FOUNDATION INVESTIGATION AND DESIGN REPORT

for

REPLACEMENT OF UNDERPASS AT HIGHWAY 401 AND CONCESSION ROAD 7
HIGHWAY 401, SITE NO. 35-351, STATION 21+042
PUSLINCH TOWNSHIP, COUNTY OF WELLINGTON
LATITUDE: 43.44977; LONGITUDE: -80.15391
ASSIGNMENT NO. 3014-E-0014
GWP 3224-15-00

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

Distribution:
1 cc: AECOM for distribution to MTO
Project Manager + One (1) Digital Copy (PDF)
1 cc: AECOM for distribution to MTO
Foundations Section
+ One (1) Digital Copy (PDF)
1 cc: AECOM + One (1) Digital Copy (PDF)
1 cc: PML Toronto

PML Ref.: 17TF006A
Index No.: 022FIR and 023FDR
GEOCRES No.: 40P8-253
June 29, 2018

Peto MacCallum Ltd.
CONSULTING ENGINEERS

PART A - FOUNDATION INVESTIGATION REPORT

for

REPLACEMENT OF UNDERPASS AT HIGHWAY 401 AND CONCESSION ROAD 7
HIGHWAY 401, SITE NO. 35-351, STATION 21+042
PUSLINCH TOWNSHIP, COUNTY OF WELLINGTON
LATITUDE: 43.44977; LONGITUDE: -80.15391
ASSIGNMENT NO. 3014-E-0014
GWP 3224-15-00

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

Distribution:
1 cc: AECOM for distribution to MTO
Project Manager + One (1) Digital Copy (PDF)
1 cc: AECOM for distribution to MTO
Foundations Section
+ One (1) Digital Copy (PDF)
1 cc: AECOM + One (1) Digital Copy (PDF)
1 cc: PML Toronto

PML Ref.: 17TF006A
Index No.: 022FIR
GEOCRES No.: 40P8-253
June 29, 2018

TABLE OF CONTENTS

PART A - FOUNDATION INVESTIGATION REPORT

PART A - FOUNDATION INVESTIGATION REPORT
for
Replacement of Underpass at Highway 401 and Concession Road 7
Highway 401, Site No. 35-351, Station 21+042
Puslinch Township, County Of Wellington
Latitude: 43.44977; Longitude: -80.15391
Assignment No. 3014-E-0014, GWP 3224-15-00

---

1. INTRODUCTION

Ministry of Transportation Ontario has retained AECOM Canada Ltd. (AECOM) as the Prime Consultant, to provide Owner's Engineer services for the re-alignment, improvement and replacement of existing structures on Highway 6 from Hamilton to Guelph. The assignment comprises of three separate projects, each of the project has been assigned different General Work Project (GWP) numbers. Each of these projects have specific delivery model and the requirements that were outlined in the Request for Proposal (RFP) are as follows:

- GWP 3042-14-00; Highway 6 and Highway 401, Hamilton to Guelph, Design-Build Project Delivery and Owner's Engineer Services
- GWP 14-00-00; Highway 6 (Hanlon Expressway), Design-Build Project Delivery
- GWP 3224-15-00; Replacement of Underpass, Highway 401 and Concession Road 7, Bridge #11, Puslinch Township, Design-Bid-Build Project Delivery

AECOM has retained Peto MacCallum Ltd. (PML) on behalf of the Ministry of Transportation Ontario (MTO) to provide geotechnical engineering services for the assignment. The geotechnical investigation work reported herein is part of the assignment under GWP 3224-15-00, to prepare a detail design for the replacement of existing underpass located at the crossing of Highway 401 and Concession Road 7.

This report presents the factual findings obtained from the geotechnical investigation carried out for the proposed replacement of the bridge to be located on a new alignment west of the existing crossing of Highway 401 and Concession Road 7. The new bridge will be constructed at approximate Sta. 21+042 (assumed by AECOM), in the Township of Puslinch, County of Wellington, Ontario.

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

2. SITE DESCRIPTION

The topography of the project area is generally flat, except for the highway embankments. Generally, the site surrounding the bridge is covered with bushes and grass. The area along the highway on both, north and south, sides is moderately vegetated with grass, trees and shrubs. The Highway 401 in this area passes through several shallow depressions, which are damp or swampy. The land uses adjacent to the project site range from industrial, commercial, residential to undeveloped lands.

Based on the observations made during the site investigation, no major sign of structural distress of the bridge due to settlement was observed other than the delamination of concrete along the deck and reinforcements are exposed at several locations. The approach embankments with a slope of about 2H:1V appear in good condition and no surface erosion is visible.

3. FIELD INVESTIGATION PROCEDURES

The PML staff visited the site on October 25, 2017, to mark out the borehole locations. The underground services at the borehole locations were cleared by the respective utility companies. Public and private utility authorities were informed and all the utility clearance documents were obtained before the commencement of drilling work.

The fieldwork was carried out between October 31, 2017 and January 17, 2018 and the location of boreholes in the field was established by PML staff using a portable GPS device. Subsequently, J.D. Barnes Limited, Ontario, under contract to PML carried out the survey of the locations and elevations of the boreholes and provided the co-ordinates for locations in MTM NAD 83 northing and easting. PML used the survey data provided by J.D. Barnes Limited for preparation of this report. All elevations reported in this report are referred to Geodetic datum and expressed in meters.

The drilling equipment from three different contractors were used for the field investigation. The equipment used were owned and operated by Landshark Drilling, Aardvark Drilling Inc., and Altech Canada, who are specialist drilling contractors. The fieldwork was carried out under the full-time supervision of a PML field supervisors.

Table 1- Borehole Information

BOREHOLE NO.	BOREHOLE LOCATION	MTM NAD 83 COORDINATES		GROUND SURFACE ELEVATION (m)	BOREHOLE DEPTH (m)
		NORTHING	EASTING		
35-351-01	North Embankment	4812484.9	251867.3	317. 0	14.3
35-351-02	North Abutment	4812456.1	251897.2	318.4	29.1
35-351-03		4812456.0	251866.4	311.1	26.1
35-351-04	Pier	4812414.8	251875.3	311.8	25.0
35-351-05		4812420.2	251860.7	311.8	29.4
35-351-06	South Abutment	4812382.6	251872.3	310.2	20.1
35-351-07		4812376.5	251875.3	311.3	27.5
35-351-08	South Embankment	4812359.4	251877.6	313.1	9.8

The investigation included advancing eight (8) boreholes numbered 35-351-01 to 35-351-08. These boreholes were advanced using hollow stem augers powered by a CME 850 track-mounted drill rig and C 57 truck-mounted drill rig. Rock coring was carried out in Boreholes 35-351-03 and 35-351-05 using a NQ size double core barrel, to confirm the presence of bedrock. The location of boreholes is shown on the attached Drawing 35-351-1.

Representative soil samples were recovered from the boreholes at 0.75 m intervals to a depth of 5.0 m, using a conventional 51 mm O.D split spoon sampler in accordance with the Standard Penetration Test (SPT) procedure. The frequency of sampling was increased to 1.5 m intervals below the depth of 5.0 m. Standard penetration tests and cone penetration tests were conducted with the sampling operation to assess the strength characteristics of the substrata. Boreholes 35-351-02, 35-351-04 and 35-351-06 were advanced to the depth of the zone of influence for shallow foundations and below the sampling depths by conducting Dynamic Cone Penetration (DCP) test to refusal. The DCPT values validated the SPT values below the water table which are susceptible to disturbance from groundwater during sampling.

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 and also water level measurements were taken in open boreholes. In addition, two monitoring wells were installed in Boreholes 35-351-03 and 35-351-07 to monitor the groundwater levels for geo-environmental purposes.

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

The recovered soil samples were returned to our laboratory for detail visual examination, and index tests.

4. LABORATORY TEST PROCEDURES

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

- Natural moisture content determinations (106)
- Grain size distribution analyses (25)
- Unconfined compression strength of rock (2)

The laboratory tests to determine the index properties were performed in accordance with the MTO test procedures, which follow American Society for Testing Materials (ASTM) test procedures, with the exception of hydrometer test (LS-702). Unconfined compressive strength of intact rock core specimens was determined in accordance with the ASTM D 7012 test procedures. The results of the grain size distribution analyses are presented on Figures 351-GS-1, 351-GS-2A, 351-GS-2B and 351-GS-2C. All of the test results are summarized on the attached Record of Borehole sheets.

One soil sample from the silty sand with gravel layer was submitted to AGAT Laboratories in Mississauga, Ontario, for testing of chemical properties relevant to exposure of concrete elements to sulphate as well as potential soil corrosion effects. Detail test results provided by AGAT laboratories are presented in Appendix A.

5. SITE GEOLOGY AND SUBSURFACE CONDITIONS

5.1 Site Geology

In general, the project area is located within the physiographic region known as Horseshoe Moraines. The area where the site is located is marked by the old spillway containing flat sand and gravel terraces and some linear, undrained swampy areas, as outlined in *The Physiography of Southern Ontario* (Chapman and Putnam, 1984). The soil deposits of the area is a coarse, open, stony till composed largely of dolostone with traces of red shale.

The Quaternary Geology map published by the MNDM indicates that the surface conditions in the vicinity of the bridge site consist of Glaciofluvial outwash deposits: gravel and sand; includes Proglacial River and deltaic deposits.

Based on the Bedrock Geology map (MRD126-REV1, 2011) published by the Ontario Ministry of Northern Development and Mines (MNDM), the bridge site lies within the Guelph bedrock formations. The project area consists mainly of sandstone, shale, dolostone, and siltstone rock.

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 Record of Borehole Sheets attached to the report. The borehole locations plan and a stratigraphic profile section are shown on Drawing 35-351-1. 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 subsurface conditions immediately below the existing ground level consist of 800 mm to 1.5 m pavement structure in the paved area and 200 mm to 300 mm of topsoil in boreholes that were advanced near the toe of the embankment. The topsoil and pavement structure are underlain by compact to very dense silty sand with gravel to the maximum depth of EL. 283.5 and the silty sand is followed by dolostone bedrock of Guelph Formation. For classification purposes, the soils encountered at this site can be divided into five distinct zones.

a) Topsoil
b) Silty Sand, With Gravel (Pavement Structure)
c) Silty Sand, With Gravel (Fill)

d) Silty Sand, With Gravel
e) Dolostone Bedrock

5.2.1 Topsoil

Topsoil was encountered in Boreholes 35-351-03, 35-351-06, 35-351-07 and 35-351-08, immediately below the existing ground surface at the locations of borehole. The thickness of the topsoil was observed to vary from 200 mm to 300 mm.

5.2.2 Silty Sand, With Gravel (Pavement Structure)

The pavement structure was encountered immediately below the existing grade in Boreholes 35-351-01, 35-351-02, 35-351-04 and 35-351-05. The pavement structure includes 100 mm to 180 mm of asphalt over 620 mm to 900 mm of silty sand, with gravel.

The moisture content of samples tested from the pavement base vary from 2.5% to 7.0% with an average value of 4.0%.

5.2.3 Silty Sand, With Gravel (Fill)

The pavement structure in Boreholes 35-351-01 and 35-351-02 are followed by compact to very dense silty sand, with gravel fill. This fill layer ranges in thickness between 3.6 m and 5.1 m and extends to a maximum depth of 6.1 m (El. 310.9) below the existing grade. The SPT values in this fill layer varies from 16 blows to 57 blows, indicating compact to very dense state of denseness. Occasional cobbles were encountered in this fill at elevation El. 312.4 in Borehole 35-351-01 and at El. 316.1 in Borehole 35-351-02.

The moisture content of samples tested from this fill vary from 3.0% to 6.8% with an average value of 4.8%. The results of the sieve analysis test performed on one representative sample from this fill is provided on Figure 351-GS-1. The test result indicates that this fill consists of 44% gravel, 37% sand, 15% silt and 4% clay sized particles.

5.2.4 Silty Sand, With Gravel

The fill and topsoil layers are immediately underlain by this silty sand with gravel deposit, which extends to a maximum depth of 29.1 m (El. 283.8). The SPT values in this deposit vary widely from as low as 11 blows/30 cm to refusal (100 blows/3 cm penetration), indicating compact to very dense state of denseness. The Boreholes 35-351-02, 35-351-04 and 35-351-06 were extended below the sampling depths ranging from 12.8 m (El. 305.6) to 13.7 m (El. 298.1) by conducting dynamic cone penetration (DCP) test and terminated at the depths where refusal to dynamic cone penetration was encountered. The termination depths range from 20.1 m (El. 290.1) to 29.1 m (El. 289.3). Occasional cobbles were encountered in this deposit below elevations ranging from El. 310.9 to El. 294.4.

In Borehole 35-351-03 and 35-351-05, the silty sand extend to bedrock at 22.6 and 26.1 m (El. 285.0 and 285.7) respectively. In Borehole 35-351-07, the lower 3.0 m of the silty sand gave SPT N-values of 100 blows for 3 cm and the borehole was terminated by practical refusal at 27.5 m depth (El. 283.8).

