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Foundation Investigation  
and Design Report

**Agreement No. 5016-E-0016**

**GWP 411-00-00**

**GEOCRES No. 410-32**

**Culvert Replacement, Stn. 14+457  
Highway 129, Reaney Township,  
District of Sudbury**

**Prepared For:**

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Northeast Region**

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# The Ministry of Transportation

## Foundation Investigation and Design Report Assignment No. 5016-E-0016 GWP 411-00-00 GEOCRES No. 41O-32

### Project Name:

Culvert Replacement, Stn. 14+457  
Highway 129, Reaney Township, District of Sudbury

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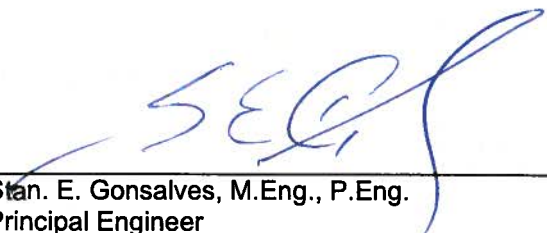
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# 1 Foundation Investigation Report

## 1.1 Introduction

This Foundation Investigation Report (FIR) presents the results of a geotechnical investigation completed by **exp** Services Inc. (**exp**) for the replacement of a centreline culvert located on Highway 129 at Station 14+457, within Reaney Township, District of Sudbury, Ministry of Transportation (MTO) Northeastern Region. This work was undertaken under Agreement No. 5016-E-0016, GWP 411-00-00. The terms of reference (TOR) were presented in the MTO Request for Quotation Document dated August 22, 2016.

The purpose of the investigation is to evaluate the subsurface conditions along the proposed culvert replacement alignment in order to provide geotechnical information necessary for the design of the culvert replacement. The site specific geotechnical investigation consisted of borings, soil sampling, borehole logging, and field and laboratory testing.

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

## 1.2 Site Description and Geological Setting

### 1.2.1 Site Description

The centreline culvert replacement site is located on Highway 129 at Station 14+457 within Reaney Township. The site is located approximately 30.5 km south of the South Junction of Highway 101. The location of the culvert and a cross section of the existing culvert alignment are shown on Dwg. No. 1 in Appendix A.

The existing culvert consists of a non-structural, corrugated steel pipe (CSP), approximately 900 mm in diameter and 32.14 m long. At this site, Highway 129 is an asphalt paved, two lane, north/south roadway having approximately 1.0 m wide granular shoulders and cable guide rails on both sides of the roadway. The highway embankment at the investigated location is approximately 5.5 m high on both sides of the roadway, having side slopes of approximately 2.2H:1V on the west embankment and 1.9H:1V on the east embankment, from the top to toe of the embankment. Photographs of the site and existing culvert are included in Appendix B.

The general site conditions were assessed on November 16, 2016. The existing waterway flows from the west to the east through the existing culvert. Immediately adjacent to the waterway on both sides of the roadway embankment, the terrain generally consists of marshy, low lying vegetation and grasses, surrounded by a thick forest consisting of both deciduous and coniferous trees. At the time of the assessment, the water levels appeared low, with standing water on the inlet side.

The side slopes of the highway embankment are covered with grass and light vegetation, with trees and larger vegetation generally located towards the embankment toes. Guardrails and signs at the top of the embankment and trees near the embankment toe all appeared to be standing vertically, suggesting there is not likely any stability issues with the current embankment. Bedrock outcrops were not observed at the site. The surface of Highway 129 near the culvert location was in poor to fair shape, with moderate wheel track rutting and moderate transverse, longitudinal, and map cracking. Immediately above the culvert, moderate to severe transverse cracking has occurred across the full width of the roadway.



## 1.2.2 Geological Setting

In accordance with Ontario Geological Survey Northern Ontario Engineering Geology Terrain Study 80, the dominant landform at the culvert site is ground moraine consisting mainly of till. Local relief is generally low ( $< 15$  m) and the terrain is generally undulating to rolling. Overall drainage is good (dry).

Ministry of Northern Development and Mines (MNDM) Map 2543, Bedrock Geology of Ontario East-Central Sheet indicates the bedrock at the culvert location consists of tonalite to granodiorite, foliated to gneissic, with minor supracrustal inclusions.

## 1.3 Investigation Procedures

### 1.3.1 Site Investigation and Field Testing

The field investigation was performed on December 6, 2016 and January 28 and 29, 2017. The field program consisted of the advancement of three (3) sampled boreholes (BH-1 to BH-3). The boreholes were located along the existing culvert alignment to provide subsurface information for the design of the proposed new culvert. Borehole BH-1 was located within the travelled northbound lane, as close as possible to the crest of the eastern embankment. Boreholes BH-2 and BH-3 were advanced at accessible locations near the outlet and inlet, respectively, of the culvert. The borehole locations are shown on Dwg. No. 1 in Appendix A.

Borehole BH-1 was advanced using a truck mounted CME-55 drill rig equipped with hollow stem augers, NW casing, and standard soil sampling equipment. Due to access restrictions, Boreholes BH-2 and BH-3 were advanced with portable tripod mounted equipment with a cathead and Hilti D200 drill. The drilling equipment was operated by a specialist drilling contractor, Landcore Drilling. Each borehole was advanced to refusal at depths ranging from 5.2 to 9.5 m below existing grades. As refusal depths were encountered greater than 2.0 m below the culvert invert, coring was not required in accordance with the TOR.

The borehole locations (referenced to MTM NAD83 coordinate system, Zone 13) and their ground surface elevations were surveyed by **exp** personnel following drilling using hand-held GPS equipment. The geodetic borehole and water elevations were surveyed using a Temporary Benchmark (TBM) established on the roadway centreline at Stn. 14+450. The TBM was assigned an elevation of 462.4 m based on a survey of the site provided to **exp** by the MTO. The borehole and TBM locations are shown on Dwg. No. 1 in Appendix A.

Soil samples were obtained using a 51 mm outside diameter 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 in Appendix C. The original field (uncorrected) SPT "N" values were recorded on the borehole logs and used to provide an assessment of the in-situ compactness condition of encountered cohesionless soils.

Upon completion of the boreholes, groundwater measurements were carried out within the boreholes in accordance with MTO guidelines. The measured groundwater levels after completion were recorded on the borehole logs as shown in Appendix C. The boreholes were decommissioned using bentonite in accordance with the Ministry of the Environment Regulation 903, as amended by Regulation 128/03 (the well regulation under the Ontario Water Resources Act).

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

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



### 1.3.2 Laboratory Testing

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

The laboratory test results are summarized on the attached borehole logs in Appendix C. The results of the particle size analyses are presented graphically in Appendix D.

## 1.4 Subsurface Conditions

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

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

In general, the subsurface conditions encountered within the embankment (BH-1) consist of asphalt overlying fill materials, peat, and native silt/sand, and assumed till. At the toes of the embankment slopes (BH-2 and BH-3), the subsurface conditions encountered consist of silt and sand fill, peat, native silt/sand, and till materials. A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

### 1.4.1 Asphalt

Asphalt was encountered at the surface of Borehole BH-1 and was approximately 76 mm thick. Asphalt thicknesses may further vary beyond the borehole location.

### 1.4.2 Fill Materials

Fill materials were encountered below the asphalt at Borehole BH-1 and at the surface of BH-3. The fill materials extended to approximately 6.3 m depth at BH-1 and 2.3 m depth at BH-3.

At BH-1, the fill material layers ranged in composition, including:

- gravelly sand fill;
- sandy silt fill;
- sand fill; and,
- silt fill.

At BH-3, the fill materials consisted of silt and sand fill, with some organics, changing to organic silt and sand fill with peat.

Further details on the fill layers are outlined in the following sections.





#### 1.4.2.1 Gravelly Sand Fill

An approximately 3.0 m thick layer of gravelly sand fill was encountered below the asphalt at Borehole BH-1. The fill material was brown in colour, moist, and contained trace silt.

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

Moisture Content:

- 3 to 7 %

Grain Size Distribution:

- 28 % gravel
- 64 % sand
- 7 % fines

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

#### 1.4.2.2 Sandy Silt Fill

Underlying the gravelly sand fill at 3.1 m depth at Borehole BH-1 was an approximately 0.7 m thick layer of sandy silt fill. The fill material was brown in colour, moist, and contained trace organics, trace gravel, trace clay, and some cobbles and boulders. One SPT was performed within the sandy silt fill, with a resulting uncorrected "N" value of 45 blows per 300 mm, classifying the soil as dense in compactness condition.

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

Moisture Content:

- 16 %

Grain Size Distribution:

- 5 % gravel
- 30 % sand
- 63 % silt
- 3 % clay

The results of the moisture content and grain size distribution tests are provided on the borehole log for BH-1 in Appendix C. The result of the grain size distribution test is also provided on Figure 2 in Appendix D.

#### 1.4.2.3 Sand Fill

Underlying the sandy silt fill at 3.8 m depth at Borehole BH-1 was an approximately 1.5 m thick layer of sand fill. The fill material was brown in colour, moist, and contained trace to some silt, trace roots, trace to some gravel, and some cobbles and boulders. Uncorrected SPT "N" values within the fill ranged from 28 to 53 blows per 300 mm, classifying the fill as compact to very dense in compactness condition.

Laboratory testing performed on selected samples consisted of two (2) moisture content tests. The test results are as follows:





Moisture Content:

- 9 and 17 %

The results of the moisture content tests are provided on the borehole log for BH-1 in Appendix C.

#### 1.4.2.4 Silt Fill

Underlying the sand fill at 5.3 m depth at Borehole BH-1 was an approximately 1.0 m thick layer of silt fill. The fill material was brown to dark brown in colour, wet, and contained some organics, some sand, and trace clay. One SPT was performed within the silt fill, with a resulting uncorrected "N" value of 9 blows per 300 mm, classifying the soil as loose in compactness condition.

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

Moisture Content:

- 23 %

Grain Size Distribution:

- 7 % gravel
- 16 % sand
- 74 % silt
- 3 % clay

The results of the moisture content and grain size distribution tests are provided on the borehole log for BH-1 in Appendix C. The result of the grain size distribution test is also provided on Figure 3 in Appendix D.

#### 1.4.2.5 Silt and Sand Fill to Organic Silt and Sand Fill

Extending from the surface of BH-3 to 2.3 m depth was silt and sand fill changing to organic silt and sand fill. The fill materials were grey to black in colour, wet, and contained some organics in the upper 0.8 m, with an increasing proportion of organics with depth. Uncorrected SPT "N" values within the fill ranged from 3 to 9 blows per 300 mm, classifying the fill as very loose to loose in compactness condition.

Laboratory testing performed on selected samples consisted of three (3) moisture content tests. The test results are as follows:

Moisture Content:

- 27 to 128 %

The results of the moisture content tests are provided on the borehole log for BH-3 in Appendix C.

#### 1.4.3 Peat

Peat was encountered at the surface of Borehole BH-2 and at 6.3 m depth at BH-1, and 2.3 m depth at BH-3. The peat layer ranged in thickness from 1.6 to 2.6 m. The peat was grey to black in colour, wet, and contained some to with silty sand, some gravel, and wood. Uncorrected SPT "N" values within the peat ranged from 1 to 6 blows per 300 mm, classifying the peat as very loose to loose in compactness condition. The lower blows counts were found in the surficial peat at BH-2.

Laboratory testing performed on selected samples consisted of nine (9) moisture content tests. The test results are as follows:



Moisture Content:

- 27 to 282 %

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

#### 1.4.4 Silt and Sand

Underlying the peat at 7.9 m depth at Borehole BH-1 and 2.6 m depth at BH-2 was an approximately 1.2 to 3.5 m thick layer of native silt and sand. The silt and sand was grey in colour, wet, and contained some organics, trace clay, and trace gravel. Uncorrected SPT "N" values within the soil ranged from 0 to 13 blows per 300 mm, classifying the soil as very loose to compact in compactness condition.

Laboratory testing performed on selected samples consisted of three (3) moisture content tests and two (2) grain size analyses. The test results are as follows:

Moisture Content:

- 15 to 45 %

Grain Size Distribution:

- 1 to 7 % gravel
- 40 to 41 % sand
- 45 to 56 % silt
- 3 to 6 % clay

The results of the moisture content and grain size distribution tests are provided on the borehole log for BH-1 and BH-2 in Appendix C. The results of the grain size distribution tests are also provided on Figure 4 in Appendix D.

#### 1.4.5 Silty Gravelly Sand Till

Underlying the peat at 4.6 m depth at Borehole BH-3 and extending to the refusal depth of 5.2 m, was native silty gravelly sand till. The till was grey in colour, wet, and contained trace clay. One SPT performed within the till resulted in an uncorrected "N" value of 110 blow per 300 mm, classifying the till as very dense in compactness condition.

At BH-1 and BH-2, very little soil material was recovered from the sampler at the refusal depths of 9.5 m and 6.2 m, respectively. The soil that was recovered from the split spoon generally consisted of gravel. It is suspected that refusal in these boreholes was within the same till layer as encountered at BH-3.

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

Moisture Content:

- 12 %

Grain Size Distribution:

- 32 % gravel
- 35 % sand
- 32 % silt
- 1 % clay



The results of the moisture content and grain size distribution test are provided on the borehole log for BH-3 in Appendix C. The result of the grain size distribution test is also provided on Figure 5 in Appendix D.

#### 1.4.6 Refusal Depths

Refusal was encountered in each borehole between approximately Elev. 450.0 m to 453.0 m. At Borehole BH-1, observations made by the field technician during drilling suggest the encountered refusal was likely on bedrock (though not confirmed). At Borehole BH-2, there were no indications during drilling that could help distinguish whether refusal was on bedrock or on very dense till. At Borehole BH-3, the split spoon sampler advanced at the refusal depth was bent when extracted from the borehole, suggesting refusal was likely on bedrock (though not confirmed).

As coring was not utilized beyond the refusal depths noted, the presence of bedrock could not be confirmed.

### 1.5 Groundwater and Surface Water Conditions

Groundwater was observed in Borehole BH-1 upon completion at approximately 2.9 m depth, Elev. 459.5 m. Note, however, that this water elevation is not likely accurate as water was pumped into the borehole for the washboring techniques utilized. Washboring techniques were also used at BH-2 and BH-3 with the portable equipment utilized, and thus, no groundwater measurements were made in these boreholes. As such, accurate groundwater measurements could not be obtained in the boreholes upon completion.

Note, however, that samples within BH-1 were generally wet below 5.3 m depth, Elev. 457.1 m. In addition, samples at BH-2 and BH-3 were wet from surface, Elev. 456.2 to 457.8 m. This could infer a groundwater level located between Elev. 456.2 and 457.8 m.

The water level within the adjacent open water was measured at the time of the investigation (January 2017) and it was at approximately Elev. 457.0 (top of ice) at the culvert inlet and at Elev. 455.8 m at the culvert outlet. This is generally at the same level as the wet samples encountered within the boreholes, which also further supports the inference above regarding the groundwater level.

Groundwater would be expected to reflect levels in the adjacent open water and to fluctuate seasonally and with weather conditions. 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.



## 2 Engineering Discussion and Recommendations

### 2.1 General

This section of the report provides geotechnical design recommendations for replacement of a non-structural centreline culvert located on Highway 129 at Station 14+457, within Reaney Township, District of Sudbury, Ministry of Transportation (MTO) Northeastern Region. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the current investigation at the site and presented in Part 1 - Foundation Investigation Report. The interpretations and recommendations provided are intended solely to permit designers to assess foundation alternatives and design the new culvert 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, etc.

Based on the TOR provided by the MTO, the existing culvert is an approximately 900 mm diameter CSP, which is approximately 32.14 m long. It is understood that the existing culvert would be replaced with a new culvert along the same alignment with minimum to no grade change anticipated at the culvert location. The size and type of the new culvert is not firmly defined at the time of writing this report. However, for preliminary design purposes, the non-structural culvert type options, such as flexible pipe, rigid pipe and concrete box less than 3 m span, are recommended to be considered in this report.

This part of the report addresses the geotechnical design of the foundation for the new culvert by providing geotechnical design parameters at the Ultimate Limit State (ULS) and Serviceability Limit States (SLS), as well as other geotechnical parameters that may be required in accordance with the latest edition of the Canadian Highway Bridge Design Code (CHBDC) (CAN/CSA-S6-14), the Canadian Foundation Engineering Manual (CFEM) (2006), MTO Gravity Pipe Design Guidelines (May 2007), and generally accepted good practice. Pertinent construction issues from a geotechnical standpoint are examined in general accordance with the terms of reference (TOR) as presented in the MTO Request for Quotation Document dated August 22, 2016. The assessment involved review of options for replacement of the existing culvert along the current alignment.

### 2.2 Expected Ground Conditions

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

- Highway 129 is an asphalt paved, two lane, north/south roadway having approximately 1.0 m wide granular shoulders and cable guide rails on both sides of the roadway at the existing culvert location. The highway embankment at the investigated location is approximately 5.5 m high on both sides of the roadway, having side slopes of approximately 2.2H:1V on the west embankment and 1.9H:1V on the east embankment, from the top to toe of the embankment. The current elevation of the crest of the roadway is approximately 462.4 m.
- The highway embankment consists of approximately 5.3 m of compact to very dense to dense granular fill with some cobbles/boulders below 3.5 m depth, underlain by approximately 1.0 m of wet, loose silt fill mixed with organics. The embankment fill is underlain by approximately 1.6 m of very loose to loose peat, followed by 1.2 m of loose to compact native silt and sand. Very dense assumed till was encountered below the silt and sand at 9.1 m depth, which extended to borehole refusal on suspected very dense till or bedrock at 9.5 m depth.



- At the existing culvert inlet on the west side of the embankment, approximately 2.3 m of very loose to loose organic mixed fill was encountered overlying approximately 2.3 m of loose peat. Underlying the peat at 4.6 m depth was a very dense silty gravelly sand till that extended for 0.6 m depth before borehole refusal on suspected very dense till or bedrock was encountered. At the outlet on the east side of the embankment, approximately 2.6 m of very loose peat was encountered overlying approximately 3.5 m of very loose native silt and sand. Borehole refusal on suspected very dense till or bedrock was encountered just below the silt and sand layer at 6.2 m depth.
- The water level within the Creek measured at the time of the investigation (January 2017) was at approximately Elev. 455.8 m at the culvert outlet. Wet samples within the boreholes were found below Elev. 456.2 to 457.8 m. As such, an inferred groundwater elevation near 456.5 m is anticipated.

## 2.3 Structure Foundations

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

- Rigid frame concrete box culvert less than 3 m span (precast or cast-in-place);
- Rigid concrete pipe culvert;
- Corrugated steel pipe (CSP) culvert; or,
- Cast-in-place, rigid frame, open footing concrete culvert supported on shallow foundations.

The choice of culvert type will depend on parameters such as the initial cost, maintenance costs, expected service life, hydraulic performance, ease of construction, and local availability of materials and equipment.

It is noted that regardless of the option selected, the existing 900 mm × 32.14 m CSP culvert is to be removed or decommissioned. In addition, the expected creek and groundwater levels are higher than the current culvert invert. This suggests the need for surface/groundwater control and a cofferdam as discussed in Section 2.9 below.

The new culvert founding level is expected to be similar to the current level (approx. Elev. 456.6 m). Below this level, very loose to loose fill materials, very loose native soils, and peat/wood were generally encountered. These materials extend as deep as Elev. 450.1 m before generally favourable very dense soils/tills are encountered. Removing these unfavourable materials from below the proposed culvert will likely result in very deep excavations (i.e. greater than 8 m below top of roadway). This would likely be considered excessive or uneconomical, and as such, these materials are anticipated to be left in place. If these materials are left in place, an engineered fill pad should be constructed below the proposed culvert. The engineered fill pad should be a minimum of 500 mm thick, and consist of clear stone gravel, Granular "A" or Granular "B" Type II. A bi-axial geogrid should be placed between the engineered fill pad and underlying in-situ soils/peat. A non-woven geotextile fabric should surround the entire fill pad to mitigate the migration of fines into the engineered fill. The new culvert must also be designed such that there is no grade raise at the culvert location and no net increase in bearing pressure on the foundation soils beyond the existing conditions. This will mitigate any significant settlement of the underlying very loose soils and peat/wood.

