FINAL

Geotechnical Investigation – Proposed Residential Development

5581 Doctor Leach Drive, Manotick, Ontario

Prepared for:

Rideau Non-Profit Housing Inc.
5581 Doctor Leach Drive
Manotick, ON K4M 1J6

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1.0 INTRODUCTION AND SCOPE

Pinchin Ltd. (Pinchin) was retained by Rideau Non-Profit Housing Inc. (Client) to conduct a Geotechnical Investigation and provide subsequent geotechnical design recommendations for the proposed residential development to be located at 5581 Doctor Leach Drive, Manotick, Ontario (Site). The Site location is shown on Figure 1.

Based on information provided by the Client, it is Pinchin's understanding that the development will consist of a two-storey residential apartment building complete with a partial basement level located in the southeast corner of the building. The proposed development will also include 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 four (4) sampled boreholes (Boreholes BH1 to BH4), at the Site.

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 soil bearing resistances at Ultimate Limit States (ULS) and Serviceability Limit States (SLS) design;
- Potential total and differential settlements;
- Foundation frost protection and engineered fill specifications and installation;
- Seismic Site classification for seismic Site response;
- Partial basement level design recommendations;
- 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 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 east side of Doctor Leach Drive, approximately 900 m south of the intersection of Rideau Valley Drive and Bankfield Road in Manotick, Ontario. The north portion of the Site is currently developed with a two-storey residential apartment building and asphalt surfaced parking areas. The south portion of the Site is currently undeveloped and consists of mature trees adjacent to Doctor Leach Drive and a well-maintained grass field where the proposed development will be located. The lands adjacent to the Site are developed with either single family residential and townhouse dwellings, or commercial buildings one to two stories in height.

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 Beekmantown Group consisting of dolostone and sandstone (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 August 25, 2022, by advancing a total of four (4) sampled boreholes (Borehole BH1 to BH4) throughout the Site. The boreholes were advanced to depths ranging from approximately 6.6 to 7.8 metres below existing ground surface (mbgs). 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 CME55 track mounted drill rig which was equipped with standard soil sampling equipment. Soil samples were collected at 0.76 and 1.52 m intervals using a 51 mm outside diameter (OD) split spoon barrel in conjunction with Standard Penetration Tests (SPT) “N” values (ASTM D1586). The SPT “N” values were used to assess the compactness condition of the non-cohesive soil. Approximate shear strengths of the cohesive deposits were measured using the field vane shear test (ASTM D2573), as well as with a handheld pocket penetrometer. The shear strengths measured are plotted on the appended borehole logs.

A monitoring well was installed in Borehole BH1 to allow measurement of groundwater levels. The location of Borehole BH1 coincides with the location of the proposed basement level . The monitoring well

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was constructed using flush-threaded 50 mm diameter Trilock pipe with 3.0 meter long 10-slot well screens, delivered to the Site in pre-cleaned individually sealed plastic bags. The screen and riser pipes were not allowed to come into contact with the ground or drilling equipment prior to installation.

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

Groundwater observations and measurements were obtained from the open boreholes during and upon completion of drilling. The groundwater level was measured in the monitoring well on September 22, 2022. 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 nut on the fire hydrant along the east side of Doctor Leach Drive, at the approximate location shown on Figure 2; and
- Elevation: 100.00 m (Local Datum)

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

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

Select soil samples collected from the boreholes were submitted to a material testing laboratory to determine the grain size distribution and moisture content of the soil. A copy of the laboratory analytical

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reports is included in Appendix III. In addition, the collected samples were compared against previous geotechnical information from the area, for consistency and calibration of results.

4.0 SUBSURFACE CONDITIONS

4.1 Borehole Soil Stratigraphy

In general, the soil stratigraphy at the Site comprises surficial organics overlying granular fill, natural silt and clay, silt, and glacial till to the maximum borehole termination depth of approximately 7.8 mbgs. Two of the boreholes were terminated on probable bedrock at 6.5 to 7.8 mbgs. The appended borehole logs provide detailed soil descriptions and stratigraphies, results of SPT, shear vane and pocket penetrometer testing, details of monitoring well installations, and groundwater measurements.

Surficial organics were encountered in all boreholes and were measured to range in thickness from approximately 100 to 150 millimetres (mm).

Granular fill was encountered underlying the surficial organics in Boreholes BH3 and BH4 and was measured to range in thickness from approximately 0.7 to 1.4 m. The granular fill typically comprised sand containing trace gravel that was brown and damp at the time of sampling. The fill material had a loose to compact relative density based on SPT ‘N’ values ranging from 6 to 17 blows per 300 mm penetration of a split spoon sampler.

Silt and clay was encountered underlying the surficial organics in Boreholes BH1 and BH2 and was noted to extend to between approximately 3.0 and 4.6 mbgs. The silt and clay material typically contained trace sand and was brown to grey in colour. The material had a stiff to very stiff consistency based on shear strengths measured with a handheld pocket penetrometer typically ranging between 50 and 112.5 kPa. The results of one particle size distribution analysis performed on a sample of the material indicates that the sample contained 2% sand, 39% silt and 59% clay. The moisture content of the sample tested was 40.8%, indicating the sample tested was at About the Plastic Limit (APL) at the time of sampling.

