



## **FINAL REPORT**

**FOUNDATION INVESTIGATION AND DESIGN REPORT**  
**Nissiamkikan Creek Culvert Replacement, Site No. 48e-123/C, Hwy 11,**  
**Township of Walters, Ontario**  
(Latitude: 49°40'9.13"N; Longitude: 87°42'44.76"W)

**Agreement No. 6017-E-0066**  
**Assignment No. 2**  
**GWP 6561-00**  
**MTO GEOCRES No. 42E-31**

**Prepared for:**  
**Ontario Ministry of Transportation**  
Regional Director's Office -NW Region  
615 James Street South  
Thunder Bay, ON P7E 6P6  
Attn: Matthew Leavitt

**Ontario Ministry of Transportation**  
Pavements and Foundations Section  
Foundations Group  
Building 'C', Room 223, 2/F,  
145 Sir William Hearst Avenue,  
Downsview, ON M3M 0B6  
Attn: K. Ahmad

**exp Services Inc.**  
May 01, 2019

# Ontario Ministry of Transportation

## Northeastern Region Geotechnical Section

### Foundation Investigation and Design Report

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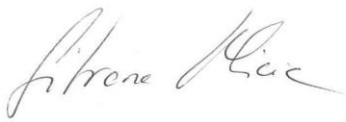
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Foundation Investigation and Design Report for Nissiamkikan Creek Culvert Replacement, Site No. 48E-123/C, Hwy 11, Township of Walters, Ontario

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**Prepared by:**  
Nimesh Tamrakar, M.Eng., P.Eng.  
Silvana Micic, Ph.D., P.Eng.

**Reviewed by:**  
TaeChul Kim, M.E.Sc., P.Eng.  
Stan E. Gonsalves, M.Eng., P.Eng.

**exp Services Inc.**  
56 Queen St, East, Suite 301  
Brampton, ON L6V 4M8  
Canada



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



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

**Date Submitted:**  
May 01, 2019

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# 1 FOUNDATION INVESTIGATION REPORT

## 1.1 Introduction

This report presents the results of a geotechnical investigation completed by **exp** Services Inc. for the replacement of Nissiamkikan Creek Culvert on Highway 11 (Lat: 49.6693; Long: -87.7126), Site No. 48E-123/C, located 19.2 km east of Highway 580 at Beardmore or 3.4 km west of Highway 801 in Nazah, within Walters Township, Ontario. The work was undertaken under Agreement No. 6017-E-0066, Assignment No. 2 (GWP 6561-00). The terms of reference (TOR) were as presented in Ministry of Transportation Ontario (MTO) email dated October 24, 2018.

The purpose of this investigation is: (i) to evaluate the subsurface condition along the proposed CSP culvert replacement alignment and the temporary widening area to provide detail design for the culvert replacement including review of all suitable replacement options; and (ii) assessment of the temporary widening areas bordering the north end of the culvert for staging purposes. The site specific geotechnical investigation consisted of a field investigation including visual inspections, drilling of boreholes, soil sampling, and laboratory testing.

This foundation investigation report has been prepared specifically and solely for the project described herein. It contains the factual results of the investigation and the laboratory testing completed for this project. Exp should be given the opportunity to confirm recommendation in the event of material change in scope of the project (concept, layout, levels) or delays exceeding 2 years from acceptance of report.

## 1.2 Site Descriptions and Geological Setting

### 1.2.1 Site Descriptions

#### 1.2.1.1 General

Nissiamkikan Creek Culvert replacement site is located on Highway 11, approximately 3.4 km west of Highway 801 in Township of Walters, Ontario. At the site, Highway 11 is a two-lane highway with a paved shoulders and cable guide rails on both sides. Based on the drawing provided by MTO, the roadway embankment at the culvert location is about 7.0 m high with side slopes of about 3.2H:1V for top 3.2 m and about 1.8H:1V for remaining embankment slope. The elevation of highway pavement centerline at the site is about 321.3 m.

Based on the information provided by MTO, the existing culvert is the twin corrugated steel pipe (CSP) culvert with each approximately 1.9 m wide by 2.6 m in height, with a total span of approximately 5.2 m and about 43 m long. From the TOR it is also understood that the existing culvert were constructed in 1899. The inverts of the culverts, as provided in the drawing, is at Elev. 313.52 m at the inlet and at Elev. 313.48 m at the outlet. Selected photographs of the site and existing culverts are presented in Appendix A. The site plan and cross-section profile are shown on Drawing 1 attached in Appendix B.

The general site conditions were assessed during the site visit on October 26, 2018 and drilling operations between December 10 and 14, 2018. However, during the drilling the surrounding area at the site location were covered by snow, so our observations were limited. Highway 11 generally runs in an east-west direction at the site location. The surrounding terrain is gently undulating with dense trees on both sides of highway. The overhead power lines were running parallel to highway at about 10 m south from the outlet end. The Nissiamkikan Creek flows in a north-south direction. At the time of investigation, the creek was frozen and the approximate creek ice levels at the inlet and outlet were about Elev. 314.5 m and 314.4 m, respectively.

## 1.2.2 Geological Setting

According to the Ministry of Northern Development and Mines, Map 2554 (Quaternary Geology of Ontario, West-Central Sheet, 1991) the surface conditions in the vicinity of the project area consists of glaciofluvial outwash deposits gravel and sand which includes proglacial river and deltaic deposits or organic deposits which includes peat, muck and marl. According to Map 2542 (Bedrock Geology of Ontario, West-Central Sheet, 1991) the surficial soils are underlain by undifferentiated igneous and metamorphic rocks. The bedrock geology of the site is of metasedimentary rocks comprised of wacke, arkose, argillite, slate, marble, chert, iron formation to mafic to intermediate metavolcanics rocks comprised of basaltic and andesitic flows, tuffs and breccias, chert, iron formation.

## 1.3 Investigation Procedures

### 1.3.1 Site Investigation and Field Testing

The field investigation was performed between December 10 and 14, 2018. Initially the investigation program was proposed of two phases:

**Phase 1:** drilling of 6 boreholes strategically located at the culvert location and along the widening embankment area to investigate and verify the consistency of the subsurface condition across the site. Upon completion of this phase, it was proposed to report the results to MTO for review and to decide if the drilling in Phase 2 was required.

**Phase 2:** drilling additional 4 boreholes.

After the completion of Phase 1 and review of the obtained results, it was decided by MTO and **exp** that Phase 2 was not necessary.

Therefore, the final field program consisted of drilling six (6) sampled boreholes (BH-1, BH-2, BH-3, BH-4, BH-5 and BH-6) which were strategically located along the existing culvert alignment and along the proposed temporary widening area to provide subsurface information for the design of proposed new culvert and for the design of embankment widening. Table 1.1 summarizes location, elevation and depth of these boreholes. BH-1 and BH-3 were advanced at the toe of the embankment on inlet and outlet side of the existing culvert, respectively. BH-2 was advanced from the embankment crest. BH-4, BH-5 and BH-6 were advanced off the road on the north side of the highway, where BH-4 and BH-5 were advanced about 110 m and 40 m west of the existing twin culverts centerline, respectively and BH-6 was advance about 113 m east of the existing twin culverts centerline. The borehole locations are shown on Drawings No. 1 in Appendix B. Table 1.1 below listed the approximate borehole locations, elevations and investigated depths.

Table 1.1 Locations, elevations and depths of BHs

BH #	Location	MTM NAD83 Northing	MTM NAD83 Easting	Ground Elevation (m)	Borehole Depth (m)
BH-1	North end (Inlet) at approx. STA 13+895	5503777.6	253365.8	316.3	7.7
BH-2	Embankment Crest at approx. STA 13+912	5503759.5	253381.5	321.3	15.5
BH-3	South end (outlet) at approx. STA 13+917	5503716.3	253385.1	316.1	8.4
BH-4	North side of Hwy at approx. STA 13+795	5503769.4	253264.3	320.9	7.0
BH-5	North side of Hwy at approx. STA 13+864	5503771.2	253333.1	319.2	4.0
BH-6	North side of Hwy at approx. STA 14+019	5503764.5	253488.0	319.9	4.8

All of the drilled boreholes were advanced using a Morooka rubber track mounted Acker MP-5 drill rig, equipped with hollow stem augers and standard soil sampling equipment operated by a specialist drilling contractor, Maple Leaf Drilling Ltd. The roadway borehole (BH-2) was advanced to depth of about 15.5 m below the ground surface, while all other off-road boreholes were advanced to a depth ranging between 4 m to 8.4 m below the ground surface.

The borehole locations (referenced to the MTM NAD83 coordinate system) and their ground surface elevations were surveyed by **exp** personnel using a handheld GPS (accuracy +/- 3m horizontal) and a level survey (accuracy +/-2mm vertical). The ground surface elevations were referenced to temporary benchmarks set up at a central line of Highway 11 along existing culvert alignment. The elevation of the TBM (321.31 m) was assumed based on the provided cross-section drawing. The temporary benchmark location is shown on Drawing No. 1 in Appendix B.

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

Upon completion of the boreholes, groundwater level measurements were carried out in open boreholes in accordance with MTO guidelines. The recorded groundwater levels after completion of drilling boreholes were presented in the borehole log sheets in Appendix C. The roadway boreholes were decommissioned by bentonite/cement mixtures in accordance with the Ministry of the Environment Regulation 903, as amended by Regulation 128/03 (the well regulation under the *Ontario Water Resources Act*).

The fieldwork was supervised by the **exp** geotechnical representative who directed the drilling and sampling operations, 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 recovered soil samples were placed in labelled moisture-proof bags and returned to **exp's** Thunder Bay laboratory for additional visual, textual and olfactory examination and selective testing.

### 1.3.2 Previous Investigation

No foundation reports are available in the MTO GEOCRES library for this particular site.

### 1.3.3 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 on all samples and particle size distribution tests for approximately 25% of the collected soil samples. All of the laboratory tests were carried out according to MTO and/or ASTM Standards as appropriate.

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

## 1.4 Subsurface Conditions

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

Borehole location plan and cross section subsurface profiles are provided in Appendix B. It should be noted that the stratigraphic boundaries indicated on the borehole logs and cross section stratigraphic profiles 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 regarded 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 site and embankment widening area consist of embankment fill layers (i.e. rockfill: crushed gravel to gravelly sand fill)/ rootmat underlain by native deposit of silty sand/silt followed by silty sand and gravel. A detailed description of the subsurface conditions encountered along the proposed culvert location and embankment widening area is discussed further in subsequent sections.



### 1.4.1 Asphalt

Asphalt was encountered at the surface of BH-2, which was advanced through the crest of the existing highway embankment. The thickness of asphalt was about 0.125 m. Asphalt thicknesses may further vary beyond the borehole locations.

### 1.4.2 Topsoil/Sandy Rootmat

Topsoil/sandy rootmat was encountered at the surface of all off-road boreholes except BH-3. The thickness of this layer ranges from about 0.1 m to 0.2 m.

The composition of this layer consists of roots and rootlets, trace to some organics, trace to some sand, trace to some silt and trace gravel. The material is dark brown to black in color, moist to wet and loose.

Laboratory testing performed on selected samples consisted of moisture content tests. The test results indicate that natural moisture content of this material ranges from 12.9% to 41.3%. The results of the moisture content tests are provided on the record of borehole sheets in Appendix C.

### 1.4.3 Fill: Gravelly Sand

A layer of gravelly sand fill was encountered below the asphalt in BH-2. It was also encountered below the rockfill in BH-2. The upper gravelly sand fill layer extended to depth of about 0.6 m below the ground surface with elevation about 320.7, whereas, the lower gravelly sand fill layer extended from a depth of about 3.7 m with elevation about 317.6 m to depth of about 5.3 m with elevation about 315.9 m. The explored thickness of this upper and lower gravelly sand fill layers were about 0.5 m and 1.6 m, respectively.

The composition of this fill layers were generally sand and gravel with some silt and trace asphalt inclusions. The material is brown in color, and moist. The SPT 'N' values obtained within this layers ranged from 34 to 40 blows per 0.3 m penetration, suggesting dense in relative density.

Laboratory testing performed on selected samples consisted of seven (5) moisture content tests and one (1) grain size distribution test. The test results are as follows:

Moisture Contents:

- 2.3% to 6.1%

Grain Size Distribution:

- 27% gravel;
- 59% sand; and
- 14% silt and clay

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

#### 1.4.4 Rockfill: Crushed Gravel

A layer of rockfill was encountered between the upper and lower gravelly sand fill layers in BH-2. The rockfill layer extended to depth of about 3.7 m below the ground surface with elevation of about 317.6 m. The explored thickness of this layer was about 3.1 m.

The composition of rockfill consists of crushed gravel, some sand and some silt. Where it was possible the split spoon tests attempted to collect samples within layer. However, in majority cases the adequate samples were not able to be recovered in the split spoon. Auger samples collected during borehole advance were brown in color. One SPT "N" value recorded within this layer was 2 blows per 0.3 m penetration. The SPT "N" value recorded within rockfill cannot be representative of actual compactness condition of rockfill. However, for information purpose, the obtained SPT "N" value within this part of the layer matrix suggesting very loose compactness condition.

Laboratory testing performed on selected samples consisted of moisture content tests. The test results indicate that the natural moisture content of this material ranges from 1.5% to 1.9%. The result of the moisture content test is provided on the record of borehole sheets in Appendix C.

#### 1.4.5 Fill: Silty Sand

A layer of silty sand fill was encountered at the surface of BH-3. The fill layer extended to depth of about 1.5 m below the ground surface with elevation 314.5 m. The explored thickness of this fill layer was 1.5 m.

The composition of this layer consist of silt and sand, trace gravel, trace organics, trace roots and rootlets. The material is brown in color, and moist to wet. The SPT 'N' values obtained within this layer ranged from 12 to 25 blows per 0.3 m penetration, suggesting compact in relative density.

Laboratory testing performed on selected samples consisted of two (2) moisture content tests and one (1) grain size distribution test. The test results are as follows:

Moisture Contents:

- 18.6% to 20.1%

Grain Size Distribution:

- 4% gravel;
- 70% sand; and
- 26% silt and clay

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

### 1.4.6 Creosote Wood

Buried creosote wood was encountered below the silty sand fill layer in BH-3. Creosote wood was extended to depth about 2.3 m below the ground surface with elevation 313.8 m, suggesting its thickness of 0.8 m.

Encountered buried wood was oily with strong creosote odours. One SPT sampler was attempted and the the SPT 'N' value recorded within this wood was 107 blows per 0.275 m penetration. The borehole was advanced auguring through the wood material.

### 1.4.7 Silty Sand/Sand

A native silty sand/sand deposit was encountered below the sandy rootmat layer BH-1, BH-5 and BH-6 and below creosote wood in BH-3. The silty sand/sand deposit was extended to depths ranging between 0.8 m to 3.1 m below the ground surface with elevations ranging between 313.0 m to 318.5 m. The explored thickness of this layer was between 0.7 m and 2.1 m.

The composition of this native layer was generally silt and sand with trace to some gravel, trace rootlets, trace peat. The material is light brown to dark brown in color, and moist to wet. The SPT 'N' values obtained within this layer was ranged from 11 to 25 blows per 0.3 m penetration, suggesting compact in relative density.

Laboratory testing performed on a selected sample consisted of seven (7) moisture content tests and three (3) grain size distribution tests. The test results are as follows:

Moisture Content:

- 7.2% to 26.8%

Grain Size Distribution:

- 0% to 14% gravel;
- 57% to 95% sand;
- 5% to 29% silt and clay

The results of the moisture content and grain size distribution tests are provided on the record of borehole sheets in Appendix C. The result of grain size distribution test is also provided on Figure 3 in Appendix D.

### 1.4.8 Sand and Silt

A native sand and silt deposit was encountered sandwiched between the silt deposit in BH-4. The sand and silt deposit was extended to depth about 2.3 m below the ground surface with elevation about 318.6 m. The explored thickness of this layer was about 1.5 m.

The composition of this layer was generally silt and sand. The material is brown in color, and wet. The SPT 'N' values obtained within this layer was ranged from 9 to 16 blows per 0.3 m penetration, suggesting loose to compact in relative density.

Laboratory testing performed on a selected sample consisted of two (2) moisture content tests and one (1) grain size distribution test. The test results are as follows:

Moisture Content:

- 23.98% to 26.6%

Grain Size Distribution:

- 0% gravel;
- 46% sand;
- 54% silt and clay

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

### 1.4.9 Silt

A native silt deposit was encountered below the silty sand/sand deposit in BH-1, BH-3, BH-5 and BH-6; below fill in BH-2 and below topsoil/ sand and silt in BH-4. The silt deposit extended to depths ranging between 4.0 m and 13.7 m below the ground surface with elevations ranging between 307.5 m and 315.2 m. BH-4 to BH-6 were terminated within this layer. The explored thickness of this layer was between 3.2 m and 8.4 m.

The composition of this layer was silt with trace to some sand, trace to few clay, trace organics. The material is light brown to grey in color, and wet. The SPT 'N' values obtained within this layer ranged from 5 blows per 0.3 m to 50 blows per 0.05 m penetration, suggesting loose to very dense, but generally loose to compact in relative density.

Laboratory testing performed on selected samples consisted of thirty two (32) moisture content tests, ten (10) grain size distribution tests and five (5) Atterberg Limit tests. The test results are as follows:

Moisture Content:

- 15.6% to 24.2%

Grain Size Distribution:

- 0% gravel;
- 0% to 38% sand;
- 55% to 94% silt;
- 6% to 15% clay; and
- 97% to 99% silt and clay

Atterberg Limits:

- Liquid Limit: 18% to 22%
- Plastic Limit: 17% to 20%
- Plasticity Index: 1% to 3%

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 grain size distribution are also provided on Figures 5 and 6 and the results of Atterberg Limit tests are provided on Figure 9 in Appendix D.

#### **1.4.10 Silty Sand and Gravel**

A native silty sand and gravel deposit was encountered below the silt deposit in BH-1 and BH-2. The silty sand and gravel deposit extended to depths ranging between 7.7 m and 15.5 m below the ground surface with elevations ranging between 305.8 m and 308.6 m. BH-1 and BH-2 were terminated within this layer. The explored thickness of this layer was between 1.0 m and 1.8 m.

The composition of this layer was silt and sand with some gravel. The material is grey in color, and wet. The SPT 'N' values obtained within this layer ranged from 50 blows per 0.075 m to 100 blows per 0.225 m penetration, suggesting very dense in relative density.

Laboratory testing performed on selected samples consisted of three (3) moisture content tests and one (1) grain size distribution test. The test results are as follows:

Moisture Content:

- 11.5% to 15.2%

Grain Size Distribution:

- 28% gravel;
- 51% sand;
- 21% silt and clay

The results of the moisture content and grain size distribution tests are provided on the record of borehole sheets in Appendix C. The result of grain size distribution test is also provided on Figure 7 in Appendix D.

#### **1.4.11 Lower Sand**

A native lower deposit of sand was encountered below the silt deposit in BH-3. The sand deposit was extended to depth about 8.4 m below ground surface with elevation about 307.6 m. BH-3 was terminated within this layer. The explored thickness of this layer was about 1.2 m.

The composition of this sand deposit was generally sand with trace to some gravel and trace silt. The material is grey in color, and wet. One SPT 'N' value obtained within this layer was 35 blows per 0.3 m penetration, suggesting dense in relative density.

Laboratory testing performed on selected samples consisted of one (1) moisture content test and one (1) grain size distribution test. The test results are as follows:

Moisture Content:

- 14.9%

Grain Size Distribution:

- 11% gravel;
- 79% sand;
- 10% silt and clay

The results of the moisture content and grain size distribution tests are provided on the record of borehole sheets in Appendix C. The result of grain size distribution test is also provided on Figure 8 in Appendix D.

## 1.5 Groundwater Conditions

Information regarding groundwater levels at the site was obtained by measuring water levels in the open holes of all the boreholes after completion of drilling. The groundwater levels measured in the boreholes are shown on Table 1.2 and on the borehole logs. Water levels measured in open boreholes might not be stabilized due to the relatively short period of observation.

Table 1.2 Groundwater data

Borehole	Date Completed	Date Measured	Ground Surface Elevation (m)	Groundwater Elevation (m)	Groundwater Depth (m)
BH-1	12/12/2018	12/12/2018	316.3	314.1	2.2
BH-2	12/14/2018	12/14/2018	321.3	311.8	9.5
BH-3	12/11/2018	12/12/2018	316.0	315.0	1.0
BH-4	12/13/2018	12/13/2018	320.9	319.5	1.4
BH-5	12/13/2018	12/13/2018	319.2	No measurable groundwater	
BH-6	12/12/2018	12/12/2018	319.9	318.5	1.4

Water in the creek was frozen at the time of investigation. The measured elevations of the top of ice at the inlet and outlet of the existing culvert were Elev. 314.5 m and Elev. 314.4 m, respectively.

Groundwater levels would be expected to reflect levels in the adjacent open water and to fluctuate seasonally. Seasonal variations in the water table should be expected, with higher levels occurring during wetter periods of the year and lower levels during drier periods.

## 1.6 Chemical Analyses

Four soil samples were selected for chemical analysis and they were sent via courier, in a secure cooler under chain of custody, to Maxxam Analytics Inc., a CALA-certified and accredited laboratory in Mississauga, Ontario. The analytical laboratory results are presented in Appendix E, and are summarized in Table 1.3, below.

Table 1.3. Corrosivity chemical analysis

Sample Identification	pH (unitless)	Soluble Chloride (ppm)	Soluble Sulphate (ppm)	Resistivity (ohm-cm)	Conductivity (mS/cm)
BH1-S4	7.61	<20	<20	8,000	0.13
BH2-S8	7.9	310	<20	1,400	0.71
BH3-S5	7.65	130	<20	2,600	0.38
BH5-S5	7.77	220	<20	2,400	0.42

## 2 ENGINEERING DISCUSSION & RECOMMENDATIONS

### 2.1 General

This section of the report provides geotechnical design recommendation for replacement of Nissiamkikan Creek Culvert on Highway 11, Site No. 48E-123/C, located 19.2 km east of Highway 580 at Beardmore or 3.4 km west of Highway 801 in Nazah, within Walters Township, Ontario, Ministry of Transportation (MTO) Northwestern Region. The recommendations are based on our interpretation of the factual data obtained from the boreholes advanced during the current investigation at the sites and presented in **Part I- Foundation Investigation Report** of this report. The interpretation and recommendations provided are intended solely to permit designers, to assess foundation alternatives and design new culvert and replacement. Comments on construction are only provided to highlight issues that could affect the design. Contractors bidding on the works should make their own assessments of the factual data and how it might affect construction means and methods, scheduling and the like. The report is subject to limitations which follows the text.

Based on the information provided by MTO, the existing culvert is a twin corrugated steel pipe (CSP) culvert with each approximately 1.9 m wide by 2.6 m high, with a total span of approximately 5.2 m and the about 43 m long. It is understood that the existing culverts would be replaced with a new culvert along the same alignment with no or minimum grade change anticipated at the culvert location. Based on the TOR, it is also understood that staged open-cut replacement with temporary highway widening and grade lowering is the preferred replacement option by MTO. The size and type of the new culvert is not firmly defined at the time of writing this report. It is also understood that, temporary widening of the existing highway embankment at north end of the culvert is considered for staging purposes.

This report addresses the geotechnical design of the foundation for the proposed culvert and for embankment widening by providing geotechnical design parameters in accordance with the latest edition of the *Canadian Highway Bridge Design Code (CHBDC)* (CAN/CSA-S6-14), the *Canadian Foundation Engineering Manual (CFEM)* (2006), MTO Gravity Pipe Design Guidelines (May 2007), and generally accepted good practice. Pertinent construction issues from a geotechnical standpoint are examined in general accordance with the Terms of Reference provided to us on October 21, 2018 together with the MTO request email. The assessment involved review of options for replacement of the existing culvert along the same alignment using a staged open-cut replacement method with temporary highway widening and grade lowering.

### 2.2 Expected Ground Conditions

The following ground conditions along the proposed culverts alignments are interpreted from the current investigation:

- a) Highway 11 is a two-lane highway with paved shoulders and cable guide rails on both sides. At the site location, Highway 11 generally runs in an east-west direction. Based on the drawing provided by MTO, the roadway embankment at the culvert location is about 7.8 m high with average side slopes of about 2.5H:1V. The elevation of highway pavement centerline at the site is about 321.3 m.



- b) The highway embankment consists of granular fill (~0.5 m thick) underlain by rockfill (~3.1 m thick) followed by gravelly sand fill (~1.6m thick). The embankment fill is underlain by native compact to dense silt (~8.4 m thick) followed by silty sand and gravel layer (1.8 m thick).
- c) At the outlet, a native deposit of compact silty sand (~2.1 m thick) was encountered below the sandy rootmat, which is underlain by loose to compact silt (~4.4 m thick) followed by very dense silty sand and gravel (~1.0 m thick). At the inlet, a layer of compact silty sand fill (~1.5m thick) was encountered which is underlain by buried creosote wood (~0.8 m thick) followed by compact silty sand (~ 0.8 m thick), compact silt (~4.1 m thick) and dense sand (~1.2 m thick) deposits.
- d) The invert of the new culvert is expected to the same as the existing invert level at approximate Elevation 313.52 m at the inlet and at Elevation 313.48 m at the outlet. The foundation soil at the invert of the new culvert is anticipated to be native compact silt.
- e) At the time of investigation, the top of the ice covering the creek water was at Elev. 314.5 m and Elev. 314.4 m at the inlet and outlet side of the existing culvert, respectively. The groundwater level measured in the boreholes drilled at the inlet and outlet was about Elev. 314.1 m and Elev. 315.0 m, respectively. The groundwater table in the embankment fill is expected to be around Elev. 315.0 m, or slightly higher. However, seasonal variations in the water table should be expected, with higher levels occurring during wetter periods of the year (such as spring thaw and late fall) and lower levels during drier periods. Some groundwater mounding within the embankment and perched water should be expected. It is noted that the provided drawings indicate an approximate creek water level of Elev. 314.41 m (inlet) in August 2016 which is slightly lower than that measured during the current investigation.

## 2.3 Structure Foundations

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

- Precast- rigid frame concrete box culvert,
- Cast-in-place rigid frame open footing culvert
- Corrugated steel pipe culvert,
- Corrugated steel plate (CSP) open footing culvert

Based on the subsurface information obtained from the site investigation, the native compact silt is considered suitable for support of all replacement options, assuming that any underlying organic soils or soft or very loose materials are to be replaced with clean and compactable soils. Beside the geotechnical criteria, the choice of culvert type also depends on parameters such as the initial cost, maintenance costs, hydraulic performance, ease of construction, water and soil corrosiveness, salvageability and local availability of material and equipment.

It is noted that regardless of the option selected, the existing twin 2.6 m high × 43 m long CSP culvert is to be removed. This will require excavation down to the existing founding elevation for all options. This suggests the need for surface/groundwater control as discussed in Section 2.7 of this report.

Any loose and/or soft soils encountered below the existing embankment should be excavated and removed to firm bearing of native soils and the grade restored with engineered fill. If the depth of excavation to remove unstable soils is excessive, using a geotextile fabric, such as Terrafix 270R or equivalent, in conjunction with engineered fill can be considered to assist in providing a stable base for the new culvert. Based on previous experience, typically a minimum of 450 mm of a clear stone accordance with OPSS. PROV 1004 over geotextile fabric would establish a stable bearing surface. The fabric should be installed a manner to mitigate the migration of fines from adjacent material.

Based on the subsoil condition, Table 2.1 below compares the possible structure options from foundation design and constructability perspectives with their advantages and disadvantages. Although the foundation soils can provide adequate support for all options listed in the table, the use of precast rigid frame box culvert is ranked highest for the criteria evaluated.

Table 2.1 Evaluation of foundation alternatives

Options	Advantages	Disadvantages	Relative Costs	Risks/ Consequences	Rank
Precast rigid frame concrete box culvert	<ul style="list-style-type: none"> <li>▪ Straightforward construction</li> <li>▪ Reduced construction period, consequently traffic management and water control period</li> <li>▪ Reduced excavation depth</li> <li>▪ Can be more readily installed during cold weather conditions</li> </ul>	<ul style="list-style-type: none"> <li>▪ If floor of box culvert is thin and poorly reinforced, it may heave and crack</li> <li>▪ During high flows, the concrete floor can be undermined</li> <li>▪ Susceptible to defects/leakage at joints</li> <li>▪ Requires bedding material</li> <li>▪ Disturbance of natural streambed</li> <li>▪ Possible sediments accumulation in the upstream of the culvert</li> </ul>	Low	<ul style="list-style-type: none"> <li>▪ Risk of unacceptable differential settlements if the entire foundation is not supported on the competent soil</li> <li>▪ Risk of leaking from joints if not properly installed</li> </ul>	1
Cast-in-place rigid frame open footing concrete culvert	<ul style="list-style-type: none"> <li>▪ Wider span may consider to maintain existing channel and so allows for natural streambed to remain intact</li> </ul>	<ul style="list-style-type: none"> <li>▪ Deeper excavation or below water excavation may required</li> <li>▪ Dewatering system required</li> <li>▪ Require placement of lean concrete</li> </ul>	Likely more expensive than Option 1	<ul style="list-style-type: none"> <li>▪ Risk of unacceptable differential settlements if the entire foundation is not supported on the competent soil</li> </ul>	3

Options	Advantages	Disadvantages	Relative Costs	Risks/ Consequences	Rank
	<ul style="list-style-type: none"> <li>Less accumulation of sediments in the upstream of culvert</li> </ul>			<ul style="list-style-type: none"> <li>Risk of delay in construction due to deeper excavation below water if proper dewatering is not maintained</li> <li>Higher scour risk</li> </ul>	
Corrugated steel pipe culvert	<ul style="list-style-type: none"> <li>Straightforward construction</li> <li>Reduce construction period, consequently traffic management and water control period</li> <li>Reduce excavation depth</li> </ul>	<ul style="list-style-type: none"> <li>Require bedding material</li> <li>Limited design life</li> <li>Potential for corrosion, hence generally shorter design life</li> <li>Disturbance of natural streambed</li> <li>Possible sediments accumulation in the upstream of the culvert</li> </ul>	Low to medium	<ul style="list-style-type: none"> <li>Risk of unacceptable differential settlements if the entire foundation is not supported on the competent soil</li> <li>Risk of structure segment loss due to corrosion</li> </ul>	2
Corrugated steel plate open footing culvert	<ul style="list-style-type: none"> <li>Wider span may consider to maintain existing channel and so allows for natural streambed to remain intact</li> <li>Less accumulation of sediments in the upstream of culvert</li> </ul>	<ul style="list-style-type: none"> <li>Deeper excavation or below water excavation may required</li> <li>Dewatering system required</li> <li>Require placement of lean concrete</li> <li>Potential for corrosion, hence generally shorter design life</li> </ul>	medium	<ul style="list-style-type: none"> <li>Risk of unacceptable differential settlements if the entire foundation is not supported on the competent soil</li> <li>Risk of structure segment loss due to corrosion</li> <li>Higher scour risk</li> </ul>	4

## 2.3.1 Shallow Foundations

### 2.3.1.1 Geotechnical Resistance

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

Table 2.2. Recommended spread footing design parameters

Culvert Type	Founding Elevation (m)	Footing Size (m)	Founding Soil Type	Factored Geotechnical Resistance at ULS (kPa)	Geotechnical Reaction at SLS (kPa)
Precast rigid frame concrete box culvert/ CSP pipe culvert	~313.3	5.2	Minimum 300 mm compacted granular material (Granular A or B Type II) over native compact silt	375	250
Cast-in-place rigid frame concrete/ CSP open footing culvert	~311.2 <sup>1</sup>	1.0	Native compact silt	375	250

Notes:

<sup>1</sup> Below the frost line.