Based on the subsoil conditions, Table 2-1 below compares the possible structure options from a foundations design and constructability perspective with their advantages and disadvantages. Although the foundation soils can provide adequate support for all options listed in the table, the use of a precast rigid frame box culvert is anticipated by the MTO to be utilized, as indicated in the Start-Up Meeting minutes for this project.

**Table 2-1: Evaluation of Foundation Alternatives**

Options	Rank	Advantages	Disadvantages	Relative Costs	Risk/Consequences
Precast Rigid Frame Concrete Box/Pipe Culvert	1	<ul style="list-style-type: none"> <li>• Straightforward construction</li> <li>• Reduced construction period, consequently traffic management and water control period reduced</li> <li>• Can be more readily installed during cold weather conditions</li> <li>• Longer service life than steel</li> </ul>	<ul style="list-style-type: none"> <li>• If floor is thin or 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> </ul>	<ul style="list-style-type: none"> <li>• Low</li> </ul>	<ul style="list-style-type: none"> <li>• Risk of unacceptable differential settlements if the entire foundation is not supported on competent soil</li> <li>• Risk of leaking from joints if not properly installed</li> </ul>
Cast-in-Place Rigid Frame Concrete Box Culvert	3	<ul style="list-style-type: none"> <li>• Suitable if site is not conducive to heavy equipment for installation of precast sections</li> <li>• Culvert design can be customized in the field for high stress or load conditions or other site specific requirements</li> <li>• Longer service life than steel</li> </ul>	<ul style="list-style-type: none"> <li>• Slower construction process</li> <li>• If floor is thin or poorly reinforced, it may heave and crack</li> <li>• During high flows, the concrete floor can be undermined</li> <li>• Requires concrete curing</li> </ul>	<ul style="list-style-type: none"> <li>• Low to medium</li> </ul>	<ul style="list-style-type: none"> <li>• Risk of unacceptable settlements if the entire foundation is not supported on competent soil</li> <li>• Risk of disturbance of base during construction</li> </ul>
Corrugated Steel Pipe (CSP) Culvert	2	<ul style="list-style-type: none"> <li>• Straightforward construction</li> <li>• Reduced construction period, consequently traffic management and water control period reduced</li> </ul>	<ul style="list-style-type: none"> <li>• Limited service life</li> <li>• Potential for corrosion</li> </ul>	<ul style="list-style-type: none"> <li>• Low to medium</li> </ul>	<ul style="list-style-type: none"> <li>• Risk of unacceptable settlements if the entire foundation is not supported on competent soil</li> <li>• Risk of structure segment loss due to corrosion</li> </ul>



Options	Rank	Advantages	Disadvantages	Relative Costs	Risk/Consequences
Cast-in-Place Rigid Frame Open Footing Concrete Culvert Supported on Shallow Foundations	4	<ul style="list-style-type: none"> <li>• Wider span may be used to maintain existing channel and allows for natural streambed to remain intact</li> <li>• Less accumulation of sediments upstream of the culvert</li> <li>• Longer service life than steel</li> </ul>	<ul style="list-style-type: none"> <li>• Slower construction process</li> <li>• Deeper excavation likely required as footings need to be below frost line</li> <li>• Requires concrete curing</li> </ul>	<ul style="list-style-type: none"> <li>• Medium</li> </ul>	<ul style="list-style-type: none"> <li>• Risk of unacceptable settlements if the entire foundation is not supported on competent soil</li> <li>• High Scour Risk</li> </ul>

### 2.3.1 Shallow Foundations

#### 2.3.1.1 Geotechnical Resistance

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

**Table 2-2:** Recommended Design Parameters

Culvert Type	Founding Elevation (m)	Assumed Footing Size (m)	Founding Soil Type	Factored Geotechnical Resistance at ULS (kPa)	Geotechnical Reaction at SLS* (kPa)
Rigid frame box culvert  Concrete Pipe Culvert  Or  CSP Pipe Culvert	~ 456.6 m or below	0.9 m	~ 0.5 m compacted engineered fill pad overlying in-situ silt and sand or peat	225	150
Cast-in-Place Open Footing Concrete Culvert	~ 453.8 m (below frost line)	1.0 m	~ 0.5 m compacted engineered fill pad overlying in-situ silt and sand or peat	250	160

\*- For Maximum Settlement of 25 mm



As it is not anticipated that the very loose soils and peat/wood will be removed from below the proposed culvert, the new culvert must be designed such that there is no grade raise at the culvert location and no net increase in bearing pressure beyond the existing conditions. This will mitigate any significant settlement of the underlying very loose soils and peat/wood. It is likely that a larger diameter culvert than existing would be required to achieve this, as it would unload the existing soils due to the larger void. Unit weights for the in-situ materials are outlined in Section 2.4. These values in addition to the weight parameters for the existing culvert can be utilized to determine the existing loading conditions.

Given that no (or minimal) grade raise and no net increase in bearing pressure is planned, the anticipated maximum total settlements for the new culvert are not expected to exceed 25 mm for construction done in accordance with these design parameters and assuming good construction practice including sound base preparation.

### 2.3.1.2 Resistance to Lateral Loads

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

**Table 2-3:** Recommended Parameters for Calculation of Unfactored Horizontal Resistance

Interface and Loading Conditions	Parameters
Between Granular "A" and pre-cast concrete	Coefficient of Friction ( $\tan \delta$ ) = 0.5
Between Granular "A" and cast-in-place concrete	Coefficient of Friction ( $\tan \delta$ ) = 0.58

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

## 2.4 Lateral Earth Pressure

Culvert walls and temporary shoring should be designed to resist lateral earth pressure. The expression for calculating lateral earth pressure "p" at any depth "h" is given by the following:

$$p = K(\gamma h + q) + \gamma_w h_w$$

where

p = Lateral earth pressure (kPa)

K = Coefficient of earth pressure

$\gamma$  = Unit weight of backfill (kN/m<sup>3</sup>)

$\gamma_w$  = Unit weight of water (kN/m<sup>3</sup>)

h = Depth to point of interest (m)

$h_w$  = Depth of water above point of interest (m)

q = Surcharge load acting adjacent to the wall at the ground surface (kPa)

Table 2.4 lists earth pressure parameters for given materials. These recommendations assume level backfill and ground surface behind the walls.

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

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

For multiple support systems refer to Canadian Foundation Engineering Manual (CFEM) for apparent earth pressure distributions (CFEM, Section 26.10.3, Figure 26.8).

**Table 2-4:** Material Types and Earth Pressure Parameters

Material	Friction Angle $\phi'$ (unfactored)	Coefficient of Active Earth Pressure ( $k_a$ )	Coefficient of Passive Earth Pressure ( $k_p$ )	Coefficient of Earth Pressure at Rest ( $k_o$ )	Unit Weight $\gamma$ (kN/m <sup>3</sup> )
Granular "A" (compact)	35°	0.27	3.7	0.43	22.8
Granular "B" Type I (compact)	32°	0.31	3.3	0.47	21.2
Granular "B" Type II (compact)	35°	0.27	3.7	0.43	22
Gravelly Sand Fill (compact)	32°	0.31	3.3	0.47	20
Sandy Silt Fill (dense)	30°	0.33	3.0	0.50	21
Sand Fill (dense to very dense)	32°	0.31	3.3	0.47	21
Silt Fill (very loose to loose)	28°	0.36	2.8	0.53	18
Silt and Sand Fill/Organic Silt and Sand Fill (very loose to loose)	28°	0.36	2.8	0.53	18
Silt and Sand (very loose to compact)	28°	0.36	2.8	0.53	19
Silty Gravelly Sand Till (very dense)	35°	0.27	3.7	0.43	21
Peat	18°	0.53	1.9	0.69	15

## 2.5 Seismic and Liquefaction Potential Consideration

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

From the Natural Resources Canada website, 2015 NBCC seismic hazard values are obtained using the site location coordinates (47.536°N, 83.211°W) and the damped reference spectral accelerations for the project site are  $S_a(0.2)=0.029g$ ,  $S_a(0.5)=0.023g$ ,  $S_a(1.0)=0.013g$ ,  $S_a(2.0)=0.0058g$  and the reference peak ground acceleration (PGA) is  $0.015g$  ( $g$ =acceleration due to gravity -  $9.81 \text{ m/s}^2$ ). These values are associated with an earthquake having 10 percent probability of exceedance in a 50-year period.

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

## 2.6 Construction Alternatives

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

1. Open cut/unsupported excavations to remove and replace culvert. The following two options of open cut/unsupported excavations were considered:
  - a. Full road closure followed by open cut/unsupported excavation
  - b. Construct temporary detour embankments at the site followed by open cut/unsupported excavation
2. Half-and-half construction using roadway protection to allow excavation and 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

Both methods 1 and 2 utilize a cut and cover approach for culvert replacement, which allows complete removal of the existing culvert. These two methods will also require disruption of traffic. For all approaches, provisions must be made to maintain surface water flow to the outlet.

Table 2-5 below summarizes the advantages and disadvantages of each considered construction method alternative. The table also shows assessed risk/consequences and relative costs of the considered methods.



**Table 2-5: Construction Alternatives for Culvert Replacement**

Installation Method	Advantages	Disadvantages	Relative Cost	Ranking
1.a. Full road closure and open cut/unsupported excavation	<ul style="list-style-type: none"> <li>Existing culvert will be completely removed and replaced with new culvert</li> <li>No detour road construction or roadway protection required</li> <li>No excavation support required</li> <li>Install entire new culvert at once</li> <li>Straightforward construction</li> <li>Short construction period</li> <li>Low capital investment; cost savings in time and materials required for construction</li> </ul>	<ul style="list-style-type: none"> <li>Traffic interruption</li> <li>No local detour available, only long distance detours available, as such cannot likely be closed</li> <li>Large amount of soil to be excavated</li> <li>Excavations will be large with likely 1H:1V sideslopes</li> <li>Need to temporarily control existing creek water and groundwater</li> <li>Potential claims to compensate vehicle occupants and local businesses for delays or time lost due to long detours</li> </ul>	<ul style="list-style-type: none"> <li>Relatively less expensive than other methods due to cost savings in time and materials required for construction</li> <li>Potential costs associated with claims to compensate vehicle occupants and local businesses for delays or time lost due to long detours</li> <li>Low risk of cost overruns</li> </ul>	4.

Installation Method	Advantages	Disadvantages	Relative Cost	Ranking
1.b. Temporary detour embankments and open cut/unsupported excavation	<ul style="list-style-type: none"> <li>• One to two lanes of traffic flow maintained at site during construction.</li> <li>• Existing culvert will be completely removed and replaced with new culvert</li> <li>• No excavation support required</li> <li>• Install entire new culvert at once</li> </ul>	<ul style="list-style-type: none"> <li>• Traffic interruption</li> <li>• Construction of detour embankments required on one side of highway</li> <li>• Difficulties to construct detours due to accessibility of surrounding terrain.</li> <li>• Increased time of construction due to detour</li> <li>• Large amount of soil to be excavated</li> <li>• Excavations will be large with likely 1H:1V sideslopes</li> <li>• Need to temporarily control existing creek water and groundwater</li> <li>• Possible settlement due to new earth fill embankment</li> <li>• Temporary detour will need to be decommissioned</li> </ul>	<ul style="list-style-type: none"> <li>• Higher cost than full road closure due to high costs associated with temporary detour embankment construction</li> <li>• Possible costs associated with purchasing private property if detour extends beyond current ROW</li> <li>• Moderate risk of cost overrun due to complexity of constructing detour embankment</li> </ul>	3.

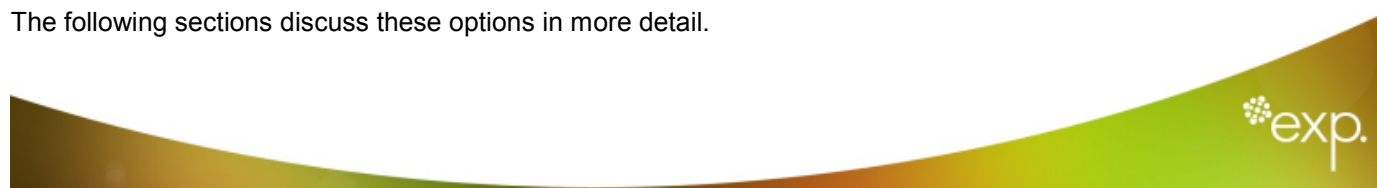
Installation Method	Advantages	Disadvantages	Relative Cost	Ranking
2.a. Half-and-half construction using roadway protection with unsupported cut sides	<ul style="list-style-type: none"> <li>• One lane of traffic flow maintained during construction</li> <li>• Straightforward construction</li> </ul>	<ul style="list-style-type: none"> <li>• Traffic interruption</li> <li>• Roadway protection required to maintain one lane of traffic</li> <li>• High cost of roadway protection system</li> <li>• Large amount of soil to be excavated</li> <li>• Culvert excavations will be large with likely 1H:1V sideslopes</li> <li>• Need to temporarily control existing creek water and groundwater</li> <li>• Narrow highway; may require temporary widening for open traffic lane</li> <li>• Boulders/cobbles within in-situ fill materials likely limits the type of roadway protection that can be used (i.e. sheet piles not likely possible)</li> </ul>	<ul style="list-style-type: none"> <li>• More expensive than road closure due to high costs of roadway protection system</li> <li>• Moderate risk of cost overrun due to complexity of roadway protection system</li> </ul>	1.

Installation Method	Advantages	Disadvantages	Relative Cost	Ranking
2.b. Half-and-half construction using roadway protection with braced cut sides	<ul style="list-style-type: none"> <li>• One lane of traffic flow maintained during construction</li> <li>• Narrow excavation for culvert</li> <li>• Less soil excavation and fill placement</li> <li>• Less roadway protection required due to small culvert excavation</li> </ul>	<ul style="list-style-type: none"> <li>• Traffic interruption</li> <li>• Roadway protection required to maintain one lane of traffic</li> <li>• Requires side shoring/bracing for culvert excavation</li> <li>• High cost of roadway protection system and side shoring</li> <li>• Bracing may interfere with culvert removal/placement</li> <li>• Shoring system will need to be decommissioned</li> <li>• Need to temporarily control existing creek water and groundwater</li> <li>• Narrow highway; may require temporary widening for open traffic lane</li> <li>• Boulders/cobbles within in-situ fill materials likely limits the type of roadway protection/shoring that can be used (i.e. sheet piles not likely possible)</li> </ul>	<ul style="list-style-type: none"> <li>• More expensive than road closure due to high costs of roadway protection system</li> <li>• More expensive than Option 2.a. due to additional shoring for braced excavation for culvert</li> <li>• Moderate risk of cost overrun due to complexity of roadway protection system</li> </ul>	2.

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.a. – Half-and-Half Construction with Unsupported Cut Sides
2. Option 2.b. – Half-and-Half Construction with Braced or Anchored Cut Sides
3. Option 1.b. – Temporary Detour Construction Followed by Open Cut/Unsupported Excavation
4. Option 1.a. – Full Road Closure Followed by Open Cut/Unsupported Excavation

The following sections discuss these options in more detail.





## 2.6.1 Open Cut/Unsupported Excavations (Options 1.a and 1.b.)

Both detour options allow for open cut, unsupported excavations to facilitate the replacement of the existing culvert. The advantages are that neither excavation support, nor roadway protection, are required with these options. The major disadvantages of both options are traffic interruption, large amounts of excavated soils, and the need for temporary construction dewatering systems (i.e. cofferdams, and sumps and pumps, etc.) to prevent existing creek water and groundwater flow into the construction area. The dewatering system would be the responsibility of the contractor. For the open cut/unsupported excavations, two methods of culvert replacement were considered suitable for this site as follows:

- a. Construction with full road closure
- b. Construction with temporary detour embankment construction

### 2.6.1.1 Option 1.a. – Full Road Closure Followed by Open Cut/Unsupported Excavation

For Option 1.a., there are no local detours available. Traffic would likely have to detour a significant distance if the highway was closed for construction of the culvert. Potential detours would likely include Highway 17 to the west or Highway 144 to east. However, as Highway 129 is a generally low volume highway, consideration may be given to this option if construction can be completed in a short time frame. Significant notice to the public would be required if the highway is closed with no local detour. This option would however be the easiest, and likely cheapest, as construction of a detour embankment will not be required. The highway however, would not likely be able to be closed.

### 2.6.1.2 Option 1.b. – Temporary Detour Construction Followed by Open Cut/Unsupported Excavation

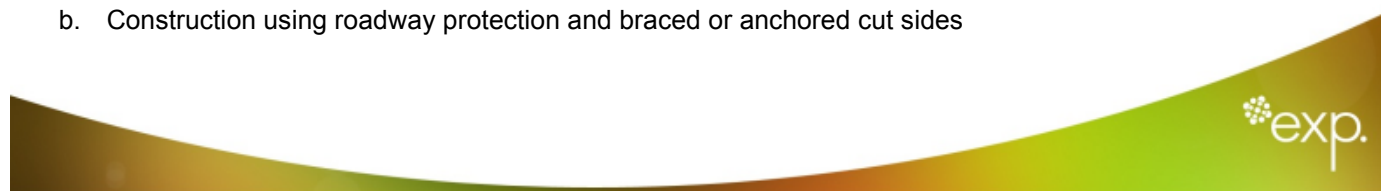
The local detour construction alternative, Option 1.b., would involve construction of a temporary on-site embankment on one side of the existing embankment depending on the available space and suitable terrain. As the creek is generally wider on the west side, it is likely that the east side is the preferred option for the detour. Compacted engineered fill for construction of the temporary detour road is recommended. Prior to construction of the temporary detour embankment, the site will need to be cleared and grubbed of any existing bushes and vegetation. All surficial topsoil, organics, and softened or loosened soil should be stripped from below the proposed temporary detour road embankment. All subgrade soils should be proofrolled prior to fill placement and embankment fill should be placed in accordance with OPSS. PROV 206 (dated November 2014).

## 2.6.2 Half-and Half Construction (Options 2.a. and 2.b.)

The half-and-half construction method could be utilized to maintain the flow of the traffic on Hwy 129. In this method, one lane of the existing highway will be used to maintain the local traffic while the other half of the existing highway will be excavated and the half of the existing culvert will be exposed. Then the excavated portion of the existing culvert will be removed and replaced with a new culvert, followed by rebuilding of that half of the embankment to grade. Upon completion of the new embankment, the traffic will be moved onto the new fill and the process will be repeated to complete the construction and culvert replacement.

The temporary excavation required to remove half of the existing embankment would be up to approximately 6.5 to 7.0 m deep. Therefore, temporary shoring, such as a soldier pile and lagging system, will be required as a roadway protection system to allow staging excavation/construction. It will be the Contractor's responsibility to design a suitable temporary support system for MTO review prior to installation. The Contractor is to follow OPSS 902, regarding excavations for structures, and OPSS.PROV 539, regarding temporary protection systems. Recommendations for a temporary roadway protection are given in Section 2.7. Using the half-and-half construction approach, two methods of culvert replacement were considered suitable for this site as follows:

- a. Construction using roadway protection and unsupported excavation of cut sides
- b. Construction using roadway protection and braced or anchored cut sides



Option 2.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 2.b will disrupt less of the embankment but would cost more, i.e. about 1.5 to 2 times that of Option 2.a. Excavation and backfilling operations will also be more challenging with Option 2.b. Both options will require temporary construction dewatering systems developed by the contractor (i.e. cofferdams, and sumps and pumps, etc.) to prevent existing creek water and groundwater flow from entering into the construction area. In addition, both options will require decommissioning of the shoring system upon completion of the work.