The moisture content of samples tested from this deposit vary widely from 2.1% to 28.5% with an average value of 11.9%. The results of the sieve analysis test performed on 23 representative samples from this deposit are provided on Figures 351-GS-2A, 351-GS-2B and 351-GS-2C. The test results indicate that this deposit consists of 3% to 70% gravel, 27% to 89% sand, 3% to 29% silt and 1% to 8% clay sized particles.

5.2.5 Dolostone Bedrock

The presence of bedrock was proven by coring in Boreholes 35-351-03 and 35-351-05 and obtaining 3.5 m and 3.3 m long rock cores, respectively. The rock coring was terminated at a depth of 26.1 m (El. 285.0) in Borehole 35-351-03 and 29.4 m (El. 282.4) in Borehole 35-351-05, below the existing ground surface.

The measured recovery of the rock cores range between 78% and 100% and the RQD measured from the rock cores retrieved range between 75% and 97%, with the exception of Sample 18 in Borehole 35-351-03. Based on the RQD values, the bedrock below about El. 288.0 may be described as good to excellent quality. For complete descriptions of the bedrock, refer to rock core description logs provided in Appendix A. Unconfined compressive strength (UCS) of rock core tested ranges from 47.6 MPa to 76.1 MPa. Based on the unconfined compression test values, the bedrock may be classified as medium to strong with respect to strength.

5.3 Groundwater

Groundwater table at the time of field investigation was encountered between approximately El. 305.2 and El. 309.3. However, the stabilised groundwater level was recorded one month after the completion of drilling at EL. 305.6 in the monitoring well installed near the proposed south abutment. The groundwater levels may fluctuate due to the influence of precipitation and seasonal changes. The groundwater level in each borehole is as follows:

Table 5.3 - Groundwater Levels

BOREHOLE NO.	ELEVATION OF WATER LEVEL (m)	DEPTH TO WATER (m)	DATE
35-351-01	305.2	11.8	Nov. 14, 2017
35-351-02	309.3	9.1	Nov. 14, 2017
35-351-03	306.4[1]	4.7[1]	Nov. 14, 2017
35-351-04	306.7	5.1	Jan. 17, 2018
35-351-05	Not Established (drill water)	Not Available	Dec. 6, 2017
35-351-06	306.4	3.8	Nov. 9, 2017
35-351-07	305.7[1]	5.7[1]	Nov. 14, 2017
35-351-08	Dry	Not Available	Nov. 3, 2017

Note 1: Groundwater level measured in monitoring wells installed.

Additional water levels in the piezometer installed in boreholes 35-351-03 and 35-351-07 are shown in the respective records of boreholes.

5.4 Chemical Analysis

A summary of the chemical test results provided by AGAT Laboratories is summarized in the table below. The detail test results provided by AGAT Laboratories are also presented in Appendix A.

Table 5.4 - Soil Chemical Analysis Results

BOREHOLE NO.	SAMPLE	DEPTH / ELEVATION (m)	SOIL TYPE	SULPHATE (µg/g)	CHLORIDE (µg/g)	pH	RESISTIVITY (Ohm-cm)
35-351-06	SS-5	3.0-3.7 / 308.8-308.1	Silty Sand, with Gravel	8	28	9.36	7630

6. CLOSURE

Mr. S. Aziz and Mr. M. Fall, carried out the field investigations under the supervision of Mr. L. Yimam, Ph.D., P. Eng., Senior Engineer and Mr. C. M. P. Nascimento, P.Eng., Project Manager. Landshark Drilling, Aardvark Drilling Inc., and Altech Canada Drilling Ltd. supplied the drilling equipment for the subsurface exploration. The laboratory testing of the selected samples was carried out in the PML laboratory in Toronto. Chemical tests on soil sample were performed by AGAT Laboratories of Mississauga, Ontario.

This report was prepared by Ms. Asieh Khadem, M.Sc. Eng., EIT, Project Supervisor and Mr. N. Rahman, P.Eng. and reviewed by Mr. Mark Vasavithasan, M.Sc. Eng., P.Eng., Senior Engineer, Geotechnical Services. Mr. C.M.P. Nascimento, P.Eng., Principal Consultant, conducted an independent review of the report.

Yours very truly

Peto MacCallum Ltd.

LICENSED PROFESSIONAL ENGINEER
A. N. RAHMAN
100189818
June 29, 2018
PROVINCE OF ONTARIO

Nazibur Rahman, P.Eng.
Senior Engineer, Geotechnical Services

REGISTERED PROFESSIONAL ENGINEER
C. M. P. Nascimento
June 29, 2018
PROVINCE OF ONTARIO

Carlos M.P. Nascimento, P. Eng.
Project Manager and
MTO Designated Principal Contact

AK/MV/NR/CN:nk

APPENDIX A

Borehole Locations Plan and Soil Strata at Structure 35-351-1

Explanation of Terms Used in Report

Record of Borehole Sheets

Results of Grain Size Distribution Analyses – Figures 351-GS-1, 351-GS-2A, 351-GS-2B and 351-GS-2C

Rock Core Description Logs

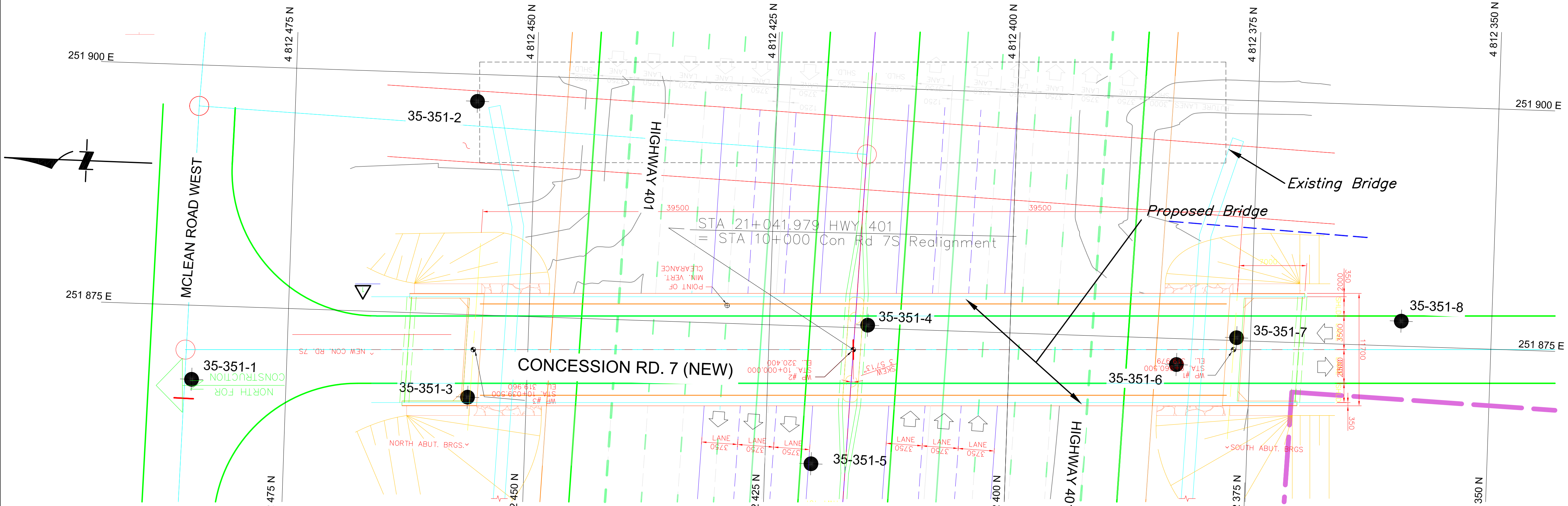
Chemical Test Results

GWP No 3224-15-00

CONCESSION ROAD 7 UNDERPASS
HWY 401 & HWY 6 REALIGNMENT
BOREHOLE LOCATION AND SOIL STRATA

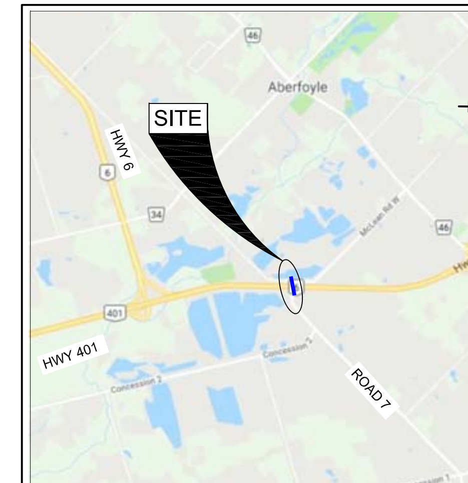
SHEET

Peto MacCallum Ltd.
CONSULTING ENGINEERS



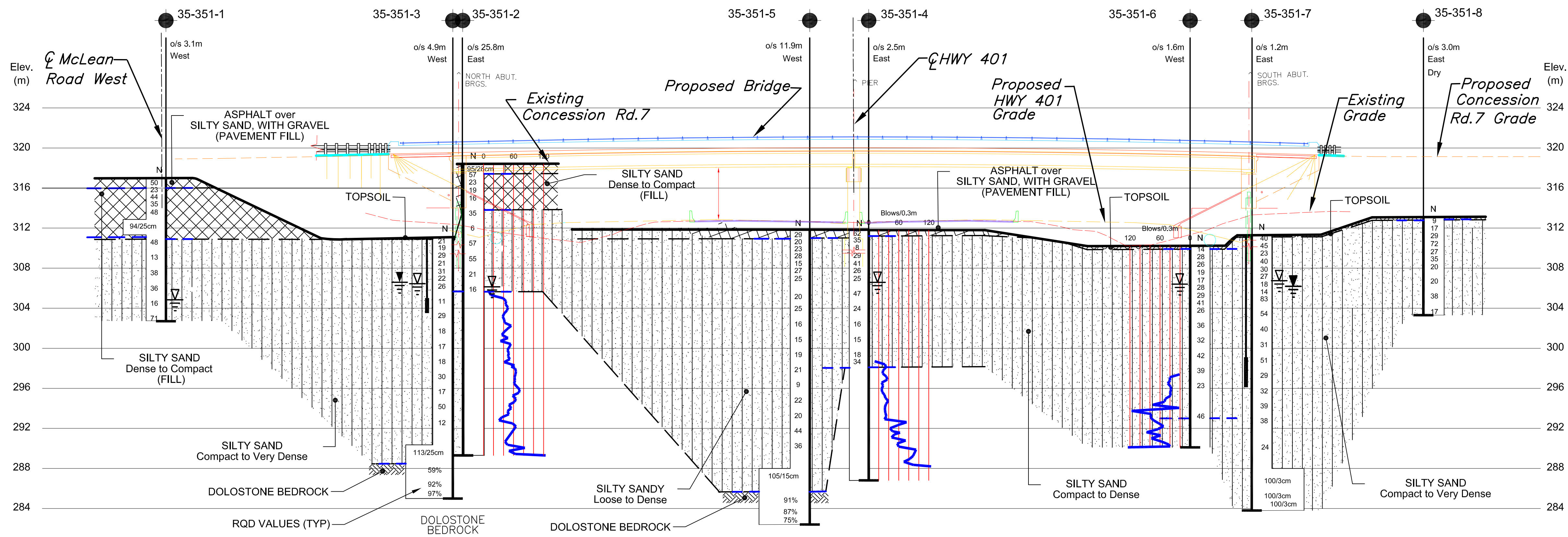
PLAN

SCALE 5 0 5 10m



KEY PLAN

500m 0 500m 1Km 2km



PROFILE ALONG ℄ CONCESSION RD 7

SCALE

HORIZONTAL 5 0 5 10m

VERTICAL 4 0 4 8m

LEGEND

- Borehole
- Cone
- Borehole by others
- N Blows/0.3m (Std. Pen Test, 475 J/blow)
- CONE Blows/0.3m (60 Cone, 475 J/blow)
- WL Encountered During Drilling
- Ground Water Measured Apr. 03, 2018
- Piezometer

BH No	ELEVATION	CO-ORDINATES NORTHINGS	CO-ORDINATES EASTINGS
35-351-1	317	4 812 484.9	251 867.3
35-351-2	318.4	4 812 456.1	251 897.2
35-351-3	311.1	4 812 456.0	251 866.4
35-351-4	311.8	4 812 414.8	251 875.3
35-351-5	311.8	4 812 420.2	251 860.7
35-351-6	310.2	4 812 382.6	251 872.3
35-351-7	311.3	4 812 376.5	251 875.3
35-351-8	313.1	4 812 359.4	251 877.6

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

NOTES:

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

Reference AECOM Drawing: 10-REFERENCE\35-351_Concession Rd_ MTO_bdr.dwg, dated June 2018

REVISIONS	DATE	BY	DESCRIPTION

Geocres No. 40P8-253

HWY No	401			DIST	CENTRAL
SUBM'D	TC	CHECKED AC	DATE JUNE 26, 2018	SITE	35-351
DRAWN	TC	CHECKED MV	APPROVED CN	DWG	35-351-1

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 (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

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

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
r_u	1	PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ϵ	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

MECHANICAL PROPERTIES OF SOIL

m_v	kPa^{-1}	COEFFICIENT OF VOLUME CHANGE
C_c	1	COMPRESSION INDEX
C_s	1	SWELLING INDEX
C_α	1	RATE OF SECONDARY CONSOLIDATION
c_v	m^2/s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	-°	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_t	1	SENSITIVITY $= \frac{c_u}{\tau_r}$

PHYSICAL PROPERTIES OF SOIL

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

RECORD OF BOREHOLE No 35-351-01

METRIC

G.W.P. 3224-15-00 LOCATION Coords: 4 812 484.9N; 251 867.3 E ORIGINATED BY S.A.
DIST Central HWY 6S BOREHOLE TYPE Hollow Stem Augers COMPILED BY A.K.
DATUM Geodetic DATE 2017.11.14 LATITUDE 43.4504 LONGITUDE -80.15401 CHECKED BY M.V.

