Foundation Investigation and Design Report

Highway 11 – 2+1 Roadway Model Project: ***Site SW1***

Assignment No. 5021-E-0038
GWP 5151-21-00
Geocres No. 31L12-002

(Latitude: 46.549100; Longitude: -79.590159)

Type of Document:
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AECOM

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1 FOUNDATION INVESTIGATION REPORT

1.1 Introduction

This report presents the results of the geotechnical investigation completed by EXP Services Inc. (EXP) for the proposed widening of Highway 11 and the corresponding embankment/roadway construction and extension of the existing culvert at site SW1. The site is located approximately 28 km north of the intersection of McKeown Avenue and Highway 11 from approximately Station 11+750 to 11+850 in the Township of Notman in the District of Nipissing, Ontario (Latitude: 46.549100; Longitude: -79.590159). The work was undertaken under Agreement No. 5021-E-0038, and the terms of reference (TOR) were provided by AECOM. The AutoCAD drawings for Highway 11 were also provided by AECOM.

The purpose of the investigation was to evaluate the subsurface condition along the proposed widening of Highway 11 and the existing culvert alignment, and based on this data, to provide a borehole location plan, cross section subsurface profile, record of boreholes, laboratory test results, and a written description of the subsurface conditions to permit detailed design and recommendations for the construction of the new proposed embankment/roadway associated with the widening of the highway and the extension of the existing culvert. The site specific geotechnical investigation consisted of a field investigation program including visual inspections, drilling, soil sampling, and laboratory testing.

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

1.2 Site Description and Geological Setting

1.2.1 Site Description

The site is located approximately 28 km north of the intersection of McKeown Avenue and Highway 11 from approximately Station 11+750 to 11+850 in the Township of Notman in the District of Nipissing, Ontario. At the site, Highway 11 generally runs in the north-south direction, with a speed limit of 90 km/h (unless otherwise posted). At the site, Highway 11 is approximately 8.6 m wide with a 2.9 m wide gravel shoulder on the east side (northbound lane) and a 2.6 m wide gravel shoulder on the west side (southbound lane). In total, the existing roadway with both shoulders included is about 14.1 m wide. The existing culvert is positioned approximately in a west-east direction and is approximately perpendicular to the highway central line. The elevation of the highway pavement centerline at the site is approximately Elev. 351.0 m as per the AutoCAD drawings provided by AECOM, and the roadway embankment above the existing ground is about 2.6 m to 2.8 m high.

Based on the information provided, the existing culvert is a concrete non-rigid frame open culvert, with width of 0.91 m, height of 0.91 m and total length of about 26.8 m, and fill cover of about 1.7 m to 1.9 m. Select photographs of the site and existing culvert are presented in Appendix A. The site plan and cross-section profiles along the existing highway and culvert alignment are shown on the drawings attached in Appendix B.

The general site conditions were assessed during a site visit September 13, 2023 as well as during the field investigation works between November 7, 2023, and January 11, 2024. During the time of the site visit as well as the field investigation works, the flow through the culvert was observed from west to east. In December 2023, the approximate top of water of the creek was measured to be at an elevation of 348.4 m on the west (inlet) side, and 348.2 m on the east (outlet) side.

Based on the CAD drawings provided by AECOM for the existing concrete culvert, the invert of the culvert is at an elevation of about 348.4 m and 348.2 m on the inlet and outlet sides, respectively.

Both sides of the embankment were observed to be mostly comprised of gravel and/or grass with boulder-sized rockfill/riprap to protect against scour or erosion around the culvert. Marshland was observed on both sides of the embankment with vegetation consisting primarily of large conifers and wild bushes. In general, the highway is founded on top of a built-up embankment while the natural terrain in the surrounding area is relatively flat.

Photographs 1 to 6 in Appendix A show the site, culvert, and road photographed between November 2023 and March 2024 by EXP. Photographs 1 and 2 show the inlet of the existing culvert and the surrounding area of the west side of the highway embankment, including the existing rip rap and vegetation. Photograph 3 shows the outlet of the existing culvert, existing rip rap, and the east side of the highway embankment at the culvert location. Photograph 4 shows the inside of the existing culvert, looking from the downstream side (outlet). Photograph 5 shows the existing road condition at the site, and Photograph 6 shows the east side of the embankment and the surrounding vegetation.

1.2.2 Geological Setting

According to the Ministry of Northern Development and Mines, Map 2555 (Quaternary Geology of Ontario, East-Central Sheet, 1991) the surface conditions in the vicinity of the project area are expected to consist of Precambrian bedrock: undifferentiated igneous and metamorphic rock, exposed at surface or covered by a discontinuous, thin layer of drift. According to Map 2543 (Bedrock Geology of Ontario, East-Central Sheet, 1991), the bedrock geology of the site is of migmatic rocks and gneisses of undetermined protolith: commonly layered biotite gneisses and migmatites; locally includes quartzofeldspathic gneisses, orthogneisses, paragneisses.

1.3 Previous Investigations

There are no available previous geotechnical reports at the location of the site in the MTO GEOCRES library; the nearest available reports on Highway 11 are approximately 7.2 km southeast and 5.1 km northwest, respectively, from the site:

- *Geocres No. 31L-209: "Foundation Investigation and Design Report, Temporary Protection System for Culvert STA 11+622, Highway 11, Blythe Township, North Bay, Ontario, GWP 5186-14-00", Prepared by Golder Associates Ltd., dated February 26, 2018.*
- *Geocres No. 31L-080: "Final Report on Detailed Foundation Investigation and Design, Tomiko River Bridge, Highway 11, North Bay, GWP 711-92-00 (WP 344-00-01), Site Number 43-10", Prepared by Golder Associates Ltd., dated December, 2001.*

1.4 Investigation Procedures

1.4.1 Site Investigation and Field Testing

A site reconnaissance was conducted by an EXP representative on September 13, 2023 to evaluate the general site conditions for the proposed borehole locations. The site investigation for the one (1) roadway borehole was performed on November 7, 2023 while the investigation of the six (6) off-road boreholes was performed between December 20, 2023 and January 11, 2024.

The entire field program consisted of drilling seven (7) sampled boreholes, numbered BH1-1 to BH1-7. The boreholes were strategically located along the highway and as close as possible to the existing culvert footprint to provide subsurface information for the widening of the highway and the culvert extension. Borehole BH1-5 was advanced on top of the embankment in the west (southbound) lane of the highway, about 9.5 m northwest of the existing culvert footprint. Boreholes BH1-7, and BH1-2 were advanced at the inlet and outlet locations of the existing culvert, respectively, at the bottom of the roadway embankment. BH1-7 was located approximately 6.5 m southwest of the culvert inlet and BH1-2 was located approximately 5.7 m north of the culvert outlet. Boreholes BH1-4, and BH1-6 were advanced beyond the toe of the existing embankment on the west side of the highway, approximately 46.5 m northwest and 47.7 m southeast of the culvert inlet, respectively. Boreholes BH1-1, and BH1-3 were advanced beyond the toe of the existing embankment on the east side of the highway, approximately 50.8 m northwest and 28.5 m southeast of the culvert outlet, respectively. The locations of the boreholes drilled during this investigation are shown on Drawing 1 in Appendix B. Roadway borehole BH1-5 was advanced to a depth of about 6.8 m below ground surface. Off-road boreholes BH1-1, BH1-2, BH1-3, BH1-4, BH1-6, and BH1-7 were advanced to depths between 3.8 m and 6.9 m below ground surface.

The roadway borehole BH1-5 drilled during this fieldwork was advanced using a truck mounted CME 75 drill rig, operated by a specialist drilling contractor, Marathon Drilling Ltd. while the off-road boreholes drilled during the site investigation were advanced using a track mounted D50 drill rig or a truck mounted CME 55 drill rig, also operated by specialist drilling contractor, Marathon Drilling Ltd. All drill rigs were equipped with hollow stem augers, NW casing/NQ coring or HW casing/HQ coring, and standard soil sampling equipment. Traffic control was provided by Demora Construction Services Inc.

The borehole locations (referenced to the MTM NAD83 Zone 10) and their ground surface elevations were surveyed by EXP personnel using a Trimble DA2 GNSS receiver with Trimble Catalyst GNSS positioning, having an accuracy of ±0.1 m in the horizontal and vertical directions. Ground surface elevations of the boreholes are summarized in Table 1.1 below.

During the drilling of the boreholes, a combination of Standard Penetration Tests (SPT) and rock coring was attempted to obtain soil and rock samples. Soil samples were obtained using a 51 mm outside diameter (O.D.) split-spoon sampler in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586) at intervals ranging from 0.75 m to 1.5 m in depth, as shown on the attached borehole logs (Appendix C). The original field (uncorrected) SPT "N" values were recorded on the borehole logs as recommended in the Canadian Foundation Engineering Manual (CFEM, pg. 103) and used to provide an assessment of the in-situ relative density of cohesionless soils. The SPT "N" values taken within the particles larger than diameter of split spoon sampler may not be reliable and collected samples are possibly not representative of the layer. When a hard stratum was reached (refusal of split spoon), sampling of hard material was performed by diamond core drilling using a 1.5 m long NQ double tube wireline core barrel.

Where possible, groundwater level measurements were carried out in the boreholes before coring and at the completion of the boreholes, in accordance with MTO guidelines. However, all boreholes at this site were advanced using diamond coring procedures. Water was used during advancement of cores from ground surface, therefore groundwater was not measured in boreholes due to the drilling method. The boreholes were decommissioned by bentonite/cement mixtures in accordance with the Ministry of the Environment Regulation 903, as amended by Regulation 128/03 (the well regulation under the Ontario Water Resources Act) upon completion of drilling.

The fieldwork was supervised by an EXP geotechnical representative who directed the drilling and sampling operations, logged borehole data in accordance with MTO and/or ASTM Standards for Soils Classification and retrieved soil samples for subsequent laboratory testing and identification.

All recovered soil samples were placed in labelled moisture-proof bags and returned to EXP's London laboratory for additional visual, textual, and olfactory examination, and selective testing. The rock cores were placed in wooden core boxes and photographed as shown in Appendix E.

Table 1.1. Summary of boreholes completed

Borehole No.	Location	Location (MTM NAD 83 Zone 10)		Latitude	Longitude	Ground Surface Elevation[1] (m)	Borehole Depth[2] (m)
		Northing	Easting				
BH1-1	Off-road beyond toe of existing slope: East side of highway	5156651.8	297863.1	46.549473	-79.590459	350.6	4.4
BH1-2	Off-road beyond toe of existing slope: East side of highway	5156622.7	297899.5	46.549212	-79.589984	348.9	4.2
BH1-3	Off-road beyond toe of existing slope: East side of highway	5156595.9	297919.2	46.548971	-79.589727	349.1	4.4
BH1-4	Off-road beyond toe of existing slope: West side of highway	5156624.8	297843.5	46.549230	-79.590715	349.1	5.3
BH1-5	West (southbound) lane of highway	5156611.1	297881.1	46.549107	-79.590224	351.0	6.8
BH1-6	Off-road beyond toe of existing slope: West side of highway	5156559.3	297908.9	46.548641	-79.589861	349.2	6.9
BH1-7	Off-road beyond toe of existing slope: West side of highway	5156592.8	297877.4	46.548942	-79.590272	348.9	3.8

Notes:

1. *The ground surface elevations are referenced from COSINE Station No. 0011993U649 (CGVD28:78).*
2. *Depths are relative to ground surface.*

1.4.2 Laboratory Testing

All samples returned to the laboratory were subjected to visual examination and classification. The laboratory testing program included the determination of natural moisture content on all samples and particle size distribution for approximately 25% of the collected soil samples where possible. Due to shallow bedrock and minimal recovery of soil samples, particle size distribution testing was limited. All the laboratory tests were carried out in accordance with MTO and/or ASTM standards as appropriate.

1.5 Subsurface Conditions

The detailed subsurface conditions encountered in the boreholes advanced during this investigation are presented on the borehole log sheets in Appendix C. 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 cross section subsurface profiles are provided in Appendix B. It should be noted that the stratigraphic boundaries indicated on the borehole log and cross section stratigraphic profiles are inferred from semi-continuous sampling, observations of drilling progress and results of Standard Penetration Tests (SPT). These boundaries typically represent transitions from one soil type to another and should not be regarded as exact planes of geological change. Furthermore, subsurface conditions may vary between and beyond the borehole locations.

Below the roadway, the subsurface conditions encountered within the investigated depths of the geotechnical investigation (BH1-5) indicates the following subsurface sequence: cohesionless fill consisting of sand and gravel, followed by native cobbles and boulders, underlain by gravelly sand, followed by bedrock. At the culvert inlet, outlet, and bottom of the embankment (BH1-1 to BH1-4, and BH1-6 to BH1-7) the encountered subsurface conditions were observed to generally consist of topsoil followed by sand to silty sand, underlain by sand and gravel or cobbles and boulders, followed by bedrock. However, layer of native silt was present in borehole BH1-4 below the sand to silty sand layer, followed by bedrock, and borehole BH1-7 consisted of topsoil followed by cobbles and boulders underlain by bedrock.

A detailed description of the subsurface conditions encountered is discussed further in subsequent sections.

1.5.1 Subsoils

1.5.1.1 Asphalt Treatment

Asphalt treatment, approximately 0.11 m thick, was encountered at the ground surface of borehole BH1-5. Asphalt thicknesses may further vary beyond the borehole location.

1.5.1.2 Topsoil

Topsoil, approximately 0.05 m to 1.5 m thick, was encountered at the ground surface of boreholes BH1-1 to BH1-4, and boreholes BH1-6 to BH1-7.

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

Moisture Content:

- 104% to 427%

Organic Content:

- 13%

The results of the moisture content, and organic content tests are provided on the record of borehole sheets in Appendix C.

1.5.1.3 Cohesionless Fill: Sand and Gravel

Non-cohesive fill material consisting of varying distributions of predominantly sand and gravel was encountered below the asphalt treatment in borehole BH1-5.The depths and elevations of this layer encountered at this borehole location are listed in Table 1.2.

Table 1.2. Summary of cohesionless fill

Borehole No.	Elevation[1] (m)		Layer Surface Depth[2] (m)	Layer Thickness (m)
	Top	Bottom		
BH1-5	350.9	348.7	0.1	2.2

Notes:
1. *The elevations referenced are geodetic.*
2. *Depths are relative to ground surface.*

The composition of this fill material generally consisted of sand and gravel in varying amounts with trace to some silt. Cobbles were also encountered in this layer. The fill was generally black to brown in colour and moist to wet. The SPT "N" values obtained within this material ranged from 6 to 36 blows per 0.3 m penetration, suggesting that this layer was loose to dense in compactness.

Laboratory testing was not performed on soil samples from this layer due to minimal sample recovery during the investigation.

1.5.1.4 Sand / Silty Sand

Native sand to silty sand was encountered below the topsoil in boreholes BH1-1 to BH1-4, and BH1-6. The depths and elevations of this layer encountered at these borehole locations are listed in Table 1.3.

Table 1.3. Summary of sand / silty sand layer

Borehole No.	Elevation[1] (m)		Layer Surface Depth[2] (m)	Layer Thickness (m)
	Top	Bottom		
BH1-1	350.5	349.8	0.1	0.7
BH1-2	348.8	247.9	0.1	0.9
BH1-3	349.0	348.5	0.1	0.5

Borehole No.	Elevation[1] (m)		Layer Surface Depth[2] (m)	Layer Thickness (m)
	Top	Bottom		
BH1-4	348.3	347.6	0.8	0.7
BH1-6	347.7	346.7	1.5	1.0

Notes:
1. *The elevations referenced are geodetic.*
2. *Depths are relative to ground surface.*

The composition of this material generally consisted of sand to silty sand with trace to some gravel. Trace organics were encountered in borehole BH1-3. This layer was generally light brown to grey in colour and wet. The SPT "N" values obtained within this material ranged from 4 to 100 blows per 0.3 m penetration to 50 blows per 0.1 m penetration, suggesting that this layer was loose to very dense in compactness, but generally loose to compact.

Laboratory testing performed on selected samples consisted of six (6) moisture content tests, and one (1) grain size distribution test. The test results are as follows:

Moisture Content:

- 18% to 42%

Grain Size Distribution:

- 3% gravel
- 93% sand
- 4% silt and clay

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

1.5.1.5 Silt

Native silt was encountered below the native sand / silty sand layer in borehole BH1-4. The depth and elevations of this layer encountered at this borehole location are listed in Table 1.4.

Table 1.4. Summary of silt layer

Borehole No.	Elevation[1] (m)		Layer Surface Depth[2] (m)	Layer Thickness (m)
	Top	Bottom		
BH1-4	347.6	346.8	1.5	0.8

Notes:

1. *The elevations referenced are geodetic.*
2. *Depths are relative to ground surface.*

The composition of this material generally consisted of silt, with trace gravel, trace sand, and trace clay. This layer was generally grey in colour and wet. The SPT "N" value obtained within this layer was 28 blows per 0.3 m penetration, suggesting that this layer was compact in compactness.

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

Moisture Content:

- 11%

Grain Size Distribution:

- 3% gravel
- 8% sand
- 88% silt
- 1% 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 test are also provided on Figure 2 in Appendix D.

1.5.1.6 Gravelly Sand / Sand and Gravel

Native gravelly sand / sand and gravel was encountered below the sand / silty sand in borehole BH1-1 and below the boulders layer in BH1-5. The depth and elevations of this layer encountered at these borehole locations are listed in Table 1.5.

Table 1.5. Summary of gravelly sand / sand and gravel layer

Borehole No.	Elevation[1] (m)		Layer Surface Depth[2] (m)	Layer Thickness (m)
	Top	Bottom		
BH1-1	349.8	349.6	0.8	0.2
BH1-5	347.9	347.7	3.1	0.2

Notes:

1. *The elevations referenced are geodetic.*
2. *Depths are relative to ground surface.*

The composition of this material generally consisted of sand and gravel in varying amounts with some silt. The material was generally brown to grey in colour and wet. The SPT "N" values obtained within this material ranged from 100 blows per 0.3 m penetration to 50 blows per 0.1 m penetration, suggesting that this layer was very dense in compactness.

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

Moisture Content:

- 9% to 35%

rain Size Distribution:

- 21% gravel
- 66% sand
- 13% silt and clay

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

1.5.1.7 Cobbles and Boulders

Native cobbles and boulders were encountered below the native sand and gravel layer in borehole BH1-1, below the cohesionless fill layer in borehole BH1-5, below the native sand / silty sand layer in borehole BH1-6, and below the topsoil in borehole BH1-7. The depths and elevations of this layer encountered at these borehole locations are listed in Table 1.6.

Table 1.6. Summary of cobbles and boulders layer

Borehole	Elevation[1] (m)		Layer Surface Depth[2] (m)	Layer Thickness (m)
	Top	Bottom		
BH1-1	349.6	349.4	1.0	0.2
BH1-5	348.7	347.9	2.3	0.8
BH1-6	346.3	345.4	2.9	0.9
BH1-7	348.7	348.1	0.2	0.6

Notes:
1. *The elevations referenced are geodetic.*
2. *Depths are relative to ground surface.*

A combination of SPT and coring was carried out during the exploration of this layer. Where possible, split spoon sampling was attempted to obtain samples from this layer. However, no split spoon samples could be obtained.

1.5.2 Bedrock

Bedrock was encountered beneath the native soils and/or cobbles and boulders in all boreholes except borehole BH1-5 where bedrock was encountered below the gravelly sand / sand and gravel layer. Elevations at the top of bedrock were between 349.4 m to 345.4 m, suggesting that the bedrock generally slopes downwards from north to south. The bedrock was investigated by coring about 3.0 m to 3.5 m into the stratum. The bedrock surface depths and elevations encountered at these borehole locations are listed in Table 1.7. Photographs of the rock cores are included in Appendix E.

Table 1.7. Summary of bedrock

Borehole No.	Elevation[1] (m)		Layer Surface Depth[2] (m)
	Top	Bottom	
BH1-1	349.4	346.2	1.2
BH1-2	347.9	344.7	1.0
BH1-3	348.5	344.7	0.6
BH1-4	346.8	343.8	2.3
BH1-5	347.7	344.2	3.3
BH1-6	345.4	342.3	3.8
BH1-7	348.1	345.1	0.8

Notes:
1. *The elevations referenced are geodetic.*
2. *Depths are relative to ground surface.*

Based on the bedrock NQ cores (~ core diameter 47 mm) recovered, the bedrock at the site consisted of quartzofeldspathic gneiss. In general, the rock samples are described as grey with pink in colour. The Rock Quality Designation (RQD) measured on the core samples typically ranged from approximately 14% to 93%, indicating a rock mass of very poor to excellent quality, but generally poor to good quality. The total core recovery (TCR) of bedrock cores ranged from 90% to 100%.

1.6 Groundwater and Surface Water Conditions

All boreholes at this site were advanced using diamond coring procedures. Water was used during advancement of cores from ground surface, therefore groundwater was not measured in boreholes due to the drilling method.

Surficial water flow was observed in the existing culvert during the site investigation. The top of creek water level at the existing culvert location was measured to be at an elevation of about 348.4 m on the west (inlet) side, and 348.1 m on the east (outlet) side during the site investigation.

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

2 ENGINEERING DISCUSSION & RECOMMENDATIONS

2.1 General

This section of the report provides geotechnical design recommendations for the design and construction of the new proposed roadway embankment, and for the extension of the existing culvert along the proposed widening of Highway 11 at site SW1. Site SW1 is located approximately 28 km north of the intersection of McKeown Avenue and Highway 11 from approximately Station 11+750 to 11+850 in the Township of Notman in the District of Nipissing, Ontario (Latitude: 46.549100; Longitude: -79.590159) in the 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 performed by EXP between November 7, 2023, and January 11, 2024. The compiled factual data is presented in **Part I-Foundation Investigation Report** of this report. The interpretation and recommendations provided are intended solely to permit designers to assess foundation alternatives for the extension of the existing culvert and for the design of the proposed embankment widening. Comments on construction are only provided to highlight issues that could affect the design. Contractors bidding on the works should make their own assessments of the factual data and how it might affect construction means and methods, scheduling and the like.

At the site, the existing concrete non-rigid frame open culvert conveys a creek below Highway 11. Based on the CAD drawings provided by AECOM, the culvert is about 0.91 m in width, 0.91 m in height, with a total length of approximately 26.8 m. The existing culvert is positioned approximately in a west to east direction, perpendicular to the highway central line. The elevation of the highway pavement at the culvert location is approximately 351.0 m. The flow through the culvert was observed from west to east, following the natural topographic conditions in the vicinity of the site.

At the time of preparing this report for the submission to the MTO (April 2024), it was unknown whether the highway would be widened on one side or both sides. However, preliminary highway cross section drawings were provided to EXP in November 2024 showing the proposed widening of about 10 m on the east side at SW1. It is also understood that the invert levels of the extension culvert will be similar to that of the existing culvert invert levels, at approximate elevation 348.4 m at the inlet and 348.2 m at the outlet locations. It is also understood that the existing highway grade is planned to remain unchanged with respect to the original ground level.

