Peto MacCallum Ltd.
CONSULTING ENGINEERS

PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT FOR DESIGN-BUILD READY ALTERNATIVE BID PACKAGE

FOR

WELLINGTON ROAD 34 CONNECTOR UNDERPASS
SITE NO. 35X-0618/B0, STATION 10+000
LATITUDE AND LONGITUDE: 43.464815, -80.183754
MIDBLOCK INTERCHANGE (MBI) AREA
HIGHWAY 6 AND HIGHWAY 401 IMPROVEMENTS
FROM HAMILTON NORTH LIMITS TO GUELPH SOUTH LIMITS
CITY OF GUELPH, ONTARIO
GWP 3059-20-00

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PML Ref.: 17TF006A
Index No.: 065FIR and 066FDR
GEOCRES No.: 40P8-291
October 14, 2021

PART A - PRELIMINARY FOUNDATION INVESTIGATION REPORT FOR DESIGN-BUILD READY ALTERNATIVE BID PACKAGE

FOR

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PART A – PRELIMINARY FOUNDATION INVESTIGATION REPORT

PART A - PRELIMINARY FOUNDATION INVESTIGATION REPORT for Wellington Road 34 Connector Underpass

Site No. 35X-0618/B0, Station 10+000
Midblock Interchange (MBI) Area
Highway 6 and Highway 401 Improvements
From Hamilton North Limits to Guelph South Limits
City of Guelph, Ontario
G.W.P. 3059-20-00

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1. INTRODUCTION

The Ministry of Transportation of Ontario (MTO), West Region retained AECOM Canada Ltd. (AECOM) as Prime Consultant, to provide Owner's Engineer services for the re-alignment, improvement and replacement of existing structures located on Highway 6 and Highway 401 from Hamilton North Limits to Guelph South Limits. The Midblock Interchange (MBI) area is part of the Hamilton to Guelph advance contract to be delivered on a design-build (DB) basis under GWP 3059-20-00. The project limits stretch from about 0.1 km north of Maltby Road to 0.3 km south of Wellington County Road 34. Drawing 35-618-1 presents the limits of the MBI area.

The scope of work in the MBI project area involves the construction of Wellington Road 34 Connector Underpass, Midblock Interchange, rehabilitation/widening of Highway 6, rehabilitation, widening and/or realignment of Concession Road 7, construction of Wellington Road 34 Underpass, widening, reconstruction and intersection improvements of Wellington County Road 34, and the construction of a new mid-concession route (Wellington Road 34 Connector route). The main foundation components of the project include the two bridge structures (Wellington Road 34 Underpass (35X-0617/B0) and Wellington Road 34 Connector Underpass (35X-0618/B0), high fill and deep cut sections along Highway 6, Concession Road 7, and the new connector route and its ramps, and overhead signs on both sides of the midblock interchange, south of Wellington Road 34 intersection, and close to the area where Concession Road 7 merges with Highway 6.

AECOM retained Peto MacCallum Ltd. (PML) on behalf of MTO to provide foundation engineering services for the project. The terms of reference and scope of work for the foundation investigation and design services are outlined in the RFP, dated November 2016. The geotechnical work reported herein is for Wellington Road 34 Connector Underpass (35X-0618/B0), and it involves the preparation of a preliminary foundation investigation report (FIR) and preliminary foundation investigation and design report (FIDR) for the DB ready package. The preliminary FIR presents the factual subsurface information obtained from the boreholes drilled by PML for this assignment. The preliminary FIDR provides preliminary design level foundation recommendations based on the findings of the subsurface investigation work carried out for this assignment.

Previously, the following technical memorandum wasprepared for the project by PML based on limited subsurface information and submitted to AECOM:

- Draft Technical Memorandum - Preliminary Review of High Fill and Deep Cut Areas, Midblock Interchange Area, Hwy 6 401 Improvement, dated April 8, 2021.

Subsequently, decision was made to conduct additional investigations for the proposed Wellington Road 34 Underpass, complete the subsurface investigation work outlined in the original RFP for Wellington Road 34 Connector Underpass, and carry out foundation investigations at the proposed locations of high fill and deep cut sections within the MBI area. Accordingly, this FIR and the associated FIDR provided in Part B, present the findings of both the original (2017) and recent (2021) foundation investigations for the proposed Wellington Road 34 Underpass (35X-0617/B0).

The FIR and FIDR for Wellington Road 34 Underpass (35X-0617/B0) and the FIR and FIDR for high fill and deep cut sections were issued as separate reports. It is understood that foundation investigations for other components of the project, such as the overhead sign support structures and retaining walls, if required, will be carried out by the design-builder.

2. SITE DESCRIPTION

The MBI area is characterized by a landform composed of several geomorphic elements, ranging from low relief and flat-lying areas near the intersection of Wellington Road 34 and Highway 6, to elevated areas at the proposed location of the Midblock Interchange. Irregular hummocky surfaces and strongly undulating and rugged topography is also present along Concession Road 7.

The proposed Wellington Road 34 Connector Underpass is located approximately 900 m north of the intersection of Wellington Road 34 and Highway 6. The surrounding area on the west and east side of Highway 6 is agricultural land. The land to the south is a wetland covered with mixed hardwood of deciduous and coniferous trees, aquatic plants, and bush. Shrubs and tall grasses are also common along roadside ditches. On the north side, a golf course exists between Highway 6 and Concession Road 7. Highway 6 at the proposed location of the underpass consists of northbound and southbound lanes (NBL and SBL) with a grassed median strip.

Refer to the Photographs 1 to 6 in Appendix A, for general site conditions.

3. FIELD INVESTIGATION PROGRAM

The original field investigation work for Wellington Road 34 Connector Underpass was carried out between November 27, 2017 and December 08, 2017. During this time, six (6) boreholes were drilled at the locations of the west approach, west abutment, the centre pier and on the east side of Highway 6. The foundation investigation in 2021 was planned with the objective of completing the original scope by advancing three (3) boreholes at the east approach and east abutment that could not be accessed in 2017 due to unavailability of permit to enter (PTE) private properties in this area.

A summary of the field investigation programs conducted in 2017 and 2021 is provided in Table 1. A borehole location plan is presented in Drawing 35-618-1. As shown in the borehole location plan, Borehole 35-618-01 was advanced at the west approach of the underpass. Boreholes 35-618-02 and 35-618-03A were drilled near or at the proposed location of the west abutment. Borehole 35-618-3B was advanced by dynamic cone penetration test (DCPT) to complement Borehole 35-618-3A drilled nearby. Boreholes 35-618-04 and 35-618-05 were drilled at the proposed location of the centre pier. Borehole 6 was drilled as part of the original field program in 2017 near the toe of the east side of Highway 6, mainly for the purpose of determining the depth of the bedrock at the location. Boreholes 35-618-07, 35-618-08 and 35-618-09 were drilled at the east approach and east abutment during the 2021 investigation to complete the foundation investigation work at the project site as outlined in the RFP.

Table 1 - Wellington Road 34 Connector Underpass Borehole Information

BOREHOLE ID	BOREHOLE LOCATION	MTM NAD 83 Coordinates (MTM Zone ON10)		GROUND SURFACE ELEVATION (m)	BOREHOLE DEPTH (m)
		NORTHING (m)	EASTING (m)		
35-618-01	West Approach	4 814 069.0	249 441.2	328.1	9.8
35-618-02	West Abutment	4 814 088.5	249 438.0	328.0	34.8
35-618-03A	West Abutment	4 814 062.3	249 445.5	328.0	13.3
35-618-03B	West Abutment	4 814 061.4	249 444.5	328.0	14.7
35-618-04	Centre Pier	4 814 119.6	249 471.6	322.3	20.5
35-618-05	Centre Pier	4 814 093.6	249 479.6	321.5	25.0

Table 1 - Wellington Road 34 Connector Underpass Borehole Information

BOREHOLE ID	BOREHOLE LOCATION	MTM NAD 83 Coordinates (MTM Zone ON10)		GROUND SURFACE ELEVATION (m)	BOREHOLE DEPTH (m)
		NORTHING (m)	EASTING (m)		
35-618-06	Near East Abutment (Toe of Road Cut)	4 814 143.6	249 487.0	323.3	25.9
35-618-07	East Approach	4 814 145.9	249 520.1	328.2	10.5
35-618-08	East Abutment	4 814 138.6	249 508.2	328.5	34.5
35-618-09	East Abutment	4 814 155.2	249 503.6	329.7	29.9

The boreholes were drilled to depths ranging from 9.8 m (El. 318.3 m) to 34.8 m (El. 293.2 m) below existing grade. The records of borehole sheets are presented in Appendix B.

Prior to commencement of the fieldwork, utility clearance procedures were implemented through Ontario One Call protocol and by contacting MTO locates. As required, fieldwork notification was also sent to MTO West Region and MTO Project Manager. In accordance with PML's work plan for the project, project specific health and safety pas well as traffic protection plans were prepared and utilized at the time of the field investigation. In addition, the borehole locations were marked by PML staff prior to drilling. All drilling activities, soil sampling and logging, and backfilling of boreholes were also conducted under the supervision of an experienced PML staff.

Both during the original (2017) and the recent (2021) field investigation programs, the boreholes were advanced using track-mounted drill rigs, equipped with continuous flight hollow and solid stem augers, supplemented by wash boring (mud rotary) techniques. For the field program carried out in 2017, the drilling equipment was owned and operated by a specialist contractor, Aardvark Drilling Inc., based in Guelph, Ontario. The drilling contractor for the additional investigation work carried out in 2021 was PML Field Services Ltd., based in Hannon, Ontario.

Soil samples were obtained at selected intervals using a split-spoon sampler in accordance with the Standard Penetration Test (SPT) procedures (ASTM D1586 – Standard Test Method for Standard Penetration Test). The results of the SPTs were reported as "N" values in the attached record of borehole sheets. Bedrock was encountered in some of the boreholes and bedrock coring was carried out by wash boring using NQ and HQ sized core barrels within NW and HW casings.

Soil samples obtained from the boreholes were inspected immediately upon retrieval to assess type, texture, and colour. All retrieved samples were sealed in clean plastic bags and transported to PML's laboratory in Toronto for visual examination and laboratory testing purposes. Preliminary rock core description was also conducted in the field after completion of rock coring operations.

Groundwater levels in the open boreholes were observed throughout the drilling operations by visual examination of soil samples, the spilt-spoon sampler, and drill rods as the samples were retrieved. In addition, monitoring wells were installed to measure stabilized groundwater levels. The monitoring wells typically consisted of 50 mm outside diameter rigid PVC pipe with a 1.5 m long screen surrounded by a sand pack and sealed at selected depths within the borehole. No artesian condition or pressurized groundwater was encountered in this project site.

Boreholes were backfilled with soil cuttings and flowing wells were decommissioned in conformance with MTO guidelines and the Ministry of Environment, Conservation and Parks (MECP) Ontario Regulation 903 (as amended by Ontario Regulation 372). In the case of wells that were used for groundwater measurements, the annular space between the borehole wall and the monitoring well pipe above the filter pack was backfilled to ground surface using bentonite pellets.

For boreholes drilled in 2017, J.D. Barnes Limited surveyed of the borehole locations, and provided coordinates and ground surface elevations. For the additional field program completed in 2021, surveying was done by Callon Dietz. The MTM NAD83 northing and easting (MTM Zone – ON10) coordinates, and the ground surface elevations (in metres, referenced to Geodetic datum) at borehole locations are presented on the record of borehole sheets given in Appendix B. The horizontal and vertical accuracies of surveying were under 5 cm and 10 cm, respectively.

4. LABORATORY TEST PROCEDURES

Laboratory tests were conducted on representative SPT soil samples recovered during the fieldwork. Testing was conducted at PML's laboratory facility located in Toronto, Ontario.

For the original (2017) foundation investigation program, the laboratory tests included the following:

- Moisture content testing (89)
- Grain size distribution analyses (24)
- Atterberg limit tests (2)

The laboratory testing work for the field program in 2021 included forty-eight (48) moisture content tests (all retrieved samples), twelve (12) grainsize distribution analyses and one (1) Atterberg limit test.

The laboratory tests to determine soil index properties were performed in accordance with MTO test procedures, which follow the American Society for Testing Materials (ASTM) standards. However, the hydrometer tests were tested based on MTO's standard LS-702.

The results of the grainsize distribution analyses of are provided in Figures 618-GS-1A, 618-GS-1B, 618-GS-1C, 618-GS-1D and 618-GS-2, in Appendix C. The Atterberg Limit test results are presented in Figure 618-PC-1, in Appendix C. All test results are provided in the record of borehole sheets.

In addition to soil index properties, two (2) soil samples were shipped to AGAT Laboratories in Mississauga, Ontario and SGS Canada Inc. in Mississauga, Ontario for corrosivity chemical testing, including determination of sulphate, sulphide, and chloride contents, as well as pH value and resistivity, required to determine the corrosivity characteristics of soils at the structure location.

Bedrock was encountered in some boreholes drilled in the original (2017) and additional (2021) investigation, and unconfined compression strength (UCS) tests were conducted on rock cores taken from Borehole 35-618-05, at a depth of 23.5 m to 23.8 m and from Borehole 35-618-08, at a depth of 32.2 m to 32.5 m. The results of the USC tests are provided in Appendix D.

5. SITE GEOLOGY AND SUBSURFACE CONDITIONS

5.1 Physiography and Regional Geology

The MBI area is located within the western flank of the northeast to southwest trending Paris Moraine. The Paris Moraine is characterized by a broad band of high-relief hummock topography with hilly irregular slopes and enclosed basins, as demonstrated by the presence of frequent small ponds and marshy areas. The geomorphic elements include hummocks, front and back slopes, as well as flat, ridge and depressions. The Paris Moraine is composed of an extensive network of coarsely stratified sand and gravel deposits on adjacent outwash plains.

The Quaternary Geology map published by the Ontario Ministry of Northern Development and Mines (MNDM), indicated that the subsurface conditions in the area contain predominantly sandy silt to silt matrix of the Wentworth Till. The bedrock in the area belongs to the Lower Silurian sandstone, shale, dolostone, and siltstone of the Guelph Formation. The Guelph Formation is identified as an important aquifer in City of Guelph and surrounding areas.

5.2 Subsurface Conditions

The subsurface conditions encountered during the 2017 and 2021 foundation investigations along with the laboratory test results are presented in the record of borehole sheets, in Appendix B. The borehole locations and updated stratigraphic profile are provided in Drawing 35-618-2. The boundaries between soil strata in this stratigraphic profile were established at borehole locations only, and using non-continuous sampling methods. The boundaries represent a transition from one soil type to another, and should not be inferred to represent an exact plane of geological change, as the subsurface conditions may vary between and beyond the boreholes.

In general, the stratigraphy at the project site (proposed location of the Wellington Road 34 Connector Underpass) consists of topsoil underlain by approximately 500 mm to 600 mm thick silty sand or sandy silt fill. Beneath the fill, a silty sand/sandy silt native material (till) with varying proportions of gravel was encountered underlain by Dolostone Bedrock.

The subsurface conditions encountered during the site investigation carried out by PML can be categorized into four (4) general soil layers. The following sections provide brief descriptions of the soil layers and subsurface conditions encountered during the investigation.

5.2.1 Topsoil

A surficial layer of topsoil was encountered in all boreholes. The thickness of the topsoil was approximately 200 mm to 300 mm.

5.2.2 Fill - Silty Sand or Sandy Silt

Fill, consisting of loose to compact silty sand/sandy silt, was encountered immediately below the topsoil in all boreholes. The fill was 500 mm to 800 mm thick, extending to a depth of approximately 0.8 m below existing ground surface in all boreholes.

The SPT ‘N’ values within the fill ranged between 4 blows and 15 blows/300 mm, indicating a loose to compact state of denseness.

The moisture contents of the samples tested from the fill ranged from 6.6% to 22.8%, with an average value of 12.9%.

5.2.3 Silty Sand/Sandy Silt (Till)

Loose to very dense silty sand/sandy silt till with occasional zones of gravel, was encountered immediately below the fill material in all boreholes. This layer extends to depths varying from 9.8 m to 31.4 m (El. 318.3 to El. 296.6) below the existing ground surface.

The SPT ‘N’ values in the silty sand/sandy silt till varied with depth from 5 blows/300 mm to spoon refusal (100 blows/300 mm penetration), indicating a loose to very dense state of denseness.

The moisture contents of samples tested from this deposit ranged from 1.4% to 23.2%.

The results of the grain size distribution analyses conducted on representative samples of the till are provided in Figures 618-GS-1A, 618-GS-1B, 618-GS-1C, 618-GS-1D and 618-GS-2 in Appendix C. The Atterberg limit test results are presented in Figure 618-PC-1, in Appendix C.

5.2.4 Dolostone Bedrock

Bedrock was encountered in three (3) boreholes (35-618-02, 35-618-05, and 35-618-06), drilled in 2017, at approximate depths ranging from 21.4 m to 31.4 m (El. 301.2 to El. 296.6), below the existing ground surfaces. Bedrock was also encountered in Borehole 35-618-08 drilled in 2021 at a depth of 31.4 m (El. 297.1). The presence of bedrock in all four (4) boreholes was confirmed by obtaining 3.4 m to 3.8 m long NQ and HQ sized rock cores using double core barrels. The depths/elevations to bedrock surface at each borehole location are summarised in Table 2.

Table 2 - Depth to Top Surface of the Bedrock

FOUNDATION ELEMENT	REFERENCE BOREHOLE	DEPTH TO BEDROCK (M)	BEDROCK ELEVATION
West Abutment	35-618-02	31.4	296.6
Centre Pier	35-618-05	21.4	300.1
Near East Abutment (Toe of the Road Cut)	35-618-06	22.1	301.2
East Abutment	35-618-08	31.4	297.1

The rock core recovery from all four boreholes ranged from 72% to 100%. The Rock Quality Designation (RQD) of the rock cores ranged from 34% to 95%. Based on the RQD values, the quality of the bedrock to about El. 297.5 to El. 295 may be described as poor to fair. Below these depths, the quality of the bedrock may be described as good to excellent.

The bedrock was identified as slightly to moderately weathered dolostone belonging to the Guelph Formation. Rock core photographs and rock core description logs are provided in Appendix D.

The unconfined compressive strength (UCS) of a rock core from the top part of the bedrock in Borehole 35-618-05 was 58.5 MPa. The UCS value of a rock core taken from Borehole 35-618-08 is 154.3 MPa. Based on these test results, the bedrock may be classified as strong to very strong with respect to strength of intact bedrock. Details of the UCS tests are provided in Appendix D.

5.3 Groundwater Conditions

Groundwater was observed in all boreholes during drilling, except borehole 35-618-03A drilled in November 2017, and 35-618-07 and 35-618-09 completed in July, 2021. The depths of groundwater levels observed during drilling varied from 5.5 m to 18.6 m (El. 322.6 to El. 307.8) below the existing ground surface. Stabilized groundwater levels were also measured in wells installed in Boreholes 35-618-02, 35-618-06 and 35-618-08. Refer to Table 3 for groundwater level readings.

Table 3 - Groundwater Monitoring Well Readings

BOREHOLE	WELL INSTALLATION DATE	WELL SCREEN DEPTH (m)	DATE	DEPTH (M)	ELEVATION (m)
35-618-02	Nov. 27, 2017	21.3 – 22.9	December 08, 2017	17.6	310.4
			January 10, 2018	17.6	310.4
			January 18, 2018	17.3	310.7
			February 15, 2018	17.3	310.7
			April 03, 2018	17.2	310.8
			August 04, 2021	17.4	310.6
			August 11, 2021	17.7	310.3
			August 17, 2021	17.8	310.2
35-618-06	Dec. 01, 2017	15.2 – 16.8	December 08, 2017	11.9	311.4
			January 10, 2018	11.6	311.7
			January 17, 2018	11.6	311.7
			April 03, 2018	11.5	311.8
			August 19, 2021	12	311.3
			August 24, 2021	12	311.3
35-618-08	August 10, 2021	32.9 – 34.4	August 11, 2021	16.1	312.4
			August 16, 2021	17.5	311.1
			August 19, 2021	16	312.5

It should be noted that the groundwater levels and gradient at the site may be influenced by the road structure, topography, underlying geology, and the water level in surrounding wetland, and may fluctuate because of seasonal changes, periods of precipitation, and temperature.

5.4 Chemical Test Results

A summary of the corrosivity test results conducted on samples of the silty sand/sandy silt till is provided in Table 4. The samples were taken from Borehole 35-618-04, at El. 319.7 and Borehole 35-618-08, at El. 325.2. The descriptions of the test methods are given in Appendix E.

Table 4 - Summary of Corrosivity Test Results

BOREHOLE NO.	ELEVATION (M)	SOIL TYPE	SULPHIDE (%)	SULPHATE (µg/g, ppm)	CHLORIDE (µg/g, ppm)	pH	RESISTIVITY (Ohm-cm)
35-618-04	319.7	Silty Sand/ Sandy Silt Till	< 0.05	6	21	8.66	9620
35-618-08	325.2		<0.04	9.4	7.1	8.45	7580

6. CLOSURE

This report was prepared by Mr. Lul Yimam, PhD, P.Eng., and reviewed by Mr. Geoffrey. Uwimana, MEng., P.Eng., Senior Engineer and MTO Designated Principal Contact.

Yours very truly

Peto MacCallum Ltd.

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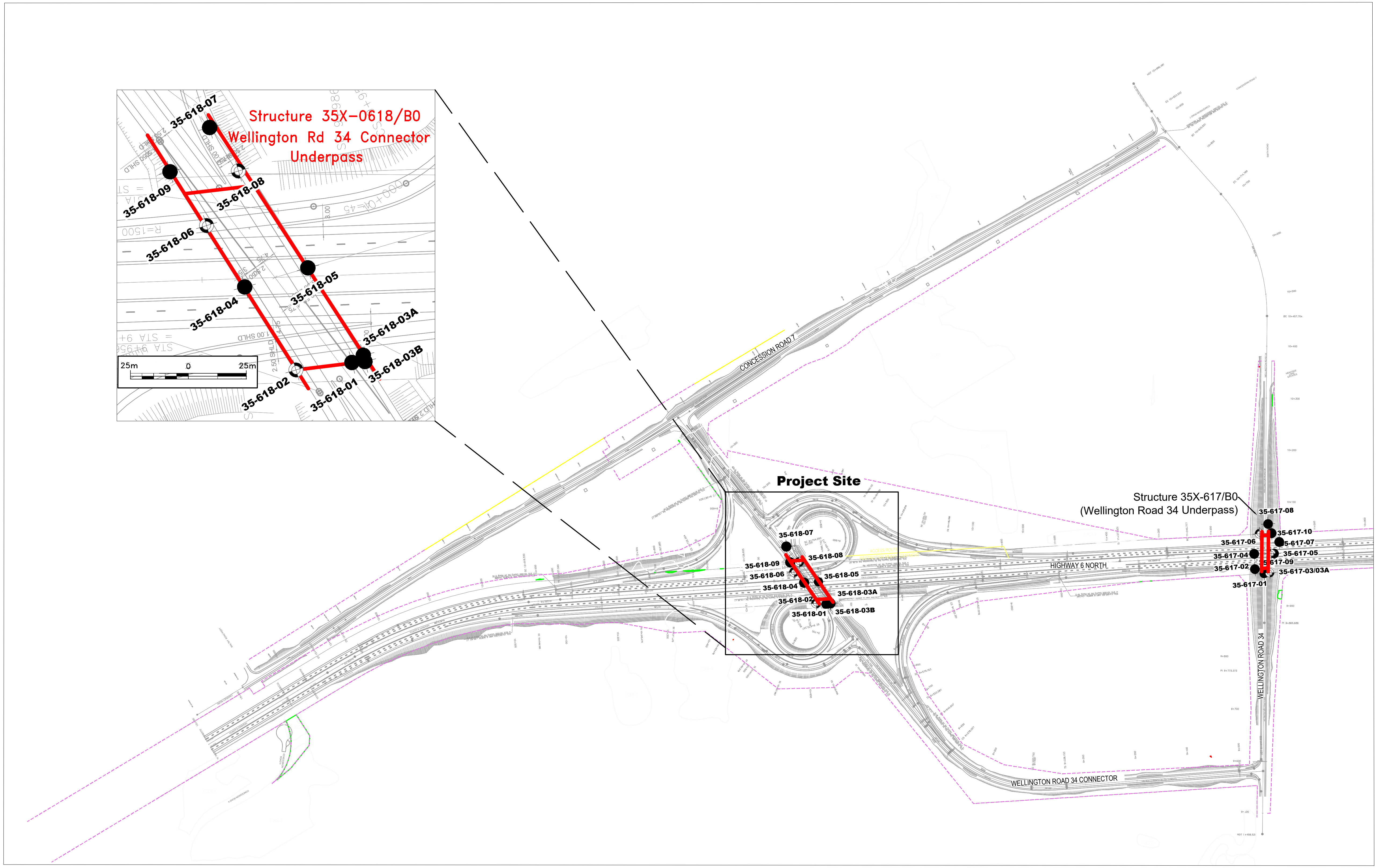
LY/GU/:ly-nk



PLAN SCALE 50m 0 50m 100m 200m

NOTES:

1. THIS DRAWING SHOULD BE READ IN CONJUNCTION WITH THE TEXT OF REPORT AND RECORD OF BOREHOLE LOGS.
2. DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS ARE IN KILOMETRES AND METRES.

