PRELIMINARY GEOTECHNICAL INVESTIGATION

581-587 Argus Road
Oakville, ON

Client

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Project Number

BIGC-GEO-490A

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1 Introduction

B.I.G. Consulting Inc. (BIG) has been retained by Oakville Argus Cross LP (the "Client") to complete a preliminary geotechnical investigation for the proposed development on the property located at 581-587 Argus Road, Oakville, Ontario (the "Site"). The Site location plan is shown on Figure 1 in Appendix A.

The investigation was authorized by Mr. Marcus Boekelman on behalf of the Client.

It is our understanding that the proposed development at the Site will comprise of a high-rise residential building with 6-Levels of underground parking structure.

The field work for this investigation was carried out in conjunction with Preliminary Hydrogeological Investigation (HG). This report addresses the geotechnical aspects of the proposed development only and the reports for the HG will be issued under separate cover.

The purpose of this preliminary geotechnical 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 of the proposed development. Final design drawings of the proposed development were not available to BIG at the time of preparation of this report. Once the final design drawings are available and the existing buildings are demolished, additional investigation and analysis will be necessary and further recommendations will be provided as appropriate.

The recommendations and comments are based on factual information 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 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 subject Site is 581-587 Argus Road in Oakville, Ontario. The Site is bound from the north and west by Argus Road, and from east and south by commercial properties.

The Site measures approximately 3,800 m^2 in area and currently occupied by two commercial buildings. The areas surrounding the buildings are covered with asphalt pavements and landscaped areas.

The topography of the Site was generally flat with an elevation difference of approximately 0.9 m between the borehole locations.

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

The fieldwork for this investigation was carried out between October 6 and 8, 2021 and consisted of advancing a total of 5-Boreholes equipped with Monitoring Wells, BH/MW1 to BH/MW5 that were drilled to the depths varying between 4.9 and 27.6 m below the existing ground surface (mBGS). It should be noted that, to confirm the presence and quality of bedrock, below the auger termination depths in boreholes BH/MW2 and BH/MW5 at 7.32 mBGS, bedrock was cored using wire line diamond coring method to the depths of 27.6 and 25.3 mBGS, respectively. The approximate borehole locations established and drilled at the Site are shown on Figure 2 in Appendix A.

The boreholes were advanced by using truck mounted, power operated solid 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.

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 5-Boreholes were equipped with Monitoring Wells upon completion of drilling. The details of the groundwater observation are presented on Section 4.5.

The ground surface elevations at borehole locations were surveyed by BIG personnel with reference to a Temporary Benchmark location (BH/MW109 as shown on BIG Geotechnical Investigation Report BIGC-ENV-349B) on the asphalt paved parking lot located at 571 Argus Road with a Geodetic Elevation of 102.890 mASL.

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 Location Plan are based on the measurements of the Site features and should be considered approximates.

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 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 Asphalt pavement overlying existing fills, which in turn was underlain by native 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 Asphalt Pavement

All boreholes were advanced through existing asphalt pavement consisting approximately 50 to 70 mm thick asphaltic concrete over 100 to 200 mm thick granular bases.

4.2 Existing Fills

Below the asphalt pavement, existing fills generally consisting of clayey silt/silty clay and silty sand with trace gravel was encountered in all boreholes that extended to depths varying between 0.9 and 1.5 mBGS. Existing fill also contained fragments of Shale and occasional cobbles.

The SPT 'N' values recorded were generally varying from 5 to over 50 blows per 300 mm of penetration indicating stiff to hard consistencies. The moisture content measurement of the recovered samples generally varied between 9 and 18 % by weight, indicating a moist condition.

4.3 Clayey Silt Till

Below existing fills in all boreholes, a glacial deposit of native clayey silt till was encountered that extended to depths varying between 2.3 and 2.6 mBGS. Clayey silt till deposit also contained trace to some sand, trace gravel and occasional Shale fragments.

The SPT 'N' values recorded in this till deposit varied from 14 to over 50 blows per 300 mm of penetration indicating stiff to hard consistencies. The moisture content measurement of the recovered samples was between 8 and 12 % by weight, indicating a moist condition.

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

4.4 Shale Bedrock

Below clayey silt till, a highly weathered to excellent quality of Georgian Bay Formation grey Shale bedrock was encountered in all boreholes. All boreholes were drilled into the Shale bedrock and sampled up to the borehole termination depths between 4.9 and 7.7 mBGS. First water strike was also recorded in all boreholes between 4.8 and 7.1 mBGS.

The SPT 'N' values recorded were generally over 50 blows per 300 mm of penetration, indicating a hard consistency. The moisture content observations were moist to damp conditions.

Further, to confirm the presence and quality of bedrock, from the auger termination depths of 7.32 mBGS in boreholes BH/MW2 and BH/MW5; HQ size bedrock samples were cored to the depths of 27.6 and 25.3 mBGS, respectively. The percentage of recoveries of rock core samples were between 83% to 100%, and the Rock Quality Designation (R.Q.D.) values were found generally between 61% and 100%, indicating fair to excellent quality of rock mass. Within the Shale bedrock core samples, occasional interbedded Limestone layers were also present.

4.5 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, sampling and installation of monitoring wells operations.

In order to obtain the 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 level recorded in the installed monitoring wells on October 18, 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	October 18, 2021
BH/MW1	104.53	7.7	7.0	3	Dry	4.38/100.15
BH/MW2	104.24	27.6	15.2	3	*	9.05/95.19
BH/MW3	104.37	4.9	4.9	3	4.72	4.24/100.13
BH/MW4	103.61	7.3	7.3	3	7.01	4.71/98.90
BH/MW5	103.75	25.3	22.8	3	*	19.04/84.71

mBGS: Meter Below Ground Surface
mASL: Meter Above Sea Level
**: Not measured due to introduced Drilling Water.*

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 at the Site will comprise of a high-rise residential building with 6-Levels of underground parking structure.

Based on preliminary design drawings prepared by Quadrangle Architects Limited, dated 2022-04-19 Issued for Rezoning, it is anticipated that the FFE of the proposed building with 6-Levels of underground parking structure will be located at an approximate elevation of 82 mASL. Once the final design drawings are available and the existing building are demolished, additional investigation and analysis will be necessary and further recommendations will be provided as appropriate.

The comments and recommendations presented in this report are based on factual information and intended only to use for the design engineers. 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 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 new 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.

