of the culvert	▪ Low to medium	▪ Risk of unacceptable differential settlements if the entire foundation is not supported on the competent soil ▪ Risk of structure segment loss due to corrosion
Steel sheet pile abutment with precast decking	5	▪ Higher geotechnical resistance value ▪ Environmentally friendly ▪ No need for dewatering and cofferdam ▪ Serve as dual purpose of support culvert foundation and retaining backfill ▪ Reduce construction period	▪ Require pile driving equipment ▪ Difficult driving condition due to cobbles and boulders encountered ▪ May require anchors to support possible later movement ▪ Durability issue with sheet pile walls	▪ Medium to High	▪ Risk of difficulties during sheet pile driving due to cobbles and boulders present
Short-span bridge	4	▪ Wider span may consider to maintain existing channel and so allows for natural streambed to remain intact ▪ Higher geotechnical resistance value ▪ Easy to construct ▪ Reduce construction period	▪ Deeper excavation or below water excavation may required ▪ Dewatering system required	▪ High	▪ Risk of delay in construction due to deeper excavation below water if proper dewatering is not maintained

2.3.1.1 Shallow Foundations

2.3.1.1.1 Geotechnical Resistance

Based on the subsurface stratigraphy encountered at this site and the assumed invert elevation of the new culvert, the recommended founding depths and geotechnical resistances for a structure founded on undisturbed competent natural soils are tabulated below.

Table 2.2. Recommended spread footing design parameters

Culvert Type	Founding Elevation (m)	Footing Size (m)	Founding Soil Type	Factored Geotechnical Resistance at ULS (kPa)	Geotechnical Reaction at SLS[2] (kPa)
Precast rigid frame concrete box culvert/ CSP pipe culvert	~306.6	5.4	Native compact silty sand with gravel	260	175
Cast-in-place rigid frame open footing concrete culvert	~304.6[1]	1.0	Native compact silty sand with gravel	300	200
Short-span bridge founded on shallow foundation	~304.6[1]	2.0	Native compact silty sand with gravel	300	200

Notes:
1. *Below the frost line. Requires deeper groundwater control.*
2. *For maximum settlement of 25 mm*

It is assumed that underlying organic soils and any other soft or very loose materials are to be replaced with clean and compactable soil such as clear stone, Granular A or Granular B Type II. Given that no grade raise is planned, the anticipated maximum total settlements for the new proposed culvert are not expected to exceed 25 mm for construction done in accordance with these design parameters and assuming good construction practice including sound base preparation.

2.3.1.1.2 Resistance to Lateral Loads

Resistance to lateral forces/ sliding should be calculated in accordance with Section 6.10.5 of the CHBDC, using the following parameters:

Table 2.3 Recommended parameters for calculation of unfactored horizontal resistance

Interface and Loading Conditions	Parameters
Between native silty sand with gravel and pre-cast concrete	Coefficient of friction (tan δ)=0.4
Between native silty sand with gravel and cast-in-place concrete	Coefficient of friction (tan δ)=0.55

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

2.3.1.1.3 Frost Protection

The frost depth in the area of the culvert is estimated to be approximately 2.3 m in accordance with OPSD 3090.100. During construction of any temporary and permanent support system using shallow foundations should be provided a minimum 2.3 m of soil cover or equivalent frost protection should be provided using thermal insulation. This frost protection requirement applies to the rigid frame open footing culvert option.

If the frost penetration line is at or above top of the culvert the backfill and cover for these culverts should be as per OPSD 803.010. Where less than 2.3 m of earth cover is provided above the top of the culvert, a frost taper should be included as per OPSD 803.010 for the concrete culverts with spans less than or equal to 3.0 m or MTOD 803.021 for the culvert with span more than 3.0 m.

2.3.1.2 Deep Foundations

2.3.1.2.1 Driven Steel Piles

Considering the site-specific conditions, steel piles driven to bedrock can be used to support a culvert footing or short-span bridge footings. Steel piles can consist of steel (minimum 350 MPa) open or closed pipe (244 m x 9mm or 324 mm x 10 mm) or H-piles (HP 310 x 79 or HP 310 x 110) sections. The piles will be installed through the gravelly sand with cobbles/cobbles and boulders fills to compact silty sand with gravel layers, and are expected to terminate on bedrock. It is anticipated that pile cap elevations would be below a frost depth of 2.3 m. Based on the depth to bedrock encountered in boreholes and probeholes drilled adjacent to proposed culvert locations, the pile tip elevation is estimated to be in the range of Elev. 297 m and Elev. 305.8 m since the bedrock is sloping. Such piles, driven into the unyielding bedrock can be designed using the factored (0.4) resistance values presented in the table below:

Table 2.4. Factored geotechnical resistance values (ULS) for driven steel piles

Pipe Size or HP Section	Pile Founding Stratum	Factored Geotechnical Axial Resistance at ULS (KN/pile)
244 mm x 9 mm	Bedrock	950
324 mm x 10 mm	Bedrock	1350
HP310 x 79	Bedrock	1450
HP310 x 110	Bedrock	2000

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

2.3.1.2.2 Caissons

Alternatively, the open footing culvert may also be supported on caissons drilled through native compact silty sand with gravel with occasional boulders and socketed into the bedrock. The high axial capacity of caissons would result in fewer units being required to support the culvert than that required for the steel piles. However, there will be difficulty in socketing the caissons within sloping bedrock and achieving an adequate seal. Temporary liners and tremie concrete will be required to install caissons at this site.

Table 2.5 below provides the factored geotechnical axial resistances for different caisson diameters socketed a minimum of 2 m in to the bedrock. The given values for caissons were results mainly from the shaft resistance of the bedrock socket. The end-bearing will be neglected due to the difficulties in cleaning and inspecting the base of sockets.

Table 2.5. Caisson's geotechnical resistance

Relevant Borehole	Foundation Elevation (m)	1.0 m Diameter Caisson		1.2 m Diameter Caisson	
		Factored ULS (kN)	SLS (kN)	Factored ULS (kN)	SLS (kN)
BH-3	300.1	5,500	N/A	6,500	N/A

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

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

a) Cross-hole acoustic testing and backscatter gamma ray (gamma-gamma) tests trough access tubes installed within the caissons during the placement of the concrete; or

(b) Sonic echo tests. The advantage of these tests is that they do not require preparation during construction of the caissons. The disadvantage is that these tests do not identify all imperfections in a caisson, but provides information about continuity, defects, such as cracks, necking, soil incursions, changes in cross section and approximate pile lengths, unless the pile is very long or the skin friction is too high.

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

Giving the uncertainties associated with the sloping bedrock, presence of cobbles and boulders at the inlet, and cleaning and inspection of the caisson base, this foundation type is not the preferred option.

2.3.2 Temporary Modular Bridge

Based on the TOR, a Temporary Modular Bridge (TMB) is considered to be built at the detour, so two boreholes were drilled at locations of proposed abutments. Considering subsurface conditions encountered in the geotechnical soil borings performed for that TBM (i.e. BH-7 and BH-8), shallow foundations (i.e spread footings) founded on bedrock is recommended as the most preferable alternative from geotechnical/foundation perspectives for the TMB.

As an alternative, it is recommended to inspect and assess the condition of existing concrete abutments by a structural engineer. If the existing abutments are in acceptable condition and can be used with some minor repair, they can be considered as abutments for the new TMB.

2.3.2.1 Shallow Foundations

Spread footings placed on the surface of the properly prepared bedrock or on mass concrete placed directly on the bedrock may be designed based on a factored geotechnical axial resistance at Ultimate Limit States (ULS) of 10,000 kPa. The recommended founding elevations of footings at north and south abutment are approximately at 309.2 m and 307.9 m, respectively. The geotechnical resistance at Serviceability Limit States (SLS) for 25 mm of settlement will be greater than factored geotechnical axial resistance at ULS, since bedrock (or mass concrete over bedrock) is considered to be an unyielding foundation and, as such, ULS conditions will govern for this foundation type. It is assumed that the mass concrete will be of compressive strength equal to or greater than the concrete footings (assumed to be 25 MPa or greater).

All loose, shattered and/or fractured rock within the footprint of the footings and at the footing level should be removed and replaced with mass concrete. OPSS 902 and SP109S12, should be included in the Contract Documents to address the requirements for construction and inspection of footings on bedrock.

If construction could be completed within one year before winter, then the TMB can be founded on existing sand and gravel fill at approximate elevation 310.8 m with the following geotechnical resistances:

- o Factored Geotechnical Resistance at ULS of 300 kPa
- o Geotechnical Reaction at SLS of 200 kPa

The geotechnical resistances provided above are given under the assumption that for concentric vertical loading condition only. Where the load is not concentric vertical loading, load eccentricity and load inclination effects need to be considered.

It is recommended to make a proper assessment if scour protection is a necessity during the period of TMB existence.

2.3.2.1.1 Resistance to Lateral Loads

Resistance to lateral forces/ sliding should be calculated in accordance with Section 6.10.5 of the CHBDC, using the following parameters:

Table 2.6. Recommended parameters for calculation of unfactored horizontal resistance

Interface and Loading Conditions	Parameters
Between cast-in-place concrete and bedrock	Coefficient of friction (tan δ)=0.7
Between cast-in-place concrete and sand and gravel fill	Coefficient of friction (tan δ)=0.5

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

For footings supported on the bedrock, the sliding resistance can be supplemented by dowelling into the bedrock. The horizontal resistance of the dowels is dependent on the strength of the bedrock, grout and steel. The dowels should have a minimum embedded length within the unfractured bedrock of 2 m. The structural strength of the dowel and compressive strength of grout should be designed in the same way as a dowel embedded into the concrete, assuming that the unconfined compressive strength of the grout will be similar to that of the concrete. If dowels are included in the design, a Non-Standard Special Provision (NSSP, see Appendix C attached) should be included to address dowelling materials, installation and testing.

For uplift resistance from the dowels, an ULS design value of 700 kPa may be assumed for the grout-to-rock bond strength, based on applying a factor of 0.4 to the ultimate bond strength estimated to be about 1,700 kPa. It is expected that ULS conditions will govern for the installation of dowels, since the geotechnical resistance at SLS assuming displacement of 25 mm is greater. The upper 0.5 m of the bond should be ignored in the calculation of required bond length since that zone of the rock is weathered or disturbed. The final bond strength for the rock-grout interface should be verified in the field by pull-out testing.

2.3.2.1.2 Frost Protection

The frost depth in the area of the temporary modular bridge is estimated to be approximately 2.3 m in accordance with OPSD 3090.100. During construction of any temporary and permanent support system using shallow foundations should be provided a minimum 2.3 m of soil cover or equivalent frost protection should be provided using thermal insulation. Since the footing for the temporary bridge will be founded on bedrock requirement of full frost protection is not applicable.

2.3.3 Temporary Cofferdam

The temporary twin pipes will be used as a bypass creek channel during the replacement of the existing culvert. However, temporary cofferdams will be required at both upstream and downstream ends to envelop the construction site and keep it free of water during replacement of the existing culvert. Considering the ground condition at the bottom of the pool/creek encountered in the boreholes drilled in the water (i.e. BH-4 and BH-6 at the inlet and BH-1 and BH-5 at the outlet side) two types of cofferdams, i) sheet pile wall and ii) rockfill dam with a impervious water barrier, could be considered.

Based on the geotechnical conditions at potential locations in the pool/creek, suitably designed steel sheet pile walls can be used as cofferdams at this site. If a cantilever system is used, an embedded depth of sheet piles can be approximately 2.0 to 2.5 times of its exposed height which depends of the surrounding water level. The proposed sheet pile wall should be at least one meter above the designed HWL defined by the Hydraulics Engineer. It should be noted that the level of water at the inlet and outlet sides could be different as measured during this investigation. The required minimum section modulus and embedment pile length should be designed based on the recommended design parameters. Cobbles and boulders were noted to be contained within the pool/creek deposits and underneath the foot print of existing embankment, therefore care has to be taken during installation of sheet piles. A Non-Standard Special Provision (NSSP) should be included in the Contract to alert the contractor the cobbles and boulders may present within the pool/creek deposit in the selection of the appropriate equipment and procedures for piling. An example of NSSP is included in Appendix I. This NSSP should be included in Contract Package. For detailed design an additional investigation should be required.

Northern Ontario is known to have areas of steeply sloping bedrock. This should be considered during design and construction of protection systems. The Contract Drawings should reflect these constrains.

Alternatively, a rockfill cofferdam can be used. This cofferdam will have to be constructed to the same topographic constraints as the sheet pile cofferdam, i.e. at each end of the existing culvert. The size of material suitable for use depends on the erosion potential, stream flow velocity, etc. The rockfill cofferdam should be designed with a more impervious water barrier at the outside face to create a more watertight enclosure. Schemes involving 2-inch minus crusher run with finer facing material upstream have been successfully used in similar settings. Any required permitting must be determined.

As mentioned, which cofferdam system is best suited depends on many technical and economic factors. The advantages and disadvantages of both cofferdam systems are summarized in Table 2.1. Given the soil conditions, topography of the surrounding terrain and available space, the use of a suitably designed steel sheet pile system of sufficiently robust cross-section is recommended for the inlet and outlet of structural culvert. However, the presence of cobbles and boulders has to be considered. In addition, driving the sheet piles perpendicular to the highway into the embankment slopes to prevent water getting in through the sides could be difficult considering that the embankment consists of cobbles and boulders fill. In that case the sheet pile cofferdam could be accompanied with a rockfill cofferdam placed at the sides.

Table 2.7 Comparison of cofferdam systems

Option	Rank	Advantages	Disadvantages	Relative Cost	Risk/ Consequence
Steel sheet piles	1	▪ Provides more watertight base ▪ Structural elements and seals easier to positively construct ▪ Increased safety with appropriate design ▪ Easily removed ▪ Less seepage ▪ Reusable	▪ More costly ▪ More likely time consuming for installation ▪ May present issues for seepage and/or piping ▪ Larger machines required ▪ May require bracing ▪ <u>May face difficulty driving through the creek deposits because of presence of cobbles and boulders</u> ▪ May require strengthening toe of sheet pile	MEDIUM TO HIGH	▪ Possible piping problem ▪ May take longer to install ▪ Difficulties in driving sheet piles due to presence of cobbles and boulders and possible steeply sloping bedrock ▪ Environmental permits
Rockfill	2	▪ Less costly ▪ Relatively less time consuming for installation ▪ Native material can be usable ▪ Not affected by presence of cobbles and boulders	▪ Require more space for installation ▪ Less safe ▪ Subjected to wave erosion ▪ Less watertight ▪ Prone to land shifts, slides and collapse ▪ More likely time consuming to remove	LOW TO MEDIUM	▪ Less stable and safe. May generate 'mud waves' ▪ May take longer to remove ▪ May require to install clay cutoff ▪ More dewatering ▪ Environmental permits

The design of these cofferdams, which are temporary retaining structures is the responsibility of the Contractor. The cofferdam must be designed to withstand the anticipated design loads and to be watertight as practically possible. The Contractor is also responsible for cofferdam's materials, construction, monitoring and removal.

As can be seen in the table, the steel sheet piling is ranked as more practical for this project, noting the possible presence of cobbles and boulders in the creek deposits. Design and construction specification for the chosen temporary cofferdam system should be prepared in accordance with OPSS 539 (Construction Specification for Temporary Protection Systems) by the Contractor. Pilling should be in accordance with OPSS 903. Cantilevered walls should be designed for the earth pressures shown in subsequent Section 2.4 and earth pressure diagram shown in CFEM Figure 26.3. Besides design and construction of the temporary cofferdam system, the Contractor is also responsible for its materials, maintenance, monitoring and removal. The temporary cofferdam shall be fully removed, unless it is specified in the Contract Documents that the cofferdam system may be partially left in place. The method and sequence of removal shall be so that there shall be no damage to the new work, existing work, and facility being protected.

2.3.3.1.1 Dewatering

Dewatering requirements behind the cofferdams to keep the construction site dry will be impacted by water levels in the creek at the time of construction activities. Dewatering shall be carried out in accordance with OPSS. PROV 517(Construction Specification for Dewatering of Pipeline, Utility, and Associated Structure Excavation) and SP NO. 517F01 (July 2017). It is responsibility of the Contractor to propose a suitable dewatering system based on the time of construction, water levels and creek flow conditions for prior approval of the MTO. The method used should not undermine the existing temporary twin culverts, highway embankment or adjacent side slopes. In this connection the provision of toe protection at side slopes during drawdown may be required to minimize sloughing and undercutting during dewatering.

Dewatering may require water taking permits (i.e. Permit To Take Water PTTW). A PTTW is required for any water taking if the volume exceeds 50,000 L/day. The rate and volume required for dewatering will be dependent on construction methods and staging chosen by the Contractor.

2.3.3.1.2 Piping

Since the sheet piles will be embedded into pervious materials (silt and sand) piping might occur when an unbalanced hydrostatic head causes large upward seepage pressures in the soil at the bottom of the inside cofferdam. Piping should be controlled by lowering the water table outside the cofferdam or driving the sheeting to sufficient depth to mitigate against piping.

In case of rockfill cofferdam, piping can be control by installing clay cutoff trench, slurry trench or impervious blanket at upstream of cofferdam.

2.4 Lateral Earth Pressure on Structures

Cofferdams, culvert walls and temporary shoring, if any, should be designed to resist lateral earth pressure. 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 cut

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

The above expression does not take into account hydrostatic pressure, which must be included for the groundwater levels measured on the site. Table 2.8 lists earth pressure parameters for given materials. These recommendations assume level backfill and ground surface behind the walls.

Table 2.8 Material types and earth pressure properties

Material	Unfactored Friction Angle φ'	Coefficient of Active Earth Pressure (K_a)	Coefficient of Passive Earth Pressure (K_p)	Coefficient of Earth Pressure At-Rest (K_o)	Unit Weight γ kN/m^3)
Compacted Granular A	35	0.27	3.69	0.43	22
Compacted Granular B, Type II	35	0.27	3.69	0.43	22
Gravelly Sand with Cobbles Fill	33	0.29	3.4	0.45	21
Cobbles and Boulders Fill	36	0.26	3.87	0.41	20
Native Loose to Compact Silty Sand with Gravel	30	0.33	3.0	0.5	19
Native Very Loose Organic Silt with Sand	24	0.42	2.46	0.59	16
Native very Loose to compact Silty Sand	28	0.36	2.13	0.53	18

Material	Unfactored Friction Angle φ'	Coefficient of Active Earth Pressure (K_a)	Coefficient of Passive Earth Pressure (K_p)	Coefficient of Earth Pressure At-Rest (K_o)	Unit Weight γ kN/m^3)
Native Loose to Dense Silty Sand with Frequent Boulders	31	0.32	1.94	0.48	19

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.

It is likely that bracing for the temporary support system will be required at a maximum interval of 5 m. For multiple support systems refer to Canadian Foundation Engineering Manual (CFEM) for apparent earth pressure distributions (CFEM, Section 26.10.3, Figure 26.8)

2.5 Seismic and Liquefaction Potential Consideration

Seismic characterization of the site must be compliant with the Canadian Highway Bridge Design Code CHBDC (CAN/CSA-S6-14). The potential for seismic loading must be considered for design in accordance with Section 4.4 of the CHDBC with respect to soil conditions encountered at the site. Table 4.1 in CHBDC (see Clause 4.4.3.2) shows site classification for seismic site response based on soil average properties in top 30 m. The borehole information at the TMB shows the presence of sand and gravel fill underlain by bedrock. Based on these soil characteristics, the site class for this site is estimated to be Class “C” according to Table 4.1.

