REVISED

Geotechnical Investigation – Proposed Mixed-Use Development

150 Kanata Avenue & 1200 Canadian Shield Avenue,
Kanata, Ontario

Prepared for:

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1.0 INTRODUCTION AND SCOPE

Pinchin Ltd. (Pinchin) was retained by Bâtimo Developpement Inc. (Client) to conduct a Geotechnical Investigation and provide subsequent geotechnical design recommendations for the proposed mixed-use development to be located on 150 Kanata Avenue & 1200 Canadian Shield Avenue, Kanata, Ontario (Site). The Site location is shown on Figure 1.

Based on information provided by the Client, it is Pinchin's understanding that the proposed development is to consist of a seven to nine storey residential/commercial building complete with a single level underground parking garage under the west portion of the Site (Phase I) and two levels of underground parking under the northeast portion of the Site (Phase II). The underside of the footing for the single level underground parking garage will be located approximately 3.5 metres below proposed finished grade, while the underside of the footing for the two-level underground parking garage will be located approximately 7.0 metres below proposed finished grade. It is noted that due to the parking garage occupying the majority of the Site footprint, no service trenches are required for the proposed development; however, an asphaltic concrete pavement structure will be required where the parking areas and access roadway is constructed outside of the parking garage footprint.

Pinchin's geotechnical comments and recommendations are based on the results of the Geotechnical Investigation and our understanding of the project scope.

The purpose of the Geotechnical Investigation was to delineate the subsurface conditions and soil engineering characteristics by advancing a total of eight (8) sampled boreholes (Boreholes BH1 to BH8), at the Site. In addition, a supplemental field investigation was completed which consisted of advancing a total of ten (10) bedrock probes at the Site. The information gathered from the Geotechnical Investigation will allow Pinchin to provide geotechnical design recommendations for the proposed development.

Based on a desk top review and the results of the Geotechnical Investigation, the following geotechnical data and engineering design recommendations are provided herein:

- A detailed description of the soil and groundwater conditions;
- Site preparation recommendations;
- Open cut excavations;
- Anticipated groundwater management;
- Foundation design recommendations including soil and bedrock bearing resistances at Serviceability Limit States (SLS) and Ultimate Limit States (ULS) design;
- Potential total and differential settlements;
- Foundation frost protection and engineered fill specifications and installation;

- Seismic Site classification for seismic Site response;
- Underground parking garage design, including concrete floor slab support recommendations;
- Asphaltic Concrete pavement structure design for access roadways;
- Visual slope condition assessment; and
- Potential construction concerns.

Abbreviations, terminology, and principle symbols commonly used throughout the report, borehole logs and appendices are enclosed in Appendix I.

2.0 SITE DESCRIPTION AND GEOLOGICAL SETTING

The Site is located on the northwest corner of the intersection of Kanata Avenue and Maritime Way, approximately 300 metres northwest of Highway 417 in Ottawa, Ontario. The Site is currently undeveloped and consists of a heavily forested area with a combination of mature trees and wild undergrowth. The Site topography varies and typically slopes down from north to south, with a low-lying area located on the southeast corner of the Site that is upwards of 5 to 6 m below the existing street level, at its deepest point. The lands adjacent to the Site are either undeveloped or developed with a combination of multi unit residential buildings and commercial retail buildings.

Data obtained from the Ontario Geological Survey Maps, as published by the Ontario Ministry of Natural Resources, indicates that the majority of the Site is located on Precambrian bedrock, with the exception of the southeast corner of the Site which is located on an organic deposit consisting of peat, much, and marl (Ontario Geological Survey 2010. Surficial geology of Southern Ontario; Ontario Geological Survey, Miscellaneous Release--Data 128-REV). The underlying bedrock at this Site is of the Grenville Supergroup and Flinton Group consisting of classic metasedimentary rocks, conglomerate, wacke, quartz arenite, arkose, limestone, siltstone, chert, minor iron formation, and minor metavolcanic rocks (Ontario Geological Survey 2011. 1:250 000 scale bedrock geology of Ontario; Ontario Geological Survey, Miscellaneous Release---Data 126-Revision 1).

3.0 GEOTECHNICAL FIELD INVESTIGATION AND METHODOLOGY

Pinchin completed a field investigation at the Site from May 18 to 20, 2021 by advancing a total of eight sampled boreholes throughout the Site. The boreholes were advanced to depths ranging from approximately 0.2 to 14.0 metres below existing ground surface (mbgs), where refusal was encountered on probable bedrock. It is noted that below the sampled depth within Borehole BH1, a Dynamic Cone Penetration Test (DCPT) was advanced to the refusal depth of approximately 14.0 mbgs to further assess the consistency of the subgrade soils with depth.

Pinchin completed a supplemental field investigation at the Site on April 13 and 14, 2022 by advancing ten bedrock probes at the approximate locations provided by the Client. The bedrock probes were advanced to depths ranging from approximately 1.2 to 9.9 mbgs, where refusal was encountered on probable bedrock.

The approximate spatial locations of the boreholes and bedrock probes advanced at the Site are shown on Figure 2.

The boreholes were advanced with the use of a Geoprobe 7822 DT direct push drill rig which was equipped with standard soil sampling equipment. Soil samples were collected at 0.76 and 1.52 m intervals using a 51 mm outside diameter (OD) split spoon barrel in conjunction with Standard Penetration Tests (SPT) “N” values (ASTM D1586). The SPT “N” values were used to assess the compactness condition of the non-cohesive soil, and to estimate the consistency of the cohesive soil.

It is noted that the bedrock probes consisted of augering through the overburden material down to the underlying bedrock surface and no soil samples of the overburden material were collected.

Bedrock was proven in Boreholes BH4, BH6, Rock Probes RP1 and RP9 by core drilling with an NQ-size double tube diamond bit core barrel. The bedrock core specimens were measured in the field to determine the Rock Quality Designation (RQD) (ASTM 6032). The core samples were returned to our offices for further visual examination and testing.

Monitoring wells were installed within Boreholes BH4 and BH6 to allow measurement of groundwater levels. The monitoring wells were constructed using flush-threaded 50 mm diameter Trilock pipe with 1.2-meter-long 10-slot well screens, delivered to the Site in pre-cleaned individually sealed plastic bags. The screen and riser pipes were not allowed to come into contact with the ground or drilling equipment prior to installation.

A completed well record was submitted to the property owner and the Ministry of the Environment, Conservation and Parks for Ontario (MECP) as per Ontario Regulation 903, as amended. A licensed well technician must properly decommission the monitoring wells prior to construction according to Regulation 903 of the Ontario Water Resources Act.

Groundwater observations and measurements were obtained from the open boreholes during and upon completion of drilling. The groundwater levels were measured in the monitoring wells on June 18, 2021, and May 6, 2022. The groundwater observations and measurements recorded are included on the appended borehole logs.

The borehole and bedrock probe locations and ground surface elevations were located at the Site by Pinchin personnel. The ground surface elevation at each borehole location was referenced to the following benchmark which was provided by the Client:

- BM: Top nut of fire hydrant, at the approximate location shown on Figure 2; and
- Elevation: 99.91 metres.

The field investigation was monitored by experienced Pinchin personnel. Pinchin logged the drilling operations and identified the soil samples as they were retrieved. The recovered soil samples were sealed into plastic bags and carefully transported to an independent and accredited materials testing laboratory for detailed analysis and testing. All soil samples were classified according to visual and index properties by the project engineer.

The field logging of the soil and groundwater conditions was performed to collect geotechnical engineering design information. The borehole logs include textural descriptions of the subsoil in accordance with a modified Unified Soil Classification System (USCS) and indicate the soil boundaries inferred from non-continuous sampling and observations made during the borehole advancement. These boundaries reflect approximate transition zones for the purpose of geotechnical design and should not be interpreted as exact planes of geological change. The modified USCS classification is explained in further detail in Appendix I. Details of the soil, bedrock and groundwater conditions encountered within the boreholes are included on the Borehole Logs within Appendix II.

Select soil samples collected from the boreholes were submitted to a material testing laboratory to determine the grain size distribution of the soil. A copy of the laboratory analytical reports is included in Appendix III. In addition, the collected samples were compared against previous geotechnical information from the area, for consistency and calibration of results.

4.0 SUBSURFACE CONDITIONS

4.1 Borehole Soil Stratigraphy

In general, the soil stratigraphy at the Site comprises surficial organics overlying natural sand, glacial till, and bedrock to the maximum borehole refusal depth of approximately 14.0 mbgs. The appended borehole logs provide detailed soil descriptions and stratigraphies, results of SPT testing, and groundwater measurements.

The surficial organics were encountered within all boreholes and were measured to range in thickness from approximately 0.15 m and 1.8 m.

The natural sand was encountered underlying the surficial organics in Boreholes BH1 and BH5 to BH8 and extended to a maximum depth of approximately 1.5 mbgs. The sand material ranged in soil matrix from sand containing some silt and trace gravel, to silty sand containing trace gravel. The non-cohesive material had a very loose to dense relative density based on SPT ‘N’ values between 1 and 31 blows per 300 mm penetration of a split spoon sampler. It is noted that a very dense gravelly sand layer containing trace silt was encountered within Borehole BH1 at approximately 7.6 mbgs. The results of one particle size distribution analysis completed on a sample of the gravelly sand material indicates that the sample contains approximately 23% gravel, 70% sand, and 7% silt sized particles.

The glacial till was encountered underlying the natural sand in Boreholes BH1, BH5, BH7 and BH8. The glacial till material ranged in soil matrix from clayey silt containing some sand and trace gravel, to sand and silt containing some gravel and trace clay. The non-cohesive glacial till had a compact to very dense relative density based on SPT ‘N’ values of 15 to 56 blows per 300 mm penetration of a split spoon sampler. The results of two particle size distribution analyses completed on samples of the glacial till indicate that the samples contained approximately 2 to 11% gravel, 18 to 44% sand, 37 to 57% silt, and 8 to 23% clay sized particles. The moisture content of the material tested ranged from 10.2 to 16.9%, indicating the material was in a moist condition at the time of sampling.

4.2 Bedrock

Refusal was encountered on bedrock within all boreholes and bedrock probes at depths ranging from approximately 0.2 to 14.0 mbgs (i.e., elevations 85.4 to 98.8 m). The bedrock cores recovered consisted of an upper layer of sandstone bedrock overlying granitic bedrock. The sandstone was noted to be moderately weathered, while the granite was noted to be faintly weathered. The bedrock was predominantly brownish grey with white and black banding, fine to medium grained, and contained some natural fractures with little oxidation. The bedrock at the fracture locations was mostly sharp and angular, which indicates minor water migration. Natural fractures were closely to moderately spaced and were generally found to occur in sets oriented at approximately 45 to 90º to the core axis. An approximate 70% wash return within the rock cores was observed. The wash return was milky white in colour. The rock core recovery ranged between 27 and 97%, with RQDs ranging from 0 to 88%, indicating a very poor to good rock quality. A photograph of the rock cores is provided in Appendix IV.

4.3 Groundwater Conditions

Groundwater observations and measurements were obtained in the open boreholes at the completion of drilling and are summarized on the appended borehole logs. The groundwater levels were measured on June 18, 2021, and May 6, 2022, in the monitoring wells installed within Boreholes BH4 and BH6. Groundwater was measured to be between approximately 0.7 and 5.3 mbgs (i.e., elevations 93.1 to 94.1 m) within the monitoring wells installed within Boreholes BH4 and BH6, respectively.

Seasonal variations in the water table should be expected, with higher levels occurring during wet weather conditions in the spring and fall and lower levels occurring during dry weather conditions.

5.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

5.1 General Information

The recommendations presented in the following sections of this report are based on the information available regarding the proposed construction, the results obtained from the geotechnical investigation, and Pinchin's experience with similar projects. Since the investigation only represents a portion of the subsurface conditions, it is possible that conditions may be encountered during construction that are substantially different than those encountered during the investigation. If these situations are encountered, adjustments to the design may be necessary. A qualified geotechnical engineer should be on-Site during the foundation preparation to ensure the subsurface conditions are the same/similar to what was observed during the investigation.

Based on information provided by the Client, it is Pinchin's understanding that the proposed development is to consist of a seven to nine storey residential/commercial building complete with a single level underground parking garage under the west portion of the Site (Phase I) and two-levels of underground parking under the northeast portion of the Site (Phase II). The proposed building will reportedly be constructed with a ground floor elevation of 100.75 m. This will result in the underside of the footing for the single level underground parking garage being located at approximate elevation 97.25 m, while the underside of the footing for the two-level underground parking garage will be located at approximate elevation 93.75 m.