This part of the report addresses the geotechnical design of the foundation for the proposed culvert extension 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-S6-19)*, the *Canadian Foundation Engineering Manual (CFEM) (2023), MTO Gravity Pipe Design Guidelines (April 2014), Guideline for MTO Foundation Engineering Services, Version 02 (October 2020) and Version 03 (April 2022),* and generally accepted good practice. Pertinent construction issues from a geotechnical standpoint were examined in general accordance with the Terms of Reference. The assessment involved a review of different types of culvert options for the extension of the existing culvert and other geotechnical and construction considerations such as an assessment of slope stability and settlement of the widened embankments, lateral earth pressure on structures, site preparation, excavation, and frost protection.

A "typical consequence level" is considered appropriate for the culvert extension and embankment widening at this site, as outline in Section 6.5 of the CHBDC (CAN/CSA-S6-19) and a MTO memorandum from Materials Engineering Research Office (MERO) #2020-01 dated March 23, 2020, respectively. Further, given the scope of work of the foundation field investigation and laboratory testing program a "typical degree of site and prediction model understanding" has been utilized. Accordingly, the appropriate corresponding ULS and SLS consequence factor, Ψ, from Table 6.1 and geotechnical resistance factors, ϕ_{gu} and ϕ_{gs} from Table 6.2 of the CHBDC (CAN/CSA-S6-19) for the culvert. Additionally, an appropriate geotechnical resistance factor, ϕ_{gu}, for global stability – permanent was obtained from Table 1 in MERO #2020-01.

2.2 Expected Ground Conditions

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

a) Highway 11 is a two-lane road (~8.6 m wide) with extended gravel shoulders (~2.6 m to 2.9 m wide). In total, the existing roadway with both shoulders included is about 14.1 m at the culvert footprint. Highway 11 is generally oriented in the north-south direction. The highway crosses the 0.91 m by 0.91 m concrete non-rigid frame open culvert with approximately 1.7 m to 1.9 m of embankment fill above the crown on the west (inlet) side and east (outlet) side, respectively; orientated approximately perpendicular to the highway central line. The elevation of the crest of the roadway embankment is about Elev. 351.1 m at the location of the culvert. Boulder-sized rock fill/riprap was observed on both sides of the embankment around the culvert to protect against scour and erosion. Marshland was observed on both sides of the embankment with vegetation consisting primarily of conifers, wild bushes, and various species other vegetative cover. The invert elevations of the existing culvert at the inlet and outlet are approximately at Elev. 348.4 m and Elev. 348.2 m, respectively.

b) Below the pavement structure (borehole BH1-5), the highway embankment consists of sand and gravel fill (~2.2 m thick) followed by cobbles and boulders (~0.8 m thick), followed by native sand (~0.2 m thick) underlain by bedrock encountered at about Elev. 347.7 m.

c) Along the west side of the highway beyond the toe of the existing embankment slope (boreholes BH1-4, and BH1-6) the subsurface conditions encountered consisted of topsoil (~0.9 to 1.5 m thick), followed by native sand/silty sand (~ 1.0 to 1.5 m thick), followed by silt (~0.8 m thick) in BH1-4, underlain by bedrock encountered at about Elev. 346.8 m to 345.4 m.

d) Along the east side of the highway beyond the toe of the existing embankment slope (boreholes BH1-1, and BH1-3), the subsurface conditions encountered consisted of topsoil (~0.1 m thick), followed by native sand/silty sand (~0.5 to 0.7 m thick) underlain by bedrock at about Elev. 349.4 m to 348.5 m. A layer of native sand and gravel (~ 0.2 m thick) followed by cobbles and boulders (~0.1 m thick) was encountered below the sand/silty sand in borehole BH1-1.

e) At the inlet (borehole BH1-7), a layer of topsoil (~0.2 m thick) was encountered at the ground surface over a layer of cobbles and boulders (~0.6 m thick) underlain by bedrock encountered at about Elev. 348.1 m. At the outlet (borehole BH1-2), a layer of topsoil (~ 0.1 m thick) was encountered at the ground surface over a layer of native silty sand (~0.9 m thick) underlain by bedrock encountered at about Elev. 347.9 m.

f) At the site, the bedrock slopes downwards from west to east along the culvert footprint, but generally slope downwards from north to south along the highway.

g) During the fieldwork, the elevation of the top of water in the creek was measured be about Elev. 348.4 m at the inlet and 348.1 m at the outlet. Water was used during coring from ground surface, therefore groundwater was not measured in boreholes due to the drilling method. Seasonal variations in the water table should be expected.

2.3 Seismic and Liquefaction Potential Consideration

2.3.1 Seismic Hazard Site Classification and Values

The potential for seismic loading must be considered for design in accordance with Section 6.14.7 of the CHBDC with respect to the soil conditions encountered at the site. Table 4.1 of the CHBDC shows site classification for seismic site response based on average soil properties in the top 30 m.

The native subsurface conditions at the embankment location generally consists of silty sand to gravel followed by cobbles and boulders and shallow bedrock (~0.6 m to 3.8 m below the existing ground surface). During the fieldwork, the top of water at the inlet and outlet of the culvert was measured to be about Elev. 348.4 m and 348.1 m, respectively. Therefore, based on these soil characteristics, the site class for this site is estimated to be Class “B” according to Table 4.1 of the CHBDC.

From the Natural Resources Canada website, 2020 NBC seismic hazard values are obtained using the site location coordinates and the site-adjusted damped reference spectral accelerations for the project site are shown in Table 2.1 below:

Table 2.1 Seismic design values

Probability of Exceedance in 50 Years (Return Period)	Sa(0.2) (g)[1]	Sa(0.5) (g)	Sa(1.0) (g)	Sa(2.0) (g)	PGA (g)
Latitude: 46.549100; Longitude: -79.590159					
2% (1 in 2475-year)	0.356	0.167	0.082	0.037	0.187

Note:

1. *g = acceleration due to gravity (9.81 m/s^2)*

These values are associated with an earthquake having a 2% probability of exceedance in a 50-year period (1 in 2475-year) for Site Class B and is also shown on the seismic hazard calculation data sheet for this site attached in Appendix G.

The site coefficients used to determine the design spectral acceleration and displacement values are a function of the Site Class and the reference peak ground acceleration (PGA_{ref}). Since Sa(0.2)/PGA is less than 2.0 at this site, PGA_{ref} is equal to 0.8*PGA=0.150 g, as per section 4.4.3.3. of the CHBDC (CAN/CSA-S6-19). The site coefficient F(PGA), for this site (Seismic Site Class B and PGA_{ref} = 0.150 g) is 0.87.

2.3.2 Liquefaction Potential

Cohesionless soils below the groundwater table (silt, silty sand, sand, cobbles and boulder) ranged from about 0.2 m to 1.7 m in thickness above the bedrock surface during EXP’s investigation. Seismically induced liquefaction is not considered to impact the overall project design due to presence of shallow bedrock and the coarse nature of the submerged soils.

2.4 Culvert Extension

2.4.1 Culvert Extension Options

The choice of culvert extension type and size depends on hydraulic performance, staging requirements, geotechnical resistance available in the foundation soils, initial cost, maintenance costs, ease of construction, water and soil corrosiveness, salvageability and local availability of materials and equipment. However, for preliminary design purposes, the following possible options for culvert extension are presented below with the advantages and disadvantages of each summarized in Table 2.2:

- Corrugated steel pipe (CSP) culvert
- Precast rigid frame concrete box culvert,
- Cast-in-place rigid frame concrete box culvert, and
- Cast-in-place rigid frame open footing concrete culvert supported on shallow foundations

The native sand to silty sand/bedrock is considered suitable for support of all options. However, the choice of culvert type also depends on parameters such as the initial cost, maintenance costs, hydraulic performance, ease of construction, water and soil corrosiveness, salvageability and local availability of material and equipment.

In general, any loose and/or soft soils encountered below the new culvert location should be excavated and replaced with engineered fill. If the depth of excavation to remove unstable soils is excessive, using a geotextile fabric in accordance with OPSS.PROV 1860, Class II Non-Woven (OPSS 1860 II-N), in conjunction with engineered fill can be considered to assist in providing a stable base for the new culvert extension. Based on previous experience, typically it should consist of Granular A or Granular B Type II (OPSS.PROV 1010) with a minimum thickness of 300 mm beneath the culvert structure and extend a minimum of 500 mm horizontally on either side of the culvert edge. The fabric should be installed a manner to mitigate the migration of fines from adjacent material.

Closed box culverts, either precast or cast-in-place, installed with appropriate granular bedding over the subgrade were determined to be feasible. Among these three options, the use of a precast box culvert is ranked highest for the criteria evaluated. It should be noted that the proposed structure must meet the required flow capacity and hydraulic requirements. An open-footed concrete culvert on spread footings is feasible at this site, however, it is likely more expensive than other options due to deeper excavations and dewatering required for casting the footings.

Table 2.2 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 concrete box culvert is ranked highest for the criteria evaluated

Table 2.2 Evaluation of structure type options for culvert extension

Option	Rank	Advantages	Disadvantages	Risk/Consequences
Precast rigid frame concrete box culvert	1	• Reduced construction period than cast-in-place open footing option, consequently water control period will be reduced • Reduced excavation depth, protection system and dewatering requirement compared to open footing option • Can be more readily installed during cold weather conditions • Existing culvert can be used to maintain flow during construction	• Greater disturbance to natural streambed than for open footing culvert option • Though founding level is higher than open footing, excavation would still require below the water level, requiring dewatering scheme to enable construction in dry conditions • Need for dowelling into existing culvert	• Risk of leaking from joints if not properly installed
Cast-in-place rigid frame concrete box culvert	2	• Suitable if precast sections are relatively larger for transportation and installation of heavier units and if site is not appropriate 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, temporary protection system requirements	• Slower construction process • Requires time for forming, placing and curing of concrete • Greater disturbance to natural streambed than for open footing culvert option	• Risk of disturbance of base during construction
Cast-in-place rigid frame open footing concrete culvert	3	• Minimum disturbance of creek channel during excavation, as compared with wider span for box culvert • Wider span may consider maintaining existing channel and so allows for natural streambed to remain intact • Less accumulation of sediments in the upstream of culvert	• Deeper excavation than box culvert options would be required, consequently increased in excavation support and dewatering requirements • Potential for undermining existing culvert during construction • Require longer duration for construction, including traffic management and water control period	• Risk of disturbance of existing structure founding soils. • Higher scour risk
Corrugated Steel Pipe (CSP) culvert	4	• Straightforward construction • Reduced construction period, consequently, traffic management and water control period • Reduced excavation depth	• Requires bedding material • Limited design life • Potential for corrosion • Difficult connection between CSP and existing concrete culvert	• Risk of structure segment loss due to corrosion

2.4.2 Shallow Foundations Options

2.4.2.1 Geotechnical Resistance for Structure Foundations

Based on the subsurface stratigraphy encountered at this site and the assumed invert elevation of the culvert extension, Table 2.3 summarizes the recommended founding levels and geotechnical resistances for a structure founded on competent cobbles/boulders, or bedrock. The geotechnical resistances provided are for vertical loading conditions only; load eccentricity and load inclination effects should be addressed in accordance with the CHBDC and its commentary. The geotechnical resistances provided in sections below were factored with typical consequence factors of 1.0 at ULS and SLS and typical degree of understanding (factor of 0.5 at ULS and factor of 0.8 at SLS) in accordance with Table 6.1 and 6.2 of the CHBDC S6-19.

Table 2.3 Recommended shallow foundation design parameters

Culvert Type	Relevant Boreholes	Founding Elevation/ Excavation Elevation (m)	Footing Size (m)	Founding Soil Type	Factored Geotechnical Resistance at ULS (kPa)	Factored Geotechnical Resistance at SLS[2] (kPa)
Precast or cast-in-place rigid frame concrete box culvert/CSP Pipe culvert	BH1-2 BH1-5 BH1-7	~348.4 m/348.1 m (inlet) to ~348.2 m/347.9 m (outlet)[3]	~1 to 2[1]	~ 0.3 m thick Granular 'A' or 'B' Type II bedding over cobbles and boulder/ bedrock	600	320
Cast-in-place rigid frame open footing concrete culvert or shallow foundations	BH1-2 BH1-5 BH1-7	348.1 m (inlet)/ to 347.9 m (outlet)[3] (Excavation to bedrock level)	1	Bedrock	1000	535

Notes:

1. *Assumed based on existing culvert*
2. *for maximum settlement of 25 mm*
3. *Excavation to bedrock*
4. *The granular material used for the granular pad shall be Granular 'A' or Granular B Type II conforming to OPSS.PROV 1010 and compacted to 98% SPMDD*

It is assumed that, if any, underlying organic soils (peat) and any other soft or very loose materials are to be replaced with clean and compactable soil such as Granular A or Granular B Type II. Given that the bedrock is shallow at the site and providing that any soft or loose material will be stripped off prior to construction of the embankment, the anticipated maximum total settlement for the proposed culvert extension is expected not to exceed 25 mm for construction done in accordance with these design parameters and assuming good construction practice including sound base preparation. Section 2.6.3 discusses site dewatering related to the installation of the culvert extension.

2.4.2.2 Resistance to Lateral Loads

Resistance to lateral forces/sliding should be calculated in accordance with Section 6.10.5 of the CHBDC (CAN/CSA S6-19), using the following parameters:

Table 2.4 Recommended parameters for calculation of unfactored horizontal resistance

Interface and Loading Conditions	Parameters
Between Granular A and precast concrete	Coefficient of friction (tan δ)=0.55
Between Granular A and cast-in-place concrete	Coefficient of friction (tan δ)=0.6
Between bedrock and cast-in-place concrete	Coefficient of friction (tan δ)=0.7

The listed values are unfactored; in accordance with the CHBDC (CAN/CSA S6-19), a factor of 0.8 is to be applied in calculating the horizontal resistance.

2.4.2.3 Frost Protection

The frost depth in the area of the culvert is estimated to be approximately 2.0 m in accordance with OPSD 3090.100. A minimum 2.0 m of soil cover or equivalent frost protection should be provided using thermal insulation only to the rigid frame open footing culvert option. However, since the footing will be placed on bedrock the requirement for frost protection might not be applicable. For the box culvert and CSP culvert options, frost protection is not required.

2.4.3 Culvert Bedding

OPSDs 803.010, 802.010, and 802.034, which are included in Appendix H provide the bedding, embedment, cover and backfill standards for concrete box and pipe culverts. According to these standards the culvert bedding should consist of Granular A (OPSS.PROV 1010) with a minimum thickness of 300 mm beneath the culvert and extend a minimum of 300 mm horizontally on either side of the culvert edge. The bedding material should be placed in layers not exceeding 200 mm in thickness, loose measurement, and compacted in accordance with OPSS.PROV 501 before a subsequent layer is placed in accordance with OPSS. PROV 401. Based on the existing conditions at the site, Granular B Type II is preferred material for the culvert bedding below the water table.

The culvert extension installation shall be carried out in accordance with OPSS.PROV 902. Therefore, prior to placing any fill material, the exposed subgrade should be inspected according to OPSS.PROV 902. A non-woven geotextile separator is to be placed between the approved subgrade and the compacted fill to assist in material placement and maintain the integrity of the founding soil along the entire length of the culvert. The geotextile separator is to be a Class II non-woven material in accordance with OPSS.PROV 1860 with a filtration opening size as specified in the contract documents.

It is recommended that at locations where the foundation will be placed on the rock (which would be the case for a cast-in-place rigid frame open footing concrete culvert), all overburden soils be stripped to expose the bedrock within the structure footprint. Since the bedrock surface in the area of the structure footprint is variable, engineered fill or mass concrete can be used to level the grade. The fill and/or mass concrete can be placed above the bedrock surface following removal of the existing overburden soils. All loose, shattered and/or fractured rock within the foundation footprint should be removed and scaled prior to placement of mass concrete or concrete foundations in accordance with OPSS.PROV 902.

2.4.4 Culvert Backfill

The selection and placing of the backfill and cover should be in accordance with OPSS.PROV 902, OPSS.PROV 421, OPSS.PROV 422 and OPSS.PROV 206, and OPSDs 802.034, 803.010 and 3101.150 for different culvert materials. The backfill should consist of free-draining, non-frost susceptible granular materials conforming to OPSS.PROV 1010.

For fills immediately below any roadway, it is recommended that Granular A or B Type II materials be used. As noted below, proper tapering as per standards should be provided. Below a depth of about 2.0 m from any finished road grade, approved compactable fill, such as select subgrade materials (OPSS.PROV 1010) or imported fill can be used.

All granular backfill materials should be placed in thin lifts (i.e., not exceeding 300 mm before compaction) and each lift should be compacted in accordance with OPSS. PROV 501. The final lift of embankment fill prior to placing pavement sub-base should be compacted to 100 % SPMDD. The Granular A base and Granular B sub-base courses (for pavement) should be compacted to 100% of the material's SPMDD.

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.

Where less than 2.0 m (the frost depth) of earth cover is provided above the top of the culvert, a frost taper should be included as per OPSD 803.031, and MTOD 803.021, whichever is applicable. If the frost taper exists at the site, it will be reinstalled within the zone of excavation in accordance with OPSD 803.031.

2.4.5 Lateral Earth Pressures

Culvert walls and temporary shoring (if any) should be designed to resist lateral earth pressure. The expression for calculating lateral earth pressure in accordance with the CHDBC is given by:

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

where,

P = earth pressure intensity at depth h, kPa
K = earth pressure coefficient
γ = unit weight of retained soil, kN/m^3
q = surcharge near wall, kPa
h = depth to point of interest, m

The above expression does not take into account hydrostatic pressure, which must be included for the groundwater levels measured on the site. Table 2.5 lists earth pressure parameters for given materials. These recommendations assume wall friction is neglected, a level ground surface in front and behind the walls and the back face of the wall is vertical.

Table 2.5 Material types and earth pressure properties under static conditions

Material	Unfactored Friction Angle ϕ' (°)	Coefficient of Active Earth Pressure (K_a)	Coefficient of Passive Earth Pressure (K_p)	Coefficient of Earth Pressure At-Rest (K_o)	Unit Weight γ (kN/m^3)
Granular A / B Type II	35	0.27	3.69	0.43	22.8
Granular B Type I	32	0.31	3.26	0.47	21
Engineered Fill (SSM)	30	0.33	3.00	0.50	21
Sand and Gravel Fill (Loose to Dense)	33	0.30	3.39	0.46	20
Sand / Silty Sand (Loose to Very Dense)	32	0.31	3.26	0.47	22
Cobbles and Boulders	36	0.26	3.85	0.41	18

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.

It is likely that bracing for the temporary support system (if any) will be required at a maximum interval of 5 m. For multiple support systems refer to Canadian Foundation Engineering Manual (CFEM) 5th Edition for apparent earth pressure distributions (CFEM, Section 20.8.1.4).

2.5 Embankment Widening

2.5.1 General

Based on the information provided to EXP by the client, the existing highway grade is planned to remain unchanged with respect to the original ground level. At the time of preparing this report for the submission to the MTO (April 2024), it was unknown whether the highway would be widened on one side or both sides. However, preliminary highway cross section drawings were provided to EXP in November 2024 showing the proposed widening of about 10 m on the east side at SW1.

Based on the existing surrounding ground, the construction of the widened embankment will require placement of approximately 2.6 m to 2.8 m high fill on the east side of the existing highway. Design analyses and recommendations for the embankment widening have been carried out using approved earth soils (i.e., Selected Subgrade Material (SSM) as per OPSS.PROV 1010) for its construction. The side slopes of new embankment fill should not be steeper than 2H:1V. Construction considerations for embankment widening within SW1 including excavation, subgrade preparation, embankment construction, etc. are provided in the following section.

2.5.2 Stability Considerations

Preliminary slope stability analyses were performed to assess the global stability of the new widened embankment to check if a minimum Factor of Safety of 1.4 for static and 1.1 for seismic conditions is achieved as per MTO criteria for typical degree of understanding (MERO #2020-01). The static and seismic slope stability analyses were performed using the Morgenstern-Price method developed on the basis of limit equilibrium. The SLOPE/W computer program developed by GeoSlope International was employed for computation.

The stratigraphy and groundwater condition at the site were developed based on the results of the geotechnical investigation presented in Part I – Foundation Investigation Report. The seismic properties given in Appendix G (Section 2.3.1) were obtained from the Natural Resources Canada website, 2020 NBC, using the site location coordinates. Tabulated below in Table 2.6 are the soil parameters used for the slope stability analyses.

Table 2.6 Soil properties used in slope stability analyses for new embankment

Material Type	Effective Stress Parameters		
	ϕ (degrees)	c (kPa)	γ (Kn/m3)
Engineered Fill (SSM)	30	0	21
Granular A/B Type II	35	0	22.8
Granular B Type I	32	0	21
Cohesionless Fill: Sand and Gravel (Loose to Dense)	33	0	20
Sand / Silty Sand (Loose to Very Dense)	32	0	22
Sand and Gravel (Very Dense)	33	0	20
Cobbles and Boulders	36	0	18
Bedrock	(Impenetrable)		

Based on the borehole information, the subsoils encountered at the work area consist of cohesionless fill and native cohesionless soils above the bedrock. Therefore, only effective stress (drained conditions) analyses of the slopes for a long-term assessment were performed taking into consideration the subsoil conditions encountered at the site. The analysis assumes that all organic material (if encountered) will be removed prior to construction. In addition, a traffic surcharge pressure of 16.8 kPa was adopted in the slope stability assessments. Table 2.7 summarizes the results of performed slope stability analyses. The SLOPE/W graphical printouts for the analyses are included in Appendix F (Figures F1 – F2).

Table 2.7 Summary of results of existing and new embankment slope stability analyses

	Locations	Max Height (m)	Conditions	Min FOS
New Embankment with SSM	East (outlet) Side (2H:1V)	~2.8 m	Drained long-term conditions, static condition	1.4 (Figure F1)
			Drained long-term conditions, seismic condition	1.2 (Figure F2)

As seen in Table 2.7, the results of the slope stability analyses for the new widened embankment (the culvert should be backfilled with Granular A/B Type II below and around the culvert followed by engineered fill (i.e., SSM) above the granular material) having side slopes no steeper than 2H:1V on the east (outlet) side can be considered stable for static and seismic conditions (i.e., calculated FOS > 1.4 for static and FOS > 1.1 for seismic with k_h = 0.5*F(PGA)*PGA=0.081g, assuming that all loose sand and silt soils below the culvert are excavated and replaced with properly compacted granular material.

2.5.3 Settlement Considerations

2.5.3.1 Settlement of Foundation Soils

Based on the ground conditions (i.e., non-cohesive foundation soils and shallow bedrock), the total settlement of the foundation soils under the widened 2.6 to 2.8 m high embankment is estimated to be less than 25 mm. The majority of this settlement is expected to occur relatively quickly following construction of the new embankments.

For new embankment approaches to structural elements, MTO settlement criteria are as follows: the post construction settlement is limited to 25 mm, 50 mm, 75 mm, and >100 mm for 0 to 20 m, 20 to 50 m, 50 to 75 m, and >75 m offsets from the abutment/culvert, respectively. These settlements are considered acceptable for 20 years post paving.

Considering that it is not planned to change the existing embankment grade at the culvert location, and the presence of shallow bedrock at the site, it is anticipated that there should be negligible additional settlements under the new widened embankment. However, a settlement of about 25 mm should be allowed.

