



FINAL REPORT

FOUNDATION INVESTIGATION AND DESIGN REPORT Locking Creek Culvert Replacement, Hwy 602, Township of Lash

**Agreement No. 6014-E-0017
Assignment No. 1
GWP 6919-12-00
Geocres No. 52C-39**

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Foundation Investigation and Design Report

Agreement No. 6014-E-0017

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Foundation Investigation and Design Report for Locking Creek Culvert Replacement
HWY 602, Township of Lash

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Part 1: 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 Locking Creek Culvert on Highway 602, located approximately 8.2 km south of the junction of Highway 602 and Highway 11 at Locking Creek, in the Township of Lash, the Ministry of Transportation (MTO) Northwestern Region. The work was undertaken under Agreement # 6014-E-0017, Assignment No. 1 (GWP 6919-12-00). The terms of reference (TOR) were as presented in the MTO letter dated December 2, 2014.

As noted in the TOR, the existing culvert is a rigid frame open footing concrete structure with a span of 3 m, depth of 1.8 m and a length of about 22 m. It is understood that the existing culvert constructed in 1935 is intended to be replaced with a new culvert along the same alignment.

The purpose of the investigation is to evaluate the subsurface conditions along the alignment, to permit detailed design for the culvert replacement. The site specific geotechnical investigation consisted of borings, soil sampling, borehole logging, and field and laboratory testing. It is noted that a Preliminary Foundation Investigation and Design Report was previously completed, by others, in November 2014, as referenced in Section 1.3.3.

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 Locking Creek Culvert replacement site is located on Hwy 602, approximately 8.2 km south of the junction of Hwy 602 and Hwy 11, in the Township of Lash. At the site, Hwy 602 is a two lane roadway with a speed limit of 80 km/h and is about 7 m wide from edge of pavement to edge of pavement, with narrow sand and gravel shoulders. Based on drawings provided, the roadway embankment is about 5 to 6 m high with side slopes of about 2H:1V.

During the fieldwork between December 17 and 20, 2014, the general site conditions were assessed; however, the site was generally snow covered which limited observations possible. Hwy 602 runs in a north to south direction and Locking Creek, flows from east to west towards the Rainy River, which is about 75 m west from Hwy 602. The Locking Creek Culvert location is located in a "valley" of the roadway. At the time of this investigation, Locking Creek was frozen and the approximate creek elevations at the inlet and outlet were 326.41 m and 326.32 m, respectively. The elevation of highway centerline pavement is 331.98 m.

The vicinity of the inlet and outlet of the culvert is heavily vegetated with trees and wild bushes. Fallen trees and branches at the inlet side were observed; however, the flow of the water did not appear to be restricted by the fallen trees.

Select photographs are provided in Appendix A.

1.2.2 Geological Setting

According to the MNR Northern Ontario Engineering Geology Terrain Data Base Map, Ontario Geological Survey Map 5069. Scale 1:100,000, dated 1978, the underlying native soil at the sites consists of clay and silt glaciolacustrine plain deposits with a subordinate landform consisting of bedrock knob; mainly low local relief, undulating to rolling and dry surface conditions.

1.3 Investigation Procedures

1.3.1 Site Investigation and Field Testing

The field investigation was performed between December 17 and 20, 2014. The field program consisted of drilling two (2) sampled boreholes (BH101 and BH102). The boreholes were strategically located about 3 m from the existing culvert; BH101 was located about 3 m north of the culvert within the southbound lane, and BH102 was located about 3 m south of the culvert within the northbound lane. The borehole locations are shown on Drawing 1 in Appendix B.

The boreholes were advanced using a rubber tire mounted CME-750 drill rig, equipped with a hollow stem continuous flight augers, standard soil sampling equipment (includes 51 mm outside diameter split spoon samplers and *in situ* shear vane testing equipment) and rock coring equipment, NQ size, operated by a specialist drilling contractor, RPM Drilling Inc. However, no rock coring equipment was used.

The boreholes BH101 and BH102 were advanced to depths of about 40.2 m and 40.1 m, respectively, at which point they were terminated.

The borehole locations were referenced to the MTM NAD83 coordinate system and their ground surface elevations were surveyed by **exp** personnel. The ground surface elevations, including top of culvert and top of water/ice at the upgradient and downgradient sides of the highway, were referenced to a temporary geodetic benchmark provided by the client (top of asphalt at Hwy 602 centreline at the culvert centerline). The elevation of the TBM was 331.98 m, and location of the TBM is detailed on Drawing 1, in Appendix B. The elevation of the TBM is based on a historical Preliminary Foundation Investigation and Design report dated, November 2014 (referenced below).

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), and were generally performed at intervals of about 0.75 m within the critical foundation zone (upper 15 m), 1.5 m intervals in the upper 21 m, and 3.0 m intervals thereafter. The original field (uncorrected) SPT "N" values were recorded on the borehole logs as recommended in the Canadian Foundation Engineering Manual and used to provide an assessment of *in-situ* compactness (cohesionless) or consistency (cohesive) soils. In addition, twelve (12) *in situ* shear field vane tests were conducted within the cohesive soils, and four (4) thin-walled, Shelby Tube samples were also collected.

Upon completion of the boreholes, ground water level measurements were carried out in boreholes in accordance with the Ministry of Transportation guidelines. The measured ground water levels after completion of drilling boreholes were recorded on borehole log sheets in Appendix C. The boreholes were backfilled with a mixture of bentonite and auger cuttings and cold patch was used to repair the asphalt surface damaged by the augers. The borehole decommissioning was in general 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 a member 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 were placed in labelled moisture-proof bags and were brought to **exp's** Thunder Bay laboratory for additional visual, textual and olfactory examination.

1.3.2 Laboratory Testing

All samples returned to the laboratory were subjected to visual examination and classification. The laboratory testing program included the determination of natural moisture content and particle size distribution for approximately 25% of the collected soil samples. Atterberg limits tests were carried out for cohesive soils. In addition, one consolidation test and a specific gravity test were performed on a representative cohesive sample within the critical foundation zone. All of the laboratory tests were carried out in accordance with MTO and/or ASTM Standards as appropriate at the **exp** laboratory in Thunder Bay, ON, with the exception of the consolidation test, which was performed by the **exp** laboratory in Brampton, ON. A summary of the laboratory testing is presented in Table 1.2 below.

The laboratory test results are provided on the attached borehole log sheets in Appendix C as well as graphically in Appendix D.

1.3.3 Previous Investigations

The following previous/historical investigations were provided by the client.

1. Preliminary Foundation Investigation and Design Report, Replacement of Locking Creek Culvert, Highway 602, Station 10+640, Township of Lash – Site No. 45-161/C; GWP 6919-12-00; Geocres No. 52C-36; Stantec Consulting Ltd; November 2014; Stantec Project No. 165000873.
2. Structural Design Report, Locking Creek Culvert, Site 45-161/C, Highway 602; GWP 6919-12-00; MTO Agreement No. 6012-E-0049; Stantec Consulting Ltd; November 2014

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. Figure 6 in Appendix D shows moisture content, total unit weight, undrained shear strength and stress history profiles with depth. 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. The historical

borehole logs from the 2014 Preliminary Foundation Investigation and Design Report, are presented in Appendix E and included in our drawings (Appendix B).

A borehole location plan and stratigraphic sections are provided in Appendix B. It should be noted that the stratigraphic boundaries indicated on the borehole log and stratigraphic sections 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 poorly graded sand with silt and gravel fill underlain by a fat clay fill. The fill layers are followed consecutively by native deposits of a lean clay with sand, underlain by a fat clay and underlain by a poorly graded sand with silt near the borehole termination depths of about 40 m below the pavement surface. A more detailed summary of the subsurface conditions encountered in the boreholes is provided in the following sections.

1.4.1 Poorly Graded Sand with Silt and Gravel (SP-SM) Fill

Poorly graded sand with silt and gravel was encountered beneath the asphalt. The asphalt thickness was about 25 mm at BH101 and BH102. The fill was generally described as frozen (in the upper zones) and brown. At BH102, asphalt treatment, about 75 mm in thickness, was observed beneath the sand with silt and gravel fill. Beneath the asphalt treatment, silty sand with gravel fill was encountered, and was described as frozen, brown and containing trace clay at depth. No SPT sampling was conducted within the fill due to frozen ground conditions; the samples were collected from the augers. This fill at BH101 and BH102 extended to about 0.5 m (331.6 m elevation) and 0.8 m (331.2 m elevation) below ground surface, respectively.

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

Grain size distribution:

- 12% to 34% gravel;
- 57% to 68% sand; and
- 9% to 20% silt and clay size.

The results of the 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 Fat Clay (CH) Fill

Fat clay fill was encountered underlying the sand with silt and gravel fill. The clay fill was generally described as stiff to soft at depth, grey, moist, containing trace to some peat. Trace wood pieces were noted at BH102 at about 2.3 m depth. Lean clay with sand fill was encountered beneath the fat clay fill at BH102. The SPT "N" values ranged between about 10 and 4 blows per 300 mm penetration, generally decreasing at depth. The clay fill extended to about 6.9 m below ground surface and about 325.2 m

elevation and 325.1 m elevation at BH101 and BH102, respectively. The thickness of this layer is between 6.1 m and 6.4 m.

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

Moisture content:

- 21.5% to 41.3%

Grain size distribution:

- 0 % gravel,
- 5% to 27% sand,
- 30% to 42% silt, and
- 35% to 64% clay size.

Total unit weights have been calculated based on the moisture contents and are estimated to range from about 18.5 to 19.5 kN/m³.

In addition, Atterberg limit testing was performed three representative samples of the clay fill (BH101-S10, BH102-S7, BH102-S11) and indicated that the soil is of medium to high plasticity. The data is shown on the plasticity chart, Figure 4, in Appendix D. The liquid limits ranged from about 46 to 63, plastic limits from 18 to 19 with corresponding plasticity index ranging from 27 to 45. The results of the moisture content, grain size distribution and Atterberg limit tests are provided on the record of borehole sheets in Appendix C. The results of the grain size distribution and Atterberg limit tests are also provided on Figure 2 and Figure 4, respectively, in Appendix D.

1.4.3 Lean Clay with Sand (CL)

Lean clay with sand was encountered beneath the fill. The lean clay with sand was generally described as firm to hard, grey, and moist to wet. At BH102, gravel and / or cobbles were noted during augering at about 17.7 m to 18.3 m depth. The SPT "N" values ranged between about 2 and 12 blows per 300 mm penetration. Four Shelby Tube samples were collected. In addition, *in situ* shear vane tests were performed and field results ranged between about 111 kPa to greater than 330 kPa (maximum instrument reading). The corrected values (based on Bjerrum, considering plasticity) ranged from about 105 kPa to greater than 310 kPa. The values of undrained shear strength of this soil layer with depth are shown on Figure 6 in Appendix D.

The lean clay with sand at BH101 and BH102 extended to about 33.6 m (298.5 m elevation) and 32.0 m (299.9 m elevation) below ground surface, respectively. The thickness of this layer is between 25.1 m and 26.7 m.

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

Moisture content:

- 20.0% to 28.6%

Grain size distribution:

- 0 % gravel,
- 20% to 28% sand,
- 34% to 41% silt, and
- 33% to 46% clay size.

Total unit weights have been calculated based on the moisture contents and are estimated to range from about 19.2 to 20.6 kN/m³. A single unit weight determination on a sample from between 11.4 m and 12.0 m in BH-101 yielded a value of about 20.4 kN/m³.

In addition, Atterberg limit testing was performed on representative samples of the lean clay with sand (BH101-S12, BH101-S17, BH101-S23, BH102-S13, BH102-S19, BH102-S24, BH102-S27, BH102-S29) and indicated that the soil is of medium plasticity. The data is shown on the plasticity chart in Figure 5, Appendix D. The liquid limits ranged from about 37 to 49, plastic limits from 14 to 16 with corresponding plasticity index ranging from 22 to 33.

The results of the moisture content, grain size distribution and Atterberg limit tests are provided on the record of borehole sheets in Appendix C. The results of the grain size distribution are also provided on Figure 3 in Appendix D, and Atterberg limit tests are provided on Figure 5 in Appendix D.

Sample BH101-S17 was tested for specific gravity, and the laboratory testing results yielded a specific gravity of 2.69.

The results of the consolidation test performed on a sample of the lean clay (BH-101, Sample 16, elevation 320.3 m) are included in Appendix D. The results are summarized below:

- Moisture content (MC) = 22.7%
- Initial void ratio (e_0) = 0.659; unit weight = 20.39 kN/m³
- Effective overburden pressure p'_0 = 164 kPa
- Pre-consolidation pressure (p'_c) = 240 kPa
- Recompression Index (C_r) = 0.025
- Compression Index (C_c) = 0.19

1.4.4 Fat Clay (CH)

Fat clay was encountered underlying the lean clay with sand. The fat clay was generally described as firm to hard, grey, and moist to wet. The SPT "N" values ranged between about 4 and 8 blows per 300 mm penetration. In addition, *in situ* shear vane tests were performed and field results ranged between about 109 kPa to greater than 277 kPa. The corrected values (based on Bjerrum, considering plasticity) ranged from about 78 kPa to 199 kPa. The undrained shear strength of this layer with depth is shown on Figure 6 in Appendix D.

The fat clay at BH101 and BH102 extended to about 39.7 m (292.4 m elevation) and 39.5 m (292.5 m elevation) below ground surface, respectively. The thickness of this layer is between 6.1 m to 7.5 m.

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

Moisture content:

- 37.2% to 64.4%

Grain size distribution:

- 0% gravel,
- 0% to 3% sand,
- 16% to 31% silt, and
- 66% to 84% clay size.

Total unit weights have been calculated based on the moisture contents and are estimated to range from about 15.9 to 18.2 kN/m³.

In addition, Atterberg limit testing was performed on two representative of the fat clay (BH101-S28, BH102-S32) and indicated that the soil is of high plasticity. The data is shown on the plasticity chart, Figure 4. The liquid limits ranged from about 55 to 91, plastic limits from 17 to 27 with corresponding plasticity index ranging from 38 to 64.

The results of the moisture content, grain size distribution and Atterberg limit tests are provided on the record of borehole sheets in Appendix C. The results of the grain size distribution are also provided on Figure 2 in Appendix D, and Atterberg limit tests are provided on Figure 4 in Appendix D.

1.4.5 Poorly Graded Sand with Silt (SP-SM)

Poorly graded sand with silt was encountered beneath the fat clay. The poorly graded sand with silt was generally described as loose to compact, grey and wet. At BH102, at the sand and clay interface, about 610 mm of blowing sand was encountered within the augers. The SPT "N" values at BH101 and BH102 were 10 and 11 blows, respectively, per 300 mm penetration. The poorly graded sand with silt at BH101 and BH102 extended to about 40.3 m (291.8 m elevation) and 40.1 m (291.9 m elevation) below ground surface, respectively.

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

Moisture content:

- 14.3% to 22.5%

Grain size distribution:

- 0% to 4% gravel,
- 89% to 92% sand, and
- 7% to 8% silt and clay size

Total unit weights have been calculated based on the moisture contents and are estimated to range from about 20.1 to 21.9 kN/m³.

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

1.5 Groundwater and Surface Water Conditions

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

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.

Table 1.1. Groundwater data

Borehole	Date Completed	Date Measured	Ground Surface Elevation ²	Depth to Water ³	Groundwater Elevation
BH101 ⁴	Dec. 20/14	Dec. 20/14	332.04	2.35	329.69
BH102 ⁴	Dec. 18/14	Dec. 18/14	331.97	4.27	327.70
BH14-1	May 21/14	May 21/14	332.1	5.9	326.2
BH14-2	May 22/14	May 22/14	331.9	5.7	326.2
BH14-3	May 26/14	May 26/14	327.9	1.7	326.2
BH14-4	May 26/14	May 26/14	328.8	2.5	326.3
Locking Creek w/ Upstream Side	--	Dec. 20/14	--	--	326.41
Locking Creek w/ Downstream Side	--	Dec. 20/14	--	--	326.32
Notes: 1) All units in metres. 2) Elevations surveyed are referenced to a geodetic temporary benchmark (TBM) provided by the client (top of asphalt at Hwy 602 centreline at the culvert centerline). The elevation of the TBM is 332.0 m, 3) Depths are relative to ground surface. 4) Artesian groundwater conditions appear to be encountered from the underlying sand with silt layer at the bottom of the boreholes.					

In addition to the groundwater depths/elevations indicated above, the recent Stantec's Preliminary Foundation Investigation and Design Report, indicates that the water level at the Locking Creek on July 20, 2012, was 326.4 m.

Part II: ENGINEERING DISCUSSIONS AND RECOMMENDATIONS

2.1 General

This section of the report provides geotechnical design recommendations for replacement of the existing culvert situated beneath Hwy 602, located approximately 8.2 km south of the junction of Highway 602 and Highway 11 at Locking Creek, in the Township of Lash. 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**, and the existing geotechnical data presented in the reports provided by MTO (see Section 1.3.3). 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.

Based on the information provided by MTO in the TOR, the existing Locking Creek culvert is a rigid-frame open footing concrete structure with a span of 3 m, depth of 1.8 m and a length of about 22 m, and provides flow from east to west under Hwy 602. It is understood that the existing culvert was constructed in about 1935, and an inspection in 2009 indicated that the culvert was in poor condition, requiring replacement with a new structure. It is also understood that the new culvert is proposed to be at the current culvert alignment. The size and type of the new culvert is not defined at the time of writing this report. However, as noted in the documents provided by MTO, several options were considered for the replacement: frame box culvert (precast or cast-in-place), rigid frame open footing culvert (precast or cast-in-place) supported on deep foundations, corrugated steel plate culvert with concrete footing supported on shallow or deep foundations, and steel sheet pile abutments with precast deck. .

This part of the 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 generally accepted good practice. Pertinent construction issues from a geotechnical standpoint are examined in general accordance with the Terms of Reference from the MTO letter dated December 2, 2014. The assessment involved review of options for replacement of the existing culvert along the same alignment with a final selection to be made by the designer, based on the optimum solution.

2.2 Expected Ground Conditions

The following ground conditions along the proposed culvert alignment are evident from the current investigation and other provided data:

- a. The height of the highway embankment at the investigation location is about 6 m with side slopes of about 2H:1V near the inlet and outlet of the existing culvert. The current elevation of the crest of the roadway is about 332.0 m.

- b. The highway embankment consists of poorly graded sand with silt and gravel fill, and silty sand with gravel fill (~ 0.5 m to 0.8 m thick) underlain by fat clay fill (~ 6.1 m to 6.4 m thick).
- c. The embankment fill is underlain by native lean clay with sand to about 32.0 m to 33.6 m (~ 25.1 m to 26.7 m thick) below ground surface, followed by a fat clay layer to about 39.5 m to 39.7 m below the ground surface (~6.1 m to 7.5 m thick). The fat clay is underlain with poorly graded sand with silt.
- d. The foundation soil at the invert of the new proposed culvert is anticipated to be native lean clay with sand located at elevations between 324.6 m and 324.9 m. Typical 'N' values ranged from 3 to 11.
- e. The groundwater table in the embankment fill is expected to be at approximate elevation 326.4 m, or slightly higher. However, seasonal variations in the water table should be expected, with higher levels occurring during wetter periods of the year (such as spring thaw and late fall) and lower levels during drier periods. Some groundwater mounding within the embankment and a perched water above the lean clay with sand layer would be anticipated.

2.3 Culvert Foundations

From the documents provided by MTO (i.e. Preliminary Foundation Investigation and Design Report by Stantec dated November 2014), it is understood that the following options were considered for the Locking Creek culvert replacement:

- frame box culvert (precast or cast-in-place),
- rigid frame open footing culvert (precast or cast-in-place) supported on deep foundations,
- corrugated steel plate culvert with concrete footing supported on shallow or deep foundations,
- steel sheet pile abutments with precast concrete deck.

The comparison of advantages and disadvantages of these options in the preliminary FIDR further indicated that the option of corrugated steel plate culvert with concrete footing supported on deep foundations was ranked as the first option. The option of steel sheet pile abutments with precast deck was ranked as a second.

Based on the subsurface information obtained from the boreholes, the native firm to stiff lean clay with sand is considered suitable for the support of all replacement options considered. However, the choice of culvert type will also depend on parameters such as the initial cost, maintenance costs, hydraulic performance, ease of construction, salvageability and local availability of material and equipment.

2.3.1 Shallow Foundations

2.3.1.1 Geotechnical Resistance

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

Table 2.1 Recommended spread footing design parameters

Culvert Type	Founding Elevation (m)	Assumed Footing Size (m)	Founding Soil Type	Factored Geotechnical Resistance at ULS (kPa)	Geotechnical Reaction at SLS**
Box Culvert	~324.7	3.0 to 4.0	Firm to Stiff Lean Clay with Sand	300	200
Rigid Frame Open Footing	~322.7*	1.0			

*below the frost line

** for maximum settlement of 25 mm

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

2.3.1.2 Resistance to Lateral Loads

Resistance to lateral forces/ sliding should be calculated in accordance with Section 6.7.5 of the CHBDC, using the following parameters:

Table 2.2 Recommended parameters for calculation of unfactored horizontal resistance

Interface and loading conditions	Parameters
Between Granular A pad and pre-cast concrete	Coefficient of friction ($\tan \delta$)=0.5
Between cast-in-place concrete and clay subgrade	Coefficient of friction ($\tan \delta$)=0.4
Between Granular A pad and clay subgrade: short term loading	Effective cohesion= 60 kPa
Between Granular A pad and clay subgrade: long term loading	Effective friction angle = 29 deg

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

2.3.1.3 Frost Protection

The frost penetration depth at the Locking Creek culvert is 2.3 m according to OPSD 3090.100. Any temporary or permanent support system using shallow foundations should be provided with a minimum 2.3 m of soil cover or equivalent thermal insulation for frost protection.

2.3.2 Deep Foundations

2.3.2.1 Driven Steel Piles

Axial Resistance in Compression

Driven steel piles can be used to support the culvert footings. Piles can consist of steel (minimum 350 MPa) pipe (open or closed) or HP sections. Such piles, driven into the underlying firm to hard lean clay with sand can be designed using the factored (0.4) resistance values in the following Table 2.3. These are only typical sizes. The values are also presented graphically on Figure F1 in Appendix F. These values result from a static analysis based on skin friction with a nominal end bearing resistance, and using the effective stress β method. The elastic compression at ULS should be less than 10 mm in all cases. Since there is no (or minimal) proposed grade raise, negative skin friction or drag loads are not a concern.

Table 2.3 Factored geotechnical resistance values (ULS) for driven steel piles

Pipe Size or HP Section	Factored ULS (kN) for Embedment (below pile cap)		
	15 m	20 m	25 m
244 mm x 9 mm	165	265	400
324 mm x 10 mm	220	365	550
HP310 x 79	260	440	650

The values given in Table 2.3, above, are based on driving the piles to various depths below the underside of the pile cap. In order to ensure that the artesian pressures encountered in the underlying sand with silt layer (at depth), do not affect the pile performance and construction, piles should not be driven to depths below 25 m (Elevation 300.0 m). This will ensure a substantial thickness (about 8 m) of fat clay between the artesian zone and the pile tips. If deeper penetration for increased capacity is required, additional analysis will be required.

Hard driving and refusal to pile penetration prior to reaching the required depth is not anticipated. Accordingly, pile shoes/tips are not necessary.

Prior to driving piles, a wave equation (WEAP) analysis should be performed in order to assess the driving stresses and the anticipated penetration resistance required to develop the required pile capacity. This analysis considers the complete driving system. Dynamic testing (PDA testing) on a number of piles with the Pile Driving Analyser must be performed near the beginning of the pile driving phase of construction to confirm the pile capacities. Alternatively, static load tests can be performed, although these are typically much more difficult to set up and are more costly.

In addition, all piles should be visually monitored by experienced personnel during installation to check for plumbness, set, internal damage, etc. All damaged piles should be rejected and if the damage is considered to be minor, the pile can be dynamically tested to determine the available pile capacity.

Piles in groups should be spaced no closer than 3 pile diameters. All piles in a group should be checked for heaving during the driving of the adjacent piles.

Lateral Resistance

In accordance with Table C6.4 of the CHBDC Commentary (2006), for HP 310x79 piles installed at this site, the lateral resistance at ULS and lateral reaction at SLS (for 10 mm) can be taken as 200 kN and 110 kN, respectively. The values for the pipe pile sizes given above will be somewhat less. The lateral resistance is shown in Table 2.4, below.