#### **2.6.2.1 Option 2.a. – Half-and-Half Construction with Unsupported Cut Sides**

This method provides roadway protection parallel to the highway between two lanes, and diverts traffic to the one side of the highway, while an open cut with sloping sides is performed on the opposite side of the highway. The roadway protection can take the form of reversible shoring such as a soldier pile and lagging with rakers or anchors for horizontal support. A sheet pile system is not likely possible due to the boulders/cobbles encountered within the in-situ fill materials. Where the cut extends below the prevailing groundwater, a suitable groundwater 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. Option 2.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 Option 2.b, since it requires excavation of a large amount of soil.

#### **2.6.2.2 Option 2.b. – Half-and-Half Construction with Braced or Anchored Cut Sides**

As with Option 2.a., this method provides roadway protection parallel to the highway between two lanes, and diverts traffic to the one side of the highway, while a braced or anchored shoring system running perpendicular to the highway is installed for face protection and to allow culvert construction to be performed on the opposite side. Excavation in this case would have to accommodate the necessary cross-bracing, such as struts. With this option, consideration would have to be given to how the new culvert sections will be installed given the relatively narrow work area and potential for obstructions from the lateral bracing using struts. Installation of tiebacks could be the solution. Temporary decking could possibly be used over the supported cut to allow for excavation of both halves prior to diverting the stream and backfilling. However, decking would be costly. Option 2.b. will disrupt less of the embankment than Option 2.a. but would cost more, i.e. about 1.5 to 2 times that of Option 2.a, due to the cost of the shoring system. Excavation and backfilling operations will also be more challenging with Option 2.b, again, due to the obstructions from the bracing. Both options require decommissioning of the shoring system upon completion of the work.

## **2.7 Temporary Roadway Protection**

Temporary roadway protection is anticipated to be a part of the half-and-half construction approach that will be required to maintain on-site traffic during the culvert construction. It is recommended that the roadway protection system be in accordance with OPSS.PROV 539. The lateral movement of the temporary shoring system should meet Performance Level 2, as specified in OPSS.PROV 539. The complete design, construction, monitoring and removal of the installed protection system should be a responsibility of the contractor. Due to the nature of this application, it is expected that much of the temporary shoring will be decommissioned in place, noting the high cost for removal. Decommissioning must be consistent with good practice to avoid interference with the highway system. The protection system should be designed to provide protection for excavations as required by the OHSA, at locations specified in the contract, and at any locations where the stability, safety or function of an existing structure and/or utility may be impaired by construction work.



At this site, a shoring system, such as soldier piles and timber lagging, may be considered for design. A sheet pile system is not likely possible due to the boulders/cobbles encountered within the in-situ fill materials. The contractor should be prepared to encounter boulders/cobbles regardless of the shoring system considered. The system should be designed based on the earth pressure 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, 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 extend fully beyond the active wedge acting on the wall. Pressure grouted soil anchors can be designed in a preliminary fashion in accordance with Section 26 of the CFEM (2006). Detailed design would be completed following the design of the wall and once 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 Performance Level 2 (for small, less important sections). The minimum requirements for monitoring should include the survey measurements of 6 m apart scaled targets attached to the shoring wall at the elevations specified. If movement approaches the allowable limit of 25 mm (Performance Level 2), suitable measures should be taken to ensure stability of the protection system and to ensure that the movement does not exceed the performance level specified.

## 2.8 Groundwater and Surface Water Control

Excavations are expected to extend below the observed groundwater level and the creek level measured during this investigation. To avoid disturbance of the founding subgrade and to allow for placement of fill in dry conditions, the groundwater must be lowered and controlled to a minimum of 0.5 m depth below the proposed excavation levels prior to excavation. The ingress of surface water must be controlled using a suitable system as well.

Diversion of the creek will be required during the culvert construction. Appropriate permitting and approvals must be in place for this work (i.e. MOECC, DFO, etc.) and work must be carried out in accordance with the approved schedules. In addition, to control water flow in the creek and for protection of the construction area, a cofferdam will likely be required for all replacement options. Dewatering requirements behind the cofferdam to keep the construction site dry will be impacted by water levels in the creek at the time of construction.

Dewatering requirements will be governed by the time of the year the construction is performed. Dewatering shall be carried out in accordance with OPSS 517 and OPSS 518. It is the responsibility of the Contractor to propose a suitable dewatering system based on the time of construction and creek/groundwater levels. The dewatering method is the responsibility of the Contractor and the Contractor should submit a proposal to the MTO for review and approval prior to construction. The method used should not undermine the existing road embankment or adjacent side slopes. The provision of toe protection at side slopes during drawdown may be required to minimize sloughing and undercutting during dewatering.

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



## 2.9 Culvert Bedding

As excavations are likely considered excessive or uneconomical to remove all organic materials and unfavourable soils from below the proposed culvert, these materials will likely be left in place. As such, an engineered fill pad should be constructed below the proposed bedding material. Prior to placing any fill material, the exposed native subgrade should be inspected in accordance with OPSS 902. The engineered fill pad should be a minimum of 500 mm thick, and consist of Granular "B" Type II (OPSS.PROV 1010) with the upper 75 mm consisting of Granular "A" for levelling purposes. A non-woven geotextile separator and bi-axial geogrid is to be placed between the approved subgrade and the engineered fill pad to assist in material placement and to maintain the integrity of the founding soil along the entire length of the culvert. The geotextile separator is to be a Class II non-woven material with an equivalent opening size of 75-150  $\mu\text{m}$ .

Bedding requirements for the various culvert materials are outlined on OPSD 802.010, 802.031, 802.032, and 803.010, which are included in Appendix F. The culvert bedding should consist of Granular "A" (OPSS.PROV. 1010) with a thickness of 300 mm beneath the culvert and extend a minimum of 500 mm horizontally on either side of the culvert edge.

The upfill and bedding material should be placed in lifts not exceeding 200 mm in thickness, loose measurement, and compacted to a minimum of 95% of the Standard Proctor Maximum Dry Density (SPMDD) in accordance with OPSS.PROV 501 before a subsequent layer is placed in accordance with OPSS.PROV 401. Particular care should be taken when compacting beneath pipe haunches. Bedding on each side of the culvert shall be completed simultaneously. At no time shall the levels on each side differ more than the 200 mm uncompacted layers.

## 2.10 Culvert Cover and Backfill

Culvert cover and backfill requirements for the various culvert materials are outlined on OPSD 802.010, 802.031, 802.032, 803.010, and 3101.150, which are included in Appendix F. Cover material should consist of Granular "A" (OPSS.PROV 1010) and shall be a minimum of 300 mm thick (compacted).

Immediately below the roadway, the backfill should consist of free-draining, non-frost susceptible granular materials, such as Granular "A" or Granular "B" Type I or II (OPSS.PROV 1010). Below the frost penetration depth of about 2.4 m from any finished road grade, approved compactable fill, such as select subgrade materials (SSM, OPSS.PROV 1010) can be used.

All granular backfill materials should be placed in lifts not exceeding 300 mm in thickness, loose measurement, and compacted to a minimum of 95% of the SPMDD in accordance with OPSS.PROV 501 before a subsequent layer is placed in accordance with OPSS.PROV 401. The final lift of embankment fill prior to placing pavement sub-base should be compacted to 100% of the SPMDD. The roadbed base and sub-base courses (for pavement) should be compacted to 100% of the material's SPMDD.

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



## 2.11 Frost Protection

The frost penetration depth in the Chapleau area is approximately 2.4 m in accordance with OPSD 3090.100 and the MTO Report titled “*Aspects of Prolonged Exposure of Pavements to Sub-Zero Temperatures*”, dated December 1981.

As the new culvert will likely be installed at a similar elevation as the existing, the frost penetration line within the embankment will be well above the top of the culvert. As such, the backfill and cover for these culverts should be as per OPSD 803.010.

At the culvert inlet and outlet, and beneath the proposed culvert, the native soils below the bedding and engineered fill pad will likely consist of silt and sand, and organics/peat. This material has a moderate to high frost susceptibility based upon the MTO Frost Classification guideline of percent particles between 5 to 75  $\mu\text{m}$ . Cold air blowing through the culvert during winter months can possibly freeze this material. Consideration may be given to utilizing insulation below the culvert bedding to prevent freezing of the underlying material. Installation details for insulation should be developed in consultation with the insulation manufacturer.

## 2.12 Embankment Design

### 2.12.1 Stability Analysis

A preliminary slope stability analysis was performed to assess the global stability of the existing embankment configuration and to check that a minimum Factor of Safety of 1.3 will be achieved for the temporary conditions for various construction configurations. The static slope stability analyses were performed using the Morgenstern-Price method developed on the basis of limit equilibrium. The SLOPE/W computer program developed by GeoSlope International was employed for modelling the embankment slopes and for computation.

Stability assessments of the existing slopes under static conditions were performed on a cross-section perpendicular to the highway at the culvert location. The cross-section of the existing embankment was established based on the topographic information provided by the MTO. The stratigraphy and groundwater conditions at the site were developed based on the results of the geotechnical investigation.

Based on the borehole information, the embankment fills and subsoils generally consist of cohesionless soil deposits. As such, an effective stress analysis for long term stability assessment was performed.

The various analyses performed are listed below. The SLOPE/W graphical printout for each analysis is shown on the noted figure in Appendix E.

- Figure E-1 – Existing Embankment Stability – Inlet Side
- Figure E-2 – Existing Embankment Stability – Outlet Side
- Figure E-3 – Proposed Embankment Stability – Inlet Side – 2H:1V Side Slopes
- Figure E-4 – Proposed Embankment Stability – Outlet Side – 2H:1V Side Slopes
- Figure E-5 – Proposed Embankment Stability – Inlet Side – 2.5H:1V Side Slopes
- Figure E-6 – Proposed Embankment Stability – Outlet Side – 2.5H:1V Side Slopes
- Figure E-7 – Temporary Detour Embankment Stability – Outlet Side, West Embankment Analysis

For the proposed final embankments, side slopes of both 2H:1V and 2.5H:1V were modelled. For the temporary detour embankment, a side slope of 2.5H:1V was modelled. In addition, it is assumed that the proposed embankments will be constructed with Granular “B” Type I material.



Tabulated below in Table 2-6 are the soil parameters used for the slope stability analyses. The soil parameters were generally estimated based on the results of the field and laboratory investigation and our past experience with similar soils.

**Table 2-6:** Soil Properties Used in Slope Stability Analysis

Soil Type	Long Term Conditions		
	$\phi'$	$c'$ (kPa)	$\gamma$ (kN/m <sup>3</sup> )
Granular "B" Type I	32°	0	21.2
Gravelly Sand Fill (compact)	32°	0	20
Sandy Silt Fill, some cobbles/boulders (dense)	32°	0	21
Sand Fill, some cobbles/boulders (dense to very dense)	32°	0	21
Silt Fill (very loose to loose)	28°	0	18
Silt and Sand Fill/Organic Silt and Sand Fill (very loose to loose)	28°	0	18
Silt and Sand (very loose to compact)	28°	0	19
Peat	18°	2	15

The results of the slope stability analyses performed are shown on Table 2-7 below. A minimum Factor of Safety (FS) of 1.3 is required to indicate that the embankment is stable. As shown on Table 2-7, the existing embankment is considered stable, however, the FS is below the minimum of 1.3 (Fig. E-1 and E-2). This is due to the relatively thick layer of very loose soils and peat/wood below the existing embankment. Reconstruction of the embankment with 2H:1V side slopes provides an improvement on the FS, however, still falls below 1.3 (Fig. E-3 and E-4). As such, in order to achieve a FS of 1.3 and ultimately be considered stable for long term conditions, the embankment side slopes need to be flattened to 2.5H:1V as indicated on Fig. E-5 to E-7. This will also result in a longer culvert than existing. For the temporary detour embankment analysis shown of Fig. E-11, the temporary detour embankment was modelled with 2.5H to 1V side slopes, resulting in a FS of 1.560.

**Table 2-7: Summary of Slope Stability Analysis Results**

Figure No.	Analysis	Factor of Safety
E-1	Existing Embankment – Inlet Side	1.211
E-2	Existing Embankment Stability – Outlet Side	1.122
E-3	Proposed Embankment Stability – Inlet Side – 2H:1V Side Slopes	1.238
E-4	Proposed Embankment Stability – Outlet Side – 2H:1V Side Slopes	1.226
E-5	Proposed Embankment Stability – Inlet Side – 2.5H:1V Side Slopes	1.336
E-6	Proposed Embankment Stability – Outlet Side – 2.5H:1V Side Slopes	1.349
E-7	Temporary Detour Embankment Stability – Outlet Side – West Embankment Analysis	1.560

## 2.12.2 Embankment Settlement and Construction

As the in-situ soils are generally cohesionless soils, a significant portion of settlement is expected to be immediate and complete by the end of construction. Post construction settlements below the existing embankment footprint are expected to be minimal (< 25 mm), provided the recommendations within this report are followed.

In order to achieve an FS of 1.3, embankment slopes of 2.5H:1V will be required. This will result in an approximately 1.5 to 2.0 m wider embankment beyond the toes of the existing embankment footprint. The new embankment will extend over the existing very loose peat and organic soils as encountered at the surface of Boreholes BH-2 and BH-3, which have not been previously compacted under the loading of the existing embankment. As such, the magnitude of settlement under these widened areas will likely be higher than the 20 to 25 mm anticipated below the existing embankment footprint.

In order to mitigate the settlement below the widened portion of the embankment, it is recommended to extend the proposed engineered fill pad outlined in Section 2.10 deeper by 1.0 m within the footprint of the widened areas. This will remove more of the very loose peat materials resulting in a 1.5 m thick engineered fill pad below the widened areas, which should help mitigate any significant settlement. Note that this deeper excavation in the widened sections may result in additional dewatering to maintain water levels 0.5 m below the excavation depth. All engineered fill pad, geogrid, and geotextile recommendations outlined in Section 2.10 will apply for the widened areas. The embankment construction should generally follow OPSD 203.010 and 203.040, included in Appendix F, with the depths of excavation as noted herein.





## 2.13 Unsupported Excavations

All excavations at this site must be conducted in accordance with the Occupational Health and Safety Act (OHSA) and Regulations for Construction (O. Reg. 213/91). All fills and native soils, with the exception of the encountered peat, may be classified as a Type 3 soil above the groundwater table in conformance with the OHSA. The encountered peat, and all soils below the groundwater table may be classified as a Type 4 soil. Temporary excavation side slopes for Type 3 soil should not exceed 1H:1V in accordance with OHSA. Temporary excavation side slopes advanced through, or terminating in, Type 4 soils should not exceed 3H:1V.

A slope stability analysis has been completed for a typical longitudinal section excavated to the base of the in-situ peat. The results of the analysis are shown on Fig. E-8 in Appendix E. The analysis resulted in a Factor of Safety of 1.685, which indicates that temporary excavations through the peat should remain stable at 3H:1V.

The need to excavate flatter side slopes if excessively wet or soft/loose materials are encountered, should not be overlooked. 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 wet weather conditions. In addition, some localized surficial sloughing may be experienced in areas of perched groundwater seepage (i.e. within the embankment fill).

## 2.14 Inlet and Outlet

### 2.14.1 Erosion Protection at Inlet and Outlet

Rip-rap protection should be provided for the culvert inlets and outlets, and the creek bed, both upstream and downstream of the culvert openings. The rip-rap should begin approximately 5 m upstream of the culvert inlet and extend 5 m downstream of the culvert outlet, and line the embankment slope to the design high water level. The size of the rip-rap is a function of the creek's hydrology, specifically the maximum projected flow velocity for the design flood event. As a rule of thumb, the thickness of the rip-rap layer should be a minimum of twice the median particle size, and 300 mm thick as a minimum. A non-woven geotextile should be placed between the rip-rap and native soils to prevent migration of the fine grained native soils into the rip-rap. The geotextile shall consist of Class II non-woven material with an equivalent opening size of 75-150  $\mu\text{m}$ . The rip-rap configuration at the creek bed should generally follow the OPSD 810.010, which is included in Appendix F of this report.

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

### 2.14.2 Seepage Cut-off Requirements

For the new culvert installation, a clay seal or cut-off wall should be constructed to prevent the migration of material along the exterior sidewalls of the culvert, the formation of flow paths, and any potential internal erosion within the roadway embankment. The type and design of cut-off utilized will be based on the creek hydraulics at the site and should be designed by the structural engineer.

Where readily available, a clay seal may be utilized. OPSS. PROV 1205 outlines the material requirements used for clay seals. The material shall be either a natural clay, clay mixture, or a geosynthetic clay liner (GCL). The coefficient of permeability shall not exceed  $1 \times 10^{-5} \text{ mm/s}$ .



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 for a minimum of 1.0 m along the side of the culvert.
- The clay seal should extend from the base of the trench to 1.0 m above the expected high water mark. The clay seal should extend laterally the full width of the trench.
- The clay should have a Liquid Limit greater than 50% and a Plasticity Index greater than  $0.75 \times (\text{Liquid Limit} - 20\%)$ .
- The clay seal is to be placed in maximum 150 mm thick lifts and compacted to 95% SPMDD within 2% of the optimum moisture content.

If the GCL is used as a clay seal, its material specifications containing the physical, mechanical and hydraulic properties shall be obtained from the manufacturer.

## 2.15 Obstructions

The in-situ fill materials within the roadway embankment were found to contain boulders and cobbles. In addition, till materials were encountered at depth. Till materials often contain cobbles and boulders, even if not indicated by the borings. These potential obstructions may impact excavations and/or the construction of temporary protection systems. A non-standard special provision is provided in Appendix G, which may form the basis for advising the contractor on this issue.

### 3 Closure

The recommendations made in this report are in accordance with our present understanding of the project and are provided solely for the design 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 noted. Should any conditions at the site be encountered that differ from those reported at the test locations, we require that we be notified immediately in order to allow reassessment of our recommendations. It may then be necessary to perform additional investigation and analysis.

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

This Foundation Investigation and Design Report has been prepared by Ian MacMillan, P.Eng. It has been reviewed by Andy Schell, M.Sc.(Eng.), P.Eng., TaeChul Kim, M.E.Sc., P.Eng., and by Stan E. Gonsalves, M.Eng., P.Eng., Designated MTO Foundation Contact. The field investigation was supervised by Shane Tobias and Nicole Wyld.

Yours truly,

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## 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

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## 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 utilized 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**.