The silt deposit was encountered underlying the granular fill in Boreholes BH3 and BH4 and extended to approximately 4.6 mbgs. The silt material was noted to contain trace gravel, trace sand and trace clay. The material had a stiff to hard consistency based on shear strengths measured with a shear vane and handheld pocket penetrometer of between 87.5 and 220 kPa. The result of one particle size distribution analysis performed on a sample of the material indicates that the sample contained 1% gravel, 2% sand, 90% silt and 7% clay. The moisture content of the sample tested was 27.4%, indicating the material tested was at APL at the time of sampling.

Glacial till was encountered in all Boreholes at depths ranging from approximately 3.0 to 4.6 mbgs and extended to the maximum borehole termination depth of approximately 7.8 mbgs. The glacial till ranged in

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soil matrix from sand and silt, containing some gravel and trace clay, to sand and gravel containing some silt and trace clay. The material had a loose to dense relative density based on SPT 'N' values of 6 to 47 blows per 300 mm penetration of a split spoon sampler. The loose portions of the glacial till were encountered within 1 m of it's upper limit. The results of two particle size distribution analyses performed on samples of the glacial till material indicate that the samples contain 17 to 50% gravel, 36 to 40% sand, 12 to 35% silt and 3 to 9% clay. The moisture content of the samples tested ranged between 7.0 and 7.6% indicating the material was in a moist condition at the time of sampling.

4.2 Bedrock

Auger refusal on probable bedrock was encountered in Boreholes BH1 and BH3 between approximately 6.6 and 7.8 mbgs. Bedrock was not encountered in Boreholes BH2 and BH4. It is noted that no bedrock cores were advanced to confirm the presence of bedrock or to evaluate the Rock Quality Designation (RQD).

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. At the completion of drilling, groundwater levels were measured to range between approximately 4.6 and 6.1 mbgs in the open boreholes. On September 22, 2022, groundwater was measured within the monitoring well installed in Borehole BH1 at a depth of approximately 3.6 mbgs.

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.

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Based on information provided by the Client, it is Pinchin's understanding that the development will consist of a two-storey residential apartment building complete with a partial basement level located in the southeast corner of the building. At this time the depth to the underside of the footings for the proposed partial basement level is unknown; as such, for the purpose of this report, Pinchin has assumed a depth of up to approximately 3.0 mbgs to the underside of the footing for the proposed partial basement level. The proposed development will also include new Site Services, and asphalt surfaced access roadways and parking areas.

5.2 Site Preparation

The existing surficial organic material and granular fill is not considered suitable to remain below the proposed building, driveways and parking areas and will need to be removed. In calculating the approximate quantity of topsoil to be stripped, we recommend that the topsoil thicknesses provided on the individual borehole logs be increased by 50 mm to account for variations and some stripping of the mineral soil below.

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)
Structural fill to support foundations and floor slabs	200	100% SPMDD	Plus 2 to minus 4
Subgrade fill beneath parking lots and access roadways	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 fill required below the proposed building comprise imported Ontario Provincial Standards and Specifications (OPSS) 1010 Granular 'B' Type I or II 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.

5.3 Open Cut Excavations and Anticipated Groundwater Management

Excavations for the proposed development will extend upwards of approximately 3.0 mbgs in order to accommodate the partial basement level foundations and new Site services.

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Based on the subsurface information obtained from within the boreholes, it is anticipated that the excavated material will predominately consist of granular fill, silt and clay, and silt materials. Bedrock is not expected to be encountered during excavations for the proposed building foundations or new Site services. The groundwater table was measured within the monitoring well installed in Borehole BH1 at a depth of approximately 3.6 mbgs and is not expected to be encountered during excavations for the proposed development. It is noted that there is potential for water to be locally perched in the granular fill above the less permeable native soils.

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. Excavations through more than one type of soil must be excavated as per the requirements of the highest numbered soil type.

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

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 inflow 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 Services

5.4.1 Pipe Bedding and Cover Materials for Flexible and Rigid Pipes

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

The pipe bedding material should consist of a minimum thickness of 150 mm Granular “A” (OPSS 1010) below the pipe and extend up the sides to the spring line. However, the bedding thickness may have to be increased depending on the pipe diameter or if wet or weak subgrade conditions are encountered. The pipe cover material from the spring line should consist of a Granular “B” Type I (OPSS 1010) and should extend to a minimum of 300 mm above the top of the pipe. All granular fill material is to be placed in maximum 200 mm thick loose lifts compacted to a minimum of 98% SPMDD.

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

Where it is difficult to stabilize the subgrade due to groundwater or the material is higher than the optimum moisture content, a Granular “B” Type II material may be required. Alternatively, if constant groundwater inflow 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.

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5.4.2 Trench Backfill

The trench backfill should be compacted in maximum 300 mm thick lifts to 98% SPMDD within 4% of the optimum moisture content. Based on the observed moisture content of the natural overburden deposits, it may be difficult to achieve the specified density on all of the trench backfill. Nevertheless, it is recommended that the natural soils be used as backfill in the trenches to prevent problems with differential frost heaving of imported subgrade material.

If necessary, compensation for wet trench backfill conditions can be made with additional Granular 'B' in the pavement structure. It should be noted, however, that the wet backfill material must be compacted to at least 90% SPMDD or detrimental post-construction settlements could occur.

