



## Foundation Investigation and Design Report

Highway 401 Eastbound Express and Collector Lanes between Victoria Park Avenue and Neilson Road - **Superstructure Replacement and High Fill for Embankment Widening at Birchmount Overpass Eastbound Core and Collectors Structure**  
(Site 37X-0212/B1 & B3)

Assignment No. 2021-E-0018  
MTO Central Region  
Geocres Number: 30M14-549  
(Latitude: 43.772431, Longitude: -79.294383)

### Type of Document:

Foundation Investigation and Design Report

### Project Number:

ADM-22000797-A0

### Prepared For:

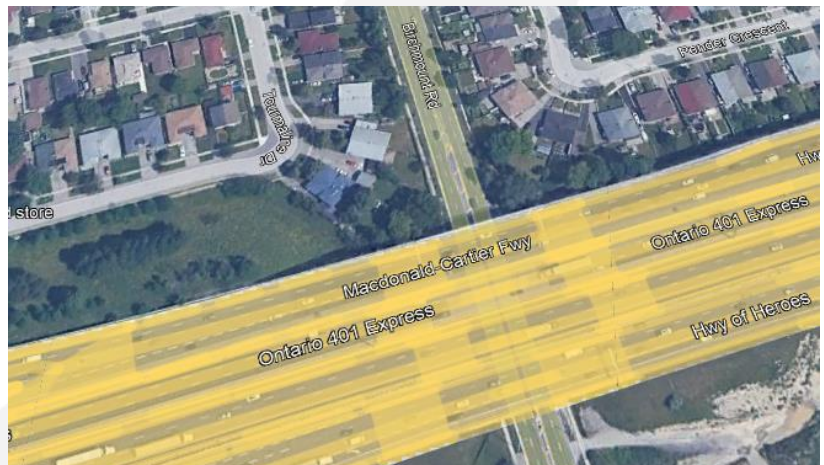
AECOM Canada Ltd.  
5090 Explorer Drive  
Mississauga, Ontario, L4W 4X6  
Attn: Taesang Ahn

### Prepared By:

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

### Date Submitted:

December 31, 2024



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## Issue and Revised Record

Rev.	Date	Format	Prepared by	Reviewed by	Approved by	Description
<b>Rev. 0</b>	September 19, 2024	pdf	E. Lu D. Mroz	T. Lardner T.C. Kim	S. Gonsalves	Report
<b>Rev. 1</b>	December 31, 2024	pdf	A.Medghalchi	T. Lardner T.C. Kim	S. Gonsalves	Report

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## Part I: Foundation Investigation Report

Highway 401 Eastbound Express and Collector Lanes between Victoria Park Avenue and Neilson Road – Birchmount Road Overpass (Site 37X-0212/B1 & B3)

## 1.0 Introduction

EXP Services Inc. (EXP) was retained by AECOM on behalf of The Ministry of Transportation (MTO) to provide detailed foundation investigation and engineering services for the proposed Highway 401 Eastbound rehabilitation and construction project. The findings, analyses and recommendations are presented in a Foundation Investigation Design Report created for each structure along the proposed highway. The work was undertaken under Assignment No. 2021-E-0018. The terms of reference (TOR) and the scope of work for the foundation investigation are outlined in Ministry of Transportation Ontario's (MTO) Request for proposal, dated June 2021. The scope of this report is specifically limited to the proposed location of the Birchmount Road Overpass structure (Site 37X-0218/B1 & B3).

The General Arrangement drawings (GA) for the bridge structure were provided to EXP by AECOM. The purpose of the investigation was to evaluate the subsurface conditions along the structure alignment to permit a detailed design for the proposed structure widening, superstructure replacement, retaining wall replacement, and high fill embankment widening associated with the bridge widening.

The site-specific geotechnical investigation consisted of borings, soil sampling, borehole logging, and field and laboratory testing. The field and laboratory work for this structure was performed by EXP. Based on collected geotechnical data, this report provides an assessment of the geotechnical issues, geotechnical design parameters, and geotechnical foundation design recommendations for the proposed structure. Geotechnical-related construction recommendations are also provided.

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.

## 2.0 Structure Description

The GA drawing titled *HWY 401 EB CORE & COLLECTOR VICTORIA PARK TO NEILSON BIRCHMONT RD. OVERPASS GENERAL ARRANGEMENT*, prepared by AECOM, dated SEP. 2022, shows the preliminarily proposed configuration of the Birchmount Road Overpass structure. A summary of the proposed structure is as follows:

1. The existing structure is a 29.72 m long two-span bridge with equal spans between the abutments (14.86 m). It is understood that the existing abutments, piers and retaining wall foundations are supported on spread footings. The existing abutments are supported on approximately 5.6 m and 5.3 m wide spread footings at the express lanes and collectors lanes, respectively. Based on the Foundation and Investigation Design Report *"Bridge Widening and Replacement Highway 401 Rehabilitation from Warden Avenue to Brock Road, Toronto, Ontario, W.O.07-20012."* produced by Golder Associates Ltd., dated April 2012, the west abutment is founded at about Elevation 173.6 m and the east abutment is founded at about Elevation 173.8 m to 174.2 m from the north side to the south side of the bridge. Both abutments were also constructed with a shear key. The center piers are founded on a 1.8 m wide footing founded at about Elevation 174.0 m.
2. The existing structure is proposed to undergo superstructure replacement, which includes replacement of the existing bridge deck and girders, and conversion to semi-integral abutments. Additionally, the bridge will be widened by 4.5 m with a new pier column and cap to attach to the existing structure.
3. The existing retaining wall structures (37x-1765/W and 37x-1766/W) will be replaced with new retaining walls along the south side of widened collector structure.
4. High fills for embankment widening and retaining walls are proposed to accommodate the additional lane of widening that will occur from about 150 m west of Birchmount Road to the CP Rail Overpass structure (located about 200 m east of Birchmount Road).

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The preliminary foundation design report and GA drawing by URS, the contract package drawings titled *Hwy 401 WB Core & Collector Lanes – Birchmount Road Overpass – Bridge Rehabilitation (Cont. No. 2019-2011, WP No. 2403/2404-15-01)*, produced by WSP Global Inc., dated July 2018, and the Foundation and Investigation Design Report (FIDR) by Golder Associates Ltd., *“Birchmount Road Overpass Rehabilitation (Site No. 37-212) Highway 401 Westbound Core and Collector Lanes, Neilson Road to Warden Avenue, City of Toronto, Ontario, MTO G.W.P. 2162-11-00”*, dated January 16, 2019 were included as part of this report is used for initial context to address the nature and scope of the investigation. It is understood that some changes might occur as a result of normal refinement or the findings of the geotechnical report.

## 3.0 Site Description and Geological Setting

### 3.1 Site Description

The site is located at the intersection of Highway 401 and Birchmount Road, approximately 3 km east of Highway 404 in the City of Toronto, Ontario. The site is adjacent to industrial zones to the south and northeast, and adjacent to residential zones to the northwest of the site. In general, the terrain in this area is relatively flat, with the natural ground surface sloping gently towards south. The Highway 401 pavement grade ranges between about Elevation 185 m to 186 m while, the Birchmount Road pavement grade is at approximate Elevation about 176 m to 179.5 m at the structure site (increasing in elevation towards the south). Based on the preliminary GA drawings by AECOM of the Eastbound Core and Collectors and by WSP of the Westbound Core and Collectors, in addition to the Foundation Investigation Design Report by Golder Associates Ltd., the fill thickness is assumed to be 9 m to 10 m.

A site location plan is presented as Drawing 1 in Appendix C.

### 3.2 Geological Setting

Based on a review of geological maps of Southern Ontario (Chapman and Putnam, 1984; 2007), the site is situated within the South Slope physiographic region where the predominate landforms are Till Plains (Drumlinized) and Drumlins. The South Slope represents the southern slope of the Oak Ridges Moraine but also includes a strip south of the Peel Plain, extending from the Niagara Escarpment to the Trent River. The South Slope gradually, fairly and uniformly slopes down towards Lake Ontario.

According to the Ministry of Northern Development and Mines, Map 2556 (Quaternary Geology of Ontario, Southern Sheet, 1991) the surface conditions in the vicinity of the project area consists of Halton Till predominately silt to silty clay matrix, high in matrix carbonate content and clast poor with occasional sand to silt zones. In addition, Map 2544 (Bedrock Geology of Ontario, Southern Sheet, 1991), the bedrock geology at the site consists of shale, limestone, dolostone and siltstone: Georgian Bay Formation, Blue Mountain Formation, Bilings Formation, Collingwood Member, Eastview Member.

## 4.0 Previous Geotechnical Investigation

During the tender design for the project, three (3) previous reports were issued which contain relevant information to the proposed Birchmount Road Overpass structure (Site 37X-0212/B1 & B3), as follows:

1. Foundation Investigation Report for Proposed Extension of the Existing Bridge at Hwy. #401 and Birchmount Road, County of York, Township of Scarborough, District #6 (Toronto), W.J. 65-F-49, W.P. 256-61, Geocres No. 30M14-073, The Ministry of Transportation Ontario (MTO), Foundation Section, Materials and Testing Div., dated August 03, 1965.
2. Geocres No. 30M14-338 *“Bridge Widening and Replacement Highway 401 Rehabilitation from Warden Avenue to Brock Road, Toronto, Ontario, W.O.07-20012.”* by Golder Associates Ltd., dated April 2012.

3. Geocres No. 30M14-492 "Birchmount Road Overpass Rehabilitation (Site No. 37-212) Highway 401 Westbound Core and Collector Lanes, Neilson Road to Warden Avenue, City of Toronto, Ontario, MTO G.W.P. 2162-11-00", by Golder Associates Ltd., dated January 16, 2019.

The applicable previous MTO borehole logs are attached as Appendix F in this report. The details of the applicable boreholes completed by the MTO are also outlined in Table 1.1.

**Table 1.1: Summary of Applicable Borehole Completed by MTO**

Borehole No.	Borehole Location	Location (MTM NAD83 Zone 10)		Latitude	Longitude	Borehole Elevation (m)	Borehole Depth (m)
		Northing	Easting				
<b>73-1</b>	East Abutment, South Side (EBL Collector)	4848065.7	321384.5	43.772366	-79.293992	176.5	11.1
<b>73-2</b>	West Abutment, South Side (EBL Collector)	4848054.1	321349.6	43.772262	-79.294426	175.9	12.6
<b>73-3</b>	Centre Pier, South Side (EBL Collector)	4848072.9	321353.0	43.772431	-79.294383	176.8	12.6

## 5.0 Field Investigation and Laboratory Analyses

### 5.1 Site Investigation and Field Testing

A site-specific investigation was undertaken by EXP between November 7, 2022, and December 12, 2022, and it included the following:

1. A walkover site assessment was carried out by a Geotechnical Engineer from EXP;
2. Subsequent to the borehole layouts in the field, existing utilities were cleared by public utility companies;
3. At the time of this report, seven (7) boreholes have been completed for this structure (BH22-1-01, BH22-1-02, BH22-1-03 and BH22-1-08 to BH22-1-11) as part of EXP's investigation. A summary of boreholes completed by EXP are listed in Table 1.2 below. The boreholes were drilled using a truck-mounted CME-75 or a MARL M10 machine (owned and operated by Drilltech drilling Ltd.) equipped with solid and hollow stem augers, mud rotary equipment, and fitted with capability for Standard Penetration Testing (SPT);
4. Boreholes were set back at least 10 m from the abutment to avoid drilling through the reinforced approach slab.
5. Soil samples in the boreholes were taken at frequent intervals of depth by the Standard Penetration Test method (SPT), in general accordance with ASTM D1586. The test consists of freely dropping a 63.5 kg hammer a vertical distance of 0.76 m to drive a 51 mm O.D. split barrel (SS-split-spoon) sampler into the ground. The number of blows of the hammer required to drive the sampler into the relatively undisturbed ground by a vertical distance of 0.30 m is recorded as the Standard Penetration Resistance, or the N-value, of the soil which is indicative of the compactness of granular (or cohesionless) soils (gravels, sands and silts) or the consistency of cohesive soils (clays and clayey soils);
6. The fieldwork was supervised by a member of EXP's engineering staff who directed the drilling and sampling operation, logged borehole data in accordance with MTO and/or ASTM Standards for Soils Classification, and retrieved soil samples for subsequent laboratory testing and identification;

7. All spoon samples obtained in the Standard Penetration Tests (SPT, ASTM D-1586) were placed in moisture proof bags after field classification. Samples were allocated from the spoon samples for moisture content testing without delay. They were subsequently re-examined under controlled laboratory conditions prior to assigning other laboratory tests;
8. Selected soil samples for corrosivity testing were sent to the Bureau Veritas Laboratories (formerly Maxxam Analytics), a CALA-certified and accredited laboratory in Mississauga, Ontario. The selected soil samples for the analytical testing were placed in a laboratory prepared glass jar, labelled, and stored in a secure cooler.
9. The borehole locations and their ground surface elevations were surveyed by EXP using a Trimble DA2 GNSS receiver with Trimble Catalyst GNSS positioning, having an accuracy of  $\pm 0.10$  m horizontal and vertical directions. MTM NAD83 Zone 10 coordinates and the geodetic elevation for the boreholes are listed in Table 1.2 below. It can also be found on the Record of Borehole Sheet (Appendix D); and
10. Upon completion of drilling and field testing, the boreholes were backfilled with a mixture of bentonite and auger cuttings. The borehole decommissioning was in general accordance with the Ministry of the Environment Regulation 903, as amended by Regulation 128/03 (the well regulation under the Ontario Water Resources Act).

**Table 1.2: Summary of boreholes completed by EXP**

Borehole No.	Borehole Location	Location (MTM NAD83 Zone 10)		Latitude	Longitude	Borehole Elevation (m)	Borehole Depth (m)
		Northing	Easting				
BH22-1-01	Centre Pier, south of O/P	4848043.0	321370.0	43.772165	-79.294168	180.0	9.8
BH22-1-02	West of West Abutment, EBL Collectors	4848051.9	321336.0	43.772243	-79.294595	185.7	15.7
BH22-1-03	East of East Abutment, EBL Collectors	4848071.2	321390.0	43.772415	-79.293924	185.9	15.8
BH22-1-08	West of West Abutment, b/w EBL and WBL Express	4848084.4	321324.7	43.772535	-79.294734	186.0	20.4
BH22-1-09	East of East Abutment, b/w EBL and WBL Express	4848102.3	321380.1	43.772695	-79.294045	186.3	20.4
BH22-1-10	West of West Abutment, b/w EBL and WBL Express	4848077.9	321305.8	43.772477	-79.294969	186.0	15.7
BH22-1-11	East of East Abutment, b/w EBL and WBL Express	4848108.6	321399.1	43.772752	-79.293809	186.3	15.8

## 5.2 Laboratory Testing

All obtained samples were submitted for natural moisture content testing. In addition, unit weight, Atterberg limits and grain size analysis (sieve and hydrometer) tests were performed on selected soil samples (performed by EXP). Chemical analyses were also carried out on three soil samples selected by EXP. The samples were tested at the Bureau Veritas Laboratories (formerly Maxxam Analytics), a CALA-certified and accredited laboratory in Mississauga, Ontario. The results of the laboratory tests are shown in Table 1.3.

**Table 1.3: List of Laboratory Test Completed by EXP**

Borehole No.	Moisture Content	Atterberg Limits	Sieve	Hydrometer	Unit Weight	Corrosivity
<b>BH22-1-01</b>	10	2	3	3	1	---
<b>BH22-1-02</b>	16	2	4	4	6	1
<b>BH22-1-03</b>	13	3	4	4	5	1
<b>BH22-1-08</b>	19	2	4	4	6	---
<b>BH22-1-09</b>	18	2	4	4	5	---
<b>BH22-1-10</b>	18	3	4	4	3	---
<b>BH22-1-11</b>	14	2	3	3	6	---

The laboratory test results are provided on the attached borehole log sheets in Appendix D as well as graphically in Appendix E.

## 6.0 Subsurface Conditions

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

A borehole location plan and stratigraphic sections are provided in Appendix C. It should be noted that the stratigraphic boundaries indicated on the borehole log and stratigraphic sections 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 interpreted as exact planes of geological change. Furthermore, subsurface conditions may vary between and beyond the borehole locations.

In general, the subsurface conditions below the roadway/pavement structure encountered within the depths of EXP’s geotechnical investigation consists of gravelly sand fill followed by silty sand to sand and silt fill which is underlain or interbedded with clayey silt fill. The embankment fill is underlain by cohesive (clayey silt) and cohesionless (sand and silt) till.

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

### 6.1 Subsoils

#### 6.1.1 Pavement Structure

A pavement structure consisting of asphalt and concrete was encountered at the surface in boreholes BH22-1-01, BH22-1-02, BH22-1-03, BH22-1-08, BH22-1-09, BH22-1-10 and BH22-1-11. The thickness of the pavement structure ranged between 255 mm and 460 mm.

#### 6.1.2 Topsoil

A thin layer of topsoil, 300 mm in thickness, was encountered at the surface in boreholes 73-1 and 73-2 during MTO’s investigation in 1965.

### 6.1.3 Cohesionless Fill: Gravelly Sand to Sand and Gravel

During EXP's geotechnical investigation, gravelly sand fill was encountered below the pavement structure (asphalt/concrete) in boreholes BH22-1-01, BH22-1-02, BH22-1-03, BH22-1-08, BH22-1-09, BH22-1-10 and BH22-1-11. Sand and gravel was also encountered at the surface at borehole 73-3 during MTO's geotechnical investigation in 1965. The approximate elevations of the surface and base of each fill layer, thickness, description and SPT "N" Values encountered in the boreholes are summarized in Table 1.4 below:

**Table 1.4: Summary of Cohesionless Fill: Gravelly Sand to Sand and Gravel Layers**

Borehole	Elevation (m)		Layer Surface Depth (m)	Layer Thickness (m)	Layer Description	SPT “N” Value Range
	Top	Bottom				
EXP (2022)						
BH22-1-01	179.7	179.6	0.3	0.1	Sand and Gravel	N/A <sup>1</sup>
BH22-1-02	185.2	184.9	0.5	0.3	Gravelly Sand	N/A <sup>1</sup>
BH22-1-03	185.6	184.5	0.3	1.1	Gravelly Sand	24
BH22-1-08	185.7	183.7	0.3	2.0	Gravelly Sand	38 – 41
BH22-1-09	185.8	184.0	0.5	1.8	Gravelly Sand	48 – 50
BH22-1-10	185.7	184.9	0.3	0.8	Gravelly Sand	N/A <sup>1</sup>
BH22-1-11	186.0	185.5	0.3	0.5	Gravelly Sand	N/A <sup>1</sup>
MTO (1965)						
73-3	176.8	176.3	0	0.5	Sand and Gravel	N/A <sup>1</sup>

Notes:

1.0 No SPT sampling within layer, only auger samples retrieved.

This layer consists of mainly sand and gravel with trace to some silt and trace clay. The material was brown to grey in colour and moist. SPT "N" values obtained within this layer range from 24 to 50 blows per 300 mm penetration, corresponding to compact to very dense in compactness.

Laboratory testing performed on selected samples consisted of moisture content and grain size distribution tests. The test results of the gravelly sand fill are as follow:

Moisture Content (EXP):

- 3% to 8%

Grain Size Distribution (EXP):

- 30% gravel;
- 53% sand;
- 13% silt;
- 4% clay

The results of the moisture content and grain size distribution tests performed by EXP are provided on the record of borehole sheets in Appendix D. The results of the grain size distribution performed by EXP are also provided in Figure 1 in Appendix E.

#### 6.1.4 Cohesionless Fill: Silty Sand to Sand and Silt

During EXP's geotechnical investigation, silty sand to sand and silt fill was encountered below the gravelly sand and sand and gravel fill in boreholes BH22-1-02, BH22-1-03, BH22-1-08, BH22-1-09, BH22-1-10 and BH22-1-11. The approximate elevations of the surface and base of each fill layer, thickness, description and SPT "N" Values encountered in the boreholes are summarized in Table 1.5 below:

**Table 1.5: Summary of Cohesionless Fill: Silty Sand to Sand and Silt Layers**

Borehole	Elevation (m)		Layer Surface Depth (m)	Layer Thickness (m)	Layer Description	SPT “N” Value Range
	Top	Bottom				
EXP (2022)						
BH22-1-02	184.9	176.6	0.8	8.3	Silty Sand	18 – 45
BH22-1-03	184.5	179.5	1.4	5.0	Silty Sand	24 – 28
BH22-1-08	183.7	176.9	2.3	6.8 <sup>1</sup>	Silty Sand	5 – 35 <sup>2</sup>
BH22-1-09	184.0	178.7	2.3	5.3	Silty Sand	17 – 86
BH22-1-10	184.9	176.4	1.1	8.5 <sup>1</sup>	Sand and Silt	25 – 54 <sup>2</sup>
BH22-1-11	185.5	178.7	0.8	6.8	Sand and Silt	31 – 59

Notes:

- 1.0 Includes cohesive fill layer within the overall cohesionless fill thickness (see Table 1.6).
- 2.0 Range for SPT "N" values only within cohesionless fill.

This layer predominately consists of sand and silt with trace to some gravel and trace to some clay. Thin layers of clayey silt were also observed in the fill in borehole BH22-1-10. The material was brown to grey in colour and damp to moist. The SPT "N" values within this layer ranged from 5 to 86 blows per 300 mm penetration, corresponding to loose to very dense, but generally compact to dense in compactness.

Laboratory testing performed on selected samples consisted of moisture content, grain size distribution and unit weight tests. The test results are as follows:

Moisture Content: (EXP)

- 3% to 17%

Grain Size Distribution: (EXP)

- 0% to 12% gravel;
- 40% to 71% sand;
- 20% to 50% silt;
- 3% to 10% clay;

Unit Weight: (EXP)



- 21.0 kN/m<sup>3</sup> to 24.2 kN/m<sup>3</sup>

The results of the moisture content, grain size distribution and unit weight tests performed by EXP are provided on the record of borehole sheets in Appendix D. The results of grain size distribution tests are also provided on Figure 2 in Appendix E.

#### 6.1.5 Cohesive Fill: Clayey Silt

During EXP's geotechnical investigation, a cohesive fill was encountered below the cohesionless fill layers in boreholes BH22-1-03, BH22-1-08, BH22-1-09, BH22-1-10 and BH22-1-11.

The approximate elevations of the surface and base of each fill layer, thickness, description and SPT (N Value) encountered in the boreholes are summarized in Table 1.6 below:

**Table 1.6: Summary of Cohesive Fill: Clayey Silt Layers**

Borehole	Elevation (m)		Layer Surface Depth (m)	Layer Thickness (m)	Layer Description	SPT “N” Value Range
	Top	Bottom				
EXP (2022)						
BH22-1-03	179.5	176.8	6.4	2.7	Clayey Silt	18 – 20
BH22-1-08	179.9	178.4	6.1	1.5	Clayey Silt	20
BH22-1-09	178.7	176.7	7.6	2.0	Clayey Silt	7 – 36
BH22-1-10	179.9	176.9	6.1	3.0	Clayey Silt	23 – 34
BH22-1-11	178.7	177.2	7.6	1.5	Clayey Silt	15

This layer predominately consists of silt and clay and can be considered sandy with trace gravel. The material was light brown to dark grey in colour and moist. The SPT "N" value within this layer ranged between 7 to 36 blows per 300 mm penetration, corresponding to firm to hard in consistency. Atterberg limits tests suggest that this cohesive fill material is low plastic.

Laboratory testing performed on selected samples consisted of moisture content, grain size distribution, Atterberg limits and unit weight tests. The test results are as follow:

#### Moisture Content (EXP):

- 8% to 15%

#### Grain Size Distribution: (EXP)

- 1% to 2% gravel;
- 30% to 45% sand;
- 39% to 58% silt;
- 11% to 17% clay;

#### Atterberg Limits: (EXP)

- Liquid Limit: 16% to 17%;

- Plastic Limit: 10% to 12%;
- Plasticity Index: 5% to 7%

Unit Weight: (EXP)

- 22.2 kN/m<sup>3</sup> to 22.5 kN/m<sup>3</sup>

The results of the moisture content, grain size distribution, Atterberg limits and unit weight tests performed by EXP are provided on the record of borehole sheets in Appendix D. The results of grain size distribution and Atterberg limits tests are also provided on Figure 3 and 6 in Appendix E.

#### 6.1.6 Sandy Silt to Silty Sand (Till)

During EXP's geotechnical investigation, a native non-cohesive till deposit was encountered below the fill in all boreholes. The approximate elevations of the surface and base of each layer, thickness, description and SPT (N Value) encountered in the boreholes are summarized in Table 1.7 below:

**Table 1.7: Summary of Cohesive Till: Clayey Silt Layers**

Borehole	Elevation (m)		Layer Surface Depth (m)	Layer Thickness (m)	Layer Description	SPT “N” Value Range
	Top	Bottom				
EXP (2022)						
BH22-1-01	179.6	170.4	0.4	9.2	Sandy Silt to Silty Sand	26 – 97
BH22-1-02	176.6	170.0	9.1	6.6	Sandy Silt to Silty Sand	10 – 106
BH22-1-03	176.8	170.1	9.1	6.7	Sandy Silt to Silty Sand	18 – 44
BH22-1-08	176.9	165.6	9.1	11.3	Sandy Silt to Silty Sand	21 – 94
BH22-1-09	176.7	165.9	9.6	10.8 <sup>1</sup>	Sandy Silt to Silty Sand	15 – 64
BH22-1-10	176.4	170.3	9.6	6.1 <sup>1</sup>	Sandy Silt to Silty Sand	13 – 180/280 mm
BH22-1-11	177.2	170.5	9.1	6.7 <sup>1</sup>	Sandy Silt to Silty Sand	18 – 37
MTO (1965)						
73-1	176.2	165.4	0.3	10.8 <sup>1</sup>	Silty Sand to Sandy Silt	80 – 162
73-2	175.6	163.2	0.3	12.4 <sup>1</sup>	Silty Sand to Sandy Silt	39 – 169/230 mm
73-3	176.3	164.1	0.5	12.2 <sup>1</sup>	Silty Sand to Sandy Silt	33 – 129

Notes:

1.0 End of borehole terminated within this layer.

This layer predominately consists of sand and silt with trace to some gravel and trace to some clay. Additionally, trace organics were observed in borehole BH22-1-08. The material was grey to brown in colour and dry to wet. The SPT "N" value within this layer ranged between 10 blows per 300 mm penetration to 180 blows per 280 mm, corresponding to compact to very dense, but generally dense to very dense in consistency. Atterberg limits tests suggest that this layer is non-plastic to low plastic.

