



DRAFT REPORT

FOUNDATION INVESTIGATION AND DESIGN REPORT
Deep Fill Culvert Replacement, Hwy 11, Township of Bowman/Carr

Agreement No. 5013-E-0008
Assignment No. 6
WO 2014-11039

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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. for the replacement of Deep Fill Culvert on Hwy 11, located approximately 325 m east of Watabeag River in the Township of Bowman/Carr, the Ministry of Transportation (MTO) Northeastern Region. The work was undertaken under Agreement # 5013-E-0008, Assignment No. 6 (WO 2014-11039). The terms of reference (TOR) were as presented in the MTO letter dated November 3, 2014.

The purpose of the investigation is to evaluate the subsurface condition along the proposed new alignment, to permit detailed design for the culvert replacement and to examine the suitability of trenchless and/or traditional open cut methods of culvert replacement. The site specific geotechnical investigation consisted of borings, soil sampling, borehole logging, and field and laboratory testing.

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

1.2 Site Description and Geological Setting

1.2.1 Site Description

The Deep Fill Culvert replacement site is located on Hwy 11 (Station 10+910) in Township of Bowman/Carr, approximately 325 m east of Watabeag River. The location of the culvert and a cross section of the existing culvert alignment are shown on Drawing 1 in Appendix B.

As noted in the drawing provided with TOR the existing corrugated steel pipe (CSP) culvert is 50 m long with 950 mm diameter. At this site Hwy 11 is two lanes, east/west roadway having approximately 2.4 m (north side) to 3.3 m (south side) wide granular shoulders and guardrails subsequently on both sides. It is estimated that the highway embankment at the investigated location is between 7.3 m (south) to 8.0 m (north) high having side slopes of approximately 2H: 1V from the top of the embankment to the toe of the embankment. Photographs of the site and inlet and outlet of the existing culvert are presented in Appendix A.

The surrounding terrain of culvert location is relatively flat, agricultural land, with depression towards the culvert stream. At the site location, water flows from south to north crossing Hwy 11 via culvert. The inlet and outlet of culvert are surrounded by trees and shrubs. During the field work, the site area was covered by snow (approximately 0.4 m thick; see photographs in Appendix A).

The general site conditions were assessed during the site reconnaissance in December 9, 2014. However, since the embankment slopes and surrounding area were covered by snow our observations were limited. On the north side (i.e., outlet side) of the embankment, it was observed that the portion of

the slope was depressed. It is estimated that the depressed slope surface is situated mostly on the west side of the culvert alignment and approximately 3 m from the edge of the pavement. The size of the depressed slope surface was estimated to be approximately 7 m to 8 m wide and 1 m deep along the slope. After removing the snow layer within a small area it was found that the slope at that location is covered with rock fill which consists mostly of cobbles and gravel. Based on this limited observation, it could be speculated that the local surficial slope failure occurred in the past and remediated with a layer of rock fill. On the south slope of the embankment (i.e., inlet side), a fan-shaped sinkhole located approximately 5 m from the inlet of culvert was observed. The size of the hole on the surface of the slope was approximately 2.0 m in diameter and the maximum depth of the hole measured in December 2014 was approximately 1 m. Photographs of the failures are presented in Appendix A.

At the time of site investigation, a thin layer of ice was formed at top of water level inside the culvert, and the stream flows below the ice layer. The water depths of the inlet and outlet streams were about 0.3 m and 0.35 m, respectively. The culvert was partially blocked at the inlet side. However, the flow of water was not restricted by the blockage (see Photographs 9 and 11, in Appendix A).

During the drilling of BH-2 at the northeast side of the existing culvert, the unexpected auger refusal was encountered at depth of 7.9 m from the ground surface, approximately at Elevation 251.0 m. Assuming that a boulder was hit, BH-2 was abandoned, and another BH-2A was drilled within an approximately 1 m perimeter of BH-2. The auger refusal was encountered again at the same level as that in BH-2. Diamond core drilling was performed in BH-2A using a 1.5 m long NQ double tube wireline corebarrel, and a 0.152 m long concrete core was retrieved in the corebarrel (Photograph 14 in Appendix A. Suspecting the presence of an abandoned old concrete culvert, the drilling in BH-2A was terminated and a new borehole, BH-2B, was drilled at the northwest side of culvert alignment (see Drawing 1 in Appendix B).

1.2.2 Geological Setting

The Map 2543 (Bedrock Geology of Ontario, East-Central Sheet, 1991) of the Ministry of Northern Development and Mines, indicates that the bedrock formation of the project area is known to be in Paleoproterozoic Group, mainly of mafic and ultramafic intrusive rocks comprised of matachewan and hearst swarms diabase dikes. The Map 2555 (Quaternary Geology of Ontario, East-Central Sheet, 1991) of the Ministry of Northern Development and Mines, indicates that the surface conditions in the vicinity of site consist of glaciolacustrine deposits consisting of silt and clay, minor sand basin and quiet water deposits.

1.3 Investigation Procedures

1.3.1 Site Investigation and Field Testing

The field investigation was performed between December 9, 2014 and December 14, 2014. The field program consisted of drilling six (6) sampled boreholes (BH-1, BH-2, BH-2A, BH-2B, BH-3 and BH-4). The boreholes were strategically located along the existing culvert alignment to provide subsurface information for the design of the proposed new culvert. BH-2, BH-2A and BH-2B were advanced from

the embankment crest on the WBL shoulder, and BH-3 from the embankment crest on the EBL shoulder. BH-1 and BH-4 were advanced at toe of embankment on the outlet side and inlet side of culvert, respectively. BH-2 and BH-2A were advanced on the east side and BH-2B on the west side of the existing culvert alignment on the WBL shoulder. Initially, it was planned to drill only BH-2 and BH-3 from the crest of the embankment to a depth of 20 m. However, after drilling BH-3 at the EBL shoulder to the depth of 20 m and premature termination of BH-2 at the WBL shoulder at the depth of 7.62 m due to auger refusal, BH-2A and BH-2B were drilled to investigate ground condition in the vicinity of BH-2. The borehole locations are shown on Drawing 1 in Appendix B.

Boreholes BH-2 and BH-3 were advanced using a track mounted CME-55 drill rig, equipped with a hollow stem auger and standard soil sampling equipment operated by a specialist drilling contractor, Landcore Drilling Inc. Due to difficulties to access the inlet and outlet sides with the drill rig, boreholes at these locations (BH-1 and BH-4) were advanced using hand drilling/sampling equipment (a portable tripod with hammer) operated also by Landcore Drilling Inc.

The boreholes drilled through the embankment BH-2, BH-2A, BH-2B and BH-3 were advanced up to depths of 7.9 m, 8.1 m, 19.5 m and 21 m, respectively. BH-2 and BH-2A were drilled until auger/ split spoon refusal (BH-2), or cored up to 0.15 m into the concrete (BH-2A), and BH-2B and BH-3 were drilled up to desired depths of 20 m. BH-1 and BH-4 were advanced to depth of 10.1 m.

The borehole locations (referenced to the MTM NAD83 coordinate system) and their ground surface elevations were surveyed by **exp** personnel. A point (BM, PT 204) on top of T Bar on the west side of the culvert was taken as a bench mark (STA 10+894.662). MTM coordinates and elevation of the bench mark (5377819.998 N and 339252.288 E; Elev. 256.97 m) was based on the horizontal and vertical control sheets (sheet no. 000, plate no. 0354-0101-033) for Hwy 11/101, Township of Bowman and Carr, provided by the MTO.

During the drilling of the boreholes, soil samples were obtained using a 51 mm outside diameter (O.D.) split-spoon sampler in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586) at intervals ranging from 0.75 m to 1.5 m in depth as shown on the attached borehole logs (Appendix C). The original field (uncorrected) SPT "N" values were recorded on the borehole logs as recommended in the Canadian Foundation Engineering Manual (CFEM, pg. 40) and used to provide an assessment of in-situ consistency or relative density of non-cohesive soils. Field vane testing was conducted in cohesive soils to measure the *in-situ* undrained shear strength of those soils. Field vane test was conducted in accordance with ASTM D2573-08. Two Shelby tube samples were obtained below the culvert invert level. Since the conventional hammer of 63.5 kg was used for sampling done by a portable tripod, the corresponding blow counts were not factored. When a hard stratum was reached sampling of hard material was performed by diamond core drilling, using a 1.5 m long NQ double tube wireline corebarrel. Since the boreholes BH-2A and BH-2B were drilled adjacent to BH-2, they were sampled below the level of BH-2 termination.

Upon completion of the boreholes, ground water level measurements were carried out from the boreholes (BH-2B and BH-3) in accordance with the Ministry of Transportation guidelines. The measured ground water levels after completion of drilling boreholes were recorded on borehole log

sheets in Appendix C. Since the wash boring technique was used to drilled boreholes (BH-1 and BH-4), the stabilized ground water level could not be established by short term observation in boreholes. The boreholes were decommissioned by bentonite/cement mixtures in accordance with the Ministry of the Environment Regulation 903, as amended by Regulation 128/03 (the well regulation under the *Ontario Water Resources Act*).

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

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

1.3.2 Laboratory Testing

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

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

1.3.3 Previous Investigation

No foundation reports are available in the MTO GEOCREs library for this particular site. However, three foundation reports related to the adjacent sites on Hwy 11 were recovered from the MTO GEOCREs library. These documents are as follows:

1. Foundation Investigation Report for Watabeag River Bridge, Hwy 11, Site No. 39E-141, District #14, New Liskeard, ON; WP 267-85-02; Geocres No. 42A-41; Ministry of Transportation and Communication; April 1989.
2. Foundation Investigation and Design Report Variable Message Sign #12, Hwy 11 Westbound, Approximately 0.9 Km East of Highway 101 West Junction Matheson, Ontario; GWP 5762-04-00; WP 5674-04-01; Geocres No. 42A-70; Golder Associates; June 2008.
3. Foundation Investigation and Design Report Hwy 11 Passing lane No. 3, Culvert Extension - STN 22+565, Embankment – STN 22+450 to 22+590, 2.7 Km North of West Junction of Hwy 101, New Liskeard, ON; GWP 5217– 08-00; WP 5219-08-01; Geocres No. 42A-89; LVM/MERLEX; February 2012.

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 gravelly sand fill underlain by sandy silt fill and clayey silt fill. Fill layers are followed consecutively by native deposits of silty clay and clay. A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

1.4.1 Fill-Gravelly Sand

Gravelly sand fill layer was encountered at the road embankment below the 76 mm asphalt in BH-2 and BH-3. The layer is approximately 1.5 m thick. It extends from Elevation 258.9 m to Elevation 257.2.0 m.

The composition of this fill layer is sand and gravel with occasional cobbles, and some silt. The material is brown in color, and moist. The SPT “N” values within this layer typically ranged from 10 to 17 blows per 300 mm penetration, suggesting compact relative density.

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

Moisture Content:

- 6.3% to 10.6%

Grain Size Distribution:

- 14% gravel;
- 60% sand; and
- 26% silt and clay.

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

1.4.2 Fill-Sandy Silt

A layer of sandy silt fill was encountered below the gravelly sand fill in BH-2 and BH-3. The thickness of this layer ranged from 1.6 m (BH-2) to 3.1 m (BH-3) extending from Elevation 257.4 m to Elevation 254.1 m.

The composition of this fill layer is sand and silt with trace gravel, few to little clay and trace rootlets. This material is brown to grey in color, and moist. The SPT “N” values within this layer ranged from 10 to 16 blows per 300 mm penetration, suggesting compact relative density.

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

Moisture Content:

- 20.6% to 28.2%

Grain Size Distribution:

- 4 % gravel,
- 22% to 31% sand, and
- 65% to 74% silt and clay.

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

1.4.3 Fill-Clayey Silt

A layer of clayey silt fill was encountered below the sandy silt fill in BH-2 and BH-3. The thickness of this layer ranged from 4.8 m (BH-2) to 2.3 m (BH-3) extending from Elevation 255.9 m to Elevation 251.0 m. BH-2 was terminated in this layer upon split spoon refusal.

The composition of this layer is silt and clay, with trace gravel, trace to little sand, occasional cobbles, trace organics. The material is brown to grey in color, and moist. The SPT “N” values within this layer ranged from 7 to 15 blows per 300 mm penetration, suggesting firm to very stiff consistency of this layer.

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

Moisture content:

- 18.9% to 27.8%

Grain Size Distribution:

- 1% gravel,
- 4% sand,

- 52% to 62% silt, and
- 33% to 43% clay.

Atterberg Limits:

- Liquid Limit: 37% to 41%
- Plastic Limit: 19%
- Plasticity Index: 18% to 22%

The results of the moisture content, grain size distribution and Atterberg Limits tests are provided on the record of borehole sheets in Appendix C. The results of the grain size distribution tests and Atterberg Limits tests are also provided on Figure 3 and Figure 6 in Appendix D.

1.4.4 Clayey Silt

A layer of clayey silt was encountered at the ground surface in BH-1 and BH-4. The thickness of this layer ranged from 2.3 m (BH-4) to 3.1 m (BH-1) extending from Elevation 252.2 m to Elevation 248 m.

The composition of this layer is silt and clay with some peat, some roots and rootlets and trace sand. This material is dark brown to black in color, and moist to wet. The SPT "N" values within this layer ranged from 1 to 9 blows per 300 mm penetration, suggesting very soft to stiff in consistency.

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

Moisture Content:

- 28.4% to 61.7%

Grain Size Distribution:

- 2% sand, and
- 98% silt and clay.

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

1.4.5 Silty Clay to Clay

A layer of silty clay to clay was encountered below the clayey silt in BH-1 and BH-4, below clayey silt fill in BH-3 and below 7.62 m from the ground surface in BH-2B. The thickness of this layer ranged from 7.5 m to 14.1 m, extending from Elevation 251.8 m to Elevation 237.7 m. BH-1, BH-2B, BH-3 and BH-4 were terminated within this layer.

The composition of this layer is silt and clay, with trace sand seams and trace rootlet. The material is brown to grey in color, and moist to wet. The SPT “N” values within this layer ranged from 1 to 12 blows per 300 mm penetration, suggesting very soft to stiff in consistency.

Field vane tests were performed to examine undrained shear strengths of the silty clay to clay layer with depth. The results of the *in-situ* field vane tests measured in BH1-1, BH-2B, BH-3 and BH-4 are shown on the Record of Borehole sheet in Appendix C. The undrained shear strength profile, which includes data from all four boreholes, is shown on Figure 8 in Appendix D. The result of field vane tests showed that the corrected undrained shear strength of this layer ranged from 11 KPa to 39 KPa suggesting very soft to firm in consistency.

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

Moisture content:

- 29.2% to 60.4%

Grain Size Distribution:

- 1% to 2% sand,
- 6% to 34% silt, and
- 65% to 93% clay.

Atterberg Limits:

- Liquid Limit: 43% to 72%
- Plastic Limit: 20% to 25%
- Plasticity Index: 23% to 47%

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 5a, Figure 5b and Figure 7 in Appendix D.

1.5 Water Conditions

Information of groundwater levels at the site was obtained by measuring the water levels in the open boreholes (BH-2B and BH-3) after completion of drilling. The groundwater levels encountered in the boreholes are shown on the borehole logs. Since the wash boring method was used for drilling of BH-1 and BH-4, an accurate groundwater level at these holes was not able to be measured in the open holes at the time of drilling operations.

The water level measured at the time of investigation (December 2014) through the open boreholes (BH-2B and BH-3) were at Elevation 240.3 m and 240.55 m respectively. Water levels measured in open

boreholes might not be stabilized due to short term observation. However, based on moisture content of the soil samples observed during drilling and measured subsequently in the lab, the inferred ground water level within the embankment was estimated to be at approximate Elevation of 250.5 m or slightly higher. This is in a good agreement with the water level observed in the culvert at the time of investigation which was at Elevation 250.6 m at the inlet and 250.1m at the outlet.

Seasonal variations in the water table should be expected, with higher levels occurring during wetter periods of the year and lower levels during drier periods. Some perched water over clayey silt layers could exist in the embankment fill as well.

PART II: ENGINEERING DISCUSSIONS AND RECOMMENATIONS

2.1 General

This section of the report provides geotechnical design recommendations for replacement of the existing corrugated steel pipe (CSP) culvert situated beneath Hwy 11, approximately 325 m east of Watabeag River in Township of Bowman/Carr. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the current investigation at the site and presented in **Part I-Foundation Investigation Report**. The interpretation and recommendations provided are intended solely to permit designers to assess foundation alternatives and design the proposed 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.

We understand that the existing culvert below Hwy 11, which is a 950 mm diameter by 50 m long CSP, is going to be replaced by a new culvert installed at a new proposed location to the west of the existing culvert as shown on Drawing 1, Appendix B. The new proposed alignment is approximately 5 m offset from the existing culvert alignment. The size and type of the new culvert is not defined at the time of writing this report.

This report addresses the geotechnical design of the foundation for the proposed 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)* (November 2006), the *Canadian Foundation Engineering Manual (CFEM)* (2006), MTO Gravity Pipe Design Guidelines (May 2007) and good practice. As requested in the RFP, this section also provides discussion about the suitability of traditional cut and cover, and trenchless methods (i.e. jack and bore, pipe jacking using a tunnel boring machine, pipe ramming, micro-tunneling, utility tunneling, pipe bursting, pipe swallowing, in-line replacement, etc.) of culvert replacement at the specific site. Pertinent construction issues for both methods from a geotechnical standpoint are examined in general accordance with the Terms of Reference from MTO Letter dated November 03, 2014.

2.2 Expected Ground Conditions

According to the results of current foundation investigation, the following ground conditions along the new proposed culvert alignment are evident:

- a. The highway embankment consists of gravely sand fill underlain by subsequent layers of sandy silt and clayey silt fills.
- b. The total thickness of the embankment fill along the proposed culvert alignment ranged from 6.9 m to 7.9 m at the investigated locations (BH-2, BH-2B and BH-3).
- c. If cut and cover method is used for the culvert replacement, the foundation soil of the new

proposed culvert is anticipated to be native silty clay to clay, assuming that approximate elevations of the culvert invert are between 250.4 m at the inlet and 249.8 m at the outlet. The actual founding level will depend on the thickness of the bottom of the culvert, design invert levels, and the depth of the granular bedding required to support the culvert.

- d. The undrained shear strength of the silty clay to clay soil located below the existing roadway is measured to be between 30 to 40 kPa, while the undrained shear strength of that soil located outside of the existing roadway is measured to be between 20 to 30 kPa.
- e. If tunnelling method (trenchless) is used for the culvert replacement, the excavation will be through clayey silt fill and underlying silty clay to clay.
- f. The groundwater levels in the open boreholes on the embankment crest were recorded between approximate elevations 240.3 m and 240.55 m at the time of investigation, which is well below the invert level. Based on moisture content of the soil samples observed during drilling and measured subsequently in the lab, the inferred ground water level within the embankment was estimated to be at approximate Elevation of 250.5 m, which is slightly above the invert level. However, seasonal variations in the water table should be expected.
- g. The water level observed in the culvert at the time of investigation was at Elevation 250.6 m at the inlet and Elevation 250.1m at the outlet.
- h. Even though the slopes of the embankment were covered by snow some slope instability on these slopes was observed. In particular, on the north side (i.e. outlet side) of the embankment, it was observed that the portion of the slope was depressed as explained in Section 1.2.1 of this report. Based on this limited observation, it could be speculated that the local surficial slope failure occurred in the past and remediated with layers of rock fill. On the south slope of the embankment (i.e. inlet side), a fan-shaped sinkhole located approximately 5 m from the inlet of culvert was observed. It is recommended that these two locations will be thoroughly investigated after melting of snow.

Assuming the founding levels given above in Point c (i.e. invert elevations: 250.4 m at the inlet and 249.8 m at the outlet), the Factored Geotechnical Resistance at ULS and the Geotechnical Reaction at SLS for the proposed culvert are given in the table below.

Table 2.1 Recommendations of Factored Geotechnical Resistance at ULS and Geotechnical Reaction at SLS for the proposed culvert

Location	Foundation Soil	Assumed Culvert Width	Geotechnical Resistance at ULS	Geotechnical Reaction at SLS (for total settlement of 25 mm)
Preloaded area under existing embankment fill	Silty Clay to Clay (Cu=30 to 40 kPa)	1 m	150 kPa	130 kPa
Other areas under and outside of existing embankment fill	Silty Clay to Clay (Cu=20 to 30 kPa)	1 m	100 kPa	75 kPa

The silty clay to clay stratum under the existing embankment would have consolidated under the stresses imposed by the embankment. Since there will be no grade raise, theoretically 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.

2.3 Construction Options

The selection of appropriate construction methods for this culvert installation considered (i) whether disruption of the traffic is acceptable or not; (ii) whether a new alignment is proposed or not; (iii) soil conditions at zone of culvert installation; and (iv) diameter and length of the new culvert. Further, several items to keep in mind during the selection were: (i) only approach that would allow removal of the existing culvert is a cut and cover method; (ii) the trenchless approaches involve construction adjacent to the current alignment with the need to decommission the existing culvert including grouting and sealing; (iii) even though cobbles and boulders were not encountered at the proposed levels of tunnelling, appropriate equipment and construction method shall be selected based on ability to accommodate these obstructions; and (iv) provision must be made to maintain surface water flow to the outlet.

Considering all above, the several options were considered as possible alternatives for the new culvert installation method:

- Traditional cut and cover methods (i.e. construct temporary detour and open cut /unsupported excavation; and half-and-half construction using shoring system); and
- Trenchless (tunnelling) methods (i.e. jack and auger bore; pipe ramming; micro-tunnelling; and TBM tunnelling)

Full road closer of Hwy 11 with long detours around the area using existing roadways likely is not acceptable, therefore it was not considered as an option for this culvert replacement. Pipe bursting, pipe swallowing and in-line replacement methods were also not considered in this project since the existing culvert is proposed to be replaced by the culvert installed along the new alignment.

Table 2.2 summarizes advantages, disadvantages and respective estimated cost of suggested methods.