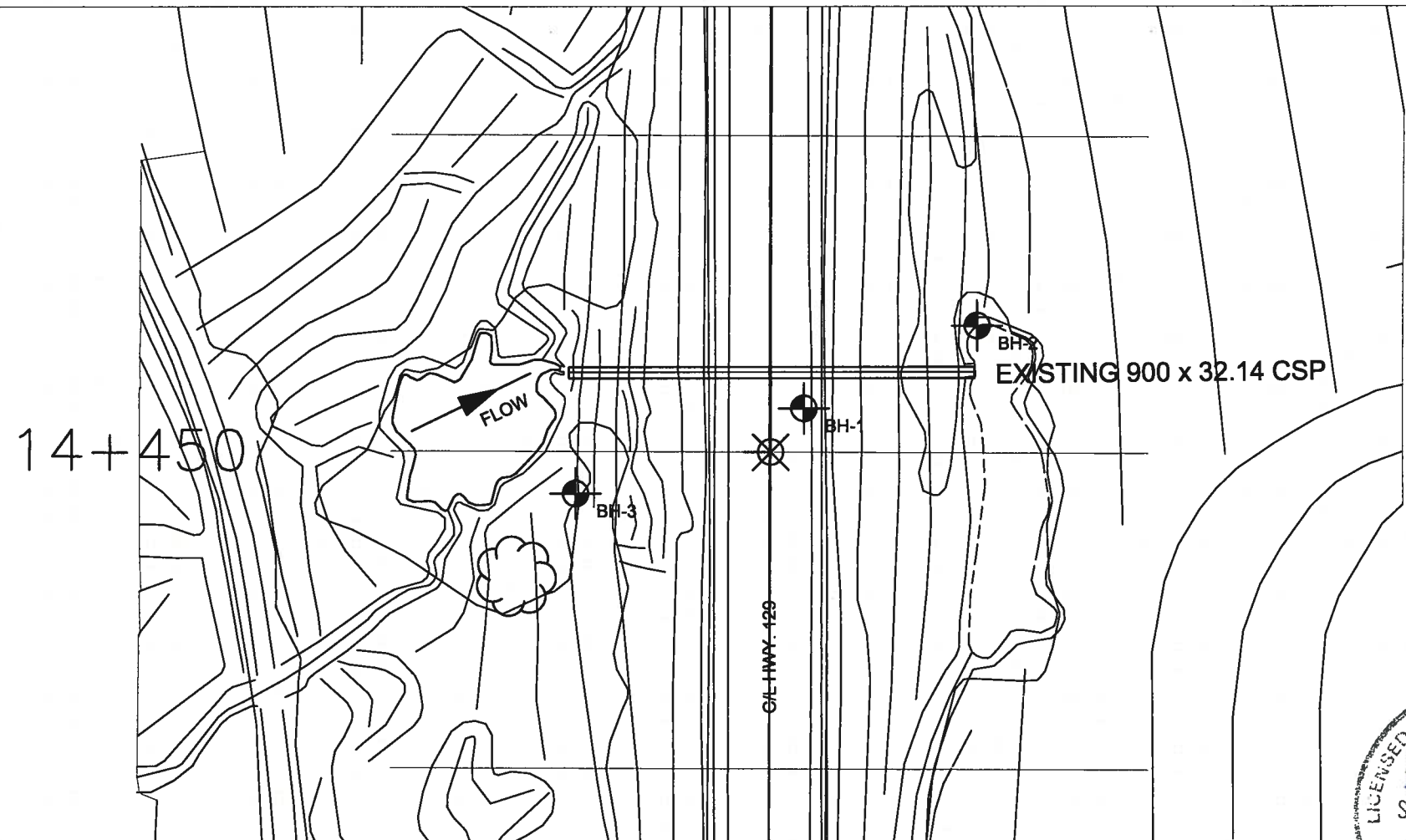


## Appendix A – Drawings

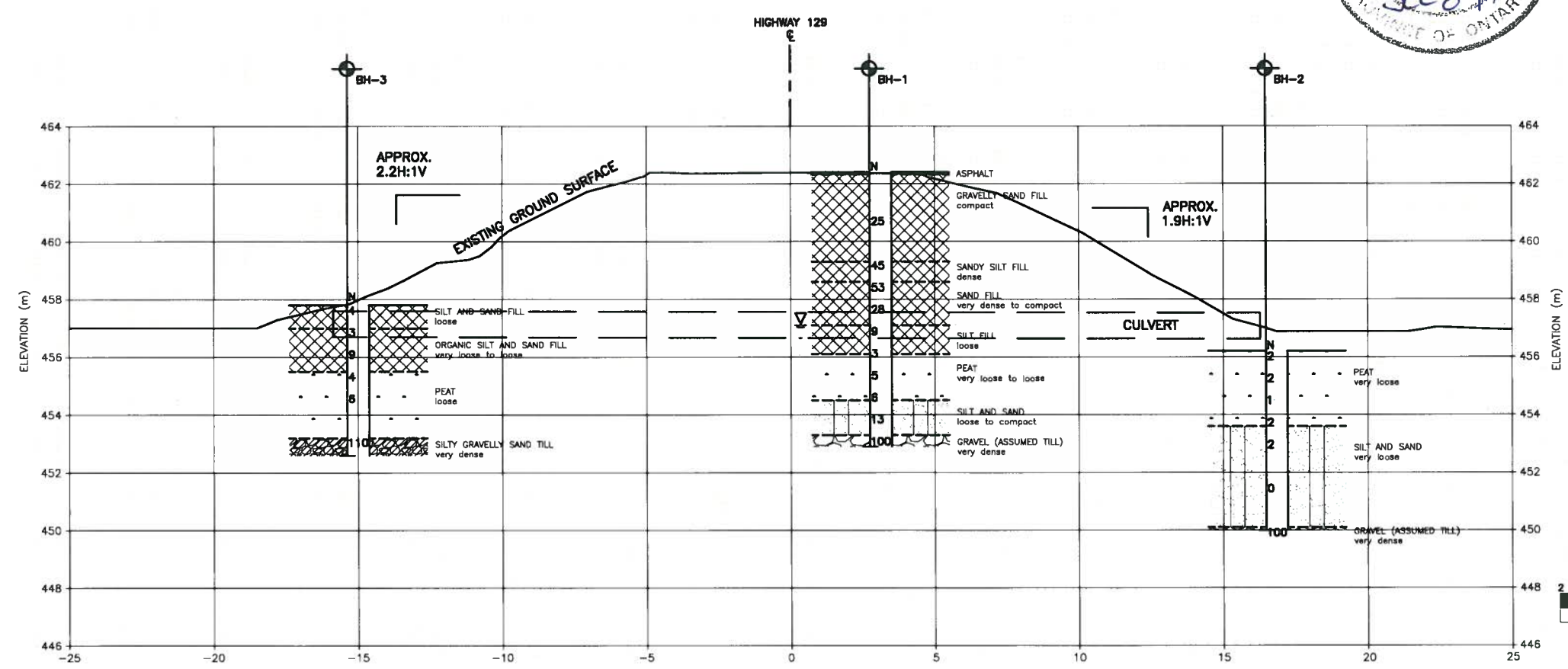




14+450



PLAN



CROSS SECTION AT CULVERT CENTRELINE

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



Agreement No. 5016-E-0016  
GWP 411-00-00  
GEOCRE No. 410-32

CULVERT REPLACEMENT, STN. 14+457  
HIGHWAY 129, REANEY TOWNSHIP  
DISTRICT OF SUDBURY  
BOREHOLE LOCATION PLAN AND SOIL  
STRATA

SHEET  
1

KEY PLAN - NTS

LEGEND

- BOREHOLE LOCATION
- STANDARD PENETRATION TEST (BLOWS/300mm)
- TEMPORARY BENCHMARK (EL. 482.4 m)
- ESTIMATED WATER LEVEL IN BOREHOLE

BOREHOLE NO.	APPROX. ELEV. (m)	MTM COORDINATES	
		NORTHING	EASTING
BH-1	482.4	5288613.6	364209.1
BH-2	456.2	5288626.0	364218.0
BH-3	457.8	5288599.0	364196.5

NOTES

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 Contract Documents.

The complete foundation investigation and design report for this project and other related documents may be examined at the Metrolia 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 GPS Gen. Cond.

SOIL STRATA SYMBOLS

ASPHALT	SILT AND SAND
FILL	GRAVEL (ASSUMED TILL)
PEAT	SILTY GRAVELLY SAND TILL

REVISIONS

DATE	BY	DESCRIPTION
2017.6.26	IM	SUBMISSION FOR MTO REVIEW
2017.11.28	IM	FINAL REPORT SUBMISSION

SCALE: AS NOTED	PROJECT NO.: SUD-00014543-AG		
SUBMD: IM	CHECKED: AS	DATE: 2017.6.26	
DRAWN: IM	CHECKED: SG	APPROVED: SG	DWG. 1

## Appendix B – Photographs







Photograph No. 1 – Highway 129 at Culvert, Stn. 14+457 (Facing North)



Photograph No. 2 – Pavement Condition at Culvert (Facing North-East)





Photograph No. 3 – Eastern Embankment at Culvert Outlet (Facing North)



Photograph No. 4 – Culvert Outlet (Facing North-East)





Photograph No. 5 – Western Embankment at Culvert Inlet (Facing South)



Photograph No. 6 – Culvert Inlet (Facing West)

## Appendix C – Borehole Logs



# Explanation of Terms Used on Borehole Records

## SOIL DESCRIPTION

Terminology describing common soil genesis:

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

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

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

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

Terminology describing soil structure:

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

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

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

*Fissured:* material breaks along plane of fracture.

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

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

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

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

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

*Homogeneous:* same color and appearance throughout.

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

*Uniformly Graded:* predominantly on grain size.

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

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

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

Table a: Percent or Proportion of Soil, Pp

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

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

Table b: Apparent Density of Cohesionless Soil

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

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

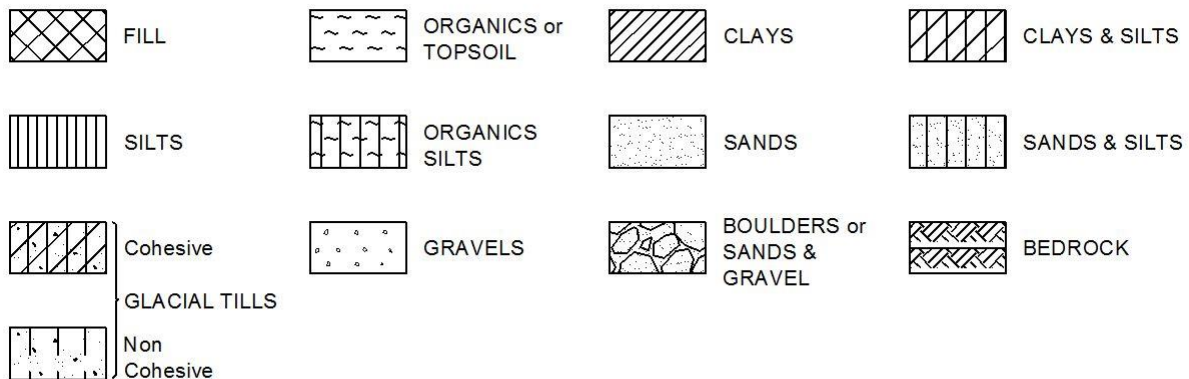
Table c: Consistency of Cohesive Soil

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

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

## STRATA PLOT

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



## WATER LEVEL MEASUREMENT



Open Borehole or Test Pit



Monitoring Well, Piezometer or Standpipe

## ABBREVIATIONS AND SYMBOLS

### FIELD SAMPLING

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

### STRESS AND STRAIN

$u_w$	kPa	Pore water pressure
$r_u$	1	Pore pressure ratio
$\sigma$	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

# RECORD OF BOREHOLE No BH-1

1 OF 1

**METRIC**

W.P. 411-00-00,5016-E-0016 LOCATION Stn. 14+455, MTM-13, 5266613.63N, 364209.1E, Non-Structural Culvert at Stn. 14+457 ORIGINATED BY NW  
 DIST Sudbury HWY 129 BOREHOLE TYPE Continuous Flight HSA and Washboring with NW Casing COMPILED BY IM  
 DATUM Geodetic DATE 2016.12.06 - 2016.12.06 LATITUDE 47.53601 LONGITUDE -83.21087 CHECKED BY IM

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									WATER CONTENT (%)	
								○ UNCONFINED		+ FIELD VANE								
							20	40	60	80	100							
							20	40	60	80	100							
462.4	Pavement Surface																	
462.0	ASPHALT (~ 76 mm thick)		1	AS												28 64 (7)		
	FILL, gravelly sand, trace silt, brown, moist, compact.		2	AS														
	No sample returned at ~ 1.5 m depth due to lodged stone in split spoon sampler. Auger sample obtained.		3	SS	25													
459.4																		
3.1	FILL, sandy silt, trace organics, trace gravel, trace clay, brown, moist, dense.		4	SS	45											5 30 63 3		
458.6	some cobbles and boulders below ~ 3.5 m depth.																	
3.8	FILL, sand, trace silt, trace to some gravel, some cobbles and boulders, brown, moist, very dense to compact.		5	SS	53													
	some silt, trace roots, grey below ~ 4.6 m depth.		6	SS	28													
457.1																		
5.3	FILL, silt, some organics, some sand, trace clay, brown to dark brown, wet, loose.		7	SS	9											7 16 74 3		
456.1																		
6.3	PEAT, with wood, black, moist to wet, very loose to loose.		8	SS	3													
	No soil sample returned at ~ 6.9 m depth due to lodged wood in split spoon sampler.		9	SS	5													
454.5																		
7.9	SILT AND SAND, trace clay, trace gravel, grey, moist, loose to compact.		10	SS	6													
453.3																		
9.1	GRAVEL, grey, moist. (Assumed Till)		12	SS	100													
453.0																		
9.5	END OF BOREHOLE Borehole terminated at ~ 9.5 m depth due to refusal on suspected very dense till or bedrock																	
	NOTES: 1. This drawing to be read with the subject report and project numbers as presented above. 2. Multiple attempts made to advance borehole beyond refusal depth. 3. Groundwater condition noted may not be accurate as water was pumped into hole due to washboring techniques utilized.																	

# RECORD OF BOREHOLE No BH-2

1 OF 1

METRIC

W.P. 411-00-00,5016-E-0016 LOCATION Stn. 14+460, MTM-13, 5266625.95N, 364218.01E, Non-Structural Culvert at Stn. 14+457 ORIGINATED BY ST  
DIST Sudbury HWY 129 BOREHOLE TYPE Portable Tripod With Cathead and Hilti D200 Drill COMPILED BY IM  
DATUM Geodetic DATE 2017.01.29 - 2017.01.29 LATITUDE 47.53612 LONGITUDE -83.21075 CHECKED BY IM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
456.2	Ground Surface													
0.0	PEAT, some to with silty sand, some gravel, grey to black, wet, very loose.  no gravel below ~ 0.8 m depth.		1	SS	2		456							
			2	SS	2		455						94.4	
			3	SS	1		454						134.7	
453.6			4	SS	2		453							
2.6	SILT AND SAND, some organics, trace gravel, trace clay, grey, wet, very loose.		5	SS	2		452							
			6	SS	0		451							
450.1			7	SS	100									
450.0	GRAVEL, grey, wet. (Assumed Till)													
6.2	END OF BOREHOLE Borehole terminated at ~ 6.2 m depth due to refusal on suspected very dense till or bedrock  NOTES: 1. This drawing to be read with the subject report and project numbers as presented above. 2. Multiple attempts made to advance borehole beyond refusal depth. 3. Groundwater level not measured within borehole as water was pumped into hole due to washboring technique utilized.													

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



# RECORD OF BOREHOLE No BH-3

1 OF 1

METRIC

W.P. 411-00-00,5016-E-0016 LOCATION Stn. 14+447, MTM-13, 5266599.04N, 364196.45E, Non-Structural Culvert at Stn. 14+457 ORIGINATED BY ST  
DIST Sudbury HWY 129 BOREHOLE TYPE Portable Tripod With Cathead and Hilti D200 Drill COMPILED BY IM  
DATUM Geodetic DATE 2017.01.28 - 2017.01.28 LATITUDE 47.53588 LONGITUDE -83.21104 CHECKED BY IM

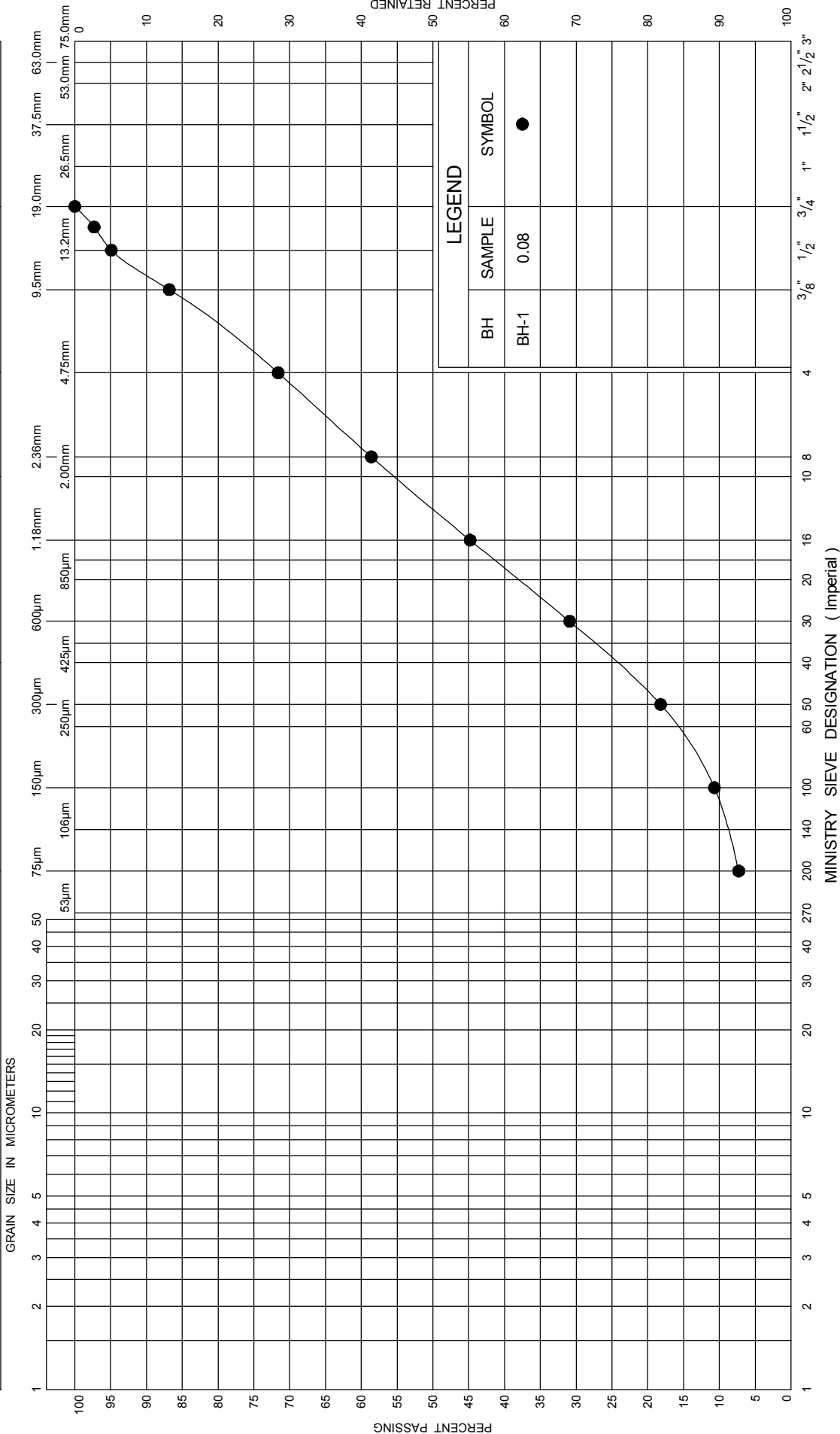
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
○ UNCONFINED      + FIELD VANE ● QUICK TRIAXIAL    x LAB VANE																	
457.8	Ground Surface						20	40	60	80	100						
0.0	FILL, silt and sand, some organics, grey, wet, loose.		1	SS	4												
457.0																	
0.8	FILL, organic silt and sand, with peat, grey to black, wet, very loose to loose.		2	SS	3												
			3	SS	9												
455.5																	
2.3	PEAT, with silty sand, black, wet, loose.		4	SS	4												
			5	SS	6												
453.2																	
4.6	TILL, silty gravelly sand, trace clay, grey, wet, very dense.		6	SS	110											32 35 32 1	
452.6																	
5.2	END OF BOREHOLE Borehole terminated at ~ 5.2 m depth due to refusal on suspected very dense till or bedrock																
	NOTES: 1. This drawing to be read with the subject report and project numbers as presented above. 2. Multiple attempts made to advance borehole beyond refusal depth. 3. Groundwater level not measured within borehole as water was pumped into hole due to washboring technique utilized.																

