APPENDIX G

GEOTECHNICAL/HYDROGEOLOGICAL REPORT

ENVISION
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PRELIMINARY GEOTECHNICAL INVESTIGATION – CRAIGLEITH WASTEWATER TREATMENT PLANT AND ASSOCIATED SANITARY SEWER

Craigleith, Ontario

EnVision Project #: 22-0165
Prepared for: WT Infrastructure Solutions Inc.
Date: February 27, 2023
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February 27, 2023

WT Infrastructure Solutions Inc.
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Attention: Jamie Witherspoon, P.Eng, LEED AP, ENV SP

SUBJECT: PRELIMINARY GEOTECHNICAL INVESTIGATION – CRAIGLEITH WASTEWATER TREATMENT PLANT AND ASSOCIATED SANITARY SEWER, CRAIGLEITH, ONTARIO

EnVision Consultants Ltd. is pleased to provide the enclosed final report on Geotechnical Investigation for the proposed pumping station and associated sanitary sewer installation along Long Point Road in Craigleigh, Town of Bule Mountains, ON.

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

Yours sincerely,

Tim Yu

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QUALITY MANAGEMENT

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1. EXECUTIVE SUMMARY

EnVision Consultants Ltd. (EnVision) was retained by WT Infrastructure Solutions Inc. (the 'Client') to conduct geotechnical investigation work in support of the preliminary design of the proposed wastewater pumping station and associated sanitary sewer installation along Long Point Road, Craigleith, Ontario (the 'Site'). EnVision understands that the proposed pumping station facilities are to consist of a lift station and septage receiving station. The base of the pumping station wet well structure is proposed to be 8m to 12m below the existing ground surface. The proposed sanitary sewer is proposed to be 525mm in diameter and the invert of the pipe will be set at depths varying from 3m to 4.5m (Alternate 1) or from 3m to 8.0m (Alternate 2) below existing ground surface, based on the plan and profile drawings provided by the Client.

EnVision conducted preliminary geotechnical investigation work which included the drilling of a total of five (5) boreholes to depths ranging from 4.6 m to 7.6 m below the existing ground surface. Three (3) monitoring wells were installed within selected boreholes for groundwater level monitoring purposes.

The subsurface conditions revealed in the boreholes generally consisted of pavement structure or topsoil at the ground surface, overlying fill materials. The native soils consisted of dense to very dense silt deposits and very dense sandy silt to silty sand till deposits. Cobbles and boulders are expected in the overburden. Boreholes were terminated on auger and sampler spoon refusal at depths ranging from 6.4m to 7.6m below the existing ground surface, corresponding to Elev. 171.0m to 171.6m, at the borehole locations within the Craigleith WWTP and at about 4.6m to 4.9m below ground surface, corresponding to Elev. 175.3m to 176.4m, along Long Point Road. Auger refusal could be on cobble/boulders or on the bedrock surface. To confirm the bedrock surface depth and elevation, bedrock coring will be required.

EnVision recommends supplementary site investigation work involving additional boreholes employing rock coring methods and detailed rock logging in order to confirm the bedrock surface elevation and to characterize the rock strength, lithology, degree of weathering, hard layer thickness and frequency, fracture frequency, RQD, etc.

Groundwater levels were found to be at depths of about 0.2m to 0.9m below existing ground surface on December 6, 2022.

EnVision recommends that test pits be dug at multiple locations along the sanitary sewer alignment as well as close to the proposed pumping station in order to further assess groundwater seepage, stability of the trenching walls, along with the frequency and size of cobbles and boulders.

For the proposed sanitary sewer installation, the dense to very dense silt and very dense sandy silt to silty sand till deposits encountered in the boreholes will provide adequate support for the sanitary sewer and will allow the use of normal Class B type bedding. Given the high groundwater levels measured in monitoring wells and cohesionless soils below the groundwater table, positive dewatering such as eductors or wellpoints will be required.

For the proposed waste water pumping station facility, a raft slab system with waterproofing is recommended. A continuous cut-off caisson wall (secant pile wall) will be required to be installed along the perimeter of the station walls to retain the overburden, temporarily control the groundwater seepage during the construction stage, and to permanently reduce groundwater flow.

PART A – GEOTECHINCAL FACTUAL DATA

2. INTRODUCTION

EnVision Consultants Ltd. (EnVision) was retained by WT Infrastructure Solutions Inc. (the 'Client') to conduct geotechnical investigation work in support of the proposed wastewater treatment plant (WWTP) and associated sanitary sewer installation along Long Point Road in the community of Craigleigh, Town of Blue Mountains, Ontario (the 'Site'). Based on the layout plan Drawing No. SP.1, dated August 10, 2022, provided by the Client, the proposed waste water pumping station structure is to be located west of Long Point Road and south of the WWTP access road. A 525mm dia. sanitary sewer has been proposed with invert depths ranging from about 3m to 4.5 m below ground surface (mbgs) (Alternate 1) or 3m to 8 mbgs (Alternate 2), based on plan and profile drawings, Drawing No. PP1.1 to PP1.4 (Alternate 1) and PP2.1 to PP2.4 (Alternate 2) respectively.

The purpose of this preliminary geotechnical investigation was to determine the subsurface soil and groundwater conditions at the borehole locations and from the findings in the boreholes, to provide preliminary geotechnical recommendations for the proposed waste water pumping station and associated sanitary sewer installation. A Borehole location plan is provided in Drawing No. 1.

This report is presented in two parts; Part A of the report includes factual data from the geotechnical investigation at the borehole locations and Part B includes geotechnical interpretation and recommendations for the proposed WWTP structures and sewer.

Preliminary hydrogeological 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, EnVision 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.

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 the most part follows ASTM or CSA Standards or modifications of these standards that have become standard practice.

This report has been prepared for the WT Infrastructure Solutions Inc. and the Town of Blue Mountains. Third party use of this report without EnVision's 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. FIELD INVESTIGATION AND TESTING

3.1. FIELDWORK

The preliminary field investigation consisted of drilling a total of five (5) boreholes (designated as BH22-1 through BH22-5). Boreholes BH22-1 to BH22-3 were drilled in the vicinity of the proposed pumping station and BH22-4 and BH22-5 were advanced along Long Point Road. The boreholes were drilled to depths ranging from 4.6m to 7.6m below the existing ground surface. The locations of boreholes are shown on the Borehole Location Plan (Drawing No. 1). Three (3) monitoring wells of 50mm diameter were installed in selected boreholes.

The as-drilled borehole locations were surveyed by EnVision personnel using differential GPS. The borehole coordinates and ground geodetic elevations at the borehole locations are summarized in Table 3-1 as well as presented in the Record of Borehole sheets in Appendix A. A summary of borehole information is provided in Table 3-1.

Table 3-1: Summary of Borehole Information

BOREHOLE ID	GROUND SURFACE ELEVATION (m)	BOREHOLE COORDINATES UTM NAD83, ZONE 17		DEPTH OF BOREHOLE (m)	NOTE
		NORTHING (m)	EASTING (m)		
BH22-1	178.56	4930610.1	556148.0	7.0	50mm MW
BH22-2	178.62	4930594.9	556183.0	7.6	-
BH22-3	177.83	4930616.8	556222.9	6.5	50mm MW
BH22-4	179.95	4930468.1	556269.7	4.6	-
BH22-5	181.3	4930317.7	556296.6	4.9	50mm MW

The field investigation work of borehole drilling was carried out on October 14 and 15, 2022, by TEC Geological Drilling Inc. with technical supervision provided by EnVision personnel. The boreholes were drilled using a Diedrich D-50 track-mounted drill rig and were advanced through the overburden soils using nominal 152 mm outer diameter, solid stem augers. 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 Appendices A).

Technical supervision of the field work was carried out by EnVision's engineering staff who arranged for the clearance of underground public utility locate services, supervised the sampling and in situ testing operations and logged the boreholes. The soil samples were identified in the field, placed in labelled containers and transported to EnVision's laboratory for further examination and testing. Rock coring (or coring of boulders above the bedrock surface) was specifically excluded from EnVision's scope of services at the request of the client.

3.2. MONITORING WELL INSTALLATION

Monitoring wells were installed in selected boreholes as summarized in *Table 3-1.* The monitoring wells were constructed using 50mm diameter environmental-grade, flush-threaded polyvinyl chloride (PVC) pipe including a screen section with a factory machined slot (10) width of 0.25mm and were completed with a PVC riser pipe. All of the pipe material and screen sections were wrapped in plastic which was removed just prior to installation to minimize the potential for contamination. The base of the monitoring well was covered with a PVC cap to prevent the influx of sediment. Clean silica supplied in bags from a commercial supplier of well sand was placed in the annular space between the pipe and the walls of the borehole. The monitoring wells were constructed in accordance with Ontario Regulation 903 (amended by O. Reg. 372/07) by extending an impermeable bentonite grout layer from approximately 0.6m above the top of the screened interval to the ground surface. The monitoring wells were completed by installing a protective well cover finished with a flush-mount casing. Well construction details are provided on the respective borehole logs presented in Appendix A.

3.3. WELL DECOMMISIONING

The monitoring wells described in Section 3.2 have not been decommissioned. The monitoring wells must be decommissioned in accordance with O. Reg. 903 (as amended) by an MECP licensed water well contractor prior to commencement of construction work.

3.4. 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 a total of six (6) selected samples and Atterberg Limits testing was conducted on one (1) selected soil sample. The gradation curves and Atterberg Limits test results are presented in Appendix B and on the respective borehole log sheets in Appendix A.

4. SUBSURFACE CONDITIONS

The borehole locations are shown in Drawing No. 1. A generalized subsurface profile at the borehole locations is presented in Drawing Nos. 2 and 3. 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 stratigraphic boundaries shown on the borehole records are inferred from observations of drilling progress and non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. The subsurface conditions will vary beyond the borehole locations.

4.1. CRAIGLEITH WWTP

Three (3) boreholes (designated as BH22-1 to BH22-3) were drilled within the property of the Craigleith WWTP for the proposed pumping station. The subsurface conditions in the boreholes consisted of pavement structure or topsoil at the ground surface, overlying fill materials or native silt deposits, which in turn are underlain by very dense, stony, glacial till deposits of sandy silt to silty sand. In Borehole BH22-1 below the silty sand till, a deposit of silt was encountered which extended to the termination depth of the borehole.

4.1.1. Topsoil / Pavement

A layer of topsoil, varying in thickness from approximately 150mm to 170mm was encountered at the ground surface at the location of BH22-2 and BH22-3.

Borehole BH22-1 was drilled along the access road where a 70mm thick layer of asphalt overlying 355mm granular base/subbase was encountered.

4.1.2. Fill Material

Below topsoil in BH22-2 and BH22-3, cohesive fill material consisting of silty clay to clayey silt was encountered which extended to depths of 0.8m to 1.5m below the existing ground surface. Standard Penetration Test (SPT) ‘N’ values measured within the cohesive fill ranged from 8 blows to 19 blows per 0.3 m of penetration, suggesting a stiff to very stiff consistency.

The natural moisture contents measured in the tested samples of fill ranged from 23 to 25%.

4.1.3. Upper Cohesionless Deposits of Silt

Below the pavement structure in BH22-1 and below fill material in BH22-2, an upper cohesionless deposit of silt was encountered which extended to a depth of 1.5m below the existing ground surface.

SPT ‘N’-values measured within silt ranged from 47 blows to over 50 blows per 0.3 m of penetration, indicating a dense to very dense state of compactness. The water content measured in samples of upper silt ranged from 10% to 12%.

4.1.4. *Glacial Till Deposits of Sandy Silt to Silty Sand*

Glacial till deposits of sandy silt to silty sand were encountered in the boreholes (BH22-1 to BH22-3) at a depth of 1.5m which extended to 3.8m in BH22-1 and to the termination depths of 6.5 to 7.6m in BH22-2 and BH22-3.

Cobbles and boulders are expected within these glacial till deposits.

SPT 'N' values measured within sandy silt till/silty sand till were found to be more than 50 blows per 0.3 m of penetration, indicating a very dense state of compactness. Some of the SPT tests revealed 'N' values exceeding 50 blows for only 125mm to 75mm of penetration, indicating *extremely dense or dense and stony conditions*. The moisture content measured in the tested samples of sandy silt till to silty sand till ranged from 5 % to 11%.

Grain size analyses were conducted on three (3) selected sandy silt till/silty sand till samples. The tested samples contained 9% to 25% gravel, 35% to 47% sand, 23% to 47% silt and 5% to 11% clay sized particles. The grain size distribution test results are summarized in *Table 4-1* and the gradation curves are presented in **Appendix B.** *Note that gravel sizes and larger particles exceeding 25mm in size are not captured in this testing method but are expected to be present in the soil material. Test pits and bulk samples would be needed in order to characterize the percentages of gravel, cobble and boulder sizes.*

Table 4-1: Summary of Grain Size Distribution Tests on Sandy Silt Till/Silty Sand Till Samples (Craigleith WWTP)

BOREHOLE NO.	SAMPLE NO.	SAMPLE DEPTH (m)	GRAIN SIZE DISTRIBUTION			
			GRAVEL (%)	SAND (%)	SILT (%)	CLAY (%)
BH22-1	SS3	1.5	9	42	38	11
BH22-2	SS5	3.0	10	35	47	8
BH22-3	SS3	1.5	25	47	23	5

Atterberg Limits testing was also carried out on the above-noted sample SS3 from Borehole BH22-1. The tested sample was found to be non-plastic.

4.1.5. *Lower Cohesionless Deposit of Silt*

In BH22-1 below the glacial till deposits of sandy silt to silty sand, a cohesionless deposit of silt was encountered which extended beyond the termination depth of 7m in the borehole.

The SPT 'N' values measured within silt were found to be more than 50 blows per 0.3 m of penetration, indicating a very dense state of compactness. Some of the SPT tests revealed 'N' values exceeding 50 blows for only 125mm to 75mm of penetration, indicating *extremely dense or potentially dense and stony conditions*.

The moisture content measured in the tested silt samples ranged from 14 % to 16%.

Grain size analysis was conducted on one (1) selected sample. The tested sample contained 0% gravel, 16% sand, 76% silt and 8% clay sized particles. The grain size distribution test result is summarized in *Table 4-2* and the gradation curve is presented in Appendix B.

