



FINAL REPORT

FOUNDATION INVESTIGATION AND DESIGN REPORT **Culvert Replacement, Highway 3, 1.89 km east of Haldimand Road 55, Jarvis**

Agreement No. 3015-E-0017
Assignment No. 3
GWP 3062-14-00
Geocres No. 40116-27

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January 23, 2017

Ministry of Transportation

Western Region – Geotechnical Section

Foundation Investigation and Design Report

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Type of Document:

Final

Project Name:

Foundation Investigation and Design Report for Culvert Replacement

Highway 3, 1.89 km East of Haldimand Road 55

Jarvis, ON

Project Number:

ADM-00235197-C0

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PART I: **FOUNDATION INVESTIGATION REPORT**

1.1. **Introduction**

This foundation investigation report presents the results of a geotechnical investigation completed by **exp** Services Inc. (**exp**) for the replacement of an existing concrete culvert located on Highway 3 at Station 15+843, approximately 1.89 km east of Haldimand Road 55 in Jarvis, part of the Ministry of Transportation (MTO) West Region. The work was undertaken under Agreement No. 3015-E-0017, Assignment No. 3. The terms of reference (TOR) were as presented in the MTO document entitled "Foundation Engineering Terms of Reference, MTO West Region – Foundations Retainer Assignment, Assignment 3 – Culvert Replacement Hwy 3 Jarvis and Hwy 6 Dundalk" provided via e-mail on October 13, 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 and bedrock 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 Highway 3 at Station 15+843, approximately 1.89 km east of Haldimand Road 55 in Jarvis, Ontario. At this site, Highway 3 is a two-lane asphalt roadway and is about 8.5 m wide from edge to edge of asphalt, with narrow sand and gravel shoulders. Based on the observations at the site, the roadway embankment is less than approximately 1.8 m high with side slope not exceeding 5H:1V.

The existing culvert is an arch concrete rigid framed structure and is assumed to have an open footing foundation, but inspection was not possible due to the presence of water. It is assessed that the culvert has a span of approximately 2.8 m and rise of 1.5 m. The culvert is approximately 18.4 m long. The existing culvert is intended to be replaced with a new culvert along the same alignment. Select photographs of the site and existing culvert are presented in Appendix A. The site plan and cross-section profiles for the culvert alignment are shown on Drawings 1 and 2 in Appendix B.

The area surrounding the culvert site generally consists of flat lying fields, but a small wooded area exists to the south (outlet side) of the culvert. A mix of shrubs and long grasses were observed on the stream bank at both inlet and outlet sides, and in the path of the stream on the inlet side. However, no visible sign of flow restriction was observed due to the vegetation.

Highway 3 runs in an east-west direction and the water in culvert flows from north to south beneath the highway. At the time of investigation, the elevation of the water in the culvert was approximately 204.9 m. The elevation of highway centerline at the culvert centerline is approximately 206.4 m. Cable guide

rails are present on both sides of the highway and overhead wires are present 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 November 1, 2016 and November 8-9, 2016, respectively. The embankments were noted to be in an overall stable configuration with no obvious indications of recent slope movement. Longitudinal cracking, some transverse cracking, and wheel rutting were observed on both lanes at the site. 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 no significant damage.

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 in a 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 in the project area consists of limestone, dolostone and shale of the Middle Devonian period.

1.3. Investigation Procedures

1.3.1. Site Investigation and Field Testing

The field investigation was performed on November 8 and 9, 2016. The field program consisted of drilling five (5) sampled boreholes, numbered BH-1 to 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. Boreholes BH-1 and BH-2 were advanced at accessible locations near the inlet and outlet of the culvert, respectively. Borehole BH-4 was advanced within the travelled eastbound lane and located about 5 m west of the culvert centerline. Two (2) additional boreholes were strategically located on the embankment to provide subsurface information for the temporary roadway protection. Boreholes BH-3 and BH-5 were advanced in the eastbound travelled lane approximately 25 m west and east side of the existing culvert, respectively. The borehole locations are shown on Drawing 1 in Appendix B.

The boreholes were advanced using a rubber track mounted Mobile B57 mechanical drill rig equipped with hollow stem augers and standard soil sampling equipment, operated by a specialist drilling contractor, Landshark Drilling. The boreholes were advanced to auger refusal on the bedrock surface at depths ranging from 4.8 to 5.8 m below the ground surface. Samples of the bedrock were retrieved at Boreholes BH-3 and BH-5 using HQ coring equipment.

The borehole locations (referenced to the MTM NAD83 coordinate system) and their ground surface elevations were surveyed by **exp** personnel using a Vertical Control Point (VCP301) located approximately 50 m west of the culvert and immediately north of the Highway 3 westbound lane shoulder (258182.258 E, 4750230.749 N). The VCP was used as a temporary benchmark (TBM) and has elevation 205.707 m as was provided on the MTO document entitled "Horizontal and Vertical

Control”, Highway 3 C/L Alignment – Walpole TWP, Contract No. 3014-E-0030-26, dated October 1, 2016. The TBM 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 tests were conducted in accordance with ASTM D2573-08. Two (2) Shelby tube samples were obtained below the culvert invert level.

Upon completion of the boreholes, ground water level measurements were carried out in boreholes in accordance with MTO 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 an **exp** geotechnical representative who directed the drilling and sampling operation, logged borehole data in accordance with MTO and/or ASTM Standards for Soils Classification, and retrieved soil and bedrock samples. The recovered soil samples were placed in labelled moisture-proof bags; bedrock samples were placed in core boxes, and all samples were 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 of all samples and particle size distribution for approximately 25% of the collected soil samples. Atterberg Limits tests were carried out on select cohesive soil samples. One soil sample was selected for corrosivity chemical analyses and was tested at AGAT Laboratories, a CALA-certified and accredited laboratory in Mississauga, Ontario. All of the laboratory tests were carried out in accordance with MTO and/or ASTM Standards, as appropriate.

The laboratory test results are provided on the attached borehole log sheets in Appendix C. The results of the grain size analyses and Atterberg Limits testing are presented graphically in Appendix D. The results of chemical analyses are also included in Appendix D.

1.4. Subsurface Conditions

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

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

In general, the subsurface conditions along the proposed culvert alignment consist of a layer of granular fill overlying silty clay fill followed by the native soils, which typically consisted of silty clay, clay, and sandy clayey silt till to silty clay till. 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 advanced on the highway, i.e. Boreholes BH-3 to BH-5, and ranged in thickness from approximately 175 to 280 mm. 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-2, and ranged in thickness from approximately 50 to 100 mm. Topsoil thicknesses may further vary beyond the borehole locations.

1.4.3. **Granular Fill**

Granular fill was encountered below the asphalt at Boreholes BH-3 to BH-5. The granular fill layer ranged in thickness from approximately 250 to 1065 mm. The granular fill was composed of sand and crushed gravel with trace silt and was noted to contain trace amounts of coal and had a hydrocarbon odour. The SPT "N" values within this layer ranged from 9 to greater than 100 blows per 305 mm penetration, suggesting loose to very dense compactness condition.

1.4.4. **Fill: Silty Clay**

Silty clay fill material was encountered at all borehole locations underlying the topsoil or granular fill and extended to depths ranging from 0.9 to 2.3 m below the ground surface and elevations ranging from 204.0 to 205.0 m. The silty clay fill contained trace to some sand, trace gravel, was brown in colour, and in a moist state. The SPT "N" values ranged from 3 to 12 blows per 305 mm penetration, suggesting soft to stiff consistency. Laboratory testing consisting of seven (7) moisture content determinations, two (2) grain size analyses, and two (2) Atterberg Limits tests were carried out with the results summarized below:

Moisture Content:

- 25% to 39%

Grain Size Analysis:

- 1% to 2% gravel
- 3% to 13% sand
- 47% to 55% silt

Atterberg Limits:

- 37% to 41% liquid limit
- 17% to 20% plastic limit
- 20% to 21% plasticity index

- 31% to 48% clay

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 Figures 3 and 7 in Appendix D.

1.4.5. Fill: Gravelly Sand with Some Silt and Clay

Gravelly sand with some silt and clay fill material was encountered at Borehole BH-4 underlying the silty clay fill at a depth of 2.3 m below the ground surface and extended to the native clay at a depth of 3.1 m below the ground surface (elevation 203.4 to 204.1 m). The gravelly sand fill contained some silt, some clay, was grey in colour and in a moist state. A single SPT "N" value of 5 blows per 305 mm penetration was obtained within the gravelly sand fill layer, suggesting loose compactness condition. Laboratory testing consisting of one (1) moisture content determination, one (1) grain size analysis, and one (1) Atterberg Limits test were carried out with the results summarized below (i.e. Atterberg Limits tests were carried out on the silty clay portion of the sample):

Moisture Content:

- 16%

Grain Size Analysis:

- 26% gravel
- 41% sand
- 20% silt
- 13% clay

Atterberg Limits:

- 48% liquid limit
- 29% plastic limit
- 19% plasticity index

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 Figures 4 and 8 in Appendix D.

1.4.6. Silty Clay

Native silty clay was encountered below the silty clay fill at Boreholes BH-1, BH-2, and BH-5 at depths ranging from 0.9 to 1.5 m (elevations 204.3 to 205.0 m) and extended to the underlying clay stratum at 1.5 to 3.2 m below the ground surface. The silty clay was varved, contained trace sand, was brown in colour, and in a moist state. The SPT "N" values ranged from 7 to 12 blows per 305 mm penetration and undrained shear strengths obtained from in-situ shear vane tests ranged from 91 to 95 kPa, classifying the silty clay as firm to stiff in consistency. Laboratory testing consisting of four (4) moisture content determinations, one (1) grain size analysis, and one (1) Atterberg Limits test were carried out with the results summarized below:

Moisture Content:

- 22% to 28%

Grain Size Analysis:

- 0% gravel
- 1% sand

Atterberg Limits:

- 33% liquid limit
- 21% plastic limit
- 12% plasticity index

- 56% silt
- 43% clay

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 Figures 1 and 5 in Appendix D.

1.4.7. Clay

Native clay was encountered at all borehole locations underlying the silty clay fill or native silty clay at depths ranging from 1.5 to 3.2 m below the ground surface (elevations 202.3 to 204.2 m) and extended to the sandy silty clay till or silty clay till at depths ranging from 3.4 to 4.6 m below the ground surface. The clay was silty, brown in colour, and in a moist state. The SPT "N" values ranged from 3 to 16 blows per 305 mm penetration and undrained shear strengths obtained from in-situ shear vane tests ranged from 43 to 107 kPa, classifying the clay as firm to very stiff in consistency. Laboratory testing consisting of eight (8) moisture content determinations, two (2) grain size analyses, and two (2) Atterberg Limits tests were carried out with the results summarized below:

Moisture Content:

- 27% to 36%

Grain Size Analysis:

- 0% gravel
- 0% to 1% sand
- 24% to 26% silt
- 73% to 76% clay

Atterberg Limits:

- 64% liquid limit
- 27% plastic limit
- 37% plasticity index

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 Figures 1 and 5 in Appendix D.

1.4.8. Sandy Clayey Silt Till / Silty Clay Till

Native sandy clayey silt till or silty clay till was encountered at all borehole locations underlying the clay at depths ranging from 3.4 to 4.6 m below the ground surface (elevations 201.3 to 202.5 m) and extended to the bedrock surface at depths ranging from 4.8 to 5.8 m below the ground surface. The till contained trace gravel, was brown in colour, and in a moist state. The SPT "N" values ranged from 4 to 10 blows per 305 mm penetration, classifying the till as firm to stiff in consistency. Laboratory testing consisting of eight (8) moisture content determinations, two (3) grain size analyses, and two (3) Atterberg Limits tests were carried out with the results summarized below:

Moisture Content:

- 7% to 24%

Grain Size Analysis:

- 5% to 8% gravel

Atterberg Limits:

- 21% to 42% liquid limit
- 13% to 20% plastic limit
- 8% to 22% plasticity index

- 13% to 30% sand
- 35% to 44% silt
- 20% to 46% clay

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 Figures 2 and 6 in Appendix D.

1.4.9. Limestone Bedrock

Bedrock or auger refusal on assumed bedrock was encountered at all borehole locations at depths ranging from 4.8 to 5.8 m below the ground surface (Elev. 200.0 to 201.3 m). The bedrock was confirmed by retrieving 0.3 to 3.2 m long HQ rock cores from Boreholes BH-5 and BH-3, respectively. The bedrock surface depth and elevation encountered at the drilled borehole locations are listed in Table 1.1 below. Photographs of the rock cores are included in Appendix E.

Table 1.1 Depth and elevation of bedrock surface

Borehole	Depth Below Ground Surface (m)	Elevation (m)	Comments
BH-1	5.5	200.0	Auger refusal on assumed bedrock surface
BH-2	4.8	200.4	Auger refusal on assumed bedrock surface
BH-3	5.8	200.5	Bedrock cored for depth of 3.2 m
BH-4	5.6	200.8	Auger refusal on assumed bedrock surface
BH-5	5.2	201.3	Bedrock cored for depth of 0.3 m

Based on the rock cores recovered, the bedrock consists of limestone. In general, the rock samples are described as light grey in colour, with narrow to wide joint spacing. The joints are flat to vertical in orientation and the joint surfaces are rough undulating to rough planar. The Rock Quality Designation (RQD) measured on the rock core samples ranged from approximately 0% to 77%, indicating a rock mass of very poor to good quality, but based on the limited sampling, was typically of fair to good quality.

1.5. Groundwater & Surface Water Conditions

Groundwater conditions were monitored in the open boreholes during and upon completion of the drilling operations. Groundwater was encountered at Boreholes BH-1, BH-2, and BH-4 at depths ranging from 4.1 to 5.5 m below grade upon completion of the drilling operations. Boreholes BH-3 and BH-5 remained dry prior to advancing the rock coring equipment. Since the soil encountered at the site is low permeable (i.e. silty clay/clay) the groundwater levels are not considered to have stabilized during the short term of the investigation.

At the time of investigation surficial flow of creek water through the culvert was observed to be at approximately Elev. 204.9 m.

Seasonal variations in the water table should be expected, with higher levels occurring during wetter periods of the year and lower levels during drier periods.

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)
BH-1 SS3 Native Silty Clay	8.31	255	51	1,770	0.565	264

PART II: **ENGINEERING DISCUSSION & 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 15+843, approximately 1.89 km east of Haldimand Road 55 in Jarvis, 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 our observation on the site, the existing culvert is an arch concrete rigid frame, probably, open footing structure having a span of approximately 2.8 m and rise of approximately 1.5 m. It is approximately 18.4 m long. It is understood that the existing culvert would be replaced with a new culvert along the same alignment with no or minimum grade change. At the time of writing this report, the type of the new culvert is not known. 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 and corrugated steel plate culvert.

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 provided to us at October 13, 2016 together with the MTO request email. 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) The highway embankment is about 1.8 m high with side slopes about 5H:1V at the culvert inlet and outlet. The current elevation of the crest of the roadway is about 206.4 m. The elevation of the bottom of the creek at the inlet and outlet was about 204.6 m.
- b) Below the asphalt layer, the highway embankment consists of granular fill (0.5 m to 1.5 m thick) underlain by firm to stiff silty clay fill (~0.8 m to 1.8 m thick) underlain by loose gravelly sand fill (BH-4; ~ 0.8 m thick).
- c) The embankment fill is underlain by native stiff silty clay (BH-5; ~0.5 m thick) and firm to very stiff clay (~1.5 m to 2.3 m thick) followed by firm sandy clayey silt/silty clay till (~0.6 m to 2.0 m

- thick). Below the till the overburden soil is underlain by bedrock encountered at elevation between 200.5 m and 201.3 m.
- d) At the inlet and outlet, the layer of topsoil (~0.05 m to 0.1 m thick) underlain by soft to firm silty clay fill (0.8 m to 1.1 m thick) and followed by firm to stiff silty clay (~0.6 m to 2.0 m thick), firm to very stiff clay (~0.8 m to 1.9 m thick) and stiff to very stiff sandy clayey silt till (~1.4 m to 1.5 m thick). The soil is underlain by bedrock at Elev. 200.0 m at the inlet and 200.4 at the outlet side.
 - e) The foundation soil at the invert of the new culvert is anticipated to be native firm to stiff clay/silty clay at about Elev. 204.0 m. The 'N' values ranged from 5 to 16.
 - f) At the time of investigation, the approximate stream water elevation at the inlet and outlet was about 204.9 m (i.e the depth of the water ~0.3 m). The groundwater table in the embankment fill is expected to be at approximate elevation 204.9 m, or slightly higher. However, seasonal variations in the water table should be expected, with higher levels occurring during wetter periods of the year (such as spring thaw and late fall) and lower levels during drier periods. Some groundwater mounding within the embankment and perched water would be anticipated.

2.3. Structure Foundations

For preliminary design purpose, several possible options are considered for the replacement of the existing culvert:

- 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,

Based on the subsurface information obtained from the site investigation, the native firm to stiff clay/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 understood that regardless of the option which will be selected, the existing arch concrete 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 2.10 below.

Areas of any loose and/or soft soils encountered below the existing embankment should be excavated and removed to firm bearing of native soils and the grade restored with engineered fill. If the depth of excavation to remove unstable soils is significant, 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 ranked highest for the criteria evaluated.

Table 2.1 Evaluation of foundation alternatives

Options	Advantages	Disadvantages	Relative Costs	Risks/Consequences	Rank
Precast rigid frame box culvert	<ul style="list-style-type: none"> ▪ Straightforward construction ▪ Reduced construction period, consequently traffic management and water control period ▪ Reduced 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 ▪ Requires 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 	<ul style="list-style-type: none"> ▪ 1
Cast-in-place rigid frame box culvert	<ul style="list-style-type: none"> ▪ Suitable if site is not conducive to heavy equipment for installation of precast sections ▪ Reduced 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 ▪ Requires 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 ▪ Risk of disturbance of base during construction 	<ul style="list-style-type: none"> ▪ 3
Rigid frame open footing concrete culvert	<ul style="list-style-type: none"> ▪ Wider span may be considered to maintain existing channel ▪ High geotechnical resistance available 	<ul style="list-style-type: none"> ▪ Deeper excavation or below water excavation may required ▪ More extensive dewatering system may required ▪ May require placement of lean concrete 	<ul style="list-style-type: none"> ▪ Likely more expensive than Option 1 	<ul style="list-style-type: none"> ▪ Risk of unacceptable differential settlements if the entire foundation is not supported on the competent soil ▪ Risk of delay in construction due to deeper excavation below water if proper dewatering is not maintained ▪ Higher risk of scour 	<ul style="list-style-type: none"> ▪ 4
Corrugated steel pipe culvert	<ul style="list-style-type: none"> ▪ Straightforward construction 	<ul style="list-style-type: none"> ▪ Requires 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 	<ul style="list-style-type: none"> ▪ 2

Options	Advantages	Disadvantages	Relative Costs	Risks/ Consequences	Rank
	<ul style="list-style-type: none"> Reduced construction period, consequently traffic management and water control period Reduced excavation depth 	<ul style="list-style-type: none"> Potential for corrosion 		is not supported on the competent soil <ul style="list-style-type: none"> Risk of structure segment loss due to corrosion 	

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 and retaining wall 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	~204.0 or below	3.0 to 4.0	Minimum 300 mm compacted granular material (Granular A or Granular B Type II) over native firm clay/silty clay	225	150
Rigid frame open footing concrete culvert and retaining wall	~203.3*	1.0	Native firm to stiff clay/silty clay	225	150

Notes:

*Below the frost line

** for maximum settlement of 25 mm

It is assumed 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. As indicated above, If the

depth of excavation to remove unstable soils is significant, 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. Given that no (or minimal) 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 clay/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. 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.

2.4. Lateral Earth Pressure

Culvert walls and temporary shoring 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. These recommendations assume level backfill and ground surface behind the walls.

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. This would normally be the case for concrete box culverts.

The effect of compaction surcharge should be taken into account in the calculations of active and at-rest earth pressures. The lateral pressure due to compaction should be taken as at least 12 kPa at the surface, and its magnitude should be assumed to diminish linearly with depth to zero at the depth where the active (or at-rest) pressure is equal to 12 kPa. This pressure distribution should be added to the calculated active (or at-rest) pressure. Notwithstanding, lighter compaction equipment and smaller lifts should be used adjacent to culvert walls to prevent overstressing.

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

Table 2.4 Material types and earth pressure properties

Material	Unfactored Friction Angle ϕ'	Coefficient of Active Earth Pressure (K_a)	Coefficient of Passive Earth Pressure (K_p)	Coefficient of Earth Pressure At-Rest (K_0)	Unit Weight γ kN/m ³
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 (stiff)	29	0.35	2.88	0.52	19
Clay (firm to stiff)	27	0.37	2.66	0.54	18
Sandy Silty Clay/Silty Clay (firm)	29	0.35	2.88	0.52	19

2.5. Seismic and Liquefaction Potential Consideration

Seismic characterization of the site must be compliant with the Canadian Highway Bridge Design Code CHBDC (CAN/CSA-S6-14). The potential for seismic loading must be considered for design in accordance with Section 4.4 of the 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 stiff silty clay/clay and firm sandy silty clay/silty clay till within first 5m below ground which are underlain by bedrock. Based on these soil characteristics, the site class for this site is estimated to be Class "C" according to Table 4.1.

From the Natural Resources Canada website, 2015 NBCC seismic hazard values are obtained using the site location coordinates (42°53'25.56"N, 80°4'13.70"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 conditions 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 two 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
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 existing culvert is 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 H.

