PRELIMINARY GEOTECHNICAL INVESTIGATION

Downtown Capital Implementation Plan, City of Guelph, Ontario

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SUBJECT: PRELIMINARY GEOTECHNICAL INVESTIGATION, DOWNTOWN CAPITAL IMPLEMENTATION PLAN, CITY OF GUELPH, ONTARIO

EnVision Consultants Ltd. is pleased to present the enclosed updated final Preliminary Geotechnical Investigation report for the above-noted site which includes additional investigation data along Wellington St. E and Wyndham St. S.

We thank you for providing EnVision an opportunity to work on this assignment. If there are any questions regarding the enclosed report, please do not hesitate to contact us.

Yours sincerely,

Maqsooda.

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QUALITY MANAGEMENT

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1. EXECUTIVE SUMMARY

EnVision Consultants Ltd. (EnVision) was retained by the City of Guelph (the 'Client') to conduct preliminary geotechnical investigation work distributed across approximately 7.0 km of roadway at various locations within downtown Guelph (the Site'). Since this is a master planning exercise, the investigation does not cater to any specific utilities, structures or road improvements at the time of this report.

EnVision conducted preliminary geotechnical investigation work consisting of drilling a total thirty-one (31) boreholes in April 2022. Sixteen (16) boreholes were advanced to a depth of 2.1m and fifteen (15) boreholes to a depth of 4m below the existing ground surface. Ten (10) monitoring wells were installed in selected 4m deep boreholes for groundwater level monitoring and for hydrogeological purposes. Bedrock was encountered in three (3) of the boreholes (BH22-12, BH22-21 and BH22-30) within the proposed 4m depth of the boreholes. Rock coring was also completed within these three (3) boreholes.

Five (5) additional boreholes were completed in August 2023 which consisted of one (1) borehole proposed to a depth of 2.1m and four (4) boreholes to a depth of 4m below the existing ground surface. Bedrock was encountered in three (3) of the proposed 4m deep boreholes and rock coring was completed within each of these boreholes. Three (3) monitoring wells were installed as part of this additional investigation.

The subsurface conditions in the boreholes generally consisted of pavement structure overlying fill material consisting of sand and gravel, gravelly sand, sand, silty sand, clayey silt and silty clay. Native soils consisting of silty clay, clayey silt, silty clay till, clayey silt till, sandy silt till to silty sand till and cohesionless deposits of gravelly sand, sandy gravel, sand and gravel, sand, silty sand and sandy silt were encountered in the boreholes. Cobbles and boulders are expected in these deposits. Bedrock of the Guelph Formation was encountered in six (6) boreholes (BH23-1, BH23-4, BH23-5, BH22-12, BH22-21 and BH22-30) at depths ranging from 2.3m to 3.8m below existing ground surface, corresponding to Elev. 307.3m to 322.5m.

Groundwater levels were measured at depths ranging from 1.7m to 4m below the existing ground surface in the monitoring wells installed in Boreholes BH23-1, BH23-3, BH23-4, BH22-4, BH22-17 and BH22-21. All other monitoring wells were found to be dry.

2. INTRODUCTION

EnVision Consultants Ltd. (EnVision) was retained by the City of Guelph (the 'Client') to conduct preliminary geotechnical investigation work within the downtown area, in association with the Downtown Capital Implementation Plan, City of Guelph, Ontario (the 'Site'). A preliminary geotechnical investigation consisting of 31 boreholes was completed in 2022 and a geotechnical report entitled "Preliminary Geotechnical Investigation, Downtown Capital Implementation Plan, City of Guelph, dated May 19, 2023" was submitted. Additional site investigation work including five (5) boreholes was proposed by the client along Wellington Street East and Wyndham Street West in May 2023. This report is an updated version of the above-mentioned geotechnical report which includes this additional investigation data. The supplementary work was undertaken in accordance with the scope outlined in EnVision's Proposal No. P23-5916 Geotechnical, Hydrogeological, & Environmental Investigations, Downtown Guelph Contract 21-18 dated June 23-2023.

These services have been requested in support of the Downtown Infrastructure Revitalization Plan, distributed over approximately 7.0km of roadways at various locations in the City of Guelph downtown. A Key Plan showing the location of the roads in the study area is depicted on Drawing No. 1.

The purpose of this preliminary geotechnical investigation was to determine the subsurface soil, bedrock and groundwater conditions at the borehole locations and from the findings in the boreholes make preliminary geotechnical recommendations for wet utility installations along the roads in the study area using open cut and/or trenchless construction methods. Preliminary comments are also provided with respect to pavement reconstruction in the study area.

This report is presented in two parts; Part A of the report includes factual data from the geotechnical investigations at the borehole locations and Part B includes preliminary geotechnical recommendations for generic utility and paving works.

Hydrogeological and Geo-environmental studies were also carried out at the site by EnVision, which are presented in a separate report.

This report is provided on the basis of the terms of reference presented above and on the assumption that the design will be in accordance with the applicable codes and standards. If there are any changes in the design features relevant to the geotechnical analyses, or if any questions arise concerning the geotechnical aspects of the codes and standards, this office should be contacted to review the design. It may then be necessary to carry out additional borings and reporting before the recommendations of this office can be relied upon. Since this is a preliminary site investigation, it is expected that individual projects will be supported in the future by detailed design stage site investigation work that caters specifically to the features of such projects.

The site investigation and recommendations follow generally accepted practice for geotechnical consultants in Ontario. The format and contents are guided by client specific needs and economics and do not conform to generalized standards for services. Laboratory testing for most part follows ASTM or CSA Standards or modifications of these standards that have become standard practice.

This report has been prepared for the City of Guelph and its civil engineering designer (R.V. Anderson Associates Limited). Third party use of this report without EnVision consent is prohibited. The limitation conditions presented in this report form an integral part of the report and must be considered in conjunction with this report.

3. REGIONAL SETTING

The following information is intended to provide an overview of the regional setting.

3.1. GEOLOGY

3.1.1. Overburden Geology

Based on a review of the public-record geological mapping of the Study Area, the surficial material consists of glaciofluvial deposits of gravel along with sandy silt to silty sand textured glacial till (Ministry of Northern Development, Mines and Forestry, 2013).

3.1.2. Bedrock Geology

Bedrock mapping of the Study Area identifies the bedrock as belonging to the Guelph Formation which includes lithologies ranging from shale, siltstone, dolostone and sandstone (Sharpe, 1980). The depth to bedrock is expected to be between 2 and 13 meters below the ground surface within the Site area.

4. FIELD INVESTIGATION AND TESTING

4.1. FIELDWORK

The field investigation consisted of drilling a total of thirty-six (36) boreholes (BH23-1 to BH23-5 and BH22-1 through BH22-31) along various roads in the City of Guelph downtown, to depths varying from 1.9m to 7.6m below the existing ground surface, as listed in Table 4-1and shown on the Borehole Location Plan (Drawing No. 1). Thirteen (13) monitoring wells of 50mm diameter were installed in selected boreholes as listed in Table 4-1. Bedrock was encountered in six (6) boreholes (BH22-12, BH22-21, BH22-30, BH23-1, BH23-4 and BH23-5) and was cored in these boreholes as per the terms of Reference provided in the RFP dated October 5, 2021.

The as-drilled borehole locations were surveyed by EnVision using differential GPS. The borehole coordinates and ground geodetic elevations at the borehole locations are summarized in Table 4-1and are presented in the Record of Borehole sheets in Appendix A.

Table 4-1: Summary of Borehole Information

		BOREHOLE COORDINATES UTM NAD83, ZONE 17			
BOREHOLE ID	**GROUND SURFACE ELEVATION (m)**	**NORTHING (m)**	**EASTING (m)**	**DEPTH OF BOREHOLE (m)**	**NOTE**
Dublin Street					
BH22-1	338.2	4821373.6	560375.5	2.1	
BH22-2	325.3	4821224.3	560482.8	4.4	50mm MW
Northumberland Street					
BH22-3	333.2	4821355.8	560556.8	1.9	
Norfolk Street					
BH22-4	333.3	4822107.1	560093.3	4.4	50mm MW
BH22-5	332.0	4821939.3	560203.3	2.1	
BH22-6	331.0	4821806.5	560299.9	4.4	
BH22-7	328.5	4821709.3	560369.7	2.1	
BH22-8	327.8	4821559.2	560489.9	2.1	
BH22-9	325.6	4821401.9	560610.1	3.9	50mm MW
Cardigan Street					

		BOREHOLE COORDINATES UTM NAD83, ZONE 17			
BOREHOLE ID	**GROUND SURFACE ELEVATION (m)**	**NORTHING (m)**	**EASTING (m)**	**DEPTH OF BOREHOLE (m)**	**NOTE**
BH22-10	322.6	4822229.6	560201.4	2.1	
BH22-11	322.8	4822048.1	560393.9	2.1	
Eramosa Road					
BH22-12	322.7	4821984.8	560524.9	6.4	
Wyndham Street N					
BH22-13	328.5	4821889.7	560537.4	2.1	
BH22-14	328.3	4821760.7	560620.6	4.4	50mm MW
BH22-15	324.1	4821626.6	560709.5	4.4	50mm MW
MacDonell Street					
BH22-16	324.3	4821616.9	560795.7	2.1	
BH22-17	322.4	4821773.7	561008.4	4.4	50mm MW
BH22-30	324.8	4821506.0	560623.7	6.5	
Woolwich Street					
BH22-18	332.1	4822013.4	560223.1	2.1	
BH22-19	330.4	4821990.5	560329.1	4.4	50mm MW
BH22-20	325.8	4821914.6	560650.1	2.1	
BH22-21	324.0	4821880.0	560801.9	6.3	50mm MW
BH22-22	318.9	4821868.6	561006.8	2.1	
Norwich Street E					
BH22-23	331.8	4822054.2	560197.6	4.1	
Suffolk Street E					
BH22-24	331.5	4821942.5	560299.3	4.2	50mm MW
Yarmouth Street					
BH22-25	330.2	4821893.7	560370.9	2.1	
Quebec Street					
BH22-26	327.0	4821673.7	560519.8	5.2	50mm MW
BH22-27	327.0	4821698.0	560615.2	2.1	

		BOREHOLE COORDINATES UTM NAD83, ZONE 17			
BOREHOLE ID	GROUND SURFACE ELEVATION (m)	NORTHING (m)	EASTING (m)	DEPTH OF BOREHOLE (m)	NOTE
Cork Street E					
BH22-28	327.5	4821550.3	560534.7	2.1	
BH22-29	324.8	4821618.1	560633.4	4.4	
Douglas Street					
BH22-31	328.1	4821759.5	560672.3	2.1	
WELLINGTON ST. E					
BH23-1	310.05	4821187.45	561046.2	6.15	50mm MW
BH23-2	310.92	4821329.23	561095.29	2.9	
BH23-3	313.40	4821534.12	561168.38	4.3	50mm MW
WYNDHAM ST. S					
BH23-4	314.51	4821413.57	561005.08	6.22	50mm MW
BH23-5	319.97	4821482.98	560843.05	7.6	

The field investigation work of borehole drilling was carried out from April 5, 2022, to April 20, 2022 by Davis Drilling Ltd. with technical supervision provided by EnVision personnel. Supplementary field investigation work (BH23-1 to BH23-5) was carried out on August 16 and 17, 2023. Boreholes were advanced using a CME75 truck mounted power auger drilling machine. Split spoon samples were retrieved at regular intervals with a hammer weighing 624 N and dropping 760 mm as per ASTM D1586. This sampling method recovers samples from the soil strata, and the number of blows required to drive the sampler 0.3m depth into the undisturbed soil (SPT 'N'-values) gives an indication of the compactness condition or consistency of the sampled soil material. The SPT 'N' values are indicated on the Borehole Log sheets (Refer to Appendix A).

Upon encountering bedrock, coring of the rock was carried out with HQ-2 size double tube wireline equipment allowing recovery of 63mm diameter rock cores. The Total Core Recovery (TCR), Solid Core Recovery (SCR), Rock Quality Designation (RQD) values, Fracture Indices (FI) and depths and thicknesses of hard layers were recorded for the rock cores in accordance with the conventions used by the International Society for Rock Mechanics (ISRM). An explanation of these terms is presented in the fly sheet at the beginning of Appendix A. Photographs of the recovered bedrock cores are provided in Appendix C.

The samples were logged in the field and returned to the EnVision laboratory for detailed examination by the geotechnical engineer and for laboratory testing.

Prior to drilling operations, all underground utilities were cleared at the borehole locations by the representatives of the public utilities locate companies.

Thirteen (13) monitoring wells of 50mm diameter were installed for groundwater level monitoring purposes.

4.2. GEOTECHNICAL LABORATORY TESTING

The laboratory testing program consisted of the measurement of the natural moisture content of all available soil samples and the results are presented on the respective borehole logs. Grain size analyses were conducted on thirty-four (34) selected samples and Atterberg Limits tests were conducted on two (2) selected soil samples. The gradation curves and Atterberg Limits tests results are presented in Appendix B and on the respective borehole log sheets in Appendix A.

Geotechnical testing of the rock cores consisted of unconfined compressive strength (UCS) tests on six (6) rock samples and CERCHAR abrasiveness tests on five (5) rock samples, The test results are presented in Appendix C.

Corrosivity tests were conducted on seven (7) soil samples in general accordance with the AWWA methodology and these test results are presented in Appendix D.

5. SUBSURFACE CONDITIONS

The borehole locations are shown on Drawing No. 1. The terms used in the record of boreholes and general notes on soil descriptions are presented in Appendix A. The subsurface conditions in the boreholes are presented in the individual borehole log sheets attached in Appendix A and are summarized in the following paragraphs.

The subsurface conditions in the boreholes generally consisted of pavement structure overlying fill material consisting of sand and gravel, gravelly sand, sand, silty sand, clayey silt and silty clay. Native soils consisting of silty clay, clayey silt, silty clay till, clayey silt till, sandy silt till to silty sand till and cohesionless deposits of sand and gravel, sandy gravel, gravelly sand, sand, silty sand and sandy silt were encountered in the boreholes. Cobbles and boulders are expected in these deposits.

