



## Foundation Investigation and Design Report

Highway 401 Eastbound Express and Collector Lanes between Victoria Park Avenue and Neilson Road - **Superstructure Replacement at Kennedy Road Overpass Eastbound Core and Collectors Structure (Site 37X-0214/B1 & B3)**

Assignment No. 2021-E-0018  
MTO Central Region  
Geocres Number: 30M14-552  
(Latitude: 43.774739, Longitude: -79.284516)

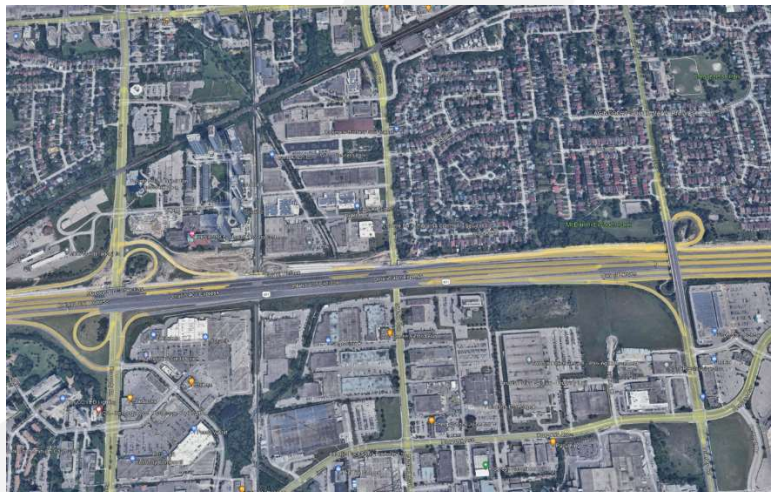
**Type of Document:**  
Foundation Investigation and Design Report

**Project Number:**  
ADM-22000797-A0

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**Date Submitted:**  
December 20, 2024



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## Issue and Revised Record

Rev.	Date	Format	Prepared by	Reviewed by	Approved by	Description
Rev. 0	September 19, 2024	pdf	E. Lu	T. Lardner T.C. Kim	S. Gonsalves	Report
Rev. 1	December 20, 2024	pdf	T. Lardner	T.C. Kim	S. Gonsalves	Report

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## Part I: Foundation Investigation Report

Highway 401 Eastbound Express and Collector Lanes between Victoria Park Avenue and Neilson Road – Kennedy Road Overpass (Site 37X-0214/B1 & B3)

## 1.0 Introduction

EXP Services Inc. (EXP) was retained by AECOM on behalf of The Ministry of Transportation (MTO) to provide detailed foundation investigation and engineering services for the proposed Highway 401 Eastbound rehabilitation and construction project. The findings, analyses and recommendations are presented in a Foundation Investigation Design Report created for each structure along the proposed highway. The work was undertaken under Assignment No. 2021-E-0018. The terms of reference (TOR) and the scope of work for the foundation investigation are outlined in Ministry of Transportation Ontario's (MTO) Request for proposal, dated June 2021. The scope of this report is specifically limited to the proposed location of the Kennedy Road Overpass structure (Site 37X-0214/B1 & B3).

The General Arrangement drawings (GA) for the bridge structure were provided to EXP by AECOM. The purpose of the investigation was to evaluate the subsurface conditions along the structure alignment to permit a detailed design for the proposed superstructure 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 Kennedy Rd O/P Bridge Rehab.*, prepared by AECOM, dated May 2023, shows the preliminarily proposed configuration of the Kennedy Road Overpass 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 *"Kennedy Road Overpass (Site No. 37-214) Rehabilitation and Northward Widening, 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 4, 2019. A summary of the proposed structure is as follows:

1. The existing structure is a 36.80 m long two-span bridge. It is understood that the existing abutments, piers and retaining wall foundations are supported on spread footings. The founding elevations of the spread footings are not known at the time of preparing this report. However, it is assumed that it is similar to the Westbound Core and Collectors Structure. Based on the previous FIDRs, the existing abutments supported on 3.4 m wide footings founded at about Elevation 167.6 m to 166.9 m and the centre piers are supported on 1.8 m wide footings founded at Elevation 167.5 m to 166.7 m. It is also noted that the existing north wingwalls/retaining walls are found on spread footings that are 4.3 m wide and 20.4 m long at Elevation 167.6 m.
2. The existing structure is proposed to undergo superstructure replacement, which includes replacement of the existing bridge deck and girders, conversion to semi-integral abutment and rehabilitation of wingwalls/retaining walls. The existing foundations will remain to support the abutments and retaining walls.
3. Initially, 0.5 m of widening on the south side of Highway 401 was proposed, however, it is understood that widening is now considered not within the proposed works.

The previous FIDRs and GA drawing by AECOM, in addition to contract package drawings titled *401 WB Core & Collector Lanes – Kennedy Rd. Overpass – Bridge Rehabilitation (Cont. No. 2019-2011, WP No. 2401/2402-15-01)*, produced by WSP Global Inc., dated February 2019, were reviewed as part of this report. These background documents were used for the initial context to

address the nature and scope of the investigation. It is understood that some changes might occur as a result of normal refinement or the findings of the geotechnical report.

## 3.0 Site Description and Geological Setting

### 3.1 Site Description

The site is located at the intersection of Highway 401 and Kennedy Road, 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 adjacent to residential zones to the northwest of the site. In general, the terrain in this area is relatively flat, with the natural ground surface sloping gently towards the south. The Highway 401 pavement grade ranges between about Elevation 176.2 m to 176.8 m while, the Kennedy Road pavement grade is at Elevation about 170 m at the structure site. Based on the FIDRs by Golder Associates Ltd., the fill thickness is assumed to be about 8 m.

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

### 3.2 Geological Setting

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

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

## 4.0 Previous Geotechnical Investigation

During the tender design for the project, four (4) previous reports were issued which contain relevant information to the proposed Kennedy Road Overpass structure (Site 37X-0214/B1 & B3), as follows:

1. Geocres No. 30M14-69 *"Foundation Investigation Highway #401 at Kennedy Road"* by Department of Highways – Ontario, dated June 14, 1954.
2. Geocres No. 30M14-71 *"Foundation Investigation Report for The Proposed New Structure at Hwy. 401 and Kennedy Intersection, Scarborough Twp., York County, District No. 6 (Toronto), W.J. 66-F-33, W.P. 858-61."* by The Ministry of Transportation Ontario (MTO), Foundation Section, Materials and Testing Div., dated June 20, 1966.
3. Geocres No. 30M14-338 *"Bridge Widening and Replacement Highway 401 Rehabilitation from Warden Avenue to Brock Road, Toronto, Ontario, W.O.07-20012."* by Golder Associates Ltd., dated April 2012.
4. Geocres No. 30M14-486 *"Kennedy Road Overpass (Site No. 37-214) Rehabilitation and Northward Widening, 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 4, 2019

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

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

Borehole No.	Borehole Location	Location (MTM NAD83 Zone 10)		Latitude	Longitude	Borehole Elevation (m)	Borehole Depth (m)
		Northing	Easting				
71-1	Eastbound Core East Approach	4848340.5	322165.4	43.774821	-79.284283	167.9	15.2
71-4	Eastbound Collector West Approach	4848307.8	322135.7	43.774528	-79.284653	170.4	14.2
71-5	Eastbound Collector East Approach	4848,322.1	322186.2	43.774655	-79.284025	168.2	18.7

## 5.0 Field Investigation and Laboratory Analyses

### 5.1 Site Investigation and Field Testing

A site-specific investigation was undertaken by EXP between October 21, 2022, and November 27, 2022, and it included the following:

1. A walkover site assessment was carried out by a Geotechnical Engineer from EXP.
2. Subsequent to the borehole layouts in the field, existing utilities were cleared by public utility companies.
3. At the time of this report, seven (7) boreholes have been completed for this structure (BH22-4-02 to BH22-4-07) as part of the additional investigation. A summary of boreholes completed by EXP is listed in Table 1.2 below. The borehole was drilled using a truck-mounted CME-75 machine (owned and operated by Drilltech drilling Ltd.) equipped with solid and hollow stem augers, mud rotary equipment, and fitted with capability for Standard Penetration Testing (SPT);
4. An obstruction was encountered in BH22-4-04A at about 4.6 m below ground surface. Therefore, a companion borehole (BH22-4-04B) was drilled about 2.8 m northeast to evaluate the subsurface in the adjacent area. It should be noted that since soil sampling was conducted to a depth of 4.6 m in BH22-4-04A, soil sampling in BH22-4-04B continued from 4.6 m below ground surface.
5. Soil samples in the boreholes were taken at frequent intervals of depth by the Standard Penetration Test method (SPT), in general accordance with ASTM D1586. The test consists of freely dropping a 63.5 kg hammer a vertical distance of 0.76 m to drive a 51 mm O.D. split barrel (SS-split-spoon) sampler into the ground. The number of blows of the hammer required to drive the sampler into the relatively undisturbed ground by a vertical distance of 0.30 m is recorded as the Standard Penetration Resistance, or the N-value, of the soil which is indicative of the compactness of granular (or cohesionless) soils (gravels, sands and silts) or the consistency of cohesive soils (clays and clayey soils).
6. The fieldwork was supervised by a member of EXP's engineering staff who directed the drilling and sampling operation, logged borehole data in accordance with MTO and/or ASTM Standards for Soils Classification, and retrieved soil samples for subsequent laboratory testing and identification.
7. All spoon samples obtained in the Standard Penetration Tests (SPT, ASTM D-1586) were placed in moisture proof bags after field classification. Samples were allocated from the spoon samples for moisture content testing without delay. They were subsequently re-examined under controlled laboratory conditions prior to assigning other laboratory tests.

8. Selected soil samples for corrosivity testing were sent to the Bureau Veritas Laboratories (formerly Maxxam Analytics), a CALA-certified and accredited laboratory in Mississauga, Ontario. The selected soil samples for the analytical testing were placed in a laboratory prepared glass jar, labelled, and stored in a secure cooler.
9. The borehole locations and their ground surface elevations were surveyed by EXP using a Trimble DA2 GNSS receiver with Trimble Catalyst GNSS positioning, having an accuracy of  $\pm 0.10$  m horizontal and vertical directions. MTM NAD83 Zone 10 coordinates and the geodetic elevation for the boreholes are listed in Table 1.2 below. It can also be found on the Record of Borehole Sheet (Appendix D); and
10. Upon completion of drilling and field testing, the boreholes were backfilled with a mixture of bentonite and auger cuttings. The borehole decommissioning was in general accordance with the Ministry of the Environment Regulation 903, as amended by Regulation 128/03 (the well regulation under the Ontario Water Resources Act).

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

Borehole No.	Borehole Location	Location (MTM NAD83 Zone 10)		Latitude	Longitude	Borehole Elevation (m)	Borehole Depth (m)
		Northing	Easting				
<b>BH22-4-02</b>	~12 m west of West Abutment, b/w EBL and WBL Express	4848340.9	322113.1	43.774826	-79.284933	176.7	15.8
<b>BH22-4-03</b>	~12 m east of East Abutment, b/w EBL and WBL Express	4848359.5	322170.8	43.774992	-79.284215	176.4	15.8
<b>BH22-4-04A</b>	~22 m east of East Abutment, South Side (EBL Collector)	4848331.9	322193.1	43.774743	-79.283939	176.2	4.3 <sup>1</sup>
<b>BH22-4-04B</b>	~25 m east of East Abutment, South Side (EBL Collector)	4848333.9	322194.2	43.774761	-79.283925	176.2	15.7
<b>BH22-4-05</b>	~22 m west of West Abutment, South Side (EBL Collector)	4848301.2	322111.2	43.774469	-79.284958	176.6	15.8
<b>BH22-4-06</b>	~45 m west of West Abutment, b/w EBL and WBL Express	4848330.0	322080.0	43.774729	-79.285344	176.8	14.2 <sup>2</sup>
<b>BH22-4-07</b>	~45 East Abutment, b/w EBL and WBL Express	4848370.0	322203.5	43.775086	-79.283809	176.2	15.3

Notes:

- 1.0 Terminated due to encountering an obstruction, BH22-4-04B drilled 2.8 m away to avoid this obstruction.
- 2.0 Terminated at refusal ( $N > 100$  blows over 1.5 m interval)

## 5.2 Laboratory Testing

All obtained samples were submitted for natural moisture content testing. Additionally, unit weight, Atterberg limits and grain size analysis (sieve and hydrometer) tests were performed on a minimum of 25% of all obtained soil samples (performed by EXP). Chemical analyses were also 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 results of the laboratory tests are shown in Table 1.3.

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

Borehole No.	Moisture Content	Atterberg Limits	Sieve	Hydrometer	Unit Weight	Corrosivity
<b>BH22-4-02</b>	15	2	4	4	3	1
<b>BH22-4-03</b>	15	4	5	5	3	1
<b>BH22-4-04A</b>	5	2	2	2	1	-
<b>BH22-4-04B</b>	11	1	3	3	5	-
<b>BH22-4-05</b>	14	3	4	4	5	-
<b>BH22-4-06</b>	13	3	5	5	1	-
<b>BH22-4-07</b>	14	3	4	4	1	-

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

## 6.0 Subsurface Conditions

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

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

In general, the subsurface conditions below the roadway/pavement structure encountered within the depths of EXP’s geotechnical investigation consists of layers of cohesionless and cohesive fill followed by native layers of sand and silt/sandy silt/silty sand/silt, silty clay to clayey silt and glacial till (mixture of clayey silt, sand and gravel).

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 of all boreholes. The thickness of the structure ranged between 300 mm and 460 mm.

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

During EXP’s geotechnical investigation, sand and gravel/gravelly sand fill was encountered below the pavement structure (asphalt/concrete) in BH22-4-03, BH22-4-04A, BH22-4-05, BH22-4-06 and BH22-4-07. The approximate elevations of the surface and base of each fill layer, thickness, description and SPT “N” Values encountered in the boreholes are summarized in Table 1.4 below:

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

Borehole	Elevation (m)		Layer Surface Depth (m)	Layer Thickness (m)	Layer Description	SPT “N” Value Range
	Top	Bottom				
EXP (2022)						
BH22-4-03	175.9	175.6	0.5	0.3	Gravelly Sand	N/A <sup>1</sup>
BH22-4-04A	175.8	174.7	0.4	1.1	Gravelly Sand	33
BH22-4-05	176.3	175.8	0.3	0.5	Sand and Gravel	N/A <sup>1</sup>
BH22-4-06	176.5	176.0	0.3	0.5	Sand and Gravel	N/A <sup>1</sup>
BH22-4-07	175.9	175.4	0.3	0.5	Sand and Gravel	N/A <sup>1</sup>

Note:

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

This layer consists of mainly sand and gravel. The material was greyish brown to brown in colour and moist to wet. SPT “N” value obtained within this layer was 33 blows per 300 mm penetration, corresponding to dense in compactness.

Laboratory testing performed on selected samples consisted of moisture content tests. The test results are as follow:

Moisture Content: (EXP)

- 6% to 17%

The results of the moisture content performed by EXP are provided on the record of borehole sheets in Appendix D.

### 6.1.3 Cohesionless Fill: Sand/Sandy Silt/Silty Sand

During EXP’s geotechnical investigation, sand/sandy silt/silty sand fill was encountered below the pavement structure (asphalt/concrete) in BH22-4-02, below the sand and gravel/gravelly sand fill in BH22-4-03, BH22-4-05, BH22-4-06 and BH22-4-07 and below the cohesive fill in BH22-4-04B. The approximate elevations of the surface and base of each fill layer, thickness, description and SPT “N” Values encountered in the boreholes are summarized in Table 1.5 below:

**Table 1.5: Summary of Cohesionless Fill: Sand/Sandy Silt/Silty Sand Layers**

Borehole	Elevation (m)		Layer Surface Depth (m)	Layer Thickness (m)	Layer Description	SPT “N” Value Range
	Top	Bottom				
EXP (2022)						
BH22-4-02	176.3	167.6	0.4	8.7 <sup>1</sup>	Sand/Silty Sand	17 – 49 <sup>2</sup>
BH22-4-03	175.6	175.2	0.8	0.4	Silty Sand	30
	168.8	167.3	7.6	1.5		7
BH22-4-04B	168.3	167.3	7.9	1.2	Sandy Silt	8
BH22-4-05	175.8	167.5	0.8	8.3	Silty Sand/Sand/Sandy Silt	7 – 51
BH22-4-06	176.0	174.5	0.8	1.5	Silty Sand	12
BH22-4-07	175.4	174.9	0.8	0.5	Silty Sand	21

**Notes:**

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

This layer consists of mainly sand and silt with varying amounts of trace to some gravel and trace to some clay. In addition, asphalt and trace organics were encountered within this material. The material was grey to brown in colour and moist to wet. The SPT "N" values within this layer ranged from 7 to 51 blows per 300 mm penetration, corresponding to loose to very dense, but generally compact to dense in compactness. Atterberg limits tests suggest that this layer was non-plastic.

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

**Moisture Content: (EXP)**

- 4% to 17%

**Grain Size Distribution: (EXP)**

- 1% to 3% gravel.
- 30% to 63% sand.
- 27% to 58% silt.
- 9% clay.

**Atterberg Limits: (EXP)**

- Non-plastic

**Unit Weight: (EXP)**

- 21.1 kN/m<sup>3</sup> to 22.9 kN/m<sup>3</sup>



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

#### 6.1.4 Cohesive Fill: Clayey Silt

During EXP's geotechnical investigation, a cohesive fill was encountered below the cohesionless fill layers in boreholes BH22-4-02, BH22-4-03, BH22-4-04A, BH22-4-06 and BH22-4-07. A cohesive fill layer was encountered at the surface in borehole 71-5 during MTO's geotechnical investigation in 1966. The approximate elevations of the surface and base of each fill layer, thickness, description and SPT (N Value) encountered in the boreholes are summarized in Table 1.6 below:

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

Borehole	Elevation (m)		Layer Surface Depth (m)	Layer Thickness (m)	Layer Description	SPT “N” Value Range
	Top	Bottom				
EXP (2022)						
BH22-4-02	172.1	170.6	4.6	1.5	Clayey Silt	13
BH22-4-03	175.2	168.8	1.2	6.4	Clayey Silt	7 – 34
BH22-4-04A	174.7	171.9	1.5	2.8 <sup>1</sup>	Clayey Silt	10 – 13
BH22-4-04B	171.6	168.3	4.6	3.3	Clayey Silt	8 – 12
BH22-4-06	174.5	169.2	2.3	5.3	Clayey Silt	11 – 18
BH22-4-07	174.9	167.8	1.3	7.1	Clayey Silt	7 – 24
MTO (1966)						
71-5	168.2	166.7	0	1.5	Clayey Silt	14

Note:

1.0 End of borehole terminated within cohesive fill layer due to encountering an obstruction.

This layer predominately consists of silt and clay and can be considered sandy with trace to some gravel. The material was grey to brown in colour and slightly moist to wet. The SPT "N" value within this layer ranged between 7 to 34 blows per 300 mm penetration, corresponding to firm to hard, but generally stiff to very stiff in consistency. Atterberg limits tests suggest that this cohesive fill material was low plastic.

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

Moisture Content (EXP and MTO):

- 10% to 23%

Grain Size Distribution: (EXP and MTO)

- 0% to 7% gravel.

