



FINAL REPORT

FOUNDATION INVESTIGATION AND DESIGN REPORT

**Melgund Creek Tributary Culvert Replacement, Highway 603, Site No. 41S-142/C,
Township of Melgund, District of Kenora**

Agreement No. 6014-E-0017

Assignment No. 8

GWP 6359-14-00

Geocres No. 52F-045

Prepared for:

Ontario Ministry of Transportation
Regional Director's Office -NW Region
615 James Street South
Thunder Bay, ON P7E 6P6
Attn: Mike Satten

Ontario Ministry of Transportation
Pavements and Foundations Section
Foundations Group
Building 'C', Room 223
1201 Wilson Avenue
Downsview, ON M3M 1J8
Attn: K.Ahmad

exp Services Inc.
December 23, 2015

Ministry of Transportation

Foundation Investigation and Design Report

Agreement No. 6014-E-0017

Assignment No. 8

GWP 6359-14-00

Geocres No. 52F-045

Type of Document:

Final

Project Name:

Foundation Investigation and Design Report for Melgund Creek Tributary Culvert
HWY 603, Site No. 41S-142/C, Township of Melgund, District of Kenora

Project Number:

ADM-00223648-G0

Prepared By:

Ian MacMillan, P.Eng.
Nimesh Tamrakar, M.Eng., EIT
Silvana Micic, Ph.D., P.Eng.

Reviewed By:

TaeChul Kim, M.E.Sc. P.Eng.
Stan E. Gonsalves, M.Eng., P.Eng.

exp Services Inc.

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



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



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

Date:

December 23, 2015

Table of Contents

Part I: 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 Laboratory Testing	3
1.4 Subsurface Conditions	3
1.4.1 Surface Treated Asphalt	3
1.4.2 Topsoil	4
1.4.3 Fill – Silty Sand/Sandy Silt	4
1.4.4 Fill – Sand	4
1.4.5 Organic Silt	5
1.4.6 Clayey Silt	5
1.4.7 Silt	6
1.4.8 Dynamic Cone	7
1.5 Water Conditions	7
1.6 Chemical Analyses	7
Part II: ENGINEERING DISCUSSION AND RECOMMENDATIONS	9
2.1 General	9
2.2 Expected Ground Conditions	9
2.3 Structure Foundations	10
2.3.1 Shallow Foundations	12
2.3.2 Steel Sheet Piles	13
2.4 Lateral Earth Pressure	14
2.5 Construction Alternatives	15
2.5.1 Detour Options (Options 1 and 2)	19
2.5.2 Half-and-Half Construction (Options 3)	20
2.6 Temporary Roadway Protection	22
2.7 Culvert Bedding	22

2.8	Culvert Backfill	23
2.9	Surface Water and Groundwater Control	24
2.10	Embankment Design	26
2.10.1	Embankment Settlement	26
2.10.2	Embankment Stability	26
2.11	Inlet and Outlet.....	27
2.11.1	Erosion Protection at Outlet.....	27
2.11.2	Stream Bed Rip-Rap.....	27
2.11.3	Seepage Cut-off Requirements	28
2.12	Corrosion Protection	29
2.13	Operational Constraints (OCs) and Non Standard Special Provisions (NSSPs)	29
Part III: CLOSURE		30
Part IV: LIMITATIONS AND USE OF REPORT		31

Appendices

APPENDIX A: PHOTOGRAPHS

APPENDIX B: DRAWINGS

APPENDIX C: BOREHOLE LOGS

APPENDIX D: LABORATORY DATA

APPENDIX E: CHEMICAL ANALYSES

APPENDIX F: SLOPE STABILITY ANALYSIS

APPENDIX G: OPSDs

APPENDIX H: SCHEMATIC SKETCHES FOR CONSTRUCTION ALTERNATIVES

APPENDIX I: OPERATIONAL CONSTRAINS AND NON STANDARD SPECIAL PROVISIONS
 (NSSPs)

Part I: FOUNDATION INVESTIGATION REPORT

1.1 Introduction

This foundation investigation report presents the results of a geotechnical investigation completed by **exp** Services Inc. for the replacement of a structural culvert on Hwy 603, located 1.5 km north of Hwy 17 at the Melgund Creek Tributary Crossing in the Township of Melgund, District of Kenora, the Ministry of Transportation (MTO) Northeastern Region. The work was undertaken under Agreement # 6014-E-0017, Assignment No. 8 (GWP 6359-14-00). The terms of reference (TOR) were as presented in the MTO letter dated August 20, 2015.

The purpose of the investigation is to evaluate the subsurface condition along the proposed culvert replacement alignment, to permit preliminary design for the culvert replacement. The site specific geotechnical investigation consisted of borings, soil sampling, borehole logging, and field and laboratory testing.

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

1.2 Site Description and Geological Setting

1.2.1 Site Description

The Culvert replacement site is located on Hwy. 603 in the Township of Melgund, at the Melgund Creek Tributary Crossing (approx. Station 10+030). The location of the culvert and a cross section of the existing culvert alignment are shown on Dwg. Nos. 1 and 2 in Appendix B.

The existing culvert consists of a structural plate corrugated steel pipe arch (SPCSPA), approximately 3.7 m span, 2.1 m high and 17.0 m long. At this site, Hwy 603 is a surface treated two lane, north/south roadway having approximately 0.5 to 1.0 m wide granular shoulders. The highway embankment at the investigated location is approximately 3.0 m high on both sides of the roadway, having side slopes of approximately 1H:1V to 2H:1V from the top of the embankment to the toe of the embankment. Photographs of the site and existing culverts are presented in Appendix A.

The general site conditions were assessed during the drilling operations between October 19 and 21, 2015. The surrounding terrain of culvert location is generally flat with some small hills with a mix of low lying vegetation/shrubs, long grasses, and treed areas with both deciduous and coniferous trees. At the site location, water flows from east to west, however, the current did not appear excessive at the time of the field investigation. The embankment surrounding the culvert inlet and outlet were vegetated. No riprap was present, however, boulders were observed near surface in the areas surrounding the culvert.

1.2.2 Geological Setting

The Map 2542 (Bedrock Geology of Ontario, West-Central Sheet, 1991) of the Ministry of Northern Development and Mines, indicates that the bedrock formation of the project area consists of mafic to intermediate metavolcanic rocks, consisting of basaltic and andesitic flows, tuffs and breccias, chert, iron formation, minor metasedimentary and intrusive rocks, and related migmatites. The Map 2554 (Quaternary Geology of Ontario, West-Central Sheet, 1991) of the Ministry of Northern Development and Mines, indicates that the surface conditions in the vicinity of site consist of glaciolacustrine deposits consisting of sand, gravelly sand and gravel; nearshore and beach deposits.

1.3 Investigation Procedures

1.3.1 Site Investigation and Field Testing

The field investigation was performed between October 19, 2015 and October 21, 2015. The field program consisted of drilling four (4) sampled boreholes (BH-M1, BH-M2, BH-M3, and BH-M4). The boreholes were strategically located along the existing culvert alignment to provide subsurface information for the design of the proposed new culvert. Boreholes BH-M1 and BH-M2 were advanced within the travelled northbound and southbound lanes, respectively, as close as possible to the embankments. Boreholes BH-M3 and BH-M4 were advanced at accessible locations near the inlet and outlet of the culvert, respectively. The borehole locations are shown on Dwg. No. 1 in Appendix B.

All of the boreholes were advanced using a track mounted CME-850 drill rig, equipped with hollow stem augers and standard soil sampling equipment operated by a specialist drilling contractor, Cartwright Drilling Inc. Each borehole was terminated approximately 10 m below the base of the existing culverts, with the exception of BH-M1, where a dynamic cone was advanced from the sampling termination depth to refusal on suspected bedrock at approximately 31.7 m depth.

The borehole locations (referenced to the MTM NAD83 coordinate system) and their ground surface elevations were surveyed by **exp** personnel. The benchmark utilized is based on information provided on Site Plan drawings provided by the MTO. The benchmark location is shown on Dwg. No. 1 in Appendix B. Borehole locations were determined using a hand-held GPS.

For the drilling program, soil samples were obtained using a 51 mm outside diameter (O.D.) split-spoon sampler in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586) at intervals ranging from 0.75 m to 1.5 m in depth as shown on the attached borehole logs (Appendix C). The original field (uncorrected) SPT "N" values were recorded on the borehole logs as recommended in the Canadian Foundation Engineering Manual (CFEM, pg. 40) and used to provide an assessment of in-situ consistency or relative density of non-cohesive soils.

Upon completion of the boreholes, ground water level measurements were carried out within the in accordance with the Ministry of Transportation guidelines. The measured ground water levels after completion of drilling boreholes were recorded on the borehole log sheets in Appendix C. The boreholes were decommissioned by bentonite/cement mixtures in accordance with the Ministry of

the Environment Regulation 903, as amended by Regulation 128/03 (the well regulation under the *Ontario Water Resources Act*).

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

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

1.3.2 Laboratory Testing

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

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

In addition, soil chemical testing was completed on one (1) as required by the TOR. The chemical testing included pH, water soluble sulphate, chloride, resistivity, sulphide, electrical conductivity analyses and redox potential. The results of the soil chemical testing are included in Appendix D.

1.4 Subsurface Conditions

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

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

In general, the subsurface conditions within the roadway embankment consist of silty sand to sand fill. The native soils consist of organic silt, silt, and clayey silt. A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections. A cross-section soil profile is included on Dwg. No. 2 in Appendix B.

1.4.1 Surface Treated Asphalt

Surface treated asphalt was encountered at the surface of boreholes BH-M1 and BH-M2 and was approximately 25 mm thick. Asphalt thicknesses may further vary beyond the borehole locations.

1.4.2 Topsoil

Topsoil was encountered at the surface of boreholes BH-M3 and BH-M4 and ranged in thickness from approximately 30 to 76 mm. Topsoil thicknesses may further vary beyond the borehole locations.

1.4.3 Fill – Silty Sand/Sandy Silt

Silty sand/sandy silt fill was encountered below the asphalt at BH-M1 and below topsoil at BH-M3. The layer was 3.6 m (BH-M1) and 1.5 m (BH-M3) thick and extended from Elev. 376.5 to 372.9 m. The silty sand/sandy silt fill contained trace to some gravel, and was brown and grey in colour, and damp to moist, becoming wet with depth. An approximately 450 mm diameter boulder was encountered within the fill at BH-M3 at 0.8 m depth. Uncorrected SPT “N” values within the fill ranged from 2 to 32 blows per 300 mm, classifying the fill as very loose to dense in compactness condition.

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

Moisture Content:

- 5.2% to 30.1%

Grain Size Distribution:

- 1 to 12% gravel;
- 24 to 53% sand; and
- 35 to 75% silt.

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

1.4.4 Fill – Sand

Sand fill was encountered below the asphalt at BH-M2. The layer was 2.7 m thick and extended from Elev. 376.6 to 373.9 m. The sand fill contained some gravel and trace silt, and was brown in colour and moist becoming wet with depth. Uncorrected SPT “N” values within the fill ranged from 6 to 17 blows per 300 mm, classifying the fill as loose to compact in compactness condition.

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

Moisture Content:

- 2.2% to 39.9%

Grain Size Distribution:

- 12% gravel;
- 80% sand; and

- 8% silt.

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

1.4.5 Organic Silt

Organic silt was encountered below the fill materials at BH-M2 and BH-M3 and below the topsoil at BH-M4. The layer was 0.7 to 1.6 m thick and extended from Elev. 375.5 to 372.3 m. The organic silt contained trace to some sand, trace to some clay, and some wood pieces. The organic silt was dark brown to grey in colour, wet, and had slight plasticity (BH-M4). Uncorrected SPT “N” values within the organic silt ranged from 0 to 8 blows per 300 mm, classifying the soil as very loose to loose in compactness condition.

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

Moisture Content:

- 20.2% to 54.5%

Grain Size Distribution:

- 0% gravel;
- 1 to 5% sand;
- 76 to 92% silt; and,
- 7 to 19% clay.

Atterberg Limits:

- Liquid Limit: 26.5%
- Plastic Limit: 21.4%
- Plasticity Index: 5.1%

The results of the moisture content and grain size distribution tests are provided on the borehole logs in Appendix C. The results of the grain size distribution tests are also provided on Figure 3 and Atterberg Limits test on Figure 5 in Appendix D.

1.4.6 Clayey Silt

Clayey silt was encountered below the organic silt at BH-M4. The layer was 7.6 m thick and extended from Elev. 373.3 to 365.7 m. The clayey silt contained trace organics near the surface of the layer and trace sand. The soil was grey in colour, wet, and had slight plasticity in the upper portion of the layer. Uncorrected SPT “N” values within the clayey silt ranged from 4 to 9 blows per 300 mm, classifying the soil as firm to stiff in consistency.

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

Moisture Content:

- 22.3% to 32.7%

Grain Size Distribution:

- 0% gravel;
- 1% sand;
- 72% silt; and,
- 27% clay.

Atterberg Limits:

- Liquid Limit: 25.0%
- Plastic Limit: 19.6%
- Plasticity Index: 5.4%

The results of the moisture content and grain size distribution tests are provided on the borehole logs in Appendix C. The results of the grain size distribution tests are also provided on Figure 2 and Atterberg Limits test on Figure 5 in Appendix D.

1.4.7 Silt

Silt was encountered below the fill at BH-M1, below the organic silt at BH-M2 and BH-M3 and below the clayey silt at BH-M4. The silt extended from as high as Elev. 372.9 m to the borehole/sampling termination depths in each borehole between Elev. 362.2 to 363.5 m. The silt contained trace to some clay, trace sand, and trace gravel and was grey in colour and wet. Uncorrected SPT "N" values within the silt ranged from 0 to 11 blows per 300 mm, classifying the soil as very loose to compact in compactness condition.

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

Moisture Content:

- 14.7% to 31.9%

Grain Size Distribution:

- 0 to 1% gravel;
- 0 to 4% sand;
- 78 to 96% silt; and,
- 4 to 18% clay.

The results of the moisture content and grain size distribution tests are provided on the borehole logs in Appendix C. The results of the grain size distribution tests are also provided on Figure 3 and 4 in Appendix D.

1.4.8 Dynamic Cone

A dynamic cone was advanced from the sampling termination depth in BH-M1 (Elev. 362.2 m). Dynamic cone values ranged from 6 to 87 blows per 300 mm, classifying the soil at loose to very dense in compactness condition. Dynamic cone values increased with depth, with refusal encountered at 31.7 m depth, Elev. 345.1 m.

1.5 Water Conditions

Groundwater levels were measured within the borehole upon completion of the drilling program and are shown on the borehole logs in Appendix C. Groundwater was encountered within each of the boreholes. However, water levels measured in open boreholes might not be stabilized due to short term observation.

Groundwater was encountered at the following Elevations at the time of the investigation (October, 2015):

- BH-M1, Elev. 375.9 m;
- BH-M2, Elev. 375.2 m;
- BH-M3, Elev. 370.9 m; and,
- BH-M4, Elev. 374.8 m.

Water levels were also measured at the culvert inlet and outlet. At both the culvert inlet and outlet, the water level was Elev. 374.7 m.

Note that water levels measured in open boreholes might not be stabilized due to short term observation.

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. Some perched water over clayey silt layers could exist in the embankment fill as well.

1.6 Chemical Analyses

One (1) representative soils sample was submitted to a CALA Certified Laboratory for chemical corrosivity analysis. The samples were analyzed for chloride, sulphate, pH, electrical conductivity, resistivity, redox potential, and sulphide concentrations. The results of the corrosivity testing are summarized below, with detailed results included in Appendix E.

Borehole BH-M1, Sample SS7 (Elev. 373.2 m):

- Sulphide: 0.02%;
- Chloride: 10 µg/g;

- Sulphate: 15 µg/g;
- pH: 8.91;
- Electrical Conductivity: 0.113 mS/cm;
- Resistivity: 8850 ohm.cm; and,
- Redox Potential: 237 mV.

Part II: ENGINEERING DISCUSSION AND RECOMMENDATIONS

2.1 General

This section of the report provides geotechnical design recommendations for replacement of the existing Melgund Creek Tributary Crossing Culvert, located on Hwy 603, about 1.5 km north of Hwy 17, in the Township of Melgund, District of Kenora, the Ministry of Transportation (MTO) Northwestern Region. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the current investigation at the site and presented in **Part I-Foundation Investigation Report**. The interpretation and recommendations provided are intended solely to permit designers to assess foundation alternatives and design the new culvert and replacement. Comments on construction are only provided to highlight issues that could affect the design. Contractors bidding on the works should make their own assessments of the factual data and how it might affect construction means and methods, scheduling and the like.

Based on information included in the TOR, it is understood that the existing culvert is a structural plate corrugated steel arch structure with a span of about 3.7 m, length of about 17 m and a height of about 2.1 m. It is understood that the existing culvert was constructed at an unknown date, and inspected in October 2013. The inspection remarked that the existing culvert was in fair to poor condition. It is noted that the culvert barrel was significantly deteriorated. It is also understood that the existing culvert will be replaced with a new culvert. The new culvert is proposed to be at the current alignment as well as that the road grade will be the same as that at the location of the existing culvert. The size and type of the new culvert is not defined at the time of writing this report. However for preliminary design purposes, the following options are being considered for the replacement: rigid frame box culvert (precast or cast-in place), rigid frame open footing culvert (precast or cast-in-place), corrugated steel plate culvert, and steel sheet pile abutment with precast concrete decking.

This part of the report addresses the geotechnical design of the foundation for the new culvert 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 from the MTO letter dated August 20, 2015. The assessment involved review of options for replacement of the existing culvert along the same alignment with a final selection to be made by the designer, based on the optimum solution.

2.2 Expected Ground Conditions

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

- a. Hwy 603 is a surface treated two lane roadway, with granular shoulders about 0.5 to 1.0 m wide.

Based on drawings provided, the roadway embankment is about 3.0 m high with side slopes of about 1H:1V to 2H:1V. The current elevation of the crest of the roadway is about 376.85 m at the culvert location.

- b. The highway embankment consists of dense to compact sand to silty sand fill (~3.1 to 3.9 m thick) underlain by loose to compact silt to about 14.6 m (~9.6 m to 10.7 m thick) below ground surface. The loose to very dense soil extends to about 31.7 m below ground surface (Elev. 345.4 m), where the refusal was encountered. A 1.6 m thick layer of loose organic silt is sandwiched between the fill and native silt layer in BH-M2.
- c. Below the fill at the inlet, a layer of native organic silt (~0.8 m thick) is underlain by loose silt to about 12.8 m below ground surface. At the outlet, a surficial layer of native organic silt (~1.2 m thick) is underlain by firm to stiff clayey silt to about 9.1 m below ground surface and followed by loose silt to about 11.3 m depth. The practical refusal is not encountered.
- d. The foundation soil at the invert of the new culvert is anticipated to be native loose to compact silt between Elev. 372.5 m to Elev. 373 m. Typical 'N' values ranged from 8 to 11.
- e. The groundwater table in the embankment fill is expected to be at approximate elevation 375.2 m, or slightly higher. However, 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. Some groundwater mounding within the embankment and perched water would be anticipated.

2.3 Structure Foundations

For preliminary design purpose, several options are being considered for the replacement:

- Rigid frame box culvert,
- Rigid frame open footing culvert supported on shallow foundations,
- Corrugated steel plate culvert supported on shallow foundations,
- Steel sheet pile abutment with precast concrete decking

Based on the subsurface information obtained from the site investigation, the native silt is considered suitable for support of all replacement options. However, the choice of culvert type will also depend on parameters such as the initial cost, maintenance costs, hydraulic performance, ease of construction, salvageability and local availability of material and equipment.

The layer of loose organic silt encountered below the existing embankment should be excavated and removed to firm bearing of native silt and grade restored with engineered fill. Since the depth of excavation to remove organic silt and/or other unstable soils could be excessive (approximately 1 m below the culvert invert), 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 over geotextile fabric would establish a stable bearing surface. The fabric should be installed a manner to mitigate the migration of fines from adjacent material.

Table 2.1 Evaluation of foundation alternatives

Options	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Rigid frame box culvert	1	<ul style="list-style-type: none"> ▪ Straightforward construction ▪ Reduce construction period, consequently traffic management and water control period ▪ Reduce excavation depth 	<ul style="list-style-type: none"> ▪ Dewatering system required ▪ Require heavy lifting equipment ▪ Require bedding material 	<ul style="list-style-type: none"> ▪ Low to medium 	<ul style="list-style-type: none"> ▪ 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
Rigid frame open footing concrete culvert	4	<ul style="list-style-type: none"> ▪ Wider span may consider to maintain existing channel ▪ High geotechnical resistance available ▪ Can incorporate dowels to enhance lateral resistance 	<ul style="list-style-type: none"> ▪ Deeper excavation or below water excavation may required ▪ Dewatering system required ▪ Possible uneven bedrock surface ▪ Require placement of lean concrete 	<ul style="list-style-type: none"> ▪ Likely more expensive than Option 1 	<ul style="list-style-type: none"> ▪ Risk of unacceptable differential settlements if the entire foundation is not supported on the competent soil ▪ Risk of delay in construction due to deeper excavation below water if proper dewatering is not maintain ▪ Higher scour risk
Corrugated Steel Pipe Culvert	2	<ul style="list-style-type: none"> ▪ Straightforward construction ▪ Reduce construction period, consequently traffic management and water control period ▪ Reduce excavation depth 	<ul style="list-style-type: none"> ▪ Dewatering system required ▪ Require bedding material ▪ Limited design life ▪ Potential for Corrosion 	<ul style="list-style-type: none"> ▪ Low to medium 	<ul style="list-style-type: none"> ▪ 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	3	<ul style="list-style-type: none"> ▪ Environmentally friendly ▪ Easy to construct ▪ No need for 	<ul style="list-style-type: none"> ▪ Require pile driving equipment ▪ May require anchors to support 	<ul style="list-style-type: none"> ▪ Medium to High 	<ul style="list-style-type: none"> ▪ Risk of frictional pile may not develop full capacity consequently risk of

Options	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
precast decking		dewatering and cofferdam <ul style="list-style-type: none"> serve as dual purpose of support culvert foundation and retaining backfill reduce construction period 	possible later movement <ul style="list-style-type: none"> Durability issue with sheet pile walls 		potential settlement <ul style="list-style-type: none"> May be limited steel sheet pile sections

Table 2.1 compares the structure options from a foundations design and constructability perspective. Although the foundation soils are generally good and will provide adequate support for all options listed in the table, the use of rigid frame box culvert is recommended.

2.3.1 Shallow Foundations

2.3.1.1 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)	Assumed Footing Size (m)	Founding Soil Type	Factored Geotechnical Resistance at ULS (kPa)	Geotechnical Reaction at SLS** (kPa)
Rigid frame box culvert	~373 or below	3.5	Minimum 300 mm compacted granular material (Granular A or Granular B Type II) over native loose silt	250	150
Rigid frame open footing concrete culvert and corrugated steel plate culvert	~370.6*	1.0	Native loose to compact silt	200	140

Notes:

*Below the frost line, based on BH-M1

** for maximum settlement of 25 mm

It is presumed that underlying organic silt and any other soft or very loose materials are to be replaced with clean and compactable soil such as Granular A or Granular B Type II. Given that no significant grade raise is planned, the anticipated maximum total settlements for the 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.

