FINAL

Geotechnical Investigation – Proposed Commercial Development

109 – 121 Willowlea Road, Ottawa, Ontario

Prepared for:

Access Property Development Inc.

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1.0 INTRODUCTION AND SCOPE

Pinchin Ltd. (Pinchin) was retained by Access Property Development Inc. (Client) to conduct a Geotechnical Investigation and provide subsequent geotechnical design recommendations for the proposed commercial development to be located at 109 – 121 Willowlea Road, 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 is to consist of two single-storey, slab-on-grade (i.e. no basement level) self-storage buildings complete with new Site services and asphalt surfaced access roadways and parking 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 six (6) sampled boreholes (Boreholes BH1 to BH6), 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;
- Foundation design recommendations including bedrock bearing resistances at Ultimate Limit States (ULS) design;
- Potential total and differential settlements;
- Foundation frost protection and engineered fill specifications and installation;
- Seismic Site classification for seismic Site response;
- Concrete floor slab-on-grade support recommendations;
- Asphaltic concrete pavement structure design for parking areas and access roadways; and
- Potential construction concerns.

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

2.0 SITE DESCRIPTION AND GEOLOGICAL SETTING

The Site is located on the south side of Willowlea Road, approximately 200 metres south of Highway 417 in Ottawa, Ontario. The west portion of the Site is currently developed with a single storey slab-on-grade self-storage building complete with a gravel surfaced storage yard which is partially occupied by storage pods. The east portion of the Site is currently undeveloped and consists of wild undergrowth with some grassed areas. The lands adjacent to the Site are developed with a combination of single family and multi unit residential buildings.

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 Paleozoic bedrock. The underlying bedrock at this Site is of the Shadow Lake Formation consisting of limestone, dolostone, shale, arkose, and sandstone (Ontario Geological Survey Map 1972, published 1978).

3.0 GEOTECHNICAL FIELD INVESTIGATION AND METHODOLOGY

Pinchin completed a field investigation at the Site on May 6, 2021 by advancing a total of six sampled boreholes throughout the Site. The boreholes were advanced to depths ranging from approximately 0.6 to 2.1 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 Geoprobe 7822 DT direct push drill rig which was equipped with standard soil sampling equipment. Soil samples 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 the foundation wall on the exterior side of the southeast corner of the existing building located on the west portion of the Site, at the approximate location shown on Figure 2; and
- Elevation: 100.0 metres (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 a combination of organics, granular fill and glacial till overlying probable bedrock to the maximum borehole refusal depth of approximately 2.1 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 to BH5 and were measured to range in thickness from approximately 100 to 600 mm.

Granular fill was encountered at the surface in Boreholes BH1 and BH6 and underlying the surficial organics in Boreholes BH2 and BH3. The fill material was measured to range in thickness from approximately 0.2 to 1.5 m thick and ranged in soil matrix from sand and gravel containing trace silt, to sand containing some gravel and trace silt. The non-cohesive material had a very loose to very dense relative density based SPT 'N' values of between 1 and greater than 50 blows per 300 mm penetration of a split spoon sampler. The result of one particle size distribution analysis completed on a sample of the fill indicates that the sample contains approximately 36% gravel, 55% sand, and 9% silt sized particles.

The glacial till was encountered underlying the granular fill and organics in Boreholes BH1 and BH4, respectively, and extended down to the probable bedrock surface in each borehole. The glacial till ranged in soil matrix from sand and silt containing some gravel, to silty sand containing some gravel. The non-cohesive glacial till had a very loose to compact relative density based SPT 'N' values of 2 to 10 blows per 300 mm penetration of a split spoon sampler. The results of two particle size distribution analyses completed on samples of the glacial till indicate that the samples contained approximately 16 to 17% gravel, 45 to 59% sand, and 25 to 38% silt sized particles.

Probable bedrock was encountered within all boreholes between approximately 0.6 and 2.1 mbgs (local elevations 97.75 to 98.95 m). It is noted that coring to confirm the presence of bedrock was not completed during the field investigation.

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 encountered between approximately 0.2 and 2.0 mbgs within Boreholes BH1, BH2, and BH4; however, was not encountered within the remainder of the boreholes at drilling completion. The water encountered is perched above the relatively impermeable probable rock surface. Seasonal variations in the water table should be expected, with higher levels occurring during wet weather conditions in the spring and fall and lower levels occurring during dry weather conditions.

5.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

5.1 General Information

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

Based on information provided by the Client, it is Pinchin's understanding that the proposed development is to consist of two single-storey, slab-on-grade (i.e. no basement level) self-storage buildings complete with new Site services and asphalt surfaced access roadways and parking areas.

Probable bedrock was encountered between approximately 0.6 and 2.1 mbgs within the boreholes advanced at the Site. As such, Pinchin recommends constructing the proposed buildings on conventional shallow strip and spread footings founded on the underlying probable bedrock surface. It is noted that depending on the proposed final grades for the Site, this may require a portion of the foundation system to be extended down to the bedrock surface.

5.2 Site Preparation

The existing organic material is not considered suitable to remain below the proposed buildings, access roadways, and parking areas and will need to be removed. In calculating the approximate quantity of organics and fill to be stripped, we recommend that the thicknesses provided on the individual borehole logs be increased by 50 mm to account for variations and some stripping of the mineral soil below. The existing granular fill and glacial till material encountered within the boreholes may remain in place in the proposed hard and soft landscaped areas, but the existing fill material is not suitable to remain below the proposed buildings.