5.1 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, 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 (pavement, topsoil, etc.) should be stripped and removed from the area of the proposed development. Similarly, all existing shrubs, vegetation, trees and scattered debris should be removed from the area of the proposed development.

b) It is our understanding that the existing building at the site will be demolished and removed. Similarly, all components of the demolished building, e.g., floor slabs, footings, walls and underground infrastructure, etc. should be sub-excavated and removed completely from the area of the proposed development.

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 in 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. 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 the excavation near the vicinity of the existing structures and any underground utility services 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) A provision of temporary groundwater control system should be available during the excavation, and the base of excavation should be kept dry all the time.

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 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 imported OPSS Granular B, OPSS Select Sub-grade and/or the on-site soils which do not contain organics and deleterious materials. Some reconditioning (i.e., drying) prior to re-use 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), before placing the next lift.

j) Portions of the excavated on-site soils which do not contain organics, debris and deleterious materials can be reused for backfills. Some reconditioning (i.e., drying) prior to reuse may require subject to weather condition.

k) The existing on-site soils are susceptible to disturbance when exposed to weather and construction traffic. Surface water runoff from the neighboring properties should not be permitted to enter and/or pond within the construction area. This is especially important to the success of the planned construction.

5.2 Foundation Options and Design Parameters

Based on the information obtained from the investigation, the Site is considered suitable for the construction of the proposed development from the geotechnical viewpoint.

Based on preliminary design drawings prepared by Quadrangle Architects Limited, it is anticipated that the foundations of the proposed building with 6-Levels of underground parking structure will be located at an approximate elevation of 81 mASL. The subsurface bedrock conditions at and below this depth, based on this investigation, is fair to excellent quality of Shale bedrock.

Considering the subsurface bedrock and groundwater conditions, following foundation options are recommended for the design and construction of the proposed development.

5.2.1 Option-1: Conventional Strip/Spread Footings

The proposed buildings may be supported by conventional spread/strip footings using the following preliminary geotechnical bearing resistance subject to inspection and adequate groundwater control during construction:

- Factored Ultimate Limit State (ULS) = 3500 kPa

It should be noted that higher bearing capacity may be available subject to additional investigation and analysis.

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 provision of temporary groundwater control system should be available during the excavation, and the base of excavation should be kept dry all the time. In no case should the footing be placed on dilated or disturbed subgrade of bedrock.

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 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.

It is well recognized that the Shale rocks found in Southern Ontario exhibit time dependent deformation (TDD) when stress change (i.e., deep excavation) occurs. In addition to the stress induced deformations, Shale may also exhibit swelling potential (rock squeeze). Allowance should, therefore, be made for these long-term characteristics of the Shale bedrock. It may be necessary to apply a layer of sprayed foam, minimum of 50 mm in thickness, separating the exposed bedrock face and exterior side of underground parking structure walls. It is recommended that a decision in this regard be made at the time of excavation.

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

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 bedrock are similar to those identified in the investigation and capable of supporting the design bearing resistance.

5.2.2 Option-2: Tanked Raft Foundation

In the event that the long-term foundation drainage for Option 1 above is not allowed to discharge into the City's sewer systems and/or if the bearing resistance provided above is not sufficient, the proposed buildings may also be designed and supported by Tanked water-proofed continuous raft foundation system avoiding permanent dewatering (i.e., avoiding permanent perimeter and under-floor drainage system) using the following geotechnical bearing resistances subject to inspection and adequate groundwater control during construction:

- Factored Ultimate Limit State (ULS) = 5000 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.

The groundwater should be kept at least 1 m below the base of excavation (i.e., lowest depth of excavation), and the base of excavation should be dry in order to preserve the structural integrity of the founding rock/soils. In no case should the footing be placed on dilated or disturbed rock/soil subgrades.

Provided the subgrade will be prepared to a uniform high density and subsequent inspection and approval by the geotechnical engineer as per Section 5.1 above, a layer of minimum 200 mm thick Granular A (OPSS 1010) or its approved equivalent, compacted to at least 98 % of its SPMDD, is recommended below the raft slab foundation. A modulus of subgrade reaction of 50,000 kN/m^3 may be used for the design of the raft slabs.

The foundation subgrade should be protected, immediately after excavation and inspection, 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.

A minimum gap of approximately 600 mm service space should be kept between the top of the raft slab and the parking level floor slab to allow for the installation and maintenance of drainpipes, sewers and any other underground utilities. The service space between the utilities may be filled with clear stone up to the underside of parking floor slab.

Designer needs to consider full hydrostatic forces acting against the perimeter basement walls and uplift pressure on the raft foundation. A high-water level approximately at the existing ground surface may be considered in the structural design purpose. Similarly, waterproofing of the raft slab and the underground perimeter walls is recommended. The manufacturer of the selected product should be consulted for their application.

It is well recognized that the Shale bedrocks found in Southern Ontario exhibit time dependent deformation (TDD) when stress change (i.e., deep excavation) occurs. In addition to the stress induced deformations, Shale may also exhibit swelling potential (rock squeeze). Allowance should, therefore, be made for these long-term characteristics of the Shale bedrock. It may be necessary to apply a layer of sprayed foam, minimum of 50 mm in thickness, separating the exposed bedrock face and exterior side of underground parking structure walls. It is recommended that a decision in this regard be made at the time of excavation.

Total and differential settlements of raft foundation founded on the competent native soils and designed as outlined above should not exceed 50 and 20 mm respectively, provided that the founding subgrade is not loosened or softened by construction activities or prolonged exposure to the weather.

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 borehole/core-hole and are capable of supporting the design bearing resistance.

5.3 Floor Slab Construction

The subsurface bedrock under 6th level of underground parking floor-slab is anticipated to be fair to excellent quality of Shale bedrock. If the foundation option-1 above is adopted, the floor-slab on these materials can be designed and constructed as a conventional slab-on-grade method provided that the proper dewatering measures are in place.

Floor bedding consisting of at least 200 mm of Granular A (OPSS 1010) or its approved equivalent, is recommended under the floor slab. The bedding should be compacted to at least 98% of SPMDD. 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. A modulus of subgrade reaction of 50,000 kN/m^3 may be used for the design and construction of the 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 designer.

5.4 Permanent Perimeter and Under-floor Drainage

If the foundation option -1 above is adopted, 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 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 pipes 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.

