PRELIMINARY GEOTECHNICAL INVESTIGATION

789-795 Brant Street
Burlington, Ontario

Client

Camarro Developments Inc.
25 Main Street West, Unit 1702
Hamilton, ON L8P 1H1

Project Number

BIGC-ENV-461A

Prepared By:

B.I.G. Consulting Inc.
12-5500 Tomken Road
T: 416.214.4880
www.bigconsultinginc.com

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1 Introduction

B.I.G. Consulting Inc. (BIG) has been retained by Camarro Developments Inc. (the Client) to conduct a preliminary geotechnical investigation for the proposed development on the property located at 789-795 Brant Street, Burlington, Ontario (the Site). The Site location plan is shown on Figure 1 in Appendix A.

The investigation was authorized by Mr. Kyle Camarro on behalf of the Client.

It is our understanding that the proposed development will comprise of a High-rise Building with 5-Levels of underground parking structure.

The field work for this investigation was carried out in conjunction with Phase I Environmental Site Assessment (ESA I), Hydrogeological Investigation (HG) and Filing of Record of Site Condition (RSC). This report addresses the geotechnical aspects of the proposed development only and the reports for the ESA I, HG and RSC will be issued under separate covers.

The purpose of this investigation was to obtain the information on the subsurface soil and groundwater conditions at the Site by means of advancing a limited number of boreholes, in-situ tests as well as laboratory tests of selected soil samples and based on this information to prepare an engineering report on geotechnical perspective pertaining to the preliminary design and construction of the proposed development. Final design drawings of the proposed development were not available to BIG at the time of preparation of this report. Therefore, once the existing buildings are demolished and final design drawings are available, additional investigation and analysis will be necessary and further recommendations will be provided as appropriate.

The recommendations and comments made in this report are based on factual information and are intended only to use for 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 development.

The report is prepared with the condition that the design will be in accordance with all applicable standards and codes, regulations of authorities having jurisdiction, and good engineering practice. On-going liaison with BIG during the final design and construction phase of the project is recommended to ensure that the 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.

The attached 'Report Limitations' is an integral part of this report.

2 Site Description

The municipal address of the Site is 789-795 Brant Street which is located at the southeast corner of Brant Street and Prospect Street in Burlington, Ontario. The Site is bound to the west by Brant Street followed by commercial properties, to the north by Prospect Street followed by commercial and residential properties, to the east by residential properties, and to the south by a commercial property. For the simplicity of the description in this report, Brant Street and Prospect Street have been considered running north-south and east-west respectively.

The Site is irregular in shape and measures approximately 2,250 m^2 in area. The Site is currently occupied by one 1-Storey slab-on-grade commercial building at 795 Brant Street, and another 2-storey

commercial building with 1-Level of basement and a parking garage at 789 Brant Street (Site buildings). The Site buildings have a combined footprint area of approximately 420 m² and occupy approximately 19 % of the Site area. The areas surrounding the Site buildings are covered with asphalt and landscaping.

The topography of the site was generally flat. Based on the ground surface elevation at borehole locations, a grade difference, in order of 0.4 m, was noted across the site.

3 Field Investigation Procedures

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 the lines will not be damaged and safety of the worker during the investigation work.

The fieldwork for this investigation was carried out between June 10 and 19, 2021 and consisted of advancing a total of 5-Boreholes BH/MW1 to BH/MW5 to depths varying between 19.8 and 24.8 m below the existing ground surface (BGS) including bedrock coring in all 5-Boreholes. Bedrock was cored using wire line diamond coring method to confirm the presence and quality of bedrock. The approximate borehole/core-hole locations established and drilled/cored at the Site are shown on Figure 2 in Appendix A.

The boreholes were advanced by using power operated solid and hollow stem continuous flight augers (truck-mounted drill rig), 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 0.76m 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 of non-cohesive soils.

The BIG's drilling supervisor examined and logged the overburden soil/rock-core samples as they were obtained from the boreholes. The recovered soil samples were sealed in clean, airtight plastic bags and rock-core samples were put in wooden box, and transferred to the BIG's Mississauga laboratory for further examination and laboratory testing.

Groundwater observations were made in all boreholes during and immediately upon completion of drilling. In order to obtain the information on stabilized groundwater levels, all boreholes were equipped with monitoring wells, one in each borehole, upon completion of drilling. Monitoring Wells installation details are shown on the Record of Borehole logs, and the details of the groundwater observation are presented on Section 4.7.

The ground surface elevations at the borehole locations were surveyed by BIG personnel with reference to Monument No. BM1045 shown on City of Burlington Monument Position Sketch, dated March 31, 2015 with a Geodetic Elevation of 93.477 mASL, which is located at Culvert over Rambo Creek on Prospect Street, 45 m Southwest of Pearson Street and 560 m Northeast of Brant Street.

It should be noted that the ground surface elevations at the borehole locations are approximate and should not be used for design and construction purpose. 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 be considered to be approximate.

4 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/rock 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 ground surface cover (topsoil and asphalt pavement) overlying existing fills, which in turn was underlain by native clayey silt till, silty sand till, clayey silt till and Shale bedrock, respectively.

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.

4.1 Ground Surface Cover

Borehole, BH/MW1 was advanced through an existing ground surface cover consisting of approximately 120 mm thick topsoil. Topsoil in general, consists of high contents of organics and rootlets. It should be noted that topsoil thickness may vary between and beyond the borehole locations, and thickness could also be changed significantly due to some on-site activities. Therefore, it is recommended that allowance be made for possible variations when making construction estimates.

Remaining boreholes BH/MW2 to BH/MW5 were advanced through existing asphalt pavement consisting of approximately 50-80 mm thick asphalt concrete over 120 to 150 mm thick granular bases.

4.2 Existing Fill

Below the ground surface cover in all boreholes, existing fill generally consisted of clayey silt with trace to some sand, and trace to some gravels were encountered in all boreholes that extended to depths varying between 1.5 and 2.3 mBGS. Fill also contained occasional Shale fragments, glass pieces, and trace organics.

The SPT 'N' values recorded were generally varied from 4 to 12 blows per 300 mm of penetration indicating soft to stiff consistencies. The moisture content measurement of the recovered samples generally varied between 8 and 22 % by weight indicating a moist condition.

4.3 Upper Clayey Silt Till

Below existing fill in all borehole, native deposit of upper clayey silt till was encountered that extended to depths varying between 3.1 and 6.1 mBGS. Clayey silt till deposit also contained trace to sand and trace gravel.

The SPT 'N' values recorded in this till deposit varied from 9 to over 50 blows per 300 mm of penetration indicating stiff to hard consistencies. The moisture content measurement of the recovered samples was between 12 and 23 % by weight, indicating moist to very moist conditions.

Due to the nature of till formation, cobbles and boulders should be anticipated within the glacial till deposits.

Geotechnical laboratory test consisting of Particle Size Analysis (Hydrometer) was conducted on selected soil sample from this deposit as summarized below:

Particle Size Analysis Test Results:

Borehole No.	Sample No.	Approx. Sample Depth (m)	% Gravel	% Sand	% Silt	% Clay
BH/MW1	SS5	3.4	6	26	48	20

Based on the above noted information, the on-Site native soil may be classified as clayey silt till, sandy with trace gravel.

Summary of test results are also included in the respective Record of Boreholes in Appendix B and Lab Test Reports are included in Appendix C.

4.4 Silty Sand Till

Below upper clayey silt till in boreholes BH/MW2 to BH/MW5, a layer on glacial silty sand till was encountered that extended to depths varying between 3.2 to 4.6 mBGS. Silty sand till also contained trace clay, and trace to some gravel.

The SPT 'N' values recorded in this layer varied from 21 to over 50 blows per 300 mm of penetration indicating compact to very dense relative density. The moisture content measurement of the recovered samples was between 10 and 16 % by weight, indicating moist to wet conditions.

Geotechnical laboratory test consisting of Particle Size Analysis (Hydrometer) test was conducted on selected soil sample from this deposit as summarized below:

Particle Size Analysis Test Results:

Borehole No.	Sample No.	Approx. Sample Depth (m)	% Gravel	% Sand	% Silt	% Clay
BH/MW3	SS5	3.4	20	41	32	7

Based on the above noted information, the on-Site native soil may be classified as silty sand till with some gravel and trace clay.

Summary of test results are also included in the respective Record of Boreholes in Appendix B and Lab Test Reports are included in Appendix C.

4.5 Lower Clayey Silt Till

Below silty sand till in boreholes BH/MW3 and BH/MW5, another glacial deposit of lower clayey silt till was encountered that extended to the depth of 6.1 mBGS. Clayey silt till deposit also contained trace to some sand and trace gravel.

The SPT 'N' values recorded in this till deposit were over 50 blows per 300 mm of penetration indicating a hard consistency. The moisture content measurement of the recovered samples varied between 5 and 12 % by weight, indicating a moist condition.

4.6 Shale Bedrock

Below upper clayey silt till at borehole BH/MW1; below sandy silt till at boreholes BH/MW2 and BH/MW4; and below lower clayey silt till at boreholes BH/MW3 and BH/MW5; highly weathered to excellent quality of reddish brown/grey Shale bedrock was encountered in all boreholes.

Further, from borehole auger termination depths into the Shale bedrock varying between 11.9 and 12.2 mBGS, HQ size samples of Shale bedrock were cored to the depths varying between 19.8 and 24.8 mBGS. The percentage of recoveries of rock core samples were between 86% and 100%, however, the Rock Quality Designation (R.Q.D.) values were found widely varying between 0% and 100%, indicating very poor to excellent quality of rock mass. Within the Shale bedrock core samples, occasional interbedded Limestone layers were also present.

More information pertaining to bedrock and rock cores are provided in the respective Record of Boreholes in Appendix B and the Pictures of Rock Core Samples are included in Appendix D.