- 26% to 42% sand.
- 41% to 52% silt.
- 11% to 26% clay.

Atterberg Limits: (EXP and MTO)

- Liquid Limit: 16% to 24%.
- Plastic Limit: 11% to 13%.
- Plasticity Index: 4% to 12%

Unit Weight: (EXP)

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

The results of the moisture content, grain size distribution, Atterberg limits and unit weight tests performed by EXP are provided on the record of borehole sheets in Appendix D. The results of grain size distribution and Atterberg limits tests are also provided on Figure 2 and 5 in Appendix E. The results of tests performed by MTO are shown on the borehole logs attached in Appendix F.

#### 6.1.5 Sandy Silt/Silt and Sand/Silty Sand/Silt

During EXP's geotechnical investigation, a native sandy silt/silt and sand/silty sand/silt deposit was encountered below fill layers in boreholes BH22-4-02, BH22-4-03, BH22-4-04B, BH22-4-05, BH22-4-06 and BH22-4-07. Sandy silt to silty sand was also encountered at the surface in boreholes 71-1 and 71-4 and below the fill layer in borehole 71-5 during MTO's geotechnical investigation in 1966. 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 Sandy Silt/Silt and Sand/Silty Sand/Silt Layers**

Borehole	Elevation (m)		Layer Surface Depth (m)	Layer Thickness (m)	Layer Description	SPT “N” Value Range
	Top	Bottom				
EXP (2022)						
BH22-4-02	167.6	160.9	9.1	6.7 <sup>1</sup>	Sandy Silt to Sand and Silt	28 – 105
BH22-4-03	167.3	160.6	9.1	6.7 <sup>1</sup>	Sand and Silt/Sandy Silt/Silt	6 – 150
BH22-4-04B	167.1	160.5	9.1	6.6 <sup>1</sup>	Sandy Silt to Sand and Silt	35 – 105
BH22-4-05	167.5	162.9	9.1	4.6	Sandy Silt	71 – 124/230 mm
BH22-4-06	169.2	162.6	7.6	6.6 <sup>1</sup>	Silty Sand/Sandy Silt/Sand and Silt	38 – 157
BH22-4-07	167.8	160.9	8.4	6.9 <sup>1</sup>	Sandy Silt	34 – 170/228 mm
MTO (1966)						
71-1	167.9	158.8	0	9.1	Sandy Silt to Silty Sand	39 – 120

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Borehole	Elevation (m)		Layer Surface Depth (m)	Layer Thickness (m)	Layer Description	SPT “N” Value Range
	Top	Bottom				
<b>71-4</b>	170.4	161.2	0	9.2	Sandy Silt to Silty Sand	12 – 170/200 mm
<b>71-5</b>	166.7	157.8	1.5	8.9	Sandy Silt to Silty Sand	28 – 189

*Note:*

*1.0 The end of borehole terminated within this layer.*

This native layer predominately consists of sand and silt with trace to some gravel, trace to some clay. Occasional clayey silt lenses were also encountered within the sandy silt/sand and silt/silty sand/silt layers. The material was grey to brown in colour and slightly moist to moist. The SPT “N” value within this layer ranged between 6 to 189 blows per 300 mm penetration, corresponding to loose to very dense, but generally dense to very dense in compactness. Atterberg limits tests suggest that this layer was non-plastic to low plastic (in one sample only).

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

Moisture Content (EXP and MTO):

- 5% to 21%

Grain Size Distribution: (EXP and MTO)

- 0% to 10% gravel.
- 8% to 54% sand.
- 32% to 82% silt.
- 2% to 12% clay

Atterberg Limits: (EXP)

- Liquid Limit: 18%.
- Plastic Limit: 10%.
- Plasticity Index: 8%

Five test results indicated non-plastic material.

Unit Weight: (EXP)

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

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

#### 6.1.6 Silty Clay to Clayey Silt

During EXP’s geotechnical investigation, a native silty clay layer was encountered below the cohesionless till in borehole BH22-4-05. Additionally, native clayey silt was encountered below the native sandy silt to silty sand in borehole 71-5 during MTO’s

geotechnical investigation in 1966. 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 1.8 below:

**Table 1.8: Summary of Silty Clay to Clayey Silt Layers**

Borehole	Elevation (m)		Layer Surface Depth (m)	Layer Thickness (m)	Layer Description	SPT “N” Value Range
	Top	Bottom				
EXP (2022)						
BH22-4-05	162.9	160.8	13.7	2.1 <sup>1</sup>	Silty Clay	32 – 44
MTO (1966)						
71-5	157.8	154.8	10.4	3.0	Clayey Silt	100/125 mm – 100/150 mm

Notes:

1.0 End of borehole terminated within this layer.

This layer predominately consists of silt and clay with trace sand to sandy and trace gravel. The material was grey in colour and slightly moist to wet. The SPT "N" values within this layer ranged from 32 to 44 blows per 300 mm penetration and 100 blows per 125 mm to 150 mm penetration, corresponding to hard in consistency. Atterberg limits tests suggest that this native silty clay/clayey silt was low to high plasticity.

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

Moisture Content (EXP and MTO):

- 11% to 29%

Grain Size Distribution: (EXP and MTO)

- 1% gravel.
- 6% to 25% sand.
- 21% to 46% silt.
- 28% to 72% clay.

Atterberg Limits: (EXP and MTO)

- Liquid Limit: 22% to 53%.
- Plastic Limit: 13% to 20%.
- Plasticity Index: 9% to 33%

Unit Weight: (EXP)

- 19.1 kN/m<sup>3</sup>

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

### 6.1.7 Glacial Till: Mixture of Clayey Silt, Sand and Gravel

A glacial till (mixture of clayey silt, sand and gravel) layer was encountered below the native sandy silt to silty sand in boreholes 71-1 and 71-4 and below the native clayey silt in borehole 71-5 during MTO's geotechnical investigation in 1966. 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 1.9 below:

**Table 1.9: Summary of Glacial Till: Mixture of Clayey Silt, Sand and Gravel Layers**

Borehole	Elevation (m)		Layer Surface Depth (m)	Layer Thickness (m)	Layer Description	SPT “N” Value Range
	Top	Bottom				
MTO (1966)						
71-1	158.8	152.7	9.1	6.1 <sup>1</sup>	Clayey Silt	100/75 mm – 100/125 mm
71-4	162.2	156.2	9.1	5.0 <sup>1</sup>	Clayey Silt	100/100 mm – 136/275 mm
71-5	154.8	149.4	13.4	5.4 <sup>1</sup>	Clayey Silt	100/100 mm – 100/150 mm

*Note:*

*1.0 The end of borehole terminated within this layer.*

This layer was described as a heterogenous mixture of clayey silt, sand and gravel. Grain size analyses of this material suggest that this layer predominately consists of sand and silt (varying from some to main fraction) with trace clay to clayey and trace gravel. Refusal SPT "N" values (100+ blows for less than 300 mm penetration) were obtained within this layer corresponding to hard in consistency. Atterberg limits tests suggest that this native clayey silt till layer was low plasticity.

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

Moisture Content (MTO):

- 9% to 14%

Grain Size Distribution: (MTO)

- 0% to 5% gravel.
- 21% to 85% sand.
- 52% to 58% silt.
- 9% to 22% clay.
- 15% silt and clay

Atterberg Limits: (MTO)

- Liquid Limit: 18% to 19%.
- Plastic Limit: 10% to 12%.
- Plasticity Index: 7% to 8%

The results of tests performed by MTO are shown on the borehole logs attached in Appendix F.

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

**Table 1.10: Summary of observed groundwater levels**

Borehole	Ground Surface Elevation (m)	Water level Depth/ Elevation (m)	Date
<b>EXP (2022)</b>			
<b>BH22-4-03</b>	176.4	11.8/164.6	October 25, 2022
<b>BH22-4-04B</b>	176.2	13.3/162.9	November 14, 2022
<b>BH22-4-06</b>	176.8	12.2/164.6 <sup>1</sup>	October 21, 2022
<b>BH22-4-07</b>	176.2	14.1/162.1	October 24, 2022
<b>MTO (1966)</b>			
<b>71-1</b>	167.9	1.7/166.2	April 4, 1966
<b>71-4</b>	170.4	4.3/166.1	April 12, 1966
<b>71-5</b>	168.2	3.7/164.5	December 6, 1966

Note:

1.0 Groundwater level inferred from split spoon observations.

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

## 6.3 Chemical Analyses

Two (2) soil samples were selected for chemical analysis during current investigation. The soils samples collected by EXP were tested at the Bureau Veritas Laboratories (formerly Maxxam Analytics), a CALA-certified and accredited laboratory in Mississauga, Ontario.

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

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

Sample Identification	pH (Unitless)	Soluble Chloride (ppm)	Soluble Sulphate (ppm)	Resistivity (ohm-cm)	Conductivity (umho/cm)	Redox Potential (mV)
BH22-4-02, SS11	7.91	470	77	970	1030	210
BH22-4-03, SS6	7.89	400	<20	1100	895	220

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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 and Thomas Lardner, Ph.D., P.Eng. It was reviewed by TaeChul Kim, M.E.Sc., P.Eng. and Stan E. Gonsalves, M.Eng., P.Eng., Designated MTO Foundation Contact.


Yours truly,


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## Part II: Foundation Design Report

Discussion and Engineering Recommendations for Kennedy Road Overpass (Site 37X-0214/B1 & B3)

## 8.0 Discussion and Recommendations

### 8.1 General

This section of the report provides geotechnical design recommendations on structure foundation, seismic and liquefaction potential, roadway protection systems, structure backfill, abutment settlement, lateral earth pressure for design, construction considerations and corrosion protection for rehabilitation of the proposed partial superstructure replacement of the Highway 401 Eastbound Core and Collectors Markham Road Overpass. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the current investigation at the site and presented in Part I-Foundation Investigation Report. Previous investigations by others, as noted in this report available through GEOCREs were used to aid in assessments. The interpretation and recommendations provided are intended solely to permit designers to assess roadway protection systems alternatives for bridge rehabilitation. Comments on construction are only provided to highlight issues that could affect the design. Contractors bidding on the works should make their own assessments of the factual data and how it might affect construction means and methods, scheduling and the like.

The existing bridge is a 36.74 m long two-span bridge structure. It is understood that the existing abutments and piers are supported on 3.4 m and 1.8 m wide spread footings at elevations ranging from Elevation 167.6 m to 166.7 m (assumed based on Geocres No. 30M14-486). The Highway 401 pavement grade ranges between about Elevation 176.2 m to 176.8 m, while the Kennedy Road pavement grade is at an Elevation of about 170 m at the structure site.

It is understood that, for the proposed rehabilitation of the Kennedy Road Overpass structure, there is no change in loading conditions on the foundation elements associated with the rehabilitation works will be negligible. Initially, widening of 0.5 m on the south side of the existing highway was part of the proposed works, however, widening at this location is now not considered as part of the works. The existing foundations will remain same and based on the contemplated traffic staging plan there will not be any unusual loads on the existing foundations. The rehabilitation program will involve replacement of the existing bridge deck and girders, conversion to semi-integral abutment; reconstruct top of wingwalls/retaining walls and barrier walls; and patch repair to abutment walls, pier structure, and wingwalls/retaining walls. The existing foundations will remain to support the abutments and retaining walls. It is anticipated that this work will require excavations of the embankment fills immediately behind the abutment walls/retaining walls to facilitate the rehabilitation work.

The depth of excavation behind the abutment/retaining wall is expected to be about 5.5 m. Additionally, the GA drawing indicates that an RSS system will be constructed immediately behind the new bridge to mitigate potential stresses on the new structure.

Based on subsoil conditions encountered at the site it is expected that excavation will be carried out through cohesionless (sand/silty sand/sandy silt) and cohesive (clayey silt) fill. Based on an assessment of the water levels observed in the borings and the subsurface conditions, groundwater depth is interpreted to be about 11.8 m to 14.1 m below existing grade with Elevation ranging between 162.1 m to 164.6 m. It should be noted that fluctuations in the level of the groundwater may occur due to seasonal variations, (precipitation, snowmelt, rainfall), local soil permeability, construction remediation activities, and other related factors. A detailed description of the soil and groundwater encountered are discussed in Part I of this report.

This part of the report addresses the geotechnical design of the foundation for the roadway protection system by providing geotechnical design parameters that may be required in accordance with the latest edition of the *Canadian Highway Bridge Design Code (CHBDC, CAN/CSA-S6-19, 2019)*, the *Canadian Foundation Engineering Manual (CFEM, 2023)*, *Guideline for MTO Foundation Engineering Services, Version 03 (April 2022)* and generally accepted good practice. This structure has the potential to significantly affect alternate transportation corridors and is considered to be of "Typical Consequences Level" associated with exceeding Limit States Design (Section 6.5 and Commentary, CHBDC, 2019). A "Typical Degree of Site and Prediction Model Understanding" is considered appropriate based on the level of foundation investigation completed. Pertinent geotechnical

resistance factors and consequence factors have been used in design. The report also addressed other geotechnical and construction considerations such as excavation, groundwater and surface water control and lateral earth pressure on structures.

## 8.2 Structure Foundations

Based on the GA drawing, it is understood that if additional foundations were constructed, the abutments would utilize shallow spread footings founded on compact to very dense sand and silt/sandy silt/silty sand for the abutments and centre piers. For completeness, several foundation options for support of abutments and piers were analyzed for this report, including micropiles and driven H-Pile foundations.

### 8.2.1 Shallow Foundations Options

#### 8.2.1.1 Geotechnical Resistance for Structure Foundations

Based on the current GA drawing no foundation remediation is proposed. The existing spread footings are estimated to be 3.4 m and 1.8 m in width for the abutment and pier, respectively. Since no information of the spread footings for the wingwalls are available, it is assumed that the spread footing width is 4.3 m for the existing wingwall (based on the north wingwalls in Geocres No. 30M14-486). Table 2.1 summarizes the evaluation of geotechnical resistances for the existing foundations. Although no additional loading is expected and foundation remediation or expansion is not anticipated, Table 2.2 provides recommended values to be used in the case of foundation extension. Given the soil conditions, it is expected that any extension would be found at the same elevation to avoid impacting the existing foundations. SLS values have been selected assuming a lower permissible settlement as it is assumed the existing structure has experienced settlement.

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 typical consequence factor of 1.0 at ULS and SLS; typical degree of understanding (factor of 0.5 at ULS) and typical degree of understanding (factor of 0.8 at SLS) in accordance with Table 6.1 and 6.2 of the CHBDC S6-19.

**Table 2.1: Evaluation of existing foundation geotechnical resistances**

Location	Founding Soil Type	Footing Width (m)	Estimated Founding Elevation (m)	Factored Geotechnical Resistance at ULS (kPa)	Factored Serviceability Geotechnical Resistance (for 25 mm settlement) (kPa)
<b>West Abutment</b> (71-4, BH22-4-02)	Compact to very dense sandy silt to silty sand	3.4	~167.6	975	550
<b>West Wingwall</b> (71-4, BH22-4-05)	Compact to very dense sandy silt to silty sand	4.3	~167.5	975	520
<b>Pier</b> (71-1)	Dense to very dense sandy silt to silty sand	1.8	~167.6	875	600

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Location	Founding Soil Type	Footing Width (m)	Estimated Founding Elevation (m)	Factored Geotechnical Resistance at ULS (kPa)	Factored Serviceability Geotechnical Resistance (for 25 mm settlement) (kPa)
<b>East Abutment</b> (71-5, BH22-4-03)	Compact to very dense sand and silt/sandy silt/silty sand	3.4	~167.6	975	550
<b>East Wingwall</b> (71-5, BH22-4-04A/04B)	Compact to very dense sandy silt to silty sand	4.3	~167.5	975	520

**Table 2.2: Recommended shallow foundation design parameters**

Location	Founding Soil Type	Footing Width (m)	Estimated Founding Elevation (m)	Factored Geotechnical Resistance at ULS (kPa)	Factored Serviceability Geotechnical Resistance (for 25 mm settlement) (kPa)
<b>West Abutment</b> (71-4, BH22-4-02)	Compact to very dense sandy silt to silty sand	5.0	~167.6	1075	500
<b>East Abutment</b> (71-5, BH22-4-03)	Compact to very dense sand and silt/sandy silt/silty sand	5.0	~167.6	1075	500

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

Wingwalls are proposed to be constructed on the embankment material behind the west and east abutment and RSS structure behind the updated abutment walls. Based on the proposed construction, the geotechnical resistances for a structure founded on the existing fill material and on an engineered granular pad are tabulated below. Per the GA drawing, the top 1 m of the wingwalls and retaining walls are to be removed and rebuilt, therefore, it is assumed that there will be no additional loading on the existing wingwall foundation.

**Table 2.3: Recommended shallow foundation design parameters for Wingwall and RSS wall**

Location	Founding Elevation <sup>1</sup> (m)	Footing Width (m)	Founding Soil Type	Factored Geotechnical Resistance at ULS (kPa)	Factored Serviceability Geotechnical Resistance (for 25 mm settlement) (kPa)
East and West Abutment	~171.7 to 171.2	>1.0	Stiff to hard clayey silt fill/compact to dense silty sand to sandy silt fill	280	150
East and West Abutment	~171.2 to 170.7	>1.0	Engineered Granular Pad compacted to 98% of SPMDD over existing fill	420	225

Note:

- (1) below frost line or minimum embedment requirements set in MTO RSS Design Guidelines, MTO Engineering Standard Branch, September 2008.

#### 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/CSA S6-19, using the following parameters:

**Table 2.4: 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	Coefficient of friction ( $\tan \delta$ )=0.45
Between pre-cast concrete and engineered fill	Coefficient of friction ( $\tan \delta$ )=0.4

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

#### 8.2.1.4 Frost Protection

Ontario Provincial Standard Drawing (OPSD) 3090.101 indicates that the frost penetration for the Scarborough area is 1.2 m. Therefore, all foundation elements should be provided with a minimum of 1.2 m of earth cover for frost protection.

## 8.2.2 Deep Foundation Options

### 8.2.2.1 General

Soil conditions at the abutments indicate that very dense silty sand/sand and silt/sandy silt (SPT 'N' values of greater than 100 blows per 300 mm) was encountered at a minimum Elevation of 161.5 m at the west abutment and at Elevation 164.0 m at the east abutment, respectively. As noted previously in Section 6.2, the top of groundwater is interpreted to be about 11.8 m to 14.1 m below existing grade with Elevation ranging between 162.1 m to 164.6 m.

Should there be a requirement for resistance to increased loading, micropiles may be considered as an alternative to increase the geotechnical resistance while minimizing the footprint of the required works. If a larger magnitude of forces are expected, driven piles may be considered. The bridge can be supported on driven Steel H-piles or steel pipe piles or drilled caissons founded in the very dense silty sand/sand and silt/sandy silt/silt. Deep foundation options provide greater control of settlements over shallow foundations, if the tie-in between the potential structure and existing structure is sensitive to differential settlements.

### 8.2.2.2 Micropiles

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

#### 8.2.2.2.1 Geotechnical Axial Resistance

Micropiles may be found in the very dense silty sand/sand and silt/sandy silt/silt stratum. Micropiles should have a minimum bond length of 3.0 m and a minimum diameter of 150 mm. Recommended values for grout-to-soil adhesion are provided in Table 2.5. The geotechnical capacities provided were factored with typical consequence factor of 1.0 at ULS and SLS; typical degree of understanding (factor of 0.5 at ULS) and typical degree of understanding (factor of 0.8 at SLS) in accordance with Table 6.1 and 6.2 of the CHBDC S6-19.

