



## Foundation Investigation and Design Report

Highway 401 Eastbound Express and Collector Lanes between Victoria Park Avenue and Neilson Road - **Full Structure Replacement and Bridge Widening at CP Rail Overhead Eastbound Core and Collectors Structure (Site 37X-0213/B1 & B3)**

Assignment No. 2021-E-0018  
MTO Central Region  
Geocres Number: 30M14-550

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Date: December 20, 2024*

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## Part I: Foundation Investigation Report

Highway 401 Eastbound Express and Collector Lanes between Victoria Park Avenue and Neilson Road – CP Rail Overhead Structure (Site 37X-0213/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 and 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 CP Rail overhead structure (Site 37X-0213/B1 & B3) and retaining wall (37X-1768/W and 37X-1769/W). Additionally proposed widening of Highway 401 (Eastbound collector) east of the CP Rail overhead structure towards Kennedy Road, will be discussed in a separate report.

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 full structure and retaining wall replacement.

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 Lane CPR Overhead Bridge Rehab.*, prepared by AECOM, dated October 2024, shows the preliminarily proposed configuration of the CP Rail overhead structure. Foundation and Investigation Design Reports (FIDR) by Golder Associates Ltd., *"Bridge Widening and Replacement Highway 401 Rehabilitation from Warden Avenue to Brock Road, Toronto, Ontario, W.O.07-20012."*, dated March 2012 and *"CP Rail Overhead Replacement (Site No. 37-213), Highway 401 Westbound Core and Collector Lanes, Neilson Road to Warden Avenue, City of Toronto, Ontario, G.W.P No. 2162-11-00."*, dated January 17, 2019. A summary of the proposed structure is as follows:

- The existing overhead structure is about 27 m long (on skew) single-span concrete structure. It is understood that the existing abutments are supported on approximately 3.0 m wide spread footings founded at approximately Elev. 173.0 m at the east abutment and 173.3 m at the west abutment. Based on the available drawings and FIDRs, the existing Highway 401 pavement grade is at approximately Elev. 184.2 m to 184.8 m at the structure location, and the CP rail track grade is at approximately Elev. 174.8 m.
- The existing structure will undergo full structure replacement. This includes replacing the superstructure (existing bridge deck and girders) and foundations (semi-integral abutments). The structure will also be widened by approximately 10 m on the south side of the EB Collector. The existing retaining walls (37X-1768/W and 37X-1769/W) will be replaced with a new RSS wall along the south side of the widened collector structure. Additional widening along the south side of the EB Collector, east of the structure towards Kennedy Road, will be discussed in a separate report.

The previous FIDRs and GA drawing by AECOM, in addition to contract package drawings titled *401 WB Core & Collector Lanes – CPR Overhead Replacement CP Mile 200.26 Belleville Subdivision (Cont. No. 2019-2011, WP No. 2394/2393-15-01)*, produced by WSP Global Inc., dated February 2019, were reviewed as part of this report. These background documents were used for initial

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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 CP Rail Overhead, approximately 5 km east of Highway 404 in the City of Toronto, Ontario. The site is adjacent to industrial zones to the south and northeast and residential zones to the northwest.

In general, the terrain in this area is relatively flat, with the natural ground surface in the immediate vicinity of the structure at about Elev. 175 m. The CP Rail tracks have been constructed near the original ground surface, with the rail grade below Highway 401 at approximately Elev. 174.8 m. The CP tracks are oriented in a northeast to southwest direction. The existing Highway 401 grade is at approximate Elev. 184.0 m to 184.8 m. The existing embankment slope is about 2H:1V.

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 predominant 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.

A surficial till sheet, which generally follows the surface topography, is generally present in this area. This till, known as Halton till, and these represent relatively recent deposits, concentrated near West and East Don River valleys formed by small glacial melt water ponds scattered throughout the Peel Plain. The recent sand, silt and clay and till deposits in this area overlie and are interbedded with stratified deposits of sand, silt and clay.

## 4.0 Previous Geotechnical Investigation

During the tender design for the project, three previous reports were issued which contain relevant information to the proposed CN Rail overhead structure (Site 37X-0215/B1 & B3), as follows:

1. Foundation Investigation Report for The Proposed Extension of Highway. #401 and C.P.R Overhead, Metropolitan Toronto, District #6, W.J. 66-F-89, W.P. 257-61, Geocres No. 30M14-075, The Ministry of Transportation Ontario (MTO), Foundation Section, Materials and Testing Div., dated January 17, 1967.
2. 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.
3. Geocres No. 30M14-493 "CP Rail Overhead Replacement (Site No. 37-213), Highway 401 Westbound Core and Collector Lanes, Neilson Road to Warden Avenue, City of Toronto, Ontario, G.W.P No. 2162-11-00." by Golder Associates Ltd., dated January 17, 2019.

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

**Table 1.1: Summary of Applicable Boreholes Completed by MTO (Geocres. No. 30M14-068)**

Borehole No.	Borehole Location	Location (MTM NAD83 Zone 10)		Latitude	Longitude	Borehole Elevation (m)	Borehole Depth (m)
		Northing	Easting				
75-10	East CP Rail	4848108.3	321539.3	43.772746	-79.292068	178.8	18.7
75-11	West CP Rail	4848102.0	321503.2	43.772690	-79.292516	177.7	15.7

## 5.0 Field Investigation and Laboratory Analyses

### 5.1 Site Investigation and Field Testing

A site-specific investigation was undertaken by EXP between October 3, 2022, to October 19, 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. Six (6) boreholes were completed for this structure (BH22-2-01, BH22-2-02, BH22-2-03, BH22-2-04, BH22-2-13, and BH22-2-14) as part of the additional investigation. A summary of boreholes completed by EXP is listed in Table 1.2 below. The boreholes were drilled using a truck-mounted CME-75 (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. 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);
5. 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;
6. 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;
7. 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.
8. 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

9. 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				
BH22-2-01	West Abut. (EBL Collector)	4848104.0	321489.0	43.772708	-79.292693	185.9	14.3
BH22-2-02	East Abut. (EBL Collector)	4848125.1	321556.0	43.772896	-79.291860	185.1	15.8
BH22-2-03	West Abut. (EBL Express)	4848146.0	321514.9	43.773086	-79.292369	185.6	21.9
BH22-2-04	East Abutment (EBL Express)	4848164.9	321573.1	43.773255	-79.291647	184.6	17.4
BH22-2-13	West Abutment (EBL Express)	4848139.0	321496.0	43.773023	-79.292605	185.8	15.9
BH22-2-14	East Abutment (EBL Express)	4848171.1	321592.9	43.773309	-79.291399	184.2	15.7

## 5.2 Laboratory Testing

Selected samples were submitted for natural moisture content testing. In addition, Atterberg Limit and Grain size analysis (sieve and hydrometer) tests were performed on selected soil samples (performed by EXP). A minimum of 25% of the samples were tested for soil classification as per the MTO guidelines. In addition, chemical analyses were carried out on two 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 laboratory testing program is listed 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
BH22-2-01	13	1	3	2	1	1
BH22-2-02	15	1	4	3	2	---
BH22-2-03	18	---	5	3	3	---
BH22-2-04	17	3	4	3	2	1
BH22-2-13	13	---	3	2	2	---
BH22-2-14	15	---	3	3	2	---

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.

The general stratigraphy encountered within the investigated depths of EXP's geotechnical investigation indicates the following sub-surface sequence: a pavement structure composed of asphalt and concrete overlaying embankment fill consisting of layers of cohesionless and cohesive material underlain by native cohesionless till with a layer of clayey silt till in borehole BH22-2-04.

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 all boreholes drilled by EXP. The thickness of the pavement structure ranged between 250 mm and 450 mm.

#### 6.1.2 Cohesionless Soil (Fill)

Layers of cohesionless fill consisting of various compositions (sand and gravel, gravelly sand, sand, sand and silt, silty sand) were encountered in all boreholes drilled by EXP. 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 1.4 below:

**Table 1.4: Summary of Cohesionless Fill Layers**

Borehole	Elevation (m)		Layer Surface Depth (m)	Layer Thickness (m)	Layer Description	SPT “N” Value Range
	Top	Bottom				
EXP (2022)						
BH22-2-01	185.5	179.8	0.4	5.7	Gravelly Sand	33 – 49
BH22-2-02	184.7	182.7	0.4	2.0	Sand	20 – 29
BH22-3-03	185.2	176.5	0.4	8.7	Sand	7 – 24
BH22-2-04	184.3	179.8	0.3	4.5	Sand	7 – 35
BH22-2-13	185.5	177.9	0.3	7.6	Silty Sand	4 – 48
	176.7	175.1	9.1	1.6	Silty Sand	9

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Borehole	Elevation (m)		Layer Surface Depth (m)	Layer Thickness (m)	Layer Description	SPT "N" Value Range
	Top	Bottom				
BH22-2-14	183.9	183.4	0.3	0.5	Sand and Gravel	N/A <sup>1</sup>
	183.4	177.8	0.8	5.6	Sand and Silt	7 – 43
	176.3	174.7	7.9	1.6	Silty Sand	22 – 28

Note:

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

The cohesionless fill consists predominantly of sand with varying amounts of gravel and silt, trace to some clay, trace clayey silt inclusions, trace organics/rootlets, and occasional cobbles. The soil is damp to moist in moisture condition and brown to greyish brown to grey in color. The SPT "N" values within this layer ranged from 4 to 49 blows per 300 mm penetration, corresponding to loose to dense in terms of compactness condition.

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

Moisture Content (EXP):

- 2% to 22%

Grain Size Distribution (EXP):

- 2% to 28% gravel
- 41% to 85% sand
- 46% silt
- 11% clay
- 5% to 36% silt and clay

Unit Weight: (EXP)

- 21.0 kN/m<sup>3</sup> to 23.2 kN/m<sup>3</sup>

The results of the moisture content, unit weight, 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 tests performed by EXP are also provided in Figure 1 in Appendix E.

### 6.1.3 Clayey Silt (Fill)

Cohesive fill was encountered in boreholes BH22-2-01, BH22-2-02, BH22-2-04, BH22-2-13, and BH22-2-14. 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 1.5 below:

**Table 1.5: Summary of Clayey Silt Fill Layers**

Borehole	Elevation (m)		Layer Surface Depth (m)	Layer Thickness (m)	Layer Description	SPT “N” Value Range
	Top	Bottom				
EXP (2022)						
BH22-2-01	179.8	176.8	6.1	3.0	Clayey Silt	12 – 27
BH22-2-02	182.7	177.5	2.4	5.2	Clayey Silt	5 – 11
BH22-2-04	179.8	175.5	4.8	4.3	Clayey Silt	6 – 19
BH22-2-13	177.9	176.7	7.9	1.2	Clayey Silt	31
BH22-2-14	177.8	176.3	6.4	1.5	Clayey Silt	23

The fill consisted predominantly of clay and silt and is considered sandy with trace to some gravel and trace organics. The soil within these layers is moist in terms of moisture condition and its color typically ranges from brown to grey. The SPT "N" values measured within those layers ranged from 5 to 31 blows per 300 mm of penetration, corresponding to firm to hard in terms of consistency.

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

**Moisture Content (EXP):**

- 8% to 20%

**Grain Size Distribution: (EXP)**

- 0% to 6% gravel
- 41% to 47% sand
- 36% to 40% silt
- 11% to 19% clay

**Atterberg Limits: (EXP)**

- Liquid Limit: 16% to 23%
- Plastic Limit: 11% to 13%
- Plasticity Index: 5% to 11%

The results of the moisture content, grain size distribution, and Atterberg limit tests performed by EXP are provided on the record of borehole sheets in Appendix D. The results of the grain size distribution and Atterberg limit tests performed by EXP are also provided on Figure 2 and 6 in Appendix E.



#### 6.1.4 Native Cohesive Till

A native cohesive till layer was encountered in borehole BH22-2-04. 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 1.6 below:

**Table 1.6: Summary of Native Cohesive Soil Layer**

Borehole	Elevation (m)		Layer Surface Depth (m)	Layer Thickness (m)	Layer Description	SPT "N" Value Range
	Top	Bottom				
EXP (2022)						
BH22-2-04	172.4	170.9	12.2	1.5	Clayey Silt (Till)	67

The native cohesive layer consisted predominantly of clay and silt with some sand and trace to some gravel. The soil is moist in terms of moisture condition and brown to grey in colour. A SPT "N" value obtained within this layer was 67 blows per 300 mm of penetration corresponding to hard in terms of consistency. Atterberg limits tests suggest that this layer was low plastic.

Laboratory testing performed on selected samples consisted of moisture content, grain size distribution, and Atterberg Limit tests. The test results of clayey silt soils are as follow:

Moisture Content (EXP):

- 8% to 15%

Grain Size Distribution: (EXP)

- 3% gravel;
- 17% sand;
- 40% silt;
- 40% clay

Atterberg Limits: (EXP)

- Liquid Limit: 29%
- Plastic Limit: 14%
- Plasticity Index: 15%

The results of the moisture content, grain size distribution and Atterberg limit tests performed by EXP are provided on the record of borehole sheets in Appendix D. The results of the grain size distribution and Atterberg limit tests performed by EXP are also provided on Figures 3 and 7 in Appendix E.

### 6.1.5 Native Cohesionless Till

Native cohesionless till consisting of sand and silt was encountered below the embankment 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 Native Cohesionless Till Layers**

Borehole	Elevation (m)		Layer Surface Depth (m)	Layer Thickness (m)	Layer Description	SPT “N” Value Range
	Top	Bottom				
EXP (2022)						
BH22-2-01	176.8	171.6	9.1	5.2 <sup>1</sup>	Sand and Silt (Till)	9 – 143
BH22-2-02	177.5	169.3	7.6	8.2 <sup>1</sup>	Sand and Silt (Till)	48 – 100
BH22-3-03	176.5	163.7	9.1	12.8 <sup>1</sup>	Sand and Silt (Till)	17 – 139
BH22-2-04	175.5	172.4	9.1	3.1 <sup>1</sup>	Sand and Silt (Till)	39 – 64
	170.9	167.2	13.7	3.7 <sup>1</sup>	Sand and Silt (Till)	62 – 100/250 mm
BH22-2-13	175.1	170.0	10.7	5.1 <sup>1</sup>	Sand and Silt (Till)	62 – 100
BH22-2-14	174.7	168.5	9.5	6.2 <sup>1</sup>	Sand and Silt (Till)	28 – 168
MTO (1967)						
78-10	178.8	160.1	0.0	18.7 <sup>1</sup>	Sand and Silt (Till)	32 – 100/127 mm
78-11	177.7	162.0	0.0	15.7 <sup>1</sup>	Sand and Silt (Till)	28 – 126

Note:

1. End of borehole terminated within this layer.

The native cohesionless layers consist predominantly of silt and sand with trace to some gravel, trace to some clay, and occasional thin sand lenses. The soil is damp to moist in terms of moisture condition and generally brown to grey in terms of color. The SPT “N” values within this layer ranged from 9 to 168 blows per 300 mm penetration, corresponding to loose to very dense but generally dense to very dense in terms of compactness condition. Atterberg limits tests suggest that this layer was non-plastic to low plastic.

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

Moisture Content (EXP and MTO):

- 2% to 21%

Grain Size Distribution: (EXP and MTO)

- 0% to 6% gravel;
- 44% to 91% sand;
- 36% to 45% silt;

- 7% to 13% clay;

Atterberg Limits: (EXP and MTO)

- Liquid Limit: 15% to 16%
- Plastic Limit: 13%
- Plasticity Index: 2% to 3%

One test result indicated non-plastic material.

Unit Weight: (EXP)

- 20.8 kN/m<sup>3</sup> to 24.1 kN/m<sup>3</sup>

The results of the moisture content, grain size distribution, and Atterberg limit tests performed by EXP are provided on the record of borehole sheets in Appendix D. The results of the grain size distribution tests performed by EXP are also provided on Figures 4 and 5 in Appendix E. The results of tests performed by MTO are shown on the borehole logs attached in Appendix J.

## 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 1.8 and are also presented on the Record of Borehole Sheets attached in Appendix D and Appendix J.

**Table 1.8: Summary of observed groundwater levels**

Borehole	Ground Surface Elevation (m)	Water level Depth/ Elevation (m)	Date
<b>EXP (2022)</b>			
<b>BH22-2-03</b>	185.6	17.3/168.3	October 18, 2022
<b>MTO (1967)</b>			
<b>75-10</b>	178.8	10.3/168.5	December 02, 1966
<b>75-11</b>	177.7	9.1/168.6	December 02, 1966

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

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

The analytical results are summarized in Table 1.9 below and are presented in Appendix D.

**Table 1.9: Summary of chemical analysis results**

Borehole I.D.	Sample I.D.	Depth (m)	pH (Unitless)	Soluble Chloride (ppm)	Soluble Sulphate (ppm)	Resistivity (ohm-cm)	Conductivity (mS/cm)	Redox Potential (mV)
BH22-2-01	SS13	13.7 – 14.3	7.98	750 – 770	41	660	1.51 – 1.53	290
BH22-2-04	SS9	9.1 – 9.7	7.91	380	<20	1500	0.659	270

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## 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, Daniel Mroz, M.E.Sc., EIT and Sugitha Anandakumar, M.Eng., P.Eng., PMP. It was reviewed by Thomas Lardner, Ph.D., P.Eng. and TaeChul Kim, M.E.Sc., P.Eng. and Stan E. Gonsalves, M.Eng., P.Eng., Designated MTO Foundation Contact.

Yours truly,

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## Part II: Foundation Design Report

Discussion and Engineering Recommendations for CP Rail Overhead Structure (Site 37X-0213/B1 & B3)

## 8.0 Discussion and Recommendations

### 8.1 General

This section of the report provides geotechnical design recommendations for the design and construction of the Highway 401 Eastbound Core and Collector CP Rail overhead structure and south retaining wall replacement. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the current investigation at the site conducted by EXP. Previous investigations by others as noted in this report, available through GEOCRESS and provided by the sponsor, were used to aid in assessments. The interpretation and recommendations provided are intended solely to permit designers to assess foundation alternatives and design the full replacement and widening of the bridge. 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 overhead structure is a 27 m long (on skew) single-span concrete structure. The existing abutments are supported on approximately 3.0 m widespread footings founded at approximately Elev. 173.0 m at the east abutment and 173.3 m at the west abutment (as per Golder Preliminary FIR, W.O. 07-20012, Report No. 09-1111-6055, April 2012). Based on the contract package drawing for Hwy 401 Westbound Core and Collector Lanes, CPR Overhead Replacement, CP Mile 200.26 Belleville Subdivision (CONT. NO. 2019-2011, WP No. 2394/2393-15-01, DWG No. 1), the existing Highway 401 pavement grade is at approximately Elev. 184.2 m to 184.8 m at the structure location, and the CP rail track grade is at approximately Elev. 174.8 m.

It is understood that the existing single span CP Rail overhead structure is to undergo full structure replacement along the same alignment. The construction of the new overhead bridge would involve widening by 10.0 m on the south side of the existing highway and replacement of the existing retaining wall structures (37x-1768/W and 37x-1769/W) with a new retaining wall along the south side of Highway 401. The new structure is proposed to be about 27.3 m long from centerline of west to east abutment (measurement parallel to highway alignment) based on the 2024 general arrangement drawings prepared by AECOM. For the widening, a permanent grade raise of up to 9.5 m with respect to original ground level on the south side of the existing highway will be required. Based on the detailed geotechnical investigation, the groundwater level is interpreted to be between approximate Elev. 168.3 m to 168.6 m across the site. 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 factors not evident at the time of measurement. A detailed description of the soil and groundwater encountered is discussed in Section 6 of this report.

This part of the report addresses the geotechnical design of the foundation for the bridge replacement and 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)*, American Railway Engineering and Maintenance-of-Way Association (AREMA) (2019) 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 assessment of slope stability and settlement of approach embankments, lateral earth pressure on structure, site preparation, excavation and frost protection.

### 8.2 Structure Foundations

At the time of preparing this report, shallow foundation type for the replacement structure is considered. 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 new abutments. 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 bridge structure**

Location	Founding Soil Type <sup>2</sup>	Excavation Elevation (m)	Recommended Founding Elevation (m)	Footing Width (m)	Factored Geotechnical Resistance at ULS (kPa) <sup>1</sup>	Factored Serviceability Geotechnical Resistance (for 25 mm settlement) (kPa)
<b>East Abutment<sup>1</sup></b> (BH22-2-2, BH22-2-4 & 75-10)	100 mm thick working slab over very dense sand and silt till	171.9	172.0 or below	3.0	1300	800
				6.5	1400	600
<b>West Abutment<sup>1</sup></b> (BH22-2-1, BH22-2-3 & 75-11)	100 mm thick working slab over compact to very dense sand and silt till	172.2	172.3 or below	3.0	1100	650
				6.5	1500	550

Typical vertical subgrade modulus ( $k_s$ ) for standard steel plate measuring 0.3 m x 0.3 m for dense to very dense non-cohesive till is 160 MPa/m above groundwater table. If below groundwater table, the typical vertical subgrade modulus for dense to very dense non-cohesive till is 96 MPa/m.

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

Wing/RSS walls are proposed to be constructed behind the (west and east) abutment, adjacent to the south side of Highway 401. Based on the proposed construction, the geotechnical resistances for a structure founded on engineered fill are tabulated below.



**Table 2.2: Recommended shallow foundation design parameters for Wing wall/RSS wall**

Location	Founding Elevation <sup>1</sup> (m)	Footing Width (m)	Founding Soil Type	Factored Geotechnical Resistance at ULS (kPa) <sup>1</sup>	Factored Serviceability Geotechnical Resistance (for 25 mm settlement) (kPa)
East Abutment Wing Wall	172.2 <sup>2</sup>	>1.0	Minimum 0.3 m thick Granular A pad <sup>3</sup> over very dense silt and sand till	750	450
West Abutment Wing Wall	172.5 <sup>2</sup>	>1.0	Minimum 0.3 m thick Granular A pad <sup>3</sup> over compact to very dense silt and sand till	750	450

**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 same as spread footings (see Section 7.2.1.1)
- (3) The granular material used for the granular pad shall be granular 'A' conforming to OPSS 1010 and compacted to 100% SPMD.

### 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 ( $\tan \delta$ ) = 0.5
Between pre-cast concrete and compacted granular fill	Coefficient of friction ( $\tan \delta$ ) = 0.45

The listed values are unfactored; in accordance with the CHBDC, a factor of 0.8 is to be applied in calculating the horizontal resistance. To achieve the full value of horizontal resistance for granular fill, a minimum thickness of 300 mm is required.

#### 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.

### 8.2.2 Deep Foundations

#### 8.2.2.1 General

The bridge can be supported on Steel H-piles or steel pipe piles driven to or into the generally dense to very dense sand and silt till.

Steel H-piles have advantages as they can be driven into a relatively strong (hard or dense) stratum offering relatively high carrying capacity, can be readily lengthened or cut to size, and they can be relatively roughly handled during delivery with little hazard of damage. These piles have minimal disturbance to neighboring piles or structures.

#### 8.2.2.2 Geotechnical Axial Capacity

Based on the subsurface conditions encountered at this site, the design parameters given in Table 2.4 are recommended for the purpose of the CHBDC/CSA S6:19. The table also provides the recommended pile tip elevations for estimating the pile lengths. 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 0.4) 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 2.4: Summary of recommended deep foundations (H-Piles)**

Foundati on Unit	Relevant Borehole	Estimated Tip Elevation (m)	Approximate Design Pile Length <sup>1</sup> (m)	Factored Axial Geotechnical Resistance at ULS (kN/pile)		Factored Serviceability Geotechnical Axial Resistance (kN/pile) <sup>2</sup>		Pile Founding Stratum
				HP310x79	HP310x110	HP310x79	HP310x110	
<b>East Abut. (Core and Collector)</b>	BH22-2-2, BH22-2-4 & 75-10	160	12.0	950	1300	800	1100	Sand and Silt Till
<b>West Abut. (Core and Collector)</b>	BH22-2-1, BH22-2-3 & 75-11	162	10.3	650	900	550	750	Sand and Silt Till

Notes:

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(1) Assuming bottom of pile cap at Elev. 163.2 m (1.2 m below existing ground at Elev. 164.4 m based on Hwy 401 Westbound Core and Collector Lanes, Bridge Replacement GO Transit/Metrolinx Mile 56.3 Uxbridge Subdivision General Arrangement Drawings, CONT. NO. 2019-2011, WP No. 2395/2397-15-01, DWG No. 1).

(2) for 25 mm total settlement.

Closed-end, concrete filled, 325 mm diameter, 9.5 mm (+) wall thickness steel pipe piles can provide similar axial resistances; however, these piles are less suitable for integral abutments and more likely to 'hang-up' during driving at levels above the desired penetration.

