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Foundation Investigation
and Design Report

Agreement No. 5016-E-0016

GWP 411-00-00

GEOCRES No. 410-35

**Culvert Replacement, Stn. 19+829
Highway 129, Birch Township,
District of Sudbury**

Prepared For:

**Ministry of Transportation
Northeast Region**

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The Ministry of Transportation

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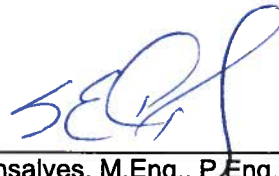
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1 Foundation Investigation Report

1.1 Introduction

This Foundation Investigation Report (FIR) presents the results of a geotechnical investigation completed by **exp** Services Inc. (**exp**) for the replacement of a centreline culvert located on Highway 129 at Station 19+829, within Birch 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 19+829 within Birch Township. The site is located approximately 45 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 1.2 m in diameter and 29.48 m long. At this site, Highway 129 is an asphalt paved, two lane, north/south roadway having approximately 1.0 m wide partially paved shoulders and cable guide rails on both sides of the roadway. The highway embankment at the investigated location is approximately 6.7 m high on the east side of the roadway and 7.7 m high on the west side of the roadway. The embankment side slopes are approximately 1.5H:1V on both sides, from the top to toe of the embankment. Photographs of the site and existing culvert are included in Appendix B.

The general site conditions were assessed on November 16, 2016. The existing waterway flows from the west to the east through the existing culvert. Immediately adjacent to the waterway on both sides of the roadway embankment, the terrain generally consists of low lying vegetation and grasses and a thick forest consisting of both deciduous and coniferous trees.

The side slopes of the highway embankment are covered with large boulder rip-rap with some grass and light vegetation near the crest of the embankment. Guardrails 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 moderate wheel track rutting and moderate transverse, longitudinal, and map cracking.

1.2.2 Geological Setting

In accordance with Ontario Geological Survey Northern Ontario Engineering Geology Terrain Study 86, the dominant landform at the culvert site is ground moraine consisting mainly of till. Local relief is generally moderate (15 to 60 m) and the terrain is generally undulating to rolling. Overall drainage is good (dry). Within Birch Township, rock knobs generally occur within the ground moraine.



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 January 15, 19, and May 3 to 4, 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 the crest of the western embankment. Boreholes BH-2 and BH-3 were advanced at accessible locations near the outlet and inlet, respectively, of the culvert. The borehole locations are shown on Dwg. No. 1 in Appendix A.

Borehole BH-1 was advanced using a truck mounted CME-55 drill rig equipped with hollow stem augers, NW casing, and standard soil sampling equipment. Due to access restrictions, Boreholes BH-2 and BH-3 were advanced with portable tripod mounted equipment with a cathead and Hilti D200 drill. The drilling equipment was operated by a specialist drilling contractor, Landcore Drilling. Each borehole was advanced to approximately 6.0 m below the culvert invert.

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. 19+825. The TBM was assigned an elevation of 464.7 m based on a survey of the site provided to **exp** by the MTO. The borehole and TBM locations are shown on Dwg. No. 1 in Appendix A.

Soil samples were obtained using a 51 mm outside diameter split-spoon sampler in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586) at intervals ranging from 0.75 m to 1.5 m in depth as shown on the attached borehole logs in Appendix C. The original field (uncorrected) SPT "N" values were recorded on the borehole logs and used to provide an assessment of the in-situ compactness condition of encountered cohesionless soils.

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

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

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

1.3.2 Laboratory Testing

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



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

1.4 Subsurface Conditions

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

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

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

1.4.1 Asphalt

Asphalt was encountered at the surface of Borehole BH-1 and was approximately 50 mm thick. Asphalt thicknesses may further vary beyond the borehole location.

1.4.2 Fill Materials

Fill materials were encountered below the asphalt at Borehole BH-1 and extended to approximately 7.6 m depth. Directly below the asphalt was an approximately 1.0 m thick layer of moist gravel and sand fill with cobbles and trace silt. Below the gravel and sand fill was very dense rock fill consisting of cobbles and boulders that extended to 6.9 m depth. The cobbles and boulders ranged in diameter from approximately 0.1 to 0.7 m. A wet, sandy gravel seam was encountered within the rock fill at approximately 5.3 m depth. Underlying the rock fill was an approximately 0.7 m thick layer of silty sand fill with some gravel. The silty sand fill was frozen at the time of the investigation. One SPT was performed within the silty sand fill, resulting in an uncorrected "N" value of 102 blows per 300 mm, classifying the silty sand fill as very dense in compactness condition.

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

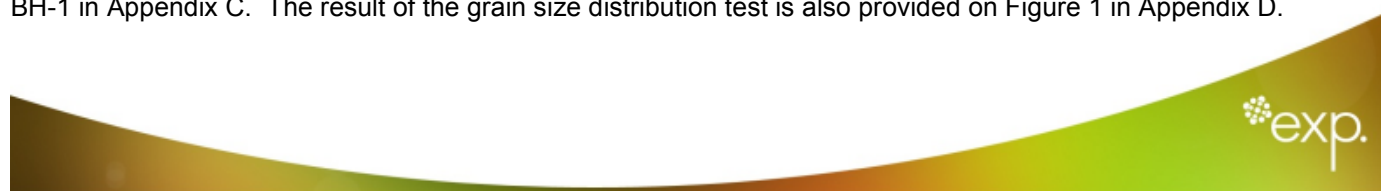
Moisture Content:

- 2 to 20 %

Grain Size Distribution:

- 52 % gravel
- 42 % sand
- 6 % fines

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



1.4.3 Peat

Peat was encountered at the surface of Boreholes BH-2 and BH-3 and was approximately 0.2 to 0.3 m thick. The peat was black in colour, and wet.

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

Moisture Content:

- 36 to 45 %

The results of the moisture content tests are provided on the Record of Borehole Sheets in Appendix C.

1.4.4 Silt

Underlying the fill material at Borehole BH-1 was an approximately 0.7 thick layer of native silt. The silt was grey in colour, and contained some sand and trace organics. One SPT performed within the silt resulted in an uncorrected "N" value of 7 blows per 300 mm, classifying the silt as loose in compactness condition.

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

Moisture Content:

- 25 %

The results of the moisture content test is provided on the Record of Borehole Sheet for BH-1 in Appendix C.

1.4.5 Sand

Underlying the peat at Borehole BH-3 was native sand that extended to approximately 4.6 m depth. The sand was grey to brown in colour, moist to wet, and contained some silt, and trace to some gravel. Uncorrected SPT "N" values within the sand ranged from 14 to 23 blows per 300 mm, classifying the sand as compact in compactness condition.

Laboratory testing performed on selected samples consisted of four (4) moisture content tests and one (1) grain size analysis. The test results are as follows:

Moisture Content:

- 11 to 38 %

Grain Size Distribution:

- 13 % gravel
- 68 % sand
- 19 % silt

The results of the moisture content and grain size distribution tests are provided on the Record of Borehole Sheet for BH-3 in Appendix C. The result of the grain size distribution test is also provided on Figure 2 in Appendix D.



1.4.6 Till

Underlying the silt at Borehole BH-1, the peat at BH-2, and the sand at BH-3 was native till that extended to the termination depth of each borehole. The till ranged in composition from sandy gravel till with trace silt; to gravelly sandy silt till; to sand and silt/silt and sand till with trace to some gravel and trace clay. The till was generally brown to grey in colour and moist to wet. Generally, uncorrected SPT "N" values within the till ranged from 14 to 110 blows per 300 mm, classifying the till as compact to very dense in compactness condition. The upper 0.3 m of the till at BH-2 was generally loose in compactness condition.

Laboratory testing performed on selected samples consisted of sixteen (16) moisture content tests and five (5) grain size analysis. The test results are as follows:

Moisture Content:

- 8 to 18 %

Grain Size Distribution:

- 1 to 66 % gravel
- 26 to 68 % sand
- 9 to 51 % silt
- 0 to 2 % clay

The results of the moisture content and grain size distribution tests are provided on the Record of Borehole Sheets in Appendix C. The results of the grain size distribution tests are also provided on Figure 3 in Appendix D.

1.5 Groundwater and Surface Water Conditions

Groundwater was observed in Borehole BH-1 at approximately 7.3 m depth, Elev. 457.3 m. Note, however, that this water elevation is not likely accurate as water was pumped into the borehole for the washboring techniques utilized. Washboring techniques were also used at BH-2 and BH-3 with the portable equipment utilized, and as such, no groundwater measurements were made in these boreholes. As such, accurate groundwater measurements could not be obtained in the boreholes upon completion.

Note, however, that samples within Borehole BH-1 were generally frozen to wet below 6.9 m depth, Elev. 457.7 m. In addition, samples at BH-2 and BH-3 were generally wet from surface, Elev. 457.1 and 458.0 m, respectively. This could infer a groundwater level located between Elev. 457.0 and 458.0 m.

The water level within the adjacent open water was measured on June 26, 2017 and it was at approximately Elev. 457.3 at the culvert inlet and at Elev. 456.6 m at the culvert outlet. This is generally at a similar level as the wet samples encountered within the boreholes, which also further supports the inference above regarding the groundwater level.

Groundwater would be expected to reflect levels in the adjacent open water and to fluctuate seasonally. 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 19+829, within Birch 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 1.2 m diameter CSP, which is approximately 29.48 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.

2.1.1 Proposed Embankment Widening

In addition to the culvert replacement, it is understood that a 250 m long, 3 to 5 m wide embankment widening is proposed by the MTO along the embankment where the culvert is located (from Stn. 19+770 in Birch Township to Stn. 10+040 in Langlois Township). The widening is proposed for the east side of the embankment. The Foundation Investigation and Design Recommendations for the proposed embankment widening has been addressed by **exp** under a separate report.

Although this FIDR provides recommendations for the culvert replacement independent of the embankment widening, if the embankment widening is to occur, it is recommended that the design and construction of the culvert replacement consider the embankment widening recommendations and construction options as well. As it would likely be more economical and easier to construct both in conjunction with one another.

In addition, if the embankment widening is to occur, a longer culvert or culvert extension beyond the existing 29.48 m long culvert will be required.



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 partially paved shoulders and cable guide rails on both sides of the roadway. The highway embankment at the investigated location is approximately 6.7 m high on the east side of the roadway and 7.7 m high on the west side of the roadway. The embankment side slopes are approximately 1.5H:1V on both sides, from the top to toe of the embankment. The current elevation of the crest of the roadway is approximately 464.5 m.
- The highway embankment consists of approximately 8.4 m of very dense fill materials generally consisting of cobbles and boulders ranging in diameter from approximately 0.1 to 0.7 m. Underlying the embankment was a thin layer (< 1.0 m thick) of loose silt with trace organics, followed by native compact to very dense till which extended to greater than 14.3 m depth (Elev. 450.3 m, borehole termination depth).
- At the existing culvert inlet on the west side of the embankment, a thin layer of peat was encountered at the surface, underlain by 4.3 m of compact native sand and compact native till that extended to greater than 6.7 m depth (Elev. 451.3 m, borehole termination depth). At the outlet on the east side of the embankment, a thin layer of peat was also encountered overlying native till that was loose in the upper 0.3 m becoming compact to dense and extending to greater than 6.7 m depth (Elev. 450.4 m, borehole termination depth).
- The water level within the Creek measured on June 26, 2017 was at approximately Elev. 456.6 m (outlet) to 457.3 (inlet) m. Wet samples within the boreholes were found below Elev. 457.0 m to 458.0 m. As such, an inferred groundwater elevation near Elev. 457.0 m is anticipated. However, the groundwater elevation will likely fluctuate seasonally.

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 1.2 m × 29.48 m CSP culvert is to be removed or decommissioned. In addition, the expected creek and groundwater levels are higher than the current culvert invert. This suggests the need for surface/groundwater control and a cofferdam as discussed in Section 2.9 below.

The new culvert founding level is expected to be similar to the current level (approx. Elev. 456.7 m at outlet). Below this elevation, a thin layer of loose silt and peat was encountered within the boreholes, before generally favourable compact to very dense native sand and till was encountered at approximately Elev. 456.0 to 457.0 m. The existing embankment fills and native soils should be excavated down to this compact to very dense native sand and till, with the grade restored with engineered fill to the proposed culvert founding level. A non-woven geotextile fabric should be placed between the prepared subgrade and any engineered fill to mitigate the migration of fines from adjacent material and to provide a stable base for material placement and compaction.



