Foundation Investigation and Design Report

Highway 401 Eastbound Express and Collector Lanes between Victoria Park Avenue and Neilson Road - **Superstructure Replacement at Markham Road Overpass Eastbound Core and Collectors Structure (Site 37X-0218/B1 & B3)**

Assignment No. 2021-E-0018
MTO Central Region
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Project Number:
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AECOM Canada Ltd.

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References

Part I: Foundation Investigation Report

Highway 401 Eastbound Express and Collector Lanes between Victoria Park Avenue and Neilson Road – Markham Road Overpass (Site 37X-0218/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 the 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 Markham Road Overpass structure (Site 37X-0218/B1 & B3).

The General Arrangement drawings (GA) for the bridge structure were provided to EXP by AECOM. The purpose of the investigation was to evaluate the subsurface conditions along the structure alignment to permit a detailed design for the proposed 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 Markham Rd OP Bridge Rehabilitation",* prepared by AECOM, dated August 2024, shows the preliminarily configuration of the Markham Road Overpass structure. Foundation Investigation and 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 *"Markham Road Overpass Rehabilitation and Northward Widening (Site No. 37-218), Highway 401 Westbound Core and Collector Lanes, Neilson Road to Warden Avenue, City of Toronto, Ontario, G.W.P No. 2162-11-00.", dated January 17, 2019,were reviewed.* A summary of the proposed structure is as follows:

1. The existing structure is a 37.28 m long two-span bridge. It is understood that the existing abutments, piers, and retaining wall foundations are supported on spread footings. However, it is assumed that they are similar to the Westbound Core and Collectors Structure. Based on the previous FIDRs, the existing abutments are supported on 3.9 m wide footings found at about Elevation 155.7 m and the centre piers are supported on 3.4 m wide footings found at about Elevation 155.8 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. No widening of Highway 401 is proposed on the Eastbound side.

The previous FIDRs and GA drawing by AECOM, in addition to contract package drawings titled *Hwy 401 WB Core & Collector Lanes – Markham Road Overpass – Bridge Rehabilitation (Cont. No. 2019-2011, WP No. 2392/2391-15-01),* produced by WSP Global Inc., dated March 2019, were reviewed as part of this report. These background documents are used for initial context to address the nature and scope of the investigation. It is understood that some changes might occur because 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 Markham Road, approximately 8 km east of Highway 404 in the City of Toronto, Ontario. The site is adjacent to industrial zones to the south and northeast, and residential zones to the northwest of the site. In general, the terrain in this area is relatively flat, with the natural ground surface sloping gently toward the south. The Highway 401 pavement grade ranges between about Elevation 164 m and 165 m while, the Markham Road pavement grade is an Elevation approximately 158 m to 159 m at the structure site. Based on the FIDRs by Golder Associates Ltd., the fill thickness is assumed to be about 7 to 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 predominant landforms are Till Plains (Drumlinized) and Drumlins. The South Slope represents the southern slope of the Oak Ridges Moraine but also includes a strip south of the Peel Plain, extending from the Niagara Escarpment to the Trent River. The South Slope gradually, fairly, and uniformly slopes down toward 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 which is predominately silt to silty clay matrix, high in matrix carbonate content and clast poor with occasional sand to silt zones. In addition, Map 2544 (Bedrock Geology of Ontario, Southern Sheet, 1991), the bedrock geology at the site consists of shale, limestone, dolostone and siltstone: Georgian Bay Formation, Blue Mountain Formation, Bilings Formation, Collingwood Member, Eastview Member.

4.0 Previous Geotechnical Investigation

During the tender design for the project, three (3) previous reports were issued which contain relevant information to the proposed Markham Road Overpass structure (Site 37X-0218/B1 & B3), as follows:

1. Geocres No. 30M14-69 *"Foundation Investigation Report for Proposed New Structure at Markham Rd. and Hwy #401, District #6 (Toronto), W.J. 67-F-40, W.P. 262-61."* by Department of Highways Ontario - Foundation Section, dated June 9, 1967.
2. Geocres No. 30M14-338 *"Bridge Widening and Replacement Highway 401 Rehabilitation from Warden Avenue to Brock Road, Toronto, Ontario, W.O.07-20012."* by Golder Associates Ltd., dated April 2012.
3. Geocres No. 30M14-484 *"Markham Road Overpass Rehabilitation and Northward Widening (Site No. 37-218), Highway 401 Westbound Core and Collector Lanes, Neilson Road to Warden Avenue, City of Toronto, Ontario, G.W.P No. 2162-11-00."* by Golder Associates Ltd., dated January 17, 2019.

The applicable previous MTO borehole logs are attached as Appendix 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				
32-1	East Abutment, South Side (EBL Collector)	4849502.2	326139.3	43.785174	-79.234873	157.7	9.6
32-2	Centre Pier, South Side (EBL Collector)	4849511.6	326117.1	43.785259	-79.235148	157.4	9.6
32-3	West Abutment, South Side (EBL Collector)	4849489.1	326104.6	43.785057	-79.235304	157.3	9.6

5.0 Field Investigation and Laboratory Analyses

5.1 Site Investigation and Field Testing

A site-specific investigation was undertaken by EXP between September 13, 2022, and September 19, 2022, and it included the following:

1. A walkover site assessment was carried out by a Geotechnical Engineer from EXP.
2. Subsequent to the borehole layouts in the field, existing utilities were cleared by public utility companies.
3. At the time of this report, four (4) boreholes have been completed for this structure (BH22-6-1 to BH22-6-4) 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. Boreholes were set back at least 14 m from the abutment to avoid drilling through the reinforced approach slab.
5. Soil samples in the boreholes were taken at frequent intervals of depth by the Standard Penetration Test method (SPT), in general accordance with ASTM D1586. The test consists of freely dropping a 63.5 kg hammer a vertical distance of 0.76 m to drive a 51 mm O.D. split barrel (SS-split-spoon) sampler into the ground. The number of blows of the hammer required to drive the sampler into the relatively undisturbed ground by a vertical distance of 0.30 m is recorded as the Standard Penetration Resistance, or the N-value, of the soil which is indicative of the compactness of granular (or cohesionless) soils (gravels, sands and silts) or the consistency of cohesive soils (clays and clayey soils).
6. The fieldwork was supervised by a member of EXP’s engineering staff who directed the drilling and sampling operation, logged borehole data in accordance with MTO and/or ASTM Standards for Soils Classification, and retrieved soil samples for subsequent laboratory testing and identification.
7. All spoon samples obtained in the Standard Penetration Tests (SPT, ASTM D-1586) were placed in moisture proof bags after field classification. Samples were allocated from the spoon samples for moisture content testing without delay. They were subsequently re-examined under controlled laboratory conditions prior to assigning other laboratory tests.
8. Selected soil samples for corrosivity testing were sent to the Bureau Veritas Laboratories (formerly Maxxam Analytics), a CALA-certified and accredited laboratory in Mississauga, Ontario. The selected soil samples for the analytical testing were placed in a laboratory prepared glass jar, labelled, and stored in a secure cooler.

9. The borehole locations and their ground surface elevations were surveyed by EXP using a Trimble DA2 GNSS receiver with Trimble Catalyst GNSS positioning, having an accuracy of ±0.10 m horizontal and vertical directions. MTM NAD83 Zone 10 coordinates and the geodetic elevation for the boreholes are listed in Table 1.2 below. It can also be found on the Record of Borehole Sheet (Appendix D); and

10. Upon completion of drilling and field testing, the boreholes were backfilled with a mixture of bentonite and auger cuttings. The borehole decommissioning was in general accordance with the Ministry of the Environment Regulation 903, as amended by Regulation 128/03 (the well regulation under the Ontario Water Resources Act).

Table 1.2: Summary of boreholes completed by EXP

Borehole No.	Borehole Location	Location (MTM NAD83 Zone 10)		Latitude	Longitude	Borehole Elevation (m)	Borehole Depth (m)
		Northing	Easting				
BH22-6-1	~14 m west of West Abutment, b/w EBL and WBL Express	4849527.5	326078.3	43.785404	-79.235629	165.3	12.0[1]
BH22-6-2	~14 m east of East Abutment, b/w EBL and WBL Express	4849551.7	326140.5	43.785620	-79.234856	164.6	12.8[1]
BH22-6-3	~29 m west of West Abutment, b/w EBL and WBL Express	4849521.4	326063.7	43.785349	-79.235811	165.4	14.3[1]
BH22-6-4	~29 m east of East Abutment, b/w EBL and WBL Express	4849559.0	326153.7	43.785685	-79.234692	164.3	12.0[1]

Note:

1.0 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
BH22-6-1	12	2	3	3	6	1
BH22-6-2	12	1	3	3	3	1
BH22-6-3	14	1	3	2	5	-
BH22-6-4	13	1	3	3	2	-

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

6.0 Subsurface Conditions

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

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

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 clayey silt and sand/sandy silt/sand and silt.

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 boreholes BH22-6-1, BH22-6-2, BH22-6-3 and BH22-6-4. The thickness of the structure ranged between 400 mm and 465 mm.

6.1.2 Cohesionless Fill: Sand and Gravel

During EXP's geotechnical investigation, sand and gravel fill was encountered below the pavement structure (asphalt/concrete) in boreholes BH22-6-1, BH22-6-3 and BH22-6-4. 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 Layers

Borehole	Elevation (m)		Layer Surface Depth (m)	Layer Thickness (m)	Layer Description	SPT "N" Value Range
	Top	Bottom				
EXP (2022)						
BH22-6-1	164.8	164.5	0.4	0.3	Sand and Gravel	N/A[1]
BH22-6-3	165.0	163.1	0.5	1.9	Sand and Gravel	54 – 102/100 mm
BH22-6-4	163.8	163.3	0.5	0.5	Sand and Gravel	N/A[1]

Note:

1. *No SPT sampling within layer, only auger samples retrieved.*

This layer consists of mainly sand and gravel with some silt. The material was greyish brown to brown in colour and moist to wet. SPT "N" values obtained within this layer range from 54 blows per 300 mm penetration to 102 blows per 100 mm penetration, corresponding to very dense in compactness.

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

Moisture Content: (EXP)

- 5% to 14%

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/Sand and Silt/Sandy Silt

During EXP's geotechnical investigation, sand/sand and silt/sandy silt fill was encountered below the pavement structure (asphalt/concrete) in borehole BH22-6-2 and below the sand and gravel fill in boreholes BH22-6-1 and BH22-6-3. 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/Sand and Silt/Sandy Silt Layers

Borehole	Elevation (m)		Layer Surface Depth (m)	Layer Thickness (m)	Layer Description	SPT "N" Value Range
	Top	Bottom				
EXP (2022)						
BH22-6-1	164.5	163.4	0.8	1.1	Sand	52
BH22-6-2	164.1	162.3	0.5	1.8	Sand to Sandy Silt	45 – 100
BH22-6-3	163.1	160.8	2.3	2.3	Sand and Silt	14 – 47
	159.3	157.8	6.1	1.5	Sandy Silt	16

This layer predominately consists of sand and silt with trace to some gravel and some clay. In addition, asphalt inclusions and topsoil/organics were encountered within this material. The material was greyish brown to grey with black inclusions in colour and slightly moist to moist. The SPT "N" values within this layer ranged from 14 to 100 blows per 300 mm penetration, corresponding to compact to very dense in compactness.

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

Moisture Content: (EXP)

- 5% to 12%

Grain Size Distribution: (EXP)

- 5% gravel;
- 37% sand;
- 40% silt;
- 18% clay;

Unit Weight: (EXP)

- 22.3 kN/m^3 to 23.1 kN/m^3

The results of the moisture content, grain size distribution and unit weight tests performed by EXP are provided on the record of borehole sheets in Appendix D. The results of grain size distribution tests are also provided on Figure 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 all boreholes (BH22-6-1 to BH22-6-4). 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)						
BH22-6-1	163.4	158.4	1.8	5.1	Clayey Silt	5 – 24
BH22-6-2	162.3	158.2	2.3	4.1	Clayey Silt	9 – 14
BH22-6-3	160.8	159.3	4.6	1.5	Clayey Silt	7
BH22-6-4	163.3	158.2	1.1	5.1	Clayey Silt	10 – 35

This layer predominately consists of silt and clay and can be considered some sand to sandy with trace gravel. Trace organics/rootlets were also encountered within this material. The material was brown to grey with black inclusions in colour and slightly moist to moist. The SPT "N" value within this layer ranged between 7 to 35 blows per 300 mm penetration, corresponding to firm to hard, but generally stiff in consistency. Atterberg limits tests suggest that this cohesive fill material is low plastic.

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

Moisture Content (EXP):

- 7% to 16%

Grain Size Distribution: (EXP)

- 2% to 7% gravel;
- 38% to 52% sand;
- 33% to 42% silt;
- 13% to 15% clay;

Atterberg Limits: (EXP)

- Liquid Limit: 17% to 19%.
- Plastic Limit: 11% to 12%.

- Plasticity Index: 5% to 8%

Unit Weight: (EXP)

- 21.0 kN/m^3 to 23.1 kN/m^3

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.

6.1.5 Clayey Silt

During EXP's geotechnical investigation, a native clayey silt layer was encountered below the cohesive fill layer in borehole BH22-6-1. Native clayey silt was also encountered at the surface in boreholes 32-1 and 32-2 and below a native sandy silt layer in borehole 32-3 during MTO's geotechnical investigation in 1967. 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 Clayey Silt Layers

Borehole	Elevation (m)		Layer Surface Depth (m)	Layer Thickness (m)	Layer Description	SPT "N" Value Range
	Top	Bottom				
EXP (2022)						
BH22-6-1	158.4	153.2	6.9	5.2[1]	Clayey Silt	11 – 155/225 mm
MTO (1967)						
32-1	157.7	148.1	0	9.6[1]	Clayey Silt	17 – 175/150 mm
32-2	157.4	147.8	0	9.6[1]	Clayey Silt	55 – 186
32-3	154.4	147.7	2.9	6.7[1]	Clayey Silt	50/13 mm – 200/115 mm

Note:
1.0 End of borehole terminated within this layer.

This layer predominately consists of silt and clay mixed with varying amounts of sand (trace to sandy) and trace gravel. The material was grey to light brown in colour and slightly moist. The SPT "N" value within this layer ranged between 11 to 186 blows per 300 mm penetration and up to 200 blows per 115 mm, corresponding to stiff to hard, but generally hard in consistency. Atterberg limits tests suggest that this layer is low plastic.

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

Moisture Content (EXP and MTO):

- 7% to 18%

Grain Size Distribution: (EXP and MTO)

- 0% gravel.
- 3% to 35% sand.

- 51% to 81% silt.
- 13% to 18% clay.

Atterberg Limits: (EXP and MTO)

- Liquid Limit: 20% to 24%.
- Plastic Limit: 12% to 17%.
- Plasticity Index: 5% to 8%

Unit Weight: (EXP)

- 21.9 kN/m^3 to 22.8 kN/m^3

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

6.1.6 Sand/Sand and Silt/Sandy Silt

During EXP's geotechnical investigation, a native sand/sand and silt/sandy silt layer below the fill in boreholes BH22-6-2, BH22-6-3 and BH22-6-4. Additionally, native sand and silt was encountered at the ground surface in borehole 32-3 during MTO's geotechnical investigation in 1967. 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 Sand/Sand and Silt/Sandy Silt Layers

Borehole	Elevation (m)		Layer Surface Depth (m)	Layer Thickness (m)	Layer Description	SPT "N" Value Range
	Top	Bottom				
			EXP (2022)			
BH22-6-2	158.2	151.8	6.4	6.4[1]	Sandy Silt to Sand	16 – 136
BH22-6-3	157.8	151.1	7.6	6.7	Sand to Sandy Silt	8 – 117/125 mm
BH22-6-4	158.2	152.3	6.1	5.9[1]	Sand and Silt to Sand	13 – 150
			MTO (1966)			
32-3	157.3	154.4	0	2.9	Sand and Silt	60 – 150/200 mm

Note:

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

This layer predominately consists of sand and/or silt with varying amounts of gravel (trace to gravelly) and trace clay. The material was grey in colour and moist to wet. The SPT "N" values within this layer ranged from 8 to 150 blows per 300 mm penetration, corresponding to loose to very dense but generally compact to very dense in compactness. Atterberg limits test results suggest that the latter is 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 and MTO):

- 7% to 21%

Grain Size Distribution: (EXP and MTO)

- 0% to 11% gravel;
- 30% to 85% sand;
- 12% to 60% silt;
- 3% to 9% clay;
- 23% to 48% silt and clay

Atterberg Limits: (EXP)

- Non-plastic

Unit Weight: (EXP)

- 20.1 kN/m^3 to 23.5 kN/m^3

The results of the moisture content, grain size distribution and unit weight tests performed by EXP are provided on the record of borehole sheets in Appendix D. The results of grain size distribution tests are also provided on Figure 4 in Appendix E. 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 1.9 and are also presented on the Record of Borehole Sheets attached in Appendix D and Appendix F.