2.5.3.2 Settlement of Embankment Fill

The fill is also expected to experience some settlement. It is estimated that the embankment itself will compress by about 0.5 to 1 percent of the embankment height under its self-weight, depending on material type and assuming placement as per MTO practices. More granular material fills would compress less and over a shorter time period, typically within the period of embankment construction. Non-granular earth fills would exhibit some additional settlement over time. In this setting, embankment fills are expected to meet the MTO approach criteria within 2 to 4 months of completion, where SSM or better materials are used for embankment widening. To minimize the post construction settlement, the fill materials should be compacted to at least 98% SPMDD. Some differential settlements can be expected at the structure/embankment interface, but these movements should be able to be accommodated during the paving process.

2.5.4 Embankment Construction

Prior to construction of the embankment widening, the site will need to be cleared and grubbed of any existing bushes, trees, and vegetation. All surficial topsoil, organics, existing fill, and softened or loosened soils should be stripped from below the proposed widening footprint. Considering the findings at the site, the anticipated stripping depths/elevations at the borehole locations are as follows:

Table 2.8 Anticipated stripping depths/elevations for embankment widening at borehole locations

Location	Borehole No.	Existing Ground Elevation at Borehole Location (m)	Recommended Stripping Depth/Elevation (m)
East Side	BH1-1	350.6	0.3/350.3
	BH1-2	348.9	0.3/348.6
	BH1-3	349.1	0.2/348.9
West Side	BH1-4	349.1	0.8/348.3
	BH1-6	349.2	1.5/347.7
	BH1-7	348.9	0.2/348.7

All subgrade soils should be proof-rolled and inspected by qualified geotechnical engineer prior to fill placement. The embankment construction should be carried out in accordance with OPSS.PROV 206. As previously noted, the existing embankment slope must be cleared of all vegetation and must be properly compacted and benched in accordance with OPSD 208.010 prior to placing the new fill materials.

The materials used for the construction of the embankment fills should consist of approved, acceptable earth fill (e.g. Select Subgrade Materials – OPSS.PROV 1010). Assuming properly compacted, acceptable inorganic earth fill materials are utilized, a minimum 2H:1V side slope can be used for the construction of the embankment fills. In accordance with OPSD 208.010, benching of the existing Highway 11 embankment side slopes should be carried out to key in the new fill materials for widening.

Care must be taken to properly compact the embankments to reduce settlements associated with fill density changes. Fill used for construction of the embankments should be in accordance with OPSS.PROV 212 and fill placement/construction should be in accordance with OPSS.PROV 206. The fill should be placed in regular lifts with loose thickness not exceeding 300 mm and compacted to at least 95% of SPMDD. The final lift of fill prior to placement of the roadway granular subbase and base courses should be compacted to 100% of SPMDD. Quality assurance should be provided as per MTO standard 501.08 (OPSS.PROV 501). Inspection and field density should be carried out by qualified personnel during placement operations to ensure that appropriate materials are used and that adequate levels of compaction have been achieved.

2.6 Construction Considerations

2.6.1 Excavation

All excavations must be carried out in accordance with the latest edition of the Ontario Occupational Health and Safety (OHSA) and good construction practice. The existing fill and native soil are considered Type 3 soils above the groundwater table and Type 4 soils below the groundwater table. Temporary excavations (i.e., those that are open only for a short period) above the groundwater table may be made with side slopes not steeper than 1H:1V, while the temporary slopes below the groundwater table must be formed at 3H:1V unless a suitable dewatering system is installed to lower the water level below the base of the excavation.

For the culvert extension, depending on the option selected, excavations for footings may be extended below the existing culvert footings. In such cases, appropriate measures will be required during construction to support the soils below the existing culvert footings during excavation, to prevent undermining of the existing culvert footings.

2.6.2 Temporary Roadway Protection

In the case that a roadway protection system is required for culvert installation, a shoring system such as a soldier pile and lagging system can be employed for temporary excavations at this site. It will be the Contractor's responsibility to design a suitable temporary support system for the MTO to review prior to installation. The Contractor is to follow OPSS.PROV 539 regarding temporary protection systems. The shoring system should be designed using the parameters recommended in Table 2.5 of this report.

The Contractor should be responsible for the complete design, construction, monitoring, and removal of the installed protection system. The protection system shall be designed to provide protection for excavations as required by the OHSA, at locations specified in the contract, and at any locations where the stability, safety or function of an existing structure and/or utility may be impaired by construction work. Decommissioning of temporary shoring must be consistent with good practice to avoid interference with highway systems and utilities, if any. The protection system shall be designed for the Performance Level 2 (for small, less important sections). The minimum requirements for monitoring should include the survey measurements of 6 m apart scaled targets attached to the shoring wall at the specified elevations. If movement approaches the allowable limit of 25 mm (Performance Level 2), suitable measures should be taken to ensure stability of the protection system and to ensure that the movement does not exceed the performance level specified.

2.6.3 Site Dewatering

2.6.3.1 Groundwater Control

The groundwater levels in the boreholes were not measured due to the introduction of water into the drilling/coring process. However, groundwater levels would be expected to reflect levels in the adjacent open water. At the time of the field investigation (November 2023 to January 2024), the approximate top of water elevations at the inlet and outlet of the existing culvert were approximately Elev. 348.4 m and 348.1 m, respectively. Construction for the widening of the embankment and extension of the culvert is recommended during the low water level season. However, considering the minimal stripping depths/excavation required on the east side for embankment widening (see Table 2.8), the groundwater control within this section mostly addresses the excavation for culvert extension.

The excavation to the foundation level for the culvert extension has to be carried out to approximately 0.8 m depth below the ground surface (Elev. 348.1 m) at the inlet side and approximately 1 m depth below the ground surface

(Elev. 347.9 m) on the outlet side. The soils encountered at the site and within potential excavation depths consist of sand to silty sand, and cobbles and boulders to bedrock. The estimated range of hydraulic conductivity (k) of these soil materials is 10^{-1}-10^{-5} m/s.

The soils encountered below the groundwater table and within potential excavation depths consist of sand and silt, and cobbles and boulders. Some of these materials are susceptible to disturbance from groundwater and mobilized equipment (i.e., silt), and in those cases the groundwater level needs to be controlled to 0.5 m below the excavation level to avoid disturbance. Any surface or groundwater seepage should be removed from the excavation prior to the placement of granular backfill in the dry condition. Granular B Type II or clear stone with geotextile wrapping can be used in the wet condition.

According to the proposed foundation depths and the water level observed at the time of the investigation, excavations for the culvert extension will extend up to approximately 0.3 m below the water level. Therefore, considering that the surface water will be controlled by cofferdams it is expected that groundwater inflow through the native soils and/or cobbles and boulders can be handled by pumping from filtered sumps located behind the cofferdams at the inlet and outlet. It should be noted that there is possibility of seepage from fissures within the bedrock which can be controlled by pumping as well.

Dewatering requirements behind the cofferdams to keep the construction site dry will be impacted by water levels in the creek at the time of construction activities. 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. Dewatering should be carried out in accordance with OPSS.PROV 517 as amended by SP517F01 (i.e., included in Appendix I for reference). It is the responsibility of the Contractor to propose a suitable dewatering system based on the time of construction, water levels and flow conditions in the creek. The method used should not undermine the existing culvert, highway embankment or adjacent side slopes. The provision of toe protection at the side slopes during drawdown may be required to minimize sloughing and undercutting during dewatering.

Dewatering may require water taking permits (i.e., Permit to Take Water PTTW). A PTTW is required for any water taking if the volume exceeds 50,000 L/day. The rate and volume required for dewatering will be dependent on the construction methods and staging chosen by the Contractor.

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.

2.6.3.2 Cofferdams

At the location of the culvert, temporary cofferdams will be required at both the upstream and downstream ends to envelop the construction site and keep it free of water during embankment construction and extension of the existing culvert. Sheet pile walls for the cofferdam is likely not feasible due to shallow bedrock. Therefore, a rockfill/earth dam can be considered. Design and construction specifications for the chosen temporary cofferdam system should be prepared in accordance with OPSS.PROV 517 as amended by SP517F01 by the Contractor.

The rockfill/earth cofferdam will have to be constructed to accommodate all topographic constraints. The size of material suitable for use depends on the erosion potential, stream flow velocity, etc. The rockfill/earth cofferdam should be designed with a more impervious water barrier at the outside face to create a more watertight enclosure.

Schemes involving 2-inch minus crusher run with finer facing material upstream have been successfully used in similar settings. Any required permitting must be determined. The proposed rockfill/earth cofferdam should be at least one meter above the designed high-water level (HWL) defined by the hydraulic engineer.

Besides design and construction of the temporary cofferdam system, the Contractor is also responsible for its materials, maintenance, monitoring and removal. The temporary cofferdam shall be fully removed, unless it is specified in the Contract Documents that the cofferdam system may be partially left in place. The method and sequence of removal shall be so that there shall be no damage to the new work, existing work, and facility being protected.

2.6.4 Scour/Erosion Protection

2.6.4.1 Embankment Side Slope

For the embankment side slopes, adequate erosion protection against surface water runoff should be provided. The native silty soil is easily disturbed by rain or surface run off. The exposed slope surface should be covered with straw or plastic sheets as soon as the slope face is exposed. The native silt and sand soil is easily disturbed from rain or surface run off. The exposed slope surface should be covered with straw or plastic sheets as soon as the slope face is exposed. To reduce surface erosion on the embankment side slopes, prompt seed and cover (OPSS.PROV 804) or sodding (OPSS.PROV 803) should be carried out as soon as possible after construction of the embankment.

2.6.4.2 Culvert Extension

Scour/erosion protection should be provided at the culvert extension. The erosion/scour protection should be designed by a specialist hydraulic engineer (as erosion and scour largely depend on the velocity of water in the watercourse and its regime) who is familiar with the findings of this report. The following are some general suggestions, considering that the boreholes indicate that the main soil type consists of silt and sand.

The need for and nature of scour and erosion protection systems must be assessed and where required, must be designed, implemented, and remain effective for the design life of the culvert. The potential for scour below foundations must be incorporated into the design.

Rip-rap protection should be provided where the culvert discharges into the open creek and where the open creek enters the culvert. The design should be finalized by the hydraulics engineer. For preliminary guidance, the rip-rap should extend approximately 5 m beyond the ends of the culvert and line the embankment slope to the spring line of the culvert. Such protection may involve 0.5 m thick rock (OPSS.PROV 511) extending from 1 m above the high-water level to the toe of the slope and into the stream bed within the plan limits of the culvert. The rip-rap configuration at the creek bed should generally follow OPSD 810.010. The slope of the riprap shall follow the embankment fill slope.

As noted above, the scour design, nature and extent of the required protection is the responsibility of a qualified hydraulic design engineer experienced in this field. Geotechnical soil parameters necessary for the scour analyses are: SPT N-value, in-situ moisture content, percent passing the No. 200 sieve (%200), mean grain size diameter (D_{50}), liquid limit (LL), plastic limit (PL), and plasticity index (PI). These parameters can be determined based on the soils encountered at the site during the investigation and presented in Part I of this report.

The erosion protection should consider the possible installation of seepage protection measures at both upstream and downstream ends. The native sand and silt soils have a high potential for migration with high seepage gradients. For the new culvert extension installation, it is prudent to examine possible methods to avoid piping of material resulting from seepage along the culvert. For culverts, the following are typical methods: (i) clay seal, (ii) steel or wooden sheet pile cutoff at the upstream end of culvert, (iii) cut-off wall incorporated in the apron slab (if one is used) of the culvert, (iv) cut-off trench constructed with geotextile, and (v) rockfill at the upstream end of the culvert barrel to terminate below the granular bedding of the culvert. Only the clay seal and cut-off trench will be addressed since the sheet pile cut-off will require the understanding of the hydraulics of the stream. However, the actual solution for the culvert extension at this site will depend on the seepage protection of the existing culvert, if any.

Clay Seal

Where readily available, a clay seal should be placed at the contact of new extension and existing culvert to prevent the migration of material along the face of the culvert, the formation of flow paths, and any potential internal erosion within the highway embankment. OPSS.PROV 1205 specifies that material used for clay seals shall be natural clay, clay mixture (1 part Bentonite powder and 3.5 parts Granular “A”) or a Geosynthetic Clay Liner (GCL). The coefficient of permeability shall not exceed 1×10^{-6} cm/s.

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

- The clay seal should be placed along the sides and top of the culvert a minimum of 1.0 m along the side of the culvert and extending out laterally 1.0 m from the culvert extension.
- The clay should have a Liquid Limit greater than 40% and a Plasticity Index greater than 0.73 x (Liquid Limit – 20%).
- The clay seal is to be placed in maximum 150 mm thick lifts and compacted to 95% SPMDD within 2% of the optimum moisture content.

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

Cut-Off Trench/Wall

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

2.7 Obstructions During Installation of Temporary Protection Systems

Cobbles and boulders were encountered during the site investigation. Therefore, care must be taken since the presence of these obstructions may affect the excavation for the culvert extension and installation of protection system elements including the temporary roadway protection system (if any) and temporary dewatering/unwatering systems. It is recommended that a NSSP be included in the Contract Documents to warn the Contractor of the presence of cobbles and boulders within the embankment. An example of a NSSP for obstructions is provided in Appendix I.

3 CLOSURE

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

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

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

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

This Foundation Investigation and Design Report has been prepared by Ciarra Alexander M.Eng, Nimesh Tamrakar, M.Eng., P.Eng., and Thomas Lardner, Ph.D., P.Eng. It was reviewed by 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 Elvis Lu, M.Eng., and Stephen Fredericks, M.Eng, P.Eng.

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Ministry of Northern Development and Mines Map 2543. Bedrock Geology of Ontario, East-Central Sheet, 1991

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Ministry of Transportation, October 2020. Guideline for MTO Foundation Engineering Services, Version 02

Ministry of Transportation, April 2022. Guideline for MTO Foundation Engineering Services, Version 03

ASTM International:

ASTM D1586 Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils

Ontario Provincial Standard Specifications (OPSS):

OPSS.PROV 206 Construction Specification for Grading

OPSS.PROV 212 Construction Specification for Earth Borrow

OPSS.PROV 401 Construction Specification for Trenching, Backfilling and Compacting

OPSS.PROV 421 Construction Specification for Pipe Culvert Installation in Open Cut

OPSS.MUNI 422 Construction Specification for Precast Reinforced Concrete Box Culverts in Open Cut

OPSS.PROV 501 Construction Specification for Compacting

OPSS.PROV 511 Rip Rap, Rock Protection and Granular Sheeting

OPSS.PROV 517 Construction Specification for Dewatering and Temporary Flow Passage Systems

OPSS.PROV 539 Construction Specification for Temporary Protection Systems

OPSS.PROV 803 Construction Specification for Vegetative Cover

OPSS.PROV 804 Temporary Erosion Control

OPSS.PROV 902 Construction Specification for Excavating and Backfilling – Structures

OPSS.PROV 1010 Material Specification for Aggregates - Base, Subbase, Select Subgrade, And Backfill Material

OPSS.PROV 1205 Material Specification for Clay Seal

OPSS.PROV 1860 Material Specification for Geotextiles

Ontario Provincial Standard Drawings (OPSD):

OPSD 208.010	Benching of Earth Slopes
OPSD 802.010	Flexible Pipe Embedment and Backfill Earth Excavation
OPSD 802.034	Rigid Pipe Bedding and Cover in Embankment, Original Ground, Earth or Rock
OPSD 803.010	Backfill and Cover for Concrete Culverts with Span Less Than or Equal to 3.0 m
MTOD 803.021	Bedding and Backfill for Precast Concrete Box Culverts
OPSD 803.031	Frost Treatment – Pipe Culverts Frost Penetration Line Between Top of Pipe and Bedding Grade
OPSD 810.010	Rip-Rap Treatment for Sewer and Culvert Outlets
OPSD 3090.100	Foundation Frost Penetration Depths for Northern Ontario
OPSD 3101.150	Walls, Abutment, Backfill, Minimum Granular Requirement

Special Provisions (SP):

SP517F01	Amendment to OPSS.PROV 517
SP109S61	Amendment to OPSS.PROV 902

Ontario Water Resources Act:

R.R.O 1990, Regulation 903 Wells, under Ontario Water Resources Act, R.S.O. 1990, c. O.40

Ontario Occupational Health and Safety Act (OHSA):

Ontario Regulation 213/91 Construction Projects

LIMITATIONS AND USE OF REPORT

BASIS OF REPORT

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

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

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

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

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

RELIANCE ON INFORMATION PROVIDED

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

STANDARD OF CARE

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

COMPLETE REPORT

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

USE OF REPORT

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

REPORT FORMAT

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

Appendix A –
Site Photographs

Photograph 1. Culvert inlet facing northeast (March 15, 2024)

Photograph 2. Culvert inlet and west side of embankment, facing northwest (March 15, 2024)

Photograph 3. Culvert outlet facing southwest (March 15, 2024)

Photograph 4. Inside of culvert at outlet facing southwest (March 15, 2024)

Photograph 5. Existing road condition facing northwest (December 12, 2023)

Photograph 6. East side of **embankment at Borehole BH1-1 Location** facing southeast (December 12, 2023)

Appendix B – Drawings

CONT No. 5021-E-0038
ASSIG No.
GWP No. 5151-21-00

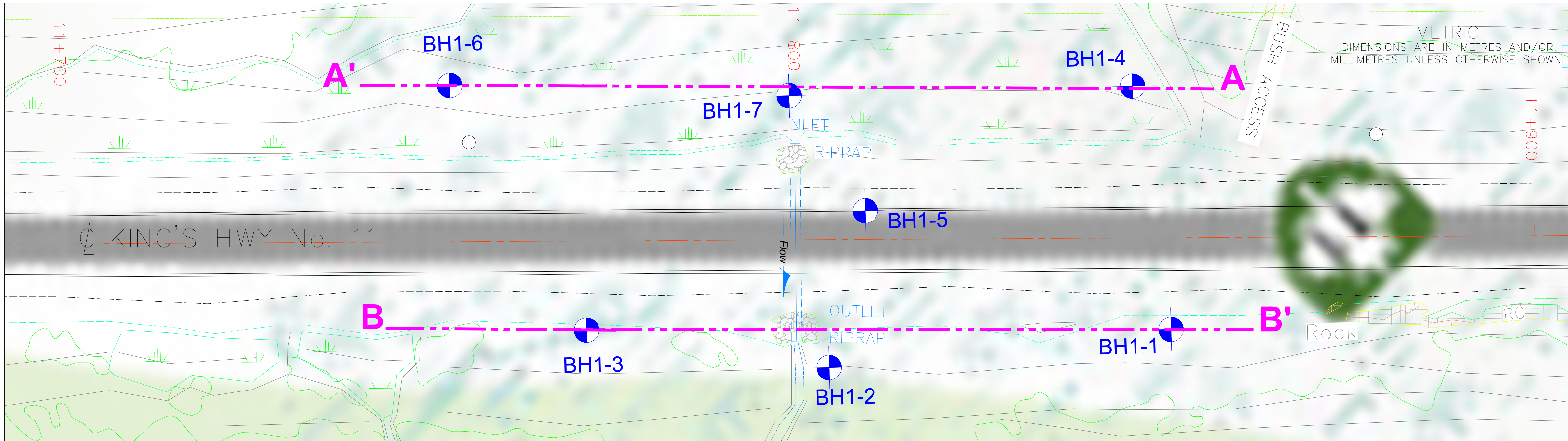
Highway 11 from Sand Dam Road Northerly 13.8 Km to Ellesmere Road (SW1)
Latitude: 46.549100°; Longitude: -79.590159°

BOREHOLE LOCATION PLAN & SOIL STRATA

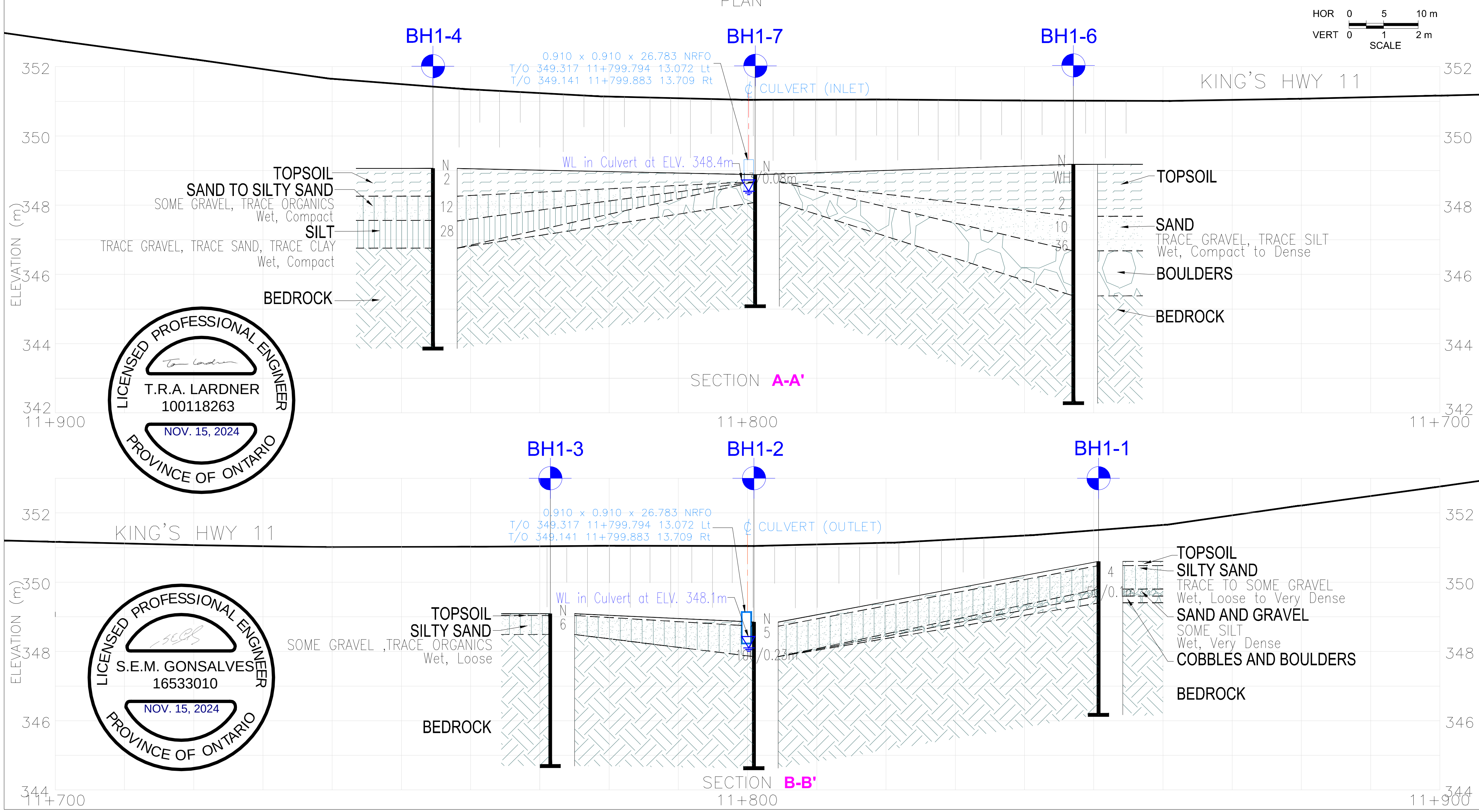
SHEET 1

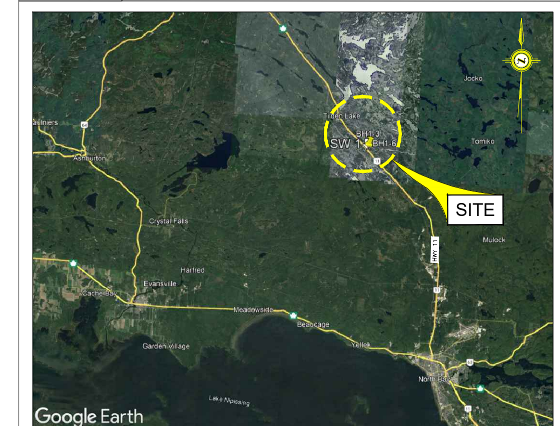
EXP SERVICES INC.

METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN.



PLAN





KEY PLAN
N.T.S.

LEGEND

- Borehole Location
- N Blows/0.3m (Std. Pen. Test, 475 J/blow)
- Water Level In Culvert

SOIL STRATA SYMBOLS

- TOPSOIL
- ASPHALT
- FILL
- SAND/ SILTY SAND
- GRAVELLY SAND/ SAND AND GRAVEL
- SILT
- COBBLES/BOULDERS
- BEDROCK

BOREHOLE COORDINATES/ NAD 83/ MTM ON-10

BH No.	ELEV.	NORTHING	EASTING
BH1-1	350.6	5156651.8	297863.1
BH1-2	348.9	5156622.7	297899.5
BH1-3	349.1	5156595.9	297919.2
BH1-4	349.1	5156624.8	297843.5
BH1-5	351.0	5156611.1	297881.1
BH1-6	349.2	5156559.3	297908.9
BH1-7	348.9	5156592.8	297877.4

NOTES

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

The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.

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.

REVISIONS

NO	DATE	BY	DESCRIPTION
			SUBMISSION FOR MTO REVIEW

PROJECT No. ADM-23010055-A0		GEOCRES No. 31L12-002	
SUBM'D SH	CHKD. TL	DATE APRIL 22, 2024	SITE -
DRAWN SH	CHKD. TL	APPRD SG	DWG 01

CONT No. 5021-E-0038
ASSIG No.
GWP No. 5151-21-00

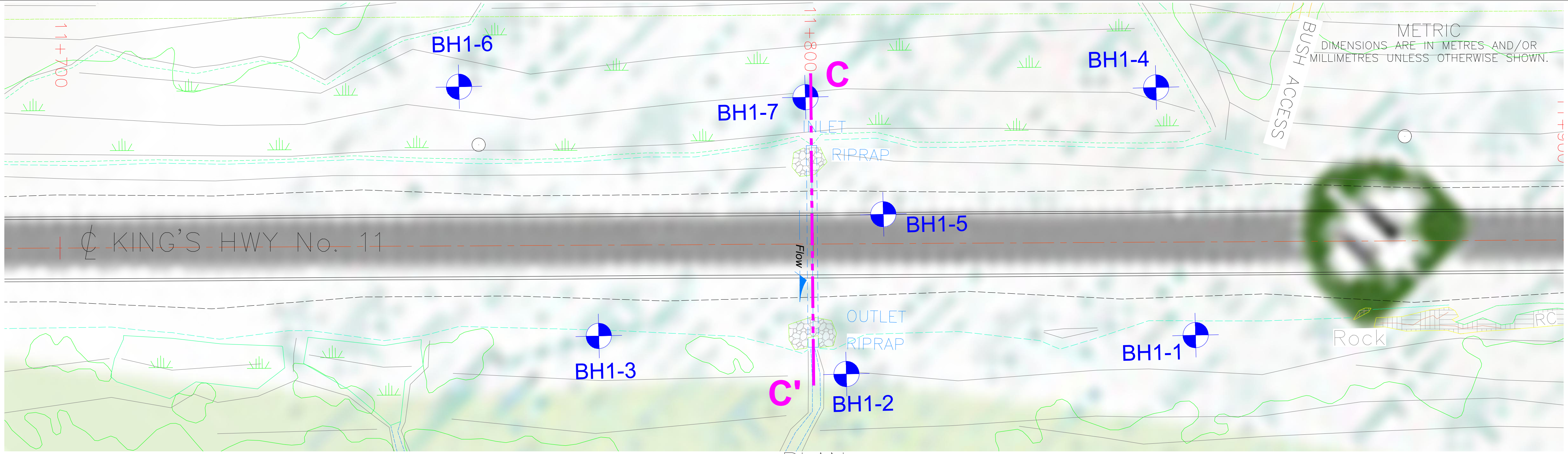
Highway 11 from Sand Dam Road Northerly 13.8 Km to Ellesmere Road (SW1)
Latitude: 46.549100°; Longitude: -79.590159°

BOREHOLE LOCATION PLAN & SOIL STRATA

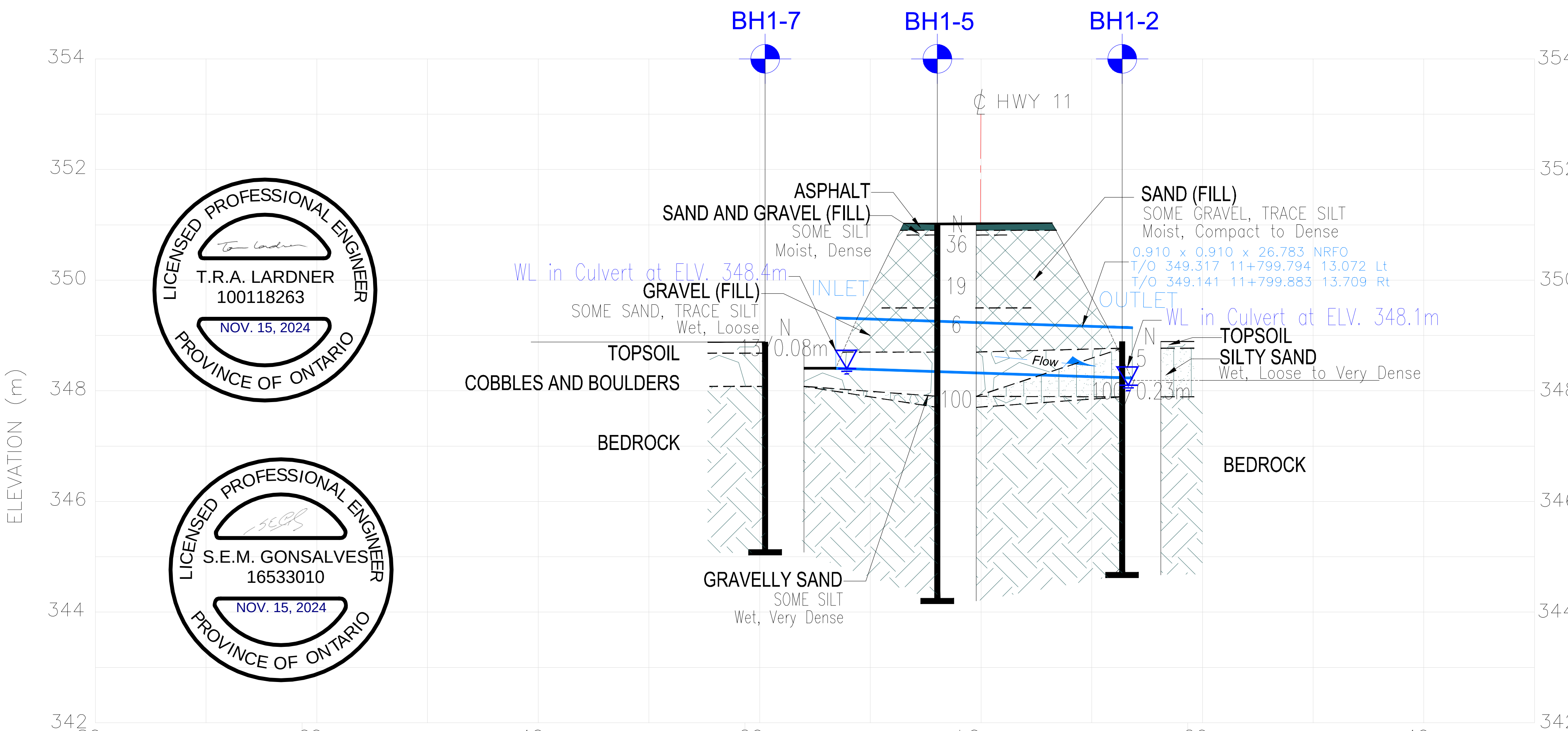
SHEET 2

EXP SERVICES INC.

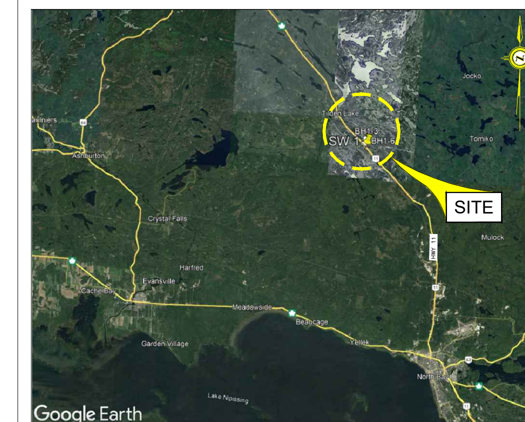
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN.



PLAN
0 5 10 m
SCALE



SECTION C-C'
HOR 0 5 7.5 m
VERT 0 1
SCALE



KEY PLAN
N.T.S.

LEGEND

Borehole Location
N Blows/0.3m (Std. Pen. Test, 475 J/blow)
Water Level In Culvert

SOIL STRATA SYMBOLS

TOPSOIL
ASPHALT
FILL
SAND/ SILTY SAND
GRAVELLY SAND/ SAND AND GRAVEL
SILT
COBBLES/BOULDERS
BEDROCK

BOREHOLE COORDINATES/ NAD 83/ MTM ON-10

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REVISIONS

NO	DATE	BY	DESCRIPTION
			SUBMISSION FOR MTO REVIEW

PROJECT No. ADM-23010055-A0		GEOCRES No. 31L12-002	
SUBM'D SH	CHKD. TL	DATE APRIL 22, 2024	SITE -
DRAWN SH	CHKD. TL	APPRD SG	DWG 02

Appendix C –
Borehole Logs

Explanation of Terms Used on Borehole Records

SOIL DESCRIPTION

Terminology describing common soil genesis:

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

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

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

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

Terminology describing soil structure:

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

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

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

Fissured: material breaks along plane of fracture.

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

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

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

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

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

Homogeneous: same color and appearance throughout.

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

Uniformly Graded: predominantly on grain size.

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

ISSMFE SOIL CLASSIFICATION

CLAY	SILT			SAND			GRAVEL			COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE		

0.002 0.006 0.02 0.06 0.2 0.6 2.0 6.0 20 60 200

EQUIVALENT GRAIN DIAMETER IN MILLIMETRES

CLAY (PLASTIC) TO	FINE	MEDIUM	CRS.	FINE	COARSE
SILT (NONPLASTIC)	SAND			GRAVEL	

UNIFIED SOIL CLASSIFICATION

Terminology describing materials outside the USCS, (e.g. particles larger than 76 mm, visible organic matter, construction debris) is based upon the proportion of these materials present and as described below in accordance with Canadian Foundation Engineering Manual (CFEM):

Table a: Percent or Proportion of Soil

Term	Description	Criteria
“trace”	trace gravel, trace sand, etc.	1% - 10%
“some”	some gravel, some sand, etc.	10% - 20%
Adjective	gravelly, sandy, silty and clayey	20% - 35%
“and”	and gravel, and sand, etc.	>35%
Noun	gravel, sand, silt, clay	>35% and main fraction

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

Table b: Apparent Density of Cohesionless Soil

	‘N’ Value (blows/0.3 m)
Very Loose	N<5
Loose	5≤N<10
Compact	10≤N<30
Dense	30≤N<50
Very Dense	50≤N

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

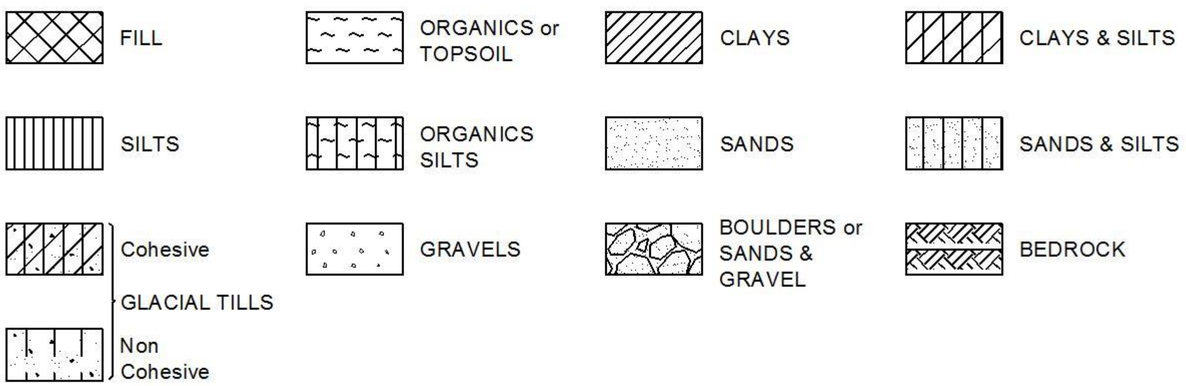
Table c: Consistency of Cohesive Soil

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

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

STRATA PLOT

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



WATER LEVEL MEASUREMENT

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

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

STRESS AND STRAIN

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

MECHANICALL PROPERIES OF SOIL

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

PHYSICAL PROPERTIES OF SOIL

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

RECORD OF BOREHOLE No BH1-1

METRIC

W.P. GWP 5151-21-00	LOCATION 5156652N, 297863E, NAD83 MTM Zone 10		ORIGINATED BY AM
DIST NER HWY 11	BOREHOLE TYPE NW Casing, NQ Core Barrel		COMPILED BY AM/CA
DATUM Geodetic	DATE 2023.12.20 - 2023.12.20	LATITUDE 46.549473 LONGITUDE -79.590459	CHECKED BY TL

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	NATURAL MOISTURE CONTENT w (%)
350.6	GROUND SURFACE				
350.5 / 0.0	**TOPSOIL,** ~ 130 mm thick				
0.1	**SILTY SAND,** trace to some gravel, light brown to grey, wet, loose to very dense	SS1	SS	4	~42
349.8 / 0.8	**SAND AND GRAVEL,** some silt, grey, wet, very dense	SS2	SS	50/ 0.10 m	~35
349.6 / 1.0	**COBBLES AND BOULDERS,** NQ coring commenced		NQ		
349.4 / 1.2	**BEDROCK,** grey with pink embedments, quartzofeldspathic gneiss Run 1: Start/End: 1.2 to 1.4 m Recovery: 100% RQD: 63% Run 2: Start/End: 1.4 to 2.9 m Recovery: 100% RQD: 93% Run 3: Start/End: 2.9 to 4.4 m Recovery: 100% RQD: 89%	RUN 1 RUN 2 RUN 3	NQ NQ NQ		
346.2 / 4.4	BOREHOLE TERMINATED AT ~ 4.4 m DEPTH Notes: 1. Groundwater level not measured due to water used for coring. 2. Borehole backfilled upon completion.				

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE (350, 349, 348, 347); DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa (○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × P. PENETROMETER; 20, 40, 60, 80, 100); PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) (20, 40, 60); UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) (GR SA SI CL)

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

EXP Services Inc.
Brampton, Ontario

Foundation Design

RECORD OF BOREHOLE No BH1-2

METRIC

W.P. GWP 5151-21-00 LOCATION 5156623N, 297900E, NAD83 MTM Zone 10 ORIGINATED BY AM

DIST NER HWY 11 BOREHOLE TYPE NW Casing, NQ Core Barrel COMPILED BY AM/CA

DATUM Geodetic DATE 2023.12.21 - 2023.12.21 LATITUDE 46.549212 LONGITUDE -79.589984 CHECKED BY TL

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
348.9	GROUND SURFACE										
348.8 / 0.0	**TOPSOIL,** ~ 130 mm thick								107.6		
0.1	**SILTY SAND,** grey, wet, loose		SS1	SS	5				~20		
347.9	- refusal on inferred bedrock at ~ 0.9 m depth, NQ coring commenced		SS2	NR	100/ 0.23 m		348				
1.0	**BEDROCK,** grey with pink embedments, quartzofeldspathic gneiss Run 1: Start/End: 1.0 to 1.5 m Recovery: 100% RQD: 47% Run 2: Start/End: 1.5 to 3.0 m Recovery: 100% RQD: 82% Run 3: Start/End: 3.0 to 4.2 m Recovery: 96% RQD: 48%		RUN 1	NQ			347				
			RUN 2	NQ			346				
344.7			RUN 3	NQ			345				
4.2	BOREHOLE TERMINATED AT ~ 4.2 m DEPTH Notes: 1. Groundwater level not measured due to water used for coring. 2. Borehole backfilled upon completion.										

Plot axes: DCPT 20 40 60 80 100; SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × P. PENETROMETER 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60

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

RECORD OF BOREHOLE No BH1-3

METRIC

W.P. GWP 5151-21-00 LOCATION 5156596N, 297919E, NAD83 MTM Zone 10 ORIGINATED BY AM

DIST NER HWY 11 BOREHOLE TYPE NW Casing, NQ Core Barrel COMPILED BY AM/CA

DATUM Geodetic DATE 2023.12.20 - 2023.12.21 LATITUDE 46.548971 LONGITUDE -79.589727 CHECKED BY TL

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	WATER CONTENT (%)	REMARKS
349.1	GROUND SURFACE					
349.0 / 0.1	**TOPSOIL,** ~ 50 mm thick					
	SILTY SAND, some gravel, trace organics, light brown, wet, loose	SS1	SS	6	o (~18)	
348.5 / 0.6	**BEDROCK,** grey with pink embedments, some oxidization, quartzofeldspathic gneiss Run 1: Start/End: 0.6 to 1.5 m Recovery: 100% RQD: 14% Run 2: Start/End: 1.5 to 3.0 m Recovery: 100% RQD: 70% Run 3: Start/End: 3.0 to 4.4 m Recovery: 100% RQD: 48%	RUN 1	NQ			
		RUN 2	NQ			
		RUN 3	NQ			
344.7 / 4.4	BOREHOLE TERMINATED AT ~ 4.4 m DEPTH Notes: 1. Groundwater level not measured due to water used for coring. 2. Borehole backfilled upon completion.					

Chart columns: STRAT PLOT; GROUND WATER CONDITIONS; ELEVATION SCALE (349, 348, 347, 346, 345); DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa — O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × P. PENETROMETER (20, 40, 60, 80, 100); PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L — WATER CONTENT (%) (20, 40, 60); UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

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

RECORD OF BOREHOLE No BH1-4

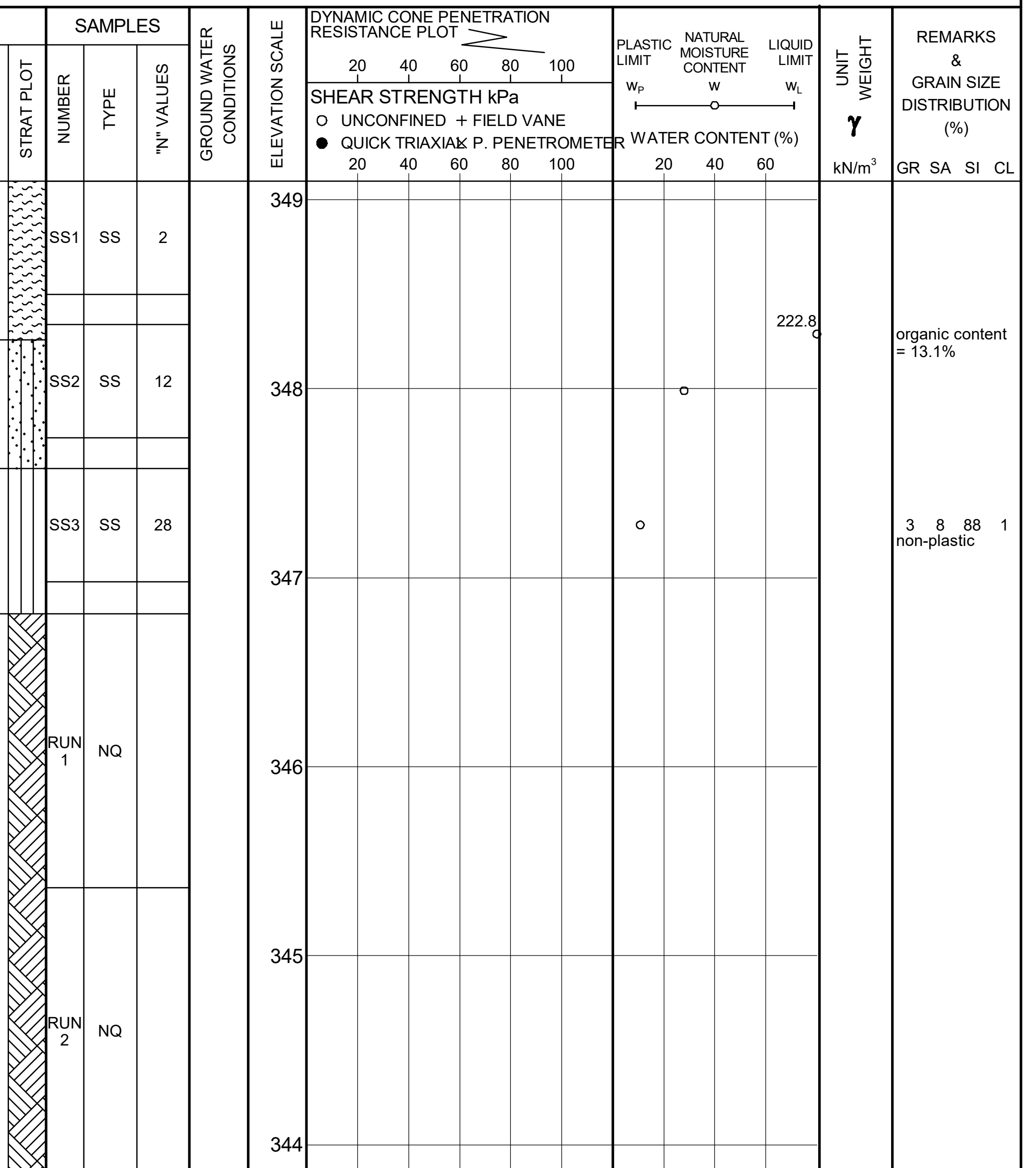
METRIC

W.P. GWP 5151-21-00 | LOCATION 5156625N, 297844E, NAD83 MTM Zone 10 | ORIGINATED BY SF/AH

DIST NER | HWY 11 | BOREHOLE TYPE NW Casing, NQ Core Barrel | COMPILED BY IL/CA

DATUM Geodetic | DATE 2024.01.09 - 2024.01.11 | LATITUDE 46.54923 | LONGITUDE -79.590715 | CHECKED BY TL

ELEV / DEPTH (m)	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
349.1	GROUND SURFACE					
0.0	**TOPSOIL** ~ 860 mm thick	SS1	SS	2	222.8	organic content = 13.1%
348.3 / 0.8	**SAND TO SILTY SAND,** some gravel, trace organics, grey, wet, compact	SS2	SS	12		
347.6 / 1.5	**SILT,** trace gravel, trace sand, trace clay, grey, wet, compact	SS3	SS	28		3 8 88 1 non-plastic
346.8 / 2.3	**BEDROCK,** grey with pink embedments, quartzofeldspathic gneiss Run 1: Start/End: 2.3 to 3.7 m Recovery: 95% RQD: 30% Run 2: Start/End: 3.7 to 5.3 m Recovery: 100% RQD: 79%	RUN 1	NQ			
		RUN 2	NQ			
343.8 / 5.3	BOREHOLE TERMINATED AT ~ 5.3 m DEPTH Notes: 1. Groundwater level not measured due to water used for coring. 2. Borehole backfilled upon completion.					