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	<ul style="list-style-type: none"> • 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 	<ul style="list-style-type: none"> • 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 	<ul style="list-style-type: none"> • Low 	<ul style="list-style-type: none"> • 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	<ul style="list-style-type: none"> • 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 	<ul style="list-style-type: none"> • 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 	<ul style="list-style-type: none"> • Low to medium 	<ul style="list-style-type: none"> • Risk of unacceptable settlements if the entire foundation is not supported on competent soil • Risk of disturbance of base during construction

Options	Rank	Advantages	Disadvantages	Relative Costs	Risk/Consequences
Corrugated Steel Pipe (CSP) Culvert	2	<ul style="list-style-type: none"> • Straightforward construction • Reduced construction period, consequently traffic management and water control period reduced 	<ul style="list-style-type: none"> • Limited service life • Potential for corrosion 	<ul style="list-style-type: none"> • Low to medium 	<ul style="list-style-type: none"> • 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	<ul style="list-style-type: none"> • 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 	<ul style="list-style-type: none"> • Slower construction process • Deeper excavation likely required as footings need to be below frost line • Requires concrete curing 	<ul style="list-style-type: none"> • Medium 	<ul style="list-style-type: none"> • 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 Design Parameters

Culvert Type	Founding Elevation (m)	Assumed Footing Size (m)	Founding Soil Type	Factored Geotechnical Resistance at ULS (kPa)	Geotechnical Reaction at SLS* (kPa)
Rigid frame box culvert Concrete Pipe Culvert Or CSP Pipe Culvert	~ 456.7 m or below	1.2 m	~ 0.5 m compacted engineered fill pad overlying in-situ compact to very dense sand and/or till	400	250
Cast-in-Place Open Footing Concrete Culvert	~ 454.3 m (below frost line)	1.0 m	~ native compact to very dense sand and/or till	450	275

*- For Maximum Settlement of 25 mm

It is assumed that any underlying organic or loose soils will be excavated and replaced with engineered fill (Granular "A" or Granular "B" Type II). 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 ($\tan \delta$) = 0.5
Between Granular "A" and cast-in-place concrete	Coefficient of Friction ($\tan \delta$) = 0.58

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

2.4 Lateral Earth Pressure

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

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

where

p = Lateral earth pressure (kPa)

K = Coefficient of earth pressure

γ = Unit weight of backfill (kN/m³)

γ_w = Unit weight of water (kN/m³)

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 ϕ' (unfactored)	Coefficient of Active Earth Pressure (k_a)	Coefficient of Passive Earth Pressure (k_p)	Coefficient of Earth Pressure at Rest (k_o)	Unit Weight γ (kN/m ³)
Granular “A” (compact)	35°	0.27	3.7	0.43	22.8
Granular “B” Type I (compact)	32°	0.31	3.3	0.47	21.2
Granular “B” Type II (compact)	35°	0.27	3.7	0.43	22
Existing Rock Fill (very dense)	40°	0.22	4.6	0.36	20
Existing Fill Materials (various, very dense)	32°	0.31	3.3	0.47	22

Material	Friction Angle ϕ' (unfactored)	Coefficient of Active Earth Pressure (k_a)	Coefficient of Passive Earth Pressure (k_p)	Coefficient of Earth Pressure at Rest (k_o)	Unit Weight γ (kN/m ³)
Sand (compact)	32°	0.31	3.3	0.47	21
Silt (loose)	28°	0.36	2.8	0.53	18
Till (various, compact to very dense)	35°	0.27	3.7	0.43	21
Peat	17°	0.59	1.7	0.74	12

2.5 Seismic and Liquefaction Potential Consideration

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

From the Natural Resources Canada website, 2015 NBCC seismic hazard values are obtained using the site location coordinates (47.410°N, 83.204°W) and the damped reference spectral accelerations for the project site are $S_a(0.2)=0.029g$, $S_a(0.5)=0.023g$, $S_a(1.0)=0.013g$, $S_a(2.0)=0.0058g$ and the reference peak ground acceleration (PGA) is 0.015g (g = acceleration due to gravity - 9.81 m/s²). These values are associated with an earthquake having 10 percent probability of exceedance in a 50-year period.

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

2.6 Construction Alternatives

As noted previously in Section 2.1.1, in addition to the culvert replacement, it is understood that a proposed embankment widening is being considered for this section of highway. Recommendations for the embankment widening have been addressed by **exp** under a separate report. If the embankment is to be widened, it would likely be appropriate to construct the culvert replacement in conjunction with the embankment widening and as such, appropriate construction alternatives should be selected accordingly.

A number of possible construction alternatives were considered for the culvert, independent of the proposed embankment widening. Due to the rock fill with boulders encountered within the existing embankment, options utilizing temporary shoring to install the culvert, such as driven sheet piles, will not likely be possible at this site. As such, the considered options would need to consist of an open cut type construction.

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 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
 - c. Stage construction by grade lowering of the existing embankment using unsupported excavation to maintain signalized one-way traffic during construction

These methods will require disruption of traffic. For all approaches, provisions must be made to maintain surface water flow to the outlet.

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

Table 2-5: Construction Alternatives for Culvert Replacement

Installation Method	Advantages	Disadvantages	Relative Cost	Ranking
1.a. Full road closure and open cut/unsupported excavation	<ul style="list-style-type: none"> 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 	<ul style="list-style-type: none"> Traffic interruption as road closure required 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 	<ul style="list-style-type: none"> 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	<ul style="list-style-type: none"> • 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 • Can be combined with proposed embankment widening, with widened area utilized as the detour 	<ul style="list-style-type: none"> • 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 	<ul style="list-style-type: none"> • 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 	1.

Installation Method	Advantages	Disadvantages	Relative Cost	Ranking
1.c. Stage construction by grade lowering	<ul style="list-style-type: none"> • One lane of traffic flow maintained during construction • Straightforward construction • No detour required • No shoring required 	<ul style="list-style-type: none"> • Traffic slowdown and congestion due to traffic lane changes • Large amount of embankment fills to be excavated and replaced • Excavation will likely extend upwards of 160 m longitudinally to maintain 10H:1V slope • Need to temporarily control existing creek water and groundwater • Additional cost for reconstruction of road • Increased time for construction of staging • Erosion Control of temporary cuts required 	<ul style="list-style-type: none"> • High expense due to large excavation and reconstruction required • Moderate risk of cost overrun due to complexity of roadway protection system 	2.

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

1. Option 1.b. – Temporary Detour Construction Followed by Open Cut/Unsupported Excavation
2. Option 1.c. – Stage Construction by Grade Lowering
3. Option 1.a. – Full Road Closure Followed by Open Cut/Unsupported Excavation

The following sections discuss these options in more detail.

2.6.1 Open Cut/Unsupported Excavations (Options 1.a to 1.c)

Each option allows 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, three 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
- c. Stage construction by Grade Lowering

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, nor will a large excavation be required to maintain traffic flow.

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 existing embankment is envisioned by the MTO to also be widened on the east side, it is likely that the east side would be the preferred option for the detour. Compacted engineered fill for construction of the temporary detour road is recommended. Prior to construction of the temporary detour embankment, the site will need to be cleared and grubbed of any existing bushes and vegetation. All surficial topsoil, organics, and softened or loosened soil should be stripped from below the proposed temporary detour road embankment. All subgrade soils should be proofrolled prior to fill placement and embankment fill should be placed in accordance with OPSS. PROV 206 (dated November 2014).

If the embankment is to be widened on the east side as proposed, the widened section can likely be used as the detour for traffic during the culvert construction, eliminating the need for a temporary embankment.

2.6.1.3 Option 1.c. – Stage Construction by Grade Lowering

The stage construction by grade lowering alternative, Option 1.c., would involve several stages of excavation of the existing road while maintaining constant one-way traffic flow. This method does not require a road protection system or detours; however, it is very disruptive to the highway due to the large overall excavation required to maintain traffic flow and the various lane changes that will be required. For this option, one lane is excavated to a maximum depth of approximately 1.75 m, while one-way traffic is maintained on the opposite lane. A temporary excavation sideslope of 1H:1V is maintained between lanes. Once cut down to the maximum excavation depth, the one-way traffic is diverted to the excavated lane, and excavation proceeds at the opposite lane. Prior to diverting traffic, a surficial layer of Granular "A" (~ 100 mm thick) should be placed and compacted on the excavated lane to allow for vehicle traffic over the rock fill embankment materials. This excavation continues to a maximum depth of approximately 1.75 m below the current travelled lane, while maintaining the 1H:1V excavation sideslope between lanes. Once complete, the process is reversed and continues until the culvert is reached. The old culvert is removed and the new culvert is then constructed in a similar fashion, by completing one side while one-way traffic is maintained on the opposite side. Once the new culvert is installed, the embankment is then reconstructed while alternating one-way traffic to each side of the embankment. In order to maintain traffic, the roadway has to be excavated longitudinally at a maximum sideslope of 10H:1V. This can result in an excavation of over 160 m in length. A general schematic diagram has been included in Appendix H for reference purposes outlining the general procedure for stage construction by grade lowering.



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 Groundwater and Surface Water Control

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

Diversion of the creek will be required during the culvert construction. Appropriate permitting and approvals must be in place for this work (i.e. 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.

2.9 Engineered Fill and Culvert Bedding

All existing fill, loose native soils, and organics should be excavated from below the proposed culvert and replaced with an engineered fill pad. Prior to placing any fill material, the exposed native subgrade should be inspected in accordance with OPSS 902. The engineered fill pad should be a minimum of 500 mm thick, and consist of 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 is to be placed between the approved subgrade and the engineered fill pad to assist in material placement and to maintain the integrity of the founding soil along the entire length of the culvert. The geotextile separator is to be a Class II non-woven material with an equivalent opening size of 75-150 µm.

Bedding requirements for the various culvert materials are outlined on OPSD 802.014, 802.034, and 803.010, which are included in Appendix F. The culvert bedding should consist of Granular "A" (OPSS.PROV. 1010) with a thickness of 500 mm beneath the culvert.



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

2.10 Culvert Cover and Backfill

Culvert cover and backfill requirements for the various culvert materials are outlined on OPSD 802.014, 802.034, 803.010, and 3101.150 which are included in Appendix F. Cover material should consist of Granular "A" (OPSS.PROV 1010) with a minimum of 500 mm (compacted). A non-woven geotextile separator is to be placed between the cover material and embankment fill. The geotextile separator is to be a Class II non-woven material with an equivalent opening size of 75-150 µm.

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, the embankment should be reconstructed with rock fill with side slopes similar to existing at 1.5H:1V.

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 roadbed base and sub-base courses (for pavement) should be compacted to 100% of the material's SPMDD. Embankment construction with rock fill shall be in accordance with OPSS.PROV 206.

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

2.11 Frost Protection

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

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

At the culvert inlet and outlet, and beneath the proposed culvert, the native soils will likely consist of the encountered native sand or till soils. These materials have generally a low to moderate frost susceptibility based upon the MTO Frost Classification guideline of percent particles between 5 to 75 µm. As 300 to 500 mm of engineered fill bedding and cover will be placed around the culvert, the soils next to the culvert should not freeze.

For open footing culverts, 2.4 m of earth cover frost protection should be provided for the culverts. If 2.4 m of earth cover frost protection cannot be provided, consideration may be given to utilizing insulation below the footings to 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.



2.12 Embankment Design

2.12.1 Stability Analysis

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

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

Based on the borehole information, the embankment fills and subsoils generally consist of cohesionless soil deposits. As such, an effective stress analysis for long term stability assessment was performed. For the analyses, it is assumed that the proposed embankments will be constructed with cobble/boulder fill similar to existing.

The various analyses performed include the following. The SLOPE/W graphical printout for each analysis is shown on the noted figure in Appendix E. As the soils conditions for a detour embankment on the east side will be similar to those shown on Figure E-4, a slope analysis for the detour embankment is not presented.

- Figure E-1 – Existing Embankment Stability – Inlet Side
- Figure E-2 – Existing Embankment Stability – Outlet Side
- Figure E-3 – Proposed Embankment Stability – Inlet Side
- Figure E-4 – Proposed Embankment Stability – Outlet Side

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		
	ϕ'	c' (kPa)	γ (kN/m ³)
Existing Fill Materials (various, very dense)	40°	0	20
New Rock Fill (compact to dense)	40°	0	20
Peat	17°	2	12
Sand (compact)	32°	0	21
Silt (loose)	28°	0	18

Soil Type	Long Term Conditions		
	ϕ'	c' (kPa)	γ (kN/m ³)
Till (various, compact to very dense)	35°	0	21

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.