Table 1.9: Summary of observed groundwater levels

Borehole	Ground Surface Elevation (m)	Water level Depth/ Elevation (m)	Date
EXP (2022)			
BH22-6-2	164.6	7.2/157.4[1]	September 18, 2022
BH22-6-3	165.4	8.1/157.3	September 14, 2022
BH22-6-4	164.3	8.7/155.6	September 19, 2022
MTO (1966)			
32-1	157.7	0.5/157.2	May 15, 1967
32-2	157.4	0.6/156.8	May 15, 1967

Borehole	Ground Surface Elevation (m)	Water level Depth/ Elevation (m)	Date
32-3	157.3	1.5/155.8	May 16, 1967

Note:

1. *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.

The analytical results are summarized in Table 1.10 below and are presented in Appendix E.

Table 1.10. Summary of chemical analysis results

Sample Identification	pH (Unitless)	Soluble Chloride (ppm)	Soluble Sulphate (ppm)	Resistivity (ohm-cm)	Conductivity (umho/cm)	Redox Potential (mV)
BH22-6-1, SS5	7.24	1000	<20	540	1870	180
BH22-6-2, SS10	7.84	90	34	4100	246	93

7.0 Closure

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. The field investigation was supervised by Elvis Lu, M.Eng.

Yours truly,

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Part II: Foundation Design Report

Discussion and Engineering Recommendations for Markham Road Overpass (Site 37X-0218/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.

Per the GA drawing and Geocres No. 30M14-484, the existing bridge is a 37.30 m long two-span bridge structure supported on 10.5 m and 3.3 m wide spread footings for the existing abutments and piers, respectively and the approximate existing footing elevations are about Elevation 155.6 m to 155.8 m (assumed based on Geocres No. 30M14-484). The Highway 401 pavement grade ranges between about Elevation 164 m to 165 m, while the Markham Road pavement grade is at an Elevation of about 158 m to 159 m at the structure site.

It is understood that, for the proposed rehabilitation of the Markham Road Overpass structure, there is no change in loading conditions on the foundation elements associated with the rehabilitation works will be negligible. The existing foundations will remain same and based on the contemplated traffic staging plan there will not be any unusual loads on the existing 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 m based on the GA drawing. 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/sand and silt/sandy silt/sand and gravel) 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 7.2 m to 8.7 m below existing grade with Elevation ranging between 155.6 m to 157.4 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 would be constructed, the abutments would utilize shallow spread footings founded on hard clayey silt on the west abutment, very dense sandy silt/hard clayey silt on the east abutment and hard clayey silt on the center piers. For completeness, several foundation options for support of abutments and piers were analyzed for this report, including micro piles 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 3.9 m, 3.4 m and 5.0 m in width for the abutment, pier and wing wall, respectively. 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)
West Abutment (32-3, BH22-6-1)	Hard clayey silt	3.9	~155.7	675	450
West Wingwall (32-3)	Very dense sandy silt to silty sand	5.0	~155.7	725	375
Pier (32-2)	Hard clayey silt	3.4	~155.7	600	475
East Abutment (32-1, BH22-6-2)	Very dense sandy silt (North) / Hard clayey silt (South)	3.9	~155.7	675	450
East Wingwall (32-1)	Very stiff to hard clayey silt	5.0	~155.7	675	400

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)
West Abutment (32-3, BH22-6-1)	Hard clayey silt	5.0	~155.7	725	375
East Abutment (32-1, BH22-6-2)	Very dense sandy silt (North) / Hard clayey silt (South)	5.0	~155.7	725	375

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[1] (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	~160.8 to 159.8	>1.0	Firm to very stiff clayey silt fill	280	150
East and West Abutment	~160.3 to 159.3	>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 δ)=0.6
Between cast-in-place concrete and compacted earth fill	Coefficient of friction (tan δ)=0.45
Between pre-cast concrete and engineered fill	Coefficient of friction (tan δ)=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 east and west abutments indicate that very dense sand/sand and silt/sandy silt or hard clayey silt (SPT 'N' values of greater than 100 blows per 300 mm) extend to a minimum Elevation of approximately 151 m. As noted previously in Section 6.2, top of groundwater is interpreted to be about 7.2 m to 8.7 m below existing grade with Elevation ranging between 155.6 m to 157.4 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 is 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 to sandy silt. Deep foundation options provide greater control of settlements over shallow foundations, if tie-in between the potential structure and existing structure is sensitive to differential settlements.

8.2.2.2 Micropiles

The proposed remedial works are expected to maintain the current loading condition, resulting in no anticipated additional loading. Should design indicate a loading of greater than approximately 10% and additional geotechnical bearing resistance be required, micropiles may be incorporated into the existing foundation structure. 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 sandy silt/sand or hard clayey 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[1]		FHWA Type C Micropile[1]		Bond Zone Stratum
		Ultimate Adhesion (kPa)	Factored Ultimate Adhesion (kPa)	**Ultimate Adhesion (kPa)**	Factored Ultimate Adhesion (kPa)	
West Abutment	(32-3, BH22-6-1)	130	55	145	65	Hard clayey silt
Pier	(32-2)	130	55	145	65	Hard clayey silt
East Abutment	(32-1, BH22-6-2)	200	80	215	85	Very dense sandy silt to sand
		130	55	145	65	Hard clayey silt

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 sandy silt to sand or hard clayey silt.

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

8.2.2.3.1 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[1] (m)	Factored Axial Geotechnical Resistance at ULS (kN/pile)[2] HP310x110	Factored Serviceability Geotechnical Axial Resistance (kN/pile)[2,3] HP310x110	Pile Founding Stratum
West Abutment	(32-3, BH22-6-1)	151.0	4.6	1400	1100	Hard clayey silt
Pier	(32-2)	151.0	4.8	1400	1100	Hard clayey silt
East Abutment	(32-1, BH22-6-2)	151.0	4.7	1400	1100	Hard clayey silt/very dense sandy silt to sand

Notes:
(1) based on an assumed bottom of pile cap a minimum of 1.2 below frost ground surface at Markham Rd. (~El. 158 m to 159 m).
(2) values as per MTO structural office policy memo 98-01, 1998
(3) for 25 mm total settlement.

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 2.6 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 MTO Integral Abutment Design Manual. A standard detail is provided in Appendix G. The piles should be set in the inner 600 mm CSP pipe, with the annual 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:
$K_{py} = 67Cu/d$

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)
Cu	Undrained Shear Strength (kPa)

As an alternative, the resistance to lateral load in front of a vertical pile may be calculated using the following geotechnical design parameters to determine a PY curve (Lateral deflection Vs resistance). The following Table 2.8 presents the estimated soil properties and their geotechnical parameters for abutments and piers. The data presented in the tables can be used for lateral load analyses using 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
γ	bulk unit weight (kN/m3)
ϕ	internal friction angle (deg)
δ	friction angle between steel pile and soils (deg)
ε_{50}	strain corresponding to 50% of the maximum principal stress difference
Kp	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 for s/b ≥ 3.75
e= 0.64 (s/b)0.34 for 1 (s/b)0<3.75

Reduction factors for leading piles in a line
e = 1 for s/b ≥ 4.0
e= 0.7 (s/b)0.26 for 1 s/b/b <4.0

Reduction factors for trailing piles in a line
e = 1 for s/b ≥ 7.0
e= 0.48 (s/b)0.38 for 1 (s/b)0<7.0

The notations used in the table are defined below:

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

Table 2.8: Parameters for lateral load analyses

Strata	Elevation (m)	Type of Soil	N_{SPT}	γ (kN/m³)	c_u (kPa)	ϕ (°)	δ (°)	K_{py} (MN/m³)		ε_{50}	n_h (MN/m³)	K_p
								Static	Cyclic			
Granular Fill	-	Cohesionless	-	21.0	-	30	14	10.0	10.0	-	6.6	3.0
West Abutment – (32-1, BH22-6-1)												
Clayey silt (stiff to hard)	157.7 – 151.0	Cohesive	11 - >50	22.8	75 - >200	-	-	135.0	55.0	0.007	-	1.0
Centre Pier (32-2)												
Clayey silt (hard)	157.4 – 152.0	Cohesive	>50	22.8	>200	-	-	135.0	55.0	0.007	-	1.0
East Abutment – North Side (BH22-6-2)												
Sandy silt to sand (compact to very dense)	158.2 – 151.0	Cohesionless	16 - >50	22.8	-	34	12	40.0	40.0	-	12.5	3.5
East Abutment – South Side (32-3)												
Sandy silt to silty sand (very dense)	157.3 – 154.4	Cohesionless	>50	22.8	-	34	12	40.0	40.0	-	12.5	3.5
Clayey silt (hard)	154.4 – 151.0	Cohesive	>50	22.8	>200	-	-	135.0	55.0	0.007	-	1.0

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. 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
   Pre-consolidation 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 Analyzer must be performed near the beginning of the pile driving phase of construction to confirm the pile capacities. Ten 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 soil/glacial tills below the GWT), relaxation after initial pile driving is possible. In the field, a number of piles should be monitored with the Pile Driving Analyzer for the end of initial driving and restrike conditions to check for relaxation as well as to confirm the ultimate bearing capacity of the piles. If the termination levels of adjacent piles penetrate deeper than a 3 horizontal to 2 vertical lines drawn down from the toe of the previously driven higher piles, the higher piles should be re-driven to the established penetration resistance. During the driving of piles in a group, the vertical elevation of the piles should be monitored. If more than 5 mm of heaving occurs during the driving of adjacent piles, the heaved piles should be re-driven to the established penetration resistance. Additionally, selected piles should be restruck to check for relaxation. The actual amount of restriking should be 10% or a minimum of 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 conducted to determine the driving requirements.

MTO permits the control of pile installation using the 'Hiley Formula'. If this method is chosen to control the pile installation, 'Hiley Formula' can apply in similar settings as shown on MTO standard drawings SS103-11 'Pile Driving Control'. Based on MTO experience with the Hiley formula, a resistance factor equal to 0.5 may be used on the ultimate resistance to verify the factors ULS design values. Assessment of the ultimate geotechnical resistance by the Hiley formular 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 2.4 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 formular 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/down drag 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 down drag 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 stiff to hard clayey silt and compact to very dense sand and silt/sand/sandy silt. Bedrock was not encountered within the investigated depth. The groundwater level is at about 1.5 m to 3.5 m depth below the existing Markham Road grade. The reported N-values for the native soils ranged from 11 to 155 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*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*PGA) are all equal to 1.00.

From Natural Resources Canada website, 2020 NBC seismic hazard values are obtained using the site location coordinates (43.785377°N, 79.235209°W), where the damped spectral accelerations are Sa(0.2)=0.330g, Sa(0.5)=0.200g, Sa(1.0)=0.105g, Sa(2.0)=0.049g the site-adjusted peak ground acceleration (PGA) is 0.179 g (g = acceleration due to gravity -9.81 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.

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 of abutments and native deposits. Difficulties with installation may occur where occasional cobbles and boulders are encountered in the fill (i.e., cobbles/boulders were not encountered in the boreholes drilled during this investigation, however auger grinding experienced during drilling through the fill might suggest the presence of cobbles and boulders), requiring their removal before further driving or 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	• Appropriate for shallow and deep installation • Easy to install through potential obstructions	• May require bracing/ tieback anchors depending on depth of excavation into overburden	• Low cost of construction	• Piles could be long • Potential for loss of soil through laggings	1
Driven Steel Sheet Piling	• Straightforward installation	• Possible obstructions within fill which may affect driving	• More expensive	• Installation may be difficult if obstructions are encountered in the fill	2

Timber lagging may be sized as per Table 20.12 of the CFEM, 5th edition (Section 20.8.9). This is provided so the center-to-center spacing of the soldier piles does not exceed 3.0 m. Soldier piles should extend a minimum depth of 3.0 m below the planned excavation depth (~8 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 8.4.1 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 clayey silt fill and compact to very dense sand/sand and silt/sandy silt/sand and gravel 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. Pilling 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:

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

where:
P = earth pressure intensity at depth h, kPa
K = earth pressure coefficient
γ = unit weight of retained soil, kN/m^3
q = surcharge near wall, kPa
h = depth to point of interest, m
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 ϕ' (o)	Coefficient of Lateral Earth Pressure(1) (K_a)	(K_p)	(K_o)	Unit Weight γ (kN/m³)	GWL (m)
West	165.0 to 163.4	Sand and gravel to sand fill (compact to very dense)	34	0.28	3.54	0.44	22.0	157.3
	163.4 to 158.4	Clayey silt fill (firm to very stiff)(2)	30	0.33	3.00	0.50	21.0	
	158.4 to 153.2	Clayey silt (stiff to hard)(2)	34	0.28	3.54	0.44	22.8	
	153.2 to 151.1	Sandy silt (very dense)	34	0.28	3.54	0.44	22.8	
East	164.1 to 163.3	Sand to sand and gravel fill (compact to very dense)	34	0.28	3.54	0.44	22.0	155.6 to 157.4
	163.3 to 158.2	Clayey silt fill (stiff to hard)(2)	30	0.33	3.00	0.50	21.0	
	158.2 to 151.8	Sandy silt (compact to very dense)	34	0.28	3.54	0.44	22.8	

Notes:
1. K_a = *active earth pressure coefficient;* K_p= *passive earth pressure coefficient;* K_o = *coefficient of earth pressure at rest*
2. *Assumes long term conditions. In short term conditions* $K_a = K_p = 1$

8.5 Structure Backfill

The selection and placing of backfill should be in accordance with OPSS.PROV 902, OPSD.3101.150 and OPSD.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 the 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 restraining walls such as boxes.
3. The mobilization of full active or passive resistance requires a measurable and perhaps significant wall movement or rotation. For active earth pressure, a rotation of 0.002 about the base of vertical walls (horizontal displacement divided by wall height) or translation of 0.001 times wall height or a combination of these is required. Therefore, unless the structural element can tolerate these deflections, the at-rest earth pressure should be used in the design.
4. For walls backfilled using granular materials in accordance with Case (b), the parameters (unfactored) given in Table 2.11 may be assumed.

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

Material	Unfactored Friction Angle ϕ' (°)	Coefficient of Active Earth Pressure (K_a)	Coefficient of Passive Earth Pressure (K_p)	Coefficient of Earth Pressure at Rest (K_o)	Unit Weight γ (kN/m³)
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)*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 0.179 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 ϕ' (°)	Coefficient of Seismic Earth Pressure - Active (K_{ae})	Coefficient of Seismic Earth Pressure - Passive (K_{pe})	Unit Weight γ (kN/m^3)
Compacted Granular A or Granular B Type II	35	0.32	3.51	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 calculating the seismic lateral earth pressure coefficient. For design purposes, the following unfactored seismic lateral earth pressure parameters for non-yielding walls are provided in Table 2.13.

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

Material	Unfactored Friction Angle ϕ' (°)	Coefficient of Seismic Earth Pressure - Active (K_{ae})	Coefficient of Seismic Earth Pressure - Passive (K_{pe})	Unit Weight γ (kN/m^3)
Compacted Granular A or Granular B Type II	35	0.38	3.33	22.8
Compacted Granular B Type I	32	0.42	2.91	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 m below the roadway (Elevation 160.3 m to 159.3 m) 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 Markham 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 7.2 m to 8.7 m below existing grade of Highway 401 with Elevation ranging between 155.6 m to 157.4 m across the Markham 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 160.3 m to 159.3 m which would be above the groundwater level (~Elevation 157.4 m to 155.6 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 infrastructure is planned at this site. However, for completeness, the analyses results have been discussed here. The analyses' results are summarized in Table 1.10.

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 7.24 to 7.84, 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 low (540 to 4100 ohm-cm) resistivity of tested soil, which suggests the severe to moderate potential for corrosion of buried metallic elements as per Table 3.2 of the MTO Gravity Pipe Design Guideline. Therefore, some level of corrosion protection for buried metallic elements is required, depending upon the material type. The test results provided in Table 1.10 may be used to aid in the selection of coatings and corrosion protection systems for buried steel objects. The measured chloride content was between 90 ppm (µg/g) to 1000 ppm which also indicates some 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 34 ppm (µg/g), i.e., 0.0034% 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 were not encountered during EXP's geotechnical investigation; however, it is noted that the presence may cause difficulties during installation. If encountered, 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 of creating a baseline of pre-existing conditions and defects. Expected structures include the existing Highway 401 roadway and accruements including the pavement surface, traffic barriers, and overhead lighting, the existing Markham Road overpass structure, Markham Road including all accruements, and potential existing utility infrastructure.