Bedrock of the Guelph Formation was encountered in six (6) boreholes (BH23-1, BH23-4, BH23-5, BH22-12, BH22-21 and BH22-30) at depths ranging from 2.3m to 3.8m below existing ground surface, corresponding to Elev. 307.3m to 322.5m.

Groundwater levels were measured in the installed monitoring wells on April 26, April 27, 2022 and on August 29, 2023. Groundwater levels were measured at depths ranging from 1.7m to 4m below the existing ground surface in the monitoring wells installed in Boreholes BH23-1, BH23-3, BH23-4, BH22-4, BH22-17 and BH22-21 at the time of observation. All other monitoring wells were found to be dry.

The subsurface conditions at each road/street are summarized in the following paragraphs.

5.1. DUBLIN STREET

Boreholes BH22-1 and BH22-2 were drilled along Dublin Street to depths of 2.1 and 4.4m below the existing ground surface. Borehole BH22-2 was completed as a monitoring well.

5.1.1. SOIL CONDITIONS

5.1.1.1 PAVEMENT STRUCTURE / FILL MATERIAL

Pavement structure consisting of 75mm of asphaltic concrete overlying 100mm of granular base/subbase was encountered at the location of boreholes. A summary of the pavement structure thicknesses at the borehole locations is listed in Table 5-1 below.

Table 5-1: Summary of Pavement structure Thicknesses at Borehole Locations

ROAD NAME	BOREHOLE NO.	PAVEMENT STRUCTURE THICKNESS	
ROAD NAME	BOREHOLE NO.	ASPHALTIC CONCRETE (m)	GRANULAR BASE/SUBBASE (m)
DUBLIN ST.	BH22-1	75	100
	BH22-2	75	100

Below pavement structure in the boreholes, fill material consisting of gravelly sand and sandy silt to silty sand was encountered which extended to depths of 1.2m to 1.5m below the existing ground surface. Fill was generally present in a compact state based on measured SPT 'N' values ranging from 14 to 20 blows per 300 mm of penetration.

5.1.1.2 NATIVE SOILS

SANDY SILT TILL

Below fill material in BH22-1, glacial till deposit of sandy silt was encountered which extended to the termination depth of 2.1m. Sandy silt till was found to be in a compact state based on measured SPT 'N' values of 14 to 29 blows per 300 mm of penetration.

Grain size analysis was conducted on a selected sandy silt till sample. The grain size distribution of the sample is indicated in Table 5-2 and the grain size distribution curve for the sample is presented in Appendix B.

Cobbles and boulders are expected in these cohesionless deposits.

Table 5-2: Summary of Grain Size Distribution on Sandy Silt Till sample

BOREHOLE NO.	SAMPLE NO.	SAMPLE DEPTH (m)	GRAIN SIZE DISTRIBUTION			
			GRAVEL (%)	SAND (%)	SILT (%)	CLAY (%)
BH22-1	SS2	0.8	7	37	46	10

COHESIONLESS DEPOSITS OF GRAVELLY SAND/SILTY SAND

Below fill material in BH22-2, cohesionless deposits of gravelly sand and sandy silt were encountered which extended to the termination depth of 4.2m. These cohesionless deposits were found to be in a dense state based on measured SPT 'N' values of 37 to 47 blows per 300 mm of penetration.

Grain size analysis was conducted on a selected gravelly sand sample. The grain size distribution of the sample is indicated in Table 5-3 and the grain size distribution curve for the sample is presented in Appendix B. Cobbles and boulders are expected in these cohesionless deposits.

Table 5-3: Summary of Grain Size Distribution on Gravelly Sand sample

BOREHOLE NO.	SAMPLE NO.	SAMPLE DEPTH (m)	GRAIN SIZE DISTRIBUTION			
			GRAVEL (%)	SAND (%)	SILT (%)	CLAY (%)
BH22-2	SS3	1.5	34	53	13	

5.1.2. GROUNDWATER CONDITIONS

Groundwater was not encountered in the boreholes during drilling. The monitoring well installed in BH22-2 was found to be dry based on the groundwater level measurements of April 26 and 27, 2022.

It should be noted that the groundwater levels can vary and are subject to seasonal fluctuations in response to weather events.

5.2. NORTHUMBERLAND STREET

Borehole BH22-3 was drilled at Northumberland Street to a depth of 1.9m below the existing ground surface.

5.2.1. SOIL CONDITIONS

5.2.1.1 PAVEMENT STRUCTURE / FILL MATERIAL

Pavement structure consisting of 75mm of asphaltic concrete overlying 100mm of granular base/subbase was encountered at the location of the borehole.

Below pavement structure in the borehole, fill material consisting of sandy silt was encountered which extended to 0.8m below the existing ground surface. Fill was present in a dense state based on measured SPT 'N' values ranging from 38 blows per 300 mm of penetration.

Grain size analysis was conducted on the granular sample, the particle size distribution is indicated in Table 5-4 and the grain size distribution curve for the sample is presented in Appendix B.

Table 5-4: Summary of Grain Size Distribution on Granular sample

BOREHOLE NO.	SAMPLE NO.	SAMPLE DEPTH (m)	GRAIN SIZE DISTRIBUTION			
			GRAVEL (%)	SAND (%)	SILT (%)	CLAY (%)
BH22-3	SS1 (Granular)	0.1	24	33	40	3

5.2.1.2 NATIVE SOILS

SILTY SAND TILL

Below fill material in BH22-3, glacial till deposit of silty sand was encountered which extended to the termination depth of 1.9m. Silty sand till was found to be in a very dense state based on measured SPT 'N' values of more than 50 blows per 300 mm of penetration.

Cobbles and boulders are expected in these glacial till deposits.

5.2.2. GROUNDWATER CONDITIONS

Groundwater was not encountered in the borehole during drilling.

5.3. NORFOLK STREET

Six (6) boreholes (BH22-4 through BH22-9) were drilled along Norfolk Street to depths varying from 2.1 to 4.4m below the existing ground surface. Monitoring wells of 50mm diameter were installed in two (2) boreholes (BH22-4 and BH22-9).

5.3.1. SOIL CONDITIONS

5.3.1.1 PAVEMENT STRUCTURE / FILL MATERIAL

Pavement structure consisting of 75mm to 200mm of asphaltic concrete overlying 150mm to 460mm of granular base/subbase was encountered at the location of boreholes (BH22-4 to BH22-9). A summary of the pavement structure thicknesses at the borehole locations is listed in Table 5-5 below.

Table 5-5: Summary of Pavement structure Thicknesses at Borehole Locations

ROAD NAME	BOREHOLE NO.	PAVEMENT STRUCTURE THICKNESS	
		ASPHALTIC CONCRETE (m)	GRANULAR BASE/SUBBASE (m)
NORFOLK ST	BH22-4	150	150
	BH22-5	150	410
	BH22-6	150	400
	BH22-7	75	300
	BH22-8	150	460
	BH22-9	200	150

Below pavement structure in BH22-4 to BH22-8, fill material consisting of silty sand to sandy silt was encountered which extended to depths varying from 2.1m to 2.3m below the existing ground surface. Boreholes BH22-5, BH22-7 and BH22-8 were terminated in fill material at a depth of 2.1m. Traces of organics were present in BH22-6 at a depth of 1.6m. Fill was present in a loose to compact state based on measured SPT 'N' values ranging from 6 to 24 blows per 300 mm of penetration.

Grain size analysis was conducted on one selected granular sample (BH22-8/SS1), the particle size distribution is indicated in Table 5-6 and the grain size distribution curve for the sample is presented in Appendix B.

Grain size analyses were conducted on three (3) selected fill samples, the particle size distribution is indicated in Table 5-6 and the grain size distribution curves for the samples are presented in Appendix B.

Table 5-6: Summary of Grain Size Distribution on Granular and Fill samples

BOREHOLE NO.	SAMPLE NO.	SAMPLE DEPTH (m)	GRAIN SIZE DISTRIBUTION			
			GRAVEL (%)	SAND (%)	SILT (%)	CLAY (%)
BH22-8	SS1 (Granular)	0.2	13	72	15	
BH22-5	SS2	0.6	16	48	30	6

BOREHOLE NO.	SAMPLE NO.	SAMPLE DEPTH (m)	GRAIN SIZE DISTRIBUTION			
			GRAVEL (%)	SAND (%)	SILT (%)	CLAY (%)
BH22-6	SS3	1.6	1	32	55	12
BH22-7	SS2	0.8	13	53	28	6

5.3.1.2 NATIVE SOILS

COHESIONLESS DEPOSITS OF SAND/SANDY SILT

Below fill material in Boreholes BH22-6 and BH22-9, cohesionless deposits of sand, sand with gravel and sandy silt were encountered which extended to depths of 3.1m to 4.4m below the existing ground surface. These deposits were found to be in a compact to very dense state based on measured SPT 'N' values of 21 to more than 50 blows per 300 mm of penetration.

Grain size analysis was conducted on a selected sandy silt sample. The grain size distribution of the sample is indicated in Table 5-7 and the grain size distribution curve for the sample is presented in Appendix B.

Cobbles and boulders are expected in these cohesionless deposits.

Table 5-7: Summary of Grain Size Distribution on Sandy Silt sample

BOREHOLE NO.	SAMPLE NO.	SAMPLE DEPTH (m)	GRAIN SIZE DISTRIBUTION			
			GRAVEL (%)	SAND (%)	SILT (%)	CLAY (%)
BH22-9	SS2	0.6	4	20	69	7

SANDY SILT TILL

Below fill material in BH22-4 and below sandy silt in BH22-9, glacial till deposits of sandy silt were encountered which extended to a depth of 3.6m in BH22-8 and to the termination depth of 3.9m in BH22-9. Sandy silt till was found to be in a very dense state based on measured SPT 'N' values of more than 50 blows per 300 mm of penetration.

Grain size analysis was conducted on a selected sandy silt till sample. The grain size distribution of the sample is indicated in Table 5-8 and the grain size distribution curve for the sample is presented in Appendix B.

Cobbles and boulders are expected in these cohesionless deposits and glacial till deposits.

Table 5-8: Summary of Grain Size Distribution on Sandy Silt Till sample

BOREHOLE NO.	SAMPLE NO.	SAMPLE DEPTH (m)	GRAIN SIZE DISTRIBUTION			
			GRAVEL (%)	SAND (%)	SILT (%)	CLAY (%)
BH22-4	SS4	2.3	8	34	50	8

SILTY CLAY

Below sandy silt till in BH22-4, cohesive deposit of silty clay was encountered which extended to the termination depth of the borehole. Silty clay was found to be in a hard consistency based on measured SPT 'N' value of 47 blows per 300 mm of penetration. The moisture content in the tested sample was found to be 13%.

5.3.2. GROUNDWATER CONDITIONS

The groundwater level measured within the monitoring well installed in BH22-4 was at 2.9m below grade (Elev. 330.5m) on April 26 and 27, 2022. The monitoring installed in BH22-9 was found to be dry at 2.9m.

It should be noted that the groundwater levels will vary and are subject to seasonal fluctuations in response to weather events. Longer term groundwater level monitoring would be needed to assess the groundwater table seasonal level variations.

5.4. CARDIGAN STREET

Two (2) boreholes (BH22-10 and BH22-11) were drilled along Cardigan Street to depths of 2.1m below the existing ground surface.

5.4.1. SOIL CONDITIONS

5.4.1.1 PAVEMENT STRUCTURE / FILL MATERIAL

Pavement structure consisting of 75mm of asphaltic concrete overlying 100mm of granular base/subbase was encountered at the location of boreholes (BH22-10 to BH22-11). A summary of the pavement structure thicknesses at the borehole locations is listed in Table 5-9 below.

Table 5-9: Summary of Pavement structure Thicknesses at Borehole Locations on Cardigan Street

ROAD NAME	BOREHOLE NO.	PAVEMENT STRUCTURE THICKNESS	
		ASPHALTIC CONCRETE (m)	GRANULAR BASE/SUBBASE (m)
CARDIGAN STREET	BH22-10	75	100
	BH22-11	75	100

Below pavement structure in the boreholes (BH22-10 and BH22-11), fill material consisting of silty sand and sandy silt was encountered which extended to the termination depths varying from 1.5m to 2.1m below the existing ground surface. Fill was present in a very loose to dense state based on measured SPT 'N' values ranging from 2 to 36 blows per 300 mm of penetration.

Grain size analysis was conducted on a selected granular sample (BH22-10/SS1), the particle size distribution is indicated in Table 5-10 and the grain size distribution curve for the sample is presented in Appendix B.

Table 5-10: Summary of Grain Size Distribution on Granular and Fill samples

BOREHOLE NO.	SAMPLE NO.	SAMPLE DEPTH (m)	GRAIN SIZE DISTRIBUTION			
			GRAVEL (%)	SAND (%)	SILT (%)	CLAY (%)
BH22-10	SS1 (Granular)	0.1	17	61	22	
BH22-11	SS2	0.6	5	49	37	9

5.4.1.2 NATIVE SOILS

SILTY SAND TILL

Below fill material in BH22-11, glacial till deposit of silty sand was encountered which extended to the termination depth of 2.1m. Silty sand till was found to be in a dense state based on measured SPT 'N' values of 41 blows per 300 mm of penetration.

Cobbles and boulders are expected in these cohesionless deposits.

5.4.2. GROUNDWATER CONDITIONS

During drilling, short-term (un-stabilized) groundwater was encountered in BH22-10 and BH22-11 at depths of 1.5m and 0.8m below ground surface.

It should be noted that the groundwater levels will vary and are subject to seasonal fluctuations in response to weather events.

5.5. ERAMOSA ROAD

One (1) borehole (BH22-12) was drilled along Eramosa Road to a depth of 6.4m below the existing ground surface.

5.5.1. SOIL CONDITIONS

5.5.1.1 PAVEMENT STRUCTURE / FILL MATERIAL

Pavement structure consisting of 150mm of asphaltic concrete overlying 410mm of granular base/subbase was encountered at the location of the borehole (BH22-12).

