FINAL

Geotechnical Investigation – Proposed Site Redevelopment

1600 James Naismith Drive, Ottawa, Ontario

Prepared for:

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1.0 INTRODUCTION AND SCOPE

Pinchin Ltd. (Pinchin) was retained by 1600 James Naismith LP (Client) to conduct a Geotechnical Investigation and provide subsequent geotechnical design recommendations for the proposed site redevelopment to be located at 1600 James Naismith Drive, 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 Site is currently developed with an eight-storey commercial office building which is proposed to be converted to a residential building. The proposed redevelopment will reportedly not include any new buried concrete structures; however, it will include the installation of new Site services, and new asphalt surfaced parking areas and access roadways. In addition, a portion of the new residential units within the basement level will be complete with walkout patios. This will require removing a portion of the existing soil cover that is being used for frost protection of the building footings. Therefore, the Client has also requested that Pinchin include geotechnical recommendations for the proposed walkout patio areas.

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 five (5) sampled boreholes (Boreholes BH1 to BH5) throughout the Site. The information gathered from the Geotechnical Investigation will allow Pinchin to provide geotechnical design recommendations for the proposed redevelopment.

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;
- Asphaltic concrete pavement structure design for parking areas and access roadways;
- Walkout patio design 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 on the north end of Telesat Court, approximately 75 meters south of Highway 174 in Ottawa, Ontario. The Site is currently developed with an eight-storey commercial office building, a single storey power plant building, and asphalt surfaced access roadways and parking areas. The lands adjacent to the Site are developed with a combination of single-family residential dwellings, multi-storey commercial buildings and a park with sports fields and facilities.

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 on May 10 and 11, 2022 by advancing a total of five sampled boreholes (Boreholes BH1 to BH5) throughout the Site. The boreholes were advanced to sampled depths ranging from approximately 2.6 to 3.5 metres below existing ground surface (mbgs) where refusal was encountered on probable bedrock. 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 in the boreholes were collected at 0.76 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.

Groundwater observations and measurements were obtained from the open boreholes during and upon completion of drilling. 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 of front nut on fire hydrant, 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 moisture content and 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 consists of either surficial asphalt or surficial organics overlying granular fill, glacial till, and probable bedrock to the maximum borehole refusal depth of approximately 3.5 mbgs. The appended borehole logs provide detailed soil descriptions and stratigraphies, results of SPT testing, and groundwater measurements.

The surficial organics were encountered in Boreholes BH2 and BH5 and were measured to be approximately 300 mm thick.

The surficial asphalt was encountered in Boreholes BH1, BH3, and BH4 and was measured to be between approximately 100 and 150 mm thick.

Granular fill was encountered underlying either the surficial asphalt or surficial organics in all boreholes with the exception of Borehole BH2. The granular fill was measured to range in thickness from approximately 0.7 to 0.9 m, and typically consisted of sand and gravel containing trace to some silt that was brown and damp at the time of sampling. The granular fill had a loose to compact relative density based on SPT ‘N’ values of between 6 and 23 blows per 300 mm penetration of a split spoon sampler.

The results of two particle size distribution analyses performed on samples of the fill indicate that the samples contain 43 to 57% gravel, 36 to 45% sand, and 7 to 12% silt sized particles.

Glacial till was encountered underlying the surficial organics within Borehole BH2 and underlying the granular fill within the remaining boreholes. The glacial till was noted to range in thickness from approximately 1.8 to 2.4 m and extended down to the underlying probable bedrock surface in all boreholes. The glacial till generally comprised silty gravelly sand containing trace clay and had a loose to very dense relative density based on SPT ‘N’ values of between 8 and 81 blows per 300 mm penetration of a split spoon sampler. The result of one particle size distribution analysis performed on a sample of the glacial till indicates that the sample contains 27% gravel, 40% sand, 24% silt and 9% clay sized particles. The moisture content of the sample tested was 6.6%, indicating that the sample was damp at the time of sampling.

4.2 Bedrock

SPT refusal on probable bedrock was encountered in all boreholes at depths ranging between approximately 2.6 and 3.5 mbgs. It is noted that bedrock coring was not completed to confirm the presence of bedrock at the Site.

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. Groundwater was not encountered within the boreholes at the time of the Site Investigation. 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 service installations 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 Site is currently developed with an eight-storey commercial office building which is proposed to be converted to a residential building. The proposed redevelopment will reportedly not include any new buried concrete structures; however, it will include the installation of new Site services, and new asphalt surfaced parking areas and access roadways. In addition, a portion of the new residential units within the basement level will be complete with walkout patios. This will require removing a portion of the existing soil cover that is being used for frost protection of the building footings. Therefore, the Client has also requested that Pinchin include geotechnical recommendations for the proposed walkout patio areas.

5.2 Site Preparation

The existing organics are not considered suitable to remain below the proposed access roads and parking areas and will need to be removed. In calculating the approximate quantity of organics to be stripped, we recommend that the organic thicknesses provided on the individual borehole logs be increased by 50 mm to account for variations and some stripping of the mineral soil below. In addition, the existing surficial asphalt and concrete surfaces will presumably be removed from the Site to allow for regrading of the new asphalt surfaced areas.

It is Pinchin's understanding that the existing loading dock is proposed to be raised with a combination of engineered fill and a new cast-in-place concrete slab. Prior to raising the loading dock area, Pinchin recommends removing the existing concrete slab to ensure there are no voids between the slab and the underlying fill material.

Pinchin recommends that any engineered fill required at the Site be compacted in accordance with the criteria stated in the following table:

Type of Engineered Fill	Maximum Loose Lift Thickness (mm)	Compaction Requirements	Moisture Content (Percent of Optimum)
Subgrade fill beneath parking lots, access roadways, and within existing loading dock	300	98% SPMDD	Plus 2 to minus 4

Prior to placing any fill material at the Site, the subgrade should be inspected by a qualified geotechnical engineer and loosened/soft pockets should be sub excavated and replaced with engineered fill.

It is recommended that any engineered fill required at the Site comprise imported Ontario Provincial Standard Specification (OPSS) 1010 Granular 'B' Type I or II material. The existing granular fill material encountered within the boreholes is also considered suitable to be used as engineered fill at the Site. In addition, the granular fill may remain in place as a subgrade material underlying the proposed pavement structure, subject to geotechnical review prior to installing the pavement structure subbase material. If the work is carried out during very dry weather, water may have to be added to the material to improve compaction.

A qualified geotechnical engineering technician should be on site to observe fill placement operations and perform field density tests at random locations throughout each lift, to indicate the specified compaction is being achieved.