Laboratory testing performed on selected samples consisted of moisture content, grain size distribution, Atterberg limits and unit weight tests. The test results are as follow:

Moisture Content (EXP):

- 2% to 20%

Grain Size Distribution: (EXP)

- 0% to 19% gravel;
- 26% to 52% sand;
- 25% to 54% silt;
- 8% to 24% clay;

Atterberg Limits: (EXP)

- Non-plastic
- Liquid Limit: 16% to 28%;
- Plastic Limit: 10% to 13%;
- Plasticity Index: 6% to 15%

Unit Weight: (EXP)

- 21.0 kN/m<sup>3</sup> to 23.6 kN/m<sup>3</sup>

The results of the moisture content, grain size distribution, Atterberg limits and unit weight tests performed by EXP are provided on the record of borehole sheets in Appendix D. The results of grain size distribution and Atterberg limits tests are also provided on Figure 4 and 7 in Appendix E.

## 6.2 Groundwater Conditions

Groundwater levels were observed upon completion of some of the boreholes. Groundwater levels measured on completion of boreholes may not be considered stabilized and therefore may not represent the established long-term average groundwater table (phreatic surface).

A summary of the groundwater levels encountered during the investigations are summarized in Table 1.9 and are also presented on the Record of Borehole Sheets attached in Appendix D and Appendix F.

**Table 1.9: Summary of observed groundwater levels**

Borehole	Ground Surface Elevation (m)	Water level Depth/ Elevation (m)	Date
<b>EXP (2022)</b>			
<b>BH22-1-08</b>	186.0	18.6/167.4 <sup>1</sup>	November 2 - 3, 2022
<b>MTO (1965)</b>			
<b>73-1</b>	176.5	8.4/168.1	May 12, 1965
<b>73-2</b>	175.9	8.3/167.6	May 12, 1965
<b>73-3</b>	176.8	8.9/167.9	May 12 - 13, 1965

**Notes:**

1.0 Groundwater level inferred from split spoon observations.

It should be noted that fluctuations in the level of the groundwater may occur due to seasonal variations, (precipitation, snowmelt, rainfall), local soil permeability, construction remediation activities, and other related factors.

### 6.3 Chemical Analyses

Three (3) soil samples were selected for chemical analysis during current investigation. The soils samples collected by EXP were tested at the Bureau Veritas Laboratories (formerly Maxxam Analytics), a CALA-certified and accredited laboratory in Mississauga, Ontario.

The analytical results are summarized in Table 1.11 below and are presented in Appendix E.

**Table 1.11. Summary of chemical analysis results**

Sample Identification	pH (Unitless)	Soluble Chloride (ppm)	Soluble Sulphate (ppm)	Resistivity (ohm-cm)	Conductivity (umho/cm)	Redox Potential (mV)
<b>BH22-1-02, SS3</b>	10.1	430	57 – 59	1100	0.852 – 0.870	37 – 58
<b>BH22-1-03, SS10</b>	7.87	560	<20	810	1.230	190

*Foundation Investigation and Design Report  
 Highway 401 Eastbound from Victoria Park Avenue to Neilson Road  
 Superstructure Replacement at Birchmount Road Overpass  
 Eastbound Core and Collectors Structure (Site 37X-0218/B1 & B3)  
 Assignment No. 2021-E-0018  
 Date: December 31, 2024*

## 7.0 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.

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


Details of the limitations of this report are presented as Appendix A, "Limitations and Use of Report".

This Foundation Investigation Report has been prepared by Elvis Lu, M.Eng., EIT and Thomas Lardner, Ph.D., P.Eng. It was reviewed by TaeChul Kim, M.E.Sc., P.Eng. and Stan E. Gonsalves, M.Eng., P.Eng., Designated MTO Foundation Contact.


Yours truly,


EXP Services Inc.

Elvis Lu, M.Eng., EIT  
 Technical Specialist

  
 Thomas Lardner, Ph.D., P.Eng.  
 Geotechnical Engineer



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

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



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## Part II: Foundation Design Report

Discussion and Engineering Recommendations for Birchmount Road Overpass (Site 37X-0212/B1 & B3)

## 8.0 Discussion and Recommendations

### 8.1 General

This section of the report provides geotechnical design recommendations on roadway protection systems for rehabilitation of the proposed superstructure and widening of the Highway 401 Eastbound Core and Collectors Birchmount Road Overpass. 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. Previous investigations by others as noted in this report available through GEOCRE were used to aid in assessments. The interpretation and recommendations provided are intended solely to permit designers to assess roadway protection systems alternatives for bridge rehabilitation and widening. Comments on construction are only provided to highlight issues that could affect the design. Contractors bidding on the works should make their own assessments of the factual data and how it might affect construction means and methods, scheduling and the like.

The existing bridge is a 29.72 m long two-span bridge with equal spans between the abutments (14.86 m). It is understood that the existing abutments, piers, and retaining wall foundations are supported on spread footings. The existing abutments are supported on approximately 5.6 m and 5.3 m wide spread footings at the express lanes and collectors lanes, respectively. Based on the Foundation and Investigation Design Report “*Bridge Widening and Replacement Highway 401 Rehabilitation from Warden Avenue to Brock Road, Toronto, Ontario, W.O.07-20012.*” produced by Golder Associates Ltd., dated April 2012, the west abutment is founded at about Elevation 173.6 m and the east abutment is founded at about Elevation 173.8 m to 174.2 m from the north side to the south side of the bridge. Both abutments were also constructed with a shear key. The centre piers are founded on a 1.8 m wide footing founded at about Elevation 174.0 m. The Highway 401 pavement grade ranges between about Elevation 185 m to 186 m, while the Birchmount Road pavement grade is at an Elevation of about 176 m to 179.5 m (from north to south) at the structure site.

It is understood that for the proposed rehabilitation of the Birchmount Road Overpass structure, the change in loading conditions on the foundation elements associated with the rehabilitation works will be negligible. The existing foundations will remain same, and based on the contemplated traffic staging plan there will not be any unusual loads on the existing foundation. The rehabilitation program will involve replacement of the existing bridge deck and girders, conversion to semi-integral abutment; and patch repair to abutment walls and wingwalls/retaining walls. The existing foundations will remain to support the abutments. It is anticipated that this work will require excavations of the embankment fills immediately behind the abutment walls/retaining walls to facilitate the rehabilitation work. The depth of excavation behind the existing abutment/retaining wall is expected to be about 4.8 m.

Additionally, the bridge will be widened by 4.5 m with a new pier column and cap to attach to the existing structure. The existing retaining wall structures (37x-1765/W and 37x-1766/W) will be replaced with new retaining walls along the south side of widened collector structure. High fills for embankment widening and retaining walls are proposed to accommodate the additional lane of widening that will occur from about 150 m west of Birchmount Road to the CP Rail Overpass structure (located about 200 m east of Birchmount Road).

Based on subsoil conditions encountered at the site it is expected that excavation will be carried out through cohesionless (gravelly sand over silty sand to sand and silt) and cohesive (clayey silt) fill for the superstructure replacement while excavation into the native soils (cohesionless and cohesive till) is expected for the proposed widened bridge abutments and pier. Based on an assessment of the water levels observed in the borings and the subsurface conditions, groundwater depth is interpreted to be about 17.9 m to 18.6 m below the existing Highway 401 grade with Elevations ranging between 167.4 m to 168.1 m. It should be noted that fluctuations in the level of the groundwater may occur due to seasonal variations, (precipitation, snowmelt, rainfall), local soil permeability, construction remediation activities, and other related factors. A detailed description of the soils and groundwater encountered are discussed in Part I of this report.

This part of the report addresses the geotechnical design of the foundation for the roadway protection system by providing geotechnical design parameters that may be required in accordance with the latest edition of the *Canadian Highway Bridge Design Code (CHBDC, CAN/CSA-S6-19, 2019)*, the *Canadian Foundation Engineering Manual (CFEM, 2023)*, *Guideline for MTO Foundation Engineering Services, Version 03 (April 2022)* and generally accepted good practice. This structure has the potential to significantly affect alternate transportation corridors and is considered to be of “Typical Consequences Level” associated with exceeding Limit States Design (Section 6.5 and Commentary, CHBDC, 2019). A “Typical Degree of Site and Prediction Model Understanding” is considered appropriate based on the level of foundation investigation completed. Pertinent geotechnical resistance factors and consequence factors have been used in design. The report also addressed other geotechnical and construction considerations such as excavation, groundwater and surface water control, lateral earth pressure on structures, embankment construction/widening, global stability, settlement, and seismic considerations.

## 8.2 Structure Foundations

At the time of preparing this report, the type of foundation for the replacement structure is not specified. However, it is assumed that the foundation for the abutments and piers for the widened portion of the Birchmount Overpass structure will be the same as the existing foundation (spread footings). However, for completeness, several foundation options for support of abutments were analyzed for this report, including spread footings, H-Piles, and caisson foundations.

### 8.2.1 Shallow Foundation Options

#### 8.2.1.1 Geotechnical Resistance for Structure Foundations

Based on the subsurface conditions encountered, the use of conventional spread footings can be considered to support the widened portion of the abutments and pier. Table 2.1 summarizes the recommended geotechnical resistances at the footing depths for the spread footing option. The geotechnical resistances provided are for vertical loading condition only; load eccentricity and load inclination effects should be addressed in accordance with the CHBDC and its commentary. The geotechnical resistances provided in sections below were factored with a typical consequence factor of 1.0 at ULS and SLS; typical degree of understanding at ULS (factor of 0.5) and high degree of understanding at SLS (factor of 0.9) in accordance with Table 6.1 and 6.2 of the CHBDC CSA S6:19 (CHBDC Section 6.5 and 6.9).

**Table 2.1 Recommended shallow foundation design parameters for widened portion of bridge structure**

Location	Founding Soil Type <sup>3</sup>	Excavation Elevation (m)	Recommended Founding Elevation (m)	Factored Geotechnical Resistance at ULS (kPa)	Factored Serviceability Geotechnical Resistance (for 25 mm settlement) (kPa)	Factored Serviceability Geotechnical Resistance (for 12 mm settlement) (kPa)
<b>West Abutment</b> (BH22-1-02, 73-2)	Silty Sand to Sandy Silt (Till) (very dense)	173.5	173.6 or below	1000 <sup>1</sup>	670 <sup>1</sup>	350 <sup>1</sup>
<b>Pier</b> (BH22-1-01, 73-3)	Silty Sand to Sandy Silt (Till) (compact to very dense) OR Engineered Granular A Pad	176.5	176.6 or below	750 <sup>2</sup>	450 <sup>2</sup>	250 <sup>2</sup>



Location	Founding Soil Type <sup>3</sup>	Excavation Elevation (m)	Recommended Founding Elevation (m)	Factored Geotechnical Resistance at ULS (kPa)	Factored Serviceability Geotechnical Resistance (for 25 mm settlement) (kPa)	Factored Serviceability Geotechnical Resistance (for 12 mm settlement) (kPa)
<b>East Abutment</b> (BH22-1-03, 73-1)	Silty Sand to Sandy Silt (Till) (very dense)	174.1	174.2 or below	1000 <sup>1</sup>	670 <sup>1</sup>	350 <sup>1</sup>

Notes:

- (1) Assumed footing width of 5.3 m to 6.3 m (similar to the existing abutments).
- (2) Assumed footing width of 1.8 m (similar to the existing piers).
- (3) ~100 mm thick working granular pad (above founding soils) consisting of Granular 'A' or Granular B Type II conforming to OPSS.PROV 1010 and compacted according to OPSS.PROV 501.

Foundations should be set at the same elevation as the existing foundations. If this is not the case, measures should be implemented such that the upper footing is not undermined during construction. The lower footing should be set above a 10H:7V line from the near bottom edge of the adjacent footing or the proposed footing should be designed to accommodate loading from the upper adjacent footing.

#### 8.2.1.2 Geotechnical Resistance for Wing/RSS Wall Foundations

The existing wing walls 37x-1765/W and 37x-1766/W are proposed to be replaced with new wing walls along the south side of the widened collector structure. Additionally, retaining walls are proposed to be constructed to accommodate the Highway 401 widening which will occur from approximately 150 m west of Birchmount Road to the CP Rail overpass structure (~200 m east of the Birchmount overpass structure). Based on the proposed construction, the geotechnical resistances for a structure founded on engineered fill and/or native till (cohesive and cohesionless) are tabulated below

**Table 2.2 Recommended shallow foundation design parameters for structure wing walls and widening retaining walls**

Location	Founding Elevation <sup>1,2</sup> (m)	Footing Width (m)	Founding Soil Type <sup>3</sup>	Factored Geotechnical Resistance at ULS (kPa) <sup>1</sup>	Factored Serviceability Geotechnical Resistance (for 25 mm settlement) (kPa)
<b>West Abutment Wing Wall</b> (BH22-1-2, 73-2)	174.5	>1.0	Silty Sand to Sandy Silt (Till) (very dense) / Clayey Silt (Till) (stiff to very stiff)	225	135
<b>East Abutment Wing Wall</b> (BH22-1-3, 73-1)	175.1	>1.0	Silty Sand to Sandy Silt (Till) (very dense) / Clayey Silt (Till) (very stiff to hard)	225	135

**Notes:**

- (1) Below frost line or for RSS wall, minimum embedment requirements set in MTO RSS Design Guidelines, MTO Engineering Standard Branch, September 2008.
- (2) Founding level assumed to be 0.9 m above the top of abutment footings (see Section 8.2.1.1).
- (3) ~100 mm thick working granular pad (above founding soils) consisting of Granular 'A' or Granular B Type II conforming to OPSS.PROV 1010 and compacted according to OPSS.PROV 501.

### 8.2.1.3 Resistance to Lateral Loads

Resistance to lateral forces/ sliding should be calculated in accordance with Section 6.10.5 of the CHBDC (CAN/CSA S6-19), using the parameters in Table 2.3.

**Table 2.3 Recommended parameters for calculation of unfactored horizontal resistance**

Interface Conditions	Parameter
Between cast-in-place concrete and compacted granular fill	Coefficient of friction ( $\tan \delta$ )=0.6
Between cast-in-place concrete and compacted earth fill (e.g. SSM)	Coefficient of friction ( $\tan \delta$ )=0.45
Between pre-cast concrete and compacted granular fill	Coefficient of friction ( $\tan \delta$ )=0.4

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

### 8.2.1.4 Frost Protection

Ontario Provincial Standard Drawing (OPSD) 3090.101 indicates that the frost penetration for the Toronto area is 1.2 m. Therefore, all foundation elements should be provided with a minimum of 1.2 m of earth cover or equivalent approved insulation for frost protection. For RSS walls, foundation elements should be below the frost line or minimum embedment requirements set in MTO RSS Design Guidelines, MTO Engineering Standard Branch, September 2008.

### 8.2.1.5 Structure Backfill

The selection and placing of backfill should be in accordance with OPSS.PROV 902. For backfilling immediately behind the abutment walls and retaining walls, it should consist of free-draining, non-frost susceptible granular materials such as Granular A or Granular B Type II conforming to OPSS. PROV 1010. Beyond this zone could consist of Granular B Type I conforming to OPSS. PROV 1010. The embankment fill should be placed and compacted in accordance with OPSS.PROV 501 and placed in accordance with OPSS.PROV 206.

## 8.2.2 Deep Foundation Options

### 8.2.2.1 General

Soil conditions at the east and west abutments indicate that a compact to very dense silty sand (SPT 'N' values of between 15 blows per 300 mm to than 100 blows per 300 mm) extend to a minimum Elevation of approximately 165 m. As noted previously in Section 6.2, top of groundwater is interpreted to be about 16.9 m to 17.6 m below existing highway grade with Elevation ranging between 167.4 m to 168.1 m.

Should there be a requirement for resistance to increased loading, micropiles may be considered as an alternative to increase the geotechnical resistance while minimizing the footprint of the required works. Deep foundation options provide greater control of settlements over shallow foundations, if tie-in between the potential structure and existing structure is sensitive to differential settlements.

#### 8.2.2.2 Micropiles

The proposed remedial works are expected to maintain the current loading condition, resulting in no anticipated additional loading. Should design indicate a loading of greater than approximately 10% and additional geotechnical bearing resistance be required, micropiles may be incorporated into the existing foundation structure. Micropiles may also be incorporated into the footings for the southern extension to increase bearing capability and/or resist settlement. Advantages of micropiles are the small construction footprint and ability to remediate the existing foundation without enlarging the footing area.

##### 8.2.2.2.1 Geotechnical Axial Resistance

Micropiles may be found in the compact to very dense Silty Sand to Sandy Silt (Till) stratum. Micropiles should have a minimum bond length of 3.0 m and a minimum diameter of 150 mm. Recommended values for grout-to-soil adhesion are provided in Table 2.4. The geotechnical capacities provided were factored with typical consequence factor of 1.0 at ULS and SLS; typical degree of understanding (factor of 0.4 at ULS) and typical degree of understanding (factor of 0.8 at SLS) in accordance with Table 6.1 and 6.2 of the CHBDC S6-14.

**Table 2.4: Summary of micropile adhesion design values**

Foundation Unit	Relevant Borehole	FHWA Type B Micropile <sup>1</sup>		FHWA Type C Micropile <sup>1</sup>		Bond Zone Stratum
		Ultimate Adhesion (kPa)	Factored Ultimate Adhesion (kPa)	Ultimate Adhesion (kPa)	Factored Ultimate Adhesion (kPa)	
West Abutment	BH22-1-02, BH22-1-08, 73-2, 73.3	200	80	250	100	Dense Sandy Silt to Silty Sand
Pier	BH22-1-01, 73.3	200	80	250	100	Dense Sandy Silt to Silty Sand
East Abutment	74-1A, 74-4, BH22-5-3, BH22-5-4	200	80	250	100	Dense Sandy Silt to Silty Sand

Note:

- (1) Micropile type as defined by FHWA Micropile Design and Construction Reference Manual (Publication No. FHWA NHI-05-039).

#### 8.2.2.2.2 Verification and proof testing

Adhesion values provided in Table 2.5 should be tested in accordance with FHWA recommendations. A minimum of one sacrificial test should be conducted to 200% the selected Ultimate Adhesion. Should the micropile type (as defined by the FHWA) be changed, or the installation means and methods be altered, a verification test must be conducted using the proposed micropile design and installation methods prior to construction of production micropiles.

Proof testing should be done on a minimum of 5% of production micropiles. Testing should be done in accordance with the FHWA requirements. Compression or tension testing is acceptable.

#### 8.2.2.2.3 Lateral resistance

Lateral resistance of micropiles is derived through casing design. To ensure adequate depth for the generation of lateral geotechnical resistance, the cased length should be approximately 20 times the diameter and may be refined through analysis. Geotechnical lateral resistance input values are provided in Table 2.11.

### 8.3 Approach Embankments

#### 8.3.1 General

Based on the information provided to EXP by the client, the existing structure is to be widened by 4.5 m on the south side. Additionally, Highway 401 will be widened to the south from west of Birchmount Road to the CP Rail overpass structure located to the east of the Birchmount overpass structure (approximately 200 m). All unretained portions of the embankment greater than 8.0 m in height requires a 2 m wide mid-height bench (OPSD 202.010). The slopes of the embankments should be provided with adequate erosion protection against surface water runoff. The global stability of forward slope has also been checked.

#### 8.3.2 Stability Considerations

Using the sub-surface information interpreted from the previous and additional boreholes and proposed embankment configuration of Highway 401 Birchmount overpass structure based on AECOM's GA drawings, stability analyses were carried out for the forward and side slopes at the widened portion of the west and east abutment. Additionally, the global stability of the most critical section of the side slopes/retaining walls where Highway 401 is proposed to be widened on either side of the structure was checked.

The analyses were carried out using a commercially available computer program (SLOPE/W) developed by Geo-Slope International. The slope stability analyses were performed using the Morgenstern-Price method developed on the basis of limit equilibrium. For the proposed slope profile, a large number of trial slip surfaces were analyzed to establish the minimum factor of safety of the slope in the proposed conditions for both static and seismic conditions. A minimum Factor of Safety of 1.5 for static (drained and undrained) and 1.1 for seismic conditions were adopted as the design criteria for abutments and embankments as per MTO requirements.

The seismic properties given in Section 8.4 were obtained from the Natural Resources Canada website, 2020 NBC, using the site location coordinates. Material parameters adopted in the slope stability analyses are summarized in Table 2.5. In addition, a traffic surcharge pressure of 16 kPa was adopted in the slope stability assessments for the abutments and approach embankment.

**Table 2.5 Soil strength parameters for slope stability assessment**

Layer Name	Bulk Unit Weight, $\gamma$ (kN/m <sup>3</sup> )	Short Term Parameters		Long Term Parameters		Water Level Elevation (m)
		$\phi$ (°)	C (kPa)	$\phi'$ (°)	C' (kPa)	
Granular A/Granular B Type II	22.8	35	-	35	-	167.9 (West Abutment) 168.1 (East Abutment)
Granular B Type I	21	32	-	32	-	
Engineered Fill	21	32	-	32	-	
Gravelly Sand to Silty Sand to Sand and Silt Fill (generally compact to dense)	21	31	-	31	-	
Clayey Silt Fill (firm to hard)	21	-	100	31	-	
Silty Sand to Sandy Silt Till (dense to very dense)	22	36	-	36	-	

The results of the slope stability analyses are shown in Figures 1 through 10 in Appendix G of this report and are summarized in Table 2.6 below. As can be shown, the determined the minimum factor of safety of critical slip surfaces meets the design criteria when the existing fill is widened with granular A and B type for static and seismic conditions for short-term (undrained) and long-term (drained) conditions with embankment slopes of 2H:1V. Therefore, based on these global stability analyses, the proposed 6.9 m to 7.1 m high (forward slope) to ~6.0 m high (embankment side slope) can be safely constructed with slopes of 2H:1V.

**Table 2.6 Summary of results of slope stability analyses**

Location	Maximum Height (m)	Conditions	Min FoS
West Abutment Forward Slope	6.9	Drained long-term conditions, static condition	1.96
		Undrained short-term conditions, seismic condition	1.96
East Abutment Forward Slope	7.1	Drained long-term conditions, static condition	2.81
		Undrained short-term conditions, static condition	2.90
		Undrained short-term conditions, seismic condition	2.09
Widened Embankment West of Structure	5.7	Drained long-term conditions, static condition	1.7
		Undrained short-term conditions, seismic condition	1.21
Widened Embankment East of Structure	5.9	Drained long-term conditions, static condition	1.67
		Undrained short-term conditions, static condition	1.67
		Undrained short-term conditions, seismic condition	1.21

### 8.3.3 Settlement Considerations

#### 8.3.3.1 Settlement of Foundation Soils

The grade raise for the widening of the existing Birchmount overpass structure will cause settlements of the founding soils, which could warrant additional measures (such as, in general, extended pre-load times and ground improvement measures) to ensure smooth performance of the road embankment.

The magnitudes of total settlement for the east and west approach embankment have been assessed based on Standard Penetration Test (SPT) results and Elastic modulus which is correlated from the available data. Parameters adopted in the settlement analyses for the east approach embankment is summarized in Table 2.7.

A computer program, Settle3D (Rocscience) was employed for settlement calculation. Settle3D is a 3-dimensional program for the analysis of immediate and consolidation settlement under foundations, embankments and surface loads. The program combines the simplicity of one-dimensional analysis with the power and visualization capabilities of more sophisticated three-dimensional programs.

**Table 2.7 Soil strength parameters for settlement analyses**

Layer Name	Unit Weight (kN/m <sup>3</sup> )	E (MPa)	Compression Index (C <sub>c</sub> )	Recompression Index (C <sub>r</sub> )	Void Ratio (e)	Preconsolidation Pressure, p' <sub>c</sub> (kPa)	CV (cm <sup>2</sup> /s)
Sandy Silt to Silty Sand Till (compact to very dense)	22	500	-	-	-	-	--

The summary of results of settlement analyses for the approached embankments is given in Table 2.8. The Settle 3D results of these cases along with settlement curve can be seen Appendix H.

**Table 2.8 Summary of results of settlement analyses**

Layer Name	Embankment Height (m)	Assumed Embankment Width (m)	Calculated Immediate Settlement (mm)	Calculated Consolidation Settlement (mm)	Calculated Total Settlement (mm)
West Approach	5.7	4.5	2.0	N/A	2.0
East Approach	5.9	4.5	2.0	N/A	2.0

The settlement analyses suggested that the total settlement will be negligible along the width of the east and west embankment widening. This settlement is expected to occur during and immediately following construction of approach embankments. It is important to note that the self-weight settlement of the embankment fill will occur within two to four months, depending on material type and placement method. Some differential settlements can be expected at the new/old embankment interface and structure/embankment interface, but these movements should be able to be accommodated during the paving process.

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

### 8.3.3.2 Settlement of Embankment Fill

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

### 8.3.4 Monitoring Program

The computed settlements are based on best assumptions and findings on the nature, consistency, compressibility, characteristics and thickness of the identified deposits within the zone of influence of loading. Actual settlements can differ somewhat, given the nature of soil and its deposition. Therefore, a program of instrumentation and monitoring is recommended to verify that the actual settlements are as predicted.

The settlement monitoring can be terminated when the following criteria are satisfied:

1. The measured settlement is equal to the predicted long-term settlement for the embankment fill.
2. The measured settlement has reached 90% of the primary settlement due to fill embankment.
3. The settlement prediction can be carried out as per "Observational Procedure of Settlement Prediction, Akira Asaoka, 1978".

### Standard Pins/ Surface Monitoring Points

The Standard Pins/Surface Monitoring Points shall be cast into concrete at the top of the embankment. The concrete will be cast in situ in a hole dug at the location of the Settlement Pins.

The Surface Monitoring Points shall be installed at a 5.0 m spacing within the embankment to a minimum of 25 m behind the abutment. Surface Monitoring Points are required whenever embankment construction occurs. For example, if the proposed abutment and embankments are constructed in a two-phase sequence, monitoring will be required for each.

The Surface Monitoring Points shall be a minimum 25 mm diameter reinforcing steel bar (OPSS.PROV 905), cut 0.4 m long. The top of the reinforcing steel bar shall be angled or rounded in such a way that a single survey point can be clearly identified and returned to.

The concrete shall (OPSS.PROV 1350) be of minimum 25 MPa compressive strength and set time sufficient to secure the nail pin within two days of pouring.