Table 4-2: Summary of Grain Size Distribution Test on Silt Sample

BOREHOLE NO.	SAMPLE NO.	SAMPLE DEPTH (m)	GRAIN SIZE DISTRIBUTION			
			GRAVEL (%)	SAND (%)	SILT (%)	CLAY (%)
BH22-1	SS6	3.8	0	16	76	8

4.2. LONG POINT ROAD

Two (2) boreholes (BH22-4 and BH22-5) were drilled along Long Point Road from the existing road surface. The subsurface conditions in these borings consisted of pavement structure overlying cohesionless fill comprised of sand underlain by stony glacial till deposits of sandy silt to silty sand texture. A description of the subsurface conditions encountered in the boreholes is provided below.

4.2.1. Pavement Structure

Boreholes BH22-4 and BH22-5 were drilled from the paved surface where a 100mm thick layer of asphalt overlying 400mm to 600mm thick granular base/subbase was encountered.

4.2.2. Fill Material

Underlying the pavement structure, cohesionless fill consisting of sand was encountered in the boreholes (BH22-4 and BH22-5) which extended to depths ranging from 1.5m to 2.3m below the existing ground surface. Fill material was found to be in a compact state based on measured SPT ‘N’ values ranging from 11 to 14 blows per 0.3 m of penetration.

The natural moisture content of the tested fill samples ranged from 17 to 25%.

4.2.3. Glacial Till Deposits of Sandy Silt to Silty Sand

Glacial till deposits of sandy silt to silty sand were encountered below fill material in the boreholes BH22-4 and BH22-5 at depths ranging from 1.5m to 2.3m which extended to the termination depths of 4.6m to 4.9m of these boreholes. These boreholes were terminated due to auger refusal.

SPT ‘N’ values measured within these glacial till deposits were found to be more than 50 blows per 0.3 m of penetration, indicating a very dense state of compactness.

Cobbles and boulders are expected within the glacial till deposits. Without diamond coring of the obstruction, it cannot be conclusively stated whether the refusal to augering occurred on the bedrock surface or on boulders/cobbles.

The moisture content measured in the tested samples of the sandy silt to silty sand till ranged from 5 % to 17%.

Grain size analysis was conducted on two (2) selected sandy silt to silty sand till samples. The tested samples contained 3% to 21% gravel, 29% to 40% sand, 34% to 58% silt and 5% to 10% clay sized particles. The grain size distribution test results are summarized in *Table 4-3* and the gradation curves are presented in Appendix B. *Note that gravel sizes and larger particles exceeding 25mm in size are not captured in this testing method but are expected to be present in the soil material. Test pits and bulk samples would be needed in order to characterize the percentages of gravel, cobble and boulder sizes.*

Table 4-3: Summary of Grain Size Distribution Tests on Sandy Silt Till/Silty Sand Till Samples (Long Point Road)

BOREHOLE NO.	SAMPLE NO.	AVERAGE SAMPLE DEPTH (m)	GRAIN SIZE DISTRIBUTION			
			GRAVEL (%)	SAND (%)	SILT (%)	CLAY (%)
BH22-4	SS3	1.5	21	40	34	5
BH22-5	SS5	3.0	3	29	58	10

4.3. GROUNDWATER CONDITIONS

The groundwater levels measured in the monitoring wells are summarized in *Table 4-4* and are also shown in the borehole log sheets attached in Appendix A. The groundwater levels measured on December 6, 2022 in the monitoring wells installed in Boreholes BH22-1, BH22-3 and BH22-5 were found to be at 0.2m to 0.9m below the existing ground surface corresponding to Elev. 177.6m to 180.4m, as listed in *Table 4-4*.

Table 4-4: Summary of Groundwater Levels Observed in the Monitoring Wells

BOREHOLE NO.	GROUND SURFACE ELEVATION (m)	WELL SCREEN DEPTH (m) [SCREENED STRATIGRAPHY]	DATE OF OBSERVATION	GROUNDWATER LEVEL DEPTH (m)	GROUNDWATER LEVEL ELEVATION (m)
BH22-1	178.6	4.0 – 7.0 [Silt, some sand]	Dec. 6, 2022	0.4	178.2
BH22-3	177.8	3.4 – 6.5 [Silty Sand Till/ Sandy Silt Till]	Dec. 6, 2022	0.2	177.6
BH22-5*	181.3	3.4 – 4.9 [Sandy Silt Till]	Dec. 6, 2022	0.9	180.4

*Borehole caved to 0.9m below the existing ground surface. Monitoring well was installed in another borehole 1m away from the original borehole location to install the monitoring well.

It should be noted that the groundwater levels are subject to seasonal fluctuations and in response to precipitation events and should be expected to be higher during wet periods of the year. Perched water should also be expected in fill material.

PART B –GEOTECHNICAL INTERPRETATION AND RECOMMENDATIONS

5. DISCUSSION AND RECOMMENDATIONS

In this section, the soil and groundwater conditions are interpreted as relevant to the preliminary design of the proposed works. The purpose of this section is to provide preliminary geotechnical recommendations based on the factual subsurface conditions (Part A of this report), in order to support the civil designers of the proposed pumping station and sanitary sewer installation at the Site.

Recommendations provided in this report must not be construed as representing specifications or directives to prospective contractors nor 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.

5.1. GENERAL DISCUSSION

Based on the plan and profile drawings, Drawing Nos. PP1.1 to PP1.4 and PP2.1 to PP2.4 dated August 10, 2022, provided to EnVision, there are two alternatives proposed by the Client for the installation of the proposed 525mm dia. sanitary sewer:

Alternative 1: The invert level of the proposed 525mm dia. pipe will be at about 3m to 4.5m (Elev. 179.31 to 174.15m) below the existing ground surface;

Alternative 2: The invert level of the proposed 525mm dia. pipe will be at about 3m to 8m (Elev.179.31 to 170.05m) below the existing ground surface:

We further understand based on a client email communication dated December 14, 2022, that the base of the proposed pumping station is proposed to be set at about 8m to 12m below the existing ground surface.

It should be noted that the recommendations provided in this report are to be considered as being preliminary in nature due to the shallow refusal depth of the boreholes. Subsurface conditions are known only to depths of 4.6m to 7.6m below the existing ground surface. Additional geotechnical investigation with bedrock coring and test pitting will be required for the detailed design of the pumping station and the sewer.

5.2. OVERVIEW OF SUBSURFACE CONDITIONS

The subsurface conditions revealed in the boreholes generally consisted of pavement structure or topsoil underlain by fill material which extend to 0.5m to 2.3m below existing ground surface at the boring locations. The native soil consisted of extremely dense, stony, silty sand till to sandy silt till and very dense cohesionless deposits of silt. Auger grinding was noted during drilling program and boreholes were terminated on auger refusal on obstructions such as cobbles or boulders or bedrock at depths ranging from 4.6m to 7.6m below the existing ground surface.

Based on BH22-1 to BH22-3 drilled within Craigleith WWTP, auger refusal encountered at depths ranging from 6.4m to 7.6m below the existing ground surface, corresponding to Elev. 171.0m to 171.6m. Based on BH225-4 and BH22-5 drilled along Long Point Road, auger refusal encountered at depths of 4.6m to 4.9m below ground surface, corresponding to Elev. 175.3m to 176.4m. To confirm the bedrock surface inferred at auger refusal, bedrock coring should be carried out. Bedrock coring was not included in EnVision's scope of work, as such the depth and elevation of bedrock is not indicated on the boring logs given the presence of cobbles and boulders, the transitional nature of overburden/bedrock contact and the method of sampling. Further investigation involving rock coring should be carried out to confirm the bedrock depth, rock quality, intact strength, rock discontinuity characteristics and lithology for detailed design.

Regional geologic mapping of the Site area suggests that bedrock may belong to the Lindsay Formation and could consist of limestone and dolostone, or alternatively, of shale.

The groundwater levels measured on December 6, 2022, in the monitoring wells installed in Boreholes BH22-1, BH22-3 and BH22-5 were found to be at 0.2m to 0.9m below the existing ground surface corresponding to Elev. 177.6m to 180.4m.

5.3. COBBLES AND BOULDERS

Boulders/cobbles were inferred based on auger grinding and extremely high SPT 'N' values measured in the glacial tills of silty sand and sandy silt textures. A very slow rate of drilling advancement was experienced during augering of these deposits given their heavily overconsolidated nature and likely 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 maximum dimension being equal to or greater than 300 mm.

The majority of boulders within the till are expected to be generally less than 1 m in diameter; however, boulders with maximum dimensions of between 2 and 3 m have been encountered in excavations in the till deposits of Southern Ontario. Cobbles and boulders shall be assumed to be comprised of Canadian Shield derived igneous or metamorphic rock of "extremely high" Cerchar abrasiveness and "very strong to extremely strong" unconfined strength (150 MPa to 250 MPa), as defined by ISRM (International Society for Rock Mechanics).

5.4. DESIGN FROST DEPTH

All pipes and foundations exposed to seasonal freezing conditions must have at least 1.7m of earth cover for frost protection purposes.

5.5. SEISMIC SITE CLASSIFICATION

Based on the borehole information and according to Table 4.1.8.4.A of OBC 2012, the subject site for the proposed pumping station can be classified as Class 'C' for seismic site response.

5.6. PUMPING STATION

5.6.1. FOUNDATIONS

We understand that the base of the proposed sewage pumping station (SPS) will be set between 8m and 12m below the existing ground surface. The depths of boreholes of the current investigation were shallower than the proposed founding depth. Further investigation by means of additional boreholes with rock coring and additional geotechnical testing must be carried out to supplement the findings of this report.

Foundations on Overburden Soils

The proposed pumping station structure can be supported by a raft foundation founded on the undisturbed native, very dense soils at the anticipated founding level using a uniformly distributed bearing pressure of 500 kPa at SLS (Serviceability Limit State) and 750 kPa at ULS (Ultimate Limit State).

For preliminary design of the raft slab, a subgrade reaction modulus of 20 MPa/m may be assumed below the raft slab on undisturbed native very dense soils.

Raft foundations designed to the specified allowable bearing capacity at the Serviceability Limit States (SLS) are expected to settle less than 25 mm total and 15 mm differential provided that the founding soils remain undisturbed and have been properly dewatered in advance of the excavation reaching the design grade.

Deeper boreholes may, however, prove that the founding level for the SPS will lie within bedrock.

Foundations on Bedrock (Inferred)

If competent limestone bedrock is confirmed by rock coring at the anticipated founding elevation of pumping station structure, it can be supported by a raft foundation founded at 0.3m below bedrock surface using a uniformly distributed bearing pressure of 3 MPa at SLS (Serviceability Limit State) and 4.5 MPa at ULS (Ultimate Limit State).

For preliminary design of the raft slab, a subgrade reaction modulus of 100MPa/m may be assumed below the raft slab at or below 0.3m below bedrock surface.

Raft foundations designed to the specified allowable bearing capacity at the Serviceability Limit States (SLS) are expected to settle less than 15 mm total and 10 mm differential.

Deeper boreholes with bedrock coring are required for final design purposes.

The raft must be designed to resist uplift water pressure, assuming the water table as 1m higher than the highest groundwater level or the regional flood level, whichever is greater. Longer term groundwater level monitoring using level loggers will be required to confirm the groundwater table and seasonal groundwater level variations.

It should be noted that the sand/silty sand till deposits are below the groundwater table and are at the founding level, therefore it must be dewatered, otherwise it will result in a loss of ground and bearing capacity. The groundwater must be lowered to 1.0 m below the deepest excavation base level by active dewatering conducted in advance of reaching the design excavation base level. The effectiveness of this dewatering must be verified by means of piezometer readings in the immediate vicinity of the excavation, provided by the Geotechnical Engineer or professional hydrogeologist of Record.

For final design, EnVision will need to review the foundation design drawings. All foundation bases must be inspected by the Geotechnical Engineer prior to placing concrete to ensure their placement on suitably competent, undisturbed founding soils/bedrock. Variations in the soil conditions may occur between and beyond the borehole locations.

It should be noted that the recommended bearing pressure provided by EnVision from the borehole information for the preliminary design stage only. The investigation and comments are necessarily on-going as additional information of the underground conditions becomes available. For example, more specific information is available with respect to conditions arising from supplementary borings to be advanced during the detailed design stage, as well as in-between boreholes and when foundation construction is underway. The interpretation between boreholes and the recommendations of this report must therefore be checked through field inspections provided by EnVision to validate the information for use during the construction stage.

5.6.2. RAFT SLAB AND PERMANENT WATERPROOFING

For a raft slab and watertight (tanked design) structure, waterproofing should be designed for hydrostatic pressures in general accordance with the concepts illustrated in Drawing No. 5. The building science engineer and the nominated waterproofing subtrade will need to prepare detailed designs for the waterproofing system including measures to accommodate tie-back heads, struts, box-outs and other projections into the wall plane.

5.6.3. UPLIFT PRESSURE

The structure of proposed pumping station will extend to about 8m to 12 m below the existing ground surface and the groundwater table measured in the monitoring wells (BH22-1 and BH22-3) was found to be about 0.2m to 0.4m below ground surface. The pumping station structure will, therefore, be subjected to hydrostatic uplift pressures. If the combination of the weight of the structure and the mobilized factored frictional resistance between the buried portion of the exterior walls and the backfill materials is insufficient to resist the uplift forces during any stage of the construction and/or during the operation of the structure, then grouted, double corrosion protected ground anchors will be required to resist buoyancy.

Friction between the exterior walls and the granular backfill materials should only be taken into account if it is absolutely certain that no excavations will be undertaken around the exterior walls any time in the future. In this case, an ultimate friction factor of 0.4 applied to the horizontal active earth pressure on the wall could be used, using a coefficient of earth pressure of 0.33 and a unit weight of 20.5 kN/m^3 above the groundwater level and 10.7 kN/m^3 below the water table can be applied.

When checking the overall stability of the structure, the design should incorporate a minimum safety factor of 1.1 when using only the dead weight of the structures. The safety factor to be used for the frictional resistance should not be less than 2.0.

It is recommended that for the design purposes, the groundwater table be assumed to be as 1.0 m above the highest groundwater level observed in the monitoring wells or the regional flood level, whichever is greater.

