Peto MacCallum Ltd.
CONSULTING ENGINEERS

FOUNDATION INVESTIGATION AND DESIGN REPORT

for

GILLES CREEK BRIDGE REPLACEMENT
SITE NO. 39E-0006/B0
HIGHWAY 579 – STATION 19+473
TOWN OF COCHRANE, ONTARIO
G.W.P. 5267-11-00
W.P. 5368-11-01
LATITUDE AND LONGITUDE: 49.11283, -81.27235

THIS REPORT SUPERSEDES THE PREVIOUS FOUNDATION INVESTIGATION AND DESIGN REPORT, DATED JULY 3, 2019.

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PML Ref.: 18TF002A
Index No.: 023FIR and 024FDR
GEOCRES No.: 42H-81
August 1, 2019

PART A - FOUNDATION INVESTIGATION REPORT

for

GILLES CREEK BRIDGE REPLACEMENT
SITE NO. 39E-0006/B0
HIGHWAY 579 – STATION 19+473
TOWN OF COCHRANE, ONTARIO
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Gilles Creek Bridge Replacement
Site No. 39E-0006/B0
Highway 579 – Station 19+473
Town of Cochrane, Ontario
G.W.P. 5267-11-00, W.P. 5368-11-01

1. INTRODUCTION

The Ministry of Transportation Ontario (MTO) has retained Parsons Corporation (Parsons) as the Prime Consultant, to provide Detail Design services for the replacement of two (2) bridges on Highway 668 and one (1) bridge on Highway 579. Parsons retained Peto MacCallum Ltd. (PML) on behalf of MTO to provide geotechnical engineering services for the assignment. This assignment involves two (2) contracts assigned to be submitted as follows:

- Contract Package 1: Replacement of Deception Creek Bridge (Site No. 39E-169) and Smith Creek Bridge (Site 39E-014) on Highway 668.
- Contract Package 2: Replacement of Gilles Creek Bridge (Site No. 39E-006) on Highway 579

The geotechnical investigation work reported herein is part of Contract Package 2, to prepare detail design for the replacement of the existing Gilles Creek Bridge, located at the crossing of Gilles Creek and Highway 579. The investigation report for Contract Package 1 will be issued under a separate cover.

Pavement investigations were also carried out in conjunction with the foundation investigation and the pavement investigation report for the proposed structure location is issued under a separate cover.

The Terms of Reference and Scope of Work for the Foundation Engineering services are outlined in MTO Assignment No. 5017-E-0030, dated August 2017.

This report presents the factual findings from the foundation investigation carried out for the proposed replacement of the existing bridge located at the crossing of Gilles Creek and Highway 579 (Station 19+473) in the Town of Cochrane, Ontario.

The purpose of the investigation was to explore the subsurface conditions expected to influence the design of the replacement bridge and to aid the designer in selecting the suitable type of foundation to support the replacement structure.

2. SITE DESCRIPTION

The location of the existing bridge is approximately 26 km north of Highway 652. Highway 579 in the area of bridge site is slightly elevated from the natural topography, and accommodates two (2) lanes of vehicular traffic. The site is generally a flat area, with the exception of the highway embankments. Gilles Creek flows from west to east, almost perpendicular to Highway 579 and meanders toward Abitibi River, located approximately 850 m east of the existing bridge. The proposed bridge site is located within farm lands and is surrounded by long grass and forestation with mature trees and shrubs.

3. FIELD INVESTIGATION PROCEDURES

The fieldwork for the foundation investigation involved advancing eight (8) boreholes. The boreholes were drilled to depths ranging from 8.2 m to 15.8 m below the existing ground surface elevation (El. 235.8 to El. 239.7), and were terminated in competent glacial till deposit.

The staff of PML visited the site on February 27 and 28, 2019 to mark out the borehole locations. The respective utility companies cleared the underground services at the borehole locations. Public and private utility authorities were informed and all of the utility clearance documents were obtained before the commencement of drilling work.

PML staff used a portable GPS device to establish the location of boreholes in the field. Subsequently, Rugged Geomatics Inc. of Timmins, Ontario, under contract to PML, carried out the survey of the as drilled borehole locations and elevations, and provided the co-ordinates for locations in MTM Northing and Easting (MTM Zone – ON12). PML used the survey data provided by Rugged Geomatics Inc. for the preparation of this report. All elevations reported in this report are referred to Geodetic datum and expressed in meters.

The equipment used for drilling was owned and operated by Landshark Drilling Inc. (Landshark), of Brantford, Ontario. Landshark is a specialist drilling contractor and worked under the full time supervision of a PML field supervisor. Boreholes numbered GC-1 to GC-4, and ED-1 to ED-4 were drilled between April 1 and 10, 2019. The boreholes were advanced using a B57 track-mounted drilling rig equipped with 200 mm diameter hollow stem augers.

Boreholes GC-1, GC-2, ED-1, and ED-2 were drilled on the south side of Gilles Creek. Boreholes GC-3, GC-4, ED-3, and ED-4 were drilled on the north side of Gilles Creek. The borehole locations are shown on Drawing GC-1 provided in Appendix A.

Boreholes ED-2 and ED-3 were relocated from the original proposed locations due to heavy snowbanks, and presence of trees and overhead hydro wires.

Representative soil samples were recovered from the boreholes at 0.75 m intervals to a depth of 6.0 m and at 1.5 m to the depth of termination, using a conventional 51 mm OD split spoon sampler in accordance with the Standard Penetration Test (SPT) procedure. Standard penetration tests were conducted simultaneously with the sampling operation to assess the strength characteristics of the substrata. In addition, attempt was made to measure in-situ vane shear strength of clayey soil at depths where SPT values were below about 8 blows/300 mm, using a N-size (MTO) vane.

The groundwater conditions at the borehole locations were observed during the drilling by visual examination of the soil samples, sampler and drill rods as the samples were retrieved. In addition, water level measurements were taken in the open boreholes upon completion of drilling. Water levels were measured using a Solinst flat tape water level reader.

The water level in the creek was observed at approximate El. 235.0 during the fieldwork.

Upon completion of drilling, the boreholes were backfilled with bentonite/cement grout in accordance with the MTO guidelines and O.Reg. 903 for borehole abandonment procedures.

The recovered soil samples were returned to the PML laboratory for detailed visual examination, and index tests.

4. LABORATORY TEST PROCEDURES

Laboratory tests on representative SPT samples recovered during the fieldwork were conducted by the laboratory owned by PML, located in Toronto. The laboratory testing program included the following:

- Natural moisture content determinations (80)
- Grain size distribution analysis (26)
- Atterberg limit tests (18)

All laboratory tests to determine the index properties were performed in accordance with the MTO test procedures, which follow the American Society for Testing Materials (ASTM) standards, with the exception of hydrometer tests (LS-702). The results of the grain size distribution analyses are presented on Figures GS-1A, GS-1B, and GS-2, and the results of the Atterberg Limit tests are presented on Figures PC-1A and PC-1B, in Appendix A. All of the test results are summarized on the attached Record of Borehole Logs provided in Appendix A.

5. SITE GEOLOGY AND SUBSURFACE CONDITIONS

5.1 Site Geology

In general, the project area is located within the Abitibi Uplands of the James physiographic region of the Canadian Shield. The Quaternary Geology map published by the Ontario Ministry of Northern Development and Mines (MNDM), indicates that the surface conditions in the area of the bridge site consist of fine grained till deposits; predominantly silty clay to silt matrix. Based on the Bedrock Geology map (MRD126-REV1, 2011) published by the MNDM, the project area mainly consists of metasedimentary supercrustal rocks of the Superior Province.

A preliminary foundation investigation was carried out by others at this site between July and August, 2014, and the report is available in the MTO Geocres Library under Geocres No.42H-60. Borehole logs and location plan from previous investigation are included in Appendix B. In general, the subsurface conditions encountered during the exploration program conducted are consistent with the geology described in the preliminary foundation investigation and design report (FIDR) dated October 9, 2015. Based on the findings reported, the bedrock surface at this site may be intercepted at a depth of about 23.5 m (El. 216.0) below the existing ground surface.

5.2 Subsurface Conditions

The subsurface conditions encountered during the course of the investigation, together with the field and laboratory test results are shown on the attached Record of Borehole Sheets. The borehole locations and stratigraphic profile sections are shown on Drawings GC-1, GC-2, and GC-3. The boundaries between soil strata have been established at the borehole locations only. The boundaries of soil strata between and beyond the boreholes are assumed and may vary from location to location.

In general, the subsoil conditions immediately below the ground surface along the alignment of proposed detour consist of 100 mm topsoil and 300 mm pavement structure in the area of the existing road. The topsoil and pavement structure are underlain by 1.4 m to 3.0 m thick fill composed of layers of silty sand and clayey silt, which is followed by firm to hard clayey silt. The clayey silt layer is underlain by very dense silty sand/sandy silt till, which extends to the maximum borehole termination depth of 15.8 m below the existing ground surface. For classification purposes, the soils encountered at this site can be divided into six (6) distinct zones:

a) Topsoil
b) Pavement Structure
c) Silty Sand/Sandy Silt, Trace/Some Gravel (Fill)
d) Clayey Silt, Some Sand, Trace Gravel (Fill)
e) Clayey Silt/Silty Clay, Trace/Some Sand, Trace Gravel
f) Silty Sand, Trace/Some Gravel (Till)

5.2.1 Topsoil

A layer of topsoil, approximately 100 mm in thickness, was encountered in all of the boreholes (ED-1 to ED-4) drilled off-road along the alignment of proposed detour.

5.2.2 Pavement Structure

A pavement structure was encountered in Boreholes GC-1 to GC-4. This pavement structure consisted of 20 mm to 30 mm of surface treated pavement (PST) over 110 mm to 130 mm of granular base, followed by 130 mm to 150 mm of granular subbase.

5.2.3 Silty Sand/Sandy Silt, Trace/Some Gravel (Fill)

A silty sand/sandy silt fill layer was encountered immediately below the pavement structure in Boreholes GC-1 to GC-4 and in Borehole ED-3, it was encountered below the topsoil. In Borehole ED-1, it was about 0.8 m thick and encountered immediately below the clayey silt fill, which is described in Section 5.2.4 below. This silty sand/sandy silt fill layer extends to 1.5 m to 3.0 m (El. 238.0 to El. 234.0) below the existing ground surface.

The SPT 'N'-values in the silty sand/sandy silt fill layer ranged from 11 to 32 blows, indicating compact to dense state of compaction. The moisture content of the samples tested ranged from 3.1% to 19.1%.

5.2.4 Clayey Silt, Some Sand, Trace Gravel (Fill)

The silty sand/sandy silt fill layer in boreholes GC-1 to GC-3 is immediately followed by this clayey silt fill layer and in Boreholes ED-1, ED-2, and ED-4, it was encountered immediately below the topsoil. This layer extends to 1.5 m to 3.3 m (El. 237.1 to El. 234.7) below the existing ground surface.

The SPT 'N'-values in the clayey silt fill in Boreholes GC-1 to GC-4 ranged from 11 to 50 blows, indicating stiff to hard consistency. Whereas, the SPT 'N'-values in the clayey silt fill layer in Boreholes ED-1 to ED-4 ranged from 2 to 11 blows, indicating soft to stiff consistency.

The moisture content of the samples tested from this layer ranged from 16.6% to 46.9%. However, the moisture content of one sample following the topsoil was found to be at 70.8%.

5.2.5 Clayey Silt/Silty Clay, Trace/Some Sand, Trace Gravel

The fill layer in all of the boreholes is underlain by this clayey silt to silty clay deposit with varying proportions of sand and gravel. Occasional seams of silty sand and sandy silt layers ranging in thickness from 0.7 m to 1.5 m were also intercepted within this clayey silt deposit in four of the boreholes (GC-3, GC-4, ED-2, and ED-3) located near the north approach. This clayey deposit extends to depths ranging from 7.6 m to 10.3 m (El. 227.1 to El. 230.0) below the existing ground surface in boreholes where this deposit was fully penetrated. It was not fully penetrated in Borehole GC-4 to establish the thickness of this deposit.

In Borehole GC-1 located near the south approach of the bridge, this deposit was again intercepted at an approximate depth of 6.6 m below the ground surface and it was approximately 2.7 m thick.

The SPT ‘N’-values in this deposit to about El. 234.5 varies from 8 blows to 14 blows, indicating stiff consistency. The clayey silt deposit below El. 234.5 was generally found to be stiff to hard consistency with SPT values ranging from 14 blows to over 100 blows, with the exception of Borehole ED-1 where this layer remains firm to stiff (N-Values 4 to 16 blows) throughout the depth. Vane shear tests were attempted at depths where low N-values were observed. The test was performed only at two (2) locations within this deposit and the vane shear strength measured at both locations was 79 kPa.

The grain size distribution results of selected clayey silt samples from this deposit are provided on Figures GS-1A and GS-1B, and the results of Atterberg limits for the same samples are provided on Figures PC-1A and PC-1B in Appendix A.

The moisture content of the samples ranged from 12.7% to 37.9%. However, the moisture content of one sample was found to be at 4.2%. Sieve analysis tests were performed on seventeen (17) representative samples and the test results indicate that this deposit consists of none to 5% gravel, none to 16% sand, 17% to 61% silt, and 24% to 83% clay. Atterberg limits were performed on those seventeen (17) representative samples and the test results indicate liquid limit values, with the exception of one sample from Borehole ED-4, range from 24 to 48, plastic limit values from 14 to 20, and corresponding plasticity index values range from 9 to 28. Based on the test results, the clayey soil may be classified as clay of low to medium plasticity (CL/CI) in the Unified Soil Classification System (USCS), i.e., clayey silt/silty clay. The liquid limit of Sample 6 from Borehole ED-4 was 55 and the corresponding plastic limit was 22.

5.2.6 Silty Sand to Sandy Silt, Trace/Some Gravel (Till)

The clayey silt deposit in all of the boreholes, except GC-4, is followed by silty sand to sandy silt till deposit to the borehole termination depths ranging from 8.2 m to 15.8 m (El. 229.1 to El. 222.0) below the existing ground surface.

A layer of boulders was encountered in one of the boreholes located near the proposed location of the north abutment (GC-3) at a depth of 8.5 m (El. 230.7), which was cored to a depth of 9.4 m (229.8) below the existing ground surface.

The SPT 'N'-values in this deposit to about El. 228.0 varies from 6 blows to 16 blows, indicating loose to compact state of compaction. The till deposit below El. 228.0 was generally found to be very dense with SPT values ranging from 97 blows to over 100 blows.

The results of grain size distribution of selected samples from till deposit are provided on Figure GS-2 in Appendix A. The moisture content of the samples ranged from 7.8% to 24.8%. Sieve analysis tests were performed on eight (8) representative samples and the test results indicate that this deposit consists of 1% to 30% gravel, 28% to 72% sand, 9% to 65% silt, and 3% to 12% clay size particles.

5.2.7 Groundwater

Groundwater was encountered during drilling in five (5) of the boreholes (GC-1, GC-4, ED-1, ED-2, and ED-4) at depths ranging from 0.8 m (El. 237.8) to 8.0 m (El. 230.5) below the ground surface. Upon completion of drilling, groundwater was encountered in four (4) of the boreholes (GC-1, GC-4, ED-1, and ED-4) at depths ranging from 4.6 m to 9.9 m below the ground surface, elevations ranging from El. 235.1 to El. 228.7. The water level in the creek was observed at approximate elevation of El. 235.0 during the fieldwork.

Groundwater levels may fluctuate due to the influence of precipitation and seasonal change. The groundwater measurements were observed and measured prior to backfilling the boreholes. Groundwater levels are shown on the Borehole Logs in Appendix A.

Refer to Table 5.2.7 for groundwater level readings observed during and upon completion of drilling.

Table 5.2.7: Groundwater Level Readings

BOREHOLE NO.	GROUND SURFACE ELEVATION (m)	GROUNDWATER LEVELS OBSERVED DURING DRILLING		GROUNDWATER LEVELS UPON COMPLETION OF DRILLING		DATE OF READING
		DEPTH (m)	ELEVATION (m)	DEPTH (m)	ELEVATION (m)	
GC-1	239.5	8.0	230.5	4.6	234.9	April 01, 2019
GC-2	239.1	Not Encountered	--	Not Encountered	--	April 10, 2019
GC-3	239.2	Not Encountered	--	Not Encountered	--	April 02, 2019
GC-4	239.7	5.2	234.5	4.6	235.1	April 01, 2019
ED-1	238.6	0.8	237.8	9.9	228.7	April 05, 2019
ED-2	236.2	2.3	233.9	Not Encountered	--	April 08, 2019
ED-3	235.8	Not Encountered	--	Not Encountered	--	April 04, 2019
ED-4	237.3	4.9	232.4	6.1	231.2	April 03, 2019

6. CHEMICAL ANALYSIS

Six (6) representative soil samples were sent to SGS Canada Inc. located in Toronto, Ontario, which is accredited by Canadian Analytical Laboratory Association (CALA). The corrosivity test results provided by SGS are presented in Appendix A. A summary of the test results are presented in the Table 6.0.

Table 6.0: Summary of Corrosivity Results

Borehole ID	Sample No.	Corrosivity Index	Sulphide (%)	Soil Redox Potential (mV)	pH	Resistivity (Ohm-cm)	Conductivity (uS/cm)	Sulphate (µg/g)	Chloride (µg/g)
GC-1	4	1	<0.02	299	8.45	7760	129	4.4	11
GC-2	3	9	<0.02	304	7.85	1750	571	30	300
GC-3	2	1	<0.02	206	8.09	4900	204	6.2	23
GC-4	3	1	<0.02	274	7.97	6210	161	6.7	6.9
ED-2	3	4.5	0.02	197	8.04	4670	214	71	3.4
ED-3	4	4.5	0.02	253	8.16	3480	287	87	14

7. CLOSURE

Mr. M. Mohamed and Mr. F. Meng carried out the field investigations under the supervision of Mr. N. Rahman, P.Eng., Project Engineer, and Ms. N. Leong-Sem, EIT. Landshark Drilling Ltd. of Brantford, Ontario supplied the drilling equipment for the subsurface exploration. The laboratory testing of the selected samples was carried out in the PML laboratory in Toronto.

This report was prepared by Ms. N. Leong-Sem, B.Eng., EIT, and Mr. N. Rahman, P.Eng., Geotechnical Services and reviewed by Mr. M. Vasavithasan, M.Sc.Eng., P.Eng., Senior Engineer, Geotechnical Services. Mr. R. Ng, MBA, PhD, P.Eng., MTO Designated Principal Contact, conducted an independent review of the report.

Yours very truly,

Peto MacCallum Ltd.