Portions of the silt and clay, and silt may have a blocky/lumpy texture. If the large interclump voids are not closed completely by thorough compaction, then long-term softening/settlement will occur. The trench backfill should be placed in thin lifts (less than 200 mm) and compacted with a sheepsfoot roller. Particular attention must be made to backfilling service connections where the trenches are narrow.

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 project 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., silt and clay or silt) 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).

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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 Manotick, 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 Discussion

It is typical construction practice to provide foundation frost protection with soil cover. For the Manotick area, foundations should be provided with a minimum of 1.8 m of soil cover frost protection above the underside of the footing.

As previously mentioned, the depth to the underside of the footing for the partial basement level is unknown. As such, for the purpose of this report, Pinchin has assumed a depth of up to approximately 3.0 mbgs to the underside of the footings for the partial basement level.

Based on the information obtained from the boreholes advanced at the Site, Pinchin recommends that the slab-on-grade portion of the building be constructed on conventional shallow strip and spread footings established on the natural silt and clay and/or natural silt encountered approximately 1.8 mbgs. The partial basement portion of the building may be constructed on conventional shallow strip and spread footings established on the natural silt and clay, natural silt, and/or glacial till encountered approximately 3.0 mbgs. There is potential that interior footings within the slab-on-grade portion of the building would be founded on engineered fill.

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5.5.2 Shallow Foundations Bearing on Natural Silt and Clay, Silt and Glacial Till

Conventional shallow strip footings established on engineered fill placed in accordance with the recommendations in Section 5.2 of this report, or the natural silt and clay, silt and/or glacial till materials may be designed using a bearing resistance for 25 mm of settlement at Serviceability Limit States (SLS) of 125 kPa, and a factored geotechnical bearing resistance of 175 kPa at Ultimate Limit States (ULS) design.

As the actual service loads were not known at the time of this report, these should be reviewed by the project structural engineer to determine if SLS or ULS governs the footing design.

It is noted that there is a potential for weaker subgrade soil to be encountered between the investigation locations. Pinchin presumes that any areas of weaker subgrade soil will consist of small pockets of soft/loose natural soil which can be compacted to match the density of the remainder of the Site. As such, the material must be compacted to a minimum of 100% Standard Proctor Maximum Dry Density (SPMDD) prior to installing the concrete formwork. Any soft/loose areas which are not able to achieve the recommended 100% SPMDD are to be removed and replaced with a low strength concrete.

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

The natural subgrade soil is sensitive to change in moisture content and can become loose/soft if subjected to additional water or precipitation. As well, it could be easily disturbed if travelled on during construction. Once it becomes disturbed it is no longer considered adequate to support the recommended design bearing pressures.

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

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

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- The subgrade should be sloped to a sump outside the excavation to promote surface drainage and the collected water pumped out of the excavation. Any potential precipitation or seepage entering the excavations should be pumped away immediately (not allowed to pond);
- The footing areas should be cleaned of all deleterious materials such as topsoil, organics, fill, disturbed, caved materials or loosened bedrock pieces;
- Any potential large cobbles or boulders (i.e. greater than 200 mm in diameter) within the subgrade material are to be removed and replaced with a similar soil type not containing particles greater than 200 mm in diameter. It is critical that particles greater than 200 mm in diameter are not in contact with the foundation to prevent point loading and overstressing; and
- If the excavated subgrade soil remains open to weather conditions and groundwater seepage, sidewall stability and suitability of the subgrade soil will need to be verified prior to construction.

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

5.5.3 Site Classification for Seismic Site Response & Soil Behaviour

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

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

The boreholes advanced at this Site extended to a maximum depth of approximately 7.8 mbgs where refusal was encountered on probable bedrock. SPT “N” values within the overburden soil deposit ranged between 3 and 47 blows per 300 mm. As such, based on Table 4.1.8.4.A of the OBC, this Site has been classified as Class D. A Site Class D has an average shear wave velocity (Vs) of between 180 and 360 m/s.

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5.5.4 Foundation Transition Zones

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

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

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

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

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

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

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

5.5.5 Estimated Settlement

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

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

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

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5.5.6 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.

5.5.7 Shallow Foundations Frost Protection & Foundation Backfill

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

Where the foundations for heated buildings do not have the minimum 1.8 m of soil cover frost protection, they should be protected from frost with a combination of soil cover and rigid polystyrene insulation, such as Dow Styrofoam or equivalent product. If required, Pinchin can provide appropriate foundation frost protection recommendations as part of the design review.

To minimize potential frost movements from soil frost adhesion, the perimeter foundation backfill should consist of a free draining granular material, such as a Granular ‘B’ Type I (OPSS 1010) or an approved sand fill, extending a minimum lateral distance of 600 mm beyond the foundation. The existing silt and clay, and silt materials are not considered suitable for reuse as foundation wall backfill. Backfill must be brought up evenly on both sides of any walls not designed to resist lateral earth pressure. All granular material is to be placed in maximum 300 mm thick lifts compacted to a minimum of 100% SPMDD below the interior of building or exterior hard landscaping areas; and, 95% SPMDD below exterior soft landscaping areas. It is recommended that inspection and testing be carried out during construction to confirm backfill quality, thickness and to ensure compaction requirements are achieved.