If an integral abutment is adopted, CSP filled with loose uniform sand in a predrilled oversized hole will be required to reduce resistance to lateral movements and reduce stresses on piles. The annular space between the pre-augured oversized hole and the pile shall be backfilled with uniformly graded sand (Ottawa type sand). The gradation for the uniformly graded sand shall be as provided in Table 2.5.

**Table 2.5: Backfill to integral abutment – augured hole**

MTO Sieve Designation		Percent Passing
2 mm	#10	100%
600 µm	#30	80% to 100%
420 µm	#40	40% to 80%
250 µm	#60	5% to 25%
150 µm	#100	0% to 6%

Commercially available materials which meet the gradation provided in Table 2.5 may be considered. The depth of such holes below the abutment shall be at least 3.0 m. Reference is made to 'Integral Abutment Manual' published by Ronen House from MTO Structural Office in which the requirements for sand fill to CSP are also presented. In addition, as per the manual, the piles for integral abutments should be in one row.

For integral abutments set within RSS walls, consideration must be given to the potential for lateral load transfer to the RSS walls system from the pile foundations. To eliminate this issue, it is recommended that the piles be set within a double CSP pipe system in accordance with Figure 7 (see Appendix H) of the MTO Integral Abutment Design Manual. The piles should be set in the inner 600 mm CSP pipe, with the annular space filled with Ottawa sand or equivalent approved uniform sand material which does not compact under cyclic loading. The annular space between the inner CSP pipe and the outer 800 mm diameter CSP pipe should be left empty to isolate the pile system from components of the RSS wall. For the detailed design, MTO Integral Abutment Bridge Design Manual and MTO RSS Design Guideline should be referenced. Should a single CSP pipe system be the preferred option, then lateral loads from the piles need to be taken into account in the design of the RSS wall.

### 8.2.2.3 Resistance to Lateral Loads

In integral abutments, the resistance to lateral load will have to be derived from the soil in front of the vertical piles. The resistance to lateral load in front of a vertical pile may be calculated using subgrade reaction theory (Broms' Method) where the coefficient of lateral subgrade reaction,  $K_{py}$  (MPa/m), is based on the following equations:

For non-cohesive soils:

$$K_{py} = n_h(z/d)$$

For cohesive soils:

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$$K_{py} = 67C_u/d$$

Where:

$K_{py}$	coefficient of horizontal subgrade reaction (MPa/m)
$d$	pile diameter/ width (m)
$n_h$	constant of horizontal subgrade reaction (MPa/m)
$z$	depth below ground surface (m)
$C_u$	Undrained Shear Strength (kPa)

As an alternative, the resistance to lateral load in front of a vertical pile may be calculated using the following geotechnical design parameters to determine a PY curve (lateral deflection vs. resistance).

Table 2.6 presents the estimated soil properties and their geotechnical parameters for abutments and piers. The data presented in the tables can be used for lateral load analyses using LPILE or equivalent software.

The notations (other than those explained above) used in the table are defined below:

NSPT	Standard Penetration Test, N-value
$\gamma$	bulk unit weight (kN/m <sup>3</sup> )
$\phi$	internal friction angle (deg)
$\delta$	friction angle between steel pile and soils (deg)
$\epsilon_{50}$	strain corresponding to 50% of the maximum principal stress difference
$K_p$	coefficient of passive earth pressure

Group action for lateral loading should be considered by Reese method using reduction factors on the single pile capacity depending on the geometry of the pile layout.

The reduction factors are as follows:

Reduction factors for the piles in a row.

$$e = 1 \quad \text{for } s/b \geq 3.75$$

$$e = 0.64(s/b)^{0.34} \quad \text{for } 1 \leq (s/b) < 3.75$$

Reduction factors for leading piles in a line.

$$e = 1 \quad \text{for } s/b \geq 4.0$$

$$e = 0.7(s/b)^{0.26} \quad \text{for } 1 \leq s/b < 4.0$$

Reduction factors for trailing piles in a line.

$$e = 1 \quad \text{for } s/b \geq 7.0$$

$$e = 0.48(s/b)^{0.38} \quad \text{for } 1 \leq (s/b) < 7.0$$

The notations used in the table are defined below:

$e$	Reduction Factor
$s$	Center-to-Center Pile Spacing
$b$	Pile Diameter

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**Table 2.6: Parameters for lateral load analyses**

Strata	Elevation (m)	Type of Soil	N <sub>SPT</sub>	$\gamma$ (kN/m <sup>3</sup> )	C <sub>u</sub> (kPa)	$\phi$ (°)	$\delta$ (°)	K <sub>py</sub> (MN/m <sup>3</sup> )		$\epsilon_{50}$	$n_h$ (MN/m <sup>3</sup> )	K <sub>p</sub>
								Static	Cyclic			
Engineered Fill	-	Cohesionless	-	21.0	-	30	14	10.0	10.0	-	6.6	3.0
<b>East Abutment Core and Collector (BH22-2-02, BH22-2-04, BH22-2-14 and 75-10)</b>												
Sand and Silt Till, A.W.T. (compact to very dense)	178.8-168.6	Cohesionless	28-168/300 mm	21	-	34	11	16	16	-	4.4	3.4
Sand and Silt Till, B.W.T. (very dense)	168.6-160.0	Cohesionless	62-100/127 mm	22	-	35	12	34	34	-	12.5	3.7
<b>West Abutment Core and Collector (BH22-2-01, BH22-2-03, BH22-2-13 and 75-11)</b>												
Sand and Silt Till, A.W.T. (compact to very dense)	174.7-172.0	Cohesionless	28-100/130mm	21	-	34	11	16	16	-	4.4	3.4
Sand and Silt Till, A.W.T. (dense to very dense)	172.0-168.6	Cohesionless	59-100/130mm	22	-	35	12	34	34	-	12.5	3.7
Sand and Silt Till, B.W.T. (dense to very dense)	168.6-160.1	Cohesionless	62-100/150mm	22	-	35	12	34	34	-	12.5	3.7

A.W.T. – Above Water Table, B.W.T. – Below Water Table

#### 8.2.2.4 Downdrag

The amount of relative settlement between soil and the pile that is necessary to mobilize negative shaft resistance/downdrag is about 10 to 12 mm. Therefore, negative shaft resistance will occur on the pile shaft in each soil layer or portion of a soil layer with a settlement greater than 10 mm. Current design involves minimal additional loading resulting in negligible expected loading. As such, downdrag is not expected to be an issue. If the proposed design results in an increase in loading greater than 10% or if a widening of the bridge is required, additional settlement analysis is required to estimate the potential loading due to downdrag.

##### **Methods for reducing negative shaft resistance forces:**

1. Use lightweight fill material

Construct structural fills using lightweight fill material such as foam concrete, geofoam, blast furnace slag, expanded shales fill to reduce the downdrag loads.

2. Use a friction reducer

Bitumen coating and plastic wrap are two methods commonly used to reduce the friction at the pile-soil interface. Bitumen coating should only be applied to the portion of the pile which will be embedded in the negative shaft resistance zone. Case histories on bitumen coatings have reported reduction in negative shaft resistance from as little as 47% to as much as 90%.

3. Reduce soil settlement

Preconsolidation of compressible soils can be achieved by preloading and consolidating the soils prior to pile installation. Wick drains are often used in conjunction with preloading in order to shorten the time required for consolidation.

#### 8.2.2.5 Pile Installation

Piles should be installed in accordance with OPSS 903 as amended by SP109F57. The possibility of piles encountering potential cobbles and boulders in the till layers should be anticipated. In addition, 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 soil. An example of a NSSP is included in Appendix K. In view of this, the piles should be stiffened as per OPSD 3000.100, Type I to minimize damage to the piles in anticipation of heavy driving conditions. It is advised that the piles incorporate pile flange reinforcement, or be fitted with a driving shoe section, offering some protection against buckling at the toe as the piles are driven through the glacial till deposits. Care must be taken to avoid overdriving and damaging the pile tip (i.e., the structural capacity of the piles should not be exceeded).

Prior to driving piles, a wave equation (WEAP) analysis should be performed in order to assess the driving stresses and the anticipated penetration resistance required to develop the required pile capacity. This analysis considers the complete driving system. The piles should be driven to adequate set cognizant of the pile driving equipment chosen for the particular piles. Development of the design capacity will depend on the chosen pile dimensions and driving techniques. Accordingly, a pile hammer will be required that can develop sufficient energy to efficiently drive the piles to the requisite driving resistance compatible with the design loads yet limit the input energy so as not to overstress the pile during driving. For the conditions at this site, piles shall be driven with an approved hammer with a manufacturer's maximum rated potential energy of not less than 95 kJ (70,000 ft-lbs) per hammer blow and measured energy >50 kJ. The final driving resistance required to achieve the design load can be determined by the Pile Driving Analyzer. Dynamic testing (PDA testing) on a number of piles with the Pile Driving Analyzer must be performed near the beginning of the pile driving phase of construction to confirm the pile capacities. Ten

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percent (10%) of the piles, but no fewer than three (3) per site, should be tested to confirm pile capacities have been achieved. Alternatively, static load tests can be performed, although these are typically much more difficult to set up and are costlier.

MTO permits the control of pile installation using the 'Hiley Formula' in similar settings.

In addition, all piles should be visually monitored by experienced personnel during installation to check for plumbness, set, internal damage, etc. All damaged piles should be rejected and if the damage is considered to be minor, the pile can be dynamically tested to determine the available pile capacity.

Piles in groups should be spaced no closer than 3 pile diameters. All piles in a group should be checked for heaving during the driving of the adjacent piles.

Given the nature of founding materials at this site (very dense sandy soil/ glacial tills below the GWT), relaxation after initial pile driving is possible. In the field, a number of piles should be monitored with the Pile Driving Analyzer for the end of initial driving and restrike conditions to check for relaxation as well as to confirm the ultimate bearing capacity of the piles. If the termination levels of adjacent piles penetrate deeper than a 3 horizontal to 2 vertical line drawn down from the toe of the previously driven higher piles, the higher piles should be re-driven to the established penetration resistance. During the driving of piles in a group, the vertical elevation of the piles should be monitored. If more than 5 mm of heaving occurs during the driving of adjacent piles, the heaved piles should be re-driven to the established penetration resistance. Additionally, selected piles should be restruck to check for relaxation. The actual amount of restriking should be 10% or a minimum of 2 piles at the site. Note that the presence or absence of relaxation will influence the need to restrike additional piles (up to 100%). In conditions where some relaxation is expected or is observed, an alternative approach is to overdrive piles (without inducing damage) to a set such that the final set after relaxation meets the established penetration resistance. This would reduce the need for restriking at locations where relaxation might occur, provided that a test program is carried out to determine the driving requirements.

MTO permits the control of pile installation using the 'Hiley Formula'. If this method is chosen to control the pile installation, 'Hiley Formula' can apply in similar settings as shown on MTO standard drawings SS103-11 'Pile Driving Control'. Based on MTO experience with the Hiley formula, a resistance factor equal to 0.5 may be used on the ultimate resistance to verify the factors ULS design values. Assessment of the ultimate geotechnical resistance by the Hiley formula should commence once the pile reaches a depth of not less than 1.5 m above the design pile tip elevation that presented in Table 2.3 and at 0.5 m intervals of depth until the ultimate axial resistance is achieved. If the ultimate capacity as determined by the Hiley formula is not achieved within the 1.5 m interval down to the design pile tip elevation, the Contractor should notify the Contract Administrator. At this depth, the pile should be allowed to rest for 48 hours and the Hiley formula should then be applied immediately upon re-striking the pile. If the ultimate capacity is still not achieved after the 48 hour wait period, the Contract Administrator should be notified, and authorization given prior to driving the pile below the design pile tip elevation.

Wherever practical, embankments should be constructed first before installing piles and other foundation elements in accordance with OPSS 903 as amended by SP109F57. If not practical due to construction sequence issues, negative skin friction/downdrag must be treated as an additional load to the piles. This is particularly important where significant consolidation settlements are anticipated based on the geometry and subsoil conditions. With this sequencing, some consolidation will occur before pile installation, thereby mitigating issues related to differential settlements at the approaches and downdrag on the piles. It will also permit better compaction conditions for embankment materials in the area of the piles.

The specific period of delay between the two events that would be required to reduce the continuing movements to levels acceptable for service and/or permit the ignoring of negative skin friction issues, must be assessed on a case-by-case basis. For those construction conditions where the piles are installed prior to embankment construction, the requirements for reducing post-construction settlements of the embankment to acceptable levels and accommodation of down drag on the piles must be

assessed and included in the design and construction. This includes such measures as the need for preloads and surcharges and/or wick drains and associated instrumentation and monitoring, as well as specific delays of final paving, as applicable.

#### 8.2.2.6 Caissons

Shallow foundation options have been selected for this bridge. The following information on caissons option is provided for the completeness of this report.

##### 8.2.2.6.1 General

All methods of caisson construction involve rotary drilling and can be classified into three broad categories. These are: (1) the dry method, (2) the casing method, and (3) the wet method. The use of augered caisson foundations may be considered at this site by using the wet method of construction, sometimes referred to as the slurry-displacement method.

The slurry displacement method is a viable option at this site where caving soil is present. It is most likely the only feasible option in a permeable, water bearing soil which is impossible to seal casing into. The placement of concrete must be made by use of a tremie (steel tube) or flexible hose or pump. However, the caissons would extend into cohesionless silty soils well below the groundwater levels measured in open holes upon completion of drilling, and potential for instability of the shaft sidewalls and caisson base during installation could happen.

Construction of the caissons would require use of a liner and or slurry methods to control groundwater, support the sidewalls of the shaft, and prevent heave in the base while augering in the saturated non-cohesive deposits. A thickness of mudcake at the shaft wall will reduce the roughness of the contact surface between the concrete and the soil and will also reduce the angle of the sliding friction between two materials. The solution to the problem of reduced skin friction due to excessive filter cake is therefore to maintain the properties of the slurry within tolerable limits and to place concrete within four hours after the excavation is completed.

The caisson excavation bottom is considered clean if a minimum of 50 % of the bottom area has less than ½ inch (13 mm) of sediment and no portion of the bottom area has more than 1 ½ inches (38 mm) of sediment.

Man entry into the caissons to hand clean the caisson bases is not generally allowed in the Province of Ontario. One or more of the following inspection procedures may be used to inspect the cleanliness of the caisson excavation bottom prior to placement of the reinforcement steel and concrete.

#### 1. Steel Probe

A steel probe can be lowered to the bottom of the caisson excavation to check whether cleaning has been satisfactorily completed. A steel probe 0.6 m in length with a flat tip on the sounding end and a weight of approximately four kilograms (#32 rebar) can be suspended from the opposite end with a non-stretch cable.

#### 2. Shaft Inspection Device (SID)

The engineer may use a SID to take sediment measurements and observe the bottom conditions of the caisson excavation at a minimum of five locations selected by the engineer. The SID is a remotely operated camera capable of observing bottom conditions and measuring sediment under water and slurry. Each SID inspection (including all 5 locations) takes approximately 1 hour after the equipment has been set up.



### 8.2.2.6.2 Geotechnical Axial Resistance

The resistance values and base elevations in Table 2.7 are recommended for caissons of 1.2 and 1.5 m diameter founded within hard till. 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 0.4) 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 2.7: Caisson geotechnical resistance**

Foundation Unit	Relevant Borehole	Founding Stratum	Recommended Foundation Elevation (m)	1.2 m Diam. Caisson		1.5 m Diam. Caisson	
				Factored ULS (KN)	Factored SLS (KN)	Factored ULS (KN)	Factored SLS (KN)
<b>East Abut. (Core and Collector)</b>	(BH22-2-04, BH22-2-02 & 75-10)	Very dense sand and silt till	160	3000	N/A	4500	N/A
<b>West Abut. (Core and Collector)</b>	(BH22-2-03, BH22-2-01 & 75-11)	Very dense sand and silt till	162	3000	N/A	4500	N/A

Given the water encountered below the caissons base, techniques such as tremie concreting and maintaining a head of concrete within the linear may be required to ensure satisfactory caisson construction. Permanent liners can be considered but would add significant cost.

To determine the structural defects in caissons, non-destructive evaluation tests which may include (a) downhole tests conducted in access tubes, including cross hole acoustic tests and backscatter gamma ray (gamma-gamma) tests, or (b) sonic echo tests can be used.

Caisson static load tests shall be completed as described in ASTM D1143-81 (compression test quick method) and ASTM D3966-90 (Lateral Test) or as per the designer's specification.

The recommended resistances are contingent on satisfactory cleaning of bases; the measures to check the base has been described above. It is recognized that under these conditions, there may be cases where the base condition may be difficult to verify.

Based on the ground water readings (dewatering and liner requirements), encountered till deposits (drilling through cobbles and boulders) and given the uncertainties associated with cleaning and inspection of the caisson base, caissons are not the preferred option for the CP Rail overhead structure, noting also that shallow foundations are currently being considered by the designer.

## 8.3 Approach Embankments

### 8.3.1 General

Based on the information provided to EXP by the client, for the replacement of the existing Highway 401 CP Rail overhead structure, the existing highway will be widened by about 10 m on the south side of the highway, which will require a grade raise of about 9.5 m where the embankment will be widened. As embankment is greater than 8.0 m in height, a 2 m wide mid-height

bench (OPSD 202.010) is required beyond the RSS walls, as applicable. The slopes of the embankments should be provided with adequate erosion protection against surface water runoff. The global stability of forward slopes 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 CP Rail overhead structure based on GA drawings, stability analyses were carried out for the most critical section at the east abutment/embankments in the south side.

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 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 7.4) 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 2.8. In addition, a traffic surcharge pressure of 16 kPa was adopted in the slope stability assessments for the abutments and approach embankment.

**Table 2.8 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	35	-	35	-	168.6
Granular B Type I	21	32	-	32	-	
Engineered Fill	21	30	-	30	-	
Gravelly Sand to Sand and Gravel Fill (compact to very dense)	20	32	-	30	-	
Silty Sand to Sand and Silt Fill (generally compact to dense)	20	30	-	30	-	
Clayey Silt Fill (firm to hard)	20	-	100	30	-	
Clayey Silt Till (hard)	22	-	150	32	-	
Sand and Silt Till (generally dense to very dense)	22	35	-	35	-	

The results of the slope stability analyses are shown in Figures 1 through 6 in Appendix F of this report and are summarized in Table 2.9 below. As can be seen, the calculated minimum factor of safety of critical slip surfaces with existing fill meets the design criteria for static and seismic conditions for long-term (drained) conditions with embankment slopes of 2H:1V with a 2 m wide mid-height for embankments greater than 8.0 m in height. Therefore, based on these global stability analyses, the proposed ~9.5 m high abutment and embankment side slopes can be safely constructed with slopes of 2H:1V.

**Table 2.9 Summary of results of slope stability analyses**

Location	Maximum Height (m)	Conditions	Min FoS
West Abutment	~9.5	Drained long-term conditions, static condition	1.9
		Drained long-term conditions, seismic condition	1.7
West Abutment Side Slope w/ Retaining Wall		Drained long-term conditions, static condition	1.8
		Drained long-term conditions, seismic condition	1.6
Widened Embankment - West of Structure		Drained long-term conditions, static condition	1.5
		Drained long-term conditions, seismic condition	1.2

### 8.3.3 Settlement Considerations

#### 8.3.3.1 Settlement of Foundation Soils

Given the nature of founding materials at this site, settlement of the replacement structure should not exceed 25 mm for footings designs. As per the GA drawing, no grade raise is proposed at locations where replacement is occurring. Therefore, settlement will be limited to self-settlement of the engineering fill (discussed in Section 7.3.3.2), with limited differential settlement between the proposed works and the existing westbound structure.

However, grade raise for the widening 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 need for additional measures will be determined through analysis of settlement monitoring results.

The magnitudes of total settlement for the critical section of the west approach embankment has been assessed based on Standard Penetration Test (SPT) results, consolidation test results, and Elastic modulus which is correlated from the available data adopted in the settlement analyses for the east approach embankment is summarized in Table 2.10.

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 2.10: Soil parameters used in settlement analyses**

Soil Layers	Unit Weight (kN/m <sup>3</sup> )	E (MPa)	Compression Index (Cc)	Recompression Index (Cr)	Void Ratio (e)	Preconsolidation Pressure (p' <sub>c</sub> ) (kPa)
Sand and silt till (Dense to Very dense)	22	120	-	-	-	-
Silty sand to sandy silt (Very dense)	22	250	-	-	-	-

The summary of results of settlement analyses for the approached embankments is given in Table 2.11. The Settle 3D results of these cases can be seen Appendix G.

**Table 2.11: Summary of results of settlement analyses**

Locations	Abutment Height (m)	Assumed Embankment Width (m)	Calculated Immediate Settlement (mm)	Calculated Consolidation Settlement (mm)	Calculated Total Settlement (mm)
<b>West approach</b>	9.5	10	19.5	0	19.5

The settlement analyses suggested that the total settlement of the east and west approach embankments under the new fill about 9.5 m high could be approximately 19.5 mm. The settlement is expected to occur relatively quickly following construction of widened approach embankments.

For new embankment approaches to structural elements, MTO settlement criteria are as follows: the post construction settlement is limited to 25 mm ; 50 mm ; 75 mm ; >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 indicated in this report. 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. To minimize the post construction settlement, the fill materials may be compacted to 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 2 months depending on the nature of embankment fill employed. As stated above, where the 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

A program of instrumentation and monitoring is recommended to verify that the actual settlements are within allowable limits. The program will include a series of surface monitoring points and survey pins.

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 Settlement Pins 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 Settlement Pins shall be installed at a standard spacing on the shoulder of the embankment. The monitoring locations shall be at a spacing of 5 m for a minimum distance of 20 m behind the proposed abutment. The Settlement Pins 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. Typical sketch of the monitoring instrumentation plan and details are attached in Appendix M.

The concrete shall (OPSS.PROV 1350) have a minimum compressive strength of 25 MPa 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 Settlement pins:

- Northing and Easting of Each Settlement Pin 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 generally dense to very dense sand and silt till and stiff to hard clayey silt till. Bedrock was not encountered within the investigated depth. The groundwater level is at about 6.2 m to 6.5 m depth below the existing CN rail grade. The reported N-values for the soil below the founding level ranged from 17 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 (43.772964°N, 79.292316°W), where the damped spectral accelerations at the project site are shown in Table 2.12 below:

**Table 2.12: Seismic design values**

Probability of Exceedance in 50 Years	$Sa(0.2)$ (g)	$Sa(0.5)$ (g)	$Sa(1.0)$ (g)	$Sa(2.0)$ (g)	$PGA$ (g)
2%	0.325	0.198	0.104	0.0481	0.176

The GSC seismic hazard calculation data sheet for this site is attached in Appendix L.

Based on soils and groundwater condition encountered (i.e. glacial tills with average corrected SPT blow count over 25 blows/305 mm in sand and non-plastic silts, CHBDC 6.14.8.1.2), no liquefaction is expected due to the ground motion from an earthquake

having a 2% probability of exceedance in a 50-year period. In addition, cyclic mobility of the native soils is also not expected for a 1 in 2475-year earthquake event.

## 8.5 Roadway Protection System

Temporary protection systems for construction are required to facilitate the structural replacement work. The temporary 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, CP Rail guidelines and AREMA Zones of Influence. 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-pile with lagging, should be practical options at this location. The subsurface condition 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 2.13.

It is considered that a sheet pile of sufficiently robust cross section could be driven through granular fill encountered at these sites, through the fill of abutments, and native deposits. Difficulties with installation may occur due to presence of cobbles and boulders in till and where occasional cobbles and boulders were encountered in the fill (i.e. the boulders were not encountered in the boreholes drilled during this investigation, however auger grinding experienced during drilling through the fill might suggest the presence of cobbles and boulders) requiring their removal before further driving or being 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 embankment fill or native very dense deposits. An example of a NSSP is included in Appendix K. Alternatively, an H-pile with lagging walls 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 2.13 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 3.0 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, 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 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 7.2.2.3. 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 26 of the CFEM (2023). The estimated factored (0.4) ULS resistance of grouted anchors would be approximately 40 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 option is ranked as more practical for this project due to possible obstructions which may be present within the fill. Design and construction specification for the chosen roadway protection system should be prepared in accordance with OPSS. PROV 539 CP Rail guidelines and AREMA Zones of Influence. Piling should be in accordance with OPSS. PROV 903. Cantilevered walls should be designed for the earth pressures coefficient presented in Section 7.2.2.3 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 loading adjacent to the wall.