Table 2.4 Assessed horizontal passive resistance and bearing reaction for driven steel piles

Pipe Size or HP Section	Assessed Horizontal Passive Resistance and Bearing Reaction (kN)	
	ULS	SLS (10 mm)
244 mm x 9 mm (estimated)	170	70
324 mm x 10 mm (estimated)	220	150
HP310 x 79 (CHBDC Commentary – Table C6.4)	200	110

2.3.2.2 Steel Sheet Piles

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

Axial Resistance in Compression

Steel sheet piles, driven into the underlying firm to hard lean clay with sand can be designed using the factored (0.4) resistance values shown graphically on Figure F2 in Appendix F. The factored resistance values (per metre width of sheet pile) have been calculated as 100 kN for 10 m embedment, 220 kN for 15 m and 380 kN for 20 m embedment. These values are based on a static analysis, considering skin friction only (end bearing resistance is negligible), using the effective stress β method, similar to the steel piles described above. It is noted that, since the sheet piles will also be retaining the approach fills, only the embedded, outside portion of the sheet piles below the level of the creek bed is considered to contribute to axial resistance. The elastic compression at ULS should be less than 6 mm in all cases. Since there is no (or minimal) proposed grade raise, negative skin friction or drag loads are not a concern.

Lateral Resistance

For relatively short (typically less than 3 m to 4 m) abutments, a cantilever sheet pile design using the earth pressure coefficients and soil parameters provided in Section 2.4, following. Note that if this design is implemented, the precast concrete deck will likely be designed to be installed such that lateral support is provided at the top of the sheet piles.

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

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

Pressure grouted soil anchors can be designed in a preliminary fashion in accordance with Section 26 of the CFEM (2006). Based on the generally stiff soils at this site, the estimated factored (0.4) ULS resistance of grouted anchors would be 12 kN/m length. Detailed design would be completed following the design of the wall and the loads have been established. Normally, such anchors are supplied and installed/tested by specialist vendors/contractors.

2.4 Lateral Earth Pressure

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

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

where P = earth pressure intensity at depth h , kPa

K = earth pressure coefficient

γ = unit weight of retained soil, kN/m³

q = surcharge near wall, kPa

h = depth to point of interest, m

The above expression does not take into account hydrostatic pressure, which must be included for the groundwater levels measured on the site. Table 2.5 lists earth pressure parameters for given materials.

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

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

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

pressure distributions (CFEM, Section 26.10.3, Figure 26.8)

Table 2.5 Material types and earth pressure properties

Material	Unfactored Friction Angle ϕ'	Coefficient of Active Earth Pressure (K_a)	Coefficient of Passive Earth Pressure (K_p)	Coefficient of Earth Pressure at Rest (K_0)	Unit Weight γ kN/m ³
Sand with Silt and Gravel Fill	32	0.31	3.25	0.47	18
Fat Clay Fill with some Sand	29	0.35	2.88	0.52	19
Lean Clay With Sand	27	0.38	2.66	0.55	20

2.5 Construction Alternatives

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

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

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

The following Table 2.6 summarize advantages and disadvantages of considered construction alternatives. The table also shows assessed risk/consequences and relative costs of the considered methods. Schematic diagrams of considered alternatives are attached in Appendix H.

Table 2.6 Construction alternatives for culvert replacement (see schematic sketches in Appendix H)

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

Installation Method	Advantages	Disadvantages	Relative Cost	Ranking
		<ul style="list-style-type: none"> • Possible extra cost to purchase of private property 		
<p>OPTION 3.A</p> <p>Half-and-half Construction with Unsupported Cut Sides</p>	<ul style="list-style-type: none"> • Traffic flow maintained at the site during construction • Short mobilization time • Straight forward construction and construction procedures 	<ul style="list-style-type: none"> • Traffic interruption • Roadway protection of up to 6 m high required to maintain one lane of traffic • High cost of roadway protection system • Large amount of soil to be excavated • Need to temporarily control existing creek water • Risk of cost overrun and instability to finish job: low to moderate 	<p>Relatively more expensive than full road closure due to high costs of roadway protection system</p>	3
<p>OPTION 3.B</p> <p>Half-and- half Construction with Braced or Anchored Cut Sides</p>	<ul style="list-style-type: none"> • One or possibly two lanes of traffic flow maintained on existing road (e.g. steel decking, but costly) • Global stability of excavation enhanced by narrow geometry • Less traffic interruption than with unsupported cut sides approach • Temporary decking could be usable over braced cut to allow for excavation of both halves prior to diverting stream and backfilling • Cost savings due to limited excavation and backfill 	<ul style="list-style-type: none"> • Traffic interruption • Roadway protection of up to 6 m high required to maintain one lane of traffic if steel docking is not possible • High cost of roadway protection system and/or decking • Require side shoring and bracing • Bracing (e.g. struts) may interfere with excavation • Excavation of material and placement of bracing required in limited space • Need to decommission the shoring system • Need to temporarily control existing creek water • Risk of cost overrun and instability to finish job: low to moderate 	<p>More expensive than full road closure and other open cut sides approach due to high costs for shoring system and temporary decking (if feasible) to maintain continuous flow of traffic</p>	6

Installation Method	Advantages	Disadvantages	Relative Cost	Ranking
OPTION 3.C Half-and-half Construction with Installation of Steel Sheet Pile Abutments with Precast Concrete Deck	<ul style="list-style-type: none"> • Environmentally friendly • Easy to construct • No need for dewatering and cofferdam • No need for detour • No need to redirect existing creek water • No need for decommissioning of shoring system • Cost effective 	<ul style="list-style-type: none"> • Traffic interruption • Roadway protection required to maintain one lane of traffic • High cost of roadway protection system • Relatively new approach for MTO • Due to possible lateral movement need an anchor system, bracing or deadman • Durability issue with sheet pile walls • Some difficulty in excavating under concrete span • Risk of cost overrun and inability to finish job: low to moderate 	Relatively more expensive than full road closer due to high costs of shoring abutments, but more practical	1
OPTION 4 Stage Construction by Grade Lowering	<ul style="list-style-type: none"> • Traffic flow maintained at the site during construction without construction of local detour • No earth embankment fill material is required for building detours • No settlement since there is no new earth embankment fill • No shoring scheme necessary 	<ul style="list-style-type: none"> • Traffic slowdown and congestion due to too many traffic lane changes • Large amount of embankment fill to be excavated and replaced; Road has to be excavated longitudinally (~130 m) to obtain 10H:1V slope • Need to temporarily control existing creek water • Additional cost for reconstruction of road • Increased time for construction of staging • Erosion control of temporary cuts required • Risk of cost overrun and inability to finish job: low to moderate 	More expensive than full road closure and half-and-half with sheet pile abutments due to large amount of embankment fill to be excavated and replaced	4

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

1. OPTION 3.C: Half-and-half construction with the steel sheet pile abutments and precast concrete deck (Figure H3.C, Appendix H)
2. OPTION 1: Full road closure using existing roadways and open cut unsupported excavation (Figure H1, Appendix H)
3. OPTION 3.A: Half-and-half construction with unsupported cut sides (Figure H3.A, Appendix H)
4. OPTION 4: Stage construction by grade lowering (Figure H4, Appendix H)
5. OPTION 2: Temporary local detour and open cut unsupported excavation (Figure H2, Appendix H)
6. OPTION 3.B: Half-and-half construction with braced or anchored cut sides (Figure H3.B, Appendix H)

The following sections discuss these options in more details.

2.5.1 Detour Options (Options 1 and 2)

Both detour options, the option with full closer of Hwy 602 and long detours around the area using existing roadways (see Figure H1, Appendix H), and the option with the local detour embankment construction at the site to maintain the local flow of traffic during the replacement (see Figure H2, Appendix H), allow for open cut, unsupported excavation to facilitate the replacement of the existing culvert. A major benefit of these options is that the existing clayey embankment fill overlaying the existing culvert will be completely removed and replaced with free-draining engineered fill material. The other advantages are that neither excavation support nor roadway protection is required with these options. The major disadvantages of both options are traffic interruption, large amounts of excavated soils and need for temporary construction unwatering and dewatering systems (i.e. cofferdams, and sumps and pumps, etc.) to prevent existing creek water and groundwater flow into the construction area which is the responsibility of the contractor.

All excavations at this site must be conducted in accordance with the Occupational Health and Safety Act (OHSA) and Regulations for Construction (O. Reg. 213/91). All fills (i.e. sand with silt and gravel fill, and fat clay fill) and firm to stiff native lean clay with sand may be classified as a Type 3 soil above the groundwater table in conformance with the OHSA. The native soils below the groundwater table may be classified as a Type 4 soil. It is expected that most of excavations will be above the groundwater levels except those at the invert level. To avoid disturbance of the founding subgrade and to allow placement of backfill in dry conditions, groundwater must be controlled to

below the proposed invert excavation levels prior to digging to final levels. As mentioned before, the ingress of surface water must be controlled using a suitable system as well.

Temporary excavation side slopes for Type 3 soil should not exceed 1H:1V in accordance with OHSA, and 2H:1V is recommended for global stability of these deep cuts (i.e. to maintain a global factor of safety greater than 1.3) where excavation will be left open for some time. Temporary excavation side slopes for Type 4 soils should not exceed 3H:1V where applicable. There is a potential for sloughing to occur if the trench remains open for an extended period of time (i.e. > 24 hours) or during a rainfall event. In addition, some localized surficial sloughing may be experienced in areas of perched groundwater seepage (i.e. within the embankment fill).

The detour construction alternative would involve construction of a temporary on-site embankment at the west or east side of the existing embankment depending on the available space and suitable terrain. Compacted engineered fill for construction of the temporary detour road is recommended. Prior to construction of the temporary detour embankment, the site will need to be cleared and grubbed of any existing bushes and vegetation. All surficial topsoil (if exists), organics and softened or loosened soil should be stripped from below the proposed temporary detour road embankment. All subgrade soils should be proof-rolled prior to fill placement and embankment fill should be placed in accordance with OPSS 206 SP 206S03 (dated March 2012).

2.5.2 Half-and-Half Construction (Options 3)

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

The temporary excavation required to remove half of the existing embankment would be up to 6 m deep. Therefore, temporary shoring such as a soldier pile and lagging or sheet pile system will be required as a roadway protection system to allow staging excavation/construction. It will be the Contractors responsibility to design a suitable temporary support system for the MTO review prior to installation. The Contractor is to follow OPSS 538 and SP No. 902S01, regarding excavations for structures, and OPSS 539 and SP No. 105S19, regarding temporary protection systems. Recommendations for a temporary roadway protection are given in Section 2.6. Using the half-and-half construction approach, several methods of culvert replacement were considered as discussed below:

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

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

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

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

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

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

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

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

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

This option provides shoring system consisting of sheet piles perpendicular to the highway, which will serve the dual purpose of retaining backfill soil during excavation and being bearing elements to support culvert foundations after excavation (see Figure H3.C, Appendix H). As shown on Figure H3.C, the sheet piles will be installed perpendicularly in the half of the embankment at both sides of

the existing culvert after installation of the roadway protection system for Stage 1 construction. Next the fill will be excavated to the designed elevation of the deck and its precast panels will be installed over the existing culvert. Then the fill below the deck panels will be excavated within construction limits for Stage 1 allowing the existing culvert to be removed. The excavation above the deck will be backfilled with a free-draining granular material up to the highway grade. The same processes will be repeated in Stage 2 construction, on the other side of the roadway protection. The contractor should be responsible for the complete design, construction and monitoring of the described system. It is their responsibility to provide the work and design that should accommodate all relevant conditions including local and global stability for all stages of installation, including any necessary groundwater or surface water controls.

A major benefit of this method is that sheet piles will be permanently installed creating abutments for the precast concrete deck, so decommissioning of the shoring system is eliminated. Further, this method allows the use of the existing culvert to convey the Locking Creek flow below Hwy 602 even during the construction work. However, due to the depth of the fill, some lateral movement of the abutments might be possible and installation of anchor systems, bracing or deadman might be required.

2.5.3 Stage Construction by Grade Lowering (Option 4)

The stage construction by grade lowering is schematically presented in Figure H4, Appendix H. As shown, the method includes several stages of excavation of the existing road maintaining the constant one-way traffic flow. This method does not require the road protection system or detours, but it is very disruptive to the highway, since approximately 130 m of the road has to be excavated longitudinally to obtain a maximum slope of 10H:1V.

2.6 Temporary Roadway Protection

Temporary roadway protection is anticipated to be a part of the half-and-half construction approach that will be required to maintain on-site traffic during the construction. It is recommended that roadway protection system be in accordance with MTO Special Provision 105S19. The complete design, construction, monitoring and removal of the installed protection system should be a responsibility of the contractor. Due to nature of this application it is expected that much of temporary shoring will be decommissioned in place noting the high cost for removal. Decommissioning must be consistent with good practice to avoid interference with highway systems and utilities, if any. The protection system should be designed to provide protection for excavations as required by the OHSA, at locations specified in the contract, and at any locations where the stability, safety or function of an existing structure and/or utility may be impaired by construction work.

Table 2.7 presents the recommended geotechnical parameters for the design of the temporary roadway protection system.

Table 2.7 Geotechnical design parameters

Soil Type	Approximate Elevation (m)		Design Parameters			
	From	To	Unit Weight γ (kN/m ³)	Friction Angle ϕ' (degrees)	Cu (kPa)	E (MPa)
Sandy Gravelly Fill	332	330.8	18	32	-	20
Fat Clay Fill	330.8	325.1	19	-	40	20
Lean Clay With Sand	325.1	298.5	20	-	100 to 150	35
Fat Clay	298.5	2924	18		150 to 200	40

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

2.7 Culvert Bedding

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

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

For the Township of Lash, a frost penetration depth of approximately 2.3 m can occur in open, unheated areas without snow cover. At the culvert inlet and outlet, and beneath the proposed culvert, the native soils consist of lean clay with sand. This material has high frost susceptibility based upon the MTO Frost Classification guideline of percent particles between 5 to 75 μm . Therefore, non-frost susceptible materials such as sand and gravel (Granular "A") need to be provided to the limit of frost penetration beneath the inlet and outlet of the culvert. However, considering that cold air blowing through the culvert during the winter season will freeze soil next to the culvert, a minimum 500 mm thick layer of non-susceptible material should be considered to be placed as a bedding along the entire culvert length.

2.8 Culvert Backfill

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 SPMDD before a subsequent layer is placed, 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 G, 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.

2.9 Groundwater and Surface Water Control

The soils encountered below the groundwater table and within potential excavation depths consist of native lean clay with sand. The materials are highly susceptible to disturbance from groundwater and mobilized equipment. The groundwater level needs to be controlled to 1 m below the excavation level to avoid disturbance, and any surface or groundwater seepage should be removed from the excavation prior to the culvert bedding material placement of granular backfill in the dry. In general, pumping using properly filtered sumps, and/or filtered drains placed along the base of the excavation should provide sufficient groundwater control during foundation works.

Provided that the existing culvert is to remain in use during construction of the new culvert, the majority of the upstream flow of the existing culvert can be diverted around the construction area. For the control of the water flow in the creek might require a cofferdam. If Option 2.C with the sheet pile abutments is selected these measures are not required, since the existing culvert will be used for dewatering. If the existing culvert is to be removed prior to completion of the new culvert, a system of sumps and pumps will be required to divert the surface water up and over the existing embankment.

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

2.10 Embankment Design

2.10.1 Embankment Settlement

It is not planned to change the existing embankment grade at the culvert location. Therefore, theoretically there should be negligible additional settlements under the existing embankment because the clay stratum under the existing embankment would have consolidated under the stresses imposed by the existing embankment. However, a settlement of about 25 mm should be allowed for due to rebound during the construction.

2.10.2 Embankment Stability

A series of slope stability analyses were performed to assess the global stability of the existing embankment and to check that a minimum Factor of Safety of 1.3 will be achieved for the new embankment at the location of the proposed culvert (Station 10+640). 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 and the topographic plan provided by MTO. The stratigraphy and groundwater condition at the site were developed based on the results of the geotechnical investigation presented in Part I - Foundation Investigation Report.

Based on the borehole information, the subsoils encountered at the work area consist of embankment fill, underlain by native clay deposits. It is expected that the undrained shear strength of the 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. Therefore, 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 beneath the existing embankment.

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

Tabulated below in Table 2.8 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 2.8 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)
Sand with Silt and Gravel Fill	32	0	18	32	0	18
Fat Clay Fill with some Sand	29	2	19	0	40	19
Lean Clay with Sand	27	2	20	0	100	20

The results of slope stability analyses for the 2H:1V west side slope of the existing embankment using drained (long term stability) and undrained (short term stability) soil parameters are presented graphically in Figures F3 and F4 in Appendix F, respectively. For the drained soil conditions, a minimum Factor of Safety of 1.43 for a deep seated rotational failure through the native lean clay with sand subgrade was calculated, while for the undrained soil conditions, a minimum Factor of Safety of 2.23 for a failure through embankment was calculated, indicating that the embankment is stable. These preliminary results of slope analyses suggest that the new embankment should be constructed with a minimum slope of 2H:1V to achieve a minimum Factor of Safety of 1.3.

2.11 Inlet and Outlet

2.11.1 Erosion Protection at Outlet

Rip-rap protection should be provided where the culvert discharges into the open creek. The rip-rap should extend approximately 5 m beyond the ends of the culvert and line the embankment slope to the spring line of the culvert. The size of the rip-rap is a function of the creek's hydrology. As a rule of thumb the thickness of the rip-rap should be a minimum of twice the median particle size, and 300 mm thick as a minimum. The rip-rap configuration at the creek bed should generally follow the OPSD 810.010, which is included in Appendix G of this report. Rip-rap placed at 1V:1H will be stable.

Where rip-rap is not required to be used outside of the culvert location but where the embankment side slopes have been scarred and/or excavated to facilitate the existing culvert replacement the scarred and/or reinstated embankment side slopes are to be vegetated with sodding, seeding or planting as necessary depending on the flow rate and volume. Should seeding be utilized, a 100 mm thick layer of topsoil should be placed along with a degradable erosion blanket to help minimize erosion until the vegetation begins to grow.

2.11.2 Stream Bed Rip-Rap

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

2.11.3 Seepage Cut-off Requirements

The existing seepage cut-off requirements should be reviewed as well in the following context. The native clay with sand soil at the inlet side and presumably below the culvert bedding have 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) steel or wooden sheet pile cutoff at the upstream end of culvert, (iii) cut-off wall incorporated in the apron slab (if one is used) of the culvert, (iv) cut-off trench constructed with geotextile, and (v) rockfill at the upstream end of the culvert barrel to terminate below the granular bedding of the culvert. Only the clay seal and cut-off trench will be addressed since the sheet pile cut-off will require the understanding of the hydraulics of the stream.

2.11.3.1 Clay Seal

Where readily available a clay seal should be placed at the inlet of the proposed culvert, to prevent the migration of material along the face of the culvert, the formation of flow paths, and any potential internal erosion within the highway embankment (OPSD 802.095, Appendix G). OPSS 1205 specifies that material used for clay seals shall be natural clay, clay mixture (1 part Bentonite powder and 3.5 parts Granular "A") or a geosynthetic clay liner (GCL). The coefficient of permeability shall not exceed 1×10^{-6} cm/s.

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

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

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

2.11.3.2 Cut-Off Trench

A cut-off trench can be used at both the upstream and downstream ends of the culvert and can be incorporated when the rip-rap apron at both ends of the culvert are being installed. In general, a trench is dug across the stream alignment to well beyond the walls of the culvert and a

geomembrane liner is laid on the side of the trench keyed into the culvert at the top and on the base of the trench. The trench is then backfilled with graded rip-rap.

February 27, 2015

Part III: Closure

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

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

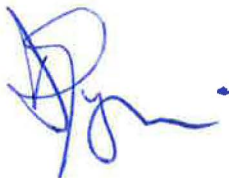
A subsurface investigation is a limited sampling of a site; the subsurface conditions have been established only at the test hole locations. Should conditions at the site be encountered which differ from those reported at the test locations, we require that we be notified immediately in order to assess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

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

This Foundation Investigation and Design Report has been prepared by Ahileas Mitsopoulos, P.Eng., Demetri N. Georgiou, M.A.Sc. P.Eng., and Silvana Micic, Ph.D., P.Eng. It was reviewed by TaeChul Kim, P.Eng. and by Stan E. Gonsalves, M.Eng., P.Eng., Designated MTO Foundation Contact. The field investigation was supervised by Elwin Farkas.

Yours truly,

exp Services Inc.



Demetri N. Georgiou, M.A.Sc., P.Eng.
Senior Geotechnical Engineer
Project Manager



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



Encl.

Appendix A – Photographs



Photo 1. Inlet of existing culvert at east side of highway



Photo 2. Outlet of existing culvert on west side of highway. Fracturing is noted.



Photo 3. Facing north on Hwy 602 before the existing culvert



Photo 4. Facing south on Hwy 602 from the existing culvert

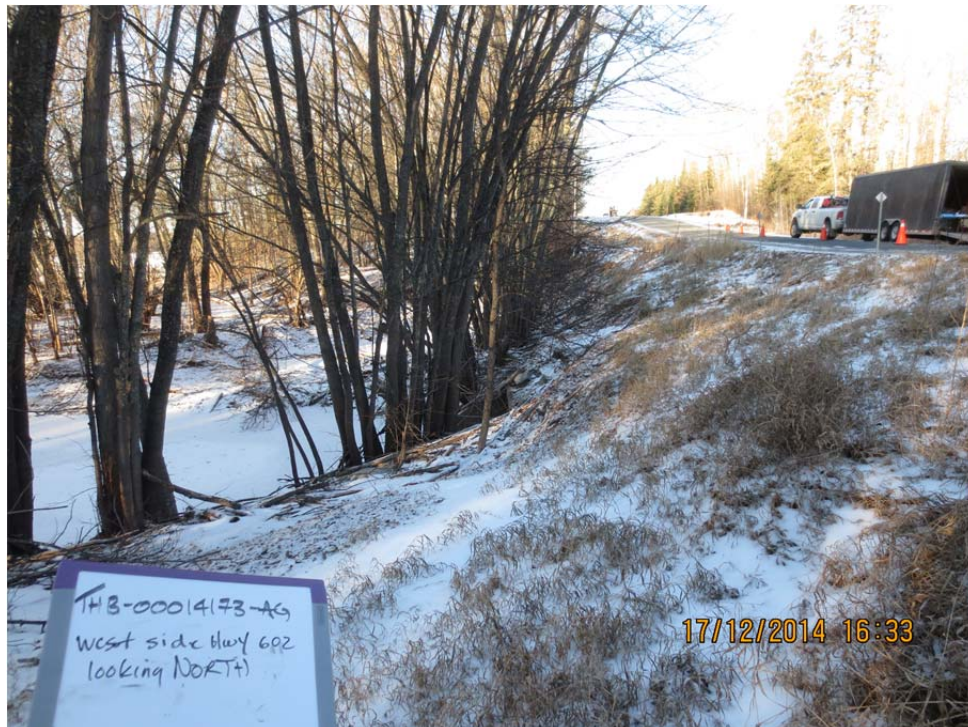
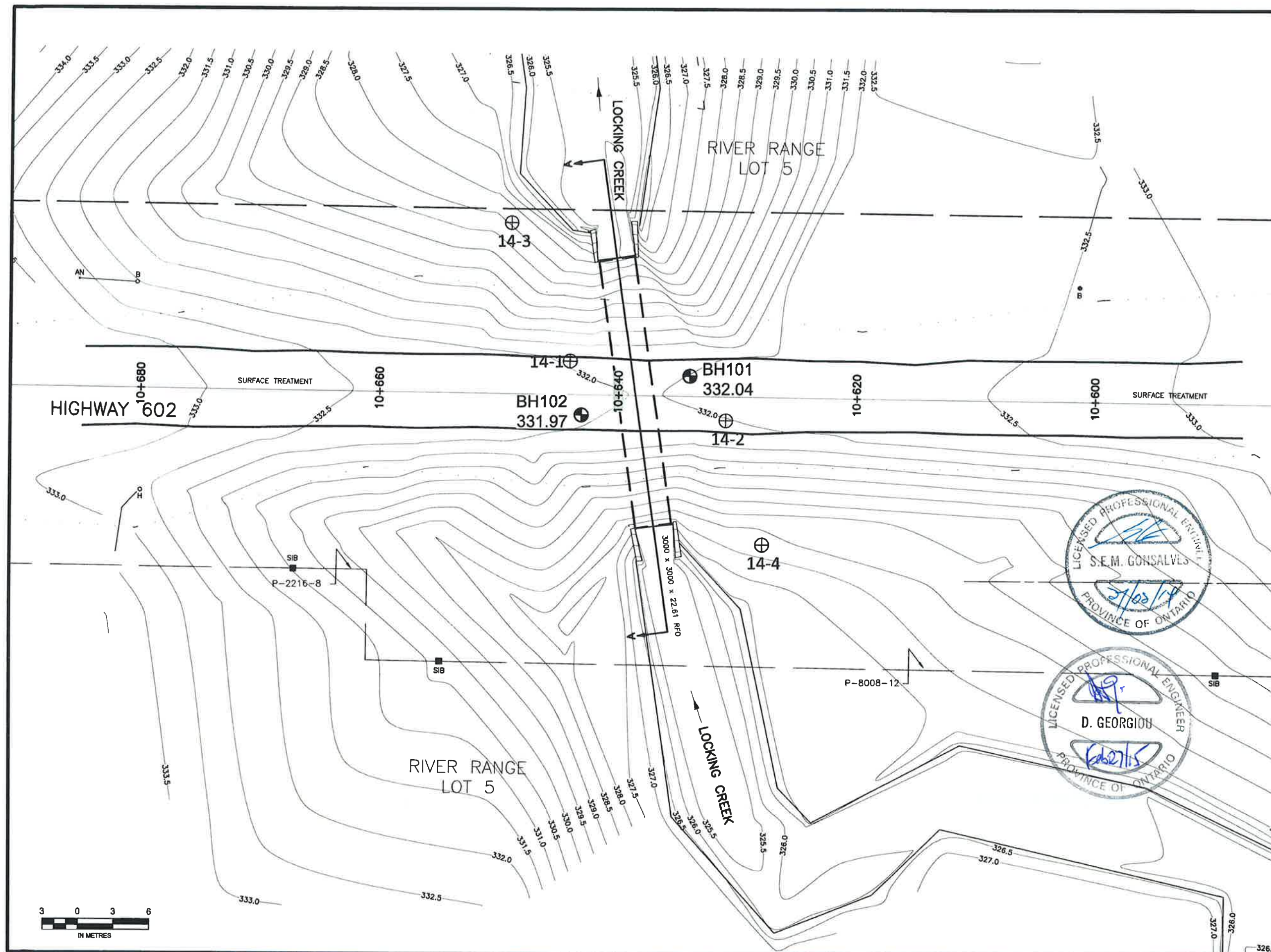


Photo 5. Embankment slope on west side facing north



Photo 6. Embankment slope on east side facing north

Appendix B – Drawings



Agreement No. 6014-E-0017
Assignment No. 1
GWP 6919-12-00

LOCKING CREEK CULVERT
(Highway 602, Township of Lash)
PLAN

DWG
1

exp

exp Services Inc.

