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

FOR

WELLINGTON COUNTY ROAD 34 UNDERPASS
SITE NO. 35X-0617/B0, STATION 10+000
LATITUDE AND LONGITUDE: 43.457324, -80.180489
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.
165 CARTWRIGHT AVENUE
TORONTO, ONTARIO
M6A 1V5
Phone: (416) 785-5110
Fax: (416) 785-5120
Email: toronto@petomaccallum.com

Distribution:
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PML Ref. 17TF006A
Index No.: 061FIR and 062FDR
GEOCRES No.: 40P8-292
October 14, 2021

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

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PART A – PRELIMINARY FOUNDATION INVESTIGATION REPORT

PART A – PRELIMINARY FOUNDATION INVESTIGATION REPORT for Wellington County Road 34 Underpass

Site No. 35X-0617/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

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 35-617-1 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 North, 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 meets Highway 6.

AECOM retained Peto MacCallum Ltd. (PML) on behalf of MTO to provide geotechnical 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 Underpass (35X-0617/B0), and 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 was prepared 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 additional (2021) foundation investigations for the proposed Wellington Road 34 Underpass (35X-0617/B0).

The FIR and FIDR for Wellington Road 34 Connector Underpass (35X-0618/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.

Highway 6 in the area has a signalized, at-grade intersection with Wellington Road 34, and consists of northbound (NBL) and southbound (SBL) lanes with a grassed median strip. Local relief at the intersection is less than 1 m. The intersection is located in a rural environment, and the surrounding land is covered by mixed hardwood of deciduous and coniferous trees, bush and wetland vegetation. Shrubs as well as tall grasses are also common along roadside ditches. A residential subdivision known as “Heritage Lake States” exists approximately 300 m east of the intersection.

Refer to Photographs 1 to 8 given in Appendix A, for general site conditions.

3. FIELD INVESTIGATION PROGRAM

The original field investigation work for Wellington Road 34 Underpass was carried out during the period November 29, 2017 to January 17, 2018. During this time, eight (8) boreholes were drilled at or near the locations of the east and west abutments, the centre pier, and the east and west approaches of the proposed underpass, within the MTO right of way (ROW). A summary of the field program is presented in Table 1. Borehole 35-617-03A was drilled approximately 1.0 m away from Borehole 35-617-03, for the purpose of installing a groundwater monitoring well. As shown in Table 1, the boreholes were advanced to depths ranged from 9.1 m (El. 300.0 m) to 19.8 m (El. 289.2 m) below existing grade,

Table 1 - Field Investigation Program for Proposed Wellington Road 34 Underpass

BOREHOLE ID	BOREHOLE LOCATION	MTM NAD 83 COORDINATES		GROUND SURFACE ELEVATION (m)	BOREHOLE DEPTH (m)
		NORTHING (m)	EASTING (m)		
35-617-01	West Approach	4 813 260.6	249 692.1	310.3	9.8
35-617-02	West Abutment	4 813 268.5	249 696.4	309.0	19.8
35-617-03	West Abutment	4 813 252.4	249 699.1	309.1	13.4
35-617-03A	West Abutment	4 813 254.4	249 700.0	309.1	9.1
35-617-04	Centre Pier	4 813 289.0	249 721.7	311.0	13.3
35-617-05	Centre Pier	4 813 247.8	249 734.0	311.0	18.2
35-617-06	East Abutment	4 813 287.4	249 756.7	308.7	17.4
35-617-07	East Abutment	4 813 255.2	249 765.1	308.8	11.6
35-617-08	East Approach	4 813 273.1	249 774.5	309.5	9.8
35-617-09	Centre Pier	4 813 257.1	249 721.4	311.2	18.3
35-617-10	East Abutment	4 813 257.5	249 767.4	309.3	14.6

In addition to the eight (8) boreholes which were part of the original scope, two (2) boreholes (35-617-09 and 35-617-10) were drilled between July 26, 2021 and July 28, 2021, at the proposed locations of the centre pier and the south side of the east abutment, to facilitate the assessment of the artesian conditions encountered during the previous investigation and to verify the depth of bedrock surface. The original plan was to drill borehole 35-617-09 on the north side of the centre pier. However, due to utility conflicts, the borehole was drilled on the south side of the proposed location of the centre pier.

The borehole location plan and updated soil strata profile are presented in Drawing 35-617-2. The record of borehole sheets for all boreholes are provided in Appendix B.

Prior to commencement of fieldwork for both the original and additional investigations, utility clearance procedures were implemented through Ontario One Call protocol and by contacting MTO locates. Fieldwork notification was also sent to MTO West Region. Further, project specific health and safety as well as traffic control plans were prepared and the borehole locations were marked by PML staff prior to commencement of the fieldwork. All drilling activities, soil sampling and logging, and backfilling of boreholes were conducted under the fulltime supervision of PML staff.

The boreholes for both the original and additional field investigation programs were advanced using truck-mounted and track-mounted drill rigs, equipped with continuous flight hollow stem augers, supplemented by wash boring (mud rotary) techniques. For the first field program carried out in 2017, the drilling equipment was owned and operated by a specialist contractor, Landshark Drilling Inc., based in Brantford, 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.

Artesian conditions were encountered in Boreholes 35-617-06 and 35-617-10 drilled on the east side of the intersection during the original and subsequent field investigation programs. Where artesian conditions and a flowing well were encountered, the artesian flows were sealed at the sources. The groundwater data, details of the artesian conditions, and the sealing operations where applicable, are provided in the record of borehole sheets.

Boreholes were backfilled with soil cuttings and flowing wells were decommissioned in accordance with MTO guidelines and the requirements of Ministry of Environment, Conservation and Parks (MECP) Ontario Regulation 903 (as amended by Ontario Regulation 372). In the case of wells used to take 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 carried out surveying of the borehole locations, and provided coordinates and ground surface elevations. For the additional field program completed in 2021, surveying was conducted 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. <u>LABORATORY TESTS</u>

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 tests (All retrieved samples,70)
- Grainsize distribution analyses (24)
- Atterberg limit tests (3)

The laboratory testing work for the additional (2021) foundation investigation program included twenty-six (26) moisture content tests, nine (9) grainsize analyses, and two (2) Atterberg limit tests.

The laboratory tests to determine soil index properties were performed in accordance with MTO test procedures, based on American Society for Testing Materials (ASTM) standards, with the exception of hydrometer tests which were tested in accordance with MTO's procedure (LS-702). The grain size distribution test results are presented in Figures 617-GS-1, 617-GS-2A, 617-GS-2B and 617-GS-2C. The Atterberg Limit test results are provided in Figure 617-PC-1, in Appendix C. All laboratory test results are included in the record of borehole sheets, provided in Appendix B.

In addition to soil index properties, three (3) soil samples were shipped to AGAT Laboratories of Mississauga, Ontario and SGS Canada Inc. of Mississauga, Ontario for corrosivity chemical testing, including the determination of sulphate, sulphide, and chloride contents, as well as pH value and resistivity, required to determine the corrosivity characteristics of soils at structure location.

Bedrock was encountered in some of the boreholes drilled in 2017 and 2021. The laboratory tests conducted on bedrock samples included unconfined compression strength (UCS) testing conducted on a core sample taken from Borehole 35-617-05, at a depth of 15.6 m to 15.9 m. UCS tests were also carried out on rock cores taken from Boreholes 35-617-09, at a depth of 16.9 m to 17.2 m, and 35-617-10, at a depth of 14.1 m to 14.4 m. The results of the UCS 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 field investigations carried out in 2017 and 2021 along with the field and laboratory test results are presented in the record of borehole sheets. The borehole locations and updated stratigraphic profile are provided in Drawing 35-617-2. The boundaries between soil strata on this profile have been established at borehole locations only and using non-continuous soil sampling, and represent a transition from one soil type to another, and should not be inferred to characterise an exact plane of geological or stratigraphical change, as the subsurface and groundwater conditions may vary between and beyond the boreholes.

In general, the stratigraphy at the project site (proposed location of Wellington Road 34 Underpass) is composed of topsoil/peat/pavement material underlain by a layer of fill, about 0.4 m to 2.3 m thick, consisting of sandy silt or silty sand. Beneath the fill, a sandy silt to silty sand native (till) material with varying proportions of gravel was encountered, over Dolostone Bedrock.

The following sections provide brief descriptions of the three (3) soil layers, the pavement structure material and the bedrock, as well as the groundwater conditions.

5.2.1 Topsoil/Peat

A layer of topsoil was encountered immediately below the existing ground surface in three (3) of the boreholes located off-road (Boreholes 35-617-06, 35-617-07, and 35-617-08). The thickness of the topsoil was in the range of 100 mm to 300 mm.

In Borehole 35-617-10, a layer of peat was encountered below the ground surface. The peat was dark brown in color and wet. In the upper part, the peat consisted of undecomposed small pieces of wood and rootlets mixed with trace to some sand and gravel. In the lower part, the peat became amorphous and spongy and was very wet and soft. The thickness of the peat was 2.3 m.

5.2.2 Pavement Structure Material

Pavement structure material was encountered in three (3) boreholes located within the median of Highway 6 and at the centre of the intersection (Boreholes 35-617-04, 35-617-05, and 35-617-09). The pavement structure material consisted of a layer of asphaltic concrete, approximately 100 mm thick, over a granular layer of silty sand with gravel, ranging in thickness from 0.6 m to 1.4 m.

The SPT 'N' values in the pavement granular material ranged from 61 blows/300 mm to 87 blows/300 mm penetration, indicating a very dense state of compactness.

The moisture contents of the samples tested from the pavement granular fill material ranged from 3.7% to 9.2% with an average value of 6.5%.

5.2.3 Fill – Sandy Silt/Silty Sand

Fill consisting of sandy silt/silty sand with varying proportions of gravel was encountered below the existing ground surface in boreholes located at the west approach and west abutment (Boreholes 35-617-01, 35-617-02 and 35-617-03). In boreholes drilled at the east approach and east abutment (Boreholes 35-617-06, 35-617-07, 35-617-08 and 35-617-10), the fill was encountered beneath the topsoil/peat. In boreholes drilled at the proposed location of the centre pier (Boreholes 35-617-04, 35-617-05 and 35-617-09), the fill was encountered beneath the pavement structure material. The thickness of the fill ranged from 0. 7 m to 2.3 m, extending to the depths of El. 308.4 to El. 306.3.

The SPT 'N' values within this layer ranged from "penetration under the weight of the hammer and rods" (WH) to 82 blows/300 mm, indicating a loose to very dense state of compactness.

The moisture contents of the samples tested from this fill material ranged from 5.7% to 81.9%.

The results of the grain size distribution analyses conducted on representative samples from this fill material are provided in Figure 617-GS-1, in Appendix C.

5.2.4 Sandy Silt to Silty Sand (Till)

A deposit of sandy silt to silty sand till with occasional silt seams, sand lenses, and gravel and cobble inclusions was encountered immediately below the fill material in all of the sampled boreholes. This till deposit extended to depths varying from 9.8 m to 16.8 m (El. 300.5 to El. 292.2) below the existing ground surface, in boreholes where this layer was fully penetrated.

The SPT 'N' values in the till deposit ranged from 1 blow/300 mm to spoon refusal (over 100 blows/300 mm penetration), indicating a very loose to very dense state of compactness.

Moisture contents of representative samples taken from the till deposit ranged from 7.7% to 29.0%, with an average value of 16.3%.

The results of the grainsize distribution analyses conducted on samples from this deposit are provided in Figures 617-GS-2A, 617-GS-2B and 617-GS-2C, in Appendix C. The Atterberg limit test results for samples from the silt seams are presented in Figure 617-PC-1, in Appendix C.

5.2.5 Dolostone Bedrock

Bedrock was encountered in five (5) boreholes (35-617-02, 35-617-05, 35-617-06, 35-617-09, and 35-617-10) and at depths ranging from 11.5 m to 16.8 m (El. 297.8 to El. 292.2) below the existing ground surface. The presence of bedrock in these boreholes was confirmed by obtaining 3.0 m to 3.3 m long rock cores using NQ and HQ size, double core barrels. The rock core recoveries (RC) from all five (5) boreholes ranged from 79% to 100%. The Rock Quality Designation (RQD) of the rock cores ranged from 52% to 100%, indicating fair to excellent quality bedrock. The low RQD values were obtained for cores taken from the upper part of the bedrock in Borehole 35-617-06. Further, the RC and RQD values of Sample 14 (Run 1) in Borehole 35-617-10 were 39% and 21%, respectively, indicating the presence of very poor rock mass quality at that particular depth.

The bedrock was identified as slightly to moderately weathered, highly fractured dolostone/dolomite belonging to the Guelph Formation. For complete descriptions of the bedrock, refer to photographs, the rock core description logs, and laboratory test results, provided in Appendix D.

The unconfined compressive strength (UCS) tests conducted on rock cores taken from Boreholes 35-617-05, 35-617-09 and 35-617-10 yielded UCS values ranging from 81.8 MPa to 165.1 MPa. Based on these values, the bedrock may be classified as strong to very strong with respect to the strength of intact rock specimens. Detailed results of the UCS tests are provided in Appendix D.

5.3 Groundwater Conditions

Groundwater was observed in all boreholes during and immediately after drilling, at depths varying from near the ground surface to 4.8 m (El. 309.0 to El. 304.3) below the existing grade. Groundwater level measurements were also taken from two (2) monitoring wells (35-617-03A and 35-617-09) installed during the original field program in 2017 and recently in 2021, respectively. A summary of groundwater level readings taken after the installation of monitoring wells is given in Table 2.

Table 2 - Groundwater Monitoring Well Readings

<table>
<tr><th>BOREHOLE</th><th>GROUND SURFACE ELEVATION</th><th>WELL INSTALLATION DATE</th><th>WELL SCREEN DEPTH (m)</th><th>DATE</th><th>DEPTH (m)</th><th>ELEVATION (m)</th></tr>
<tr><td rowspan="8">35-617-03A</td><td rowspan="8">309.1</td><td rowspan="8">11/12/2017</td><td rowspan="8">7.6 – 9.1</td><td>January 09, 2018</td><td>0.7</td><td>308.4</td></tr>
<tr><td>January 17, 2018</td><td>0.7</td><td>308.4</td></tr>
<tr><td>February 15, 2018</td><td>0.7</td><td>308.4</td></tr>
<tr><td>April 03, 2018</td><td>0.7</td><td>308.4</td></tr>
<tr><td>July 30, 2021</td><td>0.8</td><td>308.3</td></tr>
<tr><td>August 16, 2021</td><td>1.0</td><td>308.1</td></tr>
<tr><td>August 19, 2021</td><td>1.0</td><td>308.1</td></tr>
<tr><td>August 24, 2021</td><td>0.9</td><td>308.2</td></tr>
<tr><td rowspan="3">35-617-06</td><td rowspan="3">308.7</td><td rowspan="3">15/12/2017</td><td rowspan="3">9.9 – 11.5</td><td>January 9, 2018</td><td colspan="2">Frozen</td></tr>
<tr><td>January 12, 2018</td><td>-1.2 (Flowing)</td><td>309.9</td></tr>
<tr><td>February 2, 2018</td><td colspan="2">Decommissioned</td></tr>
<tr><td rowspan="2">35-617-09</td><td rowspan="2">311.2</td><td rowspan="2">26/07/2021</td><td rowspan="2">8.5 – 10.1</td><td>28/07/2021</td><td>2.6</td><td>308.7</td></tr>
<tr><td>11/08/2021</td><td>2.6</td><td>308.6</td></tr>
</table>

In a well installed in Borehole 35-617-03A, the stabilized groundwater level in 2017 was recorded to be at El. 308.4, four months after the completion of drilling. Recent measurements in the same well showed a groundwater level at El. 308.1. In addition, the groundwater level measured in a well installed in 2021 at the proposed location of the centre pier (35-617-09) was at El. 308.6.

Artesian conditions were encountered in the monitoring well installed in Borehole 35-617-06 in 2017, located on the north side of the proposed east abutment. The artesian head stabilised at approximately 1.2 m above the ground surface (El. 309.9). Artesian conditions were also encountered in Borehole 35-617-07 drilled in 2017 and in Borehole 35-617-10 drilled in 2021. The artesian head measured in Borehole 35-617-10 was approximately 1.4 m (El. 310.7) above the ditch surface. Monitoring wells were not installed in Boreholes 35-617-07 and 35-671-10 and the boreholes were backfilled with cement grout immediately after encountering artesian conditions.

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

5.4 Soil Chemical Test Results

A summary of the corrosivity test results conducted on sandy silt to silty sand sample is provided in Table 3. The samples were taken from Boreholes 35-617-02, 35-617-09 and 35-617-10. The details of the soil chemical test results and a description of the test method are provided in Appendix E.

Table 3 - A Summary of Corrosivity Test Results

BOREHOLE NO.	ELEVATION (m)	SOIL TYPE	SULPHIDE (%)	SULPHATE (μg/g, ppm)	CHLORIDE (μg/g, ppm)	pH	RESISTIVITY (Ohm-cm)
35-617-02	304.2	Sandy Silt Silty Sand Till	0.14	119	527	7.71	862
35-617-09	307.8		<0.04	23	490	8.82	358
35-617-10	306.8	Silty Sand or Sandy Silt Fill	0.85	570	2300	7.53	358

6. CLOSURE

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

Yours very truly

Peto MacCallum Ltd.

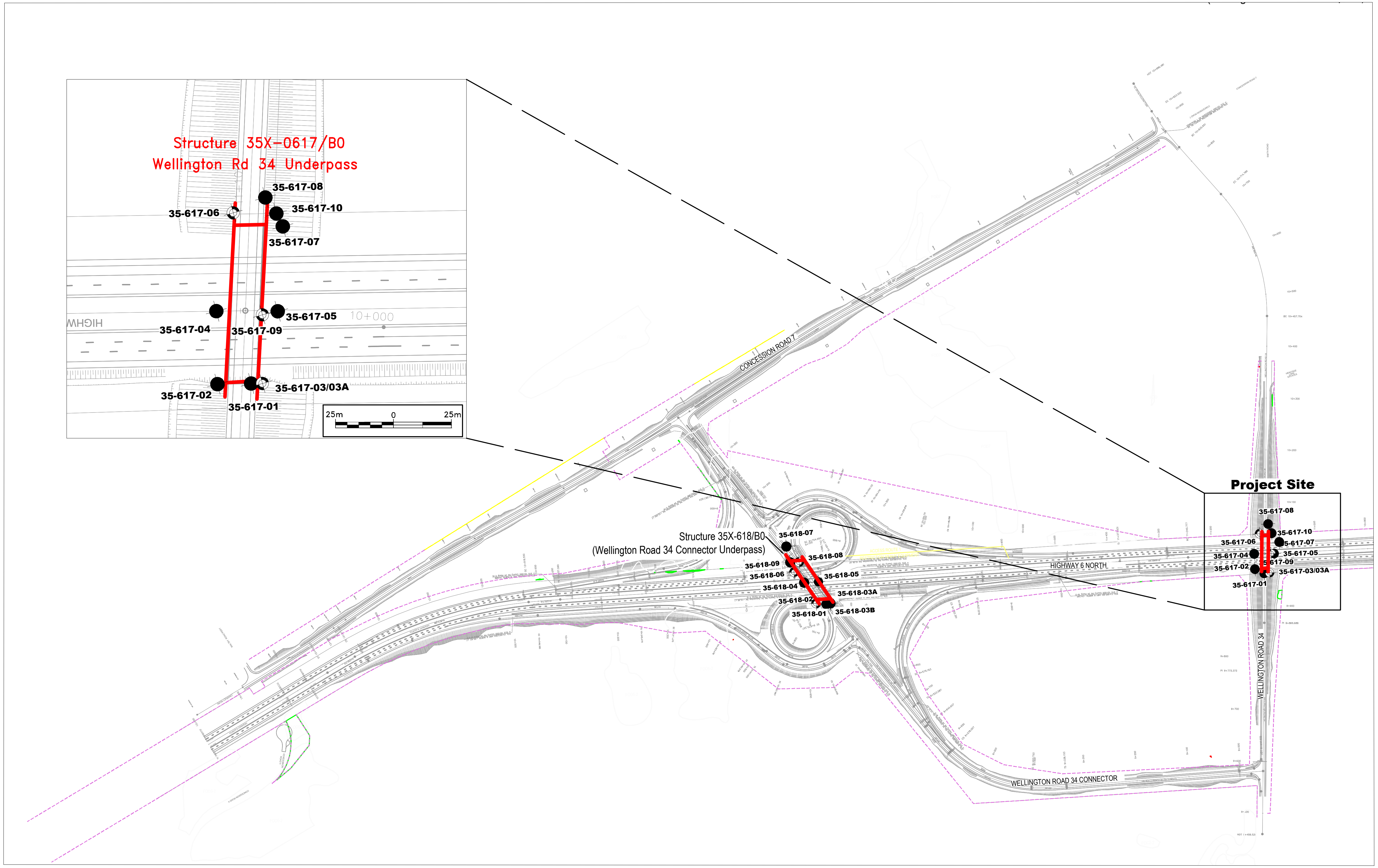
LICENSED PROFESSIONAL ENGINEER
L. A. YIMAM
100500748
Oct. 14/2021
PROVINCE OF ONTARIO

Lul Yimam, PhD, P.Eng.
Senior Engineer, Geotechnical Services

LICENSED PROFESSIONAL ENGINEER
G. G. UWIMANA
October 14, 2021
PROVINCE OF ONTARIO

Geoffrey Uwimana, MEng., P.Eng.
Discipline Head, Geotechnical Services
MTO Designated Principal Contact

LY/GU/:ly-nk



PLAN SCALE

50m 0 50m 100m 200m

GWP No 3059-20-00

Design-Build Ready Alternative Bid Package

Midblock Interchange Area

Hwy 401& Hwy 6 Improvements

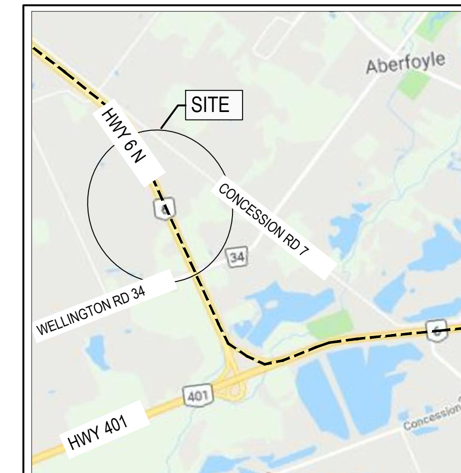
From Hamilton North Limits to Guelph South limits

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)
35-617-01	4813256.6	249692.1	310.3	9.8
35-617-02	4813273.5	249693.4	309.0	19.8
35-617-03	4813252.4	249699.1	309.1	13.4
35-617-03A	4813254.4	249700.0	309.1	9.1
35-617-04	4813289.0	249721.7	311.0	13.3
35-617-05	4813247.8	249734.0	311.0	18.2
35-617-06	4813287.4	249756.7	308.7	17.4
35-617-07	4813255.2	249765.1	308.8	11.6
35-617-08	4813273.1	249774.5	309.5	9.8
35-617-09	4813257.1	249721.4	311.2	18.3
35-617-10	4813257.5	249767.4	309.3	14.6

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.