Recommendations for the soil and rock anchors are given in Section 5.6.6 of this report.

5.6.4. EXCAVATION AND GROUNDWATER CONTROL

Excavations of overburden can be carried out with heavy hydraulic excavators. The proposed excavation depth for the pumping station structure will range from about 9m to 13m below the existing grade. For the proposed sanitary sewer, the excavation depth may range from about 3.5m to 5.1m (Alternative 1) or about 3.5m to 8.6m (Alternative 2). According to the information from the monitoring wells, the groundwater level was found to be at depths ranging from 0.2m to 0.9m (Elev. 177.6m to 180.4m) below the existing grade. Excavation of the glacial till and the native silts will prove to be very challenging due to the extremely dense packing (heavily overconsolidated nature) of these soils along with the presence of cobbles and boulders. Special excavation buckets and teeth will be needed along with the largest commercially available excavators. Slow and laboured production must be expected in the excavation of these soils.

A continuous caisson wall (secant pile wall) will be required to be installed along the perimeter of the pumping station structure to temporarily control the groundwater seepage during the construction stage, and to permanently reduce groundwater flow. The cut-off caisson wall should be sealed into the bedrock. Additional geotechnical investigation including rock coring and rock (Lugeon) packer test to determine bedrock hydraulic conductivity should be carried out during the final design stage.

Active dewatering, such as by means of closely spaced eductors, will be required to assist the excavation. For more comments on the groundwater control, reference should be made to hydrogeological study for the site. Groundwater must be lowered to at least 1m below the excavation base level in the sandy deposits below the water table. Otherwise, it will result in an unstable base and flowing sides. Even though the sandy glacial till soils are extremely dense in nature, they will tend to loosen/dilate once unconfined in an excavation and will further dilate with groundwater seepage.

It should be noted that the soils contain cobbles and boulders. The presence of obstructions such as concrete or rubble within the surficial fill material is also possible. Provisions must be made in the excavation contract for the removal of boulders in the till deposit and obstructions in the fill material.

All excavations must be carried out in accordance with the most recent Occupational Health and Safety Act (OHSA). In accordance with OHSA, the fill material and native soils (silt and glacial till deposits of sandy silt to silty sand) can be classified as Type 3 Soil above the groundwater table and Type 4 Soil below groundwater table.

Where free draining backfill is required, imported granular fill such as OPSS Granular B should be used. Imported granular fill, which can be compacted with hand held equipment, should be used in confined areas.

EXCAVATION IN BEDROCK

Should excavation in bedrock be required, hoe ramming will be needed. Depending on the rock properties, the rock removal can be arduous and time consuming, and may require use of impact breakers and line-drilling. EnVision recommends supplementary site investigation work involving additional boreholes employing rock coring methods and detailed rock logging in order to better characterize the rock strength, lithology, degree of weathering, hard layer thickness and frequency, fracture frequency, RQD, etc.

5.6.5. EARTH PRESSURES ON PERMANENT SPS WALLS IN OVERBURDEN

The static lateral earth pressures acting on walls may be calculated from the following expression:

$$p = K(\gamma h + q)$$

where p = Lateral earth pressure in kPa acting at depth h

K = Earth pressure coefficient (at rest) for vertical walls and horizontal backfill

K = 0.45 for vertical walls and horizontal site grades surrounding the SPS

γ = Unit weight of backfill, a value of 22 kN/m^3 may be assumed

h = Depth to point of interest in metres

q = Equivalent value of surcharge on the ground surface in kPa

To the above expression, hydrostatic (groundwater) pressures must be added.

5.6.6. TEMPORARY SHORING

For the temporary excavation support of the proposed pumping station, a continuous cut-off caisson wall (secant pile wall) around the perimeter exterior walls will need to be installed to retain the soils and to cut off the seepage from the sandy/silty deposits as well as any shallow perched groundwater. The cut-off caisson wall should be sealed into bedrock.

The shoring system must be designed in accordance with the 4th Edition of the Canadian Foundation Engineering Manual. The surcharge loading from construction equipment, surcharges and adjacent structures must be considered.

Soil anchors will be required to support the shoring. The anchors must be of a length that meets the Canadian Foundation Manual recommendations. It is important to note that the minimum length lies beyond the 45 - ϕ/2 + 0.15H line drawn from the base of the soldier pile and the overall stability of the system must be checked at each anchor level, where H is the shoring height.

The top anchor must not be placed lower than 3.0 meters below the top of level ground surface. Tieback bond values of 100 kPa in very dense soils and 600 kPa in bedrock are suggested but these values are preliminary since the contractor's installation procedures and the soil units that the bond zones are anchored into will determine the actual soil to concrete bond value. Tie back anchors should be post-grouted. Hence, the contractor must decide on a capacity and confirm its availability. All anchors must be tested as indicated in CFEM, 4th edition.

Movement of the shoring system is inevitable. Vertical movements will result from the vertical load on the soldier piles resulting from the inclined tiebacks and inward horizontal movement results from earth and water pressures. The magnitude of this movement can be controlled by sound construction practices, and it is anticipated that the horizontal movement will be in the range of 0.1 to 0.25%H.

To ensure that movements of the shoring are within an acceptable range, monitoring must be carried out. Vertical and horizontal targets on the soldier piles must be located and surveyed before excavation begins. Weekly readings during excavation should show that the movements will be within those predicted; if not, the monitoring results will enable directions to be given to improve the shoring. In more critical areas, such as in proximity to existing utilities or structures, the use of inclinometers to measure horizontal soil strains is also recommended.

5.6.7. PAVEMENT STRUCTURES FOR PUMPING STATION ACCESS ROAD

The recommended pavement structures for the pumping station access road provided in the table below Table 5-1 are based upon an estimate of the subgrade soil properties based on the existing borehole information. Consequently, the recommended pavement structures should be considered for preliminary design purposes only. If required, a more refined pavement structure design can be performed based on specific traffic data and design life requirements and will involve specific laboratory tests to determine frost susceptibility and strength characteristics of the subgrade soils, as well as specific data input from Client.

Table 5-1: Recommended Pavement Structure Thickness

PAVEMENT LAYER	COMPACTION REQUIREMENTS	PAVEMENT STRUCTURE[2]
Asphaltic Concrete (OPSS 1150)	93 % or higher Maximum Relative Density (MRD)	50 mm HL 3 60 mm HL 8
OPSS Granular A Base (or 20mm Crusher Run Limestone	100% SPMDD[1]	150 mm
OPSS Granular B (or 50mm Crusher Run Limestone)	100% SPMDD	450 mm

Notes:

1. Denotes Standard Proctor Maximum Dry Density, ASTM-D698
2. Pavement thickness shall match with the existing pavement, if it is thicker.

Curb-line pavement subdrainage, to the municipality's standard, must be provided in order to maintain a dry granular subbase condition.

The above pavement structure design thicknesses are not intended to support construction traffic. Asphaltic concrete placement should follow completion of all other in-plant heavy construction work.

Full time quality assurance inspection and testing by qualified geotechnical personnel is required during all subgrade preparation, granulars placement and paving operations.

5.7. SANITARY SEWER INSTALLATION USING OPEN CUT METHODS

Open-cut trenches for the proposed sanitary sewer are expected to extend to depths of up to about 5m below existing ground surface in Alternative 1 and up to about 8.6m below existing ground surface in Alternative 2. Excavations for the construction of the sanitary sewer will primarily by through pavement structure, fill materials, and into the underlying native silt and cohesionless glacial till deposits. Locally, such trenches could potentially encounter bedrock at some locations for Alternative 1. In Alternative 2, excavation will very likely extend into bedrock based on inferred bedrock surface from the current borehole information.

Excavation of the glacial till and the native silt overburden will prove to be very challenging due to the extremely dense packing (heavily overconsolidated nature) of these soils along with the presence of cobbles and boulders. Special excavation buckets and teeth will be needed along with the largest commercially available excavators. Slow and laboured production must be expected in the excavation of these soils.

Anomalous trenching conditions with greater potential for wall collapse could also occur in instances where the new sanitary 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. For comments on excavation and groundwater control, please refer to Section 5.6.4.

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 5-2* and is discussed in the following paragraphs.

Table 5-2: Soil Behavior in Open Cut

SOIL TYPE	PIPE SUPPORT	STABILITY DURING CONSTRUCTION IN OPEN CUT EXCAVATION	POSSIBLE MEANS OF GROUNDWATER CONTROL
EXISTING FILL	Not suitable to potentially suitable depending on state of compaction	Stable at 1.5H:1V (unstable below groundwater table)	Closely spaced vacuum well points for trenches <5m deep Closely spaced eductors for trenches >5m deep
NATIVE SILT	Satisfactory for Class B bedding	Stable at 1.5H:1V (unstable below groundwater table unless dewatered)	Closely spaced vacuum well points for trenches <5m deep Closely spaced eductors for trenches >5m deep
SILTY SAND TILL/ SANDY SILT TILL	Satisfactory for Class B bedding	Stable at 1H:1V (unstable below groundwater table unless dewatered)	Closely spaced vacuum well points for trenches <5m deep Closely spaced eductors for trenches >5m deep

5.7.1. Trench Stability and Dewatering

The groundwater level measured within the installed monitoring well along Long Point Road was at a depth of 0.9m below the existing ground surface (approximate Elev. 180.4m) at the time of observation. Based on the borehole information and measured groundwater level in the monitoring well, the anticipated excavation bases will be well below the groundwater levels. For any excavation in cohesionless deposits below groundwater table, groundwater control will be required. Given the low clay content and low plasticity of the on site till deposits, water seepage will dilate the soils and gradually loosen the strength, resulting in decreasing stability of the walls with time. Therefore, more elaborate dewatering procedures such as closely spaced vacuum well points or eductors will be required. The groundwater table must be lowered to at least 1m below the deepest excavation base. Otherwise, it will result in an unstable base and flowing sides. EnVision recommends that test pits be dug at multiple locations along the sanitary alignment during the detailed design stage in order to further assess groundwater seepage and the stability of the trench walls.

For comments on excavation and groundwater control, please refer to Section 5.6.4. For more comments on the groundwater control, reference should be made to hydrogeological study for the site.

Reference to Drawing No. 6 indicates theoretical zones proximal to trenches and excavations at which offset distance 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.

5.7.2. 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 higher rate of productivity in trenching, it is not without some technical drawbacks. These include:

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

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, re-compaction 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 Town 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.

5.7.3. Trenching Adjacent to Existing Services

In areas where the new sanitary sewer impinges on existing utility trenches or passes 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 pipe 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. Other pre-installed shoring measures are more suitable in such circumstances or the new utility should be offset a greater distance from the existing.

5.7.4. Pipe Bedding and Cover

It is anticipated that the existing undisturbed very dense silt and sandy silt to silty sand till deposits encountered in the boreholes will provide adequate support for the sanitary sewer 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., The Town of Blue Mountains Engineering Standards).

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*.

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 recommended 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 sanitary will lie below the groundwater table*. Use of concrete collars in place of bentonite is not recommended as this could induce point loading onto the pipe.

5.7.5. Thrust Block Bearing Resistance

An allowable (or SLS) bearing resistance of 300 kPa and factored ULS bearing resistance of 450 kPa can be used in the design of thrust blocks constructed against native soils. For thrust blocks constructed against engineered fill, the aforementioned values must be reduced by 50%. Where firm or loose fill is encountered, the thrust blocks must be bear against a minimum of 1.0 m thick engineered fill pad. This will require re-excavation of existing fill and replacement with engineered fill placed in layers and compacted to 100% SPMDD.

For design purposes, a coefficient of friction of 0.25 may be used between the granular pipe bedding and the proposed sanitary sewer, assuming that the pipe bedding and its surrounding are adequately compacted in place and in intimate contact with the pipe.

5.7.6. Backfilling and Degree of Compaction

Within the roadway, backfilling of the trenches might 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 1.6 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).

It may also be feasible to reuse some of the excavated cohesionless site soils as compacted trench backfill provided they are carefully segregated, cobbles and boulders are removed and the soils moisture-conditioned (i.e., dried or wetted, as required to be within 2 percent of the materials optimum). This should be further assessed during detail-design supplementary site investigation work when bulk samples for Proctor testing can be obtained.

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.

5.7.7. Lateral Earth Pressure Acting on Temporary Shoring

The lateral earth pressure acting on temporary shoring systems should be calculated based on the appropriate apparent earth pressure envelope as shown on Drawing No. 7.

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

5.8. GEOTECHNICAL QUALITY OF EXCAVATED SOIL FOR REUSE, PAVEMENT RESTORATION

For information related to reuse of excavation spoil at this site, further environmental Excess Soil Management investigation should be carried out at the Site during final design stage.

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.

As previously mentioned, it may also be feasible to reuse some of the excavated cohesionless site soils as compacted trench backfill provided they are carefully segregated, cobbles and boulders are removed and the soils are moisture-conditioned (i.e., dried or wetted, as required to be within 2 percent of the materials optimum). This should be further assessed during detail-design supplementary site investigation work when bulk samples for Proctor testing can be obtained.

In any case the degree of compaction of the trench backfill should be at least 98% of the material's Standard Proctor Maximum Dry Density (SPMDD). This value should be increased to at least 100% 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.

The existing road pavement structure should be reinstated in accordance with the Town's (Town of Blue Mountains) design requirements. New granulars must match into the underside of existing to ensure unimpeded cross drainage. Where a free draining backfill is needed or where the backfill is needed for structural support of overlying structures, the site soils will not be suitable and OPSS Granular "B" sand and gravel will be required. Based on the borehole information and in accordance with the Standard Drawings No. 16-STD-R2, entitled "Rural Standard Cross-Section 9.0m Road – 20m R.O.W" from Town of Blue Mountains, the recommended pavement structures are provided in *Table 5-3*.

Table 5-3: Recommended Pavement Structure Thickness for Road Restoration

PAVEMENT LAYER	COMPACTION REQUIREMENTS	PAVEMENT STRUCTURE[2]
Asphaltic Concrete	95% Maximum Relative Density (MRD)	40 mm OPSS HL-3 60 mm OPSS HL-8
OPSS Granular A Base	100% SPMDD[1]	150 mm
OPSS GRANULAR B OR 50MM Crusher Run Limestone)	100% SPMDD	450 mm but deepened to match existing sub-base

Notes:

1. Denotes Standard Proctor Maximum Dry Density, ASTM-D698
2. Pavement thickness shall match with the existing pavement, if it is thicker.