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

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 H)</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 3.0 m high required to maintain one lane of traffic High cost of roadway protection system Large amount of soil to be excavated Need to temporarily control existing creek water Risk of cost overrun and inability to finish job: low to moderate 	Relatively more expensive than full road closure due to high costs of roadway protection system	1
<p>OPTION 1.B</p> <p>Half-and-half Construction with Braced or Anchored Cut Sides (Figure H3.B, Appendix H)</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 3.0 m high required to maintain one lane of traffic if steel decking is not possible High cost of roadway protection system and/or decking Require side shoring and bracing Bracing (e.g. struts) may interfere with excavation Excavation of material and placement of bracing required in limited space Need to decommission the shoring system Need to temporarily control existing creek water Risk of cost overrun and instability to finish job: low to moderate 	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	2
<p>OPTION 2</p>	<ul style="list-style-type: none"> Existing culvert will completely remove and replaced with new culvert 	<ul style="list-style-type: none"> Traffic interruption 	Relatively less expensive than	3

Installation Method	Advantages	Disadvantages	Relative Cost	Ranking
Full Road Closure using Existing Local Roadways and Open Cut Unsupported Excavation (Figure H1, Appendix H)	<ul style="list-style-type: none"> • No construction of detour roads or roadway protection required • No excavation support required • Install entire new culvert at once • Straightforward construction • Short mobilization time • Low capital investment; cost saving in time and materials required for construction 	<ul style="list-style-type: none"> • 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 	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	
OPTION 3 Construction of Temporary Detour and Open Cut Unsupported Excavation (Figure H2, Appendix H)	<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 	More expensive than full road closure due to high costs to build local detours	4

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 H)
2. OPTION 1.B: Half-and-half construction with braced or anchored cut sides (Figure H3.B, Appendix H)
3. OPTION 2: Full road closure using existing local roadways and open cut unsupported excavation (Figure H1, Appendix H)
4. OPTION 3: Construction of temporary detour and open cut unsupported excavation (Figure H2, Appendix H)

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 Highway 3 (see Figures H3.A, and H3.B, Appendix H). 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 the excavated 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 3.0 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 2.7. Using the half-and-half construction approach, two methods of culvert replacement were considered for this site suitable 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

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 H). The roadway protection can take the form of reversible shoring such as a soldier pile and lagging or sheet pile with rakers or anchors for horizontal support. Where the cut extends below prevailing groundwater a suitable control/system is required. Once one lane is completed the

supports can be reversed and the other lane constructed in similar fashion. The shoring system would likely be decommissioned in place. Temporary surface water flow control must be developed by a 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 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 H). Excavation in this case would have to accommodate the necessary cross-bracing such as struts. With this option, consideration would have been given to how the new culvert sections will be installed given the relatively narrow work area and potential for obstructions from the lateral bracing using struts. Installation of tiebacks could be the solution. Temporary decking could possibly be used over the supported cut to allow for excavation of both halves prior to diverting stream and backfilling. However decking would be costly. As well as Option 1.A, decommissioning of the shoring system and temporary surface water flow control must be performed/developed by a 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.

2.6.2 Detour Options (Options 2 and 3)

Both detour options, the option with full closure of Highway 3 and long detours around the area using existing local roadways (see Figure H1, Appendix H), and the option with the detour embankment construction at the site to maintain the local flow of traffic during the replacement (see Figure H2, Appendix H), allow for open cut, unsupported excavation to facilitate the replacement of the existing culvert. A major benefit of these options is that the existing culvert will be completely removed once and replaced the 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 a 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) and native firm to stiff clayey soils may be classified as a Type 3 soil above the groundwater table in conformance with the OHSA. The 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. Temporary excavation side slopes for Type 4 soils should not exceed 3H:1V where applicable. There is a potential for sloughing to occur if the trench remains open for an extended period of time (i.e. > 24 hours) or during a rainfall event. In addition, some localized surficial sloughing may be experienced in areas of perched groundwater seepage (i.e. within the embankment fill).

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

2.7 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 a 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 soldier piles and timber lagging or steel sheet pile walls may be considered for design. It should be designed based on the earth pressures coefficients and soil parameters provided in Section 2.4. The actual depth of embedment should be determined by balancing moments about the pile tip. However, considering relatively shallow depth of the bedrock at Elev. 201.5 m, the piles could extend to a maximum depth of 2.5 m below the planned excavation depth (i.e. ~3.0 m). Therefore, temporary shoring system with additional anchorage or tiebacks may be required for lateral resistance. Conventional practice is to incorporate either buried deadman anchors or soil grouted anchors. Alternatively, a system of rakers can be used for support.

Deadman anchors can be designed based on the earth pressure coefficients and soil parameters provided in Section 2.4. For this project, either continuous or individual concrete block anchors would likely be appropriate. The anchor resistance is provided by a combination of the dead weight and passive resistance. For the full passive resistance to be realized with no load transfer to the wall, the anchor needs to be fully beyond the active wedge acting on the wall. Pressure grouted soil anchors can be 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.

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

For sheet piles a PZ-22 section can be considered. The factored resistance values (per metre width of sheet pile) for the sheet piles have been calculated as 20 kN/m width of sheet pile for 2.5 m embedment (i.e., approximate tip elevation of 201.5 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.

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

2.8 Culvert Bedding

OPSDs 802.010, 802.031, 802.032, 803.010 and Figure C6.20 of (CHBDC) or OPSS 3101.150 which are included in Appendix G 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 culvert (i.e. CSP pipe) shall be completed simultaneously. At no time shall the levels on each side differ more than 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 clay/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 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 803.030 and 803.031 or OPSD 3101.150 (Appendix G).

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 clay/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.

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. Given the embankment height and side slope geometry, slope stability is not considered an issue. This analysis is provided for completeness. 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 5H:1V was developed based on the cross-sections 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 clay/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 F. Since the geometry and soil stratigraphy at the north and south side slopes are similar, the result of the slope analysis performed for the north 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)
Granular Fill	32	0	21	32	0	21
Silty Clay Fill (firm to stiff)	-	65	19	29	0	19
Silty Clay (stiff)	-	85	19	29	0	19
Clay (firm to stiff)	-	60	19	27	0	19
Sandy Clayey Silt/Silty Clay Till (firm)	-	50	18	29	0	18

The results of slope stability analyses for the 5H:1V north 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 F, respectively. For the drained and undrained soil conditions, a minimum Factor of Safety was more than 1.3, indicating that the embankment is stable. In addition, the slope stability analyses were performed for the new embankment constructed of engineered fill with same slope of 5H:1V and more than a minimum Factor of Safety of 1.3 was achieved. Furthermore, stability of the new embankment constructed of engineered fill with conventional slope of 2H:1V is also checked and more than a minimum Factor of Safety of 1.3 was achieved, suggesting that the new embankment will be stable if the same slope of 5H:1V (Figure 3 and 4 in Appendix F) or conventional slope of 2H:1V is followed (Figure 5 and 6 in Appendix F).

2.11.2 Embankment Settlement

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

2.12 Inlet and Outlet

2.12.1 Erosion Protection

The requirement for and detailed design of erosion protections measures is the responsibility of and should be carried out by the hydraulics engineer. In general, rip-rap protection should be provided where the creek enters into the culvert and discharges into the open creek. The rip-rap should extend approximately 5 m beyond the ends of the culvert and line the embankment slope to the spring line of the culvert. The size of the rip-rap is a function of the creek's hydrology. As a rule of thumb the thickness of the rip-rap should be a minimum of twice the median particle size, and 300 mm thick as a minimum.

The rip-rap configuration at the creek bed should generally follow the OPSD 810.010, which is included in Appendix G of this report.

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 G 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 may have 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 G). OPSS. PROV 1205 specifies that material used for clay seals shall be natural clay, clay mixture (1 part Bentonite powder and 3.5 parts Granular "A") or a geosynthetic clay liner (GCL). The coefficient of permeability shall not exceed 1×10^{-6} cm/s.

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

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

The chemical data indicates low resistivity of the tested soil ($< 2.000 \text{ ohm-cm}$), which indicates a severe potential for corrosion of buried metallic elements, particularly pipes and appurtenances (MTO Gravity Pipe Design Guidelines, Page 25). The maximum chloride content reported is 255 ppm ($\mu\text{g/g}$) ($> 100 \text{ ppm}$) which indicates a potential for additional corrosion.

The maximum water soluble sulphate content of the soils tested is 51 ppm ($\mu\text{g/g}$), i.e. 0.05% and being less than 0.10%, does not indicate the potential to corrode normal Portland cement concrete. These data also support our local experience.

January 23, 2017

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 Jeffrey Golder, P.Eng., Nimesh Tamrakar, M.Eng., EIT. and Silvana Micic, Ph.D., P.Eng. It was reviewed by TaeChul Kim, P.Eng. and by Stan E. Gonsalves, M.Eng., P.Eng., Designated MTO Foundation Contact. The field investigation was supervised by Aziz Abdelmessih.

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Principal Engineer
Designated MTO Foundation Contact



PART IV: **LIMITATIONS AND USE OF REPORT**

BASIS OF REPORT

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

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

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

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

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

RELIANCE ON INFORMATION PROVIDED

The evaluation and conclusions contained in the Report are based on conditions in evidence at the time of site inspections and information provided to **exp** by the Client and others. The Report has been prepared for the specific site, development, building, design or building assessment objectives and purpose as communicated by the Client. **exp** has relied in good faith upon such representations, information and instructions and accepts no responsibility for any deficiency, misstatement or

inaccuracy contained in the Report as a result of any misstatements, omissions, misrepresentation or fraudulent acts of persons providing information. Unless specifically stated otherwise, the applicability and reliability of the findings, recommendations, suggestions or opinions expressed in the Report are only valid to the extent that there has been no material alteration to or variation from any of the information provided to exp.

STANDARD OF CARE

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

COMPLETE REPORT

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

USE OF REPORT

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

REPORT FORMAT

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

Appendix A – Site Photographs



Photo 1: Looking south from centreline of culvert on eastbound lane shoulder



Photo 2: Looking north at culvert from south of Highway 3



Photo 3: Looking east toward culvert



Photo 4: Looking south at culvert location from shoulder of westbound lane

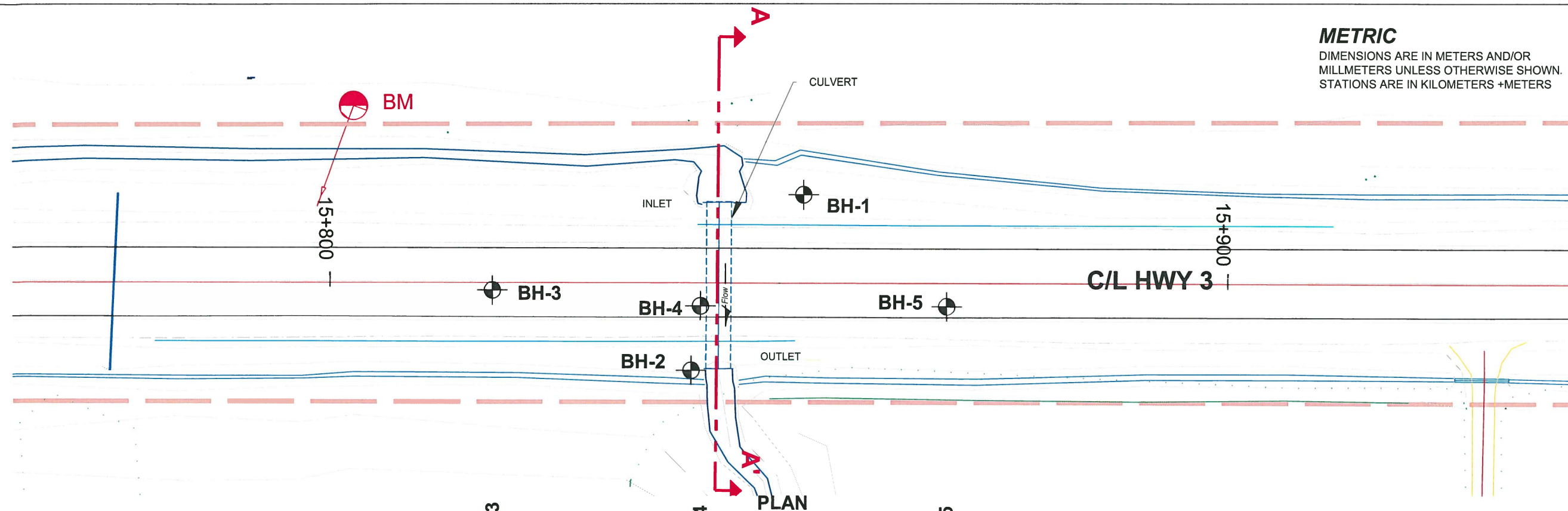


Photo 5: Looking south at culvert from north of Highway 3



Photo 6: Looking west toward culvert

Appendix B – Drawings



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

Agreement No. 3015-E-0017
Assignment No. 3
GWP - 3062-14-00

CULVERT REPLACEMENT
HWY 3, JARVIS
BOREHOLE LOCATION PLAN AND PROFILE

SHEET



LEGEND

- Location of Drilled Boreholes
- Standard Penetration Test (Blows/0.3 m)
- Water Level Upon Completion of Drilling
- Bench Mark (EL. 205.707m)

SOIL STRATA SYMBOLS

ASPHALT	CLAY	LIMESTONE BEDROCK
TOPSOIL	SILTY CLAY	
FILL	SANDY SILTY CLAY TILL	

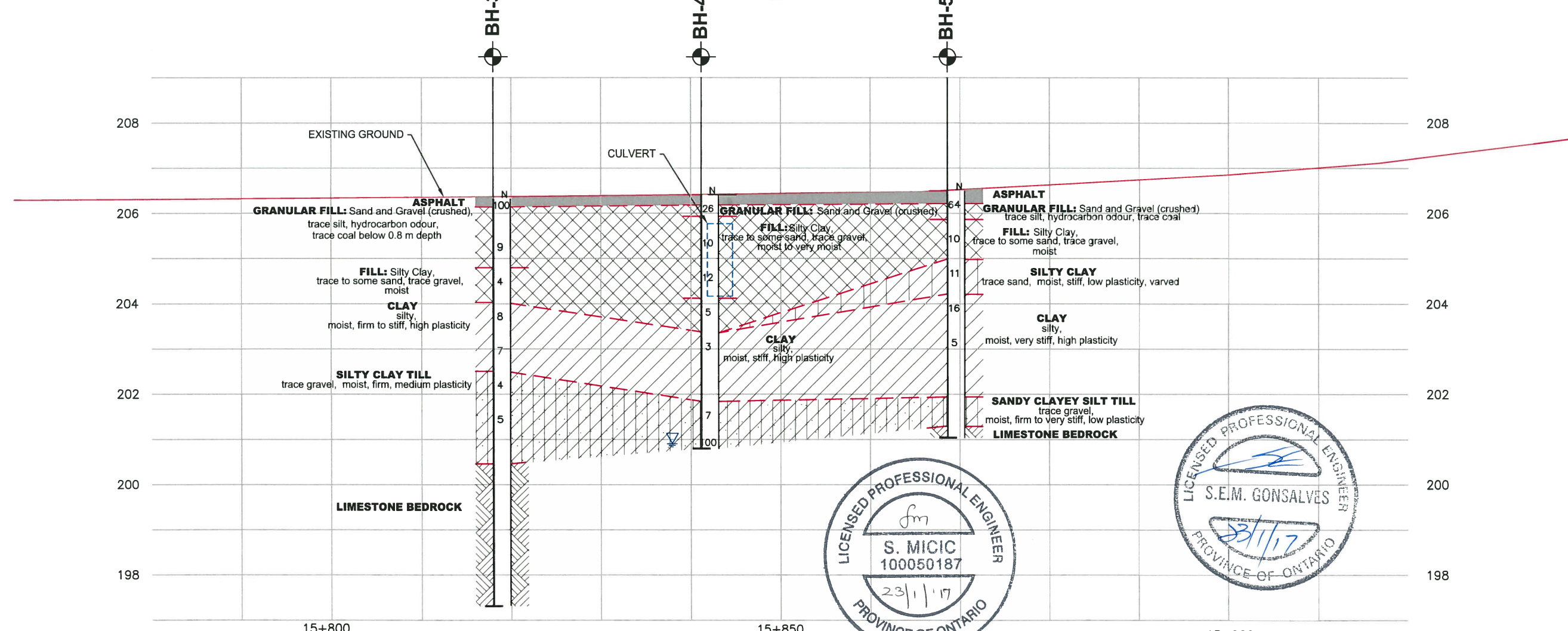
BH No.	APPROX. ELEV.	MTM CO-ORDINATES	
		NORTH	EAST
BH-1	205.5	4750241.5	258235.4
BH-2	205.2	4750220.2	258226.4
BH-3	206.3	4750225.0	258203.1
BH-4	206.4	4750227.4	258226.1
BH-5	206.5	4750232.1	258253.2

NOTE

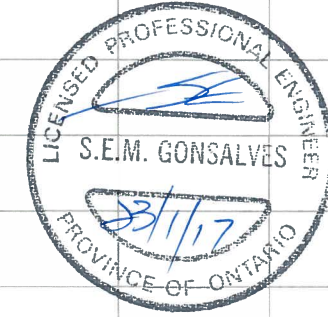
This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

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.

HOR 0 5 15 m
VERT 0 2

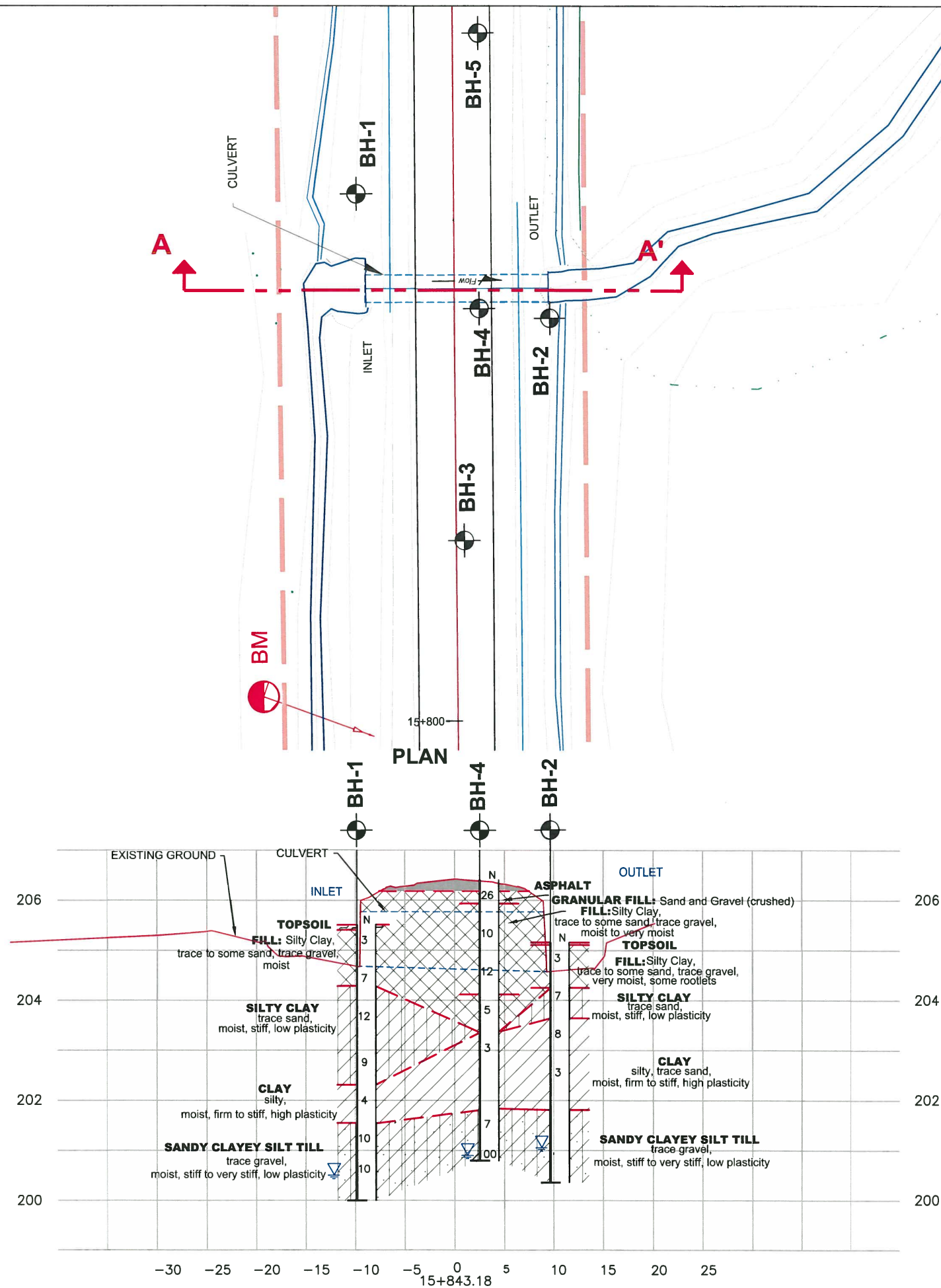


PROFILE ALONG C/L HWY 3



18/01/2017	SM	SUBMISSION FOR MTO REVIEW	
DATE	BY	DESCRIPTION	
		GEOCRES NO. 4016-27	
		PROJECT NO. ADM-00235197-C0	
SUBM'D SM	CHECKED SM	DATE	18/01/2017
DRAWN SH	CHECKED SG	APPROVED SG	DWG. 1

CON 8
LOT 10



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

Agreement No. 3015-E-0017
Assignment No. 3
GWP - 3062-14-00

**CULVERT REPLACEMENT
HWY 3, JARVIS
BOREHOLE LOCATION PLAN
AND SECTION A-A'**



SHEET

exp Services Inc.



LEGEND

- Location of Drilled Boreholes
- Standard Penetration Test (Blows/0.3 m)
- Water Level Upon Completion of Drilling
- Bench Mark (EL. 205.707m)

SOIL STRATA SYMBOLS

- ASPHALT
- TOPSOIL
- FILL
- CLAY
- SILTY CLAY
- SANDY SILTY CLAY TILL
- LIMESTONE BEDROCK

BH No.	APPROX. ELEV.	MTM CO-ORDINATES	
		NORTH	EAST
BH-1	205.5	4750241.5	258235.4
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NOTE

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18/01/2017	SM	SUBMISSION FOR MTO REVIEW	
DATE	BY	DESCRIPTION	
		GEOCRES NO. 4016-27	
		PROJECT NO. ADM-00235197-C0	
SUBMD	SM	CHECKED	SM
DRAWN	SH	CHECKED	SG
DATE	18/01/2017	APPROVED	SG
DWG.	2		

Appendix C – Borehole Logs

Explanation of Terms Used on Borehole Records

SOIL DESCRIPTION

Terminology describing common soil genesis:

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

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

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

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

Terminology describing soil structure:

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

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

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

Fissured: material breaks along plane of fracture.

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

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

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

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

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

Homogeneous: same color and appearance throughout.

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

Uniformly Graded: predominantly on grain size.