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

RECORD OF BOREHOLE No BH1-5

METRIC

W.P.	GWP 5151-21-00	LOCATION	5156611N, 297881E, NAD83 MTM Zone 10	ORIGINATED BY	ST
DIST	NER HWY 11	BOREHOLE TYPE	NW Casing, NQ Core Barrel	COMPILED BY	IL/CA
DATUM	Geodetic	DATE	2023.11.07 - 2023.11.07 LATITUDE 46.549107 LONGITUDE -79.590224	CHECKED BY	TL

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
351.0	GROUND SURFACE					
350.9 / 0.1	**ASPHALT,** ~ 115 mm thick					
350.8 / 0.2	**SAND AND GRAVEL (FILL),** some silt, black, moist, dense					
0.2	**SAND (FILL),** some gravel, trace silt, brown, moist, compact to dense	SS1	SS	36		
	- cobbles at ~ 0.8 m depth	SS2	SS	19	350	
349.5 / 1.5	**GRAVEL (FILL),** some sand, trace silt, brown, wet, loose	SS3	SS	6	349	
348.7 / 2.3	**BOULDERS,** NQ coring commenced		NQ		348	
347.9 / 3.1	**GRAVELLY SAND,** some silt, brown wet, very dense	SS4	SS	100		21 66 (13)
347.7 / 3.3	**BEDROCK,** grey with pink embedments, quartzofeldspathic gneiss Run 1: Start/End: 3.3 to 4.8 m Recovery: 92% RQD: 47% Run 2: Start/End: 4.8 to 5.7 m Recovery: 100% RQD: 92% Run 3: Start/End: 5.7 to 6.8 m Recovery: 100% RQD: 50%	RUN 1 RUN 2 RUN 3	NQ NQ NQ		347 346 345	
344.2 / 6.8	BOREHOLE TERMINATED AT ~ 6.9 m DEPTH Notes: 1. Groundwater level measured at 5.9 m depth prior to rock coring. 2. Borehole backfilled upon completion.					

Chart columns: DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × P. PENETROMETER; 20 40 60 80 100); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³; GROUND WATER CONDITIONS; STRAT PLOT.

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

RECORD OF BOREHOLE No BH1-6

 METRIC

W.P.	GWP 5151-21-00	LOCATION	5156559N, 297909E, NAD83 MTM Zone 10	ORIGINATED BY	SF/AH
DIST	NER HWY 11	BOREHOLE TYPE	NW Casing, NQ Core Barrel	COMPILED BY	IL/CA
DATUM	Geodetic	DATE	2024.01.11 - 2024.01.11 LATITUDE 46.548641 LONGITUDE -79.589861	CHECKED BY	TL

ELEV DEPTH	DESCRIPTION (SOIL PROFILE)	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
349.2	GROUND SURFACE						
0.0	**TOPSOIL,** ~ 1520 mm thick	SS1	SS	WH			
		SS2	SS	2		104.1	
347.7 / 1.5	**SAND,** trace gravel, trace silt, grey, wet, compact to dense	SS3	SS	10			3 93 (4) non-plastic
		SS4	SS	36			
346.3 / 2.9	**BOULDERS,** NQ coring commenced		NQ				
345.4 / 3.8	**BEDROCK,** grey with pink embedments, quartzofeldspathic gneiss Run 1: Start/End: 3.8 to 4.1 m Recovery: 100% RQD: 31% Run 2: Start/End: 4.1 to 5.6 m Recovery: 100% RQD: 83% Run 3: Start/End: 5.6 to 6.9 m Recovery: 100% RQD: 80%	RUN 1	NQ				
		RUN 2	NQ				
		RUN 3	NQ				
342.3 / 6.9	BOREHOLE TERMINATED AT ~ 6.9 m DEPTH Notes: 1. WH = Weight of hammer 2. Groundwater level not measured due to water used for coring. 3. Borehole backfilled upon completion.						

Chart headings: DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × P. PENETROMETER 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³; ELEVATION SCALE 349, 348, 347, 346, 345, 344, 343.

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

RECORD OF BOREHOLE No BH1-7

METRIC

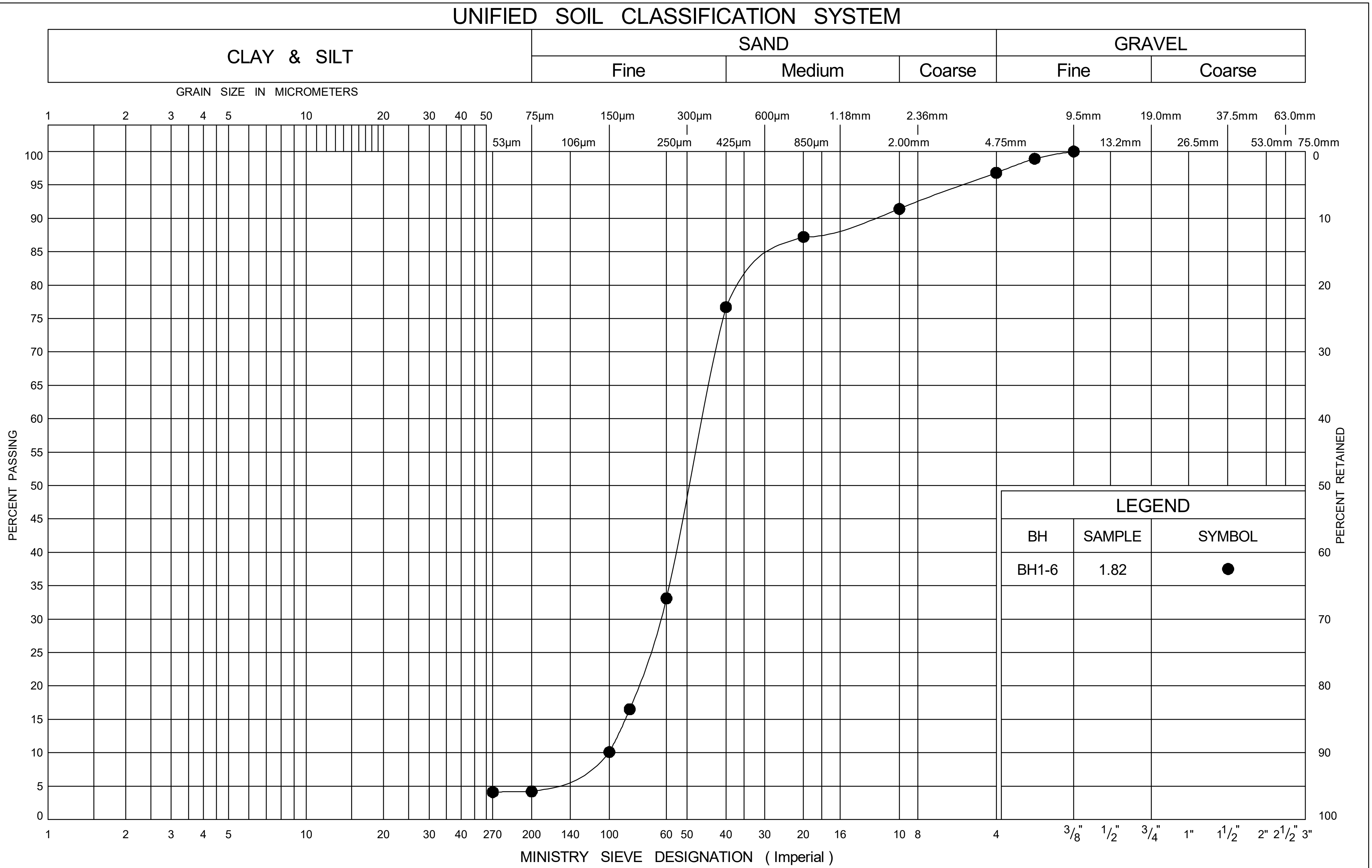
W.P. GWP 5151-21-00	LOCATION 5156593N, 297877E, NAD83 MTM Zone 10		ORIGINATED BY SF/AH
DIST NER HWY 11	BOREHOLE TYPE NW Casing, NQ Core Barrel		COMPILED BY IL/CA
DATUM Geodetic	DATE 2024.01.11 - 2024.01.11	LATITUDE 46.548942 LONGITUDE -79.590272	CHECKED BY TL

ELEV DEPTH	DESCRIPTION (SOIL PROFILE)	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%) / REMARKS
348.9	GROUND SURFACE					
0.0 / 348.7 / 0.2	**TOPSOIL,** ~ 230 mm thick - refusal on cobbles/boulders at ~ 0.2 m depth	SS1	SS	13/ 0.08 m		427.4
	COBBLES AND BOULDERS, NQ coring commenced		NQ			
348.1 / 0.8	**BEDROCK,** grey with pink embedments, quartzofeldspathic gneiss Run 1: Start/End: 0.8 to 2.3 m Recovery: 90% RQD: 40% Run 2: Start/End: 2.3 to 3.8 m Recovery: 100% RQD: 88%	RUN 1	NQ		348 / 347	
		RUN 2	NQ		346	
345.1 / 3.8	BOREHOLE TERMINATED AT ~ 3.8 m DEPTH Notes: 1. Groundwater level not measured due to water used for coring. 2. Borehole backfilled upon completion.					

Column headings: SOIL PROFILE; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × P. PENETROMETER 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT W_L, WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m^3; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

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

Appendix D –
Laboratory Data



ONTARIO MOT GRAIN SIZE HWY 11 - 2+1 - SW1.GPJ ONTARIO MOT.GDT 4/11/24

Ministry of Transportation

Ontario

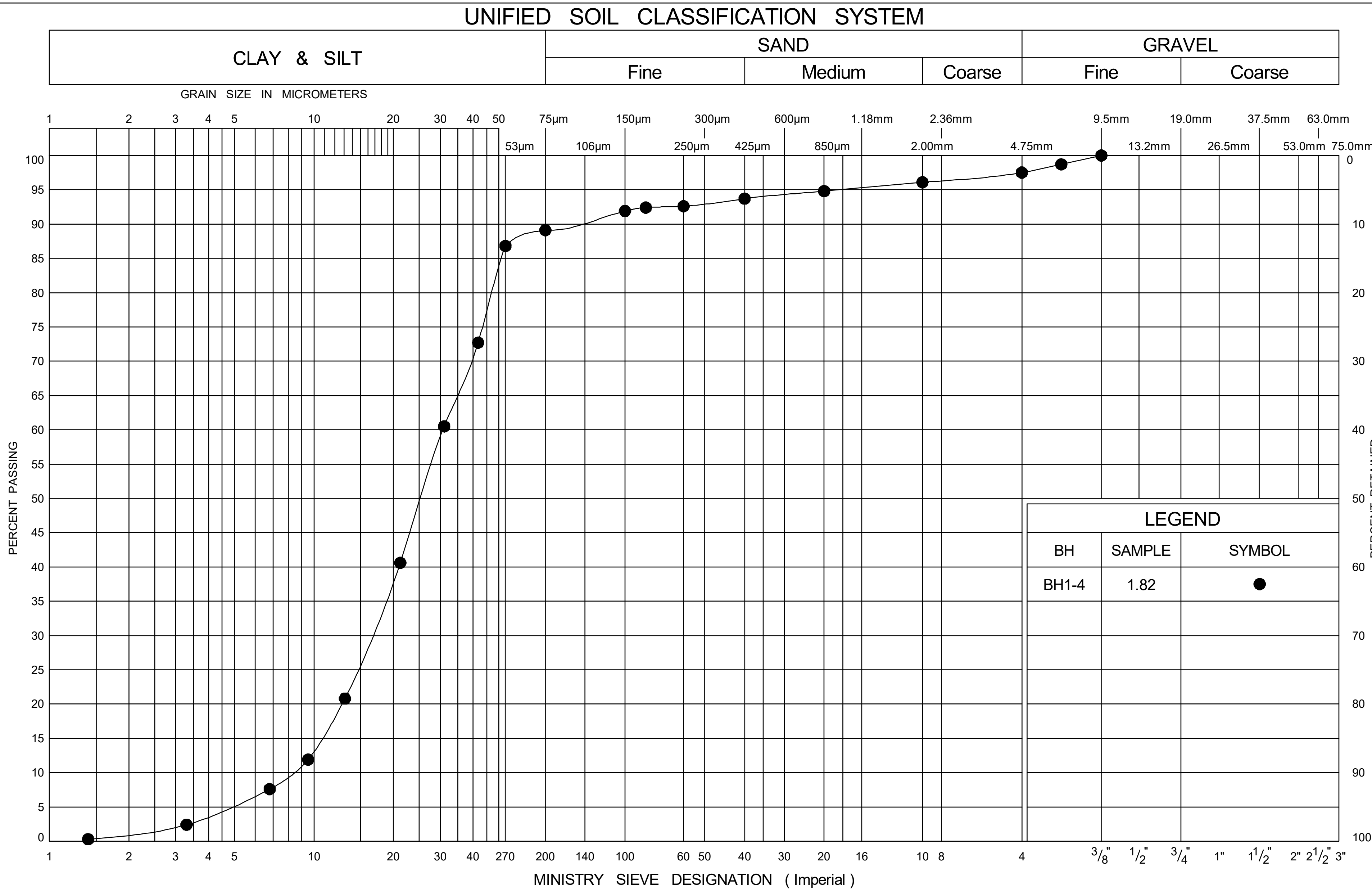
GRAIN SIZE DISTRIBUTION

Sand

FIG No 1

W P 5151-21-00

Hwy 11 2+1 - SW1



Ministry of Transportation
Ontario

GRAIN SIZE DISTRIBUTION

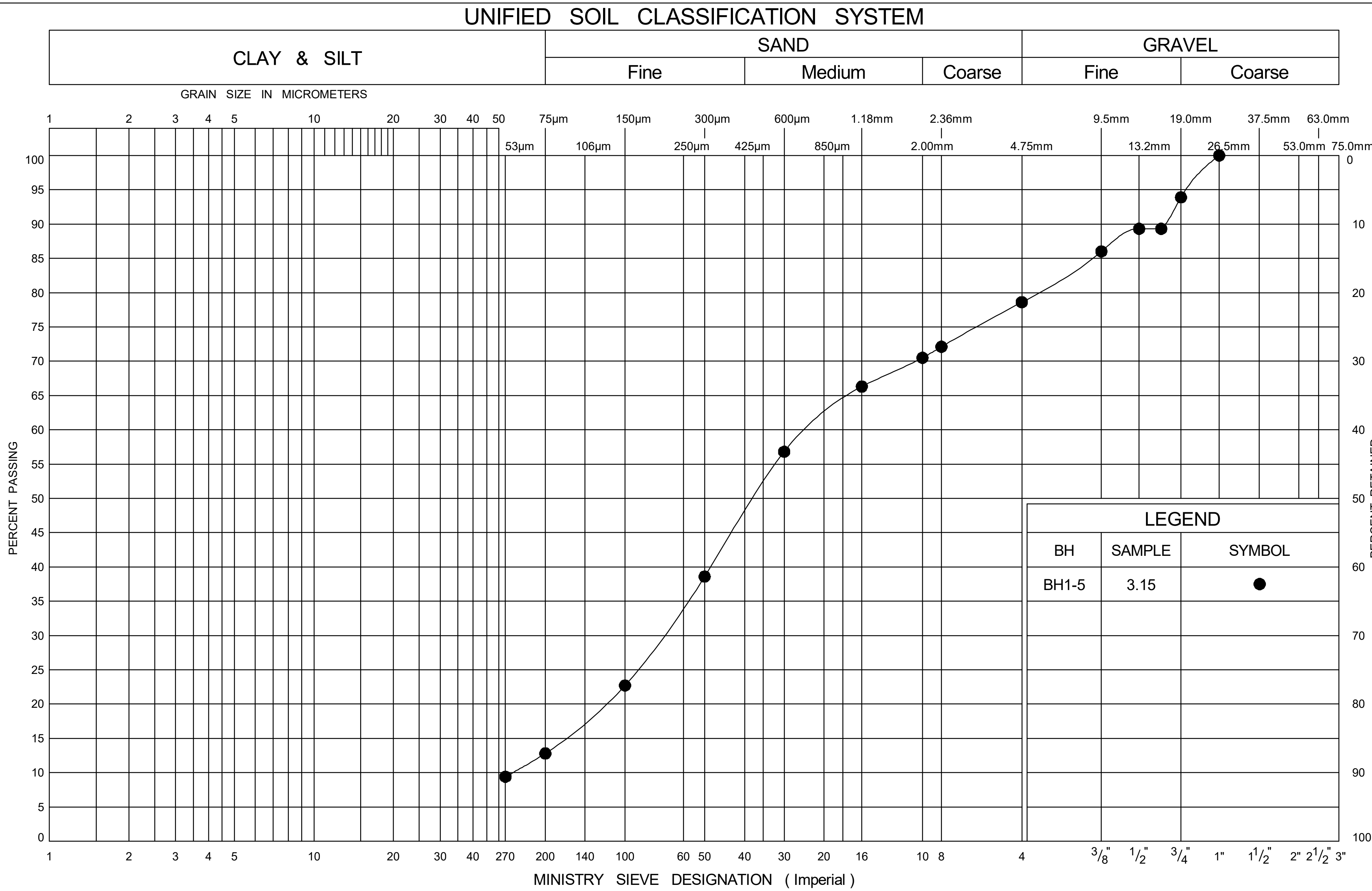
Silt

FIG No 2

W P 5151-21-00

Hwy 11 2+1 - SW1

ONTARIO MOT GRAIN SIZE HWY 11 - 2+1 - SW1.GPJ ONTARIO MOT.GDT 4/11/24



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Ontario

GRAIN SIZE DISTRIBUTION

Gravelly Sand

FIG No 3

W P 5151-21-00

Hwy 11 2+1 - SW1

Appendix E –
Bedrock Core Photographs


Photograph E1. Rock cores from BH1-1. Top: Run 1, Middle: Run 2, Bottom: Run 3.


Photograph E2. Rock cores from BH1-2. Top: Run 1, Middle: Run 2, Bottom: Run 3.


Photograph E3. Rock cores from BH1-3. Top: Run 1, Middle: Run 2, Bottom: Run 3.


Photograph E4. Rock cores from BH1-4. Top: Run 1, Middle and Bottom: Run 2.


Photograph E5. Rock cores from BH1-6. Top: Run 1, Middle: Run 2, Bottom: Run 3.


Photograph E6. Rock cores from BH1-7. Top: Run 1, Middle: Run 2.

Appendix F –
Slope Stability Analyses

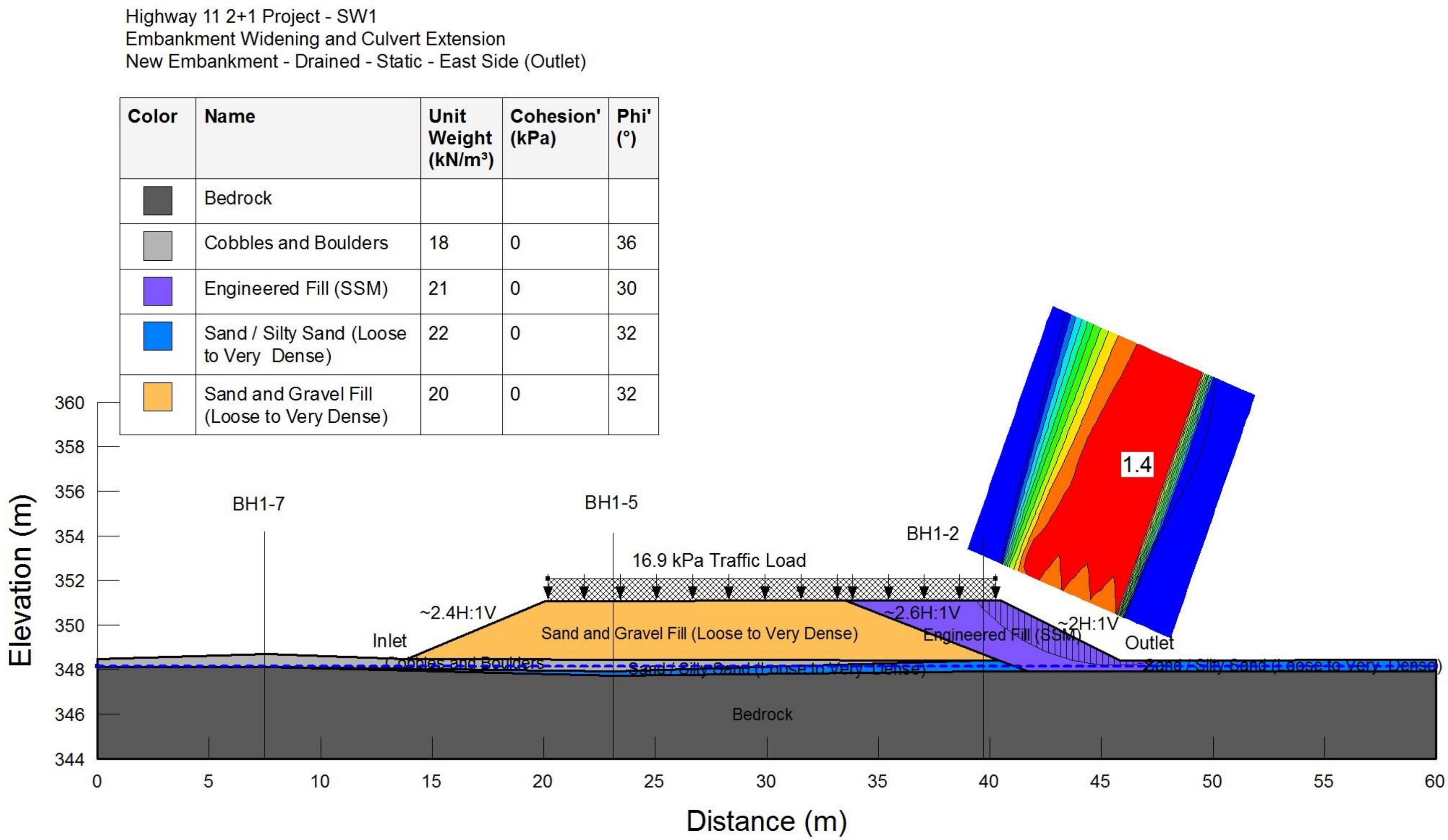


Figure F1: Slope stability analysis for new east (outlet) side of embankment (2H:1V) – drained, static condition