Below pavement structure in the borehole (BH22-12), fill material consisting of silty sand, sandy silt and silty clay was encountered which extended to 3.1m below the existing ground surface. Fill was present in a very loose to compact state or in a stiff consistency based on measured SPT 'N' values ranging from 2 to 9 blows per 300 mm of penetration.

Grain size analysis was conducted on a selected granular sample, the particle size distribution is indicated in Table 5-11 and the grain size distribution curve for the sample is presented in Appendix B.

Table 5-11: Summary of Grain Size Distribution on Granular sample

BOREHOLE NO.	SAMPLE NO.	SAMPLE DEPTH (m)	GRAIN SIZE DISTRIBUTION			
			GRAVEL (%)	SAND (%)	SILT (%)	CLAY (%)
BH22-12	SS1 (Granular)	0.8	33	57	10	

5.5.1.2 NATIVE SOILS

BEDROCK OF GUELPH FORMATION

Below the fill material in BH22-12, bedrock of the Guelph Formation was encountered at a depth of 3.1m corresponding to Elev. 319.7m.

Bedrock was proven by bedrock coring. Rock core logs are provided in Appendix A and the photographs of the rock cores are provided in Appendix C of this report.

Because of the method of drilling and sampling, the actual surface elevations of the bedrock may be different than indicated on the borehole logs. With augering or setting of HW casing into rock, the auger/casing may penetrate some of the more weathered bedrock and the coring may therefore begin below the bedrock surface. As such, the inferred bedrock surface level should not be considered accurate to better than ±0.5m.

5.5.2. GROUNDWATER CONDITIONS

Short-term (un-stabilized) groundwater was encountered in BH22-12 during drilling at a depth of 0.8m below ground surface.

It should be noted that the groundwater levels will vary and are subject to seasonal fluctuations in response to weather events.

5.6. WYNDHAM STREET

Three (3) boreholes (BH22-13 through BH22-15) were drilled along Wyndham Street to depths varying from 2.1 to 4.4m below the existing ground surface. A monitoring well of 50mm diameter was installed in one borehole (BH22-14).

5.6.1. SOIL CONDITIONS

5.6.1.1 PAVEMENT STRUCTURE / FILL MATERIAL

Pavement structure consisting of 75mm to 130mm of asphaltic concrete overlying 100mm of granular base/subbase was encountered at the location of the boreholes (BH22-13 to BH22-15). A summary of the pavement structure thicknesses at the borehole locations is listed in Table 5-12 below.

Table 5-12: Summary of Pavement structure Thicknesses at Borehole Locations

ROAD NAME	BOREHOLE NO.	PAVEMENT STRUCTURE THICKNESS	
		ASPHALTIC CONCRETE (m)	GRANULAR BASE/SUBBASE (m)
WYNDHAM ST N	BH22-13	130	100
	BH22-14	75	100
	BH22-15	75	100

Below pavement structure in the boreholes (BH22-13 to BH22-15), fill material consisting of silty sand to sand was encountered which extended to depths varying from 1.4m to 2.6m below the existing ground surface. Fill was present in a loose to very dense state based on measured SPT 'N' values ranging from 9 to more than 50 blows per 300 mm of penetration.

Grain size analyses were conducted on one (1) selected fill sample, the particle size distribution is indicated in Table 5-13 and the grain size distribution curves for the samples are presented in Appendix B.

Table 5-13: Summary of Grain Size Distribution on Fill samples

BOREHOLE NO.	SAMPLE NO.	SAMPLE DEPTH (m)	GRAIN SIZE DISTRIBUTION			
			GRAVEL (%)	SAND (%)	SILT (%)	CLAY (%)
BH22-15	SS3	1.5	3	72	19	6

5.6.1.2 NATIVE SOILS

SANDY SILT TILL / SILTY SAND TILL

Below fill material in Boreholes BH22-13 and BH22-15, native deposits of sandy silt till to silty sand till were encountered which extended to the termination depths of 2.1m to 4.4m below the existing ground surface. These deposits were found to be in a compact to very dense state based on measured SPT 'N' value of 13 to more than 50 blows per 300 mm of penetration. The moisture content in the tested samples was found to range from 5% to 13%. Cobbles and boulders are expected in the glacial till deposits.

Grain size analysis was conducted on a selected sandy silt till sample. The grain size distribution of the sample is indicated in Table 5-14 and the grain size distribution curve for the sample is presented in Appendix B.

Table 5-14: Summary of Grain Size Distribution on Sandy Silt Till sample

BOREHOLE NO.	**SAMPLE NO.**	**SAMPLE DEPTH (m)**	**GRAIN SIZE DISTRIBUTION**			
			GRAVEL (%)	SAND (%)	SILT (%)	CLAY (%)
BH22-13	SS3	1.5	4	23	61	12

COHESIONLESS DEPOSITS OF SAND / SILTY SAND

Below fill material in Borehole BH22-14, cohesionless deposits of sand and silty sand were encountered which extended to the termination depth of 4.2m below the existing ground surface. This deposit was found to be in a compact to dense state based on measured SPT 'N' value of 19 to 44 blows per 300 mm of penetration. Cobbles and boulders are expected in these cohesionless deposits.

Grain size analysis was conducted on a selected sand sample. The grain size distribution of the sample is indicated in Table 5-15 and the grain size distribution curve for the sample is presented in Appendix B.

Table 5-15: Summary of Grain Size Distribution on Sand sample

BOREHOLE NO.	**SAMPLE NO.**	**SAMPLE DEPTH (m)**	**GRAIN SIZE DISTRIBUTION**			
			GRAVEL (%)	SAND (%)	SILT (%)	CLAY (%)
BH22-14	SS4	2.6	17	67	16	

5.6.2. GROUNDWATER CONDITIONS

During drilling, short-term (un-stabilized) groundwater was encountered in BH22-13 at a depth of 1.4 below ground surface.

On April 26 and 27, 2022 the monitoring wells installed in BH22-14 and BH22-15 were found to be dry to 3.8m.

It should be noted that the groundwater levels will vary and are subject to seasonal fluctuations in response to weather events. Longer term groundwater level monitoring would be needed to assess the groundwater table seasonal level variations.

5.7. MACDONELL STREET

Three (3) boreholes (BH22-16, BH22-17 and BH22-30) were drilled along MacDonell Street to depth of 2.1m and 6.5m below the existing ground surface. A monitoring well of 50mm diameter was installed in one (1) borehole (BH22-17).

5.7.1. SOIL CONDITIONS

5.7.1.1 PAVEMENT STRUCTURE / FILL MATERIAL

Pavement structure consisting of 130mm to 150mm of asphaltic concrete overlying 460mm of granular base/subbase was encountered at the location of boreholes (BH22-16, BH22-17 and BH22-30). A summary of the pavement structure thicknesses at the borehole locations is listed in Table 5-16 below.

Table 5-16: Summary of Pavement structure Thicknesses at Borehole Locations

ROAD NAME	BOREHOLE NO.	PAVEMENT STRUCTURE THICKNESS	
		ASPHALTIC CONCRETE (m)	GRANULAR BASE/SUBBASE (m)
MACDONELL ST	BH22-16	150	460
	BH22-17	150	460
	BH22-30	130	460

Below pavement structure in the boreholes (BH22-16, BH22-17 and BH22-30), fill material consisting of sand and gravel, gravelly sand and silty sand was encountered which extended to depths varying from 1.5m to 2.3m below the existing ground surface. Fill was present in a loose to dense state based on measured SPT 'N' values ranging from 7 to 32 blows per 300 mm of penetration.

Grain size analysis was conducted on one (1) selected granular sample, the particle size distribution is indicated in Table 5-17 and the grain size distribution curve for the sample is presented in Appendix B.

Grain size analysis was conducted on one selected fill sample, the particle size distribution is indicated in Table 5-17 and the grain size distribution curve for the sample is presented in Appendix B.

Table 5-17: Summary of Grain Size Distribution on Granular and Fill samples

BOREHOLE NO.	SAMPLE NO.	SAMPLE DEPTH (m)	GRAIN SIZE DISTRIBUTION			
			GRAVEL (%)	SAND (%)	SILT (%)	CLAY (%)
BH22-16	SS1 (Granular)	0.1	30	56	14	
BH22-30	SS3	1.5	40	46	14	

5.7.1.2 NATIVE SOILS

SANDY SILT TILL

Below fill material in Borehole BH22-17, a glacial till deposit of sandy silt till was encountered which extended to 3.8m below the existing ground surface. This deposit was found to be in a dense to very dense state based on measured SPT 'N' values of 32 to more than 50 blows per 300 mm of penetration. The moisture content in the tested samples ranged from 5 to 8%.

CLAYEY SILT TILL

Below sandy silt till in Borehole BH22-17, glacial till deposits of clayey silt were encountered which extended to the termination depth of 4.4m below the existing ground surface. These deposits were found to be in a hard consistency based on measured SPT 'N' value of 42 blows per 300 mm of penetration. The moisture content in the tested sample was found to be 11%.

BEDROCK OF GUELPH FORMATION

Below the fill material in BH22-30, bedrock of the Guelph Formation was encountered at a depth of 2.3m corresponding to Elev. 322.5m.

Bedrock was proven by bedrock coring. Rock core logs are provided in Appendix A and the photographs of the rock cores are provided in Appendix C of this report.

5.7.2. GROUNDWATER CONDITIONS

The groundwater level measured within the monitoring well installed in BH22-17 was at 3.0m below grade (Elev. 319.3m) on April 26 and 27, 2022.

It should be noted that the groundwater levels will vary and are subject to seasonal fluctuations in response to weather events. Longer term groundwater level monitoring would be needed to assess the groundwater table seasonal level variations.

5.8. WOOLWICH STREET

Five (5) boreholes (BH22-18 to BH22-22) were drilled along Woolwich Street to depths of 2.1m to 6.3m below the existing ground surface. Monitoring wells of 50mm diameter were installed in two (2) boreholes BH22-19 and BH22-21.

5.8.1. SOIL CONDITIONS

5.8.1.1 PAVEMENT STRUCTURE / FILL MATERIAL

Pavement structure consisting of 75mm to 130mm of asphaltic concrete overlying 100mm to 530mm of granular base/subbase was encountered at the location of boreholes (BH22-18 to BH22-22). A summary of the pavement structure thicknesses at the borehole locations is listed in Table 5-18 below.

Table 5-18: Summary of Pavement structure Thicknesses at Borehole Locations

ROAD NAME	BOREHOLE NO.	PAVEMENT STRUCTURE THICKNESS	
		ASPHALTIC CONCRETE (m)	GRANULAR BASE/SUBBASE (m)
WOOLWICH ST	BH22-18	75	530
	BH22-19	130	460
	BH22-20	150	380
	BH22-21	150	460
	BH22-22	75	100

Below pavement structure in the boreholes (BH22-18 to BH22-22), fill material consisting of gravelly sand, gravelly silty sand, silty sand and clayey silt was encountered which extended to the termination depths of 2.1m in BH22-18 and BH22-20 and to depths of 1.5m to 1.8m in BH22-19, Bh22-21 and BH22-22. Fill was present in a loose to dense state or in a firm consistency based on measured SPT 'N' values ranging from 4 to 37 blows per 300 mm of penetration.

Grain size analyses were conducted on three (3) selected granular samples, the particle size distribution is indicated in Table 5-19 and the grain size distribution curve for the sample is presented in Appendix B.

Grain size analysis was conducted on one (1) selected fill samples, the particle size distribution is indicated in Table 5-19 and the grain size distribution curves for the samples are presented in Appendix B.

Table 5-19: Summary of Grain Size Distribution on Fill samples

BOREHOLE NO.	SAMPLE NO.	SAMPLE DEPTH (m)	GRAIN SIZE DISTRIBUTION			
			GRAVEL (%)	SAND (%)	SILT (%)	CLAY (%)
BH22-20	SS1(Granular)	0.1	33	53	14	
BH22-22	SS1 (Granular)	0.1	33	52	15	
BH22-18	SS2	0.8	20	58	22	
BH22-19	SS1 (Granular)	0.1	36	51	13	

5.8.1.2 NATIVE SOILS

SILTY CLAY TILL

Below fill material in Borehole BH22-21, a glacial till deposit of silty clay was encountered which extended to 2.6m below the existing ground surface. This deposit was found to be in a firm to hard consistency based on measured SPT 'N' values of 6 to more than 50 blows per 300 mm of penetration. The moisture contents in the tested samples were found to vary from 12% to 18%.

COHESIONLESS DEPOSITS OF SILTY SAND

Below fill material in Borehole BH22-19, a cohesionless deposit of silty sand was encountered which extended to 3.8m below the existing ground surface. This deposit was found to be in a compact state based on measured SPT 'N' value of 18 to 26 blows per 300 mm of penetration. Cobbles and boulders are expected in these cohesionless deposits.

SANDY SILT TILL

Below silty sand in BH22-19 and below fill material in BH22-2, glacial till deposit of sandy silt was encountered which extended to the termination depths of 2.1m to 4.4m of the boreholes. Sandy silt till was found to be in a compact state based on measured SPT 'N' values of 15 to 24 blows per 300 mm of penetration. The moisture content in the tested samples was found to be 9 and 10%.

BEDROCK OF GUELPH FORMATION

Below silty clay till in BH22-21, bedrock of the Guelph Formation was encountered at a depth of 2.6m corresponding to Elev. 321.4m.

Bedrock was proven by bedrock coring. Rock core logs are provided in Appendix A and the photographs of the rock cores are provided in Appendix C of this report.

5.8.2. GROUNDWATER CONDITIONS

The groundwater levels measured within the monitoring well installed in BH22-21 were at 3.7m to 4.0m below grade (Elev. 320.3m to 320.0m) on April 26 and 27, 2022.

The monitoring well installed within BH22-19 was found to be dry at 3.8m on April 26 and April 27, 2022.

