PRELIMINARY GEOTECHNICAL INVESTIGATION

1026 Cooke Boulevard, Burlington, Ontario

Client

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Project Number

BIGC-ENV-605A

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1 Introduction

B.I.G. Consulting Inc. (BIG) has been retained by Halton Standard Condominium Corporation No. 416 (the "Client") to conduct a preliminary geotechnical investigation to support the Zoning By-Law Amendment (ZBA) application for the proposed re-development at 1026 Cooke Boulevard, Burlington, Ontario (the "Site"). The Site location is present on Figures 1 and 2 in Appendix A.

The original scope of work for this preliminary investigation was outlined in BIG Proposal (Prop-BIGC-ENV-605A) dated March 2, 2023. An additional scope of work including rock coring and laboratory testing was described in an email by BIG dated June 22, 2023. Both investigations were authorized by Mr. David J. White on behalf of the Client.

Based on the architectural drawings prepared by Turner Fleischer Architects Inc. dated October 26, 2023, BIG understands that the proposed development will comprise one 30-storey building (including a mechanical penthouse) with four (4) levels of underground parking (P4). The plan of the proposed development is shown on Figure 3 in Appendix A.

It should be noted that the fieldwork for this investigation was carried out in conjunction with a Phase Two Environmental Site Assessment (ESA) and a Preliminary Hydrogeological (HG) Investigation. This report addresses the geotechnical aspects of the proposed development only and the Phase Two ESA and preliminary HG report will be issued under separate covers.

The purpose of this investigation was to characterize the subsoils and groundwater conditions across the Site by means of advancing a limited number of boreholes, conducting in-situ tests as well as laboratory tests of selected soil samples; and to prepare a geotechnical engineering report for the preliminary design of the proposed development based on the findings. It should be noted that structural design drawings of the proposed development were in progress, but not made available to BIG at the time of preparation of this report. Therefore, once the existing building on Site is demolished and preliminary structural design drawings become available, this preliminary report should be re-examined considering the latest structural design to devise the necessary additional geotechnical investigation and analysis for detailed design, as appropriate.

The general recommendations and comments made in this report are based on the limited factual geotechnical information available at the time of preparation of this report and are intended only to be used by the design engineers. The number of boreholes, depths, tests data and their interpretation presented in this report may not be sufficient to determine all the factors that may have effects on the design and construction of the proposed re-development.

The report is prepared on the condition that the design of the proposed development will be carried out in accordance with all applicable standards and codes, regulations of authorities having jurisdiction, and good engineering practice and is also subject to the attached 'Report Limitations' (which is an integral part of this report). An on-going liaison with BIG is recommended during both the detailed design and construction stages of the project to ensure that the general recommendations in this report are applicable and/or correctly interpreted and implemented. Also, any queries concerning the geotechnical aspects of the proposed development should be directed to BIG for further elaboration and/or clarification.

2 Geological Settings

For the purposes of regional characterization of the subsoil conditions in the general area of the Site, select geological publications and maps were reviewed. The findings are summarized for reference in the following paragraphs.

Physiographic mapping for Southwestern Ontario (*Chapman, L.J. and Putnam, D.F. 2007; Physiography of Southern Ontario; Ontario Geological Survey, Miscellaneous Release - Data 228*) identifies that the subject Site is located within the Physiographic Region known as Iroquois Plain and associated sand plains. Therefore, the subsoils are generally expected to be composed of sandy deposits.

In addition, quaternary geology mapping (*Ontario Geological Survey 2000. Quaternary Geology, seamless coverage of the Province of Ontario; Ontario Geological Survey, Data Set 14---Revised*) indicates that the study area is located near the boundary between Glaciolacustrine deposits (sand, gravelly sand and gravel) and bedrock (undifferentiated carbonate and clastic sedimentary rock).

Bedrock geology mapping for Southwestern Ontario (*Ontario Geological Survey. 1:250 000 scale, Bedrock Geology of Ontario. Ontario Geological Survey, Miscellaneous Release Data 126, Revised 2006*) indicates that bedrock in the general area of the Site consists of shale, limestone, dolostone and siltstone from Queenston/Georgian Bay Formations.

Geological publications and well records in the general area of the Site indicate that the bedrock surface can be expected at Elevations between 80 and 83 m above sea level (mASL). It should be noted that the bedrock was encountered and confirmed during current field investigation, as described in Section 5 of this report.

3 Site Description

The Site is located to the south of Cooke Boulevard, and west of Plains Road East, in Burlington, Ontario. The Site is currently occupied by one single storey, slab-on-grade, five-unit commercial building at the southwest, with associated asphalt pavement and landscaping area in the surrounding areas.

The Site is rectangular in shape and has an area of approximately 3,205 m^2. The Site is relatively flat-lying. The Site is bounded by Cooke Boulevard to the northeast, single dwelling residential properties to the southwest and commercial lands to the northwest and southeast.

4 Field Investigation

Prior to initiating the subsurface investigation activities, the borehole locations were marked at the Site by BIG personnel and all applicable public utility services (Gas, Bell, Rogers, Hydro, Network cables, etc.) were cleared with the assistance of Ontario-One-Call. A Private Utility Locator was also retained to locate underground private utility lines adjacent to the borehole locations to ensure that the lines will not be damaged and safety of the worker during the investigation work.

The field work, consisting of a total of five (5) boreholes (denotes as BH/MW1 to BH/MW5), was carried out from June 26th to 30th, 2023. These boreholes were advanced to depths ranging from 19.8 to 24.9 m below ground surface (mBGS). The approximate borehole locations established and drilled at the Site are shown on the Borehole Location Plan as Figure 2 in Appendix A.

The boreholes were advanced by using a track mounted drill rig and power operated hollow stem continuous flight auger, supplied and operated by a specialist drilling contractor, working under the full-time supervision of an experienced BIG geotechnical personnel.

Soil samples of the overburden were generally taken at 0.76 m or 1.5 m intervals while performing the Standard Penetration Test (SPT) in accordance with ASTM D1586. This consisted of freely dropping a 63.5 kg hammer for a vertical distance of 0.76 m to drive a 51 mm outer diameter split-barrel (split-spoon) sampler into the ground. The number of blows of the hammer required to drive the sampler into the ground by a vertical distance of 0.30 m was recorded as SPT 'N' value of the soil which indicates the consistency of cohesive soils or the relative density/compactness of non-cohesive soils. Three of the boreholes (BH/MW2, BH/MW4 and BH/MW5) were also cored using HQ size wire line diamond coring method, to confirm the presence and quality of bedrock.

The BIG's drilling supervisor examined and logged the overburden soil and rock samples as they were obtained from the boreholes. The recovered soil samples were sealed in clean, airtight plastic bags and rock-core samples were stored in wooden boxes, both transferred to the BIG's Mississauga laboratory for further examination and laboratory testing. Detailed Records of Boreholes are provided in Appendix B.

Groundwater observations were made in all boreholes during and immediately upon completion of drilling, as appropriate. In order to obtain the information on stabilized groundwater levels, upon completion of drilling, each borehole was equipped with a monitoring well for long term groundwater level observations and for hydrogeological assessment purposes. Monitoring well installation details are shown on the Record of Boreholes in Appendix B.

The ground surface geodetic elevations at the borehole locations were surveyed by BIG personnel. It should be noted that the borehole elevations were not surveyed by a professional surveyor and therefore should be considered approximate. Contractors performing the work should confirm the elevations prior to construction. The borehole locations plotted on Borehole/Monitoring Well Location Plan are based on the measurements of the Site features and should also be considered approximate.

5 Subsurface Conditions

The following summary is to assist the designers of the project with an understanding of the anticipated subsurface conditions across the Site. However, it should be noted that the subsurface soil and groundwater conditions between and beyond the drilled borehole locations may differ from those encountered at the borehole locations, and conditions may become apparent during the construction, which could not be detected or anticipated at the time of the Site investigation. The boundaries between the various strata as shown on the Record of Boreholes are based on the non-continuous sampling and represent an inferred transition between the various strata and their lateral continuation, rather than a precise plane of geological change.

Based on the subsurface conditions encountered at the borehole locations, the soil profile generally consisted of surficial asphalt pavement and existing fill materials overlying native sandy/silty/clayey soil deposits and glacial tills which in turn was underlain by silty clay/shale complex and weathered shale bedrock, as shown on Figure 4 in Appendix A.

The following table summarized the generalized subsurface conditions encountered in each borehole carried out at Site. A brief description of the subsurface stratigraphy and groundwater conditions encountered at the borehole locations are summarized, in order of depth, in the following sections and more information are provided in the Record of Boreholes presented in the Appendix B.

Generalized Stratigraphical Profile at Borehole Locations

Generalized Stratigraphical Profile	BH/MW1	BH/MW2	BH/MW3	BH/MW4	BH/MW5
Asphalt Pavement	0 – 0.2	0 – 0.3	0 – 0.4	0 – 0.3	0 – 0.3
FILL	0.2 – 1.5	0.3 – 2.3	0.4 – 1.5	0.3 – 2.3	0.3 – 4.6
Native Loose to Very Dense Sand (SW/SP)	1.5 – 7.6	2.3 – 13.7	1.5 – 2.7 & 6.1 – 12.2	2.3 – 13.7	4.6 – 12.6
Native Compact to Very Dense Silty Sand (SM)	7.6 – 13.7	-	-	-	-
Native Firm to Hard Silty Clay (CL)	13.7 – 16.8	13.7 – 15.2	2.7 – 6.1	13.7 – 15.2 & 16.8 – 18.3	12.6 – 15.2
Very Stiff to Hard Silty Clay Till (CL)	-	15.2 – 16.8	12.2 – 15.6	-	15.2 – 18.3
Very Dense Sandy Silt (SM)	-	-	15.6 – 17.2	15.2 – 16.8	-
Very Dense Silt (ML)	-	16.8 – 18.3	-	-	-
Dense to Very Dense Sandy Silt Till & Sand and Silt Till (ML)	16.8 – 19.8	18.3 – 19.8	-	18.3 – 19.8	-
Hard Silty Clay/Shale Complex	19.8 – 20.1 (EOB)	19.8 – 20.6	17.2 – 19.8	-	18.3 – 19.7
Weathered Shale Bedrock	-	20.6 – 24.6 (EOB)	19.8 – 19.8 (EOB)	19.8 – 24.9 (EOB)	19.7 – 24.8 (EOB)

5.1 Asphalt Pavement

All boreholes were advanced through the existing asphalt pavement, consisting of approximately 80 to 100 mm (average 86 mm) thick asphaltic concrete over approximately 150 to 320 mm (average 222 mm) thick granular base.

5.2 FILL

Below the asphalt pavement, FILL material was encountered to depths varying between 1.5 and 4.6 mBGS in all boreholes. The FILL material was heterogeneous, but mainly consisted of sand with trace amounts of clay, silt and gravel. It should be noted that the deep FILL encountered in BH/MW5 may be the backfill material from the construction of the underground utilities within the area.

The Standard Penetration Test (SPT) 'N' values recorded within FILL material varied between 4 and 33 blows per 300 mm of penetration, indicating loose to dense conditions. The FILL material was found to be generally moist, based on field visual evaluation. The moisture content measurements for the recovered samples ranged between 4% and 13% by weight.

5.3 Sand and Silty Sand (SM)

Below existing fill materials in all boreholes, native sand and silty sand deposits were encountered. The sandy deposits extended to depths varying from 12.2 to 13.7 mBGS. These deposits were brown to reddish brown in color and contained a trace of clay and gravel.

The SPT 'N' values recorded varied from 6 to over 100 blows per 300 mm of penetration, indicating loose to very dense, but generally compact state of compactness. The sand and silty sand were found to be generally moist to wet, based on field visual evaluation. The moisture content measurements of the recovered sample varied between 4% and 22% by weight.

5.4 Silty Clay (CL)

A stratum of silty clay was encountered below or within the sand, silty sand and sandy silt deposits in all boreholes. This deposit extended to depths ranging from approximately 6.1 to 18.3 mBGS.

The SPT 'N' values recorded in the silty clay ranged from 6 to 55 blows per 300 mm of penetration, indicating a firm to hard consistency. The silty clay was found to be moist to wet, based on visual field evaluations. The moisture content of the recovered sample, measured in laboratory, varied between 18% and 33% by weight.

Grain size analysis and Atterberg Limit test were carried out on one (1) selected soil sample recovered from BH/MW4. The test results are included in Appendix C and summarized below.

Grain Size Analysis Test Results:

Borehole No.	Sample No.	% Gravel	% Sand	% Silt	% Clay
BH/MW4	SS14	0	2	70	28

Atterberg Limits Test Results:

Borehole No.	Sample No.	LL %	PL %	PI %
BH/MW4	SS14	27	14	13

In accordance with the Unified Soil Classification System, the above results indicate the tested native soil is a silty clay of low plasticity [CL].

5.5 Silty Clay Till (CL)

A stratum of silty clay till was encountered below sand or silty clay deposits in boreholes BH/MW2, BH/MW3 and BH/MW5. This deposit extended to depths ranging from approximately 15.6 to 18.3 mBGS. Cobbles and boulders were also anticipated in these deposits.

The SPT 'N' values recorded in the till ranged from 27 to 97 blows per 300 mm of penetration, indicating a very stiff to hard consistency. The till was found to be moist, based on visual field evaluations. The moisture content of the recovered sample, measured in laboratory, varied between 12% and 20% by weight.

Grain size analysis and Atterberg Limit test were carried out on one (1) selected soil sample recovered from BH/MW5. The test results are included in Appendix C and summarized below.

Grain Size Analysis Test Results:

Borehole No.	Sample No.	% Gravel	% Sand	% Silt	% Clay
BH/MW5	SS13	2	18	48	32

Atterberg Limits Test Results:

Borehole No.	Sample No.	LL %	PL %	PI %
BH/MW5	SS13	26	15	11

In accordance with the Unified Soil Classification System, the above results indicate the tested native soil is a silty clay of low plasticity [CL].

5.6 Sandy Silt (SM) and Silt (ML)

A stratum of sandy silt/silt was encountered below silty clay or silty clay till in boreholes BH/MW2 to BH/MW4. The sandy silt/silt deposits extended to depths varying from 16.8 to 18.3 mBGS.

The SPT 'N' values recorded varied from 58 to 97 blows per 300 mm of penetration, indicating very dense condition. The measured moisture content of the recovered samples varied between 16% and 23% by weight, generally indicating moist to wet.

5.7 Sandy Silt Till and/or Sand and Silt Till (ML)

A stratum of sandy silt till and/or sand and silt till was encountered below silty clay or silt in boreholes BH/MW1, BH/MW2 and BH/MW4. The tills extended to a depth of approximately 19.8 mBGS.

The SPT 'N' values recorded varied from 31 to over 100 blows per 300 mm of penetration, indicating dense to very dense state of compactness. The tills were found to be generally moist to wet, based on field visual evaluation. The moisture content measurements of the recovered sample varied between 14% and 23% by weight.

Grain size analysis was carried out on one (1) selected soil sample recovered from BH/MW2. The test results are included in Appendix C and summarized below.

Grain Size Analysis Test Results:

Borehole No.	Sample No.	% Gravel	% Sand	% Silt	% Clay	Soil Classification
BH/MW2	SS15	9	26	58	7	ML

5.8 Silty Clay/Shale Complex

A stratum of silty clay/shale complex was encountered below the native sandy silt, sandy silt till and silty clay till in all boreholes except BH/MW4. The deposits extended to depths ranging from approximately 19.7 to 20.6 mBGS. This stratum was difficult to penetrate with the drilling augers due to the fragmented shale/limestone content and its hard consistency. Cobbles and boulders were also anticipated in these deposits.

The SPT 'N' values recorded in the silty clay/shale complex ranged from 58 to over 100 blows per 300 mm of penetration, indicating a hard consistency. These deposits were moist, based on field visual evaluation. The moisture content measurements of the recovered sample varied between 10% and 11% by weight.

5.9 Shale Bedrock

Below the sand and silt till or silty clay/shale complex in all boreholes, reddish brown to grey, highly weathered to fresh shale bedrock from Queenston/Georgian Bay Formation was encountered/inferred. Boreholes BH/MW2 to BH/MW5 were terminated within the shale bedrock at depths ranging from 19.8 to 24.8 mBGS while the split spoon got refusal within probable shale bedrock in BH/MW1. The encountered/inferred bedrock surface in the boreholes are summarized in below table.

Summary of Encountered/Inferred Bedrock Surface

Borehole No.	Ground Surface Elevation (mASL)	Approx. Bedrock Surface Depth (mBGS)	Approx. Bedrock Surface Elevation (mASL)
BH/MW1	101.98	20.07*	81.91*
BH/MW2	101.67	20.65	81.02
BH/MW3	100.64	19.84	80.80

Borehole No.	Ground Surface Elevation (mASL)	Approx. Bedrock Surface Depth (mBGS)	Approx. Bedrock Surface Elevation (mASL)
BH/MW4	101.71	19.81	81.90
BH/MW5	101.01	19.66	81.35

**Inferred probable bedrock surface due to split spoon refusal*

It should be noted that it was often difficult to distinguish where the shale bedrock began, particularly where the bedrock surface was highly weathered. As such, the inferred bedrock surface should not be considered accurate within ± 1.0 m.

The Shale bedrock was cored in boreholes BH/MW2, BH/MW4 and BH/MW5, to depths ranging from 24.6 to 24.9 mBGS by HQ size rock core drilling. Photographs of the obtained rock cores are presented in Appendix D. The total core recovery (TCR) from the core samples was 100% and the solid core recovery (SCR) ranged from 76% to 100%. The recorded Rock quality designation (RQD) values ranged from 29% to 98%, indicating poor to excellent rock quality. While the use of double tube core barrels provided reasonably good protection of the core during drilling and core retrieval, the fissile nature of the sedimentary shale greatly influences the RQD values of the rock cores. Interbedded siltstone and limestone layers were also present within the obtained core samples, with increased frequency with depth.

Two (2) representative rock cores from BH/MW2 and BH4 were subjected to Unconfined Compressive Strength (UCS) testing. The testing was carried out in general accordance with ASTM Standard Test Method D7012 – 14, entitled "Standard Test Methods for Compressive Strength and Elastic Moduli of Intact Rock Core Specimens under Varying Stated of Stress and Temperatures". The detailed results of UCS testing are included in Appendix G and are summarized in table below. The UCS values yielded from the tests conducted on core samples ranged from 47.5 MPa to 73.6 MPa, based on which the strength of the shale bedrock can be classified as medium strong to strong (Canadian Foundation Engineering Manual, 2006, 4th Edition, Table 3.5).