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

ISSMFE SOIL CLASSIFICATION											
CLAY	SILT			SAND			GRAVEL			COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE		
<div><div>0.002</div><div>0.006</div><div>0.02</div><div>0.06</div><div>0.2</div><div>0.6</div><div>2.0</div><div>6.0</div><div>20</div><div>60</div><div>200</div></div>											
EQUIVALENT GRAIN DIAMETER IN MILLIMETRES											
CLAY (PLASTIC) TO				FINE		MEDIUM		CRS.		FINE COARSE	
SILT (NONPLASTIC)				SAND				GRAVEL			
UNIFIED SOIL CLASSIFICATION											

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

Table a: Percent or Proportion of Soil, Pp

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

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

Table b: Apparent Density of Cohesionless Soil

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

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

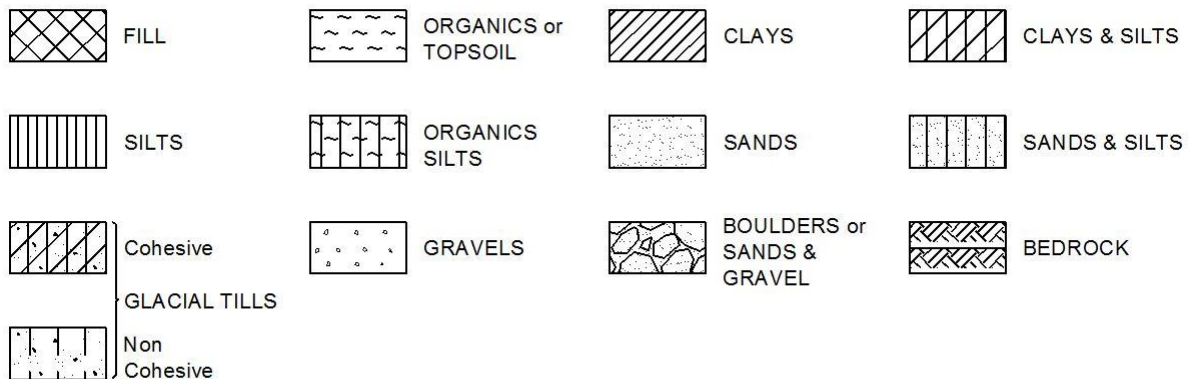
Table c: Consistency of Cohesive Soil

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

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

STRATA PLOT

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



WATER LEVEL MEASUREMENT



Open Borehole or Test Pit



Monitoring Well, Piezometer or Standpipe

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

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

STRESS AND STRAIN

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

MECHANICAL PROPERTIES OF SOIL

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

PHYSICAL PROPERTIES OF SOIL

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

Brampton, Ontario

RECORD OF BOREHOLE No BH-1

1 OF 1

METRIC

W. P. 3062-14-00 LOCATION Jarvis, ON, 258235.4 E, 4750241.5 N (MTM 10) ORIGINATED BY AA
 DIST Haldimand HWY 3 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY JG
 DATUM Geodetic DATE 2016/11/09 - 2016/11/09 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
								○ UNCONFINED	+ FIELD VANE	×	QUICK TRIAXIAL	LAB VANE					
205.5	Ground Surface																
205.4	TOPSOIL: (~100 mm thick)		1	SS	3												
0.1	FILL: silty clay, trace to some sand, trace gravel, brown, moist																
204.3			2	SS	7												
1.2	SILTY CLAY: trace sand, brown, moist, stiff, low plasticity, varved																
			3	SS	12												
			4	SS	9												
202.3																	
3.2	CLAY: silty, brown, moist, firm to very stiff, high plasticity		5	SS	4												
201.6																	
4.0	SANDY CLAYEY SILT TILL: trace gravel, brown, moist, stiff, low plasticity		6	SS	10												
	grey, increasingly sandy and gravelly below 4.6 m depth		7	SS	10												
200.0																	
5.5	Borehole terminated at 5.5 m depth due to auger refusal on assumed bedrock.																
	Notes: 1. This borehole log is to be read with the subject report and project numbers as presented above. 2. Groundwater level at 5.0 m depth upon completion of drilling.																

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

EXP RECORD OF BOREHOLE BH LOGS MTO HWY 3 GPJ ONTARIO MOT.GDT 1/18/17

Brampton, Ontario

RECORD OF BOREHOLE No BH-2

1 OF 1

METRIC

W. P. 3062-14-00 LOCATION Jarvis, ON, 258226.4 E, 4750220.2 N (MTM 10) ORIGINATED BY AA
 DIST Haldimand HWY 3 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY JG
 DATUM Geodetic DATE 2016/11/09 - 2016/11/09 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			20	40	60	80	100					
205.2	Ground Surface																
205.1	TOPSOIL: (~50 mm thick) FILL: silty clay, trace to some sand, trace gravel, dark brown, very moist, some rootlets		1	SS	3		205										
204.3																	
204.0	SILTY CLAY: trace sand, brown, moist, firm, low plasticity, varved		2	SS	7		204										
203.6																	
203.1	CLAY: silty, trace sand, brown, moist, firm to stiff, high plasticity		3	SS	8		203										
201.8			4	SS	3		202										
201.8																	
201.8	SANDY CLAYEY SILT TILL: trace gravel, brown, moist, stiff to very stiff, low plasticity		5	TW			201										
200.4																	
200.4	Borehole terminated at 4.8 m depth due to auger refusal on assumed bedrock.																
	Notes: 1. This borehole log is to be read with the subject report and project numbers as presented above. 2. Groundwater level at 4.1 m depth upon completion of drilling.																

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

EXP RECORD OF BOREHOLE BH LOGS MTO HWY 3.GPJ ONTARIO MOT.GDT 1/18/17

Brampton, Ontario

RECORD OF BOREHOLE No BH-3

1 OF 1

METRIC

W. P. 3062-14-00 LOCATION Jarvis, ON, 258203.1 E, 4750225.0 N (MTM 10) ORIGINATED BY AA
 DIST Haldimand HWY 3 BOREHOLE TYPE Continuous Flight Hollow Stem Augers/ HQ Coring COMPILED BY JG
 DATUM Geodetic DATE 2016/11/08 - 2016/11/08 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								
206.3	Road Surface																			
206.1	ASPHALT: (~175 mm thick)		1	SS	100															
0.2	GRANULAR FILL: sand and gravel (crushed), trace silt (~1065 mm thick)																			
	hydrocarbon odour, trace coal below 0.8 m depth		2	SS	9															
204.8																				
1.5	FILL: silty clay, trace to some sand, trace gravel, brown, moist		3	SS	4												1 13 55 31			
204.0																				
2.3	CLAY: silty, brown, moist, firm to stiff, high plasticity		4	SS	8															
			5	SS	7															
202.5																				
3.8	SILTY CLAY TILL: trace gravel, brown, moist, firm, medium plasticity		6	SS	4												6 13 35 46			
			7	SS	5															
200.5																				
5.8	LIMESTONE BEDROCK: light grey, bedding joints with flat orientation, joint spacing is narrow to wide (2 to 32 cm), joint surfaces are rough undulating to rough planar		8	HQ																
	RUN 1 (5.84-6.25 m): recovery 100%, RQD 70% (fair)																			
	RUN 2 (6.25-7.77 m): recovery 100%, RQD 68% (fair)		9	HQ																
	RUN 3 (7.77-8.99 m): recovery 100%, RQD 77% (good)																			
			10	HQ																
197.3																				
9.0	Borehole terminated at 9.0 m depth in bedrock.																			
	Notes: 1. This borehole log is to be read with the subject report and project numbers as presented above. 2. Borehole remained dry upon completion of auger drilling.																			

EXP RECORD OF BOREHOLE BH LOGS MTO HWY 3.GPJ ONTARIO MOT.GDT 1/18/17

+ 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 Jarvis, ON, 258226.1 E, 4750227.4 N (MTM 10) ORIGINATED BY AA
 DIST Haldimand HWY 3 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY JG
 DATUM Geodetic DATE 2016/11/08 - 2016/11/08 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			20	40	60	80	100					
206.4	Road Surface																
206.2	ASPHALT: (~225 mm thick)																
0.2	GRANULAR FILL: sand and gravel (crushed), trace silt, hydrocarbon odour, trace coal (~250 mm thick) FILL: silty clay, trace to some sand, trace gravel, grey, moist to very moist	0.2 0.5	1	SS	26		206										
205.9			2	SS	10												
0.5			3	SS	12		205										
204.1	FILL: GRAVELLY SAND with some silt and some clay grey, moist, hydrocarbon odour mixed with silty clay below 2.6 m depth	2.3	4	SS	5		204										
203.3			5	SS	3		203										
3.1	CLAY: silty, brown, moist, stiff, high plasticity																
201.8	SANDY CLAYEY SILT TILL: trace gravel, brown, moist, firm, low plasticity	4.6	6	SS	7		202										
200.8			7	SS	100		201										
5.6	Borehole terminated at 5.6 m depth due to auger refusal on assumed bedrock. Notes: 1. This borehole log is to be read with the subject report and project numbers as presented above. 2. Groundwater level at 5.5 m depth upon completion of drilling.																

EXP RECORD OF BOREHOLE BH LOGS MTO HWY 3.GPJ ONTARIO MOT.GDT 1/18/17

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

Brampton, Ontario

RECORD OF BOREHOLE No BH-5

1 OF 1

METRIC

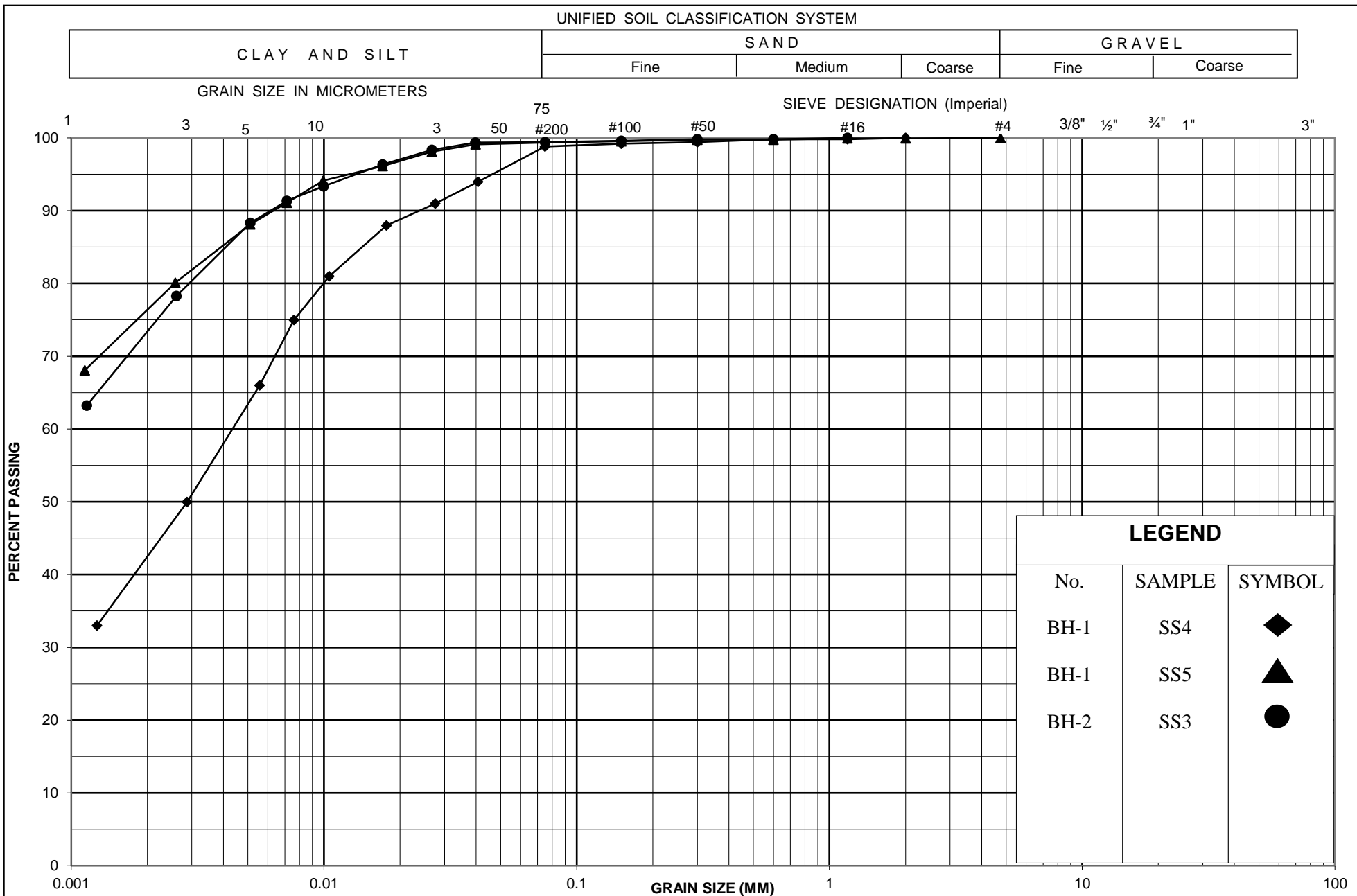
W. P. 3062-14-00 LOCATION Jarvis, ON, 258253.2 E, 4750232.1 N (MTM 10) ORIGINATED BY AA
 DIST Haldimand HWY 3 BOREHOLE TYPE Continuous Flight Hollow Stem Augers/ HQ Coring COMPILED BY JG
 DATUM Geodetic DATE 2016/11/08 - 2016/11/08 CHECKED BY SM

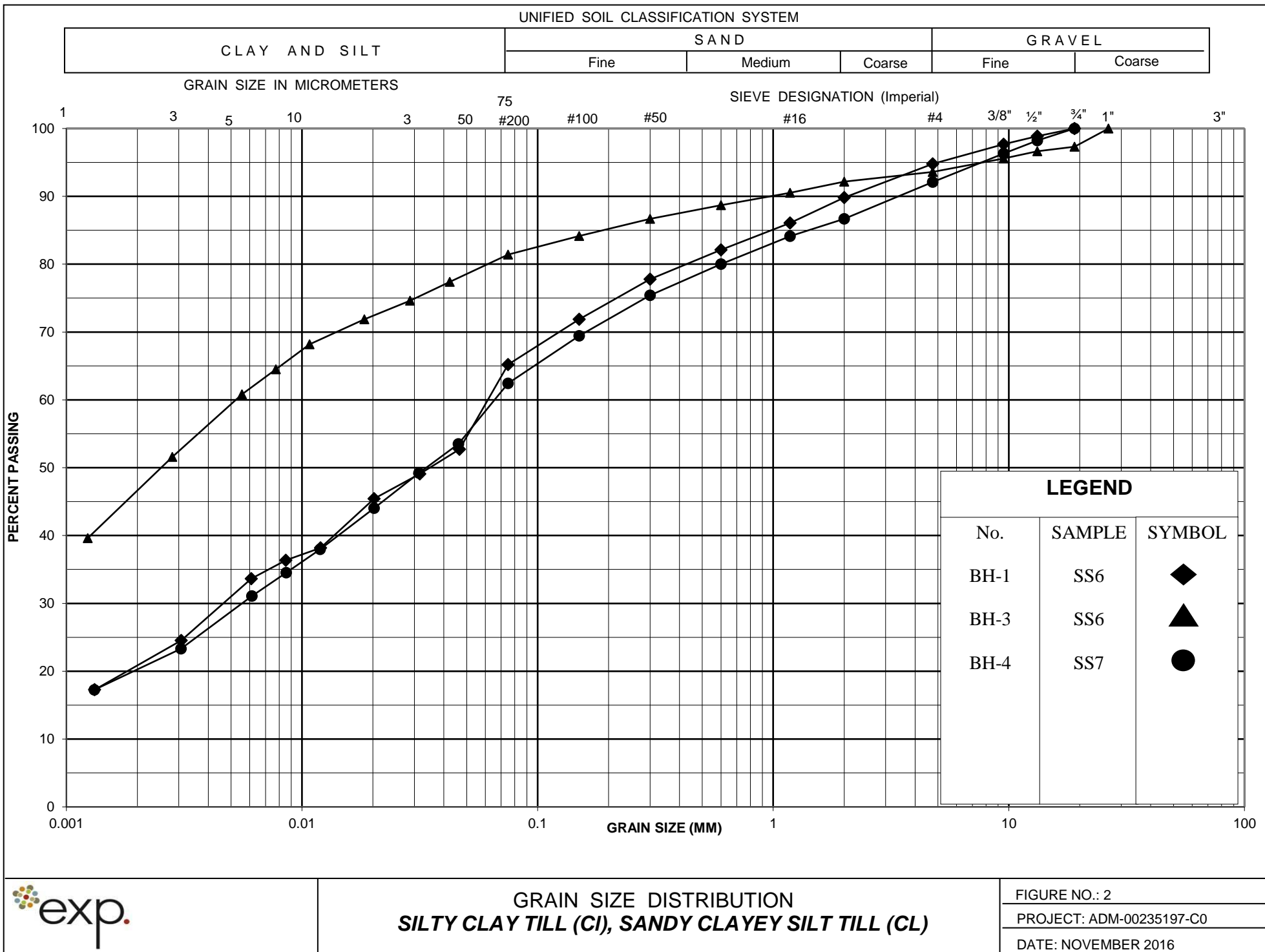
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH: Cu, KPa								
							20	40	60	80	100					
206.5	Road Surface															
	ASPHALT: (~280 mm thick)															
206.2			1	SS	64											
0.3	GRANULAR FILL: sand and gravel (crushed), trace silt, hydrocarbon odour, trace coal (~355 mm thick)															
205.9																
0.6	FILL: silty clay, trace to some sand, trace gravel, grey, moist		2	SS	10											
205.0																
1.5	SILTY CLAY: trace sand, brown, moist, stiff, low plasticity, varved		3	SS	11											
204.2																
2.3	CLAY: silty, brown, moist, very stiff, high plasticity		4	SS	16											
			5	SS	5											
201.9																
4.6	SANDY CLAYEY SILT TILL: trace gravel, brown, moist, firm to very stiff, low plasticity		6	TW												
201.3																
5.2																
201.0	LIMESTONE BEDROCK: light grey, bedding joints with flat and near vertical orientation, joint spacing is narrow to moderately wide (1 to 9 cm), joint surfaces are rough undulating to rough planar		7	HQ												
5.5																

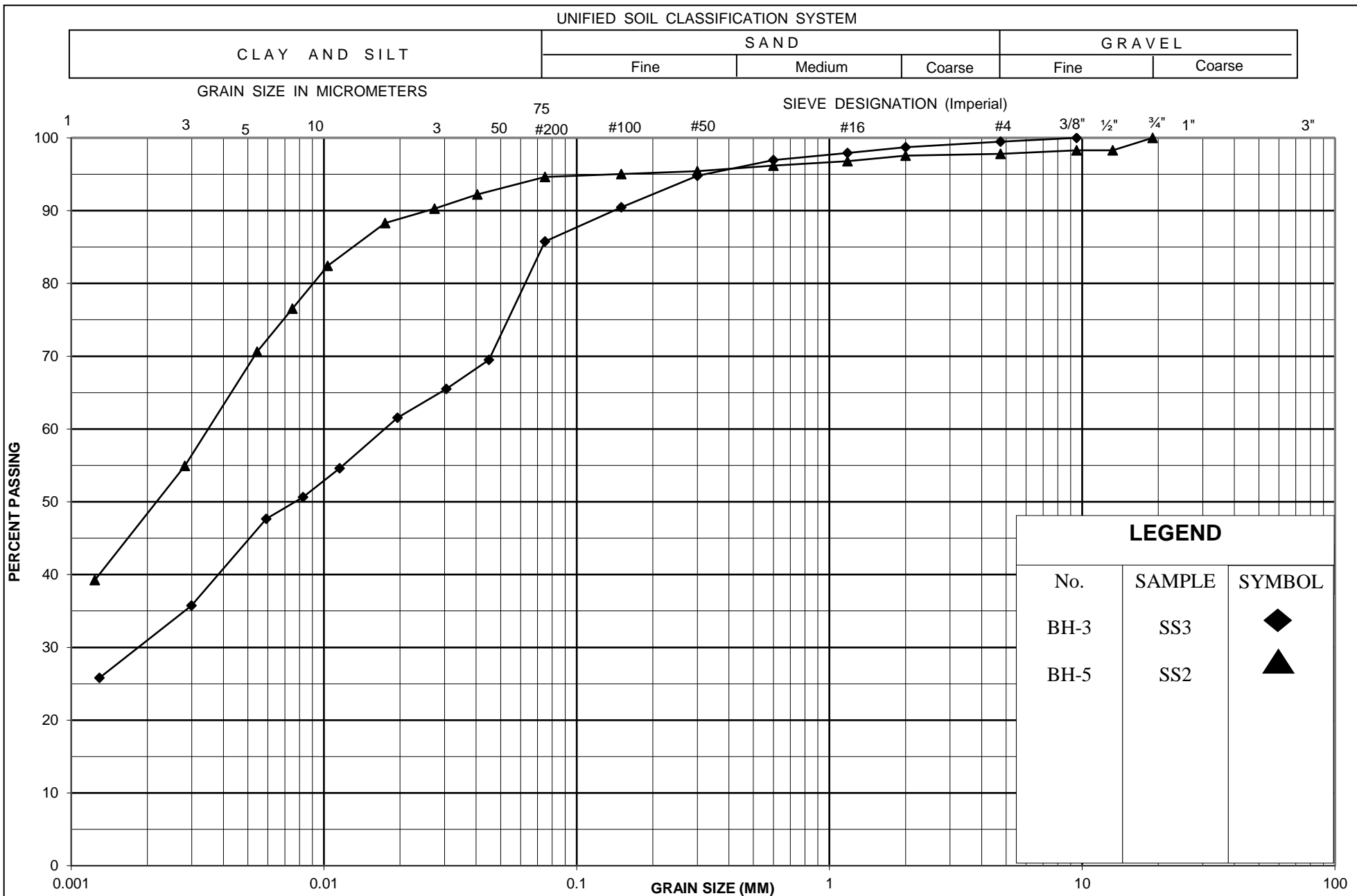
+ 3, × 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

EXP RECORD OF BOREHOLE: BH LOGS MTO HWY 3.GPJ ONTARIO MOT.GDT 1/18/17

Appendix D – Laboratory Data

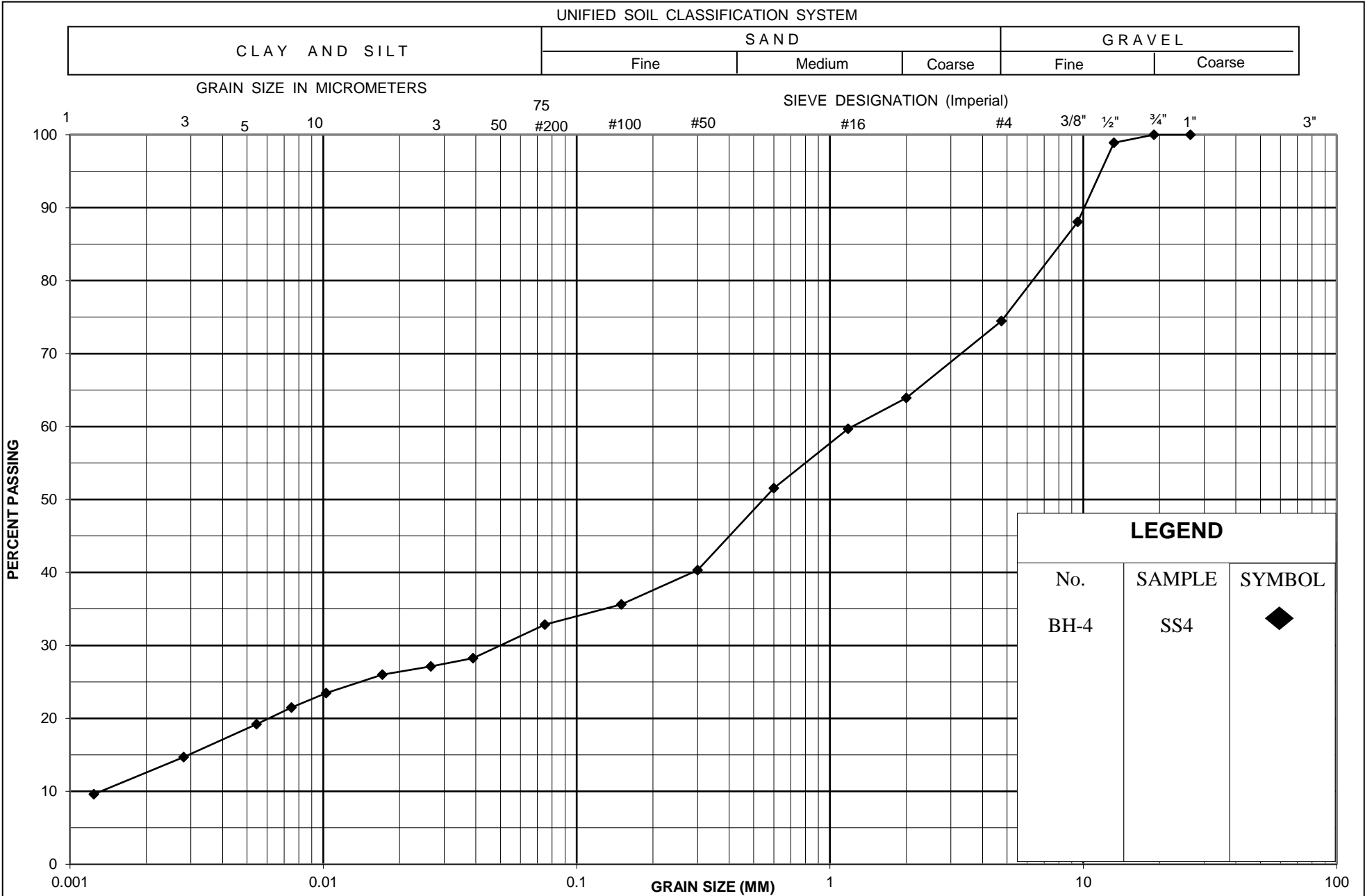






GRAIN SIZE DISTRIBUTION
FILL: SILTY CLAY (CI)