Highway 11 2+1 Project - SW1
Embankment Widening and Culvert Extension
New Embankment - Drained - Seismic - East Side (Outlet)

Color	Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
	Bedrock			
	Cobbles and Boulders	18	0	36
	Engineered Fill (SSM)	21	0	30
	Sand / Silty Sand (Loose to Very Dense)	22	0	32
	Sand and Gravel Fill (Loose to Very Dense)	20	0	32

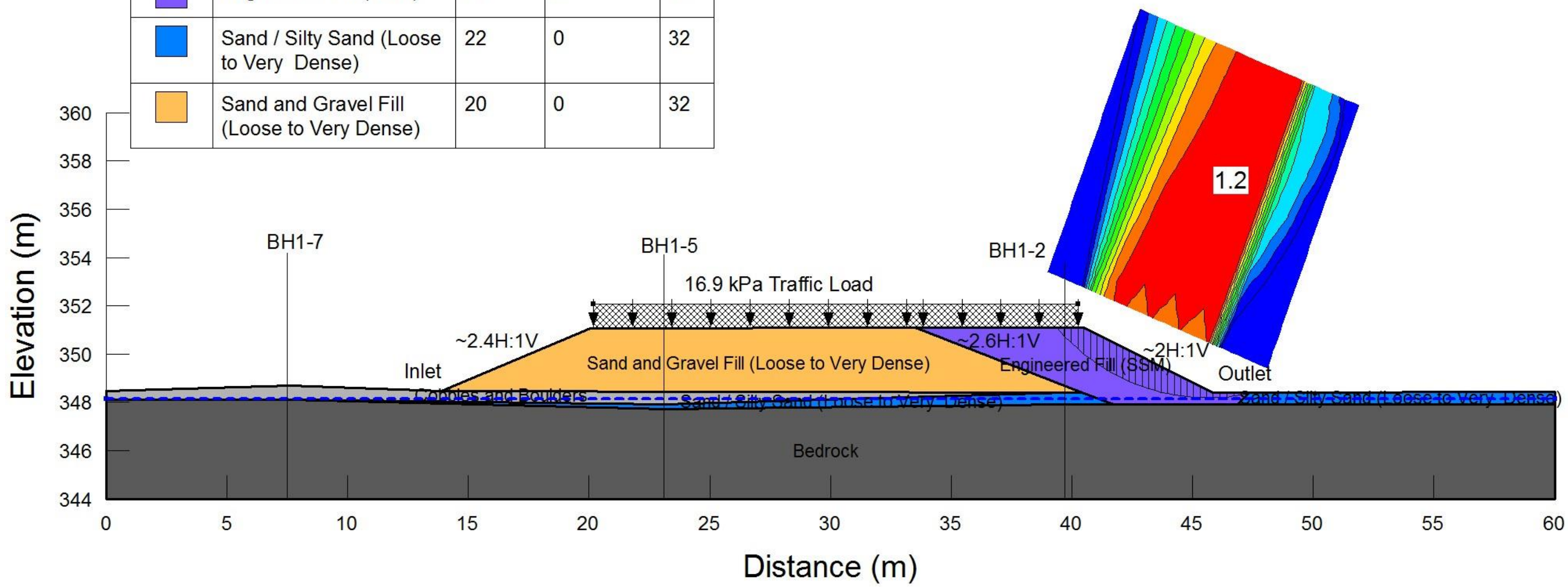


Figure F2: Slope stability analysis for new east (outlet) side of embankment (2H:1V) – drained, seismic condition

Appendix G –
Seismic Hazard Calculation

2020 National Building Code of Canada Seismic Hazard Tool

This application provides seismic values for the design of buildings in Canada under Part 4 of the National Building Code of Canada (NBC) 2020 as prescribed in Article 1.1.3.1. of Division B of the NBC 2020.

Seismic Hazard Values

User requested values

Code edition	NBC 2020
Site designation X_S	X_B
Latitude (°)	46.549
Longitude (°)	-79.59

Please select one of the tabs below.

NBC 2020 Additional Values Plots API

Background Information

The 5%-damped spectral acceleration ($S_a(T,X)$, where T is the period, in s, and X is the site designation) and peak ground acceleration (PGA(X)) values are given in units of acceleration due to gravity (g, 9.81 m/s^2). Peak ground velocity (PGV(X)) values are given in m/s. Probability is expressed in terms of percent exceedance in 50 years. Further information on the calculation of seismic hazard is provided under the *Background Information* tab.

The 2%-in-50-year seismic hazard values are provided in accordance with Article 4.1.8.4. of the NBC 2020. The 5%- and 10%-in-50-year values are provided for additional performance checks in accordance with Article 4.1.8.23. of the NBC 2020.

See the *Additional Values* tab for additional seismic hazard values, including values for other site designations, periods, and probabilities not defined in the NBC 2020.

NBC 2020 - 2%/50 years (0.000404 per annum) probability

$S_a(0.2, X_B)$	$S_a(0.5, X_B)$	$S_a(1.0, X_B)$	$S_a(2.0, X_B)$	$S_a(5.0, X_B)$	$S_a(10.0, X_B)$	$PGA(X_B)$	$PGV(X_B)$
0.356	0.167	0.0815	0.0365	0.00928	0.00332	0.187	0.111

The log-log interpolated 2%/50 year $S_a(4.0, X_B)$ value is : **0.0130**

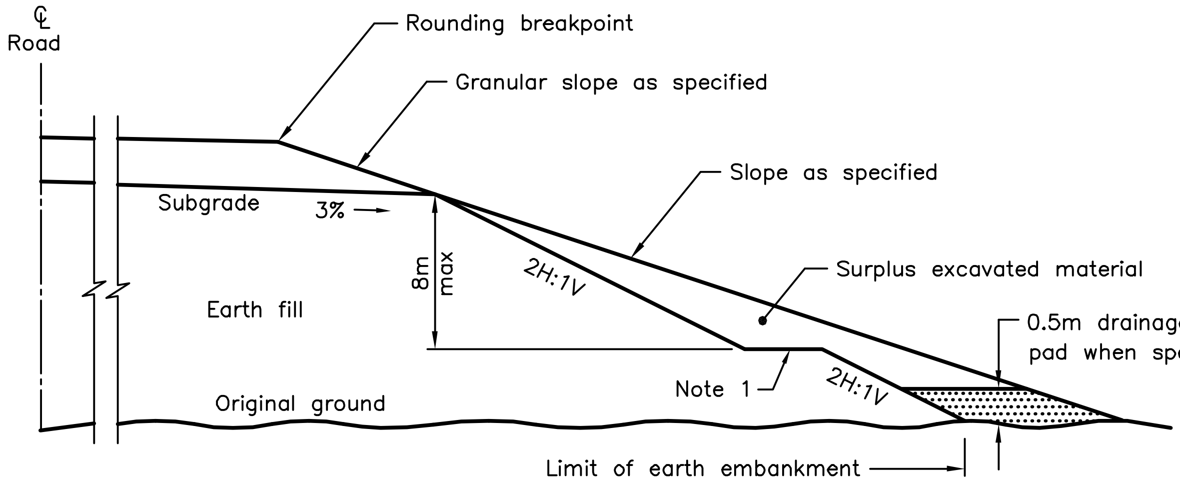
▶ Tables for 5% and 10% in 50 year values

Download CSV

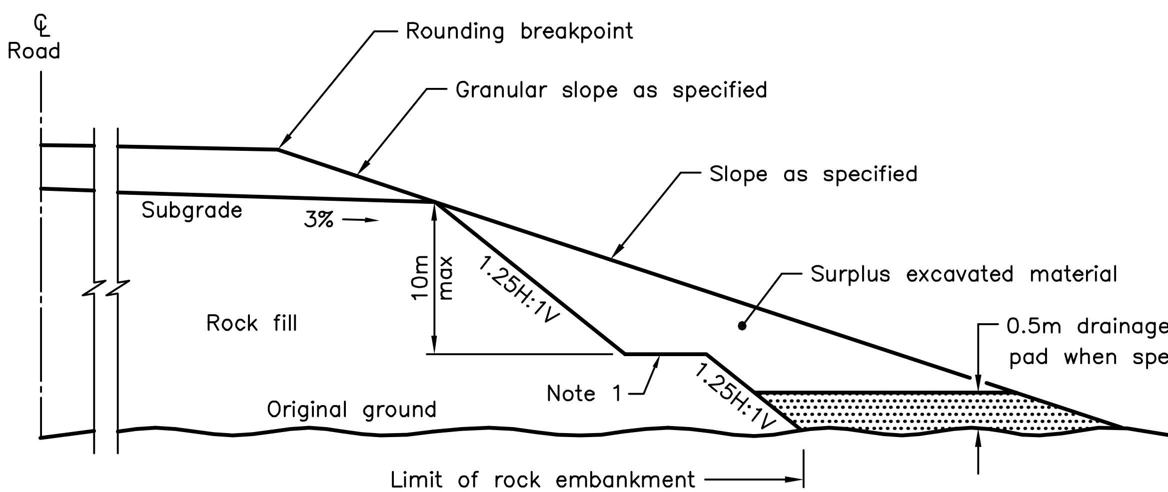
← Go back to the seismic hazard calculator form

Date modified: 2021-04-06

Appendix H – OPSDs



EARTH EMBANKMENT



ROCK EMBANKMENT

NOTES:

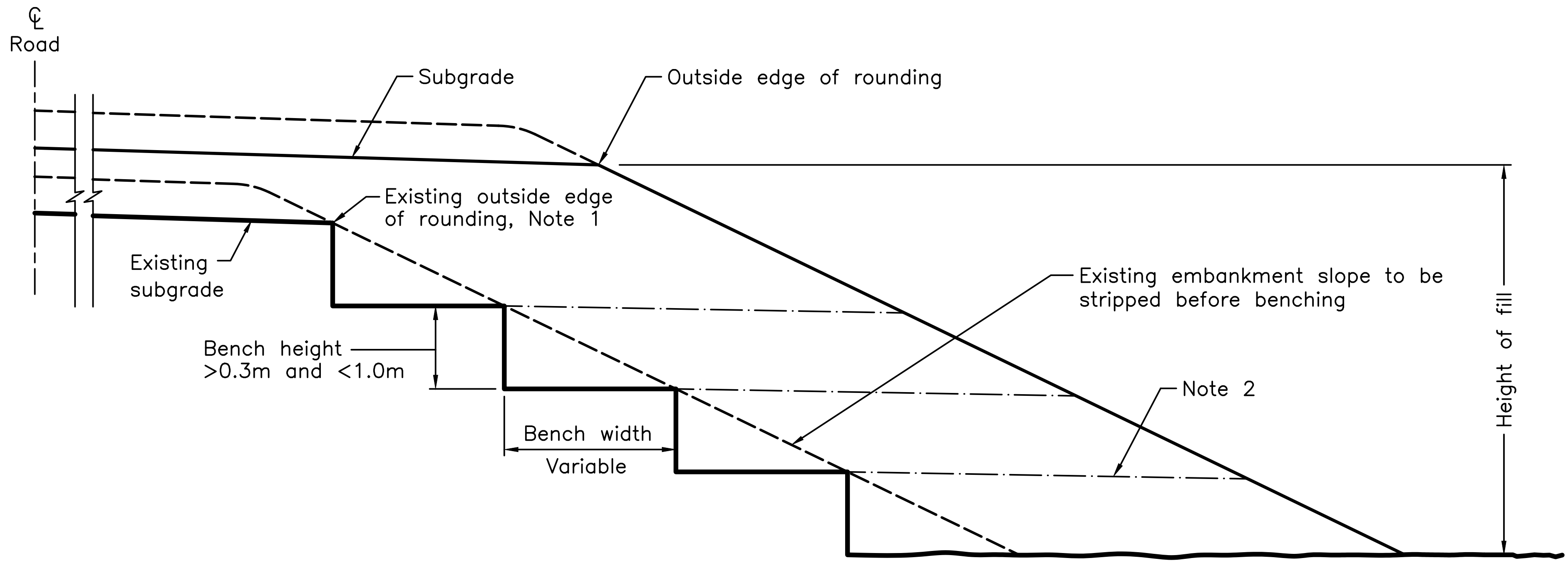
1 Benches 2m minimum in width are required along slopes at maximum vertical intervals as shown.

A Height of fill is the vertical difference between top of subgrade and top of original ground measured at new road centreline.

B Surplus excavated material placed shall not extend beyond the right-of-way.

C All dimensions are in metres unless otherwise shown.

ONTARIO PROVINCIAL STANDARD DRAWING	Nov 2016	Rev 3
SLOPE FLATTENING USING SURPLUS EXCAVATED MATERIAL ON EARTH OR ROCK EMBANKMENT		
	OPSD 202.010	



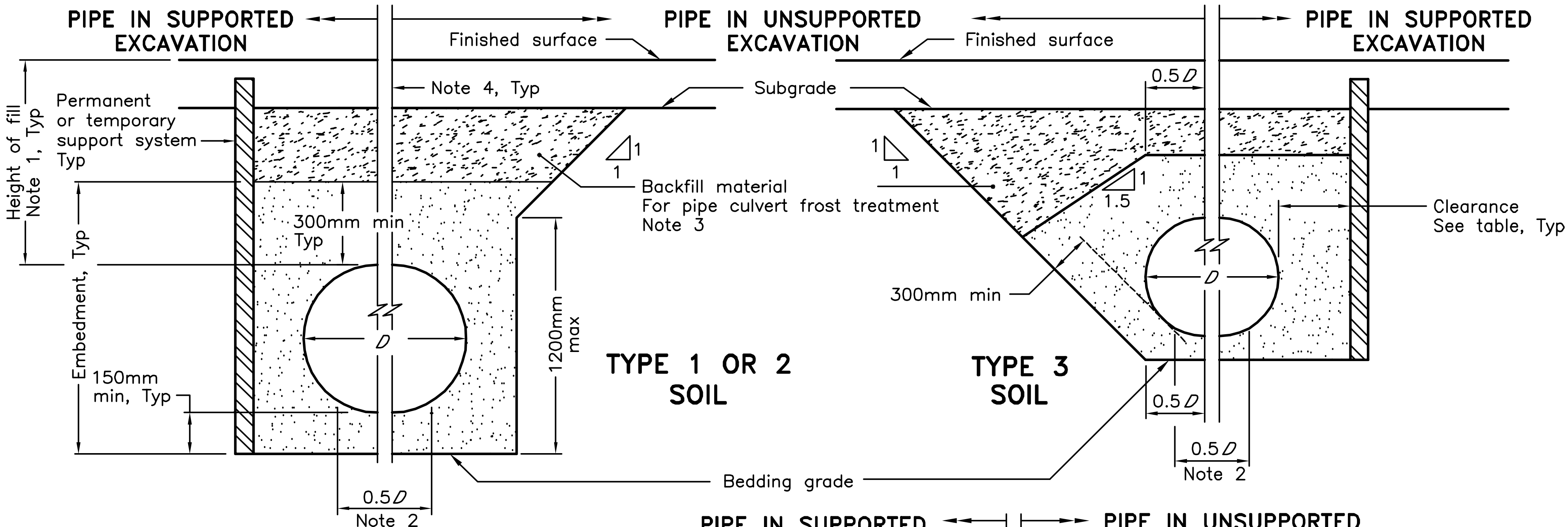
NOTES:

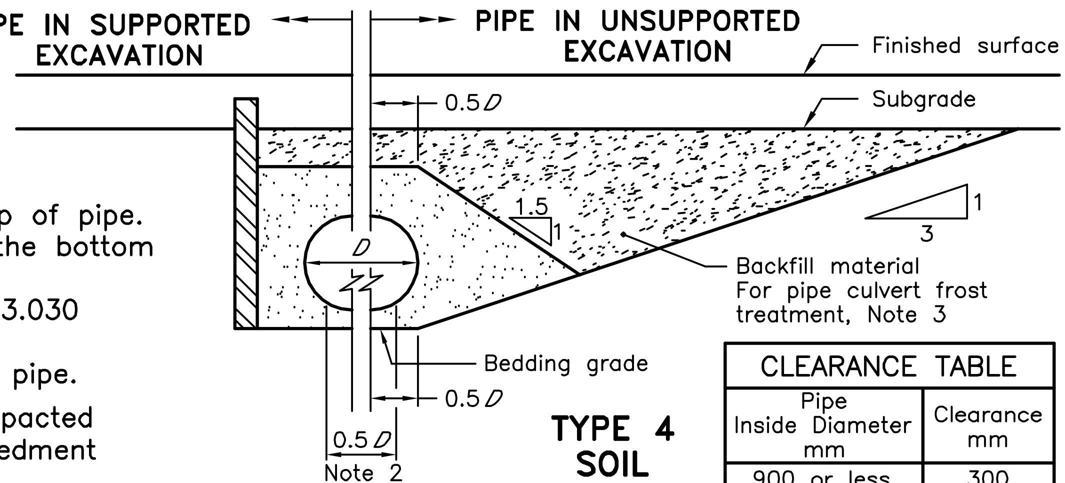
1 When the subgrade is below the existing outside edge of rounding, benching shall be carried out below the point where the subgrade intersects the existing slope.

2 Benches shall be excavated one level at a time and the fill placed and compacted before the next bench is excavated.

A Benching is not required on existing slopes flatter than 3H:1V.

ONTARIO PROVINCIAL STANDARD DRAWING	Apr 2019	Rev 4
BENCHING OF EARTH SLOPES		OPSD 208.010





LEGEND:

D – Inside diameter

NOTES:

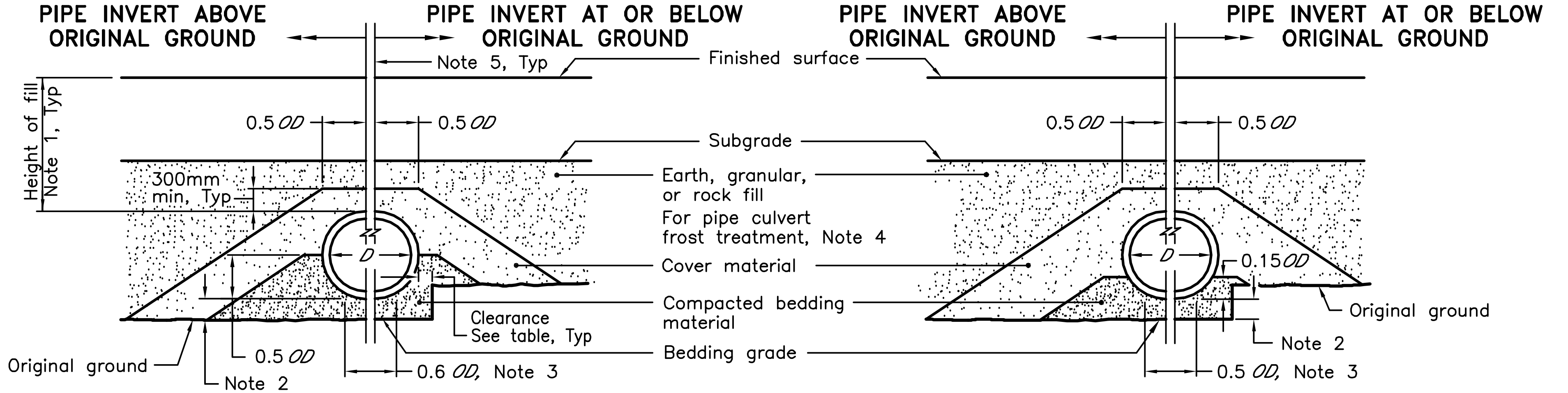
1 Height of fill is measured from the finished surface to top of pipe.
2 The pipe bed shall be compacted and shaped to receive the bottom of the pipe.
3 Pipe culvert frost treatment shall be according to OPSD 803.030 and 803.031.
4 Condition of excavation is symmetrical about centreline of pipe.

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

CLEARANCE TABLE

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

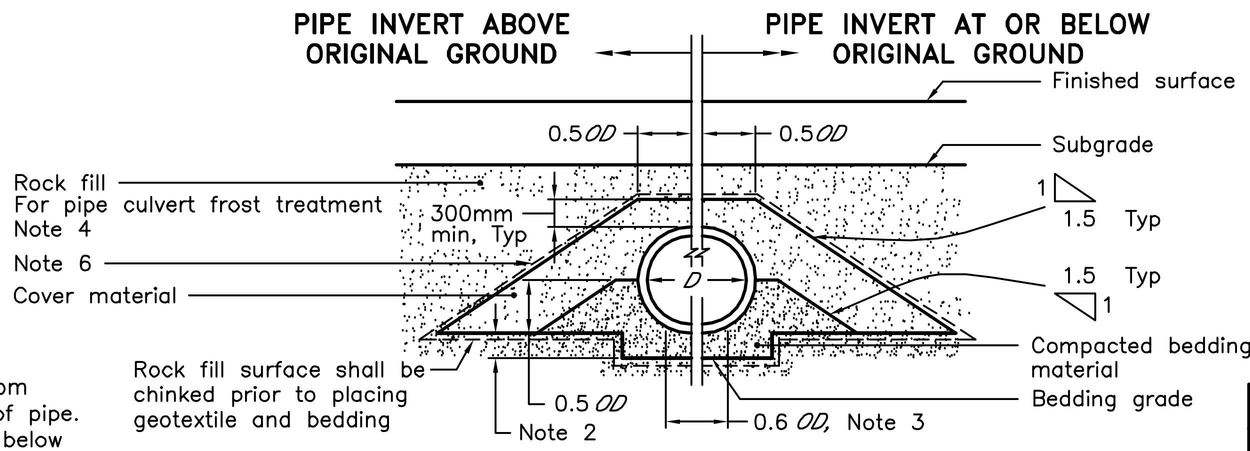
ONTARIO PROVINCIAL STANDARD DRAWING	Nov 2014	Rev 3
FLEXIBLE PIPE EMBEDMENT AND BACKFILL EARTH EXCAVATION	**OPSD 802.010**	



CLASS B BEDDING

EARTH AND ROCK EXCAVATION

CLASS C BEDDING



PIPE BEDDING AND COVER
WITH ROCK FILL UNDER AND OVER THE PIPE

LEGEND:

D – Inside diameter
OD – Outside diameter

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

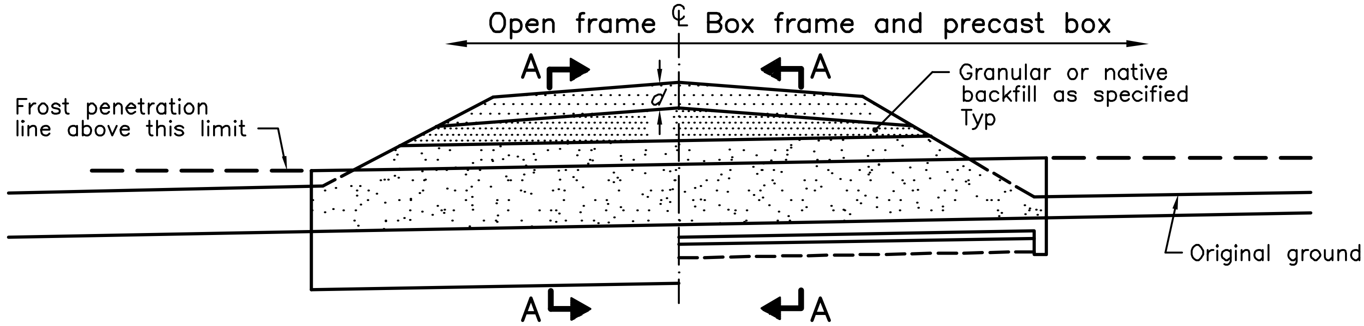
NOTES:

1 Height of fill is measured from the finished surface to top of pipe.
2 The minimum bedding depth below the pipe shall be 0.15*D*, except on a rock foundation where the minimum bedding depth shall be 0.25*D*. 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.