**Table 2.5: Summary of micropile adhesion design values**

Foundation Unit	Relevant Borehole	FHWA Type B Micropile <sup>1</sup>		FHWA Type C Micropile <sup>1</sup>		Bond Zone Stratum
		Ultimate Adhesion (kPa)	Factored Ultimate Adhesion (kPa)	Ultimate Adhesion (kPa)	Factored Ultimate Adhesion (kPa)	
West Abutment	71-4, BH22-4-02	130	55	145	65	Very dense sandy silt to silty sand
Pier	71-1	130	55	145	65	Very dense sandy silt to silty sand

Foundation Unit	Relevant Borehole	FHWA Type B Micropile <sup>1</sup>		FHWA Type C Micropile <sup>1</sup>		Bond Zone Stratum
		Ultimate Adhesion (kPa)	Factored Ultimate Adhesion (kPa)	Ultimate Adhesion (kPa)	Factored Ultimate Adhesion (kPa)	
East Abutment	71-5, BH22-4-03	130	55	145	65	Very dense sand and silt/sandy silt/silty sand

**Note:**

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

#### 8.2.2.2.2 Verification and proof testing

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

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

#### 8.2.2.2.3 Lateral resistance

Lateral resistance of micropiles is derived through casing design. To ensure adequate depth for the generation of lateral geotechnical resistance, the cased length should be approximately 20 times the diameter and may be refined through analysis. Geotechnical lateral resistance input values are provided in Table 2.8.

#### 8.2.2.3 Driven Piles

The proposed work is unlikely to include foundation extensions and is limited to the rehabilitation of the current structure. For the purpose of making this report comprehensive, the following details regarding short driven piles are included. It is assumed that the underside of the pile cap would be at the same elevation as the bottom of footing for the shallow foundation option.

Should there be a requirement for resistance to increased loading, driven piles may be considered. The bridge can be supported on Steel H-piles or steel pipe piles driven to or into the very dense silty sand/sand and silt/sandy silt/silt.

Steel H-piles have advantages as they can be driven into a relatively strong (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.

#### 8.2.2.3.1 Geotechnical Axial Resistance

Based on the subsurface conditions encountered at this site, the design parameters given in Table 2.6 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 typical consequence factor of 1.0 at ULS and SLS; typical degree of understanding (factor of 0.5 at ULS) and typical degree of understanding (factor of 0.8 at SLS) in accordance with Table 6.1 and 6.2 of the CHBDC S6-19.

**Table 2.6: Summary of recommended deep foundations**

Foundation Unit	Relevant Borehole	Estimated Tip Elevation	Approximate Design Pile Length <sup>1</sup> (m)	Factored Axial Geotechnical Resistance at ULS (kN/pile) <sup>2</sup>  HP310x110	Factored Serviceability Geotechnical Axial Resistance (kN/pile) <sup>2,3</sup>  HP310x110	Pile Founding Stratum
<b>West Abutment</b>	71-4, BH22- 4-02	163.8	3.8	1400	1200	Very dense sandy silt to silty sand
<b>Pier</b>	71-1	161.0	6.5	1400	1200	Very dense sandy silt to silty sand
<b>East Abutment</b>	71-5, BH22- 4-03	160.5	7.1	1400	1200	Very dense sandy silt/silt/silty sand

Notes:

(1) based on an assumed bottom of pile cap a minimum of 1.2 below frost ground surface at Kennedy Rd. (~El. 170 m).

(2) values as per MTO structural office policy memo 98-01, 1998

(3) for 25 mm total settlement.

(4) Piles at west abutment should be driven below Elevation 165.5 m. Piles at Pier and East Abutment should be driven below Elevation 163.5 m.

Closed-end, concrete filled, 325 mm diameter, 9.5 mm (+) wall thickness steel pile 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. Given this issue, closed-end concrete filled piles are not recommended for this application.

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 unfirmly graded sand (Ottawa type sand). The gradation for the uniformly graded sand shall be as provided in Table 2.7.



**Table 2.7: 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 2.7 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 (see Appendix G) of 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.

#### 8.2.2.3.2 Resistance to Lateral Loads

In integral abutments the resistance to the 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:

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

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)

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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). The following Table 2.8 present 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 the L-pile software or equivalent.

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.

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

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

Reduction factors for leading piles in a line

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

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

Reduction factors for trailing piles in a line

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

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

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 2.8: 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 <sub>h</sub> (MN/m <sup>3</sup> )	K <sub>p</sub>
								Static	Cyclic			
Granular Fill	-	Cohesionless	-	21.0	-	30	14	10.0	10.0	-	6.6	3.0
<b>West Abutment – 71-4, BH22-4-02</b>												
Sandy silt/sand and silt/silty sand (compact to very dense)	167.6 – 163.8	Cohesionless	12 - >50	22.0	-	33	12	40.0	40.0	-	12.5	3.4
<b>Centre Pier – 71-1</b>												
Sandy silt to silty sand (dense to very dense)	167.9 – 163.8	Cohesionless	39 - >50	22.0	-	33	12	40.0	40.0	-	12.5	3.4
<b>East Abutment – 71-5, BH22-4-03</b>												
Sandy silt/sand and silt/silty sand/silt (loose to very dense)	167.6 – 163.8	Cohesionless	6 - >50	22.0	-	33	12	40.0	40.0	-	12.5	3.4

#### 8.2.2.3.3 Downdrag

The amount of relative settlement between soil and 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. The 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. 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.
2. 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.
3. 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%.

#### 8.2.2.3.4 Pile Installation

Piles should be installed in accordance with OPSS.PROV 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 soils and an example of NSSP is included in Appendix I. 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 Analyser must be performed near the beginning of the pile driving phase of construction to confirm the pile capacities. Ten

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percent of the piles, but no fewer than three 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 three (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/silty soil 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 lines drawn down from the toe of the previously driven higher piles, the higher piles should be redriven 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 redriven 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 two (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 factored 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 2.6 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 hours 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.PROV 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.

#### 8.2.2.4 Caissons

Given the proposed remediation works and site constraints, caissons are deemed impractical for this project.

### 8.3 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 compact to very dense sandy silt/silt and sand/silty sand/silt, hard silty clay/clayey silt and hard clayey silt till. Bedrock was not encountered within the investigated depth. The groundwater level is at about 5.4 m to 7.9 m depth below the existing Kennedy Road grade. The reported N-values for the soil below the founding level ranged from 6 to over 157 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 (43.774789°N, 79.284548°W), where the damped spectral accelerations are  $Sa(0.2)=0.325g$ ,  $Sa(0.5)=0.198g$ ,  $Sa(1.0)=0.104g$ ,  $Sa(2.0)=0.048g$  the site-adjusted peak ground acceleration (PGA) is  $0.176g$  ( $g$  = acceleration due to gravity  $-9.81 \text{ m/s}^2$ ). These values are associated with an earthquake having 2 percent probability of exceedance in a 50-year period (1 in 2475-year event) for Site Class C as shown on the GSC seismic hazard calculation data sheet for this site attached in Appendix H.

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

### 8.4 Roadway Protection System

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

To safely support the excavation walls and minimize the impact to existing utilities in the embankment (if any), temporary shoring consisting of driven steel sheet piling or Soldier H-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 2.9.

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 and native deposits. Difficulties with installation may occur where occasional cobbles and boulders are encountered in the fill (i.e., an obstruction was encountered in BH22-4-04A during this investigation which may suggest the presence of cobbles and boulders), requiring their removal before further driving or fitted with a driving shoe. It is recommended that an NSSP be included in the Contract Documents to warn the Contractor of the possible presence of cobbles and/or boulders within the overburden soils or native very dense deposits and an example of NSSP is included in Appendix I. Alternatively, an H-pile with lagging wall can be used as a vertical temporary shoring system. The H-piles are installed, and lagging is inserted between installed H-piles during excavation. Space between the excavation and lagging must be suitably backfilled and drained. Lagging wall material can be selected as wood (timber), steel or concrete.

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

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

Timber lagging may be sized as per Table 20.12 of the CFEM, 5th edition (Section 20.8.9). This is provided so the center-to-center spacing of the soldier piles does not exceed 3.0 m. This is provided so the center-to-center spacing of the soldier piles does not exceed 2.5 m. Soldier piles should extend a minimum depth of 3.0 m below the planned excavation depth (~8.5 m below the roadway). The actual depth of embedment should be determined by balancing moments about the pile tip. Excavation can proceed following installation of the soldier piles. The unshored height of the excavation should not exceed 1.2 m at any given time. No excavation height should remain unshored for more than 24 hours. Any loose zones from behind the shoring should be prevented during installation of the protection system. If required, backfill Granular A should be placed and compacted behind the shoring wall.

For the relatively shallow depth of excavation anticipated, cantilevered systems may be adequate. However, depending on the actual excavation depth, embedment depth (i.e., an embedded depth of sheet piles can be approximately 2.0 to 2.5 times of its exposed height), and shoring system used, additional anchorage or tiebacks may be required. This must be confirmed by the shoring designer. Conventional practice is to incorporate either buried deadman anchors, rakers or grouted soil anchors. Deadman anchors can be designed based on the earth pressure coefficients and soil parameters provided in Section 8.3.1 following. For this project, either continuous or individual concrete block anchors would likely be appropriate. The anchor resistance is provided by a combination of the dead weight and passive resistance. For the full passive resistance to be realized with no load transfer to the wall, the anchor needs to be fully beyond the active wedge acting on the wall. Pressure grouted soil

anchors can be designed in a preliminary fashion in accordance with Section 20 of the CFEM (2023). Based on the generally stiff to very stiff clayey silt fill and compact to dense sand/sandy silt/silty sand fill at this site, 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 Table 2.9, the Soldier H-Pile and Lagging is ranked as more practical for this project due to possible obstructions that may be present within the fill layer. Design and construction specifications for the chosen roadway protection system should be prepared in accordance with OPSS. PROV 539. Piling should be in accordance with OPSS. PROV 903. Cantilevered walls should be designed for the earth pressure coefficients presented in Section 8.4.1 of this report and earth pressure diagram shown in CFEM Figure 20.14. Besides design and construction of the temporary protection system, the Contractor is also responsible for its materials, maintenance, monitoring and removal. According to OPSS.PROV 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.

#### 8.4.1 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 2.10 can be used (assuming wall friction is neglected, the back wall is vertical, and the ground surface is horizontal both on the retained side as well as in front of the toe):



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

Abutment	Elevation	Material	Unfactored Friction Angle $\phi'$ (°)	Coefficient of Lateral Earth Pressure <sup>(1)</sup>			Unit Weight $\gamma$ (kN/m <sup>3</sup> )	GWL (m)
				(K <sub>a</sub> )	(K <sub>p</sub> )	(K <sub>o</sub> )		
West	176.3 to 174.5	Sand/sand and gravel/silty sand (compact to dense)	32	0.31	3.26	0.47	21	164.6
	174.5 to 169.1	Clayey silt fill (stiff to very stiff) <sup>(2)</sup>	30	0.33	3.00	0.50	19.5	
	169.1 to 162.9	Sandy silt/silty sand/sand and silt (compact to very dense)	33	0.30	3.39	0.46	22	
	162.9 to 160.8	Silty clay (hard) <sup>(2)</sup>	30	0.33	3.00	0.50	19	
	160.8 to 156.2	Clayey silt till (hard) <sup>(2)</sup>	32	0.31	3.26	0.47	21	
East	175.9 to 175.2	Gravelly sand/sand and gravel/silty sand fill (compact to dense)	32	0.31	3.26	0.47	21	162.1 to 164.6
	175.2 to 167.8	Clayey silt fill (firm to very stiff) <sup>(2)</sup>	30	0.33	3.00	0.50	19.5	
	167.8 to 167.1	Sandy silt to silty sand fill (loose)	30	0.33	3.00	0.50	20	
	167.1 to 160.5	Sand and silt/sandy silt/silt/sand and silt (compact to very dense)	33	0.30	3.39	0.46	21.5	
	160.5 to 149.4	Clayey silt till (hard) <sup>(2)</sup>	32	0.31	3.26	0.47	21	

Notes:

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

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

## 8.5 Structure Backfill

The selection and placing of backfill should be in accordance with OPSS.PROV 902, OPSS.3101.150 and OPSS.3190.100. 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.

## 8.6 Abutment Settlement

As per the GA drawing, no highway widening or grade raise is proposed. Therefore, no settlement of the abutment or existing fill is expected as long as the highway geometry remains unchanged. Additionally, the new superstructure load is not expected to be greater than 10% of existing conditions.

If any additional loadings conditions (grade raise or widening) are proposed, the existing foundation system may require further assessment on whether it can sustain the additional loads.

The proposed fill and RSS structure is expected to experience some settlement. It is estimated that the fill itself will compress by about 0.5 to 1 percent of the fill height under its self-weight. Depending on material type and assuming placement as per MTO practices. More granular material fills would compress less and over a shorter time period, typically within the period of embankment construction. 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 4 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.

Concerning widening, the post- construction settlement criteria for embankment widening is stipulated in MTO's "Embankment Settlement Criteria for Design"; the maximum settlement limits during pavement design life of the widened embankment are 50 mm of the total settlement and 200:1 of the differential settlement rate. The differential settlement rate is applicable to both new widened embankment and, also, the differential settlement rate between the existing and the new embankment. The settlement across the widened embankment shall transition uniformly from the widening point (existing highway embankment rounding) to the new embankment rounding such that surface drainage is not impeded. The maximum settlement at structure/ embankment interface during pavement design life should be 25 mm for distance of 0 - 20 m from transition point.

## 8.7 Lateral Earth Pressures for Design

### 8.7.1 Lateral Earth Pressures for Static Design

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

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

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

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

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

**Table 2.11: Material types and unfactored earth pressure properties under static conditions**

Material	Unfactored Friction Angle $\phi'$ (°)	Coefficient of Active Earth Pressure ( $K_a$ )	Coefficient of Passive Earth Pressure ( $K_p$ )	Coefficient of Earth Pressure at Rest ( $K_o$ )	Unit Weight $\gamma$ (kN/m <sup>3</sup> )
Compacted Granular A or Granular B Type II	35	0.27	3.69	0.43	22.8
Compacted Granular B Type I	32	0.31	3.25	0.47	21
Engineered Earth Fill	30	0.33	3.00	0.50	21

The coefficients of lateral earth pressure above are provided for level backfill behind the wall (perpendicular to the wall face plane) and should be adjusted in the case of sloping backfill. For a 2 horizontal to 1 vertical (2H:1V) slope, the active earth pressure coefficients provided above should be adjusted by a factor of 1.5. The given values of active earth pressure coefficients depend on angles of friction and inclination. For preliminary design purposes, the adjustment for slopes between horizontal and 2H:1V may be linearly proportioned, however, some modification of the design pressures may be required depending on the backfill type and geometry.

## 8.7.2 Lateral Earth Pressures for Seismic Design

### 8.7.2.1 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.

When calculating seismic lateral earth pressures on walls that are capable of moving 25 to 50 mm using the M-O formulation, the seismic horizontal acceleration coefficient ( $k_h$ ) should be taken as half of the site-adjusted PGA, where, the site-adjusted PGA estimated at ground surface is given as  $F(PGA) \cdot PGA$ , where,  $F(PGA)$  is the PGA-based amplification factor that corresponds to the applicable Site Class as defined in Table 4.8 of the Code. For this site, a site-adjusted PGA of 0.176 g (Site Class C), earthquake having a 2% probability of exceedance in 50 years (1 in 2,475-year return period) can be used in the calculation of the seismic active pressure coefficient.  $k_h$  is estimated to be 0.090 g and was used for lateral earth pressures for seismic design.

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

It should be noted that in the computation of seismic earth pressure coefficients, the wall back-face geometry, backfill slope, and wall friction effects need to be addressed.

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

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

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

### 8.7.2.2 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  $F(PGA)*PGA$ . The same values for  $F(PGA)$  and  $PGA$  are used from Section 2.4.3.6.2.2. The acceleration coefficient determined at the original ground surface should be the acceleration coefficient acting at the wall base. The seismic vertical acceleration coefficient ( $k_v$ ) can be ignored when calculation the seismic lateral earth pressure coefficient. For design purposes, the following unfactored seismic lateral earth pressure parameters for non-yielding walls are provided in Table 2.13.

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

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

## 8.8 Construction Considerations

### 8.8.1 Excavation

Based on the GA drawing and correspondence with AECOM, the proposed depth of excavation is about 5.5 m below the roadway (Elevation 171.2 m to 170.7) for the RSS.

All excavations must be carried out in accordance with the latest edition of the Ontario Occupational Health and Safety Act (OHSA) and good construction practice. The existing fills which should be excavated for the rehabilitation of the Kennedy

Overpass structure (i.e., uncontrolled fill) are considered Type 3 soils above the groundwater table and Type 4 soils below the groundwater table. Temporary excavations (i.e., those that are open only for a short period) above the groundwater table may be made with side slopes not steeper than about 1H:1V, while the temporary slopes below the groundwater table have to be formed at 3H:1V unless a suitable dewatering system is installed to lower the water level below the base of the excavation. Excavation for structures should be in accordance with OPSS.PROV 902 and SP109S12. The excavation should not undermine the existing walls.

### 8.8.2 Groundwater and Surface Water Control

As mentioned in Section 6.2, based on an assessment of the water levels observed in the borings and the subsurface conditions, the groundwater levels were interpreted to be 11.8 m to 14.1 m below existing grade of Highway 401 with Elevation ranging between 162.1 m to 164.6 m across the Kennedy Road Overpass structure. Water may also be perched in the fill at higher levels during wet periods.

Based on the rehabilitation works planned at these sites, an excavation is planned to extend to about Elevation 171.2 m to 170.7 which would be above the groundwater level (~Elevation 162.1 m to 164.6 m)

However, if any rehabilitation works required within abutment stems, the possible excavation limits could be extended below the groundwater levels at this site. As such, the groundwater level needs to be controlled 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. This dewatering can likely be achieved by gravity drainage and pumping from strategically placed sumps with side ditches. Confirmation of control should be verified before general excavation to final levels.

Surface water should always be directed away from the excavation area(s). Dewatering/unwatering shall be carried out in accordance with OPSS.PROV 517 and SP517F01. It is the responsibility of the Contractor to propose a suitable dewatering system based on the time of construction, water levels, and flow conditions. The method used should not undermine the existing utilities/ structures (if any). Alternatively, and in accordance with SP 517F01, the dewatering systems may be completed by a design Engineer and design-checking Engineer with a minimum of 5 years' experience.

## 8.9 Corrosion Protection

Two (2) soil samples were selected for chemical analysis during current investigation. 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. No new infrastructures are planned at this site. However, for completeness, the analyses results have been discussed here. The analyses results are summarized in Table 1.11.