The following information should be supplied no later than three working days after installing the settlement pins:

- Northing and Easting of Each Settlement Pins in MTM NAD 83 coordinates.
- Elevation of the Settlement Pin referenced to geodetic datum.

- Dates of installation.
- Installation notes and sketches.

## 8.4 Seismic and Liquefaction Potential Consideration

Seismic characterization of the site should be compliant with the Canadian Highway Bridge Design Code (CHBDC, CSA-S6-19). Table 4.1 in the CHBDC (see Clause 4.4.3.2) shows site classification for seismic site response based on average soil properties in the top 30 m. At the site, the subsoil beneath the embankment fill generally consists of stiff to hard clayey silt till and dense to very dense sand and silt till and silty sand to sandy silt. Bedrock was not encountered within the investigated depth. The groundwater level is at about 11.4 m to 12.1 m depth below the existing Birchmount Road grade (south end of Birchmount Road). The reported N-values for the native soils ranged from 10 to over 100 blows for 300 mm of penetration, with an average value being above 50 blows per 300 mm of penetration within the drilled depth. Based on these soil characteristics, the site class for this site is estimated to be Class “C” according to Table 4.1. However, these parameters should be reviewed by the Structural Engineer.

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 ( $PGA_{ref}$ ). The  $PGA_{ref}$  is  $0.8 \cdot PGA$  if  $Sa(0.2)/PGA < 2.0$ , which holds true in this case. Therefore, as per Tables 4.2 to 4.8 of the CHBDC (CAN/CSA-S6-19), the site coefficients  $F(0.2)$ ,  $F(0.5)$ ,  $F(1.0)$ ,  $F(2.0)$  and  $F(PGA)$ , for this site (Seismic Site Class C and  $PGA_{ref}$  of  $0.8 \cdot PGA$ ) are all equal to 1.00.

From Natural Resources Canada website, 2020 NBC seismic hazard values are obtained using the site location coordinates (43.772533°N, 79.294313°W), where the damped spectral accelerations are  $Sa(0.2)=0.324g$ ,  $Sa(0.5)=0.198g$ ,  $Sa(1.0)=0.104g$ ,  $Sa(2.0)=0.0481g$  the peak ground acceleration (PGA) is  $0.176g$  ( $g$  = acceleration due to gravity  $-9.81 \text{ m/s}^2$ ). These values are associated with an earthquake having 2 percent probability of exceedance in a 50-year period (1 in 2475-year event) for Site Class C as shown on the GSC seismic hazard calculation data sheet for this site attached in Appendix J.

Based on soils and groundwater condition encountered (i.e., sands and non-plastic/low-plastic silt layers ( $PI < 12$ ) with average corrected SPT blow count over 25 blows/305 mm, CHBDC 6.14.8.1.2), no liquefaction is expected due to the ground motion from a 1 in 2475-year earthquake event. In addition, cyclic mobility of the native clayey silt till layers ( $PI > 12$ ) is also not expected for a 1 in 2475-year earthquake event.

## 8.5 Roadway Protection System

Roadway protection system for construction is required to facilitate the rehabilitation and widening work. The roadway protection system should be properly designed so that the lateral movement of any portion of the protection system will not exceed the established criterion for the structural performance level. The temporary support systems should be designed and constructed in accordance with OPSS.PROV 539 as amended by SP105S09. The lateral movement of the temporary shoring system should meet Performance Level 2 as specified in OPSS.PROV 539, provided that the existing, if any, adjacent utilities can tolerate this magnitude of deformation or re-routed away from excavation influence zone. The shoring system should be designed by a Professional Engineer, experienced in this type of work and employed by the contractor.

To safely support the excavation walls and minimize the impact to existing utilities in the embankment (if any), temporary shoring consisting of driven steel sheet piling or Soldier H-piles with lagging should be practical options at this location. The subsurface conditions at this site is suitable for both of these options. Where the depth requiring support is too much for cantilevered systems, bracing in the form of shores or deadman anchors can be considered. A comparison of these two systems based on advantages and disadvantages, risks and relative costs is provided in Table 2.9.



It is considered that a sheet pile of sufficiently robust cross section could be driven through embankment fill and native deposits encountered at these sites. Although cobbles and boulders were not encountered during the site investigation, their presence should be anticipated in the native till. Difficulties with installation may occur if occasional cobbles and boulders were to be encountered, requiring their removal before further driving or fitted with a driving shoe. It is recommended that an NSSP be included in the Contract Documents to warn the Contractor of the possible presence of cobbles and/or boulders within the native till deposits. An example of NSSP is included in Appendix K. Alternatively, an H-pile with lagging wall can be used as a vertical temporary shoring system. The H-piles are installed, and lagging is inserted between installed H-piles during excavation. Space between the excavation and lagging must be suitably backfilled and drained. Lagging wall material can be selected as wood (timber), steel or concrete.

**Table 2.9 Evaluation of temporary roadway protection system options**

Support System	Advantages	Disadvantages	Relative Cost	Risk Consequences	Rank
Soldier H-Pile and Lagging	<ul style="list-style-type: none"> <li>Appropriate for shallow and deep installation</li> <li>Easy to install through potential obstructions</li> </ul>	<ul style="list-style-type: none"> <li>May require bracing/tieback anchors depending on depth of excavation into overburden</li> </ul>	<ul style="list-style-type: none"> <li>Low cost of construction</li> </ul>	<ul style="list-style-type: none"> <li>Piles could be long</li> <li>Potential for loss of soil through laggings</li> </ul>	1
Driven Steel Sheet Piling	<ul style="list-style-type: none"> <li>Straightforward installation</li> </ul>	<ul style="list-style-type: none"> <li>Possible obstructions within fill which may affect driving</li> </ul>	<ul style="list-style-type: none"> <li>More expensive</li> </ul>	<ul style="list-style-type: none"> <li>Installation may be difficult if obstructions are encountered in the fill</li> </ul>	2

Timber lagging may be sized as per Table 20.12 of the CFEM, 5th edition (Section 20.8.9). This is provided so the center-to-center spacing of the soldier piles does not exceed 2.5 m. Soldier piles should extend a minimum depth of 3.0 m below the planned excavation depth. The actual depth of embedment should be determined by balancing moments about the pile tip. Excavation can proceed following installation of the soldier piles. The unshored height of the excavation should not exceed 1.2 m at any given time. No excavation height should remain unshored for more than 24 hours. Any loose zones from behind the shoring should be prevented during installation of the protection system. If required, backfill Granular A should be placed and compacted behind the shoring wall.

For the relatively shallow depth of excavation anticipated, cantilevered systems may be adequate. However, depending on the actual excavation depth, embedment depth (i.e., an embedded depth of sheet piles can be approximately 2.0 to 2.5 times of its exposed height), and shoring system used, additional anchorage or tiebacks may be required. This must be confirmed by the shoring designer. Conventional practice is to incorporate either buried deadman anchors, rakers or grouted soil anchors. Deadman anchors can be designed based on the earth pressure coefficients and soil parameters provided in Section 8.5.1. For this project, either continuous or individual concrete block anchors would likely be appropriate. The anchor resistance is provided by a combination of the dead weight and passive resistance. For the full passive resistance to be realized with no load transfer to the wall, the anchor needs to be fully beyond the active wedge acting on the wall. Pressure grouted soil anchors can be designed in a preliminary fashion in accordance with Section 20 of the CFEM (2023). For fill materials at this site, the estimated factored (0.4) ULS resistance of grouted anchors would be approximately 40 kN/m length. For the native Silty Sand to Sandy Silt till, the estimated factored (0.4) ULS resistance of grouted anchors would be approximately 90 kN/m length. Detailed design should be completed following the conception of the wall and when the associated loads have been established. Normally, such anchors are supplied and installed/tested by specialist vendors/contractors.

As can be seen in the table, the Soldier H-Pile and Lagging is ranked as more practical for this project due to possible obstructions that may be present within the fill layer. Design and construction specifications for the chosen roadway protection system should be prepared in accordance with OPSS. PROV 539. Piling should be in accordance with OPSS. PROV 903. Cantilevered walls should be designed for the earth pressure coefficients presented in Section 8.5.1 of this report and earth pressure diagram shown in CFEM Figure 20.14. Besides design and construction of the temporary protection system, the Contractor is also responsible for its materials, maintenance, monitoring and removal. According to OPSS 539, the protection system shall be removed from the right-of-way, unless it is specified in the Contract Documents that the protection system may be left in place. Where the piles are left in place, the top shall be removed at least 1.2 m below the finished grade level.

### 8.5.1 Lateral Earth Pressures

Temporary road protection systems should be designed to resist lateral earth pressure. The expression for calculating lateral earth pressure is given by:

$$P = K(\gamma h + q) \text{ for non-braced cut, or } K(0.65\gamma H + q) \text{ for braced support}$$

where:

P = earth pressure intensity at depth h, kPa

K = earth pressure coefficient

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

q = surcharge near wall, kPa

h = depth to point of interest, m

H = total depth of excavation, m

The above expression does not consider hydrostatic pressure, which must be included for the groundwater levels measured on the site. However, a properly designed and constructed soldier pile and lagging wall will be permeable and therefore hydrostatic pressure acting on the restrained height may be discounted. The surcharge should include soil loadings above the retained soil and other loadings adjacent to the wall.

For the design purposes, the unfactored static earth pressure parameters given in Table 2.10 can be used (assuming wall friction is neglected, the back wall is vertical, and the ground surface is horizontal both on the retained side as well as in front of the toe):

**Table 2.10: Material types and unfactored earth pressure properties under static conditions**

Abutment	Elevation	Material	Unfactored Friction Angle $\phi'$ (°)	Coefficient of Lateral Earth Pressure <sup>(1)</sup>			Unit Weight $\gamma$ (kN/m <sup>3</sup> )	GWL (m)
				(K <sub>a</sub> )	(K <sub>p</sub> )	(K <sub>o</sub> )		
West	186.0 to 176.4	Existing Fill	29	0.35	2.88	0.51	21	167.9
	176.4 to 164.0	Sandy Silt to Silty Sand Till (dense to very dense)	34	0.28	3.54	0.44	22	
East	186.3 to 176.7	Existing Fill	29	0.35	2.88	0.51	21	168.1
	176.7 to 164.0	Sandy Silt to Silty Sand Till (dense to very dense)	34	0.28	3.54	0.44	22	

Notes:

1.0  $K_a$  = active earth pressure coefficient;  $K_p$  = passive earth pressure coefficient;  $K_o$  = coefficient of earth pressure at rest

2.0 Assumes long term conditions. In short term conditions  $K_o = K_p = 1$

## 8.6 Lateral Earth Pressures for Design

### 8.6.1 Lateral Earth Pressures for Static Design

The lateral pressures acting on the abutment stems and retaining walls will depend on the type and method of placement of the backfill materials, on the nature of the soil behind the backfill, on the magnitude of surcharge including construction loadings, on the freedom of lateral movement of the structure, and on the drainage conditions behind the walls.

The following recommendations are provided concerning the design of the abutment walls or retaining walls in accordance with the CHBDC (2019). It should be noted that these design recommendations and parameters assume level backfill and ground surface behind the walls. Where there is sloping ground behind the wall, the coefficient of lateral earth pressure must be adjusted to account for the slope.

1. A compaction surcharge equal to 12 kPa should be included in the lateral earth pressures for the structural design in accordance with CHBDC S6-19 Figure 6.8.
2. If the wall support allows lateral and/or rotational yielding (unrestrained structure, such as typically the case for retaining walls), active earth pressures may be used in the geotechnical design of the structure. The granular fill should be placed in a wedged shaped zone (with a width equal to frost depth at the ground level in front of the wall) against a cut slope which begins at the footing level and extends upwards at a maximum inclination of 1 horizontal to 1 vertical (Case (b) from commentary on CHBDC S6-19 Figure C6.31). Case (a) implies to restrained walls such as boxes.
3. The mobilization of full active or passive resistance requires a measurable and perhaps significant wall movement or rotation. For active earth pressure, a rotation of 0.002 about the base of vertical walls (horizontal displacement divided

by wall height) or translation of 0.001 times wall height or a combination of these is required. Therefore, unless the structural element can tolerate these deflections, the at-rest earth pressure should be used in the design.

4. For walls backfilled using granular materials in accordance with Case (b), the parameters (unfactored) given in Table 2.11 may be assumed.

**Table 2.11: Material types and unfactored 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 ( $K_o$ )	Unit Weight $\gamma$ (kN/m <sup>3</sup> )
Compacted Granular A or Granular B Type II	35	0.27	3.69	0.43	22.8
Compacted Granular B Type I	32	0.31	3.25	0.47	21
Engineered Earth Fill	30	0.33	3.00	0.50	21

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 sloping backfill. For a 2 horizontal to 1 vertical (2H:1V) slope, the active earth pressure coefficients provided above should be adjusted by a factor of 1.5. The given values of active earth pressure coefficients depend on angles of friction and inclination. For preliminary design purposes, the adjustment for slopes between horizontal and 2H:1V may be linearly proportioned, however, some modification of the design pressures may be required depending on the backfill type and geometry. The coefficient of at-rest earth pressure for sloping granular backfill can be calculated using the equation (proposed by US Army Corps of Engineers, 1989):

$$K_o = (1 - \sin \phi') (1 + \sin \beta)$$

Where  $\beta$  is angle of sloping backfill above the horizontal.

## 8.6.2 Lateral Earth Pressures for Seismic Design

### 8.6.2.1 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 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.

For this site, a PGA of 0.176 g, 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. 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  $F(\text{PGA}) \cdot \text{PGA}$ , where,  $F(\text{PGA})$  is the PGA-based amplification factor that corresponds to the applicable Site Class as defined in Table 4.8 of the Code.

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 can be used (assuming wall friction is neglected, the back wall is vertical, and the ground surface is horizontal both on the retained side as well as in front of the toe):

**Table 2.12: Material types and earth pressure properties under seismic conditions for yielding walls**

Material	Unfactored Friction Angle $\phi'$ (°)	Coefficient of Seismic Earth Pressure - Active ( $K_{ae}$ )	Coefficient of Seismic Earth Pressure - Passive ( $K_{pe}$ )	Unit Weight $\gamma$ (kN/m <sup>3</sup> )
Compacted Granular A or Granular B Type II	35	0.32	3.52	22.8
Compacted Granular B Type I	32	0.36	3.09	21

### 8.6.2.2 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 a seismic horizontal acceleration coefficient ( $k_h$ ) equal to the site-adjusted PGA, where, the site-adjusted PGA estimated at the ground surface, given as  $F(\text{PGA}) \cdot \text{PGA}$ . The same values for  $F(\text{PGA})$  and PGA are used from Section 8.6.2.1. 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 calculation 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 2.13.

**Table 2.13: Material types and earth pressure properties under seismic conditions for non-yielding walls**

Material	Unfactored Friction Angle $\phi'$ (°)	Coefficient of Seismic Earth Pressure - Active ( $K_{ae}$ )	Coefficient of Seismic Earth Pressure - Passive ( $K_{pe}$ )	Unit Weight $\gamma$ (kN/m <sup>3</sup> )
Compacted Granular A or Granular B Type II	35	0.38	3.34	22.8
Compacted Granular B Type I	32	0.42	2.92	21

## 8.7 Construction Considerations

### 8.7.1 Excavation

All excavations must be carried out in accordance with the latest edition of the Ontario Occupational Health and Safety Act (OHSA) and good construction practice. The existing fills (i.e., uncontrolled fill) which should be excavated for the rehabilitation of the Birchmount Overpass structure are considered Type 3 soils above the groundwater table. The native soils which will be excavated for widening of the structure/embankments are considered Type 3 soils above the groundwater table and Type 4 soils below the groundwater table. Temporary excavations (i.e., those that are open only for a short period) in Type 3 and 4 soils above the groundwater table may be made with side slopes not steeper than about 1H:1V, while the temporary slopes below the groundwater table have to be formed at 3H:1V unless a suitable dewatering system is installed to lower the water level below the base of the excavation. Excavation for structures should be in accordance with OPSS.PROV 902 and SP109S12. The excavation should not undermine the existing walls.

### 8.7.2 Groundwater and Surface Water Control

As mentioned in Section 6.2, based on an assessment of the water levels observed in the borings and the subsurface conditions, the groundwater levels were interpreted to be about 17.9 m to 18.6 m below existing grade of Highway 401 with Elevations ranging between 167.4 m to 168.1 m across the Birchmount Road Overpass structure. Water may also be perched in the fill at higher levels during wet periods or in more permeable seams within the till deposits. Potential excavation depths to construct spread footings at the west abutment, east abutment and pier are expected to be carried to approximate Elev. 175.6 m, 176.1 m, and 176.6 m respectively. Therefore, it is anticipated that excavations will not extend into the groundwater table.

The soils encountered within potential excavation depths for the structures consist of existing silty sand to sand and silt and clayey silt fills, and native silty sand till. Given the conditions at this site, it is anticipated that control of seepage from water perched in the fill or in more permeable seams within the till deposit can be accomplished by conventional pumping from sumps in oversized excavations. This dewatering can likely be achieved by gravity drainage and pumping from strategically placed sumps with side ditches. Confirmation of control should be verified before general excavation to final levels.

Surface water should always be directed away from the excavation area(s). Dewatering/unwatering shall be carried out in accordance with OPSS.PROV 517 and SP517F01. It is the 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). Alternatively, and in accordance with SP 517F01, the dewatering systems may be completed by a design Engineer and design-checking Engineer with a minimum of 5 years' experience.

## 8.8 Corrosion Protection

Three (3) soil samples were selected for chemical analysis during current investigation performed by EXP. The testing was completed to determine the potential degradation of the concrete in the presence of soluble sulphates and the potential of corrosion of exposed steel used in foundations and buried infrastructure. The analyses results are summarized in Table 1.11 of this report.

The pH, resistivity, and chloride concentration provide an indication of the degree of corrosiveness of the sub-surface environment. The soil pH values measured at the site ranged between 7.87 to 10.1, which are within to above the normal range of soil pH of 5.5 to 8.5. A pH value of over 9 indicates a light potential for corrosion (Molinas and Mommandi, 2009). Additionally, the chemical data indicates low (<2000 ohm-cm) resistivity of tested soil, which suggests the severe potential for corrosion of buried metallic elements as per Table 3.2 of the MTO Gravity Pipe Design Guideline. Given the high pH and low resistivity of the soil, some level of corrosion protection for buried metallic elements is required, depending upon the material type (AASHTO,

2000/MTO Gravity Pipe Design Guidelines, April 2014). The measured chloride content was between 430 ppm ( $\mu\text{g/g}$ ) to 560 ppm which also indicates low potential for additional corrosion (Molinas and Mommandi, 2009).

Based on the results of the sample tested and given that the structure is located adjacent to the roadway and will expose to de-icing salt, consideration should be given by the designer to designing concrete for a « C » type of exposure class as defined by CSA A23.1 Table 1. Some level of corrosion protection for buried metallic elements is required, depending upon the material type. However, coating of steel H Piles is not done in general practice. It is up to the designer to determine the requirements of appropriate protective coating measures to ensure that all aspects of CHBDC 2019, Section 2 "Durability and Sustainability" requirements are followed. The test results provided in Table 1.X may be used to aid in the selection of coatings and corrosion protection systems for buried steel objects.

The maximum water-soluble sulphate content of the soils tested is less than 59 ppm ( $\mu\text{g/g}$ ), i.e., 0.0059% and being less than 0.10%, does not require sulphate resistant cement as per CSA A23.1 Table 3 "Additional requirements for concrete subjected to sulphate attack".

## 8.9 Obstructions

Although cobbles and boulders were not encountered during the site investigation, their presence should be anticipated in the native till. Additionally, although cobbles and boulders were not encountered in the fill, auger grinding experienced during drilling through the fill might reveal their presence. Therefore, care (i.e. pile flange reinforcement or be fitted with a driving shoe) has to be taken during the installation of elements of temporary protection systems or may also impact excavations. It is recommended that an NSSP be included in the Contract Documents to warn the Contractor of the possible presence of cobbles and/or boulders within the overburden soils. An example of an NSSP is included in Appendix K

## 8.10 Geotechnical Instrumentation and Monitoring

Monitoring of the effect of the construction for the rehabilitation of the existing structure should be conducted. Provided that the unwatering/dewatering (if any) and shoring are carried out in accordance with specifications and good practice, a significant impact on the existing bridge/walls foundation are not anticipated. However, monitoring of movements of the existing structure, shoring system and vibrations during rehabilitation of the structure is recommended.

The Geotechnical Instrumentation and Monitoring Plan (GIMP) shall include typical installation details, locations of installed instruments, and review procedures. Besides the existing structures, the monitoring of temporary protection systems, if any, should be performed in accordance with OPSS.PROV 539. Therefore, for this site the following elements of monitoring are anticipated:

### 8.10.1 Precondition and postcondition surveys

A precondition survey of all existing structures should be conducted prior to construction activities within the expected Zone of Influence with the goal of creating a baseline of pre-existing conditions and defects. Expected structures include the existing Highway 401 roadway and appurtenances including the pavement surface, traffic barriers, and overhead lighting, the existing Birchmount Road overpass structure, Birchmount Road including all appurtenances, and potential existing utility infrastructure.

The precondition survey should note the existing conditions of each structure, identifying existing wear-and-tear and potential deficiencies or defects. Documentation for each instance of a defect or deficiency should include the location, size, orientation, and any other relevant details. Photographic records for each occurrence is also required. The results shall be summarized and submitted as a precondition survey report. Upon review of the precondition survey report, additional monitoring, such as crack gauges, may be required.



Upon completion of the proposed works, a postcondition survey may be conducted as required to identify potential impacts on existing structures from the construction activities. A postconstruction report shall review the defects and deficiencies identified in the preconstruction survey and identify any new defects or deficiencies.

### 8.10.2 Movements of Existing Structure

Survey points should be used to monitor movements of the existing overpass structure (EBL and WBL). The monitoring plan will include the following:

- Install survey points along the existing bridge (min 6 m c/c) and the existing adjacent abutment and bridge deck (min 5m c/c).
- The location of survey points is to be coordinated with the construction team to prevent conflict during the proposed works.
- Monitoring frequency will be:
  - Preconstruction: Minimum 3 baseline readings, one month prior to construction
  - During construction: Daily readings during active construction.
  - Post construction: Biweekly after completion and then after four weeks, if there is little to no settlement continue surveying once a month for three months; or until the engineer is satisfied with performance.
- The criteria for evaluation of settlement shall be based on the following action levels:

#### Structure Limits:

1. Review Level: If a maximum value of 5 mm relative to the baseline readings is reached, the method and rate or sequence of construction shall be reviewed or modified to mitigate further ground displacements.
2. Alert Level: If a maximum value of 10 mm relative to the baseline readings is reached, the Contractor shall be required to cease construction operation or to execute pre-planned measures to secure the site to mitigate further unacceptable settlement and to assure safety of public.

#### Pavement Surface Limits:

1. Review Level: If a maximum deformation of 300 horizontal: 1 vertical relative to the baseline readings is reached, the method and rate or sequence of construction shall be reviewed or modified to mitigate further ground displacements.
2. Alert Level: If a maximum deformation of 150 horizontal: 1 vertical relative to the baseline readings is reached, the Contractor shall be required to cease construction operation or to execute pre-planned measures to secure the site to mitigate further unacceptable settlement and to assure safety of public.

### 8.10.3 Movements of Temporary Protection Systems

The minimum requirements for monitoring of temporary protection system should include the survey measurements of scaled targets attached to the shoring wall at the elevations specified. The scaled targets should be placed at a maximum spacing of 6 m with targets placed at the extreme ends and the targets distributed between the outer limits. The survey targets shall be monitored for horizontal displacement from the vertical at the frequency specified. The limit for horizontal deformation is 0.1% of the excavated height or a maximum horizontal displacement is 25 mm, and the limit of angular distortion is 1:200 (as per OPSS.PROV 539 Performance Level 2).

Shoring Limits shall follow OPSS.PROV 539, Performance Level 2:

1. Review Level: If a maximum value of 15 mm relative to the baseline readings is reached, the method and rate or sequence of construction shall be reviewed or modified to mitigate further ground displacements.



2. Alert Level: If a maximum of 25 mm relative to the baseline readings is reached, the Contractor shall be required to cease construction operation or to execute pre-planned measures to secure the site to mitigate further unacceptable settlement and to assure safety of public.

#### 8.10.4 Vibration

For bridge structures in good condition, OPSS.PROV 120 may be used to provide a limit of peak particle velocity (PPV), (noting that other entities having jurisdiction in particular settings may have more stringent regulations). Experience with monitoring of construction activities such as piling, drilling and hoe ramming have indicated that the noted threshold limit is not likely to be exceeded. However, it is recommended that site-personnel vibration monitoring takes place only during active construction of the temporary roadway protection systems.

The suggested vibration monitoring plan is described in the following.

1. The vibration monitoring should be conducted to verify the vibration levels near the existing structure and the utilities identified in the area.
2. No vibration monitoring is required for private or commercial building which is not present in the zone of influence for construction for this structure.
3. A normal background vibration reading produced by no construction related activities should be taken one month prior to construction activity.
4. Attended vibration monitoring can be conducted by a qualified technician during construction. The vibration monitoring program should include, monitoring with seismograph near the structure to confirm the magnitude of the vibration produced by construction activity. The seismograph consists of an ISEE geophone and base fitted with an internal battery can be considered. The qualified technician attended during construction activity should take readings from the seismograph and make notes of construction activities that produced the vibration events.
5. If excessive vibration levels were to be found, modifications to the construction techniques, potentially utilizing lighter or smaller equipment or less aggressive usage would be required.
6. Once construction activity is substantially complete, a final report should be prepared summarizing all vibration measurements made during that phase of construction.

The limits are as follows:

1. Review levels are any PPV of 15 mm/second at a frequency of 40 Hz or less OR a PPV of 40 mm/second at frequencies greater than 40 Hz.
2. Alert levels are any PPV of 20 mm/second at a frequency of 40 Hz or less OR a PPV of 50 mm/second at frequencies greater than 40 Hz.

*Foundation Investigation and Design Report  
Highway 401 Eastbound from Victoria Park Avenue to Neilson Road  
Superstructure Replacement at Birchmount Road Overpass  
Eastbound Core and Collectors Structure (Site 37X-0218/B1 & B3)  
Assignment No. 2021-E-0018  
Date: December 31, 2024*

## 9.0 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.

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


Details of the limitations of this report are presented as Appendix A, "Limitations and Use of Report".