Unconfined Compressive Strength (UCS) Test Results

Borehole No. & Run No.	Sample Depth (mBGS)	Sample Elevation (mASL)	UCS (MPa)
BH/MW2 R2	22.68 to 22.86	78.99 – 78.81	73.6
BH/MW4 R1	20.37 to 20.52	81.34 – 81.19	47.5

In addition, eight (8) samples of the rock cores were subjected to point load index strength testing, for both axial and diametric tests according to ASTM D5731-16. The measured axial point load index strengths correspond to unconfined compressive strengths ranging from approximately 18.65 MPa to 78.13 MPa, indicating weak to strong rock strength (Canadian Foundation Engineering Manual, 2006, 4th Edition, Table 3.5). The point load test results on the rock core samples are present in Appendix E and included on the Record of Borehole in Appendix B, as summarized in below table.

Summary of Inferred UCS Values from Point Load Tests

Borehole No. & Run No.	Sample Depth (mBGS)	Sample Elevation (mASL)	Inferred UCS from Point Load Test (MPa)
BH/MW2 R1	20.88 to 21.03	80.79 to 80.64	64.7
BH/MW2 R2	23.37 to 23.57	78.30 to 78.10	63.9
BH/MW2 R3	23.93 to 24.08	77.74 to 77.59	33.5
BH/MW4 R2	21.49 to 21.67	80.22 to 80.04	78.1

Borehole No. & Run No.	Sample Depth (mBGS)	Sample Elevation (mASL)	Inferred UCS from Point Load Test (MPa)
BH/MW4 R3	24.70 to 24.89	77.01 to 76.82	43.8
BH/MW5 R2	20.19 to 20.40	80.82 to 80.61	18.6
BH/MW5 R3	21.72 to 21.95	79.29 to 79.06	43.9
BH/MW5 R4	24.31 to 24.51	76.70 to 76.50	42.6

5.10 Groundwater Observation

Groundwater observations were made in all boreholes during drilling and upon completion of drilling, when applicable.

In order to obtain the information on stabilized groundwater levels, all boreholes were equipped with monitoring wells upon completion of drilling. The groundwater levels in the monitoring wells were recorded on July 3rd, 2023, as tabulated below:

Groundwater Observations:

Borehole No.	Ground Elevation (m)	Borehole Depth (mBGS)	MW Depth (mBGS)	Screen Length (m)	Groundwater Depth (mBGS)/ Elevation (mASL)
					July 3rd, 2023
BH/MW1	101.98	20.1	12.2	3	9.33/92.65
BH/MW2	101.67	24.6	13.7	3	9.00/92.67
BH/MW3	100.64	19.8	12.2	3	7.92/92.72
BH/MW4	101.71	24.9	12.2	3	9.06/92.65
BH/MW5	101.01	24.8	12.2	3	8.38/92.63

mBGS: Meter Below Ground Surface
mASL: Meter Above Sea Level

Records of groundwater observation of BIG investigation are also shown on the respective Record of Boreholes. It should be noted that the groundwater levels at the Site may fluctuate seasonally and may be expected to be somewhat higher during the spring months and in response to major weather events.

6 Engineering Discussion and Recommendation

As aforementioned, BIG understands that the proposed development will be comprised of one 30-storey building with four (4) levels of underground parking (P4). The P4 finished floor slab is anticipated to be at a depth of 14.45 mBGS, according to the latestArchitectural Drawing SPA401 by Turner Fleischer dated September 7, 2023.

It should be noted that no detailed structural design drawings of the proposed development were made available to BIG at the time of preparation of this report (given that the project seems to be currently in the conceptual/preliminary design stage). Therefore, once the existing building is demolished and preliminary structural design drawings (i.e., loading and proposed foundation general arrangement) becomes available, the recommendations provided under following sections should be further reviewed and additional field investigation and geotechnical analysis may then be recommended for optimization of foundation detailed design across the entire footprint of the proposed high rise, as appropriate.

Based on borehole findings, the soil profile generally consisted of surficial asphalt pavement and existing FILL materials overlying native loose to very dense sandy/silty deposits and firm to hard clayey deposits and very stiff to hard/Dense to Very Dense glacial tills which in turn was underlain by hard silty clay/shale complex and weathered shale bedrock to termination depths of most deep boreholes, as shown on Figure

4 in Appendix A. Stabilized groundwater level measurements were recorded in the monitoring wells installed across the Site at depths ranging from 7.9 to 9.3 mBGS, corresponding to Elevations between 92.63 and 92.72 mASL.

The recommendations and comments provided in this report are based on factual information obtained from this investigation and are intended only for use by the design engineers. The number of boreholes, tests data and their interpretation presented in this report may not be sufficient to determine all the factors that may have effects on the design and construction of the proposed development.

The following discussion and recommendations should be revised or supplemented where necessary when the conditions of the proposed development are different from the noted conditions/assumptions.

6.1 Grading and Site Preparation

Proper grading and site preparation are very important for the success of any planned development. As parts of effective and efficient design and construction of the proposed development, the following items highlight the fundamental geotechnical requirements to be considered during grading and site preparation. Detailed recommendations are provided in the following sections:

a) All ground surface cover (asphalt pavement, topsoil, trees, etc.) should be stripped/cut down and removed from the area of the proposed re-development.
b) It is our understanding that all existing buildings throughout the Site will be demolished, and the floor slabs, walls, foundations, etc. of the demolished building will be sub-excavated and removed completely.
c) Any existing infrastructures (e.g., manholes, catch basins, buried structures, etc.) should be sub-excavated and removed from the area of the proposed development, if they are located within the zone of influence of foundations of the proposed development. The zone of influence of the foundation is defined as an area laterally extending 1 m beyond the bottom edge of the foundation with downward slope of 1H:1V (or 1.5H:1V from the outside edge of proposed foundations). Similarly, any existing underground services, outside of the foundation influence, should be either removed or abandoned by injecting with non-shrinkable grout.
d) Care must be taken during excavation near any existing structures and underground services/utilities located within or adjacent to the excavation. Foundations of heavily loaded settlement sensitive structures and utilities located within close proximity of the proposed excavation should be accurately located and supported adequately with the suitable temporary or permanent support system where required, prior to excavation, to preserve the integrity of these structures. Similarly, excavation near the vicinity of any existing structure should be carried out without disturbing and/or undermining their foundations.
e) Where open excavation is not feasible, a properly designed perimeter shoring system should be installed prior to the mass excavation for the proposed development. For the drilling and installation of shoring system (e.g., caissons, etc.), travel path and working platform areas of the Site for drill rig must be properly prepared, inspected and approved by a geotechnical engineer from BIG prior to starting the installation of shoring system.
f) An appropriate temporary groundwater control system should be designed, installed, and operated by an experienced dewatering contractor prior to excavation and keep the base of excavation dry (at all the times for the duration of below grade construction works) to preserve the undisturbed conditions of the foundation subsoils. The groundwater level should be kept at least 1 m below the base of excavation (i.e., lowest depth of excavation). In no case should any footing be placed on dilated or disturbed subgrade.

g) The base of excavation at design subgrade level should be inspected and approved by a geotechnical engineer from BIG. During inspection, any soft/loose and wet spot identified, should be sub-excavated and replaced with approved material as directed by the geotechnical engineer.

h) Any structural backfill specified as Engineered Fill should consist of imported OPSS Granular B, OPSS Select Sub-grade material and/or portions of on-site soils which do not contain organics and deleterious materials as approved by the Geotechnical Engineer. Some reconditioning (i.e., drying) prior to reuse may be required, if the materials are found to be too wet. However, any imported soil to the Site for Engineered Fill must meet the requirements of O. Reg. 153/04 as determined by BIG.

i) To reduce the post-construction settlements, all new fills should be placed in thin lifts, not exceeding 200 mm thick loose lifts, within ±2 % of its optimum moisture content, and thoroughly compacted with suitable heavy compactors to at least 98% of Standard Proctor Maximum Dry Density (SPMDD) of the fill material, before placing the next lift.

j) Surface water runoff from the neighboring properties should not be permitted to enter and/or pond within the construction area. Consideration should be given to redirecting the surface water runoff from the neighboring properties (temporary and/or permanent) if there would be a down gradient and grade difference between site grades and existing neighboring property grades. This is especially important to the success of the planned construction.

k) The existing on-site soils are susceptible to disturbance when exposed to weather and construction traffic. Construction traffic over exposed subgrade materials should be minimized, and temporary construction hauling routes should be established. If these routes coincide with future paved areas, adequately reinforced haul roads (e.g., increased thickness of granular sub-base, geo-fabrics, etc.) should be constructed to reduce disturbance to the subgrade soils. Similarly, if the subgrade becomes excessively wet or rutted during construction activities, additional sub-base material may be required. The need for additional subbase material is best determined during construction.

6.2 Foundation Options and Design Parameters

Based on the provided architectural drawings as aforementioned, BIG understands that the proposed development will comprise one 30-storey building with four (4) levels of underground parking. The P4 floor slab is anticipated at a depth of 14.45 mBGS, corresponding to El. 86.5 ± 0.5 mASL, as shown on Figure 4 in Appendix A. The proposed founding depth for the foundation will be 1 to 2 m lower. The anticipated foundation level could be at El. 85.0 ± 1.0 mASL,

Based on the borehole findings up to date, the subgrade conditions at or below the proposed founding levels generally consist of **hard silty clay and silty clay till and very dense sandy silt till and sandy silt and silt deposits**. Therefore, the Site is generally considered suitable for the construction of proposed re-development from the geotechnical viewpoint.

Considering the proposed depth of underground parking excavation and subsurface and groundwater conditions observed at borehole locations, the following options are recommended for the design of the foundations to support the proposed 30-storey building at Site.

Option-1: Conventional Strip/Spread Foundations

Based on the BIG's borehole findings, the proposed buildings can be supported by conventional strip and spread/extended footings (up to 3 m in size) on competent undisturbed very dense/hard silty/clayey deposits and glacial tills, **provided that groundwater level is lowered to at least 1 m below the proposed foundation levels and all localized weaker subsoils discovered in advance of foundation construction shall be removed under the footprints of new foundations**. For preliminary design purpose, the recommended Serviceability Limit States (SLS) geotechnical reactions and factored Ultimate Limit States (0.5 x ULS) geotechnical resistances) for conventional strip and spread footing are summarized in the following table, subject to satisfactory settlement checks during detailed design and geotechnical field evaluations of weaker subsoils discovered and adequate groundwater control during construction.

Recommended Bearing Capacities and Anticipated Founding Elevation for Strip/Spread Footings

Borehole No.	SLS Geotechnical Reaction* (kPa)	Factored Ultimate Geotechnical Resistance 0.5 X ULS (kPa)	Highest Possible Founding Elevation (mASL)
BH/MW1	350 600	525 900	86.7 85.2
BH/MW2	350 600	525 900	86.0 84.9
BH/MW3	350 600	525 900	86.9 85.4
BH/MW4	500 1000	750 1500	86.4 81.8
BH/MW5	350 600	525 900	85.5 82.7

It should be noted that the recommended bearing values should be further reviewed once the proposed foundation general arrangement becomes available. Higher bearing capacities may be available subject to additional geotechnical investigation and satisfactory settlement analysis.

Total and differential settlements of footings founded on the competent native soils and designed as outlined above should not exceed 25 and 19 mm respectively, provided that the founding subgrade is not loosened or softened by construction activities or prolonged exposure to the weather.

The minimum footing sizes, footing thickness, excavations, and other footing requirements should be designed in accordance with the latest edition of the Ontario Building Code. However, a minimum width of 600 mm is recommended for the strip footings.

There is no official rule governing the footing depths for a fully enclosed unheated parking garage. For P4 Levels of "unheated" underground parking structures, experience in a number of structures has shown that the perimeter strip footings and the interior isolated columns footings founded at depths/covers of 0.8 and 0.9 m below the slab-on-grade, respectively, have performed satisfactorily. However, for P1-Level of "unheated" underground parking structure, we recommend installing the perimeter strip footings, and interior isolated columns footings at a minimum depth of 1.2 m below the slab-on-grade levels. Similarly, footings adjacent to fresh air ducts, entrance of the garage and any other areas which may be exposed to the seasonal freezing conditions, a minimum earth cover of 1.2 m or its equivalent thermal insulation should be provided. In addition, a nominal 50 mm thick Styrofoam insulation should be provided under the floor slab within the close proximity to the fresh air ducts.

Option-2: Tanked Raft Foundation

Alternatively, the proposed building may also be supported by continuous "tanked" raft foundation. For preliminary design purpose, the recommended Serviceability Limit States (SLS) geotechnical reactions and factored Ultimate Limit States (0.5 x ULS) geotechnical resistances for the raft foundation are summarized in the following table, subject to satisfactory settlement checks during detailed design, geotechnical field evaluations and adequate groundwater control during construction.

Recommended Bearing Capacities and Anticipated Founding Depth – Raft Foundations

Geotechnical Reaction at SLS (kPa)	0.5 x ULS Geotechnical Resistance (kPa)	Highest Possible Founding Elevation (mASL)
400	600	85.5
600	900	82.2

Assuming a raft foundation (up to 30 m in width and length) resting on undisturbed very dense/hard silty/clayey soils and glacial tills, with effective dewatering during construction, and designed for the above bearing resistance values, total and differential settlements are expected within 50 mm and 20 mm respectively. However, once more design details (such as estimated building loadings, foundation sizes, etc.) become available, a detailed settlement analysis must be carried out to validate/optimize the proposed foundation design.

The advantage of raft foundation is that the wall and column loads are distributed over the entire area of the raft slab, thereby greatly reducing the bearing pressures and the differential settlements. The thickness and reinforcement of the raft foundation should be designed by a structural engineer to account for differential settlements.

An allowance of approximately 600 mm for piping services should be incorporated in the design between the top of the raft slab and the bottom of the lowest parking floor slab to allow for the installation and maintenance of drainpipes, sewers and any other underground services. The service space should be filled with clear stone after laying the underfloor service pipes and utilities.

Waterproofing of the perimeter basement wall and slab is recommended. The walls may be dampproofed above the raft foundation level to at least 0.6 m below the proposed ground surface level. The manufacturer of the selected product should be consulted for application details for damp proofing.

Option-3: Cast-In-Place Augured Caissons

Alternatively, the proposed foundation may also be supported by deep foundation such as Augured Cast-in-place Concrete Caissons. Other deep foundation options, such as Continuous Flight Auger Piles, can also be considered, and corresponding recommendations can be provided as necessary during detailed design stage upon completion of Pile-Load Tests by the Design-Build Contractor and reviewed by Geotechnical Engineer.

Squat or Conventional Cast-In-Place augured caisson system supported on weathered or sound shale bedrock, may be used to support the proposed 30-storey building, or to provide deep foundation support or additional support for possible piled-raft foundation option. For preliminary design purpose, the recommended end bearing resistances and corresponding founding elevations of the caisson foundations are summarized in the table below.

Recommended Bearing Resistance Values for Caissons

Borehole No.	Surface Elevation (mASL)	Minimum Founding Depth (mBGS)	Minimum Founding Elevation (mASL)	Factored Bearing Resistance at ULS* (0.4 X ULS) (kPa)	Anticipated Founding Material
BH/MW2	101.67	20.7 22.2	80.97 79.47	1500 6000	Weathered Bedrock Socketed in Unweatered/Sound Bedrock
BH/MW3	100.64	19.8	80.84	1500	Weathered Bedrock
BH/MW4	101.71	19.8 21.3	81.91 80.41	1500 6000	Weathered Bedrock Socketed in Unweathered/Sound Bedrock
BH/MW5	101.01	19.7 21.2	81.31 79.81	1500 6000	Weathered Bedrock Socketed in Unweathered/Sound Bedrock

** ULS denotes Ultimate Limit States*

All conventional caissons must have a minimum length to diameter ratio of 3:1. For conventional caissons founded on sound bedrock, a minimum socket depth of three (3) caisson diameter in unweathered/sound bedrock is recommended. Higher values may be considered for conventional caissons embedded a minimum socket depth of three (3) caisson diameter in excellent quality sound bedrock below El. 78 m during detailed design.

For Shale bedrock, the foundation design is not governed by resistance at Serviceability Limit State (SLS) since the stress required to produce 25 mm of deformation will generally be much larger than the factored resistance at ULS, provided that the founding subgrade is not loosened or softened by construction activities or prolonged exposure to the weather.

It should be noted that sandy/silty seams can be anticipated within the overburden soils. This will make the installation of caissons difficult. Therefore, the caisson may require temporary liners for installation, particularly in areas where sandy/silty soils are present, to help prevent the sandy or otherwise collapsible soils from caving and to help control water seepage into the caissons. The liners must be tightly sealed into the compact to dense/very stiff to hard glacial till deposits or bedrock to help control water seepage from the wet sandy/silty seams and layers. It is expected that any water seepage occurring at this time within the caissons can be removed by conventional pumping methods. However, if the water seepage is greater than anticipated and cannot be controlled by pumping, then consideration should be given to carrying the foundation loads by shaft friction in the bedrock. In this case, the concrete must be placed in the water filled caissons by tremie methods.

Considering that auger grinding was encountered during the borehole drilling, cobbles and boulders should be expected within the glacial till deposits that may pose significant obstructions during installation of some caissons. Appropriate equipment and procedures will be required to penetrate such obstructions (cobbles and boulders) that are encountered during caisson installation.

All caisson bases must be proven to be clean. Caissons inspection and installation should be carried out under full time supervision by experienced personnel from BIG, to ensure that the design founding depth/ elevation and competent bearing surface has been established at each caisson unit.

All works should be carried out in accordance with the Occupational Health and Safety Act (Ontario Regulation 213/91) and with local regulations. The caisson liners (if any) should be carefully withdrawn after the inspection and approval of the base material and while pouring the concrete.

The interface of the weathered shale and sound bedrock would be difficult to determine in the caisson holes, and therefore, the actual founding elevations would have to be determined by field inspection during the installations of the caissons. As noted above, hard layers were encountered in the shale bedrock. Should these hard layers be encountered, they must be penetrated in order to confirm the presence of shale bedrock below the hard layers.

Concrete being placed into the caissons should have a slump of about 150 mm since it will not be practical to vibrate the concrete at depth. However, if the liners are being vibrated in and out of the hole, then normal slump concrete can probably be used. Once the method of construction is established the concrete mix must be reviewed by an experienced geotechnical consultant.

For caissons founded on bedrock, the group effect can be ignored, or group reduction factor is 1.

Consideration should be given to including O-cell testing in a selected caisson if a higher bearing resistance is required.

Foundations – General

All footings exposed to seasonal freezing conditions must have at least 1.2 meters of soil cover for frost protection.