Preparation of the Site for the proposed development will consist of removing all surficial and overburden materials down to the underlying bedrock surface in the vicinity of the proposed building footprints (below the foundations and floor slabs).

Prior to placing any fill material at the Site, the bedrock and/or subgrade soil should be inspected by a qualified geotechnical engineer and loosened/soft pockets should be sub excavated and replaced with an engineered fill. All fill material required to raise grades is to be installed in maximum 200 mm thick loose lifts, compacted to 100% of its Standard Proctor Maximum Dry Density (SPMDD), within plus 2 to minus 4 of the optimum moisture contents. It is recommended that the subgrade fill comprise material meeting Ontario Provincial Standard Specifications 1010 (OPSS 1010) Granular 'B' Type I or Type II specification.

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.

5.3 Open Cut Excavations and Anticipated Groundwater Management

Excavations for the building foundations and new Site services will extend to depths ranging from approximately 0.6 to 2.1 mbgs. As such, there is a potential for a portion of the bedrock to be removed to accommodate the 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. Groundwater was encountered between approximately 0.2 and 2.0 mbgs within Boreholes BH1, BH2, and BH4; however, was not encountered within the remainder of the boreholes at drilling completion. It is noted that the boreholes did not advance into the bedrock surface; as such, there is a potential for groundwater to be encountered during excavations into the bedrock, at the locations where groundwater was not encountered in the overburden material.

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. Excavations extending below the groundwater table would be classified as a Type 4 soil and temporary excavations will have to be sloped back at 3H to 1V from the base of the excavation.

The upper approximate 1.0 m of bedrock in this area 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.

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

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.

Minor to moderate groundwater inflow through the granular fill and glacial till materials is expected where the excavations extend less than 1.0 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.

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. Excavations to conventional design depths for the building foundations 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.

5.4 Site Servicing

5.4.1 Pipe Bedding and Cover Materials for Flexible and Rigid Pipes

The subgrade conditions beneath the Site services will consist of bedrock. No support problems are anticipated for flexible or rigid pipes founded on the 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.

All granular fill material is to be placed in maximum 200 mm thick loose lifts compacted to a minimum of 98% SPMDD.

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. 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

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.

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.

It is anticipated that imported material will be required to backfill the trenches due to minimal amount of natural soil observed at the Site. 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 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.

5.5 Foundation Design

5.5.1 Shallow Foundations Bearing on Bedrock

For conventional shallow strip and spread footings established directly on the weathered bedrock surface, a factored geotechnical bearing resistance of 500 kPa may be used at ULS. Higher bearing resistances may be available on the unweathered bedrock; however, the bedrock should be cored to confirm this recommendation.

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

The bearing resistance of 500 kPa assumes the bedrock is cleaned of all overburden material and any loose rock pieces. The bedrock should be cleaned with air or water pressure exposing clean sound bedrock. If construction proceeds during freezing weather conditions water should not be allowed to pool and freeze in bedrock depressions. All concrete should be installed and maintained above freezing temperatures as required by the concrete supplier.

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

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

5.5.2 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 2.1 mbgs where refusal was encountered on probable bedrock. SPT "N" values within the soil deposit ranged between 1 and greater than 50 blows per 300 mm. As such, based on Table 4.1.8.4.A of the OBC, this Site has been classified as Class C. A Site Class C has an average shear wave velocity (Vs) of between 360 and 760 m/s. It is recommended that shear wave velocity soundings be completed at the Site once the final design and depths of foundations are known as a higher Site Classification may be available.

5.5.3 Foundation Transition Zones

Where strip footings are founded at different elevations, the bedrock 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 bedrock, reviewed, and approved by a licensed geotechnical engineer.

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

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

5.5.5 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.

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

Where the foundations for heated buildings do not have the minimum 1.8 m of soil cover frost protection, they should be protected from frost with a combination of soil cover and rigid polystyrene insulation, such as Dow Styrofoam or equivalent product. 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. It is noted that the existing granular fill material encountered within the boreholes is suitable for reuse as a foundation wall backfill material. The backfill material used against the foundation must be placed so that the allowable lateral capacity is achieved. All granular material is to be placed in maximum 300 mm thick lifts compacted to a minimum of 100% SPMDD in hard landscaping areas and 95% SPMDD in soft landscaping areas. It is recommended that inspection and testing be carried out during construction to confirm backfill quality, thickness and to ensure compaction requirements are achieved.

5.5.7 Concrete Slab-on-Grade

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

Based on the in-situ conditions, it is recommended to establish a concrete floor slab-on-grade on a minimum 200 mm thick layer of Granular ’A’ (OPSS 1010). The purpose of the Granular ’A’ is mainly to provide a level surfaced for the concrete formwork. Alternatively, consideration may also be given to using a 200 mm thick layer of uniformly compacted 19 mm clear stone. Any required up-fill should consist of a material meeting the specifications of Granular ‘B’ Type I or Type II (OPSS 1010). The existing granular fill material encountered within the boreholes is also suitable as an up-fill material.

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
Granular Fill	45,000

5.6 Asphaltic Concrete Pavement Structure Design for Parking Lot and Access Roadways

5.6.1 Discussion

Parking areas and access roadways will be constructed around the proposed buildings. The in-situ granular fill, glacial till, and probable bedrock materials are considered sufficient bearing materials for an asphaltic concrete pavement structure provided all organics and deleterious materials are removed prior to installing the engineered fill material.