FIGURE NO.: 3
 PROJECT: ADM-00235197-C0
 DATE: NOVEMBER 2016

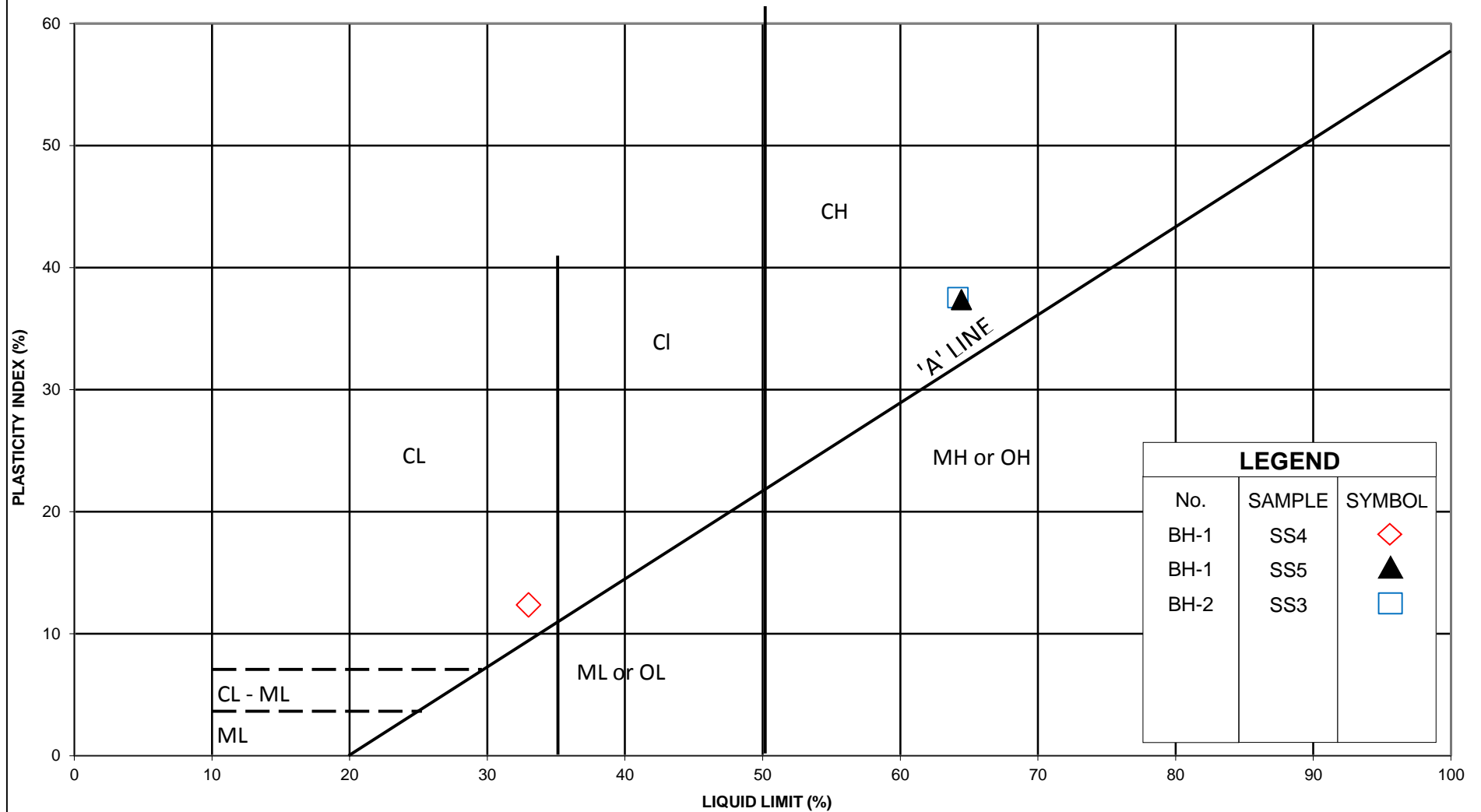


GRAIN SIZE DISTRIBUTION
FILL: GRAVELLY SAND with some silt and some clay (SC)

FIGURE NO.: 4
PROJECT: ADM-00235197-C0
DATE: NOVEMBER 2016

CULVERT REPLACEMENT

Highway 3, Jarvis, ON



PLASTICITY CHART
CLAY (CH), SILTY CLAY (CL)

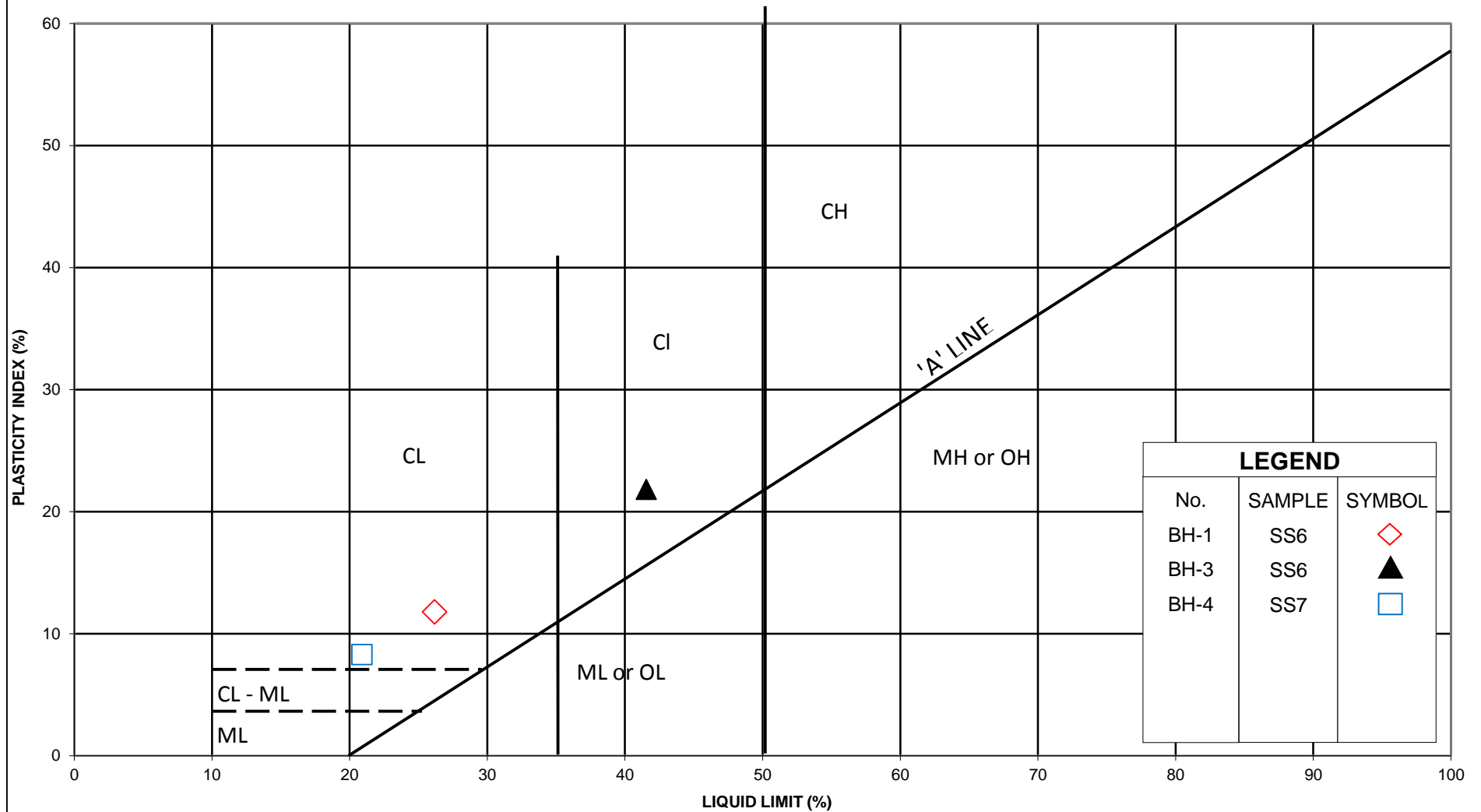
FIGURE NO.: 5

PROJECT: ADM-00235197-C0

DATE: NOVEMBER 2016

CULVERT REPLACEMENT

Highway 3, Jarvis, ON



PLASTICITY CHART

SILTY CLAY TILL (CI), SANDY CLAYEY SILT TILL (CL)

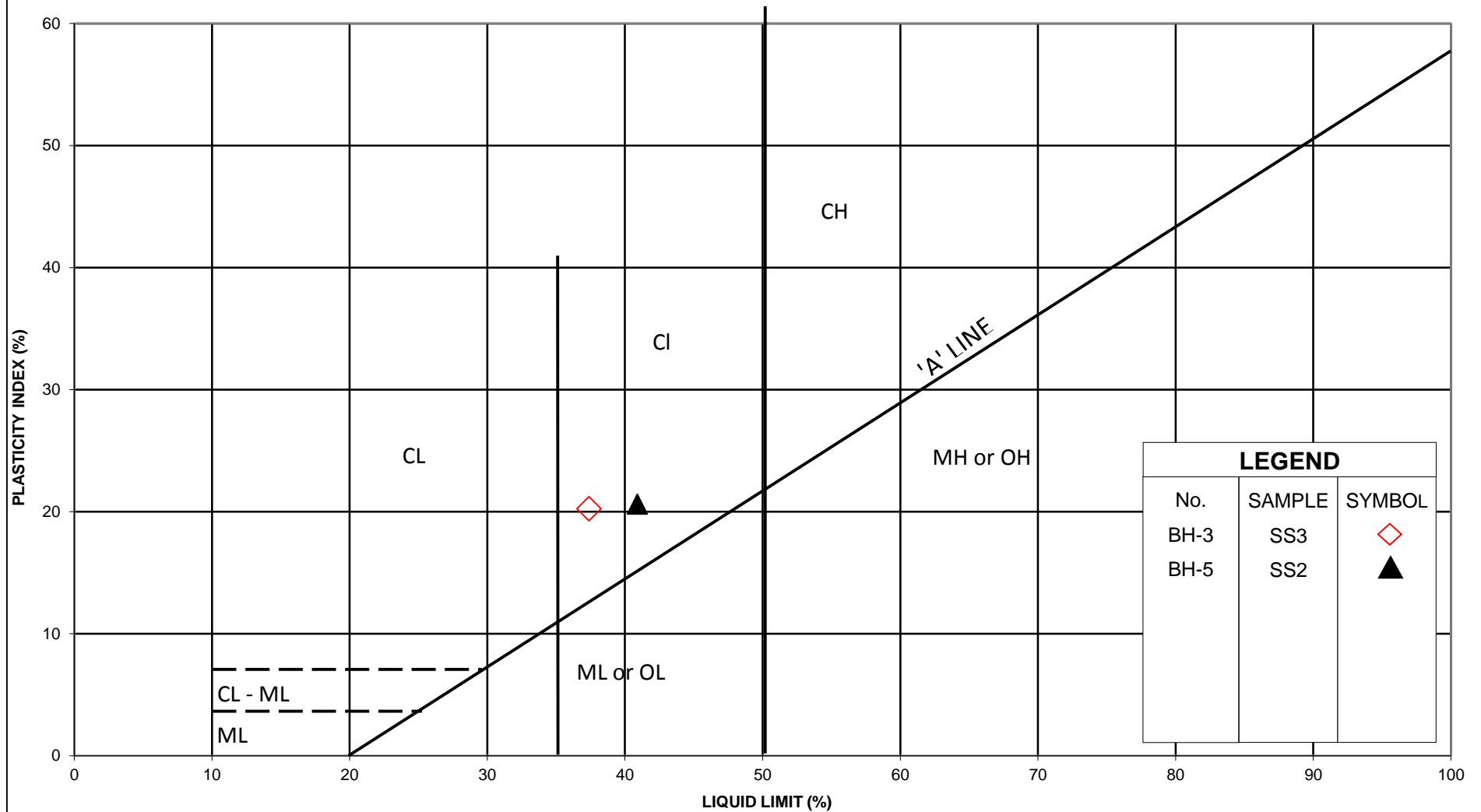
FIGURE NO.: 6

PROJECT: ADM-00235197-C0

DATE: NOVEMBER 2016

CULVERT REPLACEMENT

Highway 3, Jarvis, ON



LEGEND		
No.	SAMPLE	SYMBOL
BH-3	SS3	◇
BH-5	SS2	▲



PLASTICITY CHART

FILL: SILTY CLAY (CI)

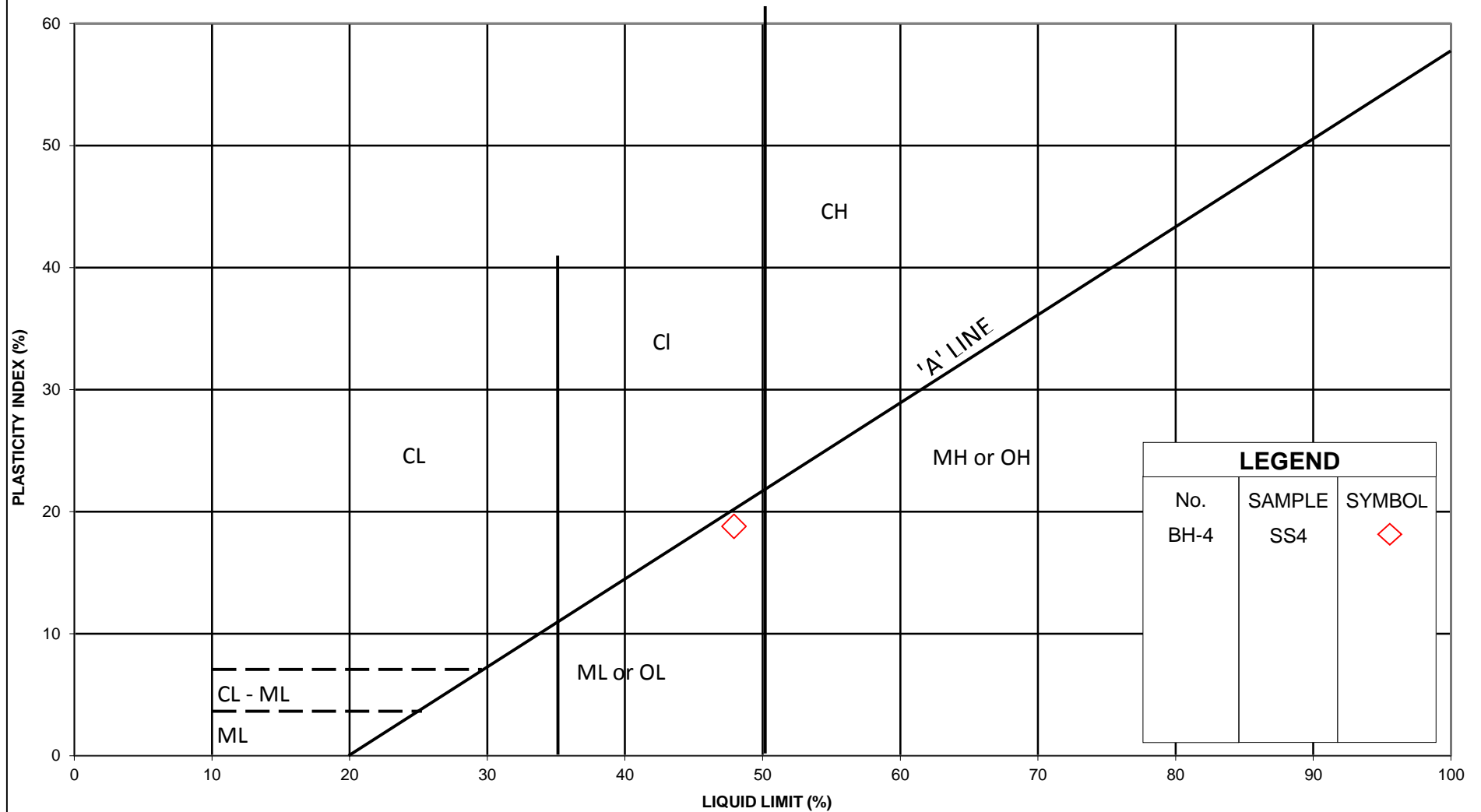
FIGURE NO.: 7

PROJECT: ADM-00235197-C0

DATE: NOVEMBER 2016

CULVERT REPLACEMENT

Highway 3, Jarvis, ON



PLASTICITY CHART

FILL: GRAVELLY SAND with some silt and some clay (SM)

FIGURE NO.: 8

PROJECT: ADM-00235197-C0

DATE: NOVEMBER 2016

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

ATTENTION TO: Jeff Golder

PROJECT: ADM-00235197-C0

AGAT WORK ORDER: 16T159963

SOIL ANALYSIS REVIEWED BY: Amanjot Bhela, Inorganic Coordinator

DATE REPORTED: Nov 18, 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: 16T159963

PROJECT: ADM-00235197-C0

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: 2016-11-14

DATE REPORTED: 2016-11-18

SAMPLE DESCRIPTION: BH-1 SS3 (5-7')

SAMPLE TYPE: Soil

DATE SAMPLED: 2016-11-08

Parameter	Unit	G / S	RDL	8012972
Sulfide	%		0.01	0.01
Chloride (2:1)	µg/g		2	255
Sulphate (2:1)	µg/g		2	51
pH (2:1)	pH Units		NA	8.31
Electrical Conductivity (2:1)	mS/cm		0.005	0.565
Resistivity (2:1)	ohm.cm		1	1770
Redox Potential (2:1)	mV		5	264

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

8012972 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).

Sulfide analysis performed by AGAT Burnaby.

Certified By:

Amanjot Bhela



Quality Assurance

CLIENT NAME: EXP. SERVICES INC.

PROJECT: ADM-00235197-C0

SAMPLING SITE:

AGAT WORK ORDER: 16T159963

ATTENTION TO: Jeff Golder

SAMPLED BY:

Soil Analysis

RPT Date: Nov 18, 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															
Sulfide	8014293		0.03	0.03	NA	< 0.01	100%	80%	120%						
Chloride (2:1)	8014326		72	72	0.0%	< 2	94%	80%	120%	104%	80%	120%	96%	70%	130%
Sulphate (2:1)	8014326		70	72	2.8%	< 2	92%	80%	120%	103%	80%	120%	98%	70%	130%
pH (2:1)	8014326		8.12	8.17	0.6%	NA	101%	90%	110%	NA			NA		
Electrical Conductivity (2:1)	8016096		0.042	0.042	0.0%	< 0.005	99%	90%	110%	NA			NA		
Redox Potential (2:1)	8014326		276	274	0.7%	< 5	102%	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:

Amanjot Bhela



Method Summary

CLIENT NAME: EXP. SERVICES INC.

AGAT WORK ORDER: 16T159963

PROJECT: ADM-00235197-C0

ATTENTION TO: Jeff Golder

SAMPLING SITE:

SAMPLED BY:

PARAMETER	AGAT S.O.P	LITERATURE REFERENCE	ANALYTICAL TECHNIQUE
Soil Analysis			
Sulfide	INOR-181-6027	modified from ASTM E1915-11	COMBUSTION
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



5835 Coopers Avenue
Mississauga, Ontario L4Z 1Y2
Ph: 905.712.5100 Fax: 905.712.5122
webearth.agatlabs.com

If this is a Drinking Water sample, please use Drinking Water Chain of Custody Form (potable water intended for human consumption)

Company: exp Services Inc.

Contact: Jeffrey Golder

Address: 80 Bancroft Street
Hamilton, ON L8E 2W5

Phone: 905.573.4000 x5022 Fax: _____

Reports to be sent to:

1. Email: jeffrey.golder@exp.com

2. Email: _____

Project:	ADM-00235197-C0
Site Location:	HWY CULVERT
Sampled By:	AA
AGAT Quote #:	PO:

Bill To Same: Yes ☒ No ☐

Company: _____
Contact: _____
Address: _____
Email: _____

(Please check all applicable boxes)

<input type="checkbox"/> Regulation 153/04	<input type="checkbox"/> Sewer Use	<input type="checkbox"/> Regulation 558
Table _____ <i>Indicate One</i>	<input type="checkbox"/> Sanitary	<input type="checkbox"/> CCME
<input type="checkbox"/> Ind/Com	<input type="checkbox"/> Storm	<input type="checkbox"/> Prov. Water Quality Objectives (PWQO)
<input type="checkbox"/> Res/Park		<input type="checkbox"/> Other
<input type="checkbox"/> Agriculture		
Soil Texture (<i>Check One</i>)	Region _____ <i>Indicate One</i>	
<input type="checkbox"/> Coarse		
<input type="checkbox"/> Fine		

Is this submission for a
Record of Site Condition?

