DRAFT

Geotechnical Investigation – Proposed Residential Development

Heron Gate 5, Ottawa, Ontario

Prepared for:

Hazelview Developments Inc.
1133 Yonge Street, 4th Floor
Toronto, ON M4T 2Y7

October 8, 2021

Pinchin File: 288344.001

Issued to: Hazelview Developments Inc.
Issued on: October 8, 2021
Pinchin File: 288344.001
Issuing Office: Mississauga, ON

Author: Wesley Tabaczuk, P.Eng.
Project Manager, Geotechnical Services
613.592.3387
wtabaczuk@pinchin.com

Reviewer: Vanessa Marshall, M.Eng., P.Eng.
National Practice Leader, Geotechnical Services
226.808.5335
vmarshall@pinchin.com

TABLE OF CONTENTS

FIGURES

APPENDICES

1.0 INTRODUCTION AND SCOPE

Pinchin Ltd. (Pinchin) was retained by Hazelview Developments Inc. (Client) to conduct a Geotechnical Investigation and provide subsequent geotechnical design recommendations for the proposed residential development to be located at Heron Gate 5, Ottawa, 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 will consist of three (3) six-storey residential apartment buildings which will be complete with a single level underground parking garage which will occupy the majority of the Site footprint. At this time the depth to the underside of the footings for the proposed underground parking level are unknown; as such, for the purpose of this report, Pinchin has assumed a depth of approximately 3.0 metres below existing ground surface (mbgs) to the underside of the footing for the proposed parking garage level.

The proposed development will also include new Site services; however, the proposed asphalt surfaced access roadway will be constructed directly atop the parking garage deck and no asphaltic concrete pavement structure is required.

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 thirteen (13) sampled boreholes (Boreholes BH1 to BH13), 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;
- Site service trench design;
- Lateral earth pressure coefficients and unit densities;
- Foundation design recommendations including soil bearing resistances at Ultimate Limit States (ULS) and Serviceability Limit States (SLS) 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; and
- Potential construction concerns.

Abbreviations, terminology, and principal 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 at the southwest corner of the intersection of Sandalwood Drive and Heron Road, approximately 5 km west of Highway 417 in Ottawa, Ontario. The Site is currently undeveloped and predominantly consists of a grassed field with isolated mature trees. The lands adjacent to the Site are developed with a combination of multi-tenant and single-family residential buildings.

It is noted that the Site was previously developed with various multi-tenant residential townhouse blocks and above grade parking garages. It is Pinchin's understanding that the previous buildings were demolished, and all associated foundations and underground services were removed from the Site.

Data obtained from the Ontario Geological Survey Maps, as published by the Ontario Ministry of Natural Resources, indicates that the Site is located on a fine textured glaciomarine deposit consisting of massive to well laminated silt and clay with minor sand and gravel deposits (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 Georgian Bay, Blue Mountain and Billings Formations consisting of shale, limestone, dolostone, and siltstone (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 between June 29 and July 7, 2021 by advancing a total of thirteen (13) sampled boreholes throughout the Site. The boreholes were advanced to depths ranging from approximately 5.2 to 7.6 mbgs, where refusal was encountered on probable bedrock or very dense glacial till. The approximate spatial locations of the boreholes advanced at the Site are shown on Figure 2.

The boreholes were advanced with the use of a truck mounted CME-55 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.

Monitoring wells were installed in Boreholes BH1, BH2, BH4, BH7, BH9, BH11, and BH12 to allow measurement of groundwater levels. The monitoring wells were constructed using flush-threaded 50 mm diameter Trilock pipe with 3.0 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. Groundwater levels were measured in the monitoring wells on August 13, 2021. The groundwater observations and measurements recorded are included on the appended borehole logs.

The borehole 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 temporary benchmark as shown on Figure 2:

- TBM: Top nut of the fire hydrant located on the south side of Heron Road, at the approximate location shown on Figure 2; and
- Elevation: 100.00 m (local datum).

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 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 granular fill and glacial till to the maximum borehole refusal depth of approximately 7.6 mbgs. The appended borehole logs provide detailed soil descriptions and stratigraphies, results of SPT testing, details of monitoring well installations, and groundwater measurements.

Surficial organics were encountered within all boreholes with the exception of Borehole BH13 and were measured to range in thickness from approximately 100 to 200 mm.

The granular fill material was encountered underlying the surficial organics within Boreholes BH1 to BH4, BH6, and BH9 to BH11, and at the surface within Borehole BH13. The fill material was measured to range in thickness from approximately 0.8 to 2.3 m, and typically consisted of sand and gravel containing trace to some silt. It is noted that trace rootlets and brick pieces were noted within the fill at isolated locations. The non-cohesive granular fill material had a variable very loose to dense relative density based SPT 'N' values of 4 to 33 blows per 300 mm penetration of a split spoon sampler.

Glacial till was encountered at depths ranging between approximately 0.2 to 2.3 mbgs and extended to maximum borehole refusal depth of approximately 7.6 mbgs. The glacial till was noted to range in soil matrix from sand containing some gravel, some silt, and some clay, to gravelly silty sand containing some clay. The non-cohesive glacial till material had a loose to very dense relative density based SPT 'N' values of 6 to greater than 50 blows per 300 mm penetration of a split spoon sampler. The results of four particle size distribution analyses completed on samples of the glacial till indicate that the samples contain 12 to 23% gravel, 42 to 56% sand, 17 to 22% silt, and 8 to 14% clay sized particles. The moisture content of the samples tested ranged from 7.3 to 9.3% indicating a damp to moist condition.

4.2 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. Groundwater was measured within the monitoring wells on August 13, 2021. Groundwater was measured between approximately 2.3 and 3.5 mbgs within the monitoring wells installed. 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 will consist of three (3) six-storey residential apartment buildings which will be complete with a single level underground parking garage which will occupy the majority of the Site footprint. At this time the depth to the underside of the footings for the proposed underground parking level are unknown; as such, for the purpose of this report, Pinchin has assumed a depth of approximately 3.0 mbgs to the underside of the footing for the proposed parking garage level.

5.2 Open Cut Excavations

Excavations for the proposed development are anticipated to extend upwards of 3.0 mbgs in order to accommodate the proposed parking garage level.

Based on the subsurface information obtained from within the boreholes, it is anticipated that the excavated material will predominately consist of organics, granular fill and glacial till materials. Groundwater was measured between approximately 2.3 and 3.5 mbgs on August 13, 2021 and is expected to be encountered during excavations for the proposed development.

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 use of trench boxes can most likely be used for temporary support of vertical side walls. The appropriate trench should be designed/confirmed for use in this soil deposit.

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, and the 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
Fill Material	20	30	0.33	3.0	0.5
Glacial Till	21	32	0.31	3.25	0.47

Based on the OHSA, the natural subgrade soil 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.

In addition to compliance with the OHSA, the excavation procedures must also be in compliance 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 2.3 and 3.5 mbgs on August 13, 2021.

Moderate groundwater inflow through the subgrade soil is expected where the excavations extend less than 0.60 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.

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. If construction commences during wet periods (typically spring or fall), 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.

It is noted that Pinchin is in the process of completing a Hydrogeological Investigation for the proposed development, and these recommendations will be further expanded on within the report.

5.4 Site Servicing

5.4.1 Pipe Bedding and Cover Materials for Flexible and Rigid Pipes

The subgrade soil conditions beneath the Site services will comprise undisturbed glacial till. No support problems are anticipated for flexible or rigid pipes founded on the undisturbed glacial till. Service pipes require an adequate base to ensure proper pipe connection and positive flow is maintained post construction. As such, pipe bedding should be placed to be of uniform thickness and compactness. The pipe bedding and cover material should conform to OPSD 802.010 and 802.013 specifications for flexible pipes and to OPSD 802.031 to 802.033 with Class “B” bedding for rigid pipes.

The pipe bedding material should consist of a minimum thickness of 150 mm Granular “A” (OPSS 1010) below the pipe and extend up the sides to the spring line. However, the bedding thickness may have to be increased depending on the pipe diameter or if wet or weak subgrade conditions are encountered.

The pipe cover material from the spring line should consist of a Granular "B" Type I (OPSS 1010) and should extend to a minimum of 300 mm above the top of the pipe. All granular fill material is to be placed in maximum 200 mm thick loose lifts compacted to a minimum of 98% SPMDD.

The bedding material, pipe and cover material should be installed as soon as practically possible after the excavation subgrade is exposed. The longer the excavated subgrade soil remains open to weather conditions and groundwater seepage, the greater the chance for construction problems to occur.

Where it is difficult to stabilize the subgrade due to groundwater or the material is higher than the optimum moisture content, a Granular "B" Type II material may be required. Alternatively, if constant groundwater infiltration becomes an issue, then an approximate 150 mm granular pad consisting of 19 mm clear stone gravel (OPSS 1004) wrapped in a non-woven geotextile (Terrafix 270R or equivalent) should be considered to maintain the integrity of the natural subgrade soils. The clear stone should contain a minimum of 50% crushed particles. Water collected within the stone should be controlled through sumps and filtered pumps.

5.4.2 Trench Backfill

Above the pipe cover material, the trench can be backfilled by re-using the excavated natural soil matching the materials exposed on the sides of the trenches. The soil should be placed to the underside of the granular subbase of the pavement structure and be compacted in maximum 300 mm thick lifts to 98% SPMDD within 4% of the optimum moisture content. This is recommended to provide soil compatibility and help minimize potential abrupt differential frost heave between surrounding natural materials similar in composition. The natural material must be free of organics or other deleterious material.