4.7 Groundwater Observation

Groundwater observations were made in all boreholes during and immediately upon completion of drilling. It should be noted that the sufficient time was not available for the groundwater to stabilize within open boreholes/monitoring wells immediately upon completion of drilling and sampling operations. In order to obtain information on the stabilized groundwater level in conjunction with hydrogeological investigation, all boreholes were equipped with Monitoring Wells, one in each borehole, upon completion of drilling.

Groundwater observation made in open boreholes during Site exploration as well as the groundwater levels recorded in the installed monitoring wells on June 28, 2021 are tabulated below:

Groundwater Observation:

Borehole No.	Ground Elevation (m)	Borehole Depth (mBGS)	MW Depth (mBGS)	Screen Length (m)	Groundwater Depth (mBGS)/Elevation (mASL)	
					Upon Completion of Drilling	June 28, 2021
BH/MW1	92.24	19.8	15.2	3	*	2.09 / 90.15
BH/MW2	92.34	20.4	15.2	3	*	2.14 / 90.20
BH/MW3	92.55	24.7	21.3	3	*	2.60 / 89.95
BH/MW4	92.12	20.3	16.8	3	*	1.94 / 90.18
BH/MW5	92.17	24.8	19.8	3	*	2.03 / 90.10

mBGS: Meter Below Ground Surface
mASL: Meter Above Sea Level
**: Not measured due to Drilling Water Introduced for Rock Coring.*

Records of groundwater observation 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.

5 Engineering Discussion and Recommendation

It is our understanding that the proposed development will comprise of a High-rise Building with 5-Levels of underground parking structure. Final design drawings of the proposed development were not available to BIG at the time of preparation of this report. Therefore, once the existing buildings are

demolished and final design drawings are available, additional investigation and analysis will be necessary and further recommendations will be provided as appropriate.

It is anticipated that the Finished Floor of 5-Levels of underground parking structure of the proposed high-rise building will be at an approximate depth of 15 mBGS, and the footings will be founded approximately between 16 and 17 mBGS (approximately elevations between 76 and 75 mASL) on Shale bedrock. However, based on very poor conditions of bedrock at few borehole locations, foundations may need to be founded on deeper depths as recommended in Foundation Options and Design Parameters section.

Considering the shallow measurements of stabilized groundwater levels in the monitoring wells varying between elevations of 90.20 and 89.95 mASLs, an appropriate temporary groundwater control system should be designed, installed and operated by an experienced dewatering contractor, prior to excavation and duration of below grade construction works, in order to preserve the structural integrity of the sidewalls and base of excavation. The hydrogeological investigation report provides details of the anticipated construction dewatering quantities and permit requirements.

The recommendations and comments provided in this report are based on factual information obtained from this preliminary investigation and are intended only to use for 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 preliminary discussion and recommendations should be revised or supplemented where necessary, when the conditions of the proposed development are different from the noted conditions/assumptions.

5.1 Grading and Site Preparation

As parts of effective and efficient design and construction of the proposed development, 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, existing fill, construction debris, soft/loose/wet materials containing organics, etc.) should be stripped and removed from the area of the proposed development.
b) It is our understanding that all existing buildings at the Site will be demolished, and the floor slabs, foundations, and underground infrastructure of the demolished buildings will be removed completely.
c) Any existing infrastructures (e.g., manholes, catch basins, buried structures, construction debris, etc.) should be sub-excavated and removed from the area of the proposed development, if they are located in the zone of influence of foundations of the proposed building. 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. Furthermore, 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 the close proximity to 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, the 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 all the times for the duration of below grade construction works, in order to preserve the structural integrity of the founding soils. 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 the footing be placed on dilated or disturbed subgrade level.

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 spots identified, should be sub-excavated and replaced with compacted approved material as directed by the geotechnical engineer.

h) Any fill, required to be used, should be used as an engineered fill. Materials used for engineered fill may consist of portion of the approved on-site soils and/or imported soils (OPSS Granular B, OPSS Select Sub-grade) which do not contain organics and deleterious materials. Some reconditioning (i.e., drying) prior to reuse may require, if the materials are found to be too wet. However, any imported soils 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 traffics. 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.

5.2 Foundation Options and Design Parameters

Based on the preliminary information obtained from the investigation, the Site is considered suitable for the construction of the proposed development from geotechnical viewpoint. Following foundation options are recommended for the design and construction of the proposed developments. The selection of the option would predominantly depend on the estimated cost, project schedule, ease of construction, etc.

OPTION-1: Conventional Strip/Spread Footings

Considering existing bedrock and groundwater conditions, the proposed building can be supported by conventional spread/strip footings with the following preliminary geotechnical bearing resistances at Serviceability Limits State (SLS) and factored geotechnical bearing resistance at Ultimate Limit State (ULS) subject to inspection and adequate groundwater control during construction:

Preliminary Geotechnical Bearing Resistances at Borehole Locations:

BH No.	Ground Elevation (m)	Approximate Footing Elevation (mASL)	Geotechnical Bearing Resistance at SLS (kPa)	Fac. Geotechnical Bearing Resistance at ULS (kPa)
BH/MW1	92.24	76 - 75	-	2000
BH/MW2	92.34	76 - 75	-	2000
BH/MW3	92.55	76 - 75	-	2000
BH/MW4	92.12	76 - 75	1000	1500
		75 or below	-	2000
BH/MW5	92.17	76 - 75	1000	1500
		75 or below	-	2000

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

It should be noted that higher bearing capacity may be available subject to review of final design drawings and additional deeper investigation and analysis after the demolition of the existing structures.

It should be noted that, in the areas, if the footing excavation has to be extended deeper than the design depth/elevation (e.g., BH/MW4 and BH/MW5) to the competent bedrock levels, then consideration may be given placing the footing on competent bedrock levels or raise the grade to the design footing level using the un-shrinkable concrete of minimum 20 MPa strength. In this case, higher factored geotechnical bearing resistance at Ultimate Limit State (ULS) can be used.

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.

A temporary groundwater control system should be in place prior to excavation and groundwater should be kept at least 1 m below the base of excavation (i.e., lowest depth of excavation). The base of excavation should be kept dry all the time for the duration of below grade construction works, in order to preserve the structural integrity of the founding bedrock. In no case should the footing be placed on disturbed bedrock subgrade.

The Shale bedrock, if left exposed, will slake. Therefore, we recommend that the foundations should be poured as soon as possible on completion of excavation, or the base of excavation should be skim coated with a lean mix concrete, minimum 75 mm thick, to level and protect the integrity of exposed subgrade.

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

Total and differential settlements for footings founded on Shale bedrock 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. However, for sound 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.

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 resistance.

OPTION-2: Raft Foundation

Alternatively, if the bearing resistance provided above is not sufficient, the proposed building may also be supported by continuous raft foundation using the following preliminary geotechnical bearing resistance at an approximate elevation of 75.0 mASL subject to inspection and adequate groundwater control during construction:

- Factored Ultimate Limit State (ULS) = 3000 kPa

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.

Groundwater (if any) should be kept at least 1 m below the base of excavation during the below grade construction works in order to preserve the structural integrity of the founding soils. In no case should the footing be placed on disturbed bedrock subgrades.

The Shale bedrock, if left exposed, will slake. Therefore, it is recommended that the foundation should be constructed as soon as possible on the completion of excavation, or the base of excavation should be skim coated with a lean mix concrete, minimum 75 mm thick, to level and protect the integrity of exposed subgrade.

A granular bed minimum 200 mm thick Granular 'A' or its equivalent, compacted to at least 98 % of SPMDD, is recommended below the raft foundation. A modulus of subgrade reaction of 40,000 kN/m^3 may be used for the design of the raft slab.

Total and differential settlements for raft footing founded on Shale bedrock and designed as outlined above should not exceed 50 and 20 mm respectively, provided that the founding subgrade is not softened by construction activities or prolonged exposure to the weather. However, for Shale bedrock, the foundation design is not governed by resistance at Serviceability Limit State (SLS) since the stress required to produce 50 mm of deformation will be much larger than the factored resistance at ULS.

Prior to the placement of concrete, foundation subgrade 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 resistance.

5.3 Floor Slab Construction

For Option-1 above, the subsoil immediately under 5th level of underground parking garage slab will consist of poor to good quality of Shale bedrock. The floor-slab on this bedrock can be designed and constructed as a conventional slab-on-grade method provided that the proper dewatering measures are in place.

A floor bedding consisting of at least 200 mm of Granular A (OPSS 1010) or its approved equivalent, is recommended to improve the support and work as a moisture barrier. The bedding should be compacted to at least 98% of SPMDD. A polyethylene vapour barrier or equivalent may be placed on the granular bedding if a moisture sensitive finish is to be placed on the floor.

The thickness of the concrete slab and the reinforcement (if any) should be designed by a structural engineer. A modulus of subgrade reaction of 40,000 kN/m^3 may be used for the design of the 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.

5.4 Permanent Perimeter and Under-floor Drainage

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 properly designed vertical shoring system should be installed to support the sides of excavation. In this case, a permanent perimeter drainage system consisting of pre-fabricated 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 pre-fabricated perimeter vertical drains with the solid collector pipe installed in a positive grade leading to a frost-free sump. The installation and connections of pre-fabricated perimeter vertical drains should be carried out as per the manufacturer's specifications.

The walls of the underground parking structure should be waterproofed/damp proofed suitably and wrapped with a continuous drainage blanket connecting to the permanent perimeter drainage system.

Underfloor drainage system may not be required, if any ingress of water under the slab is prevented. However, the need for vertical and underfloor drainage systems and the anticipated volumes of water to be pumped during and post construction should be based on the findings of the hydrogeological investigation report. The underfloor drainage system, if needed, should be kept separate from the perimeter drainage system.

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.

A conceptual design of Permanent Perimeter & Under-floor Drainage System with Shoring is shown in Appendix E.