The pH, resistivity, and chloride concentration provide an indication of the degree of corrosiveness of the sub-surface environment. In general, the soil pH values measured at the site ranged between 7.89 to 7.91, which are within the normal range of soil pH of 5.5 to 8.5 and it is not considered to be detrimental to the structure's durability (AASHTO, 2000/MTO Gravity Pipe Design Guidelines, April 2014). The chemical data indicates very low (<2000 ohm-cm) resistivity of tested soil, which suggests the severe potential for corrosion of buried metallic elements as per Table 3.2 of the MTO Gravity Pipe Design. Therefore, some level of corrosion protection for buried metallic elements is required, depending upon the material type. The test results provided in Table 1.11 may be used to aid in the selection of coatings and corrosion protection systems for buried steel objects. The measured chloride content was between 400 ppm ( $\mu\text{g/g}$ ) to 470 ppm which also indicates a very low potential for additional corrosion (Molinas and Mommandi, 2009).

Based on the results of the sample tested and given that the structure is located adjacent to the roadway and will expose to de-icing salt, consideration should be given by the designer to designing concrete for a « C » type of exposure class as defined by CSA A23.1 Table 1.

The maximum water-soluble sulphate content of the soils tested is less than 77 ppm ( $\mu\text{g/g}$ ), i.e., 0.0077% and being less than 0.10%, does not require sulphate resistant cement as per CSA A23.1 Table 3 “Additional requirements for concrete subjected to sulphate attack”.

### 8.10 Obstructions

Cobbles and boulders (i.e., an obstruction was encountered in BH22-4-04A during this investigation which may suggest the presence of cobbles and boulders) were noted to be contained within the embankment fill, therefore care has to be taken (i.e., pile flange reinforcement or be fitted with a driving shoe) during the installation of elements of temporary protection systems or may also impact excavations. It is recommended that an NSSP be included in the Contract Documents to warn the Contractor of the possible presence of cobbles and/or boulders within the overburden soil. An example of an NSSP is included in Appendix I.

### 8.11 Geotechnical Instrumentation and Monitoring

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

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

#### 8.11.1 Precondition and postcondition surveys

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

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

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

### 8.11.2 Movements of Existing Structure

Survey points should be used to monitor movements of the existing overpass structure. 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).
- 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.

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

The limits are as follows:

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



*Foundation Investigation and Design Report  
Highway 401 Eastbound from Victoria Park Avenue to Neilson Road  
Superstructure Replacement at Kennedy Road Overpass  
Eastbound Core and Collectors Structure (Site 37X-0214/B1 & B3)  
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 and Thomas Lardner, Ph.D., P.Eng. It was reviewed by TaeChul Kim, M.E.Sc., P.Eng. and Stan E. Gonsalves, M.Eng., P.Eng., Designated MTO Foundation Contact.


Yours truly,


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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**

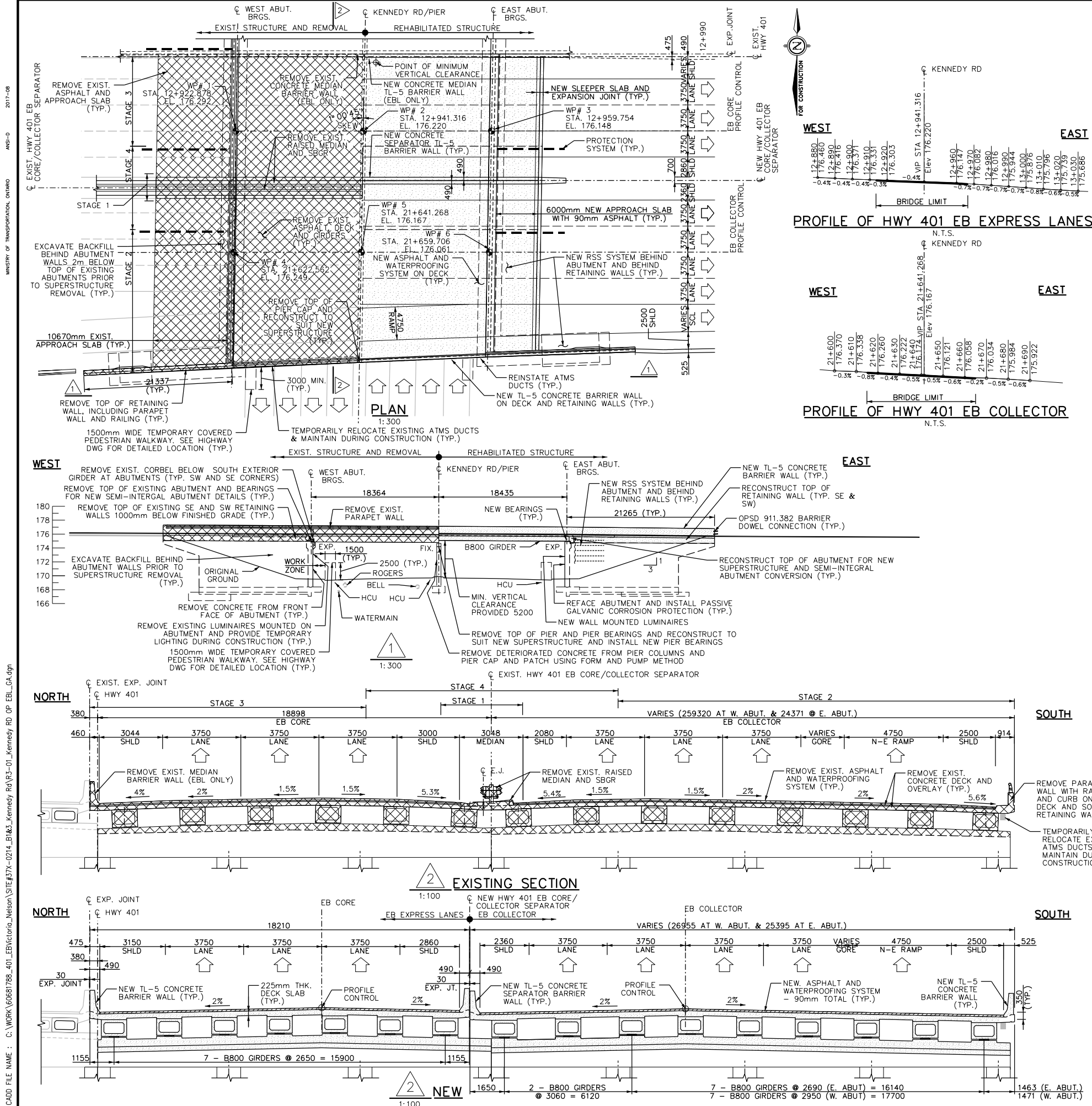
Where exp has submitted both electronic file and a hard copy of the Report, or any document forming part of the Report, only the signed and sealed hard copy shall be the original documents for record and working purposes. In the event of a dispute or discrepancy, the hard copy shall govern. Electronic 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.

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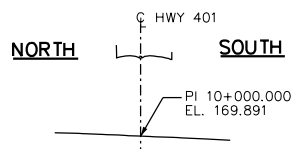
*Foundation Investigation and Design Report  
Highway 401 Eastbound from Victoria Park Avenue to Neilson Road  
Superstructure Replacement at Kennedy Road Overpass  
Eastbound Core and Collectors Structure (Site 37X-0216/B1 & B3)  
Assignment No. 2021-E-0018  
Date: September 19, 2024*

## Appendix B – General Arrangement Drawings

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



### PROFILE OF KENNEDY ROAD

### LIST OF DRAWINGS:

- R3-1. GENERAL ARRANGEMENT
- R3-2. CONSTRUCTION STAGING I
- R3-3. CONSTRUCTION STAGING II
- R3-4. REMOVALS I
- R3-5. REMOVALS II
- R3-6. ABUTMENT REHABILITATION
- R3-7. PIER REHABILITATION
- R3-8. RETAINING WALL REHABILITATION AND DETAILS
- R3-9. PRESTRESSED BOX GIRDER LAYOUT
- R3-10. PRESTRESSED BOX GIRDERS AND BEARINGS I
- R3-11. PRESTRESSED BOX GIRDERS AND BEARINGS II
- R3-12. DECK LAYOUT
- R3-13. DECK DETAILS AND REINFORCEMENT I
- R3-14. DECK DETAILS AND REINFORCEMENT II
- R3-15. EXPANSION JOINT AND SLEEPER SLAB
- R3-16. STRIP SEAL EXPANSION JOINT TYPE 'C' DETAILS
- R3-17. SEQUENCE OF EXPANSION JOINT INSTALLATION
- R3-18. NORTH AND SOUTH BARRIER WALL WITHOUT RAILING TL-5
- R3-19. E.B. SEPARATOR BARRIER WALL WITHOUT RAILING TL-5
- R3-20. 6000mm APPROACH SLAB
- R3-21. DECK DRAIN DETAILS
- R3-22. MISCELLANEOUS DETAILS
- R3-23. ELECTRICAL EMBEDDED WORK

### APPLICABLE STANDARD DRAWINGS:

- 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 3390.150 FALSEWORK CLEARANCE TO TRAFFIC LANES
- OPSD 3941.200 FIGURES IN CONCRETE SITE NUMBER AND DATE LAYOUT

### 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
- EXIST. 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

**Ontario** **Ministry of Transportation**

**CONT**  
**GWP**  
HWY 401 EB CORE & COLLECTOR  
KENNEDY RD O/P BRIDGE REHAB.  
GENERAL ARRANGEMENT

**SHEET**  
**S52**

### GENERAL NOTES:

CLASS OF CONCRETE:  
SPECIFIED 28-DAY COMPRESSIVE STRENGTH 30MPa  
UNLESS NOTED OTHERWISE  
SPECIFIED 28-DAY COMPRESSIVE STRENGTH FOR PRECAST GIRDERS ARE GIVEN ON PRESTRESSED GIRDER DRAWINGS.

CLEAR COVER:  
DECK TOP 70 ±20mm  
BOTTOM 40 ±10mm  
REMAINDER UNLESS OTHERWISE NOTED 70 ±20mm

### REINFORCING STEEL:

- REINFORCING STEEL SHALL BE GRADE 500W UNLESS OTHERWISE SPECIFIED.
- STAINLESS REINFORCING STEEL SHALL BE TYPE 316LN OR DUPLEX 2205 AND HAVE A MINIMUM YIELD STRENGTH OF 500MPa, UNLESS OTHERWISE SPECIFIED.
- BAR MARKS WITH PREFIX 'S' DENOTE STAINLESS STEEL BARS.
- UNLESS SHOWN OTHERWISE, TENSION LAP SPLICES SHALL BE CLASS 'B'.
- BAR HOOKS SHALL HAVE STANDARD HOOK DIMENSIONS USING MINIMUM BEND DIAMETERS, WHILE STIRRUPS AND TIES SHALL HAVE MINIMUM HOOK DIMENSIONS. ALL HOOKS SHALL BE IN ACCORDANCE WITH THE STRUCTURAL STANDARD DRAWING SS12-1, UNLESS INDICATED OTHERWISE.

### CONSTRUCTION NOTES:

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

REVISIONS	DATE	BY	DESCRIPTION
DESIGN J.C.	CHK U.P.	CODE CAN/CSA 56-19	LOAD CL 625-ONT
DRAWN V.A.	CHK J.C.	SITE 37X-0214/B1&B3	DWG R3-01

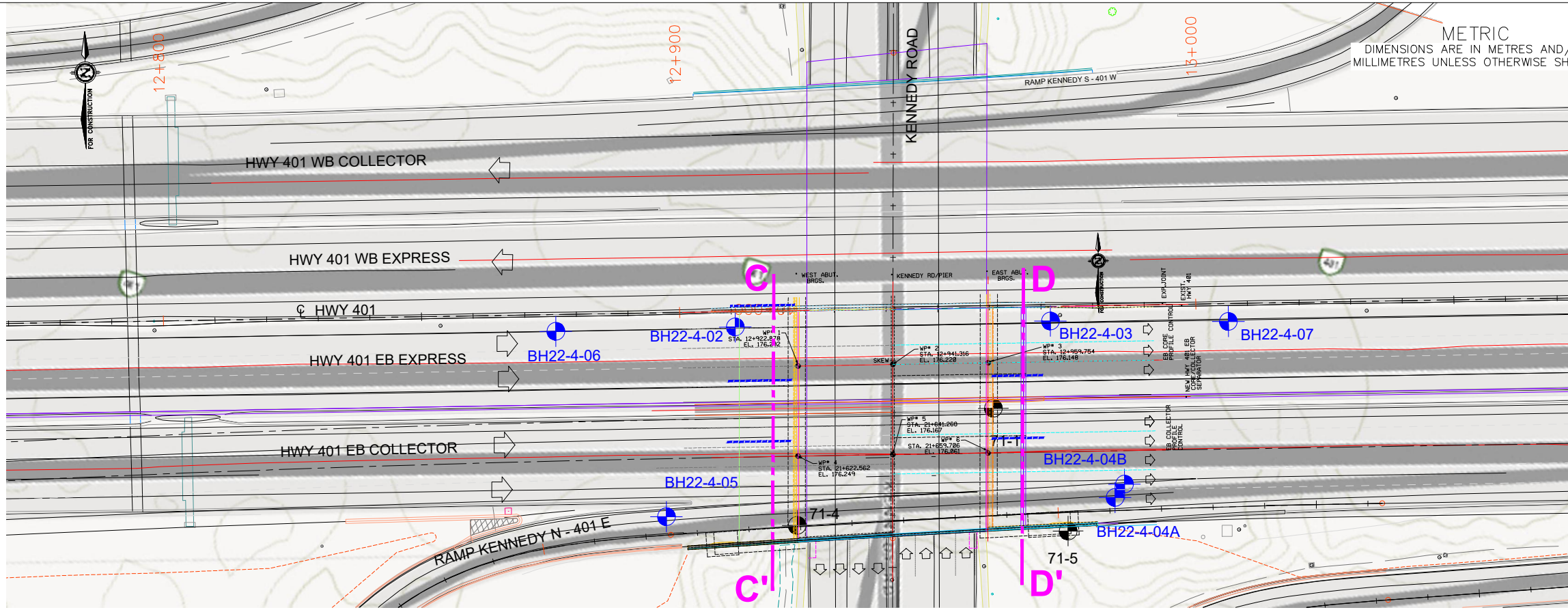
## Appendix C – Borehole Location Plan and Stratigraphic Profile



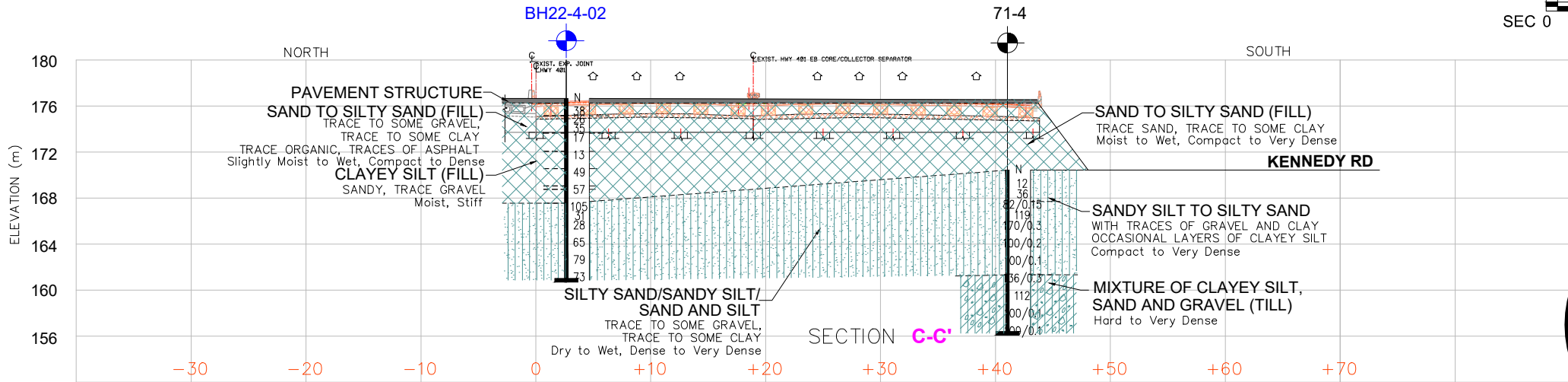
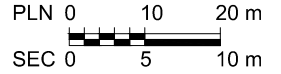




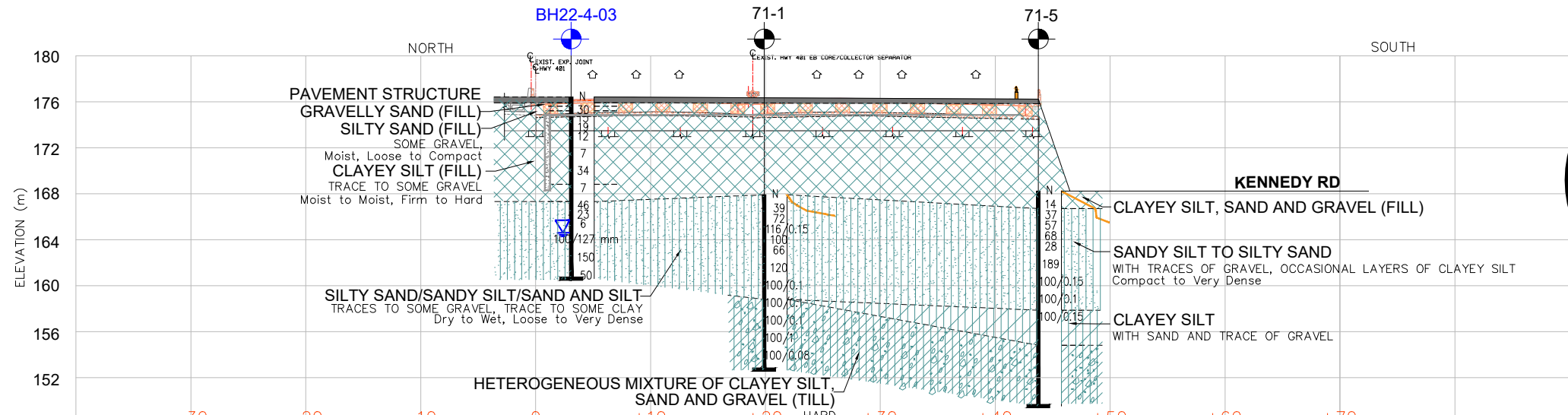
FILE NAME: I:\2003-Brompton\Proposals\Projects\International\Hwy 401 & Victoria Park Av. to Nelson\working drawings\Structure 4 - Kennedy Rd Overpass\_borehole location plan & soil strata.dwg  
MODIFIED: 2024-11-14 14:23



PLAN



SECTION C-C'



SECTION D-D'

CONT No.  
ASSIG No. 2021-E-0018  
GWP No.  
Superstructure Replacement at Kennedy Road Overpass  
Eastbound Core and Collectors Structure  
Latitude: 43.774789°; Longitude: -79.284548°  
BOREHOLE LOCATION PLAN & SOIL STRATA  
SHEET 2

exp. EXP SERVICES INC.



KEY PLAN  
N.T.S.

