



## Foundation Investigation and Design Report

*New Material Storage Facility at Dunchurch Patrol Yard,  
Highway 124, Dunchurch, ON*

Agreement No. 5021-E-0020

Assignment No. 10

Latitude: 45.658464; Longitude: -79.810954

Geocres No.: 31E12-001

**Type of Document:**

Final Report

**EXP Project Number:**

ADM-22006096-A9

**Prepared For:**

Ontario Ministry of Transportation  
Geotechnical Section, Northeastern Region  
447 McKeown Avenue, Suite 301  
North Bay, ON P1B 9S9  
Attn: Jean Pierre Perron, P.Eng.

**Prepared By:**

EXP Services Inc.  
1595 Clark Boulevard  
Brampton, ON, L6T 4V1  
Canada

**Date Submitted:**

2024-04-19

## Foundation Investigation and Design Report

**Project Name:**

***New Material Storage Facility at Dunchurch Patrol Yard, Highway 124, Dunchurch, ON***

Agreement No. 5021-E-0020

Assignment No. 10

Latitude: 45.658464; Longitude: -79.810954

Geocres No.: 31E12-001

**Type of Document:**

Final Report

**EXP Project Number:**

ADM-22006096-A9

### Issue and Revised Record

Rev.	Date	Format	Prepared by	Reviewed by	Approved by	Description
<b>A</b>	March 4, 2024	pdf	C. Alexander D. Mroz S. Micic	T.C. Kim	S. Gonsalves	Draft Report
<b>B</b>	April 19, 2024	pdf	C. Alexander D. Mroz S. Micic	T.C. Kim	S. Gonsalves	Final Report

## Table of Contents

<b>1</b>	<b>FOUNDATION INVESTIGATION REPORT.....</b>	<b>1</b>
1.1	Introduction.....	1
1.2	Site Description and Geological Setting .....	1
1.2.1	<b>Site Description</b> .....	1
1.2.2	<b>Geological Setting</b> .....	1
1.3	Available Documents of Previous Investigations .....	2
1.4	Investigation Procedures.....	2
1.4.1	<b>Fieldwork</b> .....	2
1.4.2	<b>Laboratory Testing</b> .....	3
1.5	Subsurface Conditions.....	3
1.5.1	<b>Asphalt</b> .....	4
1.5.2	<b>Cohesionless Fill: Sand</b> .....	4
1.5.3	<b>Silty Clay / Clayey Silt</b> .....	5
1.5.4	<b>Silty Sand / Sand</b> .....	6
1.5.5	<b>Bedrock</b> .....	7
1.6	Groundwater Conditions .....	7
1.7	Chemical Analyses .....	8
<b>2</b>	<b>ENGINEERING DISCUSSION &amp; RECOMMENDATIONS .....</b>	<b>9</b>
2.1	General .....	9
2.2	Geotechnical Design Considerations for Foundations .....	10
2.2.1	<b>Structure Foundation Alternatives</b> .....	10
2.2.2	<b>Evaluation of Foundation Alternatives</b> .....	10
2.2.3	<b>Shallow Foundation</b> .....	11
2.3	Seismic Potential Consideration.....	17
2.3.1	<b>Seismic Hazard Site Classification and Values</b> .....	17
2.3.2	<b>Liquefaction Considerations</b> .....	18
2.4	Perimeter Wall and Floor Construction.....	18
2.5	Stability and Settlement Analyses .....	19
2.5.1	<b>Stability</b> .....	19
2.5.2	<b>Settlement</b> .....	21
2.6	Site Preparation and Engineered Fill Construction .....	21
2.7	Excavation .....	22

2.8	Groundwater Control .....	22
2.9	Corrosion Protection .....	23
<b>3</b>	<b>CLOSURE .....</b>	<b>24</b>
	<b>REFERENCES.....</b>	<b>25</b>
	<b>LIMITATIONS AND USE OF REPORT .....</b>	<b>27</b>

## **Appendices**

APPENDIX A: SITE PHOTOGRAPHS

APPENDIX B: DRAWINGS

APPENDIX C: BOREHOLE LOGS

APPENDIX D: LABORATORY DATA

APPENDIX E: BEDROCK CORE PHOTOGRAPHS

APPENDIX F: SLOPE STABILITY ANALYSES

APPENDIX G: SETTLEMENT ANALYSES

APPENDIX H: SEISMIC HAZARD CALCULATION



# 1 FOUNDATION INVESTIGATION REPORT

## 1.1 Introduction

EXP Services Inc. (EXP) has been requested by the Ministry of Transportation, Ontario (MTO) to prepare a foundation investigation and design report for the new winter sand/salt storage structure at the Dunchurch Patrol Yard. The patrol yard is located at 6980 Highway 124 in Dunchurch, Ontario (Latitude: 45.658464; Longitude: -79.810954). The Terms of Reference (TOR) was provided by MTO. This report was undertaken under Agreement No. 5021-E-0020, Assignment No. 10.

The purpose of this investigation is to evaluate the subsurface conditions at the proposed location of the structure within the Dunchurch Patrol Yard. The proposed structure will be modelled after a recently constructed building at the Marten River Patrol Yard in Powassan, ON. The new building at the Dunchurch Patrol Yard will be approximately 24 m x 50 m. The site-specific geotechnical investigation consisted of a field investigation including visual inspection, 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 Dunchurch Patrol Yard is located at 6980 Highway 124 in Dunchurch, Ontario (Key Map on Drawing 1, Appendix B). In general, the site is bound by undeveloped land consisting of forest in all directions with a residential dwelling nearby to the west.

A paved roadway leads from the site entrance on Highway 124 to five (5) existing buildings/structures, a maintenance garage, two (2) sand domes, a shed, and a storage structure. The maintenance garage, shed, and storage structure are located approximately 46 m northwest, 92 m northwest, and 135 m north of the site entrance, respectively. The two sand domes are located adjacent to one another, about 75 m and 79 m northeast of the site entrance, respectively. The new storage building will be placed at the location of the existing sand dome that is located approximately 5.0 m east of the existing shed. Per the AutoCAD drawing of Dunchurch Patrol Yard provided by MTO, the approximate finish floor (FF) elevations of the existing maintenance garage, sand domes, shed, and storage structure are 282.310 m, 281.800 m, 281.890 m, 281.805 m, and 284.755 respectively.

The topography of the site is considered generally flat lying with borehole elevations ranging from Elev. 281.7 m to 281.9 m. The ground surface of the Dunchurch Patrol Yard is paved around the existing structures with sand and gravel in other areas along the perimeter of the patrol yard. Photographs of the site are included in Appendix A.

### 1.2.2 Geological Setting

According to the Ministry of Northern Development and Mines Map 2556, Quaternary Geology of Ontario, Southern Sheet, the site generally consists of Precambrian bedrock comprised of undifferentiated igneous and metamorphic rock, exposed at surface or covered by a discontinuous, and thin layer of drift. According to the Ministry of Northern Development and Mines Map 2544, Bedrock Geology of Ontario, Southern Sheet, the bedrock at the site consists of mafic rocks: amphibolite, gabbro, diorite, and mafic gneisses.

### 1.3 Available Documents of Previous Investigations

The nearest available previous investigation reports in the MTO GEOCREs library for the Dunchurch Patrol Yard are located about 2.7 km west, and 2.8 m east from the location of the site, respectively:

*Geocres No. 31E-322A: "Final Foundation Investigation and Design Report, Highway 124 Rehabilitation, Culvert Replacement, Station 11+225 – Twp. Of Croft, GWP 5467-09-00" prepared by LVM/Merlex Ltd., May 30, 2013.*

*Geocres No. 31E-044: "Foundation Investigation Report, Shadow River, W.P. 87-57; Hwy. 124" prepared by William A. Trow & Associates Ltd., December 1, 1964.*

### 1.4 Investigation Procedures

#### 1.4.1 Fieldwork

The field investigation was performed between January 18 and 19, 2024. The field program consisted of drilling four (4) sampled boreholes (BH23-D-1 to BH23-D-4). The boreholes were strategically located at the proposed location of the new building (i.e., at each corner of the building) to provide subsurface information for the design of the proposed material storage facility. The borehole locations are shown on Drawing 1 in Appendix B.

The borehole locations (referenced to the MTM NAD83 coordinate system) 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  $\pm 0.1$  m in the horizontal and vertical directions. A reference was made with an existing benchmark (BM), established on the finished floor at the entrance of the existing sand dome located at the center of the property, next to the existing shed, approximately 75 m northeast of the entrance to the patrol yard. The elevation of the BM was Elev. 281.800 m based on the AutoCAD drawing. The BM location is shown on Drawing 1 in Appendix B.

The boreholes were advanced using a truck mounted CME 55 drill rig equipped with hollow stem augers and diamond bit NW casing and NQ coring. All borehole drilling and sampling operations were performed by a specialist drilling contractor, Landcore Drilling Services. The locations, elevations and depths of the boreholes are shown below in Table 1.1.

Table 1.1. Locations, elevations, and depths of boreholes completed by EXP Services Inc.

BH ID	Location	MTM NAD83 Zone 10		Latitude	Longitude	Ground Surface Elevation <sup>1</sup> (m)	Borehole Depth <sup>2</sup> (m)
		Northing	Easting				
BH23-D-1	Southeast corner of proposed bldg.	5057699.7	280536.7	45.658765	-79.811346	281.7	4.5
BH23-D-2	Southwest corner of proposed bldg.	5057688.6	280515.9	45.658665	-79.811612	281.7	5.7
BH23-D-3	Northwest corner of proposed bldg.	5057736.7	280502.0	45.659097	-79.811793	281.8	6.4

BH ID	Location	MTM NAD83 Zone 10		Latitude	Longitude	Ground Surface Elevation <sup>1</sup> (m)	Borehole Depth <sup>2</sup> (m)
		Northing	Easting				
BH23-D-4	Northeast corner of proposed bldg.	5057743.4	280528.5	45.659158	-79.811453	281.9	6.4

**Notes:**

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

During the drilling of the boreholes, soil samples were obtained using a 51 mm outside diameter (O.D.) split-spoon sampler in accordance with Standard Penetration Test (SPT) procedures (ASTM D 1586), at intervals shown on the attached borehole logs (Appendix C). The original field (uncorrected) SPT “N” values were recorded on the borehole logs as recommended in the Canadian Foundation Engineering Manual (CFEM pg. 40) and used to provide an assessment of in-situ consistency of cohesive soils or compactness of non-cohesive soils. When a hard stratum was reached sampling of hard material was performed by diamond core drilling, using a 1.5 m long NQ double tube wireline core barrel.

Groundwater level measurements were carried out in the boreholes before coring procedures and at the completion of the boreholes, in accordance with MTO guidelines. The recorded groundwater levels measured prior to rock coring of the boreholes are presented in the borehole log sheets in Appendix C. The boreholes were decommissioned right after completion of drilling by bentonite/cement mixtures in accordance with the Ministry of the Environment Regulation 903, as amended by Regulation 128/03 (the well regulation under the Ontario Water Resources Act).

The fieldwork was supervised by 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 Brampton laboratory for additional visual, textual and olfactory examination, and sampling for laboratory testing.

#### 1.4.2 Laboratory Testing

All samples recovered from boreholes undertaken by EXP during this investigation were returned to EXP’s Brampton laboratory and subjected to visual examination and classification. The laboratory testing program on soil samples included the determination of natural moisture content, particle size distribution, and Atterberg limits tests for approximately 25% of the collected soil samples. One (1) soil sample was selected for chemical analysis and tested at Bureau Veritas Laboratories, a CALA-certified and accredited laboratory. All laboratory tests were carried out in accordance with MTO and/or ASTM standards as appropriate.

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

### 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. Laboratory test results of grain size analyses and Atterberg limits tests

are provided in Appendix D. The “Explanation of Terms Used in Report” preceding the borehole logs in Appendix C forms an integral part of and should be read in conjunction with this report.

A borehole location plan and 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. These boundaries typically represent transitions from one soil type to another and should not be regarded as exact planes of geological change. Furthermore, subsurface conditions may vary between and beyond the borehole locations.

In general, the stratigraphic sequence at the proposed structure site consists of pavement structure at ground surface followed by compact cohesionless fill, underlain by firm to very stiff silty clay to clayey silt, underlain by very loose to very dense silty sand to sand followed by bedrock.

A detailed description of the subsurface conditions encountered is discussed further in subsequent sections. It should be noted that the following sections are based on the geotechnical investigation conducted by EXP.

### 1.5.1 Asphalt

Asphalt, approximately 50 mm thick, was encountered at the surface of boreholes BH23-D-2, and BH23-D-3.

### 1.5.2 Cohesionless Fill: Sand

Cohesionless fill consisting of sand was encountered below the asphalt in boreholes BH23-D-2, and BH23-D-3, and at the surface of boreholes BH23-D-1, and BH23-D-4. The depths and elevations of the fill layer encountered at these borehole locations are listed in Table 1.2.

Table 1.2. Summary of cohesionless fill: sand

Borehole No.	Elevation <sup>1</sup> (m)		Layer Surface Depth <sup>2</sup> (m)	Layer Thickness (m)
	Top	Bottom		
BH23-D-1	281.7	280.9	0.0	0.8
BH23-D-2	281.6	280.6	0.1	1.0
BH23-D-3	281.7	280.3	0.1	1.4
BH23-D-4	281.9	280.3	0.0	1.6

Notes:

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

The composition of this fill material generally consisted of sand with trace gravel. The fill was generally brown in colour, and moist. The SPT “N” values obtained within this fill material ranged from 6 to 35 blows per 0.3 m penetration, suggesting that this fill layer was loose to dense in compactness, but generally compact.

Laboratory testing performed on selected samples consisted of seven (7) moisture content tests. The test results are as follows:

Moisture Content:

- 10% to 18%

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

### 1.5.3 Silty Clay / Clayey Silt

Native cohesive silty clay / clayey silt was encountered below the cohesionless fill layer in all boreholes. The depths and elevations of this layer encountered at these borehole locations are listed in Table 1.3.

Table 1.3. Summary of silty clay / clayey silt

Borehole No.	Elevation <sup>1</sup> (m)		Layer Surface Depth <sup>2</sup> (m)	Layer Thickness (m)
	Top	Bottom		
BH23-D-1	280.9	280.2	0.8	0.7
BH23-D-2	280.6	280.2	1.1	0.4
BH23-D-3	280.3	279.5	1.5	0.8
BH23-D-4	280.3	279.6	1.6	0.7

Notes:

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

The composition of this layer generally consisted of silty clay to clayey silt with trace to some sand, and trace organics. Trace oxidization was encountered in boreholes BH23-D-3, and BH23-D-4. The material was generally grey in colour and moist to wet. The SPT “N” values obtained within this layer ranged from 6 to 25 blows per 300 mm penetration, suggesting that this material was firm to very stiff in consistency, but generally stiff to very stiff in consistency. The Atterberg limits test results suggest that this cohesive layer was of low to medium plasticity, but generally low plasticity.

Laboratory testing performed on selected samples consisted of four (4) moisture content tests, three (3) grain size distribution tests, and three (3) Atterberg limits tests. The test results are as follows:

Moisture Content:

- 22% to 33%

Grain Size Distribution:

- 0% gravel;
- 5% to 11% sand;
- 61% to 69% silt;
- 22% to 32% clay

Atterberg Limits:

- Liquid Limit: 30% to 40%
- Plastic Limit: 17% to 21%
- Plasticity Index: 11% to 19%

The results of the moisture content, grain size distribution, and Atterberg limits tests are provided on the record of borehole sheets in Appendix C. The results of the grain size distribution, and Atterberg limits tests are also provided on Figure 1, and Figure 3, respectively, in Appendix D.

#### 1.5.4 Silty Sand / Sand

Native silty sand / sand was encountered below the silty clay / clayey silt layer in all boreholes. The depths and elevations of this layer encountered at these borehole locations are listed in Table 1.4.

Table 1.4. Summary of silty sand / sand

Borehole No.	Elevation <sup>1</sup> (m)		Layer Surface Depth <sup>2</sup> (m)	Layer Thickness (m)
	Top	Bottom		
BH23-D-1	280.2	278.9	1.5	1.3
BH23-D-2	280.2	279.0	1.5	1.2
BH23-D-3	279.5	277.8	2.3	1.7
BH23-D-4	279.6	277.7	2.3	1.9

Notes:

3. The elevations referenced are geodetic.
4. Depths are relative to ground surface.

The composition of this layer generally consisted of silty sand to sand with some silt, and trace clay. The material was generally brown to grey in colour and wet. The SPT "N" values within this layer ranged from 2 to 100 blows per 300 mm penetration to 100 blows per 50 mm penetration, suggesting that this material was very loose to very dense in compactness, but generally loose to compact.

Laboratory testing performed on selected samples consisted of ten (10) moisture content tests, and three (3) grain size distribution tests. The test results are as follows:

Moisture Content:

- 19% to 26%

Grain Size Distribution:

- 0% gravel;
- 65% to 88% sand;
- 10% to 32% silt;
- 2% to 3% clay

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

### 1.5.5 Bedrock

Bedrock was encountered below the silty sand / sand layer in all boreholes. The bedrock in these boreholes was investigated by coring approximately 1.7 m to 3.0 m into the stratum. Based on the encountered bedrock, it appears the rock/bedrock slopes towards the north. The bedrock surface depths and elevations encountered at these borehole locations are listed in Table 1.5. Photographs of the rock cores are included in Appendix E.

Table 1.5. Summary of Bedrock

Borehole No.	Elevation <sup>1</sup> (m)		Layer Surface Depth <sup>2</sup> (m)	Uniaxial Compressive Strength – UCS (MPa)
	Top	Bottom		
BH23-D-1	278.9	277.2	2.8	-
BH23-D-2	279.0	276.0	2.7	66.4
BH23-D-3	277.8	275.4	4.0	-
BH23-D-4	277.7	275.5	4.2	-

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 core samples are described as mafic rock (grey) with quartz veining. The Rock Quality Designation (RQD) measured on the core samples ranged from approximately 80% to 100%, indicating a rock mass of good to excellent quality. The total core recovery (TCR) of bedrock cores ranged from 83% to 100%.