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

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

8.11.2 Movements of Existing Structure

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

- Install survey points along the existing bridge (min 6 m c/c) and the existing adjacent abutment and bridge deck (min 5m c/c).
- The location of survey points is to be coordinated with the construction team to prevent conflict during the proposed works.
- Monitoring frequency will be:
  - Preconstruction: Minimum 3 baseline readings, one month prior to construction

- During construction: Daily readings during active construction.
- Post construction: Biweekly after completion and then after four weeks, if there is little to no settlement continue surveying once a month for three months; or until the engineer is satisfied with performance.

▪ The criteria for evaluation of settlement shall be based on the following action levels:

Structure Limits:

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

Pavement Surface Limits:

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

8.11.3 Movements of Temporary Protection Systems

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

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

1. Review Level: If a maximum value of 15 mm relative to the baseline readings is reached, the method and rate or sequence of construction shall be reviewed or modified to mitigate further ground displacements.
2. Alert Level: If a maximum of 25 mm relative to the baseline readings is reached, the Contractor shall be required to cease construction operation or to execute pre-planned measures to secure the site to mitigate further unacceptable settlement and to assure safety of public.

8.11.4 Vibration

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

The suggested vibration monitoring plan is described in the following.

1. Vibration monitoring should be conducted to verify the vibration levels near the existing structure and the utilities identified in the area.

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

The limits are as follows:

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.

9.0 Closure

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

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

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

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

Yours truly,

EXP Services Inc.

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DEC. 20, 2024
PROVINCE OF ONTARIO

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Encl.

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Appendix A – Limitations and Use of Report

LIMITATIONS AND USE OF REPORT

BASIS OF REPORT

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

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

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

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

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

RELIANCE ON INFORMATION PROVIDED

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

STANDARD OF CARE

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

COMPLETE REPORT

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

USE OF REPORT

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

REPORT FORMAT

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

Appendix B – General Arrangement Drawings

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

Ontario Ministry of Transportation

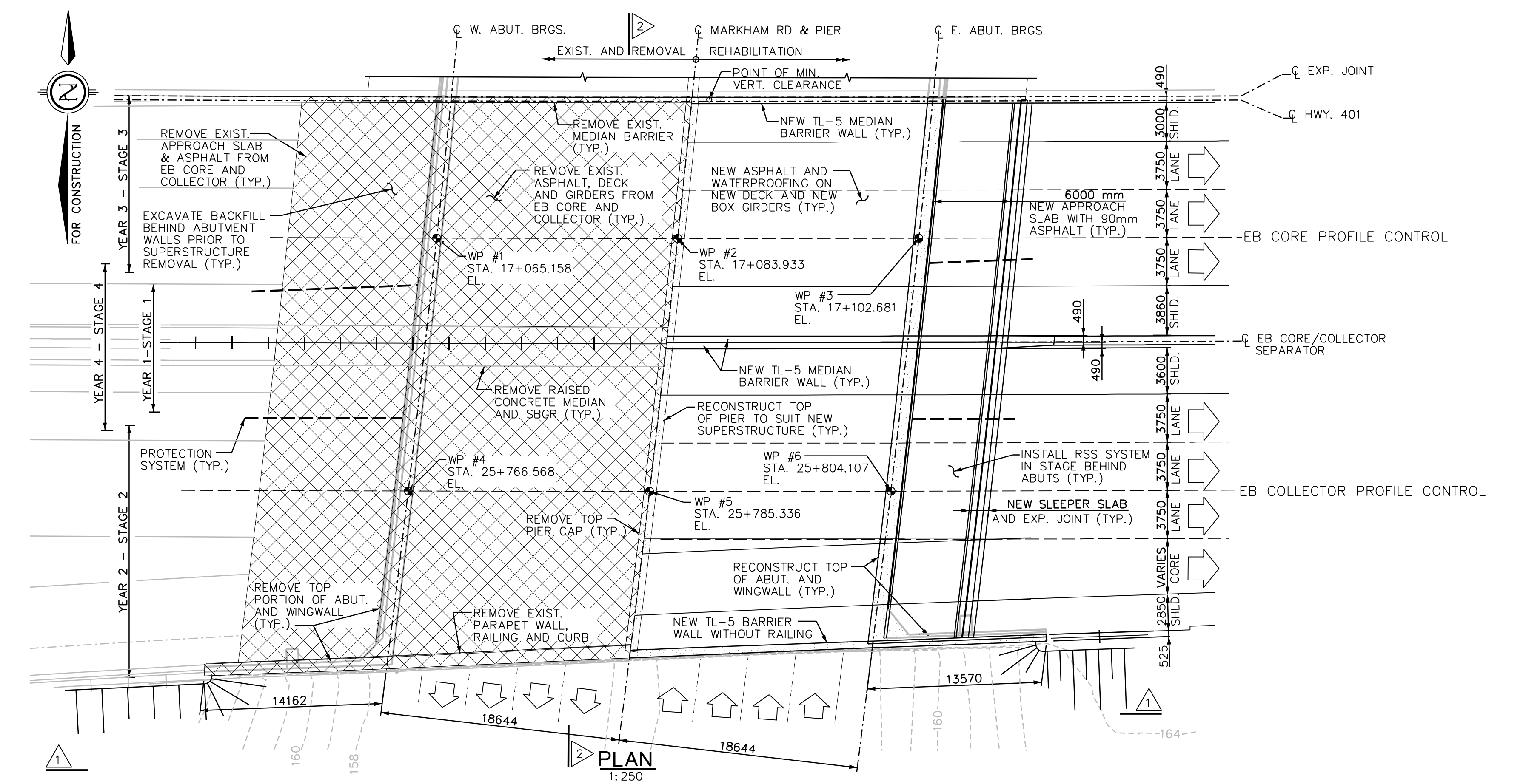
CONT
WP

HWY 401 EB CORE & COLLECTOR LANE
MARKHAM RD OP BRIDGE REHABILITATION
GENERAL ARRANGEMENT

SHEET
S127

AECOM

FOR CONSTRUCTION



PLAN
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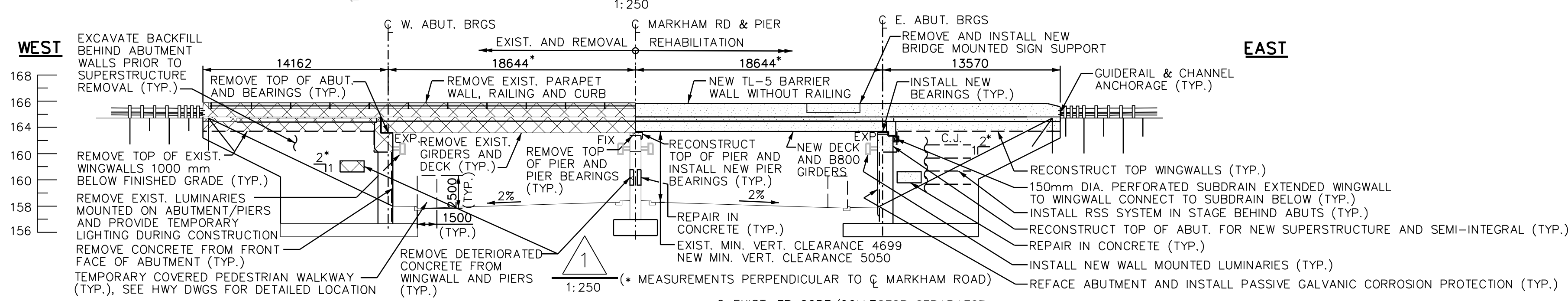

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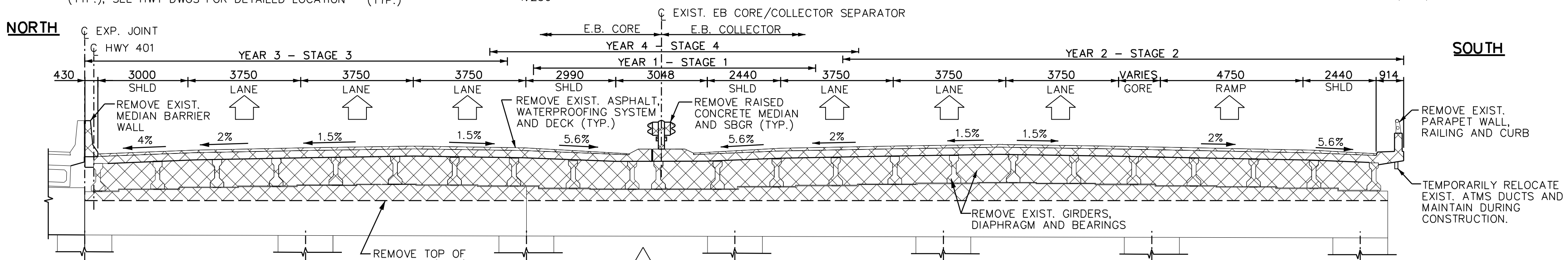
(* MEASUREMENTS PERPENDICULAR TO ℄ MARKHAM ROAD)



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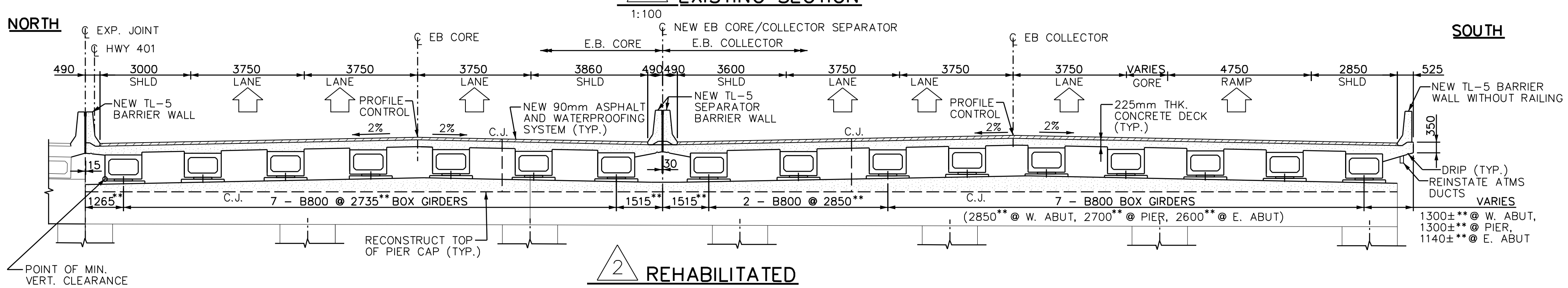

2 REHABILITATED
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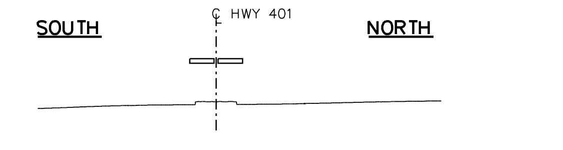
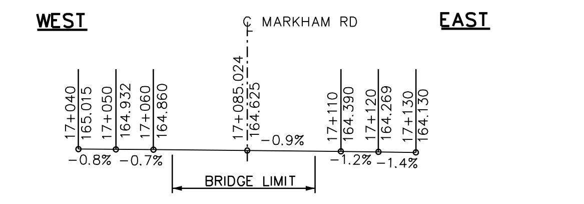
(**: MEASUREMENTS PERPENDICULAR TO ℄ PIER & ABUTMENT BRGS)



PROFILE OF MARKHAM ROAD

GENERAL TEXT
FONT 188, LEVEL 11
PLOTTED SIZE 2.5mm



PROFILE OF HWY 401 ℄ MEDIAN

LIST OF DRAWINGS:

R5–1. GENERAL ARRANGEMENT
R5–2. CONSTRUCTION STAGING – 1
R5–3. CONSTRUCTION STAGING – 2
R5–4. WEST ABUTMENT REMOVAL DETAILS
R5–5. EAST ABUTMENT REMOVAL DETAILS
R5–6. PIER REMOVAL DETAILS
R5–7. WEST ABUTMENT REHABILITATION DETAILS
R5–8. EAST ABUTMENT REHABILITATION DETAILS
R5–9. PIER REHABILITATION DETAILS
R5–10. PRESTRESSED BOX GIRDERS & BEARING LAYOUT
R5–11. PRESTRESSED BOX GIRDERS AND BEARING
R5–12. PRESTRESSED BOX GIRDERS DETAILS
R5–13. DECK LAYOUT AND SCREED ELEVATIONS
R5–14. DECK REINFORCEMENT DETAILS – I
R5–15. DECK REINFORCEMENT DETAILS – II
R5–16. SOUTH BARRIER WALL WITHOUT RAILING TL–5
R5–17. MEDIAN BARRIER WALL WITHOUT RAILING TL–5
R5–18. 6000mm APPROACH SLAB
R5–19. EXPANSION JOINT (TYPE C) AND SLEEPER SLAB
R5–20. STRIP SEAL EXPANSION JOINT – TYPE C DETAILS
R5–21. SEQUENCE OF EXPANSION JOINT INSTALLATION
R5–22. MISCELLANEOUS AND STANDARD DETAILS
R5–23. ELECTRICAL EMBEDDED WORKS

LIST OF ABBREVIATIONS

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

APPLICABLE STANDARD DRAWINGS:

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

GENERAL NOTES:

1. CLASS OF CONCRETE .... 30 MPa
   PRECAST GIRDERS .... 50 MPa
2. CLEAR COVER TO REINFORCING STEEL:
   - DECK – TOP .... 70±20
   - DECK – BOTTOM .... 40±10
   - REMAINDER .... 70±20

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

CONSTRUCTION NOTES:

1. 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.
2. TYPICAL AREAS OF REPAIRS ARE INDICATED ON THE DRAWINGS. WHERE REPAIR LIMITS ARE NOT SHOWN, LIMITS SHALL BE IDENTIFIED BY THE CONTRACT ADMINISTRATOR.
3. THE CONTRACTOR SHALL ADJUST THE BEARING SEAT ELEVATIONS AND REINFORCING STEEL TO SUIT THE ACTUAL HEIGHT OF THE BEARING SUPPLIED. THE CONTRACTOR IS RESPONSIBLE FOR PROVIDING FULL BEARING CONTACT TO 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.
4. PROTECTION SYSTEM SHALL MEET REQUIREMENTS FOR PERFORMANCE LEVEL 2. EXACT LOCATIONS AND LIMITS OF PROTECTION SYSTEM SHALL BE DETERMINED BY CONTRACTOR.
5. BACKFILL SHALL NOT BE PLACED BEHIND THE NEW SEMI–INTEGRAL ABUTMENTS UNTIL THE NEW CONCRETE HAS ACHIEVED 75% OF DESIGN COMPRESSIVE STRENGTH.
6. SAWCUT IN CONCRETE, WHERE DESIGNATED, SHALL BE 25mm DEEP OR TO THE FIRST LAYER OF REINFORCING STEEL, WHICHEVER IS LESS.
7. 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.
8. THE CONTRACTOR IS FULLY RESPONSIBLE FOR ADEQUATE PROTECTION OF ALL UTILITIES, SERVICES, ROADWAYS, ETC., DURING CONSTRUCTION OPERATIONS.
9. 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.
10. 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.
11. BACKFILL SHALL BE PLACED SIMULTANEOUSLY BEHIND BOTH DECK ENDS KEEPING THE HEIGHT OF THE BACKFILL APPROXIMATELY THE SAME. AT NO TIME SHALL THE DIFFERENCE IN ELEVATION BE GREATER THAN 300mm.