LEGEND

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

SOIL STRATA SYMBOLS

- PAVEMENT STRUCTURE
- FILL
- CLAYEY SILT
- SILTY SAND/SANDY SILT/SAND AND SILT
- MIXTURE OF CLAYEY SILT, SAND AND GRAVEL (TILL)

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

BH No.	ELEV.	NORTHING	EASTING
BH22-4-02	176.7	4848340.9	322113.1
BH22-4-03	176.4	4848359.5	322170.8
BH22-4-04A	176.2	4848331.9	322193.1
BH22-4-04B	176.2	4848333.9	322194.2
BH22-4-05	176.6	4848301.2	322111.2
BH22-4-06	176.8	4848330.0	322080.0
BH22-4-07	176.2	4848370.0	322203.5
71-1	167.9	4848340.5	322165.4
71-4	170.4	4848307.8	322135.7
71-5	168.2	4848322.1	322186.2

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	GEOCREs No.	30M14-552
SUBM'D SH	CHKD. SM	DATE	NOV. 14, 2024
DRAWN SH	CHKD. TC	APPRD SG	SITE 37X-0214/B1 & B3
		DWG	02



EXP Services Inc.

*Foundation Investigation and Design Report  
Highway 401 Eastbound from Victoria Park Avenue to Neilson Road  
Superstructure Replacement at Kennedy Road Overpass  
Eastbound Core and Collectors Structure (Site 37X-0216/B1 & B3)  
Assignment No. 2021-E-0018  
Date: September 19, 2024*

## 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.

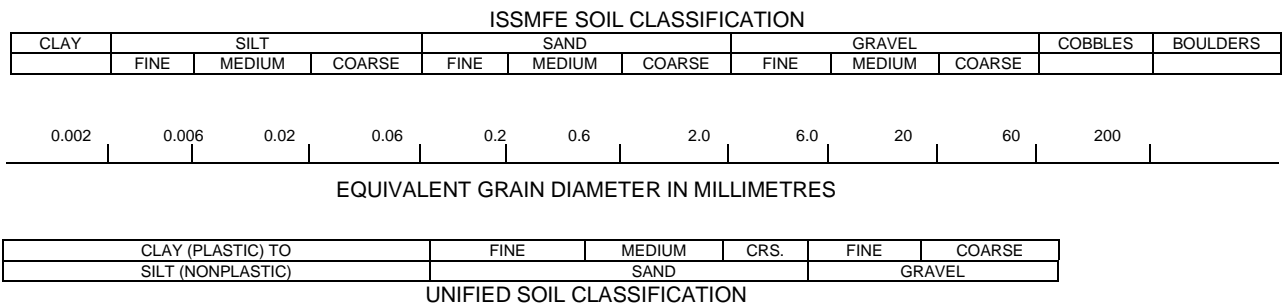
*Seam:* a thin, confined layer of soil having different particle size, texture, or color from materials above and below.

*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:

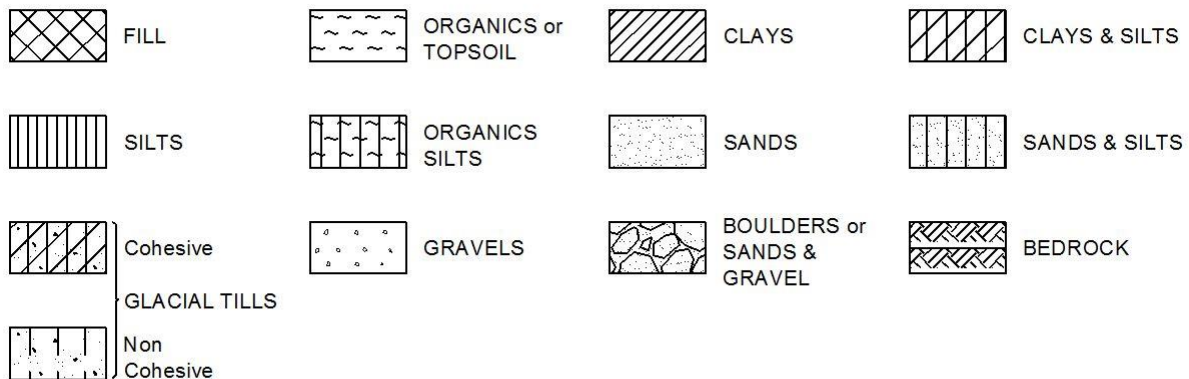
Table c: Consistency of Cohesive Soil

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 BH22-4-02

1 OF 1

METRIC

W.P. Site 37X-0214/B1&B3 LOCATION Hwy 401 - Kennedy Road O/P, Toronto, ON, MTM ON-10 322113.1E 4848340.9N ORIGINATED BY OD  
 DIST Toronto HWY 401 BOREHOLE TYPE Truck Mount CME 75 / SSA COMPILED BY OD  
 DATUM Geodetic DATE 2022.10.26 - 2022.10.26 LATITUDE 43.774826 LONGITUDE -79.284933 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/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100	W <sub>P</sub>	W	W <sub>L</sub>		GR	SA	SI	CL
176.7																				
0.0	PAVEMENT STRUCTURE - 150 mm of asphalt, and 250 mm of concrete																			
176.3																				
0.4	SAND (FILL) - some gravel, grey to brown, slightly moist, dense		AS1	AS																
			SS2	SS	38															
175.2																				
1.5	SILTY SAND (FILL) - trace gravel, trace clay, grey, moist to wet, compact to dense		SS3	SS	26															
	- Traces of asphalt were encountered at a depth of 2.3 m		SS4	SS	35															
173.7																				
3.0	SAND (FILL) - some silt, trace to some clay, grey, moist, compact		SS5	SS	17															
172.1																				
4.6	CLAYEY SILT (FILL) - sandy, trace gravel, grey, slightly moist to moist, stiff		SS6	SS	13															
170.6																				
6.1	SILTY SAND (FILL) - some gravel, trace clay, trace organic, grey, slightly moist, dense		SS7	SS	49															
168.8	-clayey silt lens																			
7.9	SAND (FILL) - trace gravel, some silt, some clay, grey to brown, slightly moist, very dense		SS8	SS	57															
167.6																				
9.1	SANDY SILT - trace gravel, trace to some clay, grey to brown, slightly moist, compact to very dense		SS9	SS	105															
			SS10	SS	31															
			SS11	SS	28															
			SS12	SS	65															
162.7																				
14.0	SAND AND SILT - trace gravel, trace to some clay, grey, slightly moist to wet, very dense		SS13	SS	79															
161.5																				
15.2	SANDY SILT - trace gravel, trace to some clay, grey to brown, slightly moist, very dense		SS14	SS	73															
160.9																				
15.8	END OF BOREHOLE																			
	NOTES: 1) No groundwater was encountered in open borehole upon completion of drilling.																			

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

ONTARIO MTO H401 - KENNEDY RD OVERPASS-090122023 GPJ ONTARIO MTO GDT 7/24/23



Brampton, Ontario

## RECORD OF BOREHOLE No BH22-4-03

1 OF 1

METRIC

W.P. Site 37X-0214/B1&B3 LOCATION Hwy 401 - Kennedy Road O/P, Toronto, ON, MTM ON-10 322170.9E 4848359.5N ORIGINATED BY OD  
 DIST Toronto HWY 401 BOREHOLE TYPE Truck Mount CME 75 / SSA COMPILED BY OD  
 DATUM Geodetic DATE 2022.10.25 - 2022.10.25 LATITUDE 43.774992 LONGITUDE -79.284215 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/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL P. PENETROMETER		W <sub>P</sub>	W	W <sub>L</sub>	WATER CONTENT (%)		GR		SA	SI	CL		
176.4								20	40	60	80	100									
0.0	PAVEMENT STRUCTURE - 150 mm of asphalt, and 310 mm of concrete		AS1	AS																	
175.9																					
175.6	GRAVELLY SAND (FILL) - brown, wet		SS2	SS	30																
175.2	- The sample is wet from coring process																				
1.2	SILTY SAND (FILL) - some gravel, brown, moist, compact		SS3	SS	15																
	CLAYEY SILT (FILL) - sandy, trace to some gravel, brown to grey, slightly moist to moist, firm to hard		SS4	SS	19																
	- Becomes grey at a depth of 1.5 m																				
	- Thin layer of sand were encountered between depths of 2.3 m and 2.7 m		SS5	SS	12																
			SS6	SS	7																
			SS7	SS	34																
168.8																					
7.6	SILTY SAND (FILL) - some gravel, greyish brown, moist, loose		SS8	SS	7																
	- Thin layer of clayey silt were encountered below 8.0 m.																				
167.3																					
9.1	SAND AND SILT - trace to some gravel, trace to some clay, brown, moist, compact to dense		SS9	SS	46																
			SS10	SS	23																
165.7	- Becomes grey at a depth of 10.3 m																				
10.7	SANDY SILT - trace gravel, some clay, greyish brown, moist, loose		SS11	SS	6																
164.2																					
12.2	SILT - trace gravel, some sand, trace to some clay, grey, moist to wet, dense to very dense		SS12	SS	100/127 mm																
			SS13	SS	150																
			SS14	SS	50																
160.6																					
15.8	END OF BOREHOLE																				
	NOTES: 1) Groundwater level was encountered at a depth of 11.8 m upon completion of drilling																				

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

ONTARIO MTO H401 - KENNEDY RD OVERPASS-090122023.GPJ ONTARIO MTO.GDT 7/24/23

Brampton, Ontario

## RECORD OF BOREHOLE No BH22-4-04A

1 OF 1

METRIC

W.P. Site 37X-0214/B1&B3 LOCATION Hwy 401 - Kennedy Road O/P, Toronto, ON, MTM ON-10 322193.1E 4848331.9N ORIGINATED BY OD  
 DIST Toronto HWY 401 BOREHOLE TYPE Truck Mount CME 75 / SSA COMPILED BY OD  
 DATUM Geodetic DATE 2022.10.31 - 2022.10.31 LATITUDE 43.774743 LONGITUDE -79.283939 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 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL P. PENETROMETER					WATER CONTENT (%)							GR	SA	SI	CL	
176.2	<b>PAVEMENT STRUCTURE</b> - 200 mm of asphalt, and 150 mm of concrete  <b>GRAVELLY SAND (FILL)</b> - brown to greyish brown, moist, dense - Becomes dark grey to black at a depth of 1.3 m <b>CLAYEY SILT (FILL)</b> - trace gravel, grey, moist, stiff						176									23.2	1	28	48	23		
0.0																						
175.8																						
0.4				AS1	AS																	
				SS2	SS			33														
174.7																						
1.5			SS3	SS	13		175															
			SS4	SS	12		174															
			SS5	SS	10		173															
171.9	<b>END OF BOREHOLE</b>						172															
4.3	NOTES: 1) Borehole terminated at 4.3 m due to encountering an obstruction at this depth. 2) A companion borehole (BH22-4-4B) was drilled 2.8 m northeast BH22-4-4A. 3) No groundwater was encountered in open borehole upon completion of drilling.																					

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

Brampton, Ontario

## RECORD OF BOREHOLE No BH22-4-04B

1 OF 1

## METRIC

W.P. Site 37X-0214/B1&B3 LOCATION Hwy 401 - Kennedy Road O/P, Toronto, ON, MTM ON-10 322194.2E 4848333.9N ORIGINATED BY OD  
 DIST Toronto HWY 401 BOREHOLE TYPE Truck Mount CME 75 / SSA COMPILED BY OD  
 DATUM Geodetic DATE 2022.11.14 - 2022.11.14 LATITUDE 43.7747611 LONGITUDE -79.283925 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								WATER CONTENT (%)		
								○ UNCONFINED + FIELD VANE	○ QUICK TRIAXIAL	● P. PENETROMETER								
176.2	<b>PAVEMENT STRUCTURE</b> - 200 mm of asphalt, and 150 mm of concrete - Continuation of BH 22-4-4A - Auger drilling to a depth of 4.6 m																	
0.0																		
175.8																		
0.4																		
171.6	<b>CLAYEY SILT (FILL)</b> - trace gravel, brown to grey, moist, stiff  - Becomes grey at a depth of 6.1 m		SS6	SS	12									23.2	3 36 42 19			
4.6																		
168.3	<b>SANDY SILT (FILL)</b> - some gravel, trace clay, brown to grey, moist, loose		SS7	SS	12									19.5				
7.9																		
167.1	<b>SANDY SILT</b> - trace gravel, trace to some clay, brown to grey, moist, dense to very dense  - Becomes grey at a depth of 10.7 m		SS8	SS	8													
9.1																		
164.0	<b>SAND AND SILT</b> - trace gravel, trace clay, grey, moist to wet, very dense		SS9	SS	35									22.3	1 31 59 9			
12.2																		
			SS10	SS	54									23.1				
			SS11	SS	49													
			SS12	SS	54													
			SS13	SS	60										0 42 53 6			

Brampton, Ontario

## RECORD OF BOREHOLE No BH22-4-05

1 OF 1

METRIC

W.P. Site 37X-0214/B1&B3 LOCATION Hwy 401 - Kennedy Road O/P, Toronto, ON, MTM ON-10 322111.2E 4848301.2N ORIGINATED BY OD  
 DIST Toronto HWY 401 BOREHOLE TYPE Truck Mount CME 75 / SSA COMPILED BY OD  
 DATUM Geodetic DATE 2022.11.27 - 2022.11.27 LATITUDE 43.774469 LONGITUDE -79.284958 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 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL P. PENETROMETER		W <sub>P</sub>	W	W <sub>L</sub>			WATER CONTENT (%)		
176.6								20	40	60	80	100					
176.9	PAVEMENT STRUCTURE - 40 mm of asphalt, and 260 mm of concrete		AS1	AS													
0.3																	
175.8	SAND AND GRAVEL (FILL) - brown, wet		SS2	SS	51												
175.8	- The sample is wet from coring process																
1.1	SILTY SAND (FILL) - grey to brown, moist to wet, very dense		SS3	SS	29												
174.2	SAND (FILL) - trace gravel, grey to brown, slightly moist, compact to very dense		SS4	SS	12												
2.4	SANDY SILT (FILL) - trace gravel, trace to some clay, grey, slightly moist to wet, loose to dense		SS5	SS	14												
			SS6	SS	7												
			SS7	SS	16												
			SS8	SS	36												

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

ONTARIO MTO H401 - KENNEDY RD OVERPASS-090122023.GPJ ONTARIO MTO.GDT 7/24/23

Brampton, Ontario

## RECORD OF BOREHOLE No BH22-4-06

1 OF 1

METRIC

W.P. Site 37X-0214/B1&B3 LOCATION Hwy 401 - Kennedy Road O/P, Toronto, ON, MTM ON-10 322080.0E 4848330.0N ORIGINATED BY OD  
 DIST Toronto HWY 401 BOREHOLE TYPE Truck Mount CME 75 / SSA COMPILED BY OD  
 DATUM Geodetic DATE 2022.10.21 - 2022.10.21 LATITUDE 43.774729 LONGITUDE -79.285344 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 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL P. PENETROMETER									
176.8								20	40	60	80	100					
176.0	PAVEMENT STRUCTURE - 90 mm of asphalt and 230 mm of concrete		AS1	AS			176										
176.0	SAND AND GRAVEL (FILL) - brown, wet - The sample is wet from coring process		SS2	SS	12		175										
174.5	SILTY SAND (FILL) - some gravel, brown, moist, compact		SS3	SS	12		174										
2.3	CLAYEY SILT (FILL) - trace gravel, brown to grey, moist, stiff to very stiff		SS4	SS	11		173										
			SS5	SS	13		172										
			SS6	SS	18		171										
			SS7	SS	17		170										
169.2							169										
7.6	SILTY SAND - trace to some gravel, trace to some clay, greyish brown to brown, dry to moist, dense to very dense		SS8	SS	38		168										
			SS9	SS	63		167										
167.7	SANDY SILT - some clay, brown to grey, moist, very dense		SS10	SS	81		166										
							165										
166.1	SAND AND SILT - trace clay, grey, moist to wet, very dense		SS11	SS	130		164										
10.7							163										
	- Becomes wet at a depth of 12.2 m		SS12	SS	138												
162.6			SS13	SS	157												
14.2	END OF BOREHOLE																
	NOTES: 1) No groundwater measured upon completion of open borehole due to cave in. 2) Groundwater level inferred to be 12.2 m based on wet split spoon sampling during drilling.																

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

ONTARIO MTO H401 - KENNEDY RD OVERPASS-090122023.GPJ ONTARIO MTO.GDT 7/24/23

Brampton, Ontario

## RECORD OF BOREHOLE No BH22-4-07

1 OF 1

METRIC

W.P. Site 37X-0214/B1&B3 LOCATION Hwy 401 - Kennedy Road O/P, Toronto, ON, MTM ON-10 322203.5E 4848370.0N ORIGINATED BY OD  
 DIST Toronto HWY 401 BOREHOLE TYPE Truck Mount CME 75 / SSA COMPILED BY OD  
 DATUM Geodetic DATE 2022.10.24 - 2022.10.24 LATITUDE 43.775086 LONGITUDE -79.283809 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	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60	W <sub>P</sub> W W <sub>L</sub>	WATER CONTENT (%)	GR SA SI CL		
176.2														
175.9	PAVEMENT STRUCTURE - 100 mm of asphalt and 200 mm of concrete		AS1	AS										
175.4	SAND AND GRAVEL (FILL) - brown, wet		SS2	SS	21									
174.9	- The sample is wet from coring process		SS3	SS	12									
174.3	SILTY SAND (FILL) - some gravel, some clay, trace organic, brown to grey, moist, compact		SS4	SS	7									
173.8	CLAYEY SILT (FILL) - trace to some gravel, brown to grey, moist to wet, firm to very stiff		SS5	SS	18									
173.3	- Black inclusions were encountered at a depth of 1.5 m													
172.8			SS6	SS	24									
172.3	- Asphalt inclusions were encountered at a depth of 5.0 m													
171.8			SS7	SS	20									
171.3	- Organic materials were encountered at a depth of 6.1 m													
170.8			SS8	SS	11									
170.3														
169.8			SS9	SS	34									
169.3			SS10	SS	36									
168.8														
168.3			SS11	SS	54									
167.8			SS12	SS	100/100 mm									
167.3														
166.8			SS13	SS	170/228 mm									
166.3	-clayey silt seam													
165.8			SS14	SS	100/76 mm									
165.3														
164.8														
164.3														
163.8														
163.3														
162.8														
162.3														
161.8														
161.3														
160.9	END OF BOREHOLE													
15.3	NOTES: 1) Groundwater level was encountered at a depth of 14.1 m upon completion of drilling.													

