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Foundation Investigation and Design Report

Agreement No. 5016-E-0016
GWP 411-00-00
GEOCRES No. 31F-195

Culvert Replacement, Stn. 12+415 Highway 129, Reaney Township, District of Sudbury

Prepared For:
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The Ministry of Transportation

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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 non-structural centreline culvert located on Highway 129 at Station 12+415, 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 12+415 within Reaney Township, at the Vincent Creek crossing. The site is located approximately 32.6 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 1.5 m in diameter and approximately 31.45 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 4.5 m high on both sides of the roadway, having side slopes of approximately 2H:1V 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 Vincent Creek flows from the east to the west through the existing culvert. Immediately adjacent to the Creek, on both sides of the roadway embankment, the terrain generally consists of marshy, low lying vegetation and grasses, with a few deciduous and coniferous trees. Further away from the Creek, the terrain changes to thick forest consisting of both coniferous and deciduous trees.

At the inlet on the east side, the Creek extends perpendicular to the highway for approximately 30 m, before turning to the north-east. Rip-rap is present along the northern shore of the Creek, extending approximately 4 to 6 m from the culvert inlet. At the outlet on the west side, the Creek changes direction towards the south, running parallel to the highway along the toe of the highway embankment for approximately 40 to 60 m before changing direction to the west. At the time of the assessment, the Creek elevation was generally near the culvert spring line.

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 fair shape, with slight rutting and localized cracking. Immediately above the culvert, moderate to severe transverse and alligator 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 consists of glaciofluvial outwash composed of sand and gravel. Local relief is generally low (< 15 m) and the terrain consists of knobby plains. Overall drainage is good (dry). Subordinate landforms through this section consist of dry sandy eolian deposits and mixed wet and dry organic peat deposits.

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 to 7, 2016 and January 14, 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 southbound lane, as close as possible to crest of the western embankment. Boreholes BH-2 and BH-3 were advanced at accessible locations near the inlet and outlet, 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 contactor, Landcore Drilling. Each borehole was advanced to approximately 6.0 m depth below the invert of the existing culvert. Refusal was not encountered within the borings.

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. 12+425. The TBM was assigned an elevation of 452.3 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 on Figures 1 and 2 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 on Dwg. No. 1 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 cohesionless fill materials and native sands/silts. At the toes of the embankment slopes (BH-2 and BH-3), the subsurface conditions encountered consist of topsoil and peat overlying possible fill, organics, native sands/silts, and till. 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 50 mm thick. The asphalt thickness may vary beyond the borehole location.

1.4.2 Topsoil

Topsoil was encountered at the surface of Borehole BH-2 and was approximately 75 mm thick. Topsoil thickness may further vary beyond the borehole location.

1.4.3 Peat

Peat was encountered at the surface of Borehole BH-3 and was approximately 0.8 m thick. The peat was black in colour, fibrous, and moist to wet. One SPT performed within the peat resulted in an uncorrected “N” value of 2 blows per 300 mm, classifying the soil as very loose in compactness condition.

Laboratory testing performed on a sample of the soil consisted of one (1) moisture content test. The test results are as follows:

Moisture Content:

- 101 %

The result of the moisture content test is provided on the borehole log for BH-3 in Appendix C.

1.4.4 Cohesionless Fill Materials

Cohesionless fill materials were encountered below the asphalt at BH-1 and extended to approximately 7.5 m depth below existing grade. The fill material layers ranged in composition, including:

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

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

1.4.4.1 Gravelly Sand Fill

An approximately 1.5 m thick layer of gravelly sand fill was encountered below the asphalt at BH-1. The fill material was brown in colour, moist, and contained trace to some silt.

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

Moisture Content:

- 3 to 5 %

Grain Size Distribution:

- 26 to 28 % gravel
- 62 to 67 % sand
- 5 to 13 % fines

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

1.4.4.2 Sand and Silt Fill

Underlying the gravelly sand fill at 1.5 m depth at BH-1 was an approximately 1.6 m thick layer of sand and silt fill. The fill material was brown to grey in colour, moist, fine grained, and contained trace clay and trace gravel. Uncorrected SPT "N" values within the sand and silt fill ranged from 16 to 17 blows per 300 mm, classifying the soil as compact in compactness condition.

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

Moisture Content:

- 9 to 11 %

Grain Size Distribution:

- 4 % gravel
- 54 % sand
- 40 % silt
- 2 % clay

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

1.4.4.3 Sand and Gravel Fill

Underlying the sand and silt fill at BH-1 at 3.1 m depth was an approximately 0.7 m thick layer of sand and gravel fill. The fill material was brown in colour and moist. One SPT performed within the fill resulted in an uncorrected "N" value of 62 blows per 300 mm, classifying the fill as being very dense in compactness condition.

Laboratory testing performed on a sample of the sand and gravel fill consisted of one (1) moisture content test. The test results are as follows:

Moisture Content:

- 5 %

The result of the moisture content test is provided on the borehole log for BH-1 in Appendix C.

1.4.4.4 Sand Fill

Underlying the sand and gravel fill at BH-1 at 3.8 m depth was an approximately 3.7 m thick layer of sand fill. The fill material was brown in colour and moist, changing to grey and wet with depth. The sand fill was medium to coarse grained and contained trace silt, trace gravel, and trace clay. Uncorrected SPT "N" values within the upper 3.1 m of the fill ranged from 10 to 21 blows per 300 mm, classifying the sand fill as compact in compactness condition. Below 6.9 m depth, an uncorrected SPT "N" of 1 blow per 300 mm was obtained, classifying the fill below this depth as very loose in compactness condition.

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

Moisture Content:

- 3 to 24 %

Grain Size Distribution:

- 3 % gravel
- 88 % sand
- 8 % silt
- 1 % 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 results of the grain size distribution test is also provided on Figure 3 in Appendix D.

1.4.5 Gravel and Sand (Possible Fill)

Underlying the topsoil at Borehole BH-2 was an approximately 1.4 m thick layer of gravel and sand. This material may be considered possible fill due to organic materials encountered below this layer. The gravel and sand was brown in colour, wet, and contained some silt. Uncorrected SPT “N” values ranged from 23 to 31 blows per 300 mm, classifying the soil as compact to dense in compactness condition.

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

Moisture Content:

- 14 to 24 %

Grain Size Distribution:

- 45 % gravel
- 44 % sand
- 12 % fines

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

1.4.6 Organic Silt

Underlying the gravel and sand at BH-2 at 1.5 m depth was an approximately 1.6 m thick layer of organic silt. The organic silt was grey in colour, wet, and contained some sand and trace gravel. Uncorrected SPT “N” values within the soil ranged from 0 to 1 blow per 300 mm, classifying the soil as very loose in compactness condition.

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

Moisture Content:

- 115 to 205 %

The results of the moisture content tests are provided on the borehole log for BH-2 in Appendix C.

1.4.7 Silty Sand

Underlying the fill materials at BH-1 at 7.5 m depth, was an approximately 1.0 m thick layer of native silty sand. The silty sand was brown to black in colour, moist to wet, and contained some organics/wood, and trace gravel. An approximately 200 mm long piece of wood was recovered in the split spoon below 8.4 m depth. One SPT performed within the soil resulted in an uncorrected "N" value of 7 blows per 300 mm, classifying the soil as loose in compactness condition.

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

Moisture Content:

- 30 to 127 %

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

1.4.8 Sandy Silt

Underlying the silty sand at BH-1 at 8.5 m depth and the organic silt at BH-2 at 3.1 m depth, was a deposit of native sandy silt. The sandy silt extended to the borehole termination depths at BH-1 and BH-2 of 14.3 m and 7.5 m, respectively. The sandy silt was grey in colour, moist to wet, and contained trace to some gravel, and trace clay. Uncorrected SPT "N" values varied between the boreholes. At BH-1, uncorrected "N" values ranged from 16 to 43 blows per 300 mm, classifying the soil as compact to dense in compactness condition. At BH-2, uncorrected "N" values ranged from 1 to 10 blows per 300 mm, classifying the soil as being very loose to loose in compactness condition. At BH-1, heaving soil were encountered within the casing at approximately 13.7 m depth.

Laboratory testing performed on selected samples consisted of seven (7) moisture content tests and two (2) grain size analyses. The test results are as follows:

Moisture Content:

- 18 to 42 %

Grain Size Distribution:

- 0 to 4 % gravel
- 20 to 32 % sand
- 64 to 69 % silt
- 4 to 7 % clay

The results of the moisture content and grain size distribution tests are provided on the borehole logs for BH-1 and BH-2 in Appendix C. The results of the grain size distribution tests are also provided on Figure 5 in Appendix D.

1.4.9 Silt

Underlying the peat at BH-3 at 0.8 m depth was native silt, which extended to approximately 5.3 m depth below existing grade. The silt was black to grey in colour, changing to grey below 1.5 m depth, and wet. The silt contained some peat in the upper 0.7 m, as well as some sand, trace to some gravel, and trace clay. Uncorrected SPT "N" values ranged from 7 to 14 blows per 300 mm, classifying the soil as loose to compact in compactness condition.

Laboratory testing performed on selected samples consisted of six (6) moisture content tests and two (2) grain size analyses. The test results are as follows:

Moisture Content:

- 18 to 26 %

Grain Size Distribution:

- 0 to 1 % gravel
- 16 to 19 % sand
- 74 to 80 % silt
- 4 to 6 % clay

The results of the moisture content and grain size distribution tests are provided on the borehole log for BH-3 in Appendix C. The results of the grain size distribution tests are also provided on Figure 6 in Appendix D.

1.4.10 Silty Gravel and Sand Till

Underlying the silt at BH-3 at 5.3 m depth and extending to the borehole termination depth of 6.7 m, was a native silty gravel and sand till. The till was grey in colour, wet, coarse grained, and contained trace clay. Uncorrected SPT "N" values within the till ranged from 14 to 18 blows per 300 mm, classifying the soil as compact in compactness condition.

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

Moisture Content:

- 9 to 11 %

Grain Size Distribution:

- 39 % gravel
- 32 % sand
- 29 % silt
- 1 % clay

The results of the moisture content and grain size distribution tests are provided on the borehole log for BH-3 in Appendix C. The results of the grain size distribution test is also provided on Figure 7 in Appendix D.

1.5 Groundwater and Surface Water Conditions

Groundwater was observed in Borehole BH-1 upon completion at approximately 5.4 m depth, Elev. 446.8 m. For BH-2 and BH-3, washboring techniques were performed for the portable equipment utilized, which required water to be pumped into the borehole. As such, accurate groundwater measurements could no be obtained in these boreholes upon completion. Note, however, that samples within these boreholes were wet near surface, which would infer a groundwater elevation at or near surface in BH-2 and BH-3, which is close to the prevailing creek level.

The water level within the Creek was measured at the time of the investigation (January 2017) and it was at approximately Elev. 447.3 m at the culvert location. This is generally at the same level as the groundwater encountered within BH-1. This would also further support the inference above that groundwater is near surface at BH-2 and BH-3.

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

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 12+415, 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 interpretation 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 1.5 m diameter CSP, which is approximately 31.45 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 4.5 m high on both sides of the roadway, having side slopes of approximately 2H:1V from the top to toe of the embankment. The current elevation of the crest of the roadway is approximately 452.2 m.

- The highway embankment consists of approximately 7.0 m of compact to dense granular fill underlain by approximately 0.5 m of wet, very loose granular fill. The embankment fill is underlain by approximately 1.0 m of loose native silty sand, mixed with some organics and wood, followed by compact to dense native sandy silt.

- At the existing culvert inlet, approximately 1.5 m of dense to compact granular soil (possible fill) was encountered overlying approximately 1.6 m of very loose native organic silt. Underlying the organic silt layer is native very loose to loose sandy silt. At the outlet, approximately 0.8 m of peat was encountered overlying approximately 4.5 m of native loose to compact silt. Below the silt was a compact silty gravel and sand till.

- The water level within the Creek measured at the time of the investigation (January 2017) was at approximately Elev. 447.3 m at the culvert inlet. The groundwater level observed within Borehole BH-1 was slightly below the creek level at approximately Elev. 446.8 m.

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 1.5 m × 31.45 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 cofferdam as discussed in Section 2.8 below.

The new culvert founding level is expected to be similar to the current level (approx. Elev. 446.5 m). As very loose fill materials and organics/wood were encountered below the current founding level between Elev. 445.3 m and 443.7 m, it is recommended that all very loose fill, organics/wood, or other deleterious materials be removed from below the proposed culvert down to the compact to dense native silty sand soils. The grade should then be restored with engineered fill.

Removing the very loose fill and organics/wood from below the proposed culvert will likely result in very deep excavations upwards of 8.5 m (or 2.8 m below the existing culvert). If this is considered excessive or uneconomical, consideration may be given to leaving these materials in place. If the very loose fill and organics/wood 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. A non-woven geotextile fabric should surround the entire fill pad to mitigate the migration of fines in the engineered fill. The new culvert must also be designed such that there is 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 fill materials and organics/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 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	• Straightforward construction • Reduced construction period, consequently traffic management and water control period reduced • Can be more readily installed during cold weather conditions • Longer service life than steel	• If floor is thin or poorly reinforced, it may heave and crack • During high flows, the concrete floor can be undermined • Susceptible to defects/leakage at joints	• Low	• Risk of unacceptable differential settlements if the entire foundation is not supported on competent soil • Risk of leaking from joints if not properly installed
Cast-in-Place Rigid Frame Concrete Box Culvert	3	• Suitable if site is not conducive to heavy equipment for installation of precast sections • Culvert design can be customized in the field for high stress or load conditions or other site specific requirements • Longer service life than steel	• Slower construction process • If floor is thin or poorly reinforced, it may heave and crack • During high flows, the concrete floor can be undermined • Requires concrete curing	• Low to medium	• Risk of unacceptable settlements if the entire foundation is not supported on competent soil • Risk of disturbance of base during construction
Corrugated Steel Pipe (CSP) Culvert	2	• Straightforward construction • Reduced construction period, consequently traffic management and water control period reduced	• Limited service life • Potential for corrosion	• Low to medium	• Risk of unacceptable settlements if the entire foundation is not supported on competent soil • Risk of structure segment loss due to corrosion

Cast-in-Place Rigid Frame Open Footing Concrete Culvert Supported on Shallow Foundations	4	• Wider span may be used to maintain existing channel and allows for natural streambed to remain intact • Less accumulation of sediments upstream of the culvert • Longer service life than steel	• Slower construction process • Deeper excavation likely required as footings need to be below frost line • Requires concrete curing	• Medium	• Risk of unacceptable settlements if the entire foundation is not supported on competent soil • High Scour Risk

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 Spread Footing Design Parameters

Culvert Type	Founding Elevation (m)	Assumed Footing Size (m)	Founding Soil Type	Factored Geotechnical Resistance at ULS (kPa)	Geotechnical Reaction at SLS* (kPa)
Rigid frame box culvert Concrete Pipe Culvert Or CSP Pipe Culvert	~ 446.5 m or below	1.5 m	~ 2.8 m compacted granular material (Granular “A” and/or “B” Type II) over native sandy silt/silty sand	450	300
	~ 446.5 m or below	1.5 m	~ 0.5 m compacted granular material (Granular “A” and/or “B” Type II) over very loose fill and/or organics/wood	300	200
Cast-in-Place Open Footing Concrete Culvert	~ 444.0 m (below frost line)	1.0 m	Native sandy silt /silty sand	450	300

*- For Maximum Settlement of 25 mm

Where the very loose soils and organics/wood are not removed from below the proposed culvert, the new culvert must also be designed such that there is no net increase in bearing pressure beyond the existing conditions. This will mitigate any significant settlement of the underlying very loose fill materials and organics/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 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 δ) = 0.5
Between Granular "A" and cast-in-place concrete	Coefficient of Friction (tan δ) = 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:

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

where
p = Lateral earth pressure (kPa)
K = Coefficient of earth pressure
γ = Unit weight of backfill (kN/m^3)
γ_w = Unit weight of water (kN/m^3)
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 ø´ (unfactored)**	**Coefficient of Active Earth Pressure (k_a)**	**Coefficient of Passive Earth Pressure (k_p)**	**Coefficient of Earth Pressure at Rest (k_o)**	**Unit Weight γ (kN/m^3)**
Granular “A” (compact)	35º	0.24	4.2	0.38	22.8
Granular “B” Type I (compact)	32º	0.27	3.7	0.43	21.2
Granular “B” Type II (compact)	35º	0.24	4.2	0.38	22
Gravelly Sand Fill, Sand and Gravel/Gravel and Sand Fill (compact to very dense)	32º	0.27	3.7	0.43	20
Sand and Silt Fill (compact)	30º	0.33	3.0	0.50	20
Sand Fill (very loose to compact)	30º	0.31	3.3	0.47	20
Sandy Silt/Silty Sand (very loose to loose)	28º	0.36	2.8	0.53	19
Sandy Silt (compact to dense)	30º	0.33	3.0	0.50	20
Silt (very loose to compact)	28º	0.36	2.8	0.53	18
Organic Silt (very loose)	25º	0.41	2.5	0.58	17
Silty Gravel and Sand Till (compact)	35º	0.27	3.7	0.43	21

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 CHDBC with respect to soil conditions encountered at the site. Table 4.1 in CHBDC (see Clause 4.4.3.2) shows site classification for seismic site response based on soil average properties in top 30 m. The borehole information shows the presence of generally loose to compact soil with no bedrock encountered at investigated depth. 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 (47.518°N, 83.2087°W) and the damped reference spectral accelerations for the project site are Sa(0.2)=0.029g, Sa(0.5)=0.023g, Sa(1.0)=0.013g, Sa(2.0)=0.0058g and the reference peak ground acceleration (PGA) is 0.015g (g=acceleration due to gravity - 9.81 m/s^2). These values are associated with an earthquake having 10 percent probability of exceedance in a 50-year period.

Based on soils and groundwater condition 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
3. Trenchless installation methods to avoid/minimize any disruption to traffic and to avoid the need to excavate the embankment. The following trenchless installation methods were considered:
    a. Jack and Bore
    b. Horizontal Directional Drilling
    c. Pipe ramming

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. Method 3 utilizes trenchless methods which will not require excavation of the existing embankment, or disruption of traffic (or very minimal disruption to move equipment in place). With trenchless methods, it may be difficult to replace the culvert at the same location and the existing culvert may or may not be removed, depending on the approach. 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	• Existing culvert will be completely removed and replaced with new culvert • No detour road construction or roadway protection required • No excavation support required • Install entire new culvert at once • Straightforward construction • Short construction period • Low capital investment; cost savings in time and materials required for construction	• Traffic interruption • No local detour available, only long distance detours available • Large amount of soil to be excavated • Excavations will be large with likely 1H:1V sideslopes • Need to temporarily control existing creek water and groundwater • Potential claims to compensate vehicle occupants and local businesses for delays or time lost due to long detours	• Relatively less expensive than other methods due to cost savings in time and materials required for construction • Potential costs associated with claims to compensate vehicle occupants and local businesses for delays or time lost due to long detours • Low risk of cost overruns	3.

Installation Method	Advantages	Disadvantages	Relative Cost	Ranking
1.b. Temporary detour embankments and open cut/unsupported excavation	• One to two lanes of traffic flow maintained at site during construction. • Existing culvert will be completely removed and replaced with new culvert • No excavation support required • Install entire new culvert at once	• Traffic interruption • Construction of detour embankments required on one side of highway • Difficulties to construct detours due to accessibility of surrounding terrain. • Increased time of construction due to detour • Large amount of soil to be excavated • Excavations will be large with likely 1H:1V sideslopes • Need to temporarily control existing creek water and groundwater • Possible settlement due to new earth fill embankment • Temporary detour will need to be decommissioned	• Higher cost than full road closure due to high costs associated with temporary detour embankment construction • Possible costs associated with purchasing private property if detour extends beyond current ROW • Moderate risk of cost overrun due to complexity of constructing detour embankment	7.