All stockpiled material should be protected from deleterious materials, additional moisture and be kept from freezing.

Quality control will be the utmost importance when selecting the material. The selection of the material should be done as early in the contract as possible to allow sufficient time for gradation and proctor testing on representative samples to ensure it meets the projects specifications.

Where the natural soil will be exposed, adequate compaction may prove difficult if the material becomes wet (i.e., above the optimum moisture content). Depending on the moisture content of the natural materials at the time of construction, they may either require moisture to be added or stockpiled and left to dry to achieve moisture content within plus 2% to minus 4% of optimum. The natural soil at this site is subject to moisture content increase during wet weather. As such, stockpiles should be protected to help minimize moisture absorption during wet weather.

Alternatively, an imported drier material of similar gradation as the soil (i.e., sand and gravel) may be mixed to decrease the overall moisture content and bring it to within plus 2% to minus 4% of optimum. Depending on weather conditions at the time of construction, an imported material may be required regardless to achieve adequate compaction. If the imported material is not the same/similar to the soil observed on the side walls of the excavation, then a horizontal transition between the materials should be sloped as per frost heave taper OPSD 205.60. Any natural material is to be placed in maximum 300 mm thick lifts compacted to 95% SPMDD within plus 2% to minus 4% optimum moisture content. Imported material should consist of a Granular "A", Granular "B" Type I, or Select Subgrade Material (OPSS 1010). Heavy construction equipment and truck traffic should not cross any pipe until at least 1 m of compacted soil is placed above the top of the pipe.

Post compaction settlement of finer grained soil can be expected, even when placed to compaction specifications. As such, fill materials should be installed as far in advance as possible before finishing the roadway in order to mitigate post compaction settlements.

5.4.3 Frost Protection

The frost penetration depth in Ottawa, Ontario is estimated to extend to approximately 2.1 mbgs in open roadways cleared of snow. As such, it is recommended to place water services at a minimum depth of 300 mm below this elevation with the top of the pipe located at 2.4 mbgs or lower as dictated by municipal service requirements. If a minimum of 2.4 m of soil cover cannot be provided, then the pipe should be insulated with a rigid polystyrene insulation (DOW Styrofoam HI40, or equivalent) or a pre-insulated pipe be utilized.

The insulation design configuration may either consist of placing horizontal insulation to a specified design distance beyond the outside edge of the pipe or an inverted "U" surrounding the top and sides of the pipe. Any method chosen requires suitable design and installation in accordance with the manufacture's recommendations. To accommodate the placement of horizontal insulation a wider excavation trench may be required.

5.5 Foundation Design

5.5.1 Shallow Foundations Bearing on Compact to Very Dense Glacial Till

Conventional shallow strip footings established on the compact to dense glacial till material encountered in the boreholes at approximately 3.0 mbgs, may be designed using a bearing resistance for 25 mm of settlement at Serviceability Limit States (SLS) of 200 kPa, and a factored geotechnical bearing resistance of 325 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 weaker pockets of subgrade soil were encountered within a portion of the boreholes; as such, there is a potential for additional areas of weaker subgrade soil to be encountered between the investigation locations. Pinchin presumes that any areas of weaker subgrade soil will consist of small pockets of soft/loose natural soil which can be compacted to match the density of the remainder of the Site. As such, the glacial till material must be compacted to a minimum of 100% Standard Proctor Maximum Dry Density (SPMDD) prior to installing the concrete formwork. Any soft/loose areas which are not able to achieve the recommended 100% SPMDD are to be removed and replaced with an OPSS 1010 Gran B Type II, or approved equivalent.

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, or caved materials;
- 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.

5.5.2 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 7.6 mbgs, where refusal was encountered on probable bedrock or very dense glacial till. SPT “N” values within the subgrade soil deposits ranged between 4 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 D. A Site Class D has an average shear wave velocity (Vs) of between 180 and 360 m/s.

It is noted that the Client has requested that a shear wave velocity sounding be completed at the Site, and these results will be provided in the Final Geotechnical Investigation Report.

5.5.3 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 engineered fill). 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 is to have a maximum slope of 2 H to 1 V, 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.

5.5.4 Estimated Settlement

All individual spread footings should be founded on uniform subgrade soils, 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.

5.5.5 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.

Exterior perimeter foundations drains are not required, where the finished floor elevation is established a minimum of 150 mm above the exterior final grades or that the exterior gradient is properly sloped to divert surface water away from the building.

5.5.6 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.

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. If required, Pinchin can provide appropriate foundation frost protection recommendations as part of the design review.

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.

5.6 Underground Parking Garage Design

As previously mentioned, the proposed development will include a single level underground parking garage which will occupy the majority of the Site footprint. Groundwater was encountered between approximately 2.3 and 3.5 mbgs within the monitoring wells installed.

As such, depending on the proposed final grades, there is a potential for the parking garage to have to be designed to either resist hydrostatic uplift or to be provided with underfloor and foundation wall drainage systems connected to a suitable frost-free outlet due to the groundwater levels at the Site. Once final design of the building is complete Pinchin should confirm this recommendation. Additional boreholes and monitoring wells may be required.

The magnitude of the hydrostatic uplift may be calculated using the following formula:

$$P = \gamma \times d$$

Where:

P = hydrostatic uplift pressure acting on the base of the structure (kPa)

γ = unit weight of water (9.8 kN/m^3)

d = depth of base of structure below the design high water level (m)

The resistance of gross uplift of the structure can be increased by simply increasing the mass of the structure, incorporating oversize footings into the structure or by installing soil anchors.

Alternatively, exterior perimeter foundation drains can be installed where subsurface walls are exposed to the interior. The foundation drains 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 water collected from the weeping tile should be directed away from the building to appropriate drainage areas; either through gravity flow or interior sump pump systems. All subsurface walls should be waterproofed.

Pinchin also recommends an underfloor drainage system be installed beneath the slab, in addition to the installation of perimeter weeping tiles at the footing level. The floor slab sub drains should be constructed in a similar fashion to the foundation drains and be connected to a suitable frost-free outlet or sump.

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. It is noted that Pinchin is in the process of completing a Hydrogeological Investigation for the proposed development, in order to determine the quantity and quality of water to be removed during and after construction.

The walls must also be designed to resist lateral earth pressure. Depending on the design of the building the earth pressure computations must consider 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.

5.7 Floor Slabs

Prior to the installation of the engineered fill material, all organics and deleterious materials should be removed to the underlying glacial till. The natural subgrade soil is to be proof roll compacted with a minimum 10 tonne non-vibratory steel drum roller to observe for weak/soft spots. It is noted that some locations will not be accessible by the steel drum roller; as such, these locations can be proof roll compacted with a minimum 450 kg vibratory plate compactor.

The in-situ glacial till material encountered within the boreholes is considered adequate for the support of the concrete floor slabs provided it is proof roll compacted as outlined above. Any soft area(s) encountered during proof rolling should be excavated and replaced with a similar soil type.

Once the subgrade soil is exposed it is to be inspected and approved by a qualified geotechnical engineering consultant to ensure that the material conforms to the soil type and consistency observed during the subsurface investigation work.

Based on the in-situ soil conditions, it is recommended to establish the concrete floor slab on a minimum 300 mm thick layer of OPSS 1010 Granular "A". Alternatively, consideration may also be given to using a 200 mm thick layer of uniformly compacted 19 mm clear stone placed over the approved subgrade. Any required up-fill should consist of an OPSS 1010 Granular "B" Type I or Type II.

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.

The following table provides the unfactored modulus of subgrade reaction values:

Material Type	Modulus of Subgrade Reaction (kN/m^3)
Granular A (OPSS 1010)	85,000
Granular "B" Type I (OPSS 1010)	75,000
Granular "B" Type II (OPSS 1010)	85,000
Glacial Till	65,000

6.0 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 undisturbed natural subgrade material prior to subgrade preparation, 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.

7.0 TERMS AND LIMITATIONS

This Geotechnical Investigation was performed for the exclusive use of Hazelview Developments Inc. (Client) in order to evaluate the subsurface conditions at Heron Gate 5, Ottawa, 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.