Table 2-7: Summary of Slope Stability Analysis Results

Figure No.	Analysis	Factor of Safety
E-1	Existing Embankment Stability – Inlet Side	1.747
E-2	Existing Embankment Stability – Outlet Side	1.607
E-3	Proposed Embankment Stability – Inlet Side	2.013
E-4	Proposed Embankment Stability – Outlet Side	1.691

2.12.2 Embankment Settlement

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

2.13 Inlet and Outlet

2.13.1 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 μ m. The rip-rap configuration at the creek bed should generally follow the OPSD 810.010, which is included in Appendix F of this report.

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

2.13.2 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 $0.75 \times (\text{Liquid Limit} - 20\%)$.
- The clay seal is to be placed in maximum 150 mm thick lifts and compacted to 95% SPMDD within 2% of the optimum moisture content.

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

2.14 Obstructions

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



3 Closure

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

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

A subsurface investigation is a limited sampling of a site. The subsurface conditions have been established only at the test hole locations noted. Should any conditions at the site be encountered that differ from those reported at the test locations, we require that we be notified immediately in order to allow reassessment of our recommendations. It may then be necessary to perform additional investigation and analysis.

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

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

Yours truly,

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4 Limitations and Use of Report

Basis of Report

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

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

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

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

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

Reliance on Information Provided

The evaluation and conclusions contained in the Report are based on conditions in evidence at the time of site inspections and information provided to **exp** by the Client and others. The Report has been prepared for the specific site, development, building, design or building assessment objectives and purpose as communicated by the Client. **Exp** has relied in good faith upon such representations, information and instructions and accepts no responsibility for any deficiency, misstatement or inaccuracy contained in the Report as a result of any misstatements, omissions, misrepresentation or fraudulent acts of persons providing information. Unless specifically stated otherwise, the applicability and reliability of the findings, recommendations, suggestions or opinions expressed in the Report are only valid to the extent that there has been no material alteration to or variation from any of the information provided to **exp**.



Standard of Care

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

Complete Report

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

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Report Format

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



Appendix A – Drawings



19+800

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



Agreement No. 5016-E-0016
GWP 411-00-00
GEOCRES No. 410-35

CULVERT REPLACEMENT, STN. 19+829
HIGHWAY 129, BIRCH TOWNSHIP
DISTRICT OF SUDBURY
**BOREHOLE LOCATION PLAN AND SOIL
STRATA**

SHEET

1

exp. exp Services Inc.

KEY PLAN - NTS



LEGEND

- BOREHOLE LOCATION
- STANDARD PENETRATION TEST (BLOWS/300mm)
- TEMPORARY BENCHMARK (EL. 484.7 m)
- ESTIMATED WATER LEVEL IN BOREHOLE

BOREHOLE COORDINATES

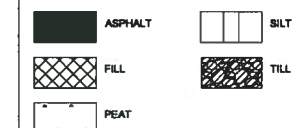
BOREHOLE NO.	APPROX. ELEV. (m)	MTM COORDINATES	
		NORTHING	EASTING
BH-1	484.6	5252554.9	364844.3
BH-2	457.1	5252558.9	364888.6
BH-3	458.0	5252545.5	364829.1

NOTES

This drawing is for subsurface information only. The proposed structure details/work 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



REVISIONS

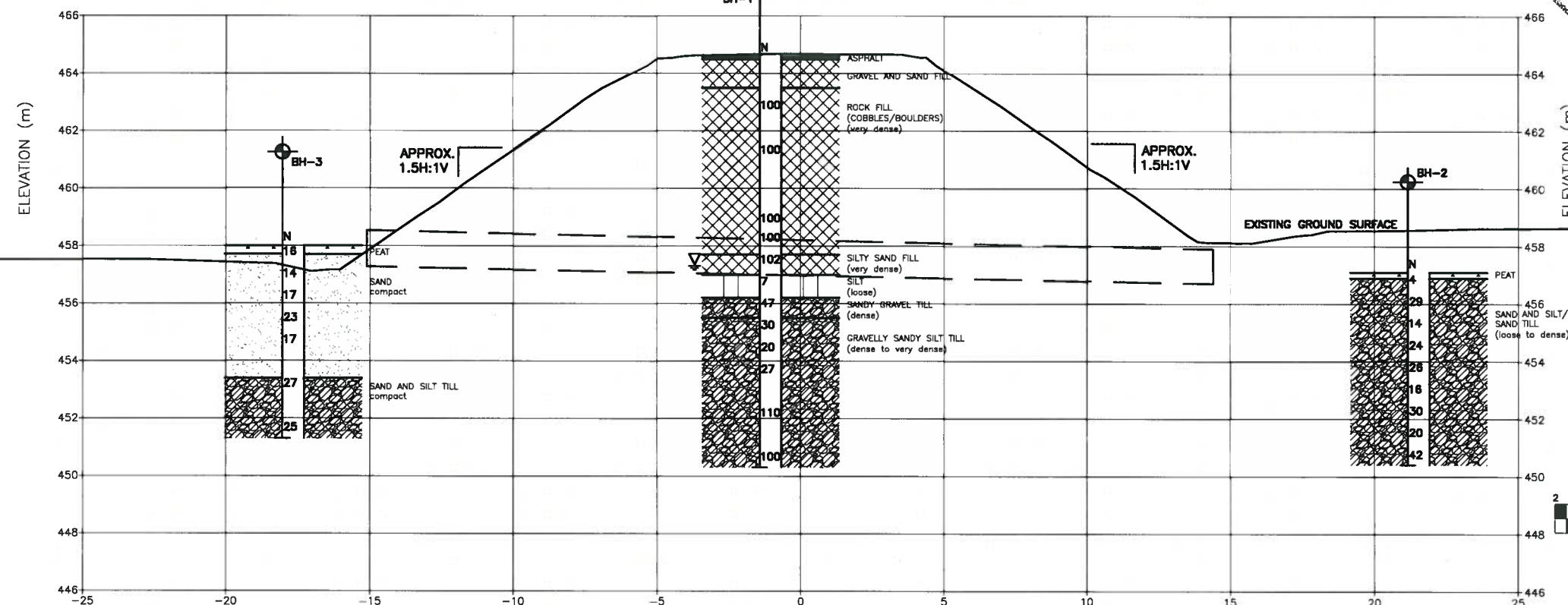
DATE	BY	DESCRIPTION
2017.10.04	IM	SUBMISSION FOR MTO REVIEW
2017.11.28	IM	FINAL REPORT SUBMISSION

SCALE: AS NOTED PROJECT NO.: SUD-00014543-AG

SUBMD: IM	CHECKED: AS	DATE: 2017.10.04
DRAWN: IM	CHECKED: SG	APPROVED: SG DWG. 1

PLAN

HIGHWAY 129



CROSS SECTION A-A AT STN. 19+825

Appendix B – Photographs





Photograph No. 1 – Highway 129 at Culvert, Stn. 19+829 (Facing North)



Photograph No. 2 – West Embankment (Facing North)



Photograph No. 3 – West Embankment at Culvert (Facing North-West)



Photograph No. 4 – Culvert Inlet (Facing West)



Photograph No. 5 – East Embankment (Facing North)



Photograph No. 6 – East Embankment at Culvert (Facing North-East)



Photograph No. 7 – Culvert Outlet (Facing East)

Appendix C – Borehole Logs

Explanation of Terms Used on Borehole Records

SOIL DESCRIPTION

Terminology describing common soil genesis:

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

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

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

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

Terminology describing soil structure:

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

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

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

Fissured: material breaks along plane of fracture.

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

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

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

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

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

Homogeneous: same color and appearance throughout.

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

Uniformly Graded: predominantly on grain size.

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

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

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

Table a: Percent or Proportion of Soil, Pp

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

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

Table b: Apparent Density of Cohesionless Soil

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

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

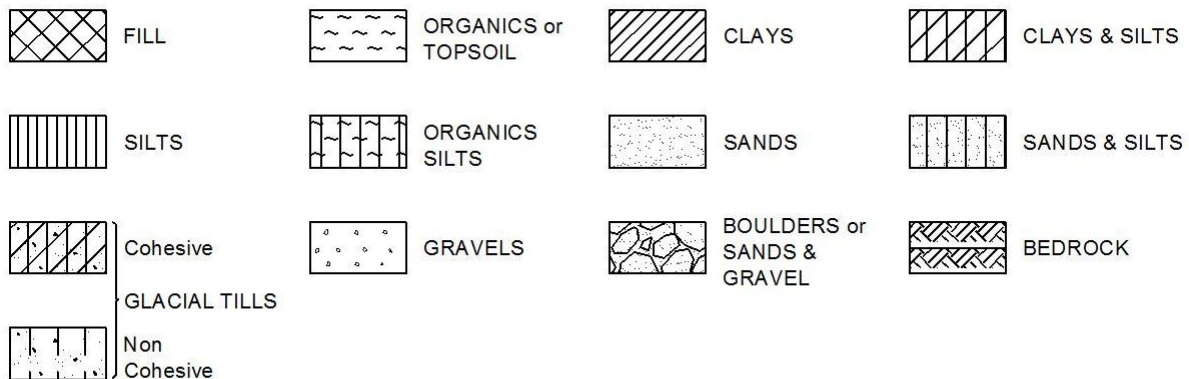
Table c: Consistency of Cohesive Soil

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

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

STRATA PLOT

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



WATER LEVEL MEASUREMENT



Open Borehole or Test Pit



Monitoring Well, Piezometer or Standpipe

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

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

STRESS AND STRAIN

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

MECHANICAL PROPERTIES OF SOIL

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

PHYSICAL PROPERTIES OF SOIL

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

METRIC

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

RECORD OF BOREHOLE No BH-2

1 OF 1

METRIC

W.P. 411-00-00,5016-E-0016 LOCATION Stn. 19+838, MTM-13, 5252558.89N, 364866.56E, Non-Structural Culvert at Stn. 19+829 ORIGINATED BY ST
DIST Sudbury HWY 129 BOREHOLE TYPE Portable Tripod With Cathead and Hilti D200 Drill COMPILED BY IM
DATUM Geodetic DATE 2017.01.15 - 2017.01.15 LATITUDE 47.409535 LONGITUDE -83.204049 CHECKED BY IM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								
								○ UNCONFINED	+	FIELD VANE	×					
457.1	Ground Surface						20	40	60	80	100					
450.0	PEAT, black, wet.		1	SS	4											
0.2	TILL, sand and silt to silt and sand, brown, wet, loose.															
	some gravel, trace clay, compact to dense below ~ 0.8 m depth.		2	SS	29											
	brown to grey below ~ 1.5 m depth.		3	SS	14											15 47 37 2
			4	SS	24											
	grey below ~ 3.1 m depth.		5	SS	26											
			6	SS	16											12 37 51 1
			7	SS	30											
			8	SS	20											
			9	SS	42											
450.4	END OF BOREHOLE Borehole terminated at ~ 6.7 m depth.															
6.7	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.															

RECORD OF BOREHOLE No BH-3

1 OF 1

METRIC

W.P. 411-00-00,5016-E-0016 LOCATION Stn. 19+826, MTM-13, 5252545.50N, 364829.11E, Non-Structural Culvert at Stn. 19+829 ORIGINATED BY ST
DIST Sudbury HWY 129 BOREHOLE TYPE Portable Tripod With Cathead and Hilti D200 Drill COMPILED BY IM
DATUM Geodetic DATE 2017.01.19 - 2017.01.19 LATITUDE 47.409418 LONGITUDE -83.204547 CHECKED BY IM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		W _p	W	W _L					
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	WATER CONTENT (%)								
458.0	Ground Surface							20	40	60	80	100					
0.0	PEAT, black, wet.		1	SS	16												
457.7	SAND, some silt, grey, wet, compact. brown, trace to some gravel below ~ 0.8 m depth.		2	SS	14		457										
0.3			3	SS	17		456										
			4	SS	23		455										
			5	SS	17		454										
							453										
453.4	TILL, sand and silt, trace gravel, trace clay, grey, wet, compact.		6	SS	27		452										
4.6																	
			7	SS	25												
451.3	END OF BOREHOLE Borehole terminated at ~ 6.7 m depth.																
6.7	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.																

