



FINAL REPORT

FOUNDATION INVESTIGATION AND DESIGN REPORT

Culvert Replacement, Highway 3, Simcoe to Renton

Agreement No. 3015-E-0017

Assignment No. 1

GWP 3062-14-00

GEOCRES No. 40116-26

Prepared for:

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Regional Director's Office -Western Region

Geotechnical Section

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Ministry of Transportation

Western Region - Geotechnical Section

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Table of Contents

Part I: FOUNDATION INVESTIGATION REPORT	31
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	4
1.4.1 Asphalt	4
1.4.2 Topsoil	4
1.4.3 Granular Fill	4
1.4.4 Fill: Silty Clay	5
1.4.5 Fill: Silty Sand	5
1.4.6 Silty Clay	6
1.5 Groundwater and Surface Water Conditions	7
1.6 Chemical Analyses	8
Part II: ENGINEERING DISCUSSIONS AND RECOMMENDATIONS	9
2.1 General	9
2.2 Expected Ground Conditions	9
2.3 Structure Foundations	10
2.3.1 Shallow Foundations	12
2.3.1.1 Geotechnical Resistance	12
2.3.1.2 Resistance to Lateral Loads	13
2.3.1.3 Frost Protection	13
2.3.2 Steel Sheet Piles	14
2.3.2.1 Axial Resistance in Compression	14
2.3.2.2 Lateral Resistance	14
2.4 Lateral Earth Pressure	15
2.5 Seismic and Liquefaction Potential Consideration	16
2.6 Construction Alternatives	16
2.6.1 Half-and-Half Construction (Options 1)	20
2.6.1.1 Option 1.A: Half-and-Half Construction with Unsupported Cut Sides	21
2.6.1.2 Option 1.B: Half-and-Half Construction with Braced or Anchored Cut Sides	21

2.6.1.3	Option 1.C: Half-end-Half Construction using Steel Sheet Pile Abutments with Precast Concrete Deck	21
2.6.2	Detour Options (Options 2 and 3)	22
2.7	Temporary Roadway Protection	23
2.8	Culvert Bedding	23
2.9	Culvert Backfill	24
2.10	Groundwater and Surface Water Control	25
2.11	Embankment Design	25
2.11.1	Embankment Stability	25
2.11.2	Embankment Settlement	27
2.12	Inlet and Outlet.....	28
2.12.1	Erosion Protection at Outlet.....	28
2.12.2	Stream Bed Rip-Rap.....	28
2.12.3	Seepage Cut-off Requirements	28
2.12.3.1	Clay Seal.....	28
2.12.3.2	Cut-Off Trench	29
2.13	Corrosion Protection	29
Part III: CLOSURE		30
Part IV: LIMITATIONS AND USE OF REPORT		31

Appendices

APPENDIX A: PHOTOGRAPHS

APPENDIX B: DRAWING

APPENDIX C: BOREHOLE LOGS

APPENDIX D: LABORATORY DATA

APPENDIX E: SLOPE STABILITY ANALYSIS

APPENDIX F: OPSDs

APPENDIX G: SCHEMATIC SKETCHES FOR CONSTRUCTION ALTERNATIVES

APPENDIX H: NON-STANDARD SPECIAL PROVISION

APPENDIX I: RESULT OF SETTLEMENT ANALYSIS

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 an existing concrete culvert located on Hwy 3 at Station 16+070, about 4.2 km east from Ireland Road, Simcoe to Renton, the Ministry of Transportation (MTO) West Region. The work was undertaken under Agreement # 3015-E-0017, Assignment No. 1 (GWP 3062-14-00). The terms of reference (TOR) were as presented in the MTO letter dated September 12, 2016.

The purpose of the investigation is to determine the subsurface conditions along the culvert alignment and to permit detailed design for the culvert replacement including temporary protection systems for 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 3 at Station 16+070, about 4.2 km east from Ireland Road, Simcoe to Renton, Ontario. At this site, Hwy 3 is a surface treated two lanes roadway and is about 7 m wide from edge to edge of pavement, with narrow sand and gravel shoulders. Based on drawings provided, the roadway embankment is about 6.5 m high with side slope of about 2H:1V.

As noted in the TOR the existing culvert is a 2.8 m × 1.5 m × 37.9 m concrete rigid framed- open footing structure. The existing culvert is intended to be replaced with a new culvert along the same alignment and it is understood that precast box culvert with an estimated span of between 4 and 5 m, and 41.5 m length is likely to be considered to replace the existing culvert. The site plan and cross-section profiles for the proposed culvert alignment are as shown on Drawings 1 and 2 in Appendix B. Select photographs of the site/ existing culvert are presented in Appendix A.

At the vicinity of the inlet and outlet of the culvert some vegetation was noted at both culvert ends. The surrounding terrain of the culvert location is generally a flat tract of land; however, at culvert location the grade is gently rolling and there are occasional trees. A mix of low lying vegetation/shrubs and long grasses were observed on the bank of the stream along the water flow path at both inlet and outlet sides; however, no visible sign of flow restriction was observed due to long grasses. Hwy 3 runs in an east-west direction and water in culvert flow from north to south beneath the highway. At the time of investigation, the approximate water elevations in culvert at inlet and outlet were about 209.4 m and 208.9 m, respectively. The elevation of highway pavement centerline at the culvert centerline

is about 215.7 m. Cable guide rails were observed on both sides of the roadway and overhead wires were observed along the north side of the roadway.

The general site conditions in the immediate vicinity of the culvert were assessed during the site reconnaissance and drilling operations on October 12, 2016 and October 14, 2016, respectively. The embankments were noted in an overall stable configuration with no obvious indications of recent slope movement. The longitudinal meandering cracks were observed on a portion of roadway along the culvert alignment and on the EBL of Hwy 3 approximately 20 m in length on east side of the culvert alignment, but major depressions in the embankment were not observed in these areas. Due to the water in the culvert, existing foundation observation was restricted. However, based on visual observation, the culvert appeared to be in satisfactory condition with some deterioration of culvert at inlet and outlet with crumbling of concrete exposed edges and revealed rebars (see Photograph 1 and 4, in Appendix A).

1.2.2 Geological Setting

The Map P.2715 (Physiography of Southern Ontario, Third Edition, 1984) Bedrock Geology of Ontario, Southern Sheet, 1991) of the Ministry of Natural Resources indicates that the project area is located at the boundary of Sand Plain and Clay Plain. The Map 2556 (Quaternary Geology of Ontario, Southern Sheet, 1991) of the Ministry of Northern Development and Mines, indicates that the surface conditions consist of glaciolaustrine deposits including silt and clay, minor sand; basin and quiet water deposits. The Map 2544 (Bedrock Geology of Ontario, Southern Sheet, 1991) of the Ministry of Northern Development and Mines, indicates that the bedrock formation of the project area consists of limestone, dolostone and shale, middle devonian.

1.3 Investigation Procedures

1.3.1 Site Investigation and Field Testing

The field investigation was performed on October 14, 2016. To expedite the site investigation program two drill rigs were used. The field program consisted of drilling five (5) sampled boreholes (BH-1, BH-2, BH-3, BH-4 and BH-5). Three (3) boreholes were strategically located along the existing culvert alignment to provide subsurface information for the design of the proposed new culvert. Borehole BH-2 was advanced within the travelled westbound lane and located about 5 m west of the culvert centerline and 2.8 m north of the highway centerline. Boreholes BH-1 and BH-3 were advanced at accessible locations near the inlet and outlet of the culvert, respectively. Two (2) additional boreholes were strategically located on the embankment to provide subsurface information for the temporary roadway protection. Boreholes BH-4 and BH-5 were advanced at approximately 25 m west and east side of the existing culvert and approximately 2.1 m and 2.4 m south of the highway centerline, respectively. The borehole locations are shown on Drawing No. 1 in Appendix B.

Boreholes on the embankment crest (BH-2 and BH-5) were advanced using a truck mounted CME-75 drill rig and, equipped with hollow stem augers and standard soil sampling equipment operated by a specialist drilling contractor, Geo-Environmental Drilling Inc. Due to difficulty in access, boreholes at the inlets and outlets (BH-1 and BH-3) were advanced using a rubber track mounted CME-55 drill rig also operated by Geo-Environmental Drilling Inc. One of the boreholes from the embankment crest

(BH-4) was also advanced using rubber track mounted CME-55 drill rig. The roadway boreholes, BH-2, BH-4 and BH-5 were advanced to depths of about 15.9 m below ground surface. The off-road boreholes (BH-1 and BH-3) were advanced to depth of about 9.8 m below ground surface.

The borehole locations (referenced to the MTM NAD83 coordinate system) and their ground surface elevations were surveyed by **exp** personnel using the Temporary Benchmark set on (MTO # 92 on concrete post, see Photograph 10 in Appendix A) approximately 120 m east of the culvert alignment on south of highway. The TBM elevation (216.3 m) is assumed based on the information provided on site plan drawings provided by the MTO. The temporary benchmark location is shown on Drawing. 1 in Appendix B.

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. Field vane testing was conducted in cohesive soils to measure the *in-situ* undrained shear strength of those soils. Field vane test was conducted in accordance with ASTM D2573-08. One Shelby tube sample was obtained below the culvert invert level.

Upon completion of the boreholes, ground water level measurements were carried out in boreholes 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 Hamilton laboratory for additional visual, textual, olfactory examination and selective testing.

1.3.2 Laboratory Testing

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

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.

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 along the proposed culvert alignment consist of a layer of granular fill overlying silty clay fill followed by native silty clay. A more detailed summary of the subsurface conditions encountered in the boreholes is provided in the following sections.

1.4.1 Asphalt

Asphalt was encountered at the surface of boreholes BH-2, BH-4 and BH-5 and ranged in thickness from approximately 0.33 m to 0.43 m. Asphalt thicknesses may further vary beyond the borehole locations.

1.4.2 Topsoil

Topsoil was encountered at the surface of boreholes BH-1 and BH-3 and ranged in thickness from approximately 0.125 m to 0.2 m. Topsoil thicknesses may further vary beyond the borehole locations.

1.4.3 Granular Fill

Granular fill was encountered below the asphalt in all boreholes drilled from road surface (BH-2, BH-4 and BH-5). The granular fill layer extended to depths ranging between 0.8 m to 1.1 m below road surface with elevations ranging between 214.5 m to 214.7 m. The explored thickness of this layer was between 0.5 m to 0.7 m.

The composition of this fill layer is sand and gravel trace silt, some asphalt inclusion. The material is brown in color, and moist. The SPT "N" values within this layer ranged from 8 to 34 blows per 300 mm penetration, suggesting loose to dense compactness condition.

Laboratory testing performed on selected samples consisted of seven (7) moisture content tests. The test results are as follows:

Moisture Content:

- 3% to 5%

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

1.4.4 Fill: Silty Clay

A layer of silty clay fill was encountered in all boreholes below granular fill in boreholes drilled from road surface (BH-2, BH-4 and BH-5) and below topsoil in boreholes drilled off-road (BH-1 and BH-3). The silty clay fill extended to depths ranging between 1.2 m to 1.5 m below ground surface in off-road boreholes with elevations ranging between 208.2 m to 208.6 m. The explored thickness of this layer in off-road boreholes was between 1.0 m to 1.3 m. In boreholes drilled from road surface it is extended to depths ranging between 3.8 m to 4.6 m below road surface with elevations ranging between 210.9 m to 211.9 m. The explored thickness of this layer was between 2.7 m to 3.8 m.

The composition of this fill layer is clay and silt and trace to few sand. The material is brown in color, and moist. The SPT “N” values within this layer ranged from 4 to 8 blows per 300 mm penetration, suggesting firm consistency.

Laboratory testing performed on selected samples consisted of sixteen (13) moisture content, four (2) grain size distribution and three (2) Atterberg Limit tests. The test results are as follows:

Moisture Content:

- 20% to 31%

Grain Size Distribution:

- 0% gravel;
- 6% to 9% sand;
- 46% to 47% silt; and
- 44% to 48% clay

Atterberg Limits:

- Liquid Limit: 36% to 43%
- Plastic Limit: 18% to 20%
- Plasticity Index: 18% to 23%

The results of the moisture content, grain size distribution and Atterberg Limits tests are provided on the record of borehole sheets in Appendix C. The results of the grain size distribution tests and Atterberg Limits tests are also provided on Figure 3, and 7 in Appendix D.

1.4.5 Fill: Silty Sand

A layer of silty sand fill was encountered below silty clay fill in boreholes drilled from road surface. The silty sand fill extended to depths ranging between 5.3 m to 6.1 m below ground surface with elevations ranging between 209.4 m to 210.3 m. The explored thickness of this layer was between 0.7 m to 2.3 m.

The composition of this fill layer is silt and sand, trace to some clay, trace to some gravel, occasional cobbles. The material is brown in color, and moist. The SPT “N” values within this layer ranged from 17 to 22 blows per 300 mm penetration, suggesting compact relative density. One SPT “N” value within

this layer in BH-5 recorded to be 100 blows per 300 mm penetration, this could be influence of presence of cobbles in the fill.

Laboratory testing performed on selected samples consisted of sixteen (4) moisture content, four (2) grain size distribution and three (1) Atterberg Limit tests. The test results are as follows:

Moisture Content:

- 10% to 19%

Grain Size Distribution:

- 12% to 19% gravel;
- 43% to 49% sand;
- 25% to 27% silt; and
- 12% to 13% clay

Atterberg Limits:

- Liquid Limit: 21%
- Plastic Limit: 15%
- Plasticity Index: 6%

The results of the moisture content, gain size distribution and Atterberg Limits tests are provided on the record of borehole sheets in Appendix C. The results of the grain size distribution tests and Atterberg Limits tests are also provided on Figure 4, and Figure 8 in Appendix D.

1.4.6 Silty Clay

A native silty clay layer was encountered in all boreholes below silty clay fill layer. The silty clay layer extended to depths ranging between 9.8 m to 15.9 m below ground surface with elevations ranging between 199.6 m to 200.3 m. All the boreholes were terminated within this layer. The explored thickness of this layer was between 8.3 m to 10.6 m.

The composition of this layer is clay and silt, trace to some sand, some organic fibers. The material is grey to brown in color, and moist to wet. The SPT "N" values within this layer ranged from 0 to 10 blows per 300 mm penetration, suggesting very soft to stiff in consistency. In addition, in situ shear vane tests were performed and field results ranged between about 28 kPa to 91 kPa.

Laboratory testing performed on selected samples consisted of thirty-six (36) moisture content, eleven (11) grain size distribution and eleven (11) Atterberg Limit tests. The test results are as follows:

Moisture Content:

- 19% to 40%

Grain Size Distribution:

- 0% to % gravel;

- 0% to 20% sand;
- 46% to 79% silt; and
- 21% to 44% clay

Atterberg Limits:

- Liquid Limit: 23% to 47%
- Plastic Limit: 17% to 26%
- Plasticity Index: 3% to 25%

The results of the moisture content, grain size distribution and Atterberg Limits tests are provided on the record of borehole sheets in Appendix C. The results of the grain size distribution tests and Atterberg Limits tests are also provided on Figure 1, 2, 5 and 6 in Appendix D.

1.5 Groundwater and Surface Water Conditions

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

Table 1.1. Groundwater data

Borehole	Date Completed	Date Measured	Ground Surface Elevation ²	Depth to Water ³	Groundwater Elevation
BH-1	Oct. 14/16	Oct. 14/16	210.1	5.8	204.2
BH-2	Oct. 14/16	Oct. 14/16	215.6	Dry	Dry
BH-3	Oct. 14/16	Oct. 14/16	209.4	4.0	205.4
BH-4	Oct. 14/16	Oct. 14/16	215.5	8.2	207.3
BH-5	Oct. 14/16	Oct. 14/16	215.7	9.0	206.7
Stream WL Upstream (North) Side	--	Oct. 14/16			209.4 ⁴
Stream WL Downstream (South) Side	--	Oct. 14/16	--	--	208.9 ⁴
Notes: 1) All units in metres. 2) Elevations surveyed are referenced to a temporary benchmark (TBM) set on (MTO # 92 concrete post, see photograph 10 in Appendix A) approximately 120 m east of the culvert alignment on south of highway. The TBM elevation (216.3 m) is assumed based on the information provided on site plan drawings provided by the MTO. 3) Depths are relative to ground surface. 4) Indicates top of surface water elevation at culvert location.					

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 could exist in the embankment fill as well.

1.6 Chemical Analyses

One soil sample was selected for chemical analyses and was sent via courier, in a secure cooler under chain of custody, to AGAT Laboratories., a CALA-certified and accredited laboratory in Mississauga, Ontario. The analytical laboratory results are presented in Appendix D, and are summarized in Table 1.2, below.

Table 1.2. Corrosivity chemical analysis

Sample Identification	pH (unitless)	Soluble Chloride (ppm)	Soluble Sulphate (ppm)	Resistivity (ohm-cm)	Conductivity (mS/cm)	Redox Potential (mV)
BH3-SS2 Silty Clay Fill	7.98	62	16	4,480	0.223	273

Part II: ENGINEERING DISCUSSIONS AND RECOMMENDATIONS

2.1 General

This section of the report provides geotechnical design recommendations for replacement of the existing culvert, located on Highway 3 at Station 16+070, about 4.2 km east from Ireland Road, Simcoe to Renton, the Ministry of Transportation (MTO) Western 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, the existing culvert is a 2.8 m × 1.5 m × 37.9 m concrete rigid frame-open footing structure. It is understood that the existing culvert would be replaced with a new culvert along the same alignment with minimum grade change is anticipated at the culvert location. It is also understood that a precast box culvert with an estimated span of between 4 m and 5 m, and 41.5 m long will likely be considered to replace the existing culvert. However, for preliminary design purposes, the following options are being considered for the replacement in this report: rigid frame box culvert (precast or cast-in place), rigid frame open footing culvert, corrugated steel plate culvert, and steel sheet pile abutments and 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 September 12, 2016. The assessment involved review of options for replacement of the existing culvert along the same alignment.

2.2 Expected Ground Conditions

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

- a. Hwy 3 is a two lanes roadway and is about 7.0 m wide from edge of pavement to edge of pavement, with narrow sand and gravel shoulders and cable guide rails on both sides of the roadway. Based on drawings provided, the roadway embankment is about 6.5 m high with side slopes about 2H:1V at the culvert inlet and outlet. The current elevation of the crest of the roadway is about 215.7 m.
- b. The highway embankment consists of granular fill (0.5 m to 0.7 m thick) underlain by firm silty

- clay fill (~2.7 m to 3.8 m thick) underlain by compact silty sand fill (~ 0.7 m to 2.3 m thick).
- c. The embankment fill is underlain by native firm to very soft silty clay (~9.8 m to 10.6 m thick). The boreholes drilled from the road surface were terminated within this layer.
 - d. At the inlet and outlet, the layer of topsoil (~0.125 m to 0.2 m thick) underlain by soft to firm silty clay fill (1.0 m to 1.3 m thick) and followed by firm to very soft silty clay (8.3 m to 8.6 m thick). The boreholes drilled at inlet and outlet locations were terminated within this layer.
 - e. The foundation soil at the invert of the new culvert is anticipated to be native soft to firm silty clay at about Elev. 208 m. Typical 'N' values ranged from 3 to 10.
 - f. At the time of investigation, the approximate stream water elevation at the inlet and outlet was about 209.4 m to 208.94 m, respectively. The groundwater table in the embankment fill is expected to be at approximate elevation 209.4 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

From the TOR it is understood that the precast frame box culvert will likely be considered for the replacement. However, for preliminary design purpose, several other possible options are being considered for the replacement:

- Rigid frame box culvert (precast or cast-in-place),
- 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 silty clay 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.

It is noted that regardless of the option selected, the existing 2.8 m × 1.5 m × 37.9 m concrete rigid frame-open footing culvert is to be removed. This will require excavation down to the existing founding elevation for all options. This suggests the need for groundwater control as discussed in Section 2.9 below.

The spots of any loose and/or soft soils encountered below the existing embankment should be excavated and removed to firm bearing of native soils and grade restored with engineered fill. If the depth of excavation to remove unstable soils is excessive, using a geotextile fabric, such as Terrafix 270R or equivalent, in conjunction with engineered fill can be considered to assist in providing a stable base for the new culvert. Based on previous experience, typically a minimum of 450 mm of a clear stone 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.

Based on the subsoil condition, Table 2.1 below compares the possible structure options from a foundations design and constructability perspective with their advantages and disadvantages.

Although the foundation soils can provide adequate support for all options listed in the table, the use of precast rigid frame box culvert is recommended.

Table 2.1 Evaluation of foundation alternatives

Options	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Precast 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"> ▪ If floor is thin and poorly reinforced, it may heave and crack ▪ During high flows, the concrete floor can be undermined ▪ Require bedding material 	<ul style="list-style-type: none"> ▪ Low 	<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
Cast-in-place rigid frame box culvert	4	<ul style="list-style-type: none"> ▪ Suitable if site is not conducive to heavy equipment for installation of precast sections ▪ Reduce excavation depth 	<ul style="list-style-type: none"> ▪ Slower construction process ▪ If floor is thin and poorly reinforced, it may heave and crack ▪ During high flows, the concrete floor can be undermined ▪ Required concrete curing 	<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
Rigid frame open footing concrete culvert	5	<ul style="list-style-type: none"> ▪ Wider span may consider to maintain existing channel ▪ High geotechnical resistance available 	<ul style="list-style-type: none"> ▪ Deeper excavation or below water excavation may required ▪ Dewatering system may required ▪ 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 if excavation require below water ▪ Risk of Scour
Corrugated Steel Pipe Culvert	3	<ul style="list-style-type: none"> ▪ Straightforward construction ▪ Reduce construction period, consequently 	<ul style="list-style-type: none"> ▪ Require bedding material ▪ Limited design life 	<ul style="list-style-type: none"> ▪ Low to medium 	<ul style="list-style-type: none"> ▪ Risk of unacceptable differential settlements if the entire foundation

Options	Rank	Advantages	Disadvantages	Relative Costs	Risks/ Consequences
		traffic management and water control period ▪ Reduce excavation depth	▪ Potential for Corrosion		is not supported on the competent soil ▪ Risk of structure segment loss due to corrosion
Steel Sheet Pile abutment with precast decking	2	▪ Higher geotechnical resistance value ▪ Environmentally friendly ▪ Easy to construct ▪ No need for dewatering and cofferdam ▪ serve as dual purpose of support culvert foundation and retaining backfill ▪ reduce construction period	▪ Require pile driving equipment ▪ May require anchors to support possible later movement ▪ Durability issue with sheet pile walls	▪ Medium to High	▪ May be limited steel sheet pile sections ▪ Risk of potential settlement

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 and CSP pipe culvert	~208.0 or below	5	Minimum 300 mm compacted granular material (Granular A or Granular B Type II) over native firm silty clay	225	150
Rigid frame open footing concrete culvert	~207.3*	1.0	Native firm silty clay	225	150

Notes:

*Below the frost line

** for maximum settlement of 25 mm

It is presumed that underlying organic fibers 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 and pre-cast concrete	Coefficient of friction ($\tan \delta$)=0.7
Between cast-in-place concrete and native silty clay	Coefficient of friction ($\tan \delta$)=0.5

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

2.3.1.3 Frost Protection

The frost depth in the area of the culvert is estimated to be approximately 1.2 m in accordance with OPSD 3090.101. During construction of any temporary and permanent support system using shallow foundations should be provided a minimum 1.2 m of soil cover or equivalent frost protection should be provided using thermal insulation. This frost protection requirement applies to the rigid frame open footing culvert option only. Frost protection is not required for the box culvert.