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100500748
Oct 14/2021

G. G. UWIMANA
Oct 14, 2021

Reference AECOM Drawing: 01_Wellington Rd._GA.dwg, dated Aug. 2017

GWP No 3059-20-00
Design-Build Ready Alternative Bid Package
Midblock Interchange Area

Hwy 401& Hwy 6 Improvements
From Hamilton North Limits to Guelph South limits
SITE and BOREHOLE LOCATION PLAN

SHEET

Peto MacCallum Ltd.
CONSULTING ENGINEERS

KEY PLAN
500m 0 500m 1Km 2km

LEGEND

- ● Borehole
- ⊕ Borehole with 50mm monitoring well

Borehole Number	Northing (MTM ON10)	Easting (MTM ON10)	Elevation	Borehole Depth(m)
35–618–01	4814069.0	249441.2	328.1	9.8
35–618–02	4814088.5	249438.0	328.0	34.8
35–618–03A	4814062.3	249445.5	328.0	13.3
35–618–03B	4814061.4	249444.5	328.0	14.7
35–618–04	4814119.6	249471.6	322.3	20.5
35–618–05	4814093.6	249479.6	321.5	25.0
35–618–06	4814143.6	249487.0	323.3	25.9
35–618–07	4814145.9	249520.1	328.2	10.5
35–618–08	4814138.6	249508.2	328.5	34.5
35–618–09	4814155.2	249503.6	329.7	29.9

– NOTE –

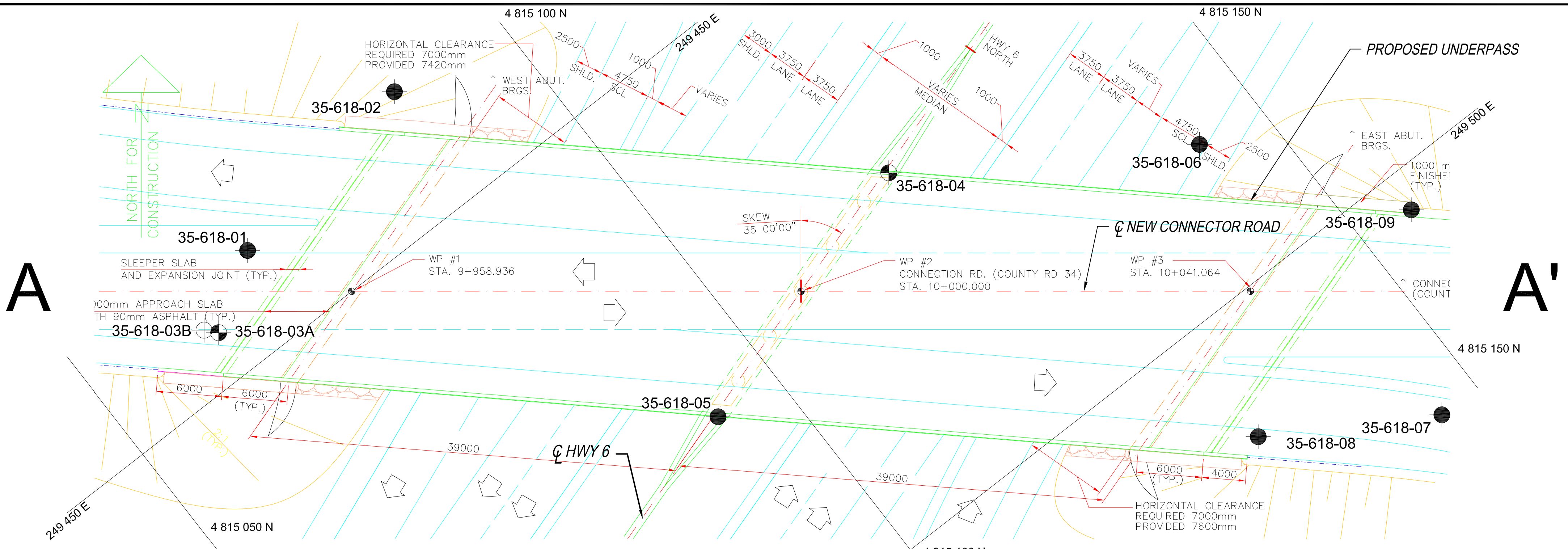
The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.

REVISIONS

DATE	BY	DESCRIPTION

Geocres No. 40P8–291

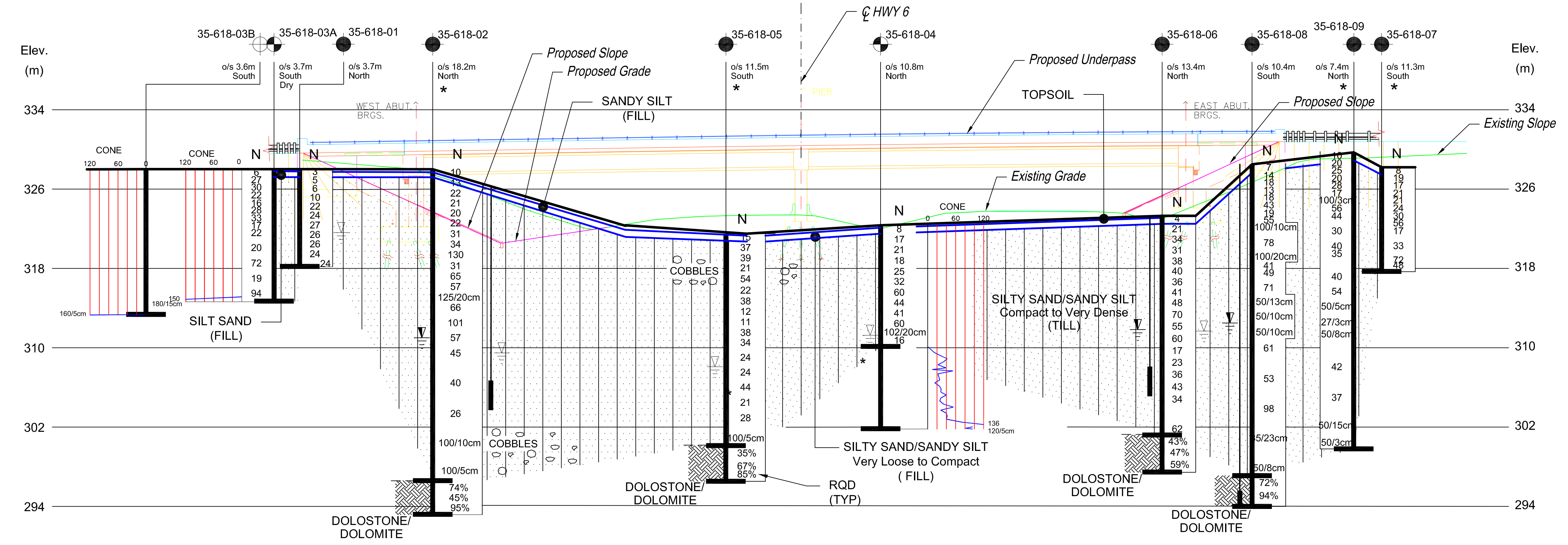
HWY No 6			DIST 31
SUBM'D LY	CHECKED LY	DATE Oct 14, 2021	SITE MBI Area
DRAWN FM	CHECKED GU	APPROVED GU	DWG 35–618–1



PLAN

SCALE

10 0 10 20m



A-A' PROFILE ALONG ℄ OF PROPOSED WELLINGTON ROAD 34 CONNECTOR UNDERPASS

SCALE

10 0 10 20m

NOTES:

1. THIS DRAWING SHOULD BE READ IN CONJUNCTION WITH THE TEXT OF REPORT AND RECORD OF BOREHOLE LOGS.
2. THIS DRAWING IS FOR SUBSURFACE INFORMATION ONLY. SURFACE DETAILS AND FEATURES ARE FOR CONCEPTUAL ILLUSTRATION.
3. DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS ARE IN KILOMETRES AND METRES.

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Oct 14/2021

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Oct 14, 2021

Reference AECOM Drawing: 01_Wellington Rd_Connector_GA.dwg, dated May 2017

GWP No 3059-20-00
Design- Build Ready Alternative Bid Package
Wellington Road. 34 Connector Underpass
SITE No. 35X - 0618/B0 STA No. 10+000
Long.: 43.464815, Lat.: -80.183.754

MIDBLOCK INTERCHANGE AREA
HWY 401 & HWY 6 IMPROVEMENTS
BOREHOLE LOCATION AND SOIL STRATA

SHEET

Peto MacCallum Ltd.
CONSULTING ENGINEERS

KEY PLAN

LEGEND

●	Borehole
◐	Borehole + DCPT
⊕	DCPT
N	Blows/0.3m (Std. Pen Test, 475 J/blow)
RQD	Rock Quality Designation
CONE	Blows/0.3m (60 Cone, 475 J/blow)
▽	WL measured in monitoring well
▽	WL observed during drilling
▽*	WL observed during drilling
*	WL could not be established
|	Piezometer

BH No	ELEVATION	CO-ORDINATES (MTM ON10) NORTHINGS	EASTINGS
35-618-01	328.1	4 814 069.0	249 441.2
35-618-02	328.0	4 814 088.5	249 438.0
35-618-03A	328.0	4 814 062.3	249 445.5
35-618-03B	328.0	4 814 061.4	249 444.5
35-618-04	322.3	4 814 119.6	249 471.6
35-618-05	321.5	4 814 093.6	249 479.6
35-618-06	323.3	4 814 143.6	249 487.0
35-618-07	328.2	4 814 145.9	249 520.1
35-618-08	328.5	4 814 138.6	249 508.2
35-618-09	329.7	4 814 155.2	249 503.6

– NOTE –

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

REVISIONS

DATE	BY	DESCRIPTION

Geocres No. 40P8-291

HWY No 6			DIST 31
SUBM'D NL	CHECKED LY	DATE October 14, 2021	SITE 35X- 0618/B0
DRAWN NL	CHECKED	APPROVED GU	DWG 35-618-2

APPENDIX A

Site Photographs

Photograph 1 – Wellington Road 34 Connector Underpass East Abutment

Photograph 2 – Wellington Road 34 Connector Underpass West Abutment

Photograph 3 - The West Abutment - Looking South

Photograph 4 - The East Abutment - Looking South

Photograph 5 - Drilling at Borehole 35-618-08 – Looking North

Photograph 6 - Drilling at Borehole 35-618-09 - Looking North

APPENDIX B

Explanation of Terms Used on Boreholes and in the Report

Record of Borehole Sheets

EXPLANATION OF TERMS USED IN REPORT

N VALUE: THE STANDARD PENETRATION TEST (SPT) N VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O.D. SPLIT BARREL SAMPLER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5kg, FALLING FREELY A DISTANCE OF 0.76m. FOR PENETRATIONS OF LESS THAN 0.3m N VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N VALUE IS DENOTED THUS $\bar{N}$.

DYNAMIC CONE PENETRATION TEST: CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60° CONE ANGLE) DRIVEN BY 475 J IMPACT ENERGY ON 'A' SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

COMPOSITION: SECONDARY SOIL COMPONENTS ARE DESCRIBED ON THE BASIS OF PERCENTAGE BY MASS OF THE WHOLE SAMPLE AS FOLLOWS:

PERCENT BY MASS	0 – 10	10 – 20	20 – 30	30 – 40	> 40
	TRACE	*SOME*	*WITH*	*ADJECTIVE (SILTY)*	*AND (AND SILT)*

CONSISTENCY: COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH (c_u) AS FOLLOWS:

c_u (kPa)	0 - 12	12 - 25	25 - 50	50 - 100	100 - 200	>200
	VERY SOFT	*SOFT*	*FIRM*	*STIFF*	*VERY STIFF*	*HARD*

DENSENESS: COHESIONLESS SOILS ARE DESCRIBED ON THE BASIS OF DENSENESS AS INDICATED BY SPT N VALUES AS FOLLOWS:

N (BLOWS/0.3m)	0 - 5	5 - 10	10 - 30	30 - 50	>50
	VERY LOOSE	*LOOSE*	*COMPACT*	*DENSE*	*VERY DENSE*

ROCKS ARE DESCRIBED BY THEIR COMPOSITION AND STRUCTURAL FEATURES AND / OR STRENGTH.

RECOVERY: SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH OF THE CORING RUN.

MODIFIED RECOVERY: SUM OF THOSE INTACT CORE PIECES, 100mm+ IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (R Q D), FOR MODIFIED RECOVERY, IS:

RQD (%)	0 - 25	25 - 50	50 - 75	75 - 90	90 - 100
	VERY POOR	*POOR.*	*FAIR*	*GOOD*	*EXCELLENT*

JOINTING AND BEDDING:

SPACING	50mm	50 - 300mm	0.3m - 1m	1m - 3m	>3m
JOINTING	*VERY CLOSE*	*CLOSE*	*MOD. CLOSE*	*WIDE*	*VERY WIDE*
BEDDING	*VERY THIN*	*THIN*	*MEDIUM*	*THICK*	*VERY THICK*

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

S S	SPLIT SPOON	T P	THINWALL PISTON
W S	WASH SAMPLE	O S	OSTERBERG SAMPLE
S T	SLOTTED TUBE SAMPLE	R C	ROCK CORE
B S	BLOCK SAMPLE	P H	T W ADVANCED HYDRAULICALLY
C S	CHUNK SAMPLE	P M	T W ADVANCED MANUALLY
T W	THINWALL OPEN	F S	FOIL SAMPLE
F V	FIELD VANE		

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
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

MECHANICAL PROPERTIES OF SOIL

m_v	kPa^{-1}	COEFFICIENT OF VOLUME CHANGE
C_c	1	COMPRESSION INDEX
C_s	1	SWELLING INDEX
C_α	1	RATE OF SECONDARY CONSOLIDATION
c_v	m^2/s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	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 $= \frac{c_u}{\tau_r}$

PHYSICAL PROPERTIES OF SOIL

ρ_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	1	LIQUIDITY INDEX $= \frac{w - w_P}{I_P}$
I_C	1	CONSISTENCY INDEX $= \frac{w_L - w}{I_P}$
DTPL		DRIER THAN PLASTIC LIMIT
APL		ABOUT PLASTIC LIMIT
WTPL		WETTER THAN PLASTIC LIMIT

e_{max}	1,%	VOID RATIO IN LOOSEST STATE
e_{min}	1,%	VOID RATIO IN DENSEST STATE
I_D	1	DENSITY INDEX $= \frac{e_{max} - e}{e_{max} - e_{min}}$
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 35-618-01

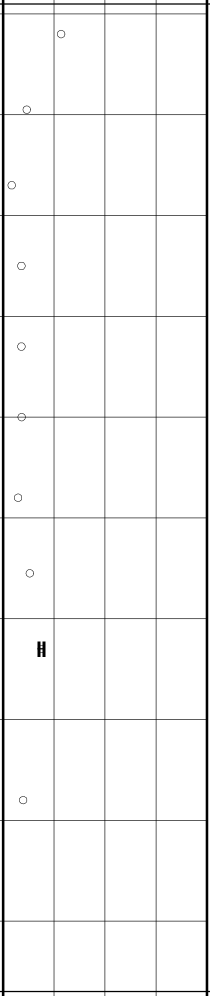
1 OF 1 **METRIC**

G.W.P. 3059-20-00
LOCATION Coords: 4 814 069.0 N; 249 441.2 E (MTM ON10)
ORIGINATED BY M.F.
DIST 31 HWY 6
BOREHOLE TYPE Hollow Stem Augers
COMPILED BY L.Y.
DATUM Geodetic
DATE 2017.11.27
LATITUDE 43.464480
LONGITUDE -80.184150
CHECKED BY G.U.

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	GR	SA	SI	CL
328.1	GROUND SURFACE					328				
327.9 / 0.0	TOPSOIL									
0.2	SILTY SAND Very loose, Brown, Moist (FILL)	1	SS	3						
327.3 / 0.8	SILTY SAND/SANDY SILT, trace/some gravel Loose to compact, Brown, Moist to wet (TILL)	2	SS	5		327				
		3	SS	6		326	2	57	37	4
		4	SS	10						
		5	SS	22		325	20	38	38	4
		6	SS	24		324				
		7	SS	27		323				
		8	SS	26	∇					
		9	SS	26		322	2	28	58	12
						321				
		10	SS	24		320				
						319				
		11	SS	24						
318.3 / 9.8	End of borehole									

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; 20 40 60 80 100



UNIT WEIGHT γ kN/m^3

REMARKS & GRAIN SIZE DISTRIBUTION (%)

∇ Groundwater level observed during drilling

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

RECORD OF BOREHOLE No 35-618-02

1 OF 3 **METRIC**

G.W.P. 3059-20-00	LOCATION Coords: 4 814 088.5 N; 249 438.0 E (MTM ON10)	ORIGINATED BY M.F.
DIST 31 HWY 6	BOREHOLE TYPE Hollow Stem Augers, Mud Rotary Drilling, NQ Rock Coring	COMPILED BY L.Y.
DATUM Geodetic	DATE 2017.11.23 - 2017.11.27 LATITUDE 43.464605 LONGITUDE -80.184004	CHECKED BY G.U.

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; 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

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
328.0	GROUND SURFACE								
0.0	TOPSOIL								
327.7 0.3	SANDY SILT Loose, Brown, Moist (FILL)	1	SS	10					
327.2 0.8	SILTY SAND/SANDY SILT, trace/some gravel Compact, Brown, Moist (TILL)	2	SS	13	327				
		3	SS	22	326	0	18	73	9
	gravel, with sand	4	SS	21	325	61	33	5	1
		5	SS	20					
		6	SS	22	324	0	5	79	16
	Dense to very dense	7	SS	31	323				
		8	SS	34	322	23	39	34	4
		9	SS	130	321				
		10	SS	31	320	17	51	28	4
					319				
		11	SS	65	318				
		12	SS	57	317				
					316				
	with gravel	13	SS	125/20cm	315				
		14	SS	66	314	45	51	(4)	
313.0									

H (plotted at sample 6, water content column)

Continued Next Page

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

RECORD OF BOREHOLE No 35-618-02

2 OF 3 **METRIC**

G.W.P. 3059-20-00
LOCATION Coords: 4 814 088.5 N; 249 438.0 E (MTM ON10)
ORIGINATED BY M.F.
DIST 31 HWY 6
BOREHOLE TYPE Hollow Stem Augers, Mud Rotary Drilling, NQ Rock Coring
COMPILED BY L.Y.
DATUM Geodetic
DATE 2017.11.23 - 2017.11.27
LATITUDE 43.464605
LONGITUDE -80.184004
CHECKED BY G.U.

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES
313.0				
15.0	SILTY SAND/SANDY SILT, with gravel Dense to very dense, Brown, Moist to wet (TILL)			
		15	SS	101
		16	SS	57
		17	SS	45
		18	SS	40
		19	SS	26
	occasional cobbles	20	SS	100/10cm
298.0				

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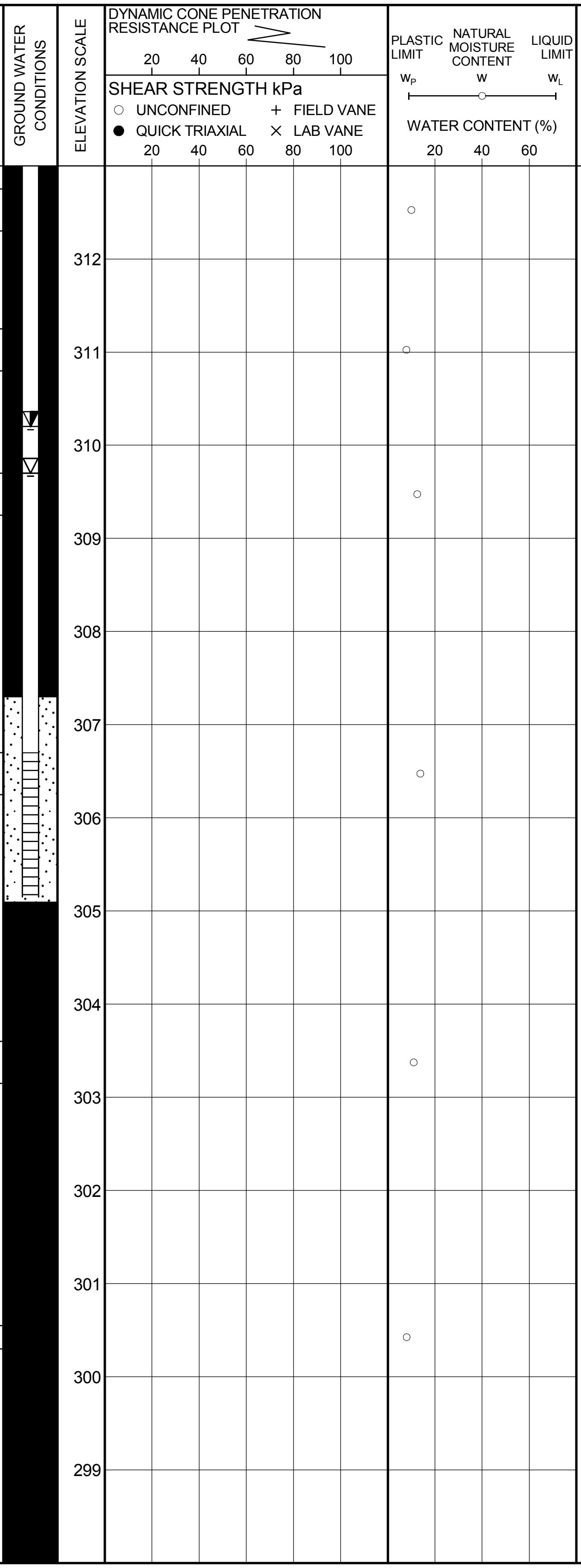

UNIT WEIGHT γ kN/m³

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

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

RECORD OF BOREHOLE No 35-618-02

3 OF 3 **METRIC**

G.W.P. 3059-20-00 LOCATION Coords: 4 814 088.5 N; 249 438.0 E (MTM ON10) ORIGINATED BY M.F.

DIST 31 HWY 6 BOREHOLE TYPE Hollow Stem Augers, Mud Rotary Drilling, NQ Rock Coring COMPILED BY L.Y.

DATUM Geodetic DATE 2017.11.23 - 2017.11.27 LATITUDE 43.464605 LONGITUDE -80.184004 CHECKED BY G.U.

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 × LAB VANE 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	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
298.0											
30.0	SILTY SAND/SANDY SILT, occasional cobbles Very dense, Brown, Wet (TILL)		21	SS	100/5cm				○		
							297				
296.6 31.4	DOLOSTONE/DOLOMITE BEDROCK Slightly weathered		22	RC NQ	REC 99%						RQD 74%
							296				
			23	RC NQ	REC 100%						RQD 45%
							295				
	Moderately weathered		24	RC NQ	REC 96%		294				RQD 95%
293.2 34.8	End of borehole										

∇ Groundwater level observed during drilling

▼ Groundwater level measured in monitoring well

NOTES:

1. Borehole charged with drilling water, thus groundwater level could not be measured upon completion of drilling
2. The presence of cobbles is inferred by auger grinding observed during drilling and is not indicative of quantity.

Monitoring Well Legend:

Date	Depth (m)	Elev.
Dec.08/'17	17.6	310.4
Jan.10/'18	17.6	310.4
Jan.18/'18	17.3	310.7
Feb.15/'18	17.3	310.7
Apr.03/'18	17.2	310.8
Jul. 30/'21	18.0	310.0
Aug. 04/'21	17.4	310.6
Aug. 11/'21	17.7	310.3
Aug. 17/'21	17.8	310.2

Monitoring Well Readings:

☐☐☐	Monument casing
■☐■	Bentonite seal
⁙☐⁙	Filter sand
⁙⊟⁙	Screen

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

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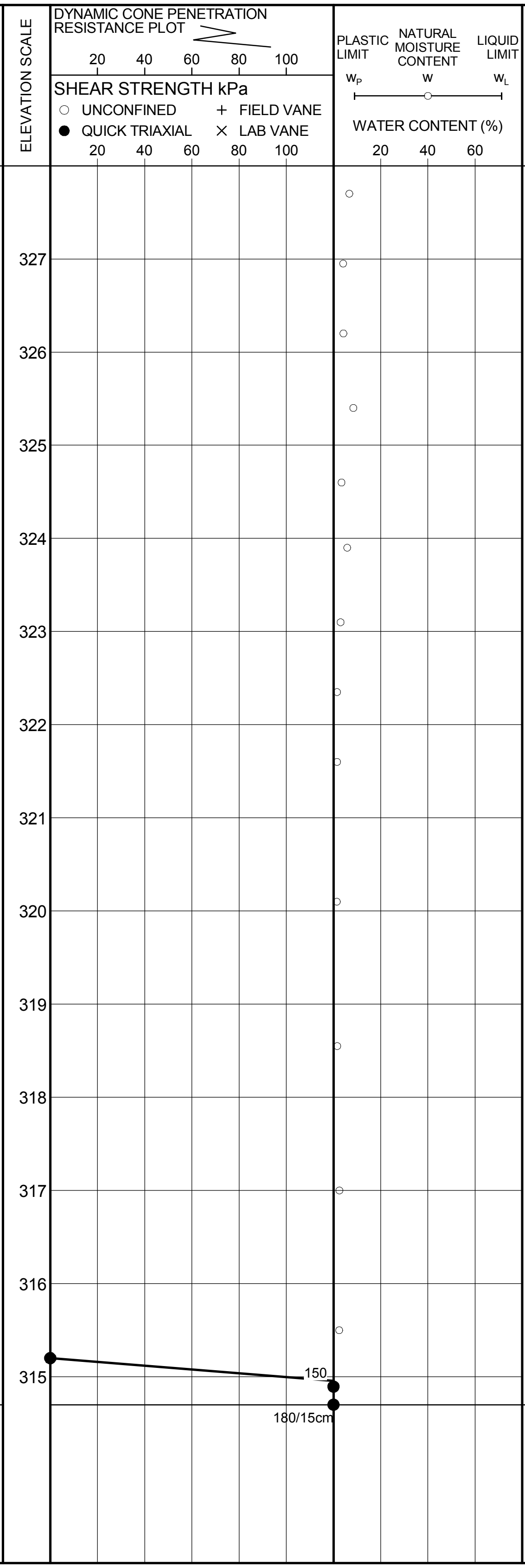
RECORD OF BOREHOLE No 35-618-03A

1 OF 1 **METRIC**

G.W.P. 3059-20-00
LOCATION Coords: 4 814 062.3 N; 249 445.5 E (MTM ON10)
ORIGINATED BY M.F.
DIST 31 HWY 6
BOREHOLE TYPE Hollow Stem Augers, Cone Penetration Test
COMPILED BY L.Y.
DATUM Geodetic
DATE 2017.11.28
LATITUDE 43.464620 LONGITUDE -80.183844
CHECKED BY G.U.

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES			GR	SA	SI	CL
328.0	GROUND SURFACE									
327.8 0.0	TOPSOIL									
0.2	SILTY SAND Loose, Brown, Moist (FILL)	1	SS	6						
327.2 0.8	SILTY SAND/SANDY SILT, some/with gravel Compact to very dense, Brown, Moist (TILL)	2	SS	27						
		3	SS	30						
		4	SS	22			17	16	60	7
		5	SS	16						
		6	SS	28						
		7	SS	33			32	46	19	3
		8	SS	17						
		9	SS	22						
		10	SS	20						
		11	SS	72			27	59	11	3
		12	SS	19						
		13	SS	94						
315.2 12.8	End of borehole Switched to cone penetration test									
314.7 13.3	End of cone penetration test									



NOTES:
1. Groundwater was not encountered during and after completion of drilling.