2.3.1.2 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 Granular A pad and pre-cast concrete	Coefficient of friction ($\tan \delta$)=0.5
Between cast-in-place concrete and native silt	Coefficient of friction ($\tan \delta$)=0.3

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

2.3.1.3 Frost Protection

The frost depth in the area of the culvert is estimated to be approximately 2.4 m in accordance with OPSD 3090.100. During construction of any temporary and permanent support system using shallow foundations should be provided a minimum 2.4 m of soil cover or equivalent frost protection should be provided using thermal insulation.

2.3.2 Steel Sheet Piles

Sheet piles can be used for retaining backfill soil during excavation, as well as bearing elements to support culvert foundations for the option culvert replacement with steel sheet pile abutments and precast concrete decking. For design, a PZ-22 section can be considered.

2.3.2.1 Axial Resistance in Compression

The factored resistance values (per metre width of sheet pile) for the sheet piles have been calculated as 100 kN for 10 m embedment. This value is based on a static analysis, considering skin friction only (end bearing resistance is negligible), using the effective stress β method, similar to the steel piles described above. It is noted that, since the sheet piles will also be retaining the approach fills, only the embedded, outside portion of the sheet piles below the level of the creek bed is considered to contribute to axial resistance. The elastic compression at ULS should be less than 6 mm in all cases. Since there is no (or minimal) proposed grade raise, negative skin friction or drag loads are not a concern.

2.3.2.2 Lateral Resistance

For relatively short (typically less than 3 m to 4 m) abutments, a cantilever sheet pile design using the earth pressure coefficients and soil parameters provided in Section 2.4, following. Note that if this design

is implemented, the precast concrete deck will likely be designed to be installed such that lateral support is provided at the top of the sheet piles.

Depending on the abutment height and steel sheet section used, additional anchorage or tiebacks may be required. Conventional practice is to incorporate either buried deadman anchors or grouted soil anchors.

Deadman anchors can be designed based on the earth pressure coefficients and soil parameters provided in Section 2.4, following. 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 designed in a preliminary fashion in accordance with Section 26 of the CFEM (2006). Based on the generally stiff soils at this site, the estimated factored (0.4) ULS resistance of grouted anchors would be 12 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.

2.4 Lateral Earth Pressure

Culvert walls at the outlet and inlet, and temporary shoring that may be required for excavation should be designed to resist lateral earth pressure. The expression for calculating lateral earth pressure is given by:

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

where P = earth pressure intensity at depth h , kPa
 K = earth pressure coefficient
 γ = unit weight of retained soil, kN/m³
 q = surcharge near wall, kPa
 h = depth to point of interest, m

The above expression does not take into account hydrostatic pressure, which must be included for the groundwater levels measured on the site. Table 2.4 lists earth pressure parameters for given materials.

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 culvert 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)

Table 2.4 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_0)	Unit Weight γ kN/m ³
Sand to Silty Sand Fill	35	0.27	3.69	0.43	21
Organic Silt	27	0.37	2.66	0.55	16
Clayey Silt	28	0.36	2.77	0.53	19
Silt	29	0.35	2.88	0.51	20

2.5 Construction Alternatives

For the proposed culvert replacement the following methods were considered as possible alternatives for the new culvert installation at this site:

1. Full road closure followed by open cut/unsupported excavation to replace culvert
2. Construct temporary detour embankments at the site followed by open cut/unsupported excavation to expose and replace culvert
3. Half-and-half construction using roadway protection to allow excavation as maintaining signalized one lane of traffic on the existing embankment during construction. The following three options of excavation and replacement using the half-and-half approach were considered:
 - A. Construction using roadway protection and unsupported excavation of cut sides
 - B. Construction using roadway protection and braced cut sides
 - C. Construction using roadway protection and steel sheet pile abutments with precast concrete deck system

All methods considered utilize a cut and cover approach for culvert replacement which allow complete removal of the existing culvert, but it requires disruption of traffic. In contrast, a trenchless approach for culvert replacement does not require disruption of traffic. However, considering the size and nature of the existing culvert and topography of the surrounding terrain, tunneling for trenchless replacement of this culvert was not considered as an applicable option. The other trenchless methods such as pipe bursting, pipe splitting, pipe swallowing and interior replacement methods were also not considered as applicable in this project, since the size of the host pipe classify this culvert as an unsuitable candidate for these techniques. For all approaches provision must be made to maintain surface water flow to the outlet.

The following Table 2.5 summarizes advantages and disadvantages of considered construction alternatives. The table also shows assessed risk/consequences and relative costs of the considered methods. Schematic diagrams of considered alternatives are attached in Appendix H.

Table 2.5 Construction alternatives for culvert replacement (see schematic sketches in Appendix H)

Installation Method	Advantages	Disadvantages	Relative Cost	Ranking
OPTION 1 Full Road Closure using Existing Roadways and Open Cut Unsupported Excavation	<ul style="list-style-type: none"> • No construction of detour roads or roadway protection required • No excavation support required • Install entire new culvert at once • Straightforward construction • Short mobilization time • Low capital investment; cost saving in time and materials required for construction 	<ul style="list-style-type: none"> • Traffic interruption • Long detour around site using other existing roads required • Large amount of soil to be excavated • Existing fills and native soils require 2H:1V side slopes to maintain stability • Erosion control of temporary cuts required • Need to temporarily control creek water • Potential claims to compensate vehicle occupants and local business for delays or time lost due to detour routes • Risk of cost overrun and inability to finish job: low 	Relatively less expensive than other methods due to cost savings in time and materials required for construction, but potential claims to compensate vehicle occupants and local business for delays or time lost due to detour routes	2
OPTION 2 Temporary Local Detour and Open Cut Unsupported Excavation	<ul style="list-style-type: none"> • Traffic flow maintained at the site during construction • Simple detour roads can be constructed • Existing culvert will be completely removed and replaced with new culvert • No excavation support required • Install entire new culvert at once 	<ul style="list-style-type: none"> • Traffic interruption • Construction of detour embankments required at east or west side of highway • Difficulties to construct detours due to inaccessible surrounding terrain • Increased time for construction of detour • Large amount of soil to be excavated • Erosion control of temporary cuts required • Need to temporarily control creek water • Possible settlement due to new earth embankment fill 	More expensive than full road closure due to high costs to build local detours	5

Installation Method	Advantages	Disadvantages	Relative Cost	Ranking
		<ul style="list-style-type: none"> • Risk of cost overrun and inability to finish job: low to moderate • Possible extra cost to purchase of private property 		
OPTION 3.A Half-and-half Construction with Unsupported Cut Sides	<ul style="list-style-type: none"> • Traffic flow maintained at the site during construction • Short mobilization time • Straight forward construction and construction procedures 	<ul style="list-style-type: none"> • Traffic interruption • Roadway protection of up to 3.0 m high required to maintain one lane of traffic • High cost of roadway protection system • Large amount of soil to be excavated • Need to decommission the shoring system • Need to temporarily control creek water • Risk of cost overrun and instability to finish job: low to moderate 	Relatively more expensive than full road closure due to high costs of roadway protection system	3
OPTION 3.B Half-and-half Construction with Braced or Anchored Cut Sides	<ul style="list-style-type: none"> • One or possibly two lanes of traffic flow maintained on existing road (e.g. steel decking, but costly) • Global stability of excavation enhanced by narrow geometry • Less traffic interruption than with unsupported cut sides approach • Temporary decking could be usable over braced cut to allow for excavation of both halves prior to diverting stream and backfilling • Cost savings due to limited excavation and backfill 	<ul style="list-style-type: none"> • Traffic interruption • Roadway protection of up to 3.0 m high required to maintain one lane of traffic if steel decking is not possible • High cost of roadway protection system and/or decking • Require side shoring and bracing • Bracing (e.g. struts) may interfere with excavation • Excavation of material and placement of bracing required in limited space • Need to decommission the shoring system 	More expensive than full road closure and other open cut sides approach due to high costs for shoring system and temporary decking (if feasible) to maintain continuous flow of traffic	4

Installation Method	Advantages	Disadvantages	Relative Cost	Ranking
		<ul style="list-style-type: none"> • Need to temporarily control creek water • Risk of cost overrun and instability to finish job: low to moderate 		
OPTION 3.C Half-and-half Construction with Installation of Steel Sheet Pile Abutments with Precast Concrete Deck	<ul style="list-style-type: none"> • Environmentally friendly • Easy to construct • No need for dewatering and cofferdam • No need for detour • No need to redirect existing creek water • No need for decommissioning of shoring system • Cost effective 	<ul style="list-style-type: none"> • Traffic interruption • Roadway protection required to maintain one lane of traffic • High cost of roadway protection system • Relatively new approach for MTO • Due to possible lateral movement need an anchor system, bracing or deadman • Durability issue with sheet pile walls • Some difficulty in excavating under concrete span • Risk of cost overrun and inability to finish job: low to moderate 	Relatively more expensive than full road closer due to high costs of shoring abutments, but more practical	1

Based on the above list of advantages and disadvantages of the possible construction methods, we recommend the following ranking of the considered options:

1. OPTION 3.C: Half-and-half construction with the steel sheet pile abutments and precast concrete deck (Figure H3.C, Appendix H)
2. OPTION 1: Full road closure using existing roadways and open cut unsupported excavation (Figure H1, Appendix H)
3. OPTION 3.A: Half-and-half construction with unsupported cut sides (Figure H3.A, Appendix H)
4. OPTION 3.B: Half-and-half construction with braced or anchored cut sides (Figure H3.B, Appendix H)
5. OPTION 2: Temporary local detour and open cut unsupported excavation (Figure H2, Appendix H)

The following sections discuss these options in more details.

2.5.1 Detour Options (Options 1 and 2)

Both detour options, the option with full closure of Hwy 603 and long detours around the area using existing roadways (see Figure H1, Appendix H), and the option with the local detour embankment construction at the site to maintain the local flow of traffic during the replacement (see Figure H2, Appendix H), allow for open cut, unsupported excavation to facilitate the replacement of the existing culverts. The major advantage is that neither excavation support nor roadway protection is required with these options. The major disadvantages of both options are traffic interruption, large amounts of excavated soils and need for temporary construction unwatering and dewatering systems (i.e. cofferdams, and sumps and pumps, etc.) to prevent creek water and groundwater flow into the construction area which is the responsibility of the contractor.

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. sand with silt and gravel fill) may be classified as a Type 3 soil above the groundwater table in conformance with the OHSA. The native soils below the groundwater table may be classified as a Type 4 soil. 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 (i.e. to maintain a global factor of safety greater than 1.3) where excavation will be left open for some time. Temporary

excavation side slopes for Type 4 soils should not exceed 3H:1V where applicable. There is a potential for sloughing to occur if the trench remains open for an extended period of time (i.e. > 24 hours) or during a rainfall event. In addition, some localized surficial sloughing may be experienced in areas of perched groundwater seepage (i.e. within the embankment fill).

The detour construction alternative would involve construction of a temporary on-site embankment at the one side of the existing embankment depending on the available space and suitable terrain. Compacted engineered fill for construction of the temporary detour road is recommended. Prior to construction of the temporary detour embankment, the site will need to be cleared and grubbed of any existing bushes and vegetation. All surficial topsoil (if exists), organics and softened or loosened soil should be stripped from below the proposed temporary detour road embankment. All subgrade soils should be proof-rolled prior to fill placement and embankment fill should be placed in accordance with OPSS. PROV 206 (dated November 2014).

2.5.2 Half-and-Half Construction (Options 3)

If a long detour using existing roadways is not available and acceptable, the half-and-half construction method should be utilized (see Figures H.3.A, H.3.B and H.3.C, Appendix H). In that method one lane of the existing highway will be used to maintain the local traffic while the other half of the existing highway will be excavated and the half of the existing culvert will be exposed. Then that portion of the existing culvert will be removed and replaced with a new culvert (or culverts), followed by rebuilding of that half of the embankment to grade. Upon completion of the new embankment, the traffic will be moved onto the new fill and the process will be repeated to complete the construction and culvert replacement.

The temporary excavation required to remove half of the existing embankment would be up to 3.4 m deep. Therefore, temporary shoring such as a soldier pile and lagging or sheet pile system will be required as a roadway protection system to allow staging excavation/construction. It will be the Contractors responsibility to design a suitable temporary support system for the MTO review prior to installation. The Contractor is to follow OPSS 902, regarding excavations for structures, and OPSS PROV. 539, regarding temporary protection systems. Recommendations for a temporary roadway protection are given in Section 2.6. Using the half-and-half construction approach, several methods of culvert replacement were considered as discussed below:

- A. Construction using roadway protection and unsupported excavation of cut sides
- B. Construction using roadway protection and braced or anchored cut sides
- C. Construction using roadway protection and a steel sheet pile abutments with precast concrete deck system

Option 3.A could be more economical due to possible cost savings for reversible wall configuration, but it will be more disruptive to the highway embankment. Option 3.B will disrupt less of the embankment but would cost more, i.e. about 1.8 times of Option 3.A. Excavation and backfilling operations will also be more challenging with Option 3.B. Both options require decommissioning of shoring system upon completion of the work.

2.5.1.1 Option 3.A: Half-and-Half Construction with Unsupported Cut Sides

This method provides roadway protection parallel to the highway between two lanes, and allows to divert traffic to the one side and undertake open cut with sloping sides at the other side (see Figure H3.A, Appendix H). The roadway protection can take the form of reversible shoring such as a soldier pile and lagging or sheet pile with rakers or anchors for horizontal support. Where the cut extends below prevailing groundwater a suitable control/system is required. Once one lane is completed the supports can be reversed and the other lane constructed in similar fashion. The shoring system would likely be decommissioned in place. Temporary surface water flow control must be developed by contractor.

Option 3.A could be more economical due to possible cost savings for reversible wall configuration, but it will be more disruptive to the highway embankment than Options 3.B and 3.C since it needs to excavate a large amount of soil.

2.5.1.2 Option 3.B: Half-and-Half Construction with Braced or Anchored Cut Sides

This method provides braced or anchored cut shoring system perpendicular to the highway for face protection and to allow culvert construction (see Figure H3.B., Appendix H). Excavation in this case would have to accommodate the necessary cross-bracing such as struts. With this option, consideration would have been given to how the new culvert sections will be installed given the relatively narrow work area and potential for obstructions from the lateral bracing using struts. Installation of tiebacks could be the solution. Temporary decking could possibly be used over the supported cut to allow for excavation of both halves prior to diverting stream and backfilling. However decking would be costly. As well as Option 3.A, decommissioning of the shoring system and temporary surface water flow control must be performed/developed by contractor.

Option 3.B will disrupt less of the embankment than Option 3.A but would cost more, i.e. about 1.8 times of Option 3.A, due to the cost of shoring system. Excavation and backfilling operations will also be more challenging with Option 3.B. Both options require decommissioning of shoring system upon completion of the work.

2.5.1.3 Option 3.C: Half-and-Half Construction using Steel Sheet Pile Abutments with Precast Concrete Deck

This option provides shoring system consisting of sheet piles perpendicular to the highway, which will serve the dual purpose of retaining backfill soil during excavation and being bearing elements to support culvert foundations after excavation (see Figure H3.C, Appendix H). As shown on Figure H3.C, the sheet piles will be installed perpendicularly in the half of the embankment at both sides of the existing culvert after installation of the roadway protection system for Stage 1 construction. Next the fill will be excavated to the designed elevation of the deck and its precast panels will be installed over the existing culverts. Then the fill below the deck panels will be excavated within construction limits for Stage 1 allowing the existing culverts to be removed. The excavation above the deck will be backfilled with a free-draining granular material up to the highway grade. The same processes will be repeated in Stage 2 construction, on the other side of the roadway protection. The contractor should be responsible for the complete design, construction and monitoring of the

described system. It is their responsibility to provide the work and design that should accommodate all relevant conditions including local and global stability for all stages of installation, including any necessary groundwater or surface water controls.

A major benefit of this method is that sheet piles will be permanently installed creating abutments for the precast concrete deck, so decommissioning of the shoring system is eliminated. Further, this method allows the use of the existing culvert to convey the creek water flow below Hwy 603 even during the construction work. However, due to the depth of the fill, some lateral movement of the abutments might be possible and installation of anchor systems, bracing or deadman might be required.

2.6 Temporary Roadway Protection

Temporary roadway protection is anticipated to be a part of the half-and-half construction approach that will be required to maintain on-site traffic during the construction. It is recommended that roadway protection system be 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 and/or utility may be impaired by construction work.

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.

2.7 Culvert Bedding

OPSDs 802.010, 802.031, 802.032, 803.010 and Figure C6.20 of (CHBDC) or OPSD 3101.150 which are included in Appendix G provide the bedding, embedment, cover and backfill standards for the different pipe material. According to these standards the culvert bedding should consist of Granular A or Granular B Type II (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 to at least 95% of the Standard Proctor Maximum Dry Density (SPMDD) before a subsequent layer is placed in accordance with OPSS 514. Bedding material placed in the haunches must be compacted prior to continued placement of cover material. Bedding on each side of the pipe shall be completed simultaneously. At no time shall the levels on

each side differ more than the 200 mm uncompacted layers.

Prior to placing any fill material, the exposed native subgrade should be inspected according to OPSS 902. 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 silt and clayey silt. This material has high 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 need to be provided to the limit of frost penetration beneath the inlet and outlet of the culvert. However, considering that cold air blowing through the culvert during the winter season will freeze soil next to the culvert, a minimum 500 mm thick layer of non-susceptible material should be considered to be placed as a bedding along the entire culvert length.

2.8 Culvert Backfill

Backfill should be placed from the base of the culvert to the full height of the culvert and extend a minimum 1.3 m horizontal distance from the outside wall (as per Figure C6.20a of the CHBDC). This horizontal distance may be reduced by the use of suitable insulation (such as a heavy duty STYROFOAM). The insulation should be placed against the outside wall of the culvert from the base of the culvert to its total height. The material should be installed as per the manufacturer's instructions.

The backfill should consist of free-draining, non-frost susceptible granular materials such as Granular A or Granular B (OPSS.PROV 1010).

All granular backfill materials should be placed in thin lifts (i.e. not exceeding 300 mm before compaction) and each lift should be compacted to at least 95% of the material's SPMDD (Standard Proctor Maximum Dry Density). The final lift of embankment fill 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.

For fills immediately below any roadway, it is recommended that Granular A or B aggregates be used. Where necessary, proper tapering as per standards should be provided. Below a depth of about 1.5 m from any finished road grade, approved compactable fill, such as select subgrade materials (SSM) can be used.

Where less than 1.3 m of earth cover is provided above the top of the culvert, a frost taper should be included as per 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 Surface Water and Groundwater Control

Cofferdams might be required at both upstream and downstream ends to envelop the construction site and keep it free of water during culvert installation. The investigation revealed that the subsurface conditions at the location of cofferdams at inlet side consist of a layer of very loose to loose organic silt (~0.8 m thick) underlain by loose silt (~9.7 m thick; borehole terminated). At the outlet side, the subsurface conditions at the location of cofferdam consist of a layer of very loose organic silt (~1.2 m thick) underlain by firm to stiff clayey silt (~7.6 m thick), followed by loose silt. The practical refusal is not encountered at both sides of the existing embankment.

Based on these geotechnical conditions, suitably designed steel sheet pile walls can be used as cofferdams at this site. Sheet piles perpendicular to the highway at least 3 m into the embankment slopes should be considered to prevent water getting in through the sides. 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. The proposed sheet pile wall should be at least one meter above 100 year flood. The required minimum section modulus and embedment pile length should be designed based on the recommended design parameters.

Alternatively, a rockfill cofferdam can be used. This cofferdam will have to be constructed to the same topographic constraints as the sheet pile cofferdam. 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 2.6. Given the soil conditions, topography of the surrounding terrain and available space, the use of a suitably designed steel sheet pile system is recommended for the inlets or outlets of these locations. The design of these cofferdams, which are temporary retaining structures is the responsibility of the Contractor. The cofferdam must be designed to withstand the anticipated design loads and to be watertight as practically possible. The Contractor is also responsible for cofferdam's materials, construction, monitoring and removal. Cofferdams should be designed in accordance with OPSS 539 by a licensed Professional Engineer experienced in shoring design. If sheet piles are employed, piling shall be according to OPSS 903.

Table 2.6 Comparison of cofferdam systems

Option	Advantages	Disadvantages	Relative Cost	Risk/Consequence
Steel sheet piles	<ul style="list-style-type: none"> • Provides more watertight base • Structural elements and seals easier to positively construct • Increased safety with appropriate design • Easily removed • Less seepage • Reusable 	<ul style="list-style-type: none"> • More costly • More likely time consuming for installation • May present issues for seepage and/or piping • Larger machines required • May require bracing 	MEDIUM TO HIGH	<ul style="list-style-type: none"> • Possible piping problem • May take longer to install • Environmental permits
Rockfill	<ul style="list-style-type: none"> • Less costly • Relatively less time consuming for installation • Native material can be usable 	<ul style="list-style-type: none"> • 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	<ul style="list-style-type: none"> • Less stable and safe. May generate 'mud waves' • May take longer to remove • May require to install clay cutoff • More dewatering • Environmental permits

The soils encountered below the groundwater table and within potential excavation depths consist of native silt and clayey silt. These soils are susceptible to disturbance from groundwater and mobilized equipment. 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, pumping using properly filtered sumps, and/or filtered drains placed along the base of the excavation should provide sufficient groundwater control during foundation works.

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

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.

2.10 Embankment Design

2.10.1 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.

2.10.2 Embankment Stability

A preliminary slope stability analysis was performed to assess the global stability of the existing embankment and to check that a minimum Factor of Safety of 1.3 will be achieved for the new embankment at the location of the proposed culvert. 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.

Stability assessments of existing slopes under static conditions were performed on the cross-section perpendicular to the highway at the proposed culvert location. The cross-section of the existing embankment with the approximate slope of 1H:1V to 2H:1V was established based on **exp's** survey data and the topographic plan 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 native silt and clayey silt deposits. Therefore, an effective stress analysis for a long term stability assessment of the embankment slope was performed taking into consideration the subsoil conditions encountered beneath the existing embankment. An organic silt layer was encountered in BH-M2, BH-M3 and BH-M4. However no organic silt layer was encountered in borehole M1; the organic layer beneath some portion of embankment slope was considered for completeness of soil layer boundary in slope stability model of east embankment slope.

The SLOPE/W graphical printouts, for analyses performed, are included in Appendix F. Tabulated below in Table 2.7 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 2.7 Soil properties used in slope stability analysis

Material	ϕ' (degrees)	c' (kPa)	γ' (kN/m ³)
Sand to Silty Sand Fill	35	0	21
Organic Silt (Very Loose to Loose)	27	0	16
Clayey Silt (Firm to Stiff)	28	0	19
Silt (Very Loose to Compact)	29	0	20
Engineered Fill	30	0	21

The results of slope stability analyses are attached as Appendix F. The results of stability analyses on the existing embankment shown on Figures F1 and F2 suggest that both slopes of the existing embankment could be on the verge of stability due to presence of the loose organic silt layer below the embankment. To achieve the factor of safety greater than 1.3 for the global stability of the new embankment with 2H:1V slopes, the loose organic silt layer has to be excavated and replaced by engineered fill, as confirmed by the analyses which results are shown in Figures F3 and F4 (Appendix F).