If the frost penetration line is at or above top of the culvert the backfill and cover for these culverts should be as per OPSD 803.010. Where less than 1.2 m of earth cover is provided above the top of the culvert, a frost taper should be included as per OPSD 803.010 for the concrete culverts with spans less than or equal to 3.0 m or OPSD 3101.150 for the culvert with span more than 3.0 m.

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 230 kN/m width of sheet pile for 10 m embedment (i.e., approximate tip elevation of 199 m). This value is based on a static analysis, considering skin friction only (end bearing resistance is negligible), using the effective stress β method. 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 firm 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, sheet pile walls 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_o)	Unit Weight γ kN/m ³
Granular Fill	32	0.31	3.25	0.47	21
Silty Clay Fill (firm to stiff)	29	0.35	2.88	0.52	19
Silty Clay (firm to stiff)	29	0.35	2.88	0.52	19
Silty Clay (very soft to soft)	25	0.40	2.46	0.58	18

2.5 Seismic and Liquefaction Potential Consideration

Seismic characterization of the site must be compliant with the Canadian Highway Bridge Design Code CHBDC (CAN/CSA-S6-14). The potential for seismic loading must be considered for design of abutment in accordance with Section 4.4 of the CHBDC with respect to soil conditions encountered at the site. Table 4.1 in CHBDC (see Clause 4.4.3.2) shows site classification for seismic site response based on soil average properties in top 30 m. The borehole information shows the presence of native firm to very soft silty clay. Based on these soil characteristics, the site class for this site is estimated to be Class "E" according to Table 4.1.

From the Natural Resources Canada website, 2015 NBCC seismic hazard values are obtained using the site location coordinates (46.2925° N, 84.0353° W) and the damped reference spectral accelerations for the project site are $S_a(0.2)=0.025g$, $S_a(0.5)=0.020g$, $S_a(1.0)=0.012g$, $S_a(2.0)=0.0052g$ and the reference peak ground acceleration (PGA) is $0.013g$ (g =acceleration due to gravity -9.81 m/s^2). These values are associated with an earthquake having 10 percent probability of exceedance in a 50-year period.

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

2.6 Construction Alternatives

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

1. 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 a steel sheet pile abutment with precast concrete deck system
2. Full road closure followed by open cut/unsupported excavation to replace culvert;
3. Construct temporary detour embankments at the site followed by open cut/unsupported excavation to expose and replace culvert;

All methods considered utilize a cut and cover approach for culvert replacement which allows 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 type of the precast box 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 summarize 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 G.

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

Installation Method	Advantages	Disadvantages	Relative Cost	Ranking
<p>OPTION 1.A</p> <p>Half-and-half Construction with Unsupported Cut Sides (Figure H3.A, Appendix G)</p>	<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 6.5 m high required to maintain one lane of traffic • High cost of roadway protection system • Large amount of soil to be excavated • Need to temporarily control existing creek water • Risk of cost overrun and inability to finish job: low to moderate 	<p>Relatively more expensive than full road closure due to high costs of roadway protection system</p>	1
<p>OPTION 1.B</p> <p>Half-and- half Construction with Braced or Anchored Cut Sides (Figure H3.B, Appendix G)</p>	<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 6.5 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 • Need to temporarily control existing creek water • Risk of cost overrun and instability to finish job: low to moderate 	<p>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</p>	3

Installation Method	Advantages	Disadvantages	Relative Cost	Ranking
<p>OPTION 1.C</p> <p>Half-and-half Construction with Installation of Steel Sheet Pile Abutments with Precast Concrete Deck (Figure H3.C, Appendix G)</p>	<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 • Due to possible lateral movement need an anchor system, bracing or deadman • Durability issue with sheet pile walls • Risk of cost overrun and inability to finish job: low to moderate 	<p>Relatively more expensive than full road closer due to high costs of shoring abutments, but more practical</p>	<p>2</p>
<p>OPTION 2</p> <p>Full Road Closure using Existing Roadways and Open Cut Unsupported Excavation (Figure H1, Appendix G)</p>	<ul style="list-style-type: none"> • Existing culvert will completely remove and replaced with new culvert • 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 existing 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 	<p>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</p>	<p>4</p>

Installation Method	Advantages	Disadvantages	Relative Cost	Ranking
<p>OPTION 3</p> <p>Temporary Local Detour and Open Cut Unsupported Excavation</p> <p>(Figure H2, Appendix G)</p>	<ul style="list-style-type: none"> • Traffic flow maintained at the site during construction • Simple detour roads can be constructed • Existing culvert will completely remove and replaced with new culvert • No excavation support required • Install entire new culvert at once 	<ul style="list-style-type: none"> • Construction of detour embankments required at one side of highway • Possible extra cost to purchase of private property • Possible settlement due to new earth embankment fill • Increased time for construction of detour • Large amount of soil to be excavated • Erosion control of temporary cuts required • Need to temporarily control existing creek water • Risk of cost overrun and inability to finish job: low to moderate • Possible extra cost to purchase of private property 	<p>More expensive than full road closure due to high costs to build local detours</p>	<p>5</p>

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

The following sections discuss these options in more details.

2.6.1 Half-and-Half Construction (Options 1)

The half-and-half construction method could be utilized to maintain the flow of the traffic on Hwy 3 (see Figures H3.A, H3.B and H3.C, Appendix G). In this 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, 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 6.5 m deep. Therefore, temporary shoring such as a soldier pile and lagging 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 steel sheet pile abutments with precast concrete deck system

Option 1.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 1.B will disrupt less of the embankment but would cost more, i.e. about 1.8 times of Option 1.A. Excavation and backfilling operations will also

be more challenging with Option 1.B. Both options require decommissioning of shoring system upon completion of the work.

2.6.1.1 Option 1.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 G). 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 1.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 1.B and 1.C since it needs to excavate a large amount of soil.

2.6.1.2 Option 1.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 G). 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 1.A, decommissioning of the shoring system and temporary surface water flow control must be performed/developed by contractor.

Option 1.B will disrupt less of the embankment than Option 1.A but would cost more, i.e. about 1.8 times of Option 1.A, due to the cost of shoring system. Excavation and backfilling operations will also be more challenging with Option 1.B. Both options require decommissioning of shoring system upon completion of the work.

2.6.1.3 Option 1.C: Half-end-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 G). 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 lake water flow below Hwy 3 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.

2.6.2 Detour Options (Options 2 and 3)

Both detour options, the option with full closure of Hwy 3 and long detours around the area using existing roadways (see Figure H1, Appendix G), 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 G), allow for open cut, unsupported excavation to facilitate the replacement of the existing culvert. A major benefit of these options is that the existing culvert will be completely removed once and replaced new culvert. The other advantages are 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 existing 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 and gravel fill and silty clay 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. It is expected that most of excavations will be above the groundwater levels except those at the invert level. 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).

Cobble fragments were encountered in the embankment fill during borehole investigation, therefore, a Non-Standard Special Provision (NSSP) to alert the contractor about the presence of cobbles and/or even boulders in the embankment fill is included in Appendix H.

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).

2.7 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.

At this site shoring system such as steel sheet pile walls and soldier piles and timber lagging may be considered for design. It should be designed based on the earth pressures coefficients and soil parameters provided in Section 2.4. For design of the timber lagging, earth pressures can be reduced by 25 percent to account for soil arching effects. This is provided that the center-to-center spacing of the soldier piles does not exceed 2.5 m. Temporary shoring system with additional anchorage or tiebacks may be required for lateral resistance. Conventional practice is to incorporate either buried deadman anchors or soil grouted anchors. Deadman anchors or soil anchors can be designed as indicated in section 2.3.2.2 of this report.

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.

2.8 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 F provide the bedding, embedment, cover and backfill standards for the different culvert material. According to these standards the culvert bedding should consist of Granular

A (OPSS.PROV. 1010) with thickness of 300 mm beneath the culvert and extend a minimum of 500 mm horizontally on either side of the culvert edge. The bedding material should be placed in layers not exceeding 200 mm in thickness, loose measurement, and compacted accordance with OPSS 501 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 1.2 m can occur in open, unheated areas without snow cover. At the culvert inlet and outlet, and beneath the proposed culvert, mostly the native soils consist of silty clay. This material has medium to 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.

2.9 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 accordance with OPSS 501. 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 1.2 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.

2.10 Groundwater and Surface Water Control

The soils encountered below the groundwater table and within potential excavation depths consist of native silty clay. 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.

Provided that the existing culvert is to remain in use during construction of the new culvert, the majority of the upstream flow of the existing culvert can be diverted around the construction area. For the control of the water flow in the creek might require a cofferdam. If the existing culvert is to be removed prior to completion of the new culvert, a system of sumps and pumps will be required to divert the surface water up and over the existing embankment.

Dewatering requirements behind the cofferdams to keep the construction site dry will be impacted by water levels in the stream 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.

2.11 Embankment Design

2.11.1 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 slopes of 2H:1V was established based on **exp's** survey data and the cross-section provided by MTO. The stratigraphy and groundwater condition at the site were developed based on the results of the geotechnical investigation presented in Part I - Foundation Investigation Report.

Based on the borehole information, the subsoils encountered at the work area consist of embankment fill, underlain by silty clay deposits. Therefore, an effective stress analysis for a long term and total stress for short term stability assessment of the embankment slope was performed taking into consideration the subsoil conditions encountered beneath the existing embankment.

The SLOPE/W graphical printout, for analysis performed is included in Appendix E. Since the geometry and soil stratigraphy at the north and south side slopes are similar, the result of the slope analysis performed for the south side slope, is only presented.

Tabulated below in Table 2.6 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 2.6 Soil properties used in slope stability analysis

Soil Type	Short-term Conditions			Long-term Conditions		
	ϕ (degrees)	c (kPa)	γ (kN/m ³)	ϕ' (degrees)	c' (kPa)	γ' (kN/m ³)
Sand and Gravel Fill	32	0	21	32	0	21
Silty Clay Fill	0	80	19	29	0	19
Silty Sand Fill	27	0	19	27	0	19
Silty Clay (firm to stiff)	0	60	19	28	0	19
Silty Clay (very soft to soft)	0	25	18	25	0	18

The results of slope stability analyses for the 2H:1V south side slope of the existing embankment using undrained (short term stability) and drained (long term stability) soil parameters are presented graphically in Figure 1 and 2 in Appendix E, respectively. For the drained soil conditions, a minimum Factor of Safety of 1.4 was calculated, while for the undrained soil conditions, a minimum Factor of Safety of 1.5 was calculated, indicating that the embankment is stable. These preliminary results of slope analyses suggest that the new embankment should be constructed with a minimum slope of 2H:1V to achieve a minimum Factor of Safety of 1.3.

2.11.2 Embankment Settlement

It is not planned to change significantly the existing embankment grade at the culvert location. However, the new culvert will be slightly longer than existing culvert (~3.6 m longer) so the additional fill will be placed. The total settlement is predicted using a computer program, Settle3D (Rocscience). Settle3D is a 3-dimensional program for the analysis of vertical consolidation and settlement under foundations, embankments and surface loads. The program combines the simplicity of one-dimensional analysis with the power and visualization capabilities of more sophisticated three-dimensional programs.

The magnitudes of total settlement for approach embankment have been assessed based on Standard Penetration Test (SPT) results with in-situ field vane shear testing, and site-specific correlations between consolidation parameters and the natural moisture content of samples obtained. Soil compressibility parameters adopted in the settlement analyses for the approach embankment is summarised in Table 2.7.

The total grade raise was assumed to be about maximum 1 m for this analysis.

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

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

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

Based on the results of moisture content tests, natural moisture contents of the clayey silt soil was between 20% and 40%. Therefore, the estimated the estimated compression index using the empirical equation is between 0.15 and 0.35.

Table 2.7. Soil parameters used in settlement analyses

Soil Layers	Unit Weight (kN/m ³)	E (MPa)	Compression Index (C_c)	Recompression Index (C_r)	Void Ratio (e)	Preconsolidation Pressure (p'_c) (kPa)
Engineered Fill	21	30	-	-	-	-
Approach Embankment						
Silty Clay Firm to Stiff	19	30	0.25	0.025	0.95	200
Silty Clay Very Soft	18	-	0.3	0.32	1.1	100

The settlement analyses suggested that the total settlement and immediate settlement could be in the order of 30 mm and 6 mm, respectively at the locations of approach embankments. It is expected that post construction settlement will be within allowable limit at the structure/embankment interface. Settlement of embankment fill itself should also expect depending on the material and placement methods. The settle 3D results of the analysis can be seen in Appendix I.

2.12 Inlet and Outlet

2.12.1 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 F of this report.

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

2.12.2 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 F of this report.

2.12.3 Seepage Cut-off Requirements

The seepage cut-off requirements should be reviewed in the following context. The native silty soils at the inlet, 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.

2.12.3.1 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 F). 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 x (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.

2.12.3.2 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.

2.13 Corrosion Protection

One soil sample was selected for chemical analyses and was sent via courier, in a secure cooler under chain of custody, to AGAT Laboratories., a CALA-certified and accredited laboratory in Mississauga, Ontario. The analytical laboratory results are summarized in section 1.6 of this report and detailed results are included in Appendix D.

Similar to our experience with the soils in the area, the chemical data indicates low to medium resistivity, which indicates a moderately potential for corrosion of buried metallic elements, particularly pipes and appurtenances. The maximum chloride content reported is 62 ppm ($\mu\text{g/g}$) i.e. 0.006% which indicates a low potential for additional corrosion.

The maximum water soluble sulphate content of the soils tested is 16 ppm ($\mu\text{g/g}$), i.e. <0.0016% and being less than 0.10%, does not indicate the potential to corrode normal Portland cement concrete. These data also support our local experience.

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 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 Nimesh Tamrakar, M.Eng and Aziz Abdelmessih.


Yours truly,

exp Services Inc.


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Principal Engineer
Designated MTO Foundation Contact

Encl.



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: South embankment slope looking west from the culvert outlet



Photo 2: Inside culvert looking north from outlet side



Photo 3: South embankment slope looking east from culvert outlet



Photo 4: Culvert inlet looking south



Photo 5: Looking north from culvert inlet



Photo 6: North embankment slope looking east from culvert inlet



Photo 7: North embankment slope looking west from culvert inlet



Photo 8: Looking east from culvert location

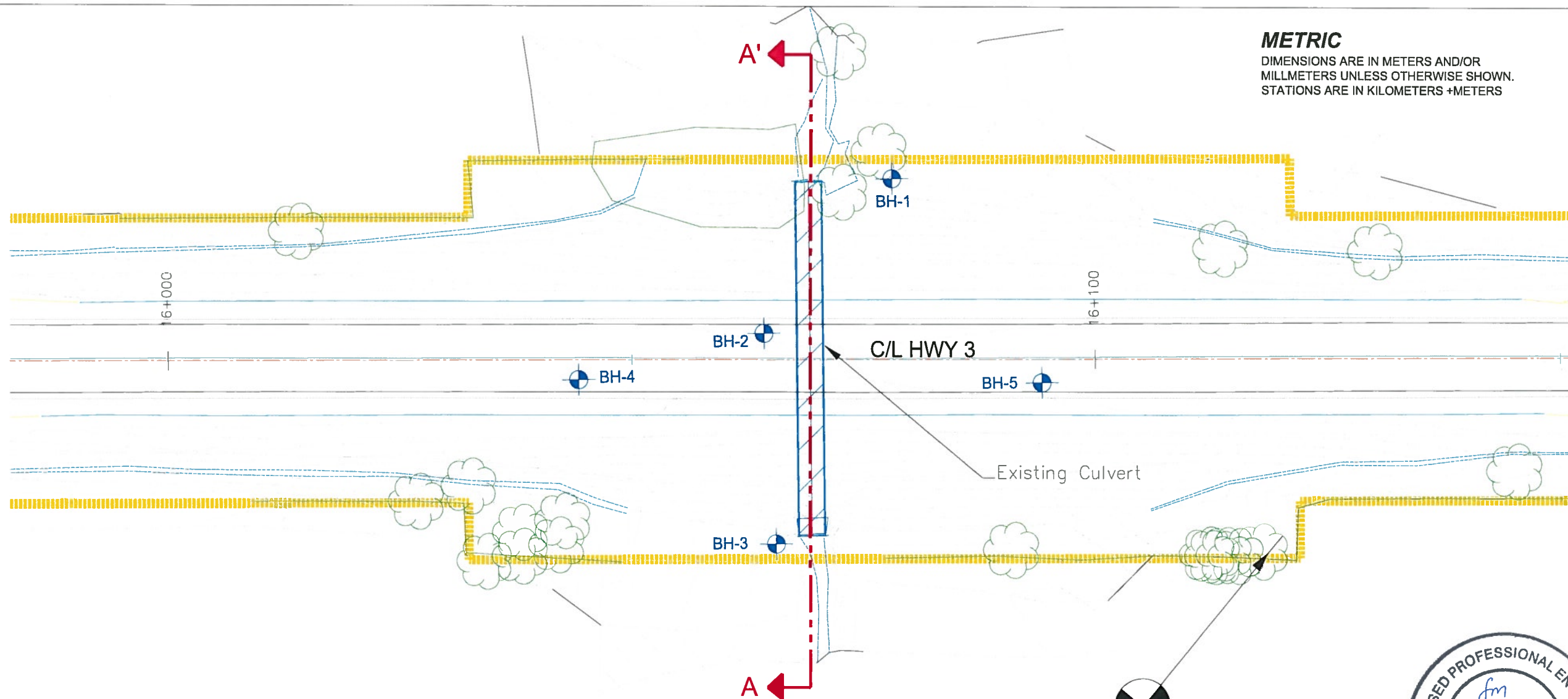


Photo 9: Looking west from culvert location

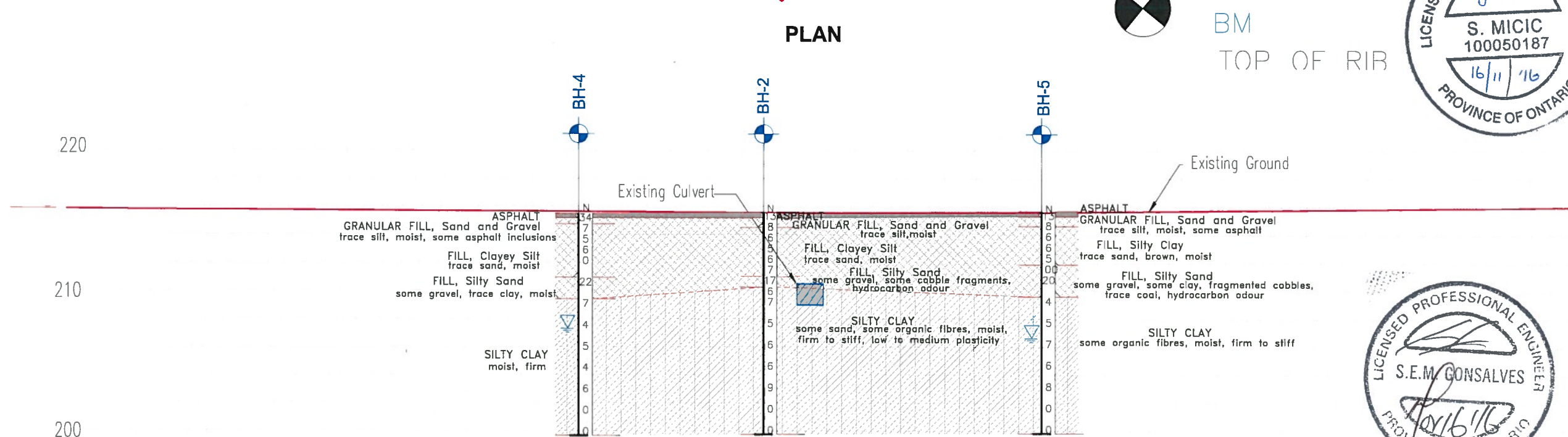


Photo 10: Temporary benchmark on MTO control point# 92

Appendix B – Drawings



PLAN



PROFILE ALONG C/L HWY 3

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

Agreement No. 3015-E-0017
Assignment No. 1
WO



**CULVERT REPLACEMENT-
STATION 16+070 HWY 3- SIMCO TO RENTON
BOREHOLE LOCATION PLAN AND PROFILE**

SHEET
1

exp Services Inc.