Any trees proposed to be planted at the Site are to be planted a minimum of 3 m from any foundation wall.

5.3 Open Cut Excavations

It is anticipated that excavations for site service trenches will extend upwards of 2.5 to 3.0 mbgs. As such, there is a potential for bedrock removal to be required to accommodate the installation of new Site services.

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 not encountered within the boreholes advanced and is not expected to be encountered during excavations for the proposed Site redevelopment.

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.

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.

Based on Pinchin's local experience in the area, the upper approximate 1.5 to 2.0 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. Often a hydraulic hammer 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.

Although not likely, drilling and blasting of the bedrock 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.

In addition, we recommend that a pre-blast survey of all neighbouring properties be undertaken prior to conducting drilling and blasting activities. The preconstruction survey will serve to protect the Client from claims unrelated to the construction activities in the development of this property.

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.

5.4 Anticipated Groundwater Management

Groundwater was not encountered in the boreholes advanced and is not expected to be encountered during excavations for the Site redevelopment. 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 by 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. Excavations for new Site services are not expected to require a Permit to Take Water or a submission to the Environmental Activity and Sector Registry (EASR). It is the responsibility of the contractor to make this application if required.

5.5 Site Service Trench Design

5.5.1 Pipe Bedding and Cover Materials for Flexible and Rigid Pipes

The subgrade conditions beneath the Site services will comprise either glacial till or bedrock. No support problems are anticipated for flexible or rigid pipes founded in the glacial till or bedrock. 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.

For pipes installed within bedrock trenches, the following is recommended:

- Install 300 mm of 19 mm clear stone gravel (OPSS 1004) or Granular “A” (OPSS 1010) below the pipe extending up the sides to the spring line;
- If clear stone is used as bedding material, then a non-woven geotextile (Terrafix 360R or equivalent) is to be placed over the clear stone and pipe extending up vertically along the side walls of the bedrock and pipe a minimum distance of 500 mm;
- The pipe cover material should consist of either a Granular “B” Type I (OPSS 1010) with a maximum particle diameter size of 26.5 mm or bedding sand and should extend to a minimum of 300 mm above the top of the pipe; and
- If rock shatter is present a non-woven geotextile (Terrafix 360R or equivalent) may be required to prevent the migration of fines from the bedding material into the rock shatter. Where blasting is required for site services, over blast of at least 600 mm of rock shatter should be performed. Over blast material may stay in the trench.

For pipes installed within the natural subgrade soil, 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

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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) with a maximum particle diameter size of 26.5 mm 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% Standard Proctor Maximum Dry Density (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 until the trench backfill material is placed.

5.5.2 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. Where the adjacent material consists of bedrock, the trench can be backfilled with well graded blast rock fill, with a gradation similar to OPSS 1010 Granular 'B' Type I. 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., silty sand or gravelly sand) 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.

5.5.3 Frost Protection

The frost penetration depth in Ottawa, Ontario for these types of soil conditions is estimated to extend to approximately 1.8 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 2.1 mbgs or lower as dictated by municipal service requirements. If a minimum of 2.1 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.

5.6 Asphaltic Concrete Pavement Structure Design for Parking Lot and Driveways

5.6.1 Discussion

As previously mentioned, the proposed Site redevelopment includes new asphalt surfaced parking areas and access roadways which will be constructed at the same/similar elevation as the current grades at the Site. The glacial till 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.

Provided the pavement structure overlies the glacial till material, the following pavement structure is recommended. It is noted that the existing granular fill encountered within the boreholes may remain in place as a subgrade material underlying the proposed pavement structure, subject to geotechnical review prior to installing the pavement structure subbase material. There is also the potential for the existing granular fill to remain in place as a subbase material; however, this would require further review during construction to confirm its composition and thickness across the Site.

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	35 mm	35 mm
Binder Course Asphaltic Concrete HL-8 (OPSS 1150)	92 % MRD as per OPSS 310	55 mm	85 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	300 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.

Portions of the glacial till may 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. Pavement subdrains should comprise 150 mm diameter perforated pipe in filter sock, bedded in concrete sand, and connected to the catch basins. The upper limit of the concrete sand bedding should be at the lower limit of the pavement subbase, with the subgrade at the bottom of the subbase sloped towards the subdrain.

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.

5.7 Walkout Patio Design Recommendations

5.7.1 Foundation Insulation Recommendations

The conversion of the existing building from commercial to residential will include the addition of residential units in the basement level which will be complete with walkout patios. In order to construct the walkout patios a portion of the existing foundation backfill material will need to be removed down to approximately 100 to 200 mm below the top of the existing basement level floor slab. At this time, Pinchin is unaware of the distance between the top of the existing floor slab and the underside of the existing footing. As such, for the purpose of this report, Pinchin has assumed that a maximum of 0.5 m of soil cover for frost protection will be remaining for the footings once walkout patios have been constructed. It is noted that the remaining soil cover thickness is to be confirmed at the Site during construction to ensure the following recommendations are adequate and do not need to be adjusted.

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. As such, the addition of walkout patios will result in areas of the building where the minimum soil cover for frost protection is not provided. In these areas the foundations should be protected from frost with a combination of soil cover and rigid polystyrene insulation, such as Dow Styrofoam or equivalent product.

To provide a combination of soil cover and rigid polystyrene insulation for frost protection, Pinchin recommends the following based on an assumed 0.5 m of soil cover provided for frost protection:

- The insulation should have a minimum thermal resistance value of R-20. If Dow Styrofoam is used this would require a total of 100 mm of insulation;
- The insulation should buttress the foundation wall and extend out a minimum horizontal distance of 1.8 m beyond the outside face of the footing;
- The insulation should be installed vertically on the perimeter of the foundation wall, from approximately 100 mm below the proposed finished grade to the top of the horizontal insulation;
- A minimum of 600 mm of soil cover is to be provided above the insulation at all times. If 600 mm of soil cover cannot be provided for the insulation, a concrete topping (i.e., 10 MPa lean mix) is to be installed over the insulation to maintain its integrity. If the walkout

patios are proposed to consist of cast-in-place concrete, the concrete topping is not required;

- The insulation placed beyond the foundation is to have a positive slope away from the building foundation; and
- All backfill material above the insulation is to consist of a Granular "A" or Granular "B" Type I (OPSS 1010) with a maximum aggregate diameter of 26.5 mm, compacted to 100% SPMDD.

These insulation recommendations assume the interior of the building is maintained at 18 degrees Celsius or higher.