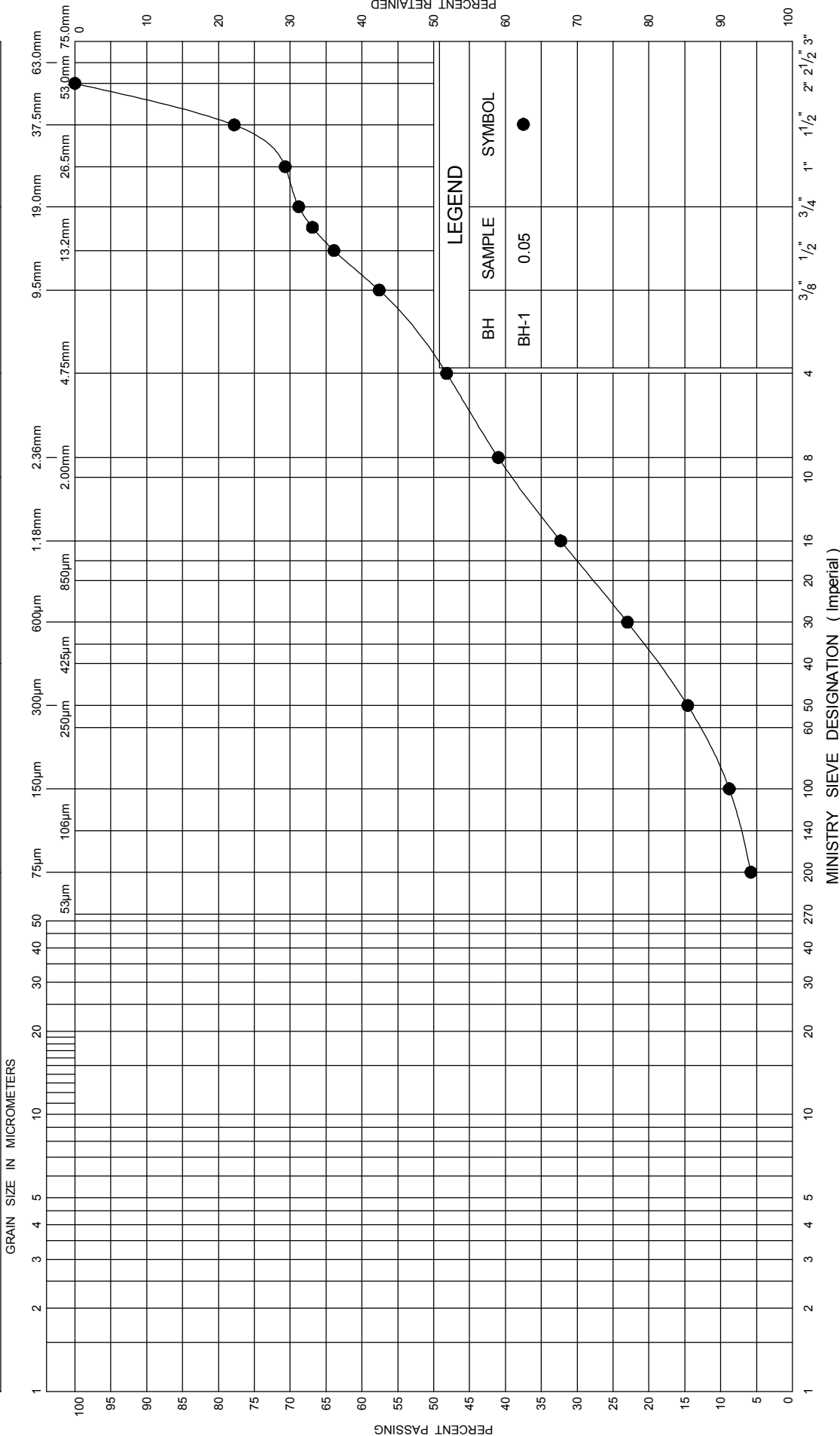
+³, X³: Numbers refer to Sensitivity O 3% STRAIN AT FAILURE

Appendix D – Laboratory Test Results



UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT			SAND			GRAVEL		
Grain Size in Micrometers			Fine			Medium		
			Fine			Coarse		



GRAIN SIZE DISTRIBUTION

Gravel and Sand Fill

FIG No 1

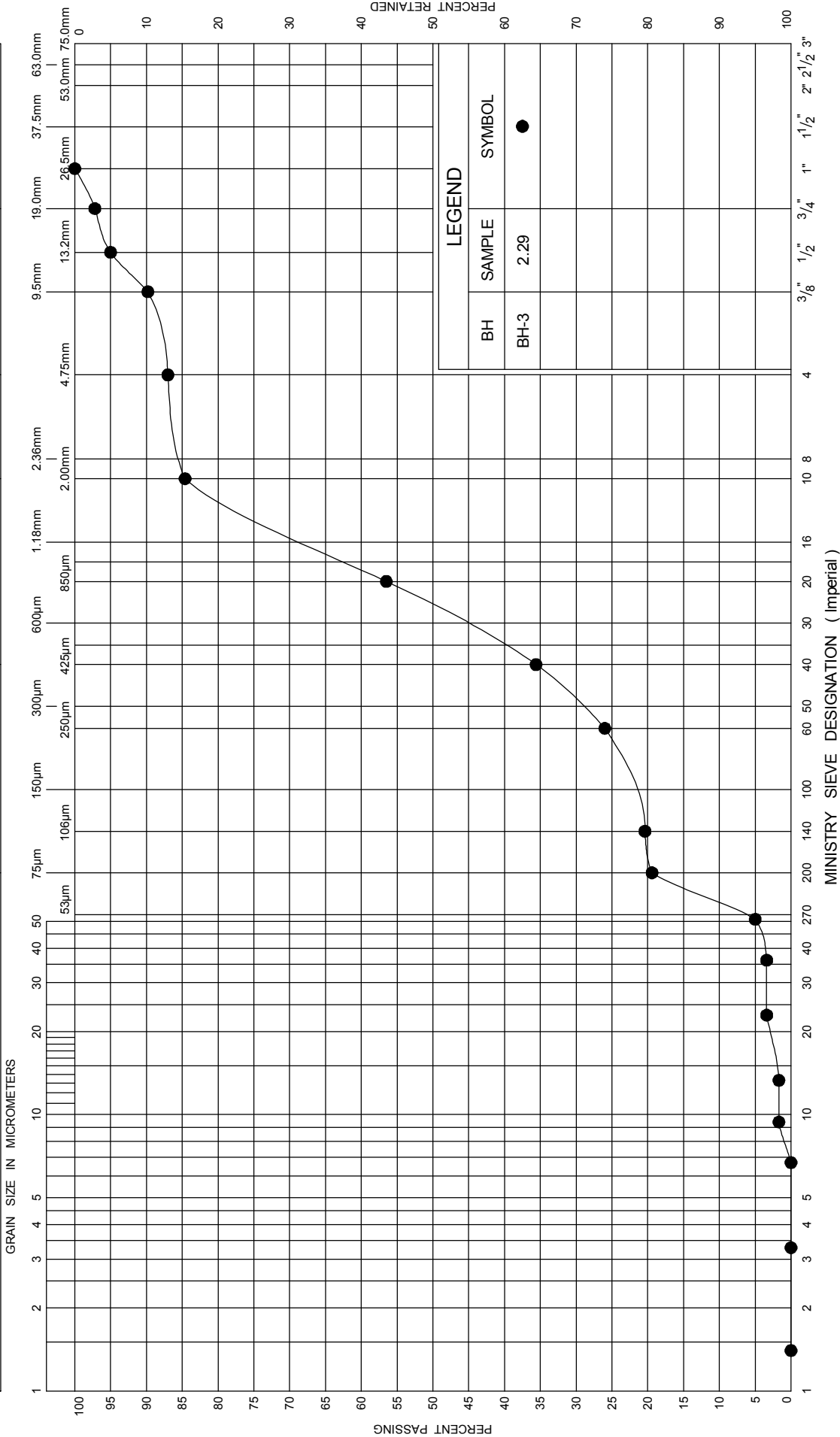
W P 411-00-00,5016-E-0016

Culvert Replacement



UNIFIED SOIL CLASSIFICATION SYSTEM

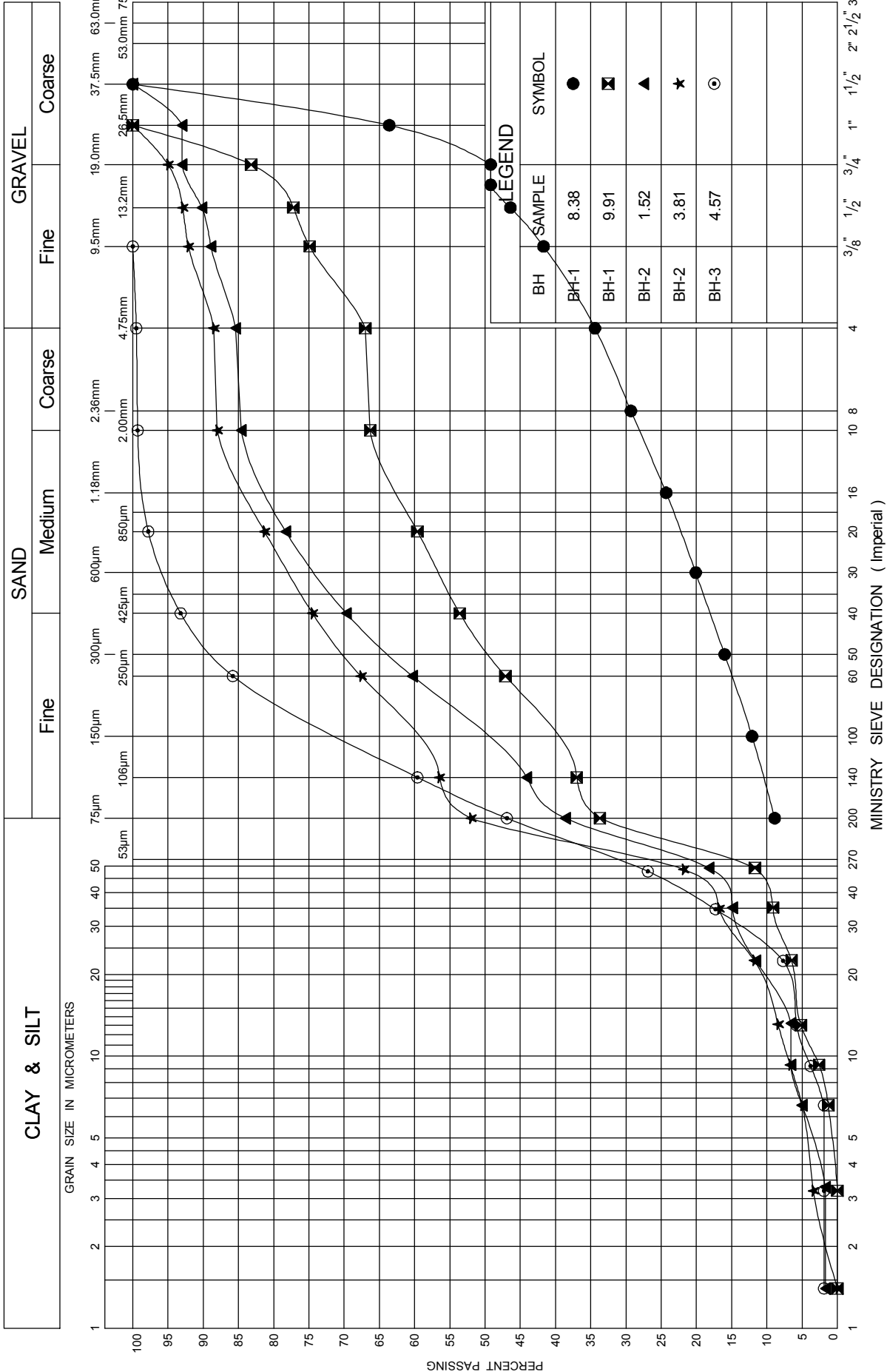
CLAY & SILT		SAND			GRAVEL		
		Fine		Medium	Coarse	Fine	Coarse



GRAIN SIZE DISTRIBUTION

Sand

UNIFIED SOIL CLASSIFICATION SYSTEM



GRAIN SIZE DISTRIBUTION

FIG No 3

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

Culvert Replacement

Ministry of
Transportation



Till

Appendix E – Slope Stability Analyses



FIGURE E-1 - Existing Embankment Stability - Inlet Side
Culvert 19+829, Highway 129, Birch Township
Drained Conditions