The geotechnical resistance at SLS normally allows for 25 mm compression of the founding medium. The subgrade along the culvert footprint should be supervised, inspected and approved by a qualified geotechnical engineer. Provided the work is executed in accordance with the recommendations herein, a subgrade modulus of 18 MPa/m is considered appropriate for design of the concrete box culvert. The recommended subgrade modulus was estimated using the equation proposed by Bowles\*  $K_s = 40(SF)q_a$  (kPa/m) where SF = Safety factor and  $q_a$  is the allowable bearing capacity.

It is assumed that underlying organic soils and any other soft or very loose materials are to be replaced with clean and compactable soil such as clear stone, Granular A or Granular B Type II. Given 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.

\*Foundation Analysis and Design (Fifth Edition) – Joseph E. Bowles

### 2.3.1.2 Resistance to Lateral Loads

Resistance to lateral forces/ sliding resistance between the base of replacement box culvert and the bedding, and cast-in-place open footing and native soils should be calculated in accordance with Section 6.10.5 of the CHBDC. The unfactored values of the coefficient of friction,  $\tan \delta$ , are presented in Table 2.3.

Table 2.3 Recommended parameters for calculation of unfactored horizontal resistance

Structure	Interaction	Coefficient of Friction, $\tan \delta$
Precast Box Culvert	Precast Concrete on Granular A or Granular B Type II bedding	0.7
Cast-in-place Open Footing Culvert	Cast-in-place concrete on native compact silt	0.55

A factor of 0.8 should be applied in calculation of the horizontal resistance in accordance with CHBDC.

### 2.3.1.3 Frost Protection

The frost depth in the area of the culvert is estimated to be approximately 2.6 m in accordance with OPSD 3090.100. During construction of any temporary and permanent support system using shallow foundations should be provided a minimum 2.6 m of soil cover or equivalent frost protection should be provided using thermal insulation. This frost protection requirement applies to the rigid frame concrete or CSP open footing culvert option. It is not necessary for a box culvert and pipe culvert since these structures are tolerant of small magnitudes of movements related to freeze-thaw cycles.

If the frost penetration line is at or above top of the culvert the backfill and cover for these culverts should be as per OPSD 803.010. Where less than 2.6 m of earth cover is provided above the top of the culvert, a frost taper should be included as per OPSD 803.010 for the concrete culverts with spans less than or equal to 3.0 m or MTOD 803.021 for the culvert with span more than 3.0 m.

## 2.4 Lateral Earth Pressure

Cofferdams, culvert walls and temporary shoring that may be required for excavation should be designed to resist lateral earth pressure. The expression for calculating lateral earth pressure is given by:

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

where

$P$  = earth pressure intensity at depth  $h$ , kPa

$K$  = earth pressure coefficient

$\gamma$  = unit weight of retained soil, kN/m<sup>3</sup>

$q$  = surcharge near wall, kPa

$h$  = depth to point of interest, m

The above expression does not take into account hydrostatic pressure, which must be included for the groundwater levels measured on the site. Table 2.4 lists earth pressure parameters for given materials. These recommendations assume level backfill and ground surface behind the walls.

Table 2.4 Material types and earth pressure properties

Material	Unfactored Friction Angle $\phi'$	Coefficient of Active Earth Pressure ( $K_a$ )	Coefficient of Passive Earth Pressure ( $K_p$ )	Coefficient of Earth Pressure at Rest ( $K_o$ )	Unit Weight $\gamma$ kN/m <sup>3</sup>
Gravelly Sand Fill	34	0.28	3.5	0.44	21
Rockfill: Crushed Gravel	35	0.27	3.69	0.43	22
Silty Sand	30	0.33	3.00	0.50	20
Silt	29	0.35	2.88	0.51	19
Sand	32	0.31	3.25	0.47	21
Silty Sand and Gravel	33	0.29	3.39	0.45	21

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).

## 2.5 Construction Alternatives

For the proposed culvert replacement, the following methods were considered as possible alternatives at

this site:

1. Full road closure followed by open cut/unsupported excavation to replace culvert
2. Temporary widening on one side of highway embankment to construct temporary detour at the site followed by staged 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 two options of excavation and replacement using the half-and-half approach were considered:
  - A. Construction using roadway protection and unsupported excavation of cut sides
  - B. Construction using roadway protection and braced cut sides
4. Staged construction using temporary modular bridge

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

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

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

Installation Method	Advantages	Disadvantages	Relative Cost	Ranking
<p>OPTION 1</p> <p>Full Road Closure and Open Cut Unsupported Excavation (see Figure H1 in Appendix H)</p>	<ul style="list-style-type: none"> <li>• No excavation support required</li> <li>• Install entire new culvert at once</li> <li>• No need to temporary divert surface water flow since the existing CSP culvert can be used to maintain the surface water flow</li> <li>• Removal of existing CSP culverts</li> <li>• Straightforward construction</li> <li>• Short mobilization time</li> <li>• Low capital investment; cost saving in time and materials required for construction</li> <li>• Adaptable to changing ground</li> <li>• Experienced contractors</li> </ul>	<ul style="list-style-type: none"> <li>• Traffic interruption</li> <li>• Long detour around site using other existing roads required, if any</li> <li>• Existing fills and native soils require 1.5H:1V side slopes to maintain stability</li> <li>• Erosion control of temporary cuts required</li> <li>• Potential claims to compensate vehicle occupants and local business for delays or time lost due to detour routes</li> <li>• Risk of cost overrun and inability to finish job: low to moderate</li> </ul>	<p>Relatively less expensive than other methods due to cost savings in time and materials required for construction, but potential claims to compensate vehicle occupants and local business for delays or time lost due to detour routes</p>	<p>N/A</p> <p><i>since there are no any possible/ acceptable detour routes and Hwy 11 is not allowed to full closure</i></p>
<p>OPTION 2</p> <p>Temporary Widening of Hwy 11 to Construct Local Detour and Open Cut Unsupported Excavation (see Figure H2 in Appendix H)</p>	<ul style="list-style-type: none"> <li>• Traffic flow maintained at the site during construction</li> <li>• No need to temporary divert surface water flow since the existing CSP culvert can be used to maintain the surface water flow</li> <li>• Removal of existing CSP culverts</li> </ul>	<ul style="list-style-type: none"> <li>• Traffic interruption</li> <li>• Construction of detour embankment required at north side of highway</li> <li>• Staged construction required- first construct the north detour embankment to replace one half of culvert and switch traffic to other half to replace the north half</li> <li>• Increased time for construction of detour and staging</li> <li>• Erosion control of temporary cuts required</li> <li>• Need to temporarily control creek water flow</li> <li>• Possible settlement due to new earth embankment fill</li> <li>• Risk of cost overrun and inability to finish job: low to moderate</li> </ul>	<p>More expensive than full road closure due to high costs to build local detours</p>	<p>1</p>



Installation Method	Advantages	Disadvantages	Relative Cost	Ranking
<p>OPTION 3.A</p> <p>Half-and-half Construction with Unsupported Cut Sides (see Figure H3.A in Appendix H)</p>	<ul style="list-style-type: none"> <li>• Traffic flow maintained at the site during construction</li> <li>• Short mobilization time</li> <li>• Straight forward construction and construction procedures</li> </ul>	<ul style="list-style-type: none"> <li>• Traffic interruption</li> <li>• Roadway protection of up to ~8 m high required to maintain one lane of traffic</li> <li>• High cost of roadway protection system</li> <li>• Installation of sheet piles could be problematic due to presence of rockfill</li> <li>• Large amount of soil to be excavated</li> <li>• Need to decommission the shoring system</li> <li>• Risk of cost overrun and instability to finish job: moderate to high</li> </ul>	<p>Relatively more expensive than full road closure due to high costs of roadway protection system</p>	<p>3</p>
<p>OPTION 3.B</p> <p>Half-and- half Construction with Braced or Anchored Cut Sides (see Figure H3.B in Appendix H)</p>	<ul style="list-style-type: none"> <li>• One or possibly two lanes of traffic flow maintained on existing road (e.g. steel decking, but costly)</li> <li>• Global stability of excavation enhanced by narrow geometry</li> <li>• Less traffic interruption than with unsupported cut sides approach</li> <li>• Temporary decking could be usable over braced cut to allow for excavation of both halves prior to diverting stream and backfilling</li> <li>• Cost savings due to limited excavation and backfill</li> </ul>	<ul style="list-style-type: none"> <li>• Traffic interruption</li> <li>• Roadway protection of up to ~8 m high required to maintain one lane of traffic if steel decking is not possible</li> <li>• High cost of roadway protection system and/or decking</li> <li>• Require side shoring and bracing</li> <li>• Installation of sheet piles could be problematic due to presence of rockfill</li> <li>• Relatively wide excavation for bracing (~5 m)</li> <li>• Bracing (e.g. struts) may interfere with excavation</li> <li>• Need to decommission the shoring system</li> <li>• Risk of cost overrun and instability to finish job: moderate to high</li> </ul>	<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>	<p>4</p>

Installation Method	Advantages	Disadvantages	Relative Cost	Ranking
<p>OPTION 4</p> <p>Staged Construction Using Temporary Modular Bridge (see Figure H4 in Appendix H)</p>	<ul style="list-style-type: none"> <li>• Traffic flow maintained at the site during construction without construction of local detour</li> <li>• No earth embankment fill material is required for building detours</li> <li>• No need for construction of the temporary support for excavation of the existing embankment which may not be possible due to rockfill: crushed gravel</li> <li>• No settlement since there is no new earth embankment fill</li> <li>• No need to temporary divert surface water flow since the existing CSP culvert can be used to maintain the surface water flow</li> <li>• Removal of existing CSP culverts</li> </ul>	<ul style="list-style-type: none"> <li>• Traffic interruption</li> <li>• Large amount of embankment fill to be excavated and replaced; Road has to be excavated longitudinally approximately 32 m at the top with forward slopes of 1.5H:1V</li> <li>• Additional cost for Temporary Modular Bridge and its foundations</li> <li>• Increased time for construction of staging</li> <li>• Erosion control of temporary cuts required</li> <li>• Risk of cost overrun and inability to finish job: low to moderate</li> </ul>	<p>More expensive than full road closer due to costs of Temporary modular bridge and foundation construction</p>	<p>2</p>

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

1. OPTION 2: Temporary widening of Hwy to construct local detour and open cut unsupported excavation (Figure H2, Appendix H)
2. OPTION 4: Staged construction using temporary modular bridge (Figure H4 in Appendix H)
3. OPTION 3.A: Half-and-half construction with unsupported cut sides (Figure H3.A, Appendix H)
4. OPTION 3.B: Half-and-half construction with braced or anchored cut sides (Figure H3.B, Appendix H)
5. OPTION 1: Full road closure using existing roadways and open cut unsupported excavation (Figure H1, Appendix H) - Since there are no any possible/ acceptable detour routes and Highway 11 is not allowed for a full closure, this option is considered not applicable. Therefore, it is not discussed further in subsequent sections.

The following sections discuss these options, except Option1, in more details.

### **2.5.1 Local Detour Option (Option 2)**

The local detour option, with embankment widening at the site to maintain the local flow of traffic during the replacement (see Figure H2, Appendix H), allows for open cut, unsupported excavation to facilitate the replacement of the existing culverts. The major advantage is that neither excavation support nor roadway protection is required with this option. The major disadvantages of this option are traffic interruption, increased time for construction of detour and staging, and need for temporary construction of local unwatering and dewatering systems (i.e. cofferdams, and sumps and pumps, etc.) to prevent creek water and groundwater flow into the construction area which is the responsibility of the Contractor.

Option 2 involves construction of temporary embankment widening at the north side of the existing embankment. Compacted engineered fill for construction of the widening is recommended. Prior to construction of widening, 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 widening footprint. All subgrade soils should be proof-rolled prior to fill placement and embankment fill should be placed in accordance with OPSS. PROV 206 (dated November 2014). The widening portion of the embankment should be key into the existing slope as indicate in OPSD 208.010.

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. rockfill: crushed gravel and gravelly sand fill) may be classified as a Type 2 soil above the groundwater table in conformance with the OHSA. The native soils below the groundwater table may be classified as a Type 4 soil. To avoid disturbance of the founding subgrade and to allow placement of backfill in dry conditions, groundwater

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

Temporary excavation side slopes for Type 2 soil should not exceed 1H:1V in accordance with OHSA, and 1.5H: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).

## **2.5.2 Half-and-Half Construction (Options 3)**

The half-and-half construction method was considered as a possible construction method at the site (see Figures H.3.A and H.3.B, Appendix H). In that method one lane of the existing highway will be used to maintain the local traffic while the other half of the existing highway will be excavated, and the half of the existing culvert will be exposed. Then that portion of the existing culvert will be removed and replaced with a new culvert (or culverts), followed by rebuilding of that half of the embankment to grade. Upon completion of the new embankment, the traffic will be moved onto the new fill and the process will be repeated to complete the construction and culvert replacement.

The temporary excavation required to remove half of the existing embankment would be up to ~8 m deep. Therefore, temporary shoring such as soldier piles and laggings or sheet piles will be required as a roadway protection system to allow staging excavation/construction. It will be the Contractor responsibility to design a suitable temporary support system for the MTO review prior to installation. The Contractor is to follow OPSS.PROV 404, regarding support systems to permit the excavation and backfilling of trenches or excavations, and OPSS.PROV 539 and SP No. 105S09, regarding temporary protection systems. Recommendations for a temporary roadway protection are given in Section 2.6. However, it should be noted that installation of sheet piles could be challenging due to presents of rockfill in the existing embankment.

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

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. Excavation and backfilling operations will also be more challenging with Option 3.B. Both options require decommissioning of shoring system upon completion of the work.

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

This method provides roadway protection parallel to the highway between two lanes, and allows to divert traffic to the one side and undertake open cut with sloping sides at the other side (see Figure H3.A, Appendix H). The roadway protection can take the form of reversible shoring such as a soldier

pile and lagging or sheet pile with rakers or anchors for horizontal support. Where the cut extends below prevailing groundwater a suitable control/system is required. Once one lane is completed the supports can be reversed and the other lane constructed in similar fashion. The shoring system would likely be decommissioned in place. Temporary surface water flow control must be developed by the 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 since it needs to excavate a large amount of soil.

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

This method provides braced or anchored cut shoring system perpendicular to the highway for face protection and to allow culvert construction (see Figure H3.B., Appendix H). Excavation in this case would have to accommodate the necessary cross-bracing such as struts, which could be challenging considering the width of the existing culverts (~5 m). With this option, consideration would have been also given to how the new culvert sections will be installed given the 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 the 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.

#### **2.5.3 Staged Construction Using Temporary Modular Bridge (Option 4)**

This method provides construction of temporary modular bridge (TMB) on one side of the highway to maintain the local flow of traffic during the replacement (see Figure H4, Appendix H), and allows for open cut, unsupported excavation to facilitate the replacement of the existing culverts. The major advantage is that neither excavation support nor roadway protection is required with these options. The major disadvantages of this option is high cost of temporary modular bridge and bridge foundation, traffic interruption, and need for temporary construction unwatering and dewatering systems (i.e. cofferdams, and sumps and pumps, etc.) to prevent creek water and groundwater flow into the construction area which is the responsibility of the contractor.

##### **2.5.3.1 Temporary Modular Bridge Foundations**

If a temporary modular bridge is considered to be built at one side of Highway 11 to facilitate design of TMB foundations, it is recommended to drill at least two additional boreholes at locations of proposed TMB abutments. For the preliminary purpose, based on the current information from BH-2 it can be expected that the shallow spread footings on mass concrete placed directly on the rockfill: crushed gravel can be recommended with the following geotechnical resistances:

- Factored Geotechnical Resistance at ULS of 300 kPa
- Geotechnical Reaction at SLS of 200 kPa

The geotechnical resistances provided above are given under the assumptions that the stratigraphy in additional boreholes and BH-2 are similar, as well as for concentric vertical loading condition only. Where the load is not concentric vertical loading, load eccentricity and load inclination effects need to be considered.

### 2.5.3.2 Resistance to Lateral Loads

Resistance to lateral forces/ sliding should be calculated in accordance with Section 6.10.5 of the CHBDC, using the following parameters:

Table 2.6. Recommended parameters for calculation of unfactored horizontal resistance

Interface and Loading Conditions	Parameters
Between cast-in-place concrete and rockfill: crushed gravel	Coefficient of friction ( $\tan \delta$ )=0.5

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

### 2.5.3.3 Frost Protection

The frost depth in the area of the temporary modular bridge is the same as that estimated in Section 2.3.1.3. It is approximately 2.6 m in accordance with OPSD 3090.100. Since the footing for the temporary bridge will be founded on rockfill: crushed gravel requirement of full frost protection is might not applicable.

## 2.6 Temporary Protection

At this site, temporary roadway protection is anticipated to be a part of the half-and-half construction approach if that method is selected to maintain on-site traffic during the construction. It is recommended that roadway protection system be in accordance with OPSS.PROV 539 and SP No. 105 S09. The lateral movement of the temporary shoring system should meet Performance Level 2 as specified in OPSS.PROV 539. The complete design, construction, monitoring and removal of the installed protection system should be a responsibility of the Contractor. Due to nature of this application it is expected that much of temporary shoring will be decommissioned in place noting the high cost for removal. Decommissioning must be consistent with good practice to avoid interference with highway systems and utilities, if any. The protection system should be designed to provide protection for excavations as required by the OHSAA, at locations specified in the contract, and at any locations where the stability, safety or function of an existing structure may be impaired by construction work.

Since rockfill: crushed gravel exists within the highway embankment at this site, the shoring system such as soldier piles and timber lagging may be considered for design. It should be designed based on the earth pressures coefficients and soil parameters provided in Section 2.4. The actual depth of embedment should be determined by balancing moments about the pile tip. However, considering the height of the roadway embankment and the depth of the refusal a temporary shoring system with additional anchorage or tiebacks may be required for lateral resistance. Conventional practice is to incorporate either buried deadman anchors or soil grouted anchors. Alternatively, a system of rakers can be used for support.

Deadman anchors can be designed based on the earth pressure coefficients and soil parameters provided in Section 2.4. For this project, either continuous or individual concrete block anchors would likely be appropriate. The anchor resistance is provided by a combination of the dead weight and passive resistance. For the full passive resistance to be realized with no load transfer to the wall, the anchor needs to be fully beyond the active wedge acting on the wall. Pressure grouted soil anchors can be also designed in a preliminary fashion in accordance with Section 26 of the CFEM (2006). Based on the generally compact soils at this site, the estimated factored (0.4) ULS resistance of grouted anchors would be 130 kN/m length. Detailed design would be completed following the design of the wall and the loads have been established. Normally, such anchors are supplied and installed/tested by specialist vendors/contractors.

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

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

After construction of the new culvert, the protection system could be removed. In that case the details of the procedures associated with the removal of the protection system indicating: method, sequence of work, and removal limits are required from the Contractor as per OPSS.PROV 539. However, if protection system is decided to be left in place the top should be removed to at least 1.2 m below the finished grade or ground level or at least 0.6 m below the streambed. All disturbed areas should be restored to an equivalent or better condition than existed prior to the commencement of construction. A non-standard special provision is provided in Appendix I which may form the basis for advise to the Contractor on this issue.



## 2.7 Surface Water and Groundwater Control

At the time of investigation (December 2018), the groundwater/water level at the embankment and in the creek was encountered between Elev. 315.0 m (BH-3) and 314.5 m (inlet side), while the excavation to the foundation level has to be carried out to Elev. 313.0 m (box culvert) to Elev. 311.2 m (open footing). Therefore, it is possible that the water table is about 2.0 m to 3.8 m above the bottom of excavation depending on the time of construction. Considering that the soils encountered below the groundwater table and within potential excavation depths consist of native silty soils, it is assessed that these soils are susceptible to disturbance from groundwater and mobilized equipment. Therefore, the groundwater level needs to be controlled to at least 0.5 m below the excavation level to avoid disturbance, and any surface or groundwater seepage should be removed from the excavation prior to the culvert bedding material placement of granular backfill in the dry. In general, where the excavation base is within 0.5 m of the prevailing groundwater level at the time of construction, it is anticipated that control of seepage can be accomplished by using properly filtered sumps. However, given the conditions at the site, it is expected that standard pumping from sumps might not be adequate to maintain the excavation unwatered, so positive dewatering systems will be required to control the groundwater seepage. Alternatively, clear stone wrapped with geotextile may be considered as a bedding material to construct culvert under the less dry condition.

Nissiamkikan Creek flows through the existing culvert will need to be controlled and the excavation areas to be protected during construction. Depending on the creek water level and surface water flow conditions at the time of construction, a portion (or both) of the existing twin CSP culvert can be used to divert the creek water flow. A cofferdam in the form of a sheet-pile cut off wall or sand bags and clay puddle may be used. Dewatering requirements behind the cofferdam to keep the construction site dry will be impacted by water levels in the stream at the time of construction activities. However, dewatering may be carried out from the sumps located along the periphery of the cofferdam.

Dewatering shall be carried out in accordance with OPSS.PROV 517 and SP No.517F01 (July 2017) attached to Appendix G. It is responsibility of the contractor to propose a suitable dewatering system based on the time of construction, water levels and flow conditions for prior approval of MTO. The method used should not undermine the existing road embankment or adjacent side slopes. In this connection, the provision of toe protection at side slopes during drawdown may be required to minimize sloughing and undercutting during dewatering. Alternatively, and in accordance with SP 517F01, the dewatering systems may be completed by a design Engineer and design-checking Engineer with a minimum of 5-year experience. For this application, this is considered a suitable approach, but the owner should make final decision. Based on the estimated permeability of sandy silt soils ( $k \sim 5 \times 10^{-6}$  m/s), the preconstruction survey distance should be approximately 50 m, if any.

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.

At this preliminary stage, an accurate prediction of the groundwater pumping volumes cannot be made, as the rate and volume required for dewatering will be dependent on construction methods and staging chosen by the contractor. However, given the creek flow will be diverted away from the proposed



excavation and the suitable dewatering system will be installed to mitigate groundwater inflows, the construction site groundwater pumping volume will not exceed 50,000 L/day. Therefore, Environmental Activity Section Registry (EASR) or Permit to Take Water (PTTW) would not be required.

## 2.8 Culvert Bedding

MTOD 803.021, OPSD 3101.150, OPSD 802.034, OPSD 802.014 and OPSD 802.024 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 or alternatively a 100 mm thick concrete working slab with 75 mm of bedding materials beneath the culvert and extend a minimum of 500 mm horizontally on either side of the culvert edge. The bedding material should be placed in layers not exceeding 200 mm in thickness, loose measurement, and compacted accordance with OPSS.PROV 501 before a subsequent layer is placed in accordance with OPSS. PROV 401.

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

## 2.9 Culvert Backfill

The selection and placing of the backfill and cover should be in accordance with OPSS 902. The backfill around the culvert should consist of free-draining, non-frost susceptible granular materials such as Granular A or Granular B (OPSS.PROV 1010).

For fills immediately below any roadway, it is recommended that Granular A or B materials be used. Where necessary, proper tapering as per standards should be provided. Below a depth of about 2.6 m from any finished road grade, approved compactable fill, such as select subgrade materials (OPSS.PROV 1010), imported fill or rockfill can be used.

All granular backfill materials should be placed in thin lifts (i.e. not exceeding 300 mm before compaction) and each lift should be compacted accordance with OPSS. PROV 501. Above the culvert cover the embankment could be constructed using rockfill. Construction of embankment using rockfill should be in accordance with OPSS.PROV 206. For rock embankment the layers should not exceed 1.5 m thickness prior to construction. Material in each layer should be fully compacted prior to the succeeding layer is placed. Each rockfill layer should be compacted with a tractor bulldozer with a minimum number of complete passes of 6 and the maximum passes of 8. A complete pass should be defined as 100% coverage of layer surface. A suitable geotextile should be placed at any contact between granular fill and rockfill.

The final lift of embankment fills prior to placing pavement sub-base should be compacted to 100 % SPMDD. The Granular A base and Granular B sub-base courses (for pavement) should be compacted to 100% of the material's SPMDD. If rockfill is used, before placing any granular fill over the rockfill,

proper chinking should be applied. Alternatively, a suitably robust geotextile can be placed for separation purposes.

The use of heavy compaction equipment should be avoided immediately adjacent and above the culvert, as per MTO practice. As specified in OPSS.PROV 401, the minimum height of fill cover above the crown of the culvert before power operated tractors or rolling equipment shall be 900 mm, unless otherwise noted by the structural engineer. During backfill placement, the height of the backfill should be maintained at approximately same level on both sides of the structure, to avoid lateral displacement of the structure.

Where less than frost depth (2.6 m) of earth cover is provided above the top of the culvert, a frost taper should be included as per OPSD 803.030, 803.031, MTOD 803.021, whichever is applicable.

Backfilling behind any retaining (wing) walls should consist of granular materials in accordance with the MTO standards. Free draining backfill materials and perforated drains (as per Figure C6.20a of the CHBDC), suitably outleted etc. should be provided in order to prevent hydrostatic pressure build-up.

## **2.10 Embankment Design**

### **2.10.1 Stability Analysis**

Slope stability analyses were performed to assess the global stability of the final embankment configuration including its temporary widening and to check that a minimum Factor of Safety of 1.3 will be achieved. The static slope stability analyses were performed using the SLOPE/W computer program and the Morgenstern-Price method developed on the basis of limit equilibrium.

The cross-section and the approximate slopes were developed based on the preliminary design drawings provided by MTO. The stratigraphy and groundwater condition at the site were developed based on the results of the geotechnical investigation presented in Part I - Foundation Investigation Report.

Based on the borehole information, the subsoils encountered at the work area consist of embankment fill, underlain by silty sand to silt deposits. Therefore, an effective stress analysis for a long term stability assessment of the embankment slope was performed taking into consideration the subsoil conditions encountered beneath the existing embankment. The analyses assume that all organic soils encountered in boreholes will be removed under the footprint of the widened section prior to construction.

Tabulated below in Table 2.7 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 2.7 Soil properties used in slope stability analyses of the culvert embankment**

Soil Type	Bulk Unit Weight, $\gamma$ (kN/m <sup>3</sup> )	$\phi'$ (degrees)	$c'$ (kPa)
Gravelly Sand Fill	21	34	-
Rockfill: Crushed Gravel	22	35	-
Sand	21	32	-
Silt	19	29	-
Silty Sand	20	30	-
Silty Sand and Gravel	21	33	-

The results of slope stability analyses are attached as Appendix F. The results of stability analyses on the new embankment slopes (north and south) are shown on Figures F1 and F2. It suggest that the global stability of new embankment slopes constructed with minimum 2H:1V slopes or matching existing slopes (which is even flatter slopes) will achieve a factor of safety greater than 1.3.

## 2.10.2 Settlement

Fill to be placed on native soils for temporary widening of the existing highway embankment will induce some settlement. However, noting no grade raise, the limited width of widening areas and presence of predominantly compact to dense non-cohesive foundation soils at the north side, the resulting settlement is expecting to occur mainly during construction in the order of 25 mm or less.

## 2.10.3 Subgrade Preparation and Embankment Construction

Prior to the placement of new fill for the embankment widening, the sites will need to be cleared and grabbed of the existing trees and bushes. All surficial topsoil, organic (i.e. muskeg/peat), loose, soft and/or deleterious materials spots bellow the footprint of the proposed embankment widening areas require to be excavated and replaced with clean and compactible soils with minimum 95% of Standard Proctor Maximum Dry Density (SPMDD).