From the Natural Resources Canada website, 2015 NBCC seismic hazard values are obtained using the site location coordinates (48° 11' 31.656" N, 79° 51' 35.675" W) and the damped reference spectral accelerations for the project site are Sa(0.2)=0.058g, Sa(0.5)=0.039g, Sa(1.0)=0.022g, Sa(2.0)=0.011g and the reference peak ground acceleration (PGA) is 0.033g (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.

Based on soils and groundwater condition encountered at the site, no liquefaction is expected due to the ground motion from an earthquake having 10% probability of exceedance in a 50-year period.

2.6 Construction Considerations

2.6.1 Excavations

It is understood that open-cut construction is being preferred as a culvert replacement option at this site and the temporary twin culverts will be used to maintain surface water flow to the outlet. The open-cut method allows complete removal of the existing culvert, but it requires disruption of traffic utilizing a local detour. To facilitate the replacement of existing culvert, and be able to maintain the local flow of traffic during that replacement, the abandon existing road located west of current Highway 672 (see Drawing 1 in Appendix B) is proposed to be used as a local detour. As mentioned before, the use of this local detour will involve construction of TMB at the location of old bridge previously existed at the abounded road.

All excavations at this site must be conducted in accordance with the Occupational Health and Safety Act (OHSA) and Regulations for Construction (O. Reg. 213/91). All fills (i.e. gravelly sand with cobbles fill) may be classified as a Type 3 soil above the groundwater table in conformance with the OHSA. The native silty sand soils below the groundwater table may be classified as a Type 4 soil. It is expected that most of excavations will be above the groundwater levels except those at the invert level for culvert replacement. To avoid disturbance of the founding subgrade and to allow placement of backfill in dry conditions, groundwater must be controlled to below the proposed invert excavation levels prior to digging to final levels. As mentioned before, the ingress of surface water must be controlled using a suitable system as well.

Temporary excavation side slopes for Type 3 soil should not exceed 1H:1V in accordance with OHSA, and 2H:1V is recommended for global stability of these deep cuts where excavation will be left open for some time. Temporary excavation side slopes for Type 4 soils should not exceed 3H:1V where applicable.

2.6.2 Temporary Protection

Temporary excavation support systems, if required to protect the existing twin culverts, 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. The complete design, construction, monitoring and removal of the installed protection system should be a responsibility of the Contractor. Due to nature of this application it is expected that much of temporary shoring will be decommissioned in place noting the high cost for removal. Decommissioning must be consistent with good practice to avoid interference with highway systems and utilities, if any. The protection system should be designed to provide protection for excavations as required by the OHSA, at locations specified in the contract, and at any locations where the stability, safety or function of an existing structure (i.e. temporary twin culverts) may be impaired by construction work.

Since cobbles and boulders fill layer ($N \geqslant 100$) exists within the highway embankment at this site, the shoring system such as soldier piles and timber lagging may be considered for design. It should be designed based on the earth pressures coefficients and soil parameters provided in Section 2.4. The actual depth of embedment should be determined by balancing moments about the pile tip. However, considering the height of the roadway embankment and the depth of the bedrock a temporary shoring system with additional anchorage or tiebacks may be required for lateral resistance. Conventional practice is to incorporate either buried deadman anchors or soil grouted anchors. Alternatively, a system of rakers can be used for support.

Deadman anchors can be designed based on the earth pressure coefficients and soil parameters provided in Section 2.4. For this project, either continuous or individual concrete block anchors would likely be appropriate. The anchor resistance is provided by a combination of the dead weight and passive resistance. For the full passive resistance to be realized with no load transfer to the wall, the anchor needs to be fully beyond the active wedge acting on the wall. Pressure grouted soil anchors can be also designed in a preliminary fashion in accordance with Section 26 of the CFEM (2006). Based on the generally compact soils at this site, the estimated factored (0.4) ULS resistance of grouted anchors would be 220 kN/m length. Detailed design would be completed following the design of the wall and the loads have been established. Normally, such anchors are supplied and installed/tested by specialist vendors/contractors.

For design of the timber lagging, earth pressures can be reduced by 25 percent to account for soil arching effects. This is provided that the center-to-center spacing of the soldier piles does not exceed 2.5 m. Excavation can proceed following installation of the soldier piles. The unshored height of the excavation should not exceed 1.2 m at any given time. No excavation height should remain unshored for more than 24 hours.

Cobbles and boulders were encountered during the investigation. Therefore, a Non-Standard Special Provision (NSSP) to alert the contractor about the presence of cobbles and boulders is included in Appendix I.

As mentioned above, the protection system should be designed for the Performance Level 2 (for small, less important sections). The minimum requirements for monitoring should include the survey measurements of 6 m apart scaled targets attached to the shoring wall at the elevations specified. If movement approaches the allowable limit of 25 mm (Performance level 2), suitable measures should be taken to ensure stability of the protection system and to ensure that the movement does not exceed the performance level specified.

2.6.3 Groundwater Control

At the time of investigation (June and July 2017), the groundwater/water level at the embankment and in the creek was encountered between Elev. 309.5 m (outlet side) and 310.3 m (inlet side), while the excavation to the foundation level has to be carried out to Elev. 306.6 m. Therefore, it is possible that the water table is about 2.9 m to 3.7 m above the bottom of excavation depending on the time of construction. Considering that the soils encountered below the groundwater table and within potential excavation depths consist of native silty sand with gravel, it is assessed that these soils are susceptible to disturbance from groundwater and mobilized equipment. Therefore, the groundwater level needs to be controlled to at least 0.5 m below the excavation level to avoid disturbance, and any surface or groundwater seepage should be removed from the excavation prior to the culvert bedding material placement of granular backfill in the dry. In general, where the excavation base is within 0.5 m of the prevailing groundwater level at the time of construction, it is anticipated that control of seepage can be accomplished by using properly filtered sumps. However, given the conditions at the site, it is expected that positive dewatering systems will be required to control the groundwater seepage. Alternatively, clear stone wrapped with geotextile may be considered as a bedding material to construct culvert under the less dry condition.

Dewatering shall be carried out in accordance with OPSS.PROV 517 and SP No.517F01 (July 2017). It is responsibility of the Contractor to propose a suitable dewatering system based on the time of construction, water levels and flow conditions for prior approval of the MTO. The method used should not undermine the existing road embankment or adjacent side slopes. In this connection, the provision of toe protection at side slopes during drawdown may be required to minimize sloughing and undercutting during dewatering. Alternatively, and in accordance with SP 517F01, the dewatering systems may be completed by a design Engineer and design-checking Engineer with a minimum of 5-year experience. For this application, this is considered a suitable approach but the owner should make final decision. Based on the estimated permeability of silty sand (k~ 5 x 10-5 m/s), the preconstruction survey distance should be approximately 100 m, if any.

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

2.7 Culvert Bedding

MTOD 803.021, OPSD 3101.150 and Figure C6.20 of (CHBDC) which are included in Appendix H provide the bedding, embedment, cover and backfill standards for the concrete box culverts. According to these standards the culvert bedding should consist of Granular A (OPSS.PROV 1010) with thickness of 300 mm beneath the culvert and extend a minimum of 500 mm horizontally on either side of the culvert edge. The bedding material should be placed in layers not exceeding 200 mm in thickness, loose measurement, and compacted accordance with OPSS.PROV 501 before a subsequent layer is placed in accordance with OPSS. PROV 401.

Prior to placing any fill material, the exposed native subgrade should be inspected according to OPSS 902 and SP109S12. A non-woven geotextile separator is to be placed between the approved subgrade and the compacted fill to assist in material placement and maintain the integrity of the founding soil along the entire length of the culvert. The geotextile separator is to be a Class II non-woven material with an equivalent opening size of 75-150 µm.

For the site area, a frost penetration depth of approximately 2.3 m can occur in open, unheated areas without snow cover. At the culvert inlet and outlet, and beneath the proposed culvert, mostly the native soils consist of silty sand and sand. This material has low to medium frost susceptibility based upon the MTO Frost Classification guideline of percent particles between 5 to 75 µm. Therefore, non-frost susceptible materials such as sand and gravel might be considered to be provided to the limit of frost penetration beneath the inlet and outlet of the culvert.

2.8 Culvert Backfill

The selection and placing of the backfill and cover should be in accordance with OPSS 902, OPSS 422 and OPSD 3101.150 for concrete box culverts. The backfill should consist of free-draining, non-frost susceptible granular materials confirming to OPSS.PROV 1010.

For fills immediately below any roadway, it is recommended that Granular A or B materials be used. Where necessary, proper tapering as per standards should be provided. Below a depth of about 2.3 m from any finished road grade, approved compactable fill, such as select subgrade materials (OPSS.PROV 1010) or imported fill can be used.

All granular backfill materials should be placed in thin lifts (i.e. not exceeding 300 mm before compaction) and each lift should be compacted accordance with OPSS. PROV 501. The final lift of embankment fills prior to placing pavement sub-base should be compacted to 100 % SPMDD. The Granular A base and Granular B sub-base courses (for pavement) should be compacted to 100% of the material's SPMDD.

The use of heavy compaction equipment should be avoided immediately adjacent and above the culvert, as per MTO practice. The minimum height of fill cover above the crown of the culvert before power operated tractors or rolling equipment shall be 900 mm, unless otherwise noted by the structural engineer. During backfill placement, the height of the backfill should be maintained at approximately same level on both sides of the structure, to avoid lateral displacement of the structure.

Where less than frost depth (2.3 m) of earth cover is provided above the top of the culvert, a frost taper should be included as per OPSD 803.030, 803.031, MTOD 803.021, whichever is applicable. Since, earth cover above the top of the culvert is less than 2.3 m so frost taper is required at this site with accordance to OPSD 3101.150.

Backfilling behind any retaining (wing) walls should consist of granular materials in accordance with the MTO standards. Free draining backfill materials and perforated drains (as per Figure C6.20a of the CHBDC), suitably outleted etc. should be provided in order to prevent hydrostatic pressure build-up.

2.9 Embankment Design

2.9.1 Stability Analysis

A preliminary slope stability analysis was performed to assess the global stability of the final embankment configuration and to check that a minimum Factor of Safety of 1.3 will be achieved. The static 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.

The cross-section and the approximate slopes were developed based on the drawing provided by MTO. The stratigraphy and groundwater condition at the site were developed based on the results of the geotechnical investigation presented in Part I - Foundation Investigation Report.

Based on the borehole information, the subsoils encountered at the work area consist of embankment fill, underlain by silty sand deposits and bedrock. Therefore, an effective stress analysis for a long term assessment of the embankment slope was performed taking into consideration the subsoil conditions encountered beneath the existing embankment. The analyses assume that all organic soils encountered in boreholes will be removed prior to construction.

The SLOPE/W graphical printout, for analysis performed is included in Appendix G. The result of the slope analysis for west and east side slopes are presented.

Tabulated below in Table 2.9 are the soil parameters used for the slope stability analysis. The soil parameters were generally estimated based on the results of field and laboratory investigation.

Table 2.9 Soil properties used in slope stability analyses

Soil Type	Effective Stress Parameters		
	ϕ' (degrees)	c' (kPa)	γ' (kN/m^3)
Granular Fill	22	-	35
Gravelly Sand with Cobbles Fill	21	-	33
Silty Sand (Loose to Compact)	19	-	29
Silty Sand with Gravel (Loose to Compact)	19	-	30
Organic Silt with Sand (Very Loose)	16	-	24
Silty Sand with Frequent Boulders (Loose to Dense)	19	-	31
Cobbles and Boulders	20	-	36

The results of slope stability analyses suggest that a minimum factor of safety above 1.3 could be obtained if the embankment with 1.7H:1V slopes is constructed after culvert replacement with the open cut method and any organic soils within its footprint is eliminated prior the embankment construction.

2.9.2 Embankment Settlement

It is not planned to change the existing embankment grade at the culvert location. Therefore, there should be negligible additional settlements under the existing embankment because the soil under the existing embankment is non-cohesive. However, a settlement of about 25 mm should be allowed for due to rebound during the construction.

2.9.3 Embankment Slope Stability and Settlement at Temporary Modular Bridge Location

Considering existing old concrete abutment will remain in place, forward slope for the temporary modular bridge founded on properly prepared bedrock or on sand and gravel fill should be safe. In addition, since there is no raise of grade, no settlement is anticipated.

2.10 Inlet and Outlet

2.10.1 Erosion Protection at Outlet

The detailed design for erosion protection should be carried out by a hydraulic engineer. However, in general, rip-rap protection should be provided where the culvert discharges into the open creek. The rip-rap should extend approximately 5 m beyond the ends of the culvert and line the embankment slope to the spring line of the culvert. The size of the rip-rap is a function of the creek's hydrology. As a rule of thumb the thickness of the rip-rap should be a minimum of twice the median particle size, and 300 mm thick as a minimum. The rip-rap configuration at the creek bed should generally follow the OPSD 810.010, which is included in Appendix H of this report.

Where the embankment side slopes have been scarred and/or excavated (beyond rip-rap limit) to facilitate the existing culvert replacement, the scarred and/or reinstated embankment side slopes are to be vegetated with sodding, seeding or planting as necessary depending on the flow rate and volume. Should seeding be utilized, a 100 mm thick layer of topsoil should be placed along with a degradable erosion blanket to help minimize erosion until the vegetation begins to grow.

2.10.2 Stream Bed Rip-Rap

The stream bed rip-rap thickness is to be at least twice the median particle size, and/or 300 mm thick as a minimum as outlined by OPSD 810.010 included in Appendix H of this report.

2.10.3 Seepage Cut-off Requirements

The seepage cut-off requirements should be reviewed in the following context. The native silty soils at the inlet side, outlet side and below the culvert bedding has a high potential for migration with high seepage gradients. For the culvert replacement and new culvert installation, it is prudent to examine possible methods to avoid piping of material resulting from seepage along the culvert. For culverts the following are typical methods: (i) clay seal, (ii) steel or wooden sheet pile cutoff at the upstream end of culvert, (iii) cut-off wall incorporated in the apron slab (if one is used) of the culvert, (iv) cut-off trench constructed with geotextile, and (v) rockfill at the upstream end of the culvert barrel to terminate below the granular bedding of the culvert. Only the clay seal and cut-off trench will be addressed since the sheet pile cut-off will require the understanding of the hydraulics of the stream.

2.10.3.1 Clay Seal

Where readily available a clay seal should be placed at the inlet of the proposed culvert, to prevent the migration of material along the face of the culvert, the formation of flow paths, and any potential internal erosion within the highway embankment. OPSS. PROV 1205 specifies that material used for clay seals shall be natural clay, clay mixture (1 part Bentonite powder and 3.5 parts Granular "A") or a geosynthetic clay liner (GCL). The coefficient of permeability shall not exceed 1 x 10^{-6} cm/s.

The following outlines the installation procedures and minimum material requirement of the clay seal:

- The clay seal should be placed along the sides and top of the culvert a minimum of 1.0 m along the side of the culvert and extending out laterally 1.0 m from the culvert.
- The clay seal should be placed from the top of the culvert footings and extend along the side and the top of the culvert. The clay must not be placed below the culvert.
- The clay should have a Liquid Limit greater than 40% and a Plasticity Index greater than 0.73 x (Liquid Limit – 20%).
- The clay seal is to be place in maximum 150 mm thick lifts and compacted to 95% SPMDD within 2% of the optimum moisture content.

If the GCL is used as a clay seal its material specifications containing the physical, mechanical and hydraulic properties shall be obtained from the manufacture. It is estimated that an approximately 12 mm thick GCL should be installed a minimum 1.0 m along the side of the culvert.

2.10.3.2 Cut-Off Trench

A cut-off trench can be used at both the upstream and downstream ends of the culvert and can be incorporated when the rip-rap apron at both ends of the culvert are being installed. In general, a trench is dug across the stream alignment to well beyond the walls of the culvert and a geomembrane liner is laid on the side of the trench keyed into the culvert at the top and on the base of the trench. The trench is then backfilled with graded rip-rap.

2.11 Obstructions

The cobbles and boulders fill layer was noted to be underneath sand and gravel fill layer at the land boreholes as well as in the underlying native soil deposits. These potential obstructions may impact excavations and/or element of temporary protection systems including cofferdams. A non-standard special provision is provided in Appendix I which may form the basis for advise to the Contractor on this issue.

2 CLOSURE

A subsurface investigation is a limited sampling of a site; the subsurface conditions have been established only at the test hole locations. Should conditions at the site be encountered which differ from those reported at the test locations, we require that we be notified immediately in order to assess this additional information.

Contractors bidding on or undertaking any proposed work at this site should, relative to the subsurface conditions, decide on their own investigations, if deemed necessary, as well as their own interpretations of the factual results provided herein, so they may draw their own conclusions as to how the subsurface conditions may affect them.