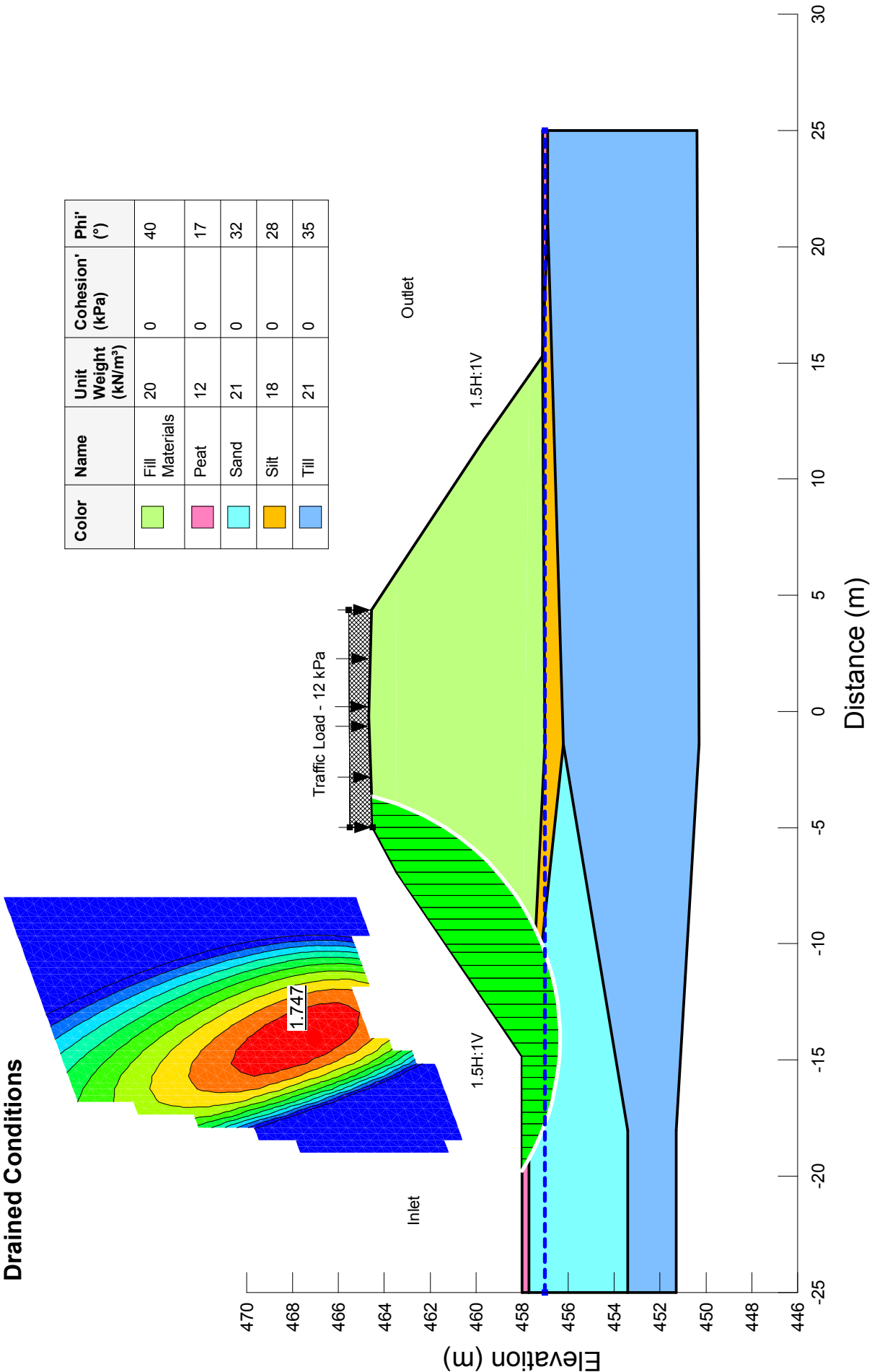


FIGURE E-2 - Existing Embankment Stability - Outlet Side
Culvert 19+829, Highway 129, Birch Township
Drained Conditions

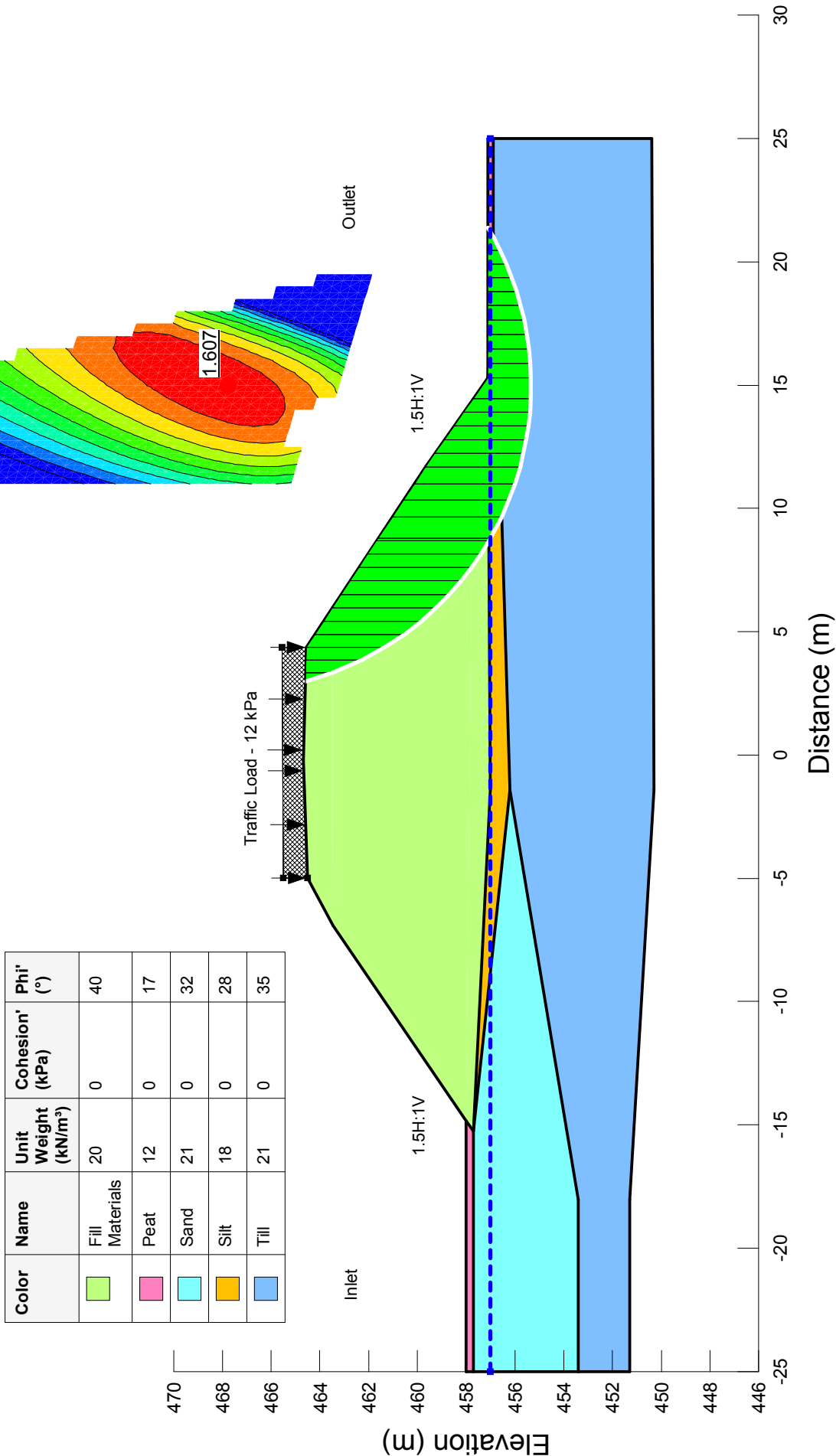


FIGURE E-3 - Proposed Embankment Stability - Inlet Side
Culvert 19+829, Highway 129, Birch Township
Drained Conditions

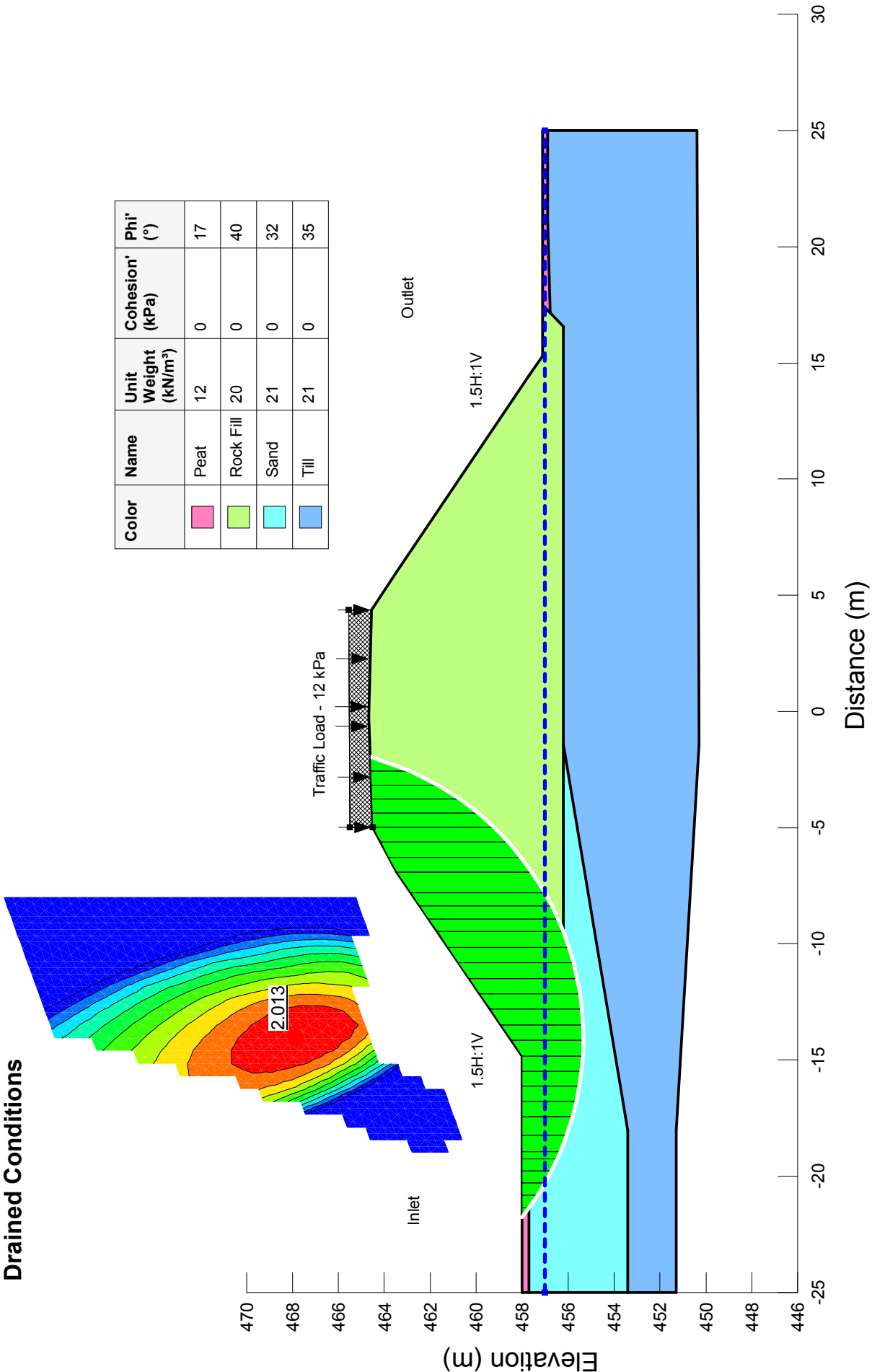
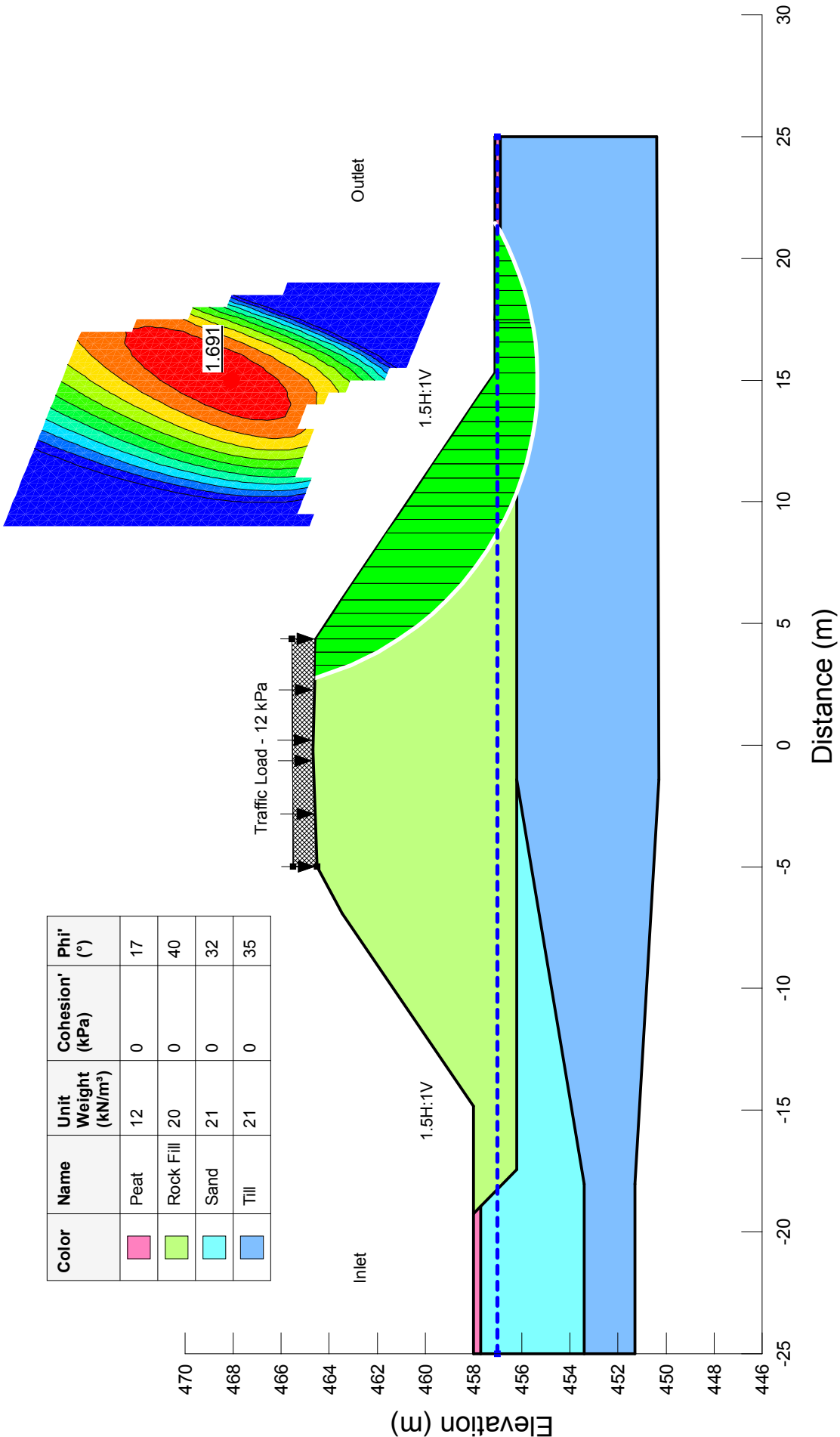
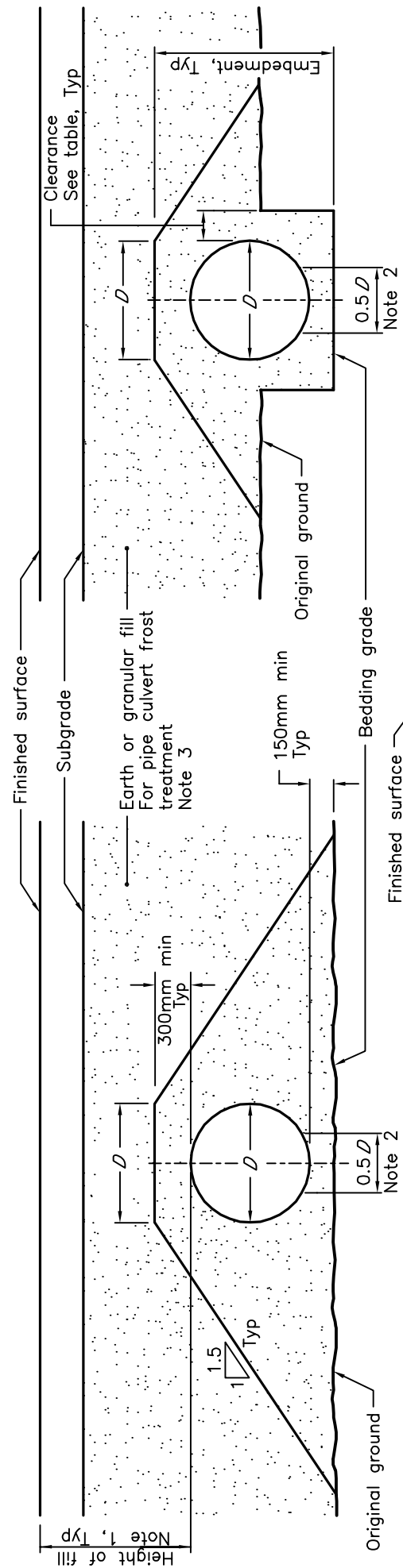