5.7.2 Walkout Patio Retaining Wall Design

5.7.2.1 Discussion

The construction of the walkout patio areas will also require the installation of retaining walls to support the remainder of the existing foundation backfill that will not be excavated. The following drawings were provided by the Client to allow Pinchin to provide the retaining wall recommendations:

- Drawing entitled "Retaining Wall Plan", prepared by D + M Structural Engineering, drawing number S-100, issued for municipal approval September 28, 2022, project number 22-110; and
- Drawing entitled "Sections", prepared by D + M Structural Engineering, drawing number S-200, issued for municipal approval September 28, 2022, project number 22-110.

Based on a review of the above referenced drawings, two types of retaining wall systems are proposed for the Site. Cast-in-place concrete retaining walls (approximately 3.0 m in height) complete with cast-in-place concrete footings will be utilized for the end walls of the walkout patios, while tiered armour stone retaining wall systems complete with granular bases will be utilized on either side of the stairs which will access the walkout patios.

The tiered walls will consist of three tiers, each approximately the height of one armour stone block (~ 500 to 600 mm high). Each tier includes a landscaped section at the top that is a minimum of 1.0 m long and is to be sloped at 3H:1V or shallower up to the next tier or armour stone. In addition, the overall slope of the tiered retaining wall systems is shown as 3H:1V or shallower. Provided the individual tier slopes and overall tiered system slope are maintained at 3H:1V or shallower, the tiered system would be considered stable.

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It is noted that Pinchin did not advance boreholes adjacent to the proposed walkout patio areas; as such, the following design recommendations have been based on the information obtained from the boreholes advanced for the proposed new Site services and asphalt surfaced parking areas.

Pinchin recommends that the subsurface soil conditions at the proposed retaining wall locations be reviewed during the construction of the retaining walls to confirm the below provided recommendations.

5.7.2.2 Retaining Wall Bearing on Engineered Fill Overlying Glacial Till

The glacial till material encountered within the boreholes is considered suitable to support the proposed retaining walls, provided it is proof roll compacted with a minimum 10 tonne non-vibratory steel drum roller to observe for weak/soft spots. It is noted that it may not be possible to access all areas with a steel drum roller; as such, these locations can be proof roll compacted with a minimum 450 kg vibratory plate compactor. Any soft area(s) encountered during proof rolling should be excavated and replaced with a similar soil type.

Pinchin notes that all walls will require to be established on engineered fill. The engineered fill is to consist of a minimum 150 mm thick layer of OPSS 1010 Granular ‘A’, compacted to 100% standard Proctor maximum dry density (SPMDD), in order to provide a uniform bearing surface for the retaining wall, as well as to protect the natural subgrade soil during construction.

For retaining walls established on a minimum 150 mm thick layer of properly compacted OPSS 1010 Granular ‘A’ overlying the glacial till, a bearing resistance for 25 mm of settlement at Serviceability Limit States (SLS) of 100 kPa, and a factored geotechnical bearing resistance of 150 kPa at Ultimate Limit States (ULS) may be used for design purposes.

Pinchin notes that a qualified geotechnical engineering consultant should be on-Site during the proof roll 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.

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

- Prior to commencing excavation, 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 promote surface drainage and any collected water is to be pumped out of the excavation. Any potential precipitation or seepage entering the excavation 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, and caved materials; 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.

5.7.2.3 Estimated Settlement

The retaining wall should be founded on a uniform layer of OPSS 1010 Granular ‘A’, compacted to 100% SPMDD and approved by a licensed geotechnical engineering consultant.

Provided the retaining wall is installed in accordance with the recommendations outlined in the preceding sections, settlements are not expected to exceed 25 mm (total settlement) and 19 mm (differential settlement).

5.7.2.4 Retaining Wall Drainage and Backfill

To assist in maximizing the service life of the retaining wall, it is recommended that grades at the bottom of the wall be sloped away at a 2% gradient or more, for a distance of at least 2.0 m. In addition, it is recommended to install a drain at the base of the retaining wall backfill material in order to allow for any potential collected water to drain from behind the wall.

The retaining wall drain should consist of a minimum 150 mm diameter fabric wrapped perforated drainage tile surrounded by OPSS 1010 19 mm diameter clear stone 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 water collected from the weeping tile should be directed through gravity flow to the end of the retaining wall furthest from the existing building.

Pinchin recommends that the retaining wall be backfilled with imported OPSS 1010 Granular 'B' Type I material, extending a minimum lateral distance of 600 mm from the edge of the retaining wall. It is recommended that inspection and testing be carried out during construction to confirm backfill quality, thickness and to ensure compaction requirements are achieved.

5.7.2.5 Lateral Earth Pressure Coefficients and Unit Densities

The retaining wall must also be designed to resist lateral earth pressure. For calculating the lateral earth pressure, the following unfactored strength properties for the in-situ glacial till and imported engineered fill are provided:

Material Type	**Effective Friction Angle ø'**	**Unit Weight ϒ kN/m^3**	**Effective Unit Weight ϒ' kN/m^3**	**Coefficient of At Rest Earth Pressure (k_o)**	**Coefficient of Active Earth Pressure (k_a)**	**Coefficient of Passive Earth Pressure (k_p)**
Glacial Till	34°	21.5	21.5	0.44	0.28	3.54
Granular "B" Type I (OPSS 1010)	32º	21.0	21.0	0.47	0.31	3.25

Pinchin also recommends that an appropriate factor of safety be applied during the design calculations.

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 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 1600 James Naismith LP (Client) in order to evaluate the subsurface conditions at 1600 James Naismith Drive, 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.

The liability of Pinchin or our officers, directors, shareholders or staff will be limited to the lesser of the fees paid or actual damages incurred by the Client. Pinchin will not be responsible for any consequential or indirect damages. Pinchin will only be 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 (Claim Period), to commence legal proceedings against Pinchin to recover such losses or damage unless the laws of the jurisdiction which governs the Claim Period which is applicable to such claim provides that the applicable Claim Period is greater than two years and cannot be abridged by the contract between the Client and Pinchin, in which case the

Claim Period shall be deemed to be extended by the shortest additional period which results in this provision being legally enforceable.