KEY PLAN

LEGEND

BH101 BOREHOLE LOCATION
332.04 GROUND SURFACE ELEVATION
IN METRES

14-1 HISTORICAL BOREHOLE
LOCATION

BH No.	APPROX. ELEV. (m)	MTM COORDINATES	
		NORTH	EAST
BH101	332.04	5,381,407	245,688
BH102	331.97	5,381,396	245,693
14-1	332.1	5,381,392	245,689
14-2	331.9	5,381,405	245,694
14-3	327.9	5,381,388	245,676
14-4	328.8	5,381,409	245,703

NOTES

- ALL DIMENSIONS ARE IN METRES.
- BASE MAP PROVIDED BY CLIENT.
- REFERENCE: THE HISTORICAL BOREHOLE LOCATIONS ARE BASED ON THE STANTEC, PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT FOR THE LOCKING CREEK CULVERT REPLACEMENT, DATED NOVEMBER 2014.
- THIS DRAWING IS FOR SUBSURFACE INFORMATION ONLY. THE PROPOSED STRUCTURE DETAILS/WORKS ARE SHOWN FOR ILLUSTRATION PURPOSES ONLY.

REVISIONS

DATE	BY	DESCRIPTION

GEOCREs No. 52C-39
Date: February 25, 2014
Drawn By: RM

Project No. ADM-00223648-AD
Scale : 1:300
Checked By: AM
Checked By: DG

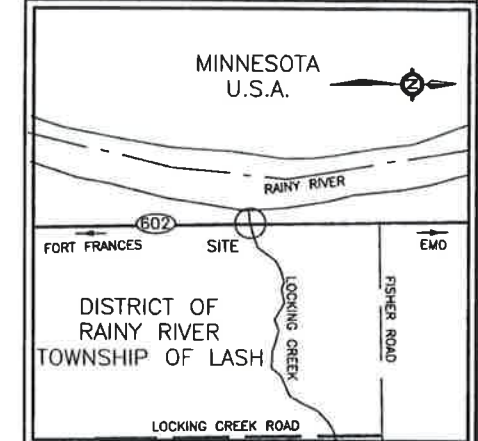
Agreement No. 6014-E-0017
Assignment No. 1
GWP 6919-12-00

LOCKING CREEK CULVERT
(Highway 602, Township of Lash)
CROSS SECTION

DWG
2

exp. **exp Services Inc.**

KEY PLAN



LEGEND

N STANDARD PENETRATION
TEST (BLOWS/0.3 m)

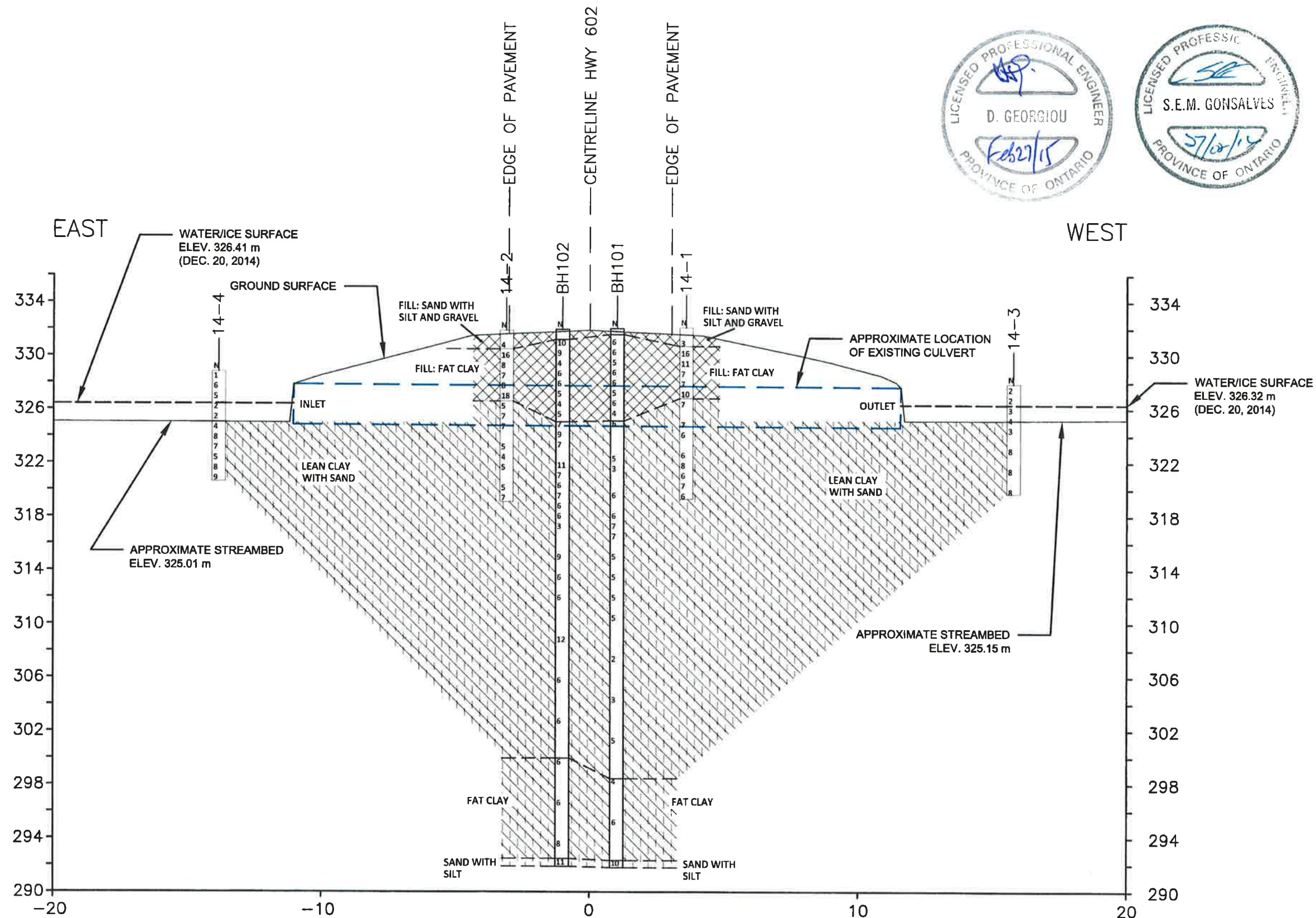
BH No.	APPROX. ELEV. (m)	MTM COORDINATES	
		NORTH	EAST
BH101	332.04	5,381,407	245,688
BH102	331.97	5,381,396	245,693
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14-2	331.9	5,381,405	245,694
14-3	327.9	5,381,388	245,676
14-4	328.8	5,381,409	245,703

NOTES

- ALL DIMENSIONS ARE IN METRES.
- ELEVATIONS OF THE CULVERT AND STREAMBED ARE DETERMINED FROM CLIENT PROVIDED DRAWINGS.
- REFERENCE: THE HISTORICAL BOREHOLE LOCATIONS AND DETAILS ARE BASED ON THE STANTEC, PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT FOR THE LOCKING CREEK CULVERT REPLACEMENT, DATED NOVEMBER 2014.
- THIS DRAWING IS FOR SUBSURFACE INFORMATION ONLY. THE PROPOSED STRUCTURE DETAILS/WORKS ARE SHOWN FOR ILLUSTRATION PURPOSES ONLY.

REVISIONS

DATE	BY	DESCRIPTION
GEOCREs No. 52C-39	Project No. ADM-00223648-A0	
Date: February 25, 2015	Scale 1:150 H 1:300 V	
Drawn By: RM	Checked By: AM	
	Checked By: DG	



A-A
PROFILE OF LOCKING CREEK CULVERT

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.

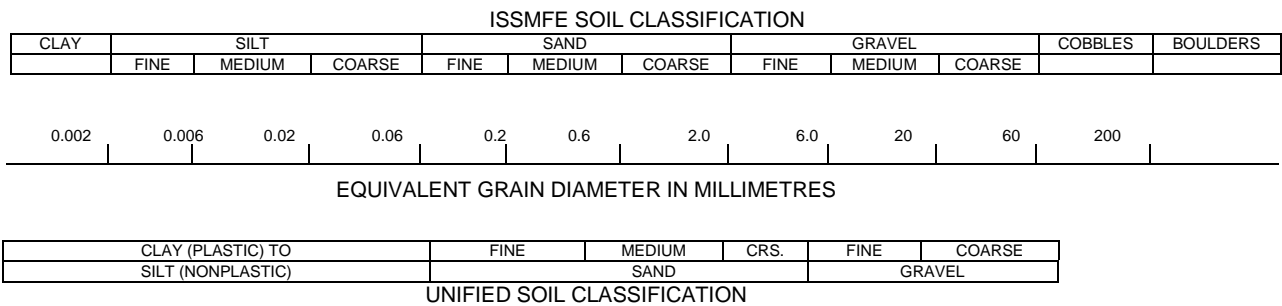
Seam: a thin, confined layer of soil having different particle size, texture, or color from materials above and below.

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:

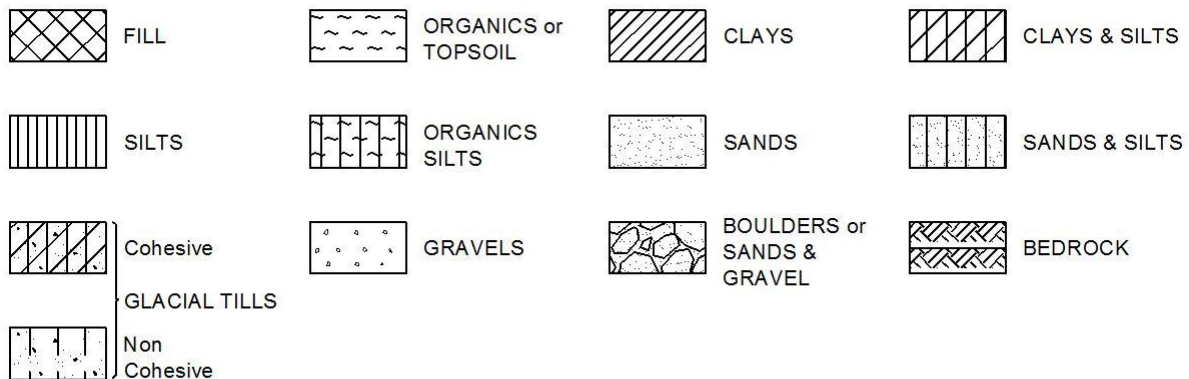
Table c: Consistency of Cohesive Soil

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'	$-\circ$	Effective angle of internal friction
c_u	kPa	Apparent cohesion intercept
ϕ_u	$-\circ$	Apparent angle of internal friction
τ_R	kPa	Residual shear strength
τ_r	kPa	Remoulded shear strength
S_t	1	Sensitivity = c_u/τ_r

PHYSICAL PROPERTIES OF SOIL

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

RECORD OF BOREHOLE No BH101

1 OF 2

METRIC

W.P. GWP No. 6919-12-00 LOCATION Locking Creek Culvert (Site No. 45-161/C) MTM ON-16 5,381,406 N 245,688 E ORIGINATED BY EF
DIST 61 HWY 602 BOREHOLE TYPE CME 750 Rubber Tire Mount / HSA COMPILED BY AM
DATUM Geodetic DATE 12.19.14 - 12.20.14 CHECKED BY DG

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 kPa									
332.0	Asphalt		S1 AUGER					20	40	60	80	100					
330.0	ASPHALT- about 25 mm		S2 AUGER														34 57 (9)
331.6	Poorly Graded SAND with Silt and Gravel (SP-SM) Fill- frozen, brown		S3 AUGER														
0.5	Fat CLAY (CH) Fill- frozen in upper 0.8 m, grey - becoming firm, moist, trace peat at about 0.8 m depth		S4	SS	6												
			S5	SS	6												
			S6	SS	5												0 6 30 64
			S7	SS	6												
			S8	SS	6												
			S9	SS	5												
			S10	SS	6												
			S11	SS	4												0 5 42 53
325.2	- becoming firm to soft, grey to brown at about 6.1 m depth		S12	SS	6												
6.9	Lean CLAY with Sand (CL) very stiff to hard, grey, moist to wet		VANE														0 23 41 36 Field Vane = 105 kPa
			S13	SH													
			S14	SS	5												
			S15	SS	3												
			VANE														Field Vane = 187 kPa
			S16	SH													
			S17	SS	6												0 24 40 36 Field Vane = 196 kPa
			VANE														
			S18	SS	6												
			S19	SS	7												
			S20	SS	7												
			VANE														Field Vane = 153 kPa
			S21	SS	5												
			S22	SS	5												
			VANE														Field Vane = 235 kPa
			S23	SS	5												0 21 39 40
			S24	SS	5												
			VANE														Field Vane = 105 kPa
			S25	SS	2												

Continued Next Page

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

METRIC

[illegible]

METRIC



ON_MOT F-14173-AG - ADM-00223648-A0 - MTO 1 - LOCKING CREEK CULVERT.GPJ ON_MOT.GDT 02/25/15

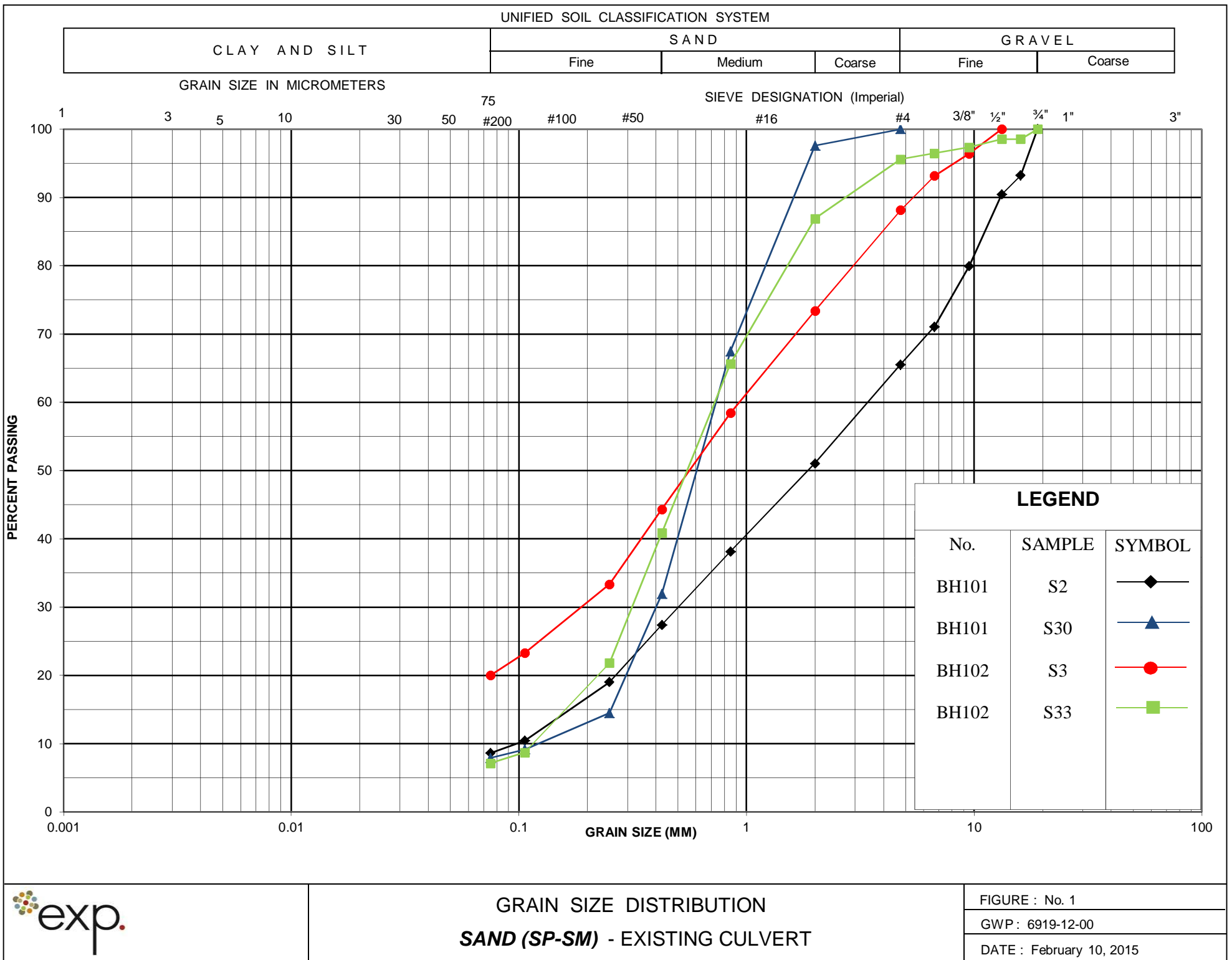
2 OF 2

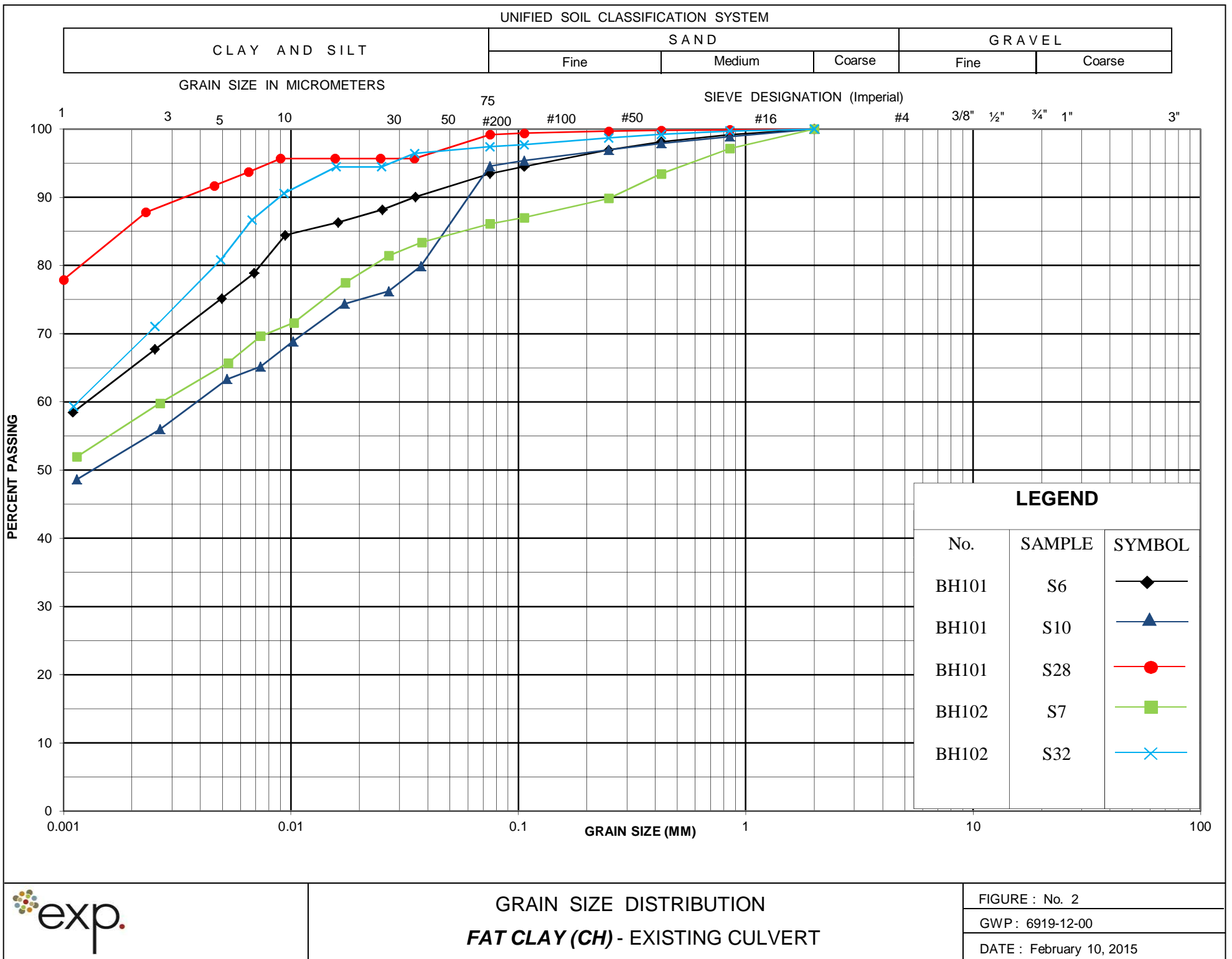
W.P.	GWP No. 6919-12-00	LOCATION	Locking Creek Culvert (Site No. 45-161/C) MTM ON-16 5,381,395 N 245,693 E	ORIGINATED BY	EF
DIST	61 HWY 602	BOREHOLE TYPE	CME 750 Rubber Tire Mount / HSA	COMPILED BY	AM
DATUM	Geodetic	DATE	12.17.14 - 12.18.14	CHECKED BY	DG

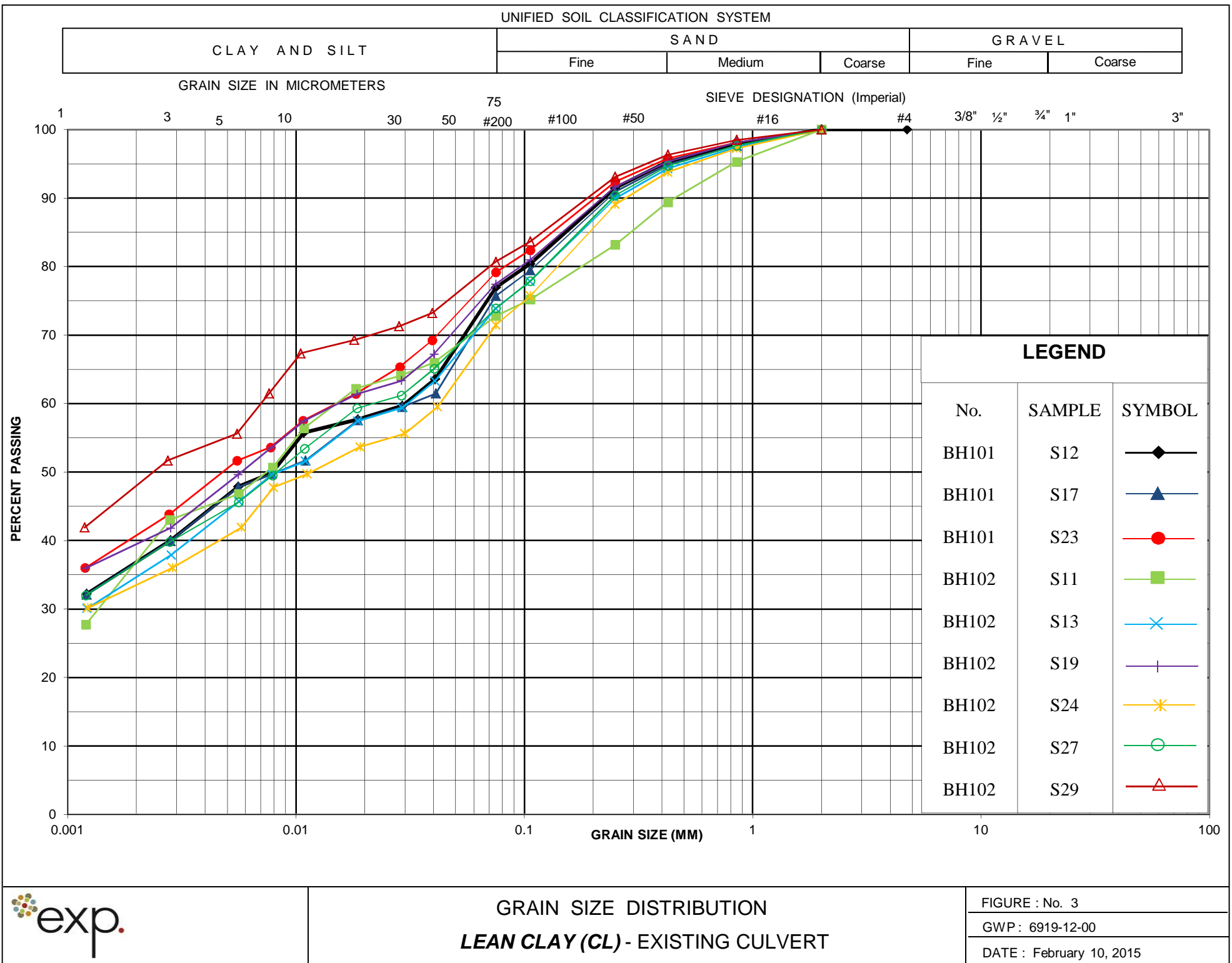
+³, ×³: Numbers refer to Sensitivity ○^{3%} STRAIN AT FAILURE