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
317.0	GROUND SURFACE					
0.0	130 mm ASPHALT over silty sand with gravel (PAVEMENT FILL)	1	SS	50		
316.0 / 1.0	SILTY SAND, with gravel. Dense to compact, Brown, Damp	2	SS	23	316	
		3	SS	44	315	
		4	SS	35	314	
		5	SS	48	313	
	occasional cobbles	6	SS	94/25cm	312	44 37 15 4
	(FILL)				311	
310.9 / 6.1	SILTY SAND, with gravel. Compact to very dense, Brown, Dry	7	SS	48	310	
		8	SS	13	309	
		9	SS	38	308 / 307	49 34 (17)
		10	SS	36	306 / 305	
		11	SS	16	304	
		12	SS	71	303	47 45 (8)
302.7 / 14.3	End of borehole					

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L, WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m^3.

Continued Next Page

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

RECORD OF BOREHOLE No 35-351-01

METRIC

G.W.P. 3224-15-00 LOCATION Coords: 4 812 484.9N; 251 867.3 E ORIGINATED BY S.A.

DIST Central HWY 6S BOREHOLE TYPE Hollow Stem Augers COMPILED BY A.K.

DATUM Geodetic DATE 2017.11.14 LATITUDE 43.4504 LONGITUDE -80.15401 CHECKED BY M.V.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
302.0											
	∇ Water level observed during drilling ▼ Water level measured after completion of drilling										

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

RECORD OF BOREHOLE No 35-351-02

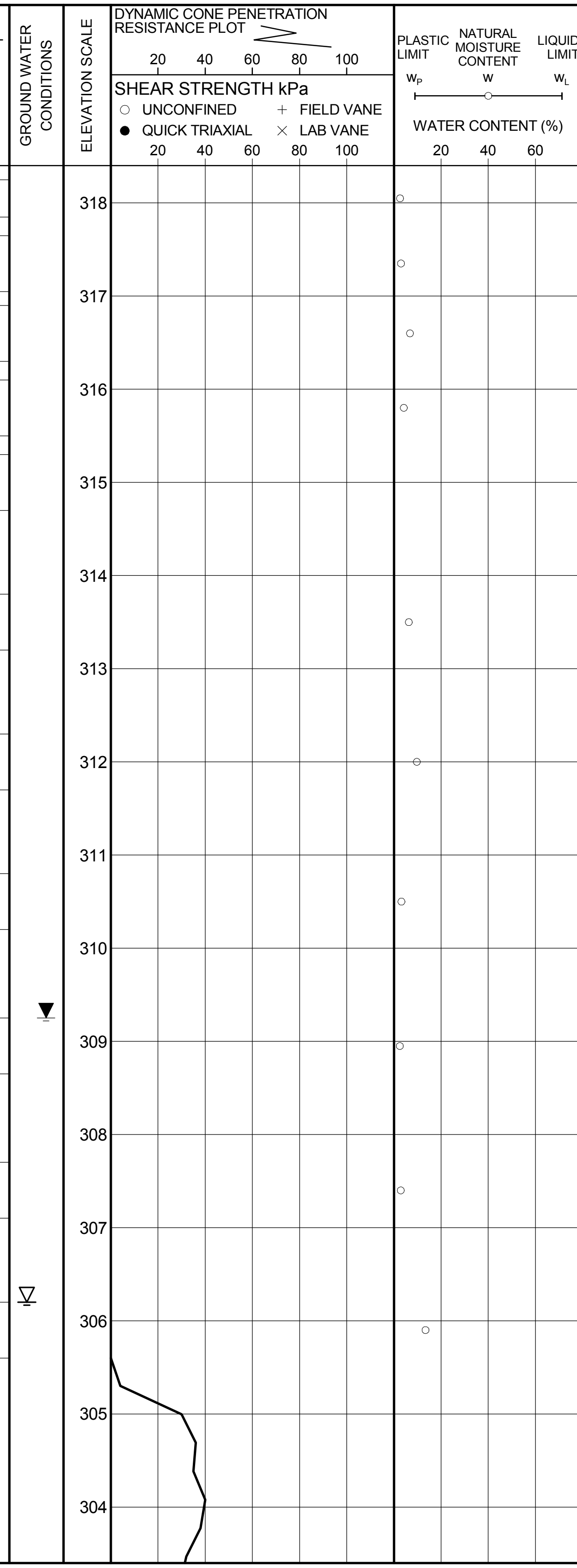
METRIC

G.W.P. 3224-15-00 LOCATION Coords: 4 812 456.1N; 251 897.2 E ORIGINATED BY S.A.

DIST Central HWY 6S BOREHOLE TYPE Hollow Stem Augers & Cone Penetration Test COMPILED BY A.K.

DATUM Geodetic DATE 2017.11.13 - 2017.11.14 LATITUDE 43.45014 LONGITUDE -80.15364 CHECKED BY M.V.

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
318.4	GROUND SURFACE				
0.0	100 mm ASPHALT over silty sand, with gravel (PAVEMENT FILL)	1	SS	95/28cm	
317.4 / 1.0	SILTY SAND, with gravel Compact to very dense, Brown, Damp	2	SS	57	
		3	SS	23	
	occasional cobbles	4	SS	19	
	(FILL)	5	SS	16	
313.8 / 4.6	SILTY SAND, with gravel Very dense to compact, Brown, Damp	6	SS	35	
		7	SS	6	
		8	SS	57	
		9	SS	55	
		10	SS	21	38 51 9 2
		11	SS	16	5 79 (16)
12.8	End of borehole				
	Probable SILTY SAND, with gravel				
303.4					



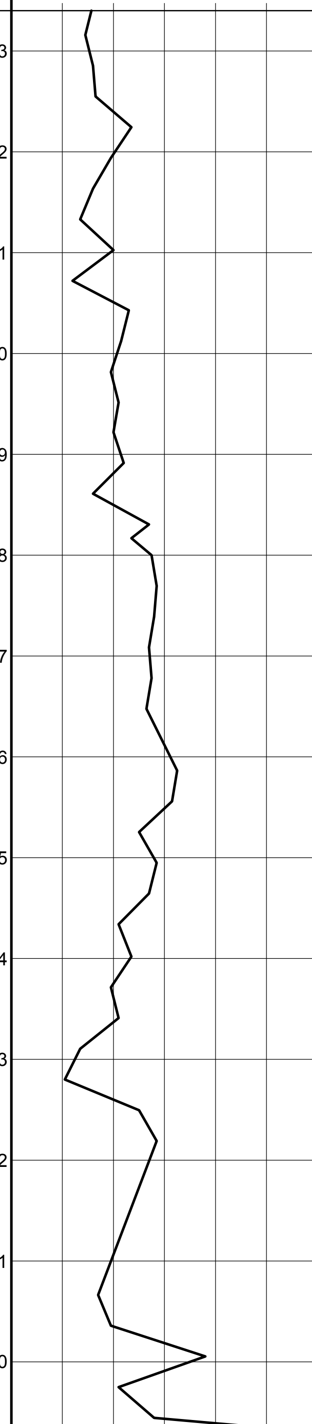
Continued Next Page

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

RECORD OF BOREHOLE No 35-351-02

METRIC

G.W.P. 3224-15-00	LOCATION Coords: 4 812 456.1N; 251 897.2 E		ORIGINATED BY S.A.
DIST Central HWY 6S	BOREHOLE TYPE Hollow Stem Augers & Cone Penetration Test		COMPILED BY A.K.
DATUM Geodetic	DATE 2017.11.13 - 2017.11.14	LATITUDE 43.45014 LONGITUDE -80.15364	CHECKED BY M.V.

SOIL PROFILE: ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: "N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE) 20 40 60 80 100	PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
303.4 15.0	End of borehole Probable SILTY SAND, with gravel *(Cont.d)*						303 302 301 300 299 298 297 296 295 294 293 292 291 290				
289.3 29.1	End of cone penetration test										

Continued Next Page

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

RECORD OF BOREHOLE No 35-351-02

METRIC

G.W.P. 3224-15-00 LOCATION Coords: 4 812 456.1N; 251 897.2 E ORIGINATED BY S.A.

DIST Central HWY 6S BOREHOLE TYPE Hollow Stem Augers & Cone Penetration Test COMPILED BY A.K.

DATUM Geodetic DATE 2017.11.13 - 2017.11.14 LATITUDE 43.45014 LONGITUDE -80.15364 CHECKED BY M.V.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
288.4											
	∇ Water level observed during drilling										.
	▼ Water level measured upon completion of drilling										.

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

RECORD OF BOREHOLE No 35-351-03

METRIC

G.W.P. 3224-15-00 | LOCATION Coords: 4 812 456.0 N; 251 866.4 E | ORIGINATED BY M.F.

DIST Central HWY 6S | BOREHOLE TYPE Hollow Stem Augers, Rotary Mud and NQ Coring | COMPILED BY A.K.

DATUM Geodetic | DATE 2017.11.06 - 2017.11.08 | LATITUDE 43.45014 | LONGITUDE -80.15402 | CHECKED BY M.V.

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

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

UNIT WEIGHT γ kN/m³

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

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	GR	SA	SI	CL
	GROUND SURFACE								
0.0	TOPSOIL:				311				
310.9 / 0.2	SILTY SAND, with gravel Compact to very dense, Brown, Moist	1	SS	21					
		2	SS	19	310				
		3	SS	29	309				
		4	SS	21					
		5	SS	31	308				
		6	SS	22	307				
		7	SS	26	306				
		8	SS	11	305	40	56	(4)	
					304				
		9	SS	29	303				
		10	SS	18	302				
					301				
		11	SS	17	300				
		12	SS	18	299				
					298				
		13	SS	30	297				
296.1									

Continued Next Page

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

RECORD OF BOREHOLE No 35-351-03

METRIC

G.W.P. 3224-15-00 LOCATION Coords: 4 812 456.0 N; 251 866.4 E ORIGINATED BY M.F.

DIST Central HWY 6S BOREHOLE TYPE Hollow Stem Augers, Rotary Mud and NQ Coring COMPILED BY A.K.

DATUM Geodetic DATE 2017.11.06 - 2017.11.08 LATITUDE 43.45014 LONGITUDE -80.15402 CHECKED BY M.V.

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
296.1 / 15.0	SILTY SAND, with gravel Compact to very dense, Brown, Moist *(Cont.d)*				296	
		14	SS	17		4 89 (7)
					295	
	occasional cobbles	15	SS	50	294	
					293	
		16	SS	12		21 42 29 8
					292	
					291	
					290	
		17	SS	113/25cm		
					289	
288.5 / 22.6	DOLOSTONE BEDROCK Slightly weathered	18	RC NQ	REC 78%	288	RQD 59%
	Unweathered	19	RC NQ	REC 100%	287	RQD 93% UCS: 47.6 Mpa
		20	RC NQ	REC 100%	286	RQD 97%
285.0 / 26.1	End of borehole				285	

Water level observed during drilling (Nov 7, 2017)

Monitoring Well Readings:

Date	Depth (m)	Elev.
Nov. 14/'17	4.7	306.4
Dec. 01/'17	4.6	306.5
Jan. 09/'18	4.8	306.3
Jan. 17/'18	4.6	306.4
Feb. 15/'18	4.6	306.5
Apr. 03/'18	4.5	306.6

Chart column headings: GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L, WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³

Continued Next Page

RECORD OF BOREHOLE No 35-351-03

METRIC

G.W.P. 3224-15-00 LOCATION Coords: 4 812 456.0 N; 251 866.4 E ORIGINATED BY M.F.

DIST Central HWY 6S BOREHOLE TYPE Hollow Stem Augers, Rotary Mud and NQ Coring COMPILED BY A.K.

DATUM Geodetic DATE 2017.11.06 - 2017.11.08 LATITUDE 43.45014 LONGITUDE -80.15402 CHECKED BY M.V.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH 281.1	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	Monitoring Well Legend: Bentonite seal Filter sand Screen										

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

Ministry of Transportation
Ontario

Foundation Design

RECORD OF BOREHOLE No 35-351-04

METRIC

G.W.P. 3224-15-00	LOCATION Coords: 4 812 414.8 N; 251 875.3E		ORIGINATED BY A.H.
DIST Central HWY 6S	BOREHOLE TYPE Hollow Stem Augers & Cone Penetration Test		COMPILED BY A.K.
DATUM Geodetic	DATE 2018.01.17	LATITUDE 43.44977 LONGITUDE -80.15391	CHECKED BY M.V.