6. GENERAL COMMENTS AND LIMITATIONS OF REPORT

This is a preliminary report submitted with generic recommendations only and is not to be used for final design purposes. EnVision will need to revise and resubmit this report once we have been provided with plan/profile drawings for the project. In areas where trench inverts may penetrate into bedrock, supplementary site investigation work will be required to better characterize the rock properties.

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(s) 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 preliminary 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 preliminary 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.

6.1. SIGNATURES

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6.2. QUALIFIER

EnVision prepared this report solely for the use of the intended recipient in accordance with the professional services agreement. In the event a contract has not been executed, the parties agree that the EnVision General Terms and Conditions, which were provided prior to the preparation of this report, shall govern their business relationship.

The report is intended to be used in its entirety. No excerpts may be taken to be representative of the findings in the assessment. The conclusions presented in this report are based on work performed by trained, professional and technical staff, in accordance with their reasonable interpretation of current and accepted engineering and scientific practices at the time the work was performed.

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.

EnVision disclaims any obligation to update this report if, after the date of this report, any conditions appear to differ significantly from those presented in this report; however, EnVision reserves the right to amend or supplement this report based on additional information, documentation or evidence.

EnVision makes no other representations whatsoever concerning the legal significance of its findings. The intended recipient is solely responsible for the disclosure of any information contained in this report. If a third party makes use of, relies on, or makes decisions in accordance with this report, said third party is solely responsible for such use, reliance or decisions. EnVision does not accept responsibility for damages, if any, suffered by any third party as a result of decisions made or actions taken by said third party based on this report.

EnVision has provided services to the intended recipient in accordance with the professional services agreement between the parties and in a manner consistent with that degree of care, skill and diligence normally provided by members of the same profession performing the same or comparable services in respect of projects of a similar nature in similar circumstances. It is understood and agreed by EnVision and the recipient of this report that EnVision provides no warranty, express or implied, of any kind. Without limiting the generality of the foregoing, it is agreed and understood by EnVision and the recipient of this report that EnVision makes no representation or warranty whatsoever as to the sufficiency of its scope of work for the purpose sought by the recipient of this report.

In preparing this report, EnVision has relied in good faith on information provided by others, as noted in the report. EnVision has reasonably assumed that the information provided is correct and EnVision is not responsible for the accuracy or completeness of such information.

Unless otherwise agreed in writing by EnVision, the Report shall not be used to express or imply warranty as to the suitability of the site for a particular purpose. EnVision disclaims any responsibility for consequential financial effects on transactions or property values, or requirements for follow-up actions /or costs.

This limitations statement is considered an integral part of this report.

Drawings

Drawing No. 1	Borehole Location Plan
Drawing Nos. 2 to 4	Subsurface Profile
Drawing No. 5	Conceptual Raft Slab System
Drawing No. 6	Risk Zones adjacent to Trench or Excavation
Drawing No. 7	Earth Pressure Distribution on Braced Excavations

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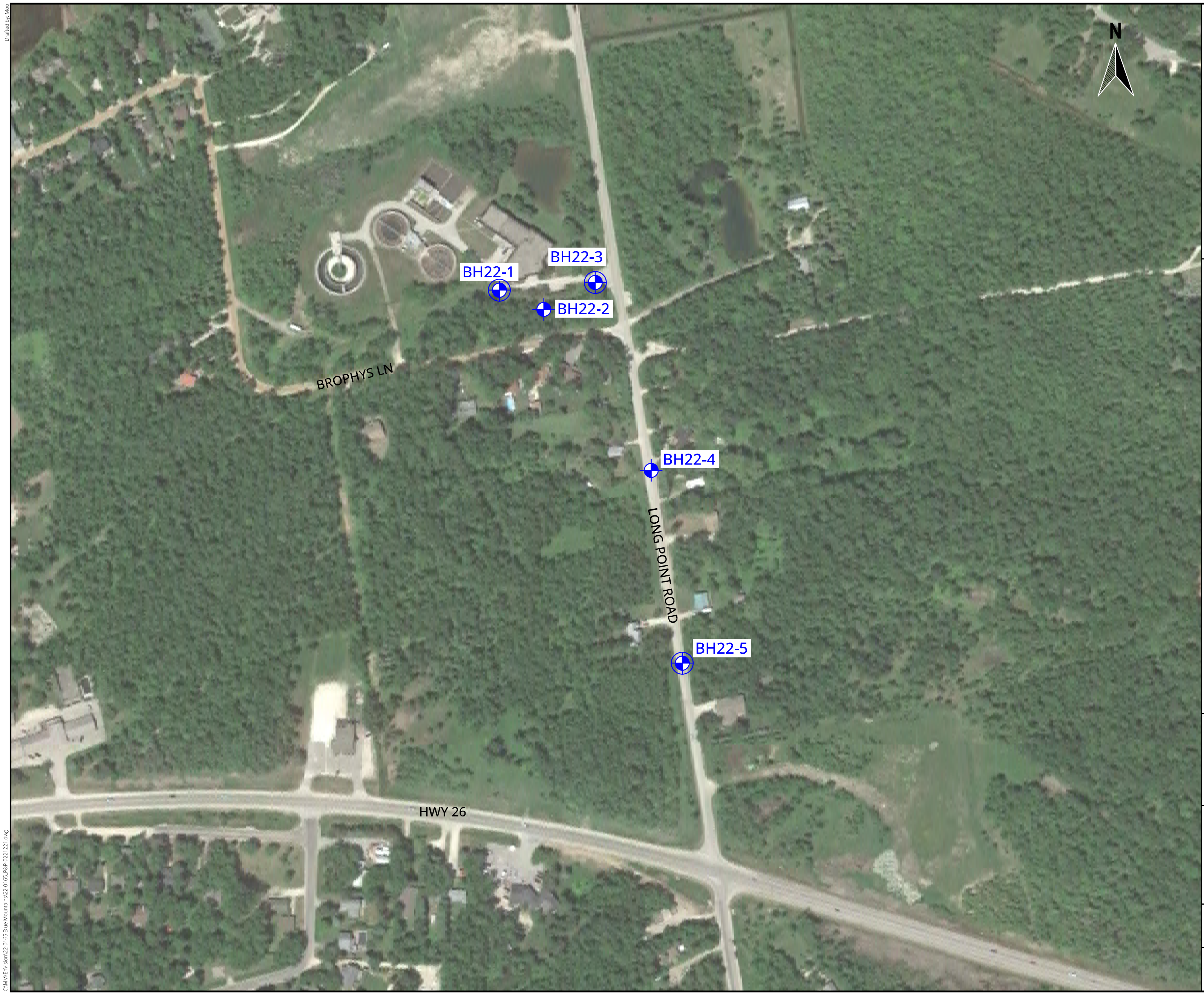
Drafted by: Moo



KEY PLAN
N.T.S

LEGEND:

- ⊕ Borehole Location
- ◉ Borehole with Monitoring Well Location

100 50 0 100m

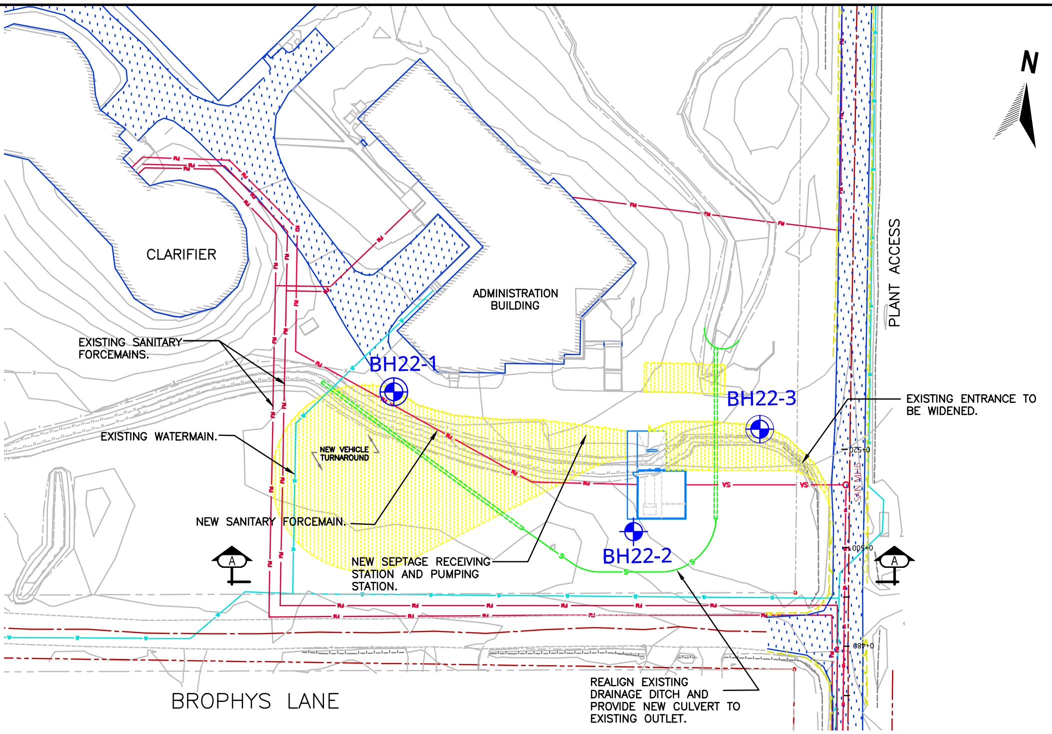
TITLE:

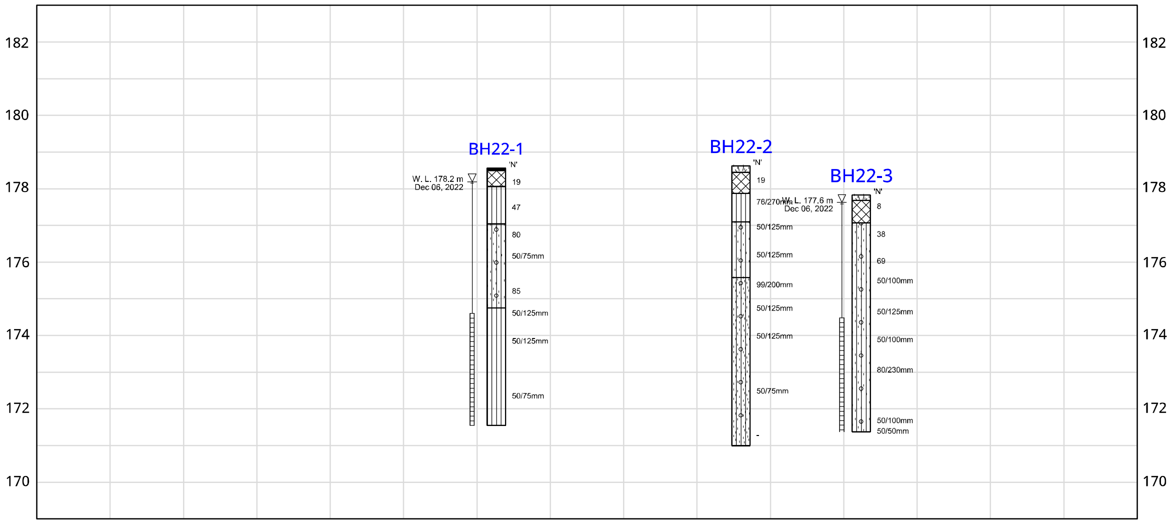
Key Plan

PROJECT :

Geotechnical Investigation
Craigleith Wastewater Treatment Plant,
Craigleith, Ontario

CLIENT :	PREPARED BY:	DATE:
WT Infrastructure Solutions Inc.	TY	December 2022
PROJECT NO:	CHECKED BY:	DRAWING NO:
22 – 0165	MB	1





LEGEND:

- Borehole Location
- Borehole with Monitoring Well Location

■ Asphalt	☒ Fill	Topsoil
Silt	Sandy Silt Till	Sand and Silt Till

40 20 0 40m

TITLE:

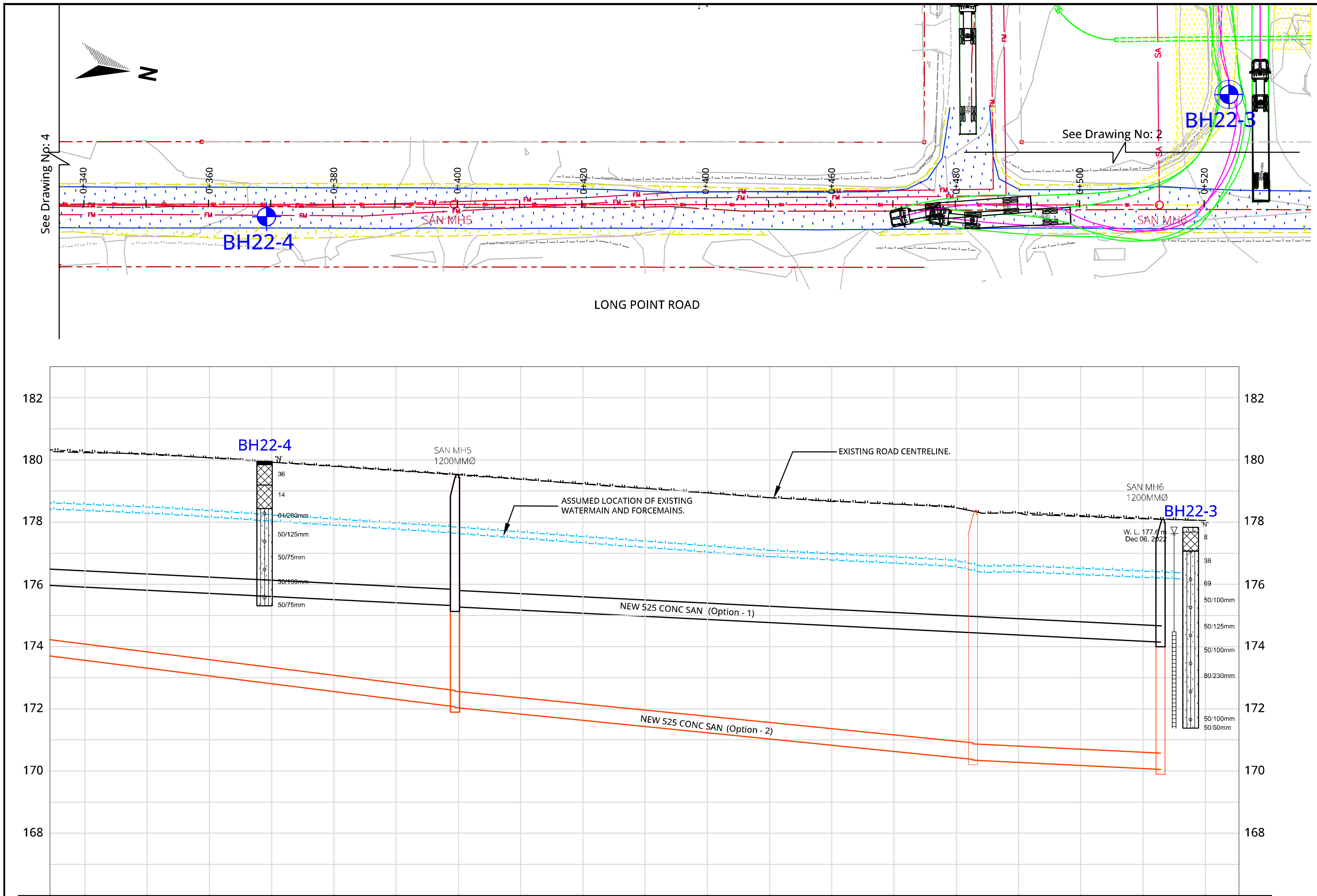
Borehole Location Plan and Geological Sections
Cross Section A-A