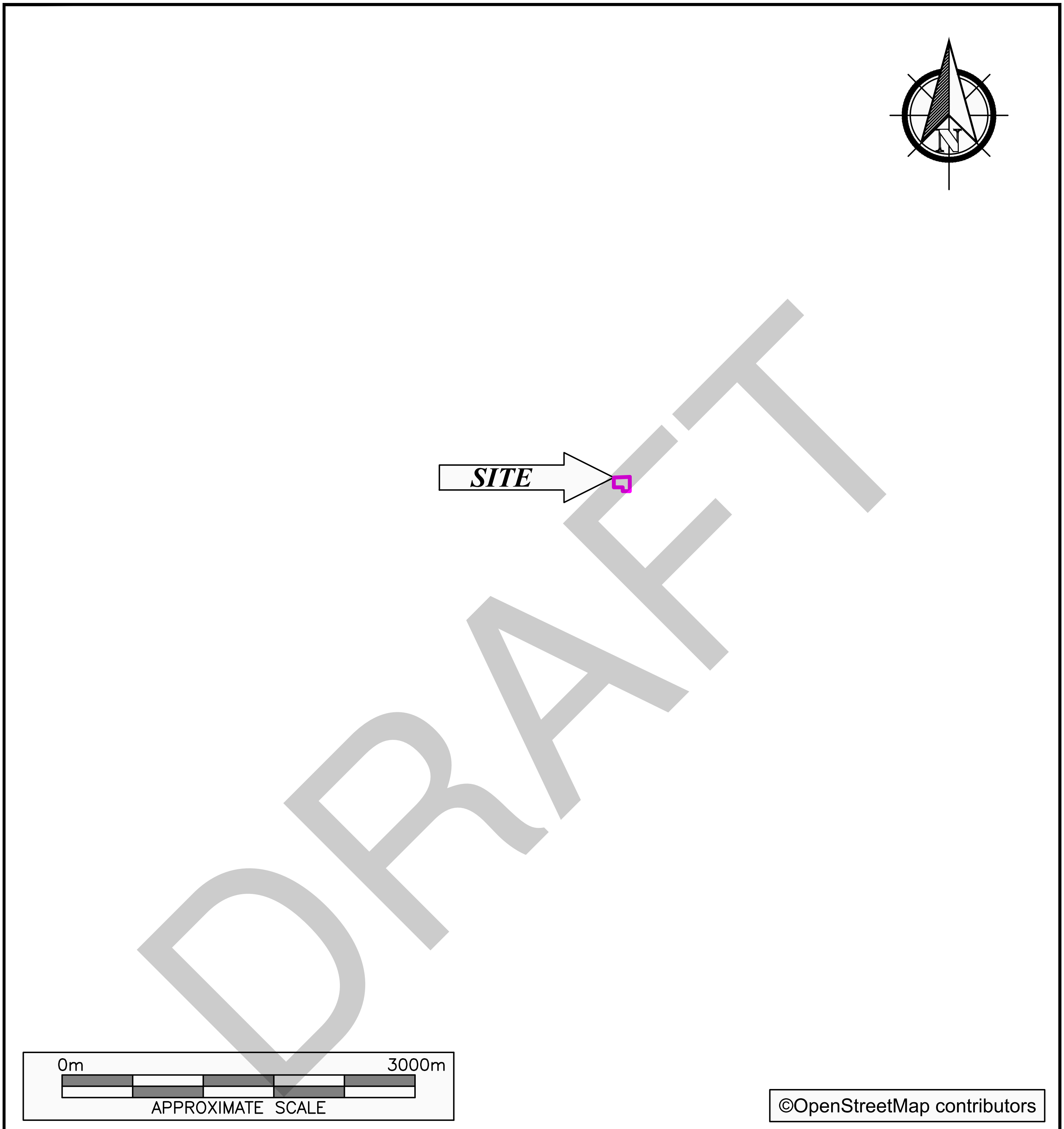
Specific limitations related to the legal and financial and limitations to the scope of the current work are outlined in our proposal, the attached Methodology and the Authorization to Proceed, Limitation of Liability and Terms of Engagement which accompanied the proposal.

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.

288344.001 Geotechnical Investigation Heron Gate 5 Ottawa ON

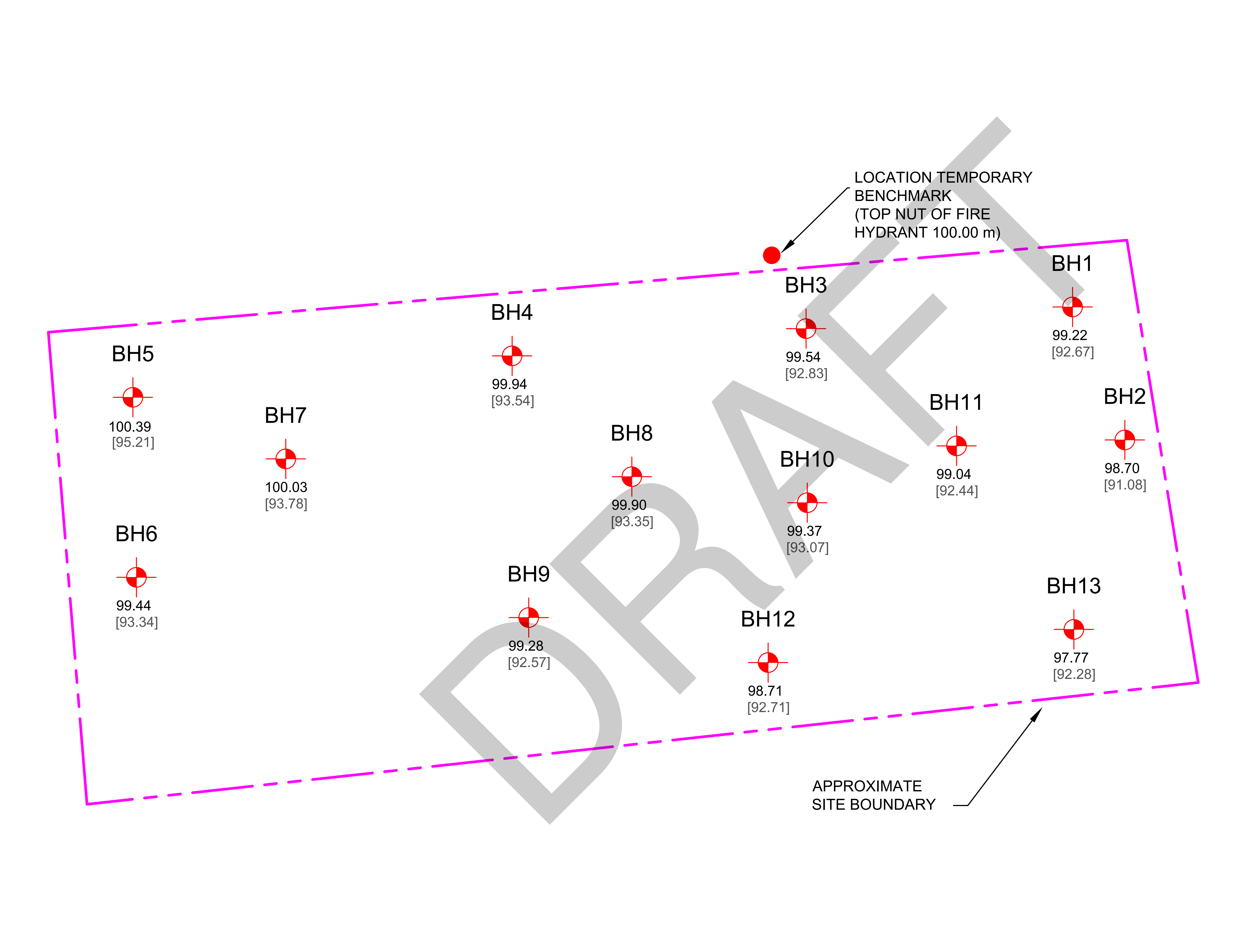
Template: Master Geotechnical Investigation Report – Ontario, GEO, September 2, 2021

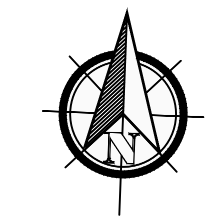
FIGURES



PINCHIN

PROJECT NAME	GEOTECHNICAL INVESTIGATION
CLIENT NAME	HAZELVIEW DEVELOPMENTS INC.
PROJECT LOCATION	HERON GATE 5, OTTAWA, ONTARIO
FIGURE NAME	KEY MAP
FIGURE NO.	1
APPROXIMATE SCALE	AS SHOWN
PROJECT NO.	288344.001
DATE	OCTOBER 2021





LEGEND

BOREHOLE / MONITORING WELL LOCATION

XX.XX APPROXIMATE LOCAL GROUND ELEVATION (m)

[XX.XX] APPROXIMATE LOCAL REFUSAL ELEVATION (m)

PINCHIN

PROJECT NAME

GEOTECHNICAL INVESTIGATION

CLIENT NAME

HAZELVIEW DEVELOPMENTS INC.

PROJECT LOCATION

HERON GATE 5, OTTAWA, ONTARIO

FIGURE NAME

BOREHOLE/MONITORING WELL LOCATION PLAN

APPROXIMATE SCALE	PROJECT NO.
AS SHOWN	288344.001
DATE	FIGURE NO.
OCTOBER 2021	2

APPENDIX I

Abbreviations, Terminology and Principal 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
σ'_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

Log of Borehole: BH1

Project #: 288344.001 **Logged By:** WT

Project: Geotechnical Investigation

Client: Hazelview Developments Inc.

Location: Heron Gate 5, Ottawa, Ontario

Drill Date: June 30, 2021 **Project Manager:** WT

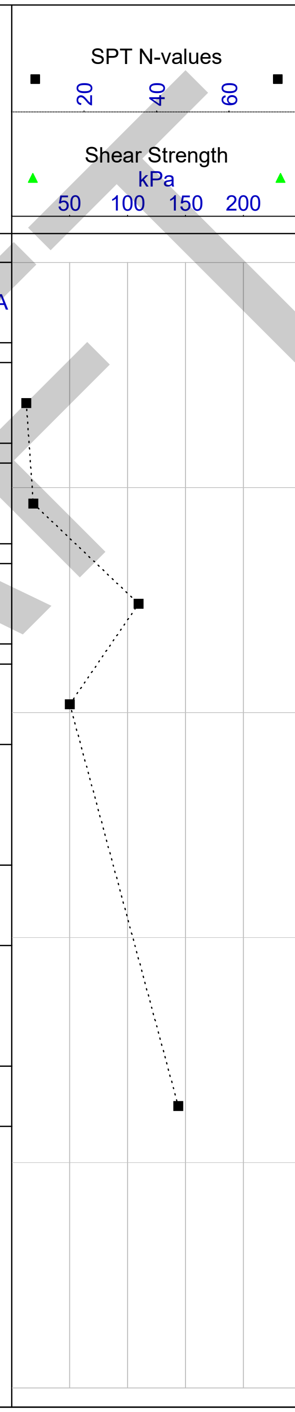
SUBSURFACE PROFILE

Depth (m)	Description	Elevation (m)
0	Ground Surface	99.22
0 – 0.2	***Organics*** ~ 200 mm	
	Fill Brown sand and gravel, trace to some silt, loose, damp	98.46
	Very loose	97.70
	Trace rootlets, loose	96.93
	Glacial Till Grey silty sand, some gravel, trace to some clay, dense, damp	96.17
	Compact	94.65
	Moist	93.12
	Dense	92.67
6.6	End of Borehole Borehole terminated at 6.6 mbgs due to auger refusal on probable bedrock or very dense glacial till.	