As previously mentioned, bedrock was encountered within all boreholes and bedrock probes at depths ranging from approximately 0.2 to 14.0 mbgs (i.e., elevations 85.4 to 98.8 m). As such, a portion of the bedrock will require removal for both the single and two-level underground parking garage levels; however, there are also areas where the bedrock surface is located deeper than the proposed underside of the footing elevation for both the single and two-level parking garages.

The existing surficial organics and loose sand deposit are not suitable to remain below the proposed buildings. The Client has indicated that the foundation walls for the single-level underground parking garage will be extended down to the bedrock surface where it is located deeper than the proposed underside of the footing level. The Client has also indicated that they would like to construct the footings for the two-level underground parking garage on a combination of the underlying bedrock surface and the very dense glacial till encountered within Borehole BH1. As such, Pinchin has provided conventional shallow foundation options for foundations founded on both bedrock and glacial till.

Pinchin notes that it may be beneficial to take advantage of the existing bedrock profile and local topography at the Site by considering additional underground parking levels where the overburden thicknesses will accommodate them.

5.2 Open Cut Excavations

Due to the varying topography at the Site, the excavation depth for the underground parking garage foundations will extend to varying depths. As such, in some areas of the Site, a portion of the bedrock will need to be removed to accommodate the underground parking garage levels. The depth of refusal on bedrock at each borehole location is illustrated on Figure 2 and generally increases with depth from the southwest corner of the Site to the northeast corner of the Site.

Based on the subsurface information obtained from within the boreholes, it is anticipated that the excavated material will predominately consist of organics, natural sand, glacial till, and bedrock.

Where workers must enter trench excavations deeper than 1.2 m, the trench excavations should be suitably sloped and/or braced in accordance with the Occupational Health and Safety Act (OHSA), Ontario Regulation 213/91, Construction Projects, July 1, 2011, Part III - Excavations, Section 226. Alternatively, the excavation walls may be supported by either closed shoring, bracing, or trench boxes complying with sections 235 to 239 and 241 under O. Reg. 231/91, s. 234(1). The shoring system may be designed as full cantilevers, or the lateral loads can be taken up to the installation of internal bracing of rakers or tie back soil anchors. The temporary shoring design must include appropriate factors of safety, and any possible surcharge loading must be considered.

The following parameters (un-factored) could be used in the shoring design against lateral loads: It should be noted that these earth pressure coefficients assume that the back of the wall is vertical; condition of the ground surface behind the wall is assumed to be flat:

Soil Layer	**Unit Weight (kN/m^3)**	**Angle of Internal Friction (°)**	**Active Earth Pressure Coefficient - K_a**	**Passive Earth Pressure Coefficient - K_p**	**At Rest Earth Pressure Coefficient - K_o**
Sand	19	30	0.33	3.0	0.5
Glacial Till	21	32	0.31	3.25	0.47

Based on the OHSA, the natural subgrade soils would be classified as Type 3 soil and temporary excavations in these soils must be sloped at an inclination of 1 horizontal to 1 vertical (H to V) from the base of the excavation. Excavations extending below the groundwater table would be classified as a Type 4 soil and temporary excavations will have to be sloped back at 3 H to 1 V from the base of the excavation.

Based on bedrock cores retrieved as well as local experience in the area, the upper approximate 2.0 to 3.0 m of bedrock is typically weathered and can usually be removed with mechanical equipment, such as a large excavator and hydraulic hammer (hoe ram) and where required, with line drilling on close centres. Specifically, the sandstone bedrock was noted to be moderately weathered and should be relatively easy to remove with a hydraulic hammer that can be utilized to create an initial opening for the excavator bucket to gain access of the layered rock. The bedrock is known to contain vertical joints and near horizontal bedding planes. Therefore, some vertical and horizontal over break of the bedrock should be expected.

Depending on the ability of the mechanical equipment to advance through the bedrock, drilling and blasting may be required. It is often difficult to blast “neat” lines using conventional drilling and blasting procedures, as such, problems with “over break” are common. This may affect quantities claimed by the contractor for rock excavations, as well as the potential for off-site disposal of the blasted rock, if necessary. Allowances should be made for over break conditions. Due consideration should also be given to controlled blasting procedures to prevent potential damage to the surrounding environment.

Drilling and blasting activities shall be carried out in accordance with the requirements outlined in Ontario Provincial Standard Specification (OPSS) 120. In addition, Pinchin has provided the following additional recommendations:

- Prior to commencing drilling and blasting activities a pre-blast survey of all buildings, utilities, structures, water wells, and facilities within a 150 m radius of the Site is to be performed. The pre-blast survey is to include but not be limited to details on the type of structure (i.e., age and type of construction), description of any existing/observed building deficiencies (i.e., differential settlement, cracks, structural and cosmetic damage, and etcetera) including dimensions when possible, and time stamped and labelled digital photographs and/or videos of areas of concern.
- Monitoring for Peak Particle Velocity (PPV) is to be completed and limited to 50 mm/s for frequencies greater than 40 Hz, 20 mm/s for frequencies equal to or less than 40 Hz, and 10 mm/s when concrete and grout has been placed within the previous 72 hours.
- Monitoring of peak sound pressure and water overpressure may also be required and are to be completed in accordance with the recommendations outline in OPSS 120 (120.07.05 Monitoring).
- A minimum of 3 trial blasts are to be completed to ensure the proposed blast design can be completed within the PPV vibration limits.
- Blasting mats and utility line shielding is to be utilized for all blasts.

- Records of each blast are to be completed which shall include but not be limited to the date, time and location of the blast, wind and atmospheric conditions at the time of the blast, blast details, and recorded values from the monitoring equipment.

Pinchin notes that, local contractors are familiar with excavating the local bedrock and have specialized knowledge and techniques for its removal. Depending on the block size and degree of weathering of the rock they may have a different approach than what is presented in the preceding paragraphs.

Construction slopes in intact bedrock should stand near vertical provided the “loose” rock is properly scaled off the face. Once the blasting is completed, if there are any permanent bedrock shear walls, they will have to be reviewed by a Rock Mechanics Specialist to determine if it is stable or if it needs reinforcing, such as rock bolting.

In addition to compliance with the OHSA, the excavation procedures must also comply to any potential other regulatory authorities, such as federal and municipal safety standards.

5.3 Anticipated Groundwater Management

As previously mentioned, groundwater was measured between approximately 0.7 and 5.3 mbgs (i.e., elevations 93.1 to 94.1 m) within the monitoring wells installed in Boreholes BH4 and BH6, respectively. It is noted that the groundwater levels were measured in May 2021 and June 2022, which are considered to be on the back end of the wet season, when groundwater levels have already lowered from the seasonal high. A conservative assumption on the seasonal high groundwater levels for the Site would be to increase the groundwater elevations by approximately 1.0 m or to grade level (i.e., elevations 94.1 m to 95.1 m).

As such, the assumed seasonal high groundwater levels are located at a greater depth than the proposed underside of footing elevation of 97.25 m for the single-level underground parking garage; however, the assumed seasonal high groundwater levels will be located above the proposed underside of footing elevation of 93.25 m for the two-level underground parking garage.

Moderate groundwater inflow through the overburden soil and/or bedrock face is expected where the excavations extend less than 0.6 m below the groundwater table. It is believed that this groundwater inflow can be controlled using a gravity dewatering system with perimeter interceptor ditches and high-capacity pumps.

For excavations extending more than 0.6 m below the stabilized groundwater table, a dewatering system installed by a specialist dewatering contractor may be required to lower the groundwater level prior to excavation. The design of the dewatering system should be left to the contractor’s discretion, and the system should meet a performance specification to maintain and control the groundwater at least 0.30 m below the excavation base. It is recommended that Pinchin review the final grading plan to confirm this recommendation.

Depending on the time of construction, there is a greater potential that the groundwater elevation could be higher and/or perched groundwater may be present. Any potential precipitation of perched groundwater should be able to be controlled from pumping from filtered sumps.

Prior to commencing excavations, it is critical that all existing surface water and potential surface water is controlled and diverted away from the Site to prevent infiltration and subgrade softening. At no time should excavations be left open for a period of time that will expose them to precipitation and cause subgrade softening.

All collected water is to discharge a sufficient distance away from the excavation to prevent re-entry. Sediment control measures, such as a silt fence should be installed at the discharge point of the dewatering system. The utmost care should be taken to avoid any potential impacts on the environment.

It is the responsibility of the contractor to propose a suitable dewatering system based on the groundwater elevation at the time of construction. The method used should not adversely impact any nearby structures. A Permit to Take Water (PTTW) or a submission to the Environmental Activity and Sector Registry (EASR) would be required if the daily water takings exceed 50,000 L/day. It is the responsibility of the contractor to make this application if required. Depending on the groundwater at the time of the excavation works, a more involved dewatering system may be required.

It is noted that if more accurate seasonal high groundwater levels are required, then a hydrogeological investigation should be completed at the Site.

5.4 Foundation Design

5.4.1 Discussion

As previously mentioned, the proposed building will reportedly be constructed with a ground floor elevation of 100.75 m. This will result in the underside of the footing for the single level underground parking garage being located at approximate elevation 97.25 m, while the underside of the footing for the two-level underground parking garage will be located at approximate elevation 93.75 m. As such, a portion of the bedrock will require removal for both the single and two-level underground parking garage levels; however, there are also areas where the bedrock surface is located deeper than the proposed underside of the footing elevation for both the single and two-level parking garages. The following tables summarizes the approximate ground surface, bedrock, and associated proposed underside of footing elevations at each individual investigation location:

Table 1: Two-Level Underground Parking Garage Locations

Investigation Location	Approximate Ground Surface Elevation (m)	Approximate Bedrock Elevation (m)	Proposed Underside of Footing Elevation at Investigation Location (m)
BH1	99.38	85.36	93.75
BH5	93.76	90.1	
BH8	99.02	93.99	
RP1	98.16	88.25	
RP2	100.03	95.76	
RP4	100.30	97.71	

Table 2: Single-Level Underground Parking Garage Locations

Investigation Location	Approximate Ground Surface Elevation (m)	Approximate Bedrock Elevation (m)	Proposed Underside of Footing Elevation at Investigation Location (m)
BH2	95.16	95.01	97.25
BH3	94.53	94.38	
BH4	93.85	92.02	
BH6	99.38	98.77	
BH7	97.68	95.7	
RP3	97.67	93.40	
RP5	96.88	95.05	
RP6	99.22	98.00	

Investigation Location	Approximate Ground Surface Elevation (m)	Approximate Bedrock Elevation (m)	Proposed Underside of Footing Elevation at Investigation Location (m)
RP7	100.21	98.84	
RP8	99.64	97.96	
RP9	99.16	97.94	
RP10	98.99	94.72	

5.4.2 Shallow Foundations Bearing on Glacial Till

As previously mentioned, the Client has indicated that they would like to construct the footings for the two-level underground parking garage on a combination of the underlying bedrock surface and the very dense glacial till encountered within Borehole BH1 at approximate elevation 93.29 m. The existing organic material, loose sand and compact glacial till are not considered suitable to support the bearing resistances provided below.

Conventional shallow strip footings established on the inorganic very dense glacial till material encountered at approximate elevation 93.29 m within Borehole BH1, may be designed using a bearing resistance for 25 mm of settlement at Serviceability Limit States (SLS) of 300 kPa, and a factored geotechnical bearing resistance of 375 kPa at Ultimate Limit States (ULS). As the actual service loads were not known at the time of this report, these should be reviewed by the project structural engineer to determine if SLS or ULS governs the footing design.

It is noted that there is a potential for weaker glacial till to be encountered between the investigation locations. Any soft/loose areas are to be removed and replaced with a low strength concrete.

Pinchin notes that a qualified geotechnical engineering consultant should be on-Site during the proof roll and foundation preparation activities to verify the recommended level of compaction is achieved and to verify the design assumptions and recommendations. This is especially critical with respect to the recommended soil bearing pressures. If variations occur in the soil conditions between the borehole locations, site verification and site review by Pinchin is recommended to provide appropriate recommendations at that time.

The natural subgrade soil is sensitive to change in moisture content and can become loose/soft if subjected to additional water or precipitation. As well, it could be easily disturbed if travelled on during construction. Once it becomes disturbed it is no longer considered adequate to support the recommended design bearing pressures. It is recommended that a working slab of lean concrete (mud slab) be placed in the footing areas immediately after excavation and inspection to protect the founding soils during placement of formwork and reinforcing steel.