2.11 Inlet and Outlet

2.11.1 Erosion Protection at Outlet

The detailed design for erosion protection should be carried out by the 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 G of this report. Rip-rap placed at 1V:1H will be stable.

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.

2.11.2 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 G of this report.

2.11.3 Seepage Cut-off Requirements

The seepage cut-off requirements should be reviewed in the following context. The native sand with silt soil at the inlet and 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.

2.11.3.1 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 (OPSD 802.095, Appendix G). 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 1×10^{-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 $0.73 \times (\text{Liquid Limit} - 20\%)$.
- The clay seal is to be placed 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.

2.11.3.2 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.

2.12 Corrosion Protection

One (1) representative soils sample was submitted to a CALA Certified Laboratory for chemical corrosivity analysis. The samples were analyzed for chloride, sulphate, pH, electrical conductivity, resistivity, redox potential, and sulphide concentrations. The results of the corrosivity testing are summarized in section 1.6 of this report and detailed results included in Appendix E.

Similar to our experience with the soils in the area, the chemical data indicates low to medium resistivity, which indicates a potential for moderate corrosion of buried metallic elements. The maximum soluble chloride content reported is 10 µg/g (i.e. 0.001%), which indicates a low potential for additional corrosion.

The water soluble sulphate content of the soil tested is 15 µg/g (i.e. 0.0015%), so since it is less than 0.1% it does not indicate the potential to corrode normal Portland cement concrete.

2.13 Operational Constraints (OCs) and Non Standard Special Provisions (NSSPs)

In assembling contract documents, a number of OCs and NSSPs should be included to address some of the foundation/geotechnical issues that might be of concern during execution of the work. It is anticipated that the following list may apply based on current information:

- (1) NSSP for mass concrete on bedrock.
- (2) NSSP for sloping rock and cobble and rock piece obstructions.
- (3) NSSP for dowelling.
- (4) NSSP for condition surveys and monitoring during any blasting.

Appendix I presents draft of the suggested NSSPs.

These should be further assessed during planning and design development when actual approaches are more defined.

December 23, 2015

Part III: CLOSURE

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

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

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

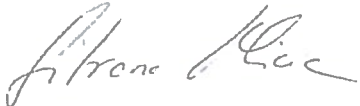
Contractors bidding on or undertaking any proposed work at this site should, relative to the subsurface conditions, decide on their own investigations, if deemed necessary, as well as their own interpretations of the factual results provided herein, so they may draw their own conclusions as to how the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

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

Yours truly,


exp Services Inc.

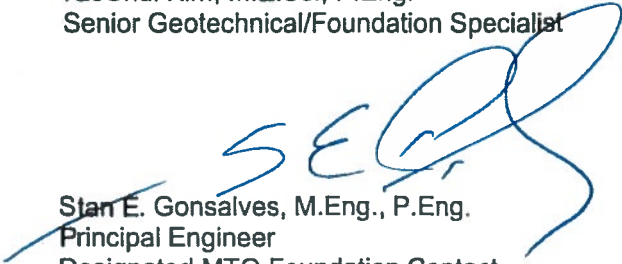

Nimesh Tamrakar, M.Eng., EIT.
Technical Specialist

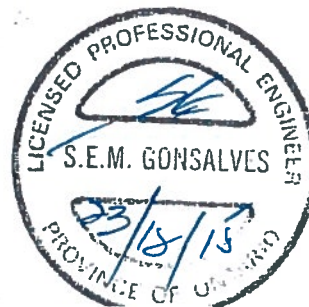

Silvana Micic, Ph.D., P.Eng.
Senior Geotechnical Engineer
Project Manager

Encl.




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


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



Part IV: 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. Facing east on inlet side of existing culvert



Photo 2. Facing west on outlet side of existing culvert



Photo 3. Embankment slope on inlet side facing northeast



Photo 4. Embankment slope on inlet side facing southeast

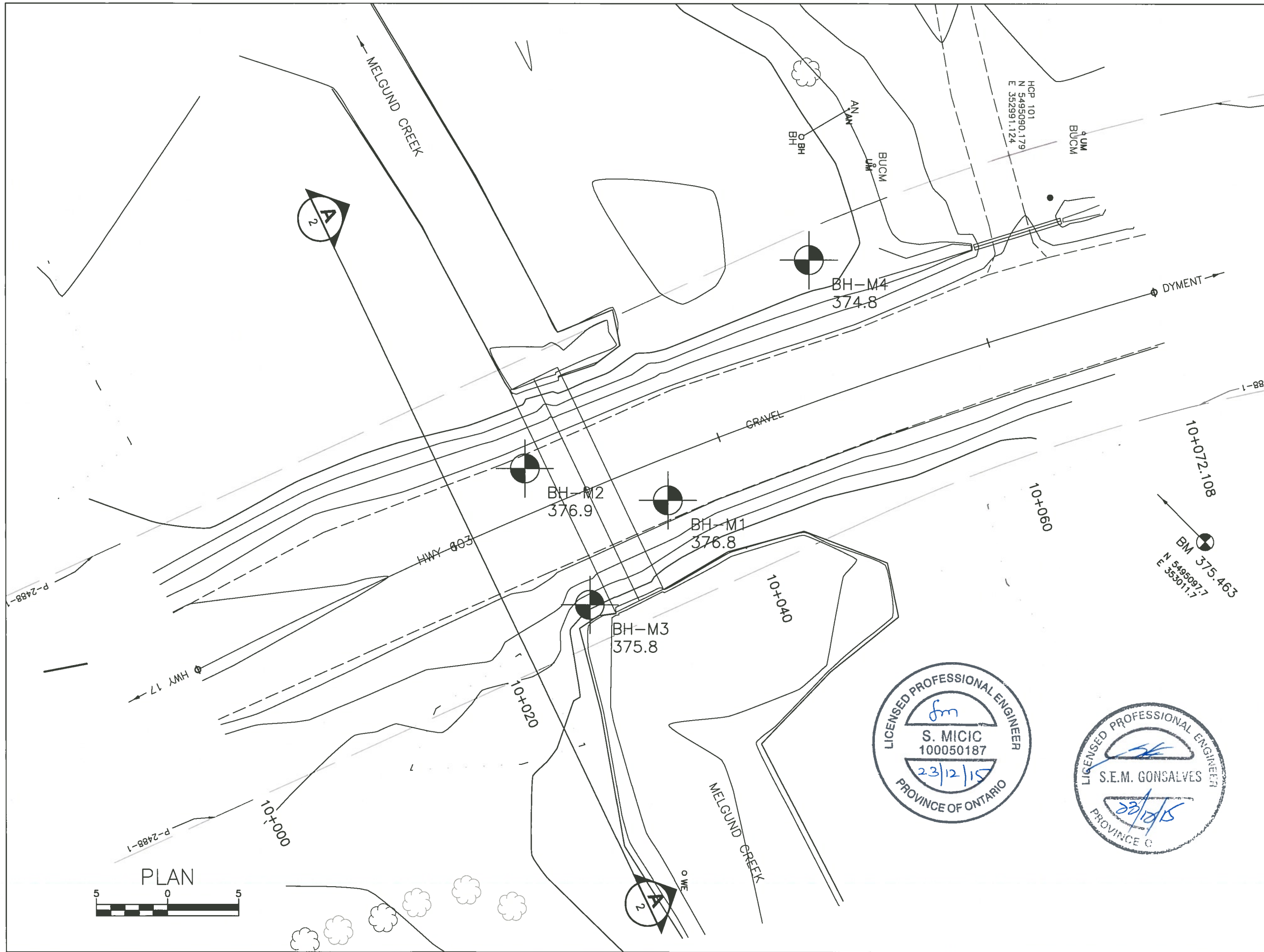


Photo 5. Embankment slope on outlet side facing southwest



Photo 6. Embankment slope on outlet side facing northwest

Appendix B – Drawings



Agreement No. 6014-E-0017
Assignment No. 8
GWP 6359-14-00

MELGUND CREEK CULVERT
(Highway 603, Township of Melgund)
PLAN

DWG
1

exp

exp Services Inc.

DISTRICT OF KENORA
TOWNSHIP OF AVERY

KEY PLAN
1.0 km 0 1.0 km

LEGEND

BOREHOLE LOCATION
GROUND SURFACE ELEVATION IN
METRES
BH-M1
376.8

BH No.	APPROX. ELEV. (m)	MTM COORDINATES	
		NORTH	EAST
BH-M1	376.8	5,495,063	353,012
BH-M2	376.9	5,495,053	353,010
BH-M3	375.8	5,495,058	353,019
BH-M4	374.8	5,495,073	352,995

NOTES

- ALL DIMENSIONS ARE IN METRES.
- BASE MAP PROVIDED BY CLIENT.
- THIS DRAWING IS FOR SUBSURFACE INFORMATION ONLY.
THE PROPOSED STRUCTURE DETAILS/WORKS ARE SHOWN
FOR ILLUSTRATION PURPOSES ONLY.

REVISIONS

	BY	DESCRIPTION

GEOCRES No.52F-045

Project No.ADM-00223648-GO

Date: Dec. 17, 2015

Scale : 1:250

Drawn By: IM

Checked By: IM

Agreement No. 6014-E-0017
Assignment No. 8
GWP 6359-14-00

MELGUND CREEK CULVERT
(Highway 603, Township of Melgund)
CROSS SECTION

DWG
2

exp

exp Services Inc.

DISTRICT OF KENORA
TOWNSHIP OF AVERY
MELGUND LAKE
Hwy 603
NATURAL GAS PIPE
SILT
Hwy 603
TOWNSHIP OF MELGUND
TOWNSHIP OF PREVELL

KEY PLAN
1.0 km 0 1.0 km

LEGEND

N STANDARD PENETRATION TEST
(BLOWS/300 mm)

MEASURED WATER LEVEL

BH No.	APPROX. ELEV. (m)	MTM COORDINATES	
		NORTH	EAST
BH-M1	376.8	5,495,063	353,012
BH-M2	376.9	5,495,053	353,010
BH-M3	375.8	5,495,058	353,019
BH-M4	374.8	5,495,073	352,995

NOTES

1. ALL DIMENSIONS ARE IN METRES.
2. BASE MAP PROVIDED BY CLIENT.
3. THIS DRAWING IS FOR SUBSURFACE INFORMATION ONLY.
THE PROPOSED STRUCTURE DETAILS/WORKS ARE SHOWN
FOR ILLUSTRATION PURPOSES ONLY.

REVISIONS

BY	DESCRIPTION

GEOCRES No.52F-045

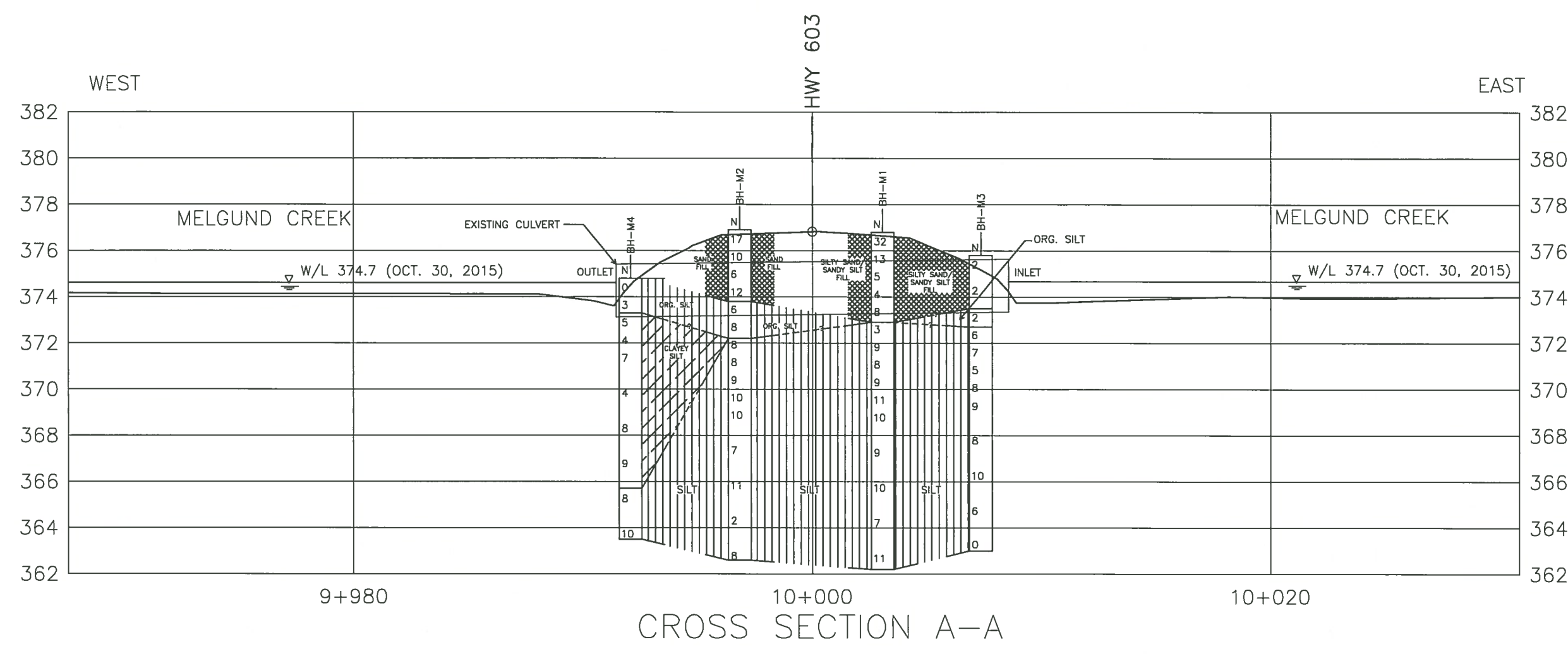
Project No.ADM-00223648-G0

Date: Dec. 17, 2015

Scale : 1:200

Drawn By: IM

Checked By: IM



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				FINE		MEDIUM	CRS.	FINE	COARSE		
SILT (NONPLASTIC)				SAND				GRAVEL			
UNIFIED SOIL CLASSIFICATION											

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

Table a: Percent or Proportion of Soil, Pp

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

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

Table b: Apparent Density of Cohesionless Soil

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

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

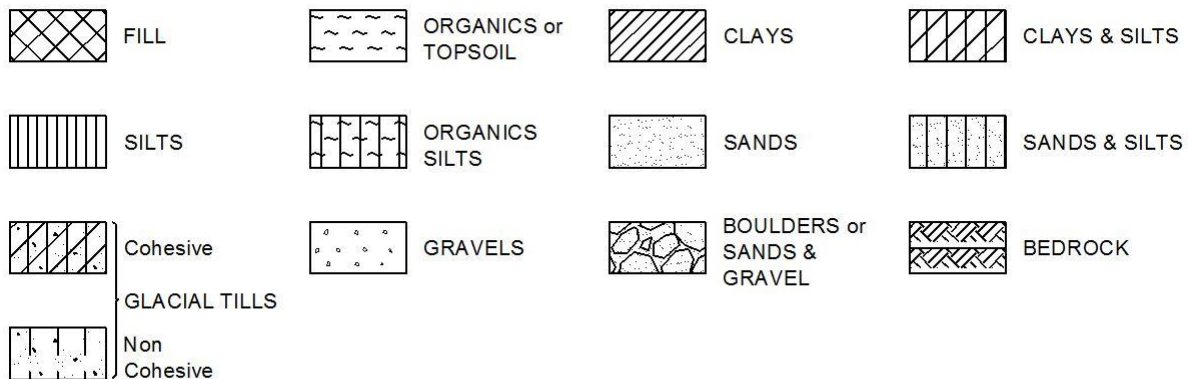
Table c: Consistency of Cohesive Soil

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

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

STRATA PLOT

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



WATER LEVEL MEASUREMENT



Open Borehole or Test Pit



Monitoring Well, Piezometer or Standpipe

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

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

STRESS AND STRAIN

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

MECHANICAL PROPERTIES OF SOIL

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

PHYSICAL PROPERTIES OF SOIL



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

RECORD OF BOREHOLE No BH-M1

SHEET 1 OF 2

METRIC

GWP No. 6359-14-00 LOCATION Melgund Crk. Culvert, (Site 41S-142/C), MTM-16, 5,495,058N, 353,013E ORIGINATED BY ST
DIST Kenora HWY 603 BOREHOLE TYPE CME 850, 200mm Dia. HSA, Dynamic Cone COMPILED BY KR
DATUM Geodetic DATE 19/10/2015 - 30/10/2015 CHECKED BY IM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION			PLASTIC LIMIT PL	NATURAL WATER CONTENT w	LIQUID LIMIT LL	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL									
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa											WATER CONTENT (%)					
								○ UNCONFINED		+ FIELD VANE									⊗ QUICK TRIAXIAL		× LAB VANE			
376.8								20	40	60	80	100												
376.8	SURFACE TREATMENT (~ 25 mm thick) FILL , silty sand, some gravel, brown, damp to moist, dense to compact. moist, loose below ~ 1.5 m depth. wet, very loose below ~ 2.3 m depth. loose below ~ 3.1 m depth.		1	SS	32		376														12	53	35	0
			2	SS	13																			
			3	SS	5			375																
			4	SS	4			374																
			5	SS	8			373																
372.9	SILT , trace clay, trace sand, grey, wet, very loose to loose.		6	SS	3		372																	
3.9			7	SS	9		371														0	3	88	9
			8	SS	8		370																	
			9	SS	9		369																	
			10	SS	11		368																	
			11	SS	10		367																	
			12	SS	9		366																	
			13	SS	10		365																	
			14	SS	7		364																	
			15	SS	11		363																	
362.2							362																	
14.6	Sampling Terminated - DCPT Commenced						361																	
							360																	
							359																	

Continued Next Page

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

RECORD OF BOREHOLE No BH-M1

SHEET 2 OF 2

METRIC

GWP No. 6359-14-00 LOCATION Melgund Crk. Culvert, (Site 41S-142/C), MTM-16, 5,495,058N, 353,013E ORIGINATED BY ST
 DIST Kenora HWY 603 BOREHOLE TYPE CME 850, 200mm Dia. HSA, Dynamic Cone COMPILED BY KR
 DATUM Geodetic DATE 19/10/2015 - 30/10/2015 CHECKED BY IM

SOIL PROFILE		SAMPLES				GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ⊗ QUICK TRIAXIAL × LAB VANE	PLASTIC LIMIT PL NATURAL WATER CONTENT w LIQUID LIMIT LL WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
345.1 31.7	END OF BOREHOLE										
	NOTES: 1. This drawing is to be read with the subject report and project numbers as presented above. 2. Interpretation assistance by exp is required before use by others.										

RECORD OF BOREHOLE No BH-M2

SHEET 1 OF 1

METRIC

GWP No. 6359-14-00 LOCATION Melgund Crk. Culvert, (Site 41S-142/C), MTM-16, 5,495,058N, 353,013E ORIGINATED BY ST
DIST Kenora HWY 603 BOREHOLE TYPE CME 850, 200mm Dia. HSA COMPILED BY KR
DATUM Geodetic DATE 20/10/2015 - 30/10/2015 CHECKED BY IM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION					PLASTIC LIMIT PL	NATURAL WATER CONTENT W	LIQUID LIMIT LL	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100					GR	SA	SI	CL
376.9																				
376.0	SURFACE TREATMENT (~ 25 mm thick) FILL , sand, some gravel, trace silt, brown, moist, compact to loose.		1	SS	17		376										12	80	8	0
			2	SS	10															
			3	SS	6		375													
	wet below ~ 2.3 m depth.		4	SS	12															
373.9							374													
3.1	ORGANIC SILT , trace sand, some wood, trace clay, dark brown, wet, loose.		5	SS	6															
			6	SS	8		373										0	1	92	7
372.3																				
4.7	SILT , trace clay, trace sand, grey, wet, loose to compact.		7	SS	8		372													
			8	SS	8															
							371													
			9	SS	9															
			10	SS	10		370										0	1	90	9
			11	SS	10		369													
			12	SS	7		368													
							367													
			13	SS	11		366										0	1	92	7
			14	SS	2		365													
							364													
			15	SS	8		363													
362.6																				
14.3	END OF BOREHOLE																			
	NOTES: 1. This drawing is to be read with the subject report and project numbers as presented above. 2. Interpretation assistance by exp is required before use by others.																			

RECORD OF BOREHOLE No BH-M3

SHEET 1 OF 1

METRIC

GWP No. 6359-14-00 LOCATION Melgund Crk. Culvert, (Site 41S-142/C), MTM-16, 5,495,058N, 353,013E ORIGINATED BY ST
DIST Kenora HWY 603 BOREHOLE TYPE CME 850, 200mm Dia. HSA COMPILED BY KR
DATUM Geodetic DATE 20/10/2015 - 30/10/2015 CHECKED BY IM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION					PLASTIC LIMIT PL	NATURAL WATER CONTENT W	LIQUID LIMIT LL	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100					
375.8																	
375.4	TOPSOIL (~ 76 mm thick)		1	SS	2		375										
	FILL, sandy silt, trace to some gravel, brown, moist to wet, very loose. 450 mm dia. boulder encountered at ~ 0.8 m depth.		2	SS	2		374										1 24 75 0
373.5							373										
2.3	ORGANIC SILT, some sand, some wood, dark brown, wet, very loose.		3	SS	2		372										
372.8							371										
3.1	SILT, some clay, trace sand, grey, wet, loose.		4	SS	6		370										0 4 78 18
			5	SS	7		369										
			6	SS	5		368										
	some sand below ~ 5.3 m depth.		7	SS	8		367										
			8	SS	9		366										
							365										
			9	SS	8		364										0 0 96 4
							363										
			10	SS	10												
			11	SS	6												
			12	SS	0												
363.0	END OF BOREHOLE																
12.8	NOTES: 1. This drawing is to be read with the subject report and project numbers as presented above. 2. Interpretation assistance by exp is required before use by others. 3. Groundwater level not considered to have stabilized in short term prior to backfilling.																