At this time Pinchin is unaware of the proposed final grades for the parking lot and access roadways. As such, provided the pavement structure overlies the in-situ granular fill, glacial till and/or probable bedrock, the following pavement structure is recommended.

5.6.2 Pavement Structure

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

Pavement Layer	Compaction Requirements	Parking Areas	Access Roadways
Surface Course Asphaltic Concrete HL-3 (OPSS 1150)	92% MRD as per OPSS 310	40 mm	40 mm
Binder Course Asphaltic Concrete HL-8 (OPSS 1150)	92 % MRD as per OPSS 310	50 mm	80 mm
Base Course: Granular “A” (OPSS 1010)	100% Standard Proctor Maximum Dry Density (ASTM-D698)	150 mm	150 mm
Subbase Course: Granular “B” Type I (OPSS 1010)	100% Standard Proctor Maximum Dry Density (ASTM D698)	300 mm	450 mm

Notes:

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

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

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

5.6.3 Pavement Structure Subgrade Preparation and Granular Up Fill

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

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

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

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

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

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

5.6.4 Drainage

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

The shallow bedrock has poor natural drainage and therefore it is recommended that pavement subdrains be installed in the lower areas and be connected to the catch basins.

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

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

6.0 SITE SUPERVISION & QUALITY CONTROL

It is recommended that all geotechnical aspects of the project be reviewed and confirmed under the appropriate geotechnical supervision, to routinely check such items. This includes but is not limited to inspection and confirmation of the bedrock surface prior to pouring any foundations or footings, backfilling, or engineered fill installation to ensure that the actual conditions are not markedly different than what was observed at the borehole locations and geotechnical components are constructed as per

Pinchin's recommendations. Compaction quality control of engineered fill material (full-time monitoring) is recommended as standard practice, as well as regular sampling and testing of aggregates and concrete, to ensure that physical characteristics of materials for compliance during installation and satisfies all specifications presented within this report.

7.0 TERMS AND LIMITATIONS

This Geotechnical Investigation was performed for the exclusive use of Access Property Development Inc. (Client) in order to evaluate the subsurface conditions at 109 – 121 Willowlea Road, 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.

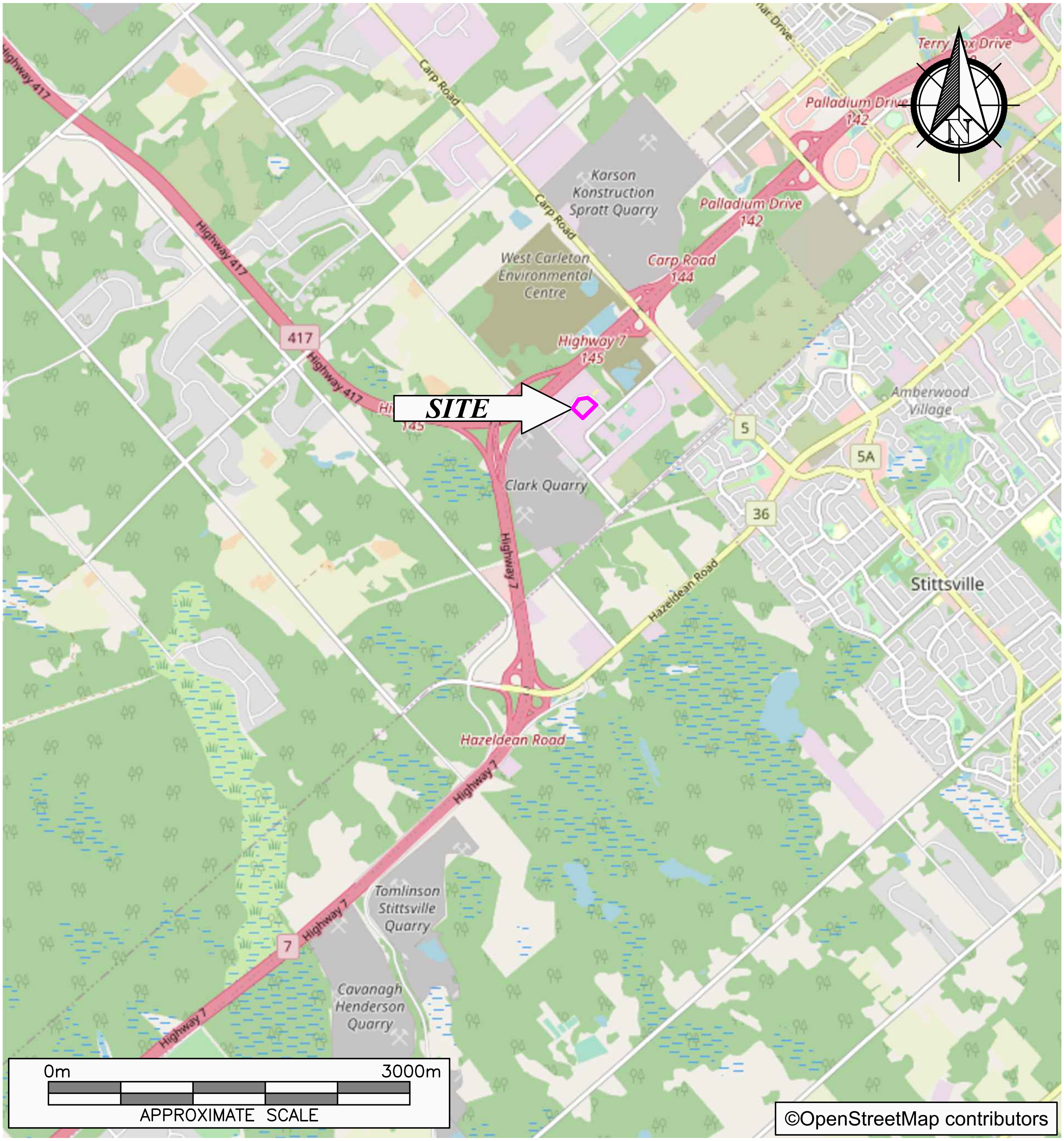
Specific limitations related to the legal and financial and limitations to the scope of the current work are outlined in our proposal, the attached Methodology, and the Authorization to Proceed, Limitation of Liability and Terms of Engagement which accompanied the proposal.