☐ Yes ☐ No

Report Guideline on Certificate of Analysis

☐ Yes ☐ No

B	Biota
GW	Ground Water
O	Oil
P	Paint
S	Soil
SD	Sediment
SW	Surface Water

[illegible]

Samples Relinquished By (Print Name and Sign)

Jeffrey Golder

Samples Relinquished By (Print Name and Sign)

Samples Relinquished By (Print Name and Sign)

Date	Time
Nov. 14, 2016	10:00

Date	Time
------	------

Date	Time
------	------

Samples Received By (Print Name and Sign)

Samples Received By (Print Name and Sign)

Samples Received By (Print Name and Sign)

Date	Time
------	------

Date	Time
------	------

Date	Time
------	------

Page _____ of _____

 $\text{N}^{0.4}$

Laboratory Use Only

Work Order #: 16T159963

Cooler Quantity: 20-721021-0
Arrival Temperatures: 14.7 15.0 15.0

Custody Seal Intact: ☒ Yes ☐ No ☐ N/A

Notes: ICE

Turnaround Time (TAT) Required:

Regular TAT ☒ 5 to 7 Business Days

Rush TAT (Rush Surcharges Apply)

☐ 3 Business Days ☐ 2 Business Days ☐ 1 Business Day

OR Date Required (Rush Surcharges May Apply):

Please provide prior notification for rush TAT
**TAT is exclusive of weekends and statutory holidays*

Appendix E – Rock Core Photographs

Borehole BH-3

Run No. 1

Sample Depth: 5.84 – 6.25 m

Sample Elevation: 199.58 – 199.17 m)

Recovery 100%, RQD 70% (fair quality)



Photo 1. Core Sample for BH3 from Elevation 199.58 m to 199.17 m

Borehole BH-3

Run No. 2

Sample Depth: 6.25 – 7.77 m

Sample Elevation: (199.17 – 197.65 m)

Recovery 100%, RQD 68% (fair quality)



Photo 2. Core Sample for BH3 from Elevation 199.17m to 197.65 m

Borehole BH-3

Run No. 3

Sample Depth: 7.77 – 8.99 m

Sample Elevation: (197.65 – 196.43 m)

Recovery 100%, RQD 77% (good quality)



Photo 3. Core Sample for BH3 from Elevation 197.65 m to 196.43 m

Borehole BH-5

Run No. 1

Sample Depth: 5.84 – 6.25 m

Sample Elevation: (199.77 – 199.36 m)

Recovery 100%, RQD 0% (very poor quality)



Photo 4. Core Sample for BH1 from Elevation 199.77 m to 199.36 m

Appendix F – Slope Stability Analysis

Culvert on Hwy 3 Stability of Embankment Slope Undrained Static Condition

Name: Granular Fill Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion': 0 kPa Phi': 32 °
Name: Silty Clay Fill (Firm to Stiff) Model: Undrained (Phi=0) Unit Weight: 19 kN/m³ Cohesion': 65 kPa
Name: Silty Clay (Stiff) Model: Undrained (Phi=0) Unit Weight: 19 kN/m³ Cohesion': 85 kPa
Name: Clay (Firm to Stiff) Model: Undrained (Phi=0) Unit Weight: 19 kN/m³ Cohesion': 60 kPa
Name: Sandy Clayey Silt Till (Firm to Stiff) Model: Undrained (Phi=0) Unit Weight: 18 kN/m³ Cohesion': 50 kPa
Name: Bedrock Model: Bedrock (Impenetrable)

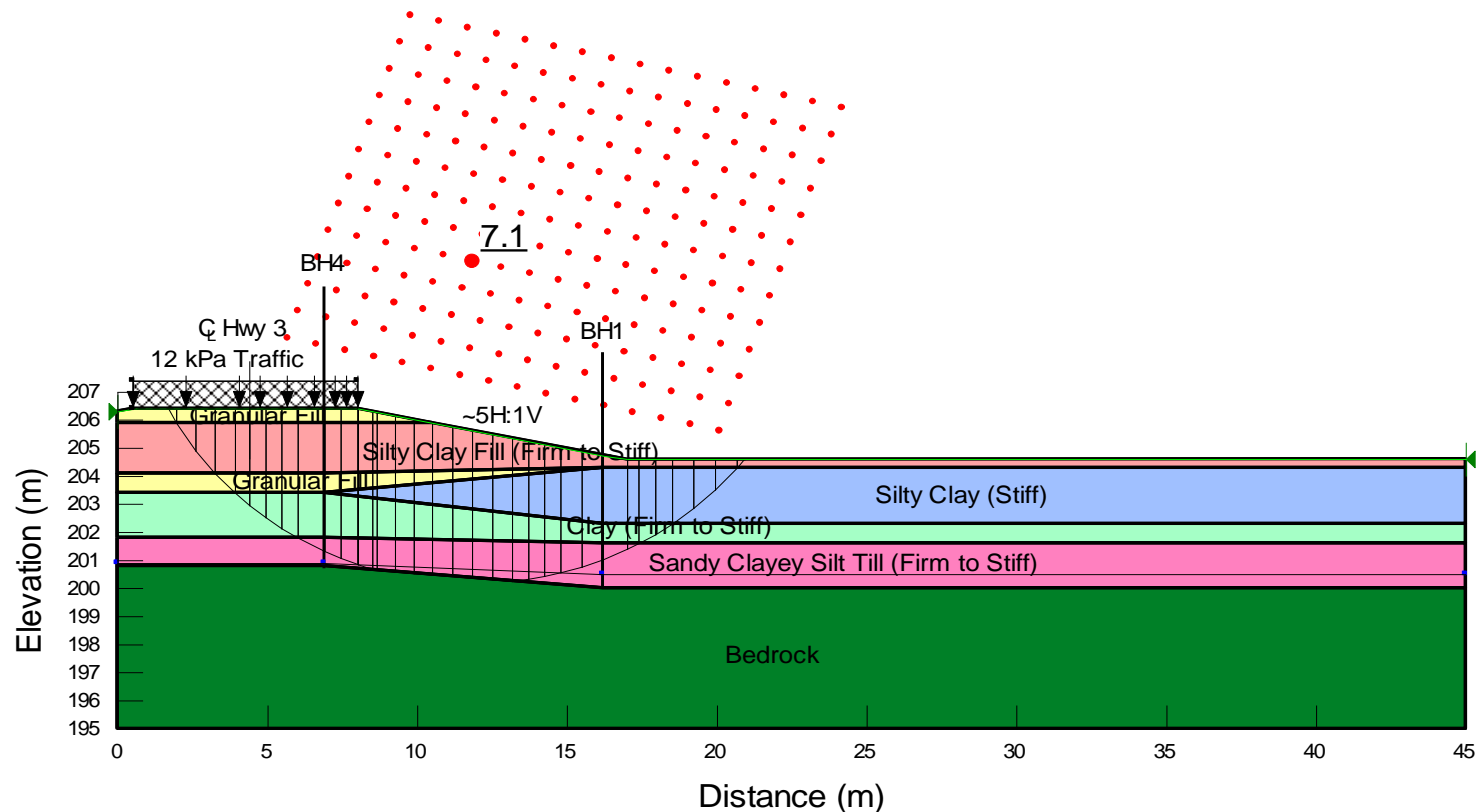


Figure 1: Slope stability analysis for embankment slope – undrained static conditions

Culvert on Hwy 3 Stability of Embankment Slope Drained Static Condition

Name: Granular Fill Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion': 0 kPa Phi': 32 °
Name: Silty Clay Fill (Firm to Stiff) Model: Mohr-Coulomb Unit Weight: 19 kN/m³ Cohesion': 0 kPa Phi': 29 °
Name: Silty Clay (Stiff) Model: Mohr-Coulomb Unit Weight: 19 kN/m³ Cohesion': 0 kPa Phi': 29 °
Name: Clay (Firm to Stiff) Model: Mohr-Coulomb Unit Weight: 19 kN/m³ Cohesion': 0 kPa Phi': 27 °
Name: Sandy Clayey Silt Till (Firm to Stiff) Model: Mohr-Coulomb Unit Weight: 18 kN/m³ Cohesion': 0 kPa Phi': 29 °
Name: Bedrock Model: Bedrock (Impenetrable)

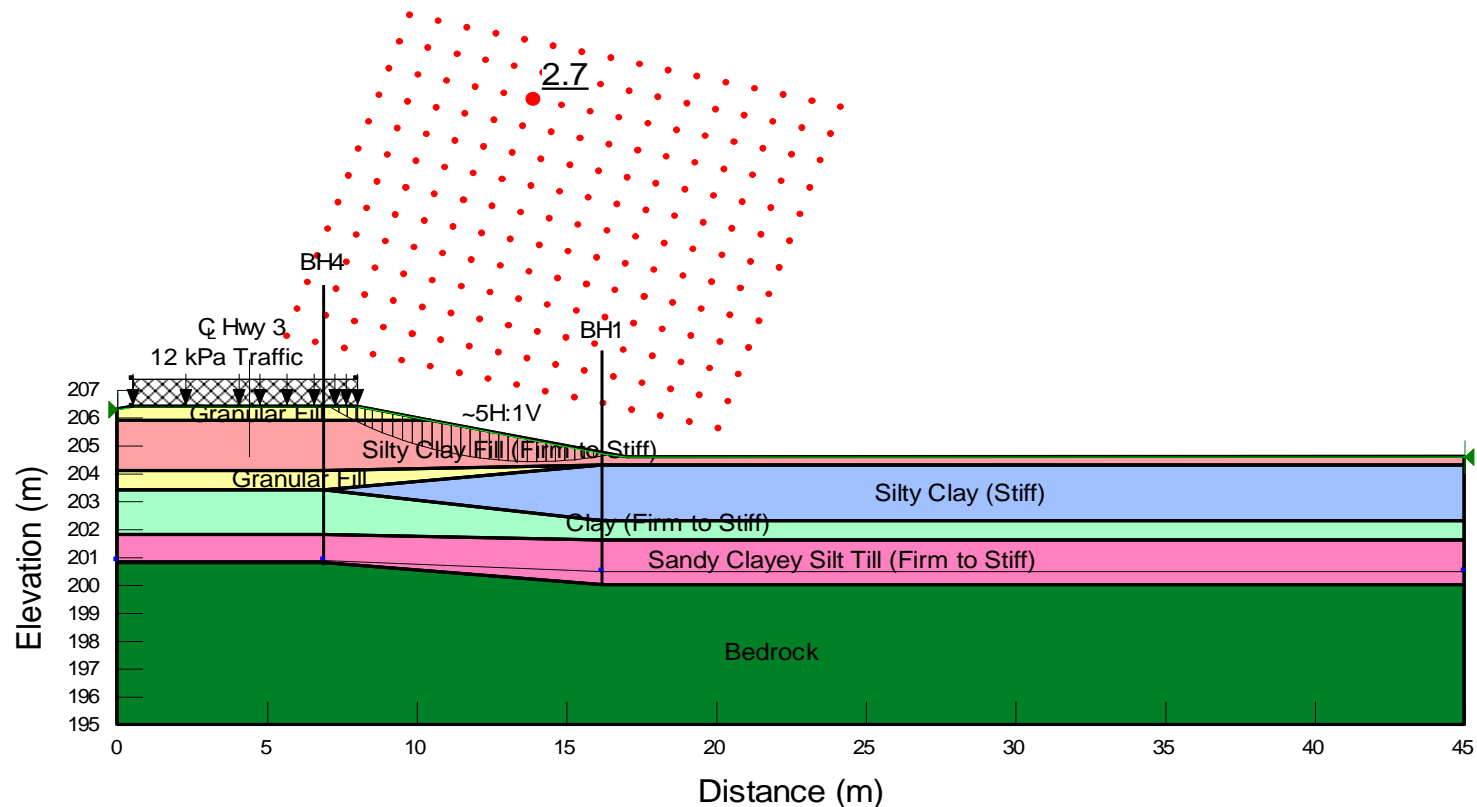


Figure 2: Slope stability analysis for embankment slope – drained static conditions

January 23, 2017

Culvert on Hwy 3 Stability of Embankment Slope Undrained Static Condition

Name: Engineered Fill Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion': 0 kPa Phi': 32 °
Name: Silty Clay Fill (Firm to Stiff) Model: Undrained (Phi=0) Unit Weight: 19 kN/m³ Cohesion': 65 kPa
Name: Silty Clay (Stiff) Model: Undrained (Phi=0) Unit Weight: 19 kN/m³ Cohesion': 85 kPa
Name: Clay (Firm to Stiff) Model: Undrained (Phi=0) Unit Weight: 19 kN/m³ Cohesion': 60 kPa
Name: Sandy Clayey Silt Till (Firm to Stiff) Model: Undrained (Phi=0) Unit Weight: 18 kN/m³ Cohesion': 50 kPa
Name: Bedrock Model: Bedrock (Impenetrable)

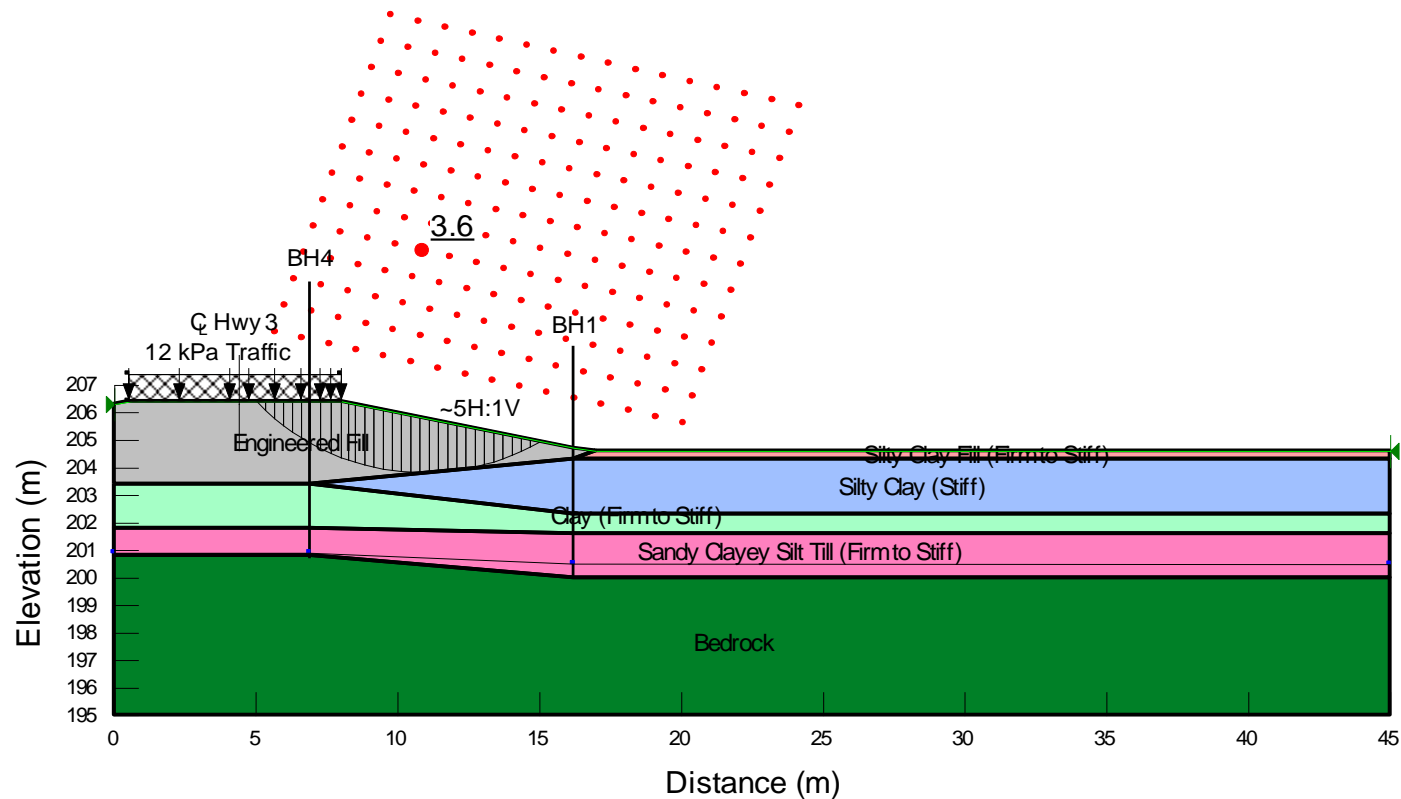


Figure 3: Slope stability analysis for embankment slope after culvert replacement – undrained static conditions

Culvert on Hwy 3 Stability of Embankment Slope Drained Static Condition

Name: Engineered Fill Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion': 0 kPa Phi': 32 °
Name: Silty Clay Fill (Firm to Stiff) Model: Mohr-Coulomb Unit Weight: 19 kN/m³ Cohesion': 0 kPa Phi': 29 °
Name: Silty Clay (Stiff) Model: Mohr-Coulomb Unit Weight: 19 kN/m³ Cohesion': 0 kPa Phi': 29 °
Name: Clay (Firm to Stiff) Model: Mohr-Coulomb Unit Weight: 19 kN/m³ Cohesion': 0 kPa Phi': 27 °
Name: Sandy Clayey Silt Till (Firm to Stiff) Model: Mohr-Coulomb Unit Weight: 18 kN/m³ Cohesion': 0 kPa Phi': 29 °
Name: Bedrock Model: Bedrock (Impenetrable)

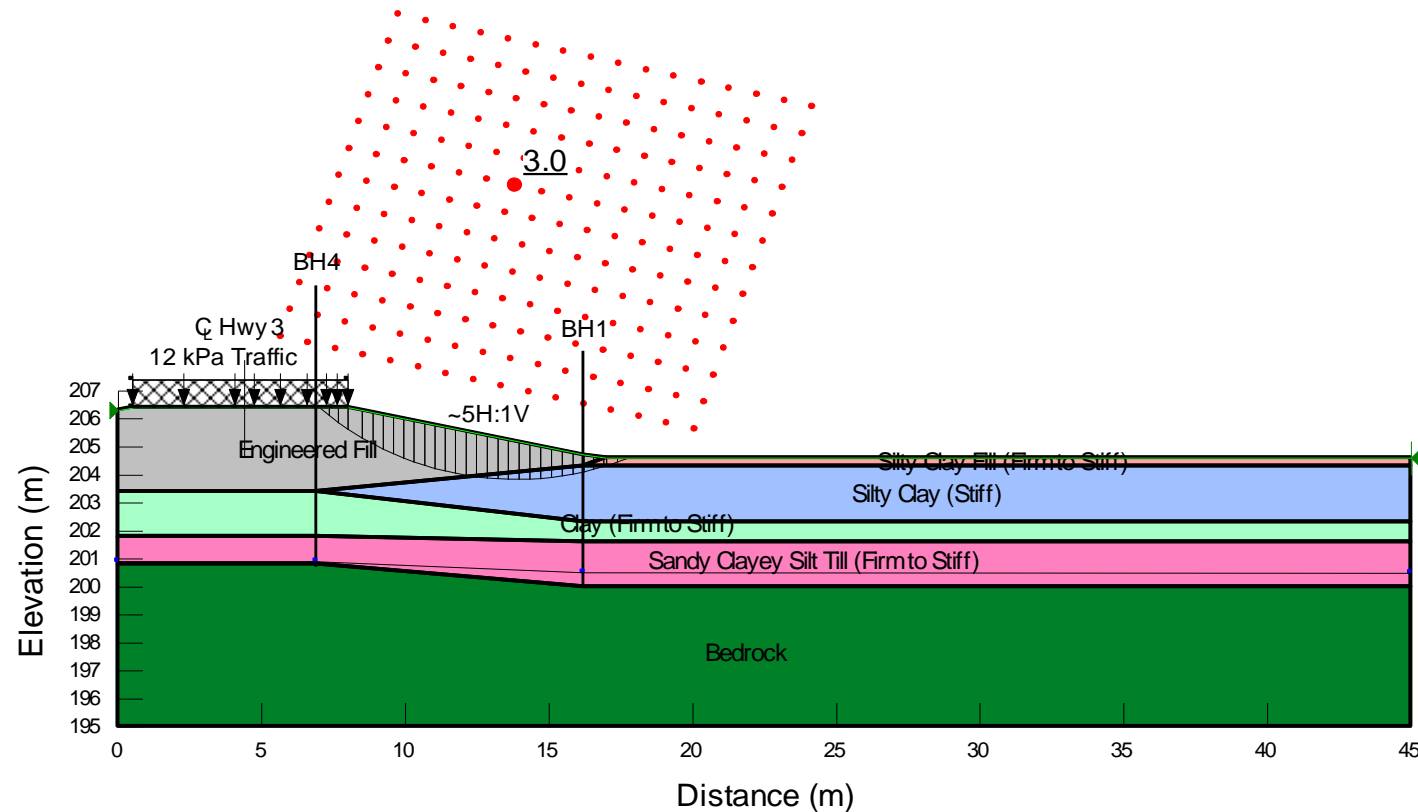


Figure 4: Slope stability analysis for embankment slope after culvert replacement – drained static conditions

Culvert on Hwy 3 Stability of Embankment Slope Undrained Static Condition

Name: Engineered Fill Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion': 0 kPa Phi': 32 °
Name: Silty Clay Fill (Firm to Stiff) Model: Undrained (Phi=0) Unit Weight: 19 kN/m³ Cohesion': 65 kPa
Name: Silty Clay (Stiff) Model: Undrained (Phi=0) Unit Weight: 19 kN/m³ Cohesion': 85 kPa
Name: Clay (Firm to Stiff) Model: Undrained (Phi=0) Unit Weight: 19 kN/m³ Cohesion': 60 kPa
Name: Sandy Clayey Silt Till (Firm to Stiff) Model: Undrained (Phi=0) Unit Weight: 18 kN/m³ Cohesion': 50 kPa
Name: Bedrock Model: Bedrock (Impenetrable)