It should be noted that the groundwater levels will vary and are subject to seasonal fluctuations in response to weather events. Longer term groundwater level monitoring would be needed to assess the groundwater table seasonal level variations.

5.9. NORWICH STREET EAST

One (1) borehole (BH22-23) was drilled along Norwich Street East to a depth of 4.1m below the existing ground surface.

5.9.1. SOIL CONDITIONS

5.9.1.1 PAVEMENT STRUCTURE / FILL MATERIAL

Pavement structure consisting of 75mm of asphaltic concrete overlying 100mm of granular base/subbase was encountered at the location of borehole (BH22-23).

Below pavement structure in the borehole (BH22-23), fill material consisting of silty sand and silty clay was encountered which extended to 2.6m below the existing ground surface. Fill was present in a very loose to compact state or in a firm consistency based on measured SPT 'N' values ranging from 7 to 17 blows per 300 mm of penetration.

Grain size analysis was conducted on a selected fill sample, the particle size distribution is indicated in Table 5-20 and the grain size distribution curve for the sample is presented in Appendix B.

Table 5-20: Summary of Grain Size Distribution on Fill sample

BOREHOLE NO.	SAMPLE NO.	SAMPLE DEPTH (m)	GRAIN SIZE DISTRIBUTION			
			GRAVEL (%)	SAND (%)	SILT (%)	CLAY (%)
BH22-23	SS1 (Granular)	0.1	16	62	22	

5.9.1.2 NATIVE SOILS

SANDY SILT TILL

Below fill material in BH22-23, a glacial till deposit of sandy silt was encountered which extended to the termination depth of 4.1m of the boreholes. Sandy silt till was found to be in a very dense state based on measured SPT ‘N’ values of more than 50 blows per 300 mm of penetration. The moisture content in the tested samples was found to be 8 to 9%.

5.9.2. GROUNDWATER CONDITIONS

During drilling, short-term (un-stabilized) groundwater was not encountered in BH22-23.

It should be noted that the groundwater levels will vary and are subject to seasonal fluctuations in response to weather events.

5.10. SUFFOLK STREET EAST

One (1) borehole (BH22-24) was drilled along Suffolk Street East to a depth of 4.2m below the existing ground surface. A monitoring well of 50mm diameter was installed within the borehole.

5.10.1. SOIL CONDITIONS

5.10.1.1 PAVEMENT STRUCTURE / FILL MATERIAL

Pavement structure consisting of 75mm of asphaltic concrete overlying 100mm of granular base/subbase was encountered at the location of the borehole (BH22-24).

Below pavement structure in the borehole (BH22-24), fill material consisting of sand and gravel, silty sand and clayey silt was encountered which extended to 2.6m below the existing ground surface. Fill was present in a loose to compact state based on measured SPT ‘N’ values ranging from 6 to 28 blows per 300 mm of penetration.

5.10.1.2 NATIVE SOILS

SANDY SILT TILL/SILTY SAND TILL

Below fill material in BH22-24, a glacial till deposit of sandy silt to silty sand were encountered which extended to 3.8m below the existing ground surface. Sandy silt till was found to be in a very dense state based on measured SPT ‘N’ values of more than 50 blows per 300 mm of penetration. The moisture content in the tested sample was found to be 10%.

Grain size analysis was conducted on a selected sandy silt till sample, the particle size distribution is indicated in Table 5-21 and the grain size distribution curve for the sample is presented in Appendix B.

Table 5-21: Summary of Grain Size Distribution on Sandy Silt Till sample

BOREHOLE NO.	SAMPLE NO.	SAMPLE DEPTH (m)	GRAIN SIZE DISTRIBUTION			
			GRAVEL (%)	SAND (%)	SILT (%)	CLAY (%)
BH22-24	SS5	3.1	3	39	46	12

COHESIONLESS DEPOSIT OF SAND

Below the glacial till deposit in BH22-24, a cohesionless deposit of sand was encountered which was found to extend to the termination depth of 4.2m of the borehole. This deposit was found to be in a very dense state based on measured SPT 'N' values of more than 50 blows per 300 mm of penetration.

5.10.2. GROUNDWATER CONDITIONS

Based on the groundwater level measurements within the monitoring well installed in BH22-24 on April 26 and 27, 2022, the monitoring well was found to be dry at 3.8m.

It should be noted that the groundwater levels will vary and are subject to seasonal fluctuations in response to weather events.

5.11. YARMOUTH STREET

One (1) borehole (BH22-25) was drilled along Yarmouth Street to a depth of 2.1m below the existing ground surface.

5.11.1. SOIL CONDITIONS

5.11.1.1 PAVEMENT STRUCTURE / FILL MATERIAL

Pavement structure consisting of 125mm of asphaltic concrete overlying 100mm of granular base/subbase was encountered at the location of the borehole (BH22-25).

Below pavement structure in the borehole (BH22-25), fill material consisting of sandy silt was encountered which extended to 1.5m. Fill was present in a very loose to a loose state based on measured SPT 'N' values ranging from 2 to 7 blows per 300 mm of penetration.

5.11.1.2 NATIVE SOILS

CLAYEY SILT TILL

Below fill material in BH22-25, a glacial till deposit of clayey silt was encountered which extended to 2.1m. Clayey silt till was found to be in a stiff consistency based on measured SPT 'N' value of

12 blows per 300 mm of penetration. The moisture content in the tested sample was found to be 11%.

Grain size analysis was conducted on a selected clayey silt till sample, the particle size distribution is indicated in Table 5-22 and the grain size distribution curve for the sample is presented in Appendix B.

Table 5-22: Summary of Grain Size Distribution on Clayey Silt Till sample

BOREHOLE NO.	SAMPLE NO.	SAMPLE DEPTH (m)	GRAIN SIZE DISTRIBUTION			
			GRAVEL (%)	SAND (%)	SILT (%)	CLAY (%)
BH22-25	SS3	1.5	3	30	48	19

5.11.2. GROUNDWATER CONDITIONS

During drilling, short-term (un-stabilized) groundwater was not encountered in BH22-25.

It should be noted that the groundwater levels will vary and are subject to seasonal fluctuations in response to weather events.

5.12. QUEBEC STREET

Two (2) boreholes (BH22-26 and BH22-27) were drilled along Quebec Street to depths of 5.2m and 2.1m below the existing ground surface. A monitoring well of 50mm diameter was installed in one (1) borehole (BH22-26).

5.12.1. SOIL CONDITIONS

5.12.1.1 PAVEMENT STRUCTURE / FILL MATERIAL

Pavement structure consisting of 75mm of asphaltic concrete overlying 100mm of granular base/subbase was encountered at the location of the boreholes (BH22-26 and BH22-27). A summary of the pavement structure thicknesses at the borehole locations is listed in Table 5-23 below.

Table 5-23: Summary of Pavement structure Thicknesses at Borehole Locations

ROAD NAME	BOREHOLE NO.	PAVEMENT STRUCTURE THICKNESS	
		ASPHALTIC CONCRETE (m)	GRANULAR BASE/SUBBASE (m)
QUEBEC ST	BH22-26	75	100
	BH22-27	75	100

Below pavement structure in the boreholes (BH22-26 and BH22-27), fill material consisting of silty sand was encountered which extended to the termination depth of 2.1m in BH22-27 and to 2.3m in BH22-26. Fill was present in a very loose to compact state based on measured SPT 'N' values ranging from 1 to 10 blows per 300 mm of penetration.

Grain size analysis was conducted on one (1) selected granular sample, the particle size distribution is indicated in Table 5-24 and the grain size distribution curves for the samples are presented in Appendix B.

Table 5-24: Summary of Grain Size Distribution on Granular sample

BOREHOLE NO.	SAMPLE NO.	SAMPLE DEPTH (m)	GRAIN SIZE DISTRIBUTION			
			GRAVEL (%)	SAND (%)	SILT (%)	CLAY (%)
BH22-26	SS1	(Granular)	32	53	15	

5.12.1.2 NATIVE SOILS

SILTY SAND TILL

Below fill material in BH22-26, glacial till deposits of sandy silt to silty sand were encountered which extended to the termination depth of 5.2m below the existing ground surface. Till was found to be in a compact to very dense state based on measured SPT 'N' values of 18 to more than 50 blows per 300 mm of penetration. The moisture content in the tested sample was found to range from 6 to 10%.

5.12.2. GROUNDWATER CONDITIONS

Based on the groundwater level measurements within the monitoring well installed in BH22-26 on April 26 and 27, 2022, the monitoring well was found to be dry at 5.2m.

It should be noted that the groundwater levels will vary and are subject to seasonal fluctuations in response to weather events. Longer term groundwater level monitoring would be needed to assess the groundwater table seasonal level variations.

5.13. CORK STREET

Two (2) boreholes (BH22-28 and BH22-29) were drilled along Cork Street to depths of 2.1m and 4.4m below the existing ground surface.

5.13.1. SOIL CONDITIONS

5.13.1.1 PAVEMENT STRUCTURE / FILL MATERIAL

Pavement structure consisting of 75mm to 125mm of asphaltic concrete overlying 100mm to 125mm of granular base/subbase was encountered at the location of boreholes (BH22-28 and BH22-29). A summary of the pavement structure thicknesses at the borehole locations is listed in Table 5-25 below.

Table 5-25: Summary of Pavement structure Thicknesses at Borehole Locations

ROAD NAME	BOREHOLE NO.	PAVEMENT STRUCTURE THICKNESS	
		ASPHALTIC CONCRETE (m)	GRANULAR BASE/SUBBASE (m)
CORK ST E	BH22-28	75	100
	BH22-29	125	125

Below pavement structure in the boreholes (BH22-28 and BH22-29), fill material consisting of gravelly sand, sand and sandy silt was encountered which extended to the termination depth of the boreholes. Fill was present in a very loose to compact state based on measured SPT 'N' values ranging from 2 to 13 blows per 300 mm of penetration.

Grain size analysis was conducted on one (1) selected granular sample, the particle size distribution is indicated in Table 4 6 and the grain size distribution curves for the samples are presented in Appendix B.

Grain size analysis was conducted on one (1) selected fill sample, the particle size distribution is indicated in Table 5-26 and the grain size distribution curve for the sample is presented in Appendix B.

Table 5-26: Summary of Grain Size Distribution on Granular and Fill samples

BOREHOLE NO.	SAMPLE NO.	SAMPLE DEPTH (m)	GRAIN SIZE DISTRIBUTION			
			GRAVEL (%)	SAND (%)	SILT (%)	CLAY (%)
BH22-28	SS1	0.1	35	53	12	
BH22-29	SS4	2.3	0	95	5	

5.13.1.2 NATIVE SOILS

SILTY SAND TILL

Below fill material in BH22-29, glacial till deposits of silty sand were encountered which extended to the termination depth of 4.4m below the existing ground surface. Till was found to be in a compact state based on measured SPT 'N' values of 15 to 26 blows per 300 mm of penetration.

5.13.2. GROUNDWATER CONDITIONS

During drilling, short-term (un-stabilized) groundwater was not encountered in the boreholes (BH22-28 and BH22-29).

It should be noted that the groundwater levels will vary and are subject to seasonal fluctuations in response to weather events.

5.14. DOUGLAS STREET

One (1) borehole (BH22-31) was drilled along Douglas Street to a depth of 2.1m below the existing ground surface.

5.14.1. SOIL CONDITIONS

5.14.1.1 PAVEMENT STRUCTURE / FILL MATERIAL

Pavement structure consisting of 125mm of asphaltic concrete overlying 460mm of granular base/subbase was encountered at the location of the borehole (BH22-31).

Below pavement structure in the borehole (BH22-25), fill material consisting of silty sand was encountered which extended to 1m below the existing ground surface. Fill was present in a compact state based on measured SPT 'N' value of 13 blows per 300 mm of penetration.

5.14.1.2 NATIVE SOILS

SILTY SAND TILL

Below fill material in BH22-31, glacial till deposit of silty sand was encountered which extended to the termination depth of 2.1m below the existing ground surface. Till was found to be in a dense state based on measured SPT ‘N’ value of 44 blows per 300 mm of penetration.

5.14.2. GROUNDWATER CONDITIONS

During drilling, short-term (un-stabilized) groundwater was not encountered in BH22-31.

It should be noted that the groundwater levels will vary and are subject to seasonal fluctuations in response to weather events.

5.15. WELLINGTON STREET EAST

Three (3) boreholes (BH23-1 to BH23-3) were drilled along Wellington Street East to depths ranging from 2.9m to 6.2m below the existing ground surface. Two boreholes (BH23-1 and BH23-3) were equipped with monitoring wells.

5.15.1. SOIL CONDITIONS

5.15.1.1 PAVEMENT STRUCTURE / FILL MATERIAL

Pavement structure, consisting of a 150mm thick layer of asphaltic concrete overlying a 260mm to 300mm thick layer of granular base/subbase, was encountered at the location of the boreholes.

Below pavement structure in the boreholes (BH23-1 to BH23-3), fill material consisting of sand and gravel and silty sand was encountered which extended to depths ranging from 0.8m to 3.1m below the existing grade. Traces of cinders/slag were also present with fill material. Fill was present in a very loose to very dense state based on measured SPT ‘N’ values ranging from 3 to 52 blows per 300 mm of penetration.

Grain size analysis was conducted on two (2) selected granular samples, the particle size distribution is indicated in

Table 5-27 and the grain size distribution curves for the samples are presented in Appendix B.

Table 5-27: Summary of Grain Size Distribution on Granular samples

BOREHOLE NO.	SAMPLE NO.	SAMPLE DEPTH (m)	GRAIN SIZE DISTRIBUTION			
			GRAVEL (%)	SAND (%)	SILT (%)	CLAY (%)
BH23-1	SS1A	0.15	52	36	12	
BH23-3	SS1A	0.15	41	47	12	

5.15.1.2 NATIVE SOILS

COHESIONLESS DEPOSITS OF SAND AND GRAVEL/SANDY GRAVEL

Below fill material in all boreholes BH23-1 to BH23-3, cohesionless deposits of sand and gravel/sandy gravel were encountered which extended to bedrock surface in BH23-1 and to the termination depths of 2.9m to 4.3m in BH23-2 and BH23-3. These cohesionless deposits were found to be in a compact to very dense state based on measured SPT 'N' values of 12 to over 50 blows per 300 mm of penetration.