LICENSED PROFESSIONAL ENGINEER
L. A. YIMAM
100500748
Oct 14/2021
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
G. G. UWIMANA
October 14, 2021
PROVINCE OF ONTARIO

– 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. 40P8-292

HWY No 6			DIST 31
SUBM'D LY	CHECKED LY	DATE OCTOBER 14, 2021	SITE MBI Area
DRAWN FM	CHECKED GU	APPROVED GU	DWG 35-617-1

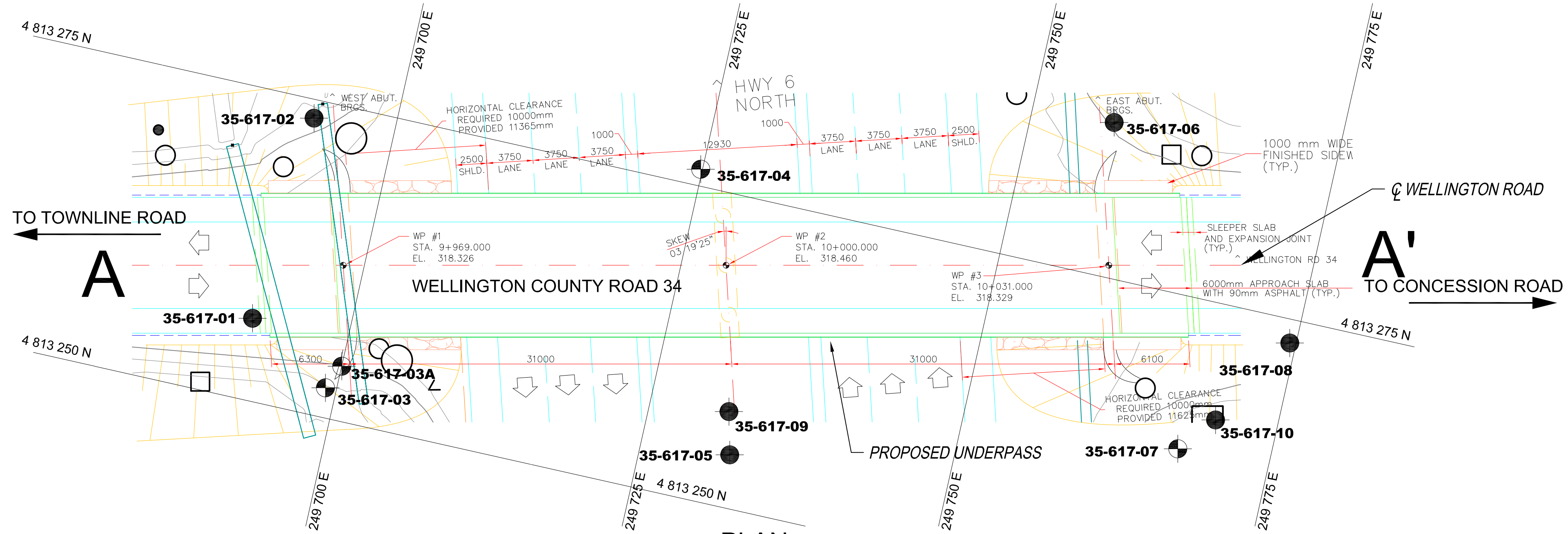
Reference AECOM Drawing: 01_Wellington Rd._GA.dwg, dated Aug. 2017

GWP No 3059-20-00
Design- Build Ready Alternative Bid Package
Wellington Road 34 Underpass
SITE No. 35X - 0617/B0 STA No. 10+000
Long.: 43.457324, Lat.: -80.180.489

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

SHEET

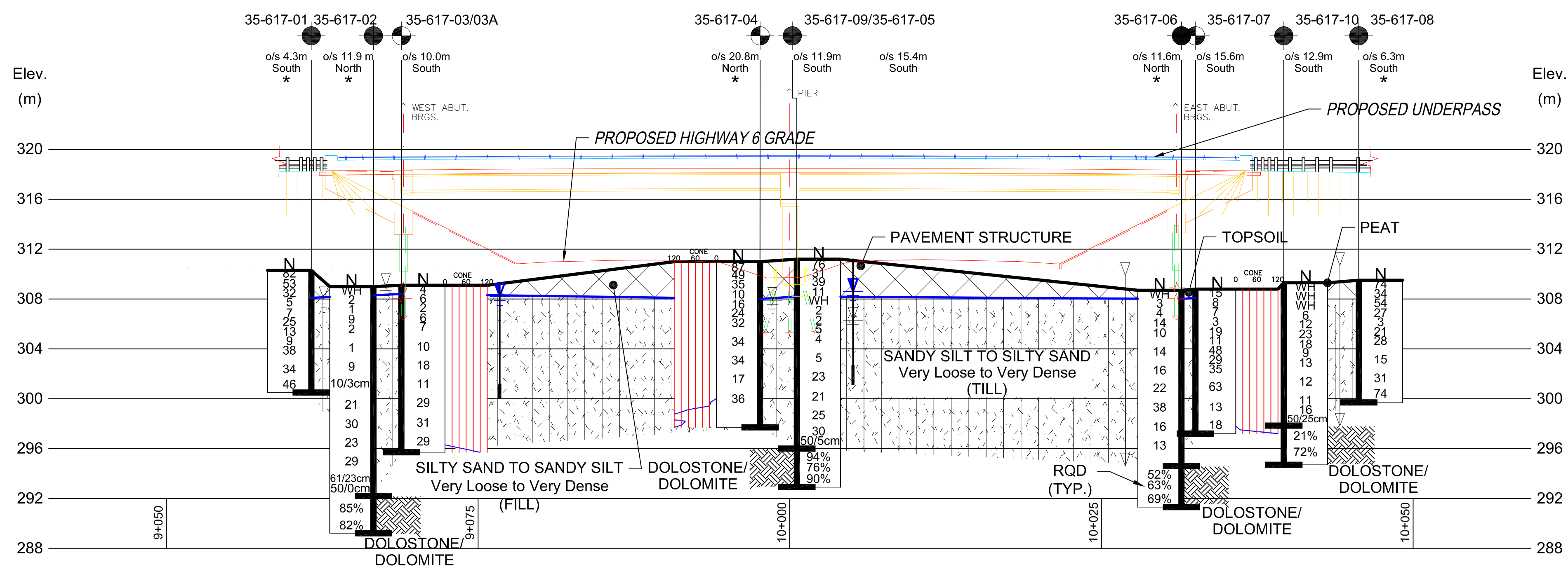
PML Peto MacCallum Ltd.
CONSULTING ENGINEERS



PLAN

SCALE

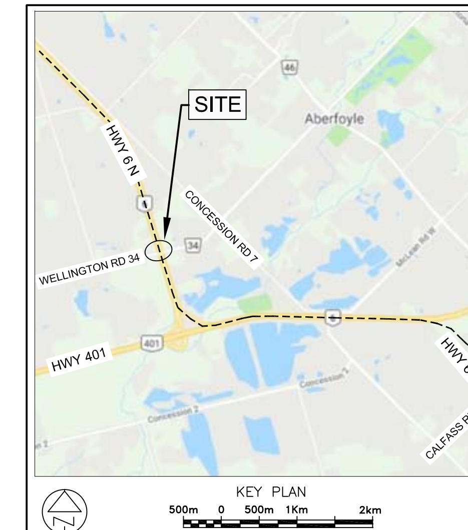
10 0 10 20m



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

SCALE

10 0 10 20m



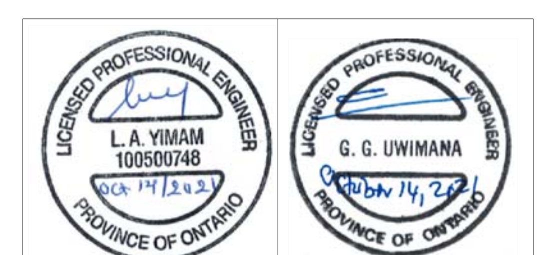
KEY PLAN

500m 0 500m 1Km 2km

LEGEND

- Borehole
- Borehole + DCPT
- CONE Blows/0.3m (60 Cone, 475 J/blow)
- N Blows/0.3m (Std. Pen Test, 475 J/blow)
- RQD Rock Quality Designation
- * Water level could not be established upon completion of drilling
- WL in Piezometer
- WL observed during drilling
- BH With artesian condition
- Piezometer

BH No	ELEVATION	CO-ORDINATES (MTM ON10)	
		NORTHINGS	EASTINGS
35-617-01	310.3	4 813 256.6	249 692.1
35-617-02	309.0	4 813 273.5	249 693.4
35-617-03	309.1	4 813 252.4	249 699.1
35-617-03A	309.1	4 813 254.4	249 700.0
35-617-04	311.0	4 813 289.0	249 721.7
35-617-05	311.0	4 813 247.8	249 734.0
35-617-06	308.7	4 813 287.4	249 756.7
35-617-07	308.8	4 813 255.2	249 765.1
35-617-08	309.5	4 813 273.1	249 774.5
35-617-09	311.2	4 813 257.7	249 721.4
35-617-10	309.3	4 813 257.5	249 767.4



– 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. 40P8-292

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

HWY No 401			DIST 31
SUBM'D NL	CHECKED LY	DATE OCTOBER 14, 2021	SITE 35X - 0617/B0
DRAWN NL	CHECKED	APPROVED GU	DWG 35-617-2

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.

APPENDIX A

Site Photographs

Photograph 1 - The Intersection of Wellington Road 34 and Highway 6, Looking North

Photograph 2 - The Intersection of Wellington Road 34 and Highway 6, Looking West

Photograph 3 - Borehole 35-617-09, Looking South

Photograph 4 - Borehole (Monitoring Well) 35-617-09, Looking North

Photograph 5 - Borehole 35-617-10, Looking West

Photograph 6 - Borehole 35-617-10, Looking East

Photograph 7 - Measuring Artesian Head in a Riser Pipe at Borehole 35-617-10

Photograph 8 - The Groundwater Level inside the Riser Pipe at Borehole 35-617-10

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/0.3 m)	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 35-617-01

METRIC

G.W.P. 3059-20-00 | LOCATION Coords: 4 813 256.6 N; 249 692.1 E (MTM ON10) | ORIGINATED BY M.F.

DIST 31 HWY 6 | BOREHOLE TYPE Hollow Stem Augers | COMPILED BY L.Y.

DATUM Geodetic | DATE 2017.12.21 | LATITUDE 43.457238 | LONGITUDE -80.180900 | CHECKED BY G.U.

ELEV / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
310.3	GROUND SURFACE					
0.0	SILTY SAND TO SAND, some gravel Very dense, Brown, Moist (FILL)	1	SS	82		
		2	SS	53		23 64 12 1
		3	SS	32		
308.1 / 2.2	SANDY SILT TO SILTY SAND, trace/some gravel Loose to compact, Brown, Wet (TILL)	4	SS	5	∇ (approx. 308.0)	
		5	SS	7		
		6	SS	25		
		7	SS	13		6 39 52 3
		8	SS	9		
	dense	9	SS	38		
		10	SS	34		
		11	SS	46		19 72 8 1
300.5 / 9.8	End of borehole					

Column headings: 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; ELEVATION SCALE 310–301.

∇ Groundwater level observed during drilling

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

RECORD OF BOREHOLE No 35-617-02

METRIC

G.W.P. 3059-20-00	LOCATION Coords: 4 813 273.5 N; 249 693.4 E (MTM ON10)		ORIGINATED BY M.Kh./S.A.
DIST 31 HWY 6	BOREHOLE TYPE Hollow stem Augers, Wash Boring, NQ Rock Coring		COMPILED BY L.Y.
DATUM Geodetic	DATE 2017.11.29 - 2017.12.04	LATITUDE 43.457271 LONGITUDE -80.180239	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
309.0	GROUND SURFACE
0.0	SILTY SAND Very loose, Brown, Wet (FILL)
308.3 0.7	SANDY SILT TO SILTY SAND, some gravel Very loose, Grey, Wet to moist (TILL) Silt, trace sand occasional cobbles compact
294.0	

NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
1	SS	WH				
2	SS	2				
3	SS	1				
4	SS	9				
5	SS	2				
6	SS	1				
7	SS	9				
8	SS	10/3cm				
9	SS	21	22	72	(6)	
10	SS	30				
11	SS	23	19	31	43	7
12	SS	29				

Elevation scale: 308, 307, 306, 305, 304, 303, 302, 301, 300, 299, 298, 297, 296, 295

Continued Next Page

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

RECORD OF BOREHOLE No 35-617-02

METRIC

G.W.P. 3059-20-00 LOCATION Coords: 4 813 273.5 N; 249 693.4 E (MTM ON10) ORIGINATED BY M.Kh./S.A.

DIST 31 HWY 6 BOREHOLE TYPE Hollow stem Augers, Wash Boring, NQ Rock Coring COMPILED BY L.Y.

DATUM Geodetic DATE 2017.11.29 - 2017.12.04 LATITUDE 43.457271 LONGITUDE -80.180239 CHECKED BY G.U.

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
294.0 15.0	SANDY SILT TO SILTY SAND, with gravel Very dense, Grey, Wet to moist (TILL)	13	SS	61/23cm			○ (~13)		
						293			
292.2		14	SS	50/0cm					
292.2 16.8	DOLOSTONE/DOLOMITE Slightly weathered					292			
		RUN 1	RC NQ	REC 99%					RQD 85%
						291			
	Slightly to moderately weathered	RUN 2	RC NQ	REC 99%		290			RQD 82%
289.2 19.8	End of borehole								

WH Split spoon penetration due to weight of hammer and rods

∇ Groundwater level observed during drilling

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.

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.

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

RECORD OF BOREHOLE No 35-617-03

METRIC

G.W.P. 3059-20-00 LOCATION Coords: 4 813 252.4 N; 249 699.1 E (MTM ON10) ORIGINATED BY S.A.

DIST 31 HWY 6 BOREHOLE TYPE Hollow Stem Augers, Wash Boring, Cone Penetration Test COMPILED BY L.Y.

DATUM Geodetic DATE 2017.12.06 LATITUDE 43.457271 LONGITUDE -80.180239 CHECKED BY G.U.

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
309.1	GROUND SURFACE						
0.0	SANDY SILT, trace/with gravel, cobbles Loose, Brown, Moist (FILL)	1	AS	4			
308.4 0.7	SANDY SILT TO SILTY SAND, some/with gravel Very loose to loose, Grey, Moist (TILL)	2	SS	6			
		3	SS	2			
		4	SS	6			
	Silt	5	SS	7			
		6	SS	10	∇		
	compact						
		7	SS	18			
	occasional cobbles	8	SS	11			31 61 7 1
		9	SS	29			
		10	SS	31			54 23 18 5
		11	SS	29			
296.3 12.8	End of borehole Start of cone penetration test						
295.7 13.4	End of cone penetration test Refusal on probable bedrock						

∇ Groundwater level observed during drilling

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100; 100/5cm

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

ELEVATION SCALE: 309 308 307 306 305 304 303 302 301 300 299 298 297 296

Continued Next Page

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

RECORD OF BOREHOLE No 35-617-03

METRIC

G.W.P. 3059-20-00 LOCATION Coords: 4 813 252.4 N; 249 699.1 E (MTM ON10) ORIGINATED BY S.A.

DIST 31 HWY 6 BOREHOLE TYPE Hollow Stem Augers, Wash Boring, Cone Penetration Test COMPILED BY L.Y.

DATUM Geodetic DATE 2017.12.06 LATITUDE 43.457271 LONGITUDE -80.180239 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						
294.1	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.										

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

RECORD OF BOREHOLE No 35-617-03A

METRIC

G.W.P. 3059-20-00 LOCATION Coords: 4 813 254.4 N; 249 700.0 E (MTM ON10) ORIGINATED BY S.A.

DIST 31 HWY 6 BOREHOLE TYPE Hollow Stem Augers COMPILED BY L.Y.

DATUM Geodetic DATE 2017.12.11 LATITUDE 43.457271 LONGITUDE -80.180239 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						
309.1	GROUND SURFACE										
0.0	Borehole 617-3A was drilled 1.0m south of Borehole 617-3 Augered to 9.1m depth without sampling (Refer to Borehole 35-617-03 for stratigraphic details)						309 308 307 306 305 304 303 302 301				
300.0 9.1	End of borehole						300				

Groundwater level measured in monitoring well

Monitoring Well Readings:

Date	Depth (m)	Elev.
Jan. 09/'18	0.7	308.4
Jan. 17/'18	0.7	308.4
Feb. 15/'18	0.7	308.4
Apr. 03/'18	0.7	308.4
Jul. 30/'21	0.8	308.3
Aug. 16/'21	1.0	308.1
Aug. 19/'21	1.0	308.1
Aug. 24/'21	0.9	308.2

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 35-617-04

METRIC

G.W.P. 3059-20-00	LOCATION Coords: 4 813 289.0 N; 249 721.7 E (MTM ON10)		ORIGINATED BY S.A.
DIST 31 HWY 6	BOREHOLE TYPE Hollow Stem Augers, Wash Boring, Cone Penetration test		COMPILED BY L.Y.
DATUM Geodetic	DATE 2018.01.17	LATITUDE 43.457271 LONGITUDE -80.180239	CHECKED BY G.U.

SOIL PROFILE

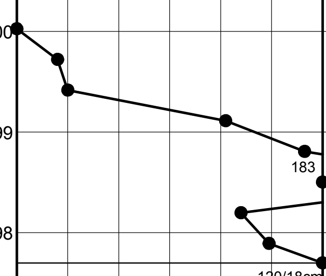
ELEV DEPTH	DESCRIPTION
311.0	GROUND SURFACE
310.9 / 0.1	100 mm ASPHALT over sand and gravel (PAVEMENT FILL)
310.3 / 0.7	SILTY SAND, trace/with gravel. Very dense to compact, Brown/grey, Moist (FILL)
308.0 / 3.0	SANDY SILT TO SILTY SAND, trace/some gravel. Compact to dense, Grey, Wet (TILL)
299.7 / 11.3	End of borehole. Start of cone penetration test
297.7 / 13.3	End of cone penetration test

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
1	SS	87				
2	SS	49				
3	SS	35				
4	SS	10				
5	SS	16				
6	SS	24				
7	SS	32	2	94	4	0
8	SS	34	0	55	41	4
9	SS	34				
10	SS	17	0	11	76	13
11	SS	36	18	71	9	2

Ground water conditions: ∇ at elev. 308.0

Column headings: GROUND WATER CONDITIONS; ELEVATION SCALE (310, 309, 308, 307, 306, 305, 304, 303, 302, 301, 300, 299, 298); 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



∇ Groundwater level observed during drilling

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

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

Ministry of Transportation Ontario

Foundation Design

RECORD OF BOREHOLE No 35-617-05

METRIC

G.W.P. 3059-20-00	LOCATION Coords: 4 813 247.8 N; 249 734.0 E (MTM ON10)		ORIGINATED BY S.A.
DIST 31 HWY 6	BOREHOLE TYPE Hollow Stem Augers, Wash Boring, NQ Rock Coring		COMPILED BY L.Y.
DATUM Geodetic	DATE 2018.01.19 - 2018.01.26	LATITUDE 43.457252 LONGITUDE -80.180943	CHECKED BY G.U.