The uniaxial compressive strength (UCS) was measured to be approximately 66.4 MPa in Run 1, indicating strong (R4) rock, according to the CFEM. The results of laboratory uniaxial compression test are presented on the borehole records in Appendix C, and in Appendix D.

### 1.6 Groundwater Conditions

Groundwater was encountered in all open boreholes prior to rock coring between approximately Elevations 279.4 to 279.9 m. The groundwater levels measured in the open boreholes prior to rock coring are shown on the borehole logs in Appendix C and are presented below in Table 1.6.

Table 1.6. Summary of groundwater levels

Borehole No.	Date Measured	Ground Surface Elevation (m)	Groundwater Depth <sup>1</sup> /Elevation <sup>2</sup> (m)
Groundwater Measured Prior to Rock Coring			
BH23-D-1	January 19, 2024	281.7	1.8/279.9
BH23-D-2	January 18, 2024	281.7	1.8/279.9

Borehole No.	Date Measured	Ground Surface Elevation (m)	Groundwater Depth <sup>1</sup> /Elevation <sup>2</sup> (m)
BH23-D-3	January 18, 2024	281.8	2.4/279.4
BH23-D-4	January 18, 2024	281.9	2.1/279.8

Notes:

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

Seasonal variations in the water table should be expected, with higher levels occurring during wetter periods of the year and lower levels during drier periods.

## 1.7 Chemical Analyses

One (1) soil sample was selected for chemical analyses during field investigation. The soil sample collected by EXP was tested at Bureau Veritas, a CALA-certified and accredited laboratory in Mississauga, Ontario.

The sample SS3 from borehole BH23-D-3 was subjected to corrosivity chemical analyses. The analytical results are summarized in Table 1.7 below and are presented in Appendix D.

Table 1.7. Summary of chemical analysis results

Sample Identification	pH (unitless)	Soluble Chloride (ppm)	Soluble Sulphate (ppm)	Resistivity (ohm-cm)	Conductivity (mS/cm)	Redox Potential (mV)
BH23-D-3 SS3	6.73	430	<20	1200	821	220



## 2 ENGINEERING DISCUSSION & RECOMMENDATIONS

### 2.1 General

This section of the report provides geotechnical design recommendations for the proposed material storage structure at the Dunchurch Patrol Yard, located in Dunchurch, Ontario. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the current investigation at the site and presented in Part I-Foundation Investigation Report. The interpretation and recommendations provided are intended solely to permit designers to assess foundation alternatives and design the proposed structure. 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.

Based on information included in the TOR and correspondence with MTO it is understood that the new material storage building will be constructed at the MTO Dunchurch Patrol Yard at the location defined by MTO. As provided in an email from MTO on November 27, 2023, it is understood that the new material storage building will have a footprint of about 24 m x 50 m (80 ft x 160 ft) and will be modelled after a recently constructed structure at the Marten River Patrol Yard. The orientation of the building will be such that the entrance doors will be facing south. As per the provided GA drawings of the Marten River Yard Salt/Sand Storage Facility, the proposed new structure at Dunchurch Patrol Yard will be about 12.3 m in height to the bottom of the trusses (underside of roof truss) and it will be encompassed by a 3.9 m high cast-in-place concrete retaining walls (above finished floor) along with 8.4 m high steel cladding walls around the perimeter of the building. A brine storage area attached to the southeast corner of the structure with a footprint of 8.3 m x 8.6 m and 9.8 m height to the bottom of the trusses and a truck loading area attached to the southwest corner of the structure with a footprint of 8.3 m x 15.5 m and 9.8 m height to the bottom of the trusses and is also included in the new proposed material storage building. The brine storage area will be encompassed by 2.8 m concrete retaining walls (above finished floor) and the truck loading area will be encompassed by a 2.8 m high concrete wall supported on strip footings. The existing ground surface at the structure location ranges from approximately Elev. 281.7 m to Elev. 281.9 m and it is assumed that finished top of floor will be at that current ground level to tie-in to the adjacent exterior paved areas.

This report addresses the geotechnical design of the foundation for the proposed structure 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 Ontario Building Code (OBC) (2012), Guidelines for MTO Foundation Engineering Services, Version 03 (April 2022), the Guideline for Professional Engineers Providing Geotechnical Engineering Service (1992), the Canadian Foundation Engineering Manual (CFEM) (2006), the provisions in the TOR and good practice. It also provides discussion about the structure foundation type, stability and settlement analyses, lateral earth pressure parameters, frost protection, drainage, construction considerations and dewatering during construction, if necessary, as requested in the TOR.

The settlement and stability analyses were completed for a scenario in which the sand/salt would be loaded to a total height of 8.1 m at the center of the stockpile on the west side of the building (~4.2 m above the outer concrete wall) and 6.6 m at the center of the stockpile in the east side of the building (~2.7 m above the concrete walls) with a maximum of 1.5H:1V slopes towards the concrete walls. It is assumed that the sand/salt will be level with concrete walls with the stockpile area covering the entire footprint of the building. The angle of repose for sand/salt was assumed to be 33°.

## 2.2 Geotechnical Design Considerations for Foundations

The subsurface conditions at the site below the pavement structure consists of compact sand fill followed by firm to very stiff clayey silt to silty clay over very loose to compact sand/silty sand which is underlain by bedrock. Bedrock was encountered and confirmed at depths ranging from 2.7 m to 4.2 m below ground surface with elevation ranging from Elev. 279.0 m to 277.7 m (i.e., sloping bedrock down towards the north). The groundwater level was measured in four (4) open holes prior to rock coring which was at approximately Elev. 279.4 m to 279.9 m.

### 2.2.1 Structure Foundation Alternatives

Based on the results of this investigation, several foundation alternatives for the structure are evaluated in this report. Advantages, disadvantages, relative cost, and risk/consequences of shallow foundations such as strip/spread footings are presented in Table 2.1. Deep foundations are not considered to be a practical option due to shallow bedrock within the proposed building location (depth less than 5.0 m below ground surface). Shallow foundation options should provide sufficient geotechnical resistances for the proposed material storage building. Considering the findings during the geotechnical investigation, as well as the high cost of pile foundations and the structure's operating life, deep foundations are not considered practical for this patrol yard structure and therefore not further discussed within the report.

### 2.2.2 Evaluation of Foundation Alternatives

In Table 2.1, the shallow foundation using strip/spread footings on the native soil (Option 1) is ranked as the preferred foundation design option. If a higher bearing capacity is preferred, a shallow foundation using strip/spread footings on 0.5 m thick engineered fill (Option 2) can be used. However, if less excavation is preferred, a shallow foundation using strip/spread footings on native very stiff clayey silt (south end) and 0.3 m to 0.4 m of engineered fill (north end) with 25 mm polystyrene placed above the footing for protection against frost action (Option 3) could be an option, as discussed in the following sections. All shallow foundation options require full excavation of any organic soil or existing fill encountered at the site.

Given the subsurface conditions at the site, the impact of settlement at the foundations of the structure will be influenced by the operating/stockpiling practices. It is our understanding that the structure will accommodate stockpiles of sand/salt within the structure. As mentioned in Section 2.1, it is assumed that the maximum loading condition is the sand/salt stockpile to be sloped a maximum of 1.5H:1V towards the concrete wall where the sand/salt is likely stockpiled to at least the level of the concrete wall over the full footprint. The center of the stockpile would be a maximum height of 8.1 m (about 4.2 m above the 3.9 m high outer concrete wall). Mounding in the center at an angle of repose of 33° beyond the height of the concrete wall is also a possibility. Due to the presence of cohesive soils at the new proposed building, a portion of the post construction settlement within the stockpile area is expected to occur immediately followed by consolidation settlement for the remaining total settlement. Consolidation settlement potential can be eliminated by sub-excavating any native cohesive soils. These types of structures generally have service lives of about 20 years.

Based on the provided typical design for the sand/salt storage structure, it is understood that the strip/spread footings for the structure will be up to about 3.5 m. As mentioned, the footings could be founded on/within the native firm to very stiff clayey silt to silty clay/compact sand to silty sand, or on free draining engineered fill, such as Granular 'A' or Granular 'B' Type II (OPSS.PROV 1010).

Table 2.1. Evaluation of shallow foundation alternatives

Options	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
<b>OPTION 1 (Deep Excavation):</b> Strip/Spread Footings on Native Soil	1	<ul style="list-style-type: none"> <li>• Straightforward construction</li> </ul>	<ul style="list-style-type: none"> <li>• Deeper excavation required to meet requirements against frost action</li> <li>• May require dewatering for the construction of footing</li> </ul>	<ul style="list-style-type: none"> <li>• Low</li> </ul>	<ul style="list-style-type: none"> <li>• Risk of differential settlements due to loading patterns in the past and during operations</li> <li>• Risk of subgrade disturbance</li> </ul>
<b>OPTION 2 (Deep Excavation):</b> Strip/Spread Footings on Granular Engineered Fill	2	<ul style="list-style-type: none"> <li>• Straightforward construction</li> <li>• Compaction control</li> <li>• Increased bearing capacity</li> <li>• Removes cohesive soils, therefore no consolidation settlement</li> </ul>	<ul style="list-style-type: none"> <li>• Deeper excavation required to meet requirements against frost action</li> <li>• May require dewatering to allow the construction of footing in dry and prevention of subgrade disturbance</li> </ul>	<ul style="list-style-type: none"> <li>• Higher cost compared to shallow foundation on native soil (Option 1)</li> </ul>	<ul style="list-style-type: none"> <li>• Risk of groundwater and subgrade disturbance</li> </ul>
<b>OPTION 3 (Shallow Excavation):</b> Strip/Spread Footings on Native Soil / Engineered Fill	3	<ul style="list-style-type: none"> <li>• Straightforward construction</li> <li>• Reduced excavation depths by placing polystyrene foam above footing to protect against frost action</li> </ul>	<ul style="list-style-type: none"> <li>• Reduced bearing capacity compared to options 1 and 2</li> </ul>	<ul style="list-style-type: none"> <li>• Higher cost than options 1 and 2 due to the cost of polystyrene foam above footing</li> </ul>	<ul style="list-style-type: none"> <li>• Risk of differential settlements due to loading patterns in the past and during operations</li> <li>• Risk of subgrade disturbance</li> </ul>

### 2.2.3 Shallow Foundation

#### 2.2.3.1 Footing Elevation

Based on the results of the geotechnical investigation and a requirement for adequate protection against frost penetration in the project area (i.e. a minimum ~1.8 m below the lowest surrounding area or equivalent thermal insulation to be provided, see Section 2.2.3.4), the following founding elevations of strip/spread footings presented in Table 2.2 are recommended:

Table 2.2. Recommendations for footing elevation

Soil at Founding Level	Foundation Elevation (m)	Footing Depth /Excavation Depth Below Existing Grade
<b>OPTION 1 (Deep Excavation):</b> Loose to compact native sand to silty sand (south)/firm to stiff clayey silt to silty clay (north)	~279.9 to 280.1	~1.8 m <sup>4</sup> / 1.8 m
<b>OPTION 2 (Deep Excavation):</b> 0.5 m thick engineered fill <sup>1</sup> over compact native sand to silty sand	~279.9 to 280.1	~1.8 m/2.3 m excavation up to 279.4 m to 279.6 m from south to north <sup>4</sup>
<b>OPTION 3 (Shallow Excavation)<sup>3,4</sup>:</b> Very stiff clayey silt (south) / 0.3 m to 0.4 m engineered fill <sup>1,2</sup> over firm to stiff clayey silt to silty clay (north)	~280.5 to 280.7	~1.2 m / 1.2 to 1.4 m excavation up to Elev. 280.5 m to 280.3 m from south to north <sup>4</sup>

## Notes:

1. Such as Granular 'A' or Granular 'B' Type II.
2. Varies in thickness depending on excavation of existing fill.
3. It is accepted that 25 mm of polystyrene foam placed above the footing provides protection against frost action which is equivalent to 600 mm of soil cover (see Section 2.2.3.4).
4. Based on frost line of 1.8 m below ground surface or 1.2 m below ground if 25 mm of polystyrene is used (see Section 2.2.3.4). The thickness of polystyrene foam is recommended based on stability analyses results (refer to Section 2.5.1).

### 2.2.3.2 Geotechnical Resistances

In the context of the CHBDC and OBC, a satisfactory foundation design would require, in terms of Limit States Design, the factored geotechnical resistance of its foundation to withstand and not exceed the imposed Ultimate Limit State (ULS) Loads Design Approach, and its ability to deform acceptably under the Service Limit State (SLS) Loads Design Approach. These associated loads are typically known as unfactored and factored loads, respectively.

Based on the subsurface stratigraphy encountered at this site and the proposed building, the following Table 2.3 summarizes the recommended resistances at the design elevation given in Table 2.2 for the strip/spread footings. The geotechnical resistances provided are for vertical loading conditions only; load eccentricity and load inclination effects should be addressed in accordance with the CFEM, OBC and the CHBDC and its commentary. The ULS and SLS consequence factor of 1.0 and a typical degree of understanding factor of 0.5 at ULS and factor of 0.8 at SLS were applied in accordance with Tables 6.1 and 6.2 in the CHBDC S6-19, respectively.

Based on the typical GA drawings provided by MTO, the width of the footings is typically ~3.7 m for most of the building with ~2.1 m footings around brine storage area and ~0.9 m strip footings for the wall at the entrance and truck loading area. Several footing widths are provided in Table 2.3. Settlement of the footings under the loading from the stockpile inside the structure which will occur after its construction is considered and discussed in Section 2.5.2. In determining the settlement characteristics of the proposed building (tolerable total and differential settlement), the unfactored loads are required to be provided by the Structural or Design Engineer. The ULS and SLS consequence factor of 1.0 and degree of site understanding of 0.8 were applied in accordance with Tables 6.1 and 6.2 in the CHBDC S6-19, respectively.

Table 2.3. Factored geotechnical resistances for different footing widths

Soil at Founding Level	Width of Footing (m)	Factored Ultimate Geotechnical Resistance (kPa)	Factored Serviceability Geotechnical Resistance (kPa) (for 25 mm settlement)
<b>OPTION 1 (Deep Excavation):</b> Loose to compact native sand to silty sand (south)/firm to stiff clayey silt to silty clay (north)	1.0	450	240
	2.0	475	250
	3.5	500	265
<b>OPTION 2 (Deep Excavation):</b> 0.5 m thick granular engineered fill <sup>1</sup> over compact native sand to silty sand	1.0	475	250
	2.0	500	265
	3.5	525	280
<b>OPTION 3 (Shallow Excavation)<sup>3,4</sup>:</b> Very stiff clayey silt (south) / 0.3 m to 0.4 m engineered fill <sup>1,2</sup> over firm to stiff clayey silt to silty clay (north)	1.0	285	150
	2.0	350	185
	3.5	375	200

## Notes:

1. Such as Granular 'A' or Granular 'B' Type II.
2. Varies in thickness depending on excavation of existing fill.
3. It is accepted that 25 mm of polystyrene foam placed above the footing provides protection against frost action which is equivalent to 600 mm of soil cover (see Section 2.2.3.4).
4. Based on frost line of 1.8 m below ground surface or 1.2 m below ground if 25 mm of polystyrene is used (see Section 2.2.3.4). The thickness of polystyrene foam is recommended based on stability analyses results (refer to Section 2.5.1).

Since the ULS resistance and the settlement depend on the footing size and depth of embedment, the geotechnical resistances given in Table 2.3 should be reviewed if the selected footing width or founding elevations differ from those given in the table. Similarly, if an inclined load is applied instead of a vertical load, which is used in these calculations, the values given in Table 2.3 should be reviewed to consider those inclinations.

Prior to placing footings, the exposed native subgrade should be inspected according to OPSS.PROV 902 and SP 109S12. A Qualified Geotechnical Engineer should check that the design foundation elevation is achieved and all unsuitable soils including fill and organics should be removed.

### 2.2.3.3 Resistance to Lateral Loads

Resistance to lateral forces/sliding resistance between the subgrade and concrete should be calculated in accordance with Section 6.10.5 of the CHBDC. The unfactored values of the coefficient of friction,  $\tan \delta$ , between the base of cast-in-place concrete footing and the native firm to stiff clayey silt/compact sand to silty sand/engineered fill (e.g., Granular 'A' or Granular 'B' Type II) are presented in Table 2.4.

Table 2.4. Recommendations for coefficient of friction

Interface	Coefficient of Friction, $\tan \delta$
Cast-in- place concrete and native loose to compact sand/silty sand	0.50
Cast-in-place concrete and native firm to very stiff silty clay to clayey silt	0.45
Cast-in-place concrete and Granular 'A' or Granular 'B' Type II	0.60

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

#### 2.2.3.4 Frost Protection

According to Ontario Provincial Standard Drawing (OPSD – 3090.101), the frost depth in the Dunchurch area is about 1.8 m. Consequently, all footings exposed to seasonal freezing conditions should be protected from frost action by at least 1.8 m of soil cover or equivalent approved insulation for frost protection. Equivalent protection could be provided by using polystyrene as suggested by the “Canadian Foundation Engineering Manual 2006, Section 13.5.2. page 196”. It is usually accepted that 25 mm of polystyrene provides frost protection which is equivalent to 600 mm of soil cover.

#### 2.2.3.5 Structure Backfill

The selection and placing of backfill should be in accordance with OPSS.PROV 902. Backfill immediately behind the walls should consist of free-draining, non-frost susceptible granular materials conforming to OPSS.PROV 1010 (Granular A, Granular B Type I or Type II, or Selected Subgrade Material (SSM)). If existing fill material is used, a review should be conducted during excavation to ensure the material conforms to SSM in OPSS.PROV 1010. All granular backfill should be placed in thick lifts (i.e., not exceeding 300 mm before compaction). Each lift should be compacted to 100% of its Standard Proctor Maximum Dry Density (SPMDD) below the floor slab, while within outside/exterior areas the fill should be compacted to 98% of its SPMDD.