PROJECT :

Geotechnical Investigation
Craigleith Wastewater Treatment Plant,
Craigleith, Ontario

CLIENT :	PREPARED BY:	DATE:
WT Infrastructure Solutions Inc.	TY	January 2023
PROJECT NO:	CHECKED BY:	DRAWING NO:
22 – 0165	MB	2

C:\MM\EnVision\22-0165 Blue Mountains\22-0165_P&P-20230104.dwg



Station	Centerline Grade Existing	Centerline Grade Finished	Top of Watermain (Existing / New)	Top of Forcemain (Existing / New)	Sanitary Sewer Invert
0+340	180.23	180.23	178.57	178.57	
0+360	180.08	180.08	178.37	178.37	
0+380	179.79	179.79	178.10	178.10	
0+400	179.53	179.53	177.83	177.83	175.31 175.28
0+420	179.24	179.24	177.53	177.53	
0+440	178.94	178.94	177.24	177.24	
0+450	178.79	178.79	177.09	177.09	174.15 175.28
0+460	178.67	178.67	176.99	176.99	
0+480	178.46	178.46	176.76	176.76	
0+500	178.19	178.19	176.51	176.51	
					174.15
0+520	178.05	178.05			

New 132.62m 525mm Ø Sanitary Sewer @1.00%

New 113.36m 525mm Ø Sanitary Sewer @1.00%

New 113.36m 525mm Ø Sanitary Sewer @1.00%

LEGEND:

- Borehole Location
- Borehole with Monitoring Well Location
- ■ Asphalt
- ▨ Fill
- Topsoil
- Silt
- Sandy Silt Till
- Sand and Silt Till

20 10 0 20m

TITLE:

Borehole Location Plan and Geological Sections
Station 0+340 to Station 0+520

PROJECT :

Geotechnical Investigation
Craigleith Wastewater Treatment Plant,
Craigleith, Ontario

CLIENT : WT Infrastructure Solutions Inc.

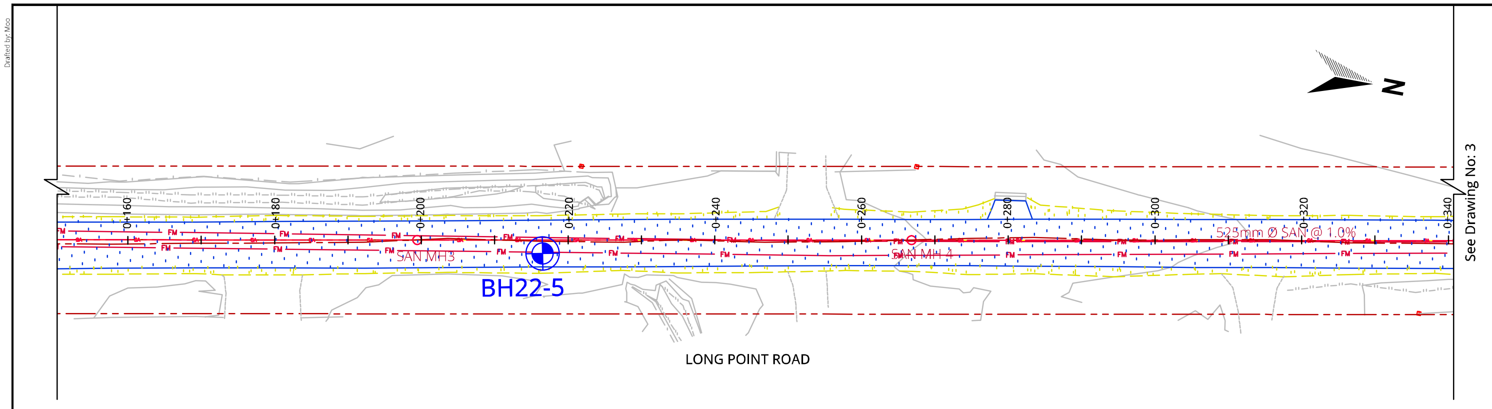
PREPARED BY: TY

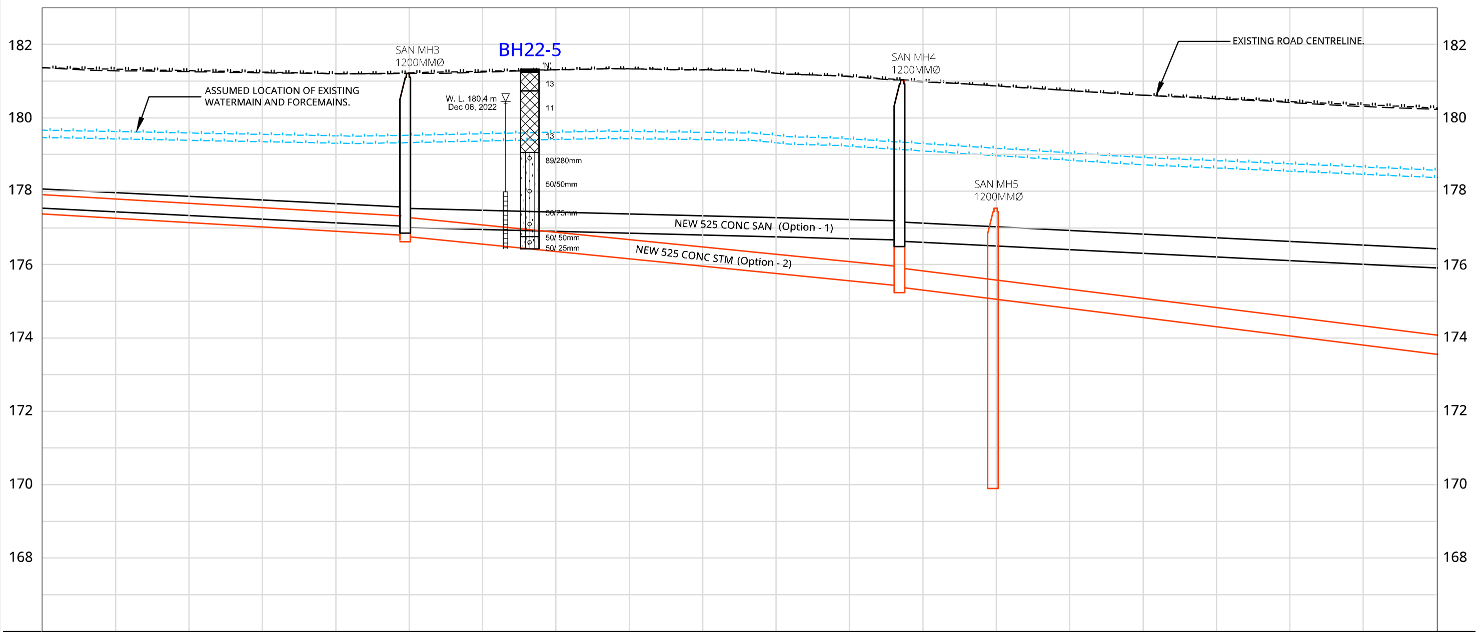
DATE: January 2023

PROJECT NO: 22 – 0165

CHECKED BY: MB

DRAWING NO: 3





	0+160	0+180	0+200	0+220	0+240	0+260	0+280	0+300	0+320	0+340
Existing / Finished Centerline Grade	181.29 178.46	181.23 181.23	181.21 181.21	181.30 181.30	181.31 181.31	181.12 181.12	180.87 180.87	180.62 180.62	180.42 180.42	180.23
Existing / New Top of Watermain	179.63	179.55	179.53	179.61	179.61	179.43	179.17	178.92	178.74	178.57
Existing / New Top of Forcemain	179.63	179.55	179.53	179.61	179.61	179.43	179.17	178.92	178.74	178.57

Sanitary Sewer Invert

New 100.00m 525mm Ø
Sanitary Sewer @1.00%

177.04
177.01

New 67.38m 525mm Ø
Sanitary Sewer @1.00%

176.67
176.64

New 132.62m 525mm Ø
Sanitary Sewer @1.00%

175.31
176.64

LEGEND:

- Borehole Location
- Borehole with Monitoring Well Location
- Asphalt
- Fill
- Topsoil
- Silt
- Sandy Silt Till
- Sand and Silt Till

20 10 0 20m

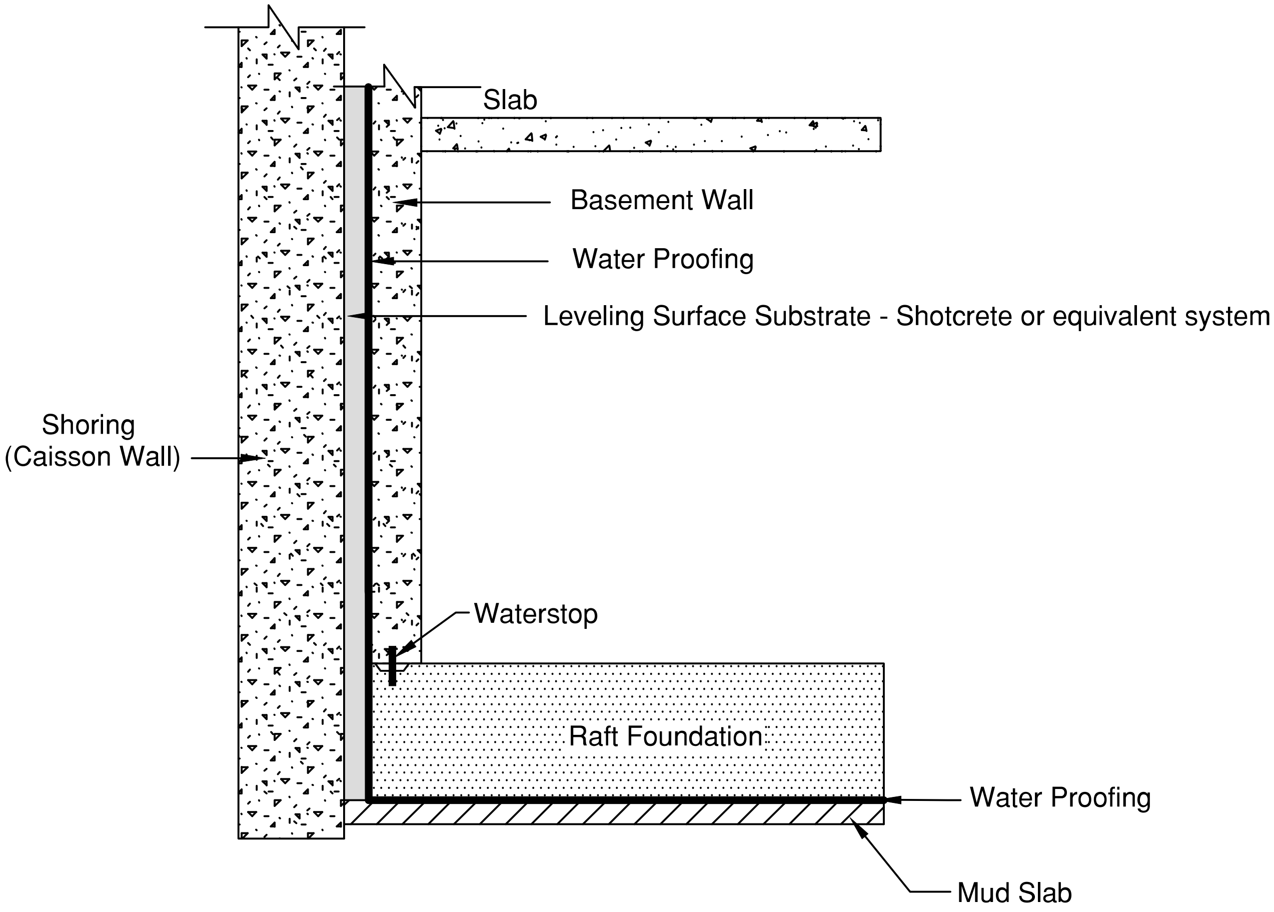
TITLE:

Borehole Location Plan and Geological Sections
Station 0+150 to Station 0+340

PROJECT :

Geotechnical Investigation
Craigleith Wastewater Treatment Plant,
Craigleith, Ontario

CLIENT :	PREPARED BY:	DATE:
WT Infrastructure Solutions Inc.	TY	January 2023
PROJECT NO:	CHECKED BY:	DRAWING NO:
22 – 0165	MB	4

