



FOUNDATION INVESTIGATION AND DESIGN REPORT

for

DECEPTION CREEK BRIDGE REPLACEMENT

SITE NO. 39E-0169/B0

HIGHWAY 668 – STATION 10+265

TOWN OF COCHRANE, ONTARIO

G.W.P. 5267-11-00

W.P. 5368-11-01

LATITUDE AND LONGITUDE: 49.27422, -81.07419

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PML Ref.: 18TF002A
Index No.: 038FIR and 039FDR
GEOCRES No.: 42H-82
August 21, 2019



PART A - FOUNDATION INVESTIGATION REPORT

for

DECEPTION CREEK BRIDGE REPLACEMENT

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HIGHWAY 668 – STATION 10+265

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August 21, 2019

Part A – Foundation Investigation Report

Deception Creek Bridge Replacement, Site No. 39E-0169/B0, Highway 668 – Station 10+265
Town of Cochrane, Ontario, G.W.P. 5267-11-00, W.P. 5368-11-01, Index No.: 038FIR
PML Ref.: 18TF002A, August 21, 2019, TOC 1 of 1



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PART A - FOUNDATION INVESTIGATION REPORT

Deception Creek Bridge Replacement

Site No. 39E-0169/B0

Highway 668 – Stations 10+265

Town of Cochrane, Ontario

G.W.P. 5267-11-00, W.P. 5267-11-01 and 5363-11-01

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 1, to prepare detail design for the replacement of the existing bridge, located along Highway 668 at the crossings of Deception Creek. Report for Contract Package 2 was 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 locations 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 bridges located along Highway 668 at the crossings of Deception Creek (Station 10+265) 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 bridges and to aid the designer in selecting the suitable type of foundation to support the replacement structures.



2. SITE DESCRIPTION

The location of the existing Deception Creek bridge is approximately 6 km north of Highway 11. Highway 668 in the area of the 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. Deception Creek flow from west to east, almost perpendicular to Highway 668. 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 field work for the foundation investigation involved advancing seven (7) boreholes at the Deception Creek bridge site. The boreholes were drilled to depths ranging from 11.3 m to 20.1 m below the existing ground surface.

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 DC-1 to DC-4 and DRW-1 to DRW-3 were drilled between April 11 and April 30, 2019. The boreholes were advanced using a B57 track-mounted drilling rig equipped with 200 mm diameter hollow stem augers.

Refer to Drawings DC-1, DC-2, DC-3 in Appendix A for borehole location details for Deception Creek Bridge.

Part A – Foundation Investigation Report

Deception Creek Bridge Replacement, Site No. 39E-0169/B0, Highway 668 – Station 10+265
Town of Cochrane, Ontario, G.W.P. 5267-11-00, W.P. 5368-11-01, Index No.: 038FIR
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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 Deception Creek was observed at approximate El. 262.0 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 (88)
- Grain size distribution analysis (26)
- Atterberg limit tests (17)
- Compressive strength of Rock Cores (4)

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 DC-GS-1 to DC-GS-3. The results of the Atterberg Limit tests are presented on Figures DC-PC-1, DC-PS-2. 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 culvert 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.

Preliminary foundation investigation was carried out by others at the site location between September 14 and September 17, 2015, and the report is available in the MTO Geocres Library under Geocres Nos. 42H-64. In general, the subsurface conditions encountered during the exploration program conducted are consistent with the geology described in the preliminary foundation investigation Report (FIR) dated September 16, 2016. The previous borehole logs and drawings are attached in Appendix B of this report.

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 DC-1, DC-2 and DC-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 retaining wall 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 0.9 m to 3.5 m thick fill composed of either silty sand or clayey silt, which is followed by soft to stiff clayey silt. Along the alignment of the existing road, the clayey silt layer is underlain by loose silt. The clayey silt and silt layers are followed by very loose to very dense silty sand to sandy silt till. In boreholes DC-2 and DC-3, the silty sand to sandy silt till is underlain by bedrock, which extends to the maximum

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borehole termination depth of 20.1 m below the existing ground surface. For classification purposes, the soils encountered at this site can be divided into eight (8) distinct zones:

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

5.2.1 Topsoil

A layer of topsoil, approximately 100 mm in thickness, was encountered in all of the boreholes (DRW-1 to DRW-3) drilled off-road along the alignment of proposed retaining wall.

5.2.2 Pavement Structure

A pavement structure was encountered in Boreholes DC-1 to DC-4. This pavement structure consisted of 20 mm to 40 mm of surface treated pavement (PST) over 110 mm of granular base, followed by 110 mm to 150 mm of granular subbase.

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

A clayey silt fill layer was encountered immediately below the pavement structure in Boreholes DC-1 to DC-4 and in Borehole DRW-1, it was encountered below the topsoil. This layer extends to 2.1 m to 3.8 m (El. 264.2 to El. 261.7) below the existing ground surface.

The SPT 'N' values in the clayey silt fill in Boreholes GC-1 to GC-4 ranged from as low as 7 blows to 36 blows, indicating firm to hard consistency. Whereas, the SPT 'N' values in the clayey silt fill layer in Boreholes DRW-1 ranged from 3 to 7 blows, indicating soft to firm consistency.

The moisture content of the samples tested from this layer ranged from 18.5% to 34.6%. However, the moisture content of one sample was found to be at 87.9%.



5.2.4 Silty Sand, Trace/With Gravel (Fill)

A silty sand fill layer was encountered immediately below the topsoil in Boreholes DRW-2 to DRW-3 and extends to 1.0 m and 1.8 m (El. 261.5 and El. 260.9) below the existing ground surface.

The SPT 'N' values in the silty sand fill layer varies between 9 and 10 blows, indicating a loose state of compaction. The moisture content of the samples tested ranged from 52.3% to 58.3%.

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

The fill layer in all of the boreholes, with the exception of Borehole DC-1, is underlain by this silty clay to clayey silt deposit with varying proportions of sand and gravel. In Borehole DC-1, it was encountered immediately below the silt deposit, which is described in Section 5.2.6 below. This silty clay to clayey silt layer extends to 3.8 m to 10.7 m (El. 262.1 to El. 254.7) below the existing ground surface.

Generally, the SPT N-Values in this deposit varies between 1 blow to 12 blows, indicating very soft to stiff consistency.

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

The moisture content of the samples ranged from 17.1% to 48.2%. However, the moisture content of one sample was found to be at 64.6%. Sieve analysis tests were performed on 10 representative samples and the test results indicate that this deposit consists of none to 10% gravel, none to 20% sand, 52% to 88% silt, and 14% to 31% clay. Atterberg limits were performed on those 10 representative samples and the test results indicate liquid limit values range from 24 to 48, plastic limit values from 17 to 31, and corresponding plasticity index values range from 5 to 17. Based on the test results, the clayey soil may be classified as clay of low to medium plasticity (CL-ML/CL/CI) in the Unified Soil Classification System (USCS), i.e., clayey silt/silty clay.

5.2.6 Silt, Tace/Some Sand, Trace Gravel

A silt layer was encountered immediately below the clayey silt layer in all boreholes, except Borehole DC-1 where it was encountered below the fill. This layer extends to 4.5 m to 10.7 m

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(El. 260.9 to El. 254.0) below the existing ground surface. It was not fully penetrated in Borehole DC-4 to establish the thickness of the deposit.

Generally, the SPT N-Values in this deposit varies between 2 blows to 10 blows, indicating a very loose to loose state of compaction.

The grain size distribution results of selected clayey silt samples from this deposit are provided on Figures DC-GS-2, and the results of Atterberg limits for the same samples are provided on Figures DC-PC-2 in Appendix A.

The moisture content of the samples ranged from 11.5% to 55.3. Sieve analysis tests were performed on 10 representative samples and the test results indicate that this deposit consists of none to none to 2% gravel, none to 3% sand, 83% to 91% silt, and 9% to 16% clay. Atterberg limits were performed on those nine (9) representative samples and the test results indicate liquid limit values range from 21 to 25, plastic limit values from 19 to 21, and corresponding plasticity index values range from 1 to 4. Based on the test results, the clayey soil may be classified as silt of slight plasticity (ML) in the Unified Soil Classification System (USCS), i.e., silt.

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

The clayey silt deposit in Boreholes DC-1, and the silt layer in all other boreholes is followed by a silty sand to sandy silt till deposit to a maximum depth of 16.8 m (El. 149.1) where bedrock was encountered.

The SPT N-values in this deposit to about El. 250.5 varies from none (penetration under the weight of hammer and rods) to 12 blows, indicating very loose to compact state of compaction. The till deposit below El. 250.5 was generally found to be very dense with SPT values ranging from 50 blows to over 100 blows.

The results of grain size distribution of selected samples from till deposit are provided on Figure GS-3 in Appendix A.

The moisture content of the samples ranged from 7.0% to 29.7%. Sieve analysis tests were performed on six (6) representative samples and the test results indicate that this deposit consists of none to 38% gravel, 47% to 58% sand, 13% to 39% silt, and 2% to 5% clay size particles.



5.2.8 Bedrock

Bedrock was encountered in Boreholes DC-2 and DC-3, immediately below the till deposit at about elevations varying from El. 250.1 to El. 249.1. The presence of bedrock was confirmed by obtaining 3.0 m of rock cores from both boreholes. These boreholes were advanced using an HQ sized double core barrel and wash boring with a 75 mm diameter NW casing. The rock core recovery ranged from 95% to 100% and the Rock Quality Designation (RQD) of the rock cores ranged from 76% to 100%. Based on the RQD value, the quality of the bedrock at this site to about El. 248.7 may be described as very poor. The quality of the bedrock below El. 248.7 may be described as fair to excellent. The bedrock was identified as unweathered diabase. For complete description of the bedrock, refer to the Rock Core Photographs and the Rock Core Description logs provided in Appendix A.

Compressive strength of selected rock core samples ranged from 108.6 MPa to 194.7 MPa. Refer to appendix A for details.

5.2.9 Groundwater

Groundwater was encountered during drilling in two (2) of the boreholes (DC-1 and DRW-3) at depths ranging from 4.6 m (El. 258.1) to 10.7 m (El. 254.7) below the ground surface. Upon completion of drilling, groundwater was encountered in two (2) of the boreholes (DC-1 and DRW-3) at depths ranging from 7.0 m (El. 258.4) to 9.9 m (El. 252.8) below the ground surface. The water level in the creek was observed at approximate elevation of El. 262.0 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.

5.2.10 Soil Corrosivity

Four (4) 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 5.2.10.

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Deception Creek Bridge Replacement, Site No. 39E-0169/B0, Highway 668 – Station 10+265

Town of Cochrane, Ontario, G.W.P. 5267-11-00, W.P. 5368-11-01, Index No.: 038FIR

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Table 5.2.10: 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)
DC-3	2	1	<0.02	284	8.29	4810	208	7.0	40
DC-3	3	1	<0.02	290	8.25	4690	213	7.4	18
DRW-2	3	4.5	0.02	218	8.15	5170	194	52	2.1
DRW-3	3	4.5	0.04	179	7.96	4460	224	4.0	5.6

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6. 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, Geotechnical Services and reviewed by Mr. K. Amatya, MEng, P.Eng., Project 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
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Project Engineer
Geotechnical Services



Robert Ng, MBA, PhD, P.Eng.
Project Manager and
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Part A – Foundation Investigation Report

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Town of Cochrane, Ontario, G.W.P. 5267-11-00, W.P. 5368-11-01, Index No.: 038FIR

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APPENDIX A

Borehole Locations Plan and Soil Strata Drawing DC-1, DC-2 and DC-3

Explanation of Terms Used in Report

Record of Borehole Sheets

Results of Grain Size Distribution Analyses – Figures DC-GS-1 to 3

Results of Atterberg Limit Tests – Figures DC-PC-1/2

Rock Core Photographs

Rock Core Descriptions

Rock Core Compressive Strength Test Results

Results of Chemical Tests provided by SGS Canada Inc.

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



DECEPTION CREEK STRUCTURE
 HIGHWAY 668-STATION 10+265
 BOREHOLE LOCATIONS

SHEET



LEGEND

- Foundation Borehole for Structure
- Foundation Borehole for Retaining Wall
- Previous Borehole (Geocres No. 42H-64)

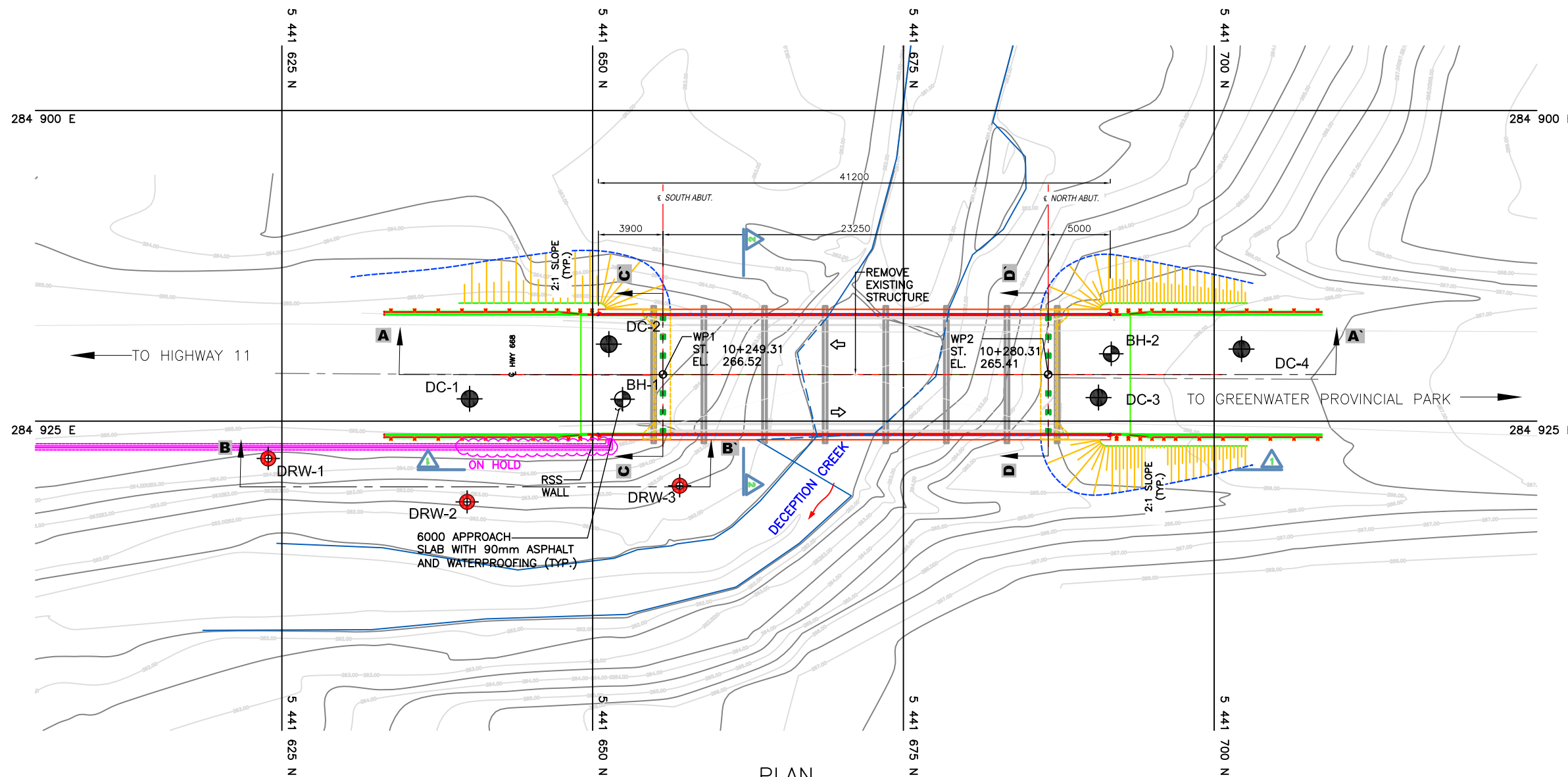
BH No	ELEVATION	EASTINGS	NORTHINGS
DC-1	265.4	5 441 640.1	284 923.2
DC-2	265.5	5 441 651.3	284 918.8
DC-3	265.9	5 441 690.7	284 923.1
DC-4	266.3	5 441 702.2	284 919.2
DRW-1	264.4	5 441 623.9	284 928.0
DWR-2	262.5	5 441 639.9	284 931.5
DWR-3	262.7	5 441 657.0	284 930.2
PREVIOUS BOREHOLES (GEOCRENS NO. 42H-64)			
BH-1	265.4	5 441 652.1	284 923.2
BH-2	265.8	5 441 691.7	284 919.6

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

DATE	BY	DESCRIPTION

Geocres No. 42H-82

HWY No 668	DIST Northern
SUBM'D TC	CHECKED NR
DATE AUG. 19, 2019	SITE 39E-0169/B0
DRAWN TC	CHECKED RN
APPROVED RN	DWG. DC-1