Installation Method	Advantages	Disadvantages	Relative Cost	Ranking
2.a. Half-and-half construction using roadway protection with unsupported cut sides	• One lane of traffic flow maintained during construction • Straightforward construction	• Traffic interruption • Roadway protection required to maintain one lane of traffic • High cost of roadway protection system • Large amount of soil to be excavated • Culvert excavations will be large with likely 1H:1V sideslopes • Need to temporarily control existing creek water and groundwater • Narrow highway; may require temporary widening for open traffic lane	• More expensive than road closure due to high costs of roadway protection system • Moderate risk of cost overrun due to complexity of roadway protection system	1.

Installation Method	Advantages	Disadvantages	Relative Cost	Ranking
2.b. Half-and-half construction using roadway protection with braced cut sides	• One lane of traffic flow maintained during construction • Narrow excavation for culvert • Less soil excavation and fill placement • Less roadway protection required due to small culvert excavation	• Traffic interruption • Roadway protection required to maintain one lane of traffic • Requires side shoring/bracing for culvert excavation • High cost of roadway protection system and side shoring • Bracing may interfere with culvert removal/placement • Shoring system will need to be decommissioned • Need to temporarily control existing creek water and groundwater • Narrow highway; may require temporary widening for open traffic lane	• More expensive than road closure due to high costs of roadway protection system • More expensive than Option 2.a. due to additional shoring for braced excavation for culvert • Moderate risk of cost overrun due to complexity of roadway protection system	2.

Installation Method	Advantages	Disadvantages	Relative Cost	Ranking
3.a. Jack and bore trenchless installation	• No disruption to traffic during installation • No need for temporary detour • Can likely be installed adjacent to existing culvert. Existing culvert can be utilized to maintain flow during construction. • Excavation of the existing embankment not required • Soil conditions within and under embankment are favourable for this type of installation	• Potential settlement of roadway as boring is typically larger than casing • Need to temporarily control existing creek water and groundwater • Specialized equipment required • May not be possible with large size culverts • Need to excavate entrance and exit pits, which may require shoring depending on required depth and location • Non-steerable installation, therefore minimal control of alignment • Existing culvert must be decommissioned • May limit the type and size of culvert that can be utilized • May be difficult to replace culvert at same location	• More expensive than road closure due to specialized equipment and construction procedures • Moderate risk of cost overrun due to complexity of installation system	5.

Installation Method	Advantages	Disadvantages	Relative Cost	Ranking
3.b. Horizontal directional drilling trenchless installation	• No disruption to traffic during installation • No need for temporary detour • Can likely be installed adjacent to existing culvert. Existing culvert can be utilized to maintain flow during construction. • Excavation of the existing embankment not required • Steerable installation method ensures accuracy of culvert placement • Soil conditions within and under embankment are favourable for this type of installation	• Potential settlement of roadway as boring is typically larger than casing • Need to temporarily control existing creek water and groundwater • Specialized equipment required • May not be possible with large size culverts • Need to excavate entrance and exit pits, which may require shoring depending on required depth and location • Existing culvert must be decommissioned • May limit the type and size of culvert that can be utilized • May be difficult to replace culvert at same location	• More expensive than road closure due to specialized equipment and construction procedures • Moderate risk of cost overrun due to complexity of installation system	6.

Installation Method	Advantages	Disadvantages	Relative Cost	Ranking
3.c. Pipe ramming trenchless installation	• No disruption to traffic during installation • No need for temporary detour • Can likely be installed adjacent to existing culvert. Existing culvert can be utilized to maintain flow during construction. • Excavation of the existing embankment not required • Little to no settlement of roadway as continuous casing support provided, and spoils materials removed likely after installation • Likely least expensive of trenchless methods • Soil conditions within and under embankment are favourable for this type of installation	• Need to temporarily control existing creek water and groundwater • Specialized equipment required • Need to excavate entrance and exit pits, which may require shoring depending on required depth and location • Potential for heave if blockage encountered • Non-steerable installation, therefore minimal control of alignment • Existing culvert must be decommissioned • May limit the type and size of culvert that can be utilized • May be difficult to replace culvert at same location	• More expensive than road closure due to specialized equipment and construction procedures • Likely least expensive of the trenchless methods • Moderate risk of cost overrun due to complexity of installation system	4.

Based on the above list of advantages and disadvantages of the possible construction methods, from a foundations perspective, 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.a. – Full Road Closure Followed by Open Cut/Unsupported Excavation
4. Option 3.c. – Pipe Ramming
5. Option 3.a. – Jack and Bore
6. Option 3.b. – Horizontal Directional Drilling (HDD)
7. Option 1.b. – Temporary Detour Construction 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.

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 runs generally parallel to the embankment on the west side, it is likely that the east side is the only 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 and in-situ organics below the embankment would be up to approximately 8.5 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 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 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 solder pile and lagging with rakers or anchors for horizontal support. Where the cut extends below prevailing groundwater, a suitable control/system is required. Once one lane is completed, the supports can be reversed and the other lane constructed in similar fashion. The shoring system would likely be decommissioned in place. 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 cut shoring system perpendicular to the highway for face protection and to allow culvert construction is 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 been given to how the new culvert sections will be installed given the relatively narrow work area and potential for obstructions from the lateral bracing using struts. Installation of tiebacks could be the solution. Temporary decking could possibly be used over the supported cut to allow for excavation of both halves prior to diverting stream and backfilling. However, decking would be costly. 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.

2.6.3 Trenchless Installation Methods (Options 3.a., 3.b., and 3.c.)

Trenchless installation methods could be utilized to maintain the flow of the traffic on Hwy 129 and avoid the need to excavate the existing embankment. For these methods, it is likely that the new culvert will be installed adjacent to the existing culvert on the south side. The will allow the existing culvert to be utilized to maintain flow of the creek during installation.

With all trenchless installation methods, entry and exit pits will need to be constructed. Depending on the depth of pits required, temporary shoring may be necessary. It will be the Contractors 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. Using the trenchless construction approach, three methods of culvert replacement were considered suitable for this site and anticipated pipe size as follows:

a. Jack and Bore
b. Horizontal Directional Drilling
c. Pipe Ramming

Option 3.c. is likely the most economical of the trenchless methods, and will likely be the least disruptive to the existing highway embankment. Options 3.a. and 3.b. are likely more complex as the construction operations are multi-stage processes. In addition, the potential for embankment settlements are higher for these options due to the installation processes. All three 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 the entry and exit pits. In addition, each option will require decommissioning of any shoring system upon completion of the work. The existing culvert will need to be decommissioned and likely grouted upon completion of the new culvert installation.

2.6.3.1 Option 3.a. – Jack and Bore

Jack and Bore, also known as auger boring, is a multi-stage process using a non-steerable auger boring machine with a temporary horizontal jacking platform and starting alignment track located at a desired elevation in an entrance pit, located on the side of the embankment. The steel casing is advanced by jacking the casing by manual control along the alignment track with simultaneous excavation of the soil being accomplished by a rotating cutting head installed in the leading edge of the pipe. The ground up soil (spoil) is transported back to the entrance pit by helical wound auger flights rotating inside the casing pipe. Auger boring machines are typically available in sizes of 900 mm to 1200 mm diameter.

The major problem associated with auger boring is the potential for subsidence or settlement of the ground above the auger hole as the cutter of the leading face of the augers generally excavates a borehole larger than the size of the following casing; therefore, settlement is a major concern, especially for the cohesionless soils within the existing embankment.

2.6.3.2 Option 3.b. – Horizontal Directional Drilling (HDD)

Horizontal Directional Drilling utilizes steerable mechanical cutting devices using drill bits or high pressure jets that drill by fluid cutting. The direction/alignment of the drilling head can be controlled remotely with tracking equipment. HDD is typically a 3-stage process involving the initial drilling of a small diameter pilot hole along the desired centerline of the culvert or pipeline. The second stage enlarges the pilot hole to the desired diameter using a reamer device pulled back through the pilot hole. The third stage involves pulling the pipe back through the drilled hole. HDD can drill lengths up to about 1500 m for pipe diameters typically ranging between 100 mm to 1500 mm, and with the steering capability, installations can be performed in a shallow arc.

2.6.3.3 Option 3.c. – Pipe Ramming

Pipe Ramming installs steel casings by utilizing the energy from a pneumatic percussion hammer attached to the end of the pipe. Continuous casing support is provided and over-excavation or drilling water is not required. Spoil is removed from inside the pipe typically after the casing has reached its destination.

Pipe ramming is a non-steerable trenchless construction technique used primarily in near horizontal applications. Installation accuracy (vertical and horizontal) is usually about +/- 1% of the length of the pipe, but subsurface obstructions or improperly aligned pipes may result in significant deviations from the desired line and grade. In an open-end application, the leading face of the culvert pipe is fitted with a cutting edge to reduce the resistance of the pipe as it moves through the ground. The spoil materials enter the cavity of the pipe as forward movement progresses. Depending on the length of pipe and impact energy provided by the ram, the spoil can be left in the pipe until the pipe penetrates through to the opposite end of the embankment or the spoil can be removed through auger boring, compressed air or pressurized water. This method could be carried out with insignificant settlement of the overlying soils.

A potential drawback is the possibility of significant soil disturbance and the potential for heave at the surface if a blockage is created at the end of the installed pipe below the travelled highway. Vibrations from the pipe hammering may also induce settlement of loose materials in the immediate vicinity of the installation, which could cause deformation at the ground surface.

This method of trenchless installation is likely the most cost effective. However, the vibration from the hammer impact may cause some settlement in any loose materials in the embankment resulting in unacceptable settlement of the overlying pavement. As the culvert site is relatively remote, the impact sound & vibration from the hammering procedure will not be a disturbance to the public.

2.7 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 may be classified as a Type 3 soil above the groundwater table in conformance with the OHSA. The soils below the groundwater table may be classified as a Type 4 soil. Temporary excavation side slopes for Type 3 soil should not exceed 1H:1V in accordance with OHSA. Temporary excavation side slopes for Type 4 soils should not exceed 3H:1V where applicable. There is a potential for sloughing to occur if the trench remains open for an extended period of time (i.e. > 24 hours) or during a rainfall event. In addition, some localized surficial sloughing may be experienced in areas of perched groundwater seepage (i.e. within the embankment fill).

2.8 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, or sheet piles may be considered for design. It 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.

2.9 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 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. MOE, 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.

2.10 Culvert Bedding

It is recommended that all fill materials, organics, and other deleterious materials be removed down to competent native soils prior to placement of the bedding material. Prior to placing any fill material, the exposed native subgrade should be inspected in accordance with OPSS 902. A non-woven geotextile separator is to be placed between the approved subgrade and the compacted fill to assist in material placement and maintain the integrity of the founding soil along the entire length of the culvert. The geotextile separator is to be a Class II non-woven material with an equivalent opening size of 75-150 µm.

As organic materials extend to a depth of approximately 8.5 m (~ Elev. 443.7 m), upfill will likely be required below the bedding material to install the culvert at a similar invert level as existing (~ Elev. 446.5 m). Upfill below the bedding should consist of Granular “B” Type II (OPSS.PROV 1010).

If excavations are considered excessive or uneconomical to remove all organic materials from below the proposed culvert, consideration may be given to leaving these materials in place. If the very loose fill and organics/wood are left in place, an engineered fill pad should be constructed below the proposed bedding material. The engineered fill pad should be a minimum of 500 mm thick, and consist of 19.0 mm Type II clear stone gravel (OPSS.PROV 1004), Granular “A” or Granular “B” Type II (OPSS.PROV 1010). 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 maintain the integrity of the founding soil along the entire length of the culvert. The geotextile separator is to be a Class II non-woven material with an equivalent opening size of 75-150 μm.

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.

2.11 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.

2.12 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 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 will likely consist of sandy silt. This material has a moderate to high frost susceptibility based upon the MTO Frost Classification guideline of percent particles between 5 to 75 µm. Cold air blowing through the culvert during winter months can freeze this material.

If excavations extend below the proposed culvert invert to remove in-situ organic materials, non-frost susceptible engineered upfill will be placed below the culvert in addition to the culvert bedding materials. Provided the thickness of the non-frost susceptible upfill and bedding materials below the culvert exceeds 2.4 m (depth of frost penetration), frost protection below any of the culvert options would not be a concern.

For box or pipe culvert, if the upfill does not exceed, 2.4 m, 300 to 500 mm of non-frost susceptible engineered fill bedding and cover will still be placed below and around the culvert which should prevent the soils from freezing next to the culvert.

For open footing culverts, if the upfill does not exceed 2.4 m, consideration may be given to utilizing insulation below the footings prevent freezing of the underlying soils. Installation details for insulation should be developed in consultation with the insulation manufacturer based on final bedding/upfill thicknesses.

2.13 Embankment Design

2.13.1 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 include the following. 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 – Final Embankment Stability – Organics Not Removed Below Culvert – Inlet Side
- Figure E-4 – Final Embankment Stability – Organics Not Removed Below Culvert – Outlet Side
- Figure E-5 – Final Embankment Stability – Organics Removed Below Culvert – Inlet Side
- Figure E-6 – Final Embankment Stability – Organics Removed Below Culvert – Outlet Side
- Figure E-7 – Temporary Detour Embankment Stability – Organics Not Removed Below Culvert - Inlet Side, West Embankment Analysis
- Figure E-8 – Temporary Detour Embankment Stability – Organics Not Removed Below Culvert - Inlet Side, East Embankment Analysis
- Figure E-9 – Temporary Detour Embankment Stability – Organics Removed Below Culvert - Inlet Side, West Embankment Analysis

For the proposed final embankments and temporary detour embankments, sideslopes of 2H:1V were 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**		
	ø´	**c' (kPa)**	**γ (kN/m³)**
Granular "B" Type I	32º	0	21.2
Gravelly Sand Fill, Sand and Gravel/Gravel and Sand Fill (compact to very dense)	32º	0	20
Sand and Silt Fill (compact)	30º	0	20
Sand Fill (very loose to compact)	30º	0	20
Sandy Silt/Silty Sand (very loose to loose)	28º	0	19
Sandy Silt (compact to dense)	30º	0	20
Silt (very loose to compact)	28º	0	18
Organic Silt (very loose)	25º	0	17
Silty Gravel and Sand Till (compact)	35º	0	21
Peat (very loose)	5º	0	15

The results of the slope stability analyses performed are shown on Table 2-7 below. A minimum Factor of Safety of 1.3 is required to indicate that the embankment is stable. As shown on Table 2-7, each analysis resulted in a Factor of Safety greater than 1.3, which indicates that the embankments would be stable for long term conditions. Note on Figures E-7 and E-9 that the resulting Factors of Safety are only slightly higher than 1.3 (1.373 and 1.361, respectively). However, as the detour embankment is only temporary, this is considered acceptable.

Table 2-7: Summary of Slope Stability Analysis Results

Figure No.	Analysis	Factor of Safety
E-1	Existing Embankment – Inlet Side	1.799
E-2	Existing Embankment Stability – Outlet Side	1.733
E-3	Final Embankment Stability – Organics Not Removed Below Culvert – Inlet Side	1.845
E-4	Final Embankment Stability – Organics Not Removed Below Culvert – Outlet Side	1.879
E-5	Final Embankment Stability – Organics Removed Below Culvert – Inlet Side	1.928
E-6	Final Embankment Stability – Organics Removed Below Culvert – Outlet Side	1.965
E-7	Temporary Detour Embankment Stability – Organics Not Removed Below Culvert - Inlet Side, West Embankment Analysis	1.373
E-8	Temporary Detour Embankment Stability – Organics Not Removed Below Culvert - Inlet Side, East Embankment Analysis	1.955
E-9	Temporary Detour Embankment Stability – Organics Removed Below Culvert - Inlet Side, West Embankment Analysis	1.361

2.13.2 Embankment Settlement

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

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 µm. 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 1×10^{-5} 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 x (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

A compact silty gravel and sand till was encountered at 5.3 m depth at borehole BH-3. 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 for 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,

exp Services Inc.

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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.

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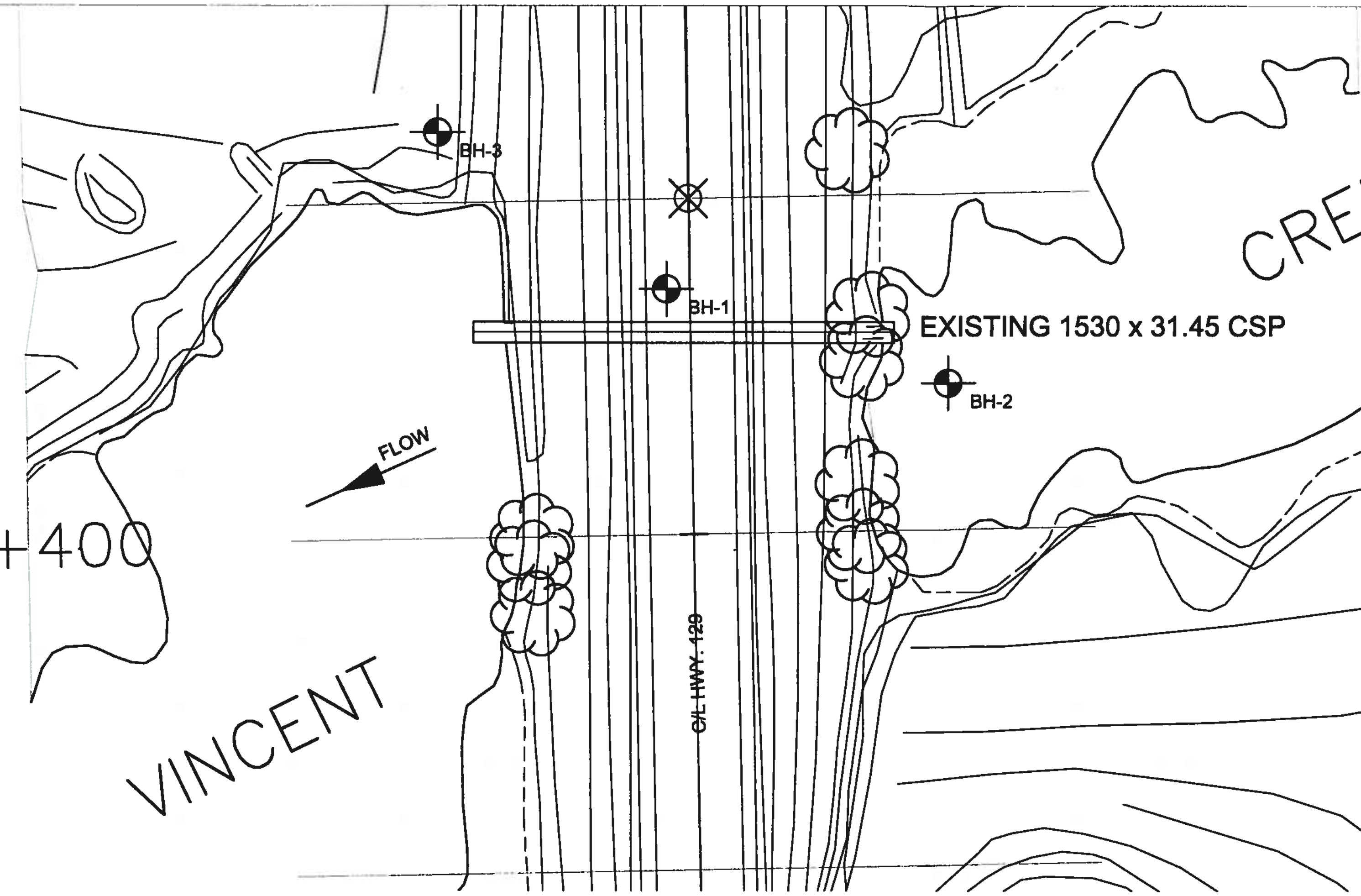
The information and opinions expressed in the Report, or any document forming part of the Report, are for the sole benefit of the Client. No other party may use or rely upon the Report in whole or in part without the written consent of **exp**. Any use of the Report, or any portion of the Report, by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