This Foundation Investigation Design Report has been prepared by Elvis Lu, M.Eng., EIT and Thomas Lardner, Ph.D., P.Eng. It was reviewed by TaeChul Kim, M.E.Sc., P.Eng. and Stan E. Gonsalves, M.Eng., P.Eng., Designated MTO Foundation Contact.


Yours truly,


EXP Services Inc.

Elvis Lu, M.Eng., EIT  
Technical Specialist

  
Thomas Lardner, Ph.D., P.Eng.  
Geotechnical Engineer



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

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



Encl.

## References

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- Canadian Geotechnical Society, 2006. *Canadian Foundation Engineering Manual*, 4th Edition. The Canadian Geotechnical Society, BiTech Publisher Ltd., British Columbia.
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- Molinas, A., and Mommandi, A., 2009. Development of New Corrosion/Abrasion Guidelines for Selection of Culvert Pipe Materials, Report No. CDOT-2009-11. Colorado Department of Transportation, DTD Applied Research and Innovation Branch.
- Ontario Geological Survey. 1991. *Bedrock geology of Ontario, southern sheet*; Ontario Geological Survey, Map 2544, scale 1:1 000 000.
- Preliminary Foundation Investigation and Design Report, Bridge Widening and Replacement, Highway 401 Rehabilitation from Warden Avenue to Brock Road, Toronto, Ontario, W.O. 07-20012, Report Number: 09-1111-6055-1, Geocres No. 30M14-338, Golders Associates Ltd., dated April 2012.
- US Army Corps of Engineers, Engineering and Design Manual for Retaining and Flood Walls, 29 September 1989.

## Appendix A – Limitations and Use of Report



## **LIMITATIONS AND USE OF REPORT**

### **BASIS OF REPORT**

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

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

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

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

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

### **RELIANCE ON INFORMATION PROVIDED**

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

### **STANDARD OF CARE**

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

### **COMPLETE REPORT**

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



## **USE OF REPORT**

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

## **REPORT FORMAT**

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

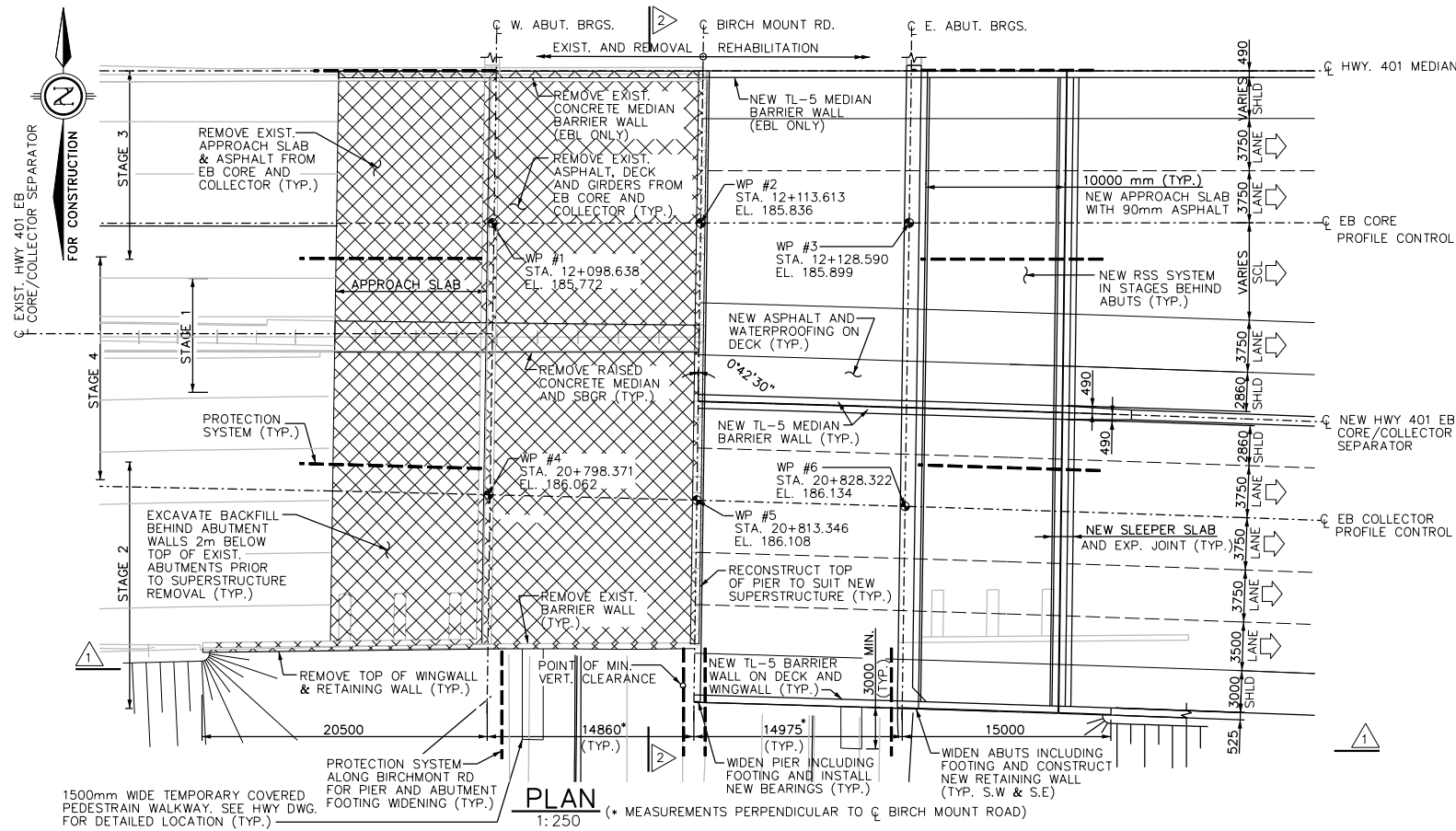
## Appendix B – General Arrangement Drawings

CADD FILE NAME : C:\Users\jia.zheng\Desktop\Hwy401\_Victoria to Nacah\Site 37X-0212-B1&B3\_Birchmount Rd OP\B4-01\_EBL\_Birchmount Rd.gxd

2017-08

ANS-D

MINISTRY OF TRANSPORTATION, ONTARIO



NORTH

SOUTH

METRIC  
DIMENSIONS ARE IN METRES AND/OR  
MILLIMETRES UNLESS OTHERWISE SHOWN  
DRAWING NOT TO BE SCALED  
100mm ON ORIGINAL DRAWING

Ontario Ministry of Transportation



CONT  
WP

HWY 401 EB CORE & COLLECTOR  
BIRCHMOUNT RD. OVERPASS  
GENERAL ARRANGEMENT

SHEET  
S1

AECOM

#### GENERAL NOTES:

- SPECIFIED 28-DAY COMPRESSIVE STRENGTH.....30 MPa  
UNLESS NOTED OTHERWISE  
SPECIFIED 28-DAY COMPRESSIVE STRENGTH FOR PRECAST  
GIRDERS ARE GIVEN ON PRESTRESSED GIRDER DRAWINGS.
- CLEAR COVER TO REINFORCING STEEL  
FOOTING.....100 ±20  
DECK TOP.....70 ±20  
BOTTOM.....40 ±10  
PIER COLUMNS, SHAFTS AND CAPS.....70 ±20  
UNLESS NOTED OTHERWISE.
- REINFORCING STEEL:
  - REINFORCING STEEL SHALL BE GRADE 500W UNLESS  
OTHERWISE SPECIFIED.
  - BAR MARKS WITH PREFIX 'S' DENOTE STAINLESS STEEL  
BARS.
  - STAINLESS REINFORCING STEEL SHALL BE TYPE 316LN  
OR DUPLEX 2205 AND HAVE MINIMUM YIELD STRENGTH  
OF 500 MPa.
  - BAR HOOKS SHALL HAVE STANDARD HOOK DIMENSIONS  
USING MINIMUM BEND DIAMETERS, WHILE STIRRUPS AND  
TIES SHALL HAVE MINIMUM HOOK DIMENSIONS.  
ALL HOOKS SHALL BE IN ACCORDANCE WITH THE  
STRUCTURAL STANDARD DRAWINGS SS12-1 UNLESS  
INDICATED OTHERWISE.
  - UNLESS SHOWN OTHERWISE TENSION LAP SPLICES SHALL  
BE CLASS B.

#### CONSTRUCTION NOTES:

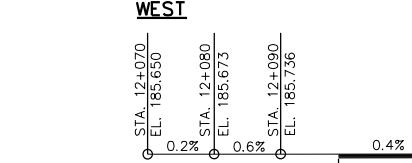
- THE CONTRACTOR SHALL VERIFY ALL RELEVANT DIMENSIONS,  
ELEVATIONS AND DETAILS ON-SITE AND REPORT ANY  
DISCREPANCIES TO THE CONTRACT ADMINISTRATOR PRIOR TO  
PROCEEDING WITH REHABILITATION WORK.
- TYPICAL AREAS OF REPAIRS ARE INDICATED ON THE DRAWINGS.  
WHERE REPAIR LIMITS ARE NOT SHOWN, LIMITS SHALL BE  
IDENTIFIED BY THE CONTRACT ADMINISTRATOR.
- THE CONTRACTOR SHALL ADJUST THE BEARING SEAT  
ELEVATIONS AND REINFORCING STEEL TO SUIT THE ACTUAL  
HEIGHT OF THE BEARING SUPPLIED. THE CONTRACTOR IS  
RESPONSIBLE FOR PROVIDING FULL BEARING CONTACT TO  
GIRDER SOFFIT AND BEARING SEAT. ADDITIONAL COST DUE  
TO ANY CHANGES IN ELEVATIONS OF THE TOP OF BEARINGS  
BY THE CONTRACTOR SHALL BE AT HIS OWN EXPENSE.
- PROTECTION SYSTEM SHALL MEET REQUIREMENTS FOR  
PERFORMANCE LEVEL 2. EXACT LOCATIONS AND LIMITS OF  
PROTECTION SYSTEM SHALL BE DETERMINED BY CONTRACTOR.
- BACKFILL SHALL NOT BE PLACED BEHIND THE NEW  
SEMI-INTEGRAL ABUTMENTS UNTIL THE NEW CONCRETE HAS  
ACHIEVED 75% OF DESIGN COMPRESSIVE STRENGTH.
- SAWCUT IN CONCRETE, WHERE DESIGNATED, SHALL BE 25mm  
DEEP OR TO THE FIRST LAYER OF REINFORCING STEEL,  
WHICHEVER IS LESS.
- ANY DAMAGE DURING CONSTRUCTION TO THE EXISTING  
STRUCTURES UTILITIES AND ADJACENT PROPERTIES NOT  
DESIGNATED FOR REPAIR SHALL BE REPAIRED GOOD BY THE  
CONTRACTOR TO THE SATISFACTION OF THE CONTRACT  
ADMINISTRATOR AND AT NO COST TO THE OWNER.
- THE CONTRACTOR IS FULLY RESPONSIBLE FOR ADEQUATE  
PROTECTION OF ALL UTILITIES, SERVICES, ROADWAYS, ETC.,  
DURING CONSTRUCTION OPERATIONS.
- THE CONTRACTOR SHALL PROVIDE DEBRIS PLATFORMS AND  
NECESSARY CONTAINMENT MEASURES TO COLLECT FALLING  
CONCRETE AND CONSTRUCTION DEBRIS SUCH THAT NO DEBRIS  
OR MATERIALS RESULTING FROM THE REMOVAL WORK FALLS IN  
AREAS BELOW THE BRIDGE.
- THE CONTRACTOR SHALL NOT REMOVE THE EXISTING  
SUPERSTRUCTURE WITHIN EACH STAGE UNTIL EXISTING  
APPROACH SLABS AND BACKFILL BEHIND BOTH ABUTMENTS  
ARE REMOVED TO THE SPECIFIED DEPTH. BACKFILL SHALL BE  
REMOVED SIMULTANEOUSLY BEHIND BOTH ABUTMENTS KEEPING  
THE HEIGHT OF BACKFILL APPROXIMATELY THE SAME. AT NO  
TIME SHALL THE DIFFERENCE IN ELEVATION BE GREATER THAN  
300mm.
- BACKFILL SHALL BE PLACED SIMULTANEOUSLY BEHIND BOTH  
DECK ENDS KEEPING THE HEIGHT OF THE BACKFILL  
APPROXIMATELY THE SAME. AT NO TIME SHALL THE  
DIFFERENCE IN ELEVATION BE GREATER THAN 300mm.

#### LEGEND:

	EXIST. CONCRETE TO REMAIN		NEW CONCRETE
	REMOVAL		NEW ASPHALT

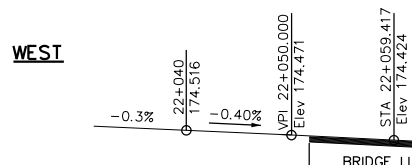
#### PROFILE OF BIRCHMOUNT RD

N.T.S.



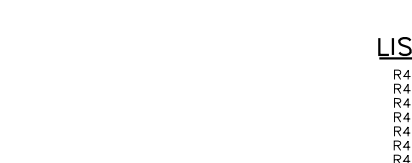
#### PROFILE OF HWY 401 C/MEDIAN

N.T.S.



#### PROFILE OF HWY 401 E.B. COLLECTOR

N.T.S.



#### LIST OF DRAWINGS:

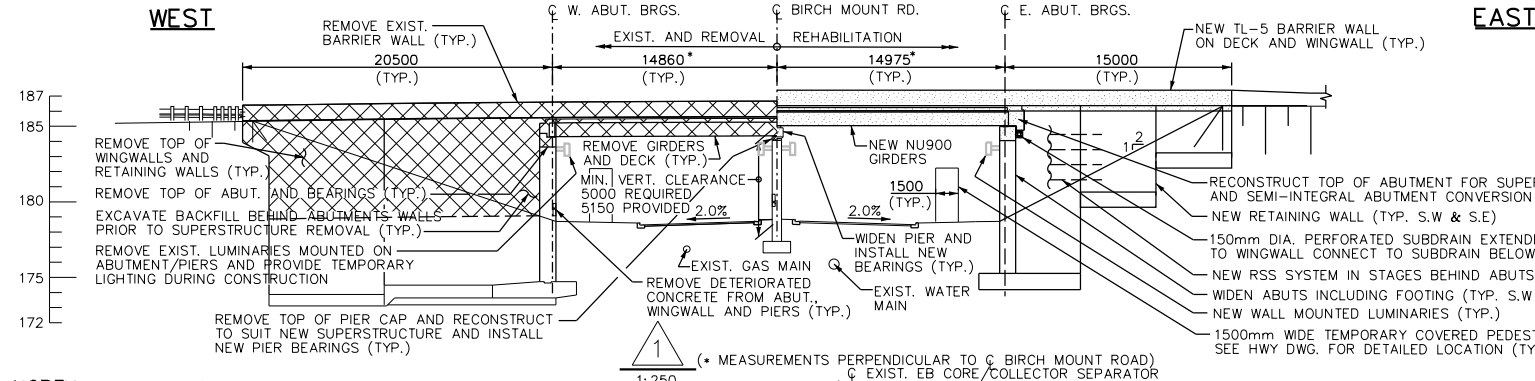
- R4-01. GENERAL ARRANGEMENT
- R4-02. BOREHOLE LOCATIONS
- R4-03. SOIL STRATA
- R4-04. CONSTRUCTION STAGING - 1
- R4-05. CONSTRUCTION STAGING - 2
- R4-06. ABUTMENT REMOVALS - I
- R4-07. ABUTMENT REMOVALS - II
- R4-08. PIER REMOVALS
- R4-09. FOUNDATION LAYOUT AND FOOTING REINFORCEMENT
- R4-10. ABUTMENT REHABILITATIONS - I
- R4-11. ABUTMENT REHABILITATIONS - II
- R4-12. WINGWALL AND RETAINING WALL DETAILS
- R4-13. PIER REHABILITATIONS
- R4-14. NU GIRDERS AND BEARINGS
- R4-15. DECK LAYOUT AND SCREED ELEVATIONS
- R4-16. DECK DETAILS AND REINFORCEMENT - 1
- R4-17. DECK DETAILS AND REINFORCEMENT - 2
- R4-18. SOUTH BARRIER WALL WITHOUT RAILING TL-5
- R4-19. MEDIAN BARRIER WALL WITHOUT RAILING TL-5
- R4-20. 10000mm APPROACH SLAB
- R4-21. EXPANSION JOINT AND SLEEPER SLAB
- R4-22. STRIP SEAL EXPANSION JOINT FOR SLEEPER SLAB
- R4-23. SEQUENCE OF EXPANSION JOINT INSTALLATION
- R4-24. MISCELLANEOUS AND STANDARD DETAILS
- R4-25. ELECTRICAL EMBEDDED WORKS

#### LIST OF ABBREVIATIONS

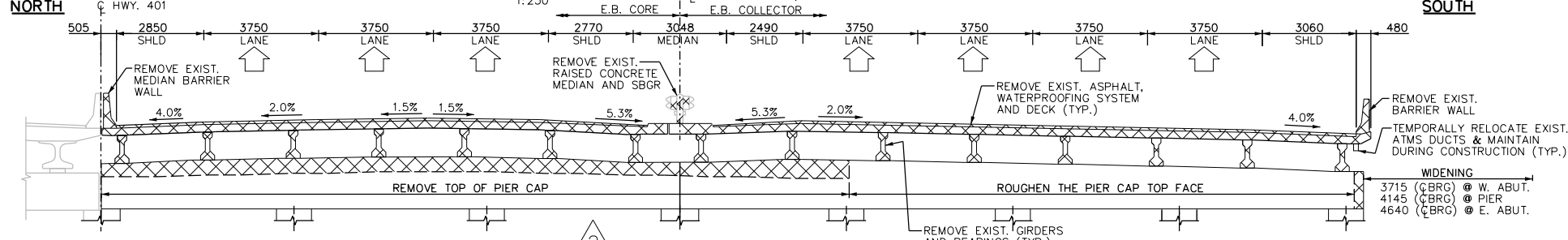
ABUT.	DENOTES ABUTMENT
BRGS.	DENOTES BEARINGS
C.J.	DENOTES CONSTRUCTION JOINT
DIA.	DENOTES DIAMETER
E.	DENOTES EAST
EB	DENOTES EAST BOUND
EL.	DENOTES ELEVATION
EXIST.	DENOTES EXISTING
EXP.	DENOTES EXPANSION
HWY	DENOTES HIGHWAY
SBGR	DENOTES STEEL BEAM GUIDE RAIL
S.E.	DENOTES SOUTH EAST
SHLD.	DENOTES SHOULDER
STA.	DENOTES STATION
S.W.	DENOTES SOUTH WEST
TYP.	DENOTES TYPICAL
W.	DENOTES WEST
WP.	DENOTES WORKING POINT

#### APPLICABLE STANDARD DRAWINGS:

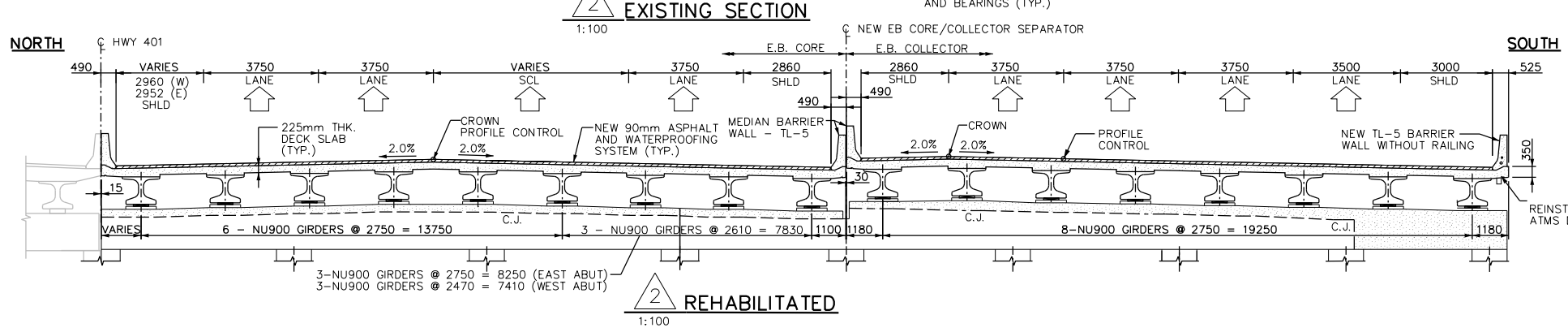
- OPSD 0914.430 GUIDE RAIL SYSTEM, STEEL BEAM STRUCTURE CONNECTION
- OPSD 3370.100 DECK WATERPROOFING, HOT APPLIED ASPHALT MEMBRANE WITH PROTECTION BOARD DETAILS
- OPSD 3370.101 DECK WATERPROOFING, HOT APPLIED ASPHALT MEMBRANE AT ACTIVE CRACKS GREATER THAN 2mm WIDE AND CONSTRUCTION JOINTS
- OPSD 3390.150 FALSEWORK CLEARANCE TO TRAFFIC LANES
- OPSD 3419.100 BARRIER AND RAILINGS STEEL BEAM GUIDE RAIL AND CHANNEL ANCHORAGE
- OPSD 3941.200 FIGURES IN CONCRETE SITE NUMBER AND DATE LAYOUT



EAST



SOUTH



#### EXISTING SECTION

1:100

#### REHABILITATED

1:100

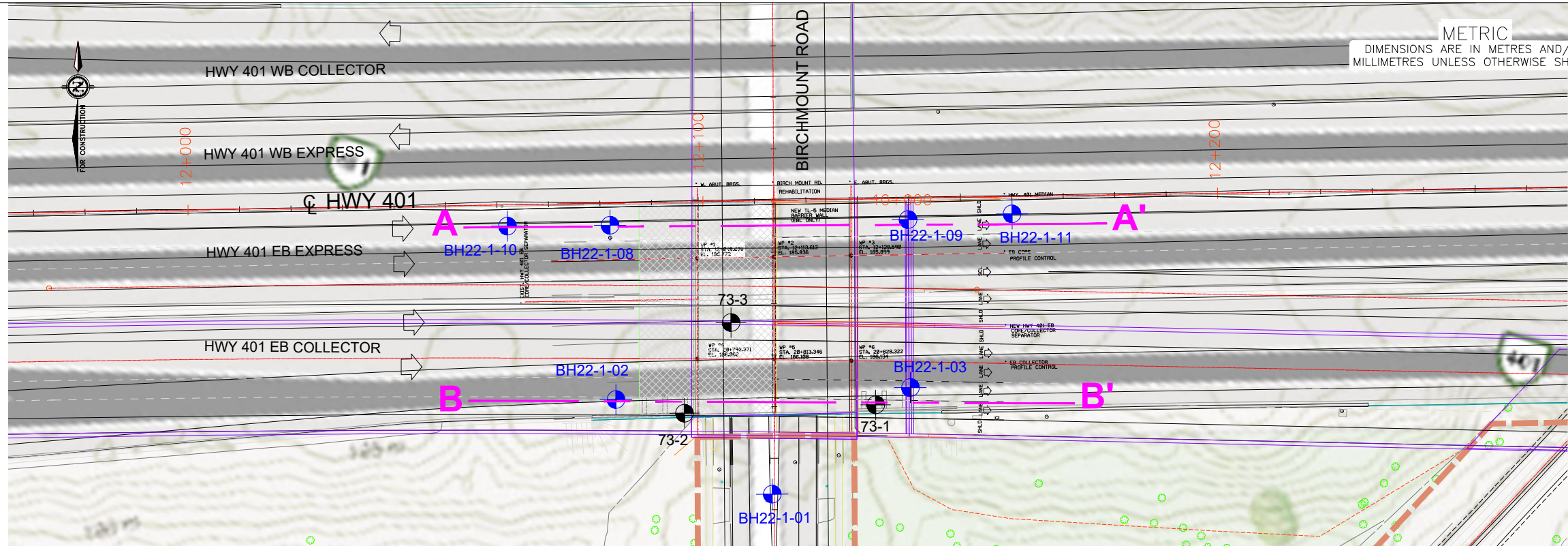
REVISIONS

DATE	BY	DESCRIPTION
DESIGN J.C.	CHK U.P.	CODE CAN/CSA S6-19
DRAWN T.K/O.Z.	CHK J.C.	SITE 37X-0212/B1&B3
LOAD CL 625-ONT		DATE OCT. 2024
DWG R4-01		

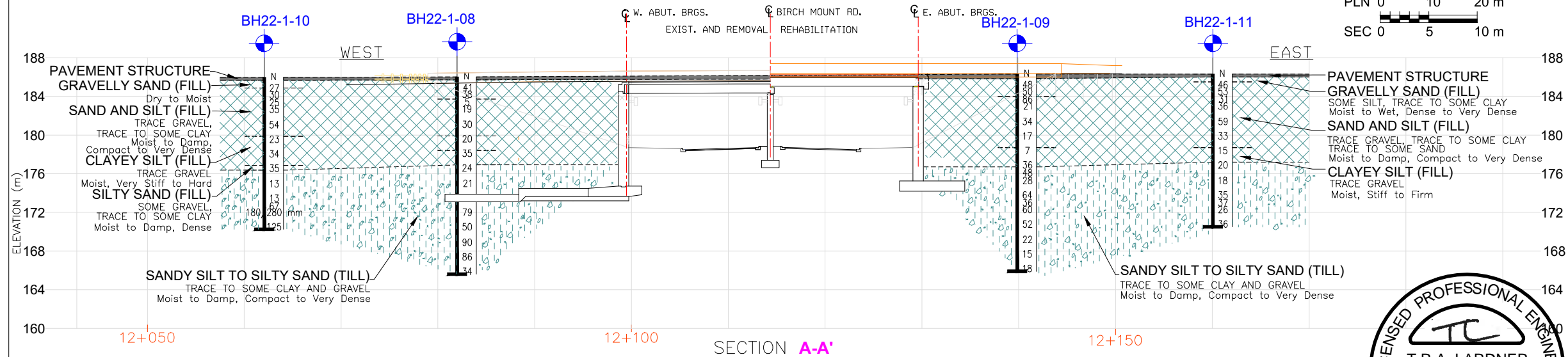


## Appendix C – Borehole Location Plan and Stratigraphic Profile

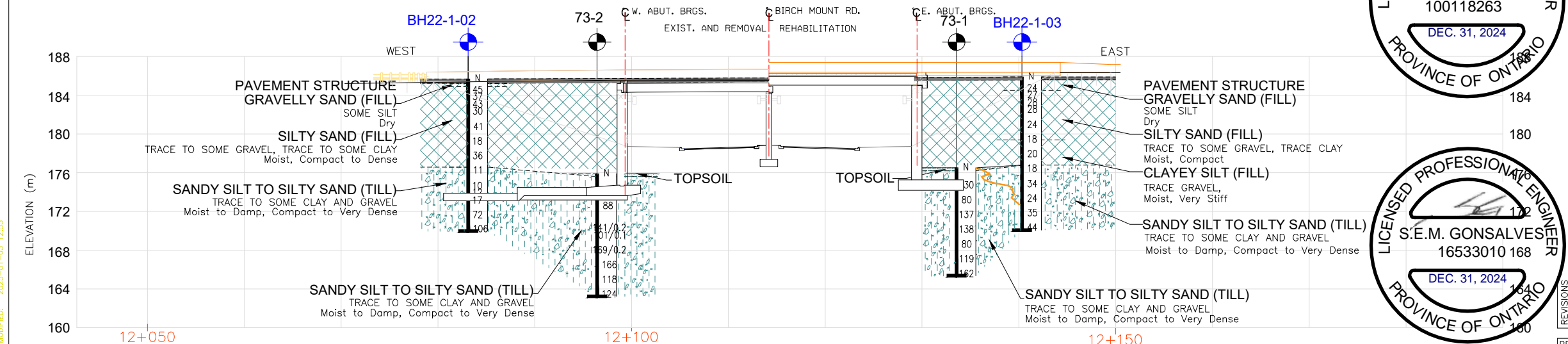
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MODIFIED: 2025-01-03 12:55



PLAN



SECTION A-A'



SECTION B-B'

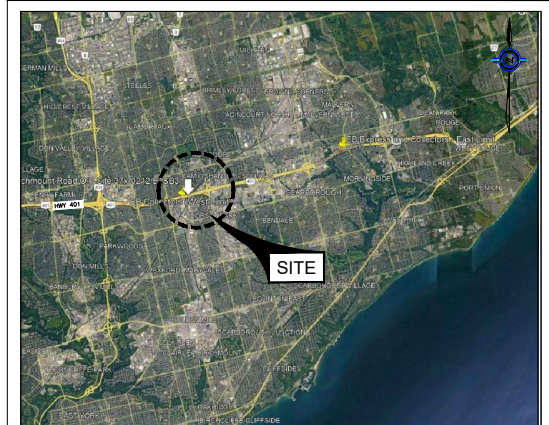
CONT No.  
ASSIG No. 2021-E-0018  
GWP No.