Grain size analysis was conducted on three selected sand and gravel/sandy gravel samples. The grain size distribution of the samples is indicated in Table 5-28 and the grain size distribution curves for the sample is presented in Appendix B. Cobbles and boulders are expected in these cohesionless deposits.

Table 5-28: Summary of Grain Size Distribution on Sand and Gravel/Sandy Gravel samples

BOREHOLE NO.	SAMPLE NO.	SAMPLE DEPTH (m)	GRAIN SIZE DISTRIBUTION			
			GRAVEL (%)	SAND (%)	SILT (%)	CLAY (%)
BH23-2	SS3	1.5	52	34	14	
BH23-2	SS4	2.3	44	44	12	
BH23-3	SS6+7	3.8	43	45	12	

BEDROCK OF GUELPH FORMATION

Below sandy gravel in BH23-1, bedrock of the Guelph Formation was encountered at a depth of 2.7m corresponding to Elev. 307.3m.

Bedrock was proven by bedrock coring. Rock core logs are provided in Appendix A and the photographs of the rock cores are provided in Appendix C of this report. In general, the recovered cores consist of moderately weathered beige to light grey dolomite, typically fossiliferous and vuggy.

5.15.2. GROUNDWATER CONDITIONS

Groundwater levels measured in the monitoring wells installed in Boreholes BH23-1 and BH23-3 were found to be at depths of 1.7m and 3.3m below the existing ground surface, respectively.

It should be noted that the groundwater levels will vary and are subject to seasonal fluctuations in response to weather events. Longer term groundwater level monitoring would be needed to assess the groundwater table seasonal level variations.

5.16. WYNDHAM STREET SOUTH

Two (2) boreholes (BH23-4 and BH23-5) were drilled along Wyndham Street South to depths of 6.2m and 7.6m below the existing ground surface. One borehole (BH23-4) was equipped with a monitoring well.

5.16.1. SOIL CONDITIONS

5.15.1.1 PAVEMENT STRUCTURE / FILL MATERIAL

Pavement structure consisting of a 150mm thick layer of asphaltic concrete overlying a 200mm to 250mm thick layer of granular base/subbase was encountered at the location of the boreholes.

Below pavement structure in the boreholes (BH23-4 and BH23-5), fill material consisting of sand and gravel and silty sand with gravel was encountered which extended to a depth of 1.5m below the existing grade. Fill was present in a compact to dense state based on measured SPT 'N' values ranging from 16 to 44 blows per 300 mm of penetration.

5.15.1.2 NATIVE SOILS

SANDY SILT TILL

Below fill material in the boreholes BH23-4 and BH23-5, glacial till deposits of sandy silt were encountered underlain by the bedrock of Guelph Formation. Sandy silt till was found to be in a loose to very dense state based on measured SPT 'N' values of 6 to over 50 blows per 300 mm of penetration.

Grain size analysis was conducted on one selected sandy silt till, gravelly sample. The grain size distribution of the sample is indicated in Table 5-29 and the grain size distribution curves for the sample is presented in Appendix B. Cobbles and boulders are expected in these glacial till deposits.

Table 5-29: Summary of Grain Size Distribution on Sandy Silt Till sample

BOREHOLE NO.	SAMPLE NO.	SAMPLE DEPTH (m)	GRAIN SIZE DISTRIBUTION			
			GRAVEL (%)	SAND (%)	SILT (%)	CLAY (%)
BH23-5	SS4	2.3	23	25	48	4

BEDROCK OF GUELPH FORMATION

Below sandy silt till in BH23-4 and BH23-5, bedrock of the Guelph Formation was encountered at a depths of 2.3m and 3.8m corresponding to Elev. 312.2m and 316.2m.

Bedrock was proven by bedrock coring. Rock core logs are provided in Appendix A and the photographs of the rock cores are provided in Appendix C of this report. In general, the core was found to consist of beige, slightly weathered to moderately weathered crystalline to fossiliferous dolostone.

5.16.2. GROUNDWATER CONDITIONS

Groundwater level measured in the monitoring well installed in Boreholes BH23-4 was found to be at a depth of 3.3m below the existing ground surface.

It should be noted that the groundwater levels will vary and are subject to seasonal fluctuations in response to weather events. Longer term groundwater level monitoring would be needed to assess the groundwater table seasonal level variations.

5.17. BEDROCK OF GUELPH FORMATION

Bedrock of Guelph Formation, consisting of dolomite or dolomitic limestone was encountered at the location of boreholes BH23-1, BH23-4, BH23-5, BH22-12, BH22-21 and BH22-30 at depths varying from 2.3m to 3.8m below the existing ground surface, corresponding to Elev. 307.3m to 322.5m. The depth and elevation of bedrock surface at the borehole locations is presented in Table 5-30.

Bedrock was proven by bedrock coring in these six (6) boreholes (BH23-1, BH23-4, BH23-5, BH22-12, BH22-21and BH22-30). The rock core log is provided in Appendix A and photographs of the rock cores are provided in Appendix C of this report.

Table 5-30: Approximate Depths and Elevations of Bedrock Surface at Borehole Locations

BOREHOLE NO.	APPROXIMATE BEDROCK SURFACE DEPTH (m)	APPROXIMATE BEDROCK SURFACE ELEVATION (m)	NOTE
BH22-12	3.1	319.7	Rock Coring
BH22-21	2.6	321.4	Rock Coring
BH22-30	2.3	322.5	Rock Coring
BH23-1	2.7	307.3	Rock Coring
BH23-4	2.3	312.2	Rock Coring
BH23-5	3.8	316.2	Rock Coring

Because of the method of drilling and sampling, the actual surface elevations of the bedrock may be different than as indicated on the borehole log. With augering or setting of HW casing into rock, the auger/casing may penetrate some of the more weathered rock and the coring may therefore begin below the bedrock surface. As such, the inferred bedrock surface level should not be considered accurate to better than ±0.5m

Visual examination of the recovered rock cores indicates that the Guelph Formation typically consists of moderately weathered to slightly weathered, light brown to white, weak to strong dolostone or dolomitic limestone. The texture and degree of weathering varies considerably with depth and location, ranging from fresh crystalline rock to moderately weathered, vuggy, fossiliferous rock.

The descriptive terms used on the record of rock cores and throughout this report are explained on the "Explanation of Terms Used in the Bedrock Core Log" sheet in Appendix A. In general, the conventions of the International Society for Rock Mechanics (ISRM) are adopted herein. Detailed descriptions of the index properties are presented in the following paragraphs.

TOTAL CORE RECOVERY (TCR)

The total core recovery indicates the total length of rock core recovered expressed as a percentage of the actual length of the core run. The total core recovery ranged from 58% to 100%, with an average value of 92%.

SOLID CORE RECOVERY (SCR)

The solid core recovery is the total length of solid, full diameter rock core that was recovered, expressed as a percentage of the length of the core run. Solid core recovery generally ranged from 0% to 100% with an average value of 75% and appears to generally improve with depth.

ROCK QUALITY DESIGNATION (RQD)

The rock quality designation index is obtained by measuring the total length of recovered rock core pieces which are longer than 100mm and expressing their sum total length as a percentage of the length of the core run. RQD is a function of the frequency of joints, bedding plane partings and fractures in the rock cores. On the basis of the recorded RQD values which range between 0 and 100% with an average value of 58%, the rock quality is estimated to be "very poor" to "excellent" quality.

UNCONFINED COMPRESSIVE STRENGTH (UCS) AND POINT LOAD INDEX STRENGTH

To determine the unconfined compressive strength (UCS) of the intact rock, a total of six (6) rock samples of suitable length core were selected for uniaxial compressive strength testing. The test results are presented in Appendix C. The unconfined compressive strength (UCS) of the tested samples of Guelph Formation ranged from 20.0MPa to 70.4MPa. Based on the above-mentioned limited number of unconfined compressive strength test results, the Guelph Formation rock samples, as tested, can be classified as "weak" to "strong" rock under ISRM strength convention.

Point load index strength tests were performed on twenty-four (24) rock samples of the Guelph Formation. The test results are presented in Table C1 in Appendix C. We have utilized the empirical approximate relationship between unconfined compressive strength (UCS) and point load index strength as follows:

UCS [MPa] ≈ 24.0 $I_{S(50)}$

where $I_{S(50)}$ is the point load index strength in MPa for a 50 mm equivalent diameter core. This is an approximate correlation after Franklin and Hoek, which may overestimate the UCS value.

For the Guelph Formation samples tested, the equivalent Point-Load derived unconfined compressive strength of the samples was inferred to range from 28 to 126MPa in the axial direction and 23 to 127MPa in the diametral direction. These values are indicative of generally "weak" to "very strong" rock under ISRM strength convention.

FRACTURE INDEX

When logging the rock cores, the fracture Index (i.e., the number of fractures for each 0.3m length of core) was also recorded. It was observed that the planes of weaknesses along which the cores tended to break. The Fracture Index is expressed as the number of discontinuities per 300 mm (1ft).

WEATHERING

The degree of weathering ranged generally from moderately weathered to slightly weathered as indicated on the Records of Rock Cores.

CERCHAR ABRASIVENESS

CERCHAR Abrasiveness index tests were conducted to measure the relative difference of hardness of a steel stylus tip and the rock specimen surface. The test procedure follows ASTM D7625-10 “Standard Test Method for Laboratory Determination of Abrasiveness of Rock Using the Cerchar Method”. Five (5) rock samples were tested for Cerchar abrasiveness index. The laboratory test results are provided in Appendix C.

5.18. GROUNDWATER CONDITIONS

Groundwater levels were measured in the installed monitoring wells on April 26, April 27, 2022 and August 29, 2023. Groundwater levels were found at depths ranging from 1.7m to 4m below the existing ground surface in the monitoring wells installed in Boreholes BH23-1, BH23-3, BH23-4, BH22-4, BH22-17 and BH22-21 corresponding to Elev. 308.4m to 330.5m as listed in Table 5-31. All other monitoring wells were found to be dry.

Table 5-31: Summary of Groundwater Level Observations in the Monitoring Wells

STREET NAME	BH NO.	GROUND SURFACE ELEVATION (m ASL)	SOIL TYPE AT SCREEN LOCATION / (DEPTH m)	DATE OF OBSERVATION	DEPTH OF GROUND-WATER (m)	GROUNDWATER TABLE ELEVATION (m ASL)
DUBLIN STREET	BH22-2	325.32	Silty sand (Fill) (2.29-3.81)	April 26, 2022 April 27, 2022	Dry Dry	Dry Dry
NORFOLK STREET	BH22-4	333.31	Sandy silt till/silty clay (2.29-3.81)	April 26, 2022 April 27, 2022	2.85 2.86	330.46 330.45
	BH22-9	325.58	Sandy silt (2,29-2.90)	April 26, 2022 April 27, 2022	Dry Dry	Dry Dry
WYNDHAM STREET N	BH22-14	328.27	Silty sand/sand (Fill) (2.29-3.81)	April 26, 2022 April 27, 2022	Dry Dry	Dry Dry
	BH22-15	324.08	Silty sand (Fill) (2.29-3.81)	April 26, 2022 April 27, 2022	Dry Dry	Dry Dry

STREET NAME	BH NO.	GROUND SURFACE ELEVATION (m ASL)	SOIL TYPE AT SCREEN LOCATION / (DEPTH m)	DATE OF OBSERVATION	DEPTH OF GROUND-WATER (m)	GROUNDWATER TABLE ELEVATION (m ASL)
MACDONELL STREET	BH22-17	322.37	Gravely sand (Fill)/clayey silt till	April 26, 2022 April 27, 2022	2.57 3.03	319.80 319.34
WOOLWICH STREET	BH22-19	330.45	Gravely sand (Fill)/sandy silt till	April 26, 2022 April 27, 2022	Dry Dry	Dry Dry
	BH22-21	323.98	Silty clay till/bedrock (2.29-3.81)	April 26, 2022 April 27, 2022	3.72 3.96	320.26 320.02
SUFFOLK STREET	BH22-24	331.53	Silty sand/sandy silt till	April 26, 2022 April 27, 2022	Dry Dry	Dry Dry
QUEBEC STREET	BH22-26	327.02	Sandy silt (Fill)/silty sand Fill (3.66-518)	April 26, 2022 April 27, 2022	Dry Dry	Dry Dry
WELLINGTON ST. E	BH23-1	310.05	Bedrock (3.1-6.1)	AUG. 29, 2023	1.68	308.37
	BH23-3	313.40	Fill/Sand and Gravel (2.6-4.3)	AUG. 29, 2023	3.33	310.07
WYNDHAM ST. S	BH23-4	314.51	Bedrock (3.1-6.1)	AUG. 29, 2023	3.31	311.20

It should be noted that the groundwater levels can vary and are subject to seasonal fluctuations in response to weather events. The long-term groundwater table may be higher than that shown on Table 5-31. Longer term groundwater level monitoring is required to confirm the groundwater table and seasonal groundwater level variations.

6. SOIL CORROSIVITY AND WATER-SOLUBLE SOIL SULPHATE TEST RESULTS

Seven (7) soil samples were analysed for corrosivity parameters and water-soluble sulphate content. The test results are presented in Appendix D and are summarized in Table 6-1 and Table 6-2.