For the design purposes, the unfactored static earth pressure parameters given in Table 2.14 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 2.14: 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	185.9 to 177.9	Silty Sand to Sand Fill (loose to dense)	30	0.33	3.0	0.50	20	168.3
	176.7 to 175.1	Clayey Silt Fill (stiff to hard) <sup>(2)</sup>	30	0.33	3.0	0.50	20	
	175.1 to 162.0	Sand and Silt Till (compact to very dense)	34	0.28	3.54	0.44	21	
East	185.1 to 179.8	Sand to Sand and Silt Fill (loose to dense)	30	0.33	3.0	0.50	20	168.5
	179.8 to 175.5	Clayey Silt Fill (firm to very stiff) <sup>(2)</sup>	29	0.35	2.9	0.52	19	
	175.5 to 174.7	Silty Sand Fill (compact)	30	0.33	3.0	0.50	20	
	174.7 to 172.0	Sand and Silt Till (compact to very dense)	34	0.28	3.54	0.44	21	
	172.0 to 160.1	Sand and Silt Till (very dense)	35	0.27	3.69	0.43	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  $K_a = K_p = 1$

## 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 restraining 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 2.15 may be assumed.

**Table 2.15: 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 2.16: 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 7.4. The acceleration coefficient determined at the original ground surface should be the acceleration coefficient acting at the wall base. The seismic vertical acceleration coefficient ( $k_v$ ) can be ignored when calculating the seismic lateral earth pressure coefficient. For design purposes, the following unfactored seismic lateral earth pressure parameters for non-yielding walls are provided in Table 2.17.

**Table 2.17: 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
Compacted Granular B Type I	32	0.42	2.92	21

## 8.7 Construction Considerations

### 8.7.1 Site Preparation and Embankment Construction

Prior to embankment construction, all heavy organic spots (topsoil, peat, organic soils, etc.), and any soft clayey silt spots below the footprint of the proposed embankments require to be excavated and replaced with clean and compactible soils with a minimum 95% of Standard Proctor Maximum Dry Density (SPMDD).

### 8.7.2 Excavation

All excavations must be carried out in accordance with the latest edition of the Ontario Occupational Health and Safety (OHSA) and good construction practice. The existing fill, which should be excavated for construction of the abutments (i.e. uncontrolled fill), are considered Type 3 soils above the groundwater table and Type 4 soils below the groundwater table.

The native dense to very dense sand and silt till which should be excavated for footings are considered Type 2 soils above groundwater and Type 3 soils below the groundwater table. Temporary excavations (i.e. those that are open only for a short period) may be made with side slopes not steeper than about 1H:1V, above and below the groundwater table 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 902 and SP109S12. The excavation should not undermine the existing walls.

### 8.7.3 Structure Backfill

The selection and placing of backfill should be in accordance with OPSS.PROV 902 and OPSD 3010.150 and OPSD3190.100. 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.

### 8.7.4 Groundwater and Surface Water Control

As mentioned in Section 6, based on an assessment of the water levels observed in the borings and the subsurface conditions, the groundwater levels are interpreted to be between approximately Elev. 168.3 m to 168.6 m across the CP Rail overhead structure. Water may also be perched in the fill at higher levels during wet periods. The soils encountered within potential excavation depths consist of sand and gravel, gravelly sand, sand, sand and silt, silty sand, and clayey silt fill, and native compact to very dense silty and sandy and stiff to hard clayey silt soils. The materials (particularly the deposits with high silt content) are highly susceptible to disturbance from groundwater and mobilized equipment. As such, the groundwater level needs to be controlled to 0.5 m below the excavation level to avoid disturbance. Given the conditions at this site, it is anticipated that control of seepage can be accomplished by conventional pumping from sumps in oversize excavations, where the excavation base is

within 0.6 m of the prevailing groundwater level at the time of construction. 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 runoff should be diverted from excavations.

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.

Detailed design of the dewatering program shall be the responsibility of the contractor.

#### 8.7.5 Embankment Construction

Assuming properly compacted acceptable inorganic earth fill materials are utilized, a minimum of 2H:1V side slopes can be used for the construction of the embankment widening approach fills. Where the embankment exceeds 8.0 m in height, a 2 m wide mid-height bench is to be provided in accordance with OPSD 202.010. In accordance with OPSD 208.010, benching of the existing Highway 401 embankment side slopes should be carried out to key in the new fill materials for the widening. The proposed 2H:1V slopes of the embankment should be provided with adequate erosion protection against surface water runoff.

The materials used for the construction of the embankment fills should consist of approved, acceptable earth fill (e.g. Select Subgrade Materials - OPSS.PROV 1010).

The native silty soil is easily disturbed during the rain or surface run off. The exposed slope surface should be covered with straw or plastic sheets as soon as the slope face is exposed. Care must be taken to properly compact the embankments to reduce settlements associated with fill density changes. Fill used for construction of the embankments should be in accordance with OPSS 212 and fill placement should meet or exceed the requirements of OPSS.PROV 206. Construction should be in accordance with OPSS.PROV 206. The fill should be placed in regular lifts with loose thickness not exceeding 300 mm and compacted to at least 95% of SPMDD. The final lift of fill prior to placement of the roadway granular subbase and base courses should be compacted to 100% of SPMDD.

Quality assurance should be provided as per MTO standard 501.08 (OPSS.PROV 501). Inspection and field density testing should be carried out by qualified personnel during placement operations to ensure that appropriate materials are used and that adequate levels of compaction have been achieved.

To reduce surface erosion on the embankment side slopes, prompt seed and cover (OPSS 804) or sodding (OPSS 803) should be carried out as soon as possible after construction of the embankment.

### 8.8 Corrosion Protection

Two (2) soil samples were selected for chemical analyses, during the 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 1.9 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 ranged from 7.91 to 7.98 which is within the normal range of soil pH of 5.5 to 8.5 (AASHTO, 2000/MTO Gravity Pipe Design Guidelines, May 2007). The chemical data indicates low resistivity of tested soil 660 – 1500 ohm-cm), which suggests severe potential for corrosion of buried metallic elements as per Table 3.2 of the MTO Gravity Pipe Design

Guideline. The measured chloride content was 380 to 770 ppm ( $\mu\text{g/g}$ ) which also indicates a low to moderate potential for additional corrosion (Molinas and Mommandi, 2009).

Based on these results, 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 2014, Section 2 "Durability" requirements are followed. The test results provided in Table 1.9 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 under 20 ppm ( $\mu\text{g/g}$ ), i.e. <0.002% and being less than 0.10%, does not require sulphate resistant cement.

## 8.9 Winter Conditions

In the event of construction during freezing temperatures, the foundation stratum should be protected from freezing by the use of loose straw, tarpaulins, propane heaters or other suitable means. In this regard, the base of the excavation should be insulated from sub-zero temperatures immediately upon exposure and until such time the abutment footings are protected with sufficient soil cover to prevent freezing at the foundation level.

## 8.10 Obstructions

Cobbles were encountered in the embankment fill during the investigation. Additionally, cobbles and boulders should be anticipated in the native till. Therefore care (i.e. pile flange reinforcement or be fitted with a driving shoe, as explained in Section 7.2.2.5) has to be taken during installation of piles. These potential obstructions may also impact excavations and/or elements of temporary protection systems. It is recommended that a NSSP be included in the Contract Documents to warn the Contractor of the possible presence of cobbles and/or boulders within the overburden soil. An example of a NSSP is included in Appendix K.

## 8.11 Geotechnical Instrumentation and Monitoring

Monitoring of the effect of the construction for the widening/replacement 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 and railway, shoring system and vibrations during replacement 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 should be performed in accordance with OPSS.PROV 539 and CP Rail guidelines. Therefore, for this site the following elements of monitoring are anticipated:

### 8.11.1 Precondition and postcondition surveys

A precondition survey of all existing structures and railways 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 accretions including the pavement surface, traffic barriers, and overhead lighting, 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 are 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.

### 8.11.2 Movements of Existing Structure

Survey points should be used to monitor movements of the existing overhead structures (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).
- Rail/track surface monitoring points to be installed on the webs of each rail at 4.0 m intervals for at least 12.0 m on either side of the proposed works.
- In-ground monitoring points shall be installed along the tracks at 4.0 m intervals for at least 12.0 m on either side of the proposed works, on both sides (approximately 150 mm from outside edge of the tie and equally spaced - if more than one track, the points can be installed along the centerline of each track. In-ground rods shall extend 1.2 m below ground surface.
- 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: Weekly 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.

#### Rail/Track Limits\*:

1. Review Level: If a maximum value of 4 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 9 mm and 12 mm relative to the baseline readings is reached horizontally and vertically, respectively 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.

*Note:*

*\* Class of track should be confirmed, it is estimated to be 3 or above.*

### 8.11.3 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.

### 8.11.4 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 has 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. 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(s) 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:

*Foundation Investigation and Design Report  
Highway 401 Eastbound from Victoria Park Avenue to Neilson Road  
Full Structure Replacement and Bridge Widening at CP Rail Overhead (Site 37X-0213/B1 & B3)  
Assignment No. 2021-E-0018  
Date: December 20, 2024*

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  
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Assignment No. 2021-E-0018  
Date: December 20, 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, Daniel Mroz, M.E.Sc., EIT and Sugitha Anandakumar, M.Eng., P.Eng., PMP. It was reviewed by Thomas Lardner, Ph.D., P.Eng. and TaeChul Kim, M.E.Sc., P.Eng. and Stan E. Gonsalves, M.Eng., P.Eng., Designated MTO Foundation Contact.


Yours truly,


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Encl.

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## 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




SHEET  
S26

## GENERAL ARRANGEMENT

GENERAL NOTES:

- CONSTRUCTION NOTES:

1. THE CONTRACTOR SHALL VERIFY ALL DIMENSIONS, DETAILS AND ELEVATIONS OF THE EXISTING STRUCTURE THAT ARE RELEVANT TO THE WORK SHOWN ON THE DRAWINGS PRIOR TO THE COMMENCEMENT OF THE WORK. ANY DISCREPANCIES SHALL BE REPORTED TO THE CONTRACT ADMINISTRATOR AND THE PROPOSED ADJUSTMENT OF THE WORK TO MATCH THE EXISTING STRUCTURE SHALL BE SUBMITTED FOR APPROVAL.
2. CONTRACTOR SHALL NOT REMOVE THE EXISTING SUPERSTRUCTURE UNTIL EXISTING APPROACH SLAB AND BACKFILL BEHIND ABUTMENT IS REMOVED. THE EXISTING BACKFILL SHALL BE REMOVED SIMULTANEOUSLY BEHIND BOTH ABUTMENTS. AT NO TIME SHALL THE DIFFERENCE IN ELEVATION BE GREATER THAN 300mm.
3. 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 DECK SOFFIT AND BEARING SEAT. ADDITIONAL COST DUE TO ANY CHANGES IN ELEVATIONS OF THE TOP OF BEARINGS BY THE CONTRACTOR SHALL BE AT THEIR OWN EXPENSE.
4. PROTECTION SYSTEMS SHALL BE DESIGNED TO PERFORMANCE LEVEL 2 CRITERIA. LIMITS OF PROTECTION SYSTEMS TO BE DETERMINED BY CONTRACTOR. PROTECTION SYSTEMS SHALL BE SUFFICIENT FOR ALL ACCESS AND WORKING PLATFORMS.
5. FOR TRAFFIC STAGING AND MAINTENANCE OF TRAFFIC, REFER TO CONSTRUCTION STAGING DRAWINGS.
6. THE CONTRACTOR WILL REQUIRE CP PROTECTION FOR ALL WORKS OCCURRING ON OR ABOUT THE RAILWAY RIGHT OF WAY.
7. TRACK SETTLEMENT MONITORING WILL BE REQUIRED FOR THE INSTALLATION OF THE TRACK PROTECTION SYSTEM AS APPROVED BY CP.

 NEW CONCRETE  
 REMOVAL  
 NEW ASPHALT

[illegible]

LIST OF ABBREVIATIONS:

LIST OF ABBREVIATIONS:




ABUT.	ABUTMENT
BRGS.	BEARINGS
C.J.	CONSTRUCTION JOINT
DIA.	DIAMETER
EB	EASTBOUND
EBL	EASTBOUND LANE
E.J.	EXPANSION JOINT
EL.	ELEVATION
EQ. SP.	EQUALLY SPACED
EXT.	EXISTING
EXP.	EXPANSION
REINF.	REINFORCING
SCL	SPEED CHANGE LANE
SHLD	SHOULDER
T/P	TOP OF PAVEMENT
TYP.	TYPICAL
WB	WESTBOUND
WBL	WESTBOUND LANE
WP	WORKING POINT

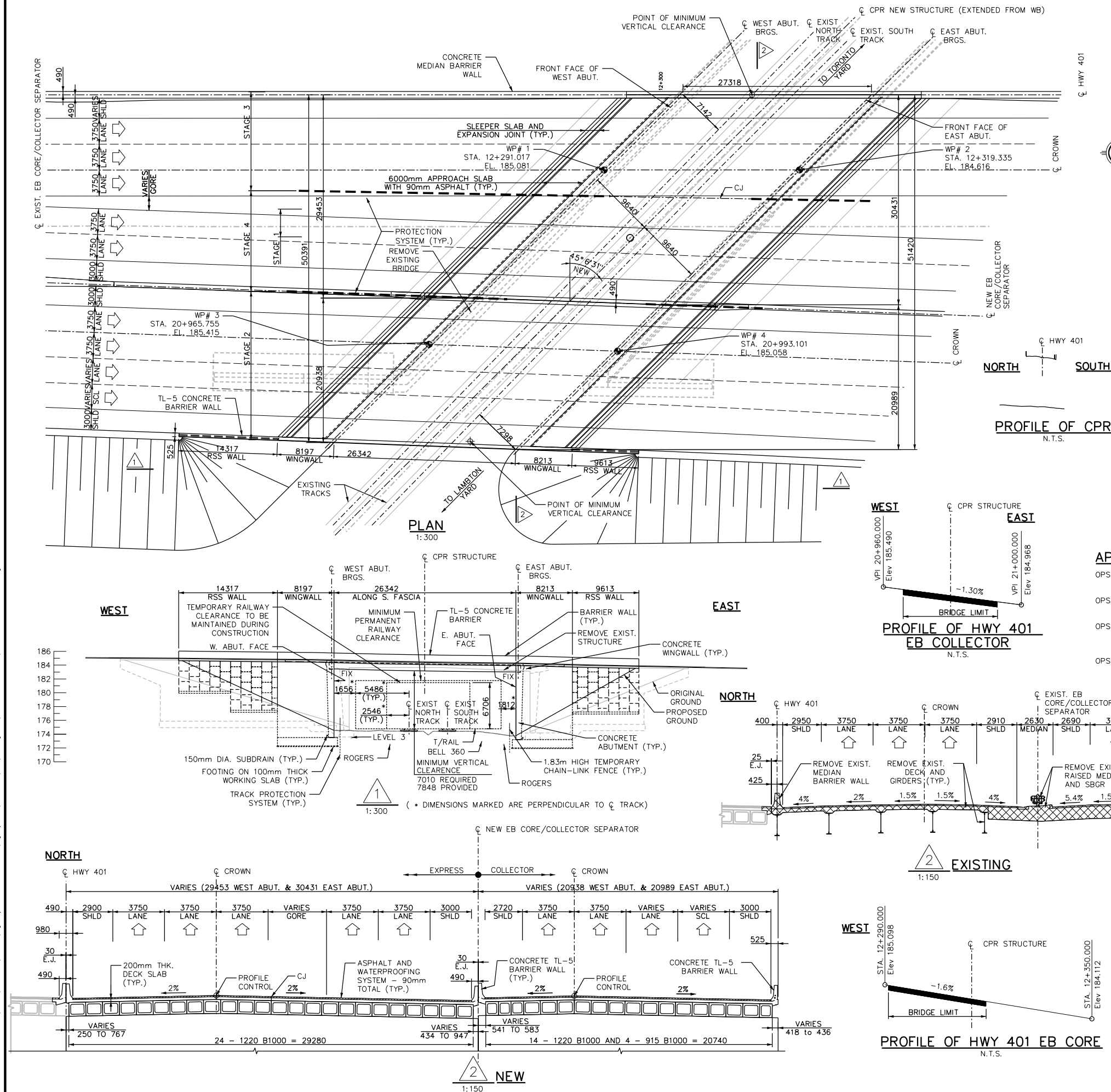
- R3-1. GENERAL ARRANGEMENT
- R3-2. BOREHOLE LOCATIONS
- R3-3. SOIL STRATA
- R3-4. CONSTRUCTION STAGING I
- R3-5. CONSTRUCTION STAGING II
- R3-6. FOOTING LAYOUT AND DETAILS
- R3-7. ABUTMENT LAYOUT
- R3-8. ABUTMENT REINFORCEMENT
- R3-9. WINGWALL REINFORCEMENT
- R3-10. RETAINING WALL LAYOUT
- R3-11. GIRDER LAYOUT
- R3-12. PRESTRESSED BOX GIRDERS AND BEARINGS I
- R3-13. PRESTRESSED BOX GIRDERS AND BEARINGS II
- R3-14. PRESTRESSED BOX GIRDERS AND BEARINGS III
- R3-15. DECK DETAILS AND REINFORCEMENT I
- R3-16. DECK DETAILS AND REINFORCEMENT II
- R3-17. SOUTH BARRIER WALL WITHOUT RAILING TL-5
- R3-18. MEDIAN BARRIER WALL WITHOUT RAILING TL-5
- R3-19. MEDIAN BARRIER WALL WITHOUT RAILING ON RSS WALL TL-5
- R3-20. 6000mm APPROACH SLAB I
- R3-21. 6000mm APPROACH SLAB II
- R3-22. EXPANSION JOINT AND SLEEPER SLAB
- R3-23. SEQUENCE OF EXPANSION JOINT INSTALLATION
- R3-24. STRIP SEAL EXPANSION JOINT FOR SLEEPER SLAB
- R3-25. TRACK PROTECTION
- R3-26. STANDARD AND MISCELLANEOUS REPAIR DETAILS

OPSD 0911.382	GUIDE RAIL SYSTEM, CONCRETE BARRIER DOWEL CONNECTION DETAIL
OPSD 3370.100	DECK, WATERPROOFING HOT APPLIED ASPHALT MEMBRANE WITH PROTECTION BOARD
OPSD 3370.101	DECK, WATERPROOFING HOT APPLIED ASPHALT MEMBRANE AT ACTIVE CRACKS GREATER THAN 2mm WIDE AND CONSTRUCTION JOINTS
OPSD 3941.200	FIGURES IN CONCRETE, SITE NUMBER AND DATE LAYOUT

CONSTRUCTION NOTES:

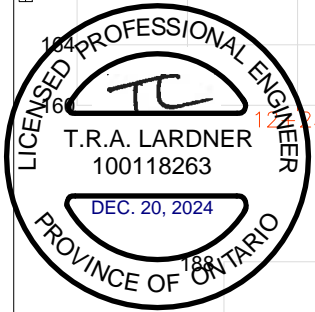
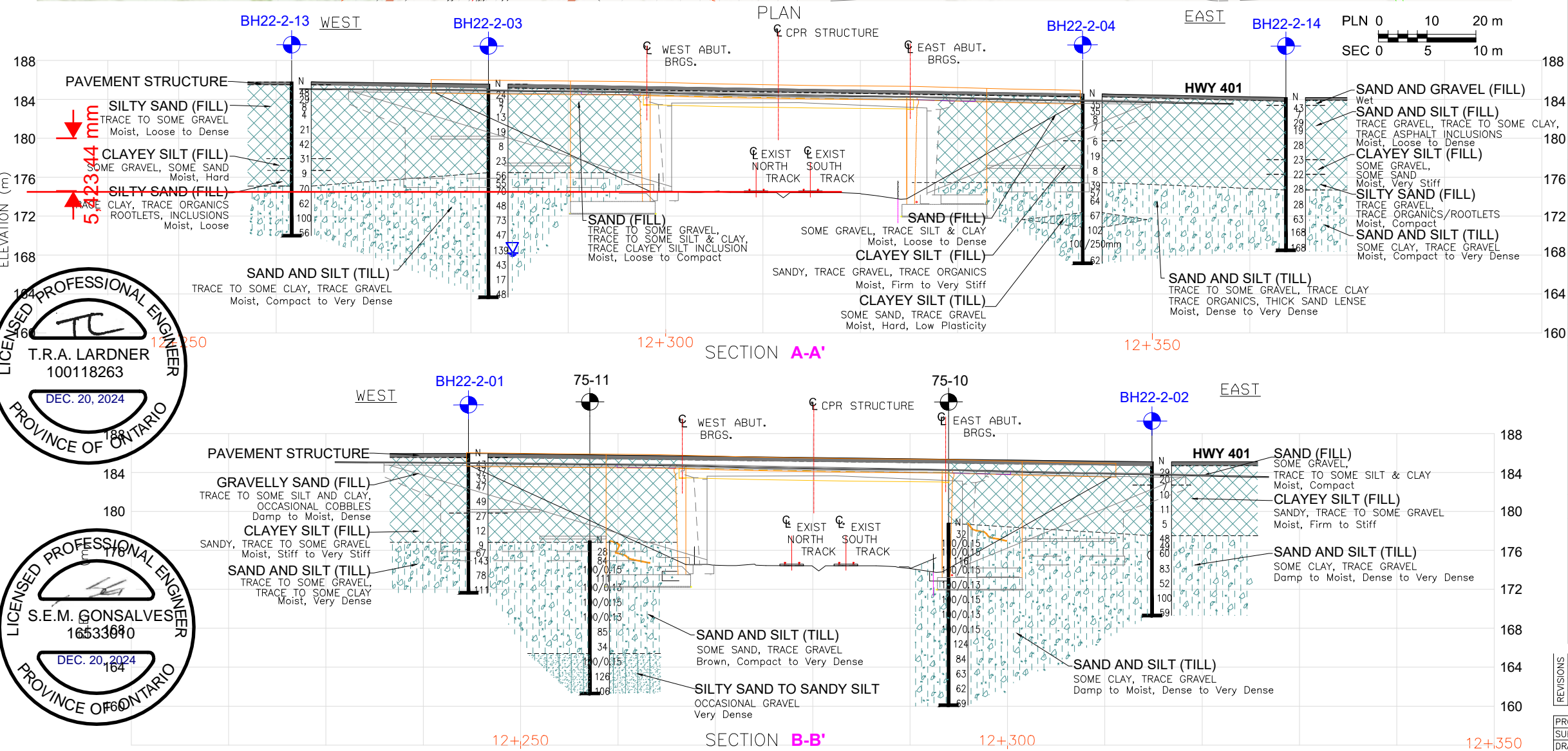
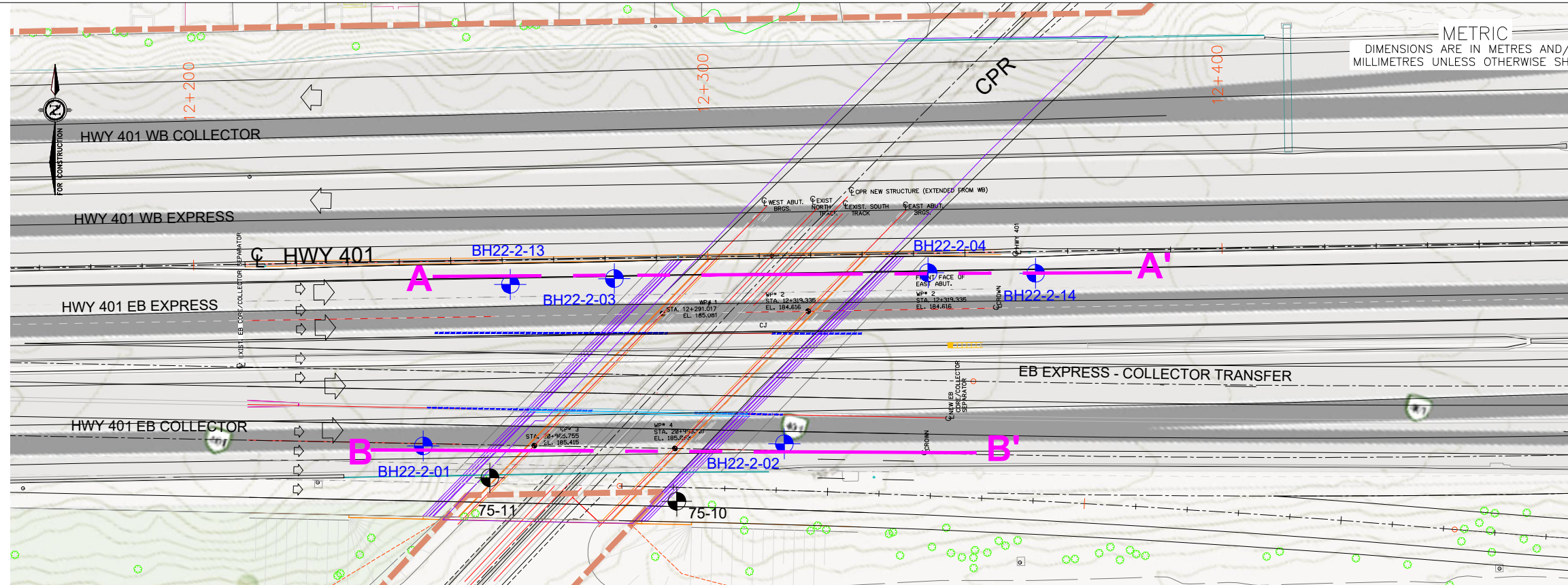
1. THE CONTRACTOR SHALL VERIFY ALL DIMENSIONS, DETAILS AND ELEVATIONS OF THE EXISTING STRUCTURE THAT ARE RELEVANT TO THE WORK SHOWN ON THE DRAWINGS PRIOR TO THE COMMENCEMENT OF THE WORK. ANY DISCREPANCIES SHALL BE REPORTED TO THE CONTRACT ADMINISTRATOR AND THE PROPOSED ADJUSTMENT OF THE WORK TO MATCH THE EXISTING STRUCTURE SHALL BE SUBMITTED FOR APPROVAL.
2. CONTRACTOR SHALL NOT REMOVE THE EXISTING SUPERSTRUCTURE UNTIL EXISTING APPROACH SLAB AND BACKFILL BEHIND ABUTMENT IS REMOVED. THE EXISTING BACKFILL SHALL BE REMOVED SIMULTANEOUSLY BEHIND BOTH ABUTMENTS. AT NO TIME SHALL THE DIFFERENCE IN ELEVATION BE GREATER THAN 300mm.
3. 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 DECK SOFFIT AND BEARING SEAT. ADDITIONAL COST DUE TO ANY CHANGES IN ELEVATIONS OF THE TOP OF BEARINGS BY THE CONTRACTOR SHALL BE AT THEIR OWN EXPENSE.
4. PROTECTION SYSTEMS SHALL BE DESIGNED TO PERFORMANCE LEVEL 2 CRITERIA. LIMITS OF PROTECTION SYSTEMS TO BE DETERMINED BY CONTRACTOR. PROTECTION SYSTEMS SHALL BE SUFFICIENT FOR ALL ACCESS AND WORKING PLATFORMS.
5. FOR TRAFFIC STAGING AND MAINTENANCE OF TRAFFIC, REFER TO CONSTRUCTION STAGING DRAWINGS.
6. THE CONTRACTOR WILL REQUIRE CP PROTECTION FOR ALL WORKS OCCURRING ON OR ABOUT THE RAILWAY RIGHT OF WAY.
7. TRACK SETTLEMENT MONITORING WILL BE REQUIRED FOR THE INSTALLATION OF THE TRACK PROTECTION SYSTEM AS APPROVED BY CP.