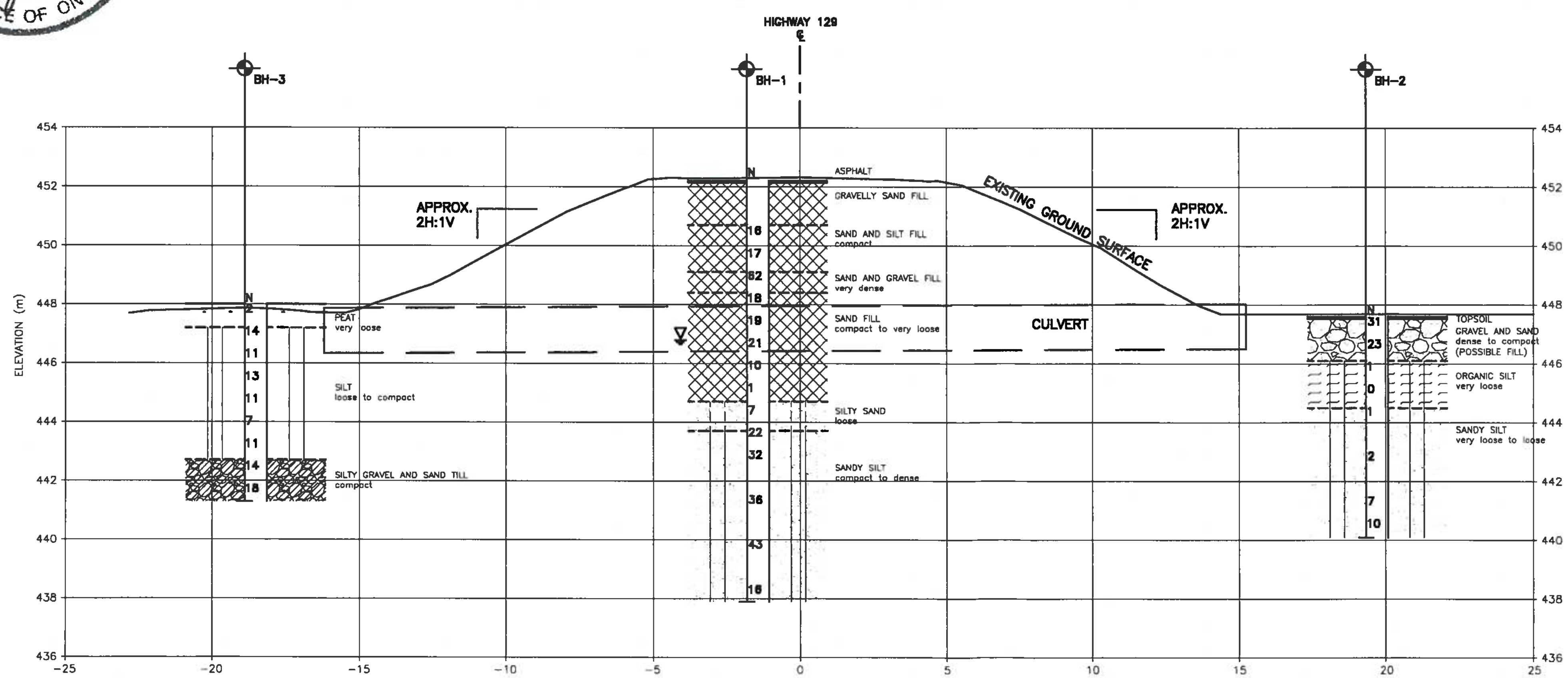
Report Format

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

Appendix A – Drawing



PLAN



CROSS SECTION AT CULVERT CENTRELINE

1:200
HORIZONTAL/VERTICAL

METRIC

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

1:500
PLAN

Agreement No. 5016-E-0016
GWP 411-00-00
GEOCRES No. 31F-195

CULVERT REPLACEMENT, STN. 12+415
HIGHWAY 129, REANEY TOWNSHIP
DISTRICT OF SUDBURY

BOREHOLE LOCATION PLAN AND SOIL STRATA

SHEET 1

exp Services Inc.

KEY PLAN - NTS

LEGEND

- BOREHOLE LOCATION
- N — STANDARD PENETRATION TEST (BLOWS/300mm)
- TEMPORARY BENCHMARK (EL. 452.3 m)
- MEASURED WATER LEVEL UPON COMPLETION OF DRILLING

BOREHOLE COORDINATES

BOREHOLE NO.	APPROX. ELEV. (m)	MTM COORDINATES	
		NORTHING	EASTING
BH-1	452.2	5264616.4	364386.8
BH-2	447.6	5264507.7	364407.2
BH-3	448.0	5264629.6	364370.9

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 Materials Engineering and Research Office, Downsview. Information contained in the report and related documents are specifically excluded in accordance with the conditions of Section GC 2.01 of OPS Gen. Cond.

SOIL STRATA SYMBOLS

ASPHALT, SILT, ORGANIC SILT, FILL, SANDY SILT, SILTY GRAVEL AND SAND TILL, PEAT, GRAVEL AND SAND, SILTY SAND

REVISIONS

DATE	BY	DESCRIPTION
2017.06.06	IM	SUBMISSION FOR MTO REVIEW
2017.08.22	IM	FINAL REPORT SUBMISSION

SCALE: AS NOTED		PROJECT NO.: SUD-00014543-AG
SUBM'D: IM	CHECKED: AS	DATE: 2017.06.06
DRAWN: IM	CHECKED: SG	APPROVED: SG / DWG. 1

Appendix B – Photographs

Photograph No. 1 – Highway 129 at Culvert, Stn. 12+415 (Facing North)

Photograph No. 2 – Pavement Cracking at Culvert (Facing South)

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

Photograph No. 4 – Culvert Inlet (Facing East)

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

Photograph No. 6 – Culvert Outlet (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		

0.002	0.006	0.02	0.06	0.2	0.6	2.0	6.0	20	60	200

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≤Pp≤10%
Little	15≤Pp≤25%
Some	30≤Pp≤45%
Mostly	50≤Pp≤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≤N<10
Compact	10≤N<30
Dense	30≤N<50
Very Dense	50≤N

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

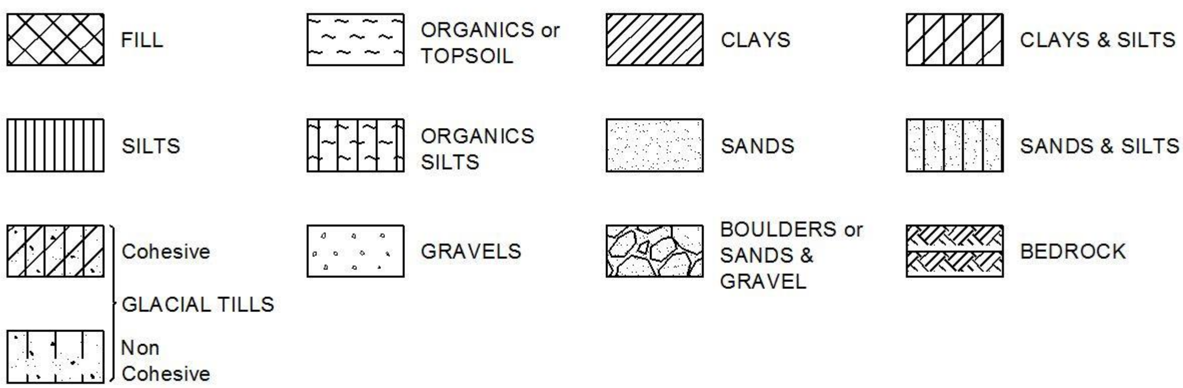
Table c: Consistency of Cohesive Soil

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

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

STRATA PLOT

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



WATER LEVEL MEASUREMENT

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

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

STRESS AND STRAIN

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

MECHANICALL PROPERIES OF SOIL

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

PHYSICAL PROPERTIES OF SOIL

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

RECORD OF BOREHOLE No BH-1

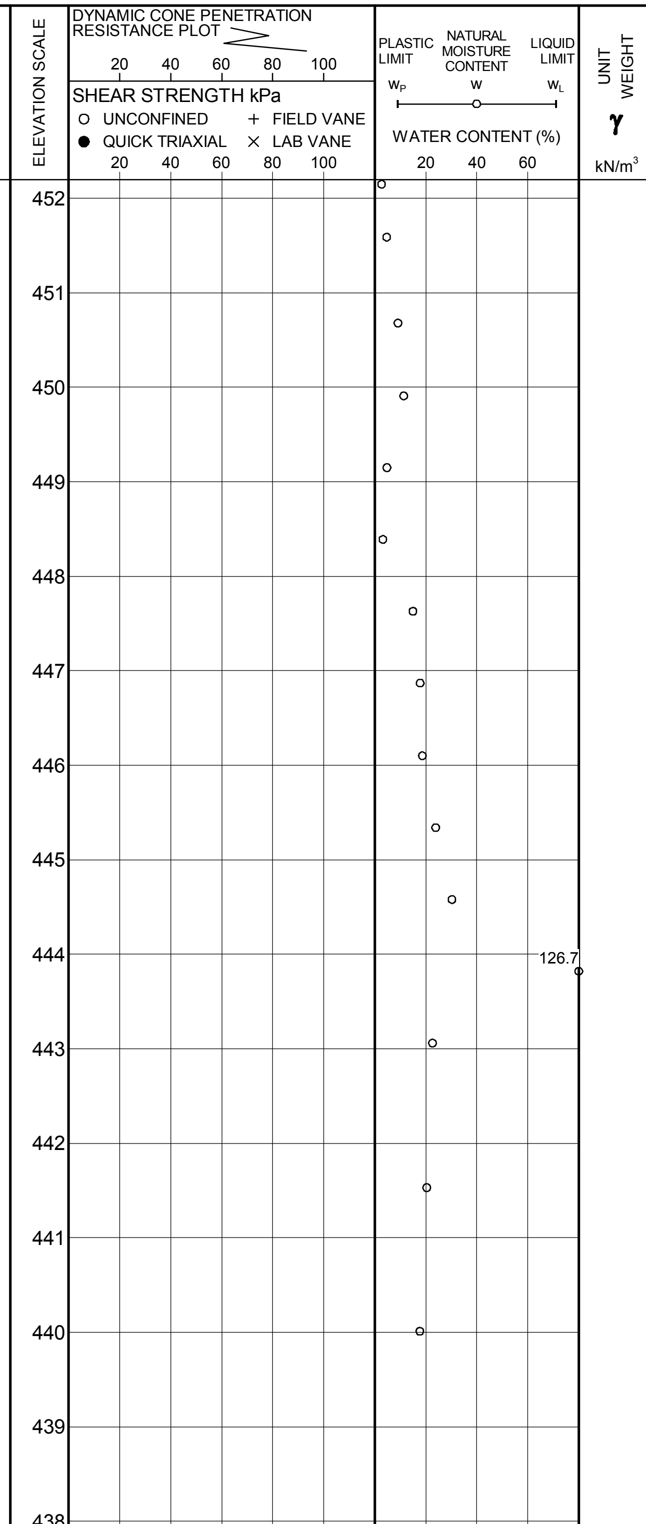
METRIC

W.P. 411-00-00,5016-E-0016 LOCATION Stn. 12+420, MTM-13, 5264616.41N, 364386.8E, Non-Structural Culvert at Stn. 12+415 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 - 2016.12.07 LATITUDE 47.51803 LONGITUDE -83.20878 CHECKED BY IM

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
452.2	Pavement Surface					
[illegible]	**ASPHALT** (~ 50 mm thick)					
	FILL, gravelly sand, trace silt, brown, moist.	1	AS			28 67 (5)
	some silt below ~ 0.6 m depth.	2	AS			26 62 (13)
450.7 / 1.5	**FILL,** sand and silt, trace clay, trace gravel, brown to grey, moist, fine grained sand, compact.	3	SS	16		4 54 40 2
		4	SS	17		
449.2 / 3.1	**FILL,** sand and gravel, brown, moist, very dense.	5	SS	62		
448.4 / 3.8	**FILL,** sand, medium to coarse grained, trace silt, trace gravel, trace clay, brown, moist, compact.	6	SS	18		3 88 8 1
		7	SS	19		
		8	SS	21	∇	
	grey, wet below ~ 6.1 m depth.	9	SS	10		
	very loose below ~ 6.9 m depth.	10	SS	1		
444.7 / 7.5	**SILTY SAND,** trace gravel, some organics/wood, brown to black, moist to wet, loose.	11	SS	7		
443.7 / 8.5	wood piece (~ 200 mm long) encountered below ~ 8.4 m depth. **SANDY SILT,** trace clay, grey, moist, compact to dense.	12	SS	22		
		13	SS	32		
		14	SS	36		
		15	SS	43		0 32 64 4
437.9 / 14.3	Heaving soils encountered, some gravel returned in split spoon at ~ 13.7 m depth.	16	SS	16		
	END OF BOREHOLE Borehole terminated at ~ 14.3 m depth.					



NOTES:
1. This drawing to be read with the subject report and project numbers as presented above.

$+^3, \times^3$: Numbers refer to Sensitivity $\bigcirc^{3\%}$ STRAIN AT FAILURE

Ministry of Transportation Ontario

Foundation Design

RECORD OF BOREHOLE No BH-2

METRIC

W.P. 411-00-00,5016-E-0016 LOCATION Stn. 12+409, MTM-13, 5264607.72N, 364407.23E, Non-Structural Culvert at Stn. 12+415 ORIGINATED BY ST

DIST Sudbury HWY 129 BOREHOLE TYPE Portable Tripod With Cathead and Hilti D200 Drill COMPILED BY IM

DATUM Geodetic DATE 2016.01.14 - 2016.01.14 LATITUDE 47.51795 LONGITUDE -83.20851 CHECKED BY IM

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
447.6	Ground Surface					
0.1	**TOPSOIL** (~ 75 mm thick)					
	GRAVEL AND SAND, some silt, brown, wet, dense to compact. (Possible Fill)	1	SS	31		45 44 (12)
		2	SS	23		
446.1 / 1.5	**ORGANIC SILT,** some sand, trace gravel, wet, grey, very loose.	3	SS	1	204.5	
		4	SS	0	115.1	
444.6 / 3.1	**SANDY SILT,** trace gravel, trace clay, wet, grey, very loose.	5	SS	1		4 20 69 7
		6	SS	2		
	loose below ~ 6.1 m depth.	7	SS	7		
		8	SS	10		
440.1 / 7.5	**END OF BOREHOLE** Borehole terminated at ~ 7.5 m depth.					

Column headings also printed: STRAT PLOT; GROUND WATER CONDITIONS; ELEVATION SCALE (447, 446, 445, 444, 443, 442, 441); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE, 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L, WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³.

NOTES:
1. This drawing to be read with the subject report and project numbers as presented above.
2. Groundwater level not measured within borehole as water was pumped into hole due to washboring technique utilized.

$+^3, \times^3$: Numbers refer to Sensitivity O $^{3\%}$ STRAIN AT FAILURE

Ministry of Transportation Ontario

Foundation Design

RECORD OF BOREHOLE No BH-3

METRIC

W.P. 411-00-00,5016-E-0016
LOCATION Stn. 12+428, MTM-13, 5264629.59N, 364370.85E, Non-Structural Culvert at Stn. 12+415
ORIGINATED BY ST

DIST Sudbury HWY 129
BOREHOLE TYPE Portable Tripod With Cathead and Hilti D200 Drill
COMPILED BY IM

DATUM Geodetic
DATE 2016.01.14 - 2016.01.14
LATITUDE 47.51815
LONGITUDE -83.20899
CHECKED BY IM

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	GR	SA	SI	CL
448.0	Ground Surface									
0.0	**PEAT,** black, fibrous, moist to wet, very loose.	1	SS	2						
447.2 / 0.8	**SILT,** some peat, some sand, black to grey, wet, loose to compact.	2	SS	14	447					
	no peat, grey, below ~ 1.5 m depth.	3	SS	11	446					
	trace gravel, trace clay below ~ 2.3 m depth.	4	SS	13			1	19	74	6
		5	SS	11	445					
		6	SS	7	444		0	16	80	4
	some gravel below ~ 4.6 m depth.	7	SS	11	443					
442.7 / 5.3	**TILL,** silty gravel and sand, trace clay, grey, wet, coarse grained sand, compact.	8	SS	14			39	32	29	1
		9	SS	18	442					
441.3 / 6.7	**END OF BOREHOLE** Borehole terminated at ~ 6.7 m depth.									

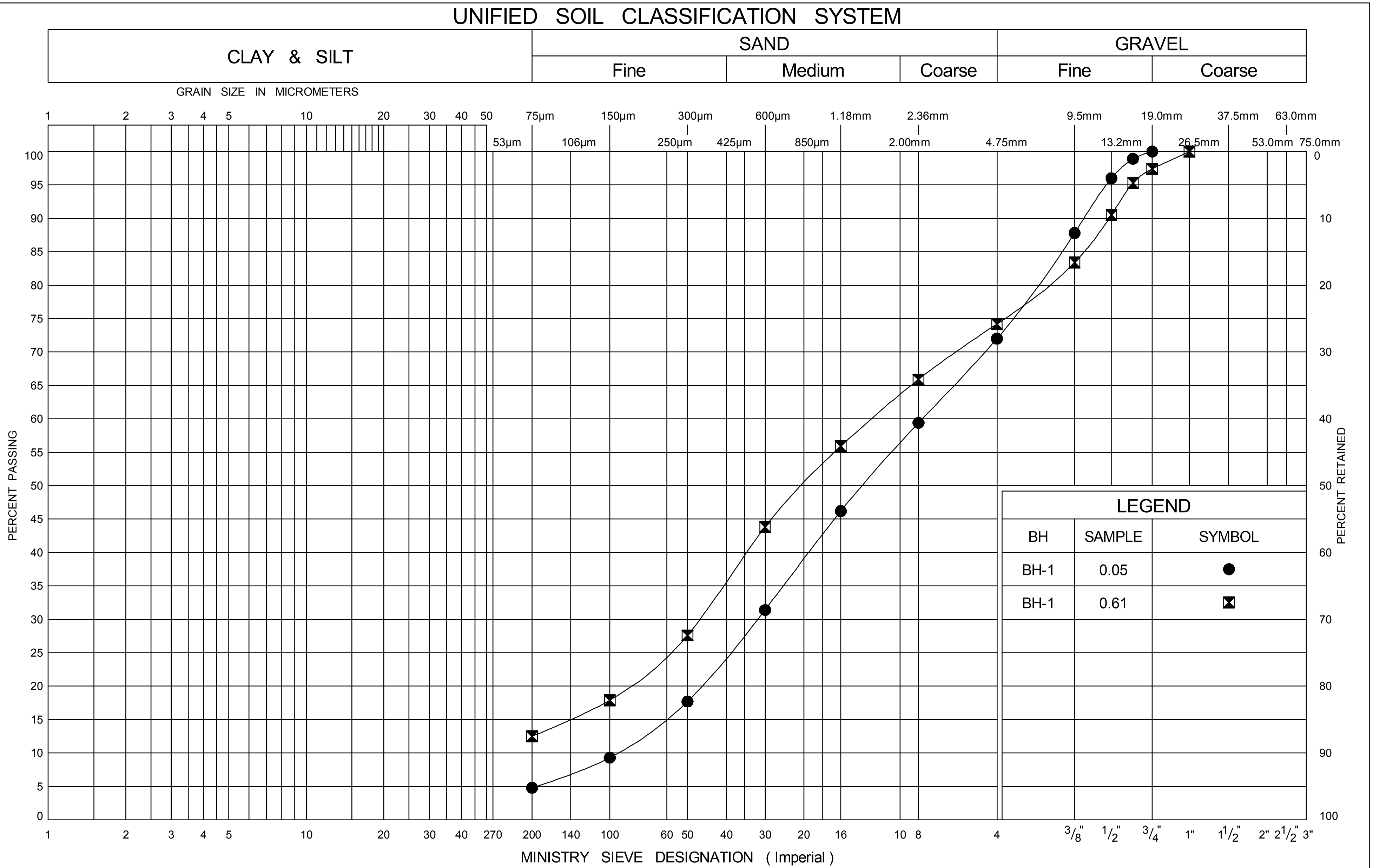
NOTES:
1. This drawing to be read with the subject report and project numbers as presented above.
2. Groundwater level not measured within borehole as water was pumped into hole due to washboring technique utilized.