Table 6-1: Summary of Soil Corrosivity Parameters

BH NO./ SAMPLE NO./ DEPTH (m)	SOIL TYPE	RESISTIVITY (ohm-cm) (POINT)	PH (POINT)	REDOX POTENTIAL (MV) POINT)	SULPHIDE (*mg/kg) (POINT)	MOISTURE CONTENT (%) (POINT)	TOTAL POINTS
BH22-4/ SS3	Silty Sand (Fill)	530 (10)	8.10 (0)	284 (0)	Trace (2)	11 (0)	12
BH22-14/ SS4	Sand (Fill)	360 (10)	8.28 (0)	236 (0)	Trace (2)	6 (0)	11
BH22-21/ SS2	Clayey Silt (Fill)	290 (10)	8.26 (0)	283 (0)	Trace (2)	17 (1)	13
BH22-30/ SS3	Sand and Gravel (Fill)	1400 (10)	8.31 (0)	232 (0)	Trace (2)	7 (0)	12
BH23-1/ SS3	Sandy Gravel	1200 (10)	8.39 (0)	208 (0)	Trace (2)	12.6 (0)	12
BH23-3/ SS5	Sand and Gravel	980 (10)	8.20 (0)	229 (0)	Trace (2)	12.1 (0)	12
BH23-4/ SS3	Sandy Silt Till	710 (10)	8.21 (0)	218 (0)	Trace (2)	12.3 (0)	12

Scoring of 10 or more on the basis of these test results is indicative, according to Table A.1 of ANSI/AWWA, C105/A21.5-10, of soil which is supportive of corrosion towards gray or ductile cast iron pipe. The scoring of all samples (shown in bold and underlined in Table 6-1exceeded 10. Based on this, these soils would be supportive of corrosion, necessitating corrosion protection measures.

Seven (7) samples were analysed for sulphate content. The test results are summarized in Table 6-2.

Table 6-2: Summary of Water-Soluble Soil Sulphate Content Test Results

BH NO./ SAMPLE NO./ DEPTH (m)	SOIL TYPE	WATER SOLUBLE SULPHATE (PPM)	PH
BH22-4/ SS3	Silty Sand (Fill)	<20	8.10
BH22-14/ SS4	Sand (Fill)	40	8.28
BH22-21/ SS2	Clayey Silt (Fill)	39	8.26
BH22-30/ SS3	Sand and Gravel (Fill)	<20	8.31
BH23-1/ SS3	Sandy Gravel	40	8.39
BH23-3/ SS5	Sand and Gravel	98	8.20
BH23-4/ SS3	Sandy Silt Till	76	8.21

The above test results indicate that the water-soluble soil sulphate degree of exposure for sulphate attack towards buried concrete is 'low' according to Table 3 of CSA Standard CAN/CSA-A23.1-09.

7. DISCUSSION AND PRELIMINARY RECONMMENDATIONS

In the following sections, the soil and bedrock conditions are interpreted as relevant to the preliminary design requirements for generic utilities. The depth of watermain/sewers is not known at the time of writing this report and as such, our recommendations remain preliminary and generalized in nature.

Preliminary recommendations described in this report must not be considered as being specifications or as being the only suitable methods. The readers of this report are also reminded that the conditions are known only at the borehole locations and in view of the limited number of the boreholes, conditions may vary significantly between the boreholes.

7.1. OVERVIEW OF SUBSURFACE CONDITIONS

The subsurface conditions revealed in the boreholes generally consisted of pavement structure overlying fill material consisting of sand and gravel, gravelly sand, sand, silty sand, clayey silt and silty clay. Native soils consisting of silty clay, clayey silt, silty clay till, clayey silt till, sandy silt till to silty sand till and cohesionless deposits of sand and gravel, sandy gravel, gravelly sand, sand, silty sand and sandy silt were encountered in the boreholes. Cobbles and boulders are expected in these deposits.

Bedrock of the Guelph Formation was encountered in three boreholes (BH23-1, BH23-4, BH23-5, BH22-12, BH22-21 and BH22-30) at depths ranging from 2.3m to 3.8m below existing ground surface, corresponding to Elev. 307.3m to 322.5m.

Groundwater levels were measured in the monitoring wells on April 26, April 27, 2022 and August 29, 2023. Groundwater levels were found to be at depths ranging from 1.7m to 4m below the existing ground surface in the monitoring wells installed in Boreholes BH23-1, BH23-3, BH23-4, BH22-4, BH22-17 and BH22-21. All other monitoring wells were found to be dry.

Perched water should be expected within shallow granular fill. Perched water should also be anticipated whenever existing utility bedding may be intercepted by new trenches. For the design purposes, the groundwater level shall be taken as 1 m higher than the measured groundwater level in the nearest monitoring well installed within the overburden or the regional flood level, whichever is higher.

7.2. COBBLES AND BOULDERS

Boulders/cobbles were inferred based on auger grinding in all of the glacial tills including those of silty sand and sandy silt and clayey silt textures and the and cohesionless soils consisting of gravelly sand, sand, silty sand and sandy silt. A very slow rate of drilling advancement was experienced during augering of these deposits given the presence of cobbles/boulders. The current investigation method of borehole drilling could not determine the size and frequency of the cobbles and boulders. Test pits would be required at the design stage to better assess cobble and boulder frequency, distribution and sizes.

Cobbles are defined as rock fragments that cannot pass through a screen with 75 mm square openings and are less than 300 mm in maximum dimension. Boulders are defined as rock fragments with their minimum dimension being equal to or greater than 300 mm. Removal of cobbles during open cut excavations is considered part of routine construction and these materials will not be considered as obstructions for this project. Accumulations of 'nested' cobbles, however, can be particularly troublesome during trenching and trenchless work and would often be considered as constituting a 'changed ground condition'. Again, such a condition could only be assessed using test pits as part of future design stage site investigations.

7.3. FROST DEPTH

Watermains must have at least 1.7 m of earth cover for frost protection purposes.

7.4. UTILITY (WATERMAINS/SEWERS) INSTALLATION USING OPEN CUT METHODS

Based on 'typical' excavation depths for open cut installation of say less than 5 to 6m below ground surface, excavations for the construction of the watermains/sewers will primarily be through pavement structure, fill, and into the underlying cohesionless deposits of sand, silty sand, sandy silt and glacial till deposits of sandy silt to silty sand and clayey silt to silt. Locally, such trenches could potentially encounter bedrock at some locations such as Eramosa Road (BH22-12), Woolwich Street (BH22-21), MacDonell Street (BH22-30), Wellington Street E (BH23-1) and Wyndham Street South (BH23-4 and BH23-5).

Trenching in the above noted soil types using conventional excavating equipment is feasible, understanding that some boulder removal could be required in the glacial till and cohesionless soils. Anomalous trenching conditions with greater potential for wall collapse could also occur in instances where the new sewer trench encroaches on existing utility trenches. Perched water might also be encountered in such cases where existing trench backfill and bedding are intercepted by the new trench. Extending of trenches into bedrock, such as might be required for a gravity sewer application, would present special challenges and site-specific evaluation would be required to determine if rock removals would be feasible or not using hoe rams and if so, what potential impacts to adjacent utilities and structures could be anticipated due to vibration. Given the medium strong to strong rock unconfined strength, rock removals in narrow trenches using mechanical methods will likely be slow, laboured and hard on equipment. The measured UCS of intact rock samples approaches the limit suggested by some examiners at which mechanical excavation is not feasible. The ability to remove rock in this manner without line drilling will likely depend on the RQD/rockmass fracturing. For future projects involving sewers where shallow rock may intercept, or lie near to the invert, closely spaced boreholes coupled with geophysical surveys (seismic refraction) would be warranted and these findings would need to be considered when making a decision on gravity flow design versus use of pumping stations/forcemains.

The anticipated behaviour of the soils as related to the support of the pipe and the stability of open cut excavations are summarized in Table 7-1 and is discussed in the following paragraphs.

Table 7-1: Soil Behaviour in Open Cut

SOIL TYPE	PIPE SUPPORT	STABILITY DURING CONSTRUCTION IN OPEN CUT EXCAVATION	POSSIBLE MEANS OF GROUNDWATER CONTROL
FILL	Not suitable to Potentially Suitable depending on state of	Stable at 1.5H:1V	Gravity drainage and pumping from filtered sumps established inside the base of trench
NATIVE COHESIONLESS SOILS / SANDY SILT TO SILTY SAND TILL/SILT/SANDY SILT/SILTY SAND/SAND AND	Satisfactory if properly dewatered	Stable at 1.5H:1V (unstable below groundwater table)	Closely spaced vacuum well points for trenches <5m deep
CLAYEY SILT TILL/SILTY CLAY TILL	Satisfactory	Stable at 1H:1V	Gravity drainage and pumping from filtered sumps established inside the base of trench

7.4.1. Trench Stability and Dewatering

Excavations in overburden can be carried out with heavy hydraulic excavators.

The groundwater levels measured in the monitoring wells installed in BH23-1, BH23-4, BH23-5, BH22-4, BH22-17 and BH22-21 were found to be at depths varying from 1.7m to 4m below the existing ground surface corresponding to Elev. 308.4m to 330.5m.

Perched water may be present in the fill material and seepage of perched water should be expected into the excavation.

Wet to saturated cohesionless deposits (sandy silt, silt and silty sand, sand and gravel, sandy gravel) were encountered in the boreholes BH23-1, BH23-2, BH23-3, BH22-4, BH22-10 and BH22-17 at depths of 1.5 to 2.3m. In areas where the trenches will reach cohesionless deposits below groundwater, positive dewatering will be required. Any excavation in wet to saturated cohesionless deposits will require groundwater control. It is expected that much of the water seepage should be controllable by the use of conventional pumping from collection sumps for trenches. However, more elaborate dewatering procedures such as closely spaced vacuum well points may be required if the flow from fill material or native cohesionless deposits is not controlled by conventional methods. The groundwater table must be lowered to at least 0.5m below the deepest excavation base. Otherwise, it will result in an unstable base and flowing sides.

All excavations must be carried out in accordance with the most recent Occupational Health and Safety Act (OHSA). In accordance with OHSA, the existing fill can be classified as Type 3 Soil above the water table and as Type 4 Soil below the groundwater table. The stiff to hard clayey silt till and silty clay till deposits can be classified as Type 2 Soil above the groundwater table and Type 3 below groundwater table. Cohesionless soils (existing granular, silt, sandy silt, silty sand, sand, gravelly sand, sandy gravel, sand and gravel, sandy silt till and silty sand till) can be classified as Type 3 Soils above groundwater and Type 4 Soil below the groundwater table. These are generic, broad-brush statements and must not be used for detail design/specifications of specific projects where closely spaced boring will be needed,

Unsupported excavations would be temporarily stable for short periods at an angle of 1.5H:1V in the existing fill material, silt, sandy silt, silty sand, sand, gravelly sand, sandy gravel, sand and gravel, sandy silt till/silty sand till above water table and 1H:1V in the cohesive clayey silt till/silty clay till. Below the groundwater table, unsupported excavations in the cohesionless soils as well as silty sand till/sandy silt till cannot safely proceed until the groundwater table is lowered to a minimum depth of 0.5m below the base of the excavation.

It should be noted that most of the Site soils will contain cobbles or boulders to some degree. Provisions must be made in excavation and trenching related contracts for the removal and disposal of boulders or other obstructions in overburden. A site-specific test pitting program would be needed to support the assessment of boulder frequency, distribution and sizes.

Reference to Drawing No. 7 indicates zones in which some degree of movement of the ground can be anticipated as a consequence of trench excavation. In this respect, it should also be noted that less ground movements will be experienced outside the excavation if the sides of the excavation are properly supported by tight, braced sheeting than if the sides are unsupported. Ground movements would be further reduced if the bracings were to be pre-stressed.

In instances where the proposed utilities will lie in separate trenches or in common trench within the zone of influence of one another, the deepest utilities must be installed first and properly backfilled under engineering supervision, prior to placement of the shallower adjacent utility. The stability of a vertical bench within a common trench, separating two new utilities needs to be assessed by this office for stability/feasibility.

7.4.2. USE OF TRENCH BOX FOR TRENCH WALL SUPPORT

Where permissible under the OSHA, contractors often elect to utilize trench boxes for temporary trench support.

While in many situations, the use of trench boxes can result in a high rate of productivity in trenching, it is not without some technical drawbacks. These include:

- Increased loss of ground relative to many other shoring methods; and
- Reduced ability to compact backfill between the trench wall and trench box.

Ground loss, raveling and/or loosening of soils will occur when using a trench box prior to its installation and while moving the box, particularly in pre-existing fill as present at this site.

Granular courses below existing pavements are particularly susceptible and significant undermining can occur. It is important that the trench not be over-excavated to ensure a tight fit between the box and the trench walls. Trench boxes need to be installed expediently. When moving the box, the void space between its outer walls and the trench must be backfilled and compacted. This may require raising the box sequentially prior to sliding it laterally. If this is not done, post- construction settlements will occur along the trench walls.

Where trench boxes are used in the existing roadways, it is prudent to expect pavement structure settlement along both sides of the trench. In such cases, following backfilling of the trench, road reconstruction should include a provision for saw cutting of the asphalt and concrete road base at least 0.3 m back from the trench walls, recompaction of the upper trench backfill and then paving.

It is recommended to follow OPSD 509.010 Pavement Reinstatement for Utility Cuts in Hot Mix Pavement (i.e., pavement step joint detail) or the equivalent City of Guelph Standards as far as the joint between new pavement patches and existing pavement is concerned.

Where trench depths exceed 6.0 m and in Type 4 Soils of any trench depth, “Engineered Support Systems” as defined under the OHSA are mandated under the OHSA.

7.4.3. Trenching Adjacent to Existing Services

In areas where a new utility trench will impinge on existing utility trenches or will pass through existing fill soils, unstable trench conditions can occur, particularly where granular backfill, clear stone, high performance backfill, or poorly compacted fill of any type are present.

In such cases, raveling of the pre- existing fill and high rates of water infiltration through utility bedding can potentially occur which can, in severe cases, put the stability of the adjacent utility in jeopardy. As such, a higher standard of care in shoring is needed where the sewer trench is located closer than 0.75H to an adjacent trench, where 'H' is the depth of the deeper cut. The use of trenching boxes is poorly suited in this instance, since they do not provide adequate intimate lateral support to the sides of the cut and considerable loss of ground can occur prior to insertion of the box. Closed sheeting, Slide Rail, or other pre-installed shoring measures are more suitable.

7.4.4. Pipe Bedding and Cover

It is anticipated that the existing undisturbed native soils will provide adequate support for sewers and will allow the use of normal Class B type bedding.