#### 2.2.3.6 Lateral Earth Pressure

##### 2.2.3.6.1 Lateral Earth Pressure for Static Design

Perimeter walls (for sand/salt stockpile), structure walls, and temporary shoring (if required) should be designed to resist lateral earth pressure. The expression for calculating lateral earth pressure is given by:

$$P = K(\gamma z + q)$$

where,

P = earth pressure intensity at depth z, kPa

K = earth pressure coefficient

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

q = surcharge near wall, kPa

z = 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 the given materials assuming that wall friction is neglected, a level ground surface in front of the wall and on the retained side, and a vertical back face of the wall. Earth pressure parameters for the sand/salt stockpile assuming a 1.5H:1V slope on the retained side are also provided.

Table 2.5. Material types and earth pressure properties under static conditions

Material	Unfactored Friction Angle $\phi'$ (°)	Coefficient of Active Earth Pressure ( $K_a$ )	Coefficient of Passive Earth Pressure ( $K_p$ )	Coefficient of Earth Pressure At- Rest ( $K_0$ )	Unit Weight $\gamma$ (kN/m <sup>3</sup> )
With level backfill slope on retained side					
Granular A/B Type II	35	0.27	3.69	0.43	22.8
Engineered Fill (Granular B Type I or SSM)	32	0.31	3.26	0.47	21
Stockpiled Sand/Salt	33	0.30	3.39	0.46	20
With 1.5H:1V backfill slope on retained side					
Stockpiled Sand/Salt	33	0.65	-	0.70	20

The mobilization of full active or passive resistance requires a measurable and perhaps significant wall movement or rotation. Therefore, unless the structural element can tolerate these deflections, the at-rest earth pressure should be used in design. The coefficients of lateral earth pressure above are provided for level backfill behind the wall (perpendicular to the wall face plane) and should be adjusted in the case of a 1.5H:1V slope for the sand/salt stockpile.

The effect of compaction surcharge should be considered in the calculations of active and at rest earth pressures during backfilling up to the finished grade. 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 retaining walls to prevent overstressing.

#### 2.2.3.6.2 Lateral Earth Pressure for Seismic Design

The total lateral earth pressure should be calculated considering the static ( $K(\gamma z + q)$ ) and seismic ( $(K_{ae} - K_a)\gamma(h - z)$ ) components using the following equation below:

$$P = K(\gamma z + q) + (K_{ae} - K_a)\gamma(h - z)$$

where,

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

$K$  = earth pressure coefficient ( $K_a$  for yielding walls,  $K_0$  for non-yielding walls)

$K_a$  = static active earth pressure coefficient

$K_{ae}$  = seismic active earth pressure coefficient

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

$q$  = surcharge near wall, kPa

$h$  = total height of wall, m

$z$  = depth to point of interest, m

Seismic lateral earth pressure parameters yielding and non-yielding walls are provided in Sections 2.2.3.6.2.1 and 2.2.3.6.2.2.

#### 2.2.3.6.2.1 Yielding Walls

Seismic loading should be taken into account in the design in accordance with Section 6.14.7 of the CHBDC. These estimates are based on the Mononobe-Okabe (M-O) pseudo-static method of analysis. The M-O method produces seismic loads that are more critical than the static loads that act prior to an earthquake. The M-O method of seismic lateral earth pressure coefficients for the structural design can be estimated in accordance with Section 6.14.7.2 and C6.14.7.2 of the CHBDC and its Commentary, respectively.

When calculating seismic lateral earth pressures on walls that are capable of moving 25 to 50 mm using the M-O formulation, the seismic horizontal acceleration coefficient ( $k_h$ ) should be taken as half of the site-adjusted PGA, where, the site-adjusted PGA estimated at ground surface is given as  $F(PGA)*PGA$ , where,  $F(PGA)$  is the PGA-based amplification factor that corresponds to the applicable Site Class as defined in Table 4.8 of the Code. For this site, a PGA of 0.113 g (Site Class C), earthquake having a 2% probability of exceedance in 50 years (1 in 2,475-year return period) can be used in the calculation of the seismic active pressure coefficient.  $k_h$  is estimated to be 0.057 g and was used for lateral earth pressures for seismic design.

The effect of the seismic vertical acceleration coefficient ( $k_v$ ) should be ignored when calculating the seismic lateral earth pressure coefficients. However, the minimum peak vertical acceleration coefficient can be taken as two-thirds of the peak horizontal acceleration coefficient, in accordance with Section 4.4.3.6 of the CHBDC when calculating the seismic lateral earth load.

It should be noted that in the computation of seismic earth pressure coefficients, the wall back-face geometry, backfill slope, and wall friction effects need to be addressed. For design purposes, the following unfactored seismic lateral earth pressure parameters in Table 2.6 can be used (assuming wall friction is neglected, a level ground surface in front of the wall and on the retained side and the back face of the wall is vertical).

Table 2.6. Material types and earth pressure properties under seismic conditions for yielding walls

Material	Unfactored Friction Angle $\phi'$ (°)	Coefficient of Seismic Active Earth Pressure ( $K_{ae}$ )	Coefficient of Seismic Passive Earth Pressure ( $K_{pe}$ )	Unit Weight $\gamma$ (kN/m <sup>3</sup> )
Granular A/B Type II	35	0.30	3.58	22.8
Engineered Fill (Granular B Type I or SSM)	32	0.34	3.15	21
Stockpiled Sand/Salt	33	0.33	3.29	20



### 2.2.3.6.2.2 Non-Yielding Walls

For walls that are restrained against lateral movement, the seismic lateral earth pressures should be obtained using the M-O formulation and using a seismic horizontal acceleration coefficient ( $k_h$ ) equal to the PGA, where the site-adjusted PGA estimated at the ground surface is given as  $F(PGA) \cdot PGA = 0.113 \text{ g}$ . The same values for  $F(PGA)$  and PGA are used from Section 2.2.3.6.2.1. The acceleration coefficient determined at the original ground surface should be the acceleration coefficient acting at the wall base. The seismic vertical acceleration coefficient ( $k_v$ ) can be ignored when calculating the seismic lateral earth pressure coefficient. For design purposes, the following unfactored seismic lateral earth pressure parameters for non-yielding walls are provided in Table 2.7.

Table 2.7. Material types and earth pressure properties under seismic conditions for non-yielding walls

Material	Unfactored Friction Angle $\phi' (^{\circ})$	Coefficient of Seismic Active Earth Pressure ( $K_{ae}$ )	Coefficient of Seismic Passive Earth Pressure ( $K_{pe}$ )	Unit Weight $\gamma \text{ (kN/m}^3\text{)}$
Granular A/B Type II	35	0.34	3.47	22.8
Engineered Fill (Granular B Type I or SSM)	32	0.38	3.04	21
Stockpiled Sand/Salt	33	0.36	3.18	20

### 2.2.3.7 Resistance to Uplift Loads

Resistance to uplift for the footings should be calculated based on the (i) dead load on the footing (ii) weight of the footing and (iii) weight of soil above the footing (i.e., burial depth and soil unit weight). Unit weights and friction angles of soil, provided in Table 2.5, can be used to determine the uplift resistances. The uplift resistance can be calculated using the methodology presented in Bowles (1997).

## 2.3 Seismic Potential Consideration

### 2.3.1 Seismic Hazard Site Classification and Values

Seismic characterization of the site should be compliant with the OBC (2012) and CHBDC. The potential for seismic loading must be considered for design in accordance with Section 4.1.8 of the OBC (2012) and Section 6.14.7 of the CHBDC with respect to the soil conditions encountered at the site. Table 4.1.8.4.A in OBC (2012) and Table 4.1 CHBDC show site classification for seismic site response based on average soil properties in the top 30 m.

At this site, the subsoil generally consists of sand fill, followed by native clayey silt to silty clay over sand to silty sand underlain by bedrock. The bedrock was confirmed to be at depths ranging from 2.7 m to 4.2 m below ground surface. The groundwater level was measured in four (4) open holes prior to the use of water for rock coring which ranged from approximately a depth of 1.8 m to 2.4 m (corresponding to Elev. 279.9 m to Elev. 279.4 m) below existing ground surface. Based on soil characteristics, the site class for this site is estimated to be Class "C" according to Table 4.1.8.4.A of the OBC (2012) and 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 2.8 below:

Table 2.8. Seismic design values

Probability of Exceedance in 50 Years (Return Period)	Sa(0.2) (g)	Sa(0.5) (g)	Sa(1.0) (g)	Sa(2.0) (g)	PGA (g)
Latitude: 45.658464; Longitude: -79.8109536					
2% (1 in 2475-year)	0.25	0.173	0.0976	0.0471	0.113

Note:

1.  $g$  = acceleration due to gravity ( $9.81 \text{ m/s}^2$ )

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 C is also shown on the seismic hazard calculation data sheet for this site attached in Appendix H.

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 greater than 2.0 at this site,  $PGA_{ref}$  is equal to  $PGA=0.113g$  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 C and  $PGA_{ref} = 0.113g$ ) is 1.00.

### 2.3.2 Liquefaction Considerations

Liquefaction of cohesionless soils below the groundwater table (assumed to be ~Elev. 279.9 m), including the native silty clay to clayey silt and sand to silty sand at the project site was evaluated through the SPT-based liquefaction triggering procedures described in Boulanger and Idriss (2014) using the site-adjusted  $PGA = 0.113 \text{ g}$  (1 in 2475-year event). This involves comparing the cyclic stress ratio (CSR), which are the cyclic shear stresses within the soil induced by seismic forces, and the cyclic resistance ratio (CRR) of the soil.

At the northeast corner of the structure, native soils below the groundwater table consists of very loose to compact sand to silty sand above the bedrock. Very loose silty sand was encountered at a depth of about 3.0 m (Elev. 278.9 m) and was determined to be marginally susceptible to liquefaction in the case of  $PGA = 0.113 \text{ g}$  (1 in 2475-year event).

## 2.4 Perimeter Wall and Floor Construction

The outer perimeter wall of the proposed structure may be constructed as a cantilever retaining wall with an extended heel toward the inside of the structure while the internal wall may be constructed as a cantilever retaining wall with a heel that extends equally on both sides. These may be founded on native soils or engineered fill. Structural steel bars should be provided in the footings and in the walls.

The concrete and/or asphalt floor slabs supported on the existing granular fill or on engineered fill could be designed inside the structure. Based on available information, the floor slab/asphalt surface elevation will be approximately 281.7 m to 281.9 m. Below the floor, a sub-floor drainage system should be placed and compacted as described later in this section. The asphalt pavement structure will need to be designed by a pavement engineer. The concrete floor slab will need to be designed by a structural engineer specialist as well. However, it is recommended that for the concrete floor slab the final lift of granular fill beneath the floor slabs should consist of a minimum thickness of 200 mm of OPSS Granular A material, uniformly compacted to 100% of SPMDD. For this condition, a modulus of subgrade reaction  $k_v$  of 50 MPa/m may be assumed for preliminary assessment purposes. For design purposes, the value provided above needs to be modified to account for size effects as per standard design methods as outlined in the CFEM 2006. The concrete floor slabs should be structurally separate from the foundation walls and columns and

saw-cut control joints should be provided at regular intervals along column lines to minimize shrinkage cracking and allow for normal differential settlement of the floor slabs. Considering that the floor will be covered by sand/salt stockpile during cold weather, frost protection is not considered necessary.

The construction of spread footings and subgrade for the floor may be carried out in accordance with the following recommendations:

Prior to construction, all obviously unsuitable material should be fully removed from the entire underfooting and underfloor area (see Section 2.6). Following rough grading, the exposed subgrade should be proof-rolled with a roller under the full-time supervision of a qualified geotechnical personnel. Any soft spots detected during proof-rolling should be sub-excavated and replaced with Granular A or Granular B, Type I or Type II materials compacted to at least 98 % of the Standard Proctor Maximum Dry Density (SPMDD). The prepared subgrade should be covered with at least 200 mm thick layer of Granular A compacted to not less than 100% of the material's SPMDD, crowned slightly in the central area.

Around the perimeter of the building the ground surface should be sloped on a positive grade away from the structure to promote surface water run-off and reduce groundwater infiltration adjacent to the foundations. Permanent perimeter drains are not required if the interior base is set at least 200 mm above the exterior grade and the grade is sloped away from the structure. However, a permanent subfloor drainage system may be required to collect salt-bearing water. To minimize contamination into the native soils and subsequently into the groundwater, a barrier such as a compacted low-permeability clay liner or geomembrane usually should be installed below a salt storage area. In practice, the use of geomembrane shows an advantage over the compacted clay liner in terms of improved performance of the barrier. The geomembrane should be installed on a minimum 75 mm thick sand layer (OPSS.PROV 1004 or OPSS.PROV 1002) and covered with a 300 mm thick layer of sand fill on top of the geomembrane to protect it from the overlying pavement structure.

## 2.5 Stability and Settlement Analyses

### 2.5.1 Stability

To assess the global stability of the material storage facility and to check that a minimum Factor of Safety (FOS) of 1.3 under static conditions and 1.1 under seismic conditions will be achieved for the maximum height winter sand/salt stockpiles, a series of slope stability analyses were performed. The slope stability analyses were performed using the Morgenstern-Price method developed based on limit equilibrium. The SLOPE/W computer program developed by GeoSlope International was employed for computation. For seismic conditions, a horizontal seismic coefficient,  $k_h$ , is half of the site-adjusted peak horizontal ground acceleration. In reference to the provided seismic design values in Section 2.3.1,  $k_h = 0.5 \cdot F(PGA) \cdot PGA = 0.057$  g was used when evaluating global stability of the stockpile under seismic conditions.

Stability assessments were performed for the proposed new structure of 50 m x 24 m dimensions assuming that the maximum sand/salt stockpile height could be about 8.1 m at the center of the stockpile (4.2 m above the outer concrete walls) having side slopes of 1.5H:1V as shown on Figures F1 to F6 in Appendix F. The soil stratigraphy and groundwater condition at the site were developed based on the results of the geotechnical investigation presented in Part I - Foundation Investigation Report.

Stability assessments were performed on recommended Option 3 with footing depth of 1.2 m which is the more critical option from a stability point of view. The stability analyses for this option show that a footing depth of 1.2 m is the minimum embedment required to achieve global stability requirements of a Factor of Safety of 1.3 under static

conditions and 1.1 under seismic conditions. The footing depth of 1.2 m would require ~25 mm of polystyrene foam placed above the footing to protect against frost action (see Section 2.2.3.4.).

Given the site subsurface conditions (i.e., cohesionless and cohesive soils), total stress analyses for short-term stability and effective stress analyses for long-term stability assessments were performed taking into consideration the subsoil conditions encountered directly beneath and adjacent to the proposed structure. The areas extending at least 1.0 meters beyond the outside edge of any footings of the building should be stripped/excavated and cleared of asphalt, surface vegetation, peat, topsoil, excessive organics, existing fill, weak/ disturbed/ deleterious/ compressible or loose materials and debris prior to construction, and should be replaced with engineered fill comprised of Granular A or Granular B, Type II.

Tabulated below in Table 2.9 are the soil parameters used for the slope stability analyses. The soil parameters were generally estimated based on the results of field and laboratory testing. Table 2.10 below summarizes results of performed slope stability analyses.

Table 2.9. Soil properties used in slope stability analyses

Material Type	Effective Stress Parameters		Undrained Shear Strength, $C_u$	Effective Stress Parameters $\gamma$
	$\phi'$ (degrees)	$\phi'$ (degrees)	(degrees)	(kN/m <sup>3</sup> )
Engineered Granular Fill	32	0	-	21
Granular A/B Type II	35	0	-	22.8
Existing Sand Fill (compact)	30	0	-	20
Clayey Silt to Silty Clay (firm to stiff)	29	0	45	19
Sand/Silty Sand (very loose to compact)	29	0	-	20
Stockpile Material (Winter sand/salt)	33	0	-	20
Bedrock	Impenetrable			

Table 2.10. Summary of results of slope stability analyses

Location	Max Height (m)	Conditions	Min FOS
North-South Section	8.1 m (4.2 m above outer concrete walls + 3.9 m high outer concrete walls)	Undrained short-term conditions, static condition	1.5 (Figure F1)
		Drained long-term conditions, static condition	1.5 (Figure F2)
		Drained long-term conditions, seismic condition	1.3 (Figure F3)
East-West Section		Undrained short-term conditions, static condition	1.6 (Figure F4)
		Drained long-term conditions, static condition	1.6 (Figure F5)
		Drained long-term conditions, seismic condition	1.4 (Figure F6)

The graphical results of these analyses can be seen in Figures F1 to F6 in Appendix F. As shown on the figures, the results of stability analyses for an approximately 8.1 m high winter sand/salt stockpile (with side slopes at 1.5H:1V) restrained with 3.9 m high outer concrete walls on both sides in the building suggest that the factor of safety of

minimum 1.3 or greater for static conditions and a minimum 1.1 for seismic conditions can be obtained for a deep-seated failure surface.

## 2.5.2 Settlement

To evaluate the maximum and differential settlement values below the sand/salt stockpile loadings in the proposed storage building, a 3D computer program; Settle3D (Rocscience) was employed. The properties for the encountered soil layers used in the settlement model are evaluated based on the results of the laboratory and field tests as per CHBDC. The estimated parameters for settlement analyses are listed in Table 2.11.

Table 2.11. Soil properties used in settlement analyses

Material Type	$\gamma$ (kN/m <sup>3</sup> )	E (MPa)	C <sub>c</sub>	C <sub>r</sub>	P <sub>c</sub> (kPa)	e <sub>0</sub>	C <sub>v</sub> (cm <sup>2</sup> /s)
Engineered Fill	21	100	-	-	-	-	-
Granular A/B Type II	22.8	100	-	-	-	-	-
Clayey silt / Silty Clay (firm to stiff)	19	20	0.22	0.022	150	0.55	0.002
Sand/Silty Sand (very loose to compact)	20	30	-	-	-	-	-

The geometry of the stockpiles was assumed based on its maximum allowable capacity which is a maximum height of approximately 8.1 m at the center and ~3.9 m (outer wall)/~2.9 m (inner wall) along the sides at the walls. The model is illustrated on Figure G1 included in Appendix G. The estimated settlements under the stockpile at the center and at the edges of the stockpile (i.e., location of footings) are presented in Table 2.12.