HIGHWAY 401 EB CORE & COLLECTOR  
BIRCHMOUNT RD. OVERPASS  
Latitude: 43.772525°, Longitude: -79.294316°  
BOREHOLE LOCATION PLAN & SOIL STRATA

SHEET  
1

exp.

EXP SERVICES INC.



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

PAVEMENT STRUCTURE	SILT AND SAND	CLAY
FILL	SANDY SILT	CLAYEY SILT
SILT	SILTY SAND	SILTY CLAY
SAND	SANDY SILT TO SILTY SAND (TILL)	CLAYEY SILT TO SILTY CLAY (TILL)

BOREHOLE CO-ORDINATES/ NAD 83/ MTM ON-10			
BH No.	ELEV.	NORTHING	EASTING
BH22-1-01	180.0	4848043	321370
BH22-1-02	185.7	4848052	321336
BH22-1-03	185.9	4848071	321390
BH22-1-08	186.0	4848084	321325
BH22-1-09	186.3	4848102	321380
BH22-1-10	186.0	4848078	321306
BH22-1-11	186.3	4848109	321399
73-1	176.5	4848066	321385
73-2	175.9	4848053	321350
73-3	176.8	4848073	321353

BOREHOLE CO-ORDINATES/ NAD 83/ MTM ON-10			
BH No.	ELEV.	NORTHING	EASTING
BH22-1-01	180.0	4848043	321370
BH22-1-02	185.7	4848052	321336
BH22-1-03	185.9	4848071	321390
BH22-1-08	186.0	4848084	321325
BH22-1-09	186.3	4848102	321380
BH22-1-10	186.0	4848078	321306
BH22-1-11	186.3	4848109	321399
73-1	176.5	4848066	321385
73-2	175.9	4848053	321350
73-3	176.8	4848073	321353

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 O.P.S. Gen. Cond.

SUBMISSION FOR MTO REVIEW			
NO	DATE	BY	DESCRIPTION
PROJECT No.	ADM-22000797-A0	GEOCREs No.	-
SUBM'D SH	CHKD. SM	DATE	JAN. 06, 2025
DRAWN SH	CHKD. TC	APPRD SG	SITE 37X-0212/B1 & B3
			DWG 01





## Appendix D – 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.

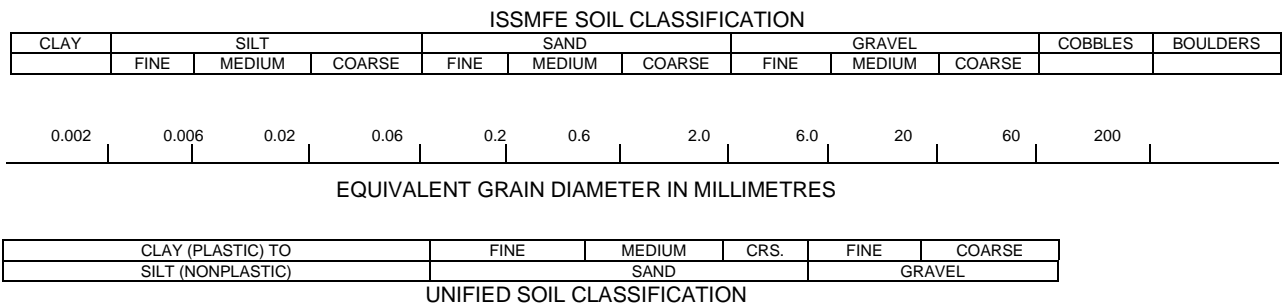
*Seam:* a thin, confined layer of soil having different particle size, texture, or color from materials above and below.

*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:

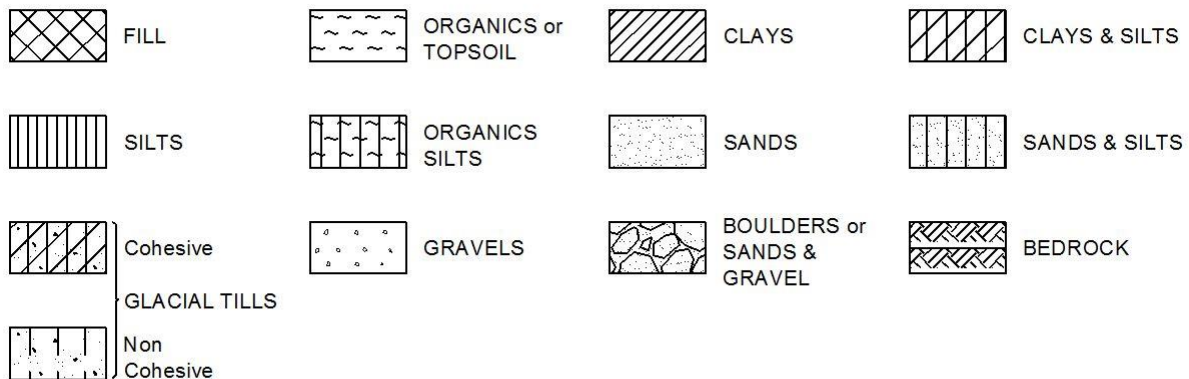
Table c: Consistency of Cohesive Soil

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

1 OF 1

**METRIC**

W.P.	Site 37X-0212/B1&B3	LOCATION	Hwy 401 - Birchmount Road O/P, Toronto, ON, MTM ON-10 321370.4E 4848043.3N			ORIGINATED BY	CA		
DIST	Toronto	HWY	401	BOREHOLE TYPE	Truck Mount MARL M10 / SSA			COMPILED BY	CA
DATUM	Geodetic	DATE	2023.03.20 - 2023.03.20	LATITUDE	43.772165	LONGITUDE	-79.294168	CHECKED BY	TL

SOIL PROFILE			SAMPLES		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES
180.0	<b>PAVEMENT STRUCTURE</b> - 25 mm of asphalt, and 230 mm of concrete <b>SAND AND GRAVEL (FILL)</b> - trace to some silt, grey, moist <b>SILTY SAND TO SAND AND SILT (TILL)</b> - trace to some gravel, trace to some clay, brown to brownish grey, moist, compact to very dense		AS1	AS	
179.6			SS2	SS	79
179.3			SS3	SS	35
0.4			SS4	SS	37
			SS5	SS	44
			SS6	SS	26
			SS7	SS	97
	- brownish grey below ~ 9.1 m depth		SS9	SS	27
170.4	<b>END OF BOREHOLE</b>				
9.6	NOTE: 1) No groundwater was encountered in open borehole upon completion of drilling.				

DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT		NATURAL MOISTURE CONTENT		LIQUID LIMIT		UNIT WEIGHT  γ  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)						
SHEAR STRENGTH kPa		w <sub>p</sub>		w		w <sub>L</sub>			WATER CONTENT (%)						
○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE								GR SA SI CL							
20	40	60	80	100	20	40	60								
					</										

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

ONTARIO MTO H401 - BIRCHMOUNT - 07022023.GPJ ONTARIO MTO.GDT 1/22/24

Brampton, Ontario

## RECORD OF BOREHOLE No BH22-1-02

1 OF 1

METRIC

W.P. Site 37X-0212/B1&B3 LOCATION Hwy 401 - Birchmount Road O/P, Toronto, ON, MTM ON-10 321336.0E 4848051.9N ORIGINATED BY SF/OD  
 DIST Toronto HWY 401 BOREHOLE TYPE Truck Mount CME 75 / SSA COMPILED BY SF/OD  
 DATUM Geodetic DATE 2022.11.28 - 2022.11.28 LATITUDE 43.772243 LONGITUDE -79.294595 CHECKED BY TL

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	60						80	100	20
185.7	<b>PAVEMENT STRUCTURE</b> - 200 mm of asphalt, and 260 mm of concrete  <b>GRAVELLY SAND (FILL)</b> - some silt, brown, moist <b>SILTY SAND (FILL)</b> - trace to some gravel, trace to some clay, brown, moist, compact to dense  <																	

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

ONTARIO MTO H401 - BIRCHMOUNT - 07022023.GPJ ONTARIO MTO.GDT 1/22/24

Brampton, Ontario

## RECORD OF BOREHOLE No BH22-1-03

1 OF 1

METRIC

W.P. Site 37X-0212/B1&B3 LOCATION Hwy 401 - Birchmount Road O/P, Toronto, ON, MTM ON-10 321390.0E 4848071.2N ORIGINATED BY SF/OD  
 DIST Toronto HWY 401 BOREHOLE TYPE Truck Mount CME 75 / SSA COMPILED BY SF/OD  
 DATUM Geodetic DATE 2022.12.12 - 2022.12.12 LATITUDE 43.772415 LONGITUDE -79.293924 CHECKED BY TL

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									WATER CONTENT (%)			GR	SA	SI	CL
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL	× LAB VANE												
185.9																							
186.0																							
0.3	PAVEMENT STRUCTURE - 90 mm of asphalt, and 210 mm of concrete		AS1	AS																			
	GRAVELLY SAND (FILL) - some silt, brown, moist, compact		SS2	SS	24																		
184.5																							
1.4	SILTY SAND (FILL) - trace to some gravel, trace clay, light brown to light grey, slightly moist, compact		SS3	SS	27																		
			SS4	SS	28																		
			SS5	SS	28																		
			SS6	SS	24																		
179.5			SS7	SS	18																		
6.4	CLAYEY SILT (FILL) - sandy, trace gravel, light brown to grey, moist, very stiff																						
			SS8	SS	20																		
176.8			SS9	SS	18																		
9.1	SILTY SAND TO SAND AND SILT (TILL) - trace to some gravel, trace to some clay, light brown to grey, moist, compact to dense																						
	- becoming dark brown to grey below a depth of 10.7 m		SS10	SS	34																		
			SS11	SS	24																		
			SS12	SS	35																		

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

ONTARIO MTO H401 - BIRCHMOUNT - 07022023.GPJ ONTARIO MTO.GDT 1/22/24

Brampton, Ontario

## RECORD OF BOREHOLE No BH22-1-08

1 OF 1

METRIC

W.P. Site 37X-0212/B1&B3 LOCATION Hwy 401 - Birchmount Road O/P, Toronto, ON, MTM ON-10 321324.7E 4848084.4N ORIGINATED BY SF/OD  
 DIST Toronto HWY 401 BOREHOLE TYPE Truck Mount CME 75 / SSA COMPILED BY SF/OD  
 DATUM Geodetic DATE 2022.11.02 - 2022.11.03 LATITUDE 43.772535 LONGITUDE -79.294734 CHECKED BY TL

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									WATER CONTENT (%)			GR	SA	SI	CL
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE															
186.0								20	40	60	80	100											
186.0	PAVEMENT STRUCTURE - 80 mm of asphalt, and 220 mm of concrete GRAVELLY SAND (FILL) - brown, moist, dense		AS1	AS			185							○									
0.3			SS2	SS	41										○								
			SS3	SS	38										○								
183.7	SILTY SAND (FILL) - trace to some clay, trace gravel, brown, damp to moist, loose to dense		SS4	SS	5		184							○									
2.3			SS5	SS	19		183							○									
							182																
			SS6	SS	30		181								○								
179.9	CLAYEY SILT (FILL) - sandy, trace gravel, brown to grey, moist, very stiff		SS7	SS	20		180							○									
6.1							179								○								
178.4	SILTY SAND (FILL) - trace to some clay, trace gravel, brown, damp to moist, dense		SS8	SS	35		178							○									
7.6							177								○								
176.9	SILTY SAND TO SAND AND SILT (TILL) - trace to some gravel, trace to some clay, brown to grey to black, damp to wet, compact to very dense  - trace organics encountered at a depth of ~11.0 m		SS9	SS	24		176							○									
9.1							175								○								
			SS10	SS	21		174								○								
							173								○								
			SS11	SS	94		172								○								
							171								○								
			SS12	SS	79		170																
							169																
			SS13	SS	50		168									○							
							167									○							
165.6	END OF BOREHOLE		SS14	SS	90		166							○									
20.4																							
	NOTE: 1) Groundwater inferred at a depth of 18.6 m based on wet split spoon retrieved during drilling.																						

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

ONTARIO MTO H401 - BIRCHMOUNT - 07022023.GPJ ONTARIO MTO.GDT 1/22/24

Brampton, Ontario

## RECORD OF BOREHOLE No BH22-1-09

1 OF 1

METRIC

W.P. Site 37X-0212/B1&B3 LOCATION Hwy 401 - Birchmount Road O/P, Toronto, ON, MTM ON-10 321380.1E 4848102.3N ORIGINATED BY SF/OD  
 DIST Toronto HWY 401 BOREHOLE TYPE Truck Mount CME 75 / SSA COMPILED BY SF/OD  
 DATUM Geodetic DATE 2022.11.07 - 2022.11.08 LATITUDE 43.772695 LONGITUDE -79.294045 CHECKED BY TL

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT NATURAL MOISTURE CONTENT			UNIT WEIGHT  <b>γ</b>  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)									
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				W <sub>P</sub>	W	W <sub>L</sub>		WATER CONTENT (%)									
								○ UNCONFINED	+	FIELD VANE	● QUICK TRIAXIAL	×	LAB VANE												
186.3							20	40	60	80	100		20	40	60		GR	SA	SI	CL					
0.0	PAVEMENT STRUCTURE - 150 mm of asphalt and 310 mm of concrete		AS1	AS									○												
185.8			SS2	SS	48									○											
0.5			SS3	SS	50										○										
184.0	GRAVELLY SAND (FILL) - some silt, trace clay, brown to grey, moist to wet, dense to very dense		SS4	SS	86								○												
2.3			SS5	SS	21									○											
			SS6	SS	34										○										
			SS7	SS	17										○										
178.7	CLAYEY SILT (FILL) - sandy, trace gravel, brown to dark grey, moist, firm to hard		SS8	SS	7								○												
7.6																									
			SS9	SS	36										○										
176.7	SILTY SAND TO SAND AND SILT (TILL) - trace to some gravel, trace to some clay, brown to dark grey, moist to wet, compact to very dense		SS10	SS	48								○				22.0								
9.6			SS11	SS	28										○				21.0						
			SS12	SS	64										4-1				23.5						
			SS13	SS	36										○										
			SS14	SS	60										○										
			SS15	SS	52										4-1				23.2						
			SS16	SS	22										○				23.4						

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

Brampton, Ontario

## RECORD OF BOREHOLE No BH22-1-10

1 OF 1

METRIC

W.P. Site 37X-0212/B1&B3 LOCATION Hwy 401 - Birchmount Road O/P, Toronto, ON, MTM ON-10 321305.8E 4848077.9N ORIGINATED BY SF/OD  
 DIST Toronto HWY 401 BOREHOLE TYPE Truck Mount CME 75 / SSA COMPILED BY SF/OD  
 DATUM Geodetic DATE 2022.11.15 - 2022.11.15 LATITUDE 43.772477 LONGITUDE -79.294969 CHECKED BY TL

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W <sub>P</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)		
								○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE × LAB VANE									
186.0							20	40	60	80	100	20	40	60				
186.0	PAVEMENT STRUCTURE - 100 mm of asphalt, and 200 mm of concrete		AS1	AS														
0.3	GRAVELLY SAND (FILL) - brown, moist		SS2	SS	27													
184.9	SAND AND SILT (FILL) - trace gravel, trace clay, brown, moist, compact to very dense		SS3	SS	30													
1.1	- Thin layers of clayey silt were encountered between a depth of 2.3 m and 3.0 m.		SS4	SS	25													
			SS5	SS	35													
			SS6	SS	54													
179.9	CLAYEY SILT (FILL) - sandy, trace gravel, brown to grey, moist, very stiff to hard		SS7	SS	23													
6.1																		
			SS8	SS	34													
176.9	SAND AND SILT (FILL) - some gravel, trace to some clay, brown, moist, dense		SS9	SS	35													
9.1	SILTY SAND TO SAND AND SILT (TILL) - trace to some gravel, trace to some clay, brown to brownish grey, moist, compact to very dense																	
176.4			SS10	SS	13													
9.6																		
			SS11	SS	13													
			SS12	SS	67													
			SS13	SS	180/ 280 mm													
170.3	END OF BOREHOLE		SS14	SS	125													
15.7	NOTE: 1) No groundwater was encountered in open borehole upon completion of drilling.																	

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

ONTARIO MTO H401 - BIRCHMOUNT - 07022023.GPJ ONTARIO MTO.GDT 1/22/24

Brampton, Ontario

## RECORD OF BOREHOLE No BH22-1-11

1 OF 1

METRIC

W.P. Site 37X-0212/B1&B3 LOCATION Hwy 401 - Birchmount Road O/P, Toronto, ON, MTM ON-10 321399.1E 4848108.6N ORIGINATED BY SF/OD  
 DIST Toronto HWY 401 BOREHOLE TYPE Truck Mount CME 75 / SSA COMPILED BY SF/OD  
 DATUM Geodetic DATE 2022.11.09 - 2022.11.09 LATITUDE 43.772752 LONGITUDE -79.293809 CHECKED BY TL

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT   NATURAL MOISTURE CONTENT   LIQUID LIMIT			UNIT WEIGHT  γ  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)							
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		W <sub>P</sub>	W	W <sub>L</sub>		WATER CONTENT (%)				GR	SA	SI	CL
								○ UNCONFINED   + FIELD VANE	● QUICK TRIAXIAL   × LAB VANE												
186.3								20	40	60	80	100									
186.0	PAVEMENT STRUCTURE - 90 mm of asphalt, and 210 mm of concrete		AS1	AS																	
0.3	GRAVELLY SAND (FILL) - grey, moist		SS2	SS	46																
185.5	SAND AND SILT (FILL) - trace to some clay, trace gravel, brown to dark grey, moist, dense to very dense		SS3	SS	53																
0.8	- A thin layer of clayey silt was encountered at a depth of 2.3 m		SS4	SS	31																
			SS5	SS	36																
			SS6	SS	59																
			SS7	SS	33																
178.7																					
7.6	CLAYEY SILT (FILL) - sandy, trace gravel, brown, moist, very stiff		SS8	SS	15																
177.2																					
9.1	SILTY SAND TO SAND AND SILT (TILL) - trace to some gravel, trace to some clay, brown to grey, dry to moist, compact to dense		SS9	SS	20																
			SS10	SS	18																
			SS11	SS	35																
			SS12	SS	37																
			SS13	SS	26																
			SS14	SS	36																
170.5																					
15.8	END OF BOREHOLE																				
	NOTE: 1) No groundwater was encountered in open borehole upon completion of drilling.																				

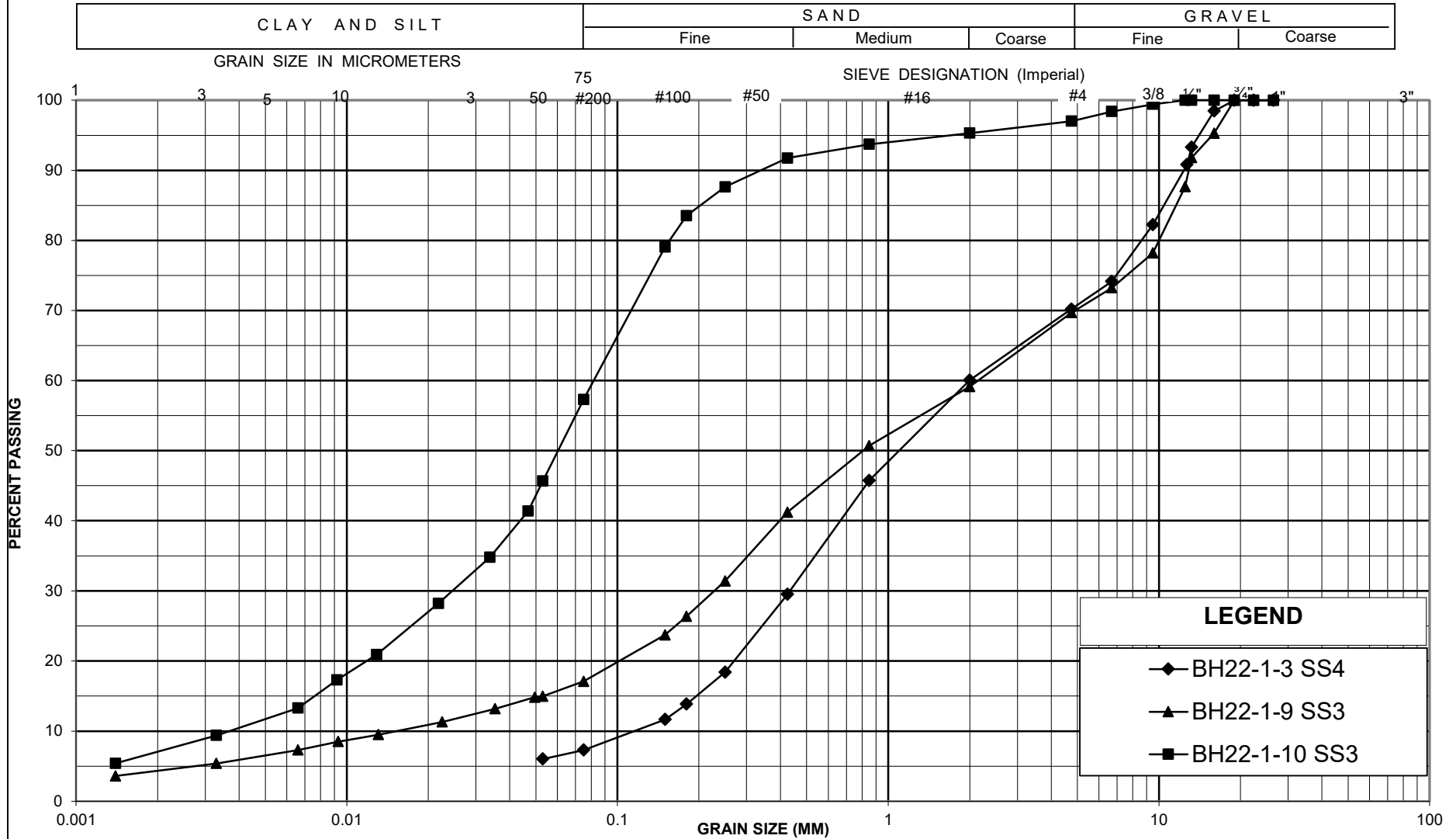
ONTARIO MTO H401 - BIRCHMOUNT - 07022023.GPJ ONTARIO MTO.GDT 1/22/24

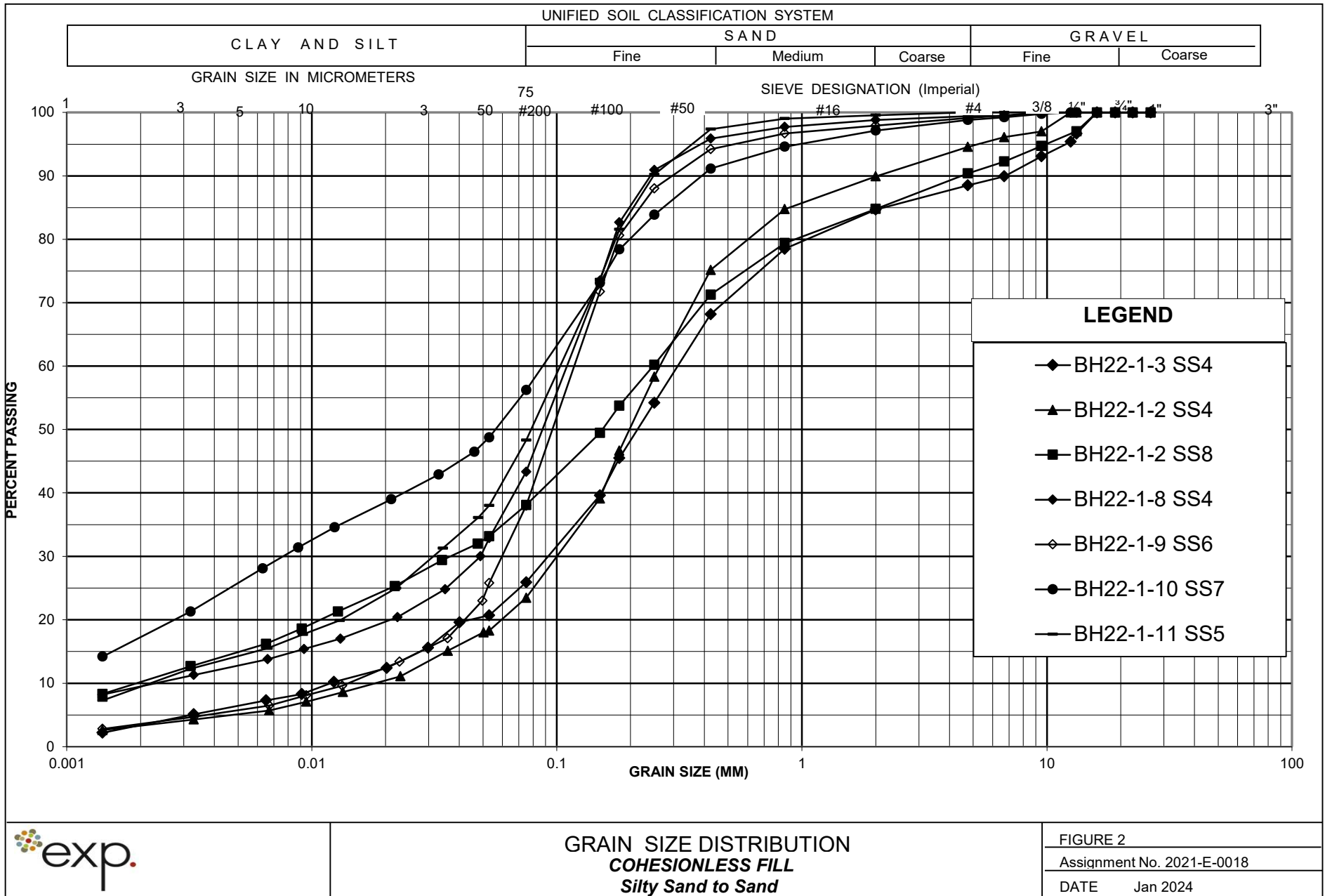
+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