RECORD OF BOREHOLE No BH-M4

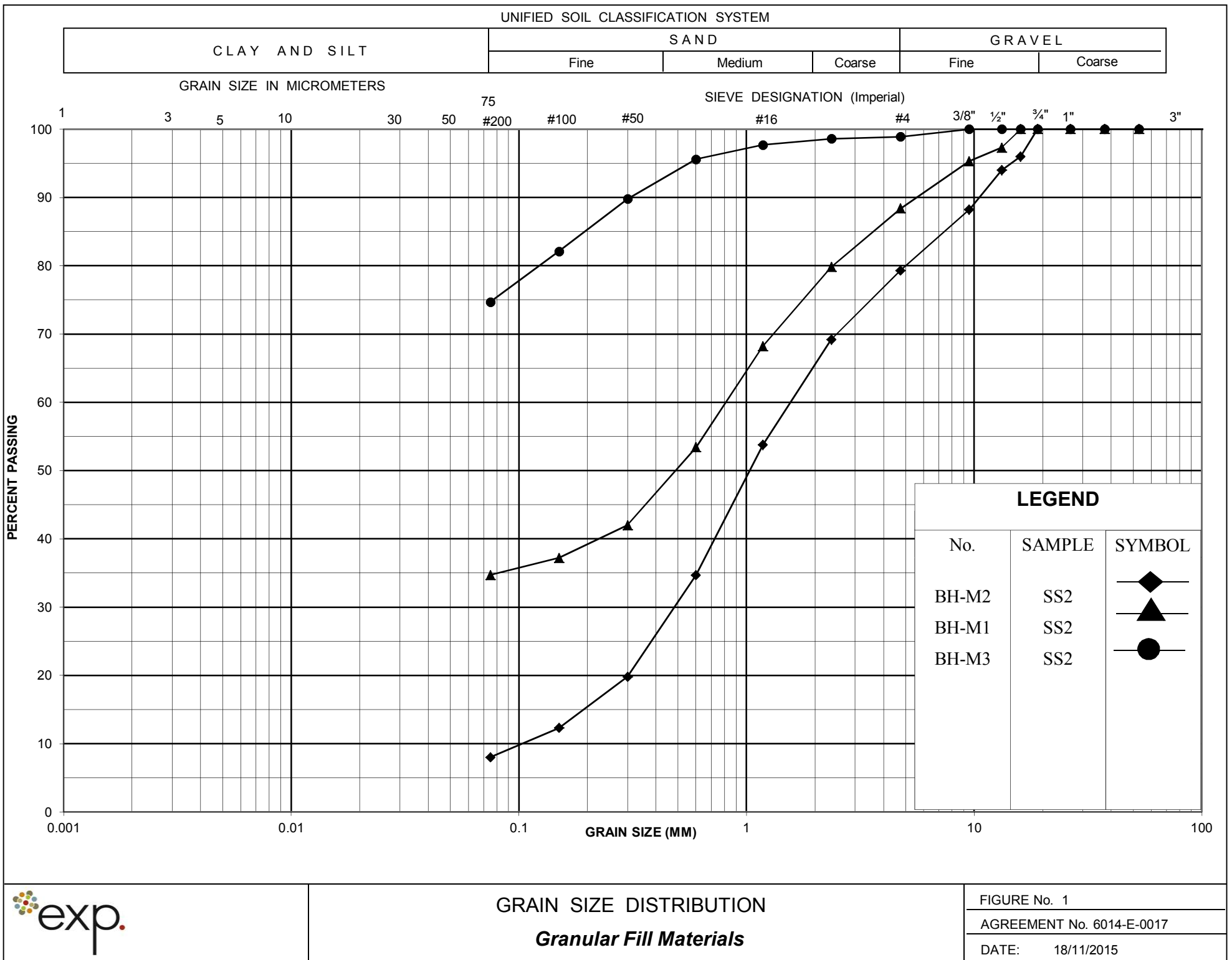
SHEET 1 OF 1

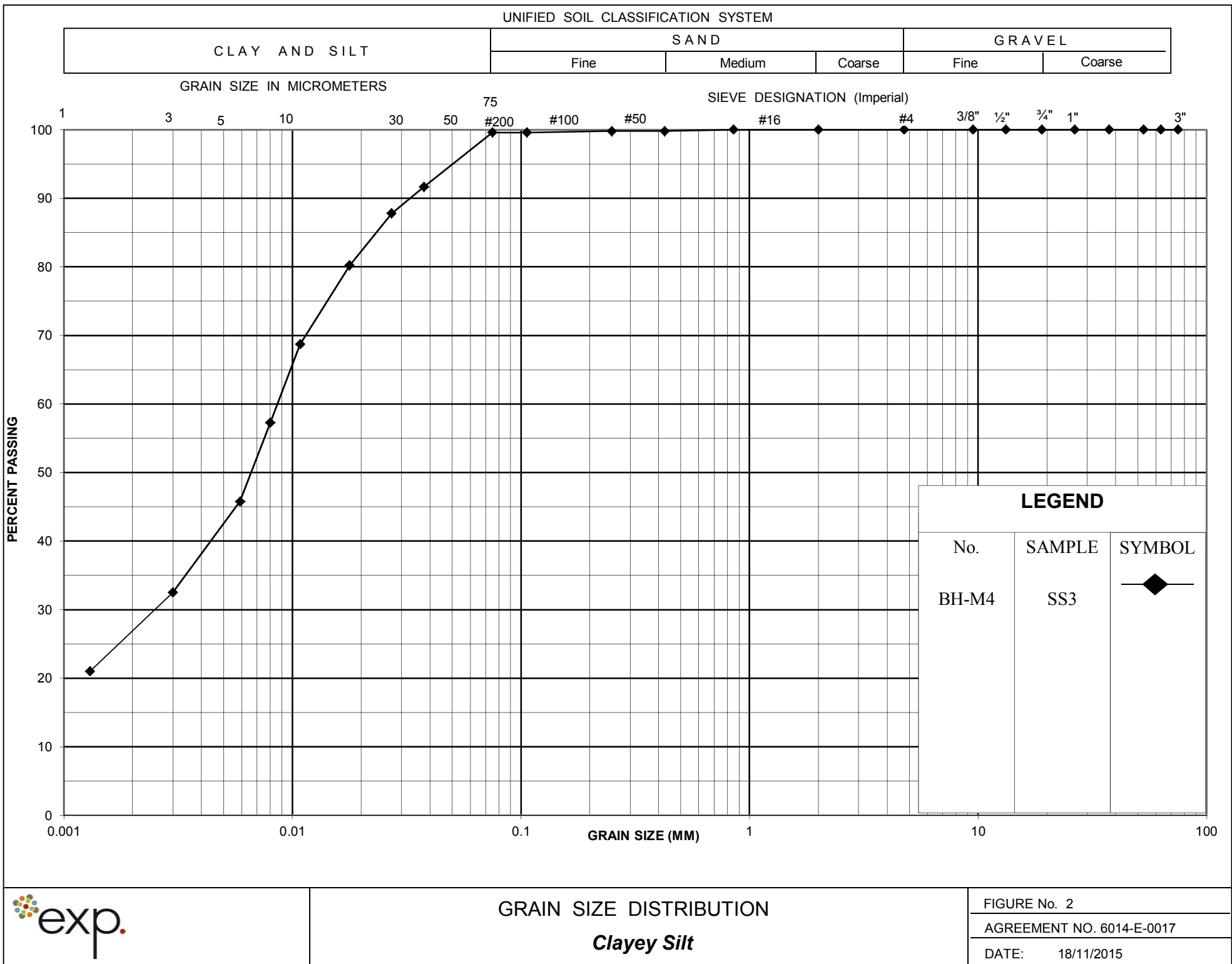
METRIC

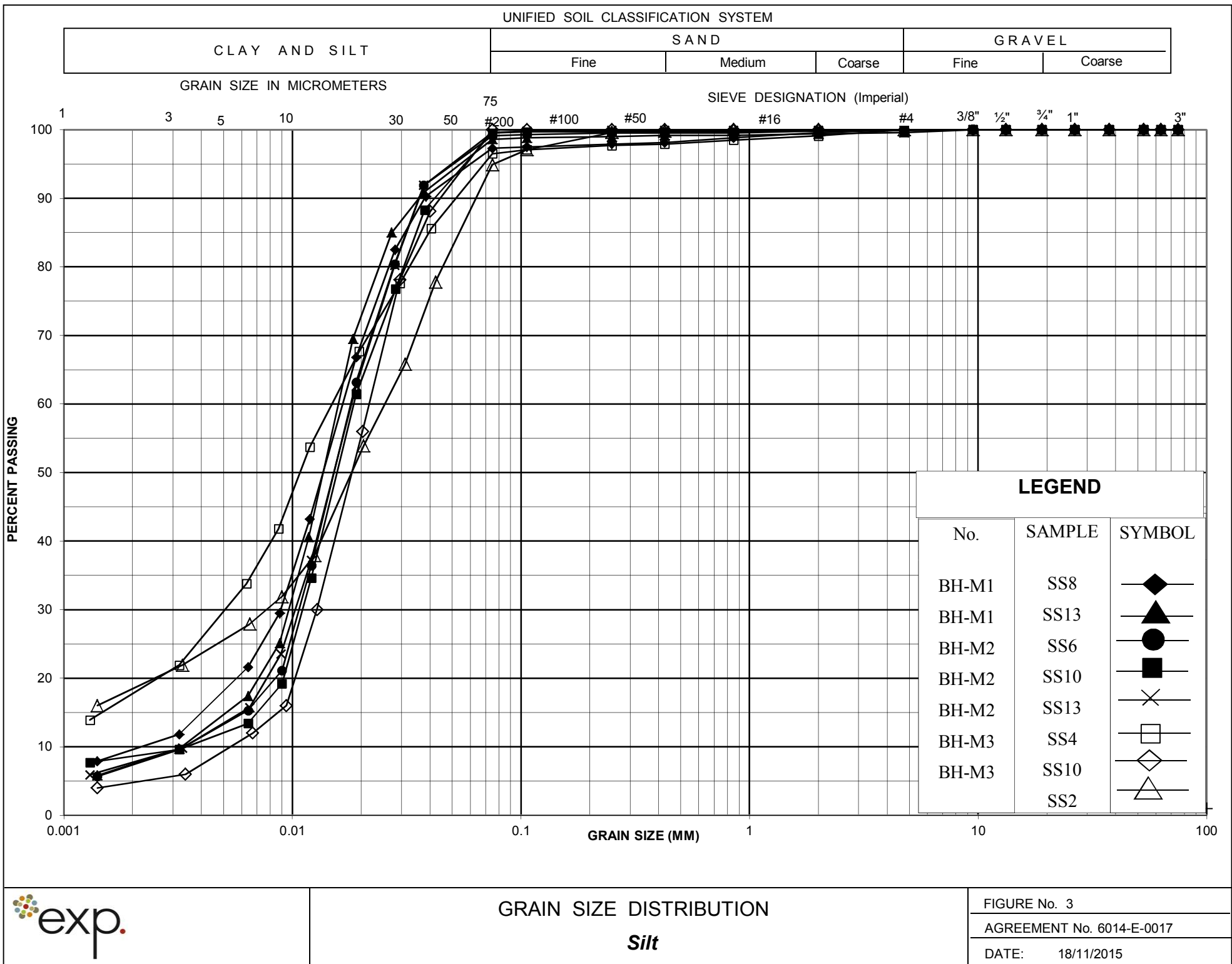
GWP No. 6359-14-00 LOCATION Melgund Crk. Culvert, (Site 41S-142/C), MTM-16, 5,495,058N, 353,013E ORIGINATED BY ST
DIST Kenora HWY 603 BOREHOLE TYPE CME 850, 200mm Dia. HSA COMPILED BY KR
DATUM Geodetic DATE 21/10/2015 - 30/10/2015 CHECKED BY IM

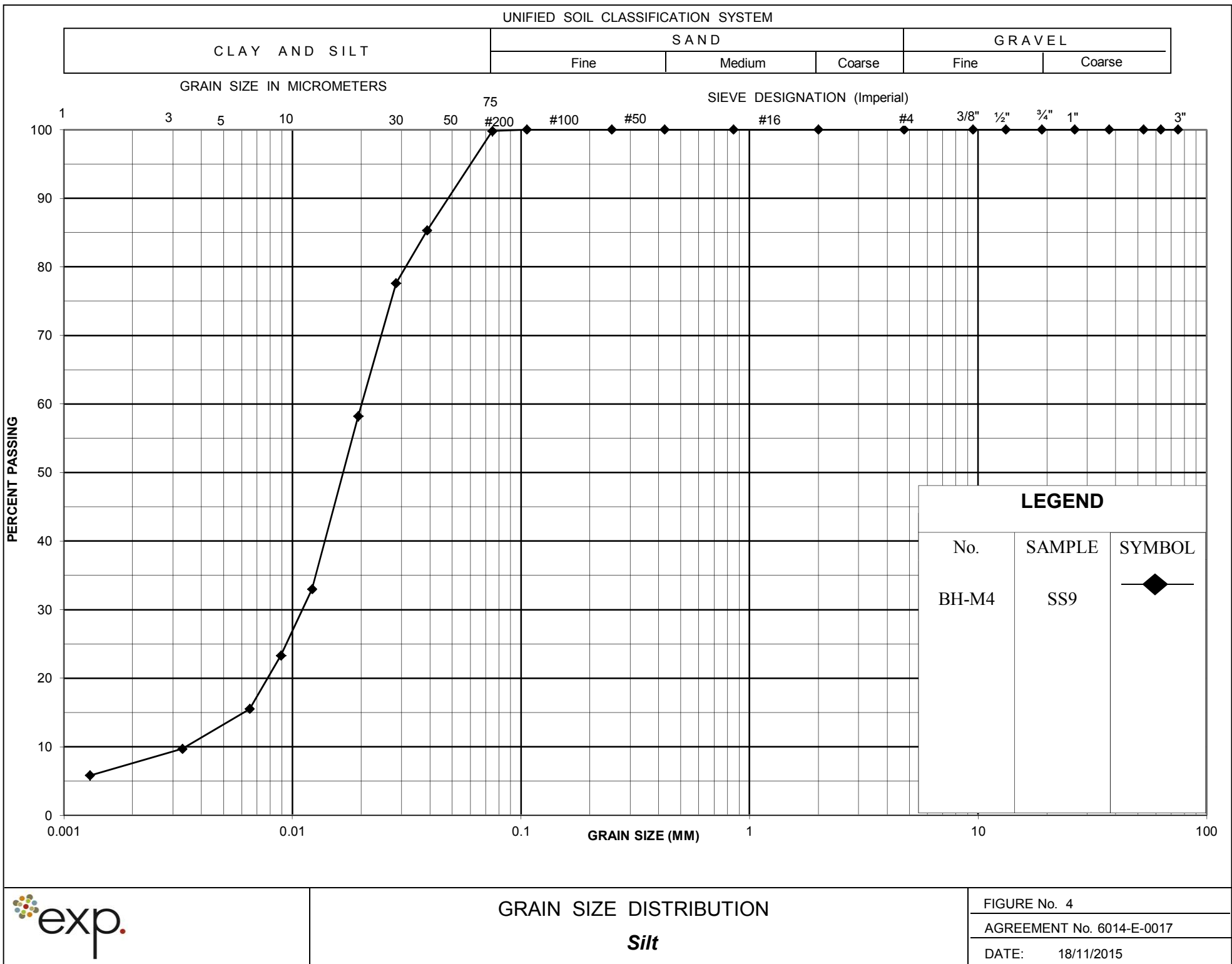
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION					PLASTIC LIMIT PL	NATURAL WATER CONTENT w	LIQUID LIMIT LL	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100					GR	SA	SI	CL
374.8																				
374.8	TOPSOIL (~ 30 mm thick)		1	SS	0		374										0	5	76	19
0.3	ORGANIC SILT, some clay, trace sand, dark brown to grey, wet, very loose, slight plasticity.		2	SS	3															
373.3							373										0	1	72	27
1.5	CLAYEY SILT, trace organics, trace sand, grey, wet, firm to stiff, slight plasticity.		3	SS	5															
			4	SS	4		372													
			5	SS	7															
							371													
			6	SS	4		370													
							369													
			7	SS	8		368													
			8	SS	9		367													
							366													
365.7							365													
9.1	SILT, trace sand, trace clay, grey, wet, loose.		9	SS	8												0	1	91	8
							364													
363.5			10	SS	10															
11.3	END OF BOREHOLE																			
	NOTES: 1. This drawing is to be read with the subject report and project numbers as presented above. 2. Interpretation assistance by exp is required before use by others.																			

Appendix D – Laboratory Data

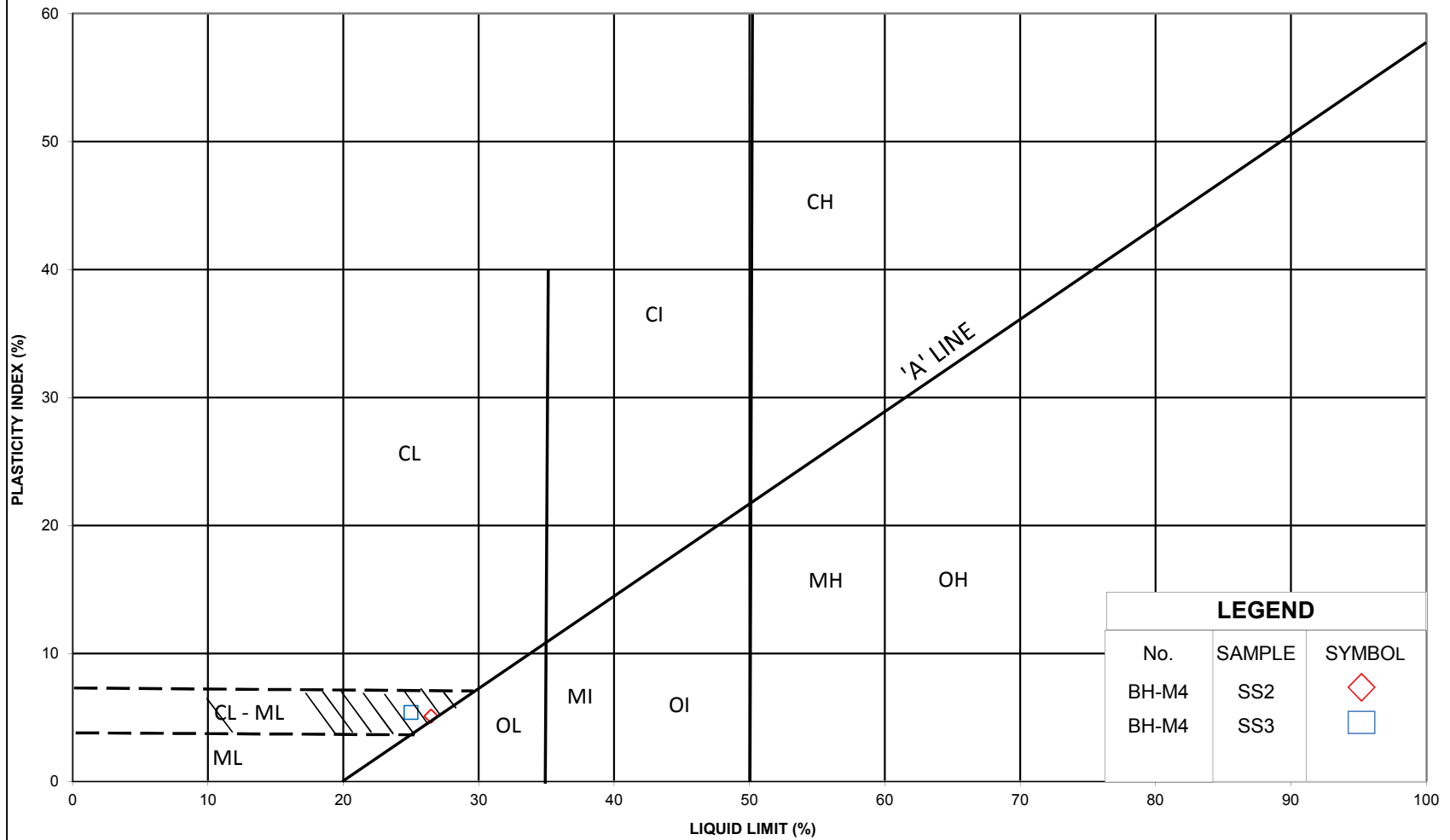








Melgund Creek Tributary Culvert, Hwy. 603, Melgund Township



Appendix E – Chemical Analyses

CLIENT NAME: EXP. SERVICES INC.
885 REGENT ST
SUDBURY, ON P3E5M4
(705) 674-9681

ATTENTION TO: Ian MacMillan

PROJECT: ADM-00223648

AGAT WORK ORDER: 15U040280

SOIL ANALYSIS REVIEWED BY: Amanjot Bhela, Inorganic Coordinator

DATE REPORTED: Nov 16, 2015

PAGES (INCLUDING COVER): 5

VERSION*: 1

Should you require any information regarding this analysis please contact your client services representative at (905) 712-5100

***NOTES**

All samples will be disposed of within 30 days following analysis. Please contact the lab if you require additional sample storage time.

**AGAT** Laboratories

Certificate of Analysis

AGAT WORK ORDER: 15U040280

PROJECT: ADM-00223648

5835 COOPERS AVENUE
MISSISSAUGA, ONTARIO
CANADA L4Z 1Y2
TEL (905)712-5100
FAX (905)712-5122
<http://www.agatlabs.com>

CLIENT NAME: EXP. SERVICES INC.

ATTENTION TO: Ian MacMillan

SAMPLING SITE:

SAMPLED BY:

Corrosivity Package

DATE RECEIVED: 2015-11-06

DATE REPORTED: 2015-11-16

BH-W4, SAMPLE SS9, BH-M1, Sample BH-A4, Sample						
SAMPLE DESCRIPTION:			22 1/2 - 24 1/2	SS7, 15-17 Ft	SS5, 10-12 Ft	
SAMPLE TYPE:			Soil	Soil	Soil	
DATE SAMPLED:			10/26/2015	10/19/2015	10/29/2015	
Parameter	Unit	G / S	RDL	7181054	7181063	7181065
Sulfide	%		0.01	0.03	0.02	0.03
Chloride (2:1)	µg/g	NA	2	174	10	66
Sulphate (2:1)	µg/g		2	5	15	12
pH (2:1)	pH Units		NA	8.00	8.91	7.10
Electrical Conductivity (2:1)	mS/cm	0.57	0.005	0.407	0.113	0.195
Resistivity (2:1)	ohm.cm		1	2460	8850	5130
Redox Potential (2:1)	mV		5	278	237	328

Comments: RDL - Reported Detection Limit; G / S - Guideline / Standard: Refers to Table 1: Full Depth Background Site Condition Standards - Soil - Residential/Parkland/Institutional/Industrial/Commercial/Community Property Use

7181054-7181065 * Sulphide analyses were performed at AGAT Laboratories Vancouver.

EC/Resistivity, pH, Chloride, Sulphate and Redox Potential were determined on the extract obtained from the 2:1 leaching procedure (2 parts DI water: 1 part soil).

Certified By:

Quality Assurance

CLIENT NAME: EXP. SERVICES INC.

