



**FOUNDATION INVESTIGATION AND DESIGN REPORT
for
PROPOSED SANITARY SEWER AND WATERMAIN
CROSSING HIGHWAY 26
STAYNER, ONTARIO**

LATITUDE: 44.422285; LONGITUDE: -80.078242

PETO MacCALLUM LTD.
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Distribution:
Electronic copy: Township of Clearview
Electronic copy: R.J. Burnside & Associates Limited
1 cc: PML Barrie
1 cc: PML Toronto

PML Ref.: 21CF013
Report: 3
August 30, 2022



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TABLE OF CONTENTS

PART A - FOUNDATION INVESTIGATION REPORT

1. INTRODUCTION	1
2. SITE DESCRIPTION	2
3. FIELD INVESTIGATION PROCEDURES	3
4. LABORATORY TEST PROCEDURES	5
5. SITE GEOLOGY AND SUBSURFACE CONDITIONS.....	5
5.1 Site Geology.....	5
5.2 Subsurface Conditions	6
5.2.1 Pavement/Shoulder	6
5.2.2 Sand Fill.....	7
5.2.3 Silt.....	7
5.2.4 Sandy Silt	8
5.2.5 Silt and Sand	8
5.2.6 Clayey Silt/Silty Clay.....	8
5.3 Ground Water.....	9
5.4 Chemical Testing.....	10
6. CLOSURE.....	11
Appendix A – Site Photographs	
Appendix B – Drawings 1 to 3 – Borehole Location Plan and Soil Strata	
Explanation of Terms Used in Report	
Log of Borehole Sheets	
Figures 3-1 to 3-7	
Results of Chemical Tests by SGS	

PART A - FOUNDATION INVESTIGATION REPORT

for
Proposed Sanitary Sewer and Watermain
Crossing Highway 26
Stayner, Ontario

1. INTRODUCTION

The Township of Clearview has retained Peto MacCallum Ltd. (PML) to conduct a foundation investigation for the planned sanitary sewer and watermain, crossing Highway 26 at the intersection with Phillips Street, in Stayner, in the Township of Clearview, Simcoe County, Ontario.

The project involves installation of a proposed sanitary sewer to be installed via open cut and trenchless methods, crossing Highway 26 at the intersection with Phillips Street. A new sanitary pipe is proposed northerly up Phillips Street from Sunnidale Road (south of Highway 26) that will cross Highway 26. The open cut sanitary sewer installation will continue westerly, within the MTO Right-Of-Way (ROW), about 108 m to a proposed manhole on Mowat Street. The trenchless sanitary sewer is proposed on a skewed angle, beginning about 2 m west of the centreline of Phillips Street on the south side of Highway 26 and ending about 5 m west of the centreline of Phillips Street on the north side of Highway 26. The sanitary sewer will comprise a 450 mm inside diameter fusible PVC pipe. The casing is to be steel and sized to accommodate no less than 50 mm of space from the outside of the watermain pipe to the casing pipe. The invert of the sanitary sewer is about 4.2 m to 5.5 m below ground surface along the open cut section and about 4.3 m to 4.6 m below the road grade along the trenchless installation.

Secondly, a proposed watermain is also to be installed via trenchless methods, crossing Highway 26 at the intersection with Phillips Street. Similar to the sanitary sewer crossing, the watermain is proposed along Phillips Street and will proceed northerly to connect with an existing watermain on the north side of Highway 26 within the Right-Of-Way (ROW). The watermain crossing is proposed on a skewed angle, beginning about 2 m east of the centreline of Phillips Street on the south side of Highway 26 and ending about 2 m west of the centreline of Phillips Street on the north side of Highway 26. The watermain will comprise of a 300 mm inside diameter fusible PVC pipe. The casing is to be steel to be sized to provide no less than 50 mm of space from the outside of the watermain pipe to the casing pipe. The obvert of the watermain is about 3.0 m to 3.1 m below the Highway 26 road grade.

Part A – Foundation Investigation Report

Proposed Sanitary Sewer and Watermain Crossing Highway 26, Stayner, Ontario
PML Ref.: 21CF013, Report: 3, August 30, 2022, Page 2



The purpose of the requested investigation was to explore the subsurface soil and ground water conditions at the site, and based on the findings, prepared a Foundation Investigation and Design report in accordance with MTO Registry, Appraisal and Qualification System (RAQS) standards to identify the subsurface conditions expected to influence the selection of trenchless method, design and construction of the proposed watermain and sanitary sewer installation.

This geotechnical investigation was conducted in conjunction with other investigations along Phillips Street and Sunnidale Street outside the MTO ROW. The report for the other roads area reported under a separate cover (Reports 1 and 2).

2. SITE DESCRIPTION

Highway 26 is generally oriented in an east/west direction and Phillips Street in a north/south direction. The MTO ROW is typically about 25 m wide in the area.

The topography of the surrounding area where the crossing is located is relatively flat and comprises urbanized curb and gutter on the south side of the Highway 26 and rural ditching/shoulders on the north side of Highway 26. Several businesses surround the intersection.

The watermain crossing will comprise an approximately 16 m long steel casing with minimum obvert level about 3.0 m below the highway centreline. The sanitary sewer crossing will comprise an approximately 16 m long steel casing with invert about 4.5 m below the highway centreline.

The sanitary sewer along the north side of Highway 26, within the MTO ROW, will have an approximate invert of about 4.5 m at the east end dropping to about 6.0 m at the west end, as shown in Drawings 1 and 2, appended. The sanitary sewer will be installed off-road by open cut construction method.

Staging pits for the trenchless installation of the sanitary sewer will be located at proposed manhole (MH) 176 on the south side of Highway 26, proposed MH 175 on the north side of Highway 26. It is likely the staging pits for both the watermain and sanitary sewer will be common on both sides due to the proximity of the pipes.

Refer to site photographs in Appendix A.



3. FIELD INVESTIGATION PROCEDURES

The field work for the proposed crossing was carried out on June 8, 13 and August 3, 2021 and comprised of four boreholes. Boreholes 1, 3 and 5 were advanced on the north side of Highway 26 along the proposed sanitary sewer alignment and at the crossing location (north side). Borehole 6 is located on the south side of Highway 26 at the crossing location. All boreholes were advanced to 9.5 to 9.6 m depth below existing grade. The borehole locations are shown on the appended Drawing 1 in Appendix B. It is noted that proposed Boreholes 2 and 4 were to be drilled on the south side of Highway 26 opposite Boreholes 1 and 3, for an alternate alignment, however the alignment was not realized and the boreholes could not be drilled due to utility conflicts.

Two (2) additional boreholes, Boreholes 5A and 7A, were investigated on July 25, 2022 based on comments provided by MTO to further assess the subsoil and groundwater conditions on either side of the proposed crossing. The boreholes were advanced 5.2 m and 9.0 m below existing grade. The borehole locations are shown on the appended Drawing 1 in Appendix B.

In general, the depth of boreholes advanced were established based on the minimum requirements of the “MTO Guidelines for Foundation Engineering – Tunnelling Speciality for Corridor Encroachment Permit Application” dated February 2021.

PML laid out the boreholes based on a site plan provided by the Client. The ground surface elevation at the borehole locations was obtained with a Sokkia SHC5000 GPS System equipped with a GCX3 (network RTK rover) Global Navigation Satellite System (GNSS) Receiver. Vertical and horizontal accuracy of this unit are 0.1 m and 0.5 m, respectively. All elevations in this report are geodetic and expressed in metres.

Co-ordination for clearances of underground utilities was provided by PML.

Both UTM coordinates (Zone 17) and Latitude and Longitude for the boreholes are provided in the table below.

Part A – Foundation Investigation Report

Proposed Sanitary Sewer and Watermain Crossing Highway 26, Stayner, Ontario
PML Ref.: 21CF013, Report: 3, August 30, 2022, Page 4



BOREHOLE NUMBER	UTM (ZONE 17)		LATITUDE	LONGITUDE
	NORTHING	EASTING		
1	4919162.0	573254.0	44.422045	-80.079766
3	4919167.6	573305.4	44.422091	-80.079120
5	4919190.0	573375.0	44.422285	-80.078242
5A	4919185.0	573376.0	44.422240	-80.078231
6	4919155.0	573372.0	44.421197	-80.078285
7A	4919174.0	573366.0	44.422142	-80.078358

Borehole log sheets and Borehole Location and Soil Strata drawings appended to the report are referenced to the UTM – Zone 17 coordinates.

The fieldwork was supervised throughout by a PML engineering staff member, who directed the drilling and sampling operations, prepared the stratigraphic logs, monitored ground water conditions and processed the recovered samples.

The boreholes were advanced using continuous flight solid stem augers, powered by a truck mounted CME-55 drill rig, equipped with an automatic hammer. The drill rig used was supplied and operated by a specialist drilling contractor (Ontario Soil Drilling).

Where required, traffic protection was provided in accordance with Ontario Traffic Manual - Temporary Conditions Manual - Book 7. The work was carried out under MTO Highway Corridor Management Encroachment Permit EC-2021-20T-00000264 V1, dated June 25, 2021.

Representative samples of the overburden were recovered at frequent depth intervals for identification purposes using a conventional 51 mm OD split spoon sampler. The sampler excludes particles larger than 38 mm. Standard penetration tests were carried out simultaneously with the sampling operations to assess the strength characteristics of the subsoil (ASTM D1586-11 Procedures). The ground water conditions in the boreholes were assessed during drilling by visual examination of the soil samples, the sampler, and drill rods as the samples were retrieved, and measurement of the water level in the open boreholes.

Monitoring wells, comprised of 50 mm diameter PVC pipe, filter sand, bentonite seal, and stick-up protection casing, were installed in three boreholes to permit monitoring of the ground water table.



The details of the monitoring well installations are shown on the applicable Log of Borehole Sheets. The borehole without a well was backfilled in accordance with O. Reg. 903, as amended. The monitoring wells/boreholes should also be decommissioned when they are no longer in use. PML will be pleased to provide assistance to the decommissioning work if requested in the future.

The ground water levels in the wells were measured on November 11, 2021 with a Huron Instruments water level tape.

Soils were identified and documented in the field according to the MTO soil classification system. The recovered soil samples were returned to our laboratory for detailed visual examination and index tests.

4. LABORATORY TEST PROCEDURES

The laboratory testing program comprised of visual examination and moisture content determination on all recovered samples. Fourteen particle size analyses tests were conducted on selected soil samples to determine the index properties of the prominent soil types encountered (25% of the samples). Atterberg Limits tests were also completed on ten samples (three samples were non-plastic). 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 standards, with the exception of hydrometer test (LS-702).

The results of the grain size distribution analyses and Atterberg Limits tests are presented on the appended Figures 3-1 to 3-7 in Appendix B.

Two samples were submitted to an external laboratory for chemical testing of soil corrosivity parameters. Results are provided in Appendix B.

5. SITE GEOLOGY AND SUBSURFACE CONDITIONS

5.1 Site Geology

According to the Ontario Division of Mines Map 2556 the quaternary geology the site is on the border of calm water deposits of silt and clay, and beach deposits of sand and sand and gravel.

Bedrock below the overburden is mapped as limestone, dolostone, shale, arkose, and sandstone of the Simcoe Group, Shadow Lake Formation from the Middle Ordovician period of the Paleozoic



era of the Phanerozoic eon, according to the Ontario Division of Mines Map 2445. Based on the nearby well records from the Ministry of Environment, Conservation and Parks (MECP), the bedrock was not encountered within 30 m of the ground surface.

5.2 Subsurface Conditions

The subsurface conditions encountered during the course of the investigation, together with the field and laboratory test results are shown on the attached Record of Borehole Sheets. The stratigraphic profile sections are shown on Drawings 2 and 3, appended. 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 soil conditions encountered along the alignment under the road and along the north side of the MTO ROW consisted of pavement/shoulder granular followed by sand fill which is underlain by native units of silt, sandy silt, silt and sand, and clayey silt.

For classification purposes, the soils encountered at this site can be divided into six distinct zones:

- a) Pavement/Shoulder Granular
- b) Sand Fill
- c) Silt
- d) Sandy Silt
- e) Silt and Sand
- f) Clayey Silt

5.2.1 Pavement/Shoulder

Road pavement was encountered at the surface of Borehole 1 (Mowat Street), and Borehole 6 (Phillips Street). The pavement structure comprised 70 to 80 mm of asphalt, over 100 mm of granular base, over 150 to 200 mm of granular subbase, for a total thickness of 400 mm.

At Boreholes 5A and 7A (Highway 26), road pavement was encountered immediately at the surface, which comprised 200 mm and 180 mm of asphalt, over 150 mm of granular base and 200 mm of granular subbase, respectively, for a total thickness of 550 mm and 530 mm. Borehole 3 was drilled in the north shoulder of Highway 26 and revealed 100 mm of granular base over 200 mm of granular subbase, for a total thickness of 300 mm.



Borehole 5 was drilled in a driveway entrance to a local business and encountered pavement comprising 100 mm asphalt over 150 mm of granular base over 250 mm of granular subbase for a total thickness of 500 mm.

5.2.2 Sand Fill

A fill layer comprised of sand was present in all boreholes beneath the pavement/shoulder granular. The fill was 1.7 to 2.6 m thick and extended to 2.1 to 2.9 m depth (elevations 209.9 to 211.0). The fill consisted sand with varying proportions of gravel and silt. The N values ranged between 4 and 20 blows per 0.3 m penetration of the split spoon sampler, indicating a loose to compact conditions. The moisture content of the samples ranged approximately from 6.0% to 22.0%.

The fill was in a moist to wet condition with depth, and the moisture contents varied from 6 to 21%. The results of the grain size distribution analysis conducted on two representative samples of the sand fill are presented on Figure 3-1, in Appendix B. The test results revealed that the fill layer consists of 1 to 18% gravel, 73 to 88% sand, and 9 to 11% silt particles. The material is classified as SP-SM based on the MTO classification system.

5.2.3 Silt

Underlying the fill in Boreholes 1, 5 and 5A a silt unit was present. The silt was 0.9 to 2.6 m thick and extended to 3.4 to 5.0 m depth (elevation 207.4 to 209.2). A lower silt unit was encountered in Borehole 5A at 4.2 m (elevation 208.4), extending to the termination depth of 5.2 m (elevation 207.4). N values in the silt were typically 35 to greater than 50, locally 4 (loose) or 13 to 17 (compact). The moisture content of the silt samples ranged approximately between 8% and 29%.

The silt unit had moisture contents of 8 and 20%, locally 29%, and was moist to wet. The results of four (4) grain size distribution analysis conducted on representative samples of the soil are presented on Figure 3-2, in Appendix B. The results showed that the samples comprised 1 to 4% gravel, 12 to 13% sand, 77 to 80% silt and 6 to 7% clay size particles. Atterberg Limits tests were also completed on the samples and showed the soil was non-plastic. The material is classified as ML based on the MTO classification system.



5.2.4 Sandy Silt

A native sandy silt layer was encountered below the silt in Borehole 1 and the silt and sand in Borehole 3. The sandy silt extended to 6.0 to 7.0 m depth (elevation 206.0 to 207.1) and was 1.7 to 2.7 m thick. This layer was encountered below clayey silt in Borehole 7A at 6.4 m (elevation 206.2), extending to the termination depth of 9.0 m (elevation 203.6) below ground surface. The layer was compact to very dense with SPT N values ranging between 15 and 83. The moisture content of the sandy silt samples ranged approximately from 7% to 20%.

The results of the grain size distribution analyses conducted on two representative samples of the sandy silt are presented on Figure 3-3 in Appendix B. The test results revealed that the sandy silt consists of 0 to 1% gravel, 20 to 27% sand, 70 to 72% silt and 3 to 7% clay sized particles. The material is classified as ML based on the MTO classification system.

5.2.5 Silt and Sand

A native silt and sand deposit was encountered in Boreholes 1, 3, 5 and 5A. In Borehole 3, the deposit was 1.4 m thick and occurred from 2.9 to 4.3 m depth (elevation 208.7 to 210.1). In Boreholes 1, 5 and 6 the silt and sand was below the upper layers and continued to the 9.5 to 9.6 m depth of exploration (elevation 202.8 to 203.5). The soil was dense to very dense with SPT N values ranging from 30 to greater than 50. The moisture content of the samples ranged between 5% and 21%.

The results of the grain size distribution analyses conducted on five representative samples of the soil are presented on Figure 3-4 in Appendix B. The test results revealed that the material consists of 1 to 17% gravel, 23 to 35% sand, 41 to 61% silt and 3 to 18% clay sized particles. Atterberg Limits testing was also carried out on all five samples, and the plasticity indices were 6 and lower, (one sample was non-plastic) and indicate that the plastic limits are 10 to 12 and the liquid limits are 14 to 17. Results are plotted on Figure 3-5 in Appendix B. The material is classified as ML to ML-CL based on the Atterberg Limits testing.

5.2.6 Clayey Silt/Silty Clay

In Boreholes 3, 5, 5A, 6 and 7A, a clayey silt unit was revealed. In Boreholes 5 and 6 the unit was an upper layer being 1.0 to 1.9 m thick, and occurring from 5.0 to 7.0 m depth (elevation 206.4 to 207.4) in Borehole 5 and from 2.4 to 4.3 m depth (elevation 208.0 to 209.9) in Borehole 6.

Part A – Foundation Investigation Report

Proposed Sanitary Sewer and Watermain Crossing Highway 26, Stayner, Ontario
PML Ref.: 21CF013, Report: 3, August 30, 2022, Page 9



In Borehole 3, the clayey silt was beneath the sandy silt and extended from 7.0 m depth to the 9.6 m termination depth of the borehole. In Borehole 5A, this layer was encountered between the silt layers from 3.4 m to 4.2 m (elevation 209.2 to 208.4). In Borehole 7A, the clayey silt/silty clay was encountered below the fill at 3.3 m (elevation 209.3), extending to 6.4 m (elevation 206.2) below ground surface. The soil consistency was very stiff to hard based on SPT N values recorded in Boreholes 3 and 5 (N values 29 or greater) and soft to stiff in Borehole 6 (N values 2 to 13). In-situ shear strengths obtained in Boreholes 5A and 7A ranged from 95 kPa to 100 kPa, indicating stiff consistency. Moisture content of the cohesive soil samples ranged approximately from 18% to 34%.

The results of the grain size distribution analyses conducted on seven representative samples of the soil are presented on Figure 3-6 in Appendix B. The test results revealed that the material consists of 0 to 7% gravel, 2 to 15% sand, to 42 to 46% silt and 38 to 52% clay sized particles. Atterberg Limits testing was also carried out on six samples and show that the plastic limits are 14 to 16 and the liquid limits are 28 to 34, with a resulting plasticity index of 13 to 18. Two of the samples at Borehole 7A exhibited the water content near or slightly higher than the liquid limits. Results are plotted on Figure 3-7 in Appendix B. The material is classified as CL based on the MTO classification system.