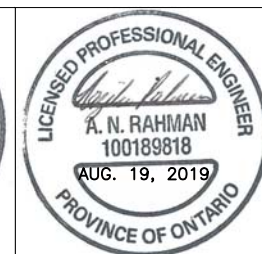
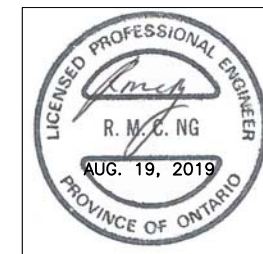


PLAN

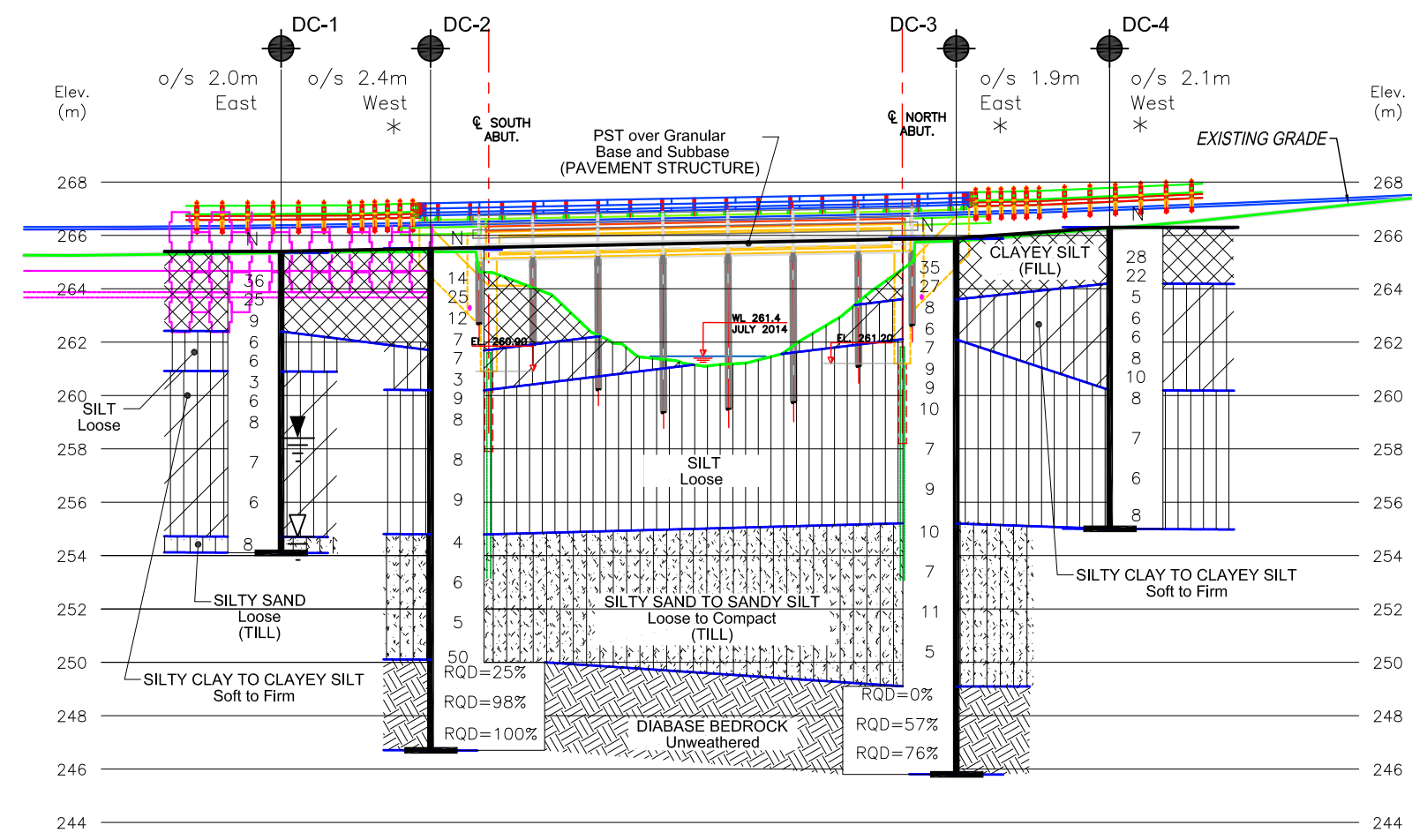
SCALE



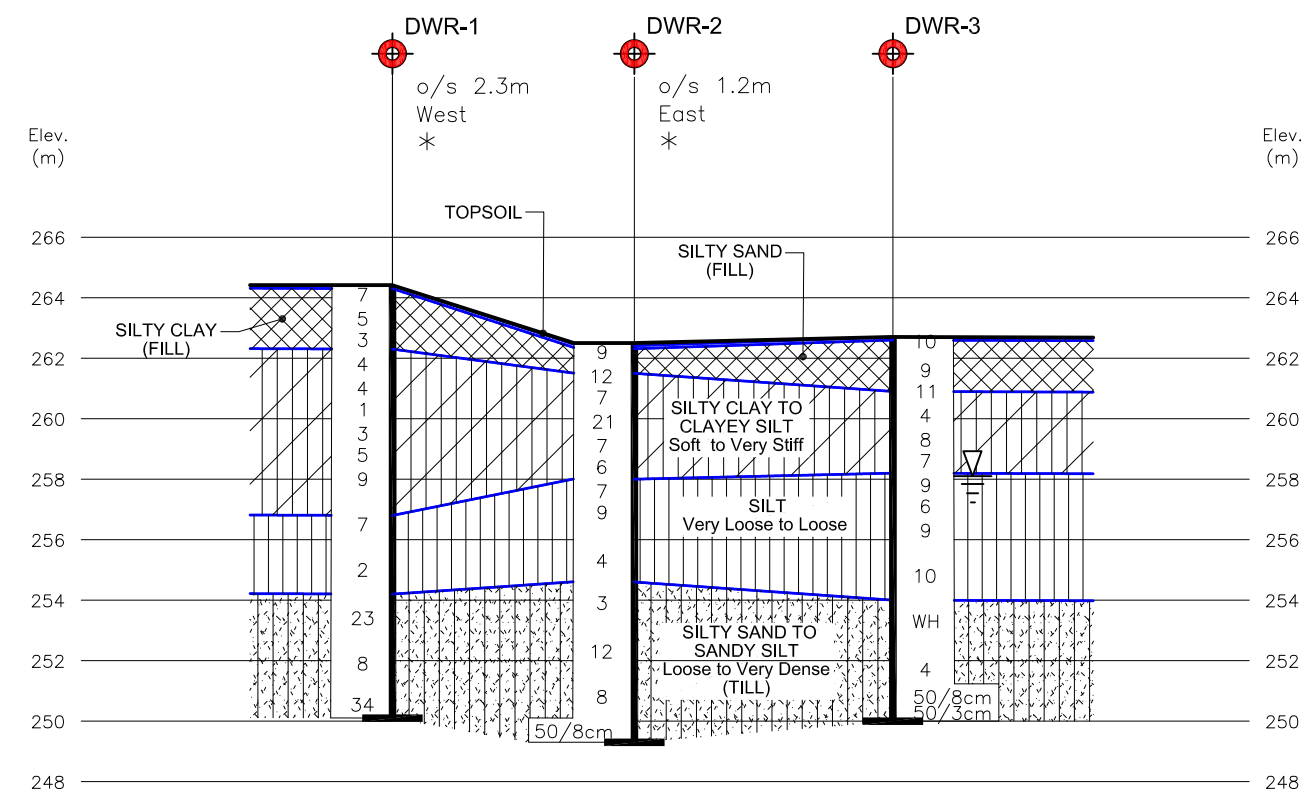
- NOTES:**
- THIS DRAWING SHOULD BE READ IN CONJUNCTION WITH THE TEXT OF REPORT AND RECORD OF BOREHOLE LOGS.
 - DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS ARE IN KILOMETRES AND METRES.
 - REFER TO DRAWING DC-2 FOR SOIL PROFILES A-A' AND B-B' AND DRAWING DC-3 FOR PROFILES C-C' AND D-D'.



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



PROFILE ALONG A-A'



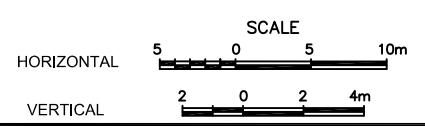
PROFILE ALONG B-B'

LEGEND

- Foundation Borehole for Structure
- Foundation Borehole for Retaining Wall
- Blows/0.3m (Std. Pen Test, 475 J/blow)
- Penetration Due to Weight of Hammer and Rods
- WL measured during drilling
- WL measured after during
- Water level could not be established

BH No	ELEVATION	NORTHINGS	EASTINGS
DC-1	265.4	5 441 640.1	284 923.2
DC-2	265.5	5 441 651.3	284 918.8
DC-3	265.9	5 441 690.7	284 923.1
DC-4	266.3	5 441 702.2	284 919.2
DRW-1	264.4	5 441 623.9	284 928.0
DRW-2	262.5	5 441 639.9	284 931.5
DRW-3	262.7	5 441 657.0	284 930.2

- NOTES:
- THIS DRAWING SHOULD BE READ IN CONJUNCTION WITH THE TEXT OF REPORT AND RECORD OF BOREHOLE LOGS.
 - DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS ARE IN KILOMETRES AND METRES.
 - REFER TO DRAWING DC-1 FOR BOREHOLE LOCATION PLAN AND DRAWING DC-3 FOR PROFILES C-C' AND D-D'.



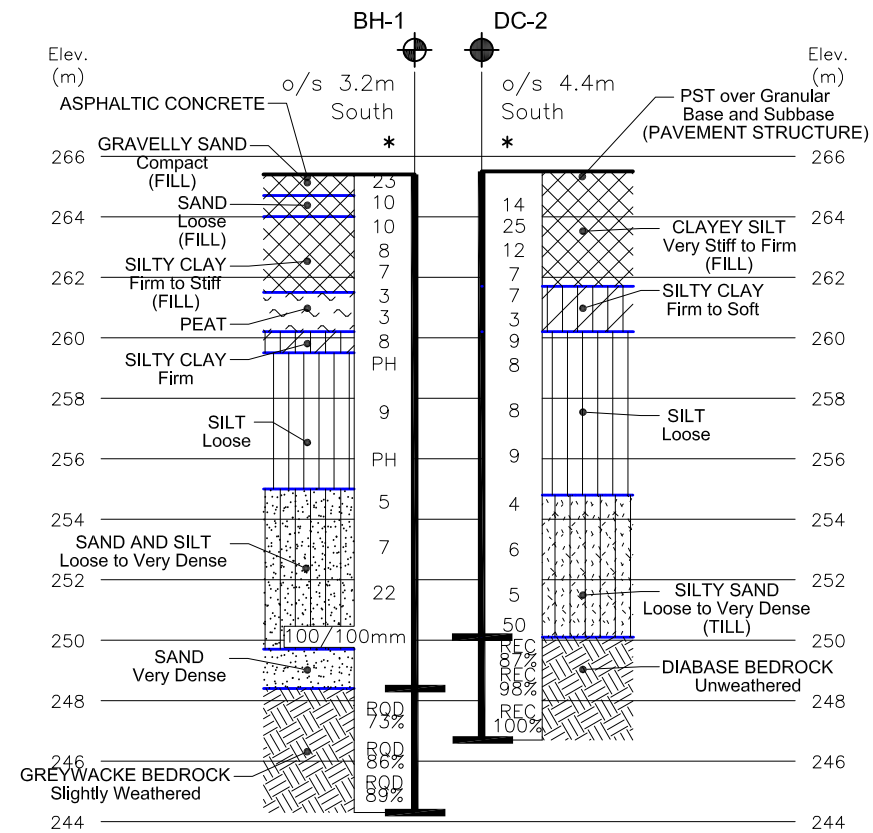
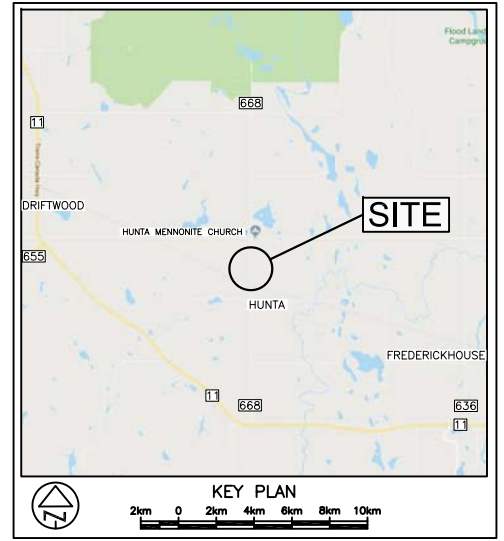
NOTE: The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.

DATE	BY	DESCRIPTION

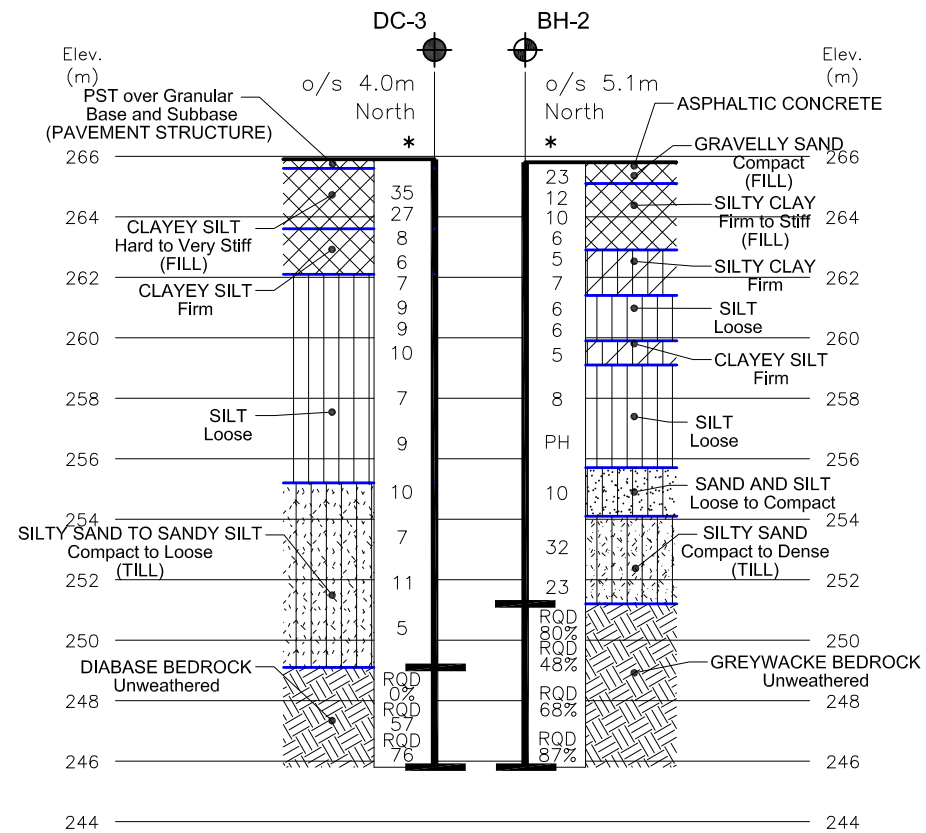
GWP No 5267-11-00
 WP No 5368-11-01

DECEPTION CREEK STRUCTURE
 HIGHWAY 668-STATION 10+265
 BOREHOLE LOCATIONS

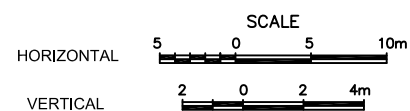
SHEET



PROFILE ALONG C-C'



PROFILE ALONG D-D'



LEGEND

- Foundation Borehole for Structure
- Previous Borehole (Geocres No. 42H-64)
- Blows/0.3m (Std. Pen Test, 475 J/blow)
- Water level could not be established

BH No	ELEVATION	NORTHINGS	EASTINGS
DC-2	265.5	5 441 651.3	284 918.8
DC-3	265.9	5 441 690.7	284 923.1
PREVIOUS BOREHOLES (GEOCRENS NO. 42H-64)			
BH-1	265.4	5 441 652.1	284 923.2
BH-2	265.8	5 441 691.7	284 919.6

NOTES:

- THIS DRAWING SHOULD BE READ IN CONJUNCTION WITH THE TEXT OF REPORT AND RECORD OF BOREHOLE LOGS.
- DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS ARE IN KILOMETRES AND METRES.
- REFER TO DRAWING DC-1 FOR BOREHOLE LOCATION PLAN AND DRAWING DC-2 FOR PROFILES A-A' AND B-B'



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

DATE	BY	DESCRIPTION

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

Geocres No. 42H-82			
HWY No 668	CHECKED KA	DATE AUG. 19, 2019	DIST Northern
SUBM'D TC	CHECKED NR	APPROVED RN	SITE 39E-0169/B0
			DWG. DC-3

EXPLANATION OF TERMS USED IN REPORT

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

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

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

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

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

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

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

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

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

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

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

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

R Q D (%)	0-25	25-50	50-75	75-90	90-100
	VERY POOR	POOR	FAIR	GOOD	EXCELLENT

JOINTING AND BEDDING:

SPACING	50mm	30-300mm	0.3m-1m	1m-3m	>3m
JOINTING	VERY CLOSE	CLOSE	MOD. CLOSE	WIDE	VERY WIDE
BEDDING	VERY THIN	THIN	MEDIUM	THICK	VERY THICK

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

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

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
i_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'_{v0}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	-°	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_l	1	SENSITIVITY = $\frac{c_u}{\tau_r}$

PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m^3	DENSITY OF SOLID PARTICLES	n	1, %	POROSITY	e_{max}	1, %	VOID RATIO IN LOOSEST STATE
γ_s	kN/m^3	UNIT WEIGHT OF SOLID PARTICLES	w	1, %	WATER CONTENT	e_{min}	1, %	VOID RATIO IN DENSEST STATE
ρ_w	kg/m^3	DENSITY OF WATER	S_r	%	DEGREE OF SATURATION	I_D	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
γ_w	kN/m^3	UNIT WEIGHT OF WATER	w_L	%	LIQUID LIMIT	D	mm	GRAIN DIAMETER
ρ	kg/m^3	DENSITY OF SOIL	w_p	%	PLASTIC LIMIT	D_n	mm	n PERCENT - DIAMETER
γ	kN/m^3	UNIT WEIGHT OF SOIL	w_s	%	SHRINKAGE LIMIT	C_u	1	UNIFORMITY COEFFICIENT
ρ_d	kg/m^3	DENSITY OF DRY SOIL	I_p	%	PLASTICITY INDEX = $w_L - w_p$	h	m	HYDRAULIC HEAD OR POTENTIAL
γ_d	kN/m^3	UNIT WEIGHT OF DRY SOIL	I_L	1	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$	q	m^3/s	RATE OF DISCHARGE
ρ_{sat}	kg/m^3	DENSITY OF SATURATED SOIL	I_C	1	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$	v	m/s	DISCHARGE VELOCITY
γ_{sat}	kN/m^3	UNIT WEIGHT OF SATURATED SOIL	DTPL		DRIER THAN PLASTIC LIMIT	i	1	HYDRAULIC GRADIENT
ρ'	kg/m^3	DENSITY OF SUBMERGED SOIL	APL		ABOUT PLASTIC LIMIT	k	m/s	HYDRAULIC CONDUCTIVITY
γ'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL	WTPL		WETTER THAN PLASTIC LIMIT	j	kN/m^2	SEEPAGE FORCE
e	1, %	VOID RATIO						

RECORD OF BOREHOLE No DC-1

1 OF 1

METRIC

G.W.P. 5267-11-00 LOCATION Coords: 5 441 640.1 N; 284 923.2 E ORIGINATED BY M.M./F.M.
 DIST Northern HWY 688 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY K.A.
 DATUM Geodetic DATE 2019.04.11 LATITUDE 49.112587 LONGITUDE -81.272287 CHECKED BY R.N.

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	20						40	60	80
265.4	Ground Surface															
0.0	40 mm PST															
265.1	over 110 mm Granular Base over 150 mm Granular Subbase (PAVEMENT STRUCTURE)															
0.3	CLAYEY SILT, some sand, trace gravel															
	Hard to very stiff, Brown, Moist (FILL)		1	SS	36											
			2	SS	25											
			3	SS	9											
262.4	SILT, some sand, trace gravel															
3.0	Loose, Grey, Wet		4	SS	6											
			5	SS	6											
260.9	SILTY CLAY TO CLAYEY SILT, some sand, trace gravel															
4.5	Soft to firm, Grey, Wet		6	SS	3							0	20	59	21	
			7	SS	6							10	16	60	14	
			8	SS	8							0	0	85	15	
			9	SS	7											
			10	SS	6											
254.7	SILTY SAND, trace gravel															
10.7	Loose, Grey, Wet (TILL)		11	SS	8								7	58	32	3
254.1	End of borehole															
11.3																

ONTARIO MTO 18TF002 DC(JUNE 2019).GPJ ONTARIO MTO.GDT 6/11/19

▽ 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 7.3 m (El. 258.1) below the existing ground surface.

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

RECORD OF BOREHOLE No DC-2

2 OF 2

METRIC

G.W.P. 5267-11-00 LOCATION Coords: 5 441 651.3 N; 284 918.8 E ORIGINATED BY M.M.
 DIST Northern HWY 688 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY K.A.
 DATUM Geodetic DATE _____ LATITUDE 49.112688 LONGITUDE -81.272348 CHECKED BY R.N.

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	20	40	60	80						100	SHEAR STRENGTH kPa	
250.5			14	SS	50														
250.1			15	RC HQ	REC 87%														RQD=25%
15.4	Unweathered, fine grained to medium grained Dark grey to black, Hard, Crystalline Diabase Bedrock		16	RC HQ	REC 98%														RQD=98% UCS=132.9 MPa
			17	RC HQ	REC 100%														RQD=100% UCS=128.9 MPa
246.7																			
18.8	End of borehole NOTES: 1. Groundwater level was not encountered inside the borehole during or upon completion of drilling. 2. No cave-in was noted in the borehole upon extraction of hollow stem augers.																		

ONTARIO MTO 18TF002 DC(JUNE 2019).GPJ ONTARIO MTO.GDT 7/30/19

RECORD OF BOREHOLE No DC-3

1 OF 2

METRIC

G.W.P. 5267-11-00 LOCATION Coords: 5 441 690.7 N; 284 923.1 E ORIGINATED BY M.M.
 DIST Northern HWY 688 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY K.A.
 DATUM Geodetic DATE _____ LATITUDE 49.113042 LONGITUDE -81.27229 CHECKED BY R.N.

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa									
						20	40	60	80	100							
265.9	Ground Surface																
0.0	20 mm PST																
265.6	20 mm PST over 110 mm Granular Base over 150 mm Granular Subbase (PAVEMENT STRUCTURE)																
0.3	CLAYEY SILT, some sand, trace gravel																
	Hard to very stiff, Brown to grey, Moist (FILL)		1	SS	35												
			2	SS	27												
263.6	CLAYEY SILT, trace sand																
2.3	Firm, Grey, Moist		3	SS	8									0	1	79	20
			4	SS	6												
262.1	SILT, trace sand																
3.8	Loose, Grey, Moist to wet		5	SS	7									0	0	86	14
			6	SS	9												
			7	SS	9												
			8	SS	10									0	2	89	9
			9	SS	7												
			10	SS	9												
255.2	SILTY SAND TO SANDY SILT, trace gravel																
10.7	Loose to compact, Grey, Wet (TILL)		11	SS	10									4	54	38	4
			12	SS	7												
			13	SS	11												
250.9																	

ONTARIO MTO 18TF002 DC(JUNE 2019).GPJ ONTARIO MTO.GDT 6/11/19

Continued Next Page

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

RECORD OF BOREHOLE No DC-3

2 OF 2

METRIC

G.W.P. 5267-11-00 LOCATION Coords: 5 441 690.7 N; 284 923.1 E ORIGINATED BY M.M.
 DIST Northern HWY 688 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY K.A.
 DATUM Geodetic DATE _____ LATITUDE 49.113042 LONGITUDE -81.27229 CHECKED BY R.N.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100						
250.9	SILTY SAND TO SANDY SILT, trace gravel Loose, Grey, Wet (TILL)		14	SS	5													
15.0																		
249.1	Unweathered, fine grained to medium grained Dark grey to black, Hard, Crystalline Diabase Bedrock		15	RC HQ	REC 76%												RQD=0%	
16.8			16	RC HQ	REC 100%												248	RQD=57% UCS=108.6 MPa
17.8			17	RC HQ	REC 100%												247	RQD=76% UCS=191.7 MPa
245.8	End of borehole																	
20.1	NOTES: 1. Groundwater level was not encountered inside the borehole during or upon completion of drilling. 2. No cave-in was noted in the borehole upon extraction of hollow stem augers.																	

ONTARIO MTO 18TF002 DC(JUNE 2019).GPJ ONTARIO MTO.GDT 7/30/19

RECORD OF BOREHOLE No DC-4

1 OF 1

METRIC

G.W.P. 5267-11-00 LOCATION Coords: 5 441 702.2 N; 284 919.2 E ORIGINATED BY M.M.
 DIST Northern HWY 688 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY K.A.
 DATUM Geodetic DATE 2019.04.15 LATITUDE 49.113147 LONGITUDE -81.272347 CHECKED BY R.N.

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	20	40	60	80					
											○ UNCONFINED	+ FIELD VANE				
											● QUICK TRIAXIAL	× LAB VANE				
											WATER CONTENT (%)					
											20	40	60			
266.3	Ground Surface															
0.0	30 mm PST															
266.0	over 110 mm Granular Base over 110 mm Granular Subbase (PAVEMENT STRUCTURE)		1	GRAB												
0.3	CLAYEY SILT, some sand															
	Very stiff, Brown to grey, Wet (FILL)		2	SS	28											
			3	SS	22											
264.2	CLAYEY SILT, trace sand															
2.1	Firm to stiff, Grey, Wet		4	SS	5											
			5	SS	6											0 1 73 26
			6	SS	6											
			7	SS	8											
			8	SS	10											
260.2	SILT, some clay, trace sand															
6.1	Loose, Grey, Wet		9	SS	8											0 0 88 12
			10	SS	7											
			11	SS	6											
			12	SS	8											0 3 85 12
255.0	End of borehole															
11.3																

NOTES:

- Groundwater level was not encountered inside the borehole during drilling.
- Borehole charged with drilling water at a depth of 4.6 m (El. 261.7) below the existing ground surface, thus groundwater level could not be established upon completion of drilling.
- No cave-in was noted in the borehole upon extraction of hollow stem augers.

ONTARIO MTO 18TF002 DC(JUNE 2019).GPJ ONTARIO MTO.GDT 6/11/19

RECORD OF BOREHOLE No DRW-1

1 OF 2

METRIC

G.W.P. 5267-11-00 LOCATION Coords: 5 441 623.9 N; 284 928.0 E ORIGINATED BY M.M./F.M.
 DIST Northern HWY 688 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY K.A.
 DATUM Geodetic DATE 2019.04.16 LATITUDE 49.112442 LONGITUDE -81.27222 CHECKED BY R.N.

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	20	40						60	80	100	20
264.4	Ground Surface																	
264.3	TOPSOIL		1	SS	7													
0.1	CLAYEY SILT, some sand, rootlets, wood debris		2	SS	5													
	Firm to very soft, Compact, Brown, Moist to wet		3	SS	3								87.9					
262.3	SILTY CLAY TO CLAYEY SILT, some to trace sand, trace gravel		4	SS	4													
2.1	Soft to stiff, Grey, Moist to wet		5	SS	4									0	20	60	20	
			6	SS	1													
			7	SS	3									6	11	52	31	
			8	SS	5													
			9	SS	9													
256.8	SILT, trace sand		10	SS	7										0	2	88	10
7.6	Loose to very loose, Grey, Wet		11	SS	2													
254.2	SANDY SILT TO SILTY SAND		12	SS	23										0	57	39	4
10.2	Loose to Dense, Grey, Wet (TILL)		13	SS	8													
250.1	End of borehole		14	SS	34													
14.3																		

ONTARIO MTO 18TF002 DC(JUNE 2019), GPJ ONTARIO MTO, GDT 6/11/19

Continued Next Page

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

RECORD OF BOREHOLE No DRW-1

2 OF 2

METRIC

G.W.P. 5267-11-00 LOCATION Coords: 5 441 623.9 N; 284 928.0 E ORIGINATED BY M.M./F.M.
 DIST Northern HWY 688 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY K.A.
 DATUM Geodetic DATE 2019.04.16 LATITUDE 49.112442 LONGITUDE -81.27222 CHECKED BY R.N.

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa								
249.4						20	40	60	80	100						
	NOTES: 1. Groundwater level was not encountered inside the borehole during or upon completion of drilling. 2. No cave-in was noted in the borehole upon extraction of hollow stem augers.															

ONTARIO MTO 18TF002 DC(JUNE 2019).GPJ ONTARIO MTO.GDT 6/11/19

RECORD OF BOREHOLE No DRW-2

1 OF 1

METRIC

G.W.P. 5267-11-00 LOCATION Coords: 5 441 639.9 N; 284 931.5 E ORIGINATED BY M.M./F.M.
 DIST Northern HWY 688 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY K.A.
 DATUM Geodetic DATE 2019.04.12 LATITUDE 49.112586 LONGITUDE -81.272173 CHECKED BY R.N.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100					
											○ UNCONFINED	+	FIELD VANE				
											● QUICK TRIAXIAL	×	LAB VANE				
											WATER CONTENT (%)						
											20	40	60				
262.5	Ground Surface																
262.4 0.1	TOPSOIL SILTY SAND wood debris, organics Dark Brown, Lose to compact, Wet (FILL)		1	SS	9												
261.5 1.0	wood debris		2	SS	12												
	CLAYEY SILT, trace sand Firm, Brown to grey, Wet		3	SS	7												
			4	SS	21												
			5	SS	7												
			6	SS	6												
258.0 4.5	SILT Loose, Grey, Wet		7	SS	7												0 1 77 22
			8	SS	9												0 0 88 12
			9	SS	4												
254.6 7.9	SILTY SAND TO SANDY SILT, some gravel Very Loose to compact, Grey, Wet		10	SS	3												13 52 32 3
			11	SS	12												
			12	SS	8												
249.3 13.2	very dense End of borehole		13	SS	50/8cm												
	NOTES: 1. Groundwater level was not encountered in the borehole during or upon completion of drilling. 2. No cave-in was noted in the borehole upon extraction of hollow stem augers.																

ONTARIO MTO 18TF002 DC(JUNE 2019), GPJ ONTARIO MTO.GDT 6/11/19

RECORD OF BOREHOLE No DRW-3

1 OF 2

METRIC

G.W.P. 5267-11-00 LOCATION Coords: 5 441 657.0 N; 284 930.2 E ORIGINATED BY M.M./F.M.
 DIST Northern HWY 688 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY K.A.
 DATUM Geodetic DATE 2019.04.11 LATITUDE 49.11274 LONGITUDE -81.272192 CHECKED BY R.N.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80						100	20	40
262.7	Ground Surface																		
262.6 0.1	TOPSOIL SILTY SAND, with gravel, wood debris, organics Lose to compact, Dark brown, Wet (FILL)		1	SS	10								o						
			2	SS	9								o						
260.9 1.8	CLAYEY SILT, trace sand Firm to very stiff, Brown to grey, Wet		3	SS	11								o						
			4	SS	4								o						
			5	SS	8								H			0	1	79	20
			6	SS	7								o						
258.2 4.5	SILT, trace sand Loose, Grey, Wet		7	SS	9								o						
			8	SS	6								H	o		0	1	83	16
			9	SS	9								o						
			10	SS	10								o			0	0	91	9
254.0 8.7	SILTY SAND TO SANDY SILT, trace gravel Very loose, Grey, Wet		11	SS	WH								o			4	53	38	5
			12	SS	4								o						
			13	SS	50/8cm								o						
250.0 12.7	End of borehole		14	SS	50/3cm								o						

ONTARIO MTO 18TF002 DC(JUNE 2019).GPJ ONTARIO MTO.GDT 6/11/19

WH Denotes penetration due to weight of hammer and rods
 Groundwater level observed during drilling

Continued Next Page

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

RECORD OF BOREHOLE No DRW-3

2 OF 2

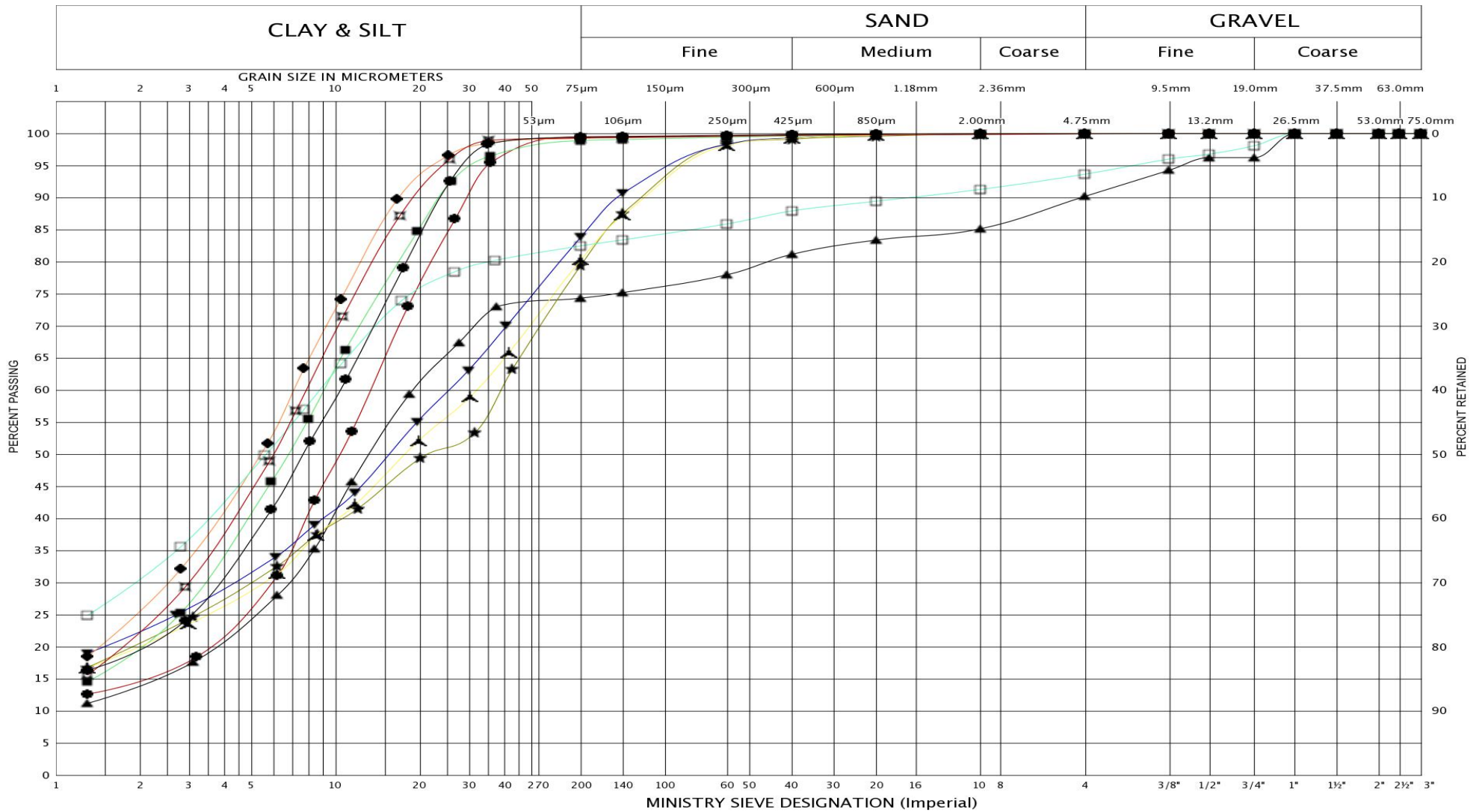
METRIC

G.W.P. 5267-11-00 LOCATION Coords: 5 441 657.0 N; 284 930.2 E ORIGINATED BY M.M./F.M.
 DIST Northern HWY 688 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY K.A.
 DATUM Geodetic DATE 2019.04.11 LATITUDE 49.11274 LONGITUDE -81.272192 CHECKED BY R.N.