LICENSED PROFESSIONAL ENGINEER
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Robert Ng, MBA, PhD, P.Eng.
Project Manager, and
MTO Designated Principal Contact

NL/NR/RN:nl-nk

APPENDIX A

Borehole Location Plan and Soil Strata Drawings GC-1, GC-2, and GC-3

Explanation of Terms Used in Report

Record of Borehole Sheets

Results of Grain Size Distribution Analyses – Figures GS-1A, GS-1B, and GS-2

Results of Atterberg Limit Tests – Figures PC-1A and PC-1B

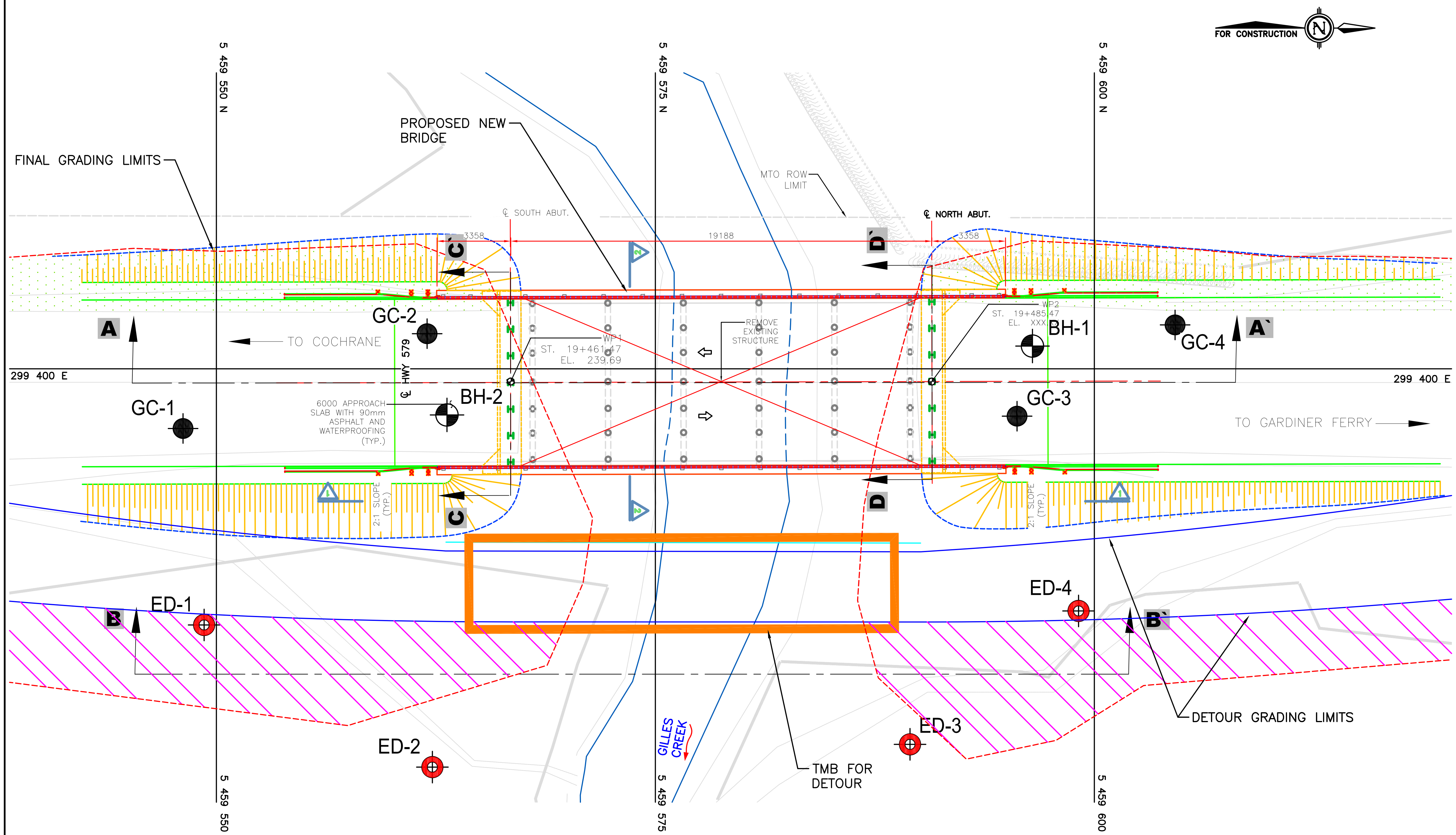
Results of Chemical Tests provided by SGS Canada Inc.

GWP No 5267-11-00
WP No 5368-11-01

GILLES CREEK STRUCTURE
HIGHWAY 579-STATION 19+473
BOREHOLE LOCATIONS

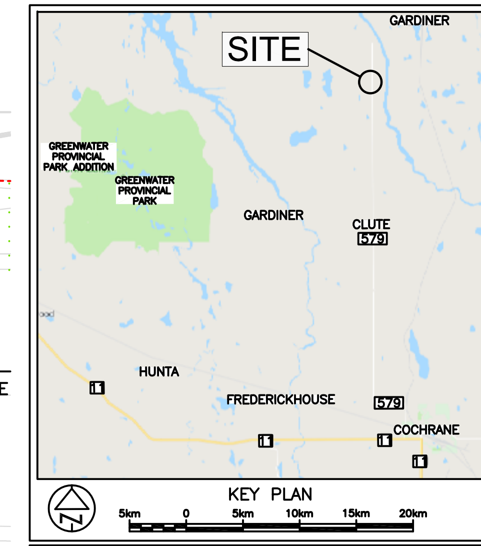
SHEET

Peto MacCallum Ltd.
CONSULTING ENGINEERS



PLAN

SCALE 2 0 2 4m



KEY PLAN

5km 0 5km 10km 15km 20km

LEGEND

- Foundation Borehole for Structure
- Foundation Borehole for Temporary Structure
- Previous Borehole (Geocres No. 42H-60)

BH No	ELEVATION	NORTHINGS	EASTINGS
GC-1	239.5	5 459 548.1	299 403.4
GC-2	239.1	5 459 562.0	299 398.0
GC-3	239.2	5 459 595.6	299 402.7
GC-4	239.7	5 459 604.6	299 397.5
ED-1	238.6	5 459 549.3	299 414.6
ED-2	236.2	5 459 562.3	299 422.7
ED-3	235.8	5 459 589.5	299 421.4
ED-4	237.3	5 459 599.1	299 413.8
PREVIOUS BOREHOLES (Geocres No. 42H-60)			
BH-1	239.1	5 459 596.25	299 398.01
BH-2	239.2	5 459 561.73	299 402.90

– NOTE –

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

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.
3. REFER TO DRAWING GC-2 FOR SOIL PROFILES A-A' AND B-B' AND DRAWING GC-3 FOR PROFILES C-C' AND D-D'.

REF Drawing: Gilles Creek-GA.dwg, dated March 2019.

REVISIONS		
08/01/19	NL	Geocres No. was corrected.
DATE	BY	DESCRIPTION

Geocres No. 42H-81

HWY No 579			DIST Northern
SUBM'D TC	CHECKED NR	DATE JULY 3, 2019	SITE 39E-0006/B0
DRAWN TC	CHECKED MV	APPROVED RN	DWG. GC-1

GWP No 5267-11-00
WP No 5368-11-01

GILLES CREEK STRUCTURE
HIGHWAY 579-STATION 19+473
SOIL STRATA

SHEET

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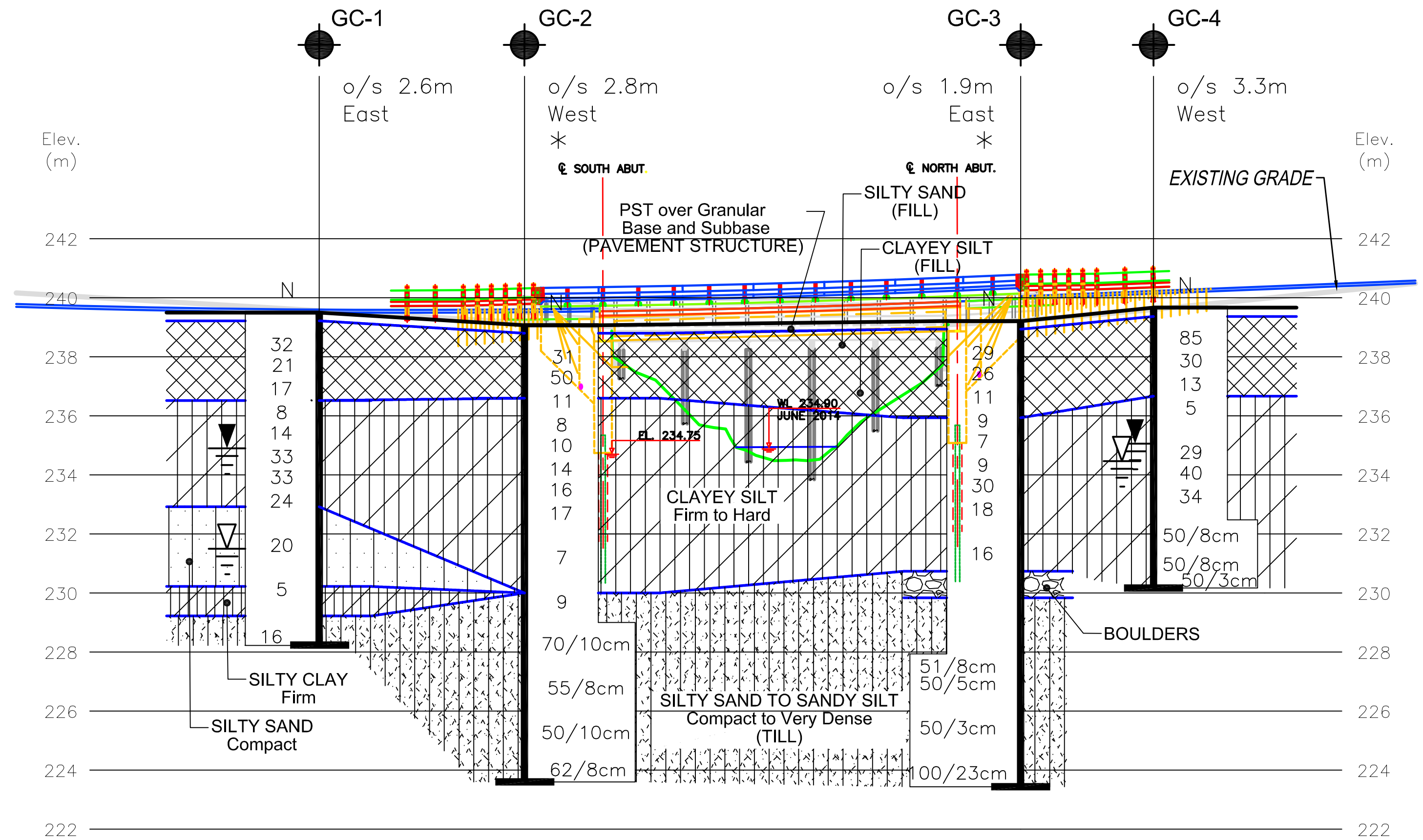
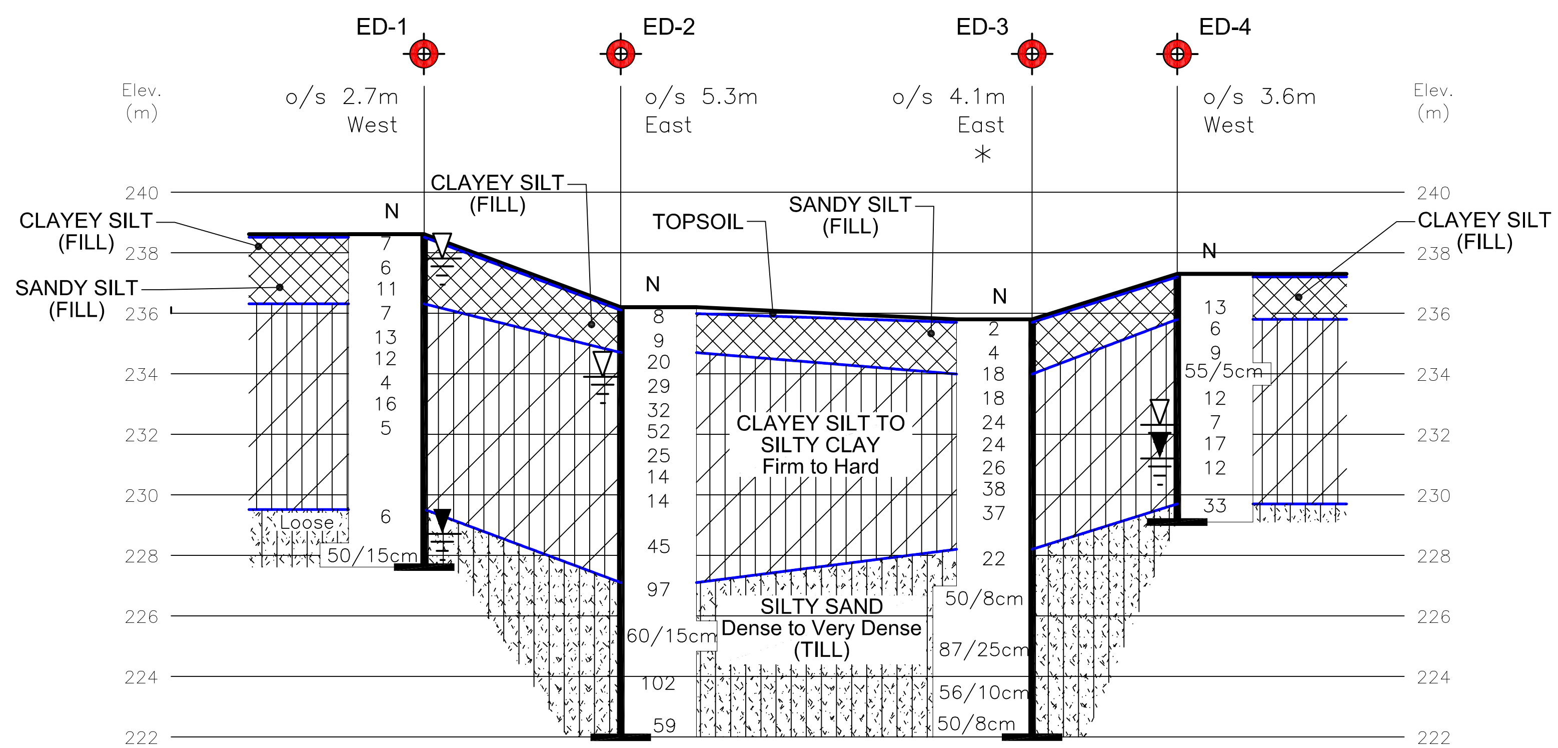
CONSULTING ENGINEERS



PROFILE ALONG A-A`

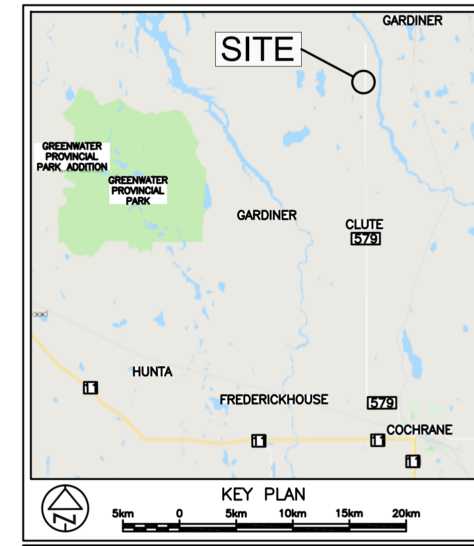


PROFILE ALONG B-B`

SCALE
HORIZONTAL 5 0 5 10m
VERTICAL 2 0 2 4m

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.
3. REFER TO DRAWING GC-1 FOR BOREHOLE PLAN AND DRAWING GC-3 FOR PROFILES C-C', AND D-D'.



KEY PLAN

LEGEND

Foundation Borehole for Structure
Foundation Borehole for Temporary Structure
N Blows/0.3m (Std. Pen Test, 475 J/blow)
WL measured during drilling
WL measured after during
* Water level could not be established

BH No	ELEVATION	NORTHINGS	EASTINGS
GC-1	239.5	5 459 548.1	299 403.4
GC-2	239.1	5 459 562.0	299 398.0
GC-3	239.2	5 459 595.6	299 402.7
GC-4	239.7	5 459 604.6	299 397.5
ED-1	238.6	5 459 549.3	299 414.6
ED-2	236.2	5 459 562.3	299 422.7
ED-3	235.8	5 459 589.5	299 421.4
ED-4	237.3	5 459 599.1	299 413.8

- NOTE -

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

LICENSED PROFESSIONAL ENGINEER R. M. C. NG JULY 3, 2019 PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER A. N. RAHMAN 100189818 JULY 3, 2019 PROVINCE OF ONTARIO

REF Drawing: Gilles Creek-GA.dwg, dated March 2019.

REVISIONS		
08/01/19	NL	Geocres No. was corrected.
DATE	BY	DESCRIPTION

Geocres No. 42H-81

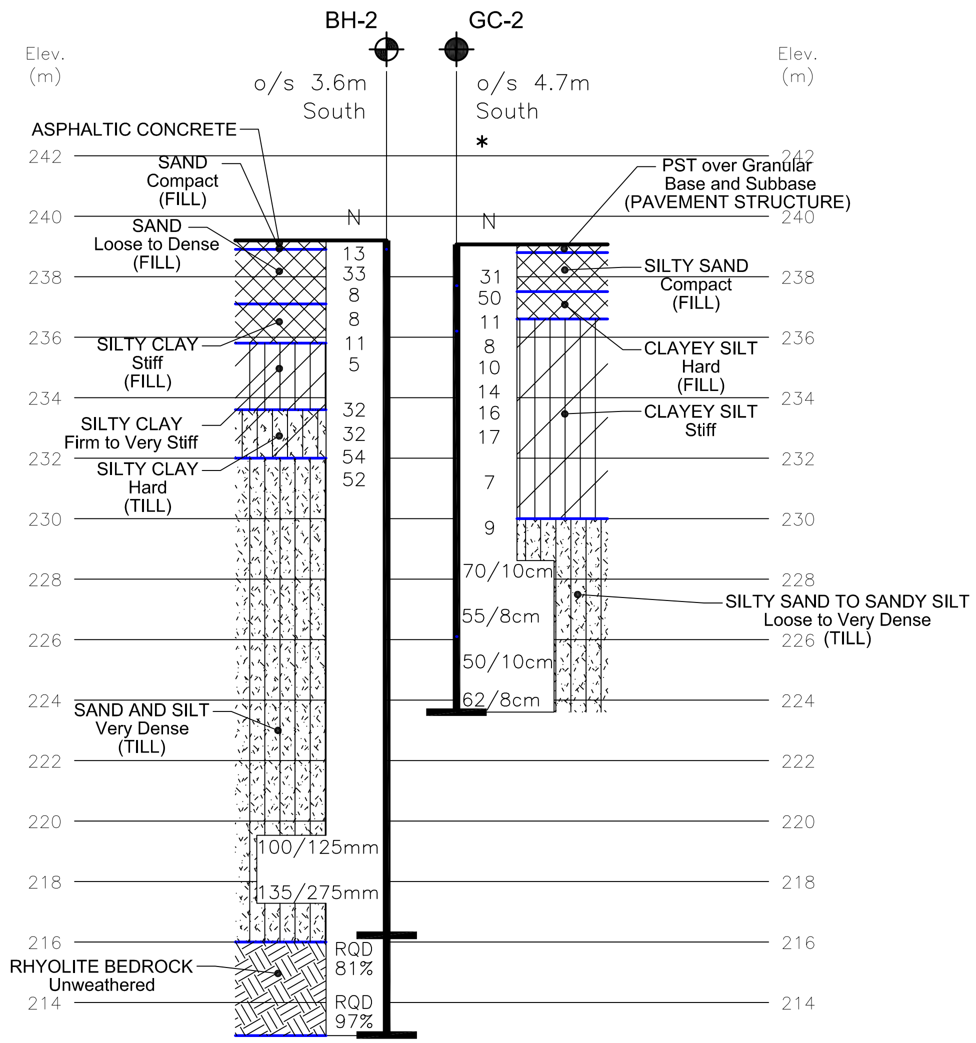
HWY No 579			DIST Northern
SUBM'D TC	CHECKED NR	DATE JULY 3, 2019	SITE 39E-0006/B0
DRAWN TC	CHECKED MV	APPROVED RN	DWG. GC-2