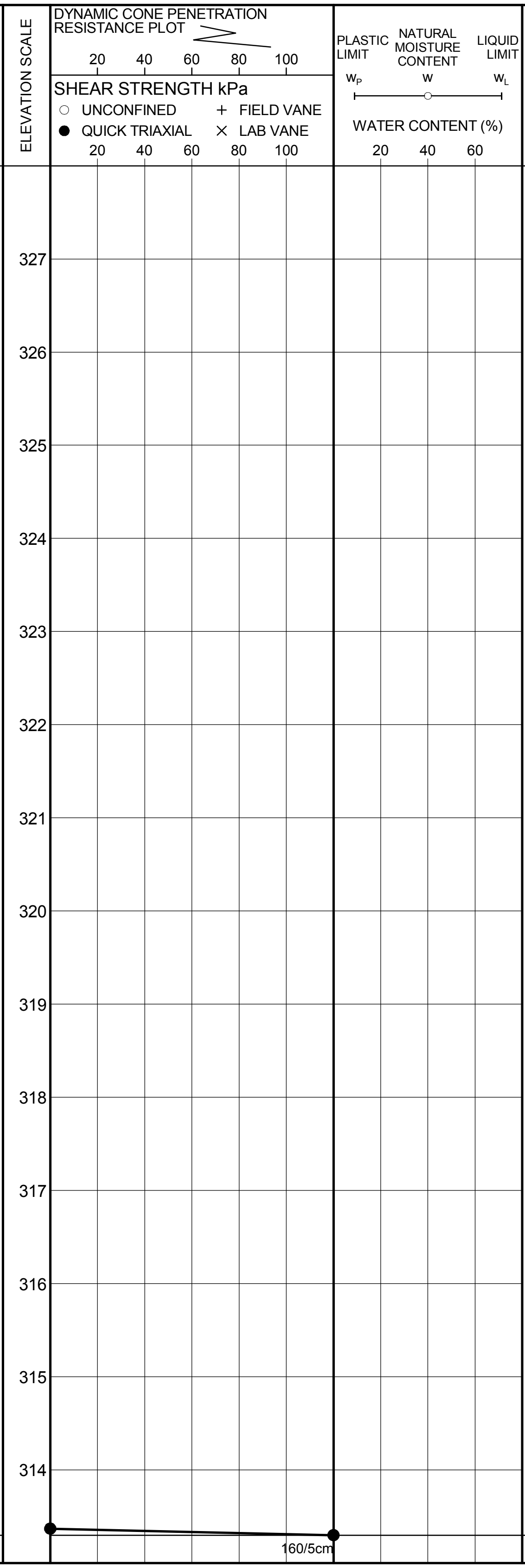
$+^3, \times^3$: Numbers refer to Sensitivity
$\bigcirc^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No 35-618-03B

1 OF 1 **METRIC**

G.W.P. 3059-20-00
LOCATION Coords: 4 814 061.4 N; 249 444.5 E (MTM ON10)
ORIGINATED BY M.F.
DIST 31 HWY 6
BOREHOLE TYPE Hollow Stem Augers, Cone Penetration Test
COMPILED BY L.Y.
DATUM Geodetic
DATE 2017.11.28
LATITUDE 43.464620
LONGITUDE -80.183844
CHECKED BY G.U.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
328.0	GROUND SURFACE										
0.0	Borehole 618-3B was drilled 1.2m west of Borehole 618-3A Augered to 14.6m depth										
313.4 14.6	Cone penetration test										
313.3 14.7	End of cone penetration test										



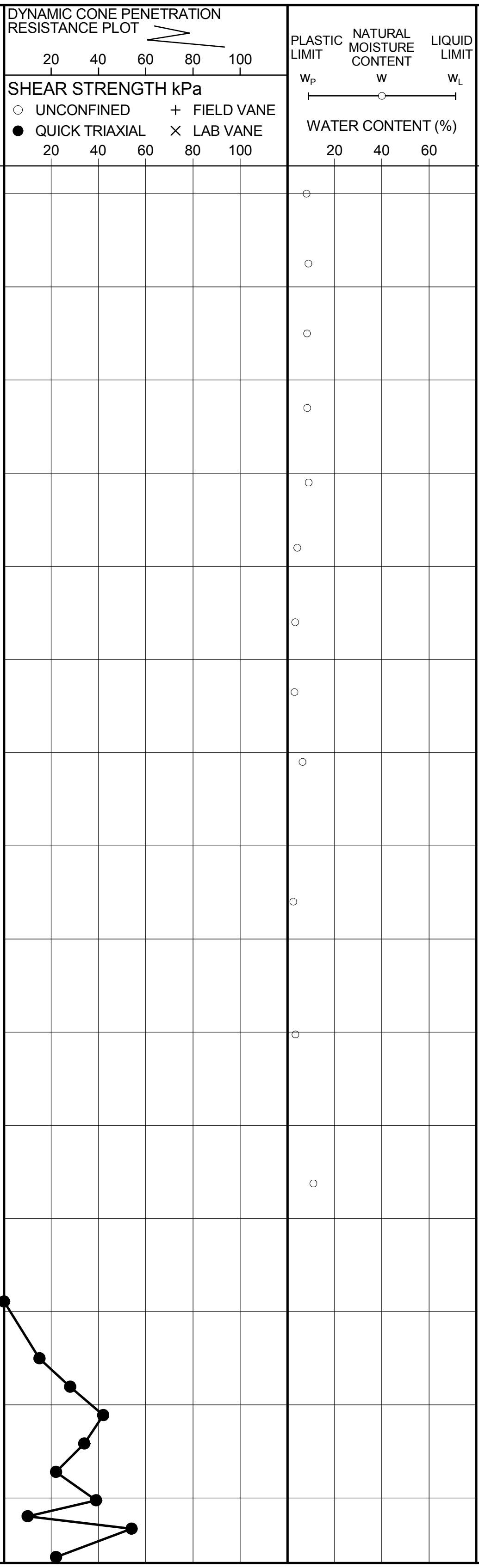
+³, ×³: Numbers refer to Sensitivity
○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No 35-618-04

1 OF 2 **METRIC**

G.W.P. 3059-20-00
LOCATION Coords: 4 814 119.6 N; 249 471.6 E (MTM ON10)
ORIGINATED BY M.F.
DIST 31 HWY 6
BOREHOLE TYPE Hollow Stem Augers, Cone Penetration Test
COMPILED BY L.Y.
DATUM Geodetic
DATE 2017.12.07 - 2017.12.08 LATITUDE 43.464940 LONGITUDE -80.183780
CHECKED BY G.U.

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	GR	SA	SI	CL
322.3	GROUND SURFACE									
322.1 0.0	TOPSOIL					322				
0.2	SANDY SILT Loose, Brown, Moist (FILL)	1	SS	8						
321.5 0.8	SILTY SAND/SANDY SILT, with gravel Compact, Brown, Moist (TILL)	2	SS	17		321	28	48	21	3
		3	SS	21		320				
		4	SS	18						
		5	SS	25		319				
	Dense to very dense	6	SS	32		318				
		7	SS	60		317	32	49	17	2
		8	SS	44						
		9	SS	41		316				
						315				
		10	SS	60		314				
		11	SS	102/20cm		313	39	48	11	2
	wet	12	SS	16	⊽	312 311				
310.1 12.2	End of borehole Switched to cone penetration test					310 309 308				
307.3										



UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%)

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$+^3, \times^3$: Numbers refer to Sensitivity

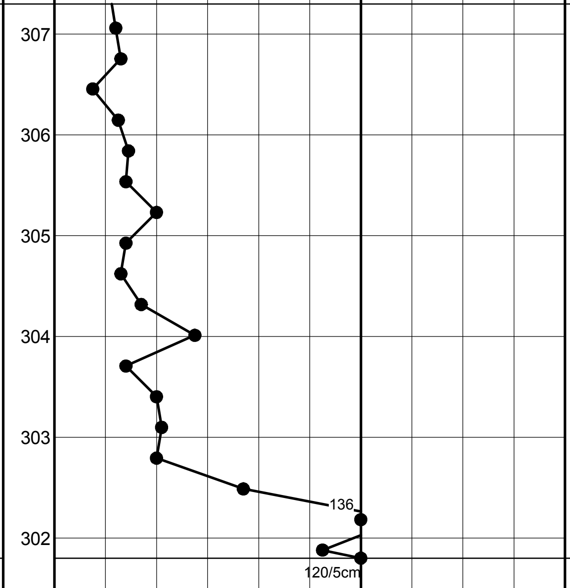
○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No 35-618-04

2 OF 2 **METRIC**

G.W.P. 3059-20-00
LOCATION Coords: 4 814 119.6 N; 249 471.6 E (MTM ON10)
ORIGINATED BY M.F.
DIST 31 HWY 6
BOREHOLE TYPE Hollow Stem Augers, Cone Penetration Test
COMPILED BY L.Y.
DATUM Geodetic
DATE 2017.12.07 - 2017.12.08
LATITUDE 43.464940
LONGITUDE -80.183780
CHECKED BY G.U.

SOIL PROFILE			SAMPLES									
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 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/m^3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
307.3											
15.0	Cone penetration test										
301.8 20.5	End of cone penetration test Refusal on probable bedrock										



∇ Groundwater level observed during drilling

NOTE: Borehole continued with cone penetration test, thus groundwater level could not be determined upon completion.

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

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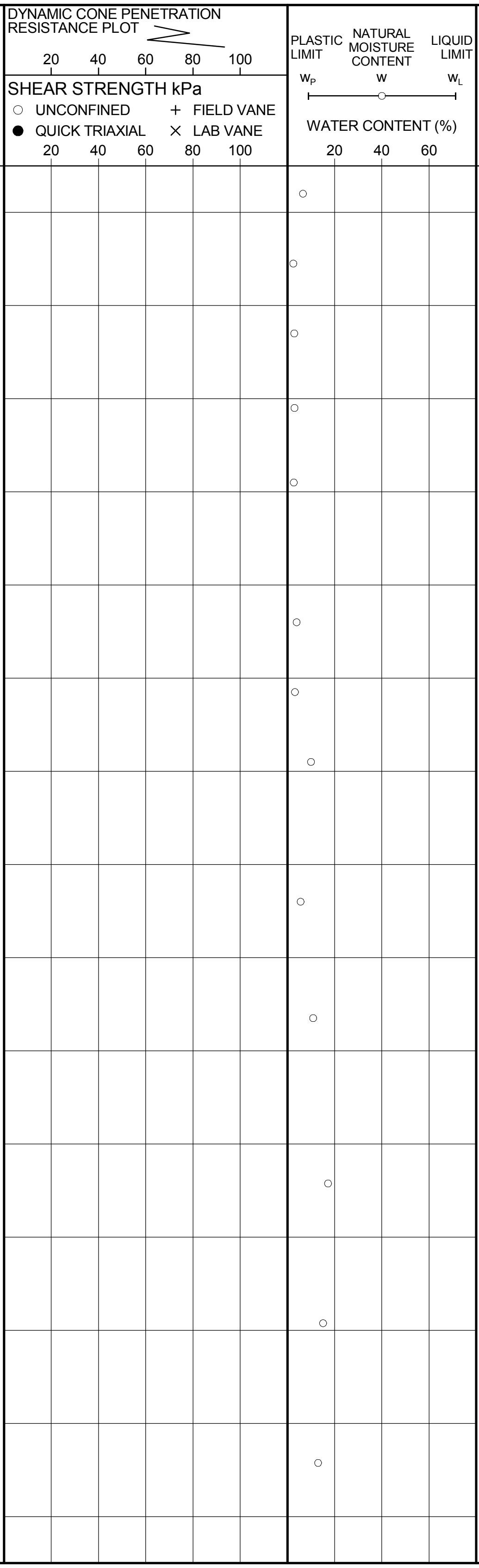
RECORD OF BOREHOLE No 35-618-05

1 OF 2 **METRIC**

G.W.P. 3059-20-00	LOCATION Coords: 4 814 093.6 N; 249 479.6 E (MTM ON10)		ORIGINATED BY M.F.
DIST 31 HWY 6	BOREHOLE TYPE Hollow Stem Augers, Mud Rotary Drilling, NQ Rock Coring		COMPILED BY L.Y.
DATUM Geodetic	DATE 2017.12.04 - 2017.12.07	LATITUDE 43.464710 LONGITUDE -80.183670	CHECKED BY G.U.



SOIL PROFILE		SAMPLES					UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE		GR	SA	SI	CL
321.5	GROUND SURFACE										
321.3 0.0	TOPSOIL										
0.2	SANDY SILT Compact, Brown, Moist (FILL)	1	SS	15		321					
320.7 0.8	SILTY SAND/SANDY SILT, with/some gravel Dense to compact, Brown, Moist (TILL)	2	SS	37		320					
	gravel, with sand	3	SS	39				57	35	(8)	
	cobbles	4	SS	21		319					
		5	SS	54		318					
		6	SS	22		317		36	45	17	2
		7	SS	38		316					
		8	SS	12				4	90	(6)	
		9	SS	11		315					
						314					
		10	SS			313		20	64	13	3
		11	SS	38		312					
						311					
		12	SS	34		310					
		13	SS	24		309					
	wet				∇	308					
		14	SS	24		307					
306.5											

Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No 35-618-05

2 OF 2 **METRIC**

G.W.P. 3059-20-00
LOCATION Coords: 4 814 093.6 N; 249 479.6 E (MTM ON10)
ORIGINATED BY M.F.
DIST 31 HWY 6
BOREHOLE TYPE Hollow Stem Augers, Mud Rotary Drilling, NQ Rock Coring
COMPILED BY L.Y.
DATUM Geodetic
DATE 2017.12.04 - 2017.12.07
LATITUDE 43.464710
LONGITUDE -80.183670
CHECKED BY G.U.

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
306.5 15.0	SILTY SAND/SANDY SILT, with gravel Dense to compact, Brown, Wet (TILL)						
		15	SS	44		306	
						305	
		16	SS	21			
						304	
		17	SS	28		303	46 34 16 4
						302	
						301	
300.1		18	SS	100/5cm			
21.4	DOLOSTONE/DOLOMITE BEDROCK Moderately weathered					300	
		19	RC NQ	REC 72%		299	RQD 34% USC=58.5 (MPa)
		20	RC NQ	REC 97%		298	RQD 67%
		21	RC NQ	REC 100%		297	RQD 85%
296.5 25.0	End of borehole						

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; 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

∇ Groundwater level observed during drilling

NOTE:
1. Borehole charged with drilling water, thus groundwater level could not be determined upon completion of drilling.
2. The presence of cobbles is inferred by auger grinding observed during drilling and is not indicative of quantity.

$+^3, \times^3$: Numbers refer to Sensitivity

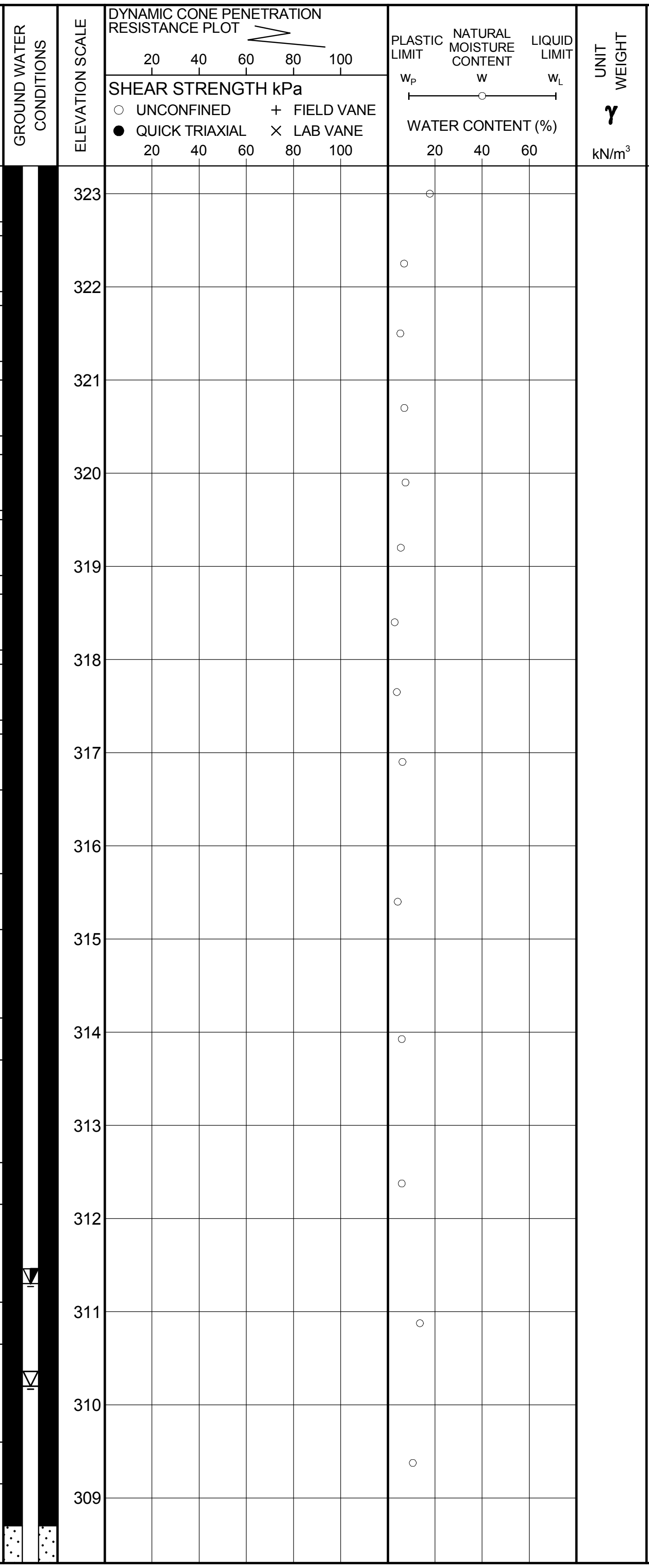
○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No 35-618-06

1 OF 3 **METRIC**

G.W.P. 3059-20-00
LOCATION Coords: 4 814 143.6 N; 249 487.0 E (MTM ON10)
ORIGINATED BY M.F.
DIST 31 HWY 6
BOREHOLE TYPE Hollow Stem Augers, Mud Rotary Drilling, NQ Rock Coring
COMPILED BY L.Y.
DATUM Geodetic
DATE 2017.11.29 - 2017.12.01
LATITUDE 43.465215
LONGITUDE -80.183163
CHECKED BY G.U.

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	GR	SA	SI	CL
323.3	GROUND SURFACE								
323.1 0.0	TOPSOIL								
0.2	SANDY SILT Loose, Brown, Moist (FILL)	1	SS	4	323				
322.5 0.8	SILTY SAND/SANDY SILT, with/trace gravel Compact to dense, Brown, Moist (TILL)	2	SS	21		30	46	19	5
					322				
		3	SS	34					
					321				
		4	SS	31					
					320				
		5	SS	38		33	43	21	3
		6	SS	40					
					319				
		7	SS	36					
					318				
		8	SS	41		0	87	11	2
					317				
		9	SS	48					
					316				
		10	SS	70					
					315				
					314				
		11	SS	55					
					313				
		12	SS	60					
					312				
	wet				311				
		13	SS	17					
					310				
	with gravel	14	SS	23					
					309				
308.3									



REMARKS & GRAIN SIZE DISTRIBUTION (%)

Continued Next Page

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

RECORD OF BOREHOLE No 35-618-06

2 OF 3 **METRIC**

G.W.P. 3059-20-00
LOCATION Coords: 4 814 143.6 N; 249 487.0 E (MTM ON10)
ORIGINATED BY M.F.
DIST 31 HWY 6
BOREHOLE TYPE Hollow Stem Augers, Mud Rotary Drilling, NQ Rock Coring
COMPILED BY L.Y.
DATUM Geodetic
DATE 2017.11.29 - 2017.12.01 LATITUDE 43.465215 LONGITUDE -80.183163
CHECKED BY G.U.

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
308.3						
15.0	SILTY SAND/SANDY SILT, with gravel Dense to very dense, Brown, Wet (TILL)	15	SS	36	308	
					307	
		16	SS	43	306	
		17	SS	34	305	35 54 9 2
					304	
					303	
		18	SS	62	302	
301.2 22.1	DOLOSTONE/DOLOMITE BEDROCK Moderately weathered	19	RC NQ	REC 95%	301	RQD 43%
		20	RC NQ	REC 90%	300	RQD 47%
		21	RC NQ	REC 72%	299 298	RQD 59%
297.4 25.9	End of borehole					

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 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

∇ Groundwater level observed during drilling.

▼ Groundwater measured in monitoring well

NOTES:
1. Borehole charged with drilling water, thus groundwater level could not be measured upon completion of drilling.

Continued Next Page

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

RECORD OF BOREHOLE No 35-618-06

3 OF 3

METRIC

G.W.P. 3059-20-00
LOCATION Coords: 4 814 143.6 N; 249 487.0 E (MTM ON10)
ORIGINATED BY M.F.
DIST 31 HWY 6
BOREHOLE TYPE Hollow Stem Augers, Mud Rotary Drilling, NQ Rock Coring
COMPILED BY L.Y.
DATUM Geodetic
DATE 2017.11.29 - 2017.12.01
LATITUDE 43.465215
LONGITUDE -80.183163
CHECKED BY G.U.

SOIL PROFILE			SAMPLES					DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 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	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE				
293.3											

Monitoring Well Legend:

Date	Depth (m)	Elev.
Dec. 08/'17	11.9	311.4
Jan. 10/'18	11.6	311.7
Jan. 17/'18	11.6	311.7
Apr. 03/'18	11.5	311.8
Aug. 19/'21	12.0	311.3

Monitoring Well Readings:

■□■ Bentonite seal
⁘□⁘ Filter sand
⁘⊟⁘ Screen

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

RECORD OF BOREHOLE No 35-618-07

1 OF 1 **METRIC**

G.W.P. 3059-20-00
LOCATION COORDS: 4 814 145.9 N; 249 520.1 E (MTM ON10)
ORIGINATED BY F.M.
DIST 31 HWY 6
BOREHOLE TYPE Solid Stem Augers
COMPILED BY L.Y.
DATUM Geodetic
DATE 2021.07.12
LATITUDE 43.465184
LONGITUDE -80.183184
CHECKED BY G.U.

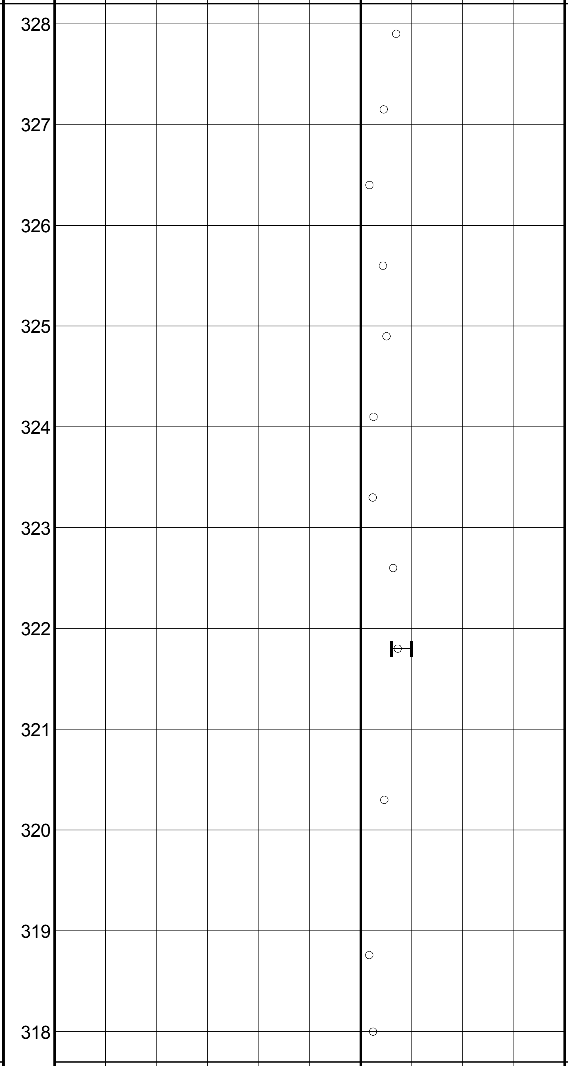
SOIL PROFILE		SAMPLES			REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
328.2	GROUND SURFACE							
328.0 / 0.0	Topsoil							
0.2	SANDY SILT, trace clay, trace gravel, organics Loose, Brown, Moist (FILL)	1	SS	8				
327.4 / 0.8	SILTY SAND/SANDY SILT, trace gravel to gravelly Compact to very dense, Brown, Moist (TILL)	2	SS	19				
		3	SS	17				
		4	SS	21	13	43	38	6
		5	SS	21				
		6	SS	24				
		7	SS	30				
		8	SS	56				
	Silt, some clay, Very stiff	9	SS	17	1	10	69	20
		10	SS	33				
		11	SS	72	31	47	17	5
		12	SS	48				
317.7 / 10.5	End of borehole							

GROUND WATER CONDITIONS

UNIT WEIGHT γ kN/m³

DYNAMIC CONE PENETRATION RESISTANCE PLOT
20 40 60 80 100
SHEAR STRENGTH kPa
○ UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × LAB VANE
20 40 60 80 100

PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L
WATER CONTENT (%)
20 40 60



NOTES:
1. Groundwater level was not encountered in the borehole during or upon completion of drilling.
2. No cave-in was noted in the borehole upon extraction of augers.

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

RECORD OF BOREHOLE No 35-618-08

1 OF 3 **METRIC**

G.W.P. 3059-20-00
LOCATION COORDS: 4 814 138.6 N; 249 508.2 E (MTM ON10)
ORIGINATED BY F.M./V.L.
DIST 31 HWY 6
BOREHOLE TYPE Solid Stem Augers, Wash Boring, HQ Rock Coring
COMPILED BY L.Y.
DATUM Geodetic
DATE 2021.08.03 - 2021.08.10 LATITUDE 43.465118 LONGITUDE -80.183330
CHECKED BY G.U.