Table 2.12. Results of settlement analyses

Location	Foundation Soil Type	Estimated Elastic Settlement (mm)		Estimated Consolidation Settlement (mm)		Estimated Total Settlement (mm)	
		Edge	Centre	Edge	Centre	Edge	Centre
Sand/Salt Storage Area	Engineered Fill Over Firm to Very Stiff Clayey Silt to Silty Clay Underlain by Very Loose to Compact Sand to Silty Sand Followed by Bedrock	5	13	8	22	13	35

The calculated settlements are anticipated to occur immediately after the stockpile loadings are applied or within a period of one month. The footings for this structure should be designed under the full allowable stockpile loadings. The geometries of stockpiles under the full allowable loadings including their possible maximum heights are recommended above.

## 2.6 Site Preparation and Engineered Fill Construction

As mentioned previously, the areas within the limits of the buildings should be stripped and cleared of asphalt, surface vegetation, peat, topsoil and debris prior to construction. Any soils containing excessive organics or loose/disturbed materials are not suitable for the subgrade of building foundations, floor slabs or engineered fill. Therefore, areas with those soils should be excavated and replaced with engineered fill comprised of Granular A or

Granular B Type II (below the groundwater table). A mud slab consisting of 0.1 m (4 inches) of concrete can be utilized under the footings if and where required to protect the foundation soils from potential disturbance.

Engineered fill (e.g., Granular A or Granular B Type II) could be placed after stripping all topsoil, peat, organic matter, fill and other compressible, weak, and deleterious materials within an area extending at least 1.0 meter beyond the outside edge of the founding level of any footings. After stripping, the entire area should be heavily proof-rolled inspected and approved by a Geotechnical Engineer. Granular A should be placed in accordance with OPSS.PROV 501 and SP105S22 while Granular B Type II should be placed in accordance with OPSS.PROV 314. The fill material should be placed in thin layers not exceeding approximately 300 mm when loose. Oversize particles larger than 120 mm should be discarded, and each fill layer should be uniformly compacted with heavy compactors, suitable for the type of fill used. The engineered fill below the footing and floor slab should be compacted to 100% of its SPMDD, while within outside/exterior areas, the fill should be compacted to 98% of its SPMDD.

Full-time geotechnical inspection and quality control (by means of frequent field density and laboratory testing) should be provided by the Geotechnical Engineer. Every lift should be evaluated by a sufficient number of tests to ensure that the level of compaction is constantly achieved, and the compaction procedure is applied.

## 2.7 Excavation

All excavations should be carried out in accordance with the latest version of the Occupational Health and Safety Act. For the act, the existing fills and native soils are considered as 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 about 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.

Qualified geotechnical personnel should be on-site during the foundation installation and for fill material placement, to verify the design assumptions, and to verify the design recommendations.

## 2.8 Groundwater Control

Based on the assessment of the water levels observed in four (4) open holes prior to rock coring, groundwater levels within the site are interpreted to be at depths ranging between approximately 1.8 m to 2.4 m (Elev. 279.4 m to 279.9 m) below the ground surface.

Since the depth of excavation for footings could range from 1.2 m to 2.3 m (Elev. 280.5 m to 279.4 m), the groundwater level could be above, at, or below the excavation depth depending on the depth of footing selected and thickness of sub-excavation and engineered fill placement. It is anticipated that active dewatering will not be required.

If groundwater is at/above the excavation depth, the groundwater level needs to be controlled to at least 0.5 m below the excavation level to avoid disturbance, and any groundwater seepage should be removed from the excavation prior to placement of granular backfill such that the granular backfill is placement in the dry. In general, where the excavation base is within 0.5 m of the prevailing groundwater level at the time of construction, it is anticipated that control of seepage can be accomplished by using properly filtered sumps, and/or filtered drains placed along the base the excavation.

Surface water should be directed away from the excavation. Dewatering shall be carried out in accordance with OPSS.PROV 517 and SP517F01. It is responsibility of the Contractor to propose a suitable dewatering system based on the time of construction, water levels and flow conditions. The method used should not undermine the existing utilities/structures (if any).

## 2.9 Corrosion Protection

One (1) soil sample was selected for analyses of pH, water soluble sulphate, chloride concentrations, resistivity, conductivity, and oxidation-reduction potential. The testing was completed to determine the potential for degradation of the concrete in the presence of soluble sulphate and the potential for corrosion of exposed steel used in foundations and buried infrastructure. The analysis results are summarized in Section 1.7 of this report and detailed results are included in Appendix D.

The chemical data presented in Section 1.7 indicates the resistivity of the tested soil is 1200 ohm-cm, which suggests a severe potential for corrosion of buried metallic elements (MTO Gravity Pipe Design Guidelines, Page 34). The maximum chloride content reported is 430 ppm ( $\mu\text{g/g}$ ) which also indicates low potential for additional corrosion. The soil pH was about 6.73 which is within what is considered the normal range for soil pH of 5.0 to 9.0. The test results in Table 1.7 may be used to aid in the selection of coatings and corrosion protection systems for buried steel objects. Based on the results of sample tested and given that the structure is a salt/sand storage, consideration should be given by the designer to designing for a «C» type of exposure concrete class as defined by CSA A23.1 Table 1.

The maximum water-soluble sulphate content of the soils tested is less than 20 ppm ( $\mu\text{g/g}$ ), i.e. 0.002% which is less than 0.10%. It indicates low potential to corrode normal Portland cement concrete. Therefore, no particular precautions are required to provide protection against sulphate attack such as special cements or mixtures.

### 3 CLOSURE

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

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

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

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

This Foundation Investigation and Design Report has been prepared by Daniel Mroz, M.E.Sc., EIT, Ciarra Alexander, M.Eng., and Silvana Micic, Ph.D., P.Eng., and reviewed by TaeChul Kim, M.E.Sc., P.Eng. and Stan E. Gonsalves, M.Eng., P.Eng., MTO Designated Foundation Contact. The field investigation was conducted by Elvis Lu, M.Eng., EIT.

EXP Services Inc.



Daniel Mroz, M.E.Sc., EIT  
Technical Specialist



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



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



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





## REFERENCES

- Boulanger, R. W. and Idriss, I. M., 2014. CPT and SPT based liquefaction triggering procedures. Department of Civil and Environmental Engineering, College of Engineering, University of California at Davis.
- Bowles, J.E., 1997. Foundation Analysis and Design, 5th Edition. McGraw-Hill.
- Canadian Geotechnical Society, 2006. Canadian Foundation Engineering Manual, 4th Edition. The Canadian Geotechnical Society, BiTech Publisher Ltd., British Columbia.
- Canadian Standards Association (CSA), 2019. Canadian Highway Bridge Design Code and Commentary on CAN/CSA-S6-19. CSA Special Publication.
- Ministry of Northern Development and Mines, Map 2556. Quaternary Geology of Ontario, Southern Sheet, 1991
- Ministry of Northern Development and Mines Map 2544. Bedrock Geology of Ontario, Southern Sheet, 1991
- Ministry of Transportation, April 2022. Guideline for MTO Foundation Engineering Services, Version 03
- Ontario Building Code, 2012. O.Reg. 332/12, Division B, Part 4 – Structural Design.

### **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 314 Construction Specification for Untreated Subbase, Base, Surface, Shoulder, Selected Subgrade, and Stockpiling
- OPSS.PROV 501 Construction Specification for Compacting
- OPSS.PROV 517 Construction Specification for Dewatering
- OPSS.PROV 902 Construction Specification for Excavating and Backfilling – Structures
- OPSS.PROV 1010 Material Specification for Aggregates - Base, Subbase, Select Subgrade, And Backfill Material

### **Ontario Provincial Standard Drawings (OPSD):**

OPSD 3090.101 Foundation Frost Depths for Southern Ontario

### **Special Provisions (SP):**

- SP 105S22 AMENDMENT TO OPSS.PROV 501
- SP 109S12 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

Project Name: Agreement No. 5021-E-0020, Assignment No. 10

Foundation Investigation and Design Report

New Material Storage Facility at Dunchurch Patrol Yard, Highway 124, Dunchurch, ON

Date: April 19, 2024

**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 utilized specific software and hardware systems. EXP makes no representation about the compatibility of these files with the Client's current or future software and hardware systems. Regardless of format, the documents described herein are EXP's instruments of professional service and shall not be altered without the written consent of EXP.

## Appendix A – Site Photographs



Photo 1. Dunchurch Patrol Yard – Drilling borehole BH23-D-1, facing southeast (January 19, 2024)



Photo 2. Dunchurch Patrol Yard – Drilling borehole BH23-D-2, facing west (January 18, 2024)





Photo 3. Dunchurch Patrol Yard – Drilling borehole BH23-D-3, facing southeast (January 18, 2024)

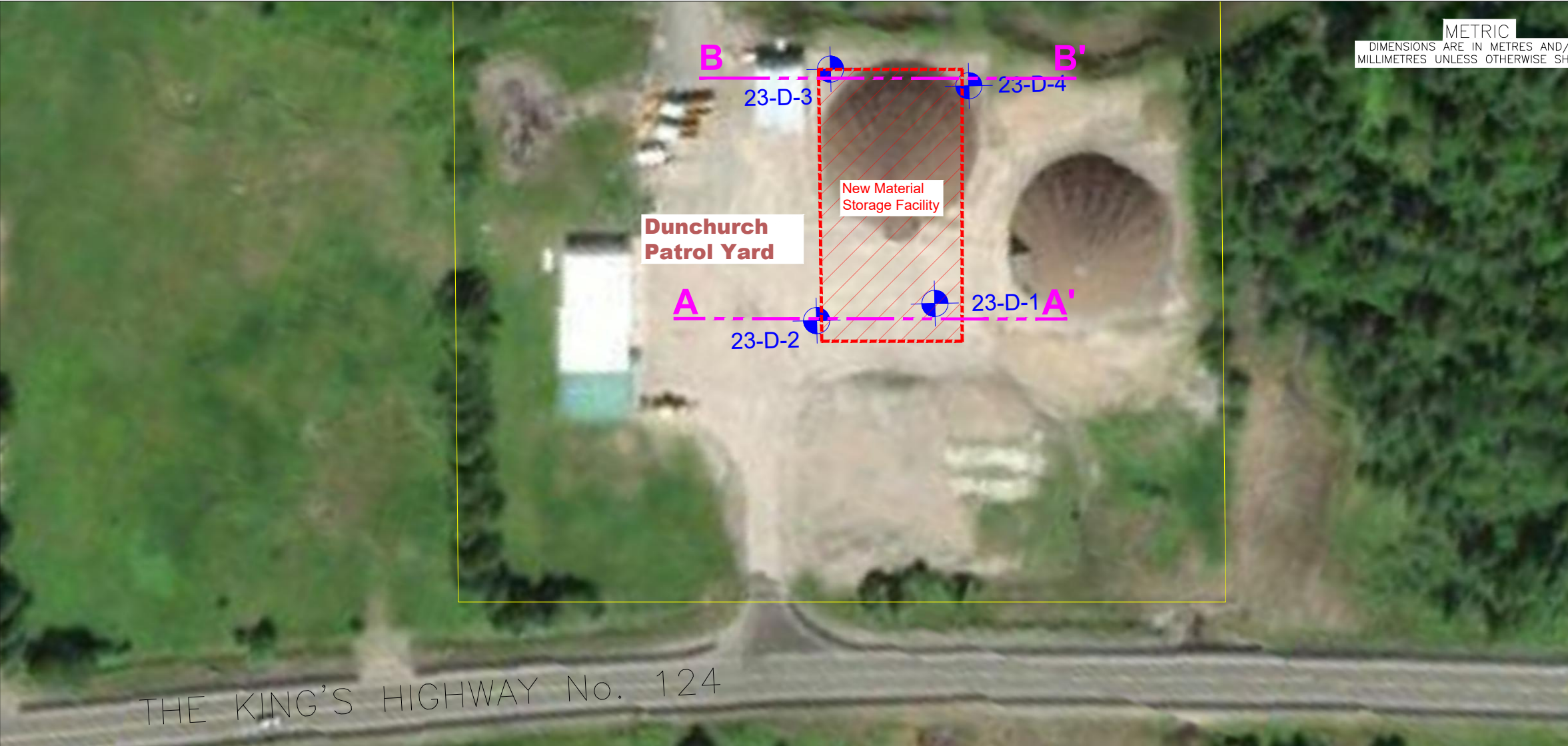


Photo 4. Dunchurch Patrol Yard – Drilling borehole BH23-D-4, facing south (January 18, 2024)

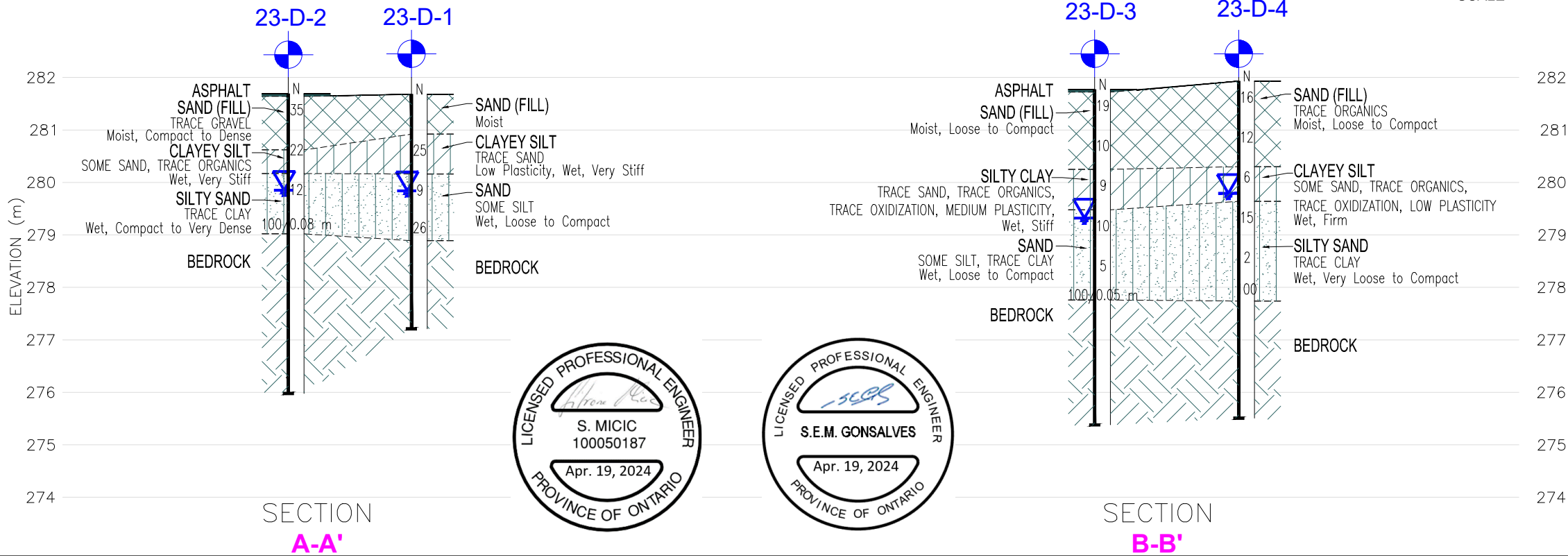
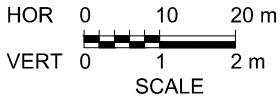
## Appendix B – Drawings



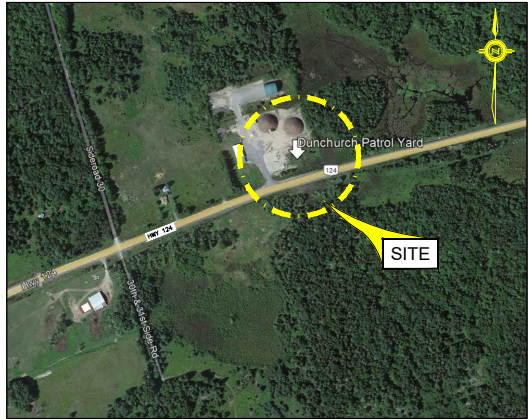
FILE NAME: I:\2003-Brampton\Proposals\Projects\International\WTO Projects\Retainer NER\5021-E-0020\Assignment 10 - Mattawa and Dunchurch Patrol Yards\AutoCAD\Working drawings\Dunchurch Patrol Yard\_plan & profile.dwg  
MODIFIED: 2024-02-26 12:35



PLAN








CONT No. 5021-E-0020	
ASSIG No. 10	
GWP No.	
New Material Storage Facility at Dunchurch Patrol Yard, Highway 124, Dunchurch, ON <i>Latitude: 45.658464°; Longitude: -79.810954°</i>	
BOREHOLE LOCATION PLAN & SOIL STRATA	
SHEET	
1	



KEY PLAN
N.T.S.
LEGEND

- Borehole Location
- Water Level Upon Completion of Drilling (W. L. NOT STABILIZED)
- N** Blows/0.3m (Std. Pen. Test, 475 J/blow)

SOIL STRATA SYMBOLS	
	ASPHALT
	FILL
	CLAYEY SILT/ SILTY CLAY
	SILTY SAND/ SAND
	BEDROCK

BOREHOLE CO-ORDINATES/ NAD 83/ MTM ON-10			
BH No.	ELEV.	NORTHING	EASTING
23-D-1	281.7	5057700	280537
23-D-2	281.7	5057689	280516
23-D-3	281.8	5057737	280502
23-D-4	281.9	5057743	280529

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.			