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

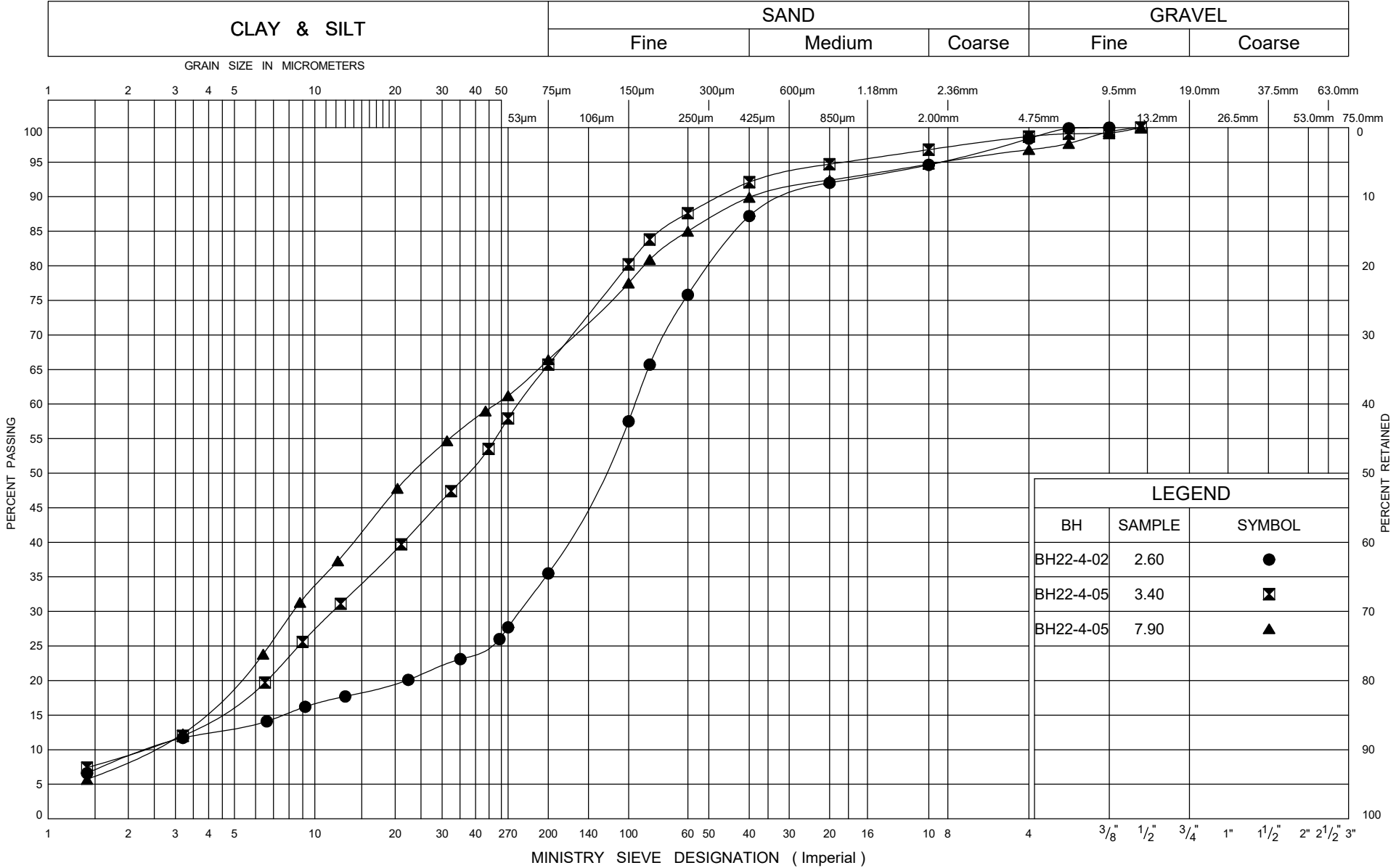
ONTARIO MTO H401 - KENNEDY RD OVERPASS-090122023.GPJ ONTARIO MTO.GDT 7/24/23

EXP Services Inc.

*Foundation Investigation and Design Report  
Highway 401 Eastbound from Victoria Park Avenue to Neilson Road  
Superstructure Replacement at Kennedy Road Overpass  
Eastbound Core and Collectors Structure (Site 37X-0216/B1 & B3)  
Assignment No. 2021-E-0018  
Date: September 19, 2024*