GWP No 5267-11-00
WP No 5368-11-01

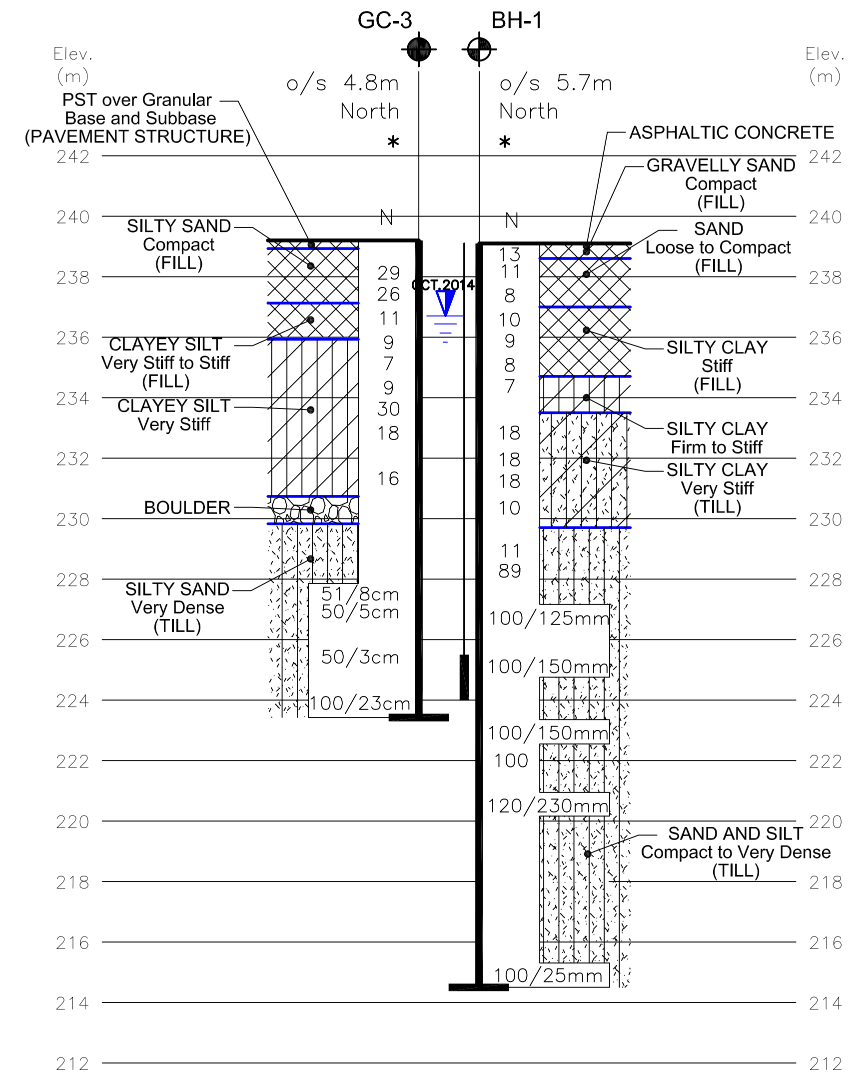
SHEET

GILLES CREEK STRUCTURE
HIGHWAY 579-STATION 19+473
SOIL STRATA

Peto MacCallum Ltd.
CONSULTING ENGINEERS



PROFILE ALONG C-C`

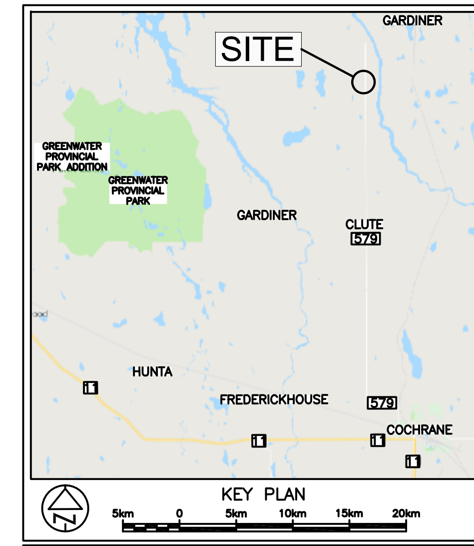


PROFILE ALONG D-D`

SCALE

HORIZONTAL 5 0 5 10m

VERTICAL 2 0 2 4m



KEY PLAN

5km 0 5km 10km 15km 20km

LEGEND

- Foundation Borehole for Structure
- Previous Borehole (Geocres No. 42H-60)
- N Blows/0.3m (Std. Pen Test, 475 J/blow)
- * Water level could not be established
- 19mm Diameter PVC Piezometer
- OCT. 2014 WL in Piezometer

BH No	ELEVATION	NORTHINGS	EASTINGS
GC-2	239.1	5 459 562.0	299 398.0
GC-3	239.2	5 459 595.6	299 402.7
PREVIOUS BOREHOLES (Geocres No. 42H-60)			
BH-1	239.1	5 459 596.25	299 398.01
BH-2	239.2	5 459 561.73	299 402.90

- NOTE -

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

LICENSED PROFESSIONAL ENGINEER
R. M. C. NG
JULY 3, 2019
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
A. N. RAHMAN
100189818
JULY 3, 2019
PROVINCE OF ONTARIO

REF Drawing: Gilles Creek-GA.dwg, dated March 2019.

REVISIONS			
08/01/19	NL	Geocres No. was corrected.	
DATE	BY	DESCRIPTION	

Geocres No. 42H-81

HWY No 579			DIST Northern
SUBM'D TC	CHECKED NR	DATE JULY 3, 2019	SITE 39E-0006/B0
DRAWN TC	CHECKED MV	APPROVED RN	DWG. GC-3

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.
3. REFEER TO DRAWING GC-1 FOR BOREHOLE PLAN AND DRAWING GC-2 FOR PROFILES A-A`, AND B-B`.

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:

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

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 GC-1

METRIC

G.W.P. 5267-11-00 LOCATION Coords: 5 459 548.1 N; 299 403.4 E ORIGINATED BY M.M./F.M.

DIST Northern HWY 668 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY K.A.

DATUM Geodetic DATE 2019.04.01 LATITUDE 49.273924 LONGITUDE -81.074167 CHECKED BY M.V.

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
239.5	Ground Surface					
0.0	20 mm PST					
239.2 / 0.3	over 130 mm Granular Base over 130 mm Granular Subbase (PAVEMENT STRUCTURE)					
	SILTY SAND, trace gravel Dense, Brown, Moist to wet	1	SS	32		
	CLAYEY SILT, some sand, trace gravel Very stiff, Brown, Moist to wet (FILL)	2	SS	21		
		3	SS	17		
236.5 / 3.0	CLAYEY SILT, some sand, trace gravel Stiff to hard, Grey, Moist to wet	4	SS	8		
		5	SS	14		5 10 61 24
					▼ (approx. 235.0)	
		6	SS	33		
		7	SS	33		
		8	SS	24		0 11 61 28
232.9 / 6.6	SILTY SAND, trace gravel Compact, Grey, Wet					
		9	SS	20	∇	
230.2 / 9.3	SILTY CLAY, trace sand Firm, Grey, Moist to wet	10	SS	5		0 2 38 60
229.2 / 10.3	SILTY SAND Compact, Grey, Moist (TILL)	11	SS	16		
228.2 / 11.3	End of borehole					

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE (239, 238, 237, 236, 235, 234, 233, 232, 231, 230, 229); 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.

∇ Groundwater level observed during drilling

▼ Groundwater level measured upon completion of drilling

NOTE: No cave-in was noted in the borehole upon extraction of hollow stem augers.

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

RECORD OF BOREHOLE No GC-2

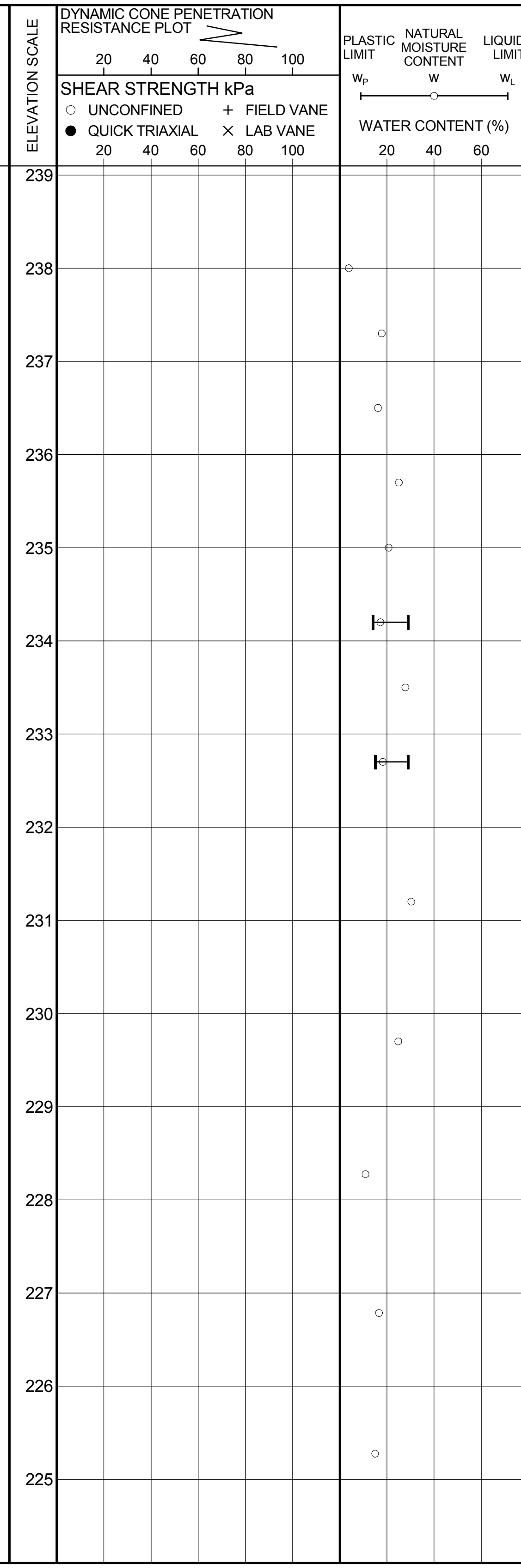
METRIC

G.W.P. 5267-11-00 LOCATION Coords: 5 459 562.0 N; 299 398.0 E ORIGINATED BY M.M./F.M.

DIST Northern HWY 668 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY K.A.

DATUM Geodetic DATE 2019.04.10 LATITUDE 49.274058 LONGITUDE -81.074241 CHECKED BY M.V.

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
239.1	Ground Surface							
0.0	30 mm PST							
238.8 / 0.3	over 140 mm Granular Base over 150 mm Granular Subbase (PAVEMENT STRUCTURE)							
	SILTY SAND, some gravel; Compact, Brown, Moist	1	SS	31				
	CLAYEY SILT, some sand, trace gravel; Hard, Brown, Moist (FILL)	2	SS	50				
236.6 / 2.5	CLAYEY SILT, trace sand, trace gravel; Stiff, Grey, Moist to wet	3	SS	11				
		4	SS	8				
		5	SS	10				
		6	SS	14	0	6	57	37
		7	SS	16				
		8	SS	17	0	6	57	37
		9	SS	7				
230.0 / 9.1	Loose	10	SS	9	8	35	52	5
	SILTY SAND TO SANDY SILT, trace to some gravel; Very dense, Grey, Wet (TILL)	11	SS	70/10cm				
		12	SS	55/8cm	16	72	9	3
		13	SS	50/10cm				
224.1								



Columns: GROUND WATER CONDITIONS; ELEVATION SCALE; UNIT WEIGHT γ kN/m^3; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

Continued Next Page

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

RECORD OF BOREHOLE No GC-2

METRIC

G.W.P. 5267-11-00 LOCATION Coords: 5 459 562.0 N; 299 398.0 E ORIGINATED BY M.M./F.M.

DIST Northern HWY 668 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY K.A.

DATUM Geodetic DATE 2019.04.10 LATITUDE 49.274058 LONGITUDE -81.074241 CHECKED BY M.V.

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE) 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
224.1											
15.0	SANDY SILT, some gravel Very dense, Grey, Wet (TILL)		14	SS	62/8cm		224		○		
223.6 15.5	End of borehole										

NOTES:

1. Groundwater was not encountered inside the borehole during or upon completion of drilling.
2. Upon extraction of hollow stem augers, the borehole caved-in at a depth of 10.1 m (El. 225.4) below the existing ground surface.

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

RECORD OF BOREHOLE No GC-3

METRIC

G.W.P. 5267-11-00 LOCATION Coords: 5 459 595.6 N; 299 402.7 E ORIGINATED BY M.M./F.M.

DIST Northern HWY 668 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY K.A.

DATUM Geodetic DATE 2019.04.02 - 2019.04.03 LATITUDE 49.27435209 LONGITUDE -81.074178 CHECKED BY M.V.

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
239.2	Ground Surface				
0.0 / 238.9 / 0.3	20 mm PST over 130 mm Granular Base over 130 mm Granular Subbase (PAVEMENT STRUCTURE)				
	SILTY SAND, trace gravel. Compact, Brown, Moist	1	SS	29	
		2	SS	26	
	CLAYEY SILT, some sand, trace gravel. Very stiff to stiff, Brown to grey, Moist to wet (FILL)	3	SS	11	
235.9 / 3.3	CLAYEY SILT, trace sand, trace gravel. Very stiff, Grey, Moist	4	SS	9	
	SILTY SAND TO SANDY SILT, trace gravel. Loose, Grey, Moist to wet	5	SS	7	
		6	SS	9	
		7	SS	30	1 9 50 40
		8	SS	18	
		9	SS	16	1 4 47 48
230.7 / 8.5	Boulder	10	RC HQ		
229.8 / 9.4	SILTY SAND, some gravel. Very dense, Grey, Moist to wet (TILL)				
		11	SS	51/8cm	
		12	SS	50/5cm	
		13	SS	50/3cm	30 46 20 4
224.2					

Column headings: SOIL PROFILE; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE (239 to 225); 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; STRAT PLOT.

Continued Next Page

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

RECORD OF BOREHOLE No GC-3

METRIC

G.W.P. 5267-11-00 LOCATION Coords: 5 459 595.6 N; 299 402.7 E ORIGINATED BY M.M./F.M.

DIST Northern HWY 668 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY K.A.

DATUM Geodetic DATE 2019.04.02 - 2019.04.03 LATITUDE 49.27435209 LONGITUDE -81.074178 CHECKED BY M.V.

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE) 20 40 60 80 100	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
224.2											
15.0	SILTY SAND, trace gravel Very dense, Grey, Moist (TILL)		14	SS	100/23cm		224		○		8 40 43 9
223.4 15.8	End of borehole										

NOTES:

1. Groundwater was not encountered inside the borehole during or upon completion of drilling.
2. Upon extraction of hollow stem augers, the borehole caved-in at a depth of 9.1 m (El. 228.2) below the existing ground surface.

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

Ministry of Transportation Ontario

Foundation Design

RECORD OF BOREHOLE No GC-4

METRIC

G.W.P. 5267-11-00 — LOCATION Coords: 5 459 604.6 N; 299 397.5 E — ORIGINATED BY M.M./F.M.

DIST Northern HWY 668 — BOREHOLE TYPE Continuous Flight Hollow Stem Augers — COMPILED BY K.A.

DATUM Geodetic — DATE 2019.04.01 — LATITUDE 49.27443273 LONGITUDE -81.074249 — CHECKED BY M.V.

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
239.7	Ground Surface												
0.0	20 mm PST												
239.4 / 0.3	over 110 mm Granular Base over 150 mm Granular Subbase (PAVEMENT STRUCTURE)					239							
	SILTY SAND, some/with gravel; Very dense to dense, Brown, Moist (FILL)	1	SS	85									
		2	SS	30		238							
		3	SS	13		237							
236.7 / 3.0	CLAYEY SILT, some sand, trace gravel; Very Stiff to hard, Grey, Moist; Firm	4	SS	5						1	12	50	37
		5	VANE			236							
	SILTY SAND, trace gravel	6	SS	29	▼ ∇	235							
		7	SS	40		234							
		8	SS	34		233				1	5	50	44
	SILTY SAND	9	SS	50/8cm		232							
						231							
		10	SS	50/8cm									
230.2 / 9.5	End of borehole. Auger refusal on probable bedrock	11	SS	50/3cm									

Chart axes: 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. REMARKS & GRAIN SIZE DISTRIBUTION (%).

∇ Groundwater level observed during drilling

▼ Groundwater level measured upon completion of drilling

NOTE: Upon extraction of hollow stem augers, the borehole caved-in at a depth of 6.1 m (El. 229.5) below the existing ground surface.

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

RECORD OF BOREHOLE No ED-1

METRIC

G.W.P. 5267-11-00 LOCATION Coords: 5 459 549.3 N; 299 414.6 E ORIGINATED BY M.M./F.M.

DIST Northern HWY 668 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY K.A.

DATUM Geodetic DATE 2019.04.05 LATITUDE 49.27394 LONGITUDE -81.074013 CHECKED BY M.V.

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
238.6	Ground Surface					
238.5 / 0.1	TOPSOIL; CLAYEY SILT, some sand, trace gravel. Firm, Brown, Moist to wet	1	SS	7		
		2	SS	6	∇ (El. ~237.9)	
	SANDY SILT, trace gravel. Compact, Brown, Moist to wet (FILL)	3	SS	11		
236.3 / 2.3	CLAYEY SILT TO SILTY CLAY, trace sand. Firm to very stiff, Grey, Moist to wet	4	SS	7		
		5	SS	13		0 1 49 50
		6	SS	12		
		7	SS	4		
		8	SS	16		
		9	SS	5		0 1 30 69
		10	VANE			
229.5 / 9.1	Loose; SANDY SILT TO SILTY SAND, trace gravel. Very dense, Grey, Moist to wet (TILL)	11	SS	6		1 28 65 6
					▼ (El. ~228.8)	
227.6 / 11.0		12	SS	50/15cm		
	End of borehole					

∇ Groundwater level observed during drilling

▼ Groundwater measured upon completion of drilling

NOTE: Upon extraction of hollow stem augers, the borehole caved-in at a depth of 9.1 m (El. 229.1) below the existing ground surface.

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: 238 237 236 235 234 233 232 231 230 229 228

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

RECORD OF BOREHOLE No ED-2

METRIC

G.W.P. 5267-11-00 LOCATION Coords: 5 459 562.3 N; 299 422.7 E ORIGINATED BY M.M./F.M.

DIST Northern HWY 668 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY K.A.

DATUM Geodetic DATE 2019.04.08 - 2019.04.09 LATITUDE 49.27405 LONGITUDE -81.073902 CHECKED BY M.V.

ELEV DEPTH (m)	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
236.2	Ground Surface					
236.1 / 0.1	TOPSOIL; CLAYEY SILT, some sand, trace gravel. Stiff, Brown, Moist (FILL)	1	SS	8		
		2	SS	9		
234.7 / 1.5	CLAYEY SILT TO SILTY CLAY, some/trace sand, trace gravel. Stiff to hard, Grey, Moist to wet	3	SS	20		
					∇ (approx. El. 234)	
		4	SS	29		
	SILTY SAND, with/some gravel	5	SS	32		11 44 14 31
		6	SS	52		
		7	SS	25		0 8 49 43
		8	SS	14		
		9	SS	14		0 1 35 64
		10	SS	45		
227.1 / 9.1	SILTY SAND/SANDY SILT, trace/some gravel. Very dense, Grey, Moist to wet (TILL)	11	SS	97		10 41 41 8
		12	SS	60/15cm		
		13	SS	102		
		14	SS	59		
222.0 / 14.2	End of borehole					

Elevation scale: 236 to 222. 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 ED-2

METRIC

G.W.P. 5267-11-00	LOCATION Coords: 5 459 562.3 N; 299 422.7 E		ORIGINATED BY M.M./F.M.
DIST Northern HWY 668	BOREHOLE TYPE Continuous Flight Hollow Stem Augers		COMPILED BY K.A.
DATUM Geodetic	DATE 2019.04.08 - 2019.04.09	LATITUDE 49.27405 LONGITUDE -81.073902	CHECKED BY M.V.

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE (20 40 60 80 100)	PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
221.2											

∇ Groundwater level observed during drilling

NOTES:

1. Groundwater level was not encountered inside the borehole upon completion of drilling.

2. Upon extraction of hollow stem augers, the borehole caved-in at a depth of 7.1 m (El. 228.1) below the existing ground surface.

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

$\bigcirc^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No ED-3

METRIC

G.W.P. 5267-11-00 LOCATION Coords: 5 459 589.5 N; 299 421.4 E ORIGINATED BY M.M./F.M.

DIST Northern HWY 668 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY K.A.

DATUM Geodetic DATE 2019.04.04 LATITUDE 49.274297 LONGITUDE -81.07392 CHECKED BY M.V.

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
235.8	Ground Surface				
235.7 / 0.1	TOPSOIL SANDY SILT, trace gravel Very loose, Brown, Wet (FILL)	1	SS	2	
		2	SS	4	
234.0 / 1.8	CLAYEY SILT TO SILTY CLAY, some/with sand, trace gravel Very stiff to hard, Grey, Moist to wet	3	SS	18	
		4	SS	18	
	SILTY SAND, some gravel	5	SS	24	15 39 36 10
		6	SS	24	
		7	SS	26	
		8	SS	38	1 16 41 42
		9	SS	37	0 3 37 60
228.2 / 7.6	Compact SILTY SAND TO SANDY SILT, some/with gravel Very dense, Grey, Wet (TILL)	10	SS	22	
		11	SS	50/8cm	
		12	SS	87/25cm	21 37 35 7
		13	SS	56/10cm	
		14	SS	50/8cm	
222.0 / 13.8	End of borehole				

Column headings: GROUND WATER CONDITIONS; ELEVATION SCALE (235, 234, 233, 232, 231, 230, 229, 228, 227, 226, 225, 224, 223, 222); 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.