SOIL PROFILE		SAMPLES			ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES		GR	SA	SI	CL
328.5	GROUND SURFACE								
328.3 0.0	Topsoil								
0.2	SANDY SILT Loose, Brown, Moist (FILL)	1	SS	7	328				
327.7 0.8	SILTY SAND/SANDY SILT Compact, Brown, Moist (TILL)	2	SS	14					
		3	SS	18	327				
		4	SS	13	326	28	37	30	5
		5	SS	18	325				
		6	SS	43	324				
		7	SS	19		20	40	31	9
	very dense	8	SS	55	323				
		9	SS	100/10cm	322				
					321				
		10	SS	78	320	20	48	26	6
		11	SS	100/20cm	319				
	Sandy gravel, Dense	12	SS	41	318	49	36	(15)	
	Sand, Dense	13	SS	49	317	10	80	(10)	
		14	SS	71	316				
		15	SS	50/13cm	315				
					314				

GROUND WATER CONDITIONS

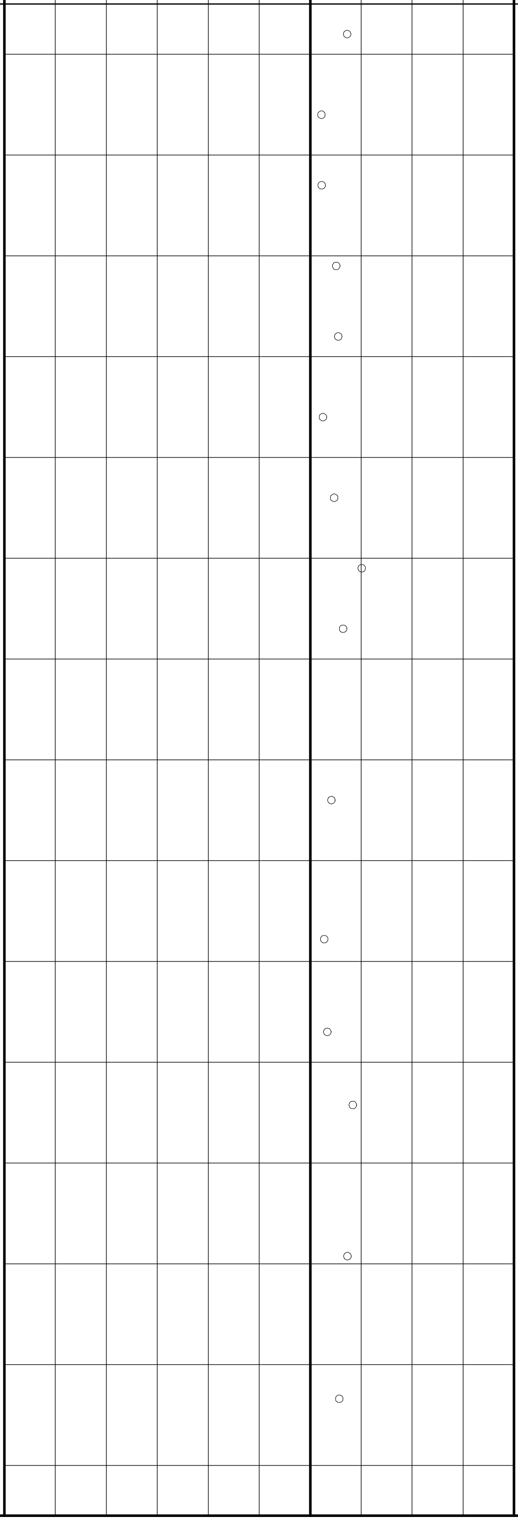
DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH kPa
○ UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × LAB VANE
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



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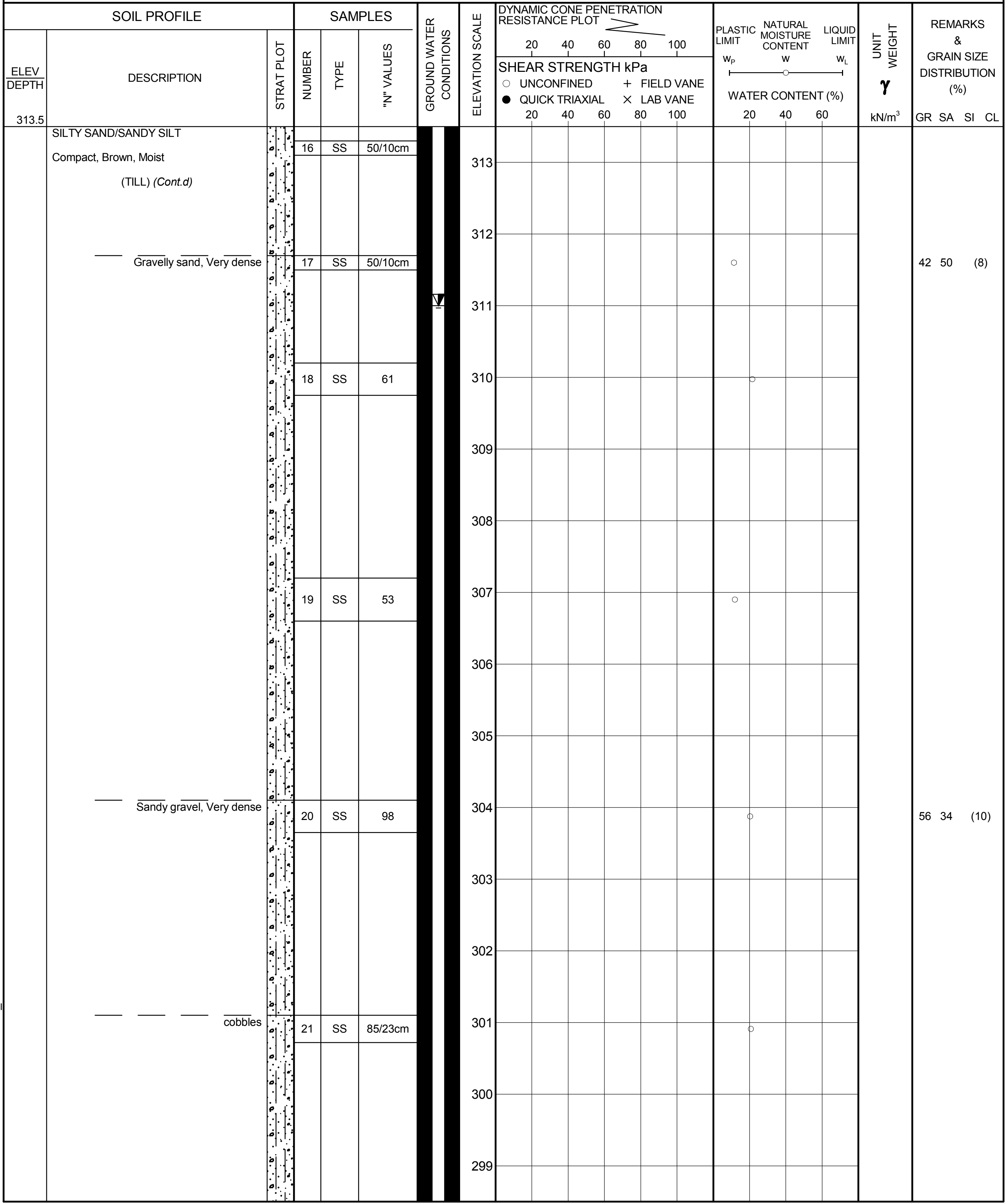
$+^3, \times^3$: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No 35-618-08

2 OF 3 **METRIC**

G.W.P. 3059-20-00
LOCATION COORDS: 4 814 138.6 N; 249 508.2 E (MTM ON10)
ORIGINATED BY F.M./V.L.
DIST 31 HWY 6
BOREHOLE TYPE Solid Stem Augers, Wash Boring, HQ Rock Coring
COMPILED BY L.Y.
DATUM Geodetic
DATE 2021.08.03 - 2021.08.10 LATITUDE 43.465118 LONGITUDE -80.183330
CHECKED BY G.U.



ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
313.5	SILTY SAND/SANDY SILT Compact, Brown, Moist (TILL) *(Cont.d)*	16	SS	50/10cm				
	Gravelly sand, Very dense	17	SS	50/10cm	42	50	(8)	
		18	SS	61				
		19	SS	53				
	Sandy gravel, Very dense	20	SS	98	56	34	(10)	
	cobbles	21	SS	85/23cm				

Continued Next Page

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

RECORD OF BOREHOLE No 35-618-08

3 OF 3 **METRIC**

G.W.P. 3059-20-00
LOCATION COORDS: 4 814 138.6 N; 249 508.2 E (MTM ON10)
ORIGINATED BY F.M./V.L.
DIST 31 HWY 6
BOREHOLE TYPE Solid Stem Augers, Wash Boring, HQ Rock Coring
COMPILED BY L.Y.
DATUM Geodetic
DATE 2021.08.03 - 2021.08.10 LATITUDE 43.465118 LONGITUDE -80.183330
CHECKED BY G.U.

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE: 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	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS
298.5	SILTY SAND/SANDY SILT Compact, Brown, Moist (TILL) *(Cont.d)*	22	SS	50/8cm	298	
297.1 31.4	DOLOSTONE/DOLOMITE	23	RC HQ	REC 100%	297 296	RQD 72% UCS=154.3 MPa
		24	RC HQ	REC 100%	295	RQD 94%
294.0 34.5	End of borehole				294	

Groundwater measured in monitoring well

NOTES:
1. Borehole charged with drilling water, thus groundwater level could not be established upon completion of drilling.
2. No cave-in was noted in the borehole upon extraction of augers.
3. The presence of cobbles is inferred by auger grinding observed during drilling and is not indicative of quantity.

Monitoring Well Readings:

Date	Depth (m)	Elev.
Aug. 11/'21	16.1	312.4
Aug. 16/'21	17.4	311.1
Aug. 19/'21	16.0	312.5

Monitoring Well Legend:
- Monument Casing
- Bentonite Seal
- Filter Sand
- Screen

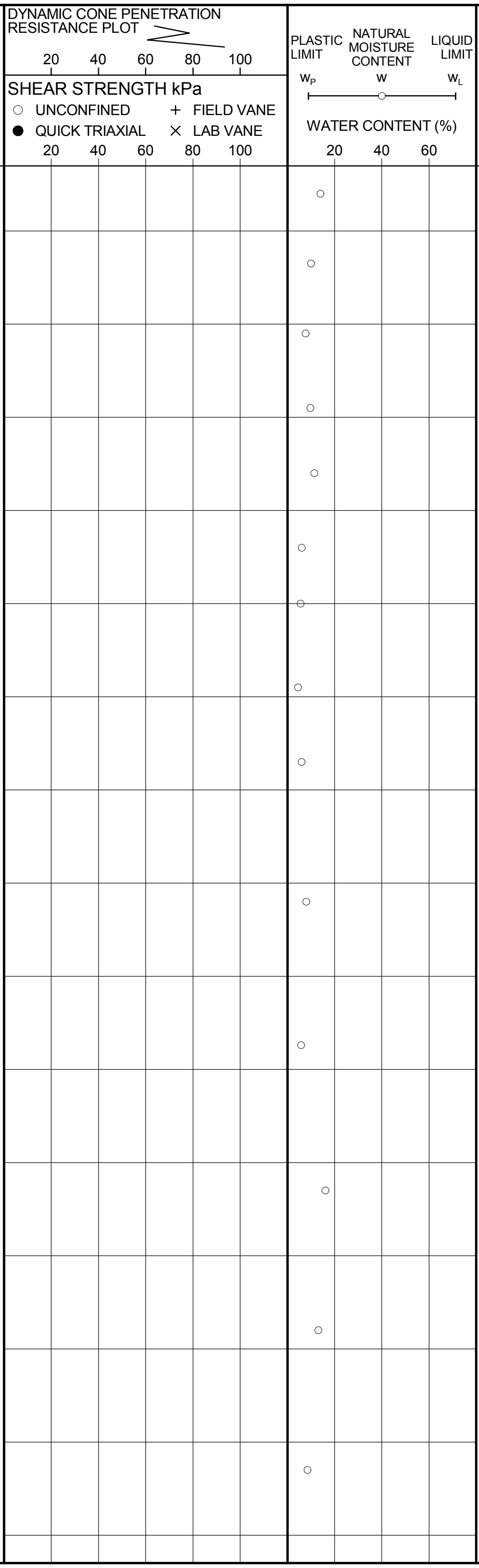
$+^3, \times^3$: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No 35-618-09

1 OF 3 **METRIC**

G.W.P. 3059-20-00
LOCATION COORDS: 4 814 155.2 N; 249 503.6 E (MTM ON10)
ORIGINATED BY F.M./P.J.
DIST 31 HWY 6
BOREHOLE TYPE Solid Stem Augers, Wash Boring
COMPILED BY L.Y.
DATUM Geodetic
DATE 2021.08.12 - 2021.08.17 LATITUDE 43.465267 LONGITUDE -80.183389
CHECKED BY G.U.

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES				GR	SA	SI	CL
329.7	GROUND SURFACE										
329.5 / 0.0	Topsoil										
0.2	SANDY SILT, trace clay, trace gravel, organics Loose, Brown, Moist (FILL)	1	SS	10							
						329					
328.9 / 0.8	SILTY SAND/SANDY SILT, trace gravel Compact to very dense, Brown, Moist (TILL)	2	SS	20				13	39	40	8
						328					
		3	SS	25							
		4	SS	20							
						327					
		5	SS	28							
						326					
		6	SS	17				1	38	56	5
	gravelly	7	SS	100/3mm		325					
		8	SS	56							
						324					
		9	SS	44							
						323					
						322					
		10	SS	30							
						321					
		11	SS	40							
						320					
						319					
		12	SS	35				8	45	43	4
						318					
		13	SS	40							
						317					
						316					
		14	SS	54				40	37	18	5
						315					



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+³, ×³: Numbers refer to Sensitivity

○³% STRAIN AT FAILURE

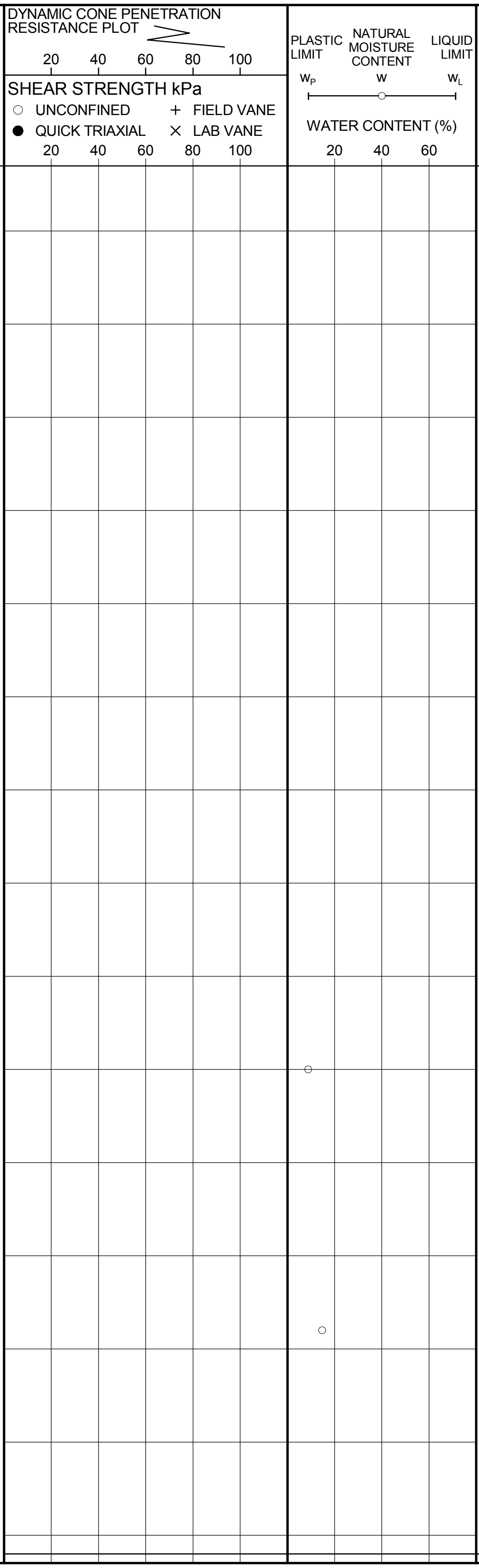
RECORD OF BOREHOLE No 35-618-09

2 OF 3

METRIC

G.W.P. 3059-20-00
LOCATION COORDS: 4 814 155.2 N; 249 503.6 E (MTM ON10)
ORIGINATED BY F.M./P.J.
DIST 31 HWY 6
BOREHOLE TYPE Solid Stem Augers, Wash Boring
COMPILED BY L.Y.
DATUM Geodetic
DATE 2021.08.12 - 2021.08.17 LATITUDE 43.465267 LONGITUDE -80.183389
CHECKED BY G.U.

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
314.7	SILTY SAND/SANDY SILT, trace gravel Compact to very dense, Brown, Moist (TILL) *(Cont.d)*	15	SS	50/5cm	314	
		16	SS	27/3cm	313	
					312	
	cobbles	17	SS	50/8cm	311	
					310	
					309	
		18	SS	42	308	
					307	
					306	
		19	SS	37	305	48 41 (11)
					304	
					303	
	cobbles	20	SS	50/15cm	302	
					301	
299.8		21	SS	50/3cm	300	



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

RECORD OF BOREHOLE No 35-618-09

3 OF 3 **METRIC**

G.W.P. 3059-20-00
LOCATION COORDS: 4 814 155.2 N; 249 503.6 E (MTM ON10)
ORIGINATED BY F.M./P.J.
DIST 31 HWY 6
BOREHOLE TYPE Solid Stem Augers, Wash Boring
COMPILED BY L.Y.
DATUM Geodetic
DATE 2021.08.12 - 2021.08.17 LATITUDE 43.465267 LONGITUDE -80.183389
CHECKED BY G.U.

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 × LAB VANE 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	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
299.7 29.9	Borehole terminated due to auger refusal										

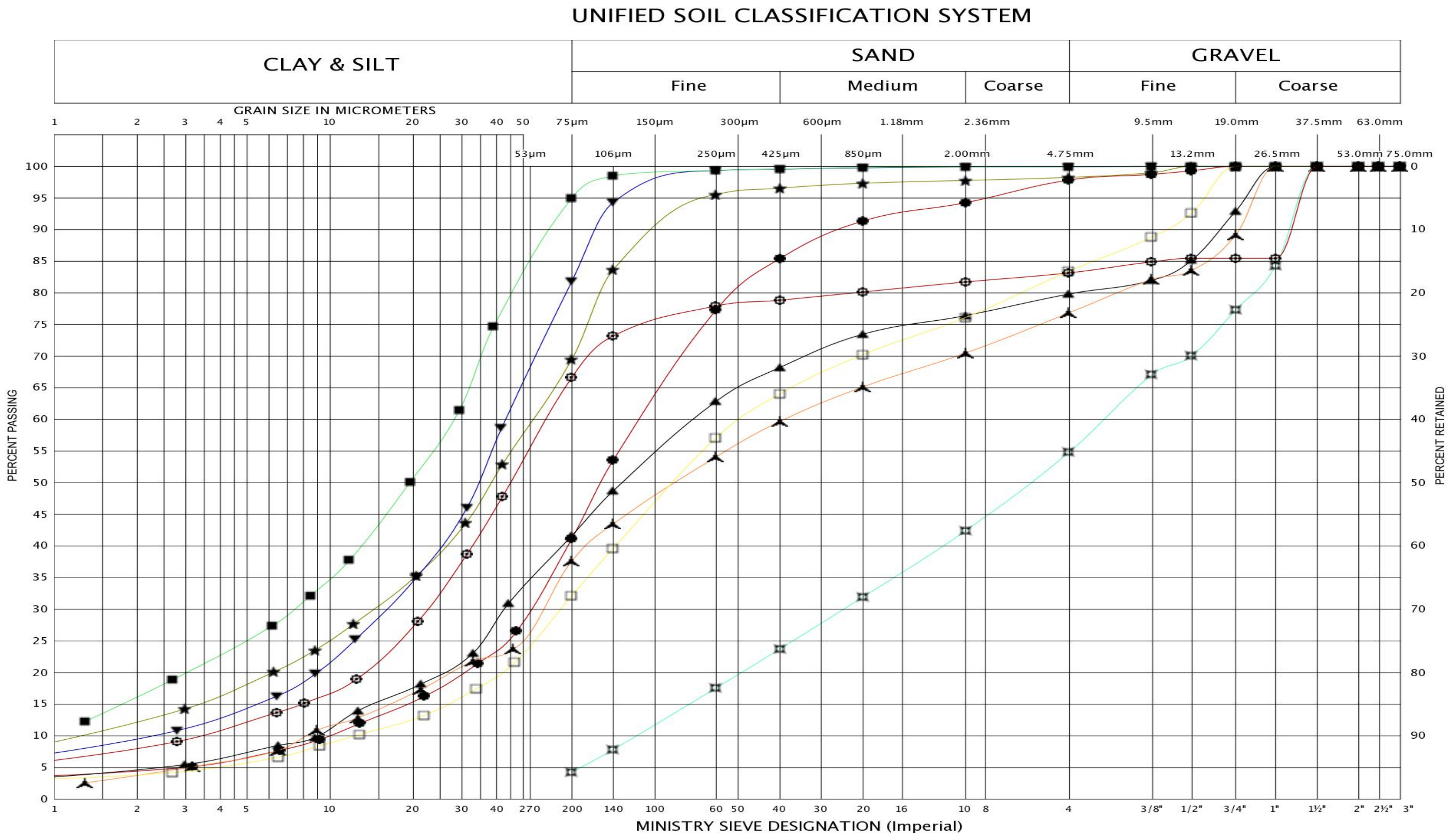
NOTES:
1. Groundwater level was not encountered during or upon completion of drilling.
2. No cave-in was noted in the borehole upon extraction of augers.
3. The presence of cobbles is inferred by auger grinding observed during drilling and is not indicative of quantity.

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

APPENDIX C

Results of Grain Size Analyses

Results of Atterberg Limit Tests



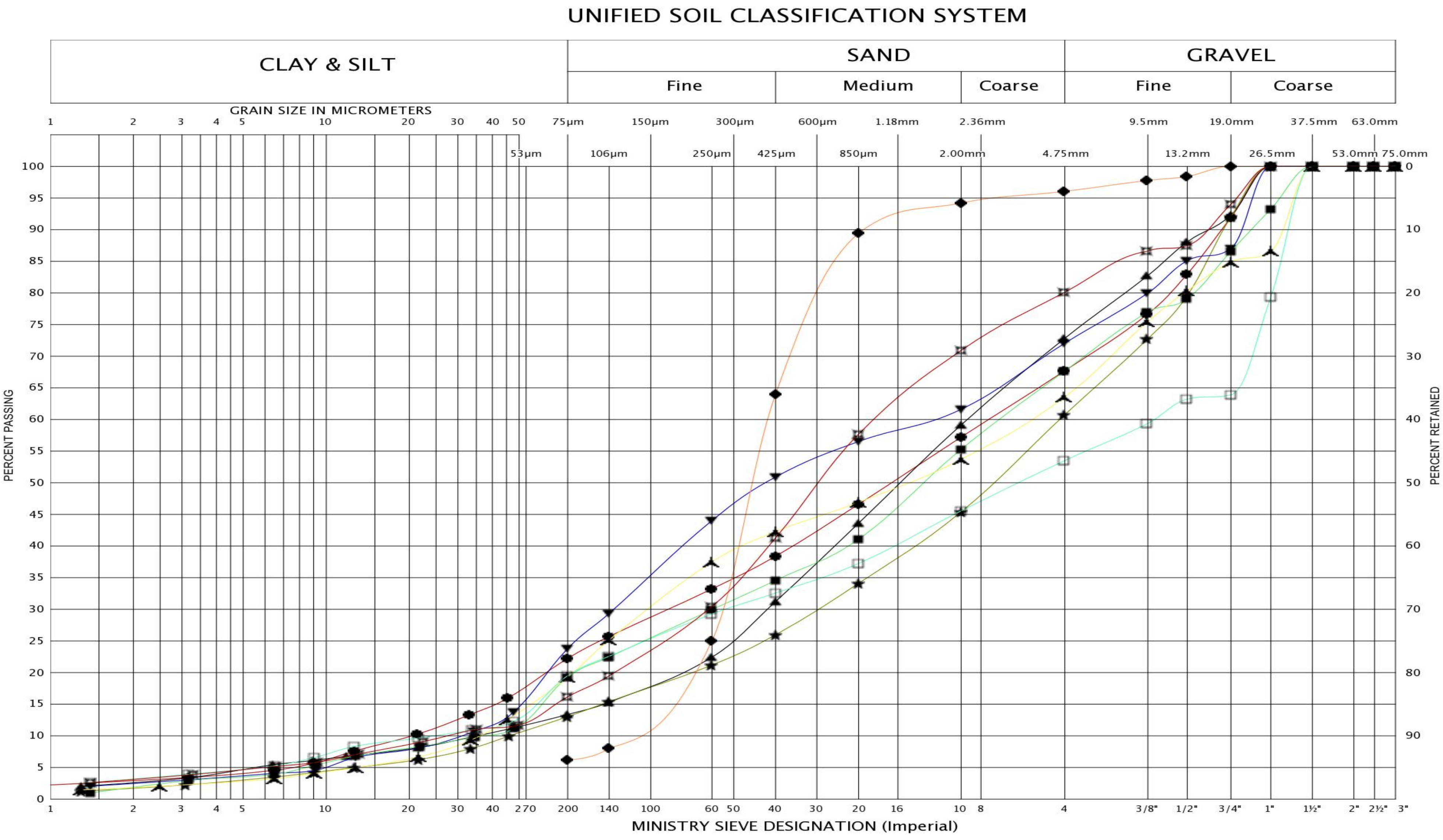
LEGEND	BH	35-618-01	35-618-01	35-618-01	35-618-02	35-618-02	35-618-02	35-618-02	35-618-02	35-618-03
	SAMPLE	3	5	9	3	6	8	10	14	4
	SYMBOL	●	▲	★	▼	■	⅄	□	⌑	⊕

Ontario

GRAIN SIZE DISTRIBUTION

Silty Sand/Sandy Silt, trace to with gravel (Till)

FIG No.:	618-GS-1A
HWY :	6
GWP	3059-20-00



LEGEND	BH	35-618-03	35-618-03	35-618-04	35-618-04	35-618-04	35-618-05	35-618-05	35-618-05	35-618-05
	SAMPLE	7	11	2	7	11	6	8	10	17
	SYMBOL	⬢	▲	▼	■	★	⅄	◆	⊠	□