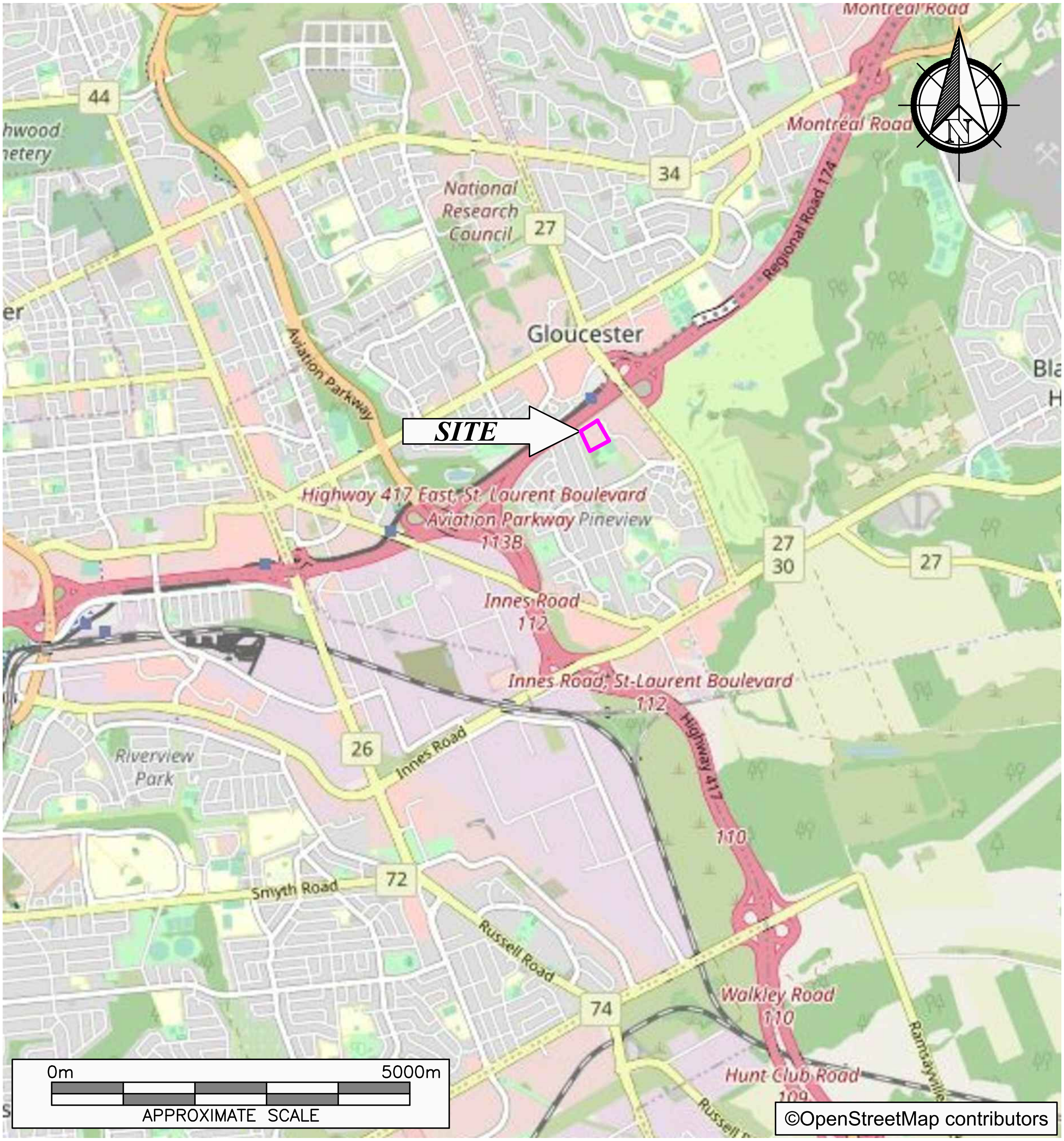
Pinchin makes no other representations whatsoever, including those concerning the legal significance of its findings, or as to other legal matters touched on in this report, including, but not limited to, ownership of any property, or the application of any law to the facts set forth herein. With respect 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.

306147 Geo Invest 1600 James Naismith Ottawa ON Moreton
Template: Master Geotechnical Investigation Report – Ontario, GEO, September 2, 2021