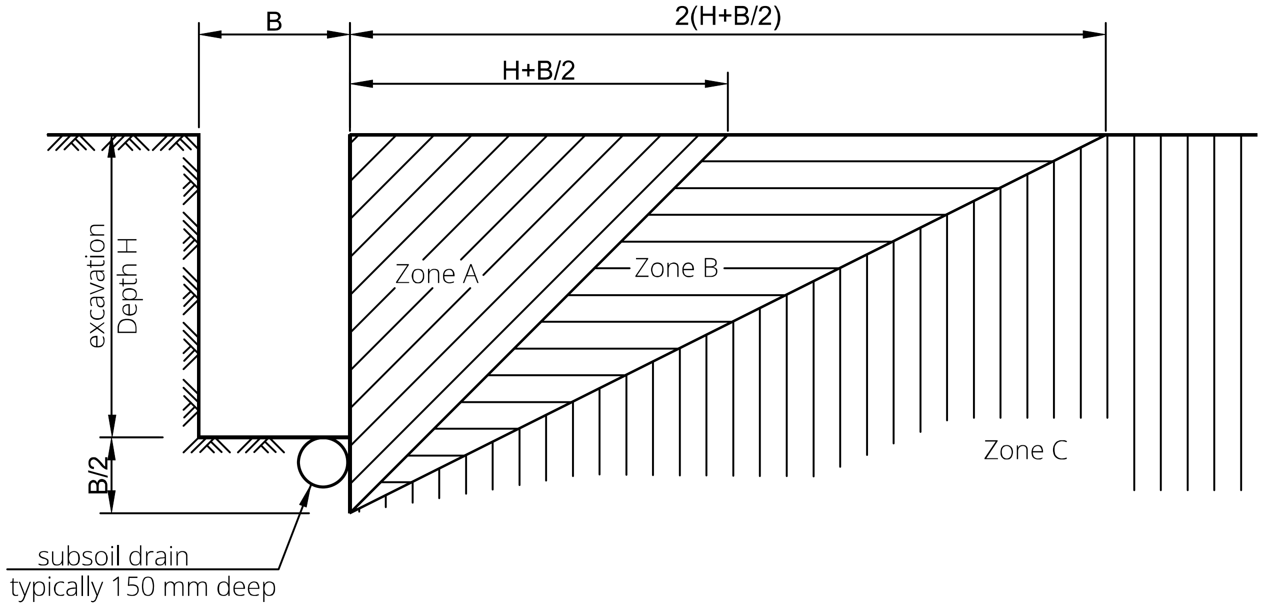


Notes

1. Detailing of the concepts shown on this drawing are to be completed by the Building Science Engineer or Architect.
2. Not for construction.

CONCEPTUAL RAFT SLAB SYSTEM
For Basement Designed as a Watertight Structure

(not to scale)



RISK ZONES (after Howe et al., 1980): Zone A is zone of long term risk, Zone B is zone of intermediate risk, Zone C is zone of no risk.

TITLE: Risk Zone

PROJECT : Geotechnical Investigation - Craigleith WWTP and Associated Sanitary Sewer, Craigleith, ON

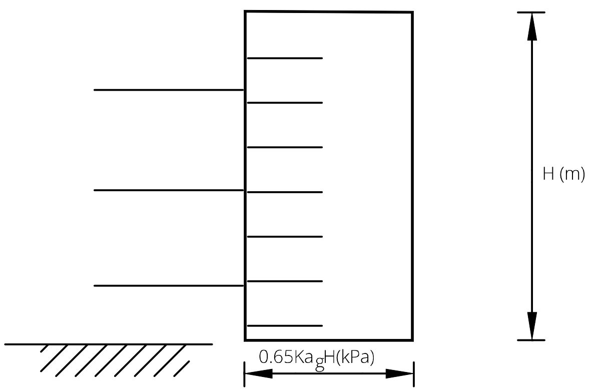
CLIENT : WT Infrastructure Solutions Inc.

PROJECT NO: 22-0165

DATE: December 2022

SCALE: N.T.S

DRAWING NO: 6

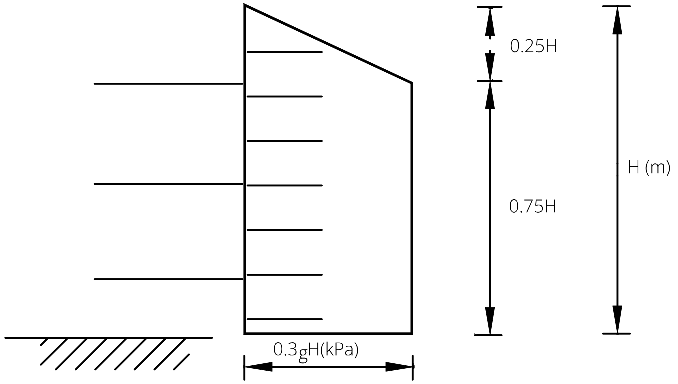


g = unit weight of soil = 21.0 kN/m

g' = submerged unit weight of soil (i.e. below ground water level)= 11.2 kN/m

Ka = 0.3

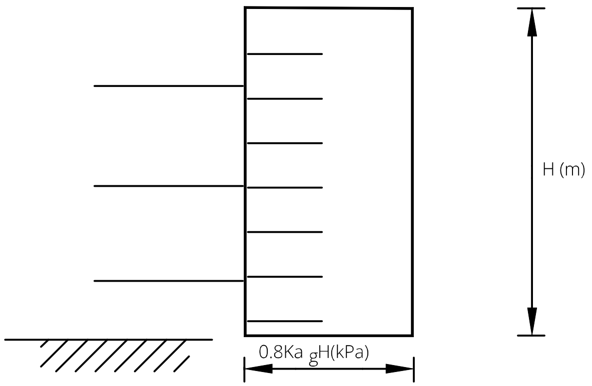
IN COMPACT TO VERY DENSE NON-COHESIVE SOILS (SANDS AND SILTS)



g = unit weight of soil = 21.5 kN/m

g' = submerged unit weight of soil (i.e. below ground water level)= 11.7 kN/m

IN COHESIVE CLAYS OR CLAYEY SOILS

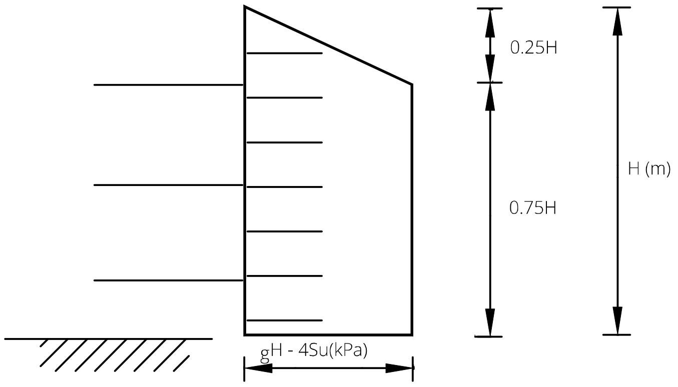


g = unit weight of soil = 19.0 kN/m

g' = submerged unit weight of soil (i.e. below ground water level)= 9.2 kN/m

Ka = 0.36

IN LOOSE OR DISTURBED NON-COHESIVE SOILS (SANDS AND SILTS)



g = unit weight of soil = 19.0 kN/m

g' = submerged unit weight of soil (i.e. below ground water level)= 9.2 kN/m

Su = 10 KPa

IN VERY SOFT TO FIRM COHESIVE CLAYS OR CLAYEY SOILS

Notes:
1. Check system for partial excavation condition.
2. If the free water level is above the base of the excavation, the hydrostatic pressure must be added to the above pressure distribution.
3. If surcharge loadings are present near the excavation, these must be included in the lateral pressure calculation.

TITLE:
Earth Pressure Distribution on Braced Excavations

PROJECT NO:
22-0165

PROJECT :
Geotechnical Investigation -
Craigleith WWTP and Associated Sanitary Sewer,
Craigleith, ON

DATE: December 2022

SCALE: N.T.S

CLIENT : WT Infrastructure Solutions Inc.

DRAWING NO: 7

APPENDIX A:

Notes on Sample Descriptions (Drawing 1A)

Terms used in the Record of Borehole Logs (Drawing 1B)

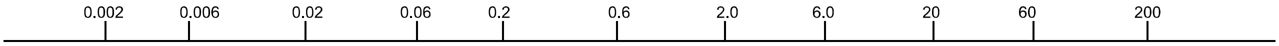
Record of Borehole Sheets (BH22-1 to BH22-5)

Notes On Sample Descriptions

1. All sample descriptions included in this report generally follow the Unified Soil Classification. Laboratory grain size analyses provided by EnVision also follow the same system. Different classification systems may be used by others, such as the system by the International Society for Soil Mechanics and Foundation Engineering (ISSMFE). Please note that, with the exception of those samples where a grain size analysis and/or Atterberg Limits testing have been made, all samples are classified visually. Visual classification is not sufficiently accurate to provide exact grain sizing or precise differentiation between size classification systems.

ISSMFE SOIL CLASSIFICATION

CLAY	SILT			SAND			GRAVEL			COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE		



EQUIVALENT GRAIN DIAMETER IN MILLIMETRES

CLAY (PLASTIC) TO	FINE	MEDIUM	CRS.	FINE	COARSE
SILT (NONPLASTIC)	SAND			GRAVEL	

UNIFIED SOIL CLASSIFICATION

2. Fill: Where fill is designated on the borehole log it is defined as indicated by the sample recovered during the boring process. The reader is cautioned that fills are heterogeneous in nature and variable in density or degree of compaction. The borehole description may therefore not be applicable as a general description of site fill materials. All fills should be expected to contain obstruction such as wood, large concrete pieces or subsurface basements, floors, tanks, etc., none of these may have been encountered in the boreholes. Since boreholes cannot accurately define the contents of the fill, test pits are recommended to provide supplementary information. Despite the use of test pits, the heterogeneous nature of fill will leave some ambiguity as to the exact composition of the fill. Most fills contain pockets, seams, or layers of organically contaminated soil. This organic material can result in the generation of methane gas and/or significant ongoing and future settlements. Fill at this site may have been monitored for the presence of methane gas and, if so, the results are given on the borehole logs. The monitoring process does not indicate the volume of gas that can be potentially generated nor does it pinpoint the source of the gas. These readings are to advise of the presence of gas only, and a detailed study is recommended for sites where any explosive gas/methane is detected. Some fill material may be contaminated by toxic/hazardous waste that renders it unacceptable for deposition in any but designated land fill sites; unless specifically stated the fill on this site has not been tested for contaminants that may be considered toxic or hazardous. This testing and a potential hazard study can be undertaken if requested. In most residential/commercial areas undergoing reconstruction, buried oil tanks are common and are generally not detected in a conventional preliminary geotechnical site investigation.

3. Till: The term till on the borehole logs indicates that the material originates from a geological process associated with glaciation. Because of this geological process the till must be considered heterogeneous in composition and as such may contain pockets and/or seams of material such as sand, gravel, silt or clay. Till often contains cobbles (60 to 200 mm) or boulders (over 200 mm). Contractors may therefore encounter cobbles and boulders during excavation, even if they are not indicated by the borings. It should be appreciated that normal sampling equipment cannot differentiate the size or type of any obstruction. Because of the horizontal and vertical variability of till, the sample description may be applicable to a very limited zone; caution is therefore essential when dealing with sensitive excavations or dewatering programs in till materials.

Drawing 1B

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Explanation of Terms Used in the Record of Borehole

Sample Type

AS	Auger sample
BS	Block sample
CS	Chunk sample
DO	Drive open
DS	Dimension type sample
FS	Foil sample
NR	No recovery
RC	Rock core
SC	Soil core
SS	Spoon sample
SH	Shelby tube sample
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample

Penetration Resistance

Standard Penetration Resistance (SPT), N:
The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in) required to drive a 50 mm (2 in) drive open sampler for a distance of 300 mm (12 in).

WH – Samples sinks under "weight of hammer"

Dynamic Cone Penetration Resistance, N_d:
The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in) to drive uncased a 50 mm (2 in) diameter, 60° cone attached to "A" size drill rods for a distance of 300 mm (12 in).

Textural Classification of Soils (ASTM D2487-10)

Classification	**Particle Size**
Boulders	> 300 mm
Cobbles	75 mm - 300 mm
Gravel	4.75 mm - 75 mm
Sand	0.075 mm - 4.75 mm
Silt	0.002 mm - 0.075 mm
Clay	<0.002 mm(*)

(*) Canadian Foundation Engineering Manual (4th Edition)

Coarse Grain Soil Description (50% greater than 0.075 mm)

Terminology	**Proportion (*)**
Trace	0-10%
Some	10-20%
Adjective (e.g. silty or sandy)	20-35%
And (e.g. sand and gravel)	> 35%

(*) Canadian Foundation Engineering Manual (4th Edition)

Soil Description

a) Cohesive Soils(*)

Consistency	**Undrained Shear Strength (kPa)**	**SPT "N" Value**
Very soft	<12	0-2
Soft	12-25	2-4
Firm	25-50	4-8
Stiff	50-100	8-15
Very stiff	100-200	15-30
Hard	>200	>30

(*) Hierarchy of Shear Strength prediction

1. Lab triaxial test
2. Field vane shear test
3. Lab. vane shear test
4. SPT "N" value
5. Pocket penetrometer

b) Cohesionless Soils

Density Index (Relative Density)	**SPT "N" Value**
Very loose	<4
Loose	4-10
Compact	10-30
Dense	30-50
Very dense	>50

Soil Tests

w	Water content
w_p	Plastic limit
w_l	Liquid limit
C	Consolidation (oedometer) test
CID	Consolidated isotropically drained triaxial test
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement
D_R	Relative density (specific gravity, Gs)
DS	Direct shear test
ENV	Environmental/ chemical analysis
M	Sieve analysis for particle size
MH	Combined sieve and hydrometer (H) analysis
MPC	Modified proctor compaction test
SPC	Standard proctor compaction test
OC	Organic content test
U	Unconsolidated Undrained Triaxial Test
V	Field vane (LV-laboratory vane test)
γ	Unit weight