FIGURE E-4 - Proposed Embankment Stability - Outlet Side
Culvert 19+829, Highway 129, Birch Township
Drained Conditions



Appendix F – Ontario Provincial Standards Drawings (OPSD)





PIPE INVERT ABOVE ORIGINAL GROUND

PIPE INVERT AT OR BELOW ORIGINAL GROUND

LEGEND:
 D - Inside diameter

PIPE EMBEDMENT WITH ROCK FILL UNDER AND OVER THE PIPE

- NOTES:**
- 1 Height of fill is measured from the finished surface to top of pipe.
 - 2 The pipe bed shall be compacted and shaped to receive the bottom of the pipe.
 - 3 Pipe culvert frost treatment shall be according to OPSD 803.030 and 803.031.
 - 4 Embedment material shall be wrapped in non-woven geotextile when specified.

- A** Granular material placed in the haunch area shall be compacted prior to placing and compacting the remainder of the embedment material.
- B** All dimensions are in metres unless otherwise shown.

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



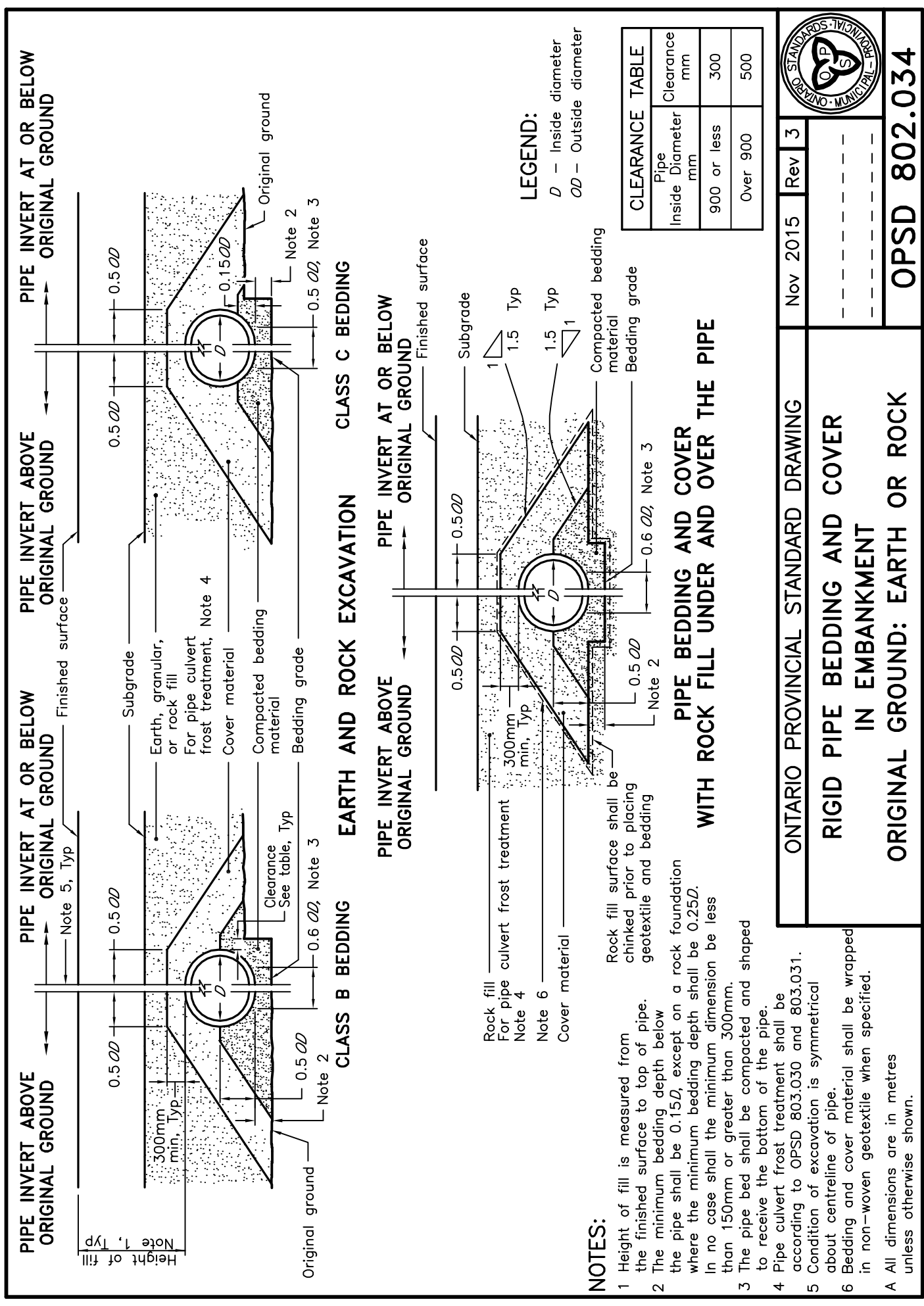
Nov 2014	Rev 3
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ONTARIO PROVINCIAL STANDARD DRAWING

FLEXIBLE PIPE EMBEDMENT IN EMBANKMENT

ORIGINAL GROUND: EARTH OR ROCK

OPSD 802.014



NOTES:

- 1 Height of fill is measured from the finished surface to top of pipe.
- 2 The minimum bedding depth below the pipe shall be 0.15D, except on a rock foundation where the minimum bedding depth shall be 0.25D. In no case shall the minimum dimension be less than 150mm or greater than 300mm.
- 3 The pipe bed shall be compacted and shaped to receive the bottom of the pipe.
- 4 Pipe culvert frost treatment shall be according to OPSD 803.030 and 803.031.
- 5 Condition of excavation is symmetrical about centreline of pipe.
- 6 Bedding and cover material shall be wrapped in non-woven geotextile when specified.
- A All dimensions are in metres unless otherwise shown.

ONTARIO PROVINCIAL STANDARD DRAWING

RIGID PIPE BEDDING AND COVER
IN EMBANKMENT

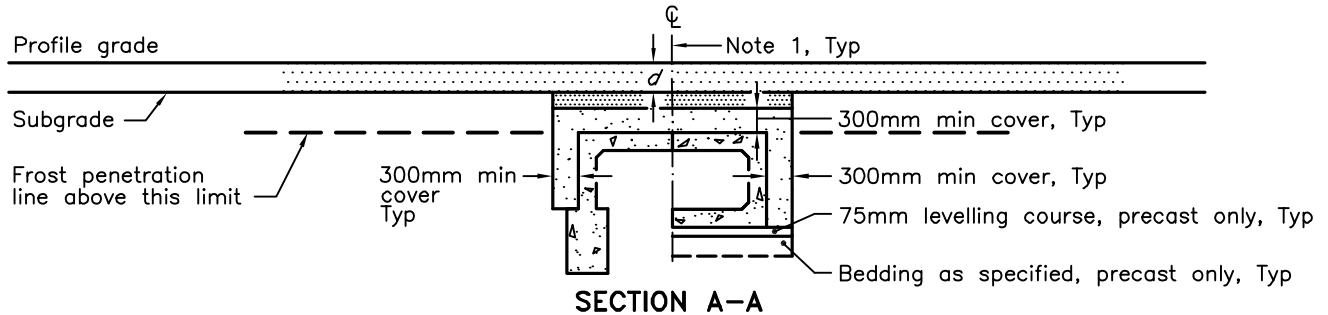
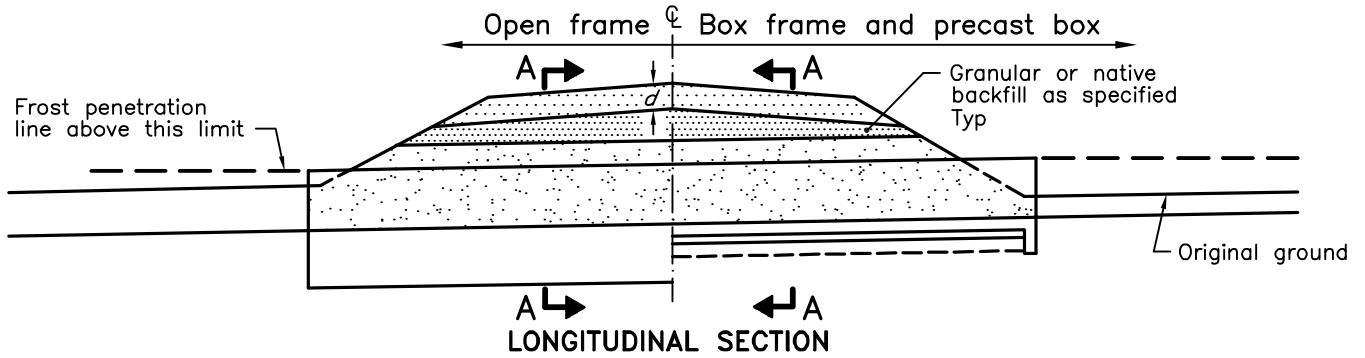
ORIGINAL GROUND: EARTH OR ROCK