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

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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 –
Site Photographs

Photo 1. Hwy 672, looking north

Photo 2. Hwy 672, looking south

Photo 3. Looking east from the outlet of the culvert

Photo 4. Looking west from the inlet of the culvert

Photo 5. East (outlet) side of the embankment looking north

Photo 6. East (outlet) side of the embankment looking south

(a) (b)

Photo 7. Water levels at outlet of existing culvert, (a) in June 2017 and (b) July 2017

Photo 8. West (inlet) side of the embankment looking north

Photo 9. West (inlet) side of the embankment looking south

Photo 10. Submerged inlet of existing culvert during Phase I investigation on June 2017

Photo 11. Temporary twin culvert, inlet side looking east

Photo 12. Water seepage through east(outlet) side of embankment north of existing culvert

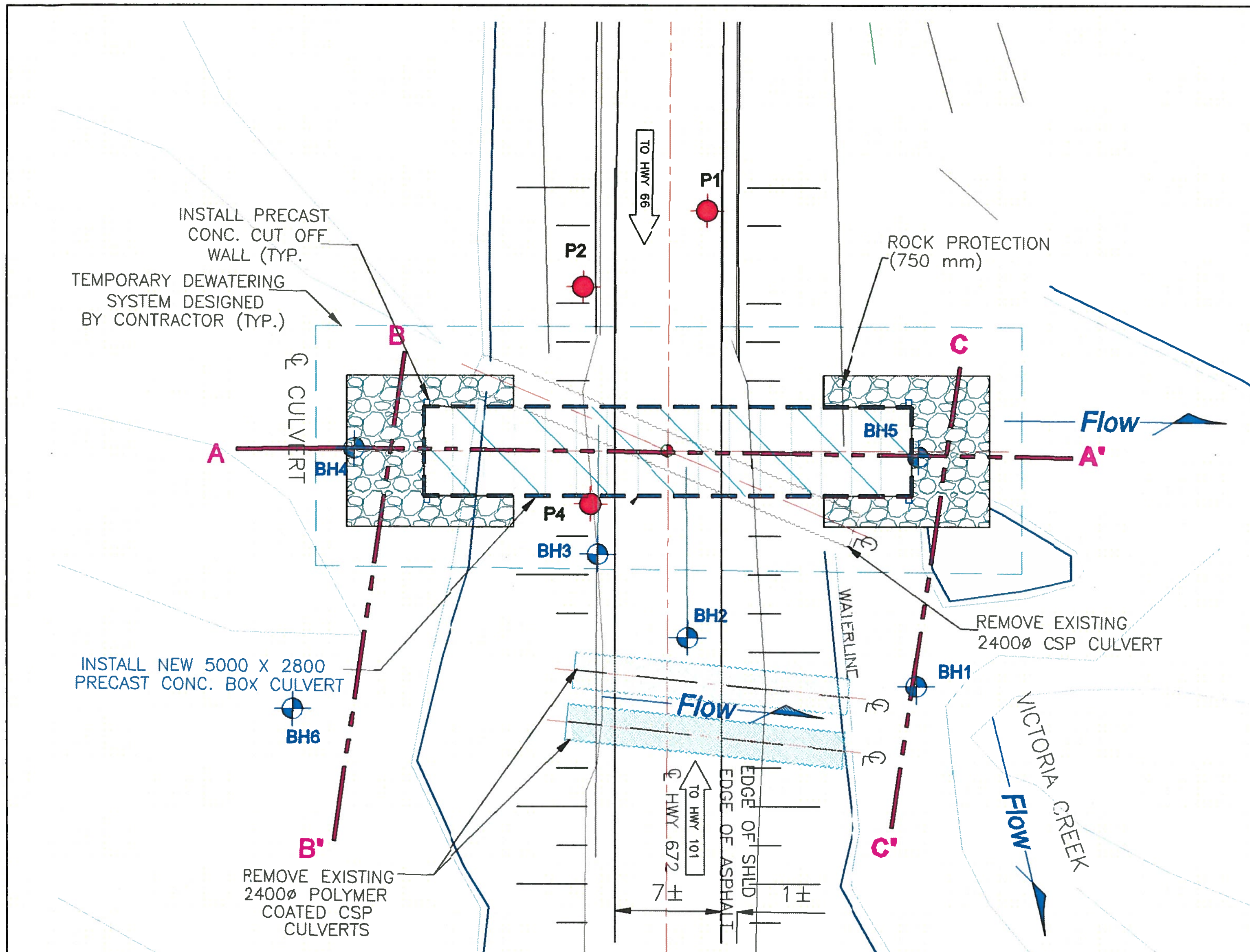
Photo 13. Temporary modular bridge location looking south, note existing abutment

Photo 14. Crain lifting of barge and rig and placing them in water

Photo 15. Borehole BH-1 drilling using a barge

Photo 16. Benchmark HCP (101)

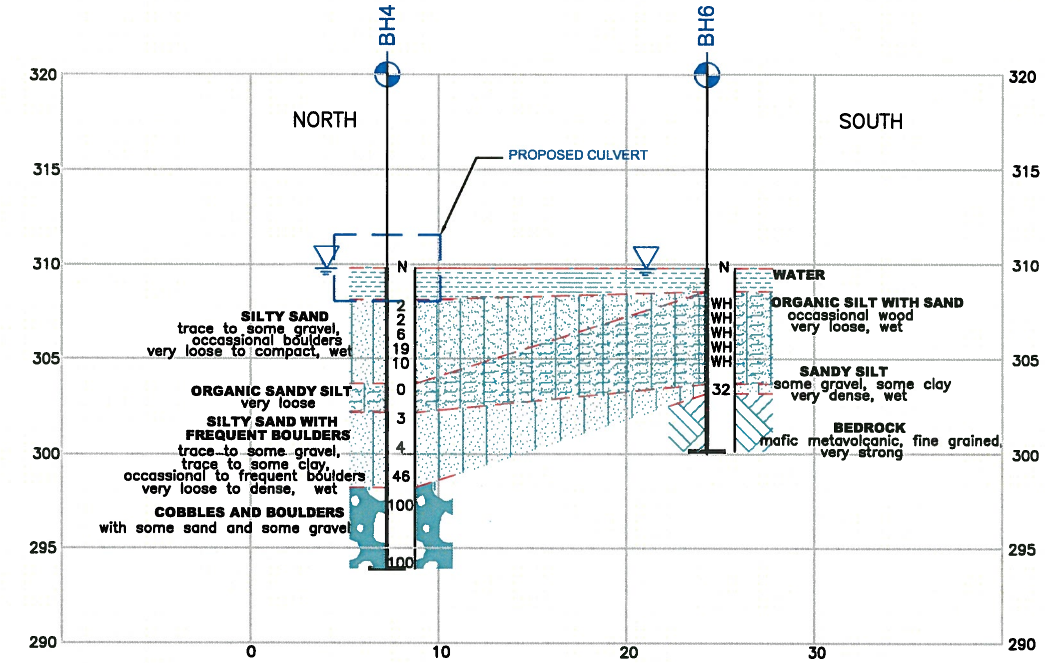
Appendix B – Drawings



PLAN

METRIC

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



SECTION B-B'

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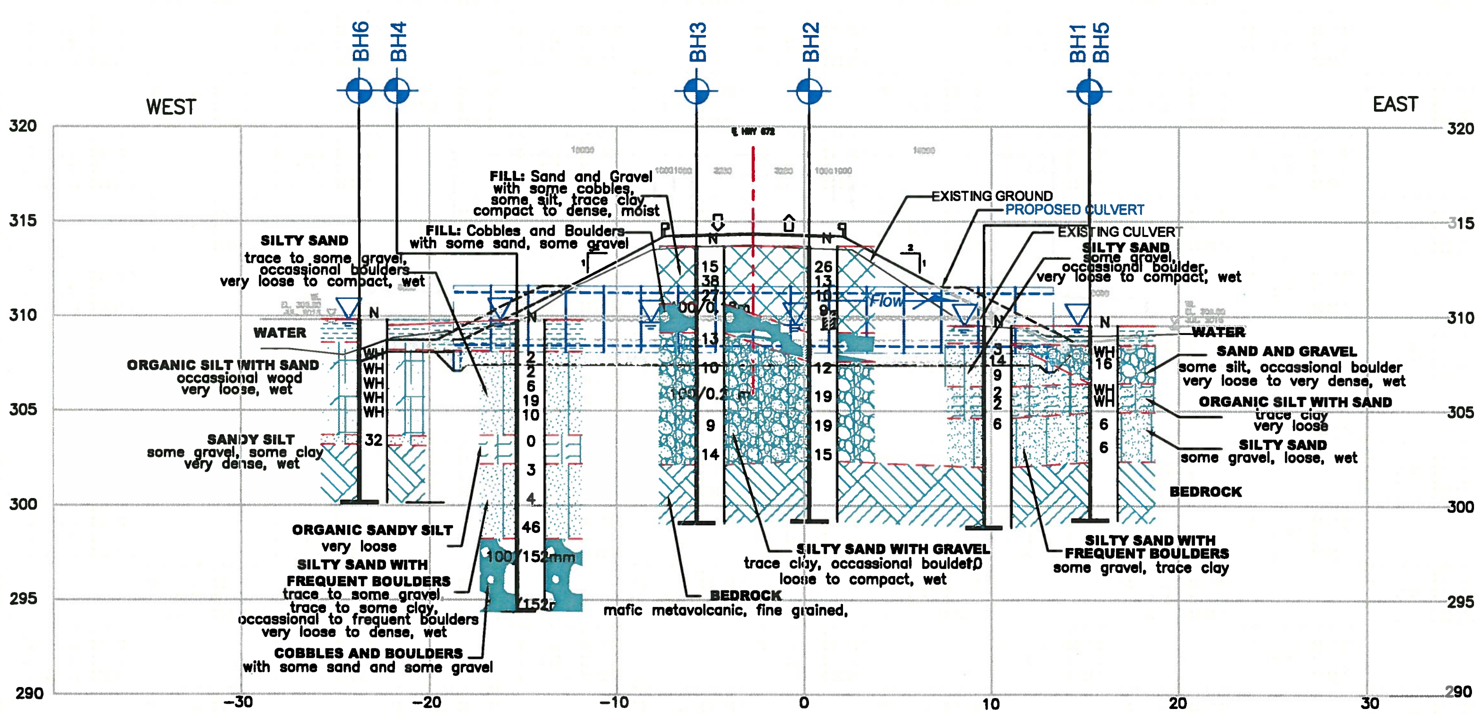
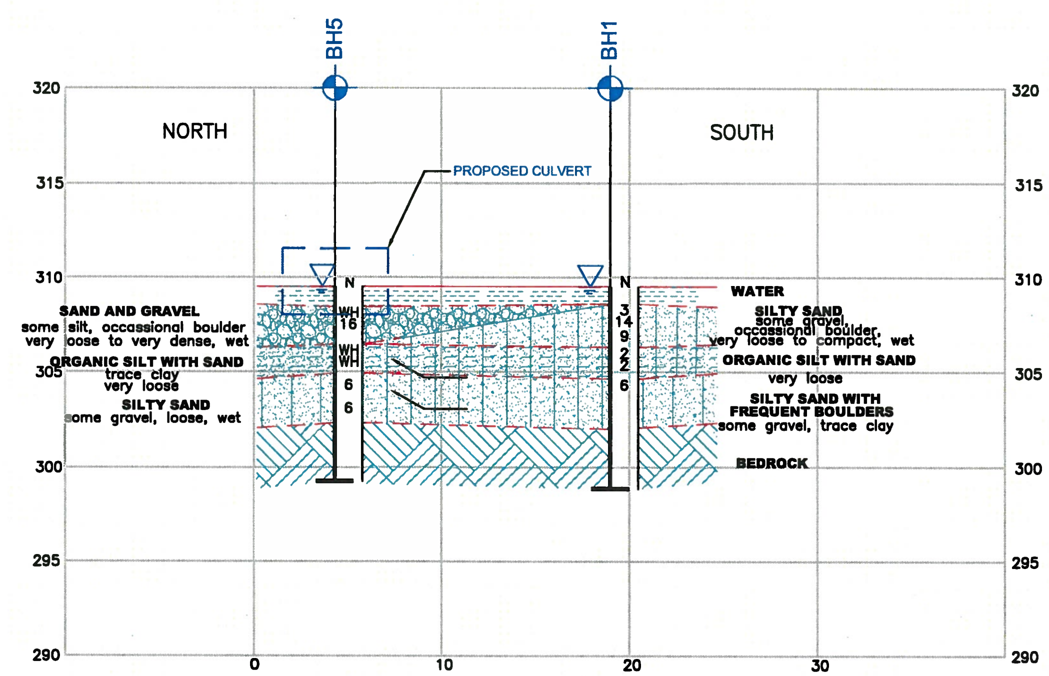


SECTION A-A'



SECTION C-C'

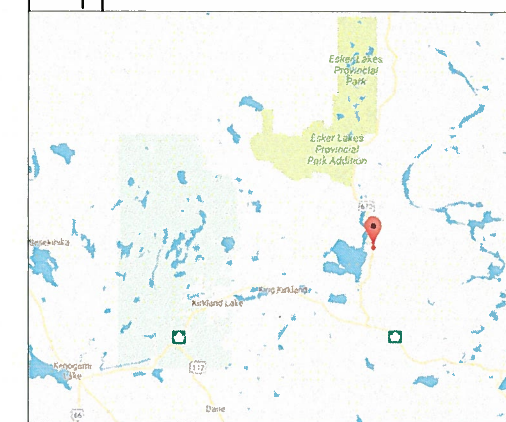
Agreement No. 5015-E-0007
Assignment No. 7
GWP 5027-17-00

VICTORIA CREEK CULVERT REPLACEMENT
HWY 672
BOREHOLE LOCATION PLAN AND SOIL STRATA

SHEET

exp.

exp Services Inc.



LEGEND

- Boreholes Done in 2017 by exp
- Probeholes Done in May 2017 by Golder
- N Standard Penetration Test (Blows/0.3 m)
- Water Level Upon Completion of Drilling
- BM (HCP101)
- BM(HCP175)

SOIL STRATA SYMBOLS

- WATER
- ORGANIC SILT WITH SAND/ ORGANIC SANDY SILT
- FILL
- SAND AND GRAVEL
- COBBLES AND BOULDERS
- SAND
- SILTY SAND/ SANDY SILT
- SILTY SAND WITH GRAVEL
- BEDROCK

BH No.	APPROX. ELEV.	MTM CO-ORDINATES	
		NORTH	EAST
BH1	309.5	5339886.1	389572.4
BH2	313.7	5339889.1	389557.4
BH3	313.7	5339894.5	389551.4
BH4	309.8	5339901.1	389535.4
BH5	309.5	5339916.1	389565.4
BH6	309.8	5339884.1	389533.4
BH7	311.6	5339985.2	389459.4
BH8	311.5	5339976.2	389448.4
P1	313.7	5339916.7	389558.4
P2	313.7	5339911.7	389550.3
P4	313.6	5339897.7	389550.9

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.

SCALE 0 5 15 m

DATE	BY	DESCRIPTION
12/01/2017	SM	SUBMISSION FOR MTO REVIEW

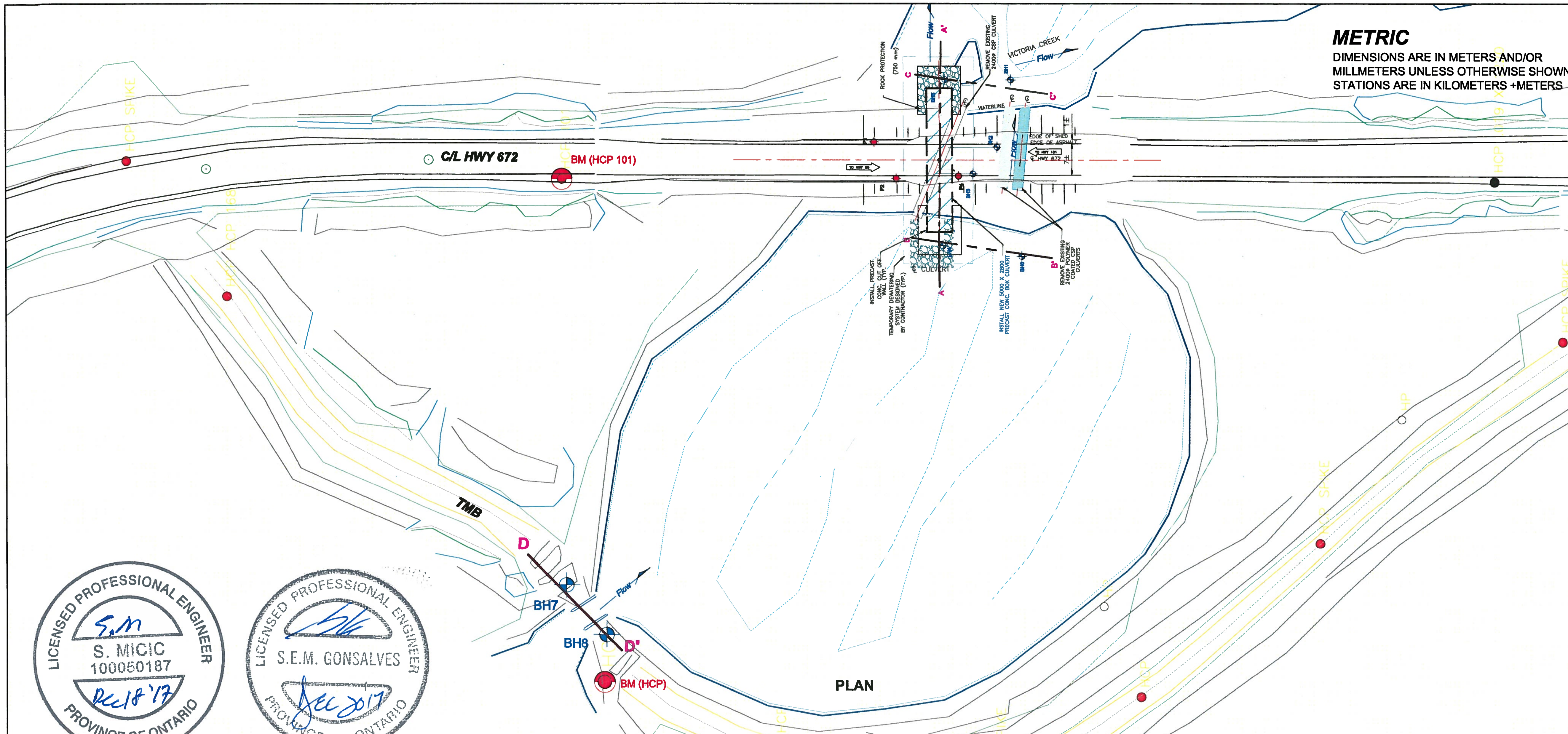
GEOCRES NO. 32D-22

PROJECT NO. ADM-00233185-H0

SUBM'D SM	CHECKED SM	DATE	Dec. 20, 17
DRAWN SH	CHECKED SG	APPROVED SG	DWG. 1

METRIC

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



Agreement No. 5015-E-0007
Assignment No. 7
GWP 5027-17-00

VICTORIA CREEK CULVERT REPLACEMENT
HWY 672
BOREHOLE LOCATION PLAN AND SOIL STRATA

SHEET

exp Services Inc.

KEY PLAN

LEGEND

- Boreholes Done in 2017 by exp
- Probeholes Done in May 2017 by Golder
- N — Standard Penetration Test (Blows/0.3 m)
- Water Level Upon Completion of Drilling
- BM (HCP101)
- BM(HCP175)

SOIL STRATA SYMBOLS

WATER; ORGANIC SILT WITH SAND/ ORGANIC SANDY SILT; FILL; SAND AND GRAVEL; COBBLES AND BOULDERS; SAND; SILTY SAND/ SANDY SILT; SILTY SAND WITH GRAVEL; BEDROCK

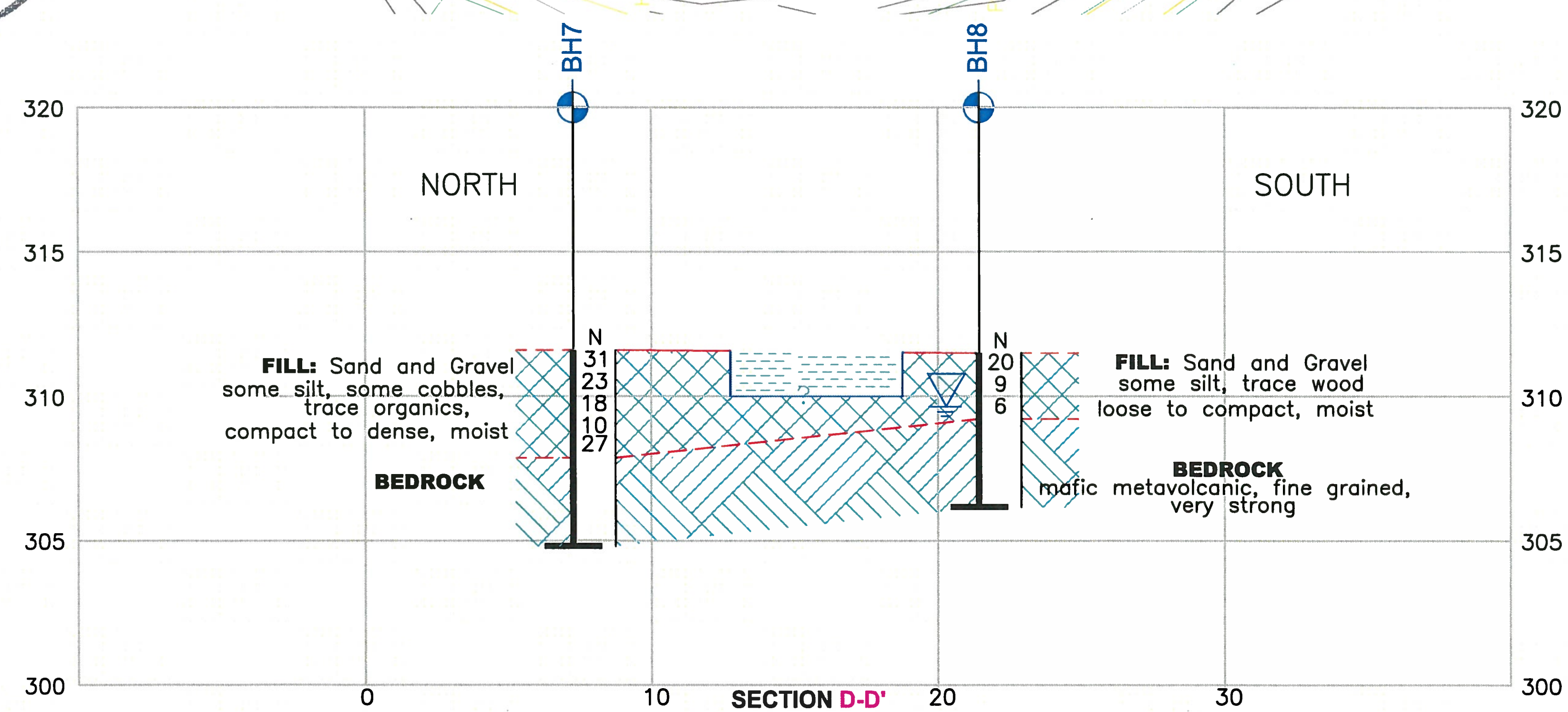
BH No.	APPROX. ELEV.	MTM CO-ORDINATES NORTH	MTM CO-ORDINATES EAST
BH1	309.5	5339886.1	389572.4
BH2	313.7	5339889.1	389557.4
BH3	313.7	5339894.5	389551.4
BH4	309.8	5339901.1	389535.4
BH5	309.5	5339916.1	389565.4
BH6	309.8	5339884.1	389533.4
BH7	311.6	5339985.2	389459.4
BH8	311.5	5339976.2	389448.4
P1	313.7	5339916.7	389558.4
P2	313.7	5339911.7	389550.3
P4	313.6	5339897.7	389550.9

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.

plan 0 10 30 m
section 0 7.5 m



SECTION D-D'

DATE	BY	DESCRIPTION
12/01/2017	SM	SUBMISSION FOR MTO REVIEW

GEOCRES NO. 32D-22
PROJECT NO. ADM-00233185-H0

SUBM'D SM	CHECKED SM	DATE	Dec. 20, 17
DRAWN SH	CHECKED SG	APPROVED SG	DWG. 2

Appendix C – Borehole Logs

Explanation of Terms Used on Borehole Records

SOIL DESCRIPTION

Terminology describing common soil genesis:

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

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

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

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

Terminology describing soil structure:

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

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

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

Fissured: material breaks along plane of fracture.