Under-floor drainage system may not be required at this Site provided 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.

Conceptual details of Permanent Perimeter and Underfloor Drainage Systems for Vertical Shoring are shown in Appendix C.

5.5 Lateral Earth Pressure

The lateral earth pressures acting on retaining structures (perimeter underground parking structure 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 = Lateral earth pressure coefficient	0.4
	γ = Bulk unit weight of the soil	21.0 kN/m^3
	H = Depth of the wall below the outer finish grade	m
	q = Equivalent value of all surcharges 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 (i.e., foundation option-1 above is adopted). If a drainage system is not provided, the buried portion of retaining structures must be designed to withstand the full hydrostatic pressure (i.e., foundation option-2 above is adopted).

Surcharge and point loads at the ground surface (e.g., from the heavy construction equipment, etc.) should also be considered in the structural design of retaining structures.

5.6 Frost Protection

The design frost penetration depth for the general Site area is 1.2 m. 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-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 with Table 4.1.8.4.B and C.

5.8 Excavation and Temporary Groundwater Control

The excavation through the existing fills, glacial till and highly to moderately weathered Shell 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 and glacial till soils. Similarly, it is expected that excavation through the fair to excellent conditions of shale bedrock may 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 be 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
Clayey Silt Till (firm to stiff)	Type 3;	When saturated	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 1 & 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 in the earth fill and upper portion of the Shale bedrock above the groundwater level (e.g., first water strike). 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. However, 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 backfilling as an engineered fill. However, depending upon the weather conditions, the excavated soils may require some reconditioning prior to reuse, i.e., maintain the moisture content close to proctor value to obtain the required minimum compaction.

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 their optimum moisture content values (i.e., moisture content at Standard Proctor Maximum Dry Density (SPMDD)), and thoroughly compacted with suitable heavy compactors to at least 98% of SPMDD for foundation support, before placing the next lift. In areas that underlie pavements and service trench, backfill should be compacted to at least 95% of SPMDD. In landscaped areas, compaction to 90% of SPMDD will be adequate.

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

Considering this investigation, the on-site soils are not considered to be free draining. The clayey soils will likely be excavated in cohesive blocks and will be difficult to handle and compact. To re-use for backfilling, the cohesive blocks will have to be reduced to smaller than 100 mm in size and placed in thin layers/lifts, provided their moisture contents are at or near the optimum moisture content (i.e., Proctor moisture values). The clayey soils will have to be compacted sufficiently using a suitable heavy equipment which may be difficult to operate in the narrow confine areas. Unless the clay soils are properly reduced in size and compacted sufficiently in thin lifts, post construction settlements could occur. Therefore, if the use of heavy equipment for compaction of the clayey soils is not possible in settlement sensitive areas and narrow confined areas (e.g., trenches), free draining on-site and/or imported granular soils (i.e., OPSS Select Subgrade Material or approved equivalent) should be used for backfilling and compacted adequately with suitable equipment.

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, glacial till deposits and highly weathered shale bedrock. Normal conventional excavation equipment will be suitable for excavating trenches within these materials.

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 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, surface water flow or wet seams within glacial 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/bedrock.

The boreholes show, the anticipated subgrade soil/rock at the base of trench for pipe bedding may comprise of stiff to hard glacial till and/or highly weathered Shale bedrock. These soils, in their undisturbed state, provide adequate support for the pipes, provided the exposed subgrades are further assessed and approved by qualified geotechnical personnel from BIG during construction.

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 and/or if weak or wet subgrade conditions are encountered. Subject to assessment by the geotechnical engineer on Site, the bedding used to support the pipes in week soils (if any) may need to be wrapped by a geotextile filter (e.g., Terrafix 270R or equivalent). Further, 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 5.1 and 5.9. The backfill 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.

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, shoring system consisting 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.

Following the Site grading and prior to the placement of granular bases, the exposed subgrade should be proof-rolled and inspected by 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 compatible to the existing subgrade soils.

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 800 mm of the pavement subgrade should be compacted to a minimum 98% of SPMDD.

Considering the proposed pavement usage, frost susceptibility and assuming adequate drainage, 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

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 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 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.

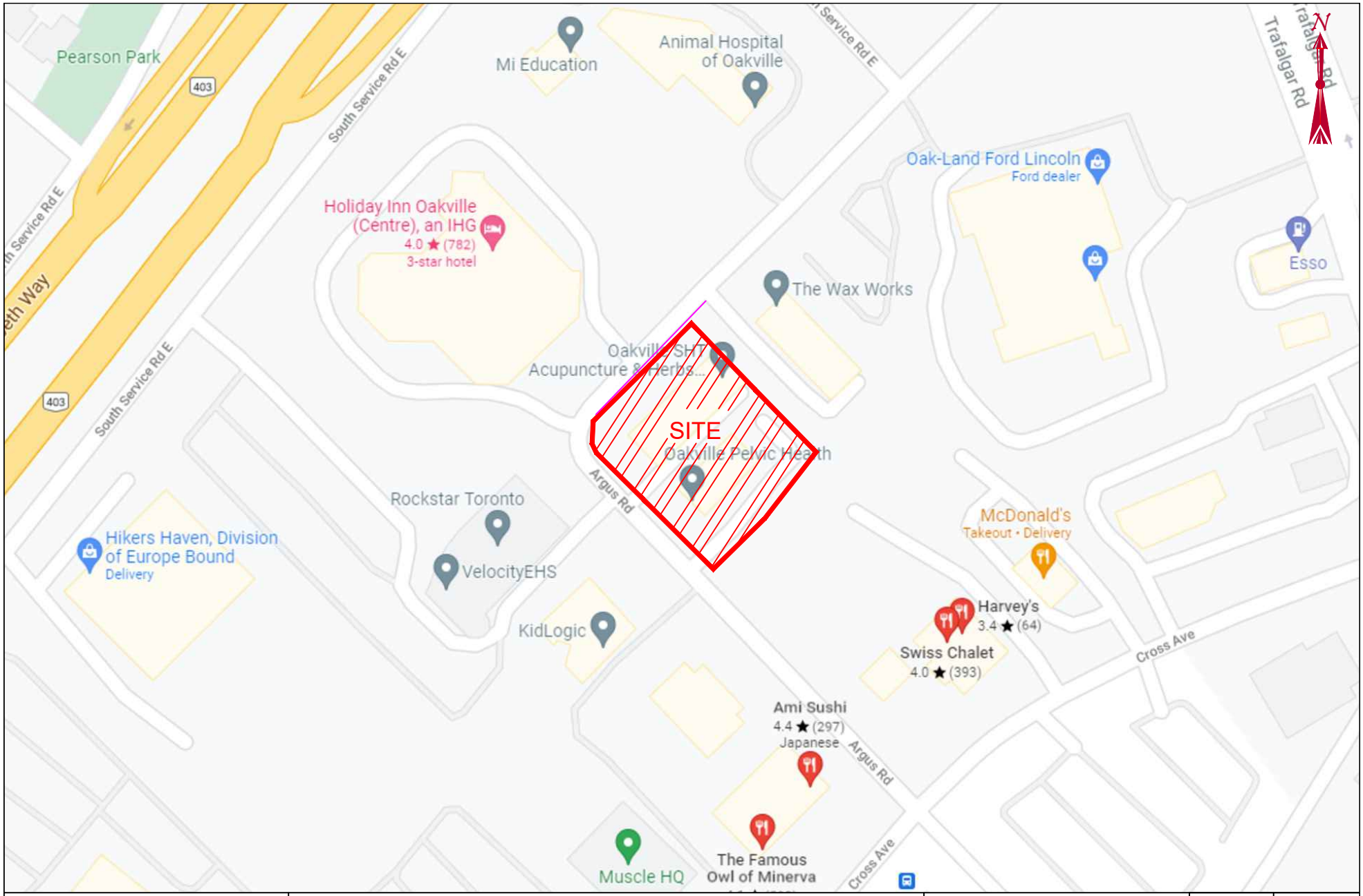
The comments made in this report relating to potential construction problems and possible methods of construction are intended only for the guidance to the designer. The number of testholes may not be sufficient to determine all the factors that may affect construction methods and costs. The anticipated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