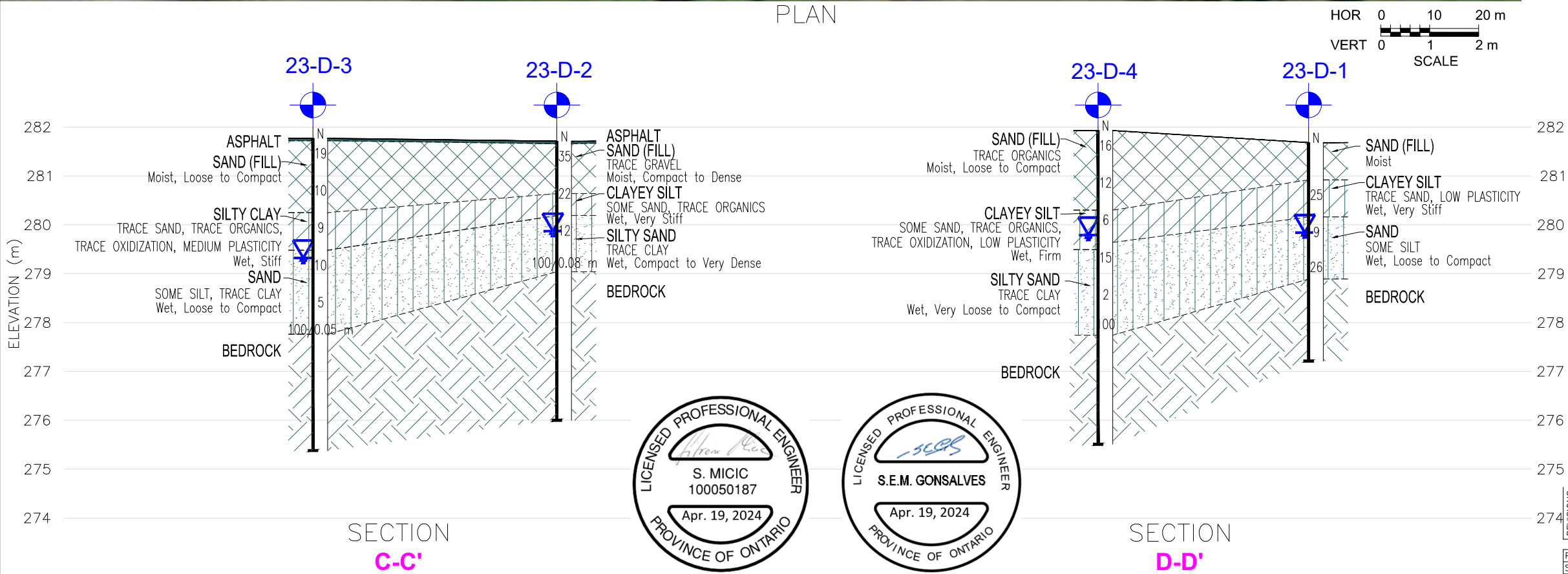
SUBMISSION FOR MTO REVIEW			
REVISIONS			
NO	DATE	BY	DESCRIPTION
PROJECT No. ADM-22006096-A9			
SUBM'D SH		CHKD. SM	GEOCREs No. 31E12-001
DRAWN SH		CHKD. TC	DATE APR. 19, 2024 SITE-
		APPRD SG	DWG 01



FILE NAME: I:\2003-Brampton\Proposals\Projects\International\WTO Projects\Retainer NER\5021-E-0020\Assignment 10 - Mattawa and Dunchurch Patrol Yards\AutoCAD\Working drawings\Patrol Yard\_plan & profile.dwg  
MODIFIED: 2024-02-26 12:35



PLAN



SECTION  
C-C'

SECTION  
D-D'

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

CONT No. 5021-E-0020  
ASSIG No. 10  
GWP No.

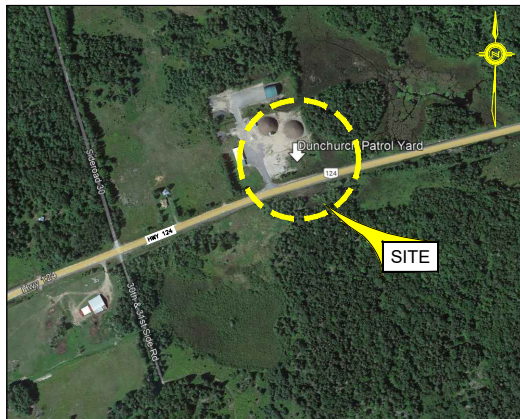


New Material Storage Facility at Dunchurch Patrol Yard,  
Highway 124, Dunchurch, ON  
Latitude: 45.658464°; Longitude: -79.810954°  
BOREHOLE LOCATION PLAN & SOIL STRATA

SHEET  
2



EXP SERVICES INC.



KEY PLAN  
N.T.S.

LEGEND

- Borehole Location
- Water Level Upon Completion of Drilling  
(W. L. NOT STABILIZED)
- Blows/0.3m (Std. Pen. Test, 475 J/blow)

SOIL STRATA SYMBOLS

- ASPHALT
- FILL
- CLAYEY SILT/  
SILTY CLAY
- SILTY SAND/ SAND
- BEDROCK

BOREHOLE CO-ORDINATES/ NAD 83/ MTM ON-10

BH No.	ELEV.	NORTHING	EASTING
23-D-1	281.7	5057700	280537
23-D-2	281.7	5057689	280516
23-D-3	281.8	5057737	280502
23-D-4	281.9	5057743	280529

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.

SUBMISSION FOR MTO REVIEW			
NO	DATE	BY	DESCRIPTION
PROJECT No.	ADM-22006096-A9	GEOCREs No.	31E12-001
SUBM'D SH	CHKD. SM	DATE	APR. 19, 2024 SITE.
DRAWN SH	CHKD. TC	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.

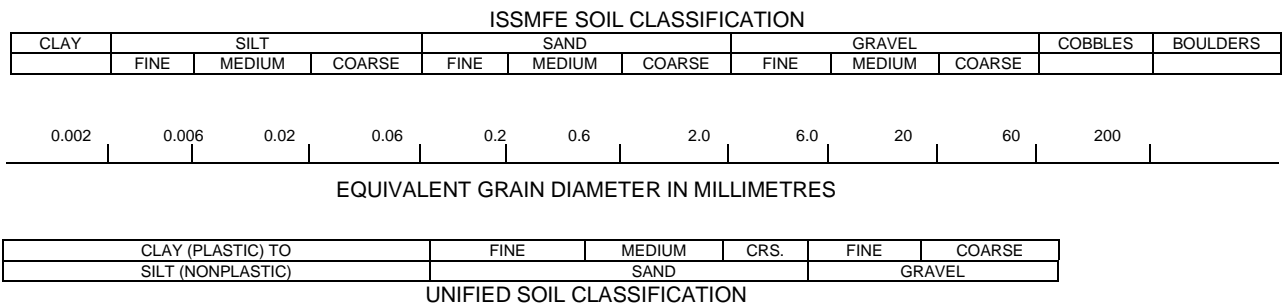
*Seam:* a thin, confined layer of soil having different particle size, texture, or color from materials above and below.

*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.



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:

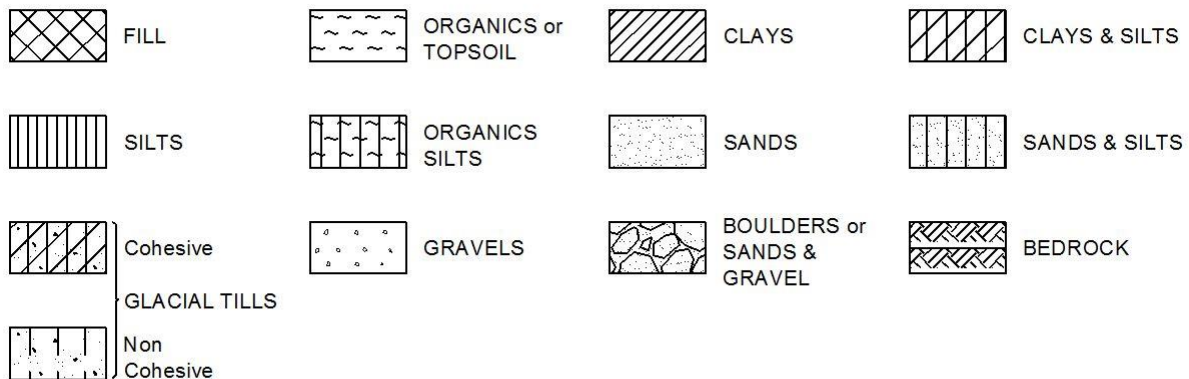
Table c: Consistency of Cohesive Soil

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



Open Borehole or Test Pit



Monitoring Well, Piezometer or Standpipe

## ABBREVIATIONS AND SYMBOLS

### FIELD SAMPLING

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

### STRESS AND STRAIN

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

### MECHANICAL PROPERTIES OF SOIL

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

### PHYSICAL PROPERTIES OF SOIL

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

Brampton, Ontario

RECORD OF BOREHOLE No 23-D-1										1 OF 1		METRIC				
W.P. _____		LOCATION 5057700N, 280537E, NAD83 MTM Zone 10				ORIGINATED BY EL										
DIST NE HWY 124		BOREHOLE TYPE Continuous Flight HSA, NW Casing, NQ Core Barrel				COMPILED BY CA										
DATUM Geodetic		DATE 2024.01.19 - 2024.01.19		LATITUDE 45.658765		LONGITUDE -79.811346		CHECKED BY SM								
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								
281.7 0.0	SAND (FILL), brown, moist		AS1	AS		▽										
280.9 0.8	CLAYEY SILT, trace sand, grey, low plasticity, wet, very stiff		SS2	SS	25											
280.2 1.5	SAND, some silt, brown, wet, loose to compact		SS3	SS	9											
			SS4	SS	26											
278.9 2.8	BEDROCK, dark grey to grey with quartz veining  Run 1: Start/End: 2.8 to 4.3 m Recovery: 100% RQD: 95%  Run 2: Start/End: 4.3 to 4.5 m Recovery: 83% RQD: 83%		RUN 1	NQ												
277.2 4.5	BOREHOLE TERMINATED AT ~ 4.5 m DEPTH  Notes: 1. Groundwater measured in open hole at 1.8 m depth prior to rock coring. 2. Borehole backfilled upon completion.		RUN2	NQ												

ONTARIO MTO DUNCHURCH.PY.GPJ ONTARIO MTO.GDT 2/26/24



Brampton, Ontario

## RECORD OF BOREHOLE No 23-D-2

1 OF 1

METRIC

W.P. \_\_\_\_\_ LOCATION 5057689N, 280516E, NAD83 MTM Zone 10 ORIGINATED BY EL  
DIST NE HWY 124 BOREHOLE TYPE Continuous Flight HSA, NW Casing, NQ Core Barrel COMPILED BY CA  
DATUM Geodetic DATE 2024.01.18 - 2024.01.18 LATITUDE 45.658665 LONGITUDE -79.811612 CHECKED BY SM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>P</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL P. PENETROMETER									
281.7							20	40	60	80	100						
280.9	ASPHALT, ~ 50 mm thick																
	SAND (FILL), trace gravel, brown, moist, compact to dense		SS1	SS	35								○				
280.6													○				
1.1	CLAYEY SILT, some sand, trace organics, grey, wet, very stiff		SS2	SS	22								○				
280.2																	
1.5	SILTY SAND, trace clay, grey, wet, compact		SS3	SS	12								○			0 71 26 3	
	- very dense at ~ 2.3 m depth																
279.0			SS4	SS	100/ 0.08 m								○				
2.7	BEDROCK, dark grey to grey with quartz veining and limestone interbeds																
	Run 1: Start/End: 2.7 to 4.2 m Recovery: 97% RQD: 88%		RUN 1	NQ												UCS test on Run 1 = 66.4 MPa	
	Run 2: Start/End: 4.2 to 5.7 m Recovery: 100% RQD: 100%																
			RUN 2	NQ													
276.0																	
5.7	BOREHOLE TERMINATED AT ~ 5.7 m DEPTH																
	Notes: 1. Groundwater measured in open hole at 1.8 m depth prior to rock coring. 2. Borehole backfilled upon completion.																





Brampton, Ontario

## RECORD OF BOREHOLE No 23-D-3

1 OF 1

METRIC

W.P. \_\_\_\_\_ LOCATION 5057737N, 280502E, NAD83 MTM Zone 10 ORIGINATED BY EL  
DIST NE HWY 124 BOREHOLE TYPE Continuous Flight HSA, NW Casing, NQ Core Barrel COMPILED BY CA  
DATUM Geodetic DATE 2024.01.18 - 2024.01.18 LATITUDE 45.659097 LONGITUDE -79.811793 CHECKED BY SM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT  γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60	W <sub>P</sub> W W <sub>L</sub>	WATER CONTENT (%)	GR SA SI CL				
281.8						▽										
280.9	ASPHALT, ~ 50 mm thick		SS1	SS	19											
	SAND (FILL), brown, moist, loose to compact															
	- grey below ~ 0.8 m depth		SS2	SS	10											
280.3																
1.5	SILTY CLAY, trace sand, trace organics, trace oxidization, grey, medium plasticity, wet, stiff		SS3	SS	9										0 5 63 32	
279.5																
2.3	SAND, some silt, trace clay, grey, wet, loose to compact		SS4	SS	10										Corrosivity Sample	
				SS5	SS		5									0 88 10 2
277.8	- very dense below ~ 3.8 m depth		SS6	SS	100/ 0.05 m											
4.0	BEDROCK, dark grey to grey with quartz veining		RUN 1	NQ												
	Run 1: Start/End: 4.0 to 4.9 m Recovery: 100% RQD: 100%															
	Run 2: Start/End: 4.9 to 6.4 m Recovery: 100% RQD: 95%		RUN 2	NQ												
275.4																
6.4	BOREHOLE TERMINATED AT ~ 6.4 m DEPTH															
	Notes: 1. Groundwater measured in open hole at 2.4 m depth prior to rock coring. 2. Borehole backfilled upon completion.															

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

ONTARIO MTO: DUNCHURCH.PY.GPJ ONTARIO MTO.GDT 2/26/24




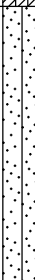

Brampton, Ontario

## RECORD OF BOREHOLE No 23-D-4

1 OF 1

METRIC

W.P. \_\_\_\_\_ LOCATION 5057743N, 280529E, NAD83 MTM Zone 10 ORIGINATED BY EL  
DIST NE HWY 124 BOREHOLE TYPE Continuous Flight HSA, NW Casing, NQ Core Barrel COMPILED BY CA  
DATUM Geodetic DATE 2024.01.18 - 2024.01.18 LATITUDE 45.659158 LONGITUDE -79.811453 CHECKED BY SM

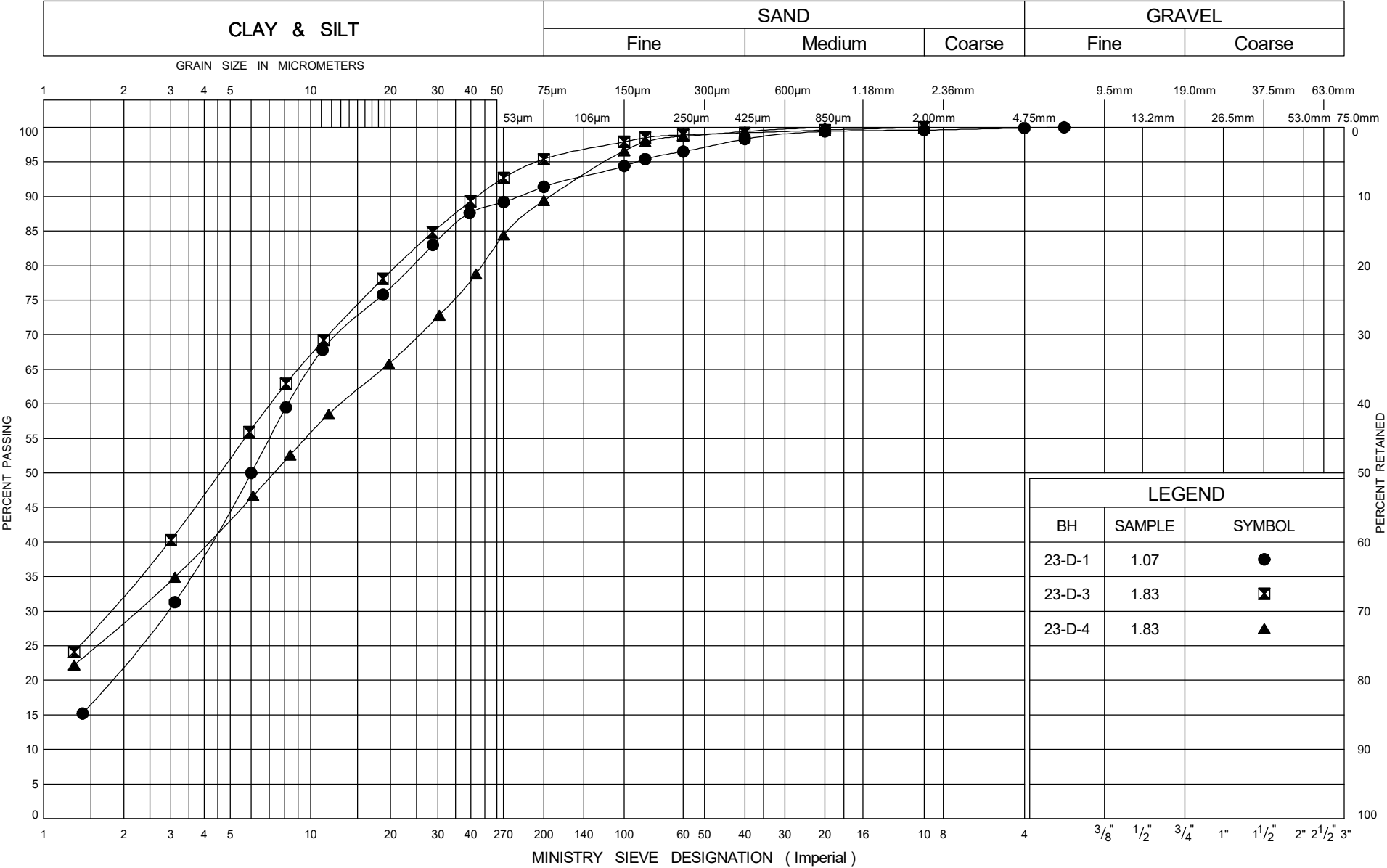
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W <sub>P</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									WATER CONTENT (%)	
																	20	40
281.9																		
0.0	SAND (FILL), brown, moist, loose to compact		SS1	SS	16		281											
			SS2	SS	12													
	- trace organics at ~ 1.4 m depth																	
280.3																		
1.6	CLAYEY SILT, some sand, trace organics, trace oxidization, grey, low plasticity, wet, firm		SS3	SS	6				280									0 11 61 28
279.6																		
2.3	SILTY SAND, trace clay, brown, wet, very loose to compact		SS4	SS	15		279											
					SS5	SS	2											
			SS6	SS	100		278											
	- very dense at ~ 4.0 m depth																	
277.7																		
4.2	BEDROCK, dark grey to grey with quartz veining  Run 1: Start/End: 4.2 to 5.0 m Recovery: 97% RQD: 80%  Run 2: Start/End: 5.0 to 6.4 m Recovery: 100% RQD: 100%		RUN 1	NQ			277											
					RUN 2	NQ												
							276											
275.5																		
6.4	BOREHOLE TERMINATED AT ~ 6.4 m DEPTH  Notes: 1. Groundwater measured in open hole at 2.1 m depth prior to rock coring. 2. Borehole backfilled upon completion.																	