Table 2.2 Installation methods for culvert replacement along the proposed new culvert alignment

Installation Method (see schematic sketches in Appendix H)		Advantages	Disadvantages	Relative Cost*	Ranking**
Cut and Cover Methods	Construct Temporary Detour and Open Cut Unsupported Excavation	<ul style="list-style-type: none"> Assessment of the foundation soil The existing CSP culvert can be used to maintain the surface water flow during the construction Removal of existing CSP culvert if the excavation includes the existing culvert location Existing embankment fill can be removed and replaced with free draining granular material Adaptable to changing ground conditions 	<ul style="list-style-type: none"> Traffic interruption Time need to build the detour road Large amount of soil to be excavated High costs to build detour road on north or south side of Hwy 11 Risk of cost overrun and inability to finish job: low to moderate 	More expensive than other cut and cover methods with shoring systems due to high costs to build detour roads	3 (if traffic interruption is acceptable)
	Half and Half Construction using shoring system Shoring System with Sloping Cuts	<ul style="list-style-type: none"> Assessment of the foundation soil Existing mixed quality embankment fill can be removed and replaced with free draining granular material The existing CSP culvert can be used to maintain the surface water flow during the construction Removal of existing CSP culvert if the excavation includes the existing culvert location Short mobilization time Straight forward construction and construction procedures 	<ul style="list-style-type: none"> Traffic interruption Roadway protection required for up to 7 m deep excavation High cost of shoring system (i.e. road protection) Decommissioning of old culvert required including grouting and sealing, if the excavation does not include the existing culvert location Large amount of soil to be excavated Risk of cost overrun and instability to finish job: low to moderate 	Likely less expensive than other cut and cover methods and trenchless methods	1 (if traffic interruption is acceptable)

Installation Method (see schematic sketches in Appendix H)			Advantages	Disadvantages	Relative Cost*	Ranking**
		<i>Shoring System with Braced Cut</i>	<ul style="list-style-type: none"> • One or possibly two lanes (e.g. steel decking, but costly) of traffic flow maintained on existing road • Assessment of the foundation soil • Global stability of excavation enhanced by narrow geometry • The existing CSP culvert can be used to maintain the surface water flow during the construction • Removal of existing CSP culvert if the excavation includes the existing culvert location • Less traffic interruption than shoring system with sloping cuts approach 	<ul style="list-style-type: none"> • Bracing (e.g. struts) may interfere with excavation • Excavation material and placement of bracing required in limited space • Limited removal of existing fill • Decommissioning of old culvert required including grouting and sealing, if the excavation does not include the existing culvert location • More expensive due to cost of shoring 	More expensive than other cut and cover methods with shoring systems due to higher costs of shoring	2 (if traffic interruption is acceptable)
<i>Trenchless Methods</i>		<i>Jack and Auger Bore</i> (Non-entry Method)	<ul style="list-style-type: none"> • No traffic interruption and requirement for detour route • Commonly used in Ontario • Relatively simple operation • Short mobilization time • Best applied in uniform soil with no free water present • Applicable for relatively short culverts • The existing CSP culvert can be used to maintain the surface water flow during the construction • Less expensive than other trenchless methods 	<ul style="list-style-type: none"> • Pipe can be difficult to steer/direct • Settlement of existing embankment due to loss of ground during jack and bore operations. Short and long term settlement • Requires decommissioning of old culvert, including grouting and sealing • Large entry pit size • Excavation and possible shoring required to achieve required resistance for advancing pipe • Fluid to support annular space • More expensive than cut and cover method • Dewatering possibly required at launching pit • Risk of cost overrun and instability to finish job: low to moderate 	Less expensive than other trenchless methods but more expensive than cut and cover method with shoring systems due to high cost associated with tunneling and constructing launching pits	1 (if traffic interruption is not acceptable)

Installation Method (see schematic sketches in Appendix H)		Advantages	Disadvantages	Relative Cost*	Ranking**
	<i>Pipe Ramming</i> (Non-entry Method)	<ul style="list-style-type: none"> • No traffic interruption and requirement for detour route • Not very sensitive to ground condition • Suitable for steel pipes up to 1.8 m in diameter and best up to 50 m long • Accommodates obstructions well • Little surface settlement • Soil removed after pipe in place • The existing CSP culvert can be used to maintain the surface water flow during the construction 	<ul style="list-style-type: none"> • Pipe can be difficult to steer/direct • Requires decommissioning of old culvert, including grouting and sealing • Excavation and possible shoring required to achieve required resistance for advancing pipe • Large entry pit size • Possible ground heave • Vibrations • Slower than other trenchless methods • Dewatering possibly required at launching and receiving pits • More expensive than cut and cover methods and jack and bore method • Risk of cost overrun and instability to finish job: moderate to high 	Slightly more expensive than jack and bore method	2 (if traffic interruption is not acceptable)
<i>Trenchless Methods</i>	<i>Micro-tunnelling</i> (Non-entry Method)	<ul style="list-style-type: none"> • No traffic interruption and requirement for detour route • Handles wide variety of ground conditions • Ability to control excavation face stability • No dewatering required • Minimum surface disruption • Very accurate • The existing CSP culvert can be used to maintain the surface water flow during the construction 	<ul style="list-style-type: none"> • High construction cost • Obstruction problematic • Requires decommissioning of old culvert, including grouting and sealing • Excavation and shoring require to achieve starting grade • Requires large area for jacking shaft and support equipment • Not suitable for short drive • Dewatering possibly required at launching and receiving pits • More expensive than cut and cover method and jack and bore method • Risk of cost overrun and instability to finish job: moderate to high 	Significantly more expensive than jack and bore method	3 (if traffic interruption is not acceptable)

Installation Method (see schematic sketches in Appendix H)		Advantages	Disadvantages	Relative Cost*	Ranking**
	<i>TBM Tunnelling (Man-entry Method)</i>	<ul style="list-style-type: none"> • No traffic interruption and requirement for detour route • Good control of settlement • Safe to use in mixed ground condition • Ability to access obstructions during tunnelling • Cost may be reduced if and where existing contractor's suitable TBM is available • The existing CSP culvert can be used to maintain the surface water flow during the construction 	<ul style="list-style-type: none"> • High capital investment • Not practical for small diameter pipe (min. 1.8 m diameter) • Requires decommissioning of old culvert, including grouting and sealing • Dewatering possibly required at launching and receiving pits • More expensive than open cut and jack and auger bore methods • Risk of cost overrun and instability to finish job: moderate to high 	Significantly more expensive than jack and bore method	4 (if traffic interruption is not acceptable)

Notes:

* *Relative Cost* is determined for 0.9 m to 1.8 m diameter pipes based on Table 1 "Average cost of trenchless techniques with more than five data records, in Construction and Rehabilitation Costs for Buried Pipe with Focus on Trenchless Technologies", NRC-CNRC report No. IRC-RR-101.

** The ranking presented distinguishes two cases: (i) disruption of traffic is acceptable, and (ii) disruption of traffic is not acceptable.

Based on the above list of advantages and disadvantages of cut and cover and trenchless construction methods, cut and cover methods might be considered as more viable methods from a geotechnical and/or foundation perspective if disruption of traffic at Hwy 11 is allowed. The major advantages of this approach are possibility to assess the foundation soil below the proposed location of the new culvert, and to remove the existing culvert if the excavation includes both locations of the proposed and existing culverts. On the other hand, the major disadvantage is disruption of traffic. Among these cut and cover methods, the half-and-half excavation/construction using shoring system with open cut sides appears to be the most economical. The detour option using open unsupported excavation is estimated as a less economical and practical option due to high costs and time required for building the detour road. The other half-and-half excavation/construction method using shoring system with braced cut also appears less economical and practical considering the high price of the shoring and distance between the existing and proposed alignments of 5 m (i.e. length of struts should be in order of 10 m to allow removal of the existing pipe).

However, if the Regional Traffic office requires replacing the culvert without disrupting traffic, then trenchless installation methods listed in Table 2.1 are more viable. The major disadvantages of these trenchless installation methods are higher cost of installation than the cut and cover methods and the need to decommission the existing culvert by grouting and sealing. Among these trenchless methods, the jack and auger bore method is assessed as the most economical followed by the pipe ramming method. Micro-tunnelling and TBM tunnelling are ranked as the least viable trenchless methods since the size of the tunnel is not favorable for these methods (i.e. the tunnel is relatively short to be economical for these two methods).

Therefore, based on the site conditions and characteristics of methods elaborated above, the following options for the culvert construction at the proposed alignment are discussed in the following sections:

- Culvert installation by cut and cover methods: open cuts with detour, and half-and-half construction using shoring system along the centerline of the road and unsupported cut sides
- Culvert installation by trenchless methods: jack and auger bore, pipe ramming, micro-tunnelling and TBM tunnelling

2.4 Culvert Installation by Cut and Cover Method

Constructing a temporary detour followed by open cut unsupported excavation appears to be one of the viable culvert replacement methods if the temporary detour approach south or north of Hwy 11 to maintain two-way traffic is acceptable and feasible (see Figure H.1 in Appendix H). A temporary detour with one-way signalized traffic could be also an option, if it is acceptable to this location. With this approach, grouting and sealing of the existing culvert (i.e. decommissioning) will be

eliminated since this option will allow removal of the existing pipe. It will also allow for the assessment of the foundation soils below the proposed location, and if any peat or soft materials are encountered they can be removed. The existing culvert could be used for maintenance and diversion of surface water flow during the construction. However, as mentioned before, the method with the temporary detour could be very expensive due to high costs to built detour roads.

Therefore, the staged construction with shoring system (i.e. half-and-half construction) along the centerline of the road and unsupported cut sides is assessed as a more viable culvert replacement method for this project (see Figure H.2 in Appendix H), assuming that an one-way signaled traffic is allowed. The construction sequences for this method may include: (i) one lane of the highway is utilized while the other lane of the highway embankment is excavated; (ii) half the new culvert is constructed and rebuilt the embankment to grade in this side; and (iii) the traffic could be moved onto the new fill and the process is repeated to complete the construction.

2.4.1 Temporary Excavations

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). The embankment fill soils (i.e. gravelly sand fill, sandy silt fill and clayey silt fill) and firm to stiff native silty clay to clay soil may be classified as a Type 3 soil above the groundwater table and Type 4 soil below the groundwater table, in conformance with the OHSA. The very soft to soft native silty clay to clay Type 4 soil may be considered as a Type 4 soil. Excavations are mostly expected to be above the groundwater level measured in the boreholes on the embankment crest during this investigation. To avoid disturbance of the founding subgrade and to allow placement of backfill in dry conditions, groundwater must be controlled to below the proposed excavation levels prior to digging to final levels.

Temporary excavations side slopes for Type 3 soil should not exceed 1H:1V in accordance with OHSA, and 2H:1V is recommended for global stability of deep cuts (i.e. to maintain a global factor of safety greater than 1.3) where excavation will be left open for some times. Temporary excavation side slopes for Type 4 soils should not exceed 3H:1V. 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. Depending on topography and overland flow drainage path beyond the crest of the proposed cut slope, a drainage ditch may be required near the crest to divert water away from the cut slope to prevent wash out and erosion. For the anticipated roadway cut, a 2 m wide bench should be incorporated into the slopes at the approximate mid-height of the slope and extend the full length of cut. Although the slope is considered to be stable without the bench, incorporation of the bench will allow for control of erosion and sloughing of surficial soil.

2.4.2 Construct Temporary Detour and Open Cut Unsupported Excavation

Based on the available space on the north and south sides of the existing highway, temporary on-site detour embankment may be constructed to maintain the two-way traffic flow during the construction of the new culvert (see Figure H.1 in Appendix H). The excavation scheme should

follow the excavation practices outlined in Section 2.4.1. The surface water flow could be maintained by the existing culvert.

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 206 SP 206S03 (dated March 2012).

2.4.3 Half-and-Half Construction using Shoring System

Since the excavation in the road embankment is relatively high (up to 8 m) a temporary shoring system will be required as a roadside protection system. A shoring system such as soldier pile and lagging or sheet piles can be employed for the temporary excavation. Given the setting, the internal bracing for horizontal support, such as a system of walers and rakers, would be required and the impact on excavation operations must be accommodated. It will be the Contractors' responsibility to design a suitable temporary support system for the MTO review prior to installation. The Contractor should follow OPSS 538 and SP No. 902S01, regarding excavations for structures, and OPSS 539 and SP No. 105S19, regarding temporary protection systems.

The Contractor shall be responsible for the complete design, construction, monitoring and removal of the installed roadside protection system (i.e. shoring). The protection system shall be designed to provide the protection for excavations as required by OHSA, at locations specified in the contract, and at any locations where the stability, safety or function of existing structures and/or utilities may be impaired by construction work. Decommissioning must be consistent with good practice and in an effective manner to avoid interference with highway systems and utilities. The protection system should be designed for the Performance Level 1a (maximum horizontal displacement allowance is 5 mm) or 1b (maximum horizontal displacement allowance is 10 mm) (OPSS 539). The minimum requirements for monitoring shall include the survey measurements of scaled targets attached to the shoring wall at the elevations specified. A maximum spacing of scaled targets is 6 m. Surface elevation points should also be established and monitored at distance of 1.5 m, and 3.0 m behind the walls at a lateral spacing of 15.0 m. All surveying should be conducted by a licenced land surveyor. If movement approaches the allowable limit, suitable measures should be taken to verify the stability of the protection system and to ensure that the movement does not exceed the performance level specified in the Contract Documents.

This method provides roadway protection parallel to the highway between EBL and WBL, diverts traffic and undertakes open cut (i.e. sloping cut parallel to the highway). The roadway protection with a parallel orientation can take the form of reversible shoring (i.e. use the same wall as support for both halves of construction). All unsupported sides of the excavation through existing fill and/or native silty clay to clay soil would require side slopes no steeper than 2H:1V as outlined in Section 2.4.1. 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 water flow diversion and/or control (e.g. pumping) must be developed by the Contractor. The existing culvert could be utilized for water control prior to its replacement.

2.4.3.1 Lateral Earth Pressure

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

$$P = K (\gamma h + q)$$

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 groundwater levels measured on the site.

Suggested soil parameters for soil types that will be encountered in the design of the shoring are provided in Table 2.3.

The mobilization of full active or passive resistance requires a measurable and perhaps significant wall movement or rotation. Therefore, unless the structural element can tolerate these deflections, the at-rest earth pressure should be used in design.

The effect of compaction surcharge should be taken into account in the calculations of active and at rest earth pressures. The lateral pressure due to compaction should be taken based on the proposed compaction equipment loadings. This lateral pressure distribution should be added to the calculated active (or at rest) pressure. Notwithstanding, hand held compaction equipment and smaller lifts should be used within 3.0 m from the face of the walls to prevent overstressing.

Table 2.3 Material types and earth pressure properties

Material	Friction Angle ϕ' (unfactored)	Coefficient of Active Earth Pressure (K_a)	Coefficient of Passive Earth Pressure (K_p)	Coefficient of Earth Pressure at Rest (K_o)	Unit Weight γ (kN/m ³)
Gravelly Sand Fill	32°	0.31	3.25	0.47	18.5
Sandy Silt Fill	30°	0.33	3.00	0.50	18.5
Clayey Silt Fill	25°	0.36	2.77	0.53	18.5
Soft Silty Clay	18°	0.53	1.89	0.59	16
Stiff Silty Clay	24°	0.42	2.37	0.59	16

Note: Values given for horizontal earth pressures are for horizontal backfill. For sloping backfill, the design requirements outline in Sec. C6.91(c) of the Canadian Highway Bridge Design Code should be used.

2.4.4 Maximum Fill Height

Pipe selections for new culverts must conform to maximum height restrictions as outlined in OPSD for Roads and Public Works (e.g. OPSDs, 805.010 for CSP, 805.020 for CSP-Arched, and 807.010 for Reinforced Concrete Pipe-Confined Trench, 807.030 for Reinforced Concrete Pipe).

2.4.5 Culvert Bedding

OPSDs 802.010, 802.031, and 802.032, which are included in Appendix F, provide the bedding, embedment, cover and backfill standards for the different pipe materials. According to these standards the culvert bedding should consist of Granular "A" (OPSS 1010) with thickness of 300 mm or alternatively a 100 mm thick concrete working slab with 75 mm of bedding materials beneath the culvert and extend a minimum of 500 mm horizontally on either side of the culvert edge. The bedding material should be placed in layers not exceeding 200 mm in thickness, loose measurement, and compacted to at least 95% of the Standard Proctor Maximum Dry Density (SPMDD) before a subsequent layer is placed in accordance with OPSS 514. Bedding material placed in the haunches must be compacted prior to continued placement of cover material. Bedding on each side of the pipe shall be completed simultaneously. At no time shall the levels on each side differ more than the 200 mm uncompacted layers.

Prior to placing any fill materials, 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 to 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 μ m.

For the Township of Bowman/Carr, a frost penetration depth of approximately 2.4 m can occur in open, unheated areas without snow cover (OPSD 3090.101). At the culvert inlet and outlet, and beneath the proposed culvert, the native soils consist of a silty clay to clay material. This material has high frost susceptibility based upon the MTO Frost Classification guideline of percent particles between 5 to 75 μm . However, to avoid any frost heave problem at inlet and outlet of the culvert, non-frost susceptible materials such as sand and gravel (Granular "A") are recommended to be provided to the limit of frost penetration beneath the inlet and outlet of the culvert. 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 bedding along the entire culvert length. Alternatively, the frost protection could be achieved using the properly designed polystyrene insulation.

2.4.6 Culvert Backfill

The culvert backfill should consist of Granular "B", Type I or Granular "A" (OPSS 1010) placed in layers not exceeding 300 mm in thickness for the full width of the trench and each layer shall be compacted to 95% of the Standard Proctor Maximum Dry Density before placing a subsequent layer, according to OPSS 514.

The culvert should be encased with a minimum of 300 mm of compacted material. Typical backfill diagrams are presented in Appendix F, OPSD 802.010, 802.031 and 802.032. The minimum height of fill cover above the crown of the pipe before power operated tractors or rolling equipment shall be 900 mm, unless otherwise noted by the structural engineer.

2.4.7 Dewatering

The soil encountered below the groundwater table and within potential excavation depths consist of native silty clay to clay. The material could be susceptible to disturbance from groundwater and mobilized equipment. In general, the groundwater level needs to be controlled at least 1 m below to the excavation level to avoid disturbance, and any surface or groundwater seepage should be removed from the excavation prior to culvert bedding material placement of granular backfill. Pumping using properly filtered sumps, and/or filtered drains placed along the base of the excavation should provide sufficient groundwater control during foundation works.

Provided that the existing CSP culvert is to remain in use during construction of the new culvert, the majority of the upstream flow of the existing culvert can be diverted around the construction area. It is the responsibility of the Contractor to propose a suitable dewatering system based on the time of construction and groundwater levels and surface water flow conditions for prior approval of the MTO. The method used should not undermine the existing road.

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

2.5 Culvert Installation by Trenchless Method

Tunnelling will be a viable installation method for culvert replacement along the proposed culvert alignment if interruption of traffic on Hwy 11 is not acceptable. Some tunnelling methods are considered and discussed in Section 2.3 based on soil and groundwater conditions at the site and the dimensions of the new culvert. For all trenchless installation methods the procedures should conform to all relevant Ontario Provincial Standard Specifications (OPSS), Non Standard Special Provisions (NSSP) such as Installation by Trenchless Method and industrial standards. According to OPSS 421, the minimum spacing allowed between new culvert and existing culvert is 600 mm (if pipe diameter <1.2 m) or a 0.5 times of pipe diameter (1.2 m < pipe diameter < 2.4 m). However, since the existing pipe is proposed to be abandoned, it is recommended that the new alignment has to be at least 3 pipe diameter offset to the west, relative to the existing culvert. The existing abandoned culvert must be properly decommissioned including grouting and sealing. Any offset of the new alignment toward the east is not recommended since the buried concrete structure was encountered during the drilling of BH-2 and BH-2A.

It is projected that the culvert trenchless excavation will be carried out through “mixed face” soil conditions (i.e. clayey silt fill and native silty clay to clay), assuming that the approximate elevation of the new culvert invert is between 250.4 m at the inlet and 249.8 m at the outlet. Based on the measurements during this investigation, the inferred ground water level within the embankment was estimated to be at approximate Elevation of 250.5 m or slightly above, which appears to be above the tunnel invert. However, seasonal variations in the groundwater table should be expected.

Cobbles and boulders were not encountered in the zone of tunnelling during borehole investigations, however it can be expected that they may be present in the embankment fill. The NSSP for these obstructions in the fill is included in Appendix G.

The best viable tunnel excavation methods assessed in Table 2.1 are discussed in the subsequent sections, as well general tunnelling construction considerations.

2.5.1 Tunnel Excavation Methods

2.5.1.1 Pipe Jack and Auger Bore

Pipe jack and auger bore method involves jacking a pipe through the soil with a hydraulic ram and removal of soil with an auger. A cutting head is fixed to the leading edge of the pipe. The auger transports spoils from the cutting head back to the bore pit. The direction of the auger head can only be controlled using a rudimentary steering system where minor adjustments can be made. One of the advantages of using the jack and bore method for the pipe installation is that the auger can be manually removed to permit cleaning of obstructions such as cobbles and boulders. Further, the auger can be adopted to use rock-cutting teeth, if necessary. However the steering ability and grade is limited.

The elevation and gradient of the pipe must be closely controlled during the course of the jack and bore. For the proposed jack and bore installation in “mixed face” soil, boring and jacking operations should be performed simultaneously. It is recommended that auger would always be maintained at least 1 m behind the cutter edge. The jacking and boring operations should be continued without stoppage until completion. To reduce (but not eliminate) loss of ground and associated disturbance, consideration may be given to jacking the pipe across the alignment as far as practical, prior auguring. However, obstacles, if encountered in the embankment fill, could make this difficult or impractical. Lubrication may be provided to reduce the friction between the pipe and the borehole walls. The characteristics of the surrounding soil should be considered in selecting the appropriate lubricant.

Considering the soil and groundwater conditions at the site and the short length of the proposed culvert (~50 m), the pipe jack and auger bore method is suitable for installation of the new culvert. Based on the current investigation data and assumed invert levels, it appears that the pipe jacking and boring will be performed through soft to stiff clayey fill and native soils, and the groundwater level will be slightly above the invert of the pipe. Therefore, a small amount of groundwater seepage into the tunnel would be expected in the zone of tunnelling. However, due to seasonal variation of groundwater levels, the dewatering might need to be carried out to temporarily lower the ground water level below the bottoms of the sanding pit. Further, surface water flow should be controlled. This would likely utilize the existing culvert prior to its decommissioning.

Excavation shoring for the bore/jacking pits can be completed as recommended in previous sections of this report.

2.5.1.2 Pipe Ramming

Pipe ramming is a trenchless method for installation of steel pipes over distances typically up to 50 m long and up to 1.8 m in diameter. The method uses pneumatic percussive blows to drive the pipe into ground. Spoil removal from the pipe can be done by auger. It typically requires excavation of two pits, but the ramming can be launched without an insertion pit if the ram is design to start at the side of a slope. The pipe ramming method is suitable for installation of the proposed culvert, since the required length of 50 m is at the upper limit of the method, and the tunnelling will be in soft to stiff clayey soils mostly above the water table. Installation is very noisy and difficult to steer. For this method surface water has to be controlled, and the existing culvert has to be properly decommissioned.

2.5.1.3 Micro-tunnelling

Micro- tunneling method is a non-entry, remotely controlled, guided 2-stage process, which provides continuous support to the excavation face. In this method a Micro Tunneling Boring Machine (MTBM) is used for soil cutting, while a pipe is jacked into place behind the cutting head with hydraulics. The MTBM is equipped with a slurry spoil removal system to control the groundwater inflow and counterbalance the earth and hydrostatic pressure while tunneling through the mixed face conditions. The cutting tool and the drilling fluid must be able to handle the different

materials and the “mixed face” condition. In order to minimize the resistance along the pipe exterior, a bentonite grout lubricant can be injected behind the cutting face. Steel, concrete or fibreglass pipes can be installed with this method.

The major advantage of micro-tunnelling method is that its performance is not affected by high groundwater levels, so the dewatering is not required. Major disadvantages of micro-tunnelling for this project are considered to be the relatively high cost of mobilization and lack of locally skilled contractors. This option may become more attractive if potential bidders have available equipment in house.

2.5.1.4 TBM Tunnelling

TBM tunnelling is a man-entry tunnelling method and encompasses the use of a tunnel boring machine (TBM). This method utilizes laser-guided targeting that achieves a very accurate line and grade to the pipe being installed. To control ground movement behind the TBM a primary liner must be installed. TBM can employ single pass or two pass system. In the two pass system the temporary liner can be ribs and lagging with the permanent liner cast-in-place afterwards. The primary liner can be provided by steel, cast iron or precast concrete liner plates. The space on the outside of the liner plates should be grouted as soon as possible, to reduce ground loss and ground settlement. Primary support can be also provided by jacking a pipe from a jacking station behind the boring machine. Pipes may be made of various materials (concrete, steel, fibreglass, etc.). Selected pipe must conform to OPS requirements for embankment depth as noted in Section 2.4.4.

The launch pit and jacking station should be constructed at the inlet side. If there is no sufficient depth to construct a thrust wall for jacking, the jack reaction can be resisted by means of a structural framework constructed above ground having adequate restraint provided by means of piles, ground anchors or other such methods for transferring horizontal loads. TBM tunnelling with pipe jacking method is limited by the jacking force available.

TBM tunnelling might be the most expensive method for the installation of the proposed culvert considering the short length of the tunnel. In addition, this method would require oversizing of pipe to minimum 1.8 m. Cost might be reduced if, and where, existing Contractor's suitable TBM is available.

2.5.2 Considerations of Tunnelling

2.5.2.1 Groundwater Control

As mentioned before, a small amount of groundwater seepage into the tunnel should be expected in the zone of tunnelling. However, the dewatering might be required in the launching pit prior to advancing the pipe to ensure dry working conditions and stabilize the excavation in that zone. The dewatering would need to be carried out to temporary lower the groundwater level to at least 1 m below the base of the excavation. Dewatering requirements will be governed by the time of the year when the construction is performed. It is the responsibility of the Contractor to propose a

suitable dewatering system based on the time of construction and groundwater levels. The method used should not undermine the existing highway. Dewatering shall conform to OPSS 517.

2.5.2.2 Ground Settlement

Settlement around the culvert is a combination of ground loss or “immediate” settlement caused by tunnelling, and consolidation settlement. The immediate settlement is a direct result of the overcut and movement of ground at the heading during tunnelling. The factors that influence the immediate settlement include the soil strength and the method of tunnelling. Based on soil characteristics of the site, an experienced Contractor should be able to keep the settlement under the MTO’s required limit of 10 mm. Technical specifications should ensure that:

- The use of over-cutters (excavating to a diameter greater than the pipe diameter) is kept under 10 mm;
- The overcut area is grouted in a timely manner (if a man-entry tunnel is constructed grout should be injected immediately after support is installed); and
- The program of instrumentation is carried out as per MTO guidelines.

In the event that the settlement is greater than the allowable 10 mm, some soil stabilization measures such as grouting or ground freezing might be applied to arrest or reduce settlement.

Before the construction of the new culvert starts, the existing culvert must be decommissioned and sinkholes, if any, made good by filling and/or grouting.