PROJECT: ADM-00223648

SAMPLING SITE:

AGAT WORK ORDER: 15U040280

ATTENTION TO: Ian MacMillan

SAMPLED BY:

Soil Analysis

RPT Date: Nov 16, 2015			DUPLICATE			Method Blank	REFERENCE MATERIAL		METHOD BLANK SPIKE			MATRIX SPIKE			
PARAMETER	Batch	Sample Id	Dup #1	Dup #2	RPD		Measured Value	Acceptable Limits		Recovery	Acceptable Limits		Recovery	Acceptable Limits	
								Lower	Upper		Lower	Upper		Lower	Upper

Corrosivity Package

Sulfide	7181043		0.03	0.03	NA	< 0.01	108%	80%	120%						
Chloride (2:1)	7181065	7181065	66	68	3.0%	< 2	94%	80%	120%	102%	80%	120%	104%	70%	130%
Sulphate (2:1)	7181065	7181065	12	13	8.0%	< 2	92%	80%	120%	101%	80%	120%	100%	70%	130%
pH (2:1)	7181065	7181065	7.10	7.00	1.4%	NA	101%	90%	110%	NA			NA		
Electrical Conductivity (2:1)	7181065	7181065	0.195	0.199	2.0%	< 0.005	98%	90%	110%	NA			NA		
Redox Potential (2:1)	7181065	7181065	328	331	0.9%	< 5	105%	70%	130%	NA			NA		

Comments: NA signifies Not Applicable.

Duplicate Qualifier: As the measured result approaches the RL, the uncertainty associated with the value increases dramatically, thus duplicate acceptance limits apply only where the average of the two duplicates is greater than five times the RL.

Certified By:


Method Summary

CLIENT NAME: EXP. SERVICES INC.

AGAT WORK ORDER: 15U040280

PROJECT: ADM-00223648

ATTENTION TO: Ian MacMillan

SAMPLING SITE:

SAMPLED BY:

PARAMETER	AGAT S.O.P	LITERATURE REFERENCE	ANALYTICAL TECHNIQUE
Soil Analysis			
Sulfide			GRAVIMETRIC
Chloride (2:1)	INOR-93-6004	McKeague 4.12 & SM 4110 B	ION CHROMATOGRAPH
Sulphate (2:1)	INOR-93-6004	McKeague 4.12 & SM 4110 B	ION CHROMATOGRAPH
pH (2:1)	INOR 93-6031	MSA part 3 & SM 4500-H+ B	PH METER
Electrical Conductivity (2:1)	INOR-93-6036	McKeague 4.12, SM 2510 B	EC METER
Resistivity (2:1)	INOR-93-6036	McKeague 4.12, SM 2510 B, SSA #5 Part 3	CALCULATION
Redox Potential (2:1)		McKeague 4.12 & SM 2510 B	REDOX POTENTIAL ELECTRODE



AGAT

Laboratories

5835 Coopers Avenue
Mississauga, ON
L4Z 1Y2
www.agatlabs.com • webearth.agatlabs.com

Chain of Custody Record

P: 905.712.5100 • F: 905.712.5122

Client Information

Company: EXP
Contact: Don Macmillan
Address: 825 Regent St.
Subway
Phone: 705 674 9681 Fax: 705 674 8271
Project: ADM-00223648 PO: _____
AGAT Quotation #: 5.0.A.

Please note, if quotation number is not provided,
client will be billed full price for analysis.

Regulatory Requirements

☒ Regulation 153/04 (reg. S11 Amend.)
Table 1 Indicate one
☒ Ind/Com
☐ Res/Park
☐ Agriculture
☐ Soil Texture (check one)
☐ Coarse ☐ Fine
☐ Sewer Use
☐ Regulation 558
☐ CCME
☐ Other (specify) _____
☐ Sanitary
☐ Storm
☐ Prow. Water Quality
☐ Objectives (PWQO)
☐ None

Invoice To

Company: _____ Same: Yes ☒ No ☐
Contact: _____
Address: _____

Report Information - reports to be sent to:

Legend Matrix
GW Ground Water **O** Oil
SW Surface Water **P** Paint
SD Sediment **S** Soil

1. Name: Don Macmillan
Email: don.macmillan@exp.com
2. Name: yes beaver park
Email: yes.beaverpark@exp.com

Is this a drinking water sampler?

(potable water intended for human consumption)
☐ Yes ☒ No
If "Yes", please use the
Drinking Water Chain of Custody Form

Is this submission for a Record of Site Condition?

☐ Yes ☒ No

Sample Identification	Date Sampled	Time Sampled	Sample Matrix	# of Containers	Comments
-----------------------	--------------	--------------	---------------	-----------------	----------

BH-W4, SAMPLES 214-244	Oct 28/15	N/A	S	2	
BH-W1, Sample 557, 15-17 FT	Oct 19/15				
BH-W4, Sample 555, 10-12 FT	Oct 29/15				

Metals and Inorganics	
Metal Scan	
Hydride Forming Metals	
Client Custom Metals	
ORPs: <input type="checkbox"/> B-HWS <input type="checkbox"/> Cl- <input type="checkbox"/> CN- <input type="checkbox"/> EC <input type="checkbox"/> FOC <input type="checkbox"/> Cr+6- <input type="checkbox"/> SAR <input type="checkbox"/> NO ₃ /NO ₂ <input type="checkbox"/> N- Total <input type="checkbox"/> Hg <input type="checkbox"/> pH	
Nutrients: <input type="checkbox"/> TP <input type="checkbox"/> NH ₃ <input type="checkbox"/> TKN <input type="checkbox"/> NO ₃ <input type="checkbox"/> NO ₂ <input type="checkbox"/> NO ₃ /NO ₂	
VOC: <input type="checkbox"/> VOC <input type="checkbox"/> THM <input type="checkbox"/> BTEX	
CCME Fractions 1 to 4	
ABNs	
PAHs	
Chlorophenols	
PCBs	
Organochlorine Pesticides	
TCLP Metals/Inorganics	
Sewer Use	

Corrosivity Package

Laboratory Use Only
Arrival Temperature: 15.0 040280
AGAT WO #: _____
Lab Temperature: 64/61/66
Notes: _____

Turnaround Time Required (TAT) Required*

Regular TAT
☒ 5 to 7 Working Days
Rush TAT (please provide prior notification)
Rush Surcharges Apply
☐ 3 Working Days
☐ 2 Working Days
☐ 1 Working Day
OR
Date Required (Rush surcharges may apply): _____

*TAT is exclusive of weekends and statutory holidays

Samples Requisitioned By: Print Name and Sign: <u>Paula Frabouze</u> <u>Trillium</u>	Date/Time: <u>Nov 6/15</u> <u>1100</u>	Samples Received By: Print Name and Sign: <u>Katherine Decker</u> <u>Shamun</u>	Date/Time: <u>Nov 6 11:30am</u>	Pink Copy - Client	Page <u>1</u> of <u>1</u>
Samples Requisitioned By: Print Name and Sign: _____	Date/Time: _____	Samples Received By: Print Name and Sign: _____	Date/Time: _____	Yellow Copy - AGAT	No: <u>40738</u>
Samples Requisitioned By: Print Name and Sign: _____	Date/Time: _____	Samples Received By: Print Name and Sign: _____	Date/Time: _____	White Copy - AGAT	

Appendix F – Slope Stability Analysis

Melgund Creek Culvert
Hwy 603
East Embankment (Inlet)
Drained Condition

Name: Sand to Silty Sand Fill Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion': 0 kPa Phi': 35 °
 Name: Organic Silt (Very Loose to Loose) Model: Mohr-Coulomb Unit Weight: 16 kN/m³ Cohesion': 0 kPa Phi': 27 °
 Name: Silt (Very Loose to Compact) Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion': 0 kPa Phi': 29 °
 Name: Clayey Silt (Firm to Stiff) Model: Mohr-Coulomb Unit Weight: 19 kN/m³ Cohesion': 0 kPa Phi': 28 °

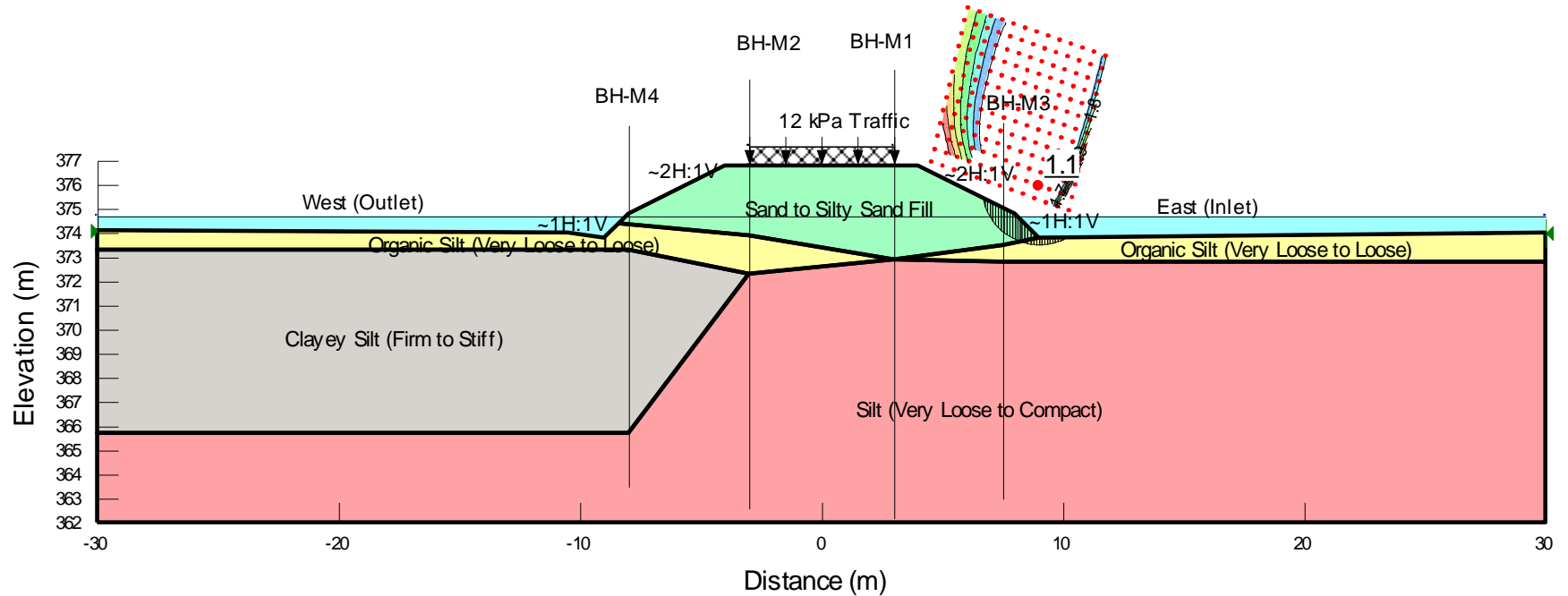


Figure F1: East slope of existing embankment (inlet) – drained static condition

Figure F2: West slope of existing embankment (outlet) – drained static condition

Melgund Creek Culvert
Hwy 603
East Embankment (Inlet)
Drained Condition

Name: Sand to Silty Sand Fill Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion': 0 kPa Phi': 35 °
Name: Organic Silt (Very Loose to Loose) Model: Mohr-Coulomb Unit Weight: 16 kN/m³ Cohesion': 0 kPa Phi': 27 °
Name: Silt (Very Loose to Compact) Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion': 0 kPa Phi': 29 °
Name: Clayey Silt (Firm to Stiff) Model: Mohr-Coulomb Unit Weight: 19 kN/m³ Cohesion': 0 kPa Phi': 28 °
Name: Engineered Fill Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion': 0 kPa Phi': 30 °

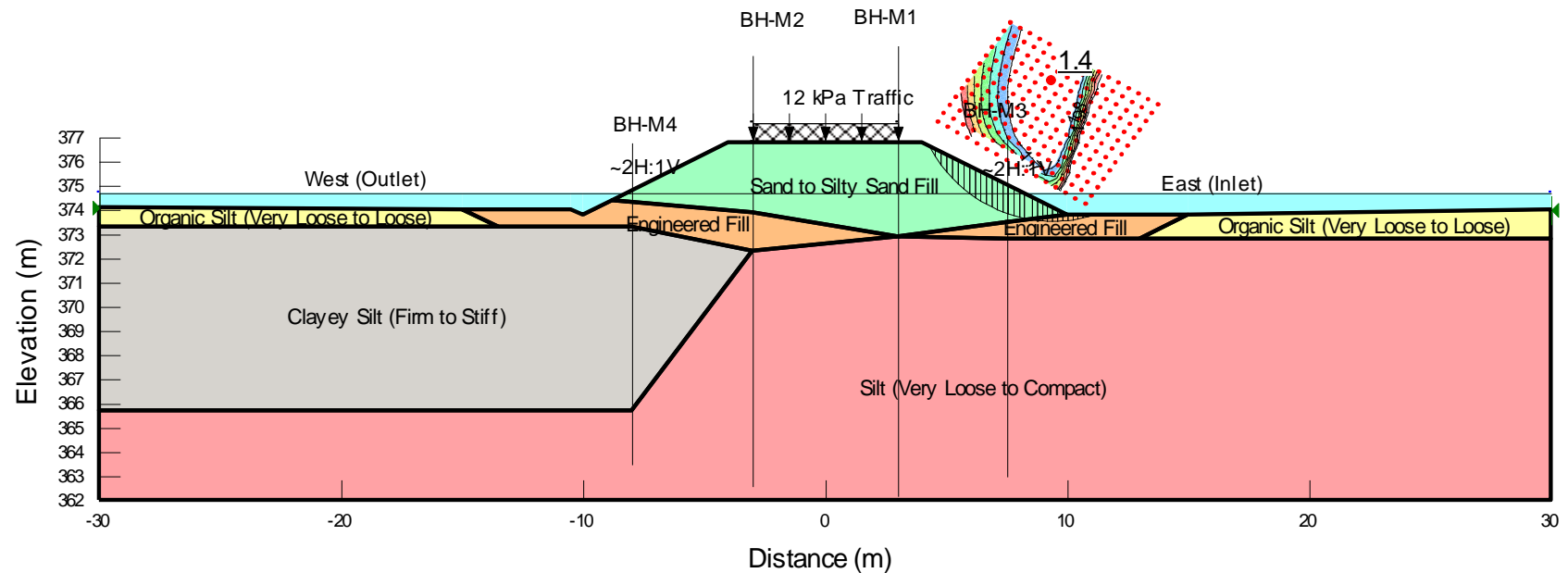


Figure F3: East slope of new embankment (inlet) – drained static condition with 2H:1V slope and engineered fill below embankment

**Melgund Creek Culvert
 Hwy 603
 West Embankment (Outlet)
 Drained Condition**

Name: Sand to Silty Sand Fill Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion': 0 kPa Phi': 35 °
 Name: Organic Silt (Very Loose to Loose) Model: Mohr-Coulomb Unit Weight: 16 kN/m³ Cohesion': 0 kPa Phi': 27 °
 Name: Silt (Very Loose to Compact) Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion': 0 kPa Phi': 29 °
 Name: Clayey Silt (Firm to Stiff) Model: Mohr-Coulomb Unit Weight: 19 kN/m³ Cohesion': 0 kPa Phi': 28 °
 Name: Engineered Fill Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion': 0 kPa Phi': 30 °

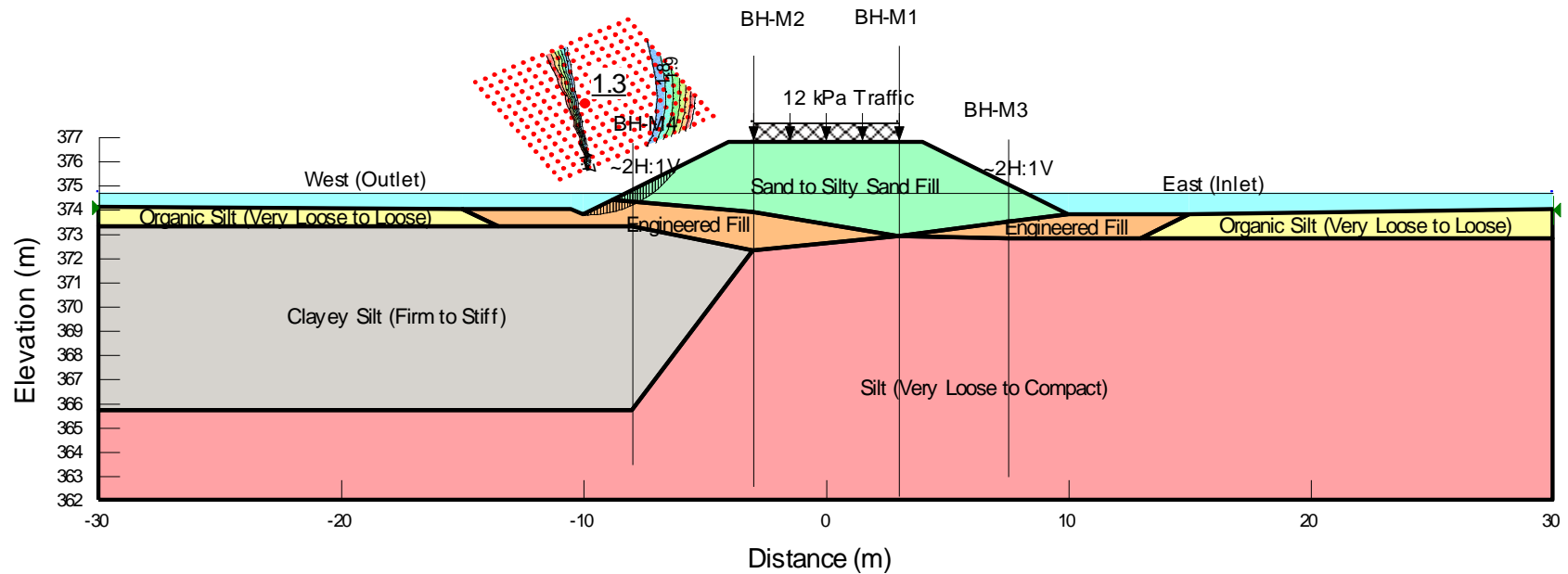
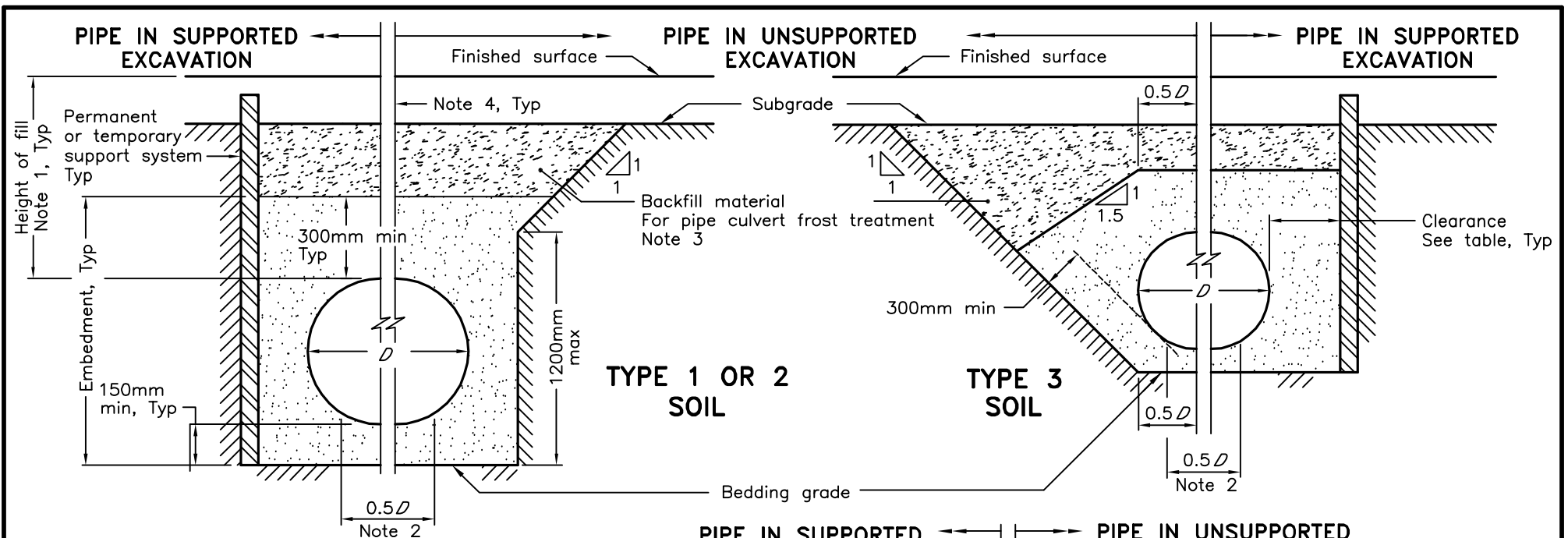


Figure F4: West slope of new embankment (outlet) – drained static condition with 2H:1V slope and engineered fill below embankment

Appendix G – OPSDs



LEGEND:

D - Inside diameter

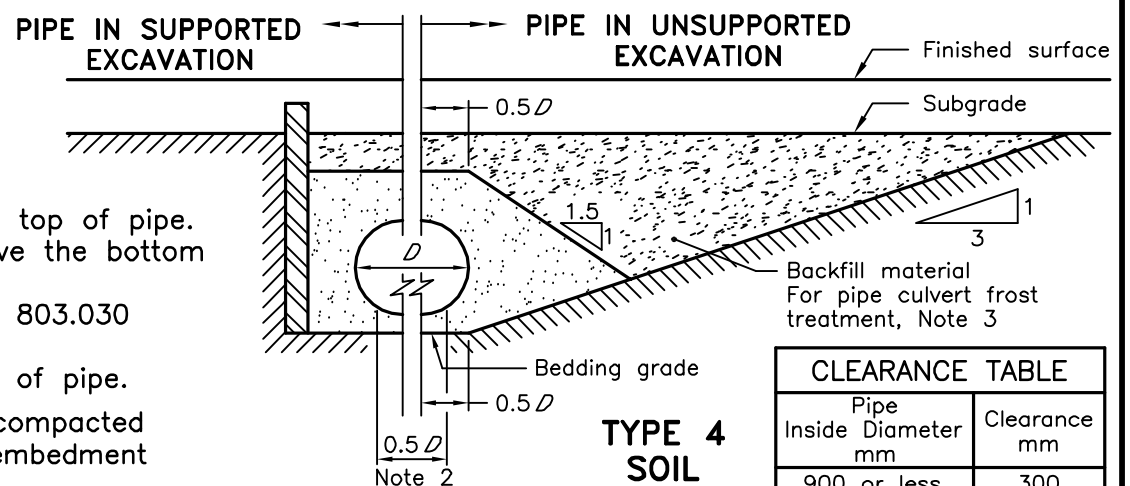
NOTES:

- 1 Height of fill is measured from the finished surface to top of pipe.
- 2 The pipe bed shall be compacted and shaped to receive the bottom of the pipe.
- 3 Pipe culvert frost treatment shall be according to OPSD 803.030 and 803.031.
- 4 Condition of excavation is symmetrical about centreline of pipe.

A Granular material placed in the haunch area shall be compacted prior to placing and compacting the remainder of the embedment material.

B Soil types as defined in the Occupational Health and Safety Act and Regulations for Construction Projects.

C All dimensions are in metres unless otherwise shown.