Column headings: SOIL PROFILE; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 20 40 60 80 100); PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60 100.8; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

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

Appendix D –
Laboratory Test Results

78 12 M



GRAIN SIZE DISTRIBUTION

GRAVELLY SAND FILL

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Ontario

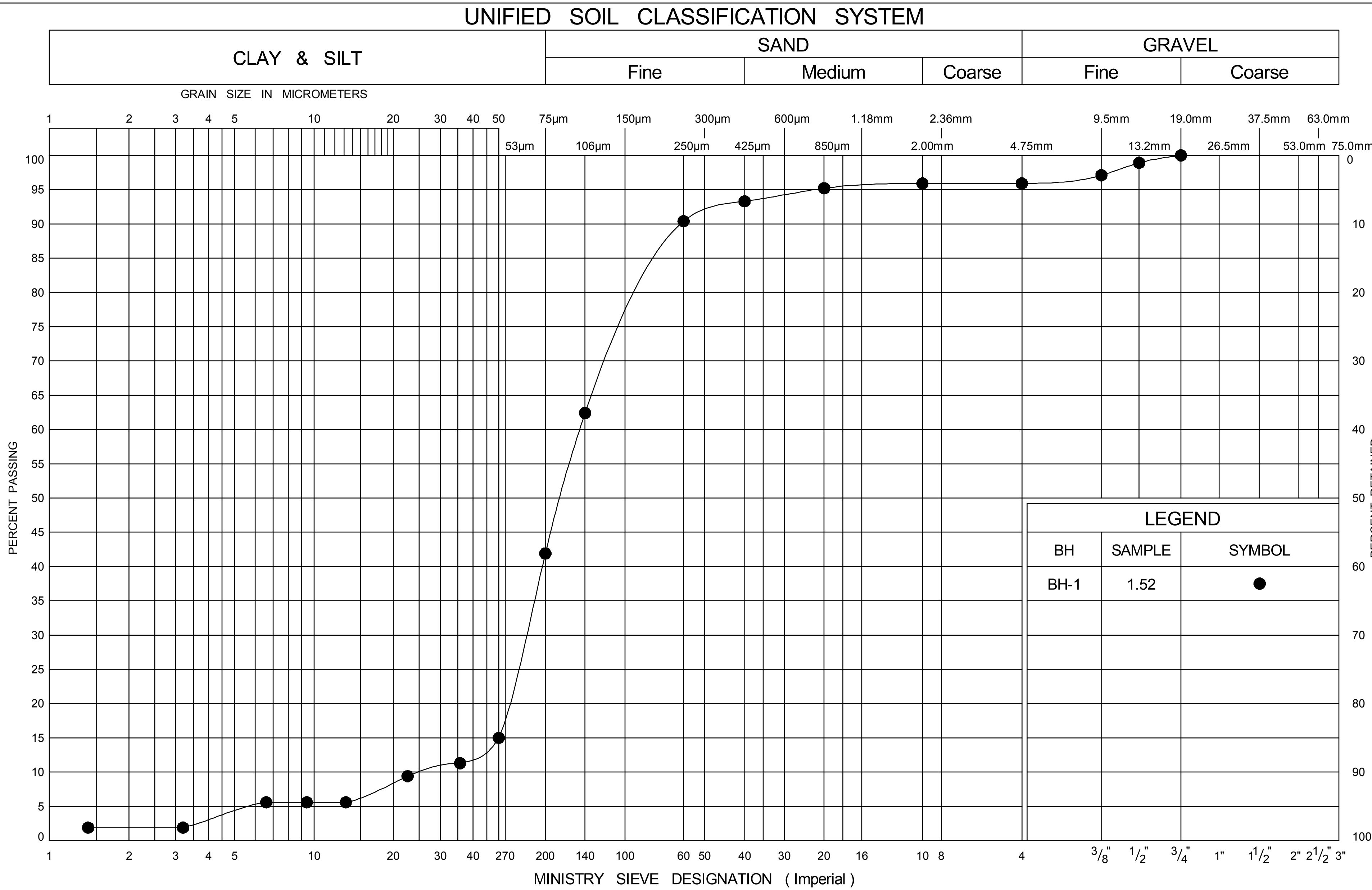
FIG No 1

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

Culvert Replacement

ONTARIO MOT GRAIN SIZE SUD-00014543-AG - HWY. 129 - CL CULVERT 12+415.GPJ ONTARIO MOT.GDT 6/6/17

78 12 M



Ministry of Transportation
Ontario

GRAIN SIZE DISTRIBUTION

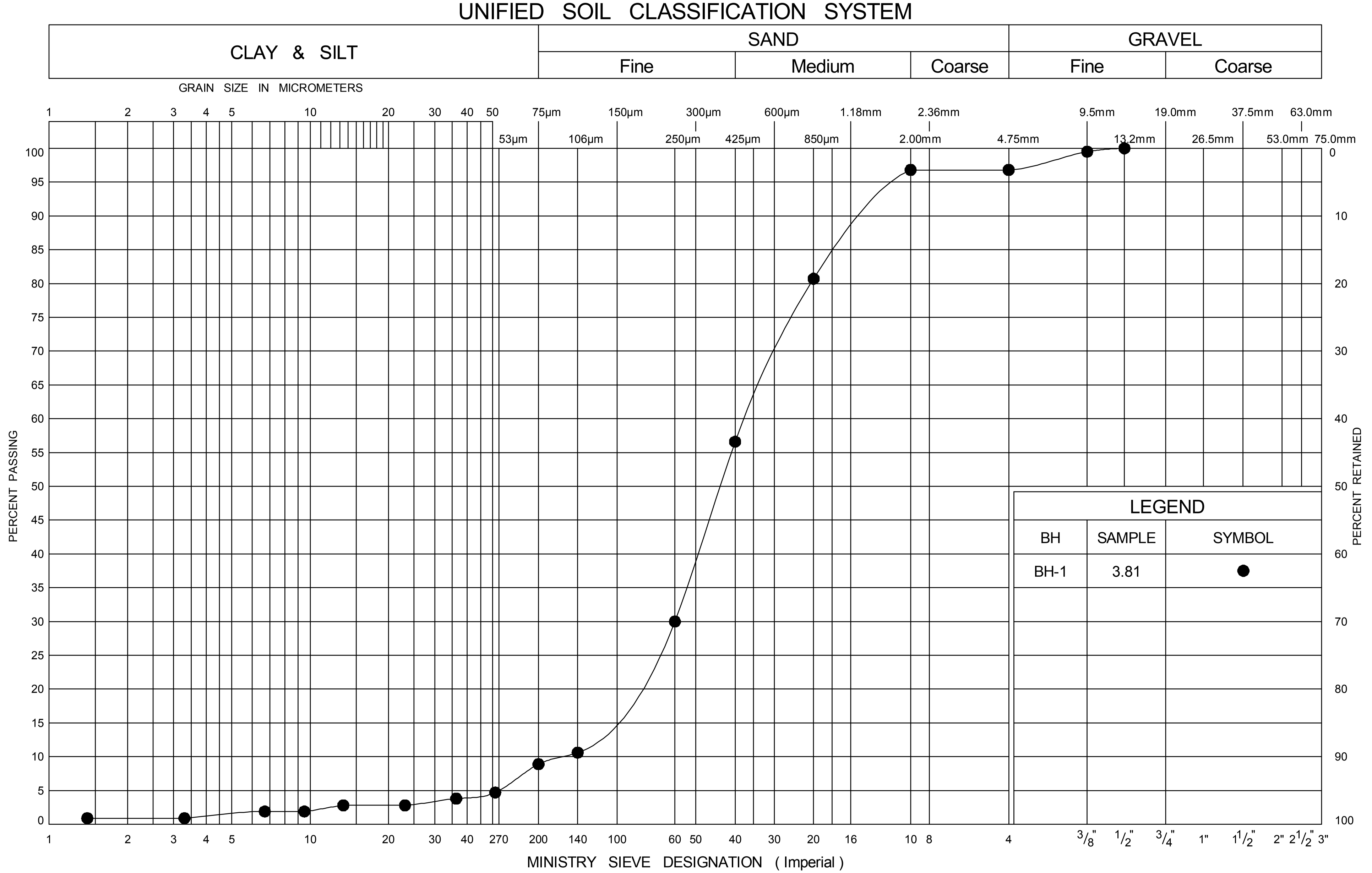
SAND AND SILT FILL

FIG No 2

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

Culvert Replacement

78 12 M



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Ontario

GRAIN SIZE DISTRIBUTION

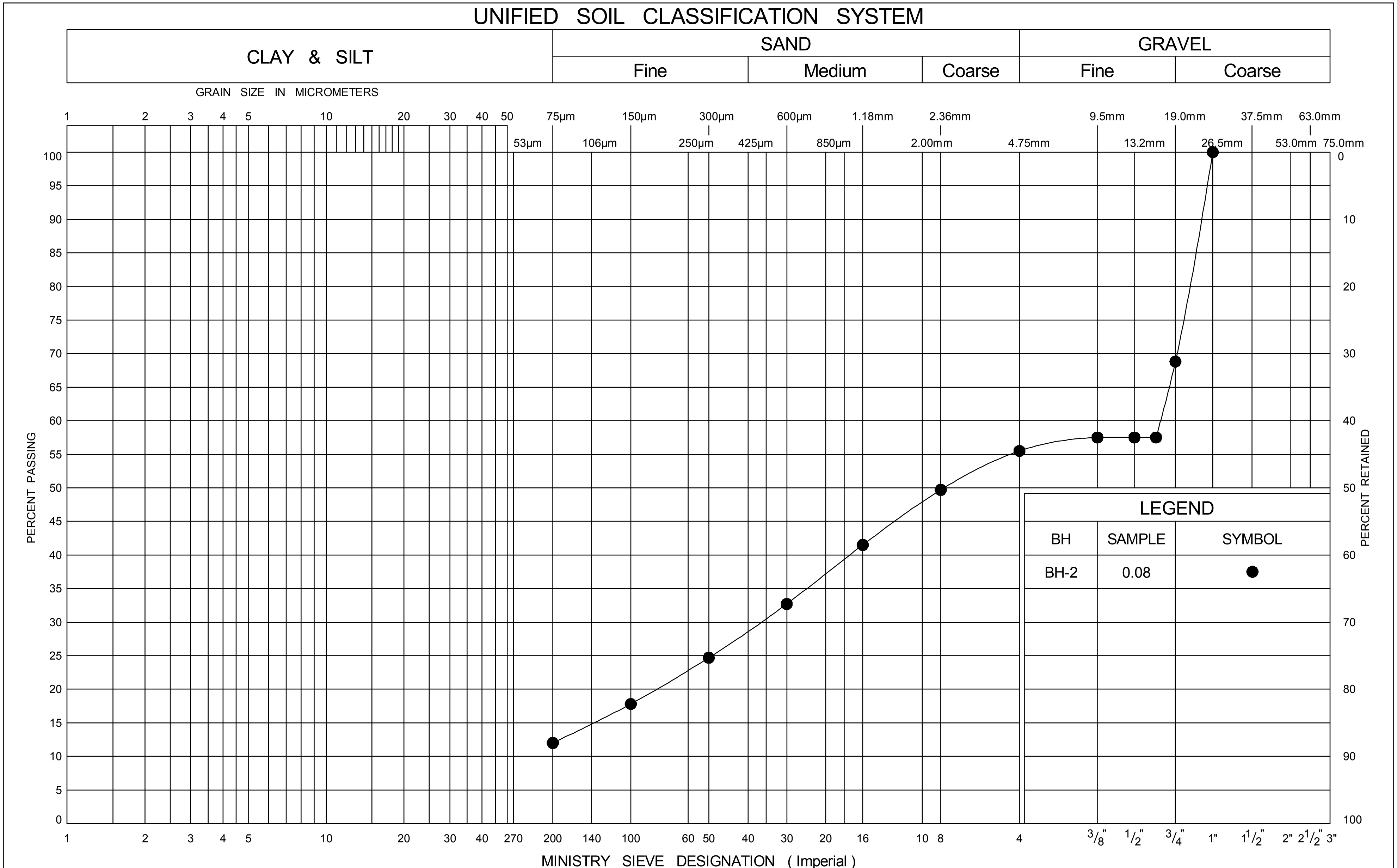
SAND FILL

FIG No 3

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

Culvert Replacement

78 12 M



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GRAIN SIZE DISTRIBUTION

GRAVEL AND SAND (POSSIBLE FILL)