5.6 Partial Basement Level Design

It is understood that a portion of the proposed building will include a basement level, with the underside of the footing presumed to be located up to approximately 3.0 mbgs. As previously mentioned, groundwater was measured in the monitoring well installed at a depth of approximately 3.6 mbgs. As such, Pinchin recommends that foundation drains be provided for the portions of the building which will have the foundation walls exposed on the interior of the building. Pinchin also recommends that these foundations drains be extended around the entire perimeter of the building to ensure proper drainage and to mitigate the potential for water to build up where drains are not installed.

The foundation drains should consist of a minimum 150 mm diameter fabric wrapped perforated drainage tile surrounded by 19 mm diameter clear stone (OPSS 1004) with a minimum cover of 150 mm on top and sides and 50 mm below the drainage tile. Since the natural soil contains a significant amount of silt sized particles, the clear stone gravel should be wrapped in a non-woven geotextile (Terrafix 270R or equivalent). The water collected from the weeping tile should be directed away from the building to appropriate drainage areas; either through gravity flow or interior sump pump systems. All subsurface walls should be waterproofed.

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

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

5.7 Floor Slabs

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

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

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

Based on the in-situ soil conditions, it is recommended to establish the concrete floor slab on a minimum 300 mm thick layer of Granular “A” (OPSS 1010) compacted to 100% SPMDD. Alternatively, consideration may also be given to using a 300 mm thick layer of uniformly compacted 19 mm clear stone placed over the approved subgrade. Any required up fill should consist of a Granular “B” Type I or Type II (OPSS 1010).

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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
Silt and Clay	15,000
Silt	20,000
Engineered Fill	30,000

The values in the table above are for loaded areas of 0.3 m x 0.3 m.

5.8 Asphaltic Concrete Pavement Structure Design for Parking Lot and Driveways

5.8.1 Discussion

Parking areas and driveway access will be constructed around the proposed building. The in-situ natural silt and clay, and natural silt 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 natural silt and clay, and silt materials, the following pavement structure is recommended.

5.8.2 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 or II (OPSS 1010)	100% Standard Proctor Maximum Dry Density (ASTM D698)	300 mm	450 mm

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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 Manotick 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 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.

5.8.3 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.

5.8.4 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 silt and silt and clay soils have poor natural drainage and therefore it is recommended that pavement subdrains be installed in the lower areas and be connected to the catch basins. Pavement subdrains should comprise 150 mm diameter perforated pipe in filter sock, bedded in concrete sand. The top of the concrete sand bedding should be at the bottom 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.

6.0 SOIL CORROSIVITY AND SULPHATE ATTACK ON CONCRETE

A soil sample from Borehole BH3 was submitted to assess the corrosivity of the soil and potential for sulphate attack on concrete. The assessment was completed using the 10-point soil evaluation procedure, provided in the Appendix to the American Water Work Association A21.5 Standard, as recommended by the Ductile Iron Pipe Research Association (DIPRA). The soil samples were evaluated for the following parameters: soil resistivity, pH, redox potential, sulfides, and moisture. Each parameter is assessed and assigned a point value, and the points are totalled. If the total is equal or greater than 10, the soil is considered corrosive to ductile iron pipe. In this case, protective measure must be undertaken. The following table summarizes the 10-point soil evaluation for the tested samples:

Borehole and Sample No.	Resistivity (ohm-cm)	Points	pH	Points	Redox Potential(mv)	Points	Sulfides	Points	Moisture	Points	Total Points
BH3 @ 1.5-2.1 m	6930	0	7.24	0	317	0	Trace	2	Poor drainage, continuously wet	2	4

In summary, the tested sample indicates a low potential for soil corrosivity, and additional protective measures are not required.

The results of the sulphate testing indicate that the Site soils do not produce a moderate or more severe degree of exposure to sulphate attack. As such no special precautions are required at the Site. The results should be reviewed by the structural engineer to ensure conformance to the concrete exposures.

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7.0 SITE SUPERVISION & QUALITY CONTROL

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

8.0 TERMS AND LIMITATIONS

This Geotechnical Investigation was performed for the exclusive use of Rideau Non-Profit Housing Inc. (Client) in order to evaluate the subsurface conditions at 5581 Doctor Leach Drive, Manotick, 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

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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.