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LEGEND

- SITE BOUNDARY
- EXISTING BUILDING FOOTPRINT
- LOCATION OF BOREHOLE/MONITORING WELL

SCALE

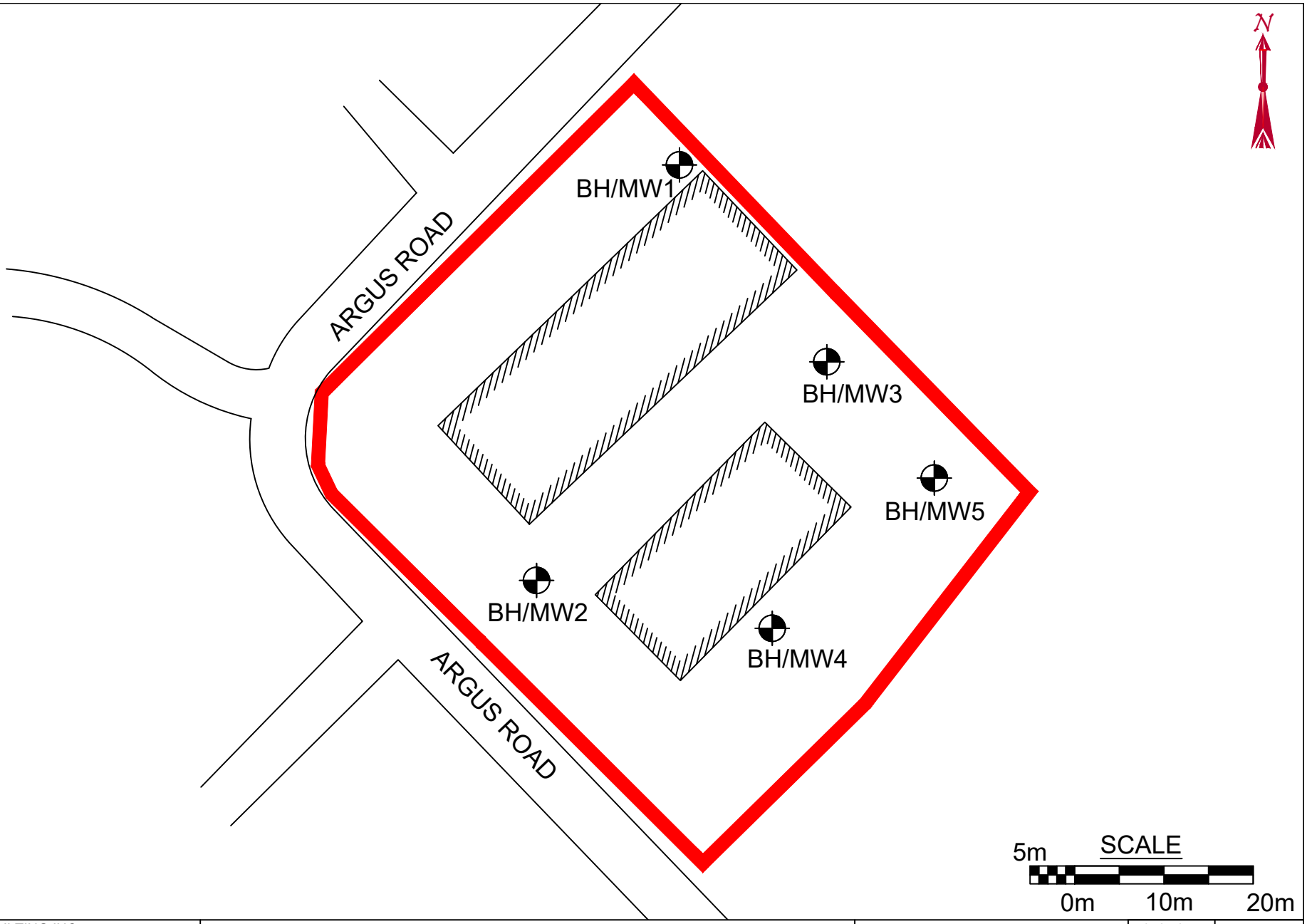
10m 10 20 30 40
0m 50m

IMAGERY OBTAINED FROM GOOGLE MAPS, 2021

TITLE AND LOCATION

SITE LOCATION PLAN
PRELIMINARY GEOTECHNICAL INVESTIGATION
581-587 ARGUS ROAD,
OAKVILLE, ONTARIO

PROJECT NO.	DWN.
BIGC-GEO-490A	O.A.
SCALE	CK.
AS NOTED	S.S.
DATE	FIG NO.
OCTOBER 2021	1



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LEGEND

- SITE BOUNDARY
- EXISTING BUILDING FOOTPRINT
- LOCATION OF BOREHOLE/MONITORING WELL

TITLE AND LOCATION

BOREHOLE/MONITORING WELL LOCATION PLAN
PRELIMINARY GEOTECHNICAL INVESTIGATION
581-587 ARGUS ROAD,
OAKVILLE, ONTARIO

PROJECT NO.	DWN.
BIGC-GEO-490A	S.M.
SCALE	CK.
AS NOTED	S.S.
DATE	FIG NO.
OCTOBER 2021	2

Appendix B – Records of Boreholes

NOTES TO RECORD OF BOREHOLES

DRILLING METHOD

SSA	Solid Stem Auger
HSA	Hollow Stem Auger
WB	Wash Boring

SAMPLE TYPE

SS	Split Spoon
AS	Auger Flight Sample
TW	Thin Wall Open
TP	Thin Wall Piston
WS	Washed Sample
VT	Vane Test
GS	Grab Sample
RC	Rock Core
PH	Sample Advanced Hydraulically
PM	Sample Advanced Manually
CC	Continuous Core

LABORATORT DATA

W	Water Content
W_p	Plastic Limit
W_l	Liquid Limit
γ	Natural Unit Weight (kN/m^3)
C_u	Undrained Shear Strength (kPa)
PP	Pocket Penetrometer
UC	Unconfined Compression
UU	Unconsolidated Undrained
CU	Consolidated Undrained