Nov 2015

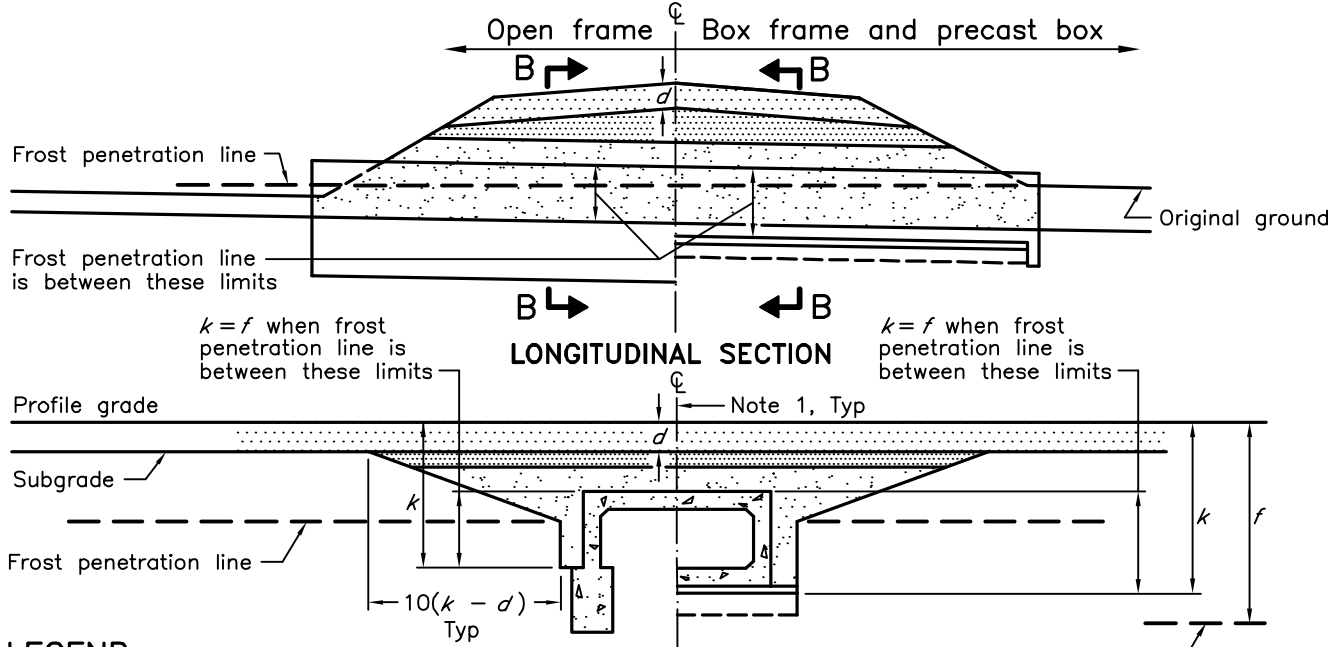
Rev 3

OPSD 802.034

FROST PENETRATION LINE AT OR ABOVE TOP OF CULVERT



FROST PENETRATION LINE BELOW TOP OF CULVERT



LEGEND:

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

NOTES:

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

ONTARIO PROVINCIAL STANDARD DRAWING

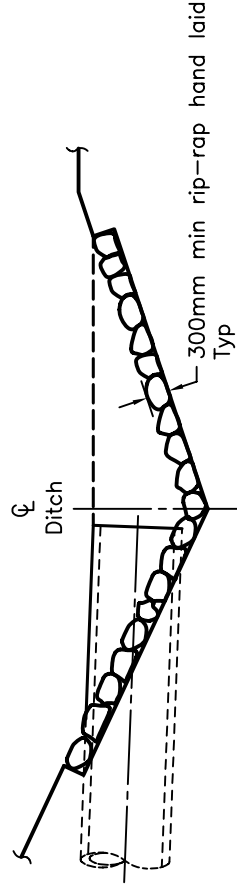
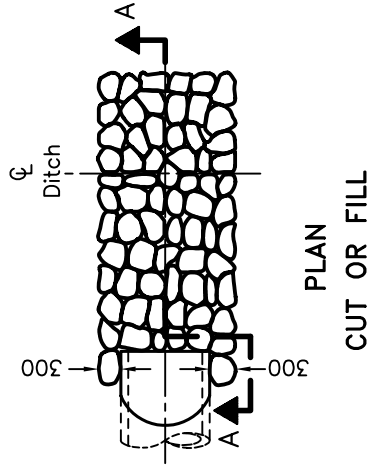
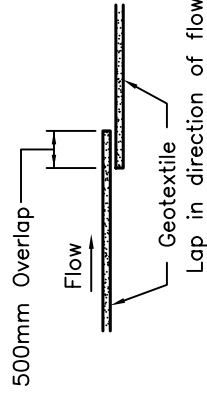
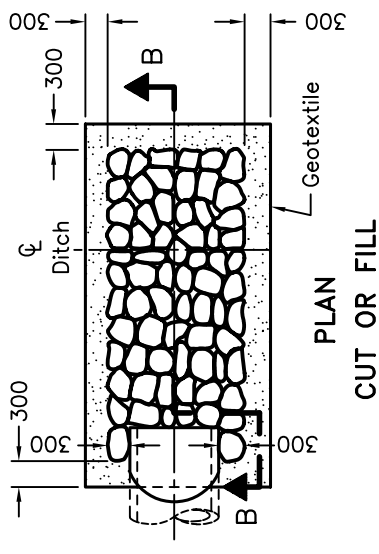
Nov 2015

Rev 3

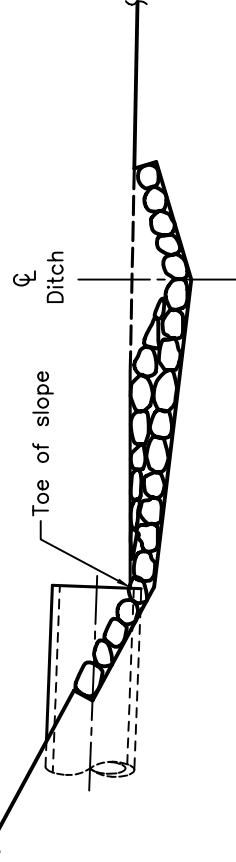
**BACKFILL AND COVER FOR
CONCRETE CULVERTS WITH SPANS
LESS THAN OR EQUAL TO 3.0M**

OPSD 803.010



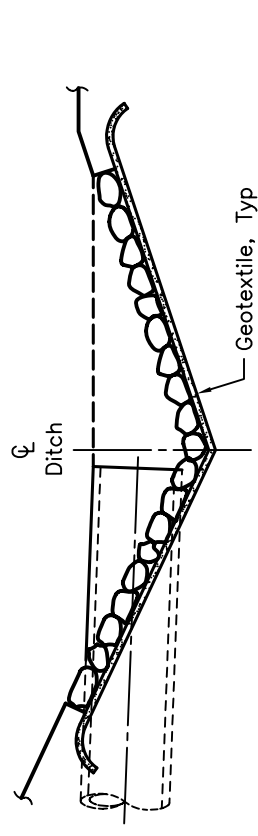


SECTION A-A CUT

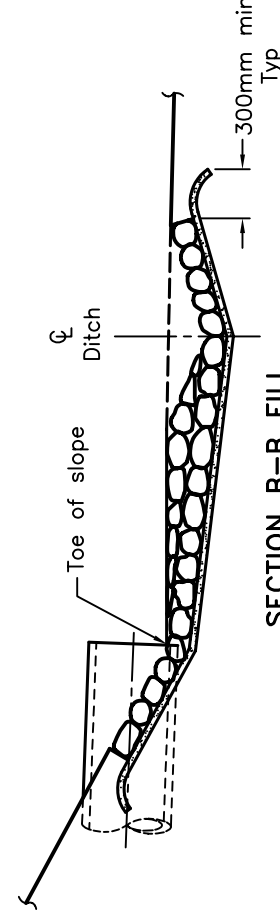


SECTION A-A FILL

TYPE A - WITHOUT GEOTEXTILE



SECTION B-B CUT



SECTION B-B FILL

TYPE B - WITH GEOTEXTILE

NOTES:

A All dimensions are in millimetres unless otherwise shown.

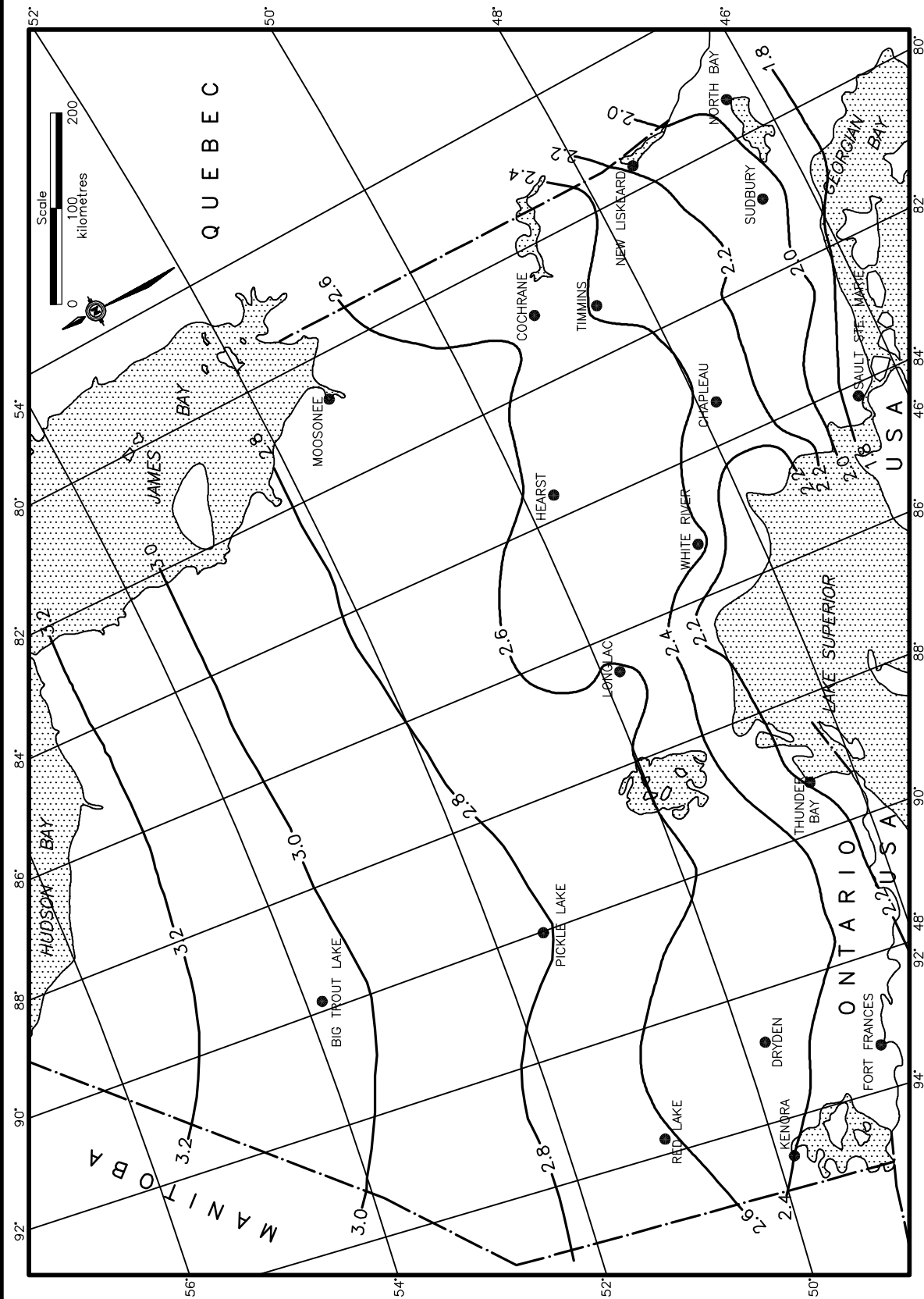
ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2001 Rev 0