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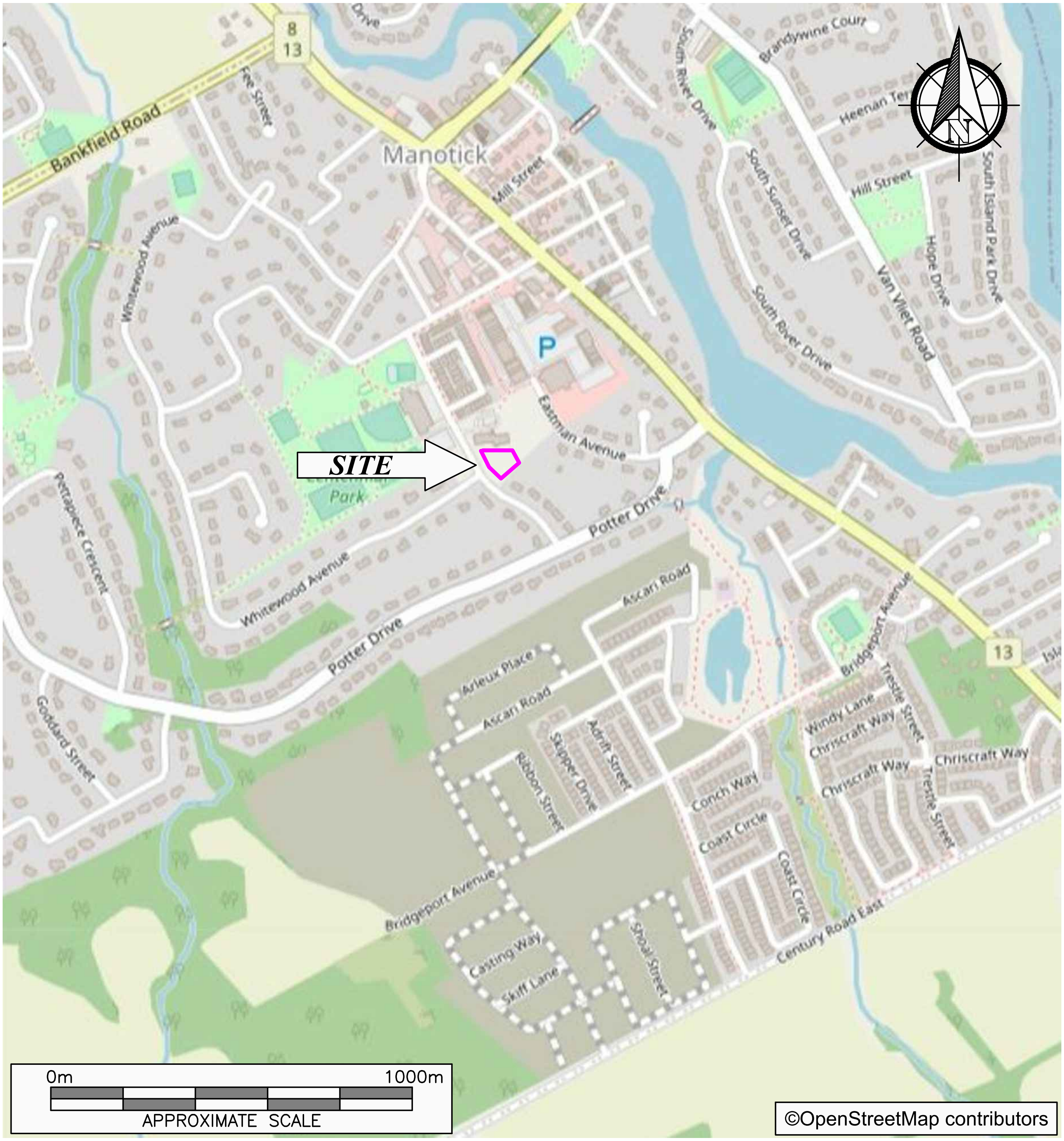
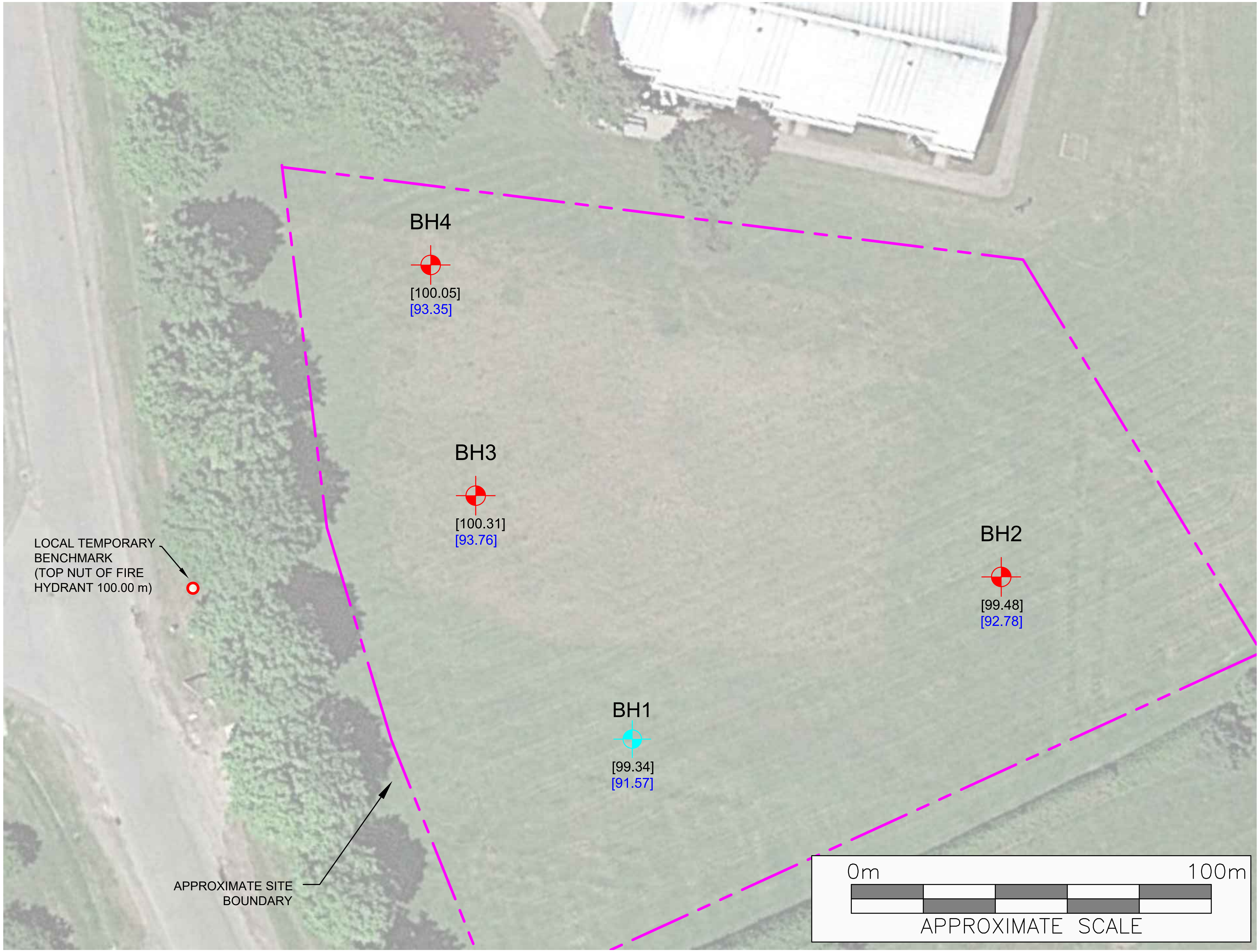
Template: Master Geotechnical Investigation Report – Ontario, GEO, September 2, 2021