5.5 Lateral Pressure

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

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

Where,

P	= Lateral earth pressure at depth H (m)	kPa
K_a	= Lateral earth pressure coefficient	0.4
γ	= Bulk unit weight of the soil	21 kN/m^3
H	= Depth of the wall below outer finish grade	m
q	= Equivalent value of all surcharge loads on the ground surface	kPa

The above equation assumes an effective drainage system to prevent the build-up of hydrostatic pressure behind the buried retaining structures. If a drainage system is not provided, the buried portion of retaining structures must be designed to withstand the full hydrostatic pressure.

5.6 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. As a general guidance, a 25 mm of insulation provides the same thermal equivalency as 600 mm of soil cover.

5.7 Earthquake Consideration

In conformance to the criteria in Table 4.1.8.4.A, Division B - Part 4 of the Ontario Building Code OBC 2012, the project site may be classified as Site Class “C-Very dense Soil and Soft Rock”. 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. The design values of F_a and F_v for the project site should be calculated in accordance to Table 4.1.8.4.B and C.

5.8 Excavation and Temporary Groundwater Control

The excavation for 5-Levels of underground parking structure depths is expected through the existing fills, glacial till and highly to moderately weathered shale bedrock can be handled by conventional mechanical excavation equipment. Allowance should be made for cobbles and boulders that may occur randomly in the earth fills, glacial till deposits and remnants of demolished buildings during excavation. Similarly, it is expected that excavation through the good to excellent quality of shale bedrock may also be possible by using a large hydraulic hoe or excavator equipped with rock or tiger-toothed bucket. A jackhammer or hoe ram may also be required to penetrate relatively harder zones within the bedrock. Progressively more difficult conditions should be anticipated with increasing depth of excavation as well as in areas where limestone layers are encountered. The actual equipment required and method of excavation within the bedrock will also dependent upon the geometry of the cut and relative depth of 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:

Fill soils	Type 3	When submerged/saturated	Type 4
Silty sand till (compact)	Type 3	When submerged	Type 4
Silty sand till (dense to very dense)	Type 2	When submerged	Type 3
Clayey silt till (stiff to firm)	Type 3	When saturated and/or fissured	Type 4
Clayey silt till (very stiff)	Type 2	when saturated and/or fissured	Type 3
Clayey Silt Till (hard)	Type 1	when saturated and/or fissured	Type 2
Weathered Shale	Type 1	when saturated and/or fissured	Type 2

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. 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. In general, above the water table, side slopes of trenches deeper than 1.2 m should be cut to a gradient no steeper than 1V:1H upon the inspection of a qualified geotechnical engineer. Similarly, vertical to near vertical slopes are feasible in weathered shale bedrock, subject to inspection by a professional engineer during construction.

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 an open excavation slope 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 be encountered during excavation in the earth fill and/or water bearing seams/pockets present in the glacial till deposits above the groundwater level. The amount of free water from this source is anticipated to be minor and the water accumulated in the excavation can readily be handled by using temporary filtered sumps and pumps. However, based on the water levels measured in the observation wells, it is anticipated that the excavation extending deeper than the elevation of 90.2 mASL may encounter groundwater during excavation. A temporary groundwater control system should, therefore, be in place prior to excavation and groundwater should be kept at least 1 m below the base of excavation (i.e., lowest depth of excavation), before the excavation deepens. The base of excavation should be kept dry all the time for the duration of below grade construction works, in order to preserve the structural integrity of the founding soils. The hydrogeological investigation report provides details of the anticipated construction dewatering quantities and permit requirements.

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.

5.9 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 backfill 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, a 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.

5.10 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 fill and glacial till deposits. Normal conventional excavation equipment will be suitable for excavating trenches in the fill and native soil deposits.

Within these soils, 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 5.8.

In areas where an open excavation slopes 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, surface water flow or wet seams within glacial till deposits. Dewatering should be achievable 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 soils at the base of trench for pipe bedding at their current state comprise of compact to very dense silty sand till and/or very stiff to hard silty clay till. The native soils, in their undisturbed state, are considered to be suitable for supporting the pipes, provided the integrity of the base of the trench is maintained during construction. However, the suitability of the materials should be further assessed by qualified geotechnical personnel from BIG during construction. This assessment consists of inspection and recommendation as required.

Pipe bedding should be in accordance with the pipe manufacturer's recommendations, appropriate local municipality requirements and standards (e.g., OPS). As a guideline, normal Class 'B' Type bedding (OPSD-802) may be considered. In general, a minimum of 150 mm thick base bedding and 300 mm above & adjacent to the pipes of OPSS Granular A is recommended for pipes 450 mm diameter or less; for large diameter pipes, the thickness of the base 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 19mm 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.

Based on visual and tactile examination of the soil samples, portion of the on-site excavated soils can be re-used to backfill the service trenches subject to the conditions noted in Sections 5.1 and 5.9.

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 the final subgrade of trench for the road pavement.

5.11 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. Where settlement sensitive structures are located at the close proximity of the proposed excavation, a series of caisson walls embedded sufficiently below the bottom of the excavation, will have to be used to prevent any movement in the adjacent properties. Shoring system consisting of soldier piles and timber laggings can be used, on the other sides, where slight movement in the ground surface can be tolerated, i.e., where non-sensitive structures exist.

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 (Manual). Similarly, the construction of the shoring system should also be carried out by a contractor, experienced in this type of construction.

The soldier piles should be installed in pre-augured holes which should be filled up to excavation level with 20 MPa (3000 psi) concrete and above that with 1-1/2 bag mix.

The following thicknesses of lagging boards have been recommended in the Manual:

Thickness of lagging	Maximum Spacing of Soldier Piles
50 mm (2 in)	2.0 m (6.5 ft)
75 mm (3 in)	2.5 m (8.0 ft)
100 mm (4 in)	3.0 m (10 ft)

Local experience has indicated that the lagging thickness of 75 mm has been adequate for soldier pile spacing of 3 m for soil conditions similar to those encountered at the subject site. However, it is important to consider all local conditions, such as the duration of excavation, the weather likely to be encountered, seasonal variations in the ground water and ice lensing causing frost heave in determining the lagging thickness.

All spaces behind the lagging must be filled with free draining granular fill. If wet conditions are encountered the space between boards should be packed with geotextile filter fabric or straw to prevent loss of ground.

The shoring system should be designed for a 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 Manual.

5.12 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 5.1.

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 a 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 recommended pavement structure may have to be adjusted according to the local regulations and/or respective city/town/region standards.

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.

6 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.

7 Closure

The subsoil information and recommendations contained in this report was prepared solely for the purpose to 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. *In order 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.

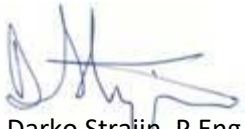
Yours truly,

B.I.G. Consulting Inc.



Subir Shrestha, M.Eng., P.Eng.
Principal Geotechnical Engineer





Darko Strajin, P.Eng.
Managing Partner

8 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 on the environmental aspects of the project, unless otherwise stated. Subsurface and groundwater conditions between and beyond the testholes may differ from those encountered at the testhole locations, and conditions may become apparent during construction which could not be detected or anticipated at the time of the site investigation. It is 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 testholes.

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.

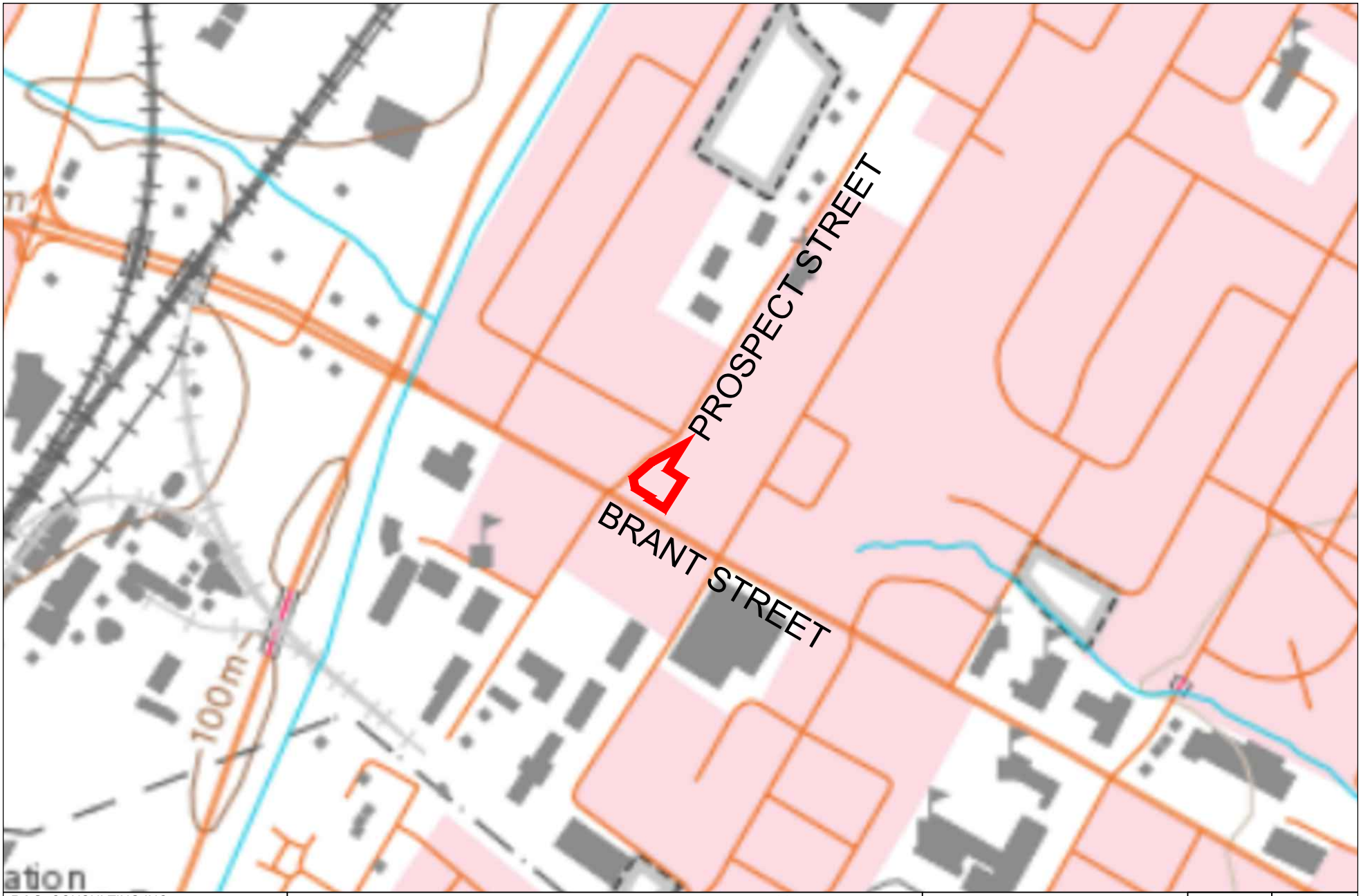
The comments made in this report relating to potential construction problems and possible methods of construction are intended only for the guidance to the designer. The number of testholes may not be sufficient to determine all the factors that may affect construction methods and costs. The anticipated 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 have an effect upon 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 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 with 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



B.I.G. CONSULTING INC.