FIGURES



PROJECT NAME
GEOTECHNICAL INVESTIGATION

CLIENT NAME
1600 JAMES NAISMITH LP

PROJECT LOCATION
1600 JAMES NAISMITH DRIVE, OTTAWA, ONTARIO

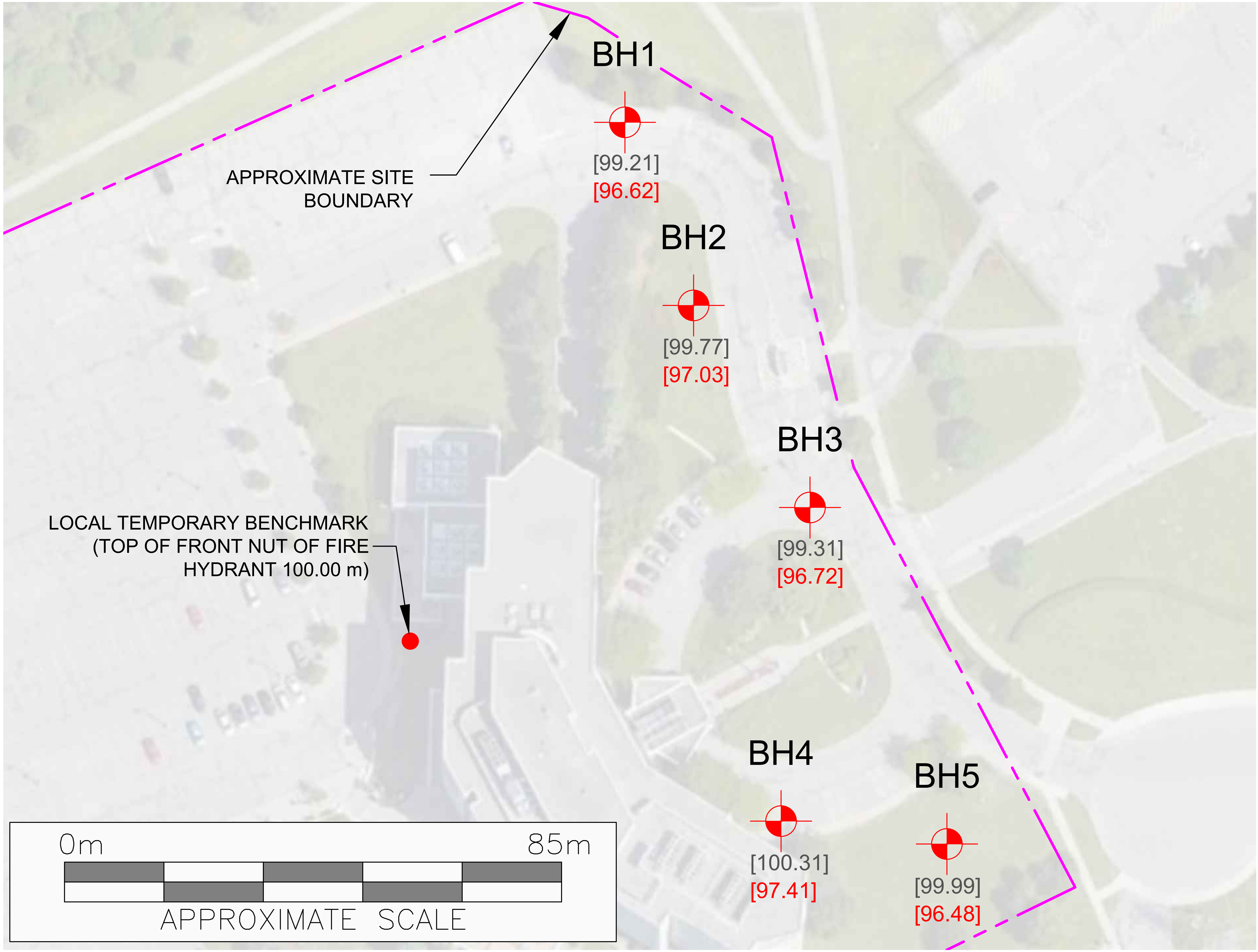
FIGURE NAME
KEY MAP

FIGURE NO.
1

APPROXIMATE SCALE
AS SHOWN

PROJECT NO.
306147

DATE
SEPTEMBER 2022



N

LEGEND

⌖	BOREHOLE LOCATION
[XX.XX]	APPROXIMATE GROUND ELEVATION (m)
[XX.XX]	APPROXIMATE REFUSAL ELEVATION (m)

PINCHIN

PROJECT NAME
GEOTECHNICAL INVESTIGATION

CLIENT NAME
1600 JAMES NAISMITH LP

PROJECT LOCATION
1600 JAMES NAISMITH DRIVE,
OTTAWA, ONTARIO

FIGURE NAME
BOREHOLE LOCATION PLAN

APPROXIMATE SCALE
AS SHOWN

PROJECT NO.
306147

DATE
SEPTEMBER 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
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:

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

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 #: 306147 **Logged By:** MK

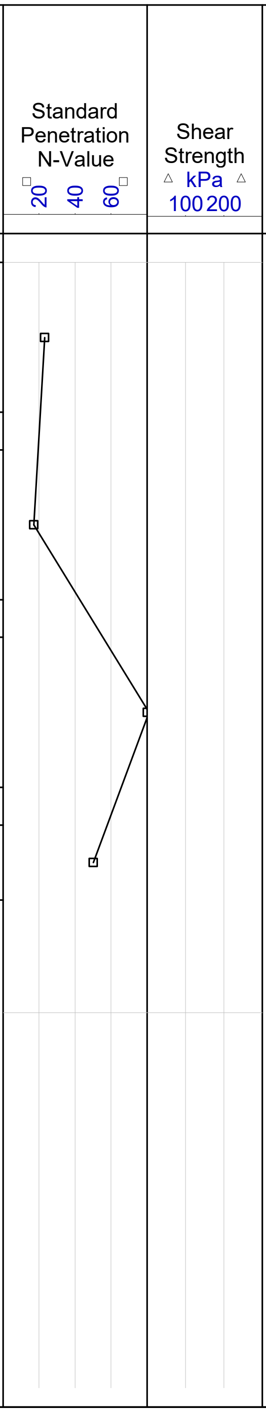
Project: Geotechnical Investigation

Client: 1600 James Naismith LP

Location: 1600 James Naismith Drive, Ottawa, ON

Drill Date: May 10, 2022 **Project Manager:** WT

SUBSURFACE PROFILE					SAMPLE							
Depth (m)	Symbol	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-Value	Water Content (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
		Ground Surface	99.21									
0		**Asphalt** ~ 150 mm		No Monitoring Well Installed								
		Fill Sand and gravel, some silt, brown, damp, compact	98.45		SS	1	60	23				GS
1		**Glacial Till** Silty gravelly sand, trace clay, brown, damp, compact	97.69		SS	2	80	17				
2		Trace to some shale bedrock fragments, very dense	96.62		SS	3	100	81				
					SS	4	100	50				
		End of Borehole										
3		Borehole was terminated at 2.59 mbgs due to SPT refusal on probable bedrock. No groundwater was encountered.										
4												



Contractor: Canadian Environmental Drilling & Contractors Inc. **Grade Elevation:** 99.21 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: BH2

Project #: 306147 **Logged By:** MK

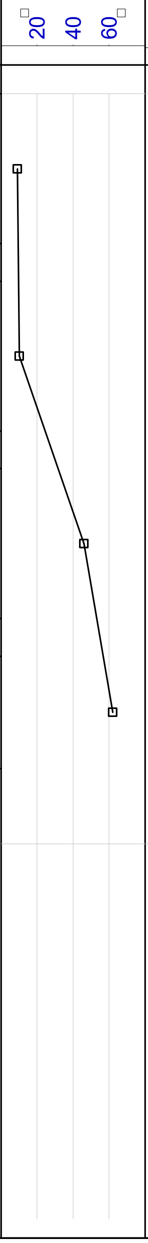
Project: Geotechnical Investigation

Client: 1600 James Naismith LP

Location: 1600 James Naismith Drive, Ottawa, ON

Drill Date: May 10, 2022 **Project Manager:** WT

SUBSURFACE PROFILE					SAMPLE									
Depth (m)	Symbol	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-Value	Standard Penetration N-Value	Shear Strength kPa	Water Content (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
		Ground Surface	99.77											
0		**Organics** ~ 300 mm	99.47	No Monitoring Well Installed										
		Glacial Till Silty gravelly sand, trace clay, brown, damp, loose to compact			SS	1	30	9						
1			98.09		SS	2	70	10						
2		Trace to some shale bedrock fragments, dense to very dense	97.03		SS	3	80	46						
					SS	4	100	62						
3		End of Borehole Borehole was terminated at 2.74 mbgs due to SPT refusal on probable bedrock. No groundwater was encountered.												
4														



Contractor: Canadian Environmental Drilling & Contractors Inc. **Grade Elevation:** 99.77 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: BH3