Chart headings: 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	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	GR	SA	SI	CL
311.0	GROUND SURFACE									
310.9 / 0.1	100mm ASPHALT over silty sand, with gravel (PAVEMENT FILL)	1	SS	61						
		2	SS	67		310				
309.5 / 1.5	SILTY SAND, trace/with gravel Compact to very dense, Grey, Moist to wet (FILL)	3	SS	11			35	45	(20)	
						309				
		4	SS	61						
		5	SS	36	∇	308				
307.4 / 3.6	SILTY SAND, with gravel Very dense to compact, Grey, Moist to wet (TILL)						4	63	29	4
						307				
		6	SS	10		306				
	Clayey silt, some sand					305				
		7	SS	78/28cm			0	6	85	9
						304				
		8	SS	50/13cm		303				
		9	SS	50/8cm		302				
						301				
		10	SS	59		300	70	27	(3)	
						299				
		11	SS	18		298				
		12	SS	50/10cm		297				

Continued Next Page

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

RECORD OF BOREHOLE No 35-617-05

METRIC

G.W.P. 3059-20-00 | LOCATION Coords: 4 813 247.8 N; 249 734.0 E (MTM ON10) | ORIGINATED BY S.A.

DIST 31 HWY 6 | BOREHOLE TYPE Hollow Stem Augers, Wash Boring, NQ Rock Coring | COMPILED BY L.Y.

DATUM Geodetic | DATE 2018.01.19 - 2018.01.26 | LATITUDE 43.457252 | LONGITUDE -80.180943 | CHECKED BY G.U.

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
296.0										
295.8 15.2	DOLOSTONE/DOLOMITE Unweathered	RUN 1	RC NQ	REC 100%						RQD 100% UCS=81.8 (MPa)
						295				
		RUN 2	RC NQ	REC 100%						RQD 72%
						294				
		RUN 3	RC NQ	REC 95%						RQD 89%
						293				
292.8 18.2	End of borehole									

Legend: ○ UNCONFINED; + FIELD VANE; ● QUICK TRIAXIAL; × LAB VANE; PLASTIC LIMIT w_P; NATURAL MOISTURE CONTENT w; LIQUID LIMIT w_L

∇ Groundwater level observed during drilling

NOTES:

1. Borehole was charged with drilling water thus water level could not be established upon completion of drilling.
2. Upon borehole decomission, free water was observed at the existing ground surface.

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

RECORD OF BOREHOLE No 35-617-06

METRIC

G.W.P. 3059-20-00 LOCATION Coords: 4 813 287.4 N; 249 756.7 E (MTM ON10) ORIGINATED BY S.A.

DIST 31 HWY 6 BOREHOLE TYPE Hollow stem Augers, Wash Boring, NQ Rock Coring COMPILED BY L.Y.

DATUM Geodetic DATE 2017.12.14 - 2017.12.15 LATITUDE 43.457271 LONGITUDE -80.180239 CHECKED BY G.U.

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION
308.7	GROUND SURFACE
0.0	TOPSOIL
308.4 / 0.3	SILTY SAND, trace/with gravel Loose, Brown/grey, Wet (FILL)
308.0 / 0.7	SILTY SAND, trace/some gravel Loose to compact, Brown/grey, Wet (TILL)
294.6 / 14.1	DOLOSTONE/DOLOMITE Slightly/moderately weathered
293.7	

SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	WH	
2	SS	3	
3	SS	4	
4	SS	14	6 72 19 3
5	SS	10	
6	SS	14	
7	SS	16	6 71 20 3
8	SS	22	
9	SS	38	19 41 37 3
10	SS	16	
11	SS	13	
RUN 1	RC NQ	REC 85%	RQD 52%

Column headings: STRAT PLOT; GROUND WATER CONDITIONS; ELEVATION SCALE (308, 307, 306, 305, 304, 303, 302, 301, 300, 299, 298, 297, 296, 295, 294); 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.

Continued Next Page

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

Ministry of Transportation Ontario

RECORD OF BOREHOLE No 35-617-06

METRIC

G.W.P. 3059-20-00 | LOCATION Coords: 4 813 287.4 N; 249 756.7 E (MTM ON10) | ORIGINATED BY S.A.

DIST 31 HWY 6 | BOREHOLE TYPE Hollow stem Augers, Wash Boring, NQ Rock Coring | COMPILED BY L.Y.

DATUM Geodetic | DATE 2017.12.14 - 2017.12.15 | LATITUDE 43.457271 | LONGITUDE -80.180239 | CHECKED BY G.U.

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%)
293.7						
15.0	DOLOSTONE/DOLOMITE Slightly/moderately weathered	RUN 2	RC NQ	REC 91%		RQD 63%
					293	
		RUN 3	RC NQ	REC 79%	292	RQD 69%
291.3 17.4	End of borehole					

Column headings: SOIL PROFILE; SAMPLES; STRAT PLOT; 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; GR SA SI CL

WH Split spoon penetration due to weight of hammer and rods

∇ Groundwater level observed during drilling

NOTES:
1. Borehole was charged with drilling water thus water level could not be established upon completion of drilling
2. Artesian water head stabilized approximately at 1.2 m above the existing ground surface. Monitoring well was decommissioned

Monitoring Well Readings:

Date	Depth (m)	Elev.
Jan. 09/'18	Frozen	---
Jan. 12/'18	-1.2 Flowing	309.9
Feb. 02/'18	Well Decommissioned	

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 35-617-07

METRIC

G.W.P. 3059-20-00 LOCATION Coords: 4 813 255.2 N; 249 765.1 E (MTM ON10) ORIGINATED BY M.F.

DIST 31 HWY 6 BOREHOLE TYPE Hollow Stem Augers, Wash Boring COMPILED BY L.Y.

DATUM Geodetic DATE 2017.12.19 - 2017.12.20 LATITUDE 43.457338 LONGITUDE -80.179838 CHECKED BY G.U.

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
308.8	GROUND SURFACE				
0.0	TOPSOIL				
308.5 / 0.3	SANDY SILT, trace/some gravel, Compact, Brown/grey, Wet (FILL)	1	SS	15	
308.1 / 0.7	SANDY SILT TO SILTY SAND, some/with gravel; Loose to compact, Brown/grey, Wet (TILL)	2	SS	8	
		3	SS	7	
		4	SS	3	
		5	SS	19	10 40 48 2
		6	SS	11	
	Silt, dense	7	SS	48	
	compact to dense	8	SS	29	15 75 9 1
		9	SS	35	
		10	SS	63	39 38 21 2
		11	SS	13	
		12	SS	18	
297.7 / 11.1	End of borehole; Start of cone penetration test				
297.2 / 11.6	End of cone penetration test; Refusal on probable bedrock				

Natural moisture content at sample 1: 88.7

Dynamic cone penetration resistance at 297.2: 160

Elevation scale: 308, 307, 306, 305, 304, 303, 302, 301, 300, 299, 298

▽ Groundwater level observed during drilling

Continued Next Page

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

Column headings: 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/m^3; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No 35-617-07

METRIC

G.W.P.	3059-20-00	LOCATION	Coords: 4 813 255.2 N; 249 765.1 E (MTM ON10)	ORIGINATED BY	M.F.
DIST 31	HWY 6	BOREHOLE TYPE	Hollow Stem Augers, Wash Boring	COMPILED BY	L.Y.
DATUM	Geodetic	DATE	2017.12.19 - 2017.12.20 LATITUDE 43.457338 LONGITUDE -80.179838	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						
293.8											
	NOTES: 1. Borehole was charged with drilling water thus water level could not be established upon completion of drilling 2. Artesian conditions were encountered at a depth of about 11.6 m (EL. 297.2) 3. Due to the artesian conditions, the borehole was backfilled with cement grout immediately after drilling was completed.										

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

Ministry of Transportation Ontario

Foundation Design

RECORD OF BOREHOLE No 35-617-08

1 OF 1 **METRIC**

G.W.P. 3059-20-00 LOCATION Coords: 4 813 273.1 N; 249 774.5 E (MTM ON10) ORIGINATED BY M.F.

DIST 31 HWY 6 BOREHOLE TYPE Hollow Stem Augers COMPILED BY L.Y.

DATUM Geodetic DATE 2017.12.18 LATITUDE 43.457338 LONGITUDE -80.179838 CHECKED BY G.U.

ELEV DEPTH	DESCRIPTION (SOIL PROFILE)	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
309.5	GROUND SURFACE					
309.4 0.1	TOPSOIL					
	SILTY SAND, with gravel Very dense to dense, Brown, Moist (FILL)	1	SS	74		
		2	SS	34		28 57 13 2
308.1 1.4	SILTY SAND, trace/some gravel Compact to dense, Brown, Wet (TILL)	3	SS	54		
					∇	
		4	SS	27		
		5	SS	3		20 59 17 4
		6	SS	21		
		7	SS	28		
		8	SS	15		
		9	SS	31		3 79 15 3
		10	SS	74		
299.7 9.8	End of borehole					

ELEVATION SCALE: 309, 308, 307, 306, 305, 304, 303, 302, 301, 300

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

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

RECORD OF BOREHOLE No 35-617-09

METRIC

G.W.P. 3059-20-00 LOCATION COORDS: 4 813 257.1 N; 249 721.4 E (MTM ON10) ORIGINATED BY F.M.

DIST 31 HWY 6 BOREHOLE TYPE Solid Stem Augers, Wash Boring, HQ Coring COMPILED BY L.Y.

DATUM Geodetic DATE 2021.07.26 LATITUDE 43.457205 LONGITUDE -80.180482 CHECKED BY G.U.

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
311.2	GROUND SURFACE				
311.0 / 0.1	100mm ASPHALT, over sand and gravel (PAVEMENT STRUCTURE)	1	SS	76	
310.4 / 0.8	SILTY SAND, trace to some gravel Compact to dense, Grey, Moist to wet (FILL)	2	SS	31	
		3	SS	39	
		4	SS	11	12 43 38 7
308.2 / 3.0	SANDY SILT to SILTY SAND, trace to some gravel Very loose to dense, Grey, Moist to wet (TILL)	5	SS	WH	
		6	SS	2	3 63 (34)
		7	SS	2	
		8	SS	5	
	Silt, loose	9	SS	4	0 21 70 9
		10	SS	5	
		11	SS	23	17 55 (28)
		12	SS	21	
		13	SS	25	
		14	SS	30	13 40 37 10

Elevation scale: 311, 310, 309, 308, 307, 306, 305, 304, 303, 302, 301, 300, 299, 298, 297

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

Continued Next Page

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

RECORD OF BOREHOLE No 35-617-09

METRIC

G.W.P. 3059-20-00 LOCATION COORDS: 4 813 257.1 N; 249 721.4 E (MTM ON10) ORIGINATED BY F.M.

DIST 31 HWY 6 BOREHOLE TYPE Solid Stem Augers, Wash Boring, HQ Coring COMPILED BY L.Y.

DATUM Geodetic DATE 2021.07.26 LATITUDE 43.457205 LONGITUDE -80.180482 CHECKED BY G.U.

ELEV DEPTH	DESCRIPTION (SOIL PROFILE)	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
296.2											
296.0			15	SS	50/5cm						
15.2	DOLOSTONE/DOLOMITE Slightly weathered		RUN 1	RC HQ	REC 100%		296				RQD 94%
			RUN 2	RC HQ	REC 97%		295				RQD 76% UCS=83.6 MPa
			RUN 3	RC HQ	REC 100%		294				RQD 90%
292.9 18.3	End of borehole						293				

Chart scales: 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

WH Split spoon penetration due to weight of hammer and rods

▽ Groundwater level observed during drilling

▼ Groundwater level 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.

Monitoring Well Readings:

Date	Depth (m)	Elev.
Jul. 28/'21	2.6	308.6
Aug. 11/'21	2.6	308.6

Monitoring Well Legend:

- Flush-mount Casing
- Bentonite Seal
- Filter Sand
- Screen

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

Ministry of Transportation Ontario

RECORD OF BOREHOLE No 35-617-10

METRIC

G.W.P. 3059-20-00	LOCATION COORDS: 4 813 257.5 N; 249 767.4 E (MTM ON10)		ORIGINATED BY V.L
DIST 31 HWY 6	BOREHOLE TYPE Solid Stem Augers, Wash Boring, NQ Coring		COMPILED BY L.Y.
DATUM Geodetic	DATE 2021.07.27 - 2021.07.29	LATITUDE 43.457280 LONGITUDE -80.180007	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	NUMBER	TYPE	"N" VALUES	WATER CONTENT (%)	REMARKS (GR SA SI CL)
309.3	GROUND SURFACE					
0.0	PEAT, fine fibrous to amorphous Dark brown, Wet	1	SS	WH		
		2	SS	WH		
		3	SS	WH	124.9	
307.0 / 2.3	SILTY SAND, trace clay, trace gravel, organics Loose, Brown/Grey, Wet (FILL)	4	SS	6	81.9	
306.3 / 3.0	SILTY SAND, trace to some gravel Loose to compact, Brown/grey, Wet (TILL)	5	SS	12		17 79 (4)
		6	SS	23		
		7	SS	18		2 61 33 4
		8	SS	9		
		9	SS	13		
		10	SS	12		1 24 63 12
		11	SS	11		
	crushed cobbles	12	SS	16		42 36 17 5
		13	SS	50/25cm		
297.8 / 11.5	DOLOSTONE/DOLOMITE Slightly to moderately weathered	RUN 1	RC NQ	REC 39%		RQD 21%
		RUN 2	RC NQ	REC 100%		RQD 72% UCS=165.1 MPa
294.7 / 14.6	End of borehole					

Elevation scale: 309, 308, 307, 306, 305, 304, 303, 302, 301, 300, 299, 298, 297, 296, 295

Continued Next Page

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

RECORD OF BOREHOLE No 35-617-10

METRIC

G.W.P. 3059-20-00 LOCATION COORDS: 4 813 257.5 N; 249 767.4 E (MTM ON10) ORIGINATED BY V.L

DIST 31 HWY 6 BOREHOLE TYPE Solid Stem Augers, Wash Boring, NQ Coring COMPILED BY L.Y.

DATUM Geodetic DATE 2021.07.27 - 2021.07.29 LATITUDE 43.457280 LONGITUDE -80.180007 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						
294.3											

WH Split spoon penetration due to weight of hammer and rods

∇ Groundwater level observed during drilling

NOTES:

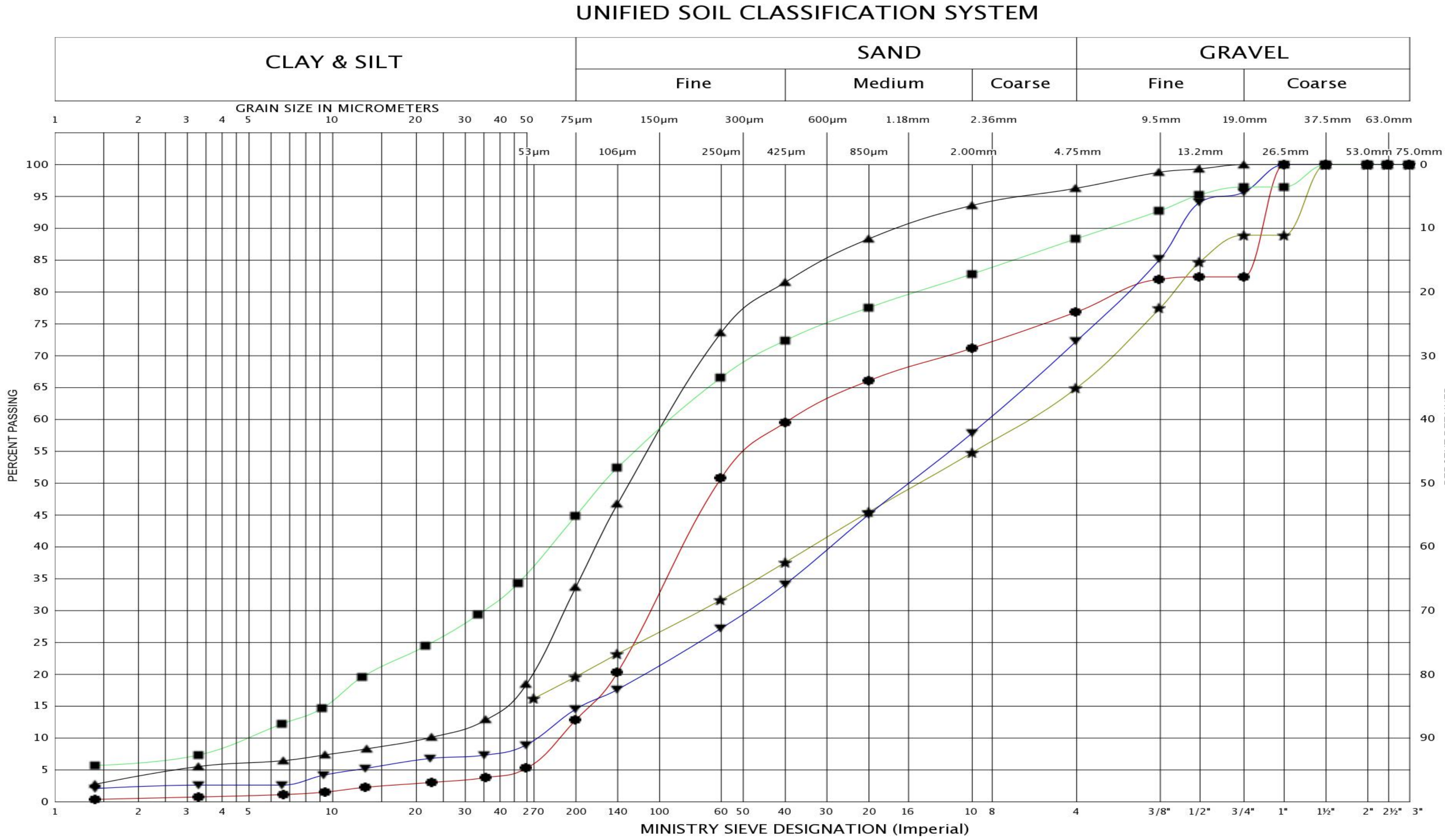
1. Artesian conditions were encountered at a depth of about 11.5 m (EL. 297.8) below the existing ground surface and the groundwater rose to an estimated height of 1.4 m (EL. 310.7) above the existing ground surface.
2. Due to the artesian conditions, a monitoring well was not installed as planned. The borehole was backfilled with cement grout immediately after drilling and rock coring was completed.
3. No cave-in was noted in the borehole upon extraction of augers.
4. 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



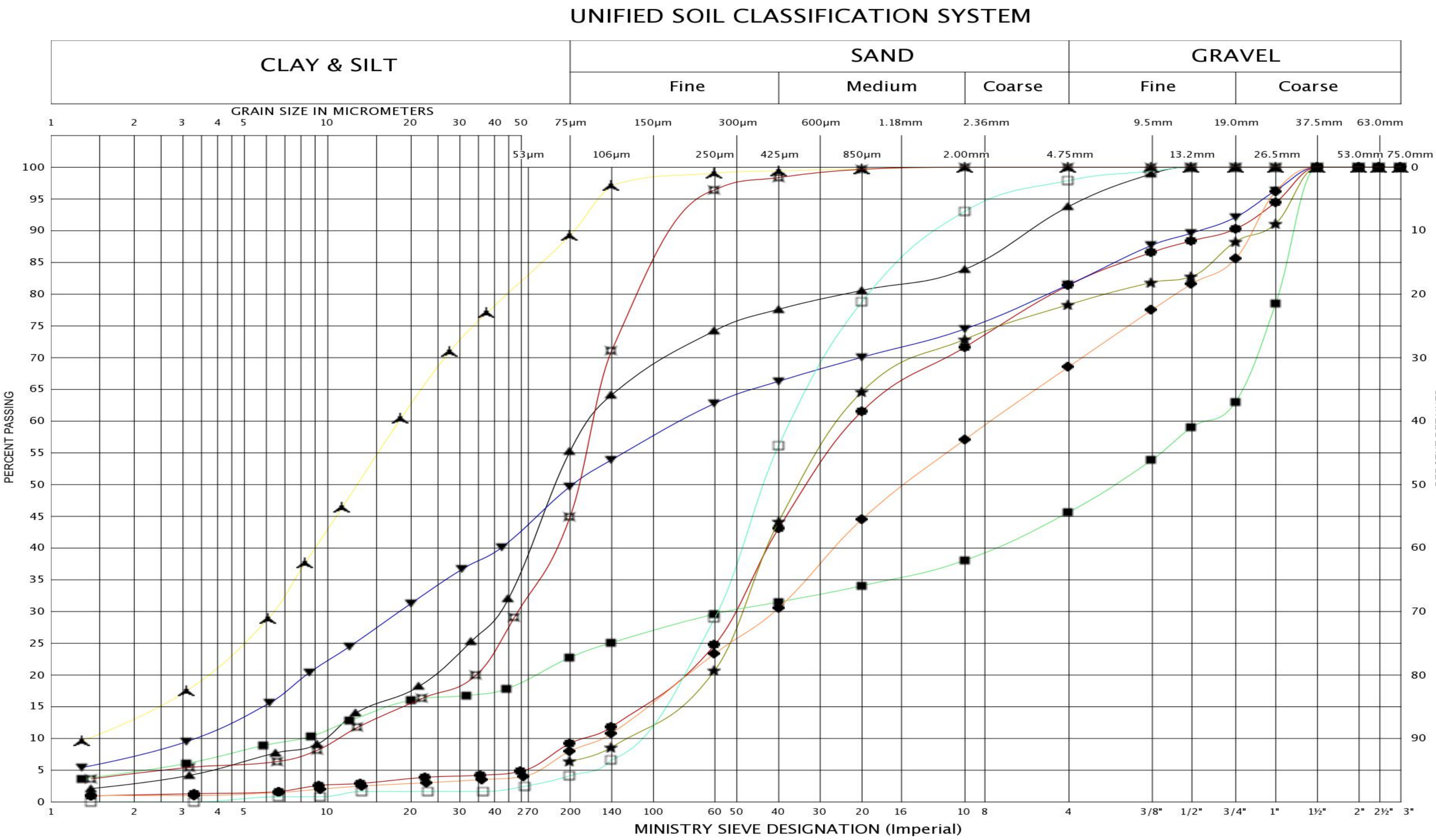
	BH	35-617-01	35-617-05	35-617-05	35-617-08	35-617-09
LEGEND	SAMPLE	2	3	5	2	4
	SYMBOL	●	★	▲	▼	■