LEGEND:

EXIST. CONCRETE TO REMAIN
NEW CONCRETE
CONCRETE REMOVAL
NEW ASPHALT

REVISIONS	DATE	BY	DESCRIPTION

DESIGN S.S.	CHK U.P.	CODE CAN/CSA S6-19	LOAD CL 625-ONT	DATE AUG. 2024
DRAWN O.Z.	CHK S.S.	SITE 37X-0218/B1&B3		DWG R5-01

Appendix C – Borehole Location Plan and Stratigraphic Profile

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

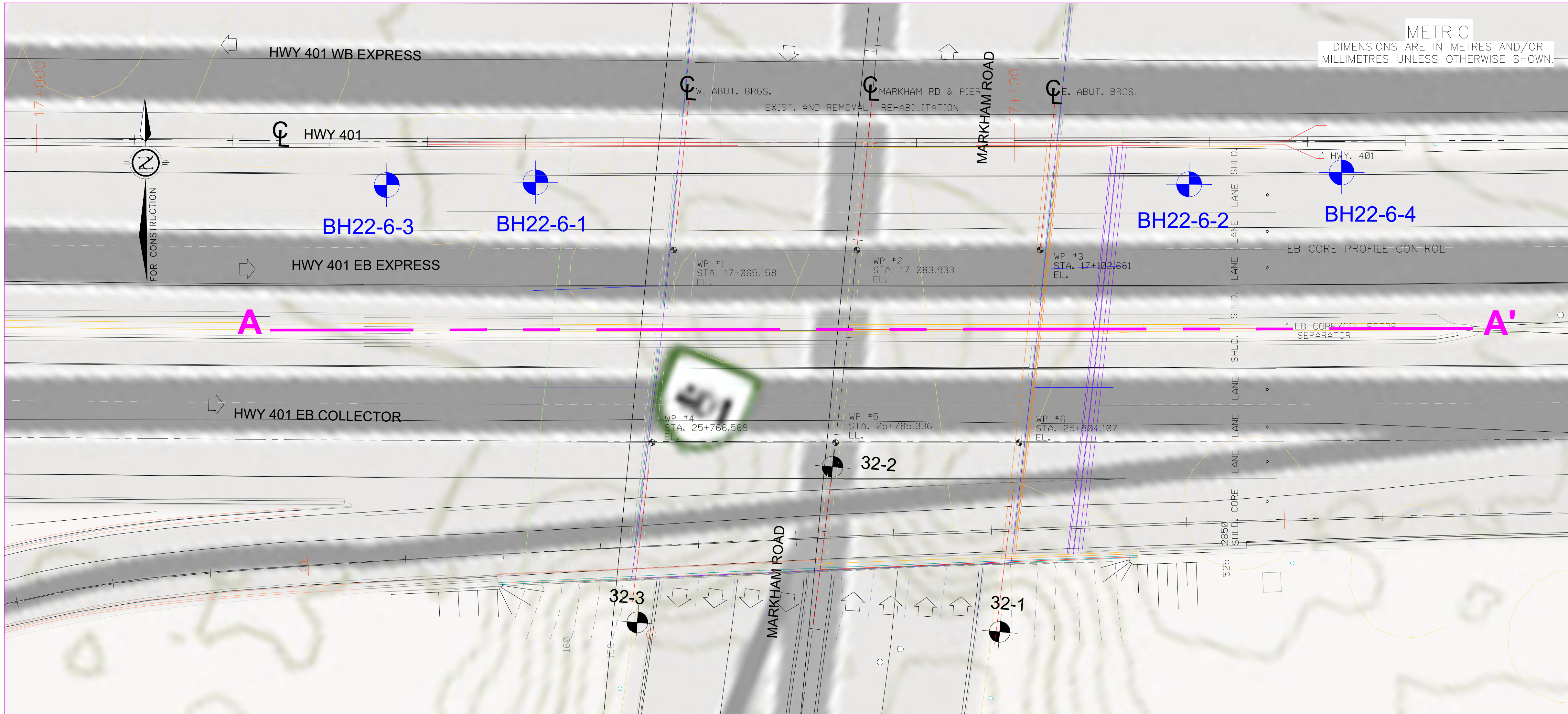
Superstructure Replacement at Markham Road Overpass
Eastbound Core and Collectors Structure
Latitude: 43.785377°; Longitude: -79.235209°

BOREHOLE LOCATION PLAN & SOIL STRATA

SHEET 1

EXP SERVICES INC.

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



PLAN

KEY PLAN
N.T.S.

LEGEND

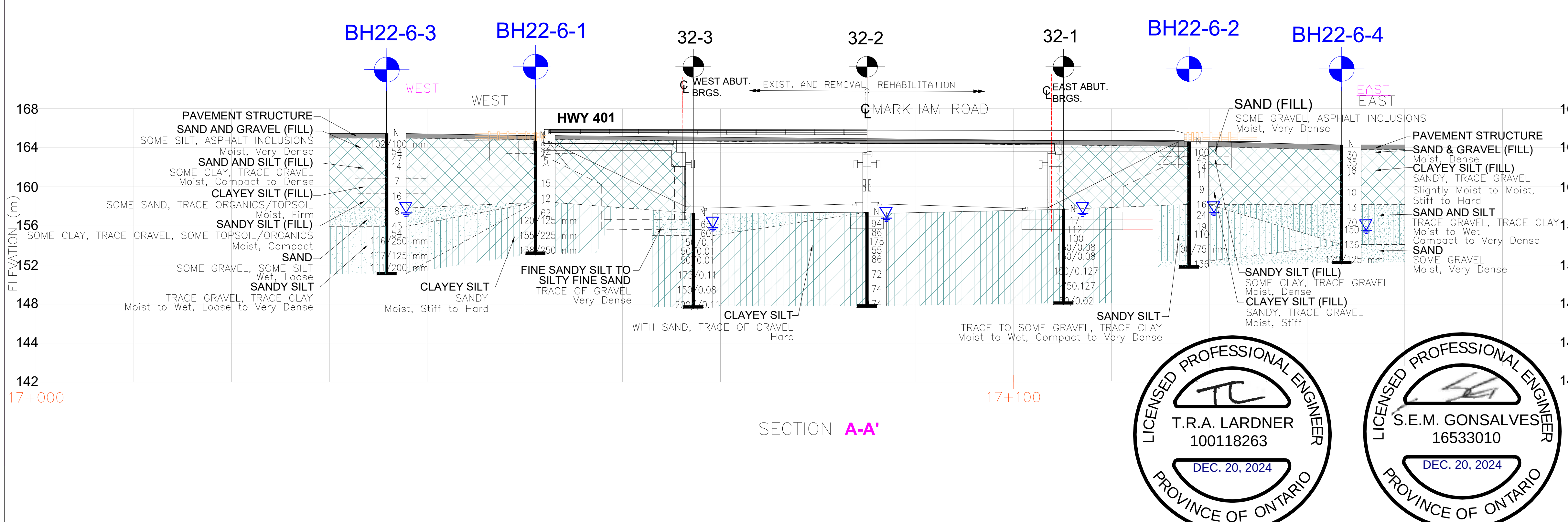
Borehole Location

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, SILT, SAND, SILT AND SAND, SANDY SILT, SILTY SAND, CLAY, CLAYEY SILT, SILTY CLAY



SECTION A-A'

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

BH No.	ELEV.	NORTHING	EASTING
BH22-6-1	165.3	4849527	326078
BH22-6-2	164.6	4849552	326140
BH22-6-3	165.4	4849521	326064
BH22-6-4	164.3	4849559	326154
32-1	157.7	4849502	326139
32-2	157.4	4849511	326117
32-3	157.3	4849489	326105

NOTES

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

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

The complete foundation investigation and design report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in the report and related documents are specifically excluded in accordance with the conditions of Section GC 2.01 of OPS Gen. Cond.

REVISIONS

NO	DATE	BY	DESCRIPTION
			SUBMISSION FOR MTO REVIEW

PROJECT No. ADM-22000797-A0		GEOCRES No. 30M14-554	
SUBM'D SH	CHKD. SM	DATE NOV. 29, 2024	SITE 37X-0218/B1 & B3
DRAWN SH	CHKD. TC	APPRD SG	DWG 01

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

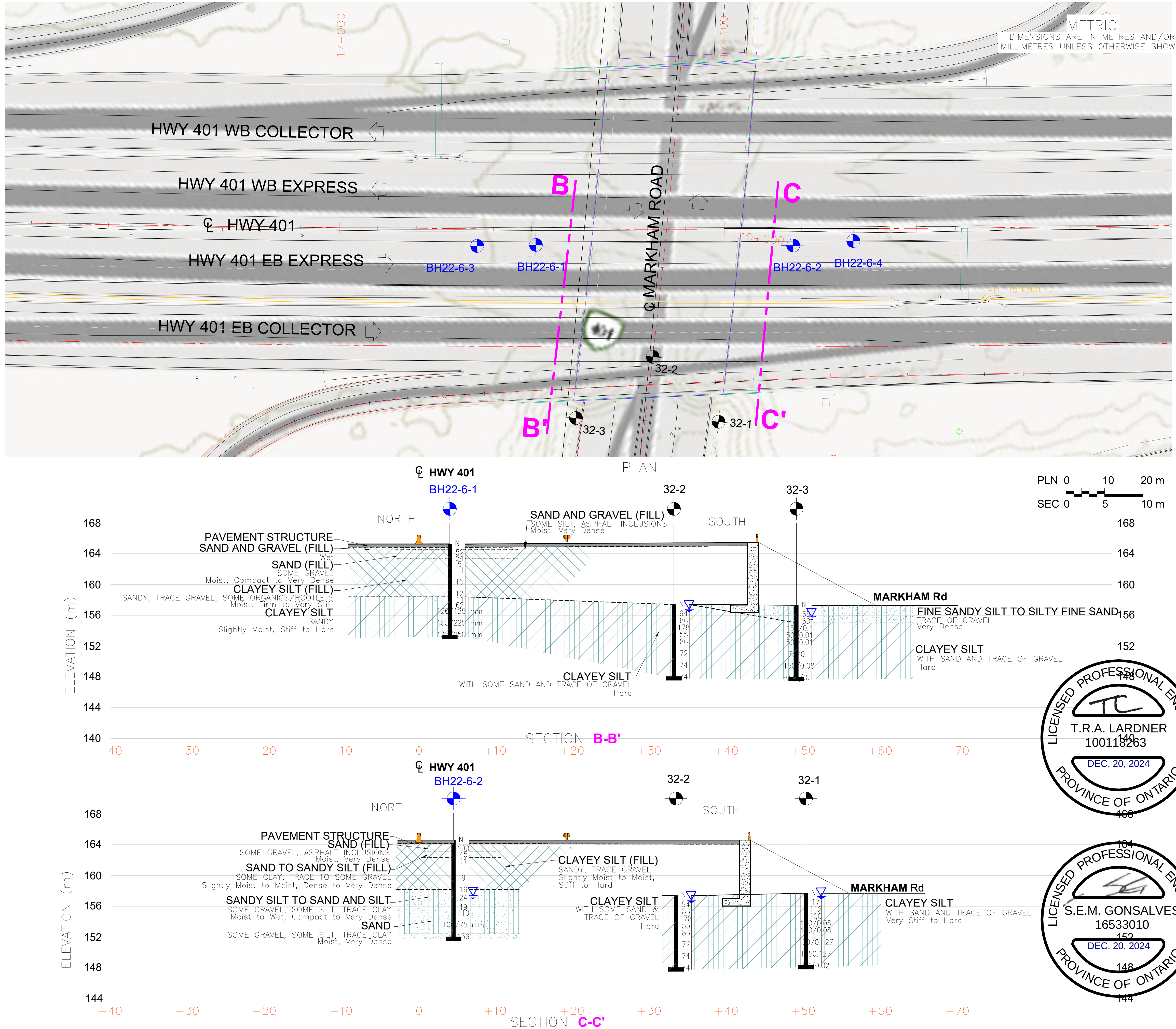
SHEET 2

Superstructure Replacement at Markham Road Overpass
Eastbound Core and Collectors Structure
Latitude: 43.785377°; Longitude: -79.235209°

BOREHOLE LOCATION PLAN & SOIL STRATA

EXP SERVICES INC.

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



KEY PLAN
N.T.S.

LEGEND

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

SOIL STRATA SYMBOLS

PAVEMENT STRUCTURE, FILL, SILT, SAND, SILT AND SAND, SANDY SILT, SILTY SAND, CLAY, CLAYEY SILT, SILTY CLAY

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

BH No.	ELEV.	NORTHING	EASTING
BH22-6-1	165.3	4849527	326078
BH22-6-2	164.6	4849552	326140
BH22-6-3	165.4	4849521	326064
BH22-6-4	162.8	4849559	326154
32-1	157.7	4849502	326139
32-2	157.4	4849511	326117
32-3	157.3	4849489	326105

NOTES

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REVISIONS NO	DATE	BY	DESCRIPTION
			SUBMISSION FOR MTO REVIEW

PROJECT No. ADM-22000797-A0		GEOCRES No. 30M14-554	
SUBM'D SH	CHKD. SM	DATE JULY 04, 2023	SITE 37X-0218/B1 & B3
DRAWN SH	CHKD. TC	APPRD SG	DWG 02

Appendix D – Borehole Logs

Explanation of Terms Used on Borehole Records

SOIL DESCRIPTION

Terminology describing common soil genesis:

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

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

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

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

Terminology describing soil structure:

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

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

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

Fissured: material breaks along plane of fracture.

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

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

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

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

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

Homogeneous: same color and appearance throughout.

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

Uniformly Graded: predominantly on grain size.

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

ISSMFE SOIL CLASSIFICATION

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

0.002 0.006 0.02 0.06 0.2 0.6 2.0 6.0 20 60 200

EQUIVALENT GRAIN DIAMETER IN MILLIMETRES

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

UNIFIED SOIL CLASSIFICATION

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

Table a: Percent or Proportion of Soil

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

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

Table b: Apparent Density of Cohesionless Soil

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

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

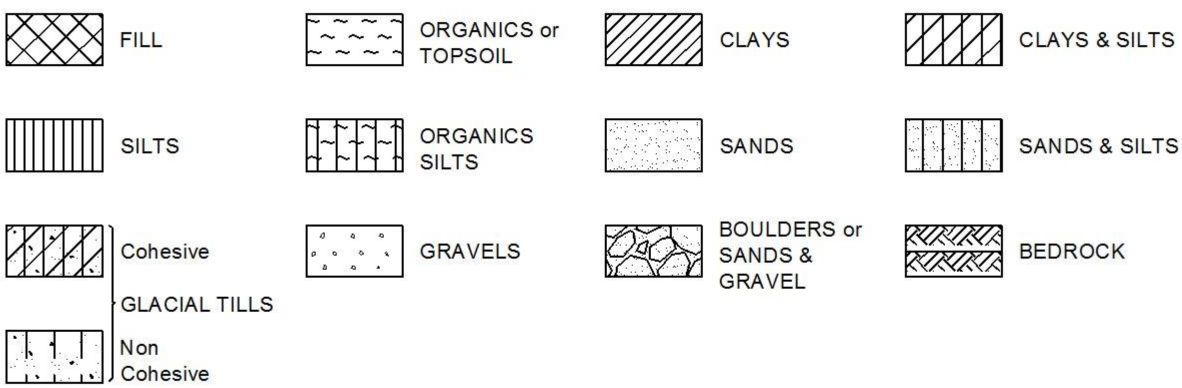
Table c: Consistency of Cohesive Soil

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

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

STRATA PLOT

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



WATER LEVEL MEASUREMENT

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

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

STRESS AND STRAIN

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

MECHANICALL PROPERIES OF SOIL

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

PHYSICAL PROPERTIES OF SOIL

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

RECORD OF BOREHOLE No BH22-6-1

METRIC

W.P. Site 37X-0218/B1 & B3	LOCATION Hwy 401 - Markham Rd. O/P, Toronto, ON, MTM ON-10 326078.3E 4849527.5N		ORIGINATED BY EL
DIST Toronto HWY 401	BOREHOLE TYPE Truck Mount CME 75 / SSA		COMPILED BY EL
DATUM Geodetic	DATE 2022.09.13 - 2022.09.13	LATITUDE 43.785404 LONGITUDE -79.235629	CHECKED BY SM/TL

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
165.3	
0.0	**PAVEMENT STRUCTURE** - 75 mm of asphalt and 325 mm of concrete
164.8 / 0.4	**SAND AND GRAVEL (FILL)** - brown, wet
164.5 / 0.8	**SAND (FILL)** - some gravel, greyish brown, slightly moist, compact to very dense
163.4 / 1.8	**CLAYEY SILT (FILL)** - sandy, trace gravel, trace organics/rootlets, brownish grey to grey with black inclusions, slightly moist to moist, firm to very stiff
158.4 / 6.9	**CLAYEY SILT** - sandy, light brown to grey, slightly moist, stiff to hard
153.2 / 12.0	**END OF BOREHOLE**

NOTES:
1) Borehole terminated at 12.0 m depth at refusal (N>100 blows over 1.5 m interval).
2) No groundwater was encountered in open borehole upon completion of drilling.