2.5.2.3 Excavation Pits

The launching and receiving pits for the tunnelling equipment are expected to be located at the inlet and outlet of the proposed culvert location, respectively. The bases of the pits are expected to be set at about 0.5 to 1 m depth from invert of the proposed culvert. Excavations for launching and receiving pits will be conducted through very soft to stiff clayey silt. In order to provide the required excavation geometry for the drilling (e.g. vertical front face for tunnel entry and a vertical rear face with a ballast system to act as a reaction force), the sides of the excavation will have to be shored. Recommendations for shoring are addressed in Section 2.4.3 of this report. Ingress of groundwater and surface water has to be controlled as explained previously. Technical specifications must ensure that the Contractor submits a groundwater and surface water control plan describing the proposed method for control.

2.5.2.4 Backfilling in Pits

It is anticipated that backfilling work will be required at the launching and receiving pits to return site condition to pre-construction grades. The following comments and recommendations are provided for backfilling such excavations.

All excavations should be backfilled with inorganic on-site soils placed in maximum 300 mm thick lifts and compacted to at least 95% of the Standard Proctor Maximum Dry Density (SPMDD). Any organic, excessively wet, compressible or otherwise deleterious materials should not be used for backfilling purposes. Any shortfall of suitable on-site excavated materials can be made up with imported and approved materials.

All backfill and compaction operations should be monitored by qualified geotechnical personnel to approve materials, to evaluate placement operations, and to verify that the specified degree of compaction is being achieved throughout the fill.

2.5.3 Monitoring and Contingency Plan

It is emphasized that the resulting performance of the installed culvert will largely be dependent upon construction procedures and techniques. However, regardless of the method of tunnelling selected for this project, it is recommended that the contractor develop a contingency plan incorporating appropriate soil volume monitoring to address loss of material from outside the pipe during the tunnelling operation. This plan should include at a minimum the following items:

- a) an "Alert" level(s), e.g. the percentage of soil in excess of 100% of the displaced soil, at which the plan would be implemented;
- b) a means to close the tunnel, and preferably to pressurize the pipe; and
- c) an emergency personnel/agency contact list.

Settlements should be monitored during construction to ensure compliance with MTO guidelines and the contract requirements. The instrumentation program should adequately verify effects of tunnelling on the overlying highway and obtain advance warning of ground movements. The scope and layout of settlement instruments should be in general accordance with the MTO guidelines (Appendix: Settlement Monitoring Guideline – Tunnelling). This should include a series of surface monitoring points placed at a maximum spacing of 5 m along the entire length of the proposed culvert. All monitoring points located in the unpaved portion of the right-of-way are to be founded below the frost penetration depth, which is typically 2.4 m in this area.

A reading schedule should be as follows:

- A minimum one set of readings prior to construction.
- A minimum three sets of readings during construction provided the movements are within the anticipated limits. Otherwise, the reading frequency may have to be increased.
- A minimum of two sets of readings on a weekly basis after completion of the work.

Instrumentation plans should be finalized once the Contractor is selected and when his construction methods are known.

As mentioned, control of ground settlement on this project depends on the behaviour of soil at the tunnel face and on the tunnelling methodology employed by the Contractor. Therefore, it is recommended that a geotechnical engineer be present during active excavation to verify that the ground conditions are consistent with those encountered in the investigation boreholes. Furthermore, it is recommended that the volume of the material removed from the tunnel be monitored and continuously compared to the rate of tunnel advance. This will provide an indication if any over-excavation is taking place.

The criteria for evaluation of settlement should be based on the following action levels:

1. *Review Level:* If a maximum value of 10 mm relative to the baseline readings is reached, the method, rate or sequence of construction, or ground stabilization measures shall be reviewed or modified to mitigate further ground displacements.
2. *Alert Level:* If a maximum of 15 mm relative to the baseline readings is reached, the Contractor shall be required to cease construction operation or to execute pre-planned measures to secure the site to mitigate further unacceptable settlement and to assure safety of public.

2.5.4 Protection Systems

Depending on the tunnelling method chosen for this project and the excavations that will be required to implement them, protection system(s) may be required for the existing roadway. The need for these systems will depend on the proposed geometry of the required excavations and their proximity to the existing highway structure. If required, protection systems (design, materials, construction, maintenance, monitoring and removal) will be required to meet the specifications set out in OPSS 539 and Special Provision No. 105S19. Recommendations for the protection systems are presented in Section 2.4.3.

2.6 Inlet and Outlet

2.6.1 Erosion Protection at Outlet

Rip-rap protection should be provided at the culvert outlet. 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 surface water 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 downstream bed should generally follow the OPSD 810.010, which is included in Appendix F of this report. Rip-rap placed at 1V:1H will be stable.

2.6.2 Stream Bed Rip-Rap

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

2.6.3 Seepage Cut-off Requirements

The existing seepage cut-off requirements should be reviewed as well in the following context. The silty soil on the site has a high potential for migration with high seepage gradients. For the culvert replacement and new 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) cut-off trench constructed with geotextile (iii) steel or wooden sheet pile cutoff at the upstream end of culvert, (iv) cut-off wall incorporated in the apron slab (if one is used) of the culvert, and (v) rockfill at the upstream end of the culvert barrel to terminate below the granular bedding of the culvert. The two most feasible methods for this particular project, 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.

2.6.3.1 Clay Seal

Where readily available, a clay seal should be placed at the inlet of the proposed culvert, to prevent the migration of material along the face of the culvert, the formation of flow paths, and any potential internal erosion within the highway embankment (OPSD 802.095, Appendix F). OPSS 1205 specifies that materials 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 requirements of the clay seal:

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

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

2.6.3.2 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 aprons 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.

2.6.4 Frost Protection

A frost penetration depth of approximately 2.4 m can occur in open unheated areas of the Township of Bowman/Carr without snow cover. At the culvert inlet and outlet, and beneath the proposed culvert, the native soils consist of a silty clay to clay material. The silty clay to clay material has a high frost susceptibility based upon the MTO Frost Classification guideline of percent particles between 5 and 75 μm . Therefore, non-frost susceptible materials such as sand and gravel (Granular "A") and/or polystyrene insulation need to be provided as outlined in Section 2.4.5.

2.7 Assessment of Slope Stability

To assess the global stability of the existing embankment (Sta.10+910) 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, a series of slope stability analyses were performed. The static slope stability analyses were performed using the Morgenstern-Price method developed on the basis of limit equilibrium. The SLOPE/W computer program developed by GeoSlope International was employed for computation.

Stability assessments of existing slopes under static conditions were performed on the cross-section perpendicular to the highway at the proposed culvert location. The cross-section of the existing embankment with the approximate slope of 2H:1V was established based on **exp's** survey data. The stratigraphy and groundwater condition at the site were developed based on the results of the geotechnical investigation presented in Part I - Foundation Investigation Report.

Based on the borehole information, the subsoils encountered at the work area consist of embankment fill, underlain by native silty clay to clay deposits. The undrained shear strength of the silty clay to clay is higher in areas that have been preloaded by the existing embankment fill, and also vary with depth, generally increasing with increased overburden pressure.

Given the above, effective stress analyses for a long term stability assessment and total stress analyses for a short term stability assessment (i.e. rapid construction) of the embankment slopes were performed taking into consideration the subsoil conditions encountered directly beneath and adjacent the existing embankment.

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

Tabulated below in Table 2.4 are the soil parameters used for the slope stability analyses. The soil parameters were generally estimated based on the results of field and laboratory investigation.

Table 2.4 Soil properties used in slope stability analyses

Material Type	Long-term Conditions			Short-term Conditions		
	ϕ (degrees)	c (kPa)	γ (kN/m ³)	ϕ (degrees)	c (kPa)	γ (kN/m ³)
Gravelly Sand Fill	32	0	18.5	32	0	18.5
Sandy Silt Fill	30	0	18.5	30	0	18.5
Clayey Silt Fill	28	5	18.5	28	5	18.5
Consolidated Upper Layer of Silty Clay to Clay (in preloaded area below existing embankment fill)	18	5	16	0	35	16
Soft Silty Clay to Clay (in other areas under and outside of existing embankment fill)	18	3	16	0	25	16
Consolidated Lower Layer of Silty Clay to Clay (in preloaded area below existing embankment fill)	20	5	16	0	45	16
Firm Silty Clay to Clay (in other areas under and outside of existing embankment fill)	20	3	16	0	30	16

The results of slope stability analyses for the 2H:1V south side slope of the existing embankment using drained (long term stability) and undrained (short term stability) soil parameters are graphically presented in Figures E1 and E2 in Appendix E, respectively. For the drained soil conditions, a Factor of Safety of 1.34 for a deep seated rotational failure through the native silty clay to clay subgrade was calculated. However, for the undrained soil conditions, a Factor of Safety of 1.2 was calculated, suggesting that the embankment could experience some instability. This is supported by the signs of possible slope instability previously mentioned, although given the snow cover, the presence or magnitude of slope instability cannot be confirmed in this moment. Therefore, the slope stability of the new embankment with the slope flatter than 2H:1V or with a rock fill berm at the embankment toe was analyzed. The results of slope analyses presented on Figure E3 indicate that the new embankment should be constructed with a minimum slope of 3H:1V to achieve a minimum Factor of Safety of 1.3 for undrained soil conditions (short term stability). Alternatively, as it shown on Figure E4, the minimum Factor of Safety of 1.3 could be also achieved by placing a rock fill berm at the toe of the 2H:1V embankment slope.

January 27, 2015

Part III: CLOSURE

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

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

We trust that these comments provide you with sufficient information to proceed with design. Should you have any questions, please do not hesitate to contact this office.

Yours truly,

exp Services Inc.

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

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

Encl.

Yours truly,

exp Services Inc.

Appendix A – Photographs



Photo 1. Facing west on Hwy 11 from the proposed culvert alignment



Photo 2. Facing east on Hwy 11 from the proposed culvert alignment



Photo 3. Facing north on outlet side of existing culvert



Photo 4. Facing south on inlet side of existing culvert



Photo 5. Water level at inlet below thin ice layer



Photo 6. Embankment slope on south side (inlet) facing north



Photo 7. Embankment slope on south side (inlet) facing west



Photo 8. Embankment slope on north side (outlet) facing south



Photo 9. Blockage at culvert inlet side



Photo 10. Sinkhole on south side (inlet) of embankment



Photo 11. Culvert at outlet side



Layer of rock fill at slope
surface

Photo 12. Depressed slope surface - north side (outlet) of embankment

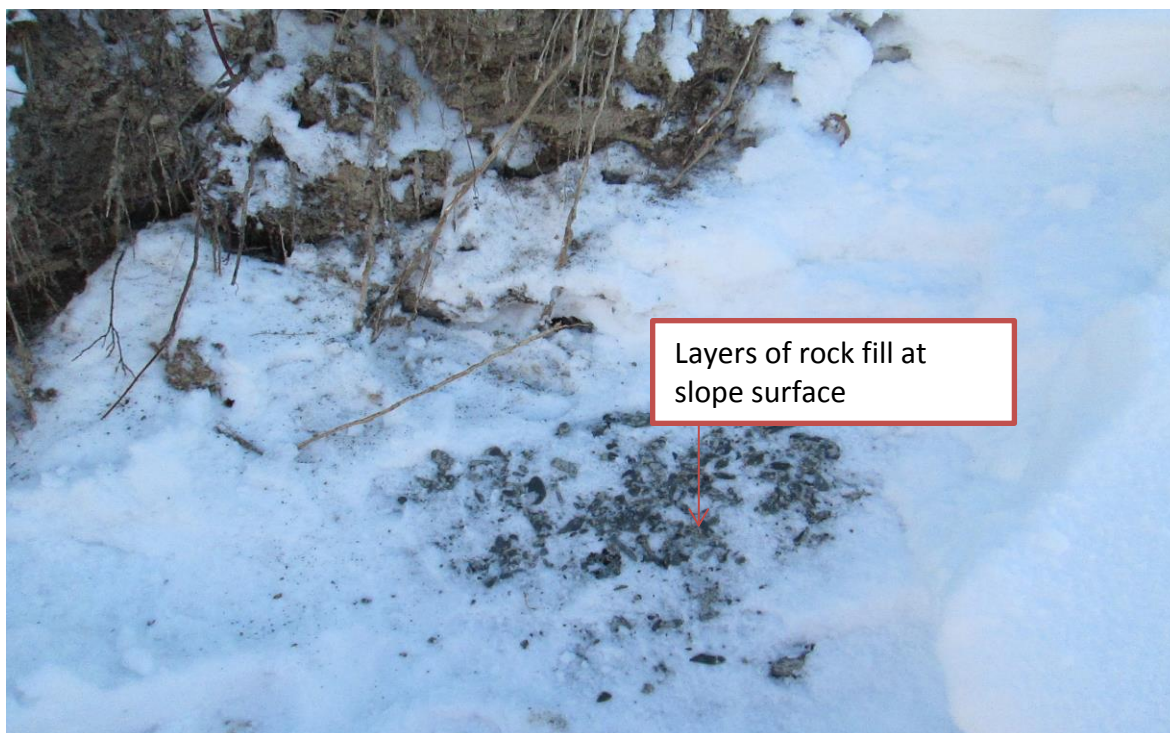


Photo 13. Layer of rock fill at depressed slope surface - north side (outlet) of embankment



Photo 14. Core sample recovered from BH-2A at Elevation 251.0 m to 250.8 m

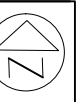
Appendix B – Drawings

DRAFT

METRIC

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

Agreement No. 5013-E-0008
Assignment No. 6
WO 2014-11039



DEEP FILL CULVERT REPLACEMENT

(HWY 11, Township of Bowman/Carr)

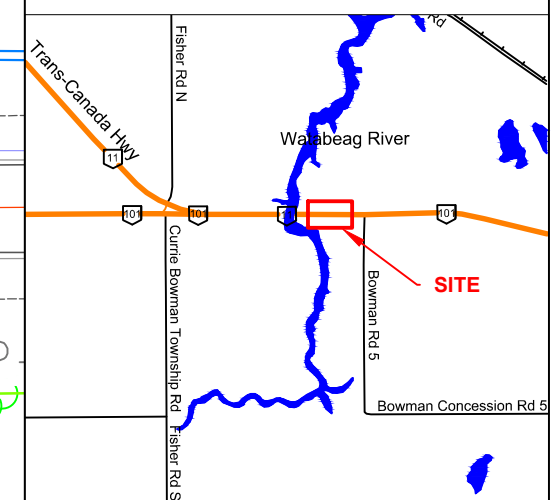
SITE PLAN AND CROSS SECTION

SHEET
1



exp Services Inc.

KEY PLAN



LEGEND



Approximate Investigated Borehole Locations



BM (Bench Mark)

N

Standard Penetration Test (Blows/0.3 m)

SOIL STRATA SYMBOLS



FILL

 TOPSOIL

SAND



SILT



SAND TO SILTY SAND

 **SAND AND GRAVEL**

BH No.	APPROX. ELEV.	MTM CO-ORDINATES	
		NORTH	EAST
BM	256.97	5377820	339252
BH 1	251.05	5377827	339276
BH 2	258.91	5377799	339275
BH 2A	258.92	5377799	339276
BH 2B	258.82	5377799	339268
BH 3	258.70	5377790	339264
BH 4	252.16	5377768	339267

NOTE

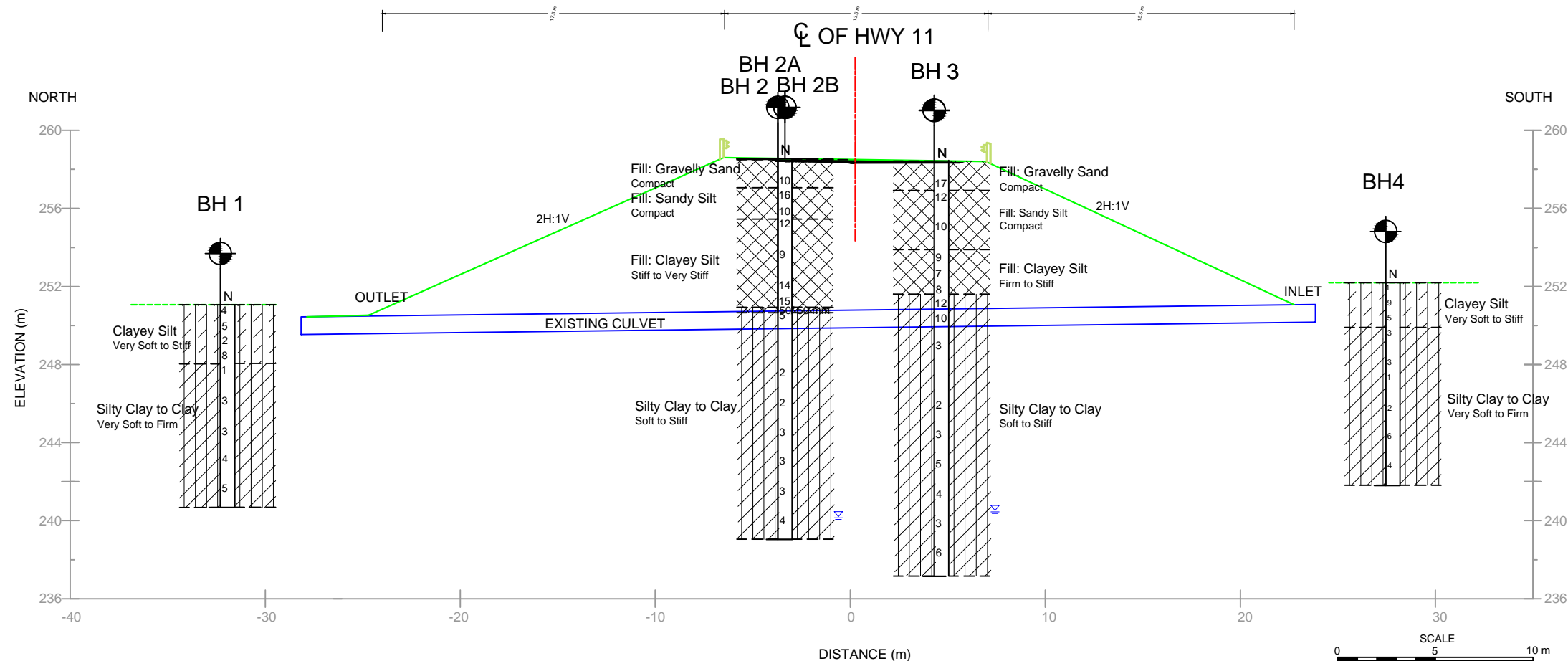
This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents

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.

2015.01.15	SM	SUBMISSION FOR MTO REVIEW
DATE	BY	DESCRIPTION
		GEOCRES NO.
SCALE	1:1000	PROJECT NO. ADM-000
SUBM'D SM	CHECKED SM	DATE 2015.01.15
DRAWN JH	CHECKED SG	APPROVED SG
		DWG. 01

Note: The plan was provided by MTO.

PLAN



CROSS SECTION A-A

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.

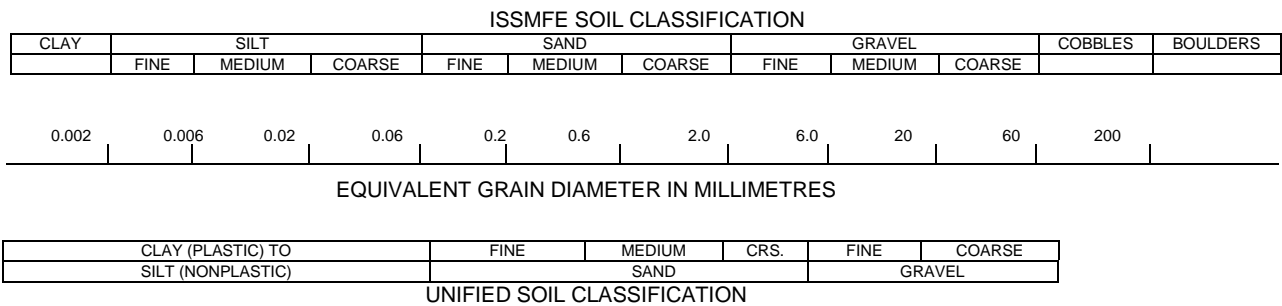
Seam: a thin, confined layer of soil having different particle size, texture, or color from materials above and below.

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



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

Table a: Percent or Proportion of Soil, Pp

	Criteria
Trace	Particles are present but estimated to be less than 5%
Few	$5 \leq Pp \leq 10\%$
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:

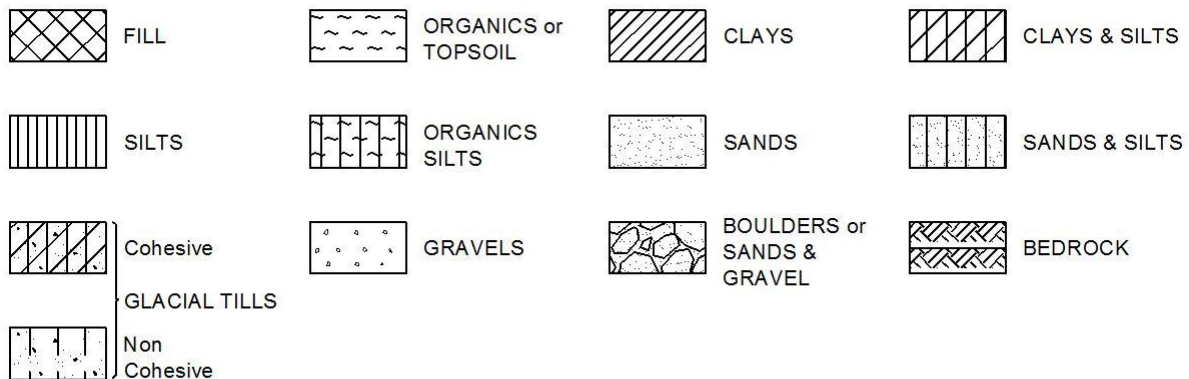
Table c: Consistency of Cohesive Soil

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. WO 2014-11039 LOCATION Township of Bowman/Carr, ON, MTM Z12 E N ORIGINATED BY NT
 DIST HWY 11 BOREHOLE TYPE Portable Tirpod SPT by hand drilling/sampling equipment COMPILED BY NT
 DATUM BM ELEV. 256.97 m MTM Z12 E N DATE 2014/12/10 - 2014/12/10 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									WATER CONTENT (%)												
								○ UNCONFINED	+ FIELD VANE	×	QUICK TRIAXIAL						LAB VANE												
251.1	Ground Surface						20	40	60	80	100	10	20	30	kN/m ³	GR SA SI CL													
	CLAYEY SILT (ML) some peat, some roots and rootlets, trace sand, dark brown to black, wet, soft to stiff		1	SS	4		250									0 2 (98)													
			2	SS	5																								
			3	SS	2																								
			4	SS	8																								
248.0	SILTY CLAY TO CLAY (CL-CH) sand seam, grey, wet, very soft to firm		5	SS	1		247	+3.3								0 1 14 85													
			VANE																										
6			SS	3																									
			VANE																										
7			SS	3																									
			VANE																										
244			+2.2																										
243			+1.8																										
242																													
241			+3.0																										
240.7			END OF BOREHOLE																										
10.4			NOTES: 1. This drawing is to be read with the subject report and project numbers as presented above. 2. Interpretation assistance by exp is required before used by others. 3. No groundwater level was measured because wash boring technique was used.																										

OPG_EXP RECORD OF BOREHOLE 5013-E-0008 ASS'G. 6 - BH LOGS.GPJ ONTARIO MOT.GDT 1/23/15

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

Brampton, Ontario

RECORD OF BOREHOLE No BH-2

1 OF 1

METRIC

W. P. WO 2014-11039 LOCATION Township of Bowman/Carr, ON, MTM Z12 E N ORIGINATED BY NT
 DIST HWY 11 BOREHOLE TYPE CME-55X, Continuous hollow stem augering COMPILED BY NT
 DATUM BM ELEV. 256.97 m MTM Z12 E N DATE 2014/12/11 - 2014/12/11 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			20	40	60	80	100					
258.9	Ground Surface																
258.8	ASPHALT 76 mm thickness		1	AG			258										14 60 (26)
	FILL: GRAVELLY SAND (SW) some silt, occasional cobbles, brown, moist, compact		2	SS	10												
257.4																	
1.5	FILL: SANDY SILT (SM) trace gravel, few clay, brown, moist, compact		3	SS	16		257										4 31 (65)
			4	SS	10												
255.9							256										
3.1	FILL: CLAYEY SILT (ML) trace gravel, trace to little sand, occasional cobbles, trace organics, brown to grey, moist, stiff to very stiff		5	SS	12												
							255										
			6	SS	9		254										
	- Vane attempted, no shear			VANE													
	- becoming sandy silt, some clay, moist, compact		7	SS	14		253										
	- becoming clayey silt, trace gravel, trace sand, moist, very stiff		8	SS	15		252										1 4 62 33
251.0			9	SS	50/50 mm		251										
7.9	END OF BOREHOLE (Split Spoon and Auger Refusal)																
	NOTES: 1. This drawing is to be read with the subject report and project numbers as presented above. 2. Interpretation assistance by exp is required before used by others. 3. Borehole abandon due to unexpected split spoon and auger refusal 4. No groundwater level was observed.																