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

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

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

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

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

Homogeneous: same color and appearance throughout.

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

Uniformly Graded: predominantly on grain size.

All soil sample descriptions included in this report follow 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 SILT (NONPLASTIC)	FINE	MEDIUM	CRS.	FINE	COARSE
	SAND			GRAVEL	

UNIFIED SOIL CLASSIFICATION

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

Table a: Percent or Proportion of Soil, Pp

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

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

Table b: Apparent Density of Cohesionless Soil

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

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

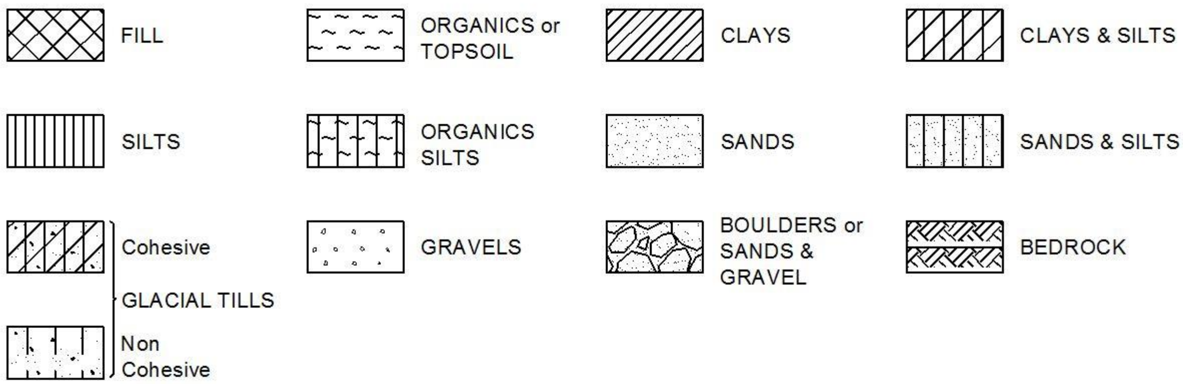
Table c: Consistency of Cohesive Soil

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

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

STRATA PLOT

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



WATER LEVEL MEASUREMENT

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

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

STRESS AND STRAIN

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

MECHANICALL PROPERIES OF SOIL

m_v	kPa^{-1}	Coefficient of volume change
c_c	1	Compression index
c_s	1	Swelling index
c_r	1	Recompression index
c_v	m^2/s	Coefficient of consolidation
H	m	Drainage path
T_V	1	Time factor
U	%	Degree of consolidation
σ'_{v0}	kPa	Effective overburden pressure
σ'_P	kPa	Preconsolidation pressure
τ_f	kPa	Shear strength
c'	kPa	Effective cohesion intercept
ϕ'	$-°$	Effective angle of internal friction
c_u	kPa	Apparent cohesion intercept
ϕ_u	$-°$	Apparent angle of internal friction
τ_R	kPa	Residual shear strength
τ_r	kPa	Remoulded shear strength
S_t	1	Sensitivity = c_u/τ_r

PHYSICAL PROPERTIES OF SOIL

P_s	kg/m^3	Density of solid particles
γ_s	kN/m^3	Unit weight of solid particles
ρ_w	kg/m^3	Density of water
γ_w	kN/m^3	Unit weight of water
ρ	kg/m^3	Density of soil
γ	kN/m^3	Unit weight of soil
ρ_d	kg/m^3	Density of dry soil
γ_d	kN/m^3	Unit weight of dry soil
ρ_{sat}	kg/m^3	Density of saturated soil
γ_{sat}	kN/m^3	Unit weight of saturated soil
ρ'	kg/m^3	Density of submerged soil
γ'	kN/m^3	Unit weight of submerged soil
e	1, %	Void ratio
n	1, %	Porosity
w	1, %	Water content
S_r	%	Degree of saturation
W_L	%	Liquid limit
W_P	%	Plastic limit
W_s	%	Shrinkage limit
I_P	%	Plasticity index = $(W_L - W_P)$
I_L	%	Liquidity index = $(W - W_P)/I_P$
I_C	%	Consistency index = $(W_L - W)/I_P$
e_{max}	1, %	Void ratio in loosest state
e_{min}	1, %	Void ratio in densest state
I_D	1	Density index = $(e_{max} - e)/(e_{max} - e_{min})$
D	mm	Grain diameter
D_n	mm	N percent - diameter
C_u	1	Uniformity coefficient
h	m	Hydraulic head or potential
q	m^3/s	Rate of discharge
v	m/s	Discharge velocity
i	1	Hydraulic gradient
k	m/s	Hydraulic conductivity
j	kN/m^3	Seepage force

RECORD OF BOREHOLE No BH-1

METRIC

W.P. 5027-17-00	LOCATION Hwy 672, Dobie, MTM ON12 N5339886.1, E389572.4		ORIGINATED BY ST
DIST Tamiskaming HWY 672	TEST PIT TYPE Continuous Flight Hollow Stem Augers		COMPILED BY NT
DATUM Geodetic	DATE 2017.07.13 - 2017.07.13	LATITUDE 48.192133 LONGITUDE -79.859664	CHECKED BY SM

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
309.5	Water Surface				
0.0	**WATER**				
308.6 / 0.9	**SILTY SAND:** some gravel, occassional boulder, brown, wet, very loose to compact	1	SS	3	
		2	SS	14	
		3	SS	9	
306.3 / 3.2	**ORGANIC SILT WITH SAND** dark brown to grey, very loose	4	SS	2	
		5	SS	2	0 6 89 5
304.6 / 4.9	**SILTY SAND WITH FREQUENT BOULDERS:** some gravel, trace clay, grey, wet, loose	6	SS	6	3 52 40 5
	- becoming frequent boulders below 6.2 m				
302.0 / 7.5	**BEDROCK:** mafic metavolcanic, fine grained, very strong, dark grey NQ Coring	7	NQ		
		8	NQ		
298.8 / 10.7	**End of borehole at 10.67 m depth.**				

	Length (m)	RQD(%)
Run1	1.52	81.7%
Run2	1.67	96.9%

Notes:
1. This drawing is to be read with the subject report and project numbers as presented above.
2. Borehole drilled through water using barge

Column headings: STRAT PLOT; GROUND WATER CONDITIONS; ELEVATION SCALE (309, 308, 307, 306, 305, 304, 303, 302, 301, 300, 299); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m^3

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

RECORD OF BOREHOLE No BH-2

METRIC

W.P. 5027-17-00 | LOCATION Hwy 672, Dobie, MTM ON12 N5339889.1, E389557.4 | ORIGINATED BY ST

DIST Tamiskaming HWY 672 | TEST PIT TYPE CME-75/NW Casing/HQ | COMPILED BY NT

DATUM Geodetic | DATE 2017.06.13 - 2017.06.14 | LATITUDE 48.19216 | LONGITUDE -79.85986 | CHECKED BY SM

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
313.7	Ground Surface				
0.0	**FILL: SAND AND GRAVEL** with some cobbles, some silt, trace clay brown, moist, compact	1	AS		
		2	SS	26	40 55 (5)
	-suspect cobbles, no recovery	3	SS	13	No sample recovery
		4	SS	10	
		5	SS	9	26 49 24 1
309.1 4.6	**FILL: COBBLES AND BOULDERS** with some sand, some gravel	6	SS	100/ 0.05 m	
307.6 6.1	**SILTY SAND WITH GRAVEL:** grey, wet, compact	7	SS	12	1 91 8 0
		8	SS	19	Very less sample recovery
		9	SS	19	Very less sample recovery
		10	SS	15	No sample recovery
302.3 11.4		11	HQ		

ELEVATION SCALE: 313, 312, 311, 310, 309, 308, 307, 306, 305, 304, 303, 302

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100

PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L

WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m³

Continued Next Page

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

RECORD OF BOREHOLE No BH-2

METRIC

W.P.	5027-17-00	LOCATION	Hwy 672, Dobie, MTM ON12 N5339889.1, E389557.4	ORIGINATED BY	ST
DIST	Tamiskaming HWY 672	TEST PIT TYPE	CME-75/NW Casing/HQ	COMPILED BY	NT
DATUM	Geodetic	DATE	2017.06.13 - 2017.06.14 LATITUDE 48.19216 LONGITUDE -79.85986	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 kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE)	PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	BEDROCK: mafic metavolcanic, fine grained, very strong, dark grey HQ Coring Length (m) RQD(%) Run1 0.91 94.4% Run2 1.52 85% Run3 0.71 92.9% *(continued)*		12	HQ			301				
			13	HQ			300				
299.2 14.5	**End of borehole at 14.53 m depth. Groundwater level measured at 4.1 m** Notes: 1. This drawing is to be read with the subject report and project numbers as presented above. 2. Groundwater level was measured in open hole upon completion of drilling.										

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

RECORD OF BOREHOLE No BH-3

METRIC

W.P. 5027-17-00	LOCATION Hwy 672, Dobie, MTM ON12 N5339894.5, E389551.4		ORIGINATED BY ST
DIST Tamiskaming HWY 672	TEST PIT TYPE CME-75/NW Casing/HQ		COMPILED BY NT
DATUM Geodetic	DATE 2017.06.14 - 2017.06.15	LATITUDE 48.19219 LONGITUDE -79.859945	CHECKED BY SM

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
313.7	Ground Surface					
0.0	**FILL: SAND AND GRAVEL** with some cobbles, some silt, trace clay brown, moist, compact to dense	1	AS			
		2	SS	15		
		3	SS	38		54 42 (4)
		4	SS	27		
310.7 / 3.1	**FILL: COBBLES AND BOULDERS** with some sand, some gravel	5	SS	100/ 0.18m		
					∇ (approx. El. 309.8)	
309.1 / 4.6	**SILTY SAND WITH GRAVEL:** trace clay, occassional boulder,grey, wet, loose to compact	6	SS	13		23 50 27 0
		7	SS	10		
		8	SS	100/ 0.2 m		
	- Spoon refusal, encountered boulder between 8 m to 8.4 m					
		9	SS	9		
		10	SS	14		19 42 36 3
302.1 / 11.6		11	HQ			

ELEVATION SCALE: 313, 312, 311, 310, 309, 308, 307, 306, 305, 304, 303, 302

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; 20 40 60 80 100

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m³

Continued Next Page

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

Exp Services Inc.
Brampton, Ontario

Foundation Design

RECORD OF BOREHOLE No BH-3

METRIC

W.P. 5027-17-00 LOCATION Hwy 672, Dobie, MTM ON12 N5339894.5, E389551.4 ORIGINATED BY ST

DIST Tamiskaming HWY 672 TEST PIT TYPE CME-75/NW Casing/HQ COMPILED BY NT

DATUM Geodetic DATE 2017.06.14 - 2017.06.15 LATITUDE 48.19219 LONGITUDE -79.859945 CHECKED BY SM

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	SAMPLES TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE) 20 40 60 80 100	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L – WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	BEDROCK: mafic metavolcanic, fine grained, very strong, dark grey HQ Coring Length (m) RQD(%) Run1 0.38 93.3% Run2 1.52 40% Run3 1.17 95.6% *(continued)*		12	HQ			301				
			13	HQ			300				
299.1 14.6	**End of borehole at 14.63 m depth. Groundwater level measured at 4 m** Notes: 1. This drawing is to be read with the subject report and project numbers as presented above. 2. Groundwater level was measured in open hole upon complition of borehole										

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

ONTARIO MTO ASSIGNMENT#7, NER .GPJ ONTARIO MTO.GDT 8/4/17

RECORD OF BOREHOLE No BH-4

METRIC

W.P. 5027-17-00	LOCATION Hwy 672, Dobie, MTM ON12 N5339901.1, E389535.4		ORIGINATED BY ST
DIST Tamiskaming HWY 672	TEST PIT TYPE Continuous Flight Hollow Stem Augers		COMPILED BY NT
DATUM Geodetic	DATE 2017.07.05 - 2017.07.05	LATITUDE 48.192273 LONGITUDE -79.860158	CHECKED BY SM

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	NATURAL MOISTURE CONTENT w (%)	GR	SA	SI	CL
309.8	Water Surface								
0.0	**WATER**								
308.1 / 1.7	**SILTY SAND :** trace to some gravel, occassional boulders grey, wet, very loose to compact	1	SS	2					
		2	SS	2		0	73	27	0
		3	SS	6					
		4	SS	19					
		5	SS	10					
303.7 / 6.1	**ORGANIC SANDY SILT** dark brown , very loose	6	SS	0					
302.2 / 7.6	**SILTY SAND WITH FREQUENT BOULDERS:** trace to some gravel, trace to some clay, occassional to frequent boulders, grey, wet, very loose to dense -becoming occassional boulders	7	SS	3		0	28	56	16
		8	SS	4					
	-becoming occassional to frequent boulders								
		9	SS	46					
298.2 / 11.6									

Column headings: SOIL PROFILE; SAMPLES; STRAT PLOT; GROUND WATER CONDITIONS; ELEVATION SCALE (309, 308, 307, 306, 305, 304, 303, 302, 301, 300, 299, 298); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m^3; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

Continued Next Page

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

RECORD OF BOREHOLE No BH-4

METRIC

W.P.	5027-17-00	LOCATION	Hwy 672, Dobie, MTM ON12 N5339901.1, E389535.4	ORIGINATED BY	ST
DIST	Tamiskaming HWY 672	TEST PIT TYPE	Continuous Flight Hollow Stem Augers	COMPILED BY	NT
DATUM	Geodetic	DATE	2017.07.05 - 2017.07.05 LATITUDE 48.192273 LONGITUDE -79.860158	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 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 (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	COBBLES AND BOULDERS: with some sand and some gravel NQ Coring Length (m) RQD(%) Run1 1.52 81.7% Run2 1.67 96.9% *(continued)*		10	SS	100/ 152 mm		297 296 295				No sample recovery
294.4			11	SS	100/ 152 mm						No sample recovery
15.4	**End of borehole at 15.4 m depth.** Notes: 1. This drawing is to be read with the subject report and project numbers as presented above. 2. Borehole drilled through water using barge										

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

RECORD OF BOREHOLE No BH-5

METRIC

W.P.	5027-17-00	LOCATION	Hwy 672, Dobie, MTM ON12 N5339916.1, E389565.4	ORIGINATED BY	ST
DIST	Tamiskaming HWY 672	TEST PIT TYPE	Continuous Flight Hollow Stem Augers	COMPILED BY	NT
DATUM	Geodetic	DATE	2017.07.11 - 2017.07.12 LATITUDE 48.192403 LONGITUDE -79.859752	CHECKED BY	SM

ELEV / DEPTH	DESCRIPTION (SOIL PROFILE)	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
309.5	Water Surface				
0.0	**WATER**				
308.4 / 1.1	**SAND AND GRAVEL:** some silt, occassional boulder, brown to grey, wet, very loose to very dense	1	SS	WH	
		2	SS	16	
		3	SS	74	
	-boulder encountered				
306.5 / 3.1	**ORGANIC SILT WITH SAND** trace clay, dark brown, very loose	4	SS	WH	0 5 92 3
		5	SS	WH	
304.9 / 4.6	**SILTY SAND :** some gravel, brown, wet, loose	6	SS	6	
		7	SS	6	10 49 41 0
302.3 / 7.2	**BEDROCK:** mafic metavolcanic, fine grained, very strong, dark grey NQ Coring Length (m) RQD(%) Run1 1.52 70% Run2 1.57 90.3%	8	NQ		
		9	NQ		
299.2 / 10.3	**End of borehole at 10.29 m depth.**				

Notes:
1. This drawing is to be read with the subject report and project numbers as presented above.
2. Borehole drilled through water using barge

Elevation scale: 309, 308, 307, 306, 305, 304, 303, 302, 301, 300

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; 20 40 60 80 100

PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60; 90.8

UNIT WEIGHT γ kN/m^3

GROUND WATER CONDITIONS

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

RECORD OF BOREHOLE No BH-6

METRIC

W.P. 5027-17-00	LOCATION Hwy 672, Dobie, MTM ON12 N5339884.1, E389533.4		ORIGINATED BY ST
DIST Tamiskaming HWY 672	TEST PIT TYPE Continuous Flight Hollow Stem Augers		COMPILED BY NT
DATUM Geodetic	DATE 2017.07.06 - 2017.07.06	LATITUDE 48.19212 LONGITUDE -79.860188	CHECKED BY SM

SOIL PROFILE | SAMPLES | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) | SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE (20 40 60 80 100) | PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L – WATER CONTENT (%) (20 40 60) | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

ELEV / DEPTH	DESCRIPTION
309.8	Water Surface
0.0	**WATER**
308.6 / 1.2	**ORGANIC SILT WITH SAND** occassional wood, dark brown, wet, very loose
303.7 / 6.1	**SANDY SILT :** some gravel, some clay, grey, wet, very dense
303.2 / 6.6	**BEDROCK:** mafic metavolcanic, fine grained, very strong, dark grey NQ Coring
300.2 / 9.7	**End of borehole at 9.65 m depth.**

Bedrock coring (NQ):

	Length (m)	RQD(%)
Run1	1.52	83.3%
Run2	1.52	91.7%

Samples:

NUMBER	TYPE	"N" VALUES	Water content (%)	GR	SA	SI	CL
1	SS	WH	~65				
2	SS	WH	112.8				
3	SS	WH	~60				
4	SS	WH	~55				
5	SS	WH	~60				
6	SS	32	~25	8	31	51	10
7	NQ						
8	NQ						

Elevation scale: 309, 308, 307, 306, 305, 304, 303, 302, 301

Notes:
1. This drawing is to be read with the subject report and project numbers as presented above.
2. Borehole drilled through water using barge

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

RECORD OF BOREHOLE No BH-7

METRIC

W.P. 5027-17-00	LOCATION Hwy 672, Dobie, MTM ON12 N5339985.16, E389459.4		ORIGINATED BY ST
DIST Tamiskaming HWY 672	TEST PIT TYPE CME-75/NW Casing/HQ		COMPILED BY NT
DATUM Geodetic	DATE 2017.06.15 - 2017.06.15	LATITUDE 48.193038 LONGITUDE -79.861164	CHECKED BY SM

ELEV / DEPTH (m)	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
311.6	Ground Surface				
0.0	**FILL: SAND AND GRAVEL** some silt, some cobbles, trace organics, brown, moist, compact to dense	1	SS	31	
		2	SS	23	
		3	SS	18	
		4	SS	10	53 15 32 0
		5	SS	27	
307.9 / 3.7	**BEDROCK:** mafic metavolcanic, fine grained, very strong, dark grey HQ Coring Length (m) RQD(%) Run1 0.61 66.7% Run2 1.22 85.4% Run3 1.22 97.9%	6	HQ		
		7	HQ		
		8	HQ		
304.8 / 6.8	**End of borehole at 6.78 m depth.**				

Plot column headings: STRAT PLOT; GROUND WATER CONDITIONS; ELEVATION SCALE (311, 310, 309, 308, 307, 306, 305); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³.