SAMPLES

NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ (kN/m³)	GR	SA	SI	CL	REMARKS
AS1	AS							
SS2	SS	52						
SS3	SS	24						
SS4	SS	5	21.3	2	52	33	13	
SS5	SS	11	21.0					Corrosivity Sample
SS6	SS	15	23.1	7	38	40	15	
SS7	SS	12	21.7					
SS8	SS	11	21.9					
SS9	SS	62	22.8	0	24	58	18	
SS10	SS	120/ 125 mm						
SS11	SS	155/ 225 mm						
SS12	SS	138/ 250 mm						

GROUND WATER CONDITIONS

ELEVATION SCALE: 165, 164, 163, 162, 161, 160, 159, 158, 157, 156, 155, 154

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × P. PENETROMETER: 20 40 60 80 100

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L

WATER CONTENT (%): 20 40 60

REMARKS & GRAIN SIZE DISTRIBUTION (%): GR SA SI CL

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

RECORD OF BOREHOLE No BH22-6-2

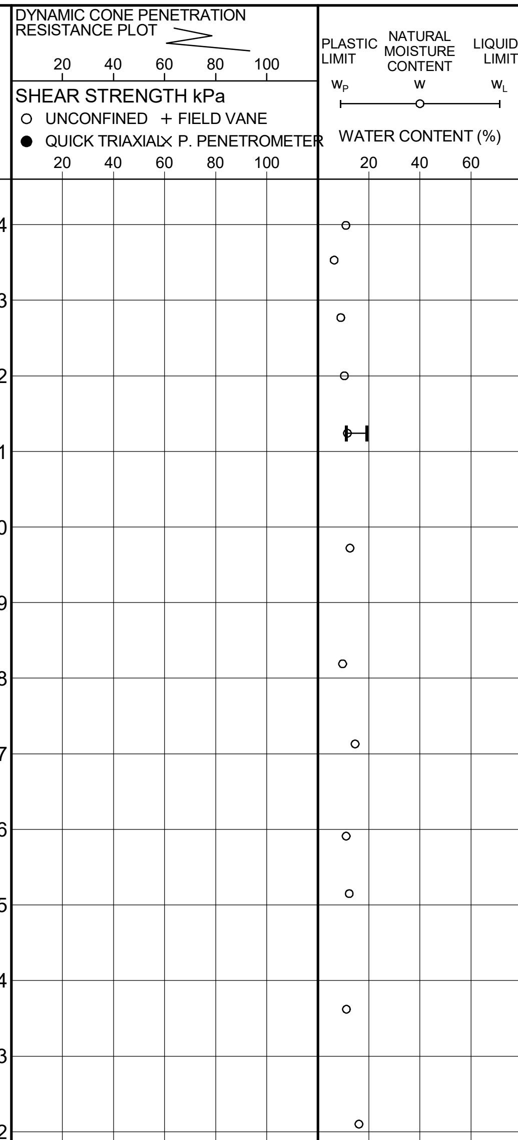
METRIC

W.P. Site 37X-0218/B1 & B3 LOCATION **Hwy 401 - Markham Rd. O/P, Toronto, ON, MTM ON-10 326140.5E 4849551.7N** ORIGINATED BY EL

DIST Toronto HWY 401 BOREHOLE TYPE Truck Mount CME 75 / SSA COMPILED BY EL

DATUM Geodetic DATE 2022.09.18 - 2022.09.18 LATITUDE 43.78562 LONGITUDE -79.234856 CHECKED BY SM/TL

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
164.6 / 0.0	**PAVEMENT STRUCTURE** - 100 mm of asphalt and 350 mm of concrete						
164.1 / 0.5	**SAND (FILL)** - some gravel, asphalt inclusions encountered, greyish brown, slightly moist to moist, very dense	AS1	AS				
		SS2	SS	100		23.1	
163.1 / 1.5	**SANDY SILT (FILL)** - some clay, trace gravel, greyish brown, slightly moist, dense	SS3	SS	45			
162.3 / 2.3	**CLAYEY SILT (FILL)** - sandy, trace gravel, dark brown to grey, slightly moist to moist, stiff	SS4	SS	14			
		SS5	SS	11			3 40 43 14
		SS6	SS	9			Corrosivity Sample
	- Topsoil/organics observed in soil cuttings from 4.6 m to 6.3 m						
158.2 / 6.4	**SANDY SILT** - trace to some gravel, trace clay, grey, moist to wet, compact to very dense	SS7	SS	16			
					∇	23.3	
		SS8	SS	24			0 32 60 8
		SS9	SS	19			
		SS10	SS	110			
		SS11	SS	100/ 75 mm			
152.4 / 12.2	**SAND** - some silt, trace clay, grey, moist, very dense					22.9	
		SS12	SS	136			0 85 12 3
151.8 / 12.8	**END OF BOREHOLE**						



NOTES:
1) Borehole terminated at 12.8 m depth at refusal (N>100 blows over 1.5 m interval).
2) Groundwater inferred at a depth of 7.2 m (Elev. 157.4 m) based on wet split spoon retrieved during drilling.

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

RECORD OF BOREHOLE No BH22-6-3

METRIC

W.P. Site 37X-0218/B1 & B3	LOCATION Hwy 401 - Markham Rd. O/P, Toronto, ON, MTM ON-10 326063.7E 4849521.4N		ORIGINATED BY EL
DIST Toronto HWY 401	BOREHOLE TYPE Truck Mount CME 75 / SSA		COMPILED BY EL
DATUM Geodetic	DATE 2022.09.14 - 2022.09.14	LATITUDE 43.785349 LONGITUDE -79.235811	CHECKED BY SM/TL

SOIL PROFILE | SAMPLES | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) | SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × P. PENETROMETER (20 40 60 80 100) | PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L — WATER CONTENT (%) (20 40 60) | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ (kN/m³)	GR	SA	SI	CL
165.4										
0.0 / 165.0	**PAVEMENT STRUCTURE** - 75 mm of asphalt and 375 mm of concrete									
0.5	**SAND AND GRAVEL (FILL)** - some silt, asphalt inclusions encountered, greyish brown, moist, very dense	AS1	AS							
		SS2	SS	102/ 100 mm						
		SS3	SS	54						
163.1 / 2.3	**SAND AND SILT (FILL)** - some clay, trace gravel, brownish grey, slightly moist, compact to dense	SS4	SS	47		23.0				
		SS5	SS	14		22.3	5	37	40	18
160.8 / 4.6	**CLAYEY SILT (FILL)** - some sand, trace organics/topsoil, grey with light brown and black inclusions, slightly moist, firm	SS6	SS	7						
159.3 / 6.1	**SANDY SILT (FILL)** - some clay, trace gravel, trace topsoil/organics, grey with black inclusions, slightly moist, compact	SS7	SS	16						
157.8 / 7.6	**SAND** - some gravel, some silt, grey, wet, loose	SS8	SS	8	∇		11	66	(23)	
156.0 / 9.5	**SANDY SILT** - trace gravel, trace clay, grey, slightly moist to wet, dense to very dense	SS9	SS	45						
		SS10	SS	54		23.5	Non-plastic 1	30	60	9
		SS11	SS	116/ 250 mm		22.6				
		SS12	SS	117/ 125 mm						
		SS13	SS	111/ 200 mm		20.1				
151.1 / 14.3	**END OF BOREHOLE**									

Elevation scale: 165, 164, 163, 162, 161, 160, 159, 158, 157, 156, 155, 154, 153, 152

NOTES:
1) Borehole terminated at 14.3 m depth at refusal (N>100 blows over 1.5 m interval).
2) Groundwater level measured at a depth of 8.1 m (Elev. 157.3 m) below the ground surface upon completion of drilling.

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

RECORD OF BOREHOLE No BH22-6-4

METRIC

W.P. Site 37X-0218/B1 & B3 — LOCATION Hwy 401 - Markham Rd. O/P, Toronto, ON, MTM ON-10 326153.7E 4849559.0N — ORIGINATED BY EL

DIST Toronto HWY 401 — BOREHOLE TYPE Truck Mount CME 75 / SSA — COMPILED BY EL

DATUM Geodetic — DATE 2022.09.19 - 2022.09.19 — LATITUDE 43.785685 — LONGITUDE -79.234692 — CHECKED BY SM/TL

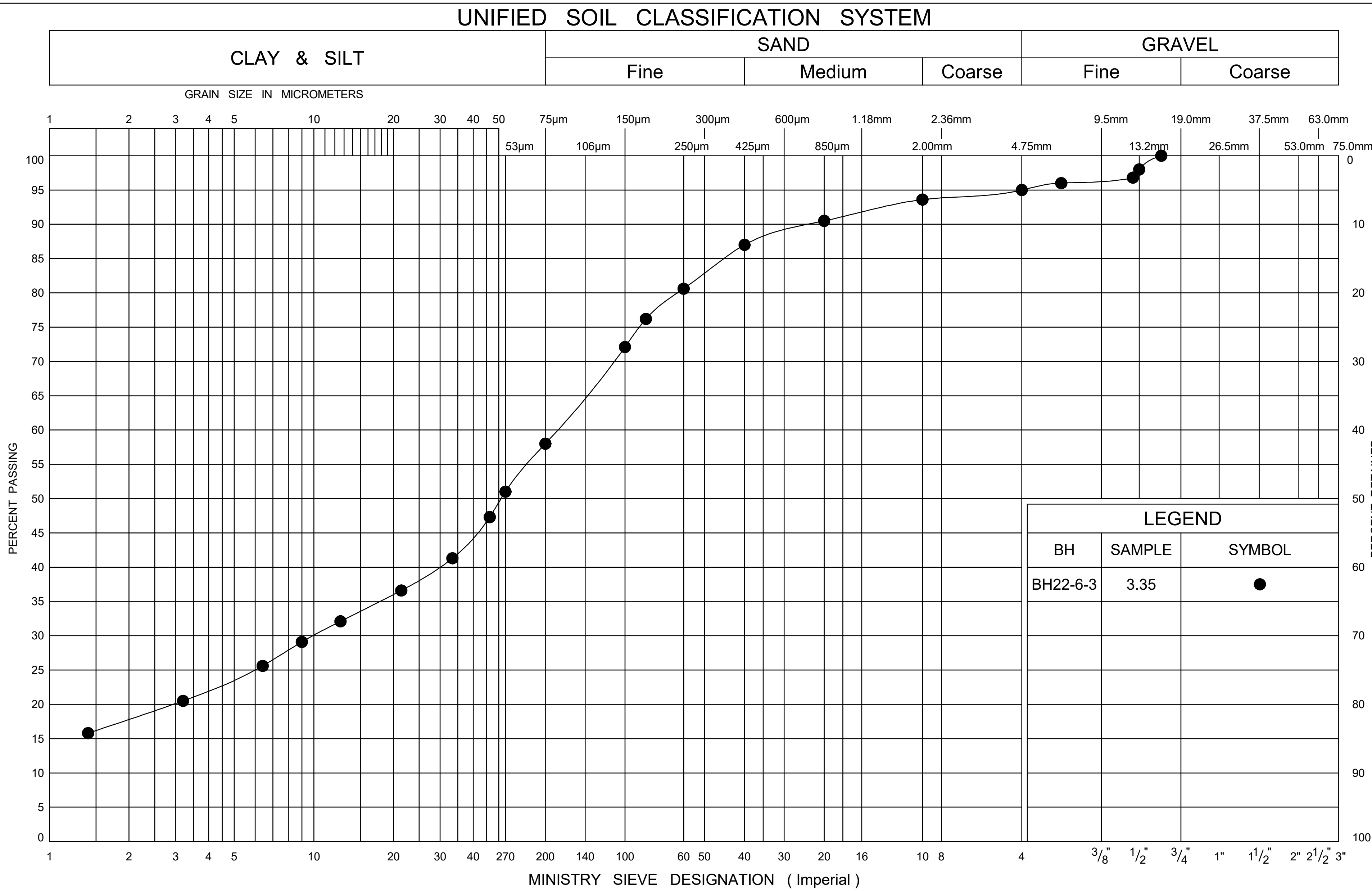
ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ (kN/m³)	GR	SA	SI	CL
164.3										
0.0 / 163.8	**PAVEMENT STRUCTURE** - 90 mm of asphalt and 375 mm of concrete									
0.5 / 163.3	**SAND AND GRAVEL (FILL)** - brown, moist, dense	AS1	AS							
		SS2	SS	30						
1.1	**CLAYEY SILT(FILL)** - sandy, trace gravel, brown to grey, slightly moist to moist, stiff to hard	SS3	SS	35			3	41	42	14
		SS4	SS	18						
		SS5	SS	11						
		SS6	SS	10		22.4	4	43	40	13
158.2 / 6.1	**SAND AND SILT** - trace gravel, trace clay, grey, moist to wet, compact to very dense	SS7	SS	13						
		SS8	SS	70			2	46	46	6
		SS9	SS	150	▽					
154.1	- becoming gravelly									
10.2	**SAND** - some gravel, grey, moist, very dense	SS10	SS	136						
152.3		SS11	SS	120/ 125 mm		22.9				
12.0	**END OF BOREHOLE**									

NOTES:
1) Borehole terminated at 12.0 m depth at refusal (N>100 blows over 1.5 m interval).
2) Groundwater level measured at a depth of 8.7 m (Elev. 155.6 m) below the ground surface upon completion of drilling.

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; ELEVATION SCALE (164, 163, 162, 161, 160, 159, 158, 157, 156, 155, 154, 153); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × P. PENETROMETER 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60; REMARKS & GRAIN SIZE DISTRIBUTION (%)

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

Appendix E – Laboratory Data



Ministry of Transportation
Ontario

GRAIN SIZE DISTRIBUTION

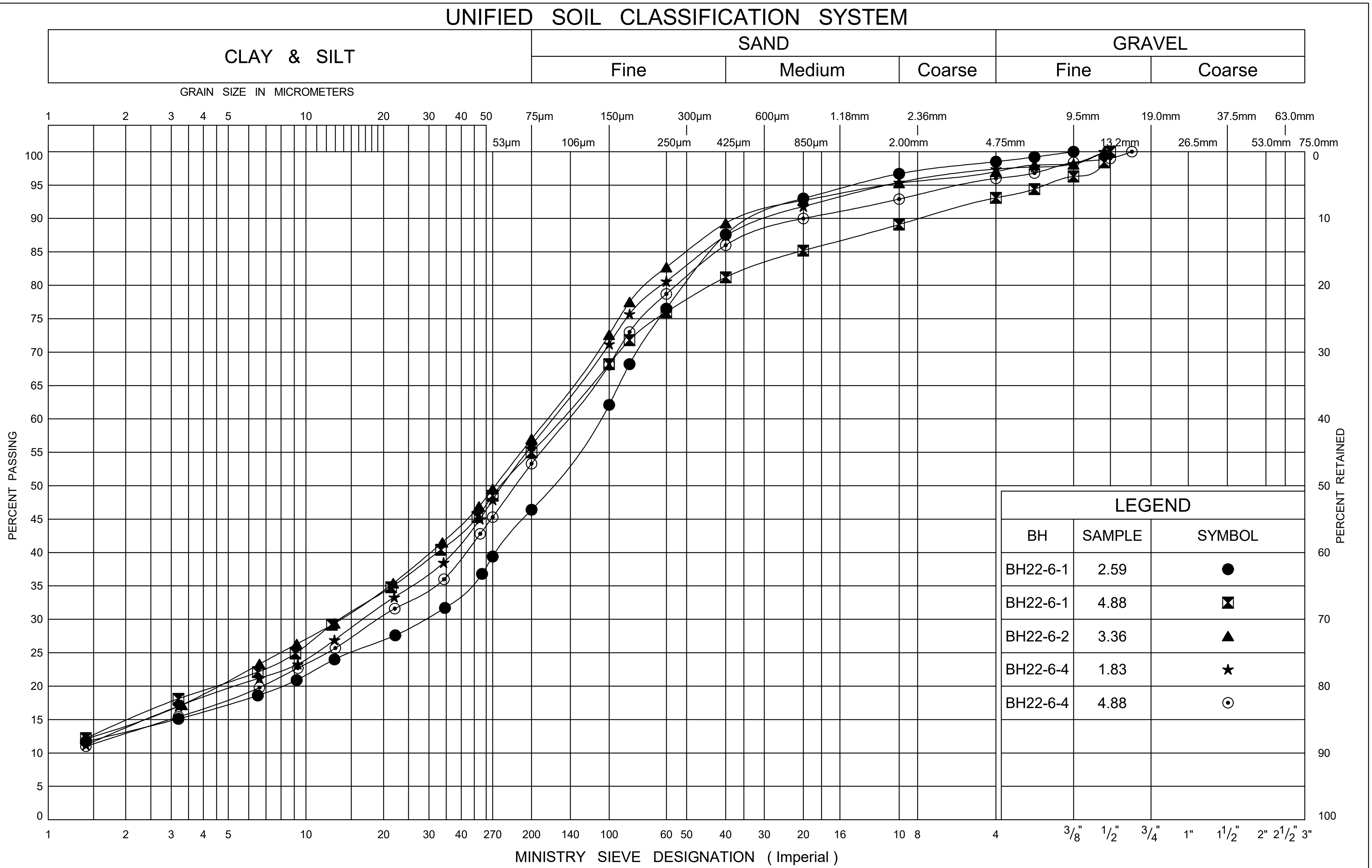
Cohesionless Fill: Sand/Sand and Silt/Sandy Silt