## Appendix D – Laboratory Test Results



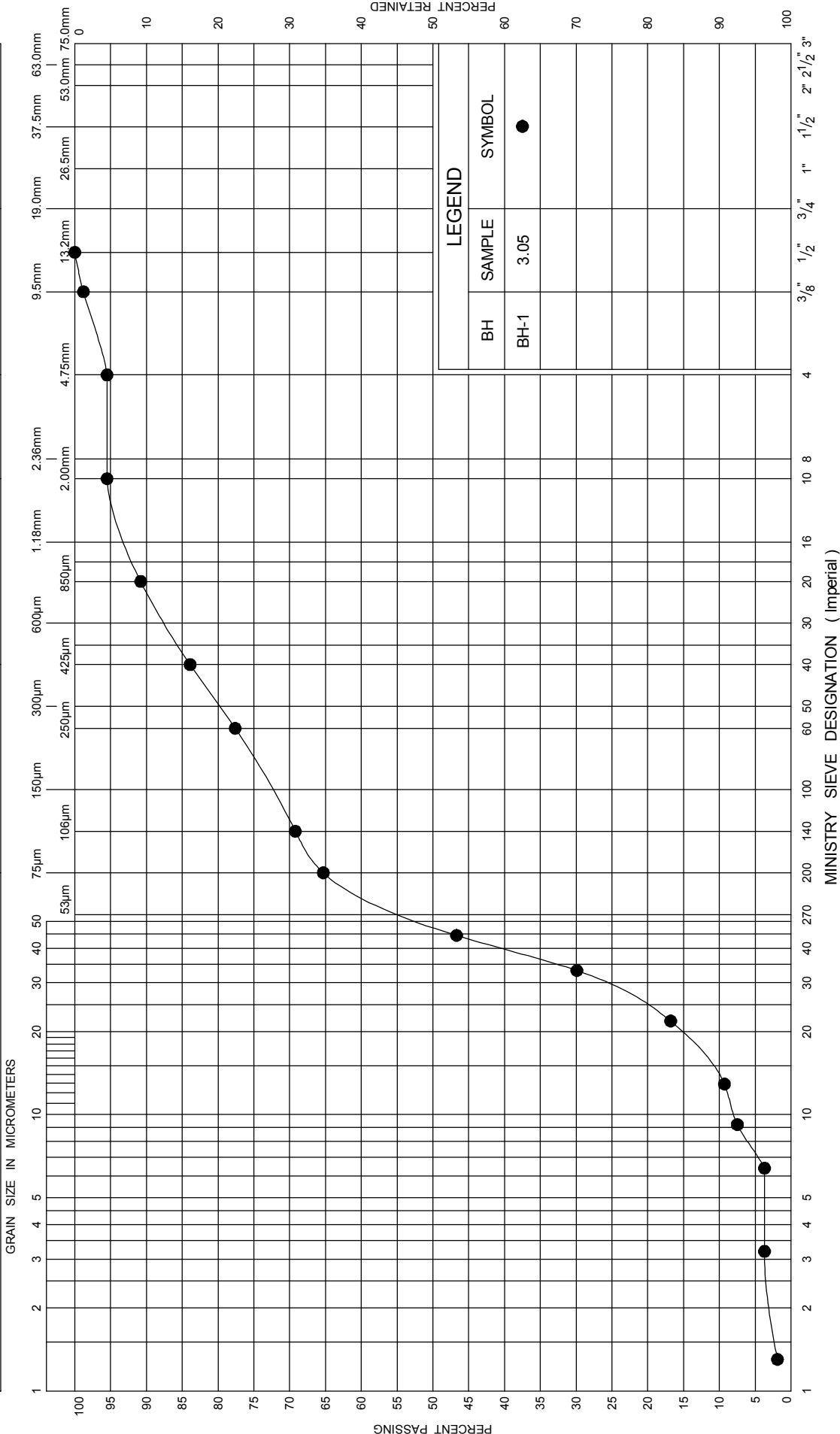
UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT		SAND			GRAVEL		
		Fine		Medium	Coarse	Fine	Coarse



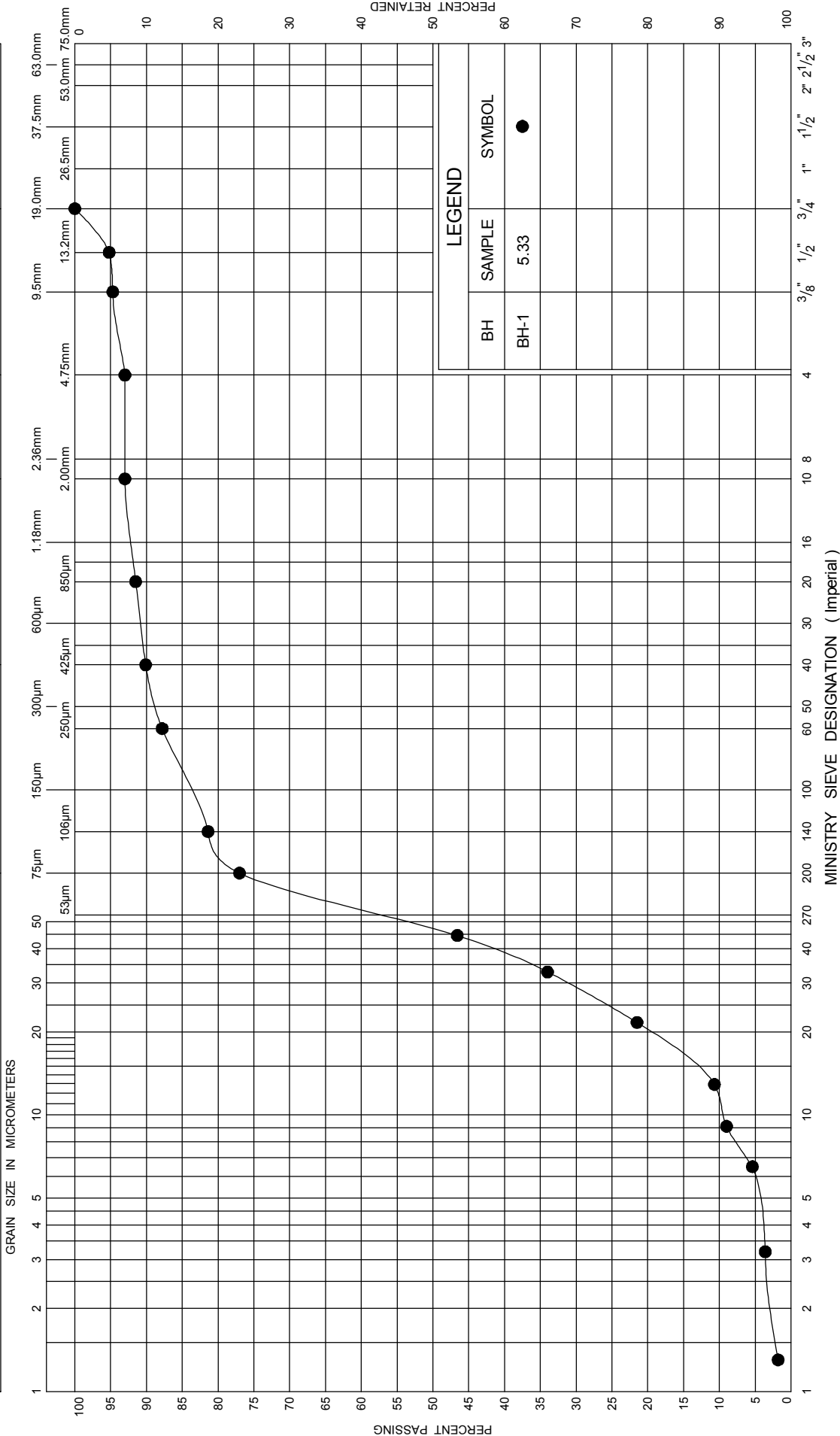
UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT		SAND			GRAVEL	
		Fine		Medium	Fine	Coarse



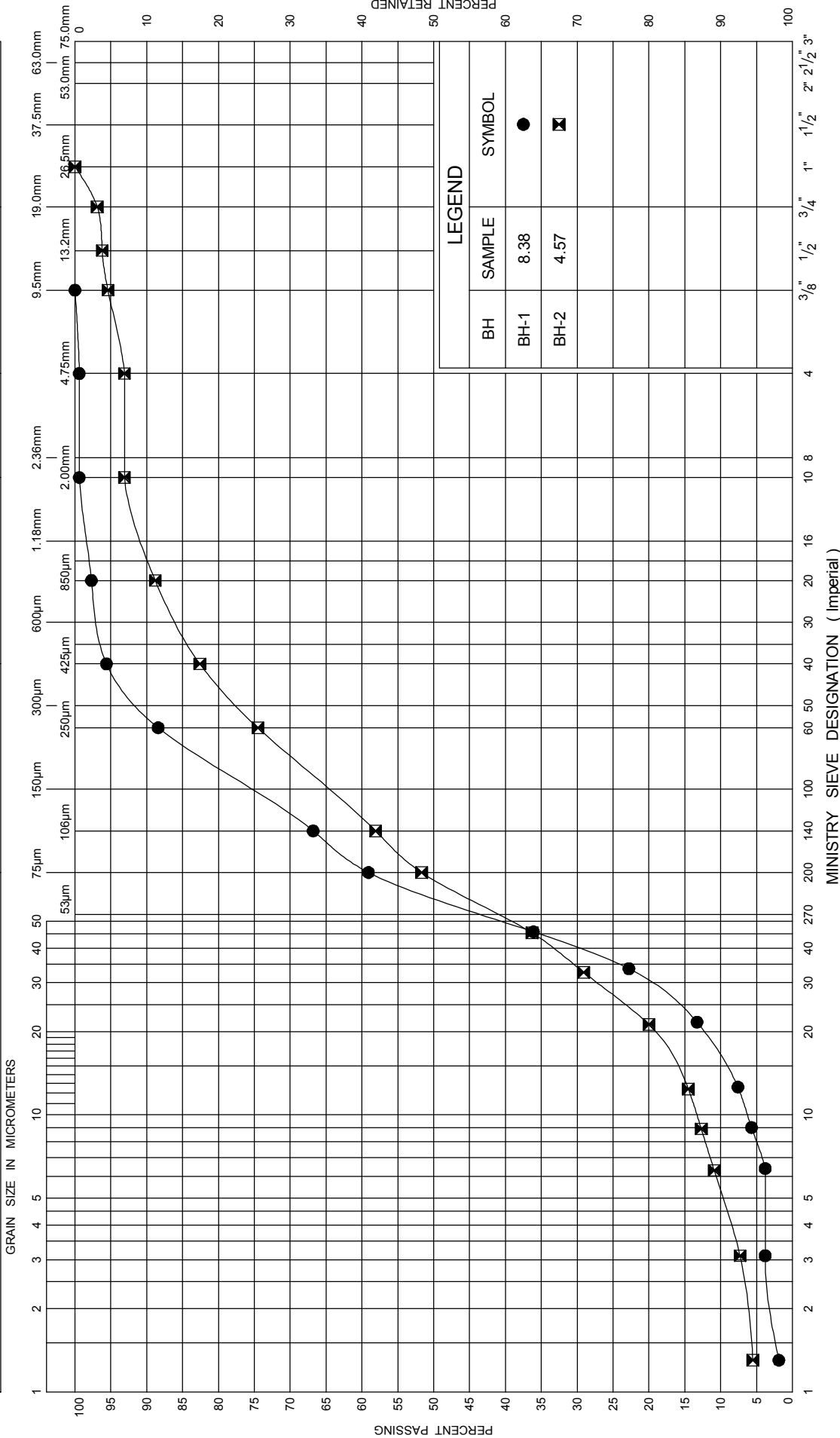
UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT		SAND			GRAVEL	
		Fine	Medium	Coarse	Fine	Coarse



UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT		SAND			GRAVEL		
		Fine		Medium	Coarse	Fine	Coarse





UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT		SAND			GRAVEL		
		Fine		Medium	Coarse	Fine	Coarse

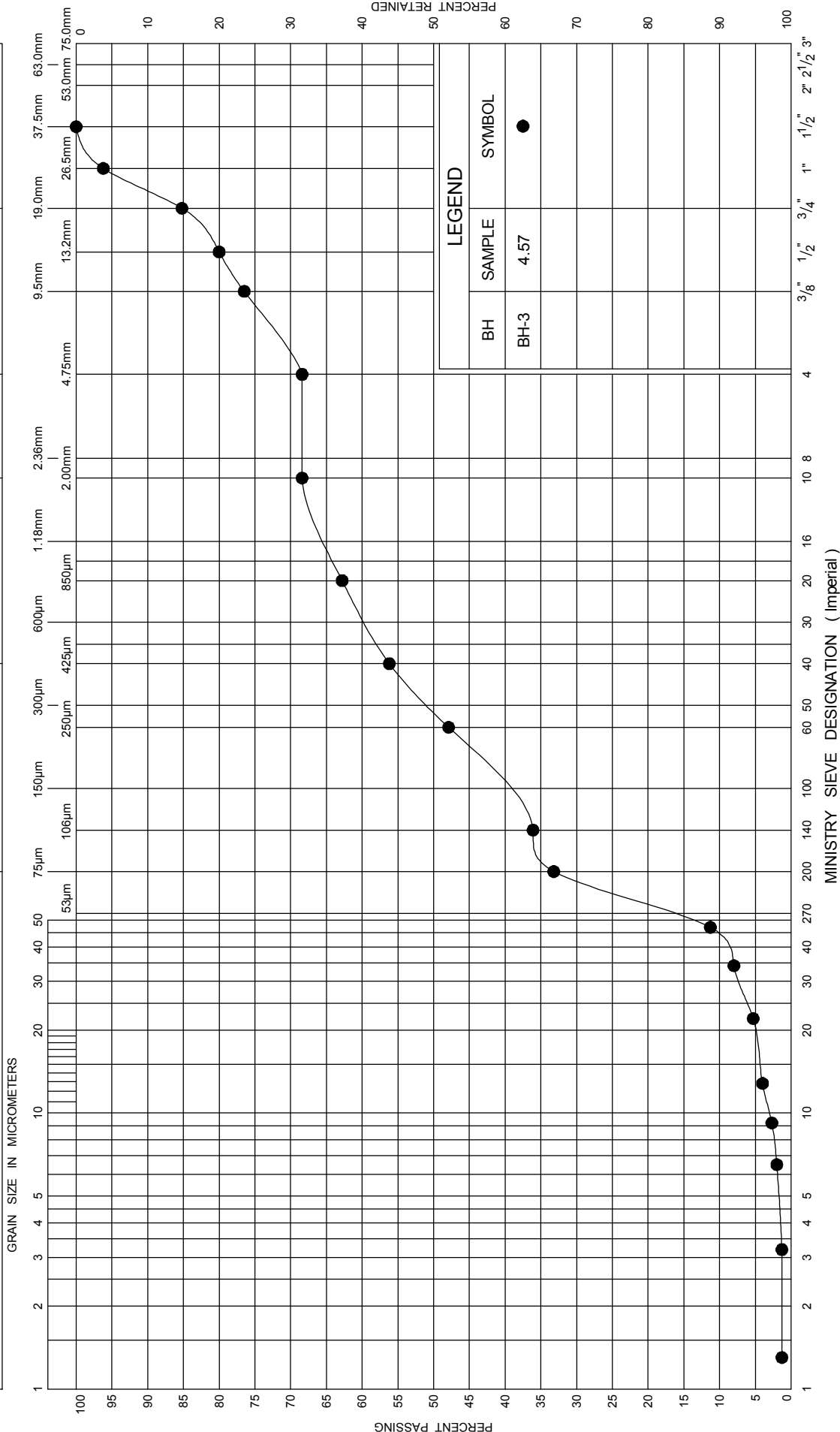


FIG No 5



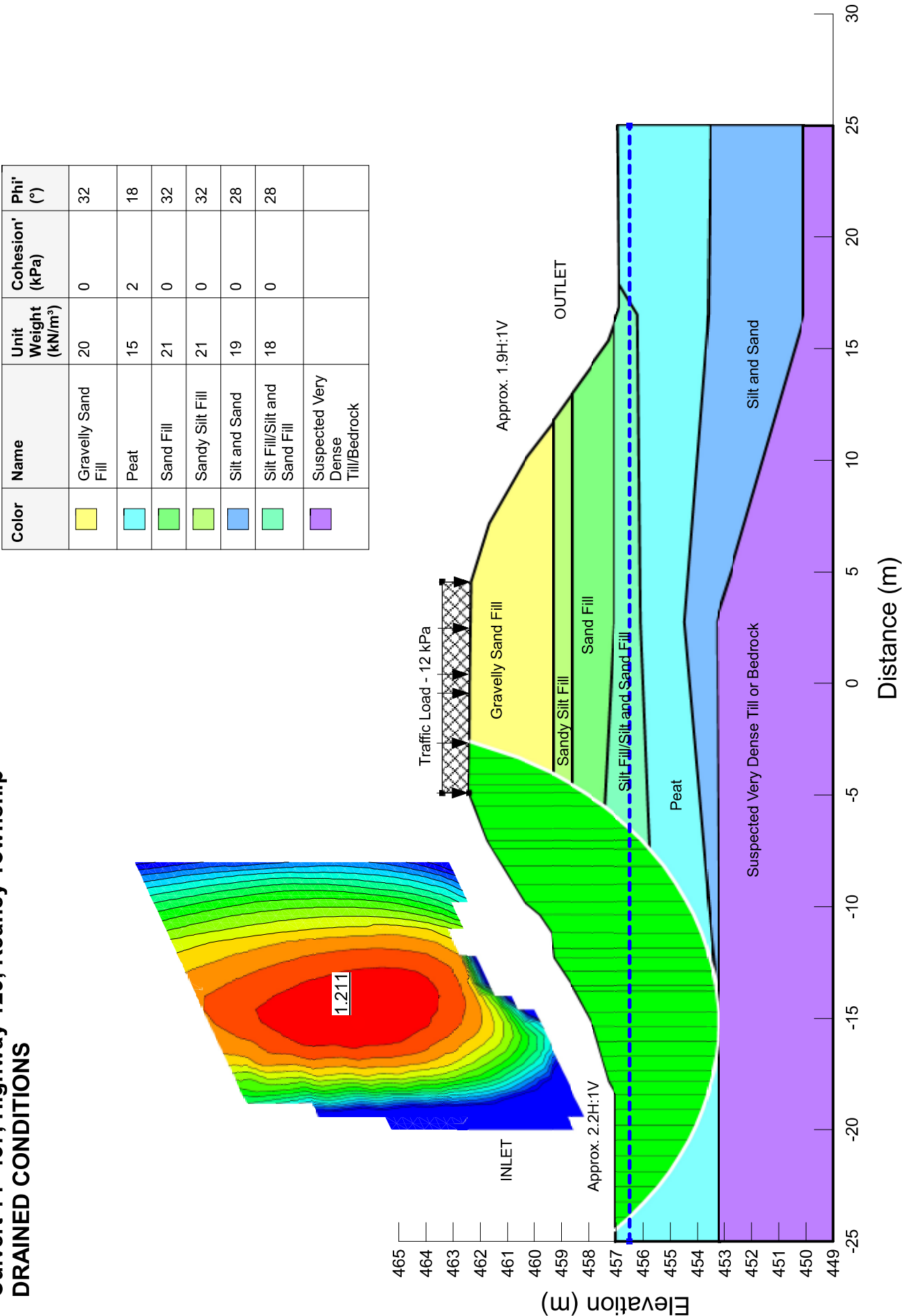
GRAIN SIZE DISTRIBUTION  
SILTY GRAVELLY SAND TILL

W P 411-00-00,5016-E-0016  
Culvert Replacement

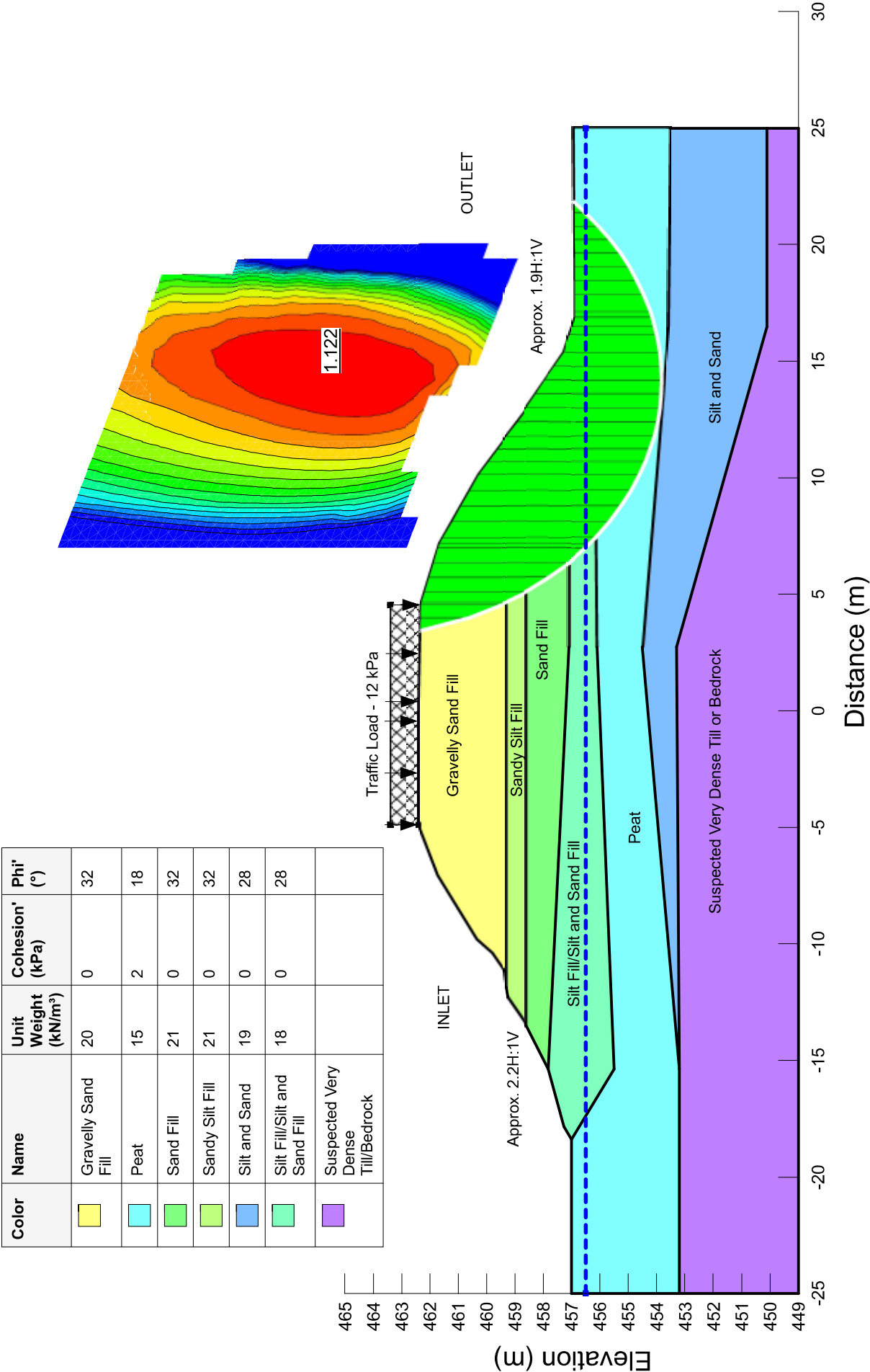
## Appendix E – Slope Stability Analyses