Considering the findings in this investigation, the anticipated stripping depths/elevations at the borehole locations are as follows:

**Table 2.8. Recommend stripping depths at borehole locations**

Borehole No.	Existing Ground Elevation at Borehole Location (m)	Recommended Stripping Depth/ Elevation (m)
BH-1	316.3	0.3/316.0
BH-3	316.0	3.1*/313.0
BH-4	320.9	0.8/320.1
BH-5	319.2	0.3/318.9
BH-6	319.9	0.3/319.6

\* Stripping depth of 3.1 m in BH-3 was recommended due to buried wood followed by trace peat mix silty sand with creosote odour. The recommended stripping depth is about 0.5 m below culvert invert, which is within excavation depth for bedding material. However, during construction this should be inspected and approved by a geotechnical engineer.

After stripping, the exposed subgrade should be inspected, approved and properly compacted (i.e. proof rolled) from the surface, using a heavy compactor. The groundwater table should be lowered to at least 0.5 m in below the subgrade level, before any proof rolling and the application of significant compaction effort.

Before placing fill materials for widening it should be properly benched into the existing embankment in accordance with OPSD 208.010 and compacted.

## 2.11 Inlet and Outlet

### 2.11.1 Erosion and Scour Protection

Permanent erosion and scour protection should be provided at the culvert inlet and outlet including slopes and sides. In addition, sediment control such as silt fences and erosion control blankets may be required during construction.

The erosion/scour protection should be designed by a specialist River Engineer/Scientist. The following are some general suggestions:

Rip-rap protection should be provided at the culvert inlet and outlet. The rip-rap should extend approximately 5 m beyond the ends of the culvert and line the embankment slope to the spring line of the culvert. The size of the rip-rap is a function of the surface water hydrology. As a rule of thumb the thickness of the rip-rap should be a minimum of twice the median particle size, and 300 mm thick as a minimum. The rip-rap configurations at the culvert outlet should generally follow the OPSD 810.010, which is included in Appendix G of this report.

### 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. Rip-rap placed at 1V:1H will be stable.

### 2.11.3 Seepage Cut-off Requirements

The seepage cut-off requirements should be reviewed in the following context. The native silty soil at the inlet and outlet side and below the culvert bedding has a high potential for migration with high seepage gradients. For the culvert replacement and new culvert installation, it is prudent to examine possible methods to avoid piping of material resulting from seepage along the culvert. For culverts the following are typical methods: (i) clay seal, (ii) steel or wooden sheet pile cutoff at the upstream end of culvert, (iii) cut-off wall incorporated in the apron slab (if one is used) of the culvert, (iv) cut-off trench constructed with geotextile, and (v) rockfill at the upstream end of the culvert barrel to terminate below the granular bedding of the culvert. Only the clay seal and cut-off trench will be addressed since the sheet pile cut-off will require the understanding of the hydraulics of the stream.

#### 2.11.3.1 Clay Seal

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

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

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

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

#### 2.11.3.2 Cut-Off Trench

A cut-off trench can be used at both the upstream and downstream ends of the culvert and can be incorporated when the rip-rap apron at both ends of the culvert are being installed. In general, a trench is dug across the stream alignment to well beyond the walls of the culvert and a geomembrane liner is laid on the side of the trench keyed into the culvert at the top and on the base of the trench. The trench is then backfilled with graded rip-rap.

## 2.12 Corrosion Protection

Based on the chemical analysis, the data in Table 1.2 indicates very low to very high resistivity of tested soil, but in general low resistivity of tested soil, which indicates a moderate potential for corrosion of

buried metallic elements, particularly pipes and appurtenances (Table 3.2, MTO Gravity Pipe Design Guidelines). Therefore, some level of pipe protection requires, depending upon the pipe material type. The maximum chloride content reported is 310 ppm i.e. 0.031%, which indicates a low to moderate potential for additional corrosion.

The maximum water soluble sulphate content of the soils tested is <20 ppm ( $\mu\text{g/g}$ ), i.e. <0.002% and being less than 0.10%, does not require sulphate resistant cement. The data supports our local experience.

## **2.13 Obstructions**

The existing embankment is made of rockfill - crushed gravel, which could be a potential obstruction to impact excavations and/or element of temporary protection systems including cofferdams. In addition, a presents of buried creosote wood at the outlet also could be a potential obstruction during the excavation. A non-standard special provision is provided in Appendix I which may form the basis for advise to the Contractor on these issues.

May 01, 2019

### 3 CLOSURE

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

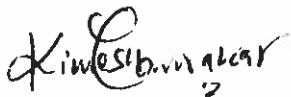
We recommend that we be retained to review our recommendations as the design nears completion to ensure that the final design is in agreement with the assumptions on which our recommendations are based and that our recommendations have been interpreted as intended. If not accorded this review, **exp**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

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

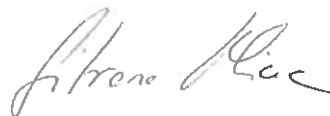
Contractors bidding on or undertaking any proposed work at this site should, relative to the subsurface conditions, decide on their own investigations, if deemed necessary, as well as their own interpretations of the factual results provided herein, so they may draw their own conclusions as to how the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

This Foundation Investigation and Design Report has been prepared by Nimesh Tamrakar, M.Eng, P.Eng. and Dr. Silvana Micic, P.Eng. It was reviewed by Mr. TaeChul Kim, M.E.Sc., P.Eng. and by Mr. Stan E. Gonsalves, M.Eng., P.Eng., Designated MTO Foundation Contact. The field investigation was supervised by Mr. Elwin Farkas.

EXP Services Inc.



Nimesh Tamrakar, M.Eng., P.Eng.  
Geotechnical Engineer



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



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



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





## **4 LIMITATIONS AND USE OF REPORT**

### **BASIS OF REPORT**

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

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

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

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

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

### **RELIANCE ON INFORMATION PROVIDED**

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



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

### **STANDARD OF CARE**

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

### **COMPLETE REPORT**

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

### **USE OF REPORT**

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

### **REPORT FORMAT**

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

## **Appendix A – Site Photographs**



Photo 1. Hwy 11 looking west



Photo 2. North side slope looking east from west side of the culvert



Photo 3. North side slope looking west from east side of the culvert



Photo 4. Looking east south from west side of the culvert (outlet side)





Photo 5. Looking north from outlet side (south) of the culvert



Photo 6. Looking south from inlet side (north) of the culvert

## **Appendix B – Drawings**







METRIC

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

Agreement No.: 6017-E-0066  
Assignment No. 2  
GWP 6561-00

NISSIAMKIKAN CREEK CULVERTS REPLACEMENT  
Hwy 11, ONTARIO, CANADA  
BOREHOLE LOCATION PLAN AND SOIL STRATA

SHEET

exp. exp Services Inc.

KEY PLAN



LEGEND

- Location of Drilled Boreholes
- N Standard Penetration Test (Blows/0.3 m)
- Water Level Upon Completion of Drilling
- Temporary Bench Mark (EL. 321.31m )

SOIL STRATA SYMBOLS

- TOPSOIL/ SANDY ROOT MAT
- ASPHALT
- FILL/ ROCK FILL
- SILT
- SILTY SAND AND GRAVEL
- SILTY SAND/ SAND AND SILT

BH No.	APPROX. ELEV.	MTM CO-ORDINATES (ZONE ON-14)	
		NORTH	EAST
BH-1	316.3	5503777.6	253365.8
BH-2	321.3	5503759.5	253381.5
BH-3	316.1	5503716.3	253385.1
BH-4	320.9	5503769.4	253264.3
BH-5	319.2	5503771.2	253333.1
BH-6	319.9	5503764.4	253488.1

NOTE

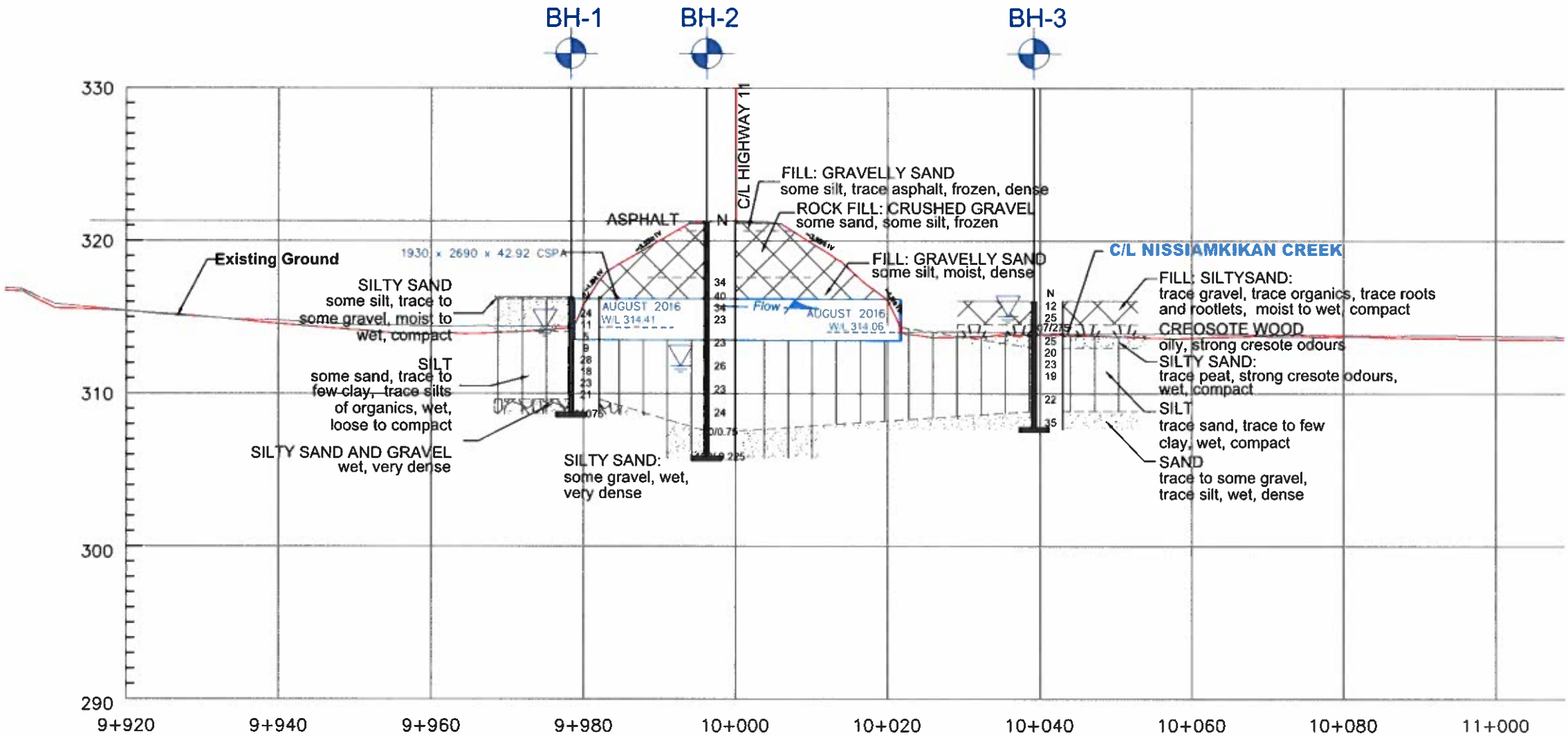
This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

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

SCALE:



29/04/2019	SM	SUBMISSION FOR MTO REVIEW	
DATE	BY	DESCRIPTION	
		GEOCRE NO. 42E-31	
		PROJECT NO. ADM-00248798-B0	
SUBMD SM	CHECKED SM	DATE	29/04/2019
DRAWN SH	CHECKED SG	APPROVED SG	DWG. 2



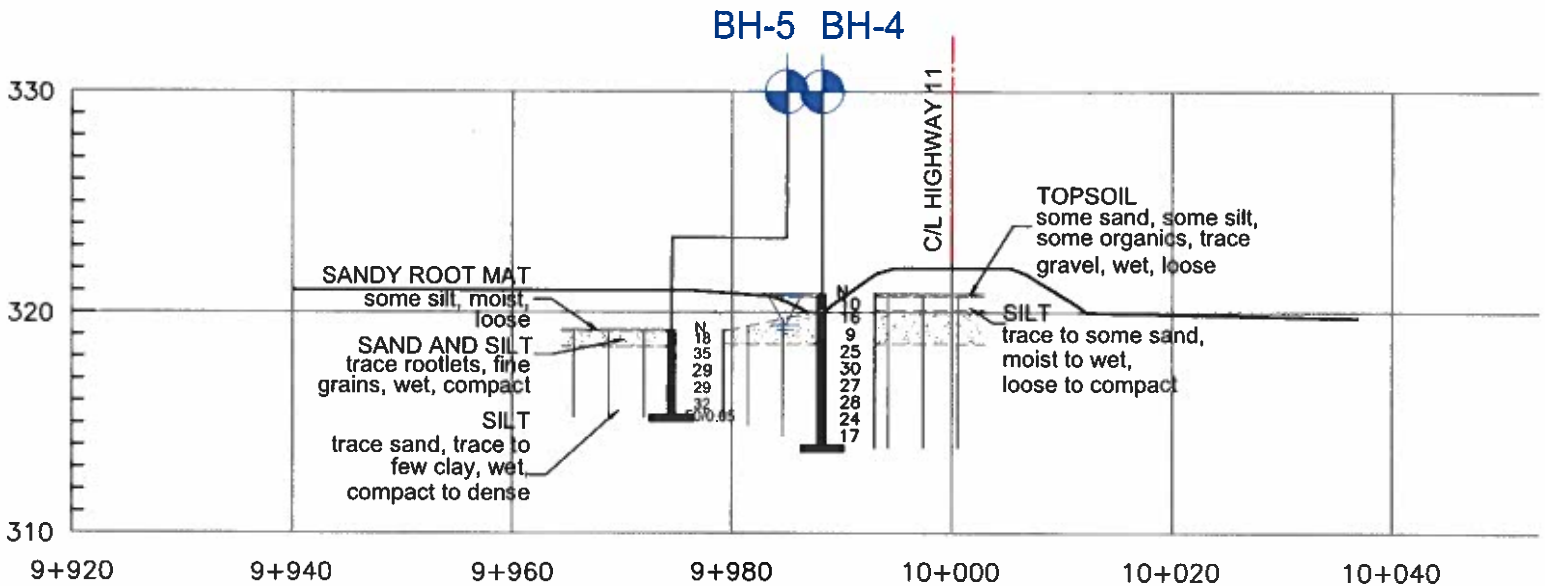
SECTION A-A' ALONG  
C/L CULVERT



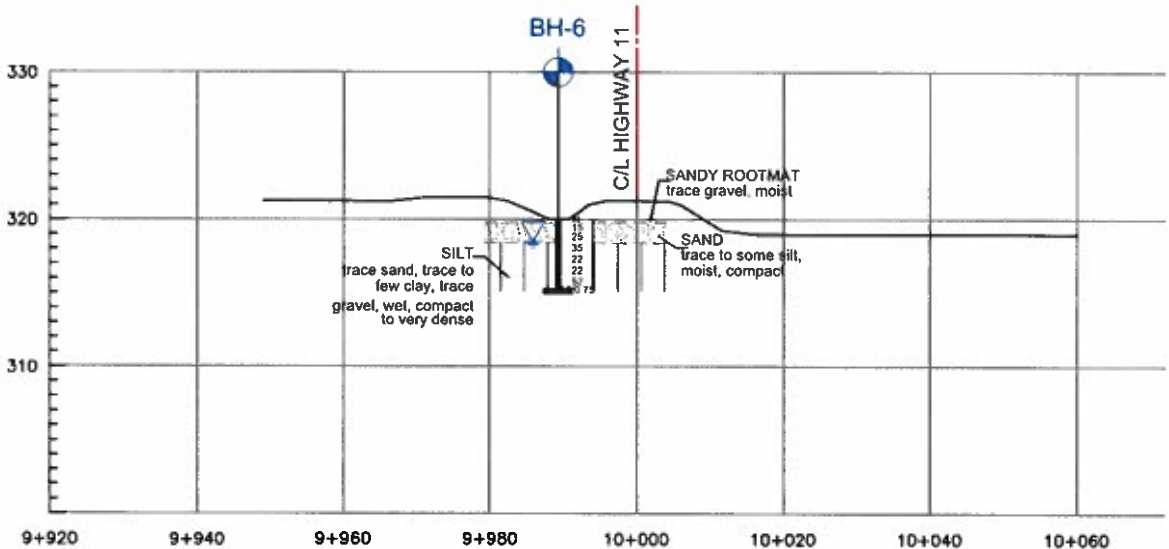


METRIC

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



SECTION B-B'



SECTION C-C'

Agreement No.: 6017-E-0066  
Assignment No. 2  
GWP 6561-00

NISSIAMKIKAN CREEK CULVERTS REPLACEMENT  
Hwy 11, ONTARIO, CANADA  
BOREHOLE LOCATION PLAN AND SOIL STRATA

SHEET

exp Services Inc.

KEY PLAN



LEGEND

- Location of Drilled Boreholes
- N Standard Penetration Test (Blows/0.3 m)
- Water Level Upon Completion of Drilling
- Temporary Bench Mark (EL. 321.31m )

SOIL STRATA SYMBOLS

- TOPSOIL/ SANDY ROOT MAT
- ASPHALT
- FILL/ROCK FILL
- SILT
- SILTY SAND AND GRAVEL
- SILTY SAND/ SAND AND SILT

BH No.	APPROX. ELEV.	MTM CO-ORDINATES (ZONE ON-14)	
		NORTH	EAST
BH-1	316.3	5503777.6	253365.8
BH-2	321.3	5503759.5	253381.5
BH-3	316.1	5503716.3	253385.1
BH-4	320.9	5503769.4	253264.3
BH-5	319.2	5503771.2	253333.1
BH-6	319.9	5503764.4	253488.1

NOTE

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SCALE:



29/04/2019	SM	SUBMISSION FOR MTO REVIEW	
DATE	BY	DESCRIPTION	
		GEOCRE NO. 42E-31	
		PROJECT NO. ADM-00248798-B0	
SUBMD SM	CHECKED SM	DATE	29/04/2019
DRAWN SH	CHECKED SG	APPROVED SG	DWG. 3

## **Appendix C – Borehole Logs**

# Explanation of Terms Used on Borehole Records

## SOIL DESCRIPTION

Terminology describing common soil genesis:

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

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

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

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

Terminology describing soil structure:

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

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

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

*Fissured:* material breaks along plane of fracture.

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

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

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

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

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

*Homogeneous:* same color and appearance throughout.

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

*Uniformly Graded:* predominantly on grain size.

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

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

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

Table a: Percent or Proportion of Soil, Pp

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

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

Table b: Apparent Density of Cohesionless Soil

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

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

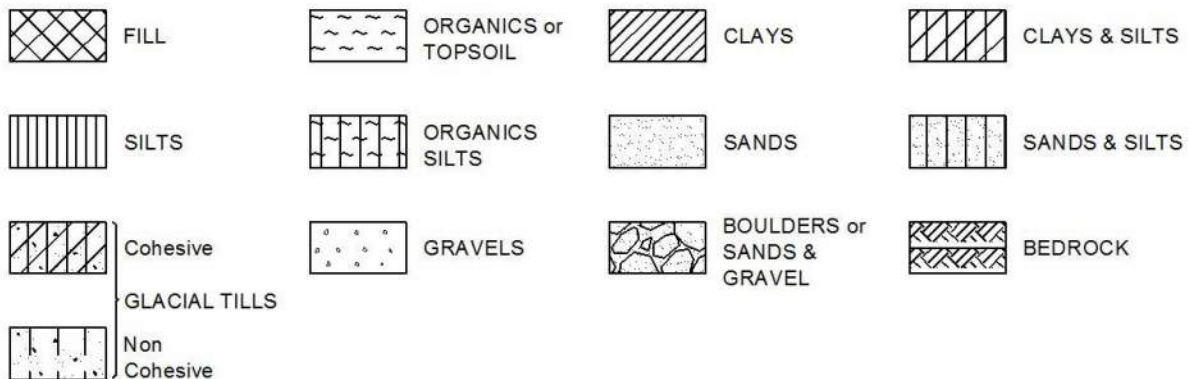
Table c: Consistency of Cohesive Soil

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

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

## STRATA PLOT

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



## WATER LEVEL MEASUREMENT



## 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
$\sigma$	kPa	Total normal stress
$\sigma'$	kPa	Effective normal stress
$\tau$	kPa	Shear stress
$\sigma_1, \sigma_2, \sigma_3$	kPa	Principal stresses
$\varepsilon$	%	Linear strain
$\varepsilon_1, \varepsilon_2, \varepsilon_3$	%	Principal strains
E	kPa	Modulus of linear deformation
G	kPa	Modulus of shear deformation
$\mu$	1	Coefficient of friction

### MECHANICAL PROPERTIES OF SOIL

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

### PHYSICAL PROPERTIES OF SOIL

$P_s$	$\text{kg}/\text{m}^3$	Density of solid particles
$\gamma_s$	$\text{kN}/\text{m}^3$	Unit weight of solid particles
$\rho_w$	$\text{kg}/\text{m}^3$	Density of water
$\gamma_w$	$\text{kN}/\text{m}^3$	Unit weight of water
$\rho$	$\text{kg}/\text{m}^3$	Density of soil
$\gamma$	$\text{kN}/\text{m}^3$	Unit weight of soil
$\rho_d$	$\text{kg}/\text{m}^3$	Density of dry soil
$\gamma_d$	$\text{kN}/\text{m}^3$	Unit weight of dry soil
$\rho_{sat}$	$\text{kg}/\text{m}^3$	Density of saturated soil
$\gamma_{sat}$	$\text{kN}/\text{m}^3$	Unit weight of saturated soil
$\rho'$	$\text{kg}/\text{m}^3$	Density of submerged soil
$\gamma'$	$\text{kN}/\text{m}^3$	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	$\text{m}^3/\text{s}$	Rate of discharge
v	m/s	Discharge velocity
i	1	Hydraulic gradient
k	m/s	Hydraulic conductivity
j	$\text{kN}/\text{m}^3$	Seepage force

Brampton, Ontario

## RECORD OF BOREHOLE No BH-1

1 OF 1

METRIC

W.P. 6561-00 LOCATION Hwy 11, Thunder Bay, 253365.784E, 5503777.576N ORIGINATED BY EF  
 DIST Thunder Bay HWY 11 BOREHOLE TYPE 108 mm I.D HSA, Acker MP5 COMPILED BY SH  
 DATUM Geodetic DATE 2018.12.12 - 2018.12.13 LATITUDE 49.6694753 LONGITUDE -87.7125968 CHECKED BY SM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT   NATURAL MOISTURE CONTENT   LIQUID LIMIT			UNIT WEIGHT  $\gamma$  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		W <sub>P</sub>	W	W <sub>L</sub>		
								20   40   60   80   100	WATER CONTENT (%)					
							○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL P. PENETROMETER							
316.3	Ground Surface		1	AS		▽								
316.0	<b>SANDY ROOTMAT:</b> trace gravel, dark brown, moist, loose <b>SILTY SAND:</b> trace to some gravel, brown to greyish brown, moist to wet, compact		2	SS			316							
0.2			3	SS	24									
			4	SS	11									
314.0	<b>SILT:</b> some sand, trace to few clay, trace organics, light brown to light grey, wet, loose to compact  -becoming sandy silt		5	SS	5		314							
2.3			6	SS	9		313							
			7	SS	28		312							
			8	SS	18		311							
			9	SS	23		310							
			10	SS	21		309							
309.6		<b>SILTY SAND AND GRAVEL:</b> grey, wet, very dense						309						
6.7														
308.6	<b>End of Borehole:</b> auger/ split-spoon refusal @ 7.7 m. <b>Notes:</b> 1. Upon completion of borehole, groundwater level was measured at 2.2 m.		11	SS	60/.075									
7.7														

ONTARIO MTO ASSIGNMENT #2.GPJ ONTARIO MTO.GDT 2/7/19

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



METRIC

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      ○<sup>3%</sup> STRAIN AT FAILURE



METRIC

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      ○<sup>3%</sup> STRAIN AT FAILURE

Brampton, Ontario

## RECORD OF BOREHOLE No BH-3

1 OF 1

METRIC

W.P. 6561-00 LOCATION Hwy 11, Thunder Bay, 253385.115E, 5503716.303N ORIGINATED BY EF  
 DIST Thunder Bay HWY 11 BOREHOLE TYPE 108 mm I.D HSA, Acker MP5 COMPILED BY SH  
 DATUM Geodetic DATE 2018.12.11 - 2018.12.12 LATITUDE 49.6689261 LONGITUDE -87.7123209 CHECKED BY SM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	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 P. PENETROMETER								
316.0	Ground Surface					▽										
0.0	<b>FILL: SILTY SAND:</b> trace gravel, trace organics, trace roots and rootlets, brown, moist to wet, compact		1	SS	12											
			2	SS	25											4 70 (26)
314.5																
1.5	<b>CREOSOTE WOOD</b> oily, strong creosote odours refusal to SPT @ 1.9 m auger through wood		3	SS	107/ 0.225											
313.8																
2.3	<b>SILTY SAND:</b> trace peat, strong creosote odours, dark brown, wet, compact		4	SS	25											
313.0																
3.1	<b>SILT:</b> trace sand, trace to few clay, grey, wet, compact		5	SS	20											0 3 (97)
			6	SS	23											
			7	SS	19										0 0 91 9	
			8	SS	22											
308.8	noted harder strata or change in layer @ approximate 7.2 m.															
7.2	<b>SAND:</b> trace to some gravel, trace silt, grey, wet, dense		9	SS	35										11 79 (10)	
307.6																
8.4	<b>End of Borehole:</b> auger/ split-spoon refusal @ 8.4 m <b>Notes:</b> 1. Upon completion of borehole, groundwater level measured at 1.0 m.															

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

ONTARIO MTO ASSIGNMENT #2.GPJ ONTARIO MTO.GDT 2/7/19

Brampton, Ontario

## RECORD OF BOREHOLE No BH-4

1 OF 1

METRIC

W.P. 6561-00 LOCATION Hwy 11, Thunder Bay, 253264.348E, 5503769.383N ORIGINATED BY EF  
 DIST Thunder Bay HWY 11 BOREHOLE TYPE 108 mm I.D HSA, Acker MP5 COMPILED BY SH  
 DATUM Geodetic DATE 2018.12.13 - 2018.12.13 LATITUDE 49.669393 LONGITUDE -87.7140016 CHECKED BY SM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT   NATURAL MOISTURE CONTENT   LIQUID LIMIT			UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL P. PENETROMETER		W <sub>P</sub> W                      W <sub>L</sub> WATER CONTENT (%)							
320.9	Ground Surface		1	AS			20	40	60	80	100	20	40	60	GR   SA   SI   CL		
320.0	<b>TOPSOIL:</b> some sand, some silt, some organics, trace gravel, dark brown to black, wet, loose		2	SS	10												0   16   (84)
0.2	<b>SILT</b> trace to some sand, light brown with oxidation, moist to wet, loose to compact		3	SS	16												0   46   (54)
320.1	<b>SAND AND SILT</b> brown, wet, loose to compact		4	SS	9												
0.8			5	SS	25												
318.6	<b>SILT</b> trace sand, trace to few clay, light brown to grey, wet, compact to dense		6	SS	30												0   1   90   9
2.3			7	SS	27												
			8	SS	28												
			9	SS	24												
			10	SS	17												
313.9	<b>End of Borehole:</b> auger/ split-spoon refusal @ 7.0 m <b>Notes:</b> 1. Upon completion of borehole, groundwater level measured at 1.4 m.																
7.0																	

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


Brampton, Ontario

## RECORD OF BOREHOLE No BH-5

1 OF 1

METRIC

W.P. 6561-00 LOCATION Hwy 11, Thunder Bay, 253333.101E, 5503771.195N ORIGINATED BY EF  
 DIST Thunder Bay HWY 11 BOREHOLE TYPE 108 mm I.D HSA, Acker MP5 COMPILED BY SH  
 DATUM Geodetic DATE 2018.12.13 - 2018.12.13 LATITUDE 49.6694152 LONGITUDE -87.7130487 CHECKED BY SM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>P</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT  γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)	
319.2	Ground Surface		1	AS		319								1 78 (21)		
318.9	<b>SANDY ROOT MAT</b> some silt, dark brown, moist, loose		2	SS	18											
	<b>SILTY SAND</b> trace rootlets, fine grains, brown to light brown, wet, compact															
318.5			3	SS	35		318									
0.8	<b>SILT</b> trace sand, trace to few clay, light brown, wet, compact to dense															
			4	SS	29			317								
			5	SS	29											
		6	SS	32	316								0 1 90 9			
		7	SS	50/ 0.05												
315.2	<b>End of Borehole:</b> auger/ split-spoon refusal @ 4.0 m. <b>Notes:</b> 1. No measurable groundwater level in open hole.															