ELEV / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
311.8	GROUND SURFACE					
0.0	180 mm ASPHALT over crushed stone silty sand, with gravel (PAVEMENT FILL)	1	SS	62		
311.0 / 0.8	SILTY SAND, with gravel Compact to dense, Brown, Moist	2	SS	35		41 48 (11)
		3	SS	8		
		4	SS	29		
		5	SS	41		
		6	SS	26		50 38 (12)
		7	SS	25		
					∇	
		8	SS	47		
		9	SS	24		
		10	SS	16		22 70 6 2
		11	SS	15		
		12	SS	18		
		13	SS	34		8 83 7 2
298.1 / 13.7	End of borehole Probable SILTY SAND, with gravel					
296.8						

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

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE: 20 40 60 80 100
PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%): 20 40 60
UNIT WEIGHT γ kN/m³

Continued Next Page

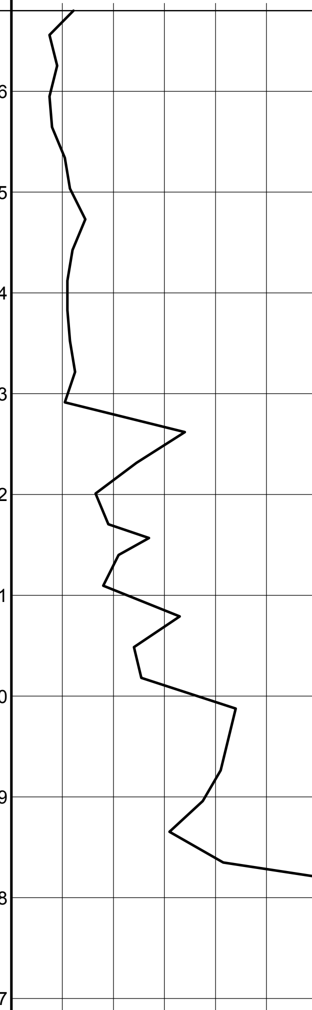
+3, ×3: Numbers refer to Sensitivity ○3% STRAIN AT FAILURE

RECORD OF BOREHOLE No 35-351-04

METRIC

G.W.P. 3224-15-00	LOCATION Coords: 4 812 414.8 N; 251 875.3E		ORIGINATED BY A.H.
DIST Central HWY 6S	BOREHOLE TYPE Hollow Stem Augers & Cone Penetration Test		COMPILED BY A.K.
DATUM Geodetic	DATE 2018.01.17	LATITUDE 43.44977 LONGITUDE -80.15391	CHECKED BY M.V.

SOIL PROFILE: ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: "N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE) (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (20 40 60)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
296.8 / 15.0	End of borehole										
	Probable SILTY SAND, with gravel *(Cont.d)*						296				
							295				
							294				
							293				
							292				
							291				
							290				
							289				
							288				
							287				
286.8 / 25.0	End of cone penetration test										
	∇ Water level observed during drilling										
	NOTE: Borehole caved-in at 5.48m										



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

RECORD OF BOREHOLE No 35-351-05

METRIC

G.W.P. 3224-15-00 LOCATION Coords: 4 812 420.2 N; 251 860.7 E ORIGINATED BY A.H.

DIST Central HWY 6S BOREHOLE TYPE Hollow Stem Augers and NQ Coring COMPILED BY A.K.

DATUM Geodetic DATE 2017.11.29 - 2017.12.06 LATITUDE 43.44982 LONGITUDE -80.15409 CHECKED BY M.V.

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
311.8	GROUND SURFACE								
0.0	180 mm ASPHALT over silty sand with gravel (PAVEMENT FILL)	1	SS	29					
311.0 0.8	SILTY SAND, with gravel Loose to dense, Brown, Moist	2	SS	20	311				
		3	SS	23	310				
		4	SS	28	309				
		5	SS	15		60	31	7	2
		6	SS	27	308				
		7	SS	25	307	52	40	6	2
					306				
		8	SS	20	305				
		9	SS	25	304				
					303				
		10	SS	16	302				
		11	SS	15	301				
					300				
		12	SS	19	299				
		13	SS	21	298				
296.8					297				

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m^3

Continued Next Page

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

RECORD OF BOREHOLE No 35-351-05

METRIC

G.W.P. 3224-15-00 LOCATION Coords: 4 812 420.2 N; 251 860.7 E ORIGINATED BY A.H.

DIST Central HWY 6S BOREHOLE TYPE Hollow Stem Augers and NQ Coring COMPILED BY A.K.

DATUM Geodetic DATE 2017.11.29 - 2017.12.06 LATITUDE 43.44982 LONGITUDE -80.15409 CHECKED BY M.V.

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
296.8	SILTY SAND, with gravel Loose to dense, Brown, Moist *(Cont.d)*						
		14	SS	9		○	70 27 (3)
					296		
		15	SS	22	295	○	
					294		
		16	SS	20	293	○	
		17	SS	44	292		
					291		
		18	SS	36	290	○	
					289		
					288		
		19	SS	105/15cm	287		
					286		
285.7 / 26.1	DOLESTONE BEDROCK Slightly weathered	20	RC NQ	REC 100%	285		RQD 91%
	Moderatyely weathered	21	RC NQ	REC 91%	284		RQD 87% UCS: 76.1 Mpa
	Moderately weathered	22	RC NQ	REC 78%	283		RQD 75%
282.4 / 29.4	End of borehole						

Column headings: SOIL PROFILE; SAMPLES; GROUND WATER CONDITIONS; STRAT PLOT; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE (20 40 60 80 100); PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³.

Continued Next Page

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

RECORD OF BOREHOLE No 35-351-05

METRIC

G.W.P. 3224-15-00 LOCATION Coords: 4 812 420.2 N; 251 860.7 E ORIGINATED BY A.H.

DIST Central HWY 6S BOREHOLE TYPE Hollow Stem Augers and NQ Coring COMPILED BY A.K.

DATUM Geodetic DATE 2017.11.29 - 2017.12.06 LATITUDE 43.44982 LONGITUDE -80.15409 CHECKED BY M.V.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE (20 40 60 80 100)	PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L / WATER CONTENT (%) (20 40 60)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
281.8											

NOTES:

1 Groundwater was not encountered during drilling

2 Borehole was charged with drilling water thus water level could not be established upon completion of drilling.

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

Ministry of Transportation
Ontario

Foundation Design

RECORD OF BOREHOLE No 35-351-06

METRIC

G.W.P. 3224-15-00
LOCATION Coords: 4 812 382.6 N; 251 872.3 E
ORIGINATED BY M.F.

DIST Central HWY 6S
BOREHOLE TYPE Hollow Stem Augers & Cone Penetration Test
COMPILED BY A.K.

DATUM Geodetic
DATE 2017.11.09
LATITUDE 43.44948
LONGITUDE -80.15394
CHECKED BY M.V.

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
310.2	GROUND SURFACE
0.0	TOPSOIL
309.9 / 0.3	SILTY SAND, with gravel Compact to dense, Brown, Wet
295.2	

SAMPLES

NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
1	SS	14				
2	SS	28				
3	SS	26				
4	SS	19	3	80	16	1
5	SS	17				
6	SS	28				
7	SS	29				
8	SS	41	45	44	10	1
9	SS	26				
10	SS	36				
11	SS	32				
12	SS	42				
13	SS	39				
14	SS	23				

GROUND WATER CONDITIONS: ∇ (at approx. elev. 306.5)

ELEVATION SCALE: 310, 309, 308, 307, 306, 305, 304, 303, 302, 301, 300, 299, 298, 297, 296

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L

WATER CONTENT (%): 20 40 60

UNIT WEIGHT γ kN/m^3

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

Continued Next Page

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

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No 35-351-06

METRIC

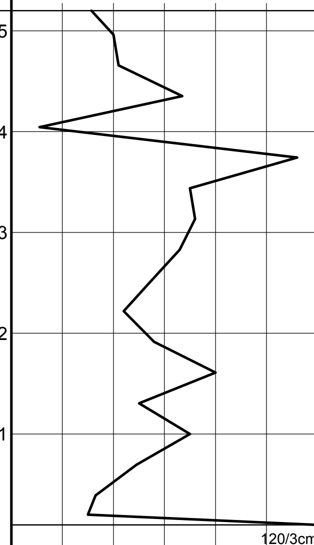
G.W.P. 3224-15-00 LOCATION Coords: 4 812 382.6 N; 251 872.3 E ORIGINATED BY M.F.

DIST Central HWY 6S BOREHOLE TYPE Hollow Stem Augers & Cone Penetration Test COMPILED BY A.K.

DATUM Geodetic DATE 2017.11.09 LATITUDE 43.44948 LONGITUDE -80.15394 CHECKED BY M.V.

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
295.2 / 15.0	SILTY SAND, with gravel Compact to dense, Brown, Wet *(Cont.d)*				295 294						
		15	SS	46		○		38	46	14	2
293.0 / 17.2	End of borehole Probable SILTY SAND, with gravel				293 292 291						
290.1 / 20.1	End of cone penetration test Refusal on probable bedrock										

DYNAMIC CONE PENETRATION RESISTANCE PLOT



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

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L

REMARKS & GRAIN SIZE DISTRIBUTION (%)

∇ Water level observed during drilling

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

RECORD OF BOREHOLE No 35-351-07

METRIC

G.W.P. 3224-15-00	LOCATION Coords: 4 812 376.5 N; 251 875.3 E		ORIGINATED BY M.F.
DIST Central HWY 6S	BOREHOLE TYPE Hollow Stem Augers		COMPILED BY A.K.
DATUM Geodetic	DATE 2017.10.31 - 2017.11.01	LATITUDE 43.44942 LONGITUDE -80.1539	CHECKED BY M.V.

SOIL PROFILE: ELEV / DEPTH, DESCRIPTION, STRAT PLOT
SAMPLES: NUMBER, TYPE, "N" VALUES
GROUND WATER CONDITIONS
ELEVATION SCALE
DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE: 20 40 60 80 100
PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%): 20 40 60
UNIT WEIGHT γ kN/m^3
REMARKS & GRAIN SIZE DISTRIBUTION (%): GR SA SI CL

ELEV DEPTH	DESCRIPTION
311.3	GROUND SURFACE
0.0	200mm Topsoil
311.1 / 0.2	SILTY SAND, some gravel Compact to very dense, Brown, Moist occasional cobbles (below approx. elev. 305.3)
296.3	

NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
1	SS	40				
2	SS	45				
3	SS	23				
4	SS	40				
5	SS	30	15	72	12	1
6	SS	27				
7	SS	18				
8	SS	14	7	88	(5)	
9	SS	83				
10	SS	54				
11	SS	40				
12	SS	31				
13	SS	51	52	39	8	1
14	SS	29				

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

Continued Next Page

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

RECORD OF BOREHOLE No 35-351-07

METRIC

G.W.P. 3224-15-00 | LOCATION Coords: 4 812 376.5 N; 251 875.3 E | ORIGINATED BY M.F.

DIST Central HWY 6S | BOREHOLE TYPE Hollow Stem Augers | COMPILED BY A.K.

DATUM Geodetic | DATE 2017.10.31 - 2017.11.01 LATITUDE 43.44942 LONGITUDE -80.1539 | CHECKED BY M.V.

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
296.3 / 15.0	SILTY SAND, some gravel Compact to very dense, Brown, Moist *(Cont.d)*				296						
		15	SS	32		○					
					295						
		16	SS	39	294	○					
		17	SS	38	293	○					
					292						
					291						
		18	SS	24	290	○		24	52	20	4
					289						
					288						
		19	SS	100/3cm	287	○					
					286						
		20	SS	100/3cm	285	○					
283.8		21	SS	100/3cm	284	○					
27.5	End of borehole ∇ Water level observed during drilling, (October 31, 2017)										

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60; REMARKS & GRAIN SIZE DISTRIBUTION (%)

Continued Next Page

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

RECORD OF BOREHOLE No 35-351-07

METRIC

G.W.P.	3224-15-00	LOCATION	Coords: 4 812 376.5 N; 251 875.3 E	ORIGINATED BY	M.F.
DIST	Central HWY 6S	BOREHOLE TYPE	Hollow Stem Augers	COMPILED BY	A.K.
DATUM	Geodetic	DATE	2017.10.31 - 2017.11.01 LATITUDE 43.44942 LONGITUDE -80.1539	CHECKED BY	M.V.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L / WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
281.3											

Monitoring Well Readings:

Date	Depth (m)	Elev.
Nov. 14/'17	5.6	305.7
Dec. 01/'17	5.75	305.6
Jan. 09/'18	5.6	305.7
Jan. 17/'18	5.4	305.9
Feb. 14/'18	5.2	306.1
Apr. 03/'18	5.1	306.2

Monitoring Well Legend:

- Bentonite seal
- Filter sand
- Screen

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

RECORD OF BOREHOLE No 35-351-08

METRIC

G.W.P. 3224-15-00 LOCATION Coords: 4 812 359.4 N; 251 877.6 E ORIGINATED BY M.F.

DIST Central HWY 6S BOREHOLE TYPE Hollow Stem Augers COMPILED BY A.K.

DATUM Geodetic DATE 2017.11.03 LATITUDE 43.44927 LONGITUDE -80.15387 CHECKED BY M.V.