Project #: 306147 **Logged By:** MK

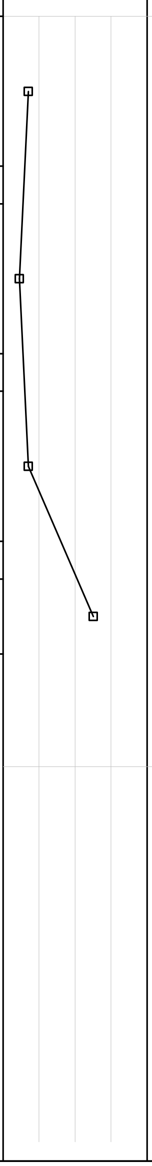
Project: Geotechnical Investigation

Client: 1600 James Naismith LP

Location: 1600 James Naismith Drive, Ottawa, ON

Drill Date: May 10, 2022 **Project Manager:** WT

SUBSURFACE PROFILE					SAMPLE									
Depth (m)	Symbol	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-Value	Standard Penetration N-Value 20 40 60	Shear Strength kPa 100 200	Water Content (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
		Ground Surface	99.31											
0		**Asphalt** ~ 150 mm		No Monitoring Well Installed										
		Fill Sand and gravel, some silt, brown, damp, compact	98.45		SS	1	60	14						
1		**Glacial Till** Silty gravelly sand, trace clay, brown, damp, loose to compact			SS	2	80	9						
2			96.92		SS	3	100	14			6.6			Hyd. MC
		Trace to some shale bedrock fragments, very dense	96.72		SS	4	100	50						
		End of Borehole												
3														
4														



Borehole was terminated at 2.59 mbgs due to SPT refusal on probable bedrock. No groundwater was encountered.

Contractor: Canadian Environmental Drilling & Contractors Inc. **Grade Elevation:** 99.21 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: BH4

Project #: 306147 **Logged By:** MK

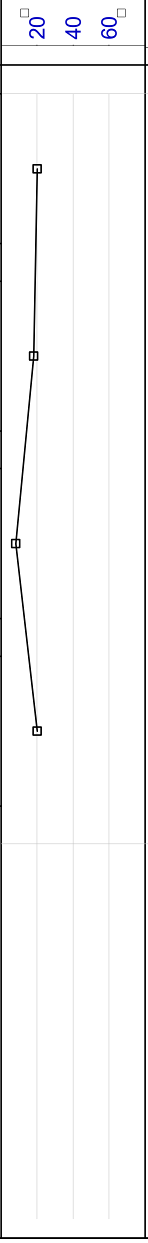
Project: Geotechnical Investigation

Client: 1600 James Naismith LP

Location: 1600 James Naismith Drive, Ottawa, ON

Drill Date: May 11, 2022 **Project Manager:** WT

SUBSURFACE PROFILE					SAMPLE									
Depth (m)	Symbol	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-Value	Standard Penetration N-Value □ 20 40 60 □	Shear Strength △ kPa △ 100 200	Water Content (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
		Ground Surface	100.31											
0		**Asphalt** ~ 100 mm		No Monitoring Well Installed										
		Fill Sand and gravel, trace silt, brown, damp, compact			SS	1	40	20						GS
1			99.24											
		Glacial Till Silty gravelly sand, trace clay, brown, damp, loose to compact			SS	2	60	18						
2					SS	3	40	8						
					SS	4	75	20						
			97.41											
3		End of Borehole Borehole was terminated at 2.90 mbgs due to SPT refusal on probable bedrock. No groundwater was encountered.												
4														



Contractor: Canadian Environmental Drilling & Contractors Inc. **Grade Elevation:** 100.31 m

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

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

PINCHIN

Log of Borehole: BH5

Project #: 306147 **Logged By:** MK

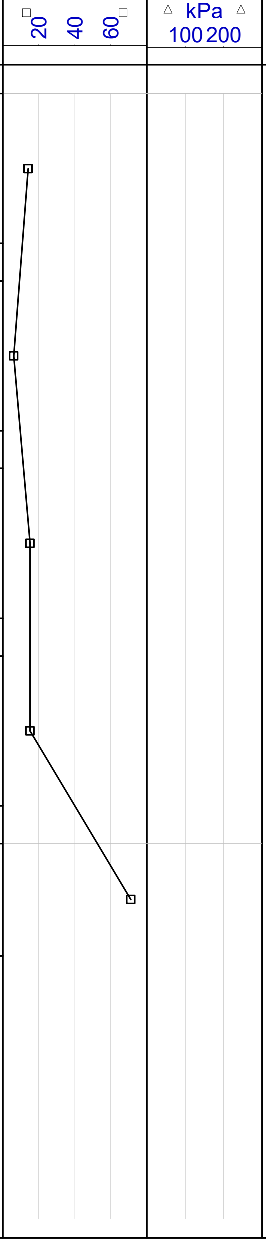
Project: Geotechnical Investigation

Client: 1600 James Naismith LP

Location: 1600 James Naismith Drive, Ottawa, ON

Drill Date: May 11, 2022 **Project Manager:** WT

SUBSURFACE PROFILE					SAMPLE									
Depth (m)	Symbol	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-Value	Standard Penetration N-Value □ 20 40 60 □	Shear Strength △ kPa △ 100 200	Water Content (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
		Ground Surface	99.99											
0		**Organics** ~ 300 mm	99.68	No Monitoring Well Installed	SS	1	30	14						
1		**Fill** Sand and gravel, trace silt, trace organics, brown, damp, loose to compact	98.77		SS	2	50	6						
2		**Glacial Till** Silty gravelly sand, trace clay, brown, damp, compact to very dense			SS	3	80	15						
3					SS	4	80	15						
			96.48		SS	5	60	71						
4		End of Borehole Borehole was terminated at 3.51 mbgs due to SPT refusal on probable bedrock. No groundwater was encountered.												



Contractor: Canadian Environmental Drilling & Contractors Inc. **Grade Elevation:** 99.99 m