FIG No 1

W P Site 37X-0218/B1 & B3

Hwy 401 - Markham Rd. O/P

ONTARIO MOT GRAIN SIZE H401 - MARKHAM.GPJ ONTARIO MOT.GDT 3/8/23



LEGEND

BH	SAMPLE	SYMBOL
BH22-6-1	2.59	●
BH22-6-1	4.88	⧗
BH22-6-2	3.36	▲
BH22-6-4	1.83	★
BH22-6-4	4.88	⊙

Ministry of Transportation
Ontario

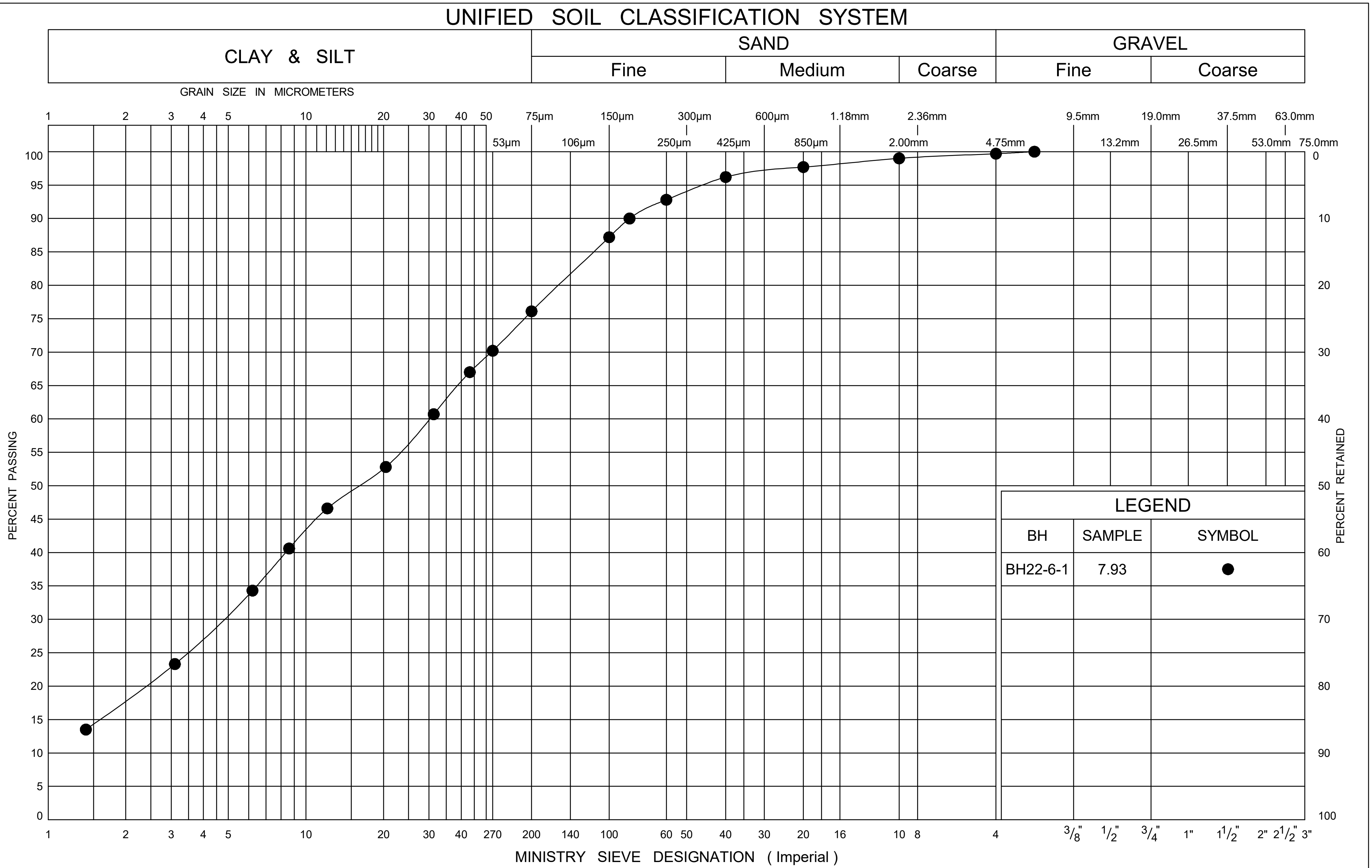
GRAIN SIZE DISTRIBUTION

Cohesive Fill: Clayey Silt

FIG No 2

W P Site 37X-0218/B1 & B3

Hwy 401 - Markham Rd. O/P



ONTARIO MOT GRAIN SIZE H401 - MARKHAM.GPJ ONTARIO MOT.GDT 3/8/23

Ministry of Transportation
Ontario

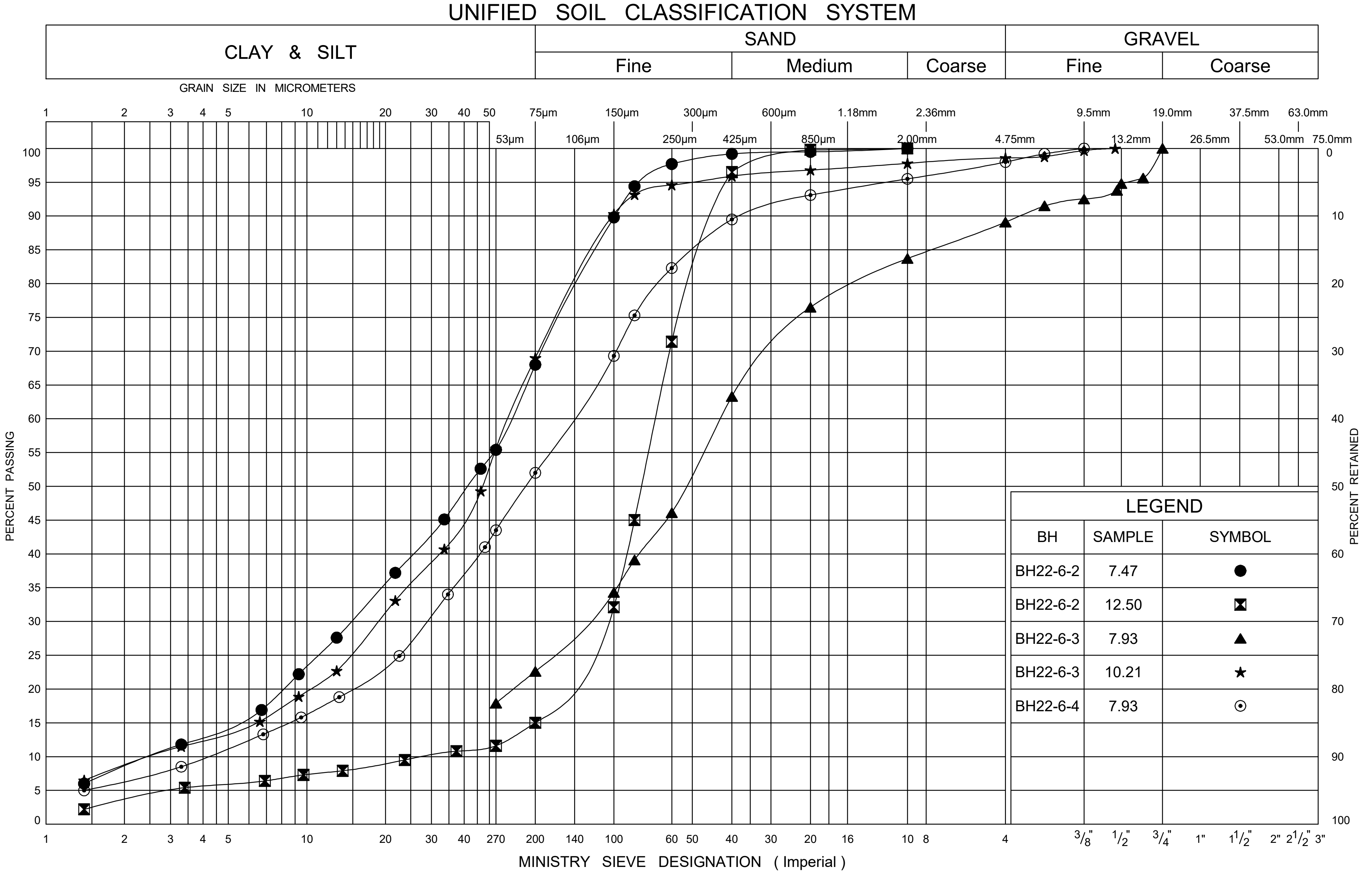
GRAIN SIZE DISTRIBUTION

Clayey Silt

FIG No 3

W P Site 37X-0218/B1 & B3

Hwy 401 - Markham Rd. O/P



Ministry of Transportation

Ontario

GRAIN SIZE DISTRIBUTION

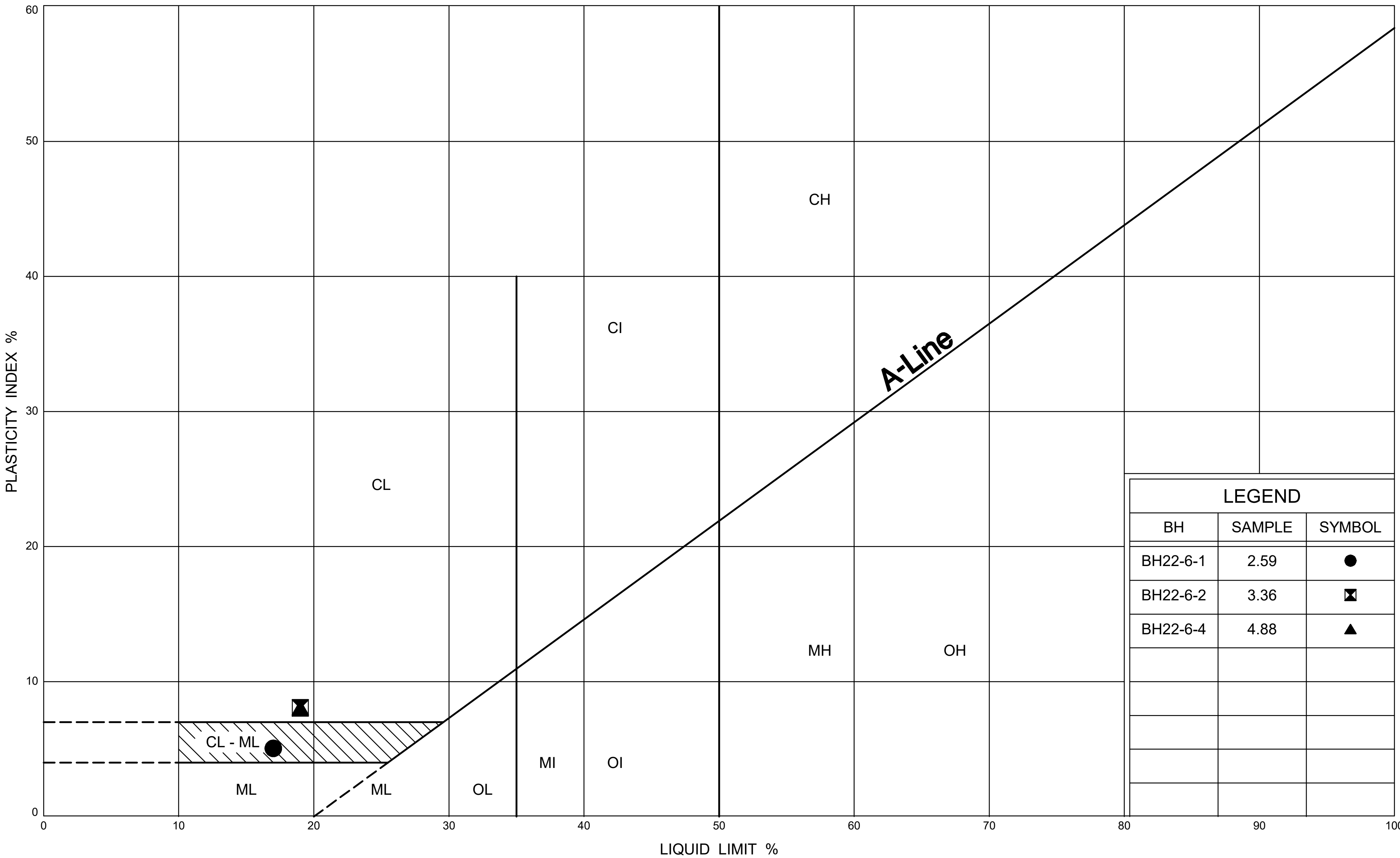
Sand/Sand and Silt/Sandy Silt

FIG No 4

W P Site 37X-0218/B1 & B3

Hwy 401 - Markham Rd. O/P

ONTARIO MOT GRAIN SIZE H401 - MARKHAM.GPJ ONTARIO MOT.GDT 6/29/23



LEGEND		
BH	SAMPLE	SYMBOL
BH22-6-1	2.59	●
BH22-6-2	3.36	⧗
BH22-6-4	4.88	▲

ONTARIO MOT PLASTICITY CHART H401 - MARKHAM.GPJ ONTARIO MOT.GDT 3/8/23

Ministry of Transportation
Ontario

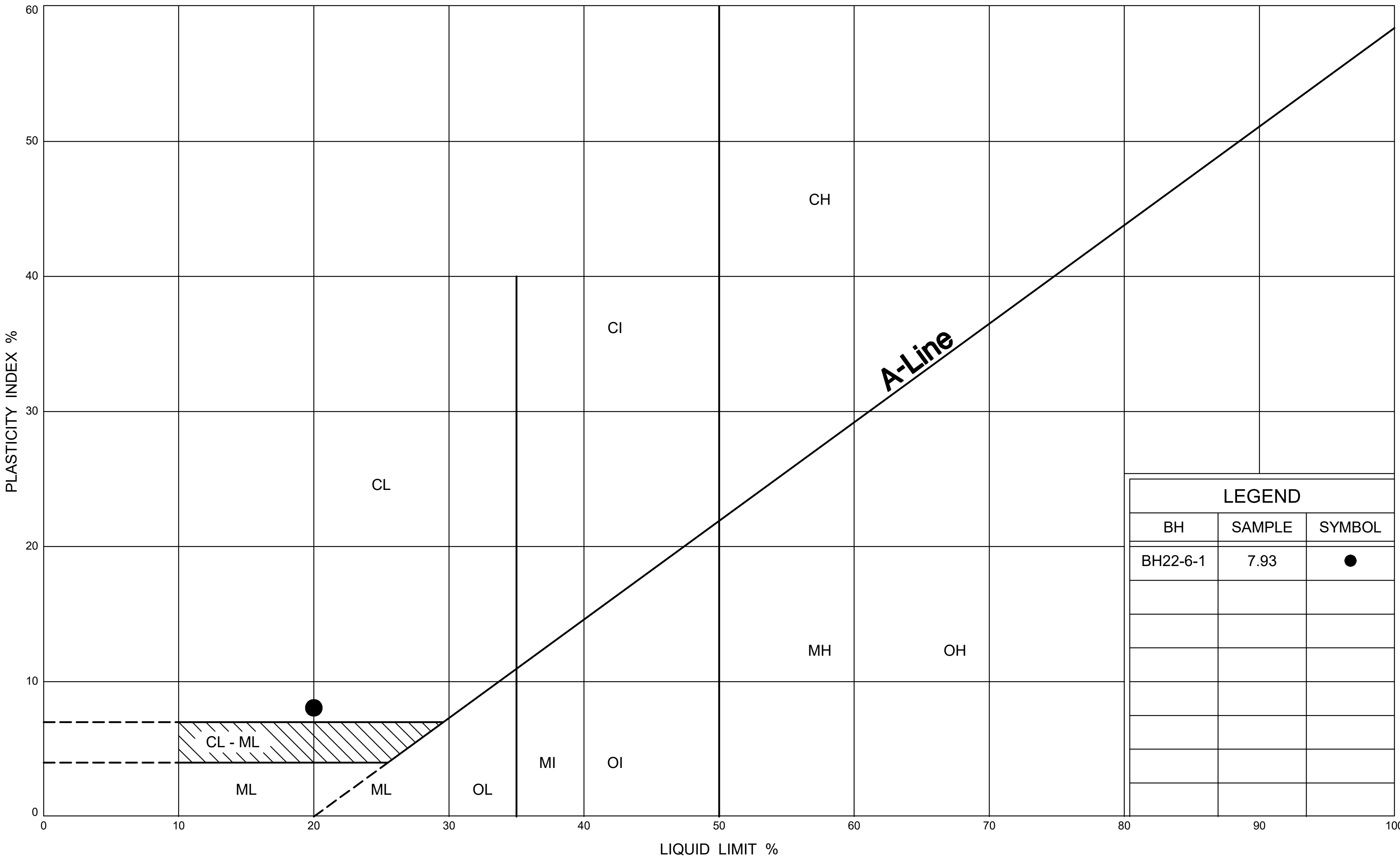
PLASTICITY CHART

Cohesive Fill: Clayey Silt

FIG No 5

W P Site 37X-0218/B1 & B3

Hwy 401 - Markham Rd. O/P



LEGEND

BH	SAMPLE	SYMBOL
BH22-6-1	7.93	●

ONTARIO MOT PLASTICITY CHART H401 - MARKHAM.GPJ ONTARIO MOT.GDT 3/8/23

Ministry of Transportation

Ontario

PLASTICITY CHART

Clayey Silt

FIG No 6

W P Site 37X-0218/B1 & B3

Hwy 401 - Markham Rd. O/P

Your Project #: ADM-22000797-A0
Site#: Hwy 401 from Victoria to Nelso
Site Location: Hwy 401 from Victoria to Nelson Ave, ON
Your C.O.C. #: 893860-02-01