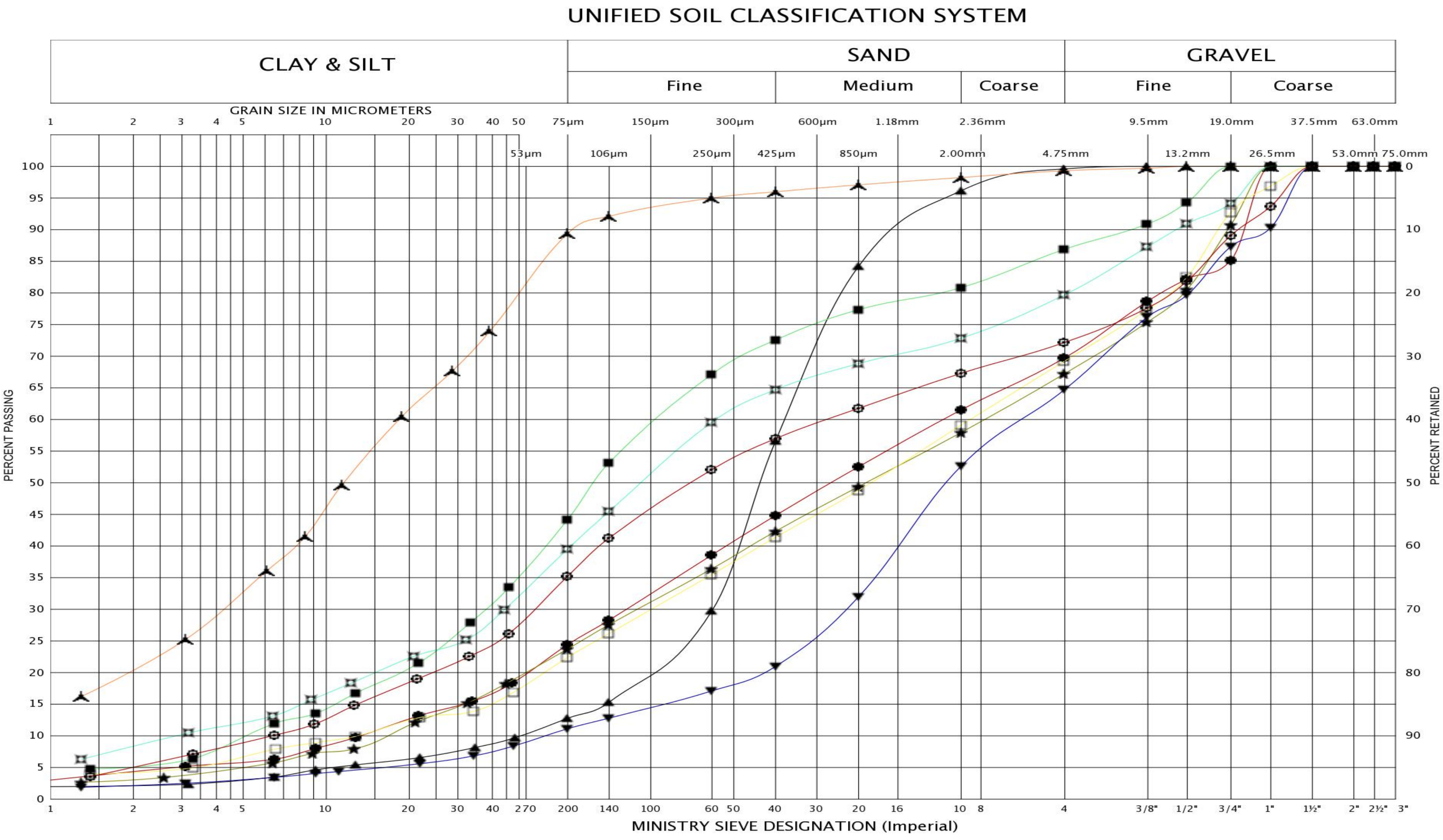
GRAIN SIZE DISTRIBUTION

Silty Sand/Sandy Silt, trace to with gravel (Till)

FIG No.: 618-GS-1B

HWY : 6

GWP 3059-20-00

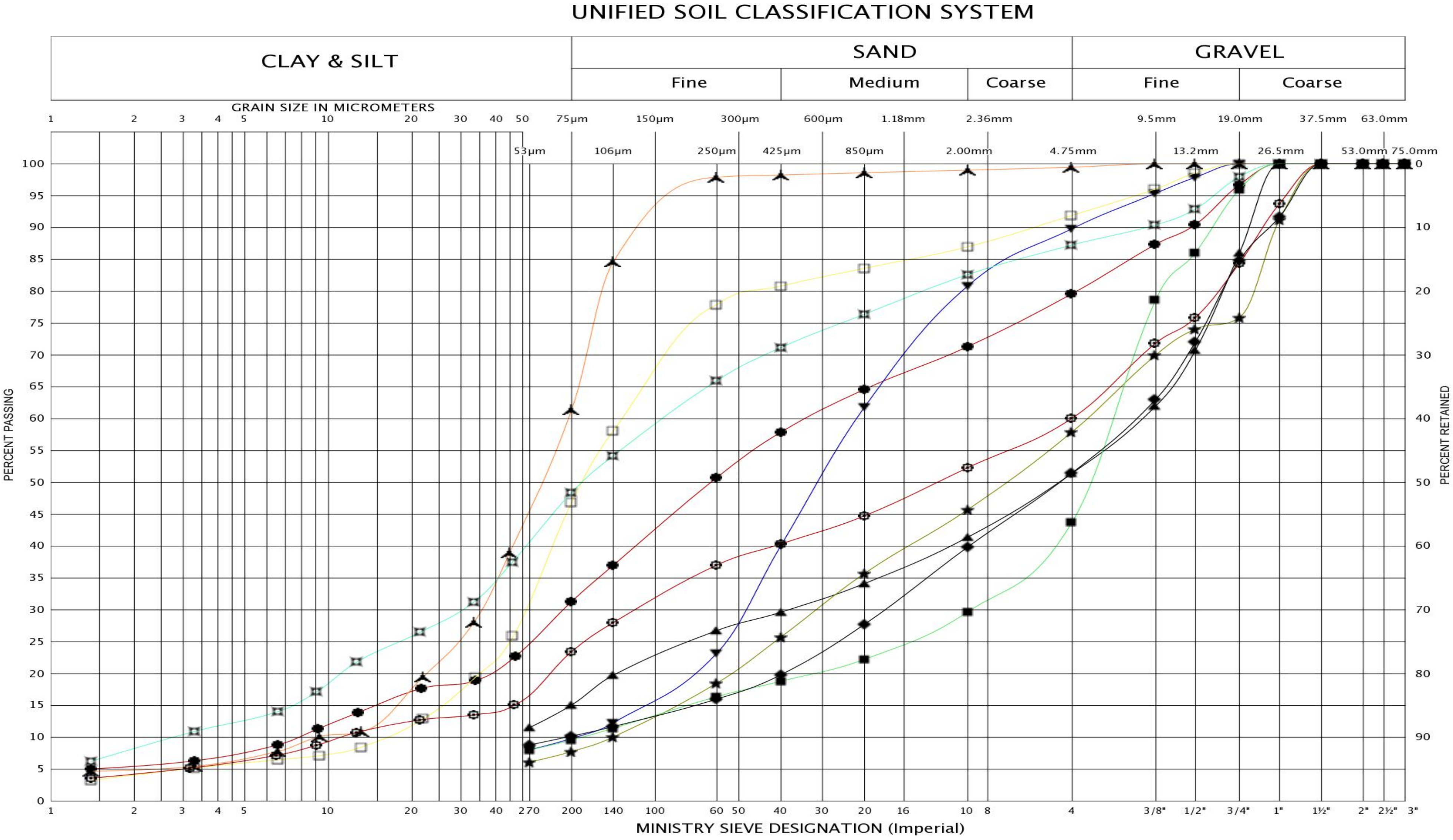


LEGEND	**BH**	35-618-06	35-618-06	35-618-06	35-618-06	35-618-07	35-618-07	35-618-07	35-618-08	35-618-08
	SAMPLE	2	5	8	17	4	9	11	4	7
	SYMBOL	●	★	▲	▼	■	⅄	□	⊕	⌘

GRAIN SIZE DISTRIBUTION

Silty Sand/Sandy Silt, trace to with gravel (Till)

FIG No.: 618-GS-1C
HWY : 6
GWP 3059-20-00



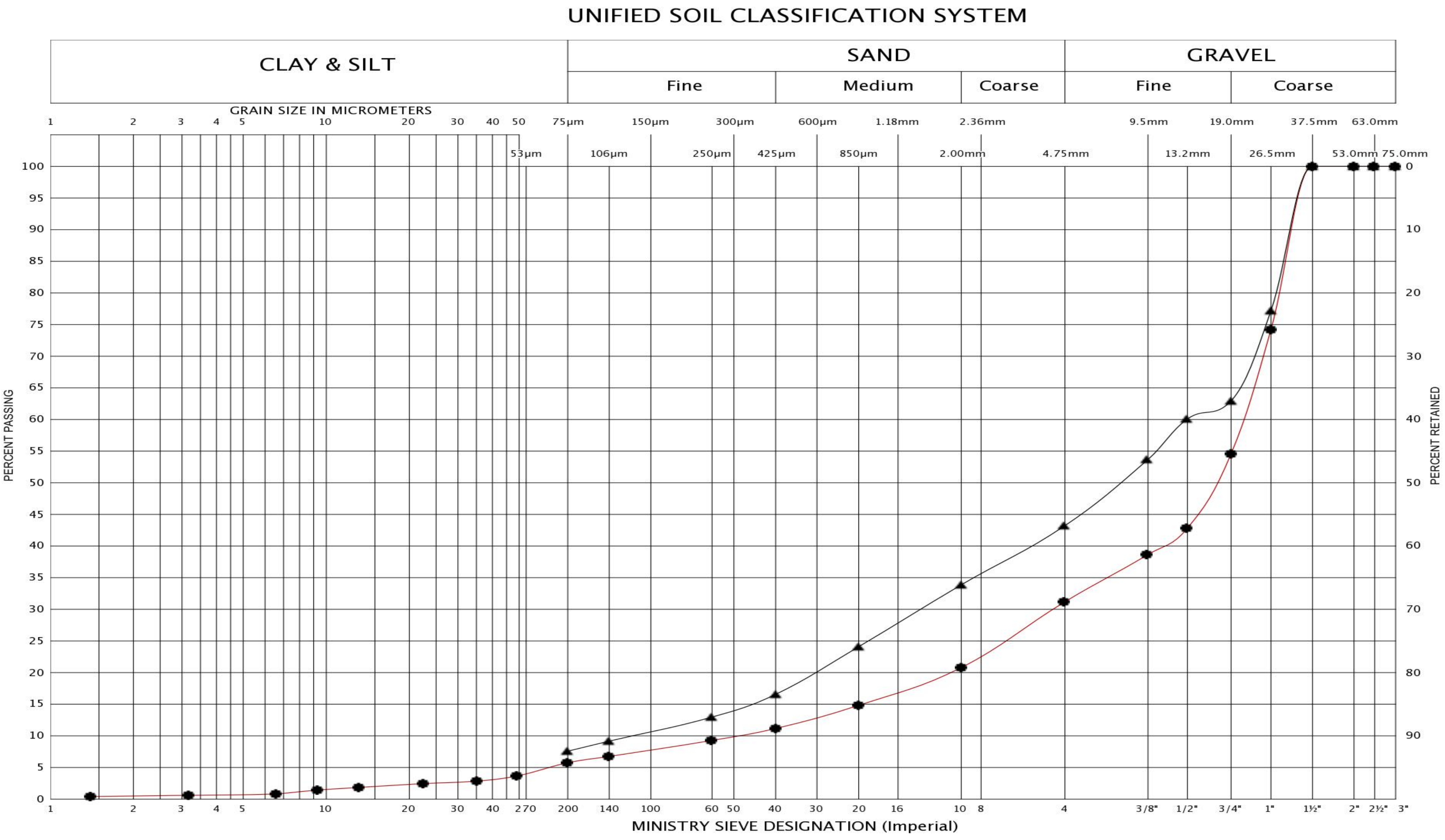
LEGEND	**BH**	35-618-08	35-618-08	35-618-08	35-618-08	35-618-08	35-618-09	35-618-09	35-618-09	35-618-09	35-618-09
	SAMPLE	10	12	13	17	20	2	6	12	14	19
	SYMBOL	●	▲	▼	★	■	⌘	⅄	□	⊕	◆

Ontario

GRAIN SIZE DISTRIBUTION

Silty Sand/Sandy Silt, trace to with gravel (Till)

FIG No.:	618-GS-1D
HWY :	6
GWP	3059-20-00



LEGEND	BH	35-618-02	35-618-05
	SAMPLE	4	3
	SYMBOL	●	▲

Ontario

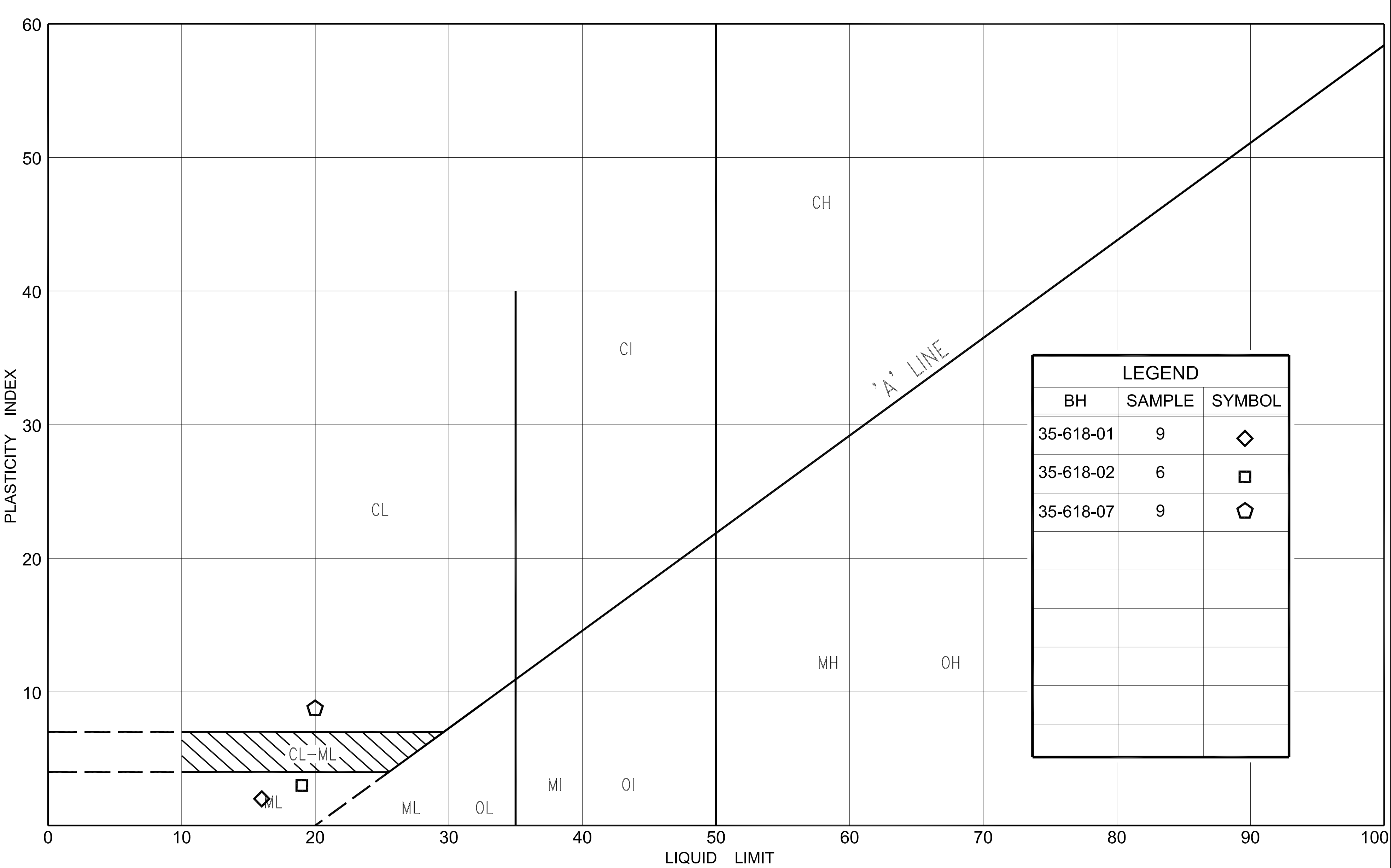
GRAIN SIZE DISTRIBUTION

GRAVEL, with sand (Till)

FIG No.: 618-GS-2

HWY : 6

GWP 3059-20-00



LEGEND

BH	SAMPLE	SYMBOL
35-618-01	9	◇
35-618-02	6	□
35-618-07	9	⬠

Ontario
Ministry of Transportation

PLASTICITY CHART

SILTY SAND to SANDY SILT, trace gravel (TILL)

FIG No.	618-PC-1
HWY:	6
G.W.P. No.	3059-20-00

APPENDIX D

Rock Core Photographs

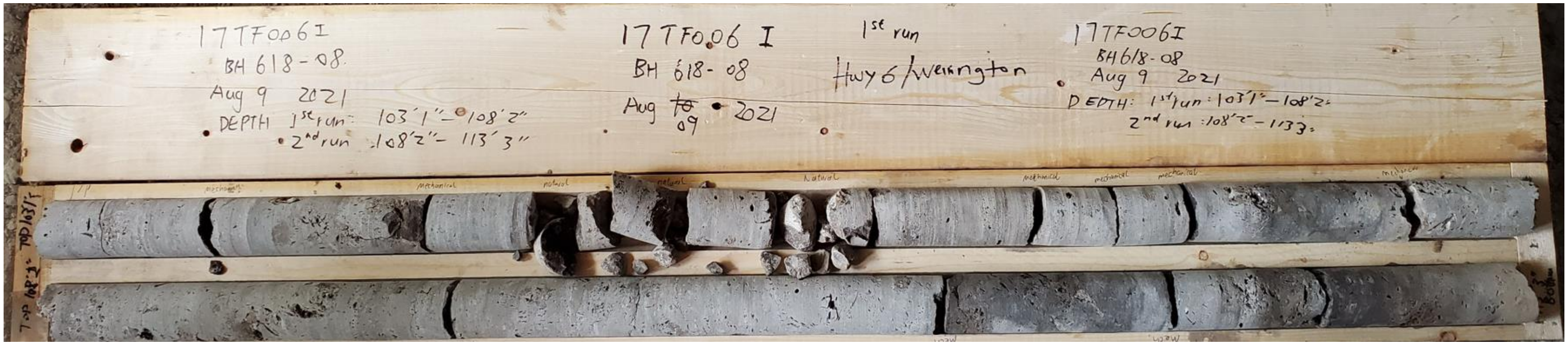
Rock Core Descriptions

Unconfined Compressive Test Results

Photographs of Rock Core Samples Retrieved from Borehole 35-618-02

Photographs of Rock Core Samples Retrieved from Borehole 35-618-05

Photograph of Rock Core Samples Retrieved from Borehole 35-618-08



Rock Core Description

Project: Highway 6/Highway 401 Improvements, From Hamilton North Limits to Guelph South Limits, Ontario

PML Ref.: 17TF006A

Borehole No.	Sample No. (Core Run)	Depth (m/ft)	% Core Recovery	% *RQD	Core Description
35-618-02	22 (1)	31.4-32.06 103’-105’2”	98.5 (0.67m)	74	**DOLOSTONE/DOLOMITE (GUELPH FORMATION)** -: medium grey, fine crystalline dolostone, sucrosic (sugary) texture, thick bedded, hard, massive, slightly weathered, slightly fractured, Minerals: dolomite, calcite grains, containing mud/silt. Common diagenesis features: irregular size and shape of vugs/voids infilled calcite crystals/grains. Common mechanical core breaks. Intact Rock Strength: R4 (Strong)
	23 (2)	32.06-33.55 105’2”-110’1”	100 (1.5m)	45 (0.71m)	**DOLOSTONE/DOLOMITE (GUELPH FORMATION)** -: medium brown to grey, fine crystalline dolostone, sucrosic texture, thick bedded, moderately hard, massive, slightly weathered, slightly fractured. Minerals: dolomite/calcite grains/crystals, containing silt & mud. Common diagenesis features: irregular size and shape of vugs/voids infilled calcite crystals/grains/silt/mud. Diagonal/vertical mechanical core breaks from 33.0m to 33.55m Intact Rock Strength: R3 (Medium Strong)
	24 (3)	33.55-34.8 110’1”-114’3”	96 (1.3m)	95	**DOLOSTONE/DOLOMITE (GUELPH FORMATION)** -: medium brown, fine crystalline dolostone, sucrosic texture, thick bedded, moderately hard, massive, moderately weathered associated with large vugs, unfractured to slightly fractured. Minerals: dolomite/calcite grains. Common diagenesis features: irregular size and shape of vugs/pores infilled calcite/dolomite crystals/grains with silt/mud. Intact Rock Strength: R**3** (Medium Strong)

*RQD – Rock Quality Designation

Compiled & logged by: Shahid Siddiqi, M.Sc. P.Geo
Reviewed by: Carlos Nascimento, P. Eng.

Note: Intact Rock Strength obtained using rock pick test in drill core from table “Rock Characterization Testing & Monitoring”, International Society of Rock Mechanics.

Rock Core Description

Project: Highway 6/Highway 401 Improvements, From Hamilton North Limits to Guelph South Limits, Ontario

PML Ref.: 17TF006A

Borehole No.	Sample No. (Core Run)	Depth (m/ft)	% Core Recovery	% *RQD	Core Description
35-618-05	19 (1)	21.4-22.76 70’3”-74’7”	72 (0.98m)	34 (0.46m)	**DOLOSTONE/DOLOMITE (GUELPH FORMATION)** -: medium brown, fine crystalline dolostone, sucrosic texture, hard, thick bedded, moderately weathered/vuggy, moderately fractured associated with mechanical breaks. Minerals: calcite/dolomite crystals/grains. Common diagenesis features: irregular size and shape of vugs infilled calcite crystals/grains, silt & mud. Mechanical core breaks. Intact Rock Strength: R3 (Medium Strong)
	20 (2)	22.76-24.3 74’7”-79’7”	97 (1.5m)	67 (1.04m)	**DOLOSTONE/DOLOMITE (GUELPH FORMATION)** -: medium brown, fine crystalline dolostone, sucrosic texture, thick bedded, hard, moderately weathered associated with vugs/voids, slightly fractured. Minerals: dolomite, calcite, containing organic material, mud/silt. Common diagenesis features: irregular size and shape of vugs/pores contain calcite crystals/grains/silt/mud & dolomitization. Mechanical core breaks. Intact Rock Strength: R4 (Strong)
	21 (3)	24.3-25.02 79’7”-82’11”	100 (0.73m)	85	**DOLOSTONE/DOLOMITE (GUELPH FORMATION) - :** medium brown, fine crystalline dolostone, sucrosic texture, thick bedded, hard, moderately weathered associated with vugs/voids, slightly fractured. Minerals: dolomite, calcite, containing organic material, mud/silt. Common diagenesis features: irregular size and shape of vugs/pores contain calcite crystals/grains/silt/mud. Mechanical core breaks. Intact Rock Strength: R4 (Strong)

*RQD – Rock Quality Designation

Compiled & logged by: Shahid Siddiqi, M.Sc. P.Geo
Reviewed by: Carlos Nascimento, P. Eng.

Notes: Intact Rock Strength obtained using rock pick test in drill core from table “Rock Characterization Testing & Monitoring”, International Society of Rock Mechanics.

Rock Core Description

Project: Highway 6/Highway 401 Improvements, From Hamilton North Limits to Guelph South Limits, Ontario

PML Ref.: 17TF006A

Borehole No.	Sample No. (Core Run)	Depth (m/ft)	% Core Recovery	% *RQD	Core Description
35-618-06	19 (1)	22.2-22.8 72'8"-75'0"	95 (0.6m)	43 (0.26m)	**DOLOSTONE/DOLOMITE (GUELPH FORMATION)** - : medium brown, fine crystalline dolostone, sucrosic texture, thick bedded, very hard, moderately weathered associated with vugs/voids, slightly fractured. Minerals: dolomite, calcite, occasional pyrite containing mud/silt. Common diagenesis features: irregular size and shape of vugs/pores contain calcite crystals/grains/silt/mud. Mechanical core breaks. Intact Rock Strength: R5 (Very Strong)
	20 (2)	22.8-24.3 75'-79'10"	90 (1.35m)	47 (0.7m)	**DOLOSTONE/DOLOMITE (GUELPH FORMATION)** - : medium brown, fine crystalline dolostone, sucrosic texture, thick bedded, hard, moderately weathered, slightly fractured. Minerals: dolomite, calcite, occasional pyrite containing mud/silt. Common diagenesis features: irregular size and shape of vugs/pores contain calcite crystals/grains/silt/mud. Mechanical core breaks. Intact Rock Strength: R4 (Strong)
	21 (3)	24.25-25.9 79'10"-85'1"	72 (1.2m)	59 (0.95m)	**DOLOSTONE/DOLOMITE (GUELPH FORMATION)** - : medium brown, fine crystalline dolostone, sucrosic texture, thick bedded, very hard, moderately weathered associated with vugs, slightly fractued. Minerals: dolomite, calcite, occasional pyrite containing mud/silt. Common diagenesis features: irregular size and shape of vugs/pores contain calcite crystals/grains/silt/mud. Mechanical core breaks. Intact Rock Strength: R5 (Very Strong)

*RQD – Rock Quality Designation

Compiled & logged by: Shahid Siddiqi, M.Sc. P.Geo
Reviewed by: Carlos Nascimento, P. Eng.

Notes: Intact Rock Strength obtained using rock pick test in drill core from table "Rock Characterization Testing & Monitoring", International Society of Rock Mechanics.

Rock Core Description

Project: Highway 6/Highway 401 Improvements, From Hamilton North Limits to Guelph South Limits, Ontario

PML Ref.: 17TF006A

Borehole No.	Sample No. (Core Run)	Depth (m/ft)	% Core Recovery	% *RQD	Core Description
35-618-08	23 (1)	31.4-33.0 103’1”-108’2”	100 (1.6 m)	72 (1.15 m)	**DOLOSTONE/DOLOMITE (GUELPH FORMATION)** - : medium brown, fine crystalline dolostone, sucrosic texture, thick bedded, very hard, moderately weathered associated with vugs/voids, slightly fractured. Minerals: dolomite, calcite, occasional pyrite containing mud/silt. Common diagenesis features: irregular size and shape of vugs/pores contain calcite crystals/grains/silt/mud. Mechanical core breaks. **Intact Rock Strength: R5 (Very Strong)**
	24 (2)	33.0-34.5 108’2”-113’3”	100 (1.5m)	94 (1.41m)	**DOLOSTONE/DOLOMITE (GUELPH FORMATION)** - : medium brown, fine crystalline dolostone, sucrosic texture, thick bedded, very hard, moderately weathered associated with vugs/voids, slightly fractured. Minerals: dolomite, calcite, occasional pyrite containing mud/silt. Common diagenesis features: irregular size and shape of vugs/pores contain calcite crystals/grains/silt/mud. Mechanical core breaks. **Intact Rock Strength: R5 (Very Strong)**

*RQD – Rock Quality Designation

Compiled & logged by: Frank Meng, M.Eng. EIT
Reviewed by: Lul Yimam, PhD., P. Eng.

Notes: Intact Rock Strength obtained using rock pick test in drill core from table “Rock Characterization Testing & Monitoring”, International Society of Rock Mechanics.

ROCK CORE DIMENSIONS

ASTM D4543

CLIENT	AECOM
PROJECT	HYW 401/6
SAMPLE IDENTIFICATION	BH 35-618-5, RN-2, 77' 1"-77' 8"

PML REF	17TF006A
LAB NO.	1801322-B
DATE SAMPLED	
DATE TESTED	2018-06-05
TESTED BY	YA/BM

DEVIATION FROM STRAIGHTNESS

DIAL READING (IN)	**TRIAL**		
	1	**2**	**3**
MINIMUM	0.0740	0.0760	0.0710
MAXIMUM	0.0860	0.0860	0.0890
DIFFERENCE	0.0120	0.0100	0.0180
MAX DIFF.	0.018	**SPEC.**	0.020 max.