KEY PLAN



- Location of Drilled Boreholes
- Standard Penetration Test (Blows/0.3 m)
- Water Level in Open Borehole

SOIL STRATA SYMBOLS

- ASPHALT
- TOPSOIL
- FILL
- SILTY CLAY

BH No.	APPROX. ELEV.	MTM CO-ORDINATES	
		NORTH	EAST
BH 1	210.1	4746671.32	244790.92
BH 2	215.6	4746652.44	244780.58
BH 3	209.4	4746630.62	244786.26
BH 4	215.5	4746643.69	244761.94
BH 5	215.7	4746653.09	244811.02

NOTE

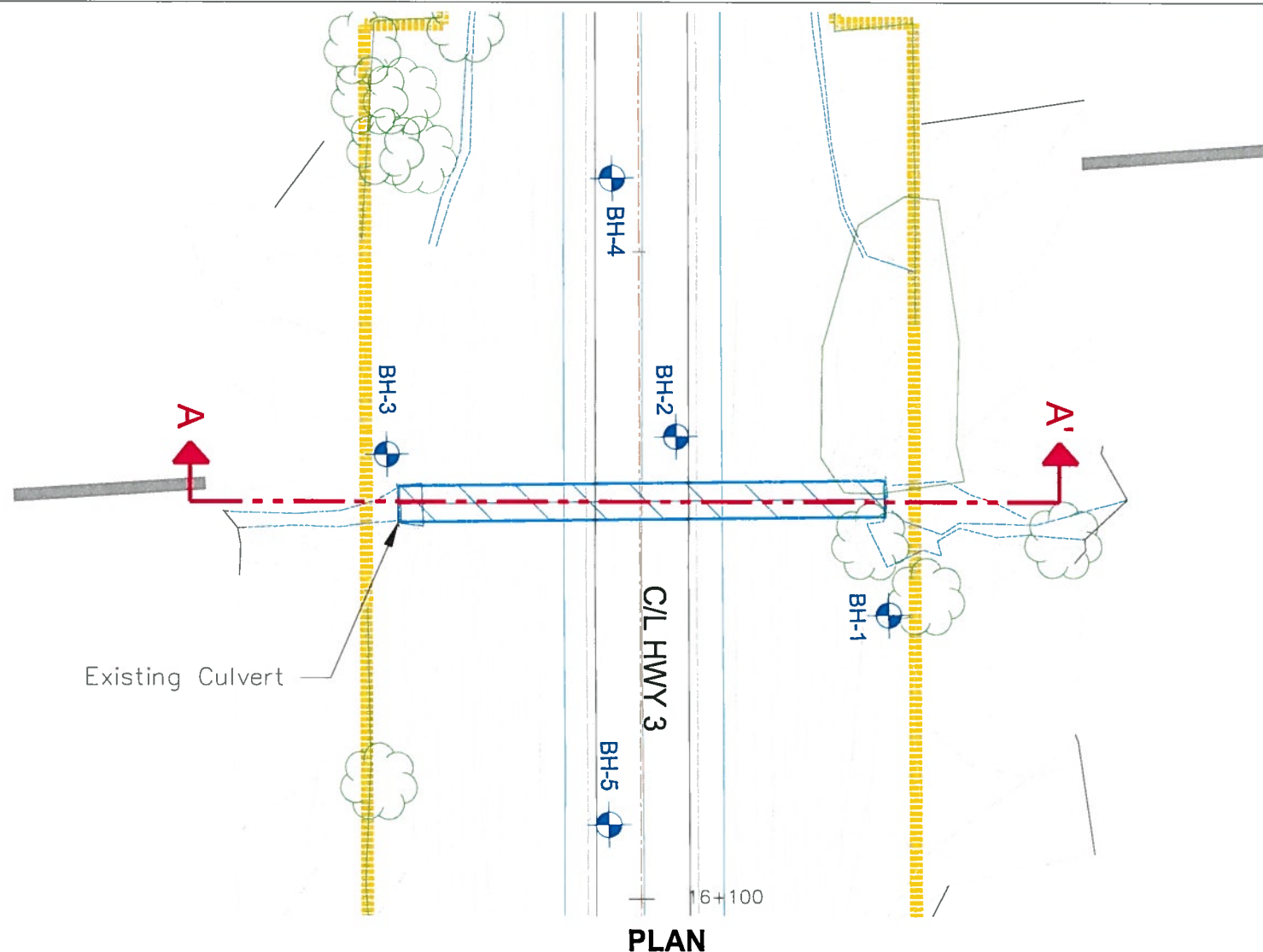
This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

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

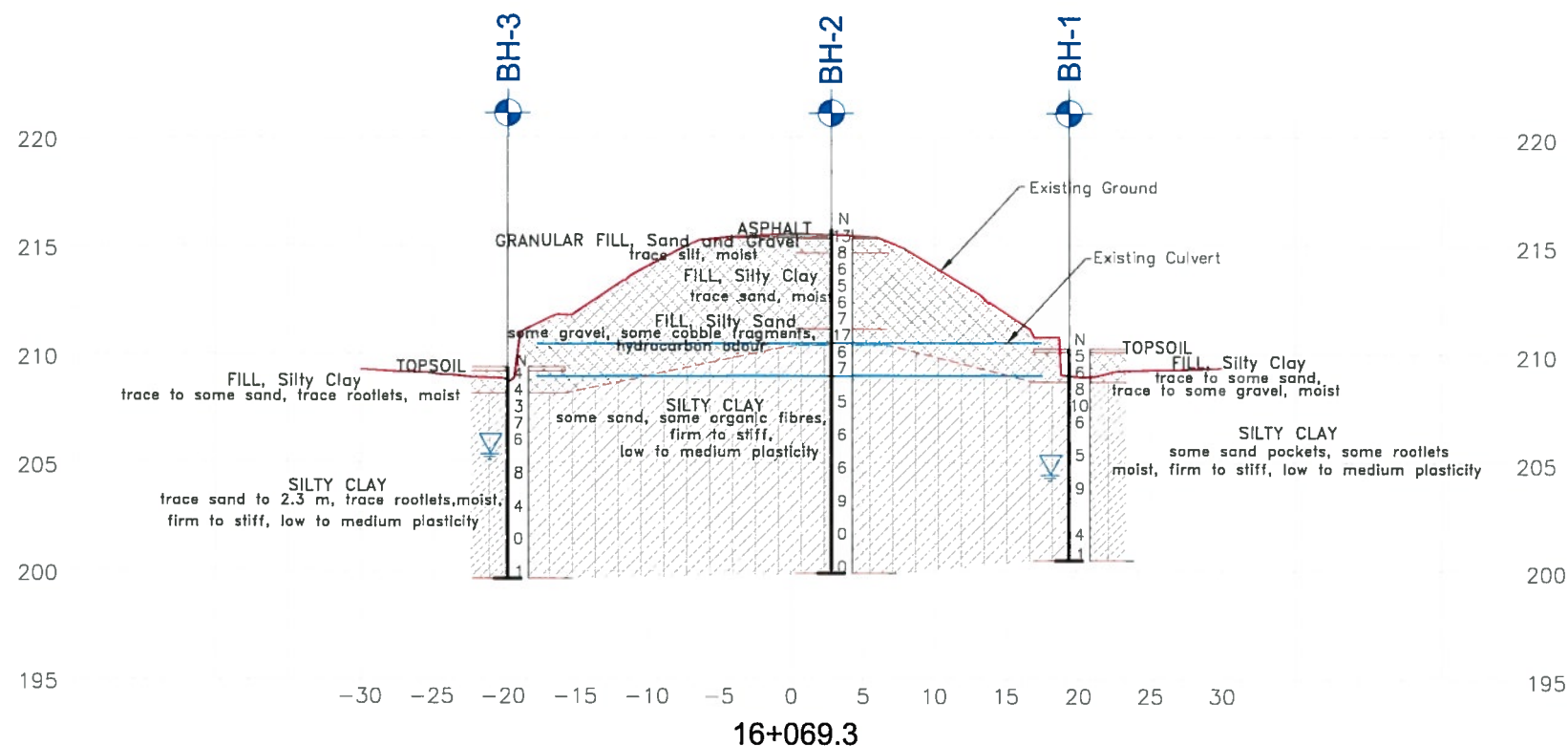


16/11/2016	SM	SUBMISSION FOR MTO REVIEW	
DATE	BY	DESCRIPTION	
		GEOCRES NO. 40116-26	
		PROJECT NO. ADM-00235197-A0	
SUBM'D SM	CHECKED SM	DATE	16/11/2016
DRAWN SH	CHECKED SG	APPROVED	DWG. 1





PLAN



SECTION A-A' Station 16+069.3

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



Agreement No. 3015-E-0017
Assignment No. 1



**CULVERT REPLACEMENT-
STATION 16+070 HWY 3- SIMCO TO RENTON
BOREHOLE LOCATION PLAN AND SECTION**

SHEET
1

exp. **exp Services Inc.**

KEY PLAN



LEGEND

- Location of Drilled Boreholes
- Standard Penetration Test (Blows/0.3 m)
- Water Level in Open Borehole

SOIL STRATA SYMBOLS

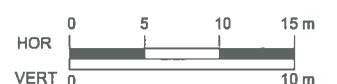
- ASPHALT
- TOPSOIL
- FILL
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16/11/2016	SM	SUBMISSION FOR MTO REVIEW	
DATE	BY	DESCRIPTION	
		GEOCRES NO. 40116-25	
		PROJECT NO. ADM-00235197-A0	
SUBM'D SM	CHECKED SM	DATE	16/11/2016
DRAWN SH	CHECKED SG	APPROVED	DWG. 2

Appendix C – Borehole Logs

Explanation of Terms Used on Borehole Records

SOIL DESCRIPTION

Terminology describing common soil genesis:

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

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

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

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

Terminology describing soil structure:

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

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

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

Fissured: material breaks along plane of fracture.

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

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

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

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

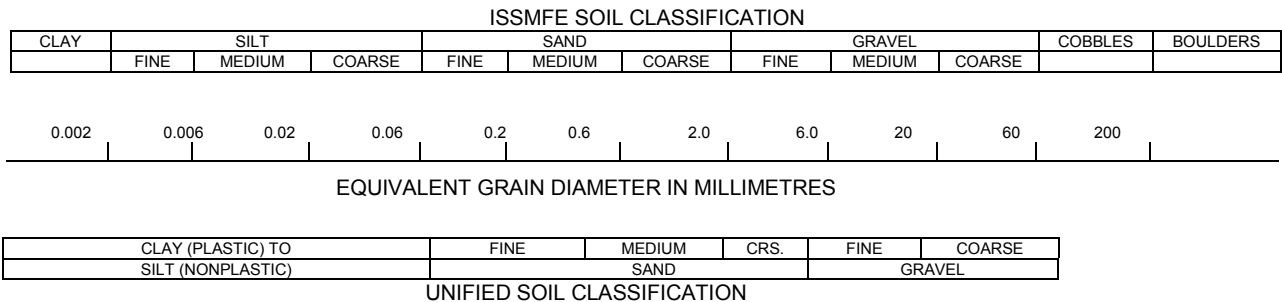
Seam: a thin, confined layer of soil having different particle size, texture, or color from materials above and below.

Homogeneous: same color and appearance throughout.

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

Uniformly Graded: predominantly on grain size.

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



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	$5 \leq P_p \leq 10\%$
Little	$15 \leq P_p \leq 25\%$
Some	$30 \leq P_p \leq 45\%$
Mostly	$50 \leq P_p \leq 100\%$

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:

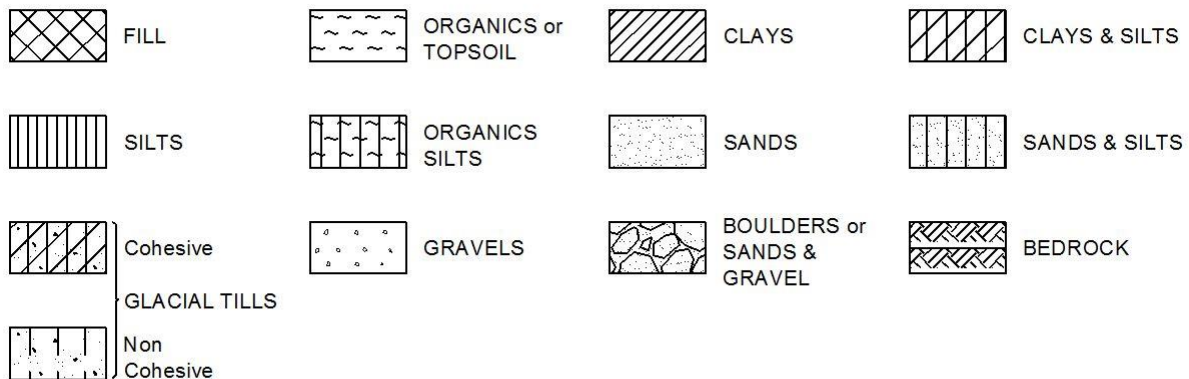
Table c: Consistency of Cohesive Soil

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

Brampton, Ontario

RECORD OF BOREHOLE No BH-1

1 OF 1

METRIC

W. P. 3062-14-00 LOCATION 244790.92 E, 4746671.32 N ORIGINATED BY NT
 DIST Weat BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY JG
 DATUM Geodetic DATE 2016/10/14 - 2016/10/14 CHECKED BY SM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)							
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH: Cu, KPa								WATER CONTENT (%)				GR	SA	SI	CL
								○ UNCONFINED	+ FIELD VANE	×	QUICK TRIAXIAL					LAB VANE							
210.1							210																
210.0	TOPSOIL: (~125 mm thick)		1	SS	5																		
	FILL: silty clay, trace to some sand, trace to some gravel, brown, moist		2	SS	6		209																
208.6																							
1.5	SILTY CLAY: trace sand pockets/veins to 3.1 m, some rootlets, brown and light brown, mottled, moist, firm to stiff, low to medium plasticity		3	SS	8		208																
			4	SS	10																		
	interbedded silt and fine sand seams, grey, wet below 3.1 m depth		5	SS	6		207																
							206																
			6	SS	5		205																
			7	SS	9		204																
							203																
			8	SH			202																
	soft to very soft below 8.2 m depth		9	SS	4																		
							201																
200.3			10	SS	1																		
9.8	End of Borehole at 9.8 m depth. Water level at 5.8 m upon completion of drilling.																						
	Notes: 1. This drawing is to be read with the subject report and project numbers as presented above. 2. Groundwater level was measured in open hole upon completion of drilling.																						

OPG_EXP RECORD OF BOREHOLE BH LOGS MTO.GPJ ONTARIO MOT.GDT 11/16/16

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

Brampton, Ontario

RECORD OF BOREHOLE No BH-2

1 OF 1

METRIC

W. P. 3062-14-00 LOCATION 244780.58 E, 4746652.44 N ORIGINATED BY AA
 DIST Weat BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY JG
 DATUM Geodetic DATE 2016/10/14 - 2016/10/14 CHECKED BY SM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH: Cu, KPa										WATER CONTENT (%)		
								○ UNCONFINED	+	FIELD VANE	×	QUICK TRIAXIAL						LAB VANE		
215.6	ASPHALT: (~430 mm thick)						20	40	60	80	100									
215.2			1	SS	13															
0.4	GRANULAR FILL: sand and gravel, trace silt, brown, moist (~660 mm thick)		2	SS	8															
214.5			3	SS	6															
1.1	FILL: silty clay, trace sand, brown, moist some black organic staining at 1.5 m depth		4	SS	5															
			5	SS	6															
			6	SS	7															
211.0	some sand pockets at 3.8 m depth		7	SS	17															
4.6	FILL: silty sand, some gravel, some cobble fragments, hydrocarbon odour below 4.6 m depth		8	SS	6															
210.3			9	SS	7															
5.3	SILTY CLAY: some sand to 7.6 m, dark grey, some organic fibres, moist, firm to stiff, low to medium plasticity		10	SS	5															
			11	SS	6															
			12	SS	6															
			13	SS	9															
			14	SS	0															
			15	SS	0															
199.8	End of Borehole at 15.9 m depth.																			
15.9	Notes: 1. This drawing is to be read with the subject report and project numbers as presented above. 2. Borehole open to 8.4 m upon completion of drilling. 3. Borehole remained dry upon completion of drilling.																			

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

OPG_EXP RECORD OF BOREHOLE BH LOGS MTO.GPJ ONTARIO MOT.GDT 11/16/16

Brampton, Ontario

RECORD OF BOREHOLE No BH-3

1 OF 1

METRIC

W. P. 3062-14-00 LOCATION 244786.26 E, 4746630.00 N ORIGINATED BY NT
 DIST Weat BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY JG
 DATUM Geodetic DATE 2016/10/14 - 2016/10/14 CHECKED BY SM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)						
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH: Cu, KPa									WATER CONTENT (%)			GR	SA	SI	CL
								○ UNCONFINED	+	FIELD VANE	×	QUICK TRIAXIAL					LAB VANE						
209.4																							
209.2	TOPSOIL: (~200 mm thick)		1	SS	4																		
0.2	FILL: silty clay, trace to some sand, trace rootlets, brown, moist																						
208.2			2	SS	4																		
1.2	SILTY CLAY: trace sand to 2.3 m, trace rootlets, brown, moist, firm to stiff, low to medium plasticity		3	SS	3													0	8	57	35		
	interbedded silt seams, grey, wet below 2.3 m depth		4	SS	7													0	1	63	36		
			5	SS	6																		
			6	SS	8																		
			7	SS	4																		
	very soft below 7.6 m depth		8	SS	0													0	0	56	44		
														</									

OPG_EXP RECORD OF BOREHOLE BH LOGS MTO.GPJ ONTARIO MOT.GDT 11/16/16

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

Brampton, Ontario

RECORD OF BOREHOLE No BH-4

1 OF 1

METRIC

W. P. 3062-14-00 LOCATION 244761.94 E, 4746643.69 N ORIGINATED BY NT
 DIST Weat BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY JG
 DATUM Geodetic DATE 2016/10/14 - 2016/10/14 CHECKED BY SM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH: Cu, KPa											
								○ UNCONFINED		+ FIELD VANE									
								× QUICK TRIAXIAL		LAB VANE									
							20 40 60 80 100				WATER CONTENT (%)								
215.5	ASPHALT: (~330 mm thick)		1	SS	34											GR SA SI CL			
215.2																			
0.3																			
214.7																			
0.8																			
	GRANULAR FILL: sand and gravel, trace silt, brown, moist, some asphalt inclusions (~430 mm thick) FILL: silty clay, trace sand, brown, moist		2	SS	7											0 9 47 44			
			3	SS	5														
			4	SS	6														
			5	SS	6														
210.9	FILL: silty sand, some gravel, trace clay, dark brown, moist		6	SS	22											19 43 25 13			
4.6																			
209.4	SILTY CLAY: brown, moist, firm interbedded silt seams, grey, wet below 9.2 m depth very soft to firm below 13.7 m depth		7	SS	7											0 0 79 21			

+ 3, × 3: Numbers refer to
Sensitivity

○ 3% STRAIN AT FAILURE

OPG_EXP RECORD OF BOREHOLE BH LOGS MTO.GPJ ONTARIO MOT.GDT 11/16/16

Brampton, Ontario

RECORD OF BOREHOLE No BH-5

1 OF 1

METRIC

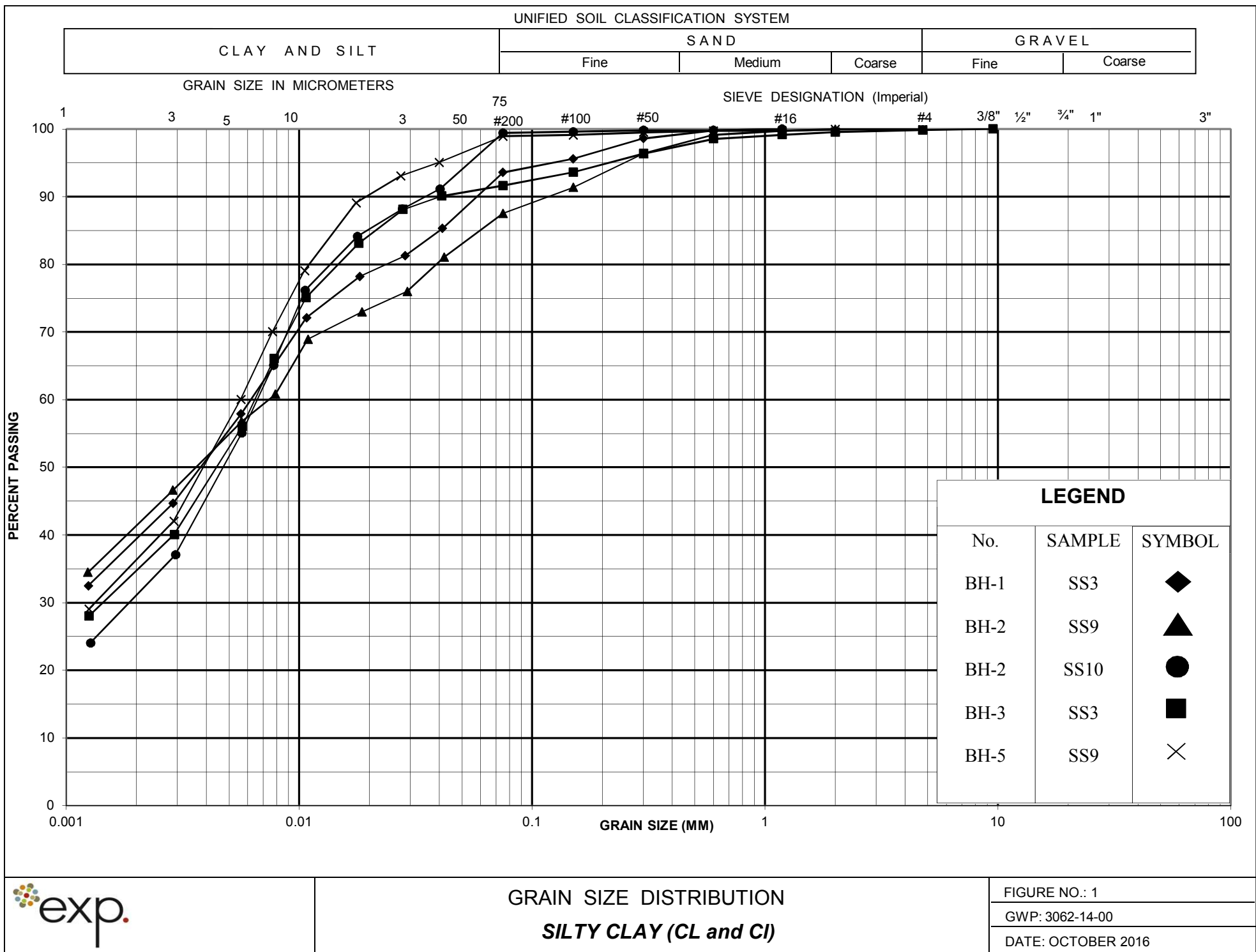
W. P. 3062-14-00 LOCATION 244811.02 E, 4746653.09 N ORIGINATED BY AA
 DIST Weat BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY JG
 DATUM Geodetic DATE 2016/10/14 - 2016/10/14 CHECKED BY SM