In addition, to ensure and protect the integrity of the subgrade soil during construction operations, the following is recommended:

- Prior to commencing excavations, it is critical that all existing surface water, potential surface water and perched groundwater are controlled and diverted away from the work Site to prevent infiltration and subgrade softening. At no time should excavations be left open for a period of time that will expose them to inclement weather conditions and cause subgrade softening;
- The subgrade should be sloped to a sump outside the excavation to promote surface drainage and the collected water pumped out of the excavation. Any potential precipitation or seepage entering the excavations should be pumped away immediately (not allowed to pond);
- The footing areas should be cleaned of all deleterious materials such as topsoil, organics, fill, disturbed, caved materials or loosened bedrock pieces;
- Any potential large cobbles or boulders (i.e., greater than 200 mm in diameter) within the subgrade material are to be removed and replaced with a similar soil type not containing particles greater than 200 mm in diameter. It is critical that particles greater than 200 mm in diameter are not in contact with the foundation to prevent point loading and overstressing; and
- If the excavated subgrade soil remains open to weather conditions and groundwater seepage, sidewall stability and suitability of the subgrade soil will need to be verified prior to construction.

If construction proceeds during freezing weather conditions, adequate temporary frost protection for the footing bases and concrete must be provided and maintained above freezing at all times.

5.4.3 Shallow Foundations Bearing on Bedrock

For conventional shallow strip and spread footings established directly on the weathered bedrock surface, a factored geotechnical bearing resistance of 900 kPa may be used at ULS. For areas where the bedrock is lower than the underside of footing, a low strength concrete could be used to raise grades to the underside of footing.

Prior to installing foundation formwork, the bedrock is to be reviewed by a geotechnical engineer. Serviceability Limit States (SLS) design does not apply to foundations bearing directly on bedrock, since the loads required for unacceptable settlements to occur would be much larger than the factored ULS and would be limited to the elastic compression of the bedrock and concrete.

The bearing resistance of 900 kPa assumes the bedrock is cleaned of all overburden material and any loose rock pieces. The bedrock should be cleaned with air or water pressure exposing clean sound bedrock. If construction proceeds during freezing weather conditions water should not be allowed to pool and freeze in bedrock depressions. All concrete should be installed and maintained above freezing temperatures as required by the concrete supplier.

The bedrock is to be relatively level with slopes not exceeding 10 degrees from the horizontal. Where the bedrock slope exceeds 10 degrees from the horizontal and does not exceed 25 degrees from the horizontal, shear dowels can be incorporated into the design to resist sliding. Where rock slopes are steeper, the bedrock is to be levelled and stepped as required. The change in vertical height will be a function of the rock quality at the proposed foundation location and will need to be determined at the time of construction.

As an alternative to levelling the bedrock, where the bedrock surface is irregular and jagged, it may be more practical to provide a level benching over these areas by pouring lean mix concrete (minimum 10 MPa) prior to constructing the foundations. This decision is made on Site since each situation will depend on the Site-specific bedrock conditions.

5.4.4 Site Classification for Seismic Site Response & Soil Behaviour

The following information has been provided to assist the building designer from a geotechnical perspective only. These geotechnical seismic design parameters should be reviewed in detail by the structural engineer and be incorporated into the design as required.

The seismic site classification has been based on the 2012 OBC. The parameters for determination of Site Classification for Seismic Site Response are set out in Table 4.1.8.4.A of the OBC. The site classification is based on the average shear wave velocity in the top 30 m of the site stratigraphy. If the average shear wave velocity is not known, the site class can be estimated from energy corrected Standard Penetration Resistance (N60) and/or the average undrained shear strength of the soil in the top 30 m.

The boreholes advanced at this Site extended to a maximum depth of approximately 14.0 mbgs where refusal was encountered on bedrock. SPT “N” values within the soil deposit ranged between 1 and greater than 50 blows per 300 mm. As such, based on Table 4.1.8.4.A of the OBC, this Site has been classified as Class C. A Site Class C has an average shear wave velocity (Vs) of between 360 and

760 m/s. It is recommended that shear wave velocity soundings be completed at the Site once the final design and depths of foundations are known as a higher Site Classification may be available.

5.4.5 Foundation Transition Zones

Excessive differential settlements can occur where the subgrade support material types differ below the underside of continuous strip footings, (i.e., glacial till to bedrock). As such, where strip footings transition from one material to another the transition between the materials should be suitably sloped or benched to mitigate differential settlements.

Pinchin also recommends the following transition precautions to mitigate/accommodate potential differential settlements:

- For strip footings, the transition zones should be adequately reinforced with additional reinforced steel lap lengths or widened footings;
- Steel reinforced poured concrete foundation walls; and
- Control joints throughout the transition zone(s).

The above recommendations should be reviewed by the structural engineer and incorporated into the design as necessary.

Where strip footings are founded at different elevations, the subgrade soil and/or bedrock is to have a maximum slope of 2H to 1V, with the concrete footing having a maximum rise of 600 mm and a minimum run of 600 mm between each step, as detailed in the 2012 Ontario Building Code (OBC). The lower footing should be installed first to mitigate the risk of undermining the upper footing.

Individual spread footings are to be spaced a minimum distance of one and a half times the largest footing width apart from each other to avoid stress bulb interaction between footings. This assumes the footings are at the same elevation.

Foundations may be placed at a higher elevation relative to one another provided that the slope between the outside face of the foundations are separated at a minimum slope of 2H to 1V with an imaginary line drawn from the underside of the foundations. The lower footing should be installed first to mitigate the risk of undermining the upper footing.

5.4.6 Estimated Settlement

All individual spread footings should be founded on bedrock, reviewed, and approved by a licensed geotechnical engineer.

Foundations installed in accordance with the recommendations outlined in the preceding sections are not expected to exceed total settlements of 25 mm and differential settlements of 19 mm.

All foundations are to be designed and constructed to the minimum widths as detailed in the 2012 OBC.

5.4.7 Building Drainage

To assist in maintaining the building dry from surface water seepage, it is recommended that exterior grades around the buildings be sloped away at a 2% gradient or more, for a distance of at least 2.0 m. Roof drains should discharge a minimum of 1.5 m away from the structure to a drainage swale or appropriate storm drainage system (i.e., interior sump pit).

5.4.8 Shallow Foundations Frost Protection & Foundation Backfill

In the Ottawa, Ontario area, exterior perimeter foundations for heated buildings require a minimum of 1.8 m of soil cover above the underside of the footing to provide soil cover for frost protection.

It is noted that for foundations established on well-draining bedrock (i.e., no ponding adjacent to the foundation), frost protection is not required. This decision is typically made on Site since each situation will depend on Site specific bedrock conditions.

Where the foundations for heated buildings do not have the minimum 1.8 m of soil cover frost protection, they should be protected from frost with a combination of soil cover and rigid polystyrene insulation, such as Dow Styrofoam or equivalent product.

To minimize potential frost movements from soil frost adhesion, the perimeter foundation backfill should consist of a free draining granular material, such as a Granular ‘B’ Type I (OPSS 1010) or an approved sand fill, extending a minimum lateral distance of 600 mm beyond the foundation. The backfill material used against the foundation must be placed so that the allowable lateral capacity is achieved. All granular material is to be placed in maximum 300 mm thick lifts compacted to a minimum of 100% SPMDD in hard landscaping areas and 95% SPMDD in soft landscaping areas. It is recommended that inspection and testing be carried out during construction to confirm backfill quality, thickness and to ensure compaction requirements are achieved.

5.5 Underground Parking Garage Design

It is understood that the building will be constructed with a single level underground parking garage under the west portion of the Site (Phase I) and two-levels of underground parking under the northeast portion of the Site (Phase II). The underside of the footing for the single level underground parking garage will be located at approximate elevation 97.25 m, while the underside of the footing for the two-level underground parking garage will be located at approximate elevation 93.75 m. As previously mentioned, Pinchin has assumed conservative seasonal high groundwater levels for the Site to be between approximate elevations 94.1 to 95.1 m.

As such, Pinchin recommends that the building be provided with underfloor and foundation wall drainage systems connected to a suitable frost-free outlet due to the groundwater levels at the Site.

The exterior perimeter foundation drains and underfloor drainage system should consist of a minimum 150 mm diameter fabric wrapped perforated drainage tile surrounded by 19 mm diameter clear stone (OPSS 1004) with a minimum cover of 150 mm on top and sides and 50 mm below the drainage tile. Since the natural soil contains a significant amount of silt sized particles, the clear stone gravel should be wrapped in a non-woven geotextile (Terrafix 270R or equivalent). The drainage system is to also include 150 mm diameter sleeves placed at the base of the foundation walls around the perimeter of the building to allow for the water collected in the foundation drains to enter into the underfloor drainage system. The sleeves are to be spaced at a maximum distance of 5.0 m center to center and are to be mechanically connected to the underfloor drainage system on the interior of the building.

The underfloor drainage system is to consist of a minimum 150 mm diameter fabric wrapped perforated drainage tile surrounded by 19 mm diameter clear stone (OPSS 1004) with a minimum cover of 150 mm on the top and sides and 50 mm below the drainage tile. The clear stone gravel should be wrapped in a non-woven geotextile (Terrafix 270R or equivalent). The drainage tile is to be installed in both directions and is to be spaced at a maximum distance of 5.0 m center to center in order to allow for connection to the foundation wall sleeves. The water collected from the drainage system is to be directed to the interior sump pump system where it will be discharged from the Site. All subsurface walls are to be waterproofed.

If the building is constructed below the groundwater table and subdrains and pumps are used to remove the groundwater from around the building footprint, there is the potential that a Permit to Take Water from the Ministry of the Environment, Conservation and Parks will be required for the long term dewatering of the Site.

The walls must also be designed to resist lateral earth pressure. Depending on the design of the building the earth pressure computations must take into account the groundwater level at the Site. For calculating the lateral earth pressure, the coefficient of at-rest earth pressure (K_0) may be assumed at 0.5 for non-cohesive sandy soil. The bulk unit weight of the retained backfill may be taken as 20 kN/m^3 for well compacted soil. An appropriate factor of safety should be applied.

5.5.1 Concrete Floor Slab

Prior to the installation of the engineered fill material, all organics and deleterious materials should be removed to the underlying bedrock surface or native soil. The underlying bedrock and native soil encountered within the boreholes is considered adequate for the support of a concrete floor slab provided it is inspected and approved by an experienced geotechnical engineering consultant.

Based on the in-situ conditions, it is recommended to establish a concrete floor slab-on-grade on a minimum 200 mm thick layer of coarse clean granular material containing not more than 10% material that will pass a 4 mm sieve. Any required up-fill should consist of a Granular 'B' Type I or Type II (OPSS 1010).

The installation of a vapour barrier may be required under the floor slab. If required, the vapour barrier should conform to the flooring manufacturer's and designer's requirements. Consideration may be given to carrying out moisture emission and/or relative humidity testing of the slab to determine the concrete condition prior to flooring installation. To minimize the potential for excess moisture in the floor slab, a concrete mixture with a low water-to-cement ratio (i.e. 0.5 to 0.55) should be used.

A modulus of subgrade reaction of 75 MPa/m can be used for the design of the floor slab founded on the bedrock. A modulus of subgrade reaction of 35 MPa/m can be used for the design of floor slabs founded on the native soil.

5.6 Asphaltic Concrete Pavement Structure Design Parking areas and Access Roadway

5.6.1 Discussion

A portion of the parking areas and access roadway will be constructed outside the underground parking garage footprint, specifically in the northeast portion of the Site where the overburden material is thicker, resulting in the pavement structure overlying the natural subgrade soil. The in-situ natural subgrade soil is considered a sufficient bearing material for an asphaltic concrete pavement structure provided all organics and deleterious materials are removed prior to installing the engineered fill material.

At this time Pinchin is unaware of the proposed final grades for the parking lot and access roadway. As such, provided the pavement structure overlies the in-situ natural soil, the following pavement structure is recommended.

5.6.2 Pavement Structure

The following table presents the minimum specifications for a flexible asphaltic concrete pavement structure:

Pavement Layer	Compaction Requirements	Parking Areas	Driveways
Surface Course Asphaltic Concrete HL-3 (OPSS 1150)	92% MRD as per OPSS 310	40 mm	40 mm

Pavement Layer	Compaction Requirements	Parking Areas	Driveways
Binder Course Asphaltic Concrete HL-8 (OPSS 1150)	92 % MRD as per OPSS 310	50 mm	80 mm
Base Course: Granular "A" (OPSS 1010)	100% Standard Proctor Maximum Dry Density (ASTM-D698)	150 mm	150 mm
Subbase Course: Granular "B" Type I (OPSS 1010)	100% Standard Proctor Maximum Dry Density (ASTM D698)	300 mm	450 mm

Notes:

I. Prior to placing the pavement structure, the subgrade soil is to be proof rolled with a smooth drum roller without vibration to observe weak spots and the deflection of the soil; and

II. The recommended pavement structure may have to be adjusted according to the City of Ottawa standards. Also, if construction takes place during times of substantial precipitation and the subgrade soil becomes wet and disturbed, the granular thickness may have to be increased to compensate for the weaker subgrade soil. In addition, the granular fill material thickness may have to be temporarily increased to allow heavy construction equipment to access the Site, in order to avoid the subgrade from "pumping" up into the granular material.