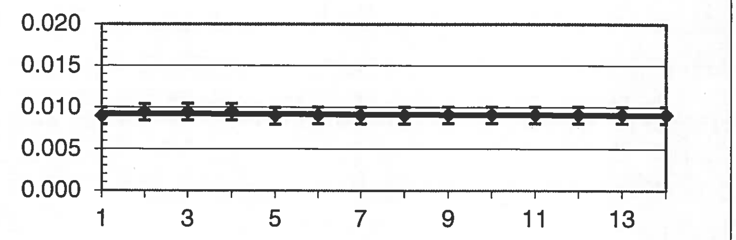
FLATNESS TOLERANCE

DIAL READING (IN)	**END 1**		**END 2**	
	SET 1	**SET 2**	**SET 1**	**SET 2**
RDG 1	0.0090	0.0071	0.0082	0.0107
RDG 2	0.0094	0.0071	0.0106	0.0110
RDG 3	0.0095	0.0076	0.0111	0.0110
RDG 4	0.0095	0.0081	0.0114	0.0108
RDG 5	0.0090	0.0083	0.0114	0.0106
RDG 6	0.0090	0.0084	0.0114	0.0102
RDG 7	0.0090	0.0088	0.0114	0.0101
RDG 8	0.0090	0.0093	0.0114	0.0101
RDG 9	0.0091	0.0100	0.0115	0.0100
RDG 10	0.0091	0.0103	0.0117	0.0098
RDG 11	0.0091	0.0106	0.0118	0.0093
RDG 12	0.0091	0.0110	0.0119	0.0093
RDG 13	0.0091	0.0114	0.0119	0.0092
RDG 14	0.0091	0.0117		0.0085
RDG 15				
RDG 16				
RDG 17				
RDG 18				
RDG 19				
RDG 20				

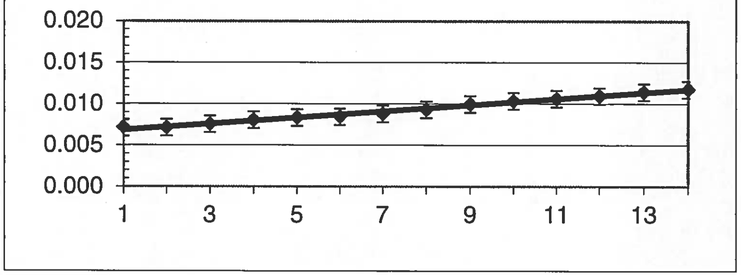
FLATNESS TOLERANCE= .001 in.

CORE DIAMETER (in.)	1.7760	1.7750	1.7810
AVE:	1.7773	**PERPENDICULARITY RATIO (Specified .0043 max.)**	
SLOPE OF BEST FIT LINE			
	MINIMUM	**MAXIMUM**	
END 1A	0.0090	0.0092	0.0001
END 2B	0.0068	0.0117	0.0028
END 2A	0.0101	0.0123	0.0012
END 2B	0.0089	0.0112	0.0013

END 1a



END 1b



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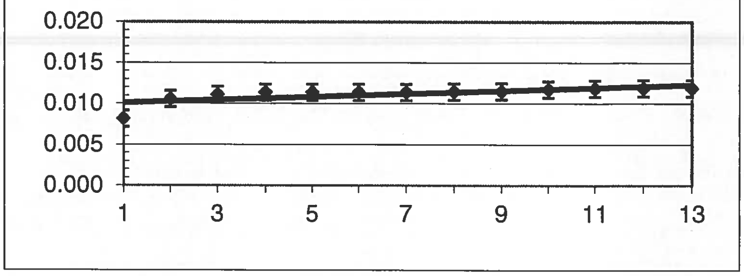

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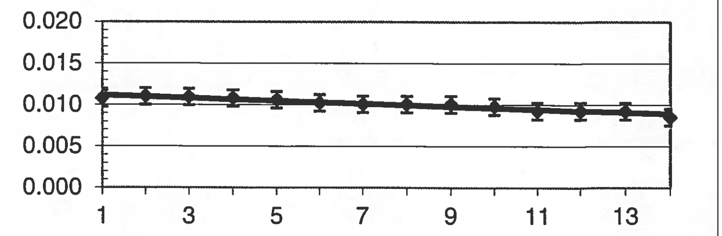


REVIEWED BY J.Noor

DATE 2018-06-08

Peto MacCallum Ltd.
CONSULTING ENGINEERS

ROCK CORE TESTING

ASTM D7012

CLIENT	AECOM	PML REF	17TF006A
PROJECT	HYW 401/6	LAB NO.	1801322-B
SAMPLE IDENTIFICATION	BH 35-618-5, RN-2, 77' 1"-77' 8"	DATE SAMPLED	
		DATE TESTED	2018-06-06
		TESTED BY	YA/BM

UNCONFINED COMPRESSIVE STRENGTH

CORE DIMENSIONS		COMPRESSIVE STRENGTH	
SPECIMEN DIAMETER (in.)	1.7773	TEST TIME (min) (spec. 2 to 15)	5:01
SPECIMEN LENGTH (in.)	4.220	MAXIMUM LOAD APPLIED (kN)	93.57
	4.221		
	4.225	COMPRESSIVE STRENGTH (MPa)	58.5
AVE.	4.222	TYPE OF FAILURE	3
SURFACE AREA (sq mm)	1601	LENGTH TO DIAMETER RATIO (spec 2-2.5)	2.38

MOISTURE CONTENT

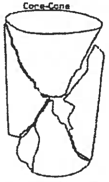
WEIGHT OF WET SAMPLE + TARE (g)	529.30
WEIGHT OF DRY SAMPLE + TARE (g)	523.20
WEIGHT OF WATER (g)	6.10
WEIGHT OF TARE (g)	116.10
WEIGHT OF DRY SAMPLE (g)	407.10
MOISTURE CONTENT (%)	1.5

UNIT WEIGHT

WEIGHT OF DRY SAMPLE IN AIR (g)	4459.00
VOLUME OF SAMPLE (cu m)	0.000172
UNIT WEIGHT (kg/cu m)	25978

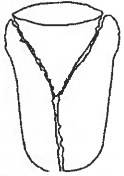
REMARKS

Type 1
Cone-Cone

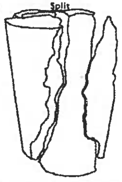


Type 2
Cone-Shear

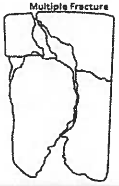
Type 3
Cone-Split



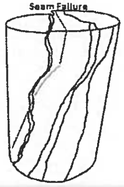
Type 4
Split



Type 5
Multiple Fracture



Type 6
Seam Failure



REVIEWED BY	J.Noor	*DATE*	2018-06-08

Peto MacCallum Ltd.

CONSULTING ENGINEERS

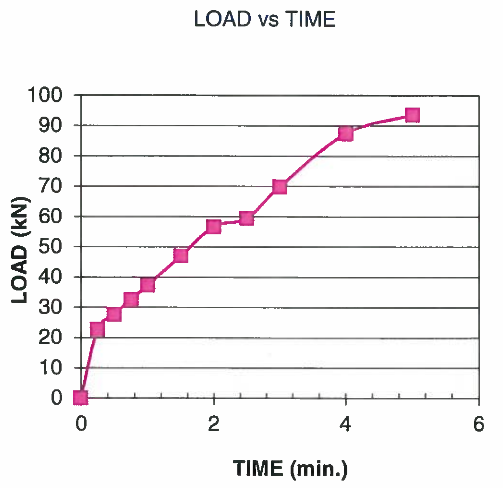
ROCK CORE TESTING

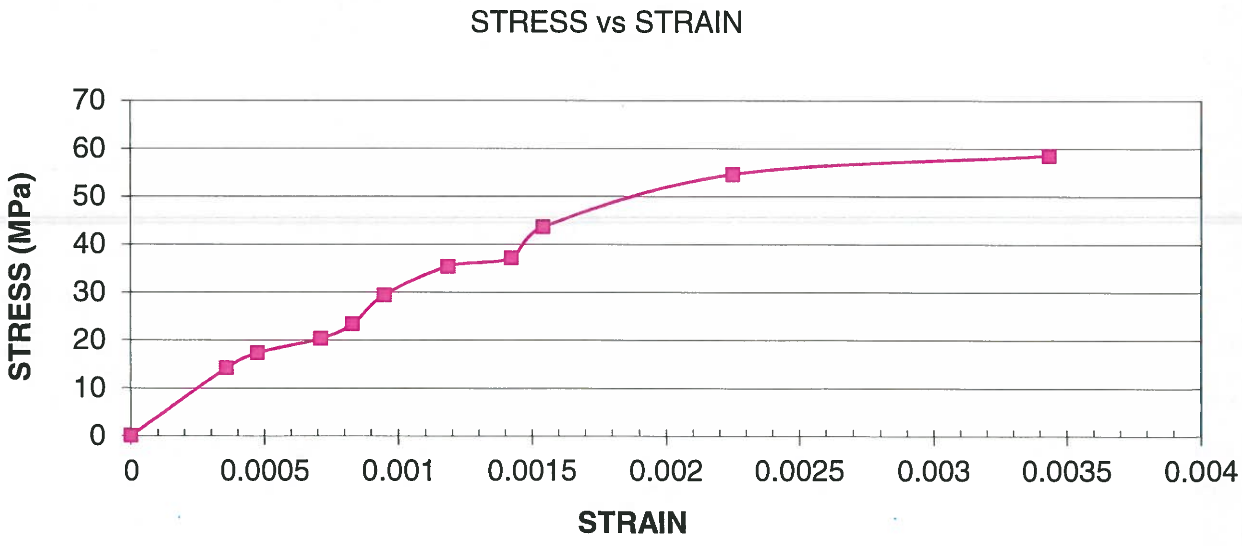
ASTM D7012

CLIENT	AECOM	*PML REF*	17TF006A
PROJECT	HYW 401/6	*LAB NO.*	1801322-B
SAMPLE IDENTIFICATION	BH 35-618-5, RN-2, 77' 1"-77' 8"	*DATE SAMPLED*	
		DATE TESTED	2018-06-06
		TESTED BY	YA/BM

UNCONFINED COMPRESSIVE STRENGTH CURVE

TIME (min.)	DEFLECTION (in)	LOAD (kN)	STRESS (MPa)	STRAIN (mm/mm)
0	0.036	0	0	0
0.25	0.0375	22.61	14.1	0.0004
0.5	0.038	27.66	17.3	0.0005
0.75	0.039	32.50	20.3	0.0007
1	0.0395	37.29	23.3	0.0008
1.5	0.04	46.94	29.3	0.0009
2	0.041	56.55	35.3	0.0012
2.5	0.042	59.34	37.1	0.0014
3	0.0425	69.74	43.6	0.0015
4	0.0455	87.34	54.6	0.0023
5	0.0505	93.57	58.5	0.0034





REVIEWED BY J.Noor *DATE* 2018-06-08

Peto MacCallum Ltd.
CONSULTING ENGINEERS

UNIAXIAL COMPRESSIVE STRENGTH OF ROCK CORE

ASTM D7012

CLIENT	AECOM Canada Ltd.	PML REF	17TF006I
PROJECT	Hwy 6 and 401 Improvements, Midblock Interchange Area	LAB NO.	2104843 C
SAMPLE IDENTIFICATION	BH35-618-8, Run1, 105'11"-106'5"	DATE SAMPLED	
		DATE TESTED	2021-08-24
		TESTED BY	L. Gowry

CORE DIMENSIONS	
SPECIMEN DIAMETER (in.)	2.3865
SPECIMEN LENGTH (in.)	5.470
	5.462
	5.465
AVE.	5.465
SURFACE AREA (sq mm)	2886

COMPRESSIVE STRENGTH	
TEST TIME (min) (spec. 2 to 15)	9:36
MAXIMUM LOAD APPLIED (kN)	445.30
COMPRESSIVE STRENGTH (MPa)	154.3
TYPE OF FAILURE	1
LENGTH TO DIAMETER RATIO (spec 2-2.5)	2.29

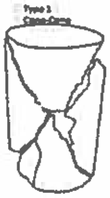
MOISTURE CONTENT

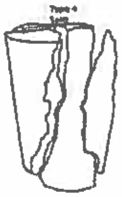
WEIGHT OF WET SAMPLE + TARE (g)	1108.45
WEIGHT OF DRY SAMPLE + TARE (g)	1107.66
WEIGHT OF WATER (g)	0.79
WEIGHT OF TARE (g)	176.27
WEIGHT OF DRY SAMPLE (g)	931.39
MOISTURE CONTENT (%)	0.1

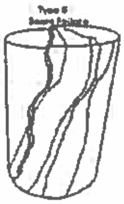
UNIT WEIGHT

WEIGHT OF DRY SAMPLE IN AIR (g)	1061.46
VOLUME OF SAMPLE (cu m)	0.000401
Density (kg/cu m)	2650
UNIT WEIGHT (γ)	25.97

REMARKS







REVIEWED BY J. Noor

DATE 2021-08-25

Peto MacCallum Ltd.
CONSULTING ENGINEERS

ROCK CORE DIMENSIONS

ASTM D4543

CLIENT	AECOM Canada Ltd.	*PML REF*	17TF006I
PROJECT	Hwy 6 and 401 Improvements, Midblock Interchange Area	*LAB NO.*	2104843 C
SAMPLE IDENTIFICATION	BH35-618-8, Run1, 105'11"-106'5"	*DATE SAMPLED*	
		DATE TESTED	2021-08-24
		TESTED BY	L. Gowry

DEVIATION FROM STRAIGHTNESS

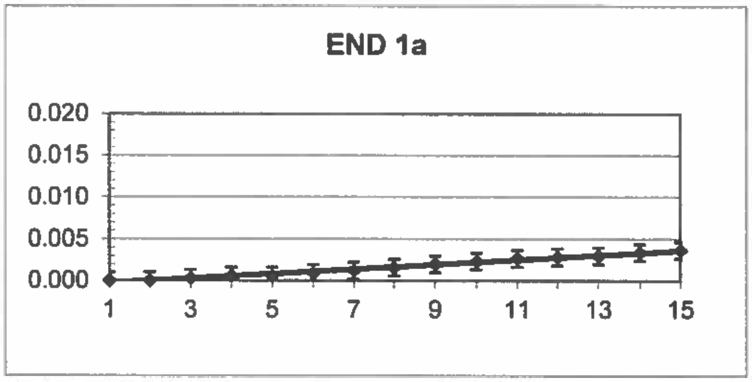
DIAL READING (IN)	TRIAL 1	2	3
MINIMUM	0.0800	0.0830	0.0840
MAXIMUM	0.0900	0.0930	0.0840
DIFFERENCE	0.0100	0.0100	0.0000
MAX DIFF.	0.01	SPEC.	0.020 max.

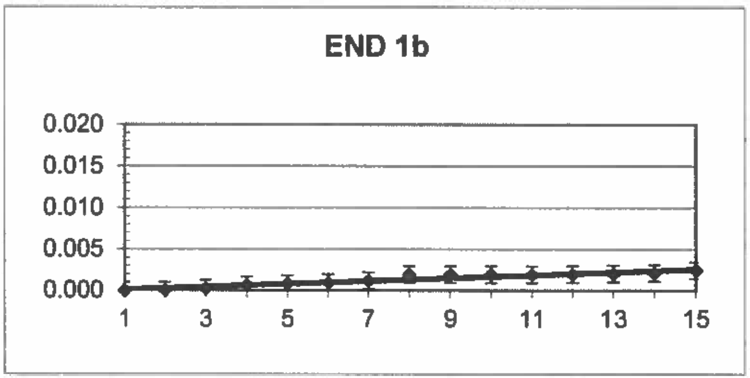
FLATNESS TOLERANCE

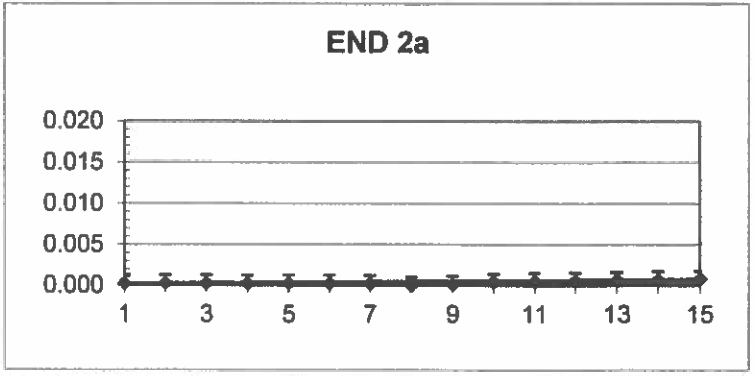
DIAL READING (IN)	END 1 SET 1	SET 2	END 2 SET 1	SET 2
RDG 1	0.0000	0.0000	0.0002	0.0107
RDG 2	0.0001	0.0000	0.0003	0.0104
RDG 3	0.0003	0.0003	0.0002	0.0104
RDG 4	0.0006	0.0007	0.0002	0.0103
RDG 5	0.0006	0.0008	0.0002	0.0104
RDG 6	0.0009	0.0010	0.0002	0.0100
RDG 7	0.0012	0.0012	0.0002	0.0099
RDG 8	0.0016	0.0019	0.0000	0.0106
RDG 9	0.0020	0.0019	0.0001	0.0111
RDG 10	0.0023	0.0019	0.0004	0.0117
RDG 11	0.0027	0.0019	0.0005	0.0122
RDG 12	0.0029	0.0020	0.0006	0.0126
RDG 13	0.0030	0.0021	0.0006	0.0125
RDG 14	0.0034	0.0021	0.0007	0.0128
RDG 15	0.0037	0.0025	0.0008	0.0145
RDG 16				0.0157
RDG 17				0.0162
RDG 18				
RDG 19				
RDG 20				

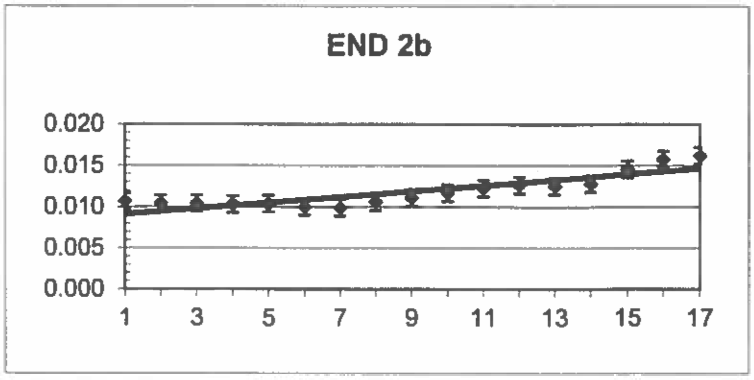
FLATNESS TOLERANCE= .001 in.

CORE DIAMETER (in.)	2.3870	2.3895	2.3830
AVE:	2.3865	PERPENDICULARITY RATIO (Specified .0043 max.)	
SLOPE OF BEST FIT LINE			
	MINIMUM	MAXIMUM	
END 1A	-0.0003	0.0036	0.0016
END 2B	0.0001	0.0026	0.0010
END 2A	0.0001	0.0006	0.0002
END 2B	0.0091	0.0146	0.0023









REVIEWED BY J. Noor

DATE 2021-08-25

APPENDIX E

Soil Chemical Test Results

Certificate of Analysis

AGAT WORK ORDER: 17T297980
PROJECT: 17TF006A

CLIENT NAME: PETO MACCALLUM LIMITED
SAMPLING SITE:Guelph, Puslinch

ATTENTION TO: Lul Yimam
SAMPLED BY:Mousa fall

Corrosivity Package

DATE RECEIVED: 2017-12-22

DATE REPORTED: 2018-01-03

			SAMPLE DESCRIPTION:	**35-617-BH2-SS6**	**35-618-BH4-SS4**	**35-352-BH6-SS6**
			SAMPLE TYPE:	**Soil**	**Soil**	**Soil**
			DATE SAMPLED:	**2017-11-29**	**2017-12-07**	**2017-11-17**
Parameter	**Unit**	**G / S**	**RDL**	**8994390**	**8994393**	**8994394**
Sulfide (S2-)	%		0.05	0.14	<0.05	<0.05
Chloride (2:1)	µg/g		2	527	21	19
Sulphate (2:1)	µg/g		2	119	6	5
pH (2:1)	pH Units		NA	7.71	8.66	8.52
Electrical Conductivity (2:1)	mS/cm		0.005	1.16	0.104	0.115
Resistivity (2:1)	ohm.cm		1	862	9620	8700
Redox Potential (2:1)	mV		5	152	138	160

Comments: RDL - Reported Detection Limit; G / S - Guideline / Standard

8994390-8994394 EC/Resistivity, pH, Chloride, Sulphate and Redox Potential were determined on the extract obtained from the 2:1 leaching procedure (2 parts DI water: 1 part soil).

*Sulphide analyzed at AGAT 5623 McAdam

Certified By: Amanjot Bhela

Quality Assurance

CLIENT NAME: PETO MACCALLUM LIMITED
PROJECT: 17TF006A
SAMPLING SITE:Guelph, Puslinch

AGAT WORK ORDER: 17T297980
ATTENTION TO: Lul Yimam
SAMPLED BY:Mousa fall

Soil Analysis

RPT Date: Jan 03, 2018			DUPLICATE			Method Blank	REFERENCE MATERIAL			METHOD BLANK SPIKE			MATRIX SPIKE		
PARAMETER	Batch	Sample Id	Dup #1	Dup #2	RPD		Measured Value	Acceptable Limits Lower	Acceptable Limits Upper	Recovery	Acceptable Limits Lower	Acceptable Limits Upper	Recovery	Acceptable Limits Lower	Acceptable Limits Upper
Corrosivity Package															
Sulfide (S2-)	8994390	8994390	0.14	0.14	NA	< 0.05	99%	80%	120%						
Chloride (2:1)	8994393	8994393	21	20	4.9%	< 2	95%	80%	120%	96%	80%	120%	98%	70%	130%
Sulphate (2:1)	8994393	8994393	6	5	NA	< 2	101%	80%	120%	102%	80%	120%	106%	70%	130%
pH (2:1)	8994393	8994393	8.66	8.59	0.8%	NA	101%	90%	110%	NA			NA		
Electrical Conductivity (2:1)	8994393	8994393	0.104	0.108	3.8%	< 0.005	98%	90%	110%	NA			NA		
Redox Potential (2:1)	8994393	8994393	138	140	1.4%	< 5	104%	70%	130%	NA			NA		

Comments: NA signifies Not Applicable.
Duplicate Qualifier: As the measured result approaches the RL, the uncertainty associated with the value increases dramatically, thus duplicate acceptance limits apply only where the average of the two duplicates is greater than five times the RL.

Certified By: Amanjot Bhela

5835 COOPERS AVENUE
MISSISSAUGA, ONTARIO
CANADA L4Z 1Y2
TEL (905)712-5100
FAX (905)712-5122
http://www.agatlabs.com

Method Summary

CLIENT NAME: PETO MACCALLUM LIMITED

AGAT WORK ORDER: 17T297980

PROJECT: 17TF006A

ATTENTION TO: Lul Yimam

SAMPLING SITE:Guelph, Puslinch

SAMPLED BY:Mousa fall

PARAMETER	AGAT S.O.P	LITERATURE REFERENCE	ANALYTICAL TECHNIQUE
Soil Analysis			
Sulfide (S2-)	MIN-200-12025	ASTM E1915-09	GRAVIMETRIC
Chloride (2:1)	INOR-93-6004	McKeague 4.12 & SM 4110 B	ION CHROMATOGRAPH
Sulphate (2:1)	INOR-93-6004	McKeague 4.12 & SM 4110 B	ION CHROMATOGRAPH
pH (2:1)	INOR 93-6031	MSA part 3 & SM 4500-H+ B	PH METER
Electrical Conductivity (2:1)	INOR-93-6036	McKeague 4.12, SM 2510 B	EC METER
Resistivity (2:1)	INOR-93-6036	McKeague 4.12, SM 2510 B,SSA #5 Part 3	CALCULATION
Redox Potential (2:1)		McKeague 4.12 & SM 2510 B	REDOX POTENTIAL ELECTRODE

FINAL REPORT

CA14274-AUG21 R1

17TF006A

Prepared for

Peto MacCallum Ltd

FINAL REPORT

First Page

CLIENT DETAILS		LABORATORY DETAILS	
Client	Peto MacCallum Ltd	Project Specialist	Jill Campbell, B.Sc.,GISAS
		Laboratory	SGS Canada Inc.
Address	165 Cartwright Ave Toronto, ON M6A 1V5, Canada	Address	185 Concession St., Lakefield ON, K0L 2H0
Contact	Lul Yimam	Telephone	2165
Telephone	416-525-5786	Facsimile	705-652-6365
Facsimile	416-785-5120	Email	jill.campbell@sgs.com
Email	lyimam@petomaccallum.com	SGS Reference	CA14274-AUG21
Project	17TF006A	Received	08/20/2021
Order Number		Approved	08/26/2021
Samples	Soil (3)	Report Number	CA14274-AUG21 R1
		Date Reported	08/26/2021

COMMENTS

Temperature of Sample upon Receipt: 9 degrees C
Cooling Agent Present:Yes
Custody Seal Present:Yes

Chain of Custody Number:022438

Corrosivity Index is based on the American Water Works Corrosivity Scale according to AWWA C-105. An index greater than 10 indicates the soil matrix may be corrosive to cast iron alloys.