For both the conventional foundation and raft options, positive dewatering of the site to at least 1.0 m below foundation elevation will be necessary prior to excavation for the duration of the below grade construction works in order to preserve the structural integrity of the natural soils. If dewatering of the subsurface is not carried out prior to excavation, the native soils will become disturbed and recommended bearing resistances will not be reliable. In no case should the footing be placed on dilated or disturbed subgrade. It will be necessary to counteract the potential uplift pressure on the underside of a raft due to hydrostatic forces. As such, positive dewatering efforts during construction must be maintained for the duration of time until the factored structural dead load exceeds the factored uplift forces.

Prior to the placement of concrete, all footing subgrades must be inspected and approved by a Geotechnical Engineer from BIG to ensure that the founding soils are similar to those identified in the boreholes and are capable of supporting the design bearing resistances.

Following the approval, footing subgrades should be protected with a 50 mm thick concrete mud-slab if water seepage is encountered and/or the excavation is to remain open for more than a day. If construction proceeds during freezing weather conditions, adequate temporary frost protection measures for footing/foundation subgrades and concrete must be provided.

Where it is necessary to place footings at different levels, the upper footing must be founded below an imaginary 10 horizontal to 7 vertical lines drawn up from the base of the lower footing. The lower footing must be installed first to help minimize the risk of undermining the upper footing.

6.3 Floor Slab Construction

Considering the Foundation Option 1 above (Conventional Strip/Spread Foundations), for 4 levels of underground parking, conventional slab-on-grade floor slab on very dense/hard silty/clayey soils and glacial tills can be adopted for the proposed residential building, provided adequate dewatering is in place, if and where required. The thickness of the concrete slab-on-grade and the reinforcement (if any) should be designed by a structural engineer.

A modulus of subgrade reaction of 15,000 kN/m^3 may be used for the design and construction of proposed P4 level of underground parking floor slab, provided that the construction is in accordance with the recommendations provided herein. The floor slab should not be tied to any load-bearing walls or columns unless they have been designed accordingly. Contraction and expansion joints should be provided for the slabs as required by the structural engineer.

Floor bedding consisting of at least 200 mm of Granular A (OPSS 1010) or its approved equivalent, is recommended as a moisture barrier under the floor slab. A polyethylene vapour barrier or equivalent may be placed at the surface of the granular bedding if a moisture sensitive finish is to be placed on the floor. The bedding should be compacted to at least 98% of SPMDD. Alternatively, if the subgrade becomes wet during earthwork and stay wet prior to construction of the slab, the 200 mm thick floor bedding should consist of 100 mm thick HL8 Coarse Aggregate underlain by 100 mm thick OPSS Granular A.

The above recommendations should be further reviewed once the detailed design is available.

6.4 Permanent Perimeter and Underfloor Drainage

If the basement is designed as a “drained” structure, a permanent perimeter drainage system should be provided around the perimeter walls of the underground parking structure. Where adequate space is not available for open-cut excavation with slopes, then a properly designed vertical shoring system should be installed to support the sides of excavation. In this case, a permanent perimeter drainage system consisting of prefabricated continuous vertical blanket, Miradrain 6000 or its equivalent, should be used at and along the shoring location, just outside of the perimeter walls of the underground parking structure. Several collection pipes, installed through the perimeter walls, connect the prefabricated perimeter vertical drains with the solid collector pipe installed in a positive grade leading to a frost-free sump. The installation and connections of prefabricated perimeter vertical drains should be carried out as per the manufacturer’s specifications.

Waterproofing of the underground perimeter walls is recommended. The manufacturer of the selected product should be consulted for application details.

Considering the subsurface soils, groundwater measurements in the monitoring wells and moisture content profiles of recovered soil samples, underfloor drainage system will also be required. The underfloor drainage system should be kept separate from the perimeter drainage system. Conceptual designs of Permanent Perimeter & Under-floor Drainage System with both open cut and shoring are provided in Appendix F.

The anticipated volumes of water to be pumped during and post construction should be based on the findings of the Preliminary Hydrogeological Investigation Report.

A provision of additional groundwater control measures, consisting of underfloor sump pumps connected to an emergency power grid, should be installed below the lowest floor level of the parking for the consequence arising from a failure of the regular system.

It should be noted that upon completion of the detailed design of the below grade structure, the underfloor drainage system design shall be reviewed and finalized.

6.5 Frost Protection

The design frost penetration depth for the general Site area is 1.2 m as per OPSD 3090.101. Therefore, any structural foundation (perimeter and other footings) and buried underground utilities exposed to seasonal freezing conditions should be provided with frost protection comprising at least 1.2 m of earth cover or its equivalent thermal insulation.

During wintertime construction, exposed subgrades to support foundations must be temporarily protected against freezing by means of loose straw, tarpaulins, heating, etc.

6.6 Earthquake Consideration

In conformance to the criteria in Table 4.1.8.4.A, Part 4, Division B of the OBC, the project site may be classified as Site Class "D – Stiff Soil". The four values of the Spectral response acceleration Sa (T) for different periods and the Peak Ground Acceleration (PGA) can be obtained from Table C-2 in Appendix C, Division B of the OBC 2012. The design values of Fa and Fv for the project site should be calculated in accordance with Table 4.1.8.4 B and C.

Seismic survey is recommended as part of further additional geotechnical investigation to confirm or upgrade the seismic site classification.

6.7 Reuse of On-Site Soils

Based on the conditions encountered in the boreholes, portions of on-site excavated soils which do not contain organics and deleterious materials can be re-used for backfilling as an Engineered Fill. However, depending upon the weather condition, the excavated soils may require some reconditioning prior to reuse, i.e., maintain the moisture contents of these materials within ±2% of SPMDD values (i.e., Proctor values) to obtain the required compaction. For reuse as an Engineered Fill for foundation support, uniform material must be used. Significant variations in fill type will require thinner lifts, more compaction effort and more field and laboratory testing. Less stringent requirements may be considered for fills placement below slab-on-grade, above footing levels, and pavement areas. Special care should be exercised to ensure compaction around columns and adjacent to foundation walls.

Unsuitable material such as organic rich pockets, frozen soils, cobbles, boulders, remnants of demolished structures, etc., should be wasted. Ideally, dissimilar materials should be stockpiled separately during excavation.

Portion of the existing soils are considered as frost susceptible (e.g., fine sand, silt) and should not be reused where a volume change in the presence of freezing conditions would have an adverse effect on the serviceability of the proposed infrastructure.

6.8 Excavation and Temporary Groundwater Control

It is expected that the excavations through the existing fills and native sandy/silty/clayey soil deposits and glacial tills for the proposed underground parking structures, can be handled by conventional mechanical excavation equipment. Allowance should be made for potential cobbles and boulders within the glacial tills, and remnants of demolished buildings during excavation.

All excavations must be carried out in accordance with the Occupational Health and Safety Act (OHSA) and Regulation 213/1991 for Construction Projects to ensure the protection of workers from on-Site contaminants of concerned impacted soil and groundwater. Under the Act, the soils to be excavated can be classified as follows:

Existing Fill	Type 3	When submerged/saturated	Type 4
Sandy/Silty Soils	Type 3	When submerged/saturated	Type 4
Clayey Soils	Type 2	When submerged/saturated	Type 3
Glacial Tills	Type 2	When submerged/saturated	Type 3

For Type 3 soils, a bank slope of 1H:1V is required. For Type 2 soils, a 1.2 m high vertical cut at the bottom of excavation may generally be used. For Type 4 soils, temporary excavation side slopes must be at a maximum inclination of 3H:1V from the base of the excavation.

Near the ground surface, occasional 3H:1V slopes may be required due to disturbed surficial soils. If an excavation contains more than one soil types, the excavation slope geometry shall be governed by the highest soil type number.

Stockpiles of excavated materials should be kept at a horizontal distance of at least the excavation depth away from the edge of any excavation to avoid the slope instability, subject to confirmation by the geotechnical engineer. Care should also be taken to avoid overloading of any underground services/structures by stockpiles.

In areas where open excavation slopes cannot be maintained due to the close proximity of the existing structures on the adjacent properties (e.g., buildings, roads, etc.), the excavation within the overburden should be supported by using a shoring system (e.g., tight wooden bracing, sheet pile, trench box, strutted soldier pile & lagging wall etc.), designed by a shoring consultant. Further, the depths of shoring walls should be extended sufficiently below the base of the excavation to ensure that toe resistance is maintained when the soil is excavated.

Perched water may also be encountered within the earth fills, or sandy/silty seams within the glacial tills above the groundwater level. The amount of free water from that source is anticipated to be minor and the water accumulated in the excavation can readily be handled by using temporary filtered sump and pump.

Stabilized groundwater level measurements were recorded in the monitoring wells installed at Site, at depths ranging from 7.9 to 9.3 mBGS, corresponding to Elevations between 92.63 and 92.72 mASL. Therefore, groundwater seepage could be expected when excavations extend below an approximate elevation of 92.72 mASL.

Depending upon the prevailing groundwater level at the time of construction, some form of positive (pro-active) groundwater control in addition to pumping from sumps, may be required. The groundwater level should be lowered to at least 1 m below the excavation base to maintain the stability of the base and side slopes of the trench excavations in these areas. OPSS.PROV 517 should be followed when groundwater control is required for this project. A significant ground settlement is not expected from the anticipated groundwater discharge. Detailed discussions on groundwater and construction dewatering including permit requirements are provide in the Preliminary Hydrogeological Investigation report.

Consideration should be given to carrying out the construction during the drier seasons of the year to reduce the need for dewatering and disturbances to the founding soils caused by the excavation below prevailing groundwater table. Dewatering requirements will be governed by the time of year the construction is performed. It is the responsibility of the contractor to propose a suitable dewatering system based on the time of construction and the groundwater levels. The method used should not undermine adjacent structures.

Seasonal variations in the water table should be anticipated, with higher levels occurring during wet weather conditions (spring thaw and late fall) and lower levels occurring during dry weather conditions.

6.9 Shoring Considerations

In areas where an open excavation slope cannot be maintained, the excavation within the overburden should be supported by using a shoring system. Shoring system consisting of soldier piles and timber laggings can be used, where slight movement in the ground surface can be tolerated, i.e., where non-sensitive structures exist. Alternatively, consideration may be given to the installation of continuous caisson walls socketed sufficiently below the bottom of the excavation, to minimize the water seepage during excavation or where settlement sensitive structures are located at the close proximity of the proposed excavation.

The shoring system should be designed by an experienced shoring consultant in accordance with the guidelines provided in the latest edition of the Canadian Foundation Engineering Manual (CFEM). Similarly, the construction of the shoring system should also be carried out by a contractor, experienced in this type of construction. The soil parameters estimated to be applicable for this design are summarized as follows:

1) Earth Pressure Coefficients
   (a) Where movement must be minimal K = 0.5
   (b) Where minor movement (0.002H) can be tolerated K = 0.33
   (c) Passive earth pressure for soldier piles (unfactored) K_p = 3.0
2) For stability check, the soil parameters estimated to be applicable for this design are summarized as follows. The surcharge is to be determined by the shoring contractor.

Earth Pressure Design Soil Parameters:

Soil Type	Unit Weight γ (kN/m3)	Effective Friction Angle ϕ (Degrees)	Cohesion c (kPa)
Existing Fill	19	28	0
Native Sandy/Silty Soils	21	33	0
Native Clayey Soils	20.5	30	5
Glacial Tills (Sandy/Silty)	21	34	3
Glacial Tills (Clayey)	21	33	5
Silty Clay/Shale Complex	22	34	5

The shoring system should be designed for a minimum factor of safety of F = 2 for soils and 3 for rocks. The overall factor of safety of the anchored block of soil must be considered. Minimum spacing and the depths of the soil anchors should be as recommended in the CFEM.

It is recommended that close monitoring of vertical and lateral movements of the shoring system should be carried out at the Site - prior to, during and after the excavation. If movements at the top of the piles are more than 12 mm (0.5 in), extra bracing may be required. In this regard, monitoring by inclinometers and by survey on targets should be instituted to ensure that the contractor maintains movements within design limit.

Subject to hydrogeological considerations, construction dewatering, or long-term underfloor drainage system may adversely impact nearby structures.

It would be best to provide a sufficient depth of shoring system cut-off below the excavation level to avoid piping, uplift and basal heave.

6.10 Lateral Earth Pressures

The lateral earth and hydrostatic pressures acting on retaining structures (e.g., perimeter basement walls, cantilever walls, etc.) may be calculated from the following expression:

$$P = K[\gamma(H-h_w) + \gamma' h_w + q] + \gamma_w h_w$$

Where,	P	= Lateral earth pressure at depth H (m)	kPa
	K	= Lateral earth pressure coefficient	
	γ	= Bulk unit weight of the soil	kN/m^3
	γ'	= Submerged unit weight of soil	kN/m^3
	γ_w	= Unit weight of water	kN/m^3

H	= Depth of the wall below the outer finish grade	m
hw	= Depth of the wall below the groundwater level	m
q	= Equivalent value of all surcharge loads on the ground surface	kPa

If the development of hydrostatic pressure behind the wall is eliminated with the installation of effective drainage system, the above expression can be simplified as follows:

$$P = K (\gamma H + q)$$

Surcharge and point loads at the ground surface including loads of heavy construction equipment should also be considered in the structural design.

6.11 Underground Services

It is considered that the sewer depths will not exceed 4.0 m below grades. Trench excavation should be carried out in accordance with the most recent version of the Ontario Occupational Health and Safety Act & Regulations for Construction Projects. The boreholes show that the trenches, generally, will be dug through existing fills and native sandy/silty/clayey soil deposits and glacial tills. Normal conventional excavation equipment will be suitable for excavating trenches in the fill and native soil deposits.

Within the native deposits, above the groundwater table, the side-slopes of excavations are expected to be temporarily stable at 1V:1H. Flatter slopes will be required for the soils located below groundwater table, if encountered as noted on Section 6.8.

In areas where an open excavation slope cannot be maintained, the excavation within the overburden should be supported using a temporary shoring system (e.g., tight wooden bracing, etc.), designed by a shoring consultant. Excavations can also be carried out at steeper side slopes by using trench box, designed in accordance with the Safety Regulations, for the protection of the workers.

Groundwater seepage into the excavations may occur from perched groundwater within fills, native sandy/silty soils and sandy/silty seams within the glacial tills, or surface water flow. Dewatering should be achieved by properly filtered sumps and pumps. The groundwater level in the trench should be kept below the bottom of the excavation by dewatering. Ideally, to prevent disturbance of the soil at the bedding level, the groundwater table must be lowered to at least 0.6 m below the base of the trench. In no case should the pipes be placed on disturbed subsoil.

The boreholes show, the anticipated subgrade in their undisturbed state, will generally comprise of loose to compact fill materials and native loose to compact sand or firm to stiff silty clay deposits, which should provide adequate support for the pipes, provided the exposed subgrade at the base of the trenches is further assessed and approved by qualified geotechnical personnel from BIG during construction.

Pipe bedding should be in accordance with the pipe manufacture recommendations, appropriate local municipality requirements and standards (e.g., OPS). However, as a guideline, normal Class ‘B’ Type bedding (OPSD-802) may be considered. The thickness of the bedding material, however, may have to be increased depending on the pipe diameter or if wet or weak subgrade conditions are encountered. Subject to assessment by the geotechnical engineer on Site, the bedding used to support the pipes in weak soils (if any) may need to be wrapped by a geotextile (e.g., Terrafix 270R or equivalent). In general, a minimum of 150 mm thick of OPSS Granular A bedding is recommended for pipes 450 mm diameter or less; for large diameter pipes, the thickness of the bedding should be increased to 200 mm. The bedding and cover materials should be compacted to a minimum of 95% of their SPMDD to provide support and protection to the pipes.

The thickness of the bedding material, however, may have to be increased depending on the pipe diameter or if wet or weak subgrade conditions are encountered. Similarly, where moist to wet conditions are encountered, the use of 'clear stone' bedding (such as 19 mm clear stone, OPSS 1004) may be considered, only in conjunction with a suitable geotextile filter (e.g., Terrafix 270R or equivalent). Without proper filtering, there may be loss of fines from native soils and/or trench backfill materials into or through the bedding materials. This loss of fines could result in loss of support to the pipes and possible surface settlements.

Portions of the excavated on-site soils which do not contain excessive organics, debris and deleterious materials can be reused for backfill in service trenches subject to the conditions noted in Sections 6.1 and 6.7.

The fills should be placed in thin lifts not exceeding 200 mm thick loose lifts, within ±2 % of its optimum moisture content, and thoroughly compacted with suitable heavy rollers to at least 95% of SPMDD, before placing the next lift. This value should be increased to at least 98 % within 0.8 m of final subgrade of trench for the road pavement construction.

6.12 Pavement Construction

Pavement design and pavement thicknesses are highly dependent on the subgrade conditions. The pavement subgrade should, therefore, be adequately prepared to receive the granular bases for the pavement construction noted in Section 6.1.

Prior to the placement of granular bases, the exposed subgrade for pavement construction should be thoroughly and consistently compacted followed by a proof-rolled operation in the presence of the qualified geotechnical personnel from BIG. Any wet/soft areas of subgrade, revealed by this process, should be sub-excavated and replaced with an approved on-site or imported fill material compatible to the existing subgrade condition. Similarly, if the grade beneath the proposed pavement areas is required to be raised, approved on-site or imported fill can be used.

All new fills should be placed in a maximum of 200 mm loose lifts, within ±2 % of its optimum moisture content, and each lift should be compacted by suitable heavy equipment to minimum 95% of SPMDD before placing the next lift. The uppermost 600 mm of the pavement subgrade should be compacted to a minimum 98% of SPMDD.

Construction traffic over exposed subgrade should be minimized, and temporary construction hauling routes should be established. If these routes coincide with future paved areas, hauling roads should be adequately reinforced with increased thickness of granular bases and/or geo-grids, etc. to reduce the disturbance to the subgrade soils.

Considering the proposed pavement usage, frost susceptibility and assuming adequate drainage provided, the following minimum pavement structure thicknesses are recommended for the long-term satisfactory performance of the pavement:

Recommended Minimum Pavement Structure Thickness:

Particulars	Heavy Duty Driveway (mm)	Standard Duty Driveway (mm)
Asphaltic Concrete: OPSS HL3	40	50
Asphaltic Concrete: OPSS HL8	70	50
Base Course - OPSS Granular A or equivalent	150	150
Sub-base Course - OPSS Granular B or equivalent	350	250

The pavement thickness should also conform to the local regulations and standards inclusive of City of Burlington. Adjustments to the above recommended thicknesses may also be required to match the existing pavement structure if full reconstruction is not being considered.