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

ONTARIO MTO DUNCHURCH.PY.GPJ ONTARIO MTO.GDT 2/26/24

## Appendix D - Laboratory Data

UNIFIED SOIL CLASSIFICATION SYSTEM



Ministry of  
Transportation

GRAIN SIZE DISTRIBUTION

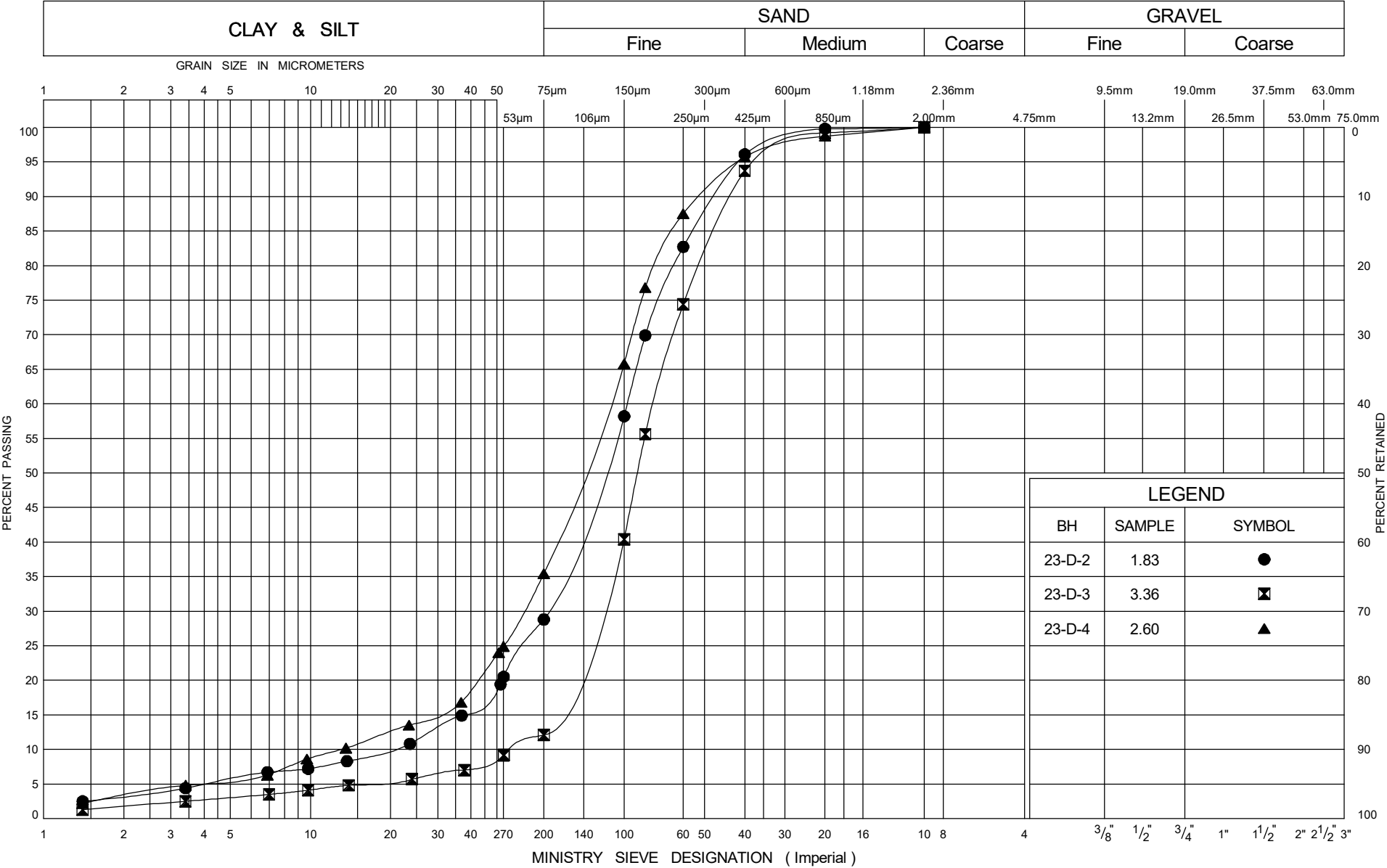
Silty Clay / Clayey Silt

FIG No 1

W P 5021-E-0020 Assign. #10

Dunchurch Patrol Yard

UNIFIED SOIL CLASSIFICATION SYSTEM



Ministry of  
Transportation

GRAIN SIZE DISTRIBUTION

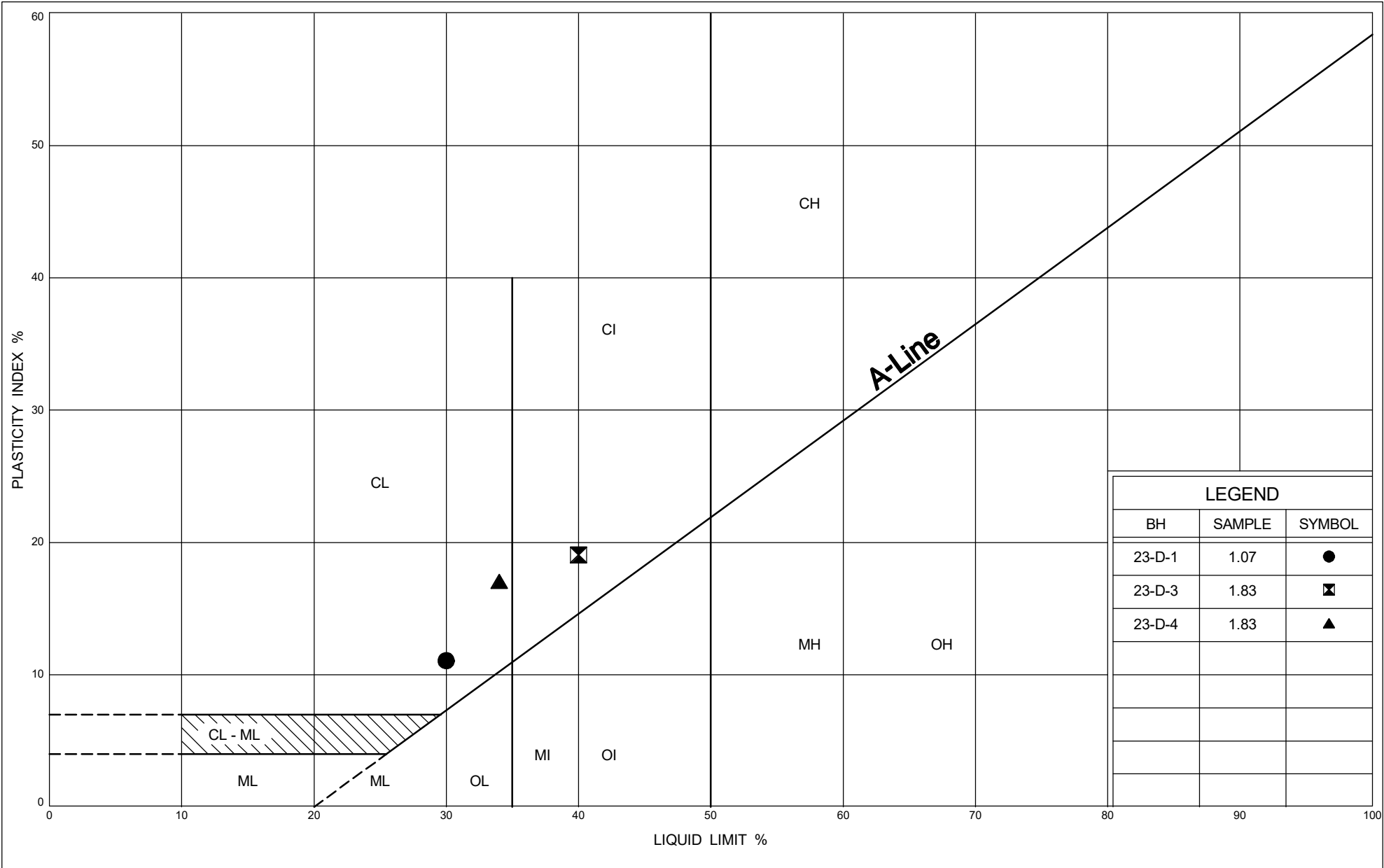
Silty Sand / Sand

FIG No 2

W P 5021-E-0020 Assign. #10

Dunchurch Patrol Yard

ONTARIO MOT PLASTICITY CHART DUNCHURCH.PY.GPJ ONTARIO MOT.GDT 2/16/24





Your Project #: ADM-22006096-A9  
Site Location: DUN CHURCH  
Your C.O.C. #: 137455

**Attention: Silvana Micic**

exp Services Inc  
Brampton Branch  
1595 Clark Blvd  
Brampton, ON  
CANADA L6T 4V1

**Report Date: 2024/01/29**  
Report #: R8007706  
Version: 1 - Final

**CERTIFICATE OF ANALYSIS**

**BUREAU VERITAS JOB #: C421555**

**Received: 2024/01/22, 14:34**

Sample Matrix: Soil  
# Samples Received: 1

Analyses	Quantity	Date	Date	Laboratory Method	Analytical Method
		Extracted	Analyzed		
Chloride (20:1 extract)	1	2024/01/26	2024/01/29	CAM SOP-00463	MOE E3013 m
Conductivity	1	2024/01/26	2024/01/26	CAM SOP-00414	OMOE E3530 v1 m
Moisture (Subcontracted) (1, 2)	1	N/A	2024/01/29	AB SOP-00002	CCME PHC-CWS m
Sulphide in Soil (1)	1	N/A	2024/01/29	AB SOP-00080	EPA9030B/SM4500S2-DF
pH CaCl2 EXTRACT	1	2024/01/25	2024/01/25	CAM SOP-00413	EPA 9045 D m
Redox Potential (3)	1	2024/01/26	2024/01/26	CAM SOP-00421	SM 24 2580 B
Resistivity of Soil	1	2024/01/23	2024/01/26	CAM SOP-00414	SM 24 2510 m
Sulphate (20:1 Extract)	1	2024/01/26	2024/01/29	CAM SOP-00464	MOE E3013 m

**Remarks:**

Bureau Veritas is accredited to ISO/IEC 17025 for specific parameters on scopes of accreditation. Unless otherwise noted, procedures used by Bureau Veritas are based upon recognized Provincial, Federal or US method compendia such as CCME, EPA, APHA or the Quebec Ministry of Environment.

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

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

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

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

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

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

\* RPDs calculated using raw data. The rounding of final results may result in the apparent difference.

(1) This test was performed by Bureau Veritas Calgary (19th), 4000 19th Street NE, Calgary, AB, T2E 6P8

(2) Offsite analysis requires that subcontracted moisture be reported.





Your Project #: ADM-22006096-A9  
Site Location: DUN CHURCH  
Your C.O.C. #: 137455

**Attention: Silvana Micic**

exp Services Inc  
Brampton Branch  
1595 Clark Blvd  
Brampton, ON  
CANADA L6T 4V1

**Report Date: 2024/01/29**  
Report #: R8007706  
Version: 1 - Final

**CERTIFICATE OF ANALYSIS**

**BUREAU VERITAS JOB #: C421555**

**Received: 2024/01/22, 14:34**

(3) Oxidation-Reduction Potential (ORP) values are determined using a Ag/AgCl reference electrode. The test is therefore, not SCC accredited for this matrix.

Encryption Key



**AUTHORIZED REPORT  
RAPPORT AUTORISÉ**

Bureau Veritas

29 Jan 2024 16:47:17

Please direct all questions regarding this Certificate of Analysis to:

Patricia Legette, Project Manager

Email: Patricia.Legette@bureauveritas.com

Phone# (905)817-5799

=====

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

Bureau Veritas has procedures in place to guard against improper use of the electronic signature and have the required "signatories", as per ISO/IEC 17025, signing the reports. For Service Group specific validation, please refer to the Validation Signatures page if included, otherwise available by request. For Department specific Analyst/Supervisor validation names, please refer to the Test Summary section if included, otherwise available by request. This report is authorized by Rodney Major, General Manager responsible for Ontario Environmental laboratory operations.

Total Cover Pages : 2

Page 2 of 9

Bureau Veritas 6740 Campobello Road, Mississauga, Ontario, L5N 2L8 Tel: (905) 817-5700 Toll-Free: 800-563-6266 Fax: (905) 817-5777 www.bvna.com

Microbiology testing is conducted at 6660 Campobello Rd. Chemistry testing is conducted at 6740 Campobello Rd.



### SOIL CORROSIVITY PACKAGE (SOIL)

Bureau Veritas ID		YEL757			YEL757		
Sampling Date		2024/01/18 15:00			2024/01/18 15:00		
COC Number		137455			137455		
	UNITS	BH-D-3, SS4	RDL	QC Batch	BH-D-3, SS4 Lab-Dup	RDL	QC Batch
<b>Calculated Parameters</b>							
Resistivity	ohm-cm	1200		9177506			
<b>CONVENTIONALS</b>							
Redox Potential	mV	220	N/A	9184247			
<b>Inorganics</b>							
Soluble (20:1) Chloride (Cl-)	ug/g	430	20	9184283			
Conductivity	umho/cm	821	2	9184309			
Available (CaCl2) pH	pH	6.73		9182015			
Soluble (20:1) Sulphate (SO4)	ug/g	<20	20	9184287			
Sulphide	mg/kg	<0.5 (1)	0.5	9188887	<0.5	0.5	9188887
RDL = Reportable Detection Limit QC Batch = Quality Control Batch Lab-Dup = Laboratory Initiated Duplicate N/A = Not Applicable (1) Matrix spike exceeds acceptance limits due to matrix interference.							



BUREAU  
VERITAS

Bureau Veritas Job #: C421555

Report Date: 2024/01/29

exp Services Inc

Client Project #: ADM-22006096-A9

Site Location: DUN CHURCH

Sampler Initials: EL

### RESULTS OF ANALYSES OF SOIL

Bureau Veritas ID		YEL757		
Sampling Date		2024/01/18 15:00		
COC Number		137455		
	UNITS	BH-D-3, SS4	RDL	QC Batch
<b>Physical Testing</b>				
Moisture-Subcontracted	%	20	0.30	9188888
RDL = Reportable Detection Limit				
QC Batch = Quality Control Batch				



BUREAU  
VERITAS

Bureau Veritas Job #: C421555

Report Date: 2024/01/29

exp Services Inc

Client Project #: ADM-22006096-A9

Site Location: DUN CHURCH

Sampler Initials: EL

## TEST SUMMARY

**Bureau Veritas ID:** YEL757  
**Sample ID:** BH-D-3, SS4  
**Matrix:** Soil

**Collected:** 2024/01/18  
**Shipped:**  
**Received:** 2024/01/22

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	SKAL/EC	9184283	2024/01/26	2024/01/29	Massarat Jan
Conductivity	AT	9184309	2024/01/26	2024/01/26	Leily Karimi
Moisture (Subcontracted)	BAL	9188888	N/A	2024/01/29	Joyce Loan Phan
Sulphide in Soil	SPEC	9188887	N/A	2024/01/29	Bailey Morrison
pH CaCl2 EXTRACT	AT	9182015	2024/01/25	2024/01/25	Gurparteek KAUR
Redox Potential	COND	9184247	2024/01/26	2024/01/26	Gurparteek KAUR
Resistivity of Soil		9177506	2024/01/26	2024/01/26	Automated Statchk
Sulphate (20:1 Extract)	SKAL/EC	9184287	2024/01/26	2024/01/29	Massarat Jan

**Bureau Veritas ID:** YEL757 Dup  
**Sample ID:** BH-D-3, SS4  
**Matrix:** Soil

**Collected:** 2024/01/18  
**Shipped:**  
**Received:** 2024/01/22

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Sulphide in Soil	SPEC	9188887	N/A	2024/01/29	Bailey Morrison



BUREAU  
VERITAS

Bureau Veritas Job #: C421555

Report Date: 2024/01/29

exp Services Inc

Client Project #: ADM-22006096-A9

Site Location: DUN CHURCH

Sampler Initials: EL

### GENERAL COMMENTS

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

Package 1	9.0°C
-----------	-------

Results relate only to the items tested.



QUALITY ASSURANCE REPORT

exp Services Inc  
Client Project #: ADM-22006096-A9  
Site Location: DUN CHURCH  
Sampler Initials: EL

QC Batch	Parameter	Date	Matrix Spike		SPIKED BLANK		Method Blank		RPD	
			% Recovery	QC Limits	% Recovery	QC Limits	Value	UNITS	Value (%)	QC Limits
9182015	Available (CaCl2) pH	2024/01/25			100	97 - 103			1.1	N/A
9184247	Redox Potential	2024/01/26			100	95 - 105			5.9	35
9184283	Soluble (20:1) Chloride (Cl-)	2024/01/29	96	80 - 120	102	80 - 120	<20	ug/g	1.8	35
9184287	Soluble (20:1) Sulphate (SO4)	2024/01/29	NC	70 - 130	97	70 - 130	<20	ug/g	0.44	35
9184309	Conductivity	2024/01/26			103	90 - 110	<2	umho/cm	6.1	10
9188887	Sulphide	2024/01/29	41 (1)	75 - 125	101	75 - 125	<0.5	mg/kg	NC	30
9188888	Moisture-Subcontracted	2024/01/29					<0.30	%		

N/A = Not Applicable

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

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

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

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

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

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

(1) Recovery or RPD for this parameter is outside control limits. The overall quality control for this analysis meets acceptability criteria.