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.

291222 Geotechnical Investigation 109-121 Willowlea Rd Ottawa ON Access Property Development.doc

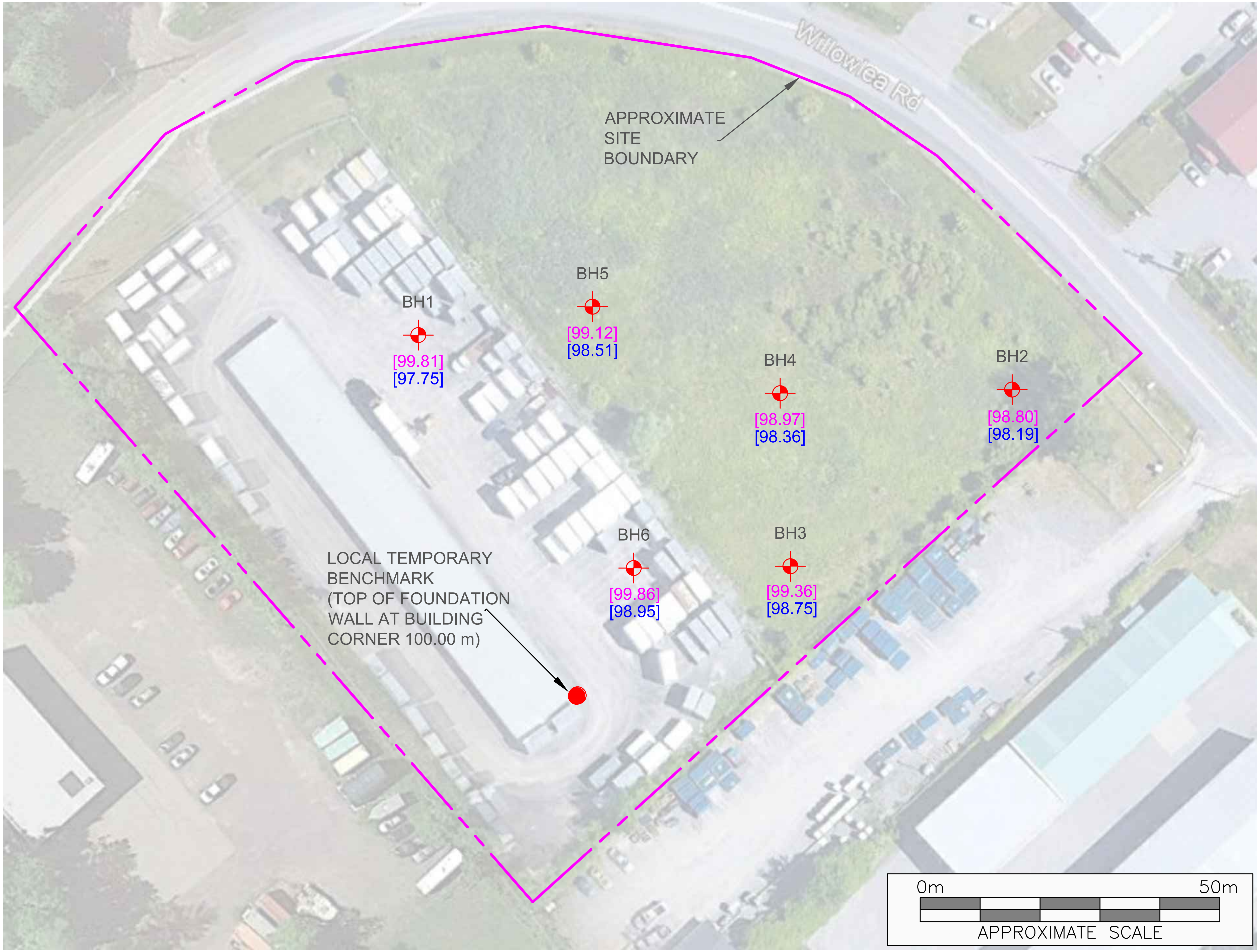
Template: Master Geotechnical Investigation Report – Ontario, GEO, April 1, 2020

FIGURES



PINCHIN

PROJECT NAME: GEOTECHNICAL INVESTIGATION			
CLIENT NAME: ACCESS PROPERTY DEVELOPMENT INC.			
PROJECT LOCATION: 109 - 121 WILLOWLEA ROAD, OTTAWA, ONTARIO			
FIGURE NAME: KEY MAP			FIGURE NO.: 1
APPROXIMATE SCALE: AS SHOWN	PROJECT NO.: 291222	DATE: JUNE 2021	



LEGEND

- BOREHOLE LOCATION
- [XX.XX] GROUND SURFACE ELEVATION (m)
- [XX.XX] BOREHOLE REFUSAL ELEVATION (m)
- m METRES

PINCHIN

PROJECT NAME
GEOTECHNICAL INVESTIGATION

CLIENT NAME
ACCESS PROPERTY DEVELOPMENT INC.