CLEARANCE TABLE	
Pipe Inside Diameter mm	Clearance mm
900 or less	300
Over 900	500

ONTARIO PROVINCIAL STANDARD DRAWING

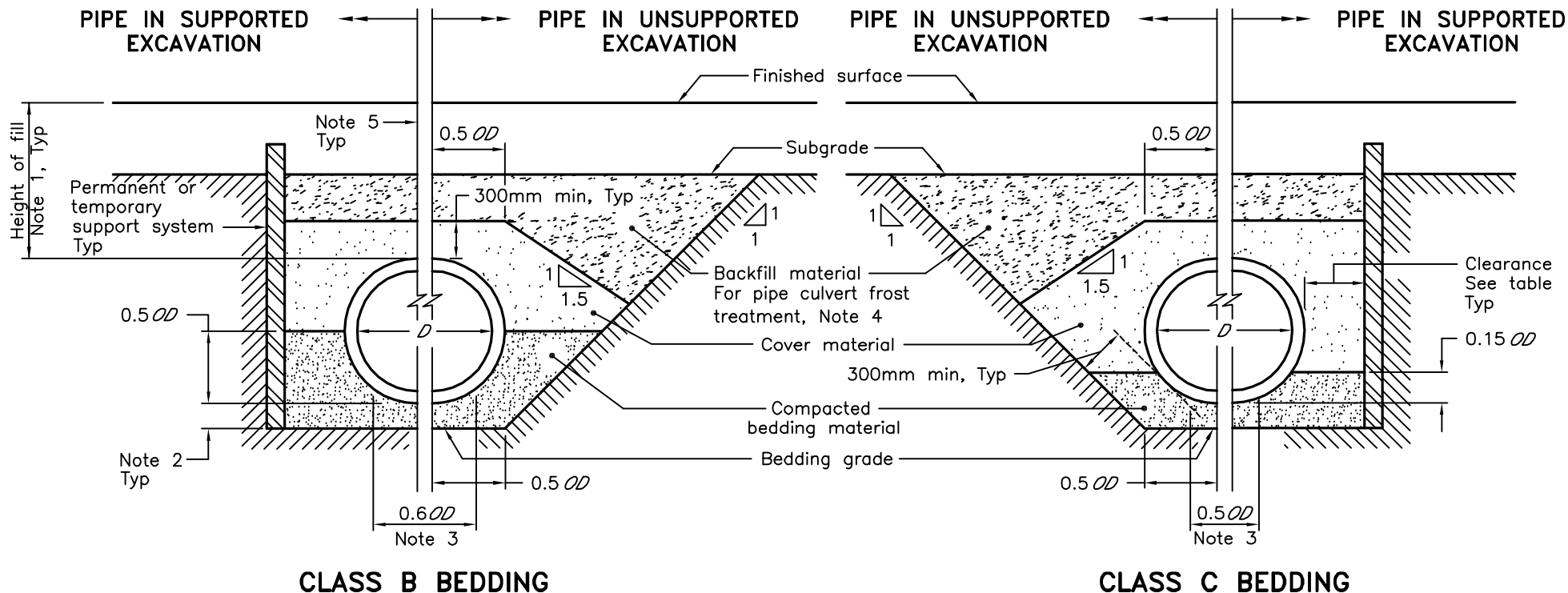
FLEXIBLE PIPE
EMBEDMENT AND BACKFILL
EARTH EXCAVATION

Nov 2010

Rev 2



OPSD 802.010



NOTES:

- 1 Height of fill is measured from the finished surface to top of pipe.
 - 2 The minimum bedding depth below the pipe shall be $0.15D$. In no case shall this dimension be less than 150mm or greater than 300mm.
 - 3 The pipe bed shall be compacted and shaped to receive the bottom of the pipe.
 - 4 Pipe culvert frost treatment shall be according to OPSD 803.030 and 803.031.
 - 5 Condition of excavation is symmetrical about centreline of pipe.
- A Soil types as defined in the Occupational Health and Safety Act and Regulations for Construction Projects.
- B All dimensions are in metres unless otherwise shown.

LEGEND:

D – Inside diameter
 OD – Outside diameter

CLEARANCE TABLE	
Pipe Inside Diameter mm	Clearance mm
900 or less	300
Over 900	500

ONTARIO PROVINCIAL STANDARD DRAWING

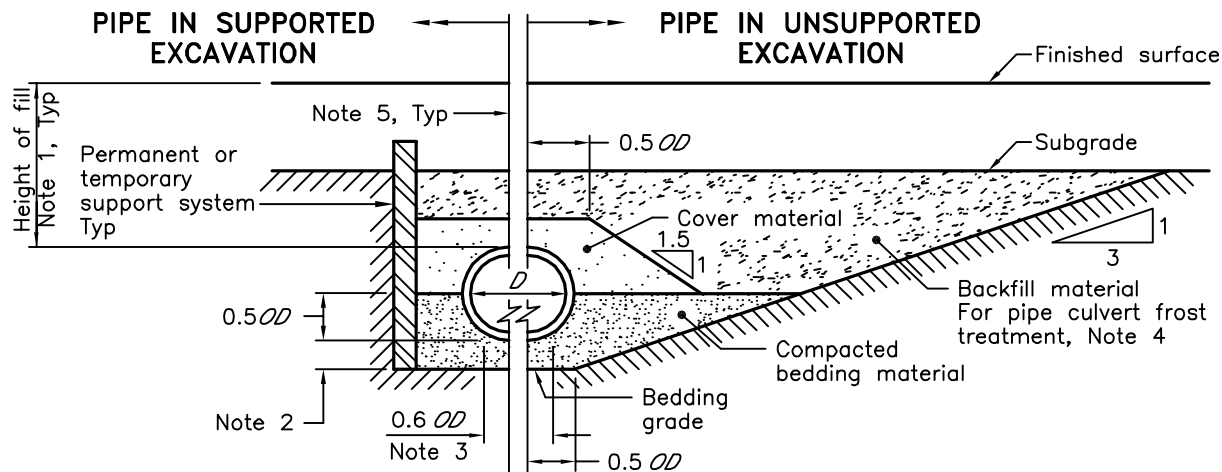
Nov 2010

Rev 2

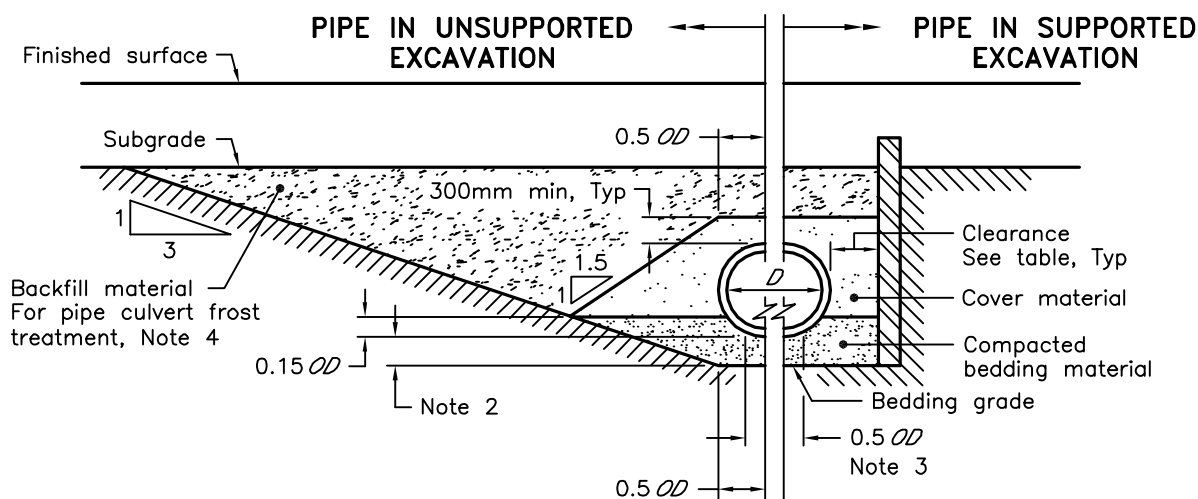
**RIGID PIPE BEDDING,
COVER, AND BACKFILL
TYPE 3 SOIL – EARTH EXCAVATION**

OPSD 802.031





CLASS B BEDDING



CLASS C BEDDING

LEGEND:

D – Inside diameter
 OD – Outside diameter

CLEARANCE TABLE	
Pipe Inside Diameter mm	Clearance mm
900 or less	300
Over 900	500

NOTES:

- Height of fill is measured from the finished surface to top of pipe.
 - The minimum bedding depth below the pipe shall be 0.15 D .
In no case shall this dimension be less than 150mm or greater than 300mm.
 - The pipe bed shall be compacted and shaped to receive the bottom of the pipe.
 - Pipe culvert frost treatment shall be according to OPSD 803.030 and 803.031.
 - Condition of excavation is symmetrical about centreline of pipe.
- A Soil types as defined in the Occupational Health and Safety Act and Regulations for Construction Projects.
- B All dimensions are in metres unless otherwise shown.

ONTARIO PROVINCIAL STANDARD DRAWING

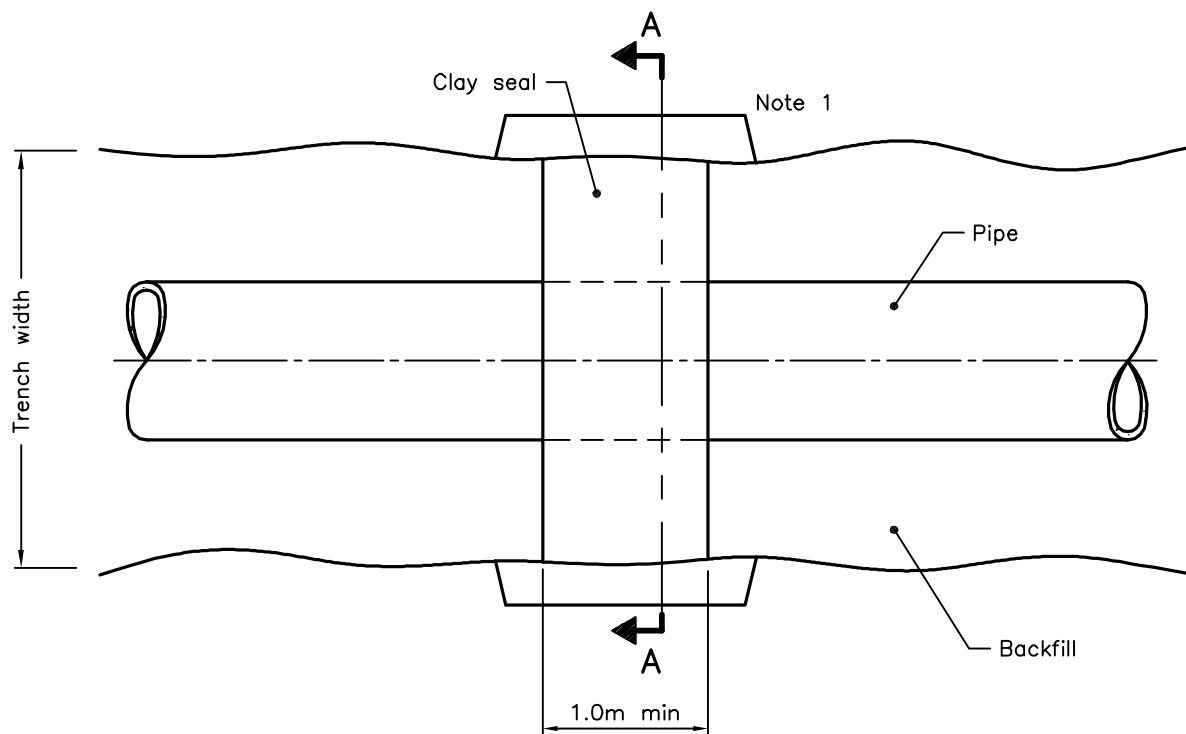
Nov 2010

Rev 2

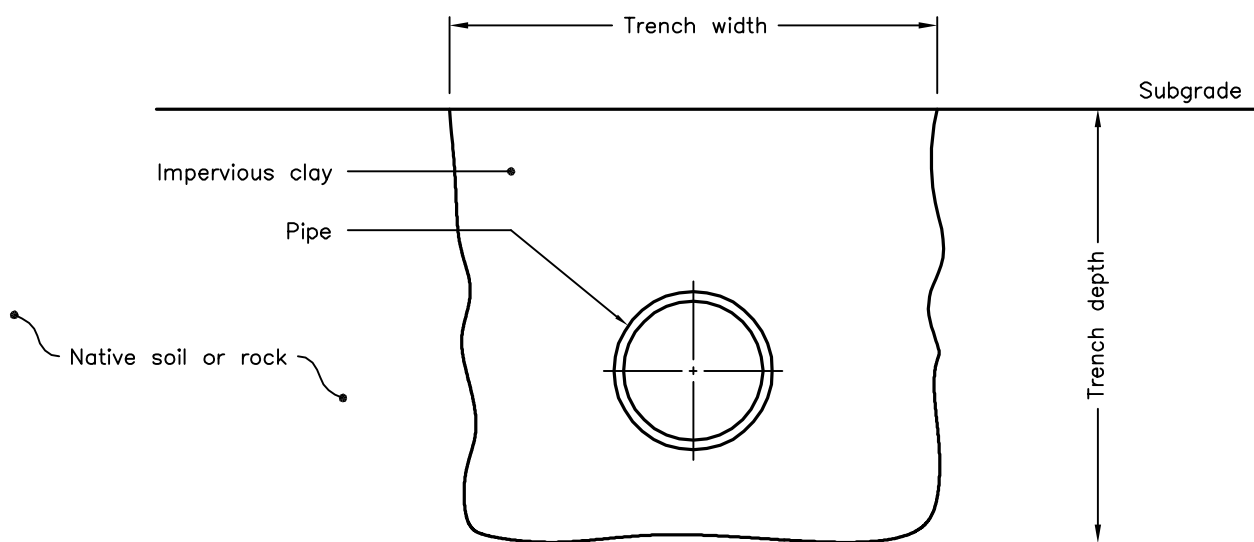
**RIGID PIPE BEDDING,
 COVER, AND BACKFILL
 TYPE 4 SOIL – EARTH EXCAVATION**

OPSD 802.032





PLAN



SECTION A-A

NOTES:

1. Key into undisturbed trench soil.

A Clay seal shall extend from bottom of trench excavation to the subgrade.

B Clay seal shall be located so that no pipe joints are within the clay seal material.

C All dimensions are in metres unless otherwise shown.

ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2011

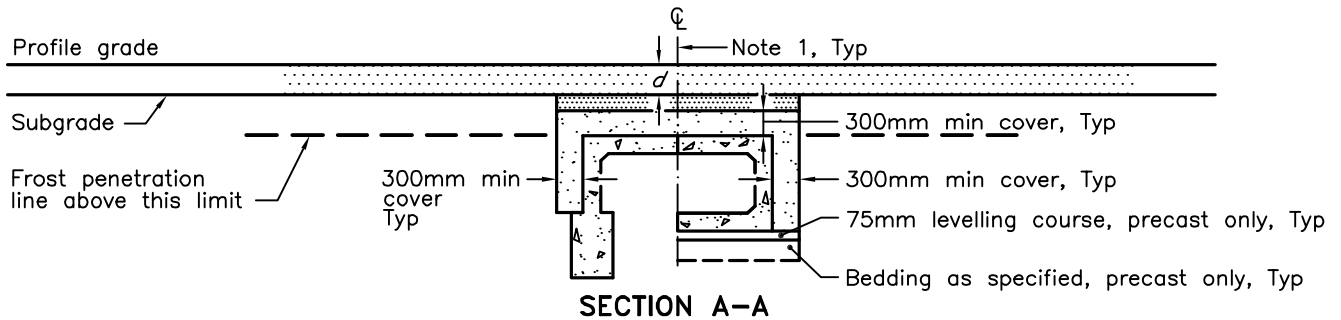
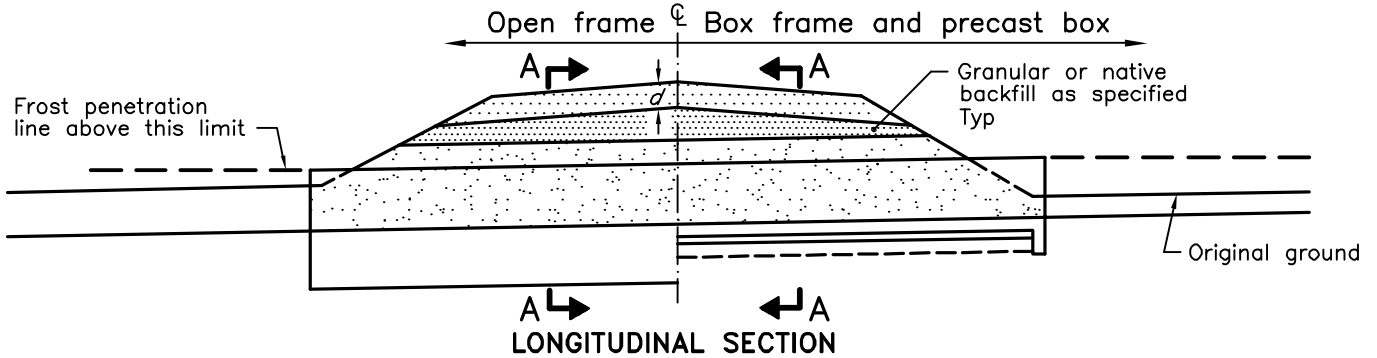
Rev 1

CLAY SEAL FOR PIPE TRENCHES

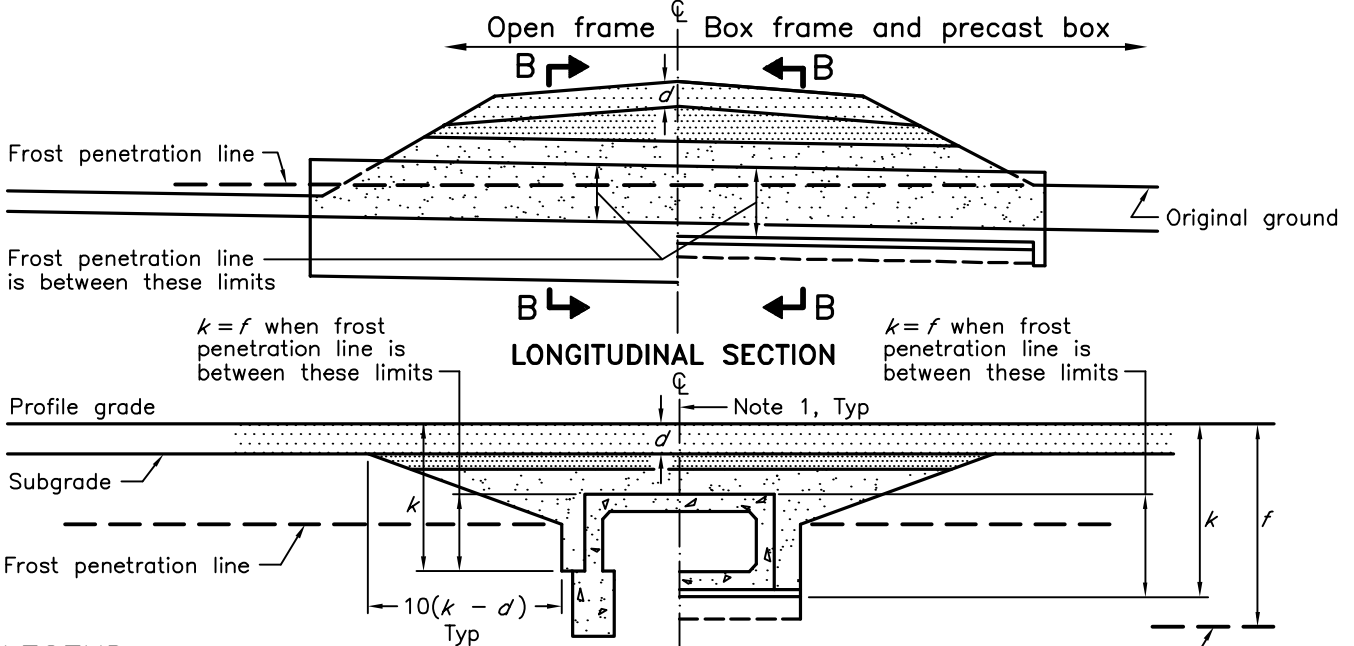
OPSD 802.095



FROST PENETRATION LINE AT OR ABOVE TOP OF CULVERT



FROST PENETRATION LINE BELOW TOP OF CULVERT



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.