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

bigconsultinginc.com

LEGEND

SITE BOUNDARY

SCALE

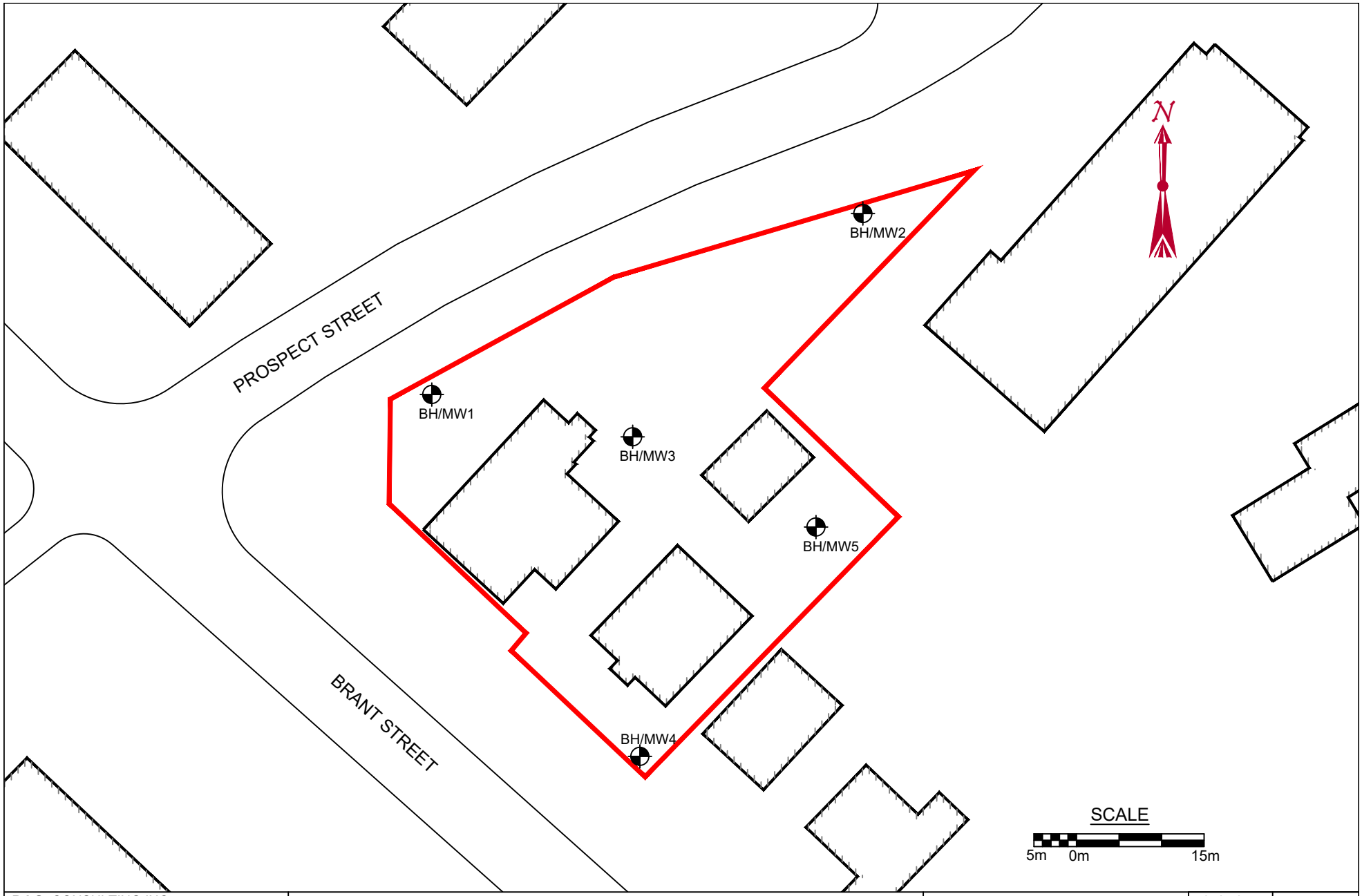
40 20 0 40 80 120 160 200

IMAGERY OBTAINED FROM TOPORAMA, NATURAL RESOURCES CANADA,2017

TITLE AND LOCATION

SITE LOCATION PLAN
PRELIMINARY GEOTECHNICAL INVESTIGATION
789-795 BRANT STREET,
BURLINGTON, ONTARIO

PROJECT NO.	DWN.
BIGC-ENV-461A	L.C.K.
SCALE	CK.
AS NOTED	S.S.
DATE	FIG NO.
JULY 2021	1



B.I.G. CONSULTING INC.

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

bigconsultinginc.com

LEGEND

- SITE BOUNDARY
- BUILDING FOOTPRINT
- BOREHOLE/MONITORING WELL

TITLE AND LOCATION

BOREHOLE/MONITORING WELL LOCATION PLAN PRELIMINARY GEOTECHNICAL INVESTIGATION

789-795 BRANT STREET,
BURLINGTON, ONTARIO

PROJECT NO.	DWN.
BIGC-ENV-461A	L.C.K.
SCALE	CK.
AS NOTED	S.S.
DATE	FIG NO.
JULY 2021	2

Appendix B – Records 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

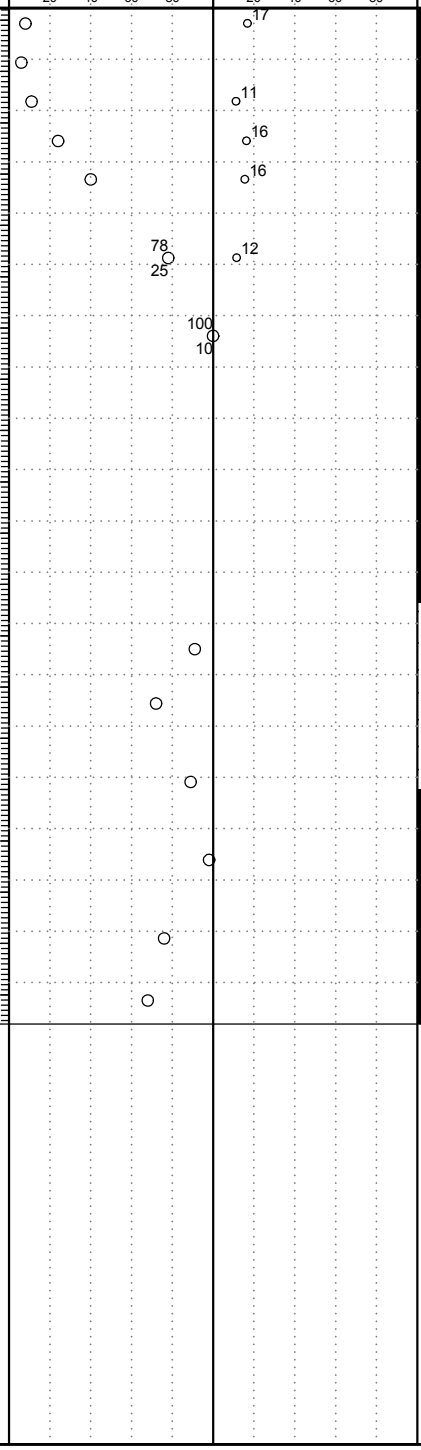
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Project Number:	BIGC-ENV-461A	Drilling Location:	See Borehole Location Plan	Logged by:	MV
Project Client:	Camarro Developments Inc.	Drilling Method:	HSA+HQ Rock Coring	Compiled by:	MV
Project Name:	Preliminary Geotechnical Investigation	Drilling Machine:	CME75	Reviewed by:	SS
Project Location:	789-785 Brant Street, Burlington, Ontario	Date Started:	14 Jun 21	Date Completed: 15 Jun 21	Revision No.: 0, 27/7/21

Geodetic Ground Surface Elevation: 92.24 m

Lithology Plot	DESCRIPTION	Sample Type	Sample Number	Recovery (%)	SPT 'N' Value/RQD%	DEPTH (m)	ELEVATION (m)
	TOPSOIL: 120mm 92.12	SS	1	67	8	0	92
	FILL: Clayey silt, trace to some sand, trace gravel, trace organics, brown, moist, firm	SS	2	0	6	1	91
	- silty sand, brown, moist to wet, compact 89.95	SS	3	100	11	2	90
	CLAYEY SILT TILL: trace to some sand, trace gravel, greyish brown, moist, very stiff to hard 2.3	SS	4	75	24	3	89
	-sandy	SS	5	100	40	4	88
	86.14	SS	6	70	78/25	5	87
	BEDROCK: Shale, highly weathered to excellen 6.1 quality, occasional limestone layers, reddish brown to grey, damp to moist	SS	7	13	100/10	6	86
		AU	8	100		7	85
		AU	9	100		8	84
		AU	10	100		9	83
		AU	11	100		10	82
	ROCK CORE BEGINS	RC	1	93	91	11	81
	- Excellent Quality - Fair Quality	RC	2	96	72	12	80
	- Good Quality	RC	3	100	89	13	79
	- Excellent Quality	RC	4	98	98	14	78
	- Good Quality	RC	5	100	76	15	77
	- Fair Quality 72.43	RC	6	100	68	16	76
	End of Borehole 19.8					17	75
						18	74
						19	73

Notes:
1. Borehole open upon completion of drilling.
2. Ground water level not measured upon completion of drilling due introduced drilling water.
3. Groundwater level reading at 2.09 m bgs on June 28, 2021.