Continued Next Page

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

RECORD OF BOREHOLE No ED-3

METRIC

G.W.P. 5267-11-00	LOCATION Coords: 5 459 589.5 N; 299 421.4 E		ORIGINATED BY M.M./F.M.
DIST Northern HWY 668	BOREHOLE TYPE Continuous Flight Hollow Stem Augers		COMPILED BY K.A.
DATUM Geodetic	DATE 2019.04.04	LATITUDE 49.274297 LONGITUDE -81.07392	CHECKED BY M.V.

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
ELEV DEPTH 220.8	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	NOTES: 1. Groundwater was not encountered inside the borehole during or upon completion of drilling. 2. No cave-in was noted inside the borehole upon extraction of hollow stem augers.										

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

RECORD OF BOREHOLE No ED-4

METRIC

G.W.P. 5267-11-00	LOCATION Coords: 5 459 599.1 N; 299 413.8 E		ORIGINATED BY M.M./F.M.
DIST Northern HWY 668	BOREHOLE TYPE Continuous Flight Hollow Stem Augers		COMPILED BY K.A.
DATUM Geodetic	DATE 2019.04.03	LATITUDE 49.27438 LONGITUDE -81.074024	CHECKED BY M.V.

SOIL PROFILE

ELEV DEPTH	DESCRIPTION
237.3	Ground Surface
237.2 0.1	TOPSOIL CLAYEY SILT, some sand, trace gravel Stiff, Brown, Moist to wet (FILL)
235.8 1.5	CLAYEY SILT TO SILTY CLAY, trace sand Firm to stiff, Grey, Moist to wet
229.7 7.6	SILTY SAND, with gravel Dense, Grey, Moist to wet (TILL)
229.1 8.2	End of borehole Auger refusal on proabable bedrock

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	13	
2	SS	6	
3	SS	9	
4	SS	55/5cm	
5	SS	12	
6	SS	7	0 0 17 83
7	SS	17	0 2 35 63
8	SS	12	
9	SS	33	23 38 27 12

ELEVATION SCALE: 237, 236, 235, 234, 233, 232, 231, 230

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

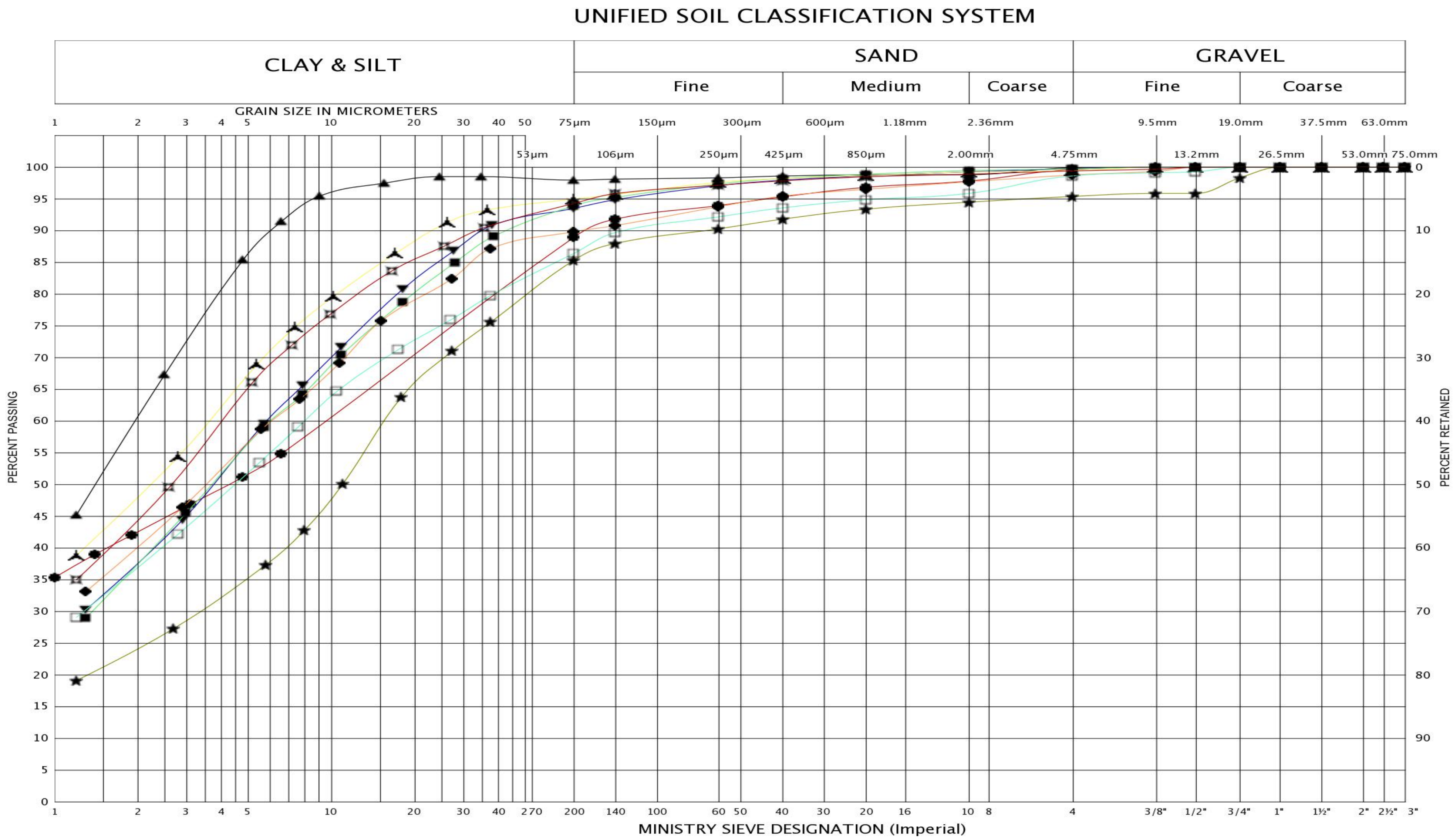
GROUND WATER CONDITIONS

∇ Groundwater level observed during drilling

▼ Groundwater level measured upon completion of drilling

NOTE: No cave-in was noted upon extraction of hollow stem augers.

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



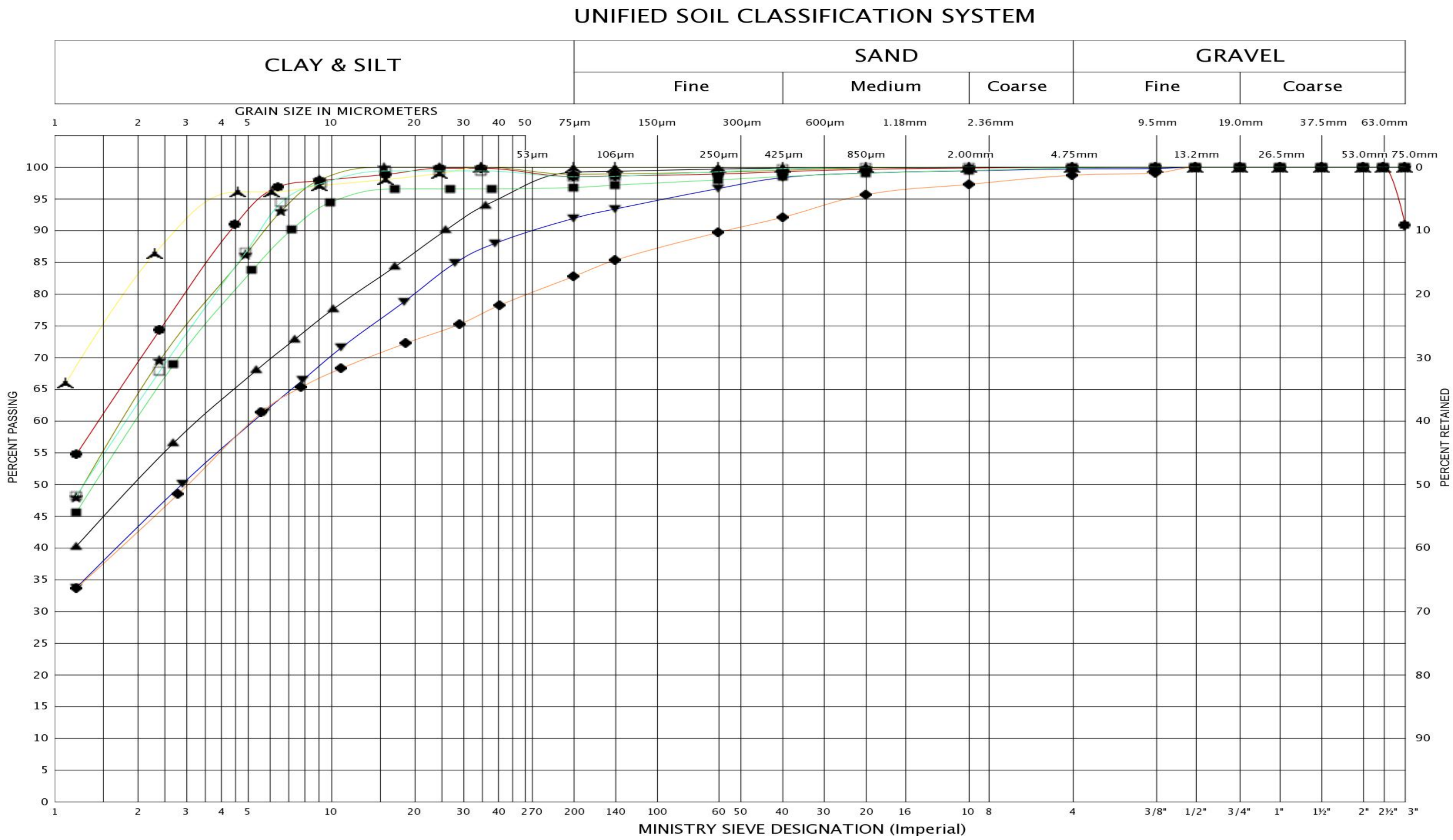
LEGEND										
	BH	GC-1	GC-1	GC-1	GC-2	GC-2	GC-3	GC-3	GC-4	GC-4
	SAMPLE	5	8	10	6	8	7	9	4	8
	SYMBOL	★	⬣	▲	▼	■	◆	⅄	□	⌑

Ontario

GRAIN SIZE DISTRIBUTION

CLAYEY SILT/SILTY CLAY, Trace/Some Sand, Trace Gravel

FIG No.:	GS-1A
HWY :	579
GWP	5267-11-00



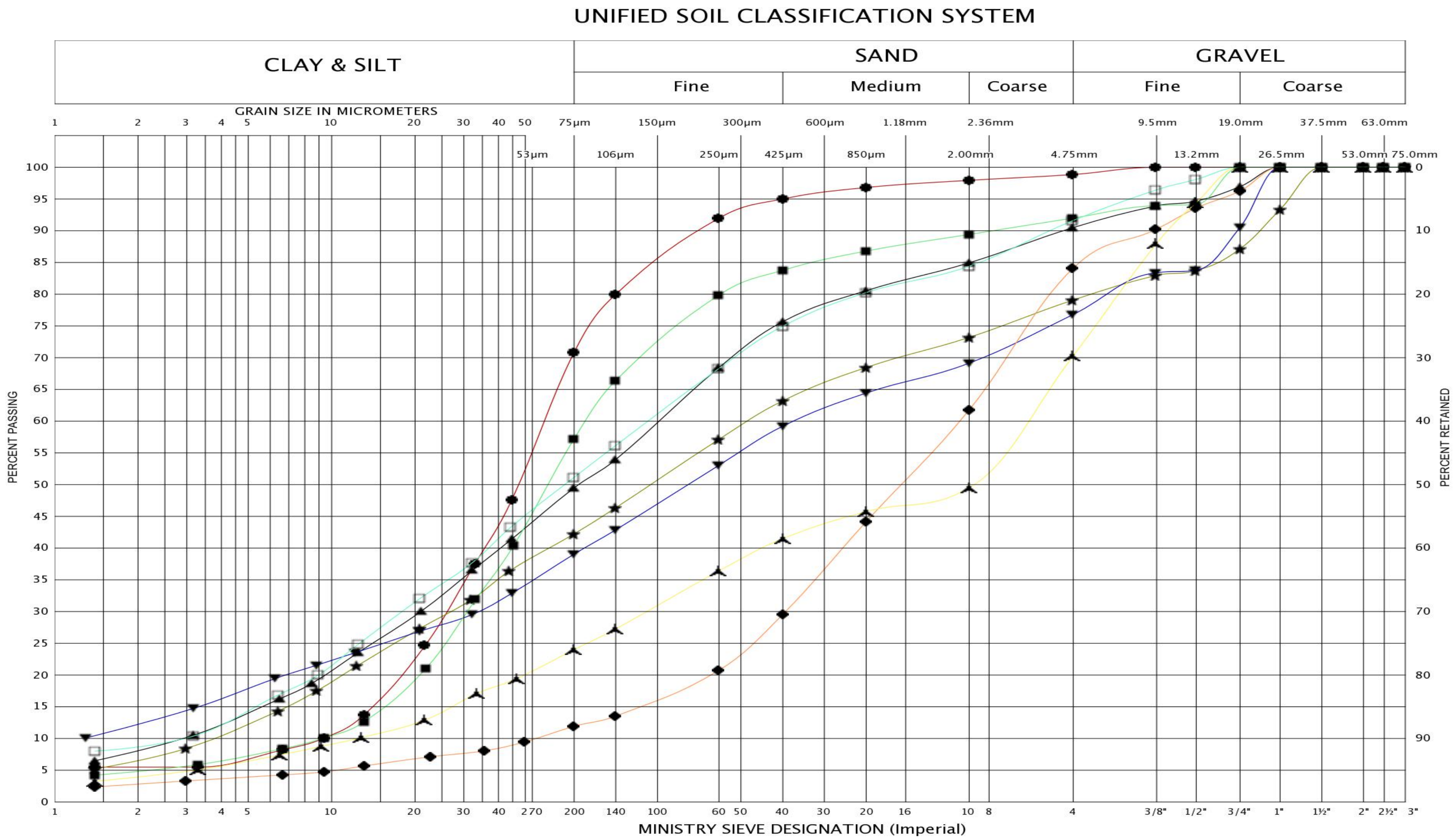
LEGEND	BH	ED-1	ED-1	ED-2	ED-2	ED-3	ED-3	ED-4	ED-4
	SAMPLE	5	9	7	9	8	9	6	7
	SYMBOL	▲	⬢	▼	★	◆	■	⅄	□

Ontario

GRAIN SIZE DISTRIBUTION

CLAYEY SILT/SILTY CLAY, Trace/Some Sand, Trace Gravel

FIG No.:	GS-1B
HWY :	579
GWP	5267-11-00



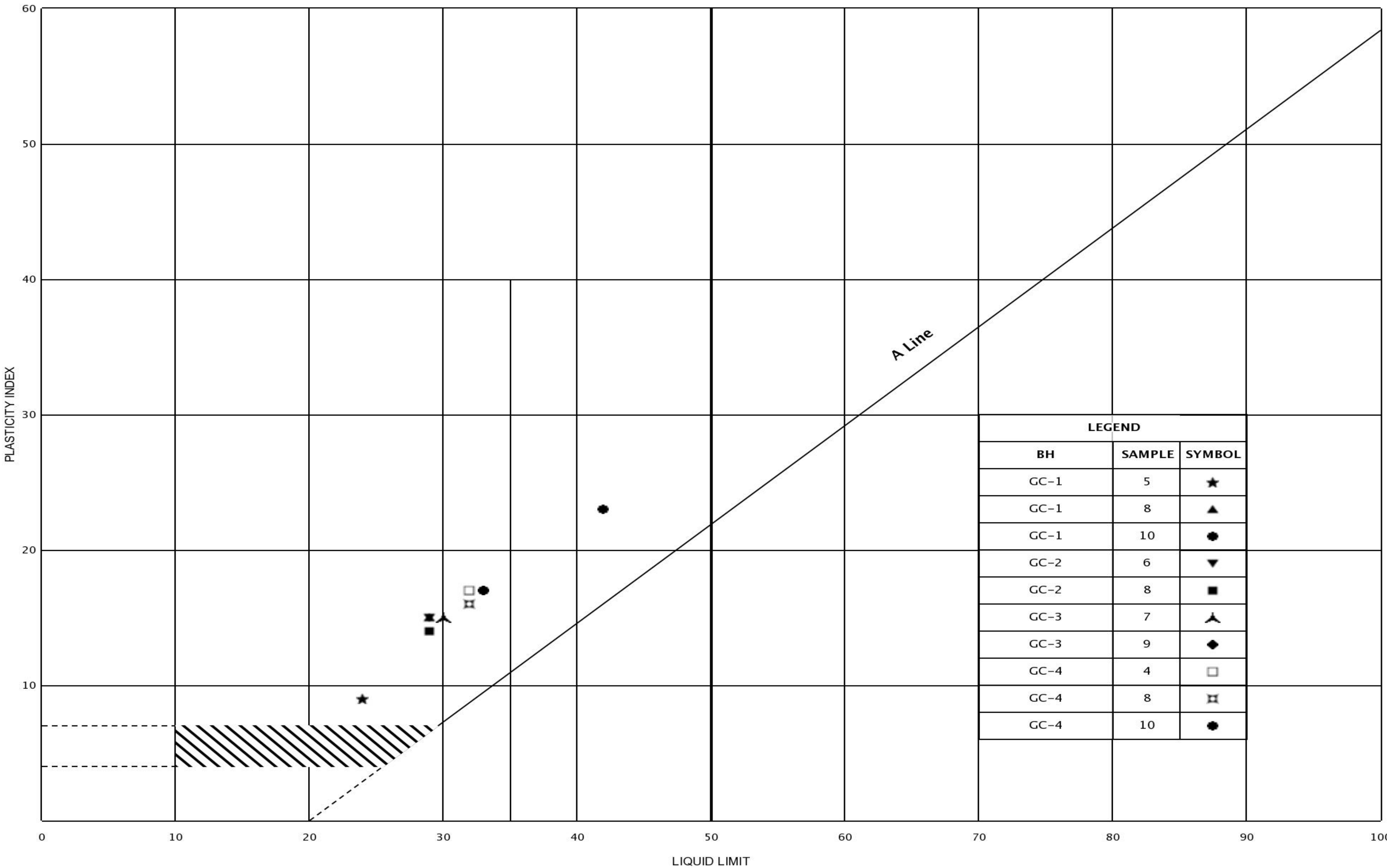
LEGEND	BH	ED-1	GC-2	ED-2	GC-2	ED-3	GC-3	GC-3	ED-4
	SAMPLE	11	10	11	12	12	13	14	9
	SYMBOL	⬣	■	▲	◆	★	⅄	□	▼

GRAIN SIZE DISTRIBUTION

SILTY SAND to SANDY SILT, Trace/Some Gravel (Till)