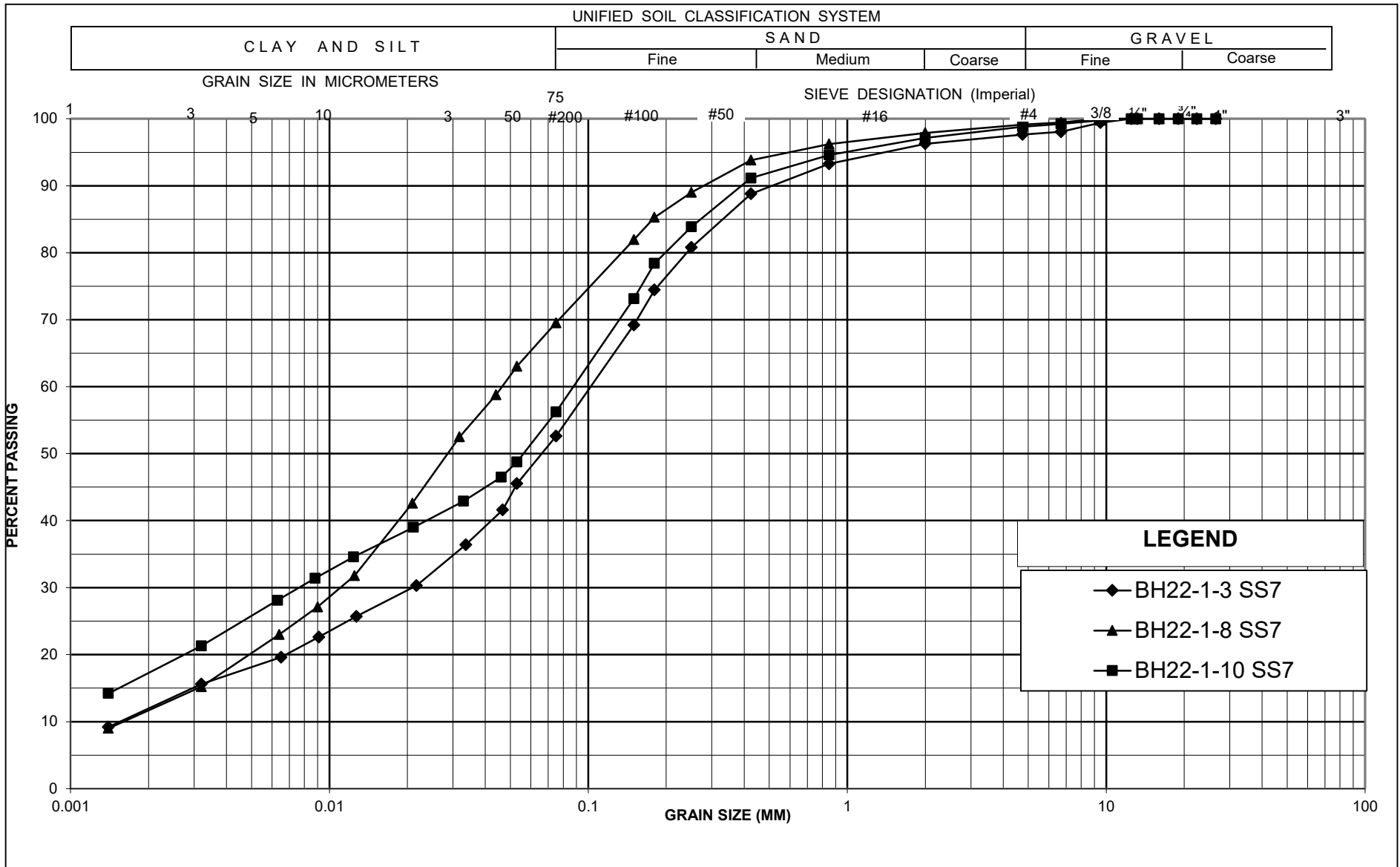
## Appendix E – Laboratory Data



# UNIFIED SOIL CLASSIFICATION SYSTEM





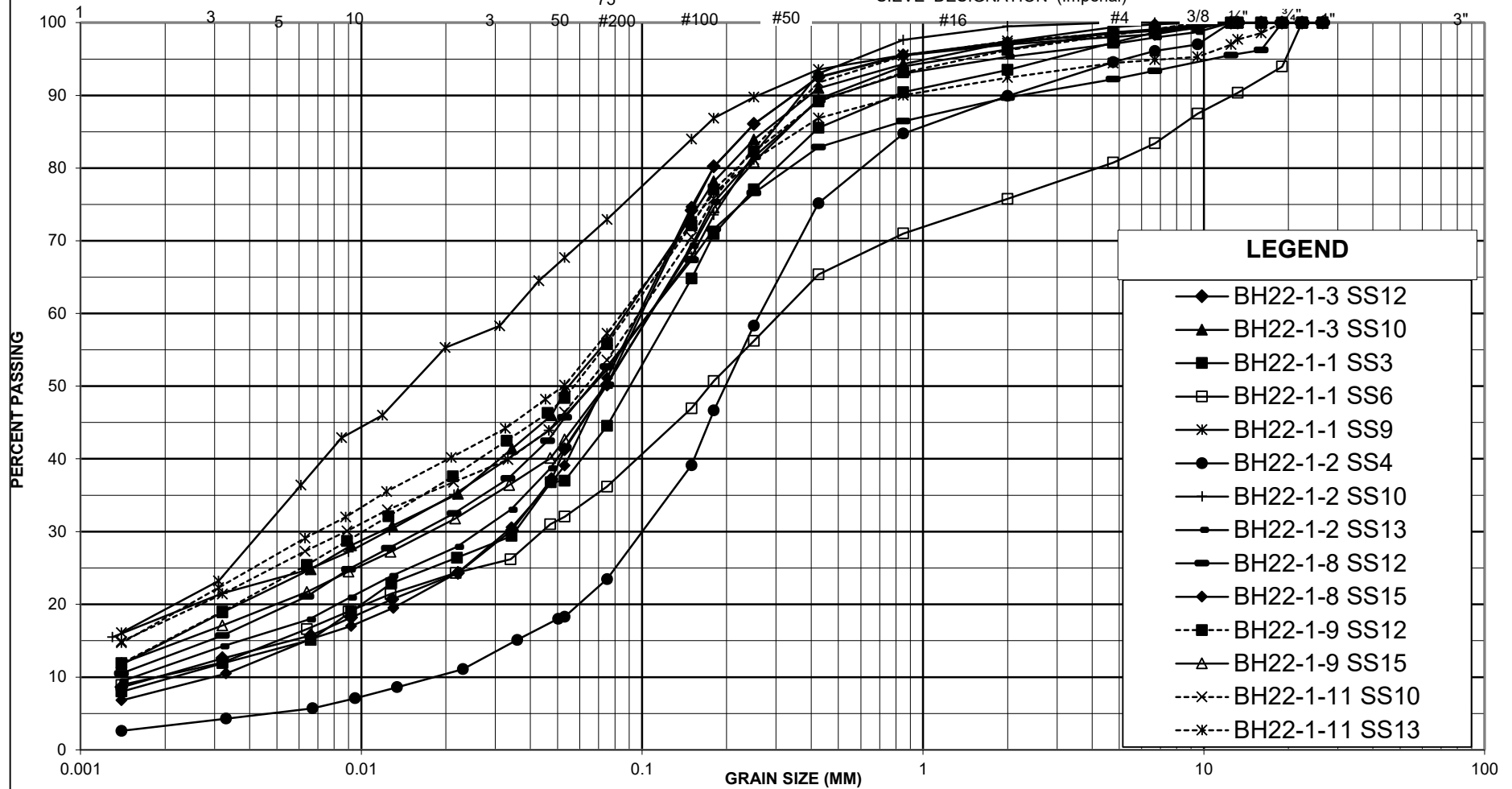


UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY AND SILT	SAND			GRAVEL	
	Fine	Medium	Coarse	Fine	Coarse

GRAIN SIZE IN MICROMETERS

SIEVE DESIGNATION (Imperial)



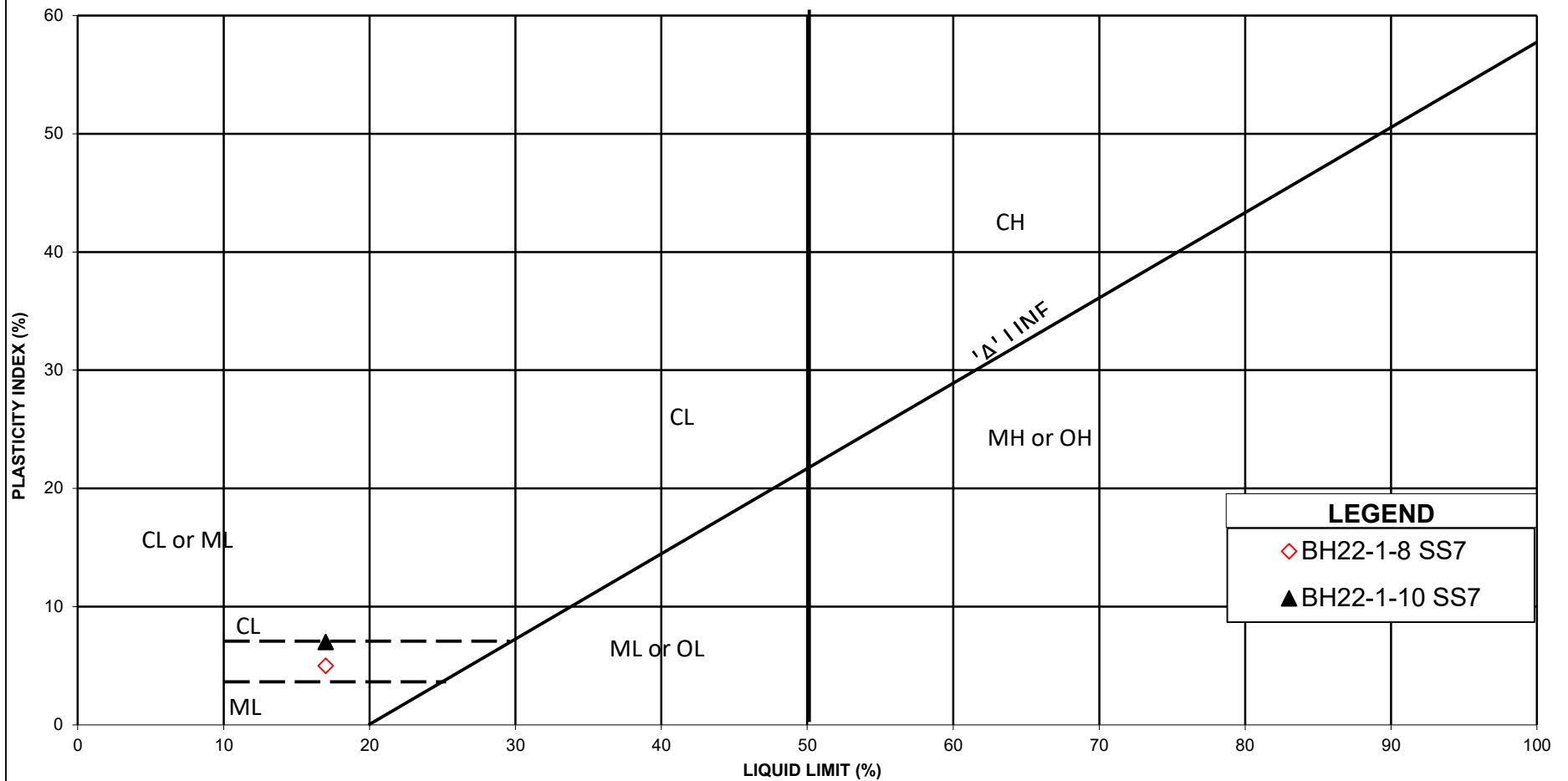
GRAIN SIZE DISTRIBUTION  
SILTY SAND TO SANDY SILT (TILL)

FIGURE 4

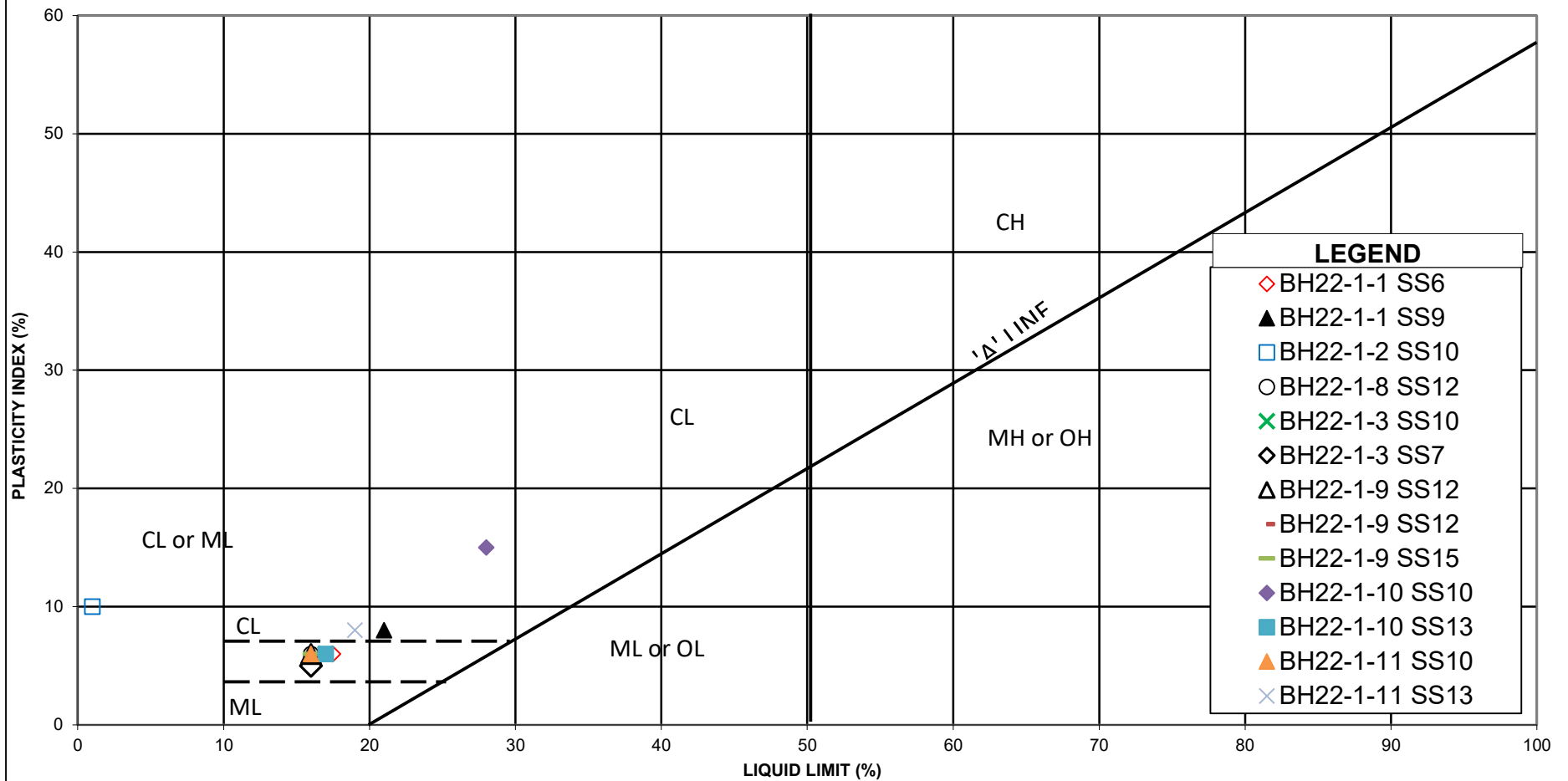
Assignment No. 2021-E-0018

DATE Jan 2024

## Highway 401 - Birchmount Road Overpass



# Highway 401 - Birchmount Road Overpass



## Appendix F – Previous Investigation - BH logs

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS &amp; TESTING DIVISION

## RECORD OF BOREHOLE NO. 1

FOUNDATION SECTION

JOB 65-F-49LOCATION Hwy. #401 & Birchmount Rd Hwy #401 Ch 316/92 130'-0" RtORIGINATED BY W.W.K.W.P. 256-61BORING DATE May 12, 1965.COMPILED BY W.W.K.DATUM 579.0BOREHOLE TYPE Penndrill 4" Auger.CHECKED BY K.G.S. *GR*

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT					LIQUID LIMIT ——— WL PLASTIC LIMIT ——— WP WATER CONTENT ——— W			BULK DENSITY  P.C.F.	REMARKS
ELEV DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT		20	40	60	80	100	WP	W	WL		
579.0	Groundlevel															
578.0	Black Org. Topsoil															
1.0	Silty sand to sandy silt with traces of clay and gravel.  Compact to very dense.		1	SS	130	570										
			2	SS	80											
				for 4"												
			3	SS	137	560										
			4	SS	138											
			5	SS	80	550										
				for 4"												
			6	SS	119											
542.5			7	SS	162	540										
36.5	End of borehole.															

W.L. El.  
551.5  
Observed in  
Borehole.



DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS &amp; TESTING DIVISION

## RECORD OF BOREHOLE NO. 2

FOUNDATION SECTION

JOB 65-F-49

LOCATION Hwy. #401 Birchmount Rd Hwy #401 Ch 315/72 130'-0" Rt.

ORIGINATED BY W.W.K.

W.P. 256-01

BORING DATE May 12, 1965.

COMPILED BY W.W.K.

DATUM 577.0

BOREHOLE TYPE Penndrill 4" Auger

CHECKED BY *AK*

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT					LIQUID LIMIT — WL PLASTIC LIMIT — WP WATER CONTENT — W			BULK DENSITY P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT		20	40	60	80	100	wp	w	wL		
577.0	Groundlevel															
576.0	Black org. topsoil															
1.0																
	Silty sand to sandy silt with traces of clay and gravel.		1	SS	39	570										
	Compact to very dense.		2	SS	88											
			3	SS	141	560										
				for 7"												
			4	SS	101											
				for 5"												
			5	SS	169	550										
				for 9"												
			6	SS	166											
			7	SS	118	540										
535.5			8	SS	124											
41.5	End of borehole.					530										

W.L. El.  
▼ 550.0  
Observed  
in  
Borehole.

DEPARTMENT OF HIGHWAYS - ONTARIO


## MATERIALS &amp; TESTING DIVISION

## RECORD OF BOREHOLE NO. 3

FOUNDATION SECTION

JOB 65-F-49 LOCATION Hwy #401 & Birchmount Rd Hwy #401 Ch 316+02 75'-0" Rt. ORIGINATED BY W.W.K.  
 W.P. 256-61 BORING DATE May 12 & 13, 1965. COMPILED BY W.W.K.  
 DATUM 580.0 BOREHOLE TYPE Penndril 4" Auger. CHECKED BY W.W.K.

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE		LIQUID LIMIT — WL PLASTIC LIMIT — WP WATER CONTENT — W		BULK DENSITY P.C.F.	REMARKS
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	BLOWS / FOOT		BLOWS / FOOT	SHEAR STRENGTH P.S.F.	WP	WL		
580.0	Groundlevel											
578.5	Sand, gravel - Fill											
1.5												
	Silty sand to sandy silt with traces of clay and gravel.		1	SS	33							
			2	SS	129	570						
			3	SS	125							
	Compact to very dense.		4	SS	120	560						
			5	SS	80							
			for 3"			550						
			6	SS	80							
			for 2 1/2"									
			7	SS	126							
538.5			8	SS	121	540						
41.5	End of borehole.											

W.L. Fl.  
 551.0  
 Observed in Borehole.

## Appendix G – Slope Stability Analysis Results

# **Highway 401 Eastbound Express and Collector Lanes between Victoria Park Avenue and Neilson Road** Birchmount Overpass Eastbound Core and Collectors Structure (Site 37X-0212/B1 & B3)

West Abutment  
Forward Slope  
Drained Conditions

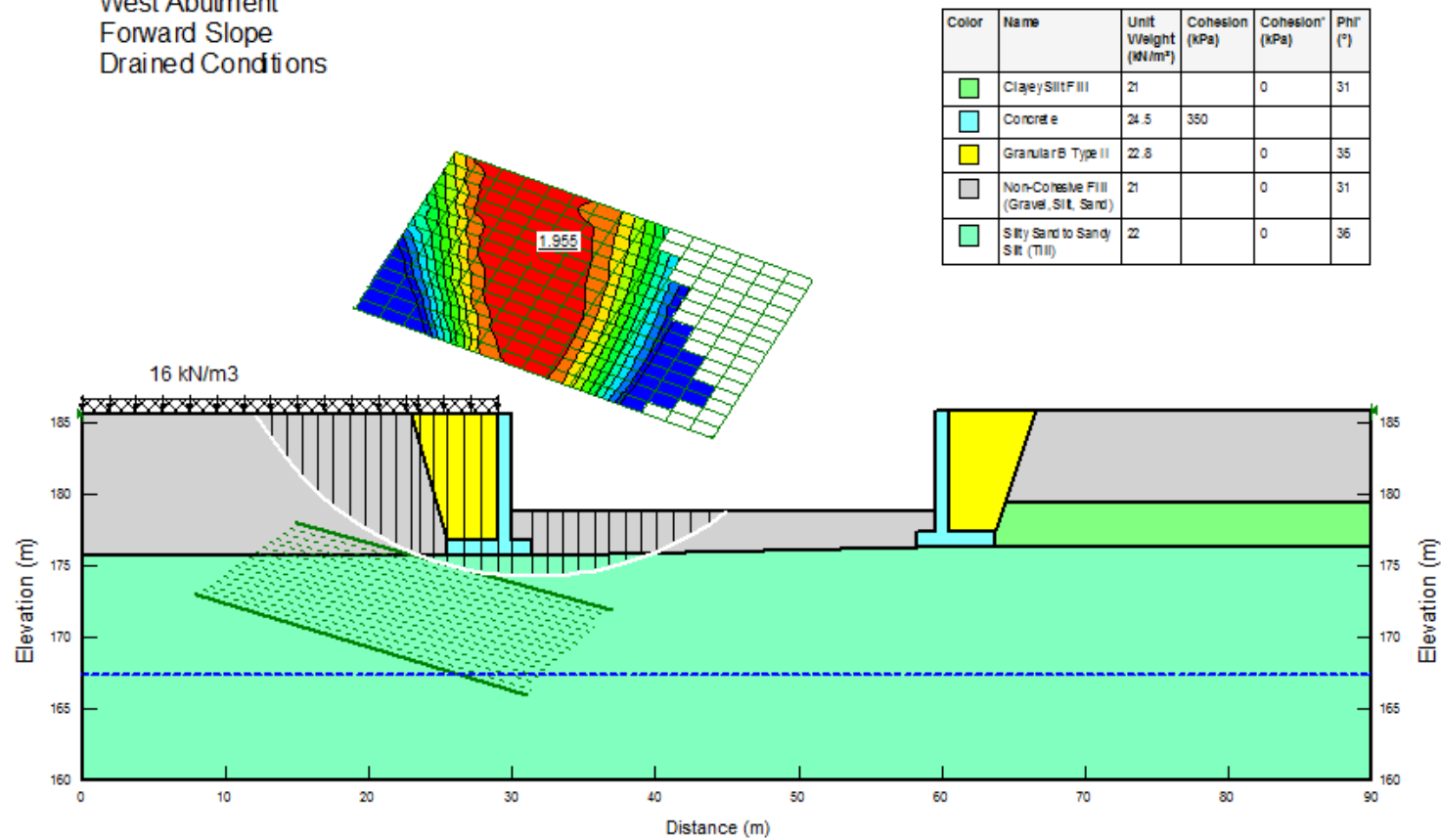


Figure 1. Slope Stability results for west abutment, forward wall, drained conditions.