PROJECT LOCATION
109 - 121 WILLOWLEA ROAD, OTTAWA, ONTARIO

FIGURE NAME
BOREHOLE LOCATION PLAN

APPROXIMATE SCALE	PROJECT NO.
AS SHOWN	291222
DATE	FIGURE NO.
JUNE 2021	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:

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

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 #: 291222 **Logged By:** WT

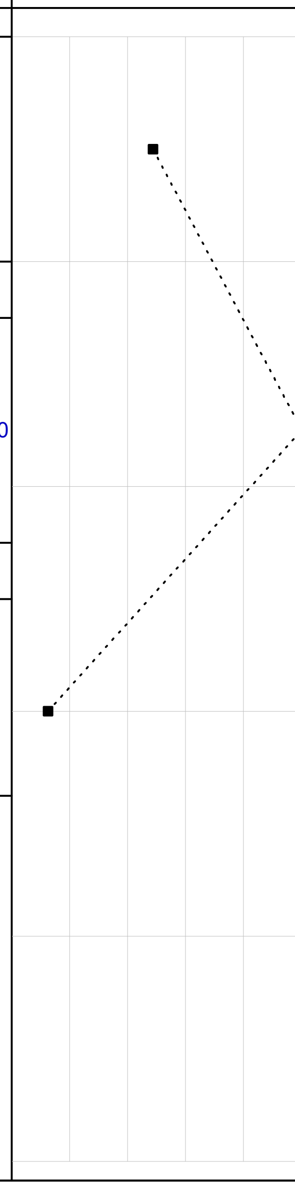
Project: Geotechnical Investigation

Client: Access Property Development Inc.

Location: 109-121 Willowlea Road, Ottawa, Ontario

Drill Date: May 6, 2021 **Project Manager:** WT

SUBSURFACE PROFILE					SAMPLE							
Depth (m)	Symbol	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-values	SPT N-values (20, 40, 60) / Shear Strength kPa (50, 100, 150, 200)	Lab Analysis	Moisture (%)	Plasticity Index
		Ground Surface	99.81									
0		***Fill*** Grey sand and gravel, trace silt, dense, damp		No Monitoring Well Installed	SS	1	65	39				
		Brown sand, some gravel, trace silt, very dense, damp	99.05		SS	2	80	>50				
1												
		Glacial Till Brown sand and silt, some gravel, compact, moist to wet	98.29		SS	3	60	10		Hyd.		
2			97.75									
		End of Borehole Borehole terminated at approximately 2.05 mbgs due to auger and split spoon refusal on probable bedrock. At drilling completion, groundwater was measured at approximately 1.98 mbgs.										
3												



Contractor: Strata Drilling Group

Drilling Method: Hollow Stem Auger / Split Spoon

Well Casing Size: N/A

Grade Elevation: 99.81 m

Top of Casing Elevation: N/A

Sheet: 1 of 1

Log of Borehole: BH2

Project #: 291222 **Logged By:** WT

Project: Geotechnical Investigation

Client: Access Property Development Inc.

Location: 109-121 Willowlea Road, Ottawa, Ontario

Drill Date: May 6, 2021 **Project Manager:** WT

Depth (m)	Symbol	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-values	Lab Analysis	Moisture (%)	Plasticity Index
		Ground Surface	98.80								
0		***Organics*** ~ 100 mm	98.70	No Monitoring Well Installed							
		Fill Brown sand and gravel, trace silt, very loose, moist to wet	98.19		SS	1	30	4			
		End of Borehole Borehole terminated at approximately 0.61mbgs due to auger and split spoon refusal on probable bedrock. At drilling completion, groundwater was measured at approximately 0.15 mbgs.									
1											
2											
3											

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

Contractor: Strata Drilling Group **Grade Elevation:** 98.80 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 #: 291222 **Logged By:** WT

Project: Geotechnical Investigation

Client: Access Property Development Inc.

Location: 109-121 Willowlea Road, Ottawa, Ontario

Drill Date: May 6, 2021 **Project Manager:** WT

SUBSURFACE PROFILE					SAMPLE							
Depth (m)	Symbol	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-values	SPT N-values 20 40 60 / Shear Strength kPa 50 100 150 200	Lab Analysis	Moisture (%)	Plasticity Index
		Ground Surface	99.36	No Monitoring Well Installed								
0		**Organics** ~ 460 mm	98.90		SS	1	30	1				
		Fill Brown sand and gravel, trace silt, very loose, wet	98.75									
		End of Borehole Borehole terminated at approximately 0.61mbgs due to auger and split spoon refusal on probable bedrock. At drilling completion, no free groundwater was encountered.										
1												
2												
3												

Contractor: Strata Drilling Group **Grade Elevation:** 99.36 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 #: 291222 **Logged By:** WT

Project: Geotechnical Investigation

Client: Access Property Development Inc.