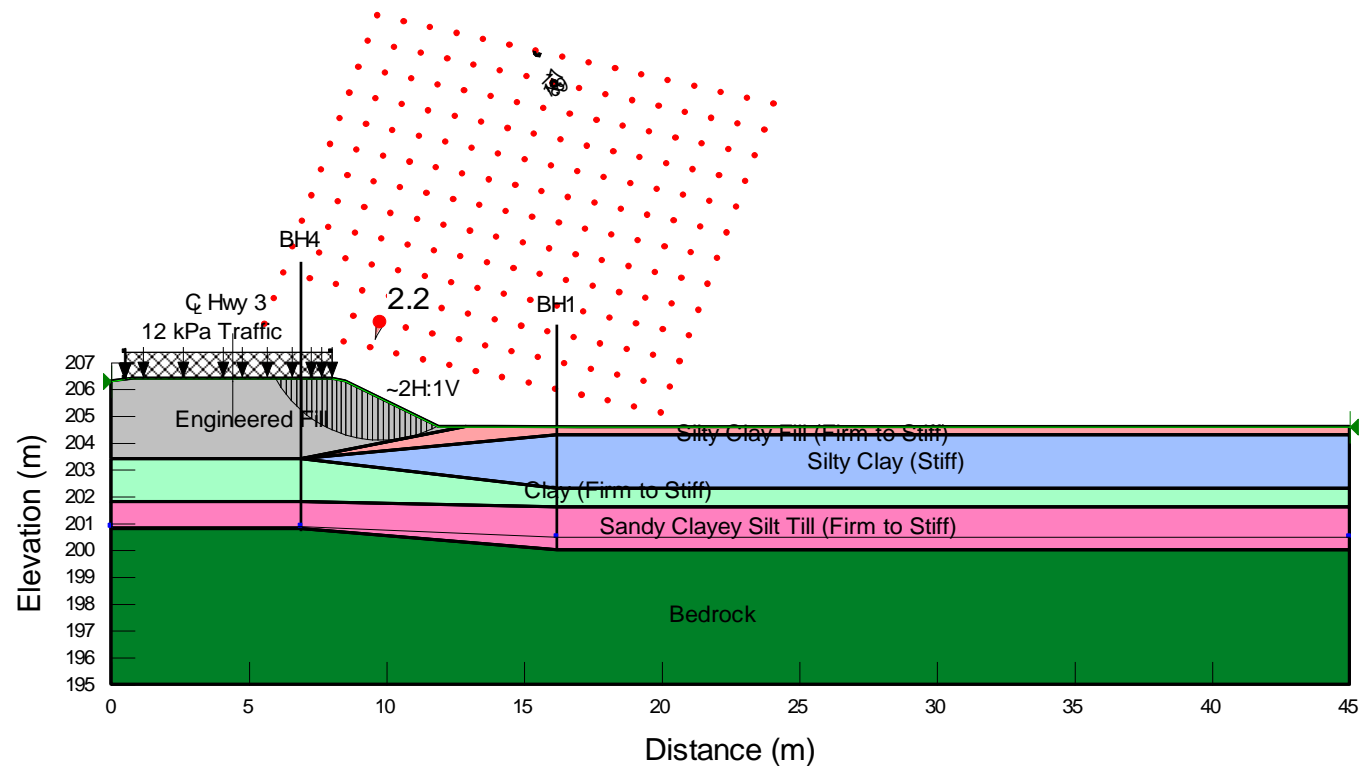


Figure 5: Slope stability analysis for embankment slope after culvert replacement with 2H:1V slope – undrained static conditions

Culvert on Hwy 3 Stability of Embankment Slope Drained Static Condition

Name: Engineered Fill Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion': 0 kPa Phi': 32 °
Name: Silty Clay Fill (Firm to Stiff) Model: Mohr-Coulomb Unit Weight: 19 kN/m³ Cohesion': 0 kPa Phi': 29 °
Name: Silty Clay (Stiff) Model: Mohr-Coulomb Unit Weight: 19 kN/m³ Cohesion': 0 kPa Phi': 29 °
Name: Clay (Firm to Stiff) Model: Mohr-Coulomb Unit Weight: 19 kN/m³ Cohesion': 0 kPa Phi': 27 °
Name: Sandy Clayey Silt Till (Firm to Stiff) Model: Mohr-Coulomb Unit Weight: 18 kN/m³ Cohesion': 0 kPa Phi': 25 °
Name: Bedrock Model: Bedrock (Impenetrable)

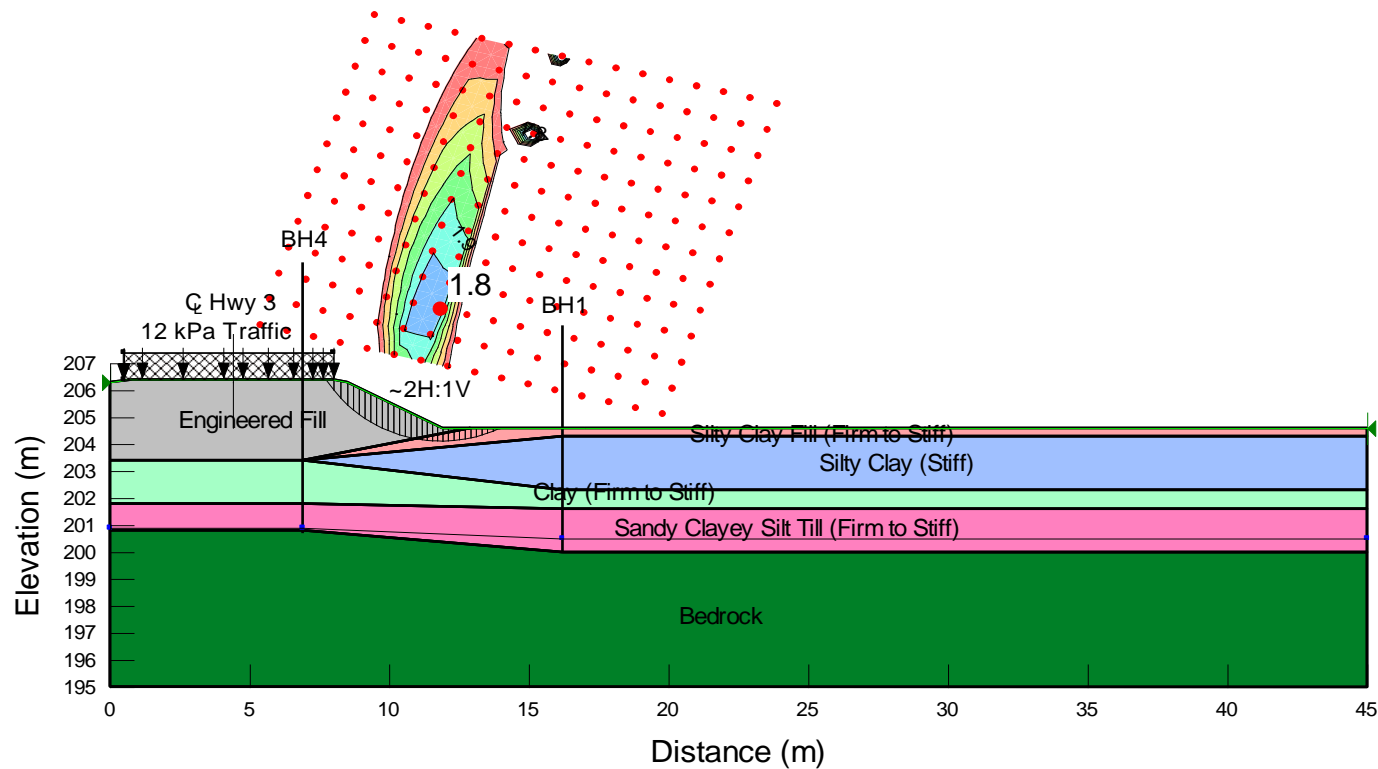
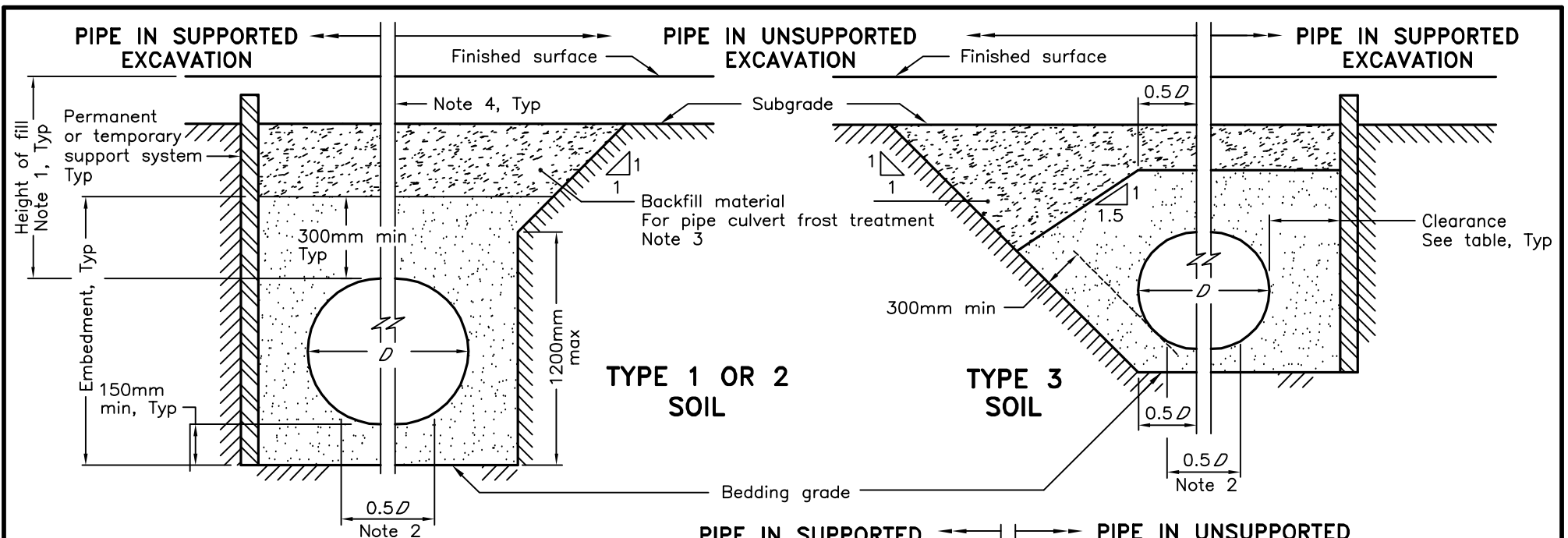


Figure 6: Slope stability analysis for embankment slope after culvert replacement with 2H:1V slope – drained static conditions

Appendix G – Ontario Provincial Standard Drawings

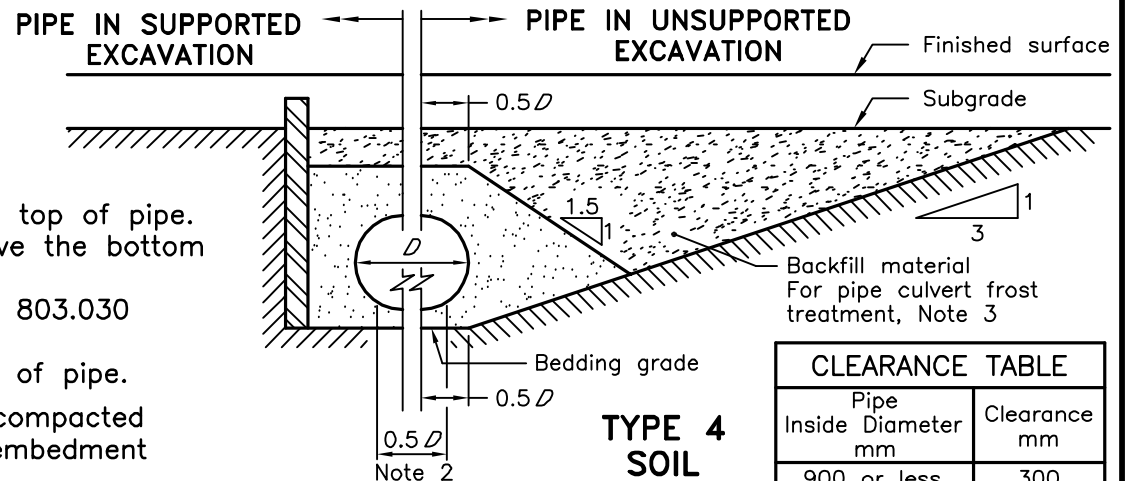


LEGEND:

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

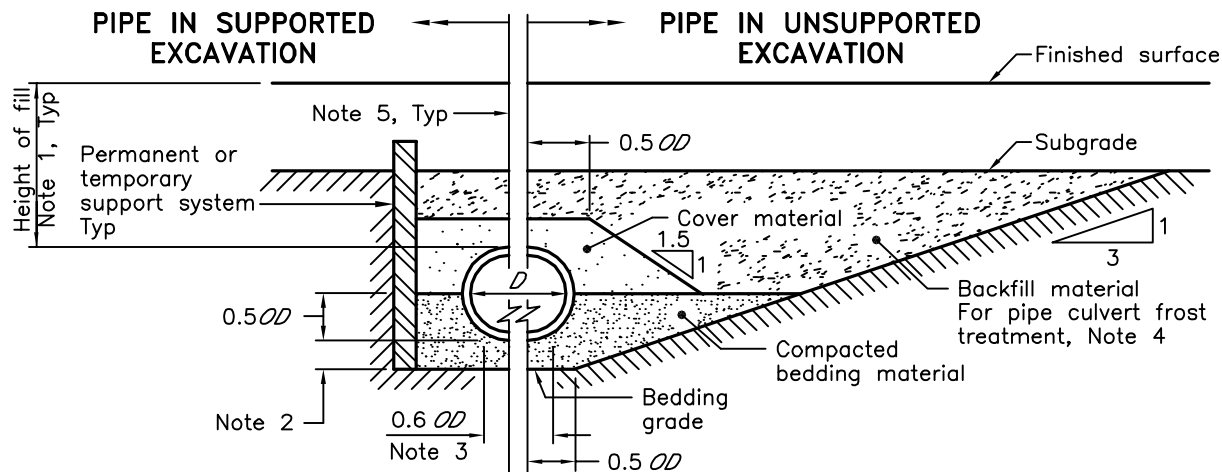
Rev 2

FLEXIBLE PIPE
EMBEDMENT AND BACKFILL
EARTH EXCAVATION

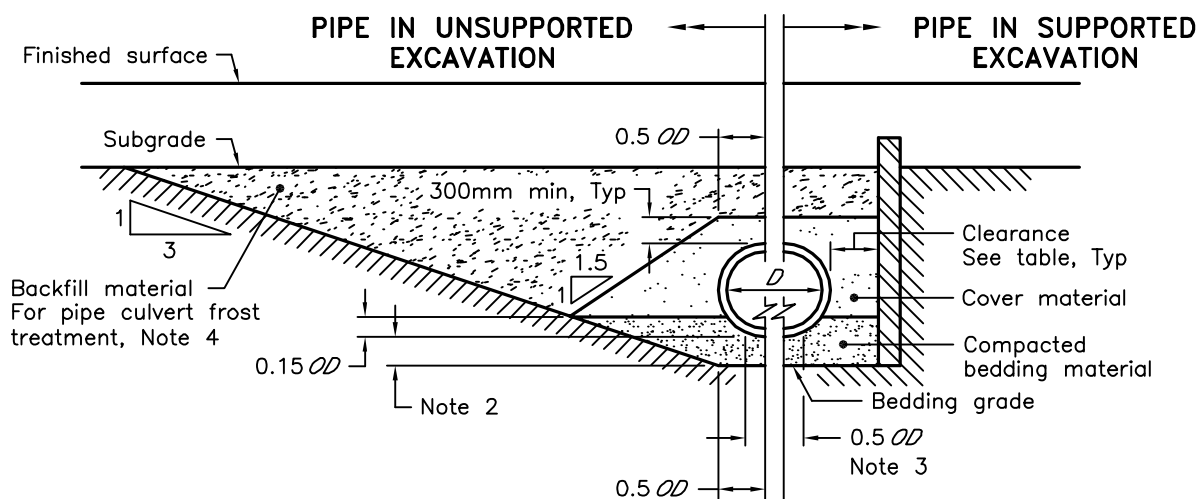
OPSD 802.010







CLASS B BEDDING



CLASS C BEDDING

LEGEND:

D – Inside diameter
 OD – Outside diameter

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

NOTES:

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

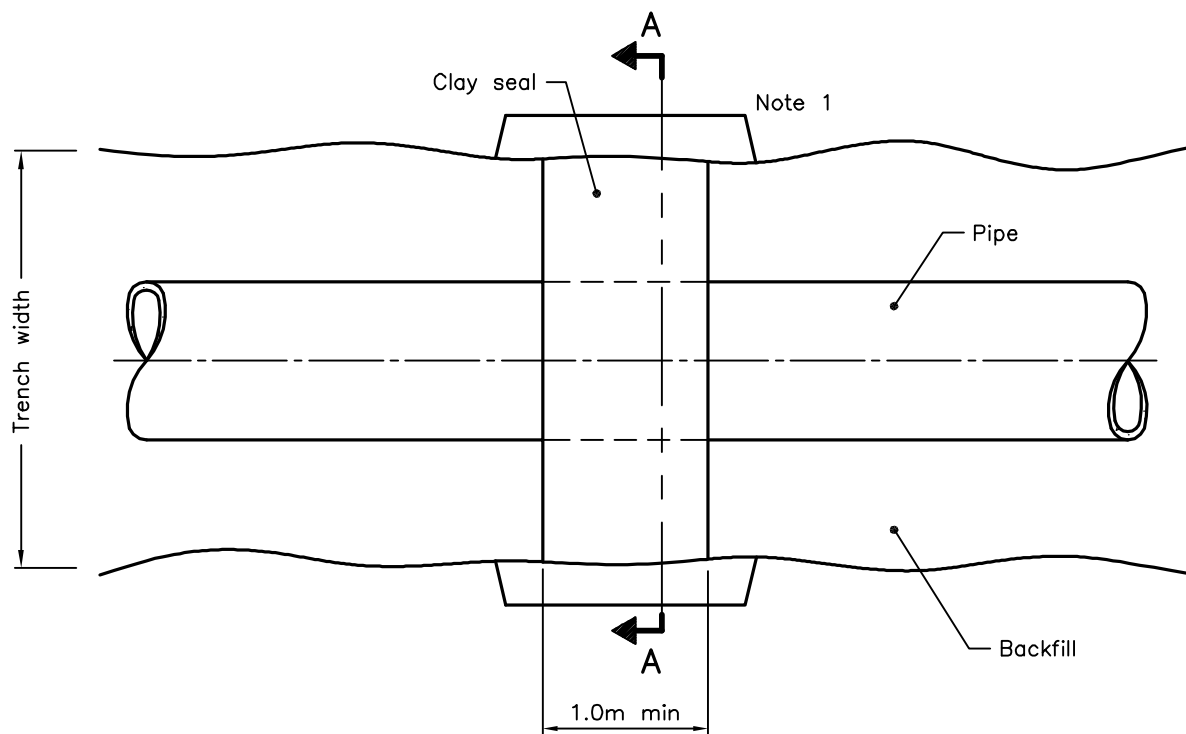
ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2010 Rev 2

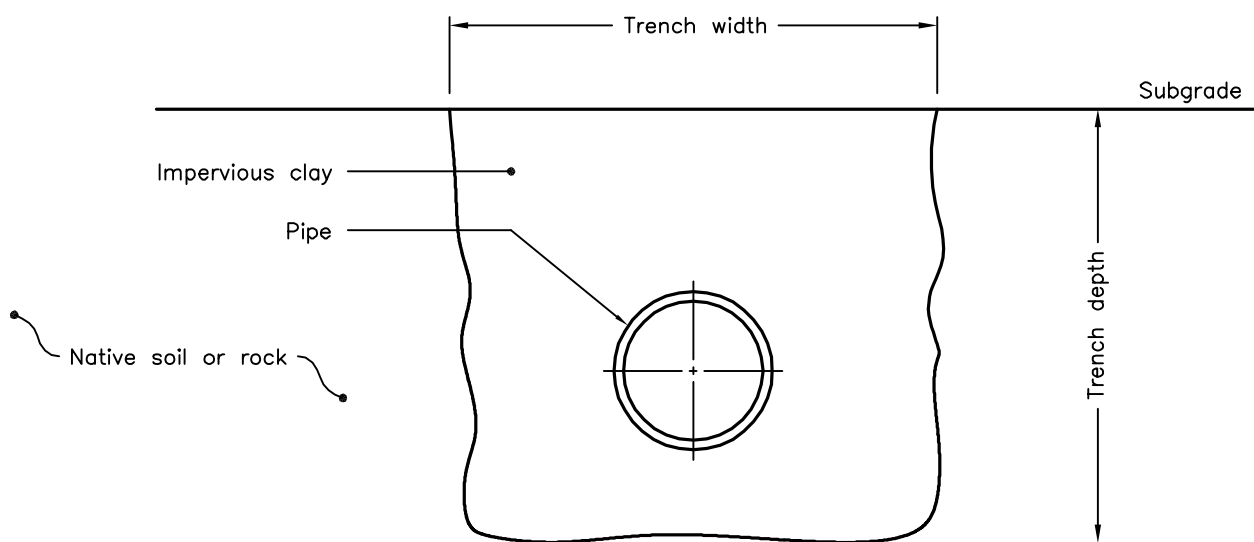
RIGID PIPE BEDDING,
 COVER, AND BACKFILL
 TYPE 4 SOIL – EARTH EXCAVATION

OPSD 802.032





PLAN



SECTION A-A

NOTES:

1. Key into undisturbed trench soil.

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

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

C All dimensions are in metres unless otherwise shown.

ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2011

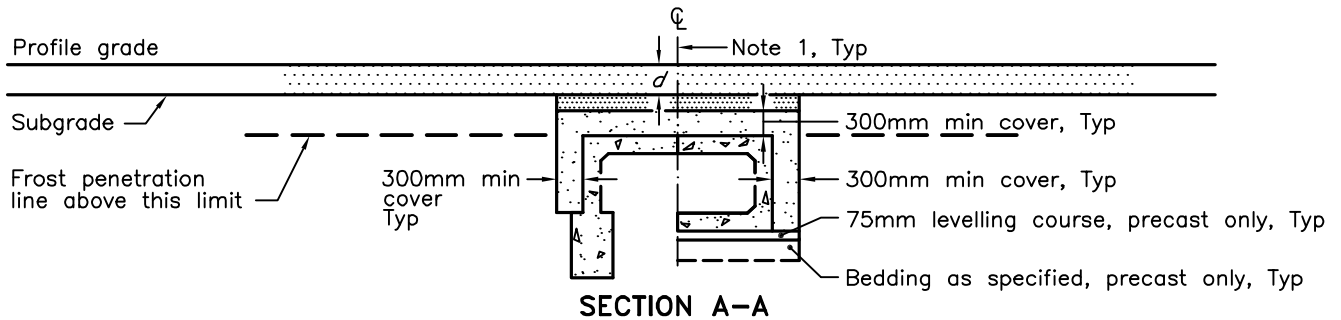
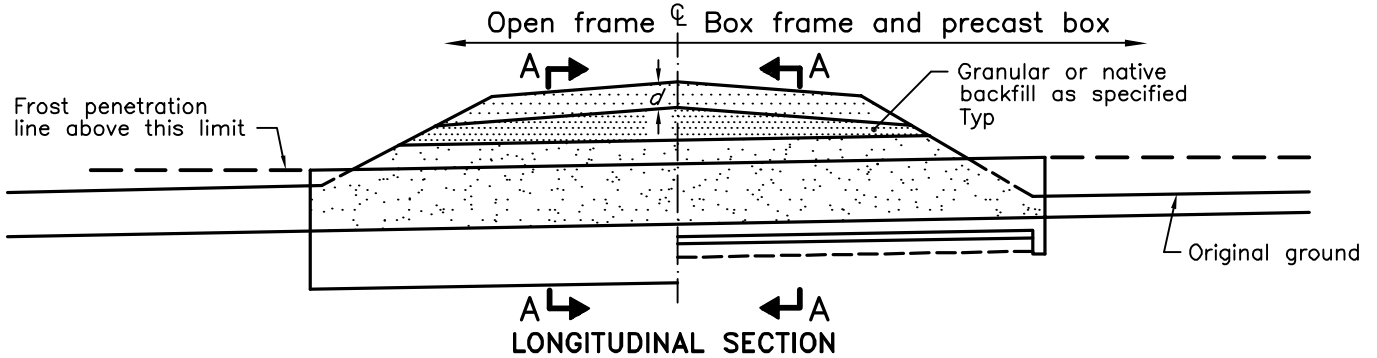
Rev 1

CLAY SEAL FOR PIPE TRENCHES

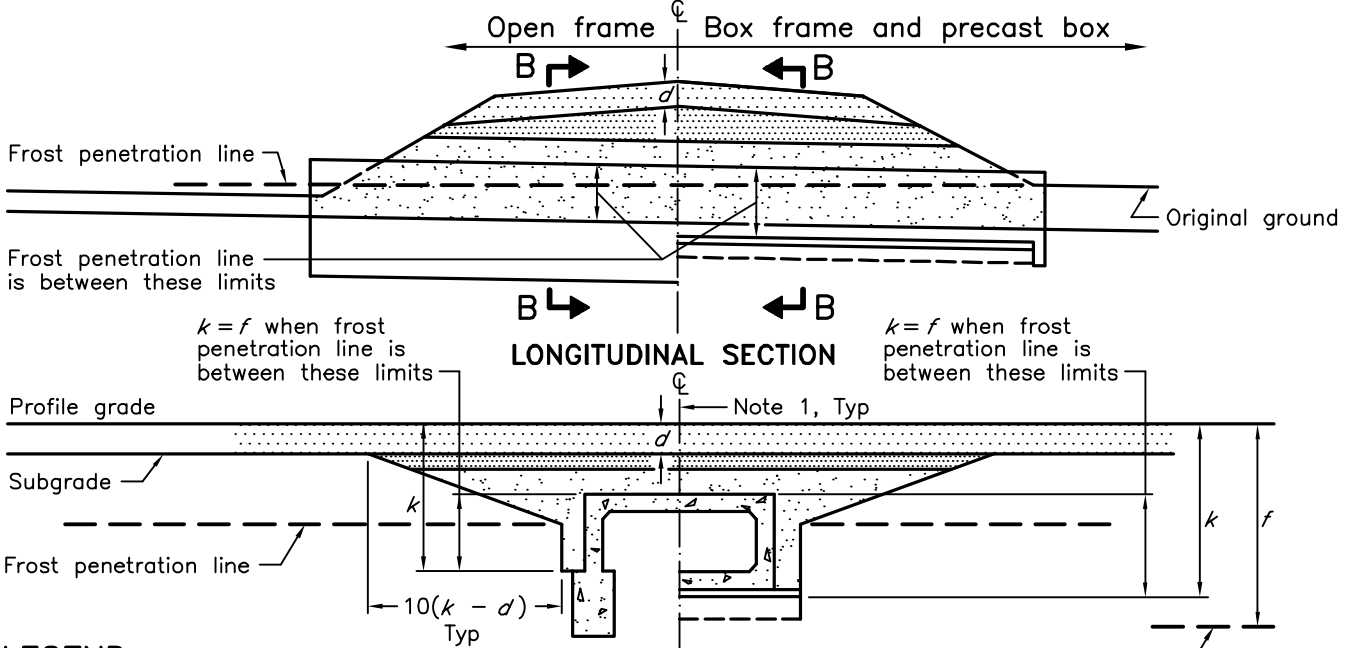
OPSD 802.095



FROST PENETRATION LINE AT OR ABOVE TOP OF CULVERT



FROST PENETRATION LINE BELOW TOP OF CULVERT



LEGEND:

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

NOTES:

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

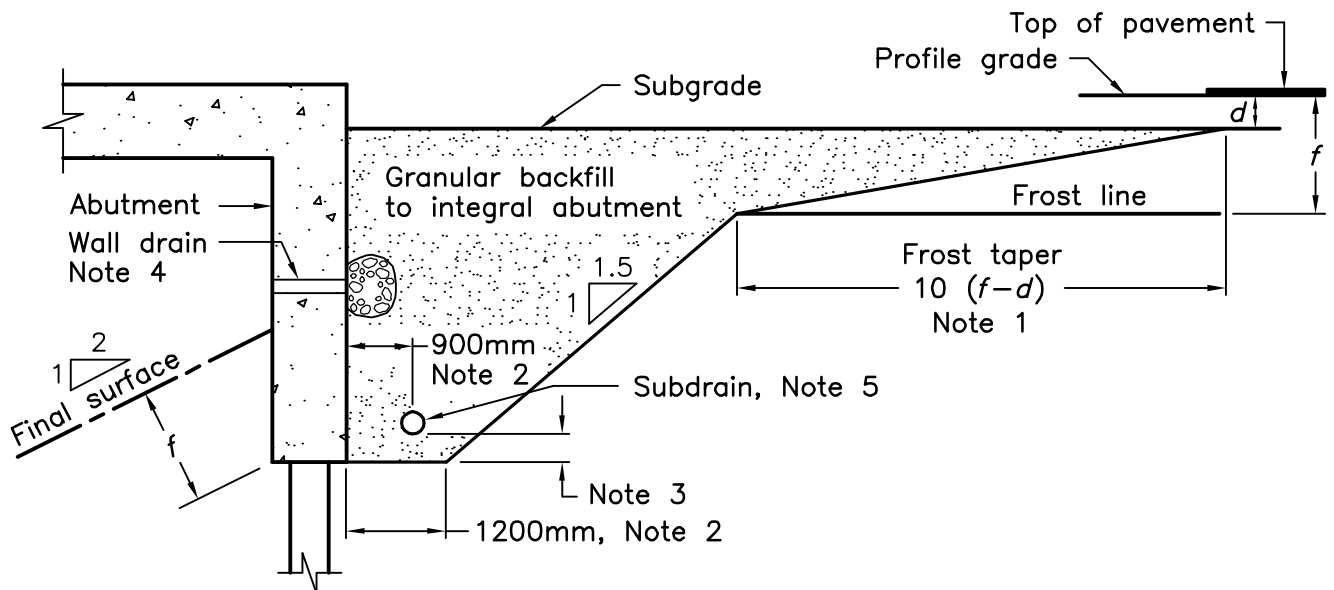
ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2010 Rev 2

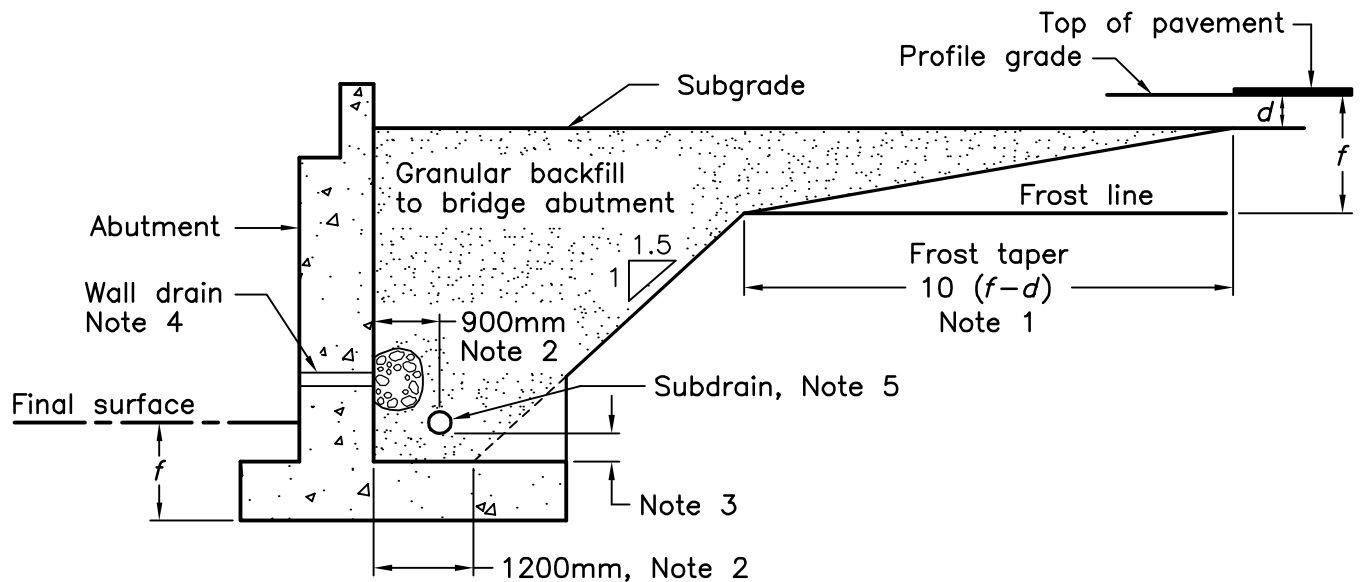
**BACKFILL AND COVER FOR
CONCRETE CULVERTS WITH SPANS
LESS THAN OR EQUAL TO 3.0M**

OPSD 803.010





INTEGRAL ABUTMENT



ABUTMENT

NOTES:

- 1 d = depth of combined base and subbase courses
 f = frost penetration depth as specified
- 2 Dimensions perpendicular to back face of abutment.
- 3 Height to be consistent with positive drainage of subdrain as specified.
- 4 Where specified, wall drains shall be installed according to OPSD 3190.100.
- 5 150mm dia perforated pipe subdrain wrapped with geotextile.
- A Lateral limits of granular backfill to bridge abutment to be inside face to inside face of retaining wall or wingwall. Frost taper shall extend the full width of the backfill unless interrupted by the retaining wall or wingwall.
- B Sections shown are parallel to centreline of roadway.
- C Subdrain shall be installed with a 2% gradient behind wall.
- D All dimensions are in millimetres unless otherwise shown.

ONTARIO PROVINCIAL STANDARD DRAWING

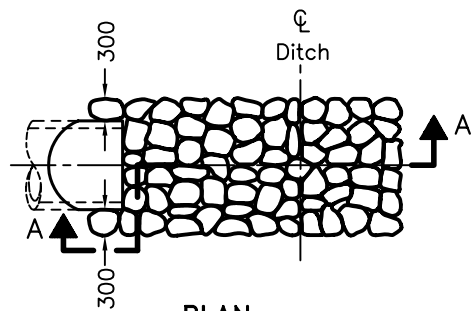
Nov 2010

Rev 1

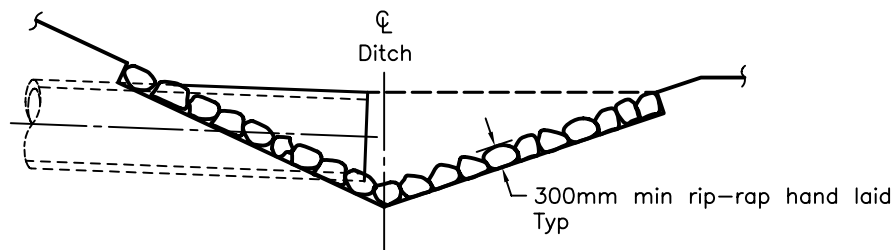


WALLS
ABUTMENT, BACKFILL
MINIMUM GRANULAR REQUIREMENT

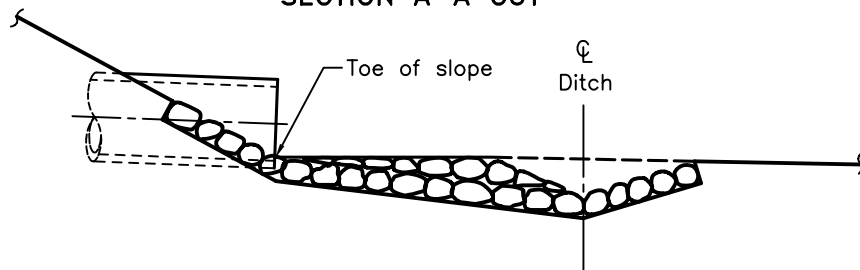
OPSD 3101.150



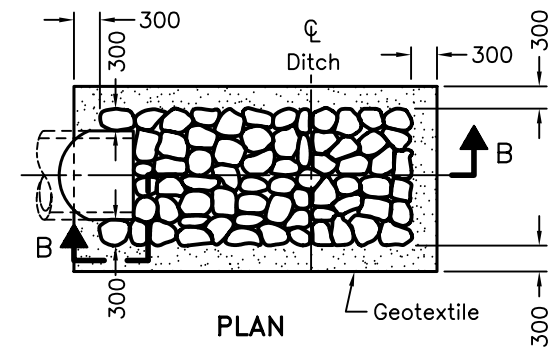
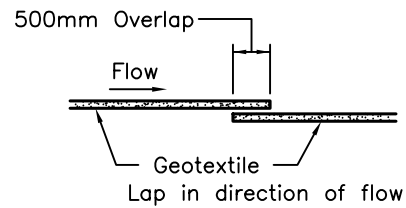
PLAN
CUT OR FILL



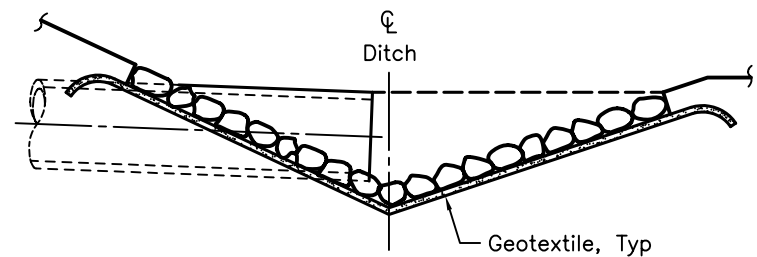
SECTION A-A CUT



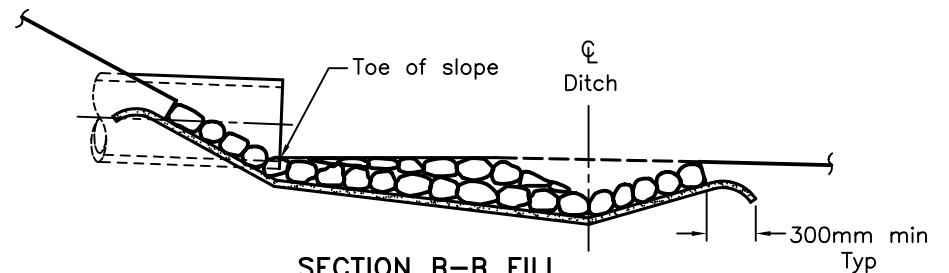
SECTION A-A FILL
TYPE A – WITHOUT GEOTEXTILE



PLAN
CUT OR FILL



SECTION B-B CUT



SECTION B-B FILL
TYPE B – WITH GEOTEXTILE

NOTES:

A All dimensions are in millimetres unless otherwise shown.

ONTARIO PROVINCIAL STANDARD DRAWING

RIP-RAP TREATMENT
FOR SEWER AND CULVERT OUTLETS

Nov 2001

Rev 0



OPSD – 810.010

- A drainage system behind a retaining structure should ensure that a groundwater table does not exist above the footing level. Preferably, the ground water level is controlled by the use of free-draining granular backfill and a collection system such as weep holes or perforated drains at the footing level. These weep holes and drains should be inspected and maintained to ensure that they do not become blocked. If free-draining, granular backfill is not employed, the permeability of the backfill and the hydrostatic head will control the extent to which the groundwater table can be depressed locally by seepage towards a footing drain. In practice, design for frost protection is best done using free-draining backfill.

The design should also consider the risk of unusually large inflows of water creating a temporary hydrostatic head of water behind the wall. An example is the overtopping of a retaining wall, adjoining a large body of water, by storm waves. Measures such as the use of quarried rock backfill, design for full hydrostatic pressure, or provision of a sloped impermeable surface layer should be considered.

Measurements have shown that earth pressures can vary seasonally, but the effects have normally been neglected in design, except for winter frost pressures. These latter can be very large if the backfill is frost susceptible and for this reason free-draining granular backfill is recommended.
- Figure C6.20 shows examples of minimum backfill requirements.

The distance, x , should be equal to or greater than the estimated vertical frost penetration. This distance may be reduced if the wall abuts a vertical face of bedrock that is not susceptible to frost. The frost penetration may be reduced by the use of suitable insulation, in which case a thermal analysis should be performed by a Geotechnical Engineer.

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

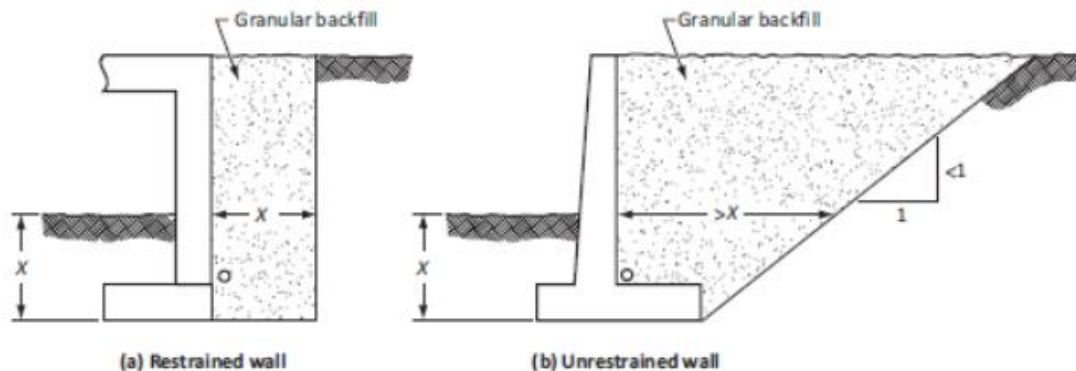


Figure C6.20
Backfill for frost protection
 (See Clause C6.12.1.)

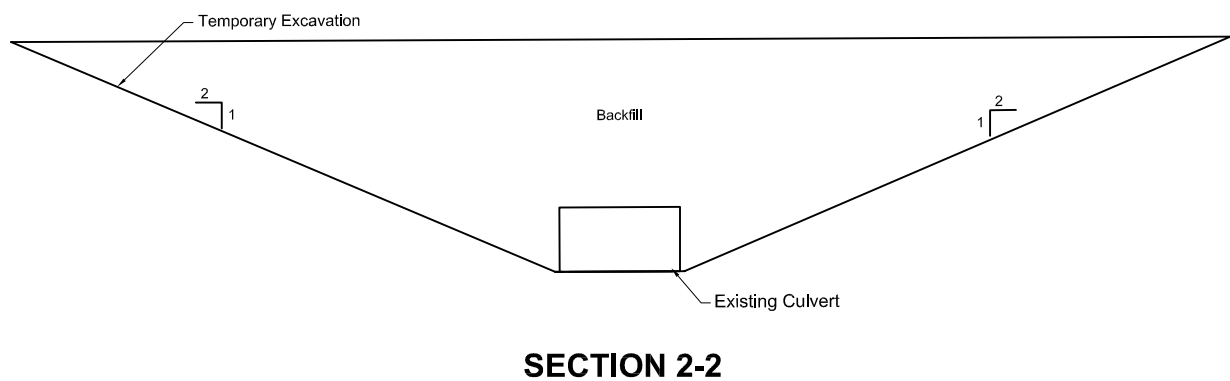
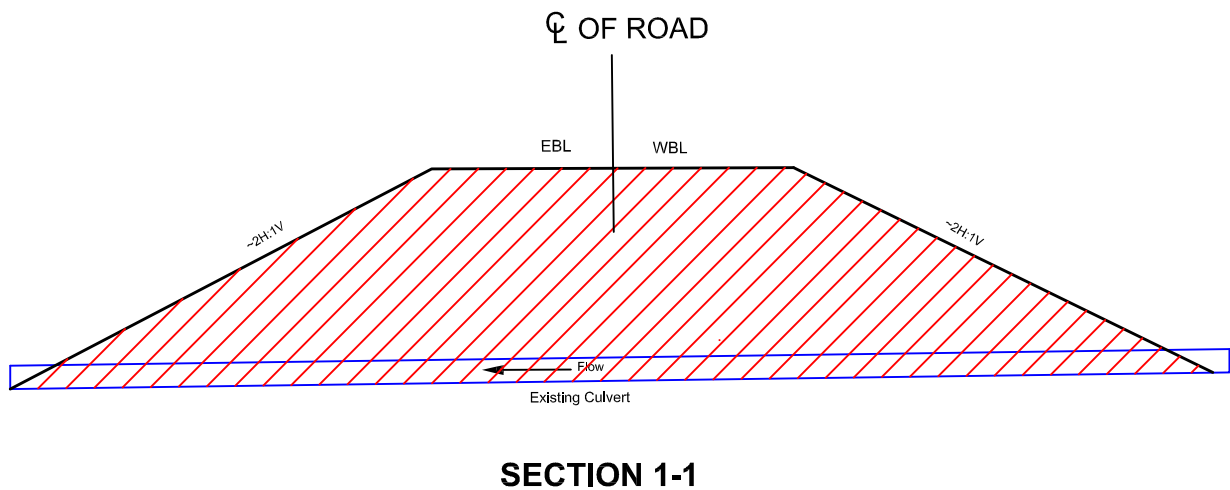
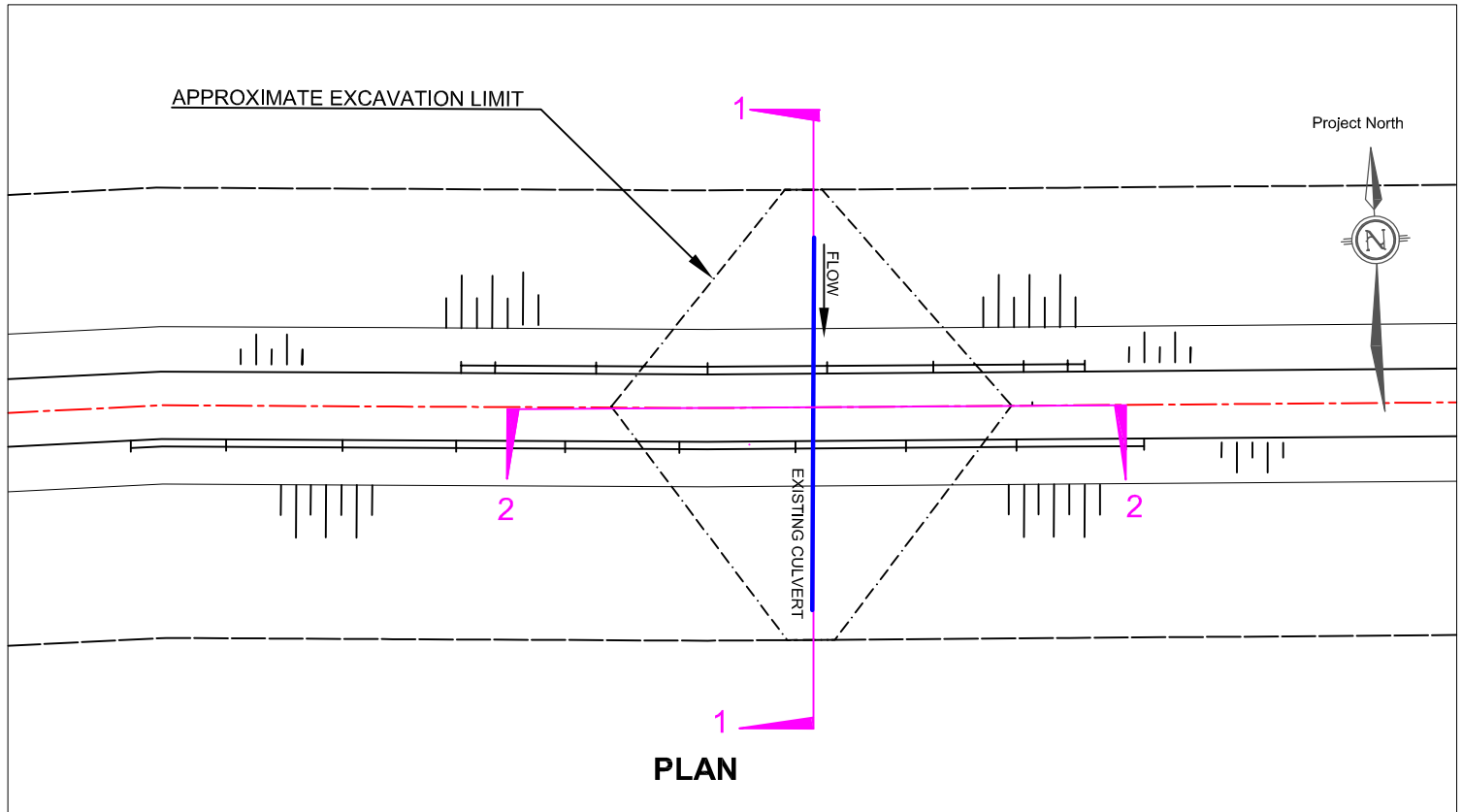
C6.12.2 Lateral ground pressures

C6.12.2.1 General

Earth pressure acting on a structure depends on the relative movement of the structure, the backfill, the type of soil adjacent to the backfill, and the soil below the footing or supporting piles. Appropriate geotechnical parameters should be chosen for the calculation of lateral pressures based on recognized geotechnical theories as specified in Clause 6.12.2.2 for the backfill behind the wall. Geotechnical parameters frequently used in allowable stress design methods are applicable in limit states design pressure calculation. Where the possibility exists, hydrostatic pressure needs to be considered, e.g., in situations where walls are partially submerged or where non-free-draining backfill is used.