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	20	40	60	80					
247.7																
	NOTES: 1. Groundwater level was not encountered in the borehole upon completion of drilling. 2. Upon extraction of hollow stem augers, the borehole caved-in at a depth of 9.1 m (El. 253.6) below the existing ground surface.															

ONTARIO MTO 18TF002 DC(JUNE 2019).GPJ ONTARIO.MTO.GDT 6/11/19

UNIFIED SOIL CLASSIFICATION SYSTEM



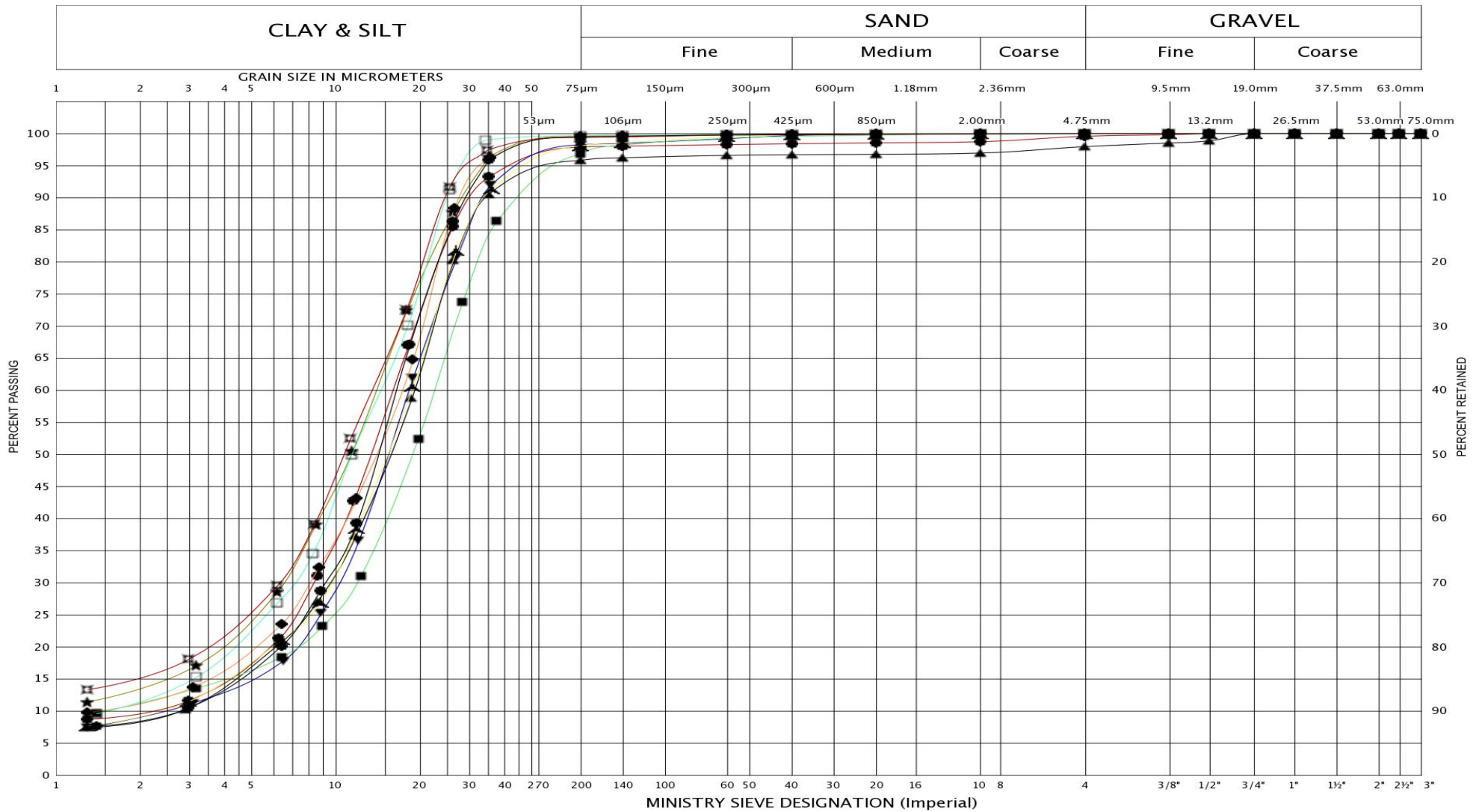
LEGEND	BH	DRW-1	DC-1	DC-1	DRW-1	DC-1	DC-2	DRW-2	DC-3	DRW-3	DC-4
SAMPLE	5	6	7	7	8	5	5	3	5	5	
SYMBOL	▲	★	▲	□	●	▼	■	◆	◆	◆	



GRAIN SIZE DISTRIBUTION
 SILTY CLAY TO CLAYEY SILT, Trace/Some Sand, Trace Gravel

FIG No.:	DC-GS-1
HWY :	668
GWP	5267-11-00

UNIFIED SOIL CLASSIFICATION SYSTEM



LEGEND	BH	DRW-1	DC-2	DRW-2	DC-2	DC-3	DC-3	DRW-3	DRW-3	DC-4	DC-4
SAMPLE	10	8	8	10	5	8	8	10	9	12	
SYMBOL	▲	●	□	▲	★	▼	⊠	●	◆	■	

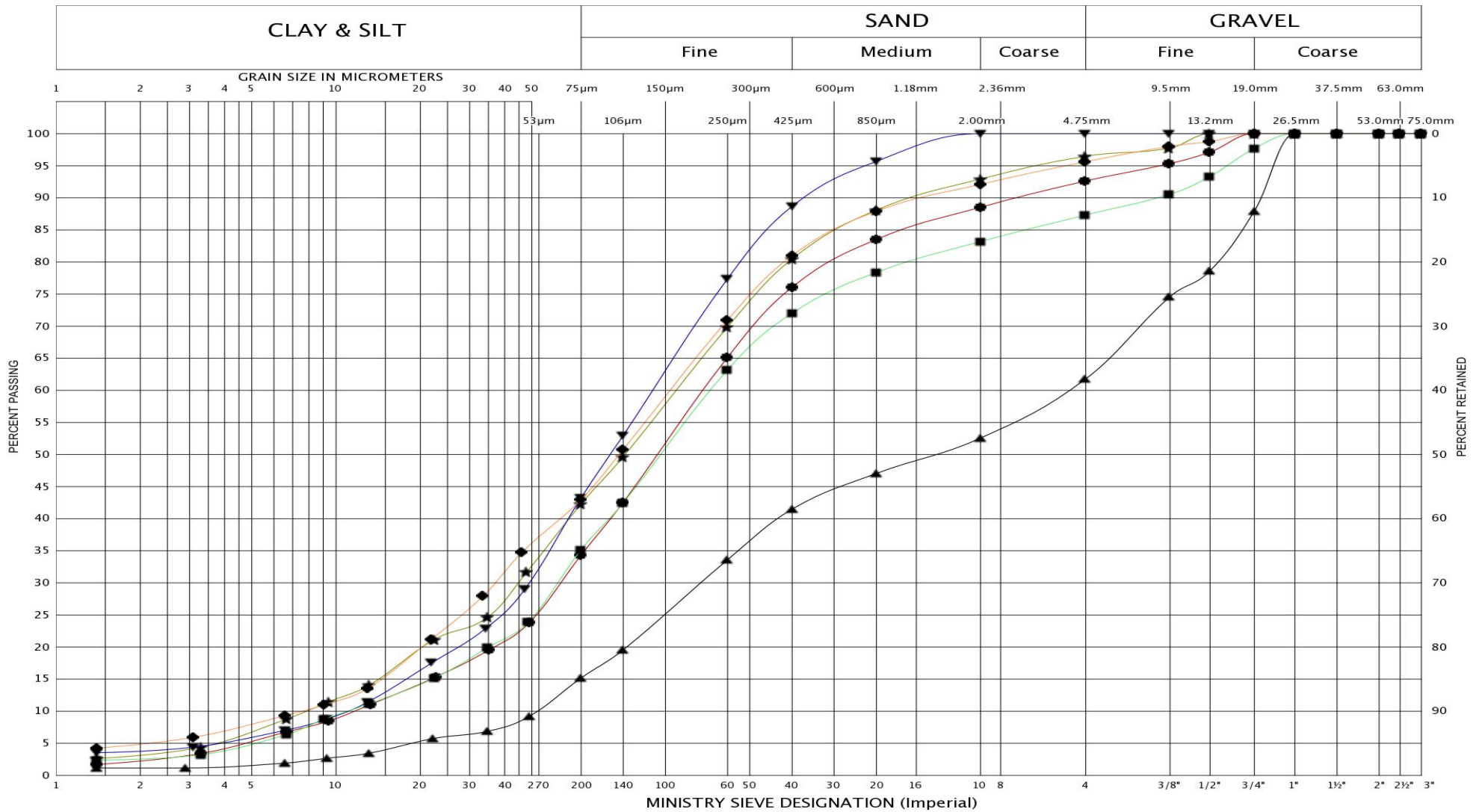


GRAIN SIZE DISTRIBUTION

SILT, Some Sand, Trace Gravel

FIG No.:	DC-GS-2
HWY :	668
GWP	5267-11-00

UNIFIED SOIL CLASSIFICATION SYSTEM



LEGEND	BH	DC-1	DRW-1	DRW-2	DC-2	DC-3	DRW-3
SAMPLE	11	12	10	12	11	11	
SYMBOL	●	▼	■	▲	★	◆	



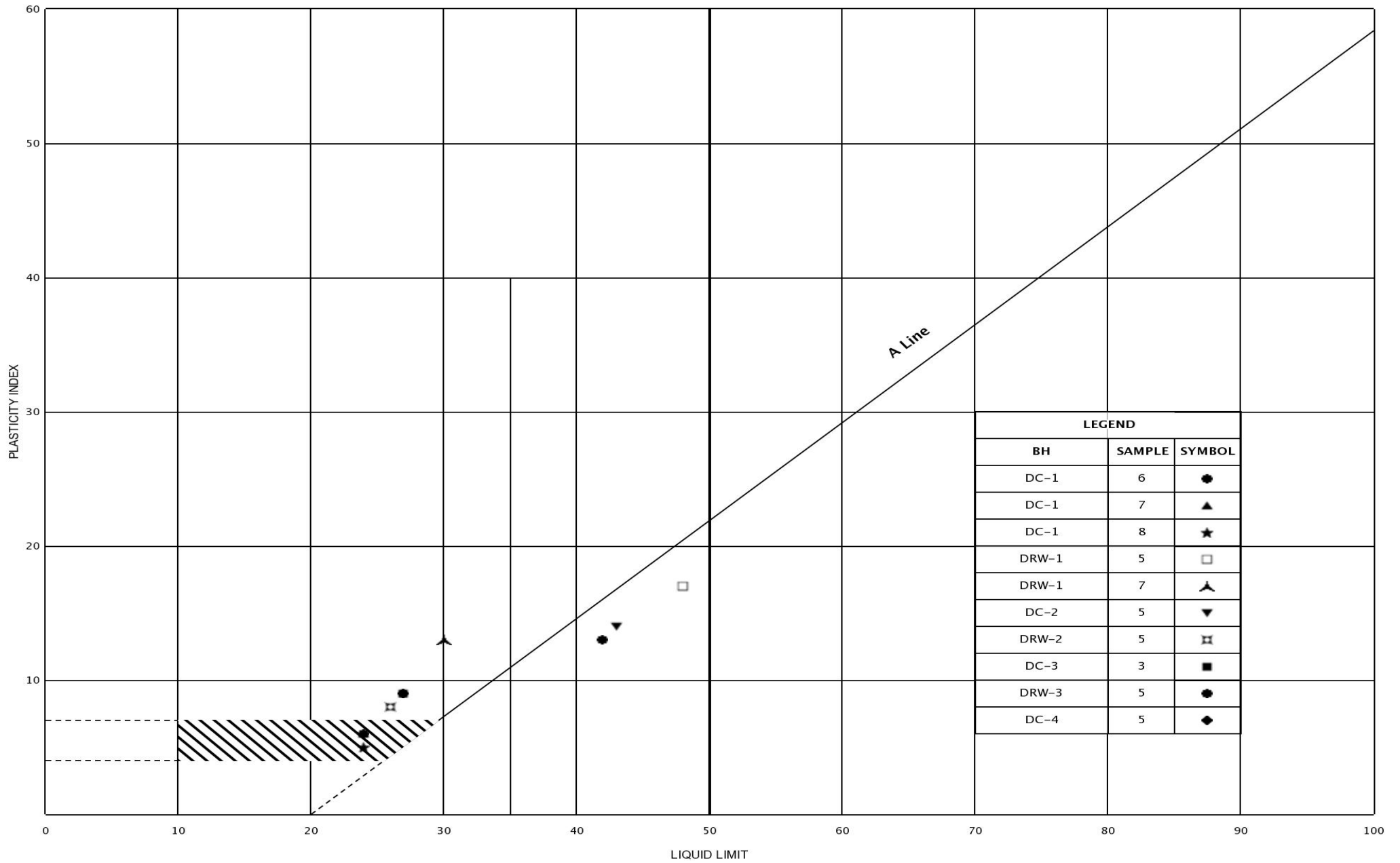
GRAIN SIZE DISTRIBUTION

SILTY SAND TO SANDY SILT, Trace/Some Sand,
Trace/With Gravel (Till)