SIGNATORIES

Jill Campbell, B.Sc.,GISAS

Jill Campbell

TABLE OF CONTENTS

PACKAGE: **- Corrosivity Index** (SOIL)

			Sample Number	5	6	7
			Sample Name	BH618-08 5SS 10-12	BH617-09 5SS 10-12	BH617-10 4SS 7.5-9.5
			Sample Matrix	Soil	Soil	Soil
			Sample Date	19/08/2021	19/08/2021	19/08/2021
Parameter	**Units**	**RL**		**Result**	**Result**	**Result**
Corrosivity Index						
Corrosivity Index	none	1		1	14	16
Soil Redox Potential	mV	-		256	249	189
Sulphide (Na2CO3)	%	0.04		< 0.04	< 0.04	0.85
pH	pH Units	0.05		8.45	8.82	7.53
Resistivity (calculated)	ohms.cm	-9999		7580	874	358

PACKAGE: **- General Chemistry** (SOIL)

			Sample Number	5	6	7
			Sample Name	BH618-08 5SS 10-12	BH617-09 5SS 10-12	BH617-10 4SS 7.5-9.5
			Sample Matrix	Soil	Soil	Soil
			Sample Date	19/08/2021	19/08/2021	19/08/2021
Parameter	**Units**	**RL**		**Result**	**Result**	**Result**
General Chemistry						
Conductivity	uS/cm	2		132	1140	2790

PACKAGE: **- Metals and Inorganics** (SOIL)

			Sample Number	5	6	7
			Sample Name	BH618-08 5SS 10-12	BH617-09 5SS 10-12	BH617-10 4SS 7.5-9.5
			Sample Matrix	Soil	Soil	Soil
			Sample Date	19/08/2021	19/08/2021	19/08/2021
Parameter	**Units**	**RL**		**Result**	**Result**	**Result**
Metals and Inorganics						
Moisture Content	%	0.1		10.3	15.4	38.6

PACKAGE: **- Metals and Inorganics** (SOIL)

			Sample Number	5	6	7
			Sample Name	BH618-08 5SS 10-12	BH617-09 5SS 10-12	BH617-10 4SS 7.5-9.5
			Sample Matrix	Soil	Soil	Soil
			Sample Date	19/08/2021	19/08/2021	19/08/2021
Parameter	**Units**	**RL**		**Result**	**Result**	**Result**
Metals and Inorganics (continued)						
Sulphate	µg/g	0.4		9.4	23	570

PACKAGE: **- Other (ORP)** (SOIL)

			Sample Number	5	6	7
			Sample Name	BH618-08 5SS 10-12	BH617-09 5SS 10-12	BH617-10 4SS 7.5-9.5
			Sample Matrix	Soil	Soil	Soil
			Sample Date	19/08/2021	19/08/2021	19/08/2021
Parameter	**Units**	**RL**		**Result**	**Result**	**Result**
Other (ORP)						
Chloride	µg/g	0.4		7.1	490	2300

QC SUMMARY

Anions by IC

Method: EPA300/MA300-Ions1.3 | Internal ref.: ME-CA-[ENV]IC-LAK-AN-001

Parameter	QC batch Reference	Units	RL	Method Blank	Duplicate		LCS/Spike Blank			Matrix Spike / Ref.		
					RPD	AC (%)	Spike Recovery (%)	Recovery Limits (%)		Spike Recovery (%)	Recovery Limits (%)	
								Low	High		Low	High
Chloride	DIO0410-AUG21	µg/g	0.4	<0.4	1	35	100	80	120	102	75	125
Sulphate	DIO0410-AUG21	µg/g	0.4	<0.4	1	35	99	80	120	99	75	125

Carbon/Sulphur

Method: ASTM E1915-07A | Internal ref.: ME-CA-[ENV]ARD-LAK-AN-020

Parameter	QC batch Reference	Units	RL	Method Blank	Duplicate		LCS/Spike Blank			Matrix Spike / Ref.		
					RPD	AC (%)	Spike Recovery (%)	Recovery Limits (%)		Spike Recovery (%)	Recovery Limits (%)	
								Low	High		Low	High
Sulphide (Na2CO3)	ECS0053-AUG21	%	0.04	< 0.04	ND	20	99	80	120			

Conductivity

Method: SM 2510 | Internal ref.: ME-CA-[ENV]EWL-LAK-AN-006

Parameter	QC batch Reference	Units	RL	Method Blank	Duplicate		LCS/Spike Blank			Matrix Spike / Ref.		
					RPD	AC (%)	Spike Recovery (%)	Recovery Limits (%)		Spike Recovery (%)	Recovery Limits (%)	
								Low	High		Low	High
Conductivity	EWL0437-AUG21	uS/cm	2	< 2	ND	20	99	90	110	NA		

QC SUMMARY

pH

Method: SM 4500 | Internal ref.: ME-CA-[ENV]EWL-LAK-AN-001

Parameter	QC batch Reference	Units	RL	Method Blank	Duplicate		LCS/Spike Blank			Matrix Spike / Ref.		
					RPD	AC (%)	Spike Recovery (%)	Recovery Limits (%)		Spike Recovery (%)	Recovery Limits (%)	
								Low	High		Low	High
pH	EWL0437-AUG21	pH Units	0.05	NA	0		100			NA		

Method Blank: a blank matrix that is carried through the entire analytical procedure. Used to assess laboratory contamination.

Duplicate: Paired analysis of a separate portion of the same sample that is carried through the entire analytical procedure. Used to evaluate measurement precision.

LCS/Spike Blank: Laboratory control sample or spike blank refer to a blank matrix to which a known amount of analyte has been added. Used to evaluate analyte recovery and laboratory accuracy without sample matrix effects.

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

Reference Material: a material or substance matrix matched to the samples that contains a known amount of the analyte of interest. A reference material may be used in place of a matrix spike.

RL: Reporting limit

RPD: Relative percent difference

AC: Acceptance criteria

Multielement Scan Qualifier: as the number of analytes in a scan increases, so does the chance of a limit exceedance by random chance as opposed to a real method problem. Thus, in multielement scans, for the LCS and matrix spike, up to 10% of the analytes may exceed the quoted limits by up to 10% absolute and the spike is considered acceptable.

Duplicate Qualifier: for duplicates as the measured result approaches the RL, the uncertainty associated with the value increases dramatically, thus duplicate acceptance limits apply only where the average of the two duplicates is greater than five times the RL.

Matrix Spike Qualifier: for matrix spikes, as the concentration of the native analyte increases, the uncertainty of the matrix spike recovery increases. Thus, the matrix spike acceptance limits apply only when the concentration of the matrix spike is greater than or equal to the concentration of the native analyte.

LEGEND

FOOTNOTES

NSS	Insufficient sample for analysis.
RL	Reporting Limit.
↑	Reporting limit raised.
↓	Reporting limit lowered.
NA	The sample was not analysed for this analyte
ND	Non Detect

Samples analysed as received. Solid samples expressed on a dry weight basis. “Temperature Upon Receipt” is representative of the whole shipment and may not reflect the temperature of individual samples.

Analysis conducted on samples submitted pursuant to or as part of Reg. 153/04, are in accordance to the Protocol for Analytical Methods Used in the Assessment of Properties under Part XV.1 of the Environmental Protection Act” published by the Ministry and dated March 9, 2004 as amended.

SGS provides criteria information (such as regulatory or guideline limits and summary of limit exceedances) as a service. Every attempt is made to ensure the criteria information in this report is accurate and current, however, it is not guaranteed. Comparison to the most current criteria is the responsibility of the client and SGS assumes no responsibility for the accuracy of the criteria levels indicated. This document is issued, on the Client's behalf, by the Company under its General Conditions of Service available on request and accessible at http://www.sgs.com/terms_and_conditions.htm. The Client's attention is drawn to the limitation of liability, indemnification and jurisdiction issues defined therein. Any other holder of this document is advised that information contained hereon reflects the Company's findings at the time of its intervention only and within the limits of Client's instructions, if any. The Company's sole responsibility is to its Client and this document does not exonerate parties to a transaction from exercising all their rights and obligations under the transaction documents.

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-- End of Analytical Report --

Peto MacCallum Ltd.
CONSULTING ENGINEERS

PART B - PRELIMINARY FOUNDATION DESIGN REPORT FOR DESIGN-BUILD READY ALTERNATIVE BID PACKAGE

FOR

WELLINGTON ROAD 34 CONNECTOR UNDERPASS
SITE NO. 35X-0618/B0, STATION 10+000
LATITUDE AND LONGITUDE: 43.464815, -80.183754
MIDBLOCK INTERCHANGE (MBI) AREA
HIGHWAY 6 AND HIGHWAY 401 IMPROVEMENTS
FROM HAMILTON NORTH LIMITS TO GUELPH SOUTH LIMITS
CITY OF GUELPH, ONTARIO
GWP 3059-20-00

PETO MacCALLUM LTD.
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One (1) Digital, PDF
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1 cc: PML Toronto

PML Ref.: 17TF006A
Index No.: 066FDR
GEOCRES No.: 40P8-291
October 14, 2021

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PART B – PRELIMINARY FOUNDATION DESIGN REPORT

Peto MacCallum Ltd.
CONSULTING ENGINEERS

PART B - PRELIMINARY FOUNDATION DESIGN REPORT for Wellington Road 34 Connector Underpass

Site No. 35X-0618/B0, Station 10+000
Midblock Interchange (MBI) Area
Highway 6 and Highway 401 Improvements
From Hamilton North Limits to Guelph South Limits
City of Guelph, Ontario
G.W.P. 3059-20-00

7. PROJECT DESCRIPTION

7.1 General

The Ministry of Transportation of Ontario (MTO) proposed the re-alignment, improvement and replacement of existing structures located on Highway 6 and Highway 401 from Hamilton North Limits to Guelph South Limits, and retained AECOM Canada Ltd. (AECOM) to provide Owner's Engineer Services. The assignment consists of separate projects to be tendered under different delivery models. The Midblock Interchange (MBI) area is part of the Highway 6 and Highway 401, Hamilton to Guelph advance contract to be delivered on a design-build (DB) basis. MTO requires a Design-Build Ready alternative package for delivery of this project.

The main foundation engineering components of the MBI area project include two underpasses (the Wellington Road 34 Connector Underpass and the Wellington Road 34 Underpass), high fill and deep cut sections along Highway 6, Concession Road 7 and the new connector route, storm ponds, and overhead signs on both sides of the midblock interchange along Highway 6, south of Wellington Road 34 intersection, and near the intersection of Concession Road 7 and Highway 6.

AECOM retained Peto MacCallum Ltd. (PML) on behalf of MTO to provide foundation engineering services for the project. This report provides recommendations for foundation design of the proposed Wellington Road 34 Connector Underpass based on interpretation of the factual geotechnical data presented in the Foundation Investigation Report (Part A) and the General Arrangement (GA) drawing provided by AECOM. During the original fieldwork carried out in 2017 and the recent foundation investigation in 2021, attempts were made to locate the boreholes within the footprints of the foundation elements as provided in AECOM's preliminary design or GA drawings. Additional foundation investigation work should be considered if significant modifications/changes are made to the preliminary design of the underpass or the road alignment, and other related structures such as retaining walls.

It should be noted that this report is intended for use by AECOM, as MTO's authorized engineer, for the purpose of designing the Wellington Road 34 Connector Underpass at the location where the foundation investigations were conducted. It shall not be used for any other purposes or at any other locations, or by any other parties including design-build contractors. Where comments are made on construction, they are provided to highlight those aspects that could affect the design of the project and, for which, special provisions could potentially be required for construction. These comments identify only some issues and are not presented as an exhaustive list of construction concerns. The design-builder will remain responsible for making its own interpretation of construction issues. Recommendations for construction aspects of the foundations should be provided in design phase of the project.

Where necessary, reference is made in this report to the Canadian Highway Bridges Design Code (CHBDC, 2019 or CSA S6:19, 2019) and its Commentary, the Ontario Provincial Standard Specifications (OPSSs), Special Provisions (amendments to OPSSs), Design-Build Special Provision (Replacements to OPSSs), and the Ontario Provincial Standard Drawings (OPSDs). The foundation design for highway structures should be carried out in accordance with the guidelines and requirements provided in CHBDC (2019). The structure consequence classification defined in CHBDC (2019) is to be specified by MTO for each structure in the MBI area. The typical consequence classification was used in this report. The list of OPSSs and OPSDs cited in this report is provided in Appendix F.

7.2 Proposed Structure

Based on the preliminary GA drawing provided by AECOM on March 25, 2021, the new underpass will provide two-way access between Wellington Road 34 and Concession Road 7, and will be constructed as a two-span northeast-southwest oriented structure. The underpass will cross over the alignment of Highway 6 at a skew angle of about 30 degrees. Each span will have a length of 45.0 m and the approach slabs at both the east and west abutments will be 6.0 m long, each. The structure will have a width of 22.5 m and will consist of through-lanes, speed change lanes and shoulders. It will accommodate two lanes of traffic, as well as traffic to and from the new ramps.

The new underpass will be supported on 310 x 110 steel H-piles driven to bedrock at the centre pier and abutments. Based on the GA drawing, the centre pier will be located in the median of Highway 6 and will consist of four columns at 6.0 m centre to centre spacing. These columns will be supported by a footing founded on two rows of battered piles. The cut-off elevation of these piles will be at El. 321.6, and the base of the pile-cap is placed at about El. 321.0. At the east and west abutments, the footings will be placed on two rows of battered and vertical piles. The cut-off elevations of these piles will be at El. 325.6 and the base of the pile cap will be at El. 325.0.

The design grade of the approach embankments at the west and east abutments are set at approximately El. 330.6 and El. 331.6, respectively. The top of the road surface of Highway 6 will be at El 323. The grade of Highway 6 at the ditch will be at El. 321 on the west and at El. 322.5 on the east side. Based on this information, the approach embankments will be approximately 9.6 m and 8.5 m high, respectively. Construction of the new approaches to the designed grade will involve cutting back the existing slope at the west abutment and filling over at the east abutment. The height of the excavation (additional cut) on the west will be approximately 5.1 m and the fill on the east will be about 4 m. The width of the cut and fill will vary across the face of the slopes.

Further, the underpass will have wingwalls and retaining walls at the west and east abutments.

8. FOUNDATION RECOMMENDATIONS

8.1 Subsoil and Groundwater Conditions

Based on the findings of the foundation investigation conducted at the west and east abutments, the centre pier, and the toe of the east side of Highway 6, the subsurface at the location of the proposed underpass consists of a thin layer of topsoil underlain by 500 mm to 800 mm thick silty sand or sandy silt fill. Beneath the fill, a layer of silty sand/sandy silt till with varying proportions of gravel and cobbles and occasional sand lenses and silt seams was encountered. Bedrock was encountered below the till, at depths ranging from 21.4 m to 31.4 m (El. 301.2 to El. 296.6).

The stabilized groundwater elevations measured in monitoring wells installed in boreholes drilled at west and east abutments (35-618-02 and 35-618-08) of the proposed underpass and at the toe of Highway 6 (35-618-06) ranged from El. 310.2 to El. 312.5.

8.2 Foundation Alternatives

Based on the subsurface and groundwater conditions of the project site, the foundation alternatives discussed below are provided to facilitate the development of the conceptual/preliminary design for the Design-Build Ready project delivery package. The foundation alternatives are the following:

- Driven Steel H-Piles
- Shallow Foundation – Spread Footings
- Drilled shaft / large diameter caissons
- Micropiles

Table 5 provides the advantages, disadvantages, risks and consequences of the foundation alternatives for the proposed structure. Construction of deep foundations (driven piles and caissons) should conform to OPSS.PROV 903, as amended by SP 109F57. For design-build projects the use of DBSP0903 (Design-Build Special Provision replacement of OPSS 903) should be considered.

Table 5 - Comparison of Foundation Options

Foundation Type	Advantages	Disadvantages	Risks/Consequences	Relative Cost
Driven Steel Piles	Higher confidence level in settlement performance May not require deep excavations for forming pile caps	Potential vibration induced during driving Potential requirement to design for corrosion protection	Steel piles may require corrosion protection, in which case the corrosion protection would need to be designed by specialists.	Moderate
Shallow Foundations	Reduced nuisance to public for noise and vibration compared to pile driving Can be constructed with limited vertical clearance	Deep excavations and roadway protection or shoring would be required to construct spread footings	Low to medium risk of settlement and resulting distresses to bridges due to differential settlements	Foundation cost relatively low

Table 5 - Comparison of Foundation Options

Foundation Type	Advantages	Disadvantages	Risks/Consequences	Relative Cost
Caissons	Reduced nuisance to public for noise and vibration compared to pile driving Ability to achieve high axial and lateral capacity Facilitates elimination of pile cap if pile bent configuration adopted Ability to install through cobbles and boulders Can be installed with limited vertical clearance	Environmental concerns for potential use of slurry to stabilise the base during caisson installation May require permanent liner Construction procedures may influence the integrity and performance of the caisson Requirement for caisson integrity testing to test for potential necking of concrete	Potential for necking of concrete in caisson could reduce the reliability to support axial or lateral loads.	Moderate
Micropiles	Minimal disturbance and can be advanced through obstruction/boulders	Need a specialist contractor for design and construction The lateral load-carrying capacity of individual vertical micropiles is limited compared to driven piles Potential requirement to design for corrosion protection	The bond length design of micropiles is based on the assumption that the load is resisted uniformly by shaft friction. Hence the micropiles should have sufficient embedment into the bedrock	Moderate

8.2.1 Option 1: Steel H-Piles Driven to Bedrock

Based on the subsoil conditions encountered at the project site, the two abutments and centre pier may be supported on HP 310 x 110 steel piles driven to bedrock surface elevations ranging from El. 300.1 to El. 296.6. The use of steel H-Piles is also the preferred and recommended option.

Steel H-piles driven to bedrock may be designed assuming a factored axial geotechnical resistance of 2000 kN at Ultimate Limit State (ULS). Geotechnical resistance at Serviceability Limit State (SLS) will not govern because the load required to produce detrimental deformation for piles driven to bedrock is anticipated to be larger than the recommended factored resistance at ULS.

Table 6 provides the approximate pile tip elevations and the lengths of piles that may be considered for design purposes based on pile cut-off elevations.

Table 6 - Approximate Pile Tip Elevations and Length for HP 310 x 110

LOCATION	PILE TIP ELEVATION	PILE CUT-OFF ELEVATION	LENGTH (m)
East Abutment	297.1 ± 1.0	323.0	25.9 ± 1.0
Centre Pier	300.1 ± 1.0	320.5	20.4 ± 1.0
West Abutment	296.6 ± 1.0	323.0	26.4 ± 1.0

It should be noted that the second borehole and DCPT (borehole 35-618-3A/B) on the west abutment encountered refusal on a possible boulder at El. 314.5; accordingly, this condition should be investigated further by drilling additional boreholes during detail design.

Considering the cobbles and gravels encountered below about El. 320.0 m, the pile tips may have to be reinforced to drive the piles through very dense soil strata to bedrock surface.

For detail foundation design, the lateral capacity of steel H-piles should be determined by load-displacement (p-y) curve analyses to model the non-linear behaviour of soils. This method allows determination of pile top deflections and bending moments of laterally loaded piles. The p-y curves are developed from pile load tests, but it is also common to use p-y curves developed by computer programs from input parameters such as pile diameter and the type of soil strata.

8.2.2 Option 2: Shallow Foundation – Spread Footings

As an alternative to H-piles, the abutments and centre pier of the proposed structure may be supported on spread footings placed at or below the depths provided in Table 7. The geotechnical resistances given in Table 7 may be assumed for minimum of 2.5 m wide spread footings.

Table 7 - Founding Elevation and Geotechnical Bearing Resistance for Shallow Foundation

LOCATION	FOUNDING ELEVATION	FACTORED GEOTECHNICAL RESISTANCE AT ULS (kN)	GEOTECHNICAL RESISTANCE AT SLS (kN)
East Abutment	323.0 ± 1.0	500	350
Center Pier	320.5 ± 1.0	450	300
West Abutment	323.0 ± 1.0	500	350

The bearing resistance for inclined loads should be reduced in accordance with the requirements of Clause 6.10.2 of the CHBDC (2019).

The total settlement under a Serviceability Limit State (SLS) load of 350 kPa and 300 kPa is expected to be in the order of 20 mm to 25 mm and the associated differential settlement will vary form 15 mm to 20 mm. Most of the total settlement is expected to come from elastic compression of the subgrade and completed immediately after completion of construction. Continuing long-term total or differential settlements under the weight of the structure may be negligible.

The silt sand to sandy silt till subgrade may be susceptible to disturbance from construction traffic and/or any ponded water. To limit the degradation, it is suggested that a concrete working slab (lean concrete) be placed on the subgrade within four hours after preparation, inspection and approval of the footing subgrade. This requirement may be addressed with a note on the structural drawing for foundation and/or with a Non-Standard Special Provision (NSSP).

The sliding resistance of footings against lateral loads between the concrete footing and subgrade should be calculated in accordance with Clause 6.10.4 of the CHBDC (2019). For cast-in-place concrete footing constructed on concrete working slab and on top of silty sand/sandy silt till subgrade, the coefficient of friction angle should be taken as follows:

- Cast-In-Place footing on concrete working slab = 0.6
- Cast-In-Place concrete working slab on compact to dense silty sand = 0.65

Considering the elevations of stabilized groundwater levels (El. 310.2 to El. 312.5), no major dewatering requirements are anticipated for footings placed at elevations given in Table 7.

8.2.3 Option 3: Caissons

The centre pier of the proposed underpass may be supported on 25.5 m to 27.5 m long caissons founded at depths of EL. 296.5 to El. 294.5 in bedrock. Caissons socketed three times the diameter into the bedrock may be designed assuming the geotechnical resistances given in Table 8.

Table 8 - Factored Geotechnical Resistance at ULS

CAISSON DIAMETER	CAISSON TIP ELEVATION	FACTORED GEOTECHNICAL RESISTANCE AT ULS (kN)
1200 mm	296.5	5000
1500 mm	295.5	7500
1800 mm	294.5	10000

A temporary liner will be required to install the caissons. The construction of the caissons should conform to OPSS.PROV 903, as amended by SP 109F57. For design-build projects, use of DBSP0903 (Design-Build Special Provision replacement of OPSS 903) should be considered.

Lateral resistances at ULS and SLS provided in Table 9 may be utilized for the design of the caissons socketed three times the diameter into the bedrock.

Table 9 - Lateral Resistance at ULS and SLS

CAISSON DIAMETER	LATERAL RESISTANCE	
	ULS (kN)	SLS (kN)
1200 mm	350	150
1500 mm	520	220
1800 mm	700	300

8.2.4 Option 4: Micropiles

Micropiles can also be considered as an option to support the proposed underpass. These piles are installed to the required depths with minimal disturbances and can be advanced through/past obstructions including boulders. The actual embedment or bond length should be decided based on the geotechnical resistance required to support the design load, the diameter of the drill hole, and the grout to ground ultimate bond strength. The bond length design is based on the assumption that the load is resisted uniformly by shaft friction. Micropile foundations must be designed and installed by a specialized contractor. The contractor should select the type of micropiles and the method of drilling and grouting, based on specific performance criteria (permissible movements).

For detailed design, site-specific load testing should be carried out to define the factor of safety or the geotechnical resistance applied to ultimate bond strength values. The lateral load-carrying capacity of individual micropiles is limited compared to driven piles. If considered, micropiles can be battered as a means to provide additional resistance to lateral loading.

9. WINGWALL AND RETAINING WALLS

The GA drawing indicates that wingwalls and retaining walls are to be constructed on both sides of the west and east approach embankments to support cut and fill slopes, respectively. Both reinforced concrete gravity (or cantilever) and reinforced soil system (RSS) walls are geotechnically feasible alternatives for retaining walls at the project site. The cantilever walls could be cast-in-place (CIP) or made up of precast elements. Cast-in-place concrete gravity and cantilever walls will require longer construction time and deeper excavations compared to RSS walls.

Concrete or RSS retaining walls should be founded on native silty sand/sandy silt till, and designed using a factored geotechnical bearing resistance at ULS of 225 kPa and a bearing resistance at SLS of 150 kPa. The factored geotechnical resistance at ULS was based on a factor of 0.5 as recommended in the Canadian Highway Bridge Design Code (CHBDC, 2019). In order to limit the degradation of the founding soil, it is recommended that a concrete working slab be placed on the subgrade within four hours of preparation, inspection and approval. Based on existing conditions, no major global stability problems are expected for the proposed retaining wall.

10. LATERAL EARTH PRESSURES

Lateral earth pressure acting on the back of a concrete retaining structure can result in sliding or overturning. Hence, earth retaining walls or abutments should be designed to resist the lateral earth pressure imposed by a backfill and any surcharge load including traffic loads. The lateral earth pressure acting on a bridge abutment or retaining structure depends on the type of backfill, the native materials, the method of placement of the backfill, the freedom of movement of the structure, the surcharge pressure, and the drainage conditions behind the walls.

Generally, the earth pressure for concrete structures should be computed as per Clause 6.12.2 of Canadian Highway Bridge Design Code (CHBDC, 2019). The lateral earth pressure, p (kPa), may be computed using the following equation, assuming a triangular pressure distribution:

$$p = K (\gamma h_1 + \gamma' h_2 + q) + \gamma_w h_2 + C_p + C_s$$

where K = Coefficient of lateral earth pressure (dimensionless)
γ = Unit weight of backfill material above assumed water level (kN/m^3)
γ' = Unit weight of submerged backfill (γ_{sat} - γ_w) material below assumed water level (kN/m^3)
γ_w = Unit weight of water (9.8 kN/m^3)
h_1 = Depth below final grade above design water level (m)
h_2 = Depth below design water level (m)
q = Surcharge load (kPa)
C_p = Compaction pressure (kPa) (Clause 6.12.3 of CHBDC, 2019)
C_s = Earth pressure from seismic events, (kPa) (Clause 6.14.7 of CHBDC, 2019)

Free draining granular material meeting the specifications of OPSS.PROV 1010 Material Specifications for Aggregates, Granular ‘A’ or Granular ‘B’ Type II should be used as backfill, and in accordance with OPSS 902. This material should be compacted according to OPSS 501, as amended by SP 105S22. Transverse drains and weep holes should be installed to prevent the build-up of hydrostatic pressure and provide positive drainage in accordance with OPSD 3190.100, where applicable. Other aspects of the granular backfill requirements with respect to sub drains and frost taper should be in accordance with the requirements OPSD 3101.150 and OPSD 3121.150.

For the case where the pressures are based on granular fill behind the wall, the parameters given in Table 10 may be assumed. The coefficient of earth pressure "at rest" should be used for design of rigid and unyielding walls where sufficient movement of the structure wall will not be permitted. For unrestrained structures, the "active" earth pressure coefficient should be employed.

It should be understood that the parameters and unfactored values of earth pressure coefficients given in Table 10 assume that the ground surface behind the structure is horizontal. If the retained ground is sloping, the coefficients must be adjusted to account for the slope based on the equations provided on Figures C6.28 and C6.29 of the Commentary (S6.1:19) to CHBDC (2019).

A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the wall stem, based on Clause 6.12.3 and Figure 6.6 of the CHBDC (2014). Compaction equipment should be used based on MTO Special Provision 105S22. In case of surcharge loadings, it should be accounted for in the design, as required.

Table 10 - Earth Pressure Coefficients for Granular Fill

PARAMETERS	OPSS GRANULAR 'A'	OPSS GRANULAR 'B' TYPE II
Internal Friction Angle, (degrees)	35°	32°
Unit weight, γ (kN/m^3)	22.5± 0.3	21.5 ± 0.3
Coefficient of Active Earth Pressure, Ka	0.27	0.33
Coefficient of Earth Pressure at Rest, Ko	0.43	0.5
Coefficient of Passive Earth Pressure, Kp	3.69	3

The granular fill may be placed either in a zone with width equal to at least 1.2 m behind the back of the wall stem (Figure C6.31(a) of the Commentary to the CHBDC (2019)) or within the wedge-shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing (Figure C6.31(b) of the Commentary to the CHBDC (2019)).