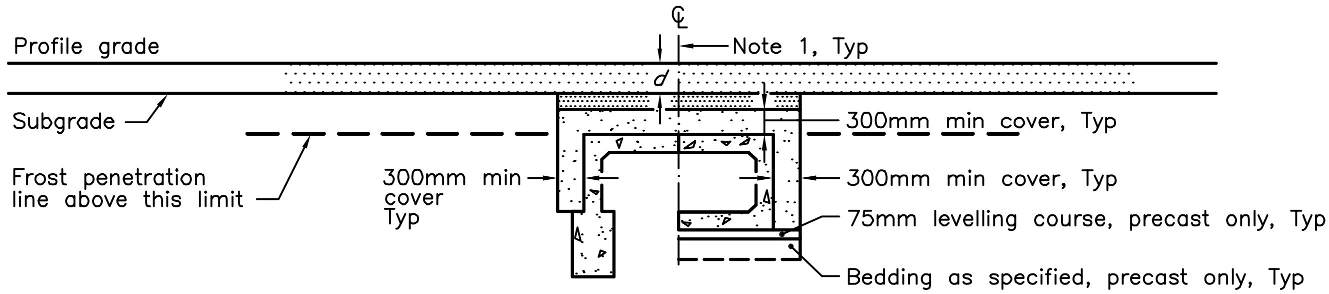
A All dimensions are in metres unless otherwise shown.

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

FROST PENETRATION LINE AT OR ABOVE TOP OF CULVERT

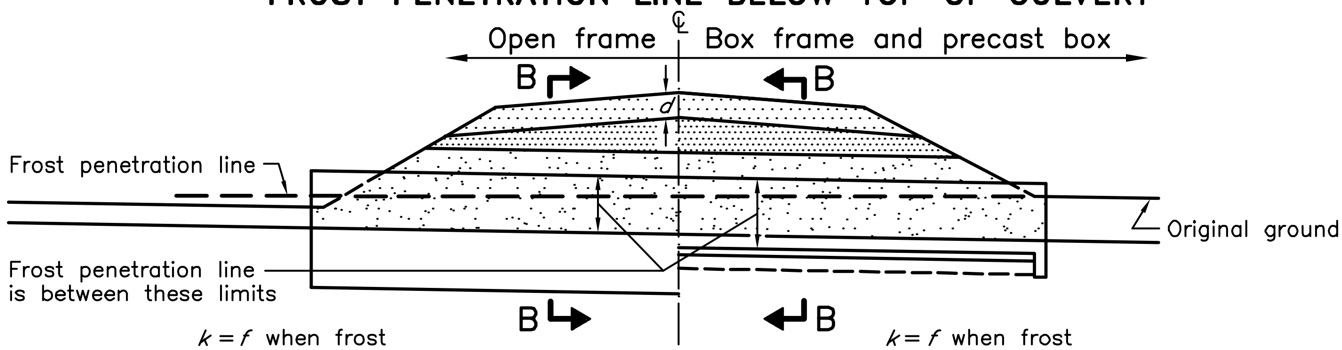


LONGITUDINAL SECTION

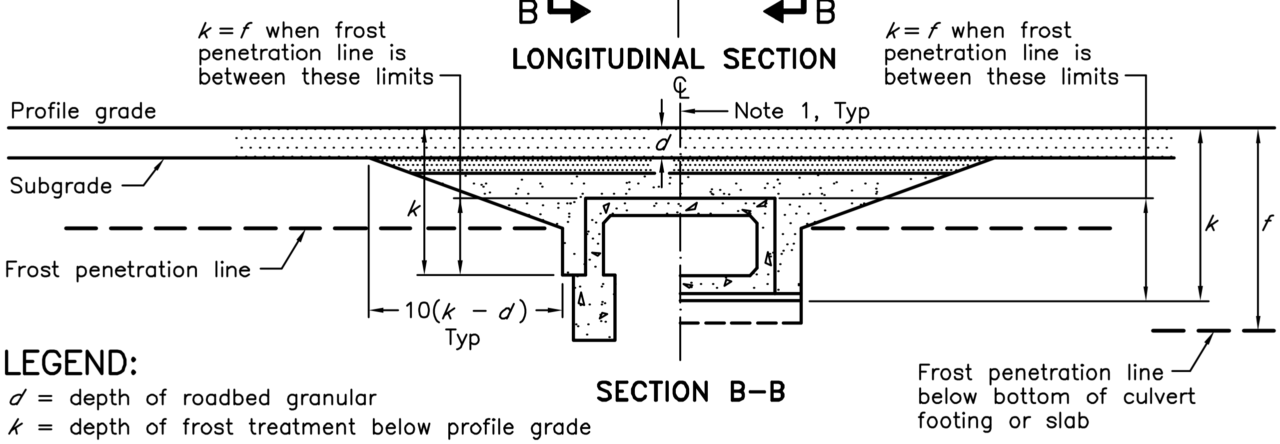


SECTION A–A

FROST PENETRATION LINE BELOW TOP OF CULVERT



LONGITUDINAL SECTION



SECTION B–B

LEGEND:

d = depth of roadbed granular

k = depth of frost treatment below profile grade

f = depth of frost penetration below profile grade

NOTES:

1 Condition of frost treatment symmetrical about centreline of culvert.

A Bedding, levelling, and cover material shall be granular as specified.

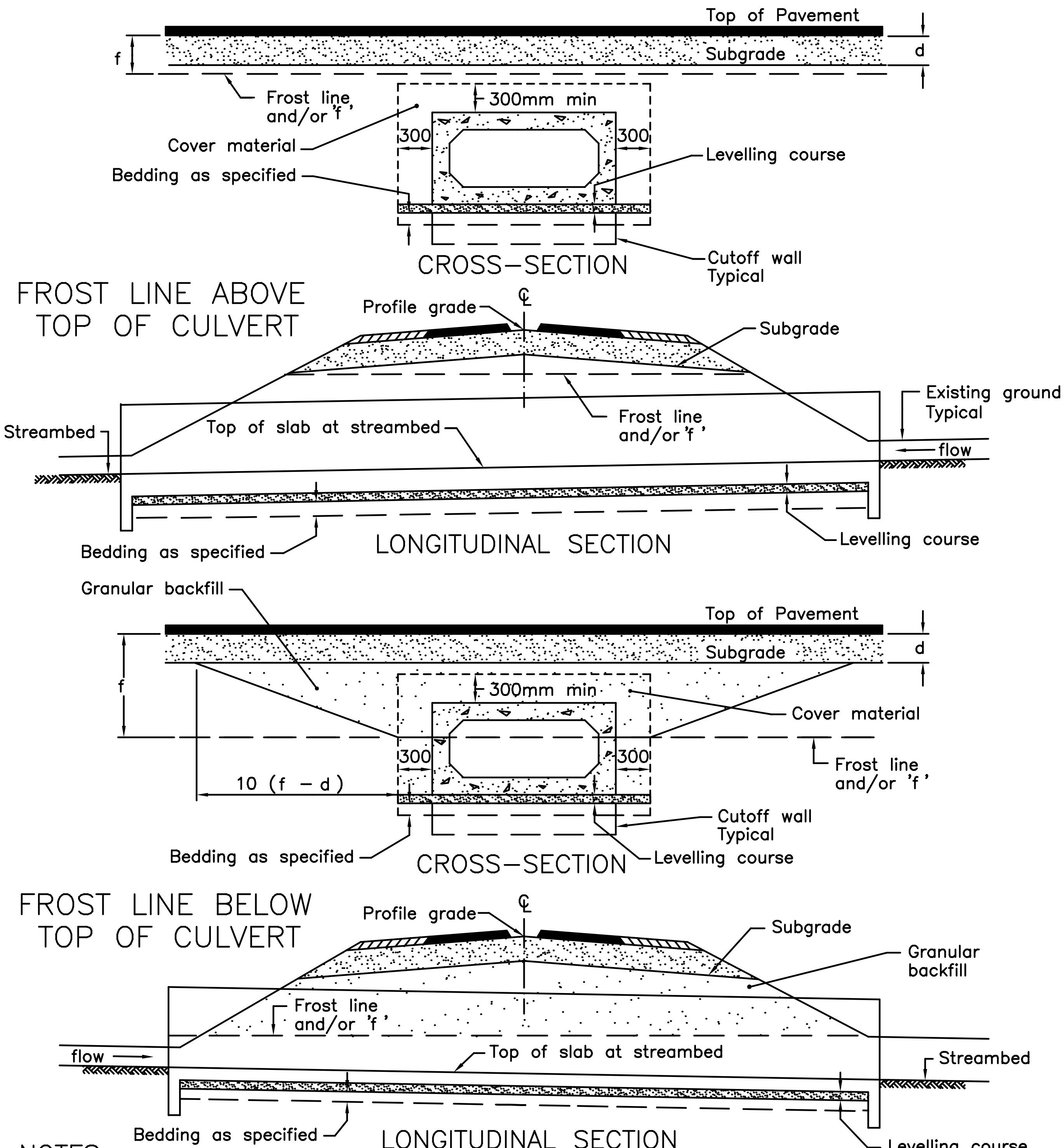
B The depth of roadbed granular shall be 600mm minimum.

C The maximum depth of frost treatment shall be bottom of box frame or top of footing.

D All dimensions are in millimetres unless otherwise shown.

ONTARIO PROVINCIAL STANDARD DRAWING	Nov 2010	Rev 2
BACKFILL AND COVER FOR CONCRETE CULVERTS WITH SPANS LESS THAN OR EQUAL TO 3.0M	OPSD 803.010	

ONTARIO STANDARDS · MUNICIPAL – PROVINCIAL · OPS



NOTES:

A Bedding, levelling, cover and backfill material to be granular as specified.

B Where frost line is below bottom of levelling course, frost tapers start at the bottom of levelling course.

C All dimensions are in millimetres unless otherwise shown.

LEGEND:

d = Denotes depth of granular (roadbed)

f = Depth of frost treatment=___ (measured from profile grade)

MINISTRY OF TRANSPORTATION ONTARIO DRAWING	Date	1994 05 25	Rev	
BEDDING AND BACKFILL FOR PRECAST CONCRETE BOX CULVERTS	Issue Date WP Issued by			
	MTOD – 803.021			

- A drainage system behind a retaining structure should ensure that a groundwater table does not exist above the footing level. Preferably, the ground water level is controlled by the use of free-draining granular backfill and a collection system such as weep holes or perforated drains at the footing level. These weep holes and drains should be inspected and maintained to ensure that they do not become blocked. If free-draining, granular backfill is not employed, the permeability of the backfill and the hydrostatic head will control the extent to which the groundwater table can be depressed locally by seepage towards a footing drain. In practice, design for frost protection is best done using free-draining backfill.
  The design should also consider the risk of unusually large inflows of water creating a temporary hydrostatic head of water behind the wall. An example is the overtopping of a retaining wall, adjoining a large body of water, by storm waves. Measures such as the use of quarried rock backfill, design for full hydrostatic pressure, or provision of a sloped impermeable surface layer should be considered.
  Measurements have shown that earth pressures can vary seasonally, but the effects have normally been neglected in design, except for winter frost pressures. These latter can be very large if the backfill is frost susceptible and for this reason free-draining granular backfill is recommended.
- Figure C6.20 shows examples of minimum backfill requirements.
  The distance, x, should be equal to or greater than the estimated vertical frost penetration. This distance may be reduced if the wall abuts a vertical face of bedrock that is not susceptible to frost. The frost penetration may be reduced by the use of suitable insulation, in which case a thermal analysis should be performed by a Geotechnical Engineer.
  If rock fill is used as a backfill material, consideration should be given to the possible deterioration of the rockfill with time, which could result in the reduction or even the total loss of free-draining properties and, hence, increased frost susceptibility.

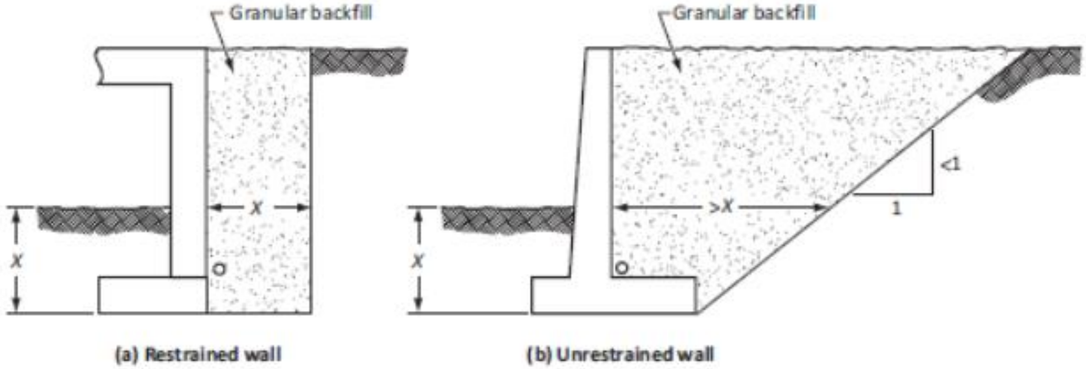
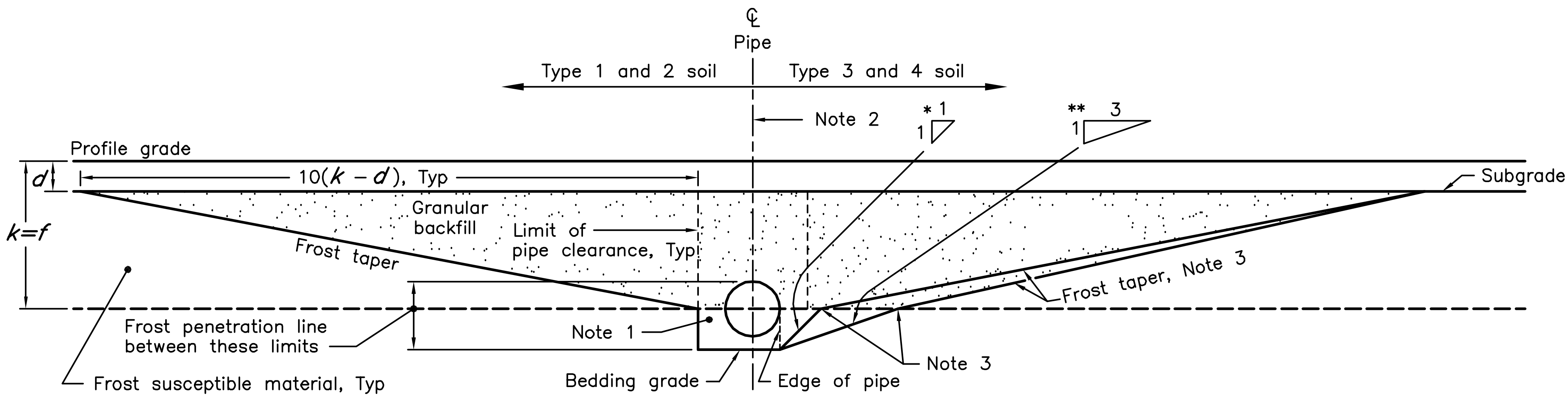


Figure C6.20
Backfill for frost protection
(See Clause C6.12.1.)

C6.12.2 Lateral ground pressures

C6.12.2.1 General

Earth pressure acting on a structure depends on the relative movement of the structure, the backfill, the type of soil adjacent to the backfill, and the soil below the footing or supporting piles. Appropriate geotechnical parameters should be chosen for the calculation of lateral pressures based on recognized geotechnical theories as specified in Clause 6.12.2.2 for the backfill behind the wall. Geotechnical parameters frequently used in allowable stress design methods are applicable in limit states design pressure calculation. Where the possibility exists, hydrostatic pressure needs to be considered, e.g., in situations where walls are partially submerged or where non-free-draining backfill is used.



FROST TREATMENT
RIGID AND FLEXIBLE PIPE

NOTES:

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

2 Condition of frost treatment symmetrical about centreline of pipe.

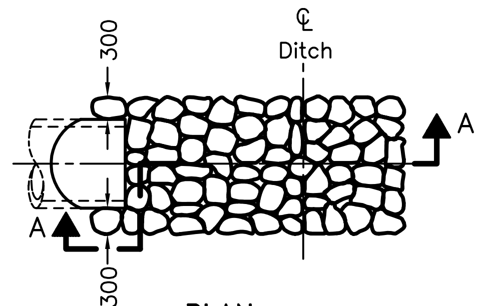
3 Frost tapers shall start at the intersection of the 1H:1V or 3H:1V slope and the frost penetration line.

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

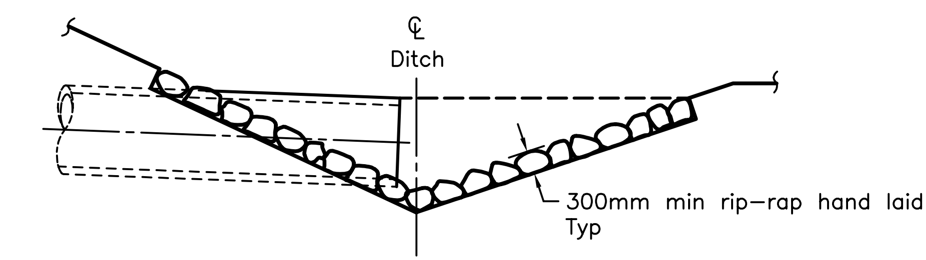
LEGEND:

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

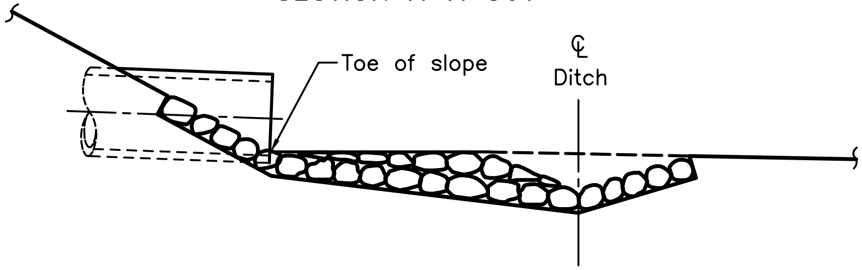
ONTARIO PROVINCIAL STANDARD DRAWING	Nov 2015	Rev 4
FROST TREATMENT – PIPE CULVERTS FROST PENETRATION LINE BETWEEN TOP OF PIPE AND BEDDING GRADE	OPSD 803.031	