INSTRUMENTATION INSTALLATION

COMMENTS

Gr-6%; Sa-26%; Si-48%; Cl-20%

B.I.G. Consulting Inc.
12-5500 Tomken Rd.
Mississauga, ON L4W 2Z4
Canada
T: 416-214-4880
F: 416-551-2633

▽ No freestanding groundwater measured in open borehole on completion of drilling.

▼ Groundwater depth observed on 28/06/2021 at a depth of: 2.09 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 : 147

RECORD OF BOREHOLE No. BH/MW2

Project Number:	BIGC-ENV-461A	Drilling Location:	See Borehole Location Plan	Logged by:	MV
Project Client:	Camarro Developments Inc.	Drilling Method:	HSA+HQ Rock Coring	Compiled by:	MV
Project Name:	Preliminary Geotechnical Investigation	Drilling Machine:	CME75	Reviewed by:	SS
Project Location:	789-785 Brant Street, Burlington, Ontario	Date Started:	11 Jun 21	Date Completed: 14 Jun 21	Revision No.: 0, 27/7/21

Geodetic Ground Surface Elevation: 92.34 m

Lithology Plot	DESCRIPTION	Sample Type	Sample Number	Recovery (%)	SPT 'N' Value/RQD%	DEPTH (m)	ELEVATION (m)	FIELD TESTING	LAB TESTING	INSTRUMENTATION INSTALLATION	COMMENTS
	ASPHALT PAVEMENT: 50 mm asphalt over 150 mm granular bases 0.2	SS	1	46	4	0	92				Sieve Analysis: 17, 25, (58)
	FILL: Clayey silt, some to with sand, trace to some gravel, occasional Shale fragments, brown, moist, soft to firm 90.84 1.5	SS	2	100	6	1	91		13		
	-sandy	SS	3	100	24	2	90		12		
	CLAYEY SILT TILL: trace to some sand, trace gravel, greyish brown, moist to very moist, very stiff 89.29 3.1	SS	4	92	28	3	89		22		
	SILTY SAND TILL: trace clay, trace to some gravel, occasional shale fragments, reddish brown, moist, dense 87.77	SS	5	59	24	4	88		16		
	BEDROCK: Shale, highly weathered to good quality, occasional limestone layers, reddish brown to grey, damp to moist 4.6	SS	6	13	50/8	5	87	50/8			
		SS	7	13	50/8	6	86	50/8			
		AU	8	100		7	85				
		AU	9	100		8	84				
		AU	10	100		9	83				
		AU	11	100		10	82				
	ROCK CORE BEGINS	RC	1	88	63	11	81				
	- Fair Quality					12	80				
	- Poor Quality	RC	2	96	45	13	79				
	- Good Quality	RC	3	100	79	14	78				
	- Good Quality	RC	4	100	78	15	77				
	- Fair Quality	RC	5	98	58	16	76				
	- Fair Quality 71.92	RC	6	100	67	17	75				
	End of Borehole 20.4					18	74				
						19	73				
						20	72				

Notes:
1. Borehole open upon completion of drilling.
2. Ground water level not measured upon completion of drilling due introduced drilling water.
3. Groundwater level reading at 2.14 m bgs on June 28, 2021.

B.I.G. Consulting Inc.
12-5500 Tomken Rd.
Mississauga, ON L4W 2Z4
Canada
T: 416-214-4880
F: 416-551-2633

No freestanding groundwater measured in open borehole on completion of drilling.

Groundwater depth observed on 28/06/2021 at a depth of: 2.14 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'.

RECORD OF BOREHOLE No. BH/MW3

Project Number:	BIGC-ENV-461A	Drilling Location:	See Borehole Location Plan	Logged by:	MV
Project Client:	Camarro Developements Inc.	Drilling Method:	HSA+HQ Rock Coring	Compiled by:	MV
Project Name:	Preliminary Geotechnical Investigation	Drilling Machine:	CME75	Reviewed by:	SS
Project Location:	789-785 Brant Street, Burlington, Ontario	Date Started:	10 Jun 21	Date Completed: 11 Jun 21	Revision No.: 0, 27/7/21

Geodetic Ground Surface Elevation: 92.55 m

Lithology Plot	DESCRIPTION	Sample Type	Sample Number	Recovery (%)	SPT 'N' Value/RQD%	DEPTH (m)	ELEVATION (m)
	ASPHALT PAVEMENT: 80 mm asphalt over 120 mm granular bases 0.2	SS	1	59	12	0	92
	FILL: Clayey silt, some sand, trace gravel, occasional glass pieces, dark brown/ black, moist, firm	SS	2	95	8	1	91
	- dark reddish brown 90.26	SS	3	59	6	2	90
	CLAYEY SILT TILL: trace to some sand, trace gravel, occaisonal cobble, brown, moist, very stiff 2.3 89.35	SS	4	92	24	3	89
	SILTY SAND TILL: gravelly, trace clay, occasional Shale fragments, reddish brown, moist, very dense 3.2 87.98	SS	5	100	64	4	88
	CLAYEY SILT TILL: trace to some sand, trace gravel, occasional cobble, brown, moist, hard 4.6 86.45	SS	6	41	50/13	5	87
	BEDROCK: Shale, highly weathered to poor quality, occasional limestone layers, reddish brown to grey, damp to moist 6.1	SS	7	13	50/8	6	86
		AU	8			7	85
		AU	9			8	84
		AU	10			9	83
		AU	11			10	82
	ROCK CORE BEGINS					11	81
	- Poor Quality	RC	1	100	41	12	80
	- Fair Quality	RC	2	99	67	13	79
	- Excellent Quality	RC	3	100	100	14	78
	- Excellent Quality	RC	4	99	90	15	77
	- Poor Quality	RC	5	100	47	16	76
	- Excellent Quality	RC	6	100	92	17	75
	- Excellent Quality *observed calcite precipitation at 22.42 m	RC	7	99	99	18	74
	- Fair Quality *observed calcite precipitation at 23.18 m 67.89	RC	8	100	51	19	73
	End of Borehole 24.7					20	72
						21	71
						22	70
						23	69
						24	68

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

Sieve Analysis: 1, 16, (83)

Gr-20%; Sa-41%; Si-32%; Cl-7%

Notes:
1. Borehole open upon completion of drilling.
2. Ground water level not measured upon completion of drilling due introduced drilling water.
3. Groundwater level reading at 2.60 m bgs on June 28, 2021.

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12-5500 Tomken Rd.
Mississauga, ON L4W 2Z4
Canada
T: 416-214-4880
F: 416-551-2633

▽ No freestanding groundwater measured in open borehole on completion of drilling.

▼ Groundwater depth observed on 28/06/2021 at a depth of: 2.60 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'.

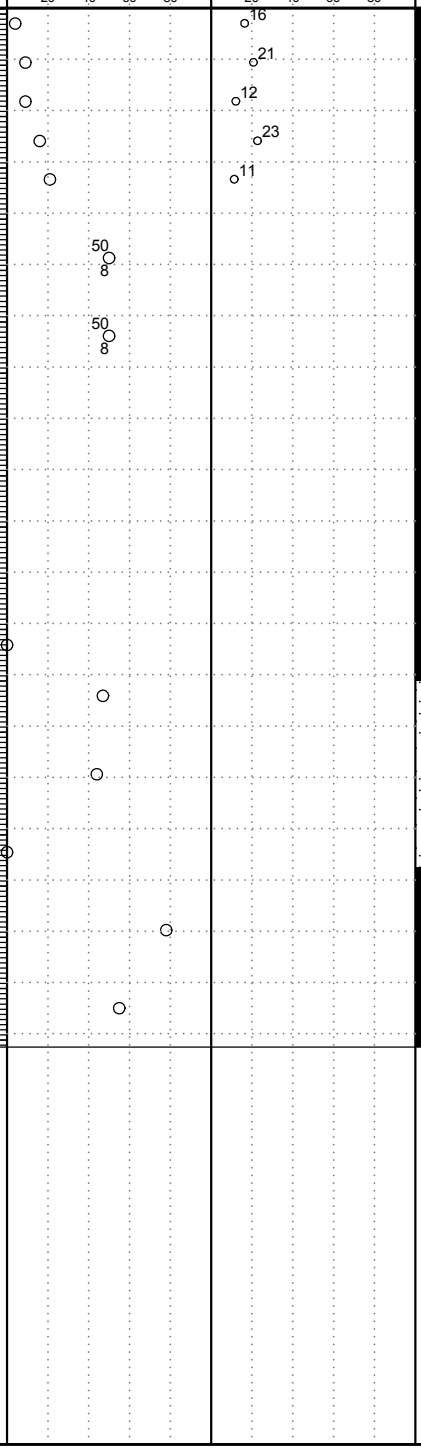
RECORD OF BOREHOLE No. BH/MW4

Project Number:	BIGC-ENV-461A	Drilling Location:	See Borehole Location Plan	Logged by:	MV
Project Client:	Camarro Developments Inc.	Drilling Method:	HSA+HQ Rock Coring	Compiled by:	MV
Project Name:	Preliminary Geotechnical Investigation	Drilling Machine:	CME75	Reviewed by:	SS
Project Location:	789-785 Brant Street, Burlington, Ontario	Date Started:	19 Jun 21	Date Completed: 19 Jun 21	Revision No.: 0, 27/7/21