# Highway 401 Eastbound Express and Collector Lanes between Victoria Park Avenue and Neilson Road

Birchmount Overpass Eastbound Core and Collectors Structure

(Site 37X-0212/B1 & B3)

West Abutment

Forward Slope

Drained Conditions

Seismic Analysis

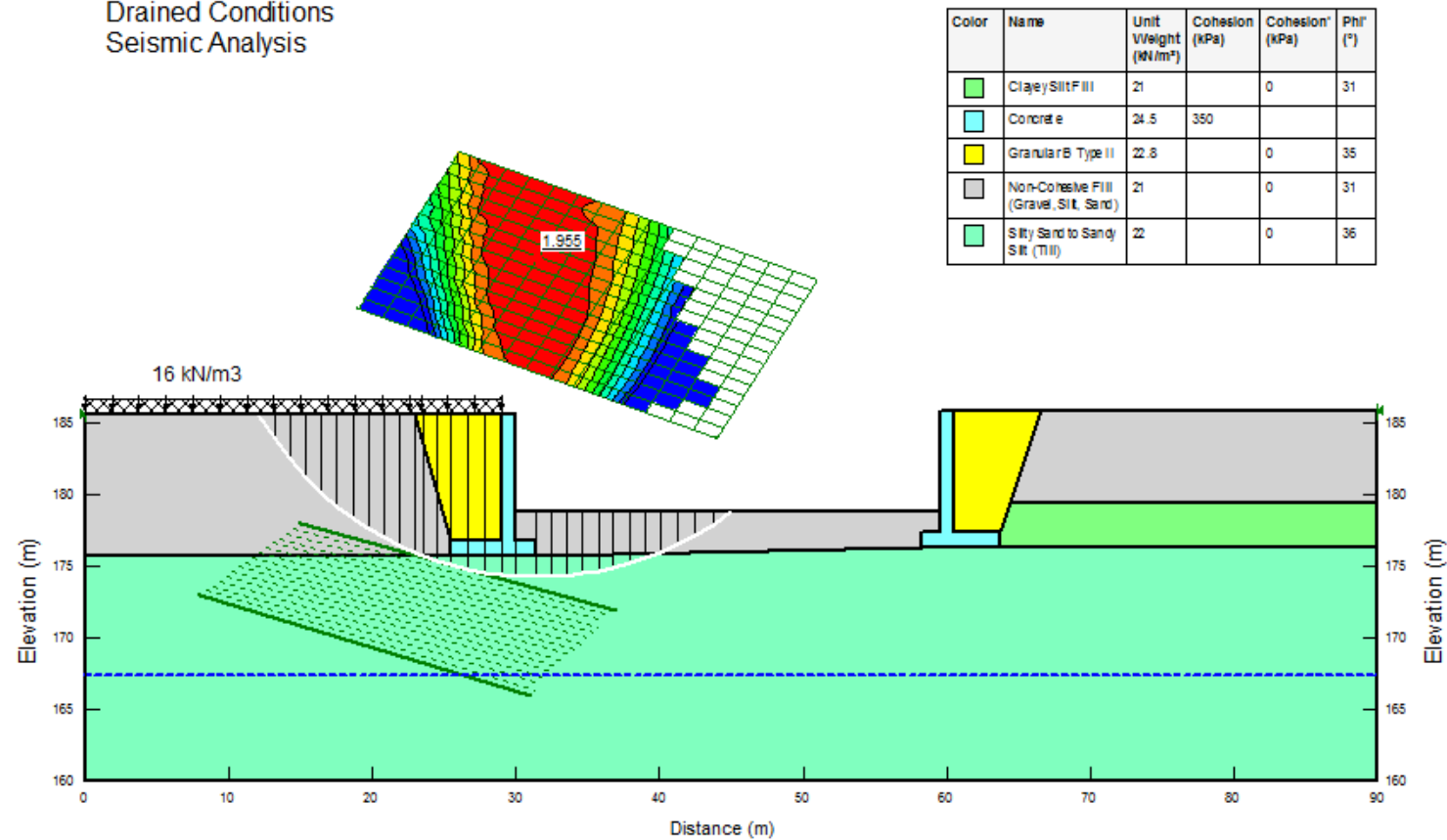


Figure 2. Slope Stability results for west abutment, forward wall, seismic conditions.

# Highway 401 Eastbound Express and Collector Lanes between Victoria Park Avenue and Neilson Road

Birchmount Overpass Eastbound Core and Collectors Structure  
(Site 37X-0212/B1 & B3)

East Abutment  
Forward Slope  
Drained Conditions

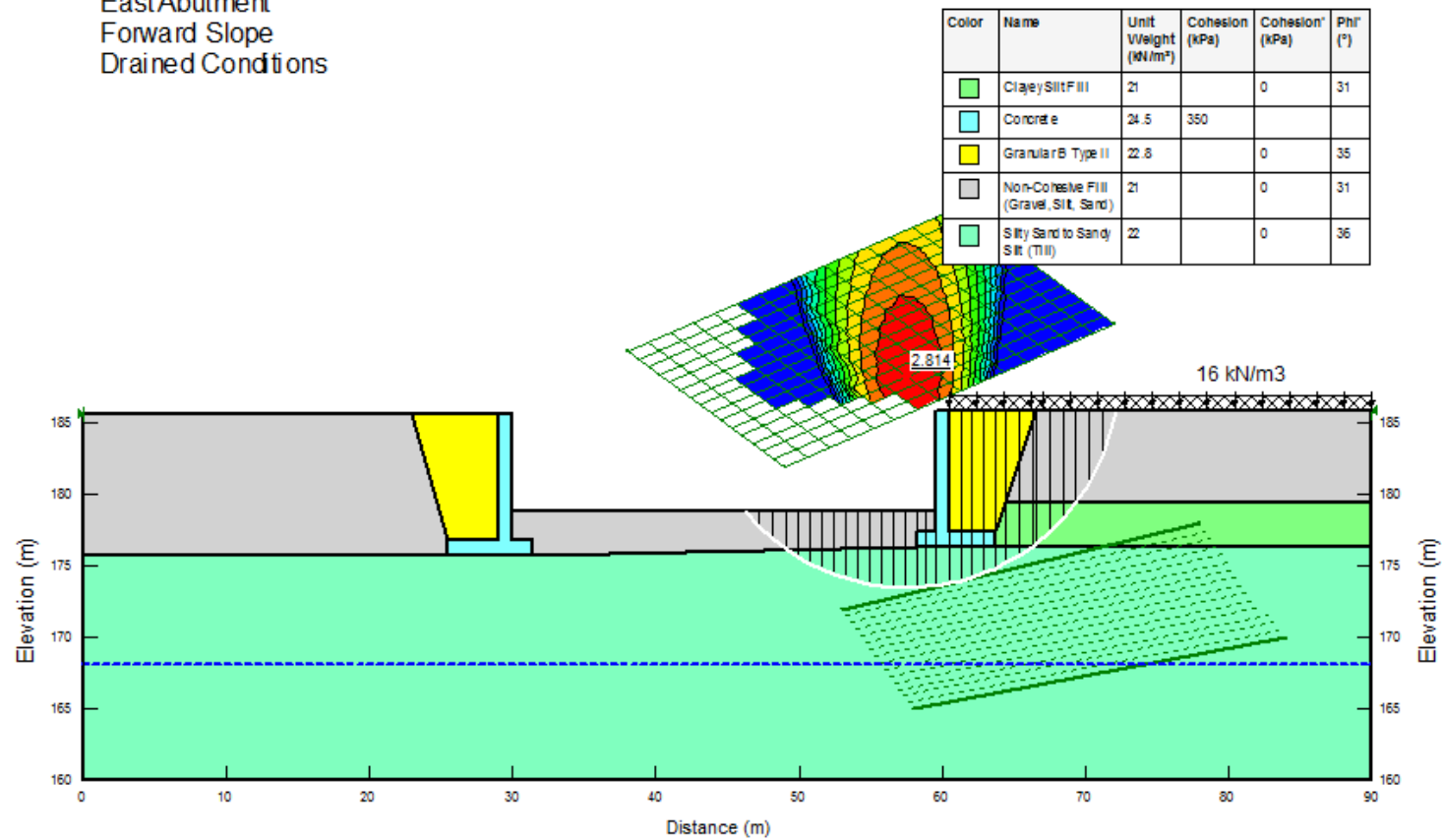


Figure 3. Slope stability results for east abutment, forward wall, drained conditions.

# Highway 401 Eastbound Express and Collector Lanes between Victoria Park Avenue and Neilson Road

Birchmount Overpass Eastbound Core and Collectors Structure  
(Site 37X-0212/B1 & B3)

East Abutment  
Forward Slope  
Undrained Conditions

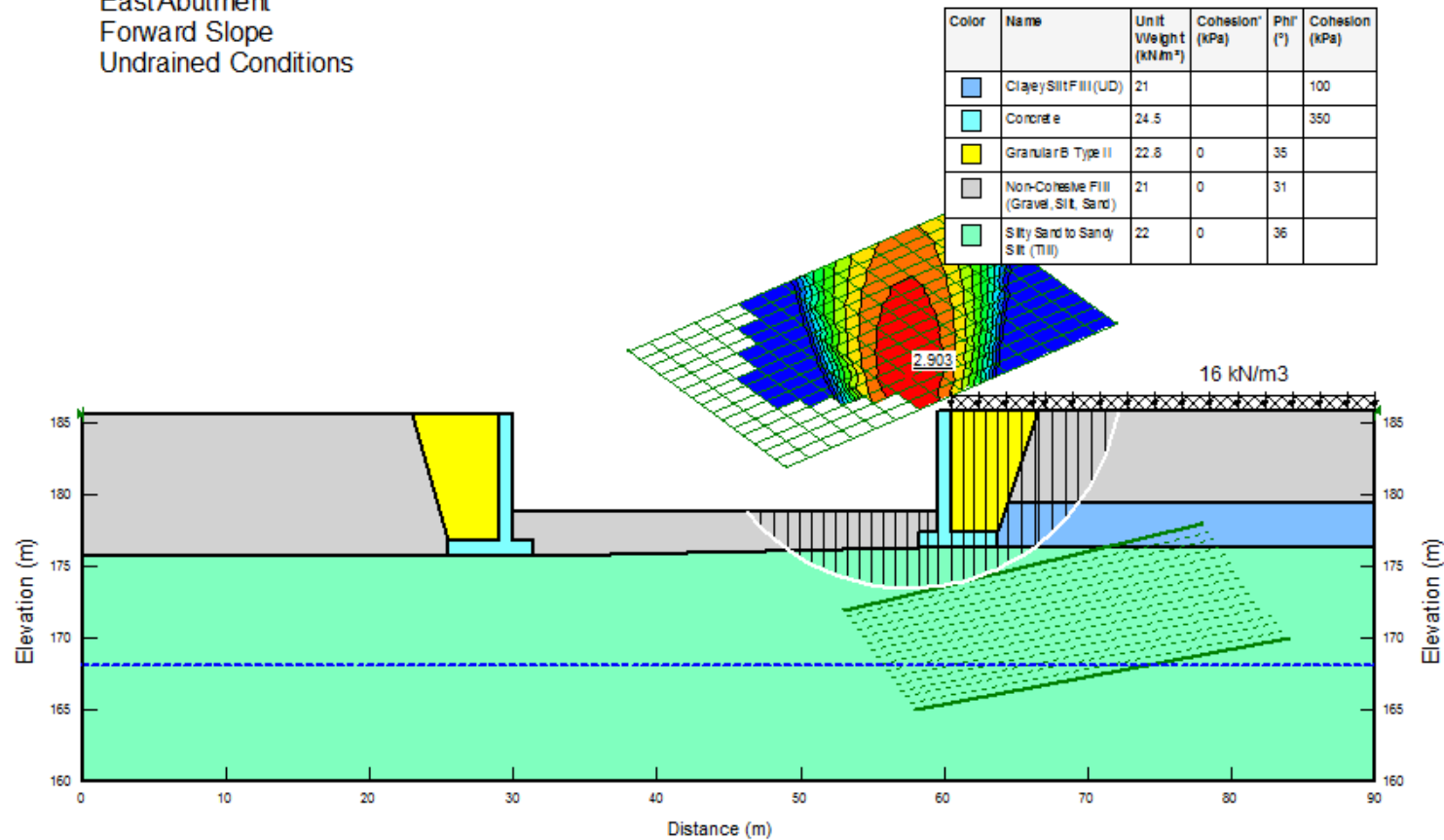


Figure 4. Slope stability results for east abutment, forward slope, undrained conditions.

# **Highway 401 Eastbound Express and Collector Lanes between Victoria Park Avenue and Neilson Road**

Birchmount Overpass Eastbound Core and Collectors Structure  
(Site 37X-0212/B1 & B3)

East Abutment  
Forward Slope  
Undrained Conditions

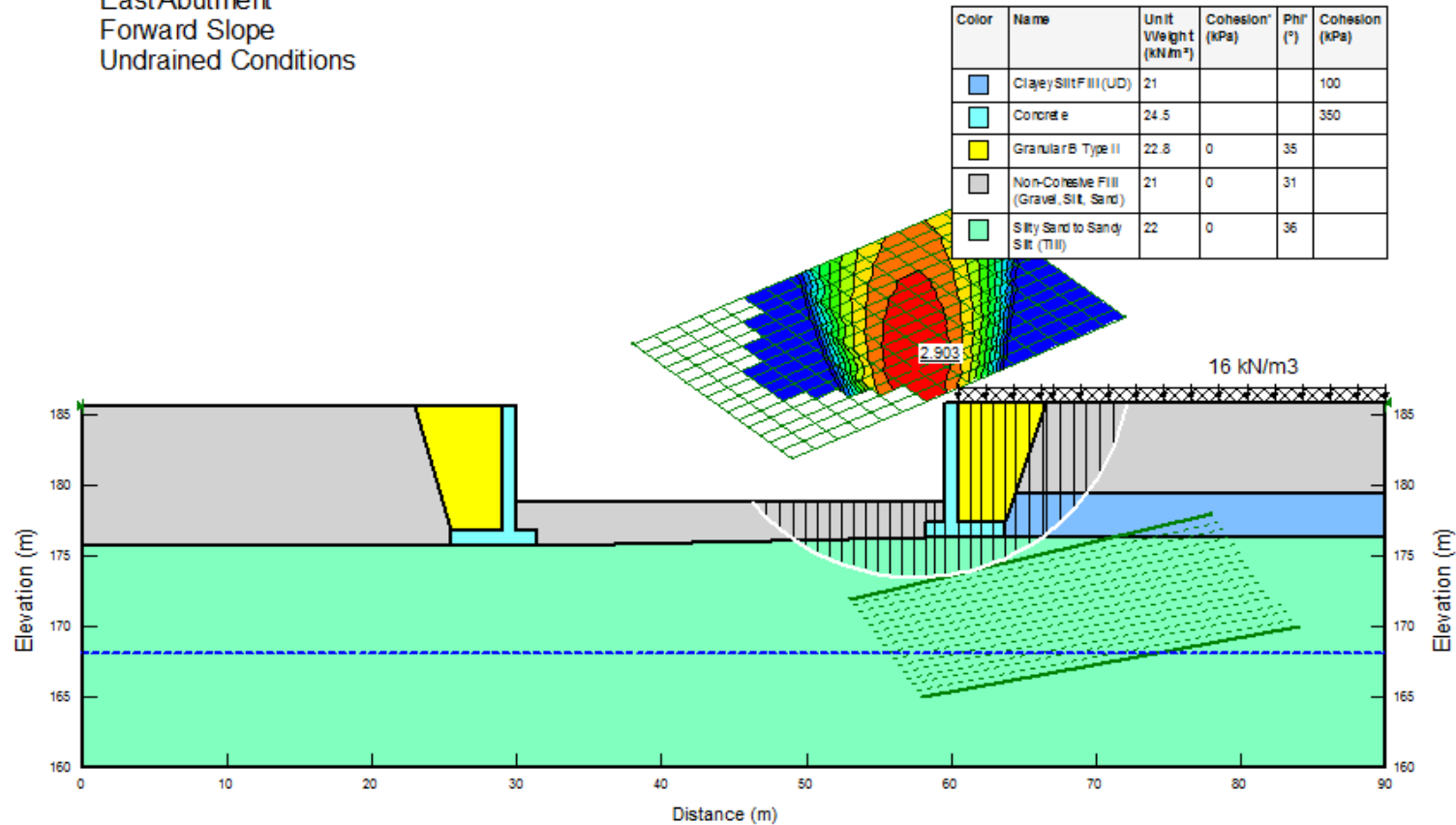


Figure 5. Slope stability results for east abutment, forward slope, seismic conditions.



# Highway 401 Eastbound Express and Collector Lanes between Victoria Park Avenue and Neilson Road

Bichmount Overpass Eastbound Core and Collector Structure  
(Site 37x-0212/B1 & B3)

West Abutment  
Side Slope  
Drained, Seismic Conditions

Color	Name	Unit Weight (kN/m <sup>3</sup> )	Cohesion' (kPa)	Phi' (°)
<span style="color: blue;">■</span>	Existing Clayey Silt Fill	21	0	31
<span style="color: lightgreen;">■</span>	Existing Granular Fill	21	0	32
<span style="color: yellow;">■</span>	Granular A/Granular B Type II	22.8	0	35
<span style="color: green;">■</span>	Silty Sand to Sandy Silt Till	22	0	36

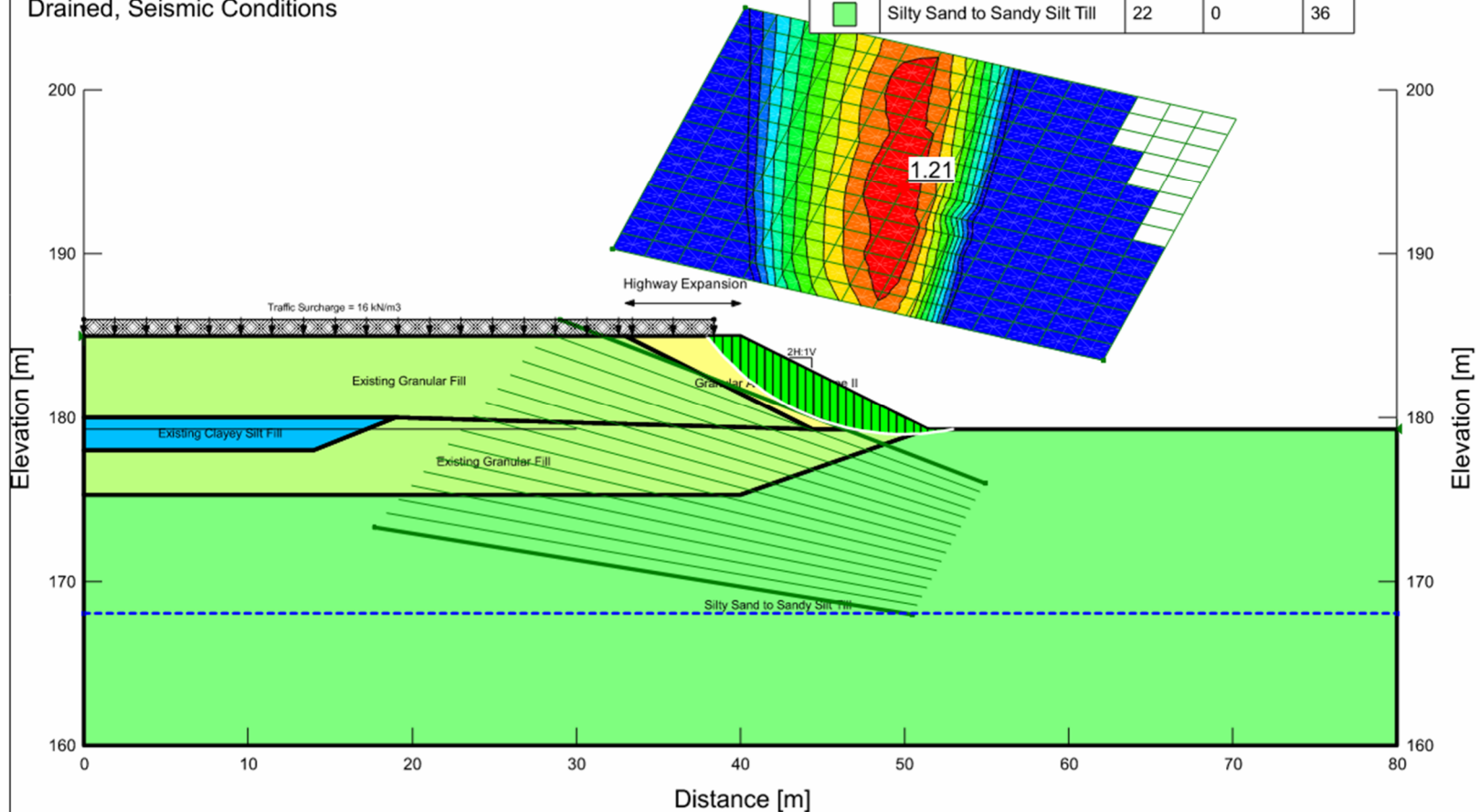


Figure 6. Slope stability results for west abutment, side slope, drained conditions.

# Highway 401 Eastbound Express and Collector Lanes between Victoria Park Avenue and Neilson Road

Bichmount Overpass Eastbound Core and Collector Structure  
(Site 37x-0212/B1 & B3)

West Abutment  
Side Slope  
Undrained, Seismic Conditions

Color	Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Cohesion (kPa)
<span style="color: blue;">■</span>	Existing Clayey Silt Fill	21			100
<span style="color: lightgreen;">■</span>	Existing Granular Fill	21	0	32	
<span style="color: yellow;">■</span>	Granular A/Granular B Type II	22.8	0	35	
<span style="color: green;">■</span>	Silty Sand to Sandy Silt Till	22	0	36	

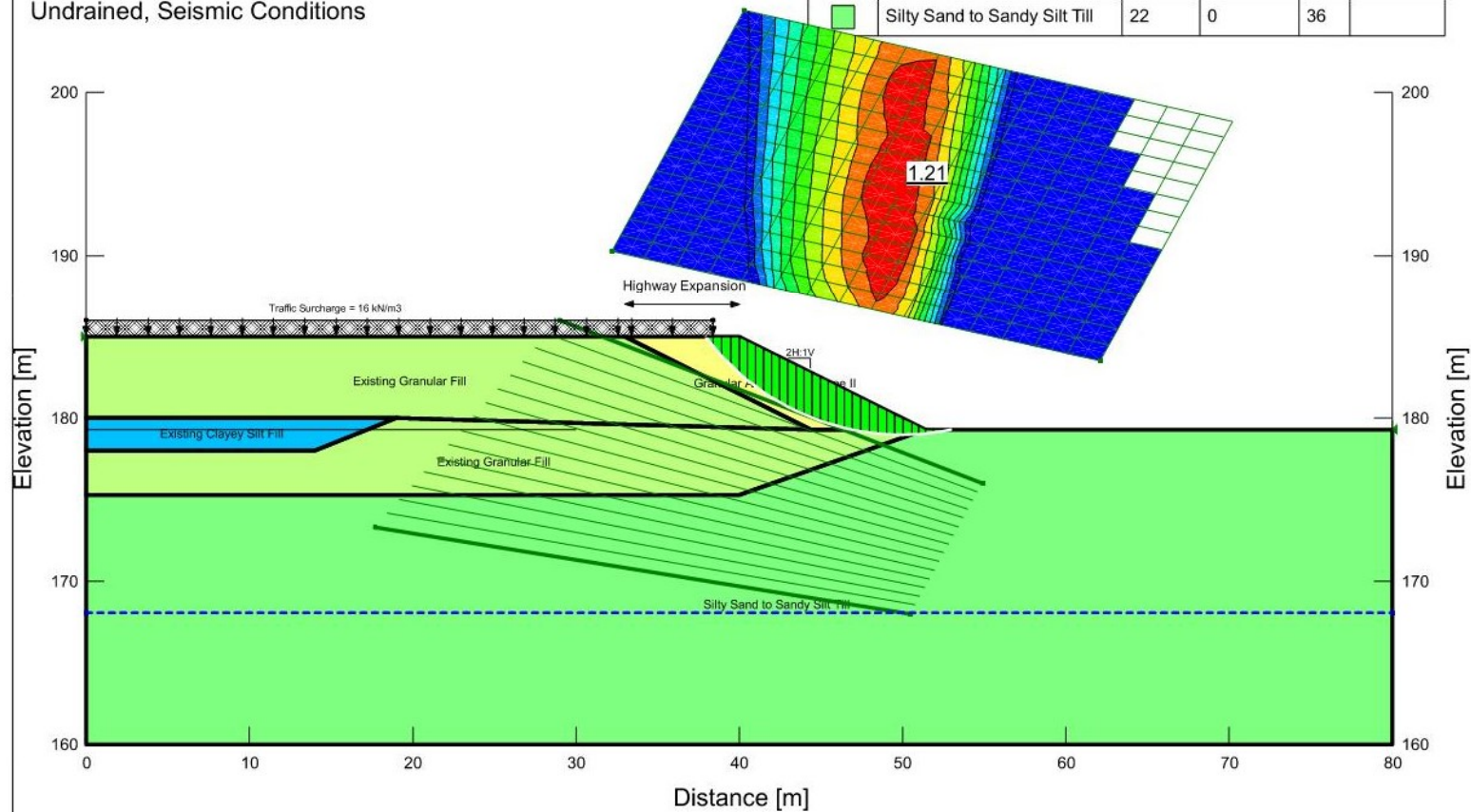


Figure 7. Slope stability results for west abutment, side slope, seismic conditions.