5.3 Ground Water

First ground water strike (wet spoon) during drilling, ground water level measured upon completion of augering, and the ground water level measured in the wells are summarized in the table below:

BOREHOLE	FIRST GROUND WATER STRIKE		UPON COMPLETION		MONITORING WELL 2021-11-11	
	DEPTH (m)	ELEVATION	DEPTH (m)	ELEVATION	DEPTH (m)	ELEVATION
1	2.9	210.2	No Water to 9.6*	No Water to 203.5*	1.1	212.0
3	2.1	210.9	1.8	211.2	--	--
5	1.4	211.0	No Water to 9.6*	No Water to 202.8*	6.6	205.8
5A	1.5	211.1	No Water to 5.2*	No Water to 207.4*	--	--



BOREHOLE	FIRST GROUND WATER STRIKE		UPON COMPLETION		MONITORING WELL 2021-11-11	
	DEPTH (m)	ELEVATION	DEPTH (m)	ELEVATION	DEPTH (m)	ELEVATION
6	2.1	210.2	No Water to 9.5*	No Water to 202.8*	8.4	203.9
7A	2.3	210.3	No Water to 9.0*	No Water to 203.6*	--	--

Note: * depth/elevation noted is to bottom of borehole.

Soil caving was encountered during drilling in Borehole 1 at 1.9m, and wet caving was observed at Boreholes 5A and 7A at 2.7m and 3.7m, respectively.

The ground water levels at the site are subject to seasonal fluctuations and precipitation patterns. It should be noted that the relatively impermeable nature of the native soil could contribute to the development of perched water conditions in the more pervious sand fill, following short term and seasonal precipitation events.

5.4 Chemical Testing

Two samples were submitted to an external laboratory for chemical testing of soil corrosivity parameters. Details of the chemical test results are presented on the certificate of analysis presented in Appendix B. The test results are summarized in the following table.

Summary of Corrosivity Test Results

BOREHOLE	SAMPLE	DEPTH (m)	SOIL TYPE	SULPHATE (µg/g)	CHLORIDE (µg/g)	pH	RESISTIVITY (ohm-cm)
1	3	1.5 to 1.9	Sand (fill)	20	92	7.86	3890
3	2	0.8 to 1.4	Sand (fill)	80	1750	8.05	395



6. CLOSURE

The field work was carried out under the supervision of Mr. N. Gardlund, EIT. The drilling equipment was supplied and operated by Ontario Soil Drilling and London Soil Test Ltd. working under subcontract to PML. Traffic control was provided by Direct Traffic Management and PML. The laboratory work was carried out in the PML Barrie laboratory. The external laboratory testing was conducted by SGS Laboratories.

This report was prepared by Mr. G. White, P.Eng., formerly with PML, and Mr. Nazibur Rahman, P.Eng. Independent review of the report was carried out by Mr. R. Ng, P.Eng., MTO Designated Principal Contact.

We trust this report has been completed within the terms of reference and is sufficient for your current needs. Should you have further questions, do not hesitate to contact our office.

Sincerely

Peto MacCallum Ltd.



Nazibur Rahman, P.Eng.
Manager, Geotechnical Services



Robert Ng, MBA, PhD, P.Eng.
MTO Designated Principal Contact



APPENDIX A

Site Photographs



Photograph 1: Looking north from Phillips Street on south side of Highway 26 at crossing location.



Photograph 2: Looking south from north side of Highway 26 at crossing location.



Photograph 3: Looking west from crossing to Mowat Street on north side of Highway 26.



Photograph 4: Looking east from Mowat Street to crossing location on north side of Highway 26.



APPENDIX B

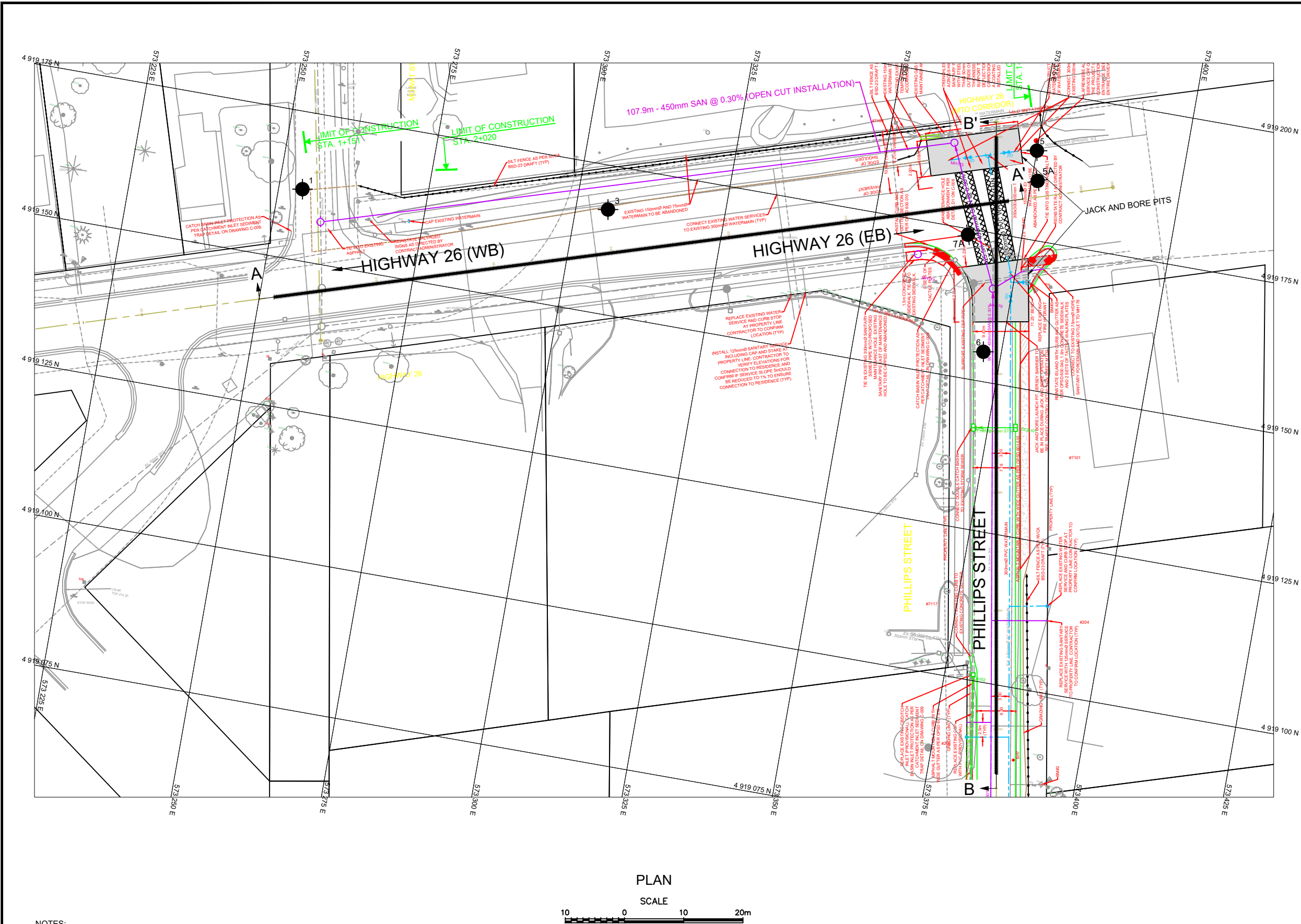
Drawings 1 to 3 – Borehole Location Plan and Soil Strata

Explanation of Terms Used in Report

Log of Borehole Sheets

Figures 3-1 to 3-7

Results of Chemical Tests by SGS



PLAN

SCALE



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.



SANITARY SEWER WATERMAIN
PHILLIPS STREET AND HIGHWAY 26
BOREHOLE LOCATION PLAN

SHEET



LEGEND

● Borehole Location

BH No.	ELEVATION	COORDINATES (UTM17)	
		NORTHING	EASTING
1	213.1	4 919 162.0	573 254.0
3	213.0	4 919 167.6	573 305.4
5	212.4	4 919 190.0	573 375.0
5A	212.6	4 919 185.0	573 376.0
6	212.3	4 919 155.0	573 372.0
7A	212.6	4 919 174.0	573 366.0

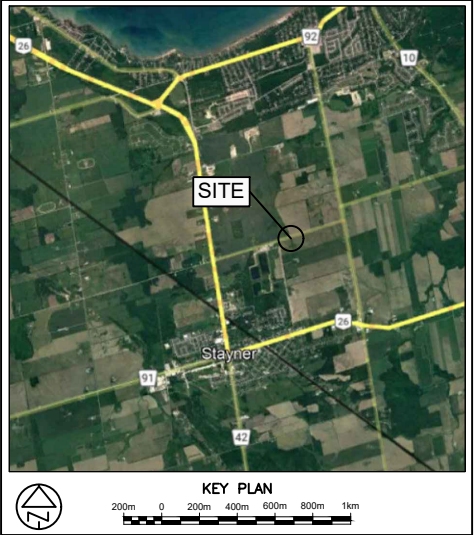
NOTE

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

REVISIONS	DATE	BY	DESCRIPTION

Geocres No. Not Assigned

HWY No.	26	DIST.	
SUBMD	NL	CHECKED	RN
DATE	AUG. 26, 2022	SITE	
DRAWN	NL	CHECKED	GU
APPROVED	GU	DWG	1

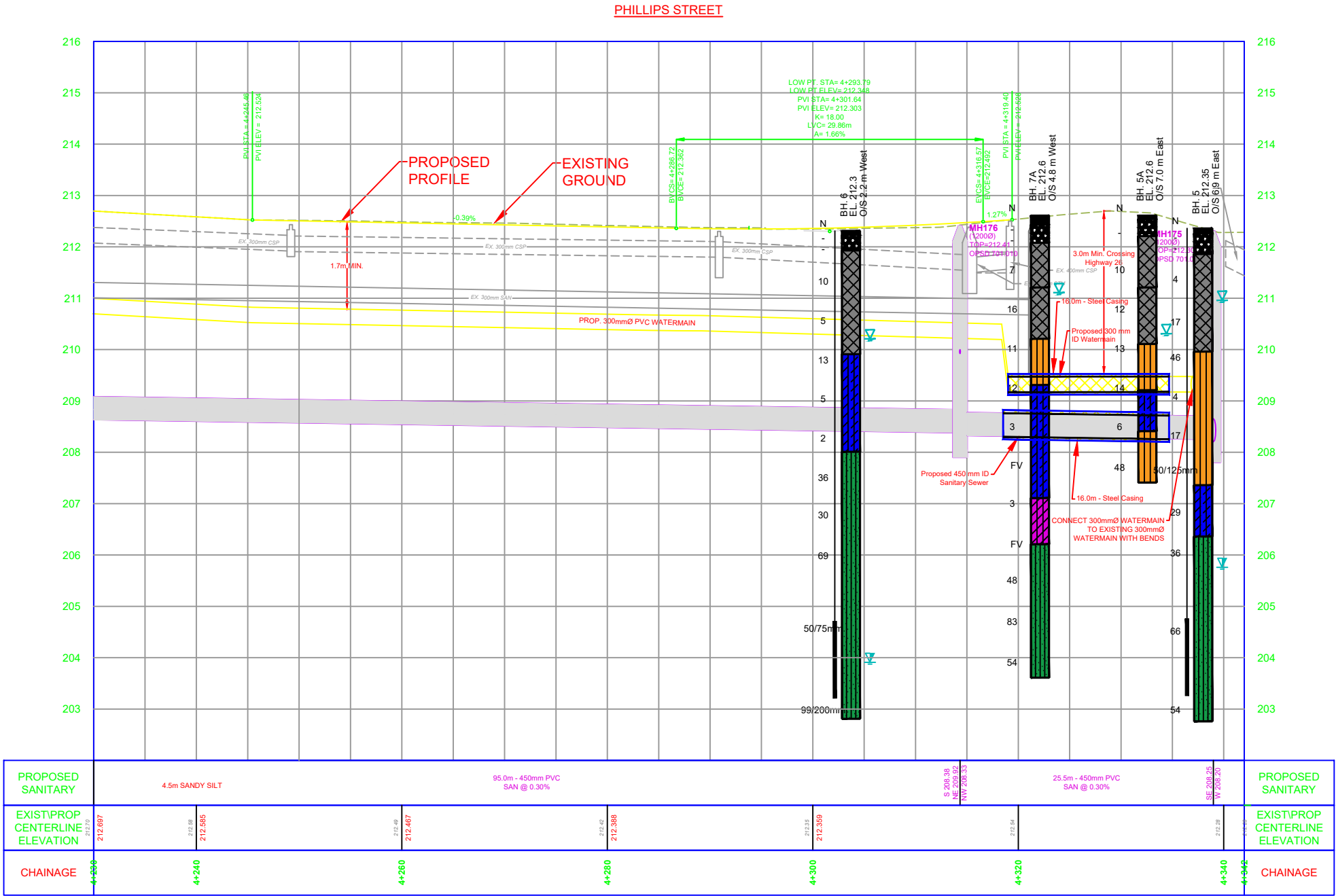


LEGEND	
N	Blows/0.3m (Std. Pen Test, 475 J/blow)
	Monitoring Well
	Groundwater Level Observed During Drilling
	Groundwater Level Measured in Monitoring Well
	Asphalt
	Granular Base/Subbase
	Fill
	Silt
	Sand and Silt
	Clayey Silt
	Silty Clay

BH No.	ELEVATION	COORDINATES (UTM17)	
		NORTHING	EASTING
5	212.4	4 919 190.0	573 375.0
5A	212.6	4 919 185.0	573 376.0
6	212.3	4 919 155.0	573 372.0
7A	212.6	4 919 174.0	573 366.0

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

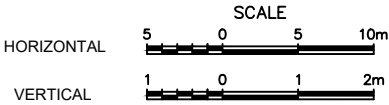
REVISIONS		DATE	BY	DESCRIPTION
Geocres No. Not Assigned				
HWY No.	26	DIST		
SUBMTD	NL	CHECKED	RN	DATE AUG. 26, 2022
DRAWN	NL	CHECKED	GU	APPROVED GU
				DWG 3



NOTES:

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PROFILE ALONG B-B'



EXPLANATION OF TERMS USED IN REPORT

SPT N VALUE: THE STANDARD PENETRATOIN TEST (SPT) N VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O.D. SPLIT-BARREL SAMPLER TO PENETRATE 0.3 m, AFTER AN INITIAL PENETRATIO OF 150 mm, INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5 kg, FALLING FREELY A DISTANCE OF 0.76 m FOR PENETRATIONS. A SPT N VALUE IS INDICATED AS THE NUMBER OF BLOWS REQUIRED TO DRIVE THE SPLIT-BARREL SAMPLER A DISTANCE OF 300 MM. AN AVERAGE SPT N VALUE IS DENOTED as \bar{N} .

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

SOILS ARE DESCRIBED BY THEIR COMPOSITION, CONSISTENCY OR COMPACTNESS.

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

PERCENTAGE BY MASS	0 - 10	10 - 20	20 -35	>35	>35 and main fraction
	'trace'	'some'	Adjective (silty, sandy, clayey etc.)	'and'	Noun (gravel, sand, silt, clay)

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

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

N (BLOWS/0.3 m PENETRATION)	0 - 4	4 - 10	10 -30	30 - 50	>50
	VERY LOOSE	LOOSE	COMPACT	DENSE	VERY DENSE

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

TOTAL CORE RECOVERY (REC): CORE RECOVERED AS A PERCENTAGE OF TOTAL CORE RUN LENGTH.

ROCK QUALITY DESIGNATION (RQD): TOTAL LENGTH OF SOUND ROCK RECEIVED IN PIECES 10 cm OR LARGER AS A PERCENTAGE OF TOTAL CORE RUN LENGTH. CLASSIFICATION OF ROCK WITH RESPECT TO RQD VALUE AS FOLLOWS:

RQD VALUE (%)	<25	25 - 50	50 -75	75 - 90	90 - 100
	VERY POOR	POOR	FAIR	GOOD	EXCELLENT

UNIAXIAL COMPRESSIVE STRENGTH (UCS): MAXIMUM AXIAL COMPRESSIVE STRESS THAT A ROCK CORE SPECIMEN CAN WITHSTAND BEFORE FAILING.

POINT LOAD STRENGTH INDEX: AN INDEX TEST TO DETERMINE POINT LOAD STRENGTH INDEX OF ROCK.

CLASSIFICATION OF ROCK WITH RESPECT TO STRENGTH IS AS FOLLOWS:

GRADE*	R0	R1	R2	R3	R4	R5	R6
UCS (MPa)	0.25 - 1	1 - 5	5 - 25	25 - 50	50 - 100	100 - 250	>250
POINT LOAD INDEX (MPa)	**	**	**	1 - 2	2 - 4	4 - 10	>10
TERM	EXTREMELY WEAK	VERY WEAK	WEAK	MEDIUM STRONG	STRONG	VERY STRONG	EXTREMELY STRONG

* - GRADE ACCORDING TO THE INTERNATIONAL SOCIETY OF ROCK MECHANICS (ISRM), 1981.

** - ROCKS WITH UNIAXIAL COMPRESSIVE STRENGTH BELOW 25 MPa ARE LIKELY TO YIELD HIGHLY AMBIGUOUS RESULTS UNDER POINT LOAD TESTING.