Geodetic Ground Surface Elevation: 92.12 m

Lithology Plot	DESCRIPTION	Sample Type	Sample Number	Recovery (%)	SPT 'N' Value/RQD%	DEPTH (m)	ELEVATION (m)
	ASPHALT PAVEMENT: 50 mm asphalt over 150 mm granular bases 91.92 / 0.2	SS	1	92	4	0	92
	FILL: Clayey silt, trace sand, trace gravel, trace Shale fragments, greyish brown, moist, soft to stiff 90.60	SS	2	84	9	1	91
	CLAYEY SILT TILL: trace to some sand, trace gravel, brownish grey, moist to very moist, stiff to very stiff 1.5 / 89.07	SS	3	75	9	2	90
		SS	4	75	16	3	89
	SILTY SAND TILL: trace clay, trace to some gravel, occasional Shale fragments, reddish brown, moist to wet, compact 3.1 / 87.65	SS	5	75	21	4	88
	BEDROCK: Shale, highly weathered to good quality, occasional limestone layers, reddish brown to grey, damp to moist 4.5	SS	6	13	50/8	5	87
		SS	7	13	50/8	6	86
		AU	8	100		7	85
		AU	9	100		8	84
		AU	10	100		9	83
		AU	11	100		10	82
	ROCK CORE BEGINS	RC	1	100	0	11	81
	- Very Poor Quality *clay infill in weathered limestone between 12.2 & 12.7 m - Poor Quality	RC	2	100	47	12	80
	- Poor Quality *clay infill between 14.4 & 14.3 m and 14.9 & 15.4 m	RC	3	86	44	13	79
	- Very Poor Quality	RC	4	92	0	14	78
	- Good Quality	RC	5	99	78	15	77
	- Fair Quality 71.86	RC	6	99	55	16	76
	End of Borehole 20.3					17	75
						18	74
						19	73
						20	72

Notes:
1. Borehole open upon completion of drilling.
2. Ground water level not measured upon completion of drilling due introduced drilling water.
3. Groundwater level reading at 1.94 m bgs on June 28, 2021.



INSTRUMENTATION INSTALLATION

COMMENTS

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

No freestanding groundwater measured in open borehole on completion of drilling.

Groundwater depth observed on 28/06/2021 at a depth of: 1.94 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 : 147

RECORD OF BOREHOLE No. BH/MW5

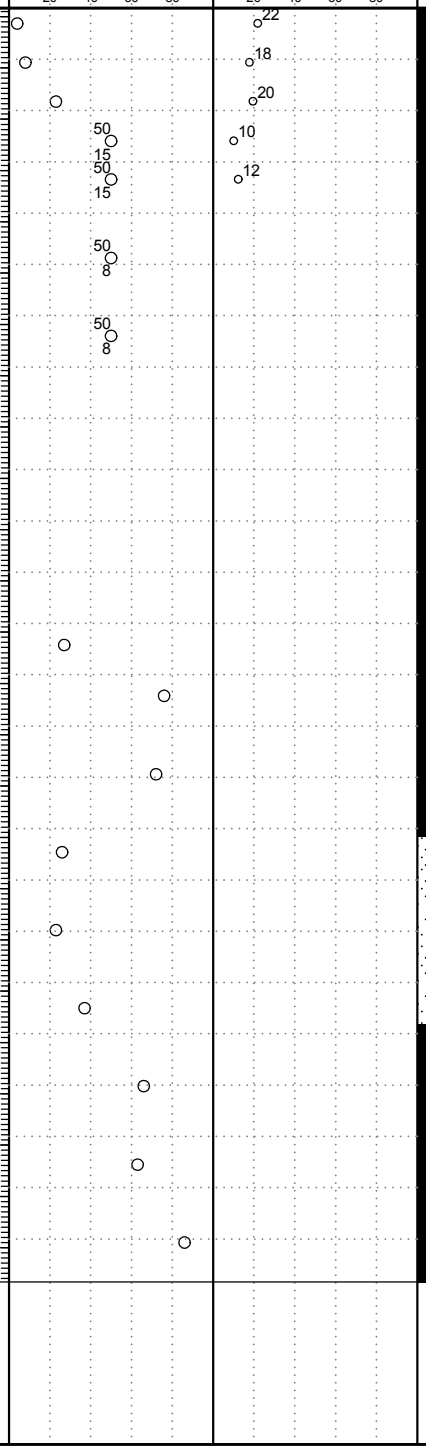
B.I.G. Consulting Inc.

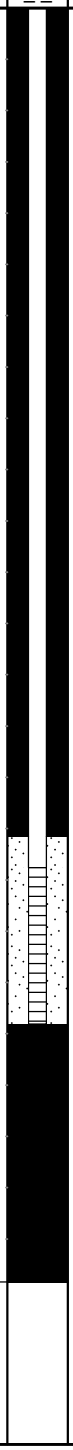
Project Number:	BIGC-ENV-461A	Drilling Location:	See Borehole Location Plan	Logged by:	MV
Project Client:	Camarro Developments Inc.	Drilling Method:	HSA+HQ Rock Coring	Compiled by:	MV
Project Name:	Preliminary Geotechnical Investigation	Drilling Machine:	CME75	Reviewed by:	SS
Project Location:	789-785 Brant Street, Burlington, Ontario	Date Started:	15 Jun 21	Date Completed: 17 Jun 21	Revision No.: 0, 27/7/21

Geodetic Ground Surface Elevation: 92.17 m

Lithology Plot	DESCRIPTION	Sample Type	Sample Number	Recovery (%)	SPT 'N' Value/RQD%	DEPTH (m)	ELEVATION (m)
	ASPHALT PAVEMENT: 50 mm asphalt over 150 mm granular bases 0.4	SS	1	84	4	0	92
	FILL: Clayey silt, trace sand, brown, moist, firm 90.67	SS	2	100	8	1	91
	CLAYEY SILT TILL: trace to some sand, trace gravel, occasional cobble, brown, moist to very moist, very stiff 89.88 1.5	SS	3	95	23	2 ▼	90
	SILTY SAND TILL: trace clay, trace to some gravel, brown, moist, very dense 88.97 2.3	SS	4	25	50/15	3	89
	CLAYEY SILT TILL: trace to some sand, trace gravel, reddish brown, moist, hard 3.2	SS	5	51	50/15	4	88
	---------- limestone fragments	SS	6	25	50/8	5	87
	86.07						
	BEDROCK: Shale, highly weathered to good quality, occasional limestone layers, reddish brown to grey, damp to moist 6.1	SS	7	25	50/8	6	86
		AU	8	100		7	85
		AU	9	100		8	84
		AU	10	100		9	83
		AU	11	100		10	82
	---------- ROCK CORE BEGINS	RC	1	85	27	11	81
	- Poor Quality Good Quality	RC	2	100	76	12	80
	- Fair Quality *clay infill between14.37 and 14.42 m	RC	3	100	72	13	79
	- Poor Quality *clay infill between 15.76 and 15.78 m	RC	4	100	26	14	78
	- Very Poor Quality *clay infill between 18.08 and 18.38 m	RC	5	100	23	15	77
	- Poor Quality *clay infill between 18.79 & 18.88 m and 19.09 & 19.19 m	RC	6	95	37	16	76
	- Fair Quality *clay infill between 20.32 and 20.36 m	RC	7	95	66	17	75
	- Fair Quality *clay infill between 23.05 and 23.17 m	RC	8	100	63	18	74
	- Good Quality 67.33	RC	9	99	86	19	73
	End of Borehole 24.8					20	72
						21	71
						22	70
						23	69
						24	68

Notes:
1. Borehole open upon completion of drilling.
2. Ground water level not measured upon completion of drilling due introduced drilling water.
3. Groundwater level reading at 2.03 m bgs on June 28, 2021.





COMMENTS

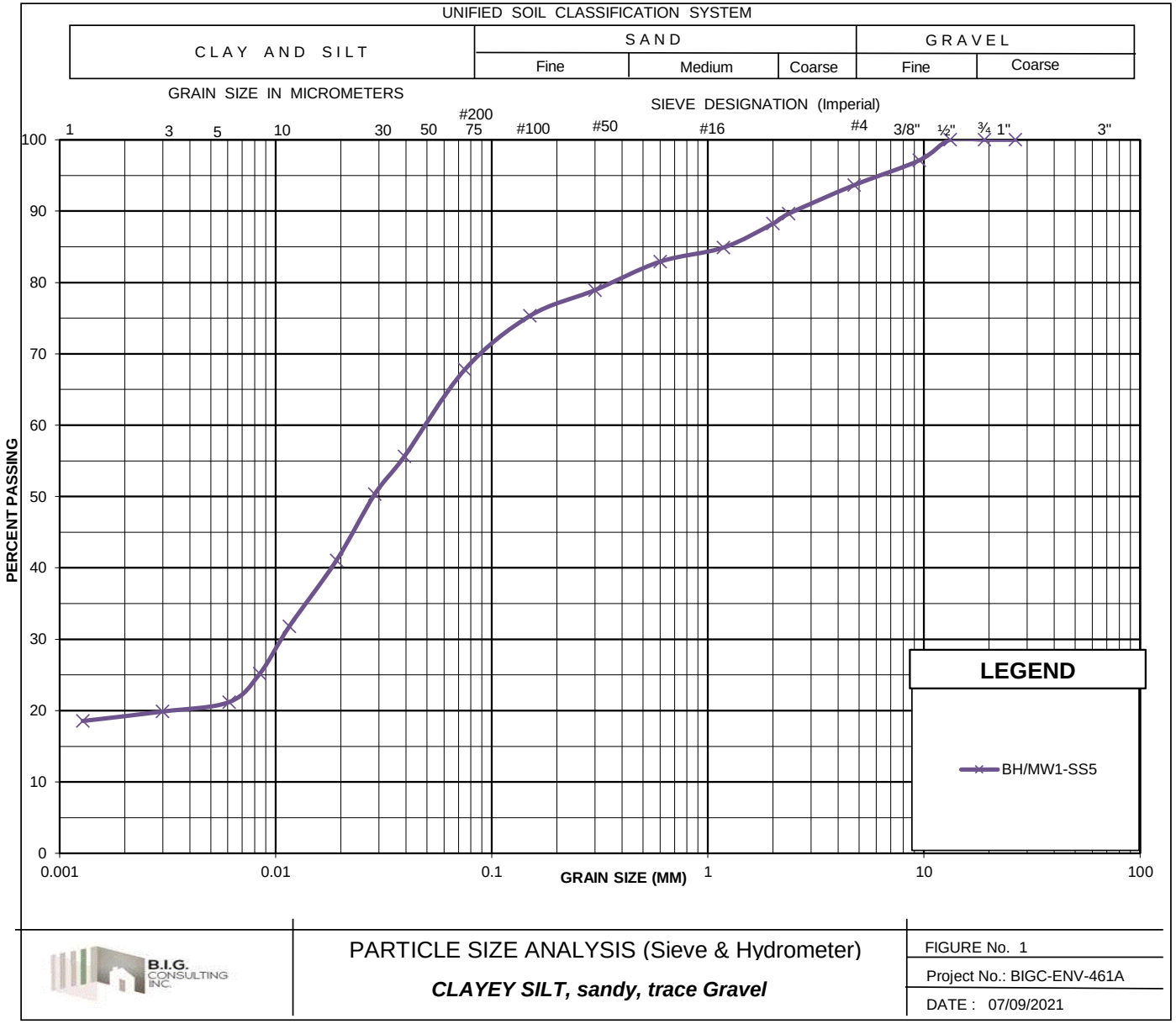
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12-5500 Tomken Rd.
Mississauga, ON L4W 2Z4
Canada
T: 416-214-4880
F: 416-551-2633