PLAN
CUT OR FILL

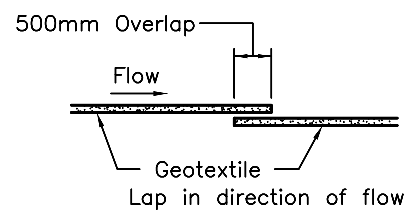


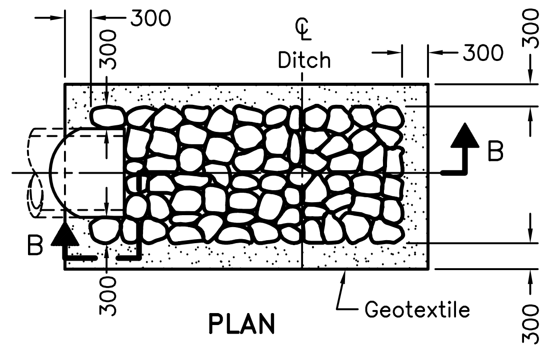
SECTION A–A CUT



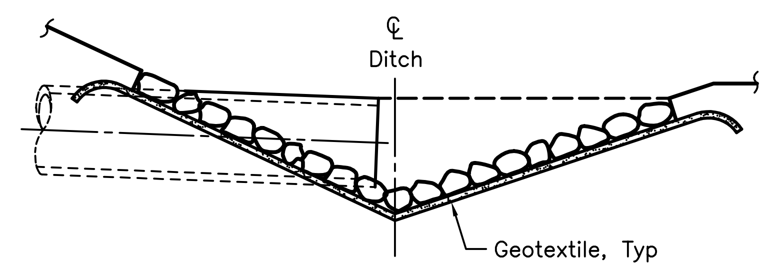
SECTION A–A FILL

TYPE A – WITHOUT GEOTEXTILE

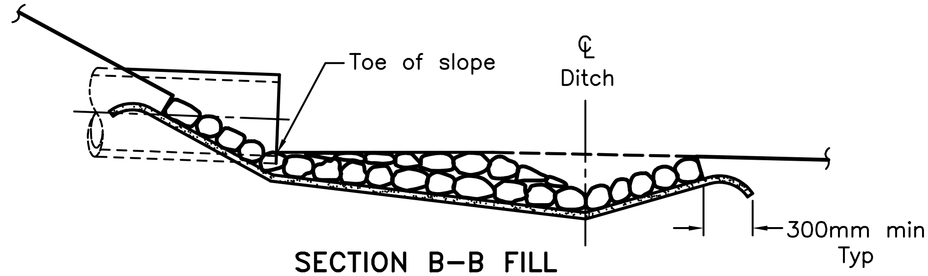




PLAN
CUT OR FILL



SECTION B–B CUT



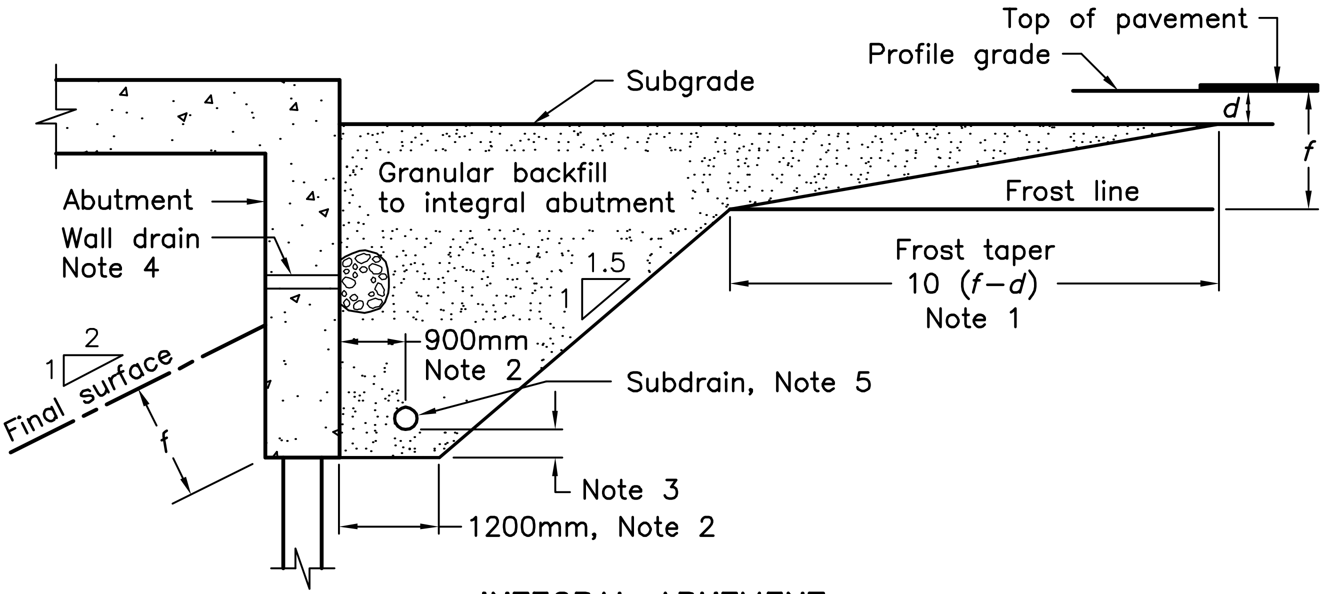
SECTION B–B FILL

TYPE B – WITH GEOTEXTILE

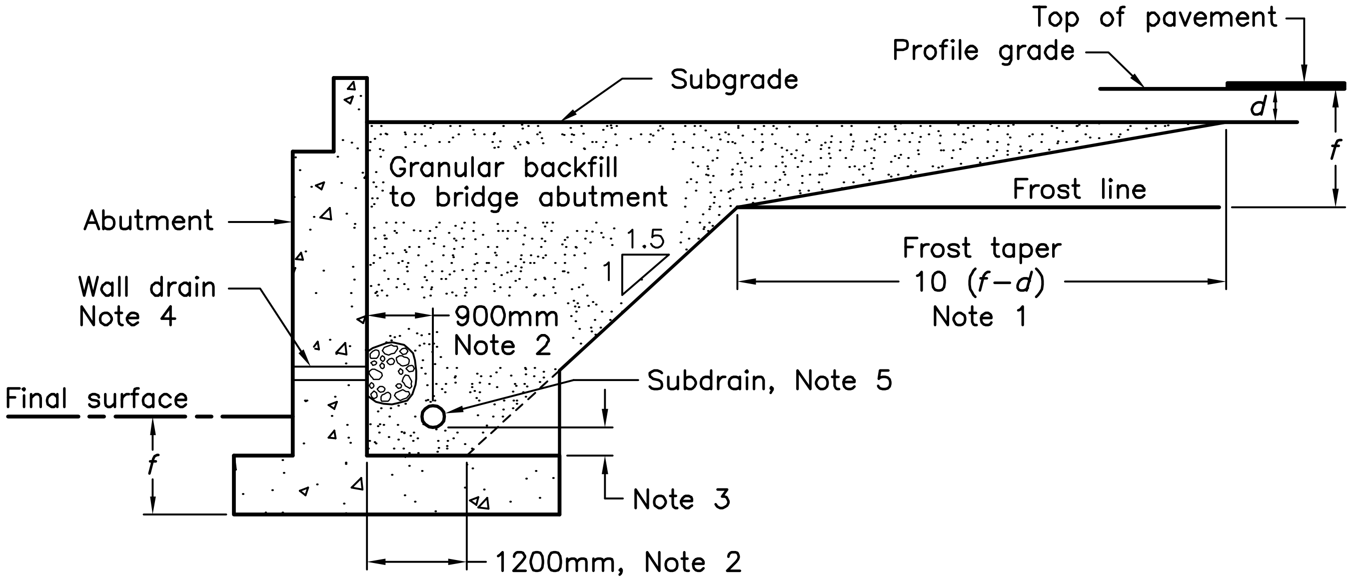
NOTES:

A All dimensions are in millimetres unless otherwise shown.

ONTARIO PROVINCIAL STANDARD DRAWING	Nov 2001	Rev 0
RIP–RAP TREATMENT FOR SEWER AND CULVERT OUTLETS		
	OPSD – 810.010	



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 I – NSSPs

NSSP FOR OBSTRUCTIONS

Scope of Work

The Contractor shall be alerted to the potential presence of cobbles and boulders in the fill and native till encountered in few boreholes advanced at the site. Therefore, appropriate equipment and procedures will be required for open cut excavation and installation of roadway protection systems and temporary dewatering/unwatering systems.

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.

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

Special Provision No. 517F01 February 2024

Amendment to OPSS 517, November 2023

Return Period Flow and Preconstruction Survey Distance

517.04 DESIGN AND SUBMISSION REQUIREMENTS

517.04.01 Design Requirements

Clause 517.04.01.01 of OPSS 517 is amended by deleting the second last paragraph in its entirety and replacing it with the following:

The temporary flow passage system shall allow the work to be conducted as specified in the Contract Documents. Design flow shall include groundwater discharge and flow resulting from a minimum 2 year return period design storm, except for the work specified in Table 1. For the work specified in Table 1, design flow shall include groundwater discharge and flow resulting from a design storm of the minimum return period specified in Table 1. A longer return period shall be used when determined appropriate for the work.

The flow estimates as specified in Table 1 do not include flow volumes from groundwater discharge.

The Owner specifically excludes flow estimates from the warranty in the Reliance on Contract Documents subsection of OPSS 100, MTO General Conditions of Contract.

TABLE 1
Site Location and Reference Information

<table>
<tr><th colspan="8">TEMPORARY FLOW PASSAGE SYSTEMS</th></tr>
<tr><th colspan="8">Source of Return Period Flow Estimates:</th></tr>
<tr><th rowspan="2">Site Name / Station Reference</th><th rowspan="2">Minimum Return Period (Years)</th><th colspan="4">Return Period Flow Estimates (m^3/s) (Note 1)</th><th rowspan="2">Design Engineer Requirements (Note 2)</th><th rowspan="2">Fish Passage Required (Note 3)</th></tr>
<tr><th>2 Year</th><th>5 Year</th><th>10 Year</th><th>25 Year</th></tr>
<tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr>
<tr><th colspan="8">DEWATERING SYSTEMS</th></tr>
<tr><th>Site Name / Station Reference</th><th colspan="2">Preconstruction Survey Distance (m) (Note 4)</th><th colspan="4">Minimum Lowered Groundwater Depth Below Base of Excavation or Work Area (m) (Note 5)</th><th colspan="1">Design Engineer Requirements (Note 2)</th></tr>
<tr><td>SW1/Sta. 11+800 Notman</td><td colspan="2"></td><td colspan="4">0.3</td><td></td></tr>
</table>

Notes:

1. a) The Design Engineer is to satisfy themselves to the accuracy and applicability of the provided flows.
   b) The intensity-duration-frequency (IDF) information can be accessed through MTO's IDF Curve Lookup web-based application tool at https://idfcurves.mto.gov.on.ca/
   c) The design, operation and maintenance of the temporary flow passage system is the sole responsibility of the Contractor.
2. "Yes" means the design Engineer and design-checking Engineer shall have a minimum of 5 years of experience in designing systems of similar nature and scope to the required work. "No" means a minimum experience level is not required for the design Engineer and design-checking Engineer.
3. "Yes" means that the design Engineer must design the temporary flow passage system to meet the fish passage requirements. "No" means fish passage is not required.
4. "N/A" means a preconstruction survey is not required.
5. Groundwater shall be lowered within the excavation or work area to below this minimum depth.

[* Designer Fill-Ins for Table 1, See Notes to Designer]

NOTES TO DESIGNER:

Designer Fill-Ins for Table 1:

1. Fill-in the source of the return period flow estimates.

2 Fill-in the site name, work, and station reference as appropriate for the dewatering system and/or temporary flow passage system item locations. Add additional rows as necessary.

3. For temporary flow passage system item locations, fill-in the minimum return period flow for each site based on MTO Drainage Design Standard TW-1. The return period flow shall not be less than 2 years.

4. For temporary flow passage system item locations, fill-in the design flow rate estimates for the various return periods.

5. Fill-in “Yes” under Design Engineer Requirements when recommended by the Foundation Engineer. Fill-in “No” otherwise.

6. For temporary flow passage system item locations, fill-in “Yes” under Fish Passage Required, when maintaining fish passage is a condition of a permit/ authorization or as recommended by the MTO Fisheries Assessment Specialist, in consultation with the MTO Environmental Planner. Fill-in “No” otherwise.

7. Fill-in the required distance under Preconstruction Survey Distance, when recommended by the Foundation Engineer. Fill-in “N/A” if not recommended.

8. Fill-in the Minimum Lowered Groundwater Depth Below Base of Excavation or Work Area provided by the Foundation Engineer.

9. When applicable, add a point d) to Note 1 of the table notes to indicate when Return Period Flow Estimates do not include base flows, for example:

   d) The Return Period Flow Estimates do not include base flows.

   d) The Return Period Flow Estimates at [enter Site Name/Description] do not include base flows.

Example Table 1

TABLE 1
Site Location and Reference Information

<table>
<tr><td colspan="9">TEMPORARY FLOW PASSAGE SYSTEMS</td></tr>
<tr><td colspan="9">Source of Return Period Flow Estimates: Longwood Channel Drainage Report (MTO 2017)</td></tr>
<tr><td rowspan="2">Site Name / Station Reference</td><td rowspan="2">Minimum Return Period (Years)</td><td colspan="4">Return Period Flow Estimates (m^3/s) (Note 1)</td><td rowspan="2" colspan="2">Design Engineer Requirements (Note 2)</td><td rowspan="2">Fish Passage Required (Note 3)</td></tr>
<tr><td>2 Year</td><td>5 Year</td><td>10 Year</td><td>25 Year</td></tr>
<tr><td>Woods Creek Culvert Rehabilitation</td><td>2</td><td>0.7</td><td>3.5</td><td>7.5</td><td>10.9</td><td colspan="2">No</td><td>No</td></tr>
<tr><td>Site 32-145 Robbs Creek Culvert Replacement</td><td>10</td><td>1.6</td><td>7.6</td><td>17.4</td><td>25.2</td><td colspan="2">Yes</td><td>Yes</td></tr>
<tr><td colspan="9">DEWATERING SYSTEMS</td></tr>
<tr><td>Site Name / Station Reference</td><td colspan="2">Preconstruction Survey Distance (m) (Note 4)</td><td colspan="4">Minimum Lowered Groundwater Depth Below Base of Excavation or Work Area (m) (Note 5)</td><td colspan="2">Design Engineer Requirements (Note 2)</td></tr>
<tr><td>Site 32-145 Robbs Creek Culvert Replacement</td><td colspan="2">300</td><td colspan="4">1.0</td><td colspan="2">Yes</td></tr>
</table>

Notes:

1. a) The Design Engineer is to satisfy themselves to the accuracy and applicability of the provided flows.
   b) The intensity-duration-frequency (IDF) information can be accessed through MTO's IDF Curve Lookup web-based application tool at https://idfcurves.mto.gov.on.ca/
   c) The design, operation and maintenance of the temporary flow passage system is the sole responsibility of the Contractor.
   d) The Return Period Flow Estimates at Site 32-145, Robbs Creek Culvert Replacement, do not include base flows.
2. "Yes" means the design Engineer and design-checking Engineer shall have a minimum of 5 years of experience in designing systems of similar nature and scope to the required work. "No" means a minimum experience level is not required for the design Engineer and design-checking Engineer.
3. "Yes" means that the design Engineer must design the temporary flow passage system to meet the fish passage requirements. "No" means fish passage is not required.
4. "N/A" means a preconstruction survey is not required.
5. Groundwater shall be lowered within the excavation or work area to below this minimum depth.

WARRANT: Always with these tender items.

DEWATERING STRUCTURE EXCAVATIONS - Item No.

Special Provision

Amendment to OPSS 902, November 2010

902.02 REFERENCES

Section 902.02 of OPSS 902 is amended by the addition of the following:

Ontario Provincial Standard Specifications, Construction

OPSS 805 Temporary Erosion and Sediment Control Measures

902.03 DEFINITIONS

Section 903.03 of OPSS 902 is amended by the addition of the following:

Automatic Transfer Switch means an electrical device that transfers power supply to a backup power source when there is an outage of the primary power source.

Cofferdam means as defined in OPSS 539.

Cut-Off Wall means a below grade wall that restricts groundwater flow and/or supports excavations, typically using soil-bentonite or cement-bentonite.

Design Storm Return Period means the average number of years based upon probability, between the occurrences of a storm event of a certain severity or greater.

Dewatering System means the components required to control water to permit construction work to proceed under specified conditions, and may include a groundwater control system, impermeable barriers, pumps, and/or equipment to carry out unwatering.

Groundwater Control System means sump pumps, oversized excavations with perimeter ditches, deep wells or well points or other systems used to lower the groundwater table.

Plug means an impervious, natural, or constructed drainage work that blocks water.

Sediment means soil particles detached from an earth surface by erosion.

Sediment Control Measure means a measure to remove sediment from water prior to discharge to the natural environment and sewer systems.

Temporary Flow Control means temporary flow control devices, channels, pipes, and other materials used to convey or divert water past an area under construction.

Unwatering means the removal of ponded or flowing surface water.

Vegetated Discharge Area means a sloped, open area of land with existing vegetation suitable to prevent erosion.

Waterbody means as any permanent or intermittent, natural or constructed body of water including lakes, ponds, wetlands and watercourses, but does not include sewage works as defined in the Ontario Water Resources Act.

Watercourse means a stream, creek, river, or channel including ditches, in which the flow of water is permanent, intermittent, or temporary.

902.04 DESIGN AND SUBMISSION REQUIREMENTS

Subsections 902.04.01 and 902.04.02 of OPSS 902 are deleted in their entirety and replaced with the following:

902.04.01 Design Requirements

902.04.01.01 Dewatering

A dewatering system shall be designed to control water and the flow of water into the excavation, prevent disturbance of the foundation, permit the placing of concrete in the dry, and complete the excavating and backfilling for structures work. The design of the system shall be sufficient to permit the work to be carried out as specified in the Contract Documents.

The design shall meet the requirements of the Contract Documents, and where a waterbody is present, shall include channel and inlet and outlet protection measures as required to protect the environment in the event of system failure or the design flow rate being exceeded.

The design shall not include the use of embankments and/or structures in public use, either existing or to be constructed as part of the Work, to control or stop water flow, unless approved by the Contract Administrator.

The design shall not result in displacement or damage to property, buildings, structures, utilities and other facilities adjacent to the Working Area, including from drawdown related settlement or other groundwater related effects.

The system shall be designed to prevent soil loss or erosion where water is removed, pumped, or discharged. The system shall be designed to prevent basal heave or instability.

Where the system involves the taking of water from a waterbody, the design shall maintain the flow of water and the natural functions of the waterbody upstream and downstream of the work area, and shall not interfere with other uses of the water.

When the system includes temporary flow control, the temporary flow control shall be designed, as a minimum, for a [* Designer Fill-In, See Notes to Designer] year design storm return period, and groundwater discharge. A longer return period shall be used when determined appropriate for the work.

Temporary flow control shall include provision for fish passage during low flows.

902.04.02 Submission Requirements

902.04.02.01 Working Drawings

Three (3) sets of Working Drawings for the dewatering system shall be submitted to the Contract Administrator at least 7 Days prior to commencement of the dewatering system installation, for information purposes only. Prior to submission of Working Drawings, the seals and signatures of a design Engineer and a design-checking Engineer shall be affixed on the Working Drawings verifying that the drawings are consistent with the Contract Documents.

One person shall not perform both the design Engineer and design-checking Engineer roles for a system.

Where multi-discipline engineering work is depicted on the same Working Drawing and the design or design-checking Engineer or both are unable to seal and sign the Working Drawing for all aspects of the work, the drawing shall be sealed and signed by as many additional design and design-checking Engineers as necessary.

The following information and details shall be shown on the Working Drawings, where applicable:

a) Plans, Elevations, and Details

   i. Type of system(s).
   ii. Design calculations demonstrating adequacy of the system and equipment.
   iii. Design flow rate(s).
   iv. Plan location, description, and dimensions of system components, including dams, cofferdams, cut-off walls, temporary channels, pipes, culverts, sewers, groundwater control systems employing wells and/or well points, sedimentation basins, tanks, pumps, power supply, and standby equipment.
   v. Method of management of pumped water and plan location of all dewatering discharge points.
   vi. Profile drawings shall extend through and immediately beyond the limits of the system.
   vii. Water elevations upstream and downstream of the system at design flow rate.
   viii. Dam height or crest elevation, cofferdam depth and tip elevation, cutoff wall depth or base elevation, pipe invert elevations, depths of wells and wellpoints, pump intake elevation, and sedimentation basin depth or base elevation.
   ix. Plan location, elevation, and dimensions of environmental protection measures.
   x. Pipe type, size, and length, pump capacity, and tank capacity.
   xi. Material and construction standards to be used for the work.
   xii. Method for establishing and monitoring construction site groundwater levels.
   xiii. Criteria and method of removal of the system.

b) Procedures for the system construction, operation, and maintenance, including daily start-up sequence where applicable, and operation shut down.

c) Procedures for the removal of the system, including the removal sequence, and well decommissioning.

d) Stand-by power or pumping system requirements and the use of automatic transfer switching, when required to protect the environment and the Work.

e) A copy of the Permit to Take Water issued by the Ministry of the Environment and Climate Change or confirmation of registration of water taking for construction dewatering, if a permit or registration is required by provincial regulation.

f) When applicable, a copy of the water taking report and discharge plan required by provincial regulation.

g) A copy of any necessary permits for the discharge of water to a sanitary sewer, or stormwater sewer system, stormwater pond, or other facility.

902.04.02.02 Preconstruction Survey

When a groundwater control system by wells or a well point system will be used, a condition survey of property and structures that may be affected by the work shall be carried out. The condition survey shall include the location and condition of adjacent properties, buildings, underground structures, water wells, Utilities, and structures, within a distance of [** Designer Fill-In, See Notes to Designer] metres from the groundwater control system. In addition, all water wells used as a supply of drinking water and located within this distance shall be tested for compliance with Ontario Drinking Water Quality Standards.

Water wells within the preconstruction survey distance can be located using the website https://www.ontario.ca/environment-and-energy/map-well-records or its successor site.

Copies of the condition survey and water quality test results shall be submitted to the Contract Administrator prior to the operation of the groundwater control system.

902.04.02.03 Milestone Inspections

The Quality Verification Engineer shall witness the following Interim Inspections of the work:

a) Dewatering of excavation for structure.

b) Completion of excavation for foundation.

c) Excavation for backfill and frost tapers.

d) Backfilling.

A copy of the written permission to proceed shall be submitted to the Contract Administrator prior to commencement of the successive operation.

902.07 CONSTRUCTION

Subsection 902.07.04 of OPSS 902 is deleted in its entirety and replaced with the following:

902.07.04 Dewatering Structure Excavation

902.07.04.01 General

The dewatering systems shall be constructed and operated according to the Working Drawings.

Activation of temporary flow control, if applicable, shall be as specified in the Contract Documents.

The dewatering system shall be continuously operational to control buoyancy forces until such forces can be resisted by backfill and structure self-weight, to keep excavations stable, to avoid erosion impacts from the release of accumulated water, and to keep the work area in the condition required to complete the associated work as specified in the Contract Documents.

When temporary flow control is to remain operational through a seasonal shutdown period, the Contractor shall be responsible for any maintenance or repair costs due to the temporary flow control during the seasonal shutdown period.

Temporary erosion and sediment control measures, including to control the discharge of water, shall be according to OPSS 805. Measures not specified in OPSS 805 shall be according to the Working Drawings. Temporary erosion and sediment control measures and cover material to protect exposed soils, as required by the Working Drawings, shall be installed as soon as is practical.

Stranded fish shall be managed as specified in the Contract Documents.

Unwatering shall be carried out as necessary.

Water suspected of being contaminated as indicated by visual or olfactory observations shall be reported to the Contract Administrator.

Dewatering and temporary flow control shall be discontinued in a manner that does not disturb any structure, pipeline, or flow channel. Operation of the dewatering system shall be shut down according to the procedures specified in the Working Drawings, where applicable.

902.07.04.02 Discharge of Water

Water from dewatering and unwatering operations shall be directed to a sediment control measure and/or a vegetated discharge area 30 m away from waterbodies or as far away as practicable from the top of the bank of any waterbody, prior to discharge to the natural environment.

Equipment and materials shall not be used or stored in vegetated discharge areas.

The discharge of water to the natural environment shall not be directed across pavements, sidewalks, curb and gutter or similar hard surfaces except through appurtenances as specified in the Contract Documents.

902.07.04.03 Monitoring

The Contract Administrator shall be notified of any complaints and any action taken or proposed to be taken in response to complaints.

Daily external visual monitoring of the surrounding area and property and structures on the preconstruction survey, if applicable, for impacts such as settlement and erosion shall be completed. Any observed impacts shall be immediately reported to the Contract Administrator. When public safety, the environment, or property is impacted or potentially impacted, the design Engineer shall, without delay, make a full assessment and direct changes to the system to eliminate impacts or potential impacts. Any changes shall be documented according to the System Amendments subsection.

When a groundwater control system is observed to negatively impact water supplies obtained from any adequate sources that were in use prior to groundwater control system operation, then water shall be supplied to the affected water users. The water shall be equivalent in quantity and quality to the normal water takings of the users. Supply shall continue until the negative impacts on the water supplies are removed, or until Contract Completion, whichever occurs first.

902.07.04.04 System Amendments

When displacement or damage to embankments and/or structures, or property adjacent to the Working Area, occurs due to the operation of the system, or soil loss or erosion occurs where water is removed, pumped, or discharged, the dewatering system or temporary flow control shall be amended to stop the displacement, damage, soil loss, or erosion.

Amendments shall be submitted to the Contract Administrator within two Business Days of the system being amended, on revised Working Drawings bearing the seal and signature of the design Engineer and design-checking Engineer.

902.07.04.05 Removal

Dewatering system and temporary flow control components shall be removed when no longer required. Removal of system components shall be according to the procedures specified on the Working Drawings, where applicable, and as specified in the Contract Documents.

Deactivation of temporary flow control shall be as specified in the Contract Documents.

Removal of temporary drainage work shall be according to OPSS 510.

Environmental protection measures and cut-off walls shall be removed, unless approved otherwise by the Contract Administrator.

Sedimentation basins and other excavations shall be backfilled with the original soil excavated, unless approved otherwise by the Contract Administrator. All disturbed areas shall be restored to an equivalent or better condition than existed prior to the commencement of construction.

NOTES TO DESIGNER:

<u>Designer Fill-Ins</u>

* Fill in the design storm return period according to MTO Drainage Design Standard TW-1.

** Fill in the preconstruction survey distance as recommended by the foundation engineer.

WARRANT: Include with this item <u>**only**</u> on the recommendation of a foundation engineer.

CUSTODIAN: Tony Sangiuliano, MERO - Foundation Group.