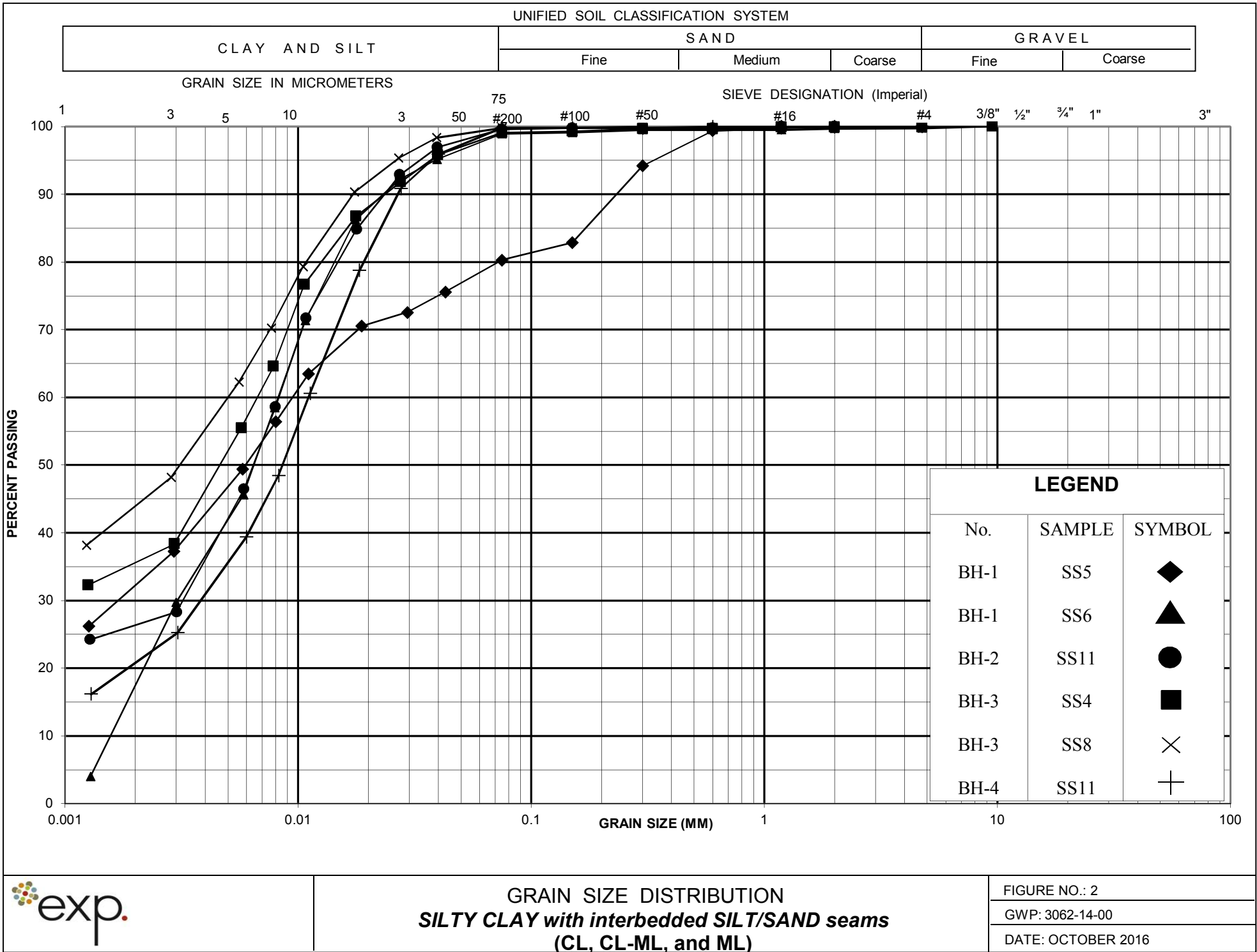
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH: Cu, KPa									WATER CONTENT (%)		
								○ UNCONFINED	+ FIELD VANE	×	QUICK TRIAXIAL						LAB VANE		
215.7	ASPHALT: (~380 mm thick)						20	40	60	80	100								
215.3	GRANULAR FILL: sand and gravel, trace silt, brown, moist, some asphalt inclusions (~685 mm thick)		1	SS	13														
214.6			2	SS	8														
1.1		FILL: silty clay, trace sand, brown, moist		3	SS	6													
				4	SS	6													
				5	SS	5													
211.9	FILL: silty sand, some gravel, some clay, fragmented cobbles, trace coal, hydrocarbon odour split spoon refusal on assumed cobble at 4.1 m depth		6	SS	100														
3.8			7	SS	20														
209.6	SILTY CLAY: dark grey, some organic fibres, moist, firm to stiff		8	SS	4														
6.1		interbedded silt seams, brown, moist below 7.6 m depth		9	SS	5													
grey, wet below 9.2 m depth			10	SS	7														
			11	SS	6														
			12	SS	8														
			13	SS	0														
very soft to stiff below 13.7 m depth			14	SS	0														
199.8	End of Borehole at 15.9 m depth. Water level at 9.0 m upon completion of drilling.																		
15.9																			
Notes: 1. This drawing is to be read with the subject report and project numbers as presented above. 2. Borehole open to 9.2 m upon completion of drilling. 3. Groundwater level was measured in open hole upon completion of drilling.																			

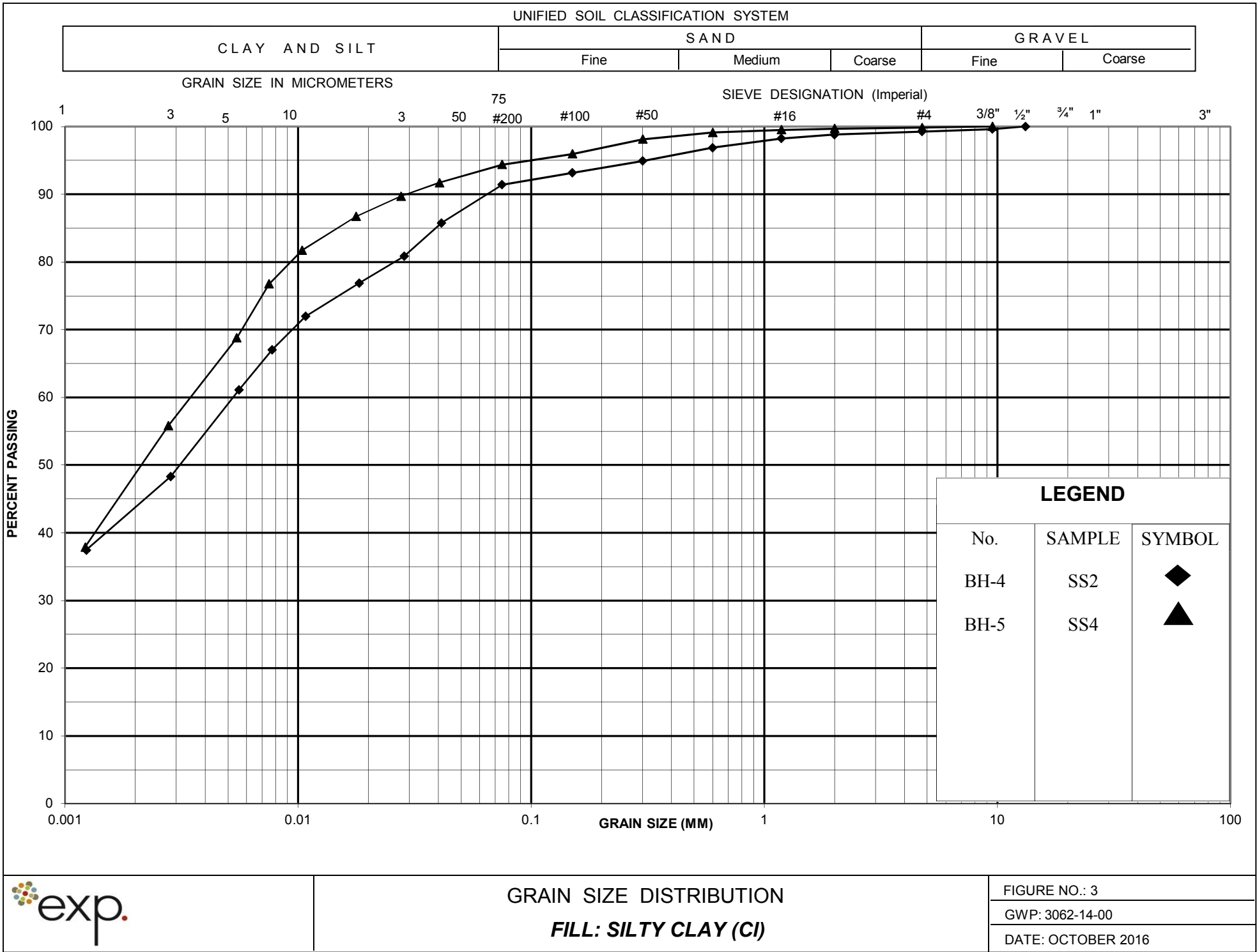
+ 3, × 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

OPG_EXP RECORD OF BOREHOLE BH LOGS MTO.GPJ ONTARIO MOT.GDT 11/16/16

Appendix D – Laboratory Data

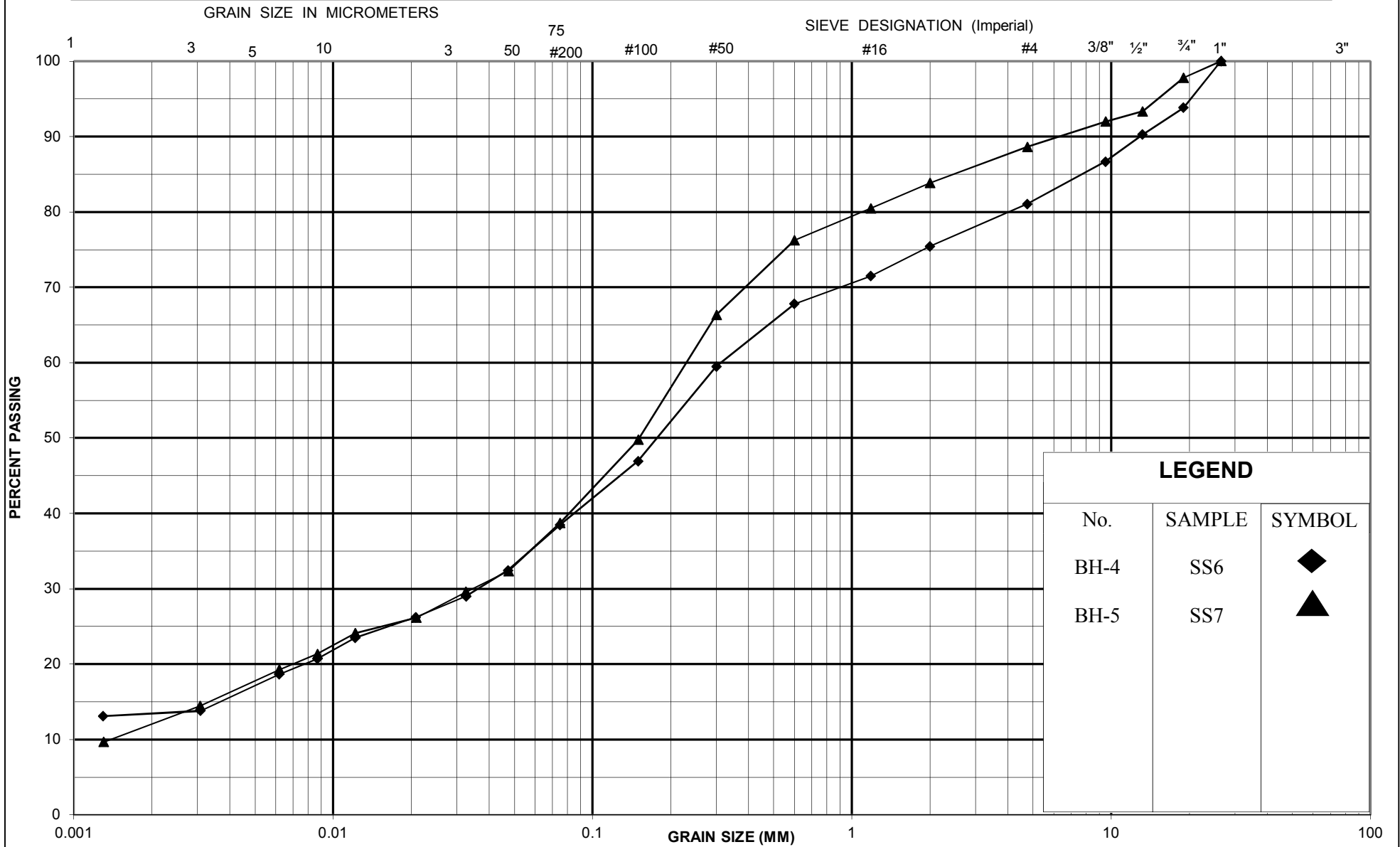






UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY AND SILT	SAND			GRAVEL	
	Fine	Medium	Coarse	Fine	Coarse



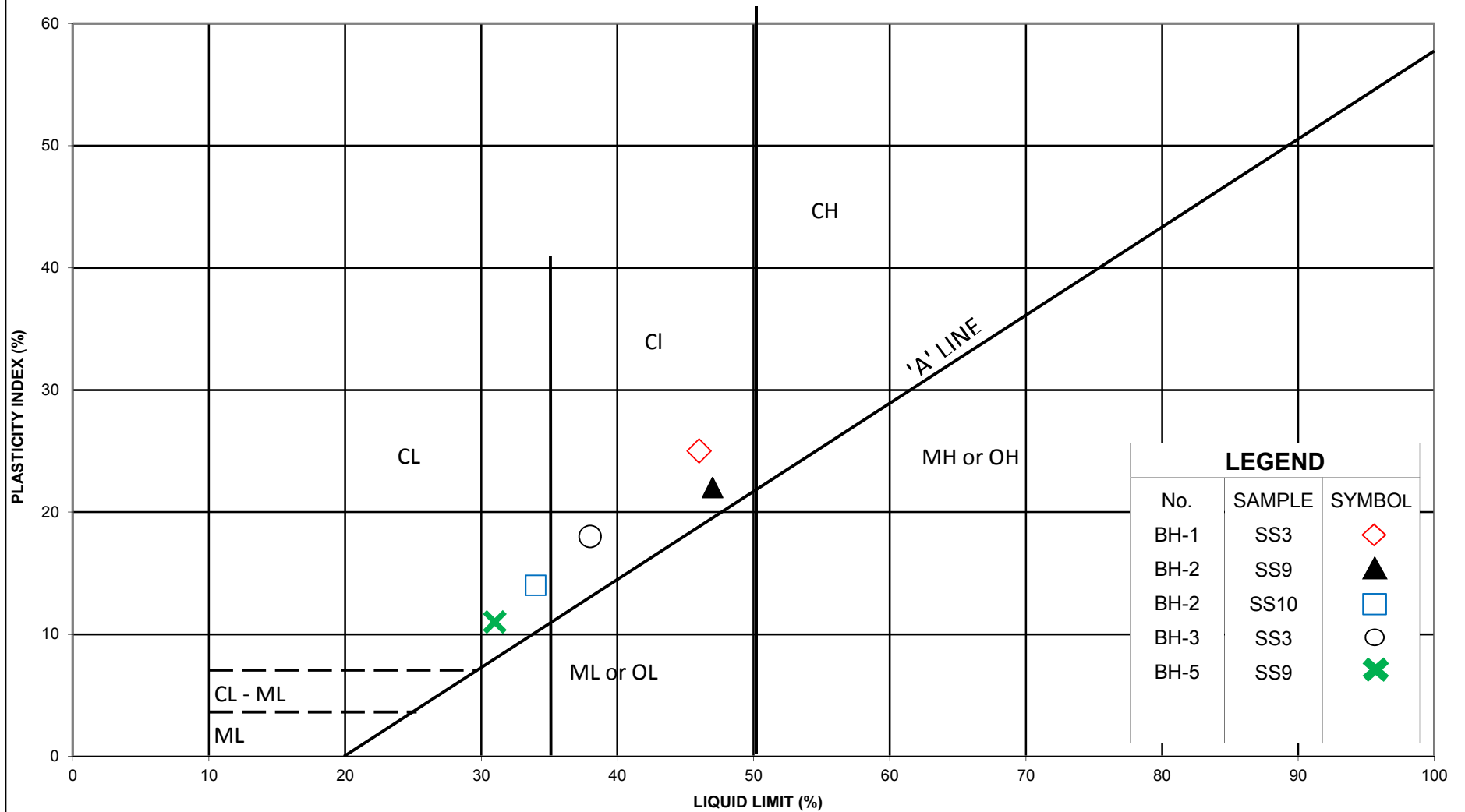
GRAIN SIZE DISTRIBUTION
FILL: SILTY SAND (SM-SC)

FIGURE NO.: 4

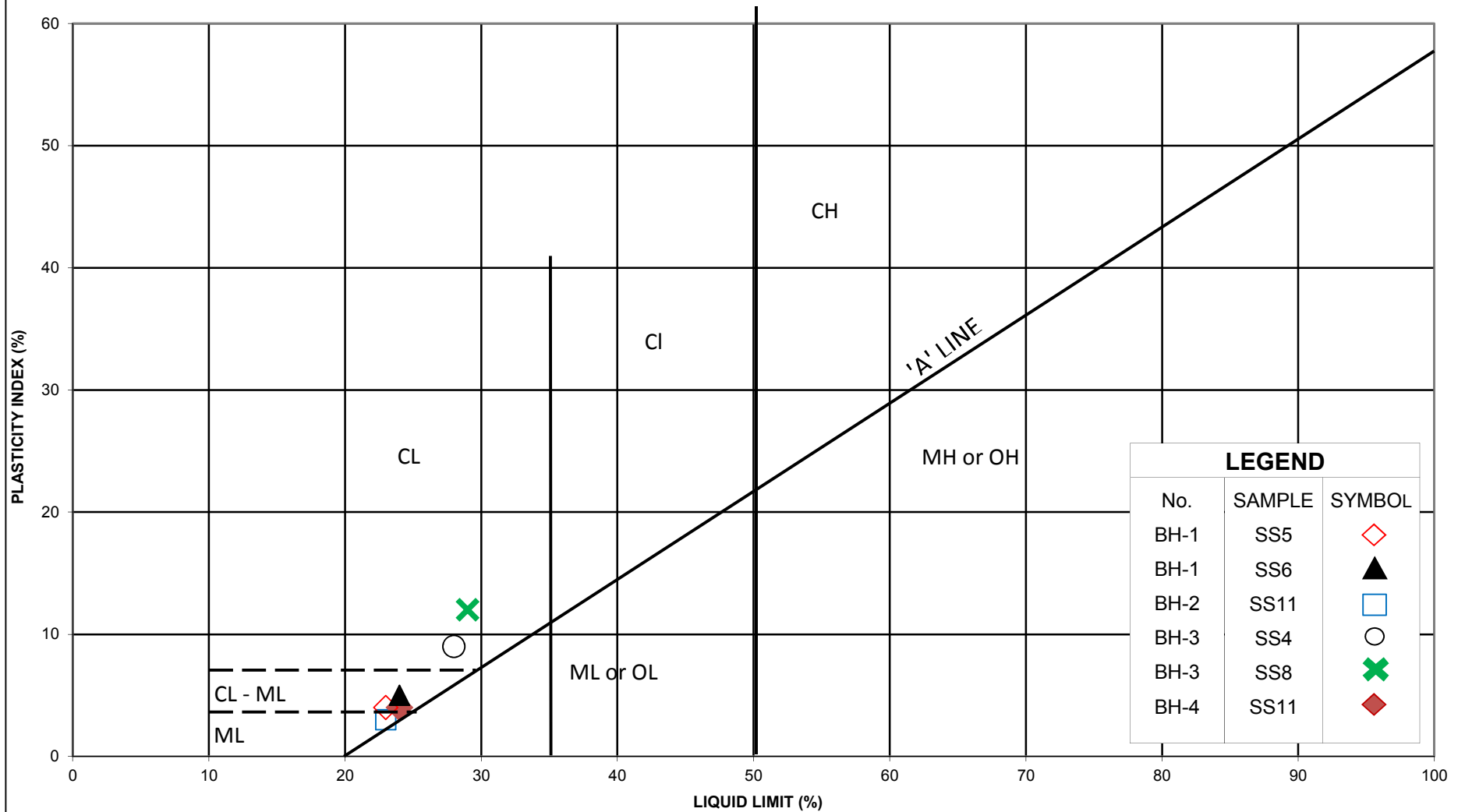
GWP: 3062-14-00

DATE: OCTOBER 2016

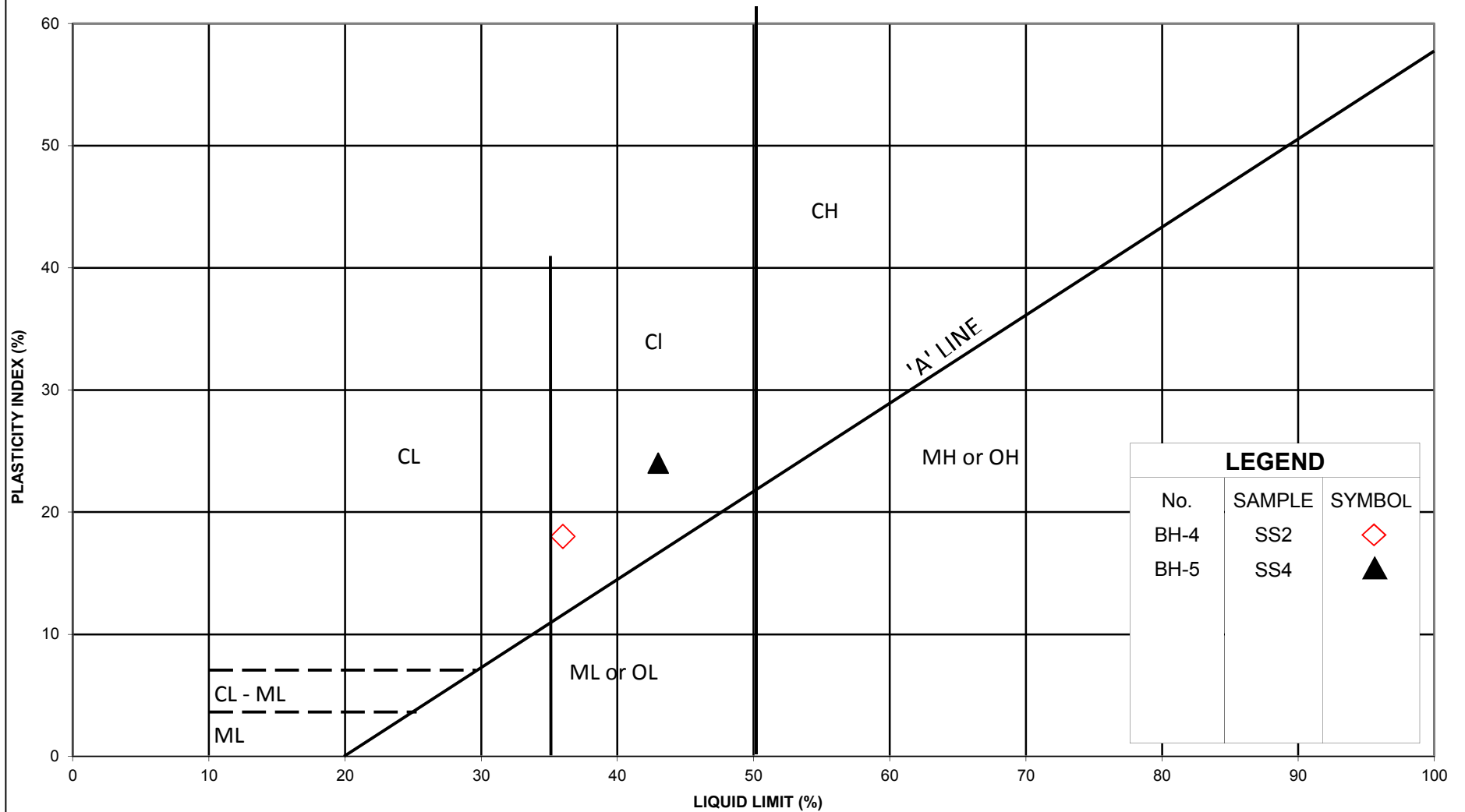
CULVERT REPLACEMENT
STA. 16+070, Highway 3, Simcoe, ON