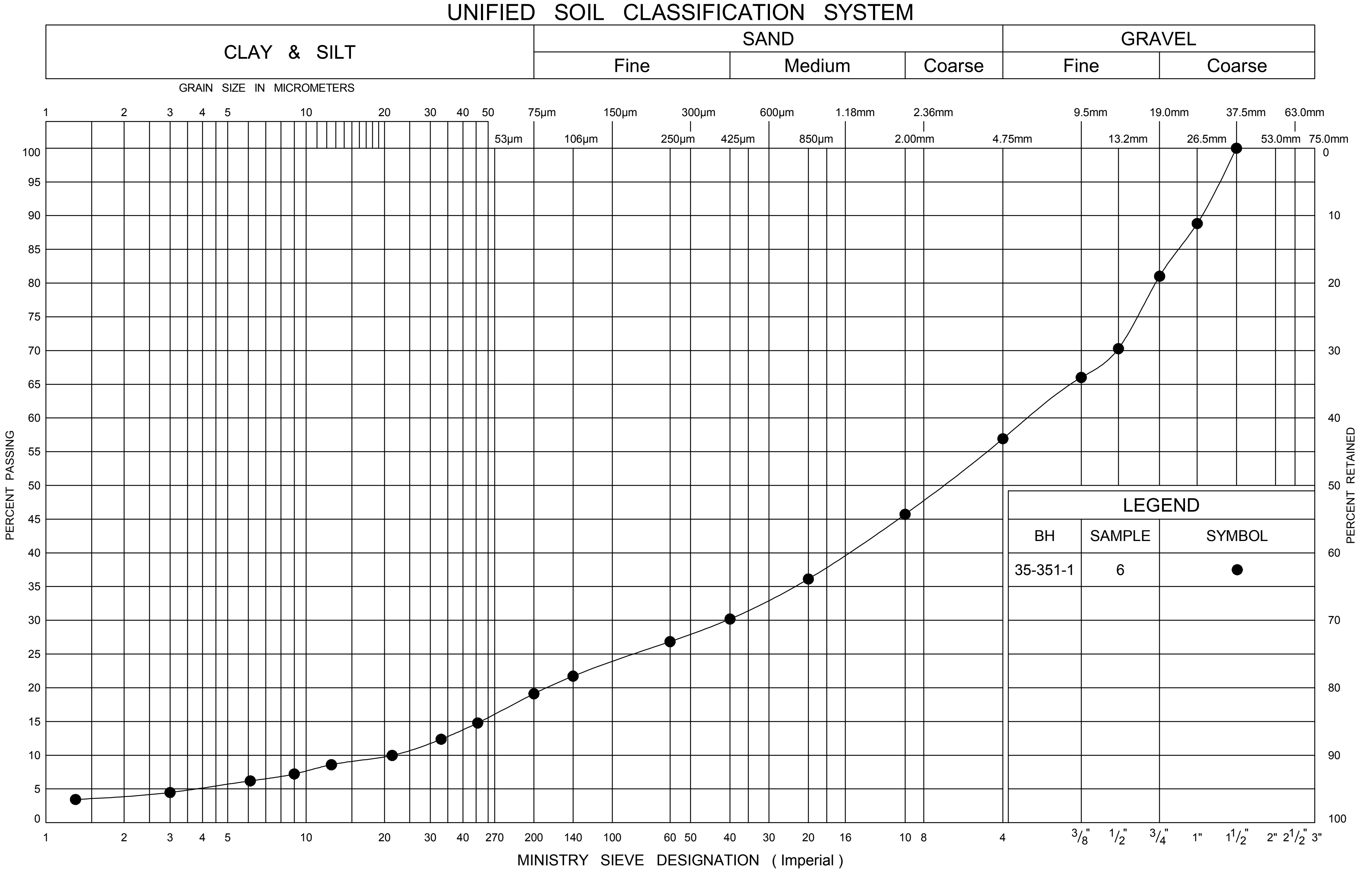
ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
313.1	GROUND SURFACE				313				
0.0	TOPSOIL								
312.8 / 0.3	SILTY SAND, with gravel Compact to dense, Brown, Moist	1	SS	9					
		2	SS	17	312				
		3	SS	29	311	50	38	11	1
	occasional cobbles	4	SS	72					
		5	SS	27	310				
		6	SS	35	309				
		7	SS	20	308	33	57	9	1
		8	SS	20	307				
					306				
		9	SS	38	305	43	44	12	1
		10	SS	17	304				
303.3 / 9.8	End of borehole								

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT W_L, WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m^3; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