FIG No.:	GS-2
HWY :	579
GWP	5267-11-00

Ontario

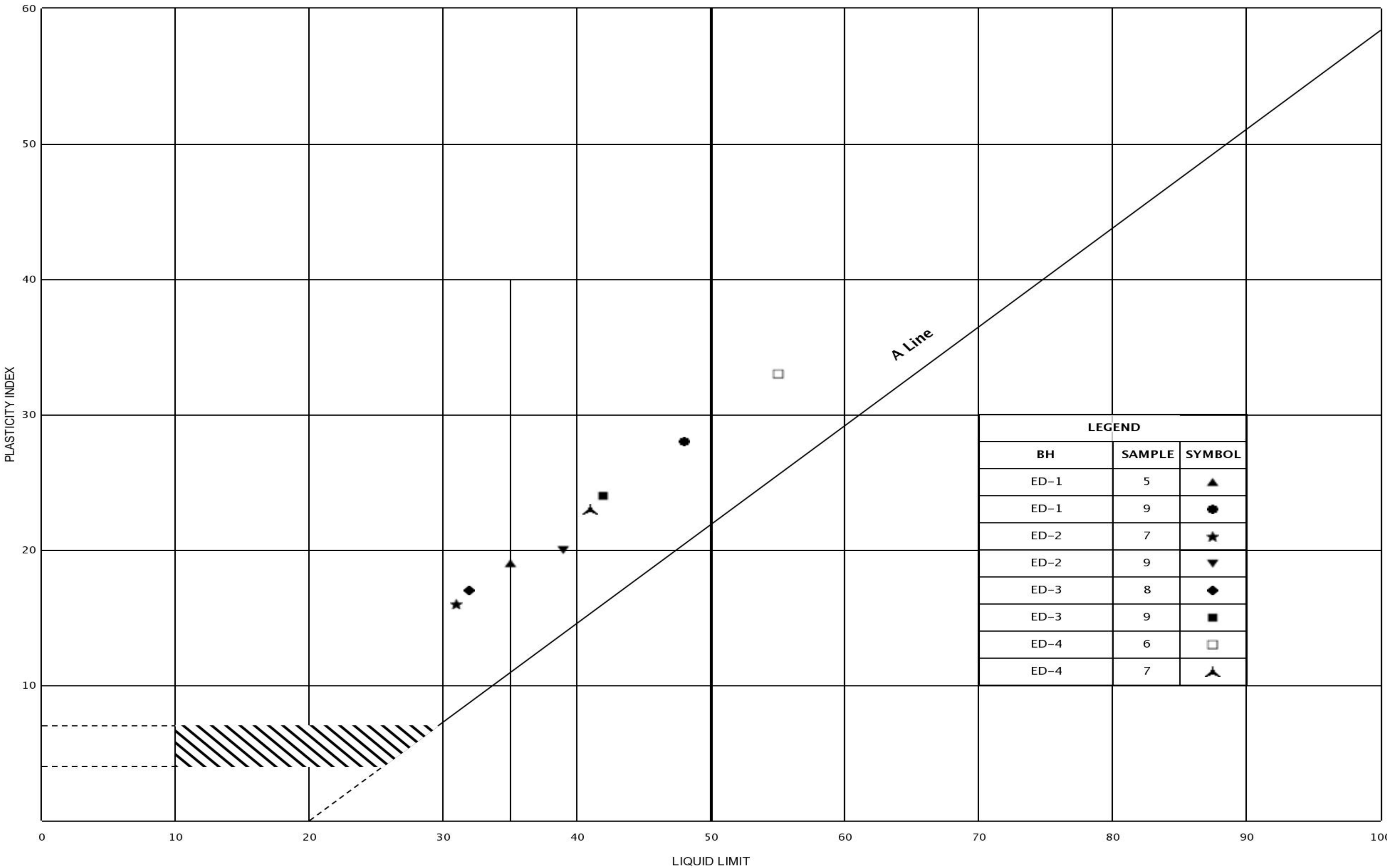


Ontario

PLASTICITY CHART

CLAYEY SILT/SILTY CLAY, Trace/Some Sand, Trace Gravel

FIG No.:	PC-1A
HWY.:	579
Assg No.	5267-11-00



Ontario

PLASTICITY CHART

CLAYEY SILT/SILTY CLAY, Trace/Some Sand, Trace Gravel

FIG No.:	PC-1B
HWY.:	579
Assg No.	5267-11-00

SGS

FINAL REPORT

CA14192-JUN19 R1

18TF002A

Prepared for

Peto MacCallum Ltd

FINAL REPORT

First Page

CLIENT DETAILS		LABORATORY DETAILS	
Client	Peto MacCallum Ltd	Project Specialist	Brad Moore Hon. B.Sc
		Laboratory	SGS Canada Inc.
Address	165 Cartwright Ave Toronto, ON M6A 1V5, Canada	Address	185 Concession St., Lakefield ON, K0L 2H0
Contact	Nazibur Rahman	Telephone	705-652-2143
Telephone	416-785-5110	Facsimile	705-652-6365
Facsimile	416-785-5120	Email	brad.moore@sgs.com
Email	nrahman@petomaccallum.com	SGS Reference	CA14192-JUN19
Project	18TF002A	Received	06/05/2019
Order Number		Approved	06/06/2019
Samples	Soil (12)	Report Number	CA14192-JUN19 R1
		Date Reported	06/06/2019

COMMENTS

Temperature of Sample upon Receipt: 17 degrees C
Cooling Agent Present:Yes
Custody Seal Present:No

Chain of Custody Number:006617

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

Brad Moore Hon. B.Sc

TABLE OF CONTENTS

SGS

FINAL REPORT

CA14192-JUN19 R1

Client: Peto MacCallum Ltd

Project: 18TF002A

Project Manager: Nazibur Rahman

Samplers: Nazibur Rahman

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

			Sample Number	5	6	7	8	9	10	11	12
			Sample Name	GC-4, SS/3, 7.5-9.5ft	ED-2, SS/3, 5.0-7.0ft	GC-3, SS/2, 5-7ft	ED-3, SS/4, 7.5-9.5ft	GC-1, SS/4, 10-12ft	DC-3, SS/2 2.5-4.5ft	GC-2, SS/3 7.5-9.5ft	DC-3, SS/3, 5-7ft
			Sample Matrix	Soil	Soil	Soil	Soil	Soil	Soil	Soil	Soil
			Sample Date	05/06/2019	05/06/2019	05/06/2019	05/06/2019	05/06/2019	05/06/2019	05/06/2019	05/06/2019
Parameter	**Units**	**RL**		**Result**	**Result**	**Result**	**Result**	**Result**	**Result**	**Result**	**Result**
Corrosivity Index											
Corrosivity Index	none	1		1	4.5	1	4.5	1	1	9	1
Soil Redox Potential	mV	-		274	197	206	253	299	284	304	290
Sulphide	%	0.02		< 0.02	0.02	< 0.02	0.02	< 0.02	< 0.02	< 0.02	< 0.02
pH	pH Units	0.05		7.97	8.04	8.09	8.16	8.45	8.29	7.85	8.25
Resistivity (calculated)	ohms.cm	-9999		6210	4670	4900	3480	7760	4810	1750	4690

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

			Sample Number	13	14	15	16
			Sample Name	SC-2, SS/4, 7.5-9.5ft	DRW-2, SS/3, 5-7ft	SC-3, SS2, 2.5-4.5ft	DRW-3, SS/3, 7.5-9.5ft
			Sample Matrix	Soil	Soil	Soil	Soil
			Sample Date	05/06/2019	05/06/2019	05/06/2019	05/06/2019
Parameter	**Units**	**RL**		**Result**	**Result**	**Result**	**Result**
Corrosivity Index							
Corrosivity Index	none	1		1	4.5	< 1	4.5
Soil Redox Potential	mV	-		225	218	193	179
Sulphide	%	0.02		< 0.02	0.02	< 0.02	0.04
pH	pH Units	0.05		8.18	8.15	8.49	7.96
Resistivity (calculated)	ohms.cm	-9999		6250	5170	10600	4460

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

Parameter	Units	RL		5	6	7	8	9	10	11	12
			Sample Name	GC-4, SS/3, 7.5-9.5ft	ED-2, SS/3, 5.0-7.0ft	GC-3, SS/2, 5-7ft	ED-3, SS/4, 7.5-9.5ft	GC-1, SS/4, 10-12ft	DC-3, SS/2 2.5-4.5ft	GC-2, SS/3 7.5-9.5ft	DC-3, SS/3, 5-7ft
			Sample Matrix	Soil	Soil	Soil	Soil	Soil	Soil	Soil	Soil
			Sample Date	05/06/2019	05/06/2019	05/06/2019	05/06/2019	05/06/2019	05/06/2019	05/06/2019	05/06/2019
				Result	**Result**	**Result**	**Result**	**Result**	**Result**	**Result**	**Result**
General Chemistry											
Conductivity	uS/cm	2		161	214	204	287	129	208	571	213

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

Parameter	Units	RL		13	14	15	16
			Sample Name	SC-2, SS/4, 7.5-9.5ft	DRW-2, SS/3, 5-7ft	SC-3, SS2, 2.5-4.5ft	DRW-3, SS/3, 7.5-9.5ft
			Sample Matrix	Soil	Soil	Soil	Soil
			Sample Date	05/06/2019	05/06/2019	05/06/2019	05/06/2019
				Result	**Result**	**Result**	**Result**
General Chemistry							
Conductivity	uS/cm	2		160	194	94	224

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

Parameter	Units	RL		5	6	7	8	9	10	11	12
			Sample Name	GC-4, SS/3, 7.5-9.5ft	ED-2, SS/3, 5.0-7.0ft	GC-3, SS/2, 5-7ft	ED-3, SS/4, 7.5-9.5ft	GC-1, SS/4, 10-12ft	DC-3, SS/2 2.5-4.5ft	GC-2, SS/3 7.5-9.5ft	DC-3, SS/3, 5-7ft
			Sample Matrix	Soil	Soil	Soil	Soil	Soil	Soil	Soil	Soil
			Sample Date	05/06/2019	05/06/2019	05/06/2019	05/06/2019	05/06/2019	05/06/2019	05/06/2019	05/06/2019
				Result	**Result**	**Result**	**Result**	**Result**	**Result**	**Result**	**Result**
Metals and Inorganics											
Moisture Content	%	0.1		18.3	19.6	13.9	17.1	16.1	18.4	15.7	18.2
Sulphate	µg/g	0.4		6.7	71	6.2	87	4.4	7.0	30	7.4

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

	13	14	15	16
Sample Name	SC-2, SS/4, 7.5-9.5ft	DRW-2, SS/3, 5-7ft	SC-3, SS2, 2.5-4.5ft	DRW-3, SS/3, 7.5-9.5ft

SGS

FINAL REPORT

CA14192-JUN19 R1

Client: Peto MacCallum Ltd

Project: 18TF002A

Project Manager: Nazibur Rahman

Samplers: Nazibur Rahman

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

			Sample Number	13	14	15	16
			Sample Name	SC-2, SS/4, 7.5-9.5ft	DRW-2, SS/3, 5-7ft	SC-3, SS2, 2.5-4.5ft	DRW-3, SS/3, 7.5-9.5ft
			Sample Matrix	Soil	Soil	Soil	Soil
			Sample Date	05/06/2019	05/06/2019	05/06/2019	05/06/2019
Parameter	**Units**	**RL**		**Result**	**Result**	**Result**	**Result**
Metals and Inorganics							
Moisture Content	%	0.1		16.9	19.3	2.6	19.4
Sulphate	µg/g	0.4		12	52	4.0	81

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

			Sample Number	5	6	7	8	9	10	11	12
			Sample Name	GC-4, SS/3, 7.5-9.5ft	ED-2, SS/3, 5.0-7.0ft	GC-3, SS/2, 5-7ft	ED-3, SS/4, 7.5-9.5ft	GC-1, SS/4, 10-12ft	DC-3, SS/2 2.5-4.5ft	GC-2, SS/3 7.5-9.5ft	DC-3, SS/3, 5-7ft
			Sample Matrix	Soil	Soil	Soil	Soil	Soil	Soil	Soil	Soil
			Sample Date	05/06/2019	05/06/2019	05/06/2019	05/06/2019	05/06/2019	05/06/2019	05/06/2019	05/06/2019
Parameter	**Units**	**RL**		**Result**	**Result**	**Result**	**Result**	**Result**	**Result**	**Result**	**Result**
Other (ORP)											
Chloride	µg/g	0.4		6.9	3.4	23	14	11	40	300	18

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

			Sample Number	13	14	15	16
			Sample Name	SC-2, SS/4, 7.5-9.5ft	DRW-2, SS/3, 5-7ft	SC-3, SS2, 2.5-4.5ft	DRW-3, SS/3, 7.5-9.5ft
			Sample Matrix	Soil	Soil	Soil	Soil
			Sample Date	05/06/2019	05/06/2019	05/06/2019	05/06/2019
Parameter	**Units**	**RL**		**Result**	**Result**	**Result**	**Result**
Other (ORP)							
Chloride	µg/g	0.4		7.3	2.1	5.8	5.6

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	DIO0085-JUN19	µg/g	0.4	<0.4	2	20	94	80	120	99	75	125
Sulphate	DIO0085-JUN19	µg/g	0.4	<0.4	4	20	96	80	120	95	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	ECS0010-JUN19	%	0.02	<0.02	7	20	108	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	EWL0082-JUN19	uS/cm	2	< 0.002	1	10	99	90	110	NA		

QC SUMMARY

pH

Method: SM 4500 l 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	EWL0082-JUN19	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 --

APPENDIX B

Previous Drawings and Record of Borehole Sheets (GEOCRES Nos. 42H-60)

Terraprobe

Consulting Geotechnical & Environmental Engineering

Construction Materials Inspection & Testing

PRELIMINARY
FOUNDATION INVESTIGATION AND DESIGN REPORT
GILLES CREEK BRIDGE REPLACEMENT
HIGHWAY 579
ASSIGNMENT No. 5013-E-0018
MINISTRY OF TRANSPORTATION, ONTARIO
G.W.P. No. 5368-11-00, SITE 39E-006
GEOCRES NO. 42H-60

PREPARED FOR: MMM Group Limited
2655 North Sheridan Way, Suite 300
Mississauga, Ontario
L5K 2P8

Attention: Mr. Trevor Small, M.Sc., P.Eng.

File No. 11-14-4066
October 09, 2015
Terraprobe Inc.

Distribution:

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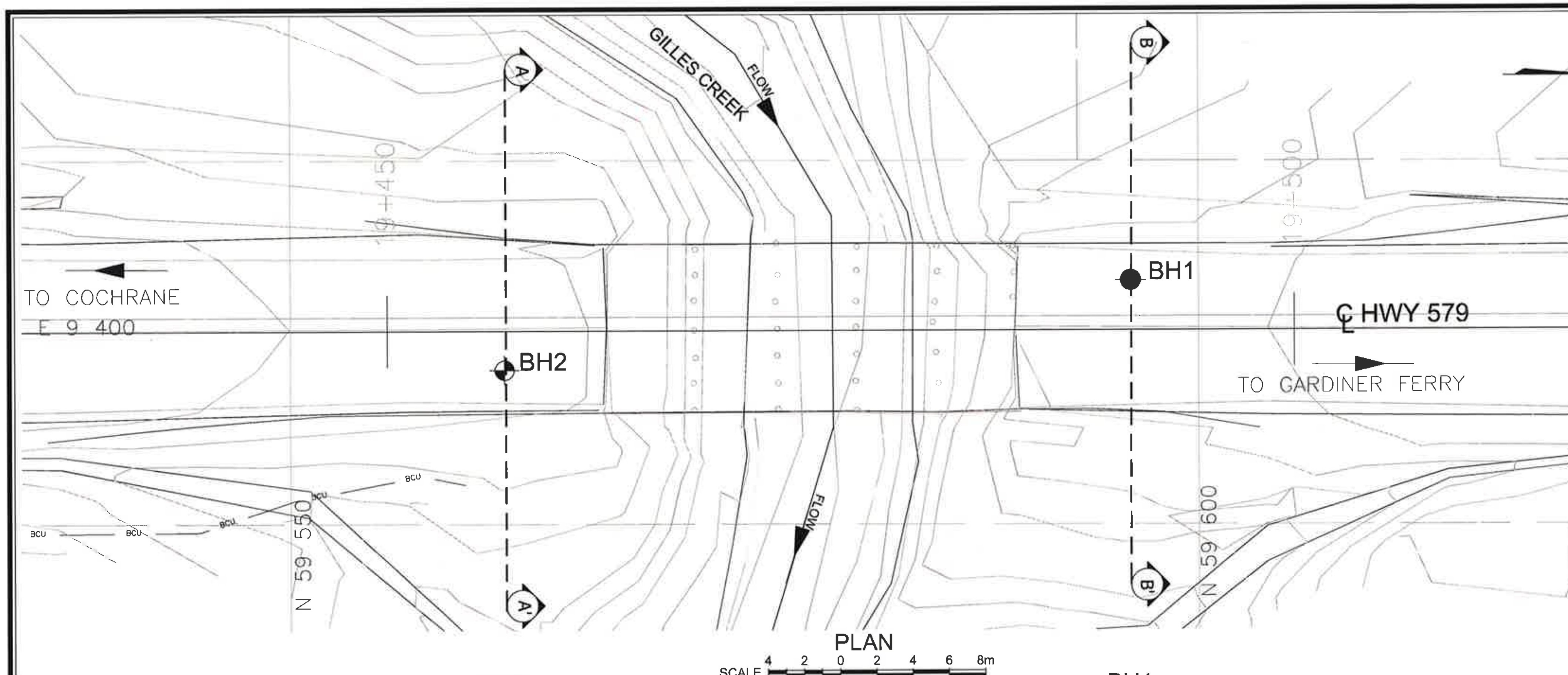
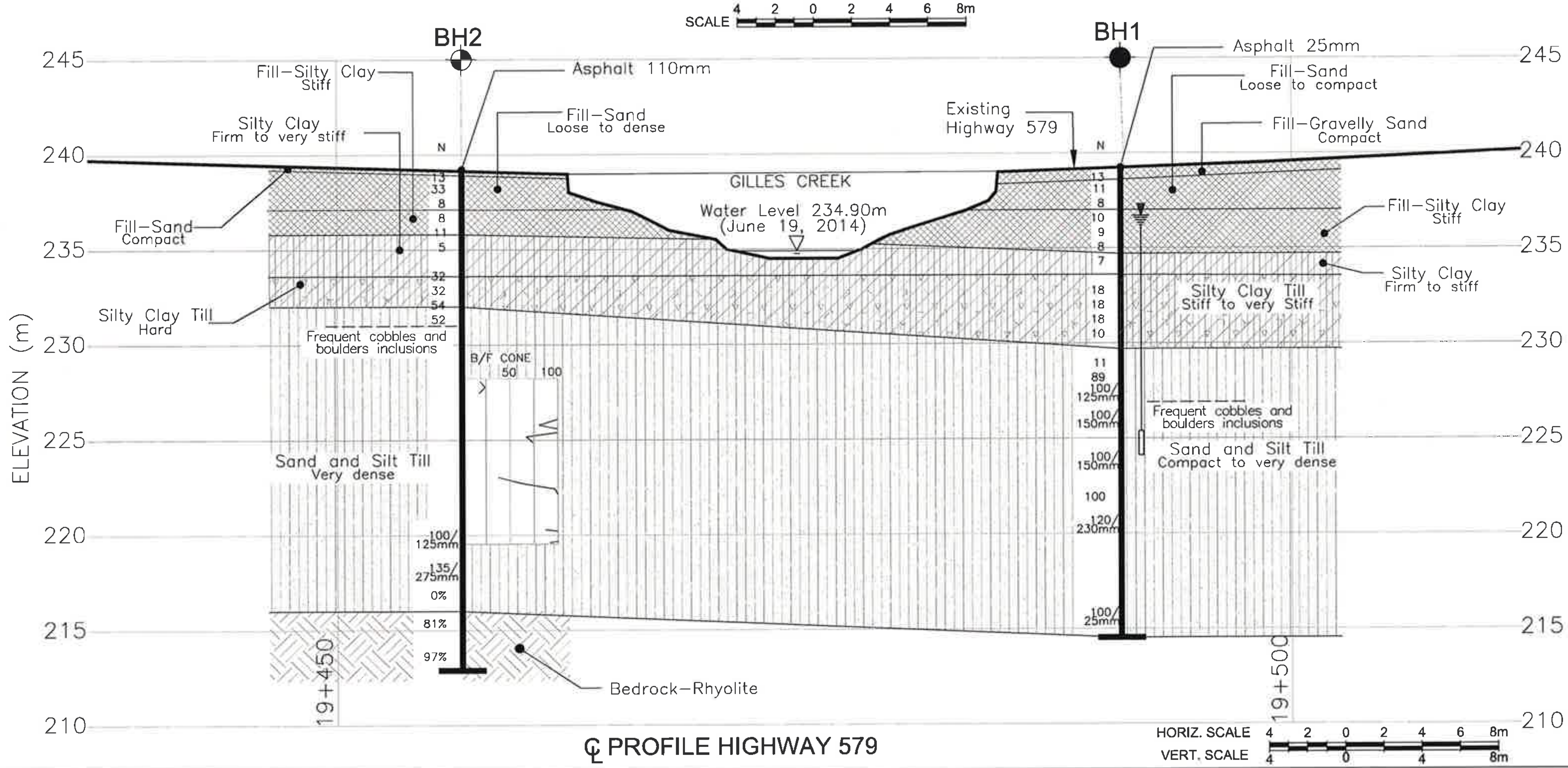




METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETERS UNLESS OTHERWISE SHOWN

GWP No 5368–11–00

HWY 579
GILLES CREEK BRIDGE
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