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

Brampton, Ontario

# RECORD OF BOREHOLE No BH-6

1 OF 1

METRIC

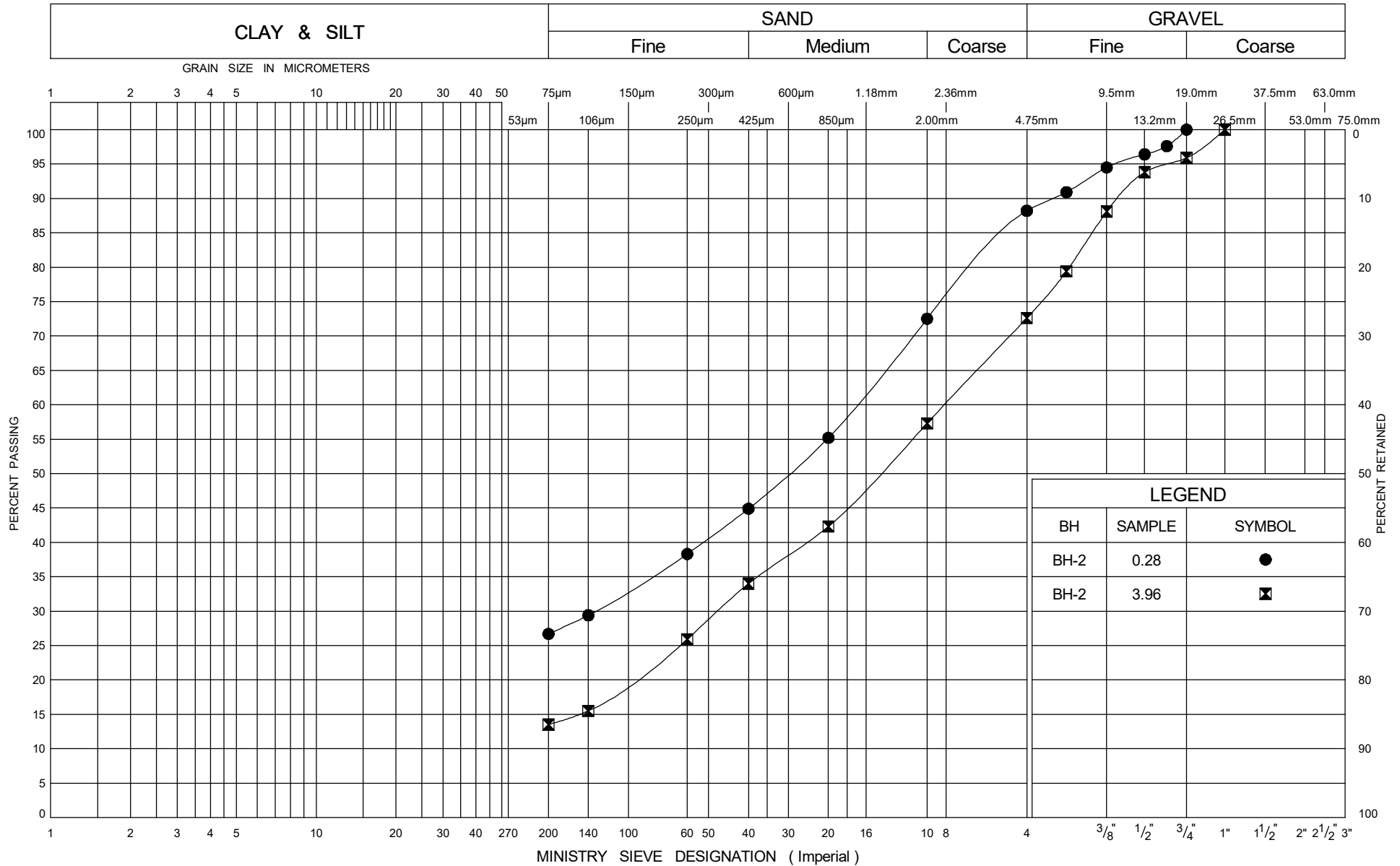
W.P. 6561-00 LOCATION Hwy 11, Thunder Bay, 253488.055E, 5503764.451N ORIGINATED BY EF  
 DIST Thunder Bay HWY 11 BOREHOLE TYPE 108 mm I.D HSA, Acker MP5 COMPILED BY SH  
 DATUM Geodetic DATE 2018.12.12 - 2018.12.12 LATITUDE 49.6693677 LONGITUDE -87.7109012 CHECKED BY SM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>P</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT  <b>γ</b>  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL & P. PENETROMETER												
319.9	Ground Surface		1	AS		▽	319	20	40	60	80	100					0 95 (5)			
319.8	<b>SANDY ROOTMAT</b> trace gravel, brown, moist		2	SS	15														No recovery	
0.2	<b>SAND</b> trace to some silt, light brown, moist, compact		3	SS	25															
318.4																				
1.5	<b>SILT</b> trace sand, trace to few clay, trace gravel, light brown to grey, wet, compact to very dense		4	SS	35				318											
			5	SS	22		317													
			6	SS	22															
							316													
			7	SS	52															
315.1																				
4.8	<b>End of Borehole:</b> auger/ split-spoon refusal @ 4.8m <b>Notes:</b> 1. Upon completion of borehole, groundwater level measured at 1.4 m.		8	SS	50/ 0.075															