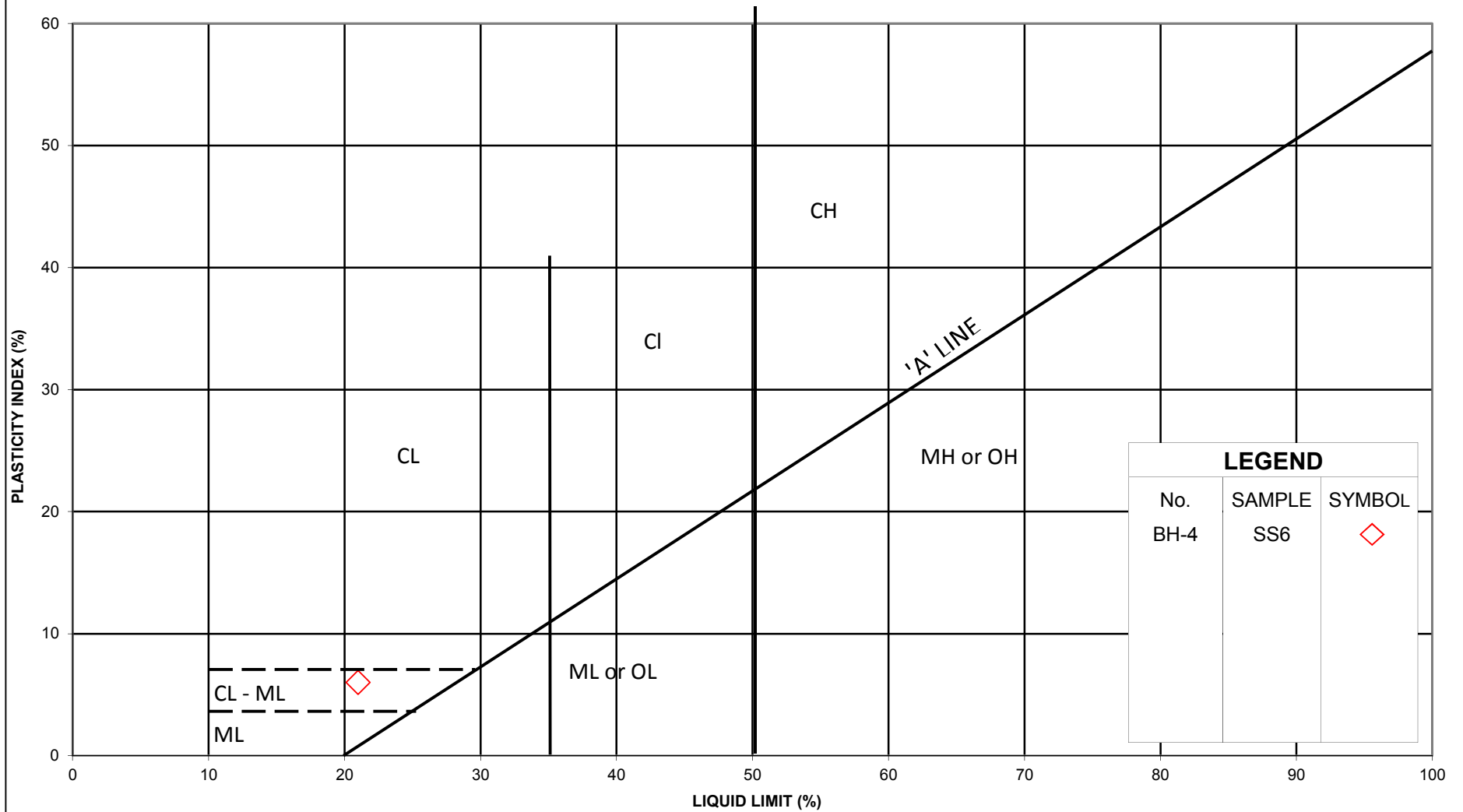
CULVERT REPLACEMENT
STA. 16+070, Highway 3, Simcoe, ON



CULVERT REPLACEMENT
STA. 16+070, Highway 3, Simcoe, ON



CULVERT REPLACEMENT
STA. 16+070, Highway 3, Simcoe, ON



LEGEND		
No.	SAMPLE	SYMBOL
BH-4	SS6	◇



PLASTICITY CHART
FILL: SILTY SAND (SM-SC)

FIGURE NO.: 8
GWP: 3062-14-00
DATE: OCTOBER 2016

CLIENT NAME: EXP. SERVICES INC.
80 BANCROFT STREET
HAMILTON, ON L8E2W5
(905) 573-4000

ATTENTION TO: Jeff Golder

PROJECT: Culvert Replacement

AGAT WORK ORDER: 16T156323

SOIL ANALYSIS REVIEWED BY: Amanjot Bhela, Inorganic Coordinator

DATE REPORTED: Nov 09, 2016

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: 16T156323

PROJECT: Culvert Replacement

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.

SAMPLING SITE:

ATTENTION TO: Jeff Golder

SAMPLED BY:

Corrosivity Package

DATE RECEIVED: 2016-11-03

DATE REPORTED: 2016-11-09

		SAMPLE DESCRIPTION:		BH-3 SS2
		SAMPLE TYPE:		Soil
		DATE SAMPLED:		2016-10-14
Parameter	Unit	G / S	RDL	7985344
Sulphide	%		0.05	<0.05
Chloride (2:1)	µg/g		2	62
Sulphate (2:1)	µg/g		2	16
pH (2:1)	pH Units		NA	7.98
Electrical Conductivity (2:1)	mS/cm		0.005	0.223
Resistivity (2:1)	ohm.cm		1	4480
Redox Potential (2:1)	mV		5	273

Comments: RDL - Reported Detection Limit; G / S - Guideline / Standard

7985344 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:

Amanjot Bhela

Quality Assurance

CLIENT NAME: EXP. SERVICES INC.

PROJECT: Colvert Replacement

SAMPLING SITE:

AGAT WORK ORDER: 16T156323

ATTENTION TO: Jeff Golder

SAMPLED BY:

Soil Analysis

RPT Date: Nov 09, 2016			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															
Sulphide	7985344	7985344	<0.05	<0.05	NA	< 0.05	98%	80%	120%	NA			NA		
Chloride (2:1)	7983417		249	249	0.0%	< 2	101%	80%	120%	104%	80%	120%	100%	70%	130%
Sulphate (2:1)	7983417		81	81	0.0%	< 2	94%	80%	120%	98%	80%	120%	105%	70%	130%
pH (2:1)	7983417		7.52	7.58	0.8%	NA	101%	90%	110%	NA			NA		
Electrical Conductivity (2:1)	7992374		0.318	0.318	0.0%	< 0.005	99%	90%	110%	NA			NA		
Redox Potential (2:1)	7983109		274	272	0.7%	< 5	101%	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: 16T156323

PROJECT: Culvert Replacement

ATTENTION TO: Jeff Golder

SAMPLING SITE:

SAMPLED BY:

PARAMETER	AGAT S.O.P	LITERATURE REFERENCE	ANALYTICAL TECHNIQUE
Soil Analysis			
Sulphide	MIN-200-12025	ASTM E1915-09	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

Appendix E – Slope Stability Analyses

Culvert Replacement on Hwy 3
Simcoe to Renton
South Embankment (Outlet)
Undrained Static Condition

Name: Engineered Fill Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion': 0 kPa Phi': 32 °
 Name: Silty Clay Fill Model: Undrained (Phi=0) Unit Weight: 19 kN/m³ Cohesion': 30 kPa
 Name: Silty Sand Fill Model: Mohr-Coulomb Unit Weight: 19 kN/m³ Cohesion': 0 kPa Phi': 27 °
 Name: Silty Clay (Firm to Stiff) Model: Undrained (Phi=0) Unit Weight: 19 kN/m³ Cohesion': 60 kPa
 Name: Silty Clay (Very Soft to Soft) Model: Undrained (Phi=0) Unit Weight: 18 kN/m³ Cohesion': 25 kPa

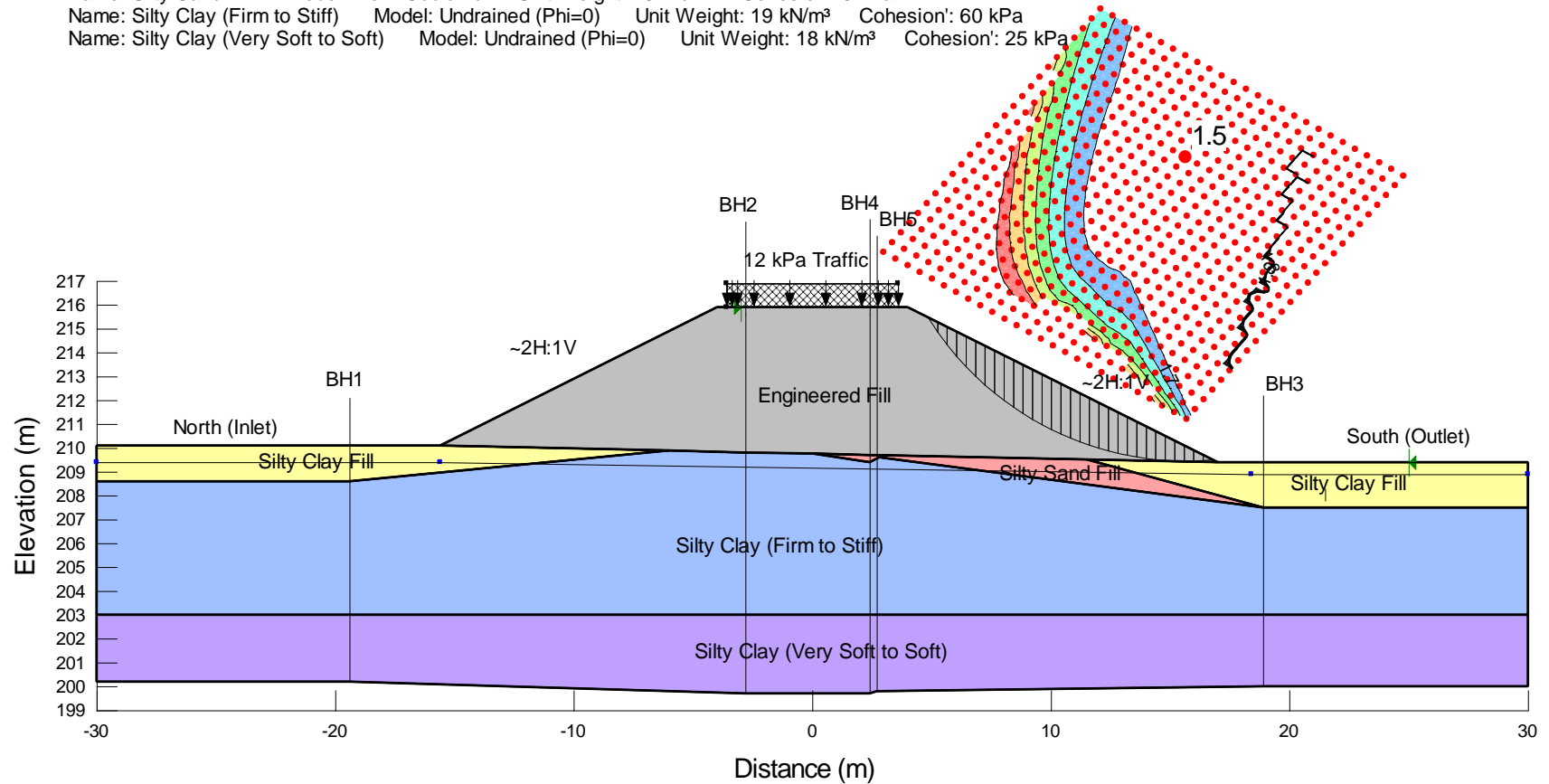


Figure 1: South embankment (outlet) – undrained static condition with 2H:1V slope and engineered fill below embankment

Culvert Replacement on Hwy 3
Simcoe to Renton
South Embankment (Outlet)
Drained Static Condition

Name: Engineered Fill Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion': 0 kPa Phi': 32 °

Name: Silty Clay Fill Model: Mohr-Coulomb Unit Weight: 19 kN/m³ Cohesion': 0 kPa Phi': 29 °

Name: Silty Sand Fill Model: Mohr-Coulomb Unit Weight: 19 kN/m³ Cohesion': 0 kPa Phi': 27 °

Name: Silty Clay (Firm to Stiff) Model: Mohr-Coulomb Unit Weight: 19 kN/m³ Cohesion': 0 kPa Phi': 28 °

Name: Silty Clay (Very Soft to Soft) Model: Mohr-Coulomb Unit Weight: 18 kN/m³ Cohesion': 0 kPa Phi': 25 °

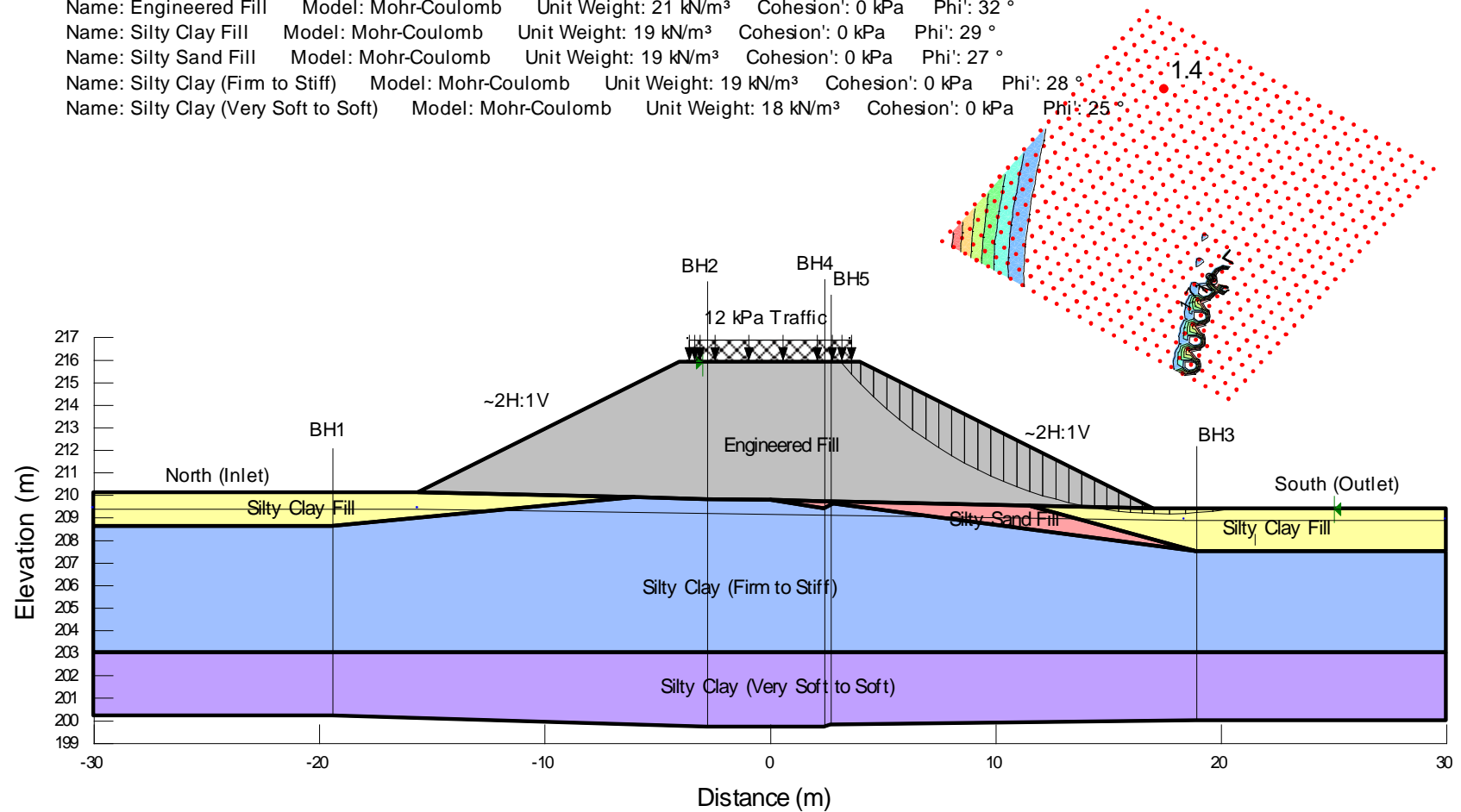
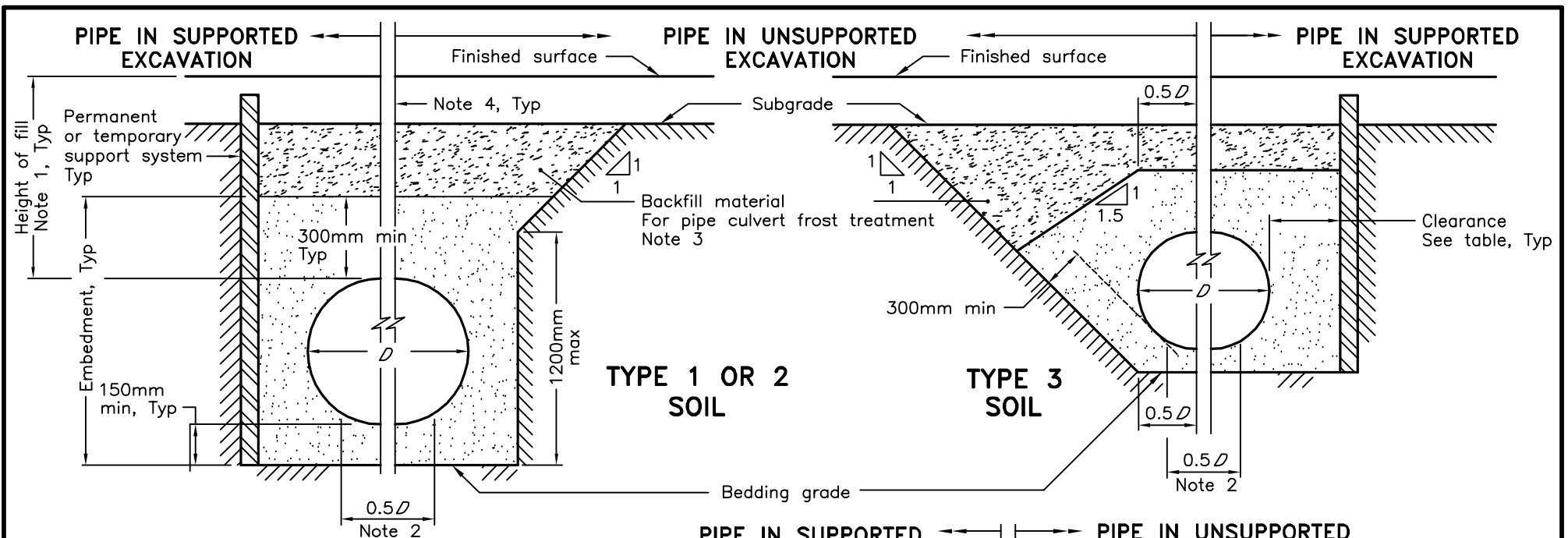


Figure 2: South embankment (outlet) – drained static condition with 2H:1V slope and engineered fill below embankment