The granular base and subbase materials should conform to the OPSS 1010 and should be compacted to 98% of the ASTM D698 SPMDD within ±2% of the optimum moisture content.

Hot mix asphalt concrete should conform to OPSS 1150 and OPSS 310 and be placed and compacted to at least 92 to 96.5 % of the Marshall Maximum Relative Density (MMRD). It is recommended that the asphalt mix design be reviewed by BIG prior to the start of the paving.

The pavement thickness considers that construction will be carried out during the drier time of the year and that the subgrade is competent. If the subgrade becomes excessively wet or rutted during construction activities, additional sub-base material may be required. The need for additional subbase material is best determined during construction.

7 Construction Monitoring

Qualified Geotechnical personnel should monitor the foundation excavation, subgrade inspection, in-situ density tests and material testing services in all stages of the proposed development, to ensure that the materials and conditions comply with this geotechnical report and project requirements. Should the condition that encountered vary from those described in this report, our office should be informed immediately so that the proper measures are undertaken. The on-Site review of the condition of the foundation soil is an integral part of the geotechnical design function and is required by Section 4.2.2.2 of the Ontario Building Code.

All backfilling should be supervised to ensure that proper materials are used, and that adequate compaction is achieved. Strict quality control guidelines should be followed during the placement of fill materials.

8 Closure

The subsoil information and recommendations contained in this report were prepared solely for the purpose of use at the specific project as described in this report and should not be used to any other project or site location. The information contained in this report is for the sole benefit of the Client and his/her consultants. *To properly understand the contents of the report, reference must be made to the whole of the report. BIG cannot be held responsible for the use of portions of the report without reference to the entire report.*

We recommend that BIG be retained to review the recommendations for this specific applicability, once the details of the proposed development are finalized and prior to the final design stage of the project.

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

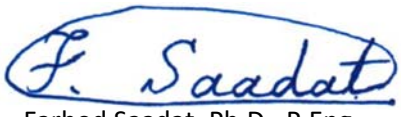
Yours truly,

B.I.G. Consulting Inc.



Andy Chen, M.Eng., P.Eng.
Senior Geotechnical Engineer





Farbod Saadat, Ph.D., P.Eng.
Principal Geotechnical Engineer

9 Report Limitations

The conclusions and recommendations given in this report are based on information determined at the test hole (borehole, test pit, probe hole, etc.) locations. The information contained herein in no way reflects the environmental 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. It is a recommended practice that the Geotechnical Engineer be retained during the construction to confirm that the subsurface conditions across the site do not deviate materially from those encountered in the test holes.

The design recommendations and opinions 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. Since all details of the design may not be known, we recommend that we be retained during the final design stage to verify that the design is consistent with our recommendations, and that assumptions made in our analysis are valid.

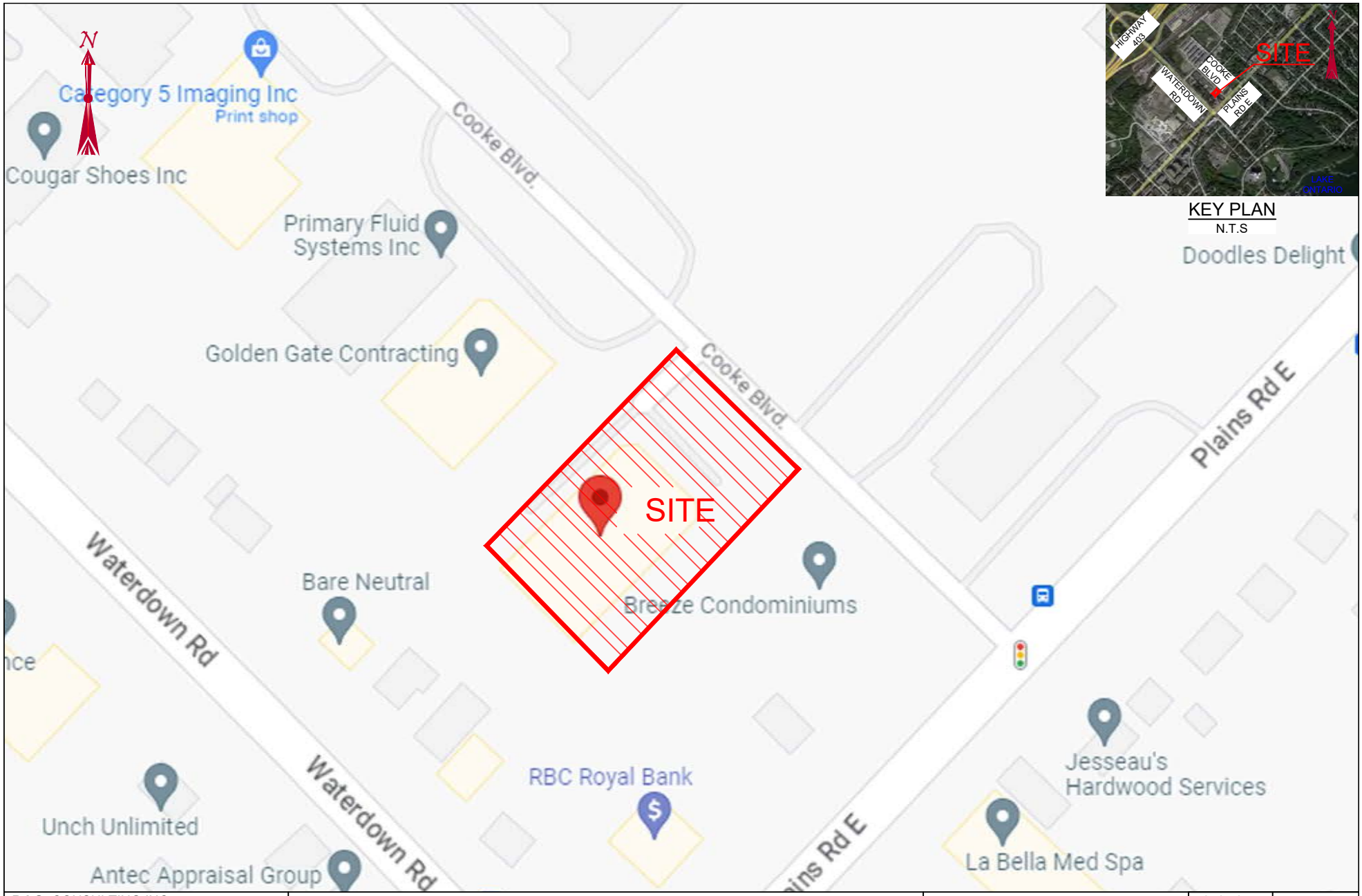
The comments made in this report relating to potential construction problems and possible methods of construction are intended only for the guidance to the designer. The number of test holes may not be sufficient to determine all the factors that may affect construction methods and costs. The anticipated construction conditions are also discussed, but only to the extent that they may influence design decisions. Construction methods discussed, however, express BIG's opinion only and are not intended to direct the contractors on how to carry out the construction. Contractors should also be aware that the data and their interpretation presented in this report may not be sufficient to assess all the factors that may influence the construction. For example, the thickness of surficial topsoil or fill layers may vary markedly and unpredictably at the site. The contractors bidding on this project or undertaking the construction should, therefore, make their own interpretation of the factual information presented in this report and draw their own conclusions as to how the subsurface conditions may affect their work. This work has been undertaken in accordance with normally accepted geotechnical engineering practices. No other warranty is expressed or implied.

The report is prepared on the condition that the design will be in accordance with all applicable standards and codes, regulations of authorities having jurisdiction, and good engineering practice.

The benchmark and elevations mentioned in this report were obtained strictly for use by this office in the geotechnical design of the project. They should not be used by any other party for any other purpose.

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. BIG accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.

Appendix A – Figures



KEY PLAN
N.T.S

B.I.G. CONSULTING INC.

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12-5500 Tomken Rd.
Mississauga, ON L4W 2Z4
Canada

bigconsultinginc.com

LEGEND
SITE BOUNDARY

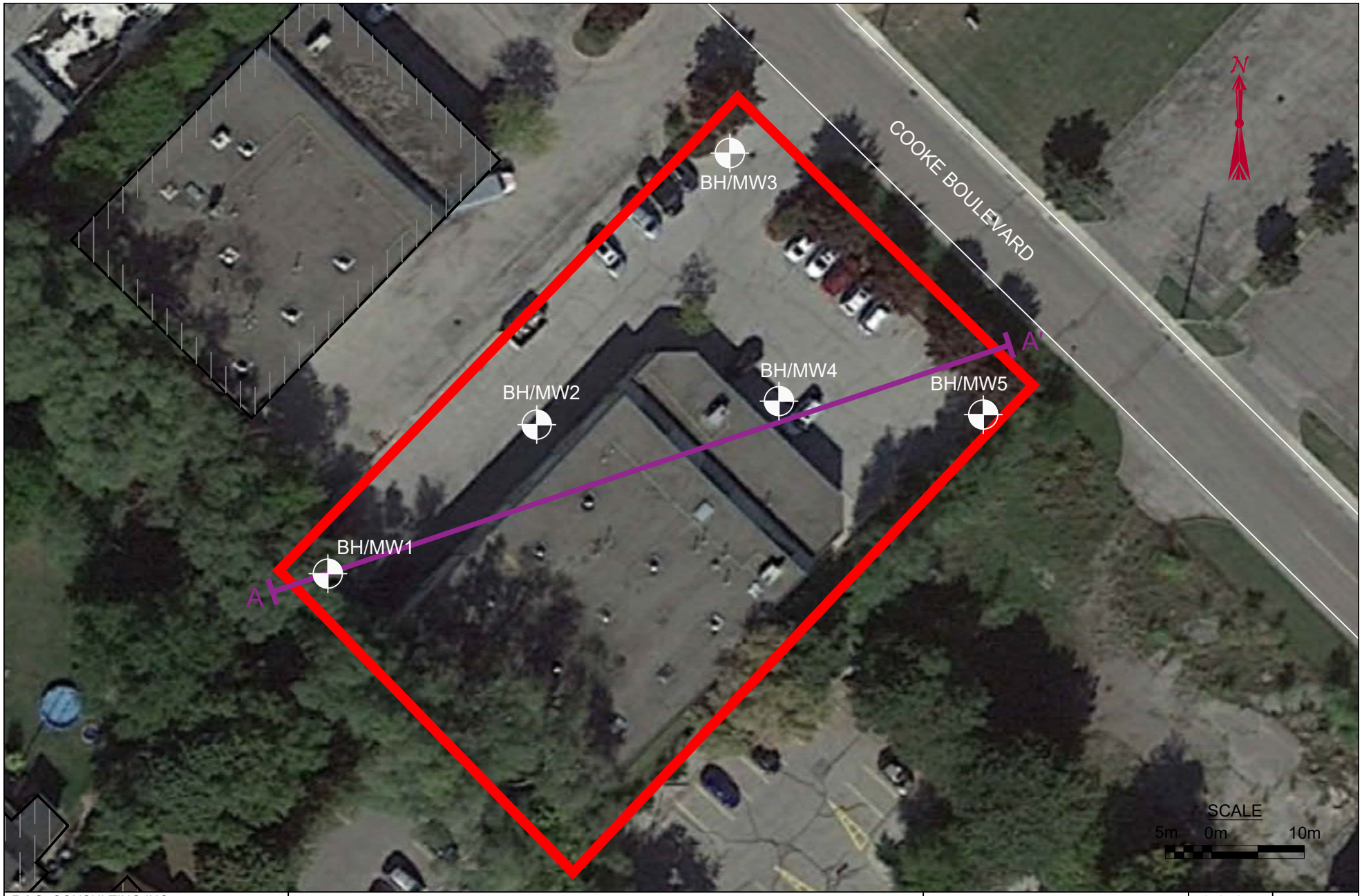
SCALE
10m 10 20 30 40
0m 50m

IMAGERY OBTAINED FROM GOOGLE MAP, DATE 2023

TITLE AND LOCATION

SITE LOCATION PLAN
PRELIMINARY GEOTECHNICAL INVESTIGATION
1026 COOKE BOULEVARD, BURLINGTON, ONTARIO

PROJECT NO.	DWN.
BIGC-ENV-605A	E.P.
SCALE	CK.
AS NOTED	A.C.
DATE	FIG NO.
JULY 2023	1



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LEGEND

SITE BOUNDARY

LOCATION OF THE BOREHOLE / MONITORING WELL

A A' GEOLOGICAL CROSS SECTION

IMAGERY OBTAINED FROM GOOGLE EARTH, 2023

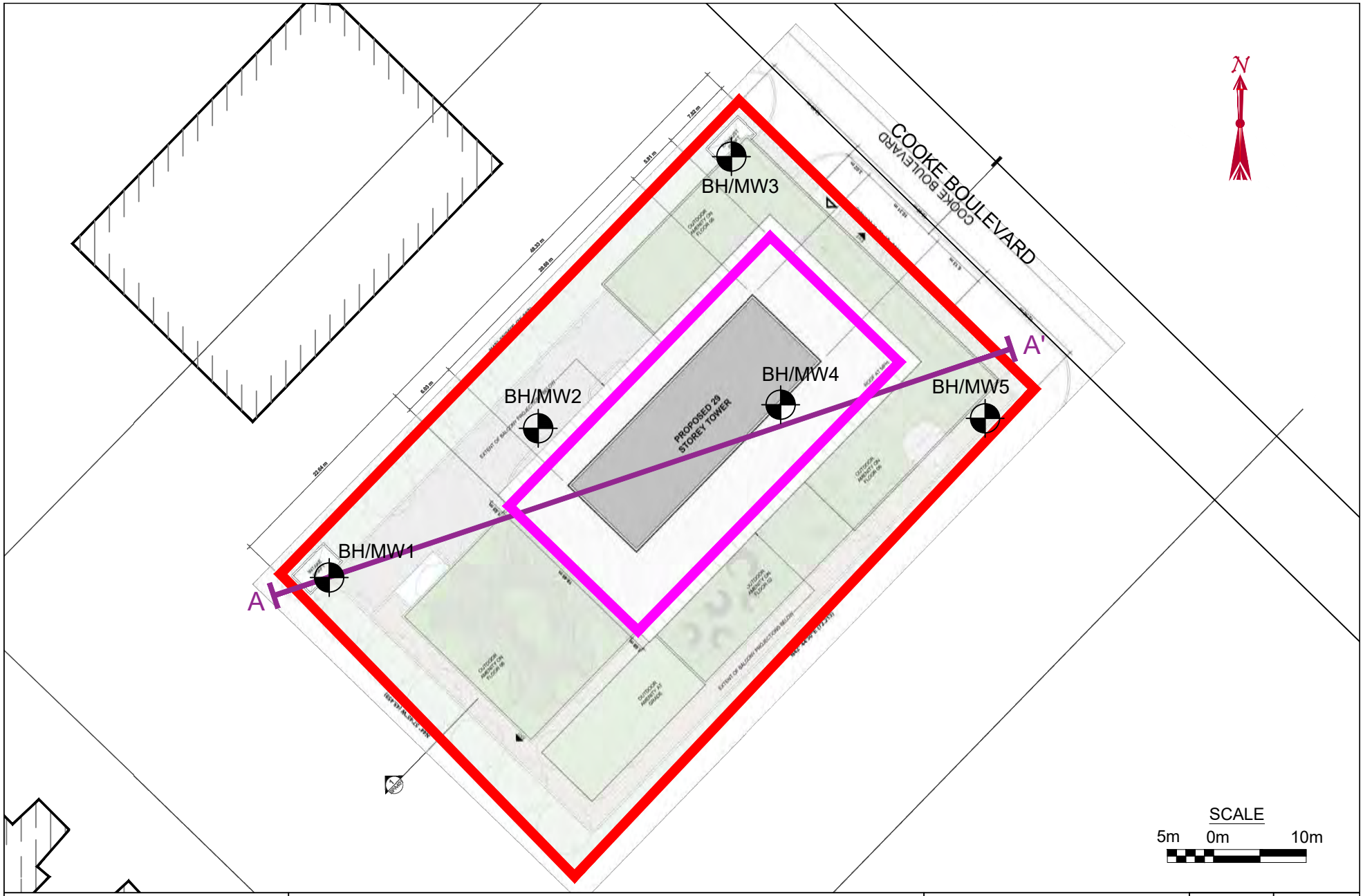
TITLE AND LOCATION

BOREHOLE/MONITORING WELL LOCATION PLAN

PRELIMINARY GEOTECHNICAL INVESTIGATION

1026 COOKE BOULEVARD, BURLINGTON, ONTARIO

PROJECT NO.	DWN.
BIGC-ENV-605A	E.P.
SCALE	CK.
AS NOTED	A.C.
DATE	FIG NO.
NOVEMBER 2023	2



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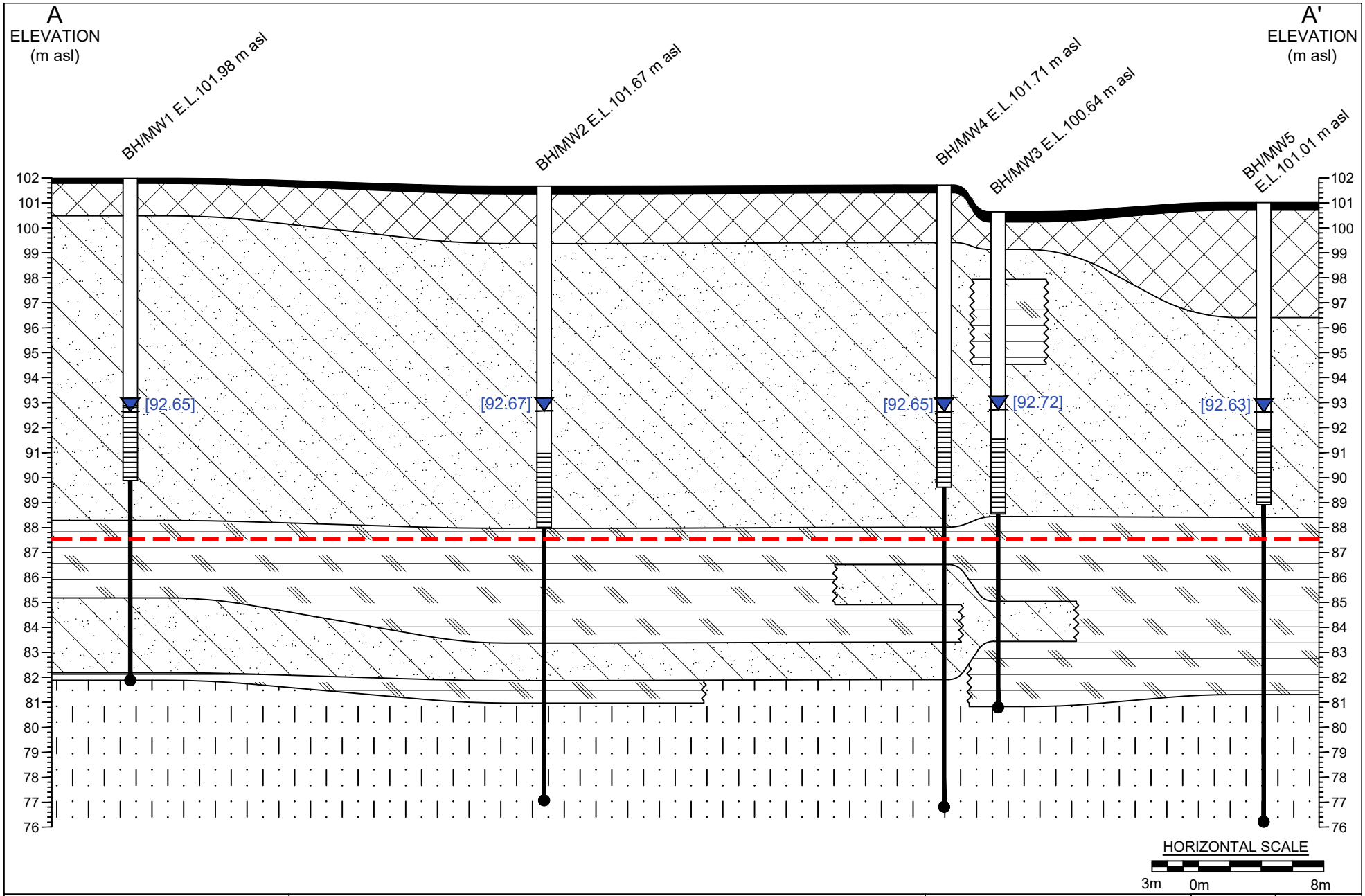
- SITE BOUNDARY
- FLOOR 29 BOUNDARY
- LOCATION OF THE BOREHOLE / MONITORING WELL
- A — A' GEOLOGICAL CROSS SECTION