ON_MOT F-14173-AG - ADM-00223648-A0 - MTO 1 - LOCKING CREEK CULVERT.GPJ ON_MOT.GDT 02/25/15

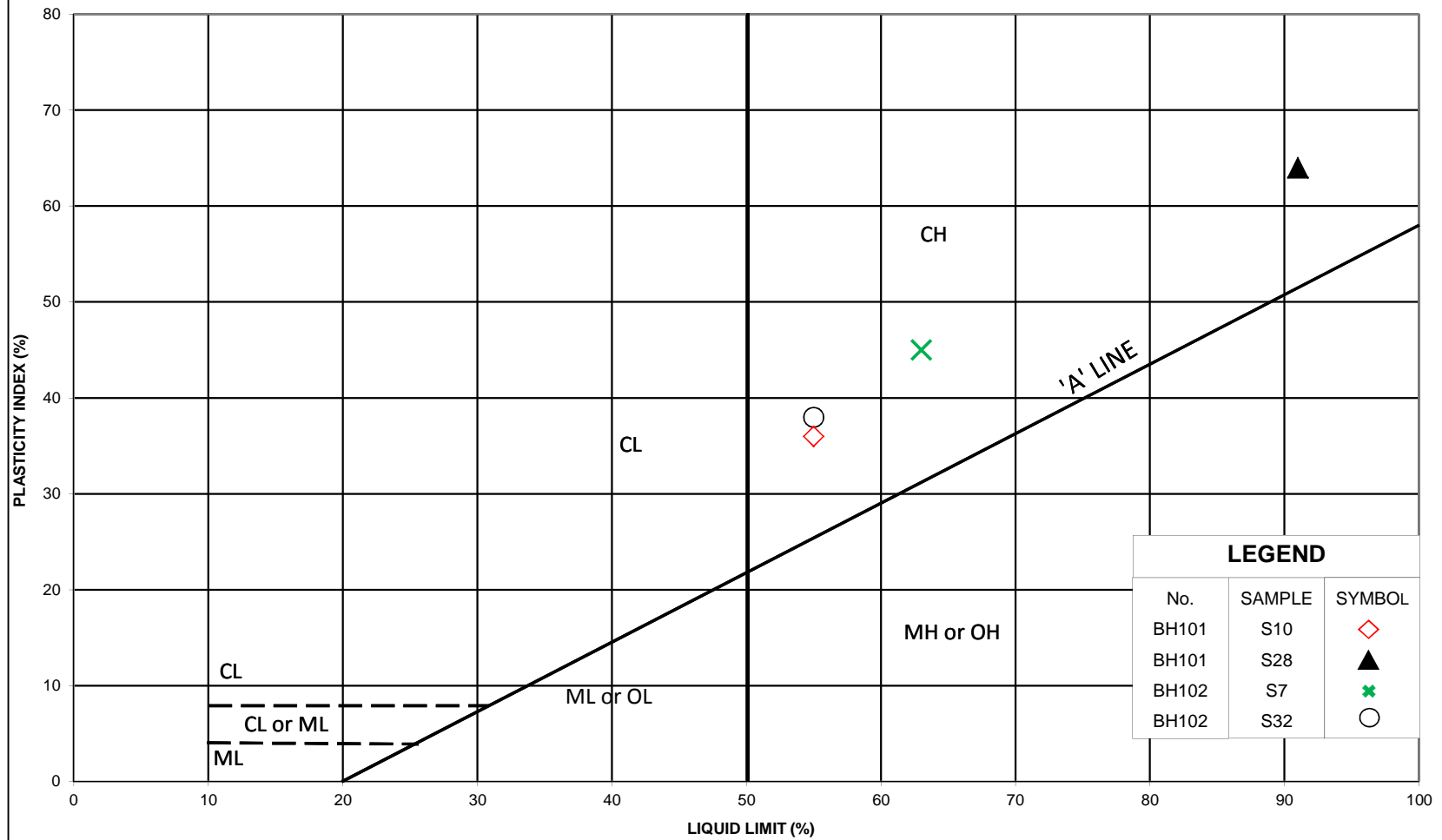
Appendix D – Laboratory Data







Locking Creek Culvert (Site No. 45-161/C)
GWP No. 6919-12-00, Highway 602, Township of Lash, Ontario



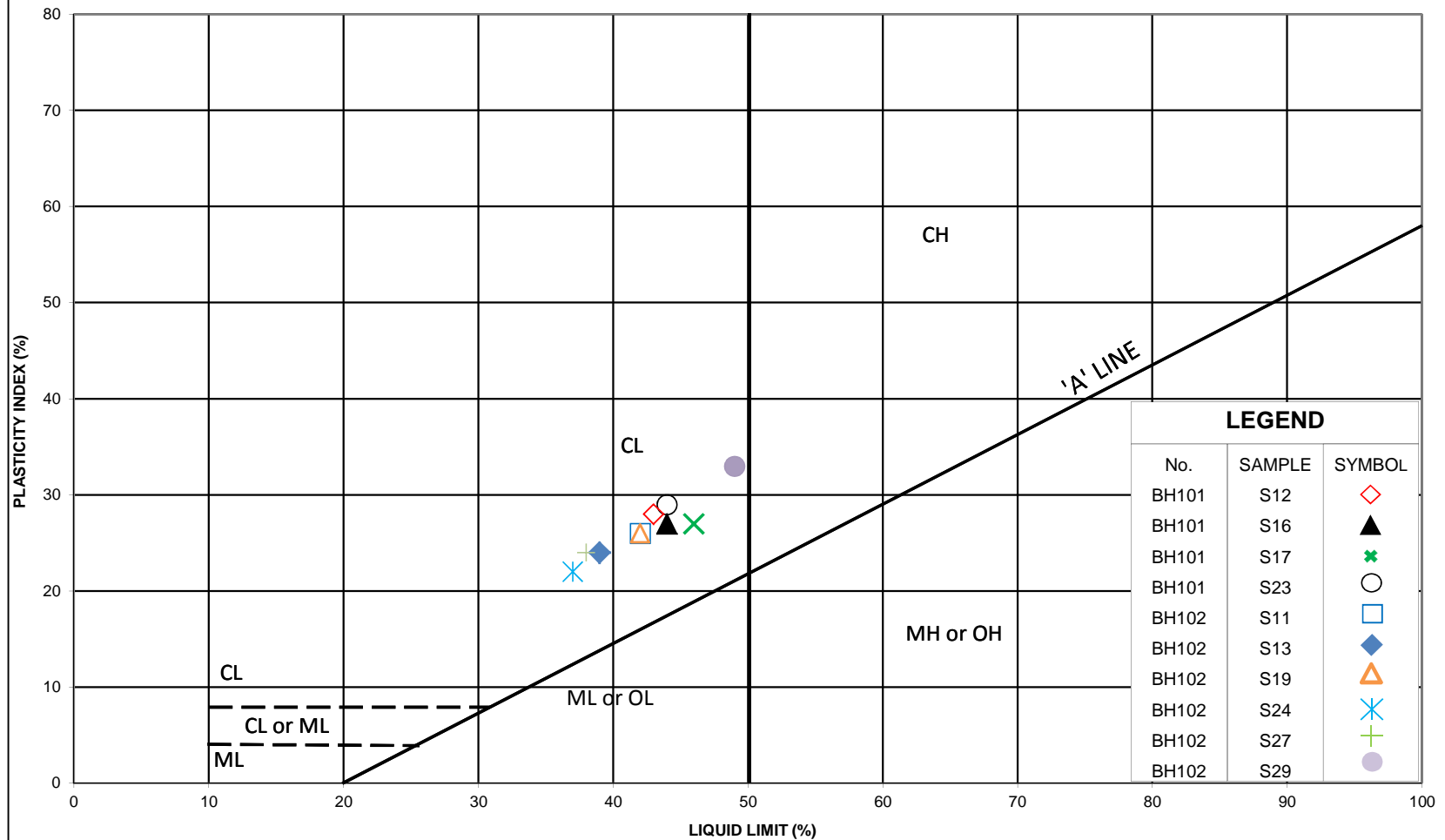
PLASTICITY CHART
FAT CLAY (CH)

FIGURE 4

ADM-00223648-A0

DATE February 25, 2015

Locking Creek Culvert (Site No. 45-161/C)
GWP No. 6919-12-00, Highway 602, Township of Lash, Ontario

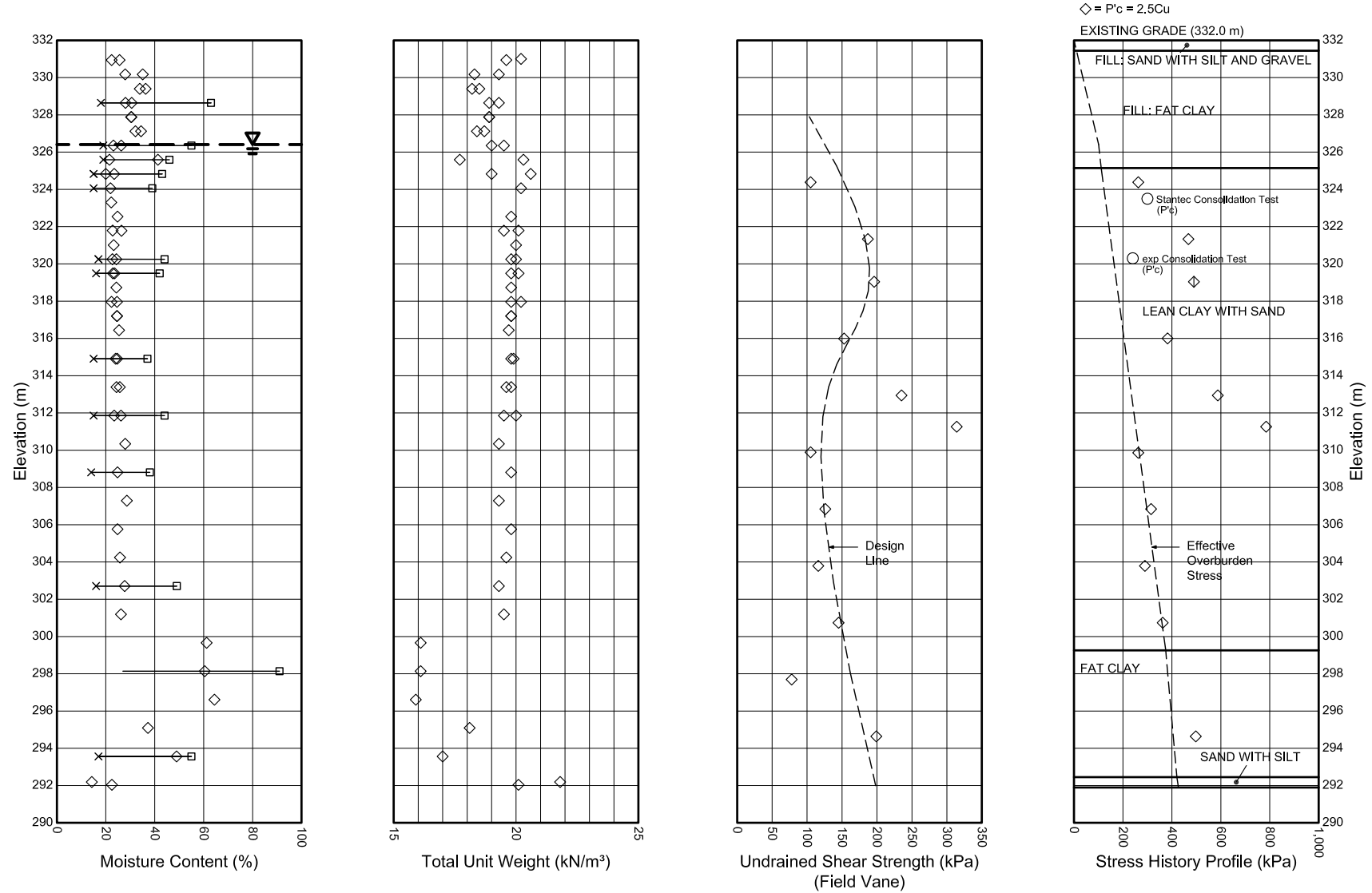


PLASTICITY CHART
LEAN CLAY (CL)

FIGURE 5

ADM-00223648-A0

DATE February 25, 2015



Thunder Bay, Ontario

FIGURE
6

SOIL PROPERTIES

Locking Creek Culvert, Site No. 45-161/C
Highway 602, Township of Lash, Ontario

PROJECT NO.:	ADM-00223648-A0
SCALE:	N/A
DRAWN BY:	RM
CHECKED BY:	DG
DATE:	February 25, 2015



exp Services Inc.

 1595 Clark Boulevard
 Brampton, ON
 L6T 4V1
 Tel.: 905-793-9800
 Fax: 905-793-0641

**Consolidation Test
 Summary Data Sheet
 (ASTM: D 2435-96)**
Project No.: adm-00223648-a0 geo-200 200-100Project Name: LabBorehole No. BH 101

Client Job No.: _____

Sample No. TW-16Sample Location: MTO Hwy 602Depth: 11.4 - 12.0mSample Description: Silty Clay Brown

Water Content Determination	Before Test		After Test
	Specimen	Trimings	Specimen
Wt. of wet sample + Ring (tare) - g	225.64	91.47	225.1
Wt. of dry sample + Ring (tare) - g	200.75	74.5	200.75
Wt. of water (W_w) - g	24.89	16.97	24.35
Wt. of Ring - g	96.17	2.09	96.17
Wt. of dry soil (W_s) - g	104.58	72.41	104.58
Water Content (W) - %	23.8	23.4	23.3
Average (W) - %	23.6		23.3

Apparatus:

Machine No.	3
Cell No.	3
Ring No.	3
Diameter of Ring (in) :	2.5
Height of Ring - H_1 (in):	0.784
Area of Ring (in^2) :	4.9087

 Load Factor:

1.55	lb. on Hanger
500	lb/ft ² on Sample

Test Data

Initial Dial Reading (in) :	0.03
Final Dial Reading (in) :	0.0577
Difference (in) :	0.0277
Machine Correction 0 to 0 (in) :	0.0043
Change in Ht., specimen, ΔH (in) :	0.0234
Final Ht. of specimen, $H_2 = H_1 - \Delta H$:	0.7606

Spec. Gr. of Solids (G) :	(estimated)	2.75
Spec. Gr. of Solids (G) :	(determined)	
Initial Height of Specimen, H_1 (in):		0.7840

Calculations	Before Test	After Test
Height of Specimen, H_1, H_2 (in):	0.7840	0.7606
Ht of Solids, H_s (in):	0.4726	0.4726
Ht. of Voids, H_v (in):	0.3114	0.2880
Ht. of Water, H_w (in):	0.3093	0.3026
Saturation, S_r (%):	99.3	100.0
Void ratio (e):	0.659	0.609

Comments:

Reported By: Willie RodychDate Reported: 19/01/2015



exp Services Inc.

1595 Clark Boulevard
Brampton, ON
L6T 4V1
Tel.: 905-793-9800
Fax: 905-793-0641

Consolidation Test Determination of Void Ratio (ASTM: D 2435-96)

Project No. adm-00223648-a0 geo-200 200-100

Project Name Lab

Client Job No.:

Sample No. BH 101 TW-16 11.4 - 12.0m

Sample Location

Height of Solids (in):	0.473
Initial Height of Voids (in):	0.311
Initial Void Ratio (e_0):	0.659
Initial Dial Reading:	0.030

Load No.	Hanger Load (lbs.)	Pressure on sample (lb/ft ²)	Final Dial Reading	Decrease in Height of Voids (in)	Machine Deflection (in)	Net Decrease in Height of Voids (in)	Height of Voids (in)	Void Ratio (e)
1	1.55	500	0.0340	0.0040	0.0014	0.0026	0.3088	0.653
2	3.1	1000	0.0382	0.0082	0.0023	0.0059	0.3055	0.646
3	6.2	2000	0.0467	0.0167	0.0036	0.0131	0.2983	0.631
4	12.4	4000	0.0599	0.0299	0.0050	0.0249	0.2865	0.606
5	24.8	8000	0.0789	0.0489	0.0070	0.0419	0.2695	0.570
6	49.6	16000	0.1057	0.0757	0.0094	0.0663	0.2451	0.519
7	99.2	32000	0.1373	0.1073	0.0118	0.0955	0.2159	0.457
8	24.8	8000	0.1266	0.0966	0.0094	0.0872	0.2242	0.474
9	6.2	2000	0.1059	0.0759	0.0071	0.0688	0.2426	0.513
10	1.55	500	0.0848	0.0548	0.0056	0.0492	0.2622	0.555
11								
12								
13								
14								
15								

Tested By:

Willie Rodych

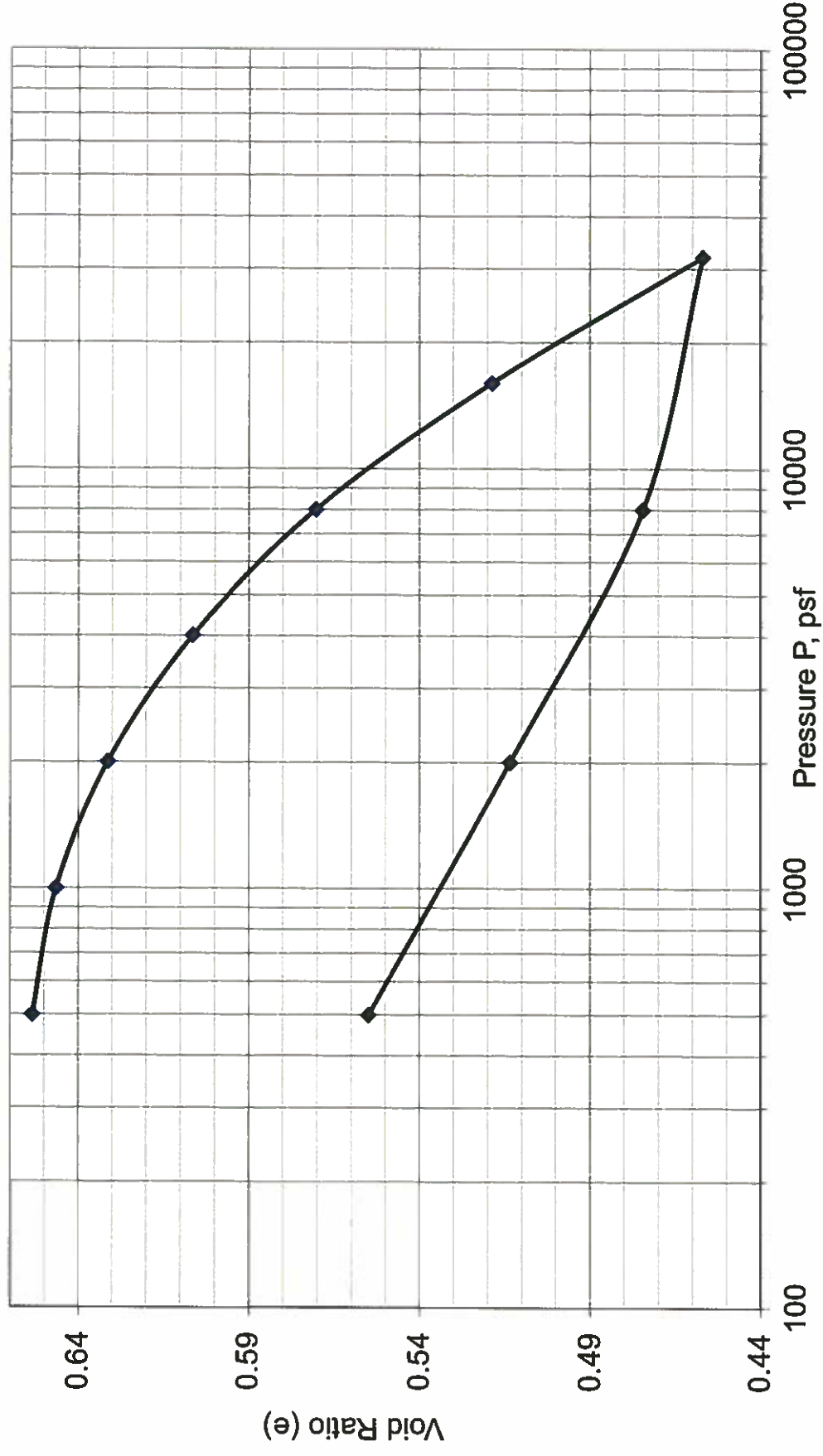
Date:

19/01/2015

Graph - e vs log P

Sample Test No.: 219686-6

Page 3 of 5





exp Services Inc.

**1595 Clark Boulevard
Brampton, ON
L6T 4V1
Tel.: 905-793-9800
Fax: 905-793-0641**

**Consolidation Test
Coefficient of Consolidation
(ASTM: D 2435-96)**

Project No.: adm-00223648-a0 geo-200 200-100

Project Name: Lab

Client Job No.:

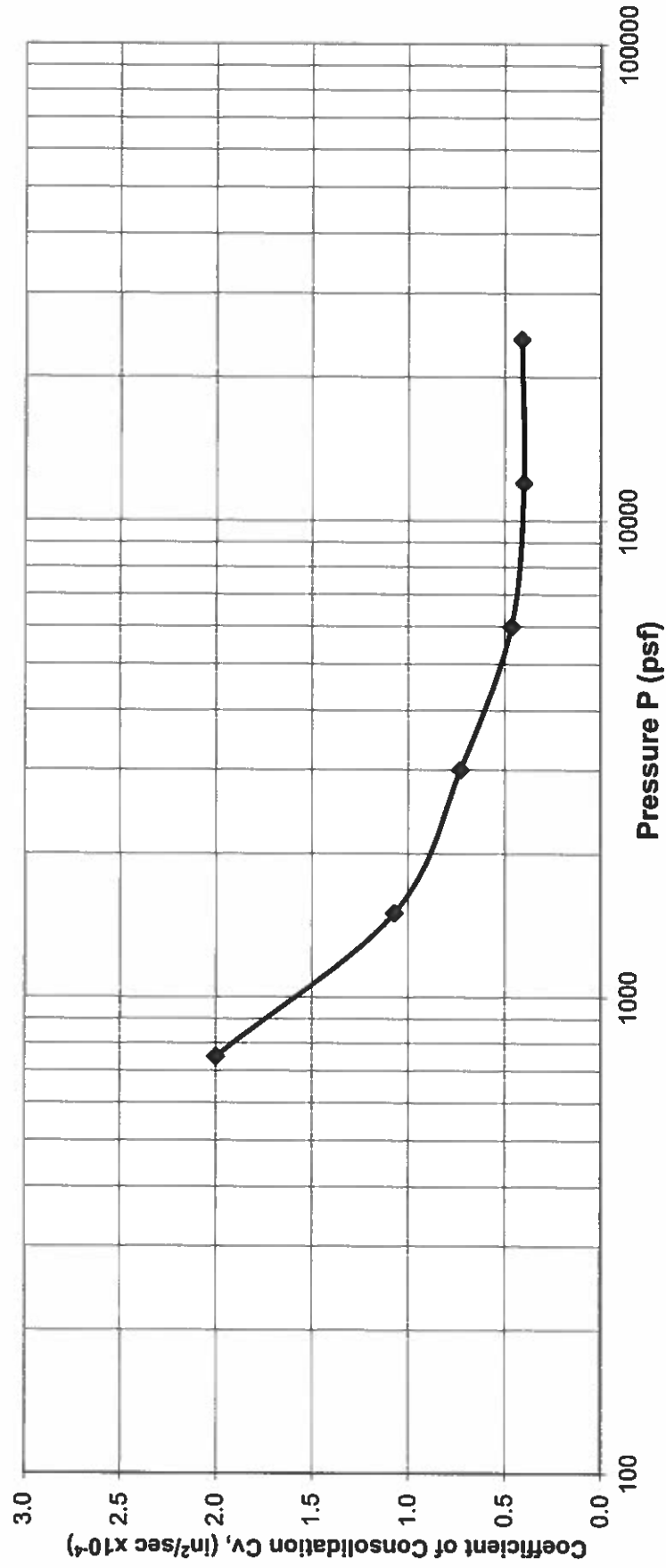
Site Location:

Sample No. BH 101 TW-16 11.4 - 12.0m

Initial Height of Sample (in):	0.7840
Initial Dial Reading:	0.0300

[illegible]

Graph - C_v vs $\log P$



Appendix E – Historical Borehole Logs

SYMBOLS AND TERMS USED ON BOREHOLE AND TEST PIT RECORDS

SOIL DESCRIPTION

Terminology describing common soil genesis:

<i>Topsoil</i>	- mixture of soil and humus capable of supporting vegetative growth
<i>Peat</i>	- mixture of visible and invisible fragments of decayed organic matter
<i>Till</i>	- unstratified glacial deposit which may range from clay to boulders
<i>Fill</i>	- material below the surface identified as placed by humans (excluding buried services)

Terminology describing soil structure:

<i>Desiccated</i>	- having visible signs of weathering by oxidization of clay minerals, shrinkage cracks, etc.
<i>Fissured</i>	- having cracks, and hence a blocky structure
<i>Varved</i>	- composed of regular alternating layers of silt and clay
<i>Stratified</i>	- composed of alternating successions of different soil types, e.g. silt and sand
<i>Layer</i>	- > 75 mm in thickness
<i>Seam</i>	- 2 mm to 75 mm in thickness
<i>Parting</i>	- < 2 mm in thickness

Terminology describing soil types:

The classification of soil types are made on the basis of grain size and plasticity in accordance with the Unified Soil Classification System (USCS) (ASTM D 2487 or D 2488). The classification excludes particles larger than 76 mm (3 inches). The USCS provides a group symbol (e.g. SM) and group name (e.g. silty sand) for identification.