The bedding should meet the standard of the current Ontario Provincial Standard Specifications (OPSS) and/or standards set by the local municipalities (i.e., Region of Waterloo, City of Guelph).

It is noted that the existing loose fill materials (e.g., in BH23-3, BH23-5, BH22-6, BH22-10, BH22-12, BH22-21, BH22-23 and BH22-24) likely require sub-excavation to reach more competent native soils. The subgrade condition must be inspected and approved by qualified geotechnical personnel prior to placing bedding. If weak/soft material is encountered, it must be sub-excavated and replaced with compacted OPSS Granular "A" material.

Cover material, at least 300 mm above the top of the pipe, should consist of Granular A or Granular B Type I with a maximum particle size of 25 mm. Finer-graded cover granulars may be required for small diameter PVC pipe.

The minimum bedding thickness should be 150 mm but this should be increased as dictated by the pipe diameter and/or aforementioned specifications.

Granular materials should be placed in maximum 200 mm thick lifts. The granular bedding and pipe cover materials should be compacted to 98% of Standard Proctor Maximum Dry Density (SPMDD) at a placement water content within 2 percent of the materials optimum. Care should be exercised when compacting the cover material on top of the pipe as well as beside them to avoid damaging them. The use of light, hand operated compaction equipment is required in these areas.

In order to minimize any long-term drainage effects caused by the granular bedding of the pipes, it is recommended that bentonite trench cut-offs (i.e. “trench plugs”) be constructed around the pipe and through the bedding at intervals of approximately 100m in areas where the pipe will lie below the groundwater table. Use of concrete collars in place of bentonite is <u>not</u> recommended as this may induce point loading onto the pipe.

7.4.5. Thrust Block Bearing Resistance

An allowable (or SLS) bearing resistance of 75 kPa and factored ULS bearing resistance of 115 kPa can be used in the design of thrust blocks constructed against native soils or against engineered fill. Where loose fill is encountered, the thrust blocks must be bear against a minimum of 1.0 m thick engineered fill. This will require re-excavation of existing fill and replacement with engineered fill placed in layers and compacted to 100% SPMDD.

7.4.6. Backfilling and Degree of Compaction

Within the roadway, backfilling of the trenches must be done using a well-graded, compacted granular soil such as Granular ‘A’ and ‘B’ material. The use of such material, if thoroughly compacted, will reduce the post construction settlements to a negligible amount and may also expedite the compaction process. In this instance, however, frost response characteristics of non-frost susceptible granular fill and the frost susceptible native soils would be different giving rise to differential frost heave or movement. In this case it would be prudent to use as backfill the on-site excavated, naturally occurring soils to match the existing conditions within the frost zone (i.e. within 1.5 m depth) or to provide a frost taper zone (i.e. to provide a zone of taper to prevent a sudden change in frost heave characteristics to reduce the effects of frost heave).

In any case, the degree of compaction of the trench should be at least 98% of the material’s Standard Proctor Maximum Dry Density (SPMDD) and the placement water content must be within 2 percent of the materials optimum water content. This value should be increased to 100% of SPMDD within 1.5 m of the road surface.

The granular pavement sub-base and base materials should be compacted to at least 100% of their respective SPMDD at a placement water content within 2 percent of the materials optimum and the boulevards should be compacted to 95% of their respective SPMDD. If future widenings are contemplated in boulevard areas, then the compaction specification in boulevards must be increased to 100% of SPMDD.

7.4.7. LATERAL EARTH PRESSURE

The lateral earth pressure on the shoring and bracing systems should be calculated based on the appropriate apparent earth pressure envelope as shown on Drawing No. 8.

If the ground surface is not horizontal, the uneven portion can be treated as an equivalent surcharge.

7.5. TRENCHLESS CONSTRUCTION METHODS

Selection of trenchless construction method will depend on the subsurface soil, bedrock and groundwater conditions within the tunnel bore horizon, length of tunnel, diameter of the tunnel and depth of earth cover above the crown, along with an assessment of boulder frequency and sizes. Based on information from the boreholes, the subsurface soils consist of a very wide range of differing soil textures including various fill materials, glacial till deposits of sandy silt to silty sand, cohesionless deposits of sand, silty sand, sandy silt and silt and bedrock of Guelph Formation. Considering the ground conditions that exist at the site, technical merits / drawbacks of various trenchless crossing alternatives are summarized Table 1 and in the following sections. These are generic discussions. Site-specific evaluation of appropriate trenchless technologies would be needed on a project-by-project basis.

7.5.1. Auger Boring

Auger boring (AB) also referred to as "jacking and boring" is a trenchless installation technique that forms a horizontal bore from a drive to a reception shaft by means of a rotating cutting head. Helical auger flights transport the spoil back to the drive shaft inside a steel casing that is being jacked in place simultaneously as the excavation progresses. AB is typically a 2-stage process: stage 1 comprises casing installation, while stage 2 is the product pipe installation. In most applications, the use of auger boring is limited to soil bores or bores in very soft rock. For the dolomitic bedrock in the Guelph area, conventional auger boring is <u>not</u> appropriate and modified drilling heads, fitted with rock disc cutters (i.e., self-boring unit or SBU) would be required, provided that the rock quality was high enough to permit this method. It is far more likely, however, that trenchless bores in the dolostone rock in Guelph would be advanced using microtunneling methods.

In most commonly track-type AB system, the track system, boring machine, casing, cutting head and augers are employed to install the pipe. The system critically depends on a properly constructed drive shaft which requires a stable foundation to support the tracks and adequate thrust block to transmit the horizontal jacking forces to the ground at the rear of the shaft.

AB has limited tracking and steering capability and it does not provide support to the excavation face and has no ability to deal with flowing unstable face conditions. With a special grade control steering head, the grade can be better maintained throughout the bore length. Alignment is most difficult to control and with the horizontal directional control the leading end of the steel casing can be installed with 150 mm accuracy. In general, an accuracy of 1% of the bore length is achievable. Adjustment and grade maintenance may be impossible in weak soils away from the launch pit.

Typical AB application involves underground pipe jacking of comparatively small diameters, from 0.1m to 1.4m with drive lengths of up to about 600m. AB system has the limited capability of handling boulders or cobbles which are smaller than 30% of the casing diameter. For bores greater than 900mm diameter, auger removal and personnel entry is needed to break up the boulders. Typical entry and exit bore pits are ~12m long and ~4m wide.

To reduce the risks associated with the AB method and improve the stability of the face, the following ground improvement method could be considered (the benefits achieved are highlighted in brackets):

1. Positive dewatering of the cohesionless deposits (this measure reduces pressure and produces slow ravelling ground conditions).
2. Grouting (change the ground behaviour from running/flowing into stable).
3. Continuous 24/7 tunneling.
4. It must be emphasized that the above measures will reduce but will not eliminate all the risks.

7.5.2. Modified Auger Boring

Modified Auger Boring (MAB) is conceptually similar to conventional Auger boring (AB), in that is uses the same technologies as traditional Auger Boring (AB) with the addition of a more specialized steering head system. A powerful auger bore machine combined with the steering system can allow for drive lengths of up to approximately 100 metres in certain soils. The specialized steering system allows for the monitoring and adjustment to lateral deviations and also allows for the monitoring and of adjustment of pitch. The line and grade can be monitored on almost a real-time basis, this allows for minor adjustments as needed. The steering head system is designed so the throat of this machine is reduced in size. In mixed faced conditions, MAB has the ability to control or adjust the position of the auger head within the casing which allows for adaptation to varying soil conditions. The combination of the auger bore machine and the steering head allows for the ability to control the location of the auger head within the casing and regulates the amount the auger turns. Pressure at the face can be maintained and reduce running ground. The steering system can be equipped with nozzles at the face that allow for injection of additives or stabilization agents to assist in controlling any ground loss as well as reducing frictional resistance between the pipe and the soil.

7.5.3. Horizontal Directional Drilling (HDD)

HDD involves the advancement of a small diameter pilot bore from a sending pit, along the proposed centroid of the carrier pipe, to a receiving pit. The bore is maintained in an open condition using a suitably viscous drilling fluid and is steered remotely by means of transmitter embedded in the bore head communicating to a manually tracked receiver at ground surface. Once the pilot bore breaks out at the far side of the bore path, the pilot bore is reamed in multiple passes using incrementally larger reamers to a diameter which is typically 1.5 times the carrier pipe outside dimension. The reaming head is then removed and the carrier pipe is pulled back into the reamed bore from the sending pit to the receiving pit. Specialized tracking systems are required in scenarios where personnel cannot manually operate a receiver above the bore path advancement (such as on the highway travelled lanes). EnVision does not recommend use of HDD for bore diameters exceeding 450mm based on experience with several failed bores or ceased carrier pipes during pullback in this size range and above. The risks of jamming the carrier pipe within the bore rises dramatically in cohesionless soils where cobbles may be present since these may be dislodged from the crown of the bore, falling onto the carrier pipe and wedging the pipe in the bore.

7.5.4. Pipe Ramming

Pipe ramming involves the direct advancement of a pipe or casing using a large pneumatic percussion hammer acting against a reaction block within the drive shaft. This method is best suited to watercourse crossings where ground surface displacement may be tolerated. Ground heave and poor bore path alignment control in plan and profile are common problems associated with this method. Loss of ground is prevented by maintaining a soil plug within the tip of the advancing casing. Displacement of the pipe volume and soil plug can lead to ground heave. Loss of the soil plug can sometimes occur in cohesionless soils below the groundwater table. Should this occur, significant ground loss and associated settlement can result. Significant friction can develop between the casing and the ground given the very small overcut and as such, generally only short drive lengths are feasible. Pipe ramming does not have suitable control mechanisms inherent in the method to be suitable for use beneath most public roadways. EnVision does not recommend this method for this application.

7.5.5. Microtunnelling

Microtunnelling (MT) is remotely controlled, guided pipeline installation technique that provides continuous support to the excavation face. Excavation is accomplished by a Micro Tunnel Boring Machine (MTBM). A slurry shield MTBM is generally more capable of handling wet, unstable ground conditions, similar to those existing at this site and dealing with cobbles and boulders. MTBM fitted with a rock head and rock cutters can also be used for trenchless bores through the bedrock, provided the rock in the bore zone has suitably high enough quality to have good standup time.

The amount of friction generated when the pipe is pushed into the ground is an important consideration. This friction contributes to the jacking resistance and is a major factor in determining the required capacity of the main thrust rams, and the requirements for intermediate jacking stations. The magnitude of the pipe friction depends on the pipe size and material, type of soil, its moisture content and grading, the details of the construction equipment and procedures employed. A pipe lubrication system may be introduced (usually based on bentonite and/or polymer slurry) to reduce jacking forces (the most common reduction factor is around 25%).

Due to relatively large jacking forces the design and construction of the jacking shaft are critical. The shaft floor and thrust reaction structure must be designed to withstand the weight of heavy pipe segments. Primary jacking pit is usually 4m to 5m long and 3m wide.

There is no theoretical limit to the length of individual pipe jacks although practical engineering considerations and economics may impose restrictions. Drives of several hundred metres either in a straight line or to a radius or a series of radii are achievable. The most common drive lengths range from 150 m to 300 m for slurry MTBM provided that intermediate jacking stations are launched every 75m.

The method is quite accurate and a tolerance of 25 mm on line and grade is attainable.

A large laydown area is needed for the subsoils separation support plant. The liner pipes must be designed for the earth, groundwater pressure and jacking forces.

In assessing construction methodology, emphasis is made to the proximity of existing buried utilities, poles and overhead wires. Consideration should be given during design and construction for the induced displacement and movements of these structures and any buried utilities.

MT is often the lowest risk (but highest cost) trenchless alternative and potentially the only technically feasible alternative in cohesionless soils below the water table for larger diameter utilities and on-grade utilities.

7.6. GEOTECHNICAL QUALITY OF EXCAVATED SOIL AND PAVEMENT RESTORATION

From a geotechnical perspective, the existing pavement structure fill, inorganic cohesionless deposits of gravelly sand, sand, sandy gravel, sand and gravel, silty sand, sandy silt and glacial till deposits of silty sand to sandy silt and clayey silt may be reused on this project as backfill within service trenches, or as subgrade material for pavements, provided that the material has a suitable water content to be compactable and is frost-compatible with the material exposed on the trench walls. For additional information related to reuse of excavation spoil at this site, the reader should refer to related environmental Excess Soil Management reports for this project. Existing fill soils containing organics, such as those encountered in BH23-3, BH22-6 and BH22-12, wet sandy silt in BH22-10, firm silty clay fill in BH22-23 are not suitable for reuse as compacted fill on the project.

As a general requirement, all backfill material should be placed in 200 to 300mm thick loose lifts and compacted to at least 98% of the SPMDD, at a placement moisture content within ±2% of the optimum. On roadway and shoulders, consideration must be given to backfilling trenches with a well-graded, compacted granular soil such as Granular 'B' material. The use of such material, if thoroughly compacted, would reduce the post construction settlements to a negligible amount and may also expedite the compaction process.

The existing sandy silt and silty sand glacial till and cohesionless soils (native material) can be considered for reuse as compacted trench backfill if approved by the City of Guelph. The silty clay and clayey silt glacial till soil will likely require some degree of air drying / moisture conditioning in order to be within 2 percent of its optimum water content at time of placement so this might involve double handling and staging of the work. As such, its reuse may be impractical.

The existing road pavement structure should be reinstated as part of any utility work. New granulars placed at the top of utility trenches must match into the underside of existing granulars to ensure unimpeded cross drainage towards the pavement edges/subdrains.

8. PAVEMENT RECOMMENDATIONS

The existing pavement structure at borehole locations and average pavement structure thickness for each road based on the boreholes is listed in Table 8-1.