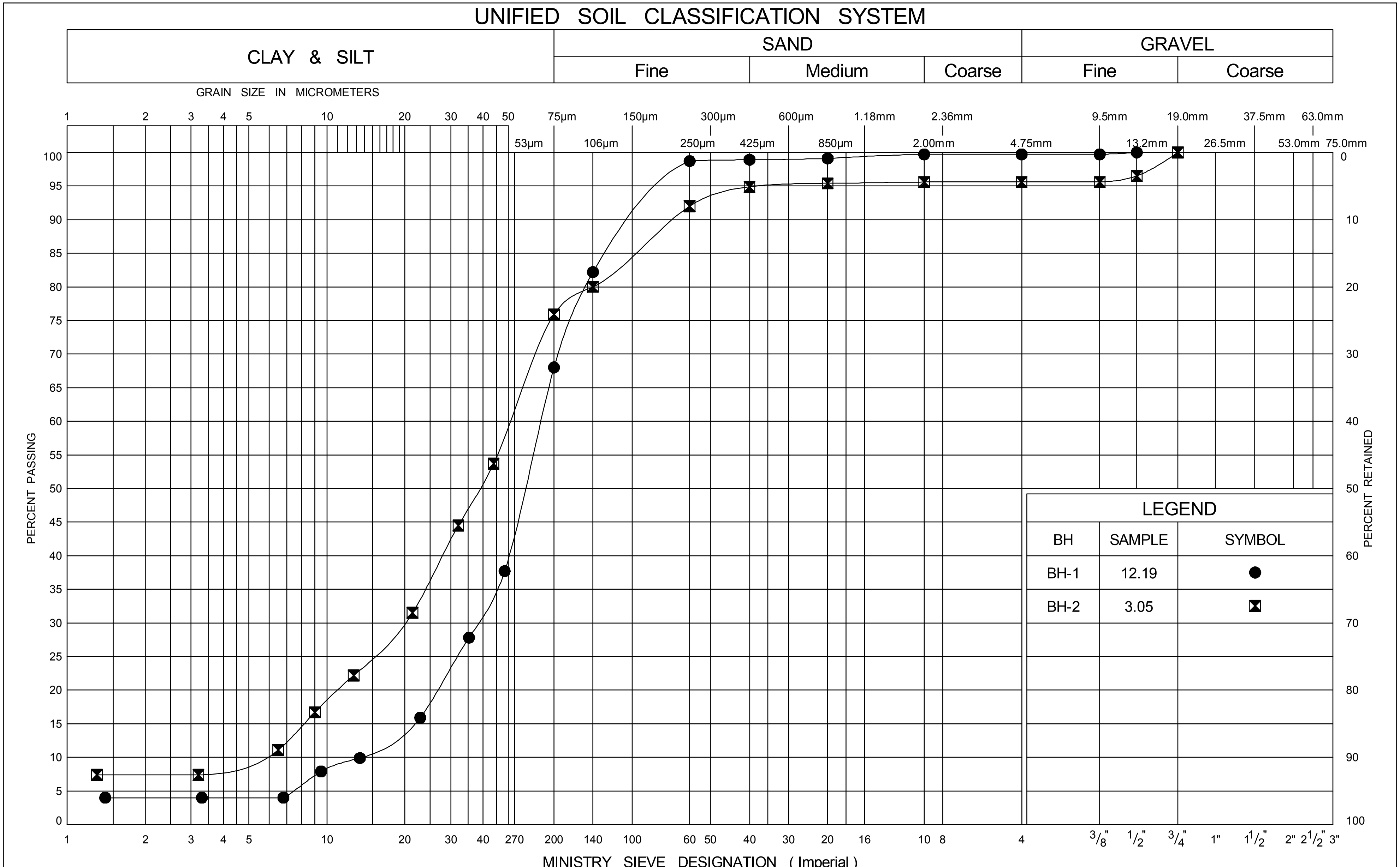
FIG No 4

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

Culvert Replacement

ONTARIO MOT GRAIN SIZE SUD-00014543-AG - HWY. 129 - CL CULVERT 12+415.GPJ ONTARIO MOT.GDT 6/6/17

78 12 M



Ministry of Transportation Ontario	GRAIN SIZE DISTRIBUTION SANDY SILT

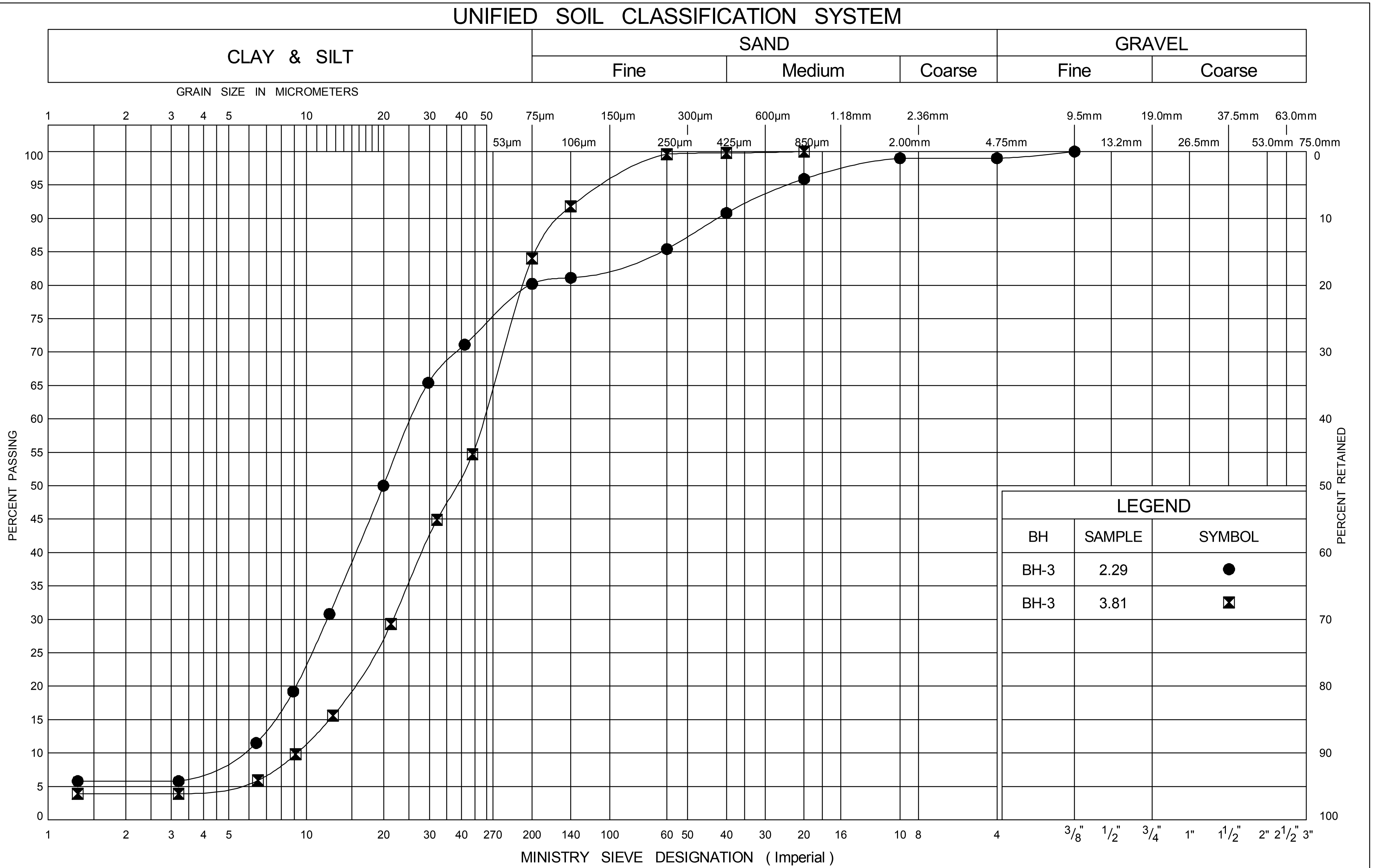
FIG No 5

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

Culvert Replacement

ONTARIO MOT GRAIN SIZE SUD-00014543-AG - HWY. 129 - CL CULVERT 12+415.GPJ ONTARIO MOT.GDT 6/6/17

78 12 M



Ministry of Transportation
Ontario

GRAIN SIZE DISTRIBUTION

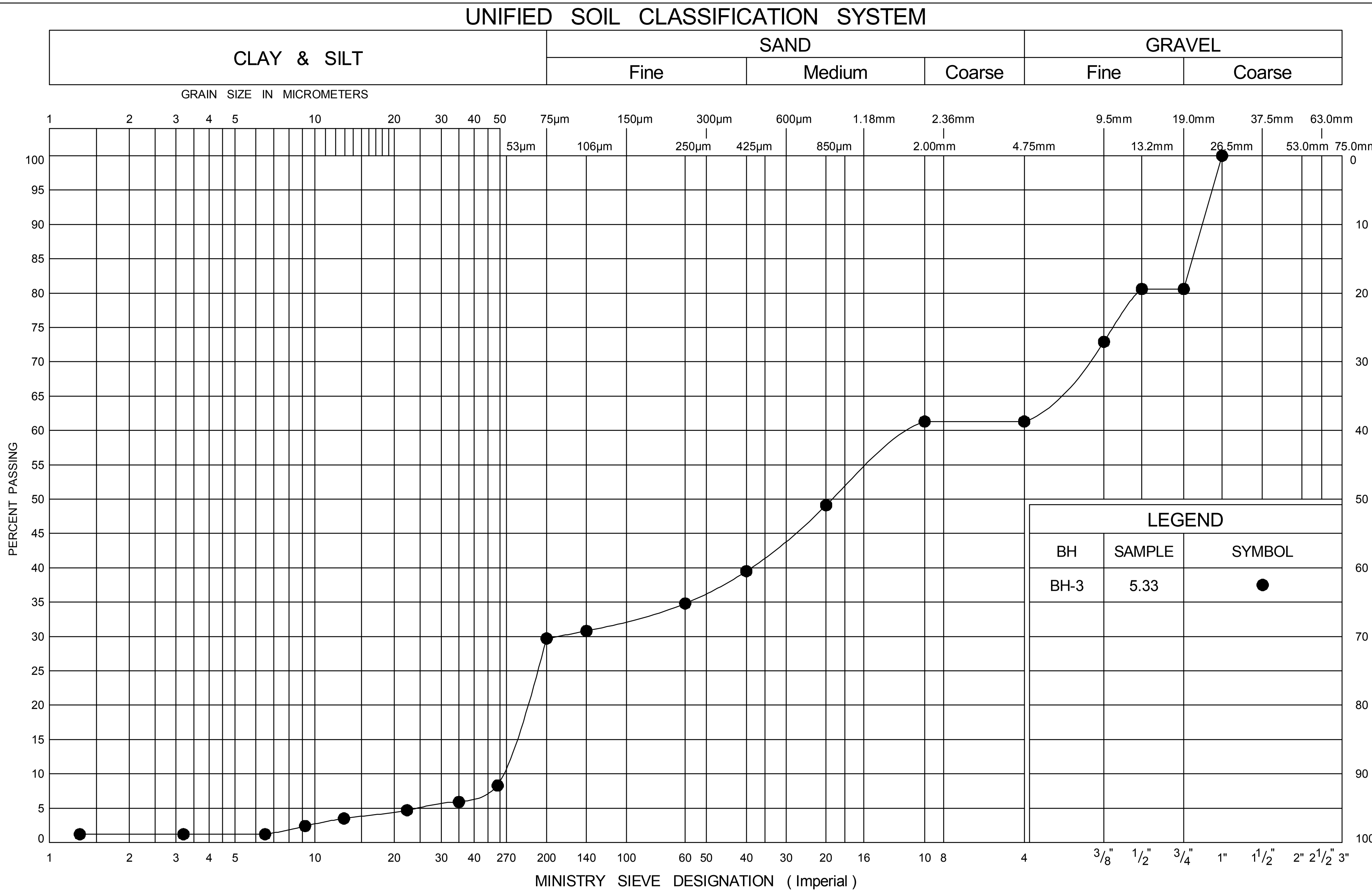
SILT

FIG No 6

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

Culvert Replacement

78 12 M



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GRAIN SIZE DISTRIBUTION

SILTY GRAVEL AND SAND TILL

FIG No 7

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

Culvert Replacement

ONTARIO MOT GRAIN SIZE SUD-00014543-AG - HWY. 129 - CL CULVERT 12+415.GPJ ONTARIO MOT.GDT 6/6/17

Appendix E –
Slope Stability Analyses

FIGURE E-1 - Existing Embankment Stability - Inlet Side
Culvert 12+415, Highway 129, Reaney Township
DRAINED CONDITIONS

Name: Gravelly Sand Fill Unit Weight: 20 kN/m³ Cohesion': 0 kPa Phi': 32 °
Name: Sand and Silt Fill Unit Weight: 20 kN/m³ Cohesion': 0 kPa Phi': 30 °
Name: Sand and Gravel Fill Unit Weight: 32 kN/m³ Cohesion': 0 kPa Phi': 35 °
Name: Sand Fill Unit Weight: 20 kN/m³ Cohesion': 0 kPa Phi': 30 °
Name: Peat Unit Weight: 15 kN/m³ Cohesion': 0 kPa Phi': 5 °
Name: Gravel and Sand Unit Weight: 21 kN/m³ Cohesion': 0 kPa Phi': 35 °
Name: Organic Silt Unit Weight: 17 kN/m³ Cohesion': 0 kPa Phi': 25 °
Name: Sandy Silt/Silty Sand - Very Loose to Loose Unit Weight: 19 kN/m³ Cohesion': 0 kPa Phi': 28 °
Name: Silt-Sandy Silt - Compact to Dense Unit Weight: 20 kN/m³ Cohesion': 0 kPa Phi': 30 °
Name: Silty Gravel and Sand Till Unit Weight: 21 kN/m³ Cohesion': 0 kPa Phi': 35 °

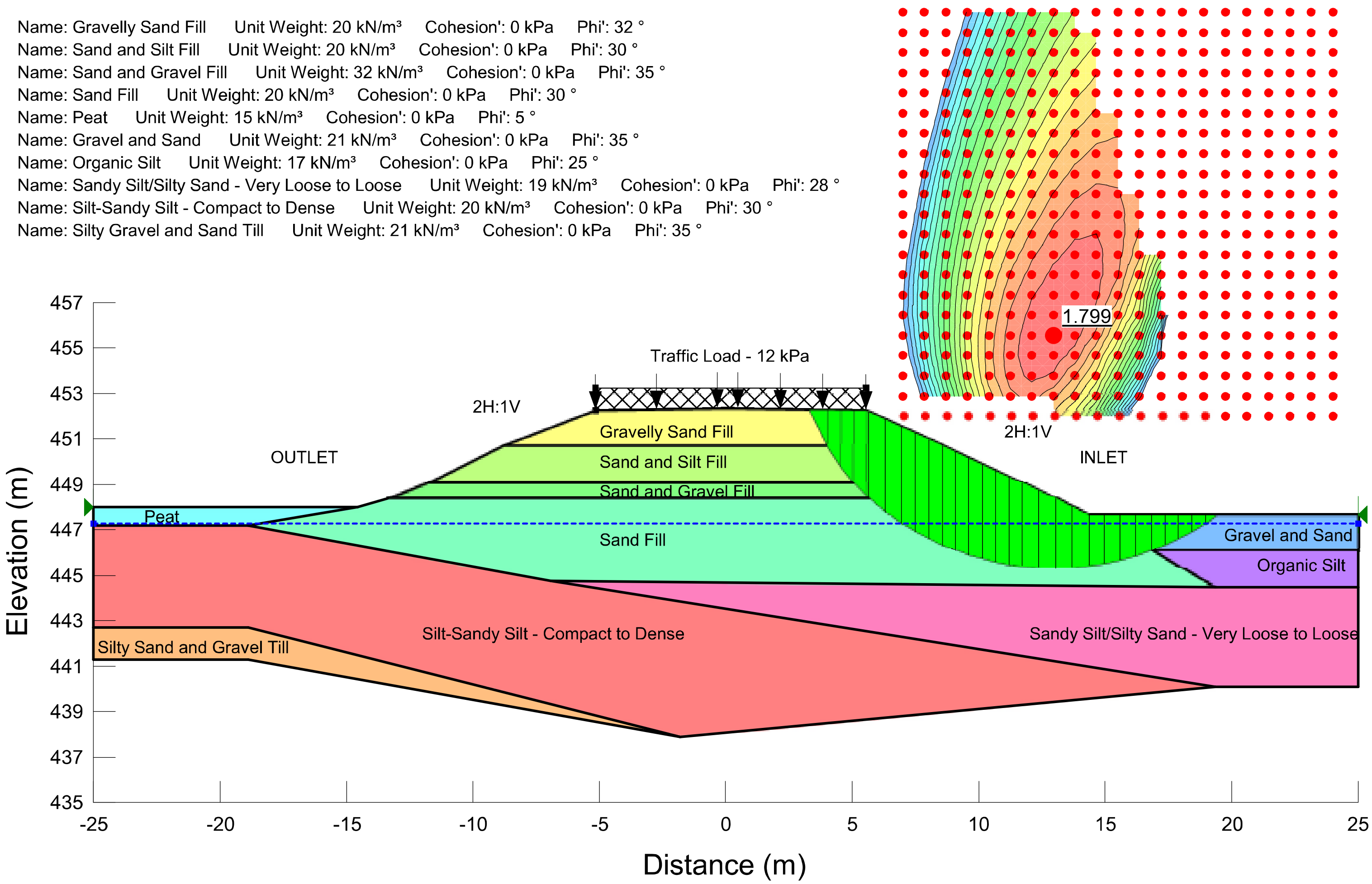


FIGURE E-2 - Existing Embankment Stability - Outlet Side
Culvert 12+415, Highway 129, Reaney Township
DRAINED CONDITIONS

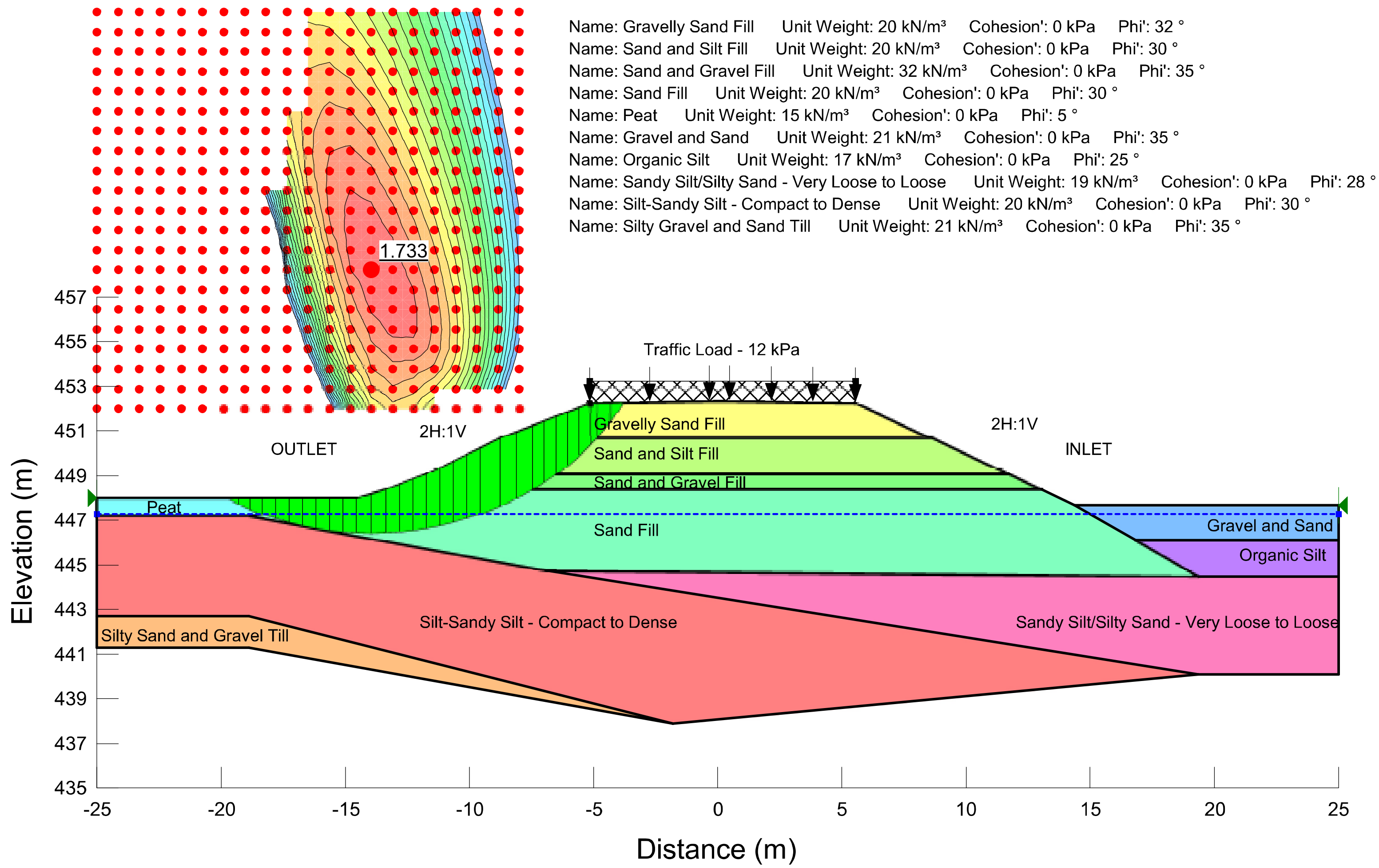


FIGURE E-3 - Proposed Embankment Stability - Inlet Side
Organics Not Removed Below Proposed Culvert
Culvert 12+415, Highway 129, Reaney Township
DRAINED CONDITIONS

Name: Sand Fill   Unit Weight: 20 kN/m³   Cohesion': 0 kPa   Phi': 30 °
Name: Peat   Unit Weight: 15 kN/m³   Cohesion': 0 kPa   Phi': 5 °
Name: Gravel and Sand   Unit Weight: 21 kN/m³   Cohesion': 0 kPa   Phi': 35 °
Name: Organic Silt   Unit Weight: 17 kN/m³   Cohesion': 0 kPa   Phi': 25 °
Name: Sandy Silt/Silty Sand - Very Loose to Loose   Unit Weight: 19 kN/m³   Cohesion': 0 kPa   Phi': 28 °
Name: Silt-Sandy Silt - Compact to Dense   Unit Weight: 20 kN/m³   Cohesion': 0 kPa   Phi': 30 °
Name: Silty Gravel and Sand Till   Unit Weight: 21 kN/m³   Cohesion': 0 kPa   Phi': 35 °
Name: Granular B Type I   Unit Weight: 21.2 kN/m³   Cohesion': 0 kPa   Phi': 32 °

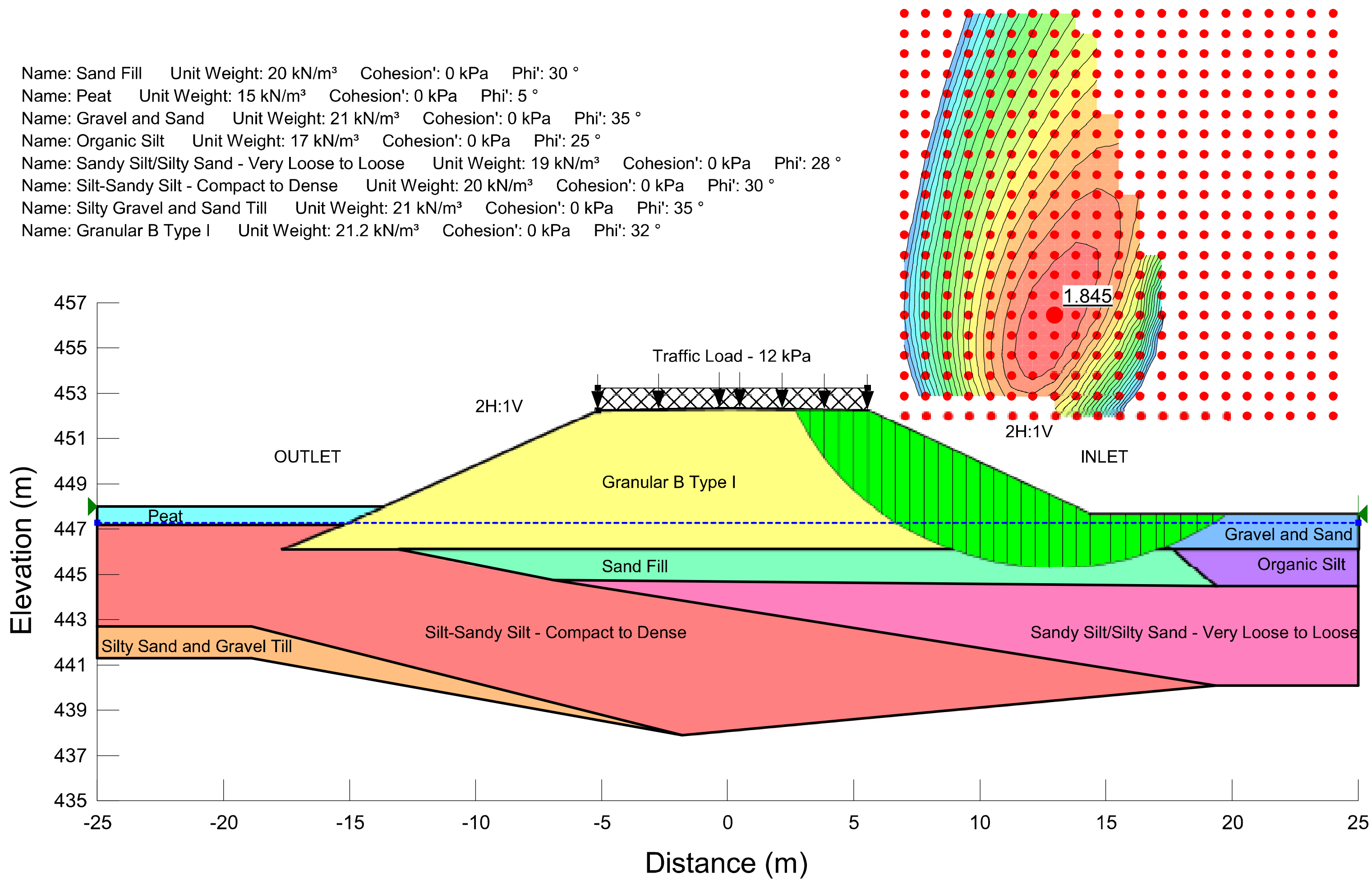


FIGURE E-4 - Proposed Embankment Stability - Outlet Side
Organics Not Removed Below Proposed Culvert
Culvert 12+415, Highway 129, Reaney Township
DRAINED CONDITIONS