Performance grade PG 58-34 asphaltic concrete should be specified for Marshall mixes.

5.6.3 Pavement Structure Subgrade Preparation and Granular up Fill

The proper placement of base and subbase fill materials becomes very important in addressing the proper load distribution to provide a durable pavement structure.

The pavement subgrade materials should be thoroughly proof rolled prior to placement of the Granular 'B' subbase course. If any unstable areas are noted, then the Granular 'B' thickness may need to be increased to support pavement construction traffic. This should be left as a field decision by a qualified geotechnical engineer at the time of construction, but it is recommended that additional Granular 'B' be carried as a provisional item under the construction contract.

Where fill material is required to increase the grade to the underside of the pavement structure it should consist of Granular 'B' Type I (OPSS 1010). The up-fill material is to be placed in maximum 300 mm thick lifts compacted to 98% SPMDD within 4% of the optimum moisture content.

Samples of both the Granular 'A' and Granular 'B' Type I aggregates should be tested for conformance to OPSS 1010 prior to utilization on Site and during construction. All stockpiled material should be protected from deleterious materials, additional moisture and be kept from freezing.

Post compaction settlement of fine-grained soil can be expected, even when placed to compaction specifications. As such, fill material should be installed as far in advance as possible before finishing the parking lot and access roadways for best grade integrity.

Where the subgrade material types differ below the underside of the pavement structure, the transition between the materials should be sloped as per frost heave taper OPSD 205.60.

5.6.4 Drainage

Control of surface water is a critical factor in achieving good pavement structure life. The pavement thickness designs are based on a drained pavement subgrade via sub-drains or ditches.

The subgrade soils have poor natural drainage and therefore it is recommended that pavement subdrains be installed in the lower areas and be connected to the catch basins.

The surface of the roadways should be free of depressions and be sloped at a minimum grade of 1% in order to drain to appropriate drainage areas. Subgrade soil should slope a minimum of 3% toward stormwater collection points. Positive slopes are very important for the proper performance of the drainage system. The granular base and subbase materials should extend horizontally to any potential ditches or swales.

In addition, routine maintenance of the drainage systems will assist with the longevity of the pavement structure. Ditches, culverts, sewers and catch basins should be regularly cleared of debris and vegetation

6.0 VISUAL SLOPE CONDITION ASSESSMENT

As requested by the City of Ottawa, Pinchin completed a visual slope condition assessment of the existing slope located adjacent to the north portion of the Site. The assessment was completed by an experienced Pinchin Geotechnical Engineer familiar with slope inspections, using the procedure outlined in the Ministry of Natural Resources (MNR) “Technical Guide – River and Stream Systems: Erosion Hazard Limit” (MNR Technical Guide). The visual inspection consisted of performing a criterion-based slope failure risk assessment using the slope stability rating chart provided by in the MNR Technical Guide.

The slope located adjacent to the north portion of the Site possess an approximate 2H to 1V to more than 3H to 1V inclination with the toe of the slope not located in close proximity to any watercourse. Based on the results of the field investigation, the slope embankment soils consist of silty sand overlying glacial till and bedrock. The slope embankment was observed to be heavily vegetated with mature trees and wild undergrowth noted throughout. The slope was dry at the time of the assessment with little to no areas of active erosion observed. It is noted that no visible signs of soil slumping, bulging, tension cracks or leaning trees on the face of the slope were observed and it is likely that the slope has not experienced any past slope movements.

Based on the above observations, the slope stability rating chart produced a rating summary score of 21. The MNR Technical Guide indicates that slopes with rating scores of less than 24 require a Site inspection only, followed by a summary outlining the observations made during the slope inspection.

It is noted that a portion of the slope embankment is currently being utilized as a downhill mountain bike course with various jumps and berms constructed on the embankment itself. Pinchin recommends that these jumps and berms be removed during the development of the Site to prevent any safety hazards associated with biking from the slope onto the proposed development Site. Once the jumps and berms are removed, healthy vegetation should be added through hydroseeding on the face of the slope.

Any disturbed sloped areas should be restored with suitable native vegetation. Periodic inspections of the slope are recommended throughout the construction process to ensure the slope stability is maintained. If slope instability is observed the slope should be remediated as soon as practically possible to its original configuration. Toe support is especially critical in maintaining a stable slope. Slope configurations should not be altered without the guidance of a qualified geotechnical engineer. In particular, the slope should not be steepened and fill materials should not be placed on the slope at any time during the construction process.

7.0 SITE SUPERVISION & QUALITY CONTROL

It is recommended that all geotechnical aspects of the project be reviewed and confirmed under the appropriate geotechnical supervision, to routinely check such items. This includes but is not limited to inspection and confirmation of the bedrock surface prior to pouring any foundations or footings, backfilling, or engineered fill installation to ensure that the actual conditions are not markedly different than what was observed at the borehole locations and geotechnical components are constructed as per Pinchin's recommendations. Compaction quality control of engineered fill material (full-time monitoring) is recommended as standard practice, as well as regular sampling and testing of aggregates and concrete, to ensure that physical characteristics of materials for compliance during installation and satisfies all specifications presented within this report.

8.0 TERMS AND LIMITATIONS

This Geotechnical Investigation was performed for the exclusive use of Bâtimo Developpement Inc. (Client) in order to evaluate the subsurface conditions at 150 Kanata Avenue & 1200 Canadian Shield Avenue, Kanata, Ontario. Within the limitations of scope, schedule and budget, our services have been executed in accordance with generally accepted practises in the field of geotechnical engineering for the

Site. Classification and identification of soil, and geologic units have been based upon commonly accepted methods employed in professional geotechnical practice. No warranty or other conditions, expressed or implied, should be understood. Conclusions derived are specific to the immediate area of study and cannot be extrapolated extensively away from sample locations.

Performance of this Geotechnical Investigation to the standards established by Pinchin is intended to reduce, but not eliminate, uncertainty regarding the subgrade soil at the Site, and recognizes reasonable limits on time and cost.

Regardless how exhaustive a Geotechnical Investigation is performed; the investigation cannot identify all the subsurface conditions. Therefore, no warranty is expressed or implied that the entire Site is representative of the subsurface information obtained at the specific locations of our investigation. If during construction, subsurface conditions differ from then what was encountered within our test location and the additional subsurface information provided to us, Pinchin should be contacted to review our recommendations. This report does not alleviate the contractor, owner, or any other parties of their respective responsibilities.

This report has been prepared for the exclusive use of the Client and their authorized agents. 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 the third parties. If additional parties require reliance on this report, written authorization from Pinchin will be required. Pinchin disclaims responsibility of consequential financial effects on transactions or property values, or requirements for follow-up actions and costs. No other warranties are implied or expressed. Furthermore, this report should not be construed as legal advice.

Pinchin makes no other representations whatsoever, including those concerning the legal significance of its findings, or as to other legal matters touched on in this report, including, but not limited to, ownership of any property, or the application of any law to the facts set forth herein. With respect to regulatory compliance issues, regulatory statutes are subject to interpretation and these interpretations may change over time. Please refer to Appendix IV, Report Limitations and Guidelines for Use, which pertains to this report.

Specific limitations related to the legal and financial and limitations to the scope of the current work are outlined in our proposal, the attached Methodology, and the Authorization to Proceed, Limitation of Liability and Terms of Engagement which accompanied the proposal.

Information provided by Pinchin is intended for Client use only. Pinchin will not provide results or information to any party unless disclosure by Pinchin is required by law. Any use by a third party of reports or documents authored by Pinchin or any reliance by a third party on or decisions made by a third party based on the findings described in said documents, is the sole responsibility of such third parties.

Pinchin accepts no responsibility for damages suffered by any third party as a result of decisions made or actions conducted. No other warranties are implied or expressed.

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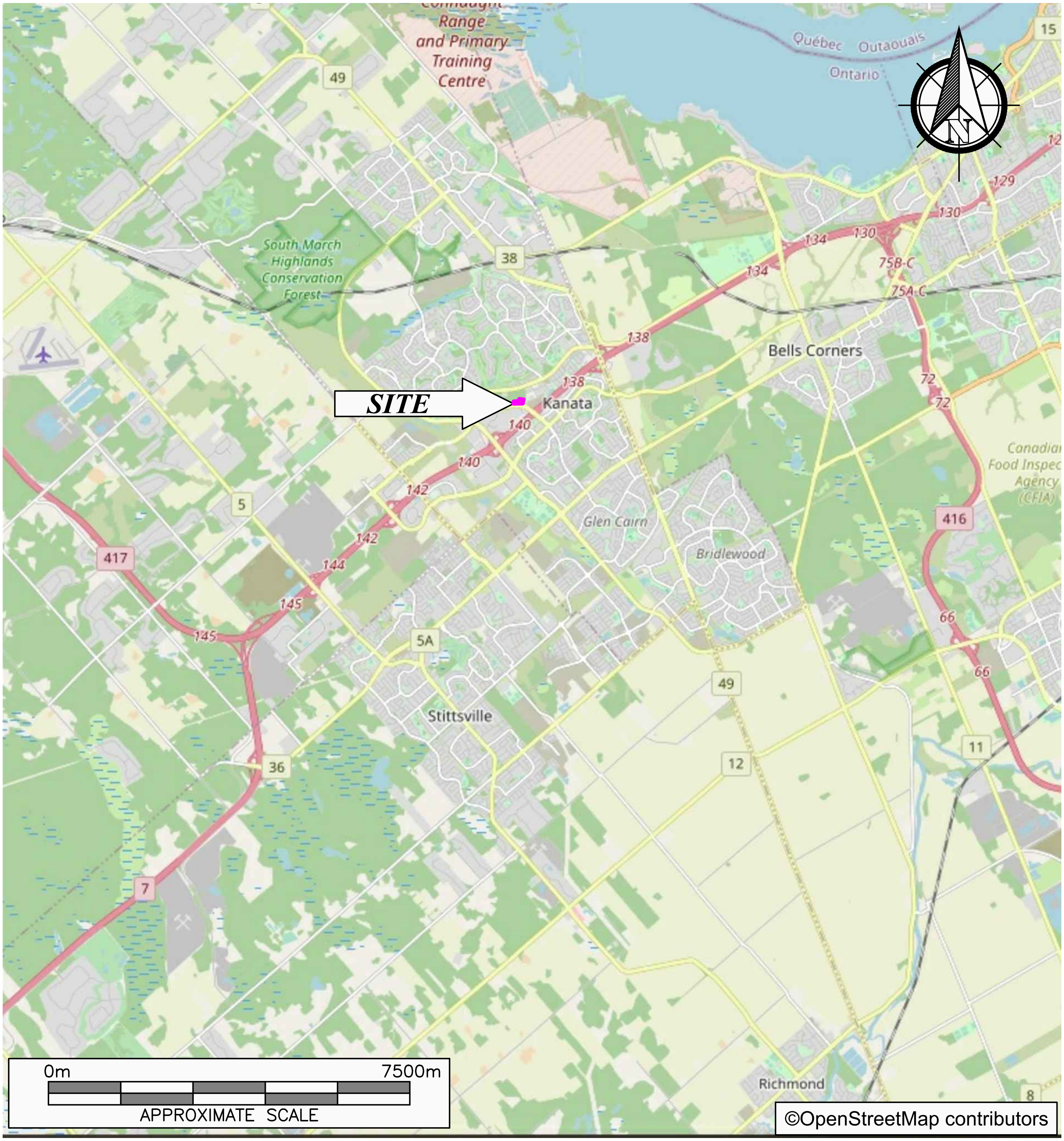
Template: Master Geotechnical Investigation Report – Ontario, GEO, April 1, 2020

FIGURES



PROJECT NAME

GEOTECHNICAL INVESTIGATION

CLIENT NAME

BATIMO DEVELOPPEMENT INC.

PROJECT LOCATION

150 KANATA AVENUE & 1200 CANADIAN SHIELD AVENUE, OTTAWA, ONTARIO

FIGURE NAME

KEY MAP

FIGURE NO.