FIGURES



PINCHIN

PROJECT NAME: GEOTECHNICAL INVESTIGATION		
CLIENT NAME: RIDEAU NON-PROFIT HOUSING INC.		
PROJECT LOCATION: 5581 DOCTOR LEACH DRIVE, MANOTICK, ONTARIO		
FIGURE NAME: KEY MAP		FIGURE NO. 1
APPROXIMATE SCALE: AS SHOWN	PROJECT NO. 306391.001	DATE: OCTOBER 2022



LEGEND

BOREHOLE LOCATION

MONITORING WELL LOCATION

[XX.XX] APPROXIMATE LOCAL GROUND ELEVATION (m

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

PINCHIN

PROJECT NAME GEOTECHNICAL INVESTIGATION	
CLIENT NAME RIDEAU NON-PROFIT HOUSING INC.	
PROJECT LOCATION 5581 DOCTOR LEACH DRIVE, MANOTICK, ONTARIO	
FIGURE NAME BOREHOLE/MONITORING WELL LOCATION PLAN	
APPROXIMATE SCALE AS SHOWN	PROJECT NO. 306391.001
DATE OCTOBER 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 #: 306391.001 **Logged By:** MK

Project: Geotechnical Investigation

Client: Rideau Non-Profit Housing Inc.

Location: 5581 Doctor Leach Drive, Ottawa, ON

Drill Date: August 25, 2022 **Project Manager:** WT

SUBSURFACE PROFILE | **SAMPLE**

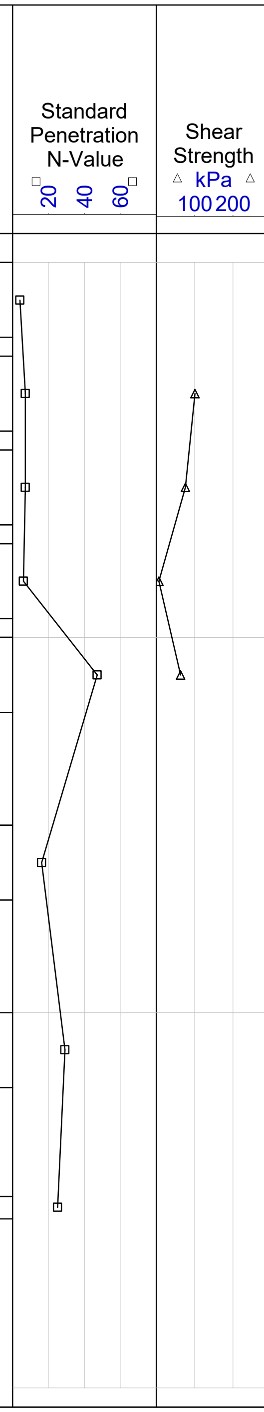
Depth (m)	Description	Elevation (m)
0	Ground Surface	99.34
0 – 0.15	**Organics** ~ 150 mm	
0.15 – 3.05	**Silt and Clay** Silt and clay, trace sand, brown, DTPL, stiff to very stiff	96.29
3.05 – 4.57	**Glacial Till** Sand and silt, some gravel, trace clay, brown, moist, dense	94.77
4.57 – 7.77	Wet, compact	91.57
7.77	End of Borehole	

Borehole terminated at 7.77 mbgs due to auger refusal on probable bedrock.

Monitoring Well Details: Riser, Bentonite, Silica Sand, Screen

Groundwater level = 3.64 mbgs, as measured on September 22, 2022.