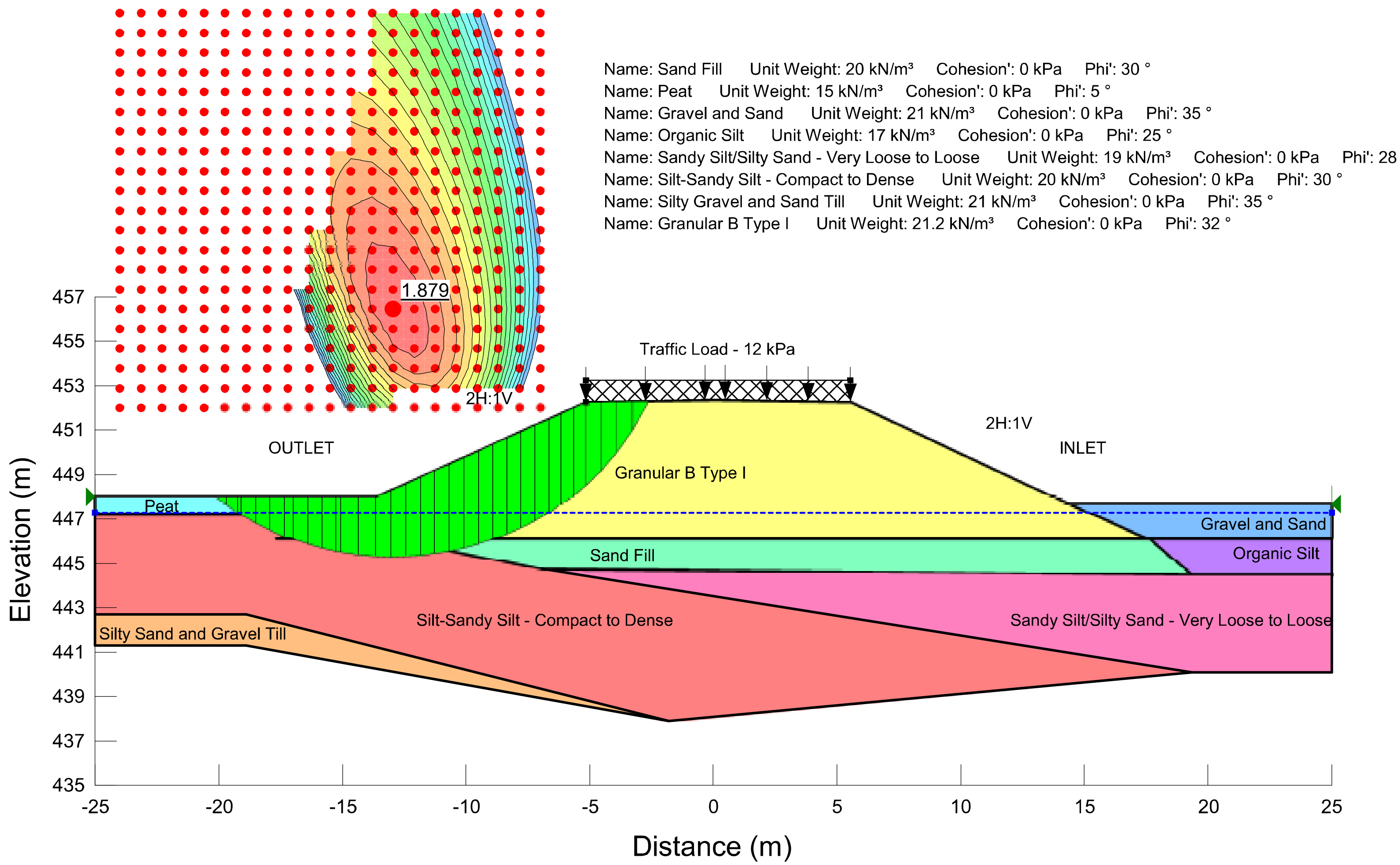


FIGURE E-5 - Proposed Embankment Stability - Inlet Side
Organics Removed Below Proposed Culvert
Culvert 12+415, Highway 129, Reaney Township
DRAINED CONDITIONS

Name: Peat Unit Weight: 15 kN/m³ Cohesion': 0 kPa Phi': 5 °
Name: Gravel and Sand Unit Weight: 21 kN/m³ Cohesion': 0 kPa Phi': 35 °
Name: Organic Silt Unit Weight: 17 kN/m³ Cohesion': 0 kPa Phi': 25 °
Name: Sandy Silt/Silty Sand - Very Loose to Loose Unit Weight: 19 kN/m³ Cohesion': 0 kPa Phi': 28 °
Name: Silt-Sandy Silt - Compact to Dense Unit Weight: 20 kN/m³ Cohesion': 0 kPa Phi': 30 °
Name: Silty Gravel and Sand Till Unit Weight: 21 kN/m³ Cohesion': 0 kPa Phi': 35 °
Name: Granular B Type I Unit Weight: 21.2 kN/m³ Cohesion': 0 kPa Phi': 32 °

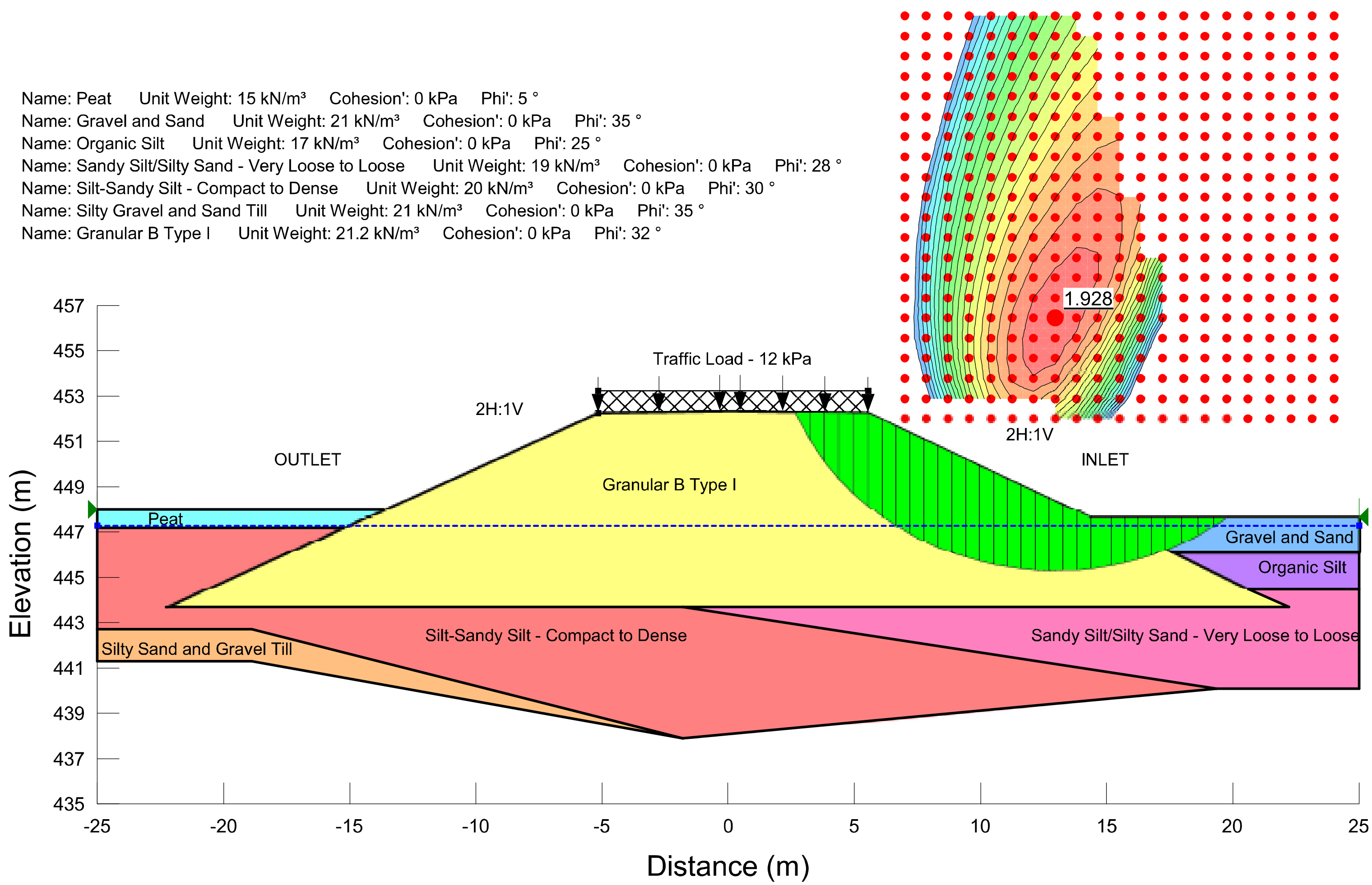


FIGURE E-6 - Proposed Embankment Stability - Outlet Side
Organics Removed Below Proposed Culvert
Culvert 12+415, Highway 129, Reaney Township
DRAINED CONDITIONS

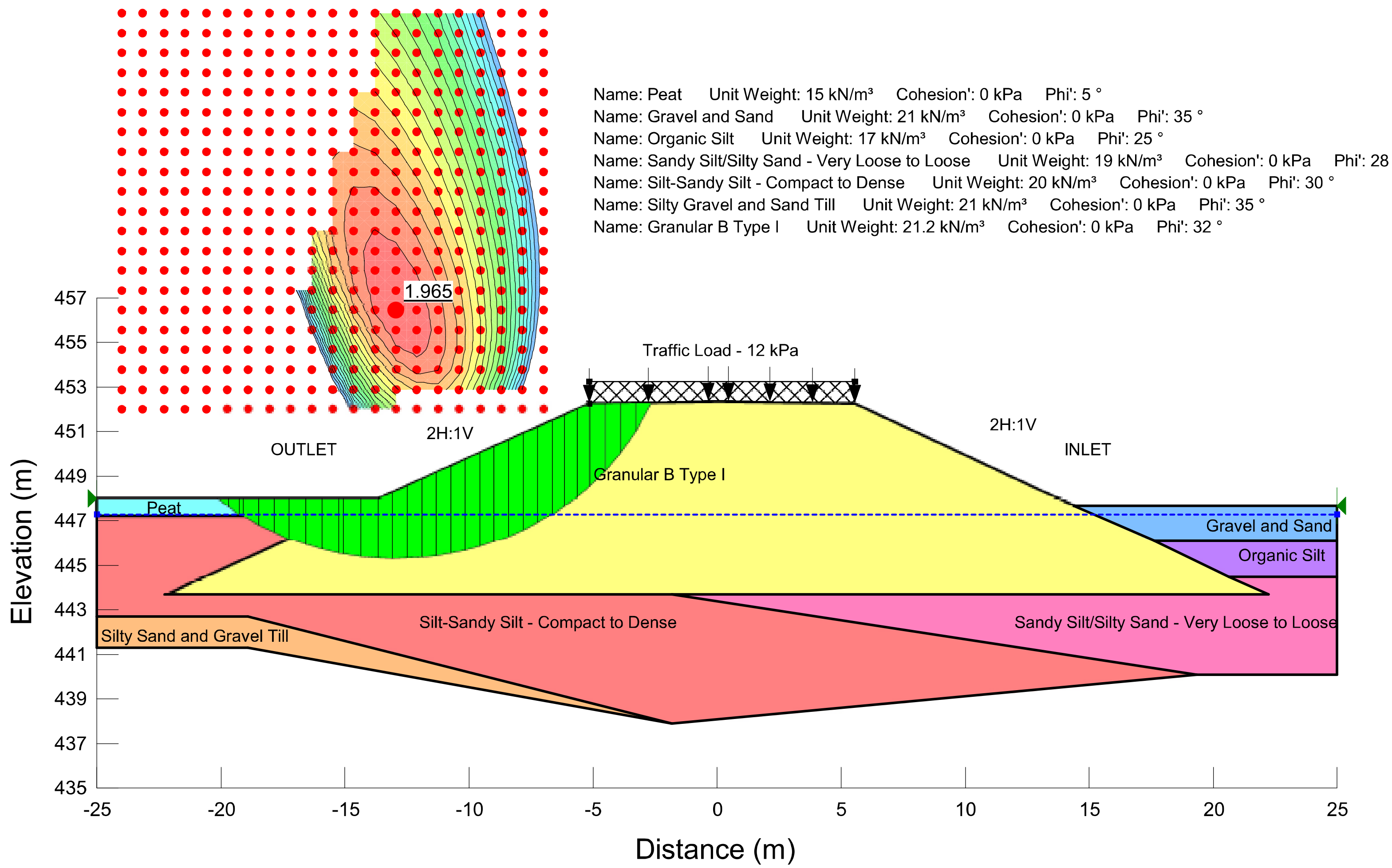


FIGURE E-7 - Detour Embankment - Inlet Side, West Embankment Analysis
Organics Not Removed Below Proposed Culvert
Culvert 12+415, Highway 129, Reaney Township
DRAINED CONDITIONS

Name: Sand Fill Unit Weight: 20 kN/m³ Cohesion': 0 kPa Phi': 30 °
Name: Peat Unit Weight: 15 kN/m³ Cohesion': 0 kPa Phi': 5 °
Name: Gravel and Sand Unit Weight: 21 kN/m³ Cohesion': 0 kPa Phi': 35 °
Name: Organic Silt Unit Weight: 17 kN/m³ Cohesion': 0 kPa Phi': 25 °
Name: Sandy Silt/Silty Sand - Very Loose to Loose Unit Weight: 19 kN/m³ Cohesion': 0 kPa Phi': 28 °
Name: Silt-Sandy Silt - Compact to Dense Unit Weight: 20 kN/m³ Cohesion': 0 kPa Phi': 30 °
Name: Silty Gravel and Sand Till Unit Weight: 21 kN/m³ Cohesion': 0 kPa Phi': 35 °
Name: Granular B Type I Unit Weight: 21.2 kN/m³ Cohesion': 0 kPa Phi': 32 °

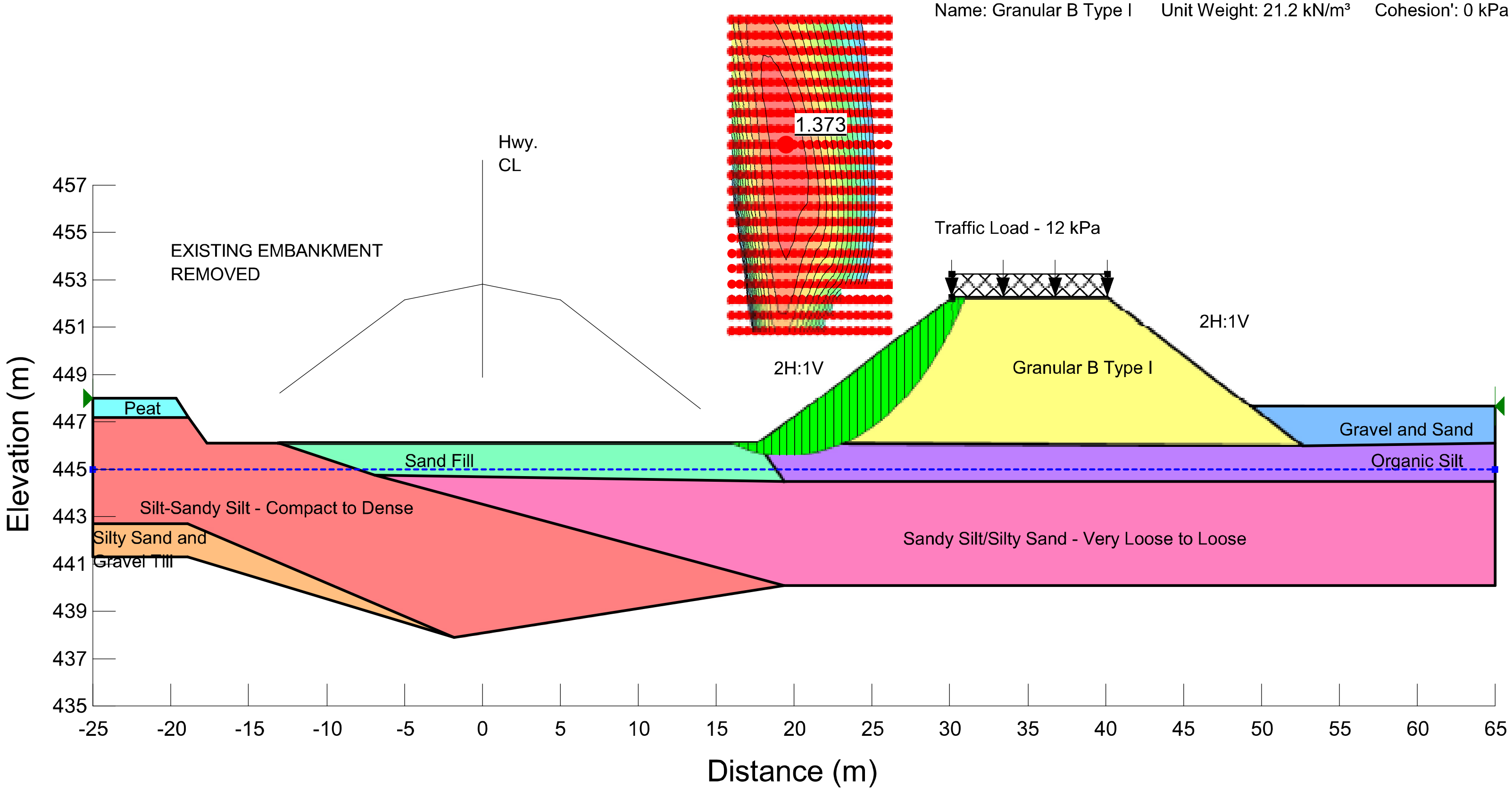


FIGURE E-8 - Detour Embankment - Inlet Side, East Embankment Analysis
Organics Not Removed Below Proposed Culvert
Culvert 12+415, Highway 129, Reaney Township
DRAINED CONDITIONS

Name: Sand Fill Unit Weight: 20 kN/m³ Cohesion': 0 kPa Phi': 30 °
Name: Peat Unit Weight: 15 kN/m³ Cohesion': 0 kPa Phi': 5 °
Name: Gravel and Sand Unit Weight: 21 kN/m³ Cohesion': 0 kPa Phi': 35 °
Name: Organic Silt Unit Weight: 17 kN/m³ Cohesion': 0 kPa Phi': 25 °
Name: Sandy Silt/Silty Sand - Very Loose to Loose Unit Weight: 19 kN/m³ Cohesion': 0 kPa Phi': 28 °
Name: Silt-Sandy Silt - Compact to Dense Unit Weight: 20 kN/m³ Cohesion': 0 kPa Phi': 30 °
Name: Silty Gravel and Sand Till Unit Weight: 21 kN/m³ Cohesion': 0 kPa Phi': 35 °
Name: Granular B Type I Unit Weight: 21.2 kN/m³ Cohesion': 0 kPa Phi': 32 °