Ontario

GRAIN SIZE DISTRIBUTION

SILTY SAND, trace/with gravel (FILL)

FIG No.:	617-GS-1
HWY :	6
GWP	3059-20-00



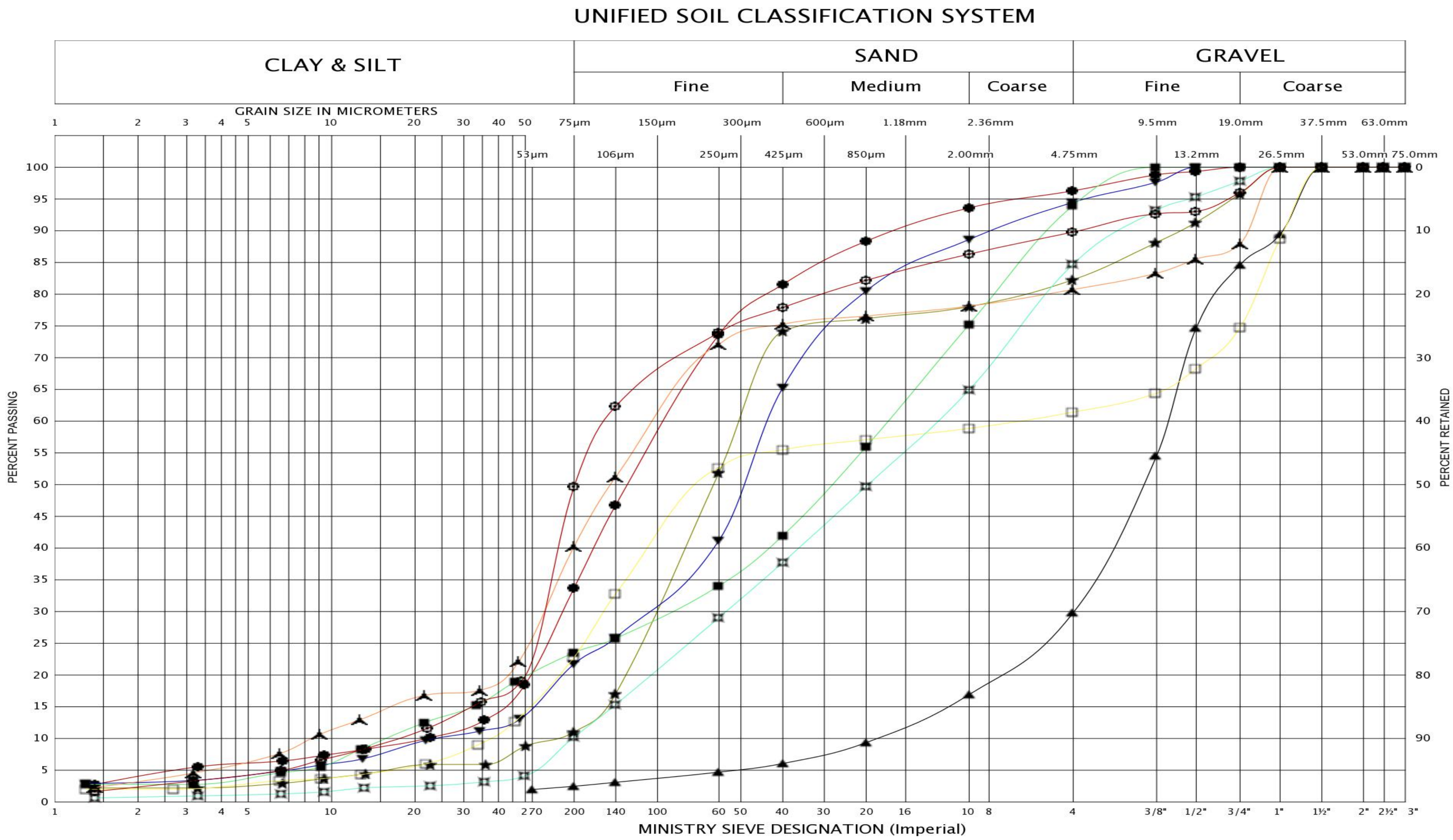
LEGEND										
	BH	35-617-01	35-617-01	35-617-02	35-617-02	35-617-03	35-617-03	35-617-04	35-617-04	35-617-04
	SAMPLE	7	11	9	11	8	10	7	8	10
	SYMBOL	▲	⬣	★	▼	◆	■	□	⌘	⅄

Ontario

GRAIN SIZE DISTRIBUTION

SANDY SILT TO SILTY SAND, trace/with gravel (Till)

FIG No.:	617-GS-2A
HWY :	6
GWP	3059-20-00



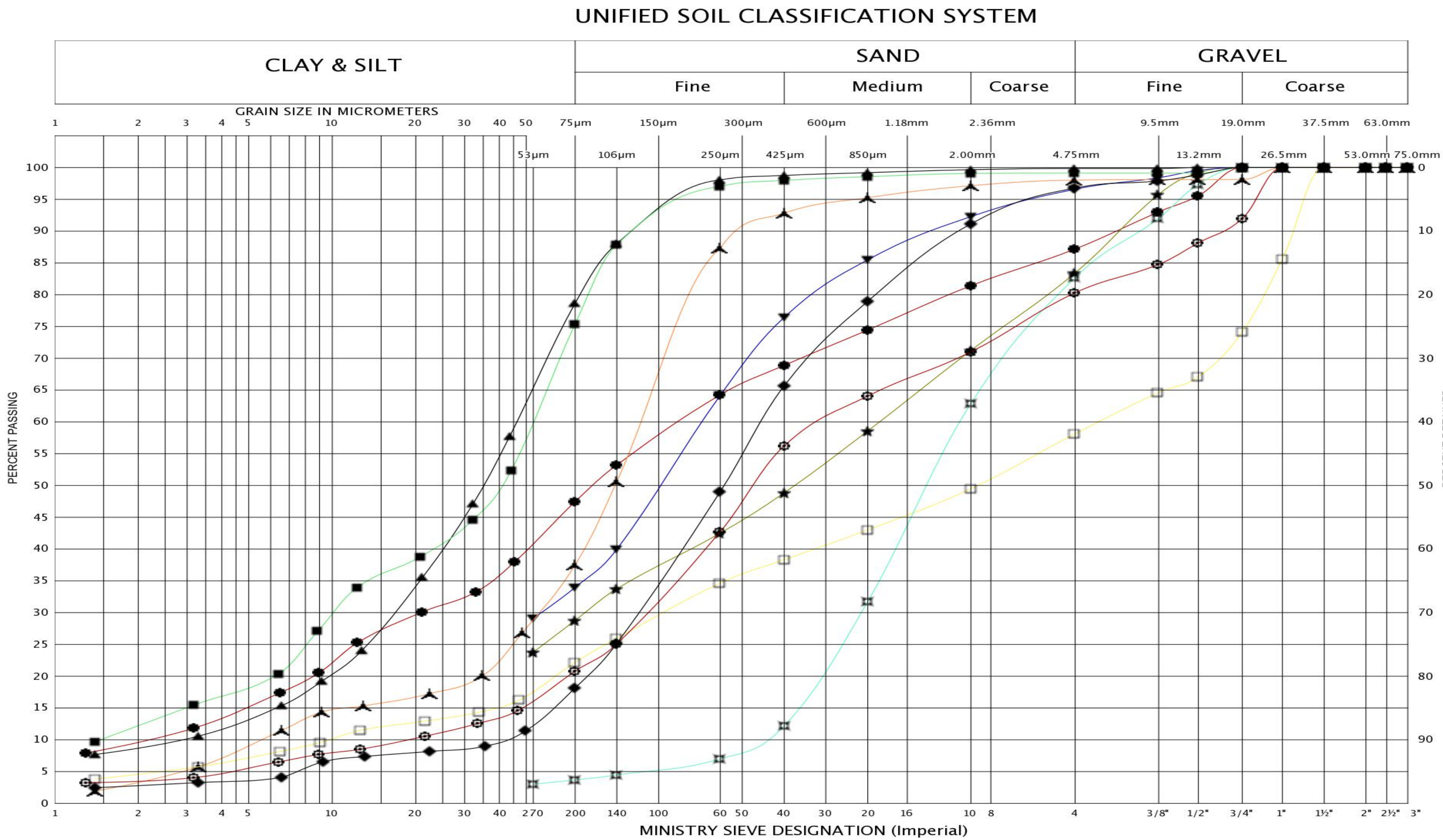
LEGEND										
	BH	35-617-04	35-617-05	35-617-05	35-617-06	35-617-06	35-617-06	35-617-07	35-617-07	35-617-07
	SAMPLE	11	5	10	4	7	9	5	8	10
	SYMBOL	★	⬢	▲	▼	■	⅄	⊕	⌘	□

Ontario

GRAIN SIZE DISTRIBUTION

SANDY SILT/SILTY SAND, trace/with gravel (TILL)

FIG No.:	617-GS-2B
HWY :	6
GWP	3059-20-00



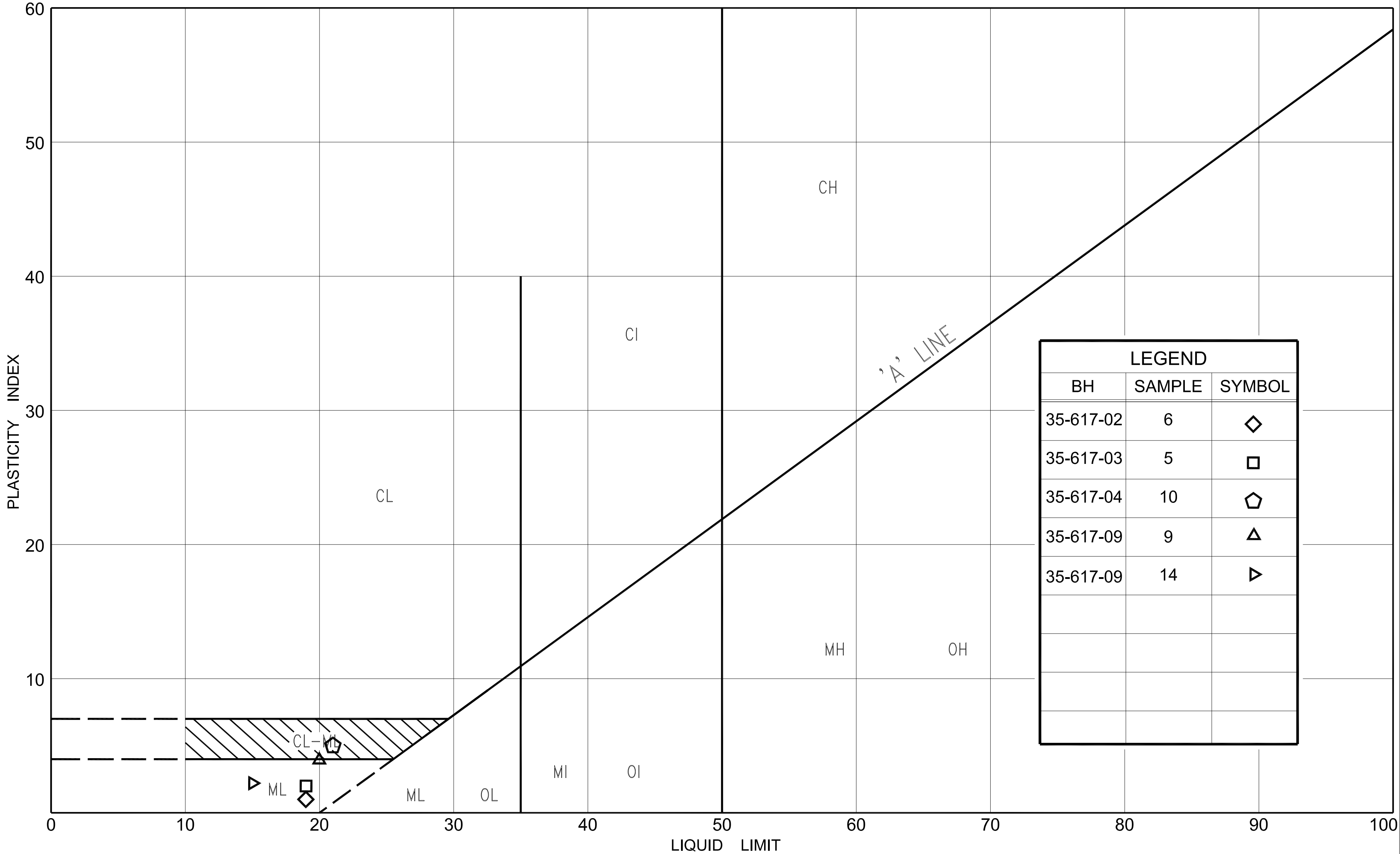
LEGEND											
	BH	35-617-08	35-617-08	35-617-09	35-617-09	35-617-09	35-617-09	35-617-10	35-617-10	35-617-10	35-617-10
	SAMPLE	5	9	6	9	11	14	5	7	10	12
	SYMBOL	⊕	◆	▼	▲	★	⬣	⌑	⅄	■	□

Ontario

GRAIN SIZE DISTRIBUTION

SANDY SILT/SILTY SAND, trace/with gravel (TILL)

FIG No.:	617-GS-2C
HWY :	6
GWP	3059-20-00



Ontario
Ministry of Transportation

PLASTICITY CHART

SILT/SANDY SILT, trace/some gravel (TILL)

FIG No.	617-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 Cores Retrieved from Borehole 35-617-02

Photographs of Rock Core Retrieved from Borehole 35-617-06

Photographs of Rock Core Samples Retrieved from Borehole 35-617-09

Photographs of Rock Core Samples Retrieved from Borehole 35-617-10

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 (Length)	% *RQD (Length)	Core Description
35-617-02	14 (1)	16.76-18.30 55’0”-60’0”	99	85 (1.32m)	**DOLOSTONE/DOLOMITE (GUELPH FORMATION)** -: medium brown, fine crystalline Dolostone, thick bedded, hard, massive, slightly weathered, unfractured to slightly fractured. Minerals: dolomite, calcite grains, containing mud/silt. Diagenesis features: vugs/voids contain calcite crystals/grains/silt/mud. Mechanical core breaks. Intact Rock Strength: R4 (Strong to Very Strong)
	15 (2)	18.3-19.8 60’0”-65’0”	99	82 (1.23m)	**DOLOSTONE/DOLOMITE (GUELPH FORMATION) - :** grey brown to brown, fine crystalline dolostone, thick bedded, hard, massive, slightly to moderately weathered, vuggy (large vugs/voids mm to cm), unfractued to silghtly fractured along horizontal bedding plane. Diagenesis features: vugs/voids infilled calcite crystals/grains/silt/mud. Mechanical core breaks. Intact Rock Strength: R4 (Strong to Very 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 (Length)	% *RQD (Length)	Core Description
35-617-05	13 (1)	15.2-15.9 49’10”-52’1”	100 (0.69m)	100 (0.69m)	**GUELPH FORMATION** Unweathered, fine to medium grained, thinly bedded, brown, moderately hard, slightly calcareous, crystalline **CARBONATE** with thin, black stylolites (<1.0 mm) and calcite lined vugs (<15.0 mm). Sample taken at 15.65-15.88 m.
	14 (2)	15.9-17.4 52’1”-57’1”	100 (1.52m)	71.7 (1.09m)	**GUELPH FORMATION** Unweathered, fine to medium grained, thinly bedded, brown to grey, moderately hard, slightly calcareous, crystalline **CARBONATE** with thin, black stylolites (<1.0 mm) and calcite lined vugs (<10.0 mm). Occasional features: Broken rock at 16.15-15.23 m; slight iron staining within natural fractures/vugs; black shale along natural fractures (<1.0 mm thick).
	15 (3)	17.4-18.2 57’1”-59’10”	95 (0.76m)	89.0 (0.71m)	**GUELPH FORMATION** Unweathered, fine to medium grained, thinly bedded, brown to grey, moderately hard, slightly calcareous, crystalline **CARBONATE** with thin, black stylolites (<1.0 mm) and calcite lined vugs (<10.0 mm). Occasional features: Broken rock at 17.40-17.45 m; slight iron staining within natural fractures/vugs; black shale along natural fractures (<1.0 mm thick).

*RQD – Rock Quality Designation

Logged by: Heather Racher, M.Sc.
Reviewed by: Carlos Nascimento, P. Eng.

Notes:
1. Intact Rock Strength obtained using rock pick test in drill core from table “Rock Characterization Testing & Monitoring”, International Society of Rock Mechanics.
2. Depths are approximated where core recovery is less than 100%. RQDs are calculated according to core recovery (less than designated 1.52 m runs).