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

## **Appendix D – Laboratory Data**

# UNIFIED SOIL CLASSIFICATION SYSTEM



Ministry of  
Transportation

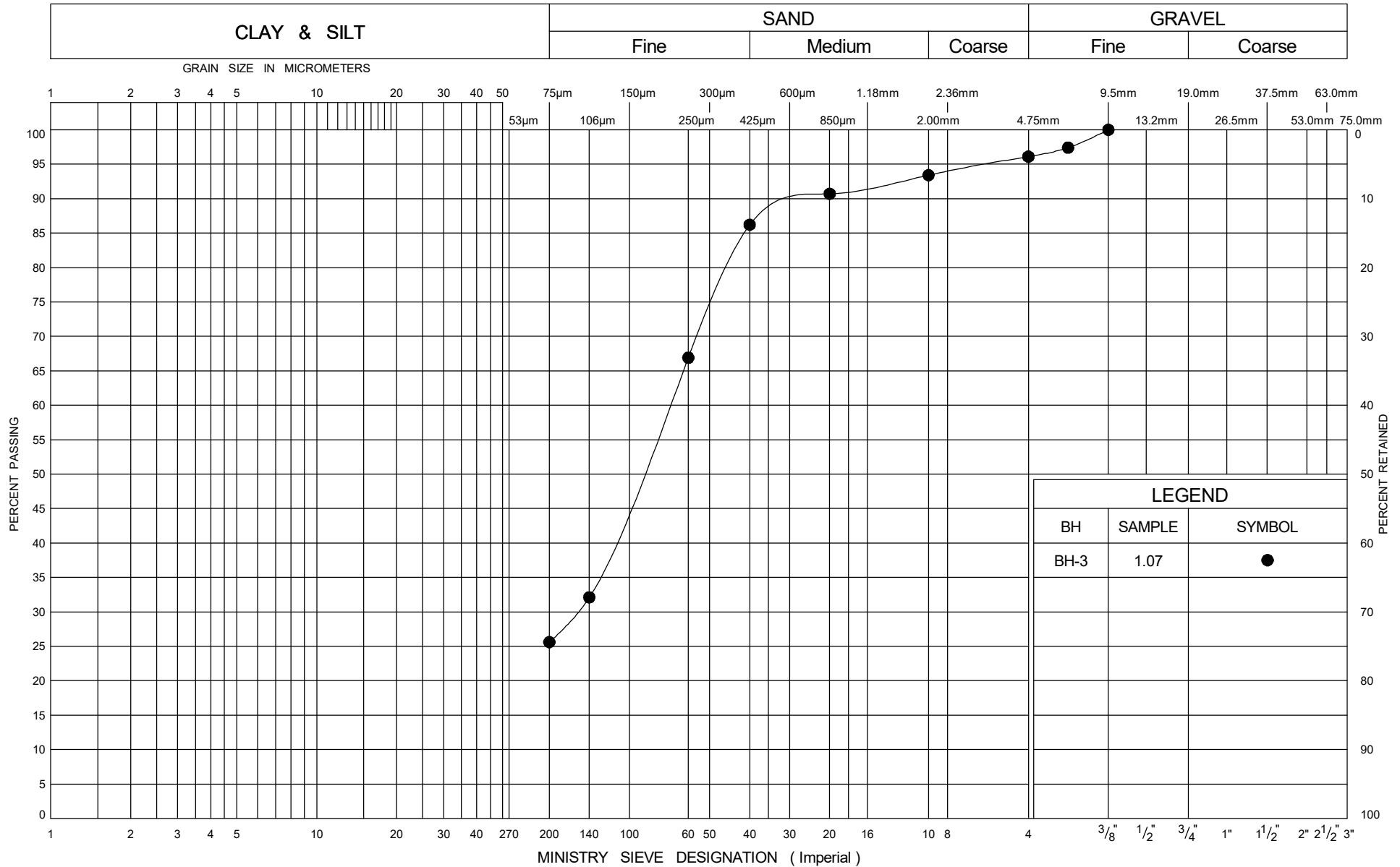
GRAIN SIZE DISTRIBUTION  
FILL: GRAVELLY SAND

FIG No 1

W P 6561-00

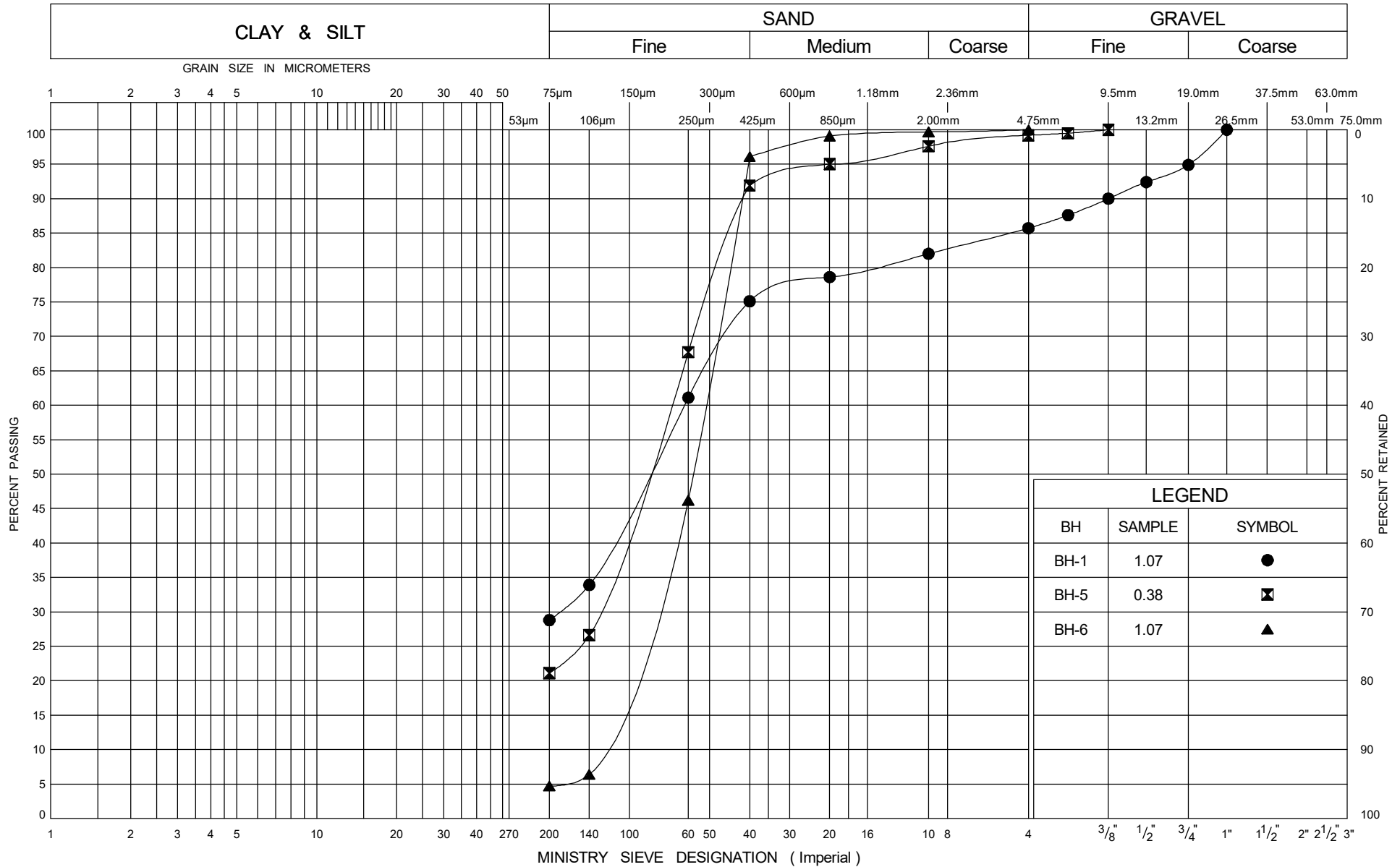
6017-E-0066

# UNIFIED SOIL CLASSIFICATION SYSTEM

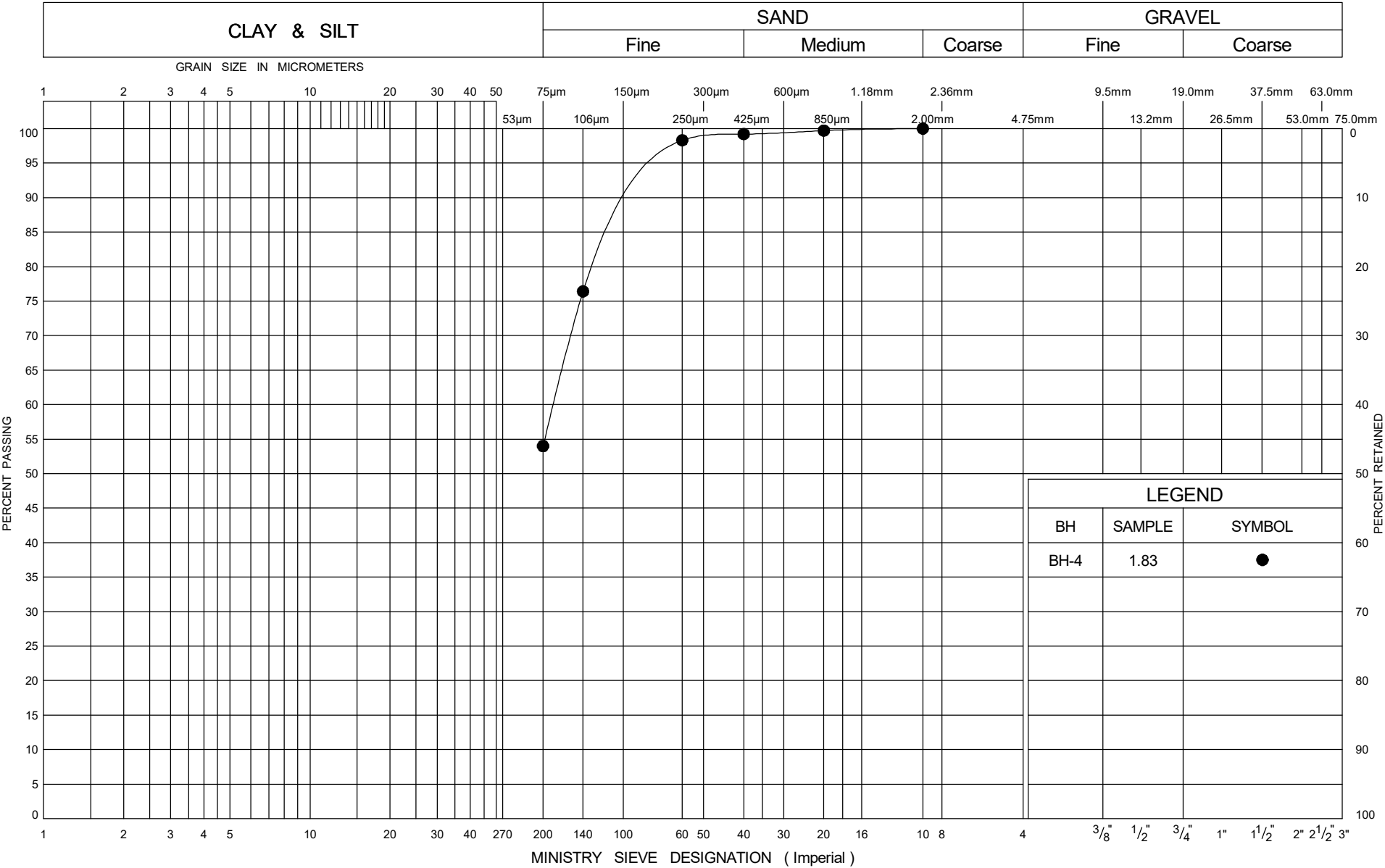




# UNIFIED SOIL CLASSIFICATION SYSTEM



UNIFIED SOIL CLASSIFICATION SYSTEM



Ministry of  
Transportation

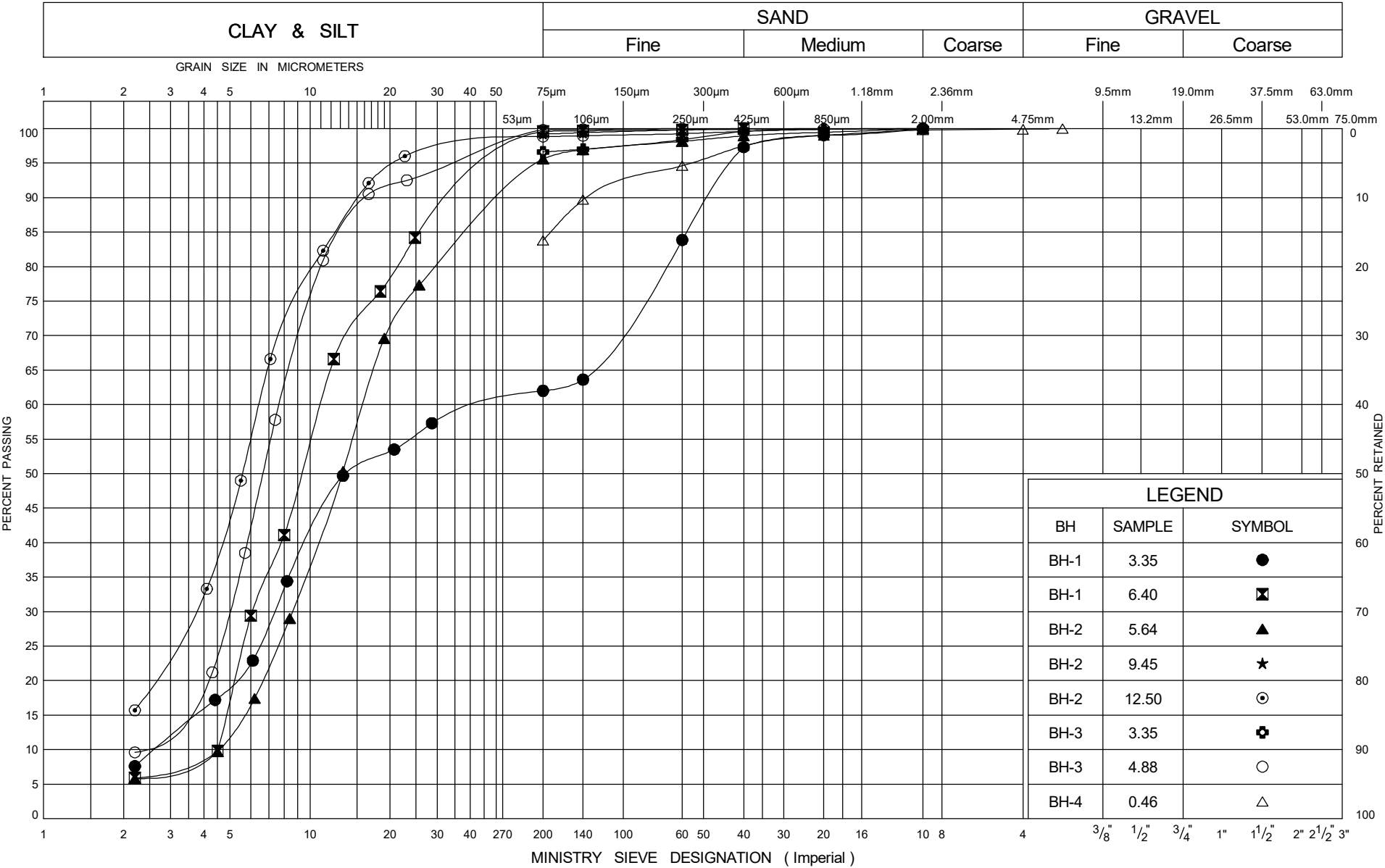
GRAIN SIZE DISTRIBUTION  
SAND AND SILT

FIG No 4

W P 6561-00

6017-E-0066

UNIFIED SOIL CLASSIFICATION SYSTEM



Ministry of  
Transportation

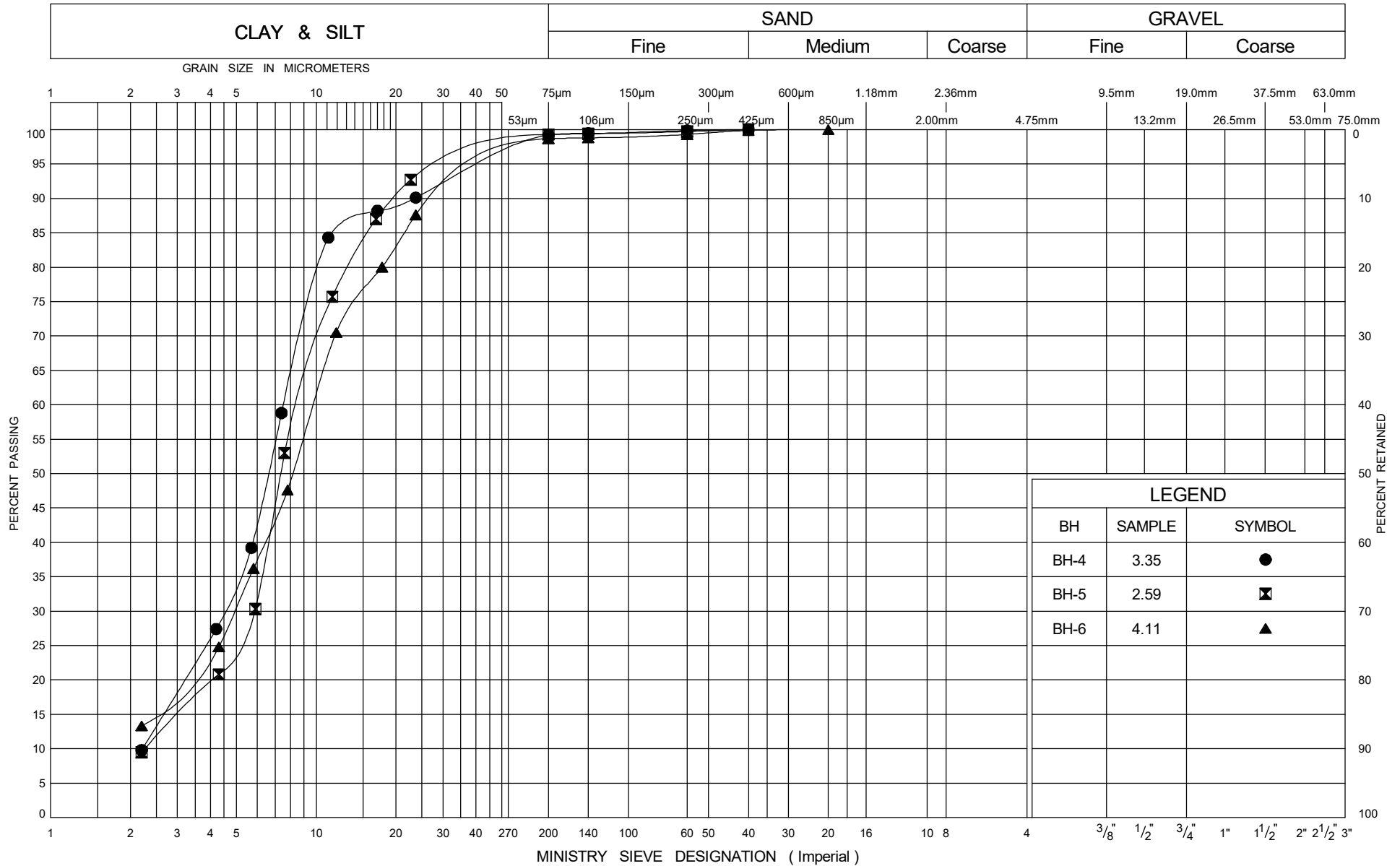
GRAIN SIZE DISTRIBUTION  
SILT

FIG No 5

W P 6561-00

6017-E-0066

# UNIFIED SOIL CLASSIFICATION SYSTEM



Ministry of  
Transportation

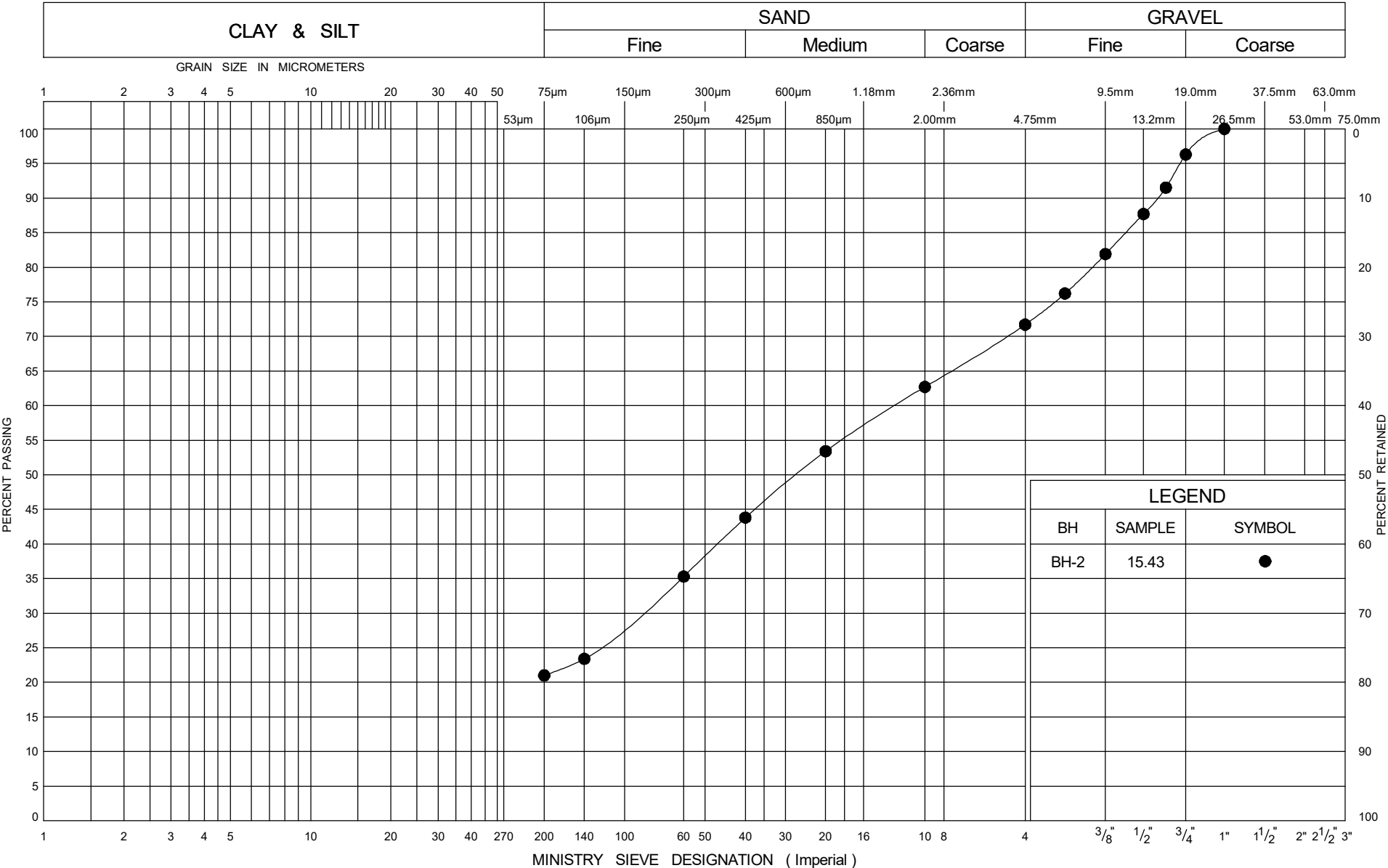
## GRAIN SIZE DISTRIBUTION SILT

FIG No 6

W P 6561-00

6017-E-0066

UNIFIED SOIL CLASSIFICATION SYSTEM



Ministry of  
Transportation

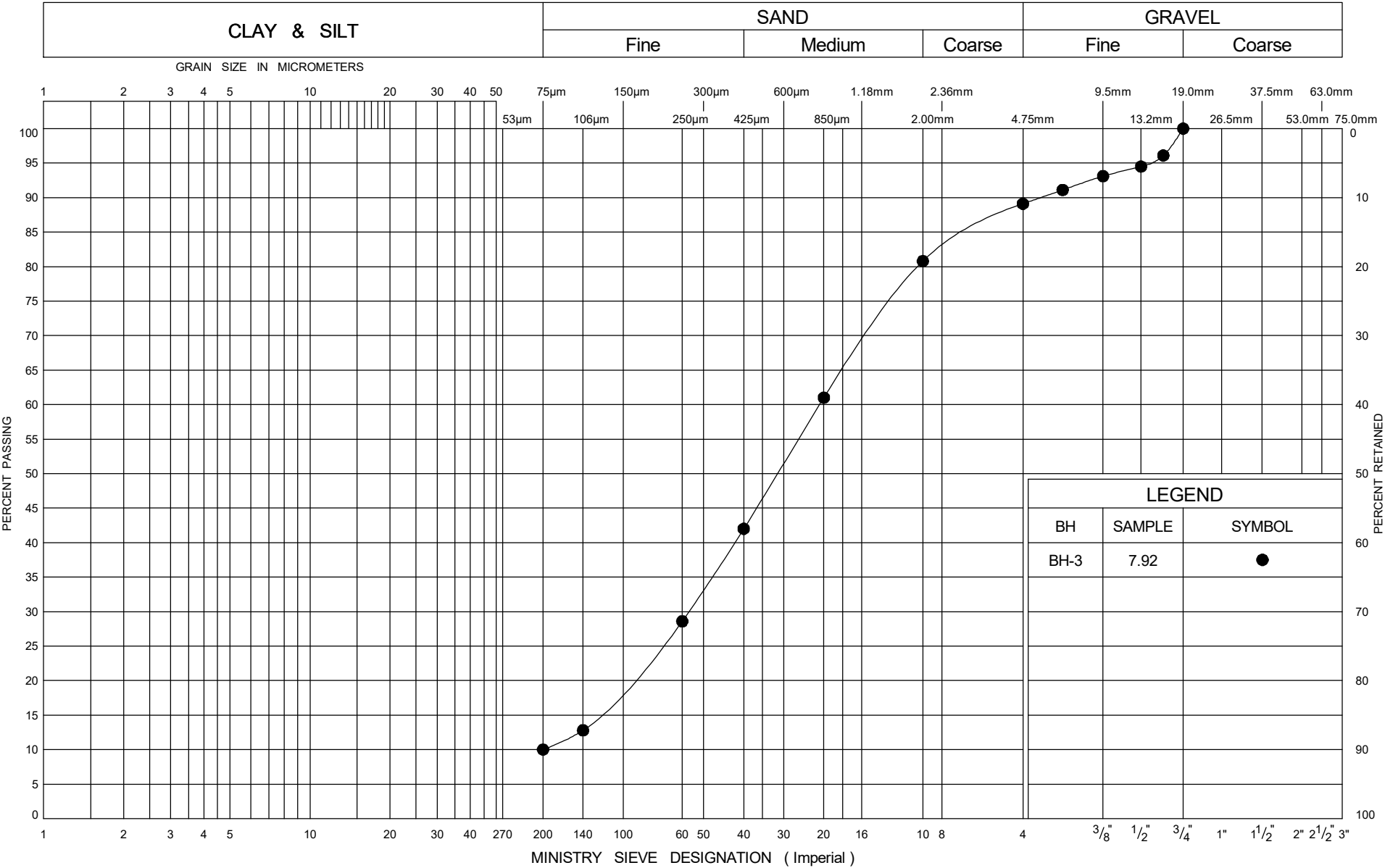
GRAIN SIZE DISTRIBUTION  
SILTY SAND AND GRAVEL

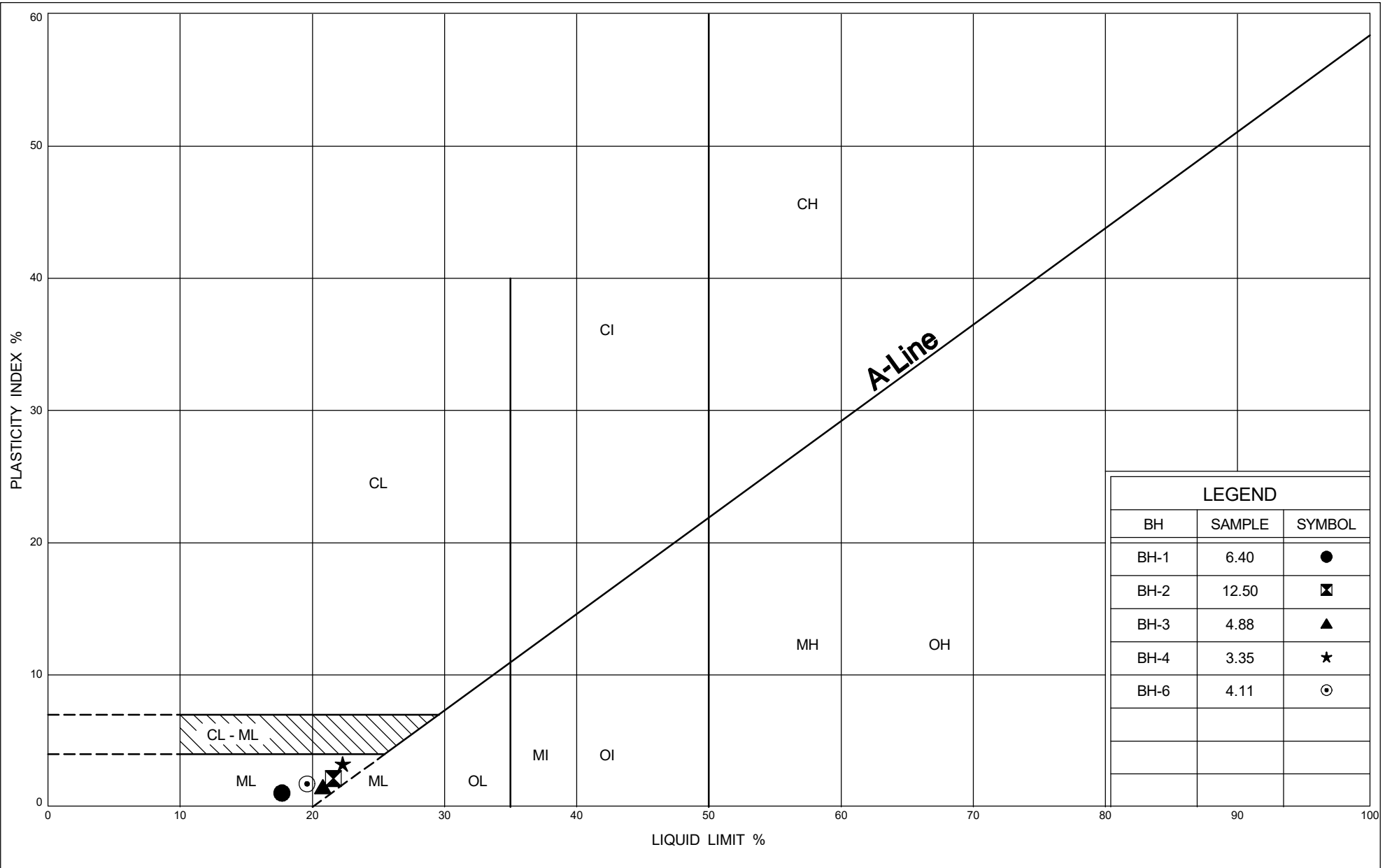
FIG No 7

W P 6561-00

6017-E-0066

UNIFIED SOIL CLASSIFICATION SYSTEM





Ministry of  
Transportation

## PLASTICITY CHART SILT

FIG No 9

W P 6561-00

6017-E-0066

## **Appendix E – Chemical Test Results**



Your Project #: ADM-00248798-B0  
Site Location: HWY 11, BEARDMORE, ON  
Your C.O.C. #: N/A

**Attention: Ahileas Mitsopoulos**

exp Services Inc  
Thunder Bay Branch  
1142 Roland St  
Thunder Bay, ON  
CANADA P7B 5M4

**Report Date: 2019/01/22**  
Report #: R5566385  
Version: 1 - Final

**CERTIFICATE OF ANALYSIS**

**MAXXAM JOB #: B914161**

**Received: 2019/01/15, 14:14**

Sample Matrix: Soil  
# Samples Received: 4

Analyses	Date		Date Analyzed	Laboratory Method	Reference
	Quantity	Extracted			
Chloride (20:1 extract)	4	N/A	2019/01/21	CAM SOP-00463	EPA 325.2 m
Conductivity	4	2019/01/21	2019/01/21	CAM SOP-00414	OMOE E3530 v1 m
pH CaCl2 EXTRACT	4	2019/01/18	2019/01/18	CAM SOP-00413	EPA 9045 D m
Resistivity of Soil	4	2019/01/17	2019/01/21	CAM SOP-00414	SM 23 2510 m
Sulphate (20:1 Extract)	4	N/A	2019/01/21	CAM SOP-00464	EPA 375.4 m

**Remarks:**

Maxxam Analytics' laboratories are accredited to ISO/IEC 17025:2005 for specific parameters on scopes of accreditation. Unless otherwise noted, procedures used by Maxxam are based upon recognized Provincial, Federal or US method compendia such as CCME, MDDELCC, EPA, APHA.

All work recorded herein has been done in accordance with procedures and practices ordinarily exercised by professionals in Maxxam's profession using accepted testing methodologies, quality assurance and quality control procedures (except where otherwise agreed by the client and Maxxam in writing). All data is in statistical control and has met quality control and method performance criteria unless otherwise noted. All method blanks are reported; unless indicated otherwise, associated sample data are not blank corrected. Where applicable, unless otherwise noted, Measurement Uncertainty has not been accounted for when stating conformity to the referenced standard.

Maxxam Analytics' liability is limited to the actual cost of the requested analyses, unless otherwise agreed in writing. There is no other warranty expressed or implied. Maxxam has been retained to provide analysis of samples provided by the Client using the testing methodology referenced in this report. Interpretation and use of test results are the sole responsibility of the Client and are not within the scope of services provided by Maxxam, unless otherwise agreed in writing. Maxxam is not responsible for the accuracy or any data impacts, that result from the information provided by the customer or their agent.

Solid sample results, except biota, are based on dry weight unless otherwise indicated. Organic analyses are not recovery corrected except for isotope dilution methods.

Results relate to samples tested. When sampling is not conducted by Maxxam, results relate to the supplied samples tested.

This Certificate shall not be reproduced except in full, without the written approval of the laboratory.

Reference Method suffix "m" indicates test methods incorporate validated modifications from specific reference methods to improve performance.

Your Project #: ADM-00248798-B0  
Site Location: HWY 11, BEARDMORE, ON  
Your C.O.C. #: N/A

**Attention: Ahileas Mitsopoulos**

exp Services Inc  
Thunder Bay Branch  
1142 Roland St  
Thunder Bay, ON  
CANADA P7B 5M4

**Report Date: 2019/01/22**  
**Report #: R5566385**  
**Version: 1 - Final**

**CERTIFICATE OF ANALYSIS**

**MAXXAM JOB #: 8914161**

**Received: 2019/01/15, 14:14**

**Encryption Key**



Maxxam

22 Jan 2019 13:27:32

Please direct all questions regarding this Certificate of Analysis to your Project Manager.

Michelle Brescacin, Project Manager Assistant - National Accounts

Email: MBrescacin@maxxam.ca

Phone# (905) 817-5700

=====

This report has been generated and distributed using a secure automated process.

Maxxam has procedures in place to guard against improper use of the electronic signature and have the required "signatories", as per section 5.10.2 of ISO/IEC 17025:2005(E), signing the reports. For Service Group specific validation please refer to the Validation Signature Page.

### RESULTS OF ANALYSES OF SOIL

Maxxam ID		ITW102	ITW103			ITW103			ITW104		
Sampling Date		2018/12/13 08:45	2018/12/14 12:15			2018/12/14 12:15			2018/12/12 09:00		
COC Number		N/A	N/A			N/A			N/A		
	<b>UNITS</b>	<b>BH1-S4</b>	<b>BH2-S8</b>	<b>RDL</b>	<b>QC Batch</b>	<b>BH2-S8 Lab-Dup</b>	<b>RDL</b>	<b>QC Batch</b>	<b>BH3-S5</b>	<b>RDL</b>	<b>QC Batch</b>

<b>Calculated Parameters</b>											
Resistivity	ohm-cm	8000	1400		5933043				2600		5933043
<b>Inorganics</b>											
Soluble (20:1) Chloride (Cl-)	ug/g	<20	310	20	5937361	280	20	5937361	130	20	5937361
Conductivity	mS/cm	0.13	0.71	0.002	5937412				0.38	0.002	5937412
Available (CaCl2) pH	pH	7.61	7.90		5934898				7.65		5934898
Soluble (20:1) Sulphate (SO4)	ug/g	<20	<20	20	5937362	<20	20	5937362	<20	20	5937362
RDL = Reportable Detection Limit QC Batch = Quality Control Batch Lab-Dup = Laboratory Initiated Duplicate											

Maxxam ID		ITW105		
Sampling Date		2018/12/13 10:00		
COC Number		N/A		
	<b>UNITS</b>	<b>BH5-S5</b>	<b>RDL</b>	<b>QC Batch</b>
<b>Calculated Parameters</b>				
Resistivity	ohm-cm	2400		5933043
<b>Inorganics</b>				
Soluble (20:1) Chloride (Cl-)	ug/g	220	20	5937361
Conductivity	mS/cm	0.42	0.002	5937412
Available (CaCl2) pH	pH	7.77		5934898
Soluble (20:1) Sulphate (SO4)	ug/g	<20	20	5937362
RDL = Reportable Detection Limit QC Batch = Quality Control Batch				

Maxxam Job #: B914161  
Report Date: 2019/01/22

exp Services Inc  
Client Project #: ADM-00248798-B0  
Site Location: HWY 11, BEARDMORE, ON  
Sampler Initials: EF

## TEST SUMMARY

**Maxxam ID:** ITW102  
**Sample ID:** BH1-S4  
**Matrix:** Soil

**Collected:** 2018/12/13  
**Shipped:**  
**Received:** 2019/01/15

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	5937361	N/A	2019/01/21	Alina Dobreanu
Conductivity	AT	5937412	2019/01/21	2019/01/21	Kazzandra Adeva
pH CaCl2 EXTRACT	AT	5934898	2019/01/18	2019/01/18	Gnana Thomas
Resistivity of Soil		5933043	2019/01/21	2019/01/21	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	5937362	N/A	2019/01/21	Alina Dobreanu

**Maxxam ID:** ITW103  
**Sample ID:** BH2-S8  
**Matrix:** Soil

**Collected:** 2018/12/14  
**Shipped:**  
**Received:** 2019/01/15

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	5937361	N/A	2019/01/21	Alina Dobreanu
Conductivity	AT	5937412	2019/01/21	2019/01/21	Kazzandra Adeva
pH CaCl2 EXTRACT	AT	5934898	2019/01/18	2019/01/18	Gnana Thomas
Resistivity of Soil		5933043	2019/01/21	2019/01/21	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	5937362	N/A	2019/01/21	Alina Dobreanu

**Maxxam ID:** ITW103 Dup  
**Sample ID:** BH2-S8  
**Matrix:** Soil

**Collected:** 2018/12/14  
**Shipped:**  
**Received:** 2019/01/15

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	5937361	N/A	2019/01/21	Alina Dobreanu
Sulphate (20:1 Extract)	KONE/EC	5937362	N/A	2019/01/21	Alina Dobreanu

**Maxxam ID:** ITW104  
**Sample ID:** BH3-S5  
**Matrix:** Soil

**Collected:** 2018/12/12  
**Shipped:**  
**Received:** 2019/01/15

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	5937361	N/A	2019/01/21	Alina Dobreanu
Conductivity	AT	5937412	2019/01/21	2019/01/21	Kazzandra Adeva
pH CaCl2 EXTRACT	AT	5934898	2019/01/18	2019/01/18	Gnana Thomas
Resistivity of Soil		5933043	2019/01/21	2019/01/21	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	5937362	N/A	2019/01/21	Alina Dobreanu

**Maxxam ID:** ITW105  
**Sample ID:** BH5-S5  
**Matrix:** Soil

**Collected:** 2018/12/13  
**Shipped:**  
**Received:** 2019/01/15

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	5937361	N/A	2019/01/21	Alina Dobreanu
Conductivity	AT	5937412	2019/01/21	2019/01/21	Kazzandra Adeva
pH CaCl2 EXTRACT	AT	5934898	2019/01/18	2019/01/18	Gnana Thomas
Resistivity of Soil		5933043	2019/01/21	2019/01/21	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	5937362	N/A	2019/01/21	Alina Dobreanu

### GENERAL COMMENTS

Each temperature is the average of up to three cooler temperatures taken at receipt

Package 1	3.3°C
-----------	-------

Chloride, Conductivity (Resistivity), pH and Sulphate analysis: The samples were received and analyzed past the recommended hold time.

**Results relate only to the items tested.**

## QUALITY ASSURANCE REPORT

exp Services Inc  
Client Project #: ADM-00248798-B0  
Site Location: HWY 11, BEARDMORE, ON  
Sampler Initials: EF

QC Batch	Parameter	Date	Matrix Spike		SPIKED BLANK		Method Blank		RPD	
			% Recovery	QC Limits	% Recovery	QC Limits	Value	UNITS	Value (%)	QC Limits
5934898	Available (CaCl <sub>2</sub> ) pH	2019/01/18			100	97 - 103			0.59	N/A
5937361	Soluble (20:1) Chloride (Cl <sup>-</sup> )	2019/01/21	NC	70 - 130	101	70 - 130	<20	ug/g	8.1	35
5937362	Soluble (20:1) Sulphate (SO <sub>4</sub> )	2019/01/21	117	70 - 130	104	70 - 130	<20	ug/g	NC	35
5937412	Conductivity	2019/01/21			103	90 - 110	<0.002	mS/cm	0.26	10

N/A = Not Applicable

Duplicate: Paired analysis of a separate portion of the same sample. Used to evaluate the variance in the measurement.

Matrix Spike: A sample to which a known amount of the analyte of interest has been added. Used to evaluate sample matrix interference.

Spiked Blank: A blank matrix sample to which a known amount of the analyte, usually from a second source, has been added. Used to evaluate method accuracy.


Method Blank: A blank matrix containing all reagents used in the analytical procedure. Used to identify laboratory contamination.

NC (Matrix Spike): The recovery in the matrix spike was not calculated. The relative difference between the concentration in the parent sample and the spike amount was too small to permit a reliable recovery calculation (matrix spike concentration was less than the native sample concentration)

NC (Duplicate RPD): The duplicate RPD was not calculated. The concentration in the sample and/or duplicate was too low to permit a reliable RPD calculation (absolute difference <= 2x RDL).

### VALIDATION SIGNATURE PAGE

The analytical data and all QC contained in this report were reviewed and validated by the following individual(s).



---

Brad Newman, Scientific Service Specialist

---

Maxxam has procedures in place to guard against improper use of the electronic signature and have the required "signatories", as per section 5.10.2 of ISO/IEC 17025:2005(E), signing the reports. For Service Group specific validation please refer to the Validation Signature Page.



6740 Campobello Road, Mississauga, Ontario L5N 2L8 www.maxxam.ca  
Phone: 905-817-5700 Fax: 905-817-5779 Toll Free: 800-563-6266

# CHAIN OF CUSTODY RECORD

Page 1 of 1

INVOICE INFORMATION		REPORT INFORMATION (if differs from invoice)		PROJECT INFORMATION		TURNAROUND TIME (TAT) REQUIRED	
Company Name: <b>exp Services Inc.</b>	Company Name:	Quotation #:	<input checked="" type="checkbox"/> Regular TAT (5-7 days)		<input type="checkbox"/> Rush TAT (Applicable Surcharge)		
Contact Name: <b>Ahileas Mitsopoulos</b>	Contact Name:	P.O. #:	PLEASE PROVIDE ADVANCE NOTICE FOR RUSH PROJECTS				
Address: <b>1142 Roland Street</b>	Address:	Project #: <b>ADM-00248798-B0</b>					
<b>Thunder Bay, ON P7B 5M4</b>		Site Location: <b>Hwy 11, Beardmore, ON</b>					
Phone: <b>807.623.9495</b> Fax: <b>807.623.8070</b>	Phone: Fax:	Site #:	<input type="checkbox"/> 1 Day (100%)				
Email: <b>jason.goodman@exp.com</b>	Email:	Sampled By: <b>Elwin Farkas</b>	<input type="checkbox"/> 2 Days (50%)				
			<input type="checkbox"/> 3-4 Days (25%)				
<b>MOE REGULATED DRINKING WATER OR WATER INTENDED FOR HUMAN CONSUMPTION MUST BE SUBMITTED ON THE MAXXAM DRINKING WATER CHAIN OF CUSTODY</b>			<b>ANALYSIS REQUESTED</b>		<b>Rush Confirmation #:</b>		
<b>REGULATION 153 (2011)</b>		<b>OTHER REGULATIONS</b>		<b>Date Required:</b>		<b>LABORATORY USE ONLY</b>	
<input type="checkbox"/> Table 1 <input type="checkbox"/> Res/Park <input type="checkbox"/> Med/Fine	<input type="checkbox"/> CCME <input type="checkbox"/> Sanitary Sewer Bylaw					<b>CUSTODY SEAL (Y/N)</b> Temperature (°C) on Receipt	
<input type="checkbox"/> Table 2 <input type="checkbox"/> Ind/Comm <input type="checkbox"/> Coarse	<input type="checkbox"/> MISA <input type="checkbox"/> Storm Sewer Bylaw					<input type="checkbox"/> Present <b>N</b> <b>313/14</b>	
<input type="checkbox"/> Table 3 <input type="checkbox"/> Agri/Other	<input type="checkbox"/> PWQO Municipality:					<input type="checkbox"/> Intact <b>N/A</b>	
<input type="checkbox"/> Table	<input type="checkbox"/> Other (Specify):					<input type="checkbox"/> COOLING MEDIA PRESENT (Y/N) <b>YES</b>	
FOR RSC (PLEASE CIRCLE) Yes / <b>No</b>		<input type="checkbox"/> REG 558 (MINIMUM 3 DAY TAT REQUIRED)					
<b>Include Criteria on Certificate of Analysis (Y/N)? <u>Y</u></b>							
<b>SAMPLES MUST BE KEPT COOL (&lt; 10 °C) FROM TIME OF SAMPLING UNTIL DELIVERY TO MAXXAM</b>							
<b>SAMPLE IDENTIFICATION</b>	<b>DATE SAMPLED</b>	<b>TIME SAMPLED</b>	<b>MATRIX</b>	<b># OF CONT.</b>	<b>COMMENTS / TAT COMMENTS</b>		
1 BH1-S4	13-Dec-18	8:45	Soil	1			
2 BH2-S8	14-Dec-18	12:15	Soil	1			
3 BH3-S5	12-Dec-18	9:00	Soil	1			
4 BH5-S5	13-Dec-18	10:00	Soil	1			
5							
6							
7							
8							
9							
10							
<b>RELINQUISHED BY: (Signature/Print)</b>		<b>DATE: (YYYY/MM/DD)</b>	<b>TIME:</b>	<b>RECEIVED BY: (Signature/Print)</b>	<b>DATE: (YYYY/MM/DD)</b>	<b>TIME:</b>	<b># JARS USED AND NOT SUBMITTED</b>
Michael Suslyk		2019/01/15	12:00	<i>Michelle Brescacin</i>	2019/01/15	14:14	
				<i>Jason Goodman</i>	2019/01/17	09:43	






Maxxam Analytics International Corporation o/a Maxxam Analytics

1/17/2



## **Appendix F – Results of Stability Analysis**

Nissiamkikan Creek Culvert  
Hwy 11  
New North Side Slope (Inlet) with 2H:1V Slope  
Drained Condition

Color	Name	Model	Unit Weight (kN/m <sup>3</sup> )	Cohesion' (kPa)	Phi' (°)
	Gravelly Sand Fill	Mohr-Coulomb	21	0	34
	Rockfill: Crushed Gravel	Mohr-Coulomb	22	0	35
	Sand (Dense)	Mohr-Coulomb	21	0	32
	Silt (Compact)	Mohr-Coulomb	19	0	29
	Silty Sand	Mohr-Coulomb	20	0	30
	Silty Sand and Gravel	Mohr-Coulomb	21	0	33

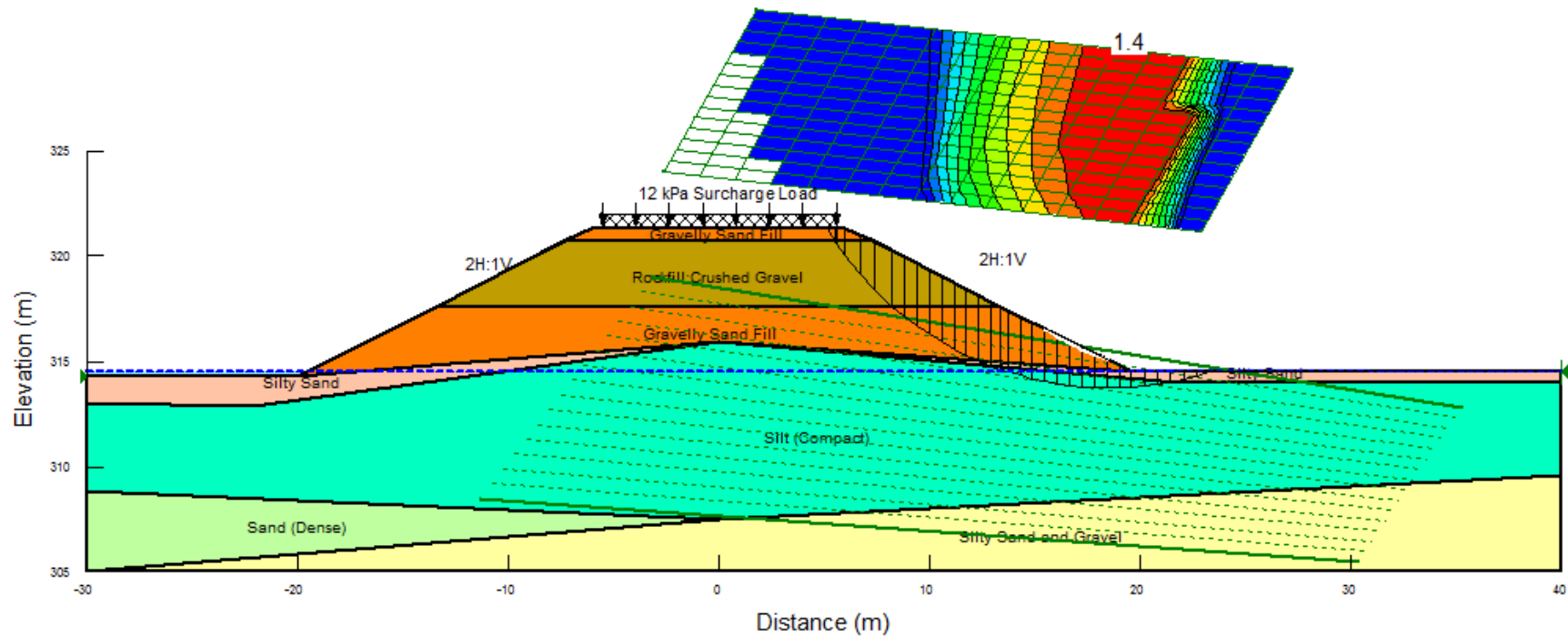


Figure F1: Drained condition- North side of embankment with 2H:1V slope geometry

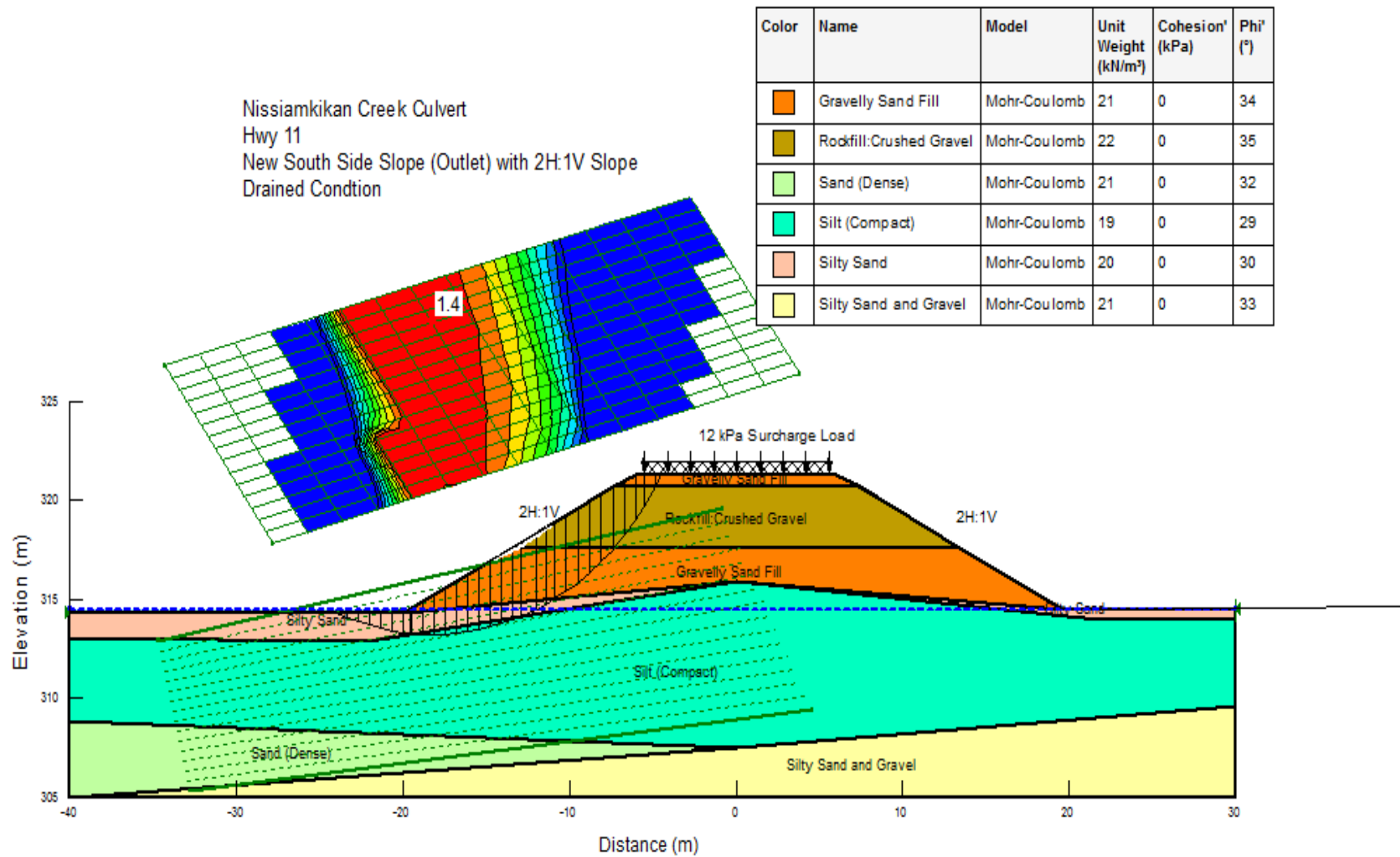
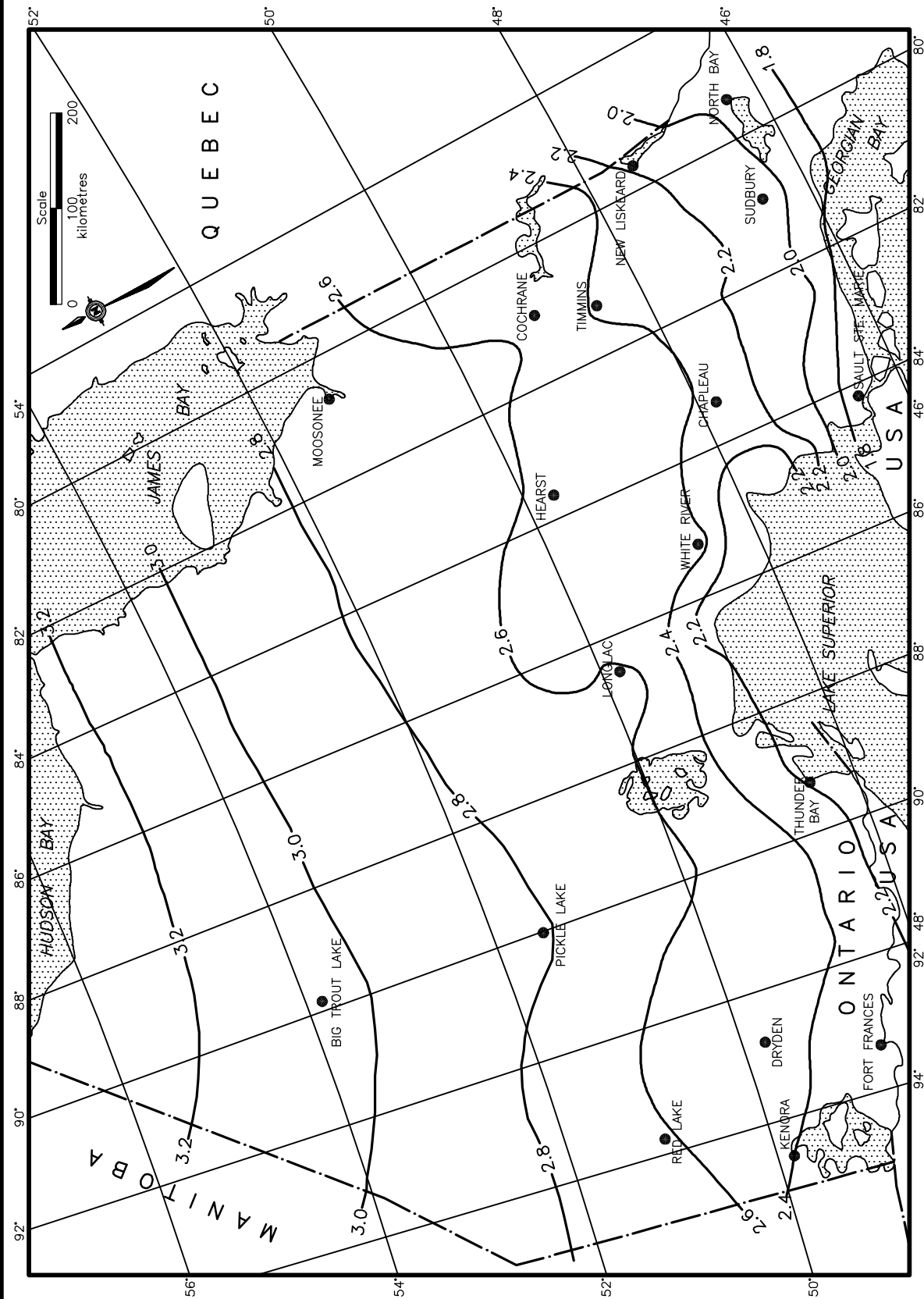