OPG_EXP RECORD OF BOREHOLE 5013-E-0008 ASSIG. 6 - BH LOGS.GPJ ONTARIO MOT.GDT 1/23/15

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

Brampton, Ontario

RECORD OF BOREHOLE No BH-2A

1 OF 1

METRIC

W. P. WO 2014-11039 LOCATION Township of Bowman/Carr, ON, MTM Z12 E N ORIGINATED BY NT
 DIST HWY 11 BOREHOLE TYPE CME-55X, Continuous hollow stem augering COMPILED BY NT
 DATUM BM ELEV. 256.97 m MTM Z12 E N DATE 2014/12/12 - 2014/12/12 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 ○ UNCONFINED + FIELD VANE × QUICK TRIAXIAL LAB VANE									
258.9	Ground Surface							20	40	60	80	100					
258.8	ASPHALT 76 mm thickness																
	FILL: GRAVELLY SAND (SW) some silt, occasional cobbles, brown, moist																
	FILL: SANDY SILT (SM) trace gravel, few clay, brown , moist																
	FILL: CLAYEY SILT (ML) trace gravel, trace to little sand, occasional cobbles, brown to grey, moist																

OPG_EXP RECORD OF BOREHOLE 5013-E-0008 ASSIG. 6 - BH LOGS.GPJ ONTARIO MOT.GDT 1/23/15

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

Brampton, Ontario

RECORD OF BOREHOLE No BH-2B

1 OF 2

METRIC

W. P. WO 2014-11039 LOCATION Township of Bowman/Carr, ON, MTM Z12 E N ORIGINATED BY NT
 DIST HWY 11 BOREHOLE TYPE CME-55X, Continuous hollow stem augering COMPILED BY NT
 DATUM BM ELEV. 256.97 m MTM Z12 E N DATE 2014/12/14 - 2014/12/14 CHECKED BY SM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH: Cu, KPa									
								○ UNCONFINED	+ FIELD VANE	×	QUICK TRIAXIAL	LAB VANE					
							20	40	60	80	100						
258.8	Ground Surface																
258.7	ASPHALT 76 mm thickness																
	FILL: GRAVELLY SAND (SW) some silt, occasional cobbles, brown, moist																
	FILL: SANDY SILT (SM) trace gravel, few clay, brown, moist																
	FILL: CLAYEY SILT (ML) trace gravel, trace to little sand, occasional cobbles, brown to grey, moist																
251.2																	
7.6	SILTY CLAY TO CLAY (CL-CH) trace sand, trace rootlets, brown to grey, moist to wet, soft to firm		9B	SS	5												
			10	TW													
				VANE													
			11	SS	2												
				VANE													
	- becoming sand seam		12	SS	2												
				VANE													
			13	SS	3												
				VANE													
	- becoming more sand seam, at every 1" interval		14	SS	3												
				VANE													
			15	SS	3												
				VANE													
			16	SS	4												
				VANE													
239.3																	
19.5	END OF BOREHOLE																
	NOTES: 1. This drawing is to be read with the subject report and project numbers as presented above																

Continued Next Page

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

OPG_EXP RECORD OF BOREHOLE 5013-E-0008 ASSIG. 6 - BH LOGS.GPJ ONTARIO MOT.GDT 1/23/15

Brampton, Ontario

RECORD OF BOREHOLE No BH-2B

2 OF 2

METRIC

W. P. WO 2014-11039 LOCATION Township of Bowman/Carr, ON, MTM Z12 E N ORIGINATED BY NT
 DIST HWY 11 BOREHOLE TYPE CME-55X, Continuous hollow stem augering COMPILED BY NT
 DATUM BM ELEV. 256.97 m MTM Z12 E N DATE 2014/12/14 - 2014/12/14 CHECKED BY SM

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	20	40	60	80	100	W _p	W		
	2. Interpretation assistance by exp is required before used by others. 3. BH 2B was drilled at west side of existing culvert alignment due to unexpected split spoon and auger refusal at BH 2 and BH 2A on east side of existing culvert at depth of 7.9 m. similar soil was visually observed as that in BH 2. Therefore sampling was performed after depth of 7.62 m. 4. Groundwater level was measured in open hole.															

OPG_EXP RECORD OF BOREHOLE 5013-E-0008 ASSIG. 6 - BH LOGS.GPJ ONTARIO MOT.GDT 1/23/15

Brampton, Ontario

RECORD OF BOREHOLE No BH-3

1 OF 2

METRIC

W. P. WO 2014-11039 LOCATION Township of Bowman/Carr, ON, MTM Z12 E N ORIGINATED BY NT
 DIST HWY 11 BOREHOLE TYPE CME-55X, Continuous hollow stem augering COMPILED BY NT
 DATUM BM ELEV. 256.97 m MTM Z12 E N DATE 2014/12/12 - 2014/12/13 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									
								○ UNCONFINED + FIELD VANE									
								× QUICK TRIAXIAL LAB VANE									
							WATER CONTENT (%)										
							20	40	60	80	100	10	20	30			
258.7	Ground Surface																
258.6	ASPHALT 76 mm thickness		1	AG			258						○				
	FILL: GRAVELLY SAND (SW) trace to some silt, occasional cobbles, brown, moist, compact		2	SS	17								○				
257.2							257								○		
1.5	FILL: SANDY SILT (SM) trace gravel, few clay, trace rootlets, brown to grey , moist, compact		3	SS	12												
							256										
			4	SS	10		255								○	4 22 (74)	
							254										
254.1							253								○		
4.6	FILL: CLAYEY SILT (ML) trace gravel, trace sand , trace rootlets, brown to grey, moist, firm to stiff		5	SS	9		252										
			6	SS	7		251										
			7	SS	8		250										
251.8	- Vane attempted, no shear			VANE			249										
6.9	SILTY CLAY TO CLAY (CL-CH) sand seam, brown to grey , moist to wet, soft to stiff		8	SS	12		248								○		
			9	SS	10		247								○		
							246										
	- becoming clay, soft		10	SS	3		245										
				VANE			244	+1.6									
			11	TW			243										
				VANE			242	+2.0									
			12	SS	2		241										
				VANE			240	+1.8									
			13	SS	3		239										
				VANE			238	+1.8									
	- becoming silty		14	SS	5		237										
				VANE													
			15	SS	4												
				VANE													
			16	SS	3												
				VANE													
			17	SS	6												
				VANE													
237.7																	

Continued Next Page

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

OPG_EXP RECORD OF BOREHOLE 5013-E-0008 ASS'G. 6 - BH LOGS.GPJ ONTARIO MOT.GDT 1/23/15

Brampton, Ontario

RECORD OF BOREHOLE No BH-3

2 OF 2

METRIC

W. P. WO 2014-11039 LOCATION Township of Bowman/Carr, ON, MTM Z12 E N ORIGINATED BY NT
 DIST HWY 11 BOREHOLE TYPE CME-55X, Continuous hollow stem augering COMPILED BY NT
 DATUM BM ELEV. 256.97 m MTM Z12 E N DATE 2014/12/12 - 2014/12/13 CHECKED BY SM

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	20	40	60	80	100	W _p	W		
21.0	END OF BOREHOLE NOTES: 1. This drawing is to be read with the subject report and project numbers as presented above. 2. Interpretation assistance by exp is required before used by others. 3. Groundwater level was measured in open hole.															

OPG_EXP RECORD OF BOREHOLE 5013-E-0008 ASSIG. 6 - BH LOGS.GPJ ONTARIO MOT.GDT 1/23/15




Brampton, Ontario

RECORD OF BOREHOLE No BH-4

1 OF 1

METRIC

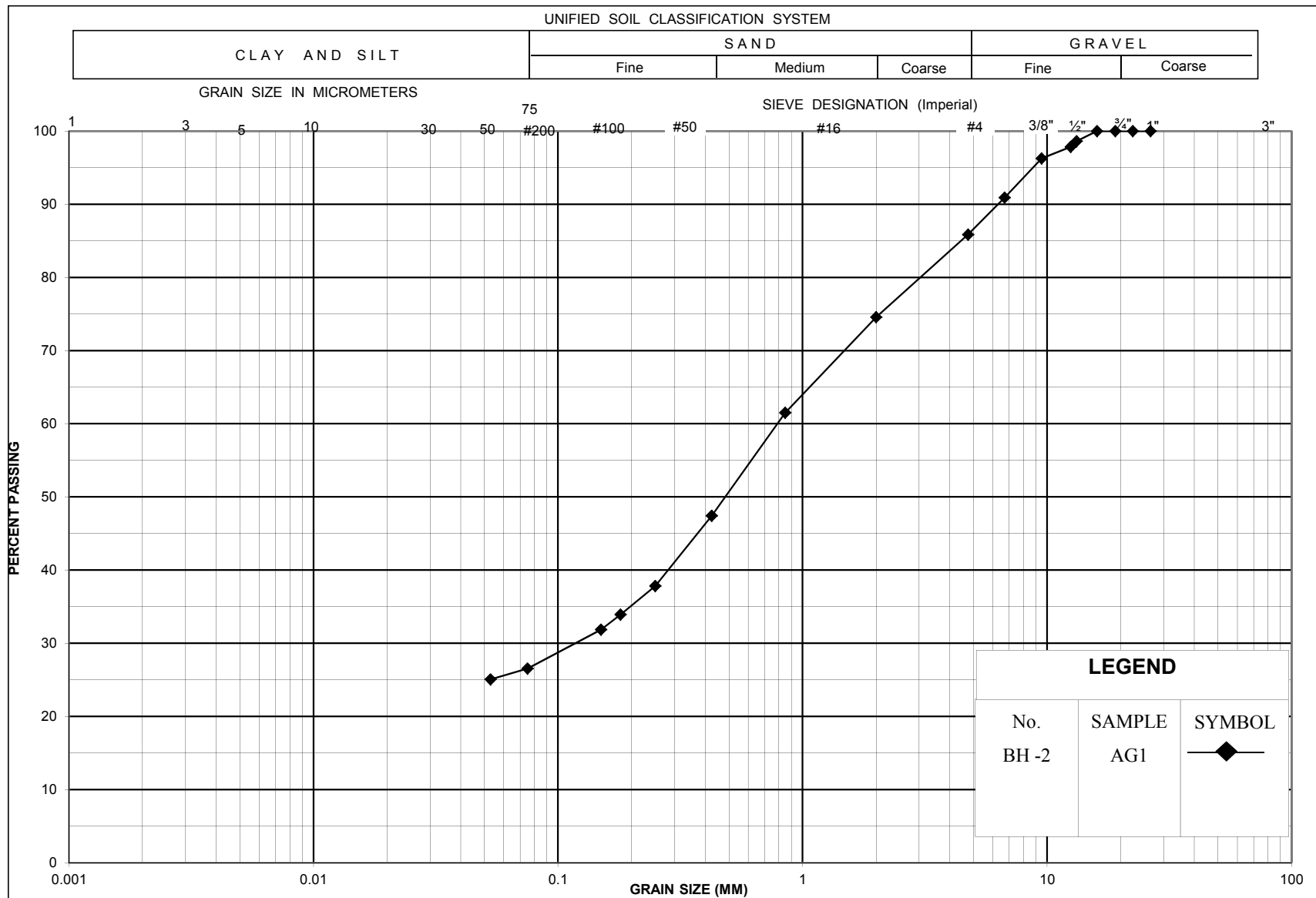
W. P. WO 2014-11039 LOCATION Township of Bowman/Carr, ON, MTM Z12 E N ORIGINATED BY NT
 DIST HWY 11 BOREHOLE TYPE Portable Tirpod SPT by hand drilling/sampling equipment COMPILED BY NT
 DATUM BM ELEV. 256.97 m MTM Z12 E N DATE 2014/12/09 - 2014/12/09 CHECKED BY SM

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	REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH: Cu, KPa ○ UNCONFINED + FIELD VANE × QUICK TRIAXIAL LAB VANE										WATER CONTENT (%)	
252.2	Ground Surface							20	40	60	80	100	10	20	30	kN/m ³	GR SA SI CL		
	CLAYEY SILT (ML) some peat, some roots and rootlets, trace sand, occasional gravel, brown to black, moist, very soft to stiff		1	SS	1		252									43.9	0 2 (98)		
			2	SS	9		251												
			3	SS	5		250												
249.9	SILTY CLAY TO CLAY (CL-CH) sand seam, grey, wet, very soft to firm		4	SS	3		249										45	0 1 25 74	
			VANE					+2.0											
5			SS	3	248												56 40.5		
6			SS	1	247												42.4		
			VANE					+2.3											
7			SS	2	246												45.9		
			VANE					+2.6											
8			SS	6	244												59 47.5		0 0 (100)
			VANE					+1.2											
9			SS	4	243												48.3		
241.8			END OF BOREHOLE		VANE				242										
10.4																			
NOTES: 1. This drawing is to be read with the subject report and project numbers as presented above. 2. Interpretation assistance by exp is required before used by others. 3. No groundwater level was measured because wash boring technique was used.																			

OPG_EXP RECORD OF BOREHOLE 5013-E-0008 ASSIG. 6 - BH LOGS.GPJ ONTARIO MOT.GDT 1/23/15

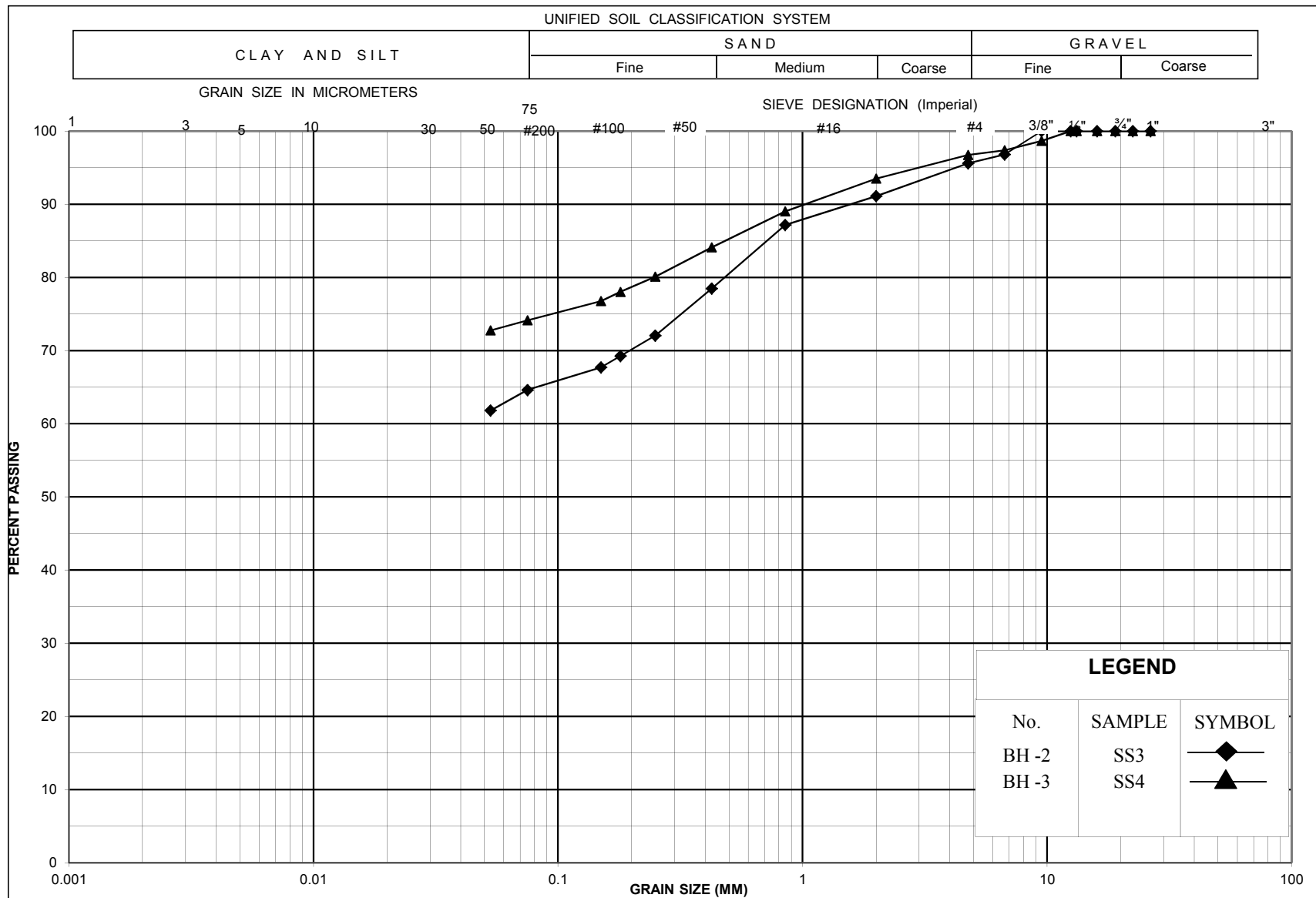
+ 3, × 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