1

APPROXIMATE SCALE

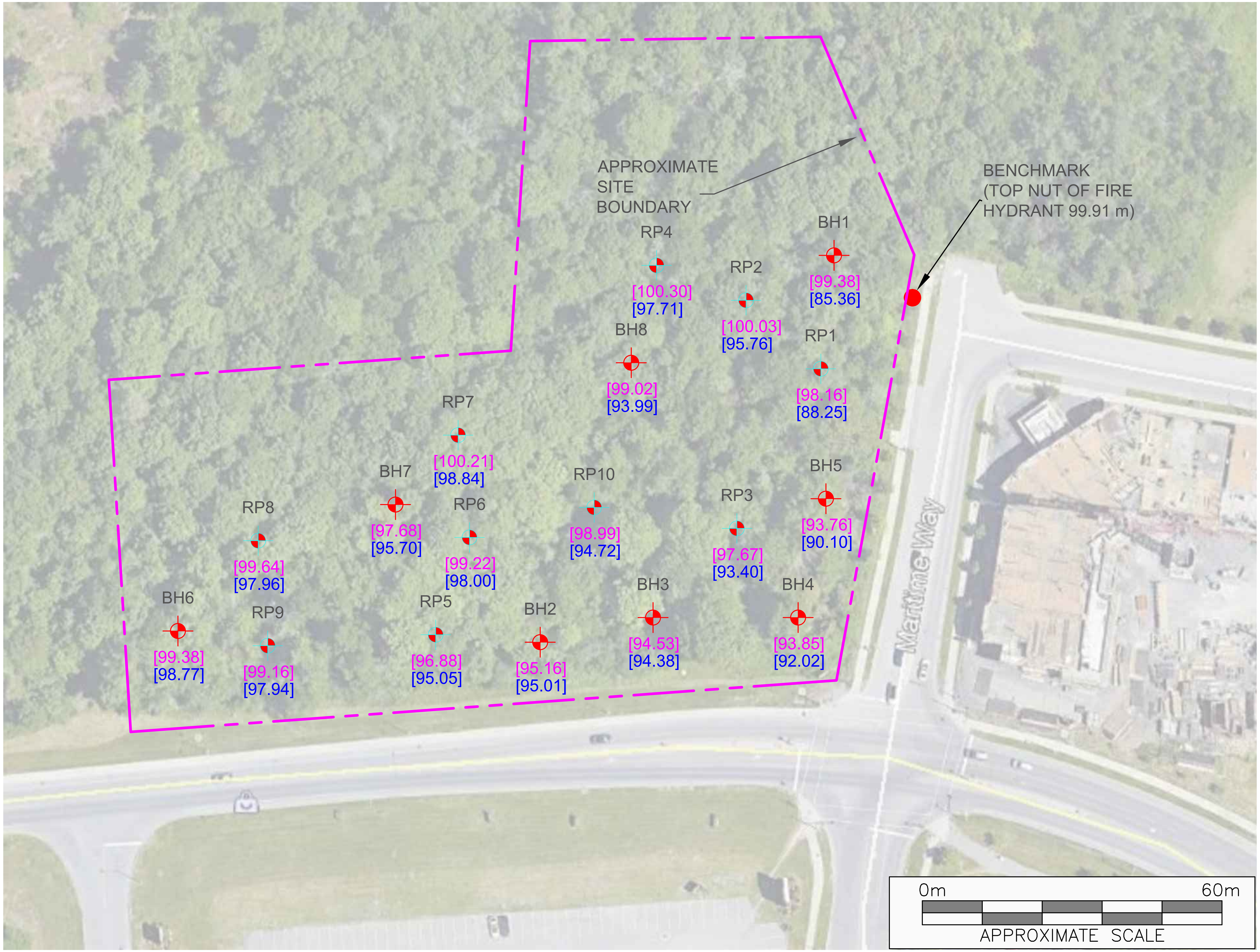
AS SHOWN

PROJECT NO.

290435.001

DATE

MAY 2022



LEGEND

⊕	BOREHOLE LOCATION
⊕	BEDROCK PROBE LOCATION
[XX.XX]	GROUND SURFACE ELEVATION (m)
[XX.XX]	BOREHOLE REFUSAL ELEVATION (m)
m	METRES

PINCHIN

PROJECT NAME
GEOTECHNICAL INVESTIGATION

CLIENT NAME
BATIMO DEVELOPPEMENT INC.

PROJECT LOCATION
150 KANATA AVENUE & 1200 CANADIAN SHIELD AVENUE, OTTAWA, ONTARIO

FIGURE NAME
BOREHOLE LOCATION PLAN

APPROXIMATE SCALE AS SHOWN	PROJECT NO. 290435.001
DATE MAY 2022	FIGURE NO. 2

APPENDIX I

Abbreviations, Terminology and Principle Symbols used in Report and Borehole Logs

ABBREVIATIONS, TERMINOLOGY & PRINCIPAL SYMBOLS USED

Sampling Method

AS	Auger Sample	**w**	Washed Sample
SS	Split Spoon Sample	**HQ**	Rock Core (63.5 mm diam.)
ST	Thin Walled Shelby Tube	**NQ**	Rock Core (47.5 mm diam.)
BS	Block Sample	**BQ**	Rock Core (36.5 mm diam.)

In-Situ Soil Testing

Standard Penetration Test (SPT), "N" value is the number of blows required to drive a 51 mm outside diameter spilt barrel sampler into the soil a distance of 300 mm with a 63.5 kg weight free falling a distance of 760 mm after an initial penetration of 150 mm has been achieved. The SPT, "N" value is a qualitative term used to interpret the compactness condition of cohesionless soils and is used only as a very approximation to estimate the consistency and undrained shear strength of cohesive soils.

Dynamic Cone Penetration Test (DCPT) is the number of blows required to drive a cone with a 60 degree apex attached to "A" size drill rods continuously into the soil for each 300 mm penetration with a 63.5 kg weight free falling a distance of 760 mm.

Cone Penetration Test (CPT) is an electronic cone point with a 10 cm2 base area with a 60 degree apex pushed through the soil at a penetration rate of 2 cm/s.

Field Vane Test (FVT) consists of a vane blade, a set of rods and torque measuring apparatus used to determine the undrained shear strength of cohesive soils.

Soil Descriptions

The soil descriptions and classifications are based on an expanded Unified Soil Classification System (USCS). The USCS classifies soils on the basis of engineering properties. The system divides soils into three major categories; coarse grained, fine grained and highly organic soils. The soil is then subdivided based on either gradation or plasticity characteristics. The classification excludes particles larger than 75 mm. To aid in quantifying material amounts by weight within the respective grain size fractions the following terms have been included to expand the USCS:

Soil Classification		Terminology	Proportion
Clay	< 0.002 mm		
Silt	0.002 to 0.06 mm	“trace”, trace sand, etc.	1 to 10%
Sand	0.075 to 4.75 mm	“some”, some sand, etc.	10 to 20%
Gravel	4.75 to 75 mm	Adjective, sandy, gravelly, etc.	20 to 35%
Cobbles	75 to 200 mm	And, and gravel, and silt, etc.	>35%
Boulders	>200 mm	Noun, Sand, Gravel, Silt, etc.	>35% and main fraction

Notes:

- Soil properties, such as strength, gradation, plasticity, structure, etcetera, dictate the soils engineering behaviour over grain size fractions; and
- With the exception of soil samples tested for grain size distribution or plasticity, all soil samples have been classified based on visual and tactile observations. The accuracy of visual and tactile observation is not sufficient to differentiate between changes in soil classification or precise grain size and is therefore an approximate description.

The following table outlines the qualitative terms used to describe the compactness condition of cohesionless soil:

Cohesionless Soil	
Compactness Condition	**SPT N-Index (blows per 300 mm)**
Very Loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	> 50

The following table outlines the qualitative terms used to describe the consistency of cohesive soils related to undrained shear strength and SPT, N-Index:

Cohesive Soil

Consistency	Undrained Shear Strength (kPa)	SPT N-Index (blows per 300 mm)
Very Soft	<12	<2
Soft	12 to 25	2 to 4
Firm	25 to 50	4 to 8
Stiff	50 to 100	8 to 15
Very Stiff	100 to 200	15 to 30
Hard	>200	>30

Note: Utilizing the SPT, N-Index value to correlate the consistency and undrained shear strength of cohesive soils is only very approximate and needs to be used with caution.

Soil & Rock Physical Properties

General

W	Natural water content or moisture content within soil sample
γ	Unit weight
γ'	Effective unit weight
γ_d	Dry unit weight
γ_{sat}	Saturated unit weight
ρ	Density
ρ_s	Density of solid particles
ρ_w	Density of Water
ρ_d	Dry density
ρ_{sat}	Saturated density e Void ratio
n	Porosity
S_r	Degree of saturation
E_{50}	Strain at 50% maximum stress (cohesive soil)

Consistency

W_L	Liquid limit
W_P	Plastic Limit
I_P	Plasticity Index
W_S	Shrinkage Limit
I_L	Liquidity Index
I_C	Consistency Index
e_{max}	Void ratio in loosest state
e_{min}	Void ratio in densest state
I_D	Density Index (formerly relative density)

Shear Strength

C_u, S_u	Undrained shear strength parameter (total stress)
C'_d	Drained shear strength parameter (effective stress)
r	Remolded shear strength
τ_p	Peak residual shear strength
τ_r	Residual shear strength
ø’	Angle of interface friction, coefficient of friction = tan ø’

Consolidation (One Dimensional)

C_C	Compression index (normally consolidated range)
C_r	Recompression index (over consolidated range)
C_S	Swelling index
m_V	Coefficient of volume change
c_V	Coefficient of consolidation
T_V	Time factor (vertical direction)
U	Degree of consolidation
σ'_o	Overburden pressure
$\sigma' p$	Preconsolidation pressure (most probable)
OCR	Overconsolidation ratio

Permeability

The following table outlines the terms used to describe the degree of permeability of soil and common soil types associated with the permeability rates**:**

Permeability (k cm/s)	**Degree of Permeability**	**Common Associated Soil Type**
$> 10^{-1}$	Very High	Clean gravel
10^{-1} to 10^{-3}	High	Clean sand, Clean sand and gravel
10^{-3} to 10^{-5}	Medium	Fine sand to silty sand
10^{-5} to 10^{-7}	Low	Silt and clayey silt (low plasticity)
$>10^{-7}$	Practically Impermeable	Silty clay (medium to high plasticity)

Rock Coring

Rock Quality Designation (RQD) is an indirect measure of the number of fractures within a rock mass, Deere et al. (1967). It is the sum of sound pieces of rock core equal to or greater than 100 mm recovered from the core run, divided by the total length of the core run, expressed as a percentage. If the core section is broken due to mechanical or handling, the pieces are fitted together and if 100 mm or greater included in the total sum**.**

RQD is calculated as follows:

$$\text{RQD (\%)} = \frac{\Sigma \text{ Length of core pieces} > 100 \text{ mm} \times 100}{\text{Total length of core run}}$$

The following is the Classification of Rock with Respect to RQD Value:

RQD Classification	**RQD Value (%)**
Very poor quality	**<25**
Poor quality	25 to 50
Fair quality	50 to 75
Good quality	75 to 90
Excellent quality	90 to 100

APPENDIX II

Pinchin's Borehole Logs

PINCHIN

Log of Borehole: BH1

Project #: 290435.001 **Logged By:** WT

Project: Geotechnical Investigation

Client: Bâtimo Developpement Inc.

Location: 150 Kanata Ave & 1200 Canadian Shield Ave, Kanata, Ontario

Drill Date: May 18, 2021 **Project Manager:** WT

SUBSURFACE PROFILE

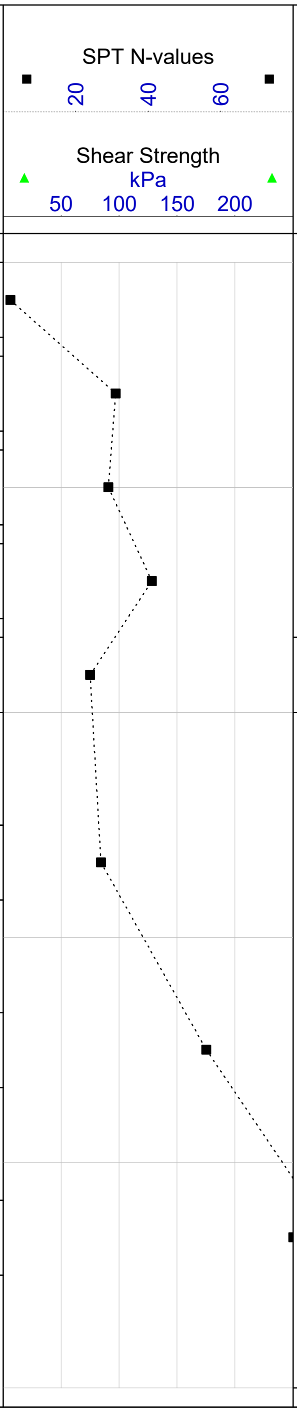
Description	Elevation (m)
Ground Surface	99.38
Organics ~ 300 mm	99.08
Silty Sand Brown silty sand, trace gravel, very loose, damp	98.62
Sand Brown sand, some silt, trace gravel, dense, damp	97.86
Glacial Till Grey clayey silt, some sand. trace gravel, compact, moist	97.09
Dense	96.33
Compact	93.28
Very dense	91.76
Brown gravelly sand, trace silt, trace clay, very dense, wet	