BUREAU  
VERITAS

Bureau Veritas Job #: C421555  
Report Date: 2024/01/29

exp Services Inc  
Client Project #: ADM-22006096-A9  
Site Location: DUN CHURCH  
Sampler Initials: EL

### VALIDATION SIGNATURE PAGE

The analytical data and all QC contained in this report were reviewed and validated by:

Anastassia Hamanov, Scientific Specialist

Veronica Falk, B.Sc., P.Chem., QP, Scientific Specialist, Organics

Suwan (Sze Yeung) Fock, B.Sc., Scientific Specialist

---

Bureau Veritas has procedures in place to guard against improper use of the electronic signature and have the required "signatories", as per ISO/IEC 17025, signing the reports. For Service Group specific validation, please refer to the Validation Signatures page if included, otherwise available by request. For Department specific Analyst/Supervisor validation names, please refer to the Test Summary section if included, otherwise available by request. This report is authorized by Rodney Major, General Manager responsible for Ontario Environmental laboratory operations.





# REC'D IN LONDON

## CHAIN OF CUSTODY RECORD

137455

Page \_\_\_\_\_ of \_\_\_\_\_



NCNT-2024-01

NOT-2014-01-1188

Unless otherwise agreed to in writing, work submitted on this Clinch Custody is subject to Eurovet Verkeas Laboratories' standard Terms and Conditions. Signing of this Clinch of Custody document is acknowledgement with acceptance of our Terms available at: <http://www.eurovet.com/terms-and-conditions>

COC-1004 (06/15)

724934

White: Mexican - Yellow: Cile

727934 White: Maxxam - Yellow Client





**exp Services Inc.**  
1595 Clark Boulevard  
Brampton, Ontario, L6T 4V1  
Tel: (905) 793-9800  
Fax: (905) 793-0641  
[www.exp.com](http://www.exp.com)

## Rock Core Test UCS

Project No.: ADM-22006096-A9

Project Name: Dunchurch

### Uniaxial Compressive Strength of Intact Rock Core Specimens

Core No.	1				
Depth (m)	BH-D-2 2.7				
Date Sampled	January 22 <sup>nd</sup> , 2024				
Date Received	February 20 <sup>th</sup> , 2024				
Date Tested	February 21 <sup>st</sup> , 2024				
Lithology	N/A				
Length - (mm)	104.0				
Average Diameter - (mm)	63.0				
L/D Ratio	2.21				
Rate of Loading (MPa/Sec)	0.30				
Uniaxial Compressive Strength - (MPa)	66.4				
Moisture Condition at Time Of Test	Dry				
Remarks	Tested by R. Chavez				

Tested in accordance with ASTM D7012, unless otherwise indicated.

  
\_\_\_\_\_  
Testing Laboratory Representative Signature  
Eric Jageshur - Concrete Supervisor

February 21<sup>st</sup>, 2024  
\_\_\_\_\_  
Date

## Appendix E – Bedrock Core Photographs



Figure E1. Bedrock core samples, BH23-D-1, Run 1 (top), Run 2 (middle), January 19, 2024



Figure E2. Bedrock core samples, BH23-D-2, Run 1 (top), Run 2 (middle), January 18, 2024



Figure E3. Bedrock core samples, BH23-D-3, Run 1 (top), Run 2 (middle), January 18, 2024



Figure E4. Bedrock core samples, BH23-D-4, Run 1 (top), Run 2 (middle), January 18, 2024

## Appendix F – Results of Stability Analyses

5021-E-0020 Northeastern Region  
 Work Order # 10 - New Material Storage Structure at Dunchurch Patrol Yard  
 Undrained Static Analysis - North-South Section - Side Wall

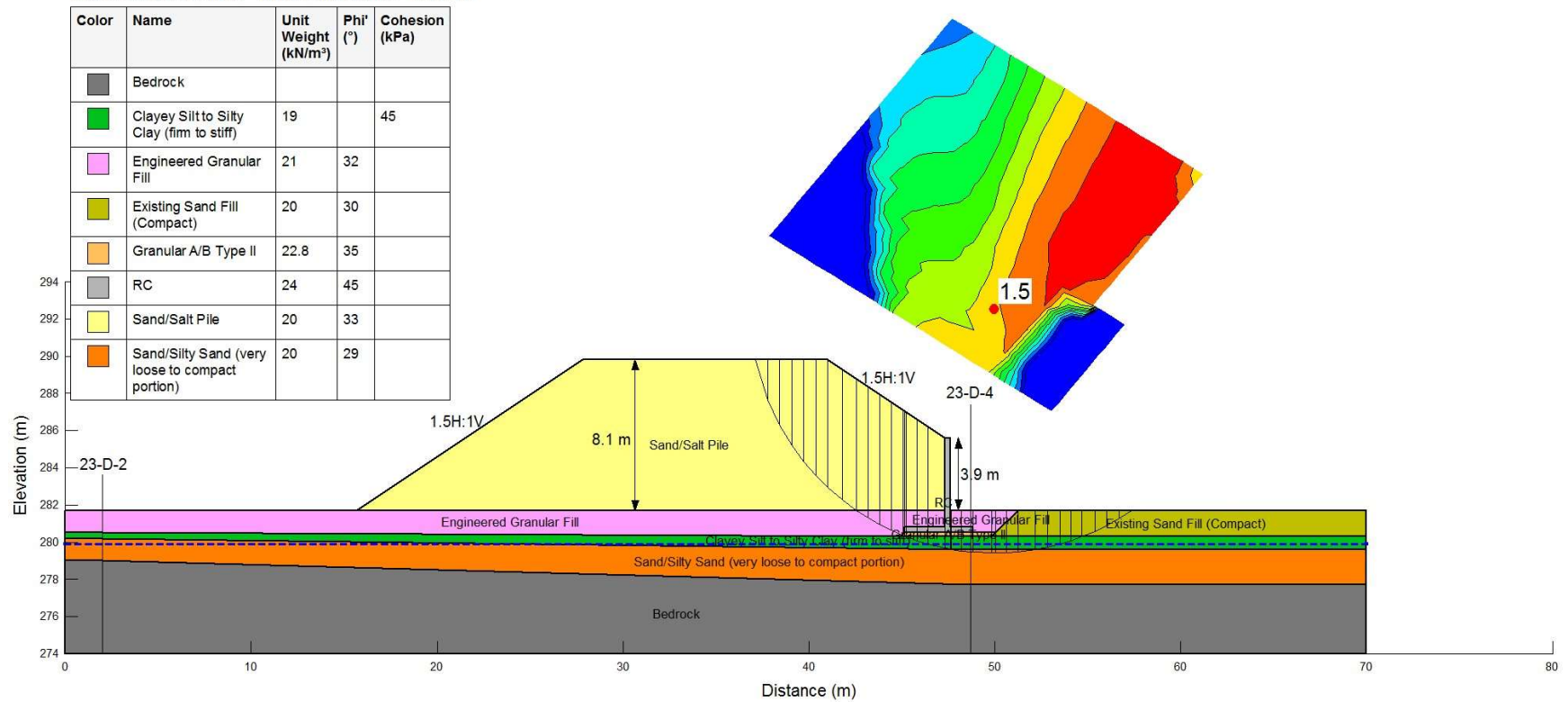


Figure F1: Slope stability analysis of new material storage building – N-S section, north side wall – undrained static analysis



5021-E-0020 Northeastern Region  
 Work Order # 10 - New Material Storage Structure at Dunchurch Patrol Yard  
 Drained Static Analysis - North-South Section - Side Wall

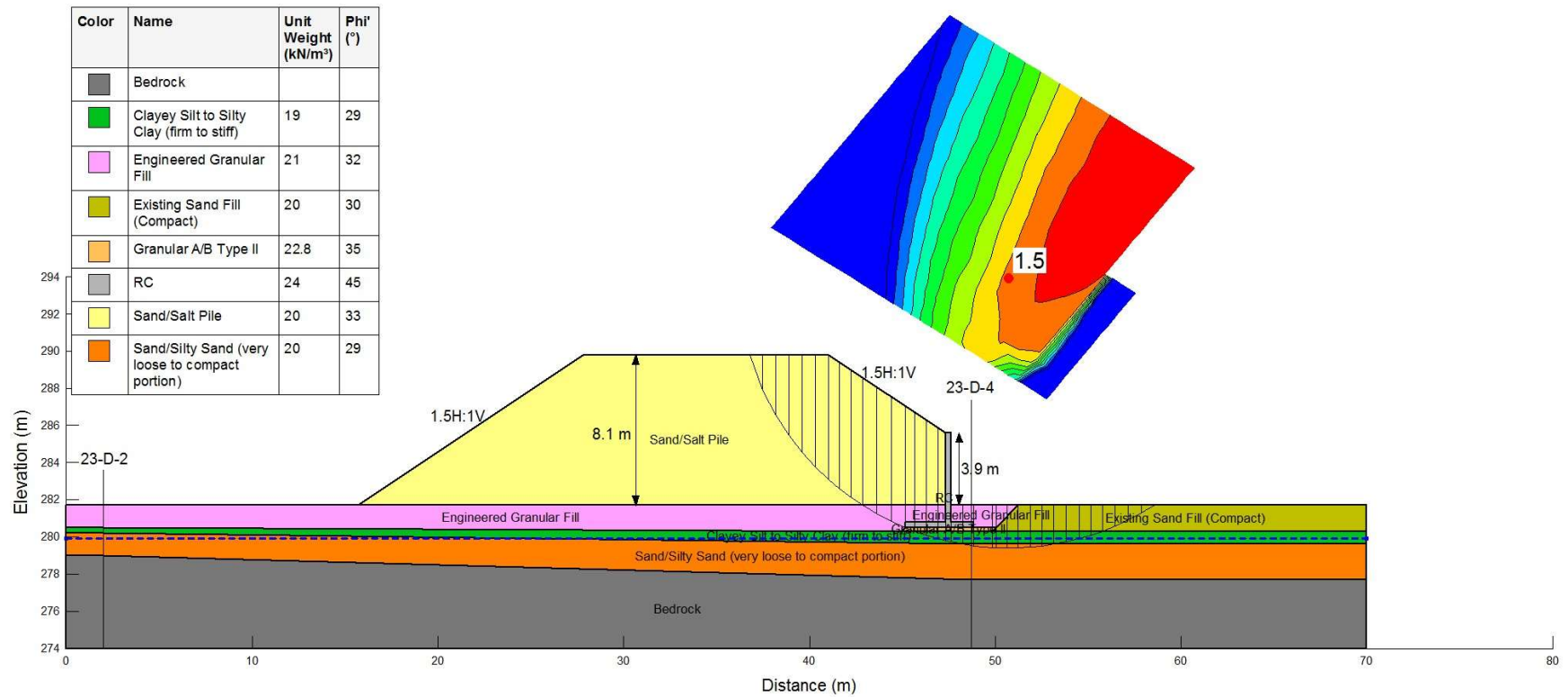


Figure F2: Slope stability analysis of new material storage building – N-S section, north side wall – drained static analysis

5021-E-0020 Northeastern Region  
 Work Order # 10 - New Material Storage Structure at Dunchurch Patrol Yard  
 Drained Seismic Analysis - North-South Section - Side Wall

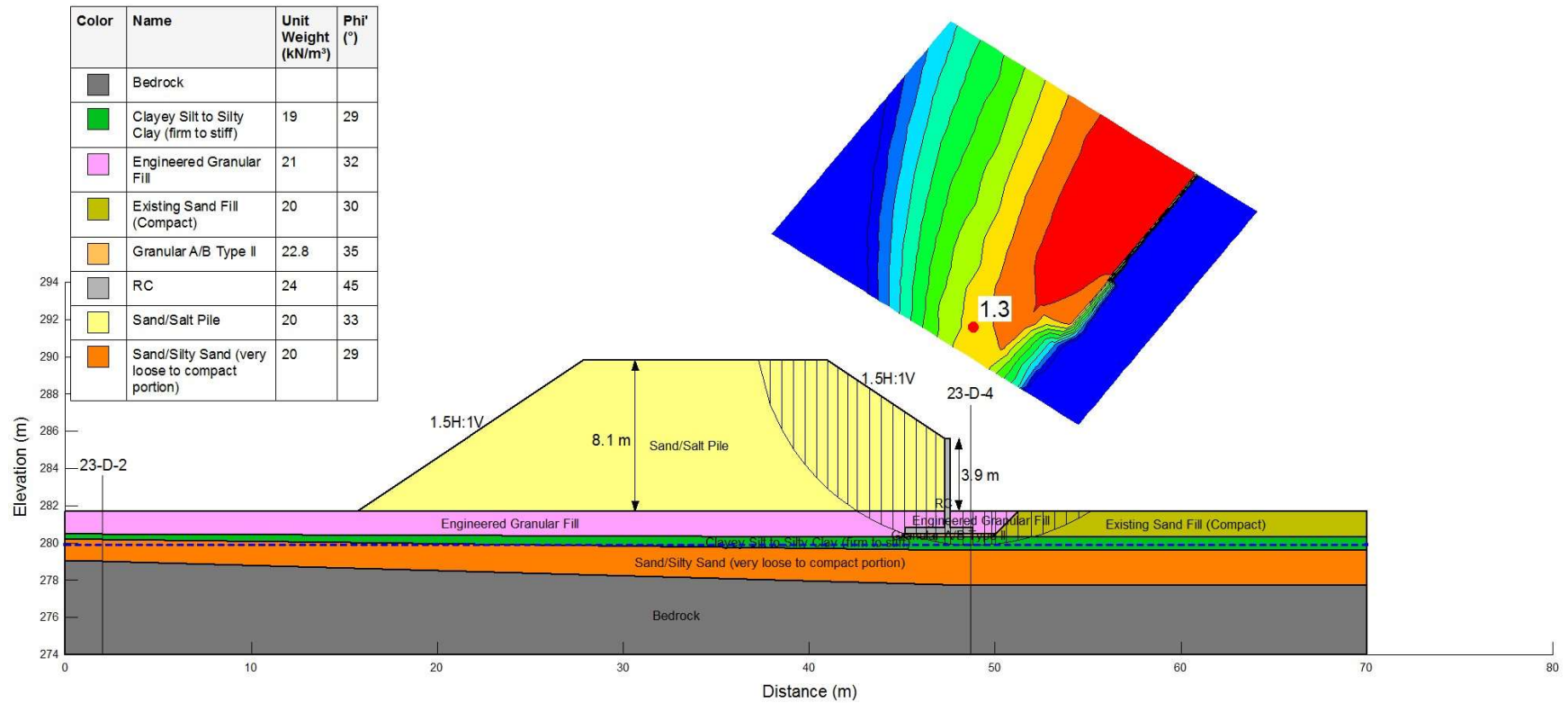


Figure F3: Slope stability analysis of new material storage building – N-S section, north side wall – drained seismic analysis

5021-E-0020 Northeastern Region  
 Work Order # 10 - New Material Storage Structure at Dunchurch Patrol Yard  
 Undrained Static Analysis - East-West Section - Side Wall

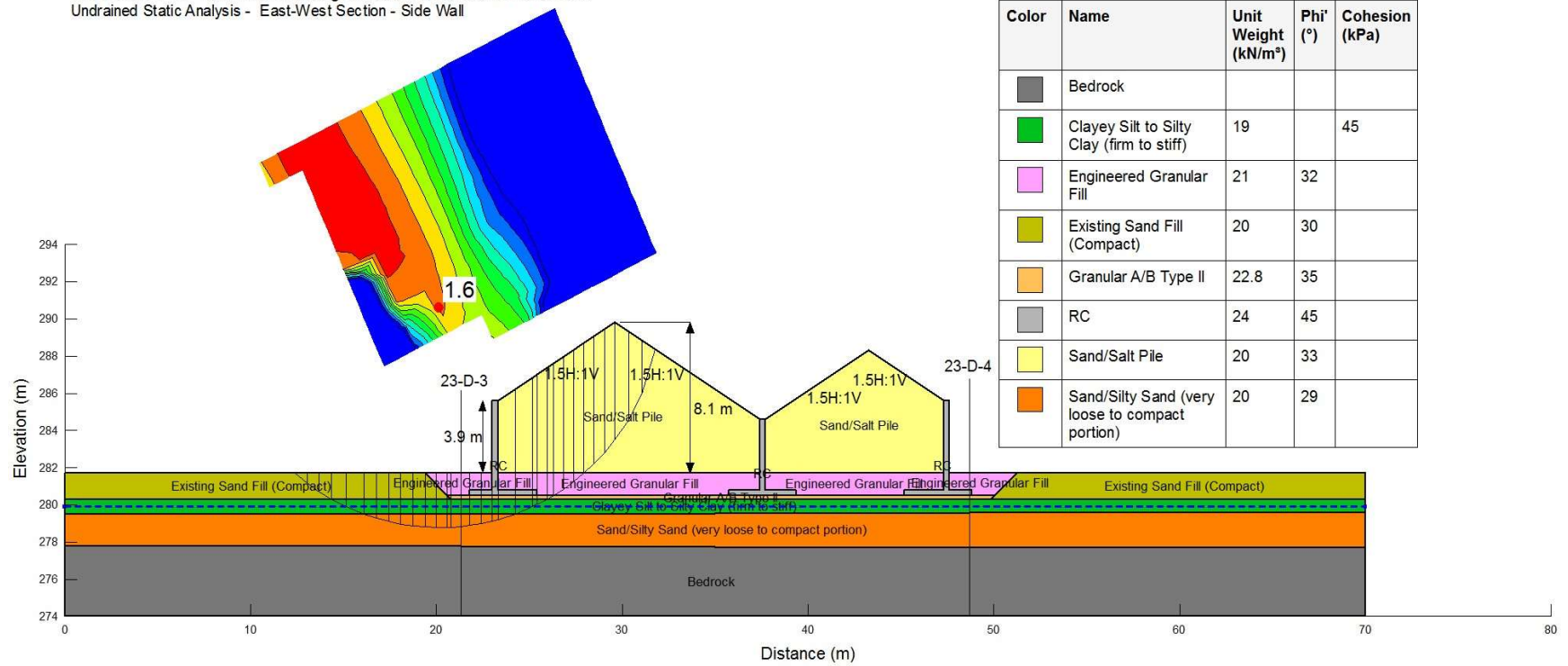


Figure F4: Slope stability analysis of new material storage building – E-W section, west side wall – undrained static analysis