Appendix D – Laboratory Data



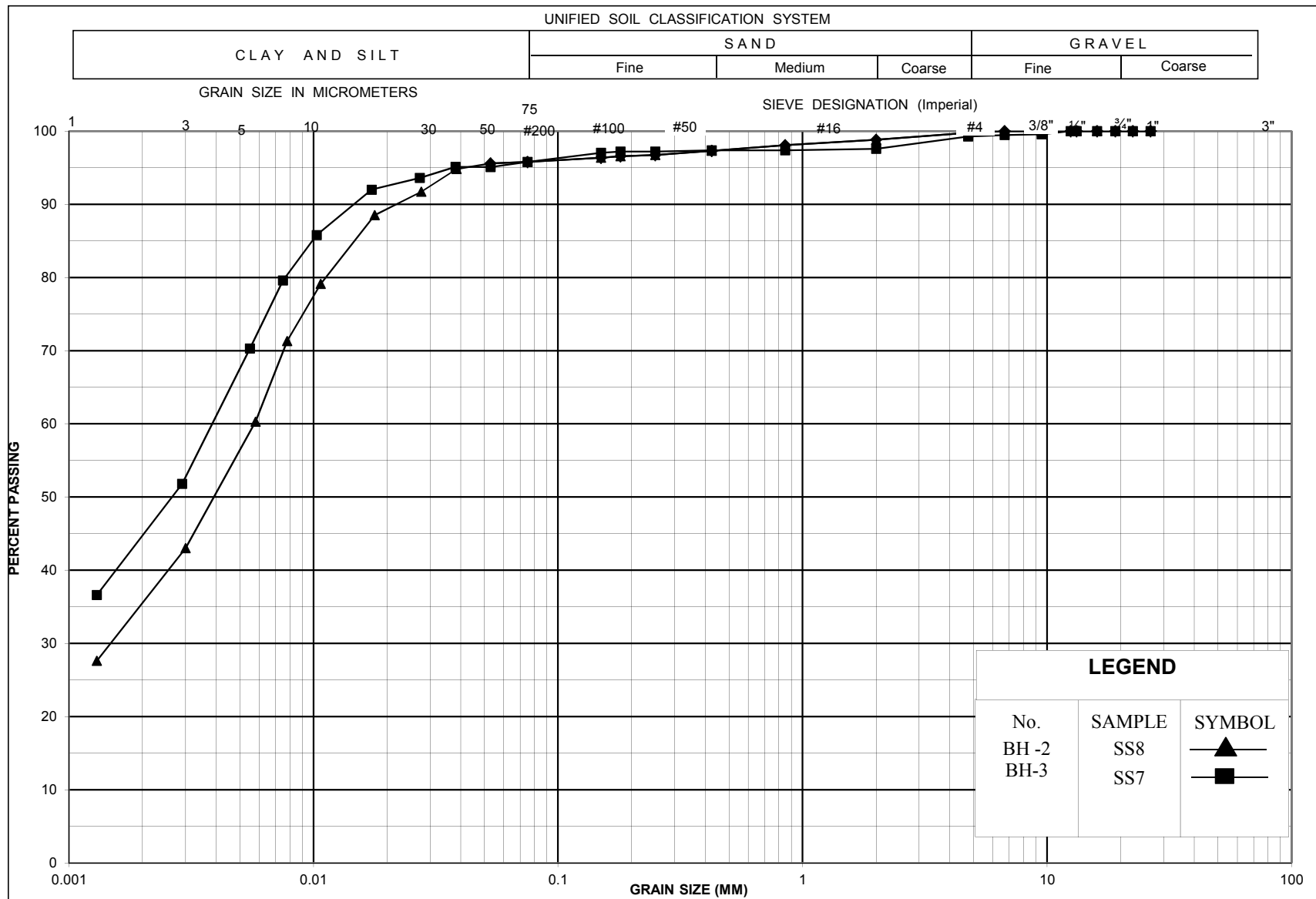
GRAIN SIZE DISTRIBUTION
FILL: GRAVELLY SAND

FIGURE 1
WO-2014-11039
DATE January 2015



GRAIN SIZE DISTRIBUTION
FILL: SANDY SILT

FIGURE 2
WO-2014-11039
DATE January 2015

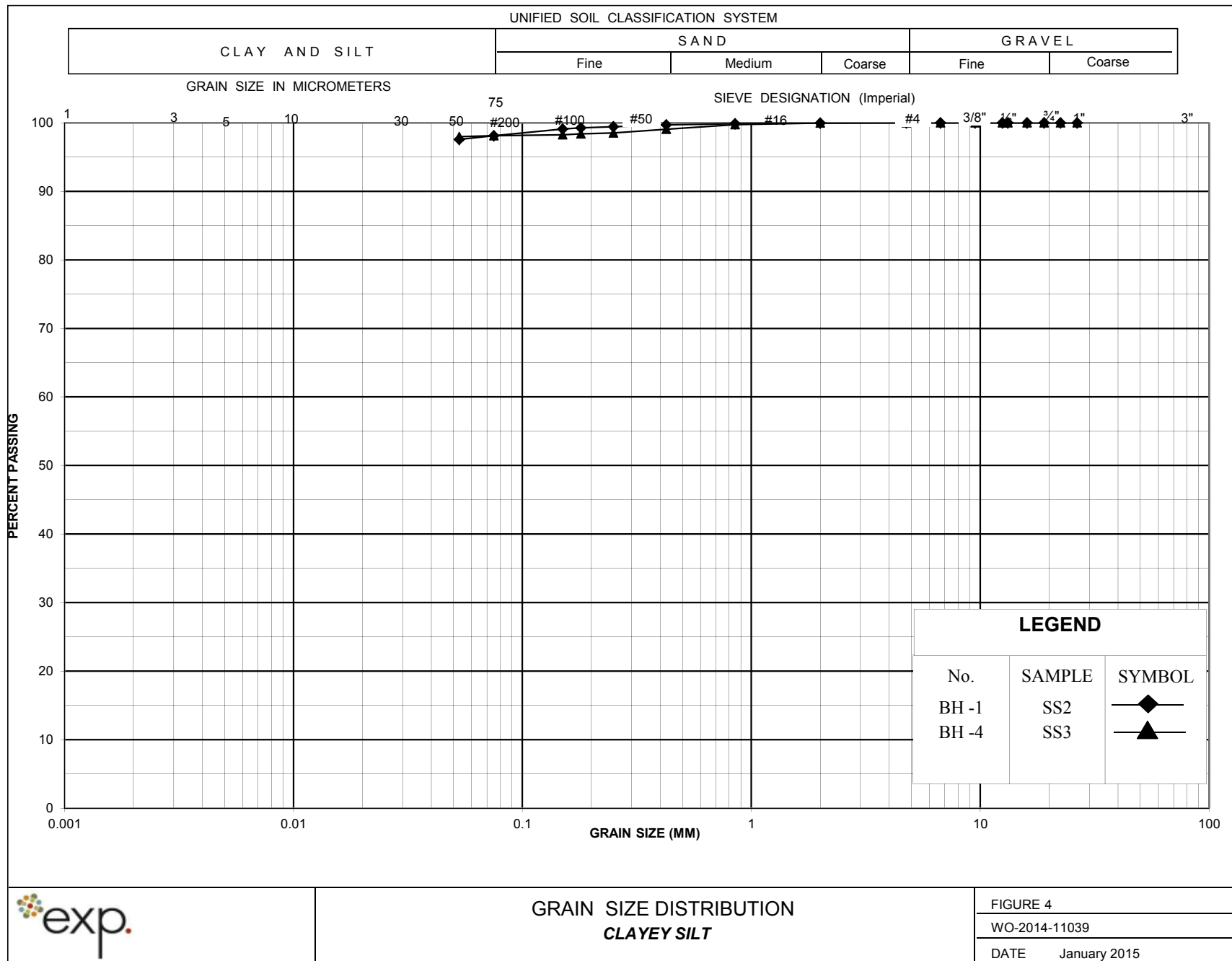


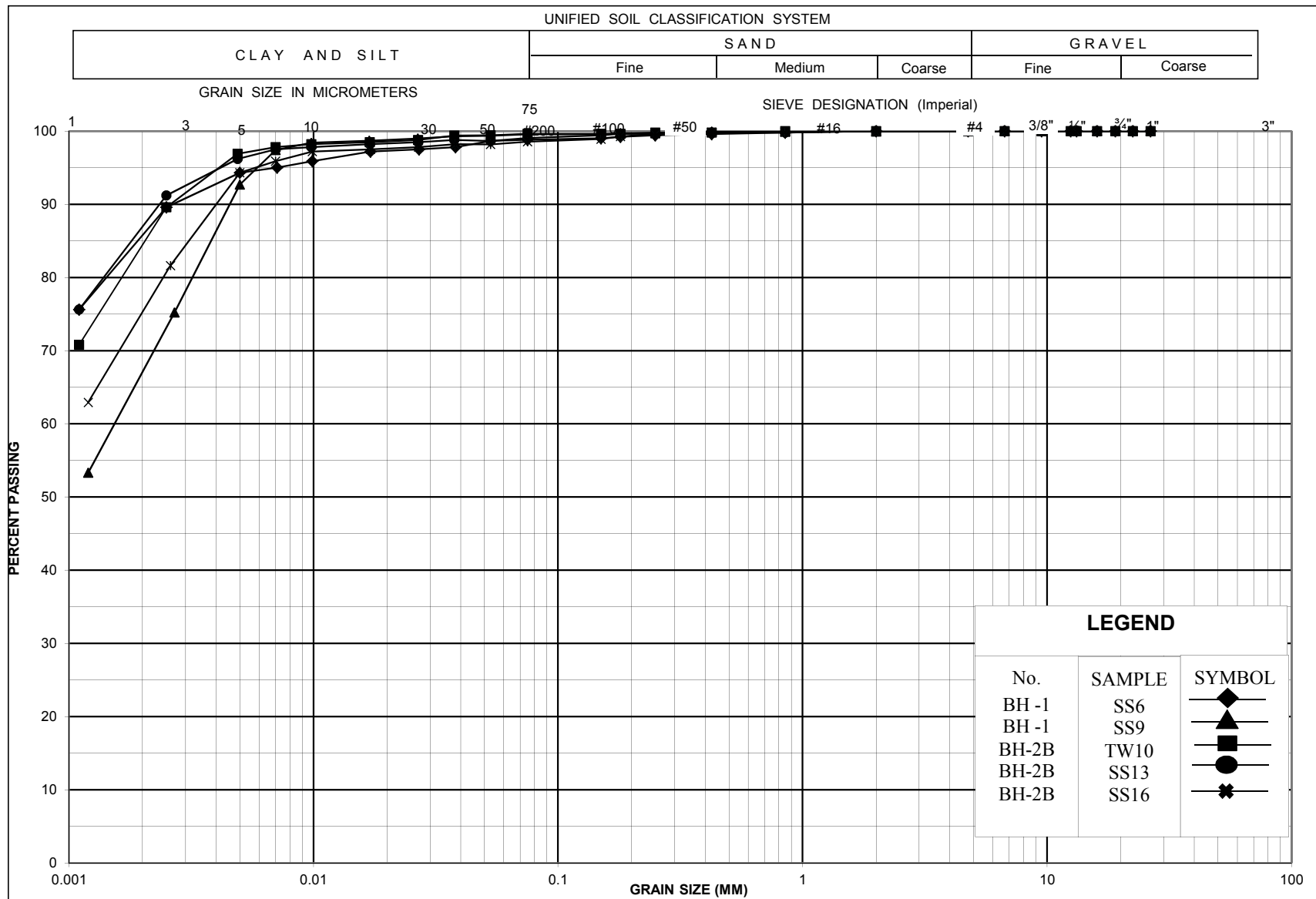
GRAIN SIZE DISTRIBUTION
FILL: CLAYEY SILT

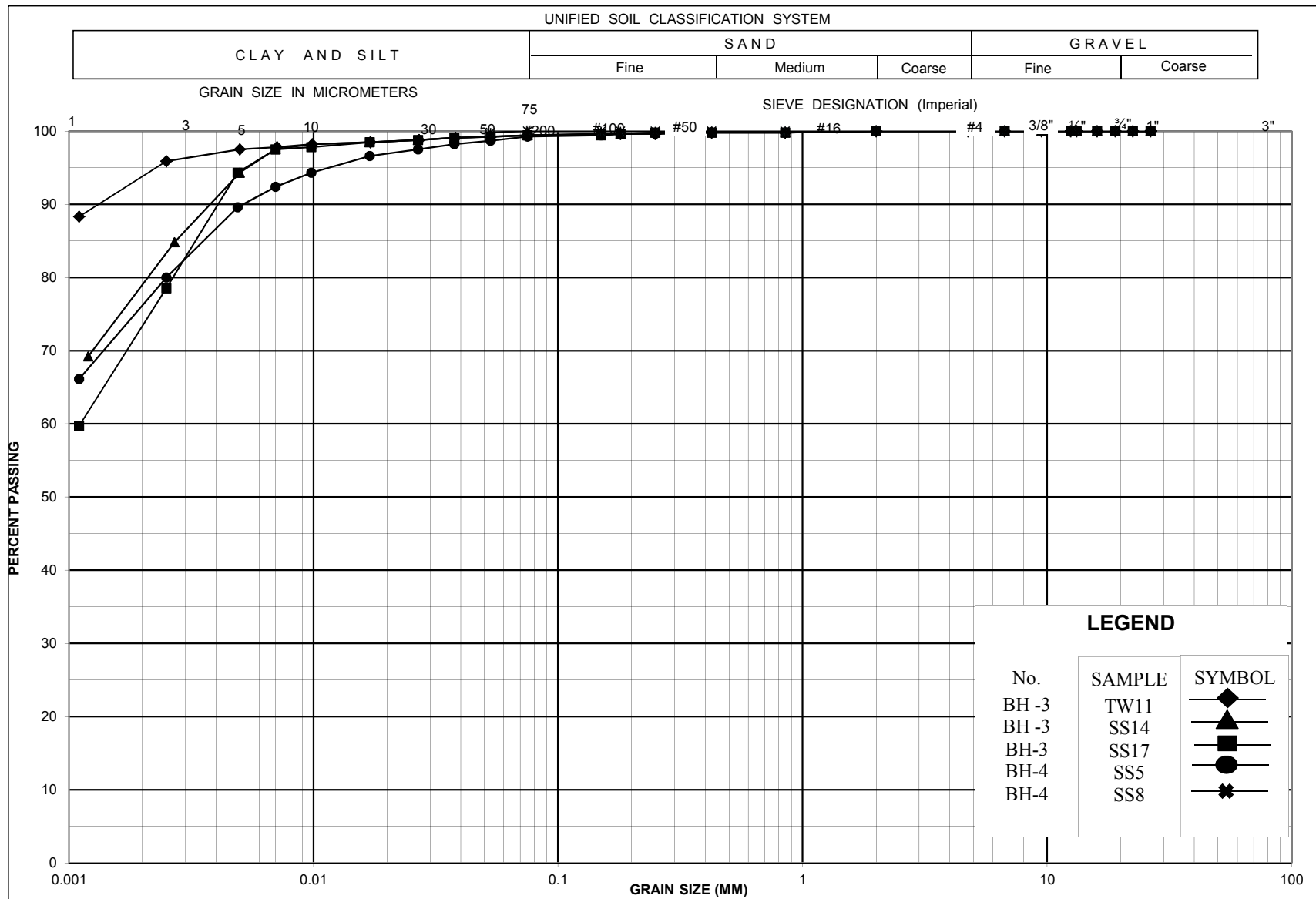
FIGURE 3

WO-2014-11039

DATE January 2015







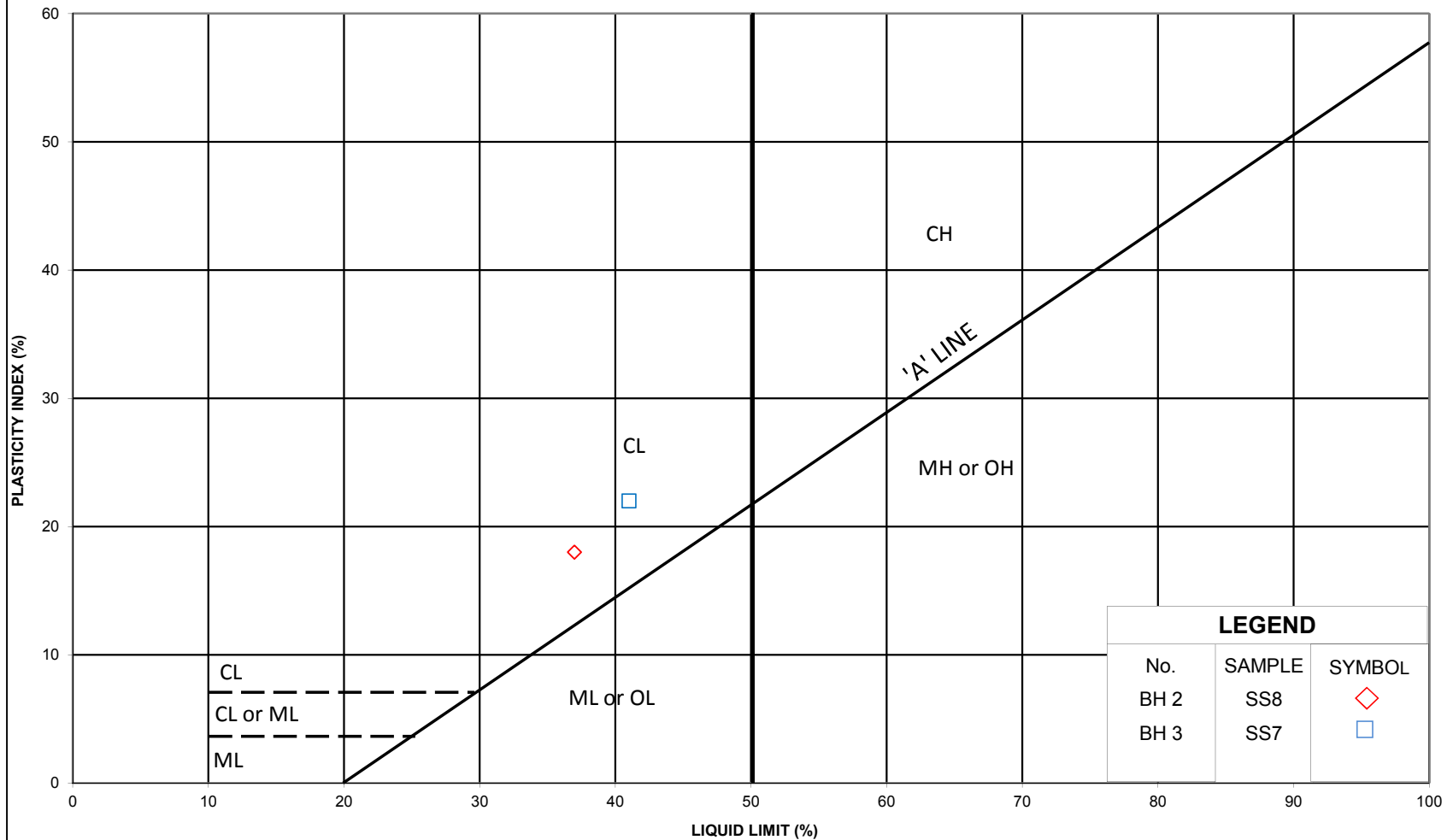
GRAIN SIZE DISTRIBUTION
SILTY CLAY TO CLAY

FIGURE 5b

WO-2014-11039

DATE January 2015

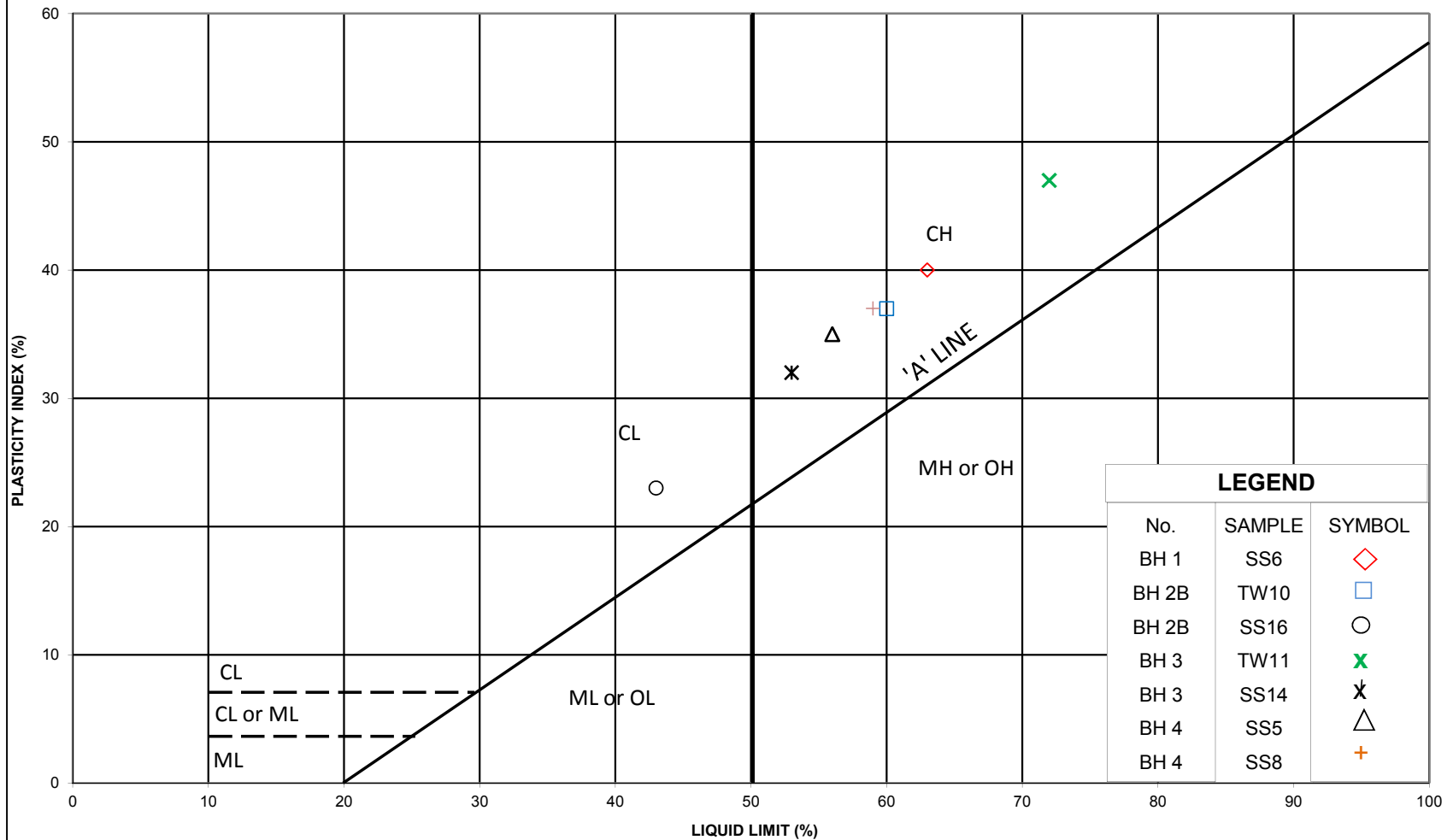
**Deep Fill Culvert Replacement , Hwy 11,
Township of Bowman/Carr**



PLASTICITY CHART
FILL: CLAYEY SILT (ML)

FIGURE 6
WO 2014-11039
DATE January 2015

**Deep Fill Culvert Replacement , Hwy 11,
Township of Bowman/Carr**



**PLASTICITY CHART
SILTY CLAY TO CLAY (CL-CH)**

FIGURE 7

WO 2014-11039

DATE January 2015

FIGURE 8

Corrected Undrained Shear Strength (kPa)

Elevation (mASL)

Legend:

- ◇ BH-1
- ▲ BH-2B
- BH-3
- BH-4

Borehole	Elevation (mASL)	Corrected Undrained Shear Strength (kPa)
BH-1	241.0	20.5
BH-1	244.0	19.5
BH-1	245.5	21.5
BH-1	247.0	24.5
BH-2B	242.5	22.5
BH-2B	244.0	23.5
BH-2B	245.0	22.5
BH-2B	247.0	22.5
BH-3	238.0	38.0
BH-3	241.0	33.0
BH-3	245.5	16.5
BH-3	247.0	19.5
BH-4	242.0	12.0
BH-4	243.5	13.0
BH-4	245.0	23.0
BH-4	246.5	29.5
BH-4	248.5	10.5



Prepared By :NT
Checked By :SM

Appendix E – Slope Stability Analyses Results

January 27, 2015

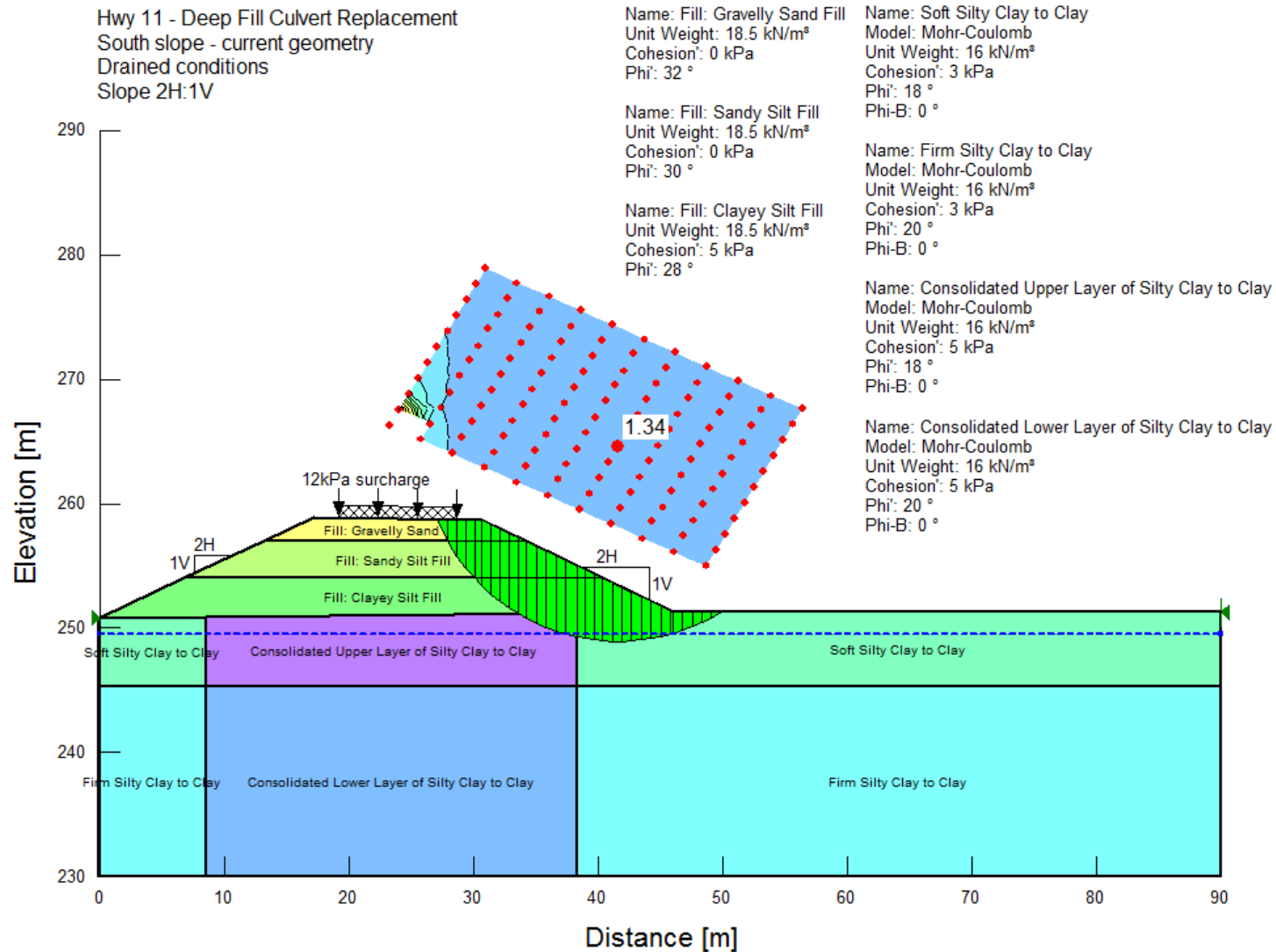


Figure E1. South side of embankment with ~ 2H:1V slope-current geometry, effective stress analyses

January 27, 2015

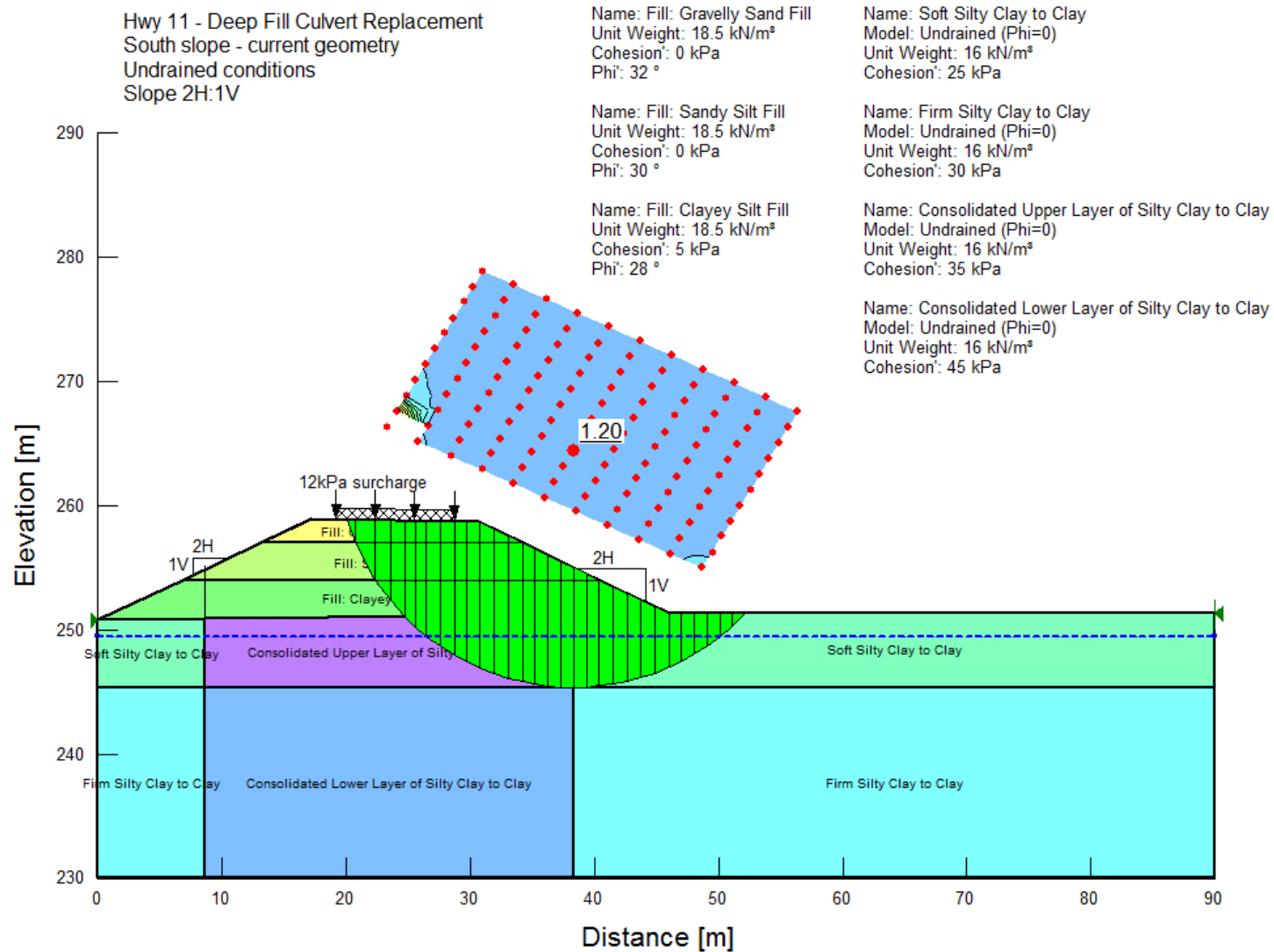


Figure E2. South side of embankment with ~ 2H:1V slope-current geometry, total stress analyses