Depth (m): 0 – 9

Monitoring Well Details: No Monitoring Well Installed

SAMPLE

Sample Type	Sampler #	Recovery (%)	SPT N-values	Lab Analysis	Moisture (%)	Plasticity Index
SS	1	80	2			
SS	2	60	31			
SS	3	100	29			
SS	4	100	41			
SS	5	80	24	Hyd.	16.9	
SS	6	20	27			
SS	7	80	56			
SS	8	80	91			



Contractor: Strata Drilling Group

Grade Elevation: 99.38 m

Drilling Method: Hollow Stem Augers / Split Spoon / DCPT

Top of Casing Elevation: N/A

Well Casing Size: N/A

Sheet: 1 of 2

Log of Borehole: BH1

Project #: 290435.001 **Logged By:** WT

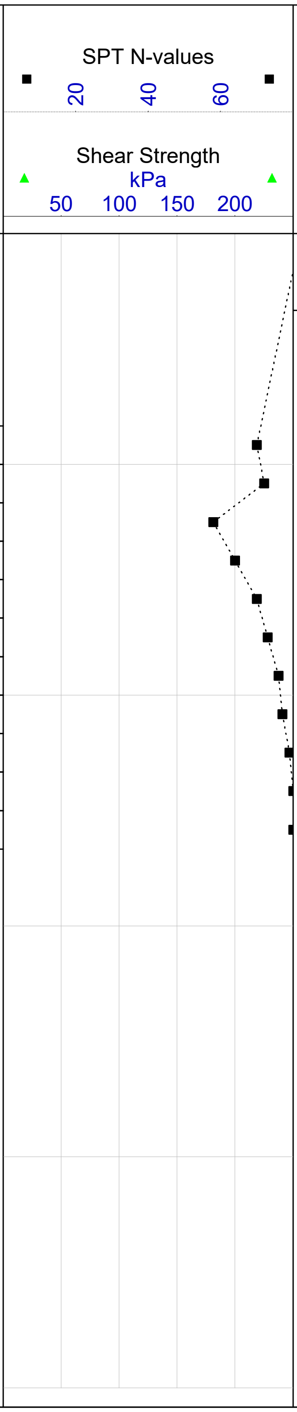
Project: Geotechnical Investigation

Client: Bâtimo Developpement Inc.

Location: 150 Kanata Ave & 1200 Canadian Shield Ave, Kanata, Ontario

Drill Date: May 18, 2021 **Project Manager:** WT

SUBSURFACE PROFILE					SAMPLE						
Depth (m)	Symbol	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-values	Lab Analysis	Moisture (%)	Plasticity Index
									Hyd.		
10											
		Start DCPT - Probable Glacial Till	88.71		DCPT	9	N/A	70			
11					DCPT	10	N/A	72			
					DCPT	11	N/A	58			
					DCPT	12	N/A	64			
12					DCPT	13	N/A	70			
					DCPT	14	N/A	73			
					DCPT	15	N/A	76			
13					DCPT	16	N/A	77			
					DCPT	17	N/A	79			
					DCPT	18	N/A	80			
			85.36		DCPT	19	N/A	100			
14		End of Borehole Borehole terminated at approximately 14.0 mbgs, due to DCPT refusal on probable bedrock.									
15											
16											
17											
18											



Contractor: Strata Drilling Group

Grade Elevation: 99.38 m

Drilling Method: Hollow Stem Augers / Split Spoon / DCPT

Top of Casing Elevation: N/A

Well Casing Size: N/A

Sheet: 2 of 2

Log of Borehole: BH2

Project #: 290435.001 **Logged By:** WT

Project: Geotechnical Investigation

Client: Bâtimo Developpement Inc.

Location: 150 Kanata Ave & 1200 Canadian Shield Ave, Kanata, Ontario

Drill Date: May 18, 2021 **Project Manager:** WT

SUBSURFACE PROFILE					SAMPLE							
Depth (m)	Symbol	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-values	SPT N-values ■ 20 40 60 ■ Shear Strength kPa ▲ 50 100 150 200 ▲	Lab Analysis	Moisture (%)	Plasticity Index
		Ground Surface	95.16									
0		***Organics*** ~ 150 mm	95.01	No Monitoring Well Installed								
		Borehole terminated at 0.2 mbgs due to auger refusal on probable bedrock.										
1												

Contractor: Strata Drilling Group

Drilling Method: Hollow Stem Augers / Split Spoon

Well Casing Size: N/A

Grade Elevation: 95.16 m

Top of Casing Elevation: N/A

Sheet: 1 of 1

Log of Borehole: BH3

Project #: 290435.001 **Logged By:** WT

Project: Geotechnical Investigation

Client: Bâtimo Developpement Inc.

Location: 150 Kanata Ave & 1200 Canadian Shield Ave, Kanata, Ontario

Drill Date: May 18, 2021 **Project Manager:** WT

SUBSURFACE PROFILE					SAMPLE							
Depth (m)	Symbol	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-values	SPT N-values ■ 20 40 60 ■ Shear Strength kPa ▲ 50 100 150 200 ▲	Lab Analysis	Moisture (%)	Plasticity Index
		Ground Surface	94.53	No Monitoring Well Installed								
0		**Organics** ~ 150 mm	94.38									
		Borehole terminated at 0.2 mbgs due to auger refusal on probable bedrock.										
1												

Contractor: Strata Drilling Group

Grade Elevation: 94.53 m

Drilling Method: Hollow Stem Augers / Split Spoon

Top of Casing Elevation: N/A

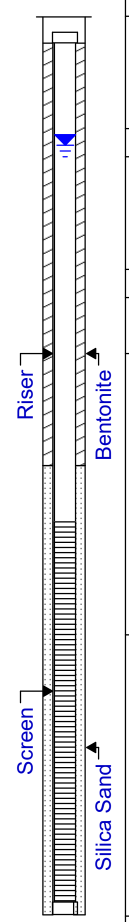
Well Casing Size: N/A

Sheet: 1 of 1

Log of Borehole: BH4

Project #: 290435.001
Logged By: WT
Project: Geotechnical Investigation
Client: Bâtimo Developpement Inc.
Location: 150 Kanata Ave & 1200 Canadian Shield Ave, Kanata, Ontario
Drill Date: May 18, 2021
Project Manager: WT

SUBSURFACE PROFILE				SAMPLE						
Depth (m)	Symbol	Description	Elevation (m)	Sample Type	Sampler #	Recovery (%)	SPT N-values	Lab Analysis	Moisture (%)	Plasticity Index
0		Ground Surface	93.85							
0	≈	**Organics** Black organics, very loose to loose, wet		SS	1	80	0			
				SS	2	60	0			
			92.02	SS	3	60	5			
2	▦	**Bedrock** Limestone		RC	4	79	N/A			38
			88.97	RC	5	92	N/A			88
5		End of Borehole Borehole terminated at 4.9 mbgs.								



Groundwater level = 0.7 mbgs, as measured on June 18, 2021.

Contractor: Strata Drilling Group
Grade Elevation: 93.85 m
Drilling Method: Hollow Stem Augers / Split Spoon / Diamond Bit
Top of Casing Elevation: 94.75 m
Well Casing Size: N/A
Sheet: 1 of 1

Log of Borehole: BH5

Project #: 290435.001 **Logged By:** WT

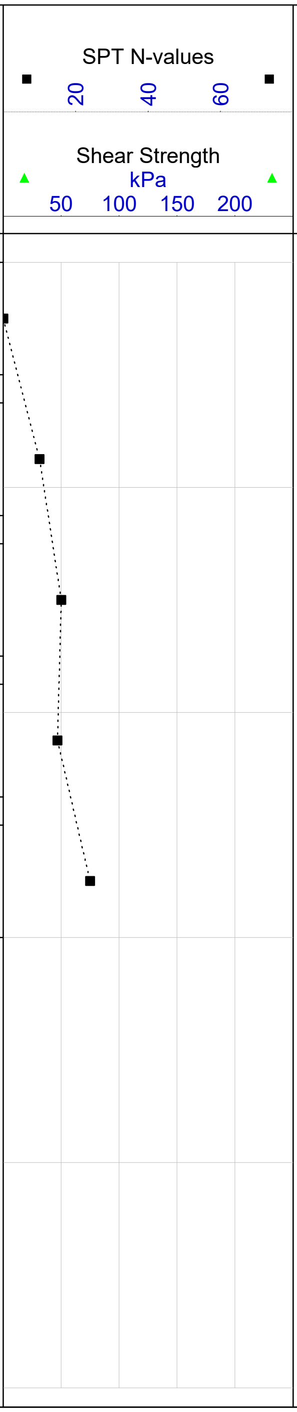
Project: Geotechnical Investigation

Client: Bâtimo Developpement Inc.

Location: 150 Kanata Ave & 1200 Canadian Shield Ave, Kanata, Ontario

Drill Date: May 18, 2021 **Project Manager:** WT

SUBSURFACE PROFILE					SAMPLE						
Depth (m)	Symbol	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-values	Lab Analysis	Moisture (%)	Plasticity Index
0		Ground Surface	93.76								
		Organics Black organics, very loose, wet	93.15	No Monitoring Well Installed	SS	1	80	0			
1		**Sand** Grey sand, trace silt, trace grave, compact, moist	92.24		SS	2	80	10			
2 3		**Glacial Till** Brown gravelly sand, trace silt, trace clay, compact, wet	90.10		SS SS SS	3 4 5	80 70 60	16 15 24			
4 5 6		End of Borehole Borehole terminated at 3.7 mbgs due to auger refusal on probable bedrock.									



Contractor: Strata Drilling Group **Grade Elevation:** 93.76 m

Drilling Method: Hollow Stem Augers / Split Spoon **Top of Casing Elevation:** N/A

Well Casing Size: N/A **Sheet:** 1 of 1

Log of Borehole: BH6

Project #: 290435.001 **Logged By:** WT

Project: Geotechnical Investigation

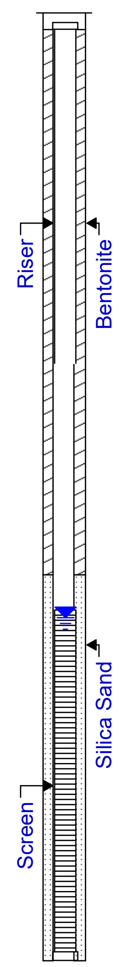
Client: Bâtimo Developpement Inc.

Location: 150 Kanata Ave & 1200 Canadian Shield Ave, Kanata, Ontario

Drill Date: May 18, 2021 **Project Manager:** WT

SUBSURFACE PROFILE

Depth (m)	Description	Elevation (m)
0	Ground Surface	99.38
0 – 0.30	**Organics** Black organics, loose, wet	99.08
0.30 – 0.61	**Sand** Brown sand, trace gravel, trace silt, loose, damp	98.77
0.61 – 8.23	**Bedrock** Limestone bedrock	91.15
	End of Borehole Borehole terminated at 4.9 mbgs.	



Groundwater level = 5.25 mbgs, as measured on June 18, 2021.

SAMPLE

Sample Type	Sampler #	Recovery (%)	SPT N-values	Plasticity Index
SS	1	60	9	
RC	2	100	N/A	37
RC	3	100	N/A	63
RC	4	100	N/A	19
RC	5	100	N/A	13
RC	6	100	N/A	0

SPT N-values: 20 40 60

Shear Strength kPa: 50 100 150 200

Lab Analysis

Moisture (%)

Contractor: Strata Drilling Group

Grade Elevation: 99.38 m

Drilling Method: Hollow Stem Augers / Split Spoon / Diamond Bit

Top of Casing Elevation: 100.28 m

Well Casing Size: N/A

Sheet: 1 of 1

Log of Borehole: BH7

Project #: 290435.001 **Logged By:** WT

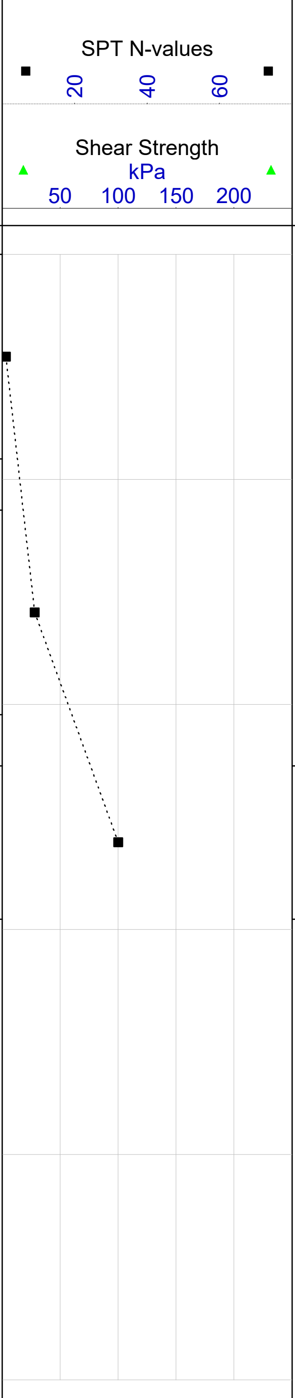
Project: Geotechnical Investigation

Client: Bâtimo Developpement Inc.