Attention: Nimesh Tamrakar

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

Report Date: 2022/10/04
Report #: R7328399
Version: 1 - Final

CERTIFICATE OF ANALYSIS

BUREAU VERITAS JOB #: C2R8525
Received: 2022/09/27, 08:33

Sample Matrix: Soil
Samples Received: 2

Analyses	Quantity	Date Extracted	Date Analyzed	Laboratory Method	Analytical Method
Chloride (20:1 extract)	2	2022/10/03	2022/10/04	CAM SOP-00463	SM 23 4500-Cl E m
Conductivity	2	2022/10/03	2022/10/03	CAM SOP-00414	OMOE E3530 v1 m
Moisture (Subcontracted) (1, 2)	2	N/A	2022/10/01	AB SOP-00002	CCME PHC-CWS m
Sulphide in Soil (1)	2	N/A	2022/09/30	AB SOP-00080	EPA9030B/SM4500S2-DF
pH CaCl2 EXTRACT	2	2022/09/30	2022/09/30	CAM SOP-00413	EPA 9045 D m
Redox Potential (3)	2	2022/10/03	2022/10/04	CAM SOP-00421	SM 2580 B
Resistivity of Soil	2	2022/09/27	2022/10/03	CAM SOP-00414	SM 23 2510 m
Sulphate (20:1 Extract)	2	2022/10/03	2022/10/03	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.

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.

Reference Method suffix “m” indicates test methods incorporate validated modifications from specific reference methods to improve performance.
* RPDs calculated using raw data. The rounding of final results may result in the apparent difference.
(1) This test was performed by Bureau Veritas Calgary (19th), 4000 19th Street NE , Calgary, AB, T2E 6P8
(2) Offsite analysis requires that subcontracted moisture be reported.

Your Project #: ADM-22000797-A0
Site#: Hwy 401 from Victoria to Nelso
Site Location: Hwy 401 from Victoria to Nelson Ave, ON
Your C.O.C. #: 893860-02-01

Attention: Nimesh Tamrakar

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

Report Date: 2022/10/04
Report #: R7328399
Version: 1 - Final

CERTIFICATE OF ANALYSIS

BUREAU VERITAS JOB #: C2R8525
Received: 2022/09/27, 08:33
(3) Oxidation-Reduction Potential (ORP) values are determined using a Ag/AgCl reference electrode. The test is therefore, not SCC accredited for this matrix.

Encryption Key

Please direct all questions regarding this Certificate of Analysis to your Project Manager.
Patricia Legette, Project Manager
Email: Patricia.Legette@bureauveritas.com
Phone# (905)817-5799
==
This report has been generated and distributed using a secure automated process.
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 Signature Page.

Bureau Veritas Job #: C2R8525
Report Date: 2022/10/04

exp Services Inc
Client Project #: ADM-22000797-A0
Site Location: Hwy 401 from Victoria to Nelson Ave, ON
Sampler Initials: NT

SOIL CORROSIVITY PACKAGE (SOIL)

Bureau Veritas ID		TVP610			TVP610			TVP611		
Sampling Date		2022/09/18 03:00			2022/09/18 03:00			2022/09/23 01:00		
COC Number		893860-02-01			893860-02-01			893860-02-01		
	UNITS	**BH22-6-2 SS10**	**RDL**	**QC Batch**	**BH22-6-2 SS10 Lab-Dup**	**RDL**	**QC Batch**	**BH22-3-4 SS5**	**RDL**	**QC Batch**
Calculated Parameters										
Resistivity	ohm-cm	4100		8249951				1100		8249951
CONVENTIONALS										
Redox Potential	mV	93	N/A	8260394	77	N/A	8260394	190	N/A	8260394
Inorganics										
Soluble (20:1) Chloride (Cl-)	ug/g	90	20	8260593				480	20	8260593
Conductivity	umho/cm	246	2	8260420				945	2	8260420
Available (CaCl2) pH	pH	7.84		8257456				7.75		8257456
Soluble (20:1) Sulphate (SO4)	ug/g	34	20	8260601	31	20	8260601	<20	20	8260601
Sulphide	mg/kg	2.1 (1)	0.5	8259069				2.6	0.5	8259069
Physical Testing										
Moisture-Subcontracted	%	15	0.30	8264759				10	0.30	8264759

RDL = Reportable Detection Limit
QC Batch = Quality Control Batch
Lab-Dup = Laboratory Initiated Duplicate
N/A = Not Applicable
(1) Analyzed past method specified hold time

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

Bureau Veritas Job #: C2R8525
Report Date: 2022/10/04

exp Services Inc
Client Project #: ADM-22000797-A0
Site Location: Hwy 401 from Victoria to Nelson Ave, ON
Sampler Initials: NT

TEST SUMMARY

Bureau Veritas ID: TVP610
Sample ID: BH22-6-2 SS10
Matrix: Soil

Collected: 2022/09/18
Shipped:
Received: 2022/09/27

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	8260593	2022/10/03	2022/10/04	Alina Dobreanu
Conductivity	AT	8260420	2022/10/03	2022/10/03	Roya Fathitil
Moisture (Subcontracted)	BAL	8264759	N/A	2022/10/01	Simranjeet Batth
Sulphide in Soil	SPEC	8259069	N/A	2022/09/30	Dafne Strozake Maximo
pH CaCl2 EXTRACT	AT	8257456	2022/09/30	2022/09/30	Taslima Aktar
Redox Potential	COND	8260394	2022/10/03	2022/10/04	Surinder Rai
Resistivity of Soil		8249951	2022/10/03	2022/10/03	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	8260601	2022/10/03	2022/10/03	Samuel Law

Bureau Veritas ID: TVP610 Dup
Sample ID: BH22-6-2 SS10
Matrix: Soil

Collected: 2022/09/18
Shipped:
Received: 2022/09/27

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Redox Potential	COND	8260394	2022/10/03	2022/10/04	Surinder Rai
Sulphate (20:1 Extract)	KONE/EC	8260601	2022/10/03	2022/10/03	Samuel Law

Bureau Veritas ID: TVP611
Sample ID: BH22-3-4 SS5
Matrix: Soil

Collected: 2022/09/23
Shipped:
Received: 2022/09/27

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	8260593	2022/10/03	2022/10/04	Alina Dobreanu
Conductivity	AT	8260420	2022/10/03	2022/10/03	Roya Fathitil
Moisture (Subcontracted)	BAL	8264759	N/A	2022/10/01	Simranjeet Batth
Sulphide in Soil	SPEC	8259069	N/A	2022/09/30	Dafne Strozake Maximo
pH CaCl2 EXTRACT	AT	8257456	2022/09/30	2022/09/30	Taslima Aktar
Redox Potential	COND	8260394	2022/10/03	2022/10/04	Surinder Rai
Resistivity of Soil		8249951	2022/10/03	2022/10/03	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	8260601	2022/10/03	2022/10/03	Samuel Law

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

GENERAL COMMENTS

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

Package 1	7.0°C

Results relate only to the items tested.

Bureau Veritas Job #: C2R8525
Report Date: 2022/10/04

QUALITY ASSURANCE REPORT

exp Services Inc
Client Project #: ADM-22000797-A0
Site Location: Hwy 401 from Victoria to Nelson Ave, ON
Sampler Initials: NT

			Matrix Spike		SPIKED BLANK		Method Blank		RPD	
QC Batch	Parameter	Date	% Recovery	QC Limits	% Recovery	QC Limits	Value	UNITS	Value (%)	QC Limits
8257456	Available (CaCl2) pH	2022/09/30			100	97 - 103			0.13	N/A
8259069	Sulphide	2022/09/30	124	75 - 125	85	75 - 125	<0.5	mg/kg	NC	30
8260394	Redox Potential	2022/10/04			100	95 - 105			18	N/A
8260420	Conductivity	2022/10/03			101	90 - 110	<2	umho/cm	0.67	10
8260593	Soluble (20:1) Chloride (Cl-)	2022/10/04	130	70 - 130	101	70 - 130	<20	ug/g	NC	35
8260601	Soluble (20:1) Sulphate (SO4)	2022/10/03	NC	70 - 130	104	70 - 130	<20	ug/g	9.1	35
8264759	Moisture-Subcontracted	2022/10/01					<0.30	%		

N/A = Not Applicable

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

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

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

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

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

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

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

VALIDATION SIGNATURE PAGE

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

Anastassia Hamanov, Scientific Specialist

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

CHARTERED
Eva Pranjic
CHEMIST

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

Janet Gao, B.Sc., QP, Supervisor, Organics

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 Signature Page.

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

27-Sep-22 08:33
Patricia Legette
C2R8525
MUM ENV-1743

Page of

INVOICE TO:		REPORT TO:		PROJECT INFORMATION:	
Company Name:	#17488 exp Services Inc	Company Name:		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 #:	Hwy 401 from Victoria to Nelso
Email:	AP@exp.com; Karen.Burke@exp.com	Email:	Nimesh.Tamrakar@exp.com	Sampled By:	

Bottle Order #: 893860
COC #: C#893860-02-01
Project Manager: Patricia Legette

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): ☐ Table 1 ☐ Res/Park ☐ Medium/Fine ☐ Table 2 ☐ Ind/Comm ☐ Coarse ☐ Table 3 ☐ Agri/Other ☐ For RSC ☐ Table ___

Other Regulations: ☐ CCME ☐ Sanitary Sewer Bylaw ☐ Reg 558 ☐ Storm Sewer Bylaw ☐ MISA Municipality ___ ☐ PWQO ☐ Reg 406 Table ___ ☐ Other

Special Instructions

Include Criteria on Certificate of Analysis (Y/N)? ___

ANALYSIS REQUESTED (PLEASE BE SPECIFIC): Field Filtered (please circle): Metals / Hg / Cr VI; Soil Corrosivity Package

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

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

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

	Sample Barcode Label	Sample (Location) Identification	Date Sampled	Time Sampled	Matrix	Field Filtered	Soil Corrosivity Package	# of Bottles	Comments
1		BH22-6-2 SS10	2022-09-19	3:30am	SOIL		✓	2	
2		BH22-3-4 SS5	2022-9-23	1:00Am	SOIL		✓	2	
3					SOIL				
4					SOIL				
5					SOIL				
6					SOIL				
7					SOIL				
8					SOIL				
9					SOIL				
10					SOIL				

* RELINQUISHED BY: (Signature/Print)	Date: (YY/MM/DD)	Time	RECEIVED BY: (Signature/Print)	Date: (YY/MM/DD)	Time	# jars used and not submitted
Nimesh Tamrakar	2022-09-27		Z VITRINO	2022/09/27	08:37	

Laboratory Use Only: Time Sensitive; Temperature (°C) on Recei ___; Custody Seal: Present / Intact — Yes / No

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

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

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

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

White: Bureau Veritas Yellow: Client

6/9/6

Bureau Veritas Canada (2019) Inc.

Your Project #: ADM-22000797-A0
Site Location: MIDLAND/MARKHAM - 401 HWY
Your C.O.C. #: n/a

Attention: Silvana Micic

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

Report Date: 2022/09/22
Report #: R7308420
Version: 1 - Final

CERTIFICATE OF ANALYSIS

BUREAU VERITAS JOB #: C2Q5822
Received: 2022/09/15, 10:29

Sample Matrix: Soil
Samples Received: 2

Analyses	Quantity	Date Extracted	Date Analyzed	Laboratory Method	Analytical Method
Chloride (20:1 extract)	2	2022/09/20	2022/09/21	CAM SOP-00463	SM 23 4500-Cl E m
Conductivity	2	2022/09/20	2022/09/20	CAM SOP-00414	OMOE E3530 v1 m
Moisture (Subcontracted) (1, 2)	2	N/A	2022/09/21	AB SOP-00002	CCME PHC-CWS m
Sulphide in Soil (1)	2	N/A	2022/09/21	AB SOP-00080	EPA9030B/SM4500S2-DF
pH CaCl2 EXTRACT	2	2022/09/21	2022/09/21	CAM SOP-00413	EPA 9045 D m
Redox Potential (3)	2	2022/09/20	2022/09/20	CAM SOP-00421	SM 2580 B
Resistivity of Soil	2	2022/09/16	2022/09/20	CAM SOP-00414	SM 23 2510 m
Sulphate (20:1 Extract)	2	2022/09/20	2022/09/20	CAM SOP-00464	EPA 375.4 m

Remarks:

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

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

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

Solid sample results, except biota, are based on dry weight unless otherwise indicated. Organic analyses are not recovery corrected except for isotope dilution methods.
Results relate to samples tested. When sampling is not conducted by Bureau Veritas, results relate to the supplied samples tested.
This Certificate shall not be reproduced except in full, without the written approval of the laboratory.
Reference Method suffix “m” indicates test methods incorporate validated modifications from specific reference methods to improve performance.
* RPDs calculated using raw data. The rounding of final results may result in the apparent difference.
(1) This test was performed by Bureau Veritas Calgary (19th), 4000 19th Street NE , Calgary, AB, T2E 6P8
(2) Offsite analysis requires that subcontracted moisture be reported.

Your Project #: ADM-22000797-A0
Site Location: MIDLAND/MARKHAM - 401 HWY
Your C.O.C. #: n/a

Attention: Silvana Micic

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

Report Date: 2022/09/22
Report #: R7308420
Version: 1 - Final

CERTIFICATE OF ANALYSIS

BUREAU VERITAS JOB #: C2Q5822
Received: 2022/09/15, 10:29

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

Encryption Key

Please direct all questions regarding this Certificate of Analysis to your Project Manager.
Patricia Legette, Project Manager
Email: Patricia.Legette@bureauveritas.com
Phone# (905)817-5799
==
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 Signature Page.