Appendix F – OPSDs

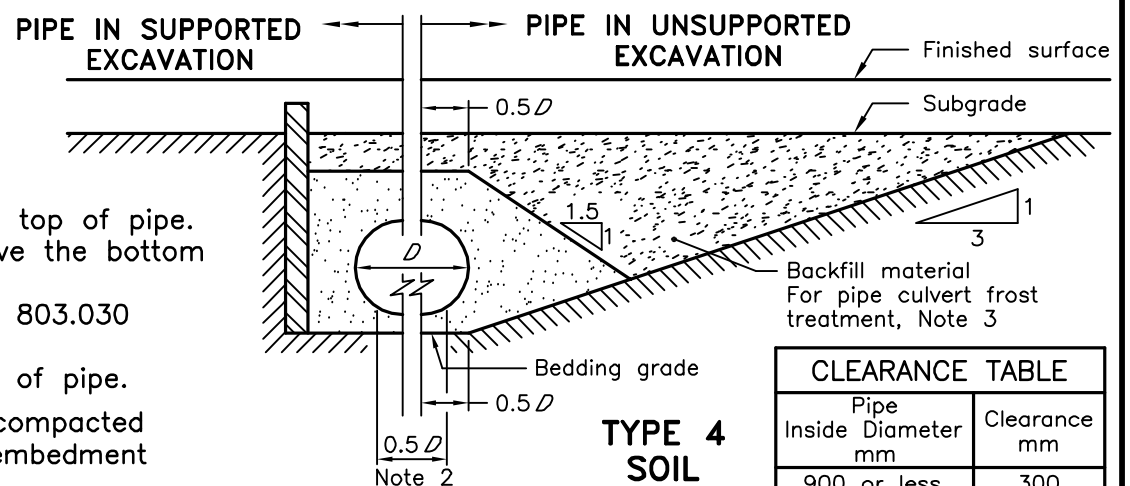


LEGEND:

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

Nov 2010

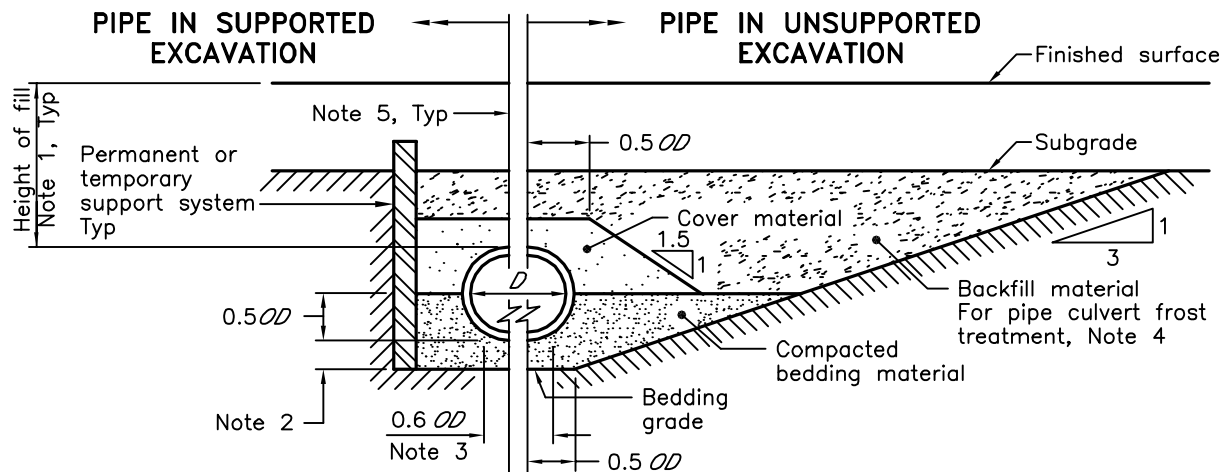
Rev 2

FLEXIBLE PIPE
EMBEDMENT AND BACKFILL
EARTH EXCAVATION

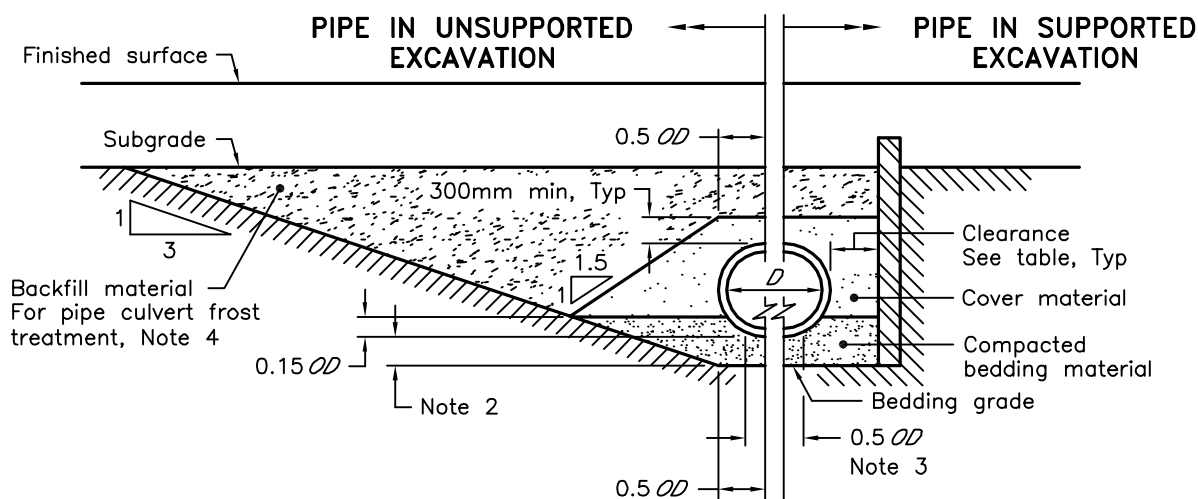
OPSD 802.010







CLASS B BEDDING



CLASS C BEDDING

LEGEND:

D – Inside diameter
 OD – Outside diameter

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.

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

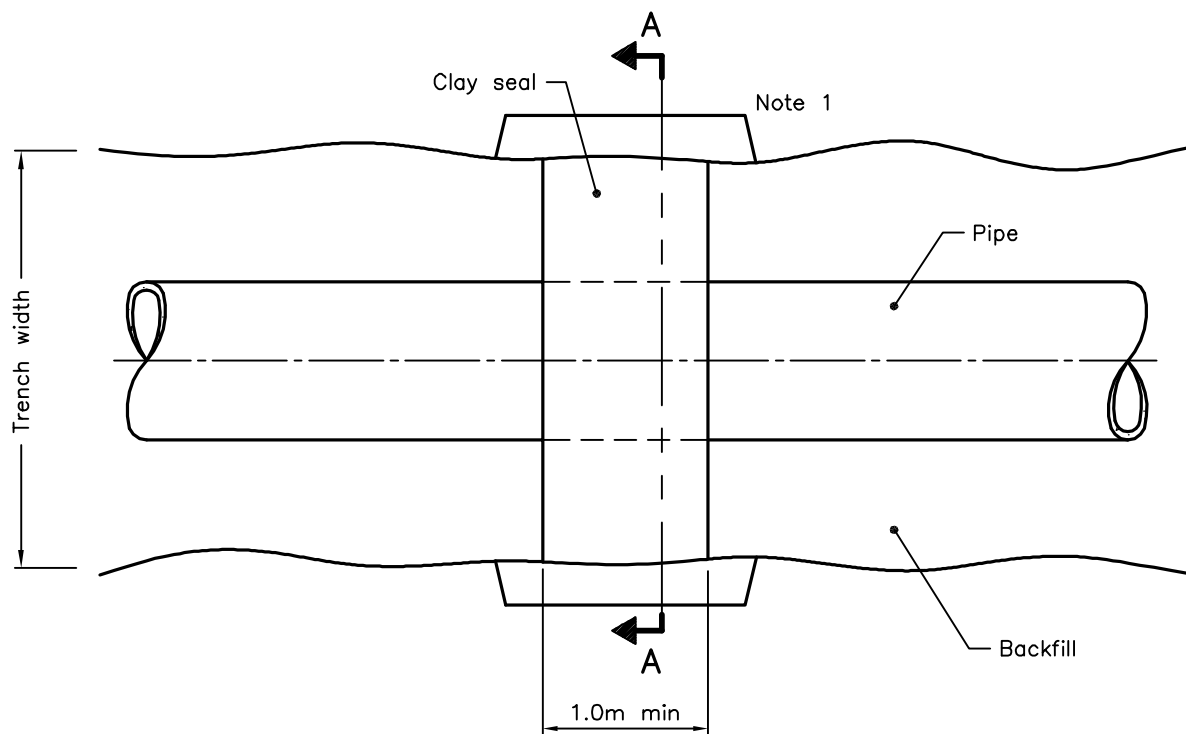
ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2010 Rev 2

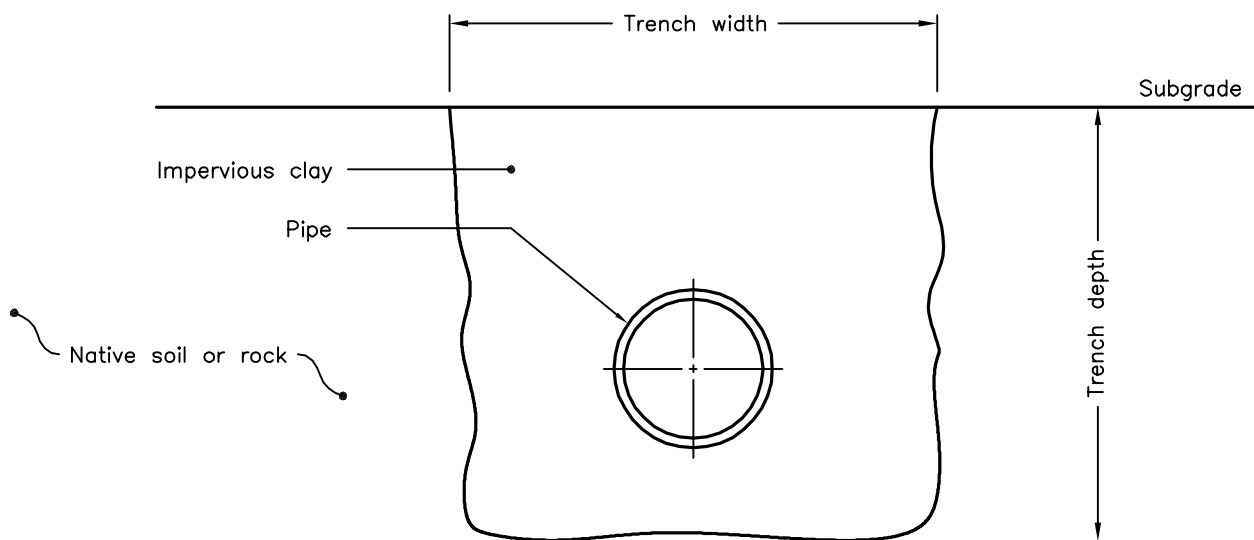
**RIGID PIPE BEDDING,
 COVER, AND BACKFILL
 TYPE 4 SOIL – EARTH EXCAVATION**

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

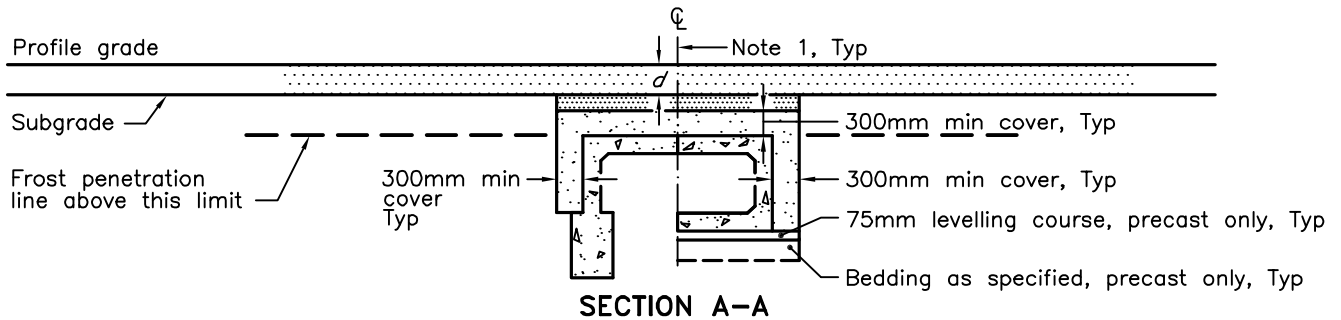
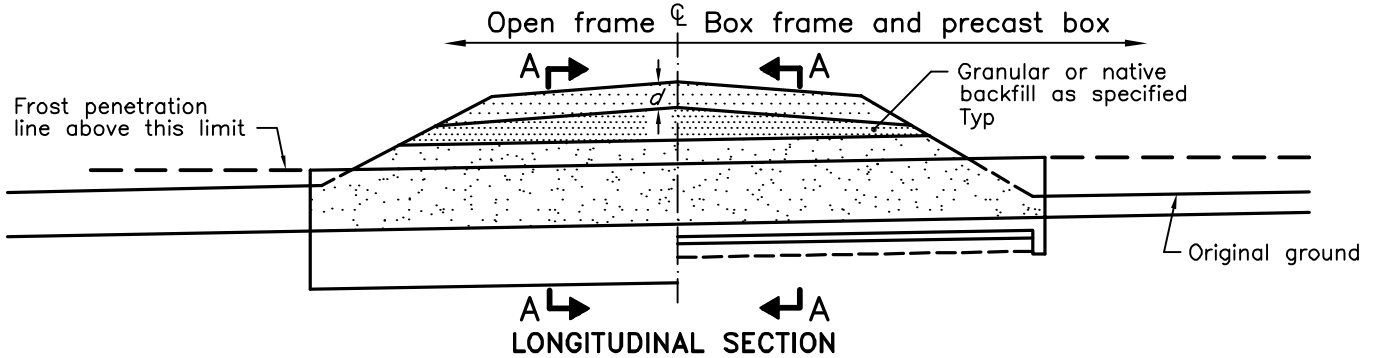
Rev 1

CLAY SEAL FOR PIPE TRENCHES

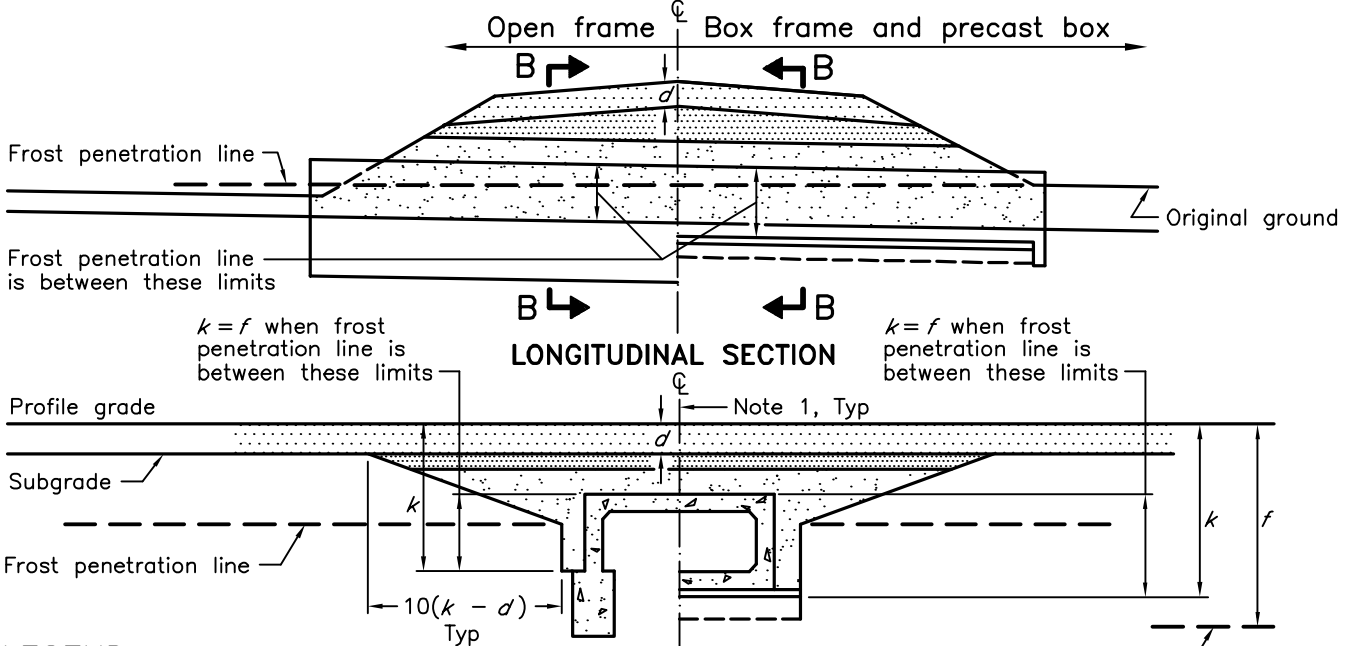
OPSD 802.095



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.

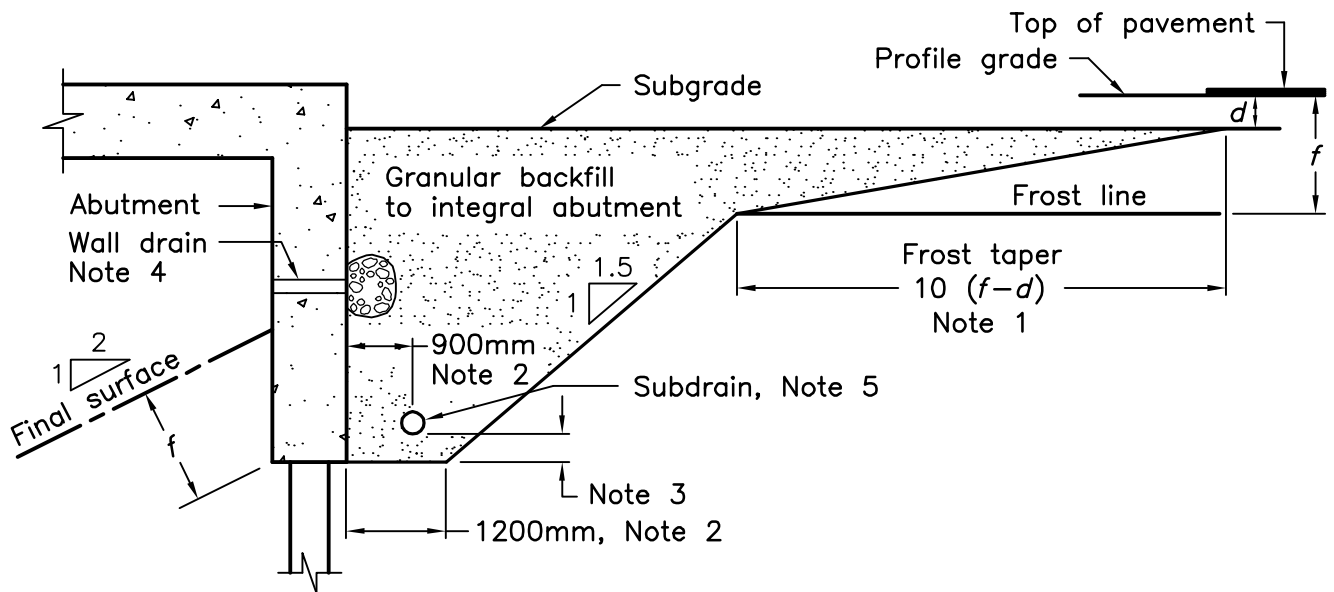
ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2010 Rev 2

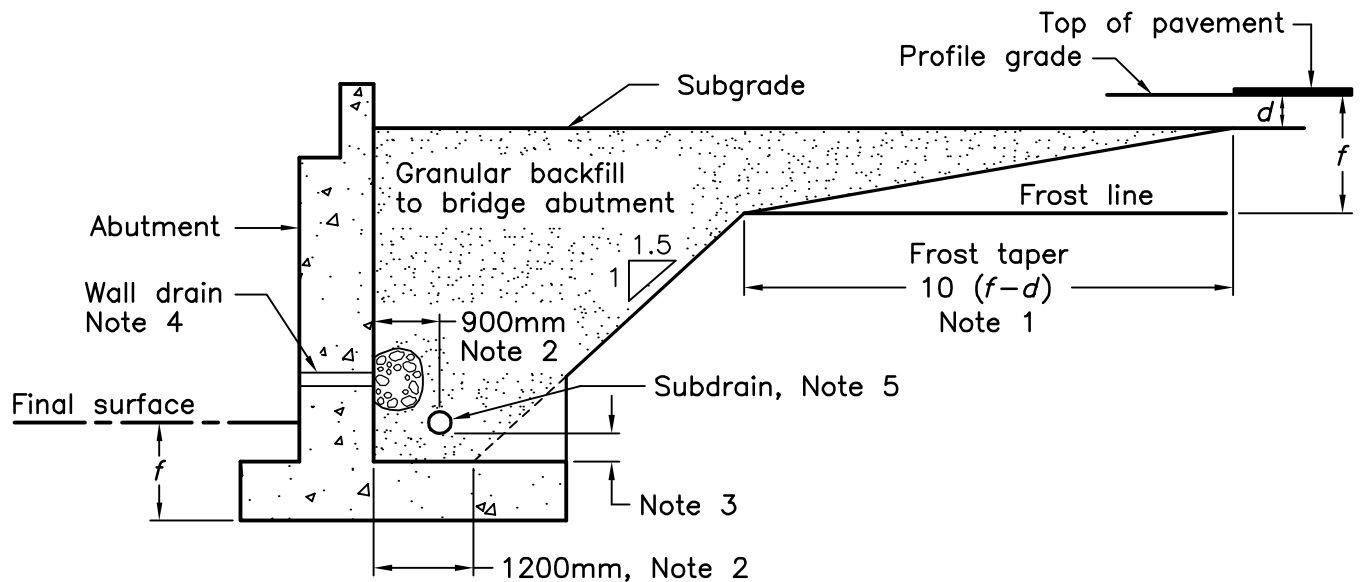
**BACKFILL AND COVER FOR
CONCRETE CULVERTS WITH SPANS
LESS THAN OR EQUAL TO 3.0M**