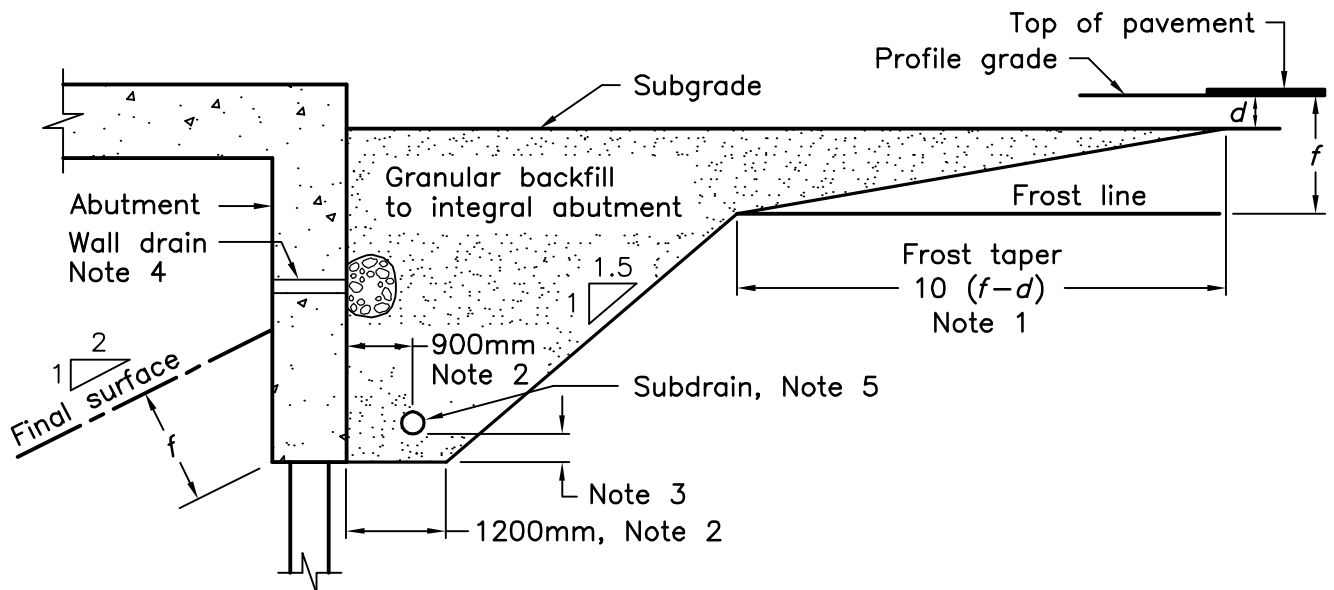
ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2010 Rev 2

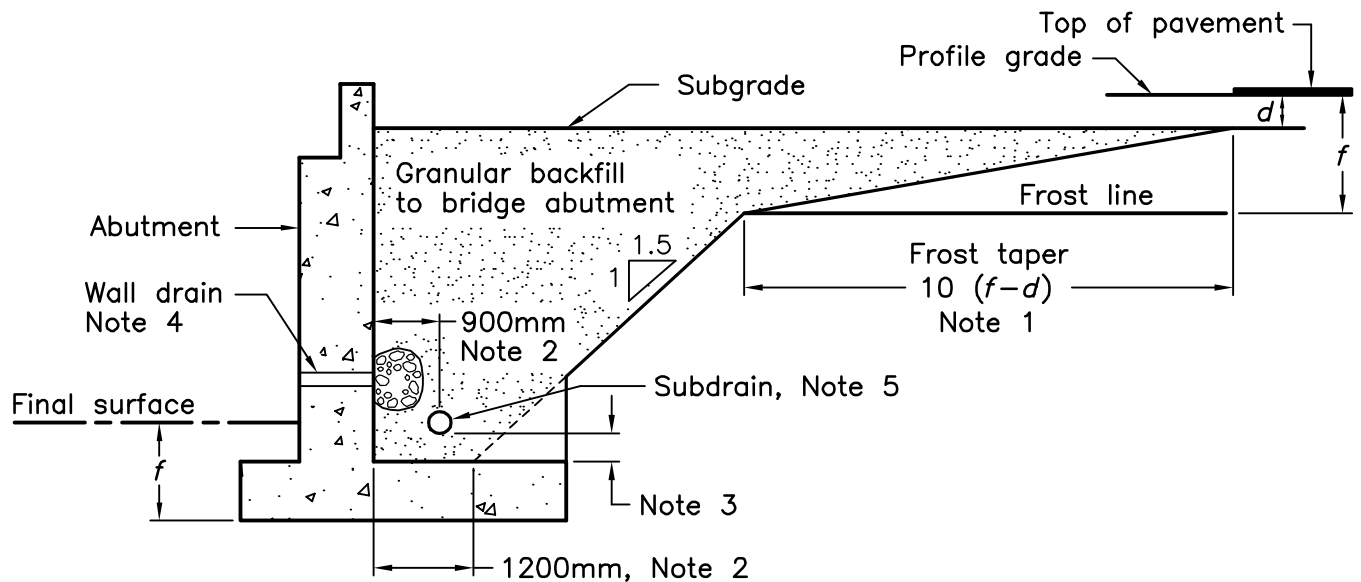
**BACKFILL AND COVER FOR
CONCRETE CULVERTS WITH SPANS
LESS THAN OR EQUAL TO 3.0M**

OPSD 803.010





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

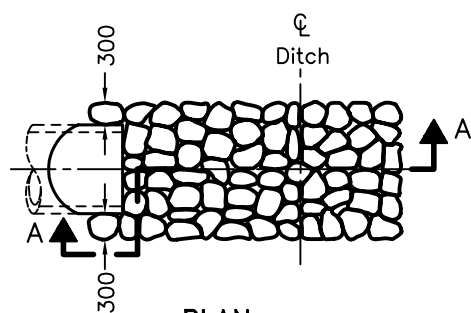
Nov 2010

Rev 1

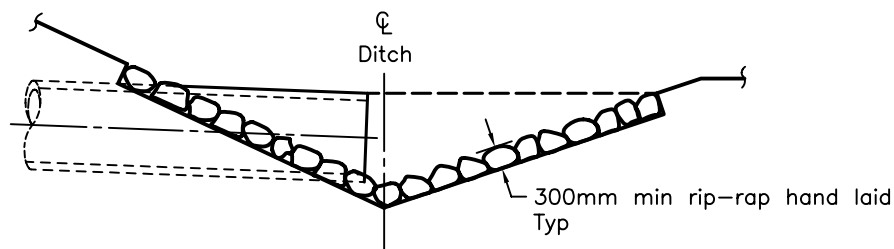


WALLS
ABUTMENT, BACKFILL
MINIMUM GRANULAR REQUIREMENT

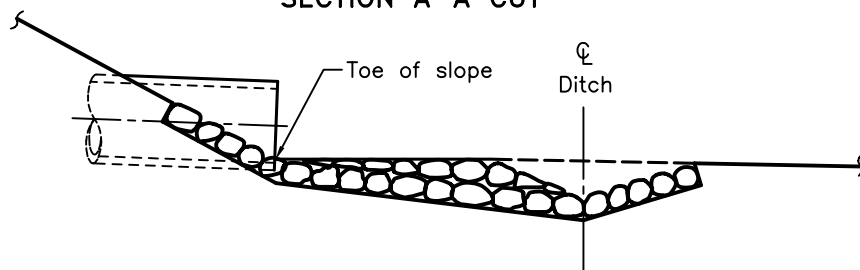
OPSD 3101.150



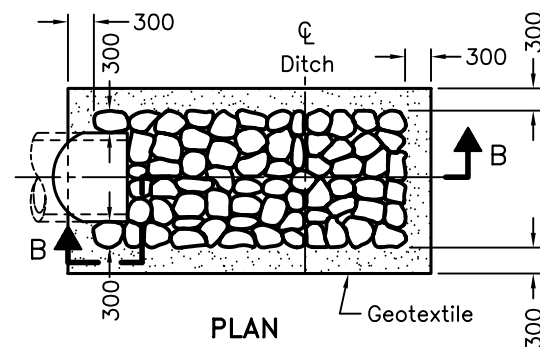
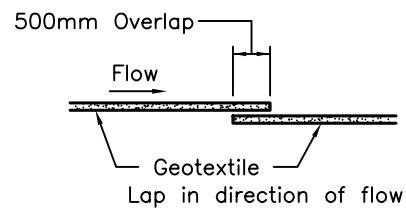
PLAN
CUT OR FILL



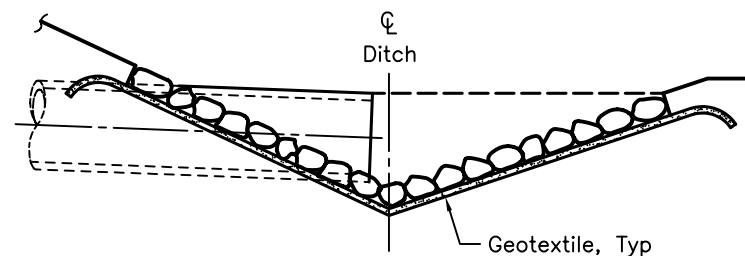
SECTION A-A CUT



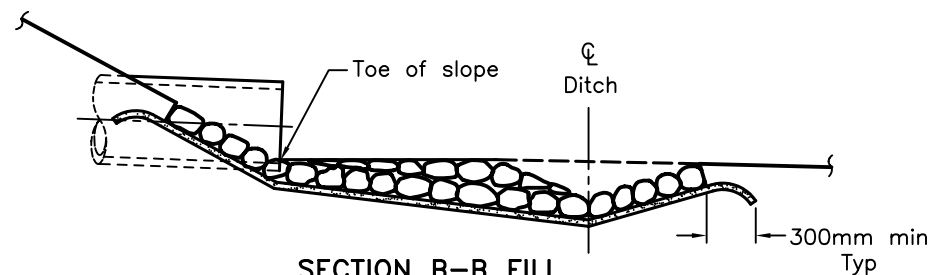
SECTION A-A FILL
TYPE A – WITHOUT GEOTEXTILE



PLAN
CUT OR FILL



SECTION B-B CUT



SECTION B-B FILL
TYPE B – WITH GEOTEXTILE

NOTES:

A All dimensions are in millimetres unless otherwise shown.

ONTARIO PROVINCIAL STANDARD DRAWING

RIP-RAP TREATMENT
FOR SEWER AND CULVERT OUTLETS

Nov 2001

Rev 0



OPSD – 810.010

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

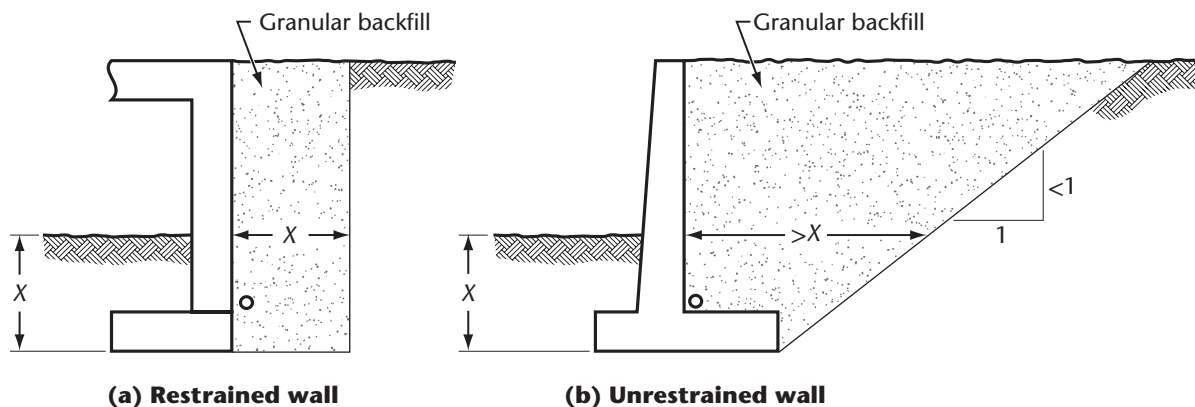


Figure C6.20
Backfill for frost protection
(See Clause C6.9.1.)

C6.9.2 Lateral pressures

C6.9.2.1 General

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

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

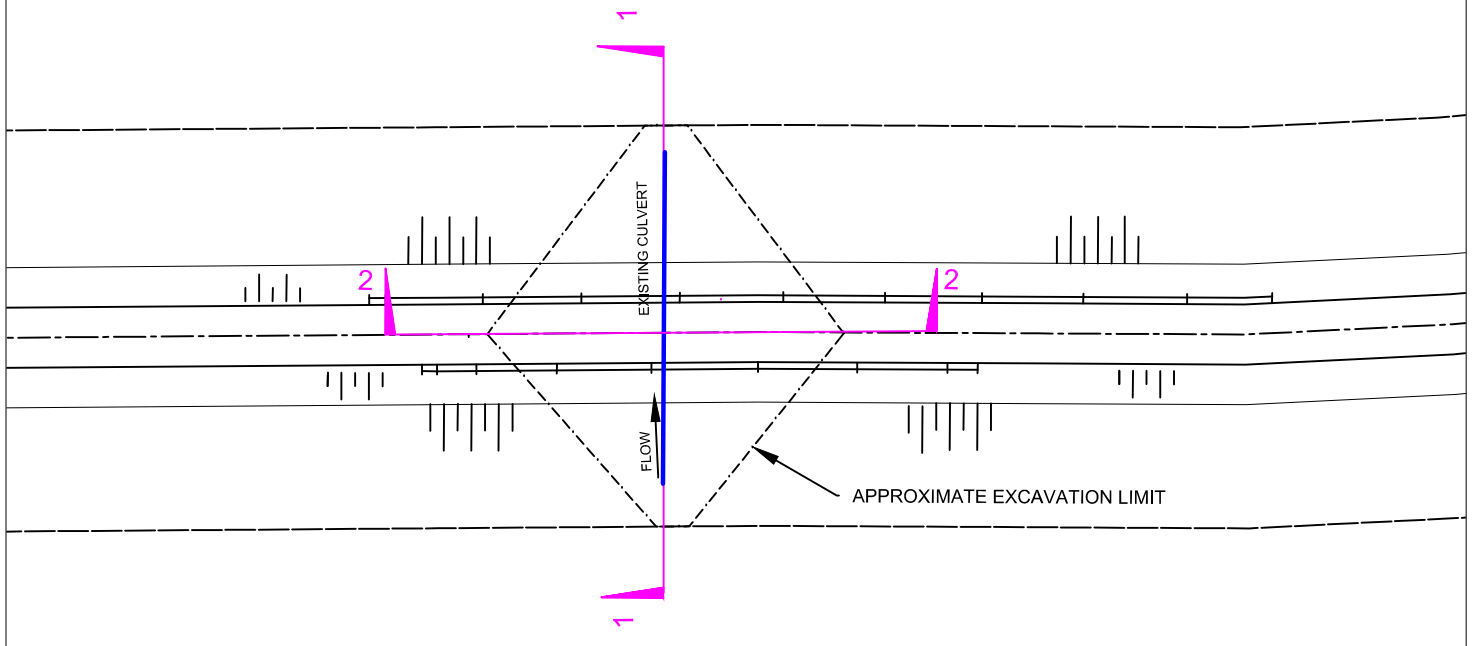
An unrestrained structure is one in which active pressure is mobilized in the backfill due to movement in the supporting structure. This movement corresponds to a rotation of approximately 0.002 about the base of a vertical wall, a horizontal translation of 0.001 times the height of the wall, or a combination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

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

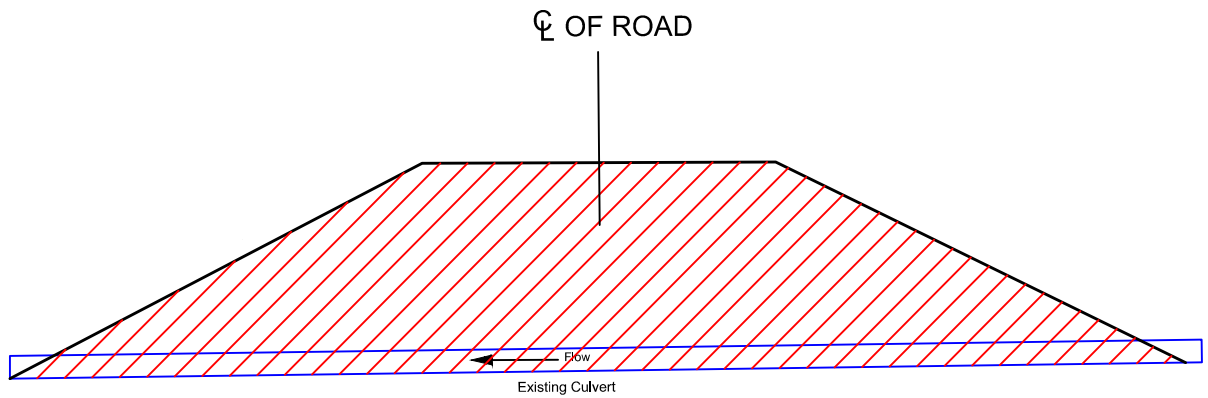
Appendix H – Schematic Sketches for Construction Alternatives

**FIGURE H.1: FULL ROAD CLOSURE USING EXISTING ROADWAYS AND OPEN CUT
UNSUPPORTED EXCAVATION OPTION1**

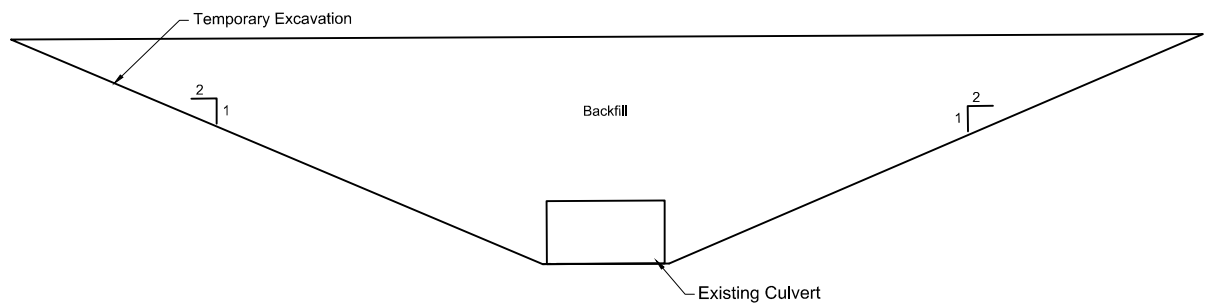
SCHEMATIC DIAGRAMS (NTS)



PLAN



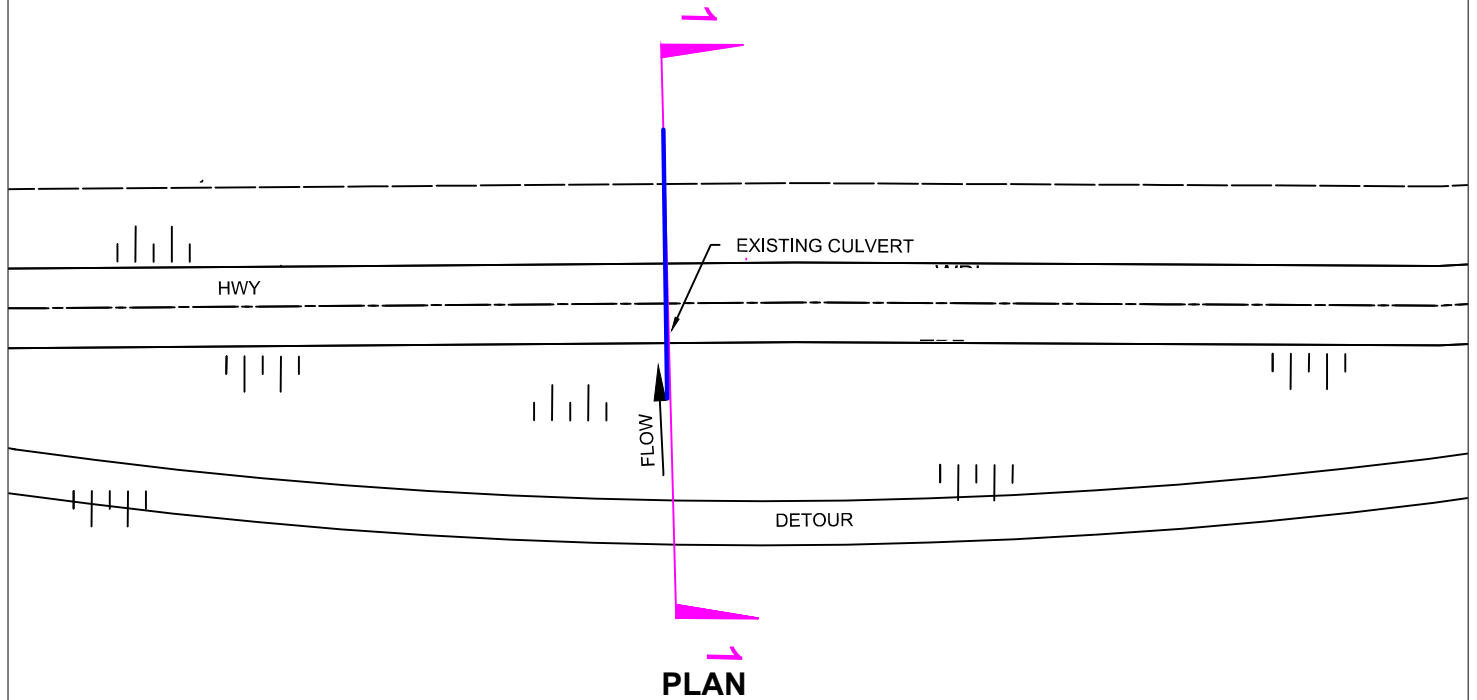
SECTION 1-1



SECTION 2-2

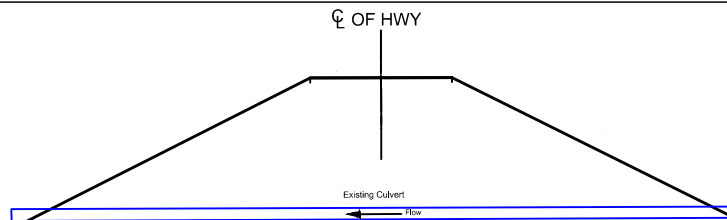
FIGURE H.2: TEMPORARY LOCAL DETOUR AND OPEN CUT UNSUPPORTED EXCAVATION (OPTION 2)

SCHEMATIC DIAGRAMS (NTS)

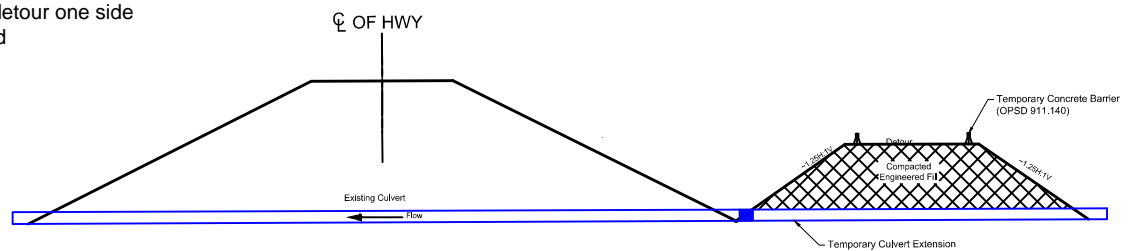


RECOMMENDED STAGES

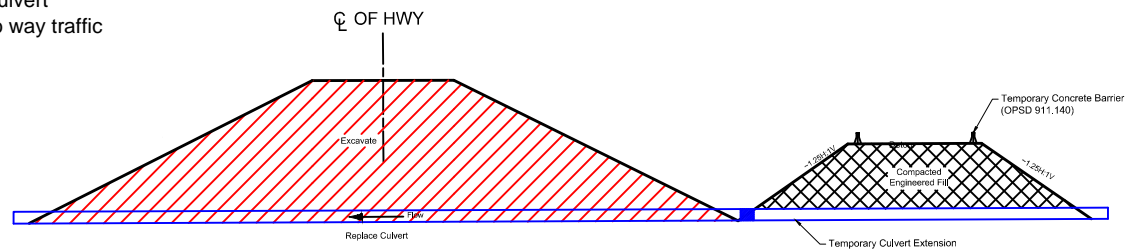
1.0 Stage 1 - Current condition



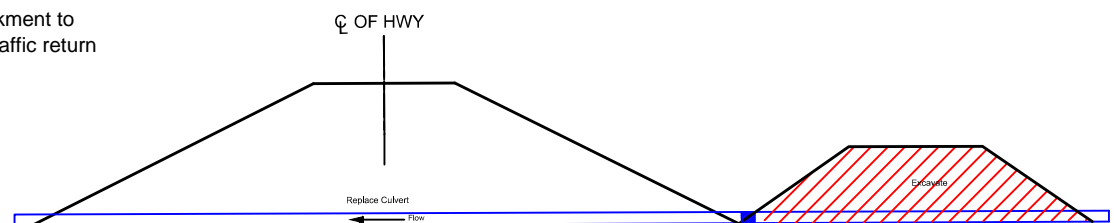
2.0 Stage 2 - Build temporary detour one side
Two-way traffic on existing road