RIP-RAP TREATMENT
FOR SEWER AND CULVERT OUTLETS

OPSD - 810.010



NOTES:

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

ONTARIO PROVINCIAL STANDARD DRAWING

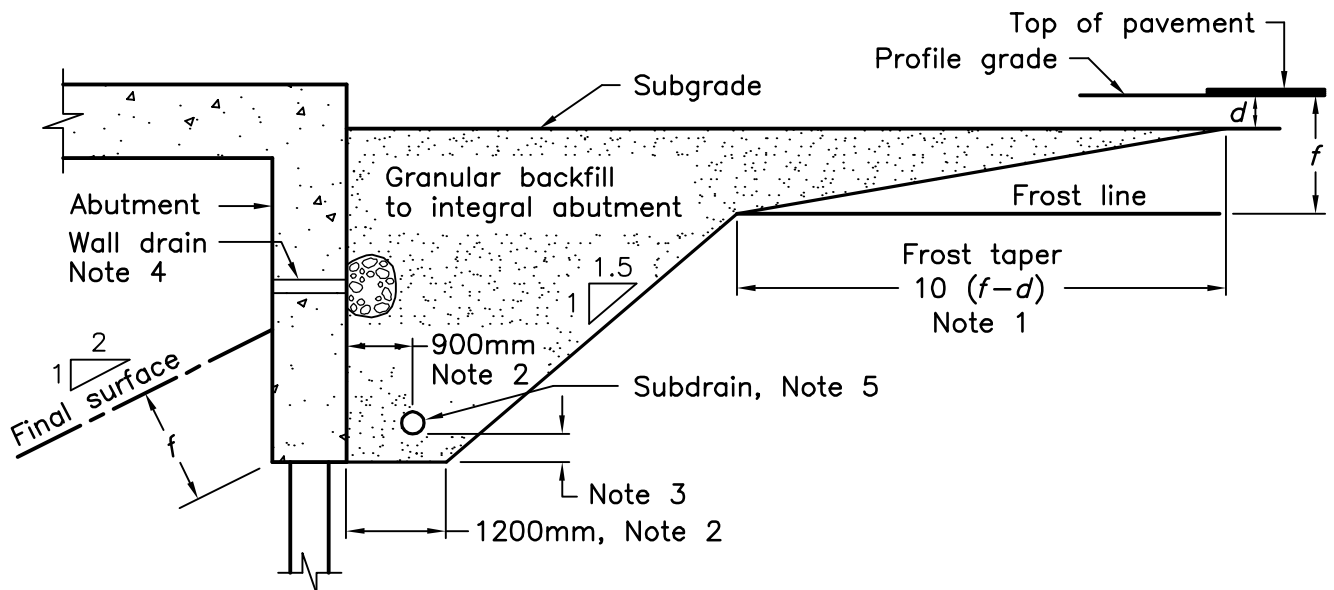
Nov 2010

Rev 1

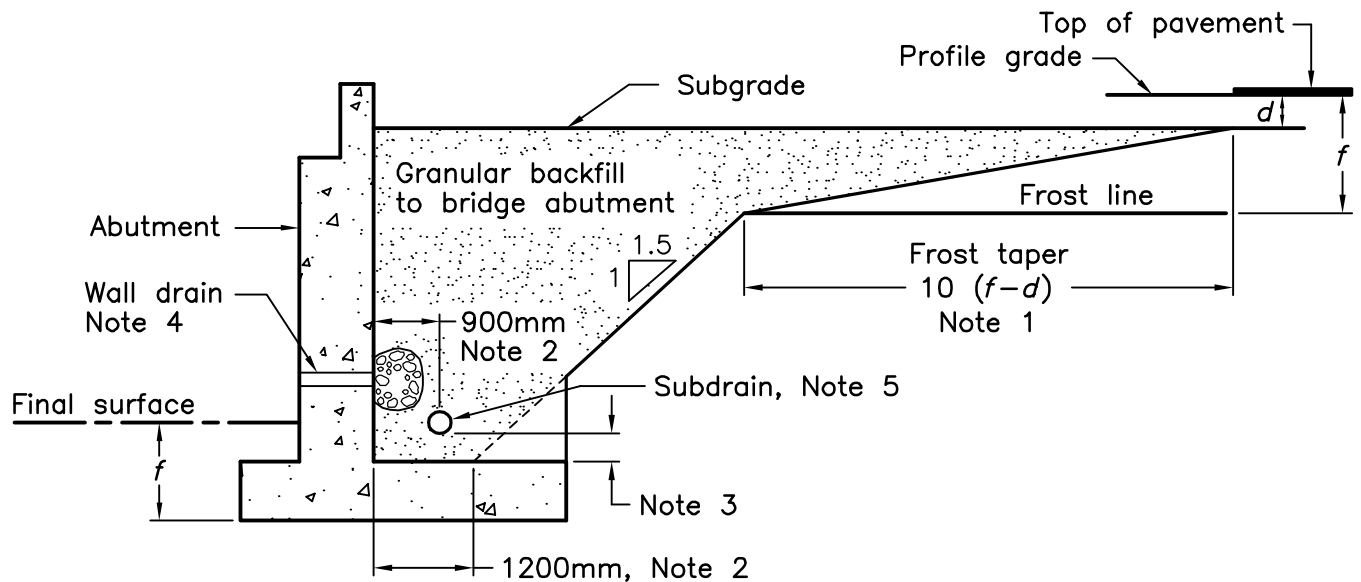
FOUNDATION FROST PENETRATION DEPTHS FOR NORTHERN ONTARIO

OPSD 3090.100





INTEGRAL ABUTMENT



ABUTMENT

NOTES:

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

ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2010

Rev 1



WALLS
ABUTMENT, BACKFILL
MINIMUM GRANULAR REQUIREMENT

OPSD 3101.150

Appendix G – Non-Standard Special Provisions (NSSP)



NSSP FOR COBBLES AND/OR BOULDERS OBSTRUCTIONS

Scope of Work

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

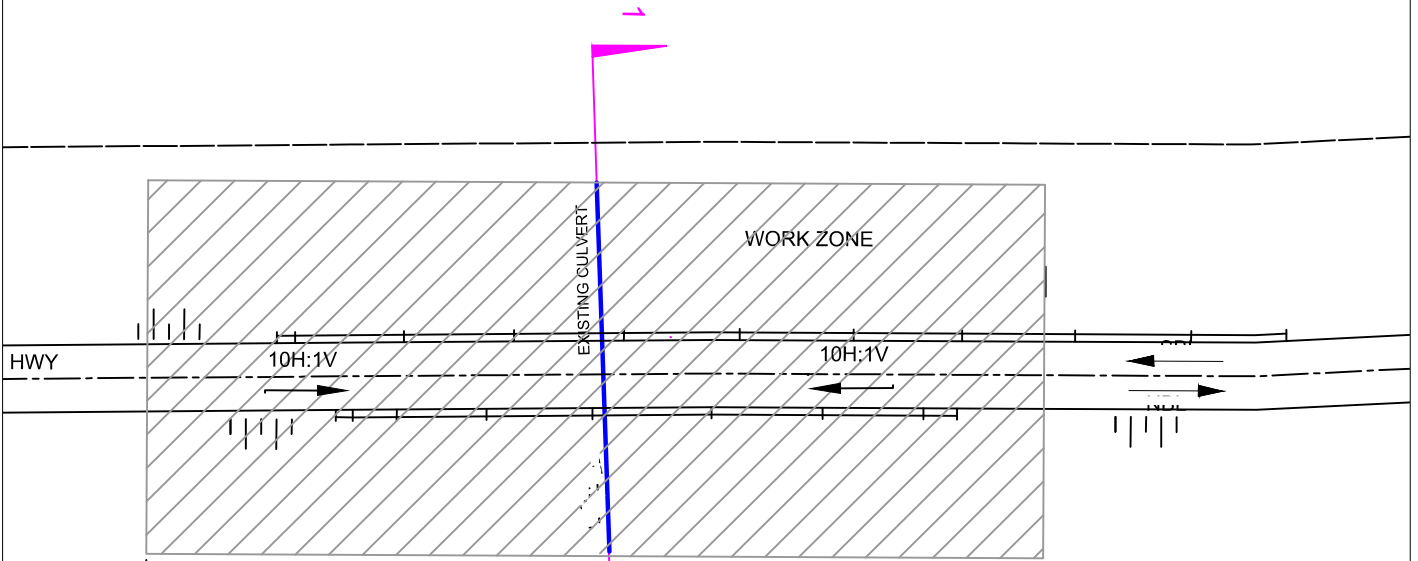
Basis of Payment

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

Appendix H – Schematic Diagram



STAGE CONSTRUCTION BY GRADE LOWERING SCHEMATIC DIAGRAMS (NTS)

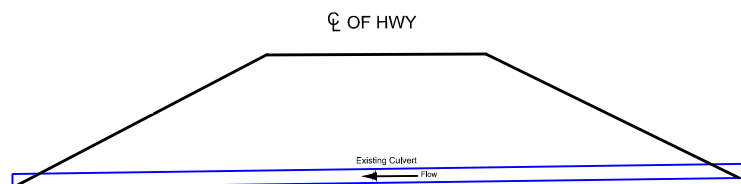


Note:
Highway should be longitudinally excavated as the work zone shows in the plan using the slope 10H:1V.

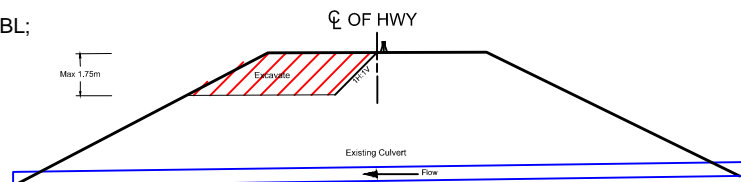
PLAN

RECOMMENDED STAGES

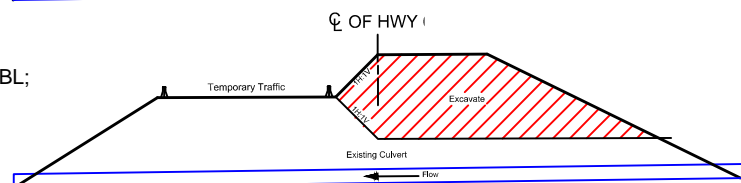
1.0 Stage 1 - Current condition



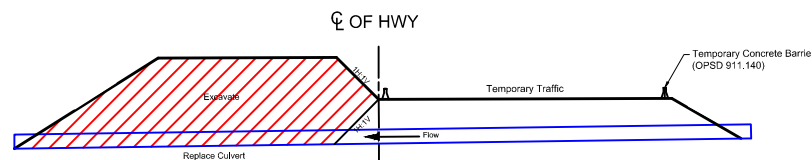
2.0 Stage 2 - Excavation on existing SBL;
One-way traffic on existing road



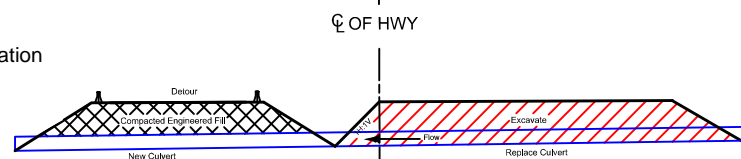
3.0 Stage 3 - Excavation on existing NBL;
One-way traffic shifted to west side



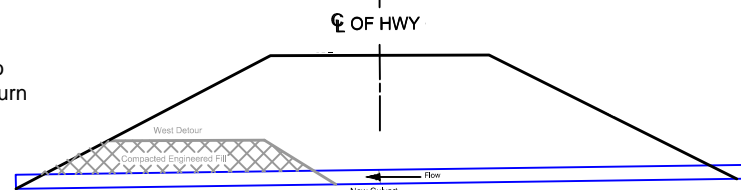
4.0 Stage 4 - Excavation and
culvert construction on west side;
One-way traffic shifted to east side



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



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



SECTION 1-1