Bureau Veritas Job #: C2Q5822
Report Date: 2022/09/22

exp Services Inc
Client Project #: ADM-22000797-A0
Site Location: MIDLAND/MARKHAM - 401 HWY
Sampler Initials: EL

SOIL CORROSIVITY PACKAGE (SOIL)

Bureau Veritas ID		TSY134			TSY134			TSY135		
Sampling Date		2022/09/12 13:00			2022/09/12 13:00			2022/09/13 13:00		
COC Number		n/a			n/a			n/a		
	UNITS	**BH22-5-4 SS5**	**RDL**	**QC Batch**	**BH22-5-4 SS5 Lab-Dup**	**RDL**	**QC Batch**	**BH22-6-1 SS5**	**RDL**	**QC Batch**
Calculated Parameters										
Resistivity	ohm-cm	1500		8229384				540		8229384
CONVENTIONALS										
Redox Potential	mV	110	N/A	8234488				180	N/A	8234488
Inorganics										
Soluble (20:1) Chloride (Cl-)	ug/g	330	20	8234709				1000	40	8234709
Conductivity	umho/cm	660	2	8235211				1870	2	8235211
Available (CaCl2) pH	pH	8.17		8237272				7.24		8237272
Soluble (20:1) Sulphate (SO4)	ug/g	<20	20	8234714				<20	20	8234714
Sulphide	mg/kg	2.0 (1)	0.5	8241140	2.6	0.5	8241140	<0.5 (1)	0.5	8241140
Physical Testing										
Moisture-Subcontracted	%	5.6	0.30	8241139				15	0.30	8241139

RDL = Reportable Detection Limit
QC Batch = Quality Control Batch
Lab-Dup = Laboratory Initiated Duplicate
N/A = Not Applicable
(1) Sample extracted past method-specified hold time.
Sample contained greater than 10% headspace at time of extraction.
Analyzed past method specified hold time

Bureau Veritas ID		TSY135		
Sampling Date		2022/09/13 13:00		
COC Number		n/a		
	UNITS	**BH22-6-1 SS5 Lab-Dup**	**RDL**	**QC Batch**
Inorganics				
Conductivity	umho/cm	1850	2	8235211

RDL = Reportable Detection Limit
QC Batch = Quality Control Batch
Lab-Dup = Laboratory Initiated Duplicate

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

Bureau Veritas Job #: C2Q5822
Report Date: 2022/09/22

exp Services Inc
Client Project #: ADM-22000797-A0
Site Location: MIDLAND/MARKHAM - 401 HWY
Sampler Initials: EL

TEST SUMMARY

Bureau Veritas ID: TSY134
Sample ID: BH22-5-4 SS5
Matrix: Soil

Collected: 2022/09/12
Shipped:
Received: 2022/09/15

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	8234709	2022/09/20	2022/09/21	Samuel Law
Conductivity	AT	8235211	2022/09/20	2022/09/20	Roya Fathitil
Moisture (Subcontracted)	BAL	8241139	N/A	2022/09/21	Eric Tse
Sulphide in Soil	SPEC	8241140	N/A	2022/09/21	Ly Vu
pH CaCl2 EXTRACT	AT	8237272	2022/09/21	2022/09/21	Taslima Aktar
Redox Potential	COND	8234488	2022/09/20	2022/09/20	Surinder Rai
Resistivity of Soil		8229384	2022/09/20	2022/09/20	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	8234714	2022/09/20	2022/09/20	Samuel Law

Bureau Veritas ID: TSY134 Dup
Sample ID: BH22-5-4 SS5
Matrix: Soil

Collected: 2022/09/12
Shipped:
Received: 2022/09/15

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Sulphide in Soil	SPEC	8241140	N/A	2022/09/21	Ly Vu

Bureau Veritas ID: TSY135
Sample ID: BH22-6-1 SS5
Matrix: Soil

Collected: 2022/09/13
Shipped:
Received: 2022/09/15

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	8234709	2022/09/20	2022/09/21	Samuel Law
Conductivity	AT	8235211	2022/09/20	2022/09/20	Roya Fathitil
Moisture (Subcontracted)	BAL	8241139	N/A	2022/09/21	Eric Tse
Sulphide in Soil	SPEC	8241140	N/A	2022/09/21	Ly Vu
pH CaCl2 EXTRACT	AT	8237272	2022/09/21	2022/09/21	Taslima Aktar
Redox Potential	COND	8234488	2022/09/20	2022/09/20	Surinder Rai
Resistivity of Soil		8229384	2022/09/20	2022/09/20	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	8234714	2022/09/20	2022/09/20	Samuel Law

Bureau Veritas ID: TSY135 Dup
Sample ID: BH22-6-1 SS5
Matrix: Soil

Collected: 2022/09/13
Shipped:
Received: 2022/09/15

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Conductivity	AT	8235211	2022/09/20	2022/09/20	Roya Fathitil

GENERAL COMMENTS

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

Package 1	1.3°C

Results relate only to the items tested.

QUALITY ASSURANCE REPORT

			Matrix Spike		SPIKED BLANK		Method Blank		RPD	
QC Batch	Parameter	Date	% Recovery	QC Limits	% Recovery	QC Limits	Value	UNITS	Value (%)	QC Limits
8234488	Redox Potential	2022/09/20			100	95 - 105			21	N/A
8234709	Soluble (20:1) Chloride (Cl-)	2022/09/21	116	70 - 130	108	70 - 130	<20	ug/g	NC	35
8234714	Soluble (20:1) Sulphate (SO4)	2022/09/20	NC	70 - 130	99	70 - 130	<20	ug/g	NC (1)	35
8235211	Conductivity	2022/09/20			101	90 - 110	<2	umho/cm	1.1	10
8237272	Available (CaCl2) pH	2022/09/21			100	97 - 103			1.1	N/A
8241139	Moisture-Subcontracted	2022/09/21					<0.30	%		
8241140	Sulphide	2022/09/21	113	75 - 125	114	75 - 125	<0.5	mg/kg	25	30

N/A = Not Applicable

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

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

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

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

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

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

(1) Due to colour interferences, sample required dilution. Detection limit was adjusted accordingly.

VALIDATION SIGNATURE PAGE

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

Cristina Carriere

Cristina Carriere, Senior Scientific Specialist

Veronica Falk

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

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

6740 Campobello Road, Mississauga, Ontario L5N 2L8
Phone: 905-817-5700 Fax: 905-817-5779 Toll Free: 800-563-6266

CHAIN OF CUSTODY RECORD
ENV COC - 00014v3

Page 1 of 1

15-Sep-22 10:29
Patricia Legette
C2Q5822
SPJ ENV-919

Invoice Information	Invoice to (requires report) ☐	Report Information (if differs from invoice)		Project Information	
Company:	EXP Services	Company:	EXP SERVICES	Quotation #:	
Contact Name:	Elvis Lu	Contact Name:	Elvis Lu	P.O. #/ AFE#:	
Street Address:	1595 Clark Blvd	Street Address:		Project #:	ADM-22000797-AO
City:	BRAMPTON Prov: ON Postal Code: L6T 4V1	City:	Prov: Postal Code:	Site #:	401 HWY
Phone:	519 870 4510	Phone:		Site Location:	MIDLAND / MARKHAM
Email:	Elvis.Lu@exp.com	Email:		Site Location Province:	ON
Copies:		Copies:		Sampled By:	Elvis Lu

Regulatory Criteria

REG 153: ☐ Table 1 ☐ Table 2 ☐ Table 3 ☐ Table ____ ☐ Res/Park ☐ Ind/Comm ☐ Agri/other ☐ Med/Fine ☐ Course ☐ For RSC

OTHER: ☐ CCME ☐ Reg 558* *min 3 day TAT ☐ MISA ☐ PWQO ☐ Reg 406, Table: ☐ Sanitary Sewer Bylaw ☐ Storm Sewer Bylaw Municipality ____ ☐ Other:

Include Criteria on Certificate of Analysis (check if yes): ☐

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

Analyses: 1 FIELD FILTERED; 2 FIELD PRESERVED; 3 LAB FILTRATION REQUIRED; 4 BTEX/ F1; 5 F2 - F4; 6 VOCs; 7 Reg 153 metals and inorganics; 8 Reg 153 ICPMS metals; 9 Reg 153 metals (Hg, Cr VI, ICPMS metals, HWS - B); 10 Corrosivity / PH; 11–20 ; 21 # OF CONTAINERS SUBMITTED; 22 HOLD - DO NOT ANALYZE

#	Sample Identification	YY	MM	DD	HH	MM	Matrix	10 Corrosivity / PH	# of containers	Hold
1	ADM- BH22-5-4 SSS	22	09	12				✓		
2	BH22-5-4 SSS	22	09	12				✓		
3	BH22-6-1 SSS	22	09	13				✓		
4	BH22-6-1 SSS	22	09	13				✓		
5										
6										
7										
8										
9										
10										
11										
12										

Regular Turnaround Time (TAT): ☐ 5 to 7 Day ☐ 10 Day

Rush Turnaround Time (TAT) Surcharges apply: ☐ Same Day ☐ 1 Day ☐ 2 Day ☐ 3 Day ☐ 4 Day

Date Required: YY MM DD

Comments

LAB USE ONLY	Yes	No	°C	1	2	3
Seal present		✓		\	|	2
Seal intact						
Cooling media present	✓					

LAB USE ONLY	Yes	No	°C	1	2	3
Seal present						
Seal intact						
Cooling media present						

LAB USE ONLY	Yes	No	°C	1	2	3	Temperature reading by:
Seal present							
Seal intact							
Cooling media present							

	Relinquished by: (Signature/ Print)	YY	MM	DD	HH	MM	Received by: (Signature/ Print)	YY	MM	DD	HH	MM	Special instructions
1	Juan Barua	22	09	15	10	24	[signature]	2022	09	15	10	29	
2													

Appendix F – Previous Investigation - BH logs

FORM OB-MT-126
66-4546

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 1

FOUNDATION SECTION

JOB 67-F-40

W.P. 262-61

DATUM Geodetic

LOCATION Co-ordinates 62,143 N. 102,822 E.

BORING DATE May 15, 1967

BOREHOLE TYPE Continuous Flight Auger

ORIGINATED BY AMS

COMPILED BY AMS

CHECKED BY [signature]

ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	REMARKS Gr.Sa.Si.Cl
517.5	GROUND LEVEL						
0.0	Clayey silt with sand and trace of gravel. Very Stiff to Hard						▼516.0
			1	SS	17		
			2	SS	112	510	0 25 62 13
			3	SS	100		
			4	SS	150/3"		
			5	SS	150/3½"	500	
			6	SS	150/6"		
			7	SS	175/6"	490	
486.0			8	SS	50/¾"		
31.5	End of Borehole						

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT

SHEAR STRENGTH P.S.F.

LIQUID LIMIT — w_L
PLASTIC LIMIT — w_P
WATER CONTENT — w

WATER CONTENT % 10 20 30

BULK DENSITY γ P.C.F.

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OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 2

FOUNDATION SECTION

JOB 67-F-40
W.P. 262-61
DATUM Geodetic

LOCATION Co-ordinates 62,170 N; 102,750 E.
BORING DATE May 15, 1967
BOREHOLE TYPE Continuous Flight Auger

ORIGINATED BY AMS
COMPILED BY AMS
CHECKED BY [illegible]

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT
SHEAR STRENGTH P.S.F.

LIQUID LIMIT —— w_L
PLASTIC LIMIT —— w_P
WATER CONTENT —— w

WATER CONTENT % 10 20 30

ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	BULK DENSITY γ P.C.F.	REMARKS Gr.Sa.Si.Cl
516.4	GROUND LEVEL							
0.0	Clayey silt with some sand and trace of gravel.							▼ 514.4
			1	SS	94			
			2	SS	86	510		0 35 51 14
			3	SS	178			
			4	SS	55			
	Hard		5	SS	86	500		
			6	SS	72			
			7	SS	74	490		0 3 81 16
484.9			8	SS	186			
31.5	End of Borehole							

FORM OB-MT-126
66-4546

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 3

FOUNDATION SECTION

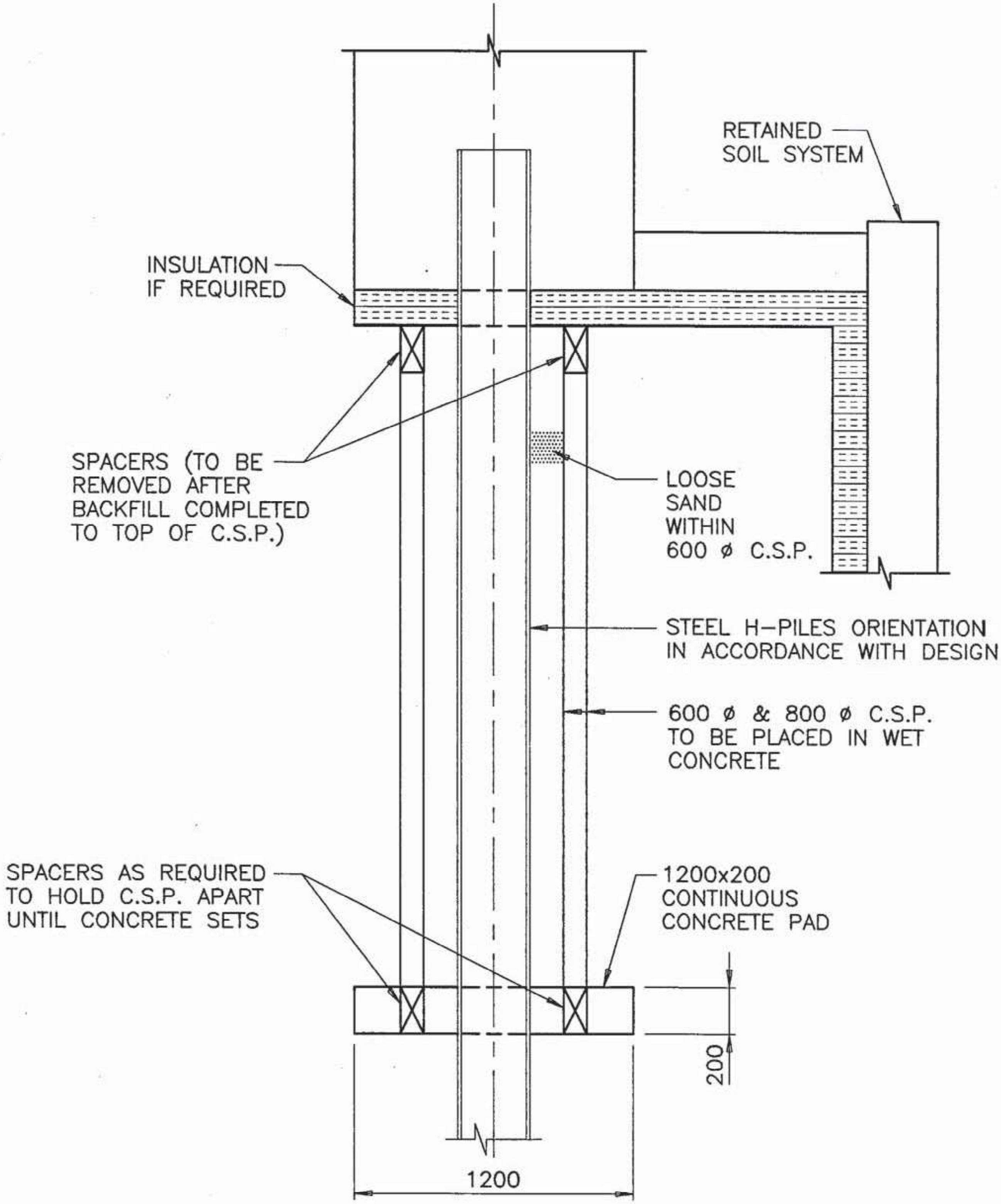
JOB 67-F-40 LOCATION CoOrdinates 62,098 N; 102,710 E. ORIGINATED BY AMS

W.P. 262-61 BORING DATE May 16, 1967 COMPILED BY AMS

DATUM Geodetic BOREHOLE TYPE Continuous Flight Auger CHECKED BY [signature]

ELEV. / DEPTH	DESCRIPTION	STRAT. PLOT	SAMPLES NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT; SHEAR STRENGTH P.S.F.	WATER CONTENT % (LIQUID LIMIT w_L, PLASTIC LIMIT w_P, WATER CONTENT w) 10 20 30	BULK DENSITY γ P.C.F.	REMARKS Gr.Sa.Si.Cl
516.0	GROUND LEVEL									
0.0	Fine sandy silt to silty fine sand and trace of gravel. Very Dense.		1	SS	65					2 50 (48) ▼511.0
			2	SS	60	510				
506.5 / 9.5	Clayey silt with sand and trace of gravel. Hard.		3	SS	150/4"					
			4	SS	50/½"					
			5	SS	50/½"	500				0 10 73 17
			6	SS	175/4½"					
			7	SS	150/3"	490				0 24 59 17
484.5			8	SS	200/4½"					
31.5	End of Borehole									

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

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 (°)	43.785
Longitude (°)	-79.235

Please select one of the tabs below.

NBC 2020 | Additional Values | Plots | API | Background Information

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

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

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

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

$S_a(0.2, X_C)$	$S_a(0.5, X_C)$	$S_a(1.0, X_C)$	$S_a(2.0, X_C)$	$S_a(5.0, X_C)$	$S_a(10.0, X_C)$	$PGA(X_C)$	$PGV(X_C)$
0.33	0.2	0.105	0.0485	0.0126	0.00427	0.179	0.131

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

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

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.