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

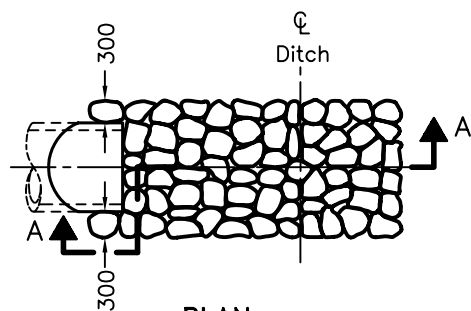
Nov 2010

Rev 1

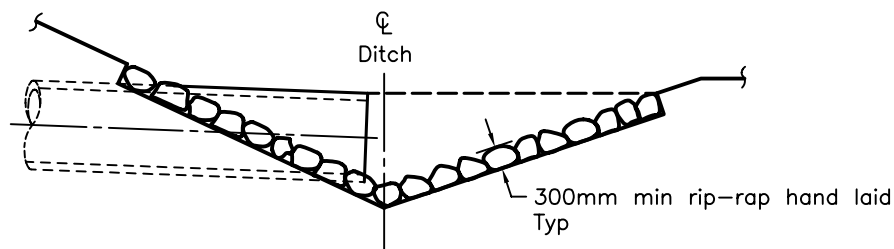


WALLS
ABUTMENT, BACKFILL
MINIMUM GRANULAR REQUIREMENT

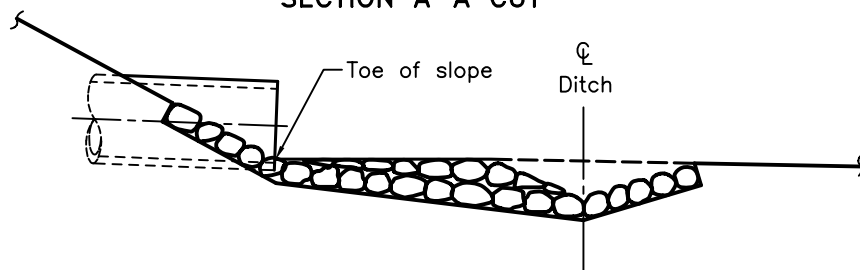
OPSD 3101.150



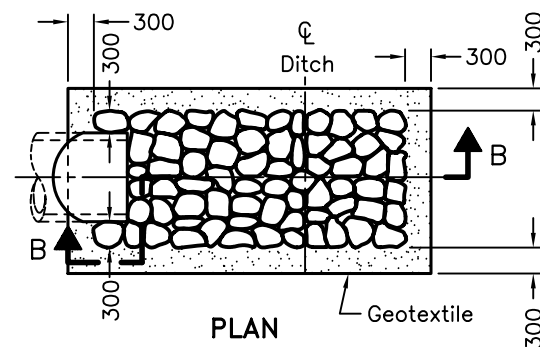
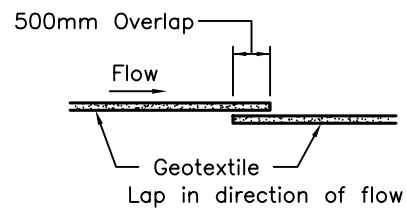
PLAN
CUT OR FILL



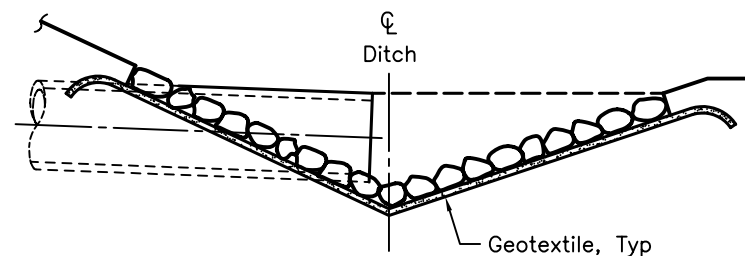
SECTION A-A CUT



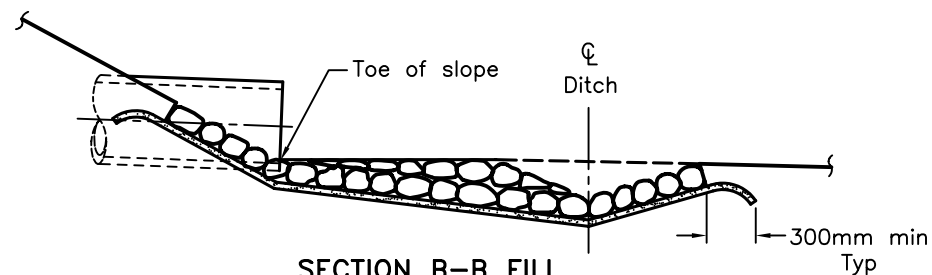
SECTION A-A FILL
TYPE A – WITHOUT GEOTEXTILE



PLAN
CUT OR FILL



SECTION B-B CUT



SECTION B-B FILL
TYPE B – WITH GEOTEXTILE

NOTES:

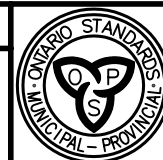
A All dimensions are in millimetres unless otherwise shown.

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.

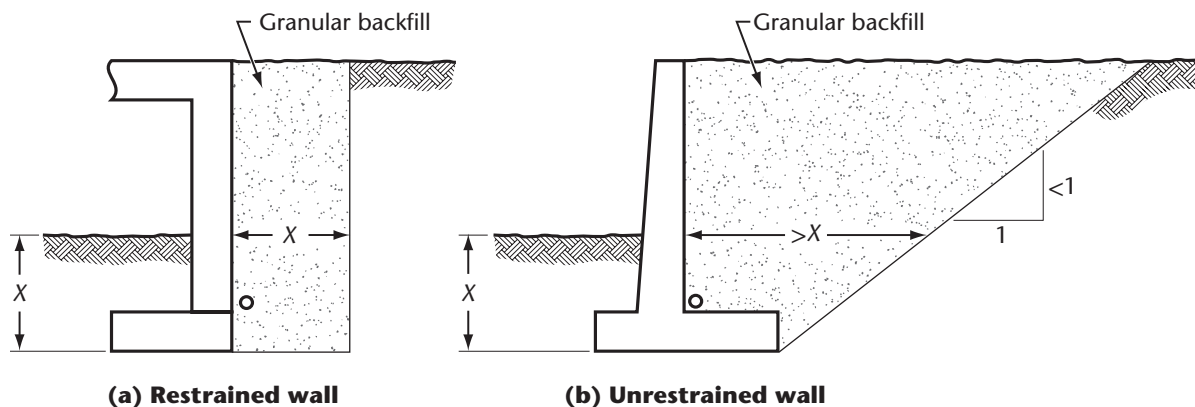


Figure C6.20
Backfill for frost protection
(See Clause C6.9.1.)

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.

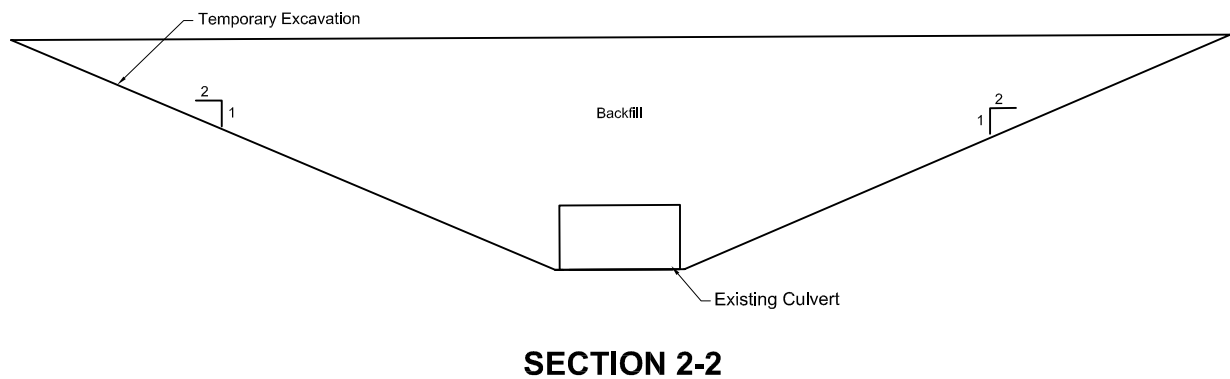
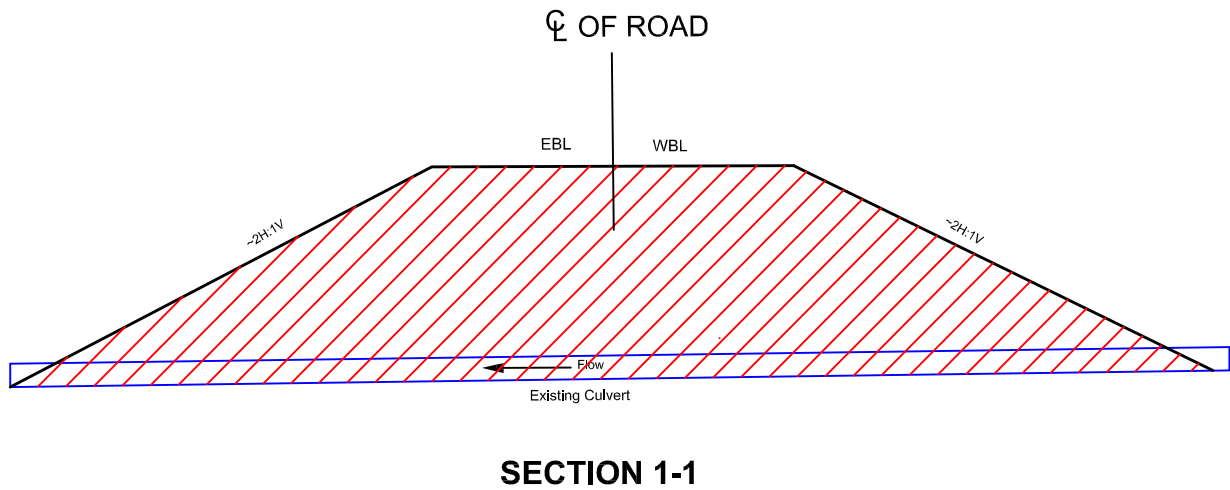
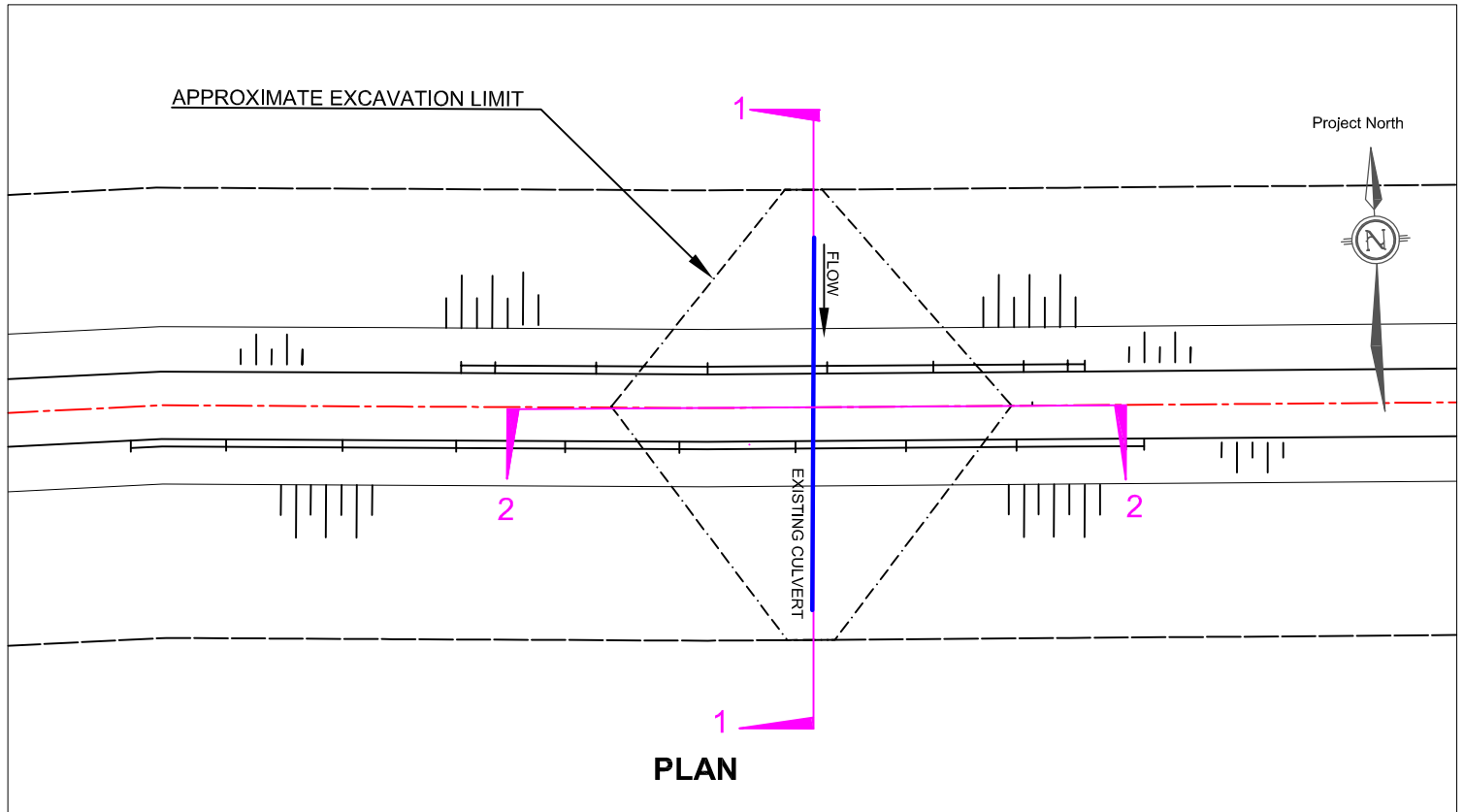
Clause 6.9.2.1 includes the specification of four lateral pressure conditions for design. The first two cases apply to unrestrained structures, with Item (a) applying to the sizing of the base or pile arrangement with respect to external stability, and Item (b) to the sizing of the structural sections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

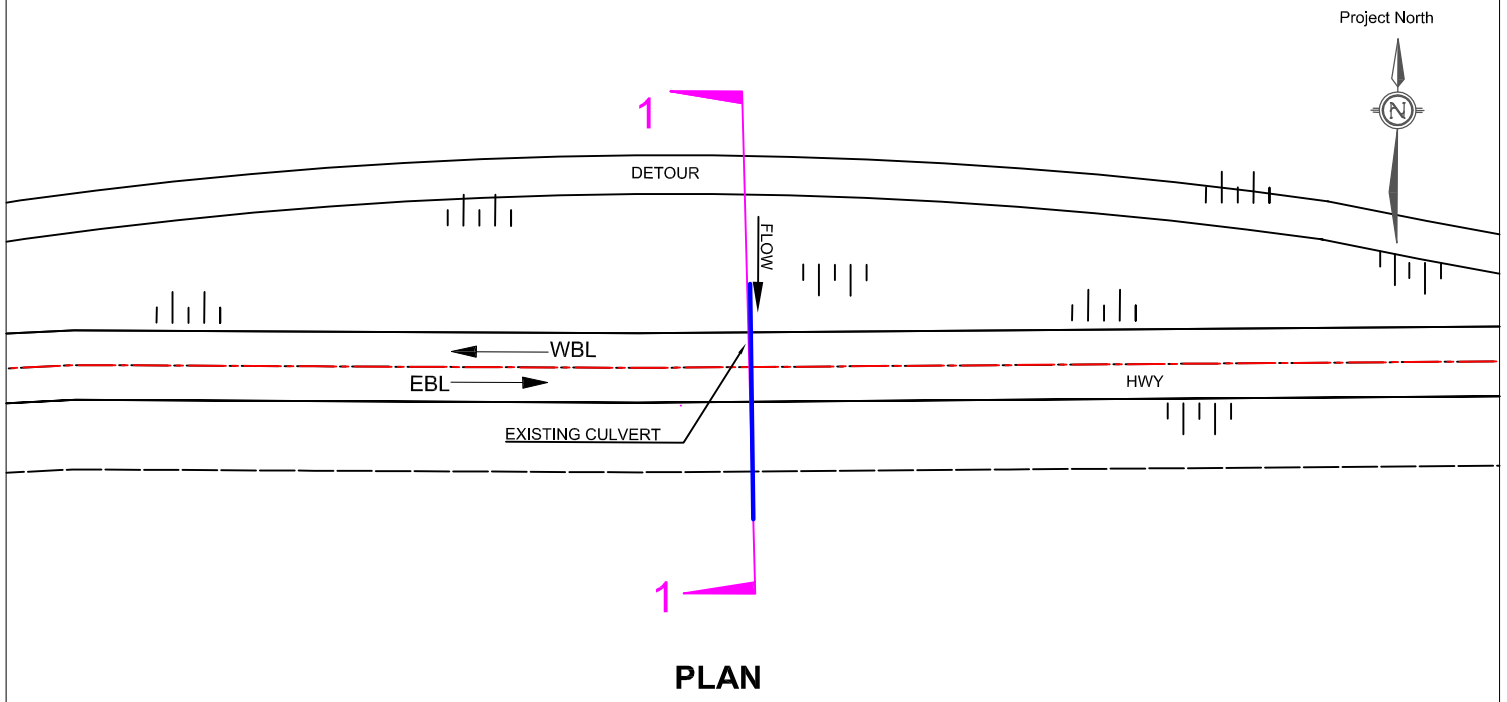
Appendix G – Schematic Sketches for Construction Alternative

**FIGURE H.1: FULL ROAD CLOSURE USING EXISTING ROADWAYS AND OPEN CUT
UNSUPPORTED EXCAVATION OPTION1**
SCHEMATIC DIAGRAMS (NTS)



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

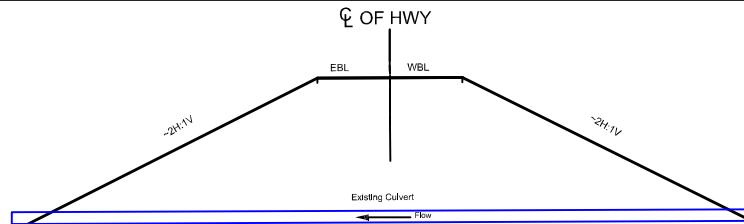
SCHEMATIC DIAGRAMS (NTS)



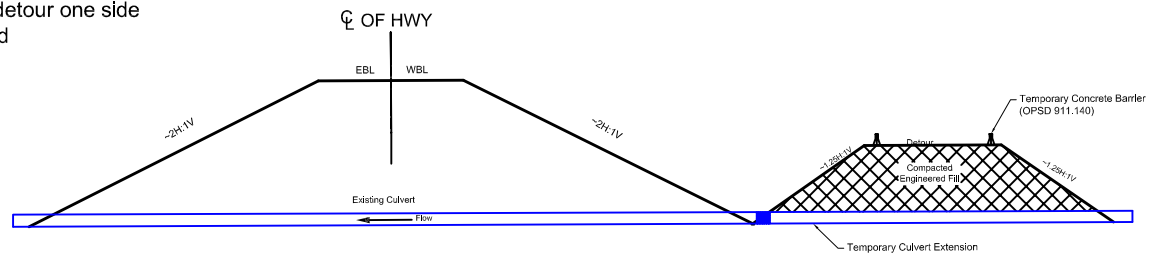
PLAN

RECOMMENDED STAGES

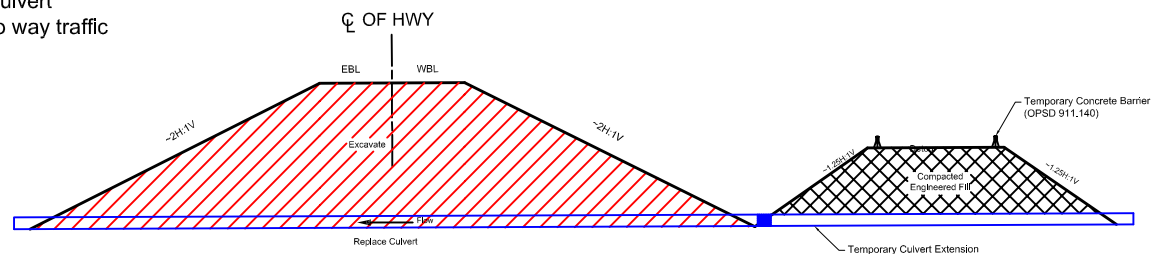
1.0 Stage 1 - Current condition



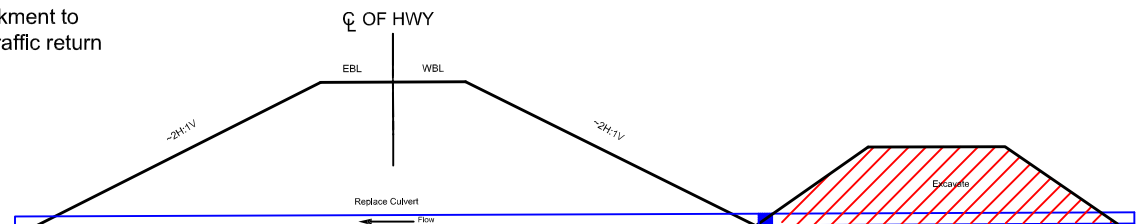
2.0 Stage 2 - Build temporary detour one side
Two-way traffic on existing road