CD	Consolidated Drained
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

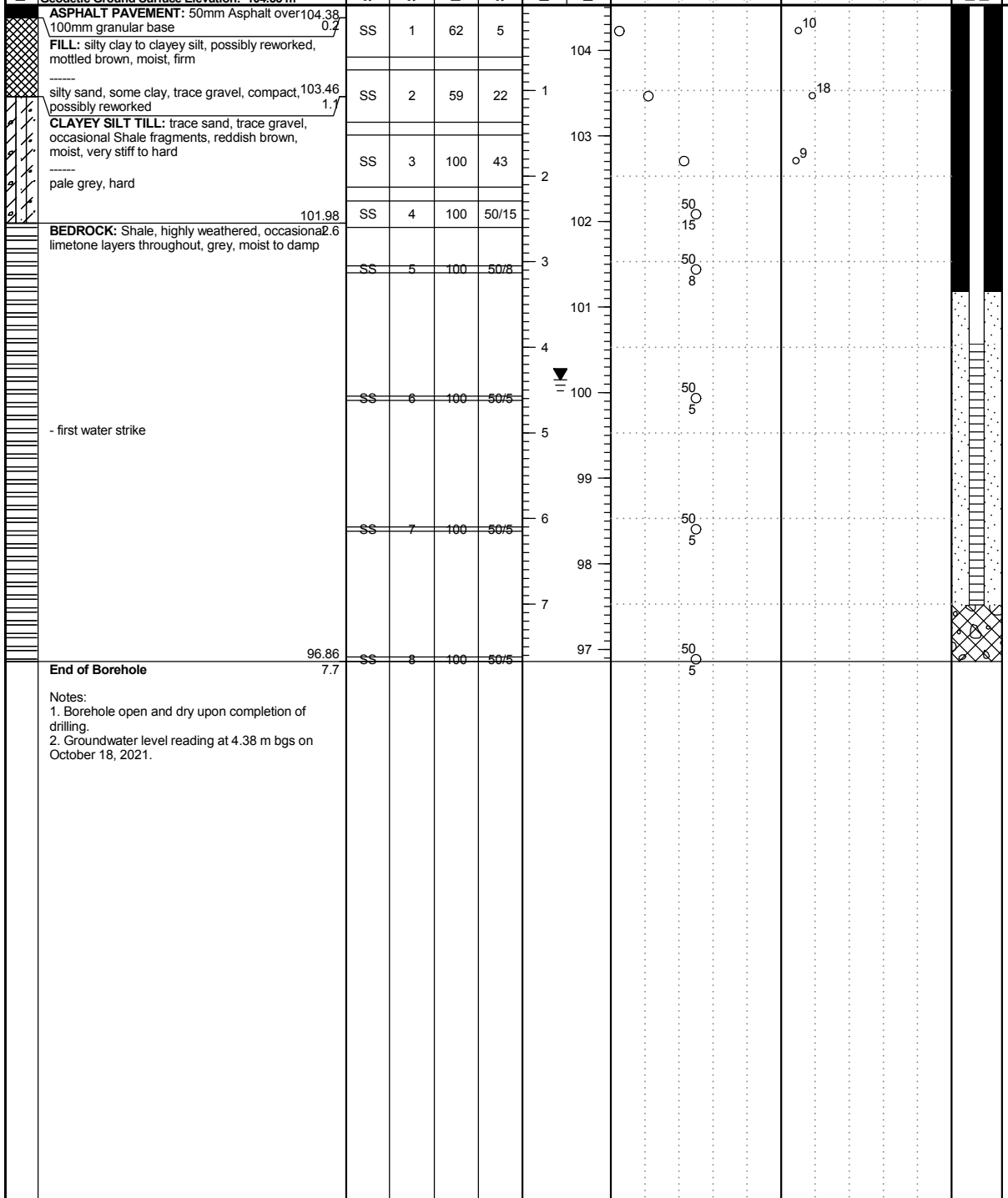
Project Number:	BIGC-GEO-490A	Drilling Location:	See Borehole Location Plan	Logged by:	MV
Project Client:	Oakville Argus Cross LP	Drilling Method:	150 mm Mud Rotary/ HQ Core	Compiled by:	MV
Project Name:	Preliminary Geotechnical Investigation	Drilling Machine:	Truck Mounted Drill	Reviewed by:	SS
Project Location:	581-587 Argus Road, Oakville	Date Started:	8 Oct 21	Date Completed: 8 Oct 21	Revision No.: 0, 26/10/21

Geodetic Ground Surface Elevation: 104.53 m

Depth (m)	Description	Elevation (m)	Sample Type	Sample Number	Recovery (%)	SPT 'N' Value/RQD%
0	**ASPHALT PAVEMENT:** 50mm Asphalt over 100mm granular base	104.38	SS	1	62	5
0.2	**FILL:** silty clay to clayey silt, possibly reworked, mottled brown, moist, firm					

1.1	silty sand, some clay, trace gravel, compact, possibly reworked	103.46	SS	2	59	22
	CLAYEY SILT TILL: trace sand, trace gravel, occasional Shale fragments, reddish brown, moist, very stiff to hard		SS	3	100	43
	------					
	pale grey, hard					
2.6		101.98	SS	4	100	50/15
	BEDROCK: Shale, highly weathered, occasional limetone layers throughout, grey, moist to damp		SS	5	100	50/8
			SS	6	100	50/5
	- first water strike					
			SS	7	100	50/5
7.7		96.86	SS	8	100	50/5
	End of Borehole					

Notes:
1. Borehole open and dry upon completion of drilling.
2. Groundwater level reading at 4.38 m bgs on October 18, 2021.