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 (Length)	% *RQD (Length)	Core Description
35-617-06	12 (1)	14.1-14.8 46’2”-48’6”	85	52 (0.75m)	**DOLOSTONE/DOLOMITE (GUELPH FORMATION)** -: medium brown, fine crystalline, thick bedded, hard, massive, slightly - moderately weathered (mm to cm vugs/voids), slightly fractured, Minerals: dolomite, calcite grains, containing mud/silt. Diagenesis features: vugs/voids contain calcite/dolomite crystals/grains & silt/mud. Mechanical core breaks. Intact Rock Strength: R4 (Strong to Very Strong)
35-617-06	13 (2)	14.8-15.6 48’6”-51’3”	91	63	**DOLOSTONE/DOLOMTE (GUELPH FORMATION) - :** medium brown, fine crystalline dolostone, hard, thick bedded, moderately weathered/vuggy, slightly to moderately fractured mostly parralel to bedding with occational bitumen staining. Mineral: calcite/dolomite. Diagenesis features: vugs infilled calcite crystals/grains. Mechanical core breaks. Intact Rock Strength: R4 (Strong to Very Strong)
	14 (3)	15.6-17.4 51’3”-57’0”	79	69	**DOLOSTONE/DOLOMITE (GUELPH FORMATION) - :** medium brown, occational offwhite, fine crystalline dolostone, thick bedded, hard, slightly weathered associated with voids/vugs, slightly fractured parallel to bedding with occational bitumen staining. Minerals: dolomite, calcite, containing organic material, mud/silt. Mineral: Dolomite. Diagenesis features: voids infilled calcite crystals/grains/silt/mud. Mechanical core breaks. Intact Rock Strength: R4 (Strong to Very 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 (Length)	% *RQD (Length)	Core Description
35-617-09	16 (1)	15.2-15.8 50-51'9"	100 (0.60m)	94 (0.60m)	**DOLOSTONE/DOLOMITE (GUELPH FORMATION)** -: medium brown, fine crystalline, thick bedded, hard, massive, slightly - moderately weathered (mm to cm vugs/voids), slightly fractured, Minerals: dolomite, calcite grains, containing mud/silt. Diagenesis features: vugs/voids contain calcite/dolomite crystals/grains & silt/mud. Mechanical core breaks. Intact Rock Strength: R4 (Strong to Very Strong)
35-617-09	17 (2)	15.8-17.3 51'9"-56'9"	97 (1.46m)	76 (1.14m)	**DOLOSTONE/DOLOMTE (GUELPH FORMATION) - :** medium brown, fine crystalline dolostone, hard, thick bedded, moderately weathered/vuggy, slightly to moderately fractured mostly parralel to bedding with occational bitumen staining. Mineral: calcite/dolomite. Diagenesis features: vugs infilled calcite crystals/grains. Mechanical core breaks. Intact Rock Strength: R4 (Strong to Very Strong)
	18 (3)	17.3-18.3 51'3"-57'0"	100 (1.0m)	90 (0.9m)	**DOLOSTONE/DOLOMITE (GUELPH FORMATION) - :** medium brown, occational offwhite, fine crystalline dolostone, thick bedded, hard, slightly weathered associated with voids/vugs, slightly fractured parallel to bedding with occational bitumen staining. Minerals: dolomite, calcite, containing organic material, mud/silt. Mineral: Dolomite. Diagenesis features: voids infilled calcite crystals/grains/silt/mud. Mechanical core breaks. Intact Rock Strength: R4 (Strong to Very Strong)

*RQD – Rock Quality Designation

Compiled & logged by: Frank Meng, M.Eng. EIT
Reviewed by: Lul Yimam, PhD, 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 (Length)	% *RQD (Length)	Core Description
35-617-10	14 (1)	11.5-12.6 37’7”-41’2”	39 (0.43m)	21 (0.23m)	**DOLOSTONE/DOLOMITE (GUELPH FORMATION)** -: medium brown, fine crystalline Dolostone, thick bedded, hard, massive, slightly weathered, unfractured to slightly fractured. Minerals: dolomite, calcite grains, containing mud/silt. Diagenesis features: vugs/voids contain calcite crystals/grains/silt/mud. Mechanical core breaks. Intact Rock Strength: R4 (Strong to Very Strong)
	15 (2)	12.6-14.6 41’2”-47’10”	100 (2.0m	72 (1.44m)	**DOLOSTONE/DOLOMITE (GUELPH FORMATION) - :** grey brown to brown, fine crystalline dolostone, thick bedded, hard, massive, slightly to moderately weathered, vuggy (large vugs/voids mm to cm), unfractued to silghtly fractured along horizontal bedding plane. Diagenesis features: vugs/voids infilled calcite crystals/grains/silt/mud. Mechanical core breaks. Intact Rock Strength: R4 (Strong to Very Strong)

*RQD – Rock Quality Designation

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

Peto MacCallum Ltd.
CONSULTING ENGINEERS

ROCK CORE DIMENSIONS
ASTM D4543

CLIENT	AECOM	*PML REF*	17TF006A
PROJECT	HYW 401/6	*LAB NO.*	1801322-A
SAMPLE IDENTIFICATION	BH 35-617-5, RN-1, 51' 4"-52' 1"	*DATE SAMPLED*	
		DATE TESTED	2018-06-05
		TESTED BY	YA/BM

DEVIATION FROM STRAIGHTNESS

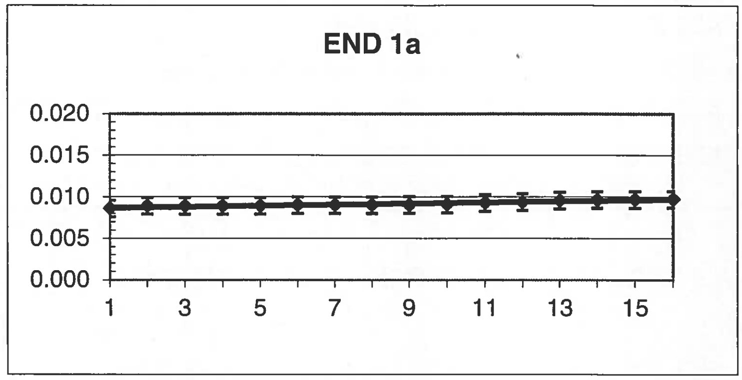
DIAL READING (IN)	TRIAL 1	TRIAL 2	TRIAL 3
MINIMUM	0.0180	0.0150	0.0090
MAXIMUM	0.0270	0.0290	0.0290
DIFFERENCE	0.0090	0.0140	0.0200
MAX DIFF.	0.02	SPEC.	0.020 max.

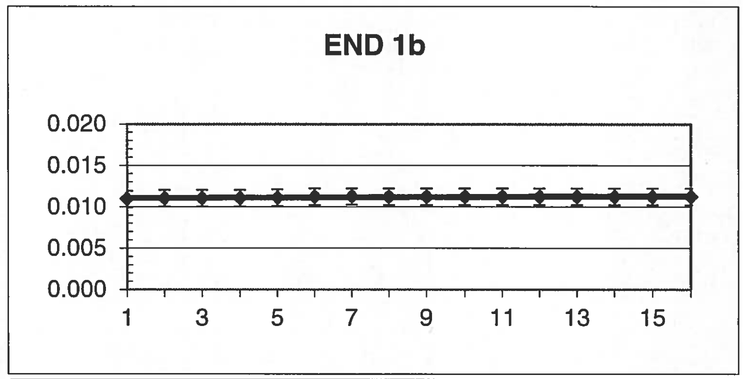
FLATNESS TOLERANCE

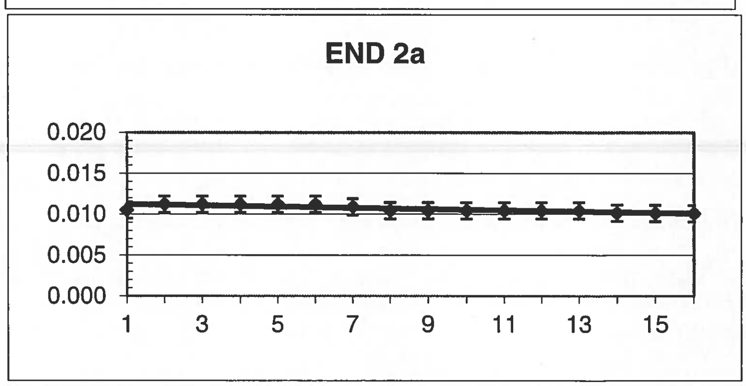
DIAL READING (IN)	END 1		END 2	
	SET 1	SET 2	SET 1	SET 2
RDG 1	0.0086	0.0109	0.0105	0.0083
RDG 2	0.0089	0.0110	0.0112	0.0082
RDG 3	0.0089	0.0110	0.0112	0.0082
RDG 4	0.0089	0.0110	0.0112	0.0083
RDG 5	0.0089	0.0111	0.0112	0.0087
RDG 6	0.0090	0.0112	0.0112	0.0090
RDG 7	0.0090	0.0113	0.0109	0.0090
RDG 8	0.0090	0.0112	0.0104	0.0090
RDG 9	0.0090	0.0112	0.0104	0.0095
RDG 10	0.0091	0.0112	0.0104	0.0098
RDG 11	0.0093	0.0112	0.0104	0.0102
RDG 12	0.0094	0.0112	0.0104	0.0104
RDG 13	0.0096	0.0112	0.0104	0.0106
RDG 14	0.0096	0.0112	0.0102	0.0109
RDG 15	0.0097	0.0112	0.0101	0.0113
RDG 16	0.0097	0.0112	0.0101	0.0116
RDG 17				
RDG 18				
RDG 19				
RDG 20				

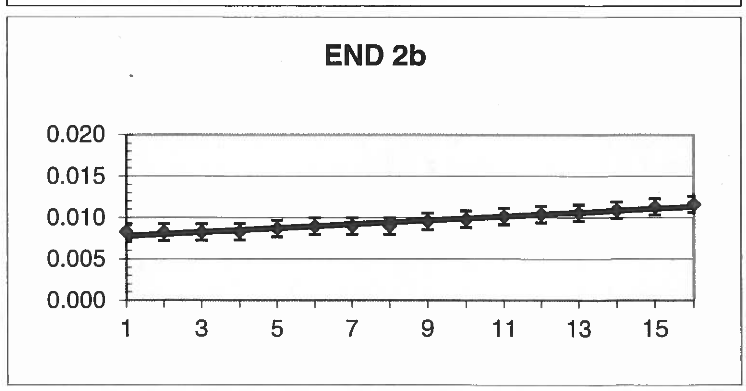
FLATNESS TOLERANCE= .001 in.

CORE DIAMETER (in.)		2.4750	2.4870	2.4840
AVE:		2.4820	PERPENDICULARITY RATIO (Specified .0043 max.)	
SLOPE OF BEST FIT LINE	MINIMUM	MAXIMUM		
END 1A	0.0087	0.0097	0.0004	
END 2B	0.0110	0.0113	0.0001	
END 2A	0.0101	0.0112	0.0004	
END 2B	0.0078	0.0113	0.0014	









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-A
SAMPLE IDENTIFICATION	BH 35-617-5, RN-1, 51' 4"-52' 1"	DATE SAMPLED	
		DATE TESTED	2018-06-06
		TESTED BY	YA/BM

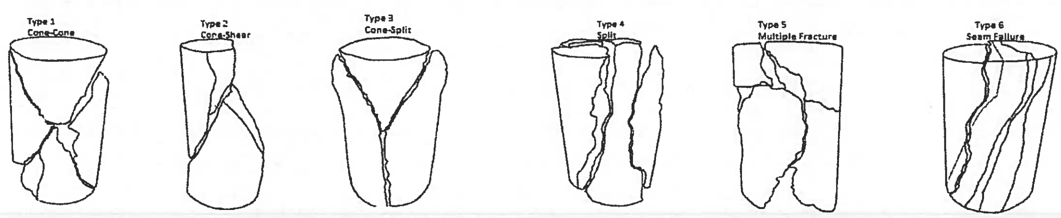
UNCONFINED COMPRESSIVE STRENGTH

CORE DIMENSIONS		COMPRESSIVE STRENGTH	
SPECIMEN DIAMETER (in.)	2.4820	TEST TIME (min) (spec. 2 to 15)	5:21
SPECIMEN LENGTH (in.)	5.560	MAXIMUM LOAD APPLIED (kN)	255.33
	5.547		
	5.546	COMPRESSIVE STRENGTH (MPa)	81.8
AVE.	5.551	TYPE OF FAILURE	3
SURFACE AREA (sq mm)	3121	LENGTH TO DIAMETER RATIO (spec 2-2.5)	2.24

MOISTURE CONTENT / UNIT WEIGHT

MOISTURE CONTENT		UNIT WEIGHT	
WEIGHT OF WET SAMPLE + TARE (g)	1220.00	WEIGHT OF DRY SAMPLE IN AIR (g)	1132.90
WEIGHT OF DRY SAMPLE + TARE (g)	1216.29	VOLUME OF SAMPLE (cu m)	0.000440
WEIGHT OF WATER (g)	3.71	UNIT WEIGHT (kg/cu m)	2574
WEIGHT OF TARE (g)	107.60		
WEIGHT OF DRY SAMPLE (g)	1108.69		
MOISTURE CONTENT (%)	0.3		

REMARKS



REVIEWED BY J.Noor

DATE 2018-06-08

Peto MacCallum Ltd.
CONSULTING ENGINEERS

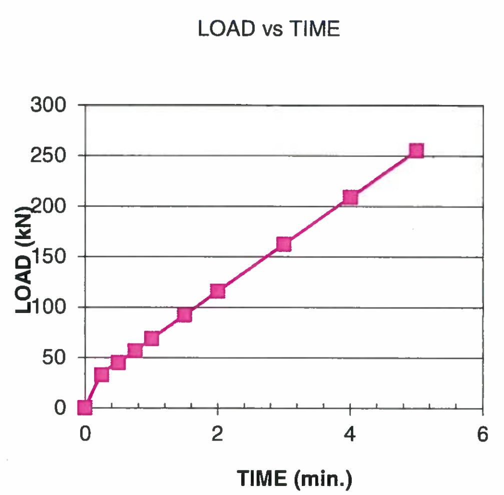
ROCK CORE TESTING

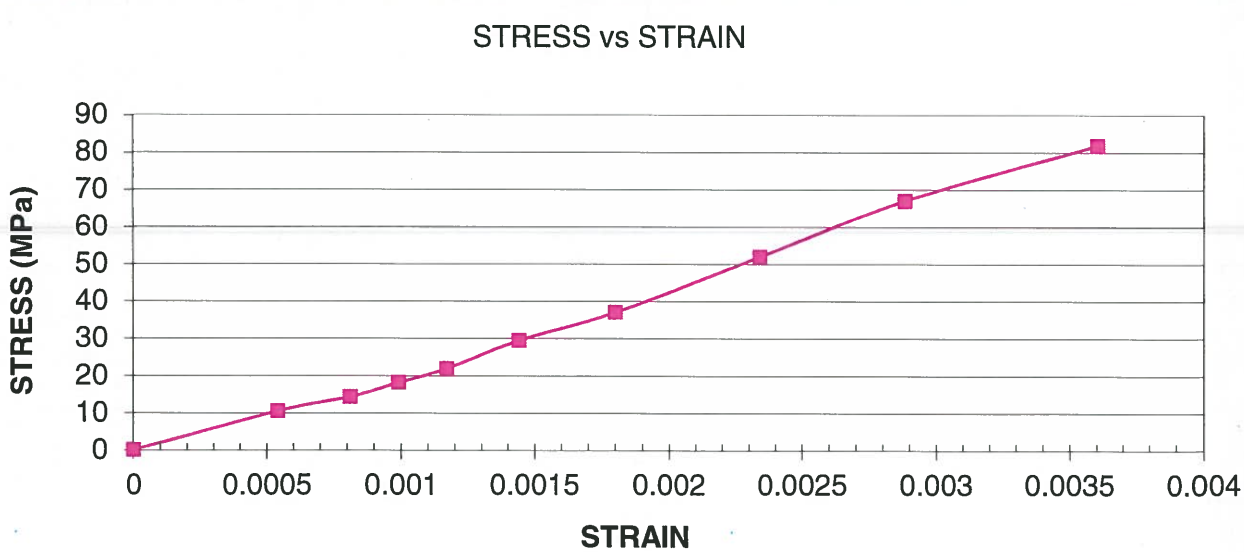
ASTM D7012

CLIENT	AECOM	*PML REF*	17TF006A
PROJECT	HYW 401/6	*LAB NO.*	1801322-A
SAMPLE IDENTIFICATION	BH 35-617-5, RN-1, 51' 4"-52' 1"	*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	0.474	0	0	0
0.25	0.477	32.78	10.5	0.0005
0.5	0.4785	44.76	14.3	0.0008
0.75	0.4795	56.65	18.1	0.0010
1	0.4805	68.44	21.9	0.0012
1.5	0.482	92.00	29.5	0.0014
2	0.484	115.56	37.0	0.0018
3	0.487	162.13	51.9	0.0023
4	0.49	208.90	66.9	0.0029
5	0.494	255.33	81.8	0.0036





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

Peto MacCallum Ltd.
CONSULTING ENGINEERS

UNIAXIAL COMPRESSIVE STRENGTH OF ROCK CORE

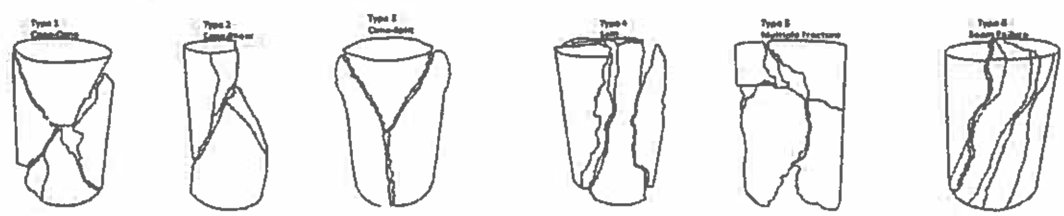
ASTM D7012

CLIENT	AECOM Canada Ltd.	PML REF	17TF006 I
PROJECT	Hwy 6 and 401 Improvements, Midblock Interchange Area	LAB NO.	2104843 A
SAMPLE IDENTIFICATION	BH 35-617-9, Run2, 55'7"-56'5"	DATE SAMPLED	
		DATE TESTED	2021-08-24
		TESTED BY	L. Gowry

CORE DIMENSIONS		COMPRESSIVE STRENGTH	
SPECIMEN DIAMETER (in.)	2.4957	TEST TIME (min) (spec. 2 to 15)	9:36
SPECIMEN LENGTH (in.)	5.571	MAXIMUM LOAD APPLIED (kN)	263.90
	5.571		
	5.574	COMPRESSIVE STRENGTH (MPa)	83.6
AVE.	5.572	TYPE OF FAILURE	1
SURFACE AREA (sq mm)	3156	LENGTH TO DIAMETER RATIO (spec 2-2.5)	2.23

MOISTURE CONTENT		*UNIT WEIGHT*	
WEIGHT OF WET SAMPLE + TARE (g)	1277.56	WEIGHT OF DRY SAMPLE IN AIR (g)	1127.85
WEIGHT OF DRY SAMPLE + TARE (g)	1276.84	VOLUME OF SAMPLE (cu m)	0.000447
WEIGHT OF WATER (g)	0.72	Density (kg/cu m)	2525
WEIGHT OF TARE (g)	176.12	UNIT WEIGHT (γ)	24.75
WEIGHT OF DRY SAMPLE (g)	1100.72		
MOISTURE CONTENT (%)	0.1		

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*	17TF006 I
PROJECT	Hwy 6 and 401 Improvements, Midblock Interchange Area	*LAB NO.*	2104843 A
SAMPLE IDENTIFICATION	BH 35-617-9, Run2, 55'7"-56'5"	*DATE SAMPLED*	
		DATE TESTED	2021-08-24
		TESTED BY	L. Gowry

DEVIATION FROM STRAIGHTNESS

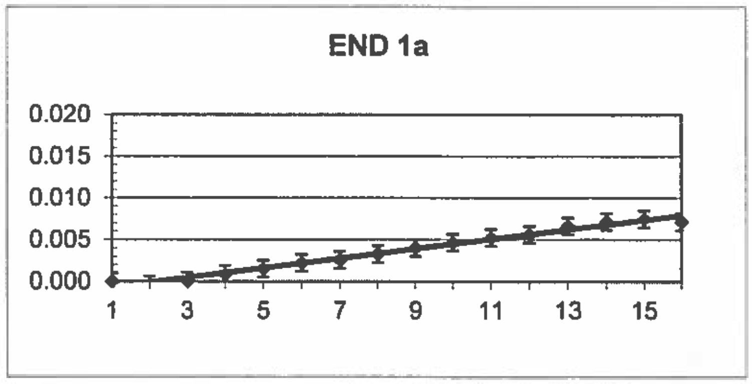
DIAL READING (IN)	TRIAL 1	TRIAL 2	TRIAL 3
MINIMUM	0.0910	0.0820	0.0890
MAXIMUM	0.0990	0.0870	0.0930
DIFFERENCE	0.0080	0.0050	0.0040
MAX DIFF.	0.008	SPEC.	0.020 max.

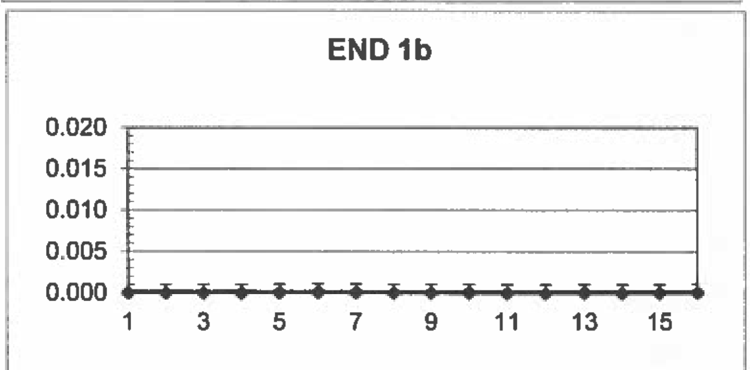
FLATNESS TOLERANCE

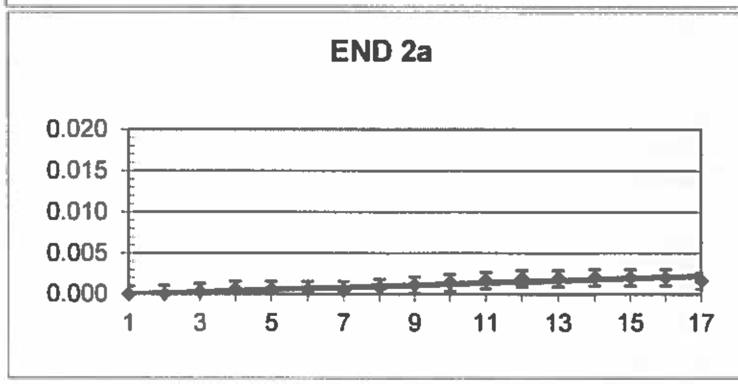
DIAL READING (IN)	END 1 SET 1	END 1 SET 2	END 2 SET 1	END 2 SET 2
RDG 1	0.0000	0.0000	0.0000	0.0000
RDG 2	-0.0004	0.0000	0.0001	0.0001
RDG 3	0.0001	0.0000	0.0003	0.0003
RDG 4	0.0009	0.0000	0.0006	0.0009
RDG 5	0.0016	0.0001	0.0006	0.0013
RDG 6	0.0022	0.0001	0.0005	0.0018
RDG 7	0.0026	0.0001	0.0005	0.0024
RDG 8	0.0033	0.0001	0.0008	0.0028
RDG 9	0.0040	0.0001	0.0011	0.0036
RDG 10	0.0047	0.0001	0.0014	0.0045
RDG 11	0.0052	0.0000	0.0017	0.0049
RDG 12	0.0056	0.0000	0.0019	0.0055
RDG 13	0.0066	0.0001	0.0019	0.0060
RDG 14	0.0072	0.0000	0.0020	0.0064
RDG 15	0.0075	0.0000	0.0020	0.0069
RDG 16	0.0071	0.0001	0.0020	0.0072
RDG 17			0.0016	0.0070
RDG 18				
RDG 19				
RDG 20				

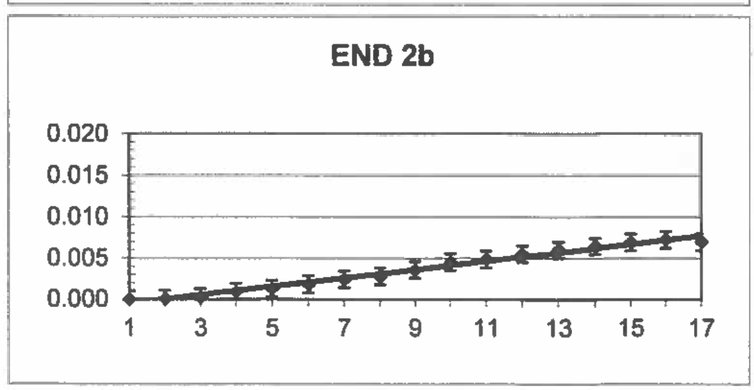
FLATNESS TOLERANCE= .001 in.