SITE PLAN / ROOF PLAN OBTAINED FROM TURNER FLEISCHER ARCHITECTS INC. DRAWING NO. SPA005

TITLE AND LOCATION

BOREHOLE/MONITORING WELL LOCATION PLAN
PRELIMINARY GEOTECHNICAL INVESTIGATION
1026 COOKE BOULEVARD, BURLINGTON, ONTARIO

PROJECT NO.	DWN.
BIGC-ENV-605A	E.P.
SCALE	CK.
AS NOTED	A.C.
DATE	FIG NO.
NOVEMBER 2023	3



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LEGEND

- ASPHALT PAVEMENT
- FILL
- SAND / SILTY SAND / SANDY SILT / SANDY SILT TILL / SAND AND SILT TILL
- SILTY CLAY / SILTY CLAY TILL / SILT / SHALE COMPLEX
- BEDROCK
- WATER LEVEL
- [XX.XX] WATER LEVEL MEASUREMENT (m asl) (JULY 03, 2023)
- APPROXIMATE P4 FINISHED FLOOR SLAB (14.45 m BELOW GROUND SURFACE)

TITLE AND LOCATION

GEOLOGICAL CROSS SECTION A-A'
PRELIMINARY GEOTECHNICAL INVESTIGATION
1026 COOKE BOULEVARD,
BURLINGTON, ONTARIO

PROJECT NO.	DWN.
BIGC-ENV-605A	E.P.
SCALE	CK.
AS NOTED	A.C.
DATE	FIG NO.
NOVEMBER 2023	4

Appendix B – Notes to Record of Boreholes
Record of Boreholes

NOTES TO RECORD OF BOREHOLES

DRILLING METHOD		SAMPLE TYPE		LABORATORT DATA	
SSA	Solid Stem Auger	SS	Split Spoon	W	Water Content
HSA	Hollow Stem Auger	AS	Auger Flight Sample	W_p	Plastic Limit
WB	Wash Boring	TW	Thin Wall Open	W_l	Liquid Limit
RM	Rotary Mud Drilling	TP	Thin Wall Piston	γ	Natural Unit Weight (kN/m^3)
		WS	Washed Sample	C_u	Undrained Shear Strength (kPa)
		VT	Vane Test	PP	Pocket Penetrometer
		GS	Grab Sample	UC	Unconfined Compression
		RC	Rock Core	UU	Unconsolidated Undrained
		PH	Sample Advanced Hydraulically	CU	Consolidated Undrained
		PM	Sample Advanced Manually	CD	Consolidated Drained
		CC	Continuous Core	TOV	Total Organic Vapors

STANDARD PENETRATION TEST (SPT 'N'): The number of blows required to advance a standard 51 mm outer diameter split spoon sampler to penetrate 0.3 m distance into the undisturbed ground in a borehole driven by means of a 63.5 kg hammer falling freely from a distance of 0.76m.

DYNAMIC CONE PENETRATION TEST (DCPT): The number of blows required to advance a 51 mm diameter – 60 degree cone fitted to the end of the drill rods to penetrate 0.3 m distance into the undisturbed ground driven by 475 Joules driving energy per blow.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR RELATIVE DENSITY

CONSISTENCY: Cohesive soils are described on the basis of their undrained shear strength (Cu) or 'N' values as follows:

N (blows/0.3m)	0 - 2	2 - 4	4 - 8	8 - 15	15 - 30	>30
Consistency	VERY SOFT	SOFT	FIRM	STIFF	VERY STIFF	HARD
Cu (kPa)	0 - 12	12 - 25	25 - 50	50 - 100	100 - 200	>200

RELATIVE DENSITY: Cohesionless soils are described on the basis of their relative density as indicated by 'N' values as follows:

N (blows/0.3m)	0 - 4	4 - 10	10 - 30	30 - 50	>50
Relative Density	VERY LOOSE	LOOSE	COMPACT	DENSE	VERY DENSE

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

RECOVERY: Sum of the lengths of all recovered rock core pieces divided by the total length of the core run (expressed as a percent).

ROCK QUALITY DESIGNATION (RQD): Sum of the lengths of intact rock core pieces, 100 mm or more in lengths, divided by the total length of the core run (expressed as a percent). Classifications of a rock based on the RQD value are as follows:

RQD (%)	0 - 25	25 - 50	50 - 75	75 - 90	90 - 100
Quality	VERY POOR	POOR	FAIR	GOOD	EXCELLENT

JOINTING AND BEDDING:

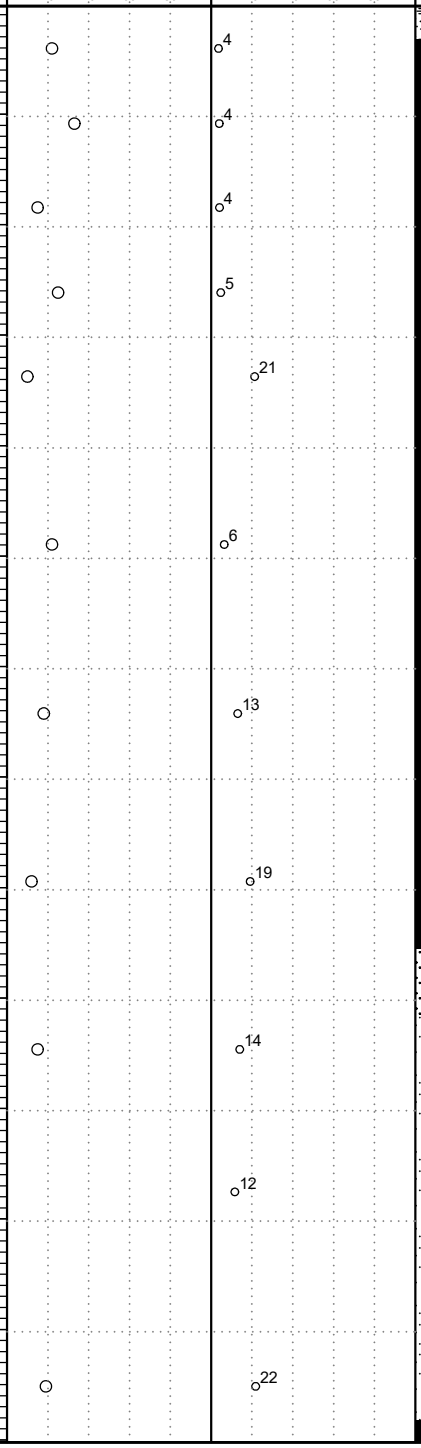
SPACING	50 Millimeters	50 - 300 Millimeters	0.3 – 1.0 Metres	1.0 – 3.0 Metres	> 3.0 Metres
JOINTING	VERY CLOSE	CLOSE	MOD. CLOSE	WIDE	VERY WIDE
BEDDING	VERY THIN	THIN	MEDIUM	THICK	VERY THICK

RECORD OF BOREHOLE No. BH/MW1

Project Number:	BIGC-ENV-605A	Drilling Location:	See BH Location Plan	Logged by:	AD
Project Client:	Halton Standard Condominium Corporation No. 416	Drilling Method:	190 mm Hollow Stem Augering	Compiled by:	KH
Project Name:	Geotechnical and Hydrogeological Investigations	Drilling Machine:	Track Mounted Drill	Reviewed by:	AC
Project Location:	1026 Cooke Blvd, Burlington, Ontario	Date Started:	30 Jun 23	Date Completed: 30 Jun 23	Revision No.: 0, 14/9/23

Geodetic Ground Surface Elevation: 101.98 m

DESCRIPTION	Sample Type	Sample Number	Recovery (%)	SPT 'N' Value/RQD%	DEPTH (m)	ELEVATION (m)
ASPHALT PAVEMENT: 80 mm asphalt over 150 mm granular base (101.75 / 0.2)						
FILL: sand, trace clay, trace silt, trace gravel, brown to redish brown, moist, compact to dense	SS	1	84	22		
	SS	2	79	33	1	101
(100.46)						
SAND: trace clay, trace silt, trace gravel, brown to reddish brown, moist to wet, compact	SS	3	75	15	2	100
	SS	4	84	25		
	SS	5	87	10	3	99
					4	98
	SS	6	87	22	5	97
					6	96
	SS	7	67	18		
(94.36)					7	95
SILTY SAND: trace clay, trace gravel, brown to reddish brown, wet, compact to very dense	SS	8	75	12	8	94
					9	93
	SS	9	75	15		
					10	92
- layers of gravel	SS	10	100	50/ 130 mm		
					11	91
					12	90
	SS	11	75	19		
					13	89



INSTRUMENTATION INSTALLATION

COMMENTS

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Mississauga, ON L4W 2Z4
Canada
T: 416-214-4880
F: 416-551-2633

Groundwater depth on completion of drilling: N/A m.

Groundwater depth observed on July 3, 2023 at a depth of: 9.33 m.

Borehole details as presented, do not constitute a thorough understanding of all potential conditions present and requires interpretative assistance from a qualified Geotechnical Engineer. Also, borehole information should be read in conjunction with the geotechnical report for which it was commisioned and the accompanying'Notes to Record of Boreholes'.

Scale: 1 : 68

Continued on Next Page

RECORD OF BOREHOLE No. BH/MW1

Project Number: **BIGC-ENV-605A** Drilling Location: **See BH Location Plan** Logged by: **AD**

Lithology Plot	DESCRIPTION	Sample Type	Sample Number	Recovery (%)	SPT 'N' Value/RQD%	DEPTH (m)	ELEVATION (m)	FIELD TESTING: Penetration Testing (○ SPT ● DCPT; MTO Vane* △ Intact ▲ Remould; Nilcon Vane* ◇ Intact ◆ Remould; * Undrained Shear Strength (kPa) 20 40 60 80)	LAB TESTING: ★ Rinse pH Values 2 4 6 8 10 12; Soil Vapour Reading △ parts per million (ppm) 100 200 300 400; ▲ Lower Explosive Limit (LEL) W_P W W_L Plastic Liquid 20 40 60 80	INSTRUMENTATION INSTALLATION	COMMENTS
	SILTY SAND: trace clay, trace gravel, brown to reddish brown, wet, compact to very dense										
	88.26										
	SILTY CLAY: trace sand, grey, wet, stiff to hard 13.7	SS	12	100	14	14	88	○	○ 28		
						15	87				
		SS	13	59	35			○	○ 33		
						16	86				
	85.22										
	SANDY SILT TILL: trace to some gravel, trace clay, reddish brown, moist, very dense - layers of silty clay till 16.8	SS	14	100	50/ 80 mm	17	85		○ 15		
						18	84				
		SS	15	89	50/ 130 mm				○ 14		
						19	83				
	82.17										
	SILTY CLAY / SHALE COMPLEX: reddish brown, moist, hard 19.8	SS	16	100	50/ 100 mm	20	82		○ 11		
	81.91 20.1										
	Spoon Refusal on Probable Bedrock										
	1. Borehole was open upon completion of drilling.										
	2. Groundwater level was measured at 9.33 m below ground surface on July 3, 2023.										

Borehole details as presented, do not constitute a thorough understanding of all potential conditions present and requires interpretative assistance from a qualified Geotechnical Engineer. Also, borehole information should be read in conjunction with the geotechnical report for which it was commisioned and the accompanying'Notes to Record of Boreholes'.

Scale: 1 : 68

RECORD OF BOREHOLE No. BH/MW2

Project Number:	BIGC-ENV-605A	Drilling Location:	See BH Location Plan	Logged by:	CE
Project Client:	Halton Standard Condominium Corporation No. 416	Drilling Method:	190 / 63.5 mm Hollow Stem Augering / HQ Coring	Compiled by:	KH
Project Name:	Geotechnical and Hydrogeological Investigations	Drilling Machine:	Track Mounted Drill	Reviewed by:	AC
Project Location:	1026 Cooke Blvd, Burlington, Ontario	Date Started:	26 Jun 23	Date Completed: 26 Jun 23	Revision No.: 0, 14/9/23

Geodetic Ground Surface Elevation: 101.67 m

Depth (m)	Elevation (m)	Description	Sample Type	Sample Number	Recovery (%)	SPT 'N' Value/RQD%	Field Testing (SPT ○ / DCPT ●)	Lab Testing (Soil Vapour Reading, parts per million (ppm))	Instrumentation Installation
0	101.67	**ASPHALT PAVEMENT:** 80 mm asphalt over 210 mm granular base							
0.3	101.38	**FILL:** sand, trace clay, trace silt, trace gravel, brown to redish brown, moist, loose to compact	SS	1	67	22		11	
1	101		SS	2	62	17		5	
2	100		SS	3	59	9		9	
2.3	99.38	**SAND:** trace clay, trace silt, trace gravel, brown to reddish brown, moist to wet, compact	SS	4	75	10		7	
3	99		SS	5	70	15		18	
4	98								
5	97	- layers of silty clay	SS	6	67	18		18	
6	96								
7	95		SS	7	67	28		8	
8	94		SS	8	70	24		5	
9	93								
10	92		SS	9	70	27		22	
11	91	- layers of gravel	SS	10	75	29		12	
12	90								
13	89		SS	11	79	18		21	

Field Testing: Penetration Testing ○ SPT ● DCPT; MTO Vane* △ Intact ▲ Remould; Nilcon Vane* ◇ Intact ◆ Remould; * Undrained Shear Strength (kPa) 20 40 60 80

Lab Testing: ★ Rinse pH Values 2 4 6 8 10 12; Soil Vapour Reading △ parts per million (ppm) 100 200 300 400; ▲ Lower Explosive Limit (LEL) W_P W W_L Plastic Liquid 20 40 60 80

Instrumentation Installation: Bentonite; Sand; Slotted Pipe; Groundwater Level

Continued on Next Page

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F: 416-551-2633

▽ Groundwater depth on completion of drilling: N/A m.

▼ Groundwater depth observed on July 3, 2023 at a depth of: 9.00 m.

Borehole details as presented, do not constitute a thorough understanding of all potential conditions present and requires interpretative assistance from a qualified Geotechnical Engineer. Also, borehole information should be read in conjunction with the geotechnical report for which it was commisioned and the accompanying'Notes to Record of Boreholes'.

Scale: 1 : 68

RECORD OF BOREHOLE No. BH/MW2

Project Number: **BIGC-ENV-605A** Drilling Location: **See BH Location Plan** Logged by: **CE**

Lithology Plot	DESCRIPTION	Sample Type	Sample Number	Recovery (%)	SPT 'N' Value/RQD%	DEPTH (m)	ELEVATION (m)	FIELD TESTING	LAB TESTING	INSTRUMENTATION INSTALLATION	COMMENTS
	SAND: trace clay, trace silt, trace gravel, brown to reddish brown, moist to wet, compact 87.95										
	SILTY CLAY: grey, moist, stiff 13.7	SS	12	95	12	14	88		23		
	86.43					15	87				
	SILTY CLAY TILL: some sand to sandy, trace gravel, containing occasional cobbles and boulders, grey to redish grey, moist, hard 15.2	SS	13	100	33	16	86		17		
	84.91										
	SILT: trace clay, trace sand, layers of silty clay, seams of silty sand, brown to grey, wet, very dense 16.8	SS	14	100	67	17	85		21		
	83.38					18	84				
	SANDY SILT TILL: trace clay, trace gravel, brown to reddish brown, moist to wet, very dense 18.3	SS	15	95	64	19	83		21		Gr: 9%; Sa: 26%; Si: 58%; Cl: 7%
	81.86										
	SILTY CLAY / SHALE COMPLEX: reddish brown, moist, hard 19.8	SS	16	100	50/ 80 mm	20	82				
	81.02										
	BEDROCK: Shale, Queenston Formation, highly weathered to slightly weathered, fair to good quality, medium strong to strong, occasional limestone layers, reddish brown to grey 20.7 - Fair Quality	RC	1	100	70	21	81				Point Load Test UCS (Axial): 64.73 MPa
	- Fair Quality	RC	2	100	75	22	80				UCS: 73.6 MPa
	- Good Quality	RC	3	100	82	23	79				Point Load Test UCS (Axial): 63.90 MPa
						24	78				Point Load Test UCS (Axial): 33.46 MPa
	77.06										
	End of Borehole 24.6 1. Borehole was open upon completion of drilling. 2. Groundwater level was measured at 9.00 m below ground surface on July 3, 2023.										

Borehole details as presented, do not constitute a thorough understanding of all potential conditions present and requires interpretative assistance from a qualified Geotechnical Engineer. Also, borehole information should be read in conjunction with the geotechnical report for which it was commisioned and the accompanying'Notes to Record of Boreholes'.