Terminology describing cobbles, boulders, and non-matrix materials (organic matter or debris):

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:

<i>Trace, or occasional</i>	Less than 10%
<i>Some</i>	10-20%
<i>Frequent</i>	> 20%

Terminology describing compactness of cohesionless soils:

The standard terminology to describe cohesionless soils includes compactness (formerly "relative density"), as determined by the Standard Penetration Test N-Value (also known as N-Index). A relationship between compactness condition and N-Value is shown in the following table.

Compactness Condition	SPT N-Value
<i>Very Loose</i>	<4
<i>Loose</i>	4-10
<i>Compact</i>	10-30
<i>Dense</i>	30-50
<i>Very Dense</i>	>50

Terminology describing consistency of cohesive soils:

The standard terminology to describe cohesive soils includes the consistency, which is based on undrained shear strength as measured by *in situ* vane tests, penetrometer tests, or unconfined compression tests.

Consistency	Undrained Shear Strength	
	kips/sq.ft.	kPa
<i>Very Soft</i>	<0.25	<12.5
<i>Soft</i>	0.25 - 0.5	12.5 - 25
<i>Firm</i>	0.5 - 1.0	25 - 50
<i>Stiff</i>	1.0 - 2.0	50 - 100
<i>Very Stiff</i>	2.0 - 4.0	100 - 200
<i>Hard</i>	>4.0	>200

ROCK DESCRIPTION

Terminology describing rock quality:

RQD	Rock Mass Quality
0-25	<i>Very Poor</i>
25-50	<i>Poor</i>
50-75	<i>Fair</i>
75-90	<i>Good</i>
90-100	<i>Excellent</i>

Rock quality classification is based on a modified core recovery percentage (RQD) in which all pieces of sound core over 100 mm long are counted as recovery. The smaller pieces are considered to be due to close shearing, jointing, faulting, or weathering in the rock mass and are not counted. RQD was originally intended to be done on NW core; however, it can be used on different core sizes if the bulk of the fractures caused by drilling stresses are easily distinguishable from *in situ* fractures. The terminology describing rock mass quality based on RQD is subjective and is underlain by the presumption that sound strong rock is of higher engineering value than fractured weak rock.

Terminology describing rock mass:

Spacing (mm)	Joint Classification	Bedding, Laminations, Bands
> 6000	<i>Extremely Wide</i>	-
2000-6000	<i>Very Wide</i>	<i>Very Thick</i>
600-2000	<i>Wide</i>	<i>Thick</i>
200-600	<i>Moderate</i>	<i>Medium</i>
60-200	<i>Close</i>	<i>Thin</i>
20-60	<i>Very Close</i>	<i>Very Thin</i>
<20	<i>Extremely Close</i>	<i>Laminated</i>
<6	-	<i>Thinly Laminated</i>

Terminology describing rock strength:

Strength Classification	Unconfined Compressive Strength (MPa)
<i>Extremely Weak</i>	< 1
<i>Very Weak</i>	1 – 5
<i>Weak</i>	5 – 25
<i>Medium Strong</i>	25 – 50
<i>Strong</i>	50 – 100
<i>Very Strong</i>	100 – 250
<i>Extremely Strong</i>	> 250

Terminology describing rock weathering:

Term	Description
<i>Fresh</i>	No visible signs of rock weathering. Slight discolouration along major discontinuities
<i>Slightly Weathered</i>	Discolouration indicates weathering of rock on discontinuity surfaces. All the rock material may be discoloured.
<i>Moderately Weathered</i>	Less than half the rock is decomposed and/or disintegrated into soil.
<i>Highly Weathered</i>	More than half the rock is decomposed and/or disintegrated into soil.
<i>Completely Weathered</i>	All the rock material is decomposed and/or disintegrated into soil. The original mass structure is still largely intact.

STRATA PLOT

Strata plots symbolize the soil or bedrock description. They are combinations of the following basic symbols. The dimensions within the strata symbols are not indicative of the particle size, layer thickness, etc.



Boulders
Cobbles
Gravel



Sand



Silt



Clay



Organics



Asphalt



Concrete



Fill



Bedrock

SAMPLE TYPE

SS	Split spoon sample (obtained by performing the Standard Penetration Test)
ST	Shelby tube or thin wall tube
DP	Direct-Push sample (small diameter tube sampler hydraulically advanced)
PS	Piston sample
BS	Bulk sample
WS	Wash sample
HQ, NQ, BQ, etc.	Rock core samples obtained with the use of standard size diamond coring bits.

WATER LEVEL MEASUREMENT



measured in standpipe,
piezometer, or well



inferred

RECOVERY

For soil samples, the recovery is recorded as the length of the soil sample recovered. For rock core, recovery is defined as the total cumulative length of all core recovered in the core barrel divided by the length drilled and is recorded as a percentage on a per run basis.

N-VALUE





Numbers in this column are the field results of the Standard Penetration Test: the number of blows of a 140 pound (64 kg) hammer falling 30 inches (760 mm), required to drive a 2 inch (50.8 mm) O.D. split spoon sampler one foot (305 mm) into the soil. For split spoon samples where insufficient penetration was achieved and N-values cannot be presented, the number of blows are reported over sampler penetration in millimetres (e.g. 50/75). Some design methods make use of N value corrected for various factors such as overburden pressure, energy ratio, borehole diameter, etc. No corrections have been applied to the N-values presented on the log.

DYNAMIC CONE PENETRATION TEST (DCPT)

Dynamic cone penetration tests are performed using a standard 60 degree apex cone connected to A size drill rods with the same standard fall height and weight as the Standard Penetration Test. The DCPT value is the number of blows of the hammer required to drive the cone one foot (305 mm) into the soil. The DCPT is used as a probe to assess soil variability.

OTHER TESTS

S	Sieve analysis
H	Hydrometer analysis
k	Laboratory permeability
γ	Unit weight
G_s	Specific gravity of soil particles
CD	Consolidated drained triaxial
CU	Consolidated undrained triaxial with pore pressure measurements
UU	Unconsolidated undrained triaxial
DS	Direct Shear
C	Consolidation
Q_u	Unconfined compression
I_p	Point Load Index (I_p on Borehole Record equals $I_{p(50)}$ in which the index is corrected to a reference diameter of 50 mm)

	Single packer permeability test; test interval from depth shown to bottom of borehole
	Double packer permeability test; test interval as indicated
	Falling head permeability test using casing
	Falling head permeability test using well point or piezometer

W.P. 6919-12-00

LOCATION Highway 602 Locking Creek (Site 45-161/C)

N: 5 381 392 E: 245 689 ORIGINATED BY JHJ

DIST _____ HWY 602

BOREHOLE TYPE Hollow-stem Augers, Splittspoon Sampler

COMPILED BY KF

DATUM Geodetic

DATE 2014 05 21 - 2014 05 21

CHECKED BY SG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT W _p — W — W _L WATER CONTENT (%)	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					
								○ UNCONFINED × FIELD VANE ● QUICK TRIAXIAL × LAB VANE					
332.1	Asphalt												
332.0	25 mm Asphalt												
	FILL: silty sand with gravel, dark brown to black		1	BS									
			2	SS	3								PP=62 kPa
330.7													
1.4	FILL: silty clay, trace wood, roots, sand and gravel, grey		3	SS	16								
			4	SS	11								PP=62 kPa
			5	SS	7								5 16 28 51 PP=100 kPa
			6	SS	7								PP= 62 kPa
			7	SS	10								PP= 62 kPa
326.8													
5.3	SILTY CLAY (CI)		8	SS	7								0 21 44 35
	Stiff to very stiff												
	Grey												S _u = 167 kPa
	- cobbles encountered at 7.6 m depth		9	ST									
			10	SS	7								
	- trace gravel below 8 m depth		11	SS	6								PP=50 kPa
			12	ST								20.1	
			13	SS	6								S _u = 154 kPa

Continued Next Page

x³, x³: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

STN13-ONTARIO MTO STANTEC 165000873 - MTO 13 STRUCTURES_LOCKINGCK GPJ ONTARIO MOT.GDT 14/8/25



RECORD OF BOREHOLE No BH14-1

2 OF 2

METRIC

W.P. 6919-12-00 LOCATION Highway 602 Locking Creek (Site 45-161/C) N: 5 381 392 E: 245 689 ORIGINATED BY JHJ
DIST HWY 602 BOREHOLE TYPE Hollow-stem Augers, Spittspon Sampler COMPILED BY KF
DATUM Geodetic DATE 2014 05 21 - 2014 05 21 CHECKED BY SG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)		
					20 40 60 80 100					w _p w w _L							
					○ UNCONFINED × FIELD VANE ● QUICK TRIAXIAL × LAB VANE												
					20 40 60 80 100					10 20 30							
319.3 12.8	SILTY CLAY (CI) Stiff to very stiff Grey (continued)		14	SS	8		322								PP=62 kPa		
			15	SS	6		321					>> 4.1			S _u = 146 kPa		
											>> 2.1			S _u = 159 kPa			
			16	SS	7										1 27 36 36		
						320											
			17	SS	6												

×³ ×³Numbers refer to
Sensitivity○^{3%}

STRAIN AT FAILURE

**Stantec****RECORD OF BOREHOLE No BH14-2**

1 OF 2

METRIC

W.P. 6919-12-00

LOCATION

Highway 602 Locking Creek (Site 45-161/C)

N: 5 381 405 E: 246 694

ORIGINATED BY JHJ

DIST HWY 602

BOREHOLE TYPE Hollow-stem Augers, Splittspoon Sampler

COMPILED BY KF

DATUM Geodetic

DATE

2014 05 22 - 2014 05 22

CHECKED BY SG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT LIMIT		UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)				
								○ UNCONFINED ● QUICK TRIAXIAL	× FIELD VANE × LAB VANE	w _p	w			w _L
331.9 0.0	Granular Fill		1	BS										
	FILL: clayey sand and gravel, brown to black - occasional wood pieces		2	SS	4									
330.5 1.4	FILL: silty clay, some sand, trace gravel, grey		3	SS	16									1 18 41 40
			4	SS	8									
			5	SS	7									PP=75 kPa
			6	SS	8									
			7	SS	18									0 12 35 53
	- wood encountered at 5.0 m deep													
326.6 5.3	SILTY CLAY (Cl), trace gravel		8	SS	5								PP=50 kPa	
	Very stiff		9	SS	7									1 35 32 32
	Grey		10	SS	7									PP=75 kPa
			11	ST										
			12	SS	5									PP=75 kPa
			13	SS	4									S _u = 134 kPa S _u = 123 kPa
	PP = Pocket penetrometer S _u = Undrained shear strength													

PP = Pocket penetrometer
S_u = Undrained shear strength

Continued Next Page

× 3 × 3

Numbers refer to Sensitivity

○ 3%

STRAIN AT FAILURE

STN13-ONTARIO MTO-STANTEC 165000873 - MTO 13 STRUCTURES LOCKINGCK GPJ ONTARIO MOT GDT 14/8/25

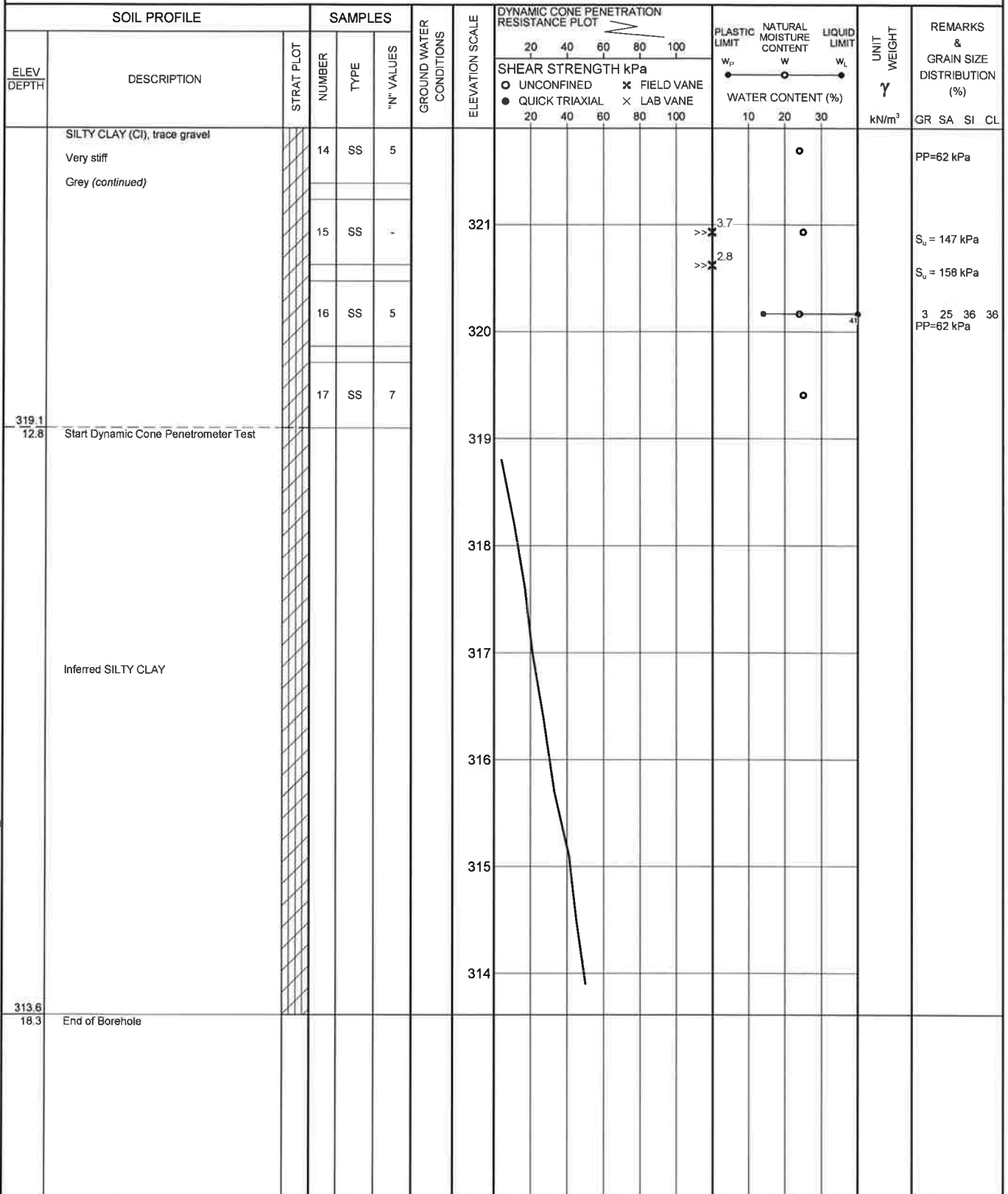


RECORD OF BOREHOLE No BH14-2

2 OF 2

METRIC

W.P. 6919-12-00 LOCATION Highway 602 Locking Creek (Site 45-161/C) N: 5 381 405 E: 246 694 ORIGINATED BY JHJ
DIST HWY 602 BOREHOLE TYPE Hollow-stem Augers, Split Spoon Sampler COMPILED BY KF
DATUM Geodetic DATE 2014 05 22 - 2014 05 22 CHECKED BY SG



STN13-ONTARIO MTO STANTEC 165000873 - MTO 13 STRUCTURES_LOCKINGCK.GPJ ONTARIO.MOT.GDT 14/8/25

x 3 x 3

Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

W.P. 6919-12-00

LOCATION Highway 602 Locking Creek (Site 45-161/C)

N: 5 381 388 E: 245 676

ORIGINATED BY JHJ

DIST _____ HWY 602

BOREHOLE TYPE Portable Drilling Equipment, Splittspoon Sampler

COMPILED BY KF

DATUM Geodetic

DATE 2014 05 26 - 2014 05 26

CHECKED BY SG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)				
								○ UNCONFINED × FIELD VANE ● QUICK TRIAXIAL × LAB VANE						
327.9	Peat													
327.0	PEAT													
0.1	SILTY SAND (SM), with clay, some gravel, trace organics		1	BS	2									
	Very loose													
	Dark brown to grey													
			2	SS	2		327							
			3	SS	3		326							
325.6	SILTY CLAY (CI), with sand, some gravel and cobbles													
2.3	Firm to very stiff		4	SS	4		325							PP=38 kPa
	Grey		5	SS	3									2 28 36 34 PP=50 kPa
	- Shelby tube refusal - Soil strength exceeded capacity of vane (200 kPa) at 4.1 m		6	BS	-		324							
			7	SS	8		323							PP=50 kPa
							322							
			8	SS	8									2 25 35 38 PP=75 kPa
							321							
	- Soil strength exceeded capacity of vane (200 kPa)		9	SS	8		320							PP=50 kPa
319.7	End of Borehole													
8.2														
	PP = Pocket penetrometer													

**Stantec****RECORD OF BOREHOLE No BH14-4**

1 OF 1

METRIC

W.P. 6919-12-00 LOCATION Highway 602 Locking Creek (Site 45-161/C) N: 5 381 409 E: 245 703 ORIGINATED BY JHJ
 DIST HWY 602 BOREHOLE TYPE Portable Drilling Equipment, Splitspoon Sampler COMPILED BY KF
 DATUM Geodetic DATE 2014 05 26 - 2014 05 26 CHECKED BY SG

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 kPa							WATER CONTENT (%)
								○ UNCONFINED ● QUICK TRIAXIAL	✕ FIELD VANE ✕ LAB VANE						
328.8	Peat							20 40 60 80 100	10 20 30						
328.9	PEAT														
0.1	SILTY CLAY (Cl), with sand, trace gravel		1	SS	1		328							PP=50 kPa	
	Firm		2	SS	6										
	Brown to Grey		3	SS	5		327							PP=50 kPa	
			4	SS	2		326							0 39 28 33	
325.7	SILTY CLAY (Cl), with sand, trace gravel		5	SS	2		325							PP=75 kPa	
3.1	Very stiff		6	SS	4									PP=75 kPa	
	Grey		7	SS	8		324							1 24 37 38 PP=62 kPa	
	- Soil strength exceeded capacity of vane (200 kPa) vane at 4.1 m		8	SS	7		323							PP=62 kPa	
			9	SS	5		322							PP=38 kPa	
			10	SS	8									1 25 36 38 PP=38 kPa	
			11	SS	9		321								
320.6	End of Borehole														
8.2	Note: 63.6 kg at 762 mm drop														
	PP = Pocket penetrometer														

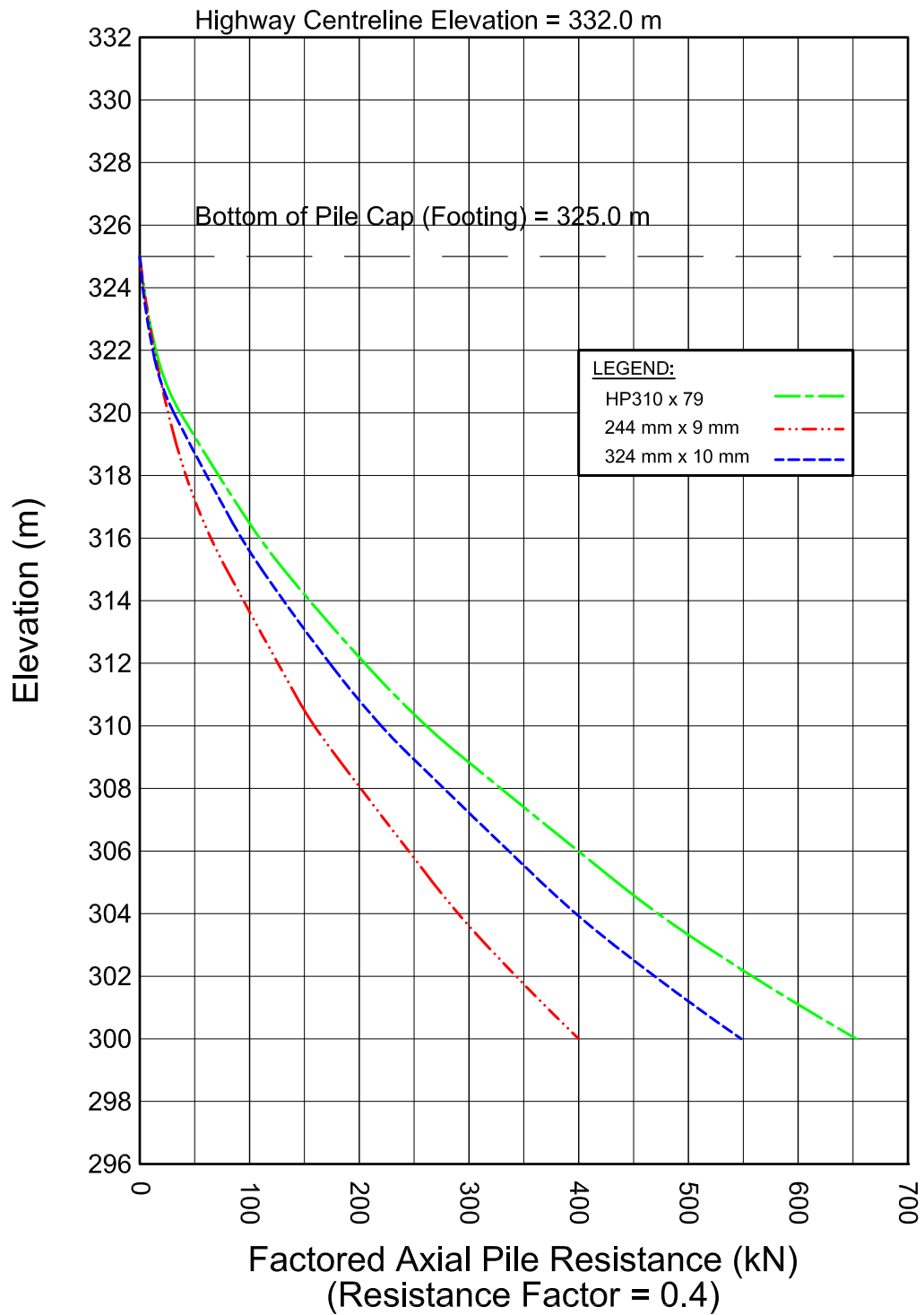
STN13-ONTARIO MTO STANTEC 165000873 - MTO 13 STRUCTURES_LOCKINGCK.GPJ ONTARIO MOT GDT 14/8/25