Location: 150 Kanata Ave & 1200 Canadian Shield Ave, Kanata, Ontario

Drill Date: May 19, 2021 **Project Manager:** WT

SUBSURFACE PROFILE					SAMPLE							
Depth (m)	Symbol	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-values	SPT N-values ■ 20 40 60 ■ Shear Strength ▲ kPa ▲ 50 100 150 200	Lab Analysis	Moisture (%)	Plasticity Index
		Ground Surface	97.68									
0		***Organics*** Black organics, loose, moist	96.92	No Monitoring Well Installed	SS	1	30	1				
1		***Silty Sand*** Brown silty sand, trace gravel, loose, moist	96.16		SS	2	80	9				
		Glacial Till Brown sand and silt, some gravel, trace clay, dense, moist	95.70		SS	3	80	32		Hyd.		
2 3		End of Borehole Borehole terminated at 2.0 mbgs due to auger refusal on probable bedrock. No free groundwater was encountered at drilling										



Contractor: Strata Drilling Group **Grade Elevation:** 97.68 m

Drilling Method: Hollow Stem Augers / Split Spoon **Top of Casing Elevation:** N/A

Well Casing Size: N/A **Sheet:** 1 of 1

Log of Borehole: BH8

Project #: 290435.001 **Logged By:** WT

Project: Geotechnical Investigation

Client: Bâtimo Developpement Inc.

Location: 150 Kanata Ave & 1200 Canadian Shield Ave, Kanata, Ontario

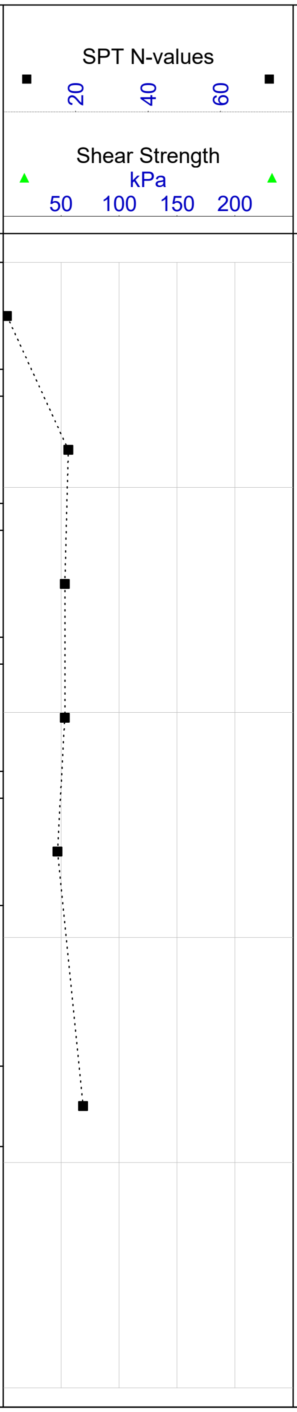
Drill Date: May 20, 2021 **Project Manager:** WT

SUBSURFACE PROFILE

Depth (m)	Description	Elevation (m)	Monitoring Well Details
	Ground Surface	99.02	
0	**Organics** Black organics, loose, moist	98.87	No Monitoring Well Installed
	Silty Sand Brown silty sand, trace gravel, very loose, damp	98.26	
1	**Glacial Till** Brown sand and silt, some gravel, trace clay, compact, moist	96.73	
2	Grey silt, trace sand, trace gravel, compact, damp to moist	95.97	
3	Grey sand and silt, some gravel, trace clay, compact, moist	93.99	
5	End of Borehole Borehole terminated at 5.0 mbgs due to auger refusal on probable bedrock. No free groundwater was encountered at drilling completion.		

SAMPLE

Sample Type	Sampler #	Recovery (%)	SPT N-values	Lab Analysis	Moisture (%)	Plasticity Index
SS	1	50	1			
SS	2	80	18			
SS	3	80	17			
SS	4	90	17			
SS	5	60	15			
SS	6	80	22			



Contractor: Strata Drilling Group

Drilling Method: Hollow Stem Augers / Split Spoon

Well Casing Size: N/A

Grade Elevation: 99.02 m

Top of Casing Elevation: N/A

Sheet: 1 of 1

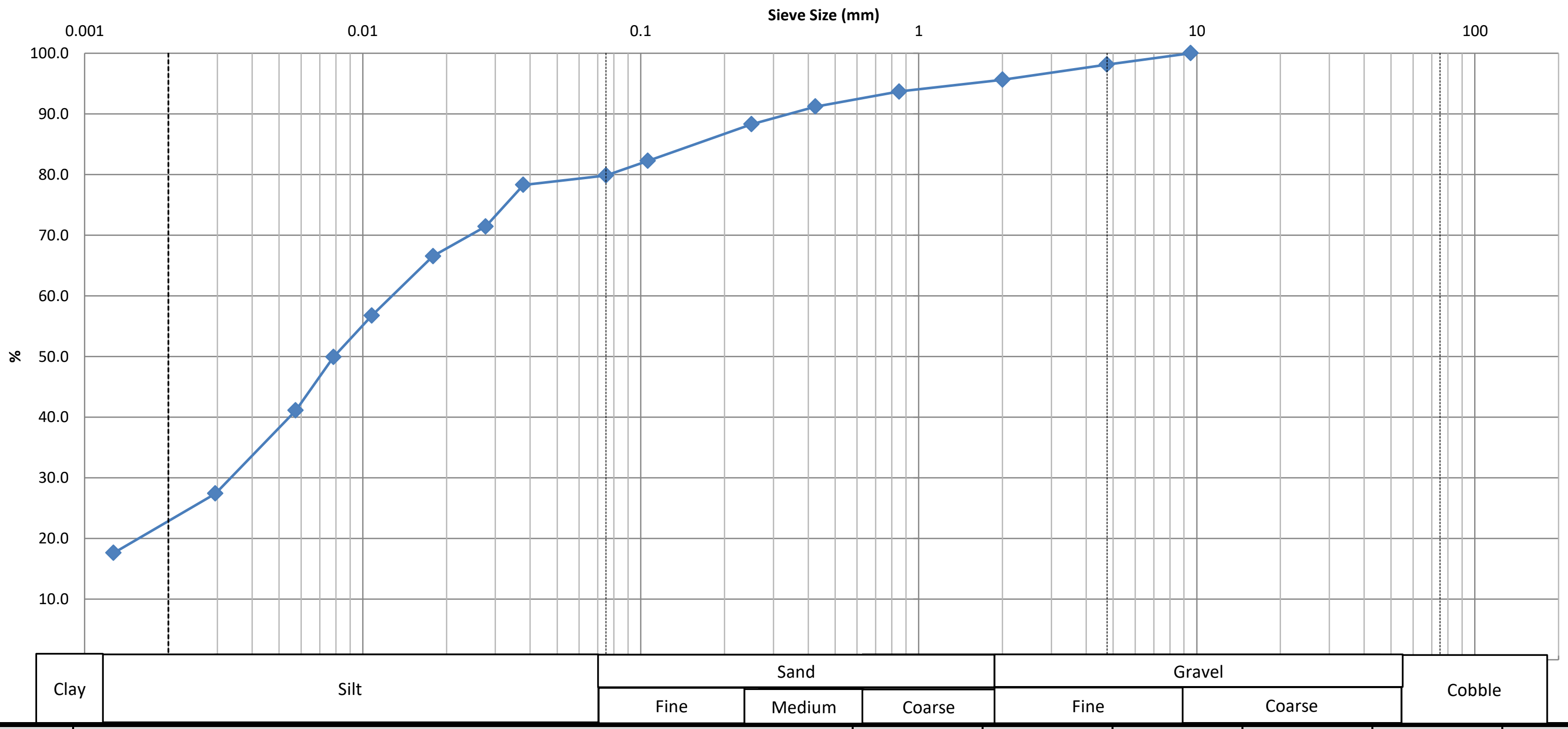
APPENDIX III

Laboratory Testing Reports for Soil Samples

patersongroup
consulting engineers

SIEVE ANALYSIS
ASTM C136

CLIENT:	Pinchin	DEPTH:	10' - 12'	FILE NO:	PM4184
CONTRACT NO.:		BH OR TP No.:	BH1	LAB NO:	24772
PROJECT:	290435.001			DATE RECEIVED:	1-Jun-21
				DATE TESTED:	3-Jun-21
DATE SAMPLED:	31-May-21			DATE REPORTED:	7-Jun-21
SAMPLED BY:	Client			TESTED BY:	D.B



Identification	Soil Classification	MC(%)	LL	PL	PI	Cc	Cu
		16.9					

D100	D60	D30	D10	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
				1.9	18.2	56.9	23.0

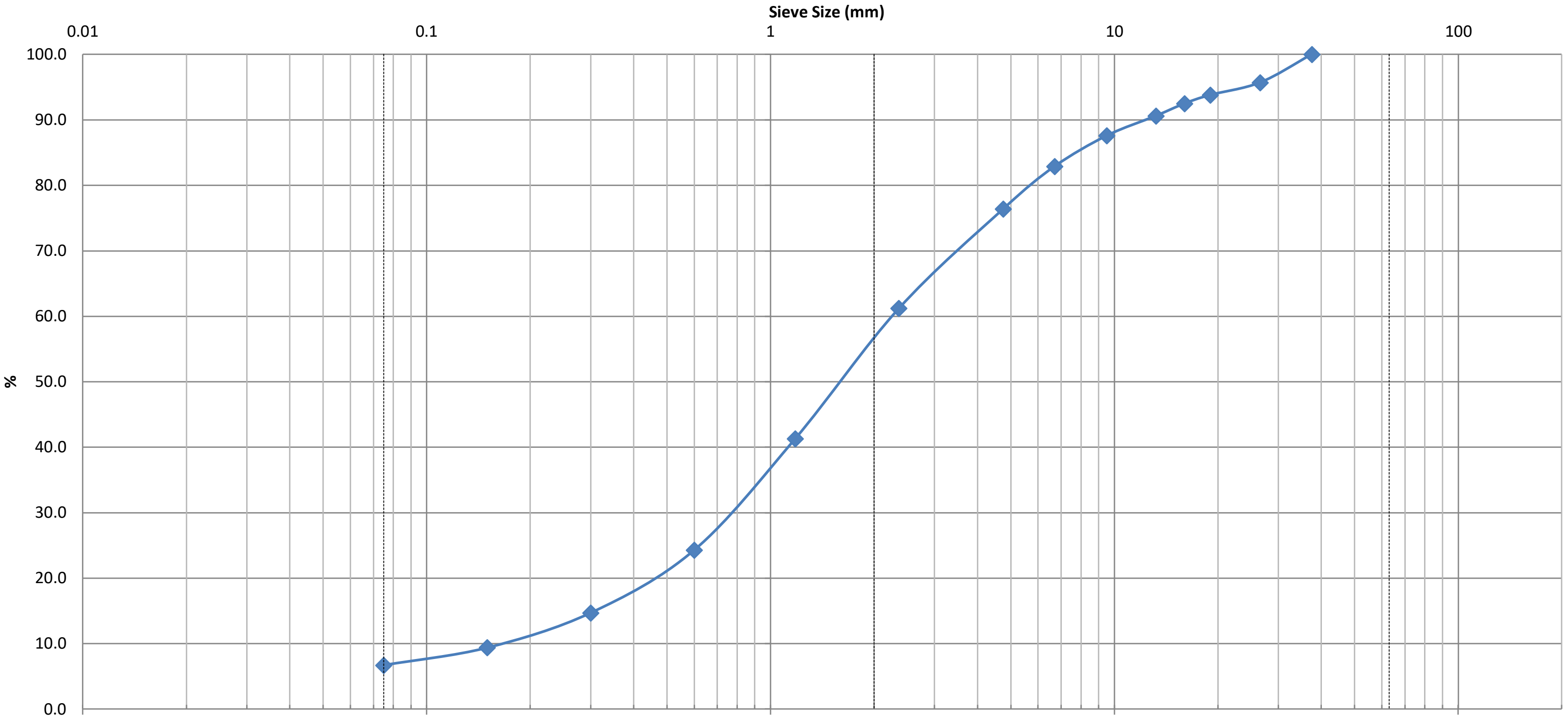
Comments:

REVIEWED BY:	**Curtis Beadow**	**Joe Forsyth, P. Eng.**

patersongroup
consulting engineers

SIEVE ANALYSIS
ASTM C136

CLIENT:	Pinchin	DESCRIPTION:	Sand / Gravel	FILE NO:	PM4184
CONTRACT NO.:	-	SPECIFICATION:	**Sand w Gravel**	LAB NO:	24771
PROJECT:	290435.001	INTENDED USE:	-	DATE RECEIVED:	31-May-21
		PIT OR QUARRY:	-	DATE TESTED:	1-Jun-21
DATE SAMPLED:	31-May-21	SOURCE LOCATION:	BH1	DATE REPORTED:	7-Jun-21
SAMPLED BY:	client	SAMPLE LOCATION:	30' - 32'	TESTED BY:	DK/CP



Silt and Clay	Sand			Gravel		Cobble
	Fine	Medium	Coarse	Fine	Coarse	

Identification	**Soil Classification**	**MC(%)**	**LL**	**PL**	**PI**	**Cc**	**Cu**
						1.56	13.5
D100	D60	D30	D10	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
37.5	2.3	0.78	0.17	23.6	69.7	6.7	

Comments:

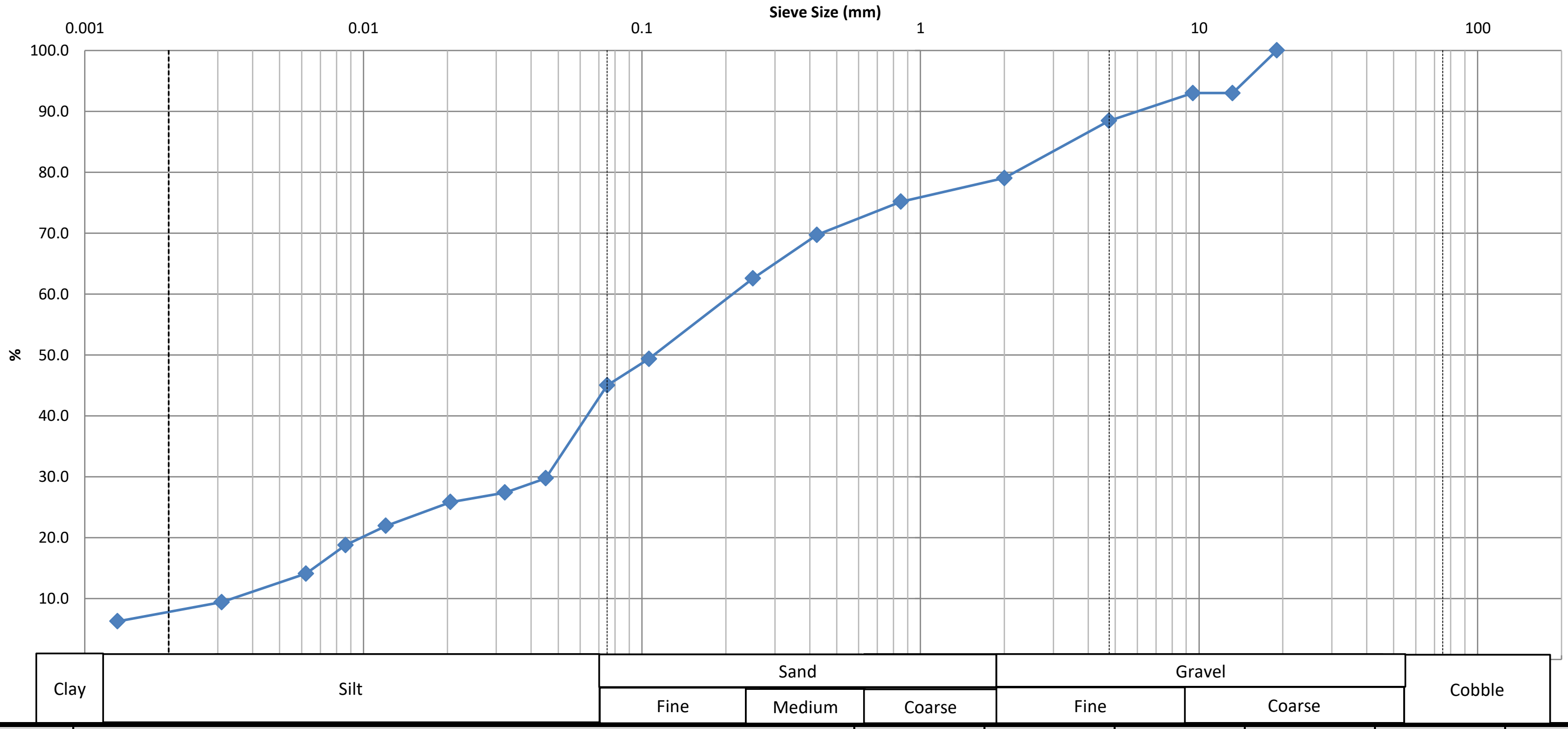
Curtis Beadow

Joe Fosyth, P. Eng.

patersongroup
consulting engineers

SIEVE ANALYSIS
ASTM C136

CLIENT:	Pinchin	DEPTH:	5' - 6.5'	FILE NO:	PM4184
CONTRACT NO.:		BH OR TP No.:	BH7	LAB NO:	24773
PROJECT:	290435.001			DATE RECEIVED:	1-Jun-21
				DATE TESTED:	3-Jun-21
DATE SAMPLED:	31-May-21			DATE REPORTED:	7-Jun-21
SAMPLED BY:	Client			TESTED BY:	D.B.



Identification	Soil Classification				MC(%)	LL	PL	PI	Cc	Cu
					10.2					
	D100	D60	D30	D10	Gravel (%)		Sand (%)	Silt (%)	Clay (%)	
					11.5		43.5	37.0	8.0	

Comments:

REVIEWED BY:	**Curtis Beadow**	**Joe Forsyth, P. Eng.**

APPENDIX IV

Rock Core Photographs

Photo 1 – Borehole BH4, Rock Core (Runs 1 and 2)

Photo 2 – Borehole BH6, Rock Core (Runs 1 to 3)

Photo 3 – Borehole BH6, Rock Core (Runs 4 and 5)

Photo 4 – Bedrock Probe RP4, Rock Core (Runs 1 to 3)

Photo 5 – Bedrock Probe RP4, Rock Core (Run 4)

Photo 6 – Bedrock Probe RP9, Rock Core (Runs 1 to 3)

Photo 6 – Bedrock Probe RP9, Rock Core (Runs 4 and 5)

APPENDIX V

Slope Stability Rating Chart

TABLE 4.2 - SLOPE STABILITY RATING CHART

Site Location: 150 Kanata Ave & 1200 Canadian Shield Ave
File No. 290435.001
Property Owner:
Inspection Date: April. 14, 2022
Inspected By: Wesley Tabaczuk
Weather: Sunny

1. SLOPE INCLINATION

	degrees	horiz. : vert.	
a)	18 or less	3 : 1 or flatter	0
b)	18 - 26	2 : 1 to more than 3 : 1	[✓ 6]
c)	more than 26	steeper than 2 : 1	16

2. SOIL STRATIGRAPHY

a)	Shale, Limestone, Granite (Bedrock)	0
b)	Sand, Gravel	6
c)	Glacial Till	[✓ 9]
d)	Clay, Silt	12
e)	Fill	16
f)	Leda Clay	24

3. SEEPAGE FROM SLOPE FACE

a)	None or Near bottom only	[✓ 0]
b)	Near mid-slope only	6
c)	Near crest only or, From several levels	12

4. SLOPE HEIGHT

a)	2 m or less	0
b)	2.1 to 5 m	2
c)	5.1 to 10 m	[✓ 4]
d)	more than 10 m	8

5. VEGETATION COVER ON SLOPE FACE

a)	Well vegetated; heavy shrubs or forested with mature trees	[✓ 0]
b)	Light vegetation; Mostly grass, weeds, occasional trees, shrubs	4
c)	No vegetation, bare	8

6. TABLE LAND DRAINAGE

a)	Table land flat, no apparent drainage over slope	0
b)	Minor drainage over slope, no active erosion	[✓ 2]
c)	Drainage over slope, active erosion, gullies	4

7. PROXIMITY OF WATERCOURSE TO SLOPE TOE

a)15 metres or more from slope toe	[✓ 0]
b)Less than 15 metres from slope toe	6

8. PREVIOUS LANDSLIDE ACTIVITY

a)	No	[✓ 0]
b)	Yes	6

SLOPE INSTABILITY RATING VALUES INVESTIGATION RATING SUMMARY **TOTAL** 21

APPENDIX VI

Report Limitations and Guidelines for Use

REPORT LIMITATIONS & GUIDELINES FOR USE

This information has been provided to help manage risks with respect to the use of this report.

GEOTECHNICAL SERVICES ARE PERFORMED FOR SPECIFIC PURPOSES, PERSONS AND PROJECTS

This report was prepared for the exclusive use of the Client and their authorized agents, subject to the conditions and limitations contained within the duly authorized work plan. 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 the third parties. If additional parties require reliance on this report, written authorization from Pinchin will be required. Pinchin disclaims responsibility of consequential financial effects on transactions or property values, or requirements for follow-up actions and costs. No other warranties are implied or expressed. Furthermore, this report should not be construed as legal advice.

SUBSURFACE CONDITIONS CAN CHANGE

This geotechnical report is based on the existing conditions at the time the study was performed, and Pinchin's opinion of soil conditions are strictly based on soil samples collected at specific test hole locations. The findings and conclusions of Pinchin's reports may be affected by the passage of time, by manmade events such as construction on or adjacent to the Site, or by natural events such as floods, earthquakes, slope instability or groundwater fluctuations.

LIMITATIONS TO PROFESSIONAL OPINIONS

Interpretations of subsurface conditions are based on field observations from test holes that were spaced to capture a 'representative' snap shot of subsurface conditions. Site exploration identifies subsurface conditions only at points of sampling. Pinchin reviews field and laboratory data and then applies professional judgment to formulate an opinion of subsurface conditions throughout the Site. Actual subsurface conditions may differ, between sampling locations, from those indicated in this report.

LIMITATIONS OF RECOMMENDATIONS

Subsurface soil conditions should be verified by a qualified geotechnical engineer during construction. Pinchin should be notified if any discrepancies to this report or unusual conditions are found during construction.

Sufficient monitoring, testing and consultation should be provided by Pinchin during construction and/or excavation activities, to confirm that the conditions encountered are consistent with those indicated by the test hole investigation, and to provide recommendations for design changes should the conditions revealed during the work differ from those anticipated. In addition, monitoring, testing and consultation by Pinchin should be completed to evaluate whether or not earthwork activities are completed in accordance with our recommendations.   Retaining Pinchin for construction observation for this project is the most effective method of managing the risks associated with unanticipated conditions. However, please be advised that any construction/excavation observations by Pinchin is over and above the mandate of this geotechnical evaluation and therefore, additional fees would apply.

MISINTERPRETATION OF GEOTECHNICAL ENGINEERING REPORT

Misinterpretation of this report by other design team members can result in costly problems. You could lower that risk by having Pinchin confer with appropriate members of the design team after submitting the report. Also retain Pinchin to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering or geologic report. Reduce that risk by having Pinchin participate in pre-bid and preconstruction conferences, and by providing construction observation. Please be advised that retaining Pinchin to participation in any ‘other’ activities associated with this project is over and above the mandate of this geotechnical investigation and therefore, additional fees would apply.

CONTRACTORS RESPONSIBILITY FOR SITE SAFETY

This geotechnical report is not intended to direct the contractor's procedures, methods, schedule or management of the work Site. The contractor is solely responsible for job Site safety and for managing construction operations to minimize risks to on-Site personnel and to adjacent properties. It is ultimately the contractor’s responsibility that the Ontario Occupational Health and Safety Act is adhered to, and Site conditions satisfy all ‘other’ acts, regulations and/or legislation that may be mandated by federal, provincial and/or municipal authorities.

SUBSURFACE SOIL AND/OR GROUNDWATER CONTAMINATION

This report is geotechnical in nature and was not performed in accordance with any environmental guidelines. As such, any environmental comments are very preliminary in nature and based solely on field observations. Accordingly, the scope of services do not include any interpretations, recommendations, findings, or conclusions regarding the, assessment, prevention or abatement of contaminants, and no conclusions or inferences should be drawn regarding contamination, as they may relate to this project. The term "contamination" includes, but is not limited to, molds, fungi, spores, bacteria, viruses, PCBs, petroleum hydrocarbons, inorganics, pesticides/insecticides, volatile organic compounds, polycyclic aromatic hydrocarbons and/or any of their by-products.

Pinchin will not be responsible for any consequential or indirect damages. Pinchin will only be held liable for damages resulting from the negligence of Pinchin. Pinchin will not be liable for any losses or damage if the Client has failed, within a period of two years following the date upon which the claim is discovered within the meaning of the Limitations Act, 2002 (Ontario), to commence legal proceedings against Pinchin to recover such losses or damage.