Table 8-1: Summary of Existing Pavement Structure Thicknesses at Borehole Locations and Average for Each Road

ROAD NAME	BOREHOLE NO.	PAVEMENT STRUCTURE THICKNESS AT EACH BOREHOLE LOCATION		AVERAGE PAVEMENT STRUCTURE THICKNESS	
		ASPHALTIC CONCRETE (m)	GRANULAR BASE/SUBBASE (m)	ASPHALTIC CONCRETE (m)	GRANULAR BASE/SUBBASE (m)
DUBLIN ST.	BH22-1	75	100	75	100
	BH22-2	75	100		
NORFOLK ST	BH22-4	150	150	146	311
	BH22-5	150	410		
	BH22-6	150	400		
	BH22-7	75	300		
	BH22-8	150	460		
	BH22-9	200	150		
CARDIGAN STREET	BH22-10	75	100	75	100
	BH22-11	75	100		
ERAMOSA ROAD	BHJ22-12	150	410	150	410
WYNDHAM ST N	BH22-13	130	100	93	100
	BH22-14	75	100		
	BH22-15	75	100		

ROAD NAME	BOREHOLE NO.	PAVEMENT STRUCTURE THICKNESS AT EACH BOREHOLE LOCATION		AVERAGE PAVEMENT STRUCTURE THICKNESS	
		ASPHALTIC CONCRETE (m)	GRANULAR BASE/SUBBASE (m)	ASPHALTIC CONCRETE (m)	GRANULAR BASE/SUBBASE (m)
MACDONELL ST	BH22-16	150	460	143	460
	BH22-17	150	460		
	BH22-30	130	460		
WOOLWICH ST	BH22-18	75	530	116	386
	BH22-19	130	460		
	BH22-20	150	380		
	BH22-21	150	460		
	BH22-22	75	100		
SUFFOLK STREET EAST	BH22-24	75	100	75	100
YARMOUTH STREET	BH22-25	125	100	125	100
QUEBEC ST	BH22-26	75	100	75	100
	BH22-27	75	100		
CORK ST E	BH22-28	75	100	100	113
	BH22-29	125	125		
DOUGLAS STREET	BH22-31	125	460	125	460
WELLINGTON ST. E	BH23-1	150	280	150	280
	BH23-2	150	260		
	BH23-3	150	300		

ROAD NAME	BOREHOLE NO.	PAVEMENT STRUCTURE THICKNESS AT EACH BOREHOLE LOCATION		AVERAGE PAVEMENT STRUCTURE THICKNESS	
		ASPHALTIC CONCRETE (m)	GRANULAR BASE/SUBBASE (m)	ASPHALTIC CONCRETE (m)	GRANULAR BASE/SUBBASE (m)
WYNDHAM ST. S	BH23-4	150	200	150	225
	BH23-5	150	250		

8.1. GEOTECHNICAL LABORATORY TEST RESULTS

Selected samples of granular base/subbase and subgrade material were subjected to grain size analysis for the purpose of assessing existing structural function and compliance against OPSS 1010 gradation specifications.

8.1.1. GRAIN SIZE ANALYSIS OF BASE/SUBBASE MATERIAL

Grain size analysis was conducted on thirteen (13) selected granular samples. The grain size distribution curves for the samples are summarized in

Table 8-2 and are presented in Figure 1 in Appendix B.

Table 8-2: Summary of Grain Size Distribution on Pavement Granular Base samples

BOREHOLE NO.	SAMPLE NO.	SAMPLE DEPTH (M)	GRAIN SIZE DISTRIBUTION			
			GRAVEL (%)	SAND (%)	SILT (%)	CLAY (%)
BH22-3	SS1 (Granular)	0.1	24	33	40	
BH22-8	SS1 (Granular)	0.2	13	72	15	
BH22-10	SS1 (Granular)	0.1	17	61	22	
BH22-12	SS1 (Granular)	0.8	33	57	10	
BH22-16	SS1 (Granular)	0.1	30	56	14	
BH22-19	SS1 (Granular)	0.1	36	51	13	
BH22-20	SS1(Granular)	0.1	33	53	14	
BH22-22	SS1 (Granular)	0.1	33	52	15	
BH22-23	SS1 (Granular)	0.1	16	62	22	
BH22-26	SS1 (Granular)	0.1	32	53	15	
BH22-28	SS1 (Granular)	0.1	35	53	12	
BH23-1	SS1 (Granular)	0.15	52	36	12	
BH23-3	SS1 (Granular)	0.15	41	47	12	

None of the tested samples of the existing granular base and subbase materials meet the OPSS 1010 granular A and B Type I gradation specifications owing to excessive fines content.

As such, these granular layers would be expected to have impeded drainage function and reduced structural number relative to new, OPSS 1010-compliant granulars.

8.1.2. GRAIN SIZE ANALYSIS OF SUBGRADE MATERIAL

Sieve and hydrometer analyses were conducted on five (5) selected fill (subgrade) samples for the purposes of assessing frost susceptibility. The grain size distribution curves for the samples are summarized in Table 8-3 and are presented in Figure 2 in Appendix B.

Table 8-3: Summary of Grain Size Distribution on Subgrade samples

BOREHOLE NO.	SAMPLE NO.	SAMPLE DEPTH (M)	GRAIN SIZE DISTRIBUTION			
			GRAVEL (%)	SAND (%)	SILT (%)	CLAY (%)
BH22-5	SS2	0.6	16	48	30	6
BH22-6	SS3	1.6	1	32	55	12
BH22-7	SS2	0.8	13	53	28	6
BH22-11	SS2	0.6	5	49	37	9
BH22-15	SS3	1.5	3	72	19	6

The tested samples indicate the pavement subgrade soils are poorly drained and are moderately to highly frost susceptible.

8.2. PRELIMINARY RECOMMENDATIONS ON PAVEMENT RECONSTRUCTION

Preliminary recommended pavement structures are provided in Table 8-4 based upon an estimate of the subgrade soil properties determined from visual examination and textural classification of the soil samples. The values may need to be adjusted based on the City standards. These recommended pavement structures should be considered for preliminary design purposes only since EnVision has not been provided with traffic counts/percentage of truck traffic/expected traffic growth and other requisite design inputs. A functional design ten years has been used to establish the pavement recommendations. This represents the number of years to the first rehabilitation, assuming regular maintenance is carried out.

Table 8-4: Preliminary Recommended Pavement Structure Thicknesses

ROAD NAME	PAVEMENT LAYER	COMPACTION REQUIREMENTS	PAVEMENT STRUCTURE
NORFOLK STREET ERAMOSA ROAD WYNDHAM STREET N MACDONELL STREET WOOLWICH STREET QUEBEC STREET WELLINGTON ST. E WYNDHAM ST. S (15m and 16m wide roads)	Asphaltic Concrete	96% Maximum Relative Density (MRD)	45 mm HL 3 or SP 12.5 90 mm HL 8 or SP 19.0
	OPSS Granular A Base (or 20mm Crusher Run Limestone	100% SPMDD*	175 mm
	OPSS Granular B (or 50mm Crusher Run Limestone)	100% SPMDD	450 mm
DUBLIN ST CARDIGAN STREET SUFFOLK STREET E YARMOUTH STREET CORK STREET E DOUGLAS STREET (8.4m, 8.8m and 10m wide roads)	Asphaltic Concrete	96% Maximum Relative Density (MRD)	40 mm HL 3 or SP 12.5 50 mm HL 8 or SP 19.0
	OPSS Granular A Base (or 20mm Crusher Run Limestone	100% SPMDD*	175 mm
	OPSS Granular B (or 50mm Crusher Run Limestone)	100% SPMDD	350 mm

* Denotes Standard Proctor Maximum Dry Density, ASTM-D698

** Base of granular sub-base must be adjusted to match in with adjacent sub-base in order to promote cross drainage across the roadway.

8.2.1. Subgrade Preparation

The long-term performance of the pavement structure is highly dependent upon the subgrade support conditions. Stringent construction control procedures should be maintained to ensure uniform subgrade moisture and density conditions are achieved.

The subgrade is expected to consist of native soils or clean cohesionless fill materials. The fill materials encountered on the site may be utilized for subgrade preparation provided they are environmentally acceptable and do not contain organics, deleterious materials and fine soils, as well as their in-situ moisture content is within 2 percent of the optimum moisture content such as. The pavement subgrade should be proof-rolled; and any loose, soft, wet or unstable areas should be sub-excavated, and backfilled with clean earth fill placed in 200 mm thick lifts and compacted to a minimum of 98% SPMDD and 100% SPMDD for top 1.5m below the sub-grade. Local sub-excavation may be required in areas where incompetent (loose/firm) subgrade conditions and significant organic inclusions (if any) are encountered. The entire pavement subgrade should be compacted to a minimum of 98 percent SPMDD with minimum cross-fall of 3 percent prior to the granular sub-base placement.

Additional comments on the subgrade preparation and construction of roadways are as follows:

- As part of the subgrade preparation, proposed roadways should be stripped of topsoil and other obvious objectionable material in areas having no existing pavement. Fill required to raise the grades to design elevations should conform to backfill requirements outlined in previous sections of this report. The subgrade should be properly shaped, crowned then proof-rolled in the full-time presence of a representative of this office. Soft or spongy subgrade areas should be sub-excavated and properly replaced with suitable approved backfill compacted to 98% SPMDD.
- The most severe loading conditions may occur during construction. Consequently, special provisions such as restricted access lanes, half-loads during paving, etc., may be required, especially if construction is carried out during unfavourable weather.
- Once the subgrade has been inspected and approved, the granular base and sub-base course materials should be placed in layers not exceeding 200 mm loose thickness and should be compacted to at least 100% of their respective SPMDD. The grading of the material should conform to current requirement of City of Guelph.
- The placing, spreading and rolling of the asphalt should be in accordance with OPS specifications or, as required by the local authorities. Frequent field compaction tests should be carried out on both the asphalt and granular base and sub-base materials to ensure that the required degree of compaction is achieved.

8.2.2. Drainage Requirements

Control of surface water is an important factor in achieving a good pavement service life. Therefore, we recommend that provisions be made to drain the new pavement subgrade and its granular layers. It is understood that the proposed improvements are anticipated to consist of typical urban section (concrete curb/gutter and catchbasins). To provide positive drainage across the pavement platform, the surface of pavement should be sloped at a grade of 2 percent and the pavement subgrade should be sloped at a grade of 3 percent towards the subdrains. Subdrains should be designed and constructed in accordance with OPSS or local municipality specifications, and the subdrain pipe should be connected to a positive outlet.

8.2.3. Reuse of Existing Pavement Materials

It should be noted that gradation analyses of the tested samples of the existing granular base and subbase materials do not meet the OPSS 1010 granular A and B Type I gradation specifications as a result of excessive fines content. Therefore, the existing excavated granular materials cannot be reused as subbase/base materials, however, some of this material, if carefully stripped without fouling, may be reused as subgrade material to replace soft, wet or otherwise disturbed areas identified during proofrolling.

9. GENERAL COMMENTS AND LIMITATIONS OF REPORT

EnVision Consultants Ltd. should be retained for a general review of the final design and specifications to verify that this report has been properly interpreted and implemented. If not accorded the privilege of making this review, EnVision will assume no responsibility for interpretation of the recommendations in the report.

The comments given in this report are intended only for the preliminary guidance of design engineers. The number of boreholes required to determine the localized underground conditions between boreholes affecting construction costs, techniques, sequencing, equipment, scheduling, etc., would be much greater than has been carried out for design purposes. Contractors bidding on or undertaking the works should, in this light, decide on their own investigations, as well as their own interpretations of the factual borehole and test pit results, so that they may draw their own conclusions as to how the subsurface conditions may affect them.

This report is intended solely for the Client named. The material in it reflects our best judgment in light of the information available to EnVision at the time of preparation. Unless otherwise agreed in writing by EnVision Consultants Ltd., it shall not be used to express or imply warranty as to the fitness of the property for a particular purpose. No portion of this report may be used as a separate entity, it is written to be read in its entirety.

The conclusions and recommendations given in this report are based on information determined at the test hole locations. The information contained herein in no way reflects on the environment aspects of the project, unless otherwise stated. Subsurface and groundwater conditions between and beyond the test holes may differ from those encountered at the test hole locations, and conditions may become apparent during construction, which could not be detected or anticipated at the time of the site investigation. The benchmark and elevations used in this report are primarily to establish relative elevation differences between the test hole locations and should not be used for other purposes, such as grading, excavating, planning, development, etc.

The design recommendations given in this report are applicable only to the project described in the text and then only if constructed substantially in accordance with the details stated in this report.

Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of such third parties. EnVision Consultants Ltd. accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.

We accept no responsibility for any decisions made or actions taken as a result of this report unless we are specifically advised of and participate in such action, in which case our responsibility will be as agreed to at that time.

We trust that the information contained in this report is satisfactory. Should you have any questions, please do not hesitate to contact this office.

9.1. SIGNATURES

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9.2. QUALIFIER

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The content and opinions contained in the report are based on the observations and/or information available to EnVision at the time of preparation, using investigation techniques and engineering analysis methods consistent with those ordinarily exercised by EnVision and other engineering/scientific practitioners working under similar conditions, and subject to the same time, financial and physical constraints applicable to this project.

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10. REFERENCES

1. City of Guelph Linear Infrastructure Standards, March 3, 2021.

DRAWINGS

Drawing No. 1	Borehole Location Plan
Drawing Nos. 2 to 8	Borehole Location Plan and Stratigraphic Profiles
Drawing No. 9	Risk Zones
Drawing No. 10	Earth Pressure Distribution on Braced Excavations

APPENDIX A:

Notes on Sample Descriptions (Drawing 1A); Terms used in the Record of Borehole Logs (Drawing 1B); Terms used in the Record of Rock Core Logs (Drawing 1C); Record of Borehole Sheets (BH22-1 to BH22-31& BH23-1 to BH23-5)

APPENDIX B:

Grain Size Analysis Test Results (Figures 1 to 7)

APPENDIX C:

Uniaxial Compressive Strength Test Results on Rock Samples; Cerchar Abrasiveness Test Results on Rock Samples; Point Load Index Test Results ; Rock Core Photos

APPENDIX D: Soil Corrosivity Laboratory Certificates of Analyses