# **Highway 401 Eastbound Express and Collector Lanes between Victoria Park Avenue and Neilson Road**

Birchmount Overpass Eastbound Core and Collectors Structure  
(Site 37X-0212/B1 & B3)

East Abutment  
Side Slope  
Drain Static Condition

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
Blue	Existing Clayey Silt Fill	Mohr-Coulomb	21	0	31
Light Green	Existing Granular Fill	Mohr-Coulomb	21	0	32
Yellow	Granular A/Granular B Type II	Mohr-Coulomb	22.8	0	35
Light Blue	Silty Sand to Sandy Silt Till	Mohr-Coulomb	22	0	36

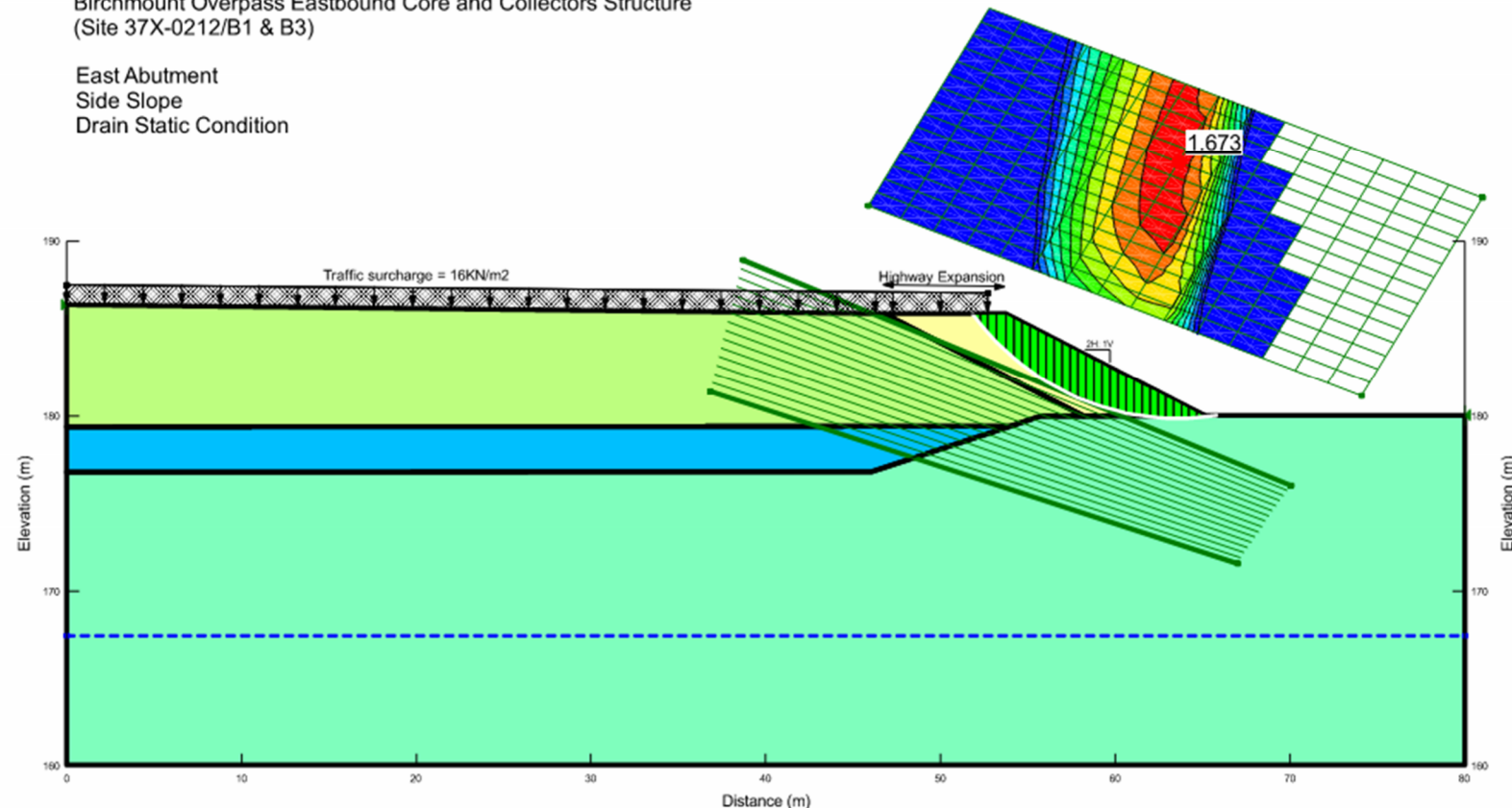


Figure 8. Slope stability results for east abutment, side slope, drained conditions.

# **Highway 401 Eastbound Express and Collector Lanes between Victoria Park Avenue and Neilson Road**

Birchmount Overpass Eastbound Core and Collectors Structure  
(Site 37X-0212/B1 & B3)

East Abutment  
Side Slope  
Undrained Seismic Conditions

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Cohesion (kPa)
<span style="color: blue;">■</span>	Existing Clayey Silt Fill	Undrained (Phi=0)	21			100
<span style="color: lightgreen;">■</span>	Existing Granular Fill	Mohr-Coulomb	21	0	32	
<span style="color: yellow;">■</span>	Granular A/Granular B Type II	Mohr-Coulomb	22.8	0	35	
<span style="color: lightblue;">■</span>	Silty Sand to Sandy Silt Till	Mohr-Coulomb	22	0	36	

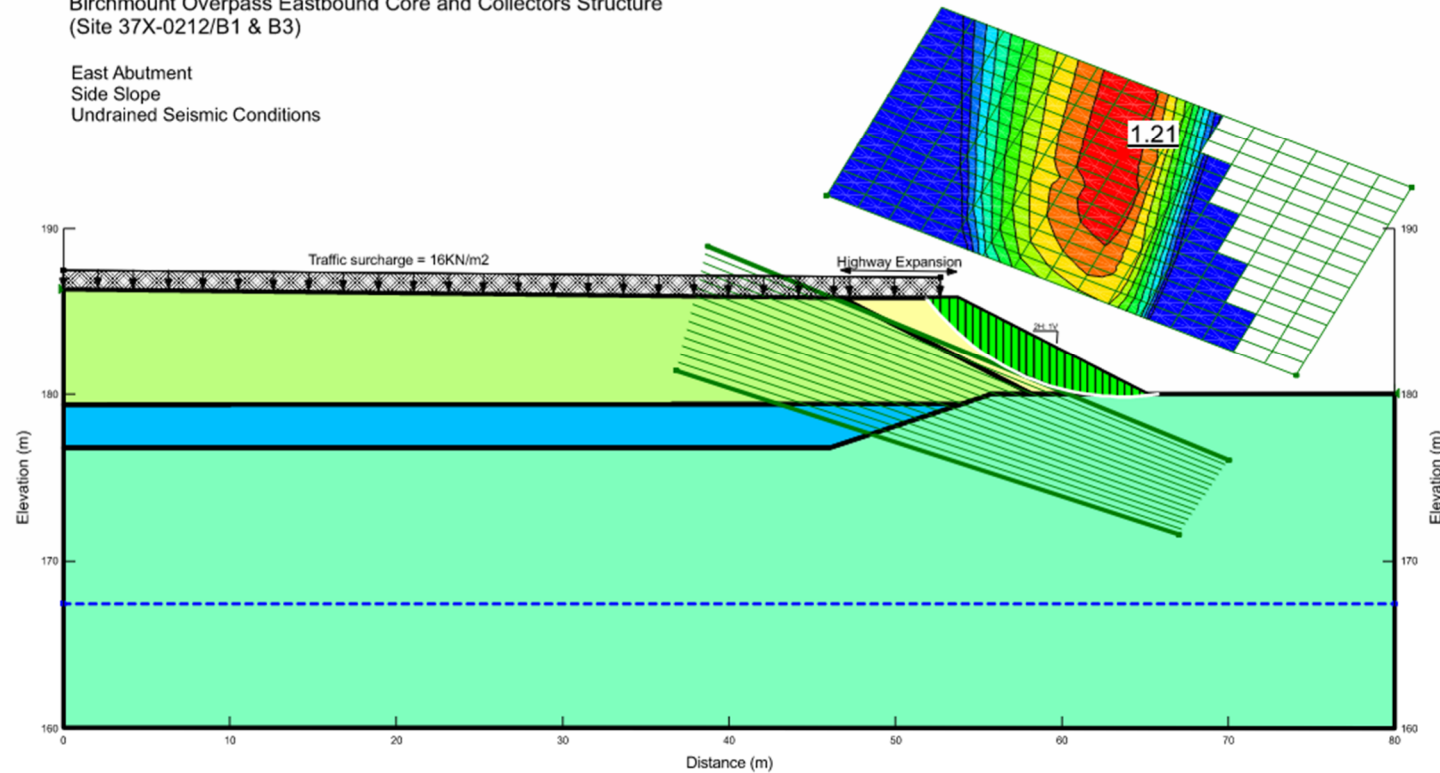


Figure 9. Slope stability results for east abutment, side slope, Undrained conditions.



# **Highway 401 Eastbound Express and Collector Lanes between Victoria Park Avenue and Neilson Road**

Birchmount Overpass Eastbound Core and Collectors Structure  
(Site 37X-0212/B1 & B3)

East Abutment  
Side Slope  
Undrained Seismic Conditions

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Cohesion (kPa)
<span style="color: blue;">■</span>	Existing Clayey Silt Fill	Undrained (Phi=0)	21			100
<span style="color: green;">■</span>	Existing Granular Fill	Mohr-Coulomb	21	0	32	
<span style="color: yellow;">■</span>	Granular A/Granular B Type II	Mohr-Coulomb	22.8	0	35	
<span style="color: cyan;">■</span>	Silty Sand to Sandy Silt Till	Mohr-Coulomb	22	0	36	

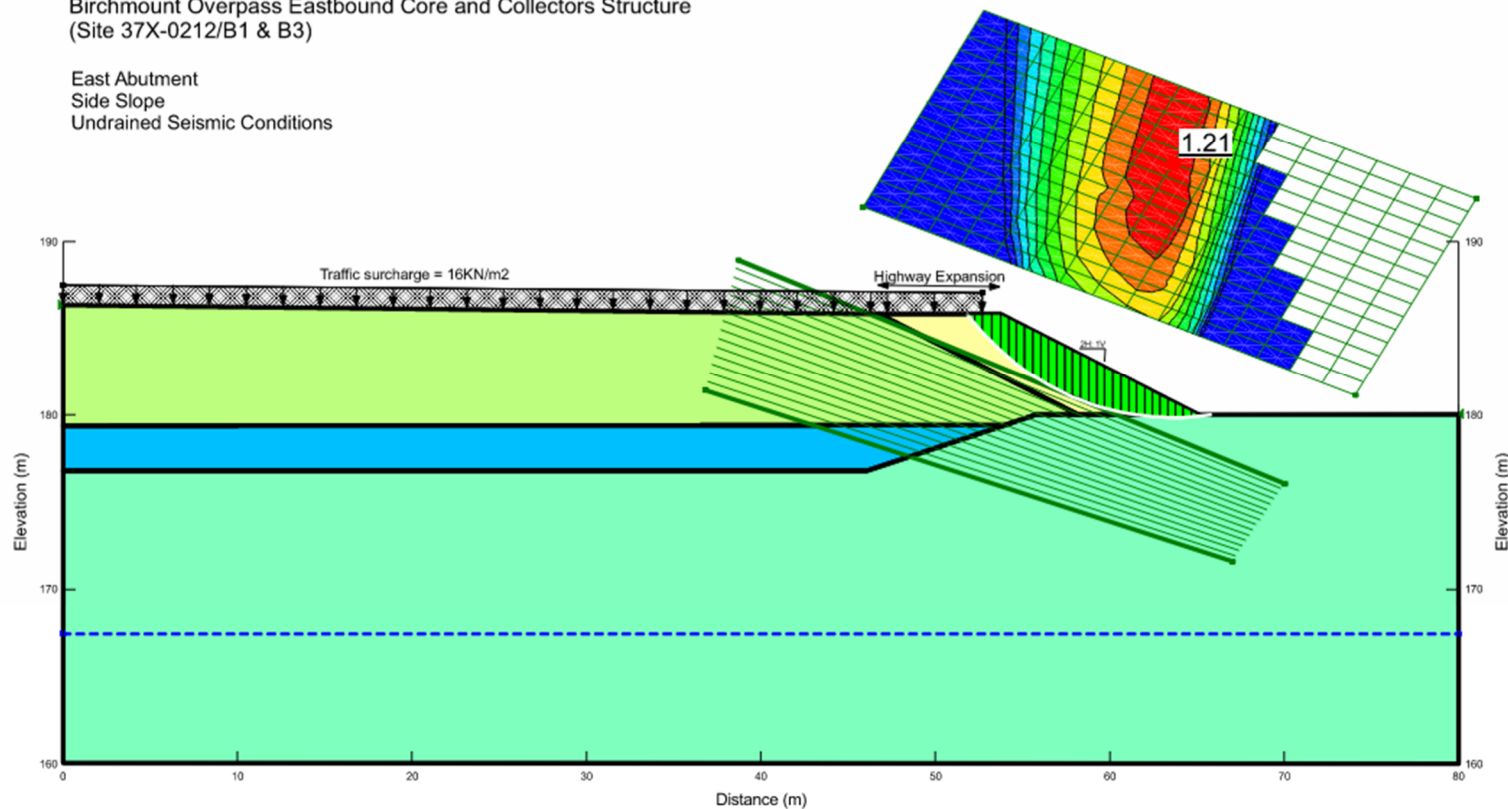
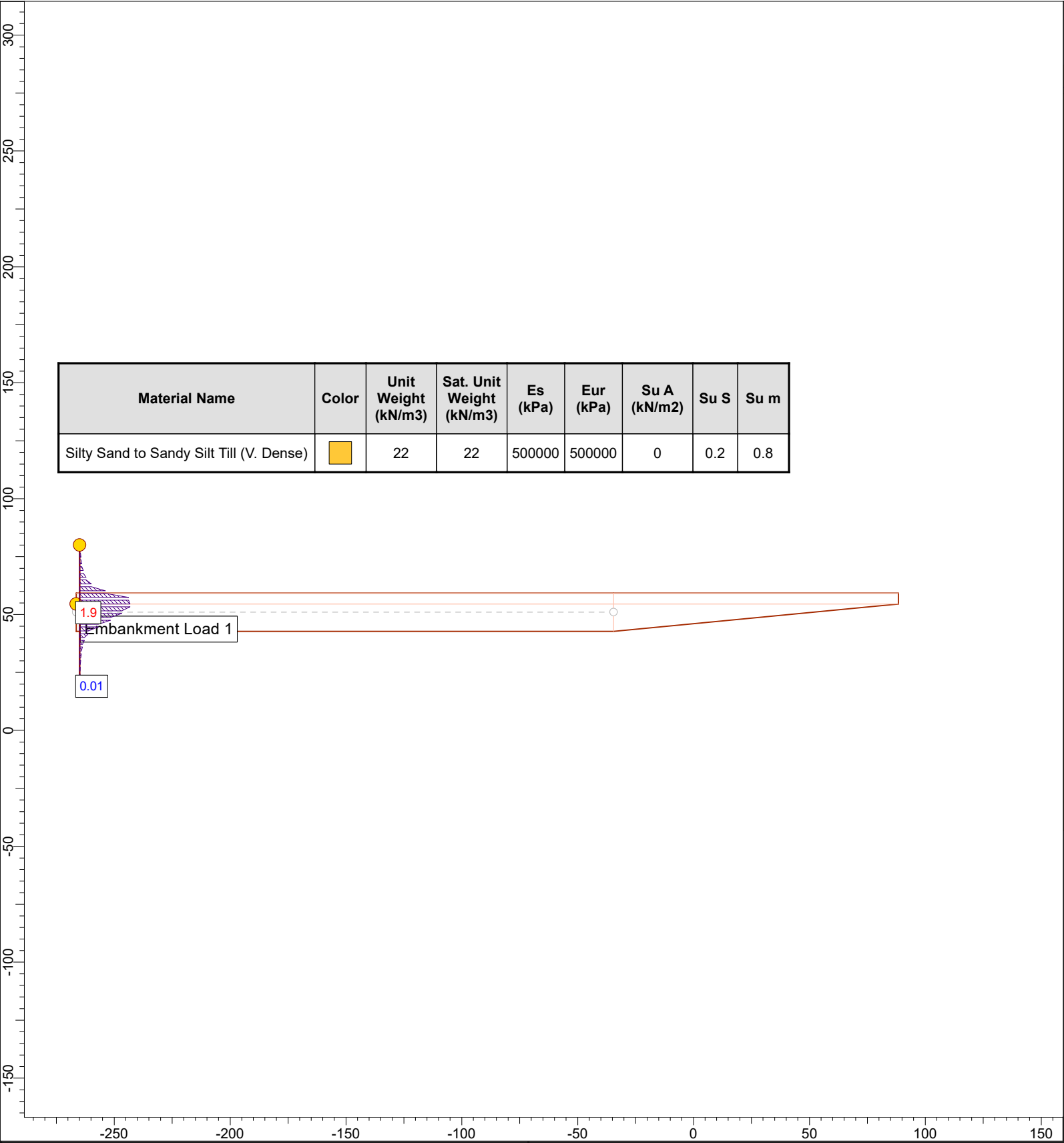


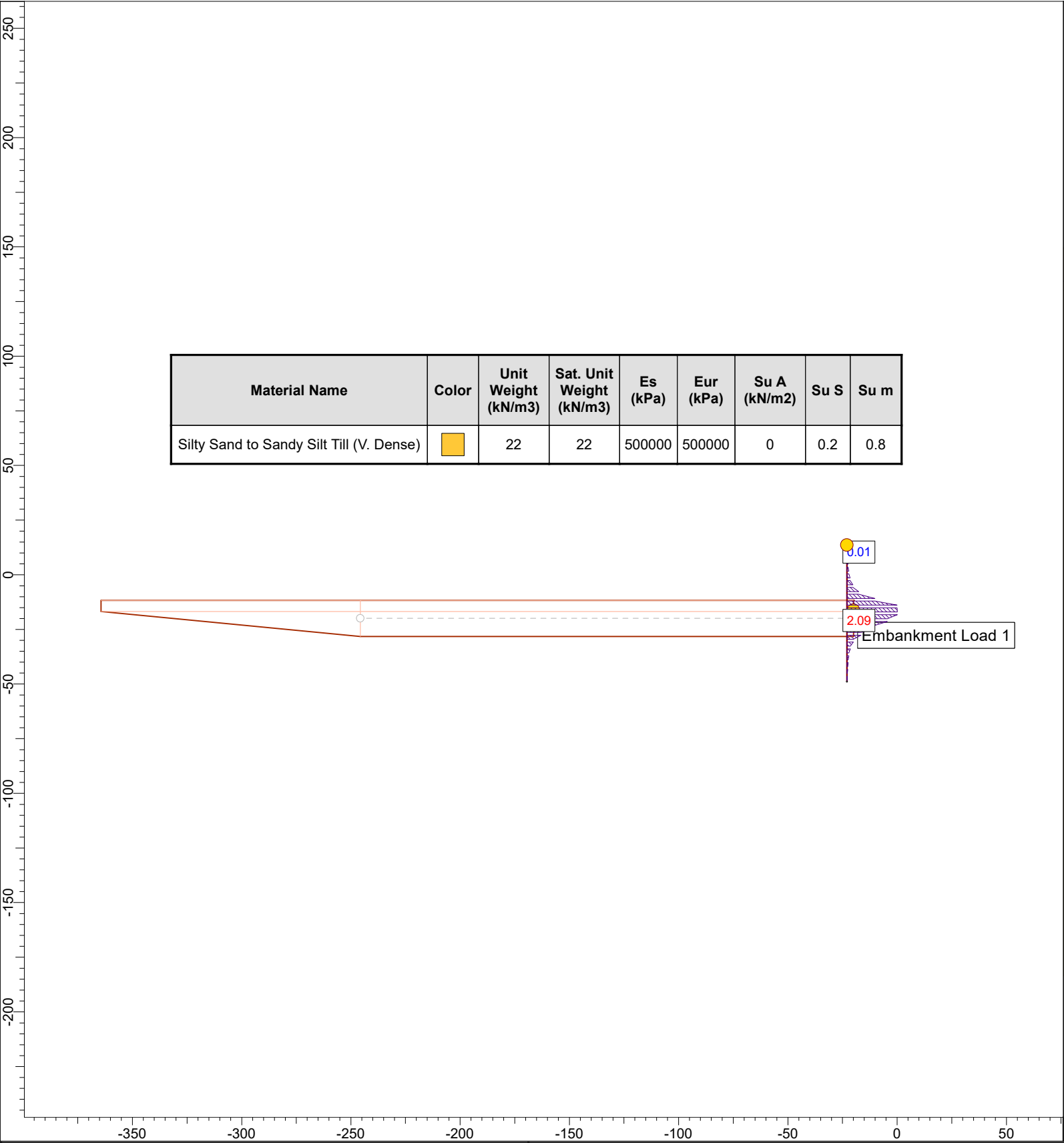
Figure 10. Slope stability results for east abutment, side slope, seismic conditions.

## Appendix H – Settlement Analysis Results

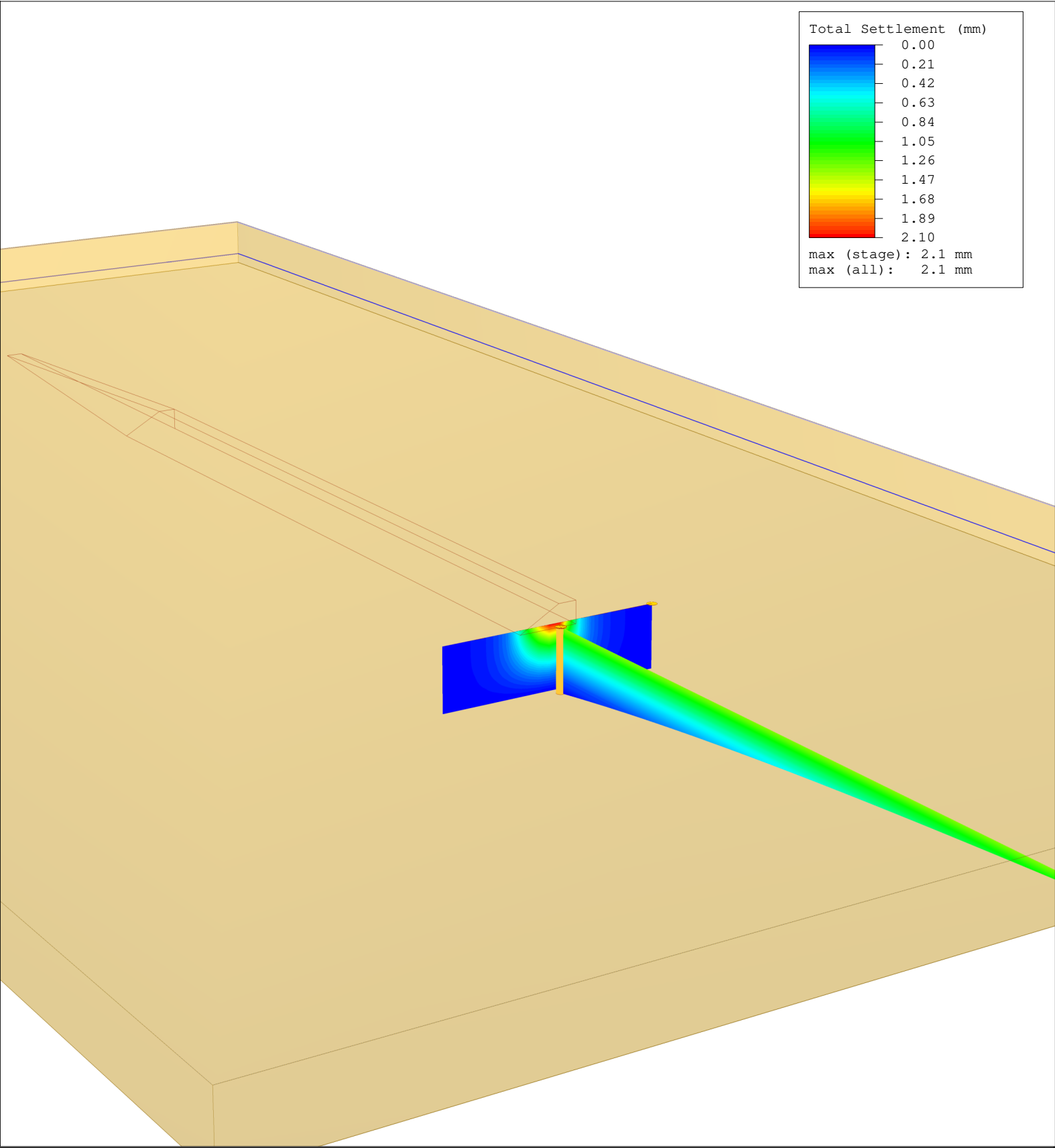


Material Name	Color	Unit Weight (kN/m3)	Sat. Unit Weight (kN/m3)	Es (kPa)	Eur (kPa)	Su A (kN/m2)	Su S	Su m
Silty Sand to Sandy Silt Till (V. Dense)	<div></div>	22	22	500000	500000	0	0.2	0.8





Material Name	Color	Unit Weight (kN/m3)	Sat. Unit Weight (kN/m3)	Es (kPa)	Eur (kPa)	Su A (kN/m2)	Su S	Su m
Silty Sand to Sandy Silt Till (V. Dense)	<div></div>	22	22	500000	500000	0	0.2	0.8





## Appendix J – Seismic Hazard Values



Government  
of Canada

Gouvernement  
du Canada

[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 (°)	43.773
Longitude (°)	-79.294

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( $X_c$ )) 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( $X_c$ )
0.324	0.198	0.104	0.0481	0.0125	0.00424	0.176	0.129

The log-log interpolated 2%/50 year  $S_a(4.0, X_c)$  value is : **0.0174**

#### ▼ Tables for 5% and 10% in 50 year values

##### NBC 2020 - 5%/50 years (0.001 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( $X_c$ )
0.18	0.113	0.0589	0.0267	0.00658	0.00227	0.0932	0.0697

The log-log interpolated 5%/50 year  $S_a(4.0, X_c)$  value is : **0.0093**

##### NBC 2020 - 10%/50 years (0.0021 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( $X_c$ )
-----------------	-----------------	-----------------	-----------------	-----------------	------------------	--------------	--------------

$S_a(0.2, X_C)$	$S_a(0.5, X_C)$	$S_a(1.0, X_C)$	$S_a(2.0, X_C)$	$S_a(5.0, X_C)$	$S_a(10.0, X_C)$	PGA( $X_C$ )	PGV( $X_C$ )
0.108	0.0696	0.036	0.0159	0.00367	0.00127	0.0533	0.0405

The log-log interpolated 10%/50 year  $S_a(4.0, X_C)$  value is : **0.0052**

Download CSV

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

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

## Appendix K – Non-Standard Special Provisions (NSSP)s

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

---

### **Scope of Work**

The Contractor should be aware that the existing fill and native soil could contain cobbles and boulders as inferred from the obstruction that was encountered and difficulties in advancing augers/auger grinding. Consideration of the presence of these obstructions must be made in the selection of appropriate equipment and procedures for piling or for temporary shoring through these materials.

### **Basis of Payment**

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

## **NSSP FOR DELAY OF PAVEMENT AT HIGH FILL EMBANKMENT**

---

### **Scope of Work**

The Contractor should be aware that High Fill embankment construction will result in settlements of the native material and the selected fill.

Embankment construction using Granular A fill and compacted to 98% SPMDD will require a minimum delay of pavement of 30 days. Embankment construction using SSM and compacted to 98% SPMDD will require a minimum delay of pavement of 90 days.

Prior to placing the pavement granular sub-base material and paving, the Contractor shall survey the embankment to confirm the elevation and place additional fill as required to achieve design requirements.

The Contractor shall not proceed with final granular base placement and paving until approval has been given by the Contracting Authority.