DISCONTINUITY SPACING: DISTANCE BETWEEN A PAIR OF DISCONTINUITIES MEASURED ALONG A LINE OF SPECIFIED LOCATION AND ORIENTATION. CLASSIFICATION OF ROCK WITH RESPECT TO DISCONTINUITY SPACING IS AS FOLLOWS (ISRM, 1981):

SPACING WIDTH (m)	<0.02	0.02 - 0.06	0.06 - 0.20	0.20 - 0.6	0.6 - 2.0	2.0 - 6.0	>6.0
SPACING CLASSIFICATION	EXTREMELY CLOSE	VERY CLOSE	CLOSE	MODERATELY CLOSE	WIDE	VERY WIDE	EXTREMELY WIDE

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

SS - SPLIT SPOON TP - THINWALL PISTON SAMPLE
WS - WASH SAMPLE OS - OSTERBERG SAMPLE
AS - AUGER SAMPLE RC - ROCK CORE
FV - FIELD VANE BS - BLOCK SAMPLE
CS - CHUNK SAMPLE FS - FOIL SAMPLE
TW - THINWALL SHELBY TUBE SAMPLE
PH - TW ADVANCED HYDRAULICALLY
PM - TW ADVANCED MANUALLY

STRESS AND STRAIN

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

MECHANICAL PROPERTIES OF SOIL

C_c	COMPRESSION INDEX
C_{cr}	RECOMPRESSION INDEX
C_s	SWELL INDEX
c_v	COEFFICIENT OF CONSOLIDATION - VERTICAL (cm ² /s)
c_h	COEFFICIENT OF CONSOLIDATION - HORIZONTAL (cm ² /s)
C_α	COEFFICIENT OF SECONDARY CONSOLIDATION
m_v	COEFFICIENT OF VOLUME CHANGE (kPa ⁻¹)
σ'_p	PRECONSOLIDATION PRESSURE (kPa)
σ'_{vo}	EFFECTIVE OVERBURDEN PRESSURE (kPa)
H	DRAINAGE PATH (m)
U	DEGREE OF CONSOLIDATION
T_v	TIME FACTOR; VERTICAL DRAINAGE
T_h	TIME FACTOR; HORIZONTAL DRAINAGE
S_{u, c_u}	UNDRAINED SHEAR STRENGTH (kPa)
S_R	RESIDUAL SHEAR STRENGTH (kPa)
S_r	REMOULDED SHEAR STRENGTH (kPa)
σ_c	UNIAXIAL COMPRESSIVE STRENGTH (kPa)
c'	EFFECTIVE COHESION INTERCEPT (kPa)
c	APPARENT COHESION INTERCEPT (kPa)
Φ'	EFFECTIVE ANGLE OF INTERNAL FRICTION (Degrees)
S_t	SENSITIVITY (= c_u/S_c)
I_p	POINT LOAD STRENGTH INDEX

PHYSICAL PROPERTIES

W _p - PLASTIC LIMIT (%)	W _L - LIQUID LIMIT (%)	W - MOISTURE CONTENT (%)
W _s - SHRINKAGE LIMIT (%)	I _p - PLASTICITY INDEX (%)	γ_w - UNIT WEIGHT OF WATER (kg/m ³)
γ - UNIT WEIGHT OF SOIL (kg/m ³)	γ_{sat} - UNIT WEIGHT OF SATURATED SOIL (kg/m ³)	γ_d - UNIT WEIGHT OF DRY SOIL (kg/m ³)
ρ_w - DENSITY OF WATER (kN/m ³)	ρ - DENSITY OF SOIL (kN/m ³)	ρ_{sat} - DENSITY OF SATURATED SOIL (kN/m ³)
ρ_d - DENSITY OF DRY SOIL (kN/m ³)	S_r - DEGREE OF SATURATION (%)	D_r, SG - RELATIVE DENSITY (FORMERLY SPECIFIC GRAVITY)
C_u - UNIFORMITY COEFFICIENT	C_c - CURVATURE COEFFICIENT	

RECORD OF BOREHOLE No 1

1 OF 1

METRIC

G.W.P. _____ LOCATION Coordinates: 4 919 162 N, 573 254 E (UTM17) ORIGINATED BY NG
 DIST _____ HWY 26 BOREHOLE TYPE Solid Stem Augers COMPILED BY GW
 DATUM Geodetic DATE 2021.06.13 LATITUDE 43.764723 LONGITUDE -80.470385 CHECKED BY GW

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/ GAS READING γ _{kN/m³} / ppm/%	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								
213.1	GROUND SURFACE															
0.0	PAVEMENT: 70 mm asphalt, over		1A	GS	-											
212.7	100 mm granular base, over		1B	GS	-											
0.4	200 mm granular subbase, moist															
	SAND, trace silt, trace gravel															
	Compact, Brown, Moist to very moist		2	SS	16											
	(FILL)		3	SS	20											
211.0	SILT, some sand, trace clay, trace gravel															
2.1	Very dense to dense, Brown to grey, Very moist to wet		4	SS	62											
			5	SS	47											
			6	SS	35											
208.8	SANDY SILT, trace clay															
4.3	Dense to compact, Brown, Very moist to moist		7	SS	37											
			8	SS	15											
207.1	SILT AND SAND, trace clay															
6.0	Very dense to dense, Grey, Very moist		9	SS	67											
			10	SS	68											
203.5	End of borehole		11	SS	48											
9.6	NOTES: 1. First water strike noted at 2.9 m during drilling. 2. No water was noted upon completion of drilling. 3. Cave was noted at 1.9 m upon completion of drilling. 4. Water level in well on November 11, 2021 was at 1.1 m/ elevation 212.0															

RECORD OF BOREHOLE No 3

1 OF 1

METRIC

G.W.P. _____ LOCATION Coordinates: 4 919 168 N, 573 306 E (UTM17) ORIGINATED BY NG
 DIST _____ HWY 26 BOREHOLE TYPE Solid Stem Augers COMPILED BY GW
 DATUM Geodetic DATE 2021.06.13 LATITUDE 43.764633 LONGITUDE -80.470394 CHECKED BY GW

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/ GAS READING γ _{kN/m³} / ppm/%	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)		
								20	40	60							80	100
213.0	GROUND SURFACE		1A	GS	-													
0.0	SHOULDER GRANULAR:		1B	GS	-													
212.7	100 mm granular base, over																	
0.3	200 mm granular subbase, moist																	
	SAND, some silt, trace gravel																	
	Compact, Brown, Very moist to wet		2	SS	10													
			3	SS	18													
			4	SS	16													
210.1	(FILL)																	
2.9	SILT AND SAND, some clay, trace gravel		5	SS	39													
	Compact to very dense, Brown to grey, Moist		6	SS	50/140mm													
208.7																		
4.3	SANDY SILT, trace clay		7	SS	49													
	Dense, Grey, Very moist		8	SS	47													
			9	SS	45													
206.0																		
7.0	CLAYEY SILT		10	SS	51													
	Hard, Grey, APL																	
203.4			11	SS	34													
9.6	End of borehole																	
	NOTES: 1. First water strike noted at 2.1 m during drilling. 2. Water level noted at 1.8 m upon completion of drilling.																	

RECORD OF BOREHOLE No 5

1 OF 1

METRIC

G.W.P. _____ LOCATION Coordinates: 4 919 190 N, 573 375 E (UTM17) ORIGINATED BY NG
 DIST _____ HWY 26 BOREHOLE TYPE Solid Stem Augers COMPILED BY GW
 DATUM Geodetic DATE 2021.06.08 LATITUDE 43.764813 LONGITUDE -80.470323 CHECKED BY GW

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/ GAS READING γ _{kN/m³} / ppm/%	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	SHEAR STRENGTH kPa														
212.4	GROUND SURFACE		1A	GS	-		212					20	40	60	80	100	GR	SA	SI	CL
0.0	PAVEMENT: 100 mm asphalt, over 150 mm granular base, over 250 mm granular subbase, moist		1B	GS	-			20	40	60	80									
211.9	SAND, some silt, trace gravel		2	SS	4		211					20	40	60	1	88	(11)			
0.5	Loose to compact, Brown, Moist to wet		3	SS	17		210													
	(FILL)		4	SS	46		209													
210.0	SILT, some sand, trace clay, trace gravel		5	SS	4		210					20	40	60	4	12	77	7		
2.4	Loose to very dense, Grey, Wet		6	SS	17		209													
			7	SS	50/125mm		208													
			8	SS	29		207													
			9	SS	36		206													
207.4	CLAYEY SILT, some sand, trace gravel		10	SS	66		207					20	40	60	10	31	44	15		
5.0	Very stiff, Grey, APL		11	SS	54		206													
206.4	SILT AND SAND, some clay, trace gravel						205													
6.0	Very dense, Grey, Moist						205					20	40	60						
							204													
							203													
202.8	End of borehole						203					20	40	60						
9.6																				
	NOTES:																			
	1. First water strike noted at 1.4 m during drilling.																			
	2. No water was noted upon completion of drilling.																			
	3. Water level in well on November 11, 2021 was at 6.6 m / elevation 205.8																			

RECORD OF BOREHOLE No 5A

1 OF 1

METRIC

G.W.P. _____ LOCATION Coordinates: 4 919 185 N; 573 376 E (UTM17) ORIGINATED BY NG
 DIST _____ HWY 26 BOREHOLE TYPE Hollow Stem Augers COMPILED BY NR
 DATUM Geodetic DATE 2022.07.25 LATITUDE 44.42224 LONGITUDE -80.07823 CHECKED BY NR

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/ GAS READING γ _{kN/m³} / ppm/%	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									WATER CONTENT (%)
								20	40	60	80	100					
212.6	GROUND SURFACE																
0.0	PAVEMENT: 200 mm asphalt, over 150 mm granular base, over 200 mm granular subbase, moist		1A 1B	GS GS	- -												
212.1	SAND AND GRAVEL, trace silt to silty																
0.6	Compact, Brown to black/grey, Moist		2	SS	10												
211.2	SAND, trace to some silt, trace gravel																
1.4	Compact, Brown, Wet		3	SS	12												
	(FILL)																
210.1	SILT, some sand, trace clay		4	SS	13												
2.5	Compact, Grey, Very moist																
209.2	CLAYEY SILT, trace sand		5	SS	14												
3.4	Stiff, Grey, APL																
208.4	SILT, trace sand, trace clay		6	SS	6												
4.2	Dense, Grey, Moist																
207.4	End of borehole		7	SS	48												
5.2	Notes: 1. First water strike noted at 1.5 m during drilling. 2. Wet cave was noted at 2.7 m upon completion of drilling and following removal of the hollow augers 3. Moved 2.0 m west and re-drilled to 3.8 m depth for in-situ fieldvane test 4. Where in-situ fieldvane test was carried out, the consistency was based on the shear strength and was not based on SPT N value.																

ONTARIO MTO - W/GAS READING 21CF013 MTO LOGS (BH 1-6) 2021-11-17.GPJ ONTARIO MTO.GDT 22-8-30

RECORD OF BOREHOLE No 6

1 OF 1

METRIC

G.W.P. _____ LOCATION Coordinates: 4 919 155 N, 573 372 E (UTM17) ORIGINATED BY NG
 DIST _____ HWY 26 BOREHOLE TYPE Solid Stem Augers COMPILED BY GW
 DATUM Geodetic DATE 2021.06.13 LATITUDE 43.764544 LONGITUDE -80.470467 CHECKED BY GW

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/ GAS READING γ _{kN/m³} / ppm/%	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)
212.3	GROUND SURFACE							20	40	60	80	100				
0.0	PAVEMENT: 80 mm asphalt, over		1A	GS	-		212									
212.0	100 mm granular base, over		1B	GS	-											
0.3	150 mm granular subbase, moist															
	SAND, some gravel, trace silt															
	Loose to compact, Brown, Moist to wet		2	SS	10										18	73 (9)
							211									
			3	SS	5											
	(FILL)						210									
209.9	CLAYEY SILT, trace sand		4	SS	13											
2.4	Stiff to soft, Grey, APL		5	SS	5		209								1	15 44 40
			6	SS	2											
208.0	SILT AND SAND, trace to some gravel, trace clay		7	SS	36		208								7	35 44 14
4.3	Dense to very dense, Grey, Moist		8	SS	30		207									
			9	SS	69		206								17	29 41 13
			10	SS	50/75mm		205									
			11	SS	99/200mm		204									
202.8	End of borehole						203									
9.5	NOTES: 1. First water strike noted at 2.1 m during drilling. 2. No water was noted upon completion of drilling. 3. Water level in well on November 11, 2021 was at 8.4 m / elevation 203.9															

RECORD OF BOREHOLE No 7A

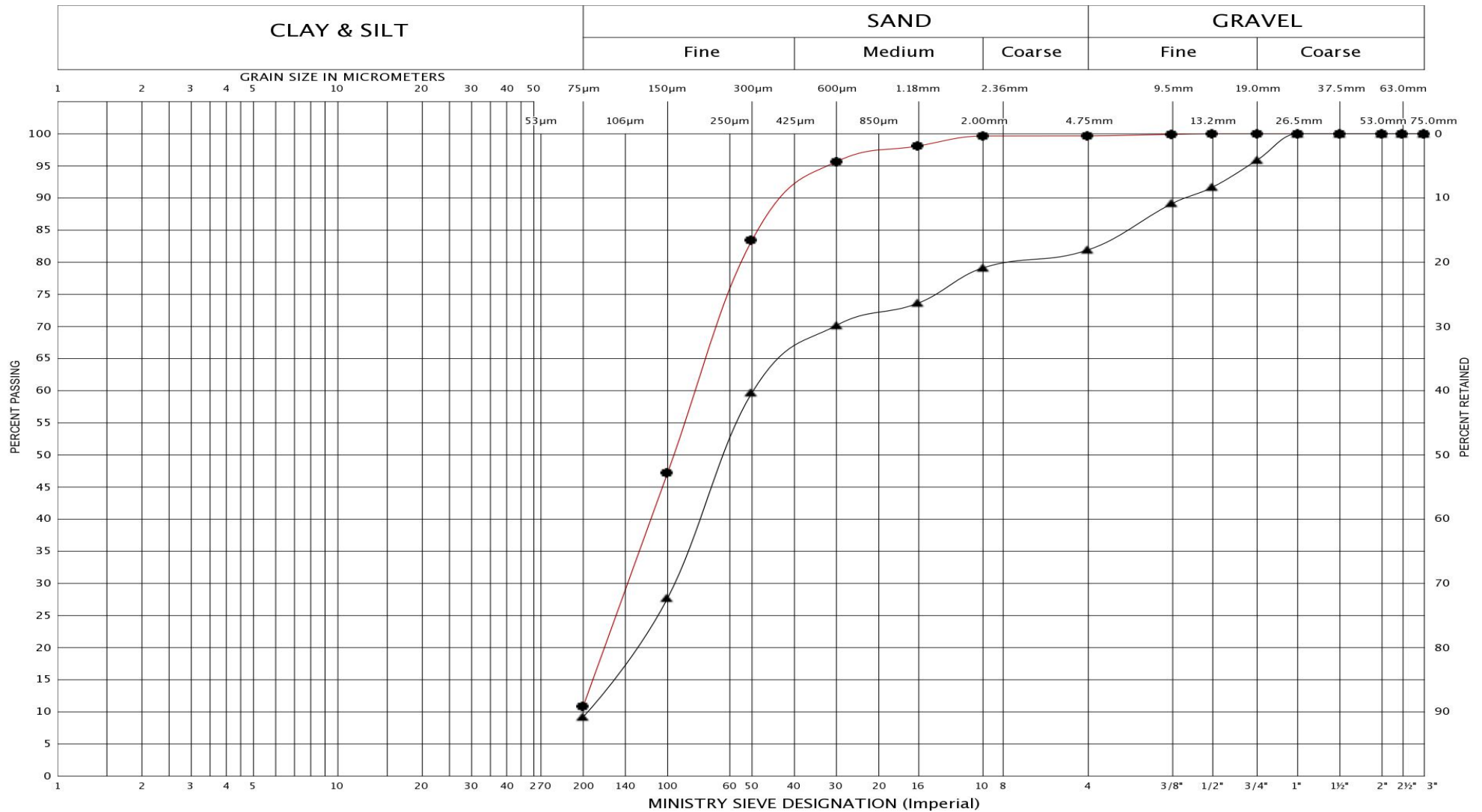
1 OF 1

METRIC

G.W.P. _____ LOCATION Coordinates: 4 919 174 N; 573 366 E (UTM17) ORIGINATED BY NG
 DIST _____ HWY 26 BOREHOLE TYPE Hollow Stem Augers COMPILED BY NR
 DATUM Geodetic DATE 2022.07.25 LATITUDE 44.422142 LONGITUDE -80.078358 CHECKED BY NR

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/ GAS READING γ _{kN/m³} / ppm/%	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								○ UNCONFINED + FIELD VANE												
								● QUICK TRIAXIAL × LAB VANE												
212.6	GROUND SURFACE						20	40	60	80	100									
0.0	PAVEMENT: 180 mm asphalt, over 150 mm granular base, over 200 mm granular subbase, moist		1A	GS	-															
212.1			1B	GS	-															
0.5	SAND, some silt to silty, trace to some gravel																			
	Loose, Brown, Moist		2	SS	7															
211.2																				
1.4	SAND, trace to some silt																			
	Compact, Brown, Very moist to wet		3	SS	16															
210.2	(FILL)																			
2.4	SILT, trace to some sand, some clay		4	SS	11															
	Compact, Brown, Moist to very moist																			
209.3																				
3.3	CLAYEY SILT, trace sand, trace gravel		5	SS	12											0 8 76 16				
	Stiff, Brown to grey, APL		6	SS	3											2 11 44 43				
			7	VANE	-															
207.1																				
5.5	SILTY CLAY, trace sand, trace gravel		8	SS	3											2 7 42 49				
	Stiff, Grey, APL																			
206.2			9	VANE	-															
6.4	SANDY SILT, trace to some gravel																			
	Dense to very dense, Grey, Wet		10	SS	48															
			11	SS	83															
			12	SS	54															
203.6																				
9.0	End of borehole																			
	Notes: 1. First water strike noted at 2.3 m during drilling. 2. Wet cave was noted at 3.7 m upon completion of drilling and following the removal the hollow augers 3. Where in-situ fieldvane test was carried out, the consistency was based on the shear strength and was not based on SPT N value.																			