CORE DIAMETER (in.)	2.4980	2.4940	2.4950
AVE:	2.4957	PERPENDICULARITY RATIO (Specified .0043 max.)	
SLOPE OF BEST FIT LINE	MINIMUM	MAXIMUM	
END 1A	-0.0007	0.0079	0.0034
END 2B	0.0000	0.0000	0.0000
END 2A	0.0000	0.0022	0.0009
END 2B	-0.0005	0.0077	0.0033









REVIEWED BY J. Noor *DATE* 2021-08-25

Peto MacCallum Ltd.
CONSULTING ENGINEERS

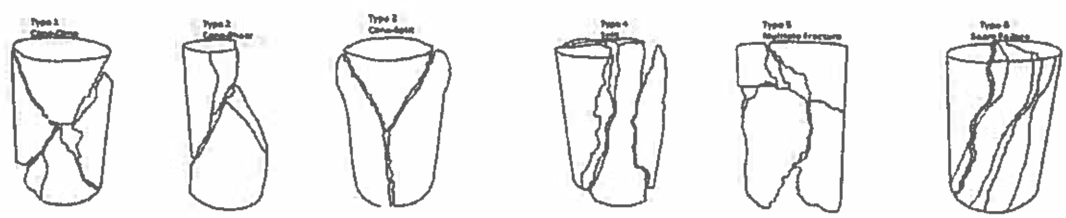
UNIAXIAL COMPRESSIVE STRENGTH OF ROCK CORE

ASTM D7012

CLIENT	AECOM Canada Ltd.	PML REF	17TF006I
PROJECT	Hwy 6 and 401 Improvements, Midblock Interchange Area	LAB NO.	2104843 B
SAMPLE IDENTIFICATION	BH35-617-10, Run2, 46'4"-47'2"	DATE SAMPLED	
		DATE TESTED	2021-08-24
		TESTED BY	L. Gowry

CORE DIMENSIONS		COMPRESSIVE STRENGTH	
SPECIMEN DIAMETER (in.)	1.8570	TEST TIME (min) (spec. 2 to 15)	9:36
SPECIMEN LENGTH (in.)	4.242	MAXIMUM LOAD APPLIED (kN)	288.50
	4.241		
	4.240	COMPRESSIVE STRENGTH (MPa)	165.1
AVE.	4.241	TYPE OF FAILURE	2
SURFACE AREA (sq mm)	1747	LENGTH TO DIAMETER RATIO (spec 2-2.5)	2.28

MOISTURE CONTENT		*UNIT WEIGHT*	
WEIGHT OF WET SAMPLE + TARE (g)	704.55	WEIGHT OF DRY SAMPLE IN AIR (g)	525.62
WEIGHT OF DRY SAMPLE + TARE (g)	704.37	VOLUME OF SAMPLE (cu m)	0.000188
WEIGHT OF WATER (g)	0.18	Density (kg/cu m)	2792
WEIGHT OF TARE (g)	180.31	UNIT WEIGHT (γ)	27.37
WEIGHT OF DRY SAMPLE (g)	524.06		
MOISTURE CONTENT (%)	0.0		
REMARKS			



REVIEWED BY J. Noor *DATE* 2021-08-24

Peto MacCallum Ltd.
CONSULTING ENGINEERS

ROCK CORE DIMENSIONS
ASTM D4543

CLIENT	AECOM Canada Ltd.
PROJECT	Hwy 6 and 401 Improvements, Midblock Interchange Area
SAMPLE IDENTIFICATION	BH35-617-10, Run2, 46'4"-47'2"

PML REF	17TF006I
LAB NO.	2104843 B
DATE SAMPLED	
DATE TESTED	2021-08-24
TESTED BY	L. Gowry

DEVIATION FROM STRAIGHTNESS

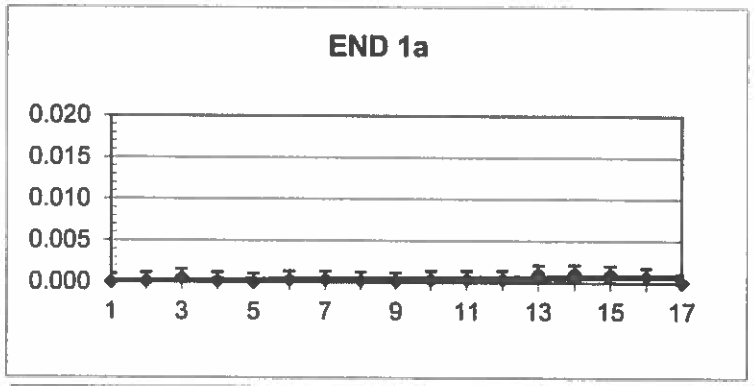
DIAL READING (IN)	TRIAL 1	TRIAL 2	TRIAL 3
MINIMUM	0.0250	0.0330	0.0410
MAXIMUM	0.0280	0.0390	0.0470
DIFFERENCE	0.0030	0.0060	0.0060
MAX DIFF.	0.006	SPEC.	0.020 max.

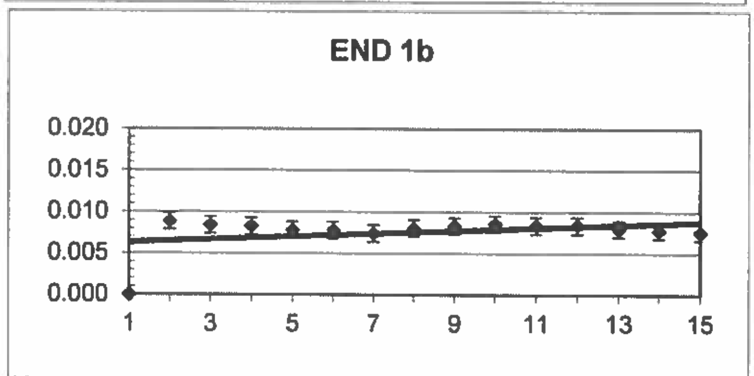
FLATNESS TOLERANCE

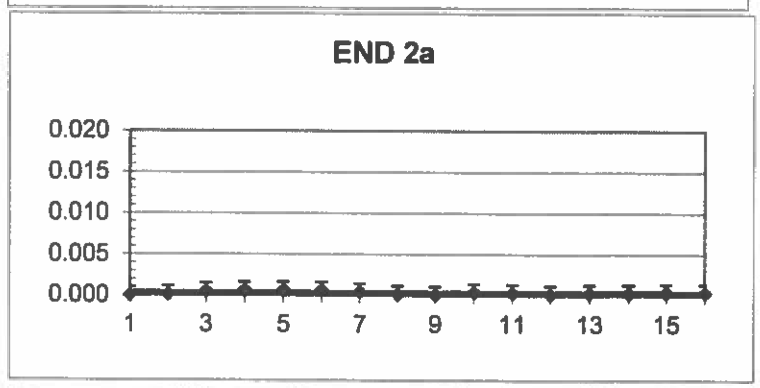
DIAL READING (IN)	END 1 SET 1	END 1 SET 2	END 2 SET 1	END 2 SET 2
RDG 1	0.0000	0.0000	0.0000	0.0000
RDG 2	0.0002	0.0089	0.0001	0.0045
RDG 3	0.0006	0.0084	0.0005	0.0047
RDG 4	0.0002	0.0083	0.0006	0.0053
RDG 5	0.0001	0.0078	0.0006	0.0056
RDG 6	0.0003	0.0078	0.0006	0.0054
RDG 7	0.0004	0.0074	0.0005	0.0047
RDG 8	0.0002	0.0080	0.0002	0.0044
RDG 9	0.0002	0.0083	0.0001	0.0040
RDG 10	0.0004	0.0086	0.0004	0.0055
RDG 11	0.0004	0.0084	0.0003	0.0055
RDG 12	0.0004	0.0084	0.0002	0.0049
RDG 13	0.0011	0.0080	0.0003	0.0055
RDG 14	0.0011	0.0078	0.0003	0.0059
RDG 15	0.0010	0.0076	0.0004	
RDG 16	0.0008		0.0004	
RDG 17	0.0001			
RDG 18				
RDG 19				
RDG 20				

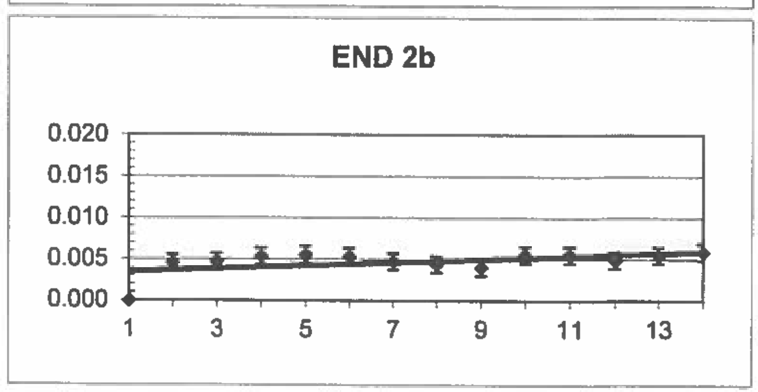
FLATNESS TOLERANCE= .001 in.

CORE DIAMETER (in.)	1.8580	1.8570	1.8560
AVE:	1.8570	PERPENDICULARITY RATIO (Specified .0043 max.)	
SLOPE OF BEST FIT LINE	MINIMUM	MAXIMUM	
END 1A	0.0001	0.0008	0.0004
END 2B	0.0064	0.0088	0.0013
END 2A	0.0003	0.0004	0.0001
END 2B	0.0035	0.0059	0.0013









REVIEWED BY J. Noor *DATE* 2021-08-24

APPENDIX E

Soil Chemical Test Results

Certificate of Analysis

AGAT WORK ORDER: 17T297980

PROJECT: 17TF006A

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

CLIENT NAME: PETO MACCALLUM LIMITED

ATTENTION TO: Lul Yimam

SAMPLING SITE:Guelph, Puslinch

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

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

Quality Assurance

CLIENT NAME: PETO MACCALLUM LIMITED | **AGAT WORK ORDER: 17T297980**
PROJECT: 17TF006A | **ATTENTION TO: Lul Yimam**
SAMPLING SITE:Guelph, Puslinch | **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

FINAL REPORT

CA14274-AUG21 R1

Client: Peto MacCallum Ltd
Project: 17TF006A
Project Manager: Lul Yimam
Samplers: N/A

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

FINAL REPORT

CA14274-AUG21 R1

Client: Peto MacCallum Ltd
Project: 17TF006A
Project Manager: Lul Yimam
Samplers: N/A

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 I 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.

-- End of Analytical Report --

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

FOR

WELLINGTON COUNTY ROAD 34 UNDERPASS
SITE NO. 35X-0617/B0, STATION 10+000
LATITUDE AND LONGITUDE: 43.457324, -80.180489
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.
165 CARTWRIGHT AVENUE
TORONTO, ONTARIO
M6A 1V5
Phone: (416) 785-5110
Fax: (416) 785-5120
Email: toronto@petomaccallum.com

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1 cc: PML Toronto

PML Ref. 17TF006A
Index No.: 062FDR
GEOCRES No.: 40P8-292
October 14, 2021

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PART B – PRELIMINARY FOUNDATION DESIGN REPORT

PART B – PRELIMINARY FOUNDATION DESIGN REPORT for Wellington County Road 34 Underpass

Site No. 35X-0617/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), West Region has 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 components of the MBI area project include two bridge structures (the Wellington Road 34 Connector Underpass and the Wellington Road 34 Underpass), and the high fills and deep cuts along Highway 6, Concession Road 7 and the new connector route. The overhead signs on the north and south sides of the midblock interchange along Highway 6 and the construction of retaining walls, if required, and the embankments and approach earthworks, may also need site specific foundation investigation and design recommendations.

This report provides recommendations for foundation design of the proposed Wellington Road 34 Underpass based on the factual foundation investigation data presented in the Preliminary Foundation Investigation Report (Part A) and the General Arrangement (GA) drawing provided by AECOM on March 25, 2021. During the original foundation investigation carried out in 2017 and the recent additional investigation in 2021, attempts were made to locate the boreholes within the footprints of the foundation elements as provided in AECOM's preliminary design drawings. Additional foundation investigation work should be considered if significant modifications/changes are made to the conceptual/preliminary design of the underpass or the road alignment. Further, it should be noted that some boreholes were drilled away from the originally proposed locations because of utility conflicts and access constraints, and additional investigations may be required if unforeseen conditions are observed during construction.

It should be understood that this report is intended for use by AECOM, as MTO's authorized engineer, for the purpose of designing the proposed Wellington Road 34 Underpass at the location where the foundation investigation was conducted. This report shall not be used for any other purpose or for any other locations, or by any other parties including design-build contractors. Where comments are made on construction, they are provided to highlight 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. Recommendations regarding construction aspects of the foundation elements should be provided during the detail design phase of the project.

Wherever it is 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 all highway structures should be carried out in accordance with the guiltiness 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, the plan is to construct the new underpass as a two-span east-west oriented structure, with each span 32.0 m long. In addition, the underpass will consist of 6.5 m and 7.0 m long cantilevered sections, extending from the west and east abutments, respectively. The total width of the underpass will be 12.7 m, and the structure is designed to accommodate two lanes of traffic, one in each direction. The underpass will be supported on integral abutments and a centre pier. The centre pier will be located in the median of Highway 6, and will consist of three (3) columns located at 4.2 m centre-to-centre spacing.

The GA drawing indicates that steel H-piles are the preferred option to support the centre pier and abutment footings of the new underpass. The steel H-piles for the integral abutments will be lowered in pre-augered holes supported with 600 mm diameter and 3.0 m long corrugated steel pipes (CSP) and backfilled with loose sand. Based on the GA drawing, the cut-off elevations of the piles to support the west and east abutments will be at El. 313.5. The cut-off elevation of the piles for the centre pier will be at El. 308.5, and the pile-cap is proposed to be placed at about El. 308.1.

The approach slabs of the new underpass will be 6.0 m long at both the west and east sides. The design grade of the approach embankments will be set at El. 318.0, which will result in fills of 9.1 m high on the west side (Sta. 9+960), and about 9.2 m high on the east side (Sta. 10+040).

8. FOUNDATION RECOMMENDATIONS

8.1 Subsoil and Groundwater Conditions

Based on the findings of the foundation investigation presented in Part A, the subsurface at the location of the proposed underpass consists of a thin layer of topsoil underlain by 0.7 m to 2.3 m thick sandy silt or silty sand fill. In Borehole 35-617-10 drilled as part of the additional investigation in 2021, a layer of fine fibrous to amorphous peat, 2.3 m thick, was encountered underlain by a thin layer of sandy silt/silty sand fill. Beneath the fill, very loose to very dense sandy silt/silty sand till with varying proportions of gravel was encountered. Bedrock was encountered in five (5) boreholes at depths ranging from 11.5 m to 16.8 m (El. 297.8 to El. 292.2) below existing grade.

The stabilized groundwater elevation measured after the completion of drilling in a monitoring well installed in 2017 on the south side of the proposed west abutment (35-617-03A) was at El. 308.4. Recent groundwater level measurements in this well showed slight changes at El. 308.2. Another well was installed during the additional investigation in 2021 at Borehole 35-617-09 (centre pier). Subsequent readings in this well indicate stabilized groundwater elevation of El. 308.6.

Artesian groundwater conditions were encountered in the monitoring well installed in 2017 in a borehole located on the north side of the proposed east abutment (Borehole 35-617-06). The artesian head stabilised at approximately 1.2 m above the ground surface (El. 309.9). Artesian conditions were also encountered in Borehole 35-617-07 drilled in 2017. This Borehole was backfilled with cement grout immediately after the artesian condition was encountered. Recently in 2021, artesian condition was encountered in Borehole 35-617-10 and the stabilized head was measured to be approximately 1.4 m above the ditch surface. Low head artesian conditions and the existence of a confined and pressurized aquifer from the zone of medium to coarse silty sand basal layer, and/or the underlying fractured bedrock, were also confirmed in other boreholes drilled recently by PML near the intersection as part of this project. The water wells in the surrounding area also suggest the existence of pressurised groundwater. Generally, the groundwater condition in the region is characterized by high perched water levels exhibited by pockets of sand layers.

The presence of uncontrolled artesian conditions within the subsurface/foundation soils can loosen or soften subsurface/foundation materials as artesian water flows/travels through the subsurface/foundation soils/ and can cause intermittent soft spots in the wet silt seams, loose gravels or sand at various depths, as observed at the project site.

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 4 provides the advantages, disadvantages, risks and consequences of the foundation alternatives to support the proposed structure. Construction of the deep foundations (driven piles and caissons) should conform to OPSS.PROV 903, as amended by SP 109F57.

Table 4 - 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. Deep foundations embedded in artesian aquifers can be the cause of serious problems during, and after their installation, due to a disproportionately or post-construction soil loss in the case of driven displacement, which can in turn lead to settlement due to upward migration and eventual loss of eroded soil particles.	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	Major dewatering scheme is required to construct the footing under high groundwater table. Artesian conditions at the site may loosen soils below the footing causing settlement during construction. Higher risk of settlement and resulting distresses to bridges due to differential settlements.	Foundation cost relatively low, but more cost uncertainty due to deeper excavation and shoring requirements

Table 4 - 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 Ability to install through cobbles and boulders Can be installed with limited vertical clearance	High potential for construction difficulties in artesian conditions and cost overrun 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 of caisson to support axial or lateral loads. Considering the existing artesian groundwater conditions and the challenging construction procedures required to maintain stability of caisson excavation as a result of high risk of washout, complicated by potentially unstable, non-cohesive ground and the environmental concerns for potential use of slurry in an area surrounded by a wetland, caissons are not recommended at the site for supporting the proposed underpass.	Relatively high
Micropiles	Minimal disturbance and can be advanced through obstruction/boulders Can be drilled with grout to minimize the impact of artesian conditions	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, both the east and west abutments and centre pier may be supported on HP 310 x 110 steel piles driven to bedrock elevations ranging from El. 297.8 to El. 292.2. This 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 the performance of piles because the loads required to produce detrimental deformation is anticipated to be larger than the recommended factored resistance at ULS.

Table 5 summarizes the approximate pile tip elevations and the length of piles that may be considered for design purposes at the east and west abutments and centre pier.

Table 5 - Approximate Pile Tip Elevation and Length for HP 310 x 110 Steel Pile

LOCATION	PILE TIP ELEVATION	PILE CUT-OFF ELEVATION	LENGTH (m)
West Abutment	292.2 ± 1.0	313.8	21.6 ± 1.0
Centre Pier	295.8 ± 1.0	308.6	12.8 ± 1.0
East Abutment	294.6 ± 1.0	313.9	19.3 ± 1.0

Considering occasional cobbles encountered below approximately El. 305.0 m, the pile tips may have to be reinforced to drive the piles through layers consisting of cobbles and gravels.

The coefficient of horizontal subgrade reaction, k_s (kN/m^3), may be computed using the following equation for cohesionless soils (Terzaghi, 1955), and the n_h values given in Table 6.

$k_s = (n_h)\ z/b$; where n_h = coefficient related to soil density; z (m) = depth; b (m) = pile width

Table 6 - Coefficient n_h Values for Computation of Horizontal Subgrade Reaction

LOCATION	SOIL TYPE	APPROXIMATE ELEVATIONS		n_h VALUES (kN/m^3)	
		FROM	TO	DRY/MOIST	SATURATED
West Abutment	Loose Sand	313.8	310.8	1,000	750
	Very Loose to Loose Silty Sand/Sandy Silt Till	310.8	301.5	1,200	900
	Compact Silty Sand to Sandy Silt Till	301.5	292.2	4,900	3000
Center Pier	Compact Silty Sand to Sandy Silt Till	308.2	295.8	4,900	3000
East Abutment	Loose Sand	313.9	310.9	1,000	750
	Loose Silty Sand to Sandy Silt Till	310.9	306.5	1,200	900
	Compact Silty Sand to Sandy Silt Till	306.5	294.6	4,900	3000

However, for detail design, the lateral capacity of piles should be determined by load-displacement (p-y) curve analyses to model the non-linear behavior of soils. This method allows determining pile top deflections and bending moments of laterally loaded piles. The p-y curves would be 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 material type to determine pile top deflections.