 × 3 × 3
 Sensitivity

Numbers refer to

○ 3% STRAIN AT FAILURE

Appendix F – Pile Capacity and Slope Stability Analyses Results



Thunder Bay, Ontario

FIGURE
F1

Factored Axial Pile Resistance

Locking Creek Culvert, Site No. 45-161/C
Highway 602, Township of Lash, Ontario

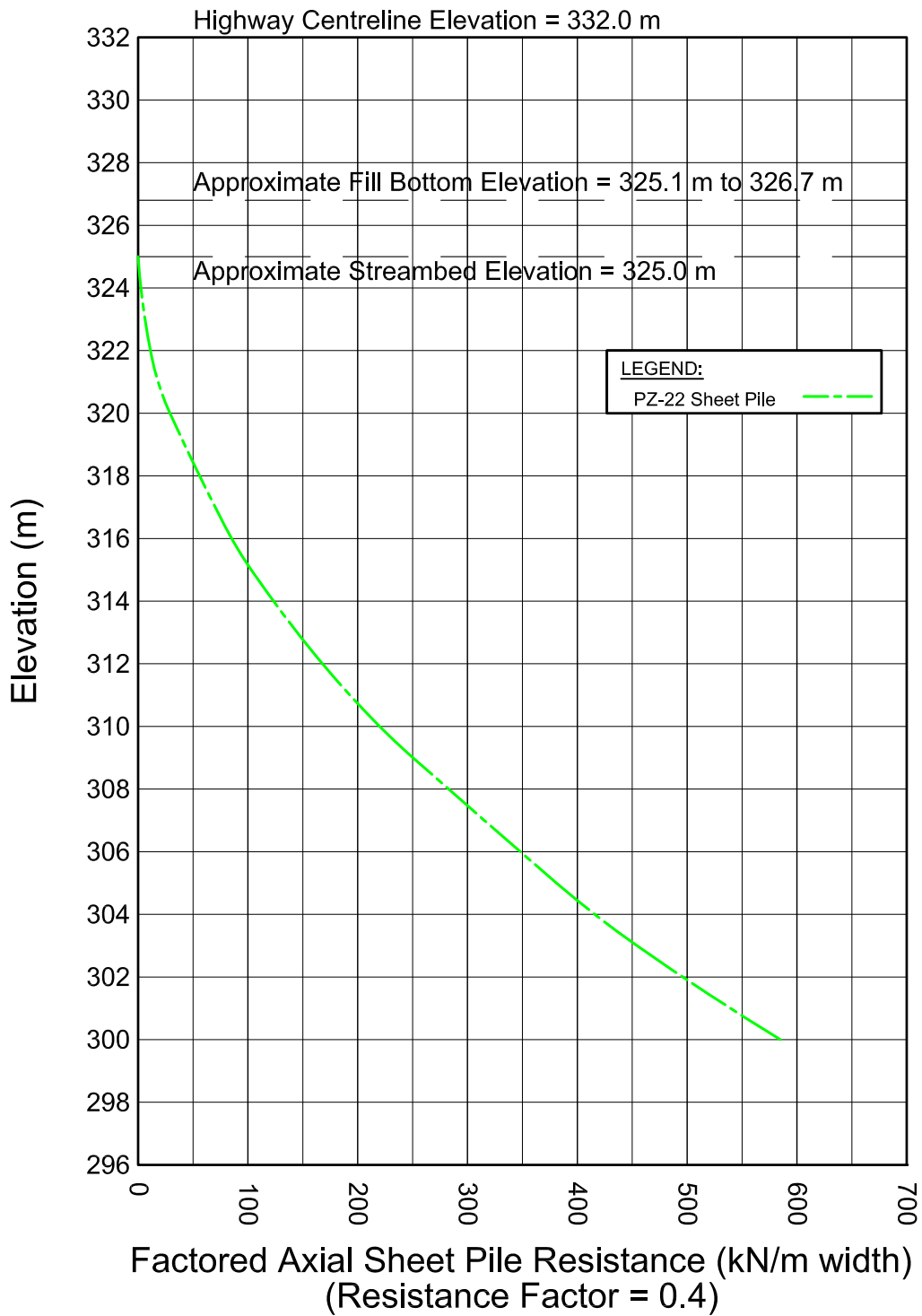
PROJECT NO.: ADM-00223648-A0

SCALE: N/A

DRAWN BY: RM

CHECKED BY: DG

DATE: February 25, 2015



Thunder Bay, Ontario

FIGURE
F2

Factored Axial Sheet Pile Resistance

Locking Creek Culvert, Site No. 45-161/C
Highway 602, Township of Lash, Ontario

PROJECT NO.: ADM-00223648-A0

SCALE: N/A

DRAWN BY: RM

CHECKED BY: DG

DATE: February 25, 2015

Highway 602 - Locking Creek Culvert Replacement
West Slope ~2H:1V
Drained Conditions

Proposed Tunnel Section
Sta. 10+640

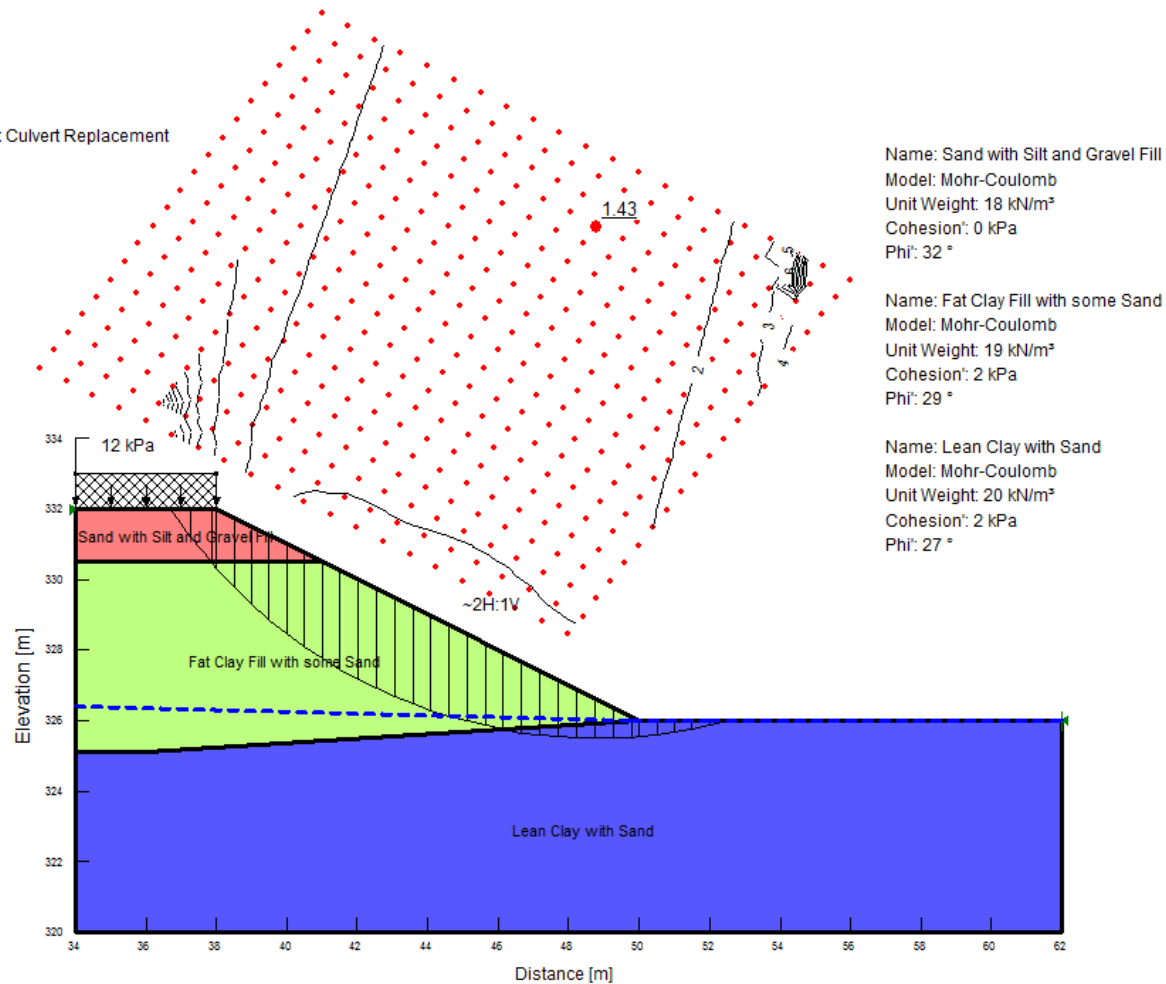


Figure F3. West side of embankment with ~ 2H:1V slope-current geometry, Effective stress analyses

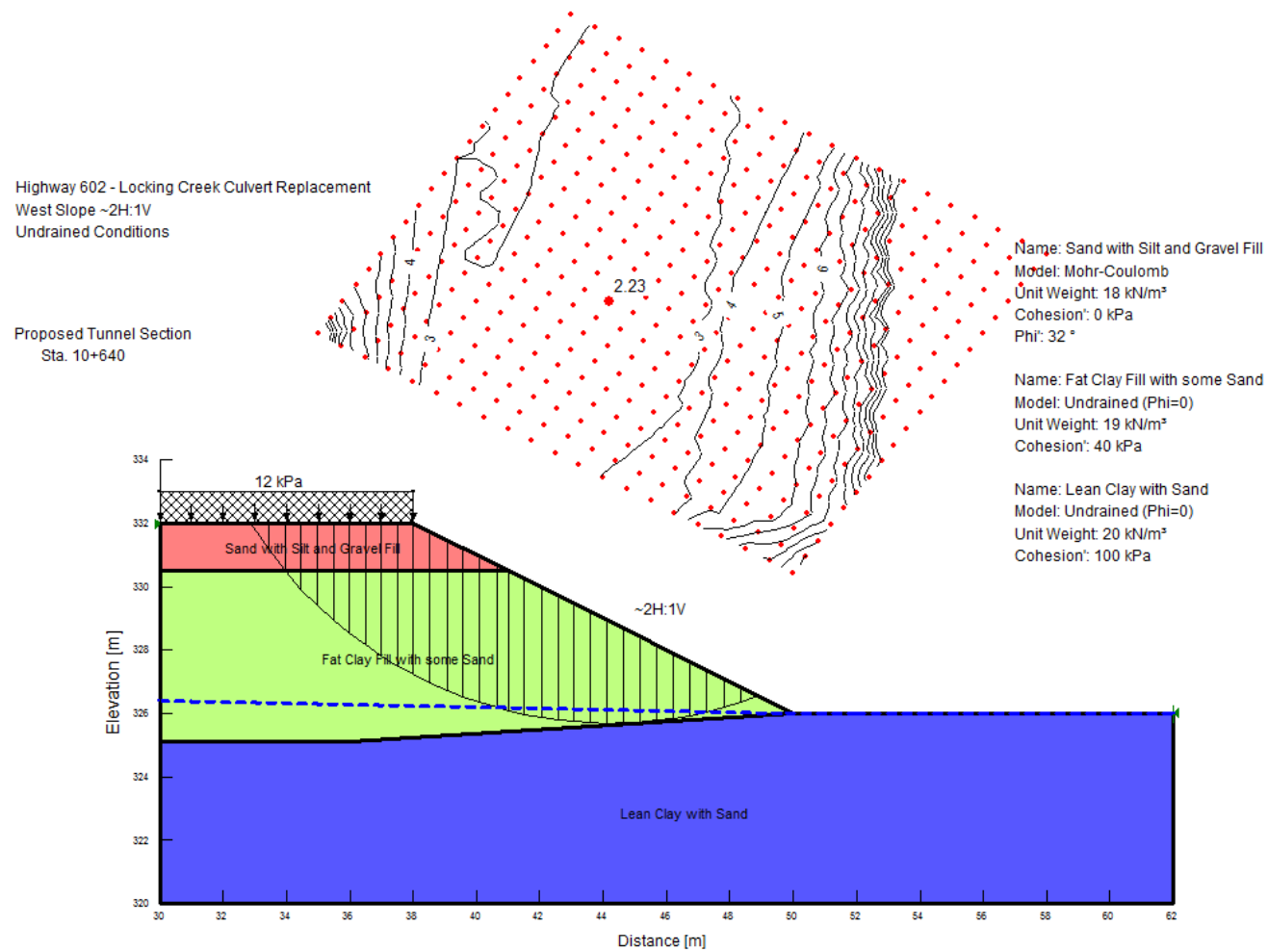
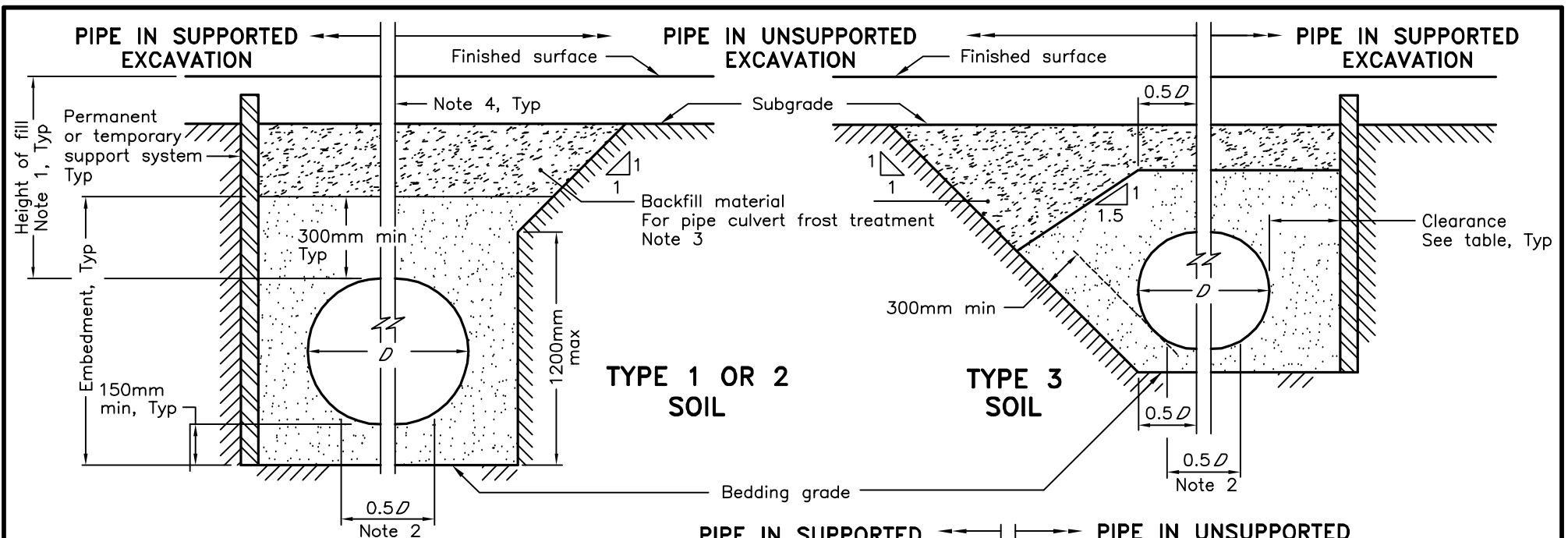


Figure F4. West side of embankment with ~ 2H:1V slope-current geometry, Total stress analyses

Appendix G – OPSDs

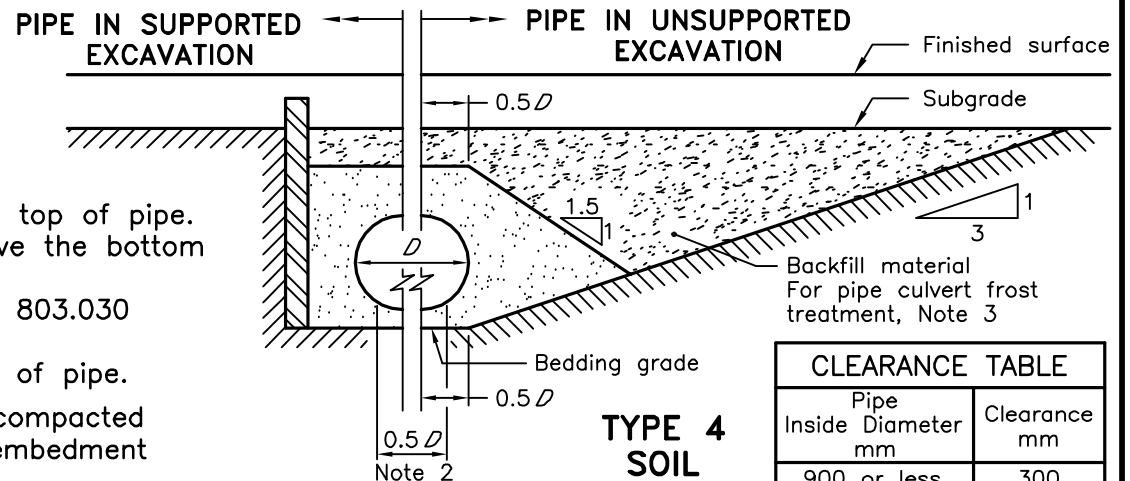


LEGEND:

D - Inside diameter

NOTES:

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



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

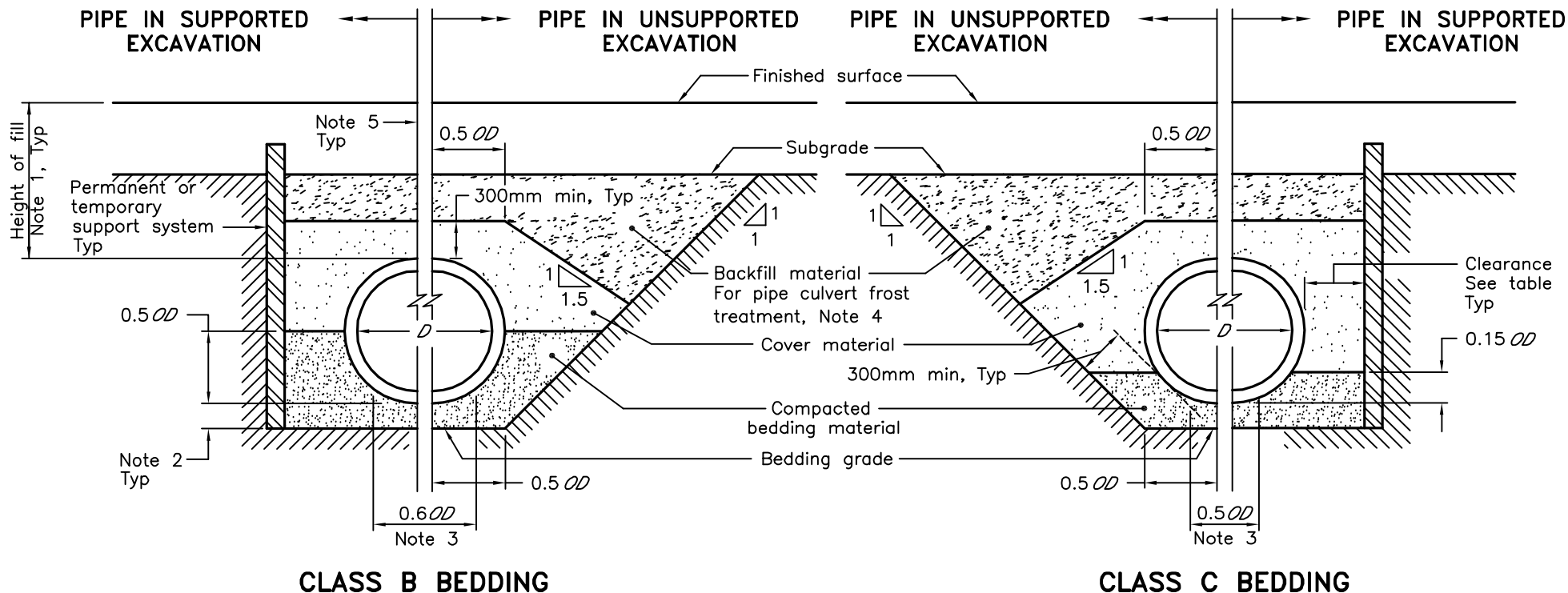
ONTARIO PROVINCIAL STANDARD DRAWING

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

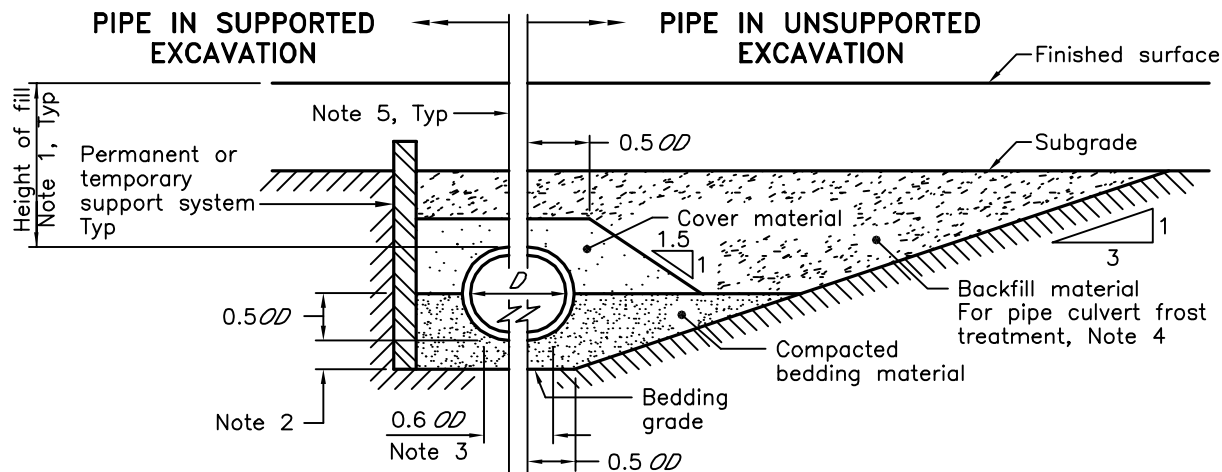
Nov 2010

Rev 2

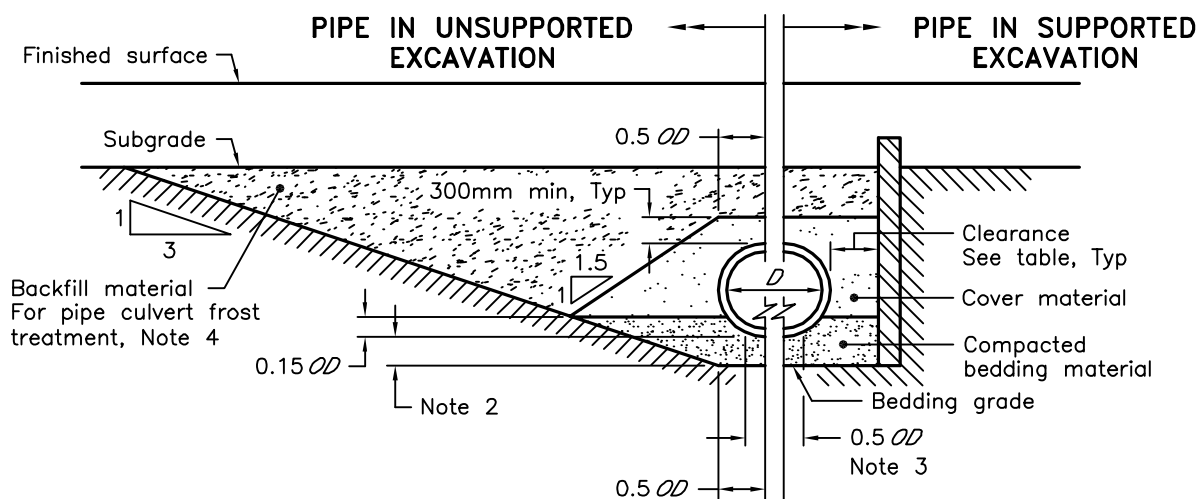
**RIGID PIPE BEDDING,
COVER, AND BACKFILL
TYPE 3 SOIL – EARTH EXCAVATION**

OPSD 802.031





CLASS B BEDDING



CLASS C BEDDING

LEGEND:

D – Inside diameter
 OD – Outside diameter

NOTES:

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

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

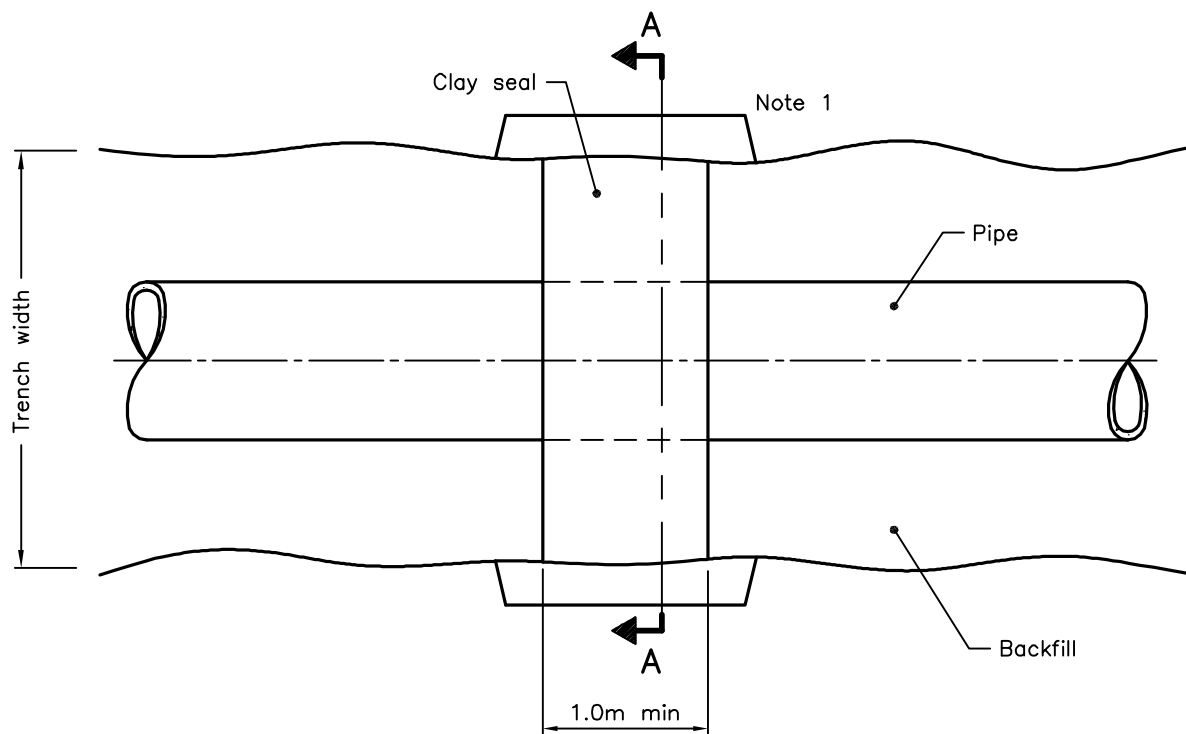
ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2010 Rev 2

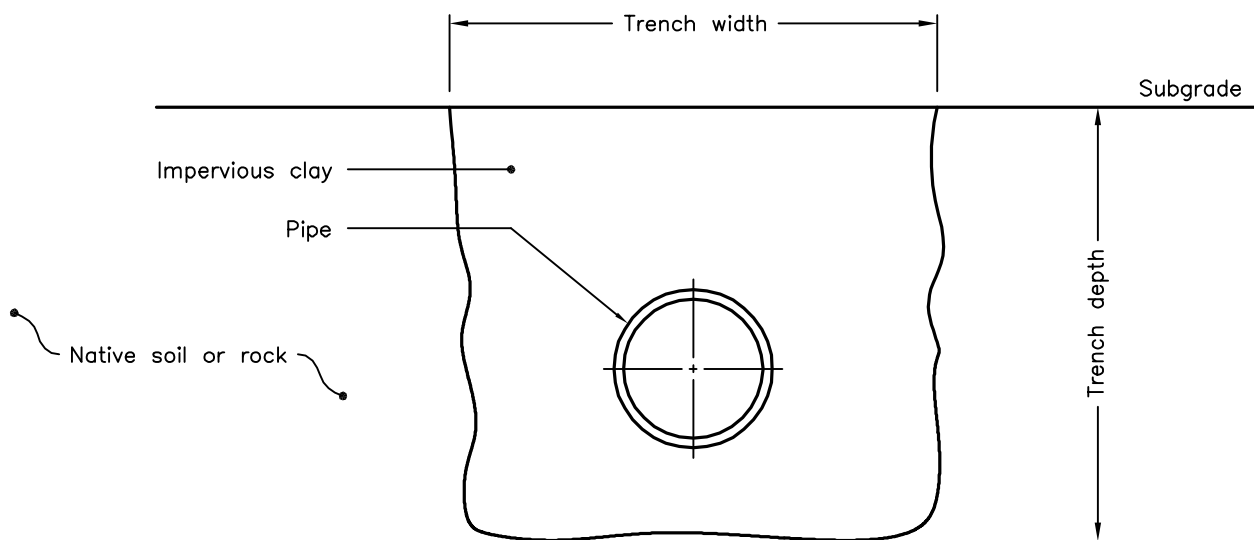
**RIGID PIPE BEDDING,
 COVER, AND BACKFILL
 TYPE 4 SOIL – EARTH EXCAVATION**

OPSD 802.032





PLAN



SECTION A-A

NOTES:

1. Key into undisturbed trench soil.

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

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

C All dimensions are in metres unless otherwise shown.

ONTARIO PROVINCIAL STANDARD DRAWING

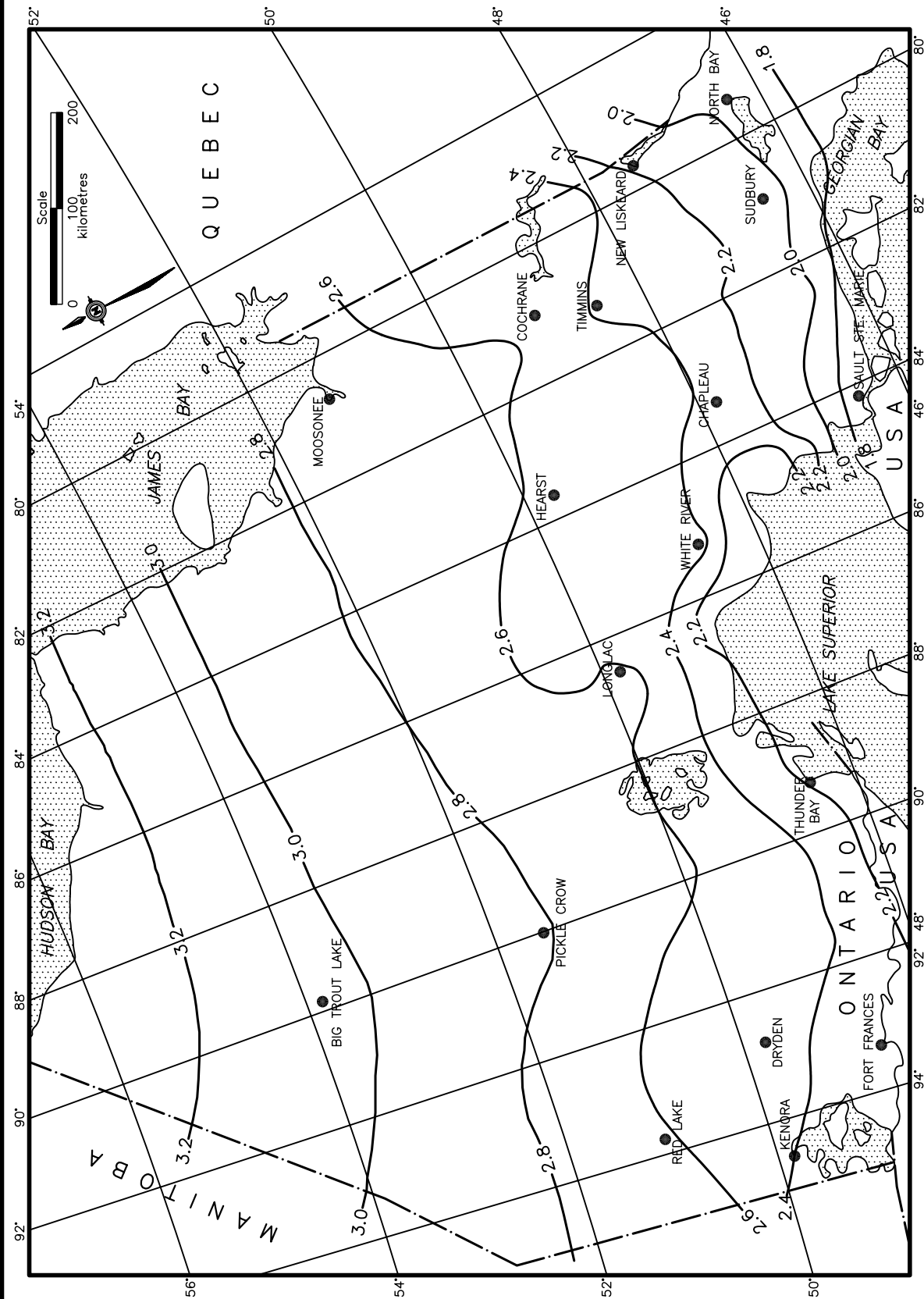
Nov 2011

Rev 1

CLAY SEAL FOR PIPE TRENCHES

OPSD 802.095





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 RR 225 "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 depths are in metres.

ONTARIO PROVINCIAL STANDARD DRAWING

FOUNDATION FROST DEPTHS FOR NORTHERN ONTARIO

Nov 2005

Rev 0

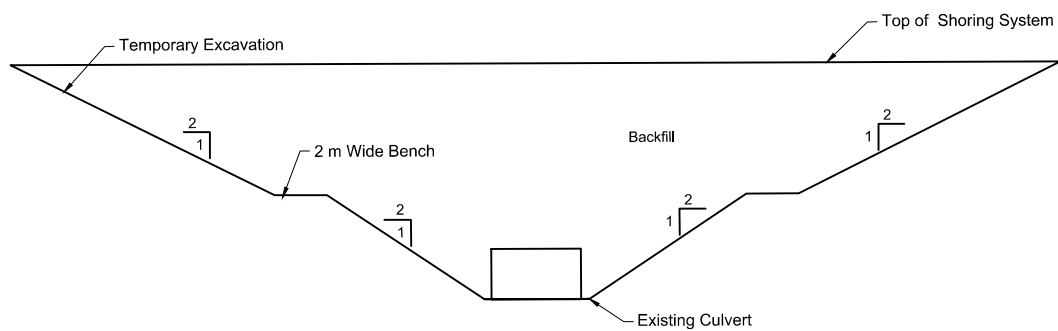
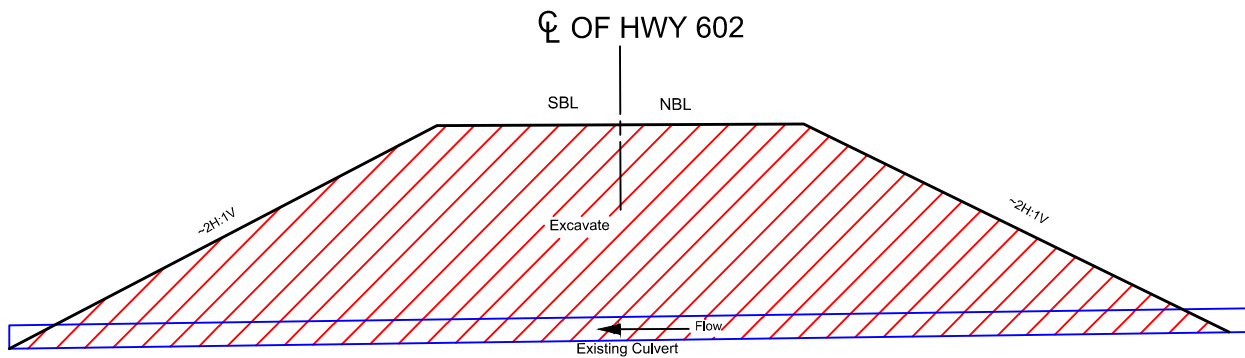
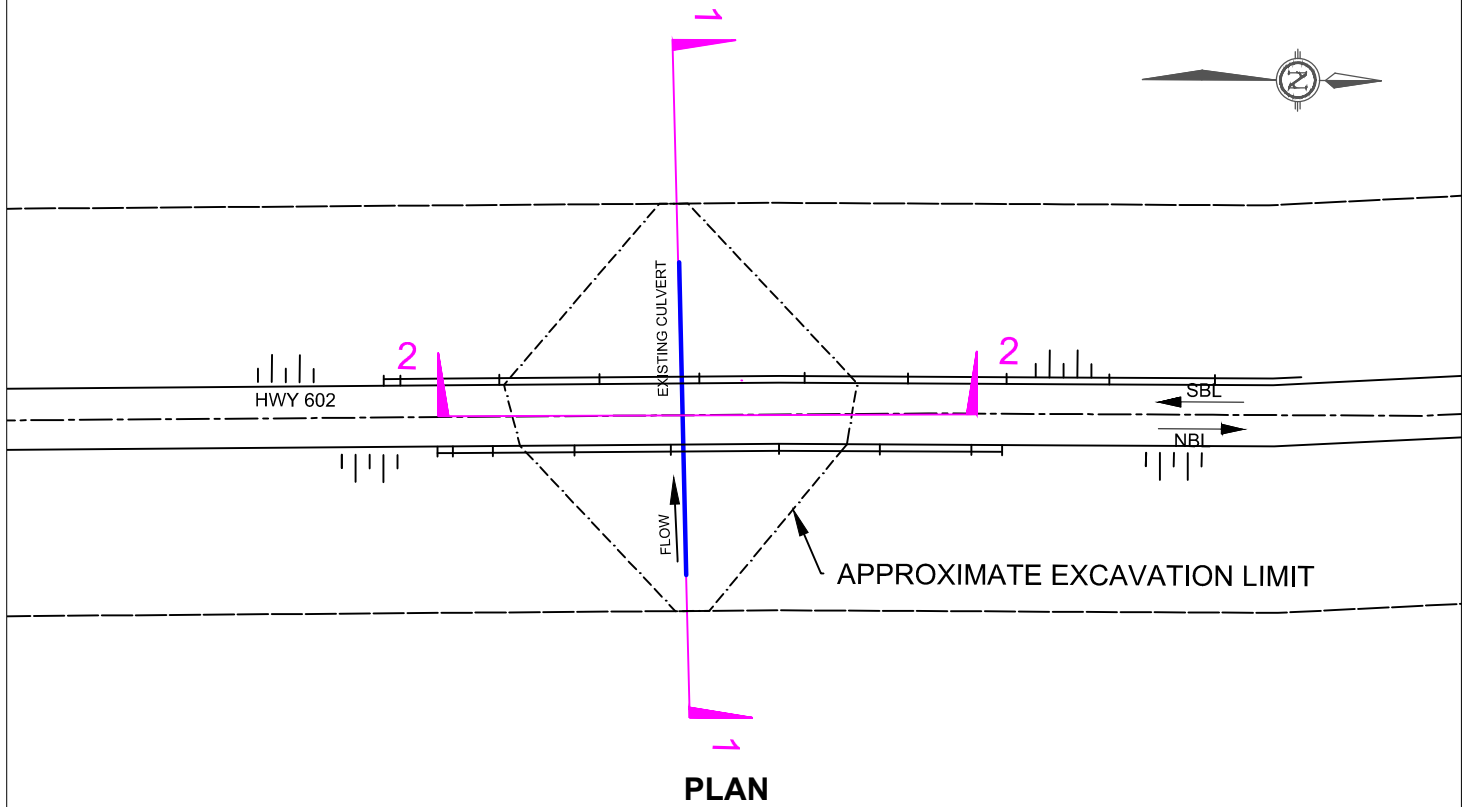


OPSD - 3090.100

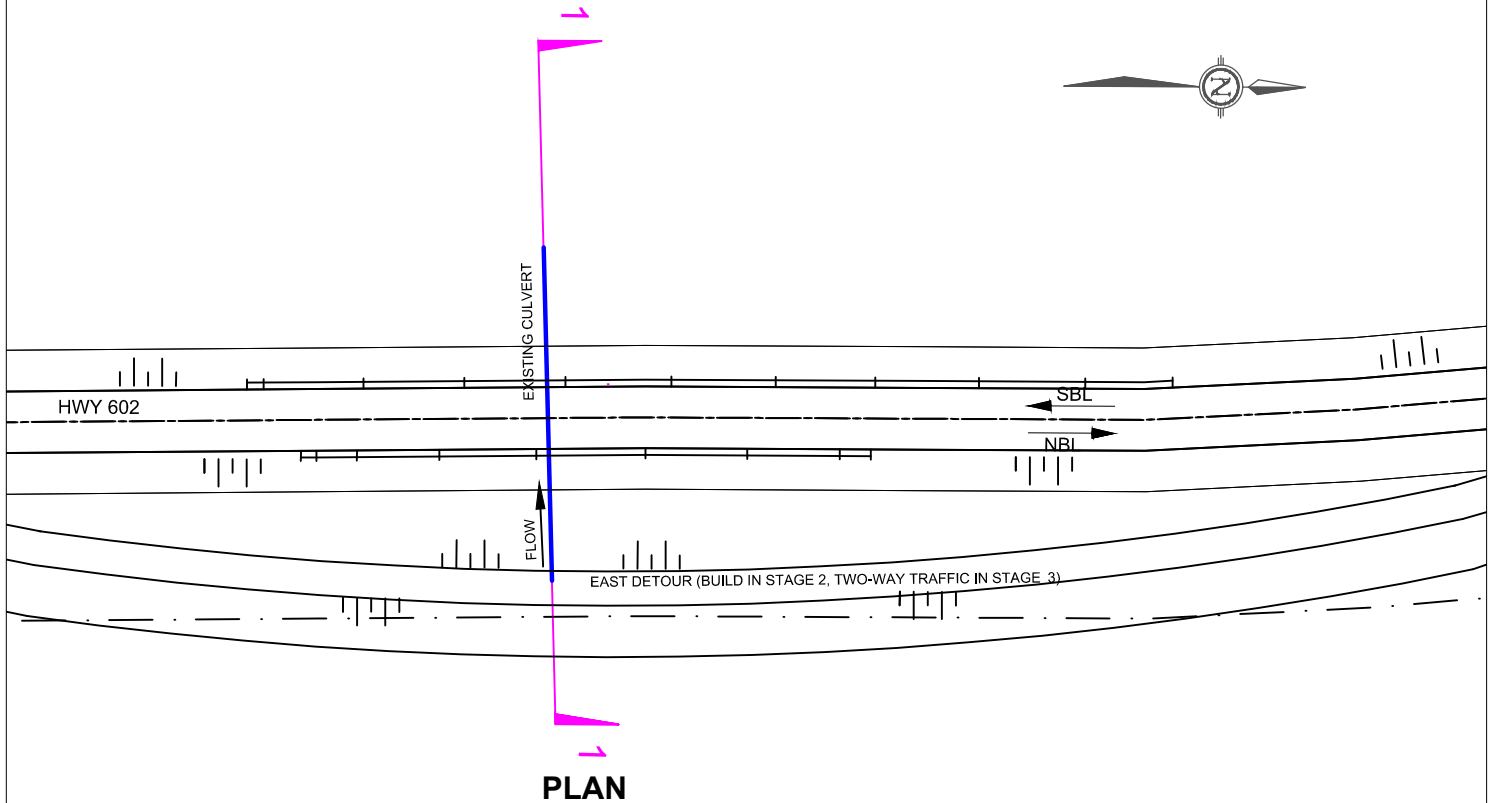
Appendix H – Schematic Sketches for Construction Alternatives

FIGURE H.1: FULL ROAD CLOSURE USING EXISTING ROADWAYS AND OPEN CUT UNSUPPORTED EXCAVATION (OPTION 1)

SCHEMATIC DIAGRAMS (NTS)

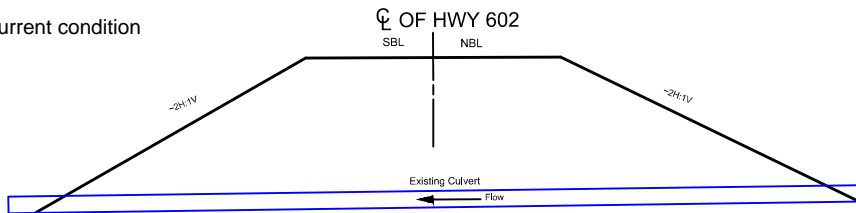


**FIGURE H.2: TEMPORARY LOCAL DETOUR AND OPEN CUT UNSUPPORTED EXCAVATION
(OPTION 2)**
SCHEMATIC DIAGRAMS (NTS)

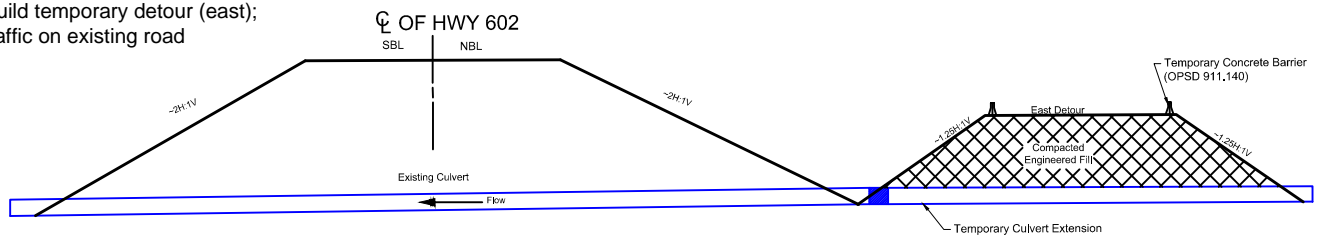


RECOMMENDED STAGES

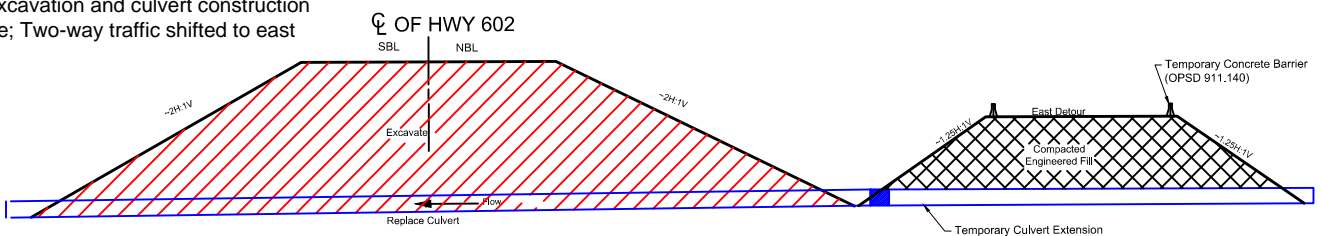
Stage 1 - Current condition



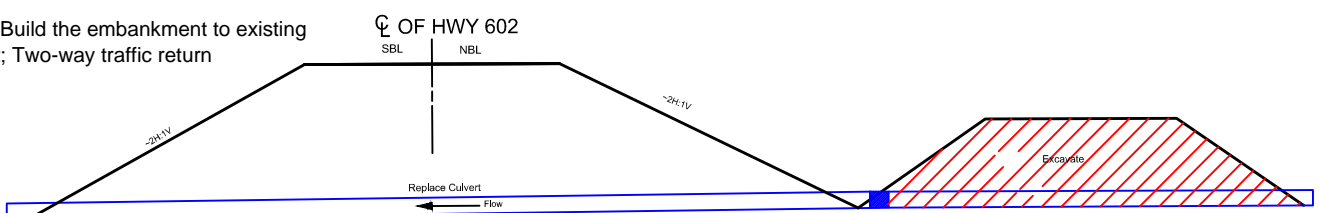
Stage 2 - Build temporary detour (east);
Two-way traffic on existing road



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



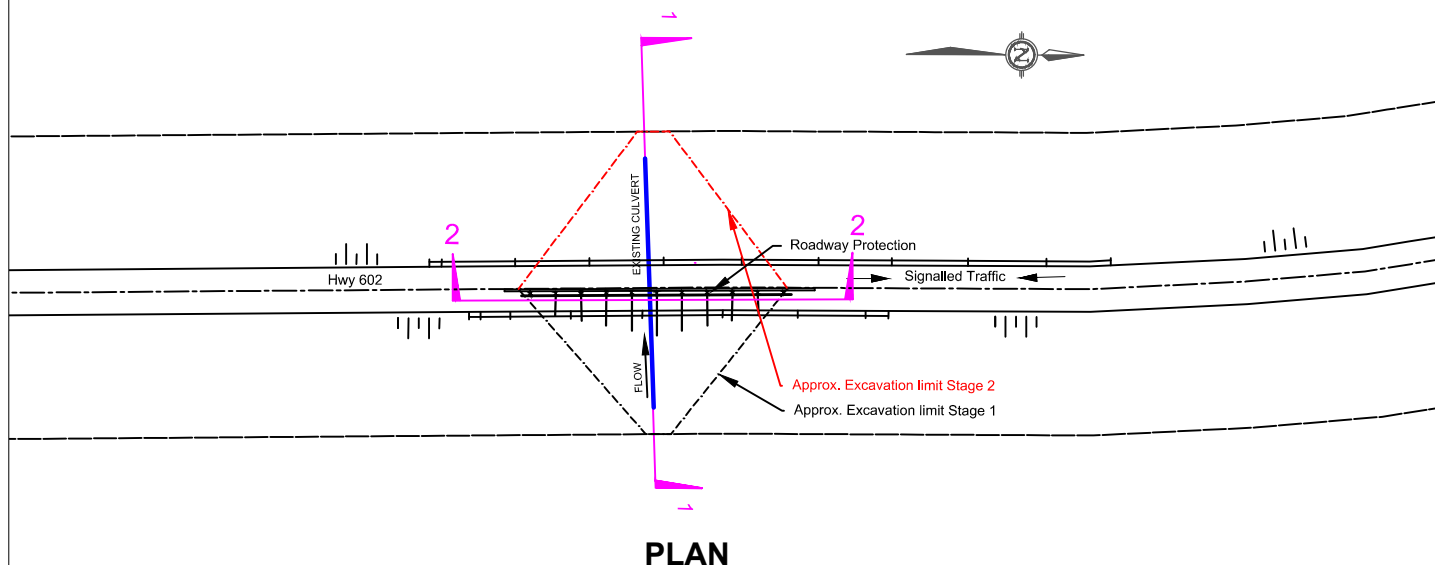
Stage 4 - Build the embankment to existing
alignment; Two-way traffic return