January 27, 2015

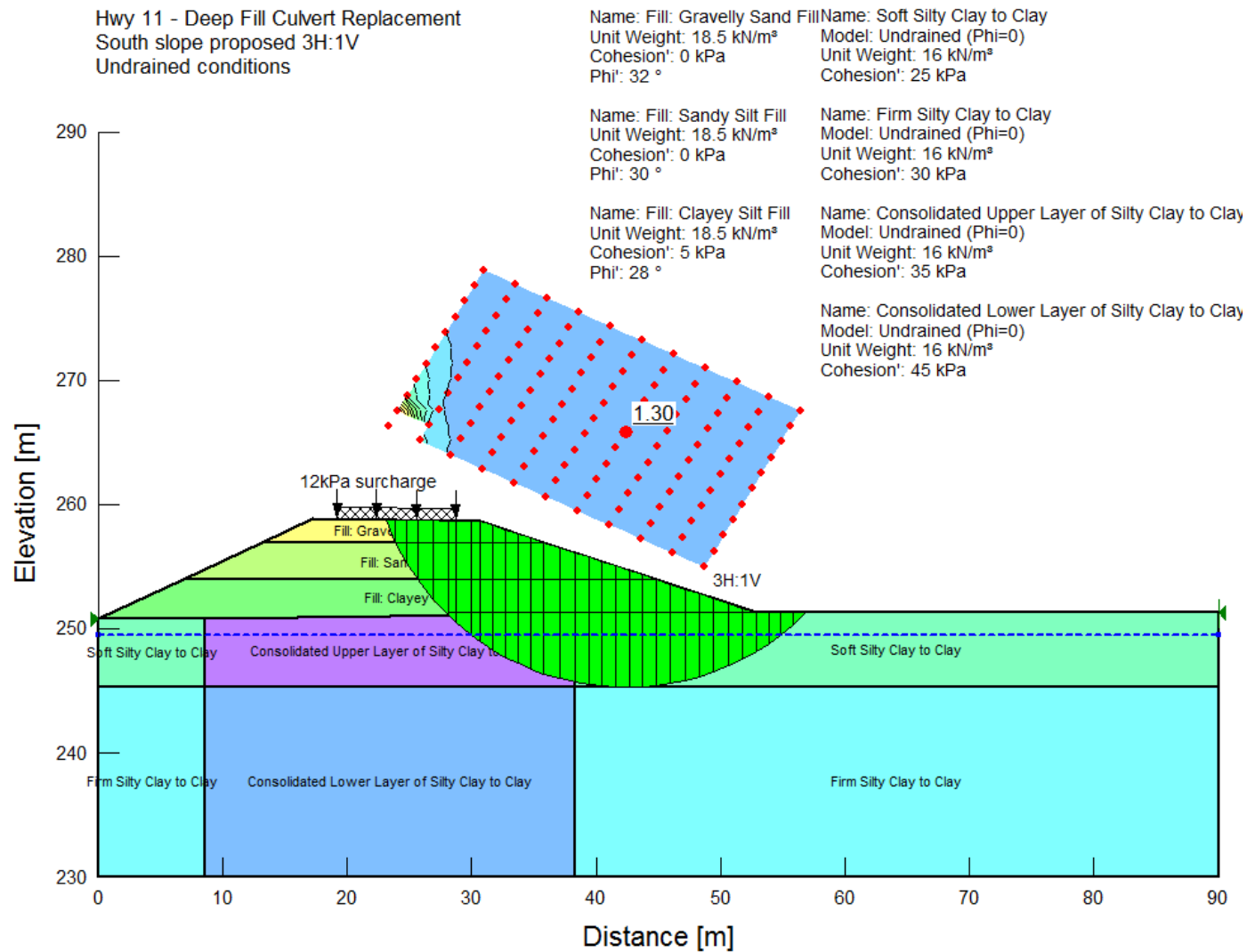


Figure E3. South side of embankment with proposed 3H:1V slope-total stress analyses

January 27, 2015

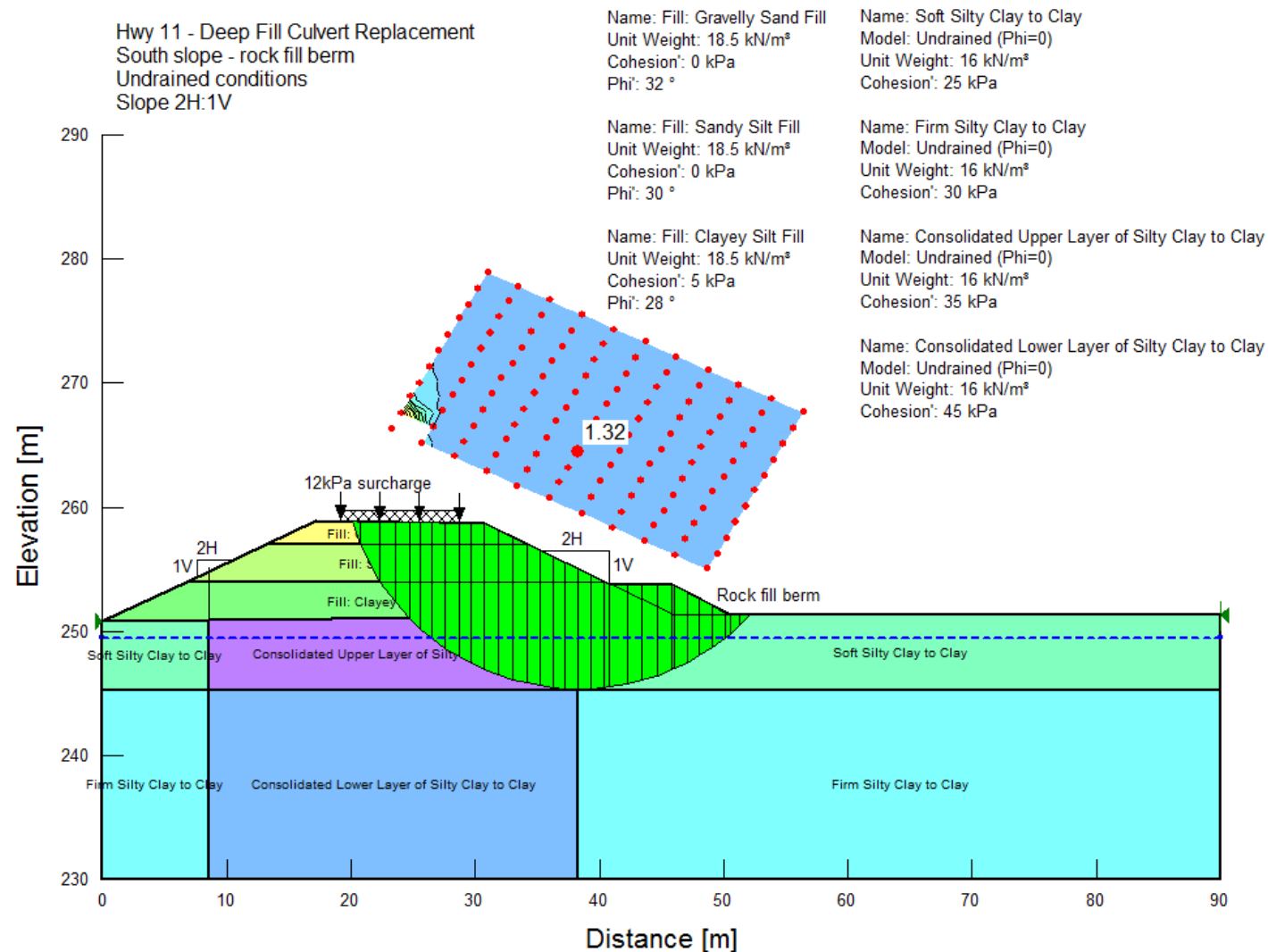
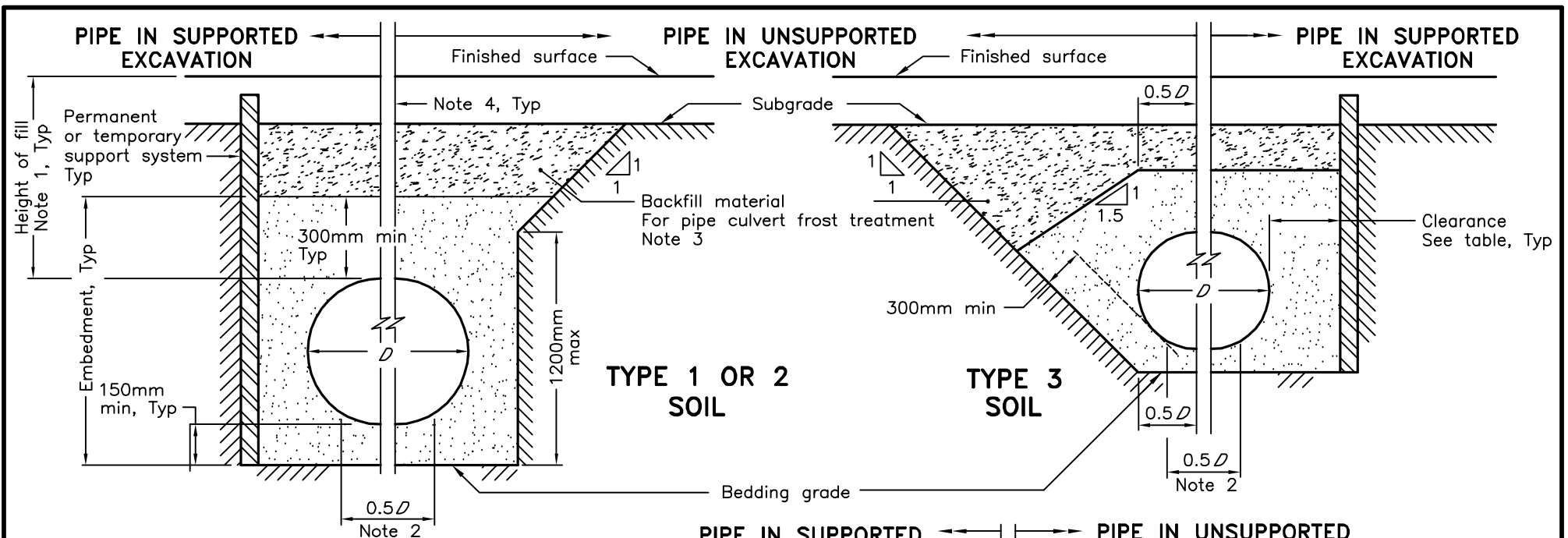


Figure E2. South side of embankment with proposed 2H:1V slope and rock fill berm at the toe-total stress analyses

Appendix F – OPSDs

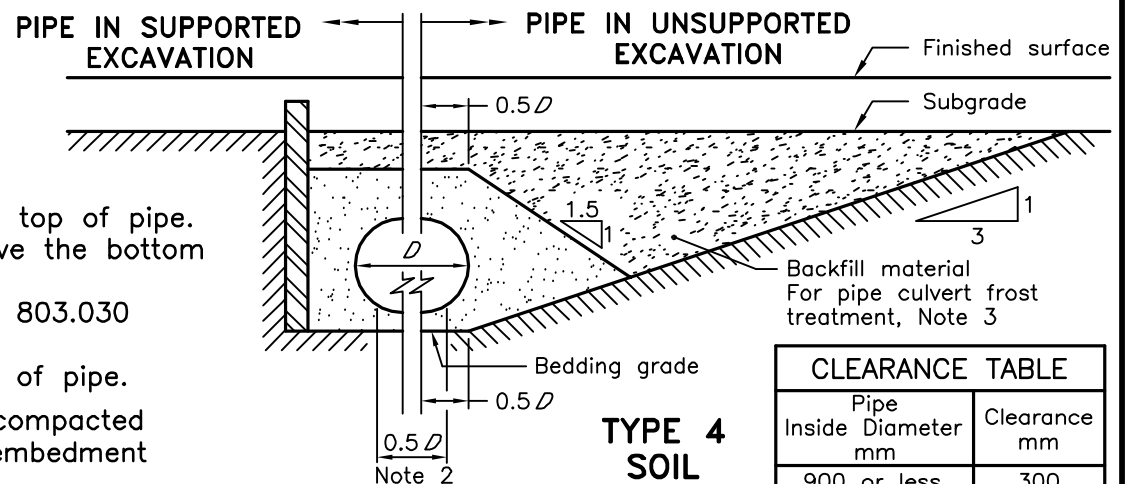


LEGEND:

D - Inside diameter

NOTES:

- 1 Height of fill is measured from the finished surface to top of pipe.
 - 2 The pipe bed shall be compacted and shaped to receive the bottom of the pipe.
 - 3 Pipe culvert frost treatment shall be according to OPSD 803.030 and 803.031.
 - 4 Condition of excavation is symmetrical about centreline of pipe.
- A Granular material placed in the haunch area shall be compacted prior to placing and compacting the remainder of the embedment material.
- B Soil types as defined in the Occupational Health and Safety Act and Regulations for Construction Projects.
- C All dimensions are in metres unless otherwise shown.



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

ONTARIO PROVINCIAL STANDARD DRAWING

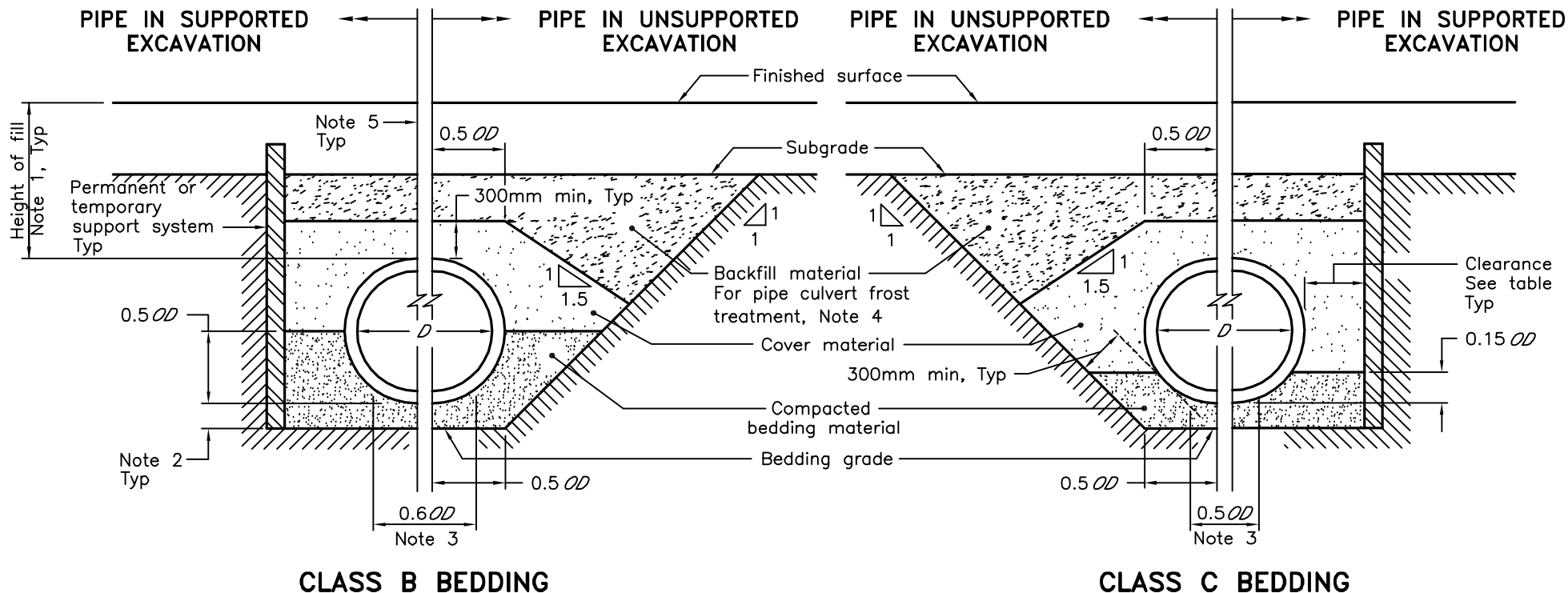
Nov 2010

Rev 2

FLEXIBLE PIPE
EMBEDMENT AND BACKFILL
EARTH EXCAVATION



OPSD 802.010



NOTES:

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

LEGEND:

D – Inside diameter
 OD – Outside diameter

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

ONTARIO PROVINCIAL STANDARD DRAWING

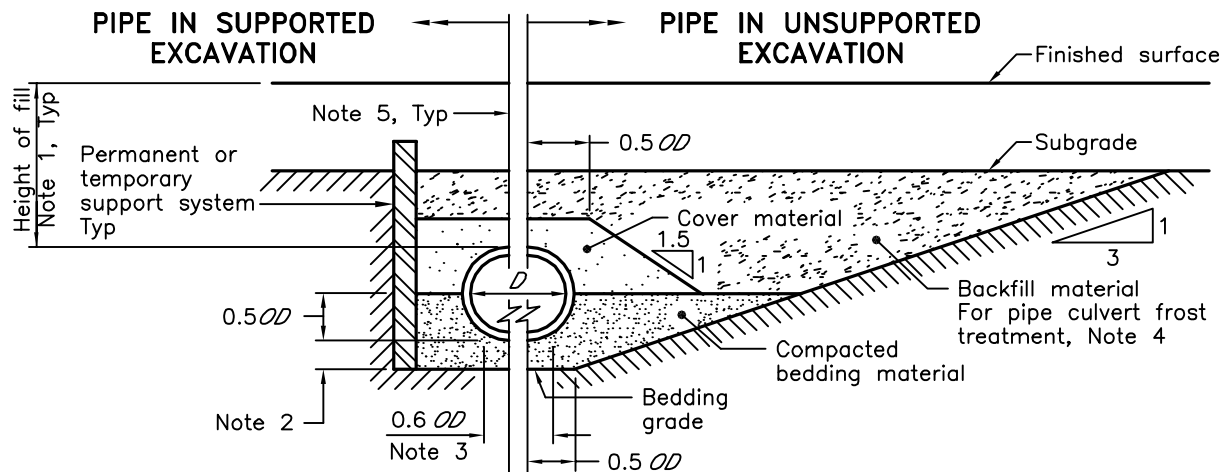
Nov 2010

Rev 2

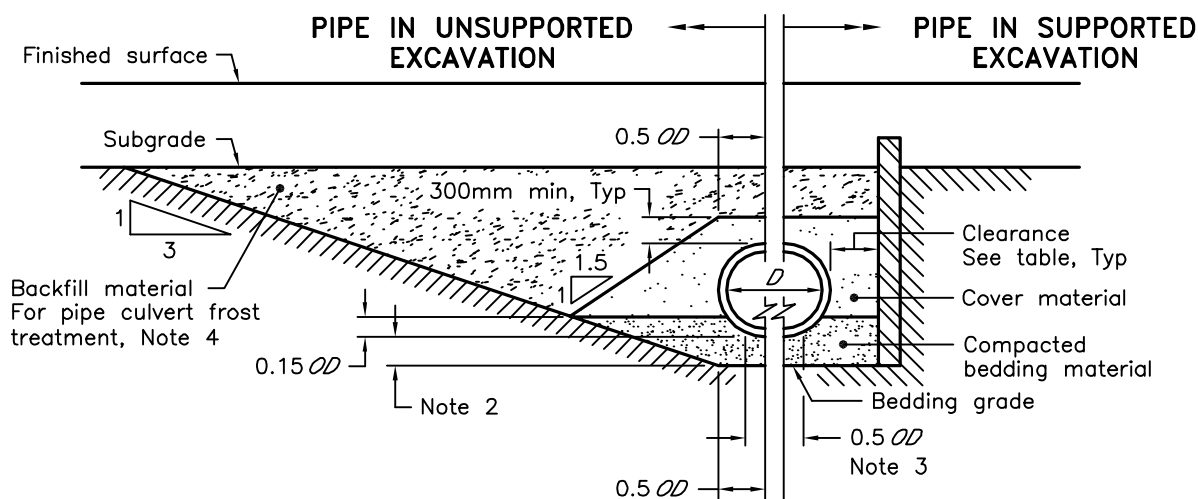
**RIGID PIPE BEDDING,
COVER, AND BACKFILL
TYPE 3 SOIL – EARTH EXCAVATION**

OPSD 802.031





CLASS B BEDDING



CLASS C BEDDING

LEGEND:

D – Inside diameter
 OD – Outside diameter

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

NOTES:

- Height of fill is measured from the finished surface to top of pipe.
 - The minimum bedding depth below the pipe shall be $0.15D$.
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

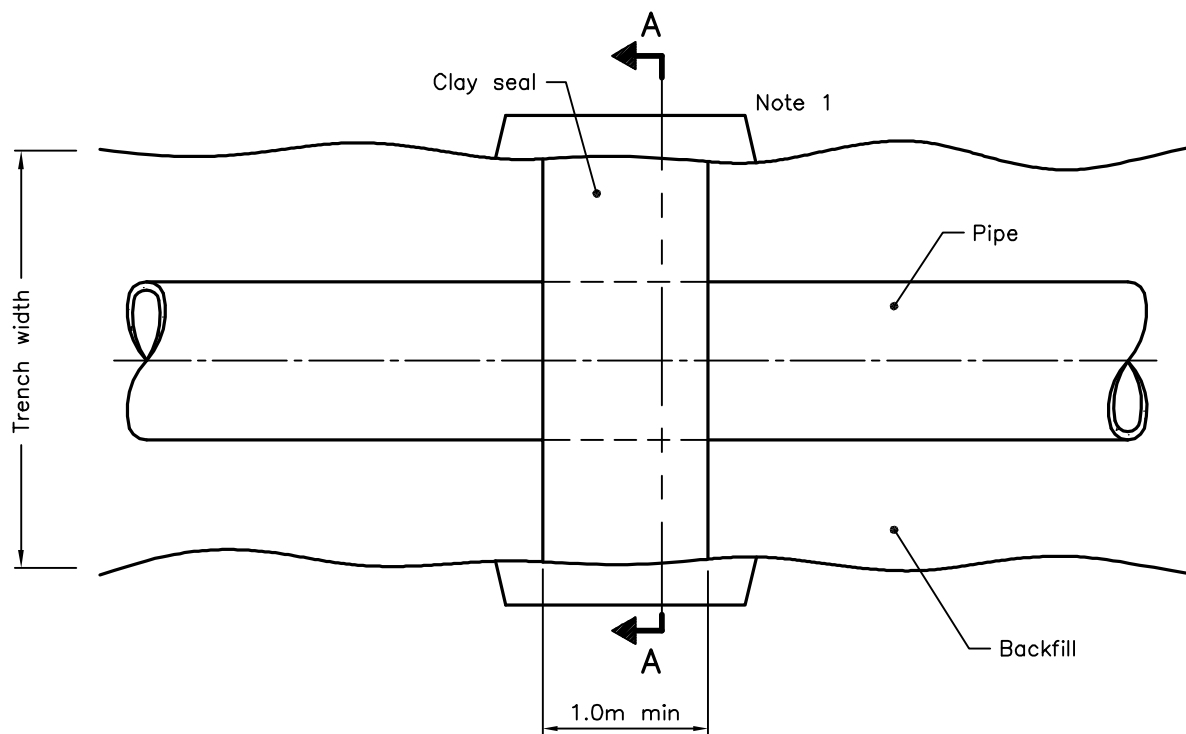
Nov 2010

Rev 2

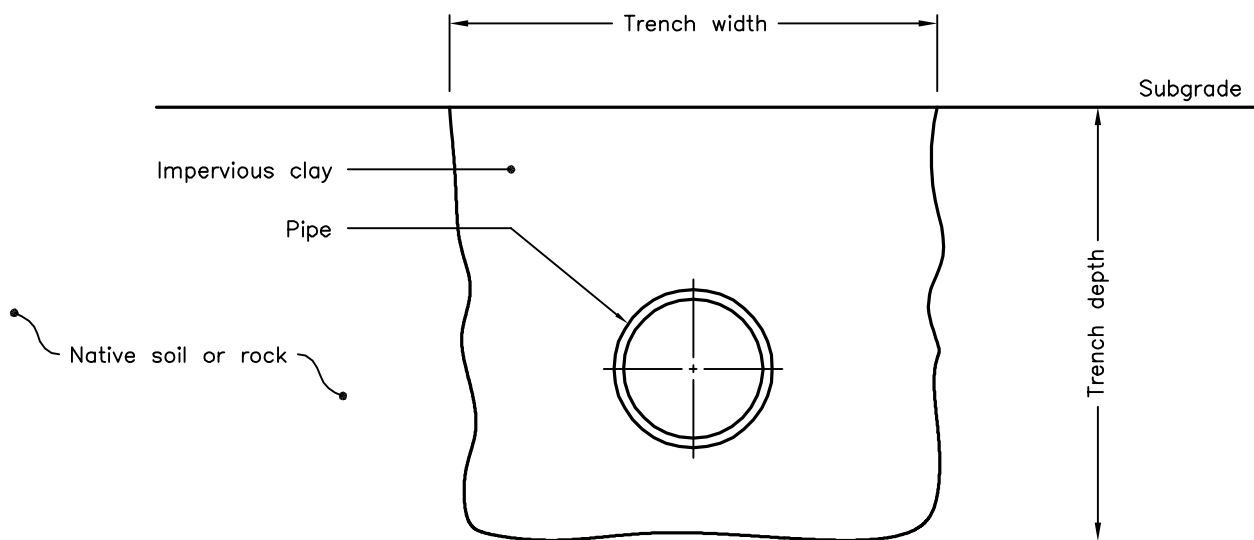
RIGID PIPE BEDDING,
 COVER, AND BACKFILL
 TYPE 4 SOIL – EARTH EXCAVATION

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.