FIG No.: DC-GS-3

HWY : 668

GWP 5267-11-00



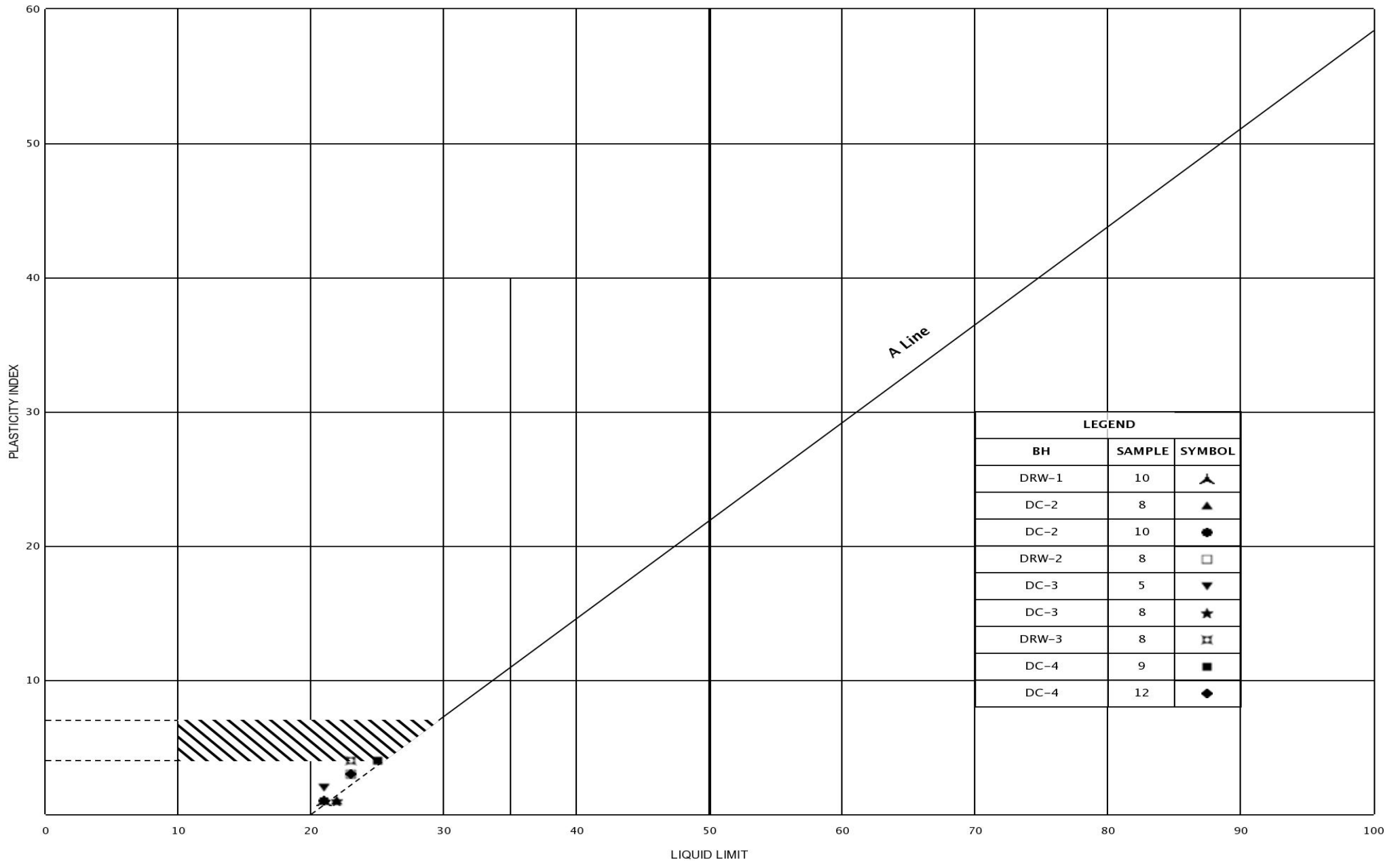
PLASTICITY CHART

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

FIG No.: DC-PC-1

HWY.: 668

GWP 5267-11-00



PLASTICITY CHART
SILT, Some Sand, Trace Gravel

FIG No.:	DC-PC-2
HWY.:	668
GWP	5267-11-00

Part A – Foundation Investigation Report

Deception Creek Bridge Replacement, Site No. 39E-0169/B0, Highway 668 – 10+265
Town of Cochrane, Ontario, G.W.P. 5267-11-00, W.P. 5368-11-01, Index No.: 038FIR
PML Ref.: 18TF002A, August 21, 2019



Photograph 1: Borehole DC-2, Runs 1 and 2.



Photograph 2: Borehole DC-2, Run 3.



Photograph 3: Borehole DC-3, Runs 1 and 2.



Photograph 4: Borehole DC-3, Run 3.

Part A – Foundation Investigation Report

Deception Creek Bridge Replacement, Site No. 39E-0169/B0, Highway 668 – 10+265
 Town of Cochrane, Ontario, G.W.P. 5267-11-00, W.P. 5368-11-01, Index No.: 038FIR
 PML Ref.: 18TF002A, August 21, 2019



ROCK CORE DESCRIPTION

Location: Deception Creek, Cochrane, ON

Site Name: Hwy 668

BH No.	CORE RECOVERY				CORE DESCRIPTION	
	RC No.	DEPTH (m)	% CR*	% RQD*	DEPTH (m)	DESCRIPTION
DC 2	1	15.24	86.8% (0.46 m)	24.5% (0.13 m)	15.77	<p style="text-align: center;">NEO-MESOARCHEAN (2.5-3.4 GA) MAFIC TO INTERMEDIATE METAVOLCANIC</p> Unweathered, fine to medium grained, massive, dark grey to black, hard, crystalline; DIABASE with white; translucent quartz veins (2.0 cm thick). Occasional features: broken rock at 15.24-15.47 m.
DC 2	2	15.77	98.1% (1.52 m)	98.1% (1.52)	17.32	<p style="text-align: center;">NEO-MESOARCHEAN (2.5-3.4 GA) MAFIC TO INTERMEDIATE METAVOLCANIC</p> Unweathered, fine to medium grained, massive, dark grey to black, hard, crystalline; DIABASE . Sample taken at 15.93-16.18 m.
DC 2	3	17.32	+100.0% (1.52 m)	100.0% (1.45 m)	18.75	<p style="text-align: center;">NEO-MESOARCHEAN (2.5-3.4 GA) MAFIC TO INTERMEDIATE METAVOLCANIC</p> Unweathered, fine to medium grained, massive, dark grey to black, hard, crystalline; DIABASE with white; translucent quartz veins (4.0 cm thick). Sample taken at 17.32-17.55 m.

CR* - Core Recovery

Logged by: Heather Racher, M.Sc.

RQD* - Rock Quality Designation

Note: Depths are approximated where core recovery is less than 100%. RQDs are calculated according to core recovery (less than designated 1.52 m runs).

Part A – Foundation Investigation Report

Deception Creek Bridge Replacement, Site No. 39E-0169/B0, Highway 668 – 10+265
 Town of Cochrane, Ontario, G.W.P. 5267-11-00, W.P. 5368-11-01, Index No.: 038FIR
 PML Ref.: 18TF002A, August 21, 2019



ROCK CORE DESCRIPTION

Location: Deception Creek, Cochrane, ON

Site Name: Hwy 668

BH No.	CORE RECOVERY				CORE DESCRIPTION	
	RC No.	DEPTH (m)	% CR*	% RQD*	DEPTH (m)	DESCRIPTION
DC 3	1	16.76	75.6% (0.31 m)	0.00% (0.00 m)	17.17	<p style="text-align: center;">NEO-MESOARCHEAN (2.5-3.4 GA) MAFIC TO INTERMEDIATE METAVOLCANIC</p> Unweathered, medium to coarse grained, massive, dark grey/white, hard, crystalline; DIABASE . Occasional features: broken rock at 16.76 -17.17 m (entire run).
DC 3	2	17.17	+100.0% (1.52 m)	57.0% (0.81)	18.59	<p style="text-align: center;">NEO-MESOARCHEAN (2.5-3.4 GA) MAFIC TO INTERMEDIATE METAVOLCANIC</p> Unweathered, medium to coarse grained, massive, dark grey/white, hard, crystalline; DIABASE . Occasional features: broken rock at 17.96-18.06 m, 18.52-18.57 m; vertical fracture at 18.57-18.69 m; sulphides within natural fractures. Sample taken at 18.06-18.34 m.
DC 3	3	18.59	+100.0% (1.70 m)	75.5% (1.17 m)	20.14	<p style="text-align: center;">NEO-MESOARCHEAN (2.5-3.4 GA) MAFIC TO INTERMEDIATE METAVOLCANIC</p> Unweathered, medium to coarse grained, massive, dark grey/white, hard, crystalline; DIABASE . Occasional features: broken rock at 18.92-18.95 m, 19.30-19.33 m; olivine and calcite lined within natural fractures; colour change to solid dark grey, fine to medium grained at 19.08-19.28 m. Sample taken at 19.86-20.14 m.

CR* - Core Recovery

Logged by: Heather Racher, M.Sc.

RQD* - Rock Quality Designation

Note: Depths are approximated where core recovery is less than 100%. RQDs are calculated according to core recovery (less than designated 1.52 m runs).

Peto MacCallum Ltd.

CONSULTING ENGINEERS

UNIAXIAL COMPRESSIVE STRENGTH OF ROCK CORE

ASTM D7012

CLIENT Parsons
 PROJECT Agreement 5017-E-0030 GWP 5267-11-00
 SAMPLE IDENTIFICATION DC-2 RUN2 15.9m-16.2m

PML REF 18TF002A
 LAB NO. 1902356 F
 DATE SAMPLED
 DATE TESTED 2019-06-04
 TESTED BY FP/BM

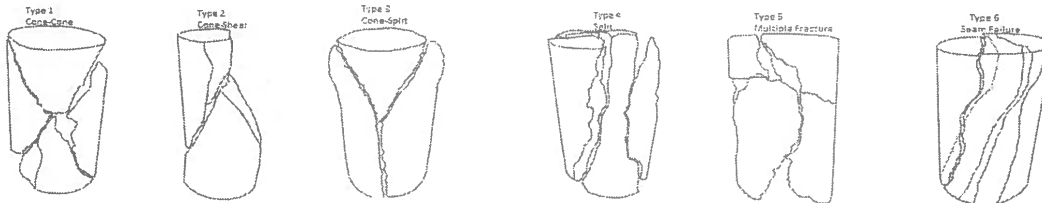
CORE DIMENSIONS		COMPRESSIVE STRENGTH	
SPECIMEN DIAMETER (in.)	2.4773	TEST TIME (min) (spec. 2 to 15)	7;52
SPECIMEN LENGTH (in.)	6.032	MAXIMUM LOAD APPLIED (kN)	413.40
	6.034		
	6.031	COMPRESSIVE STRENGTH (MPa)	132.9
AVE.	6.032	TYPE OF FAILURE	TYPE IV
SURFACE AREA (sq mm)	3110	LENGTH TO DIAMETER RATIO (spec 2-2.5)	2.43

MOISTURE CONTENT

UNIT WEIGHT

WEIGHT OF WET SAMPLE + TARE (g)	1372.10	WEIGHT OF DRY SAMPLE IN AIR (g)	1296.09
WEIGHT OF DRY SAMPLE + TARE (g)	1370.98	VOLUME OF SAMPLE (cu m)	0.000476
WEIGHT OF WATER (g)	1.12	UNIT WEIGHT (kg/cu m)	2720
WEIGHT OF TARE (g)	170.30		
WEIGHT OF DRY SAMPLE (g)	1200.68		
MOISTURE CONTENT (%)	0.1		

REMARKS



REVIEWED BY _____

DATE _____

Peto MacCallum Ltd.

CONSULTING ENGINEERS

UNIAXIAL COMPRESSIVE STRENGTH OF ROCK CORE ASTM D7012

CLIENT Parsons
PROJECT Agreement 5017-E-0030 GWP 5267-11-00
SAMPLE IDENTIFICATION DC-2 RUN3 17.3m-17.6m

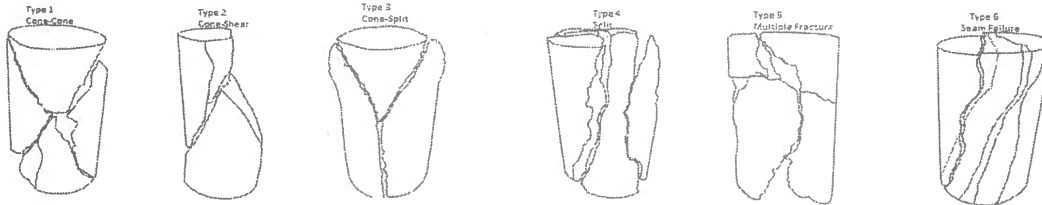
PML REF 18TF002A
LAB NO. 1902356 E
DATE SAMPLED
DATE TESTED 2019-06-04
TESTED BY BM/FP

CORE DIMENSIONS		COMPRESSIVE STRENGTH	
SPECIMEN DIAMETER (in.)	2.4820	TEST TIME (min) (spec. 2 to 15)	6:27
SPECIMEN LENGTH (in.)	6.046	MAXIMUM LOAD APPLIED (kN)	402.30
	6.046		
	6.046	COMPRESSIVE STRENGTH (MPa)	128.9
AVE.	6.046	TYPE OF FAILURE	TYPE I
SURFACE AREA (sq mm)	3121	LENGTH TO DIAMETER RATIO (spec 2-2.5)	2.44

MOISTURE CONTENT

UNIT WEIGHT

WEIGHT OF WET SAMPLE + TARE (g)	1447.00	WEIGHT OF DRY SAMPLE IN AIR (g)	1312.13
WEIGHT OF DRY SAMPLE + TARE (g)	1445.95	VOLUME OF SAMPLE (cu m)	0.000479
WEIGHT OF WATER (g)	1.05	UNIT WEIGHT (kg/cu m)	2737
WEIGHT OF TARE (g)	149.70		
WEIGHT OF DRY SAMPLE (g)	1296.25		
MOISTURE CONTENT (%)	0.1		
REMARKS			



REVIEWED BY _____

DATE _____

Peto MacCallum Ltd.

CONSULTING ENGINEERS

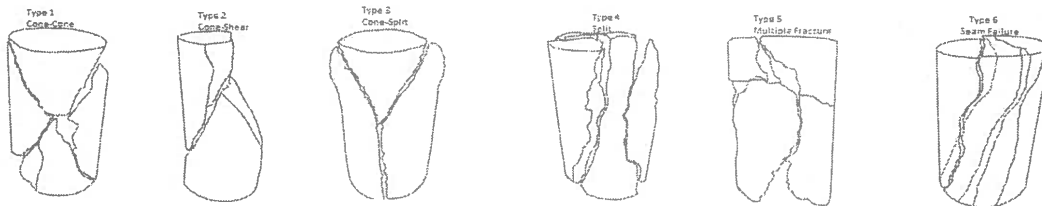
UNIAXIAL COMPRESSIVE STRENGTH OF ROCK CORE

ASTM D7012

CLIENT	Parsons	PML REF	18TF002A
PROJECT	Replacement of 2 Bridges on Hwy 668 & 1 Bridge on Hwy	LAB NO.	1902356 A
SAMPLE IDENTIFICATION	DC -3 RUN 2 18.1m-18.3m.	DATE SAMPLED	
		DATE TESTED	2019-06-04
		TESTED BY	FP/BM

CORE DIMENSIONS		COMPRESSIVE STRENGTH	
SPECIMEN DIAMETER (in.)	2.4787	TEST TIME (min) (spec. 2 to 15)	8:45
SPECIMEN LENGTH (in.)	6.025	MAXIMUM LOAD APPLIED (kN)	338.00
	6.026		
	6.027	COMPRESSIVE STRENGTH (MPa)	108.6
AVE.	6.026	TYPE OF FAILURE	Type -2
SURFACE AREA (sq mm)	3113	LENGTH TO DIAMETER RATIO (spec 2-2.5)	2.43

MOISTURE CONTENT		UNIT WEIGHT	
WEIGHT OF WET SAMPLE + TARE (g)	1089.50	WEIGHT OF DRY SAMPLE IN AIR (g)	1259.41
WEIGHT OF DRY SAMPLE + TARE (g)	1087.76	VOLUME OF SAMPLE (cu m)	0.000477
WEIGHT OF WATER (g)	1.74	UNIT WEIGHT (kg/cu m)	2643
WEIGHT OF TARE (g)	133.80		
WEIGHT OF DRY SAMPLE (g)	953.96		
MOISTURE CONTENT (%)	0.2		
REMARKS			



REVIEWED BY _____

DATE _____

Peto MacCallum Ltd.
CONSULTING ENGINEERS

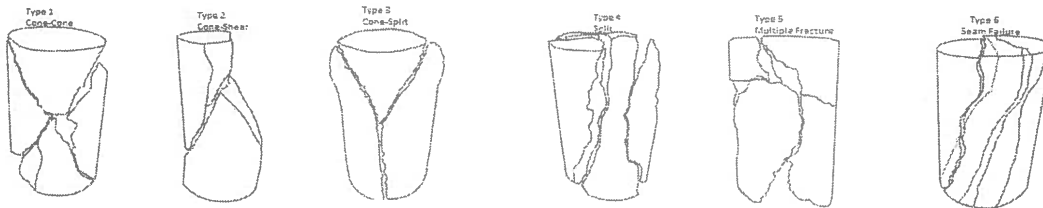
UNIAXIAL COMPRESSIVE STRENGTH OF ROCK CORE
ASTM D7012

CLIENT Parsons
PROJECT Agreement 5017-E-0030 GWP 5267-11-00
SAMPLE IDENTIFICATION DC-3 RUN3 19.9m.-20.1m.