UNIFIED SOIL CLASSIFICATION SYSTEM



LEGEND	BH	5	6
	SAMPLE	3	2
	SYMBOL	●	▲



GRAIN SIZE DISTRIBUTION

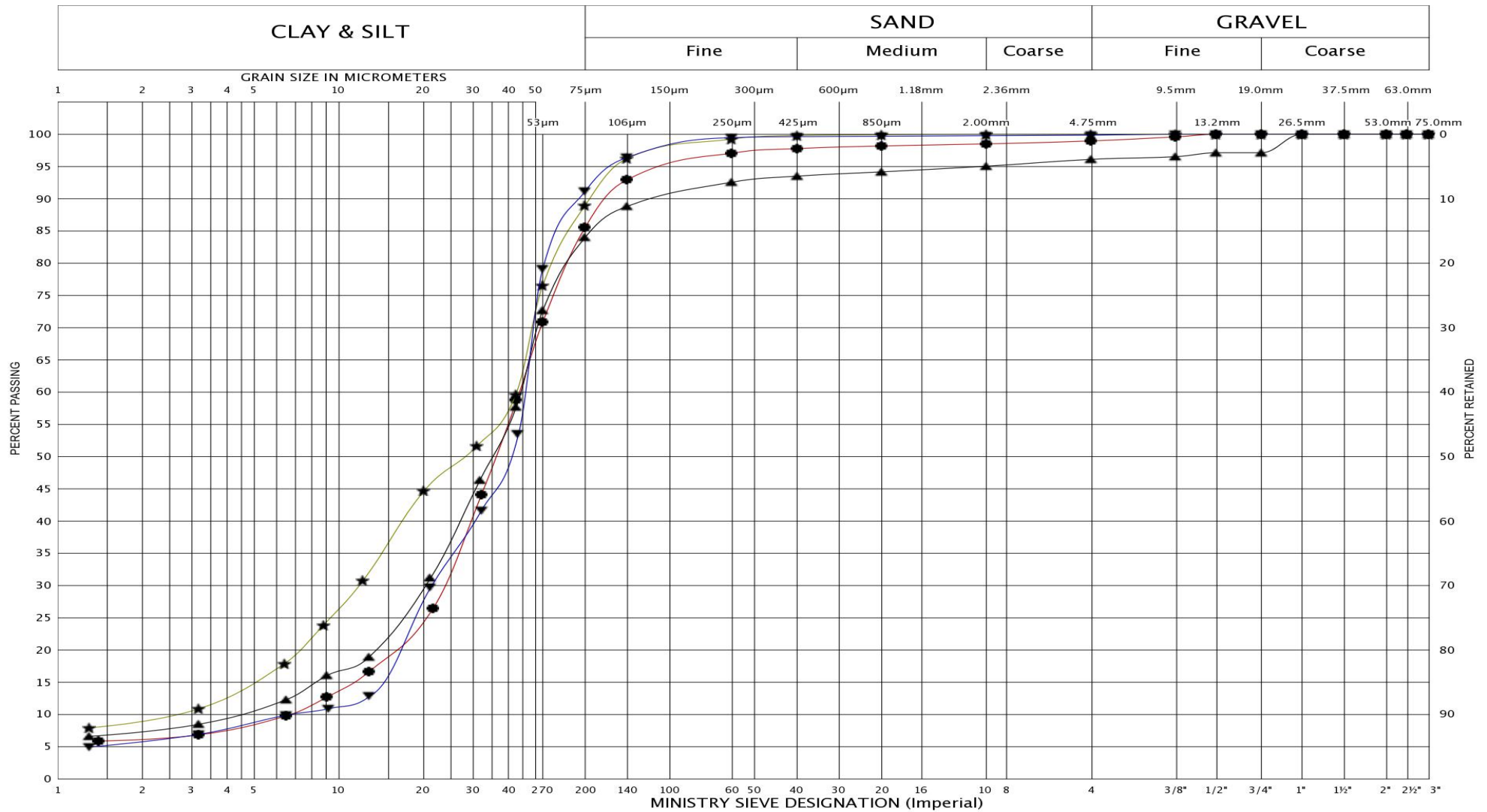
FILL: Sand, Trace/Some Silt, Trace/Some Gravel

FIG No.: 3-1

HWY : 26

Project No.: 21CF013

UNIFIED SOIL CLASSIFICATION SYSTEM



LEGEND	BH	1	5	5A	5A
	SAMPLE	5	6	4	7
	SYMBOL	●	▲	★	▼



GRAIN SIZE DISTRIBUTION

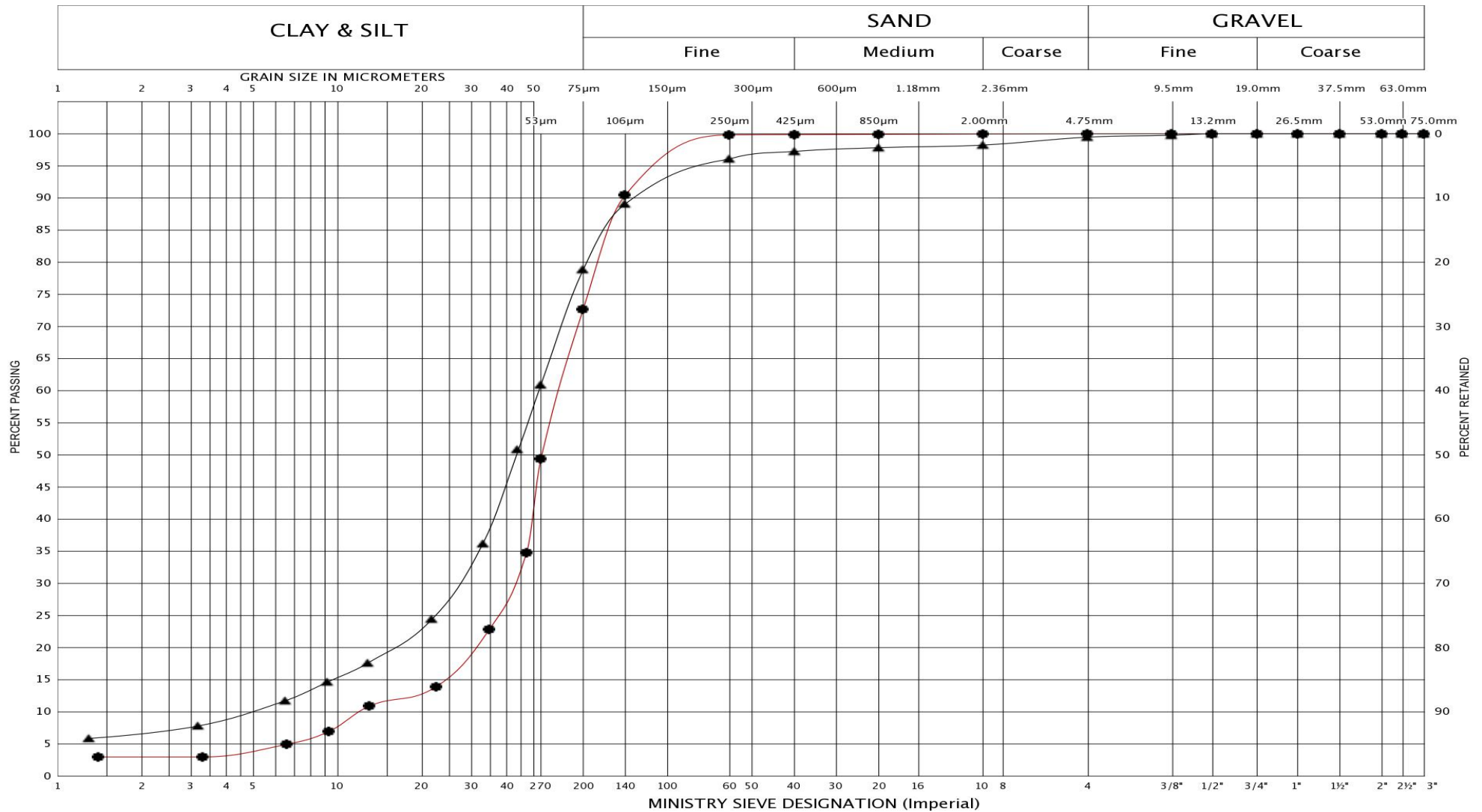
Silt, Some Sand, Trace Clay, Trace Gravel