▽ No freestanding groundwater measured in open borehole on completion of drilling.

▼ Groundwater depth observed on 28/06/2021 at a depth of: 2.03 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'.

Appendix C – Geotechnical Laboratory Test Results



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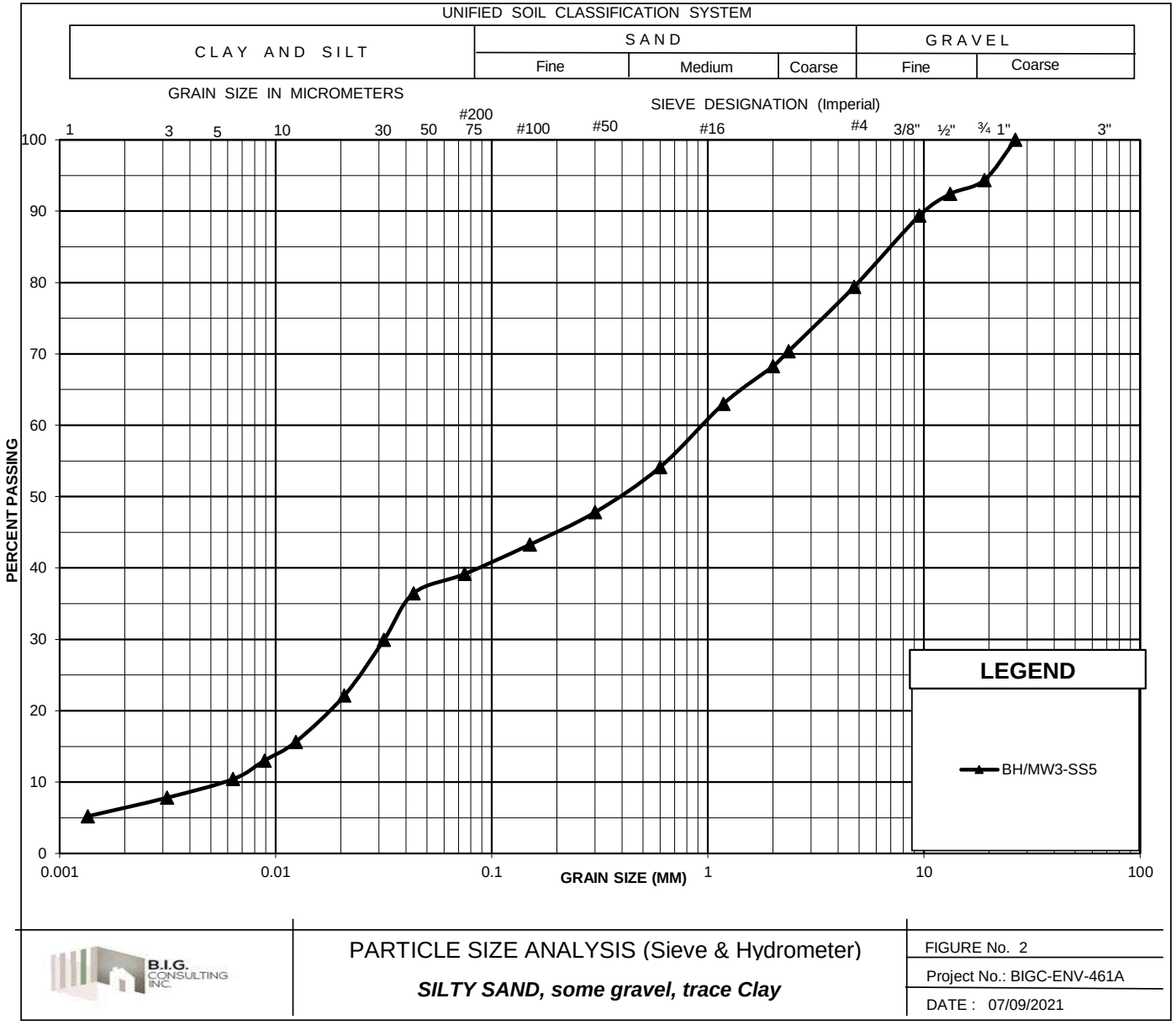
PARTICLE SIZE ANALYSIS (Sieve & Hydrometer)

CLAYEY SILT, sandy, trace Gravel

FIGURE No. 1

Project No.: BIGC-ENV-461A

DATE : 07/09/2021



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PARTICLE SIZE ANALYSIS (Sieve & Hydrometer)

SILTY SAND, some gravel, trace Clay

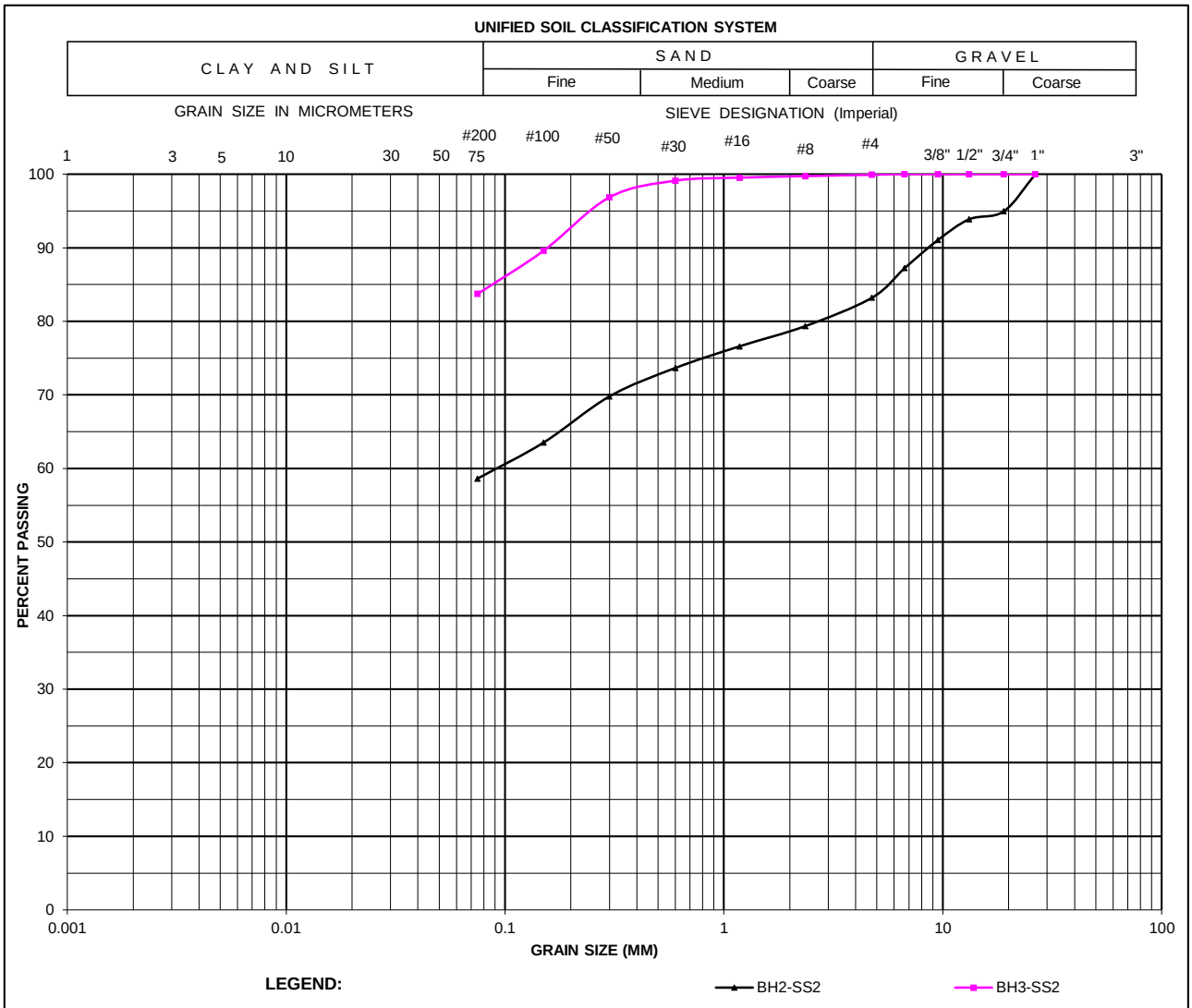
FIGURE No. 2

Project No.: BIGC-ENV-461A

DATE : 07/09/2021

PARTICLE SIZE ANALYSIS
(Sieve)