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

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

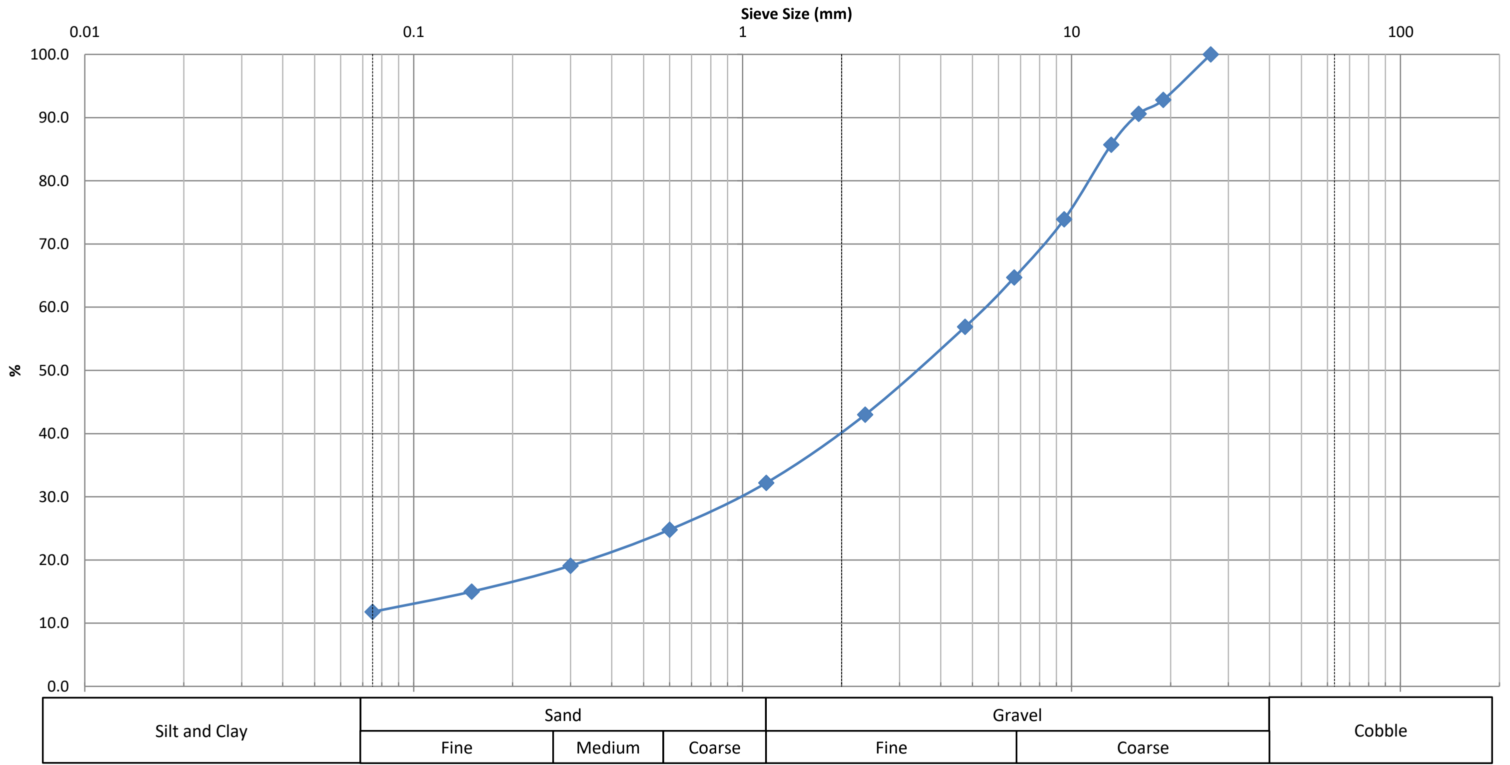
APPENDIX III

Laboratory Testing Reports for Soil Samples

patersongroup
consulting engineers

SIEVE ANALYSIS ASTM C136

CLIENT:	Pinchin	DESCRIPTION:	Crushed Stone	FILE NO:	PM4184
CONTRACT NO.:	306147	SPECIFICATION:	-	LAB NO:	34085
PROJECT:	Lab Testing	INTENDED USE:	Misc.	DATE RECEIVED:	20-May-22
		PIT OR QUARRY:	-	DATE TESTED:	30-May-22
DATE SAMPLED:	10-May-22	SOURCE LOCATION:	BH1	DATE REPORTED:	14-Jun-22
SAMPLED BY:	Client	SAMPLE LOCATION:	0' - 2'	TESTED BY:	CP



Identification	Soil Classification	MC(%)	LL	PL	PI	Cc	Cu
						3.49	110.0

D100	D60	D30	D10	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
26.5	5.5	0.98	0.05	43.1	45.1	11.8	

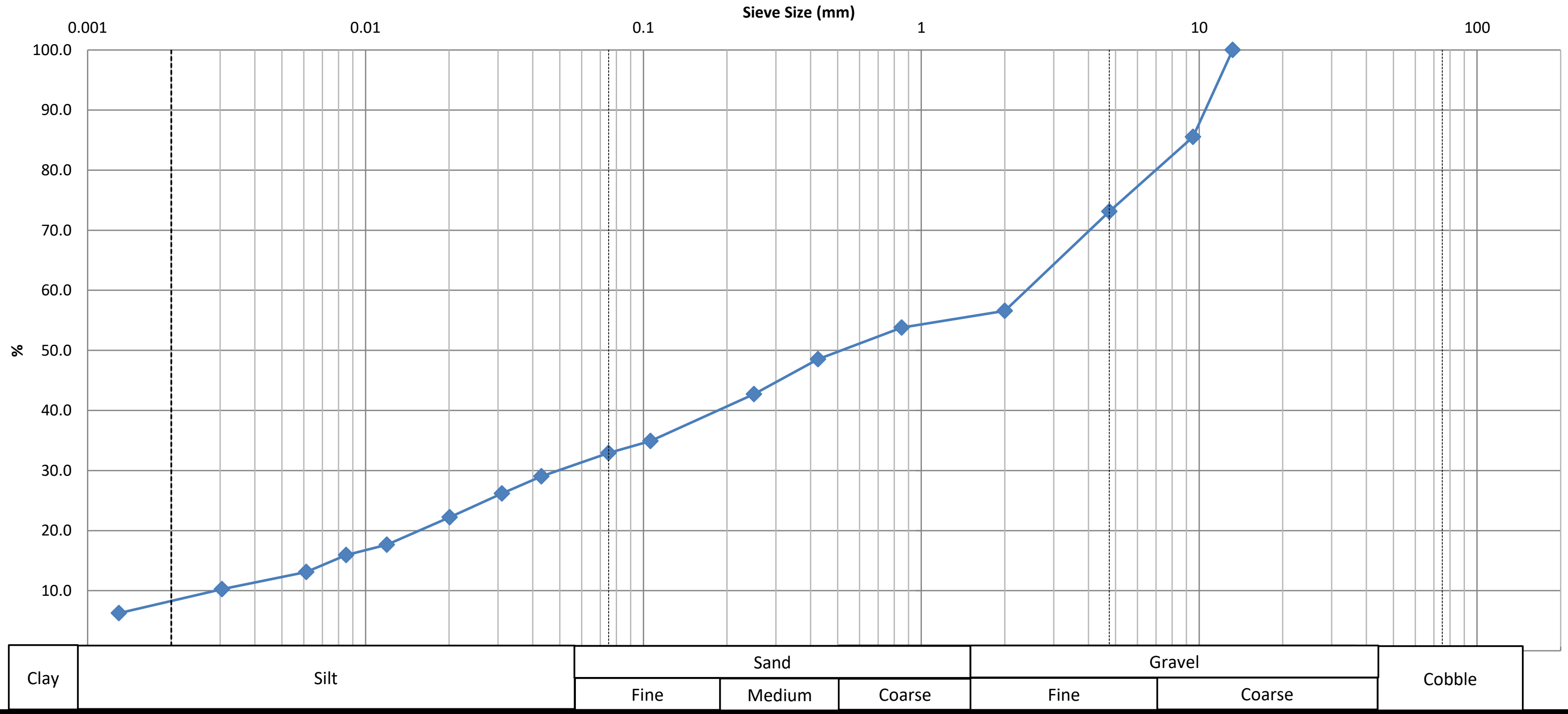
Comments:

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

patersongroup
consulting engineers

SIEVE ANALYSIS
ASTM C136

CLIENT:	Pinchin	DEPTH:	5.0'-7.0'	FILE NO:	PM4184
CONTRACT NO.:		BH OR TP No.:	BH3	LAB NO:	34086
PROJECT:	306147			DATE RECEIVED:	20-May-22
				DATE TESTED:	22-May-22
DATE SAMPLED:	-			DATE REPORTED:	13-Jun-22
SAMPLED BY:	Client			TESTED BY:	DK



Identification	Soil Classification				MC(%)	LL	PL	PI	Cc	Cu
					6.6					
	D100	D60	D30	D10	Gravel (%)	Sand (%)	Silt (%)	Clay (%)		
					26.9	40.2	23.9	9.0		

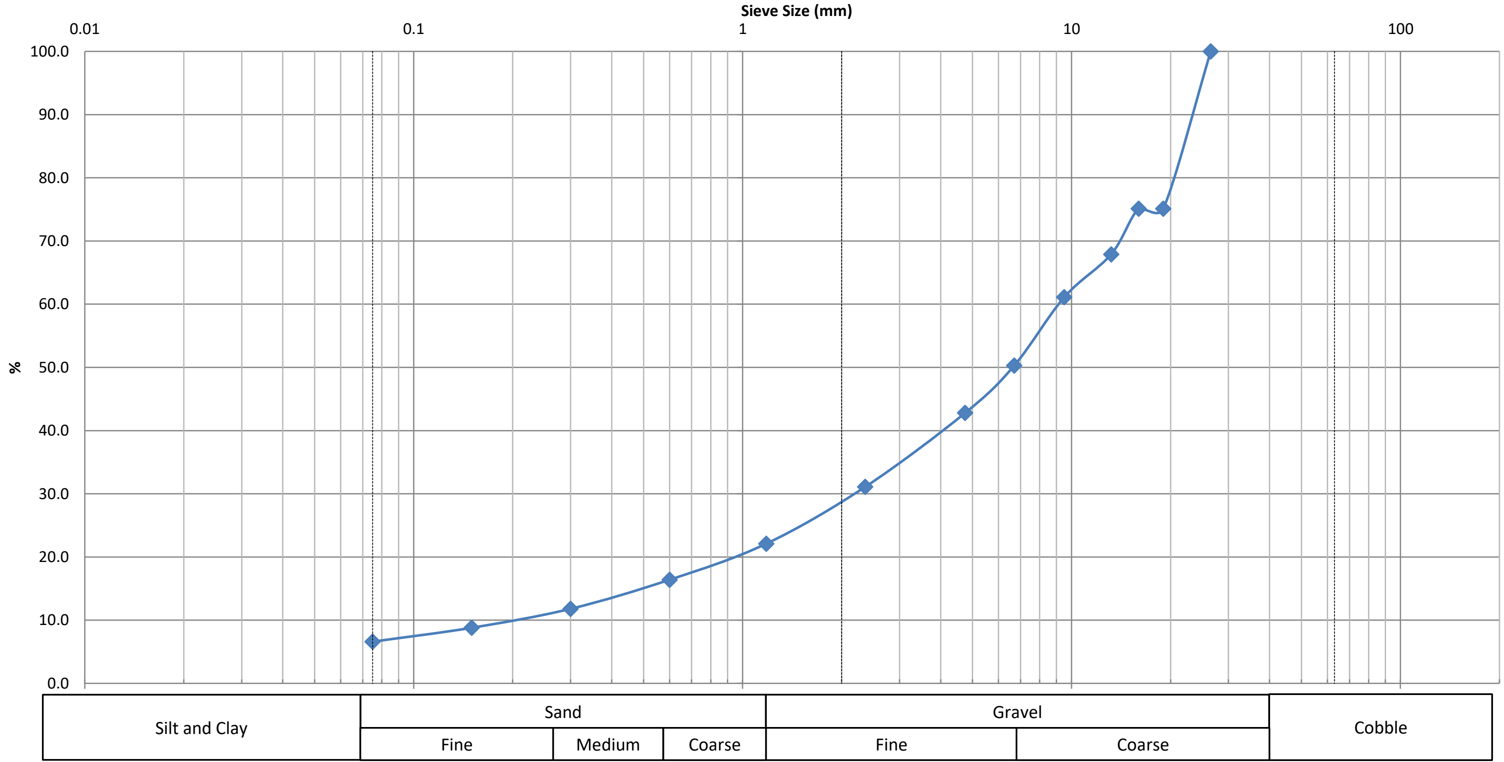
Comments:

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

patersongroup
consulting engineers

SIEVE ANALYSIS ASTM C136

CLIENT:	Pinchin	DESCRIPTION:	Crushed Stone	FILE NO:	PM4184
CONTRACT NO.:	306147	SPECIFICATION:	-	LAB NO:	34087
PROJECT:	Lab Testing	INTENDED USE:	Misc.	DATE RECEIVED:	20-May-22
		PIT OR QUARRY:	-	DATE TESTED:	30-May-22
DATE SAMPLED:	10-May-22	SOURCE LOCATION:	BH4	DATE REPORTED:	14-Jun-22
SAMPLED BY:	Client	SAMPLE LOCATION:	0' - 2'	TESTED BY:	CP



Identification	Soil Classification	MC(%)	LL	PL	PI	Cc	Cu
						2.63	46.0

D100	D60	D30	D10	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
26.5	9.2	2.2	0.2	57.2	36.2	6.6	

Comments:

REVIEWED BY:	Curtis Beadow	Joe Fosyth, 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.