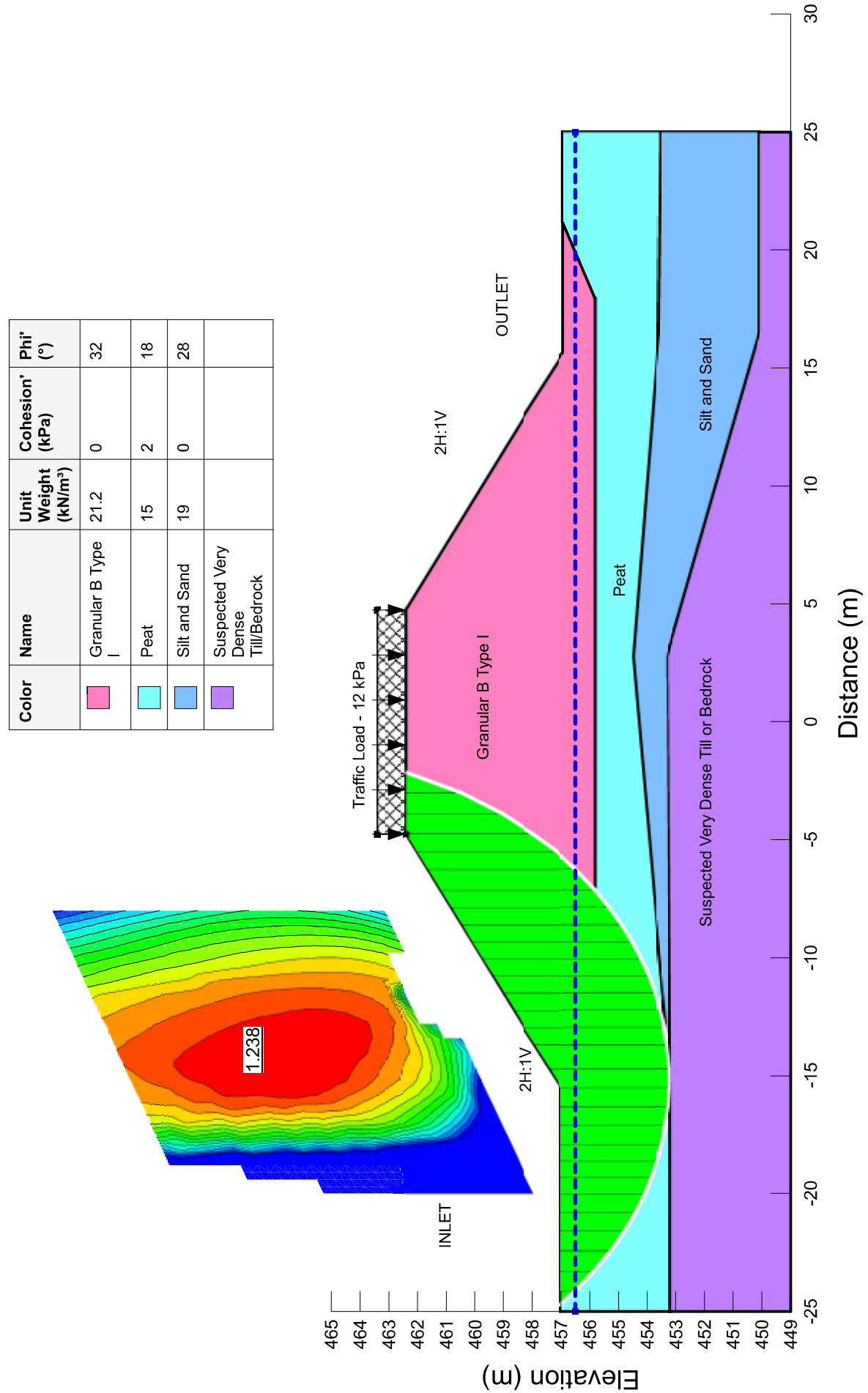
FIGURE E-1 - Existing Embankment Stability - Inlet Side  
Culvert 14+457, Highway 129, Reaney Township  
DRAINED CONDITIONS



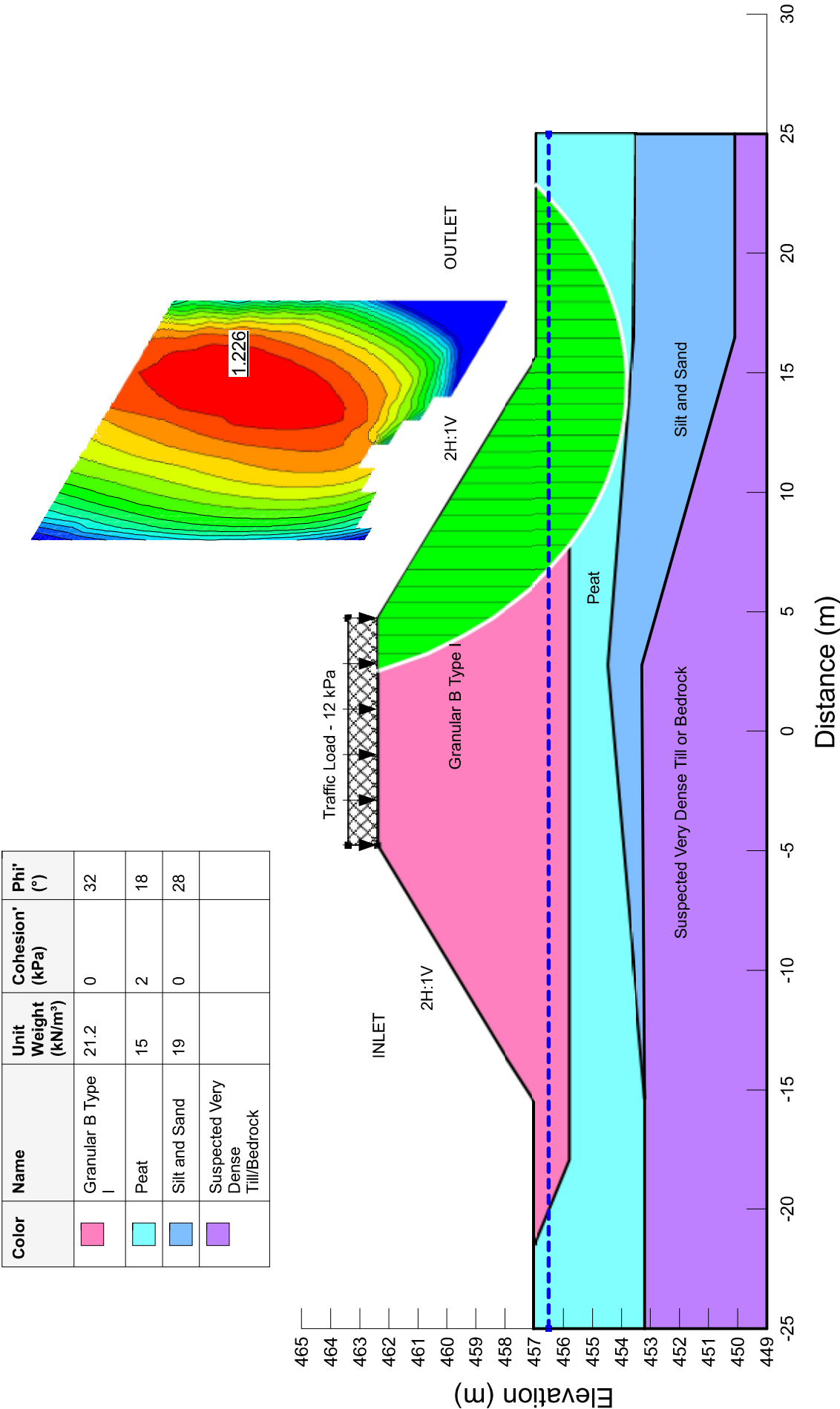
**FIGURE E-2 - Existing Embankment Stability - Outlet Side**  
**Culvert 14+457, Highway 129, Reaney Township**  
**DRAINED CONDITIONS**



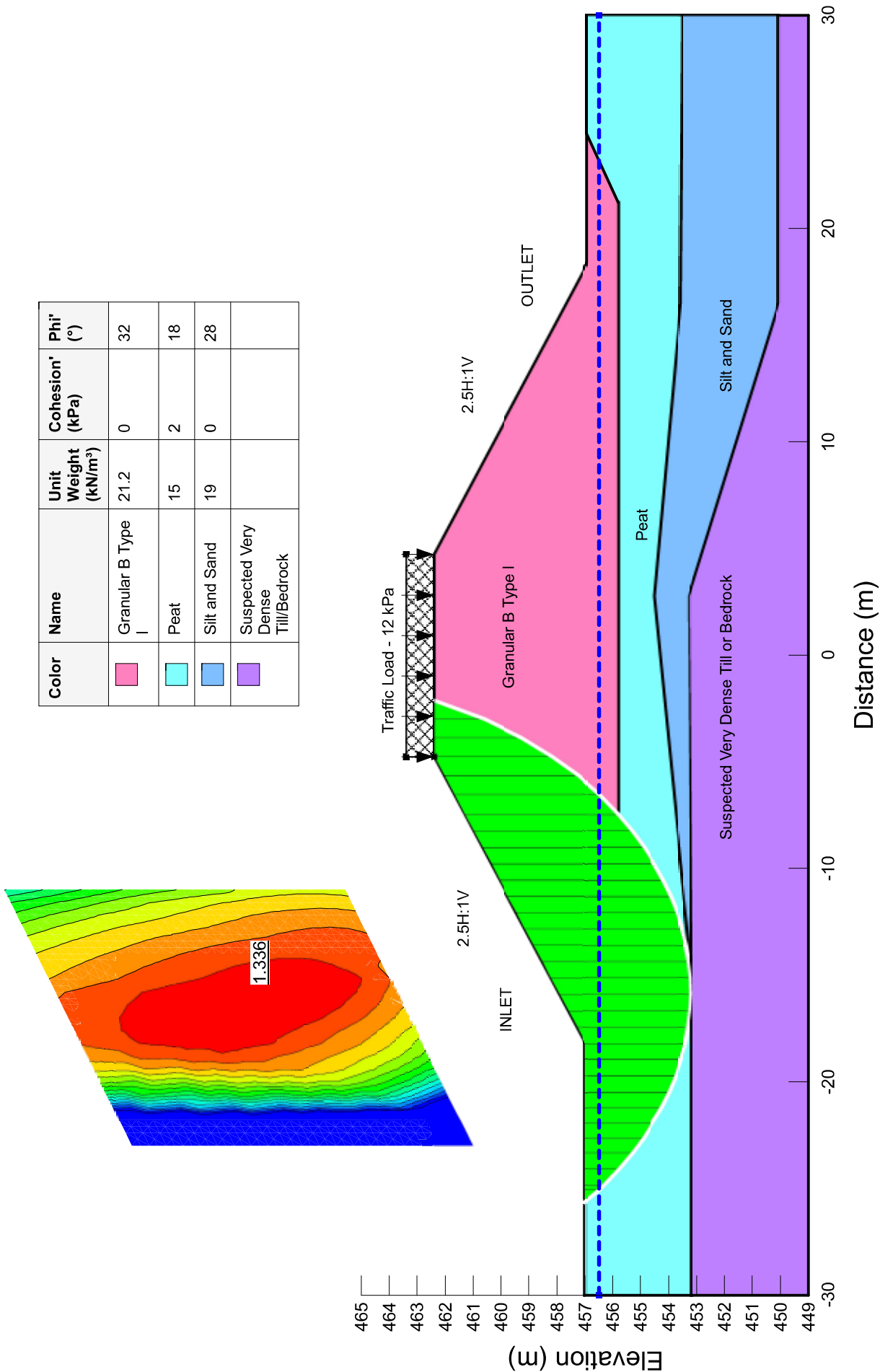
**FIGURE E-3 - Proposed Embankment Stability - Inlet Side - 2H:1V Side Slopes**  
**Culvert 14+457, Highway 129, Reaney Township**  
**DRAINED CONDITIONS**



**FIGURE E-4 - Proposed Embankment Stability - Outlet Side - 2H:1V Side Slopes**  
**Culvert 14+457, Highway 129, Reaney Township**  
**DRAINED CONDITIONS**



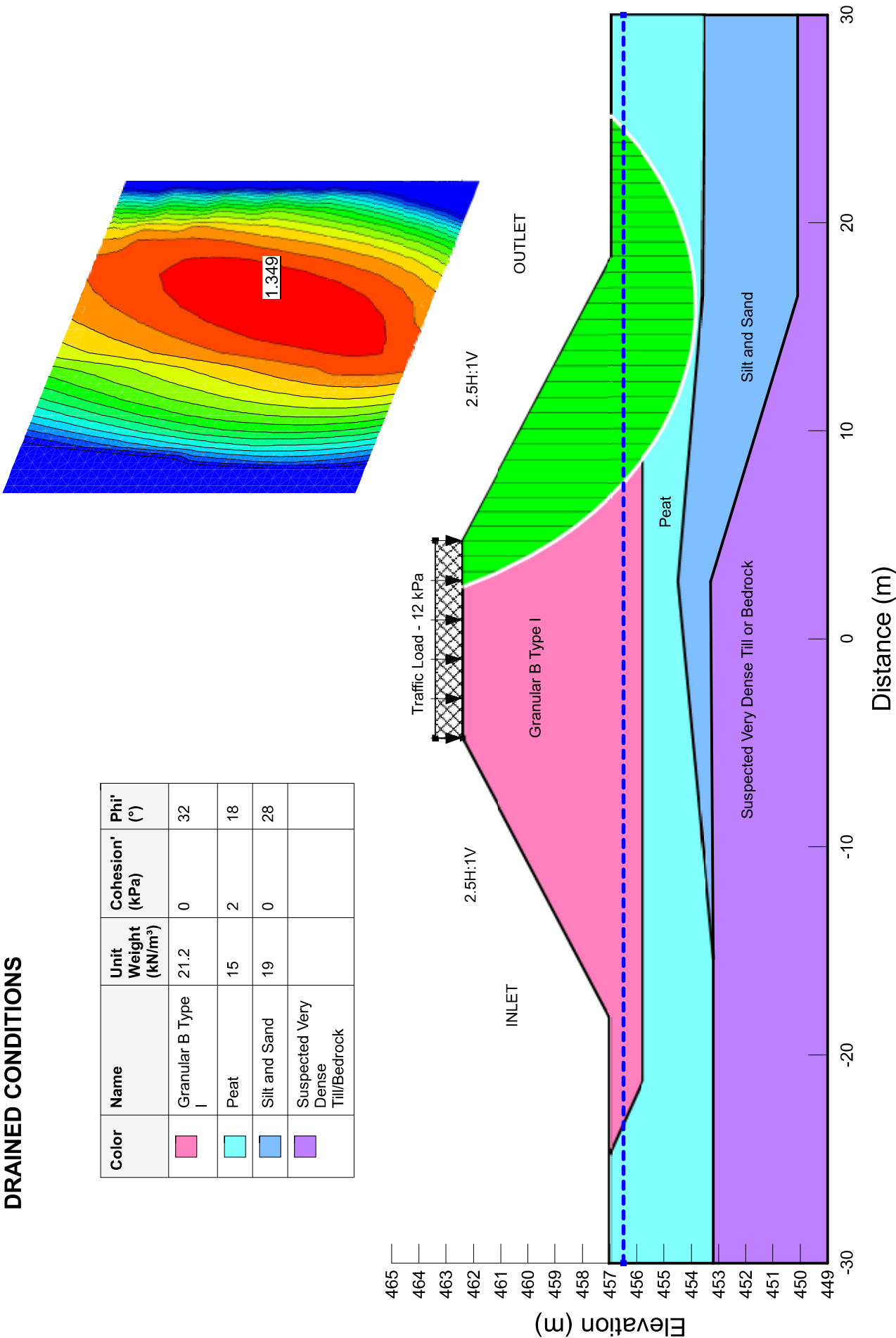
**FIGURE E-5 - Proposed Embankment Stability - Inlet Side - 2.5H:1V Side Slopes**  
**Culvert 14+457, Highway 129, Reaney Township**  
**DRAINED CONDITIONS**









**FIGURE E-6 - Proposed Embankment Stability - Outlet Side - 2.5H:1V Side Slopes**  
**Culvert 14+457, Highway 129, Reaney Township**  
**DRAINED CONDITIONS**

Color	Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
<div></div>	Granular B Type I	21.2	0	32
<div></div>	Peat	15	2	18
<div></div>	Silt and Sand	19	0	28
<div></div>	Suspected Very Dense Till/Bedrock			



**FIGURE E-7 - Detour Embankment Stability**  
**Outlet Side - West Embankment**  
**Culvert 14+457, Highway 129, Reaney Township**  
**DRAINED CONDITIONS**

Color	Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
	Granular B Type I	21.2	0	32
	Peat	15	2	18
	Silt and Sand	19	0	28
	Suspected Very Dense Till/Bedrock			