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Groundwater depth on completion of drilling: Dry m.

Groundwater depth observed on 18/10/2021 at a depth of: 4.38 m.

Scale: 1 : 74

RECORD OF BOREHOLE No. BM/MW2

Project Number:	BIGC-GEO-490A	Drilling Location:	See Borehole Location Plan	Logged by:	MV
Project Client:	Oakville Argus Cross LP	Drilling Method:	96 mm Mud Rotary/ HQ Core	Compiled by:	MV
Project Name:	Preliminary Geotechnical Investigation	Drilling Machine:	Truck Mounted Drill	Reviewed by:	SS
Project Location:	581-587 Argus Road, Oakville	Date Started:	7 Oct 21	Date Completed: 7 Oct 21	Revision No.: 0, 26/10/21

Lithology Plot	DESCRIPTION (Geodetic Ground Surface Elevation: 104.24 m)	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)	LAB TESTING: Rinse pH Values; Soil Vapour Reading (parts per million (ppm)); Lower Explosive Limit (LEL); Plastic / Liquid	INSTRUMENTATION INSTALLATION	COMMENTS
	ASPHALT PAVEMENT: 70mm Asphalt over 200mm granular bases (103.97)	SS	1	70	16	0	104	50	15		
	0.3 **FILL:** silty clay to clayey silt, trace gravel, dark greenish black, damp, very stiff										
	------ mottled greenish brown, stiff (102.72)	SS	2	75	12	1	103		12		
	CLAYEY SILT TILL: trace sand, trace gravel, grey to reddish brown, damp, hard (101.65) 1.5	SS	3	79	34	2	102		8		
		SS	4	100	50/23			50/23			
	BEDROCK: Shale, highly weathered to excellent quality, occasional limetone layers throughout, grey, moist to damp 2.6	SS	5	100	50/5	3	101	50/5			
		SS	6	100	50/8	4	100	50/8			
	- first water strike	SS	7	100	50/5	5	99	50/5			
	ROCK CORE BEGINS at 7.32 m	RC	1	83	0	6	98				
	- Very Poor Quality - Fair Quality	RC	2	100	70	7	97				
	- Fair Quality	RC	3	100	72	8	96				
	- Good Quality soft zone from 12.06 to 12.2 m	RC	4	97	78	9 ▼	95				
	- Good Quality	RC	5	100	77	10	94				
						11	93				
						12	92				
						13	91				
						14					

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▽ Groundwater depth on completion of drilling: Not measured m.

▼ Groundwater depth observed on 18/10/2021 at a depth of: 9.05 m.

Scale: 1 : 74

Continued on Next Page

RECORD OF BOREHOLE No. BM/MW2

Project Number: **BIGC-GEO-490A** Drilling Location: **See Borehole Location Plan** Logged by: **MV**

Lithology Plot	DESCRIPTION	Sample Type	Sample Number	Recovery (%)	SPT 'N' Value/RQD%	DEPTH (m)	ELEVATION (m)	INSTRUMENTATION INSTALLATION	COMMENTS
	BEDROCK: Shale, highly weathered to excellent quality, occasional limetone layers throughout, grey, moist to damp	RC	6	100	79	15	90		
	- Good Quality some oxidised laminae at 13.87 m soft zone from 14.38 to 14.54 m						89		
	- Excellent Quality	RC	7	100	90	16	88		
	- Excellent Quality some oxidised laminae at 16.92 m	RC	8	97	95	17	87		
	- Good Quality	RC	9	97	89	18	86		
	- Excellent Quality	RC	10	100	100	19	85		
	- Excellent Quality	RC	11	100	99	20	84		- cement grout
	- Good Quality fracture zone with slickenside from 24.01 to 24.29 m	RC	12	97	79	21	83		
	- Good Quality	RC	13	97	88	22	82		
	- Good Quality soft zones at 26.25 m and 27.02 to 27.07 m	RC	14	100	84	23	81		
	76.69					24	80		
	End of Borehole 27.6					25	79		
	Notes: 1. Borehole open completion of drilling. 2. Groundwater level reading not measured upon completion of drilling due to introduced drilling water. 3. Groundwater level reading at 9.05 m bgs on October 18, 2021.					26	78		
						27	77		

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

Scale: 1 : 74

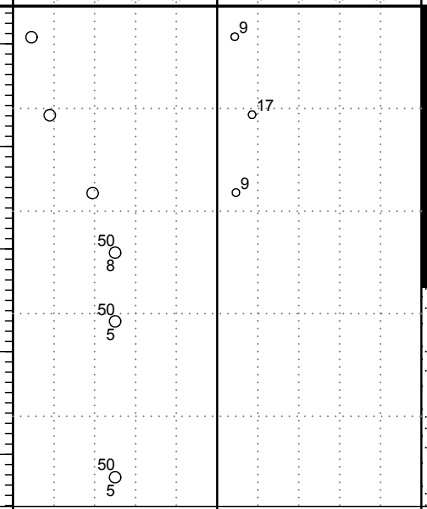
RECORD OF BOREHOLE No. BM/MW3

Project Number:	BIGC-GEO-490A	Drilling Location:	See Borehole Location Plan	Logged by:	MV
Project Client:	Oakville Argus Cross LP	Drilling Method:	150 mm Solid Stem Augering	Compiled by:	MV
Project Name:	Preliminary Geotechnical Investigation	Drilling Machine:	Truck Mounted Drill	Reviewed by:	SS
Project Location:	581-587 Argus Road, Oakville	Date Started:	8 Oct 21	Date Completed: 8 Oct 21	Revision No.: 0, 26/10/21