NOTE: Groundwater was not encountered during and on completion of drilling

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

78 12 M



Ministry of Transportation Ontario	

GRAIN SIZE DISTRIBUTION

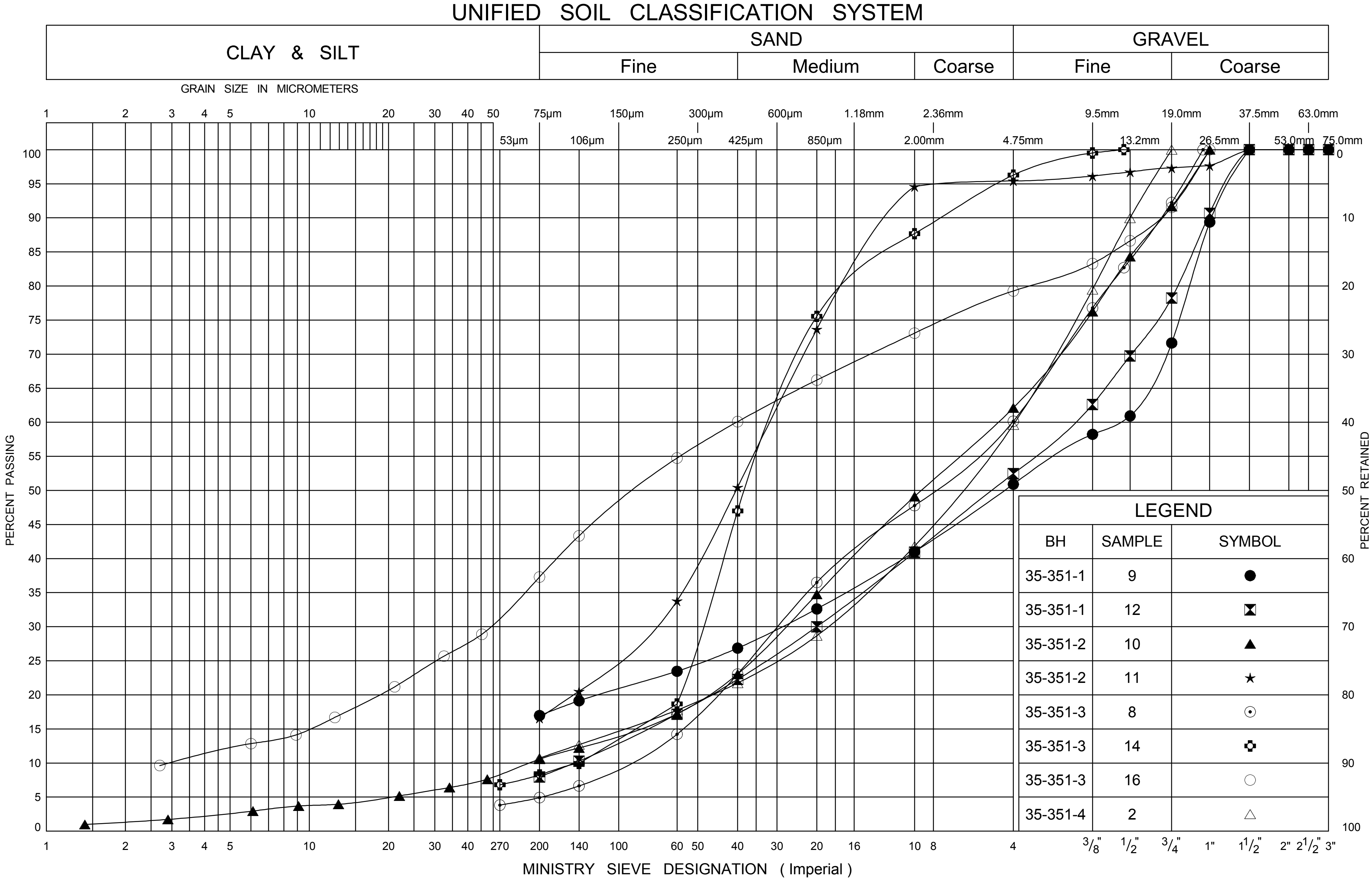
SILTY SAND, with gravel

FIG No: 351-GS-1

HWY: 401

W P: 3224-15-00

78 12 M



Ministry of Transportation
Ontario

GRAIN SIZE DISTRIBUTION

SILTY SAND, with gravel

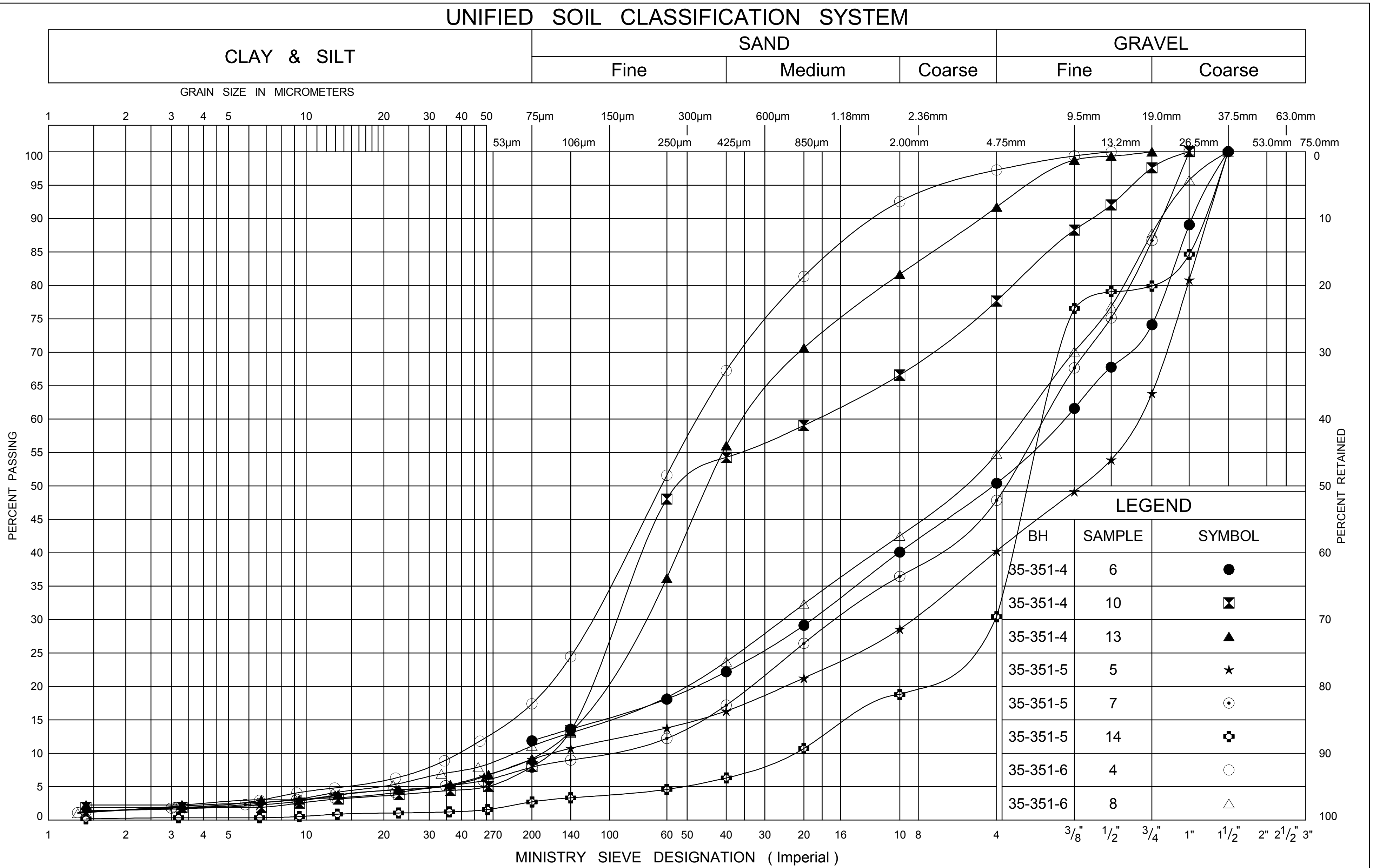
FIG No: 351-GS-2A

HWY: 401

W P: 3224-15-00

ONTARIO MOT GRAIN SIZE 17TF006A-LAB-MAR 7 2018.GPJ ONTARIO MOT.GDT 3/8/18

78 12 M



Ministry of Transportation

Ontario

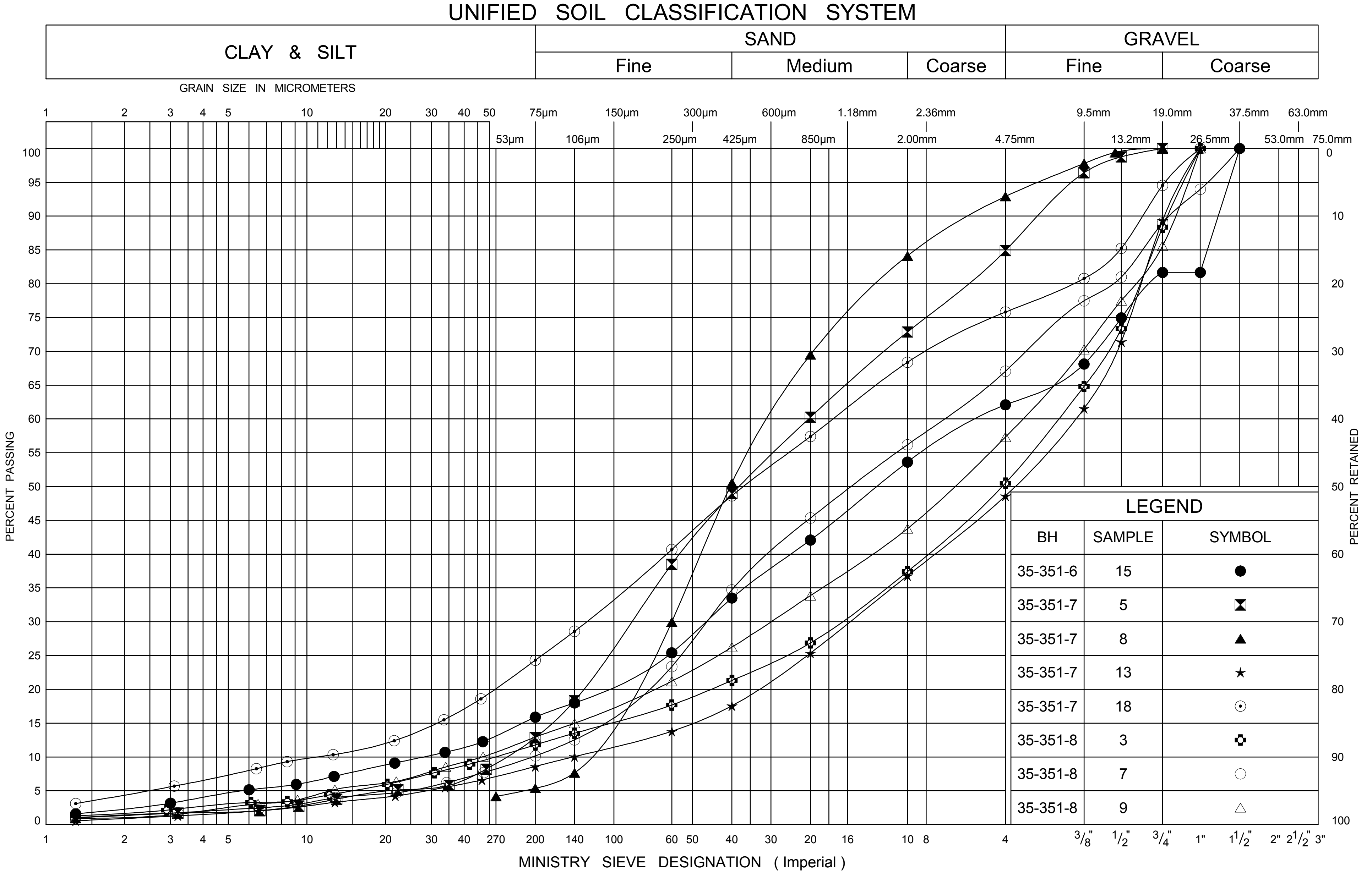
GRAIN SIZE DISTRIBUTION

SILTY SAND, with gravel

FIG No:	351-GS-2B
HWT:	401
W P:	3224-15-00

ONTARIO MOT GRAIN SIZE 17TF006A-LAB-MAR 7 2018.GPJ ONTARIO MOT.GDT 3/8/18

78 12 M



LEGEND		
BH	SAMPLE	SYMBOL
35-351-6	15	●
35-351-7	5	⧗
35-351-7	8	▲
35-351-7	13	★
35-351-7	18	⊙
35-351-8	3	✤
35-351-8	7	○
35-351-8	9	△

Ministry of Transportation, Ontario	GRAIN SIZE DISTRIBUTION SILTY SAND, with gravel	FIG No: 351-GS-2C HWY: 401 W P: 3224-15-00

ROCK CORE DESCRIPTION LOGS

BOREHOLE NO.	Sample No. Core Run	DEPTH (m)	% CR	% RQD	DESCRIPTION
35-351-03	18 (1)	22.6-23.16	78.5	59	**DOLOSTONE/DOLOMITE (GUELPH FORMATION):** medium brown, fine crystalline sucrosic (sugary) texture, hard, thick bedded, slightly weathered, unfractured to slightly fractured with bitumen stainging along bedding plane. Common mechanical core breaks. Vugs/voids: Irregular sizes & shapes filled with fine grain calcite from 2 mm to 10 mm. Diagenesis features: voids/vugs, dolomitization.
	19 (2)	23.16-24.62	100	92	Medium brown, fine crystalline dolostone, sucrosic (sugary) texture, thick bedded, massive, hard, unweathered to slightly weathered with irregular vugs/voids, slightly fractured. Minerals: dolomite, calcite, containing organic material, mud/silt. Diagenesis features: vugs/voids infilled calcite crystals/grains/dolomitization. Mechanical core breaks.
	20 (3)	23.16-26.15	100	97	Medium brown, fine crystalline sucrosic texture, thick bedded, hard, slightly to moderately weathered with various irregular size & shapes of vugs from 25.96 m to 26.09 m, slightly fractured. Minerals: dolomite, calcite, containing organic/bitumen material, mud/silt. Mechanical breaks

CR* - Core Recovery
RQD* - Rock Quality Designation

Logged by: S. Siddiqi, P.Geo.

ROCK CORE DESCRIPTION LOGS

BOREHOLE NO.	Sample No. Core Run	DEPTH (m)	% CR	% RQD	DESCRIPTION
35-351-05	21 (1)	26.15-27.06	100	91	DOLOSTONE/DOLOMITE (GUELPH FORMATION)**:** medium brown to light grey, fine crystalline sucrosic (sugary) texture, thick bedded, hard, vugs/porous zones contains coarse grained calcite, slightly weathered, slightly fractured with occasional bitumen staining along bedding plane. Minerals: dolomite, calcite, containing organic material, mud/silt. diagenesis features: vugs infilled calcite crystals/grains. Natural lateral/ horizontal joints parallel to bedding plane at 26.15 m, 26.51 m & 26.85 m.
	22 (2)	27.06-28.6	91	87	Medium brown to light grey, fine crystalline sucrosic texture, thick bedded, hard, moderately weathered, various irregular size and shapes of vugs infilled calcite crystals/grains with mud. highly fractured associated with mechanical breaks.
	23 (3)	28.6-29.2	78	75	Medium grey, fine crystalline sucrosic texture, thick bedded, hard, moderately weathered, large vugs/voids infilled coarse grain dolomite/calcite crystals/grains and fine mud, slightly fractured along lateral joint bedding plane with bitumen staining. 5 mm to 20 mm irregular vugs/voids from 28.62 m - 28.64 m & 28.83 m

CR* - Core Recovery

RQD* - Rock Quality Designation

Logged by: S. Siddiqi, P.Geo.

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

Certificate of Analysis

AGAT WORK ORDER: 17T294823

PROJECT: 17TF006A-hwy 401/hwy6

CLIENT NAME: PETO MACCALLUM LIMITED

ATTENTION TO: Lul Yimam

SAMPLING SITE:

SAMPLED BY:

Corrosivity Package

DATE RECEIVED: 2017-12-13

DATE REPORTED: 2017-12-28

			SAMPLE DESCRIPTION:	35-611-5 SS6A	35-612-4 SS5	35-351-6 SS5
			SAMPLE TYPE:	Soil	Soil	Soil
			DATE SAMPLED:	2017-10-10	2017-10-25	2017-11-09
Parameter	**Unit**	**G / S**	**RDL**	**8975159**	**8975162**	**8975163**
Sulfide (S2-)	%		0.05	<0.05	<0.05	<0.05
Chloride (2:1)	µg/g		2	79	30	28
Sulphate (2:1)	µg/g		2	5	6	8
pH (2:1)	pH Units		NA	8.42	8.73	9.36
Electrical Conductivity (2:1)	mS/cm		0.005	0.196	0.137	0.131
Resistivity (2:1)	ohm.cm		1	5100	7300	7630
Redox Potential (2:1)	mV		5	189	180	158

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

8975159-8975163 EC, pH, Chloride, Sulphate and Redox Potential were determined on the extract obtained from the 2:1 leaching procedure (2 parts DI water: 1 part soil).

*Sulphide analyzed at AGAT 5623 McAdam

Certified By: Nivine Basily

PART B –FOUNDATION DESIGN REPORT

for

REPLACEMENT OF UNDERPASS AT HIGHWAY 401 AND CONCESSION ROAD 7
HIGHWAY 401, SITE NO. 35-351, STATION 21+042
PUSLINCH TOWNSHIP, COUNTY OF WELLINGTON
LATITUDE: 43.44977; LONGITUDE: -80.15391
ASSIGNMENT NO. 3014-E-0014
GWP 3224-15-00

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

Distribution:
1 cc: AECOM for distribution to MTO
Project Manager + One (1) Digital Copy (PDF)
1 cc: AECOM for distribution to MTO
Foundations Section
+ One (1) Digital Copy (PDF)
1 cc: AECOM + One (1) Digital Copy (PDF)
1 cc: PML Toronto

PML Ref.: 17TF006A
Index No.: 023FDR
GEOCRES No.: 40P8-253
June 29, 2018

TABLE OF CONTENTS

PART B - FOUNDATION DESIGN REPORT

PART B - FOUNDATION DESIGN REPORT
for
Replacement of Underpass at Highway 401 and Concession Road 7
Highway 401, Site No. 35-351, Station 21+042
Puslinch Township, County Of Wellington
Latitude: 43.44977; Longitude: -80.15391
Assignment No. 3014-E-0014, GWP 3224-15-00

7. INTRODUCTION

This foundation investigation and design report with the interpretation and recommendations are intended for the use of AECOM Canada Ltd. (AECOM) on behalf of the Ministry of Transportation and shall not be used or relied upon for any other purposes or by any other parties including the contractor. The contractors must make their own interpretation based on the factual data in Part A of the report. 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 foundation design recommendations based on interpretation of the geotechnical data presented in the factual report (Part A). This report is to assist the design team in the selection of a suitable type of foundation for the proposed replacement bridge located at the crossing of Highway 401 and Concession Road 7 at the approximate Sta. 21+042 (assumed by AECOM) in the Township of Puslinch, County of Wellington, Ontario.

The discussions and recommendations are based on interpretation of the geotechnical data presented in the factual report (Part A) and a review of relevant existing reports and borehole logs obtained from the MTO GEOCRES Library.

8.2 Existing Bridge

Based on the Foundation Investigation and Design Report, dated September 17, 1958 (Geocres No. 40P08-013), the existing underpass at the crossing of Concession Road 7 and Highway 401 is a two-span structure with a center pier and the road accommodates two lanes of vehicular traffic. The spans of the bridge on the westbound and eastbound lanes are approximately 16.8 m and 18.3 m, respectively. The abutments and the center pier are supported on spread footings placed below El. 312.4 and the footings were designed assuming an allowable bearing resistance (SLS) of 430 kPa (4 tons/sq. ft.).

Based on the observations made during the site investigation, no major sign of structural distress of the bridge due to settlement was observed other than the delamination of concrete along the deck and reinforcements are exposed at several locations. The approach embankments with a slope of about 2H:1V appear in good condition and no surface erosion is visible.

8.3 Proposed Bridge

Based on the final General Arrangement (GA) drawing received from AECOM on June 13, 2018, the centerline of the proposed underpass will be located approximately 22.0 m west of the centerline of the existing Concession Road 7 Bridge. The proposed bridge will be a two span, each 39.5 m long, structure supported on integral abutments and a center pier located at the median.

The GA drawing indicates that the center pier will consists of two columns, which are proposed to be supported on caissons, located at a center to center spacing of 6.6 m. Further the drawing indicates that the steel H-piles for the abutments will be lowered in pre-augered holes supported with 600 mm diameter corrugated steel pipes (CSP) and backfilled with loose sand. The drawing also indicates that the cut-off elevations of the piles to support the north and south abutments are proposed to be at El. 314.5 and El. 315.6, respectively. The caissons to support the columns will be extended to El. 310.9. Based on the information provided by AECOM on May 7, 2018, each column at the center pier will transfer a factored load of about 8,100 KN at the founding level of the footing or on top of the caissons. In view of space limitations at the median of Highway 401, a pile cap will not be constructed and each of the caisson will support a column that will extend to a bent under the bridge deck.

The new underpass will accommodate six traffic lanes and shoulders on each direction of Highway 401. The replacement structure will accommodate two 3.5 m wide lanes and a 2.0 m wide shoulder on Concession Road 7. The approach slabs at both abutments will be 6.0 m long and the embankments will be about 7.0 m high. The design grade of the approach at both abutments will be set at about El. 320.0.