PML REF 18TF002A
LAB NO. 1902356 G
DATE SAMPLED
DATE TESTED 2019-05-04
TESTED BY FP/BM

CORE DIMENSIONS		COMPRESSIVE STRENGTH	
SPECIMEN DIAMETER (in.)	2.4840	TEST TIME (min) (spec. 2 to 15)	11:23
SPECIMEN LENGTH (in.)	5.838	MAXIMUM LOAD APPLIED (kN)	608.60
	5.839		
	5.839	COMPRESSIVE STRENGTH (MPa)	194.7
AVE.	5.839	TYPE OF FAILURE	TYPE 1
SURFACE AREA (sq mm)	3127	LENGTH TO DIAMETER RATIO (spec 2-2.5)	2.35

MOISTURE CONTENT		UNIT WEIGHT	
WEIGHT OF WET SAMPLE + TARE (g)	1295.60	WEIGHT OF DRY SAMPLE IN AIR (g)	1227.98
WEIGHT OF DRY SAMPLE + TARE (g)	1294.47	VOLUME OF SAMPLE (cu m)	0.000464
WEIGHT OF WATER (g)	1.13	UNIT WEIGHT (kg/cu m)	2648
WEIGHT OF TARE (g)	187.00		
WEIGHT OF DRY SAMPLE (g)	1107.47		
MOISTURE CONTENT (%)	0.1		
REMARKS			



REVIEWED BY _____

DATE _____



FINAL REPORT

CA14192-JUN19 R1

18TF002A

Prepared for

Peto MacCallum Ltd

First Page

CLIENT DETAILS

Client Peto MacCallum Ltd
 Address 165 Cartwright Ave
 Toronto, ON
 M6A 1V5, Canada
 Contact Nazibur Rahman
 Telephone 416-785-5110
 Facsimile 416-785-5120
 Email nrahman@petomacallum.com
 Project 18TF002A
 Order Number
 Samples Soil (12)

LABORATORY DETAILS

Project Specialist Brad Moore Hon. B.Sc
 Laboratory SGS Canada Inc.
 Address 185 Concession St., Lakefield ON, K0L 2H0
 Telephone 705-652-2143
 Facsimile 705-652-6365
 Email brad.moore@sgs.com
 SGS Reference CA14192-JUN19
 Received 06/05/2019
 Approved 06/06/2019
 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




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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



FINAL REPORT

CA14192-JUN19 R1

Client: Peto MacCallum Ltd

Project: 18TF002A

Project Manager: Nazibur Rahman

Samplers: Nazibur Rahman

PACKAGE: - General Chemistry (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
Conductivity	uS/cm	2	161	214	204	287	129	208	571	213

PACKAGE: - General Chemistry (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
Conductivity	uS/cm	2	160	194	94	224

PACKAGE: - Metals and Inorganics (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
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)

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



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-IENVIIC-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-IENVIARD-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-IENVIEWL-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 | Internal ref.: ME-CA-ENVIEWL-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.

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

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-- End of Analytical Report --

Part A – Foundation Investigation Report

Deception Creek Bridge Replacement, Site No. 39E-0169/B0, Highway 668 – Station 10+265

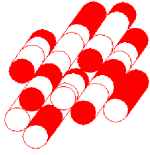
Town of Cochrane, Ontario, G.W.P. 5267-11-00, W.P. 5368-11-01, Index No.: 038FIR

PML Ref.: 18TF002A, August 21, 2019



APPENDIX B

Previous Borehole Logs and Drawings (GEOCRETS No. 42H-64)



Terraprobe

Consulting Geotechnical & Environmental Engineering

Construction Materials Inspection & Testing

**PRELIMINARY
FOUNDATION INVESTIGATION REPORT
DECEPTION CREEK BRIDGE REPLACEMENT
HIGHWAY 668
ASSIGNMENT No. 5013-E-0018
MINISTRY OF TRANSPORTATION, ONTARIO
G.W.P. No. 5267-11-00, SITE 39E-169
GEOCRES NO. 42H-64**

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. 1-15-0509
September 16, 2016

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stoneycreek@terraprobe.ca

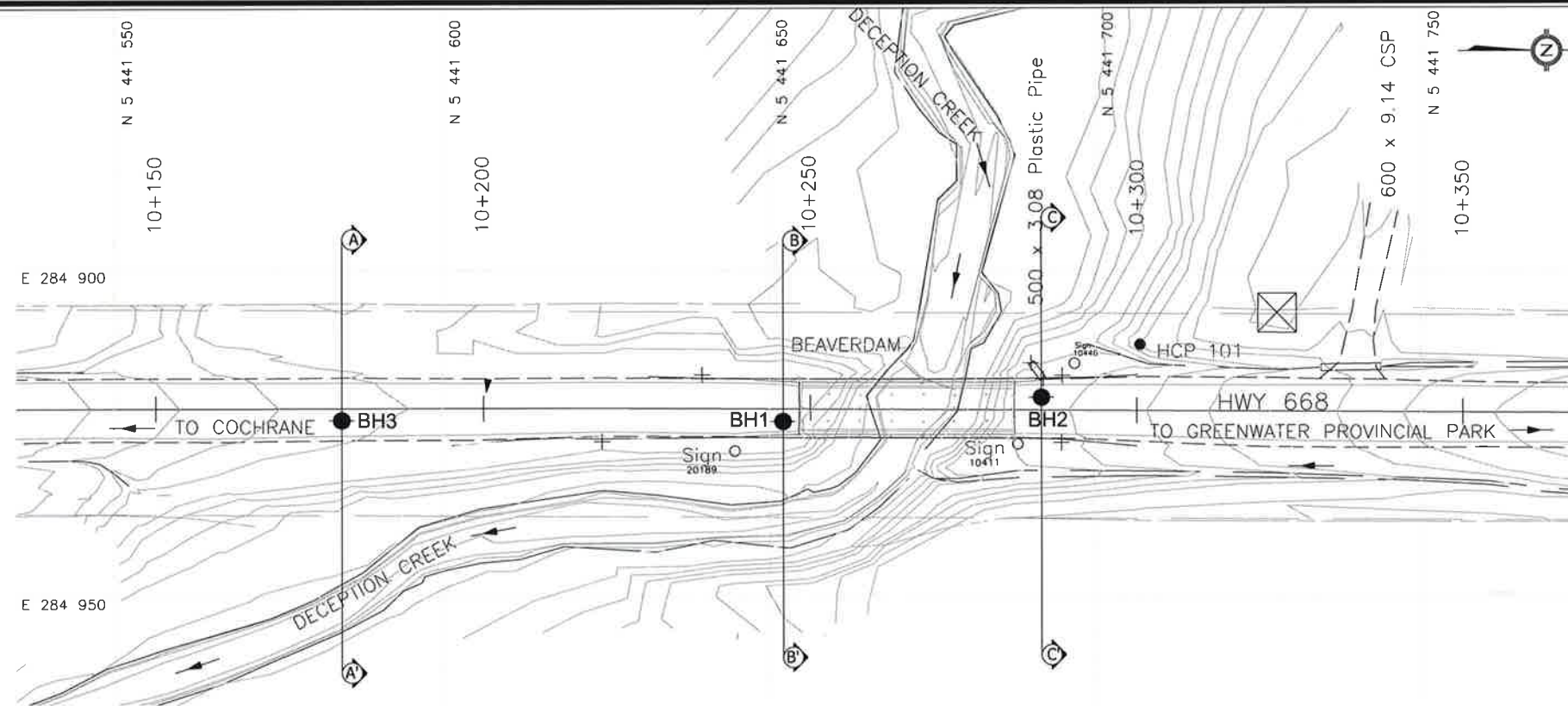
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www.terraprobe.ca



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AND/OR MILLIMETERS UNLESS
OTHERWISE SHOWN

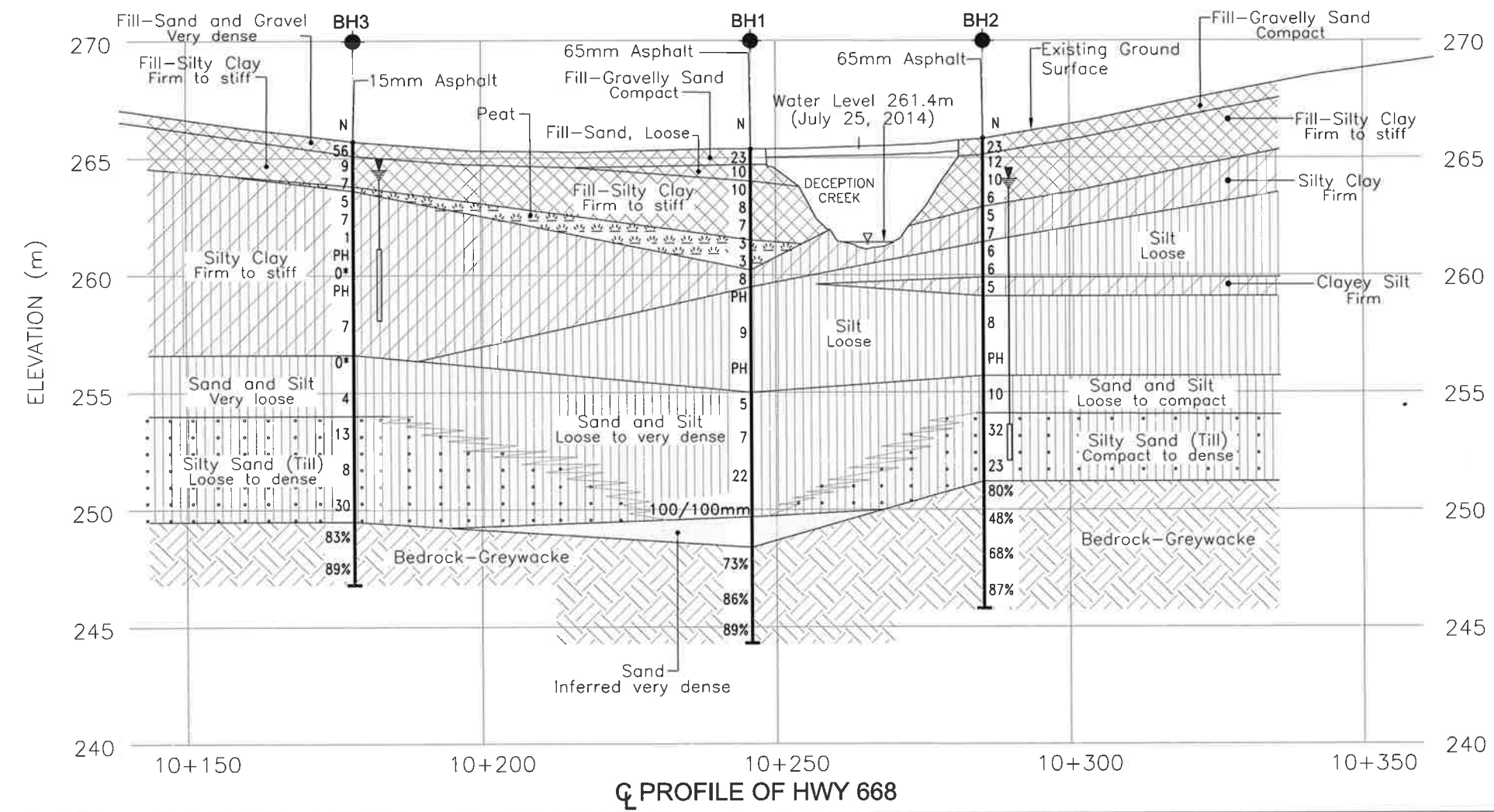
GWP No.: 5267-11-00

HWY 668
DECEPTION CREEK BRIDGE
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET
1 OF 2



PLAN



KEY PLAN

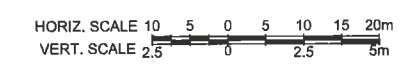
LEGEND

- Bore Hole
- Dynamic Cone Penetration Test
- Bore Hole And Cone
- Blows/0.3m (Std Pen Test, 475 J/blow)
- Blows/0.3m (60' Cone, 475 J/blow)
- WL at Time of Investigation
- WL in Piezometer
- Piezometer
- Rock Quality Designation
- Auger Refusal

No	ELEV.	LOCAL COORDINATES	
		NORTHING	EASTING
1	265.4	5 441 652.1	284 923.2
2	265.8	5 441 691.7	284 919.6
3	265.7	5 441 584.6	284 922.9

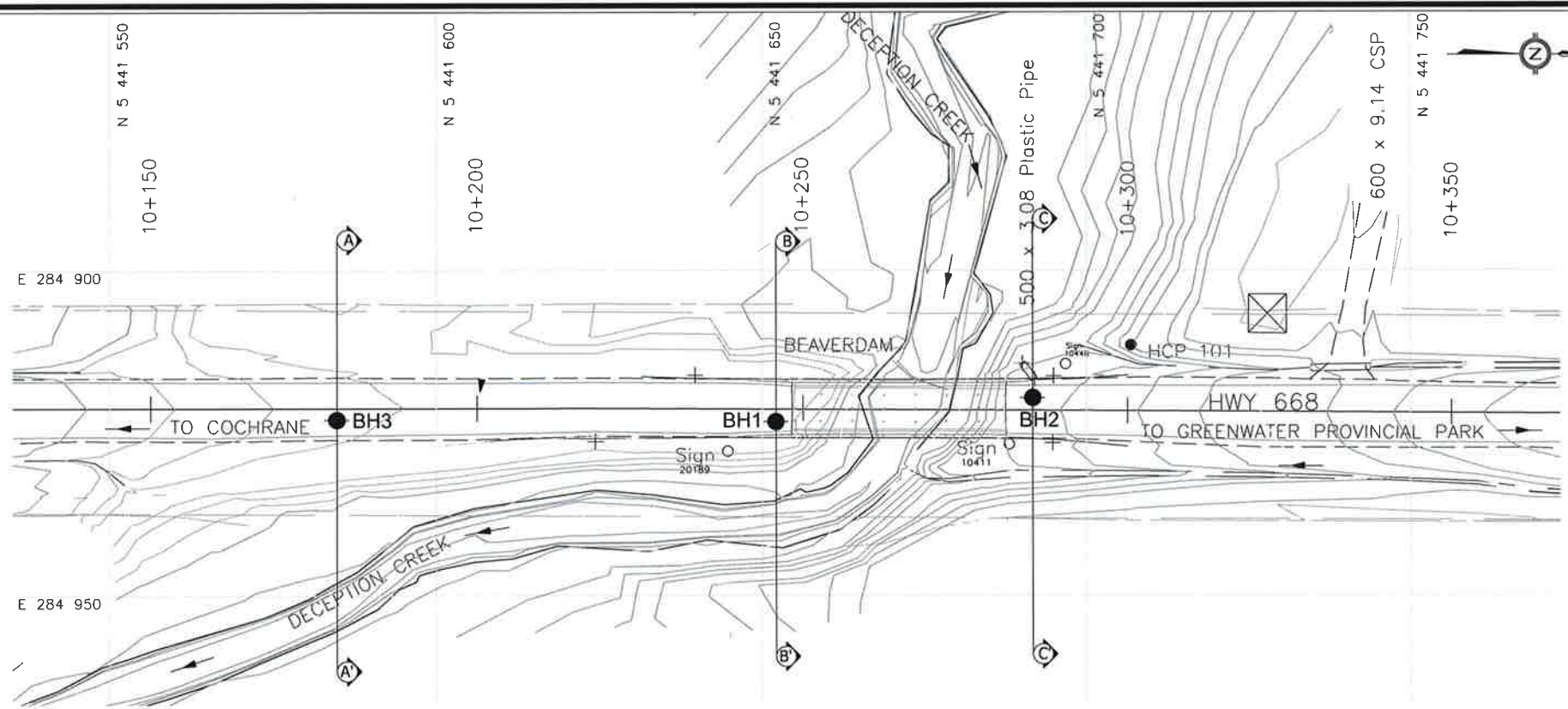
NOTE
This drawing is for subsurface information only. The proposed structure details/works if shown are for illustration purposes only and may not be consistent with final design configuration as shown elsewhere in the contract documents.
The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.
The complete foundation investigation 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 B5280668001, DTM5280668001, received September 11, 2014



REVISIONS	DATE	BY	DESCRIPTION

HWY: 668	PROJECT No.: 1-15-0509	Geocres No. 42H-64
SUBM'D. SD	CHKD. RA	DATE: Sept. 2016
DRAWN: KC	CHKD. RA	APPD: MT



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DIMENSIONS ARE IN METRES
AND/OR MILLIMETERS UNLESS
OTHERWISE SHOWN

REGISTERED PROFESSIONAL ENGINEER
M. Tanos
M. TANOS
16.09.2016
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
R.A. Abdul
R.A. ABDUL
Sept. 16, 2016
PROVINCE OF ONTARIO