Sample Type	Sampler #	Recovery (%)	SPT N-Value	Water Content (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
SS	1	70	4				
SS	2	90	7				
SS	3	100	7				
SS	4	100	6				
SS	5	90	47				
SS	6	40	16				
SS	7	70	29	7.6			Hyd.
SS	8	10	25				



Contractor: Canadian Environmental Drilling & Contractors Inc. **Grade Elevation:** 99.34 m

Drilling Method: Hollow Stem Augers/Split Spoons **Top of Casing Elevation:** 100.30 m

Well Casing Size: 55 mm **Sheet:** 1 of 1

Log of Borehole: BH2

Project #: 306391.001 **Logged By:** MK

Project: Geotechnical Investigation

Client: Rideau Non-Profit Housing Inc.

Location: 5581 Doctor Leach Drive, Ottawa, ON

Drill Date: August 25, 2022 **Project Manager:** WT

Depth (m)	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-Value	Water Content (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
0	Ground Surface	99.48	No Monitoring Well Installed								
	Organics ~ 100 mm			SS	1	60	5				
	Silt and Clay Silt and clay, trace sand, brown, DTPL to APL, stiff to very stiff			SS	2	80	9				
				SS	3	100	7	40.8			Hyd.
				SS	4	100	4				
		94.91		SS	5	100	3				
	Glacial Till Sand and silt, some gravel, trace clay, brown, wet, loose to compact			SS	6	70	6				
		92.78		SS	7	40	23	7.0			Hyd.
	End of Borehole										

Standard Penetration N-Value (20, 40, 60); Shear Strength kPa (100, 200)

Borehole terminated in glacial till at 6.71 mbgs. At drilling completion, groundwater was encountered at 4.88 mbgs.

Contractor: Canadian Environmental Drilling & Contractors Inc. **Grade Elevation:** 99.48 m

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

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

Log of Borehole: BH3

Project #: 306391.001 ***Logged By:*** MK

Project: Geotechnical Investigation

Client: Rideau Non-Profit Housing Inc.

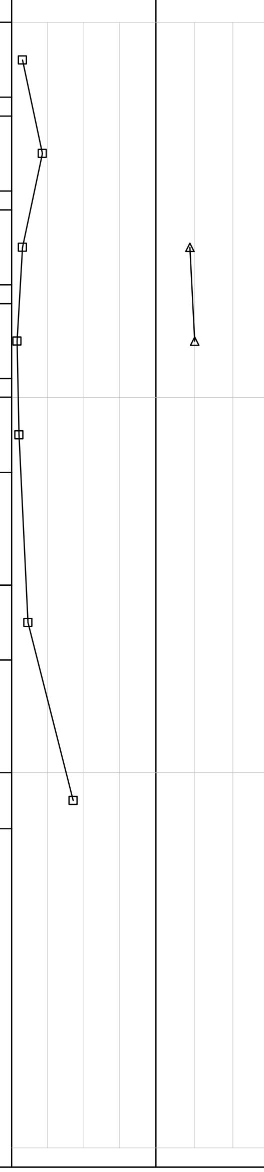
Location: 5581 Doctor Leach Drive, Ottawa, ON

Drill Date: August 25, 2022 ***Project Manager:*** WT

SUBSURFACE PROFILE / **SAMPLE**

Depth (m)	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-Value	Water Content (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
0	Ground Surface	100.31	No Monitoring Well Installed								
0	**Organics** ~ 100 mm			SS	1	60	6				
	Granular Fill Sand, trace gravel, brown, damp, loose to compact	98.79		SS	2	50	17				
	Silt Silt, trace gravel, trace sand, trace clay, brown, DTPL to APL, stiff to hard			SS	3	100	6				
				SS	4	100	3				
		95.74		SS	5	100	4	27.4			Hyd.
	Glacial Till Sand and silt, some gravel, trace clay, brown, wet, loose to dense			SS	6	20	9				
		93.76		SS	7	30	34				
	End of Borehole										

Standard Penetration N-Value (20 40 60) / Shear Strength kPa (100 200)



Borehole terminated at 6.55 mbgs due to auger refusal on probable bedrock. At drilling completion, groundwater was encountered at 4.57 mbgs.

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

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

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

Log of Borehole: BH4

Project #: 306391.001 **Logged By:** MK

Project: Geotechnical Investigation

Client: Rideau Non-Profit Housing Inc.

Location: 5581 Doctor Leach Drive, Ottawa, ON

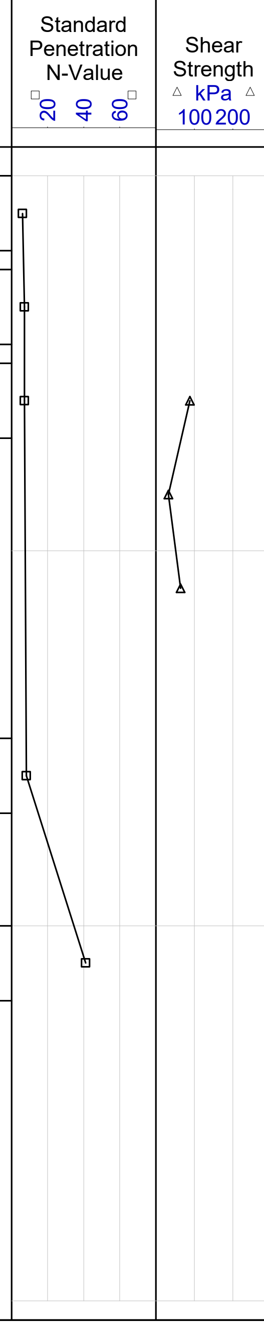
Drill Date: August 25, 2022 **Project Manager:** WT