Figure F2: Drained condition- South side of embankment with 2H:1V slope geometry

## **Appendix G – OPSD and SP**



# NOTES:

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

ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2010

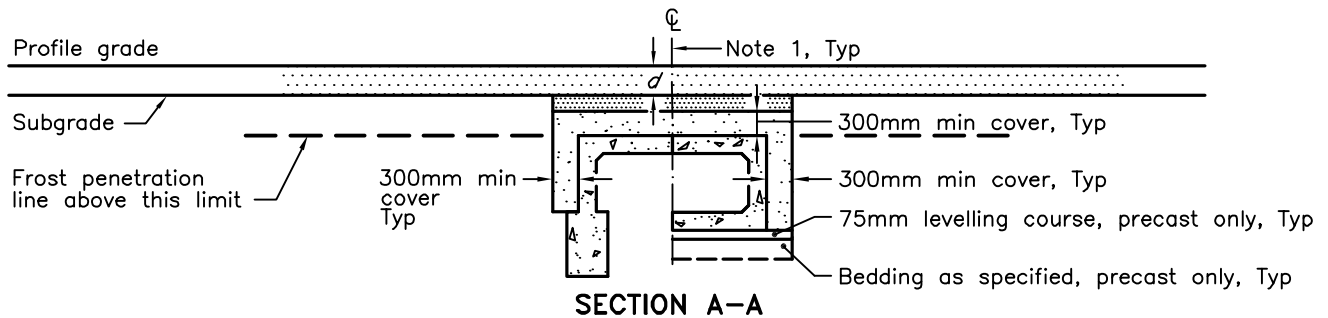
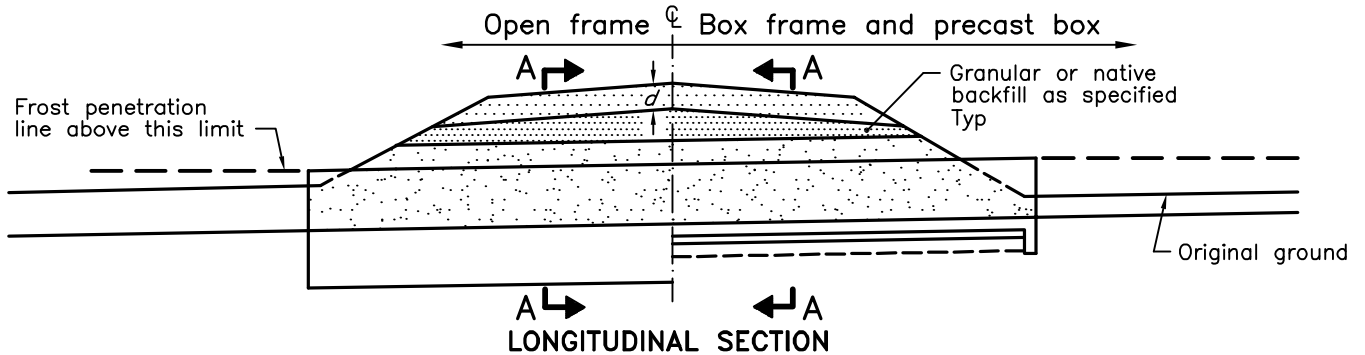
Rev 1

## FOUNDATION FROST PENETRATION DEPTHS FOR NORTHERN ONTARIO

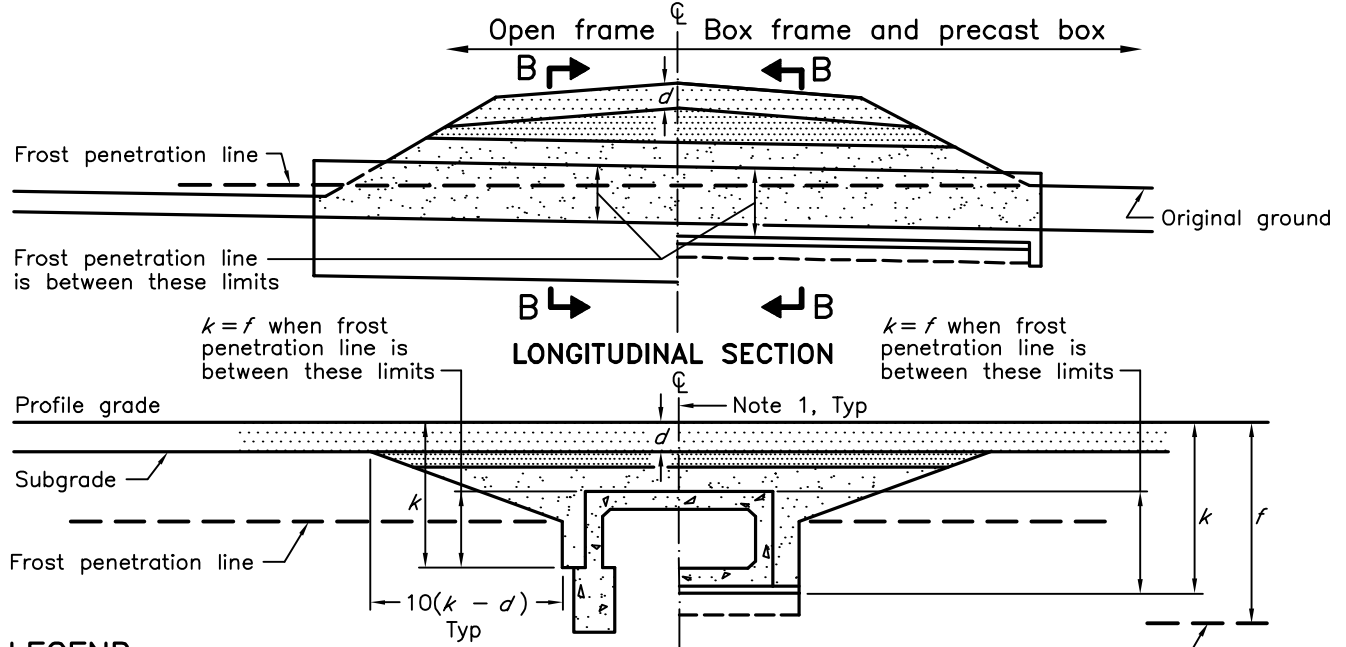
OPSD 3090.100



## FROST PENETRATION LINE AT OR ABOVE TOP OF CULVERT



## FROST PENETRATION LINE BELOW TOP OF CULVERT



### LEGEND:

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

### NOTES:

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

ONTARIO PROVINCIAL STANDARD DRAWING

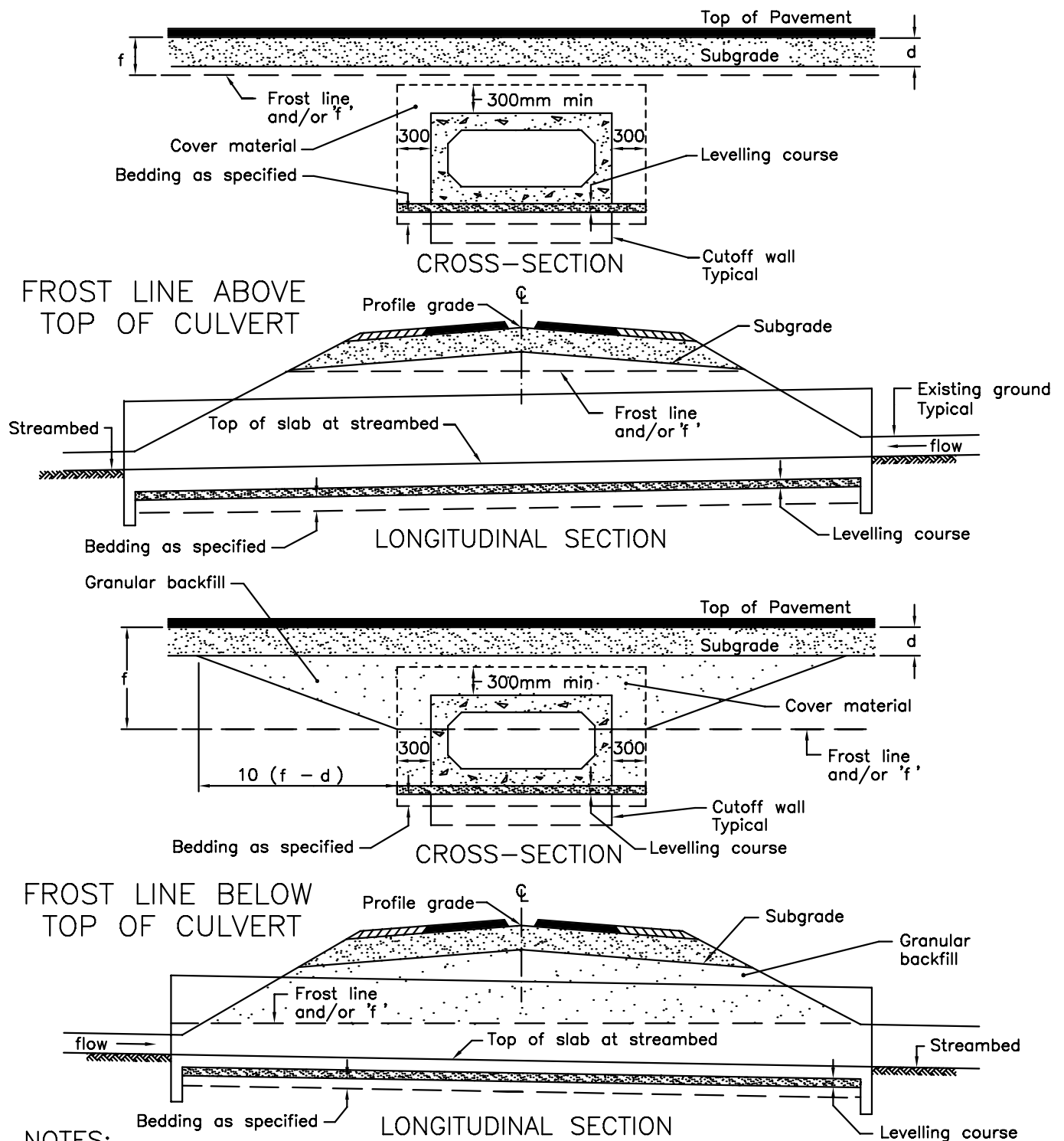
Nov 2015

Rev 3

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

**OPSD 803.010**





NOTES:

- A Bedding, levelling, cover and backfill material to be granular as specified.
- B Where frost line is below bottom of levelling course, frost tapers start at the bottom of levelling course.
- C All dimensions are in millimetres unless otherwise shown.

LEGEND:

d = Denotes depth of granular (roadbed)  
 f = Depth of frost treatment=\_\_\_\_  
 (measured from profile grade)

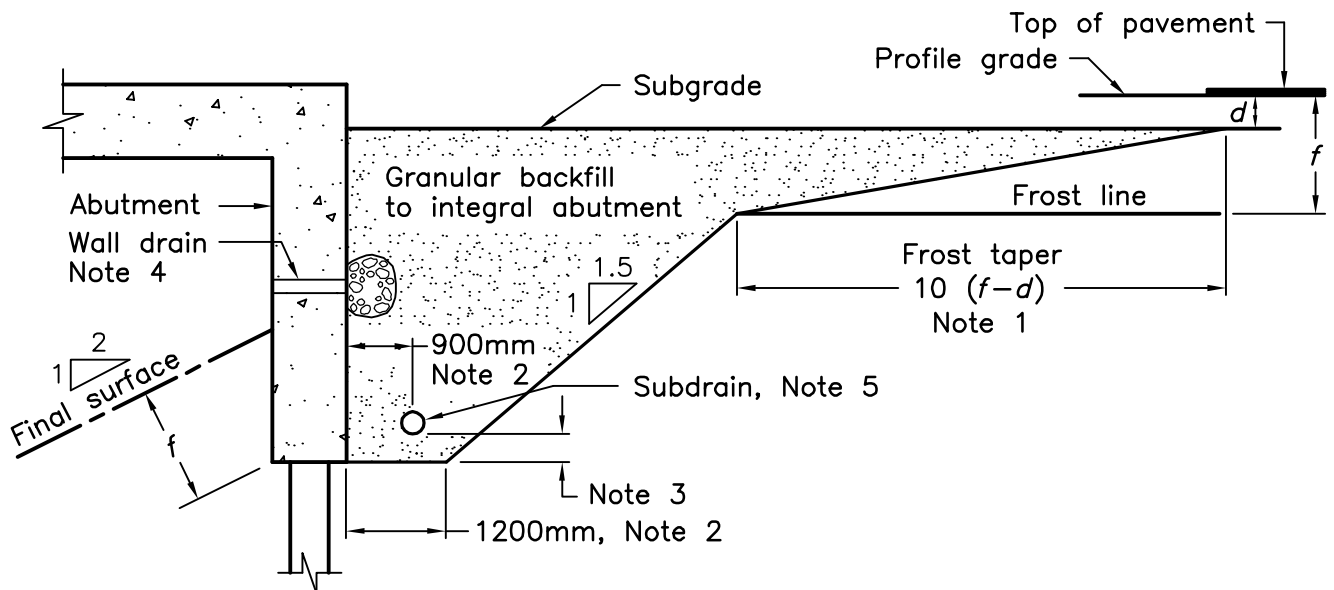
MINISTRY OF TRANSPORTATION ONTARIO DRAWING

Date | 1994 05 25 | Rev |

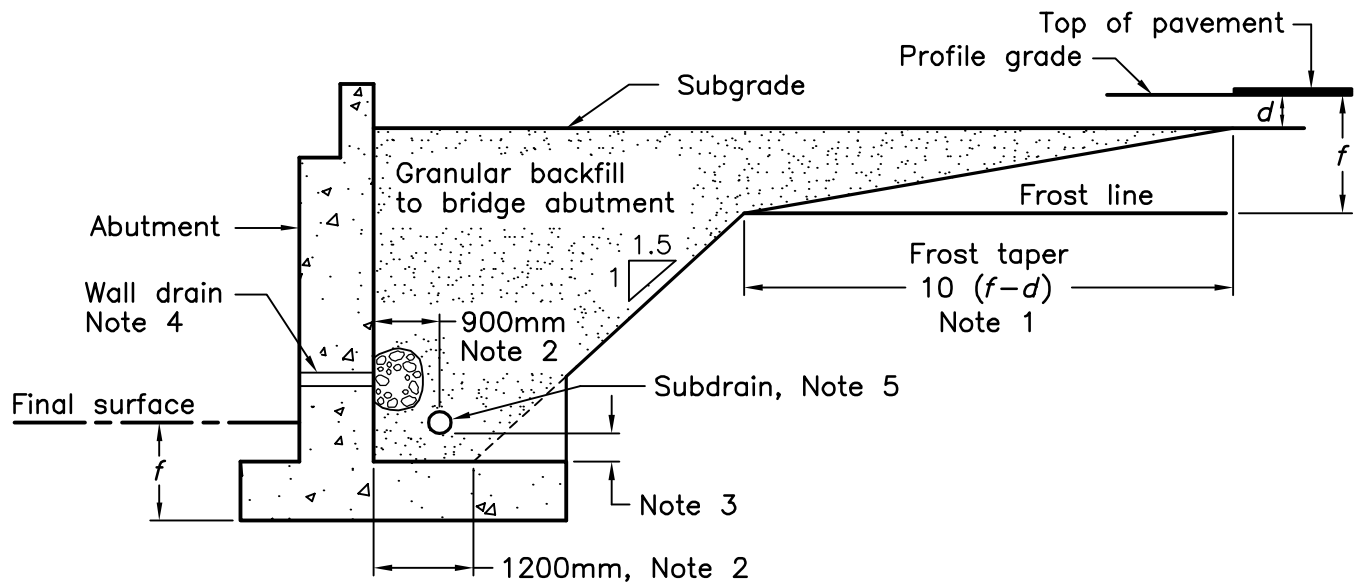
**BEDDING AND BACKFILL**  
 FOR PRECAST CONCRETE BOX CULVERTS

Issue Date  
 WP  
 Issued by

MTOD - 803.021



### INTEGRAL ABUTMENT



### ABUTMENT

#### NOTES:

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

ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2010

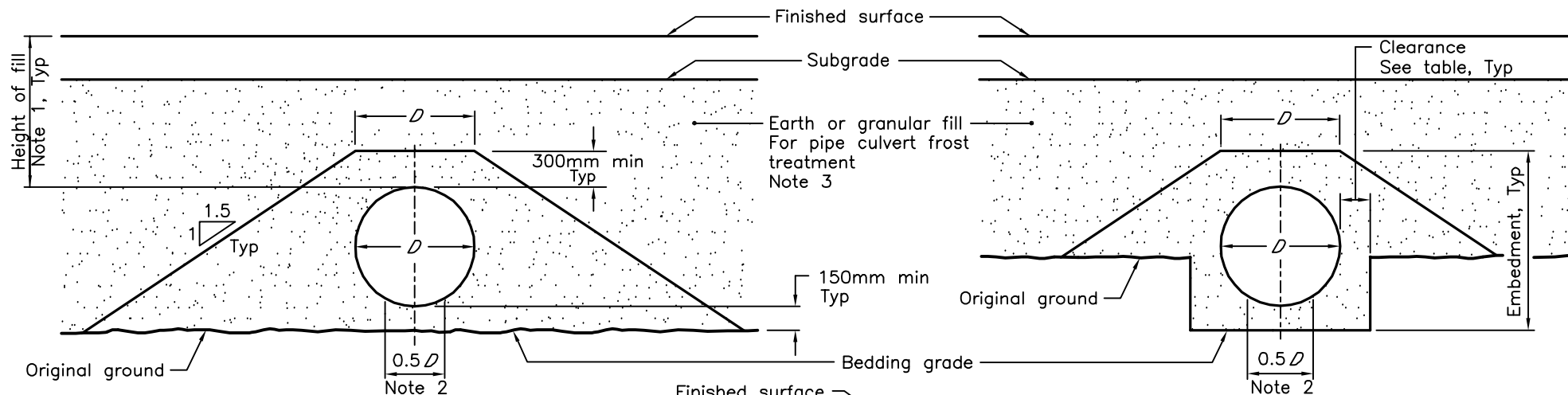
Rev 1



**WALLS**  
**ABUTMENT, BACKFILL**  
**MINIMUM GRANULAR REQUIREMENT**

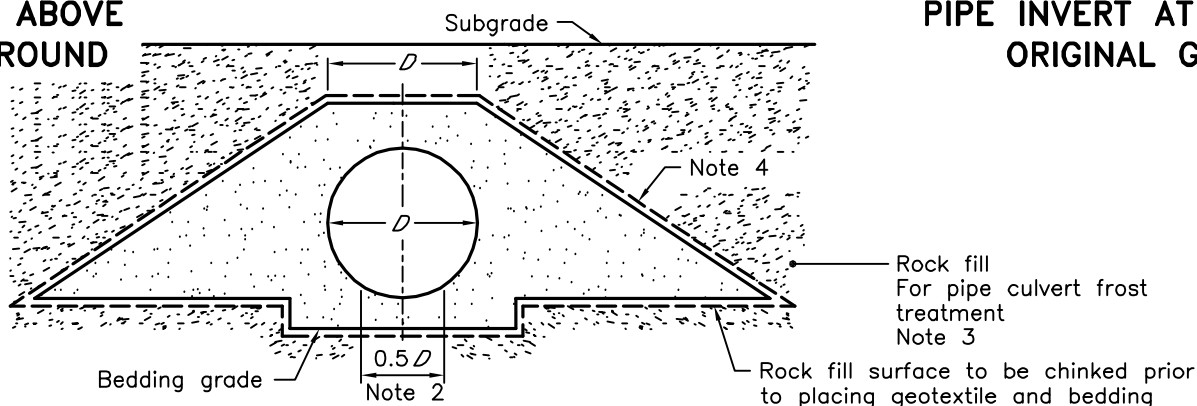
**OPSD 3101.150**





**PIPE INVERT ABOVE  
ORIGINAL GROUND**

**PIPE INVERT AT OR BELOW  
ORIGINAL GROUND**



**PIPE EMBEDMENT**

**WITH ROCK FILL UNDER AND OVER THE PIPE**

**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 Embedment material shall be wrapped in non-woven geotextile when specified.

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

B All dimensions are in metres unless otherwise shown.

**CLEARANCE TABLE**

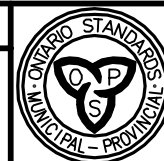
Pipe Inside Diameter mm	Clearance mm
900 or less	300
Over 900	500

ONTARIO PROVINCIAL STANDARD DRAWING

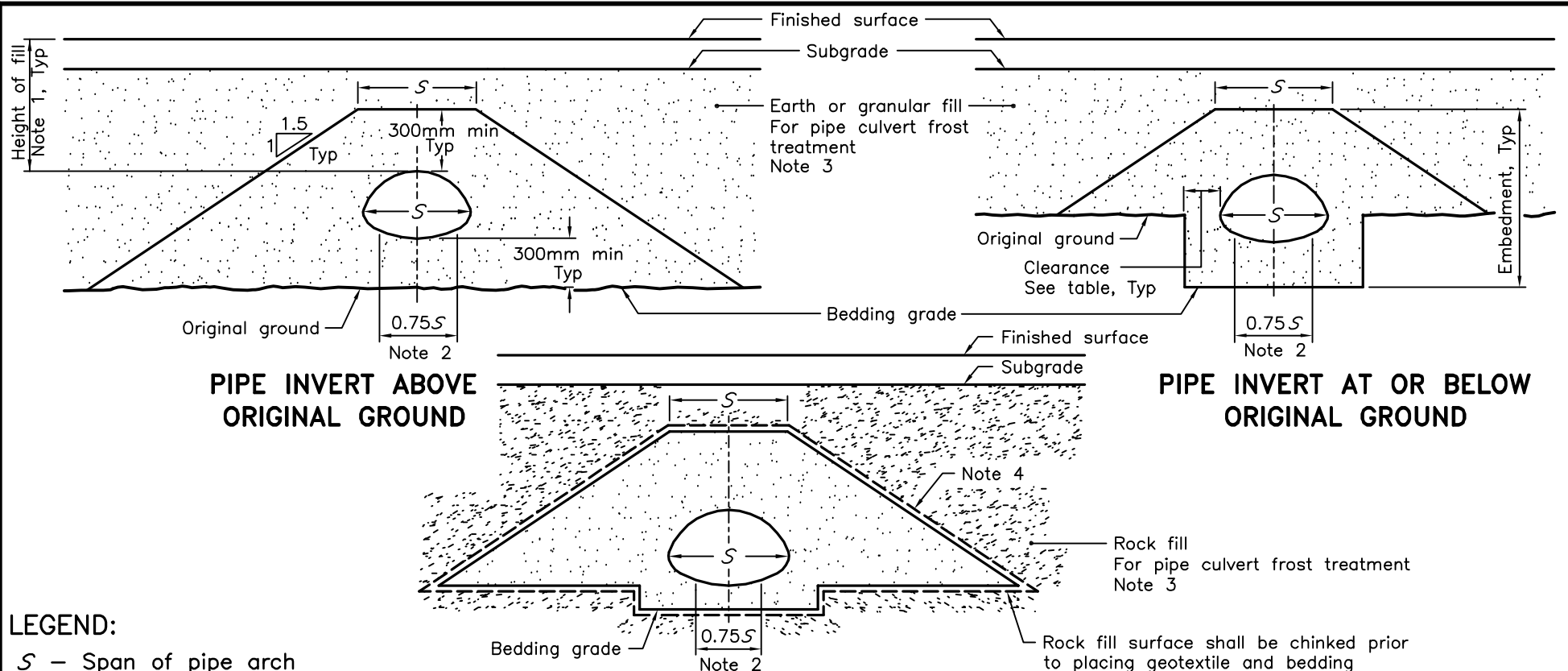
Nov 2014 Rev 3

**FLEXIBLE PIPE EMBEDMENT  
IN EMBANKMENT**

**ORIGINAL GROUND: EARTH OR ROCK**



**OPSD 802.014**



## NOTES:

- 1 Height of fill is measured from the finished surface to top of pipe. Height of fill over pipe shall be according to OPSD 805.020.
- 2 For width  $0.75S$ , granular material shall be uncompacted for its full depth and fine graded to shape of bottom of the pipe arch.
- 3 Pipe culvert frost treatment shall be according to OPSD 803.030 and 803.031.
- 4 Embedment material shall be wrapped in non-woven geotextile when specified.
- A Granular material placed in the haunch area, other than the area included in Note 2, shall be compacted prior to placing and compacting the remainder of the embedment material.

- B All dimensions are in metres unless otherwise shown.