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CLAY SEAL FOR PIPE TRENCHES

OPSD 802.095



Pipe Type	Corrugation Profile	Diameter	Min. Height of Fill	Area m ²	Metal Thickness									
					Corrugated Steel Pipe					Structural Plate				
					1.6	2.0	2.8	3.5	4.2	3.0	4.0	5.0	6.0	7.0
					Maximum Height of Fill Over Pipe m									
Round Corrugated Steel Pipe (Notes 1 and 2)	68 x 13mm	300	300	0.07	61.3	79.7								
		400	300	0.13	45.9	59.7								
		450	300	0.16	40.8	53.1								
		500	300	0.20	36.8	47.8	69.3							
		600	300	0.28	30.6	39.8	57.8							
		700	300	0.38	26.2	34.1	49.5							
		750	300	0.44	24.5	31.9	46.5							
		800	300	0.50	23.0	29.9	43.3							
		900	300	0.64	20.4	26.5	38.5	48.9						
		1000	300	0.79	18.4	23.9	34.7	44.0	53.6					
		1200	300	1.13		19.9	28.9	36.7	44.7					
		1400	300	1.54			23.8	30.3	37.0					
		1500	300	1.77			21.3	27.2	33.2					
		1600	300	2.01			19.1	24.4	29.8					
		1800	300	2.54				19.5	23.9					
	2000	350	3.14					18.9						
	125 x 25mm	1200	300	1.13			20.4	29.6	37.6	45.8				
		1400	300	1.54			17.5	25.4	32.2	39.2				
		1500	300	1.77			16.3	23.7	30.1	36.6				
		1600	300	2.01			15.3	22.2	28.2	34.3				
		1800	300	2.54			13.6	19.7	25.0	30.5				
		2000	350	3.14			12.2	17.7	22.5	27.5				
		2200	400	3.80			11.1	16.1	20.5	25.0				
		2400	400	4.52			10.0	14.8	18.8	22.9				
		2700	450	5.73				13.1	16.7	20.3				
		3000	500	7.07				11.3	14.4	17.5				
Round Structural Plate Corrugated Steel Pipe (Note 3)	152 x 51mm	1500	300	1.77						26.2	39.1	49.8	60.5	70.6
		1660	300	2.16						23.7	35.3	45.0	54.6	63.8
		1810	350	2.58						21.7	32.4	41.3	50.1	58.5
		1970	350	3.04						19.9	29.8	37.9	46.0	53.8
		2120	400	3.54						18.5	27.7	35.3	42.8	50.0
		2280	400	4.07						17.2	25.7	32.8	39.8	46.4
		2430	450	4.65						16.1	24.1	30.8	37.3	43.6
		2590	450	5.26						15.1	22.6	28.8	35.0	40.9
		2740	500	5.91						14.3	21.4	27.3	33.1	38.6

NOTES:

- 1 Fill heights apply to galvanized, aluminized Type 2, or polymer coated round corrugated steel pipe.
 - 2 Refer to manufacturer for custom sizes. For custom sizes use next larger standard diameter for estimating maximum height of fill over pipe.
 - 3 Fill heights apply to galvanized round structural plate corrugated steel pipe.
- A Fill height values are based on the American Iron and Steel Institute design method.
B This OPSD shall be read in conjunction with OPSD 802.010, 802.013, and 802.014.
C All dimensions are in millimetres unless otherwise shown.

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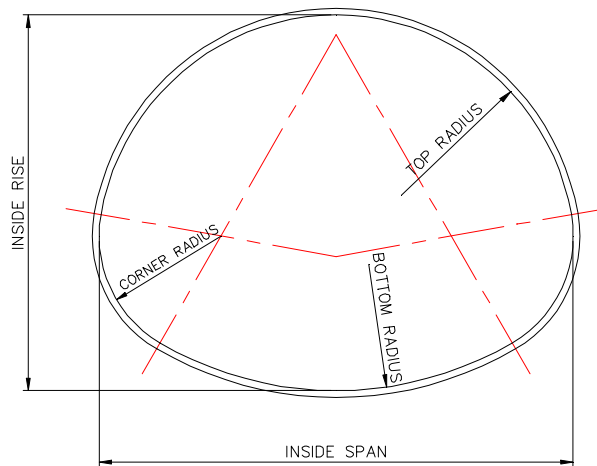
Rev 1



HEIGHT OF FILL TABLE
ROUND CORRUGATED STEEL PIPE AND
STRUCTURAL PLATE CORRUGATED STEEL PIPE

OPSD 805.010

Corrugation Profile	Inside Span	Inside Rise	Area m ²	Equivalent Round	Min Height of Fill	Metal Thickness									
						Corrugated Steel Pipe					Structural Plate				
						1.6	2.0	2.8	3.5	4.2	3.0	4.0	5.0	6.0	7.0
						Max Height of Fill Over Pipe m Corner Bearing Pressure Limited to 200 kPa (Note 3)									
68 x 13 mm (Note 1)	560	420	0.19	500	300	4.1									
	680	500	0.27	600	300	4.2									
	800	580	0.37	700	300	4.1									
	910	660	0.48	800	300	4.1									
	1030	740	0.61	900	300	4.0									
	1150	820	0.74	1000	300		4.0								
	1390	970	1.06	1200	300		3.9								
	1630	1120	1.44	1400	300			3.9							
	1880	1260	1.87	1600	350				3.8						
	2130	1400	2.36	1800	400				3.7						
125 x 25mm (Note 1)	1330	1030	1.09	1200	300			4.4							
	1550	1200	1.48	1400	300			4.4							
	1780	1360	1.93	1600	300			4.4							
	2010	1530	2.44	1800	350			4.3							
	2230	1700	2.97	2000	400			4.6							
	2500	1830	3.44	2200	450			4.5							
	2800	1950	4.27	2400	500			4.4							
	3300	2080	5.39	2700	550			4.3							
	3650	2280	6.60	3000	650			4.2							
	3890	2690	8.29	3300	650				3.5						
152 x 51mm (Note 2)	4370	2870	9.76	3600	750					3.0					
	2060	1520	2.49		350							5.1			
	2240	1630	2.90		400							4.8			
	2440	1750	3.36		450							4.6			
	2590	1880	3.87		450							4.6			
	2690	2080	4.49		450							5.0			
	3100	1980	4.83		550							3.4			
	3400	2010	5.28		600							2.7			
	3730	2290	6.61		650							2.9			
	3890	2690	8.29		650							3.5			
	4370	2870	9.76		750							3.0			



NOTES:

- 1 Fill heights apply to galvanized, aluminized Type 2, or polymer coated corrugated steel pipe arch.
 - 2 Fill heights apply to galvanized structural plate corrugated steel pipe arch.
 - 3 For heights of fill approaching the above maximums for 200kPa corner bearing pressure, consideration should be given to round pipes since they reduce the load being applied to the foundation.
- A Fill height values are based on American Iron and Steel Institute design method.
 B This OPSD shall be read in conjunction with OPSD 802.020, 802.023, and 802.024.
 C All dimensions are in millimetres unless otherwise shown.

ONTARIO PROVINCIAL STANDARD DRAWING

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HEIGHT OF FILL TABLE
CORRUGATED STEEL PIPE-ARCH AND
STRUCTURAL PLATE CORRUGATED STEEL PIPE-ARCH

OPSD 805.020



PIPE SIZE mm	BEDDING CLASS	CONFINED TRENCH			
		50-D	65-D	100-D	140-D
		MAXIMUM HEIGHT OF FILL			
300	A	5.1	11.8	NL	NL
	B	2.9	3.9	7.4	NL
	C	2.2	3.1	5.1	10.2
375	A	6.5	NL	NL	NL
	B	3.0	4.1	8.8	NL
	C	2.4	3.2	5.7	12.8
450	A	7.6	NL	NL	NL
	B	3.1	4.4	10.0	NL
	C	2.5	3.4	6.2	13.2
525	A	8.5	NL	NL	NL
	B	3.3	4.7	10.9	NL
	C	2.6	3.5	6.6	17.6
600	A	9.2	NL	NL	NL
	B	3.4	4.9	11.5	NL
	C	2.6	3.7	7.0	18.6
675	A	9.7	NL	NL	NL
	B	3.5	5.1	11.8	NL
	C	2.7	3.8	7.2	18.6
750	A	10.0	NL	NL	NL
	B	3.6	5.2	12.0	NL
	C	2.8	3.8	7.2	18.0
825	A	10.2	NL	NL	NL
	B	3.6	5.2	12.0	NL
	C	2.8	3.8	7.4	17.4
900	A	9.8	NL	NL	NL
	B	3.8	5.4	11.6	NL
	C	2.8	4.0	7.4	16.0
975	A	6.2	9.6	NL	NL
	B	3.4	4.6	8.0	14.4
	C	2.8	3.8	6.0	9.6
1050	A	6.4	9.8	NL	NL
	B	3.6	4.6	8.2	14.6
	C	2.8	3.8	6.2	9.8
1200	A	6.8	10.4	NL	NL
	B	3.6	4.8	8.4	15.0
	C	3.0	3.8	6.4	10.2
1350	A	7.0	10.8	NL	NL
	B	3.8	5.0	8.6	15.2
	C	3.2	4.0	6.4	10.4
1500	A	7.2	11.0	NL	NL
	B	3.8	5.0	8.8	15.2
	C	3.2	4.0	6.6	10.4
1650	A	7.4	11.2	NL	NL
	B	4.0	5.2	9.0	15.2
	C	3.4	4.2	6.8	10.6
1800	A	7.6	11.4	NL	NL
	B	4.0	5.2	9.0	15.2
	C	3.4	4.2	6.8	10.6

LEGEND:

NL = No Limit

NOTES:

A Height of fill is measured from the finished surface to top of pipe.

B The load factor used for the calculation of the variable bedding factor is: Class A bedding=2.8, Class B bedding=1.9, and Class C bedding=1.5.

C The excavation width is based on OPSD 802.030 and is the width of the excavation measured at the top of the pipe.

D Minimum height of fill over top of pipe shall be 600mm.

E Strength designations are 50-D, 65-D, 100-D, and 140-D according to CSA A257.2.

F The table is based on backfill density of 1900 kg/m³.

G For confined trench $K_u=0.130$.

H Conditions other than those indicated should be calculated from first principles.

I All dimensions are in metres unless otherwise shown.

ONTARIO PROVINCIAL STANDARD DRAWING

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HEIGHT OF FILL TABLE

REINFORCED CONCRETE PIPE – CONFINED TRENCH
CLASS 50-D, 65-D, 100-D, and 140-D

OPSD 807.010

PIPE SIZE mm	BEDDING CLASS	POSITIVE PROJECTING EMBANKMENT			
		50-D	65-D	100-D	140-D
		MAXIMUM HEIGHT OF FILL			
300	A	4.5	6.0	9.0	12.9
	B	2.4	3.6	5.7	7.8
	C	1.8	2.7	4.5	6.3
375	A	4.8	6.0	9.6	13.5
	B	2.7	3.6	5.7	8.1
	C	2.1	3.0	4.8	6.6
450	A	4.8	6.3	9.9	13.8
	B	2.7	3.9	6.0	8.4
	C	2.1	3.0	4.8	6.9
525	A	4.8	6.6	9.9	14.1
	B	3.0	3.9	6.0	8.7
	C	2.4	3.0	5.1	7.2
600	A	5.1	6.6	10.2	14.4
	B	3.0	3.9	6.3	8.7
	C	2.4	3.3	5.1	7.2
675	A	5.1	6.6	10.2	14.4
	B	3.0	3.9	6.3	9.0
	C	2.4	3.3	5.1	7.2
750	A	5.1	6.6	10.5	14.7
	B	3.0	4.2	6.3	9.0
	C	2.4	3.3	5.1	7.5
825	A	5.1	6.9	10.5	14.7
	B	3.0	4.2	6.6	9.0
	C	2.4	3.3	5.4	7.5
900	A	5.1	6.9	10.5	15.0
	B	3.3	4.2	6.6	9.3
	C	2.4	3.3	5.4	7.5
975	A	5.4	6.9	10.8	15.0
	B	3.3	4.2	6.6	9.3
	C	2.4	3.3	5.4	7.5
1050	A	5.4	6.9	10.8	15.0
	B	3.3	4.2	6.6	9.3
	C	2.7	3.3	5.4	7.5
1200	A	5.4	7.2	11.1	15.6
	B	3.3	4.5	6.9	9.6
	C	2.7	3.6	5.7	7.8
1350	A	5.4	7.2	11.1	15.6
	B	3.3	4.5	6.9	9.6
	C	2.7	3.6	5.7	8.1
1500	A	5.7	7.2	11.1	15.6
	B	3.3	4.5	6.9	9.9
	C	3.0	3.6	5.7	8.1
1650	A	5.7	7.2	11.4	15.9
	B	3.6	4.5	6.9	9.9
	C	3.0	3.6	5.7	8.1
1800	A	5.7	7.2	11.4	15.9
	B	3.6	4.5	7.2	9.9
	C	3.0	3.9	5.7	8.1

NOTES:

- A Height of fill is measured from the finished surface to top of pipe.
 B For the positive projecting embankment, the load factor used for the calculation of the variable bedding factor is:
 Class A bedding=2.8, Class B bedding=1.9, and Class C bedding=1.5.
 C Minimum height of fill over top of pipe shall be 600mm.
 D Strength designations are 50-D, 65-D, 100-D, and 140-D according to CSA A257.2.
 E The table is based on backfill density of 1900 kg/m³.
 F For positive projecting embankment $K_u=0.190$, $p=0.7$, and $r_{sd}=0.7$.
 G Conditions other than those indicated should be calculated from first principles.
 H All dimensions are in metres unless otherwise shown.

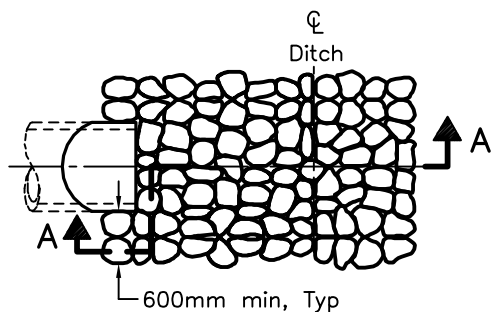
ONTARIO PROVINCIAL STANDARD DRAWING

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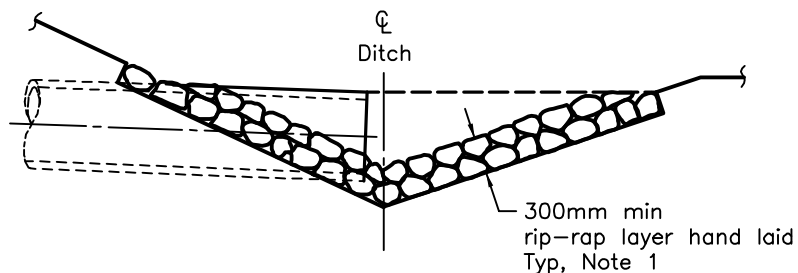
HEIGHT OF FILL TABLE
 REINFORCED CONCRETE PIPE – EMBANKMENT
 CLASS 50-D, 65-D, 100-D, and 140-D

OPSD 807.030

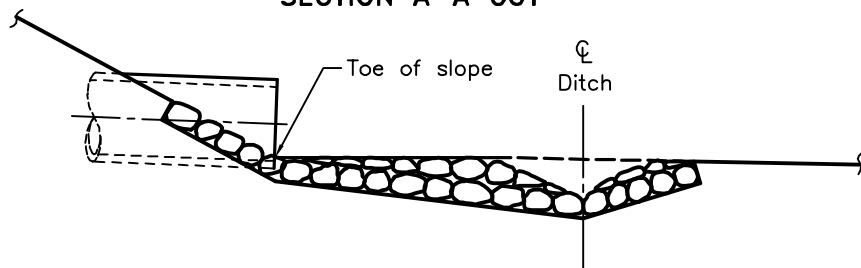




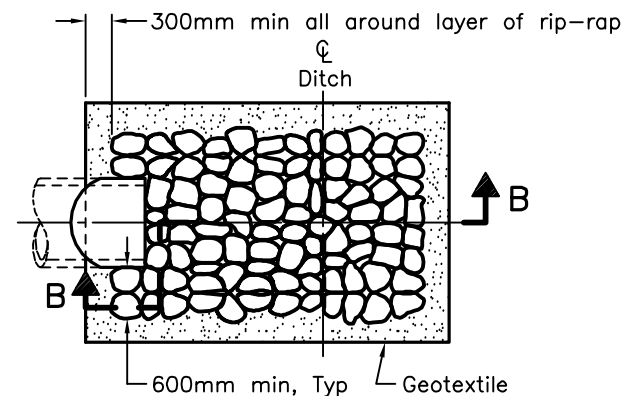
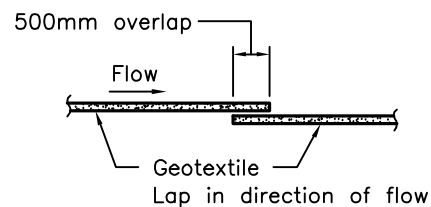
PLAN
CUT OR FILL



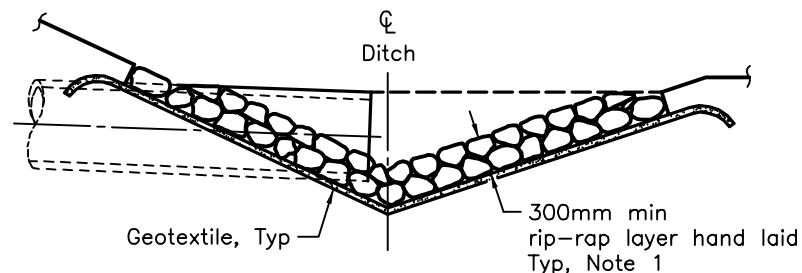
SECTION A-A CUT



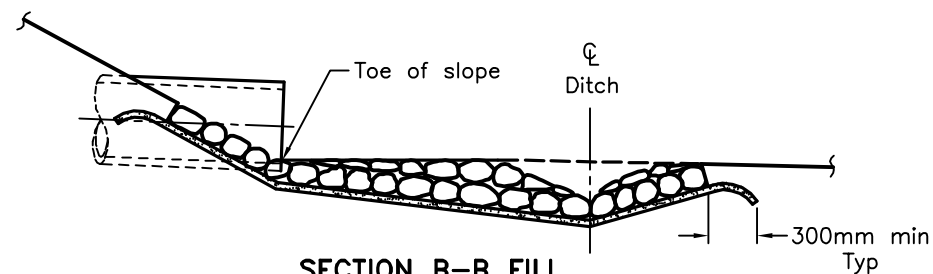
SECTION A-A FILL
TYPE A – WITHOUT GEOTEXTILE



PLAN
CUT OR FILL



SECTION B-B CUT



SECTION B-B FILL
TYPE B – WITH GEOTEXTILE

NOTES:

1 The thickness of the rip-rap layer shall be at least 1.5 times the rip-rap mean diameter.

A All dimensions are in millimetres unless otherwise shown.

ONTARIO PROVINCIAL STANDARD DRAWING

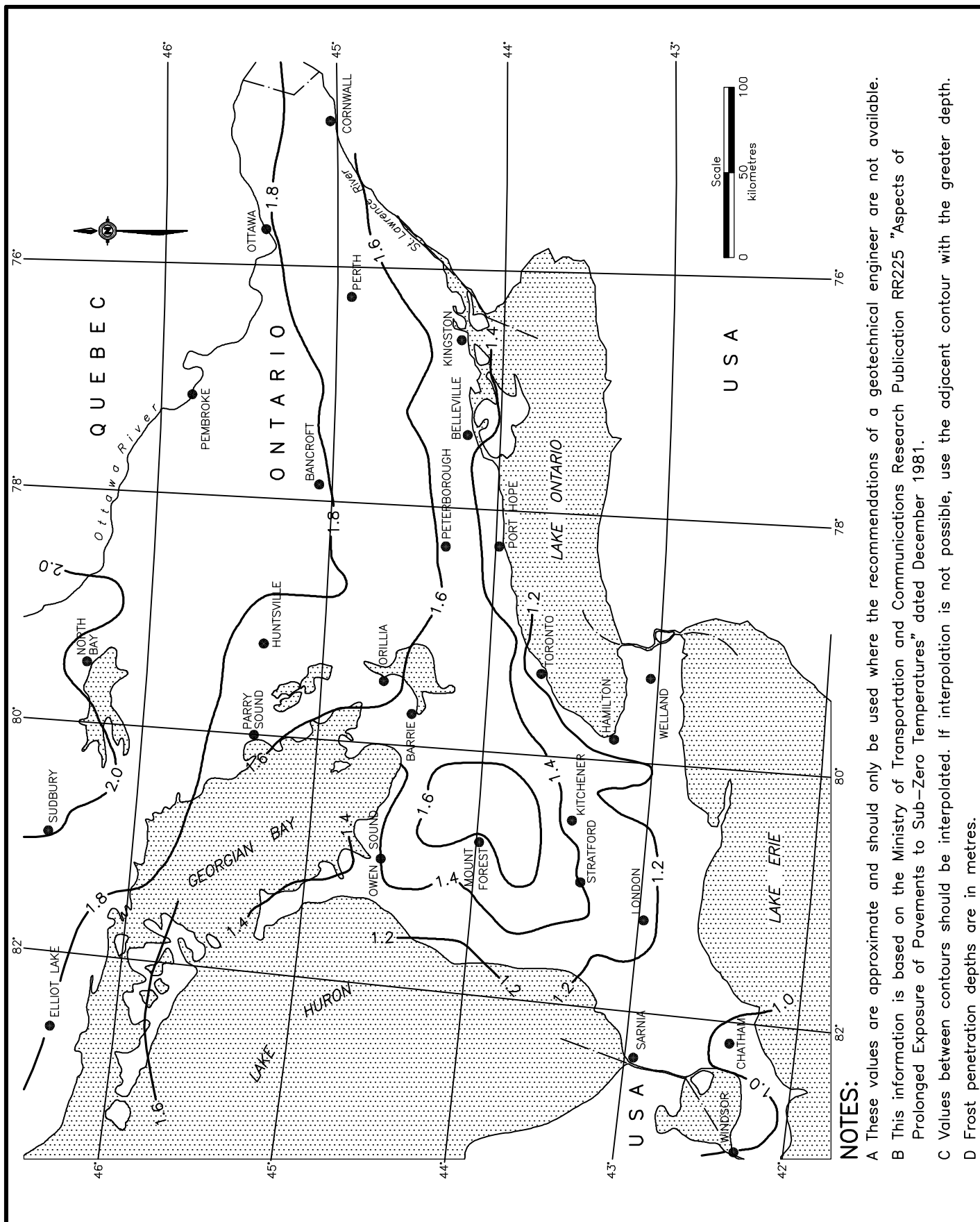
Nov 2013

Rev 2

**GENERAL RIP-RAP LAYOUT
FOR SEWER AND CULVERT OUTLETS**

OPSD 810.010





NOTES:

- A These values are approximate and should only be used where the recommendations of a geotechnical engineer are not available.
- B This information is based on the Ministry of Transportation and Communications Research Publication RR225 "Aspects of Prolonged Exposure of Pavements to Sub-Zero Temperatures" dated December 1981.
- C Values between contours should be interpolated. If interpolation is not possible, use the adjacent contour with the greater depth.
- D Frost penetration depths are in metres.