Location: 109-121 Willowlea Road, Ottawa, Ontario

Drill Date: May 6, 2021 **Project Manager:** WT

Depth (m)	Symbol	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-values	SPT N-values (20 40 60) / Shear Strength kPa (50 100 150 200)	Lab Analysis	Moisture (%)	Plasticity Index
		Ground Surface	98.97	No Monitoring Well Installed								
0		**Organics** ~ 300 mm	98.67									
		Glacial Till Brown silty sand, some gravel, very loose, moist to wet	98.36		SS	1	30	2	■	Hyd.		
		End of Borehole Borehole terminated at approximately 0.61mbgs due to auger and split spoon refusal on probable bedrock. At drilling completion, groundwater was encountered at 0.45 mbgs.										
1												
2												
3												

Contractor: Strata Drilling Group

Grade Elevation: 98.97 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: BH5

Project #: 291222 **Logged By:** WT

Project: Geotechnical Investigation

Client: Access Property Development Inc.

Location: 109-121 Willowlea Road, Ottawa, Ontario

Drill Date: May 6, 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	99.12	No Monitoring Well Installed								
		Organics ~ 600 mm	98.51		SS	1	30	2	■			
1		End of Borehole Borehole terminated at approximately 0.61mbgs due to auger and split spoon refusal on probable bedrock. At drilling completion, no free groundwater was encountered.										
2												
3												

Contractor: Strata Drilling Group

Drilling Method: Hollow Stem Auger / Split Spoon

Well Casing Size: N/A

Grade Elevation: 99.12 m

Top of Casing Elevation: N/A

Sheet: 1 of 1

Log of Borehole: BH6

Project #: 291222 **Logged By:** WT

Project: Geotechnical Investigation

Client: Access Property Development Inc.

Location: 109-121 Willowlea Road, Ottawa, Ontario

Drill Date: May 6, 2021 **Project Manager:** WT

SUBSURFACE PROFILE					SAMPLE							
Depth (m)	Symbol	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-values	SPT N-values 20 40 60 / Shear Strength kPa 50 100 150 200	Lab Analysis	Moisture (%)	Plasticity Index
		Ground Surface	99.86									
0		**Fill** Grey sand and gravel, trace silt, dense, damp		No Monitoring Well Installed	SS	1	60	31		Hyd.		
		Brown sand, some gravel, trace silt, very dense, damp	99.10		SS	2	30	>50				
			98.95									
1		End of Borehole Borehole terminated at approximately 0.91mbgs due to auger and split spoon refusal on probable bedrock. At drilling completion, no free groundwater was encountered.										
2												
3												