 NEW CONCRETE  
 REMOVAL  
 NEW ASPHALT

[illegible]

## Appendix C – Borehole Location Plan and Stratigraphic Profile





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

HIGHWAY 401 EB CORE & COLLECTOR  
CPR OVERHEAD BRIDGE REHAB  
Latitude: 43.772964°, Longitude: -79.292316°

BOREHOLE LOCATION PLAN & SOIL STRATA

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


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

BH No.	ELEV.	NORTHING	EASTING
BH22-2-01	185.9	4848104	321489
BH22-2-02	185.1	4848125	321556
BH22-2-03	185.6	4848146	321515
BH22-2-04	184.6	4848165	321573
BH22-2-13	185.8	4848139	321496
BH22-2-14	184.2	4848171	321593
75-10	178.8	4848108	321539
75-11	177.7	4848102	321503

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 the OPS Gen. Cond.

SUBMISSION FOR MTO REVIEW

NO	DATE	BY	DESCRIPTION

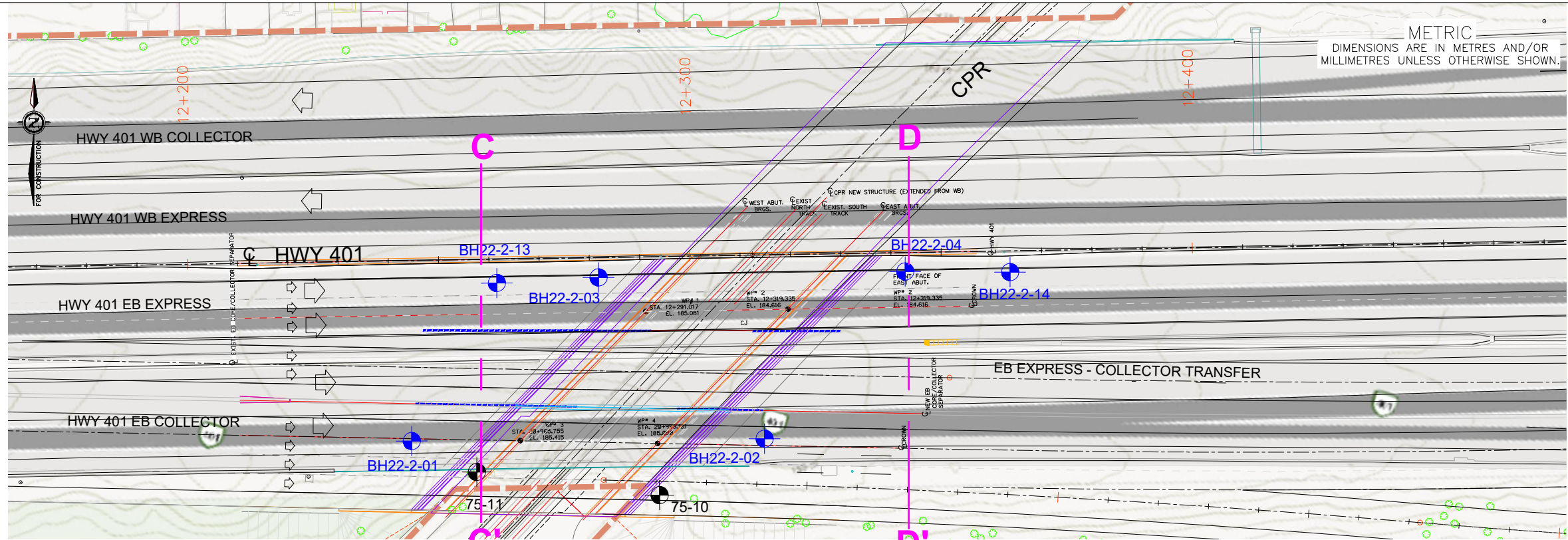
PROJECT No. ADM-22000797-A0  
SUBM'D SH CHKD. SM  
DRAWN SH CHKD. TC

GEOCREs No. 30M14-338  
DATE NOV. 13, 2024  
APPRD SG

SITE 37X-0213/B1 & B3  
DWG 01

FILE NAME: I:\2003-Brampton\Proposals\Proposals\International\Hwy 401 & Victoria Park Av. to Nelson\working drawings\Structure 2 - CPR Overpass.dwg  
MODIFIED: 2024-11-13 10:47





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

SOIL STRATA SYMBOLS


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

BH No.	ELEV.	NORTHING	EASTING
BH22-2-01	185.9	4848104	321489
BH22-2-02	185.1	4848125	321556
BH22-2-03	185.6	4848146	321515
BH22-2-04	184.6	4848165	321573
BH22-2-13	185.8	4848139	321496
BH22-2-14	184.2	4848171	321593
68-3	165.0	4848481	322536
68-4	165.6	4848444	322557
68-5	165.9	4848431	322537
2011-01	166.3	4848414	322538

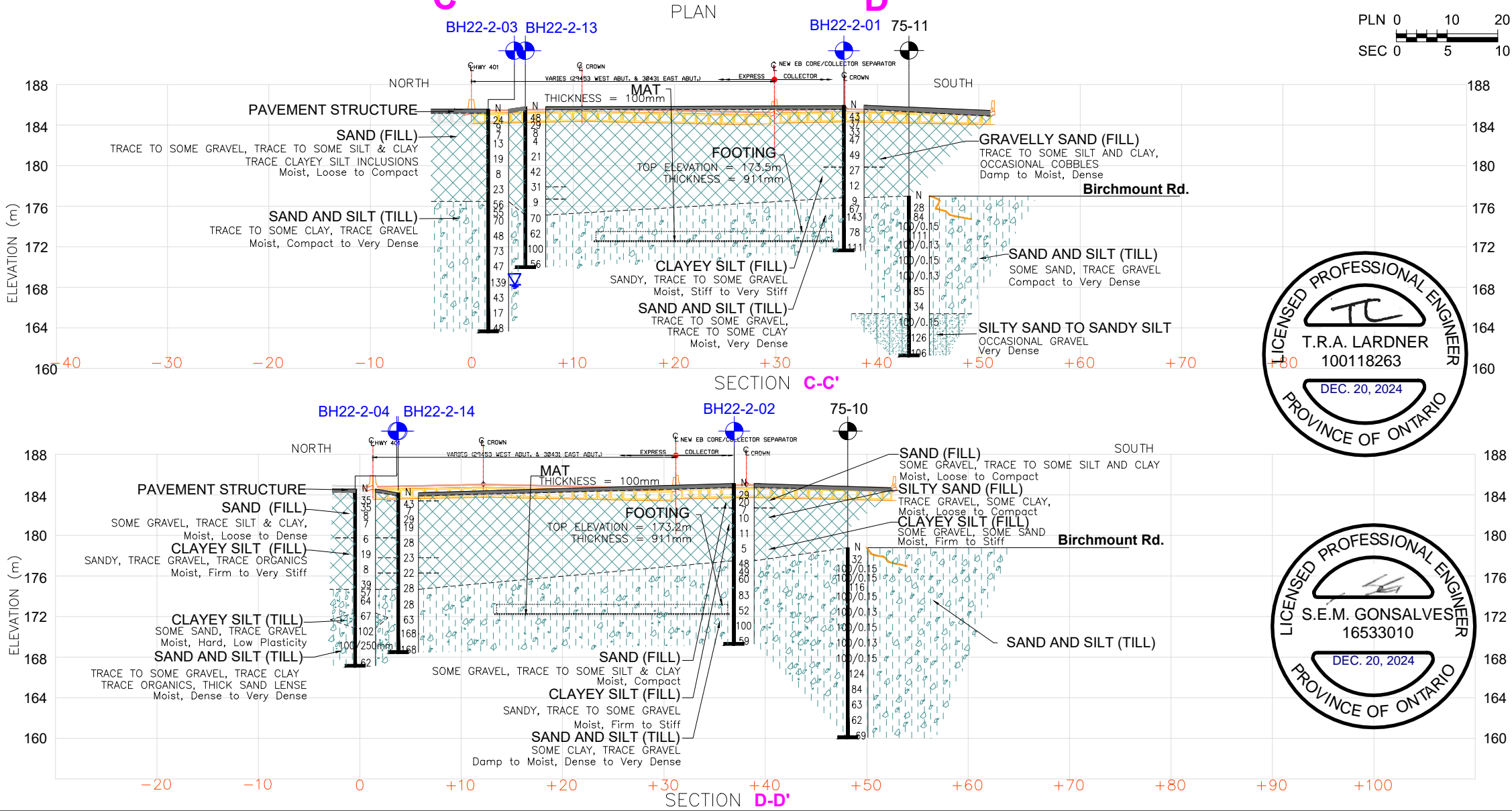
NOTES

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SUBMISSION FOR MTO REVIEW			
NO	DATE	BY	DESCRIPTION
PROJECT No. ADM-22000797-A0			
SUBM'D SH		CHKD. SM	GEOCREs No. -
DRAWN SH		CHKD. TC	DATE NOV. 11, 2024
		APPRD SG	SITE 37X-0213/B1 & B3
		DWG 02	



## 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.

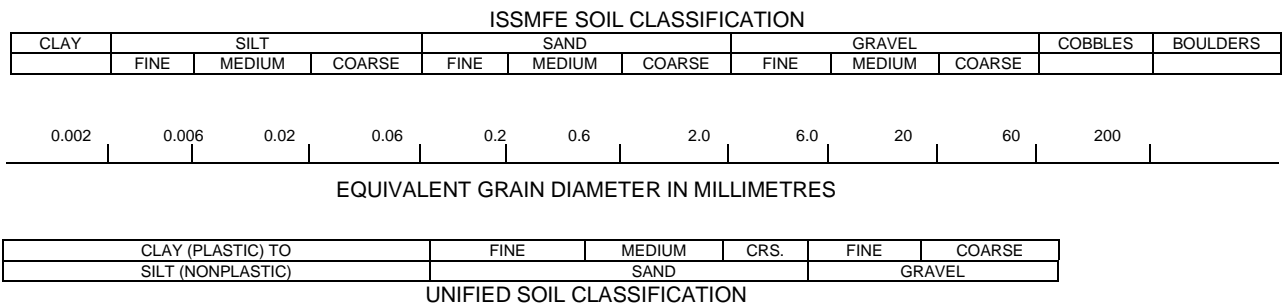
*Seam:* a thin, confined layer of soil having different particle size, texture, or color from materials above and below.

*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:

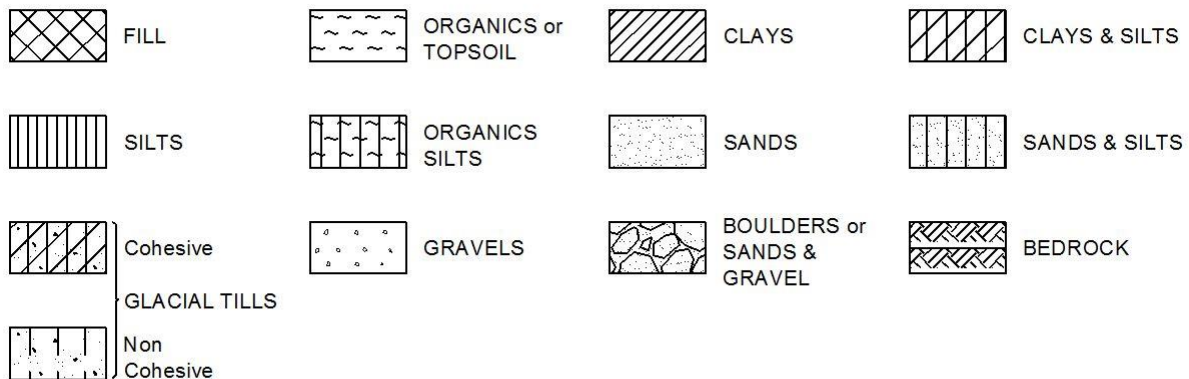
Table c: Consistency of Cohesive Soil

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

## RECORD OF BOREHOLE No BH22-2-01

1 OF 1

METRIC

W.P. Site 37X-0212/B1 & B3 LOCATION Hwy 401 - CPR O/P, Toronto, ON, MTM ON-10 321489E 4848104N ORIGINATED BY EL  
 DIST Toronto HWY 401 BOREHOLE TYPE Truck Mount CME 75 / SSA COMPILED BY EL/OD  
 DATUM Geodetic DATE 2022.10.12 - 2022.10.12 LATITUDE 43.77270818 LONGITUDE -79.29269277 CHECKED BY SM/TL

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT			UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			SHEAR STRENGTH kPa					W <sub>p</sub>	W	W <sub>L</sub>		
185.9 0.0 185.5 0.4	PAVEMENT STRUCTURE - 90 mm of asphalt and 340 mm of concrete GRAVELLY SAND (FILL) - trace to some silt and clay, brown to greyish brown, damp to moist, dense - occasional cobbles encountered		AS1	AS		185										28 60 (12)
			SS2	SS		184										
			SS3	SS		183										
			SS4	SS		182										
			SS5	SS		181										
			SS6	SS		180										
179.8 6.1	CLAYEY SILT (FILL) - sandy, trace to some gravel, brown with black inclusions, moist, stiff to very stiff		SS7	SS		179										
			SS8	SS		178										0 41 40 19
176.8 9.1	SAND AND SILT (TILL) - trace to some gravel, trace to some clay, brown to brownish grey, moist, very dense		SS9	SS		177										
			SS10	SS		176										1 52 40 7
			SS11	SS		175										
			SS12	SS		174										
			SS13	SS		173										
171.6 14.3	END OF BOREHOLE  NOTE: 1) No groundwater was encountered in open borehole upon completion of drilling.					172										Corrosivity Sample

ONTARIO MTO H401 - CP - 14122020.GPJ ONTARIO MTO GDT 8/4/23

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



Brampton, Ontario

## RECORD OF BOREHOLE No BH22-2-02

1 OF 1

METRIC

W.P. Site 37X-0212/B1 & B3 LOCATION Hwy 401 - CPR O/P, Toronto, ON, MTM ON-10 321556E 4848125N ORIGINATED BY EL  
 DIST Toronto HWY 401 BOREHOLE TYPE Truck Mount CME 75 / SSA COMPILED BY EL/OD  
 DATUM Geodetic DATE 2022.10.19 - 2022.10.19 LATITUDE 43.77289569 LONGITUDE -79.29185986 CHECKED BY SM/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 (%)	
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIALX P. PENETROMETER										
185.1 0.0 184.7 0.4	PAVEMENT STRUCTURE - 190 mm of asphalt and 215 mm of concrete  SAND (FILL) - some gravel, trace to some silt and clay, brown, moist, compact		AS1	AS														
			SS2	SS	29													17 69 (14)
			SS3	SS	20													
182.7 2.4	CLAYEY SILT (FILL) - sandy, trace to some gravel, brown to grey, moist, firm to stiff		SS4	SS	7													
			SS5	SS	10													6 47 36 11
			SS6	SS	11													
			SS7	SS	5													
177.5 7.6	SAND AND SILT (TILL) - some clay, trace gravel, light brown to greyish brown, damp to moist, dense to very dense - thin sand lenses interbedded within till layer		SS8	SS	48												2 46 40 12	
			SS9	SS	49													
			SS10	SS	60													1 45 43 11
			SS11	SS	83													
			SS12	SS	52													
			SS13	SS	100													
169.3 15.8	END OF BOREHOLE		SS14	SS	59													
	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 - CP - 14122020.GPJ ONTARIO MTO.GDT 8/4/23

Brampton, Ontario

## RECORD OF BOREHOLE No BH22-2-03

1 OF 2

METRIC

W.P. Site 37X-0212/B1 & B3 LOCATION Hwy 401 - CPR O/P, Toronto, ON, MTM ON-10 321515E 4848146N ORIGINATED BY EL  
 DIST Toronto HWY 401 BOREHOLE TYPE Truck Mount CME 75 / SSA / Mud Rotary COMPILED BY EL/OD  
 DATUM Geodetic DATE 2022.10.11 - 2022.10.18 LATITUDE 43.77308936 LONGITUDE -79.29236375 CHECKED BY SM/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 (%)		
								○ UNCONFINED + FIELD VANE	○ UNCONFINED	+ FIELD VANE						● QUICK TRIAXIAL	× P. PENETROMETER	
185.6							20	40	60	80	100	20	40	60	kN/m <sup>3</sup>	GR SA SI CL		
0.0	PAVEMENT STRUCTURE - 90 mm of asphalt and 280 mm of concrete																	
185.2			AS1	AS								○						
0.4		SAND (FILL) - trace to some gravel, trace to some silt and clay, brown, moist, loose to compact		SS2	SS	24							○					
				SS3	SS	9							○				9 85 (6)	
				SS4	SS	7							○					
				SS5	SS	13							○					
				SS6	SS	19							○				9 70 (21)	
				SS7	SS	8							○					
				SS8	SS	23							○					
176.5	SAND AND SILT (TILL) - trace to some clay, trace gravel, greyish brown with trace reddish brown inclusions, moist, very dense		SS9	SS	56							○				2 48 40 10		
9.1			SS10	SS	55							○						
		SS11	SS	70								○						
		SS12	SS	48								○						
		SS13	SS	73								○				1 49 42 8		
		SS14	SS	47								○				20.8		
	- wet below 18.3 m depth		SS15	SS	139							○				23.7		
			SS16	SS	43							○				5 49 37 9		

Continued Next Page

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

ONTARIO MTO H401 - CP - 14122020.GPJ ONTARIO MTO.GDT 8/4/23

**METRIC**

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

Brampton, Ontario

## RECORD OF BOREHOLE No BH22-2-04

1 OF 1

METRIC

W.P. Site 37X-0212/B1 & B3 LOCATION Hwy 401 - CPR O/P, Toronto, ON, MTM ON-10 321573E 4848165N ORIGINATED BY EL  
 DIST Toronto HWY 401 BOREHOLE TYPE Truck Mount CME 75 / SSA COMPILED BY EL  
 DATUM Geodetic DATE 2022.10.05 - 2022.10.05 LATITUDE 43.77325556 LONGITUDE -79.29164722 CHECKED BY SM/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
184.6																		
184.9																		
0.3	PAVEMENT STRUCTURE - 50 mm of asphalt and 200 mm of concrete		AS1	AS			184											
	SAND (FILL) - some gravel, trace silt and clay, greyish brown, slightly moist, loose to dense		SS2	SS	35													
			SS3	SS	35		183											
			SS4	SS	8		182											
			SS5	SS	7		181									13 82 (5)		
							180											
179.8	CLAYEY SILT (FILL) - sandy, trace gravel, greyish brown to grey, moist, firm to very stiff		SS6	SS	6													
							179											
			SS7	SS	19		178											
							177											
	- trace organics encountered		SS8	SS	8											1 45 38 16		
							176											
175.5	SAND AND SILT (TILL) - some gravel, trace clay, trace organics, light brown to greyish brown, slightly moist, dense to very dense		SS9	SS	39		175									24.1		
	- no organics below 9.9 m depth		SS10	SS	57											Corrosivity Sample		
	- 0.1 m thick sand lens		SS11	SS	64		174											
							173											
172.4	CLAYEY SILT (TILL) - some sand, trace gravel, grey, moist, hard, low plasticity		SS12	SS	67		172									3 17 40 40		
							171											
170.9	SAND AND SILT (TILL) - trace gravel, trace clay, greyish brown, slightly moist to moist, very dense		SS13	SS	102											1 45 45 9		
			SS14	SS	100/ 250mm		170									Non-Plastic		
							169											
							168											
			SS15	SS	62											23.8		
167.2	END OF BOREHOLE																	
17.4	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 - CP - 14122020.GPJ ONTARIO MTO GDT 8/4/23




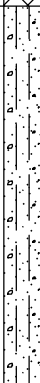

Brampton, Ontario

## RECORD OF BOREHOLE No BH22-2-13

1 OF 1

METRIC

W.P. Site 37X-0212/B1 & B3 LOCATION Hwy 401 - CPR O/P, Toronto, ON, MTM ON-10 321496E 4848139N ORIGINATED BY EL  
 DIST Toronto HWY 401 BOREHOLE TYPE Truck Mount CME 75 / SSA COMPILED BY EL  
 DATUM Geodetic DATE 2022.10.03 - 2022.10.03 LATITUDE 43.77302617 LONGITUDE -79.29259997 CHECKED BY SM/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 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL X P. PENETROMETER									WATER CONTENT (%)			
								20	40	60	80						100	20	40	60
185.8	<b>PAVEMENT STRUCTURE</b> - 100 mm of asphalt and 200 mm of concrete <b>SILTY SAND (FILL)</b> - trace to some gravel, brownish grey to grey, slightly moist to moist, loose to dense		AS1	AS											21.0	2 62 (36)				
189.9			SS2	SS	48															
0.3			SS3	SS	29															
			SS4	SS	8															
			SS5	SS	4															
			SS6	SS	21															
			SS7	SS	42															
177.9	<b>CLAYEY SILT (FILL)</b> - some gravel, some sand, grey, slightly moist, hard		SS8	SS	31															
7.9																				
176.7	<b>SILTY SAND (FILL)</b> - trace clay, trace organics/rootlets, grey with black inclusions, slightly moist, loose		SS9	SS	9															
9.1																				
175.1	<b>SAND AND SILT (TILL)</b> - trace to some clay, trace gravel, brownish grey to grey, slightly moist to moist, very dense		SS10	SS	70									4 44 40 12						
10.7																				
			SS11	SS	62															
			SS12	SS	100															
			SS13	SS	56															
170.0	<b>END OF BOREHOLE</b>  NOTE: 1) No groundwater was encountered in open borehole upon completion of drilling.													22.6						
15.8																				

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

ONTARIO MTO H401 - CP - 14122020.GPJ ONTARIO MTO.GDT 8/4/23

Brampton, Ontario

## RECORD OF BOREHOLE No BH22-2-14

1 OF 1

METRIC

W.P. Site 37X-0212/B1 & B3 LOCATION Hwy 401 - CPR O/P, Toronto, ON, MTM ON-10 321593E 4848171N ORIGINATED BY EL  
 DIST Toronto HWY 401 BOREHOLE TYPE Truck Mount CME 75 / SSA COMPILED BY EL  
 DATUM Geodetic DATE 2022.10.04 - 2022.10.04 LATITUDE 43.77330897 LONGITUDE -79.29140389 CHECKED BY SM/TL