Monitoring Well Details: Groundwater level = 3.55 mbgs, as measured on August 13, 2021.

SAMPLE

Sample Type	Sampler #	Recovery (%)	SPT N-values	Lab Analysis	Moisture (%)	Plasticity Index
AS	1	100	N/A			
SS	2	10	4			
SS	3	10	6			
SS	4	80	35			
SS	5	100	16	Hyd.	7.8	
SS	6	100	21			
SS	7	100	46			



Contractor: Strata Drilling Group **Grade Elevation:** 99.22 m

Drilling Method: Hollow Stem / Split Spoon **Top of Casing Elevation:** 100.00 m

Well Casing Size: 5.08 cm **Sheet:** 1 of 1

Log of Borehole: BH2

Project #: 288344.001 **Logged By:** WT

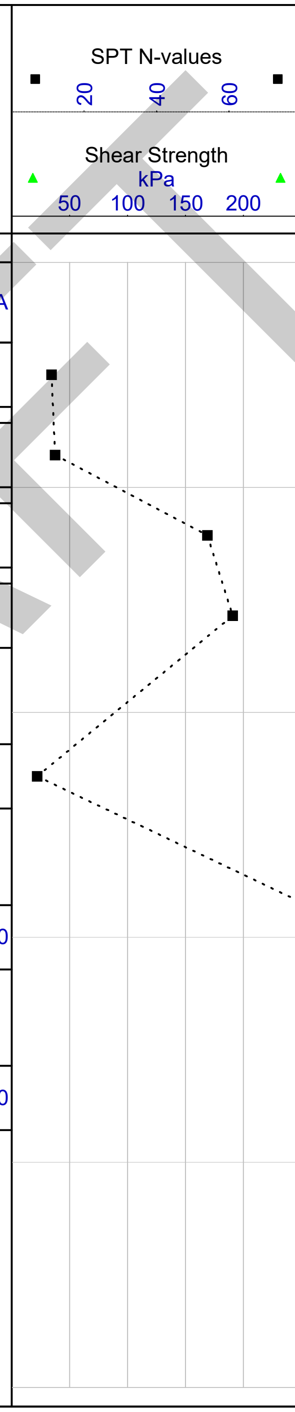
Project: Geotechnical Investigation

Client: Hazelview Developments Inc.

Location: Heron Gate 5, Ottawa, Ontario

Drill Date: June 30, 2021 **Project Manager:** WT

SUBSURFACE PROFILE				SAMPLE							
Depth (m)	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
0	Ground Surface	98.70									
	Organics ~ 150 mm	97.94		AS	1	100	N/A				
1	***Fill*** Brown sand and gravel, trace to some silt, loose, damp	97.18		SS	2	80	11				
2	Brown silty sand, some gravel, compact, damp	96.41		SS	3	60	12				
	Glacial Till Brown silty sand, some gravel, trace clay, compact, damp			SS	4	10	54				
3	Very dense, moist	94.13		SS	5	5	61				
5	Grey, loose, wet	92.60		SS	6	30	7				
6	Grey, very dense, wet	91.08		SS	7	80	>50				
8	End of Borehole Borehole terminated at 7.6 mbgs due to auger refusal on probable bedrock or very dense glacial till.		Groundwater level = 3.50 mbgs, as measured on August 13, 2021.	SS	8	N/A	>50				



Contractor: Strata Drilling Group

Grade Elevation: 98.70 m

Drilling Method: Hollow Stem / Split Spoon

Top of Casing Elevation: 99.60 m

Well Casing Size: 5.08 cm

Sheet: 1 of 1

Log of Borehole: BH3

Project #: 288344.001 **Logged By:** WT

Project: Geotechnical Investigation

Client: Hazelview Developments Inc.

Location: Heron Gate 5, Ottawa, Ontario

Drill Date: June 30, 2021 **Project Manager:** WT

SUBSURFACE PROFILE

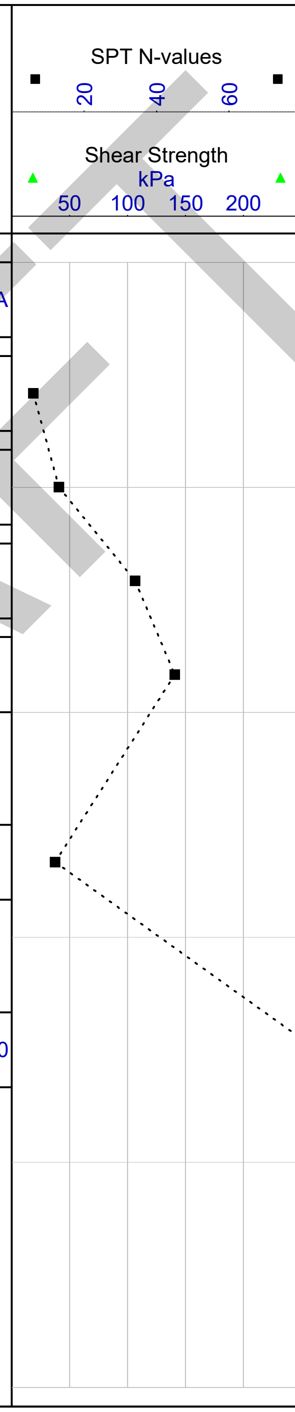
Depth (m)	Description	Elevation (m)
0	Ground Surface	99.54
0–0.6	**Organics** ~ 150 mm.	
0.6–1.0	**Fill** Brown sand and gravel, trace to some silt, loose, damp	98.78
1.5	**Glacial Till** Brown, silty sand, some gravel, trace to some clay, loose, damp	98.02
	Compact	
2.3	Dense	97.25
4.6	Grey, compact, wet	94.97
6.1	Grey, very dense, wet	93.44
6.7	End of Borehole	92.83

Borehole terminated at 6.7 mbgs due to auger refusal on probable bedrock or very dense glacial till.

SAMPLE

Sample Type	Sampler #	Recovery (%)	SPT N-values
AS	1	100	N/A
SS	2	30	6
SS	3	40	13
SS	4	100	34
SS	5	100	45
SS	6	50	12
SS	7	100	>50

Monitoring Well Details | Lab Analysis | Moisture (%) | Plasticity Index



Contractor: Strata Drilling Group **Grade Elevation:** N/A

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

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

Log of Borehole: BH4

Project #: 288344.001 **Logged By:** WT

Project: Geotechnical Investigation

Client: Hazelview Developments Inc.

Location: Heron Gate 5, Ottawa, Ontario

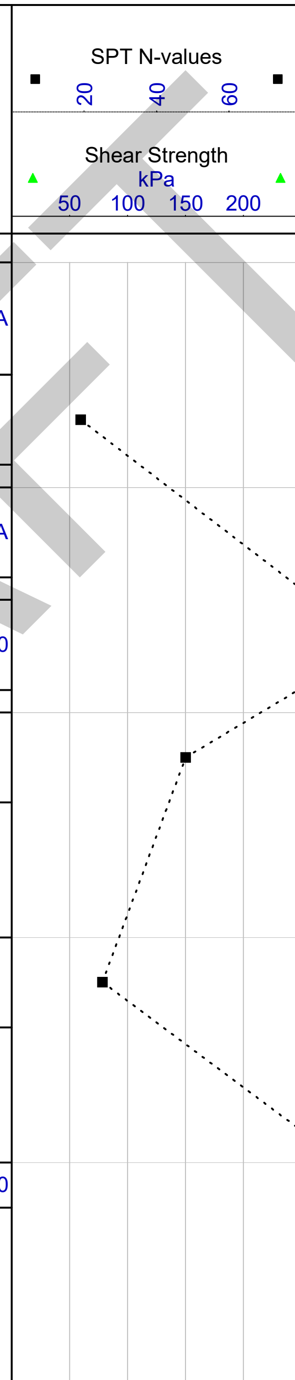
Drill Date: June 30, 2021 **Project Manager:** WT

SUBSURFACE PROFILE

Depth (m)	Description	Elevation (m)	Monitoring Well Details
0	Ground Surface	99.94	
	Organics ~ 150 mm.	99.79	
	Fill Brown sand and gravel, trace silt, trace brick pieces, loose, damp	99.18	
1	***Glacial Till*** Grey silty sand, some gravel, trace clay, compact, damp	98.42	
2	Some clay, very loose	97.65	
	Grey silty sand and gravel, trace clay, very dense, damp	96.89	
3	Dense	95.37	
4–5	Compact, wet	93.84	
6	Very dense, wet.	93.54	
7	End of Borehole Borehole terminated at 6.4 mbgs due to auger refusal on probable bedrock or very dense glacial till.		Groundwater level = 3.45 mbgs, as measured on August 13, 2021.