Notes:
1. This drawing is to be read with the subject report and project numbers as presented above.
2. Groundwater level was not measured in open hole upon complition of borehole

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

RECORD OF BOREHOLE No BH-8

METRIC

W.P. 5027-17-00	LOCATION Hwy 672, Dobie, MTM ON12 N5339976.15, E389448.4		ORIGINATED BY ST
DIST Tamiskaming HWY 672	TEST PIT TYPE CME-75/NW Casing/HQ		COMPILED BY NT
DATUM Geodetic	DATE 2017.06.13 - 2017.06.13	LATITUDE 48.19296 LONGITUDE -79.861314	CHECKED BY SM

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	NATURAL MOISTURE CONTENT w (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
311.5	Ground Surface									
0.0	**FILL: SAND AND GRAVEL** some silt, trace wood, brown, moist, loose to compact		1	SS	20		311	○		
			2	SS	9		310	○		
			3	SS	6	∇		○		43 54 (3)
309.2 / 2.3	**BEDROCK:** mafic metavolcanic, fine grained, very strong, dark grey HQ Coring Length (m) RQD(%) Run1 1.27 94% Run2 0.91 94.4% Run3 0.86 88.2%		6	HQ			309			
			7	HQ			308			
			8	HQ			307			
306.2 / 5.3	**End of borehole at 5.33 m depth. Groundwater level was measured at 1.83 m** Notes: 1. This drawing is to be read with the subject report and project numbers as presented above. 2. Groundwater level was measured in open hole upon complition of borehole									

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

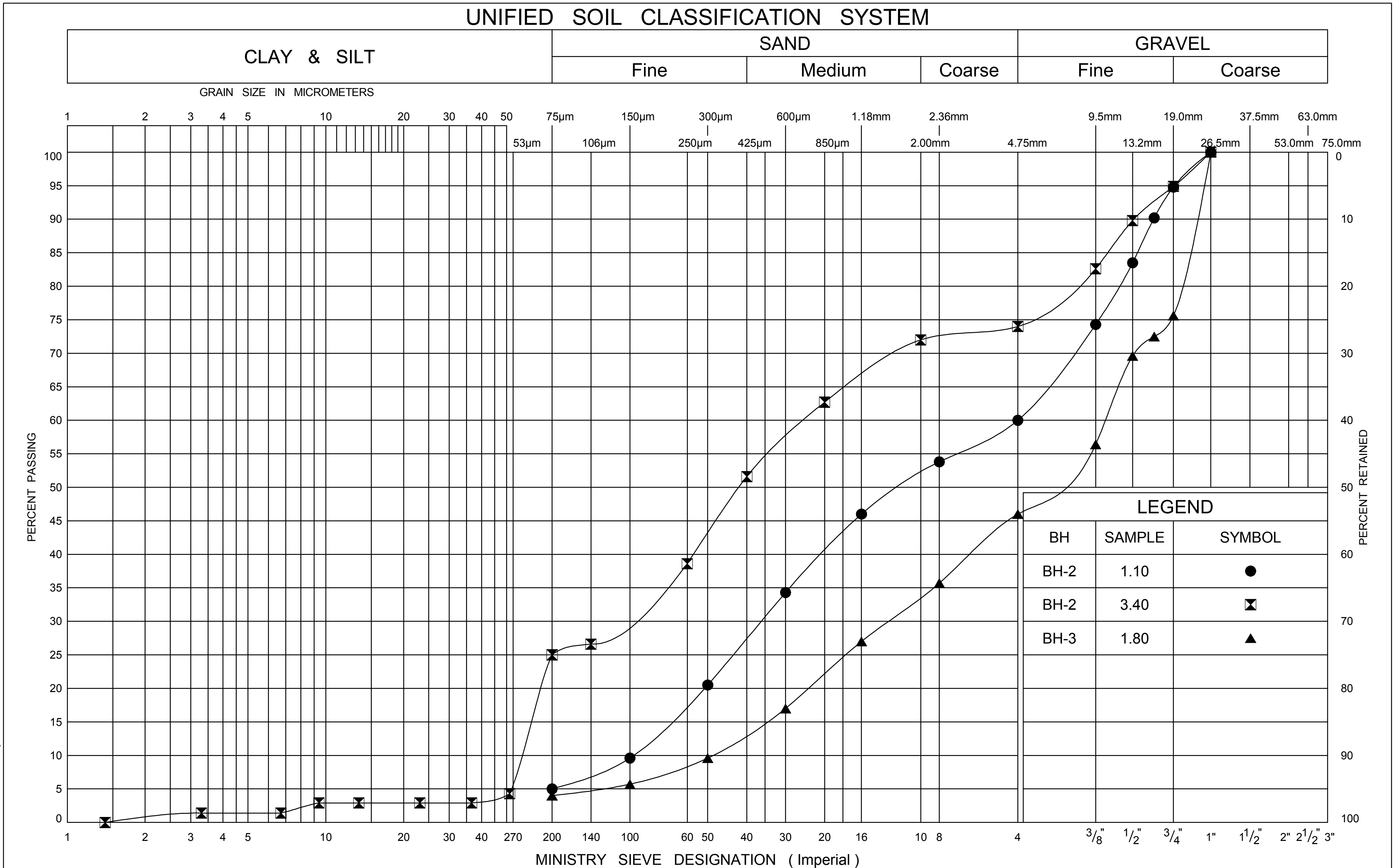
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE — 20 40 60 80 100