FIG No.: 3-2

HWY : 26

Project No.: 21CF013

UNIFIED SOIL CLASSIFICATION SYSTEM



LEGEND	BH	1	3
	SAMPLE	7	7
	SYMBOL	●	▲

GRAIN SIZE DISTRIBUTION

SANDY SILT, Trace Clay

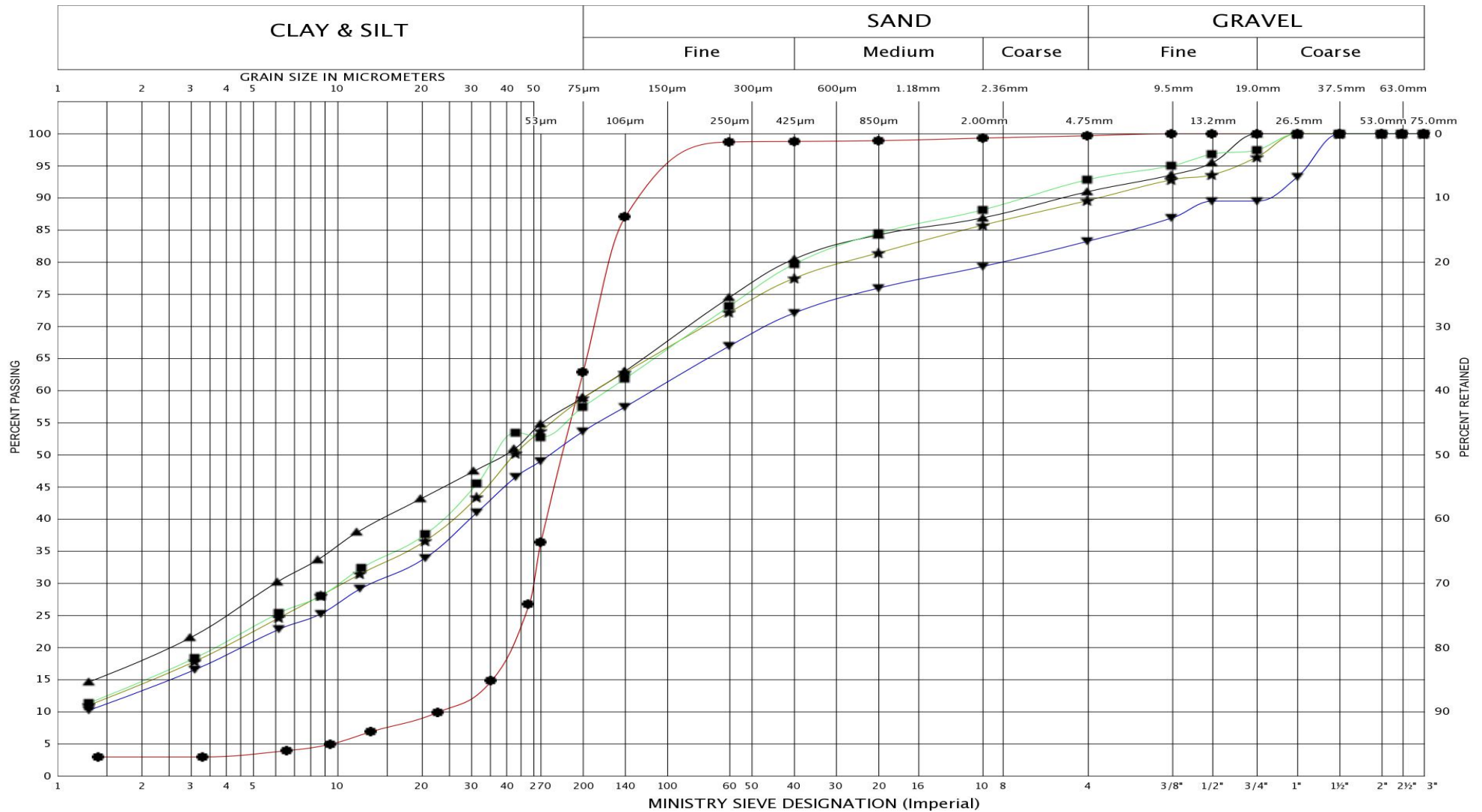
FIG No.: 3-3

HWY : 26

Project No.: 21CF013



UNIFIED SOIL CLASSIFICATION SYSTEM



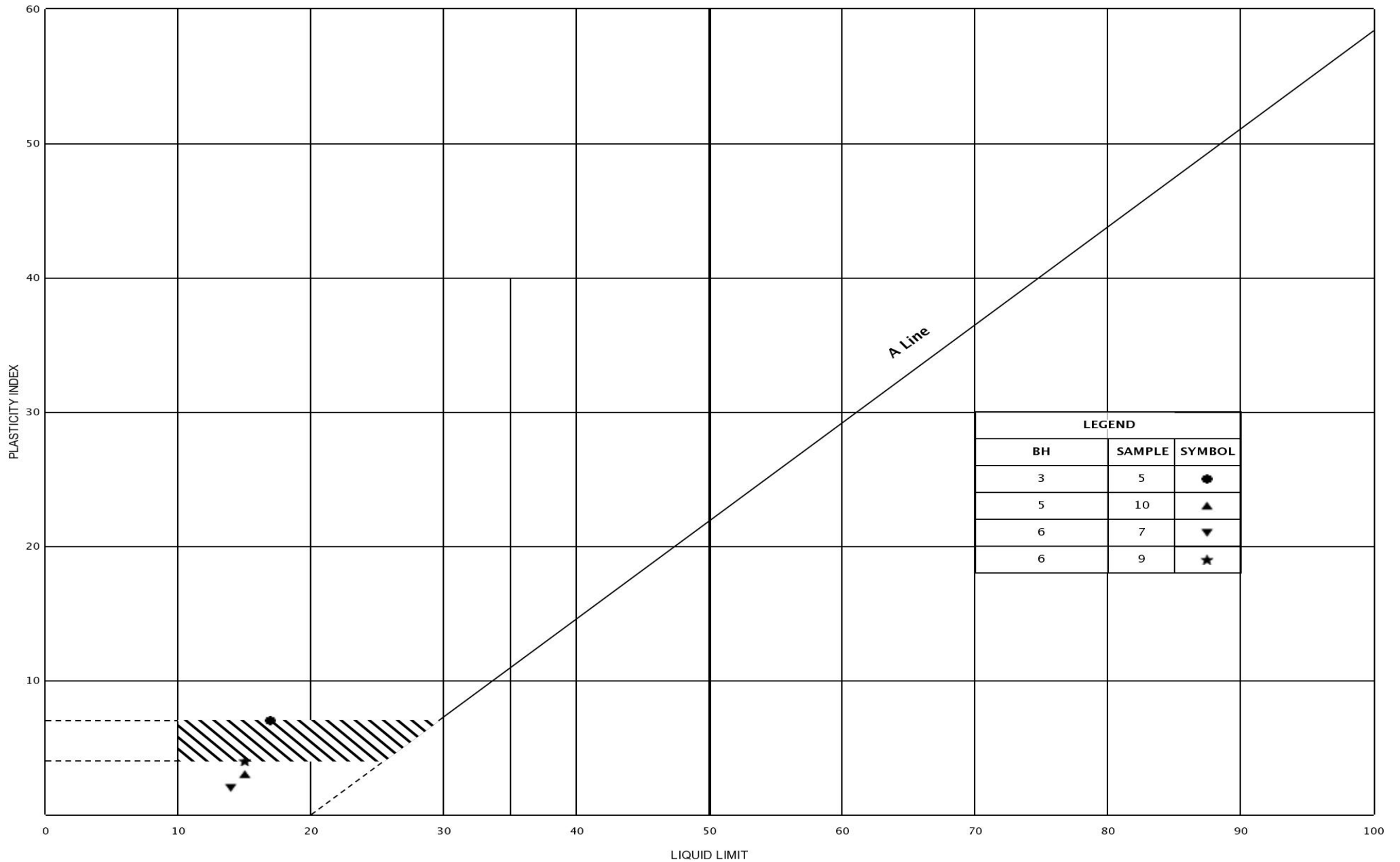
GRAIN SIZE DISTRIBUTION

SILT AND SAND, Trace/Some Clay, Trace/Some Gravel

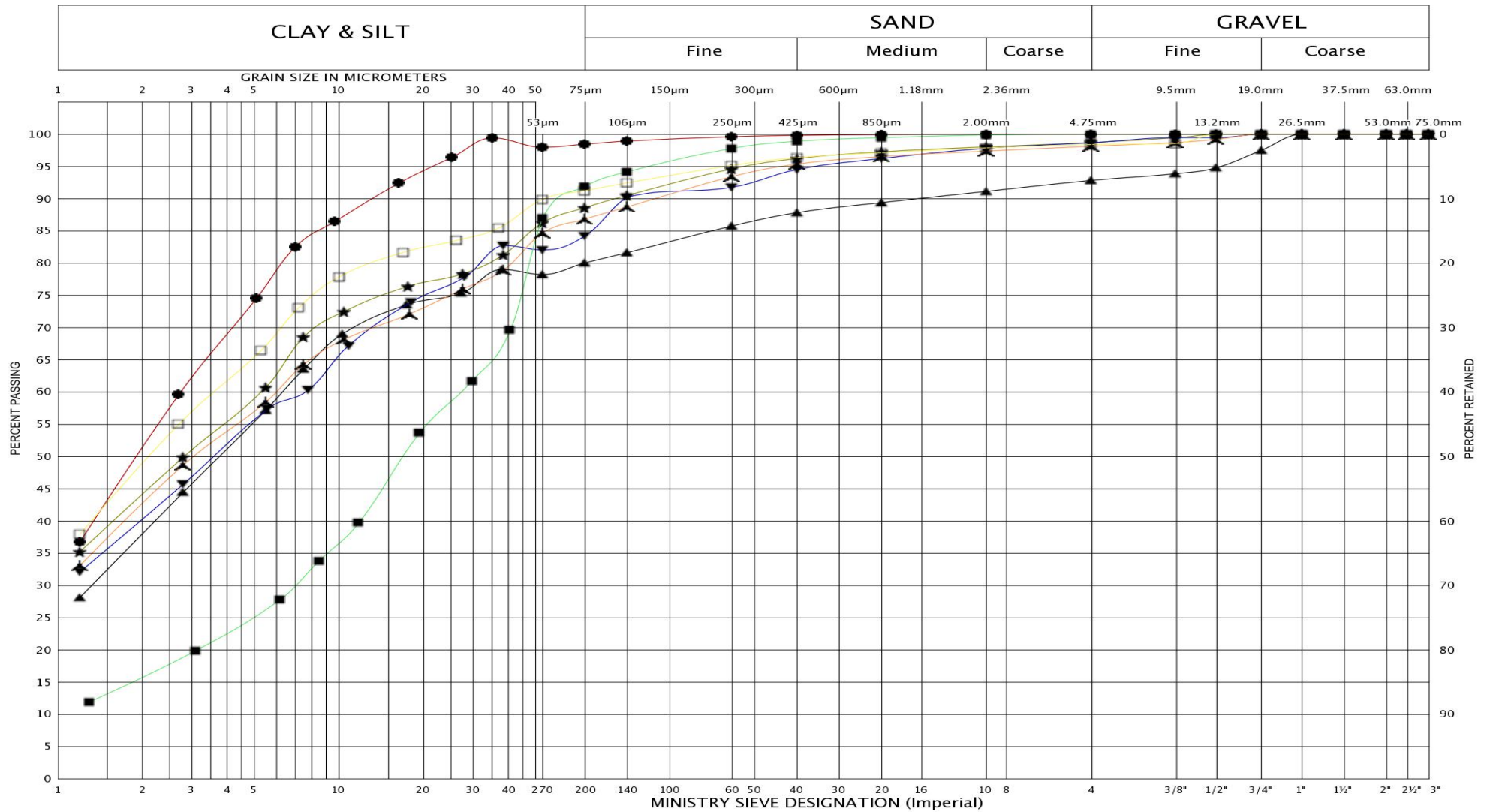
FIG No.: 3-4

HWY : 26

Project No.: 21CF013



UNIFIED SOIL CLASSIFICATION SYSTEM



LEGEND	BH	3	5	5A	6	7A	7A	7A
	SAMPLE	11	8	6	5	5	6	8
	SYMBOL	●	▲	★	▼	■	▲	□



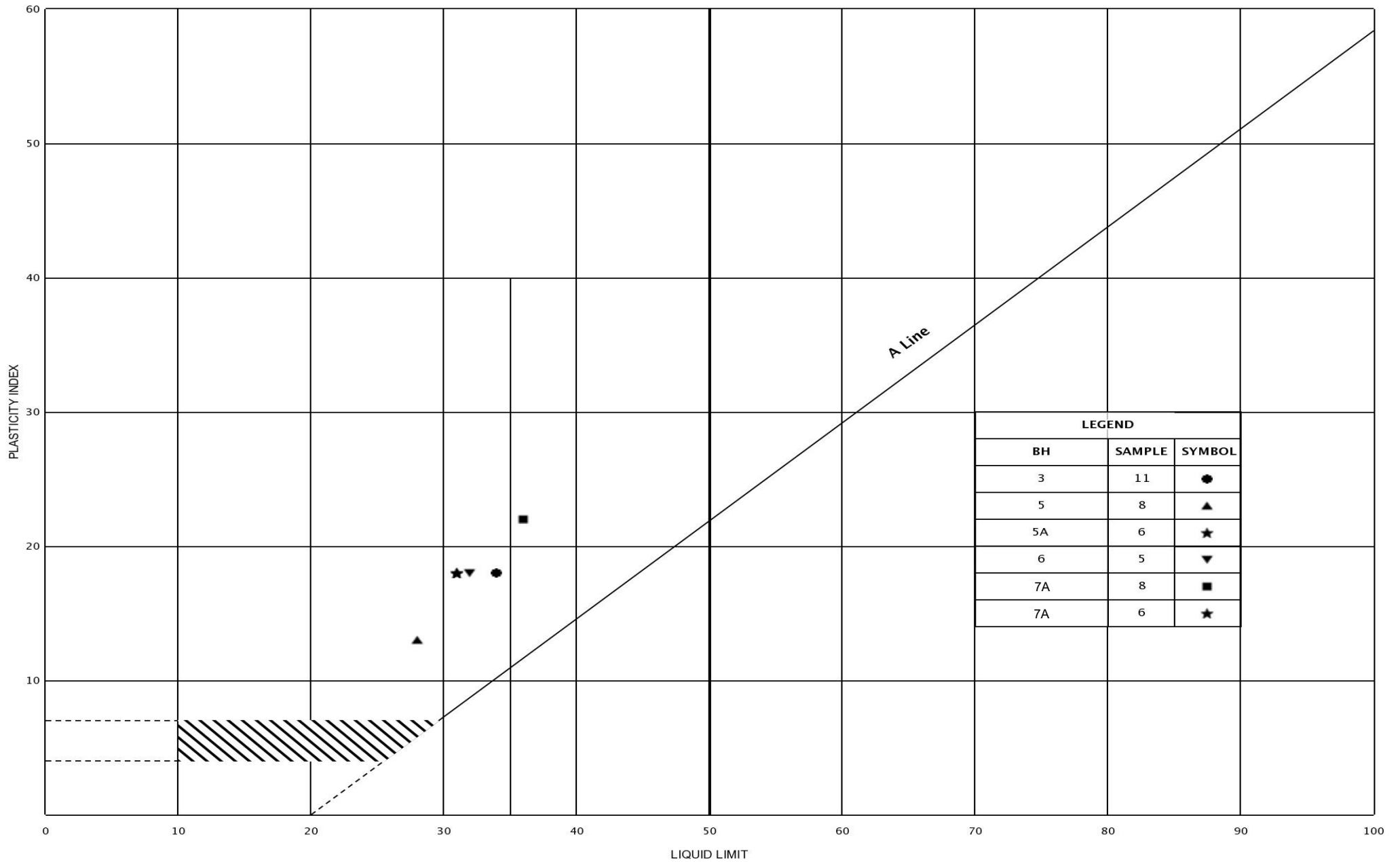
GRAIN SIZE DISTRIBUTION

Clayey Silt/Silty Clay, Trace/Some Sand, Trace gravel

FIG No.: 3-6

HWY : 26

Project No.: 21CF013



C.O.C.: GH4494

REPORT No. B21-38115

Report To:

Peto MacCallum Ltd

19 Churchill Drive,

Barrie ON L4N 8Z5

Attention: Geoff White

Caduceon Environmental Laboratories

112 Commerce Park Drive

Barrie ON L4N 8W8

Tel: 705-252-5743

Fax: 705-252-5746

DATE RECEIVED: 19-Nov-21

JOB/PROJECT NO.:

DATE REPORTED: 25-Nov-21

P.O. NUMBER: 21CF013

SAMPLE MATRIX: Soil

WATERWORKS NO.

			Client I.D.	BH 1 SS 3	BH 3 SS 2		
			Sample I.D.	B21-38115-1	B21-38115-2		
			Date Collected	12-Jul-21	12-Jul-21		
Parameter	Units	R.L.	Reference Method	Date/Site Analyzed			
pH @25°C	pH Units		MOEE3530	19-Jul-21/R	7.86	8.05	
Resistivity	ohms·cm		SM 2510B	21-Jul-21/O	3890	395	
REDOX potential	mV		In-House	23-Nov-21/R	316	313	
Chloride	µg/g	5	SM4110C	22-Nov-21/O	92	1750	
Sulphate	µg/g	10	SM4110C	22-Nov-21/O	20	80	
Sulfide	µg/g	0.4	In-House	25-Nov-21	< 0.3 ¹	< 0.4 ¹	

¹ Subcontracted to Testmark Labs



R.L. = Reporting Limit

Test methods may be modified from specified reference method unless indicated by an *

Site Analyzed=K-Kingston,W-Windsor,O-Ottawa,R-Richmond Hill,B-Barrie

Christine Burke
 Lab Manager

The analytical results reported herein refer to the samples as received. Reproduction of this analytical report in full or in part is prohibited without prior consent from

[illegible]



PART B – FOUNDATION DESIGN REPORT

for

**PROPOSED SANITARY SEWER AND WATERMAIN
CROSSING HIGHWAY 26
STAYNER, ONTARIO**

LATITUDE: 44.422285; LONGITUDE: -80.078242

PETO MacCALLUM LTD.
25 SANDFORD FLEMING DRIVE
UNIT 2
COLLINGWOOD, ONTARIO
L9Y 5A6
PHONE: (705) 445-0005
EMAIL: collingwood@petomaccallum.com

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PML Ref.: 21CF013
Report: 3
August 30, 2022



TABLE OF CONTENTS

PART B - FOUNDATION DESIGN REPORT

7. INTRODUCTION	12
8. DISCUSSION AND RECOMMENDATIONS.....	12
8.1 General	12
8.2 Proposed Installation.....	13
8.3 MTO Requirements and Policy for Encroachments and Utilities	14
9. SUBSOIL CONDITIONS.....	15
10. FROST PENETRATION DEPTH	16
11. OPEN CUT INSTALLATION OF SANITARY SEWER	16
11.1 General	16
11.2 Excavation and Temporary Protection System	16
11.3 Ground Water Control	19
11.4 Installation of Sanitary Sewer.....	20
12. INSTALLATION USING TRENCHLESS TECHNOLOGY	20
12.1 Selection of Installation Method.....	20
12.1.1 Jack and Bore.....	22
12.1.2 Pipe Ramming	24
12.1.3 Horizontal Directional Drilling (HDD).....	25
12.1.4 Micro-tunneling	26
12.1.5 Comparison of Alternate Trenchless Methods.....	27
12.1.6 Recommended Method	29
12.1.7 Ground Classification Discussions.....	29
12.1.8 Anticipated Settlement.....	30
12.1.9 Entry and Receiving Pits.....	30
12.2 Seismic Zone and Site Response	32
12.3 Soil Corrosivity	32
12.4 Ground Water Control at the Pits	33
13. CONSTRUCTION CONSIDERATIONS.....	34
13.1 Settlement Monitoring	34
14. CLOSURE.....	37



Appendix C –

Copy of Drawings provided by R.J. Burnside & Associates Limited (Drawing No. C-005 Plan and Profile Phillips Street STA. 4+230 to STA. 4+ 329 and Drawing No. C-006 Plan and Profile Highway 26 STA. 1+151 to STA. 1+274).

Copy of Ministry of Transportation's "Guidelines for Foundation Engineering – Tunnelling Specialty for Corridor Encroachment Permit Application"

DWG A – Settlement Monitoring Plan

DWG B – Settlement Instrumentation

PART B — FOUNDATION DESIGN REPORT

for
Proposed Sanitary Sewer and Watermain
Crossing Highway 26
Stayner, Ontario

7. INTRODUCTION

This foundation design report with the interpretations and recommendations are intended for the use of the Township of Clearview (the Town) and R.J. Burnside & Associates Limited (RJB), and shall not be used or relied upon for any other purposes or by any other parties. Where comments are made on construction, they are provided only to highlight aspects, which could affect the design of the structure. Contractors must make their own interpretation of the factual data provided in the foundation investigation report (Part A), as it may affect equipment selection, proposed construction methods and scheduling.

8. DISCUSSION AND RECOMMENDATIONS

8.1 General

This section of the report provides recommendations for the design of the proposed installation of the proposed 450 mm internal diameter (I.D.) sanitary sewer and 300 mm I.D. PVC watermain in the Township of Clearview, Simcoe County, Ontario. The proposed sanitary sewer is anticipated to be installed by open cut method for a length of about 108 m from Mowat Street North to Phillips Street along the Highway 26 north embankment, and then across Highway 26 by a trenchless method at the intersection with Phillips Street. The installation of the proposed 300 mm I.D. PVC watermain is anticipated to be carried out by a trenchless method across Highway 26, at the intersection with Phillips Street.

The recommendations are based on interpretation of the factual information obtained from the boreholes drilled during the site investigation. The discussions and recommendations presented are intended to provide information to the designer of the casing and watermain/sanitary sewer pipes, and identify the geotechnical constraints, for the proposed installation of the services along the highway, and crossing the highway by trenchless methods.

The profile and detail drawings provided by R.J. Burnside & Associates Limited and utility locates carried out prior to the field work, identified various underground utilities located within the Highway 26 ROW, where the proposed watermain and sanitary sewer are planned. Prior to

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construction, a survey to locate the underground utilities and buried structures should be undertaken and the influence of the existing facilities to the proposed services should be evaluated. Further, the existing underground utilities, may impose potential conflict with the proposed alignment of the services and precautionary measures to prevent damage must be arranged with the respective utility owners.

The scope of foundation investigation work carried out by PML does not cover or include accessing beyond the location of boreholes and easement requirements. Therefore, the specialty contractor and owner of the services (Township of Clearview) should confirm the existence of any utility or obstructions that may impose potential conflict with the proposed alignment and advised to obtain necessary permits, prior to the commencement of construction.

Based on our assessment and observations of the site during the field work. It is considered as a preliminary assessment, there is satisfactory space for setback from the road on the south and north side of the highway to set up entry and exit pits for trenchless installation. Local businesses will have to be contacted to arrange for access for pits on the north side of the highway.

8.2 Proposed Installation

Based on information provided by RJB, the Town is proposing to construct a watermain and a sanitary sewer crossing under Highway 26 to upgrade connections/serviceability.

RJB has provided drawings pertaining to the installation of the watermain and sanitary sewer to PML via email on January 27, 2022. Copy of the drawings are appended in Appendix C.

Based on the drawings, the 450 mm I.D. PVC sanitary sewer pipe will be installed by open cut method for 107.9 m in length along Highway 26 north embankment, beyond the edge of pavement, from manhole (MH) 930 to MH 175. Refer to Drawing No. C-006 for details. From south side of Highway 26 and Phillips Street intersection, trenchless method will be utilized to install the sanitary sewer pipe to the north side of Highway 26. The proposed sanitary sewer crossing will comprise an approximately 16 m long casing with approximate invert elevation 208.2, about 4.3 m to 4.6 m below Highway 26 travel lanes. The pipe will eventually be connected from MH 176 to MH 175. Refer to Drawing No. C-005 in Appendix C for details.

The proposed 300 mm I.D. PVC watermain crossing will comprise an approximately 16 m long casing with approximate invert elevation 209.1, about 3.5 m to 3.7 m below Highway 26 travel



lanes. Trenchless method will also be utilized to connect the proposed watermain crossing from south of Phillips Street intersection to the north across Highway 26 travel lanes. The proposed pipe will be connected to existing 300 mm watermain on the south and north of Highway 26. Refer to Drawing No. C-005 in Appendix C.

Staging pits for the trenchless installation of the proposed sanitary sewer and watermain will be as shown in Drawing Nos. C-005 and C-006. The same launch and receiving pits will likely be used for the proposed sanitary sewer and watermain due to the proximity of the pipes. The sanitary sewer will be connected to proposed Manhole 176 on the south side of Highway 26 and to the proposed Manhole 175 on the north side of Highway 26, and to Manhole 930 on Mowat Street.

8.3 MTO Requirements and Policy for Encroachments and Utilities

As the project involves the crossing of Highway 26, the investigation must comply with the Ministry of Transportation (MTO) "Guidelines for Foundation Engineering - Tunnelling Specialty for Corridor Encroachment Permit Application" dated February 2021, a copy of which has been provided in Appendix C. This foundation design report has been prepared as per the project requirements and the above noted MTO guidelines.

Reference is also given to the Transportation Association of Canada (TAC) Guidelines for Underground Utility Installations Crossing Highway Rights-of-Way, dated March 2013.

MTO does not permit open cut or trenching for installation of utility pipe or casing across the highway corridor, except where in the opinion of the Field Service Engineer other methods are not possible because of the size of the pipe or the nature of the subsoil conditions. Entry and receiving pits are required to be located at the bottom of the ditch line and back slope of the ditch. In a fill area, pits should be located beyond the toe of the slope of embankment. Due to existing gas main and culvert conflict, only 2.0 m offset from the edge of the Highway 26 pavement can be provided to the pit location of the north side. Refer to Drawings C-005 and C-006.

The standard depth of cover for buried utility pipes under the travelled portion of the highway should not be less than 1.2 m. In Southern Ontario, the depth of cover for buried pipes should not be less than 0.75 m below the bottom of highway ditch. TAC requires 1.8 m of cover below the paved area and 1.2 m of cover in the ditch area. Referring to Drawing C-005, the proposed depth of cover along the alignment of the proposed sanitary sewer meets the MTO requirements for encroachments and utilities



below the travelled lanes and ditches, and meets the TAC requirements. The proposed cover for the watermain meets the MTO requirements for encroachments and utilities below the travelled lanes and ditches, and meets the TAC requirements.

9. SUBSOIL CONDITIONS

Boreholes 1, 3, and 5 along the proposed sanitary alignment on the north side of Highway 26 contacted a sand fill beneath the pavement/shoulder granular, overlying native units of silt, sandy silt, clayey silt and silt and sand. First strike ground water was noted at 1.4 to 2.9 m depth (elevation 210.2 to 211.0) with stabilized ground water in the wells measured at 1.1 and 6.6 m depth (elevation 212.0 and 205.8).

The proposed sanitary sewer north of Highway 26 will have about 4.0 m of cover, and with pipe invert dropping from elevation 208.2 in the east (MH175) to elevation 207.9 in the west (MH 930). The open cut installation will encounter the pavement structure (at MH 930), fill, and silt/silty sand/sandy silt. The invert will be below the perched water throughout, and will be above the stabilized water level in the east and within the stabilized ground water level in the west.

The subsurface conditions encountered in the area of the crossing (Boreholes 5, 5A, 6 and 7A) typically comprises sand fill underlain by native units of silt and clayey silt over a silt and sand deposit. First strike ground water was noted at 1.4 to 2.3 m depth (elevation 210.2 to 211.1) with stabilized ground water in the wells installed in Boreholes 5 and 6 at 6.6 and 8.4 m depth (elevation 205.8 and 208.9), respectively.

The proposed watermain pipe will have about 3.0 m to 3.1 m of cover from Highway 26 existing surface as shown in Drawing Nos. C-005 with invert at about elevation 209.1. The proposed watermain will generally anticipate to encounter silt (Borehole Nos. 5 and 5A) at the north end and silt and clayey silt (Borehole Nos. 6 and 7A) at the south end. The invert will be below the perched water and above the stabilized water level in the wells.

The proposed sanitary sewer crossing will have about 3.8 m to 3.9 m of cover from Highway 26 existing surface as shown in Drawing Nos. C-005 with invert at about elevation 208.2. The installation will generally anticipate to encounter silt and clayey silt (Borehole Nos. 5 and 5A) at the north end and clayey silt (Borehole No. 6 and 7A) at the south end. The invert will be below the perched water and above the stabilized water level in the wells.



10. FROST PENETRATION DEPTH

In accordance with OPSD 3090.101, the frost penetration depth for design purposes in the area where the site is located is 1.6 m. The average annual freezing index in the area is in the range of 750 degree days Celsius.

11. OPEN CUT INSTALLATION OF SANITARY SEWER

11.1 General

The proposed 450 I.D. mm diameter PVC sanitary sewer pipe will be installed by open cut or trenching method approximately from 1+152.1 to Station 1+260 (approximately 107.9 m linear length). Refer to Drawing No. C-006. Boreholes 1, 3 and 5 were investigated within the portion of the watermain by open cut method. The proposed invert of the sanitary sewer will be from elevation 208.2 in the east (MH175) to elevation 207.9 in the west (MH 930) along north side of Highway 26, beyond edge of pavement. The open cut or trenching method will be carried out within the Ministry of Transportation of Ontario (MTO) Right-of-Way.

11.2 Excavation and Temporary Protection System

General Reference is given to Ontario Provincial Standard Specifications (OPSS). PROV. 201, 490 and 801 for specifications associated with site preparation.

Prior to excavation, the locations and depths of existing underground utilities should be verified by the Contractor. All underground utilities that might be exposed and become unsupported as a result of the excavation should be properly supported to avoid potential damage. If there may be conflicts it may be necessary to consider relocation of the existing underground utilities.

For the open cut or trenching method section, excavation is anticipated to extend through the pavement structure, fill and native overburden consisting of silt/silt and sand/silty sand/sandy silt to about 4.5 m to 5.5 m below the ground surface at Boreholes 1, 3 and 5 locations. The table below summarizes the borehole locations, ground surface elevation, proposed invert depth, anticipated excavation depth, and anticipating bearing soils at the invert level. It should be noted that the final base of excavation will be lower than the pipe invert to allow for pipe bedding



materials and any subgrade excavation due to local poor soils as approved by the Engineer. Pipe embedment shall be in accordance with OPSD 802.010.

Summary of Invert, Excavation Depth and Soil

	NEAREST BOREHOLE NO. ALONG THE OPEN CUT INSTALLATION ALIGNMENT		
	1	3	5
Ground Surface Elevation (m)	213.1	213.0	212.4
Proposed Approximate Sanitary Sewer Invert Elevation (m)	207.9	208.0	208.2
Anticipated Excavation Depth to Subgrade Level, (m) ¹	5.5	5.3	4.5
Anticipated Bearing Subgrade Soils	Dense to Compact Sandy Silt	Dense Sandy Silt	Compact Silt

Note: (1) – The excavation depth includes 300 mm depth below invert level for bedding purposes.

Any fill, spongy or soft area observed within the base of the excavation should be removed and replaced with suitable fill material and compacted in accordance with OPSS 401.

All work should be carried out in accordance with the Occupational Health and Safety Act (Ontario Regulation 213/91) and with MTO/the Town regulations.

Excavated material shall not be stockpiled in the areas immediately adjacent to the top of the excavation slopes. 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.

According to the Ontario's Occupational Health and Safety Act, Ontario Regulation 213/91 amended to regulation 628/05, the existing fill and compact soils can be classified as Type 3 soils. Dense soils can be classified as Type 2 soil. Soils below the groundwater level which take on the characteristics of a Type 4 soil should be classified as Type 4 soils. Open cut excavations are governed by soils with the highest soil type number. For excavations through multiple soil types, the side slope geometry is governed by the soil with the highest number designation. The slope of excavation walls should



conform to as described in Ont. Reg. 213/92, S. 234. Temporary shoring systems may be required if slopes as described in Ont. Reg. 213/92, S. 234 cannot be provided.