SAMPLE

Sample Type	Sampler #	Recovery (%)	SPT N-values	Lab Analysis	Moisture (%)	Plasticity Index
AS	1	100	N/A			
SS	2	25	19			
SS	3	100	N/A			
SS	4	100	>50			
SS	5	50	48			
SS	6	65	25			
SS	7	-	>50			



Contractor: Strata Drilling Group **Grade Elevation:** 99.94 m

Drilling Method: Hollow Stem Auger / Split Spoon **Top of Casing Elevation:** 100.91 m

Well Casing Size: 5.08 cm **Sheet:** 1 of 1

Log of Borehole: BH5

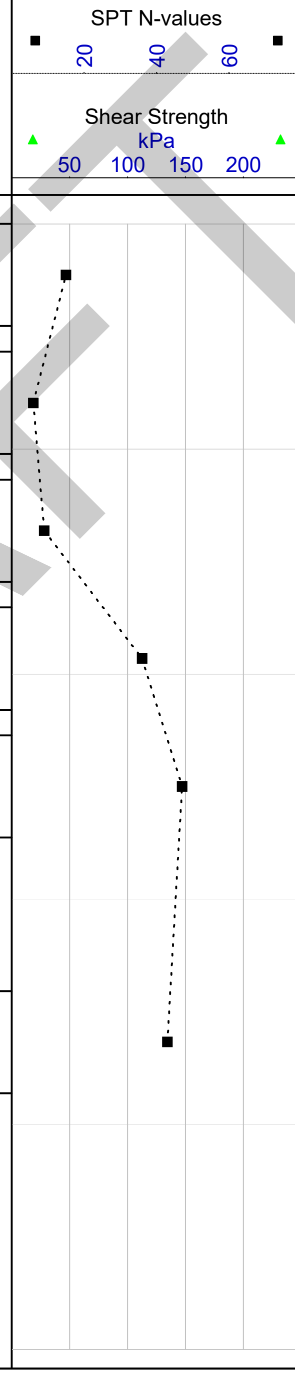
Project #: 288344.001
Logged By: WT
Project: Geotechnical Investigation
Client: Hazelview Developments Inc.
Location: Heron Gate 5, Ottawa, Ontario
Drill Date: June 30, 2021
Project Manager: WT

SUBSURFACE PROFILE

Depth (m)	Symbol	Description	Elevation (m)	Monitoring Well Details
0		Ground Surface	100.39	
		Organics ~ 200 mm	100.19	
		Glacial Till Brown silty sand and gravel, trace clay, compact, damp	99.63	
1		Loose, moist		
2		Dense	98.10	
5			95.21	
		End of Borehole Borehole terminated at 5.2 mbgs due to auger refusal on probable bedrock or very dense glacial till.		

SAMPLE

Sample Type	Sampler #	Recovery (%)	SPT N-values	Lab Analysis	Moisture (%)	Plasticity Index
AS	1	100	15			
SS	2	75	6			
SS	3	30	9			
SS	4	100	36			
SS	5	30	47			
SS	6	45	43			



Contractor: Strata Drilling Group
Grade Elevation: 100.39 m
Drilling Method: Hollow Stem / Split Spoon
Top of Casing Elevation: N/A
Well Casing Size: N/A
Sheet: 1 of 1

Log of Borehole: BH6

Project #: 288344.001 **Logged By:** WT

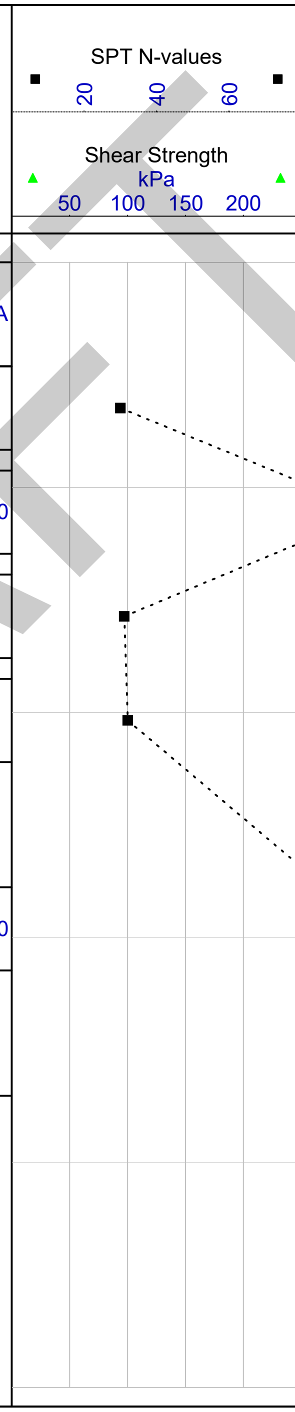
Project: Geotechnical Investigation

Client: Hazelview Developments Inc.

Location: Heron Gate 5, Ottawa, Ontario

Drill Date: June 30, 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 / Shear Strength kPa	Lab Analysis	Moisture (%)	Plasticity Index
		Ground Surface	99.44									
0		***Organics*** ~ 200 mm.	99.24		AS	1	100	N/A				
		Fill Brown sand and gravel, trace silt, dense, damp	98.68									
1		***Glacial Till*** Brown sand, some gravel, some silt, trace to some clay, dense, damp	97.92		SS	2	70	30				
		Very dense	97.15		SS	3	30	>50				
2		Grey, dense, moist			SS	4	100	31				
3					SS	5	80	32				
4												
		Very dense	94.87		SS	6	100	>50		Hyd.	7.3	
5												
6		End of Borehole Borehole terminated at 6.1 mbgs due to auger refusal on probable bedrock or very dense glacial till.	93.34									
7												
8												



Contractor: Strata Drilling Group **Grade Elevation:** 99.44 m

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

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

Log of Borehole: BH7

Project #: 288344.001 **Logged By:** WT

Project: Geotechnical Investigation

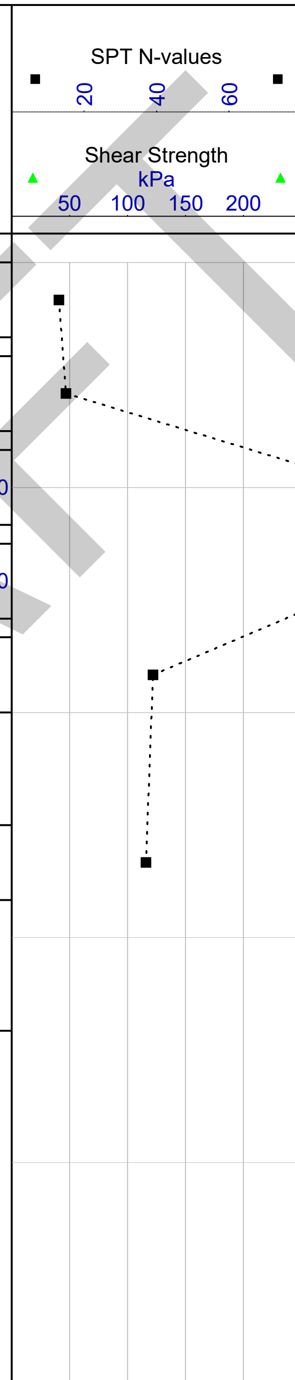
Client: Hazelview Developments Inc.

Location: Heron Gate 5, Ottawa, Ontario

Drill Date: June 30, 2021 **Project Manager:** WT

SUBSURFACE PROFILE / SAMPLE

Depth (m)	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-values
0	Ground Surface	100.03					
0–0.2	**Organics** ~ 200 mm	99.83		SS	1	50	13
0.2–1.5	**Glacial Till** Brown silty sand and gravel, trace clay, compact, damp	98.51		SS	2	80	15
1.5–3.05	Very dense, moist	96.98		SS	3	100	>50
				SS	4	100	>50
3.05–6.25	Dense, wet	93.78		SS	5	80	39
				SS	6	100	37
6.25	End of Borehole Borehole terminated at 6.3 mbgs due to auger refusal on probable bedrock or very dense glacial till.		Groundwater level = 3.0 mbgs, as measured on August 13, 2021.				