CLEARANCE TABLE	
Pipe Span mm	Clearance mm
900 or less	300
Over 900	500

ONTARIO PROVINCIAL STANDARD DRAWING

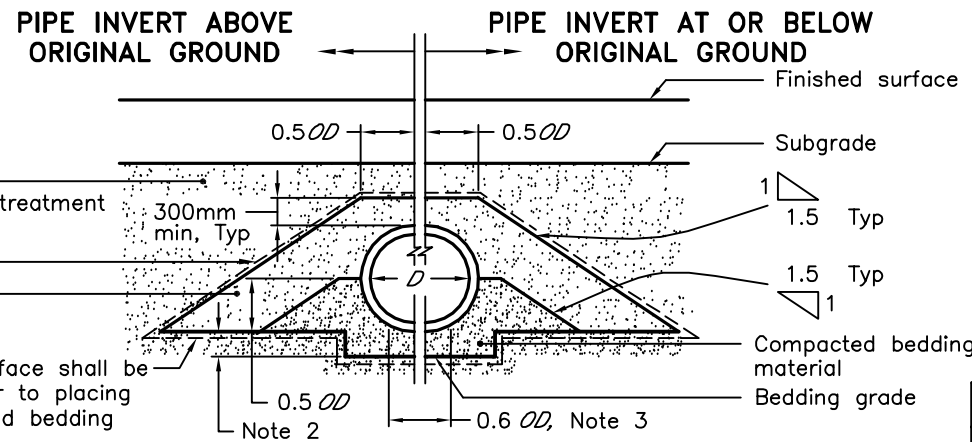
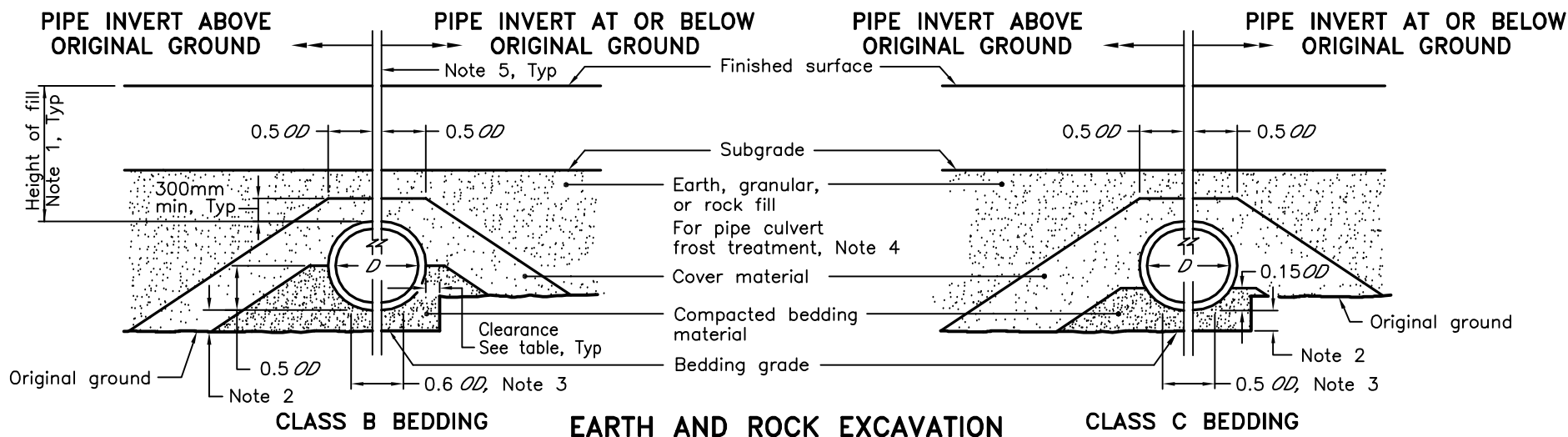
**FLEXIBLE PIPE ARCH EMBEDMENT  
IN EMBANKMENT**  
**ORIGINAL GROUND: EARTH OR ROCK**

Nov 2015

Rev 3



**OPSD 802.024**



**NOTES:**

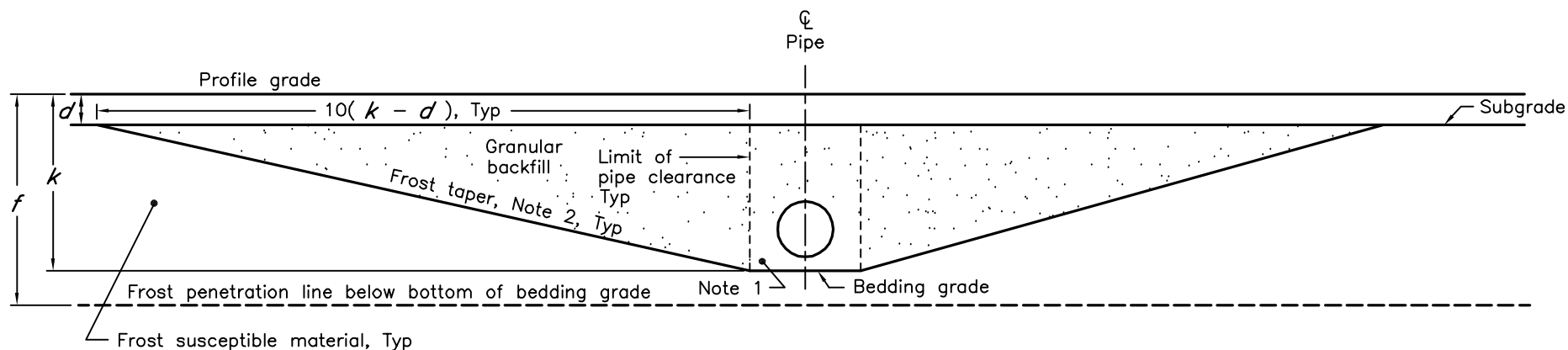
- 1 Height of fill is measured from the finished surface to top of pipe.
- 2 The minimum bedding depth below the pipe shall be  $0.15D$ , except on a rock foundation where the minimum bedding depth shall be  $0.25D$ . In no case shall the minimum dimension be less than 150mm or greater than 300mm.
- 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.
- 6 Bedding and cover material shall be wrapped in non-woven geotextile when specified.
- A All dimensions are in metres unless otherwise shown.

Rock fill surface shall be chinked prior to placing geotextile and bedding

**LEGEND:**  
 $D$  – Inside diameter  
 $OD$  – Outside diameter

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

<b>ONTARIO PROVINCIAL STANDARD DRAWING</b>		Nov 2015    Rev 3	
<b>RIGID PIPE BEDDING AND COVER IN EMBANKMENT</b>		-----	
<b>ORIGINAL GROUND: EARTH OR ROCK</b>		<b>OPSD 802.034</b>	



## FROST TREATMENT RIGID AND FLEXIBLE PIPE

### NOTES:

- 1 Pipe embedment or bedding, cover, and backfill shall be according to:
  - a) Flexible OPSD 802.010, 802.013, 802.014, 802.020, 802.023, and 802.024.
  - b) Rigid – OPSD 802.030, 802.031, 802.032, 802.033, 802.034, 802.050, 802.051, 802.052, 802.053, and 802.054.
- 2 Frost tapers shall start at bedding grade.

### LEGEND:

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

ONTARIO PROVINCIAL STANDARD DRAWING

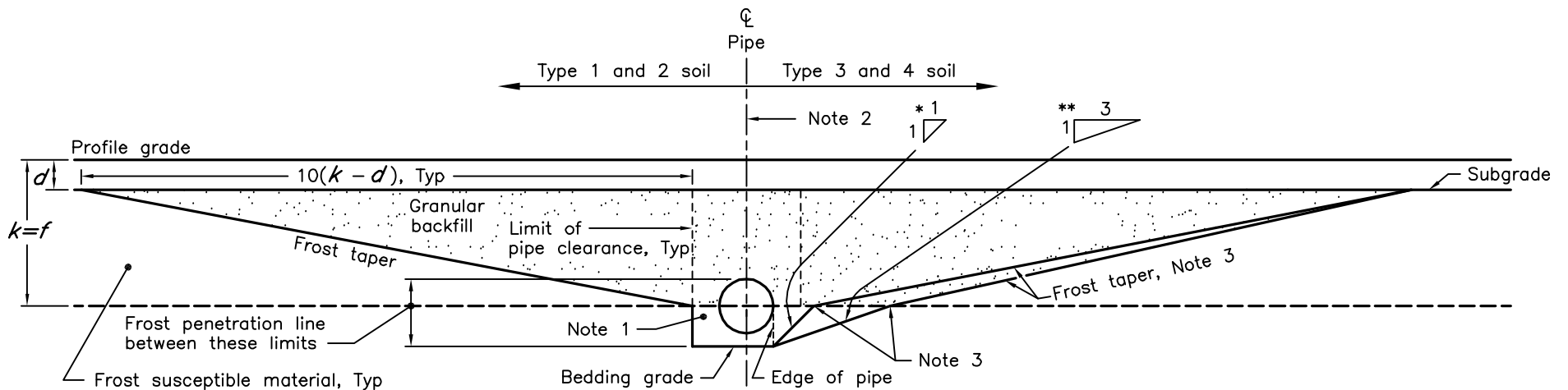
Nov 2015

Rev 3

FROST TREATMENT – PIPE CULVERTS  
FROST PENETRATION LINE BELOW  
BEDDING GRADE

OPSD 803.030





### FROST TREATMENT RIGID AND FLEXIBLE PIPE

#### NOTES:

- 1 Pipe embedment or bedding, cover, and backfill shall be according to:
  - a) Flexible OPSD 802.010, 802.013, 802.014, 802.020, 802.023, and 802.024.
  - b) Rigid – OPSD 802.030, 802.031, 802.032, 802.033, 802.034, 802.050, 802.051, 802.052, 802.053, and 802.054.
- 2 Condition of frost treatment symmetrical about centreline of pipe.
- 3 Frost tapers shall start at the intersection of the 1H:1V or 3H:1V slope and the frost penetration line.
- A Soil types as defined in the Occupational Health and Safety Act and Regulations for Construction Projects.

#### LEGEND:

- $d$  – depth of roadbed granular  
 $k$  – depth of frost treatment below profile grade  
 $f$  – depth of frost penetration below profile grade  
 $*$  – Type 3 soil  
 $**$  – Type 4 soil

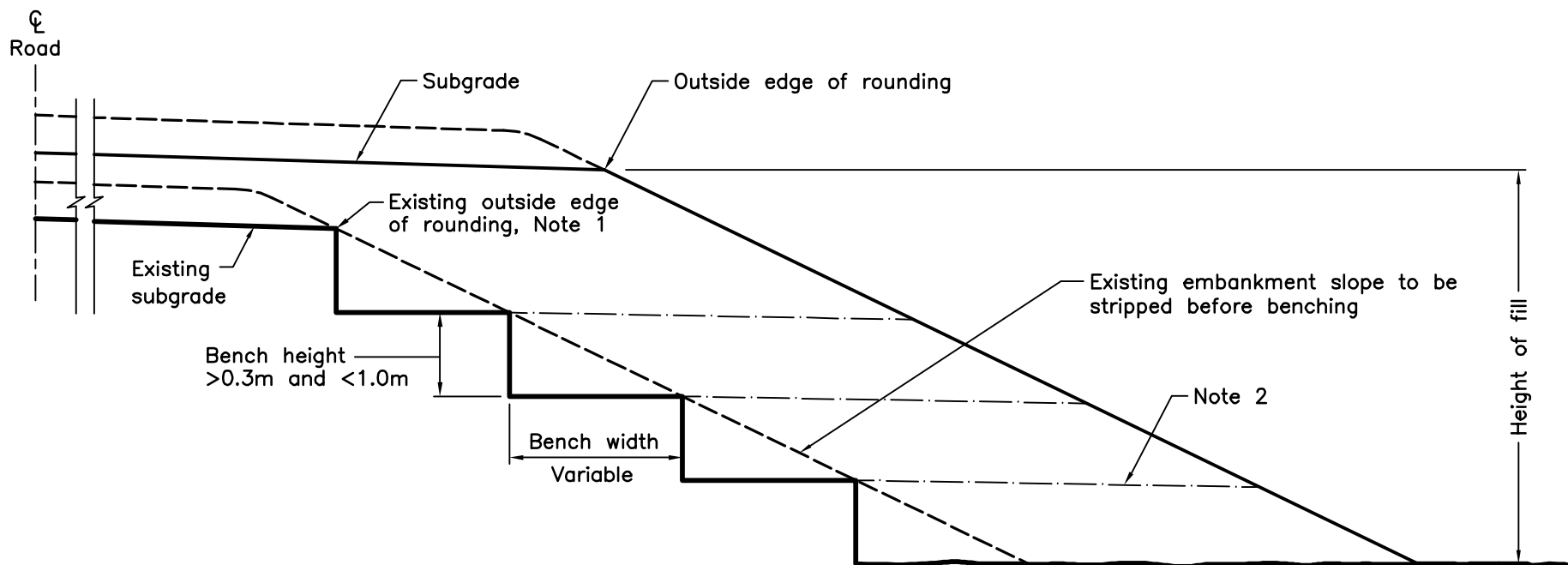
ONTARIO PROVINCIAL STANDARD DRAWING

FROST TREATMENT – PIPE CULVERTS  
FROST PENETRATION LINE BETWEEN  
TOP OF PIPE AND BEDDING GRADE

Nov 2015 Rev 4



OPSD 803.031



# NOTES:

- 1 When the subgrade is below the existing outside edge of rounding, benching shall be carried out below the point where the subgrade intersects the existing slope.
  - 2 Benches shall be excavated one level at a time and the fill placed and compacted before the next bench is excavated.
- A Benching is not required on existing slopes flatter than 3H:1V.
- B All dimensions are in metres unless otherwise shown.

ONTARIO PROVINCIAL STANDARD DRAWING

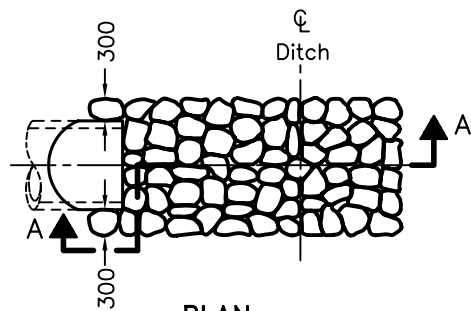
Nov 2013

Rev 3

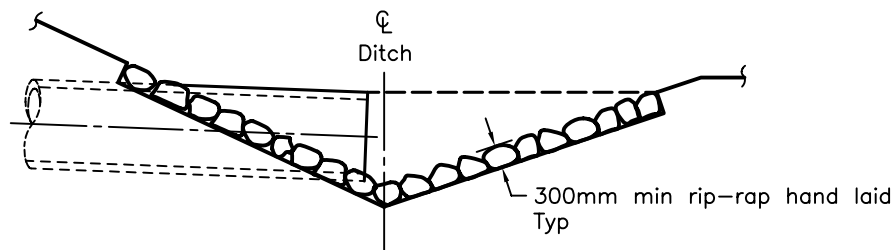
BENCHING OF EARTH SLOPES

OPSD 208.010

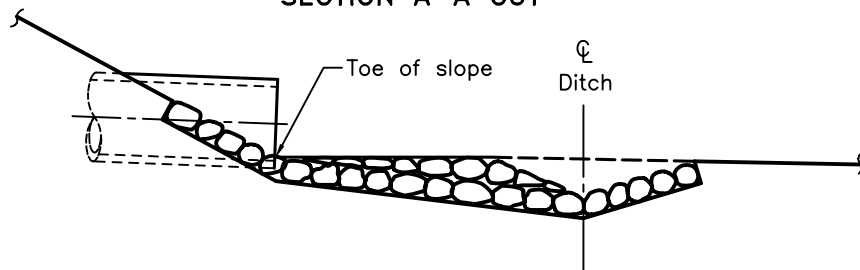




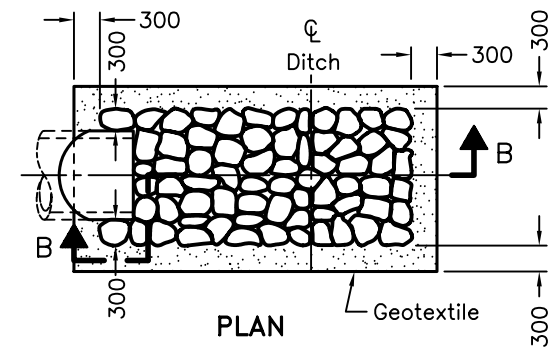
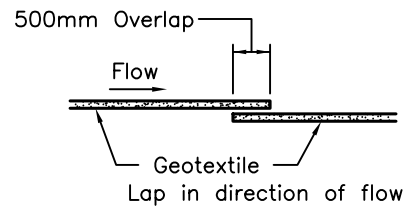
PLAN  
CUT OR FILL



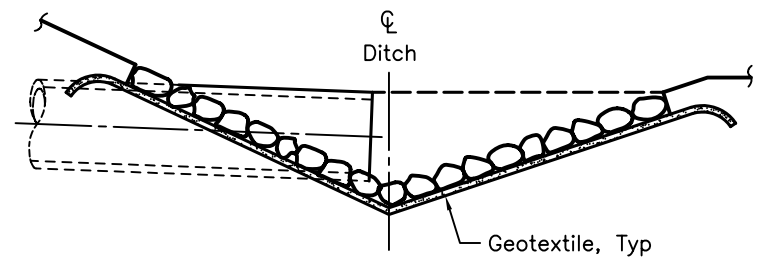
SECTION A-A CUT



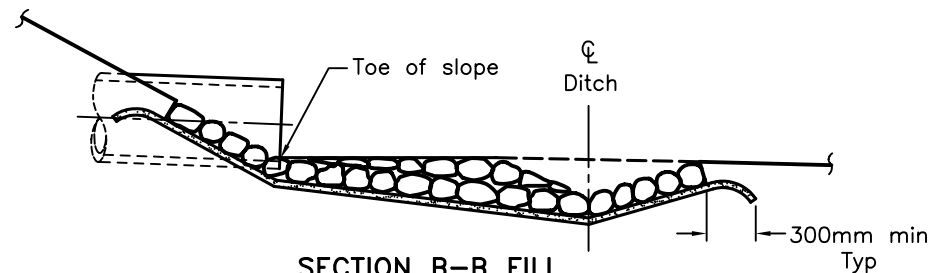
SECTION A-A FILL  
TYPE A – WITHOUT GEOTEXTILE



PLAN  
CUT OR FILL



SECTION B-B CUT



SECTION B-B FILL  
TYPE B – WITH GEOTEXTILE

**NOTES:**

A All dimensions are in millimetres unless otherwise shown.

ONTARIO PROVINCIAL STANDARD DRAWING

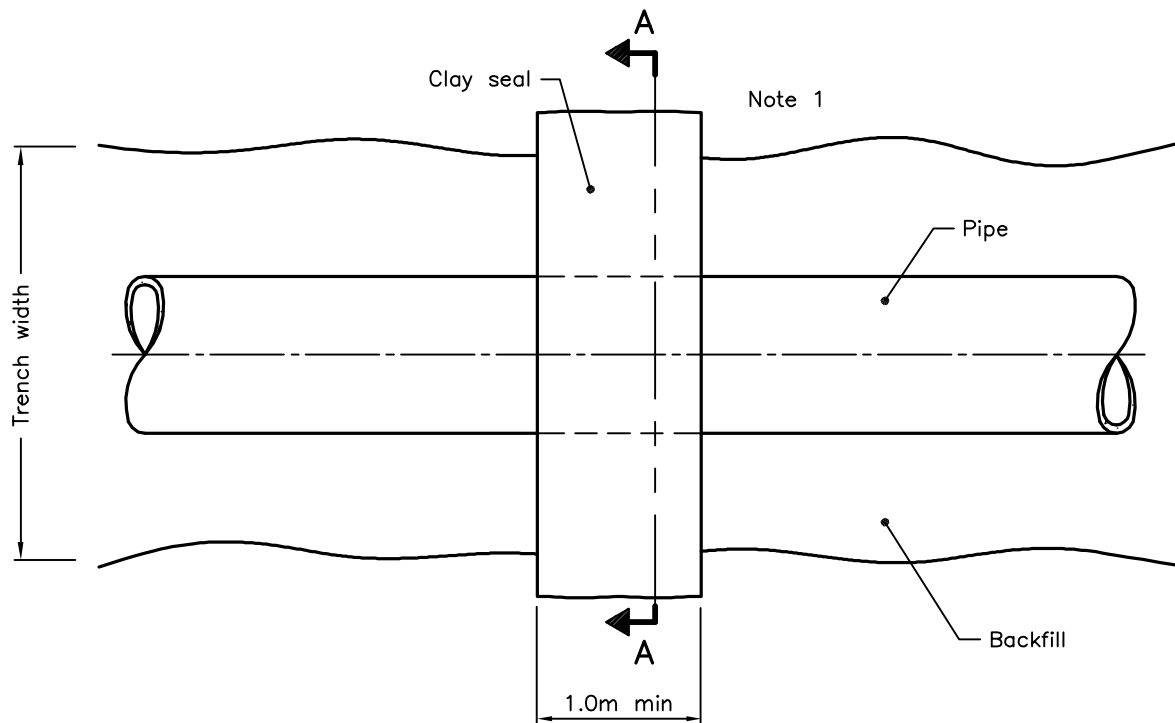
Nov 2001

Rev 0

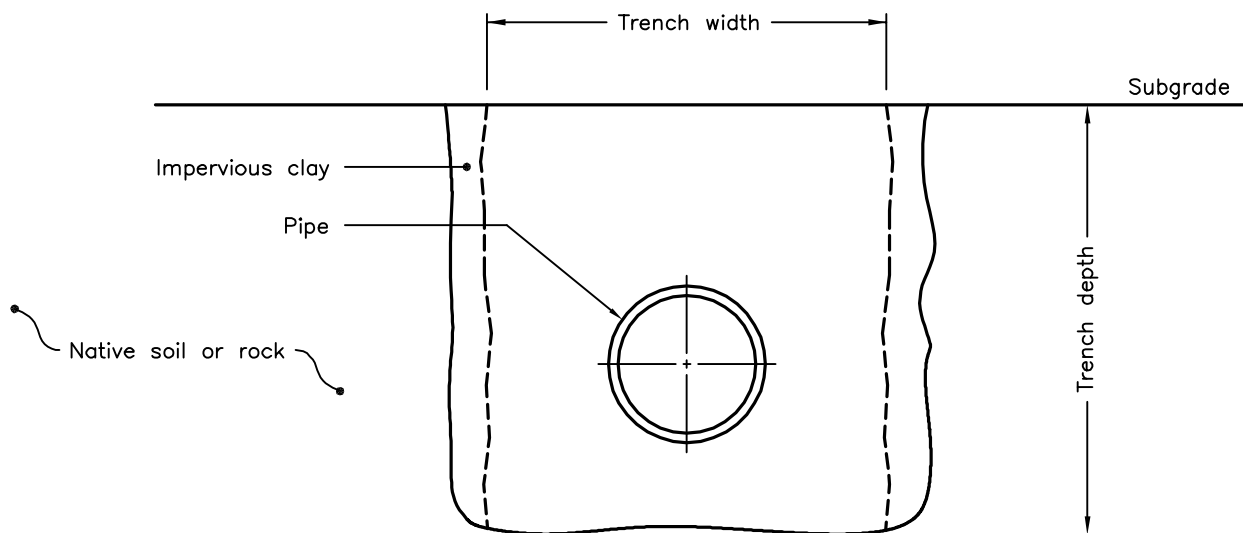
**RIP-RAP TREATMENT**  
FOR SEWER AND CULVERT OUTLETS



**OPSD – 810.010**



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 2016

Rev 2

CLAY SEAL FOR PIPE TRENCHES

OPSD 802.095





**DEWATERING SYSTEM - Item No.**  
**TEMPORARY FLOW PASSAGE SYSTEM - Item No.**

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Special Provision No. 517F01

July 2017

**Amendment to OPSS 517, November 2016**

**Design Storm Return Period and Preconstruction Survey Distance**

**517.01 SCOPE**

Section 517.01 of OPSS 517 is deleted in its entirety and replaced with the following:

This specification covers the requirements for the design, operation, and removal of a dewatering or temporary flow passage system or both to control water during construction, and the control of the water prior to discharge to the natural environment and sewer systems.

**517.04 DESIGN AND SUBMISSION REQUIREMENTS**

**517.04.01 Design Requirements**

Subsection 517.04.01 of OPSS 517 is amended by deleting the first paragraph in its entirety and replacing it with the following:

A dewatering or temporary flow passage system or both shall be designed to control water at the locations specified in the Contract Documents and at any other location where a system is necessary to complete the work. The design of the system shall be sufficient to permit the work at each location to be carried out as specified in the Contract Documents.

Subsection 517.04.01 of OPSS 517 is further amended by deleting the second last paragraph in its entirety and replacing it with the following:

Temporary flow passage systems shall be designed, as a minimum, for a 2 year design storm return period and groundwater discharge, except for the work specified in Table A. For the work specified in Table A, the temporary flow passage system shall be designed, as a minimum, for the design storm return period specified in Table A and groundwater discharge. A longer return period shall be used when determined appropriate for the work.

Intensity-Duration Factor (IDF) curve location, site specific minimum return period, return period flow estimates, and other information is provided in Table A. The IDF information can be accessed through the MTO IDF Curve Look up Tool on the Drainage and Hydrology page of MTO's website. The return period flow estimates do not include flow volumes from groundwater discharge. The Owner specifically excludes these flow estimates from the warranty in the Reliance on Contract Documents subsection of OPSS 100, MTO General Conditions of Contract.

**Table A**

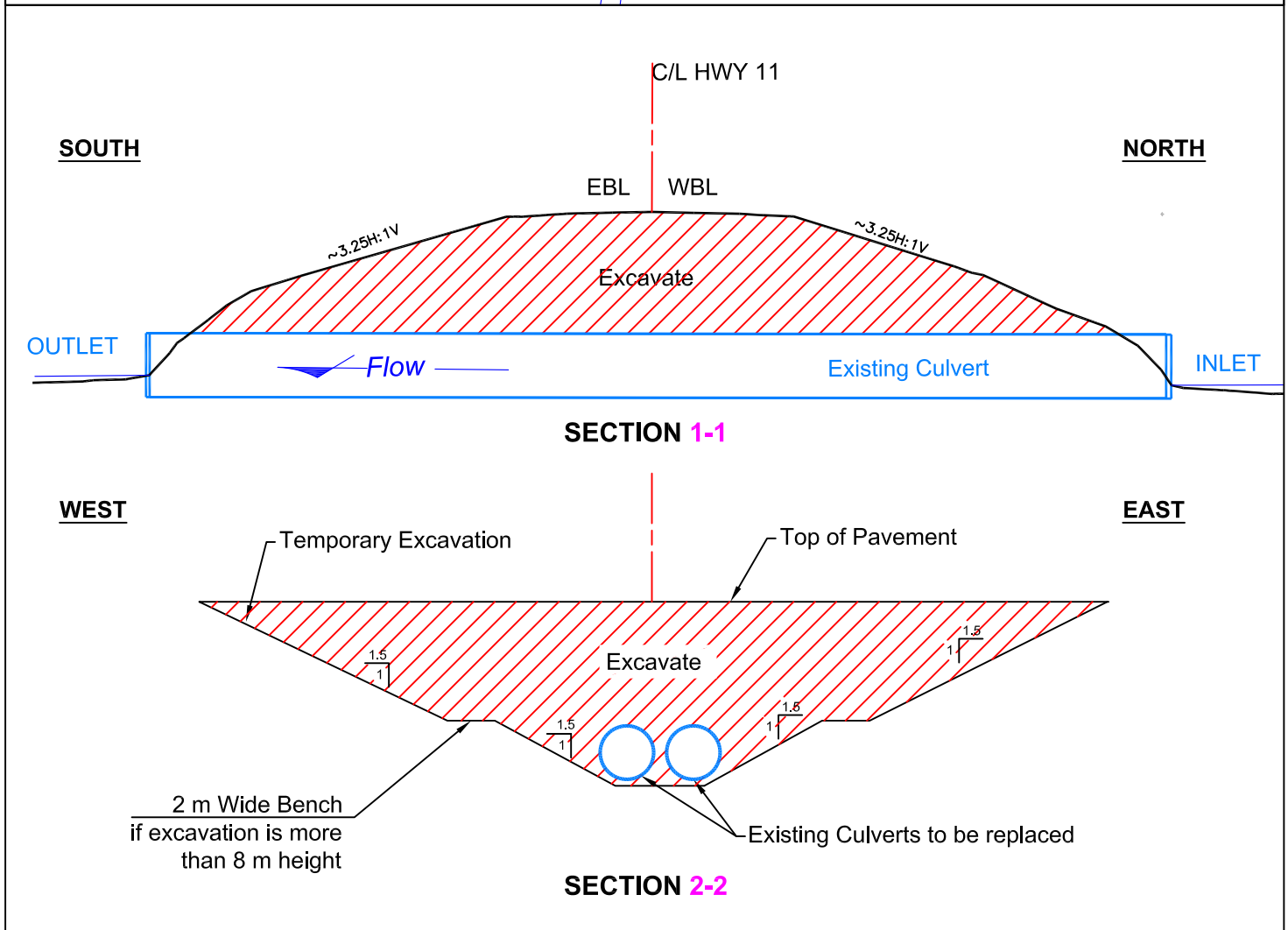
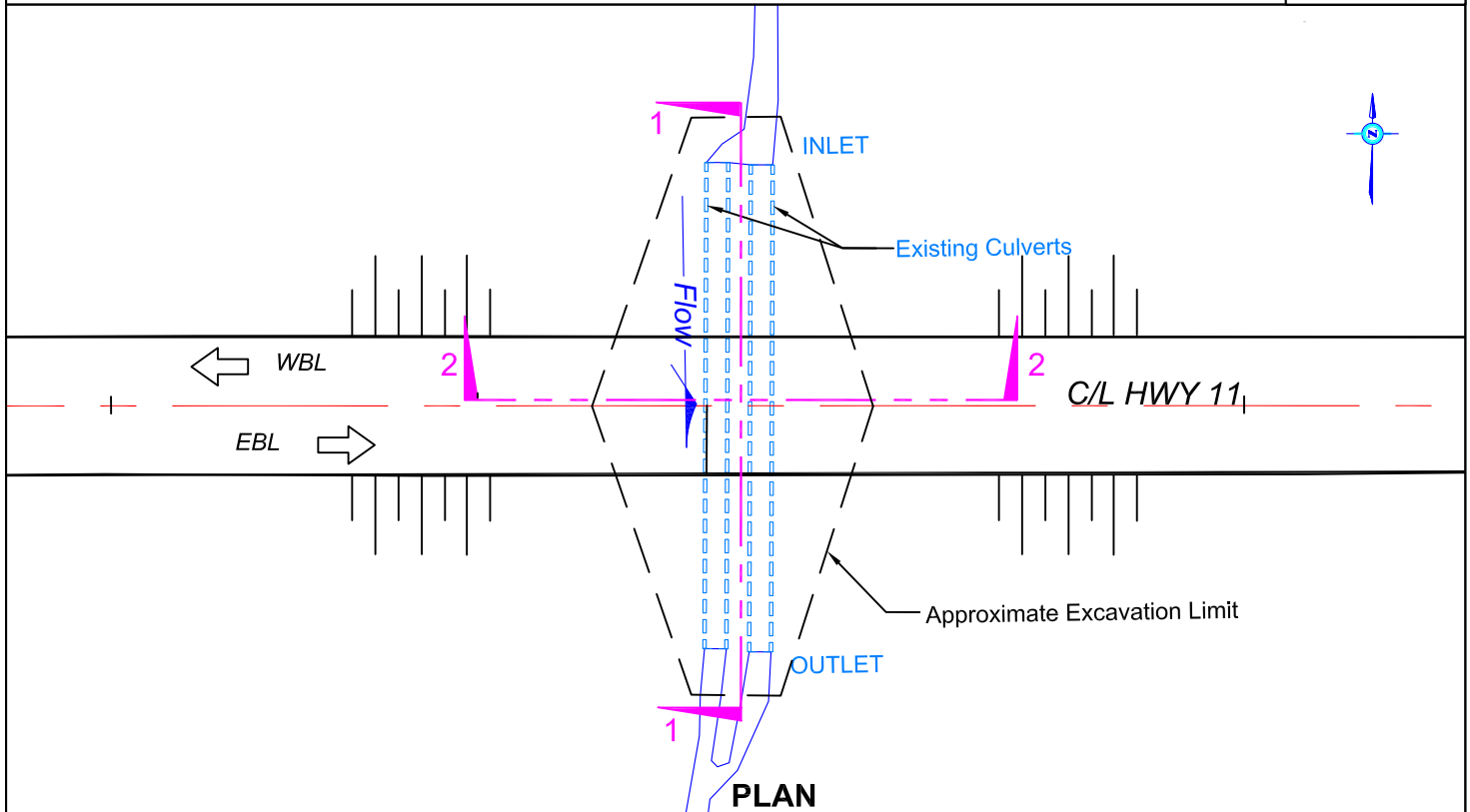
<b>IDF Curve Location</b>	Latitude: *		Longitude: *			
<b>Temporary Flow Passage Systems</b>						
Site Name / Station Reference	Minimum Return Period (Years)	Return Period Flow Estimates (m <sup>3</sup> /s)				Design Engineer Requirements (Note 1)
		2 Year	5 Year	10 Year	25 Year	
**	***	****	****	****	****	*****
<b>Dewatering Systems</b>						
Site Name / Station Reference	Preconstruction Survey Distance (Note 2) (m)				Design Engineer Requirements (Note 1)	
**	*****				*****	
<p>Note:</p> <p>1. “Yes” means the design Engineer and design-checking Engineer shall have a minimum of 5 years of experience in designing systems of similar nature and scope to the required work. “No” means a minimum experience level is not required for the design Engineer and design-checking Engineer.</p> <p>2. “N/A” indicates a preconstruction survey is not required.</p>						