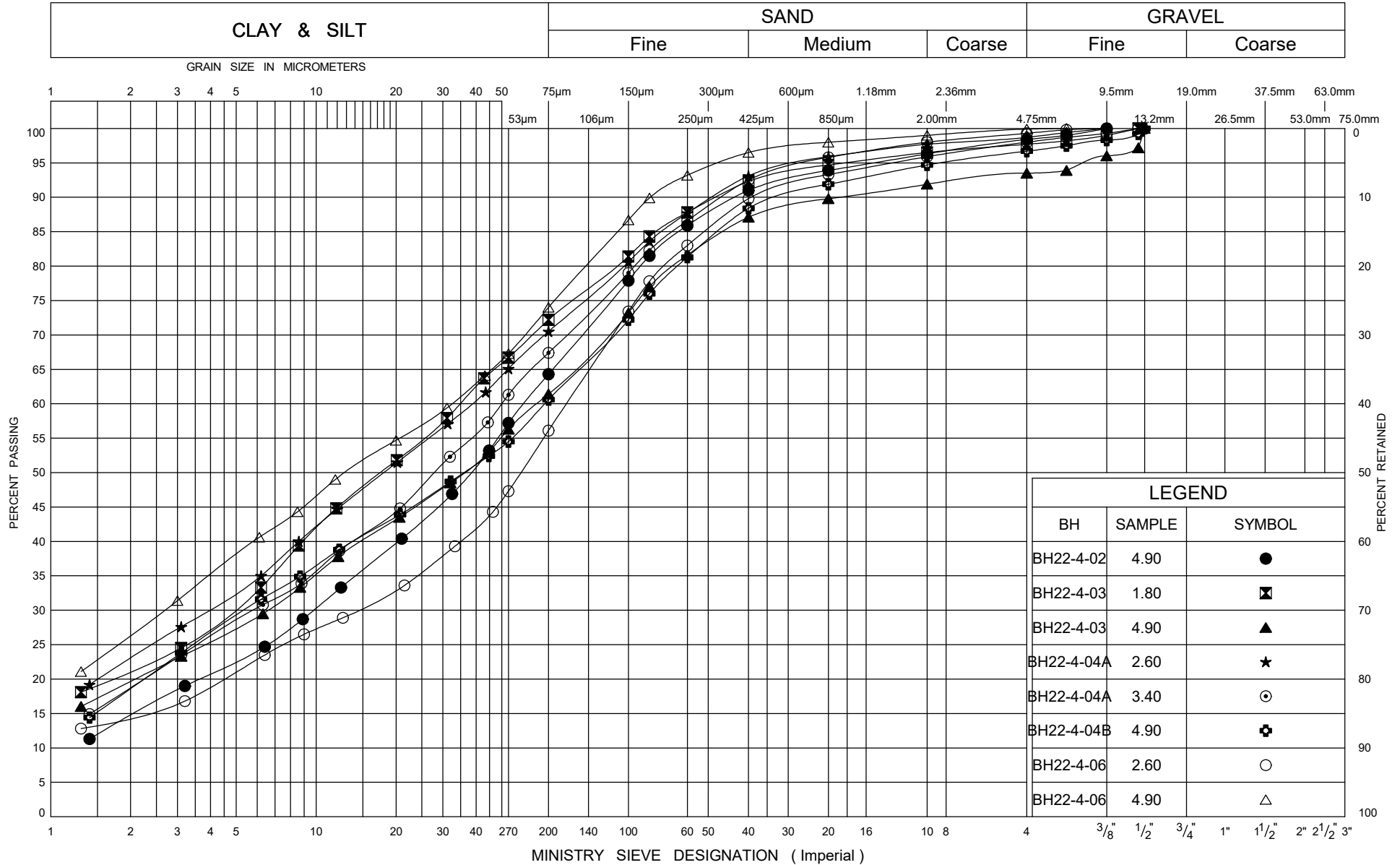
## Appendix E – Laboratory Data

UNIFIED SOIL CLASSIFICATION SYSTEM





# UNIFIED SOIL CLASSIFICATION SYSTEM



Ministry of  
Transportation

## GRAIN SIZE DISTRIBUTION

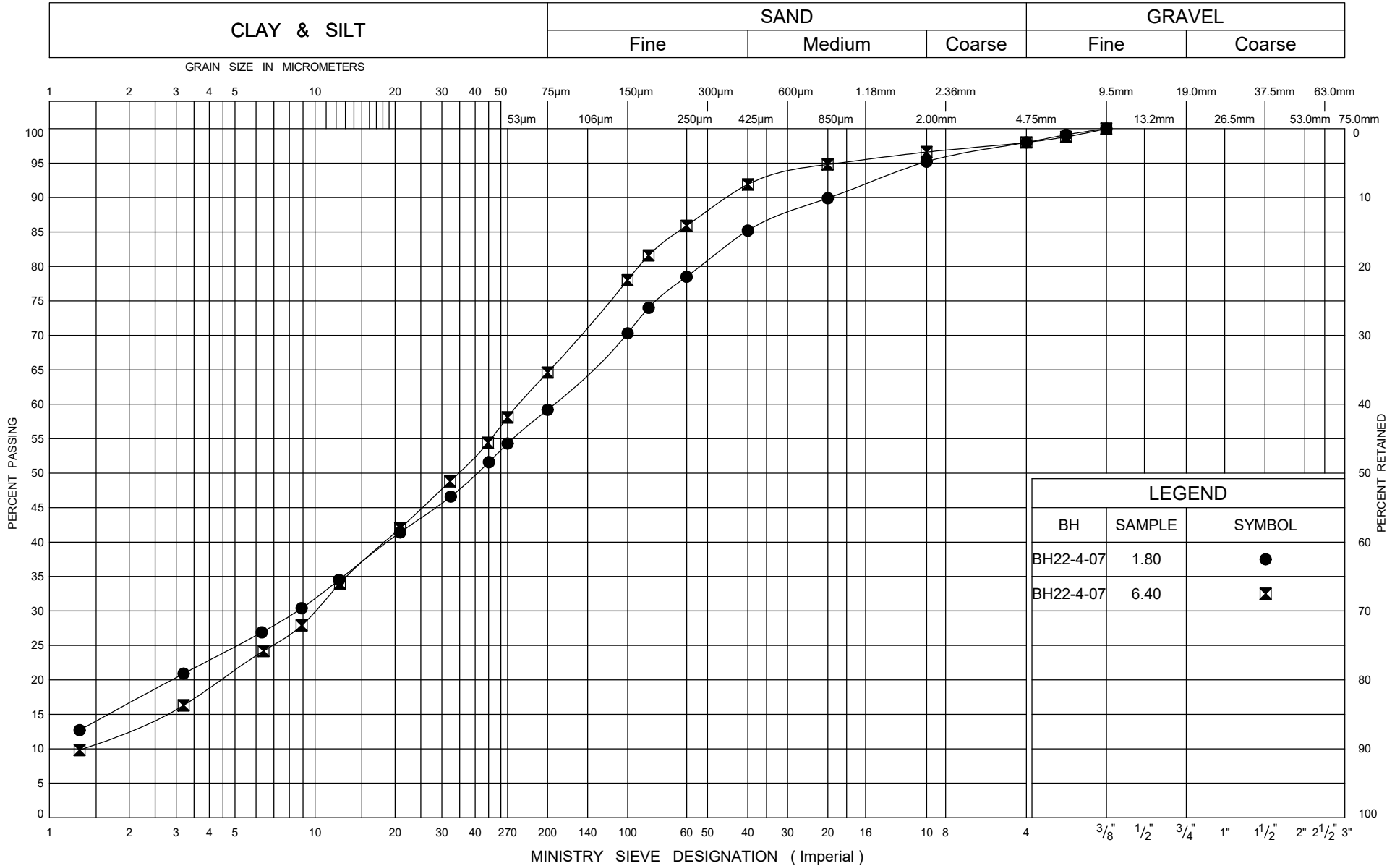
Cohesive Fill

FIG No 2a

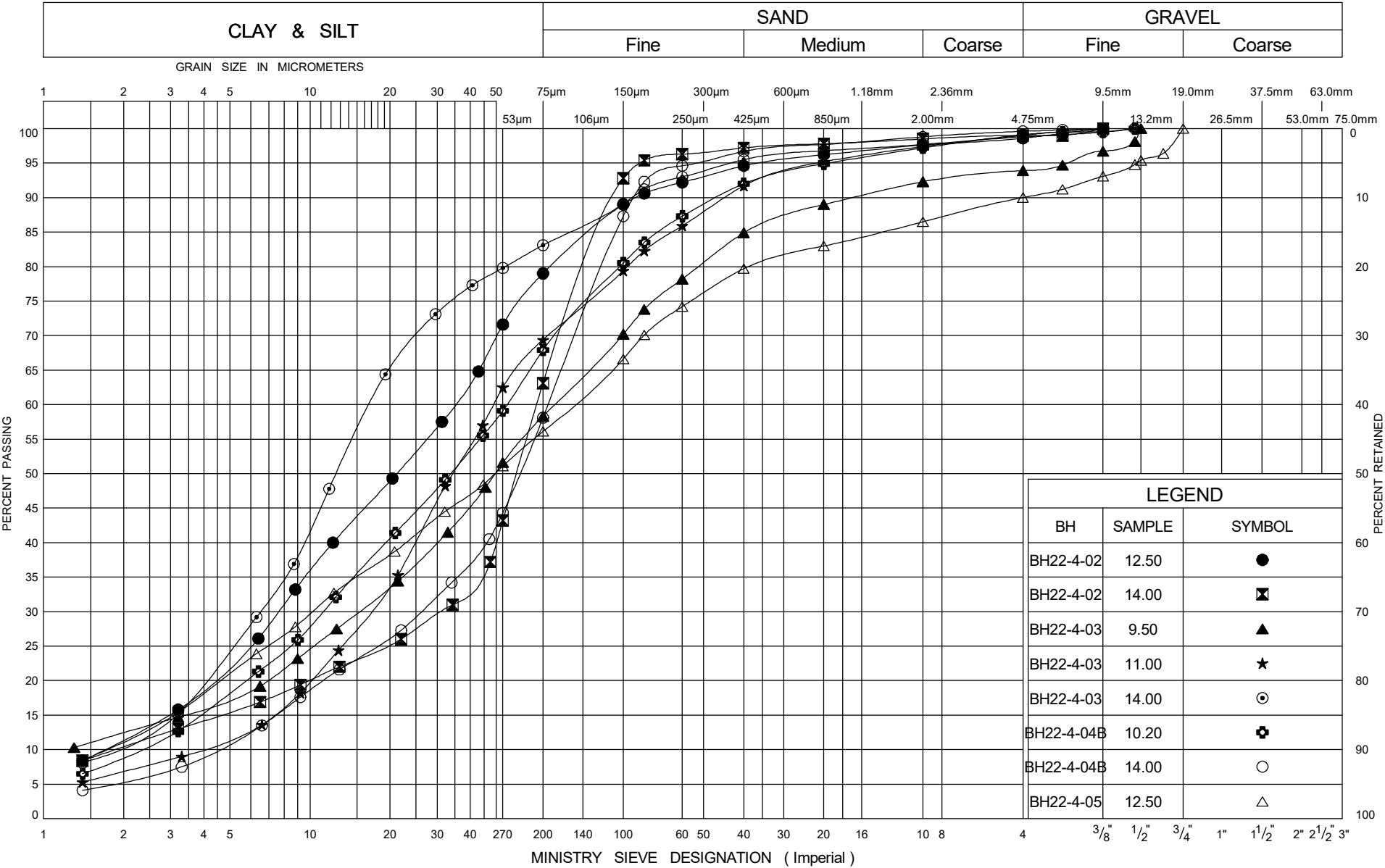
W P Site 37X-0214/B1&B3

Hwy 401 - Kennedy Road O/P

# UNIFIED SOIL CLASSIFICATION SYSTEM



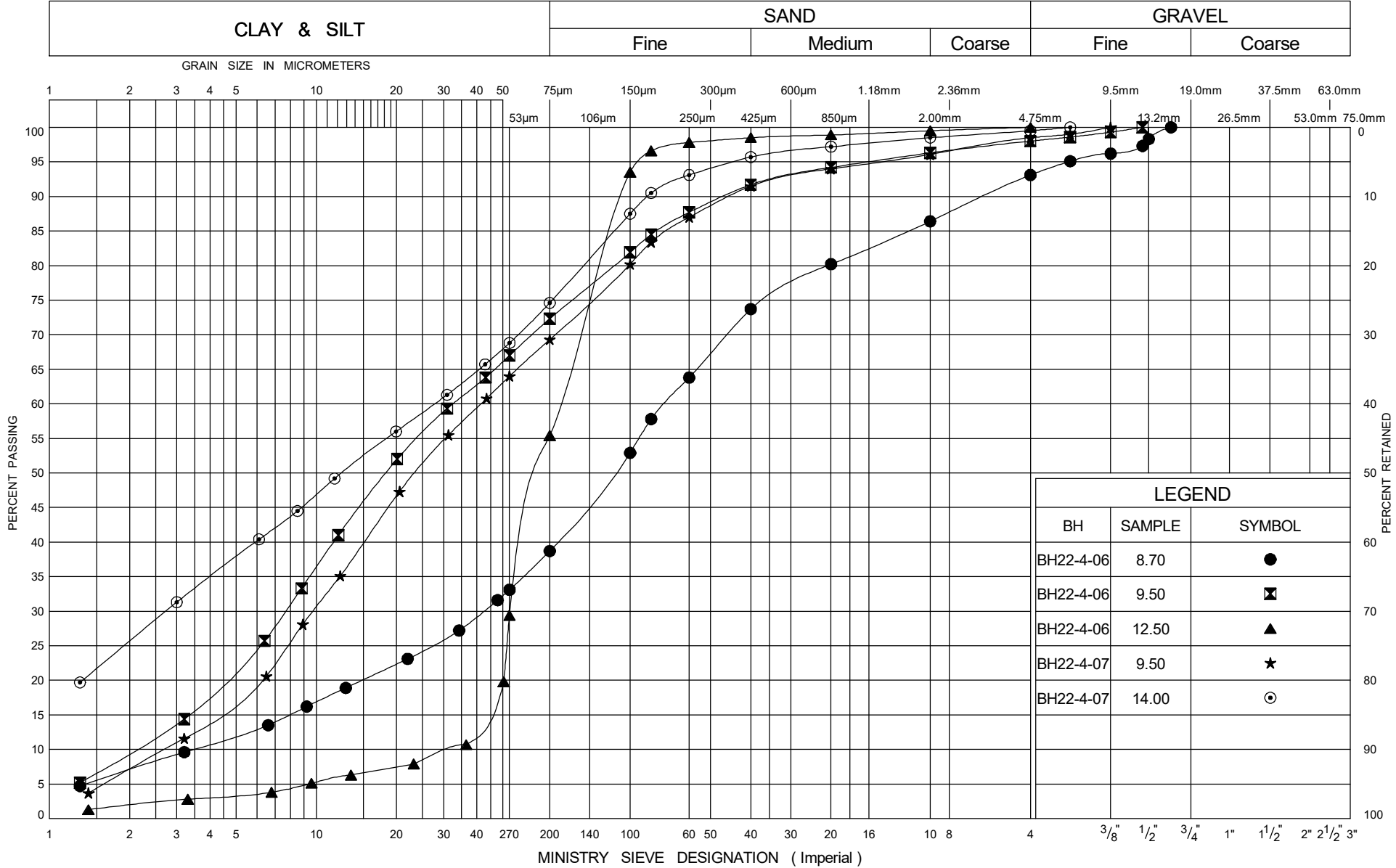
UNIFIED SOIL CLASSIFICATION SYSTEM



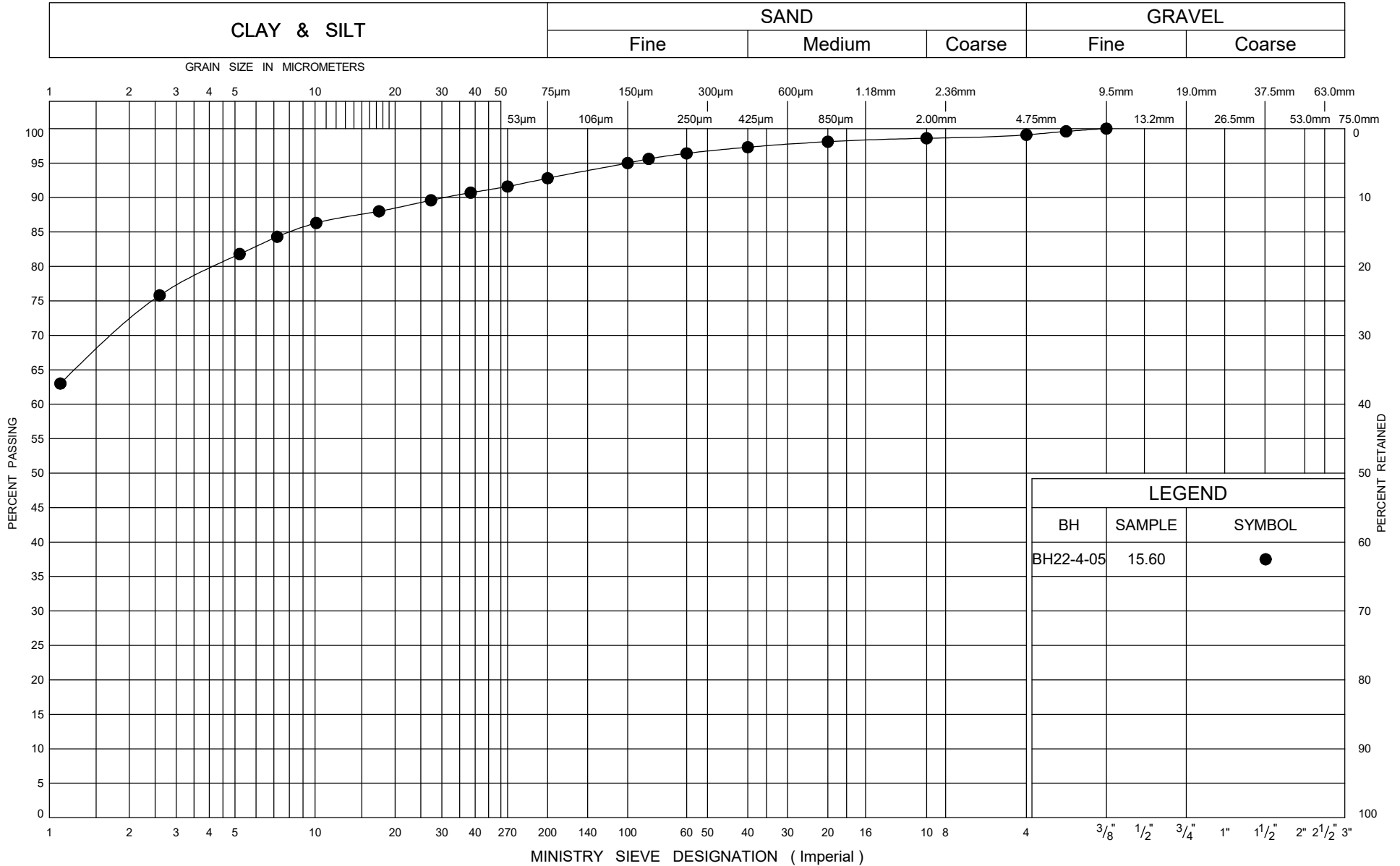
GRAIN SIZE DISTRIBUTION  
Sandy Silt/Sand and Silt/Silty Sand/Silt

FIG No 3a  
W P Site 37X-0214/B1&B3  
Hwy 401 - Kennedy Road O/P

UNIFIED SOIL CLASSIFICATION SYSTEM



# UNIFIED SOIL CLASSIFICATION SYSTEM



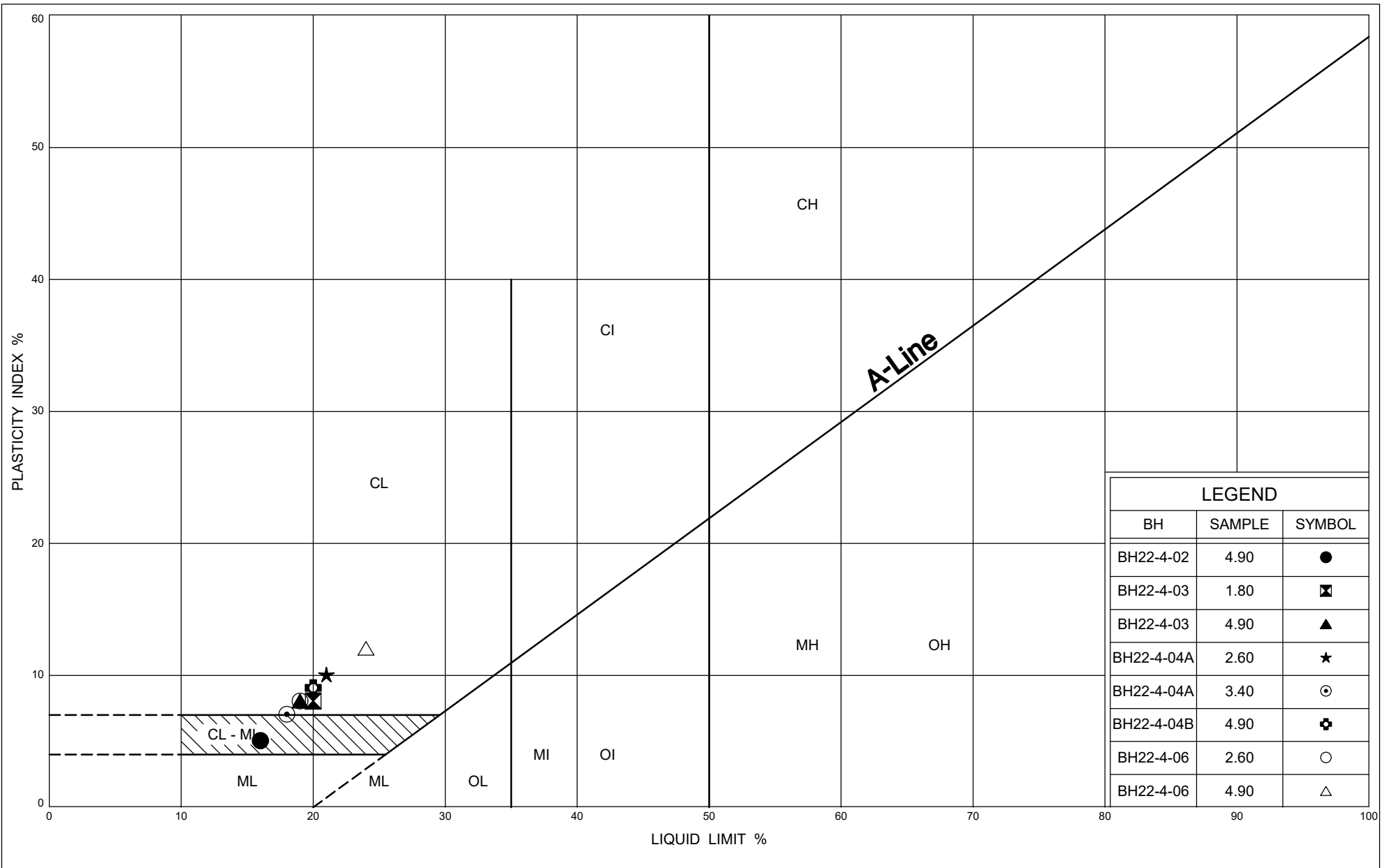
## GRAIN SIZE DISTRIBUTION

Silty Clay

FIG No 4

W P Site 37X-0214/B1&B3

Hwy 401 - Kennedy Road O/P



Ministry of  
Transportation

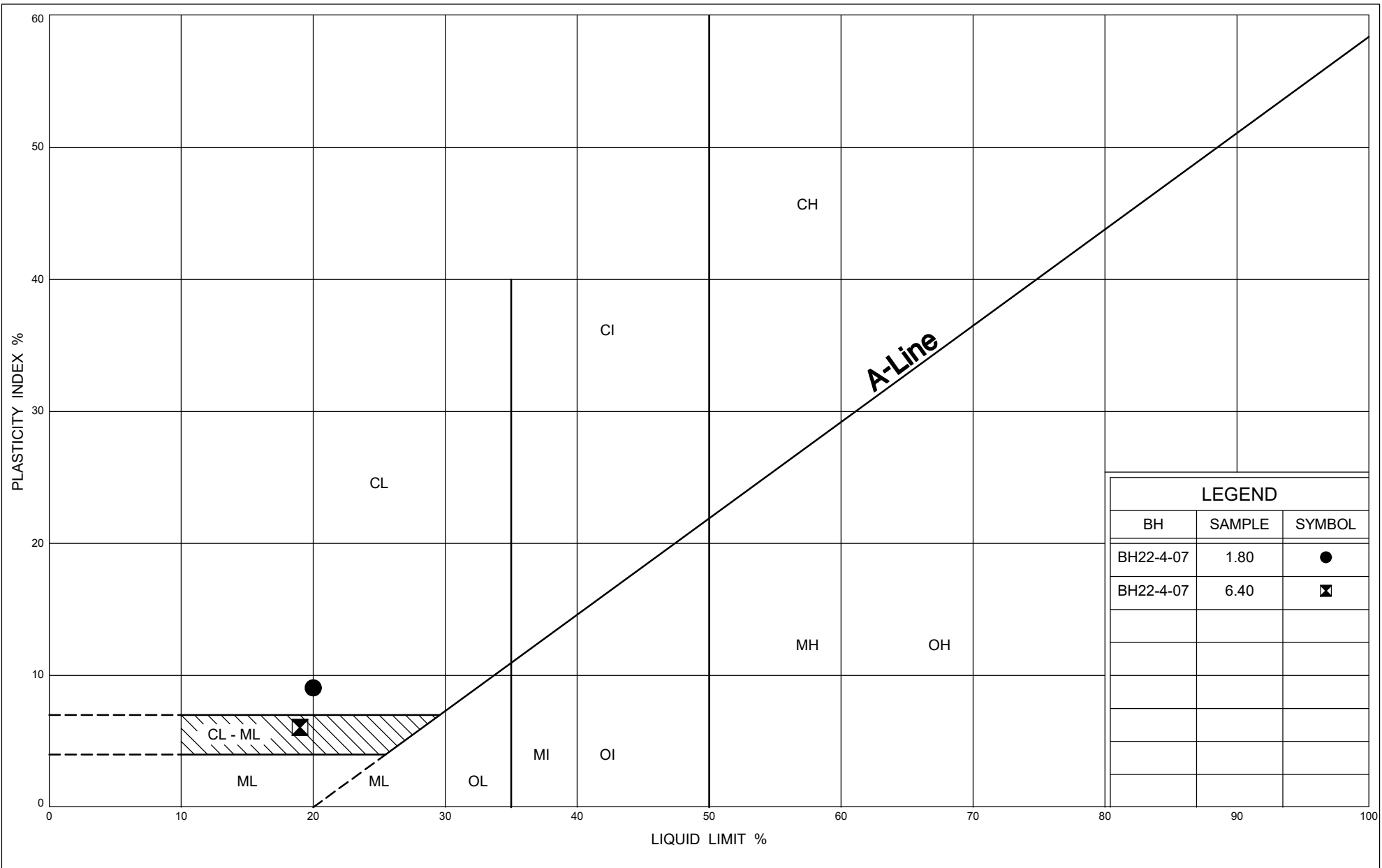
## PLASTICITY CHART

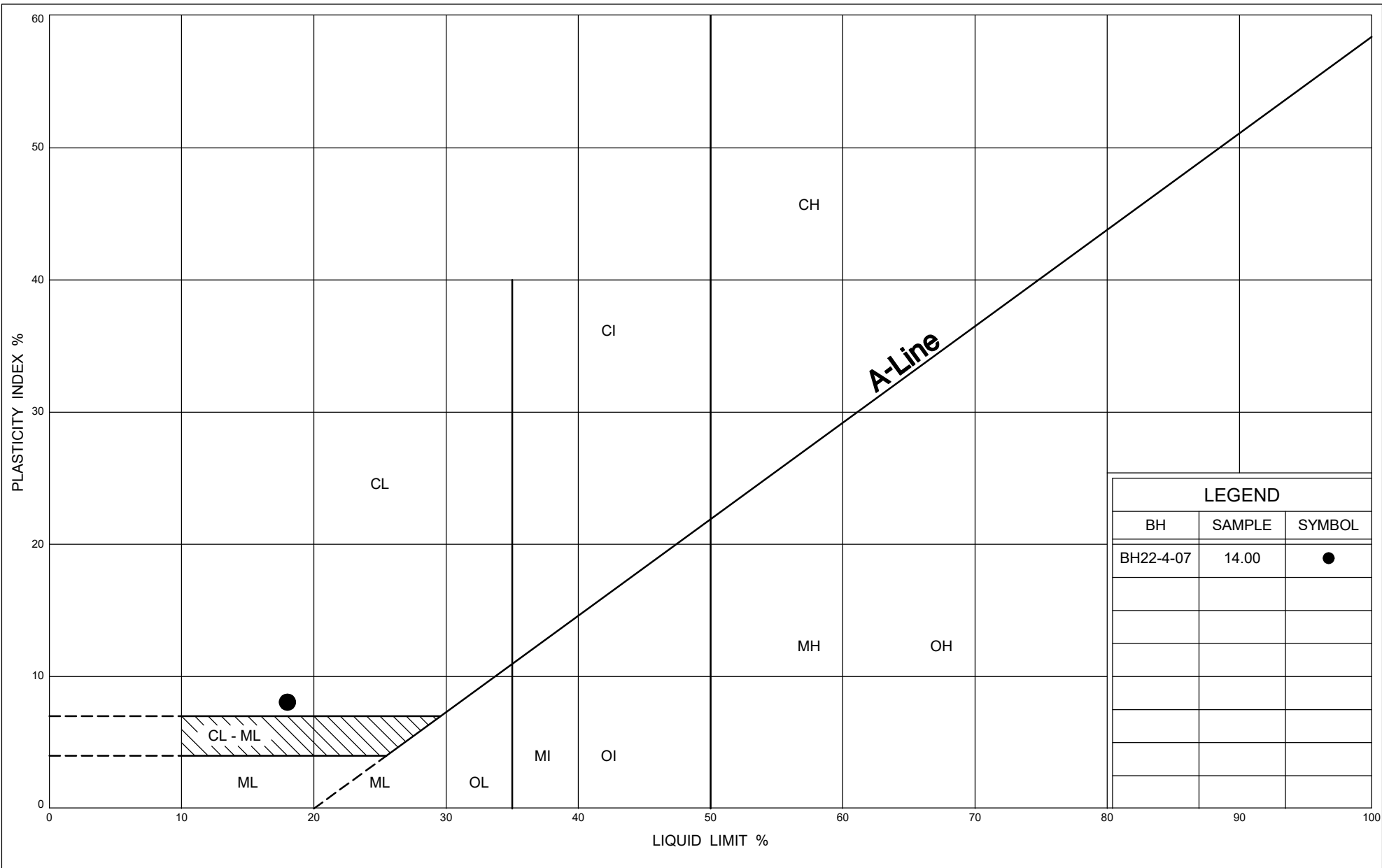
Cohesive Fill

FIG No 5a

W P Site 37X-0214/B1&B3

Hwy 401 - Kennedy Road O/P





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Transportation

## PLASTICITY CHART

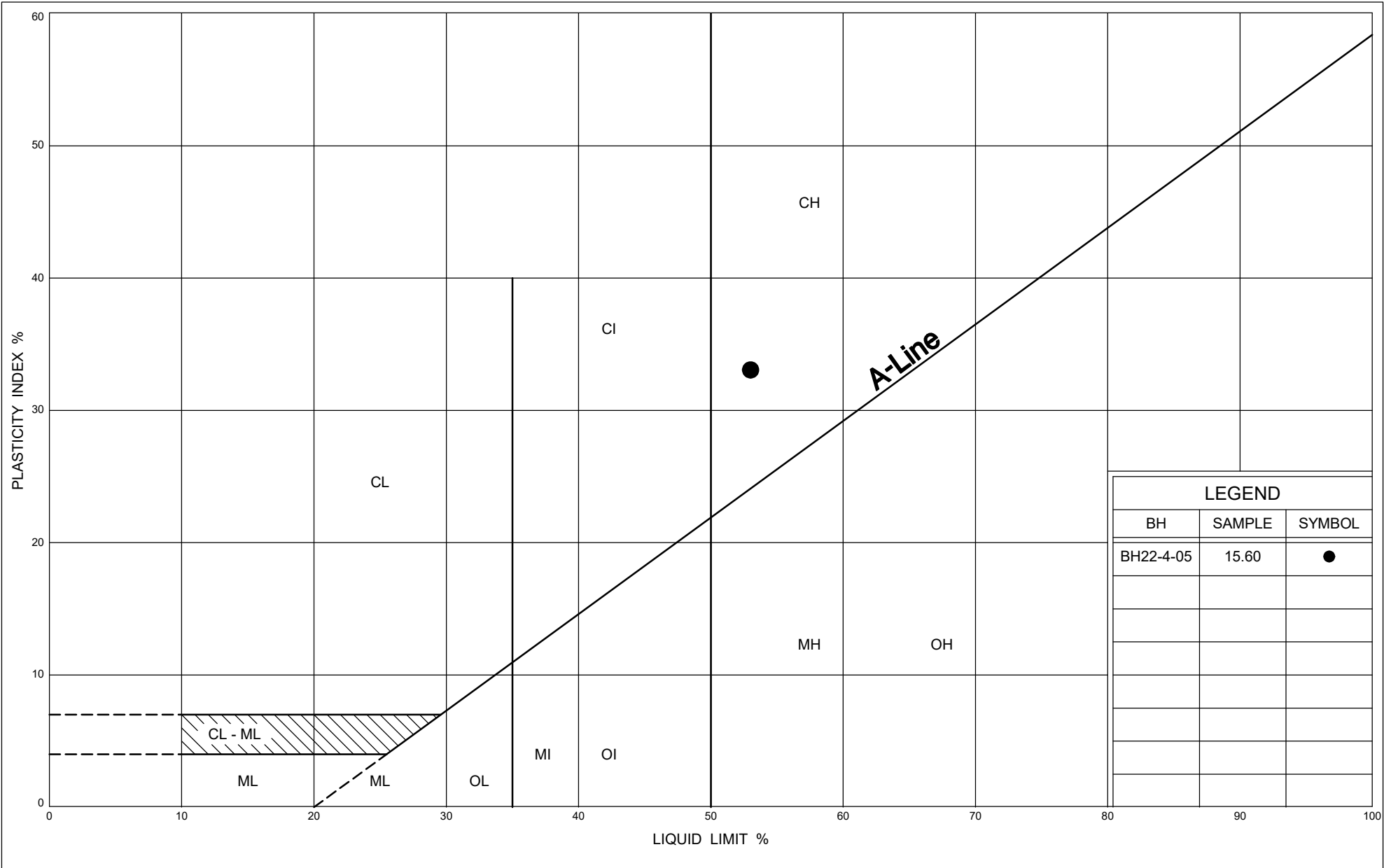
Sandy Silt/Sand and Silt/Silty Sand/Silt