Contractor: Strata Drilling Group

Drilling Method: Hollow Stem Auger / Split Spoon

Well Casing Size: N/A

Grade Elevation: 99.86 m

Top of Casing Elevation: N/A

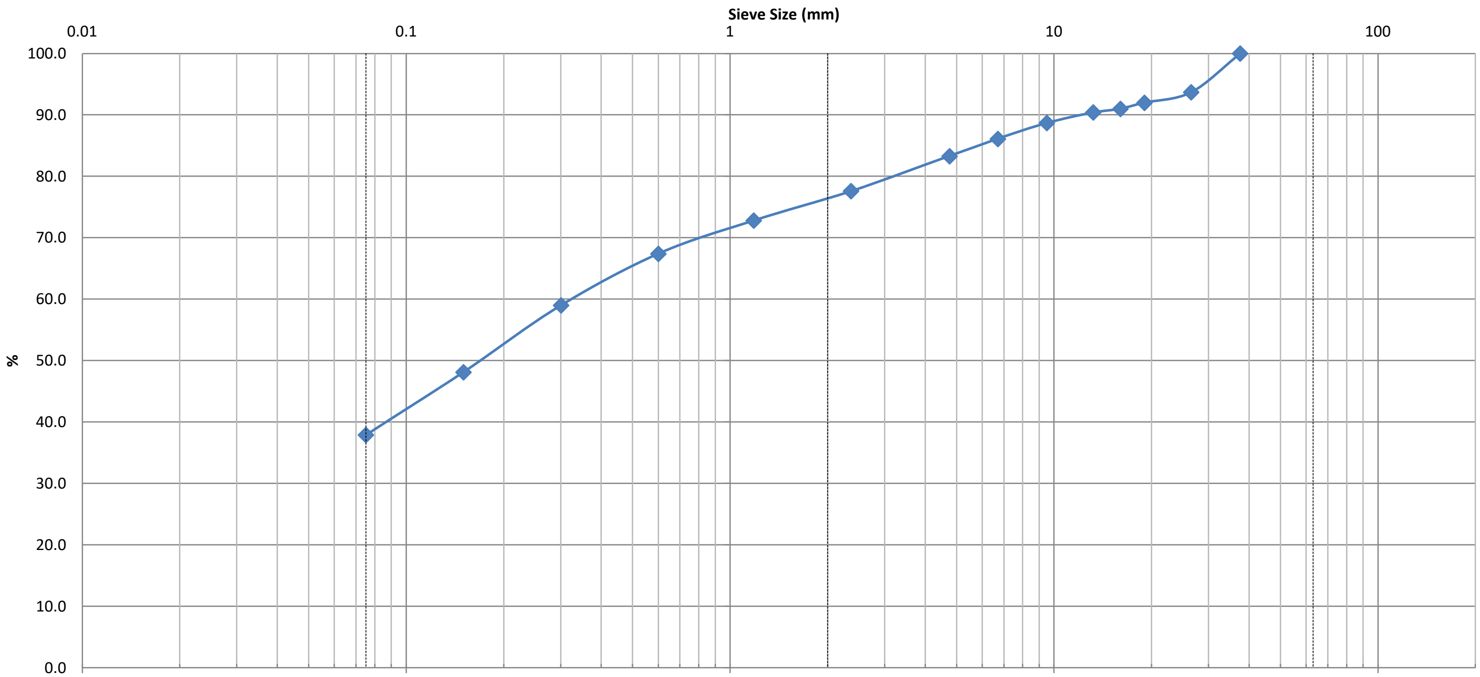
APPENDIX III

Laboratory Testing Reports for Soil Samples

patersongroup
consulting engineers

SIEVE ANALYSIS
ASTM C136

CLIENT:	Pinchin	DESCRIPTION:	Silty Clay w Gravel	FILE NO:	PM4184
CONTRACT NO.:	-	SPECIFICATION:	**Clay / Gravel**	LAB NO:	24362
PROJECT:	291222	INTENDED USE:	-	DATE RECEIVED:	14-May-21
		PIT OR QUARRY:	-	DATE TESTED:	17-May-21
DATE SAMPLED:	-	SOURCE LOCATION:	BH1	DATE REPORTED:	19-May-21
SAMPLED BY:	Client	SAMPLE LOCATION:	15' - 7'	TESTED BY:	D.K/D.B



Silt and Clay	Sand			Gravel		Cobble
	Fine	Medium	Coarse	Fine	Coarse	

Identification	Soil Classification				MC(%)	LL	PL	PI	Cc	Cu
									0.23	16.0
	D100	D60	D30	D10	Gravel (%)	Sand (%)	Silt (%)		Clay (%)	
	37.5	0.32	0.038	0.02	16.7	45.4	37.9			

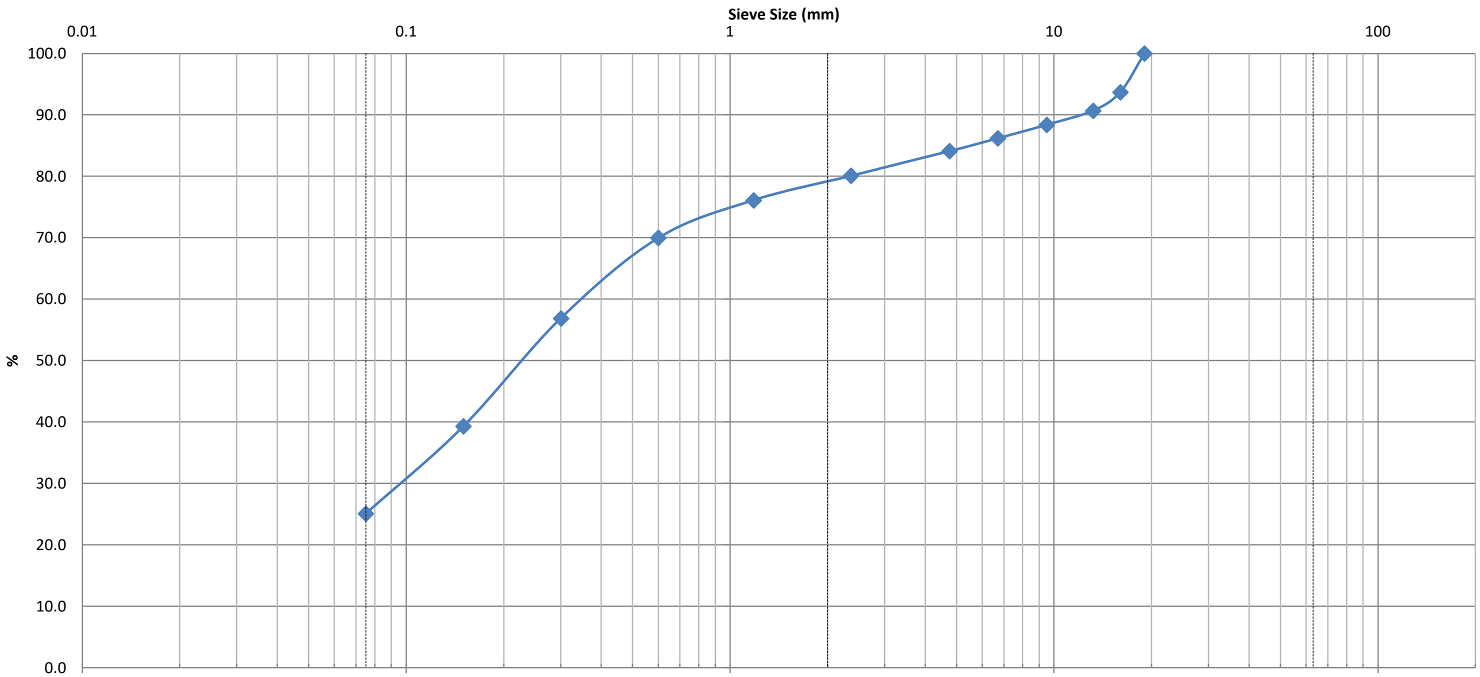
Comments:

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

patersongroup
consulting engineers

SIEVE ANALYSIS
ASTM C136

CLIENT:	Pinchin	DESCRIPTION:	Silty Clay w Gravel	FILE NO:	PM4184
CONTRACT NO.:	-	SPECIFICATION:	**Clay / Gravel**	LAB NO:	24363
PROJECT:	291222	INTENDED USE:	-	DATE RECEIVED:	14-May-21
		PIT OR QUARRY:	-	DATE TESTED:	17-May-21
DATE SAMPLED:	-	SOURCE LOCATION:	BH4	DATE REPORTED:	19-May-21
SAMPLED BY:	Client	SAMPLE LOCATION:	10' - 2'	TESTED BY:	D.K/D.B



Silt and Clay	Sand			Gravel		Cobble
	Fine	Medium	Coarse	Fine	Coarse	

Identification	Soil Classification				MC(%)		LL	PL	PI	Cc	Cu
										0.96	12.5
	D100	D60	D30	D10	Gravel (%)		Sand (%)		Silt (%)		Clay (%)
	19.0	0.35	0.097	0.028	15.9		59.0		25.1		

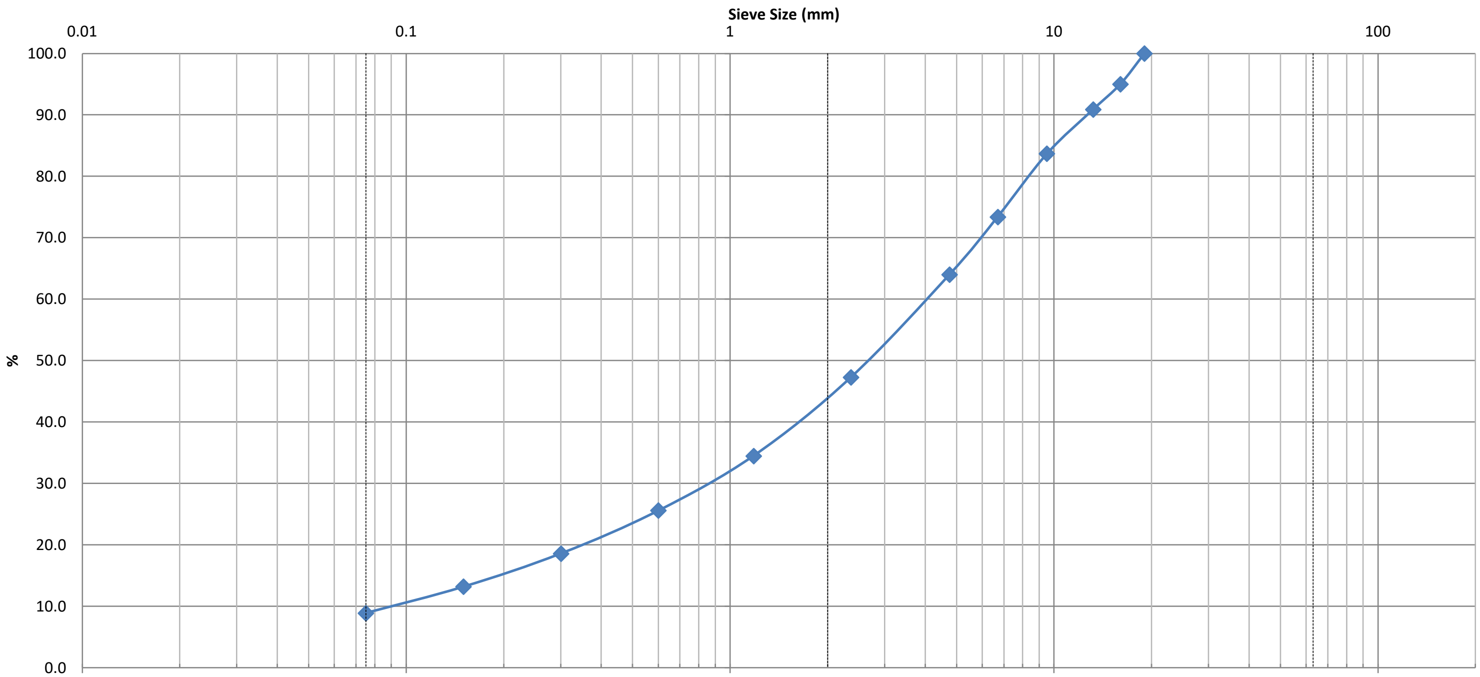
Comments:

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

patersongroup
consulting engineers

SIEVE ANALYSIS
ASTM C136

CLIENT:	Pinchin	DESCRIPTION:	Sand and Gravel	FILE NO:	PM4184
CONTRACT NO.:	-	SPECIFICATION:	**Sand / Gravel**	LAB NO:	24364
PROJECT:	291222	INTENDED USE:	-	DATE RECEIVED:	14-May-21
		PIT OR QUARRY:	-	DATE TESTED:	17-May-21
DATE SAMPLED:	-	SOURCE LOCATION:	BH6	DATE REPORTED:	19-May-21
SAMPLED BY:	Client	SAMPLE LOCATION:	10' - 2'	TESTED BY:	D.K/D.B



Silt and Clay	Sand			Gravel		Cobble
	Fine	Medium	Coarse	Fine	Coarse	

Identification	Soil Classification				MC(%)	LL	PL	PI	Cc	Cu
									2.03	45.1
	D100	D60	D30	D10	Gravel (%)	Sand (%)	Silt (%)		Clay (%)	
	19.0	4.1	0.87	0.091	36.0	55.1	8.9			

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.