Lab Analysis | Moisture (%) | Plasticity Index

Contractor: Strata Drilling Group

Drilling Method: Hollow Stem / Split Spoon

Well Casing Size: 5.08 cm

Grade Elevation: 100.03 m

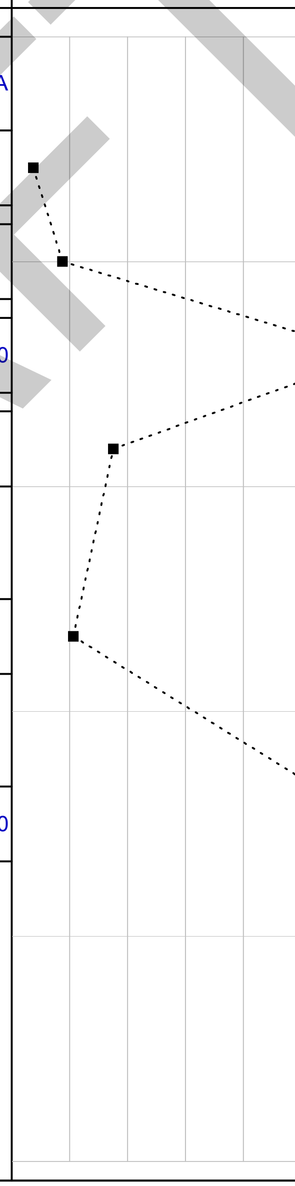
Top of Casing Elevation: 101.13 m

Sheet: 1 of 1

Log of Borehole: BH8

Project #: 288344.001
Logged By: WT
Project: Geotechnical Investigation
Client: Hazelview Developments Inc.
Location: Heron Gate 5, Ottawa, Ontario
Drill Date: June 30, 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
0		Ground Surface	99.90									
		Organics ~ 200 mm	99.70		AS	1	100	N/A				
1		***Glacial Till*** Brown silty sand and gravel, trace clay, loose, damp	98.38		SS	2	40	6				
2		Compact, moist	97.61		SS	3	60	14				
3		Very dense	96.85		SS	4	100	>50				
4		Grey, compact, moist			SS	5	90	28				
5					SS	6	80	17				
6			93.80									
		Grey, very dense, wet	93.35		SS	7	100	>50				
7		End of Borehole. Borehole terminated at 6.6 mbgs due to auger refusal on probable bedrock or very dense glacial till.										
8												
9												



Contractor: Strata Drilling Group
Grade Elevation: 99.90 m
Drilling Method: Hollow Stem / Split Spoon
Top of Casing Elevation: N/A
Well Casing Size: N/A
Sheet: 1 of 1

Log of Borehole: BH9

Project #: 288344.001
Logged By: WT
Project: Geotecchnical Investigation
Client: Hazelview Developments Inc.
Location: Heron Gate 5, Ottawa, Ontario
Drill Date: June 30, 2021
Project Manager: WT

SUBSURFACE PROFILE

Depth (m)	Description	Elevation (m)	Monitoring Well Details
0	Ground Surface	99.28	
0 – 0.1	***Organics*** ~ 100 mm		
	Fill Brown sand and gravel, trace silt, dense, damp	98.21	
	Glacial Till Brown silty sand, some gravel, trace clay, compact, damp	97.76	
	Very dense	96.99	
	Grey, dense	96.23	
	Grey, compact	94.71	
	Grey, compact, wet	93.18	
	Gravelly sand, some silt, trace clay, very dense, wet	92.57	
6.7	End of Borehole. Borehole terminated at 6.7 mbgs due to auger refusal on probable bedrock or very dense glacial till.		Groundwater level = 2.31 mbgs, as measured on August 13, 2021.

SAMPLE

Sample Type	Sampler #	Recovery (%)	SPT N-values	Lab Analysis	Moisture (%)	Plasticity Index
SS	1	40	33			
SS	2	80	15			
SS	3	70	53			
SS	4	80	36			
SS	5	60	19			
SS	6	30	20			
SS	7	30	>50	Hyd.	9.3	

SPT N-values: 20, 40, 60
Shear Strength kPa: 50, 100, 150, 200

Contractor: Strata Drilling Group
Grade Elevation: 99.28 m
Drilling Method: Hollow Stem / Split Spoon
Top of Casing Elevation: 100.08 m
Well Casing Size: 5.08 cm
Sheet: 1 of 1

Log of Borehole: BH10

Project #: 288344.001 **Logged By:** WT

Project: Geotechnical Investigation

Client: Hazelview Developments Inc.

Location: Heron Gate 5, Ottawa, Ontario

Drill Date: June 30, 2021 **Project Manager:** WT

SUBSURFACE PROFILE			SAMPLE							
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	99.37									
Organics ~ 150 mm			SS	1	60	29				
Fill Brown sand and gravel, trace silt, compact, damp.	98.61		SS	2	50	12				
Glacial Till Brown silty sand, some gravel, trace clay, compact, damp			SS	3	80	23				
Dense	97.08		SS	4	80	49				
Grey, dense	96.32		SS	5	30	34				
Grey, compact, wet	94.80		SS	6	40	18				
End of Borehole Borehole terminated at 6.3 mbgs due to auger refusal on probable bedrock or very dense glacial till.	93.07									

Contractor: Strata Drilling Group **Grade Elevation:** 99.37 m

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

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

Log of Borehole: BH11

PINCHIN

Project #: 288344.001
Logged By: WT
Project: Geotechnical Investigation
Client: Hazelview Developments Inc.
Location: Heron Gate 5, Ottawa, Ontario
Drill Date: June 30, 2021
Project Manager: WT

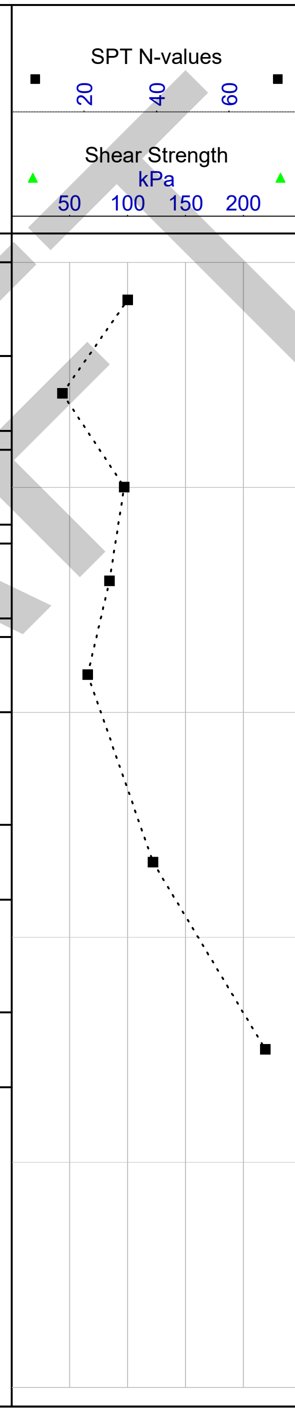
SUBSURFACE PROFILE

Depth (m)	Description	Elevation (m)
0	Ground Surface	99.04
0–0.2	***Organics*** ~ 200 mm	98.84
0.2–0.76	***Fill*** Brown sand and gravel, trace silt, dense, damp	98.28
0.76–1.52	***Glacial Till*** Brown gravelly silty sand, trace to some clay, compact, damp	97.52
1.52–2.29	Dense	96.75
2.29–3.05	Compact	95.99
3.05–4.57	Grey, compact	94.47
4.57–6.10	Grey, compact, moist	92.94
6.10–6.7	Grey, very dense	92.44
6.7	End of Borehole Borehole terminated at 6.7 mbgs due to auger refusal on probable bedrock or very dense glacial till.	

Monitoring Well Details: Groundwater level = 3.05 mbgs, as measured on August 13, 2021.

SAMPLE

Sample Type	Sampler #	Recovery (%)	SPT N-values	Lab Analysis	Moisture (%)	Plasticity Index
SS	1	40	32			
SS	2	70	14			
SS	3	70	31			
SS	4	60	27			
SS	5	80	21			
SS	6	60	39			
SS	7	60	70			



Contractor: Strata Drilling Group
Grade Elevation: 99.04 m
Drilling Method: Hollow Stem / Split Spoon
Top of Casing Elevation: 99.89 m
Well Casing Size: N/A
Sheet: 1 of 1

Log of Borehole: BH12

Project #: 288344.001 **Logged By:** WT

Project: Geotechnical Investigation

Client: Hazelview Developments Inc.

Location: Heron Gate 5, Ottawa, Ontario

Drill Date: June 30, 2021 **Project Manager:** WT

Depth (m)	Description (SUBSURFACE PROFILE)	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-values
0	Ground Surface	98.71					
0–0.15	**Organics** ~ 150 mm			SS	1	30	19
0.15–1.52	**Glacial Till** Brown gravelly silty sand, trace to some clay, compact, damp	97.19		SS	2	50	16
1.52–2.29	Very dense	96.42		SS	3	100	50
2.29–3.05	Compact	95.66		SS	4	80	24
3.05–4.57	Grey, dense	94.14	Groundwater level (2.85 mbgs)	SS	5	60	32
4.57–6.0	Grey, compact, wet	92.71		SS	6	45	35
6.1	End of Borehole: Borehole terminated at 6.1 mbgs due to auger refusal on probable bedrock or very dense glacial till.		Groundwater level = 2.85 mbgs, as measured on August 13, 2021.				

SPT N-values: 20, 40, 60
Shear Strength kPa: 50, 100, 150, 200

Contractor: Strata Drilling Group **Grade Elevation:** *98.71 m*

Drilling Method: Hollow Stem / Split Spoon **Top of Casing Elevation:** 99.61 m

Well Casing Size: 5.08 cm **Sheet:** 1 of 1

Log of Borehole: BH13

Project #: 288344.001 **Logged By:** WT

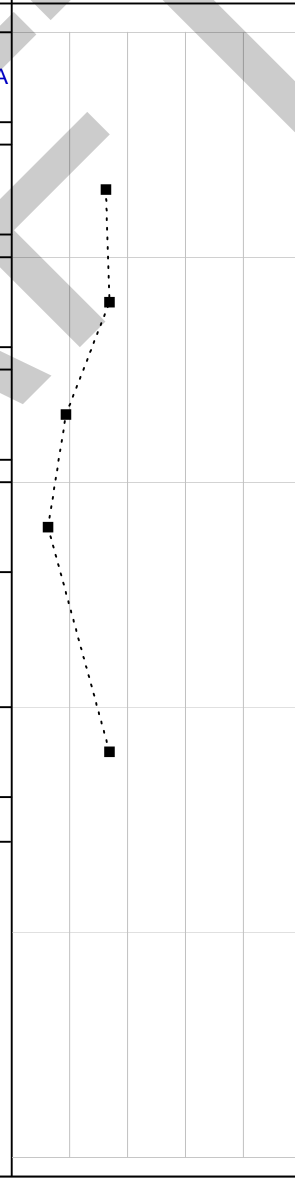
Project: Geotechnical Investigation

Client: Hazelview Developments Inc.