If excavations steeper than approximately 1H:1V are required and open excavation is restricted such that a road protection system is required, it should be designed to meet minimum Performance Level 2 in accordance with OPSS 539 and SSP 105S09. It should be noted that the use of shoring may conflict with existing utilities in some locations. The Contractor is responsible for the selection, design, construction, and performance of the temporary protection systems. It should be noted that temporary shoring installed within MTO ROW should be removed after the pipe backfill and installation has been completed. Geotechnical parameters provided in the table below may be used for the temporary protection systems.

Soil Parameters

ELEVATION (m)		SOIL TYPE	SOIL PARAMETERS *		
FROM	TO		FRICTION ANGLE (Ø')	UNIT WEIGHT (γ) kN/m ³	C _u , (kPa)
Borehole 1					
212.7	211.0	Sand (Fill)	28	19	-
211.0	208.8	Silt	28	19	-
208.8	207.1	Sandy Silt	30	19	-
207.1	203.5	Silt and Sand	32	20	-
Borehole 3					
212.7	210.1	Sand (Fill)	28	19	-
210.1	208.7	Silt and Sand	32	20	-
208.7	206.0	Sandy Silt	30	19	-
206.0	203.4	Clayey Silt	24	19	150
Borehole 5					
211.9	210.0	Sand (Fill)	28	19	-
210.0	207.4	Silt	28	19	-
207.4	206.4	Clayey Silt	20	19	100
206.4	202.8	Silt and Sand	32	20	-

Note: Submerged unit weight should be used below the water level.



If the shoring system does not provide for drainage, the groundwater pressure should be added to the lateral earth pressure for design

11.3 Ground Water Control

First strike ground water was noted at 1.4 to 2.9 m depth (elevation 210.2 to 211.0) in Boreholes 1, 3 and 5, with stabilized ground water in the wells installed in Boreholes 1 and 5 at 1.1 m and 6.6 m depth (elevation 212.0 and 205.8), respectively.

A dewatering scheme together is required to lower the water level a minimum of 0.5 m below the base of excavation. Alternatively, water-tight shaft construction could be employed. The hydraulic head and ground water inflow must be properly controlled to ensure a stable and safe excavation and to facilitate construction. Construction during the dry time of the year is also recommended in order to reduce the ground water control requirements.

The contractor should be responsible for the selection, performance and detailed design of the dewatering system. The dewatering system should be designed to conform to the requirements of OPSS 517 and SSP 517F01.

In accordance with SSP 517F01, the dewatering system should be designed by a designer with a minimum 5 years of experience in the field. A preconstruction survey of 20 m in all directions along the open cut section should be considered due to depth of dewatering required, and existing private properties.

Water taking in Ontario is governed by the Ontario Water Resources Act (OWRA) and the Water Taking and Transfer Regulation O.Reg. 387/040. Section 34 of the OWRA requires that any one taking more than 50,000 L/d to notify the Ministry of Environment, Conservation and Parks (MECP). This requirement applies to all withdrawals, whether for consumption, dewatering for temporary construction or permanent drainage improvements. Projects that are assessed to be taking more than 50,000 L/d but less than 400,000 L/d of ground water can obtain a permit/permission online via the Environmental Activity and Sector Registry (EASR) system. If it is assessed that more than 400,000 L/d is required, then a Category 3 PTTW will be required.

In general pumping less than 50,000 L/d is anticipated from the pits with sheet piling in place. If deeper excavation is required, the details must be reviewed to assess the need for a PTTW or registry on the EASR.



11.4 Installation of Sanitary Sewer

The installation of the 450 I.D. mm diameter sanitary sewer shall be in accordance with OPSS 410. The native soils consisting of compact to dense silt/sandy silt are considered adequate for support of the sanitary sewer. Where soft or loose and other deleterious materials are encountered at the proposed invert level, the materials should be removed completely and replaced with Granular A material compacted to 100% SPMDD. The bedding material should be Granular A meeting the requirement of OPSS 1010, amended by SS9 110S06. Reference to OPSDs 802.030, 802.031, 802.032 should be made for rigid pipe, and reference to OPSDs 802.010 should be made for flexible pipe with regards to embedment, bedding, cover and backfill.

Trenching, backfilling and compacting shall be in accordance with OPSS 401. The backfill material may be Granular A or Granular B Type II meeting the requirements of OPSS 1010, amended by SSP 110S06. Site restoration following installation should be in accordance with OPSS 492.

The silt/sandy silt layer at the bedding level may be susceptible to disturbance due to construction activity and any ponded water. In order to limit the degradation, it is recommended that the granular bedding be placed on the subgrade within four hours after preparation, inspection and approval of the subgrade.

12. INSTALLATION USING TRENCHLESS TECHNOLOGY

12.1 Selection of Installation Method

Oversize casings are sometimes installed so that a carrier pipe can be inserted within the casing to fine tune or correct the vertical and horizontal alignment.

Open cut or trenching to install the casing does not comply with the MTO policy for encroachments and utilities within the MTO ROW. It should be noted that the MTO Corridor Management Office (CMO) generally requires pressurized pipes under the highway to be encased within a steel liner.

There are a number of trenchless technologies employed in the industry, depending on the site conditions and the size of the casing to be installed. The installation of casings at this site requires boring through wet sand fill over clayey silt/silt for the watermain and silt/sandy silt/clayey silt for the sanitary sewer. Most of the trenchless methods are feasible through the subsoil conditions encountered.



The casing pipes to be installed are assumed to be about 0.5 m to 0.6 m in diameter and Pipe Jacking method is generally unsuitable to install utility pipe of small diameter, and requires a minimum boring diameter of 1.43 m to employ this method.

There appears to be four possible trenchless methods to consider for the installation of the casings. The pipe ramming method is inexpensive and the technology is available for installation of casing with diameter as small as 305 mm. Jack and Bore is also an option, which can be used to install pipes with diameters ranging from as large as 1.8 m to as small as 203 mm and for lengths as much as 150 m. Horizontal Direction Drilling (HDD) is also an option to be considered. This method can be used to install pipes with diameter ranging from 50 to 1200 mm. Micro-tunnelling can be employed to advance the casing; scheduling and cost efficiency will need to be considered.

Once the specifics and installation method have been confirmed, the general requirements for the installation may be addressed with a site specific Non-Standard Special Provision (NSSP). PML will be able to assist in developing a site specific NSSP, if required and as requested, with reference to MTO NSSP "Pipe Installation by Trenchless Method", June 2021. General reference is also made to OPSS415, Construction Specifications for Pipeline Installation by Tunneling, amended by SSP 415S01.

As recommended by MTO guidelines, all the practical installation methods were to be considered and evaluated. The four feasible trenchless methods, i.e., Jack and Bore, HDD, Pipe Ramming, and Micro-tunnelling, are discussed in this report. A comparison of the technical advantages and disadvantages of the four trenchless methods, for the installation of proposed casing, is presented in Section 12.1.5 of this report.

It should be noted that the following discussion and recommendations are presented in relation to the crossing details provided by RJB. Alternative configurations and installation methods may also be considered, provided that they meet the needs of the project proponent and the MTO policy for encroachment. Further, the recommendations presented are based on the boreholes drilled along the currently proposed alignment. Additional subsurface investigation will be required if the crossing alignment is altered or shifted. Regardless of the method used, it is recommended that the contractor prepare a plan in advance of construction outlining the details of the installation to provide instructions for the construction crews and provide a possible contingency action plan should difficulties occur during the tunnelling operations. Since the tunnelling process should be



continuous, any stoppage in the tunnel advancement under the travelled portion of Highway 26 must be avoided and a mitigation action plan prior to tunnelling, provided. The plan should also be reviewed by the project proponent prior to construction. Upon request, PML can assist in reviewing the plan to check the assumptions made for soil and ground water are appropriate.

The presence of buried utilities must be verified, and measures should be implemented to prevent damage.

12.1.1 Jack and Bore

Jack and Bore typically involves the simultaneous advancement of a continuous flight auger and conduit pipe. The auger is used to excavate soil in advance of the casing and transport cuttings back to the jacking/entry pit where they are removed. Rotary power to auger and pushing force is provided by a drill rig located within the jacking/entry pit. Jack and Bore is a common method of trenchless installation and in appropriate site and soil conditions may be preferable from a cost perspective.

The Jack and Bore is generally compatible with a variety of stable soil conditions, although there are geotechnical constraints when used below the ground water table, in running soils (sands), or in soils with cobbles and large boulders.

The Jack and bore method is feasible for the soil at the site. This method is applicable for all types of crossings to install sewer or utility pipes to a maximum length of 150 m, which is longer than the length of the proposed casing of 16 m. The diameter of the proposed casings is expected to be about 1 m, which is larger than the minimum diameter of 203 mm required to employ this method. The pipe for employing this method should resist abrasion caused by the rotation of augers and steel is the typical material used, although concrete pipe may also be used in a corrosive environment for buried metallic pipes.

In this method, surface subsidence and heave during installation may pose major problems. Heave occurs when excessive force is applied to the face of boring and surface subsidence occurs when over excavation is permitted. Conversely, if the rate of advance is too slow in loose or wet deposits then the risk of over cut and loss due to raveling at the cutting face increases. The workers are not required to enter the shaft to remove the spoil, however, adequate working space for entry (jacking) and receiving (exit) pits will be required.



The most critical part of this method is positioning the track system on the same line and grade as the bore of the casing. The jacking forces should be estimated to select the appropriate jacking system. Suitable jacking head and bracing between jacks and jacking head should be used to assure that pressure will be applied to the pipe uniformly around the ring of the pipe. The drive shaft should have a stable foundation and an adequate thrust block designed to transmit the horizontal jacking force.

The Jack and Bore method has a limited steering ability, which can affect the line and accuracy of grade. It is typically unguided once it is launched and any subsurface obstructions can cause large deflections. Accuracy of one percent (1%) of the length of drive may be achieved in vertical with a steering head and grade monitoring system combined with good workmanship and suitable equipment. However, horizontal alignment is generally not controlled. As appropriate and if applicable, a guided auger boring system or pilot tube guided system may be employed to facilitate the installations.

Utilization of an effective lubrication system may minimize potential casing friction during advance. A suitable face pressure or soil plug should be maintained to minimize loss of ground during advance. The conventional Jack and Bore system of auger boring is generally open face and in certain soil conditions such as cohesionless running sand and silts, and would necessitate the conventional system to include a closed face articulated steering cutter head equipped with grout injection ports and as necessary, and if feasible, provision of slurry pressure. Any over cutting during augering and casing advance, which may potentially create soil disturbance, space or void outside the casing should be grouted to avoid potential ground movements.

Jack and Bore installation(s) should be conducted in accordance with OPSS 416, Construction Specifications for Pipeline and Utility Installation by Jacking and Boring.

Reference is given to the Excavation and Ground Water Control for Staging Areas for recommendations pertaining to the construction of jacking/entry and receiving pits.

Distress at the ground surface is generally prevented or minimized by proper planning and good construction practices. The contractor should submit a plan for review indicating the planned processes/methods.



The Jack and Bore method is feasible for both the sanitary sewer and watermain crossings across Highway 26 at Phillips Street intersection, with geotechnical constraints to be managed and controlled by the contractor to deal with local ground water seepages and possible wet sand fill.

12.1.2 Pipe Ramming

Pipe ramming installation is analogous to driving an open-ended tube pile horizontally. Impact forces from a percussive hammer are used to advance a conduit pipe from an entry pit to a receiving pit. During the advance, most of the soil being penetrated fills the conduit rather than being excavated. The rammed conduit is terminated in a receiving pit at which point the soil contained in the pipe is removed. When the driving has been completed, soil within the pipe can be removed via augering or a pipe shovel. Augering is typically the preferred method. If soil within the pipe cannot be augured, use of a pipe shovel will be necessary. A pipe shovel is essentially a special scoop made from a pipe which fits inside the liner. Excavation via pipe shovel involves advancing the shovel into the soil plug using impact hammer, then pulling the shovel and its contents out with a chain or cable. This process is repeated as required.

In general, compared to Jack and Bore, less extensive ground water control measures should be needed along the installation path because the soil within the pipe is typically not removed until after the crossing has been completed. The retained soil will tend to act as a plug, reducing the potential for ground water seepage and soil flowing through the pipe. Ground water control via sump pumping will only be required at the entry and receiving pits. Also, pipe ramming is able to accommodate cobbles and boulders more easily than jack and bore, provided that the boulders are small enough to fit inside the casing. However, significantly thicker steel casing is generally required due to the intrinsic driving forces needed to advance the casing using pipe ramming.

The initial set-up is the critical factor in the success of any pipe ramming project. The drive shaft must be located on very stable ground or a concrete slab must be placed below the casing. In this method, the pipe is unguided, therefore the floor of the drive shaft must be engineered to be on the same line and grade as the pipe to provide the accuracy needed. Reference is given to the Entry and Receiving Pits section for recommendations pertaining to the construction of entry and receiving pits.

In the ditch area to the north and south of the highway, the tunneling depth is close to the ground surface with minimal cover. The inadequate cover in some of the ditch areas may result in deflection of the casing during the installation and the pipe may drift upward. In addition,



subsidence of the fill under the road may occur due to the compaction resulting from the vibratory action of the hammer.

In general, it will be more difficult to install the proposed conduits where grades are less than about 1% because of the limited ability to adjust grades during pipe ramming. Pipe ramming does not allow for significant alignment corrections during installation.

Pipe ramming is considered feasible for the proposed installation of casing, however the vibration may be impractical for the local businesses that surround the site.

12.1.3 Horizontal Directional Drilling (HDD)

The HDD method involves the boring and enlargement of an uncased near horizontal tunnel which is kept open through use of drilling fluids. Upon completion of boring, a conduit pipe is pulled through the bore. The process starts by advancing a relatively small diameter hole along the proposed path. During the pilot bore the cutter head at the lead of the drill string is steered by the drill rig. After the pilot hole has been completed, the borehole is enlarged using reaming tools until the desired bore diameter is achieved. The conduit is typically pulled through the borehole on the final reaming pass. Water based drilling fluids containing bentonite and/or polymers are used during the pilot bore and reaming processes to convey cuttings out of the borehole and to stabilize the bore. The bore is typically oversized to minimize the friction resistance along the conduit during installation. During the pull through pass the annulus between the pipe and the surrounding soil is typically filled by displaced drilling fluids that were not displaced out of the bore.

With HDD, there is potential for inadvertent drilling fluid return to the ground surface through hydrofracture of the soil surrounding the bore or if the bore crosses pre-existing fissures/preferential seepage paths. Inadvertent drilling fluid return could cause loss of drilling fluid circulation along the bore which may hinder or prevent completion of an HDD installation. Inadvertent fluid return could be a potential environmental concern if drilling fluid migrated to a wetland environment or other sensitive areas. For the relative shallow cover of the proposed pipes may be a less desirable option since there may be the risk of frac-out of the drilling fluid in the highway ROW if the drilling and fluid pressure are not well controlled. Therefore, mitigation of inadvertent drilling fluid returns should be part of planning and construction for any HDD installation. The HDD contractor should carry out their own tests as necessary to evaluate that the appropriate design and type of fluid is selected for soil conditions.



HDD boring is typically carried out from the ground surface without the use of staging pits, reducing the extent of ground water control, if required. However, the HDD bore requires a long slot approach trench to reach the design invert level and a sizable layout staging area which may not be readily available in the narrow Highway 26 ROW.

HDD installations should be carried out in accordance with OPSS 450, Construction Specifications for Pipeline and Utility Installation in Soil by Horizontal Directional Drilling.

The size of the watermain, length of the drilling run, consistency of the subsurface soils and any potential ground water will dictate the size of the HDD equipment used.

Considering that the bore would be unlined during the HDD process (until the final pass is made and the casing is pulled through), there would be a potential for loss of ground and sink holes to develop at surface. The ground surface over the tunnel route may become distorted and distressed by tunnelling. The most common type of distress is settlement caused by loss of ground around the tunnel. Heave of the ground surface and/or inadvertent drilling fluid returns are also possible.

Distress at the ground surface is generally prevented or minimized by proper planning and good construction practices. The contractor should submit a plan for review indicating the planned processes/methods.

HDD is considered feasible, but with a number of factors to consider as mentioned above, which makes the method less attractive for this project.

12.1.4 Micro-tunneling

Micro-tunnelling involves the advancement of a tunnel boring machine from the jacking pit to the receiving pit. The Micro-Tunnel Boring Machine (MTBM) is remotely controlled and offers good grade control. The tunnel segments are pushed from the jacking pit while line and grade are controlled by the MTBM as it advances. These machines may be designed with, cutter head and hydraulic controlled floors, and also utilize pressurized bentonite slurry to counterbalance the earth and water pressures acting at the tunnel face. More recent Earth-Pressure-Balance (EPB) MTBM do not need a slurry system to balance the pressure at the face of the tunnel. The excavated soil is withdrawn in a controlled manner to prevent loss of ground during tunnel advance. Where a slurry is used, the slurry is circulated back through the tunnel to transport



cuttings to a settling tank. Given the machines ability to control soil and water pressures at the face, dewatering prior to advancing the tunnel would not be necessary with this tunnelling method. However, dewatering of the staging and receiving pits (if required) may still be needed.

Cognizant of the tunnel size, grade requirements, and subsurface conditions, micro-tunnelling may be considered for the proposed crossing. Micro-tunnelling may pose a disadvantage from a cost perspective and will largely depend on availability of a machine of the same size. The method provides better control when encountering potential ground water seepage, and potential obstructions such as potential boulders during tunnelling. The costs and project scheduling may outweigh its benefits and poses risk to the project feasibility. Utilization of micro-tunnelling is therefore not recommended.

12.1.5 Comparison of Alternate Trenchless Methods

The table below summarizes the advantages and disadvantages of the trenchless methods.

TUNNELLING METHOD	ADVANTAGES	DISADVANTAGES
Jack and Bore	<ul style="list-style-type: none"> • Contractor Availability; • Relatively cost effective compared to other methods; • Can accommodate variable soils without major tool adjustment; • Spoils are removed by auger through casing being placed; • Lubricant or drilling fluid is optional and could be applied; • Mixed face conditions may be dealt with by good workmanship for control; • Immediate ground support by casing and temporary face support by auger position/soil cutter head and soil plug. 	<ul style="list-style-type: none"> • Ground water control is required for the entry and exit pits (recommended for drier seasons); • It is typically unguided and horizontal alignment not controlled; • Subsurface obstructions such as large boulders pose problems in advancing and may require new drive path if worker cannot enter to remove; • Requires operators with relatively high skills; • Requires auger removal and additional tools to break up very large boulders, if encountered; • Subsurface obstruction and mixed face may cause deviation from tunnel path (acceptable if sufficient space is available for carrier pipe adjustment).