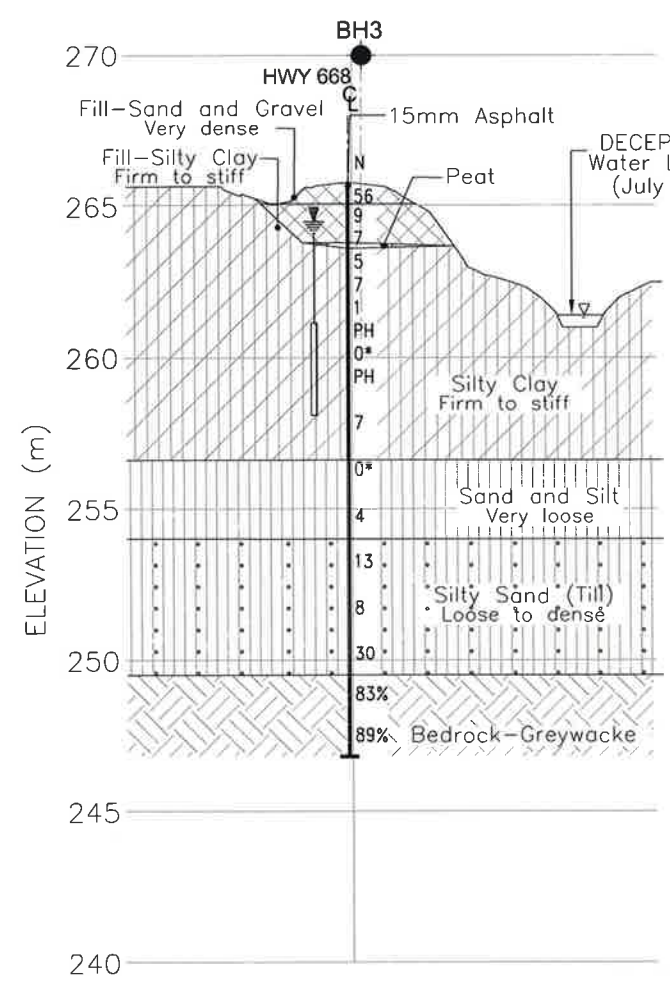
PLAN

SCALE 10 0 10 20m

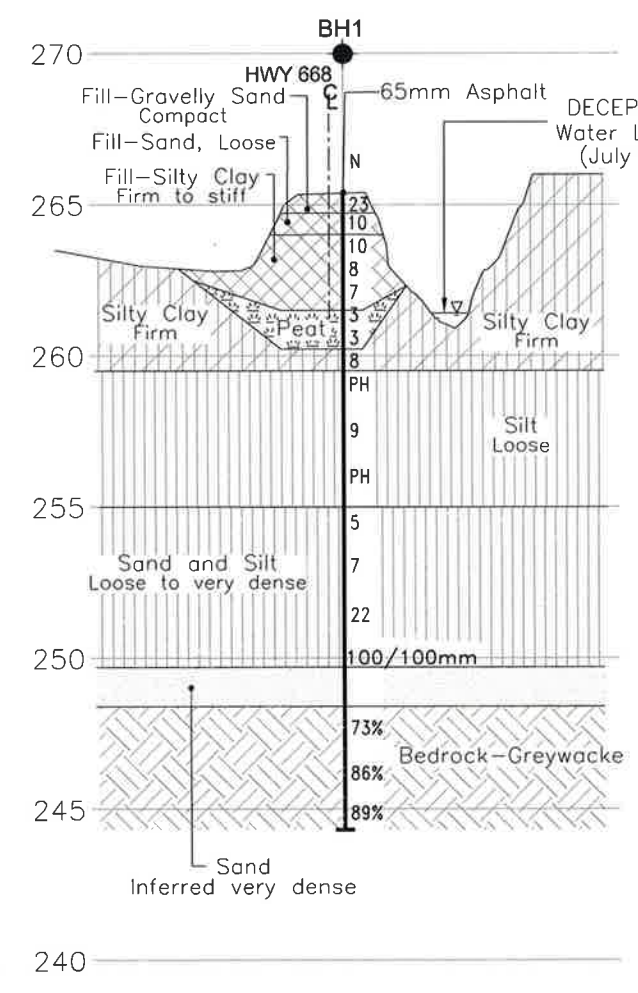
GWP No.: 5267-11-00

HWY 668
DECEPTION CREEK BRIDGE
BOREHOLE LOCATIONS AND SOIL STRATA

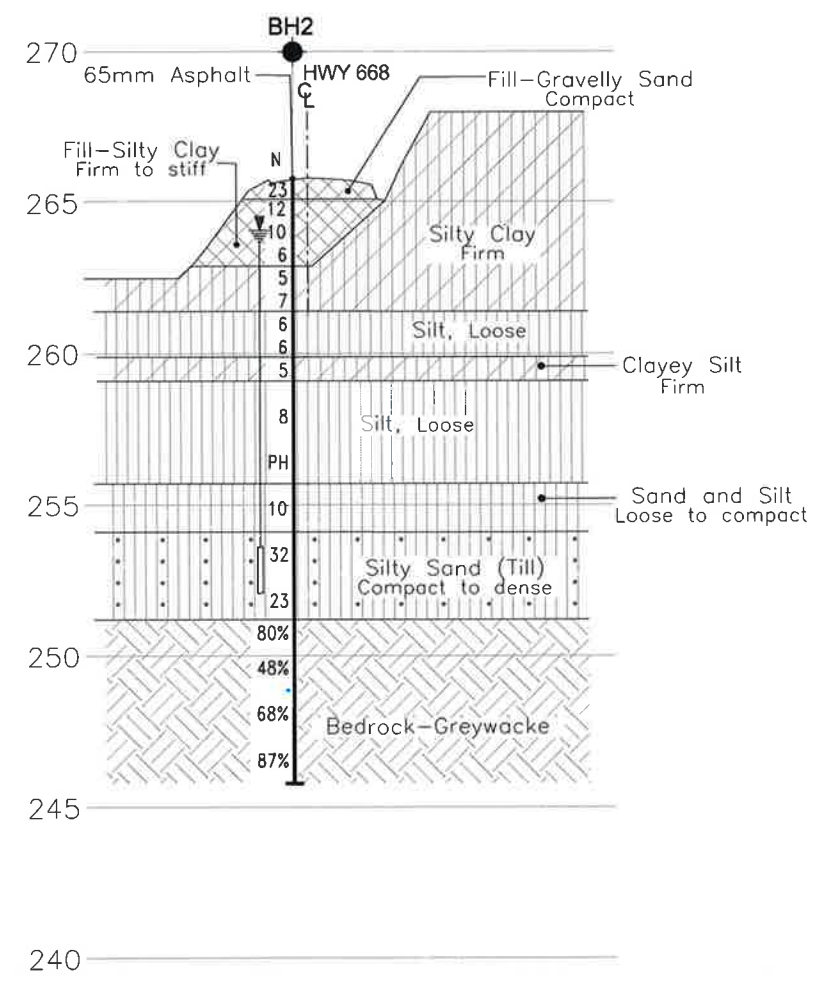
SHEET
2 OF 2



SECTION A-A'



SECTION B-B'



SECTION C-C'

HORIZ. SCALE 10 5 0 5 10 15 20m
VERT. SCALE 2.5 0 2.5 5m

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
- 90% Rock Quality Designation
- A/R Auger Refusal

No	ELEV.	LOCAL COORDINATES	
		NORTHING	EASTING
1	265.4	5 441 652.1	284 923.2
2	265.8	5 441 691.7	284 919.6
3	265.7	5 441 584.6	284 922.9

NOTE
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REVISIONS	DATE	BY	DESCRIPTION

HWY. 668	PROJECT No. 1-15-0509	Geocres No. 42H-64
SUBM'D. SD	CHKD. RA	DATE: Sept. 2016
DRAWN: KC	CHKD. RA	APPD: MT
		SITE: 39E-169
		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 \bar{u} .

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

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_r	1	SENSITIVITY = c_u / τ_r

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

PHYSICAL PROPERTIES OF SOIL

r_s	kg/m^3	DENSITY OF SOLID PARTICLES	e	1.0%	VOID RATIO	e_{min}	1.0%	VOID RATIO IN DENSEST STATE
γ_s	kN/m^3	UNIT WEIGHT OF SOLID PARTICLES	n	1.0%	POROSITY	I_D	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
r_w	kg/m^3	DENSITY OF WATER	w	1.0%	WATER CONTENT	D	mm	GRAIN DIAMETER
γ_w	kN/m^3	UNIT WEIGHT OF WATER	S_r	%	DEGREE OF SATURATION	D_n	mm	n PERCENT - DIAMETER
r	kg/m^3	DENSITY OF SOIL	w_L	%	LIQUID LIMIT	C_u	1	UNIFORMITY COEFFICIENT
γ	kN/m^3	UNIT WEIGHT OF SOIL	w_p	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
r_d	kg/m^3	DENSITY OF DRY SOIL	w_s	%	SHRINKAGE LIMIT	q	m^3/s	RATE OF DISCHARGE
γ_d	kN/m^3	UNIT WEIGHT OF DRY SOIL	I_p	%	PLASTICITY INDEX = $(w_L - w_p)$	v	m/s	DISCHARGE VELOCITY
r_{sat}	kg/m^3	DENSITY OF SATURATED SOIL	I_L	1	LIQUIDITY INDEX = $(w - w_p)/I_p$	i	1	HYDRAULIC GRADIENT
γ_{sat}	kN/m^3	UNIT WEIGHT OF SATURATED SOIL	I_c	1	CONSISTENCY INDEX = $(w_L - w)/I_p$	k	m/s	HYDRAULIC CONDUCTIVITY
r'	kg/m^3	DENSITY OF SUBMERGED SOIL	e_{max}	1.0%	VOID RATIO IN LOOSEST STATE	j	kN/m^3	SEEPAGE FORCE
γ'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL						

RECORD OF BOREHOLE No 1

2 of 2

METRIC

G.W.P. 5267-11-00 LOCATION Coords: E:284923.2 N:5441652.1 ORIGINATED BY SD
 DIST HWY 668 BOREHOLE TYPE SOLID STEM AUGERS / NW CASING AND WASH BORING / NQ CORING COMPILED BY SD
 DATUM GEODETIC DATE 2015-9-15 - 2015-9-16 CHECKED BY RA

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			SPT 'N' VALUE	SHEAR STRENGTH (kPa)					W _p	W		
249.7	(continued) SAND AND SILT, trace to some gravel, trace clay, with cobbles and boulders, very dense, grey, wet		16	SS	100/100mm											
15.7	SAND, trace to some gravel, frequent cobbles and boulders, inferred very dense, black, wet		17	RC												
248.4			18	WS												13 75 (12)
17.0	BEDROCK - GREYWACKE containing quartz veins, slightly weathered, thickly bedded, light grey to grey, strong to extremely strong		1	RUN												Run #1 TCR: 97% SCR: 95% RQD: 73% UCS* = 196 - 303 (MPa)
			2	RUN												Run #2 TCR: 100% SCR: 100% RQD: 86% UCS* = 88 - 253 (MPa)
			3	RUN												Run #3 TCR: 100% SCR: 96% RQD: 89% UCS* = 141 - 230 (MPa)

END OF BOREHOLE

Borehole filled with drill water upon completion of drilling.

Soil heaved into casings after completing RC17.

Borehole grouted and sealed with bentonite slurry after drilling was completed.

Atterberg Limits test attempted on TW9, SS10 and TW11. Samples are non-plastic.

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

file: 1-15-0509-01_deception_creek_bh_logs.gpj

RECORD OF BOREHOLE No 2

1 of 2

METRIC

G.W.P. 5267-11-00 LOCATION Coords: E:284919.6 N:5441691.7 ORIGINATED BY SD
 DIST HWY 668 BOREHOLE TYPE SOLID STEM AUGERS / NW CASING AND WASH BORING / NQ CORING COMPILED BY SD
 DATUM GEODETIC DATE 2015-9-14 CHECKED BY RA

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			SPT 'N' VALUE	SHEAR STRENGTH (kPa)							
265.8	GROUND SURFACE														
265.1	65mm ASPHALTIC CONCRETE		1	SS	23										24 58 12 6
0.7	605mm FILL, gravelly sand, some silt, trace clay, compact, brown, moist		2	SS	12										
	FILL, silty clay, trace sand, firm to stiff, brown, moist		3	SS	10										
			4	SS	6										
262.9															
2.9	SILTY CLAY, trace sand, firm, grey, wet		5	SS	5										0 8 38 54
			6	SS	7										
261.4															
4.4	SILT, occasional clay seams and partings, loose, grey, wet		7	SS	6										0 0 86 14
			8	SS	6										
259.9															
5.9	CLAYEY SILT, trace sand, firm, grey, wet		9	SS	5										0 1 76 23
259.1															
6.7	SILT, trace to some clay, loose, grey, wet		10	SS	8										
			11	TW	PH										0 3 82 15
255.7															
10.1	SAND AND SILT, trace to some gravel, loose to compact, grey, wet		12	SS	10										
254.1															
11.7	SILTY SAND, trace to some gravel, occasional cobbles, compact to dense, grey, wet (GLACIAL TILL)		13	SS	32										
			14	SS	23										
251.2															
14.6			1	RUN											Run #1 TCR: 100%

file: 1-15-0509-01_deception_creek_bh_logs.gpj

Continued Next Page

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

RECORD OF BOREHOLE No 2

2 of 2

METRIC

G.W.P. 5267-11-00 LOCATION Coords: E:284919.6 N:5441691.7 ORIGINATED BY SD
 DIST HWY 668 BOREHOLE TYPE SOLID STEM AUGERS / NW CASING AND WASH BORING / NQ CORING COMPILED BY SD
 DATUM GEODETTIC DATE 2015-9-14 CHECKED BY RA

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT	PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE									SPT 'N' VALUE	SHEAR STRENGTH (kPa)
(continued)														
	BEDROCK - GREYWACKE containing quartz veins, unweathered, thickly bedded, light grey to grey, strong to extremely strong		1	RUN								SCR: 82% RQD: 80% UCS*= 97 - 248 (MPa) Run #2 TCR: 81% SCR: 77% RQD: 48% UCS*= 150 - 257 (MPa)		
250			2	RUN									Run #3 TCR: 100% SCR: 93% RQD: 68% UCS*= 112 - 225 (MPa)	
249			3	RUN										Run #4 TCR: 94% SCR: 92% RQD: 87% UCS*= 90 - 195 (MPa)
248			4	RUN										
247														
246														
245.8 20.0														

END OF BOREHOLE

Piezometer installation consists of a 50mm diameter PVC pipe with a 1.5m slotted screen.

Unable to push vane below 7.0m.

Atterberg Limits test attempted on SS7 and TW11. Samples are non-plastic

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

WATER LEVEL READINGS

Date	Water Depth (m)	Elevation (m)
Oct 1, 2015	1.7	264.1
Oct 7, 2015	1.8	264.0

RECORD OF BOREHOLE No 3

1 of 2

METRIC

G.W.P. 5267-11-00 LOCATION Coords: E:284922.9 N:5441584.6 ORIGINATED BY SD
 DIST HWY 668 BOREHOLE TYPE HOLLOW STEM AUGERS / NW CASING AND WASH BORING / NQ CORING COMPILED BY SD
 DATUM GEODETIC DATE 2015-9-16 - 2015-9-17 CHECKED BY RA

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			SPT 'N' VALUE	SHEAR STRENGTH (kPa)								
265.7	GROUND SURFACE															
265.1	15mm ASPHALTIC CONCRETE		1	SS	56											
0.6	590mm FILL, sand and gravel, trace silt, very dense, brown, dry		2	SS	9											
	FILL, silty clay, trace sand, trace gravel, firm to stiff, brown, moist		3	SS	7											
263.8	PEAT, amorphous, black		4	SS	5											
1.9			5	SS	7											
263.6	SILTY CLAY, trace sand, trace gravel, firm to stiff, brown to 4.1m, grey below, occasional silt seams and partings, moist to wet		6	SS	1											0 7 53 40
2.1			7	TW	PH											1 9 46 44
			8	SS	0*											0 6 47 47
			9	TW	PH											0 1 26 73
			10	SS	7											0 0 53 47
			11	SS	0*											
256.6	SAND AND SILT, trace gravel, trace clay, very loose, grey, wet		12	SS	4											4 59 33 4
9.1			13	SS	13											commence NW casing and wash boring
254.0	SILTY SAND, trace to some gravel, occasional cobbles, loose to dense, grey, wet (GLACIAL TILL)		14	SS	8											
11.7																

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Continued Next Page





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

RECORD OF BOREHOLE No 3

2 of 2

METRIC

G.W.P. 5267-11-00 LOCATION Coords: E:284922.9 N:5441584.6 ORIGINATED BY SD
 DIST HWY 668 BOREHOLE TYPE HOLLOW STEM AUGERS / NW CASING AND WASH BORING / NQ CORING COMPILED BY SD
 DATUM GEODETIC DATE 2015-9-16 - 2015-9-17 CHECKED BY RA

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 	PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE								
(continued)												
249.5	SILTY SAND , trace to some gravel, occasional cobbles, loose to dense, grey, wet (GLACIAL TILL)		15	SS	30	250						29 37 28 6
16.2	BEDROCK - GREYWACKE containing quartz veins, unweathered, thickly bedded, light grey to grey, strong to extremely strong		1	RUN		249						Run #1 TCR: 99% SCR: 99% RQD: 83% UCS**= 99 - 285 (MPa)
246.8			2	RUN		248						Run #2 TCR: 100% SCR: 100% RQD: 89% UCS**= 90 - 156 (MPa)
18.9						247						

END OF BOREHOLE

*Sampler sinking under weight of hammer and/ or rods.

Piezometer installation consists of a 50mm diameter PVC pipe with a 3.0m slotted screen.

Piezometer installed 0.3m east and 1.8m north of this borehole on September 17, 2015.

Consolidation test performed on TW9.