Scale: 1 : 68

RECORD OF BOREHOLE No. BH/MW3

Project Number:	BIGC-ENV-605A	Drilling Location:	See BH Location Plan	Logged by:	CE
Project Client:	Halton Standard Condominium Corporation No. 416	Drilling Method:	190 / 100 mm Hollow Stem Augering / Mud Rotary	Compiled by:	KH
Project Name:	Geotechnical and Hydrogeological Investigations	Drilling Machine:	Track Mounted Drill	Reviewed by:	AC
Project Location:	1026 Cooke Blvd, Burlington, Ontario	Date Started:	27 Jun 23	Date Completed: 27 Jun 23	Revision No.: 0, 14/9/23

Geodetic Ground Surface Elevation: 100.64 m

Lithology Plot	DESCRIPTION	Sample Type	Sample Number	Recovery (%)	SPT 'N' Value/RQD%	DEPTH (m)	ELEVATION (m)	FIELD TESTING: Penetration Testing (SPT ○ / DCPT ●), MTO Vane* (Intact △ / Remould ▲), Nilcon Vane* (Intact ◇ / Remould ◆), * Undrained Shear Strength (kPa) 20 40 60 80	LAB TESTING: ★ Rinse pH Values 2 4 6 8 10 12; Soil Vapour Reading △ parts per million (ppm) 100 200 300 400; ▲ Lower Explosive Limit (LEL) W_P W W_L; Plastic 20 40 60 80 Liquid	INSTRUMENTATION INSTALLATION	COMMENTS
	ASPHALT PAVEMENT: 100 mm asphalt over 320 mm granular base (100.22)										
	FILL: sand, trace clay, trace silt, trace gravel, brown, moist, compact (0.4)	SS	1	67	16			○	6		
	(99.12)	SS	2	67	13	1	100	○	8		
	SAND: trace clay, trace silt, trace gravel, brown, moist to wet, loose to compact (1.5)	SS	3	67	19	2	99	○	6		
	(97.95)	SS	4	100	6		98	○	22		
	SILTY CLAY: trace sand, trace gravel, redish brown, moist to wet, firm to very stiff (2.7)					3					
	- seams of sand	SS	5	84	14		97	○	18		
	(94.54)	SS	6	92	29	4	96	○	20		
	SAND: trace clay, trace silt, trace gravel, brown to redish brown, moist to wet, compact to very dense (6.1)	SS	7	79	22	5	95	○	11		
	- layers of sand and gravel					6					
	- cobbles and boulders - layers of silty sand	SS	8	41	67	7	94	○	10		
		SS	9	92	36	8	93	○	21		
	- layers of sandy silt	SS	10	95	42	9	92	○	20		
	(88.45)					10	91				
	SILTY CLAY TILL: trace to some sand, trace gravel, layers of silty clay, grey, moist, very stiff to hard (12.2)	SS	11	87	27	11	90	○	20		
						12	89				
						13	88				

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▽ Groundwater depth on completion of drilling: N/A m.

▼ Groundwater depth observed on July 3, 2023 at a depth of: 7.92 m.

Borehole details as presented, do not constitute a thorough understanding of all potential conditions present and requires interpretative assistance from a qualified Geotechnical Engineer. Also, borehole information should be read in conjunction with the geotechnical report for which it was commisioned and the accompanying'Notes to Record of Boreholes'.

Scale: 1 : 68

Continued on Next Page

RECORD OF BOREHOLE No. BH/MW3

Project Number: **BIGC-ENV-605A** Drilling Location: **See BH Location Plan** Logged by: **CE**

Lithology Plot	DESCRIPTION	Sample Type	Sample Number	Recovery (%)	SPT 'N' Value/RQD%	DEPTH (m)	ELEVATION (m)	FIELD TESTING	LAB TESTING	INSTRUMENTATION INSTALLATION	COMMENTS
	SILTY CLAY TILL: trace to some sand, trace gravel, layers of silty clay, grey, moist, very stiff to hard										
		SS	12	100	38	14	87		15		
							86				
						15					
	85.02	SS	13	100	97		85		16		
	SANDY SILT: trace clay, reddish brown, moist, very dense 15.6					16					
	83.42	SS	14	100	58	17	84		23		
	SILTY CLAY / SHALE COMPLEX: reddish brown, moist, hard 17.2						83				
		SS	15	100	50/ 80 mm	18					
							82				
						19					
	80.83						81				
	BEDROCK: Shale, Queenston Formation 19.8	SS	16	100	50/ 30 mm						
	End of Borehole										

1. Borehole was open upon completion of drilling.
2. Groundwater level was measured at 7.92 m below ground surface on July 3, 2023.

FIELD TESTING: Penetration Testing — ○ SPT ● DCPT; MTO Vane* △ Intact ▲ Remould; Nilcon Vane* ◇ Intact ◆ Remould; * Undrained Shear Strength (kPa) 20 40 60 80

LAB TESTING: ★ Rinse pH Values 2 4 6 8 10 12; Soil Vapour Reading △ parts per million (ppm) 100 200 300 400; ▲ Lower Explosive Limit (LEL) W_P W W_L Plastic Liquid 20 40 60 80

Borehole details as presented, do not constitute a thorough understanding of all potential conditions present and requires interpretative assistance from a qualified Geotechnical Engineer. Also, borehole information should be read in conjunction with the geotechnical report for which it was commisioned and the accompanying'Notes to Record of Boreholes'.

Scale: 1 : 68

RECORD OF BOREHOLE No. BH/MW4

Project Number:	BIGC-ENV-605A	Drilling Location:	See BH Location Plan	Logged by:	CE
Project Client:	Halton Standard Condominium Corporation No. 416	Drilling Method:	190 / 63.5 mm Hollow Stem Augering / HQ Coring	Compiled by:	KH
Project Name:	Geotechnical and Hydrogeological Investigations	Drilling Machine:	Track Mounted Drill	Reviewed by:	AC
Project Location:	1026 Cooke Blvd, Burlington, Ontario	Date Started:	29 Jun 23 Date Completed: 29 Jun 23	Revision No.:	0, 14/9/23

Lithology Plot	DESCRIPTION	Sample Type	Sample Number	Recovery (%)	SPT 'N' Value/RQD%	DEPTH (m)	ELEVATION (m)	FIELD TESTING	LAB TESTING	INSTRUMENTATION INSTALLATION	COMMENTS
	Geodetic Ground Surface Elevation: 101.71 m							Penetration Testing: ○ SPT ● DCPT; MTO Vane* △ Intact ▲ Remould; Nilcon Vane* ◇ Intact ◆ Remould; * Undrained Shear Strength (kPa) 20 40 60 80	★ Rinse pH Values 2 4 6 8 10 12; Soil Vapour Reading △ parts per million (ppm) 100 200 300 400; ▲ Lower Explosive Limit (LEL) W_P W W_L Plastic Liquid 20 40 60 80		
	ASPHALT PAVEMENT: 90 mm asphalt over 200 mm granular base 101.42										
	FILL: sand, trace clay, trace silt, trace gravel, brown, moist, loose to compact 0.3	SS	1	30	10			○			
	- layers of silty sand	SS	2	79	7	1	101	○	○13		
	99.42	SS	3	79	11	2	100	○	○5		
	SAND: trace clay, trace silt, trace gravel, brown to reddish brown, moist to wet, loose to very dense 2.3	SS	4	75	8	3	99	○	○15		
	- layers of silty sand - cobbles and boulders	SS	5	75	25	4	98	○	○10		
		SS	6	75	15	5	97	○	○7		
		SS	7	70	20	6	96	○	○12		
		SS	8	62	12	7	95	○	○5		
	- layers of gravel	SS	9	84	63	8	94	○	○11		
	- layers of gravel	SS	10	92	43	9	93	○	○12		
	- layers of silty sand	SS	11	75	19	10	92	○	○19		
						11	91				
						12	90				
						13	89				

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▽ Groundwater depth on completion of drilling: N/A m.

▼ Groundwater depth observed on July 3, 2023 at a depth of: 9.06 m.

Borehole details as presented, do not constitute a thorough understanding of all potential conditions present and requires interpretative assistance from a qualified Geotechnical Engineer. Also, borehole information should be read in conjunction with the geotechnical report for which it was commisioned and the accompanying'Notes to Record of Boreholes'.

Scale: 1 : 68

Continued on Next Page

RECORD OF BOREHOLE No. BH/MW4

Project Number: **BIGC-ENV-605A** Drilling Location: **See BH Location Plan** Logged by: **CE**

Lithology Plot	DESCRIPTION	Sample Type	Sample Number	Recovery (%)	SPT 'N' Value/RQD%	DEPTH (m)	ELEVATION (m)	FIELD TESTING: Penetration Testing (○ SPT ● DCPT); MTO Vane* (△ Intact ▲ Remould); Nilcon Vane* (◇ Intact ◆ Remould); * Undrained Shear Strength (kPa) 20 40 60 80	LAB TESTING: ★ Rinse pH Values 2 4 6 8 10 12; Soil Vapour Reading △ parts per million (ppm) 100 200 300 400; ▲ Lower Explosive Limit (LEL) W_P W W_L Plastic Liquid 20 40 60 80	INSTRUMENTATION INSTALLATION	COMMENTS
	SAND: trace clay, trace silt, trace gravel, brown to reddish brown, moist to wet, loose to very dense										
	87.99										
	SILTY CLAY: grey, moist, very stiff 13.7	SS	12	100	20	14	88	○	23		
	86.47					15	87				
	SANDY SILT: trace clay, grey, moist, very dense 15.2 - layers of silty clay till	SS	13	100	64	16	86	○	16		
	84.95										
	SILTY CLAY: trace sand, grey, moist, hard 16.8	SS	14	100	55	17	85	○	20		Gr: 0%; Sa: 2%; Si: 70%; Cl: 28% LL: 27%; PL: 14%; PI: 13%
	83.42										
	SAND AND SILT TILL: trace to some gravel, trace clay, reddish brown, moist, dense 18.3 - layers of silty sand to sandy silt	SS	15	100	31	18	84	○	23		
	81.90										
	BEDROCK: Shale, Queenston Formation, highly weathered to fresh, poor to excellent quality, medium strong to strong, occasional limestone layers, reddish brown to grey 19.8 - Poor Quality	SS	16	100	50/ 100 mm	19	83		8		UCS: 47.5 MPa
		RC	1	100	30	20	82	○			Point Load Test UCS (Axial): 78.13 MPa
	- Good Quality	RC	2	100	78	21	81				
	- Excellent Quality	RC	3	100	94	22	80	○			
	76.84					23	79				
	End of Borehole 24.9 1. Borehole was open upon completion of drilling. 2. Groundwater level was measured at 9.06 m below ground surface on July 3, 2023.					24	78	○			Point Load Test UCS (Axial): 43.82 MPa
							77				

Borehole details as presented, do not constitute a thorough understanding of all potential conditions present and requires interpretative assistance from a qualified Geotechnical Engineer. Also, borehole information should be read in conjunction with the geotechnical report for which it was commisioned and the accompanying'Notes to Record of Boreholes'.

Scale: 1 : 68

RECORD OF BOREHOLE No. BH/MW5

Project Number:	BIGC-ENV-605A	Drilling Location:	See BH Location Plan	Logged by:	CE
Project Client:	Halton Standard Condominium Corporation No. 416	Drilling Method:	190 / 63.5 mm Hollow Stem Augering / HQ Coring	Compiled by:	KH
Project Name:	Geotechnical and Hydrogeological Investigations	Drilling Machine:	Track Mounted Drill	Reviewed by:	AC
Project Location:	1026 Cooke Blvd, Burlington, Ontario	Date Started:	28 Jun 23	Date Completed: 28 Jun 23	Revision No.: 0, 14/9/23

Geodetic Ground Surface Elevation: 101.01 m

Lithology Plot	DESCRIPTION	Sample Type	Sample Number	Recovery (%)	SPT 'N' Value/RQD%	DEPTH (m)	ELEVATION (m)
	ASPHALT PAVEMENT: 80 mm asphalt over 230 mm granular base 100.70						
	FILL: sand, trace clay, trace silt, trace gravel, brown to redish brown, moist, loose to compact 0.3	SS	1	67	10		
	- layers of silty clay	SS	2	79	9	1	100
		SS	3	84	14	2	99
		SS	4	54	4	3	98
		SS	5	75	9	4	97
	96.44						
	SAND: trace clay, trace silt, trace gravel, brown to reddish brown, moist to wet, loose to compact 4.6	SS	6	75	6	5	96
	- cobbles and boulders	SS	7	79	40	6	95
		SS	8	8	16	7	94
		SS	9	59	17	8	93
		SS	10	67	10	9	92
						10	91
						11	90
	88.46	SS	11	100	14	12	89
	SILTY CLAY: trace sand, grey, moist, stiff 12.6						
	- layers of silty clay till					13	

FIELD TESTING: Penetration Testing (○ SPT ● DCPT); MTO Vane* (△ Intact ▲ Remould); Nilcon Vane* (◇ Intact ◆ Remould); * Undrained Shear Strength (kPa) 20 40 60 80

LAB TESTING: ★ Rinse pH Values 2 4 6 8 10 12; Soil Vapour Reading △ parts per million (ppm) 100 200 300 400; ▲ Lower Explosive Limit (LEL) W_P W W_L; Plastic Liquid; 20 40 60 80

1	12
2	13
3	12
4	11
5	8
6	6
7	10
8	22
9	21
10	22

INSTRUMENTATION INSTALLATION

COMMENTS

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Mississauga, ON L4W 2Z4
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F: 416-551-2633

▽ Groundwater depth on completion of drilling: N/A m.

▼ Groundwater depth observed on July 3, 2023 at a depth of: 8.38 m.

Borehole details as presented, do not constitute a thorough understanding of all potential conditions present and requires interpretative assistance from a qualified Geotechnical Engineer. Also, borehole information should be read in conjunction with the geotechnical report for which it was commisioned and the accompanying'Notes to Record of Boreholes'.

Scale: 1 : 68

Continued on Next Page

RECORD OF BOREHOLE No. BH/MW5

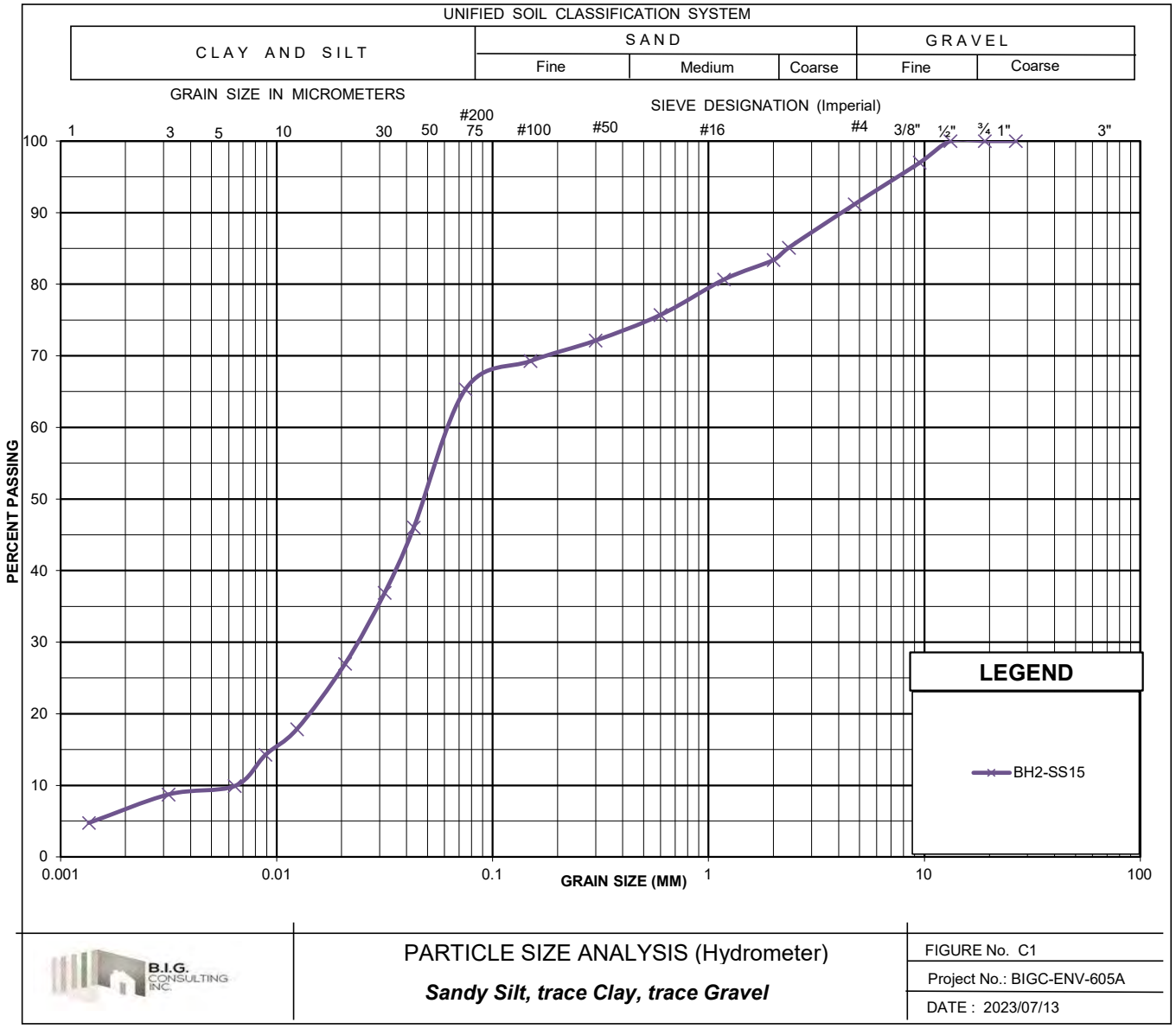
Project Number: **BIGC-ENV-605A** Drilling Location: **See BH Location Plan** Logged by: **CE**

Lithology Plot	DESCRIPTION	Sample Type	Sample Number	Recovery (%)	SPT 'N' Value/RQD%	DEPTH (m)	ELEVATION (m)	FIELD TESTING	LAB TESTING	INSTRUMENTATION INSTALLATION	COMMENTS
	SILTY CLAY: trace sand, grey, moist, stiff							Penetration Testing: ○ SPT ● DCPT; MTO Vane* △ Intact ▲ Remould; Nilcon Vane* ◇ Intact ◆ Remould; * Undrained Shear Strength (kPa) 20 40 60 80	★ Rinse pH Values 2 4 6 8 10 12; Soil Vapour Reading △ parts per million (ppm) 100 200 300 400; ▲ Lower Explosive Limit (LEL) W_P W W_L Plastic Liquid 20 40 60 80		
		SS	12	100	15	14	87	○	24		
	85.77					15	86				
	SILTY CLAY TILL: some sand, trace gravel, redish grey, moist, hard 15.2	SS	13	100	34			○	17		Gr: 2%; Sa: 18%; Si: 48%; Cl: 32% LL: 26%; PL: 15%; PI: 11%
						16	85				
		SS	14	100	41	17	84	○	12		
	- containing shale pieces										
	82.72					18	83				
	SILTY CLAY / SHALE COMPLEX: reddish brown, moist, hard 18.3 - layers of sandy silt till	SS	15	100	63			○	10		
	81.35					19	82				
	BEDROCK: Shale, Queenston Formation, highly weathered to fresh, poor to excellent quality, very weak to medium strong, occasional limestone layers, reddish brown to grey 19.7 - Poor Quality	SS	16	100	50/ 100 mm						
	- Good Quality	RC	1	100	29	20	81	○			Point Load Test UCS (Axial): 18.65 MPa
		RC	2	100	80	21	80	○			
	- Excellent Quality	RC	3	100	98	22	79	○			Point Load Test UCS (Axial): 43.88 MPa
						23	78				
	- Excellent Quality	RC	4	100	94	24	77	○			Point Load Test UCS (Axial): 42.61 MPa
	76.22										
	End of Borehole 24.8 1. Borehole was open upon completion of drilling. 2. Groundwater level was measured at 8.38 m below ground surface on July 3, 2023.										