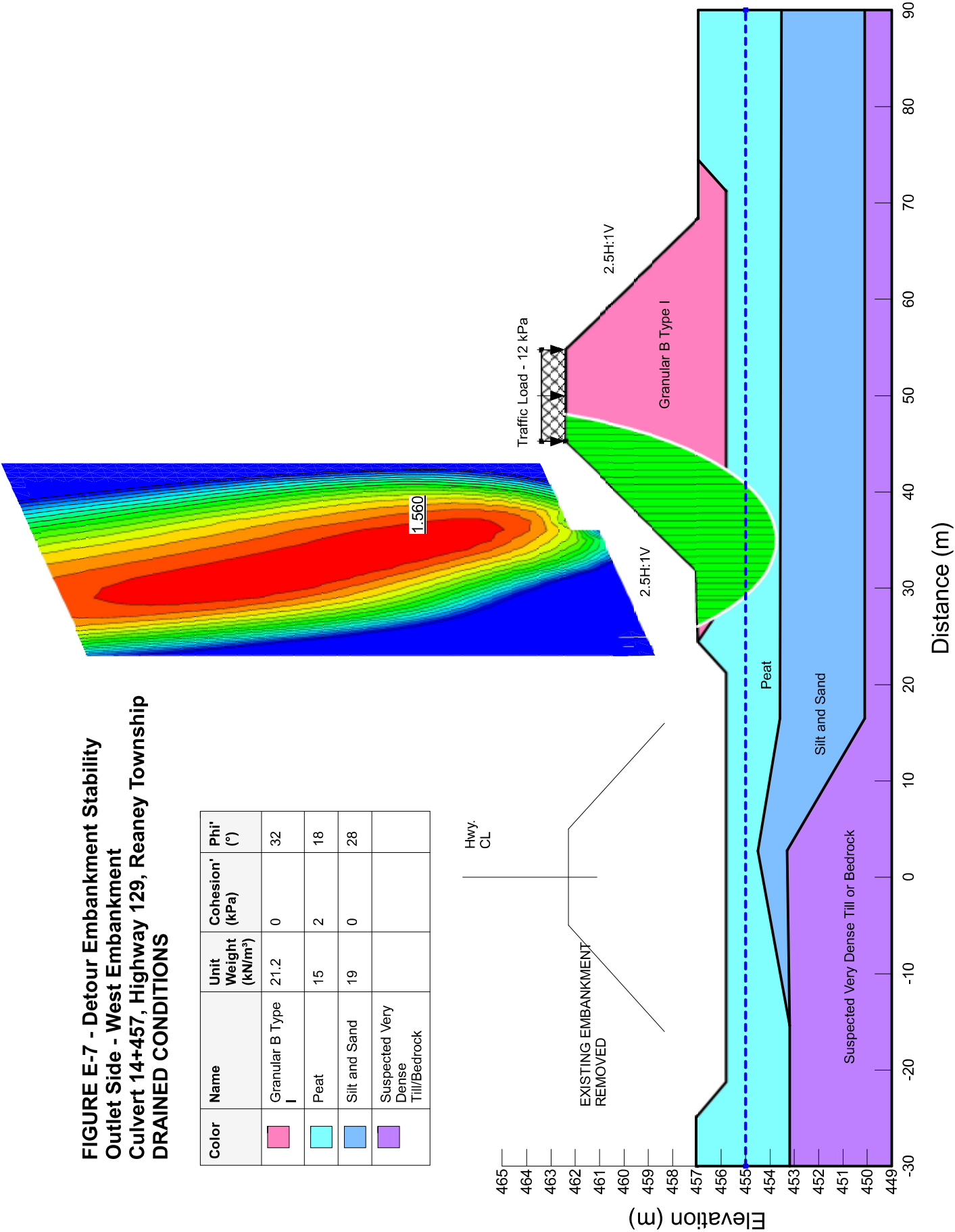
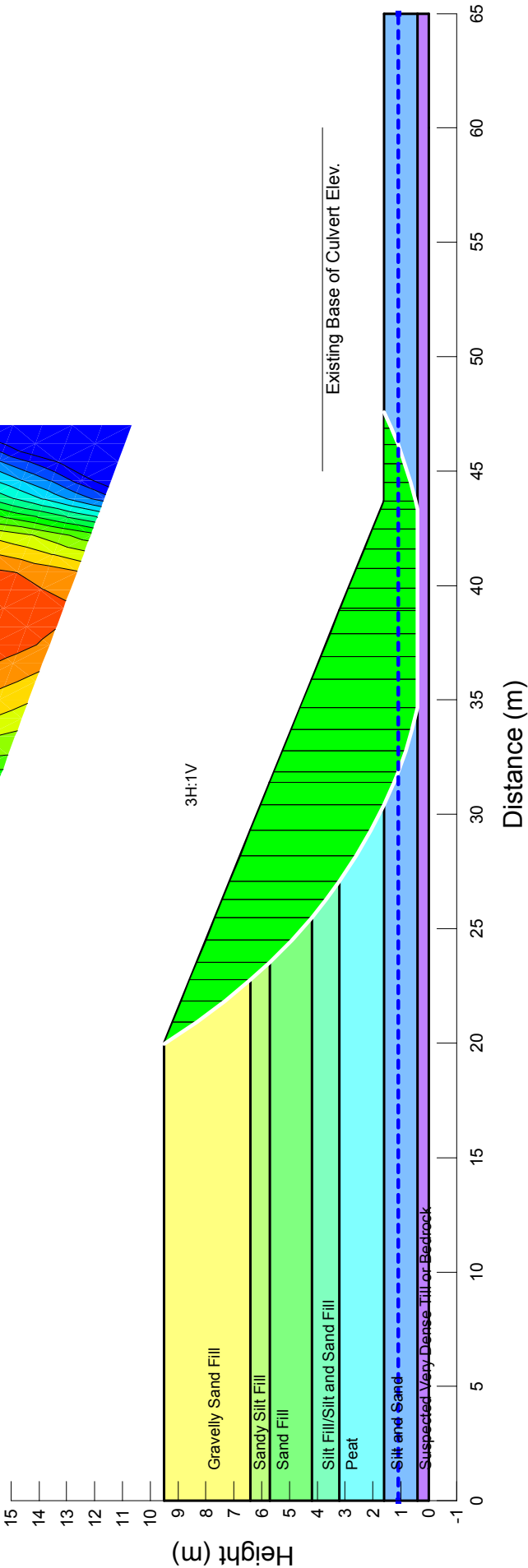
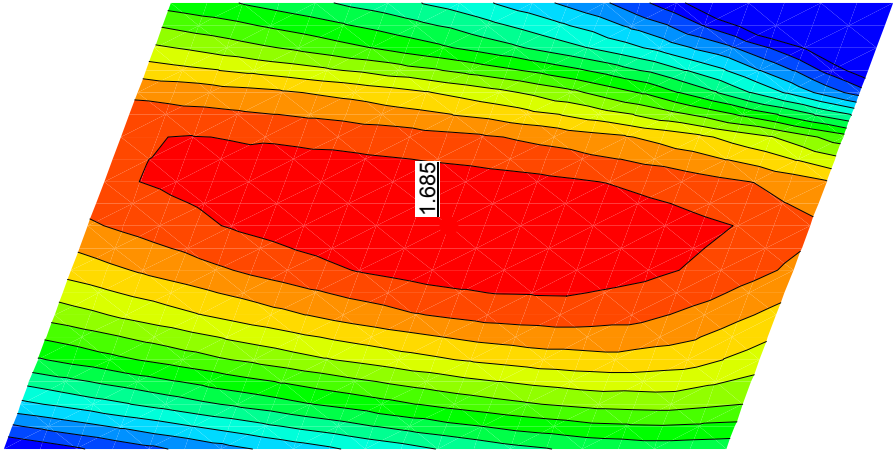


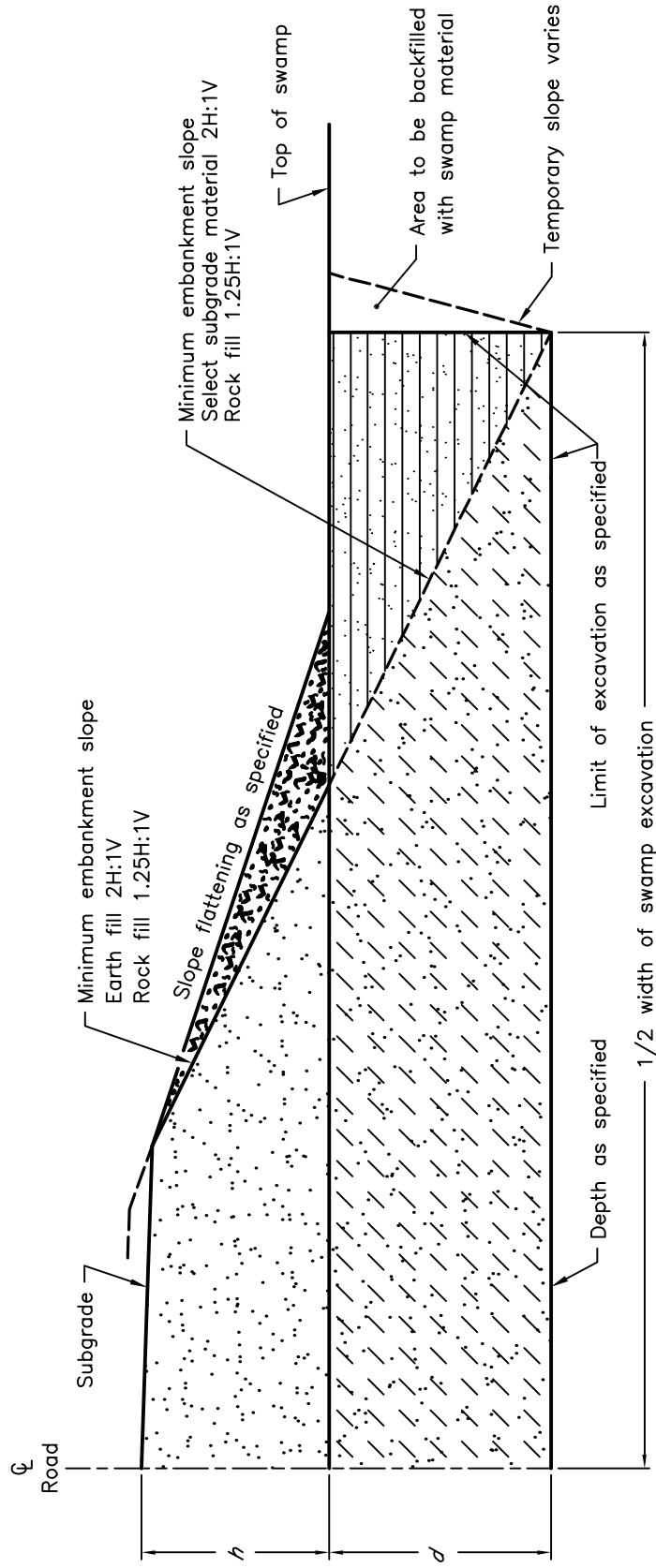
FIGURE E-8 - Temporary Excavation Stability  
Culvert 14+457, Highway 129, Reaney Township  
DRAINED CONDITIONS

Color	Name	Unit Weight (kN/m³)	Cohesion* (kPa)	Phi* (°)
<div></div>	Gravelly Sand Fill	20	0	32
<div></div>	Peat	15	2	18
<div></div>	Sand Fill	21	0	32
<div></div>	Sandy Silt Fill	21	0	32
<div></div>	Silt and Sand	19	0	28
<div></div>	Silt Fill/Silt and Sand Fill	18	0	28
<div></div>	Suspected Very Dense Till/Bedrock			



## Appendix F – Ontario Provincial Standards Drawings (OPSD)

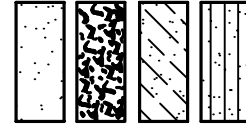




# NOTES:

- A For this OPSD,  $h \leq 4.5\text{m}$  and  $d \leq 6.0\text{m}$ .
- B Height of fill is the vertical difference between subgrade and top of swamp measured at new road centreline.
- C For divided roads with median  $< 10\text{m}$ , excavate swamp material full width.
- D For divided roads with median  $\geq 10\text{m}$ , excavate swamp material to limits as specified.

# LEGEND:



$h$  - Height of fill  
 $d$  - Depth of sub-excavation

ONTARIO PROVINCIAL STANDARD DRAWING

# EMBANKMENTS OVER SWAMP

NEW CONSTRUCTION

Nov 2010 Rev 3

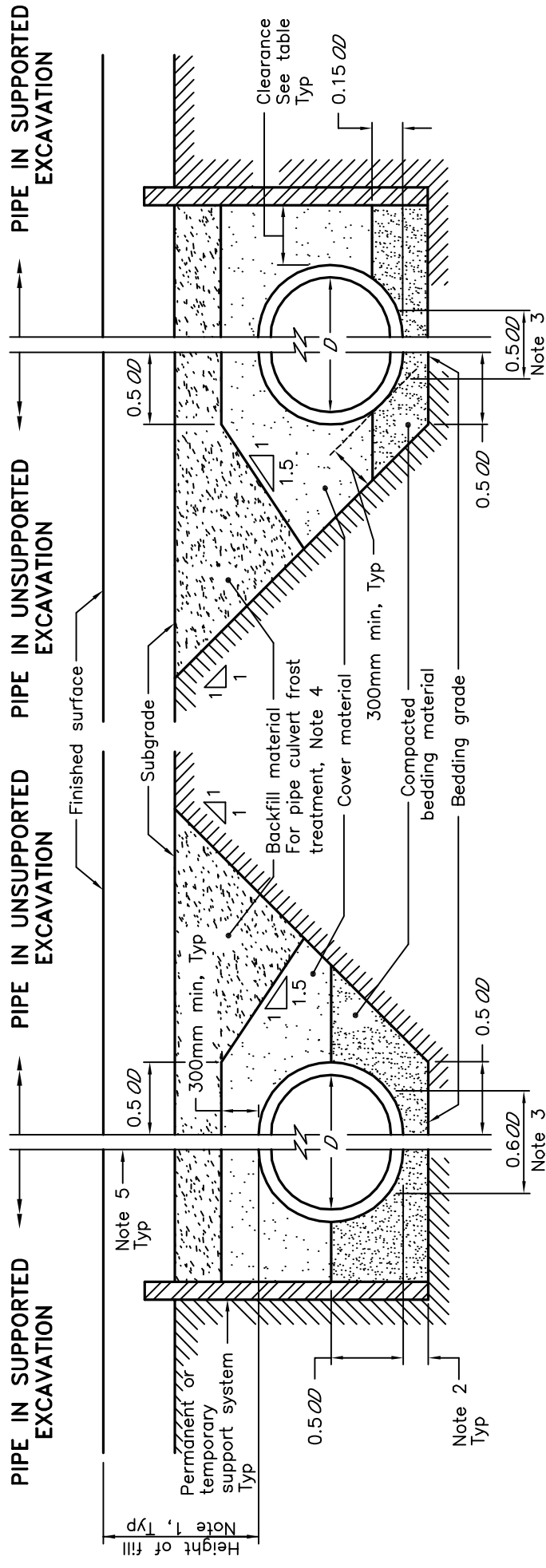


OPSD 203.010









### CLASS B BEDDING

### CLASS C BEDDING

#### NOTES:

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

B All dimensions are in metres unless otherwise shown.

#### LEGEND:

- D – Inside diameter  
OD – Outside diameter

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

ONTARIO PROVINCIAL STANDARD DRAWING

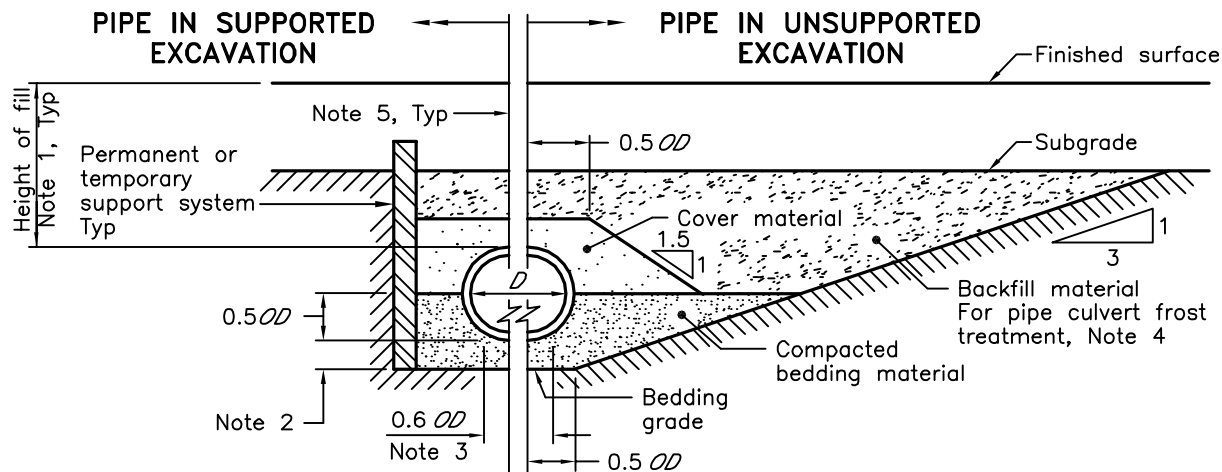
RIGID PIPE BEDDING,  
COVER, AND BACKFILL

TYPE 3 SOIL – EARTH EXCAVATION

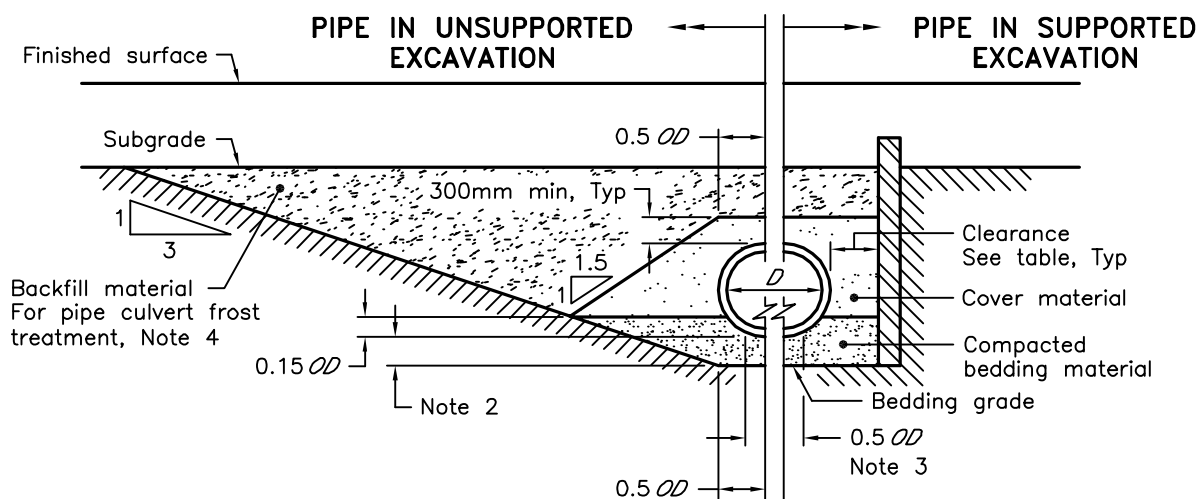
Nov 2010 Rev 2



OPSD 802.031



### CLASS B BEDDING



### CLASS C BEDDING

#### LEGEND:

$D$  – Inside diameter  
 $OD$  – Outside diameter

#### NOTES:

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

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

ONTARIO PROVINCIAL STANDARD DRAWING

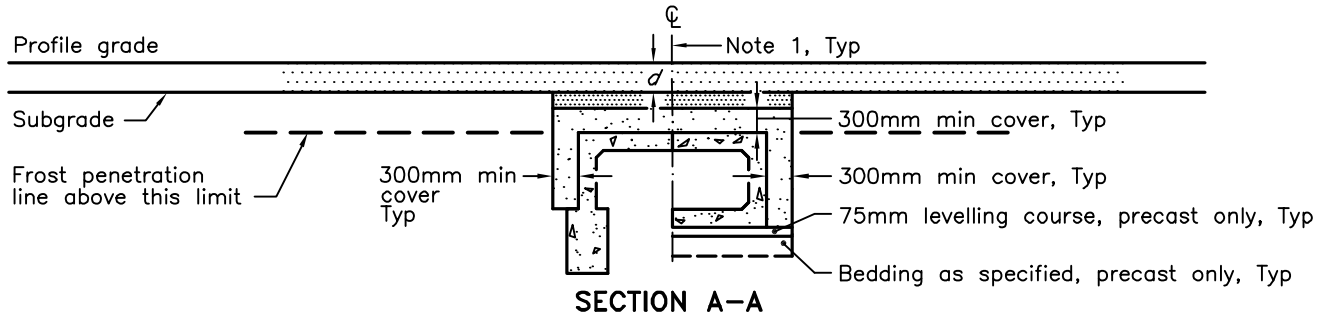
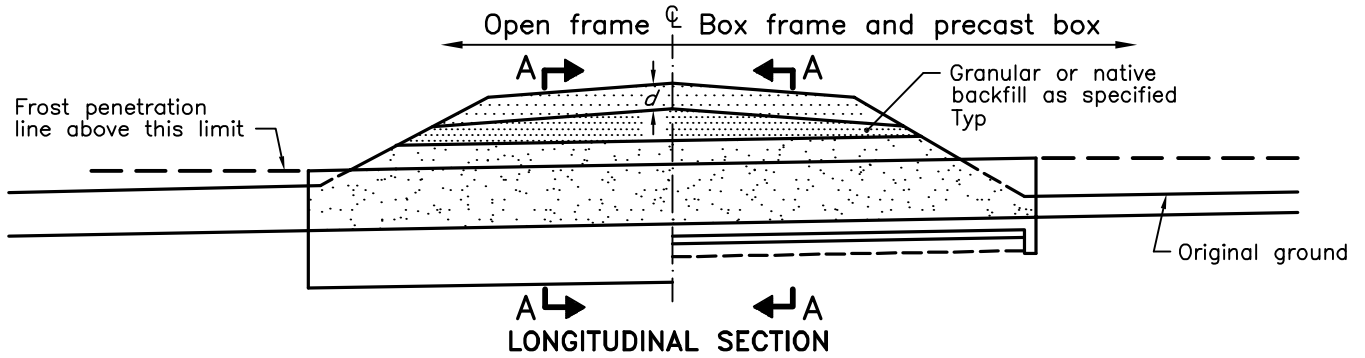
Nov 2010 Rev 2