MMM Group Limited
2655 North Sheridan Way, Suite 300
Mississauga, ON Canada L5K 2P8
t: 905.823.8500, f: 905.823.8503

Terraprobe Inc.
Consulting Geotechnical & Environmental Engineering
Construction Materials Engineering, Inspection & Testing
11 Indell Lane • Brampton Ontario L6T 3Y3 (905) 796-2650

KEY PLAN

	LEGEND
	Bore Hole
	Dynamic Cone Penetration Test
	Bore Hole And Cone
'N'	Blows/0.3m (Std Pen Test, 475 J/blow)
CONE	Blows/0.3m (60° Cone, 475 J/blow)
	WL at Time of Investigation
	WL in Piezometer (October 2014)
	Piezometer
90%	Rock Quality Designation
A/R	Auger Refusal

No	ELEV.	COORDINATES	
		NORTHING	EASTING
1	239.1	59 596.25	9 398.01
2	239.2	59 561.73	9 402.90

REGISTERED PROFESSIONAL ENGINEER
M. TANOS
9.10.2015
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
R.A. ABDUL
Oct. 09, 2015
PROVINCE OF ONTARIO

NOTE

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

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

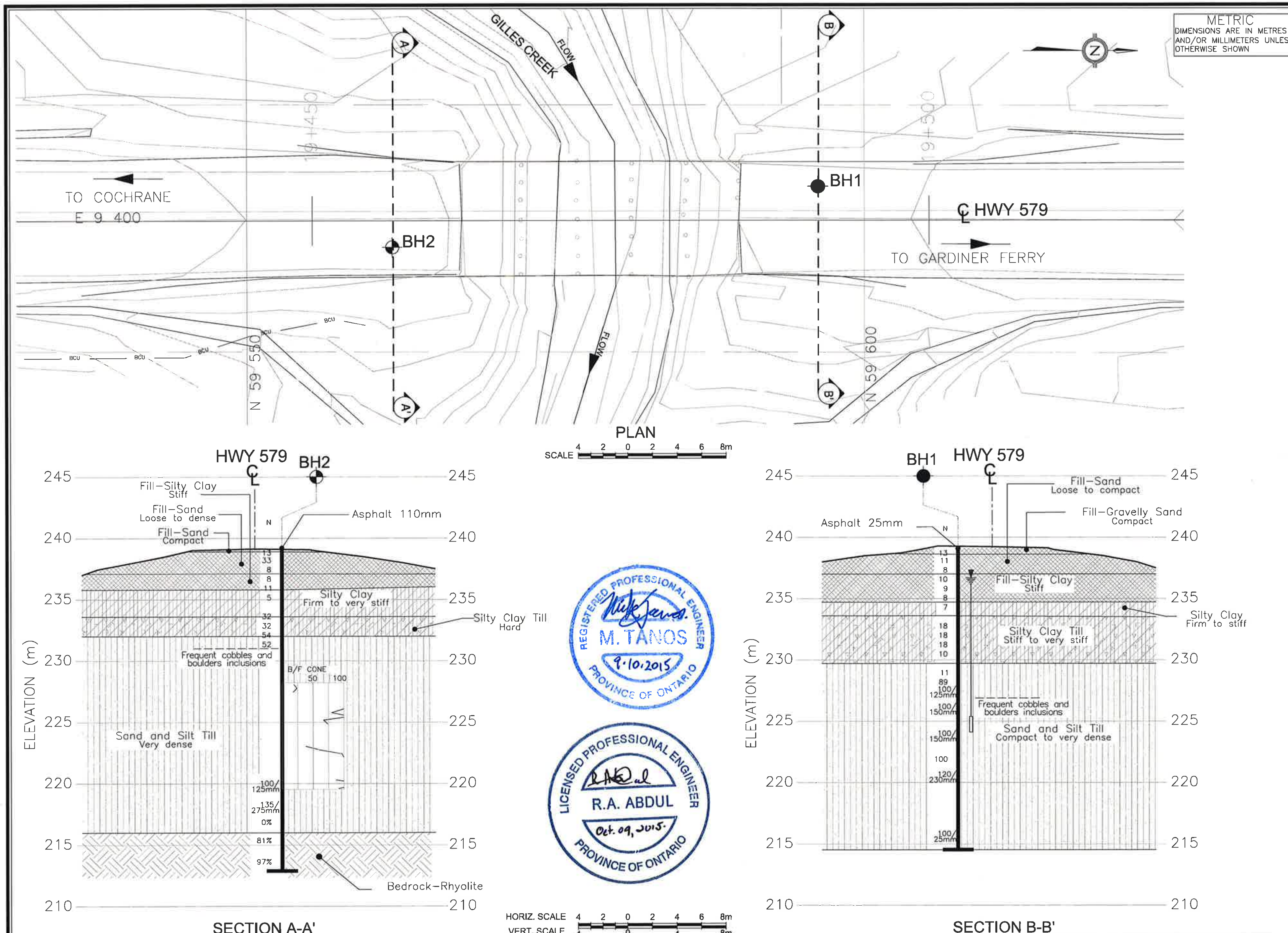
The complete foundation investigation and design report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents are specifically excluded in accordance with Section GC 2.01 of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

REFERENCE

Drawings provided in digital format by MMM Group Ltd. by CD (Assignment 5013–E–0018 Preliminary Design for Rehab/Replacement of 12 Structures on Highways in New Liskeard Area) drawing files B13420579001, DTM13420579001, received September 11, 2014.

REVISIONS	DATE	BY	DESCRIPTION

HWY. 579	PROJECT No. 11–14–4066	GEOCRES No.: 42H–60
SUBM'D. HA	CHKD. RA / DATE: October 2015	SITE: 39E–006
DRAWN: KC	CHKD. RA / APPD: MT	DWG. 1



GWP No 5368–11–00

HWY 579
GILLES CREEK BRIDGE
BOREHOLE LOCATIONS AND SOIL STRATA

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KEY PLAN

	LEGEND
	Bore Hole
	Dynamic Cone Penetration Test
	Bore Hole And Cone
'N'	Blows/0.3m (Std Pen Test, 475 J/blow)
CONE	Blows/0.3m (60° Cone, 475 J/blow)
	WL at Time of Investigation
	WL in Piezometer (October 2014)
	Piezometer
90%	Rock Quality Designation
A/R	Auger Refusal

No	ELEV.	COORDINATES	
		NORTHING	EASTING
1	239.1	59 596.25	9 398.01
2	239.2	59 561.73	9 402.90

NOTE

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

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REVISIONS	DATE	BY	DESCRIPTION

HWY. 579	PROJECT No. 11–14–4066		GEOCRES No.: 42H–60
SUBM'D. HA	CHKD. RA	DATE: October 2015	SITE: 39E–006
DRAWN: KC	CHKD. RA	APPD: MT	DWG. 2

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 ú.

DYNAMIC CONE PENETRATION TEST: CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60/ CONE ANGLE) DRIVEN BY 475J 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.

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

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

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

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

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

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

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

SS	SPLIT SPOON	TP	THINWALL PISTON
WS	WASH SAMPLE	OS	OSTERBERG SAMPLE
ST	SLOTTED TUBE SAMPLE	RC	ROCK CORE
BS	BLOCK SAMPLE	PH	TW ADVANCED HYDRAULICALLY
CS	CHUNK SAMPLE	PM	TW ADVANCED MANUALLY
TW	THINWALL OPEN	FS	FOIL SAMPLE

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
r_u	1	PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ε	%	LINEAR STRAIN
$\varepsilon_1, \varepsilon_2, \varepsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

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 = c_u / τ_r

PHYSICAL PROPERTIES OF SOIL

r_s	kg/m^3	DENSITY OF SOLID PARTICLES	e	1,%	VOID RATIO	e_{min}	1,%	VOID RATIO IN DENSEST STATE
γ_s	kN/m^3	UNIT WEIGHT OF SOLID PARTICLES	n	1,%	POROSITY	I_D	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
r_w	kg/m^3	DENSITY OF WATER						
γ_w	kN/m^3	UNIT WEIGHT OF WATER	w	1,%	WATER CONTENT	D	mm	GRAIN DIAMETER
r	kg/m^3	DENSITY OF SOIL	S_r	%	DEGREE OF SATURATION	D_n	mm	n PERCENT - DIAMETER
γ	kN/m^3	UNIT WEIGHT OF SOIL	w_L	%	LIQUID LIMIT	C_u	1	UNIFORMITY COEFFICIENT
r_d	kg/m^3	DENSITY OF DRY SOIL	w_P	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
γ_d	kN/m^3	UNIT WEIGHT OF DRY SOIL	w_S	%	SHRINKAGE LIMIT	q	m^3/s	RATE OF DISCHARGE
r_{sat}	kg/m^3	DENSITY OF SATURATED SOIL	I_P	%	PLASTICITY INDEX = $(w_L - w_P)$	v	m/s	DISCHARGE VELOCITY
γ_{sat}	kN/m^3	UNIT WEIGHT OF SATURATED SOIL	I_L	1	LIQUIDITY INDEX = $(w - w_P)/I_P$	i	1	HYDRAULIC GRADIENT
r'	kg/m^3	DENSITY OF SUBMERGED SOIL	I_C	1	CONSISTENCY INDEX = $(w_L - w)/I_P$	k	m/s	HYDRAULIC CONDUCTIVITY
γ'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL	e_{max}	1,%	VOID RATIO IN LOOSEST STATE	j	kN/m^3	SEEPAGE FORCE

RECORD OF BOREHOLE No 1

METRIC

G.W.P. 5368-11-00	LOCATION Coords: E:9398.01 N:59596.25	ORIGINATED BY W.Z
DIST ____ HWY 579	BOREHOLE TYPE HOLLOW STEM AUGERS/CASING AND WASH BORING/NQ CORING	COMPILED BY H.A
DATUM GEODETIC	DATE 2014-7-14 - 2014-8-6	CHECKED BY R.A

SOIL PROFILE

ELEV / DEPTH (m)	DESCRIPTION
239.1	**GROUND SURFACE**
	25mm **ASPHALTIC CONCRETE**
238.6 / 0.5	455mm **FILL-GRAVELLY SAND**, trace silt, compact, brown, dry
	FILL, sand, trace gravel, trace silt, loose to compact, brown, dry
237.0 / 2.1	**FILL**, silty clay, trace sand, trace gravel, containing wood fragments, stiff, brown, moist to wet
234.7 / 4.4	**SILTY CLAY**. some sand, firm to stiff, grey, moist to wet
233.5 / 5.6	**SILTY CLAY**, trace sand, stiff to very stiff, grey, moist to wet (GLACIAL TILL)
229.7 / 9.4	**SAND AND SILT**, trace to some clay, trace gravel, containing cobbles and boulders below 12.2m, compact to very dense, grey, moist (GLACIAL TILL)

SAMPLES

NUMBER	TYPE	SPT 'N' VALUE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	13	
2	SS	11	2 92 (6)
3	SS	8	
4	SS	10	
5	SS	9	
6	SS	8	
7	SS	7	0 11 47 42
8	SS	18	
9	SS	18	0 5 57 38
10	SS	18	
11	SS	10	
12	SS	11	sampler wet at 9.9m
13	SS	89	6 48 39 7
14	SS	100 / 125mm	July 14, 2014 ----------------- July 31, 2014 NQ Coring
15	RC		
16	SS	100 / 150mm	
17	RC		

Column headings: STRAT PLOT; GROUND WATER CONDITIONS; ELEVATION SCALE (239, 238, 237, 236, 235, 234, 233, 232, 231, 230, 229, 228, 227, 226, 225); 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 (%) 10 20 30; UNIT WEIGHT γ kN/m³

8.3

>+

Continued Next Page

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

RECORD OF BOREHOLE No 1

METRIC

G.W.P. 5368-11-00 | LOCATION Coords: E:9398.01 N:59596.25 | ORIGINATED BY W.Z

DIST ____ HWY 579 | BOREHOLE TYPE HOLLOW STEM AUGERS/CASING AND WASH BORING/NQ CORING | COMPILED BY H.A

DATUM GEODETIC | DATE 2014-7-14 - 2014-8-6 | CHECKED BY R.A

ELEV DEPTH (m)	DESCRIPTION	SAMPLE NUMBER	TYPE	SPT 'N' VALUE	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	(continued) **SAND AND SILT**, trace to some clay, trace gravel, containing cobbles and boulders below 12.2m, compact to very dense, grey, moist (GLACIAL TILL)	17	RC		224		
		18	SS	100 / 150mm	223	○	34 43 19 4
	containing gravelly sand layers below 15.8m						
		19	SS	100	222		
		20	SS	120/ 230mm	221	○	July 31, 2014 ------------------ Aug. 05, 2014
					220		
		21	RC		219		
		22	RC		218 / 217		
					216 / 215		Aug. 05, 2014 ------------------ Aug. 06, 2014
214.5 24.6		23	SS	100 / 25mm			

END OF BOREHOLE

Piezometer installation consists of a 19mm diameter schedule 40PVC pipe with a 1.52m slotted screen.

WATER LEVEL READINGS

Date	Water Depth (m)	Elevation (m)
Sep 16, 2014	2.6	236.5
Oct 27, 2014	2.4	236.7

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 (%) 10 20 30; UNIT WEIGHT γ kN/m³

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

Ministry of Transportation Ontario

Terraprobe

RECORD OF BOREHOLE No 2

METRIC

G.W.P.	5368-11-00	LOCATION	Coords: E:9402.9 N:59561.73	ORIGINATED BY	W.Z
DIST		HWY 579			
		BOREHOLE TYPE	HOLLOW STEM AUGERS/CASING AND WASH BORING/NQ CORING	COMPILED BY	H.A
DATUM	GEODETIC	DATE	2014-7-15 - 2014-7-16	CHECKED BY	R.A

SOIL PROFILE | SAMPLES | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) | SHEAR STRENGTH (kPa): ○ UNCONFINED, ● QUICK TRIAXIAL, + FIELD VANE, × LAB VANE (20 40 60 80 100) | PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L — WATER CONTENT (%) (10 20 30) | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

ELEV DEPTH (m)	DESCRIPTION	NUMBER	TYPE	SPT 'N' VALUE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
239.2	**GROUND SURFACE**				
238.9 / 0.3	110mm **ASPHALTIC CONCRETE**; 190mm **FILL-SAND**, some gravel, some silt, compact, brown, dry	1	SS	13	19 71 (10)
	FILL, sand, trace to some gravel, trace silt, loose to dense, brown, dry	2	SS	33	
		3	SS	8	
237.1 / 2.1	**FILL**, silty clay, some sand, trace gravel, stiff, brown, moist; 50mm amorphous peat layer	4	SS	8	1 13 50 36
		5	SS	11	
235.8 / 3.4	**SILTY CLAY**, some sand, trace gravel, trace organics, firm to very stiff, grey, moist to wet; 50mm sand and silt layer, grey, wet	6	SS	5	0 18 54 28
233.6 / 5.6	**SILTY CLAY**, some sand, trace gravel, occasional silty sand to sand and gravel layers, hard, grey, moist to wet (GLACIAL TILL)	7	SS	32	
		8	SS	32	sampler wet at 6.1m
		9	SS	54	1 20 18 61
232.0 / 7.2	**SAND AND SILT**, some clay, trace gravel, containing cobbles and boulders, very dense, grey, moist to wet (GLACIAL TILL)	10	SS	52	commence casing and washboring
		11	RC		NQ Coring
		12	RC		
		13	RC		

Field vane: 2.2 (at approx. elev. 234.5)

LL=49

Continued Next Page

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

Ministry of Transportation Ontario

Terraprobe

RECORD OF BOREHOLE No 2

METRIC

G.W.P. 5368-11-00	LOCATION Coords: E:9402.9 N:59561.73		ORIGINATED BY W.Z
DIST	HWY 579	BOREHOLE TYPE HOLLOW STEM AUGERS/CASING AND WASH BORING/NQ CORING	COMPILED BY H.A
DATUM GEODETIC	DATE 2014-7-15 - 2014-7-16		CHECKED BY R.A

ELEV DEPTH (m)	DESCRIPTION *(continued)*	STRAT PLOT	NUMBER	TYPE	SPT 'N' VALUE	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SAND AND SILT, some clay, trace gravel, containing cobbles and boulders, very dense, grey, moist to wet (GLACIAL TILL)		13	RC			224	
			14	RC			223	
			15	RC			222	July 15, 2014 ----------------- July 16, 2014
			16	RC			221 / 220	
			17	SS	100 / 125mm		219	
			18	SS	135/ 275mm		218	2 51 35 12
			19	RC			217	
216.0 23.2	**BEDROCK-RHYOLITE**, with quartz intrusions, unweathered massive, grey with white (quartz) intrusions, Strong to Very Strong. Quartz layers range from 30mm to 35mm in thickness		1	RUN	NQ		216 / 215	RUN# 1 TCR=97% SCR=91% RQD=81% UCS*= 111 - 147 (MPa)
			2	RUN	NQ		214 / 213	RUN# 2 TCR=97% SCR=97% RQD=97% UCS*= 84 - 187 (MPa)
212.9 26.3								

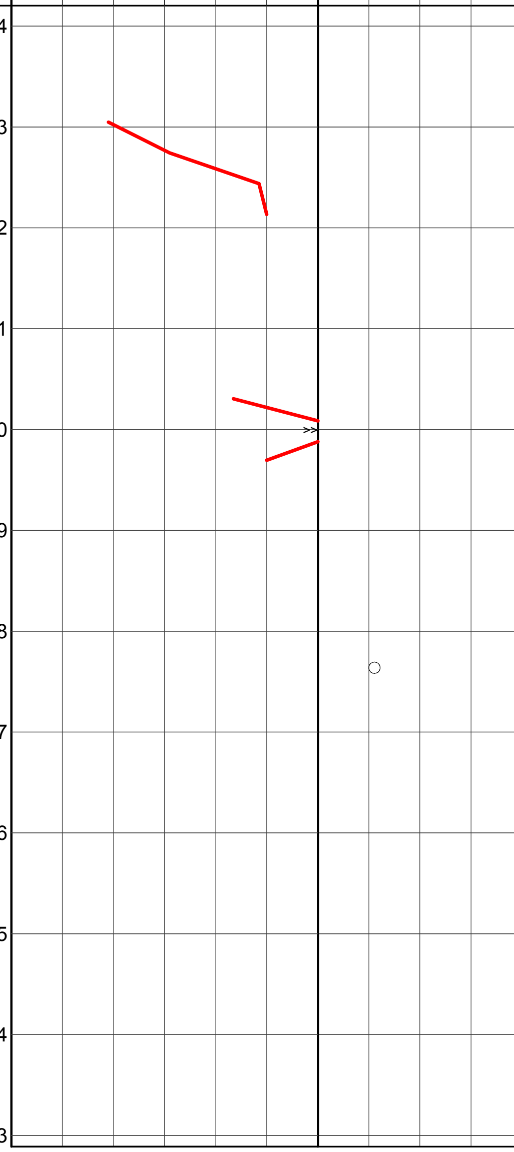
DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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 (%): 10 20 30

UNIT WEIGHT γ kN/m³



END OF BOREHOLE

Dynamic cone penetration test (DCPT) performed from 10.7m to 11.7m, 13.0m to 14.2m, 16.0m to 16.9m and 18.7m to 19.4m.

*Uniaxial Compressive Strength determined from Point Load Strength Index values.

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

PART B – FOUNDATION DESIGN REPORT

for

GILLES CREEK BRIDGE REPLACEMENT
SITE NO. 39E-0006/B0
HIGHWAY 579 – STATION 19+473
TOWN OF COCHRANE, ONTARIO
G.W.P. 5267-11-00
W.P. 5368-11-01
LATITUDE AND LONGITUDE: 49.11283, -81.27235

PETO MacCALLUM LTD.
165 CARTWRIGHT AVENUE
TORONTO, ONTARIO
M6A 1V5
Phone: (416) 785-5110
Fax: (416) 785-5120
Email:toronto@petomaccallum.com

Distribution:
Electronic Copies:

Parsons for distribution to MTO, Project Manager
(PDF)

Parsons for distribution to MTO,
Pavements and Foundations Section
(PDF, AutoCAD, gINT (.gpj))

Parsons (PDF)

1 cc: PML Toronto

PML Ref.: 18TF002A
Index No.: 024FDR
GEOCRES No.: 42H-81
August 1, 2019

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PART B - FOUNDATION DESIGN REPORT

PART B - FOUNDATION DESIGN REPORT
Gilles Creek Bridge Replacement
Site No. 39E-0006/B0
Highway 579 – Station 19+473
Town of Cochrane, Ontario
G.W.P. 5267-11-00, W.P. 5368-11-01

8. INTRODUCTION

This foundation investigation and design report with the interpretation and recommendations are intended for the use of Parsons on behalf of MTO, and shall not be used or relied upon for any other purposes or by any other parties including the construction or design-build contractor. Where comments are made on construction, they are provided only to highlight those aspects, which could affect the design of the project. Contractors must make their own interpretation of the factual information provided in Part A of the report, as it may affect equipment selection, proposed construction methods and scheduling.