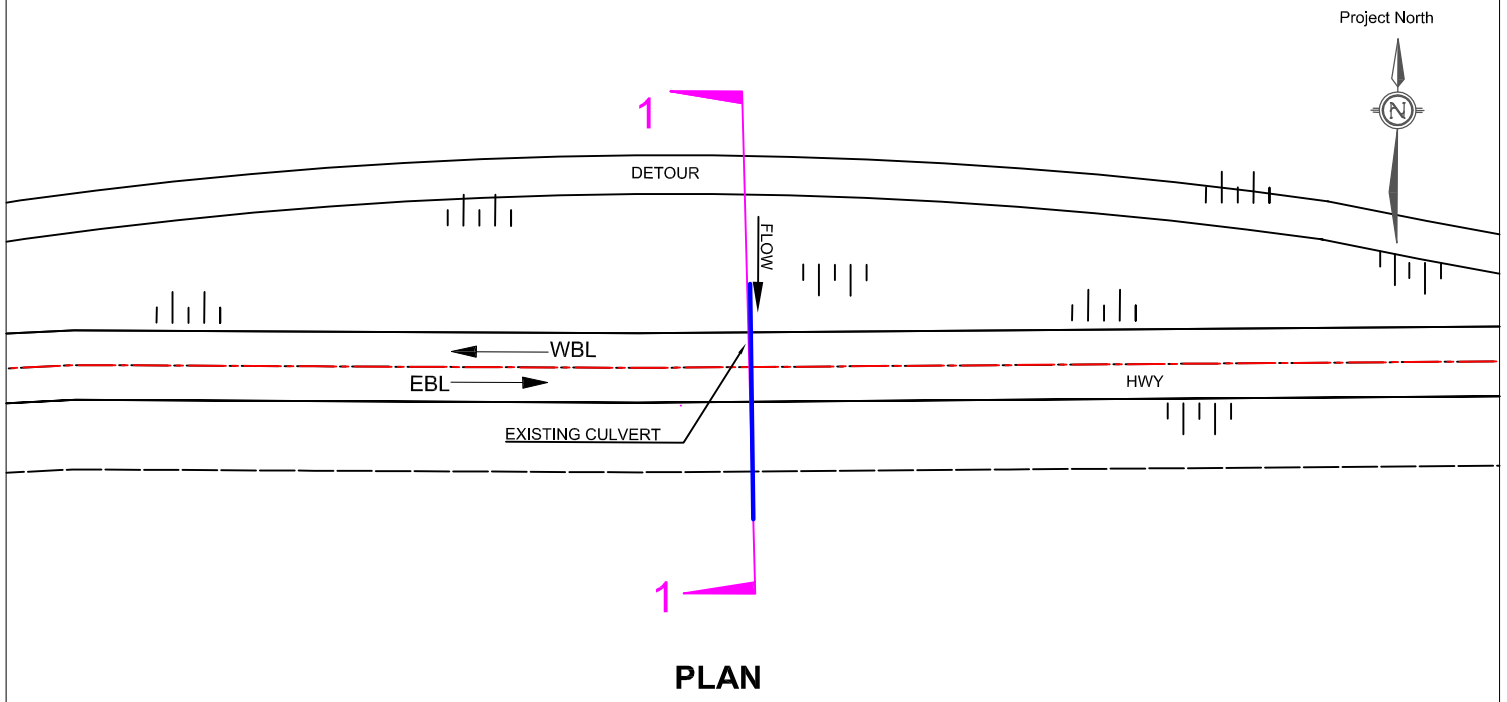
Appendix H – Schematic Sketches for Construction Alternatives

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



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

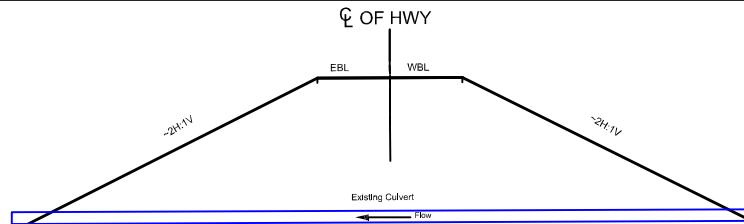
SCHEMATIC DIAGRAMS (NTS)



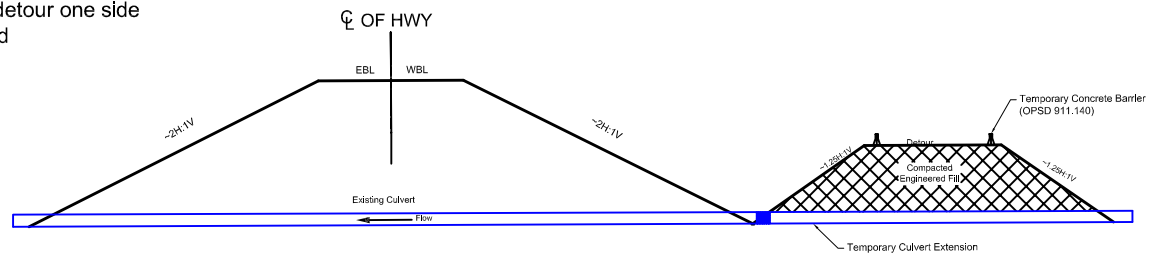
PLAN

RECOMMENDED STAGES

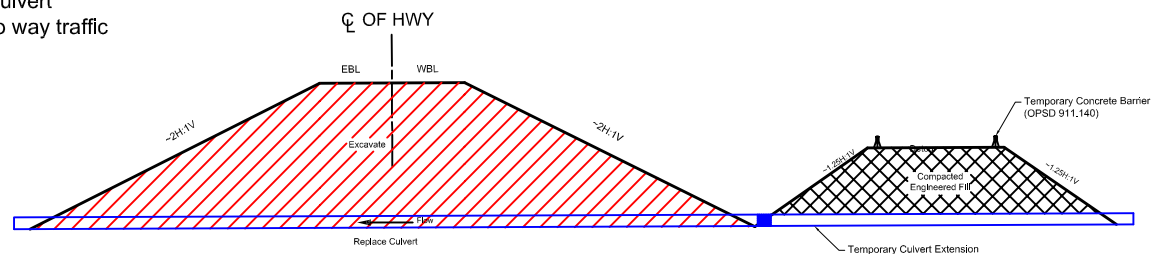
1.0 Stage 1 - Current condition



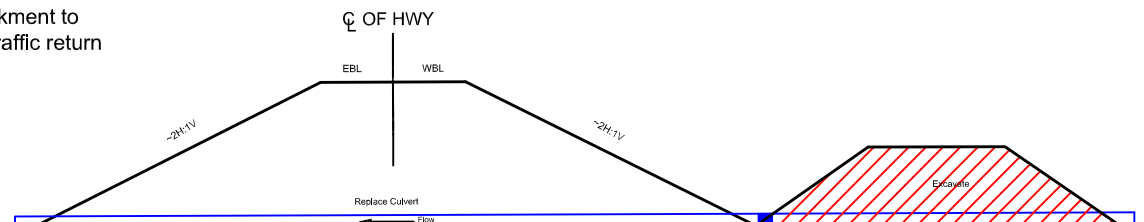
2.0 Stage 2 - Build temporary detour one side
Two-way traffic on existing road



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



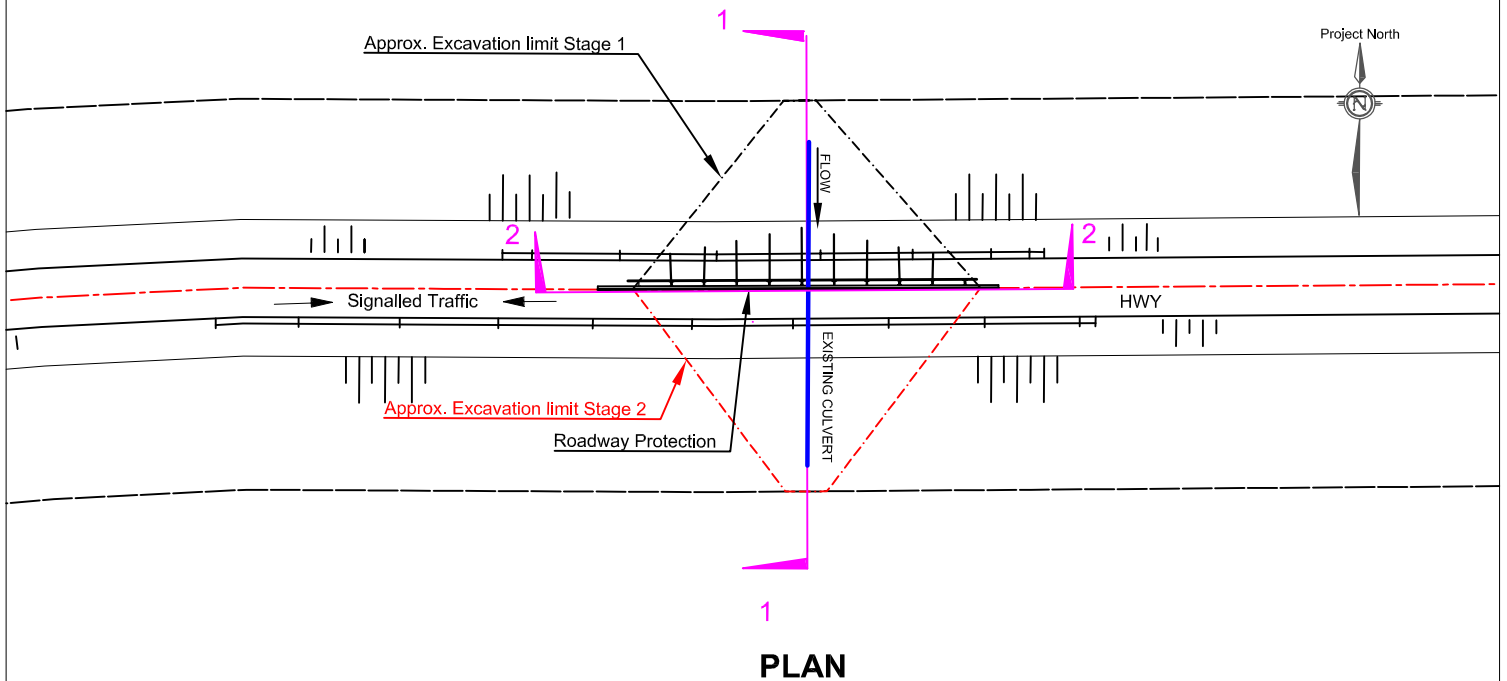
4.0 Stage 4 - Build the embankment to existing alignment; Two-way traffic return



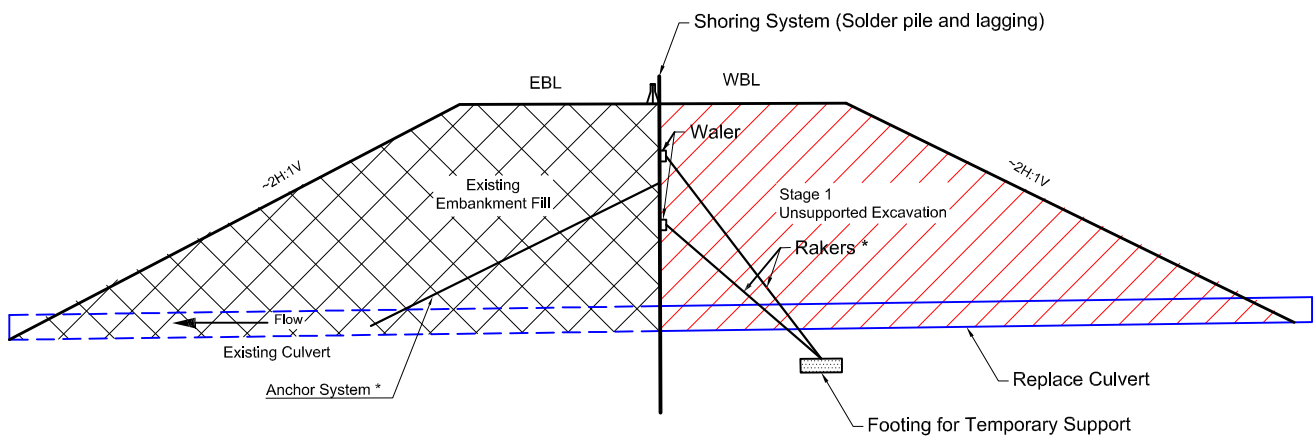
SECTION 1-1

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

SCHEMATIC DIAGRAMS (NTS)

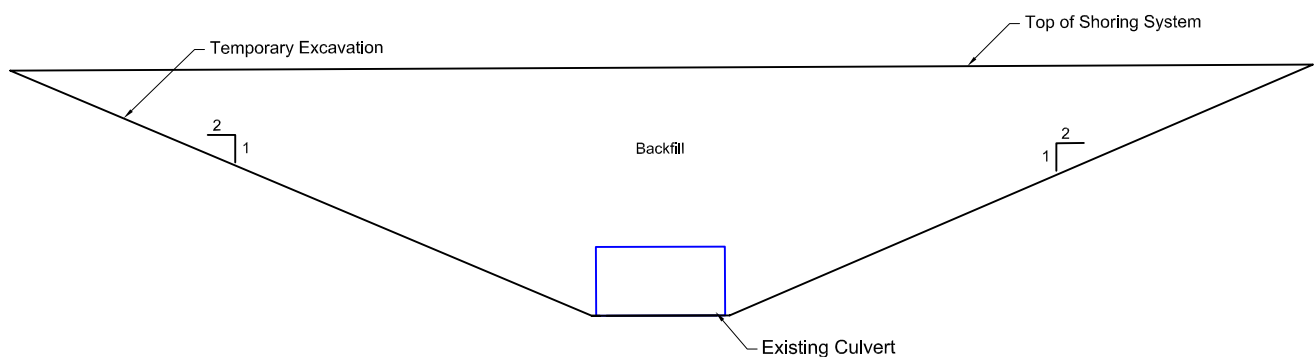


Half and Half Construction, Shoring system with either rakers or anchor system - Unsupported Excavation



* Rakers or Anchor System

SECTION 1-1



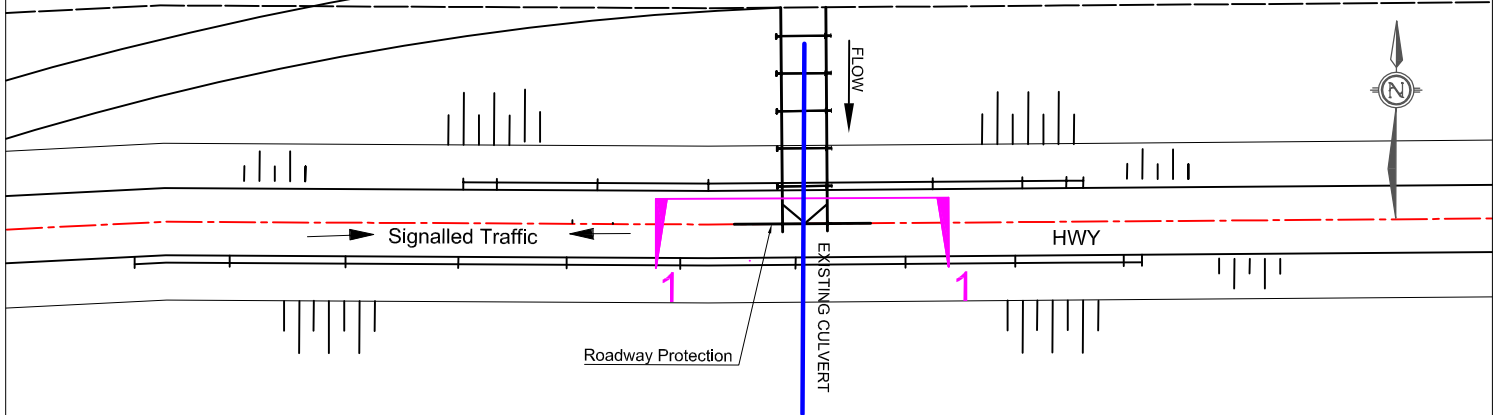
SECTION 2-2

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

SCHEMATIC DIAGRAMS (NTS)

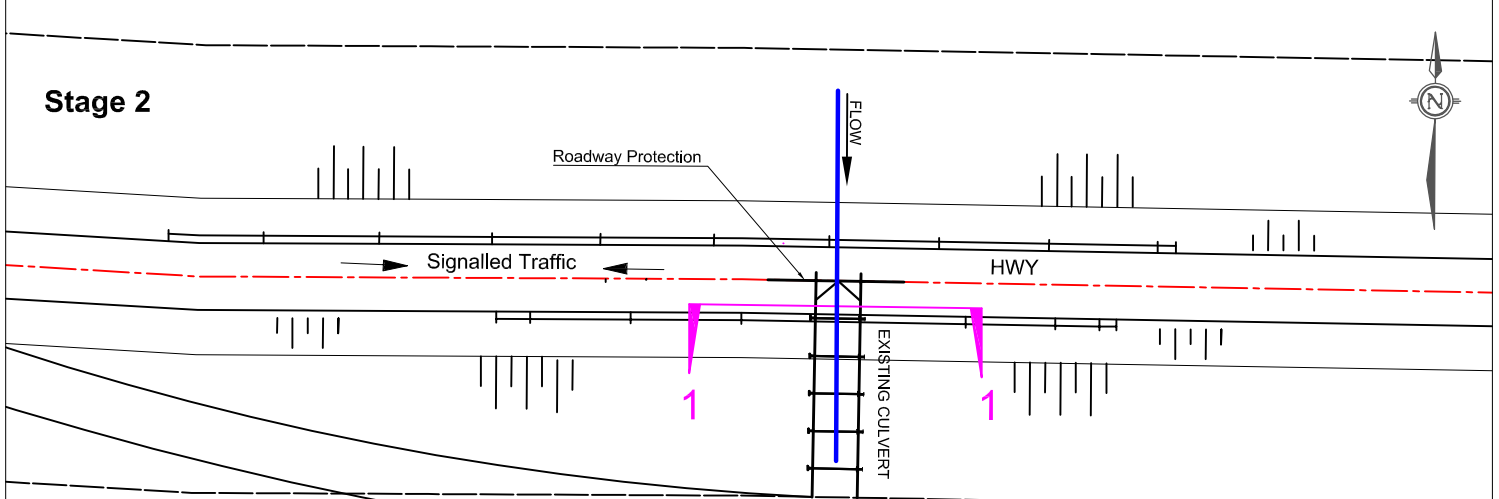
Stage 1

Project North



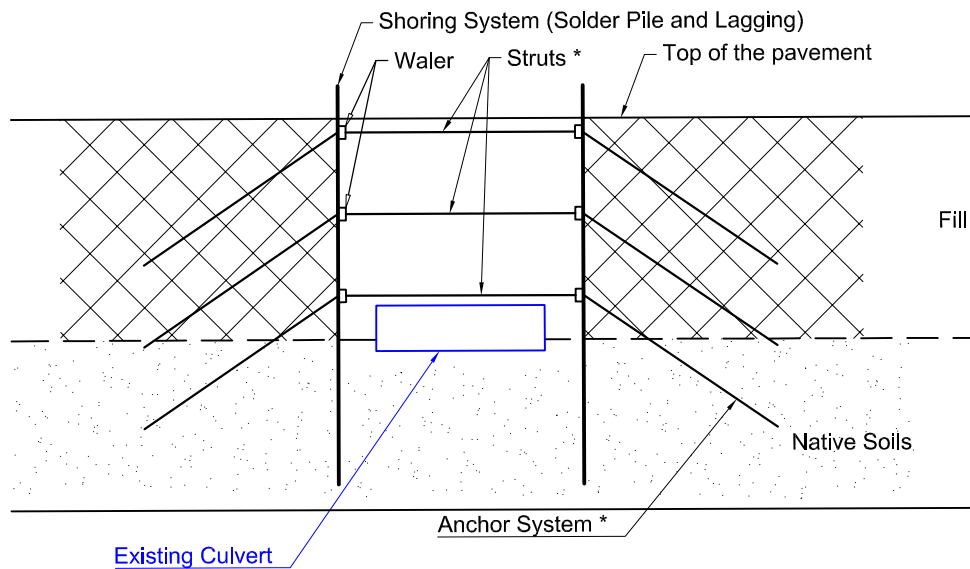
Stage 2

Project North



PLAN

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



* Struts or Anchor System

SECTION 1-1