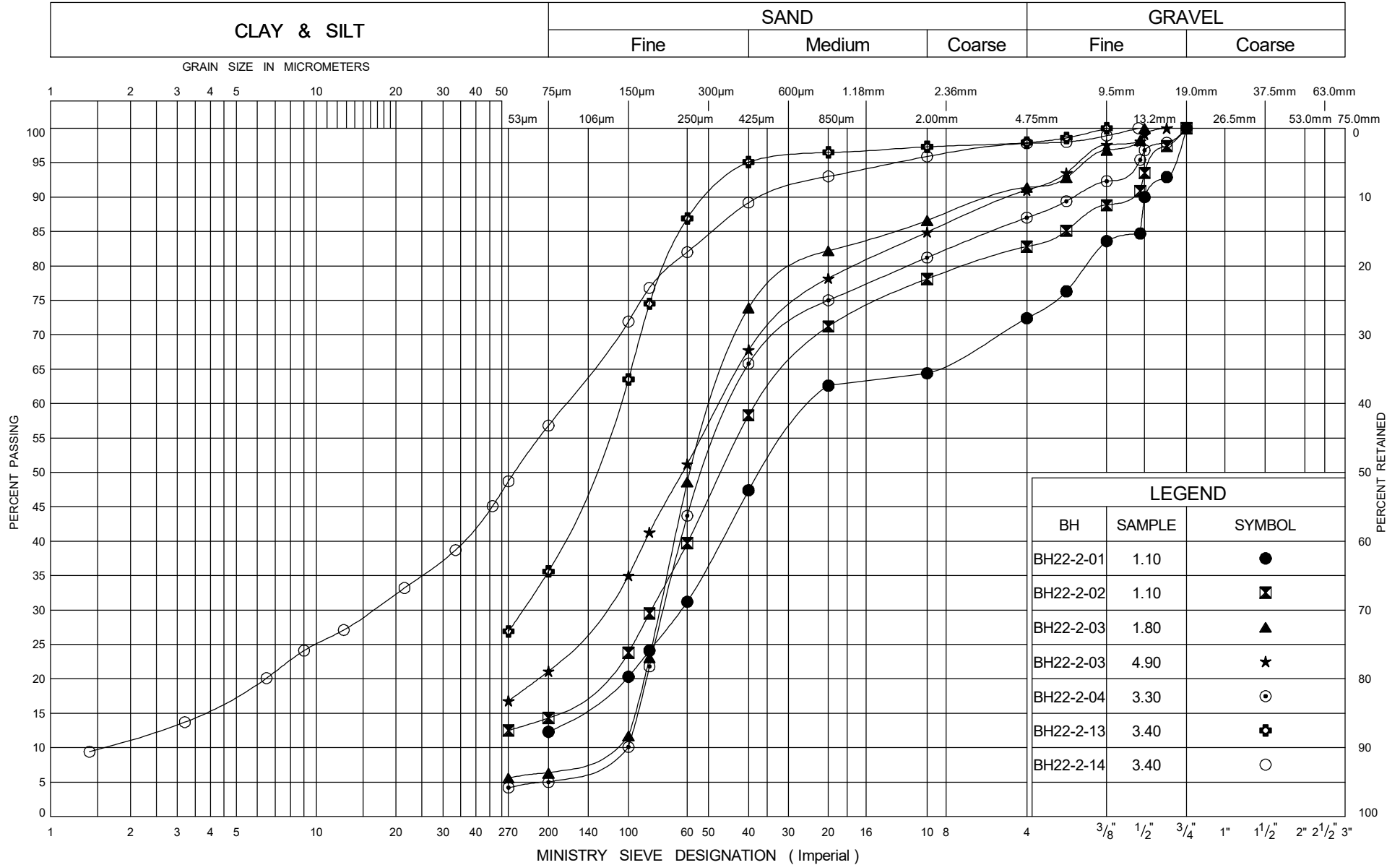
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT  γ  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				WATER CONTENT (%)				
								20	40	60	80	100	W <sub>P</sub>	W		
184.2																
189.9																
0.3																
183.4																
0.8																

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

ONTARIO MTO H401 - CP - 14122020.GPJ ONTARIO MTO.GDT 8/4/23

## Appendix E – Laboratory Data

# UNIFIED SOIL CLASSIFICATION SYSTEM



## GRAIN SIZE DISTRIBUTION

Cohesionless Fill

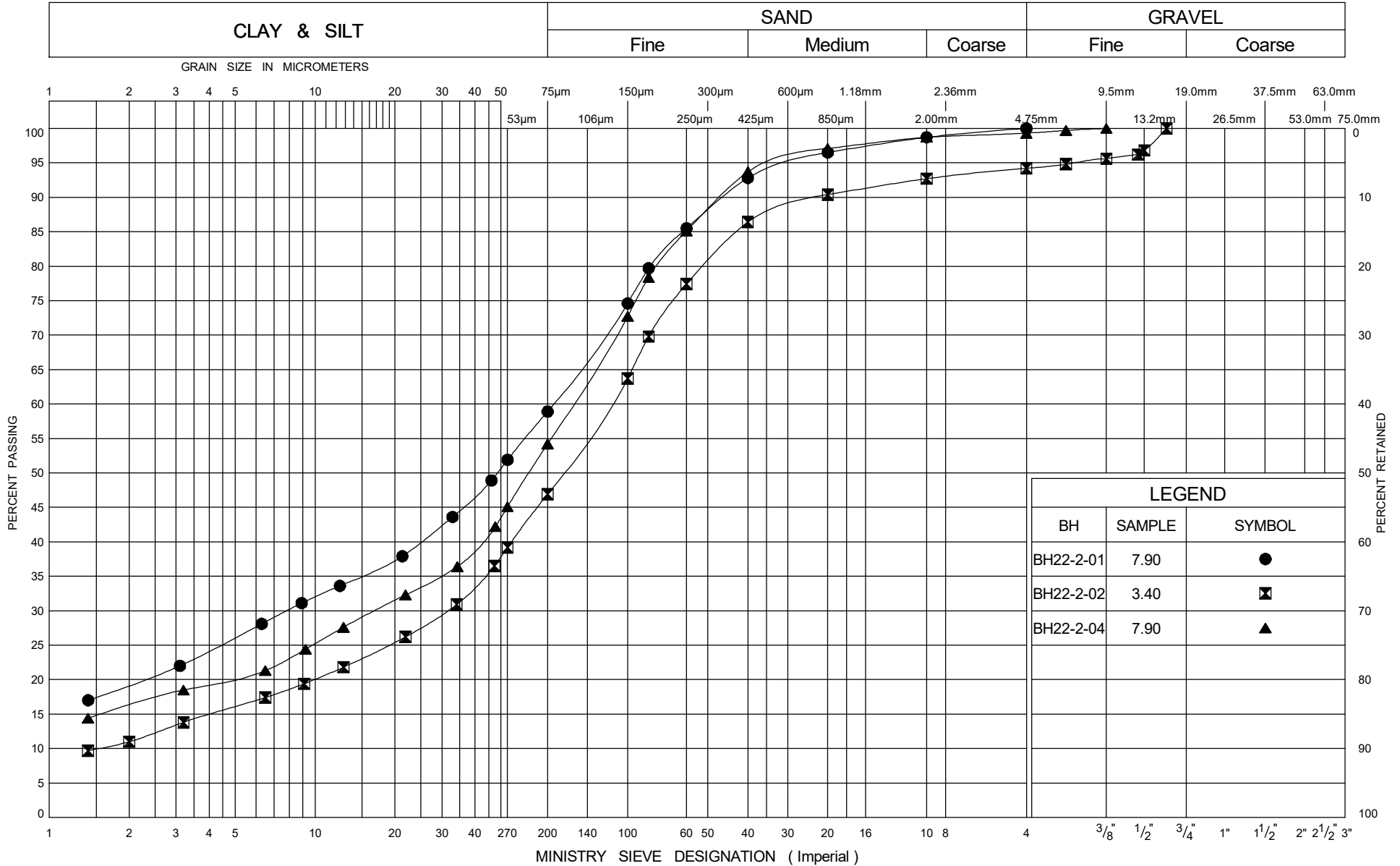
FIG No 1

CP Rail Overpass Replacement  
(Site 37X-0213/B1 & B3)

Hwy 401 Eastbound Express and  
Collector Lanes

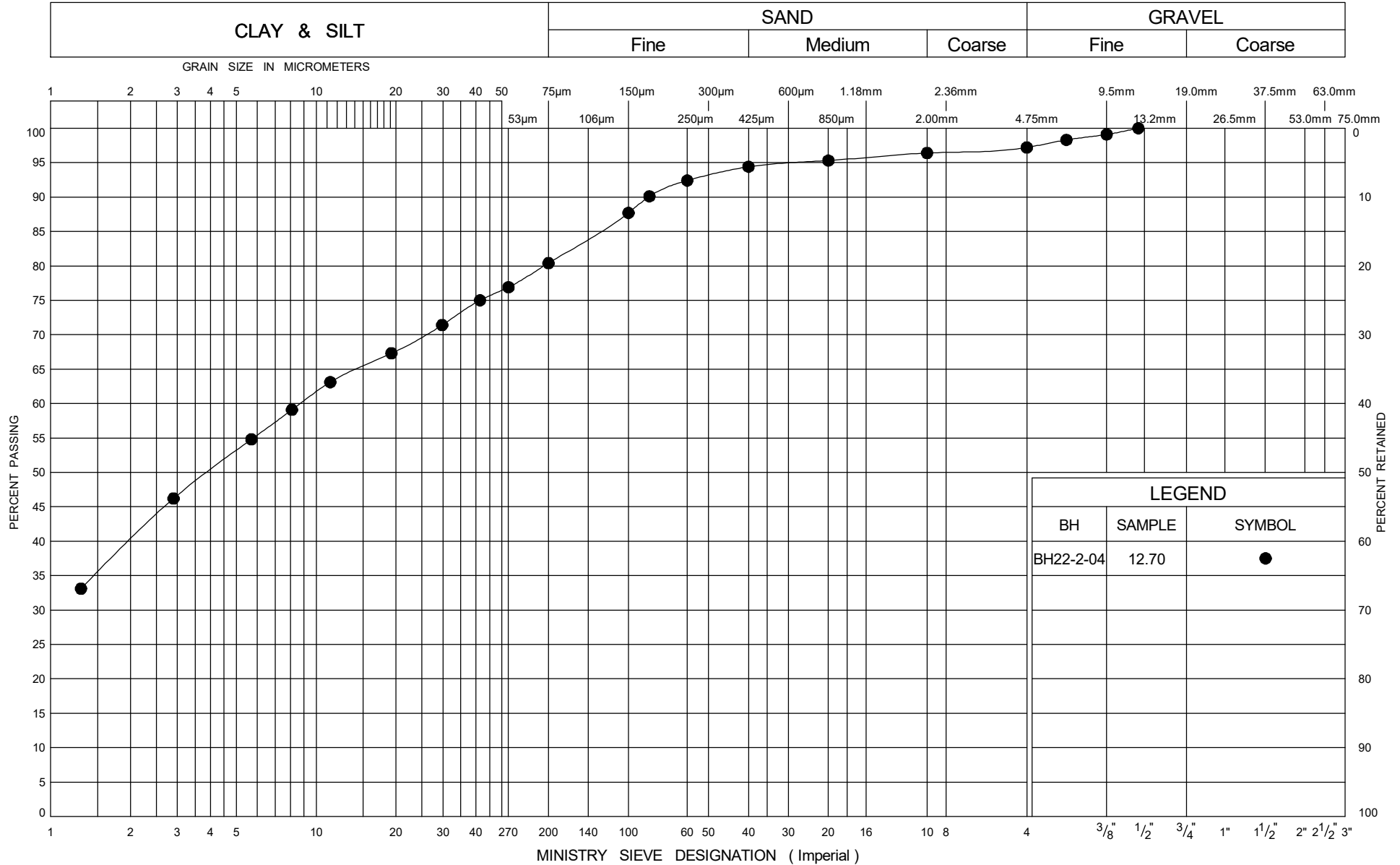


# UNIFIED SOIL CLASSIFICATION SYSTEM



LEGEND		
BH	SAMPLE	SYMBOL
BH22-2-01	7.90	●
BH22-2-02	3.40	⊠
BH22-2-04	7.90	▲

# UNIFIED SOIL CLASSIFICATION SYSTEM



Ministry of  
Transportation

## GRAIN SIZE DISTRIBUTION

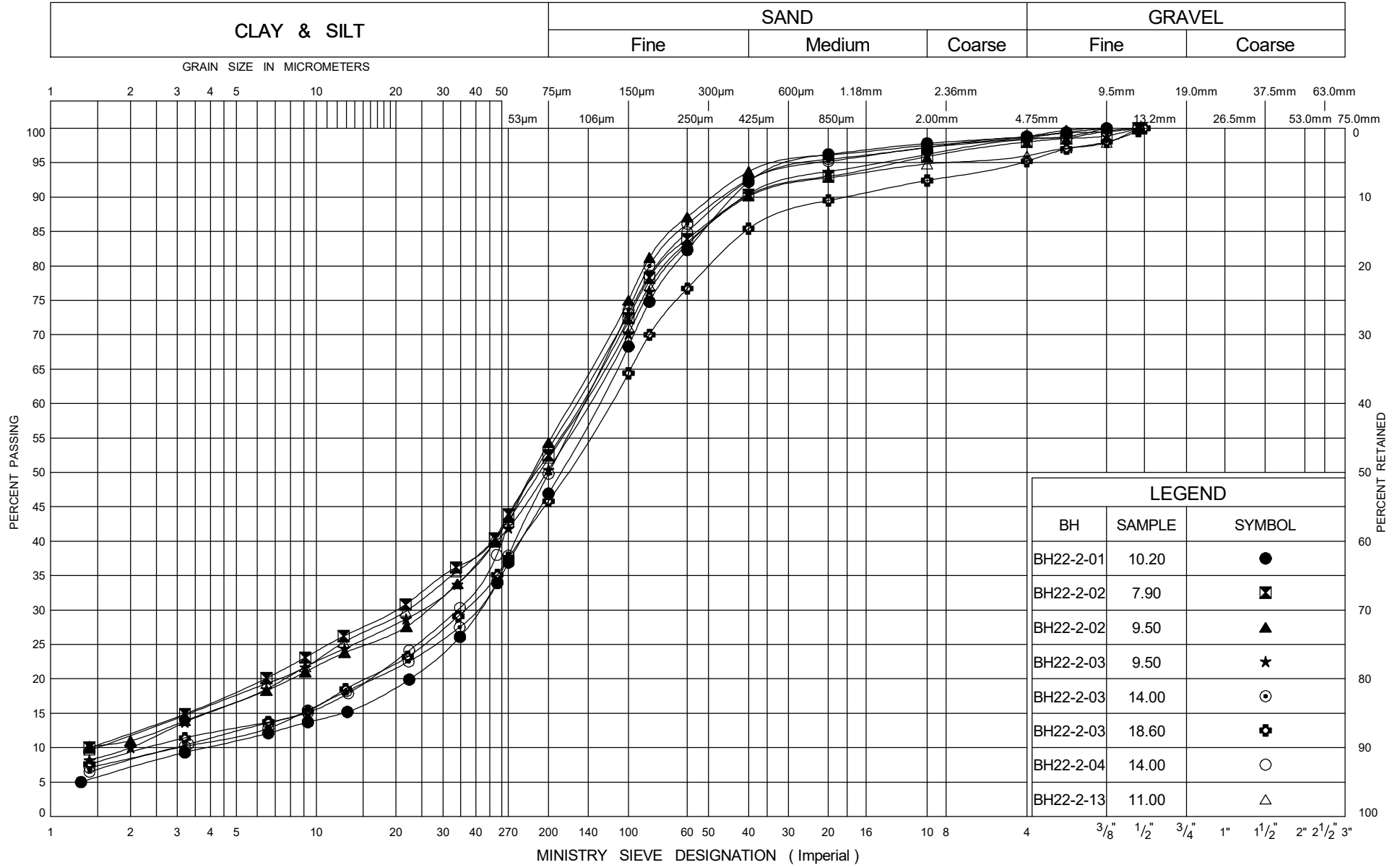
Native Cohesive Till

FIG No 3

CP Rail Overpass Replacement  
(Site 37X-0213/B1 & B3)

Hwy 401 Eastbound Express and  
Collector Lanes

# UNIFIED SOIL CLASSIFICATION SYSTEM



Ministry of  
Transportation

## GRAIN SIZE DISTRIBUTION

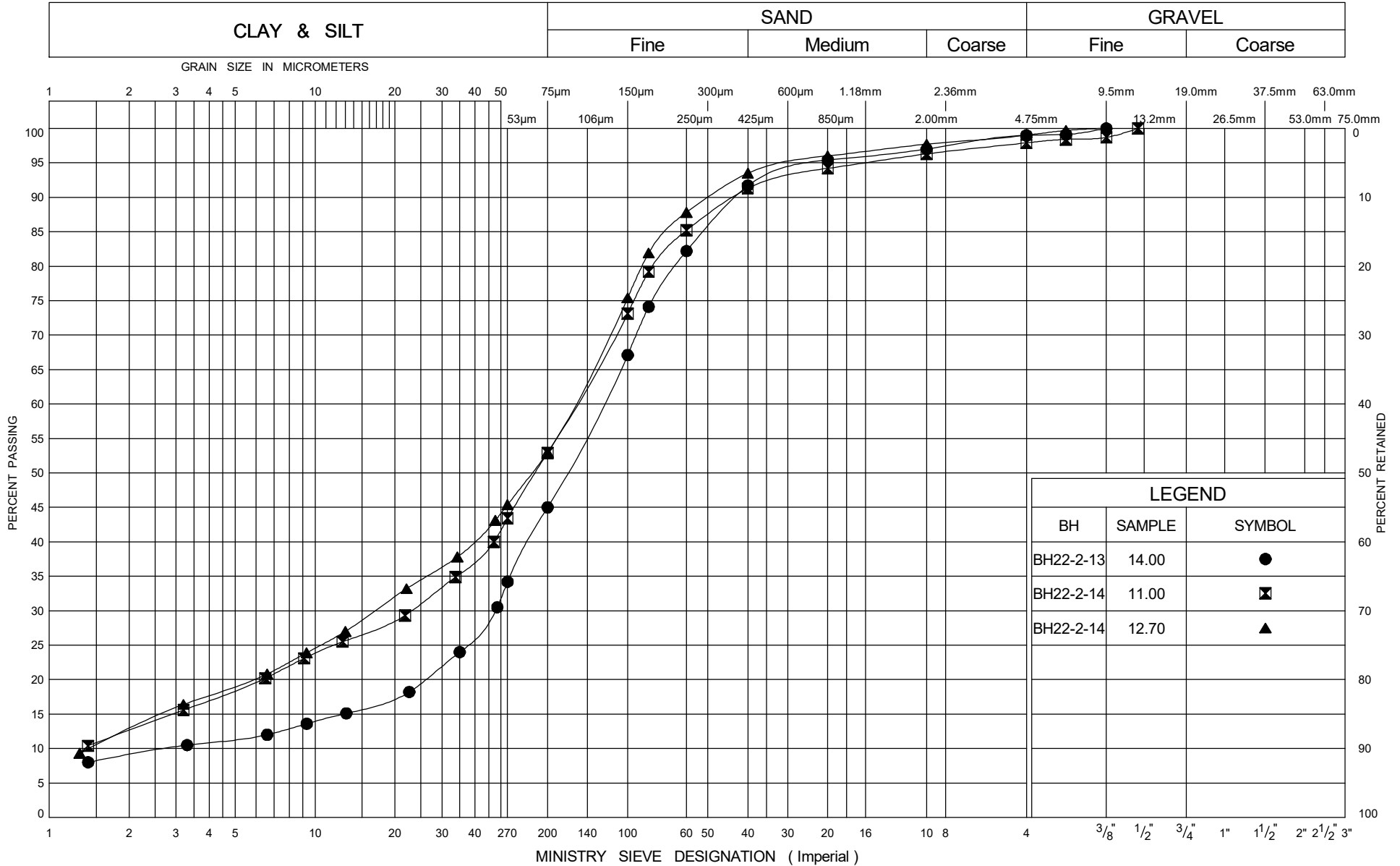
Native Cohesionless Till

FIG No 4

CP Rail Overpass Replacement  
(Site 37X-0213/B1 & B3)

Hwy 401 Eastbound Express and  
Collector Lanes

# UNIFIED SOIL CLASSIFICATION SYSTEM



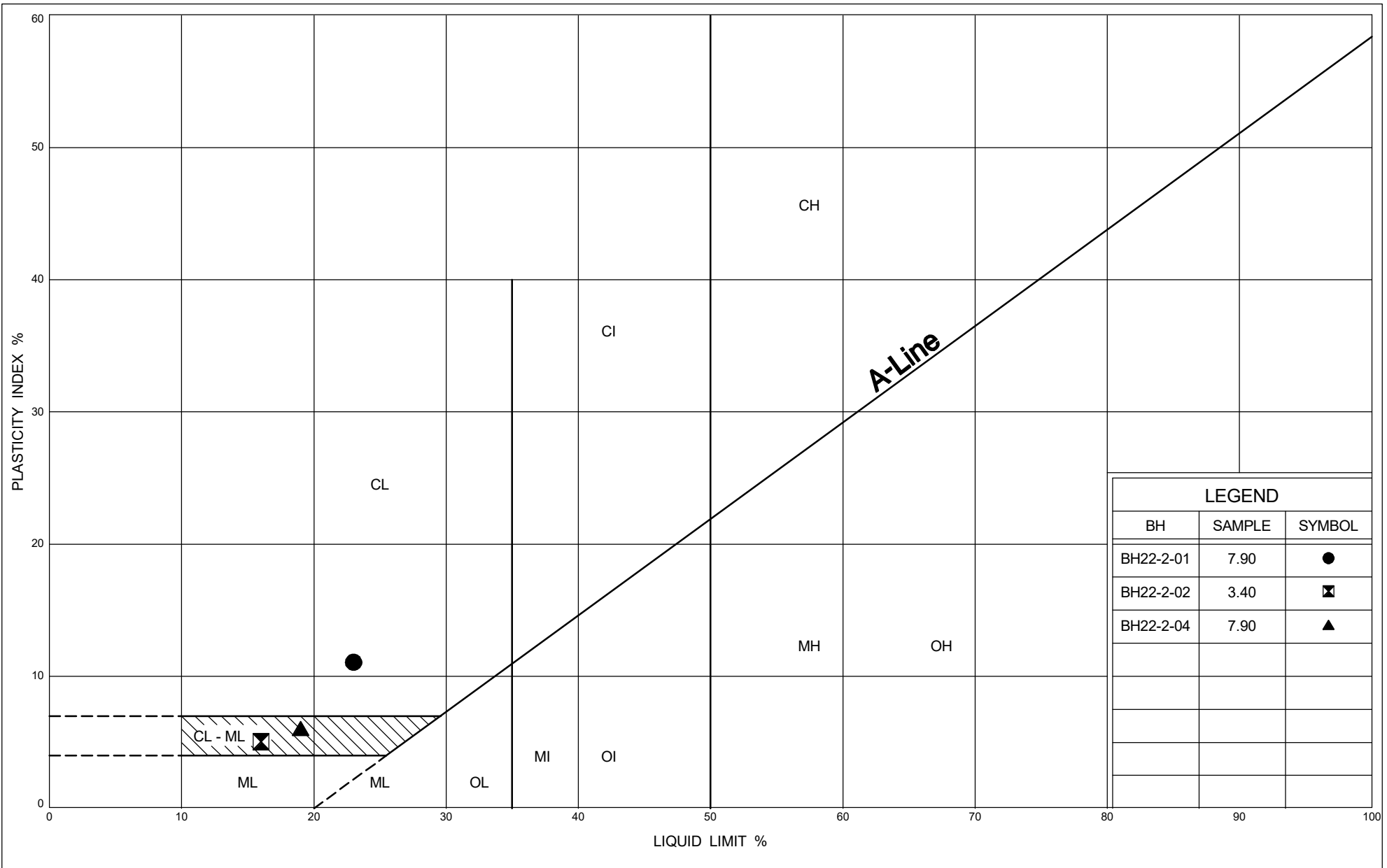
## GRAIN SIZE DISTRIBUTION

Native Cohesionless Till

FIG No 5

CP Rail Overpass Replacement  
(Site 37X-0213/B1 & B3)