TUNNELLING METHOD	ADVANTAGES	DISADVANTAGES
Pipe Ramming	<ul style="list-style-type: none"> • Minimal ground water control required along the installation route; • Relatively faster installation than Jack and Bore; • Can advance through soil with cobbles and small boulders; • Low sensitivity to ground water seepage compared to Jack and Bore; • Can accommodate variable soils without major tool adjustment; • Small Staging areas as compared to HDD. 	<ul style="list-style-type: none"> • Ground water control is required for the entry and exit pits (recommended for drier seasons); • Poor grade control (cannot be corrected once installation has started); • Effects of vibrations will have to be assessed/monitored for subsidence; • Require thicker steel casing to withstand driving forces; • High risk of excessive ground loss/settlements/ground heave during casing drive if large obstruction encountered.
HDD	<ul style="list-style-type: none"> • Does not require deep staging pits; • Minimal ground water control required during drilling; • No wet season restrictions; • Grade control can be adjusted during drilling. 	<ul style="list-style-type: none"> • Largest tunnel diameter envisioned is about 1200 mm (large equipment); • Requires long slot trench and layout area; • Potential for inadvertent drilling fluid returns; • Requires drilling fluid to maintain the bore which could allow subsidence; • Potential oval tunnel cross section; • Gravel can hinder grade control; • Steel casing requirement around carrier pipe renders the installation impractical.
Micro-tunnelling	<ul style="list-style-type: none"> • Remotely controlled and positional control is accurate; • Better accuracy on-line and grade compared to other methods; • Some smaller obstructions can be overcome by reverse rotation; • Capable of balancing soil and hydrostatic face pressure, and ground water control may not be required; 	<ul style="list-style-type: none"> • Limited contractor availability; • Significant cost and availability of contractors for the MTBM or EPB MTBM method; • Require larger excavation for entry and exit pits; • Ground water control is required for the entry and exit pits (recommended for drier seasons);



12.1.6 Recommended Method

Cognizant of the, site conditions and grade requirements for the services, it is recommended that Jack and Bore be used for the installation of casings. This method meets with the requirements of MTO policy for encroachments and utilities.

It should be noted that annular void between the steel casing and the carrier pipe (watermain or sanitary pipe) should be pressure grouted after the installations.

As noted previously, the presence of nearby buried utilities must be verified, and measures should be implemented to prevent damage.

12.1.7 Ground Classification Discussions

The trenchless operation for the sanitary sewer will advance through loose to compact silt (Boreholes 5 and 5A) to firm to stiff clayey silt (Borehole 7A) and for the proposed watermain will advance through dense to compact silt (Borehole 5A) to stiff to firm clayey silt (Borehole 6) within the Highway 26 ROW. It is expected that the soil deposits will exhibit different resistances across the boring diameter along the tunnel length.

The Tunnelman's Ground Classification System (Heuer, 1974, Deere et.al. 1969, Terzaghi, 1950) has been used as a basis to describe the anticipated behaviour of the ground. Considering the encountered subsurface soil and groundwater conditions, the soils may be classified as 'slowly ravelling'. The initially "slowly ravelling" ground may change into "rapidly or fast ravelling" ground to 'cohesive running' where less compactness conditions or fissured zones are encountered, requiring the excavation to be supported at the crown, perimeter and face.

The stand-up time for these soils is shown in the table below. The stand-up time is based on the behavioristic classification of various soils by Deere et. al. (1969).

Stand-Up Time for Soil Type

TUNNELMAN'S SOIL TYPE	STAND-UP TIME RANGE
Slow Ravelling	100 minutes to 30 hours
Fast Ravelling	7 minutes to 100 min
Cohesive Running	0.5 minute to 7 min

Clean gravel and sands have practically no stand-up time.



12.1.8 Anticipated Settlement

The empirical calculations based on the available pipe configurations and soils information, and coupled with experiences and case records of trenchless installation indicate that the settlements could be controlled to within acceptable tolerance of about 15 mm settlement over the two pipes provided that the suitable and proper equipment and workmanship has been used. The Designer and Contractor should carry out their own settlement and horizontal displacement calculations based on their own purposes.

The scope of the foundation investigation does not include assessing the impact on the pavement and utilities that may be in the vicinity of the alignment. The contractor will be responsible for any associated impact on the existing structures and underground utilities in the vicinity due to settlement and horizontal displacement along the bore length.

12.1.9 Entry and Receiving Pits

It is anticipated that supported excavations will be utilized for the staging areas considering the requirement for near vertical face to initially advance the pipe into the soil. Considering the proximity of the services, a common pit may be employed on each side of the road.

General Reference is given to OPSS 201, 206 and 490, for specifications associated with site preparation.

Due to the proximity, it is assumed that the staging pits, on both sides, for both services, will be a common pit. Pits are assumed to extend a maximum 1 m below the proposed casing invert. The proposed sanitary sewer invert will govern the depth of the pits. Excavation will extend to an anticipated depth of about 5.0 to 5.5 m depth below existing grade at the crossing.

Excavations, as noted above will extend through surficial pavement and underlying fill, and into the native silt, clayey silt, sandy silt and the silt and sand. Although not encountered in the boreholes, the site borders a geologic area that has beach deposits and the presence of cobbles and boulders should not be disregarded.



Provided adequate ground water control is achieved, the on-site soils are classified as Type 3 material as defined in the Occupational Health and Safety Act (OHSA). Excavations within Type 3 soil that are to be entered by workers, may not be steeper than one horizontal to one vertical (1H:1V) from the base. Workers should not enter an unprotected excavation if there is evidence of ongoing ground water seepage in the banks.

Any unsupported open excavation side slopes should be continuously examined and reviewed for evidence of instability, particularly following periods of heavy rain or thawing. When required, remedial action must be taken to ensure the continued stability of the excavation slope and the safety of the workers.

Based on the water levels observed in the boreholes, excavation described above will encountered the perched water condition at the crossing and the ground water table at the west end of the proposed sanitary sewer alignment. Seepage volume is not anticipated to be large and conventional sump pumping techniques in conjunction with proper designed shoring should suffice to control ground water seepages (OPSS 517 amended by SP 517F01). Further reference is made to Section 11.4.

It is envisioned that sheet piling will be employed to support the excavations, which will aid with ground water control. For design of temporary sheet piling for excavations, the following parameters may be assumed (wall friction ignored):

PARAMETER	FILL	CLAYEY SILT	CLAYEY SILT (BH 6)	SILT/SILT AND SAND/SANDY SILT
Angle of Internal Friction, ϕ , (degrees)	28	--	--	30
Shear Strength, S_u (kPa)	--	150	5	--
Bulk Unit Weight (kN/m ³)	20	19	19	20

The shoring system should be designed by an experienced Professional Engineer and should also include design check for basal stability, and it is recommended that the shoring system should extend at least about 2 to 2.5 m below the base of excavation into native dense materials.



The performance level for the temporary protection system should be OPSS 539 level 2 (amended by SSP 105S09). Any sheet piling within the MTO ROW will have to be removed once the installation is complete.

The temporary exist and entry pits after the completion of pipe installations shall be decommissioned by backfilling with unshrinkable fill.

12.2 Seismic Zone and Site Response

The site specific spectral and Peak Ground Acceleration (PGA) numbers for the project site for the 2% in 50 year probability of exceedance, are $S_a(0.2)=0.099$, $S_a(0.5)=0.072$, $S_a(1.0)=0.045$, $S_a(2.0)=0.023$ and $PGA=0.058$ (National Building Code 2015).

The native soil below the proposed inverts is typically competent, with N values typically greater than 20. Based on the type of soil, the site for seismic design purposes may be classified as Site Class D in accordance with Clause 4.4.3.2 (Table 4.1) of the Canadian Highway Bridge Design Code, 2019.

12.3 Soil Corrosivity

Two samples of the site soil were submitted to an external laboratory for chemical testing of soil corrosivity parameters. Details of the chemical test results are presented on the certificate of analysis presented in Appendix B.

As shown by the analysis, the sulphate concentration in the samples were 20 and 80 $\mu\text{g/g}$. According to clause 4.1.1.6 of the Canadian Standard Association (CSA) standard A23.1-19/CSA A23.2-19, soluble sulphate concentrations less than 1000 $\mu\text{g/g}$ generally indicate a low degree of sulphate attack for concrete when in contact with soil or ground water.

Generally, a chlorine concentration value in excess of 250 ppm (0.025%) leads to corrosive environment for buried metals or reinforced steel. The chloride content was 92 $\mu\text{g/g}$ and 1750 $\mu\text{g/g}$ (0.0092% and 0.175%). The potential for corrosive environment due to chlorine is assessed to be low near Borehole 1 and high near Borehole 3.

Electrical resistivity less than 2,000 ohm-cm generally lead to highly corrosive environment for steel element on contact with the soil. The Borehole 1 sample has a resistivity value of 3890 ohm-cm and



the Boreholes 3 sample has a value of 395 ohm-cm. The results suggest a corrosive environment for steel, exists near Borehole 3 and non-corrosive environment exists near Borehole 1.

In general, no sulphate attack is expected for concrete. However, corrosion protection measures are indicated at one location and should be provided for the entire steel casing. Alternative, corrosion protection could comprise a protective coating over the pipe, or cathodic protection, or providing a sufficiently thick pipe to accommodate sacrificial corrosion of the steel over the casing's design life.

12.4 Ground Water Control at the Pits

Ground water during drilling ranged from 1.4 to 2.3 m depth (elevation 210.2 to 211.1) in Borehole Nos. 5, 5A, 6 and 7A, and stabilized levels measured in the wells were 6.6 and 8.4 m depth (elevation 203.9 to 205.8) in Borehole Nos. 5 and 6.

Perched water was present in the sand fill above the less pervious native soil, with volumes varying depending on the seasonal conditions. Seepage from perched water, ground water or surface water which enters the excavation should be readily handled by temporary surface water diversion and conventional sump pumping techniques, provided sheet piling is in place as described above.

The actual dewatering methods should be established at the contractor's discretion within the context of a performance specification for the project. Regardless of the dewatering method chosen, the hydraulic head and ground water inflow must be properly controlled to ensure a stable and safe excavation and to facilitate construction. It is recommended that the ground water level be lowered, as necessary, to at least 0.5 m below the lowest excavation level. Reference is also given to OPSS 517 which pertains to construction dewatering.

Ideally, any dewatering measures/ground water control measures for the staging works should be established such that the zone of influence includes as much of the tunnel area as possible as this will generally reduce potential ground water seepage through the tunnel during construction. Lowering of groundwater during construction is expected to have negligible impact on existing infrastructure, provided that the existing infrastructure is founded on competent native soils. Construction during the dry time of the year is also recommended in order to reduce the ground water control requirements.



The contractor should be responsible for the selection, performance and detailed design of the dewatering system. The dewatering system should be designed to conform to the requirements of OPSS 517 and SSP 517F01.

13. CONSTRUCTION CONSIDERATIONS

If the contractor encounters obstructions, and boulders and cobbles such that further advance is not possible, and at the City's staff and/or the CA's direction, it is the responsibility of the contractor to abandon the drive and advance a new pipe alignment at no additional costs. The abandoned drive shall be fully grouted. Open cut on the Highway 26 shall not be permitted unless approval is provided by MTO.

The contractor shall be responsible to check and confirm all the underground utilities and structures in the tunnel path and its vicinity to assure that there is no conflict with the tunnelling operations, and will not impact the existing underground utilities or structures. Any damages as a result of the pipe installations shall be restored to their original conditions or better.

It is the responsibility of the contractor to ensure that potential loss of ground is minimized and any excessive movements and settlements resulting from the jack and bore operations are to be dealt with immediately at no additional cost to the owner.

13.1 Settlement Monitoring

The ground surface over the tunnel route may become distorted and distressed by tunnelling. The most common type of distress is settlement caused by loss of ground around the tunnel. Heave of the ground surface and or inadvertent drilling fluid returns are also possible depending on the type of installation. Mitigation of the distress or distortion on the travelled lanes of Highway 26 would be a major inconvenience to highway users and possibly a safety issue.

Distress at the ground surface is generally prevented or minimized by good construction practices and proper planning. In this regard, preparation of an installation plan as noted above is recommended.

The tunnelling process should be continuous, such that a stoppage in the advancement of the tunnel is not programmed when the end of the tunnel is under the travelled lanes of Highway 26. Such a stoppage would provide greater potential for loss of ground/settlement around the tunnel.



Monitoring during tunnelling will provide feedback to the engineer and contractor to adjust the construction procedures to control ground movements. Accordingly, changes to the progress and procedures can be made before the construction reaches locations where ground movements could be potentially damaging to the highway.

It is recommended that the project proponent implement a monitoring program to check the condition of the ground over the tunnel before, during and upon completion of construction. The monitoring program should be carried out by a qualified geotechnical consulting firm that is MTO RAQS approved and should conform to the MTO Settlement Monitoring Guidelines for Tunnelling which are presented in Appendix C. Generally, the CMO requires submission of a Geotechnical Instrumentation and Monitoring Plan (GIMP) for the encroachment permit applications. As noted in the Appendix, monitoring points should be installed over the proposed tunnelling route at a maximum interval of 5 m (Five arrays of monitoring points at typical 4 m intervals shown on drawing). Monitoring period should begin prior to tunnelling, extend throughout the duration and continue at least two weeks after completion of tunnelling. Measurement of the monitoring points should be done at least three times a day for everyday in the monitoring period. A Settlement Monitoring Plan (DWG A) and a Settlement Instrumentation (DWG B) are appended in Appendix C for the tunnelling section.

A pavement condition survey of the pavement directly above the pipe alignment should be carried out before, during and after the installation.

Monitoring points should be marked using a method approved by MTO. Monitoring points should also be functional throughout the monitoring period and should not deteriorate because of highway traffic, maintenance activities, and weather conditions.

If distress is observed during construction, the contractor should be informed and corrective action should be undertaken immediately. Specific corrective action will be dependent on the nature of the distress and type of installation. Regardless, the process should be outlined in the monitoring program and be part of the contingency actions in the contractor's installation plan. It should be noted that the ground movement monitoring does not relieve the contractor's responsibility to undertake the necessary action and additional instrumentation and independent reading of the instrumentation to ensure that work is carried out in a safe and acceptable manner.



At review level (10 mm relative to baseline readings), the method, rate or sequence of construction or ground stabilization measures should be reviewed or modified to mitigate further ground displacements. If alert level is reached (15 mm relative to the baseline readings), the contractor shall cease construction operations and execute pre-planned measures to secure the site, mitigate further movements and assure the safety of the public and maintain traffic.

All actions to prevent, secure, or mitigate destruction or damage to the highway and associated features should be done in accordance with and approved by MTO.

With regard to the cut and cover area, the method to be utilized for the temporary road protection is not available. The Performance Level of the temporary protection should meet Level 2 in accordance with OPSS 539, as amended. If sheet piling method is used, monitoring by survey on target should be conducted at every 5.0 m interval at the top of the shoring. If pile and lagging method is used, monitoring should be conducted at every pile location.



14. CLOSURE

This report was prepared by Mr. G. White, P.Eng , formerly with PML, and Mr. Nazibur Rahman, P.Eng. Independent review of the report was carried out by Mr. R. Ng, P.Eng., MTO Designated Principal Contact.

We trust this report has been completed within the terms of reference and is sufficient for your current needs. Should you have further questions, do not hesitate to contact our office.

Sincerely

Peto MacCallum Ltd.



Nazibur Rahman, P.Eng.
Manager, Geotechnical Services

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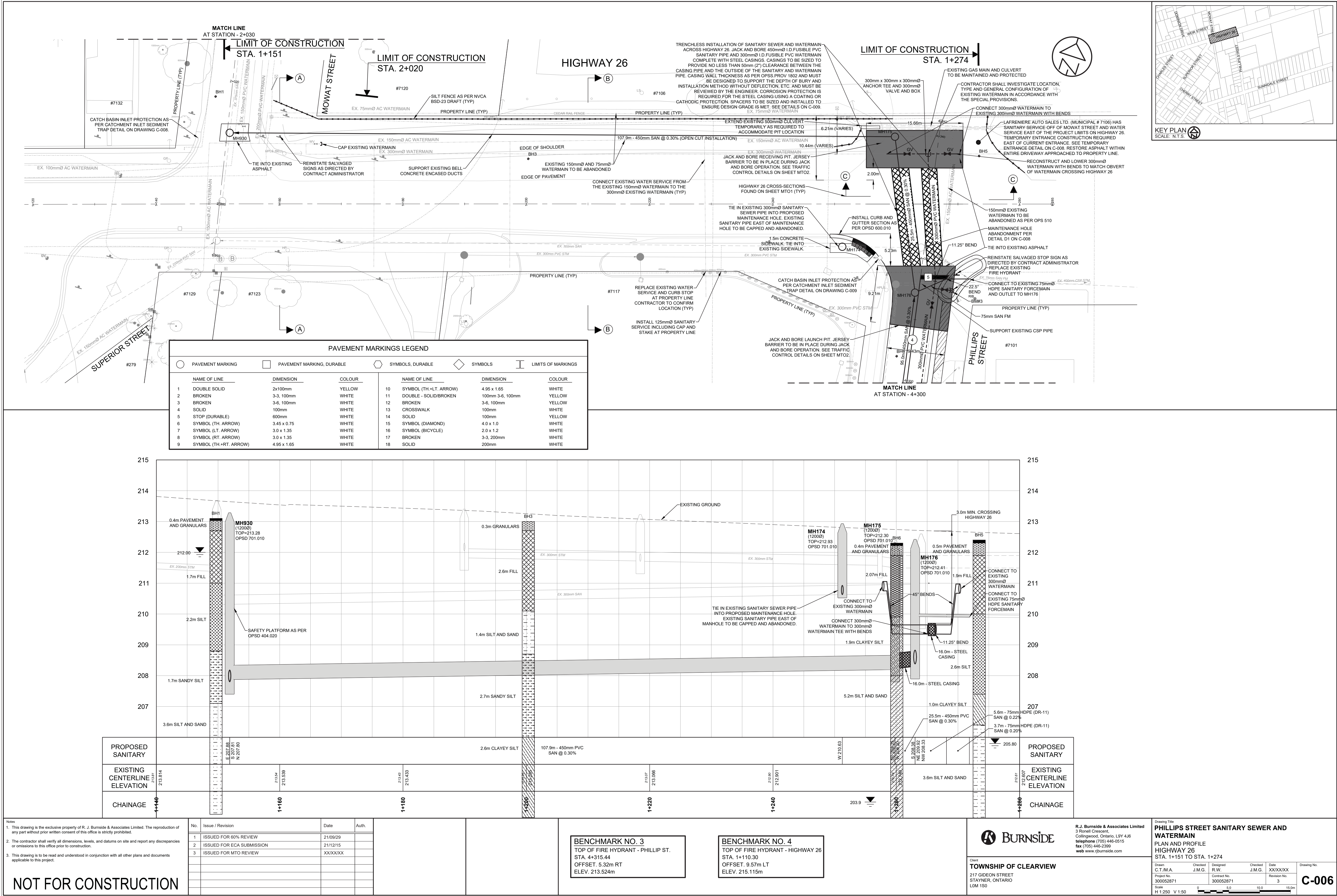
APPENDIX C

Copy of Drawings provided by R.J. Burnside & Associates Limited –
(Drawing No. C-005 Plan and Profile Phillips Street STA. 4+230 to STA. 4+ 329 and
Drawing No. C-006 Plan and Profile Highway 26 STA. 1+151 to STA. 1+274).