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60

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

Appendix D – Laboratory Data

78 12 M



ONTARIO MOT GRAIN SIZE ASSIGNMENT#7, NER .GPJ ONTARIO MOT.GDT 8/4/17

Ministry of Transportation

Ontario

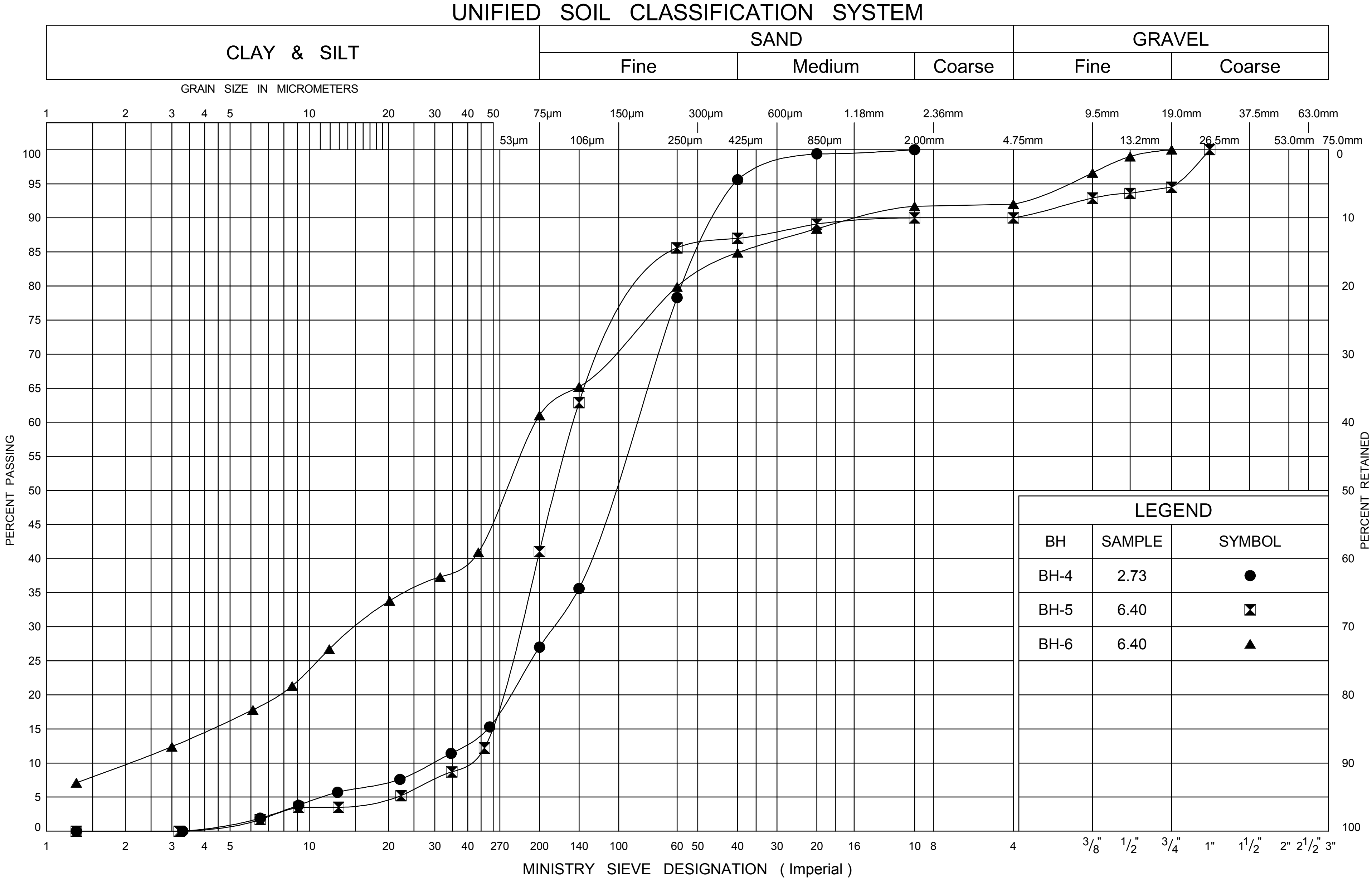
GRAIN SIZE DISTRIBUTION

FIG No 1

W P 5027-17-00

5015-E-0007, Assignment 7

78 12 M



Ministry of Transportation

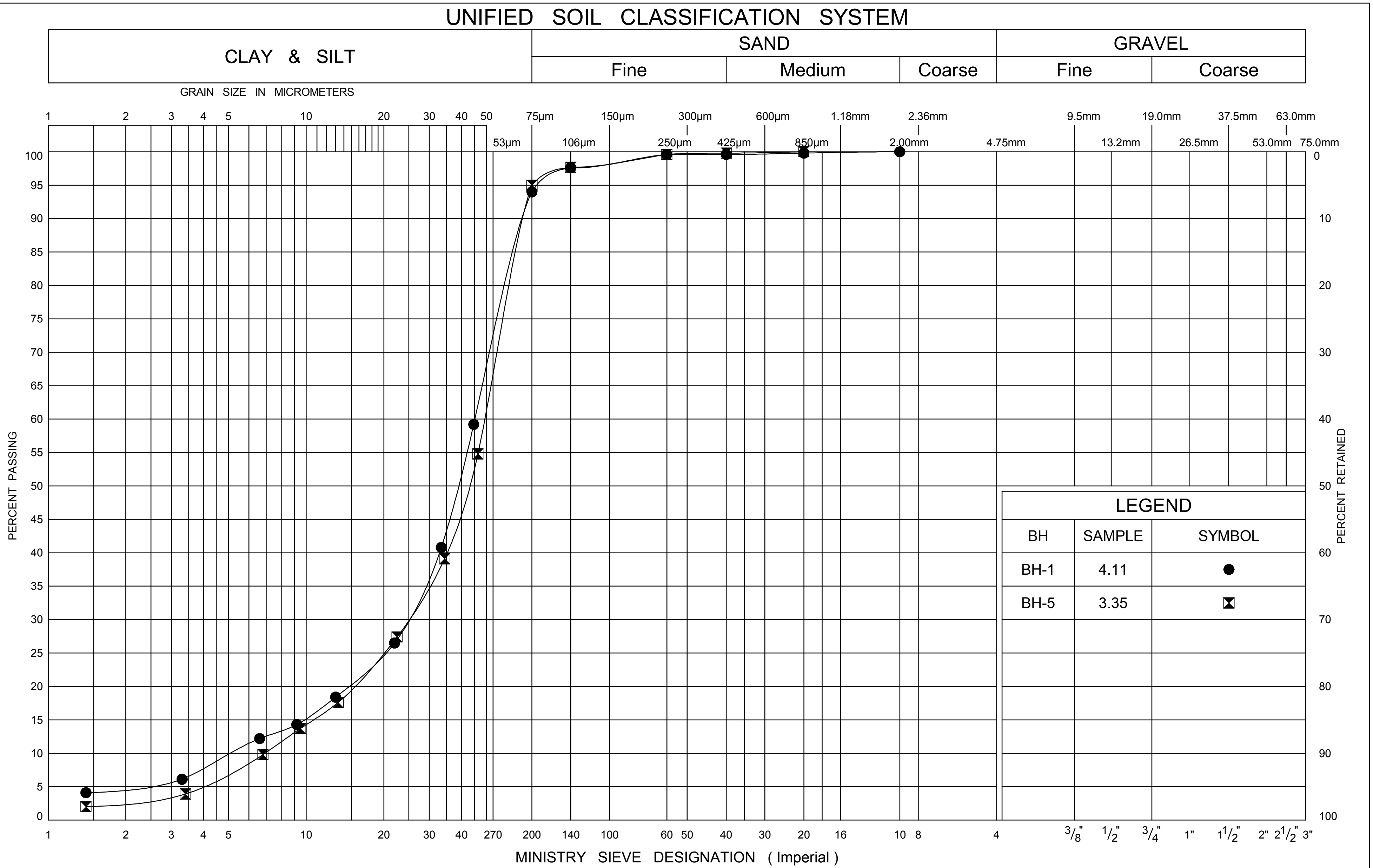
Ontario

GRAIN SIZE DISTRIBUTION

FIG No 2

W P 5027-17-00

5015-E-0007, Assignment 7



BH	SAMPLE	SYMBOL
BH-1	4.11	●
BH-5	3.35	⧗

Ministry of Transportation
Ontario

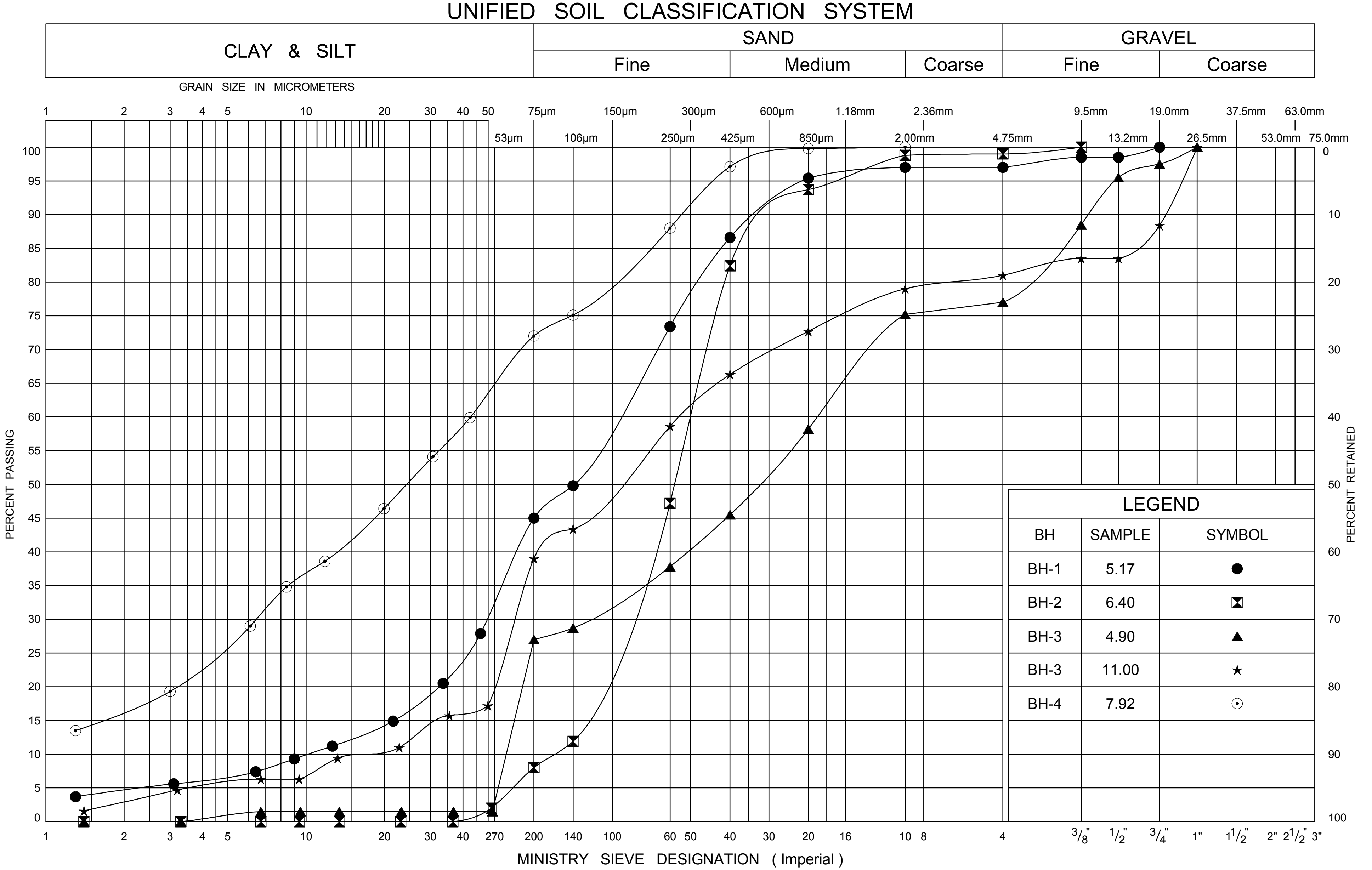
GRAIN SIZE DISTRIBUTION

FIG No 3

W P 5027-17-00

5015-E-0007, Assignment 7

78 12 M



LEGEND

BH	SAMPLE	SYMBOL
BH-1	5.17	●
BH-2	6.40	⧗
BH-3	4.90	▲
BH-3	11.00	★
BH-4	7.92	⊙

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GRAIN SIZE DISTRIBUTION

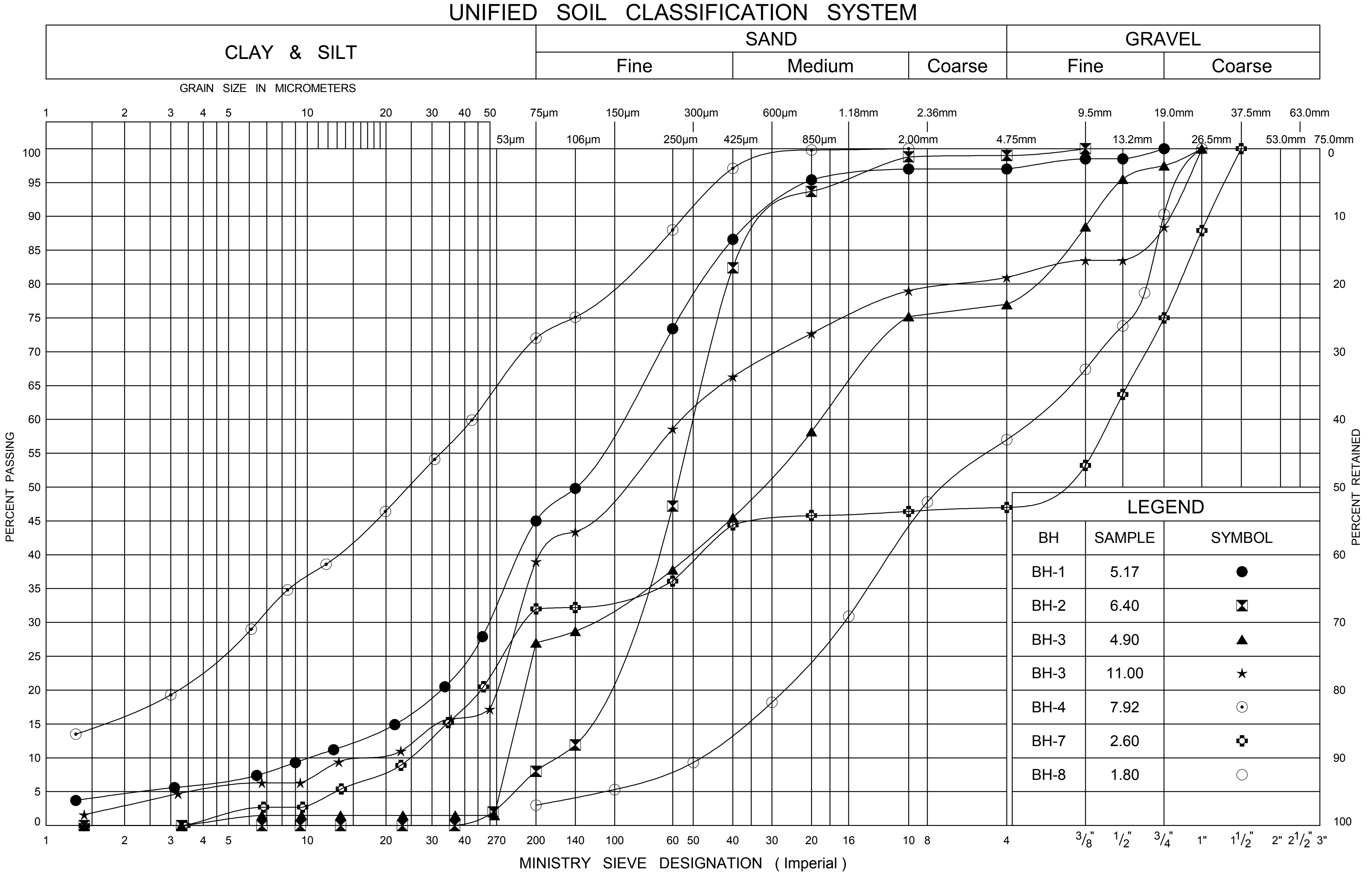
FIG No 4

W P 5027-17-00

5015-E-0007, Assignment 7

ONTARIO MOT GRAIN SIZE ASSIGNMENT#7, NER .GPJ ONTARIO MOT.GDT 8/4/17

78 12 M



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GRAIN SIZE DISTRIBUTION

FIG No 5

W P 5027-17-00

5015-E-0007, Assignment 7

ONTARIO MOT GRAIN SIZE ASSIGNMENT#7, NER .GPJ ONTARIO MOT.GDT 8/4/17

Appendix E –
Rock Core Photographs

Project NO: ADM 0002331875-H0
BH NO: 1
Run NO: 1 & 2
Sample Depth: 7.5 m to 10.7 m
Elevation: 302.0 m to 298.8 m
Date: July 13, 2017

Photo 1. Bedrock Core Sample for BH1 from Elevation 302.0 m to 298.8 m

Project NO: ADM 0002331875-H0
BH NO: 2
Run NO: 1,2 & 3
Sample Depth: 11.4 m to 14.5
Elevation: 302.3 m to 299.2 m
Date: June 14, 2017

Photo 2. Bedrock Core Sample for BH2 from Elevation 302.3 m to 299.2 m

Project NO: ADM 0002331875-H0
BH NO: 3
Run NO: 1,2 & 3
Sample Depth: 11.6 m to 14.6 m
Elevation: 302.1 m to 299.8 m
Date: June 15, 2017

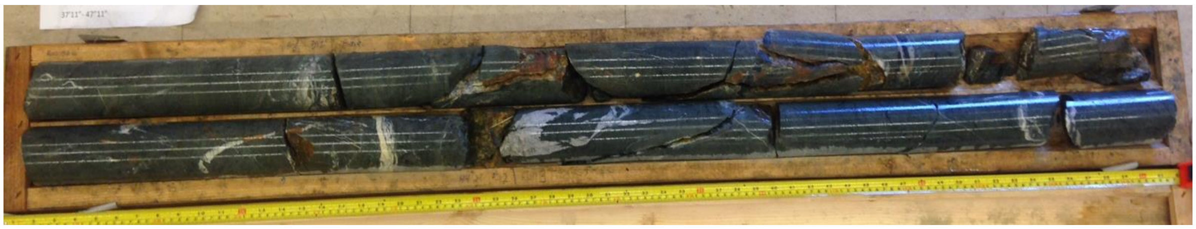


Photo 3. Bedrock Core Sample for BH3 from Elevation 302.1 m to 299.8 m

Project NO: ADM 0002331875-H0
BH NO: 5
Run NO: 1&2
Sample Depth: 7.2 m to 10.3 m
Elevation: 302.3 m to 299.2 m
Date: July 12, 2017

Photo 4. Bedrock Core Sample for BH5 from Elevation 302.3 m to 299.2 m

Project NO: ADM 0002331875-H0
BH NO: 6
Run NO: 1&2
Sample Depth: 6.6 m to 9.7 m
Elevation: 303.2 m to 300.2 m
Date: July 6, 2017

Photo 5. Bedrock Core Sample for BH6 from Elevation 303.2 m to 300.2 m

Project NO: ADM 0002331875-H0
BH NO: 7
Run NO: 1,2 &3
Sample Depth: 3.7 m to 6.8 m
Elevation: 307.9 m to 304.8 m
Date: June 15, 2017

Photo 6. Bedrock Core Sample for BH7 from Elevation 307.9 m to 304.8 m

Project NO: ADM 0002331875-H0
BH NO: 8
Run NO: 1,2 &3
Sample Depth: 2.3 m to 5.3 m
Elevation: 309.2 m to 306.2 m
Date: June 13, 2017

Photo 7. Bedrock Core Sample for BH8 from Elevation 309.2 m to 306.2 m

Appendix F –
Golder's Technical Memorandum

TECHNICAL MEMORANDUM

DATE May 25, 2017 **PROJECT No.** 1541608-10

TO J.P. Perron, P.Eng.
Ministry of Transportation Geotechnical Section

FROM John Hagan, P.Eng.
Andrew Balasundaram, P.Eng

EMAIL John_Hagan@golder.com
Andrew_Balasundaram@golder.com

PAVEMENT ENGINEERING RETAINER - AGREEMENT NUMBER 5015-E-0012
ASSIGNMENT NO. 10 - FIELD INVESTIGATION FOR HIGHWAY 672
CENTRELINE CULVERT AT APPROXIMATELY STA 10+150,
ARNOLD TOWNSHIP, ONTARIO

Golder Associates Ltd. (Golder) is pleased to provide this Technical Memorandum summarizing the results of the field investigation completed for the replacement of the existing centreline culvert located along Highway 672, at approximately Sta. 10+150 in the Township of Arnold, northeast of Kirkland Lake Ontario. We understand that the highway has previously washed out and that the MTO requires factual geotechnical information to support the design of the replacement culvert.

The field investigation completed on May 16, 2017 consisted of advancing three rock probes (P1, P2 and P4) to depths ranging from 12.5 m to 17.4 m below existing ground surface and 4.5 m to 4.6 m into inferred bedrock. The probeholes were advanced with a Junjin JD-800E Air-track using a 75 mm diameter drill bit. The approximate probehole locations are shown on the attached pedological sketch. The high rate of water flowing through the embankment resulted in drill sediments downstream, and it was also difficult to effectively manage drill cuttings using airtrack drilling methods. These issues were discussed with MTO and the decision made not to advance Probehole P3.

The as-drilled probehole locations were referenced in the field to the highway centreline and existing culvert, and converted into northing/easting coordinates on the plan drawing. The ground surface elevations were surveyed to HCP 101 as provided by MTO (ExplodedVictoriaCreekDTM.dwg). The MTM NAD83 Zone 10 northing and easting coordinates, the ground surface elevations referenced to Geodetic datum, and the probehole depths at each location are presented on the attached Record of Probehole sheets.

The inferred stratigraphy presented on the Record of Probehole sheets and shown on the stratigraphic profile on Drawing 1 is based on visual observations of drill cuttings, progress/advancement of the drill string through the subsurface materials encountered in the probehole, and audible observations of the drill string head striking the drill rods. Given the composition of the embankment fill materials, it was not possible to identify the approximate transition from embankment material to native soils (if present), present above the inferred bedrock surface.

We trust this Technical Memo meets MTO's approval. Please do not hesitate to contact the undersigned with any questions or concerns.

Yours very truly,

GOLDER ASSOCIATES LTD.

John Hagan

Andrew C. Balasundaram

John Hagan, P.Eng.
Geotechnical Engineer

Andrew Balasundaram, P.Eng.
Pavement Engineering Project Manager

Attachments: Pedological Sketch
Record of Probeholes P1, P2 and P4

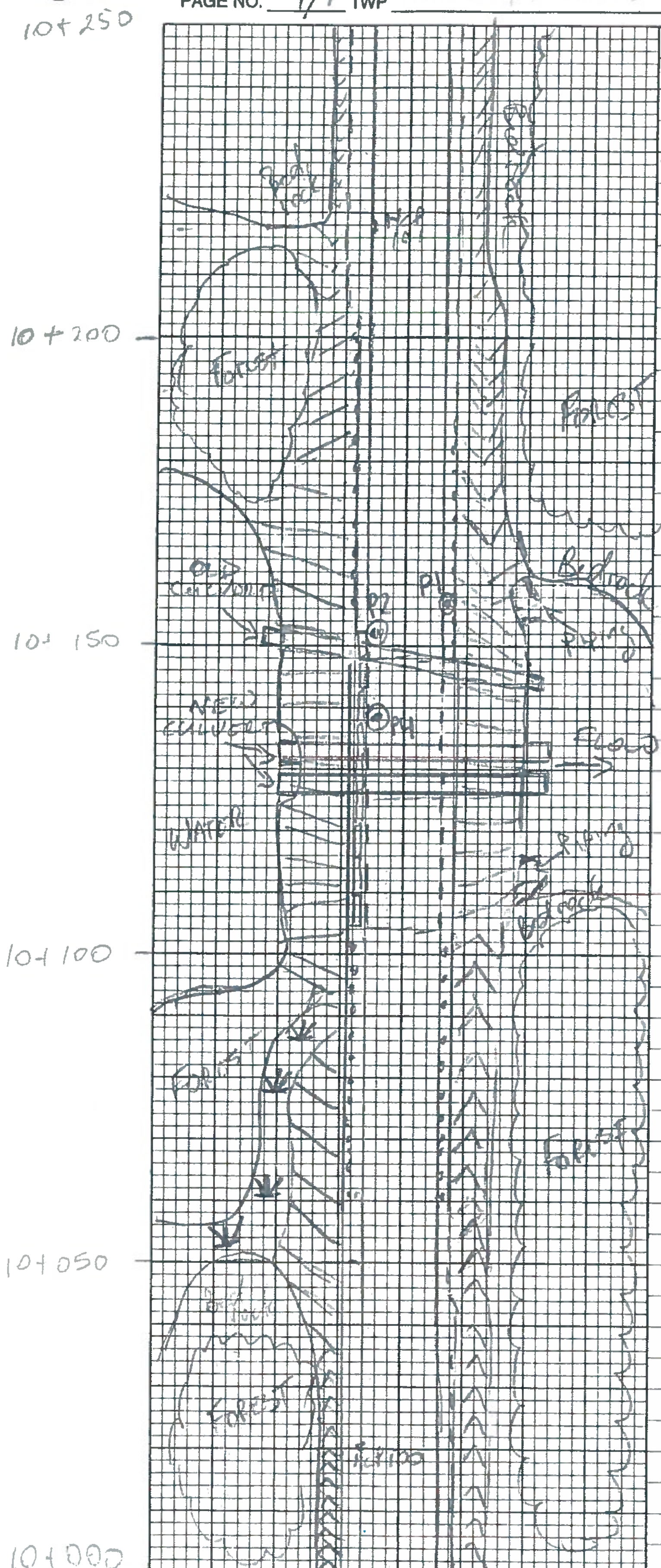
AC/JBH/ACB/us

w:\active\2015\3 proj\1541608 mto_5015-e-0012_ner retainer (east)\assignment 10\reporting\draft\1541608 (10000) tm assignment 10 - culvert hwy 672 - 2017'05'25.docx

PEDOLOGICAL SKETCH

Golder Associates

PAGE NO. 1/1 TWP ______ WP NO. ______ DATE ______



- Exposed bedrock on all four sides of creek crossing

- Water flowing west to east

[illegible] culvert [illegible] outlet, [illegible] spill [illegible] (a [illegible] seep [illegible]) under [illegible]

HWY NO. 672 LOCATION Veto Cre[illegible] ENGINEER M[illegible]

RECORD OF PROBEHOLES P1, P2 AND P4

RECORD OF PROBEHOLE No P1

 METRIC

PROJECT 1541608

LOCATION N 5339916.7; E 389558.4

ORIGINATED BY MR

DIST HWY 672

BOREHOLE TYPE 75 mm Diameter Airtrack Probehole

COMPILED BY AC

DATUM GEODETIC

DATE May 16, 2017

CHECKED BY

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 kPa (○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED) 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L / WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
313.7	GROUND SURFACE										
0.0	Inferred Embankment FILL						313				
			1	Probe	-		312				
							311				
	Boulder between 3.0 m and 3.5 m depth.						310				
	Cobbles between 4.6 m and 4.9 m depth.		2	Probe	-		309				
							308				
							307				
			3	Probe	-		306				
305.8 / 7.9	Inferred BEDROCK						305				
							304				
			4	Probe	-		303				
							302				

Continued Next Page

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

RECORD OF PROBEHOLE No P1

 METRIC

PROJECT 1541608

LOCATION N 5339916.7; E 389558.4

ORIGINATED BY MR

DIST HWY 672

BOREHOLE TYPE 75 mm Diameter Airtrack Probehole

COMPILED BY AC

DATUM GEODETIC

DATE May 16, 2017

CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	— CONTINUED FROM PREVIOUS PAGE —										
	Inferred BEDROCK										
301.2			5	Probe	-						
12.5	END OF PROBEHOLE										

SUD-MTO 001 1542608 ASSIGNMENT_10.GPJ GAL-MISS.GDT 23/05/17 DATA INPUT:

$+^{3}, \times^{3}$: Numbers refer to Sensitivity

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

RECORD OF PROBEHOLE No P2

 METRIC

PROJECT 1541608

LOCATION N 5339911.7; E 389550.3

ORIGINATED BY MR

DIST HWY 672

BOREHOLE TYPE 75 mm Diameter Airtrack Probehole

COMPILED BY AC

DATUM GEODETIC

DATE May 16, 2017

CHECKED BY

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 kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED (20 40 60 80 100)	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
313.7	GROUND SURFACE										
0.0	Inferred Embankment FILL										
							313				
			1	Probe	-						
							312				
							311				
							310				
	Cobbles between 3.8 m and 4.0 m depth.										
			2	Probe	-						
							309				
							308				
							307				
			3	Probe	-						
							306				
							305				
	Boulder between 9.1 m and 9.4 m depth.										
							304				
	Boulder between 10.1 m and 10.7 m depth.										
			4	Probe	-		303				
302.4 / 11.3	Inferred BEDROCK										
							302				

Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity

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

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RECORD OF PROBEHOLE No P2

 METRIC

PROJECT 1541608

LOCATION N 5339911.7; E 389550.3

ORIGINATED BY MR

DIST HWY 672

BOREHOLE TYPE 75 mm Diameter Airtrack Probehole

COMPILED BY AC

DATUM GEODETIC

DATE May 16, 2017

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 kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED) 20 40 60 80 100	PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	— CONTINUED FROM PREVIOUS PAGE —										
	Inferred BEDROCK						301				
			5	Probe	-		300				
							299				
297.9			6	Probe	-		298				
15.8	END OF PROBEHOLE										

SUD-MTO 001 1542608 ASSIGNMENT_10.GPJ GAL-MISS.GDT 23/05/17 DATA INPUT:

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

Golder Associates

RECORD OF PROBEHOLE No P4

 METRIC

PROJECT 1541608

LOCATION N 5339897.7; E 389550.9

ORIGINATED BY MR

DIST ____ HWY 672

BOREHOLE TYPE 75 mm Diameter Airtrack Probehole

COMPILED BY AC

DATUM GEODETIC

DATE May 16, 2017

CHECKED BY ____

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
313.6	GROUND SURFACE										
0.0	Inferred Embankment FILL										
							313				
			1	Probe	-		312				
							311				
							310				
	Cobble between 0.6 m and 0.8 m depth.										
			2	Probe	-		309				
							308				
							307				
			3	Probe	-		306				
							305				
	Boulder between 9.1 m and 9.4 m depth.						304				
	Boulder between 10.1 m and 10.7 m depth.										
			4	Probe	-		303				
							302				

Plot legend: DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED; 20 40 60 80 100