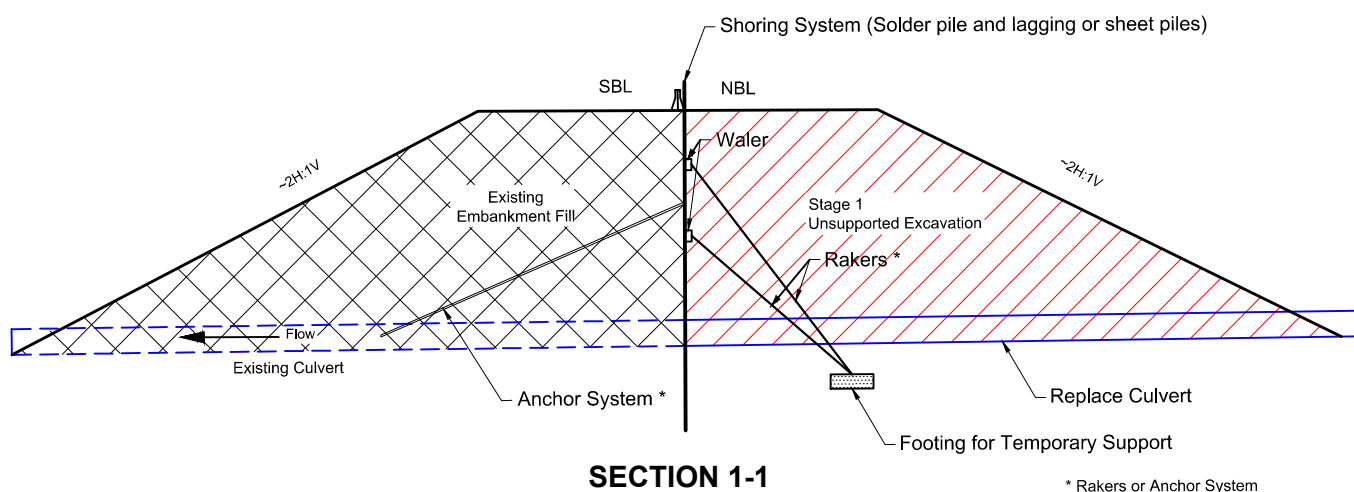
NOT TO SCALE

SECTION 1-1

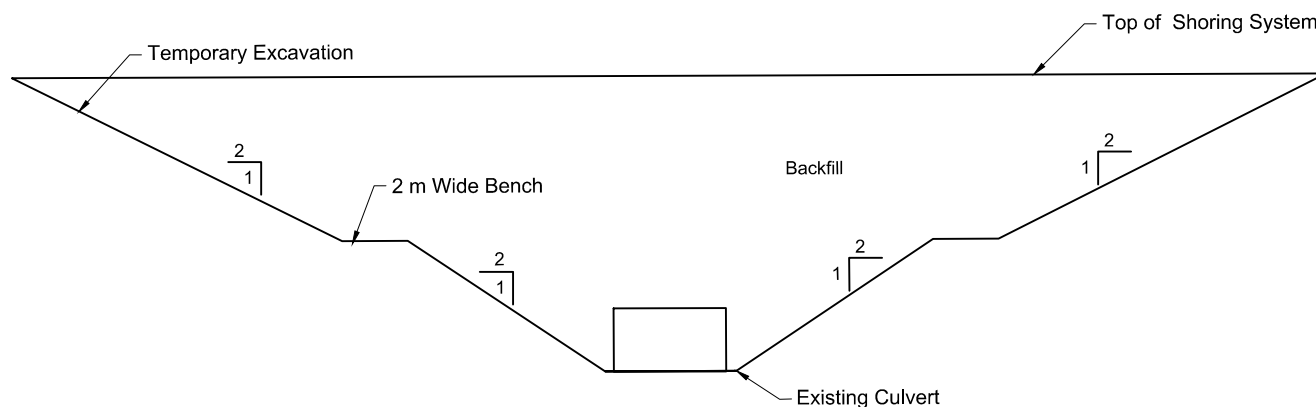
**FIGURE H.3.A: HALF AND HALF CONSTRUCTION WITH UNSUPPORTED CUT SIDES
(OPTION 3.A)
SCHEMATIC DIAGRAMS (NTS)**



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



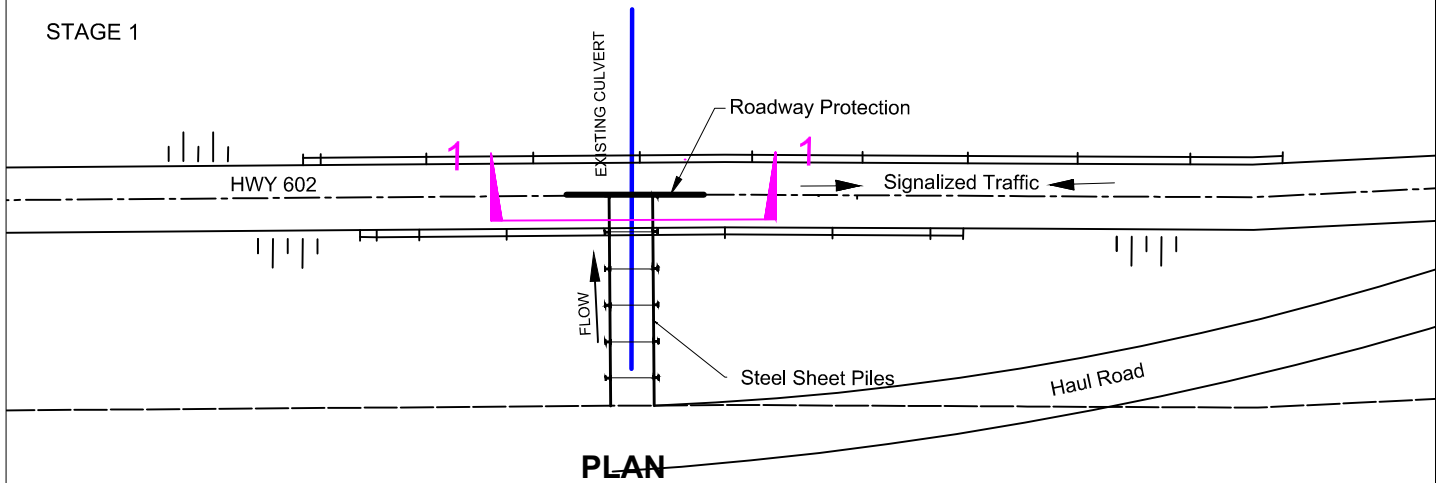
* Rakers or Anchor System



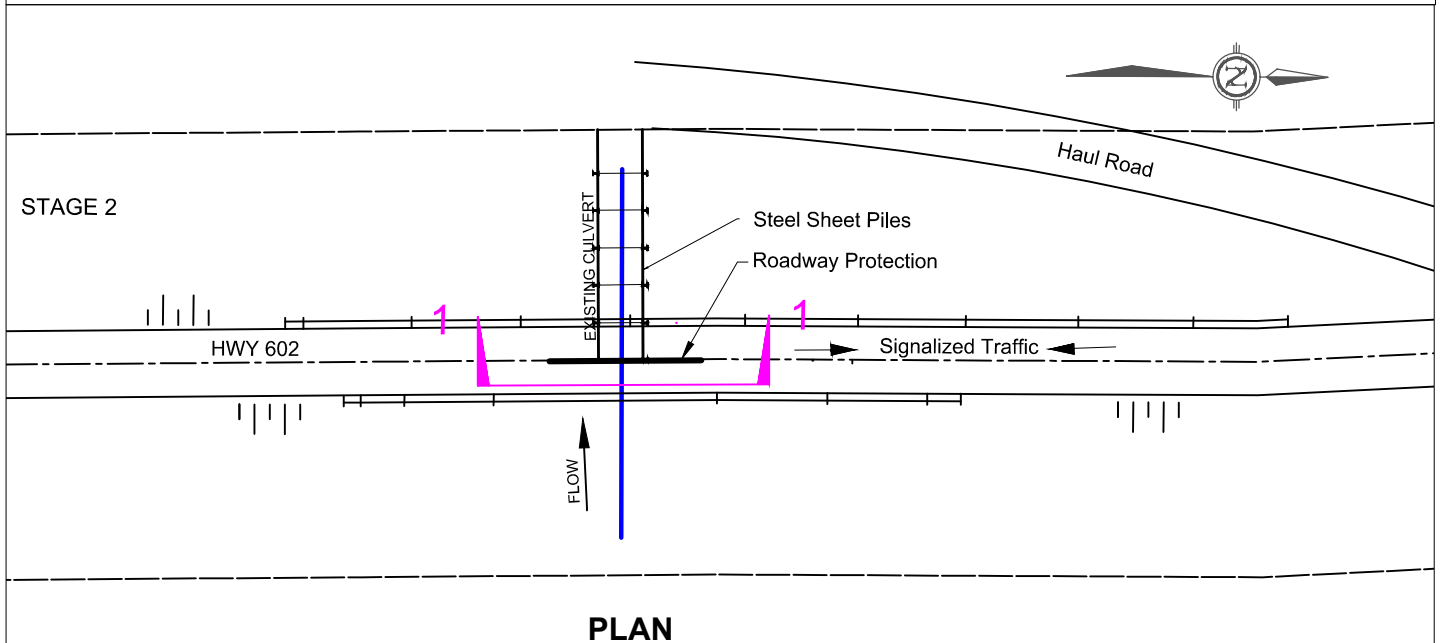
**FIGURE H.3.B: HALF AND HALF CONSTRUCTION WITH BRACED CUT SIDES
OR ANCHOR SYSTEM (OPTION 3.B)**
SCHEMATIC DIAGRAMS (NTS)



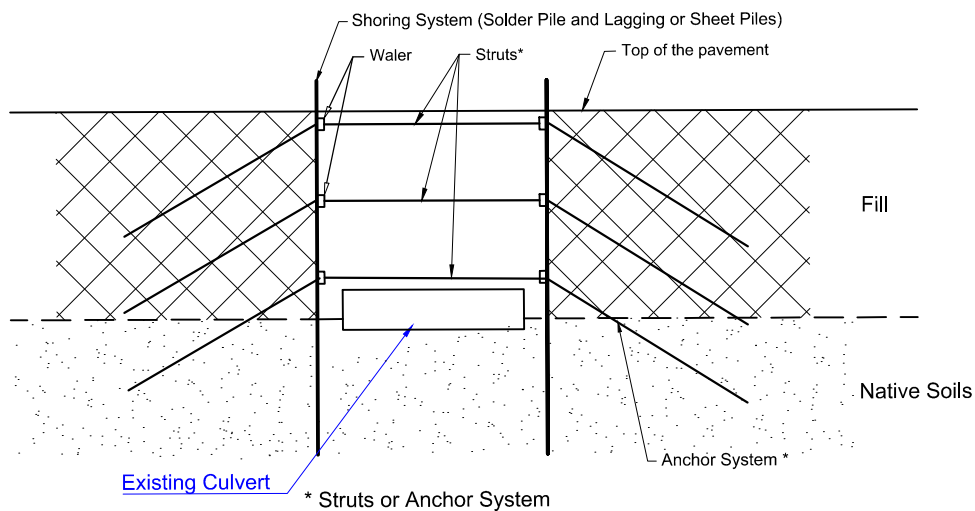
STAGE 1



STAGE 2



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



* Struts or Anchor System

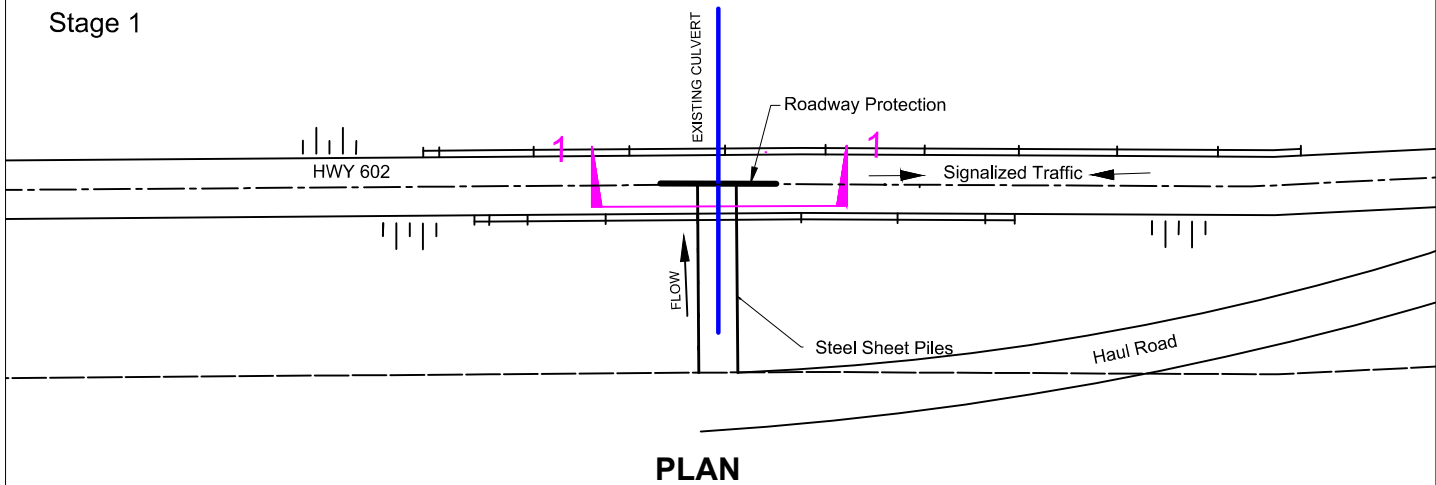
SECTION 1-1

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

SCHEMATIC DIAGRAMS (NTS)



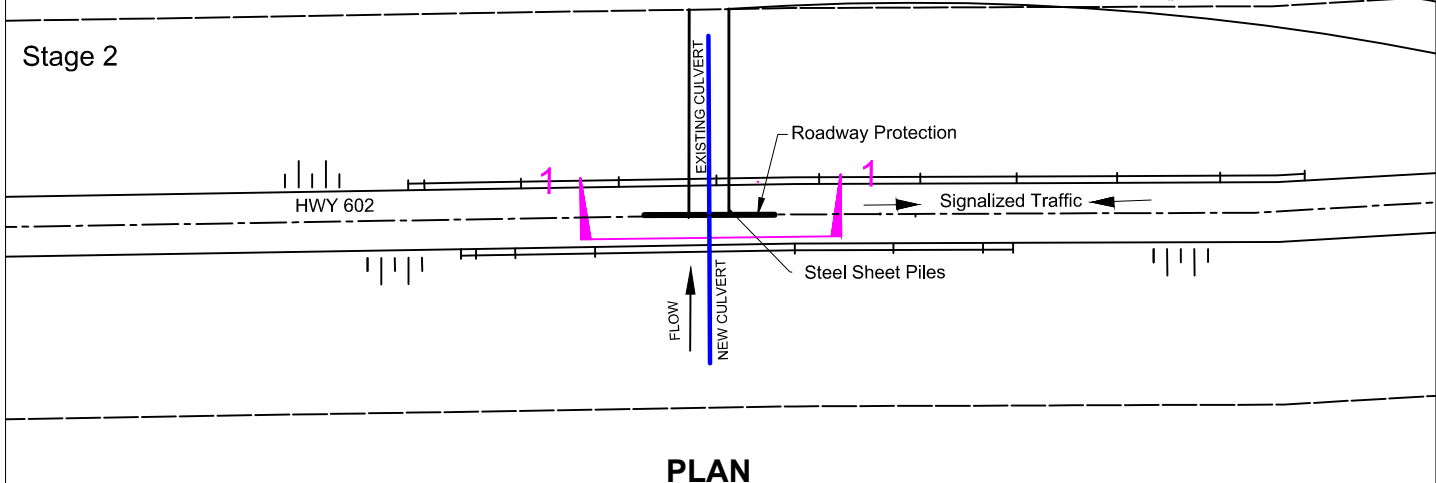
Stage 1



PLAN

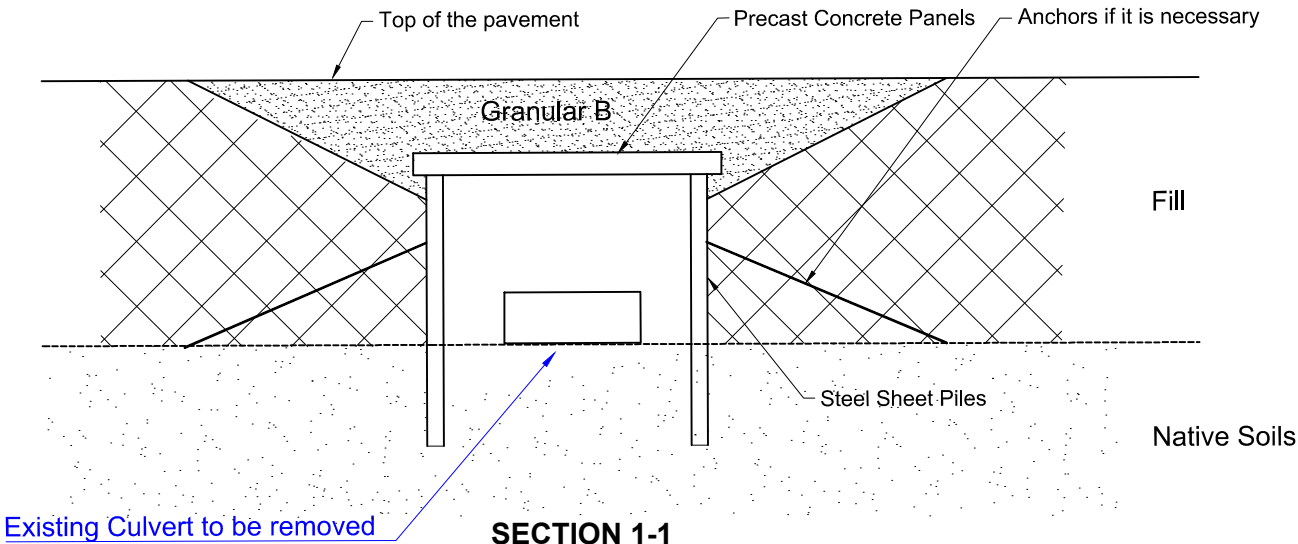


Stage 2



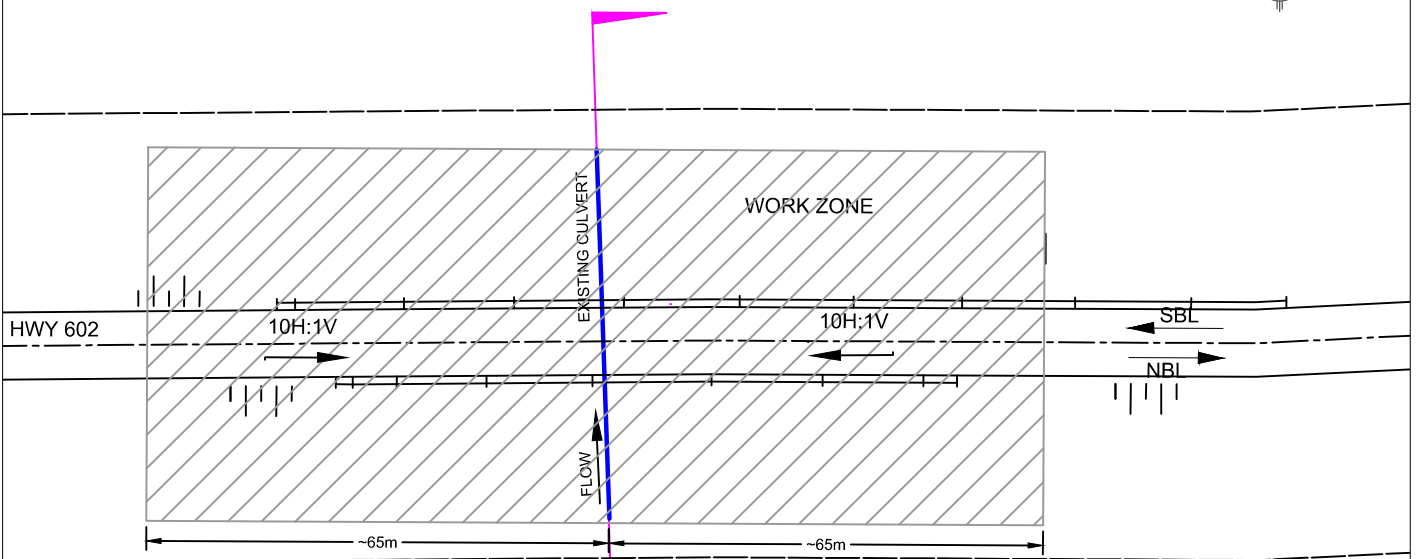
PLAN

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



SECTION 1-1

FIGURE H.4: STAGE CONSTRUCTION BY GRADE LOWERING (OPTION 4)
SCHEMATIC DIAGRAMS (NTS)

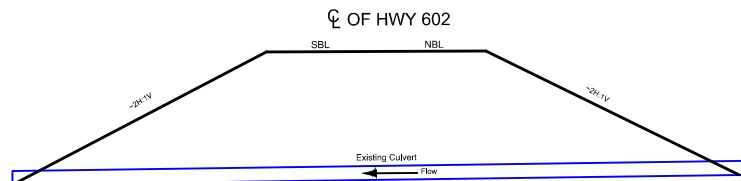


Note:
 Highway should be longitudinally excavated as the work zone shows in the plan using the slope 10H:1V.

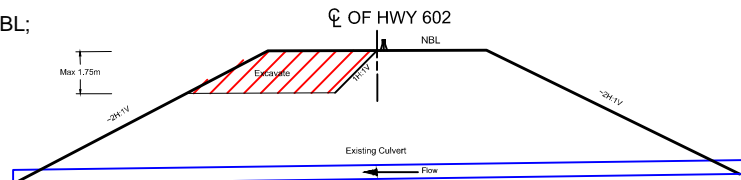
PLAN

RECOMMENDED STAGES

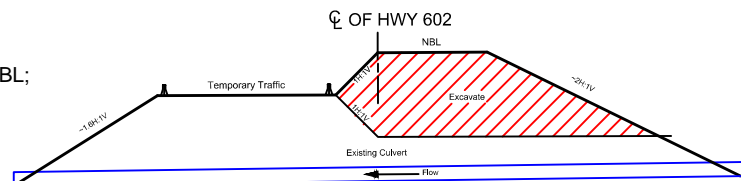
1.0 Stage 1 - Current condition



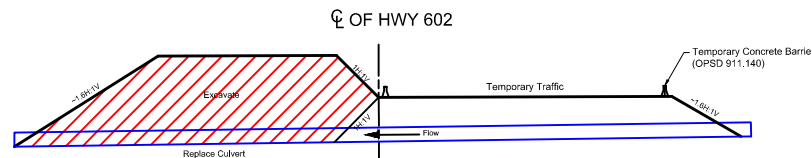
2.0 Stage 2 - Excavation on existing SBL;
 One-way traffic on existing road



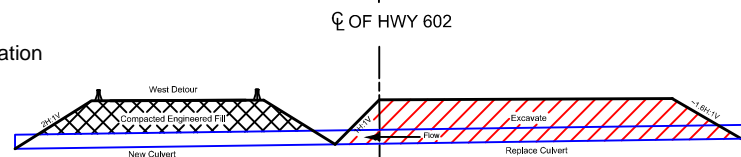
3.0 Stage 3 - Excavation on existing NBL;
 One-way traffic shifted to west side



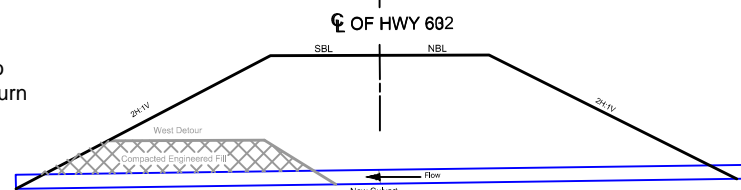
4.0 Stage 4 - Excavation and
 culvert construction on west side;
 One-way traffic shifted to east side



5.0 Stage 5 - Build west detour, excavation
 and culvert construction on east side;
 One-way traffic shifted to west detour



6.0 Stage 6 - Build the embankment to
 existing alignment; Two-way traffic return



SECTION 1-1