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CLAYEY SILT, some to with sand, trace to some gravel

FIGURE No.	3
Project No.:	BIGC-ENV-461A
DATE:	July 9, 2021

Appendix D - Pictures of Rock Core Samples

ROCK CORE PICTURES

BH/MW1

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-461A	**Prepared By:**	KK
Project:	Preliminary Geotechnical Investigation	**Checked By:**	SS
Location:	789-795 Brant Street, Burlington, Ontario	**Date:**	Jul. 07'21

Rock Core No.	Core Depth From (m)	Core Length (m)	Core Recovery (%)	RQD (%)
RC-1	12.20	0.61	93	92
RC-2	12.81	1.52	96	72
RC-3	14.33	1.52	100	89
RC-4	15.86	1.52	98	98
RC-5	17.38	1.52	100	76
RC-6	18.90	0.91	100	68



FIGURE- D1

ROCK CORE PICTURES
BH/MW2

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Tel: (416) 214-4880
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Project No.:	BIGC-ENV-461A	**Prepared By:**	KK
Project:	Preliminary Geotechnical Investigation	**Checked By:**	SS
Location:	789-795 Brant Street, Burlington, Ontario	**Date:**	Jul. 07'21

Rock Core No.	Core Depth From (m)	Core Length (m)	Core Recovery (%)	RQD (%)
RC-1	12.20	0.92	87	63
RC-2	13.12	1.52	96	45
RC-3	14.64	1.52	100	79
RC-4	16.17	1.52	100	78
RC-5	17.69	1.52	97	58
RC-6	18.90	1.52	100	67



FIGURE- D2

ROCK CORE PICTURES
BH/MW3

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Project No.:	BIGC-ENV-461A	**Prepared By:**	KK
Project:	Preliminary Geotechnical Investigation	**Checked By:**	SS
Location:	789-795 Brant Street, Burlington, Ontario	**Date:**	Jul. 07'21

Rock Core No.	Core Depth From (m)	Core Length (m)	Core Recovery (%)	RQD (%)
RC-1	12.50	1.52	100	40
RC-2	14.02	1.52	99	78
RC-3	15.55	1.52	100	100
RC-4	17.07	1.52	99	90
RC-5	18.60	1.52	100	47
RC-6	20.12	1.52	100	92
RC-7	21.64	1.52	99	99
RC-8	23.14	1.52	100	51



FIGURE- D3

ROCK CORE PICTURES

BH/MW4

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

Project No.:	BIGC-ENV-461A	**Prepared By:**	KK
Project:	Preliminary Geotechnical & Hydrogeological Investigation	**Checked By:**	SS
Location:	789-795 Brant Street, Burlington, Ontario	**Date:**	Jul. 07'21

Rock Core No.	Core Depth From (m)	Core Length (m)	Core Recovery (%)	RQD (%)
RC-1	12.19	0.46	113	0
RC-2	12.65	1.52	100	47
RC-3	14.17	1.52	85	44
RC-4	15.70	1.52	92	0
RC-5	17.22	1.52	99	78
RC-6	18.74	1.52	99	55



FIGURE- D4

ROCK CORE PICTURES
BH/MW5

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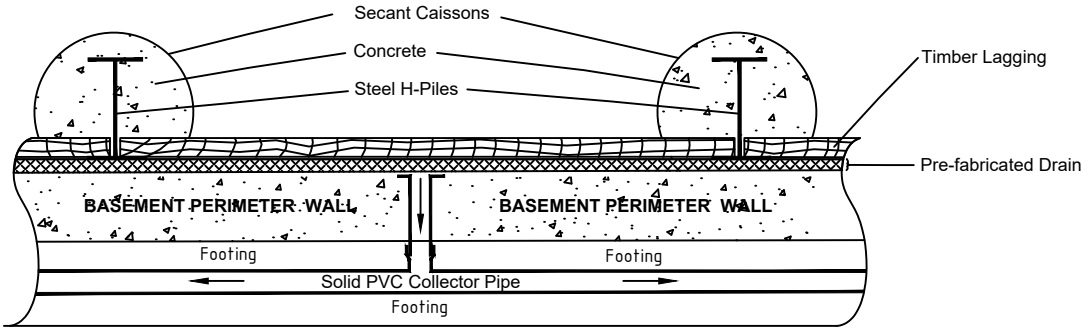
Project No.:	BIGC-ENV-461A	**Prepared By:**	KK
Project:	Preliminary Geotechnical & Hydrogeological Investigation	**Checked By:**	SS
Location:	789-795 Brant Street, Burlington, Ontario	**Date:**	Jul. 07'21

Rock Core No.	Core Depth From (m)	Core Length (m)	Core Recovery (%)	RQD (%)
RC-1	12.19	0.41	95	27
RC-2	12.60	1.52	69	76
RC-3	14.12	1.52	98	72
RC-4	15.65	1.52	100	36
RC-5	17.17	1.52	100	47
RC-6	18.70	1.52	95	37
RC-7	20.22	1.52	95	66
RC-8	21.74	1.52	100	63
RC-9	23.32	1.52	99	87

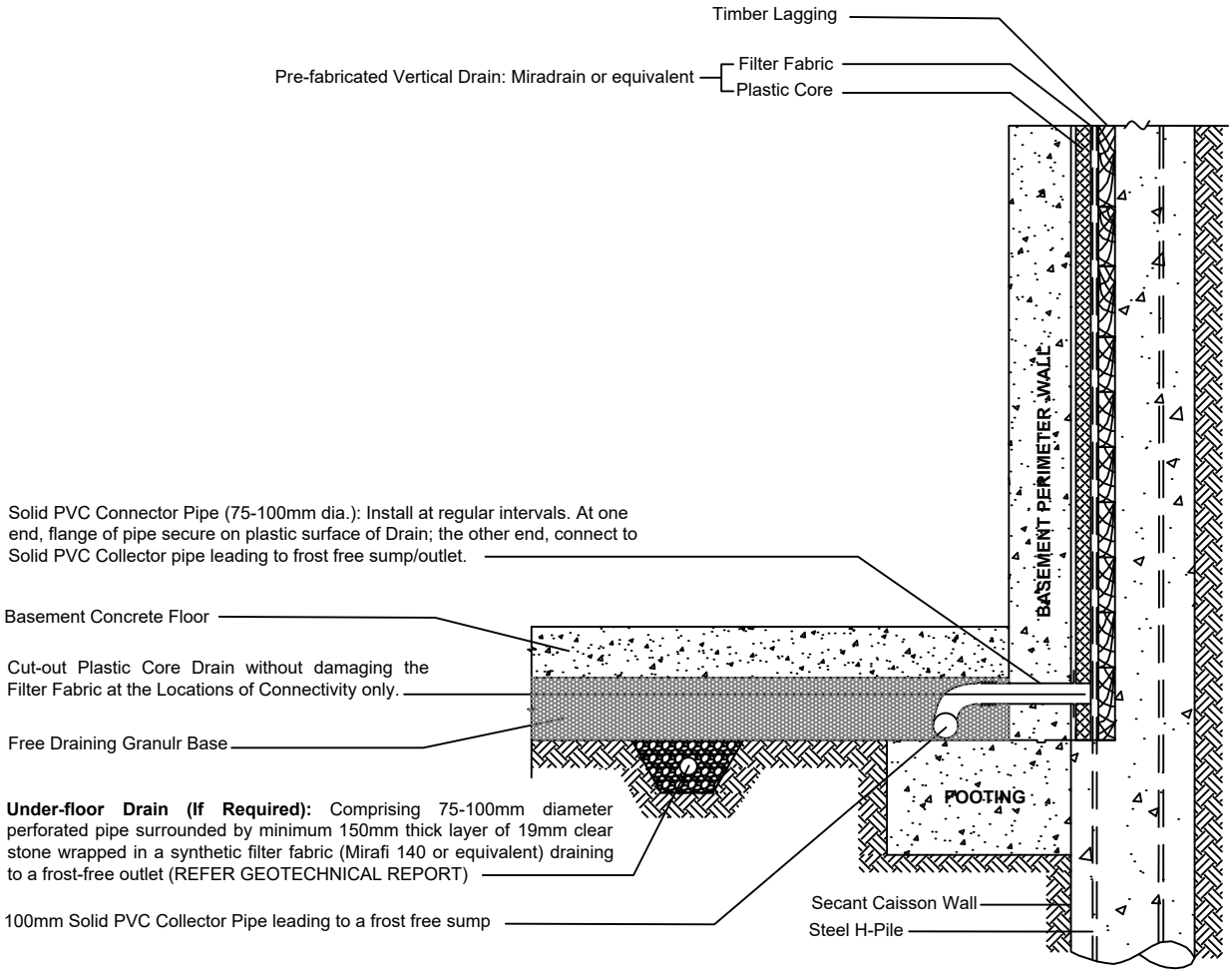


FIGURE- D5

Appendix E - Conceptual Permanent Perimeter and Underfloor Drainage Systems with Shoring



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 flter 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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t: (416) 214 - 4880 f: (416) 551 - 2633
12-5500 Tomken Rd.
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Canada

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LEGEND

TITLE AND LOCATION

PERMANENT PERIMETER & UNDER-FLOOR DRAINAGE SYSTEM
(CONCEPTUAL-FOR SHORING)
PRELIMINARY GEOTECHNICAL INVESTIGATION
789-795 BRANT STREET,
BURLINGTON, ONTARIO

PROJECT NO.	DWN.
BIGC-ENV-461A	O.A.
SCALE	CK.
AS NOTED	S.S.
DATE	FIG NO.
JUly 2021	E