Borehole details as presented, do not constitute a thorough understanding of all potential conditions present and requires interpretative assistance from a qualified Geotechnical Engineer. Also, borehole information should be read in conjunction with the geotechnical report for which it was commisioned and the accompanying'Notes to Record of Boreholes'.

Scale: 1 : 68

Appendix C – Geotechnical Laboratory Test Results



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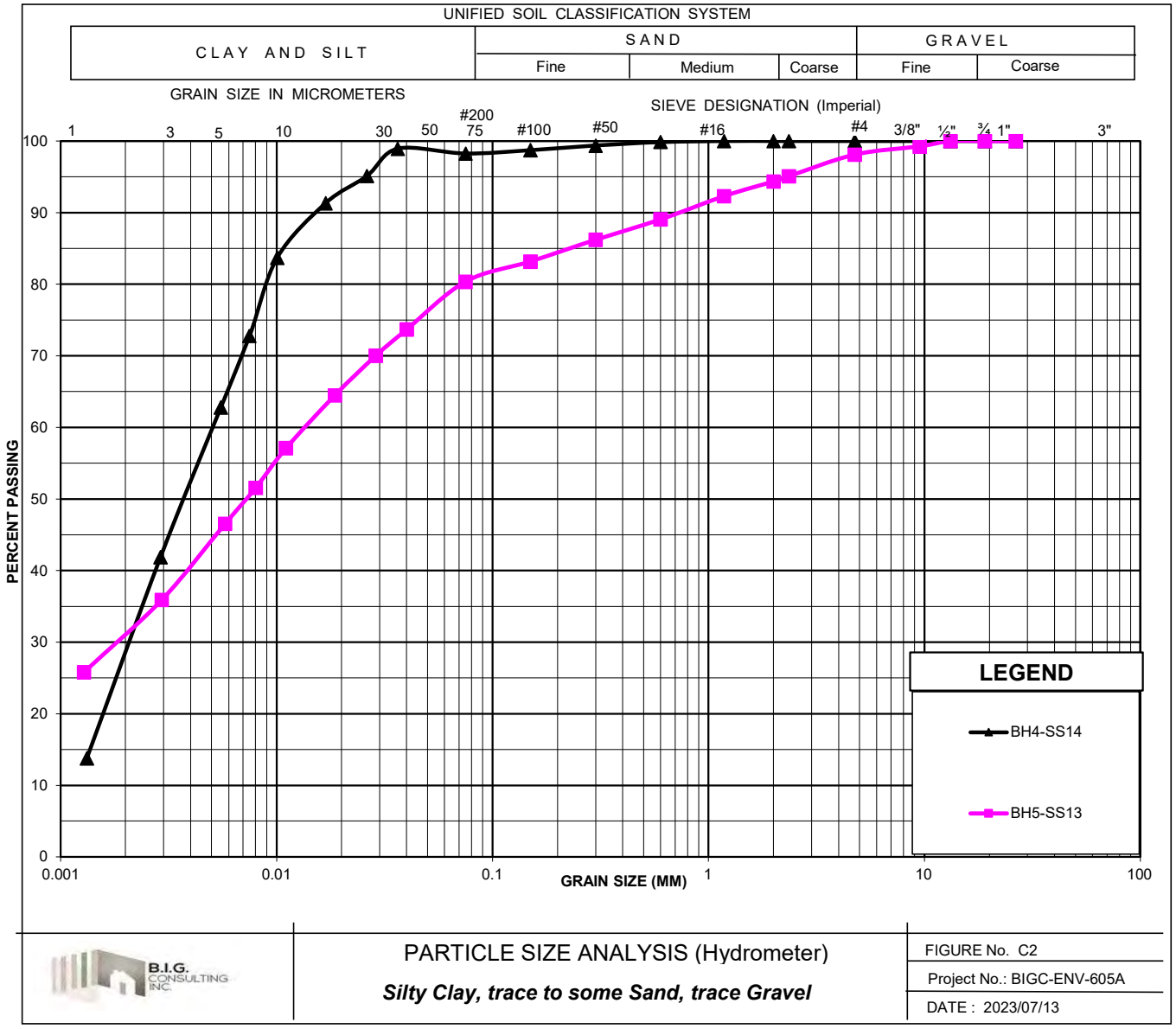
PARTICLE SIZE ANALYSIS (Hydrometer)

Sandy Silt, trace Clay, trace Gravel

FIGURE No. C1

Project No.: BIGC-ENV-605A

DATE : 2023/07/13



B.I.G. CONSULTING INC.

PARTICLE SIZE ANALYSIS (Hydrometer)

Silty Clay, trace to some Sand, trace Gravel

FIGURE No. C2

Project No.: BIGC-ENV-605A

DATE : 2023/07/13

B.I.G CONSULTING INC.

PLASTICITY CHART

Job # :	BIGC-ENV-605A	**Lab #** :	
Project Client:		**Technician** :	PS
Project :	Preliminary Geotechnical Investigation	**Supervisor** :	SS
Location :	1026 Cooke Blvs, Burlington	**Date** :	07/13/2023

TEST RESULTS

Specimen #	Sample #	Depth (m)	LL%	PL%	PI	Fines%	W%	Classification	Remarks
BH4	SS14		27	14	13	98		CL	Native
BH5	SS13		26	15	11	80		CL	Native

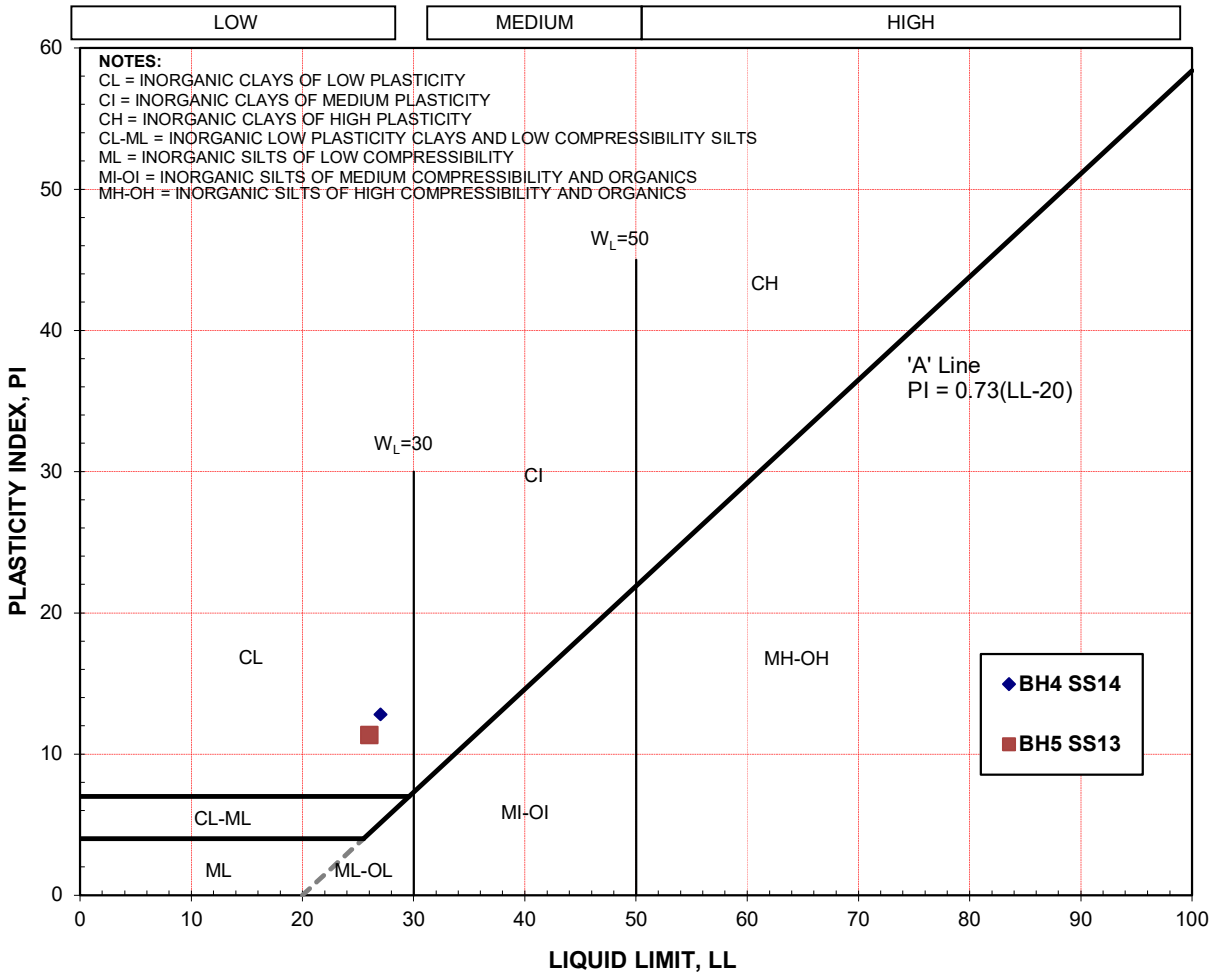


FIGURE - C3

Appendix D – Picture of Rock Core Samples

ROCK CORE PICTURES

BH/MW2

B.I.G. Consulting Inc.
12-5500 Tomken Road
Mississauga, Ontario, L4W 2Z4
Tel: (416) 214-4880
Fax: (416) 551-2633
www.bigconsultinginc.com

Project No.:	BIGC-ENV-605A	**Prepared By:**	KH
Project:	Geotechnical Investigation	**Checked By:**	AC
Location:	1026 Cooke Blvd, Burlington, ON	**Date:**	06/26/2023

Rock Core No.	Core Depth From (m)	Core Length (m)	TCR (%)	SCR (%)	RQD (%)
RC-1	20.65	1.22	100	92	70
RC-2	21.87	1.57	100	96	75
RC-3	23.44	1.17	100	82	82



ROCK CORE PICTURES

BH/MW4

B.I.G. Consulting Inc.
12-5500 Tomken Road
Mississauga, Ontario, L4W 2Z4
Tel: (416) 214-4880
Fax: (416) 551-2633
www.bigconsultinginc.com

Project No.:	BIGC-ENV-605A	**Prepared By:**	KH
Project:	Geotechnical Investigation	**Checked By:**	AC
Location:	1026 Cooke Blvd, Burlington, ON	**Date:**	06/29/2023

Rock Core No.	Core Depth From (m)	Core Length (m)	TCR (%)	SCR (%)	RQD (%)
RC-1	20.27	1.52	100	78	30
RC-2	21.79	1.53	100	93	78
RC-3	23.32	1.55	100	100	94



ROCK CORE PICTURES

BH/MW5

B.I.G. Consulting Inc.
12-5500 Tomken Road
Mississauga, Ontario, L4W 2Z4
Tel: (416) 214-4880
Fax: (416) 551-2633
www.bigconsultinginc.com

Project No.:	BIGC-ENV-605A	**Prepared By:**	KH
Project:	Geotechnical Investigation	**Checked By:**	AC
Location:	1026 Cooke Blvd, Burlington, ON	**Date:**	06/28/2023

Rock Core No.	Core Depth From (m)	Core Length (m)	TCR (%)	SCR (%)	RQD (%)
RC-1	19.76	0.43	100	76	29
RC-2	20.19	1.53	100	93	80
RC-3	21.72	1.52	100	98	98
RC-4	23.24	1.55	100	98	94



Appendix E – Point Load Test Results

Point Load Strength Index of Rock Test Results

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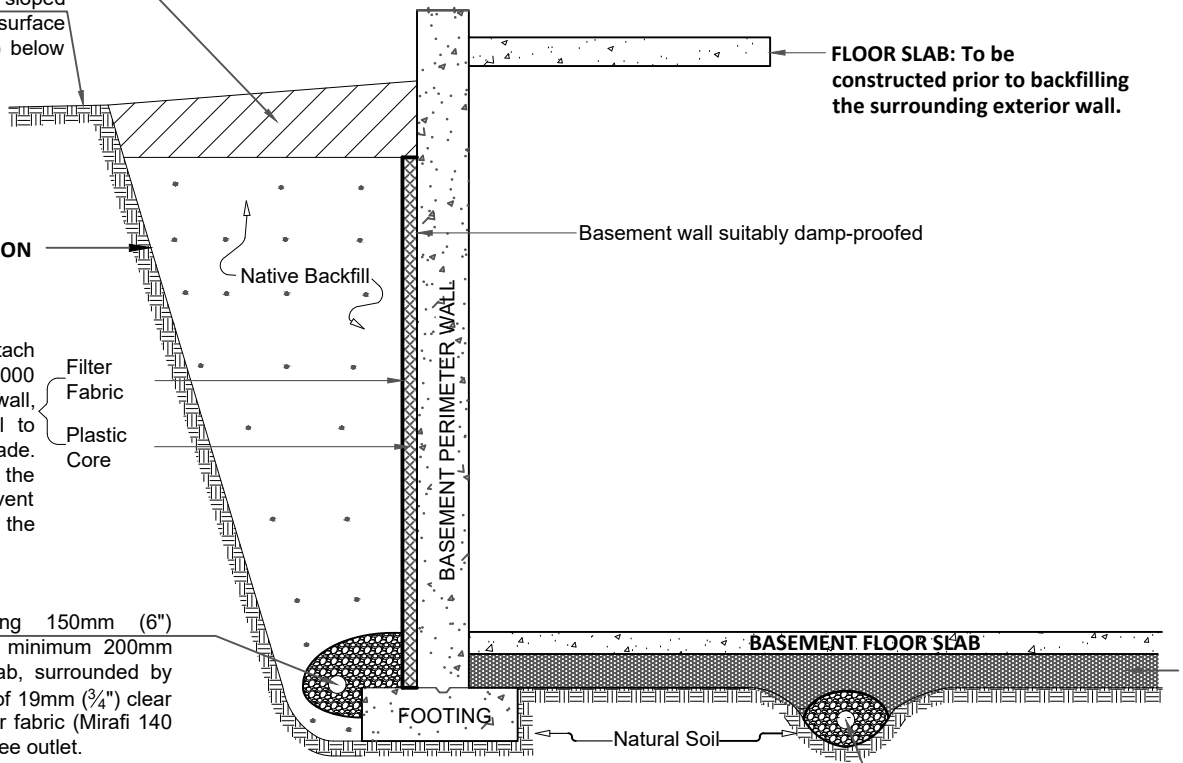
ASTM D5731-16

Project No: BIGC-ENV-605A
Project Name: Phase Two ESA, Preliminary Geo & Hydrogeological Inv.
Project Location: 1026 Cooke Boulevard, Burlington, Ontario