Geodetic Ground Surface Elevation: 104.37 m

Description	Depth (m)	Elevation (m)	Sample Type	Sample Number	Recovery (%)	SPT 'N' Value/RQD%
ASPHALT PAVEMENT: 50mm Asphalt over 150mm granular bases	0.2	104.17	SS	1	38	9
FILL: silty clay to clayey silt, possibly reworked, trace sand, trace gravel, mottled brown, moist, stiff to very stiff						
------		103.30	SS	2	70	18
silty sand, some clay, trace gravel, mottled pale grey, possibly reworked, compact	1.1					
CLAYEY SILT TILL: trace sand, trace gravel, occasional Shale fragments, reddish brown to grey, moist, very stiff to hard			SS	3	100	39
		101.93	SS	4	100	50/8
BEDROCK: Shale, highly weathered, occasional limetone layers throughout, grey, moist to damp	2.4		SS	5	100	50/5
- first water strike		99.49	SS	6	100	50/5
End of Borehole on Auger Refusal	4.9					

Notes:
1. Borehole open upon completion of drilling.
2. Groundwater level at 4.72 m bgs upon completion of drilling.
3. Groundwater level reading at 4.24 m bgs on October 18, 2021.



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Groundwater depth on completion of drilling: 4.72 m.

Groundwater depth observed on 18/10/2021 at a depth of: 4.24 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 : 74

RECORD OF BOREHOLE No. BM/MW4

Project Number:	BIGC-GEO-490A	Drilling Location:	See Borehole Location Plan	Logged by:	MV
Project Client:	Oakville Argus Cross LP	Drilling Method:	150 mm Solid Stem Augering	Compiled by:	MV
Project Name:	Preliminary Geotechnical Investigation	Drilling Machine:	Truck Mounted Drill	Reviewed by:	SS
Project Location:	581-587 Argus Road, Oakville	Date Started:	8 Oct 21	Date Completed: 8 Oct 21	Revision No.: 0, 26/10/21

Geodetic Ground Surface Elevation: 103.61 m

Description	Elevation	Depth	Sample Type	Sample Number	Recovery (%)	SPT 'N' Value/RQD%
ASPHALT PAVEMENT: 50mm Asphalt over 150mm granular bases	103.41	0.2	SS	1	75	14
FILL: silty clay to clayey silt, Shale fragments, brown to grey, moist, stiff	102.70	0.9	SS	2	51	31
CLAYEY SILT TILL: trace sand, trace gravel, pale slightly mottled brown to grey, moist, stiff to hard			SS	3	82	14
	101.02	2.6	SS	4	47	75/23
BEDROCK: Shale, highly weathered, occasional limetone layers throughout, grey, moist to damp			SS	5	100	50/8
			SS	6	100	50/8
- first water strike						
	96.29	7.3	SS	7	100	50/8
End of Borehole on Auger Refusal						

Notes:
1. Borehole open upon completion of drilling.
2. Groundwater level at 7.01 m bgs upon completion of drilling.
3. Groundwater level reading at 4.71 m bgs on October 18, 2021.

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

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Groundwater depth on completion of drilling: 7.01 m.

Groundwater depth observed on 18/10/2021 at a depth of: 4.71 m.

RECORD OF BOREHOLE No. BM/MW5

Project Number:	BIGC-GEO-490A	Drilling Location:	See Borehole Location Plan	Logged by:	MV
Project Client:	Oakville Argus Cross LP	Drilling Method:	96 mm Solid Stem Augers	Compiled by:	MV
Project Name:	Preliminary Geotechnical Investigation	Drilling Machine:	Truck Mounted Drill	Reviewed by:	SS
Project Location:	581-587 Argus Road, Oakville	Date Started:	6 Oct 21	Date Completed: 6 Oct 21	Revision No.: 0, 26/10/21

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: 103.75 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: 70mm Asphalt over 130mm granular base						103.55				
	0.2										
	FILL: silty clay to clayey silt, trace sand, trace gravel, mottled pale grey, moist, stiff to hard	SS	1	70	9						
	------										
	occasional cobble, mottled greenish brown, hard	SS	2	48	50/15	1	103	50/15			
	102.23										
	CLAYEY SILT TILL: trace gravel and pebbles, pale grey, moist, hard	1.5	SS	3	62	32	2	102	50/8	12	
	101.46										
	BEDROCK: Shale, highly weathered to excellent quality, occasional limetone layers throughout, grey, moist to damp	2.3	SS	4	100	50/8	3	101	50/8		
			SS	5	100	50/8	4	100	50/8		
			SS	6	100	50/10	5	99	50/10		
	- first water strike		SS	7	100	50/8	6	98	50/8		
	---------- ROCK CORE BEGINS at 7.32 m		RC	1	87	0	7	97			
	- Very Poor Quality						8	96			
	- Fair Quality fracture zone from 8.16 to 8.72 m some conglomeratic layers throughout run		RC	2	100	61	9	95			
	- Fair Quality		RC	3	94	70	10	94			
	- Good Quality		RC	4	100	87	11	93			
	- Fair Quality some oxidised laminae from 12.34 to 15.39 m		RC	5	98	72	12	92			
							13	91			
							14	90			

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▽ Groundwater depth on completion of drilling: Not measured m.

▼ Groundwater depth observed on 18/10/2021 at a depth of: 19.04 m.