Location: Heron Gate 5, Ottawa, Ontario

Drill Date: June 30, 2021 **Project Manager:** WT

SUBSURFACE PROFILE				SAMPLE							
Depth (m)	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
0	Ground Surface	97.77									
0–0.76	**Fill** Brown sand and gravel, trace silt, loose, damp.	97.01		AS	1	100	N/A				
0.76–2.29	**Glacial Till** Grey gravelly sitly sand, trace to some clay, compact, damp	95.48		SS	2	40	26				
				SS	3	80	27				
2.29–5.49	Moist	92.28		SS	4	70	15		Hyd.	8.3	
				SS	5	60	10				
				SS	6	60	27				
	End of Borehole Borehole terminated at 5.5 mbgs due to auger refusal on probable bedrock or very dense glacial till.										



Contractor: Strata Drilling Group **Grade Elevation:** 97.77 m

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

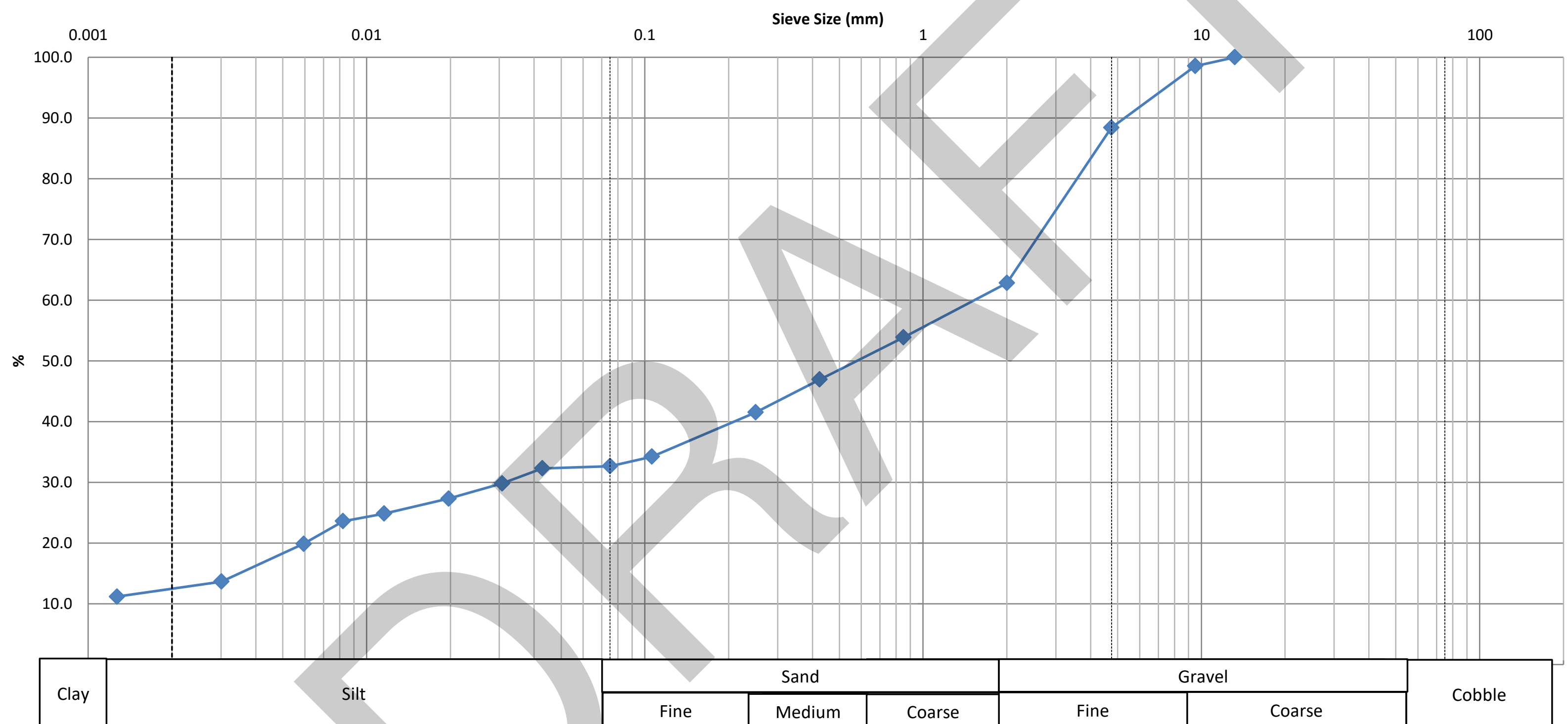
Well Casing Size: N/A **Sheet:** 1 of 1

APPENDIX III

Laboratory Testing Reports for Soil Samples

SIEVE ANALYSIS
ASTM C136

CLIENT:	Pinchin	DEPTH:	10 - 12'	FILE NO:	PM4184
CONTRACT NO.:		BH OR TP No.:	BH1	LAB NO:	26372
PROJECT:	288344.001			DATE RECEIVED:	15-Jul-21
				DATE TESTED:	19-Jul-21
DATE SAMPLED:	-			DATE REPORTED:	22-Jul-21
SAMPLED BY:	Client			TESTED BY:	DB



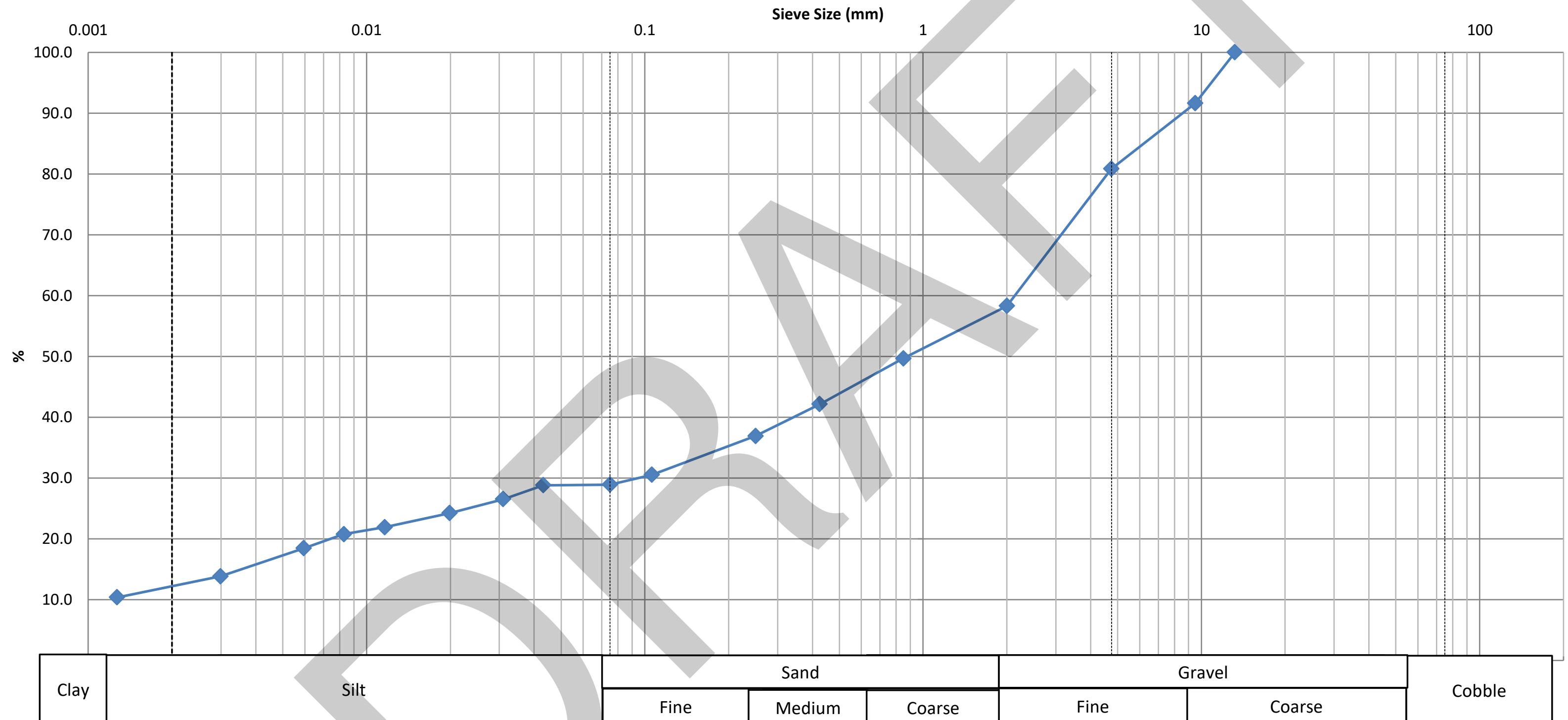
Identification	Soil Classification					MC(%)	LL	PL	PI	Cc	Cu
						7.8					
	D100	D60	D30	D10	Gravel (%)		Sand (%)		Silt (%)		Clay (%)
					11.6		55.7		20.3		12.4

Comments:

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

SIEVE ANALYSIS
ASTM C136

CLIENT:	Pinchin	DEPTH:	15- 17'	FILE NO:	PM4184
CONTRACT NO.:		BH OR TP No.:	BH6	LAB NO:	26371
PROJECT:	288344.001			DATE RECEIVED:	15-Jul-21
				DATE TESTED:	19-Jul-21
DATE SAMPLED:	-			DATE REPORTED:	22-Jul-21
SAMPLED BY:	Client			TESTED BY:	DB



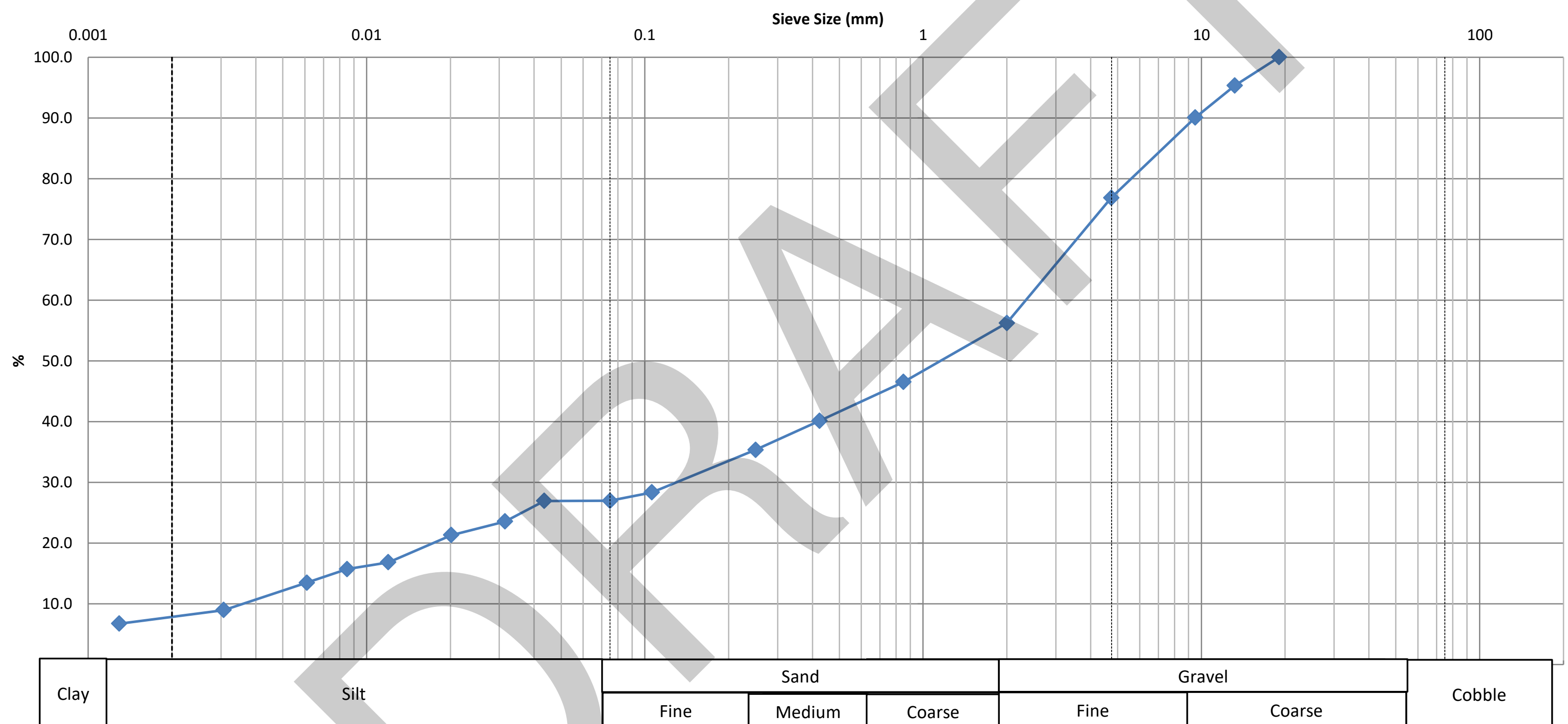
Identification	Soil Classification				MC(%)	LL	PL	PI	Cc	Cu
					7.3					
	D100	D60	D30	D10	Gravel (%)	Sand (%)		Silt (%)		Clay (%)
					19.2	51.9		16.9		12.0

Comments:

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

	SIEVE ANALYSIS **ASTM C136**

CLIENT:	Pinchin	DEPTH:	20 - 22'	FILE NO:	PM4184
CONTRACT NO.:		BH OR TP No.:	BH9	LAB NO:	26370
PROJECT:	288344.001			DATE RECEIVED:	15-Jul-21
				DATE TESTED:	19-Jul-21
DATE SAMPLED:	-			DATE REPORTED:	22-Jul-21
SAMPLED BY:	Client			TESTED BY:	DB



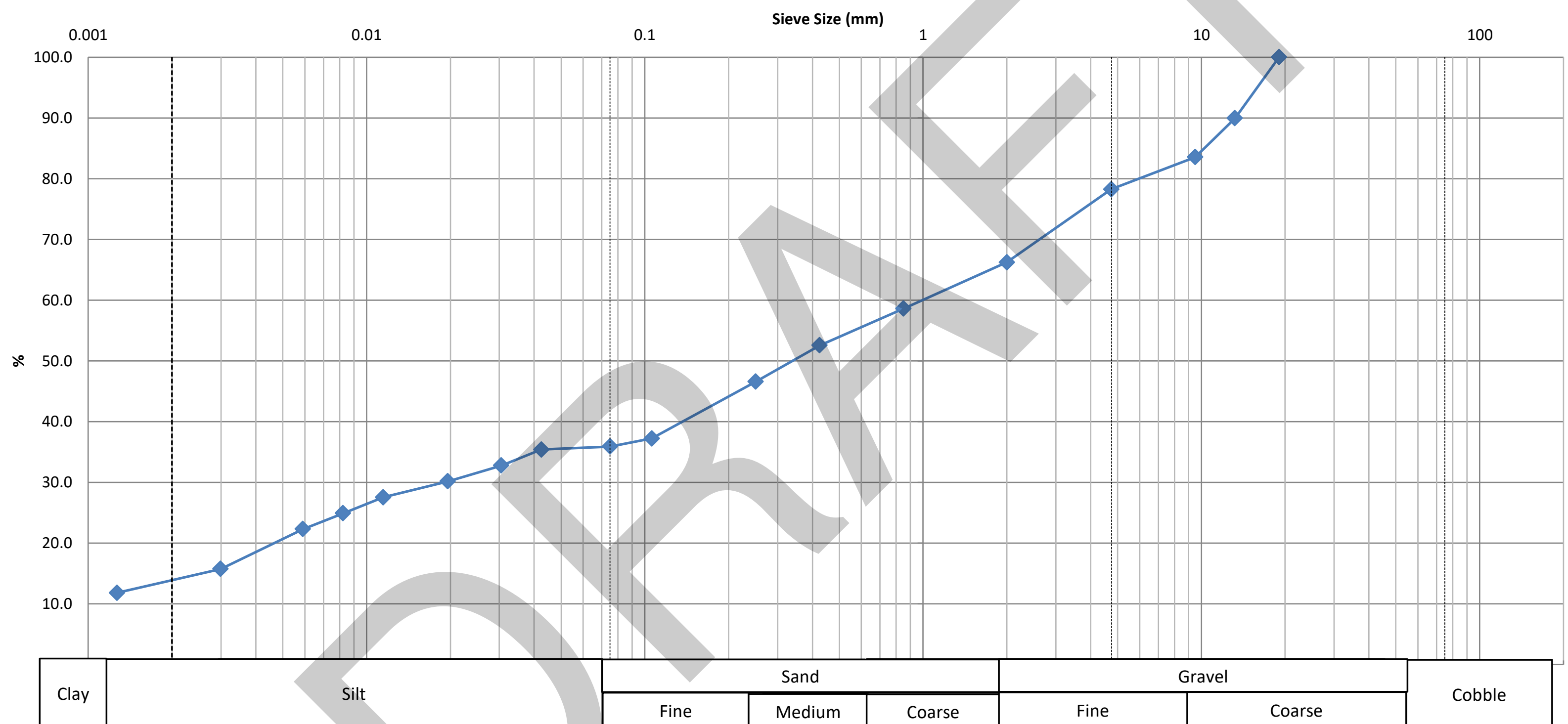
Identification	Soil Classification					MC(%)	LL	PL	PI	Cc	Cu
						9.3					
	D100	D60	D30	D10	Gravel (%)	Sand (%)		Silt (%)		Clay (%)	
					23.2	49.8		19.2		7.8	

Comments:

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

	SIEVE ANALYSIS **ASTM C136**

CLIENT:	Pinchin	DEPTH:	7.5 - 9.5 '	FILE NO:	PM4184
CONTRACT NO.:		BH OR TP No.:	BH13	LAB NO:	26369
PROJECT:	288344.001			DATE RECEIVED:	15-Jul-21
				DATE TESTED:	19-Jul-21
DATE SAMPLED:	-			DATE REPORTED:	22-Jul-21
SAMPLED BY:	Client			TESTED BY:	DB



Identification	Soil Classification				MC(%)	LL	PL	PI	Cc	Cu
					8.3					
	D100	D60	D30	D10	Gravel (%)	Sand (%)		Silt (%)	Clay (%)	
					21.7	42.4		22.4	13.5	

Comments:

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

APPENDIX IV

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.