Date(s) Tested: August 11 & 15, 2023
Tested By: VB
Reviewed By: FS

Borehole ID	RUN No.	Sample Depth (m)	Test Type, D-diametric A-axial	Distance Between Platens, (mm)	Failure Load, P (kN)	Effective Diameter, D_e^2	Uncorrected Point Strength, Is (MPa)	Size Correction Factor, F	Size Uncorrected Point Load Strength Index, Is(50) (MPa)	Infered Rock's Estimated Uniaxial Compressive Strength, UCS (MPa)	Material Description
			D	63.30	0.43	4006.89	0.107	1.11	0.12	2.92	shale
			D	62.80	0.89	3943.84	0.226	1.11	0.25	6.13	shale
			D	62.60	1.23	3918.76	0.314	1.11	0.35	8.51	shale
BH/MW2	RUN1	20.9 - 21.05	A	29.20	2.46	2334.82	1.054	1.11	1.17	28.65	shale and limestone
			A	24.07	10.20	1933.82	5.275	1.11	5.85	143.44	limestone
			A	62.80	1.50	1843.86	0.814	1.11	0.90	22.08	shale and limestone
								Diametric Mean	0.24	5.85	
								Axial Mean	2.64	64.73	
			D	63.10	1.07	3981.61	0.269	1.11	0.30	7.31	shale
			D	63.10	0.83	3981.61	0.208	1.11	0.23	5.67	shale
			D	63.10	0.05	3981.61	0.012	1.11	0.01	0.31	shale
BH/MW2	RUN2	23.40 - 23.60	A	23.00	9.80	1850.78	5.295	1.11	5.88	144.00	limestone
			A	35.00	1.60	2825.32	0.566	1.11	0.63	15.40	shale
			A	28.30	2.70	2273.66	1.188	1.11	1.32	32.29	shale and limestone
								Diametric Mean	0.18	4.43	
								Axial Mean	2.61	63.90	
			D	63.10	0.72	3981.61	0.181	1.11	0.20	4.92	shale
			D	63.10	0.92	3981.61	0.231	1.11	0.26	6.29	shale
			D	63.20	1.20	3994.24	0.300	1.11	0.33	8.18	shale
BH/MW2	RUN3	23.90 - 24.10	A	29.94	2.91	2409.23	1.208	1.11	1.34	32.85	shale
			A	30.01	3.30	2422.51	1.362	1.11	1.51	37.05	shale
			A	42.20	3.80	3390.41	1.121	1.11	1.24	30.48	shale
								Diametric Mean	0.26	6.46	
								Axial Mean	1.37	33.46	
			D	63.10	2.25	3981.61	0.565	1.11	0.63	15.37	shale
			D	62.95	2.70	3962.70	0.681	1.11	0.76	18.52	shale
			D	63.10	2.82	3981.61	0.708	1.11	0.79	19.27	shale
BH/MW4	RUN2	21.50 - 21.65	A	16.90	5.90	1355.62	4.352	1.11	4.83	118.36	shale and limestone
			A	19.30	3.88	1555.50	2.494	1.11	2.77	67.83	limestone
			A	40.00	5.75	3244.21	1.772	1.11	1.97	48.20	shale and limestone
								Diametric Mean	0.72	17.72	
								Axial Mean	3.19	78.13	
			D	63.10	0.18	3981.61	0.044	1.11	0.05	1.20	shale
			D	63.10	0.77	3981.61	0.193	1.11	0.21	5.25	shale
			D	63.10	0.80	3981.61	0.202	1.11	0.22	5.49	shale
BH/MW4	RUN3	21.70 - 21.90	A	22.73	2.91	1826.16	1.594	1.11	1.77	43.34	shale
			A	32.20	3.30	2582.89	1.278	1.11	1.42	34.75	shale
			A	31.90	5.03	2562.89	1.963	1.11	2.18	53.37	shale
								Diametric Mean	0.16	3.98	
								Axial Mean	1.79	43.82	
			D	63.00	0.29	3969.00	0.073	1.11	0.08	1.99	shale
			D	63.10	0.24	3981.61	0.060	1.11	0.07	1.64	shale
			D	63.20	0.18	3994.24	0.045	1.11	0.05	1.23	shale
BH/MW5	RUN2	20.20 - 20.40	A	28.50	2.30	2286.10	1.006	1.11	1.12	27.36	shale and limestone
			A	33.08	2.16	2653.48	0.814	1.11	0.90	22.14	shale
			A	47.80	0.91	3834.23	0.237	1.11	0.26	6.45	shale and limestone
								Diametric Mean	0.07	1.62	
								Axial Mean	0.76	18.65	
			D	63.30	1.20	4006.89	0.299	1.11	0.33	8.16	shale
			D	63.10	1.59	3981.61	0.399	1.11	0.44	10.86	shale
			D	63.20	1.19	3994.24	0.298	1.11	0.33	8.11	shale
BH/MW5	RUN3	21.70 - 21.90	A	36.20	3.97	2912.97	1.363	1.11	1.51	37.06	shale
			A	54.80	6.03	4409.69	1.366	1.11	1.52	37.16	shale
			A	53.30	9.06	4288.98	2.112	1.11	2.34	57.43	shale
								Diametric Mean	0.37	9.04	
								Axial Mean	1.79	43.88	
			D	63.10	0.44	3981.61	0.111	1.11	0.12	3.01	shale
			D	63.10	0.25	3981.61	0.063	1.11	0.07	1.71	shale
			D	63.10	0.86	3981.61	0.216	1.11	0.24	5.88	shale
BH/MW5	RUN4	24.30 - 24.50	A	33.04	3.08	2658.69	1.158	1.11	1.29	31.50	shale
			A	59.20	10.30	4756.21	2.166	1.11	2.40	58.89	shale and limestone
			A	44.30	4.90	3559.12	1.377	1.11	1.53	37.44	shale and limestone
								Diametric Mean	0.14	3.53	
								Axial Mean	1.74	42.61	

Appendix F – Permanent Perimeter & Under-Floor Drainage System for Open Excavation (Conceptual)
Permanent Perimeter & Under-Floor Drainage System with Vertical Shoring (Conceptual)



SURFACE SEAL: 600 (2') thick Relatively impervious clayey soils. May be deleted, if adjacent native soil is sand/gravel or if the surface is paved tight to the building.

EXTERIOR GRADE: Positively sloped away from building to promote surface drainage. Minimum 300mm (12") below the interior floor slab.

PRE-FABRICATED DRAIN: Attach pre-fabricated drain (Miradran 6000 or equivalent) to basement wall, continuous from bottom of wall to 0.6m (2') below the exterior grade. Fold all terminal fabric flaps over the core edge and fasten to prevent intrusion of soils/gravels into the drainage core.

PERIMETER DRAIN: Comprising 150mm (6") diameter perforated pipe, invert minimum 200mm (8") below underside of floor slab, surrounded by minimum 150mm (6") thick layer of 19mm (¾") clear stone wrapped in a synthetic filter fabric (Mirafi 140 or equivalent) draining to a frost free outlet.

GRANULAR BASE: Minimum 150mm (6") thick well compacted Granular A or approved eqivalent.

UNDER-FLOOR DRAIN (If Required): Comprising 75 (3") - 100mm (4") diameter perforated pipes placed in regular intervals, surrounded by minimum 150mm (6") thick layer of 19mm (¾") clear stone wrapped in a synthetic filter fabric (Mirafi 140 or equivalent) draining to a frost free outlet. Invert should be at least 300mm (12") below the underside of the floor slab. (REFER GEOTECHNICAL REPORT).

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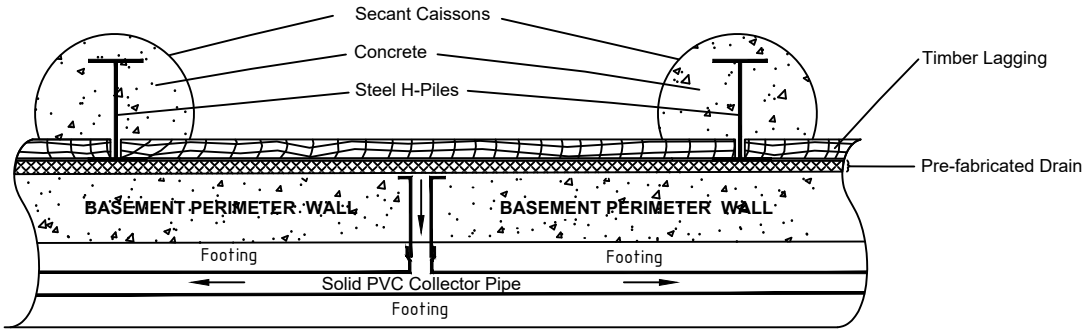
t: (416) 214 - 4880 f: (416) 551 - 2633
12-5500 Tomken Rd.
Misissauga, ON L4W 2Z4
Canada

bigconsultinginc.com

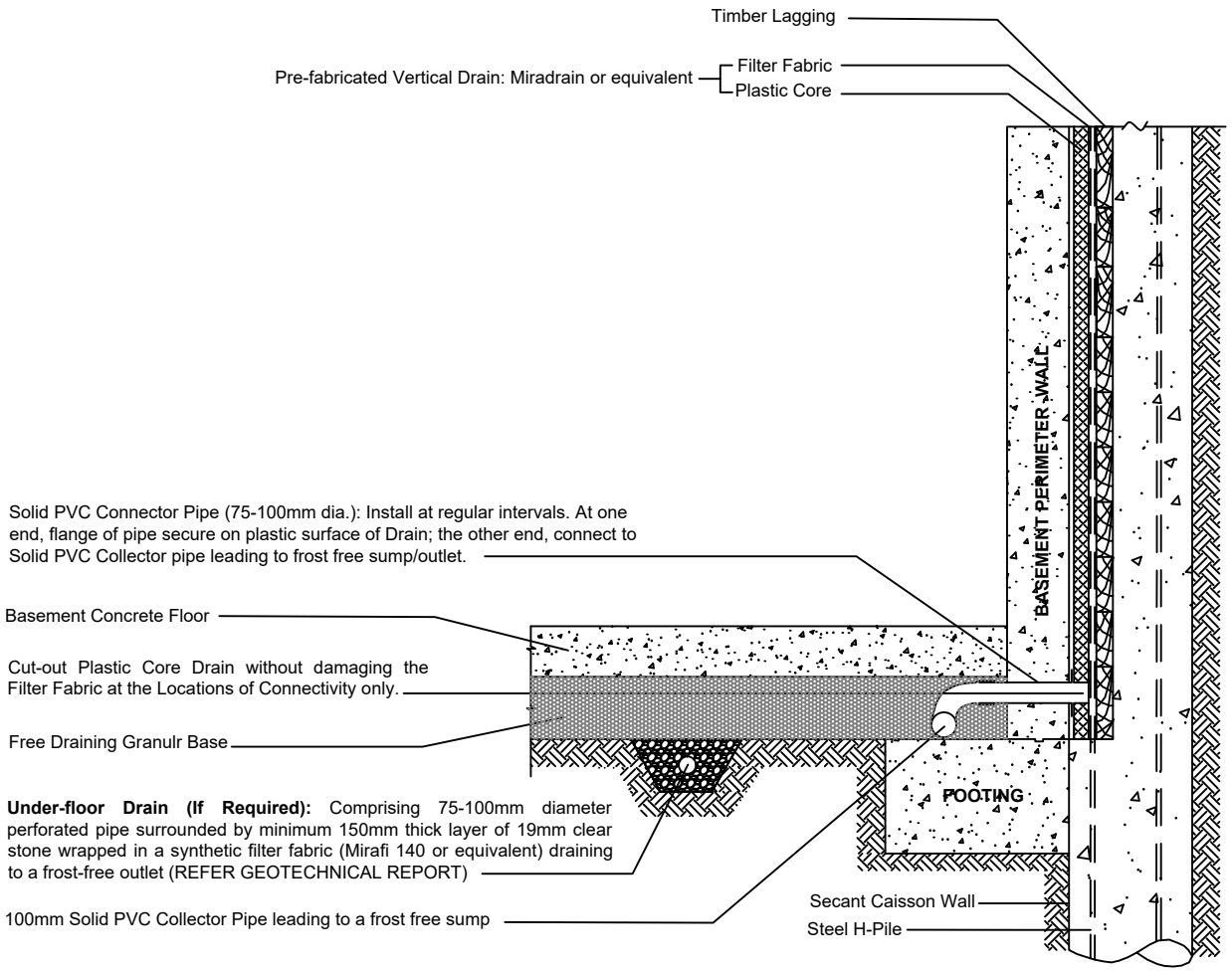
TITLE AND LOCATION

PERMANENT PERIMETER & UNDER-FLOOR DRAINAGE SYSTEM FOR OPEN EXCAVATION (CONCEPTUAL)
PRELIMINARY GEOTECHNICAL INVESTIGATION
1026 COOKE BOULEVARD,
BURLINGTON, ONTARIO

PROJECT NO.	DWN.
BIGC-ENV-605A	E.P.
SCALE	CK.
AS NOTED	A.C.
DATE	FIG NO.
JULY 2023	F1



PLAN



TYPICAL SECTION

Note:

1. A continuous blanket of prefabricated drainage system, Miradrain 6000 or equivalent, should extend continuously from the top of footings to approximately 1.2m below the ground surface.
2. All terminal end openings (top, bottom & sides) of drain must be covered with terminal fabic flaps and fasten to prevent intrusion of concrete and soils into the drainage core.
3. All surface joints of the Miradrain should be sealed with tape.
3. The backfill materials behind the lagging should be free draining. If wet conditions are encountered, geotextile filter fabric or straw should be used to prevent loss of ground.
4. Subfloor drainage system (if required) should keep/treat separate from the perimeter drainage system.

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12-5500 Tomken Rd.
Misissauga, ON L4W 2Z4
Canada

bigconsultinginc.com

TITLE AND LOCATION

PERMANENT PERIMETER & UNDER-FLOOR DRAINAGE SYSTEM WITH VERTICAL SHORING (CONCEPTUAL)
PRELIMINARY GEOTECHNICAL INVESTIGATION
1026 COOKE BOULEVARD,
BURLINGTON, ONTARIO

PROJECT NO.	DWN.
BIGC-ENV-605A	E.P.
SCALE	CK.
AS NOTED	A.C.
DATE	FIG NO.
JULY 2023	F2

Appendix G - UCS Rock Laboratory Testing Results

Geomechanica Inc.
Unit 14 – 1240 Speers Rd.
Oakville Ontario
Canada L6L 2X4

September 13, 2023

Khagendra Kandel
B.I.G. Consulting Inc.
12-5500 Tomken Road
Mississauga, ON
Canada, L4W 2Z4

Re: Rush UCS Testing
(B.I.G. Project No. BIGC-ENV-650A)

Dear Khagendra:

On September 7th, 2023 a series of two (2) core samples (HQ-sized) were received by Geomechanica Inc. via drop-off by B.I.G. personnel. These samples were identified as being from B.I.G. project BIGC-ENV-650A. From these samples, 2 Uniaxial Compressive Strength (UCS) tests were completed.

Details regarding the steps of specimen preparation and testing along with the test results are presented in the accompanying laboratory report and summary spreadsheet.

Sincerely,



Bryan Tatone Ph.D., P. Eng.

Geomechanica Inc.
Tel: (647) 478-9767
Email: bryan.tatone@geomechanica.com

GEOMECHANICA

Rock Laboratory Testing Results

A report submitted to:

Khagendra Kandel
B.I.G. Consulting Inc.
12-5500 Tomken Road
Mississauga, ON
Canada, L4W 2Z4

Prepared by:

Bryan Tatone, PhD, PEng

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Geomechanica Inc.
#14-1240 Speers Rd.
Oakville ON
L6L 2X4 Canada
Tel: +1-647-478-9767
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September 13, 2023
Project number: BIGC-ENV-605a

Abstract

This document summarizes the results of laboratory testing, including 2 Uniaxial Compressive Strength (UCS) tests. The UCS values along with photographs of specimens before and after testing are presented herein.

In this document:

1 Uniaxial Compressive Strength Tests 1
Appendices 3

Disclaimer: This report was prepared by Geomechanica Inc. for B.I.G. Consulting Inc.. The material herein reflects Geomechanica Inc.'s best judgment given the information available at the time of preparation. Any use which a third party makes of this report, any reliance on or decision to be made based on it, are the responsibility of such third parties. Geomechanica Inc. accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.

1 Uniaxial Compressive Strength Tests

1.1 Overview

This section summarizes the results of uniaxial compressive strength (UCS) testing. The testing was performed in Geomechanica's rock testing laboratory using a 150 ton (1.3 MN) Forney loading frame equipped with pressure-compensated control valve to maintain an axial displacement rate of approximately 0.15 mm/min (Figure 1). The preparation and testing procedure for each specimen included the following:

1. Unwrapping the core sample, inspecting it for damage, and re-wrapping it in electrical tape to minimize exposure to moisture and possible damage during subsequent specimen preparation.
2. Diamond cutting the core sample to obtain a cylindrical specimen with an appropriate length (length:diameter = 2:1) and nearly parallel end faces.
3. Diamond grinding the specimen to obtain flat (within ± 0.025 mm) and parallel end faces (within 0.25°).
4. Placing the specimen into the loading frame, applying a 1 kN axial load, and removing the electrical tape.
5. Axially loading the specimen to rupture while continuously recording axial force and axial deformation to determine the peak strength (UCS).



Figure 1: Forney loading frame setup for UCS testing.

Using a precision V-block mounted on the magnetic chuck of the surface grinder, test specimens met the end flatness, end parallelism, and perpendicularity criteria set out in ASTM D4543-19. The side straightness criteria, as checked with a feeler gauge, and the minimum length:diameter criteria were met for all specimens unless noted otherwise in Table 1. Testing of the specimens followed ASTM D7012-14 Method C.

1.2 Results

The results of UCS testing are summarized in Table 1. Additional specimens and testing details are included in the summary spreadsheet that accompanies this report.

Table 1: Summary of UCS test results.

Sample	Depth (ft' in")	Bulk density ρ (g/cm^3)	UCS (MPa)	Lithology	Failure description
BH/MW2	74'5" - 75'0"	2.630	73.6	Red Shale and Limestone	1, 2
BH/MW4	66'10" - 67'4"	2.618	47.5	Red Shale and Limestone	1

[1] Inclined shear fracture and axial splitting failure

[2] Partial hourglass failure

1.3 Specimen photographs

Photographs of the specimens before and after testing are presented in the Appendix of this report.

Appendices

Specimen sheets

- BH/MW2
- BH/MW4

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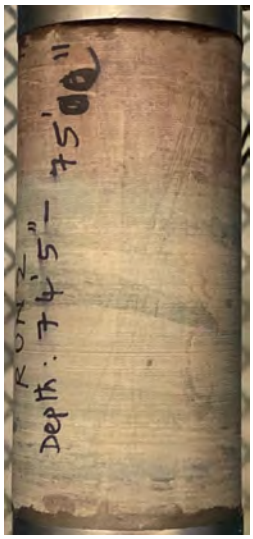
Uniaxial Compression Test

Client	B.I.G. Consulting Inc.	**Project**	BIGC-ENV-605a
Sample	BH/MW2	**Depth**	74’5” - 75’0”

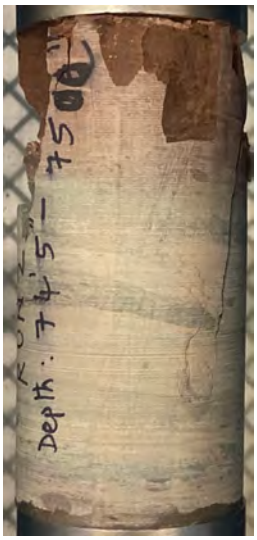
Specimen parameters	
Diameter (mm) [a]	62.93
Length (mm) [a]	129.44
Bulk density ρ (g/cm^3)	2.630
UCS (MPa)	73.6
Lithology	Red Shale and Limestone
Failure description [b]	1, 2

[a] Additional specimen measurement/details provided in accompanying summary spreadsheet.
[b] Failure description: [1] Inclined shear fracture and axial splitting failure; [2] Partial hourglass failure;

Prior to testing



After testing



Remarks: Loading rate: 0.15 mm/min.

Performed by	AB/SD	**Date**	2023-09-11

GEOMECHANICA

Uniaxial Compression Test

Client	B.I.G. Consulting Inc.	**Project**	BIGC-ENV-605a
Sample	BH/MW4	**Depth**	66'10" - 67'4"

Specimen parameters	
Diameter (mm) [a]	62.77
Length (mm) [a]	130.65
Bulk density ρ (g/cm^3)	2.618
UCS (MPa)	47.5
Lithology	Red Shale and Limestone
Failure description [b]	1

[a] Additional specimen measurement/details provided in accompanying summary spreadsheet.
[b] Failure description: [1] Inclined shear fracture and axial splitting failure;

Prior to testing



After testing



Remarks: Loading rate: 0.15 mm/min.

Performed by	AB/SD	**Date**	2023-09-11