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

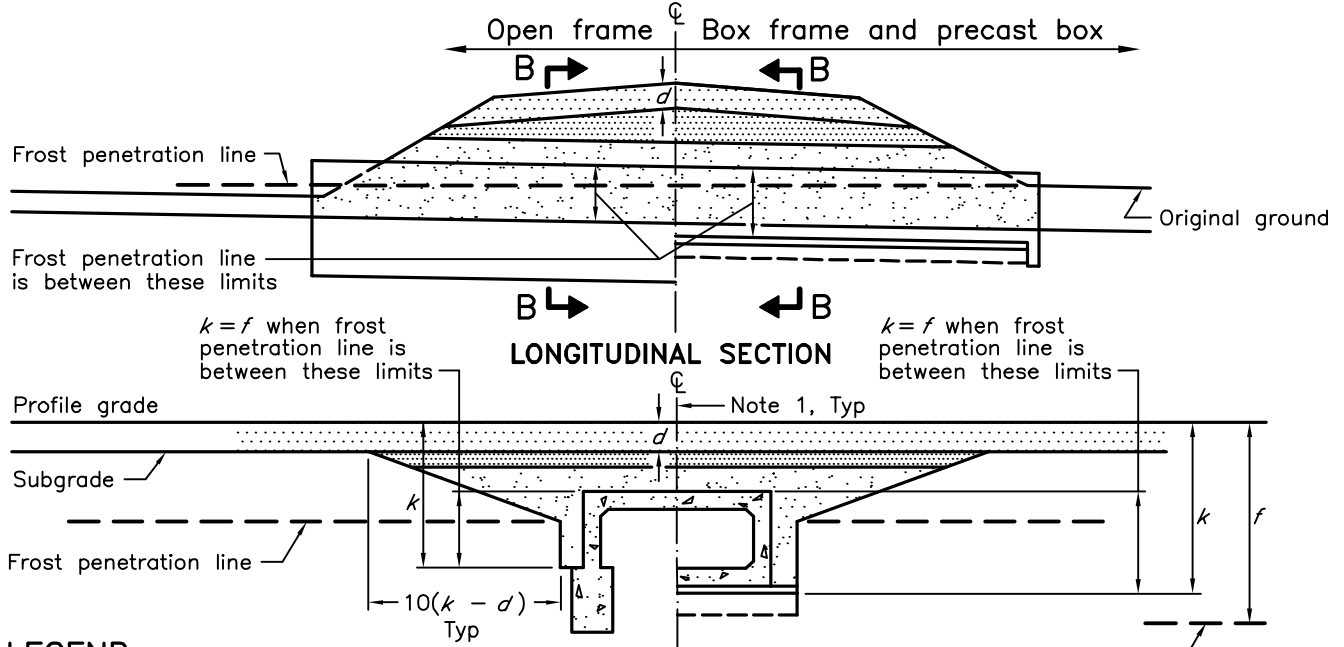
OPSD 802.032



## 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

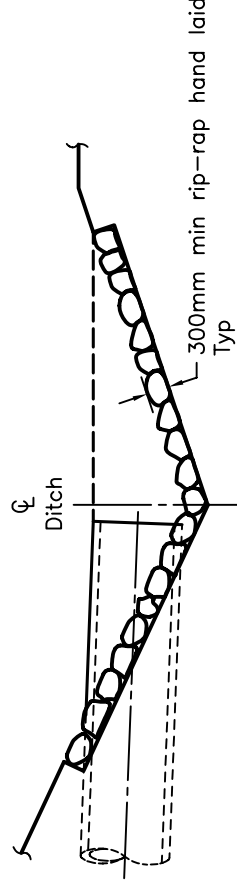
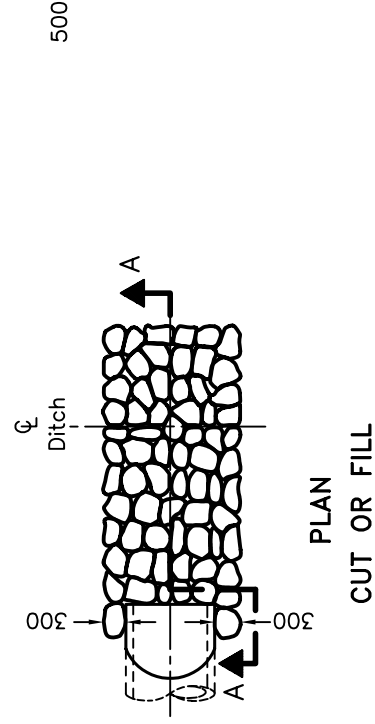
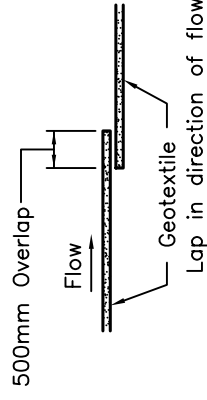
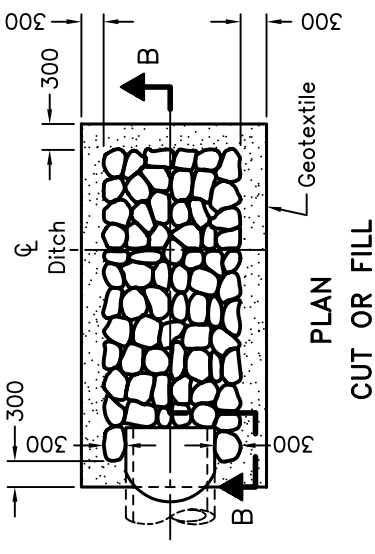
Nov 2015

Rev 3

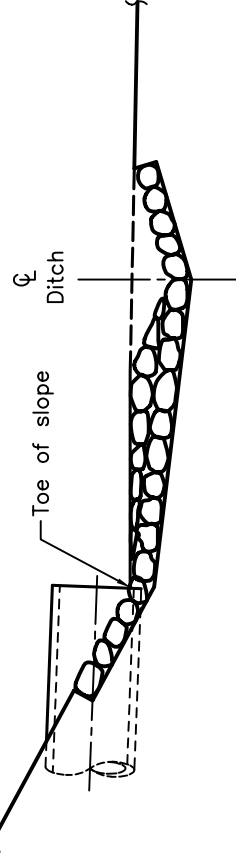
**BACKFILL AND COVER FOR  
CONCRETE CULVERTS WITH SPANS  
LESS THAN OR EQUAL TO 3.0M**

**OPSD 803.010**



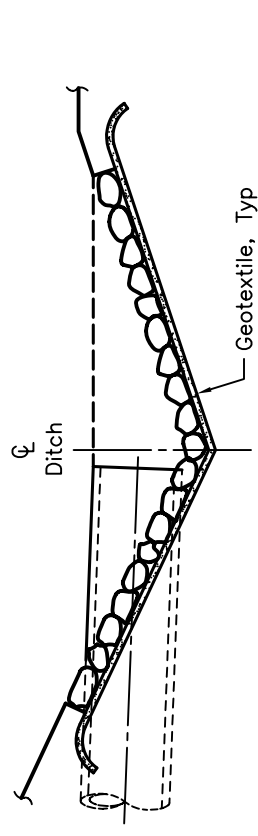


SECTION A-A CUT

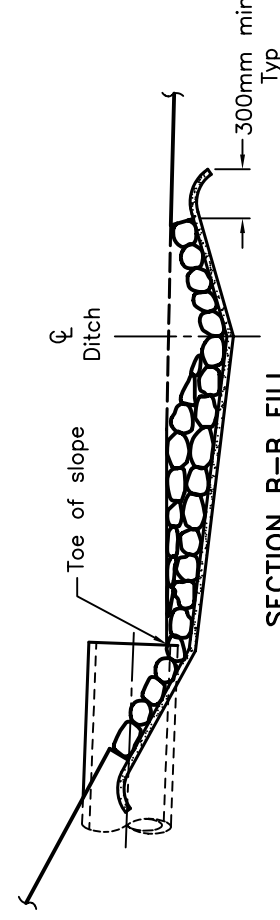


SECTION A-A FILL

TYPE A - WITHOUT GEOTEXTILE



SECTION B-B CUT



SECTION B-B FILL

TYPE B - WITH GEOTEXTILE

NOTES:

A All dimensions are in millimetres unless otherwise shown.

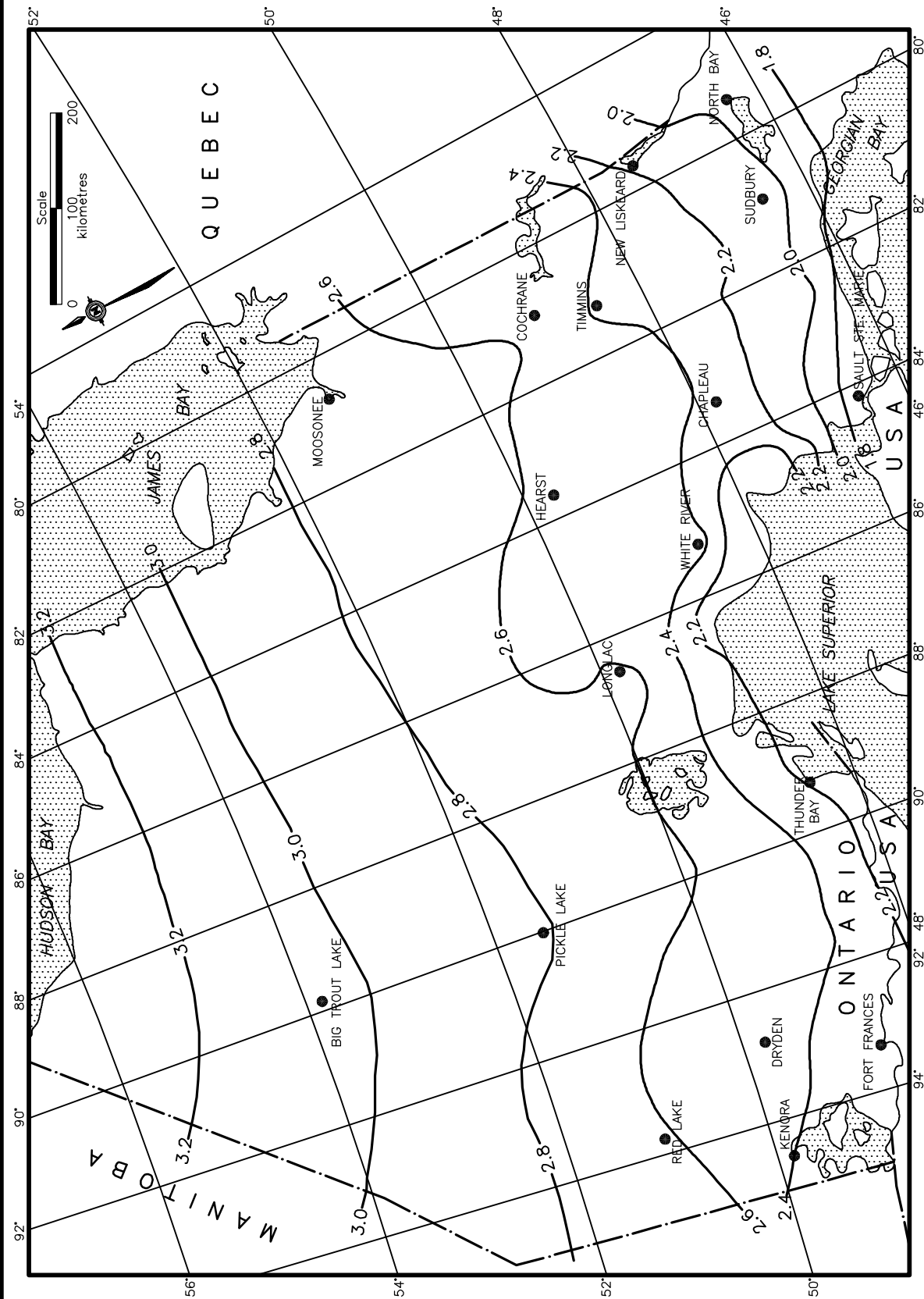
ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2001 Rev 0



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

OPSD - 810.010



# 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

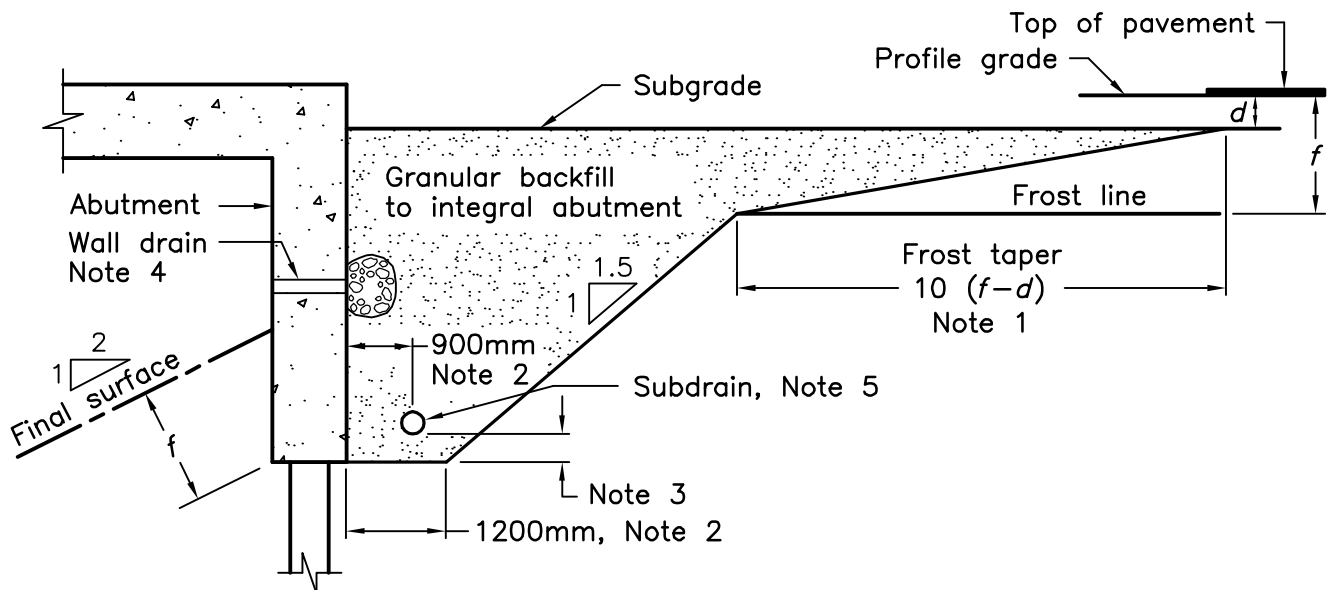
Nov 2010

Rev 1

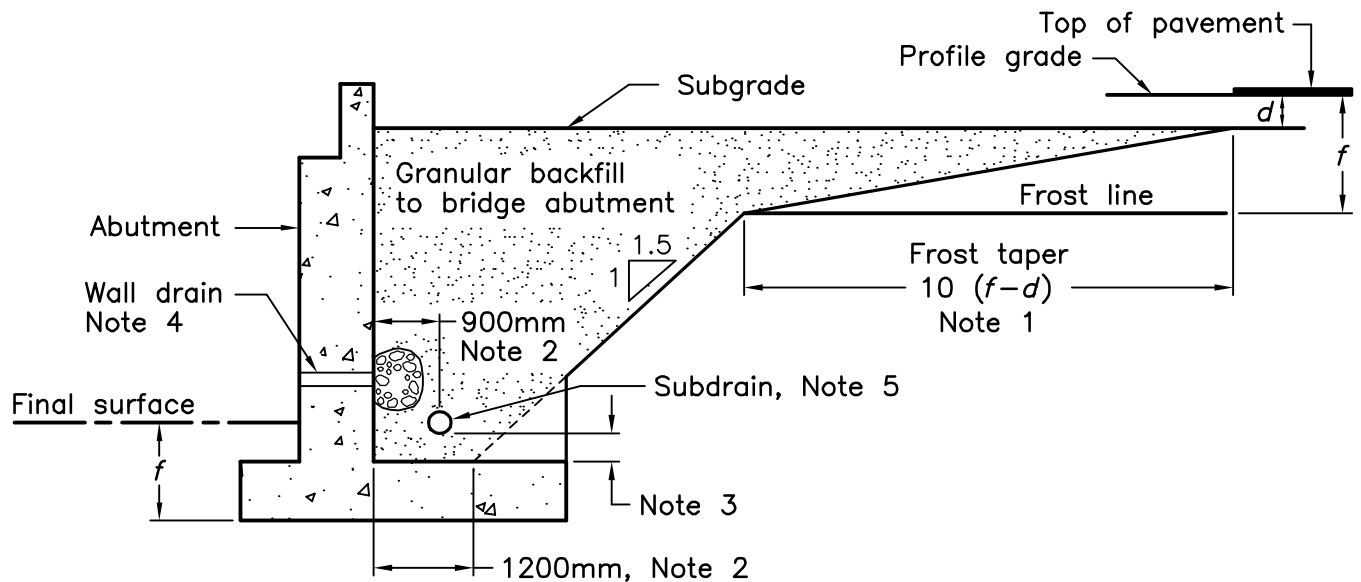
## FOUNDATION FROST PENETRATION DEPTHS FOR NORTHERN ONTARIO

OPSD 3090.100





### 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**



## Appendix G – Non-Standard Special Provisions (NSSP)



## **NSSP FOR COBBLES AND/OR BOULDERS OBSTRUCTIONS**

### **Scope of Work**

The Contractor should be aware that cobbles and/or boulders may be encountered during the installation of shoring elements and during excavations of the in-situ soils and embankment fill. Appropriate equipment and procedures will be required to penetrate/remove cobbles and/or boulders that may be encountered during installation of shoring and excavation,

### **Basis of Payment**

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