FIG No 6

W P Site 37X-0214/B1&B3

Hwy 401 - Kennedy Road O/P







Your Project #: ADM-22000797-A0  
Your C.O.C. #: 903374-01-01

**Attention: Nimesh Tamrakar**

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

**Report Date: 2022/11/08**  
Report #: R7378236  
Version: 1 - Final

**CERTIFICATE OF ANALYSIS**

**BUREAU VERITAS JOB #: C2V5874**

**Received: 2022/10/28, 11:51**

Sample Matrix: Soil  
# Samples Received: 2

Analyses	Quantity	Date	Date	Laboratory Method	Analytical Method
		Extracted	Analyzed		
Chloride (20:1 extract)	2	2022/11/03	2022/11/04	CAM SOP-00463	SM 23 4500-Cl E m
Conductivity	2	2022/11/03	2022/11/03	CAM SOP-00414	OMOE E3530 v1 m
Moisture (Subcontracted) (1, 2)	2	N/A	2022/11/03	AB SOP-00002	CCME PHC-CWS m
Sulphide in Soil (1)	2	N/A	2022/11/02	AB SOP-00080	EPA9030B/SM4500S2-DF
pH CaCl2 EXTRACT	2	2022/11/03	2022/11/03	CAM SOP-00413	EPA 9045 D m
Redox Potential (3)	2	2022/11/03	2022/11/04	CAM SOP-00421	SM 2580 B
Resistivity of Soil	2	2022/10/28	2022/11/04	CAM SOP-00414	SM 23 2510 m
Sulphate (20:1 Extract)	2	2022/11/03	2022/11/07	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  
Your C.O.C. #: 903374-01-01

**Attention: Nimesh Tamrakar**

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

**Report Date: 2022/11/08**  
Report #: R7378236  
Version: 1 - Final

**CERTIFICATE OF ANALYSIS**

**BUREAU VERITAS JOB #: C2V5874**

**Received: 2022/10/28, 11:51**

(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

=====

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

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



BUREAU  
VERITAS

Bureau Veritas Job #: C2V5874  
Report Date: 2022/11/08

exp Services Inc  
Client Project #: ADM-22000797-A0  
Sampler Initials: NT

### SOIL CORROSIVITY PACKAGE (SOIL)

Bureau Veritas ID		UDK776		UDK777			UDK777		
Sampling Date		2022/10/25 15:00		2022/10/23 15:00			2022/10/23 15:00		
COC Number		903374-01-01		903374-01-01			903374-01-01		
	<b>UNITS</b>	<b>22-4-2 SS11</b>	<b>QC Batch</b>	<b>22-4-3 SS6</b>	<b>RDL</b>	<b>QC Batch</b>	<b>22-4-3 SS6 Lab-Dup</b>	<b>RDL</b>	<b>QC Batch</b>
<b>Calculated Parameters</b>									
Resistivity	ohm-cm	970	8313871	1100		8313871			
<b>CONVENTIONALS</b>									
Redox Potential	mV	210	8325057	220	N/A	8325057			
<b>Inorganics</b>									
Soluble (20:1) Chloride (Cl-)	ug/g	470	8324034	400	20	8324034			
Conductivity	umho/cm	1030	8325071	895	2	8325071	889	2	8325071
Available (CaCl2) pH	pH	7.91	8324644	7.89		8324213			
Soluble (20:1) Sulphate (SO4)	ug/g	77	8324043	<20	20	8324043			
Sulphide	mg/kg	<0.5 (1)	8331974	<0.5 (2)	0.5	8331974			
<b>Physical Testing</b>									
Moisture-Subcontracted	%	8.6	8331973	12	0.30	8331973			
RDL = Reportable Detection Limit QC Batch = Quality Control Batch Lab-Dup = Laboratory Initiated Duplicate N/A = Not Applicable (1) Sample contained greater than 10% headspace at time of extraction. Sample extracted past method-specified hold time. Analyzed past method specified hold time (2) Sample contained greater than 10% headspace at time of extraction. Sample extracted past method-specified hold time. Analyzed past method specified hold time									



**BUREAU  
VERITAS**

Bureau Veritas Job #: C2V5874

Report Date: 2022/11/08

exp Services Inc

Client Project #: ADM-22000797-A0

Sampler Initials: NT

## TEST SUMMARY

**Bureau Veritas ID:** UDK776  
**Sample ID:** 22-4-2 SS11  
**Matrix:** Soil

**Collected:** 2022/10/25  
**Shipped:**  
**Received:** 2022/10/28

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	8324034	2022/11/03	2022/11/04	Samuel Law
Conductivity	AT	8325071	2022/11/03	2022/11/03	Surinder Rai
Moisture (Subcontracted)	BAL	8331973	N/A	2022/11/03	Winston Lee
Sulphide in Soil	SPEC	8331974	N/A	2022/11/02	Ly Vu
pH CaCl <sub>2</sub> EXTRACT	AT	8324644	2022/11/03	2022/11/03	Taslina Aktar
Redox Potential	COND	8325057	2022/11/03	2022/11/04	Surinder Rai
Resistivity of Soil		8313871	2022/11/04	2022/11/04	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	8324043	2022/11/03	2022/11/07	Samuel Law

**Bureau Veritas ID:** UDK777  
**Sample ID:** 22-4-3 SS6  
**Matrix:** Soil

**Collected:** 2022/10/23  
**Shipped:**  
**Received:** 2022/10/28

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	8324034	2022/11/03	2022/11/04	Samuel Law
Conductivity	AT	8325071	2022/11/03	2022/11/03	Surinder Rai
Moisture (Subcontracted)	BAL	8331973	N/A	2022/11/03	Winston Lee
Sulphide in Soil	SPEC	8331974	N/A	2022/11/02	Ly Vu
pH CaCl <sub>2</sub> EXTRACT	AT	8324213	2022/11/03	2022/11/03	Taslina Aktar
Redox Potential	COND	8325057	2022/11/03	2022/11/04	Surinder Rai
Resistivity of Soil		8313871	2022/11/04	2022/11/04	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	8324043	2022/11/03	2022/11/07	Samuel Law

**Bureau Veritas ID:** UDK777 Dup  
**Sample ID:** 22-4-3 SS6  
**Matrix:** Soil

**Collected:** 2022/10/23  
**Shipped:**  
**Received:** 2022/10/28

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Conductivity	AT	8325071	2022/11/03	2022/11/03	Surinder Rai



BUREAU  
VERITAS

Bureau Veritas Job #: C2V5874

Report Date: 2022/11/08

exp Services Inc

Client Project #: ADM-22000797-A0

Sampler Initials: NT

### GENERAL COMMENTS

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

Package 1	0.3°C
-----------	-------

Results relate only to the items tested.

BUREAU  
VERITAS

Bureau Veritas Job #: C2V5874

Report Date: 2022/11/08

## QUALITY ASSURANCE REPORT

exp Services Inc

Client Project #: ADM-22000797-A0

Sampler Initials: NT

QC Batch	Parameter	Date	Matrix Spike		SPIKED BLANK		Method Blank		RPD	
			% Recovery	QC Limits	% Recovery	QC Limits	Value	UNITS	Value (%)	QC Limits
8324034	Soluble (20:1) Chloride (Cl <sup>-</sup> )	2022/11/04	111	70 - 130	108	70 - 130	<20	ug/g	NC	35
8324043	Soluble (20:1) Sulphate (SO <sub>4</sub> )	2022/11/07	119	70 - 130	109	70 - 130	<20	ug/g	NC	35
8324213	Available (CaCl <sub>2</sub> ) pH	2022/11/03			100	97 - 103			0.38	N/A
8324644	Available (CaCl <sub>2</sub> ) pH	2022/11/03			100	97 - 103			0.86	N/A
8325057	Redox Potential	2022/11/04			101	95 - 105			3.9	N/A
8325071	Conductivity	2022/11/03			104	90 - 110	<2	umho/cm	0.68	10
8331973	Moisture-Subcontracted	2022/11/03					<0.30	%		
8331974	Sulphide	2022/11/02	65 (1)	75 - 125	108	75 - 125	<0.5	mg/kg		

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

(1) Matrix spike exceeds acceptance limits due to matrix interference.



BUREAU  
VERITAS

Bureau Veritas Job #: C2V5874

Report Date: 2022/11/08

exp Services Inc

Client Project #: ADM-22000797-A0

Sampler Initials: NT

### VALIDATION SIGNATURE PAGE

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

Anastassia Hamanov, Scientific Specialist

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

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

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Bureau Veritas has procedures in place to guard against improper use of the electronic signature and have the required "signatories", as per ISO/IEC 17025, signing the reports. For Service Group specific validation, please refer to the Validation Signatures page if included, otherwise available by request. For Department specific Analyst/Supervisor validation names, please refer to the Test Summary section if included, otherwise available by request. This report is authorized by {0}, {1} responsible for {2} {3} laboratory operations.





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

28-Oct-22 11:51

Patricia Legette

C2V5874

AN4 ENV-778

903374

COC #:

Project Manager:

C#903374-01-01

Patricia Legette

INVOICE TO:		REPORT TO:		PROJECT INFORMATION:	
Company Name: #17488 exp Services Inc	Company Name: Nimesh Tamrakar	Quotation #: C20328			
Attention: Accounts Payable	Attention: Nimesh Tamrakar	P.O. #:			
Address: 1595 Clark Blvd	Address:	Project: ADM-22000797-A0			
Brampton ON L6T 4V1		Project Name:			
Tel: (905) 793-9800 Fax: (905) 793-0641	Tel: (905) 796-3200 Ext: 3026 Fax:	Site #:			
Email: AP@exp.com; Karen.Burke@exp.com	Email: Nimesh.Tamrakar@exp.com	Sampled By:			

MOE REGULATED DRINKING WATER OR WATER INTENDED FOR HUMAN CONSUMPTION MUST BE SUBMITTED ON THE BUREAU VERITAS DRINKING WATER CHAIN OF CUSTODY					
Regulation 153 (2011)		Other Regulations		Special Instructions	
<input type="checkbox"/> Table 1	<input type="checkbox"/> Res/Park	<input type="checkbox"/> CCME	<input type="checkbox"/> Sanitary Sewer Bylaw		
<input type="checkbox"/> Table 2	<input type="checkbox"/> Ind/Comm	<input type="checkbox"/> Reg 558	<input type="checkbox"/> Storm Sewer Bylaw		
<input type="checkbox"/> Table 3	<input type="checkbox"/> Agri/Other	<input type="checkbox"/> MISA	<input type="checkbox"/> Municipality		
<input type="checkbox"/> Table	<input type="checkbox"/> For RSC	<input type="checkbox"/> PWQO	<input type="checkbox"/> 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	
1	22-4-2 SS11	Oct 25/23	03:00 AM		Field Filtered (please circle): Metals / Hg / Cr-VI 50.1 CONTAMINANT PACKAGE
2	22-4-2 SS11	Oct 25/23	03:00 AM		
3	22-4-3 SS6	Oct 24/23	03:00 AM		
4	22-4-3 SS6	Oct 24/23	03:00 AM		
5					
6					
7					
8					
9					
10					

* 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		
								Time Sensitive	Temperature (°C) on Reel	Custody Seal
									0/0/1	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/ENVIRONMENTAL-LABORATORIES/RESOURCES/COC-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/ENVIRONMENTAL-LABORATORIES/RESOURCES/CHAIN-CUSTODY-FORMS-COCS.

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

White: Bureau Veritas Yellow: Client

Bureau Veritas Canada (2019) Inc.

## Appendix F – Previous Investigation - BH logs

DEPARTMENT OF HIGHWAYS - ONTARIO

## MATERIALS &amp; TESTING DIVISION

JOB 66-E-33

LOCATION  Hwy. 401 & Kennedy Rd. Sta. 344/11 64' Rt.

ORIGINATED BY V.K.

W.P. \_\_\_\_\_ BORING DATE April 4, 1966

COMPILED BY V.K.

DATUM Geodetic

BOREHOLE TYPE Penn-Drill

CHECKED BY 2/5

## RECORD OF BOREHOLE NO. 1.

FOUNDATION SECTION

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT	LIQUID LIMIT — WL PLASTIC LIMIT — WP WATER CONTENT — W			BULK DENSITY P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT		20 40 60 80 100	Wp ———— W ———— WL				
<b>167.9</b>	Ground Level											
521.0												
0.0						550						Gr-2 Sa-14 Si-78 Cl-6
	Bandy Silt to Silty Sand with traces of Gravel and Clay		1	SS	39							W.L. 545.5 5.5
			2	SS	72							
	Dense to V. Dense		3	SS	116/6"	540						Gr-2 Sa-8 Si-82 Cl-3
			4	SS	100							
			5	SS	66							
			6	SS	120	530						
			7	SS	100/4"							
<b>158.8</b>												
521.0												
30.0												Gr-5 Sa-21 Si-52 Cl-22
	Heterogeneous mixture of Clayey Silt, Sand and Gravel		8	SS	100/4"	520						
	GLACIAL TILL		9	SS	100/4"							
	Hard		10	SS	100/5"	510						
			11	SS	100/3"							
201.0												
50.0	End of Borehole					500						

FOUNDATION SECTION

MATERIALS &amp; TESTING DIVISION

JOB 66-F-33

LOCATION Hwy. 401 & Kennedy Rd. Sta. 342+84 140' Rt.

ORIGINATED BY V.K.

W. P. \_\_\_\_\_ BORING DATE April 12, 1966

COMPILED BY \_\_\_\_\_ V.K.

DATUM Guadalupe

BOREHOLE TYPE Penn - Drill

CHECKED BY

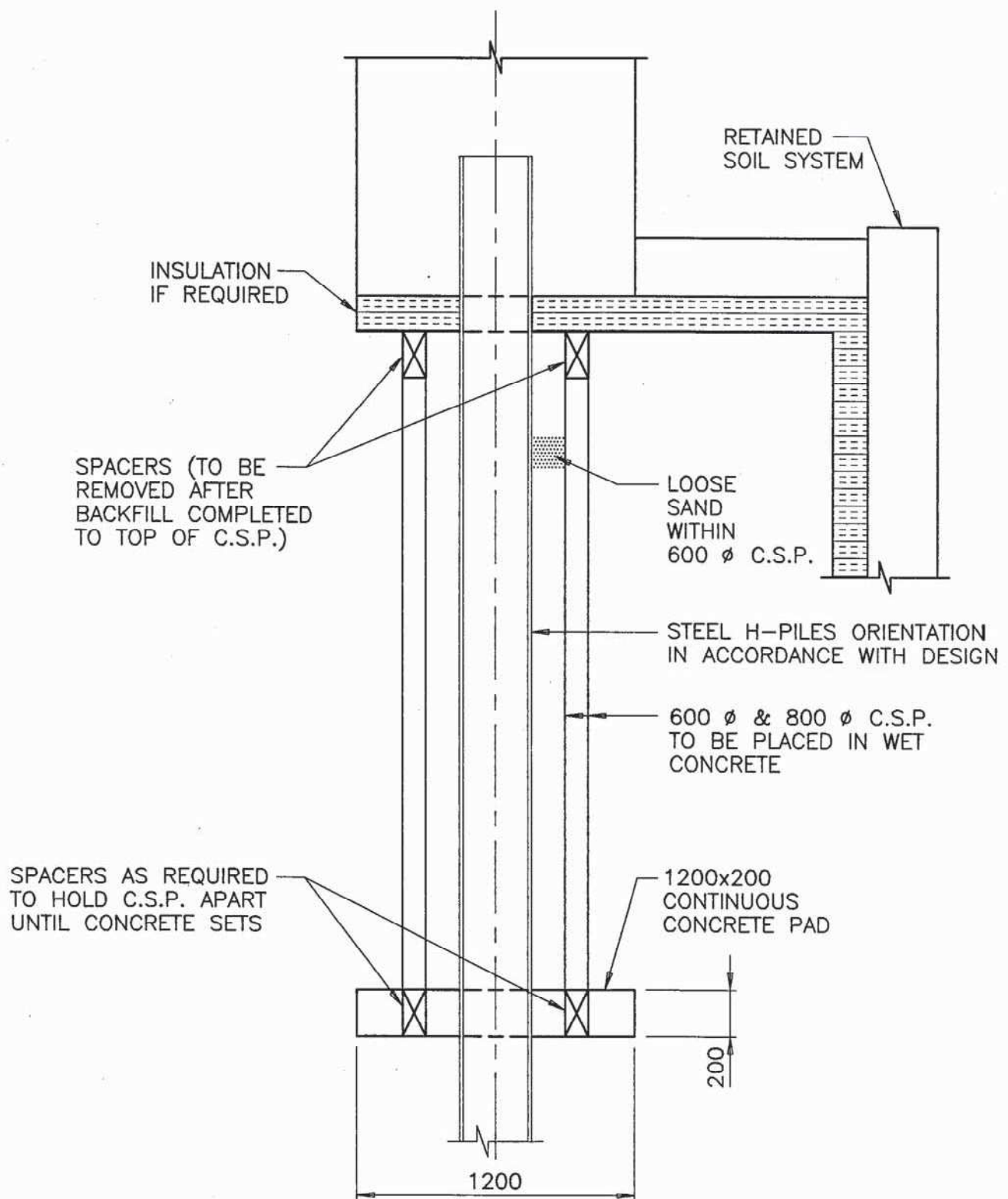
SOIL PROFILE		SAMPLES	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT	LIQUID LIMIT ——— WL PLASTIC LIMIT ——— WP WATER CONTENT ——— W	BULK DENSITY	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT		$\frac{w}{L}$ $\frac{w}{P} \quad w \quad w/L$	P.C.F.	
			20    40    60    80    100			
			SHEAR STRENGTH P.S.F.			
		NUMBER	TYPE	BLOWS / FOOT	ELEV SCALE	
559.0 0.0	Ground Level					
	Fandy Silt to Silty Sand with traces of Gravel and Clay	1 SS 12			550	Gr-2 Sa-52 Si-39 Cl-7
		2 SS 36				
		3 SS 62/6"				Gr-1 Sa-30 Si-65 Cl-4
		4 SS 119				w L.L. = 245.0 14.0
		5 SS 170/10"			540	
	Compact to very Dense	6 SS 100/7"				
		7 SS 100/5"				
529.0 30.0	Heterogeneous Mixture of Clayey Silt, Sand and Gravel	8 SS 136/11"			530	
		9 SS 112				Gr-5 Sa-28 Si-58 Cl-9
	GLACIAL TILL	10 SS 100/4"			520	
512.5 46.5	Hard	11 SS 100/A"				

[illegible]

EXP Services Inc.

*Foundation Investigation and Design Report  
Highway 401 Eastbound from Victoria Park Avenue to Neilson Road  
Superstructure Replacement at Kennedy Road Overpass  
Eastbound Core and Collectors Structure (Site 37X-0216/B1 & B3)  
Assignment No. 2021-E-0018  
Date: September 19, 2024*

## Appendix G – Standard Detail for Double CSP at Integral Abutment



INTEGRAL ABUTMENT WITH RETAINED SOIL SYSTEM

FIG. 7



## Appendix H – 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



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 (°)	43.775
Longitude (°)	-79.285

**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	0.0482	0.0125	0.00424	0.176	0.13

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

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

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

---

### **Scope of Work**

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

### **Basis of Payment**

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

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

---

### **Scope of Work**

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

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

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

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