Hwy 401 Eastbound Express and  
Collector Lanes



Ministry of  
Transportation

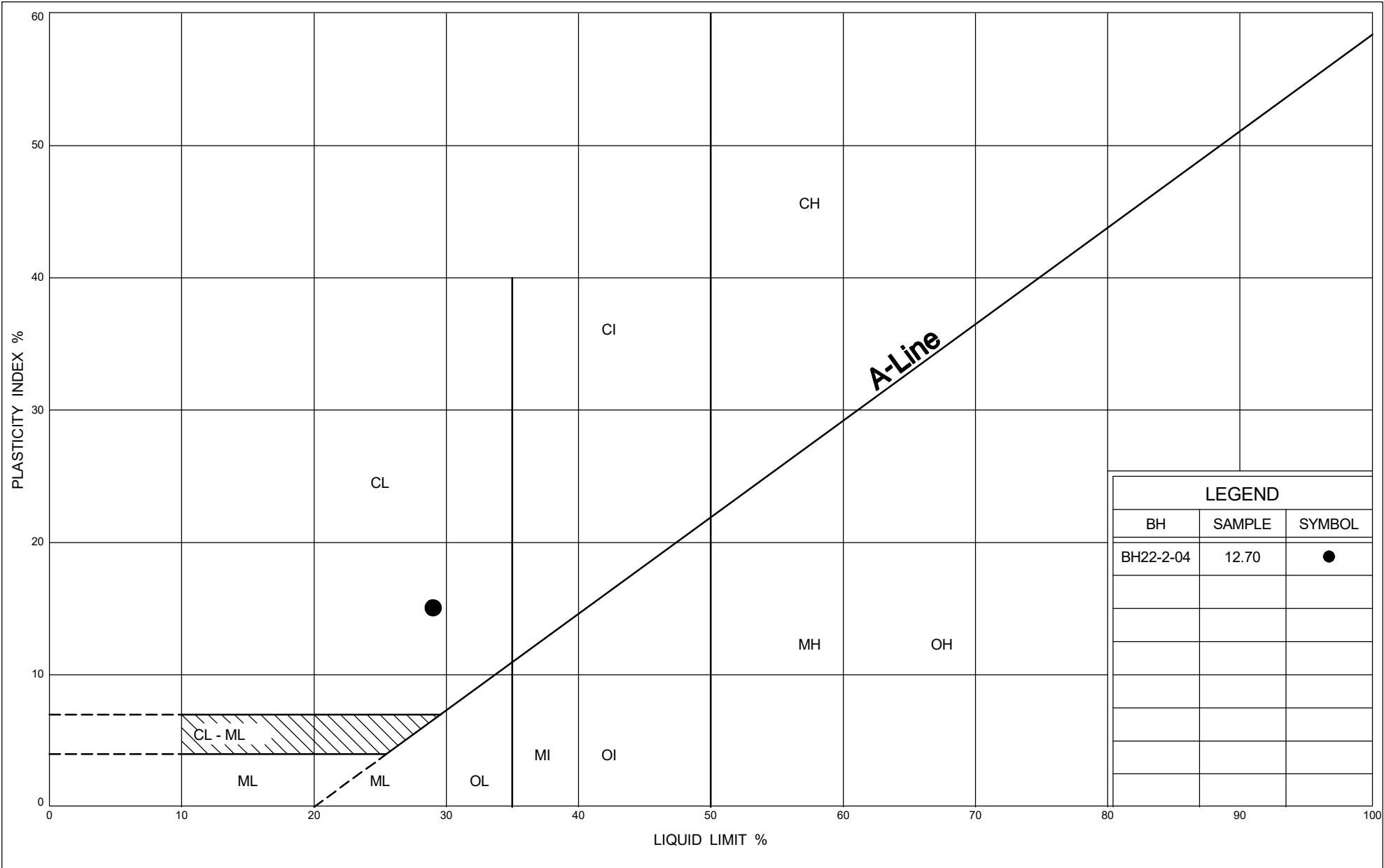
## PLASTICITY CHART

Clayey Silt (FILL)

FIG No 6

CP Rail Overpass Replacement  
(Site 37X-0213/B1 & B3)

Hwy 401 Eastbound Express and  
Collector Lanes





Your Project #: ADM-22000797-A0  
Site Location: Hwy 401 from Victoria to Nelson Ave, ON  
Your C.O.C. #: 893860-03-01

**Attention: Nimesh Tamrakar**

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

**Report Date: 2022/10/19**  
Report #: R7348456  
Version: 1 - Final

**CERTIFICATE OF ANALYSIS**

**BUREAU VERITAS JOB #: C2T2765**

**Received: 2022/10/07, 10:56**

Sample Matrix: Soil  
# Samples Received: 2

Analyses	Quantity	Date	Date	Laboratory Method	Analytical Method
		Extracted	Analyzed		
Chloride (20:1 extract)	2	2022/10/14	2022/10/19	CAM SOP-00463	SM 23 4500-Cl E m
Conductivity	2	2022/10/12	2022/10/13	CAM SOP-00414	OMOE E3530 v1 m
Moisture (Subcontracted) (1, 2)	2	N/A	2022/10/15	AB SOP-00002	CCME PHC-CWS m
Sulphide in Soil (1)	2	N/A	2022/10/13	AB SOP-00080	EPA9030B/SM4500S2-DF
pH CaCl2 EXTRACT	2	2022/10/15	2022/10/15	CAM SOP-00413	EPA 9045 D m
Redox Potential (3)	2	2022/10/17	2022/10/19	CAM SOP-00421	SM 2580 B
Resistivity of Soil	2	2022/10/07	2022/10/13	CAM SOP-00414	SM 23 2510 m
Sulphate (20:1 Extract)	2	2022/10/14	2022/10/17	CAM SOP-00464	EPA 375.4 m

**Remarks:**

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

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

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

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

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

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

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

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

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

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



Your Project #: ADM-22000797-A0  
Site Location: Hwy 401 from Victoria to Nelson Ave, ON  
Your C.O.C. #: 893860-03-01

**Attention: Nimesh Tamrakar**

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

**Report Date: 2022/10/19**  
Report #: R7348456  
Version: 1 - Final

**CERTIFICATE OF ANALYSIS**

**BUREAU VERITAS JOB #: C2T2765**

**Received: 2022/10/07, 10:56**

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

**Encryption Key**

Please direct all questions regarding this Certificate of Analysis to your Project Manager.

Patricia Legette, Project Manager

Email: Patricia.Legette@bureauveritas.com

Phone# (905)817-5799

=====

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

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For Service Group specific validation please refer to the Validation Signature Page.





### SOIL CORROSIVITY PACKAGE (SOIL)

Bureau Veritas ID		TYO346	TYO347		
Sampling Date		2022/10/06 02:30	2022/09/26 02:00		
COC Number		893860-03-01	893860-03-01		
	<b>UNITS</b>	<b>BH22-2-4 SS9</b>	<b>BH22-3-3 SS10</b>	<b>RDL</b>	<b>QC Batch</b>
<b>Calculated Parameters</b>					
Resistivity	ohm-cm	1500	3600		8271092
<b>CONVENTIONALS</b>					
Redox Potential	mV	270	270	N/A	8286961
<b>Inorganics</b>					
Soluble (20:1) Chloride (Cl-)	ug/g	380	130	20	8283307
Conductivity	umho/cm	659	279	2	8278171
Available (CaCl2) pH	pH	7.91	7.92		8285499
Soluble (20:1) Sulphate (SO4)	ug/g	<20	<20	20	8283308
Sulphide	mg/kg	2.7	3.7	0.5	8290499
<b>Physical Testing</b>					
Moisture-Subcontracted	%	8.1	9.9	0.30	8290500
RDL = Reportable Detection Limit QC Batch = Quality Control Batch N/A = Not Applicable					



BUREAU  
VERITAS

Bureau Veritas Job #: C2T2765

Report Date: 2022/10/19

exp Services Inc

Client Project #: ADM-22000797-A0

Site Location: Hwy 401 from Victoria to Nelson Ave, ON

Sampler Initials: IB

## TEST SUMMARY

**Bureau Veritas ID:** TYO346  
**Sample ID:** BH22-2-4 SS9  
**Matrix:** Soil

**Collected:** 2022/10/06  
**Shipped:**  
**Received:** 2022/10/07

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	8283307	2022/10/14	2022/10/19	Alina Dobreanu
Conductivity	AT	8278171	2022/10/12	2022/10/13	Surinder Rai
Moisture (Subcontracted)	BAL	8290500	N/A	2022/10/15	Winston Lee
Sulphide in Soil	SPEC	8290499	N/A	2022/10/13	Bailey Morrison
pH CaCl2 EXTRACT	AT	8285499	2022/10/15	2022/10/15	Kien Tran
Redox Potential	COND	8286961	2022/10/17	2022/10/19	Surinder Rai
Resistivity of Soil		8271092	2022/10/13	2022/10/13	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	8283308	2022/10/14	2022/10/17	Alina Dobreanu

**Bureau Veritas ID:** TYO347  
**Sample ID:** BH22-3-3 SS10  
**Matrix:** Soil

**Collected:** 2022/09/26  
**Shipped:**  
**Received:** 2022/10/07

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	8283307	2022/10/14	2022/10/19	Alina Dobreanu
Conductivity	AT	8278171	2022/10/12	2022/10/13	Surinder Rai
Moisture (Subcontracted)	BAL	8290500	N/A	2022/10/15	Winston Lee
Sulphide in Soil	SPEC	8290499	N/A	2022/10/13	Bailey Morrison
pH CaCl2 EXTRACT	AT	8285499	2022/10/15	2022/10/15	Kien Tran
Redox Potential	COND	8286961	2022/10/17	2022/10/19	Surinder Rai
Resistivity of Soil		8271092	2022/10/13	2022/10/13	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	8283308	2022/10/14	2022/10/17	Alina Dobreanu



### GENERAL COMMENTS

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

Package 1	0.7°C
-----------	-------

Results relate only to the items tested.

BUREAU  
VERITAS

Bureau Veritas Job #: C2T2765

Report Date: 2022/10/19

## QUALITY ASSURANCE REPORT

exp Services Inc

Client Project #: ADM-22000797-A0

Site Location: Hwy 401 from Victoria to Nelson Ave, ON

Sampler Initials: IB

QC Batch	Parameter	Date	Matrix Spike		SPIKED BLANK		Method Blank		RPD	
			% Recovery	QC Limits	% Recovery	QC Limits	Value	UNITS	Value (%)	QC Limits
8278171	Conductivity	2022/10/13			99	90 - 110	<2	umho/cm	0.39	10
8283307	Soluble (20:1) Chloride (Cl-)	2022/10/19	NC	70 - 130	105	70 - 130	<20	ug/g	28	35
8283308	Soluble (20:1) Sulphate (SO4)	2022/10/17	NC	70 - 130	105	70 - 130	<20	ug/g	3.4	35
8285499	Available (CaCl2) pH	2022/10/15			100	97 - 103			2.7	N/A
8286961	Redox Potential	2022/10/19			100	95 - 105			7.3	N/A
8290499	Sulphide	2022/10/13	48 (1)	75 - 125	107	75 - 125	<0.5	mg/kg		
8290500	Moisture-Subcontracted	2022/10/15					<0.30	%		

N/A = Not Applicable

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

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

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

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

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

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



BUREAU  
VERITAS

Bureau Veritas Job #: C2T2765

Report Date: 2022/10/19

exp Services Inc

Client Project #: ADM-22000797-A0

Site Location: Hwy 401 from Victoria to Nelson Ave, ON

Sampler Initials: IB

### VALIDATION SIGNATURE PAGE

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

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



Ewa Pranjić, M.Sc., C.Chem, Scientific Specialist

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

---

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

# CHAIN OF CUSTODY RECORD

Page of

INVOICE TO:		REPORT TO:		PROJECT INFORMATION:		Laboratory Use Only:	
Company Name:	#17488 exp Services Inc	Company Name:		Quotation #:	C20328	Bureau Veritas Job #:	Bottle Order #:
Attention:	Accounts Payable	Attention:	Nimesh Tamrakar	P.O. #:			
Address:	1595 Clark Blvd Brampton ON L6T 4V1	Address:		Project:	ADM-22000797-A0		
Tel:	(905) 793-9800	Tel:	(905) 796-3200 Ext: 3026	Project Name:	Hwy 401 from Victoria to Nelso	COC #:	Project Manager:
Email:	AP@exp.com; Karen.Burke@exp.com	Email:	Nimesh.Tamrakar@exp.com	Site #:			Patricia Legette
				Sampled By:			

MOE REGULATED DRINKING WATER OR WATER INTENDED FOR HUMAN CONSUMPTION MUST BE SUBMITTED ON THE BUREAU VERITAS DRINKING WATER CHAIN OF CUSTODY						ANALYSIS REQUESTED (PLEASE BE SPECIFIC)		Turnaround Time (TAT) Required:	
Regulation 153 (2011)			Other Regulations			Special Instructions		Please provide advance notice for rush projects	
<input type="checkbox"/> Table 1	<input type="checkbox"/> Res/Park	<input type="checkbox"/> Medium/Fine	<input type="checkbox"/> CCME	<input type="checkbox"/> Sanitary Sewer Bylaw				<b>Regular (Standard) TAT:</b>	
<input type="checkbox"/> Table 2	<input type="checkbox"/> Ind/Comm	<input type="checkbox"/> Coarse	<input type="checkbox"/> Reg 558	<input type="checkbox"/> Storm Sewer Bylaw				(will be applied if Rush TAT is not specified):	
<input type="checkbox"/> Table 3	<input type="checkbox"/> Agri/Other	<input type="checkbox"/> For RSC	<input type="checkbox"/> MISA	<input type="checkbox"/> Municipality				Standard TAT = 5-7 Working days for most tests.	
<input type="checkbox"/> Table			<input type="checkbox"/> PWOO	<input type="checkbox"/> Reg 406 Table				Please note: Standard TAT for certain tests such as BOD and Dioxins/Furans are > 5 days - contact your Project Manager for details.	
<input type="checkbox"/> Other			<input type="checkbox"/> Other					<b>Job Specific Rush TAT (if applies to entire submission)</b>	
Include Criteria on Certificate of Analysis (Y/N)?								Date Required:	Time Required:
								Rush Confirmation Number:	(call lab for #)
Sample Barcode Label	Sample (Location) Identification	Date Sampled	Time Sampled	Matrix	Field Filtered (please circle): Metals / Hg / Cr VI	Soil Corrosivity Package		# of Bottles	Comments
1	BH22-2-4 SS9	2022/10/06	02:30am	SOIL		/			
2	BH22-2-4 SS9	2022/10/06	02:30am	SOIL		/			
3	BH22-3-3 SS10	2022/09/26	02:00am	SOIL		/			
4	BH22-3-3 SS10	2022/09/26	02:00	SOIL		/			
5				SOIL					
6				SOIL					
7				SOIL					
8				SOIL					
9				SOIL					
10				SOIL					

07-Oct-22 10:56

Patricia Legette

C2T2765

RPK ENV-925

* RELINQUISHED BY: (Signature/Print)		Date: (YY/MM/DD)	Time	RECEIVED BY: (Signature/Print)		Date: (YY/MM/DD)	Time	# jars used and not submitted	Laboratory Use Only				
Ivan Barva		22/10/07	10:55am			20/10/07	10:56		Time Sensitive	Temperature (°C) on Recl	Custody Seal	Yes	No
										17/10	Present		
											Intact		

\* UNLESS OTHERWISE AGREED TO IN WRITING, WORK SUBMITTED ON THIS CHAIN OF CUSTODY IS SUBJECT TO BUREAU VERITAS'S STANDARD TERMS AND CONDITIONS. SIGNING OF THIS CHAIN OF CUSTODY DOCUMENT IS ACKNOWLEDGMENT AND ACCEPTANCE OF OUR TERMS WHICH ARE AVAILABLE FOR VIEWING AT WWW.BVNA.COM/TERMS-AND-CONDITIONS.

\*\* IT IS THE RESPONSIBILITY OF THE RELINQUISHER TO ENSURE THE ACCURACY OF THE CHAIN OF CUSTODY RECORD. AN INCOMPLETE CHAIN OF CUSTODY MAY RESULT IN ANALYTICAL TAT DELAYS.

\*\* SAMPLE CONTAINER, PRESERVATION, HOLD TIME AND PACKAGE INFORMATION CAN BE VIEWED AT WWW.BVNA.COM/RESOURCES/CHAIN-OF-CUSTODY-FORMS.

White: Bureau Veritas Yellow: Client

SAMPLES MUST BE KEPT COOL (< 10° C) FROM TIME OF SAMPLING UNTIL DELIVERY TO BUREAU VERITAS

Bureau Veritas Canada (2019) Inc.



Your Project #: Campobello job# C2T2765

**Attention: Patricia Legette**

BUREAU VERITAS  
CAMPOBELLO  
6740 CAMPOBELLO ROAD  
MISSISSAUGA, ON  
CANADA L5N 2L8

**Report Date: 2022/10/18**

Report #: R3250114

Version: 1 - Final

**CERTIFICATE OF ANALYSIS**

**BUREAU VERITAS JOB #: C278786**

**Received: 2022/10/07, 21:59**

Sample Matrix: Soil  
# Samples Received: 2

Analyses	Date		Date Analyzed	Laboratory Method	Analytical Method
	Quantity	Extracted			
Moisture	2	N/A	2022/10/15	AB SOP-00002	CCME PHC-CWS m
Sulphide	2	2022/10/13	2022/10/13	AB SOP-00080	EPA9030B/SM4500S2-DF

**Remarks:**

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

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

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

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

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

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

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

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



Your Project #: Campobello job# C2T2765

**Attention: Patricia Legette**

BUREAU VERITAS  
CAMPOBELLO  
6740 CAMPOBELLO ROAD  
MISSISSAUGA, ON  
CANADA L5N 2L8

**Report Date: 2022/10/18**

Report #: R3250114

Version: 1 - Final

**CERTIFICATE OF ANALYSIS**

**BUREAU VERITAS JOB #: C278786**

**Received: 2022/10/07, 21:59**

Encryption Key

Please direct all questions regarding this Certificate of Analysis to your Project Manager.

Customer Solutions, Western Canada Customer Experience Team

Email: customersolutionswest@bureauveritas.com

Phone# (403) 291-3077

=====

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For Service Group specific validation please refer to the Validation Signature Page.





BUREAU  
VERITAS

Bureau Veritas Job #: C278786

Report Date: 2022/10/18

BUREAU VERITAS

Client Project #: Campobello job# C2T2765

Sampler Initials: IB

### RESULTS OF CHEMICAL ANALYSES OF SOIL

<b>Bureau Veritas ID</b>		BDY771	BDY772		
<b>Sampling Date</b>		2022/10/07 21:56	2022/10/07 21:56		
	<b>UNITS</b>	<b>BH22-2-4 SS9</b>	<b>BH22-3-3 SS10</b>	<b>RDL</b>	<b>QC Batch</b>
<b>Misc. Inorganics</b>					
Sulphide	mg/kg	2.7	3.7	0.5	A754056
RDL = Reportable Detection Limit					



BUREAU  
VERITAS

Bureau Veritas Job #: C278786

Report Date: 2022/10/18

BUREAU VERITAS

Client Project #: Campobello job# C2T2765

Sampler Initials: IB

### PHYSICAL TESTING (SOIL)

<b>Bureau Veritas ID</b>		BDY771	BDY772		
<b>Sampling Date</b>		2022/10/07 21:56	2022/10/07 21:56		
	<b>UNITS</b>	<b>BH22-2-4 SS9</b>	<b>BH22-3-3 SS10</b>	<b>RDL</b>	<b>QC Batch</b>
<b>Physical Properties</b>					
Moisture	%	8.1	9.9	0.30	A757459
RDL = Reportable Detection Limit					



Your Project #: ADM-22000797-A0  
Site Location: Hwy 401 from Victoria to Nelson Ave, ON  
Your C.O.C. #: 893860-04-01

**Attention: Nimesh Tamrakar**

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

**Report Date: 2022/11/01**  
Report #: R7367904  
Version: 1 - Final

**CERTIFICATE OF ANALYSIS**

**BUREAU VERITAS JOB #: C2U6020**

**Received: 2022/10/20, 12:51**

Sample Matrix: Soil  
# Samples Received: 1

Analyses	Quantity	Date	Date	Laboratory Method	Analytical Method
		Extracted	Analyzed		
Chloride (20:1 extract)	1	2022/10/25	2022/10/28	CAM SOP-00463	SM 23 4500-Cl E m
Conductivity	1	2022/10/25	2022/10/25	CAM SOP-00414	OMOE E3530 v1 m
Moisture (Subcontracted) (1, 2)	1	N/A	2022/10/27	AB SOP-00002	CCME PHC-CWS m
Sulphide in Soil (1)	1	N/A	2022/10/27	AB SOP-00080	EPA9030B/SM4500S2-DF
pH CaCl2 EXTRACT	1	2022/10/25	2022/10/25	CAM SOP-00413	EPA 9045 D m
Redox Potential (3)	1	2022/10/25	2022/10/26	CAM SOP-00421	SM 2580 B
Resistivity of Soil	1	2022/10/21	2022/10/26	CAM SOP-00414	SM 23 2510 m
Sulphate (20:1 Extract)	1	2022/10/25	2022/10/26	CAM SOP-00464	EPA 375.4 m

**Remarks:**

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

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

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

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

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



Your Project #: ADM-22000797-A0  
Site Location: Hwy 401 from Victoria to Nelson Ave, ON  
Your C.O.C. #: 893860-04-01

**Attention: Nimesh Tamrakar**

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

**Report Date: 2022/11/01**  
Report #: R7367904  
Version: 1 - Final

**CERTIFICATE OF ANALYSIS**

**BUREAU VERITAS JOB #: C2U6020**

**Received: 2022/10/20, 12:51**

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

**Encryption Key**

Please direct all questions regarding this Certificate of Analysis to:

Patricia Legette, Project Manager

Email: Patricia.Legette@bureauveritas.com

Phone# (905)817-5799

=====

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### SOIL CORROSIVITY PACKAGE (SOIL)

Bureau Veritas ID		UBI927			UBI927		
Sampling Date		2022/10/12			2022/10/12		
COC Number		893860-04-01			893860-04-01		
	<b>UNITS</b>	<b>BH22-2-1 SS13</b>	<b>RDL</b>	<b>QC Batch</b>	<b>BH22-2-1 SS13 Lab-Dup</b>	<b>RDL</b>	<b>QC Batch</b>
<b>Calculated Parameters</b>							
Resistivity	ohm-cm	660		8298711			
<b>CONVENTIONALS</b>							
Redox Potential	mV	290	N/A	8303878			
<b>Inorganics</b>							
Soluble (20:1) Chloride (Cl-)	ug/g	750	20	8303802	770	20	8303802
Conductivity	umho/cm	1510	2	8303555	1530	2	8303555
Available (CaCl2) pH	pH	7.98		8303401			
Soluble (20:1) Sulphate (SO4)	ug/g	41	20	8303809	41	20	8303809
Sulphide	mg/kg	2.8 (1)	0.5	8310746			
<b>Physical Testing</b>							
Moisture-Subcontracted	%	9.2	0.30	8319509	9.9	0.30	8319509
RDL = Reportable Detection Limit QC Batch = Quality Control Batch Lab-Dup = Laboratory Initiated Duplicate N/A = Not Applicable (1) Sample extracted past method-specified hold time. Analyzed past method specified hold time							



BUREAU  
VERITAS

Bureau Veritas Job #: C2U6020

Report Date: 2022/11/01

exp Services Inc

Client Project #: ADM-22000797-A0

Site Location: Hwy 401 from Victoria to Nelson Ave, ON

Sampler Initials: IB

## TEST SUMMARY

**Bureau Veritas ID:** UBI927  
**Sample ID:** BH22-2-1 SS13  
**Matrix:** Soil

**Collected:** 2022/10/12  
**Shipped:**  
**Received:** 2022/10/20

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	8303802	2022/10/25	2022/10/28	Alina Dobreanu
Conductivity	AT	8303555	2022/10/25	2022/10/25	Surinder Rai
Moisture (Subcontracted)	BAL	8319509	N/A	2022/10/27	Richard Ly
Sulphide in Soil	SPEC	8310746	N/A	2022/10/27	Bailey Morrison
pH CaCl <sub>2</sub> EXTRACT	AT	8303401	2022/10/25	2022/10/25	Taslina Aktar
Redox Potential	COND	8303878	2022/10/25	2022/10/26	Surinder Rai
Resistivity of Soil		8298711	2022/10/26	2022/10/26	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	8303809	2022/10/25	2022/10/26	Samuel Law

**Bureau Veritas ID:** UBI927 Dup  
**Sample ID:** BH22-2-1 SS13  
**Matrix:** Soil

**Collected:** 2022/10/12  
**Shipped:**  
**Received:** 2022/10/20

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	8303802	2022/10/25	2022/10/28	Alina Dobreanu
Conductivity	AT	8303555	2022/10/25	2022/10/25	Surinder Rai
Moisture (Subcontracted)	BAL	8319509	N/A	2022/10/27	Richard Ly
Sulphate (20:1 Extract)	KONE/EC	8303809	2022/10/25	2022/10/26	Samuel Law



### GENERAL COMMENTS

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

Package 1	1.3°C
-----------	-------

Results relate only to the items tested.