11. APPROACH EMBANKMENTS

11.1 West Approach Embankment

The existing ground surface on the west side of the proposed underpass varies from El. 322.5 at the bottom of the roadside ditch to El. 328.5 at the top of the slope. Hence, the height of the existing cut slope is about 6.0 m. On the other hand, the new grade of the west approach embankment is set at El. 330.6, and the grade at the ditch line of Highway 6 will be at El. 321, resulting in 9.6 m high approach embankment with 2H:1V or flatter abutment fore slope.

Generally, the fore slopes of abutments or side slopes of approach embankments exceeding 8.0 m in height will require a 2.0 m wide mid-height bench or berm to control surficial erosion and improve stability. These embankments should be constructed in accordance with OPSD 202.010.

The preliminary GA drawing provided by AECOM indicated that the abutment fore slope on the west side of the underpass will involve construction of a mid-height berm. In addition, the soils encountered in boreholes drilled in the area are dense to very dense, and the groundwater level is relatively deep. Furthermore, construction of wingwalls and retaining walls on both sides of the structure will contribute to the stability of the slope and provide additional strength to the abutment. Hence, the occurrence of slope failures that can affect the performance of the proposed structure is assumed to be negligible for an approach embankment with 2H:1V or flatter abutment fore slope.

Further, slope stability analyses were conducted to assess the performance of the abutment fore slope constructed with 2H:1V side slopes or flatter and mid-height berm. All stability analyses were carried out with a computer program, *Slope-W*. For all stability analyses, the Spencer method was used to estimate the factor of safety against rotational shear failures. The Spencer method analyzes potential circular shear surfaces by separating the materials above the failure plane into multiple segments and then using force and moment equilibrium to balance the forces in each segment.

For the purpose of conducting slope stability analyses for abutment fore slope, slope geometries were obtained from preliminary GA drawing provided by AECOM. The subsurface and groundwater conditions were obtained from boreholes drilled in the area. Since the subsurface encountered in boreholes consisted mainly of cohesionless (granular) soils, a drained condition was assumed and the stability analyses were conducted using effective stress approach.

A summary of the assumed shear strength parameters for soil layers encountered in boreholes drilled at or near the proposed locations of the proposed structure is provided in Table 11. The parameters were determined using the information from boreholes, the soil profiles presented in Foundation Investigation Report (Part A), and based on literature and previous experiences.

Table 11 – Soil Strength Parameters

MATERIAL	SOIL PROPERTY		
	BULK UNIT WEIGHT (kN/m^3)	INTERNAL FRICTION ANGLE (ϕ’)	DRAINED SHEAR STRENGTH (c’) kPa
Pavement Fill	21	32	-
Embankment Fill	20	30	-
Compact Silty Sand/Sandy Silt (Till)	19	28	-
Very Dense Silty Sand/Sandy Silt (Till)	19	34	-

The result of the slope stability analysis is given in Drawing 35-618-2 in Appendix G, and the Factor of Safety (FOS) value for a rotational slip plane was found to be 2.0. This value is higher than the 1.5 that is often used to design highway approach embankments with no risk slope instability.

Further, the proposed embankment at the west approach may be expected to induce settlement of the founding soil in the order of 10 mm to 15 mm. In addition, any fill required is expected to settle by 0.5% to 1.0% (50 mm to 100 mm) of the fill height, depending on the type of fill material and the method of placement. However, the majority of the estimated settlement will be in the form of elastic compression and will be completed immediately after the construction of the embankment. To mitigate post-construction settlements, paving of the road may need to be delayed by two to four weeks after the placement of a fill to the designed grade of the embankment.

Organic or soft soils were not encountered in boreholes drilled in the area. However, any soft/weak materials/soils observed within the base (design subgrade level) footprint of the embankment should be removed before placing fill. In addition, the slopes of the approach embankment should be protected against surface erosion by sodding and suitable vegetation. Refer to OPSS 803 and OPSS 804 for the type of seed and mulch required. Further, granular sheeting meeting the requirements of OPSS.PROV 1004 may be provided in areas where seepage is identified.

11.2 East Approach Embankment

The existing ground surface at the east approach embankment varies from El 323.1 to El. 330.2. Hence, the height of the existing cut slope is approximately 7.1 m. The new grade at the east approach of the proposed underpass is set at El. 331.6 and the embankment at this location is required to be constructed by placing additional fill over the existing slope. The benching for the placement of fill on an existing slope should be in accordance with OPSD 208.010.

The height of the approach embankment/abutment slope is expected to be about 8.5 m above the grade of Highway 6 at the location of the east abutment. Generally, abutment for slopes or side slopes exceeding 8.0 m in height require a 2.0 m wide mid-height bench to control surficial erosion and improve stability. These embankments should be constructed in accordance with OPSD 202.010.

However, at the project site, it is understood that the abutment structure on the east side will be constructed in the upper part of the abutment slope and this will reduce the height of the uninterrupted slope to about 5 m. Hence, major instability problems are unlikely to occur. In addition, since the soils encountered in boreholes drilled in the area are dense and the groundwater level is relatively deep, the probability of occurrence of slope failures that can affect the performance of the proposed structure is assumed to be negligible. Furthermore, construction of wingwalls and retaining walls in the area will help in stabilizing and providing additional strength to the abutment slope.

Further, slope stability analyses were conducted to assess the performance of the abutment fore slope constructed with 2H:1V side slopes or flatter. Slope geometries for the abutment fore slope were obtained from preliminary GA drawing provided by AECOM. The subsurface and groundwater conditions were obtained from boreholes drilled in the area. Since the subsurface encountered in boreholes consisted mainly of cohesionless (granular) soils, a drained condition was assumed and the stability analyses were conducted using effective stress approach.

A summary of the assumed shear strength parameters for soil layers encountered in boreholes drilled at or near the proposed locations of the proposed structure is provided in Table 11. The parameters were determined using the information from boreholes, the soil profiles presented in Foundation Investigation Report (Part A), and based on literature and previous experiences.

The result of the slope stability analysis is given in Drawing 35-618-3 in Appendix G, and the Factor of Safety (FOS) value for a rotational slip plane was found to be 2.1. This value is higher than the 1.5 that is often used to design highway approach embankments with no risk slope instability.

The proposed embankment at the east approach embankment is expected to induce settlement of the founding soil in the order of 10 mm to 15 mm. In addition, any fill required is expected to settle by 0.5% to 1.0% (50 mm to 100 mm) of the fill height, depending on the type of fill material and the method of placement. However, the majority of the estimated settlement will be in the form of elastic compression and will be completed immediately after the construction of the embankment. To mitigate post-construction settlements, paving of the road may need to be delayed by two to four weeks after the placement of the fill to the designed grade of the embankment.

Organic or soft soils were not encountered in boreholes drilled at the east approach. However, any soft/weak materials observed within the base (design subgrade level) footprint of the embankment should be removed before placing the fill. In addition, the slopes of the approach embankment should be protected against surface erosion by sodding and suitable vegetation. Refer to OPSS 803 and OPSS 804 for the type of seed and mulch required. Further, granular sheeting meeting the requirements of OPSS.PROV 1004 may be provided in areas where seepage is identified.

12. SEISMIC CONSIDERATIONS

The Spectral (Sa(T), where T is in seconds) and Peak Ground Acceleration (PGA) for the project site are 0.151 (Sa (0.2)) and 0.094 (2%/50 years), respectively, based on the longitude and latitude coordinates of the proposed structure and the procedures provided in the National Building Code of Canada (2015). The PGA for the site is 0.075 in accordance with Clause 4.4.3.3, CHBDC (2019).

The seismic site classification uses the conditions of soils encountered in the upper 30 m of the subsurface. Based on the average SPT "N" values of the silty sand to sandy silt fill and the underlying till deposit and the bedrock, the subsurface at the site is classified as Type D for seismic design purposes, based on Clause 4.4.3.2 of CHBDC, 2019.

In accordance with Clause 4.4.4, CHBDC (2019), a seismic performance category of 1 (major–route and other bridges) is considered for the site.

The site class as well as the history of seismicity indicates that the risk of seismic activity at this location is low. Hence, no seismic design considerations are anticipated for this structure.

13. FROST PROTECTION

Based on OPSD 3090.101, the frost penetration depth for the region is approximately 1.2 m. All pile caps or footings shall be provided with a minimum of 1.2 m earth cover or equivalent thermal insulation as protection against frost action. A section of any footing exposed for frost action should be covered by non-frost susceptible granular material.

14. CONSTRUCTION CONSIDERATIONS

14.1 Roadway Protection

Based on the preliminary GA drawing, the construction of temporary roadway protection systems may be required at the location of the centre pier. A roadway protection system shall be designed and constructed by the contractor and should meet Performance Level 2, in accordance with OPSS.PROV 539, amended by SP 105S09. The use of DBSP0539 (design-build special provision replacement of OPSS.PROV 539) should also be considered. Performance Level 2 requires a Maximum Angular Distortion of 1:200 and a Maximum Horizontal Displacement of 25 mm.

The soil parameters given in Table 12 may be used for the design of roadway protection systems required at the project site. The requirements provided in OPSS.PROV 539 or DBSP0539 also call for monitoring of the roadway protection system used in the area by the contractor (design-builder) to check the horizontal and vertical displacements of the existing roadway.

Table 12 - Soil Parameters for the Preliminary Design of Roadway Protection System

ELEVATION		SOIL TYPE	UNIT WEIGHT, (kN/m^3)	PRELIMINARY DESIGN PARAMETERS	
FROM	TO			EFFECTIVE FRICTION ANGLE, (Φ)	UNDRAINED SHEAR STRENGTH, S_u (kPa)
321.0	318.0	Compact Sandy Silt to Silty Sand (Fill)	19	30	-
318.0	312.0	Dense Silty Sand/Sandy Silt Till	20	34	-

14.2 Excavation

Based on the preliminary GA drawing, the excavation depth to facilitate construction of the pile cap at the proposed pier location is not expected to exceed 2.0 m below the existing grade of Highway 6. In case the shallow foundation option is used, the excavation depth to the recommended founding elevation (El.320.0) will be about 2.5 m below the grade of the highway.

The existing fill may be considered as Type 3 soil and the dense silty sand/sandy silt till may be taken as Type 2 soil in accordance with Occupational Health and Safety Act (OHSA). In dense till deposits, temporary excavation slopes of 1H:1V over the full depth of excavation or flatter slopes can be constructed assuming adequate drainage measures are in place. Temporary shoring systems may be required if such drainage measures cannot be provided.

Considering the depth of the expected excavation, an open cut may be feasible depending on the setback or availability of space from the edge of the shoulders of Highway 6. A slope of 2H:1V may be used for an open cut excavation. The assessment of space for open cut excavation should take into consideration of the Ministry of Labour requirements. It requires a minimum of 1.0 m horizontal space behind Temporary Concrete Barrier adjacent to excavations for safety purposes. Generally, it is assumed that space will not be a problem at the site. It should be noted that, as per OHSA, the open-cut excavation procedures are governed by soils with the highest soil type number.

The contractor is responsible for the design of temporary shoring required for excavation walls. It is anticipated that excavation walls will be shored using trench boxes and sheet piles or a combination of shoring systems, depending on the depth of excavation. The design of temporary shoring should account for lateral pressures exerted by the soil, surcharge load from construction traffic, and temporary stockpiles adjacent to the excavation. If dewatering is not considered, the design should also include the hydrostatic pressure exerted by groundwater behind the shoring, if applicable.

Excavating and backfilling in the area should be in accordance with OPSS 902, or DBSP0902 (the design-build special provision replacement of OPSS 902).

Excavation of the soils should be feasible using conventional excavation equipment. All excavated surfaces should be kept free of frost and water during the period of construction. Runoff shall be directed away from open excavations and should not be allowed to flow into the excavation. Excavated material shall not be stockpiled on top of the excavation.

Prior to excavation, the locations and depths of existing underground utilities, if applicable, should be verified. All underground utilities that might be exposed and become unsupported as a result of the excavation should be properly supported to avoid potential damage.

14.3 Groundwater Control

At the structure location, the stabilized groundwater elevations measured in monitoring wells installed in boreholes drilled at west and east abutments (35-618-02 and 35-618-08) of the proposed underpass and at the toe of Highway 6 (35-618-06) ranged from El. 310.2 to El. 312.5.

Considering the depth of excavation and the stabilized groundwater levels at the site, no major dewatering requirements are anticipated. Seepage from soil fissures or surface run-off that enters the excavations can be handled by conventional sump pumping techniques.

Generally, if required, the contractor should be responsible for selection, performance and detailed design of the dewatering system. The dewatering system should be designed to conform to the requirements of OPSS.PROV 517. If pumping of groundwater at volumes greater than 50,000 L/day and less than 400,000 L/day is required, the Environmental Activity Sector Registry (EASR) must be completed. An EASR may not be required to temporarily pump surface water from behind a dewatering system, as long as the water is returned to the original source. If water taking in excess of 400,000 litres/day is required, a Permit to Take Water (PTTW) must be obtained in advance from the Ministry of the Environment, Conservation, and Parks (MECP). If sheet piles are installed to adequate depths to cut-off groundwater inflows, pumping volumes are anticipated to be less than 400,000 litres/day and PTTW applications would not be required. The actual rate of groundwater taking will be a function of the final design, time of the year, and the contractor’s schedule, equipment, and techniques.

14.4 Subgrade Preparation and Embankment Construction

The approach embankment fills should consist of well compacted and acceptable native or granular material. Organic or soft soils were not encountered in the boreholes drilled at both the west and east approaches. However, any soft/weak materials/soils observed within the base (design subgrade level) footprints of the approach embankments should be removed before placing the fill. The depth and extent of stripped material should be determined during the detail design phase.

After stripping of soft and compressible materials to the specified depth, the exposed subgrade should be proof rolled to identify any area requiring sub-excavation and the sub-excavation shall be backfilled using acceptable fill compacted as specified. Embankment fill should be placed and compacted in accordance with OPSS.PROV 206. Sod application and vegetation cover should be in conformance with OPSS.PROV 803. Measures to reduce erosion of the embankment slopes due to surface runoff should be considered during the detail design phase of the project and may include placement of topsoil and sod as soon as practicable after the construction of the embankments is completed. Erosion protection measures should be in accordance with OPSS.PROV 804.

Design provisions to manage differential settlements of immediate approaches to structures and the use of wider embankments should be addressed during detail design. Consideration should be given to surcharging/preloading to reduce settlements before paving.

14.5 Pile Installation and Obstructions

Due to the depositional process for glacial till, there is a risk that cobbles and boulders will be encountered within these deposits. The presence of cobbles and boulders may pose installation challenges for driven piles and caissons. Pile tip reinforcement is recommended for driven piles to prevent damage when driving through till soils with boulders and/or cobbles. Caisson installation equipment must be capable of penetrating dense glacial till and dislodging, removing or penetrating any obstructions such as cobbles and boulders. Design-Build contract documents should include a Non-standard Special Provision (NSSP) to alert bidders of the potential difficulty and/or issues pertaining to driving steel H-piles or excavating caissons in till soils with cobbles and boulders.

14.6 Soil Corrosivity

Two samples from the silty sand/sandy silt till were tested for soil corrosivity and potential exposure of concrete to sulphate attack. The samples were taken from Borehole 35-618-04 at a depth of about 2.6 m (El. 319.7 m) and Borehole 35-618-08 at a depth of 3.3 m (El. 325.2). The boreholes were drilled near or at the proposed location of the centre pier and east abutment. A summary of the test results is provided in Table 4 given in Section 5.4 of Part A of this report.

As shown in Table 4, the sulphate concentration was reported to be 0.0006% (6 µg/g) and 0.0009% (9.4 µg/g). According to Clause 4.1.1.1 of the Canadian Standards Association (CSA) A23.1-19/A23.2-19 Standard (2019), soluble sulphate concentrations less than 0.1% (1000 µg/g) generally indicate a low degree of sulphate attack when concrete is in contact with soil or groundwater. Hence, the potential for sulphate attack of concrete at the project site is low.

Moreover, a resistivity value of higher than 2000 ohm-cm is generally considered not corrosive environment for soil in contact with steel. The resistivity value provided in Table 4 for the till is higher than 2000 ohm-cm, and indicates a non-corrosive environment for buried metal.

The pH value in Table 4 is within the normal range expected for soil pH.

It is expected that granular backfill will be used at structure location for embankment construction. Generally, no sulphate attack is expected from granular backfill materials. However, it may be advisable to test backfill material for corrosion potential if it is imported from unknown sources. Additional corrosivity tests of native soils and reference to MTO Gravity Pipe Design Manual (2014), are recommended before the selection of cement types and steel products.

15. CLOSURE

This report was prepared by Mr. Lul Yimam, PhD, P.Eng., and reviewed by Mr. Geoffrey. Uwimana, MEng., P.Eng., Senior Engineer and MTO Designated Principal Contact.p

Yours very truly

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APPENDIX F

List of Ontario Provincial Standard Specifications (OPSSs), Drawings (OPSDs), Special Provisions (SPs) and Design-Build Special Provisions (DBSPs) Cited in the Report

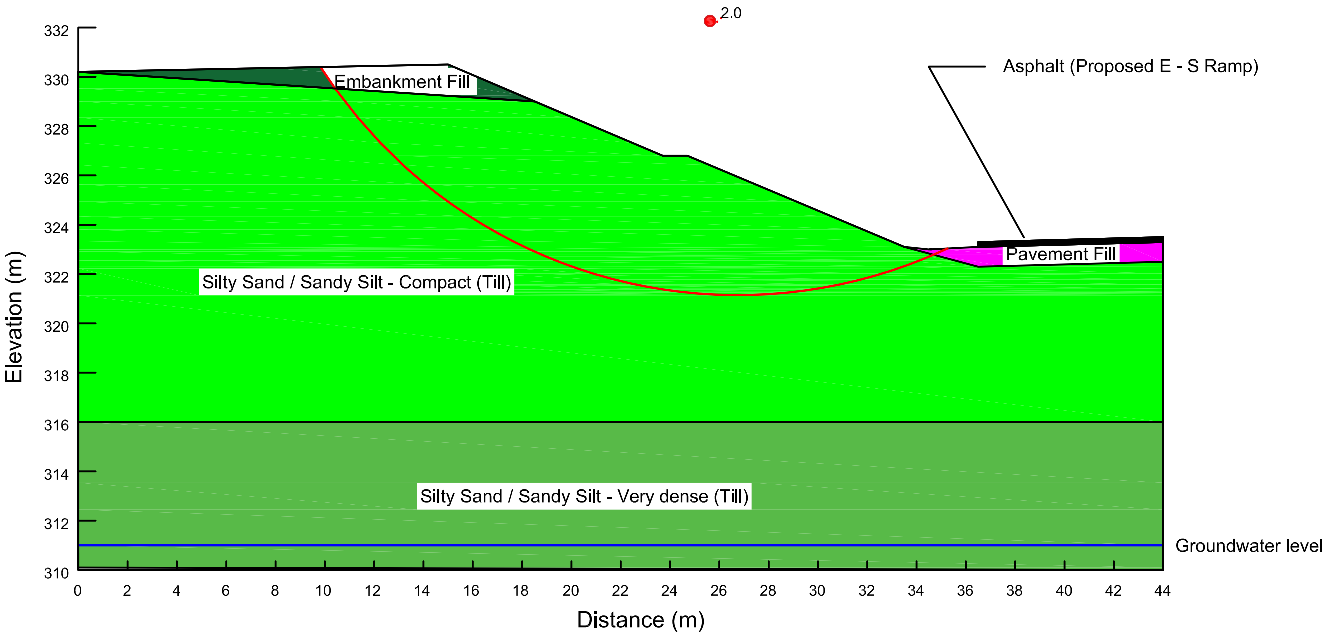
DOCUMENT	TITLE	REVISION DATE
OPSS.PROV 903	Construction Specification for Deep Foundations	April, 2016
SP 109F57	Special Provision (Amendment) to OPSS 903	January, 2020
DBSP0903	Design-Build Special Provision (Replacement) to OPSS 903	August, 2020
OPSS.PROV 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade, and Backfill Material	April, 2013
OPSS 902	Construction Specification for Excavation and Backfilling of Structures	November, 2019
DBSP0902	Design-Build Special Provision (Replacement) to OPSS 902	August, 2020
OPSS 501	Construction Specification for Compacting	November, 2014
SP 105S22	Amendment to OPSS 501, Target Density Control Strip	June, 2016
OPSD 3190.100	Walls Retaining, and Abutment Wall Drain	November, 2010
OPSD 3101.150	Walls, Abutment, Backfill, Minimum Granular Requirement	November, 2010
OPSD 3121.150	Walls Retaining, Backfill Minimum Granular Requirement	November, 2010
OPSS.PROV 803	Construction Specification for Vegetative Cover	November, 2020
OPSS.PROV 804	Construction Specification for Temporary Erosion Control	November, 2020
OPSS.PROV 1004	Material Specification for Aggregates	November, 2012
OPSD 202.010	Embankment Construction using Excess Material Outside of Earth or Rock Fill	November, 2010
OPSD 208.010	Benching of Earth Slopes	April, 2019
OPSD 3090.101	Foundation, Frost Penetration depths for Southern Ontario	November, 2010
OPSS.PROV 539	Construction Specification for Temporary Protection Systems	November, 2014
SP 105S09	Amendment to OPSS 539	March, 2018
DBSP0539	Design-Build Special Provision (Replacement) to OPSS 539	August, 2020
OPSS.PROV 517	Construction Specification for Dewatering	November, 2016
OPSS.PROV 206	Construction Specification for Grading	November, 2014

APPENDIX G

Results of Slope Stability Analyses

SOIL STRENGTH PARAMETERS

PROPERTY / MATERIAL	ASPHALT	PAVEMENT FILL	EMBANKMENT FILL	SILTY SAND / SANDY SILT, COMPACT (TILL)	SILTY SAND / SANDY SILT, VERY DENSE (TILL)
Bulk Unit Weight (kN/m^3)	–	21	20	19	19
Internal Friction Angle (°)	–	32	30	28	34
Drained Shear Strength, (Cohesion) (kPa)	–	–	–	–	–



Proposed Wellington Road 34 Connector Underpass
(SITE NO. 35X–0618/B0)
West Abutment Foreslope Slope Stability Analysis
MIDBLOCK INTERCHANGE AREA, HIGHWAY 6 & HIGHWAY 401 IMPROVEMENTS
CITY OF GUELPH, ONTARIO

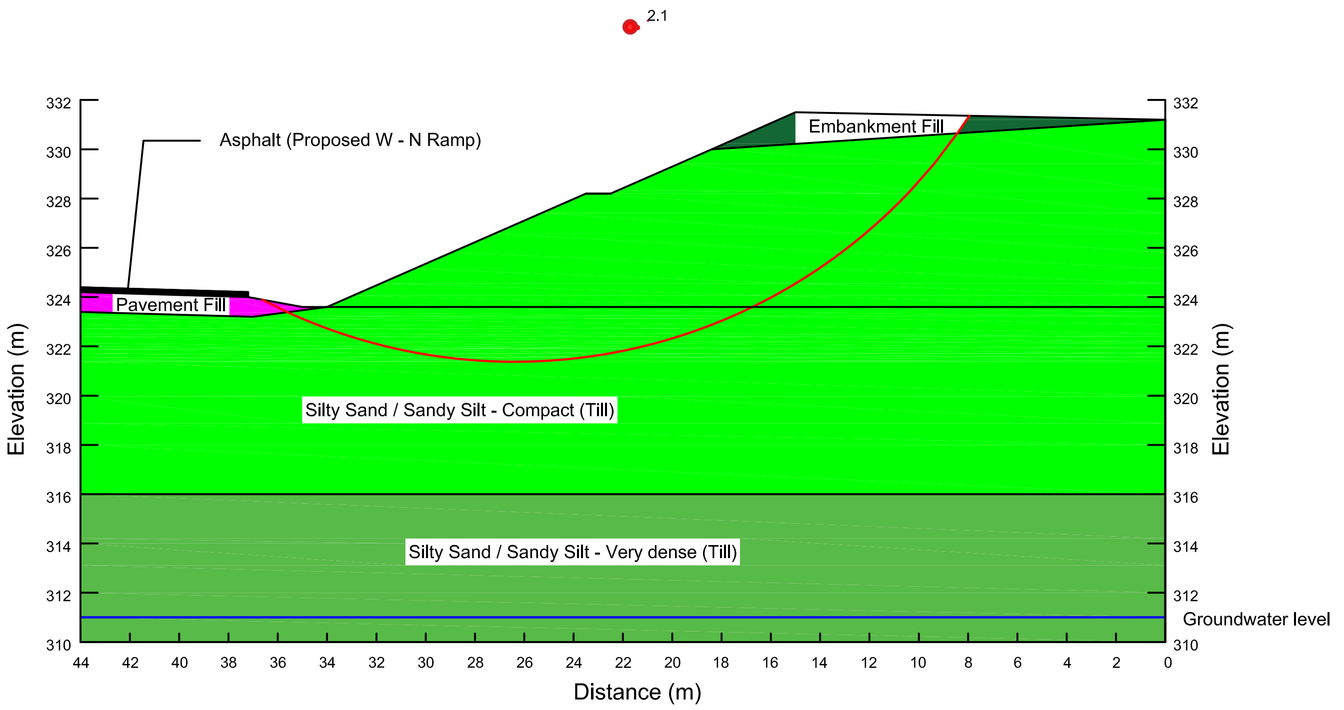
CONTRACT NO:	
G.W.P.:	3059–20–00
HWY NO.:	6
DISTRICT:	31

Peto MacCallum Ltd.
CONSULTING ENGINEERS

		JOB NO.	DATE	DRAWING NO.
DRAWN:	P.J.	17TF006A	OCT 14, 2021	35–618–3
CHECKED:	L.Y.			
APPROVED:	G.U.			

SOIL STRENGTH PARAMETERS

PROPERTY \ MATERIAL	ASPHALT	PAVEMENT FILL	EMBANKMENT FILL	SILTY SAND / SANDY SILT, COMPACT (TILL)	SILTY SAND / SANDY SILT, VERY DENSE (TILL)
Bulk Unit Weight (kN/m^3)	–	21	20	19	19
Internal Friction Angle (°)	–	32	30	28	34
Drained Shear Strength, (Cohesion) (kPa)	–	–	–	–	–



Proposed Wellington Road 34 Connector Underpass
(SITE NO. 35X–0618/B0)
East Aburment Foreslope Slope Stability Analysis
MIDBLOCK INTERCHANGE AREA, HIGHWAY 6 & HIGHWAY 401 IMPROVEMENTS
CITY OF GUELPH, ONTARIO

CONTRACT NO:	
G.W.P.:	3059–20–00
HWY NO.:	6
DISTRICT:	31

PML Peto MacCallum Ltd.
CONSULTING ENGINEERS

		JOB NO.	DATE	DRAWING NO.
DRAWN:	P.J.	17TF006A	OCT 14, 2021	35–618–4
CHECKED:	L.Y.			
APPROVED:	G.U.			