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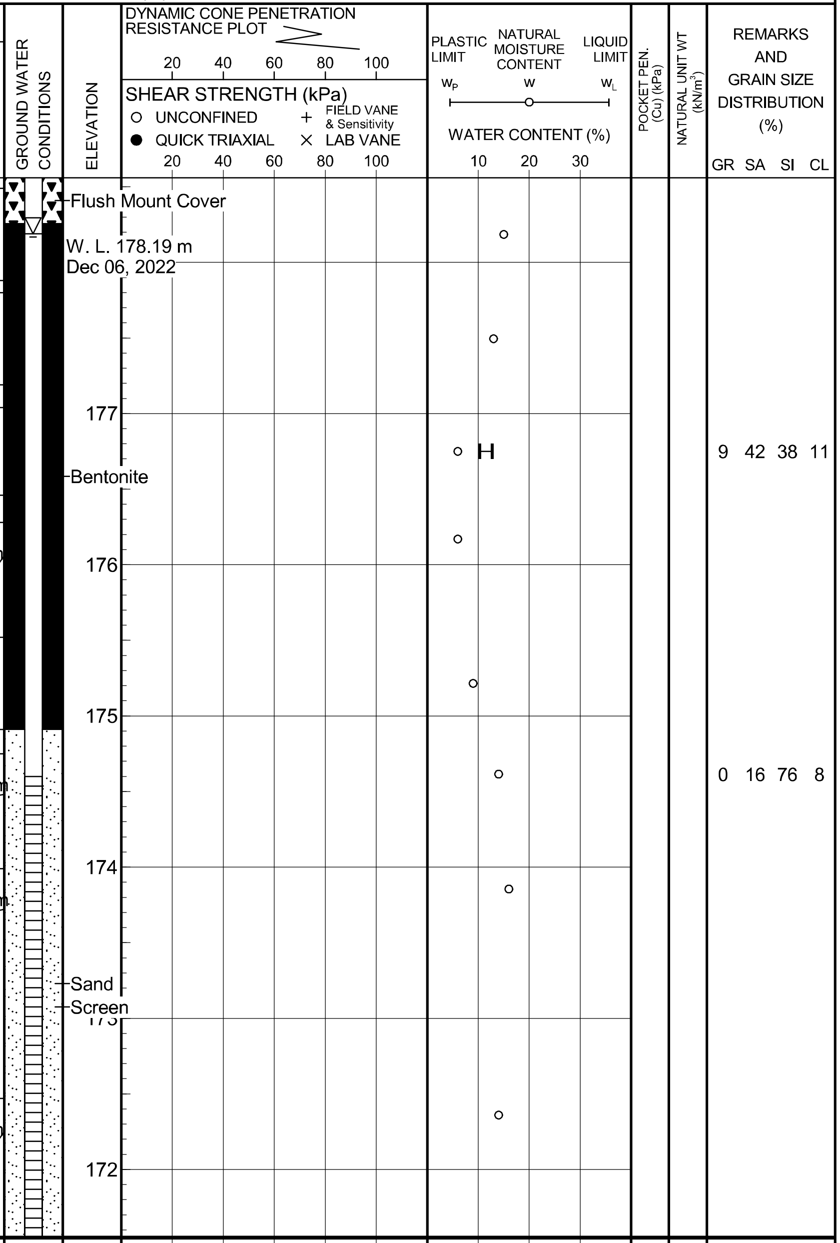
LOG OF BOREHOLE BH22-1

1 OF 1

PROJECT: Geotechnical Investigation - Wastewater Treatment Plant		REF. NO.: 22-0165
CLIENT: WT Infrastructure Solutions Inc.	Method: Solid Stem Augers	ENCL NO.:
PROJECT LOCATION: Longpoint Rd, Craigleith, ON.	Diameter: 152mm	ORIGINATED BY RG
DATUM: Geodetic	Date: Oct/15/2022 to Oct/15/2022	COMPILED BY RG
BH LOCATION: N 4930610.1 E 556148	Equipment: Drill Tech Diedrich D-50	

SOIL PROFILE (m) ELEV DEPTH	DESCRIPTION	SAMPLES NUMBER	TYPE	"N" BLOWS/0.3 m	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
178.56	Ground Surface				
178.49 0.07	**ASPHALT:** 70mm				
	GRANULAR: 355mm, sand and gravel	1	SS	19	
178.06 0.50	**SILT:** some sand, trace gravel, brown, moist, compact to dense.	2	SS	47	
177.04 1.52	**SANDY SILT TILL/SILTY SAND TILL:** trace to some gravel, trace to some clay, inferred cobbles and boulders, contains lenses of silty sand, grey, moist to wet, very dense.	3	SS	80	9 42 38 11
		4	SS	50/75mm	
		5	SS	85	
174.75 3.81	**SILT:** some sand to sandy, trace clay, occasional gravel, grey, moist to wet, very dense.	6	SS	50/125mm	0 16 76 8
		7	SS	50/125mm	
		8	SS	50/75mm	
171.55 7.01	**END OF BOREHOLE**				



Notes:
1) Auger refusal at 7.01m.
2) Borehole was open with wet bottom upon completion of drilling.
3) A 50mm dia. monitoring well was installed upon completion of drilling, screened from 3.96m to 7.01m.

Date	Water level Depth (m)
Dec. 6, 2022	0.37

GROUNDWATER ELEVATIONS
Measurement 1st ▽ 2nd ▼ 3rd ▼ 4th ▼

GRAPH NOTES $+^3, \times^3$: Numbers refer to Sensitivity ○ ^{8}=3% Strain at Failure

LOG OF BOREHOLE BH22-2

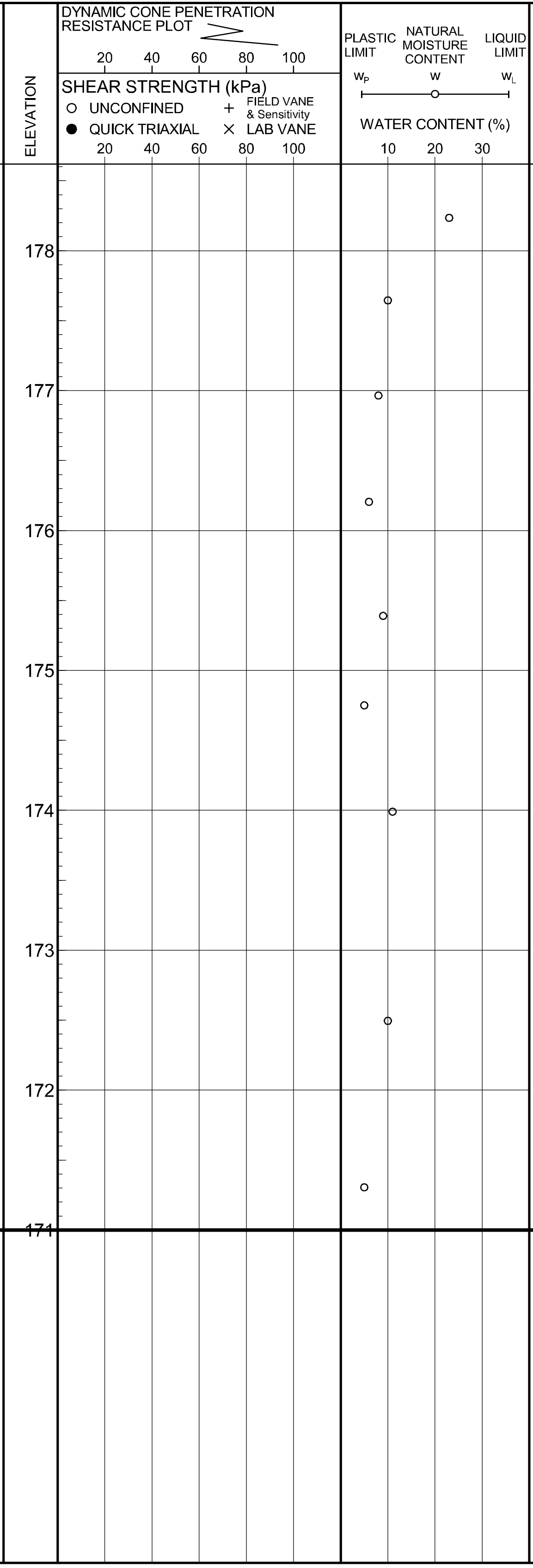
PROJECT: Geotechnical Investigation - Wastewater Treatment Plant
REF. NO.: 22-0165
CLIENT: WT Infrastructure Solutions Inc.
Method: Solid Stem Augers
ENCL NO.:
PROJECT LOCATION: Longpoint Rd, Craigleith, ON.
Diameter: 152mm
ORIGINATED BY RG
DATUM: Geodetic
Date: Nov/15/2022 to Nov/15/2022
COMPILED BY RG
BH LOCATION: N 4930594.9 E 556183
Equipment: Drill Tech Diedrich D-50

SOIL PROFILE

(m) ELEV / DEPTH	DESCRIPTION	STRATA PLOT
178.62	Ground Surface	
178.45 / 0.00	**TOPSOIL:** 170mm	
0.17	**FILL:** clayey silt, brown, moist, very stiff.	
177.87 / 0.75	**SILT:** some sand, trace gravel, brown, moist, very dense.	
177.10 / 1.52	**SANDY SILT TILL:** some gravel, brown to grey, moist, very dense. lenses of silty sand	
175.58 / 3.04	**SAND AND SILT TILL:** trace to some gravel, trace clay, inferred cobbles and boulders, contains lenses of silty sand /sandy silt, grey, moist to wet, very dense.	
171.00 / 7.62	**END OF BOREHOLE** Notes: 1) Auger refusal at 7.62m. 2) Borehole was open and water at a depth of 3.0m completion of drilling.	

SAMPLES

NUMBER	TYPE	"N" BLOWS / 0.3 m	GROUND WATER CONDITIONS	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	19		
2	SS	76/ 270mm		
3	SS	50/ 125mm		
4	SS	50/ 125mm		
5	SS	99/ 200mm		wet spoon 10 35 47 8
6	SS	50/ 125mm		
7	SS	50/ 125mm		
8	SS	50/ 75mm		
9	AS	-		



POCKET PEN. (Cu) (kPa)

NATURAL UNIT WT (kN/m³)

GROUNDWATER ELEVATIONS

Measurement 1st ▽ 2nd ▼ 3rd ▼ 4th ▼

GRAPH NOTES
+ 3, × 3: Numbers refer to Sensitivity
○ 8 =3% Strain at Failure

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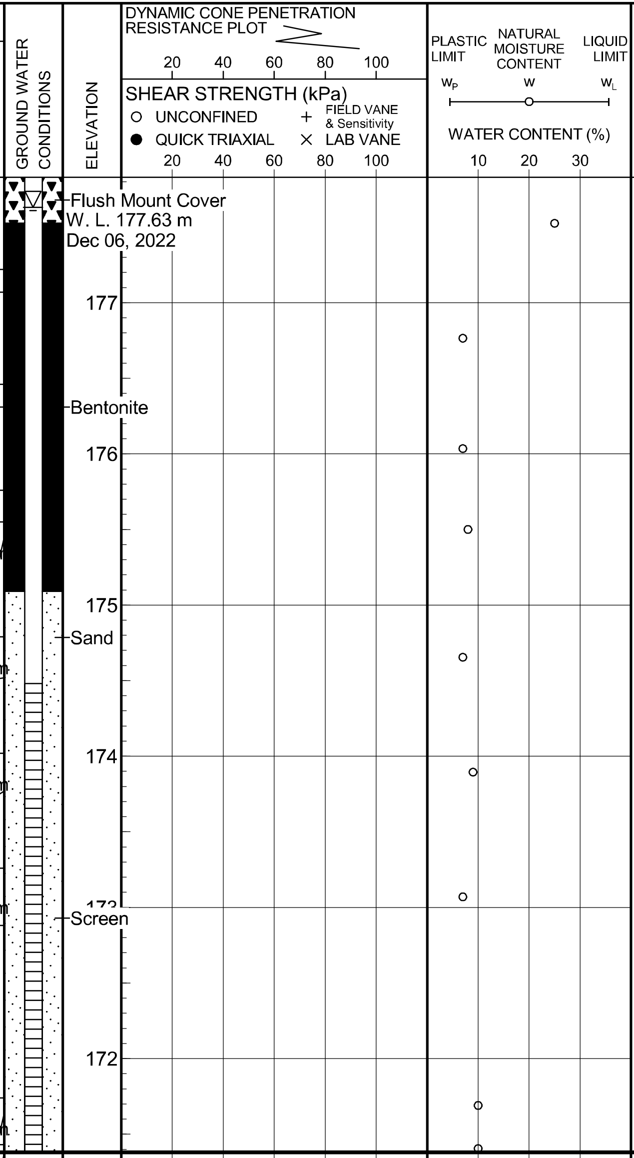
LOG OF BOREHOLE BH22-3

1 OF 1

PROJECT: Geotechnical Investigation - Wastewater Treatment Plant
REF. NO.: 22-0165
CLIENT: WT Infrastructure Solutions Inc.
Method: Solid Stem Augers
ENCL NO.:
PROJECT LOCATION: Longpoint Rd, Craigleith, ON.
Diameter: 152mm
ORIGINATED BY RG
DATUM: Geodetic
Date: Nov/14/2022 to Nov/14/2022
COMPILED BY RG
BH LOCATION: N 4930616.8 E 556222.9
Equipment: Drill Tech Diedrich D-50

SOIL PROFILE		SAMPLES			REMARKS AND GRAIN SIZE DISTRIBUTION (%)			
(m) ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" BLOWS / 0.3 m	GR	SA	SI	CL
177.83	Ground Surface							
177.68 0.00	**TOPSOIL:** 150mm							
0.15	**FILL:** silty clay, some sand, trace gravel, contains rootlets and organics, brown, moist, stiff.	1	SS	8				
177.07 0.76	**SANDY SILT TILL/SILTY SAND TILL:** trace to some gravel, trace clay, contains lenses of silty sand, grey, moist to wet, dense to very dense.	2	SS	38				
	gravelly sandy silt till at 1.7m	3	SS	69	25	47	23	5
	inferred cobbles and boulders below 2.3m	4	SS	50/100mm				
		5	SS	50/125mm				
					wet spoon			
		6	SS	50/100mm				
		7	SS	80/230mm				
		8	SS	50/100mm				
171.38 6.45	**END OF BOREHOLE**	9	SS	50/50mm				



POCKET PEN. (Cu) (kPa)
NATURAL UNIT WT (kN/m^3)

Notes:
1) Auger refusal at 6.4m.
2) Borehole was open and water at a depth of 3.6m upon completion of drilling.
3) A 50mm dia. monitoring well was installed upon completion of drilling, screened from 3.35m to 6.45m.