5021-E-0020 Northeastern Region  
 Work Order # 10 - New Material Storage Structure at Dunchurch Patrol Yard  
 Drained Static Analysis - East-West Section - Side Wall

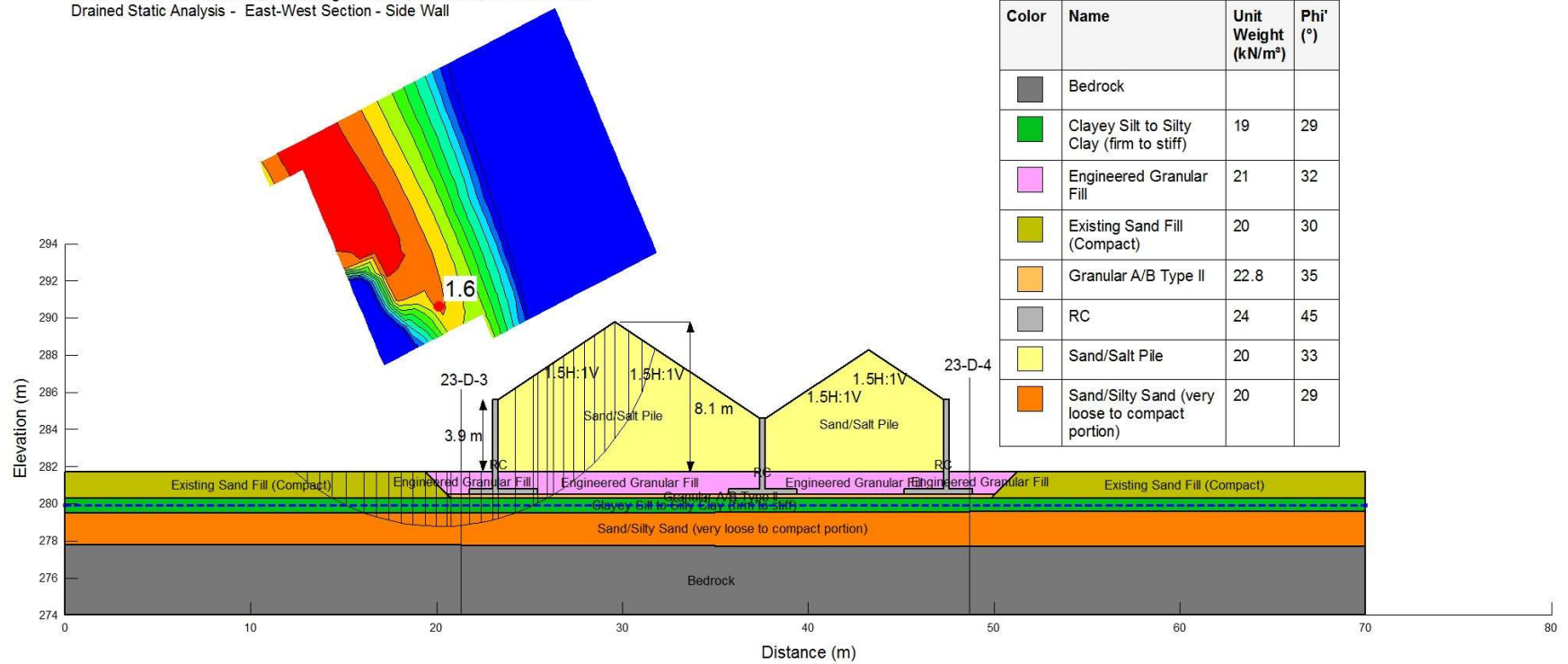


Figure F5: Slope stability analysis of new material storage building – E-W section, west side wall – drained static analysis

5021-E-0020 Northeastern Region  
 Work Order # 10 - New Material Storage Structure at Dunchurch Patrol Yard  
 Drained Seismic Analysis - East-West Section - Side Wall

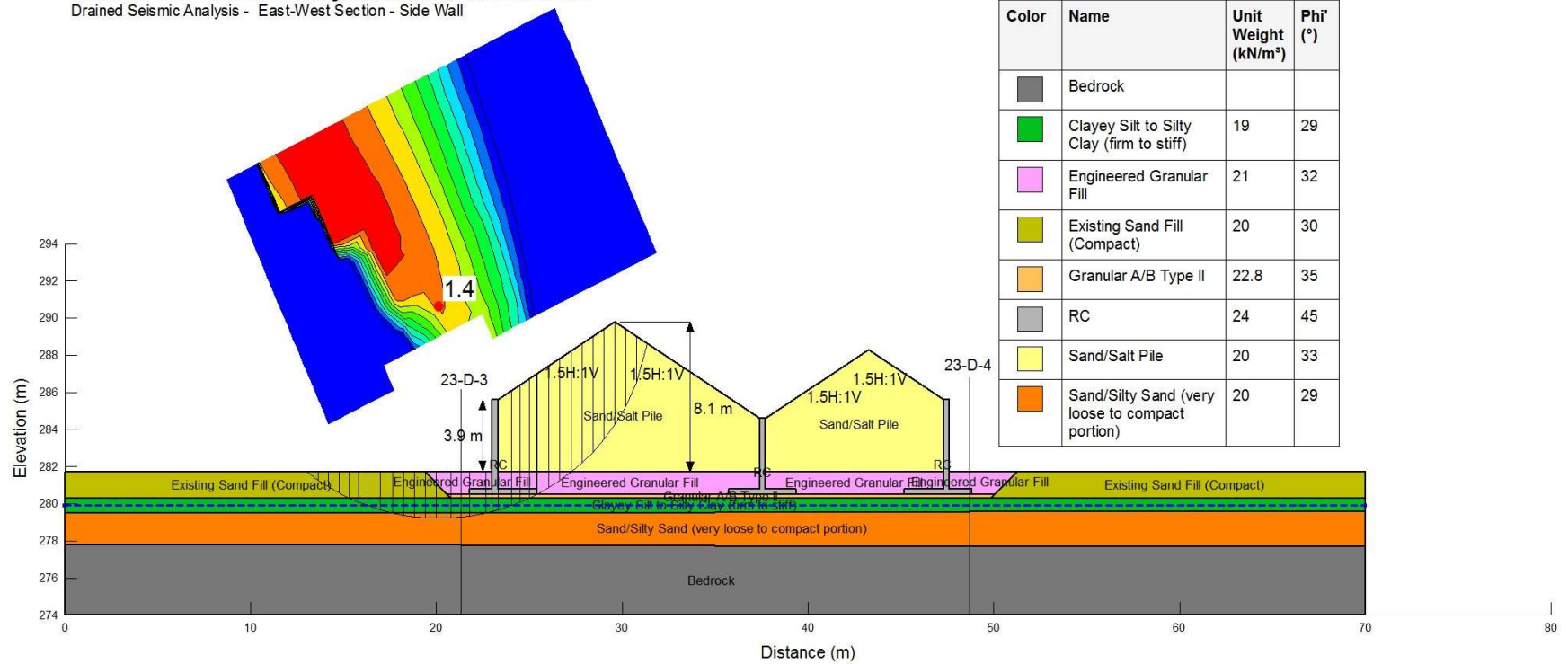
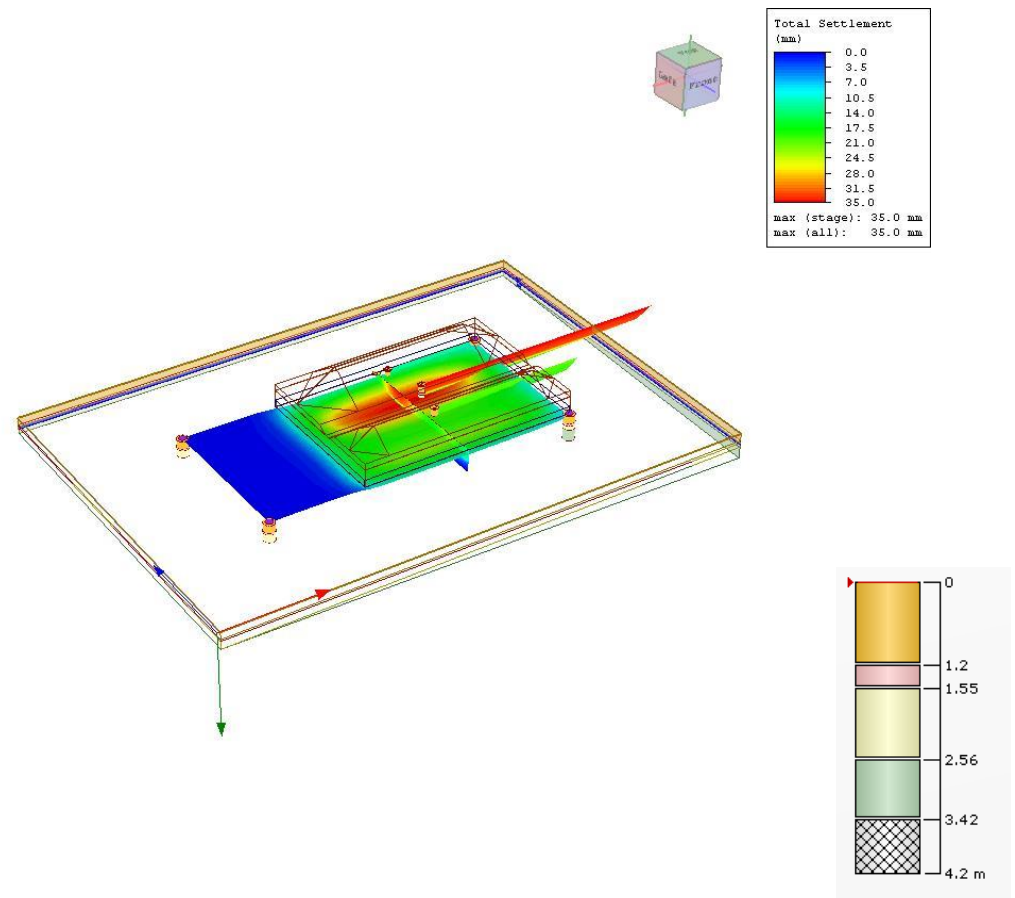
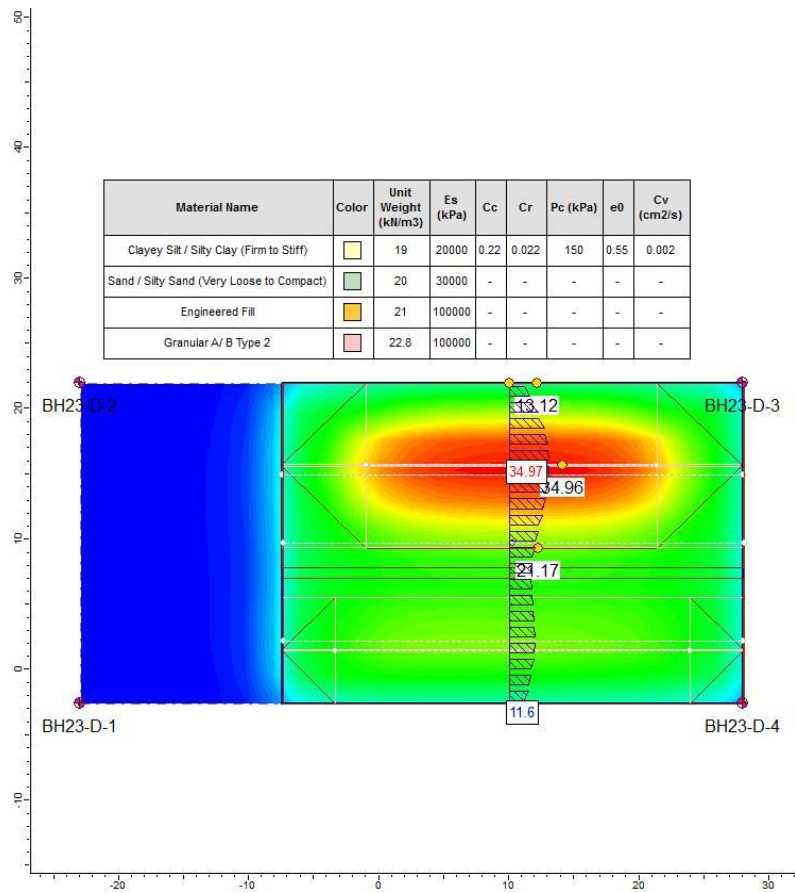


Figure F6: Slope stability analysis of new material storage building – E-W section, west side wall – drained seismic analysis

## Appendix G – Results of Settlement Analyses



*Project:* Dunchurch Patrol Yard

*Analysis Description:* Full loading at Salt/Sand Storage Area – **Total Settlement**

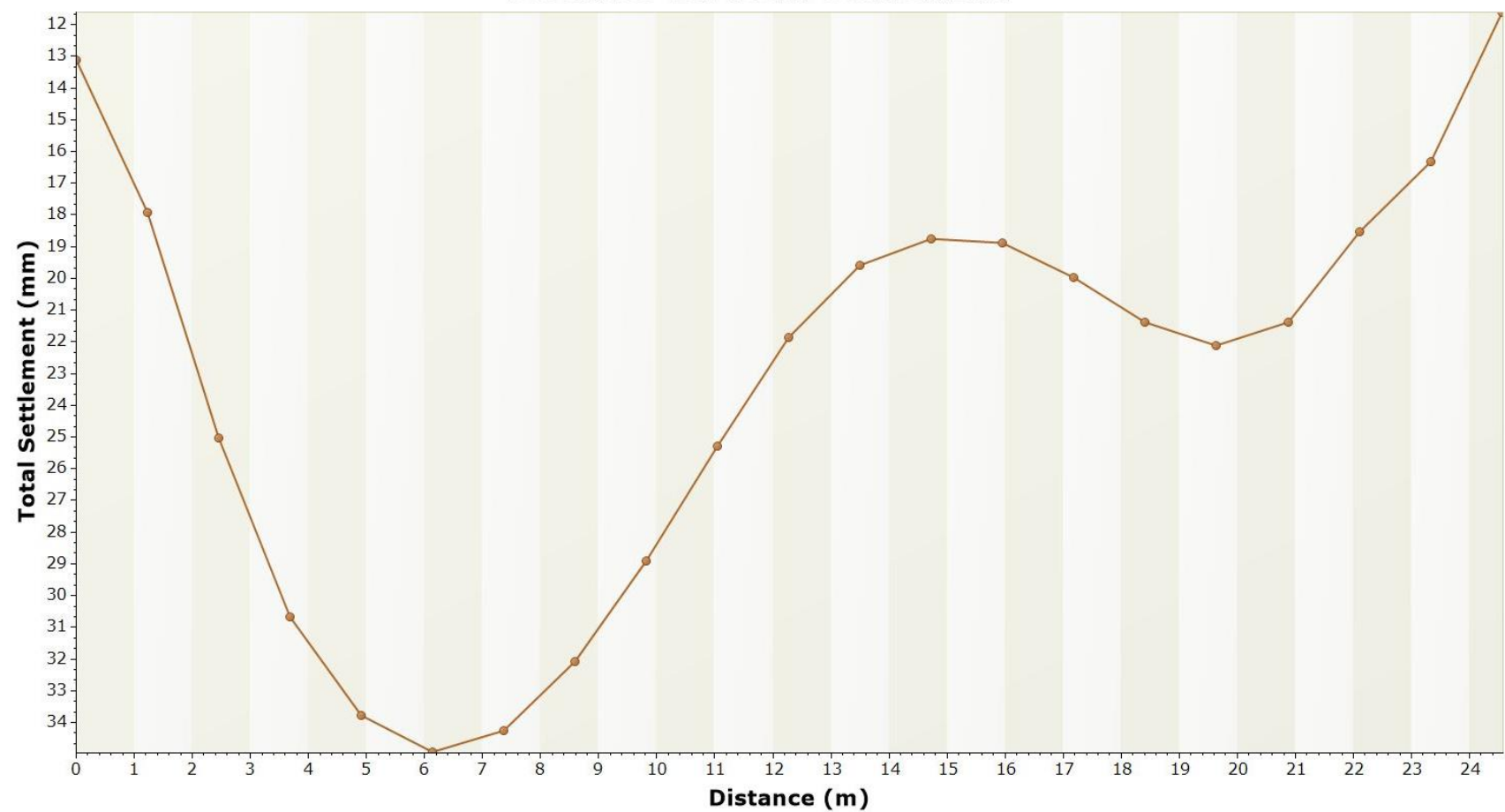
*Figure No:* G1


*Company:* EXP Services Inc.

*Date:* March, 2024

*File Name:* Settlement Analysis – Assignment 10

Distance vs. Total Settlement



	Project: Dunchurch Patrol Yard	
	Analysis Description: Full loading at Salt/Sand Storage Area – <b>Total Settlement</b>	
	Figure No: G2	Company: EXP Services Inc.
	Date: March, 2024	File Name: Settlement Analysis – Assignment 10

## Appendix H – Seismic Hazard Calculation



Government  
of Canada

Gouvernement  
du Canada

[Canada.ca](https://Canada.ca) › [Natural Resources Canada](#) › [Earthquakes Canada](#)

# 2020 National Building Code of Canada Seismic Hazard Tool

---

**i** 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 $X_s$	$X_C$
Latitude (°)	45.658
Longitude (°)	-79.811

**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 ( $g$ ,  $9.81 \text{ m/s}^2$ ). 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**

$S_a(0.2, X_C)$	$S_a(0.5, X_C)$	$S_a(1.0, X_C)$	$S_a(2.0, X_C)$	$S_a(5.0, X_C)$	$S_a(10.0, X_C)$	PGA( $X_C$ )	PGV( $X_C$ )
0.25	0.173	0.0976	0.0471	0.0127	0.0044	0.113	0.111

The log-log interpolated 2%/50 year  $S_a(4.0, X_C)$  value is : **0.0175**

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

Download CSV

← Go back to the [seismic hazard calculator form](#)

**Date modified:** 2021-04-06