BUREAU  
VERITAS

Bureau Veritas Job #: C2U6020

Report Date: 2022/11/01

## QUALITY ASSURANCE REPORT

exp Services Inc

Client Project #: ADM-22000797-A0

Site Location: Hwy 401 from Victoria to Nelson Ave, ON

Sampler Initials: IB

QC Batch	Parameter	Date	Matrix Spike		SPIKED BLANK		Method Blank		RPD	
			% Recovery	QC Limits	% Recovery	QC Limits	Value	UNITS	Value (%)	QC Limits
8303401	Available (CaCl <sub>2</sub> ) pH	2022/10/25			100	97 - 103			0.75	N/A
8303555	Conductivity	2022/10/25			106	90 - 110	<2	umho/cm	1.3	10
8303802	Soluble (20:1) Chloride (Cl <sup>-</sup> )	2022/10/28	NC	70 - 130	104	70 - 130	<20	ug/g	3.0	35
8303809	Soluble (20:1) Sulphate (SO <sub>4</sub> )	2022/10/26	NC	70 - 130	107	70 - 130	<20	ug/g	0.89	35
8303878	Redox Potential	2022/10/26			100	95 - 105			5.4	N/A
8310746	Sulphide	2022/10/27	52 (1)	75 - 125	100	75 - 125	<0.5	mg/kg		
8319509	Moisture-Subcontracted	2022/10/27					<0.30	%	7.3	20

N/A = Not Applicable

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

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

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

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

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

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





BUREAU  
VERITAS

Bureau Veritas Job #: C2U6020

Report Date: 2022/11/01

exp Services Inc

Client Project #: ADM-22000797-A0

Site Location: Hwy 401 from Victoria to Nelson Ave, ON

Sampler Initials: IB

## VALIDATION SIGNATURE PAGE

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

---

Cristina Carriere, Senior Scientific Specialist

---

Ghayasuddin Khan, M.Sc., P.Chem., QP, Scientific Specialist, Inorganics

---

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

---

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

# CHAIN OF CUSTODY RECORD

Page 1 of 1

<b>INVOICE TO:</b>		<b>REPORT TO:</b>		<b>PROJECT INFORMATION:</b>		<b>Laboratory Use Only:</b>	
Company Name: #17488 exp Services Inc		Company Name: <u>EXP Services</u>		Quotation #: C20328		Bureau Veritas Job #:	
Attention: Accounts Payable		Attention: Nimesh Tamrakar		P.O. #:		Bottle Order #:	
Address: 1595 Clark Blvd		Address:		Project: ADM-22000797-A0		COC #:	
Brampton ON L6T 4V1				Project Name: Hwy 401 from Victoria to Nelso		Project Manager:	
Tel: (905) 793-9800 Fax: (905) 793-0641		Tel: (905) 796-3200 Ext: 3026 Fax:		Site #:		Patricia Legette	
Email: AP@exp.com; Karen.Burke@exp.com		Email: Nimesh.Tamrakar@exp.com		Sampled By:		C#893860-04-01	

MOE REGULATED DRINKING WATER OR WATER INTENDED FOR HUMAN CONSUMPTION MUST BE SUBMITTED ON THE BUREAU VERITAS DRINKING WATER CHAIN OF CUSTODY

<b>Regulation 153 (2011)</b>			<b>Other Regulations</b>			<b>Special Instructions</b>		
<input type="checkbox"/> Table 1	<input type="checkbox"/> Res/Park	<input type="checkbox"/> Medium/Fine	<input type="checkbox"/> CCME	<input type="checkbox"/> Sanitary Sewer Bylaw				
<input type="checkbox"/> Table 2	<input type="checkbox"/> Ind/Comm	<input type="checkbox"/> Coarse	<input type="checkbox"/> Reg 558	<input type="checkbox"/> Storm Sewer Bylaw				
<input type="checkbox"/> Table 3	<input type="checkbox"/> Agri/Other	<input type="checkbox"/> For RSC	<input type="checkbox"/> MISA	Municipality				
<input type="checkbox"/> Table			<input type="checkbox"/> PWQO	Reg 406 Table				
<input type="checkbox"/> Other								
Include Criteria on Certificate of Analysis (Y/N)?								
Sample Barcode Label	Sample (Location) Identification	Date Sampled	Time Sampled	Matrix	Field Filtered (please circle):	Metals / Hg / Cr VI	Soil Corrosivity Package	ANALYSIS REQUESTED (PLEASE BE SPECIFIC)
1	BH22-2-1 SS13	2022/10/12		SOIL	✓		✓	
2	BH22-2-1 SS13	2022/10/12		SOIL	✓		✓	
3				SOIL				
4				SOIL				
5				SOIL				
6				SOIL				
7								
8								
9								
10								

Turnaround Time (TAT) Required:  
Please provide advance notice for rush projects

**Regular (Standard) TAT:**  
(will be applied if Rush TAT is not specified):  
Standard TAT = 5-7 Working days for most tests.  
Please note: Standard TAT for certain tests such as BOD and Dioxins/Furans are > 5 days - contact your Project Manager for details.

**Job Specific Rush TAT (if applies to entire submission)**  
Date Required: Time Required:  
Rush Confirmation Number: (call lab for #)

# of Bottles: Comments:

20-Oct-22 12:51

Patricia Legette



C2U6020

ALCO ENV 511

* RELINQUISHED BY: (Signature/Print)		Date: (YY/MM/DD)	Time	RECEIVED BY: (Signature/Print)		Date: (YY/MM/DD)	Time	# Jars used and not submitted	Laboratory Use Only		
<u>[Signature]</u> Juan Barua		22/10/20		<u>[Signature]</u> / Anmol		22/10/20	12:51		Time Sensitive	Temperature (°C) on Reel	Custody Seal
										1/2/1	Present
											Intact
											Yes
											No

\* UNLESS OTHERWISE AGREED TO IN WRITING, WORK SUBMITTED ON THIS CHAIN OF CUSTODY IS SUBJECT TO BUREAU VERITAS'S STANDARD TERMS AND CONDITIONS. SIGNING OF THIS CHAIN OF CUSTODY DOCUMENT IS ACKNOWLEDGMENT AND ACCEPTANCE OF OUR TERMS WHICH ARE AVAILABLE FOR VIEWING AT WWW.BVNA.COM/TERMS-AND-CONDITIONS.

\* IT IS THE RESPONSIBILITY OF THE RELINQUISHER TO ENSURE THE ACCURACY OF THE CHAIN OF CUSTODY RECORD. AN INCOMPLETE CHAIN OF CUSTODY MAY RESULT IN ANALYTICAL TAT DELAYS.

\*\* SAMPLE CONTAINER, PRESERVATION, HOLD TIME AND PACKAGE INFORMATION CAN BE VIEWED AT WWW.BVNA.COM/RESOURCES/CHAIN-OF-CUSTODY-FORMS.

SAMPLES MUST BE KEPT COOL (< 10° C) FROM TIME OF SAMPLING UNTIL DELIVERY TO BUREAU VERITAS

White: Bureau Veritas Yellow: Client



Your Project #: Campobello job# C2U6020

**Attention: Patricia Legette**

BUREAU VERITAS  
CAMPOBELLO  
6740 CAMPOBELLO ROAD  
MISSISSAUGA, ON  
CANADA L5N 2L8

**Report Date: 2022/11/01**

Report #: R3257319

Version: 1 - Final

**CERTIFICATE OF ANALYSIS**

**BUREAU VERITAS JOB #: C282985**

**Received: 2022/10/21, 16:00**

Sample Matrix: Soil  
# Samples Received: 1

Analyses	Date		Date Analyzed	Laboratory Method	Analytical Method
	Quantity	Extracted			
Moisture	1	N/A	2022/10/27	AB SOP-00002	CCME PHC-CWS m
Sulphide	1	2022/10/26	2022/10/27	AB SOP-00080	EPA9030B/SM4500S2-DF

**Remarks:**

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

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

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



Your Project #: Campobello job# C2U6020

**Attention: Patricia Legette**

BUREAU VERITAS  
CAMPOBELLO  
6740 CAMPOBELLO ROAD  
MISSISSAUGA, ON  
CANADA L5N 2L8

**Report Date: 2022/11/01**

Report #: R3257319

Version: 1 - Final

**CERTIFICATE OF ANALYSIS**

**BUREAU VERITAS JOB #: C282985**

**Received: 2022/10/21, 16:00**

**Encryption Key**

Please direct all questions regarding this Certificate of Analysis to:  
Customer Solutions, Western Canada Customer Experience Team  
Email: customersolutionswest@bureauveritas.com  
Phone# (403) 291-3077

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### RESULTS OF CHEMICAL ANALYSES OF SOIL

<b>Bureau Veritas ID</b>		BFB941		
<b>Sampling Date</b>		2022/10/12		
	<b>UNITS</b>	<b>BH22-2-1 SS13</b>	<b>RDL</b>	<b>QC Batch</b>
<b>Misc. Inorganics</b>				
Sulphide	mg/kg	2.8 (1)	0.5	A773361
RDL = Reportable Detection Limit				
(1) Sample extracted past method-specified hold time. Analyzed past method specified hold time				



BUREAU  
VERITAS

Bureau Veritas Job #: C282985

Report Date: 2022/11/01

BUREAU VERITAS





Client Project #: Campobello job# C2U6020

Sampler Initials: I.B

### PHYSICAL TESTING (SOIL)

<b>Bureau Veritas ID</b>		BFB941	BFB941		
<b>Sampling Date</b>		2022/10/12	2022/10/12		
	<b>UNITS</b>	<b>BH22-2-1 SS13</b>	<b>BH22-2-1 SS13 Lab-Dup</b>	<b>RDL</b>	<b>QC Batch</b>
<b>Physical Properties</b>					
Moisture	%	9.2	9.9	0.30	A774882
RDL = Reportable Detection Limit					
Lab-Dup = Laboratory Initiated Duplicate					

## Appendix F – Slope Stability Analysis Results

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
	Abutment	Mohr-Coulomb	24	100	45
	Granular A/B Type II	Mohr-Coulomb	22	0	35
	Sand and silt Till (generally dense to very dense)	Mohr-Coulomb	22	0	35
	Silty Sand to Sand and Silt Fill (generally compact to dense)	Mohr-Coulomb	20	0	30

Hwy 401 Victoria Park Ave. to Neilson Rd.  
CPR Overhead structure  
West Abutment  
Drained Static Conditions

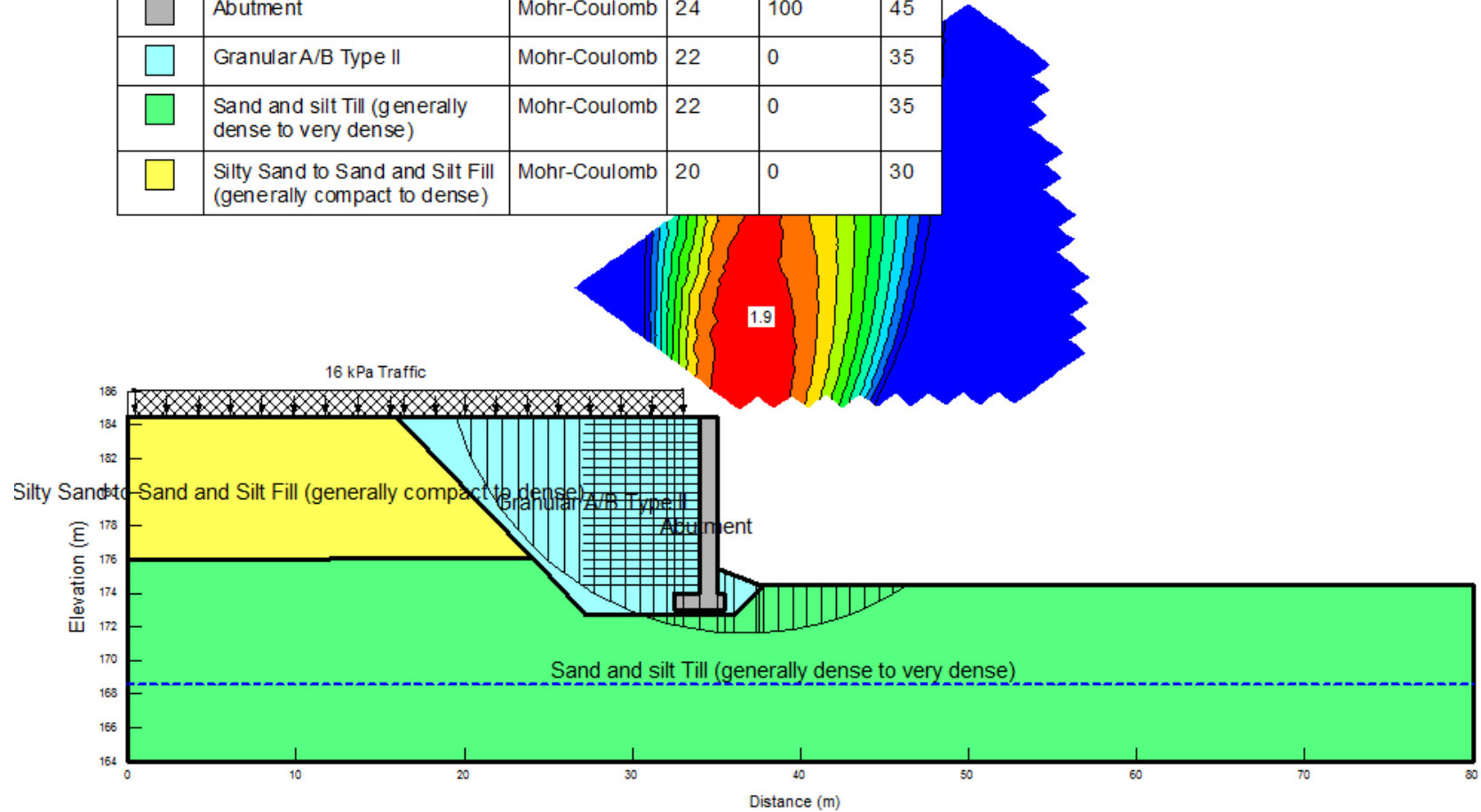






Figure 1: Slope stability analysis for west abutment – drained static condition



Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
	Abutment	Mohr-Coulomb	24	100	45
	Granular A/B Type II	Mohr-Coulomb	22	0	35
	Sand and silt Till (generally dense to very dense)	Mohr-Coulomb	22	0	35
	Silty Sand to Sand and Silt Fill (generally compact to dense)	Mohr-Coulomb	20	0	30

Hwy 401 Victoria Park Ave. to Neilson Rd.  
CPR Overhead structure  
West Abutment  
Drained Static Conditions

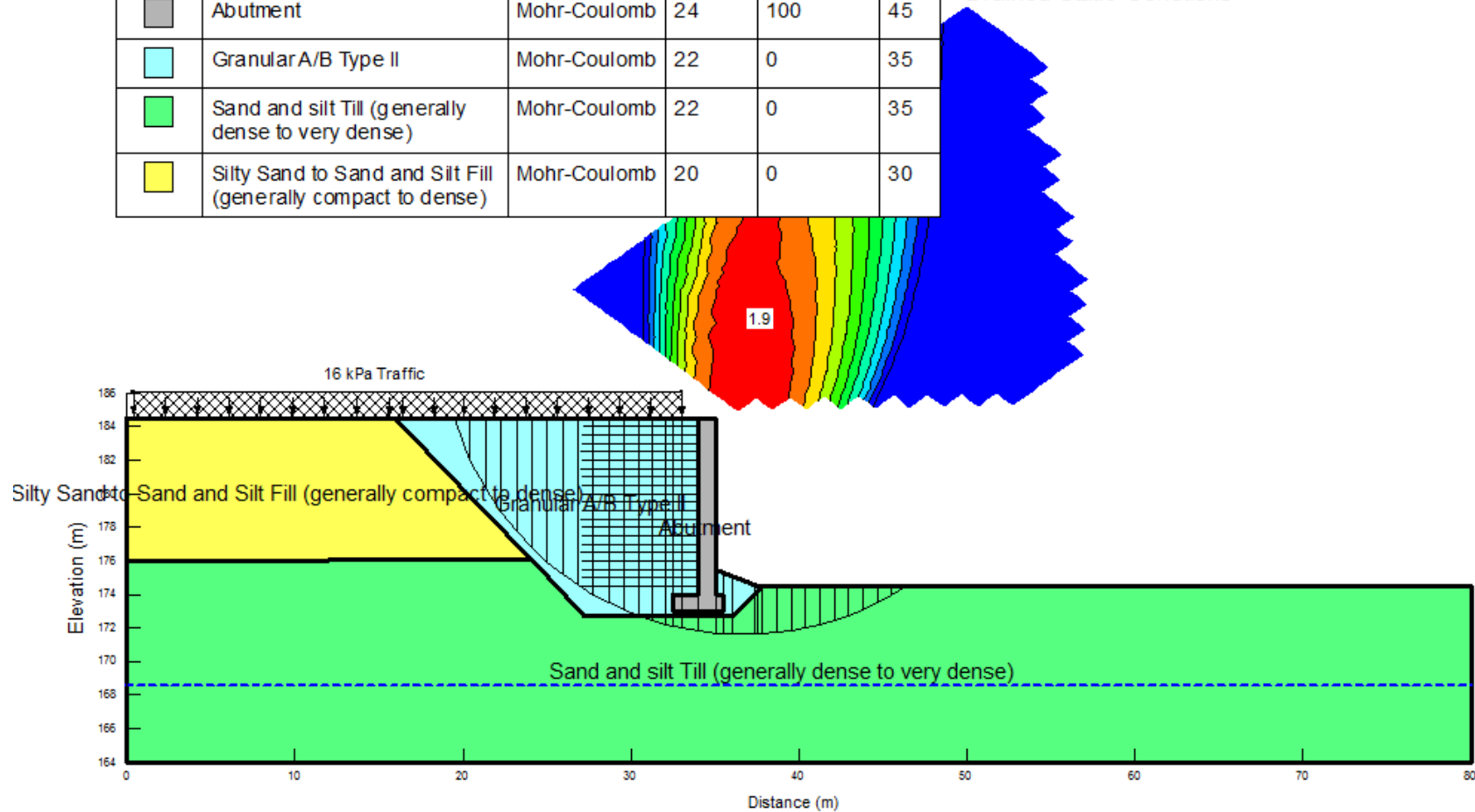





Figure 2: Slope stability analysis for west abutment – drained seismic condition

Hwy 401 Victoria Park Ave. to Neilson Rd.  
 CPR Overhead structure  
 RSS wall - West  
 Drained Static Conditions

Color	Name	Model	Unit Weight (kN/m <sup>3</sup> )	Cohesion' (kPa)	Phi' (°)
	RSS wall	Mohr-Coulomb	24	100	45
	Sand and silt Till (generally dense to very dense)	Mohr-Coulomb	22	0	35
	Silty Sand to Sand and Silt Fill (generally compact to dense)	Mohr-Coulomb	20	0	30

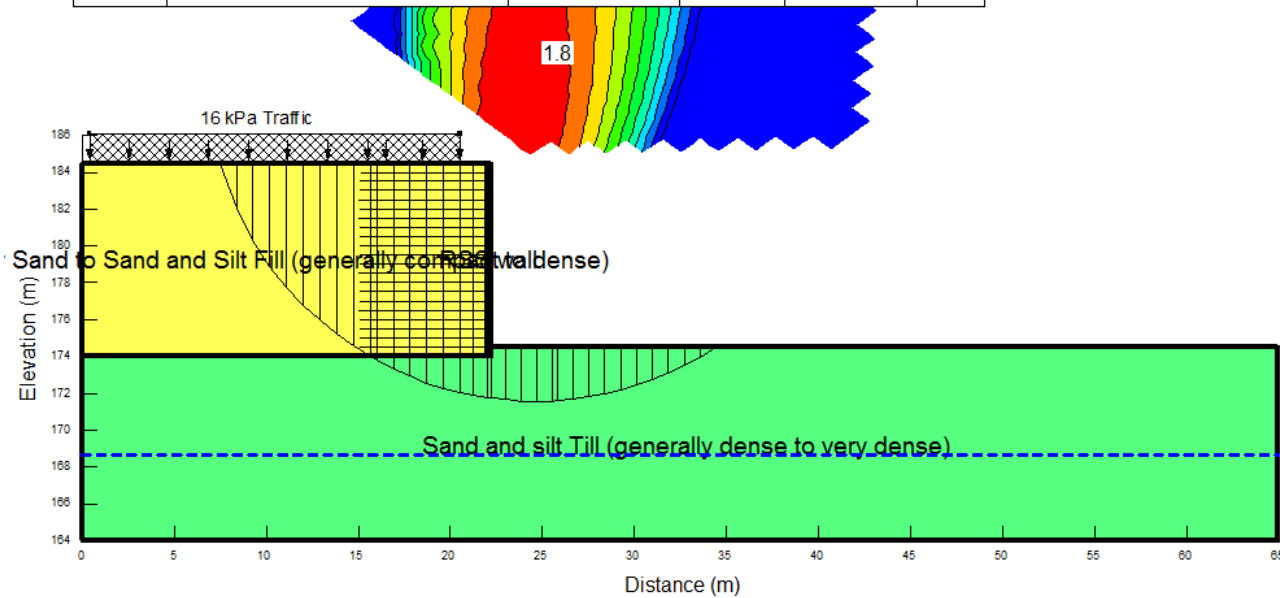





Figure 3: Slope stability analysis for retaining wall - west – drained static condition

Hwy 401 Victoria Park Ave. to Neilson Rd.  
 CPR Overhead structure  
 RSS wall - West  
 Drained Seismic Conditions

Color	Name	Model	Unit Weight (kN/m <sup>3</sup> )	Cohesion' (kPa)	Phi' (°)
	RSS wall	Mohr-Coulomb	24	100	45
	Sand and silt Till (generally dense to very dense)	Mohr-Coulomb	22	0	35
	Silty Sand to Sand and Silt Fill (generally compact to dense)	Mohr-Coulomb	20	0	30

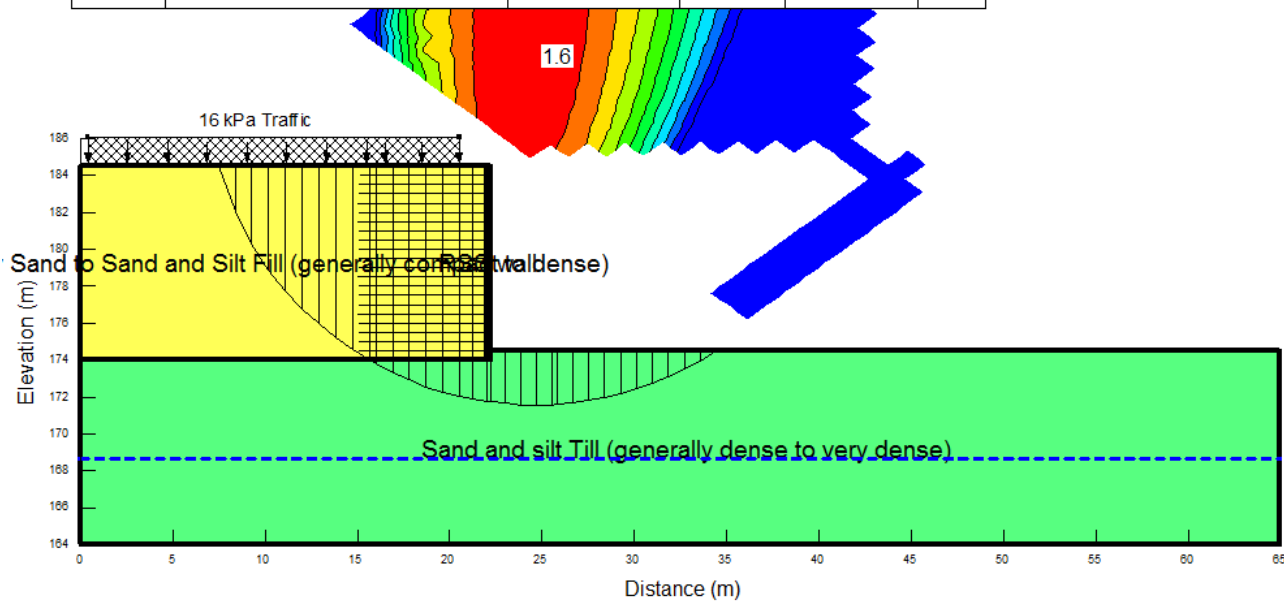




Figure 4: Slope stability analysis for for retaining wall - west – drained seismic condition