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RECORD OF BOREHOLE No. BM/MW5

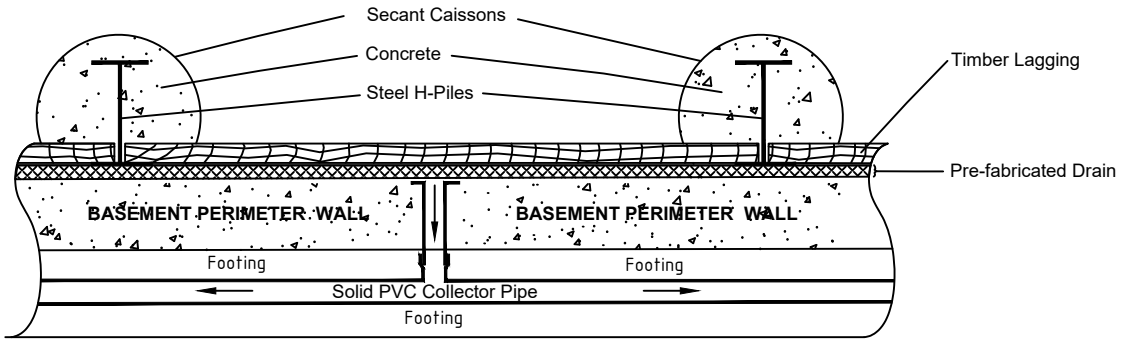
Project Number: **BIGC-GEO-490A** Drilling Location: **See Borehole Location Plan** Logged by: **MV**

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
	BEDROCK: Shale, highly weathered to excellent quality, occasional limetone layers throughout, grey, moist to damp										
	- Excellent Quality	RC	6	100	93	15	89	○			
	- Fair Quality sub vertical fracture from from 15.84 to 15.92 m	RC	7	100	74	16	88	○			
	- Excellent Quality	RC	8	94	93	17	87	○			
	- Excellent Quality	RC	9	100	92	18 19 ▼	86	○			
	- Excellent Quality	RC	10	98	90	20	85	○			
	- Fair Quality	RC	11	94	70	21 22	84 83	○			
	- Excellent Quality fracture zone from 23.81 to 23.91 m	RC	12	100	99	23 24	82 81	○			
	- Good Quality 78.49	RC	13	100	88	25	80	○			
	End of Borehole 25.3						79				
	Notes: 1. Borehole open upon completion of drilling. 2. Groundwater level reading not measured upon completion of drilling due to introduced drilling water. 3. Groundwater level reading at 19.04 m bgs on October 18, 2021.										

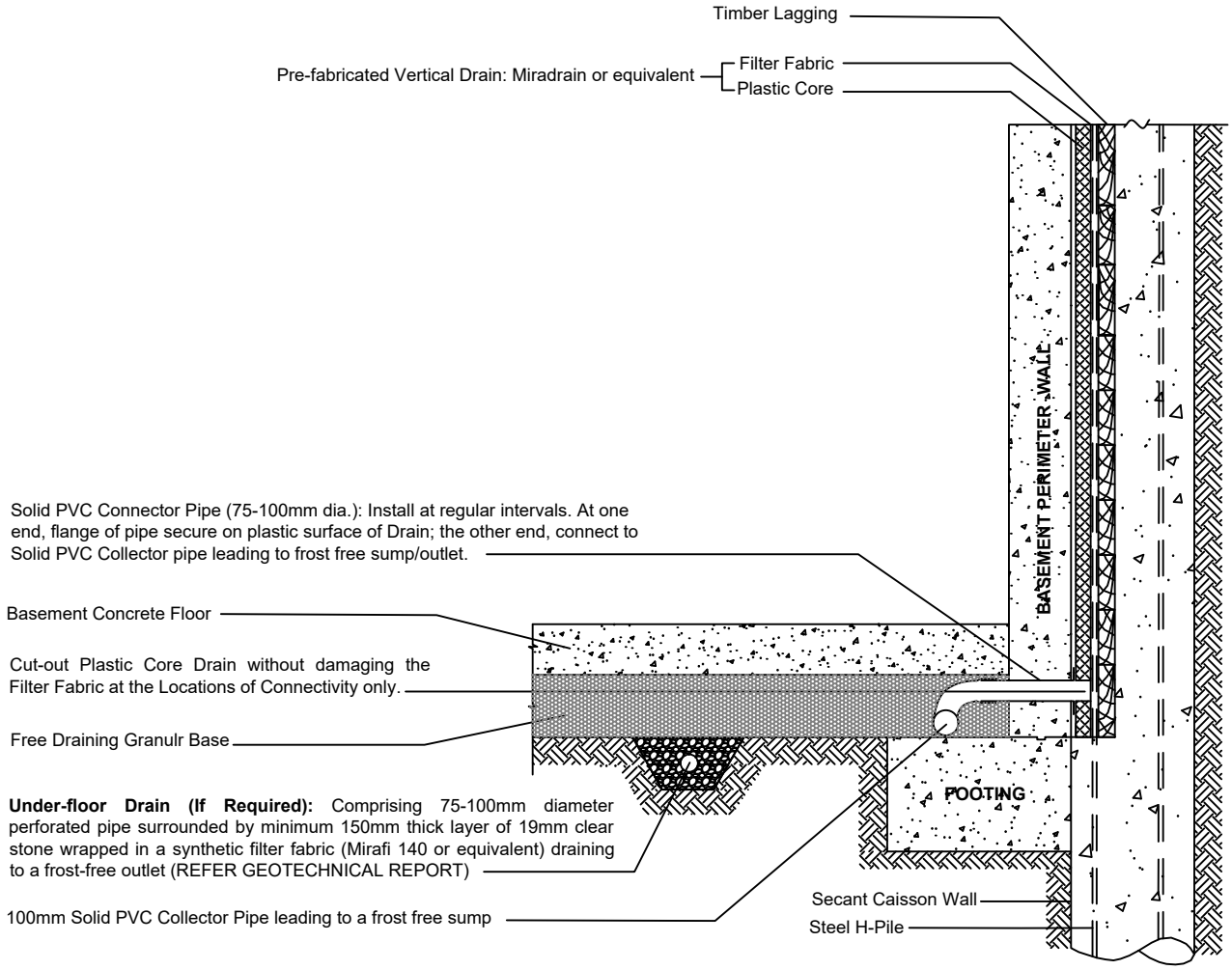
Borehole details as presented, do not constitute a thorough understanding of all potential conditions present and requires interpretative assistance from a qualified Geotechnical Engineer. Also, borehole information should be read in conjunction with the geotechnical report for which it was commisioned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

Scale: 1 : 74

Appendix C – Conceptual Permanent Perimeter and Underfloor Drainage System 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 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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TITLE AND LOCATION

PERMANENT PERIMETER & UNDER-FLOOR DRAINAGE SYSTEM
(CONCEPTUAL-FOR SHORING)
PRELIMINARY GEOTECHNICAL INVESTIGATION
581-587 ARGUS ROAD, OAKVILLE, ONTARIO

PROJECT NO.	DWN.
BIGC-GEO-490A	S.M.
SCALE	CK.
AS NOTED	S.S.
DATE	FIG NO.
OCTOBER 2021	C