It is noted that integral abutments will be used to support the proposed structure. Since integral abutments are subjected to horizontal movements induced by temperature cycles, the design of the proposed underpass requires the need to focus on methods that reduce restraints. To accommodate the movement of the integral abutment system, two concentric corrugated steel pipes (CSP), extending at least 3 m below the bottom of the abutment footing, be placed around the steel H-piles to create an annular space. The inner CSP of 600 mm in diameter should be filled with sand meeting the requirements of Granular B Type I, with a maximum particle size of 37.5 mm. The outer CSP will have a diameter of about 800 mm so that the inner CSP and the piles will have enough unrestricted space for movement. The outer CSP is pushed in place and is not allowed to move.

The MTO guideline for integral abutment bridges (1996) suggested the use of a single row of vertical steel H-piles to support abutment footings. The guideline recommends battering the end piles at a minimum of 1H:10V in transverse direction to provide additional lateral resistance.

In addition, any embankment backfill should be placed before the piling work begins at the abutment so that no allowance for negative shaft friction will be required during pile design.

Artesian conditions were observed from a zone of medium to coarse silty sand and/or the upper, fractured part of the bedrock in Borehole 35-617-06 and the artesian head stabilised at about 1.2 m (El. 309.9) above the ground level. Artesian conditions were also observed in Borehole 35-617-10 with a head of around 1.4 m (El. 310.7) and in some water wells of the region. Deep foundations under the artesian conditions may cause problems with pile stability during and after installation. This is due to the high risk of soil loss resulting from the foundation elements inadvertently acting as wicks or openings potentially resulting in an upward migration of soil particles carried by groundwater following the newly introduced pressure relief pathway to the ground surface.

In general, considering the artesian and pressurized groundwater conditions at the site, use of drainage blankets may be considered to control any flow of water during pile installation and prevent loss of fines from the subsurface. Alliteratively, dewatering can be performed to lower the water pressure beneath the confining layer (bedrock level) using deep wells and/or well point systems. Using these systems, the water pressure can be reduced to a level where it is less than the total weight of the confining layer, until at least the installation is completed.

Further, the construction of the proposed underpass may require the replacement of existing non-structural culverts near the abutments. Generally, H-piles are low displacement piles, and the culvert foundations will not be significantly affected by pile driving. However, if offset is less than 3 m, it is advisable to regularly check the adjacent culvert foundations during pile installation.

8.2.2 Option 2: Shallow Foundation – Spread Footings

The centre pier of the proposed underpass may be supported on spread footings placed at or below El. 306.0. The following geotechnical resistances may be assumed for minimum of 2.5 m wide spread footings placed at or below El. 306.0, in the upper part of the sandy silt/silty sand till.

- Factored Geotechnical Bearing Resistance at ULS = 300 kPa
- Geotechnical Bearing Resistance at SLS = 200 kPa

During construction, it is advisable to verify the geotechnical bearing resistance based on the conditions encountered at the site. The base of the footings should be inspected and the presence of compact to very dense silty sand/sandy silt till along the entire length of the footing should be confirmed. Any loose or soft subgrade should be excavated (removed) and replaced with competent material. Further, the geotechnical resistances are dependent on the size of the footings and if considerations are made to use different footing dimensions, then the ULS and SLS geotechnical resistances should be reviewed. In addition, 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 300 kPa is expected in the order of 20 mm to 25 mm and the associated differential settlement is expected to be in the range of 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 sandy silt to silty sand till subgrade is susceptible to disturbance from construction traffic and/or any ponded water. To limit the degradation, it is suggested that a minimum of 100 mm 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 to 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 average depth of groundwater level of El. 308.4 at the project site, and the recommended elevation of El. 306.0 for the placement of footing, the shallow foundation option may not be cost effective due to the need to excavate by dewatering the 2.4 m high groundwater.

8.2.3 Option 3: Caissons

The proposed centre pier of the underpass may be supported on caissons founded in bedrock at about El. 292.2 to El. 290.4. Caissons socketed three times the diameter into the bedrock may be designed assuming the geotechnical resistances given in Table 7.

Table 7 - Factored Geotechnical Resistance at ULS

CAISSON DIAMETER	CAISSON TIP ELEVATION	FACTORED GEOTECHNICAL RESISTANCE AT ULS (kN)
1200 mm	292.2±1.0	5000
1500 mm	291.3±1.0	7500
1800 mm	290.4±1.0	10000

The presence of artesian groundwater conditions and high perched water levels exhibited by pockets of sand layers can be problematic to caisson construction, leading to heave effects that could compromise end bearing capacity and lead to excessive settlement under the design loads. Hence, a temporary liner or casing will be required to install the caissons. Heave can be mitigated by ensuring that the casings are topped up with water to maintain a positive pressure head throughout construction. If the artesian pressure leads to a flowing situation, extending the casings above ground level or the use of support fluids to increase the weight of the fluid inside the casing, may be effective in mitigating potential heave effects at the base of the pile.

Lateral resistances at ULS and at SLS provided in Table 8 may be utilized for the design of the caissons socketed at least three times the diameter of the pile, into the bedrock.

Table 8 - 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

Micropile foundation can be considered as an option to support the proposed underpass. These piles can be installed to the required depths with minimal disturbances and can be advanced through/past obstructions including cobbles and boulders. The actual embedment or bond length of these micropiles 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.

Micropiles 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.

At the project site, micropiles can be installed with grout to minimize the impact of artesian conditions. However, the lateral load-carrying capacity of individual vertical micropiles is limited compared to driven piles and may not be as effective as driven steel H-piles.

9. LATERAL EARTH PRESSURES

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

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 provide positive drainage of the granular backfill in accordance with the requirements of 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 9 may be assumed. The parameters and unfactored values given in Table 9 assume that the ground surface behind the structure is horizontal. If the retained ground is sloping, the lateral earth pressure 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).

Table 9 - 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

A minimum compaction surcharge of 12 kPa should be included in the calculation of lateral earth pressure for structural design of the wall stem, based on Clause 6.12.3 and Figure 6.8 of the CHBDC (2019). Surcharge loads, if present, should be accounted for in the design, as required.

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)).

The use of integral abutment at the project site may induce lateral yielding of the abutment stem and retaining walls, and active earth pressures may be used in the design of the structure. The use of active pressure within the backfill, assuming an unrestrained structure, should be done as presented in Section C6.12 and Table C6.12 of the Commentary (S6.1-14) to the CHBDC (2019).

10. APPROACH EMBANKMENTS

10.1 Bearing and Embankment Stability

PML received draft design cross sections and profiles of the road alignments in the MBI area from AECOM, in an email sent on March 09, 2021. Review of these cross sections and profiles indicated that the work at the proposed underpass will involve the construction of about 9.1 m to 9.2 m high approach embankments on the west and east abutments, respectively. The cross sections also show that the embankment bases will be about 50 m wide, and this will require widening of the roadway on both the south and north sides by up to 20 m from the edge of the existing shoulder. Generally, it is anticipated that the proposed embankment construction and the accompanying roadway widening at the approaches will involve the excavation of unsuitable materials such as topsoil, peat, soft and loose soils, and the replacement of the unsuitable subgrade material with appropriate fill to achieve the required grade. In addition, the embankment construction may require drainage works and erosion control measures.

Considering the heights of the embankments, the fills are expected to impose a load of 180 kPa to 190 kPa at the subgrade level, assuming a compacted fill density of 20 kN/m^3. The ultimate bearing capacities of the soil layers encountered at the subgrade level beneath the peat, soft and loose soils, are higher than the estimated loads, and no failure against bearing is anticipated.

Further, slope stability analyses were conducted to assess the performance of the new approach embankments constructed with 2H:1V side slopes or flatter. 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.

The factors governing the stability and performance of new high fill embankments include the geometry of proposed high fills, the type and thickness of embankment fill material, the thickness and extent of peat, organic, soft and loose materials within the footprints of embankments, thickness and engineering properties of foundation soils, surcharge loads, and groundwater conditions.

For the purpose of conducting slope stability analyses for this assignment, fill slope geometries were obtained from preliminary cross sections provided by AECOM. The fill slopes are assumed to be constructed with 2H:1V side slopes or flatter. Further, use of a well compacted granular fill was assumed so that post-construction settlement of the embankment itself is negligible. It was also assumed that any peat/organic material, existing asphalt, and near-surface soft and loose soil layers beneath the footprint of new embankments will be removed prior to construction. The subsurface and groundwater conditions were obtained from boreholes drilled in the area. Further, a surcharge load of 12 kN/m^2 was considered in all analyses to account for a typical highway traffic load.

Since the subsurface encountered in boreholes consisted of soils of widely differing permeabilities, the more permeable soils were generally considered as drained whereas the less permeable soils were taken as undrained. With this assumption, the drained soils were treated in terms of effective stresses and the undrained soils were analyzed using total stress approaches.

A summary of the assumed shear strength parameters for soil layers encountered in boreholes drilled at or near the proposed locations of high fills are given in Table 10. 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 10 – Soil Strength Parameters

MATERIAL	SOIL PROPERTY			
	BULK UNIT WEIGHT (kN/m^3)	INTERNAL FRICTION ANGLE (ϕ')	DRAINED SHEAR STRENGTH (c') kPa	UNDRAINED SHEAR STRENGTH (Cu) kPa
Pavement Fill	21	32	-	-
Embankment Fill	20	30	-	-
Peat	10	20	1	5
Compact Silty Sand/Sandy Silt (Till)	19	28	-	-
Clayey Silt (Till)	19	20	5	30
Very Dense Silty Sand/Sandy Silt (Till)	19	34	-	-

On the west side, embankment stability with respect to rotational shear failure extending through the new approach embankment and out into the proposed widening, was analyzed. The stability analyses were performed for a "typical section" near the abutment or at Sta. 9+960 and considered an embankment with a height of 9.1 m ("likely case") and 10 m ("worst case") scenario.

A layer of fine fibrous to amorphous peat/organics, about 3 m thick, was encountered in the boreholes. Strength parameters of peat/organics were estimated based on experience. Generally, amorphous peat has lower shear strength parameters than fibrous peat. The peat outside of the existing embankment was assumed to be in an undrained condition. The drained or long-term condition reflects no excess pore water pressure present within the peat. Thus, its strength is represented by the angle of internal friction. However, the undisturbed peat strength was assumed to be controlled by cohesion or undrained shear strength. A groundwater level near the surface was assumed as a “likely case” because of the presence of wetlands in the area.

On the east side, the stability analyses considered the section through Sta. 10+040, where the thickness of the proposed embankment is about 9.2 m. Like the case on the west side, an analysis was also conducted for a 10 m high approach embankment. The road widening and embankment construction on both the south and north sides of Wellington Road 34 at this location will be on top of native silt sand and gravelly sand materials. The stability analyses also involved the topsoil, peat (organic), and soft and loose soils beyond the footprint of the embankment. The soil strength parameters assumed for these materials and the embankment fill are summarized in Table 10. A groundwater level near the ground surface was used as a “likely case” for the analyses.

The results of the slope stability analyses are given in Drawings 35-617-3, 35-617-4, 35-617-5, and 35-617-6 in Appendix G. For the west approach embankment, the Factor of Safety (FS) value for a rotational slip plane that passes through a 9.1 m high fill was found to be 1.5. This value is equal to the 1.5 that is often used to design highway approach embankments with no risk of slope instability. For a 10 m high embankment, the FS value reduced to 1.4. For the east approach embankment, the FS values were 1.5 and 1.3 for 9.2 m and 10 m high embankments, respectively.

The results indicated that deep-seated failures are not anticipated for embankments constructed on both west and east sides of the proposed underpass with 2H:1V side slopes or flatter and as high as 9.2 m, if any peat, organic, and soft and loose materials within the footprints of the embankments are removed, and the embankments are constructed of well compacted granular fill.

However, although embankment instability typically occurs either during embankment construction or shortly after, it can also happen over the long-term. Embankment instability occurs in the form of either a planar slide extending out into the adjacent area, or a deep-seated, rotational failure plane extending through the embankment and the underlying soft soils. Deep seated failure plane development can occur either quickly with the formation of a large head scarp and heaving of the adjacent ground near the toe of the slope, or very slowly in a creeping type failure, where the failure plane may move only a small amount over a long period of time. Hence, proper level of visual inspection is required at all times. Visual observation is especially required if the embankments are higher than the proposed heights (9.1 m and 9.2 m). If failure is expected based on visual observation, then instrument monitoring using inclinometers should be implemented.

Further, the assessment of the stability of the approach embankment slopes should be reviewed and confirmed during design and construction, based on the actual subsoil conditions encountered within the proposed embankment footprint. Mitigation measures to improve slope stability include use of lightweight fill materials, wick drains, preloading (surcharging) and staged construction or a combination of these options, which will also control magnitude and time rate of settlements.

The earth fill slopes should be protected against surface erosion by sodding and vegetation. Refer to OPSS.PROV 803 and OPSS.PROV 804 for time constraints and the required type of seed and mulch.

10.2 Embankment Settlement

In addition to stability analyses, the potential for the occurrence of settlement of the new approach embankments was also assessed. Generally, since any peat, organic and soft materials encountered within the footprints of the new embankment will be removed, the proposed embankments at the east and west approaches are expected to induce settlement of the founding soil in the order of 20 mm to 30 mm. In addition, the fill itself may be expected to settle by 0.5% to 1.0% (45 mm to 90 mm) of the fill height (9.2 m), 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 construction. To mitigate post-construction settlements, the paving of the road on both approaches may need to be delayed by two to four weeks after the placement of the fill to the designed grade of the embankment.

Generally, any peat, organic or deleterious material, spongy or soft area observed under the plan limits of both approach embankments of the proposed underpass should be sub-excavated before the placement of any fill and the exposed subgrade surface should be proof-rolled and backfilled with acceptable fill material. If possible, the removal of the peat, organic and soft or loose soils beyond the footprints of the new embankments is recommended where the Right of Way (ROW) permits. In addition, the placement of the fill beyond the edge of the existing roadway may need preloading for as long as possible, but at least for a period of one month prior to paving of the new road (immediately preceded by final fine-grading of the Granular A pavement base to achieve the design grade).

11. SEISMIC CONSIDERATIONS

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

The seismic site classification is based on the conditions of soils encountered in the upper 30 m of the subsurface. Based on the average SPT "N" values of the sandy silt to silty sand materials and the underlying till deposit and 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 affecting the underpass is low. Hence, no seismic design considerations are anticipated for this site.

12. FROST PROTECTION

Based on OPSD 3090.101, the frost penetration depth for the region is approximately 1.2 m. All pile caps or footings of shallow foundations 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.

13. CONSTRUCTION CONSIDERATIONS

13.1 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. Generally, 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 11 may be used for the design of roadway protection systems at the project site. However, additional boreholes will be required during design to confirm these values and to ensure obstructions don't exist along the alignment of the roadway protection system. OPSS.PROV 539 and DBSP0539 also call for monitoring of the roadway protection system by the contractor to check the horizontal and vertical displacements of the roadway.

Table 11 - 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)
311.0	305.8	Silty Sand to Sandy Silt (Fill)	18	30	-
305.8	292.2	Silty Sand to Sandy Silt Till	20	34	-

13.2 Excavation

The GA drawing indicates that the depth of the excavation required to facilitate construction of the pile cap at the proposed pier location may not exceed 2.6 m below existing grade of Highway 6. In case the shallow foundation option is preferred, the excavation depth to the recommended founding elevation (El.306.0) will be about 4.8 m below the grade of Highway 6 (El. 310.8).

All excavations should be carried out in accordance with the Occupational Health and Safety Act (OHSA) and MTO Regulations for Construction Projects. The existing compact to dense silty sand fill and native soil should be considered as Type 3 soil in accordance with OHSA. As per OHSA regulations, the open-cut excavation procedures are governed by soils with the highest soil type number. Temporary excavation slopes of 1H:1V over the full depth of excavation or flatter slopes should be provided assuming that adequate drainage or dewatering measures are in place. Temporary shoring systems will be required if such slopes cannot be provided. This is especially true if the groundwater levels at the structure location remain high during construction. Below the groundwater level, caving is anticipated and thus shoring is likely be required.

Like roadway protection system, the contractor is responsible for the design of temporary shoring for excavation walls. It is anticipated that excavation walls will be shored using trench boxes, 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 behind the shoring.

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 at structure location 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 should be verified. All underground utilities that might be exposed and become unsupported as a result of the excavation should be properly supported and managed to avoid potential damage.

13.3 Groundwater Control

The stabilized groundwater level measured in a monitoring well installed in a borehole drilled on the west side of the proposed underpass (Borehole 35-617-03A) was at El. 308.4. Recent measurements in this well indicated groundwater level at El. 308.2. The measurement of groundwater level in a well installed in Borehole 35-617-09 completed in 2021 showed a stabilized groundwater level at El. 308.6. Generally, for design purposes, the stabilized groundwater level may be taken as El. 308.4.

At the structure location, all excavations may have to be carried out under wet conditions because of high groundwater table. For construction in the dry, groundwater should be lowered a minimum of 0.5 m below the base of excavation (proposed footings) using different techniques of dewatering. Dewatering could be carried out by oversize excavations and sump pumping, pumping of well points or deep wells, sheet pile cofferdams or a combination of these. The contractor (design-builder) should be responsible for selection, design, installation and performance of the dewatering system. The dewatering system should be designed to conform to the requirements of OPSS.PROV 517. In addition to this standard specification, the inclusion of the Non-Standard Special Provision for Dewatering Structure Excavations (NSP FOUN0003) into contract documents should be considered.

The use of deep foundations is expected to penetrate artesian groundwater conditions at the project site, and a drainage blanket, possibly in combination with a geotextile, should be considered for placement beneath the pile caps to prevent the migration of fines that may be transported along the piles or caisson liner during and after construction. Alliteratively, dewatering can be performed to lower the water pressure beneath the confining layer (bedrock level) using deep wells and/or well point systems. Using these systems, the water pressure can be reduced to a level where it is less than the total weight of the confining layer, until at least the installation is completed. Appropriate recommendations should be developed during the detail design phase of the project along with NSSP, as appropriate, for construction.

Further, caissons constructed in granular subsoils subjected to unbalanced hydrostatic head will require special measures to prevent loss of ground, boiling of the base or basal heave. Such special measures could include the use of drilling mud and tremie concreting techniques.

If pumping of groundwater at volumes greater than 50,000 L/day and less than 400,000 L/day is required during construction, 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 (sheet pile or a sand bag cofferdam), 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. In general, the actual rate of groundwater taking will be a function of the final design, time of year, and the contractor’s schedule, equipment, and techniques, and this should be investigated further during detail design. At the time of processing, it is advisable to check any other requirements for taking water including municipality permits and conservation authorities.

13.4 Subgrade Preparation and Embankment Construction

The approach embankment fills should consist of well compacted and acceptable native or granular material. The topsoil/peat as well as any spongy or soft area and organic deposits observed within the base of the embankment should be removed before placing embankment fill. The depth and extent of stripped material should be determined during detail design of the underpass.

After stripping of soft and compressible materials to the specified depths, the exposed subgrade should be proof rolled to identify any soft and compressible materials requiring sub excavation. Excavated areas shall be backfilled with well compacted approved/acceptable fill. 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 embankment slopes due to 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 construction of the embankments. Erosion protection measures should be in accordance with OPSS.PROV 804.

Since the groundwater levels are at shallow depths, measures will be needed to prevent embankment slope instability due to seepage. These measures should be addressed during the detail design phase and may include an observational engineering approach in which specific areas requiring treatment are identified based on their performance after they have been constructed.

Design provisions to manage differential settlements of immediate approaches to the proposed structure should be addressed during the detail design phase of the project. Consideration should be given to surcharging/preloading to reduce settlements before paving.

13.5 Pile Installation and Obstructions

Construction of deep foundations (driven piles and caissons) should conform to OPSS.PROV 903, as amended by SP 109F57. For design-build projects, use of DBSP0903 (the design-build special provision replacement of OPSS.PROV 903) should be considered.

Generally, 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, if encountered, may pose installation challenges for driven piles and caissons. Based on the subsurface information at the site, the risk of encountering cobbles and boulders to the extent that they will significantly affect pile driving and caisson installation is assumed to be low. However, this aspect should be investigated further during the detail design phase of the project.

13.6 Soil Corrosivity

A sample from the silty sand to sandy silt till taken from Borehole 35-617-02 at a depth of approximately 4.8 m (El. 304.2 m), at the location of the west abutment of the proposed underpass, was tested for soil corrosivity and potential exposure of concrete to sulphate attack. Additional soil chemical corrosivity tests were also conducted recently using a sandy silt silty sand till sample from Boreholes 35-617-09 (centre pier), and a fill sample from 35-617-10 (east abutment). A summary of the results of chemical tests are provided in Table 3 given in Section 5.4 of Part A of this report.