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



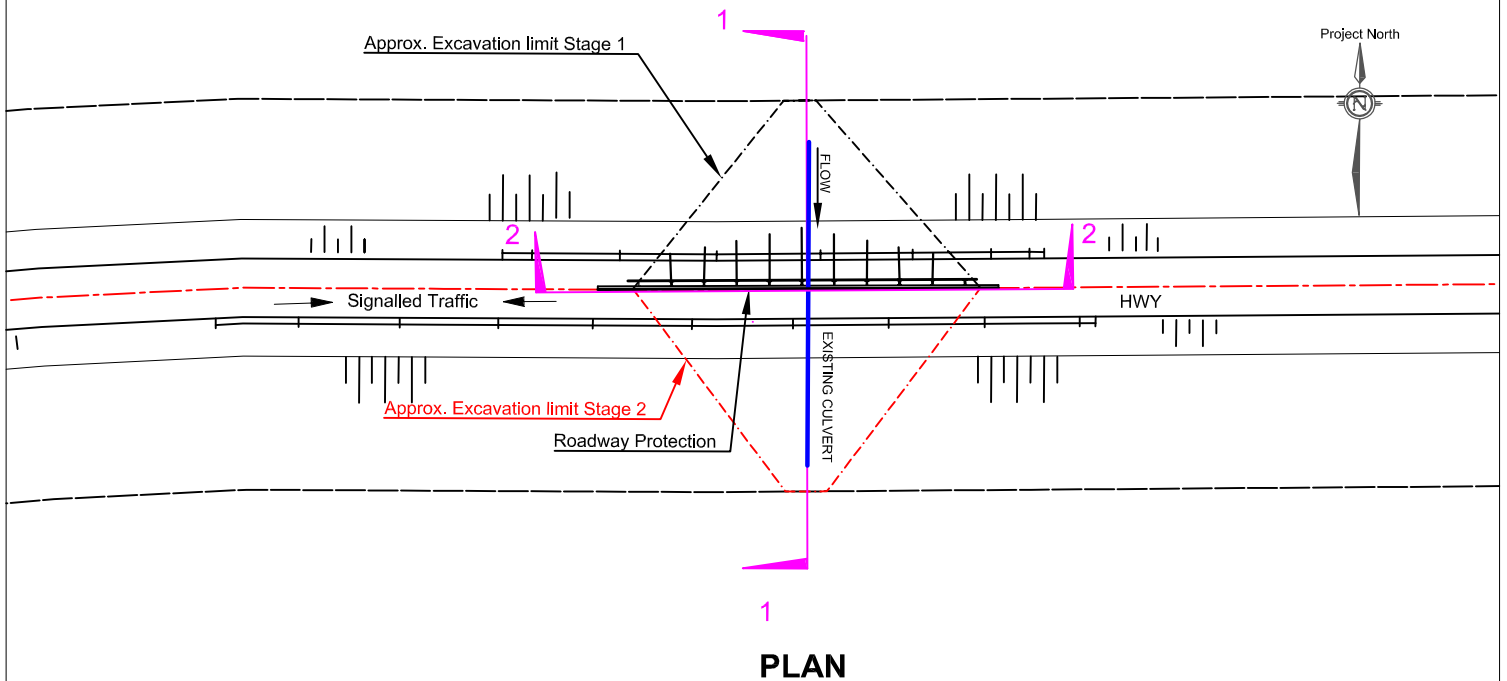
4.0 Stage 4 - Build the embankment to existing alignment; Two-way traffic return



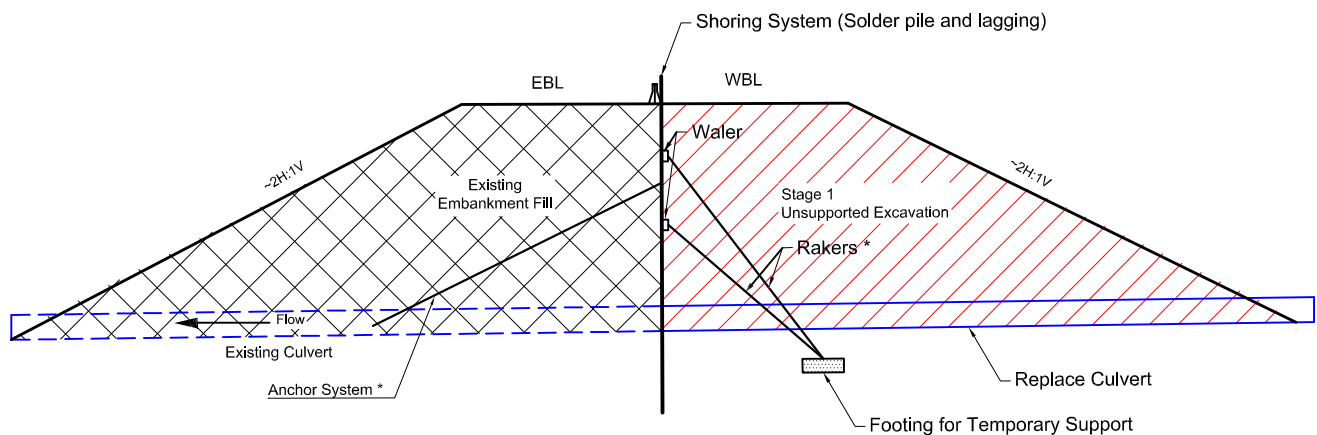
SECTION 1-1

FIGURE H.3.A: HALF AND HALF CONSTRUCTION WITH UNSUPPORTED CUT SIDES

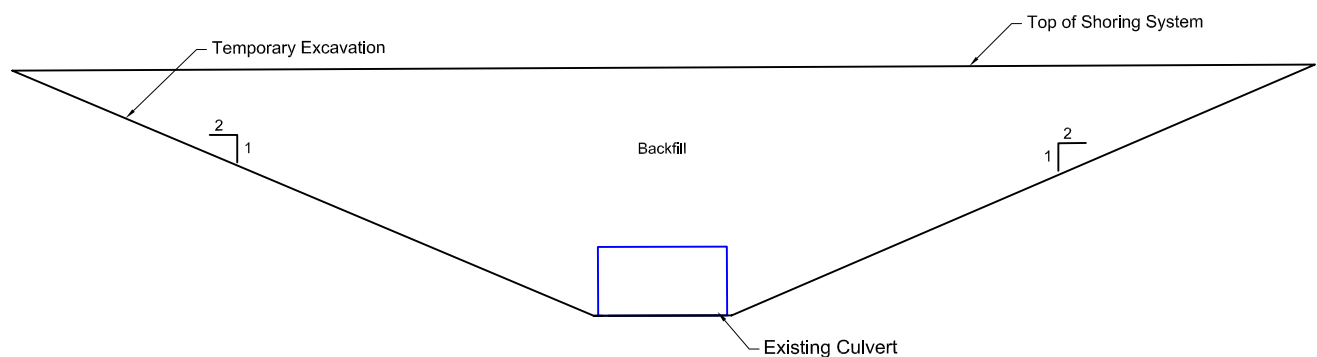
SCHEMATIC DIAGRAMS (NTS)



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



SECTION 1-1

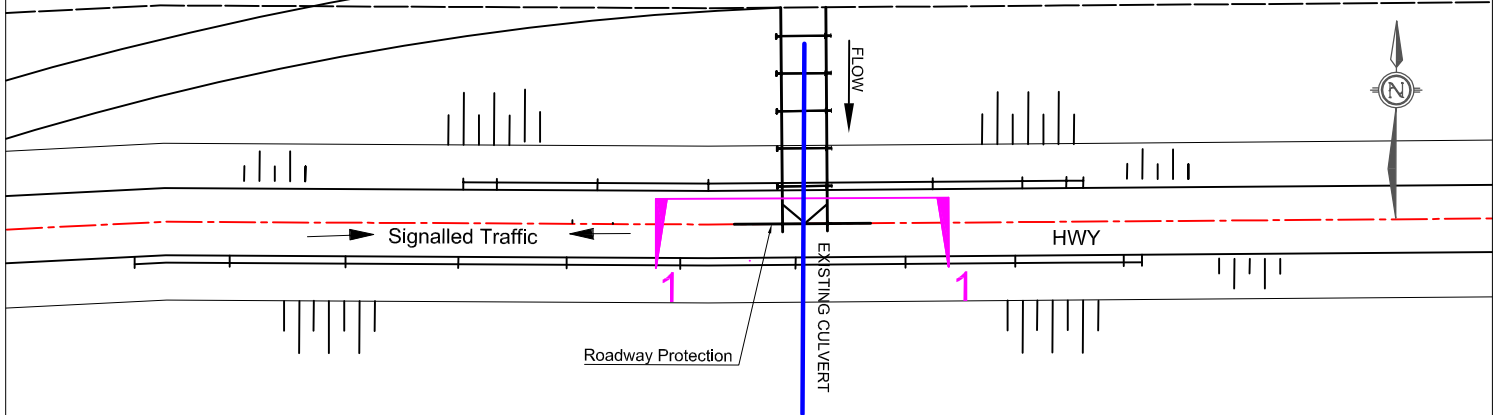


SECTION 2-2

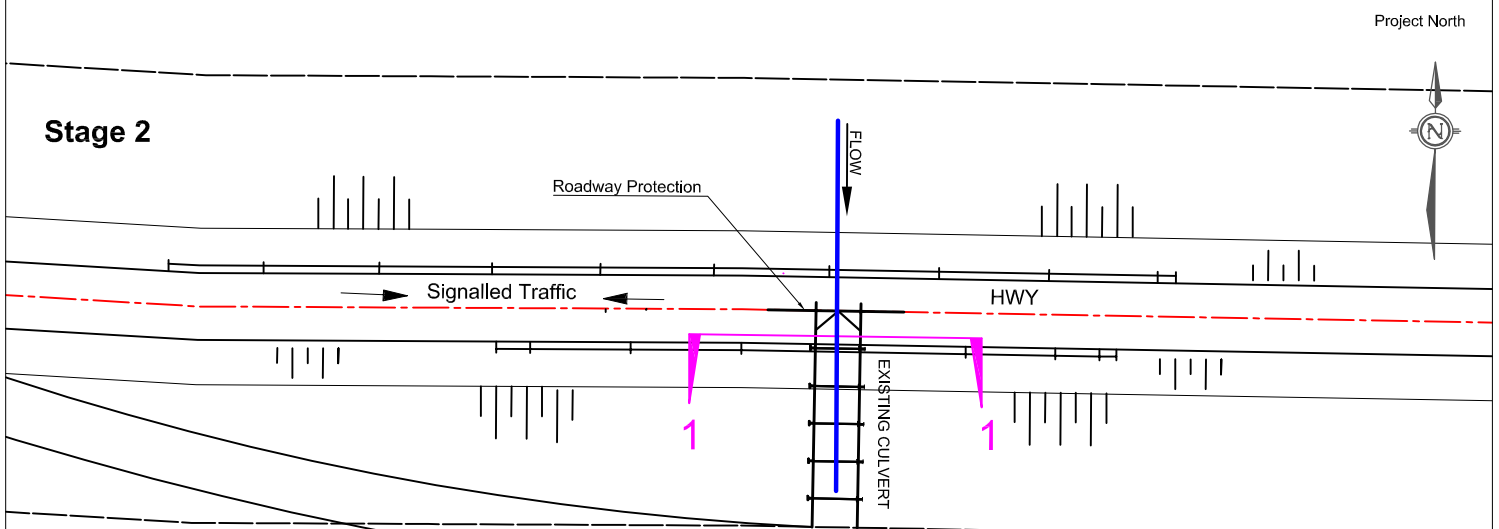
FIGURE H.3.B: HALF AND HALF CONSTRUCTION WITH BRACED CUT SIDES OR ANCHOR SYSTEM OPTION 3.B

SCHEMATIC DIAGRAMS (NTS)

Stage 1

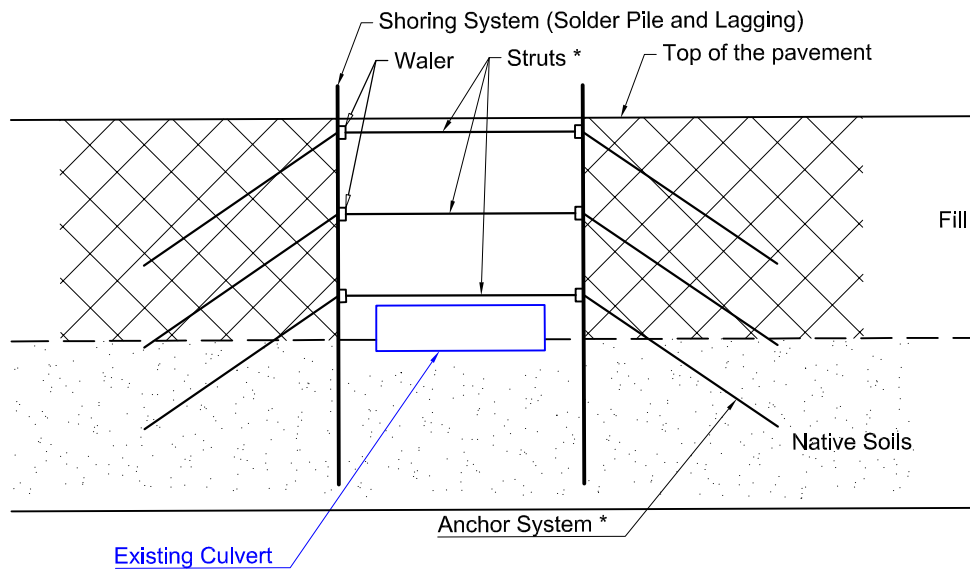


Stage 2



PLAN

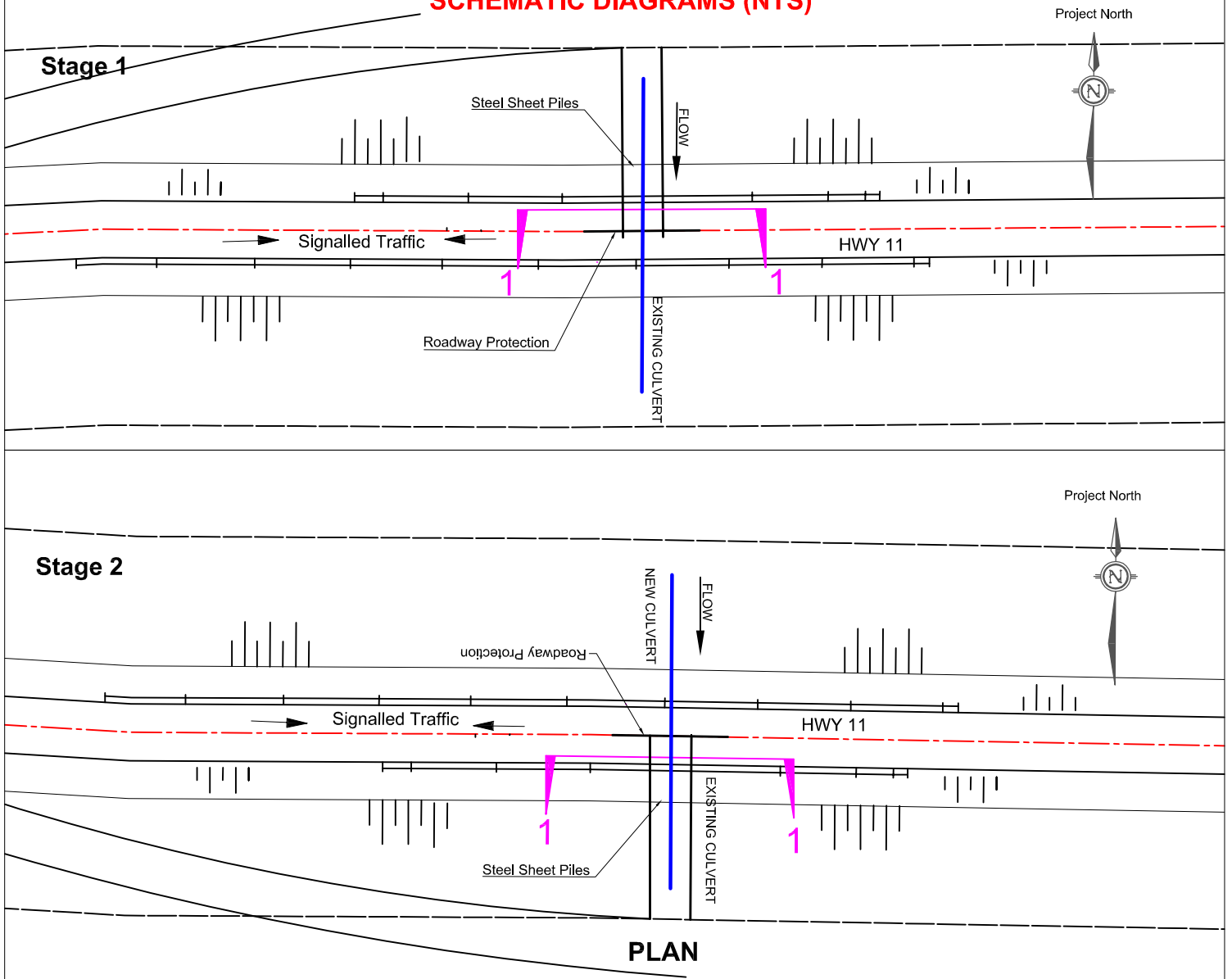
Half and Half Construction, Shoring System - Braced Cut Struts or Anchor System



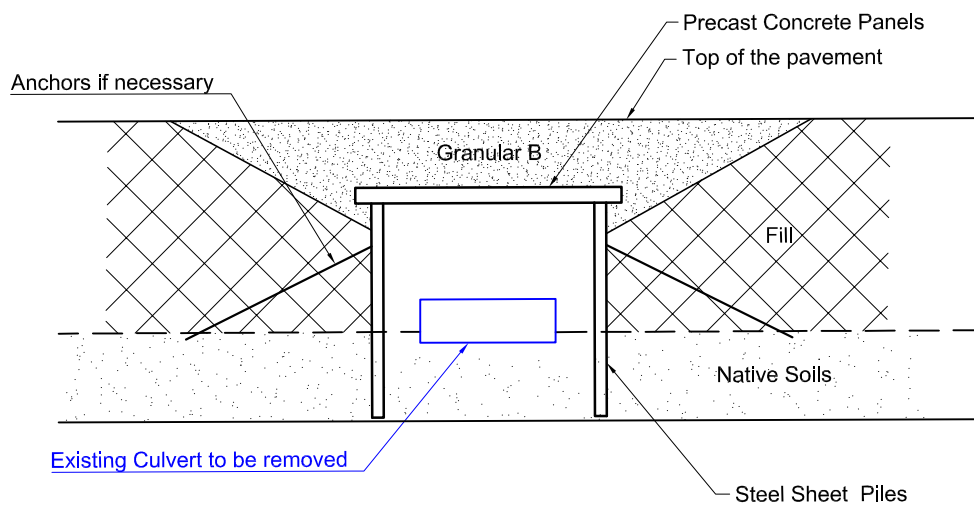
* Struts or Anchor System

SECTION 1-1

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



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



* Struts or Anchor System

SECTION 1-1

Appendix H – Non - Standard Special Provision

NSSP FOR COBBLES AND/OR BOULDERS OBSTRUCTIONS

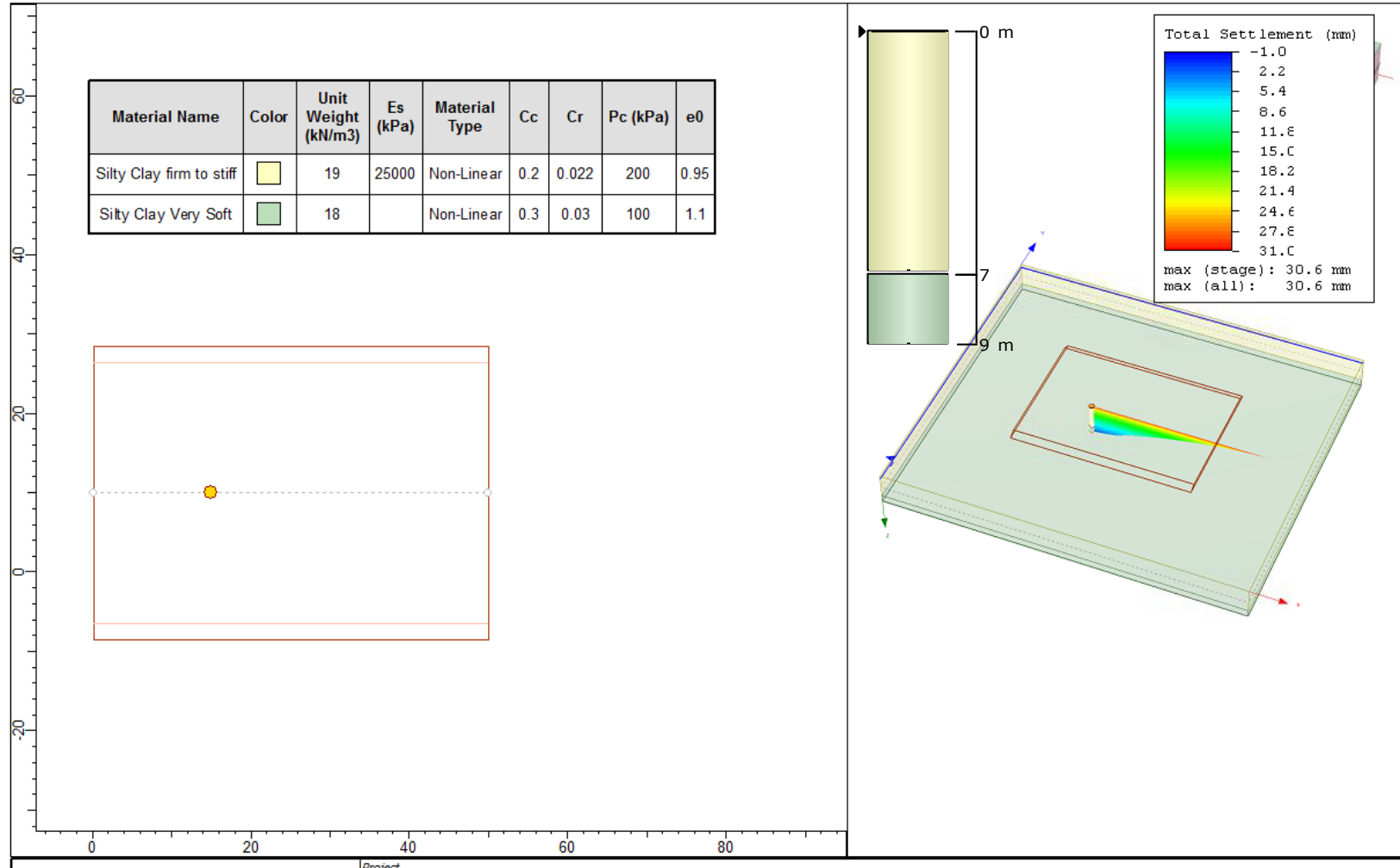
Scope of Work

The Contractor should be aware that the embankment at the site consists of granular fill underlain by firm to very stiff silty clay fill materials which may contain cobbles and/or boulders. Appropriate equipment and procedures will be required to penetrate/remove cobbles and/or boulders 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.

Appendix I – Result of Settlement Analysis



Project: Foundation Investigation and design report for Culvert Replacement, Hwy 3, Renton

Analysis Description: Approach Embankment – **Total Settlement**

Figure No: I-1

Company: exp Services Inc.

Date: November, 2016

File Name: Settlement Analysis – Assignment 1