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60

Continued Next Page

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

Golder Associates

RECORD OF PROBEHOLE No P4

 METRIC

PROJECT 1541608

LOCATION N 5339897.7; E 389550.9

ORIGINATED BY MR

DIST HWY 672

BOREHOLE TYPE 75 mm Diameter Airtrack Probehole

COMPILED BY AC

DATUM GEODETIC

DATE May 16, 2017

CHECKED BY

ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: "N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED) (20 40 60 80 100)	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%) (20 40 60)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	— CONTINUED FROM PREVIOUS PAGE —										
	Inferred Gravel (FILL)						301				
300.8 / 12.8	Inferred BEDROCK		5	Probe	-		300				
							299				
							298				
			6	Probe	-		297				
296.2 / 17.4	END OF PROBEHOLE										

$+^3, \times^3$: Numbers refer to Sensitivity

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

Appendix G –
Slope Stability Analyses

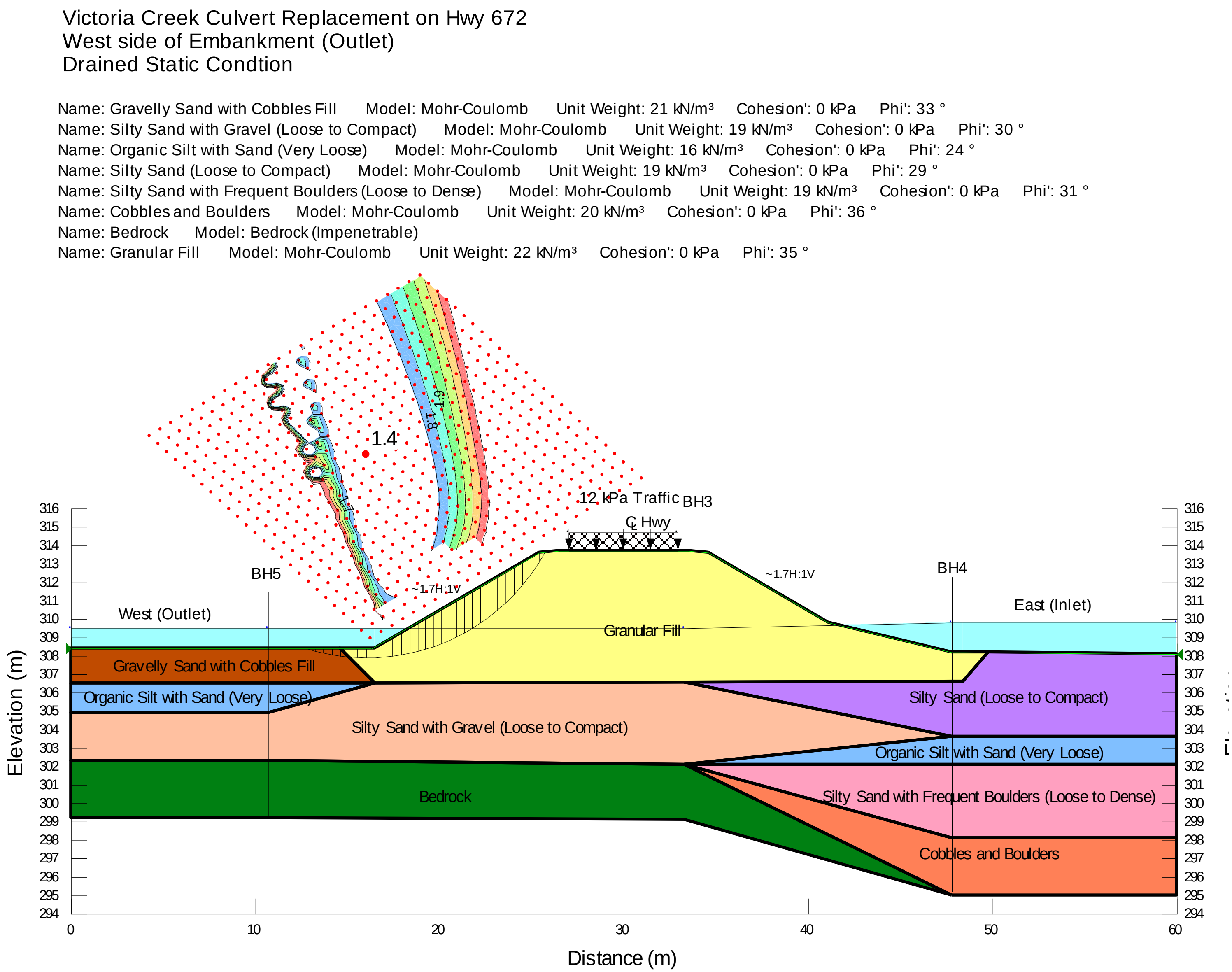


Figure 1: Slope stability analysis – Drained static conditions for west side of embankment

Victoria Creek Culvert Replacement on Hwy 672
East side of Embankment (Inlet)
Drained Static Condtion

Name: Gravelly Sand with Cobbles Fill Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion': 0 kPa Phi': 33 °
Name: Silty Sand with Gravel (Loose to Compact) Model: Mohr-Coulomb Unit Weight: 19 kN/m³ Cohesion': 0 kPa Phi': 30 °
Name: Organic Silt with Sand (Very Loose) Model: Mohr-Coulomb Unit Weight: 16 kN/m³ Cohesion': 0 kPa Phi': 24 °
Name: Silty Sand (Loose to Compact) Model: Mohr-Coulomb Unit Weight: 19 kN/m³ Cohesion': 0 kPa Phi': 29 °
Name: Silty Sand with Frequent Boulders (Loose to Dense) Model: Mohr-Coulomb Unit Weight: 19 kN/m³ Cohesion': 0 kPa Phi': 31 °
Name: Cobbles and Boulders Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion': 0 kPa Phi': 36 °
Name: Bedrock Model: Bedrock (Impenetrable)
Name: Granular Fill Model: Mohr-Coulomb Unit Weight: 22 kN/m³ Cohesion': 0 kPa Phi': 35 °

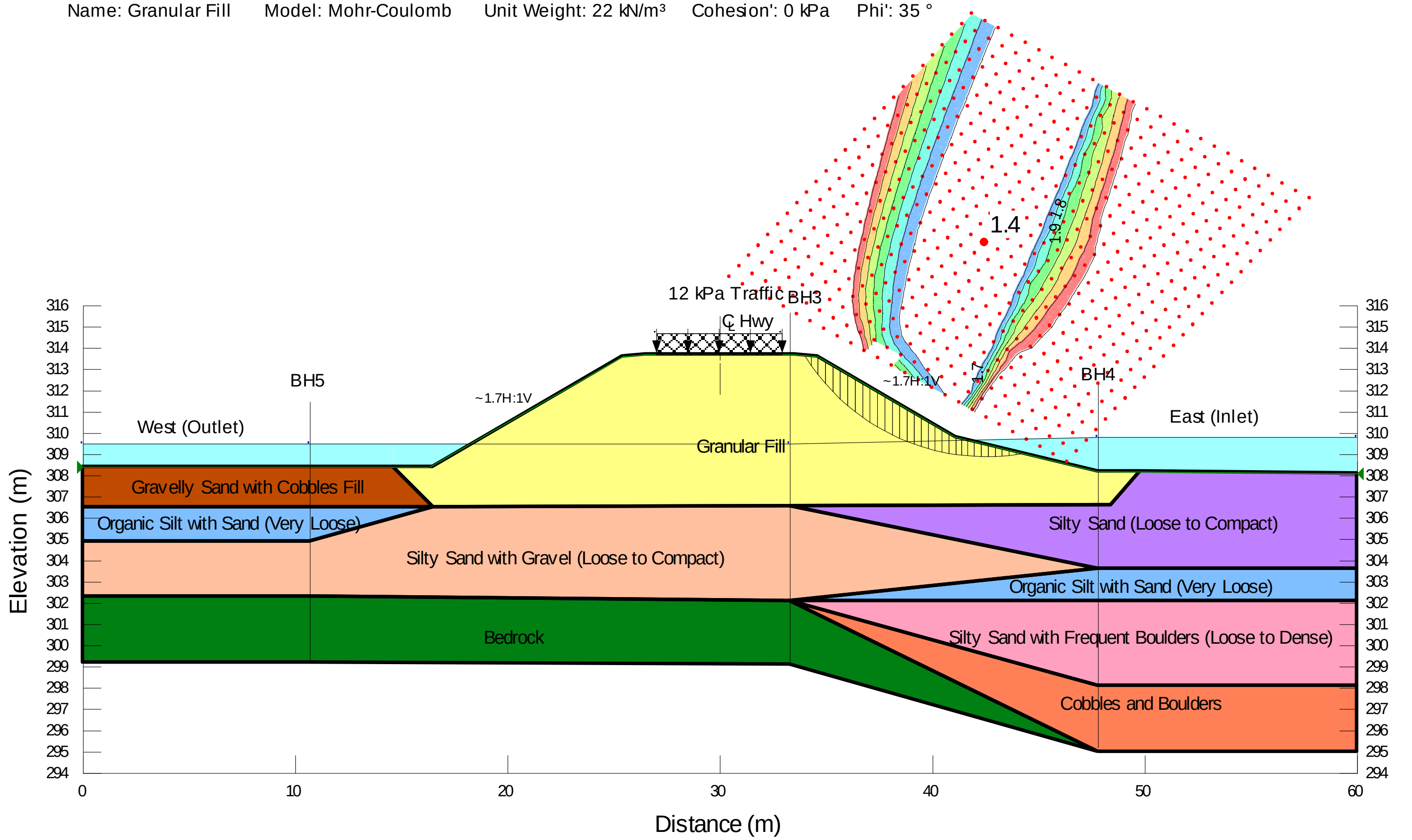
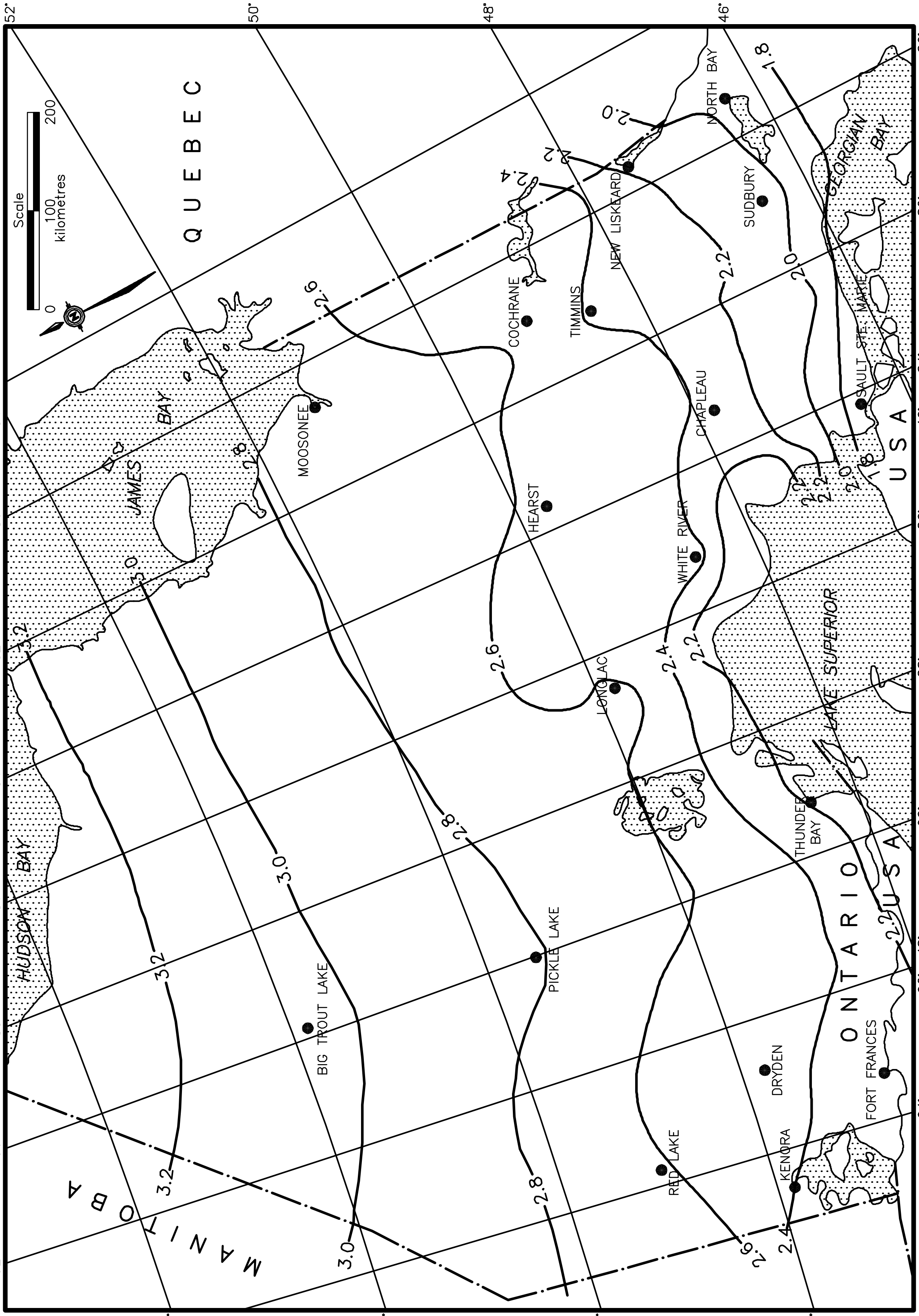


Figure 2: Slope stability analysis – Drained static conditions for east side of embankment

Appendix H – OPSDs



NOTES:

A These values are approximate and should only be used where the recommendations of a geotechnical engineer are not available.

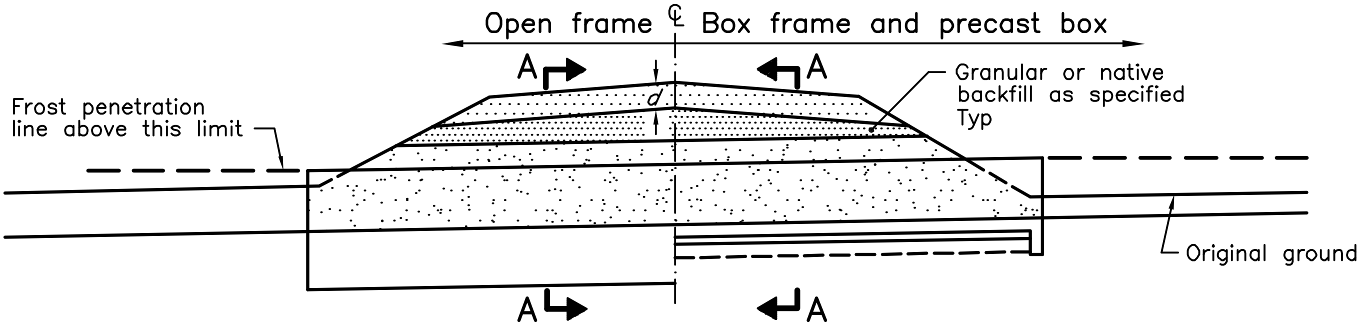
B This information is based on the Ministry of Transportation and Communications Research Publication RR225 "Aspects of Prolonged Exposure of Pavements to Sub-Zero Temperatures" dated December 1981.

C Values between contours should be interpolated. If interpolation is not possible, use the adjacent contour with the greater depth.

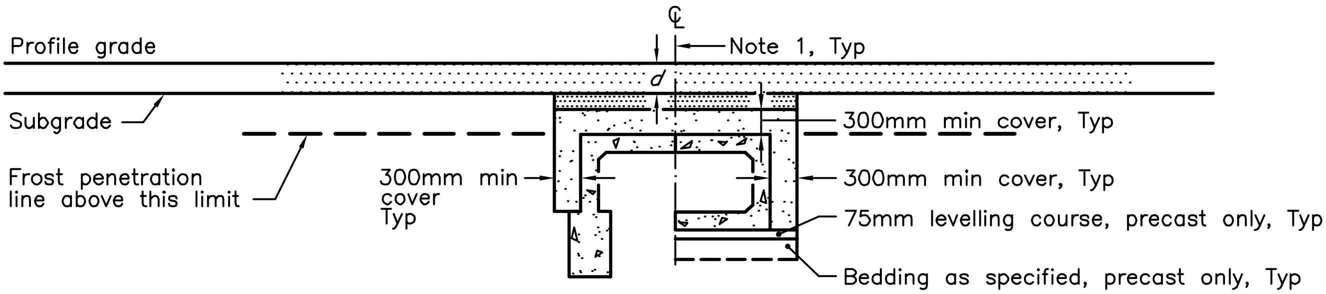
D Frost penetration depths are in metres.

ONTARIO PROVINCIAL STANDARD DRAWING	Nov 2010	Rev 1
FOUNDATION FROST PENETRATION DEPTHS FOR NORTHERN ONTARIO	OPSD 3090.100	

FROST PENETRATION LINE AT OR ABOVE TOP OF CULVERT

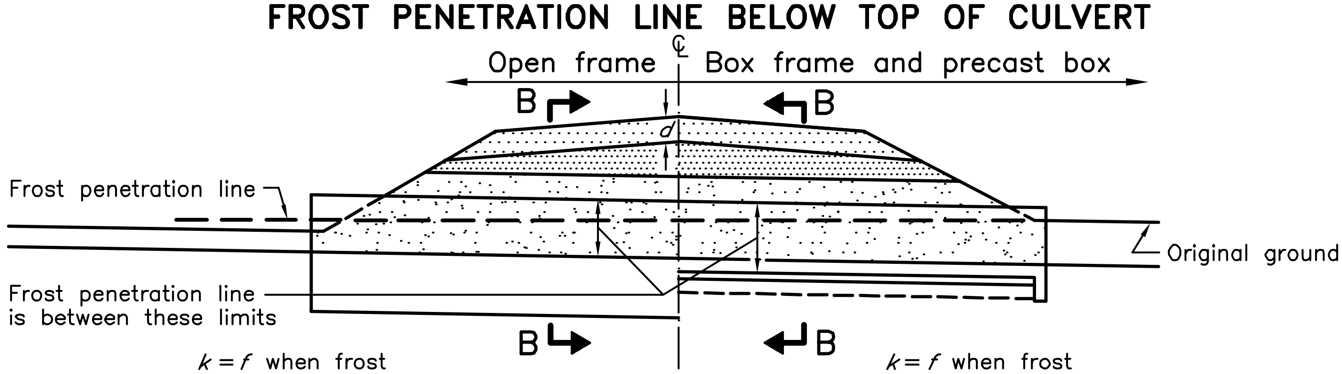


LONGITUDINAL SECTION

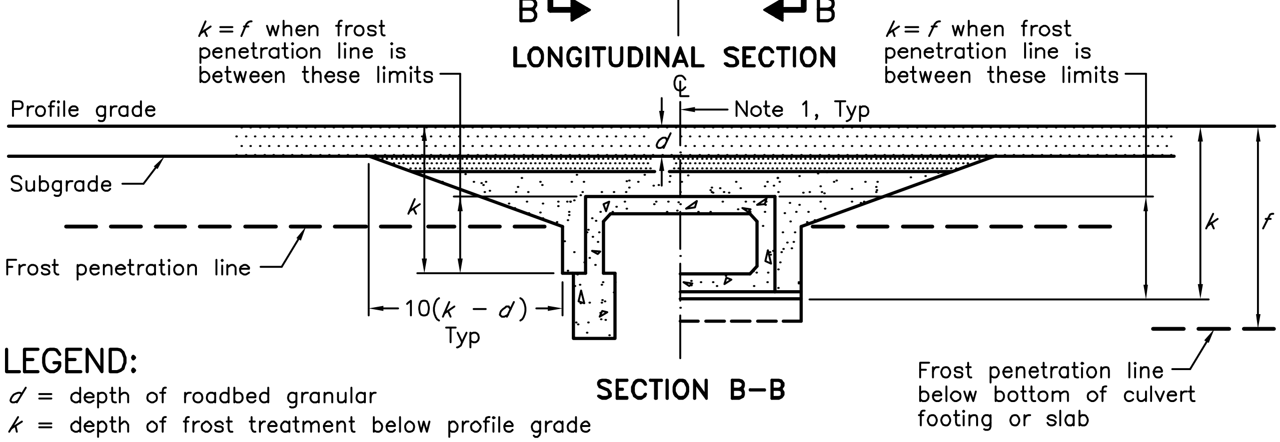


SECTION A–A

FROST PENETRATION LINE BELOW TOP OF CULVERT



LONGITUDINAL SECTION



SECTION B–B

LEGEND:

d = depth of roadbed granular
k = depth of frost treatment below profile grade
f = depth of frost penetration below profile grade