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**FOUNDATION
FROST PENETRATION DEPTHS
FOR SOUTHERN ONTARIO**



OPSD 3090.101

Appendix G – Operational Constrains and Non Standard Special Provisions

NSSP FOR BOULDER AND COBBLE OBSTRUCTIONS IN FILL

Scope of Work

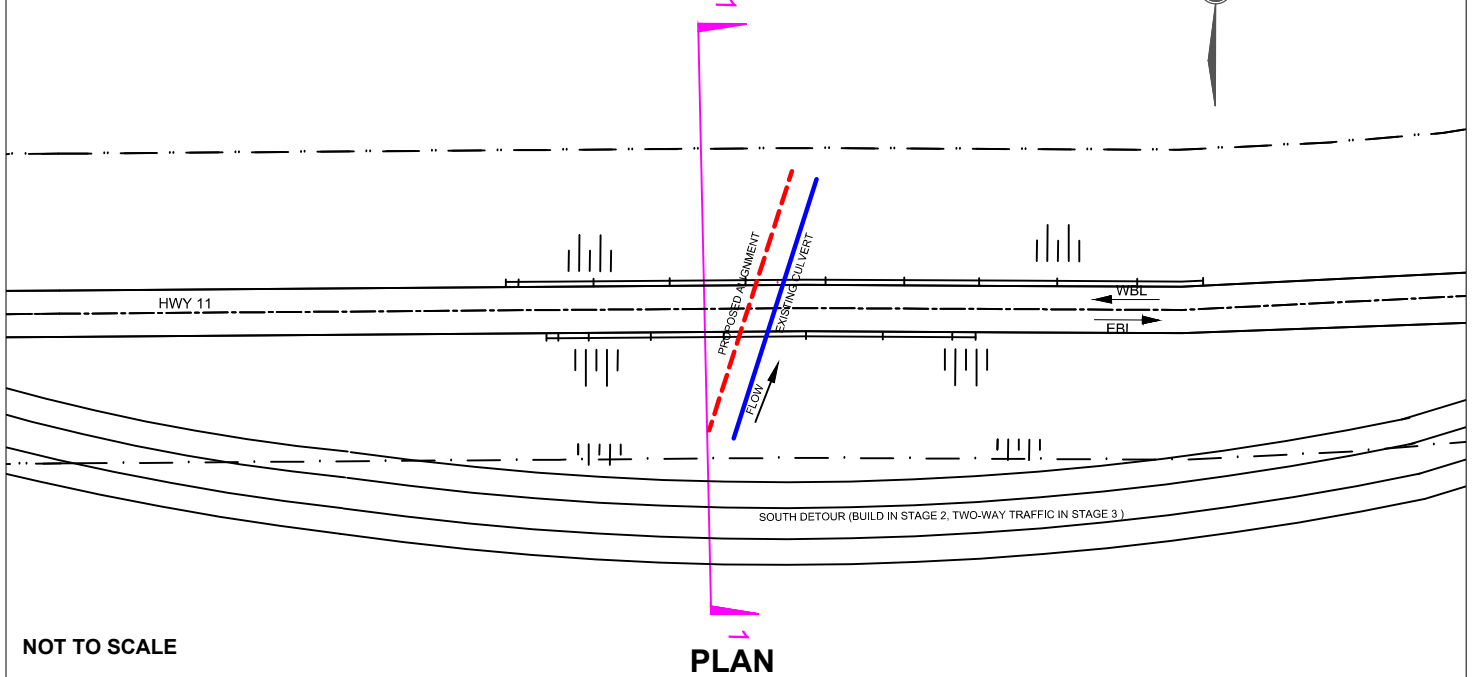
The Contractor should be aware that the overburden soils at the site consist of gravelly sand to sand fill materials which may contain boulders and cobbles. Appropriate equipment and procedures will be required to penetrate/remove boulders and cobbles that are encountered during excavation.

Basis of Payment

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

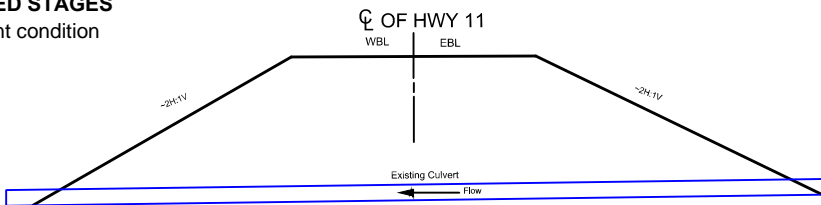
Appendix H – Schematic Sketches for Construction Alternatives

FIGURE H.1: TEMPORARY LOCAL DETOUR AND OPEN CUT UNSUPPORTED EXCAVATION SCHEMATIC DIAGRAMS

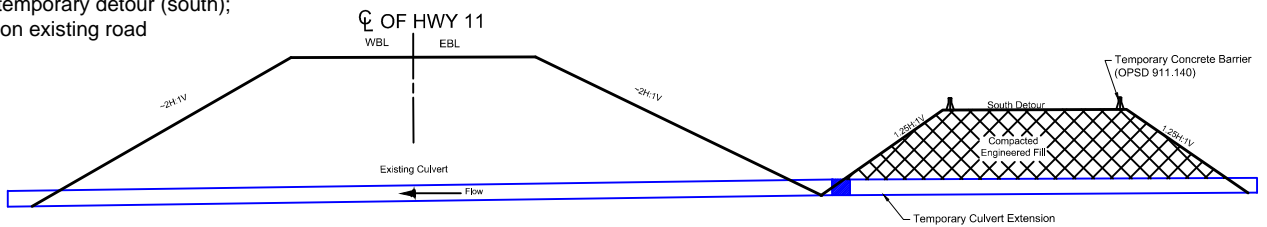


RECOMMENDED STAGES

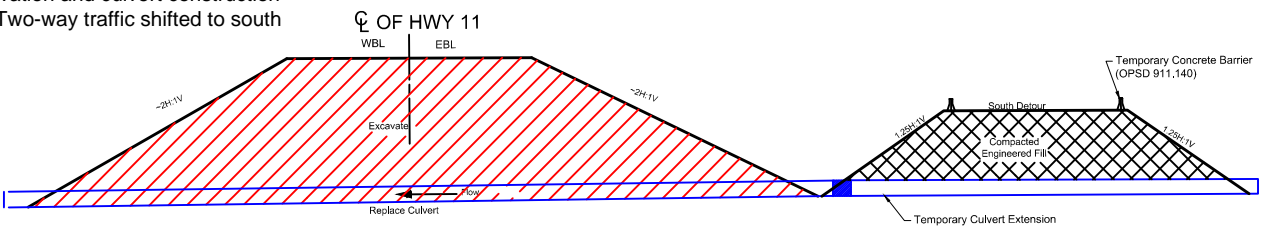
Stage 1 - Current condition



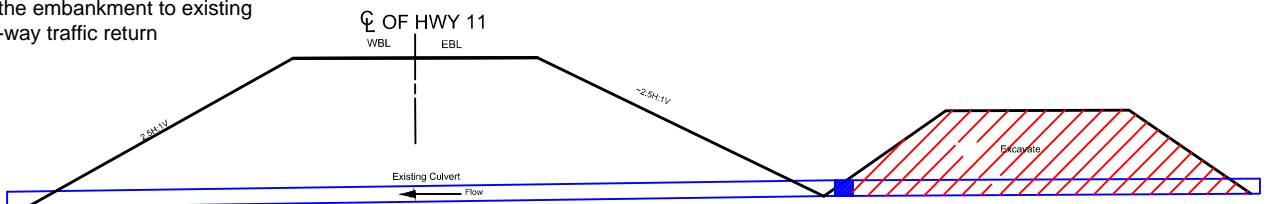
Stage 2 - Build temporary detour (south);
Two-way traffic on existing road



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



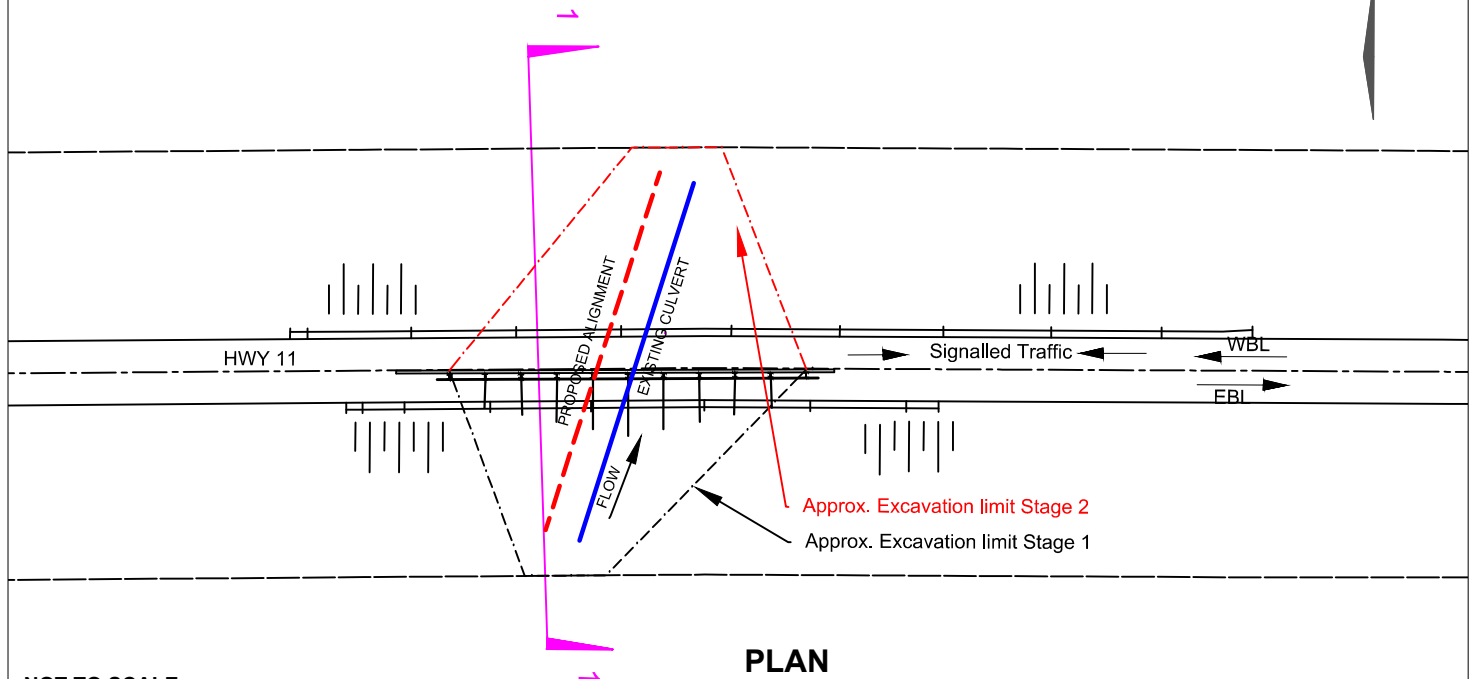
Stage 4 - Build the embankment to existing
alignment; Two-way traffic return



NOT TO SCALE

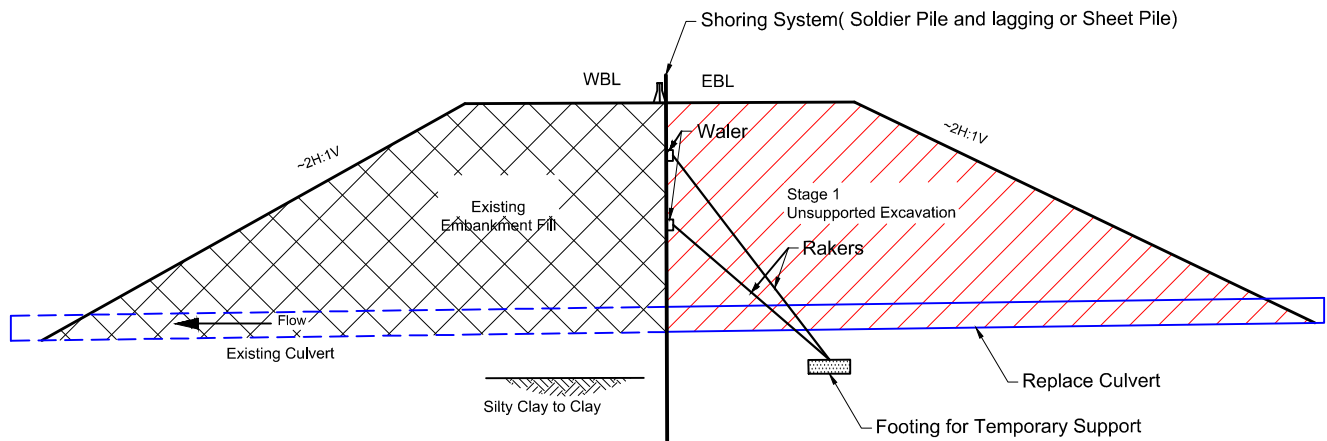
SECTION 1-1

FIGURE H.2: HALF AND HALF CONSTRUCTION USING SHORING SYSTEM WITH UNSUPPORTED CUT SIDES
SCHEMATIC DIAGRAMS



NOT TO SCALE

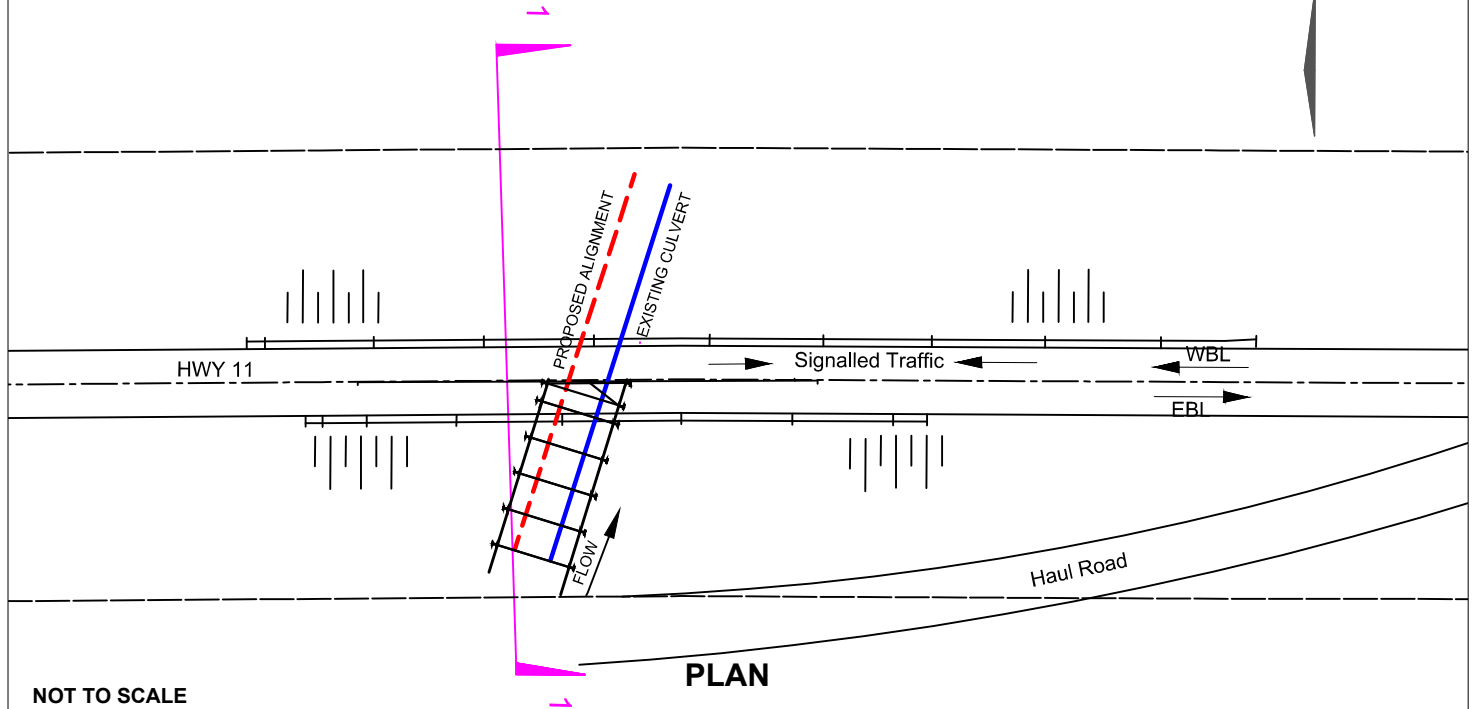
Half and Half Construction, Temporary Shoring - Unsupported Excavation



NOT TO SCALE

SECTION 1-1

**FIGURE H.3: HALF AND HALF CONSTRUCTION USING SHORING SYSTEM
WITH BRACED CUT SIDES
SCHEMATIC DIAGRAMS**



Half and Half Construction, Temporary Shoring - Braced Cut

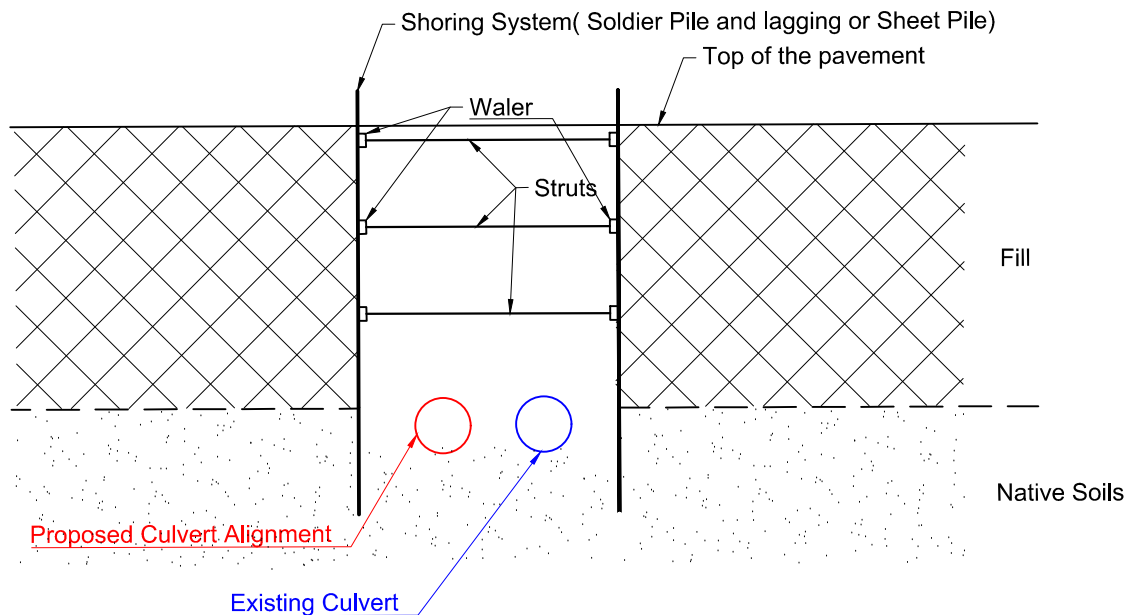
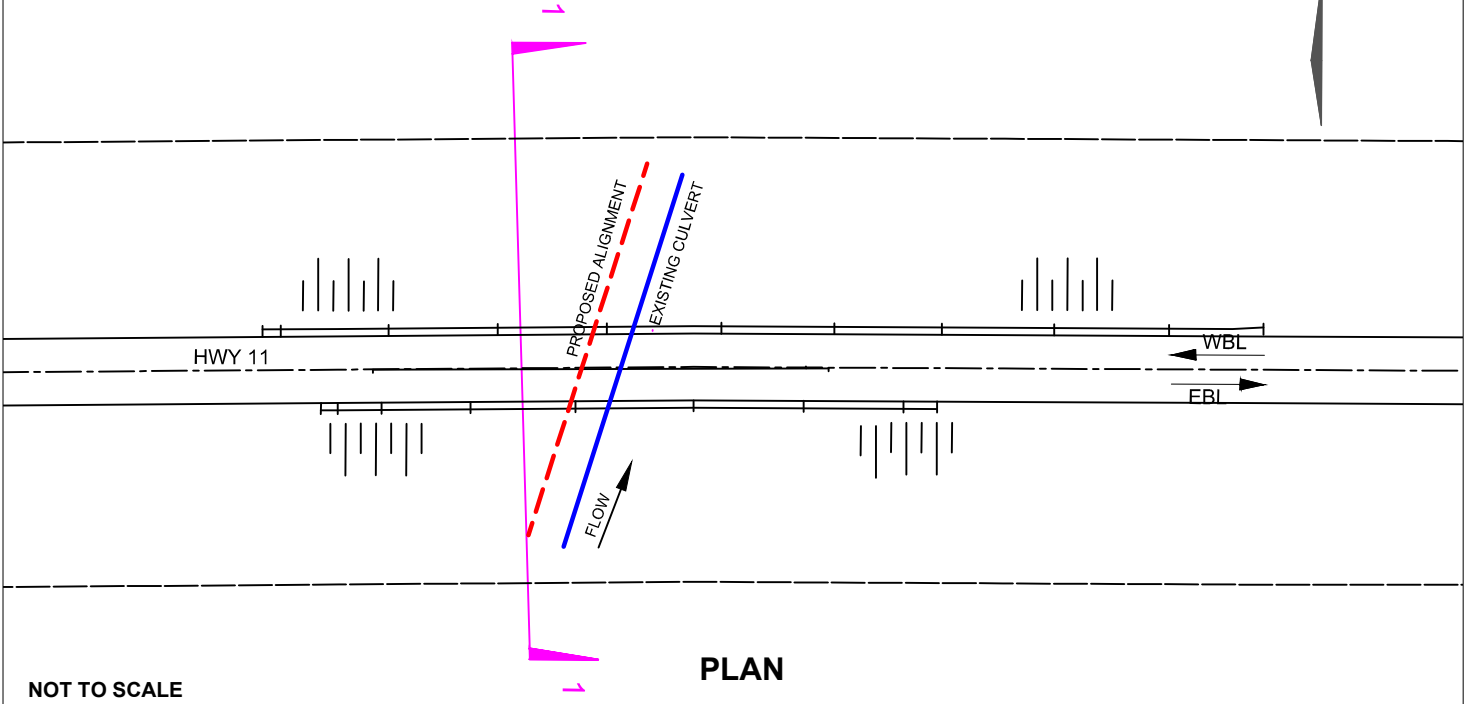
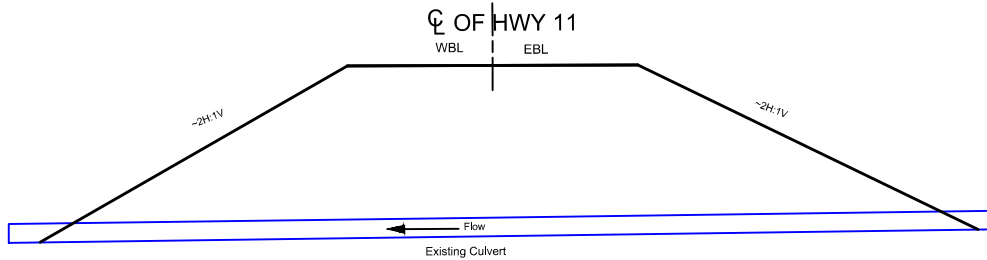


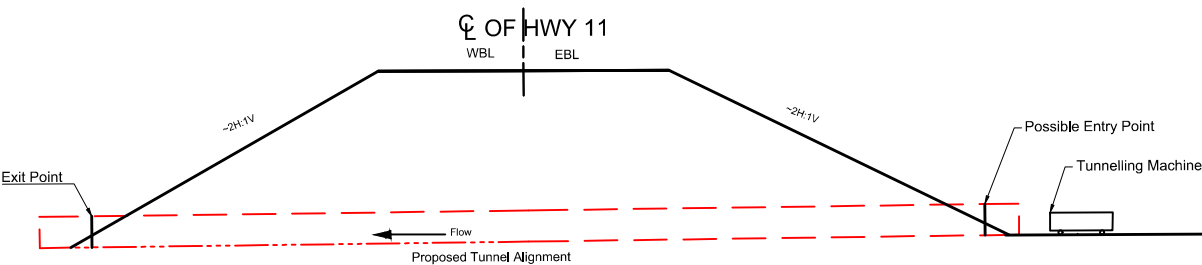
FIGURE H.4: TUNNELLING OPTION
SCHEMATIC DIAGRAMS



Current Condition



Tunnelling



NOT TO SCALE

SECTION 1-1