9. PROJECT DESCRIPTION

9.1 General

This report provides recommendations for foundation design based on interpretation of the geotechnical data presented in the factual report (Part A) and the details provided on the General Arrangement (GA) drawing provided for the proposed replacement of bridge at the crossing of Highway 579 and Gilles creek in the Town of Cochrane, Ontario.

Based on the (GA) drawing, it is proposed to construct the single-span replacement bridge supported on integral abutments. A temporary modular bridge will be constructed on the east side of the existing bridge to detour vehicular traffic during the construction of the replacement bridge. The structural details of the modular bridge was not available at the time preparing this report.

The discussions and recommendations presented in this report are based on the information provided by Parsons and the factual data obtained during the geotechnical investigation carried out by PML.

9.2 Existing Structure

The existing bridge is a 5-span, each 4.3 m long timber structure with a total length of 23.0m. The existing bridge is 10 m wide and accommodates two (2) lanes of vehicular traffic (northbound and southbound), over the Gilles Creek. The bridge was constructed in 1941 and has no record of major rehabilitation.

The Ontario Bridge Management System (OBMS) inspection report, dated October 24, 2017, identified light to severe cracks and splits on planks, caps, and piles, and minor undermining of both abutment walls. Light weathering and checks, narrow to minor gaps, and light to severe splits were noted along the bridge deck exterior and exterior soffits, along with loss of contact between the deck and girder at the northwest corner. Vertical misalignment between boards was also reported.

Evidence of pile movement was noted in the inspection report. Light to severe splits and checks, and loss of contact between pier cap and piles up to 30 mm at one location, weathering of pile caps, localized rot, land loss of creosote was noted at the locations of pier caps. It was reported that the girders at north pier are not fully in contact with pier cap, and the second pier cap from the south side has been replaced with a steel section and exhibits light corrosion. The report reveals that longitudinal struts had been added between pier caps and severe split on pier cap reinforcing was also noted.

The site was covered with about 1.2 m of snow during the fieldwork and reconnaissance of the existing structure or approach embankment could not be carried out.

9.3 Proposed Structures

9.3.1 Replacement Bridge

Based on the GA drawing dated March 2019, it is proposed to construct the replacement bridge with a 24.0 m long single-span structure supported on integral abutments. The proposed structure will consist of 4.2 m long cantilevered sections, extending from the north and south abutments. The drawing indicates that the steel H-piles for the 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.

The GA drawing indicates that the cut-off elevations of the piles to support the north and south abutments are proposed to be at El. 235.3 and El. 235.6, respectively.

The approach slabs will be 6.0 m long at both abutments. The design grade of the approach embankments at the west and east abutments will be set at about El. 239.7 and 240.1, respectively, which will result in grade raise of approach embankments up to approximately 0.6 m at the south abutment and about 0.9 m at the north abutment.

9.3.2 Structure for Detour

Based on the GA drawing, the construction of replacement bridge will be carried out by transferring the traffic onto a temporary modular bridge (TMB) located east of the existing bridge. The TMB will be constructed with concrete barriers on each side, along an alignment approximately 13.3 m from the centerline of the existing bridge.

9.4 Structure Foundations

Based on the GA drawing, the abutments of the proposed bridge are to be supported on steel H-piles. For comparison purposes the following Table 9.4 provides the advantages, disadvantages, risks and consequences of the foundation alternatives to support the proposed structure.

Table 9.4: Comparison of Foundation Types

FOUNDATION TYPE	ADVANTAGES	DISADVANTAGES	RISKS / CONSEQUENCES
Driven Piles	•High geotechnical resistance available •Allows for integral abutment design •Ability to drive through cobbles or dense gravel •Does not require deep excavation	•Higher cost compared to footings •Vibration induced during driving •May require pile tip reinforcement •Individual piles may encounter refusal at varying depths •Limitations on location of pile splicing	•Piles may hung-up at varying elevations •Possible pile tip damage if piles are not adequately protected while driving to till/bedrock
Spread Footings	•Ease of construction •No dewatering or deep excavation is required •Less cost compared to deep foundations •Adequate bearing resistance available at reasonably shallow depth	•Lower bearing resistance than for driven piles or caissons •May require shoring or roadway protection for excavation	•Immediate settlements due to elastic compression may be expected •Limited support for increase in loading

9.4.1 Replacement Bridge on Integral Abutments

In summary, the subsoil conditions immediately below the existing road consists of 300 mm of pavement structure underlain by 2.2 m to 3.0 m thick fill composed of layers of silty sand and clayey silt, which is followed by firm to hard clayey silt. The clayey silt layer is underlain by very dense silty sand/sandy silt till, which extends to the maximum borehole termination depth of 15.8 m below the existing ground surface.

The groundwater level was encountered only in two of the boreholes advanced in the area of the replacement structure at a depth of 5.2 m and 8.0 m during drilling. Upon completion of drilling, groundwater level was observed at a depth of 4.6 m (El. 235.1) below the ground surface. The water level in Gilles Creek was observed at approximately El. 235.0 during the fieldwork.

Based on the subsoil conditions encountered at this site and discussed in Part A of this report, it is feasible to support the proposed replacement bridge on HP 310 x 110 steel piles driven to approximate elevation El. 225.5±1.0. The piles may be lowered into a 600 mm diameter pre-augered hole supported by CSP to a depth of 3.0 m from the pile cut-off elevation and driven to El. 225.5±1.0. The steel H-piles driven to this elevation may be designed assuming a factored axial geotechnical resistance of 1600 kN at Ultimate Limit State (ULS) and 1200 kN at Serviceability Limit State (SLS).

Table 9.4.1a below summarizes the approximate pile tip elevations and length of piles that may be considered for design purposes.

Table 9.4.1a: Pile Tip Elevations and Length for HP 310 x 110

LOCATION	PILE TIP ELEVATION	PILE CUT-OFF ELEVATION	LENGTH (m)
North Abutment	225.5 ± 1.0	235.3	9.8 ± 1.0
South Abutment	225.5 ± 1.0	235.6	10.1 ± 1.0

A layer of boulders was encountered in one of the boreholes located near the proposed location of the north abutment (GC-3) at a depth of 8.5 m (El. 230.7), which was cored to a depth of 9.4 m (229.8) below the existing ground surface. Provision in the contract should be allowed to pre-auger to El. 229.8 at this location prior to driving of piles. The annular space of pre-augured holes to remove the boulders should be filled with concrete to the point of contraflexture after the installation of pile to the required tip elevation. The contractor should be alerted to anticipate presence of boulders by incorporating a Non Standard Special Provision (NSSP) in the contract. The NSSP is provided under the List of Standards may be used for this purpose. The pile tips need to be reinforced to drive the piles through dense to very dense silty sand till deposit to avoid damage. Oversized driving shoes, similar to Ontario Provincial Standard Design (OPSD) 3000.100 (Foundation Piles Steel H-Pile Driving Shoe) or Titus H bearing, are not recommended. These types of pile tip reinforcement may reduce the shaft friction and may lead to overrun, especially when the pile capacity is partly derived from shaft friction. The pile tip reinforcement shown on the attached Sketch No. PML-1 is recommended.

Considering the height of proposed grade raise, no major settlement of the approaches is anticipated to allow for negative skin friction (down drag) loads at this site.

To evaluate the point of contraflexture, the coefficient of horizontal subgrade reaction, ks (MN/m3) may be computed using the following equations:

a) Cohesionless Soils (Terzaghi, 1955)

k_s = n_h z/b

where; n_h = coefficient related to soil density

z = depth, m

b = pile width, m

b) Cohesive Soils (Davison, 1970)

k_s = 67 τ_u/d

where; τ_u = Undrained shear strength

d = Pile diameter or width, m

The coefficient of horizontal subgrade reaction values provided in Table 9.4.1b, may be used to determine the point of contraflexture for HP 310 x 110 steel H-piles:

Table 9.4.1b: Coefficient of Horizontal Subgrade Reaction for Piles

STRUCTURE	SOIL BOUNDARY ELEVATION (m)	SOIL TYPE	nh VALUES (kN/m3)	UNDRAINED SHEAR STRENGTH (kN/m2)
South Abutment	234.8 to 231.8	Loose Sand	1,000	-
	231.8 to 230.0	Stiff Clayey Silt	-	50
	230.0 to 223.6	Very Dense Silty Sand to Sandy Silt Till	12,000	-
North Abutment	235.0 to 232.0	Loose Sand	1,000	
	232.0 to 230.7	Very Stiff Clayey Silt	-	100
	230.7 to 223.4	Very Dense Silty Sand to Sandy Silt Till	12,000	

The construction of pile foundation should be in accordance with OPSS.PROV 903. Pile splices within 6.0 m below the cut-off elevation should not be permitted. This requirement should be addressed with a note on the structural drawing for foundations.

The driving of piles shall be carefully monitored and controlled by employing the Hiley Dynamic Pile Driving Formula driven in accordance with MTO Standards SS103-11 assuming an ultimate pile capacity of 3,200 kN. In case the pile is designed for a structural design load at ULS less that of the factored geotechnical resistance of 1600 kN at ULS recommended, two times the structural design load should be specified on the contract drawing. The pile driving criteria should be established based on two times the structural design load at ULS to avoid overrun of pile length.

The driving criteria employing Hiley Formula may be established on first pile of every ten piles driven in a group, as per the OPSS.PROV 903. At least ten percent (10%) with a minimum of two (2) piles in a group should be checked using Hiley Formula no sooner than 48 hours after the installation, as per the OPSS.PROV 903. This requirement should be addressed with a note on the structural drawing for foundation.

Alternatively, the ultimate resistance of piles may be determined by Dynamic Formula and validated using High-Strain Dynamic Testing, in accordance with SP 109F57, appended in Appendix C.

9.4.2 Shallow Foundation

Alternatively, the proposed north and south abutments may be supported on spread footings placed on stiff to hard clayey silt at approximate elevation 234.5. The geotechnical resistances provided on Table 9.4.2 for a 2.5 m wide footing are recommended for the design of the proposed bridge.

Table 9.4.2: Founding Elevation and Geotechnical Resistance for Shallow Foundation

LOCATION	FOUNDING ELEVATION	FACTORED GEOTECHNICAL RESISTANCE AT ULS (kN)	GEOTECHNICAL RESISTANCE AT SLS (kN)	SUBGRADE SOIL
South Abutment	234.5	250	175	Stiff to Hard Clayey Silt
North Abutment				

The bearing resistance for inclined loads should be reduced in accordance with the requirements of clause 6.10.4 of the CHBDC (2014). The total settlement under a Serviceability Limit State (SLS) loads recommended is expected to be in the order of 20 mm to 25 mm and the associated differential settlement may be expected to be in the order of 15 mm to 20 mm. Most of the estimated total settlement is expected to result from elastic compression of the subgrade and be completed immediately after completion of construction. Continuing total or differential settlements under the weight of the structure may be negligible.

The sliding resistance of footings against lateral loads between the concrete footing and subgrade should be calculated in accordance with Section 6.10.5 of the CHBDC (2014). For cast in place concrete footings constructed on concrete working slabs and on top of very stiff clayey silt:

- Cast-In-Place footing on concrete working slab: = 0.6
- Cast-In-Place concrete working slab on very stiff clayey silt: = 0.6

Considering the depth of groundwater level (El. 228.8 to El. 231.2) at this site, no major dewatering problems are anticipated for footings placed at the recommended elevations of El. 234.5 to 235.0.

9.5 Foundation For Detour Structure

In summary, the subsoil conditions immediately below the ground surface along the alignment of proposed detour consist of 100 mm of topsoil. The topsoil is underlain by 1.4 m to 2.2 m thick fill composed of layers of silty sand and clayey silt, which is followed by firm to hard clayey silt. The clayey silt layer is underlain by dense to very dense silty sand till, which extends to the maximum borehole termination depth of 14.2 m below the existing ground surface.

The groundwater level was observed in three of the boreholes drilled in the area of proposed detour at depths varying between of 0.8 m and 4.9 m during drilling. Upon completion of drilling, it was observed only in two of the boreholes at a depth of 6.1 m (El. 231.3) and 9.9 (El. 228.7) below the ground level. The water level in Gilles Creek was observed at approximately El. 235.0 during the fieldwork.

There is no detail available regarding the duration of the detour or the structural details of the proposed TMB. However, if it is proposed to be temporary during the summer months, the foundation may be placed on stiff clayey silt fill at about El. 236.0 and compact sandy silt fill at about El. 234.5 at the south and north abutment locations, respectively, after removing all the spongy and soft area observed within the foot print of the footings. The geotechnical resistances provided on Table 9.5 for a 2.0 m wide footing are recommended for the design of the TMB supported on conventional timber crib or equivalent abutments founded at a level not higher than El. 236.0.

Table 9.5: Founding Elevation and Geotechnical Resistance for Shallow Foundation

<table>
<tr><th>LOCATION</th><th>FOUNDING ELEVATION</th><th>FACTORED GEOTECHNICAL RESISTANCE AT ULS (kN)</th><th>GEOTECHNICAL RESISTANCE AT SLS (kN)</th><th>SUBGRADE SOIL</th></tr>
<tr><td>South Abutment</td><td>236.0</td><td rowspan="2">250</td><td rowspan="2">175</td><td>Stiff Clayey Silt Fill</td></tr>
<tr><td>North Abutment</td><td>234.5</td><td>Compact Sandy Silt Fill</td></tr>
</table>

In case the TMB is required to be maintained throughout the winter months, the depth of frost should be taken into consideration in the determination of the founding level of the footings.

The bearing resistance for inclined loads should be reduced in accordance with the requirements of clause 6.10.4 of the CHBDC (2014). The total settlement under a Serviceability Limit State (SLS) loads recommended is expected to be in the order of 20 mm to 25 mm and the associated differential settlement may be expected to be in the order of 15 mm to 20 mm. Most of the estimated total settlement is expected to result from elastic compression of the subgrade and be completed immediately after completion of construction. Continuing total or differential settlements under the weight of the structure may be negligible.

The sliding resistance of footings against lateral loads between the concrete footing and subgrade should be calculated in accordance with Section 6.10.5 of the CHBDC (2014). For cast in place concrete footings constructed on concrete working slabs and on top of very stiff clayey silt:

- Cast-In-Place footing on concrete working slab: = 0.6
- Cast-In-Place concrete working slab on very stiff clayey silt: = 0.6

Considering the depth of groundwater level (El. 228.8 to El. 231.2) at this site, no major dewatering problems are anticipated for footings placed at the recommended elevation of El. 236.0.

10. LATERAL EARTH PRESSURE

Earth pressure for the concrete structure should be computed as per the Clause 6.12.2 (b) of Canadian Highway Bridge Design Code (CHBDC, 2014). Sufficient movement of the structure wall may not be permitted and “at rest” conditions may be assumed for the calculation of earth pressure. The earth pressure calculation should include maximum water level expected in the creek. The lateral earth and water pressure, p (kPa), may be computed using the equivalent fluid pressures presented in Section 6.12 of the CHBDC 2014 or employing 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, P = lateral earth pressure (kPa)
K = lateral earth pressure coefficient
γ = unit weight of backfill material above assumed water level (kN/m^3)
γ' = unit weight of submerged backfill ($\gamma - \gamma_w$) 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 (m), above assumed water level
h_2 = depth below design water level (m)
q = surcharge load (kPa)
C_p = compaction pressure (refer to clause 6.12.3 of CHBDC 2014)
C_s = earth pressure induced by seismic events, kPa (refer to clause 4.6.5 of CHBDC 2014)

Where Ø = angle of internal friction of retained soil (35º for Granular A or 30º for Granular B Type II)
δ = angle of friction between soil and wall (24º for Granular A or B Type II)

The seismic site coefficient for the conditions at this site is provided in Section 10 of this report. Granular 'A' or 'B' should be utilized as backfill material and should be carried out in accordance with the requirements specified in the OPSS 902. The following parameters are recommended for the granular backfill:

Table 10.0: Recommended Geotechnical Parameters

GEOTECHNICAL PARAMETER	GRANULAR A	GRANULAR B TYPE II
Angle of Internal Friction, degrees	35°	30°
Unit Weight, kN/m^3	22.5	21.5
Coefficient of Active Earth Pressure (K_a)	0.27	0.33
Coefficient of Earth Pressure at Rest (K_o)	0.43	0.5
Coefficient of Passive Earth Pressure (K_p)	3.69	3

A weeping tile system (OPSS.PROV 405 and OPSD 3190.100) and/or weep holes should be installed to minimize the build-up of hydrostatic pressure behind the wall. The weeping tiles should be surrounded by a properly designed granular filter or geotextile to prevent migration of fines into the system. The drainage pipe should be installed on a positive grade.

Backfilling adjacent to abutment and retaining structures should be carried out in conformance with OPSS 902. The minimum requirement of granular backfill material behind abutment should be in accordance with OPSD 3101.150 and for retaining walls should be in accordance with OPSD 3121.150. The granular material should be in accordance OPSS.PROV 1010.

11. APPROACH EMBANKMENT

Based on the GA drawing, it is anticipated that the grades of approach embankments will be set at approximately 0.6 m and 0.9 m above the existing south and north abutments, respectively. The existing bridge was constructed in 1941 and the approach embankment fill is in place for more than seventy-five (75) years. The approaches or structure has no record of rehabilitation. Considering the consistency of fill material and underlying clayey silt, slope stability problems are not anticipated and stability analysis was not carried out to confirm the stability of approach embankments.

Considering the subsoil conditions at this site, no major instability problems are anticipated for the embankments constructed with 2H:1V side slope or flatter. Any spongy or soft area observed within the base of the embankment should be removed before placing the fill.

As indicated above, the existing fill and the clayey silt deposit is not expected to settle appreciably under the imposed load from a fill height ranging from 0.6 m to 0.9 m to report. Hence settlement problems are not anticipated with the height of fill required to achieve the proposed grade. However, it is suggested that the approach embankments fill be placed at least two to three weeks in advance of the construction of bridge and the paving of the road should be delayed by four to six weeks after placement of fill to the designed grade of the embankment, to mitigate the effects of post-construction settlement.

12. FOUNDATION FROST DEPTH

In accordance with OPSD 3090.100, a minimum of 2.5 m earth cover is required to protect against the frost penetration in the area where the site is located.

Frost tapers within the granular backfill should be constructed in accordance with OPSD 3101.150. The frost penetration depth, f, is measured from the top of the grade to the bottom of the footing.

13. SEISMIC CONSIDERATIONS

The Spectral (Sa(T), where T is in seconds) and Peak Ground Acceleration (PGA) for the project site is 0.216 (Sa(0.2)) and 0.140 (2%/50 years), respectively, based on the longitude and latitude coordinates of the proposed structure (National Building Code of Canada, 2015). The soil below the founding level at this site for seismic design purposes is classified as Type C in accordance with Clause 4.4.3.2, CHBDC 2014.

The Seismic Performance Category should be determined by the Regulatory Authority (MTO) and no information was provided in the RFP with regards to the category. In the absence of any information, it was assumed that the proposed replacement bridge is located on a Major Route and classified as Seismic Performance Category 2.

14. CONSTRUCTION CONSIDERATIONS

14.1 Excavation

All the excavations should be carried out in accordance with the Occupational Health and Safety Act (OHSA) and MTO Regulations for Construction Projects. In accordance with Ont. Reg.213/91, S. 226., the very stiff clayey silt may be classified as Type 2 soils. The fill soils may be classified as Type 3 soils. The slope of excavation walls should conform to as described in Ont. Reg. 213/92, S. 234. Since side slope geometry is governed by the soil with the highest number designation, temporary cut slopes over the full depth of excavation inclined at 1H:1V should be provided assuming adequate drainage measures are in place. Temporary shoring systems may be required if such slopes cannot be provided.