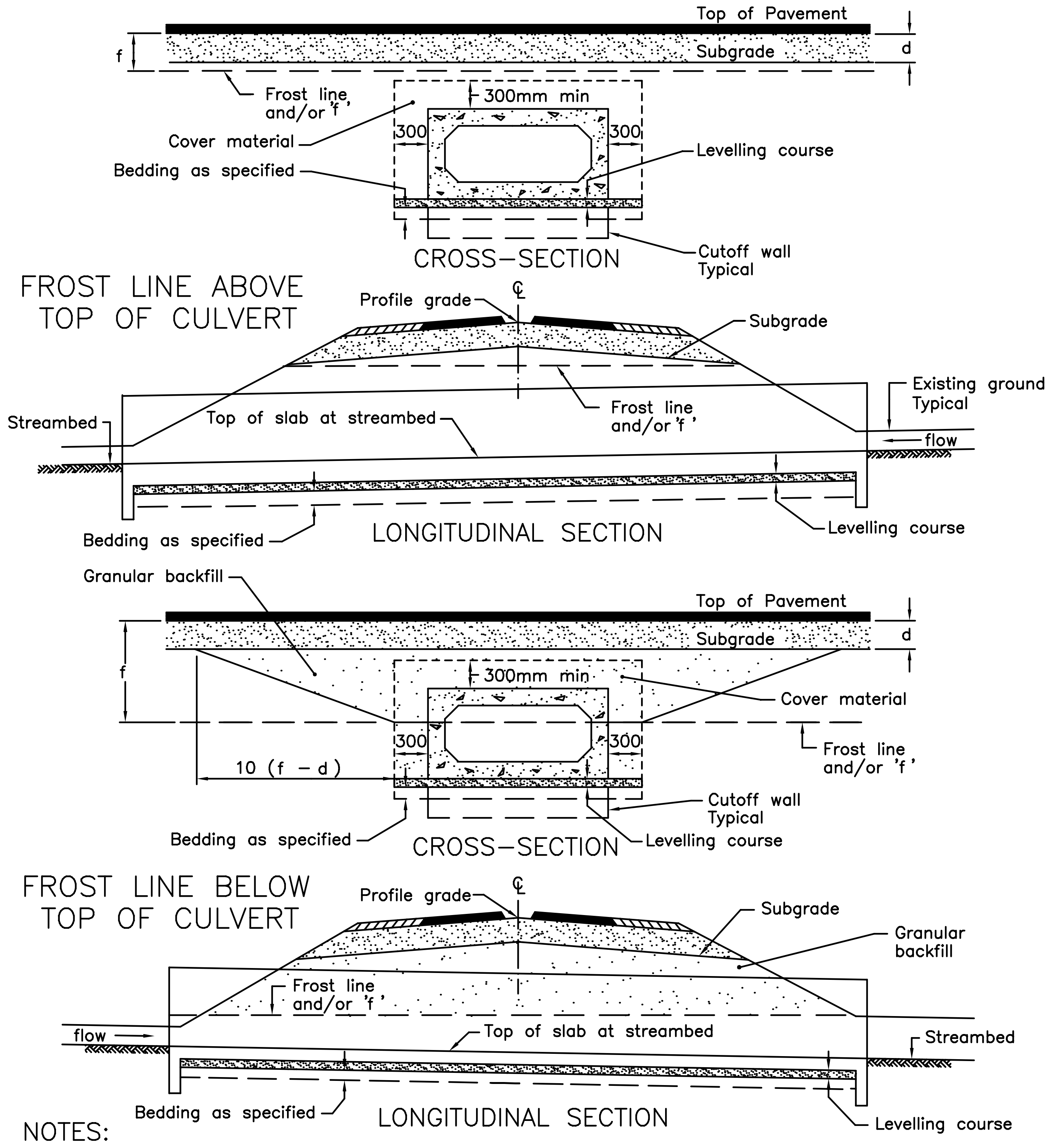
NOTES:

1 Condition of frost treatment symmetrical about centreline of culvert.

A Bedding, levelling, and cover material shall be granular as specified.
B The depth of roadbed granular shall be 600mm minimum.
C The maximum depth of frost treatment shall be bottom of box frame or top of footing.
D All dimensions are in millimetres unless otherwise shown.

ONTARIO PROVINCIAL STANDARD DRAWING	Nov 2010	Rev 2
BACKFILL AND COVER FOR CONCRETE CULVERTS WITH SPANS LESS THAN OR EQUAL TO 3.0M		
	OPSD 803.010	

ONTARIO STANDARDS · MUNICIPAL – PROVINCIAL · OPS



NOTES:

A Bedding, levelling, cover and backfill material to be granular as specified.

B Where frost line is below bottom of levelling course, frost tapers start at the bottom of levelling course.

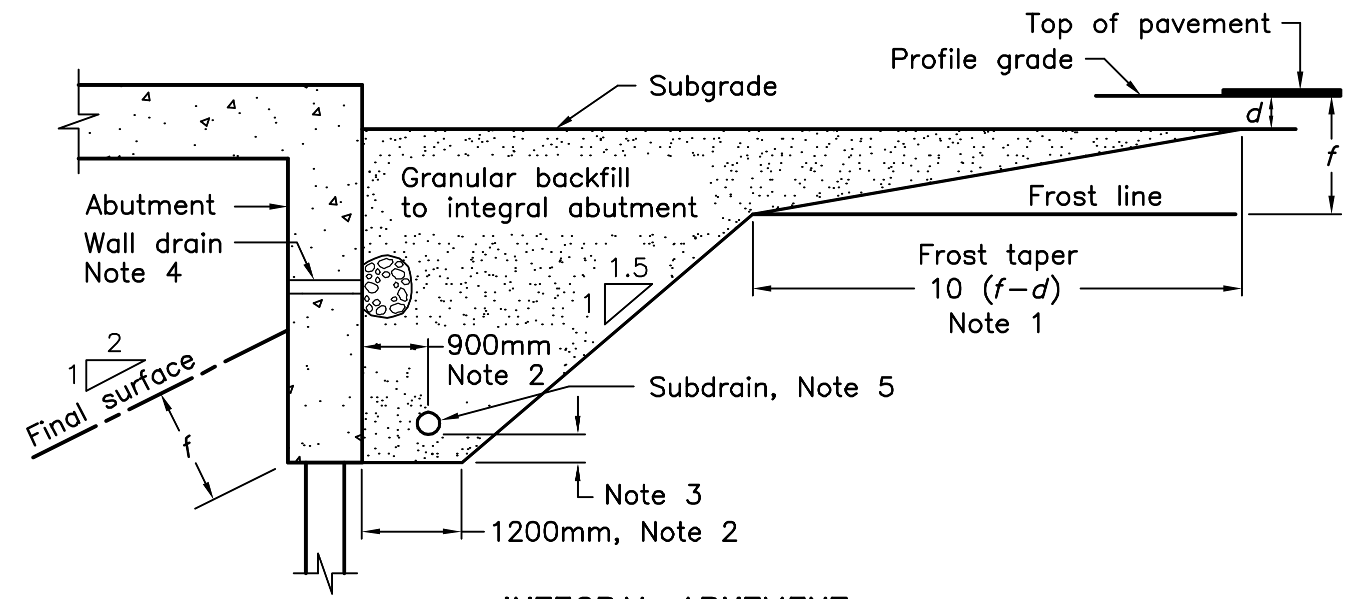
C All dimensions are in millimetres unless otherwise shown.

LEGEND:

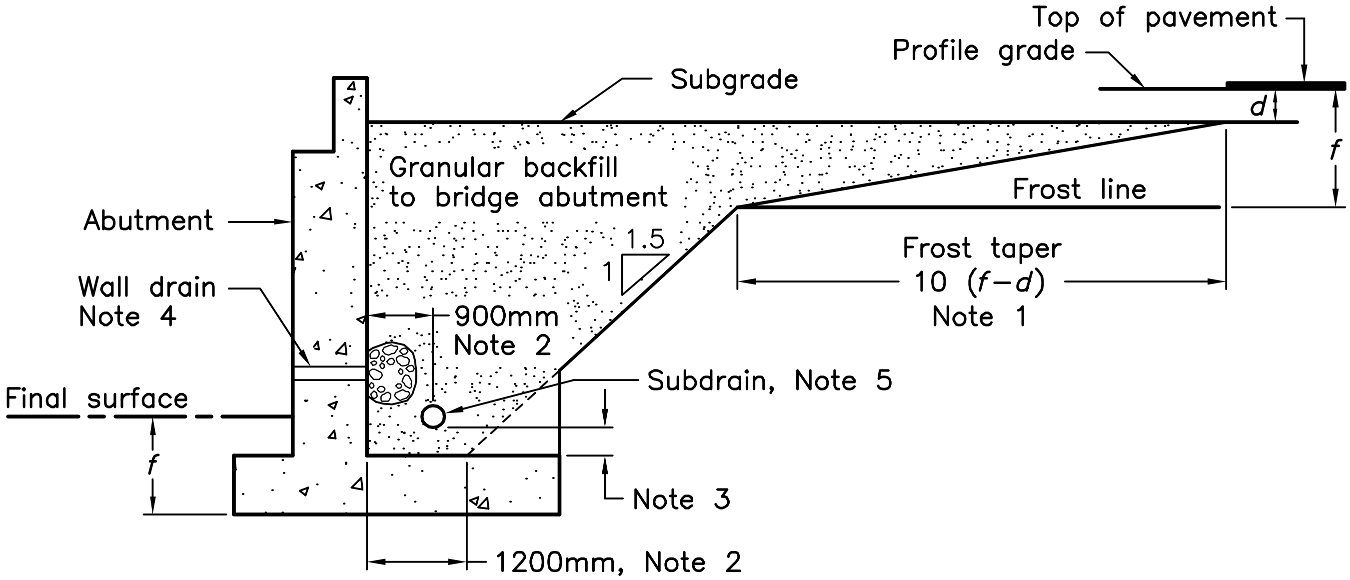
d = Denotes depth of granular (roadbed)

f = Depth of frost treatment=____ (measured from profile grade)

MINISTRY OF TRANSPORTATION ONTARIO DRAWING	Date	1994 05 25	Rev
BEDDING AND BACKFILL FOR PRECAST CONCRETE BOX CULVERTS	Issue Date WP Issued by		
	MTOD – 803.021		



INTEGRAL ABUTMENT



ABUTMENT

NOTES:

1 d = depth of combined base and subbase courses
  f = frost penetration depth 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 backfill unless interrupted by the retaining wall or wingwall.

B Sections shown are parallel to centreline of roadway.

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

D All dimensions are in millimetres unless otherwise shown.

ONTARIO PROVINCIAL STANDARD DRAWING	Nov 2010	Rev 1
WALLS ABUTMENT, BACKFILL MINIMUM GRANULAR REQUIREMENT	OPSD 3101.150	

ONTARIO STANDARDS MUNICIPAL – PROVINCIAL

- A drainage system behind a retaining structure should ensure that a groundwater table does not exist above the footing level. Preferably, the ground water level is controlled by the use of free-draining granular backfill and a collection system such as weep holes or perforated drains at the footing level. These weep holes and drains should be inspected and maintained to ensure that they do not become blocked. If free-draining, granular backfill is not employed, the permeability of the backfill and the hydrostatic head will control the extent to which the groundwater table can be depressed locally by seepage towards a footing drain. In practice, design for frost protection is best done using free-draining backfill.
  The design should also consider the risk of unusually large inflows of water creating a temporary hydrostatic head of water behind the wall. An example is the overtopping of a retaining wall, adjoining a large body of water, by storm waves. Measures such as the use of quarried rock backfill, design for full hydrostatic pressure, or provision of a sloped impermeable surface layer should be considered.
  Measurements have shown that earth pressures can vary seasonally, but the effects have normally been neglected in design, except for winter frost pressures. These latter can be very large if the backfill is frost susceptible and for this reason free-draining granular backfill is recommended.
- Figure C6.20 shows examples of minimum backfill requirements.
  The distance, x, should be equal to or greater than the estimated vertical frost penetration. This distance may be reduced if the wall abuts a vertical face of bedrock that is not susceptible to frost. The frost penetration may be reduced by the use of suitable insulation, in which case a thermal analysis should be performed by a Geotechnical Engineer.
  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.

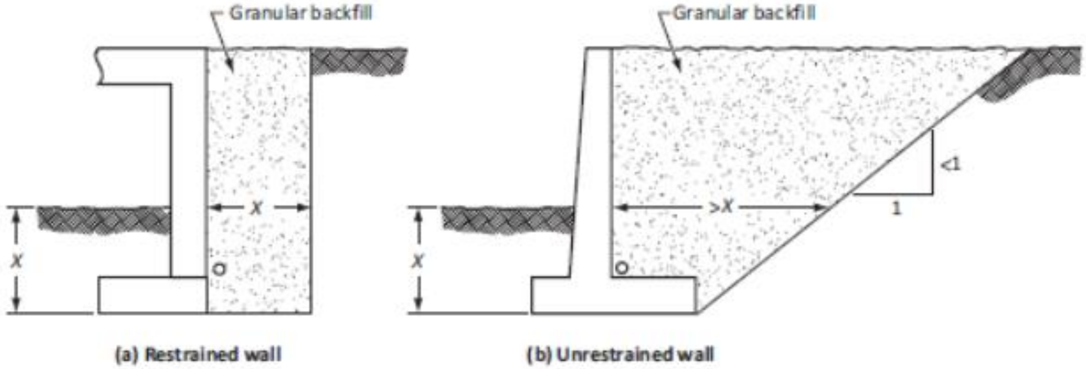
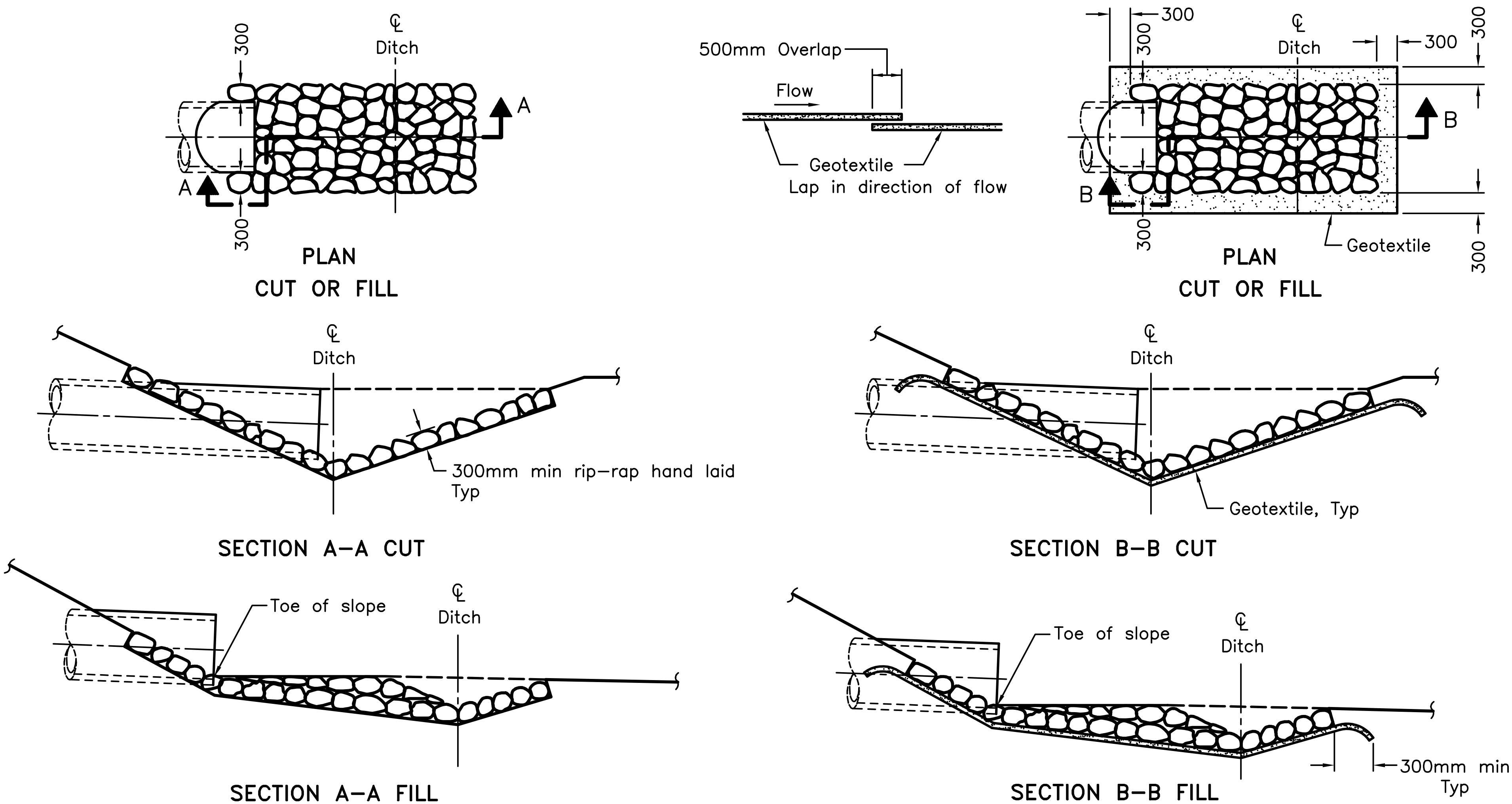


Figure C6.20
Backfill for frost protection
(See Clause C6.12.1.)

C6.12.2 Lateral ground pressures

C6.12.2.1 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 6.12.2.2 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.



TYPE A – WITHOUT GEOTEXTILE

TYPE B – WITH GEOTEXTILE

NOTES:

A All dimensions are in millimetres unless otherwise shown.

ONTARIO PROVINCIAL STANDARD DRAWING	Nov 2001	Rev 0

RIP–RAP TREATMENT

FOR SEWER AND CULVERT OUTLETS

OPSD – 810.010

Appendix I –
Non-Standard Special Provisions
(NSSPs)

NSSP FOR COBBLES AND/ BOULDERs OBSTRUCTIONS

Scope of Work

The Contractor should be aware that cobbles and boulders were encountered within the silty sand layer or overlying the bedrock at the boreholes advanced at the site. It is also encountered in embankment fill at the boreholes advanced through the road surface and underneath of foot print of existing embankment. Consideration of the presence of these obstructions must be made in the selection of appropriate equipment and procedures for piling /or for temporary shoring through these materials.

Basis of Payment

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