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

WATER LEVEL READINGS

Date	Water Depth (m)	Elevation (m)
Sep 28, 2015	1.2	264.5
Oct 9, 2015	1.3	264.4

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PART B – FOUNDATION DESIGN REPORT

for

DECEPTION CREEK BRIDGE REPLACEMENT

SITE NO. 39E-0169/B0

HIGHWAY 668 – STATION 10+265

TOWN OF COCHRANE, ONTARIO

G.W.P. 5267-11-00

W.P. 5368-11-01

LATITUDE AND LONGITUDE: 49.27422, -81.07419

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Index No.: 039FDR
GEOCRES No.: 42H-82
August 21, 2019

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Town of Cochrane, Ontario, G.W.P. 5267-11-00, W.P. 5368-11-01, Index No.: 039FDR
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Appendix C – Sketch No. PML-1

List of Standard Specifications Relevant to Report
Non-Standard Special Provisions (NSSP)

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Deception Creek Bridge Replacement
Site Nos. 39E-0169/B0
Highway 668 – Stations 10+265
Town of Cochrane, Ontario
G.W.P. 5267-11-00, W.P. 5267-11-01 and 5363-01

7. 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.

8. PROJECT DESCRIPTION

8.1 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) drawings for the proposed replacement of bridge on Highway 668 at the crossing of Deception Creek in the Town of Cochrane, Ontario.

Based on the GA drawing, it is proposed to construct the replacement bridge with a single-span structure supported on integral abutments.

PML understands that the highway during the construction of both bridges across Smith Creek and Deception Creek will be closed and the traffic will be diverted through a local detour.

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.

8.2 Existing Structure

The existing bridge is a seven (7) span, timber structure with a total length of 33.0 m. Five of the seven spans are 4.9 m long and the other two are 4.1 m long. The existing bridge is 9.0 m wide

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and accommodates two (2) lanes of vehicular traffic (northbound and southbound), over the Deception Creek. This bridge was constructed in 1962.

The Ontario Bridge Management System (OBMS) inspection report, dated July 11, 2017, identified light weathering, split and checks, localized loss of fill behind lagging, wide gap between some lagging and piles shimmed at the abutment walls. Minor splintering of pile cap at the abutments was also identified. At the pier locations, Medium to severe rot at end of several pier caps with light checking, loss of preservation (creosote) and splintering were noted at the pier locations. Rotation of multiple number of piles away from the caps were identified. Report indicates that extensive repair and reinforcing work was carried out for the pier caps and piles. Two (2) braces were reported missing, one at pile bent 3 and other at bent 6. Settlement with narrow to wide cracking at edge of approach, minor rutting and ponding near the southwest quadrant were identified at the approaches. It was noted that the surface treatment that was carried out did not extend to the full width between curbs. Minor displacement of timber with minor splitting and checking were identified in the interior soffit. Localized spalls and delaminations of concrete with narrow to medium cracks, light scaling and light honeycombing were observed on the exterior soffit. Evidence of severe erosion was noted in front of the north abutment. A beaver dam was identified in the watercourse below the bridge.

8.3 Proposed Structure

Based on the GA drawing dated March 2019, it is proposed to construct the replacement bridge with a 31.0 m long single-span structure supported on integral abutments. The proposed structure will consist of 5.2 m long cantilevered sections, extending from the 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. 261.7 and El. 261.6, respectively.

The approach slabs will be 6.0 m long at both abutments. The design grade of the approach embankments at the north and south abutments will be set at about El. 266.9 and 266.6, respectively, which will result in a grade raise of approach embankments by approximately 1.2 m at both abutments.

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8.4 Structure Foundations

In general, the subsoil conditions immediately below the ground surface consist of 300 mm pavement structure in the area of the existing road. The pavement structure is underlain by 2.1 m to 3.8 m thick fill composed of clayey silt. The fill, with the exception of borehole located on the south approach, is followed by firm to stiff clayey silt. The fill in the south approach area is followed by 1.5 m of loose silt, which is underlain by soft to firm silty clay to clayey silt. The clayey silt layer in other three boreholes is underlain by loose silt. The clayey silt and silt layers are followed by loose to compact till deposit composed of silty sand to sandy silt. The silty sand to sandy silt deposit is underlain by unweathered diabase bedrock. The bedrock surface at the south and north abutments were intercepted at El. 250.1 and El. 249.1, respectively, compared to the bedrock elevations 248.4 and 251.2 reported in the preliminary investigation report.

The water level in Deception Creek was observed at approximately El. 262.0.0 during the fieldwork. The groundwater level was observed at depths varying between 4.6 m and 10.7 m during drilling, and was measured at depths varying from 7.0 m to 9.9 m upon completion of drilling.

8.4.1 Foundation Recommendation

Considering the subsoil conditions encountered at this site, supporting the proposed replacement bridge on a shallow foundation is not considered favourable and require deep foundations. It is recommended that the proposed replacement bridge be supported on HP 310 x 110 steel piles driven to bedrock. The factored geotechnical resistance at ULS of the diabase bedrock encountered at this site may be in the order of 10 MPa.

The piles may be lowered into a 600mm diameter pre-augered hole supported by CSP to a depth of 3.0 m from the pile cut-off elevation and driven to bedrock. The steel H-piles driven to bedrock may be designed assuming a factored axial geotechnical resistance of 2000 kN at Ultimate Limit State (ULS), in accordance with the Structural Office Policy Memo 98-01 dated April 15, 1998. Axial capacity at SLS will not govern because the loads required to produce detrimental deformation is anticipated to be larger than the factored resistance at ULS.

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Table 8.4.1a below summarizes the approximate pile tip elevations and length of piles that may be considered for design purposes.

Table 8.4.1a: Pile Tip Elevations and Length

STRUCTURE SITE	LOCATION	PILE TIP ELEVATION	PILE CUT-OFF ELEVATION	LENGTH (m)
Deception Creek Bridge (Site No. 39E-0169/B0)	North Abutment	250.1 to 248.4* \pm 1.0	261.7	11.6 to 13.3 \pm 1.0
	South Abutment	251.2* to 248.7 \pm 1.0	261.6	10.4 to 12.9 \pm 1.0

Note : (*) – GEOCRETS 42H-64, dated September 16, 2016.

The construction of pile foundation should be in accordance with OPSS.PROV 903 and SP 109F57. 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 pile tips need to be reinforced to drive the piles through 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. The pile tip reinforcement shown on the attached Sketch No. PML-1 is recommended.

Boulders or cobbles were not encountered in the boreholes drilled during the current investigation. However, the borehole data from the preliminary foundation investigation (Geocres No. 42H-64) reveals that cobbles and boulders were encountered. Considering the nature of glacial till deposits and data from the preliminary foundation investigation, there may be potential for intercepting boulders or cobbles within this deposit during the installation of piles. For this reason, a NSSP is included in the appendix to alert the contractor for potential obstruction from cobbles or boulders during the pile installation.

Considering the proposed grade raise of about 1.2 m at the approaches, appreciable settlement of the approaches is anticipated to allow for negative skin friction (down drag) loads on the piles.

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To evaluate the point of contraflexure, the coefficient of horizontal subgrade reaction, k_s (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)

$$k_s = 67 \tau_u/d$$

where; τ_u = Undrained shear strength
 d = Pile diameter or width, m

The coefficient of horizontal subgrade reaction values provided in Table 8.4.1b, may be used to determine the point of contraflexure for HP 310 x 110 steel H-piles:

Table 8.4.1b: Coefficient of Horizontal Subgrade Reaction for Piles

STRUCTURE SITE	STRUCTURE	SOIL BOUNDARY ELEVATION (m)	SOIL TYPE	nh VALUES (kN/m ³)	UNDRAINED SHEAR STRENGTH (kN/m ²)
Deception Creek Bridge (Site No. 39E-0169/B0)	South Abutment	260.9 to 257.9	Loose Sand	1,000	-
		257.9 to 254.8	Loose Silt	1,000	-
		254.8 to 250.1	Loose Silty Sand	1400	-
	North Abutment	261.2 to 258.2	Loose Sand	1,000	-
		258.2 to 255.2	Loose Silt	1,000	-
		255.2 to 249.1	Loose to Compact Silty Sand to Sandy Silt	2000	-

8.5 Retaining Soil System (RSS) Wall

At the time of the preparation of this report, the requirement for an approximate 2.0 m high RSS wall at the southeast embankment was being assessed. The RSS wall constructed at this site should be “High Performance” and “High Appearance”. The design of the RSS wall shall be the

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responsibility of the proprietary RSS supplier and the design should meet the MTO RSS Design Guideline, Special Provision 599S22 and Special Provision 599S23.

The internal stability and sliding resistance of the RSS wall will be assessed by the designer of proprietary product and the structural integrity of the wall will remain the responsibility of the supplier.

The final design of the retaining structures should be checked against CFEM (4th edition), Chapter 27: Reinforced Soil Walls.

The proposed RSS wall may be founded on clayey silt fill or compacted granular fill at approximate elevation 264.0. The RSS wall placed at or below the elevation recommended may be designed assuming a factored geotechnical resistance at ULS of 150 kPa and factored geotechnical resistance of 100 kPa at SLS. Any soft or spongy area observed below the founding elevation recommended should be removed and replaced with granular fill in accordance with OPSS 501.

9. 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/m³)

γ' = unit weight of submerged backfill ($\gamma - \gamma_w$) material below assumed water level (kN/m³)

γ_w = unit weight of water (9.8 kN/m³)

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 \emptyset = angle of internal friction of retained soil (35° for Granular A or 30° for Granular B Type II)

δ = angle of friction between soil and wall (24° for Granular A or B Type II)

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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 9.0: 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 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.

10. APPROACH EMBANKMENT

Based on the GA drawing, it is anticipated that the approach embankments will be raised approximately 1.2 m at the south and north abutments from existing grade consisting of rock fill.

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The existing bridge was constructed in 1962 and the approach embankment fill has been in place for almost 57 years.

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.

The estimate of the magnitude of total settlements induced by the new fill are based only on the elastic compression of the newly placed rock fill itself, elastic compression of subgrade, and the primary consolidation of the subgrade under the embankment. Considering the geometry of the rock slope (1.25H:1V), the new fill is not expected to impose appreciable load on the subgrade as well as on the virgin ground. The maximum height of the proposed embankments is expected to be about 1.2 m above the grade of existing surface and is expected to impose a maximum load of about 22 kPa at the existing road surface, assuming a compacted density of rock fill about 18 kN/m³. Elastic compression of the subgrade soil was estimated assuming a modulus value of 3500 kPa to 4500 kPa and a Poisson ratio of 0.4 for the existing embankment with no added base width from the new rock fill. Based on the estimate, The primary consolidation settlement resulting from the underlined clayey layer was estimated assuming a C_c value of 0.26 and a void ratio of 0.83, based on the preliminary investigation report. it is anticipated that approximately 40 mm to 50 mm of primary settlement of the clayey subsoil and 10 mm to 15 mm of elastic compression settlement of cohesionless soil will occur due to the additional embankment loading. It is estimated that 90% of the primary compression will complete approximately in three months. The elastic compression will occur during the construction of the embankments.

Considering the subsoil conditions at this site, no major instability problems are anticipated for the embankments constructed with 1.25H:1V side slope consisting of rockfill. Any spongy or soft area observed within the base of the embankment should be removed before placing the fill.

In order to prevent loss of fines from the existing earth slopes, it is recommended to place 0.5 m thick Granular A on the existing slopes followed by 0.5 m thick crushed stone (20 mm to 25 mm) under the proposed rockfill.

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It is suggested that the approach embankments fill be placed at least three months in advance of the construction of bridge and the paving of the road should be delayed by two to three weeks after placement of fill to the designed grade of the embankment, to mitigate post-construction settlement effects.

11. 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 two sites are 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.

12. SEISMIC CONSIDERATIONS

The Spectral ($S_a(T)$, where T is in seconds) and Peak Ground Acceleration (PGA) for the project site is 0.221 ($S_a(0.2)$) and 0.144 (2%/50 years) 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.

13. CONSTRUCTION CONSIDERATIONS

13.1 Excavation

Based on the cut-off elevations of the piles, the depth of excavations are expected to be about 3.9 m and 4.2 m at the south and north abutment locations, respectively. 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 stiff to firm clayey soils and compact to loose cohesionless soils may be classified as Type 3 soils.

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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. For excavations through multiple soil types, the side slope geometry is governed by the soil with the highest number designation. Since the 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 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.

13.2 Groundwater Control

The cut-off elevations of the piles to support the north and south abutments are proposed to be at El. 261.7 and El. 261.6, respectively. The depth of excavations at the south and north abutment locations for construction of integral abutments are expected to be about 3.9 m to 4.2 m, respectively, below the existing ground level.

Considering the depth of excavation, no major dewatering problems are anticipated at both sites. 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. It is suggested that construction of the bridge replacements should be carried out during the drier season.

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13.3 Soil Corrosivity

A total of four (4) samples 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 5.2.10 of Part A of this report. The sulphate concentration varied from 4.0 µg/g to as high as 52 µg/g (0.0004% to 0.0052%), 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 soils on buried concrete structures may be negligible. The chloride contents of the samples from the fill ranged from as low as 2.1 µg/g to 40 µg/g (0.00021% to 0.004%). 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 is assessed to be low.

Electrical resistivity less than 2000 ohm-cm generally leads to highly corrosive environment for steel elements in contact with soil. The resistivity values ranged from 4460 ohm-cm to 5170 ohm-cm, indicating negligible to low corrosive environment for steel elements in contact with soils. The pH values of soil samples ranged from 7.96 to 8.29 compared to the value of 5.5 that generally leads to corrosion.

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14. CLOSURE

This Foundation Investigation and Design Report was prepared by Mr. K. Amatya, MEng, P.Eng., Project Engineer Geotechnical Services, and reviewed by Mr. N.Rahman P.Eng. 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.



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Project Manager and
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KA/NR/RN.nr-nk

Part B – Foundation Design Report

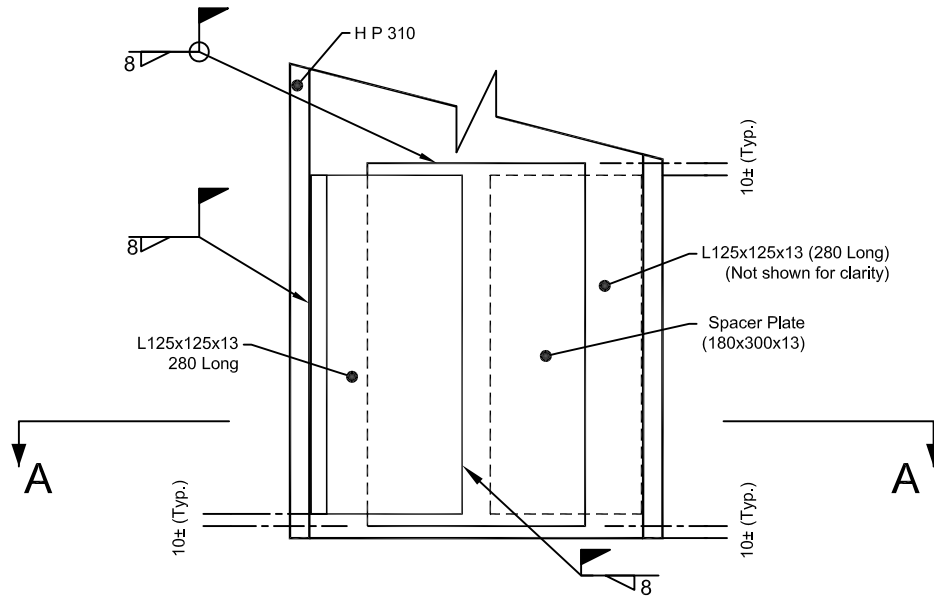
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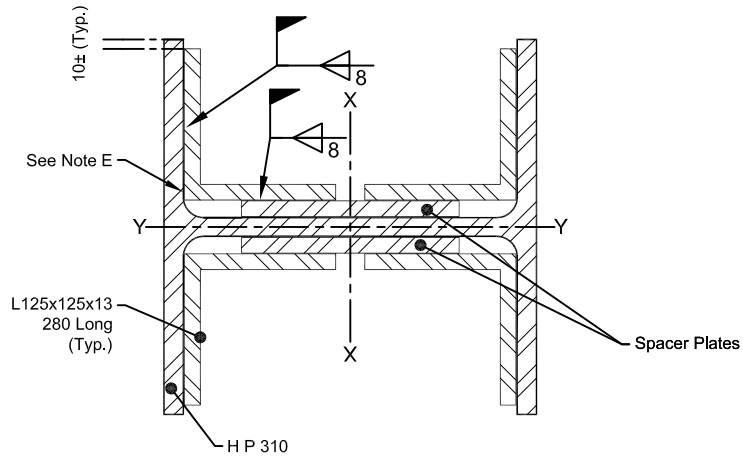
APPENDIX C

Sketch No. 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



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

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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 109F57	Amendment to OPSS 903, Special Provision
NSSP FOUN0003	Dewatering Structure Excavations, Amendment to OPSS 902

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NON-STANDARD SPECIAL PROVISIONS (NSSP)

NSSP – Potential for Cobbles and Boulders during Pile Driving

Glacial till deposit overlies the bedrock encountered at this site. Considering the nature of the glacial till deposits, there is potential for presence of cobbles or boulders within this deposit.

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. Piles meeting refusal on a boulder may need to be relocated, have their capacity reduced and / or require additional piles to be installed.

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

AMENDMENT TO OPSS 903, APRIL 2016

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.

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.