Date	Water level Depth (m)
Dec. 6, 2022	0.20

ENVISION-SOIL-ROCK-OCTOBER-12-2021.GLB
ENVISION SOIL LOG 2DIG 22-0165 - JAN 4-2023.GPJ 1/4/23

GROUNDWATER ELEVATIONS
Measurement 1st ▽ 2nd ▼ 3rd ▼ 4th ▼

GRAPH NOTES
$+^3, \times^3$: Numbers refer to Sensitivity
○ ^{8}=3% Strain at Failure

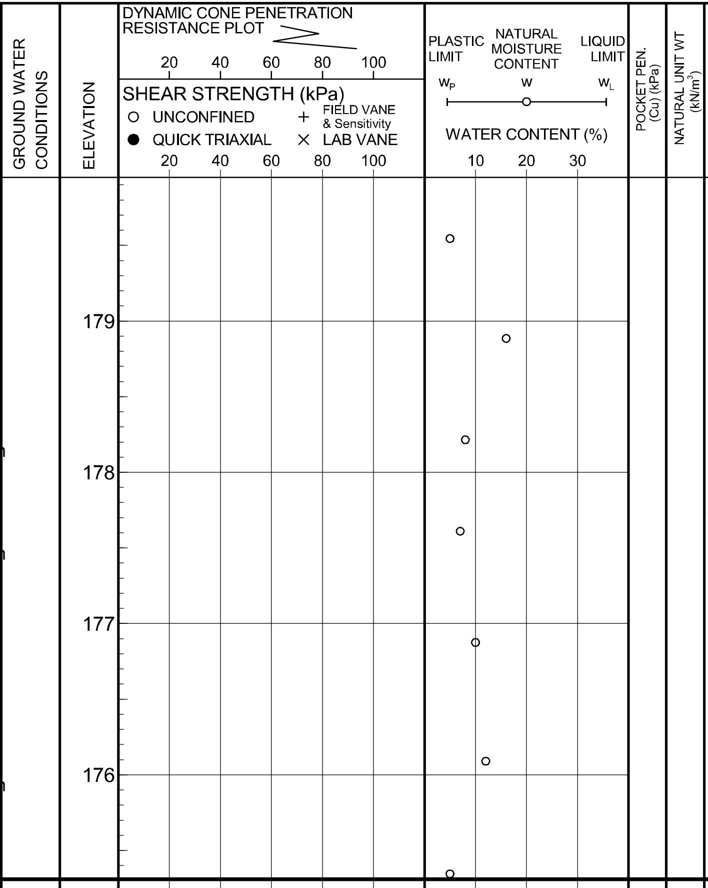
ENVISION
CONSULTANTS LTD

LOG OF BOREHOLE BH22-4

1 OF 1

PROJECT: Geotechnical Investigation - Wastewater Treatment Plant
REF. NO.: 22-0165
CLIENT: WT Infrastructure Solutions Inc.
Method: Solid Stem Augers
ENCL NO.:
PROJECT LOCATION: Longpoint Rd, Craigleith, ON.
Diameter: 152mm
ORIGINATED BY RG
DATUM: Geodetic
Date: Nov/14/2022 to Nov/14/2022
COMPILED BY RG
BH LOCATION: N 4930468.1 E 556269.7
Equipment: Drill Tech Diedrich D-50

(m) ELEV / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" BLOWS / 0.3 m	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
179.95	Ground Surface				
0.00	**ASPHALT:** 100mm				
179.85 0.10	**GRANULAR:** 600mm, sand and gravel	1	SS	36	
179.19 0.76	**FILL:** sand, trace silt, brown, moist,compact.	2	SS	14	
178.43 1.52	**SANDY SILT TILL/SILTY SAND TILL:** trace to some gravel, trace clay, inferred cobbles and boulders, contains lenses of silty sand, grey, moist to wet, very dense.	3	SS	64/ 280mm	21 40 34 5
	gravelly sandy silt till at 1.7m	4	SS	50/ 125mm	Auger grinding
		5	SS	50/ 75mm	
	wet below 3.8m	6	SS	50/ 100mm	wet spoon
175.31 4.64	**END OF BOREHOLE** Notes: 1) Auger refusal at 4.60m. 2) Borehole caved to 2.1m upon completion of drilling.	7	SS	50/ 75mm	



GROUNDWATER ELEVATIONS

Measurement 1st ▽ 2nd ▼ 3rd ▼ 4th ▼

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

○ $^{\varepsilon}$=3% Strain at Failure

ENVISION-SOIL-ROCK-OCTOBER-12-2021.GLB
ENVISION SOIL LOG 2DIG 22-0165 - JAN 4-2023.GPJ 1/4/23

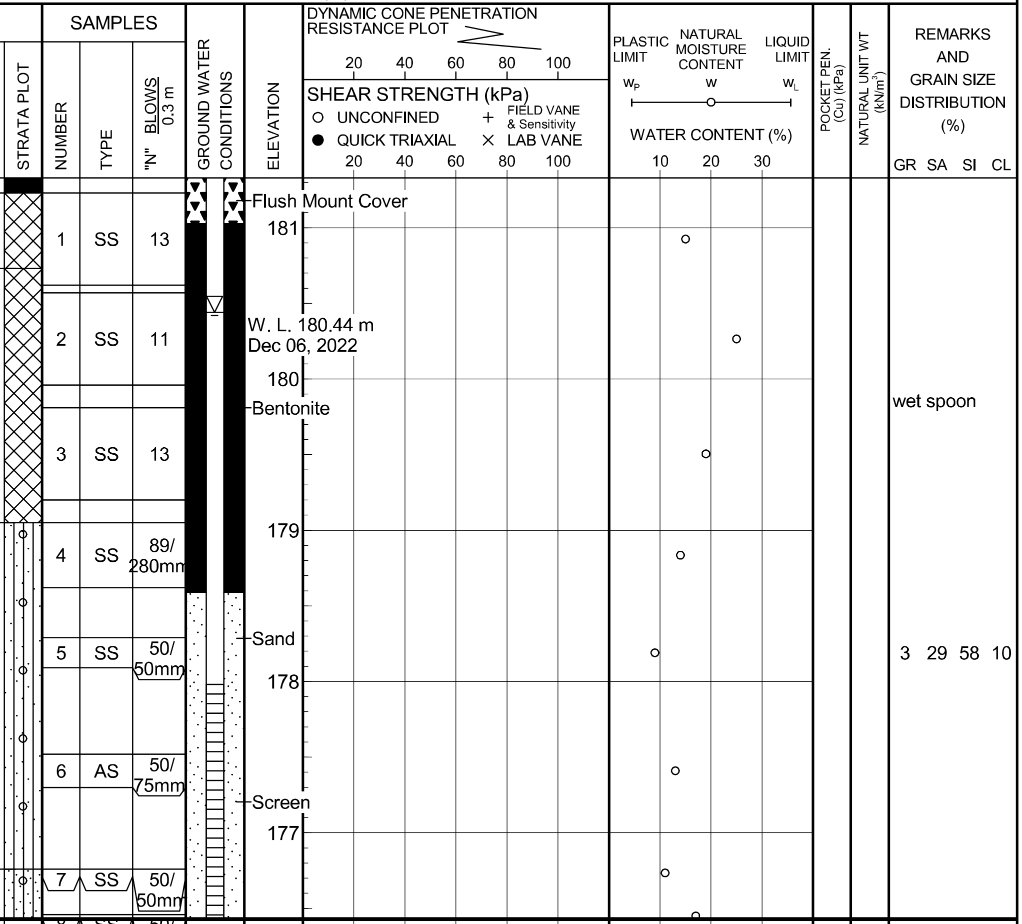
ENVISION
CONSULTANTS LTD

LOG OF BOREHOLE BH22-5

1 OF 1

PROJECT: Geotechnical Investigation - Wastewater Treatment Plant
REF. NO.: 22-0165
CLIENT: WT Infrastructure Solutions Inc.
Method: Solid Stem Augers
ENCL NO.:
PROJECT LOCATION: Longpoint Rd, Craigleith, ON.
Diameter: 152mm
ORIGINATED BY RG
DATUM: Geodetic
Date: Nov/14/2022 to Nov/14/2022
COMPILED BY RG
BH LOCATION: N 4930317.7 E 556296.6
Equipment: Drill Tech Diedrich D-50

ELEV (m) DEPTH	DESCRIPTION	NUMBER	TYPE	"N" BLOWS/0.3 m	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
181.33	Ground Surface				
181.23 0.10	**ASPHALT:** 100mm **GRANULAR:** 400mm, sand and gravel	1	SS	13	
180.73 0.60	**FILL:** sand, trace silt, brown, wet, compact.	2	SS	11	
		3	SS	13	wet spoon
179.05 2.28	**SANDY SILT TILL:** trace gravel, trace clay, inferred cobbles and boulders, contains lenses of silty sand, grey, moist to wet, very dense.	4	SS	89/ 280mm	
		5	SS	50/ 50mm	3 29 58 10
		6	AS	50/ 75mm	
176.76 4.57	**SAND AND SILT TILL:** trace gravel, trace clay, inferred cobbles and boulders, grey, moist to wet, very dense.	7	SS	50/ 50mm	
176.43 4.90	**END OF BOREHOLE**	8	SS	50/ 25mm	



Notes:
1) Augar refusal at 4.9m.
2) Borehole caved to 0.9m upon completion of drilling.
3) A 50mm dia. monitoring well was installed upon completion of drilling, screened at 3.35m to 4.90m.

Date	Water level Depth (m)
Dec. 6, 2022	0.89

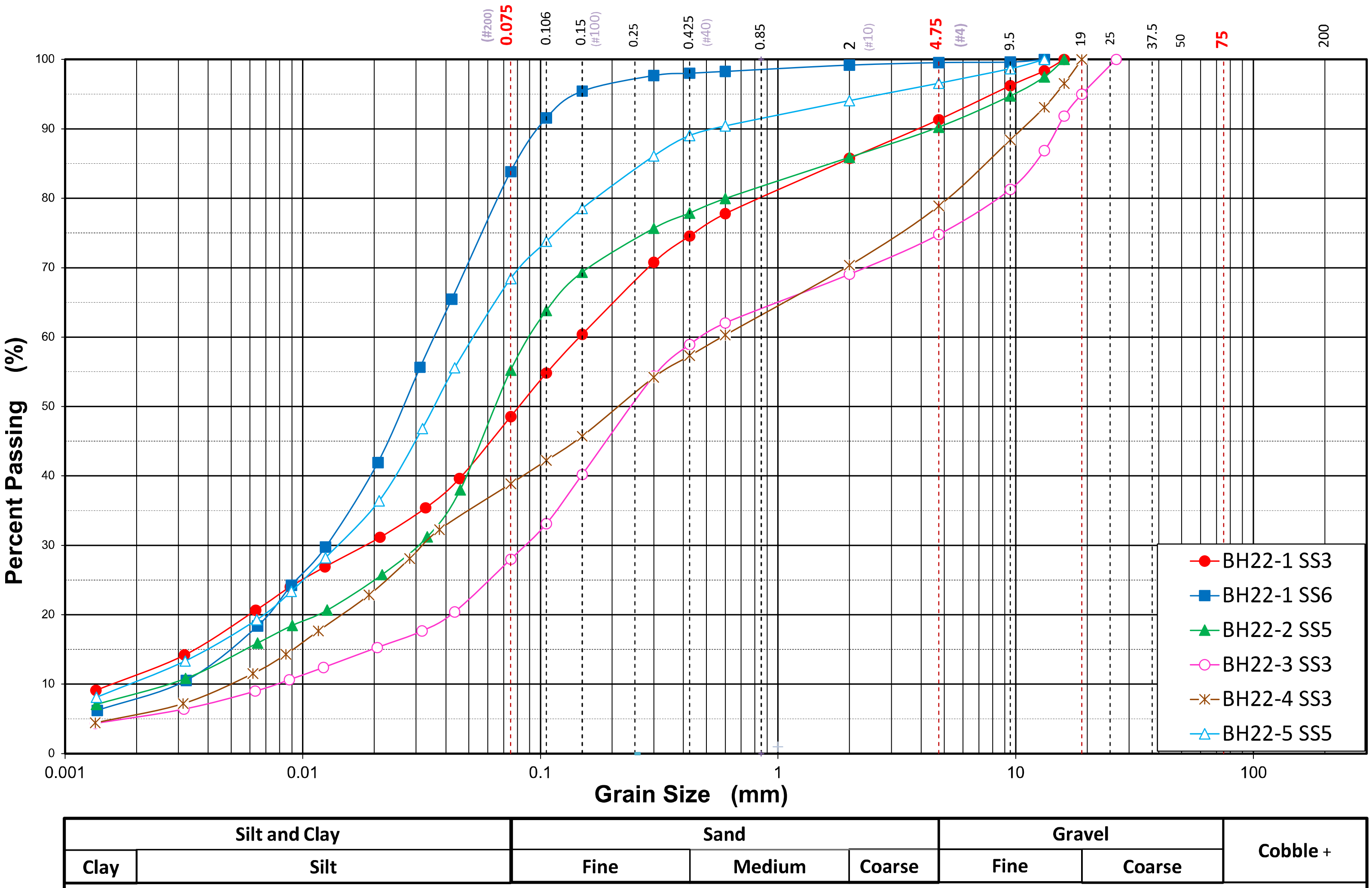
GROUNDWATER ELEVATIONS
Measurement 1st ▽ 2nd ▼ 3rd ▼ 4th ▼

GRAPH NOTES $+^3, \times^3$: Numbers refer to Sensitivity
○ ^{8}=3% Strain at Failure

APPENDIX B:

Grain Size Analyses and Atterberg Limits Test Results

Particle Size Distribution (ASTM-D421/D422)



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GRAIN SIZE DISTRIBUTION

Figure No	1
Project No	22-0165
Date	Dec. 05/2022