## **Appendix H – Schematic Sketches for Construction Alternatives**

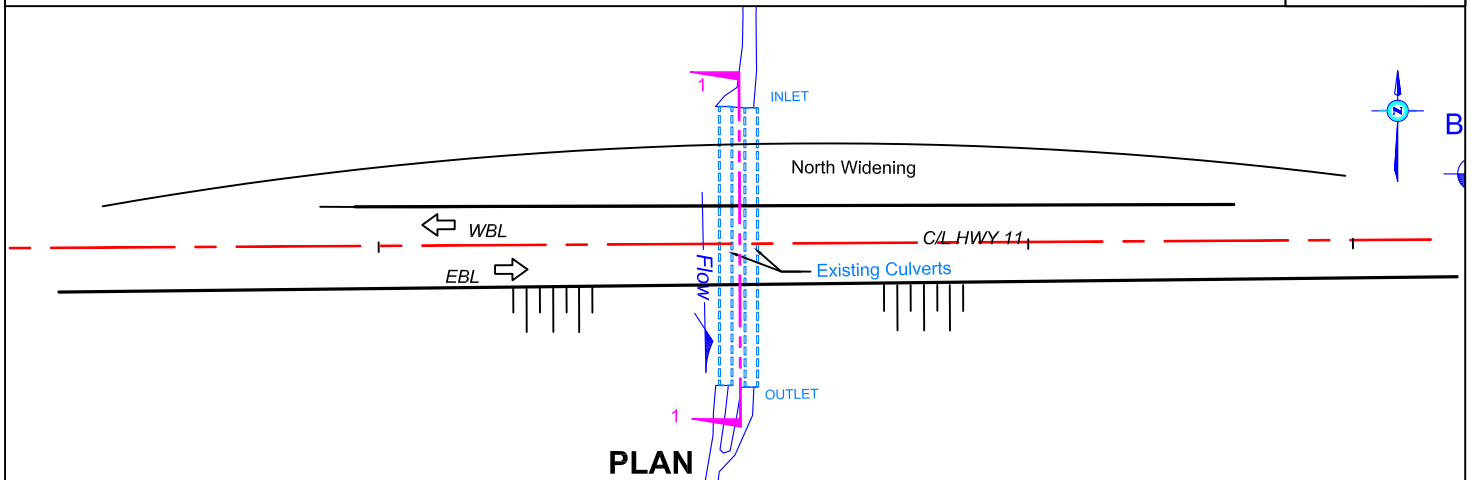
# FIGURE H1: FULL ROAD CLOSURE AND OPEN CUT UNSUPPORTED EXCAVATION OPTION



## SCHEMATIC DIAGRAMS (NTS)

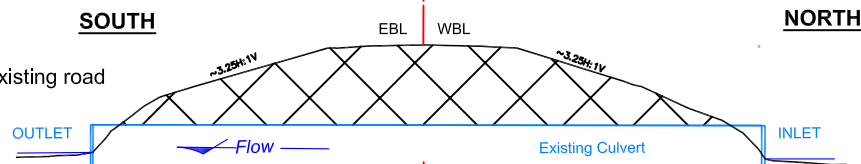


# FIGURE H2: TEMPORARY LOCAL DETOUR AND OPENT CUT UNSUPPORTED EXCAVATION SCHEMATIC DIAGRAMS (NTS)

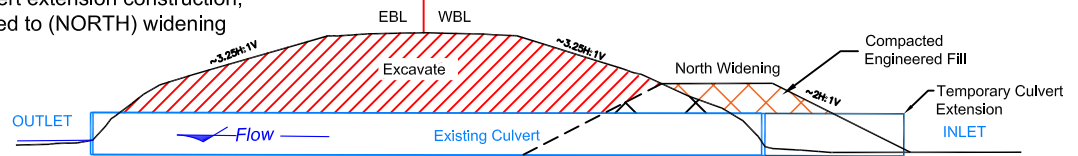


## RECOMMENDED STAGES

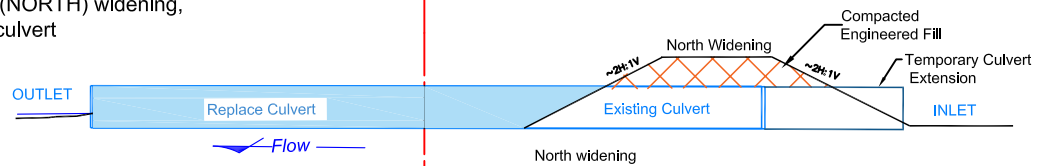
**1.0 Stage 1** - Current condition  
Two-way traffic on existing road



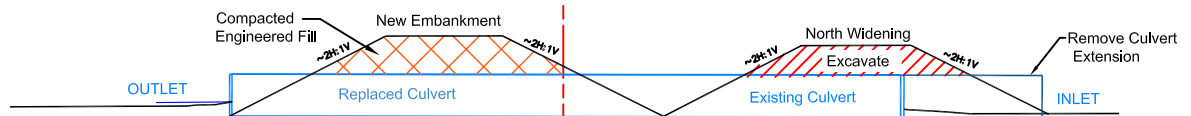
**2.0 Stage 2** - Build temporary widening (NORTH);  
Excavation and culvert extension construction;  
One-way traffic shifted to (NORTH) widening



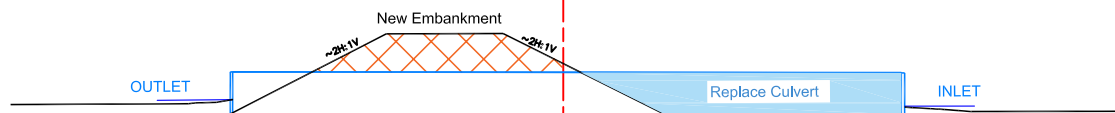
**3.0 Stage 3** One-way traffic on (NORTH) widening,  
installation of new culvert



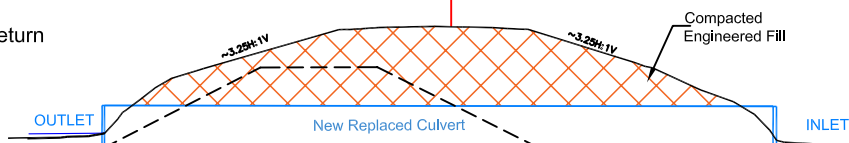
**4.0 Stage 4** - Excavation on (NORTH) widening  
One-way traffic shifted to new embankment



**5.0 Stage 5** - One-way traffic on new embankment, complete  
culvert replacement

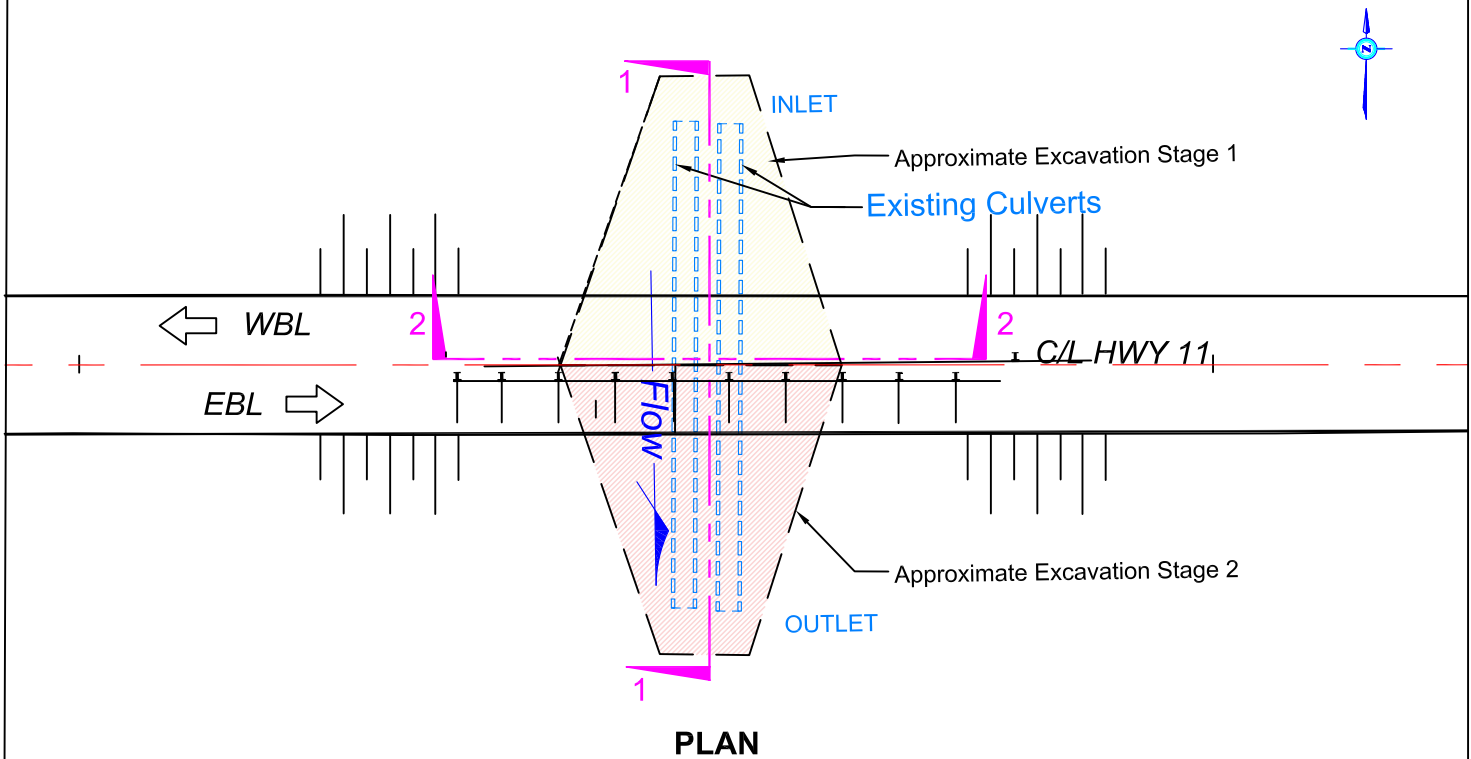


**6.0 Stage 6** - Build new embankment to original state, switching  
traffic between EBL and WBL during full height embankment  
construction  
Two-way traffic return

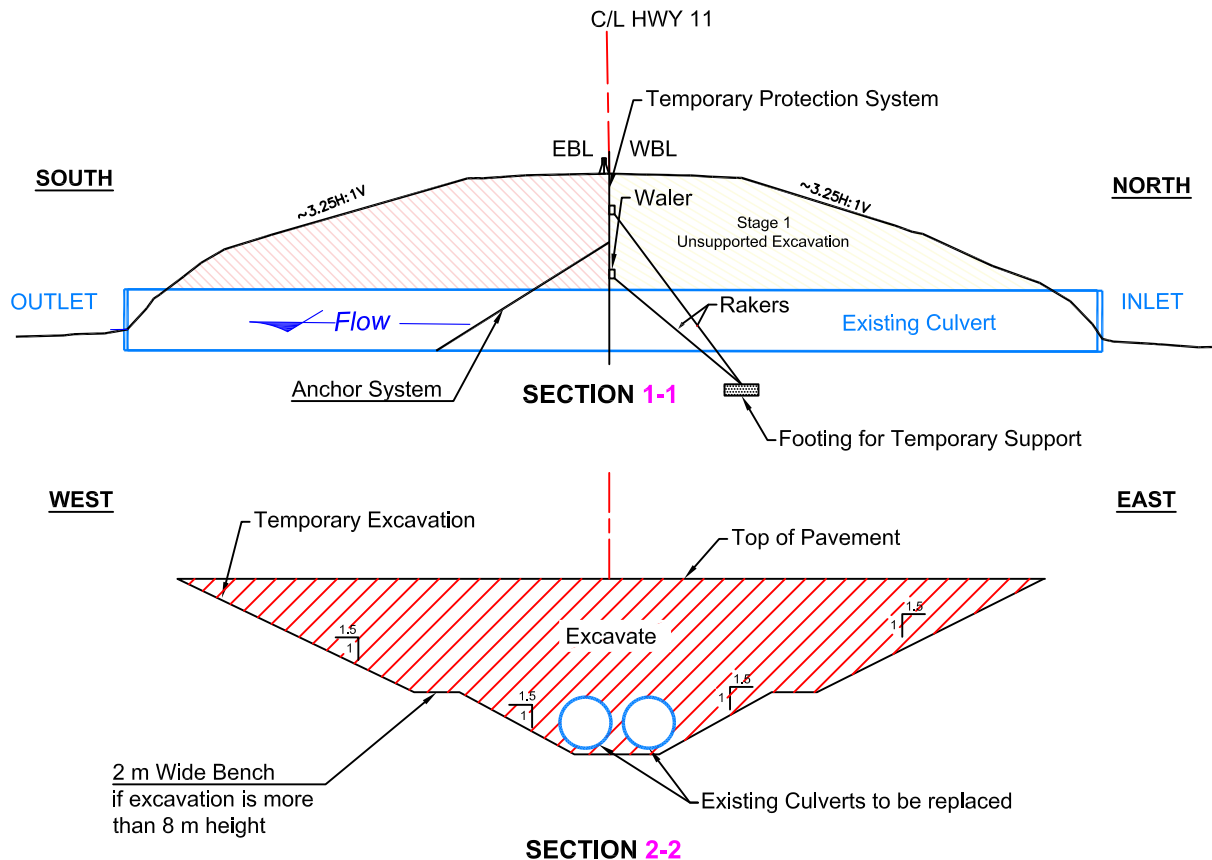


## SECTION 1-1

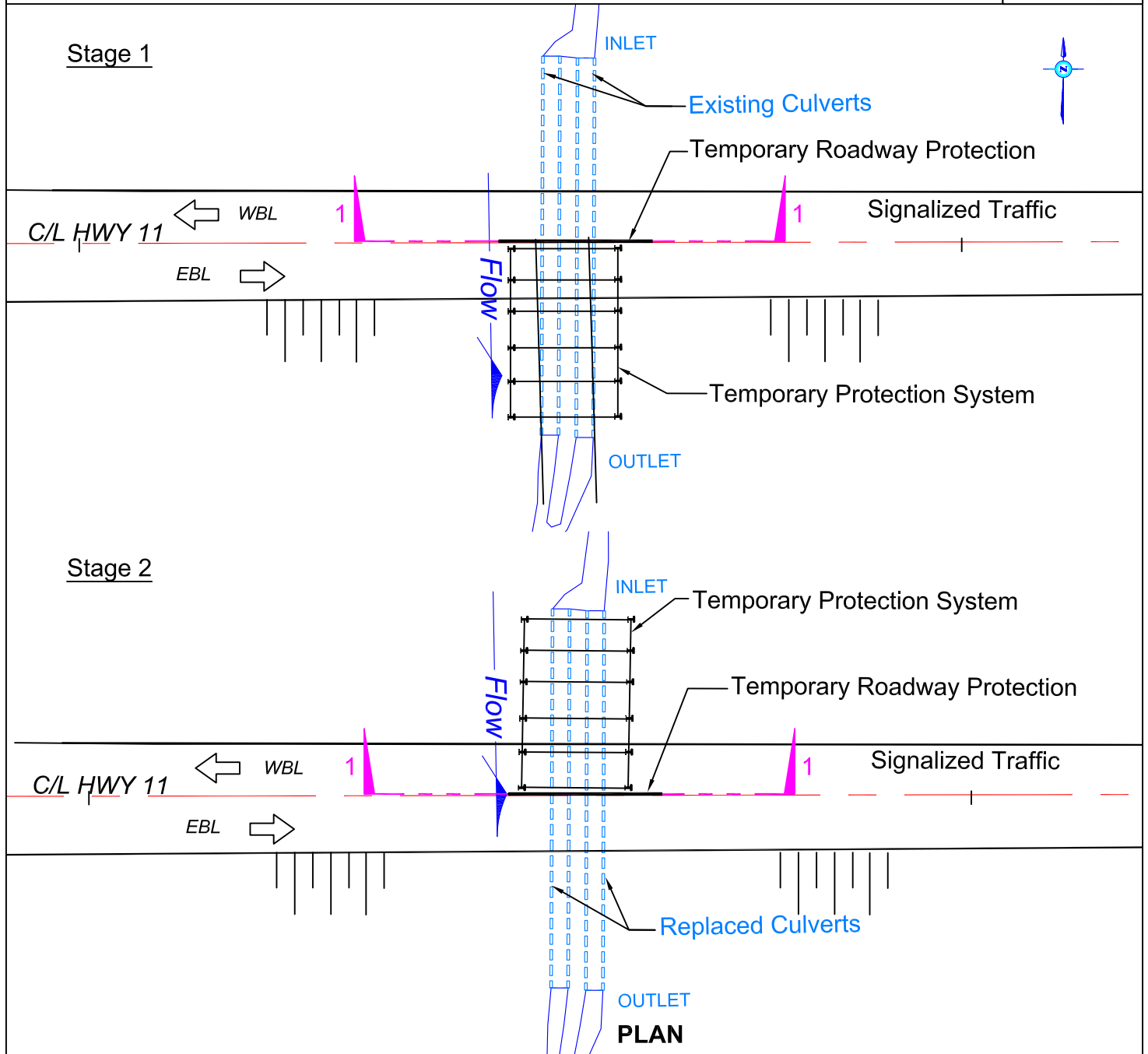
**FIGURE H3A: HALF AND HALF CONSTRUCTION, SORING SYSTEM WITH EITHER  
RAKERS OR ANCHOR  
SCHEMATIC DIAGRAMS (NST)**



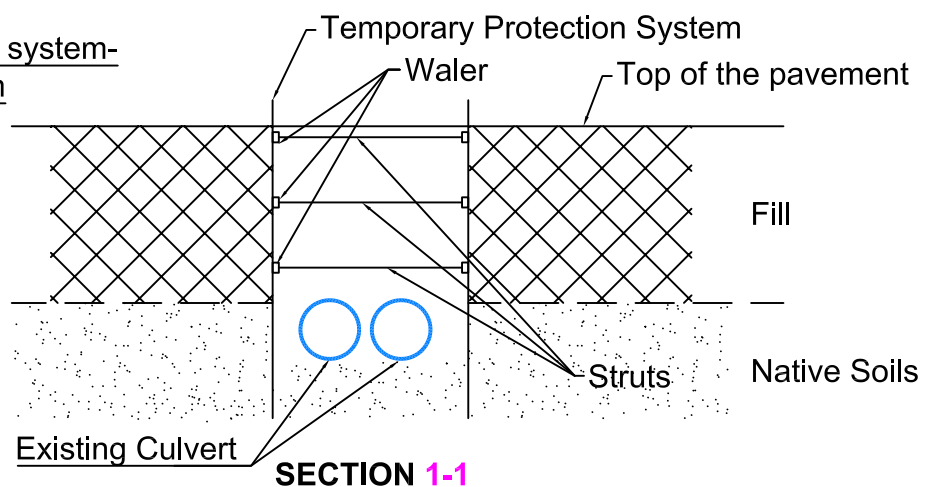
Half and Half Construction, shoring system with either rakers or anchor system- Unsupported Excavation



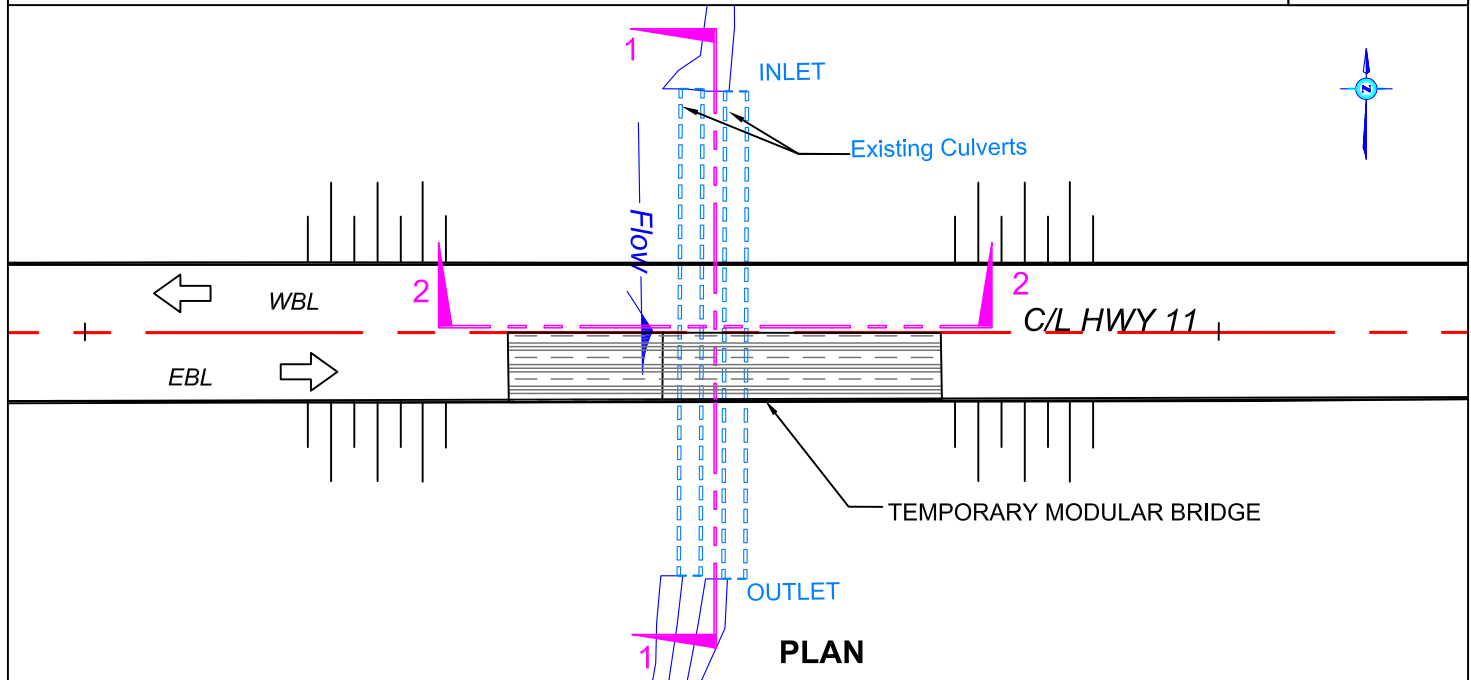
**FIGURE H3B: HALF AND HALF CONSTRUCTION WITH BRACED CUT SIDES OR ANCHOR SYSTEM**  
**SCHEMATIC DIAGRAMS (NST)**



Half and half construction, shoring system-  
braced cut struts or anchor system

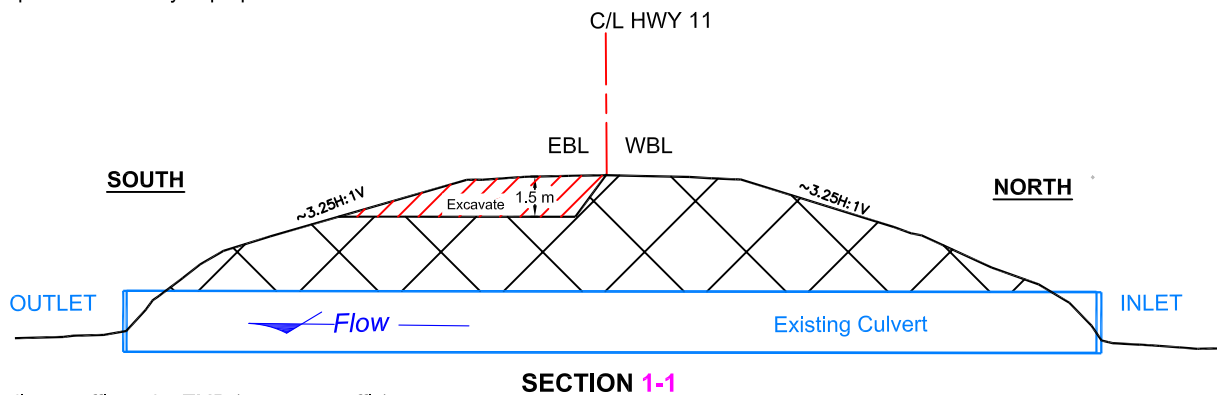


**FIGURE H4: STAGED CONSTRUCTION WITH TEMPORARY MODULAR BRIDGE**  
**SCHEMATIC DIAGRAMS (NST)**

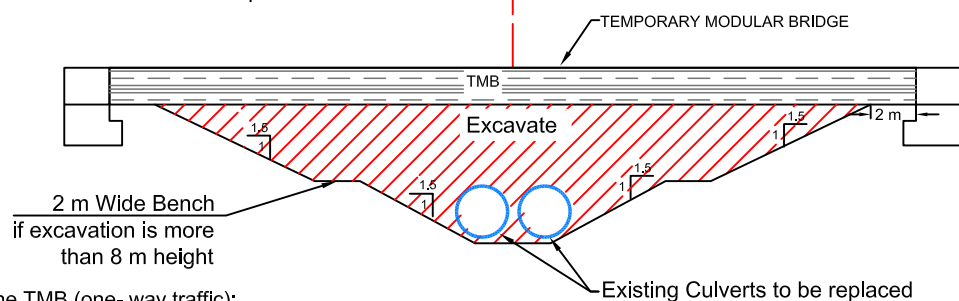


**RECOMMENDED STAGES**

**1.0 Stage 1** - Close EBL with traffic signal; one- way traffic shifted to WBL;  
 Excavation of the EBL approximately 1.5m deep under the  
 proposed Temporary Modular Bridge (TMB) with 1.5H:1V  
 slope from 2m away of proposed abutment locations for TMB.



**2.0 Stage 2** - Redirect traffic to the TMB (one- way traffic);  
 Excavate rockfill embankment beneath the TMB and the  
 other side of embankment towards slope 1.5H:1V



**3.0 Stage 3** - Traffic on the TMB (one- way traffic);  
 Construct proposed Culvert.  
 Backfill reverse sequences.

**SECTION 2-2**

**4.0 Stage 4** - Switch traffic to the reconstructed side (WBL)- one- way traffic;  
 Remove the TMB;  
 Backfill under the bridge location.



## **Appendix I – Non-Standard Special Provisions (NSSPs)**

## **NSSP FOR OBSTRUCTIONS**

### **Scope of Work**

The Contractor should be aware that the existing rockfill embankment consist of crushed gravel which will be encountered during the installation of shoring elements and during excavation of the embankment fill. Therefore, appropriate equipment and procedures will be required to penetrate through crushed gravel that may encountered during installation of shoring and excavation.

The Contractor also should be aware a presence of buried creosote wood which should be encountered during the excavation at the outlet side.

### **Basis of Payment**

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

## **NSSP FOR REMOVAL OF PROTECTION SYSTEM**

### **Scope of Work**

If protection systems are specified for removal or the Contractor elects to remove, the method and sequence of removal; should be such that there will be no damage to the new work, existing work, and facility being protected.

If protection systems are left in place, the top should be removed to at least 1.2 m below the finished grade or ground level or at least 0.6 m below the streambed.

All disturbance areas have to be restored to an equivalent or better condition than existing prior to the commencement of construction.

### **Basis of Payment**

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