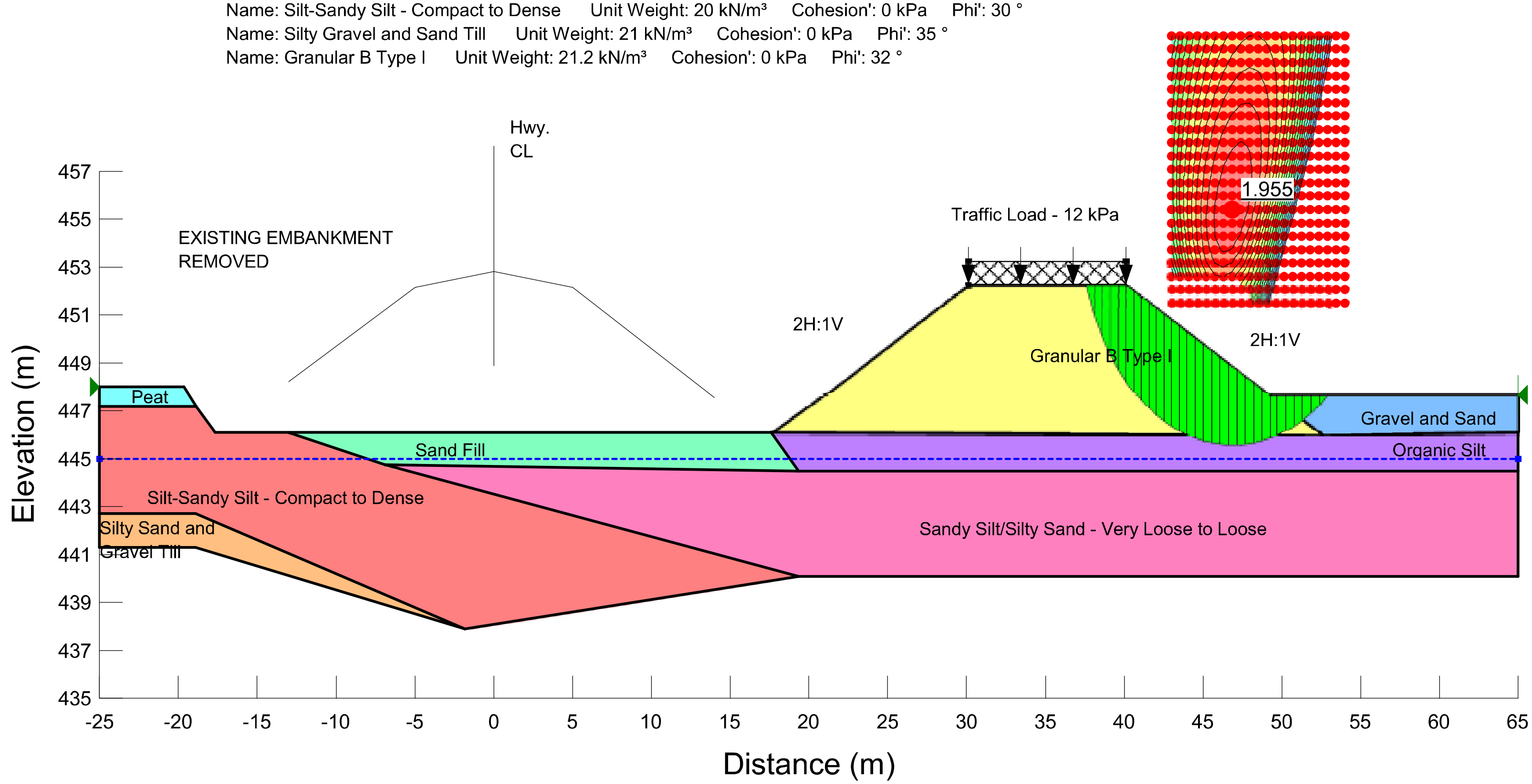
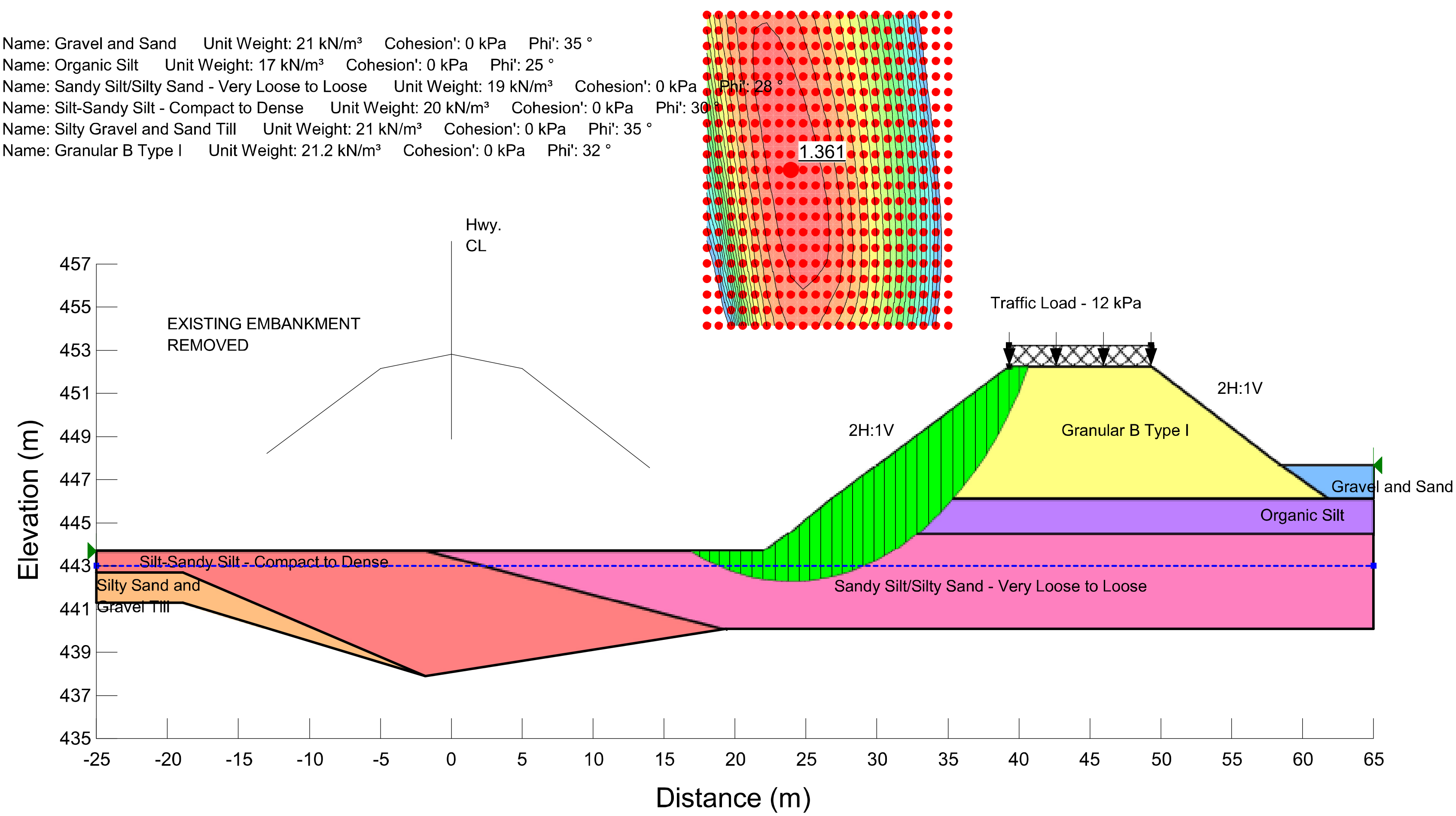
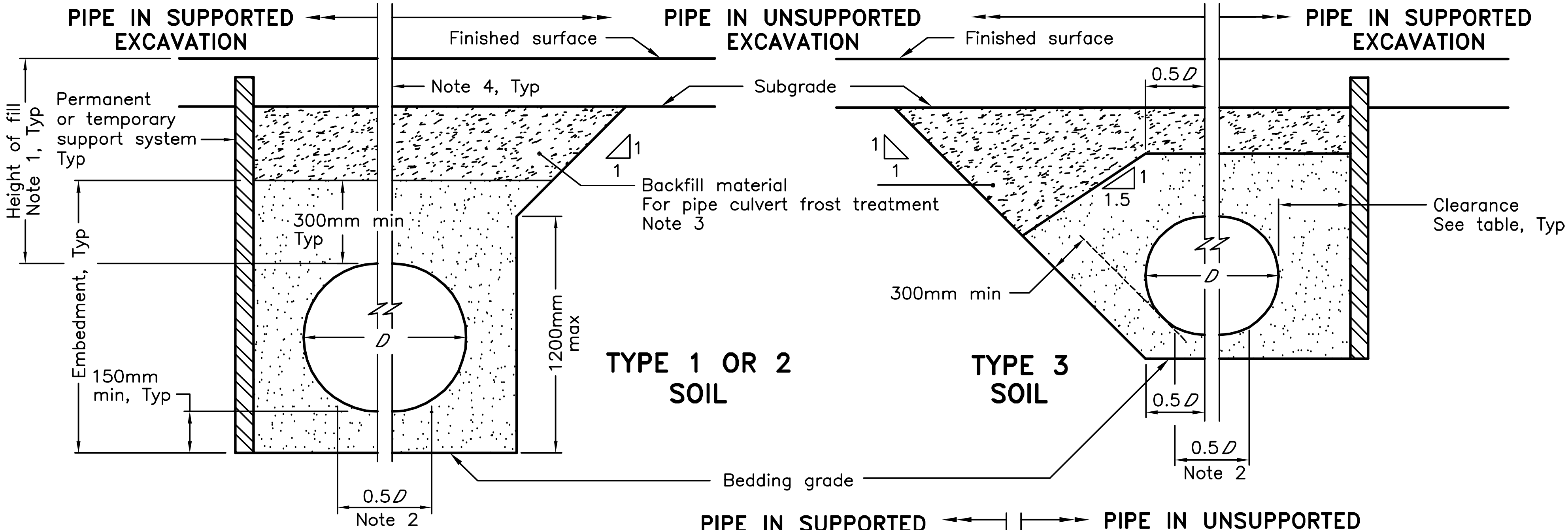


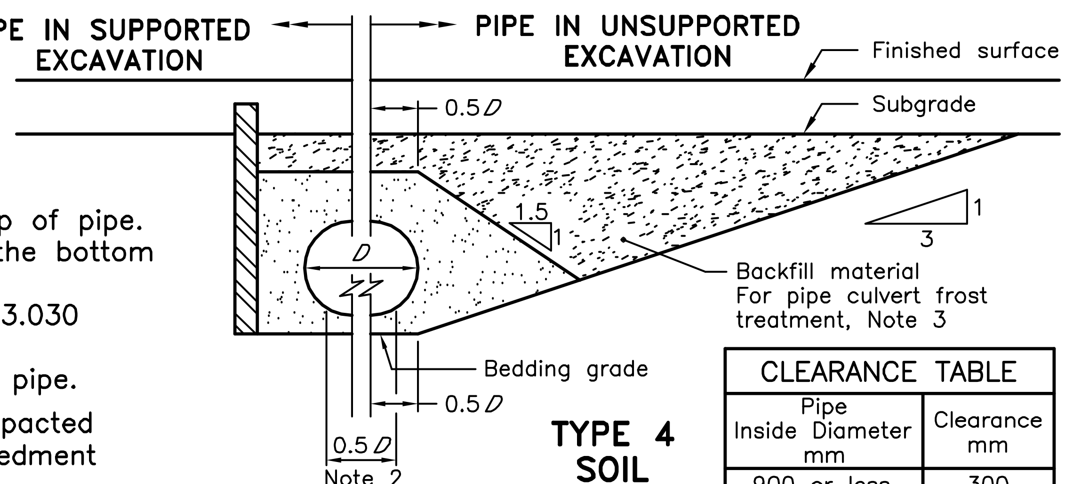
FIGURE E-9 - Detour Embankment - Inlet Side, West Embankment Analysis
Organics Removed Below Proposed Culvert
Culvert 12+415, Highway 129, Reaney Township
DRAINED CONDITIONS

Name: Gravel and Sand   Unit Weight: 21 kN/m³   Cohesion': 0 kPa   Phi': 35 °
Name: Organic Silt   Unit Weight: 17 kN/m³   Cohesion': 0 kPa   Phi': 25 °
Name: Sandy Silt/Silty Sand - Very Loose to Loose   Unit Weight: 19 kN/m³   Cohesion': 0 kPa   Phi': 28 °
Name: Silt-Sandy Silt - Compact to Dense   Unit Weight: 20 kN/m³   Cohesion': 0 kPa   Phi': 30 °
Name: Silty Gravel and Sand Till   Unit Weight: 21 kN/m³   Cohesion': 0 kPa   Phi': 35 °
Name: Granular B Type I   Unit Weight: 21.2 kN/m³   Cohesion': 0 kPa   Phi': 32 °



Appendix F –
Ontario Provincial Standards Drawings





LEGEND:

D – Inside diameter

NOTES:

1 Height of fill is measured from the finished surface to top of pipe.
2 The pipe bed shall be compacted and shaped to receive the bottom of the pipe.
3 Pipe culvert frost treatment shall be according to OPSD 803.030 and 803.031.
4 Condition of excavation is symmetrical about centreline of pipe.

A Granular material placed in the haunch area shall be compacted prior to placing and compacting the remainder of the embedment material.
B Soil types as defined in the Occupational Health and Safety Act and Regulations for Construction Projects.
C All dimensions are in metres unless otherwise shown.

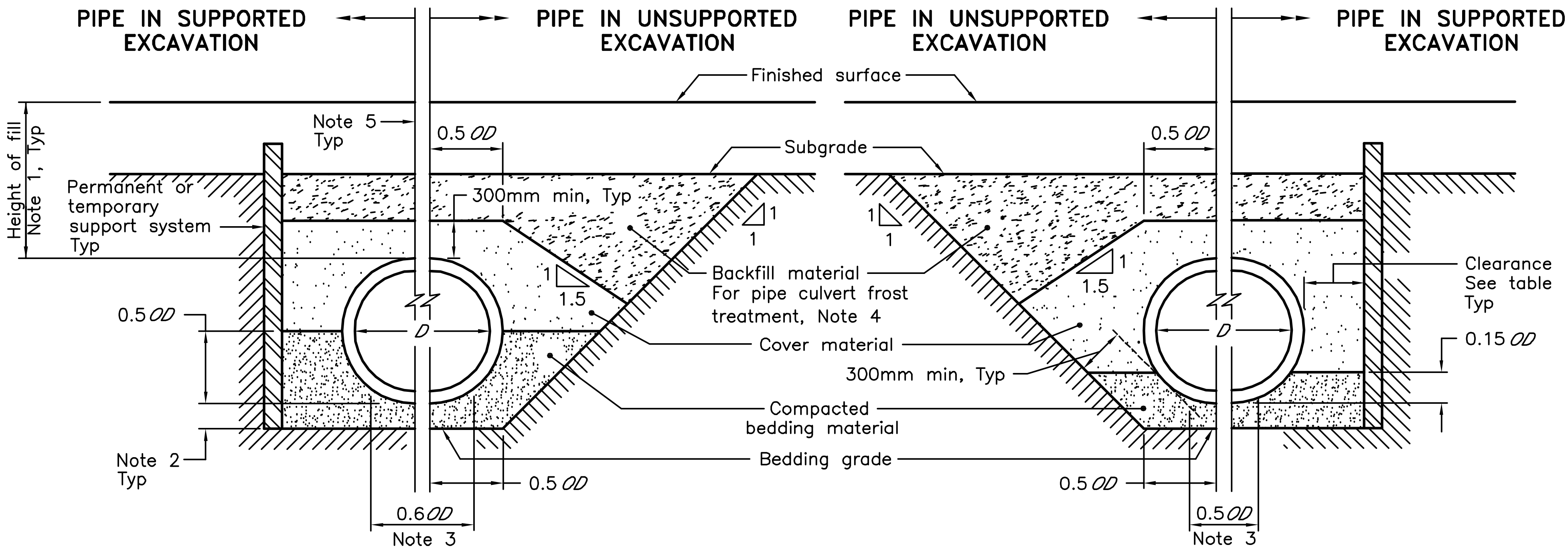
CLEARANCE TABLE

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

ONTARIO PROVINCIAL STANDARD DRAWING | Nov 2014 | Rev 3

FLEXIBLE PIPE
EMBEDMENT AND BACKFILL
EARTH EXCAVATION

OPSD 802.010



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 0.15*D*. In no case shall this dimension be less than 150mm or greater than 300mm.
3 The pipe bed shall be compacted and shaped to receive the bottom of the pipe.
4 Pipe culvert frost treatment shall be according to OPSD 803.030 and 803.031.
5 Condition of excavation is symmetrical about centreline of pipe.

A Soil types as defined in the Occupational Health and Safety Act and Regulations for Construction Projects.

B All dimensions are in metres unless otherwise shown.

LEGEND:

D – Inside diameter
OD – Outside diameter

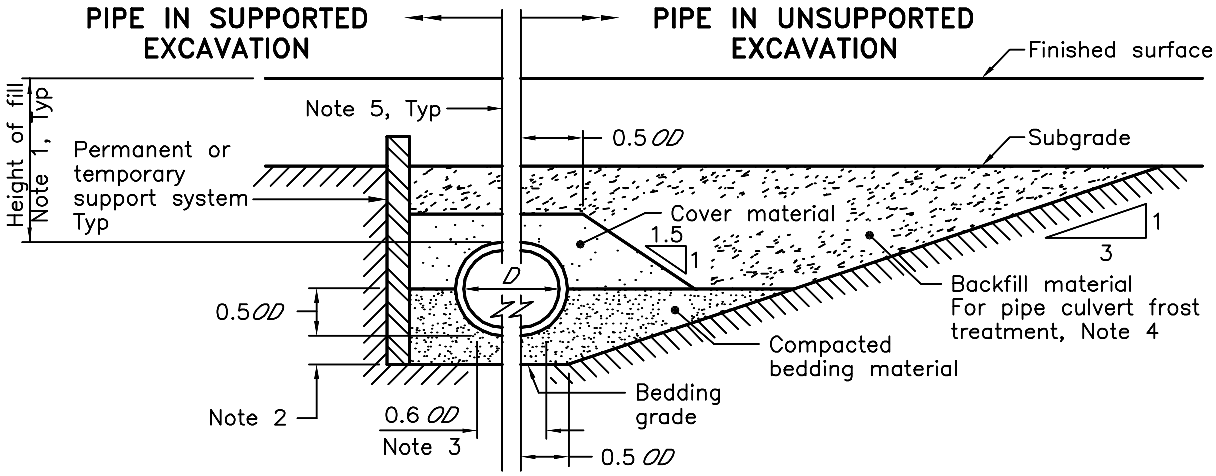
CLEARANCE TABLE	
Pipe Inside Diameter mm	Clearance mm
900 or less	300
Over 900	500

ONTARIO PROVINCIAL STANDARD DRAWING

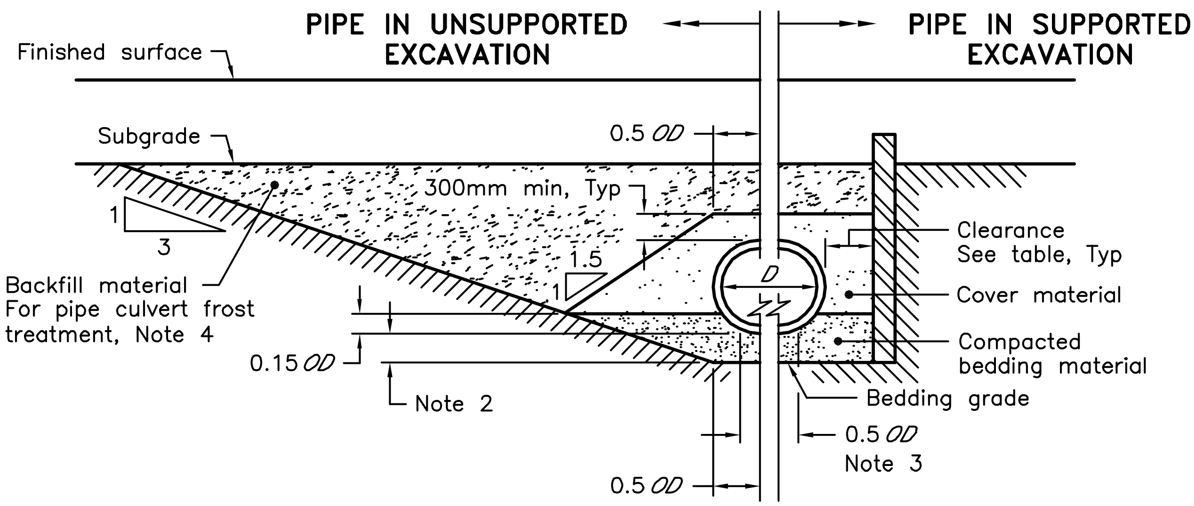
Nov 2010 | Rev 2

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

OPSD 802.031



CLASS B BEDDING



CLASS C BEDDING

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

LEGEND:

D – Inside diameter
OD – Outside diameter

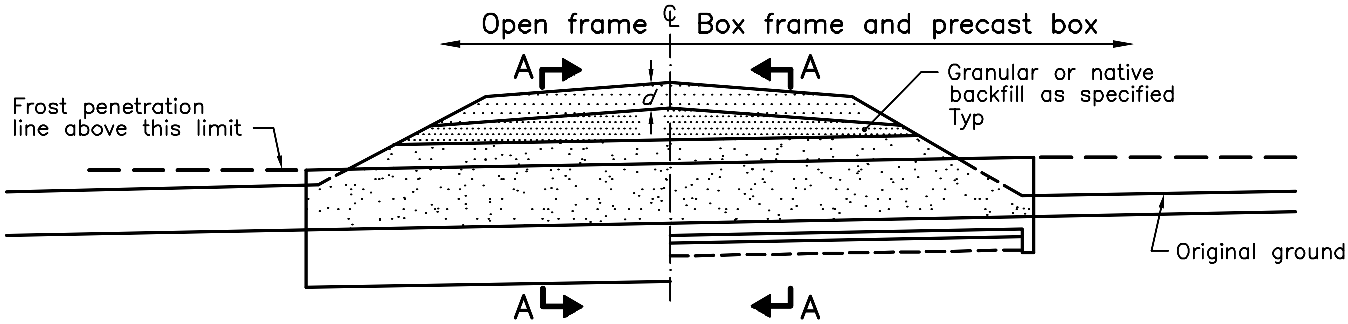
NOTES:

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

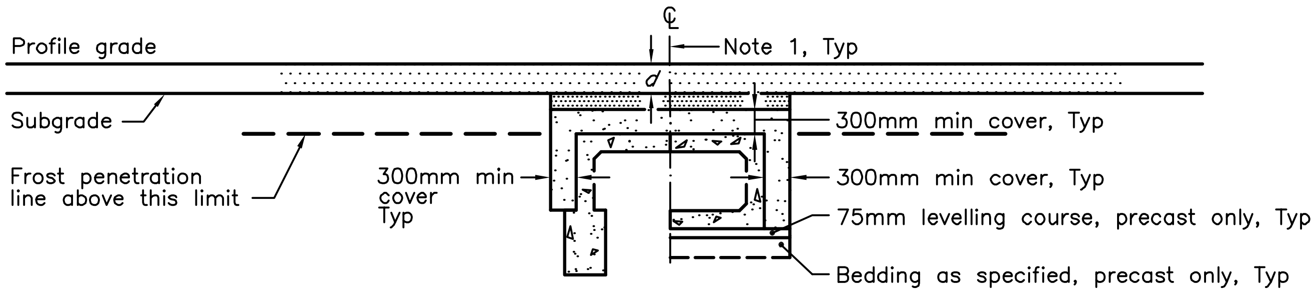
ONTARIO PROVINCIAL STANDARD DRAWING	Nov 2010	Rev 2
RIGID PIPE BEDDING, COVER, AND BACKFILL TYPE 4 SOIL – EARTH EXCAVATION	OPSD 802.032	

ONTARIO STANDARDS · MUNICIPAL – PROVINCIAL · OPS

FROST PENETRATION LINE AT OR ABOVE TOP OF CULVERT

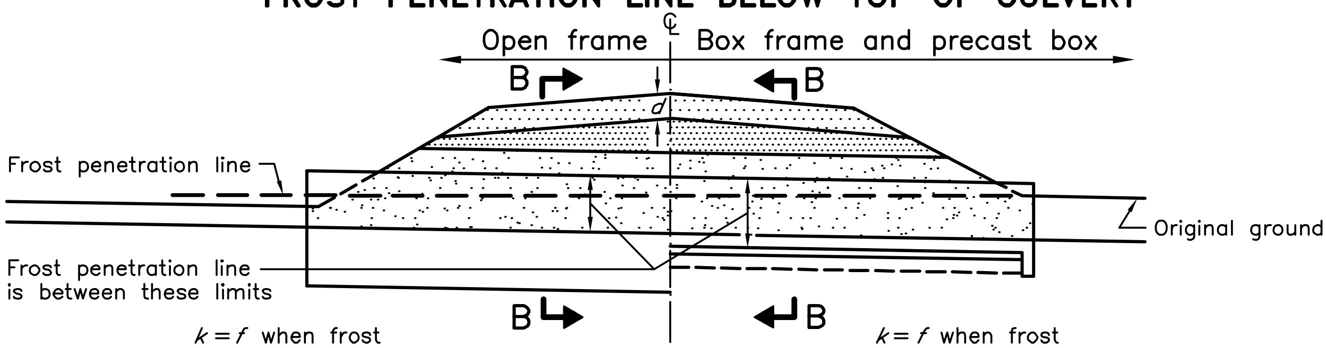


LONGITUDINAL SECTION

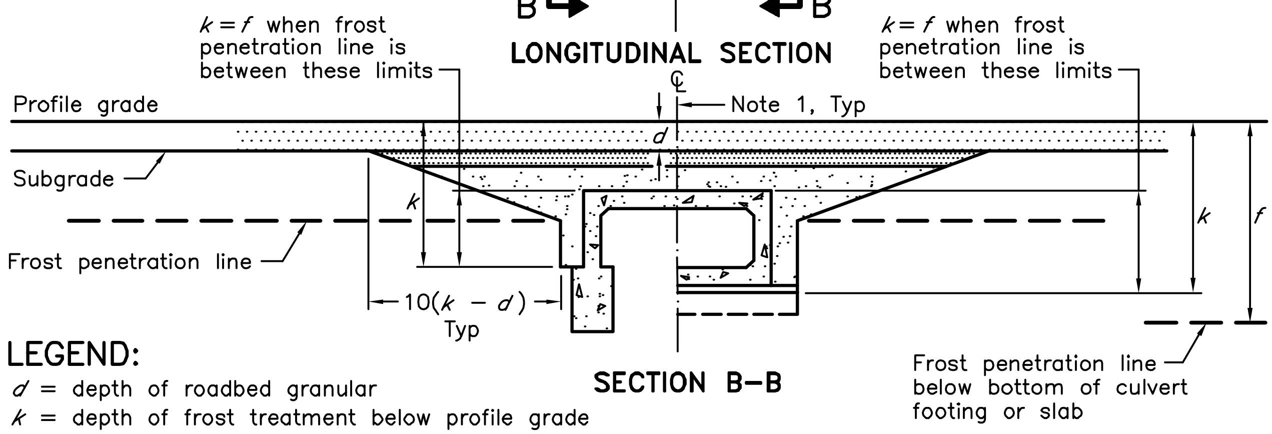


SECTION A–A

FROST PENETRATION LINE BELOW TOP OF CULVERT



LONGITUDINAL SECTION



SECTION B–B

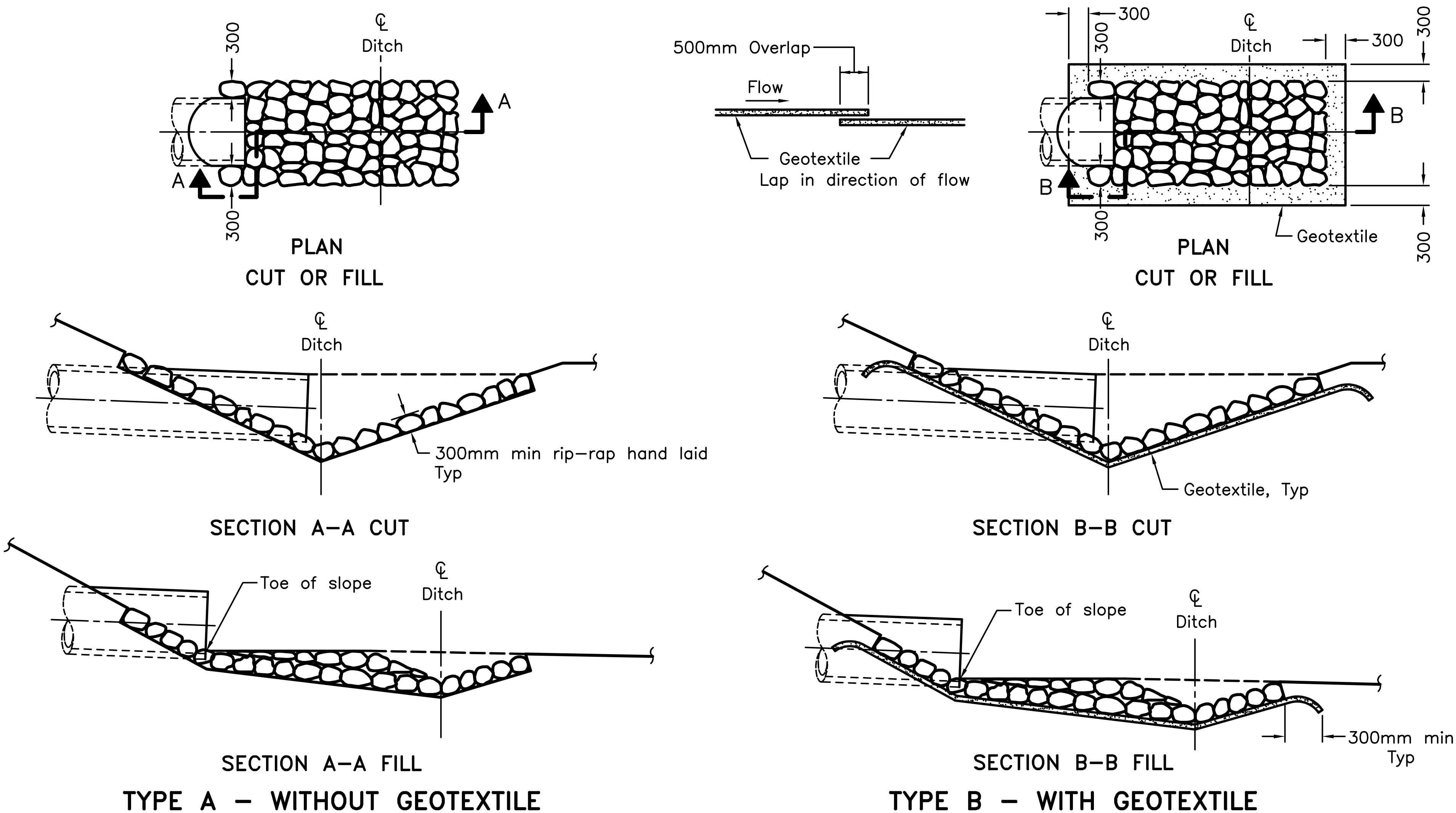
LEGEND:

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

NOTES:

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

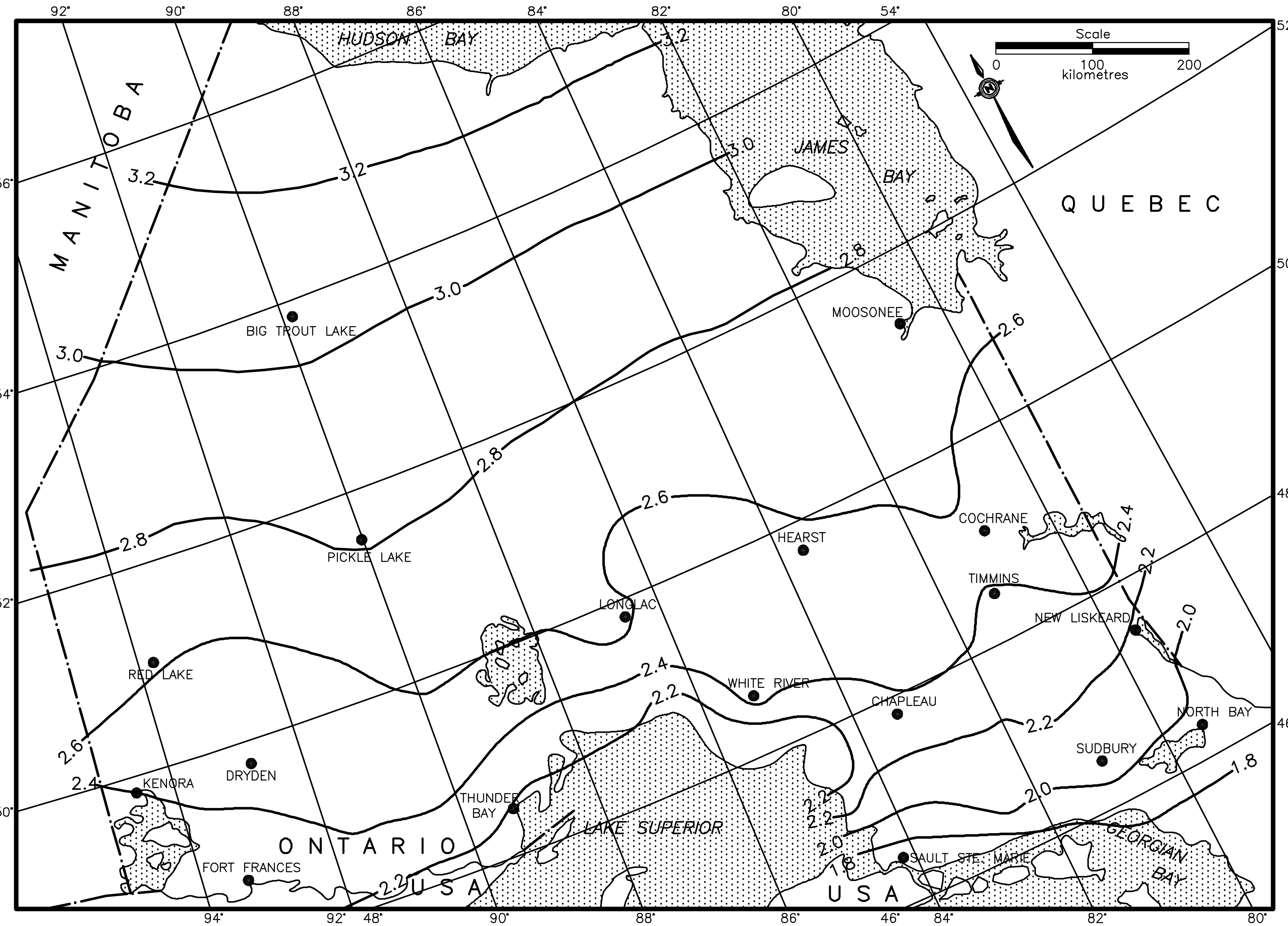
ONTARIO PROVINCIAL STANDARD DRAWING	Nov 2015	Rev 3
BACKFILL AND COVER FOR CONCRETE CULVERTS WITH SPANS LESS THAN OR EQUAL TO 3.0M		
	OPSD 803.010	



NOTES:

A 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.

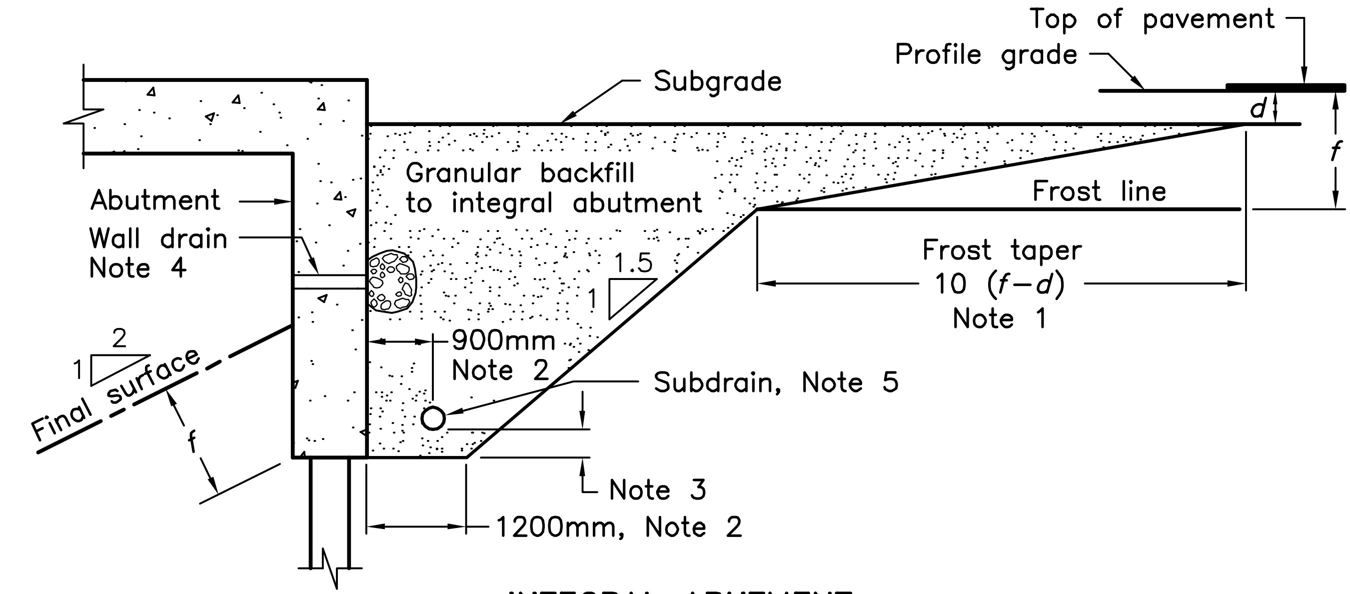
ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2010 | Rev 1

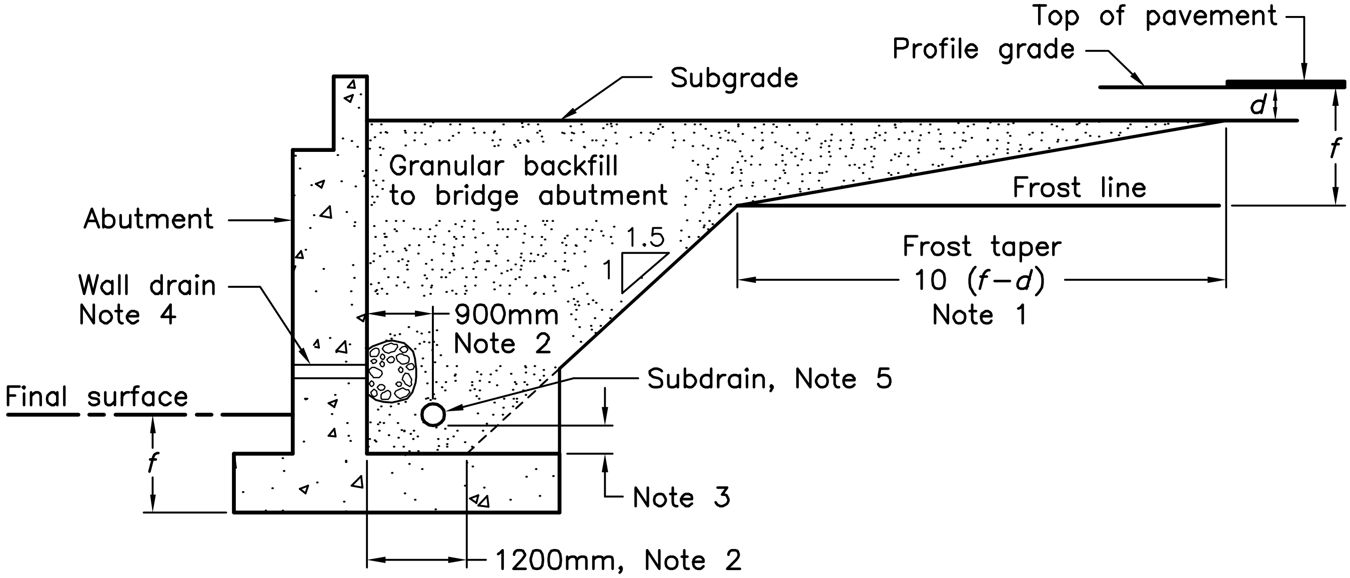
FOUNDATION FROST PENETRATION DEPTHS FOR NORTHERN ONTARIO

ONTARIO STANDARDS MUNICIPAL-PROVINCIAL

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 Provision (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.