As shown in Table 3, the sulphate concentration was reported to be 119 µg/g (0.012%) in Borehole 35-617-02, 23 µg/g (0.002%) in Borehole 35-617-09 and 570 µg/g (0.057%) in Borehole 35-617-10. 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 proposed structure is considered to be negligible.

However, a resistivity value of less than 2000 ohm-cm is generally considered corrosive for soil in contact with steel. The resistivity values provided in Table 3 for the sandy silt and silty sand till are less than 2000 ohm-cm, and indicates a moderately corrosive environment for buried metal.

The pH values in Table 3 are 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.

14. CLOSURE

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

Yours very truly

Peto MacCallum Ltd.

LICENSED PROFESSIONAL ENGINEER
L. A. YIMAM
100500748
Oct 14/2021
PROVINCE OF ONTARIO

Lul Yimam, PhD, P.Eng.
Senior Engineer, Geotechnical Services

LICENSED PROFESSIONAL ENGINEER
G. G. UWIMANA
October 14, 2021
PROVINCE OF ONTARIO

Geoffrey Uwimana, MEng., P.Eng.
Discipline Head, Geotechnical Services
MTO Designated Principal Contact

LY/GU/:ly-nk

APPENDIX F

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

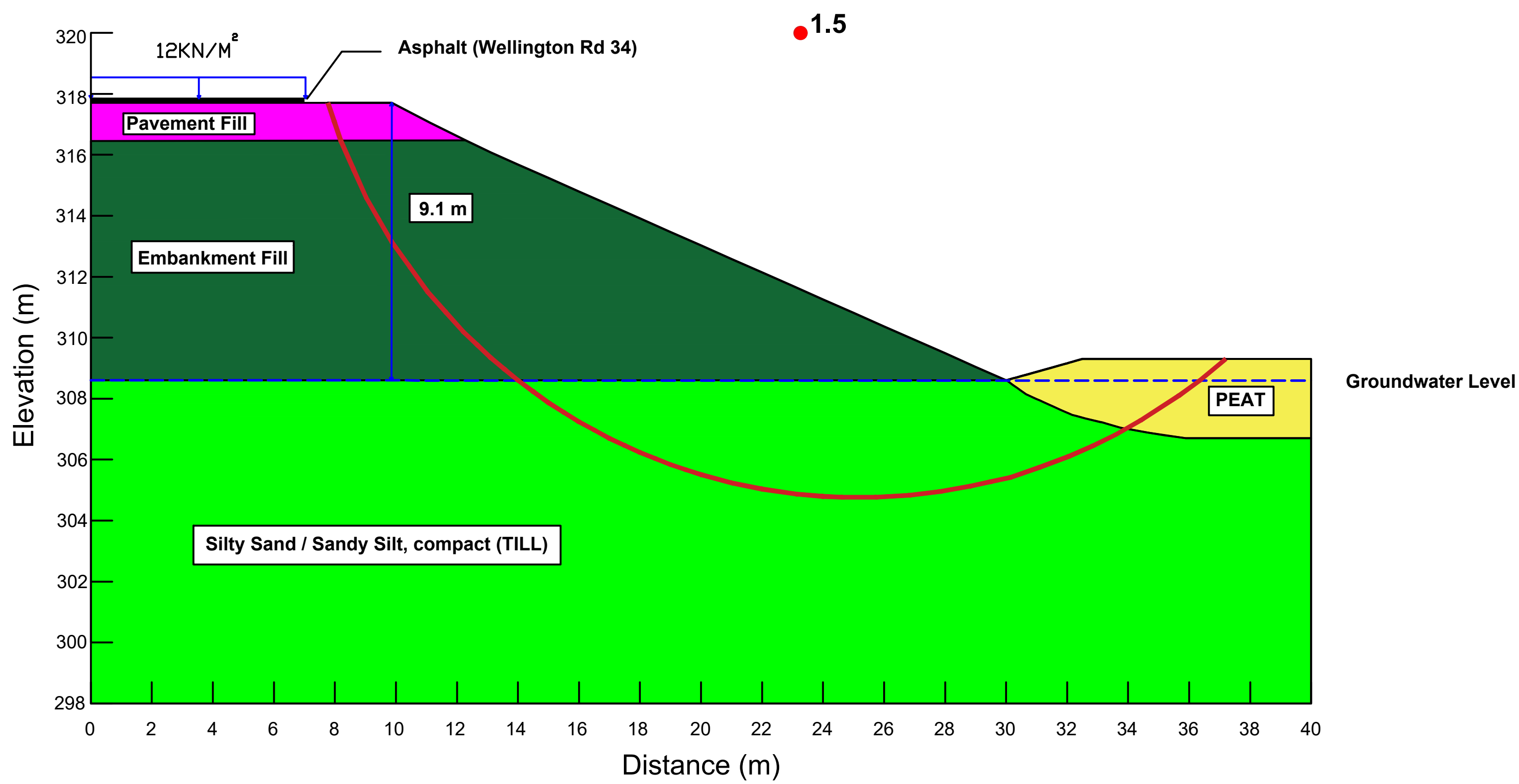
DOCUMENT	TITLE	REVISION DATE
OPSS.PROV 903	Construction Specification for Deep Foundations	April, 2016
SP 109F57	Special Provision (Amendment) to OPSS 903	January, 2020
DBSP0903	Design-Build Special Provision (Replacement) to OPSS 903	August, 2020
OPSS.PROV 1010	Material Specification for Aggregates – 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	August, 2021
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
OPSD 202.010	Embankment Construction using Excess Material Outside of Earth or Rock Fill	November, 2010
OPSD 3090.101	Foundation, Frost Penetration depths for Southern Ontario	November, 2010
OPSS.PROV 539	Construction Specification for Temporary Protection Systems	November, 2014
DBSP0539	Design-Build Special Provision (Replacement) to OPSS 539	August, 2020
SP 105S09	Amendment to OPSS 539	March, 2018
OPSS.PROV 517	Construction Specification for Dewatering	November, 2016
OPSS.PROV 206	Construction Specification for Grading	November, 2014

APPENDIX G

Results of Approach Embankments Slope Stability Analyses

SOIL STRENGTH PARAMETERS

PROPERTY \ MATERIAL	ASPHALT	PAVEMENT FILL	EMBANKMENT FILL	PEAT	SILTY SAND / SANDY SILT, COMPACT (TILL)
Bulk Unit Weight (kN/m^3)	–	21	20	10	19
Internal Friction Angle (°)	–	32	30	20	28
Drained Shear Strength, (Cohesion) (kPa)	–	–	–	1	–
Undrained Shear Strength, (Cohesion) (kPa)	–	–	–	5	–



Wellington Road 34 Proposed Underpass (SITE NO. 35X-0617/B0)
West Approach Embankment (STATION 9+960) Slope Stability Analysis
Embankment Height = 9.1 m
MIDBLOCK INTERCHANGE AREA, HIGHWAY 6 & HIGHWAY 401 IMPROVEMENTS
CITY OF GUELPH, ONTARIO

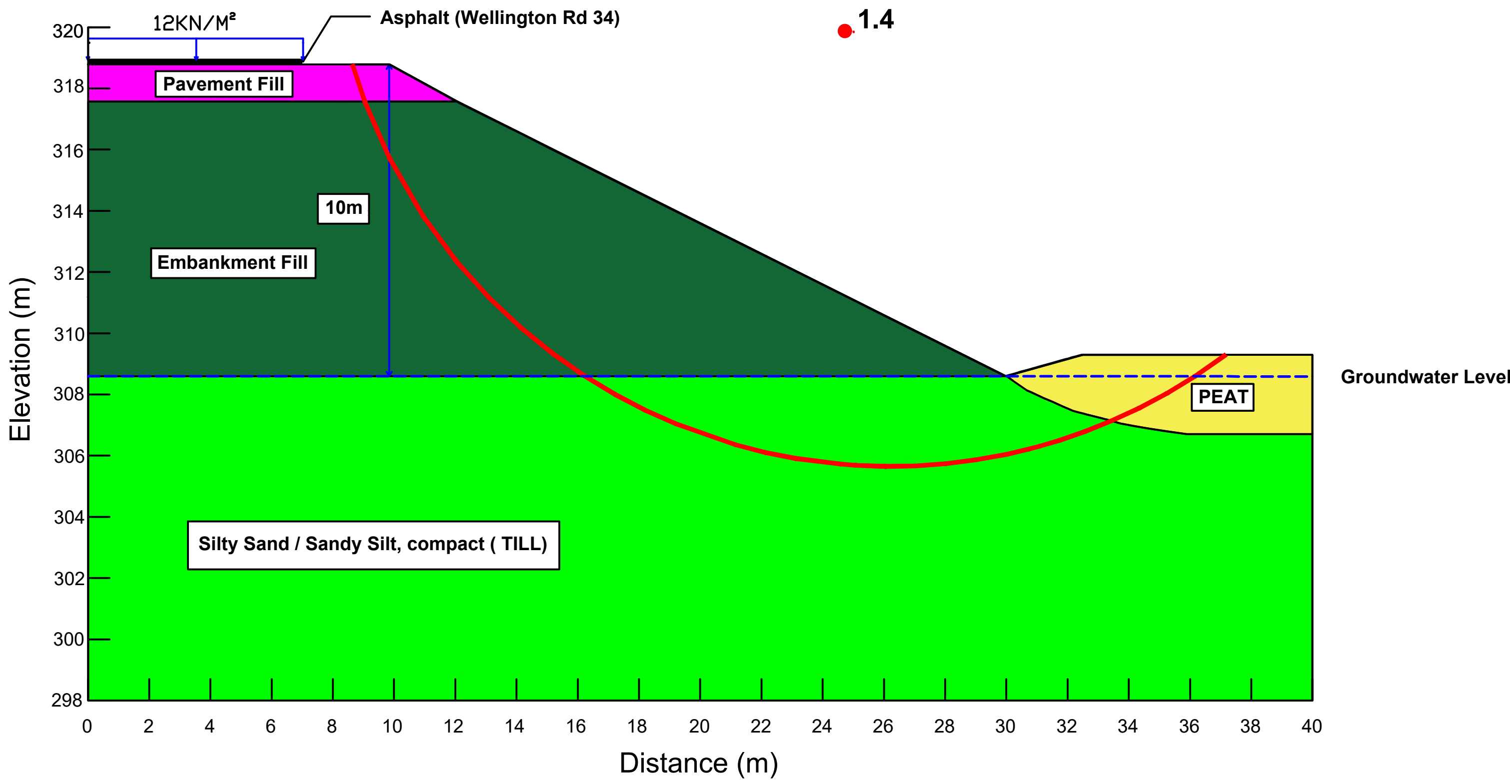
CONTRACT NO:	
G.W.P.:	3059-20-00
HWY NO.:	6
DISTRICT:	31

Peto MacCallum Ltd.
CONSULTING ENGINEERS

DRAWN:	P.J.	JOB NO.	DATE	DRAWING NO.
CHECKED:	L.Y.	17TF006A	OCT 01, 2021	35-617-3
APPROVED:	G.U.			

SOIL STRENGTH PARAMETERS

PROPERTY \ MATERIAL	ASPHALT	PAVEMENT FILL	EMBANKMENT FILL	PEAT	SILTY SAND / SANDY SILT, COMPACT (TILL)
Bulk Unit Weight (kN/m^3)	–	21	20	10	19
Internal Friction Angle (°)	–	32	30	20	28
Drained Shear Strength, (Cohesion) (kPa)	–	–	–	1	–
Undrained Shear Strength, (Cohesion) (kPa)	–	–	–	5	–



Wellington Road 34 Proposed Underpass (SITE NO. 35X–0617/B0)
West Approach Embankment (STATION 9+960) Slope Stability Analysis
Embankment Height = 10 m
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

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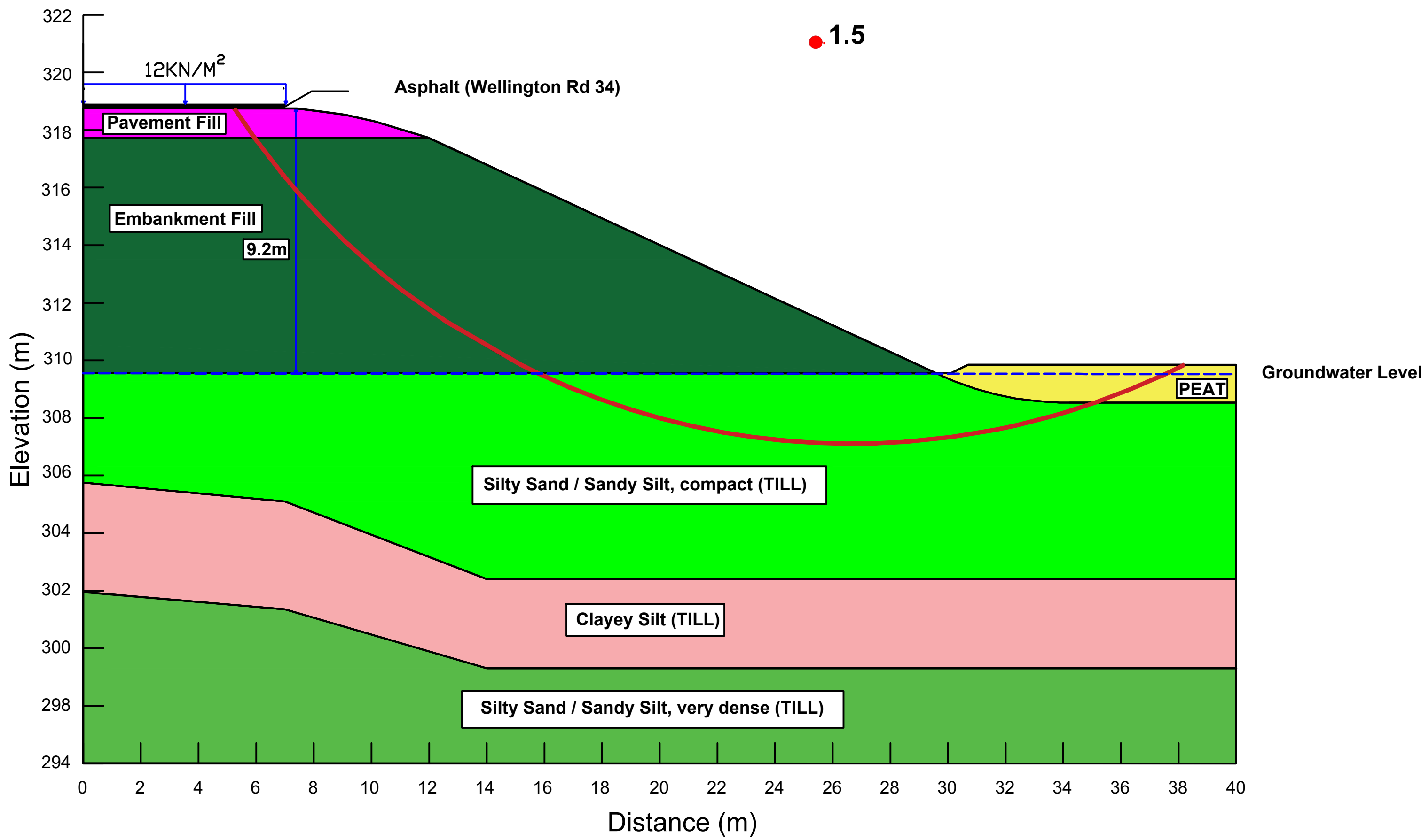
CHECKED: L.Y.	17TF006A	OCT 01, 2021	35–617–4
APPROVED: G.U.			

SOIL STRENGTH PARAMETERS

PROPERTY / MATERIAL	ASPHALT	PAVEMENT FILL	EMBANKMENT FILL	PEAT	SILTY SAND / SANDY SILT, COMPACT (TILL)	CLAYEY SILT (TILL)	SILTY SAND / SANDY SILT, VERY DENSE (TILL)
Bulk Unit Weight (kN/m^3)	–	21	20	10	19	19	19
Internal Friction Angle (°)	–	32	30	20	28	20	34
Drained Shear Strength, (Cohesion) (kPa)	–	–	–	1	–	5	–
Undrained Shear Strength, (Cohesion) (kPa)	–	–	–	5	–	30	–



Wellington Road 34 Proposed Underpass (SITE NO. 35X–0617/B0)
East Approach Embankment (STATION 10+040) Slope Stability Analysis
Embankment Height = 9.2 m
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

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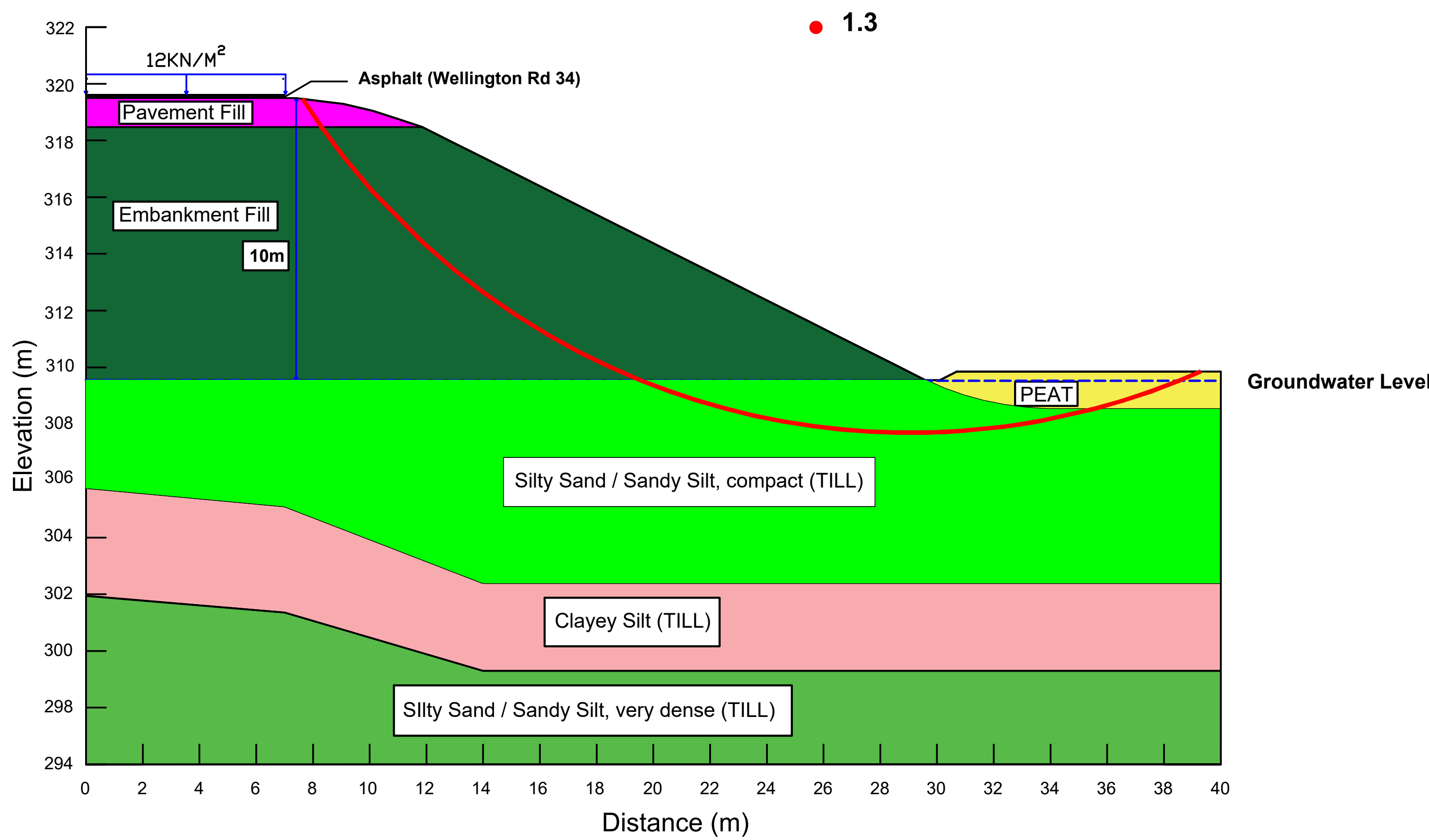
CHECKED: L.Y.	17TF006A	OCT 01, 2021	35–617–5
APPROVED: G.U.			

SOIL STRENGTH PARAMETERS

PROPERTY / MATERIAL	ASPHALT	PAVEMENT FILL	EMBANKMENT FILL	PEAT	SILTY SAND / SANDY SILT, COMPACT (TILL)	CLAYEY SILT (TILL)	SILTY SAND / SANDY SILT, VERY DENSE (TILL)
Bulk Unit Weight (kN/m^3)	–	21	20	10	19	19	19
Internal Friction Angle (°)	–	32	30	20	28	20	34
Drained Shear Strength, (Cohesion) (kPa)	–	–	–	1	–	5	–
Undrained Shear Strength, (Cohesion) (kPa)	–	–	–	5	–	30	–



Wellington Road 34 Proposed Underpass (SITE NO. 35X-0617/B0)
East Approach Embankment (STATION 10+040) Slope Stability Analysis
Embankment Height = 10 m
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.
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CHECKED:	L.Y.	17TF006A	OCT 01, 2021	35-617-6
APPROVED:	G.U.			