8.4 Structure Foundation

Since the input for the consequence classification was not specified by the Regulatory Agency it was assumed for this report that the consequence level is a typical level and the consequence level of 1.0 was used to estimate the ULS and SLS resistances. The site for the proposed structure is located in an area where information from the previous investigations and performance of the existing structures are available. In view of this, a "Typical Understanding" of prediction model as identified in Clause 6.5.3.2 (b) was used for this site.

In summary, the subsurface conditions immediately below the existing ground level consist of 800 mm to 1.0 m pavement structure in the paved area and 200 mm to 300 mm of topsoil where boreholes were advanced near the toe of the embankment. The topsoil and pavement structure are underlain by compact to dense silty sand with gravel and occasional cobbles to a maximum depth of EL. 283.5 and the silty sand is followed by dolostone bedrock.

Based on the GA drawing, the abutments of the proposed underpass are to be supported on steel H-piles and the columns for the center pier are to be supported on caissons. For comparison purposes the following Table 8.4 provides the advantages, disadvantages, risks and consequences of the foundation alternatives to support the proposed structure.

Table 8.4: Comparison of Foundation Types

FOUNDATION TYPE	ADVANTAGES	DISADVANTAGES	RISKS / CONSEQUENCES
Driven piles	• High geotechnical resistance available • Allows for integral abutment design • Ability to drive through cobbles or dense gravel • Does not require deep excavation	• Higher cost compared to footings • Vibration induced during driving • May require pile tip reinforcement • Individual piles may encounter refusal at varying depths • Limitations on location of pile splicing	• Piles may hung-up at varying elevations • Possible pile tip damage if piles are not adequately protected while driving to bedrock
Spread footings	• Ease of construction • No dewatering or deep excavation is required • Less cost compared to deep foundations • Adequate bearing resistance available at reasonably shallow depth	• Lower bearing resistance than for driven piles or caissons • May require shoring or roadway protection for excavation	• Immediate settlements due to elastic compression may be expected • Limited support for increase in loading
Caissons	• Higher bearing resistance available for caissons founded in bedrock • Space limitations at the median for pier construction may be addressed • can be readily extended with special adaptation to support the deck	• High cost relative to footings • Require temporary lining • Construction procedures may influence the integrity and performance of the caisson • Concrete in shaft liable to squeezing or necking where conventional type of construction is used	• Potential for necking of concrete while withdrawing temporary liner • May require caisson integrity testing for potential necking of concrete

8.4.1 Option 1: Steel H-Piles

Based on the subsoil conditions encountered at this site, it is recommended that both abutments and center pier are supported on 310 x 110 steel H-piles. The steel H-piles will be driven to bedrock at the center pier and north abutment and may be designed assuming a factored axial geotechnical resistance of 2,000 kN at Ultimate Limit State (ULS). Piles for the south abutment driven into very dense silty sand to about El. 284.5 and may be designed assuming factored axial geotechnical resistance of 1,600 kN at ULS. Geotechnical resistance at Serviceability Limit State (SLS) will not govern because the loads required to produce detrimental deformation are anticipated to be larger than the factored resistance at ULS recommended.

The table below summarizes the approximate pile tip elevations and the length of piles from the proposed cut-off indicated on the GA drawing and summarized in Section 8.3.

Table 8.4.1 Pile Tip Elevation and Length for HP 310 x 110

LOCATION	APPROXIMATE PILE TIP ELEVATION AND LENGTH	
	ELEVATION	LENGTH (m)
North Abutment	288.5 ± 1.0	26.0 ± 1.0
Center Pier	285.5 ± 1.0	25.5 ± 1.0
South Abutment	284.5 ± 1.5	31.0 ± 1.5

The driven pile installations should follow the OPSS.PROV 903 and section 8.4.1.2 in this report.

As indicated on Table 8.4.1, an allowance should be made in the contract to allow for local variations and pile tip penetration.

8.4.1.1 Horizontal Subgrade Reaction for Piles

The coefficient of horizontal subgrade reaction, k_s (kN/m^3), provided in the table below 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

Table 8.4.1.1: Coefficient n_h Values for Computation of Horizontal Subgrade Reaction

LOCATION	SOIL TYPE	ELEVATIONS (m)		n_h values (kN/m^3)
		FROM	TO	
North Abutment	Loose Sand	315.5 ±	312.5 ±	1,000
	Compact to Dense Sand	312.5 ±	305.6 ±	13,600
		305.6 ±	288.5 ±	8,450
South Abutment	Loose Sand	314.5 ±	311.5 ±	1,000
	Compact to Dense Sand	311.5 ±	305.6 ±	13,600
		305.6 ±	284.5 ±	8,450
Center Pier	Compact to Dense Sand	311.0 ±	305.6 ±	13,600
		305.6 ±	285.5 ±	8,450

Piles for the proposed integral abutments may be lowered in pre-augered holes to the depth of contraflexture point and driven to the pile tip elevations recommended. The annular space to the contraflexture should be backfilled with uniformly graded loose sand as required by MTO Report SO-96-01. Consideration should be given to MTO report SO-96-01 (Integral Abutment Bridges) for design guidelines.

8.4.1.2 Driven Pile Installation Notes

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

Piles driven to refusal on bedrock for the north abutment will derive their resistance from tip resistance and will typically achieve their design resistance at end of driving. The same assumption would apply for the centre pier if the pile foundation is selected. For the south abutment piles will be driven to a very dense sand deposit and a combination of shaft friction and tip resistance will be required to achieve the design resistance. In view of past experience with current pile driving equipment, it is recommended that the piles be driven to the recommended tip elevation and a soil set-up period of a minimum of 5 days be allowed before the resistance is checked using a dynamic test such as the Hiley Formula. The test piles should not be disturbed by driving adjacent piles, re-tapping or re-driving during the set-up time period. An NSSP was prepared for to instruct the Contractor and Contract Administration of this requirement.

Considering the occasional cobbles encountered below about E. 310.9 to El. 294.4, the pile tips should be reinforced to drive the piles through cobbles and to avoid damage to the tip of the piles driven to bedrock. Oversized driving shoes similar to Ontario Provincial Standard Drawing (OPSD 3000.100, Foundation Piles Steel H-Pile Driving Shoe) or Titus H bearing pile point are not recommended. These types of pile tip reinforcement may reduce the shaft friction and may lead to overruns, especially when the pile capacity is partly derived from shaft friction. The pile tip reinforcement shown on the attached Figure PML-1 is recommended.

8.4.2 Option 2: Shallow Foundation – Strip Footings

Alternatively, the proposed abutments may be supported on spread footings placed at or below El. 310.0. The following geotechnical resistances may be assumed for 2.5 m wide strip footings placed at or below El. 310.0.

Factored Geotechnical Bearing Resistance at ULS = 650 kPa

Geotechnical Bearing Resistance at SLS = 400 kPa

The bearing resistance for inclined loads should be reduced in accordance with the requirements of clause 6.10.4 of the CHBDC (2014).

The total settlement under a Serviceability Limit State (SLS) load of 400 kPa is expected to be in the order of 20 mm and the associated differential settlement may be expected to be in the order of 15 mm. Most of the total settlement estimated is expected to result from elastic compression of the subgrade and completed shortly after completion of road construction. Continuing total or differential settlements under the weight of the structure may be negligible.

The silty sand subgrade will be susceptible to disturbance from construction traffic and any ponded water. In order to limit the degradation of the founding soil, it is suggested that a concrete working slab (lean concrete) be placed on the subgrade within four hours after preparation, inspection and approval of the footing subgrade. This requirement should be addressed with a Non-Standard Special Provision (NSSP) or with a note on the structural drawing for foundations.

The sliding resistance of footings against lateral loads between the concrete footing and subgrade should be calculated in accordance with Section 6.10.5 of the CHBDC (2014). For cast-in-place concrete footing constructed on concrete working slab or on top of silty sand subgrade, the coefficient of friction tan δ should be taken as follows:

Cast-In-Place footing on smooth concrete working slab: tan δ = 0.55

Cast-In-Place concrete working slab on compact to dense silty sand: tan δ = 0.55

Considering the depth of groundwater level (El. 306.5) at this site, no major dewatering problems are anticipated for footings placed at the recommended elevation of El. 310.0.

8.4.3 Option 3: Caissons

The proposed center pier of the underpass may be supported on approximately 29 m to 31 m long caissons founded at about EL. 282.0 to El. 280.0 in dolostone bedrock as indicated in Table 8.4.3 below. The caissons socketed three times the diameter into the bedrock may be designed assuming the following geotechnical resistances:

Table 8.4.3: Caisson Tip Elevation and Geotechnical Resistance

DIAMETER (mm)	CAISSON TIP ELEVATION (m)	FACTORED GEOTECHNICAL RESISTANCE AT ULS (kN)
1200	282.0 ± 0.5	5,000
1500	281.0 ± 0.5	7,500
1800	280.0 ± 0.5	10,000

The construction of the deep foundation should conform to OPSS.PROV 903. The installation of caissons through the silty sand deposit and approximately 24.5 m to 26.5 m of groundwater will require the use of temporary steel liners extending to the bedrock to preserve the structural integrity of the caissons.

8.4.3.1 Lateral Resistances for Caissons

Lateral resistances at ULS and at SLS provided in Table 8.4.3.1a may be utilized for the design of the caissons socketed at least three times the diameter of the caisson or founded at the tip elevations provided in Table 8.4.3.

Table 8.4.3.1a: Lateral Resistance of Caissons

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

In case the lateral resistance recommended above is not adequate, it is recommended that the caissons be socketed at least four times the diameter into the bedrock and designed assuming the lateral resistance provided in Table 8.4.3.1b below. If required, a detail analysis may be carried out to determine the lateral resistance, using the horizontal subgrade reaction values provided in Table 8.4.1.1 and assuming a coefficient of horizontal subgrade reaction, n_h, value of 25 MN/m^3 for bedrock.

Table 8.4.3.1b: Lateral Resistance of Caisson

CAISSON DIAMETER (mm)	LATERAL RESISTANCE	
	AT ULS (kN)	AT SLS (kN)
1200	450	180
1500	650	260
1800	900	360

8.4.3.2 Caisson Installation Notes

Caissons should be installed in accordance with OPSS.PROV 903. The Contractor should select the installation procedure based on the groundwater conditions and on the cohesionless type of soil cover at this site, as well as the limited space available for construction at the centre median.

An NSSP was prepared to advise the contractor that temporary steel liners will need to be used to advance to the caisson founding levels. If vibration is utilized to advance and extract the liners, care should be taken to avoid damage to the existing bridge foundations from the vibrations.

In addition, the inspection of the caisson base and rock socket will need to be carried out using a shaft inspection device.

8.4.4 Recommended Option

The proposed structural arrangements for integral abutments require that the bridge be supported on steel H-piles. With this in view, driven steel H-piles are the recommended option for both abutments.

Considering the ease of construction and from a geotechnical perspective based on the subsoil conditions, shallow foundation (strip footing) is the recommended option for supporting the center pier.

Alternatively, the installation of caissons founded in bedrock is feasible and recommended to address the space limitations at the median of the Highway 401 where a pile cap is not readily feasible.

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). The earth pressure calculation should include maximum water level expected in the creek. The lateral earth and water pressure, p (kPa), may be computed using the equivalent fluid pressures presented in Section 6.12 of the CHBDC 2014 or employing the following equation assuming a triangular pressure distribution.

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

Where, P = lateral earth pressure (kPa)
K = lateral earth pressure coefficient
γ = unit weight of backfill material above assumed water level (kN/m^3)
γ' = unit weight of submerged backfill (γ - γ_w) material below assumed water level (kN/m^3)
γ_w = unit weight of water (9.8 kN/m^3)
h_1 = depth below final grade (m), above assumed water level
h_2 = depth below assumed water level (m)
q = surcharge load (kPa)
C_p = compaction pressure (refer to clause 6.12.3 of CHBDC 2014)
C_s = earth pressure induced by seismic events, kPa (refer to clause 4.6.5 of CHBDC 2014)

Where Ø = angle of internal friction of retained soil
δ = angle of friction between soil and wall

The seismic site coefficient for the conditions at this site is provided in Section 14 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: Earth Pressure Coefficients

GEOTECHNICAL PARAMETER	OPSS Granular ‘A’ or ‘B’ Type II	OPSS Granular ‘B’ Type I
Internal Friction Angle, (degrees)	35	30
Unit weight, γ (kN/m^3)	22.5± 0.3	21.5 ± 0.3
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

The coefficient of earth pressure “at rest” should be used for design of rigid and unyielding walls where sufficient movement of the structure wall is not permitted. For unrestrained structures, the active earth pressure coefficient should be employed.

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 EMBANKMENTS

The height of the proposed approach embankments is expected to be about 7.0 m above the grade of Highway 401 and is expected to impose a load of about 140 kPa to 150 kPa at the existing subgrade level, assuming compacted density of fill in the range of 20 kN/m^3 to 22 kN/m^3. Considering the typically cohesionless and compact to very dense subsoil conditions at this site, or major instability problems are anticipated for the embankments constructed with 2H:1V side slope of flatter. Any spongy or soft area observed within the base of the embankment should be removed before placing the fill. The new embankment should be placed by benching into the existing embankments in accordance with OPSD 208.01.