Copy of Ministry of Transportation's "Guidelines for Foundation Engineering –
Tunnelling Specialty for Corridor Encroachment Permit Application"

DWG A – Settlement Monitoring Plan

DWG B – Settlement Instrumentation



Guidelines for Foundation Engineering – Tunnelling Specialty For Corridor Encroachment Permit Application

General

These guidelines specify MTO requirements for the Foundation Engineering – Tunnelling Specialty component of submissions from proponents of development within the Ministry of Transportation's (MTO) corridor permit control area. The Foundation Engineering – Tunnelling Specialty component of submissions is a requirement for the permit application only and does not cover all the design requirements.

All applications containing tunnelling proposals shall be forwarded to the regional Geotechnical Section for review. Applications containing Low Complexity tunnelling proposals will typically be reviewed by the regional Geotechnical Section. The Geotechnical Section will forward applications involving Medium and High Complexity tunnelling proposals to the Foundation Section of the Structures Office for review.

Foundations Engineering consultants that are registered in the MTO consultant acquisition system (RAQS) at complexity ratings identified in Table 1 are eligible to provide Foundations Engineering services for this project. Alternatively, the proponents may propose a Foundations Engineering consultant that is not registered in RAQS, in which case, the proponent must submit sufficient documentation to demonstrate that the consultant's qualifications meet or exceed the RAQS complexity requirements. The submission for RAQS exemption shall demonstrate that the proponent has successfully completed tunnelling/trenchless projects on projects of similar scope and complexity. The proponent shall submit a minimum of three (3) Foundation Investigation and Design Reports on projects of similar scope and complexity produced in the last five (5) years. The proponent shall submit any supplementary engineering and construction experience to demonstrate their qualifications.

For Engineering Materials Testing and Evaluation, the consultant shall be qualified for Soil and Rock testing of complexity level at least equal to that identified for this project.

Please refer to Table 1 on Page 2 for the Foundation Engineering Complexity of Work guideline.

Table 1: Complexity ratings for tunnelling specialty services

Excavation Diameter (ø)	≤ 300 mm		$1\text{ m} \geq \text{ø} > 300$ mm		$2\text{ m} \geq \text{ø} > 1\text{ m}$		$\text{ø} > 2\text{ m}$
Design Cover* (m)	≥ 1.5 m	< 1.5 m	$\geq 3\text{ ø}$ and > 1.5 m	$< 3\text{ ø}$ or < 1.5 m	$\geq 3\text{ ø}$	$< 3\text{ ø}$	N/A
King's Highway	Low	Medium	Low	Medium	Medium	High	High
400 Series Freeway	Low	High	Medium	High	High	High	High

* Design cover is the proposed vertical distance measured from the lowest ground elevation to the crown of the tunnel

Site Investigation, Field Testing and Monitoring

General

This section describes requirements for site investigation, field/laboratory testing and monitoring programs for a proposed tunnelling projects. For low complexity projects, some or all of these requirements may not be necessary. Foundation field investigation, laboratory analyses and monitoring for low complexity projects with an excavation diameter of 300 mm or less will generally only be required on an exception basis. The applicant's Foundation Engineering service can contact MTO Geotechnical staff for clarification regarding appropriate levels of investigation, testing and monitoring.

Field Testing

A minimum of one borehole is required at each end of tunnel crossing. The boreholes shall be located outside but within two metres of the tunnel's excavated footprint.

Spacing between the boreholes shall not exceed 50 m. In case of larger spacing between the boreholes, additional boreholes shall be advanced except where significant traffic disruptions might occur and where consistent conditions are evident.

Boreholes shall be advanced to 3 tunnel diameters (excavated diameters) below invert. If bedrock is encountered earlier, the borehole shall advance to at least 3 m below the invert of tunnel into the bedrock.

The investigations, if required, shall be supplemented with additional and deeper boreholes to verify consistent conditions and existence of boulders within critical foundation zones.

Sampling and testing, consisting of Standard Penetration Test, thin wall tube sample, rock cores, and MTO Field Vane Test where appropriate, shall be conducted to develop a comprehensive subsurface model. Semi-continuous sampling at 0.75m (2.5ft) intervals is required within overburden; whereas, sampling interval of 1.5m (5.0ft) is required below the tunnel invert.

Where encountered, the bedrock-soil interface shall be determined by geological definition and not by the material properties.

All aspects of implementation of means of subsurface investigations including, but not limited to, planning, licensing, construction, maintenance, abandonment, and reporting, shall be in accordance with Ministry of the Environment Regulation 903 and its amendments (the water well regulation under the OWRA).

Boreholes and piezometer tubes shall be backfilled with a suitable bentonite/cement mixture. Test pits shall be backfilled with suitable material and either re-vegetated or otherwise protected from erosion. Temporary open holes shall be adequately covered. Holes in roads shall be backfilled as required to prevent future settlement and acceptably patched where pavement surfaces have been damaged. Backfilling requirements shall be described in the Foundation Investigation and Design Report.

Where encountered, artesian groundwater conditions shall be sealed. Details of the artesian condition and the sealing operation shall be included in the Foundation Investigation Report.

Fieldwork, including any Traffic Protection Plans required, shall be carried out in accordance with the Occupational Health and Safety Act.

Traffic Control in accordance with Ontario Traffic Manual Book 7 shall be provided during the course of any field investigations. However, where significant traffic disruptions might occur, boreholes may be relocated or numbers reduced with MTO's approval.

The locations and ground surface elevations of all boreholes, test pits and soundings shall be surveyed and referred to fixed reference points and data. Locations are to be identified by co-ordinates (Northing and Easting). The vertical accuracy of survey readings shall be within 0.1m; whereas, horizontal accuracy shall be within 0.5m.

The site investigation shall be of sufficient scope to verify design assumptions and to provide the contractor with adequate subsurface information for design and construction planning.

Sufficient subsurface (factual) information is required to determine the vertical and horizontal extent of subsurface materials (including both soil and rock) and their pertinent engineering properties and groundwater conditions.

Subsurface information is usually acquired by advancing boreholes, laboratory testing of soil samples and rock core samples, performing in-situ tests such as standard penetration tests, dynamic cone tests, and piezocone tests (CPTU) and test pits.

Minimum Laboratory Testing Requirements

Laboratory testing shall consist of routine testing of 25% of samples. One routine lab test is defined as natural water content plus Atterberg Limits plus grain size distribution tests. Complex laboratory testing is defined by all other tests including compressive strength, shear strength, consolidation, permeability and triaxial testing. Laboratory testing requirements shall be supplemented with additional routine and complex tests if required to verify strata boundaries and properties and behaviour of critical subsurface zones.

A minimum of one (1) soil chemical test shall be conducted at maximum of 100 m spacing. A soil chemical test includes pH, water soluble sulphate, sulphide, chloride, resistivity and electrical conductivity analyses.

Borehole Log Preparation and Foundation Drawing

Borehole log sheets, figures and drawings shall be prepared in accordance with MTO standards. The Foundation Drawing shall consist of a plan showing the locations of all borings, test pits and soundings and various stratigraphical longitudinal profiles and stratigraphical cross-sections at each tunnel structure foundation element and groundwater levels.

Requirements for the Foundation Investigation and Design Report

A Foundation Investigation and Design Report shall consist of the factual subsurface information (including the field and laboratory test information) and the recommendations required for foundation design.

Service Provider services shall be in accordance with the most recent editions of the Canadian Highway Bridge Design Code (CHBDC), and the 'Guideline for Professional Engineers Providing Geotechnical Engineering Services' published by the Professional Engineers of Ontario.

The designated principal contact identified for Foundations Engineering services by MTO shall sign, and where required, seal, all submissions and correspondence that are submitted to MTO.

The report shall be signed and sealed by two professional engineers, registered with the Professional Engineers of Ontario, representing the consulting firm; one of them shall be the firm's designated principal contact for MTO's Foundations Engineering projects.

The Foundation Investigation component of the report shall contain:

- Site Description - including topography, vegetation, drainage, existing land use, and structures.
- Investigation Procedures - including site investigation and lab testing procedures.
- Description of Subsurface Conditions - including soil, boulders, rock and groundwater conditions.
- Miscellaneous Section - that identifies the name of the drilling company, the laboratory where testing was performed, the persons who carried out the field supervision, and those who wrote and reviewed the report.

The Foundation Design component of the report shall present discussion and recommendations for design. The Service Provider shall analyse field data and test results and make comprehensive and practical recommendations pertaining to temporary, interim and permanent conditions at the Project.

The Service Provider shall identify and evaluate all reasonable and appropriate alternatives for the proposed tunnel crossing. Alternatives may include, but not limited to, jack & bore, pipe jacking using TBM, pipe ramming, micro-tunnelling, utility tunnelling using TBM (two pass system), Horizontal Directional Drilling (HDD) and cut and cover methods.

The Service Provider shall identify and present overview assessments of the advantages, disadvantages, relative costs and risks/consequences of alternative tunnelling methods in a table. The report should conclude a preferred alternative from foundation engineering and cost effectiveness perspective.

In the development and design of the preferred alternative, the Service Provider shall, as applicable, address:

- impacts on the land use and property, traffic and transportation, and environment,
- length and diameter constraints
- control of face stability
- capability of boulder excavation
- evaluation of temporary and permanent support
- alignment control
- estimated settlements and heave and management of these deformations

- special access and egress requirements for TBM's and other similar equipment such as those used for the Jack & Bore method including recommendations for vertical shafts and jacking pits;
- shored and un-shored alternatives for open-cut excavation;
- groundwater control & dewatering;
- the long-term stability of the tunnel;
- relative costs; and
- traffic management and contractor access for each alternative.

If borehole logs available from previous projects are included to meet the requirements of field investigations then the accuracy of subsurface information from these boreholes remains the responsibility of Service Provider except in situations where MTO specify the use of previous boreholes. Borehole logs from previous studies that are appended to the report shall be reformatted to meet the MTO's requirements.

The final foundation recommendations shall detail the geometric, material and strength properties of the new tunnel crossing plus the liner, bedding and backfill requirements, and slope and embankment restoration requirements. The invert elevation should be assessed in view of the subsurface conditions and the anticipated open face stability control.

The Service Provider is responsible for developing contract documents sufficient to implement the design. This typically includes:

- Contract specifications for materials and specialized construction activities, and
- Recommendations for methods of overcoming anticipated construction problems, in particular, those relating to dewatering, boulder excavation, alignment control and the stability of excavations and embankments.

The Service Provider shall develop a detailed instrumentation and monitoring program that meets the requirements of these guidelines. (see Appendix for typical settlement monitoring guidelines).

The Service Provider is responsible for preparing Traffic Control Plans, Traffic Protection Plans and to obtain approvals and an Encroachment Permit from the Ministry, which are required for lane closures necessary to install the settlement monitoring points.

The tunnelling Service Provider shall ensure that the foundations engineering component of the project is adequately reflected in the design drawings, specifications and related contract documents.

Written confirmation is required from the Proponent and the tunnelling Service Provider that the design package submitted to MTO have been reviewed by the tunnelling Service Provider and that all recommendations have been satisfactorily incorporated in the contract package.

APPENDIX: SETTLEMENT MONITORING GUIDELINES - TUNNELING

The purpose of settlement monitoring is to prevent damage to existing utilities and highway structures along the tunnel alignment. Ground settlement include settlement due to lost ground and dewatering/drainage.

Daily visual monitoring of the road surface and shoulders shall be carried out for any evidence of movements (e.g. cracks, bulges, heaves, depressions, ponding, etc.)

Instrumentation Arrays

All measurement points shall be installed and surveyed before the start of excavation to establish benchmarks/baseline.

Surface Monitoring Points

Surface monitoring points will be installed to cover the whole length of the tunnel with in the right of way under the jurisdiction of MTO (Figure 1).

Surface monitoring points will be located at not greater than 5m intervals along the tunnel alignment. The surface monitoring will be identified using paint marks on the pavement. Surface monitoring points installed on the unpaved right of way shall be founded below frost penetration depths. The interval and/or marking of the points should be changed with MTO's approval where traffic disruptions might occur.

The final instrumentation plan should be finalised when Contractor's proposed construction method is available.

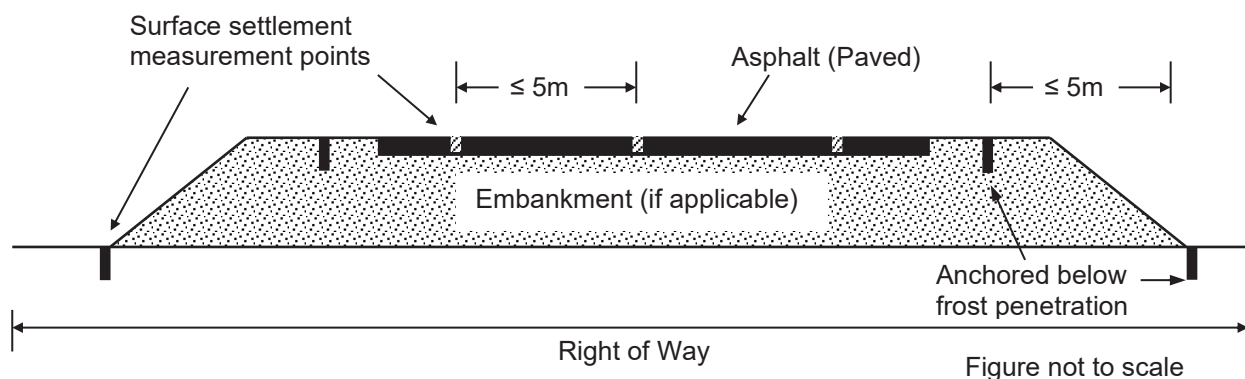


Figure 1: Typical configuration of surface settlement monitoring points along the tunnel alignment.

Condition Survey

A condition survey for the pavement will be carried out prior to commencement of construction and documented for the purpose of requirement of restoration. The condition survey shall document visible flaws such as cracks, distortions and deviations, heaves, and depressions. This surface survey will be completed during the installation of the monitors and again once the tunnel has been completed.

Reading Frequency

An average of at least two readings shall be taken to establish the initial conditions.

The reading and collection of data from the surface monitoring points shall be read and recorded by the Contractor during the construction period and after construction for period of at least 2 weeks provided that further settlement has stopped.

A minimum of three (3) sets of reading be taken daily, provided that movements are within anticipated limits. Otherwise, the frequencies should increase according to a pre-planned interval.

Monitoring of movements is required during work stoppages, such as during non-operation period (off-shifts) or weekends. A minimum of three (3) sets of readings should be taken daily.

Measurements of the monitoring points shall be reported promptly to MTO for review.

Data Collection and Data Transfer

A procedure is required to be established in consultation with MTO so that the monitoring data and the interpreted data will reach all parties as soon as necessary. The contract administrator/Service Provider and the Contractor should interpret monitoring data as needed for the purpose of on-going construction. The Foundation Engineer should be contacted for technical support to the prime Service Provider in the interpretation of ground movements and review of the Contractor's response when Review and Alert Levels are reached.

Criteria for Assessment

The acceptable surface settlement (or heave) will be according to criteria as specified below.

Baseline Reading – A baseline reading of the instrumentation shall be taken prior to commencement of the work. An average of at least two initial readings shall be recorded as baseline reading.

Review Level – A maximum value of 10 mm relative to the baseline readings is suggested for this project. If this level is reached, the method, rate or sequence of construction, or ground stabilization measures should be reviewed or modified to mitigate further ground displacements.

Alert Level – A maximum value of 15mm relative to the baseline readings is suggested for this project. If this level is reached, the Contractor shall cease construction operations and to execute pre-planned measures to secure the site, to mitigate further movements and to assure safety of public and maintain traffic.

Review of Contractor's Proposed Method

MTO, the Proponent's prime Service Provider and Foundation Engineer should review the Contractor's proposed method of construction. The proposed method should include a description of the potential loss of ground, and calculation of the maximum settlement in relation to the Contractor's procedure and equipment, alternative/remedial measures when review level of measurement is reached; and contingency/remedial measures when alert level of measurement is reached.

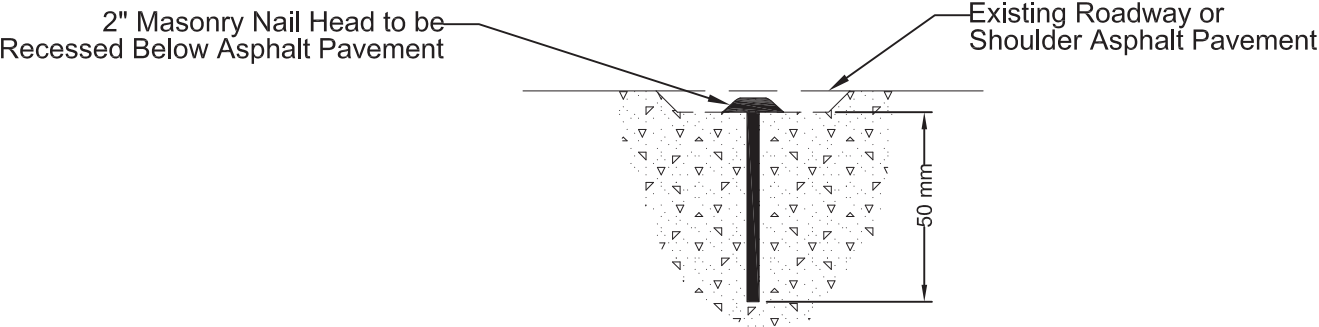
Contractor's Responsibility for Restoration and Warranty Provision

In addition to the monitoring program to assess the adequacy of the construction method to control potential ground movements and groundwater, the Contractor is responsible for reinstatement (such as surface paving) should movements or other surface distress occur, and provide a reasonable warranty period acceptable to MTO. Remedial measures shall be approved by MTO; however, MTO maintains the right to perform the maintenance at the proponent's expense.

Construction Monitoring

The Proponent shall retain a RAQS qualified Geotechnical Service Provider – Medium Complexity to supervise the installation of surface settlement points on site and to provide direction, technical input and field inspection on this project.

CONT No	
GWP No	
WP No	
SANITARY SEWER WATERMAIN PHILLIPS STREET AND HIGHWAY 26 SETTLEMENT INSTRUMENTATION	SHEET



SURFACE MONITORING POINT(SMP) (TYP.)
(NTS)

REVISIONS			
	DATE	BY	DESCRIPTION

HWY No	40				DIST
SUBMD	NL	CHECKED	NR	DATE	AUG. 2022
DRAWN	NL	CHECKED	RN	APPROVED	RN
				DWG	B