Construction Specifications for Excavating and Backfilling–Structures should be in accordance with OPSS 902. 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 to avoid potential damage.

14.2 Groundwater Control

The cut-off elevations of the piles to support the north and south abutments are proposed to be at El. 235.3 and El. 235.6, respectively. The depth of excavations at the abutment locations for construction of integral abutments are expected to be about 3.6 m below the existing ground level. In case, shallow foundations are opted, the depth of excavation may be in the order of 4.6 m.

The founding elevation of 236.0 recommended for TMB will not involve any excavation other than for removal of spongy or soft area and replacing with granular backfill.

Groundwater was encountered during drilling only in two (2) of the boreholes drilled in the area of replacement structure at a depth of 5.2 m (El. 234.5) and 8.0 m (El. 230.5) below the ground surface. Upon completion of drilling, groundwater was observed in both boreholes at a depth of 4.6 m (El. 235.1) below the ground surface. The water level in the creek was observed at approximate elevation of El. 235.0 during the fieldwork. Considering the depth of excavation and groundwater level in the area of the proposed replacement structure, no major dewatering problems are anticipated.

The groundwater level was observed in three of the boreholes drilled in the area of proposed detour at depths varying between of 0.8 m and 4.9 m during drilling. Upon completion of drilling, it was observed only in two of the boreholes at a depth of 6.1 m (El. 231.3) and 9.9 (El. 228.7) below the ground level. Considering the recommended footing elevation of the TMB, depth of excavation required and the groundwater level in the area of the proposed detour structure, no major dewatering problems are anticipated. However, the groundwater levels may fluctuate due to the influence of precipitation and seasonal changes.

It is considered that seepage from soil fissures or surface run-off that enters the excavations can be handled by conventional sump pumping techniques. The groundwater level should be lowered to a minimum of 0.5 m below the base of excavation. Refer to OPSS.PROV 517 and NSSP FOUN0003.

14.3 Soil Corrosivity

A total of three (3) samples from the fill were tested for soil corrosivity and potential exposure of concrete to sulphate attack. A summary of the results of chemical analyses are provided in section 6.0 of Part A of this report. The sulphate concentration varied from 6.2 µg/g to as high as 30 µg/g (0.00062% to 0.003%), which is less than 0.1% (1000 µg/g) generally indicates a low degree of sulphate attack. Compared to the values suggested in Canadian Standard A23.1-14, the effect of fill material on buried concrete structures may be negligible. The chloride contents of the samples from the fill ranged from as low as 6.9 µg/g to 300 µg/g (0.00069% to 0.03%). Generally, the concentration value in excess of 250 ppm (0.025%) leads to corrosive environment for buried metals or reinforcing steel. The potential for corrosive environment of this fill is assessed to be low to moderate.

Electrical resistivity less than 2000 ohm-cm generally leads to highly corrosive environment for steel elements in contact with soil. The resistivity values of fill samples ranged from 1750 ohm-cm to 6210 ohm-cm. The test results suggest that a corrosive environment exists at this site for steel elements in contact with fill where the resistivity was less than 2000 ohm-cm. The pH values of fill samples ranged from 7.85 to 8.09 compared to the value of 5.5 that generally leads to corrosion.

A total of three (3) samples from the clayey silt to silty clay deposit were tested for soil corrosivity and potential exposure of concrete to sulphate attack. A summary of the results of chemical analyses are provided in section 6.0 of Part A of this report. The sulphate concentration of the samples varied from 4.4 µg/g to 87 µg/g (0.00044% to 0.0087%), which is less than 0.1% (1000 µg/g) generally indicates a low degree of sulphate attack. Compared to the values suggested in Canadian Standard A23.1-14, the clayey soil may have negligible effect on buried concrete structures. The chloride content of the samples ranged from 3.4 µg/g to 14 µg/g (0.00034% to 0.0014%). Compared to the concentration value of 250 ppm (0.025%) that generally leads to corrosive environment for buried metals, the potential for this clayey soil is assessed to be low.

The resistivity values of samples from clayey silt to silty clay were higher than 2000 ohm-cm indicating negligible to low corrosive environment for steel elements in contact with this clayey soil. The pH values ranged from 8.04 to 8.45 compared to the value of 5.5 that generally leads to corrosion.

15. CLOSURE

This report was prepared by Ms. N. Leong-Sem, EIT, and Mr. N. Rahman, P.Eng., Geotechnical Services, and reviewed by Mark Vasavithasan, MSc.Eng., P.Eng. Senior Engineer, Geotechnical Services. Mr. R. Ng, MBA, PhD, P.Eng., MTO Designated Principal Contact, conducted an independent review of the report.

Yours very truly,

Peto MacCallum Ltd.

Natasha Leong-Sem
EIT
Geotechnical Services

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Project Manager, and
MTO Designated Principal Contac

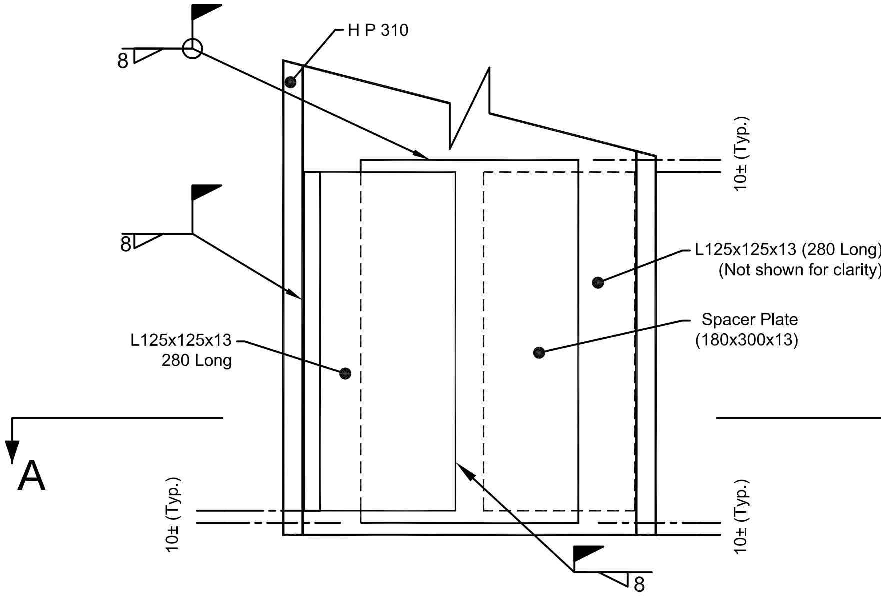
NL/NR/RN:nl-nk

APPENDIX C

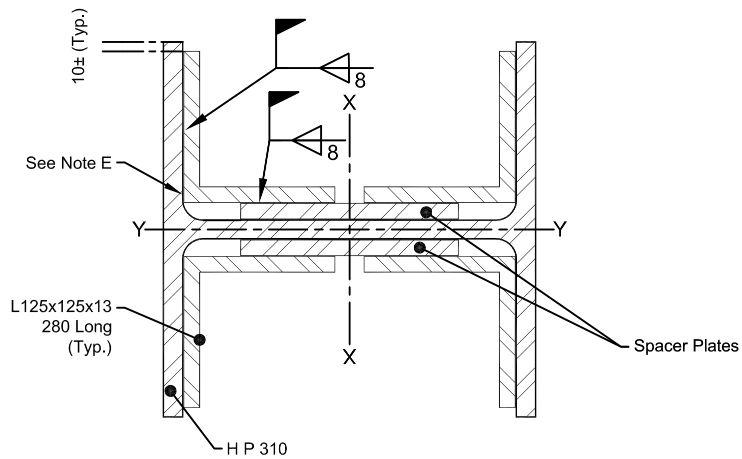
Sketch PML-1

List of Standard Specifications Relevant to Report

Non-Standard Special Provisions (NSSP)



ELEVATION



SECTION A - A

NOTES:

A. Pile tip reinforcement applies to piles HP310x79, HP310x110 & HP310x132.

B. Reinforcement steel shall be according to CSA G40.20/G40.21, Grade 300W.

C. Welding shall be according to CSA W59.

D. Spacer plate shall be 13 mm thick.

E. Chamfer corner of L-shape as required to be flat on flange.

F. Welds are symmetrical about both axis.

G. All dimensions are in millimetres unless otherwise shown.

H-PILE TIP REINFORCEMENT

BRIDGE FOUNDATION

PML Peto MacCallum Ltd.
CONSULTING ENGINEERS

DRAWN:	T.C.	DATE	SCALE	JOB NO.	SKETCH NO.
CHECKED:	M.V.	Jun. 2019	N.T.S	18TF002A	PML-1
APPROVED:	R.N.				

LIST OF STANDARD SPECIFICATIONS RELEVANT TO REPORT

DOCUMENT	TITLE
OPSS.PROV 405	Construction Specification for Pipe Subdrains
OPSS 501	Construction Specification for Compacting
OPSS 902	Excavation and Backfilling of Structures
OPSS.PROV 903	Construction Specification For Deep Foundations
OPSS.PROV 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade, and Backfill Material
OPSD 3090.100	Foundation, Frost Penetration depths for Northern Ontario
OPSD 3101.150	Walls, Abutment, Backfill, Minimum Granular Requirement
OPSD 3121.150	Wall, Retaining, Backfill, Minimum Granular Requirement
SP 109F12	Amendment to OPSS 902, November 2010
SP 109F57	Amendment to OPSS 903, April 2016
NSSP FOUN0003	Amendment to OPSS 902, November 2010

Special Provision No. 109F57 April 2018

903.03 DEFINITIONS

Section 903.03 of OPSS 903 is amended by the deletion of the definitions for Certificate of Conformance and Quality Verification Engineer.

903.04 DESIGN AND SUBMISSION REQUIREMENTS

903.04.02.04.02.01 Milestone Inspections

Clause 903.04.02.04.02.01 of OPSS 903 is deleted in its entirety.

903.04.02.06 Review of Splice Test Results and Permission to Proceed

Clause 903.04.02.06 of OPSS 903 is deleted in its entirety.

903.07 CONSTRUCTION

903.07.02.07.01 General

Clause 903.07.02.07.01 of OPSS 903 is amended by deleting the first paragraph in its entirety and replacing it with the following:

The driving of piles shall be carefully monitored and controlled and pile driving records produced for each pile under the direction of the Contractor. A pile driving record shall be submitted to the Contract Administrator.

903.07.02.07.03 Driving to a Specified Ultimate Resistance

903.07.02.07.03.01 General

Clause 903.07.02.07.03.01 of OPSS 903 is deleted in its entirety and replaced with the following:

When piles are specified to be driven to a specified ultimate resistance, the specified ultimate resistance shall be determined using the [* Designer Fill-In, See Notes to Designer] at end of initial driving as specified in the Contract Documents. If the specified ultimate resistance is not achieved, retap/restrike shall be conducted after initial driving as specified in the Contract Documents.

A Request to Proceed shall be submitted to the Contract Administrator after the design ultimate resistance is achieved.

The next operation shall not proceed until a Notice to Proceed has been received from the Contract Administrator.

903.07.02.07.03.03 Driving to Bedrock

Clause 903.07.02.07.03.03 of OPSS 903 is amended by deleting the last sentence in its entirety.

903.07.02.07.04 Wave Equation Analysis

Clause 903.07.02.07.04 of OPSS 903 is deleted in its entirety and replaced with the following:

When requested by the Contract Administrator, all equipment, material, and personnel shall be supplied to conduct the wave equation analysis procedure.

903.07.03.07 Concrete

903.07.03.07.01 General

Clause 903.07.03.07.01 of OPSS 903 is deleted in its entirety and replaced with the following:

A Request to Proceed shall be submitted to the Contract Administrator before the concrete placement.

The reinforcement shall not be displaced or distorted during the construction of the caisson.

The placement of concrete shall not proceed until the Contract Administrator has inspected the caisson hole and issued to the Contractor a Notice to Proceed.

Concrete shall be placed immediately after the Notice to Proceed has been received and shall be placed in the caisson according to OPSS 904 and as specified herein.

Arching of concrete during casing withdrawal shall be prevented.

903.07.03.07.05 Founding Elevation

Clause 903.07.03.07.05 of OPSS 903 is amended by deleting the last paragraph in its entirety and replacing it with the following:

Complete access to inspect the bearing area of the caisson pile prior to the placement of concrete shall be given to the Contract Administrator.

903.07.06 Load Test

Subsection 903.07.06 of OPSS 903 is amended by deleting the first paragraph in its entirety and replacing it with the following:

When a load test is specified in the Contract Documents, the testing shall be according to ASTM D 1143M for piles under vertical static load, ASTM D 3689 for piles under tensile load, and ASTM D 3966 for piles under lateral loads. The Contract Administrator shall witness the pile load test. All records and results of the pile load test shall be submitted to the Contract Administrator.

903.07.08.01.02 Visual Inspection of Welds

Clause 903.07.08.01.02 of OPSS 903 is deleted in its entirety and replaced with the following:

Complete access to visually inspect the welds shall be given to the Contract Administrator.

A representative sample of not less than 30% of the welds, as determined by the Contract Administrator, shall be visually inspected for conformance to the requirements of CSA W59 and the Contract Documents.

903.07.08.01.03 Non-Destructive Testing of Welds

Clause 903.07.08.01.03 of OPSS 903 is deleted in its entirety and replaced with the following:

Radiographic or ultrasonic testing shall be carried out using procedures according to CSA W59.

Ultrasonic or radiographic testing shall be carried out on the entire length of selected splice welds chosen at random by the Contractor's welding inspector assigned to carry out visual inspections.

Selection shall be based on the following criteria:

a) For pile groups other than at integral abutments, 10% of the splice welds, rounded to the next highest number, but no fewer than two.

b) For pile groups at integral abutments, 10% of the splice welds, rounded to the next highest number, but no fewer than two of when the welds are below 6 m of the pile cut-off elevation.

c) For pile groups at integral abutments, all splice welds within 6 m of the pile cut-off elevation.

903.07.08.03 Certificate of Conformance

Clause 903.07.08.03 of OPSS 903 is deleted in its entirety.

903.10 BASIS FOR PAYMENT

903.10.01 Supply Equipment for Installing Driven Piles - Item
Supply Equipment for Installing Caisson Piles - Item
Supply Equipment for Installing Displacement Caisson Piles - Item

Subsection 903.10.01 of OPSS 903 is amended by deleting the second paragraph in its entirety and replacing it with the following:

For payment purposes, 50% of the work under this item shall be paid when the satisfactory performance of the equipment has been demonstrated to the Contract Administrator by the installation of 1% of piles.

Another 40% shall be paid by progress payments proportional to the work completed. The remaining 10% shall be paid on the satisfactory completion of the installation of piles.

NON-STANDARD SPECIAL PROVISIONS (NSSP)

NSSP-1 – Potential for Cobbles and Boulders during Pile Driving

The Contractor shall be advised that cobbles and boulders were identified at the interface of clayey silt layer and silty sand glacial till deposit during the advancement of boreholes in the area of proposed abutments.

Hence, the Contractor shall allow for these obstructions during the installation of piles. If during pile driving there is evidence that a pile meets refusal on a boulder, the contractor shall inform the Contract Administrator. The obstructions at the pile locations where encountered shall be removed by pre-auguring and the pile shall be driven to the ultimate capacity or to the tip elevation specified in the contract. After the installation of pile to the required depth, the annular space of pre-augured holes to remove the boulders shall be filled with concrete up to the depth where loose sand fill required to be placed.

Alternatively, piles meeting refusal on a boulder may be relocated, have their capacity reduced, and / or require additional piles to be installed as directed by the Contract Administrator.

The contractor shall also consider the difficulties associated with the excavation for drilled shafts because of the presence of cobbles and boulders within the silty sand glacial till deposit.

DEWATERING STRUCTURE EXCAVATIONS - Item No.

Special Provision No. FOUN0003

Amendment to OPSS 902, November 2010

902.02 REFERENCES

Section 902.02 of OPSS 902 is amended by the addition of the following:

Ontario Provincial Standard Specifications, Construction

OPSS 517 Dewatering
OPSS 805 Temporary Erosion and Sediment Control Measures

902.03 DEFINITIONS

Section 903.03 of OPSS 902 is amended by the addition of the following:

Automatic Transfer Switch means as defined in OPSS 517.

Cofferdam means as defined in OPSS 539.

Cut-Off Wall means as defined in OPSS 517.

Design Storm Return Period means as defined in OPSS 517.

Groundwater Control System means as defined in OPSS 517.

Plug means as defined in OPSS 517.

Sediment means as defined in OPSS 517.

Sediment Control Measure means as defined in OPSS 517.

Temporary Flow Passage System means as defined in OPSS 517.

Unwatering means as defined in OPSS 517.

Vegetated Discharge Area means as defined in OPSS 517.

Waterbody means as defined in OPSS 517.

Watercourse means as defined in OPSS 517.

902.04 DESIGN AND SUBMISSION REQUIREMENTS

902.04.01 Design Requirements

902.04.01.01 Dewatering

Clause 902.04.01.01 of OPSS 902 is deleted in its entirety and replaced with the following:

A dewatering system shall be designed to control water and the flow of water into the excavation, prevent disturbance of the foundation, permit the placing of concrete in the dry, and complete the excavating and backfilling for structures work.

When the system includes temporary flow passage system, the system shall be designed, as a minimum, for a 5-year design storm return period, and groundwater discharge. A longer return period shall be used when determined appropriate for the work.

The dewatering system shall be according to the design requirements specified in OPSS 517.

902.04.02 Submission Requirements

Subsection 902.04.02 of OPSS 902 is deleted in its entirety and replaced with the following:

902.04.02.01 Preconstruction Survey

When a groundwater control system by wells or a well point system will be used, a condition survey of property and structures that may be affected by the work shall be carried out. The condition survey shall include the location and condition of adjacent properties, buildings, underground structures, water wells, Utilities, and structures, within a distance of 100 metres from the groundwater control system. In addition, all water wells used as a supply of drinking water and located within this distance shall be tested for compliance with Ontario Drinking Water Quality Standards.

Water wells within the preconstruction survey distance can be located using the website https://www.ontario.ca/environment-and-energy/map-well-records or its successor site.

Copies of the condition survey and water quality test results shall be submitted to the Contract Administrator prior to the operation of the groundwater control system.

902.04.02.02 Working Drawings

Working Drawings for the dewatering system shall be according to OPSS 517.

902.07 CONSTRUCTION

902.07.04 Dewatering Structure Excavation

Subsection 902.07.04 of OPSS 902 is amended by the addition of the following clauses:

902.07.04.01 General

The dewatering systems shall be constructed and operated according to the Working Drawings.

Activation and deactivation of a temporary flow passage system, if applicable, shall be according to OPSS 517.

The dewatering system shall be continuously operational to control buoyancy forces until such forces can be resisted by backfill and structure self-weight, to keep excavations stable, to avoid erosion impacts from the release of accumulated water, and to keep the work area in the condition required to complete the associated work as specified in the Contract Documents.

When a temporary flow passage system is to remain operational through a seasonal shutdown period, the Contractor shall be responsible for any maintenance or repair costs due to the system during the seasonal shutdown period.

Temporary erosion and sediment control measures, including controlling the discharge of water, shall be according to OPSS 805. Measures not specified in OPSS 805 shall be according to the Working Drawings. Temporary erosion and sediment control measures and cover material to protect exposed soils, as required by the Working Drawings, shall be installed as soon as is practical.

Stranded fish shall be managed as specified in the Contract Documents.

Unwatering shall be carried out as necessary.

Water suspected of being contaminated as indicated by visual or olfactory observations shall be reported to the Contract Administrator.

Dewatering and temporary flow passage systems shall be discontinued in a manner that does not disturb any structure, pipeline, or flow channel. Operation of the dewatering system shall be shut down according to the procedures specified in the Working Drawings, where applicable.

902.07.04.02 Discharge of Water

The discharge of water shall be according to OPSS 517.

902.07.04.03 Monitoring

Monitoring shall be according to OPSS 517.

902.07.04.04 System Amendments

Amendments to stop any displacement, damage, soil loss or erosion due to the operation of the dewatering system shall be according to OPSS 517.

902.07.04.05 Removal

Removal of dewatering system and temporary flow passage system components shall be according to OPSS 517.

NOTES TO DESIGNER:

* Fill in the design storm return period according to MTO Drainage Design Standard TW-1.

** Fill in the preconstruction survey distance as recommended by the foundation engineer.