Color	Name	Model	Unit Weight (kN/m <sup>3</sup> )	Cohesion' (kPa)	Phi' (°)
	Sand and silt Till (generally dense to very dense)	Mohr-Coulomb	22	0	35
	Silty Sand to Sand and Silt Fill (generally compact to dense)	Mohr-Coulomb	20	0	30

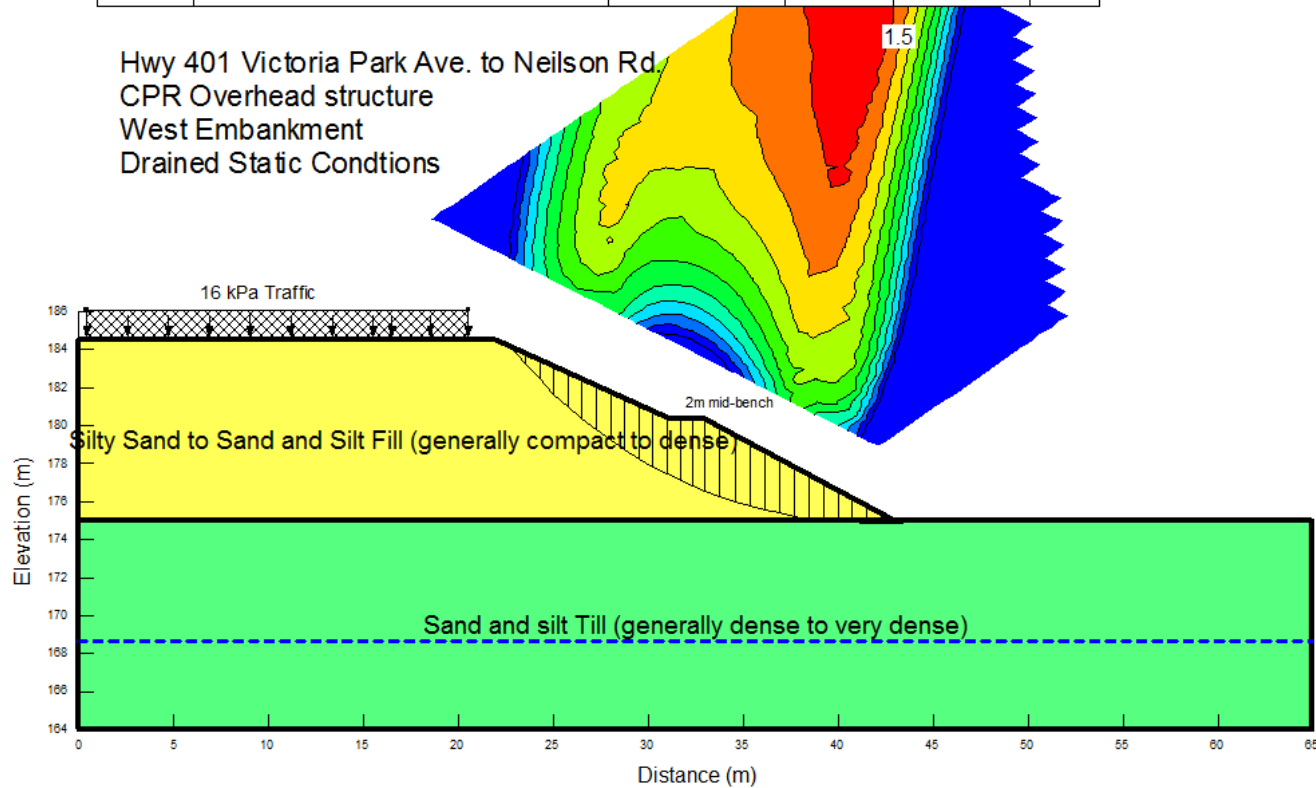




Figure 5: Slope stability analysis for west embankment– drained static condition

Color	Name	Model	Unit Weight (kN/m <sup>3</sup> )	Cohesion' (kPa)	Phi' (°)
	Sand and silt Till (generally dense to very dense)	Mohr-Coulomb	22	0	35
	Silty Sand to Sand and Silt Fill (generally compact to dense)	Mohr-Coulomb	20	0	30

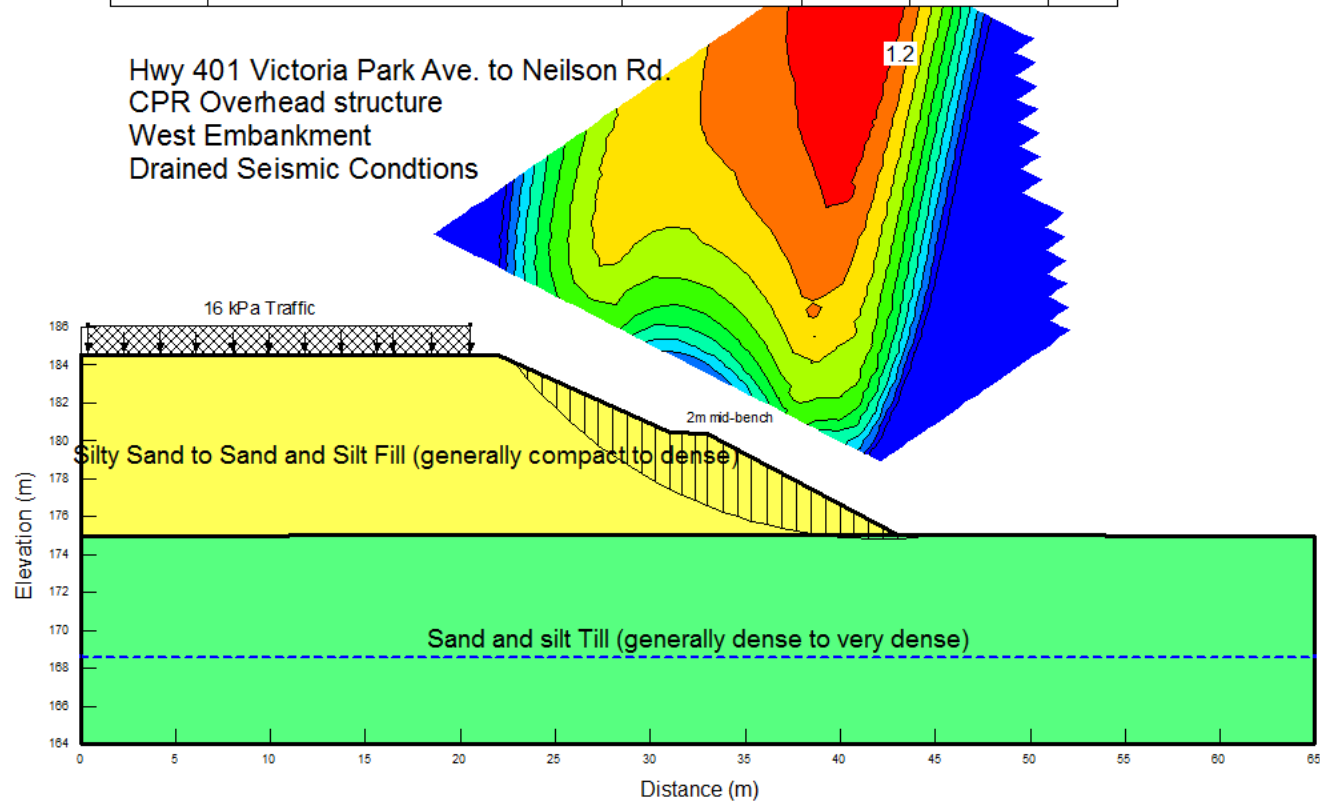
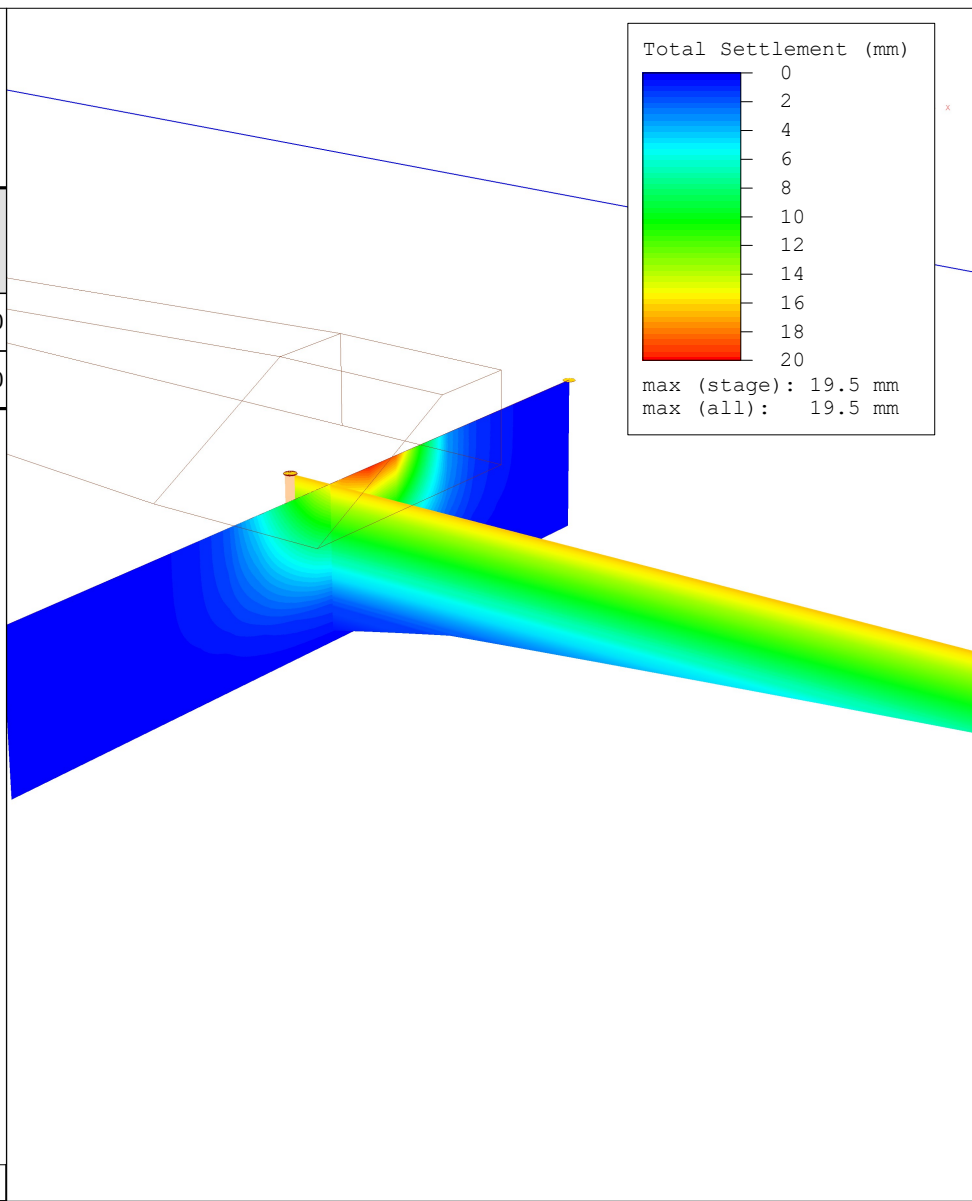
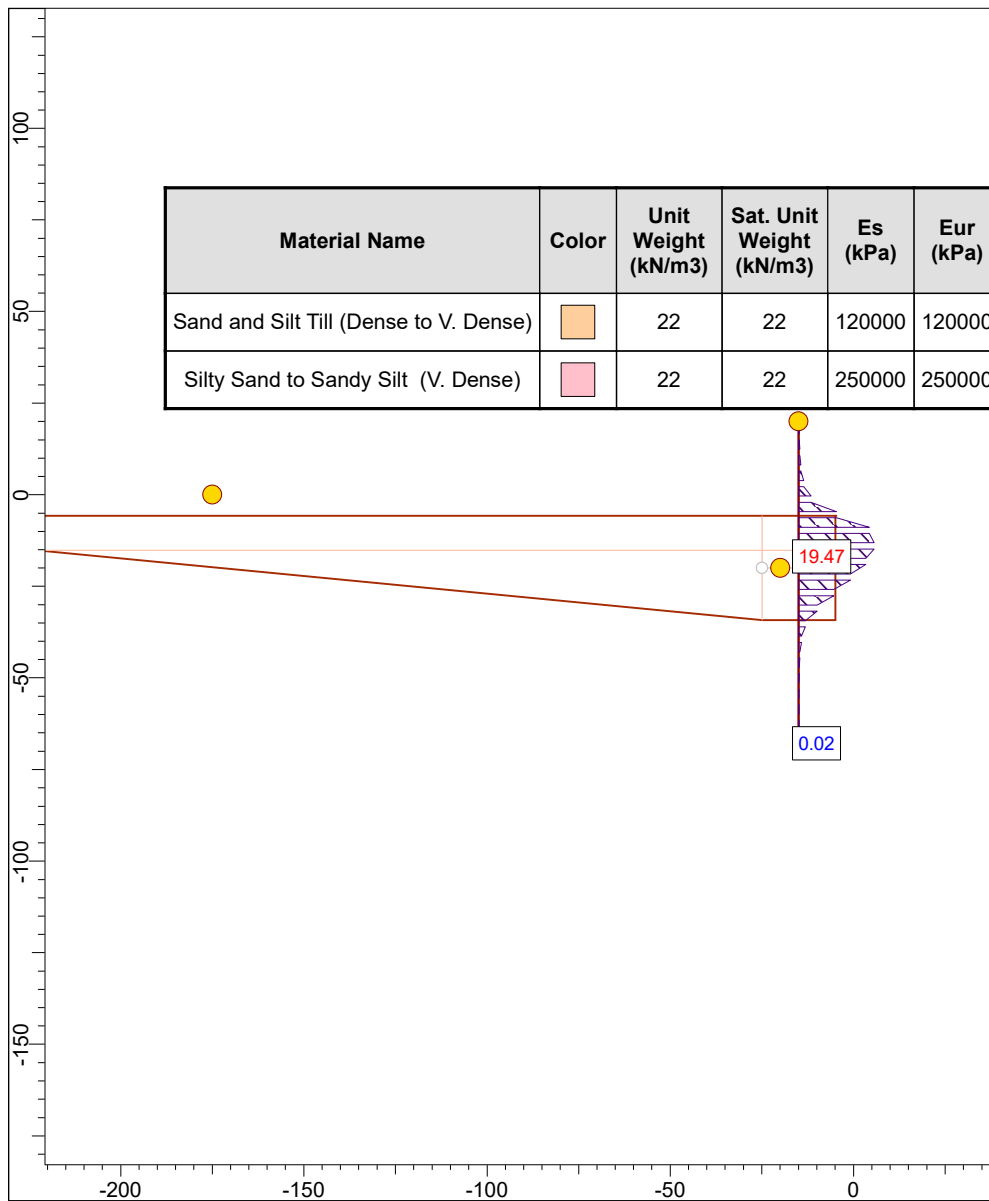


Figure 6: Slope stability analysis for west embankment – drained seismic condition

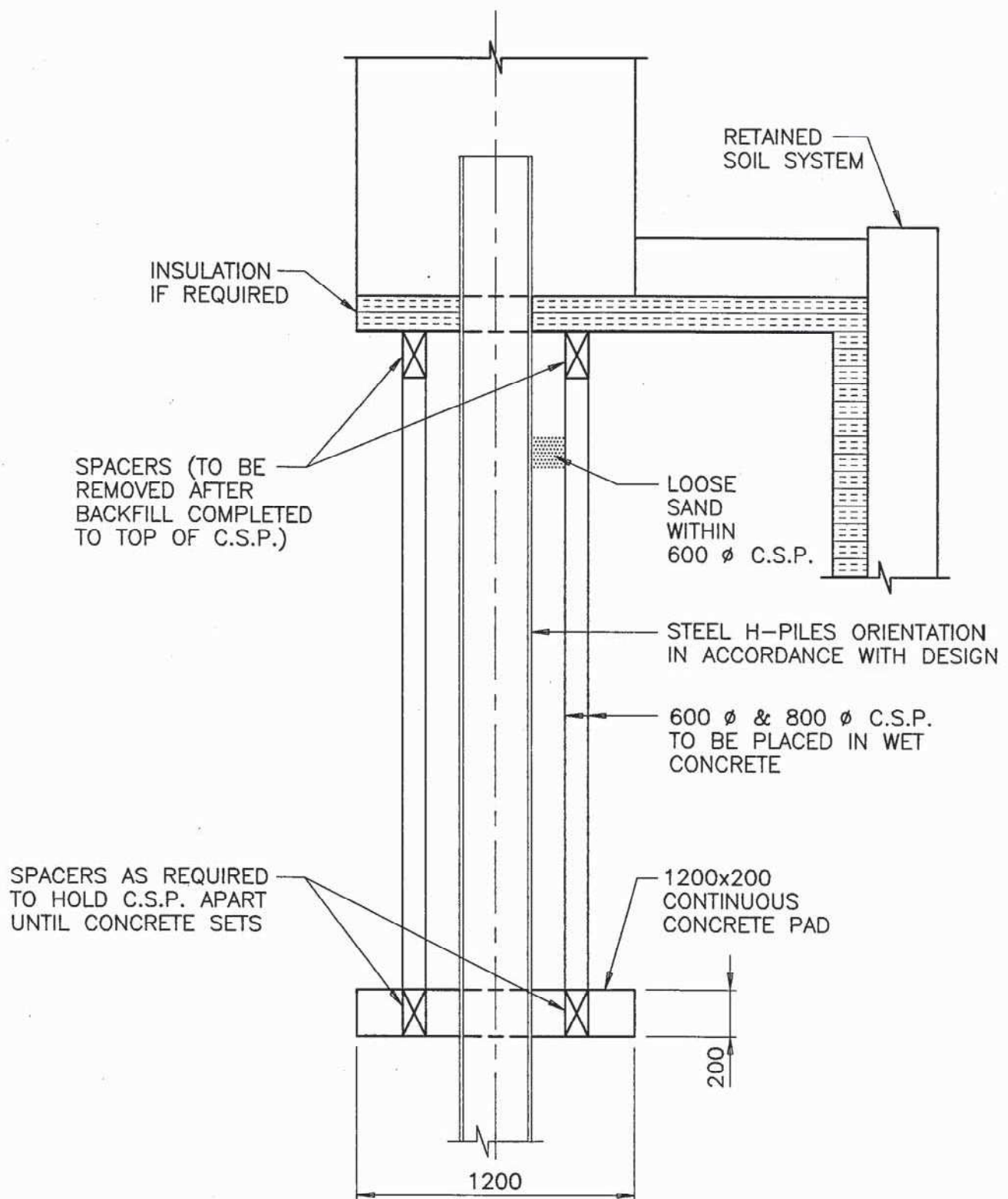
## Appendix G – Settlement Analysis Results



	Project		Hwy 11 2+1 Roadway Model GWP 5033-22-00	
	Analysis Description		SW10 Settlement of East Embankment - East Extension	
	Drawn By		T Lardner	Company EXP Services Inc.
	Date		8/5/2014, 2:41:38 PM	File Name CPR West approach Embankment_ with Engineered Fill.s3z

## Appendix H – Integral Abutment with Retained Soil System (Fig.7)





INTEGRAL ABUTMENT WITH RETAINED SOIL SYSTEM

FIG. 7

## Appendix I – Suggested Vibration Monitoring Plan

### **Suggested Vibration Monitoring Plan – CP Rail Overhead Structure**

- The vibration monitoring should be conducted to verify the vibration levels near the existing structure and the utilities identified in the area during the pile driving, hoe ramming for demolition of the first element of the bridge. For bridge structures in good condition a limit of peak particle velocity (PPV) 50 mm/s is generally considered to be applicable for control MTO bridges, (noting that other entities having jurisdiction in particular settings may have more stringent regulations).
- No vibration monitoring is required for private or commercial building(s) which is not present in the zone of influence for construction for this structure.
- A normal background vibration reading produced by no construction related activities should be taken one week prior to driving operations.
- Attended vibration monitoring can be conducted by a qualified technician during pile driving. The vibration monitoring program should include monitoring with seismograph near the structure to confirm the magnitude of the vibration produced by the pile driving operation. The seismograph consists of an ISEE geophone and base fitted with an internal battery can be considered. The qualified technician attended during pile driving operation should take readings from the seismograph and make notes of construction activities that produced the vibration events.
- 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.
- Once pile driving is substantially complete, a final report should be prepared summarizing all vibration measurements made during that phase of construction.

END OF SECTION

## Appendix J – Previous Investigation Borehole Logs

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS &amp; TESTING DIVISION

## RECORD OF BOREHOLE NO. 10

FOUNDATION SECTION

JOB 66-F-89 LOCATION Hwy. 401 & C.P.R. (Sta. 322 + 22 155' Rt.) ORIGINATED BY V.K.  
W.P. 257-61 BORING DATE December 2, 1966 COMPILED BY V.K.  
DATUM Geodetic BOREHOLE TYPE Pen-Drill Auger CHECKED BY \_\_\_\_\_

SOIL PROFILE		STRAT. PLT	SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE		LIQUID LIMIT — WL PLASTIC LIMIT — WP		BULK DENSITY P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION		NUMBER	TYPE	BLOWS / FOOT		BLOWS / FOOT	SHEAR STRENGTH P.S.F.	WATER CONTENT — W	WATER CONTENT %		
586.7	GROUND LEVEL											
0.0	(Brown)											
			1	SS	32							
			2	SS	100/6"	580						Gr. 2, Sa. 48 Si. 38 Cl. 12
			3	SS	100/6"							
	Silty sand to sandy silt with trace of clay and occasional gravel.		4	SS	116							Gr. 1, Sa. 46 Si. 44 Cl. 9
			5	SS	100/6"	570						
			6	SS	100/5"							
	Dense to very dense.		7	SS	100/6"	560						
			8	SS	100/5"							
			9	SS	100/6"	550						
			10	SS	124							
			11	SS	84	540						
			12	SS	63							
			13	SS	62	530						
525.2			14	SS	69							
61.5	End of Borehole					520						

## RECORD OF BOREHOLE NO. 11

FOUNDATION SECTION

## MATERIALS &amp; TESTING DIVISION

JOB 66-F-89LOCATION Hwy. 401 & C.P.R., Sta. 321 / 02, 131' Rt.ORIGINATED BY V.K.W.P. 257-61BORING DATE December 2, 1966COMPILED BY V.K.DATUM GeodeticBOREHOLE TYPE Drive BX Casing & WashCHECKED BY ✓

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE					LIQUID LIMIT ——— w <sub>L</sub> PLASTIC LIMIT ——— w <sub>p</sub> WATER CONTENT ——— w			BULK DENSITY Y P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT		BLOWS / FOOT					w <sub>p</sub> ——— w <sub>L</sub>				
							20	40	60	80	100	10	20	30		
							SHEAR STRENGTH P.S.F.					WATER CONTENT %				
583.0	GROUND LEVEL															
0.0	(Brown)					580										
	Clayey silt with pockets of silty sand and trace of gravel.  Very Stiff to Hard		1	SS	28											
			2	SS	84											
			3	SS	100/5"											
			4	SS	111	570										
			5	SS	100/5"											
			6	SS	100/6"	560										
			7	SS	100/5"											
			8	SS	85	550										
			9	SS	34											
545.0																
38.0	Silty sand to sandy silt, occasional gravel.  Very Dense.		10	SS	100/6"	540										
			11	SS	126											
531.5			12	SS	106											
51.5	End of Borehole					530										

Clayey silt with pockets of silty sand and trace of gravel.

Very Stiff to Hard

Gr.2, Sa.47  
Si.38  
Cl.13Gr.6, Sa.45  
Si.38  
Cl.11

El.553.

Gr.0, Sa.91  
Si.&Cl. 9

## Appendix K – Non-Standard Special Provision (NSSP)

## **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.

## Appendix L – 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 (°)	-79.292

Please select one of the tabs below.

NBC 2020

Additional Values

Plots

API

Background Information

The 5%-damped spectral acceleration ( $S_a(T, X)$ , where  $T$  is the period, in s, and  $X$  is the site designation) and peak ground acceleration ( $PGA(X)$ ) values are given in units of acceleration due to gravity ( $g$ ,  $9.81 \text{ m/s}^2$ ). Peak

ground velocity (PGV(X)) values are given in m/s. Probability is expressed in terms of percent exceedance in 50 years. Further information on the calculation of seismic hazard is provided under the *Background Information* tab.

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

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

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

$S_a(0.2, X_C)$	$S_a(0.5, X_C)$	$S_a(1.0, X_C)$	$S_a(2.0, X_C)$	$S_a(5.0, X_C)$	$S_a(10.0, X_C)$	PGA( $X_C$ )	PGV( $X_C$ )
0.325	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	0.059	0.0267	0.00659	0.00227	0.0933	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	0.0534	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