Elastic compression of the subgrade soil was estimated assuming a modulus value of 40 MPa to 50 MPa and a Poisson ratio of 0.4 for the embankment with a base width of 24.0 m. Based on the estimation, the proposed embankment may be expected to induce settlement in the order of 15 mm to 25 mm of the founding soil. In addition, the fill itself may be expected to settle by 0.5% to 1.0% (35 mm to 70 mm) of the fill height, depending on the type of fill material and placement. However, majority of the settlement will be in the form of elastic compression and will be completed shortly after completion of construction. The paving of the road should be delayed by four to six weeks after placement of fill to the designed grade of the embankment to mitigate long term differential settlements.

11. EXCAVATIONS, BACKFILL AND EROSION CONTROL

It is envisaged that no major excavations will be required at this site. However, any excavation should conform to the Occupational Health and Safety Act (OHSA) considering that the existing fills are classified as Type 3 soils as well as the upper layers of the typically cohesionless native soils.

The backfill behind the abutments or retaining walls if incorporated should be in conformance with OPSD 3101.150 and OPSD 3121.150 and the accepted proprietary design of the RSS, if applicable.

Backfilling adjacent to the abutments and possible retaining walls at the site should be carried out with conformance to OPSS.PROV 501. Operation of compaction equipment at the retaining structures should be restricted to limit the compaction pressure noted in clause 6.12.3 of the CHBDC. Refer to OPSS.PROV 501 for additional information in this regard.

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

12. ROADWAY PROTECTION SYSTEM

Design parameters based on the information obtained from Borehole 35-351-02 are provided below subject to verification of the parameters prior to the installation of a roadway protection system if required at the south abutment approach embankment.

The subsoil conditions encountered at this site is favourable for driving sheet piles to design and construct a shoring system to maintain traffic on Concession Road 7. A shoring system consisting of sheet pile wall with tie-backs may be feasible. The Contractor should be responsible for selection, design and performance of the temporary roadway protection scheme.

Temporary roadway protection should be designed to meet a Performance Level of 2 and constructed in accordance with OPSS.PROV 539 (Temporary Protection Systems). The following soil parameters are recommended for the design of the roadway protection system.

Table 12 - Soil Parameters

Elevation		SOIL TYPE	SOIL PARAMETERS		
From	To		FRICTION ANGLE (Ø°)	UNIT WEIGHT (Ɣ) KN/m^3	C_U, KN/m^2
318.4	313.8	Compact to Dense Silty Sand (Fill)	30	19	0
313.8	305.6	Compact to Dense Silty Sand	34	20	0

It is expected that the existing bridge will be removed once the construction of new bridge is completed. There is no foundation design requirement to leave the roadway protection system in place, once the construction of bridge is completed. It may be removed as specified in Section 539.07.02 of OPSS.PROV 539. The method and sequence of removal should not cause any damage to the new work, existing work, and facility being protected. This requirement may be addressed by a note on the construction staging drawing.

13. FOUNDATION FROST DEPTH

In accordance with OPSD 3090.101, a minimum of 1.2 m earth cover is required to protect against the frost penetration in the area where the site is located.

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

14. SEISMIC CONSIDERATIONS

The Spectral and Peak Ground Accelerations (Sa (0.2) and PGA) for the project site, based on the Town of Guelph, Ontario, is 0.019 and 0.067, respectively (National Building Code of Canada, 2015). The soil at the site for seismic design purposes is classified as Type C in accordance with Clause 4.4.3.2 of CHBDC, 2014.

15. GROUNDWATER CONTROL

Groundwater table at the time of field investigation was encountered between approximately El. 305.2 and El. 309.3. However, the stabilised groundwater level was recorded at EL. 305.6, one month after the installation of monitoring well near the proposed south abutment. The groundwater levels may fluctuate due to the influence of precipitation and seasonal changes.

It is considered that seepage from soil fissures or surface water run-off that enters the excavations can be handled by conventional sump pumping techniques. The groundwater level should be lowered a minimum of 0.5 m below the base of excavation. The groundwater levels at the site are subject to seasonal fluctuations and precipitation patterns.

16. SOIL CORROSION

One sample from the silty sand, with gravel deposit was tested for soil corrosivity and potential exposure of concrete to sulphate attack. A summary of the chemical test results is provided in Appendix A of this report. The sulphate concentration of 8 µg/g (0.0008%) reported in Table 5-4 for the silty sand, with gravel is far too low compared to the value of 0.1% suggested in Canadian Standard A23.1-14 to have any effect on buried concrete structures. Therefore, the potential for sulphate attack will be mild or relatively low. The chloride content of 0.0028% (28 µg/g) reported in Appendix A is significantly lower than the concentration value of 250 ppm (0.025%) that generally leads to the corrosive environment for buried metals. The potential for the corrosive environment at this site is relatively low.

Electrical resistivity less than 2000 ohm-cm generally leads to the highly corrosive environment for steel elements in contact with soil. The resistivity value of 7630 ohm-cm reported is significantly higher than 2000 and suggests a moderately or non-corrosive environment at this site for steel elements. However, the reported pH value of 9.36 is slightly higher than the value of 5.5 that generally leads to corrosion.

Generally, no sulphate attack is expected from selected backfill materials. However, it may be advisable to test backfill material for corrosion potential if the material is imported from unknown sources.

17. CLOSURE

This Foundation Investigation and Design Report was prepared by Ms. Asieh Khadem, M.Sc. Eng., EIT., Project Supervisor, and Nazibur Rahman and reviewed by Mr. Mark Vasavithasan, MSc. Eng., P.Eng. Senior Engineer, Geotechnical Services. Mr. C.M.P. Nascimento, P. Eng., Principal Consultant, conducted an independent review of the report.

Yours very truly

Peto MacCallum Ltd.

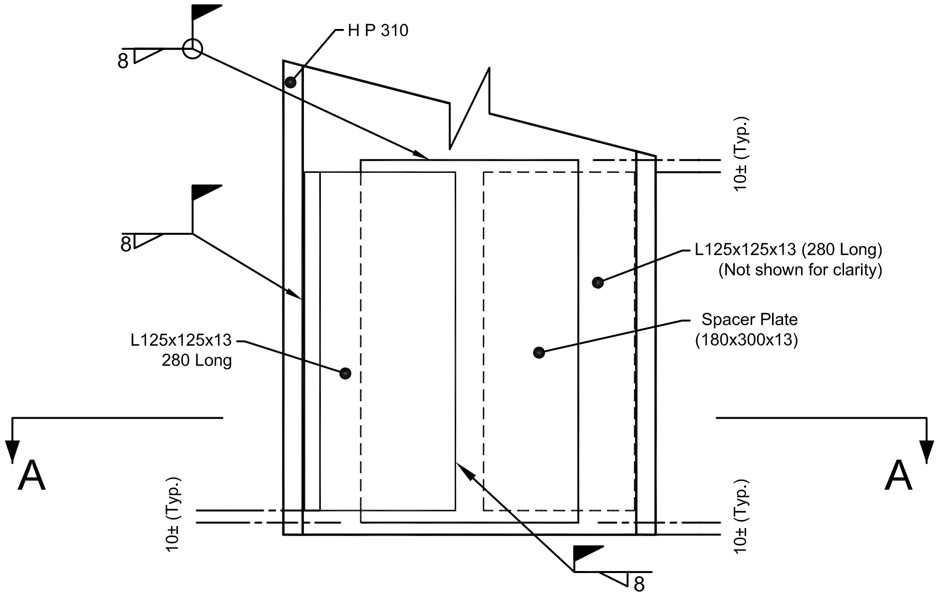
LICENSED PROFESSIONAL ENGINEER
A. N. RAHMAN
100189818
June 29, 2018
PROVINCE OF ONTARIO

Nazibur Rahman, P.Eng.
Senior Engineer, Geotechnical Services

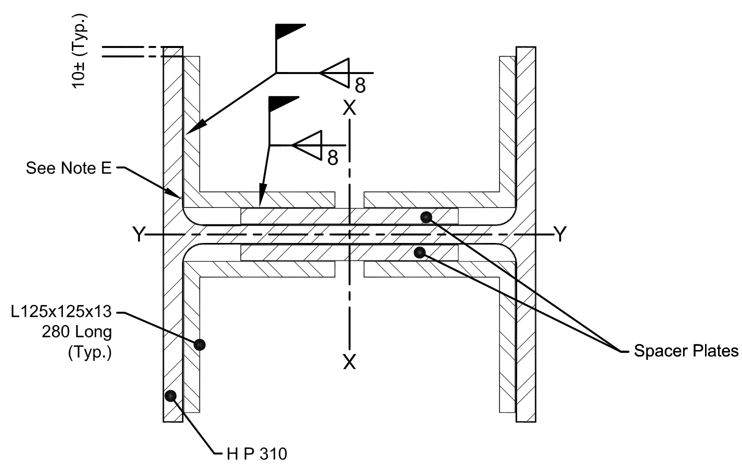
REGISTERED PROFESSIONAL ENGINEER
C M P Nascimento
June 29, 2018
PROVINCE OF ONTARIO

Carlos M.P. Nascimento, P.Eng.
Project Manager and
MTO Designated Principal Contact

AK/MV/NR/CN:nk



ELEVATION



SECTION A - A

NOTES:

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

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

C. Welding shall be according to CSA W59.

D. Spacer plate shall be 13 mm thick.

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

F. Welds are symmetrical about both axis.

G. All dimensions are in millimetres unless otherwise shown.

H-PILE TIP REINFORCEMENT

BRIDGE FOUNDATION

PML Peto MacCallum Ltd.
CONSULTING ENGINEERS

		DATE	SCALE	JOB NO.	SKETCH NO.
DRAWN:	N.A.	JUNE 2018	N.T.S.	17TF006A	PML-1
CHECKED:	M.V.				
APPROVED:	C.N.				

APPENDIX FDR-A

List of Standard Specifications Relevant to Report

Non-Standard Special Provisions (NSSP)

LIST OF STANDARD SPECIFICATIONS RELEVANT TO REPORT

DOCUMENT	TITLE
OPSS.PROV 903	Construction Specification for Deep Foundations
OPSS 405	Construction Specification for Pipe Subdrains
OPSS.PROV 501	Construction Specification for Compacting
OPSS.PROV 539	Construction Specification for Temporary Protection Systems
OPSS 902	Excavation and Backfilling of Structures
OPSS.PROV 1010	Material Specification for Aggregates - Base, Subbase, Select Subgrade, and Backfill Material
OPSS.PROV 804	Construction Specification for Seed and Cover
OPSS 803	Construction Specification for Sodding
OPSD 208.01	Benching of Earth Slopes
OPSD 3090.101	Foundation, Frost Penetration depths for Southern Ontario
OPSD 3101.150	Walls Abutment, Backfill Minimum Granular Requirement
OPSD 3121.150	Walls, Retaining, Backfill, Minimum Granular Requirement
OPSD 3000.100	Steel H-Pile Driving Shoe
OPSD 3190.100	Walls, Retaining and Abutment, Wall Drain

NON-STANDARD SPECIAL PROVISIONS (NSSP)

NSSP 1 – Installation of Shoring for Roadway Protection and Excavation (Addition to OPSS 539)

The Contractor is advised that cobbles and/or boulders may be encountered during the excavation. The Contractor shall select and use the appropriate methods and equipment to account for obstructions from cobbles, during the shoring installations or excavations.

NSSP 2 – Lean Concrete to Protect Subgrade of Shallow Foundation

The contractor is advised that the silty sand subgrade encountered at this site will be susceptible to disturbance from construction traffic and any ponded water. In order to limit the degradation of the founding soil, the contractor shall take necessary measures to protect the subgrade within four hours after preparation, inspection and approval of the footing subgrade.

NSSP 3 – Surface Water Control (Addition to OPSS 517)

The Contractor shall take necessary measures for diversion of surface water and drainage, and to lower the water level to a minimum of 0.5 m below the base of the excavations to allow for construction work within the overburden in-the-dry.

The subsoil conditions encountered at this site are relatively pervious in nature. The Contractor shall be responsible for designing and implementing measures for surface water control, if required.

NSSP 4 – Soil Set-up Time for Driven Piles

Sufficient soil set-up time for Driven Piles shall be allowed when performing Hiley Formula testing for verification of load carrying capacity of steel H-piles driven into a soil deposit. The waiting time for completion of soil set-up before carrying out confirmatory dynamic testing, such as the Hiley Formula testing, shall be a minimum of 5 days. The pile(s) being tested shall remain undisturbed after the end of driving until the pile is tested. This restriction includes re-taping, further driving or splicing of the pile.

NSSP 5 – Temporary Steel Liners for Caisson Installation

The Contractor is advised that temporary steel liners are likely required to advance the caissons through the cohesionless soil typically encountered at this site. If vibration is utilized to advance and extract the liners, care should be taken to avoid damage to the existing bridge foundations from the vibrations.

In addition, the inspection of the caisson base and rock socket will need to be carried out using a shaft inspection device.