3.0 Stage 3 - Excavation and culvert construction on other side; Two way traffic shifted to detour

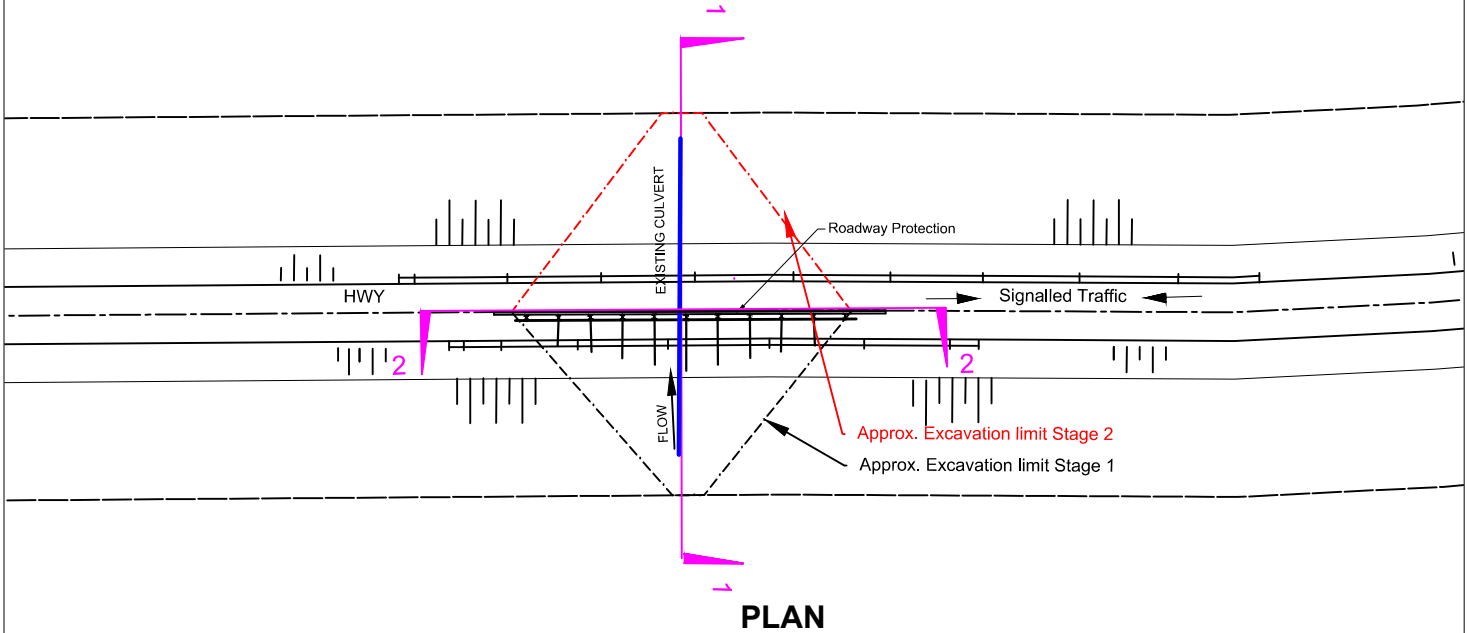


4.0 Stage 4 - Build the embankment to existing alignment; Two-way traffic return

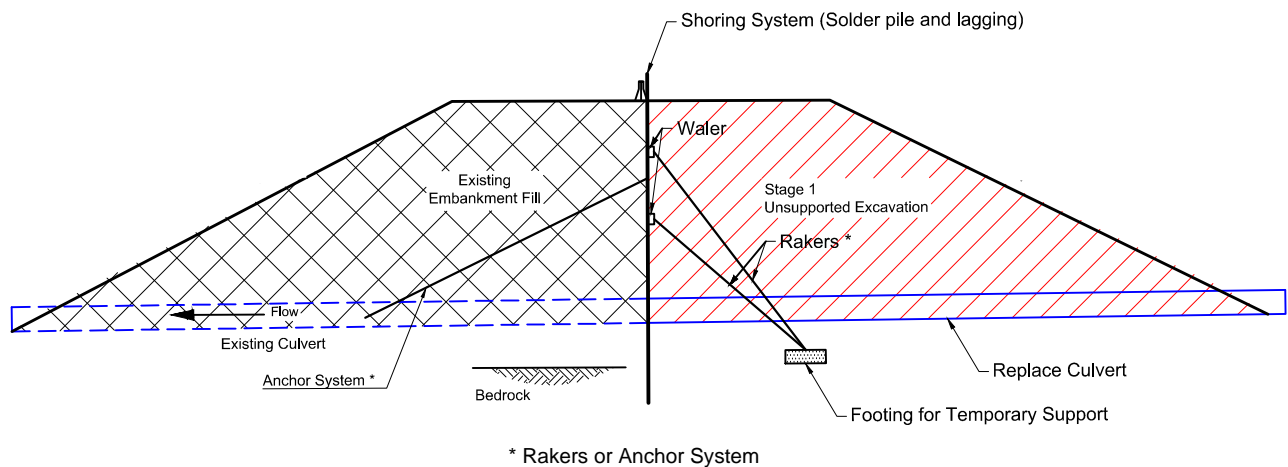


SECTION 1-1

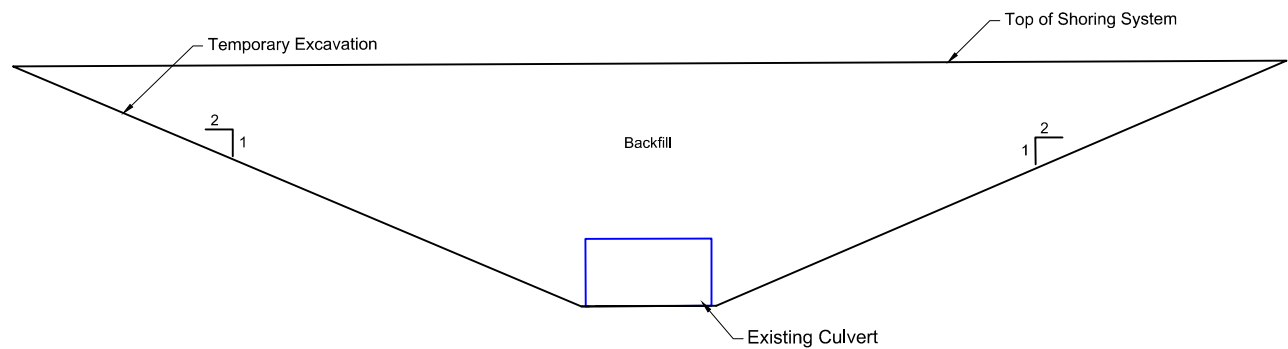
SCHEMATIC DIAGRAMS (NTS)



Half and Half Construction, Shoring system with either rakers or anchor system - Unsupported Excavation



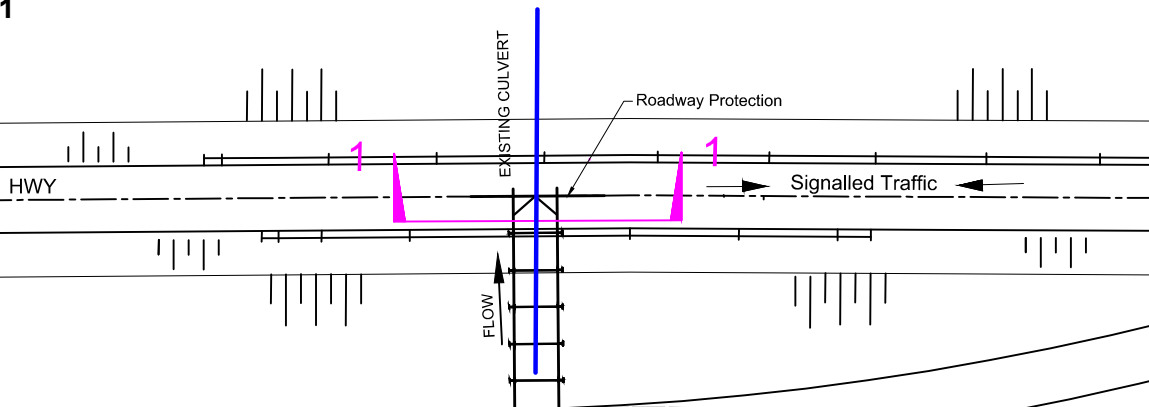
SECTION 1-1



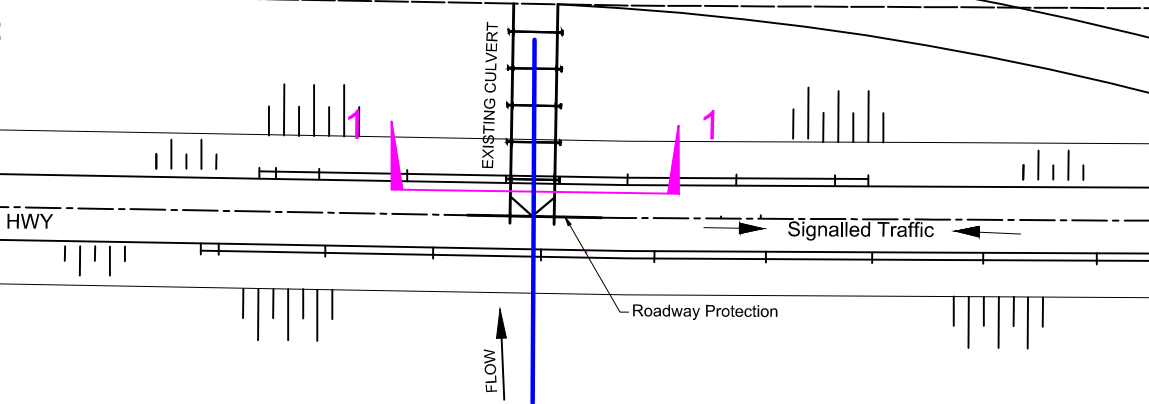
SECTION 2-2

**FIGURE H.3.B: HALF AND HALF CONSTRUCTION WITH BRACED CUT SIDES
OR ANCHOR SYSTEM OPTION 3.B
SCHEMATIC DIAGRAMS (NTS)**

Stage 1

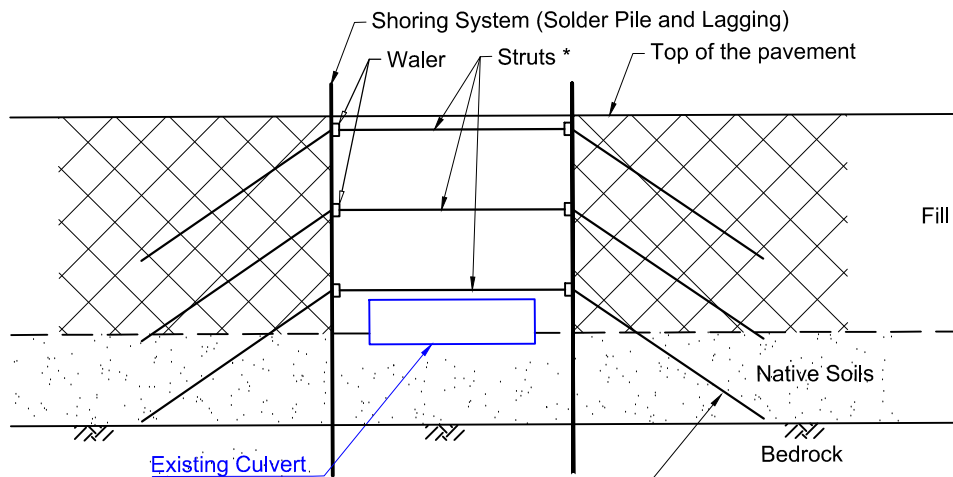


Stage 2



PLAN

Half and Half Construction, Shoring System - Braced Cut Struts or Anchor System



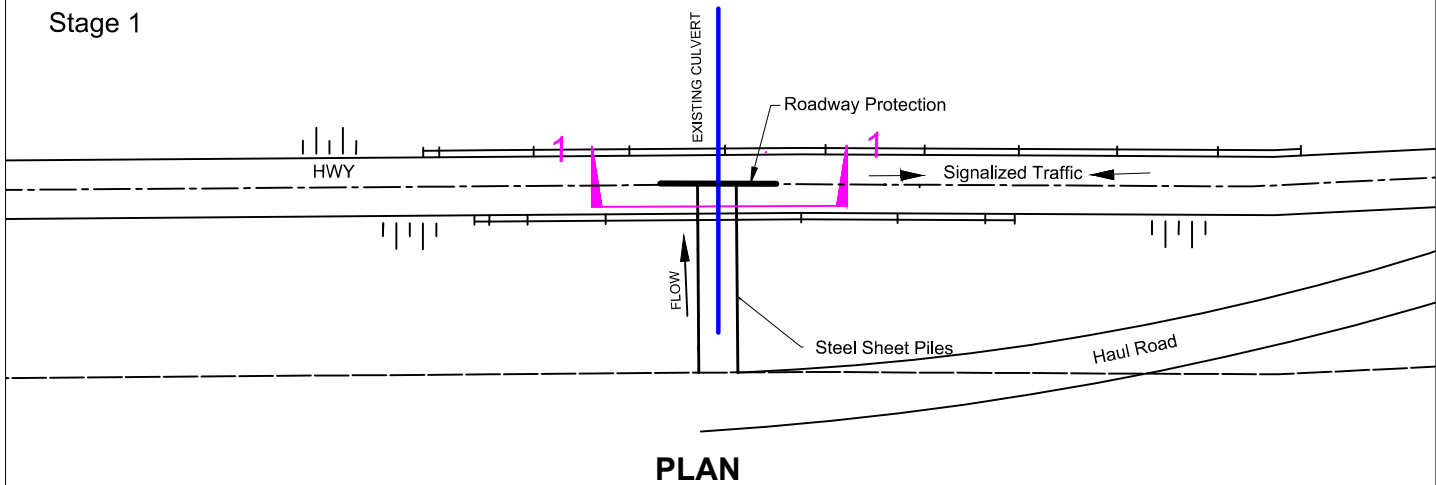
* Struts or Anchor System

SECTION 1-1

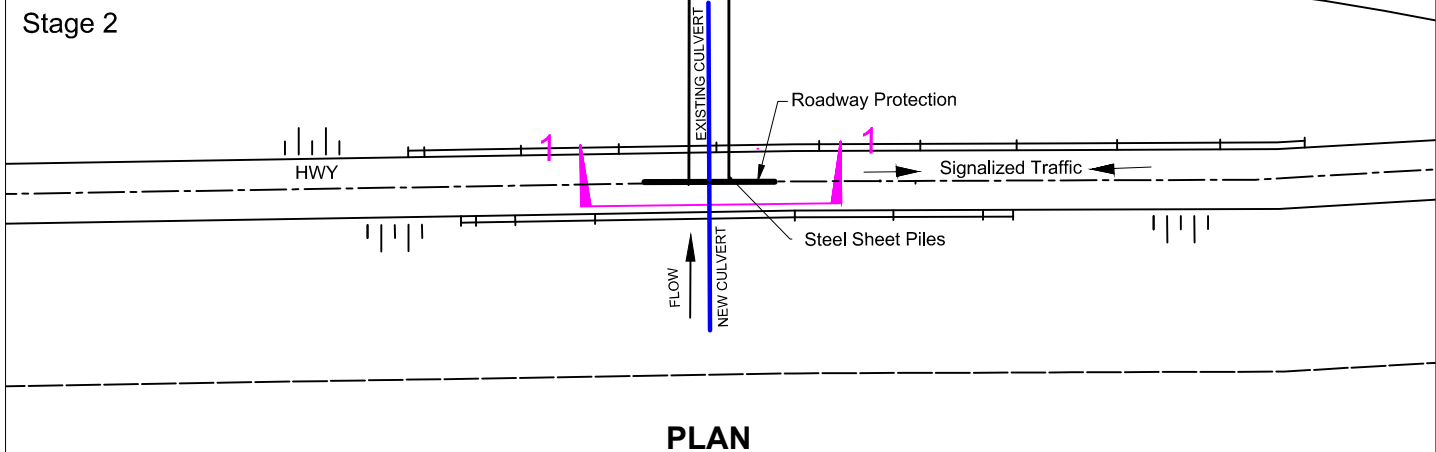
FIGURE H.3.C: HALF AND HALF CONSTRUCTION WITH INSTALLATION OF STEEL SHEET PILE ABUTMENTS WITH PRECAST CONCRETE DECK (OPTION 3.C)

SCHEMATIC DIAGRAMS (NTS)

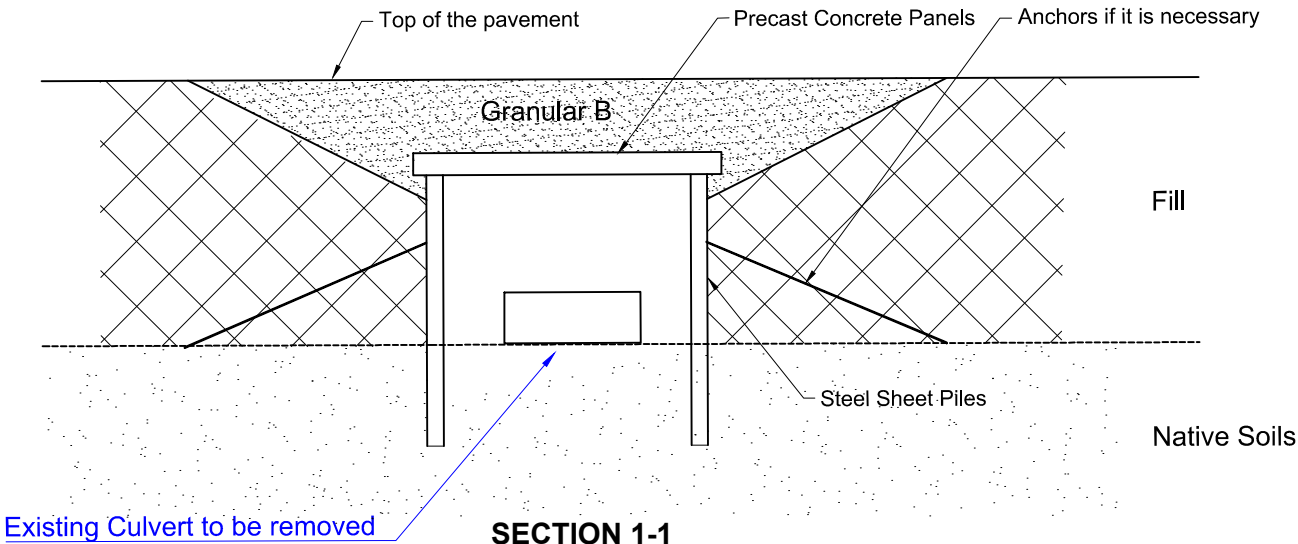
Stage 1



Stage 2



Half and Half Construction, Steel Sheet Pile Abutments with Precast Concrete Deck



Appendix I- Operational Constrains and Non Standard Special Provisions

NSSP FOR MASS CONCRETE ON BEDROCK

Scope of Work

The scope of work for the above noted tender item includes the mass concrete under the strip footings for the Flood Creek East Timber Culvert Replacement.

The Contractor should be aware that there is sloping bedrock in the area and fractured bedrock was encountered within the upper 1 m of the bedrock surface. Mass concrete volumes will vary depending on the variable intact bedrock surface.

Construction

Concrete shall be of the same strength as the footing concrete and placed in accordance with OPSS.PROV 904 "CONSTRUCTION SPECIFICATION FOR CONCRETE STRUCTURES".

Basis of Payment

Payment at the contract price for the above noted tender item includes full compensation for all labour, equipment and materials to do the required work.

NSSP FOR SLOPING ROCK AND COBBLES AND ROCK PIECE OBSTRUCTIONS

Scope of Work

The Contractor should be aware that there is sloping bedrock in the area and fractured bedrock was encountered within the upper 1 m of the bedrock surface. The overburden soils at the site consist of gravelly sand and silty sand fill materials which may contain cobbles and rock fragments especially near the bedrock interface.

Appropriate equipment and procedures will be required to penetrate/remove cobbles and fractured bedrock that are encountered during excavation.

Basis of Payment

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

NSSP FOR DOWELLING

Scope of Work

Work under this item is for the placement and field testing of dowels into rock.

Materials and Installation

Dowels into rock shall be constructed in accordance with OPSS.PROV 904 "CONSTRUCTION SPECIFICATION FOR CONCRETE STRUCTURES". All reinforcing steel supplied shall be in accordance with OPSS.PROV 1440 "MATERIAL SPECIFICATION FOR STEEL REINFORCEMENT FOR CONCRETE" (dowel bars conforming to CSA Standard CSAG30.18, Grade 400).

Where dowels are to be placed in rock, holes shall be drilled to the required depth and size. Hole diameter shall be two times the nominal diameter of the dowel. Each hole shall be cleaned out, grouted and the dowel set in place. Grout shall be of the same strength as the footing concrete (or at least 25 MPa at 28 days).

If the hole contains water, the contractor shall remove the water otherwise a tremie procedure shall be used to completely fill the hole with grout. The dowel shall be installed into the hole after the grout has been placed and while it is still fresh.

Dowel Testing

All proposed testing procedures shall be in general conformance with ASTM D 3689-90 and ASTM D 114381 (Re-approved 1994). Field testing must be carried out in the presence of, and the results reviewed and approved by, the Contract Administrator.

Performance Tests

Performance tests shall be carried out on two rock dowels to confirm that the design load of the rock can be achieved. Performance test shall be by axial tensioning using a hydraulic jack with a capacity of at least 1.5 times the ultimate strength of the dowels.

Rock dowels shall be loaded and unloaded in 3 cycles and measurements of the dowel displacement shall be carried out at each load step in accordance with the following schedule:

Cycle-Step	1-1	1-2	1-3	2-1	2-2	2-3	2-4	3-1	3-2
% of Design Load	50	75	25	50	75	100	25	50	75

Cycle-Step	3-3	3-4	3-5
% of Design Load	100	110	25

Displacement measurements shall be carried out at each load step using displacement gauges with precision of 0.0025 cm. Measurements shall be referenced to an independent fixed referenced pint.

Rock dowels which do not meet the acceptance criteria shall be replaced at the Contractor's expense and re-tested. If a rock dowel fails, three additional rock dowels shall be tested at or near the same footing location as directed by the Contract Administrator.

Acceptance criteria for the rock dowels will be in accordance with the Post-testing Institute (1985) as follows:

The dowels are acceptable if the total elastic movement is greater than 80% of the theoretical elastic elongation of the free stressing and is less than the theoretical elongation of the free stressing length plus 50% of the bond length.

Basis of Payment

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

NSSP FOR CONDITION SURVEYS AND MONITORING DURING ANY BLASTING

Scope of Work

If any blasting is required, the Blast Contractor must be fully qualified and experienced. The Blast Contractor shall outline the procedure and extent of the pre-blast survey. The blast methodology, including drill hole patterns, hole size and depths, size of blast, explosive and initiation product details, as well as all blast control procedures shall be required. Blast control procedures would include details on controlling flyrock, temporary road closures, blast signaling and site clearing procedures. Details on instrumentation, number and location of monitoring sites, blast recording and reporting procedures, and procedures to be followed in the event of excessive vibration readings are required as well.

Instrumentation or monitoring ground and air vibration effects from the blasting should be set up in accordance with the International Society of Explosives Engineering field practice guidelines (1999).

Basis of Payment

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