SUBSURFACE PROFILE

Depth (m)	Description	Elevation (m)	Monitoring Well Details
0	Ground Surface	100.05	No Monitoring Well Installed
0 – 0.1	**Organics** ~ 100 mm		
0.1 – 0.76	**Granular Fill** Sand, trace gravel, brown, damp, loose	99.29	
0.76 – 4.57	**Silt** Silt, trace gravel, trace sand, trace clay, grey to brown, DTPL to APL, stiff to hard	95.48	
4.57 – 6.71	**Glacial Till** Sand and silt, some gravel, trace clay, brown, moist to wet, loose to dense	93.35	
6.71	End of Borehole Borehole terminated at 6.71 mbgs in dense glacial till. At drilling completion, groundwater was encountered at 6.1 mbgs.		

SAMPLE

Sample Type	Sampler #	Recovery (%)	SPT N-Value	Water Content (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
SS	1	60	6				
SS	2	50	7				
SS	3	100	7				
SS	4	100	8				
SS	5	100	41				



Contractor: Canadian Environmental Drilling & Contractors Inc. **Grade Elevation:** 100.05 m

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

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

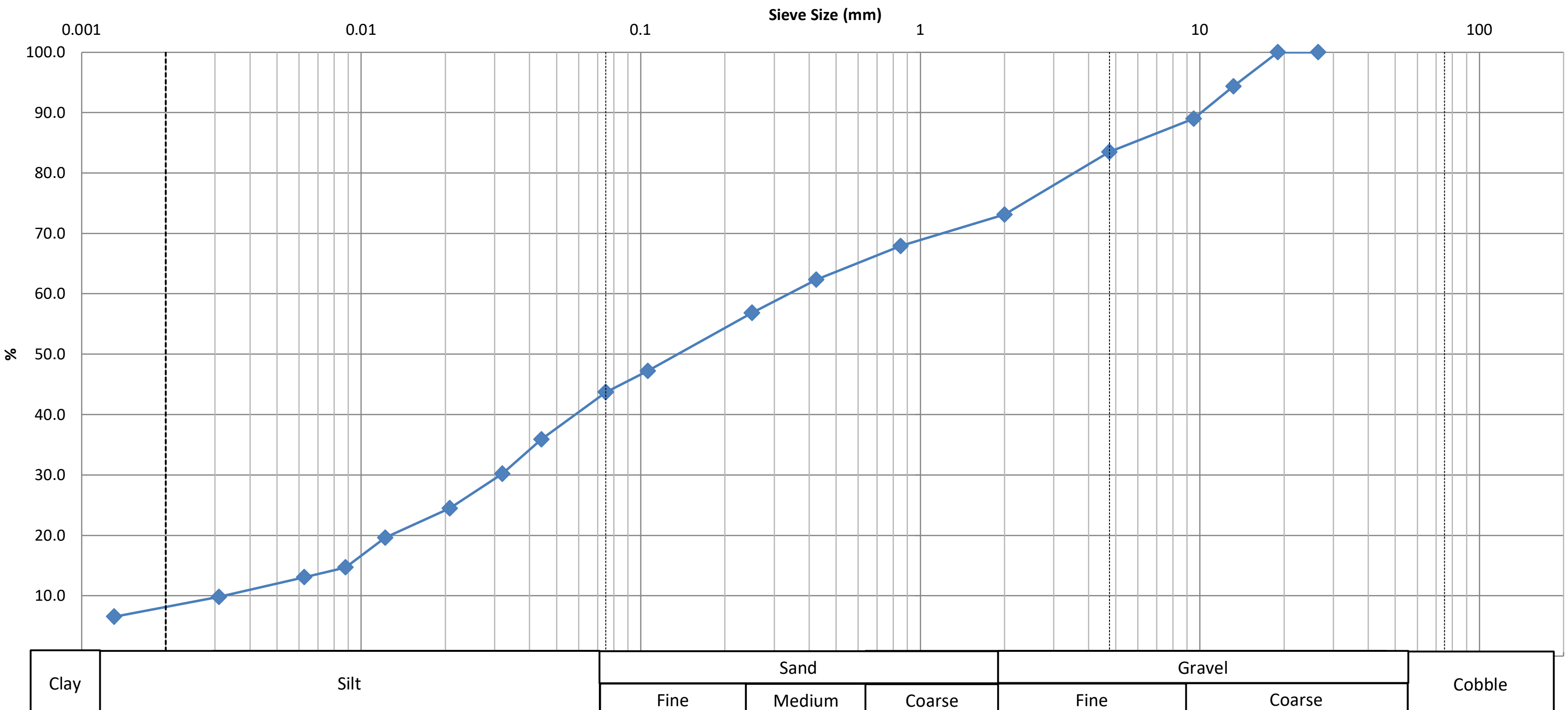
APPENDIX III

Laboratory Testing Reports for Soil Samples

patersongroup
consulting engineers

SIEVE ANALYSIS
ASTM C136

CLIENT:	Pinchin	DEPTH:	20'-22'	FILE NO:	PM4184
CONTRACT NO.:		BH OR TP No.:	BH1	LAB NO:	37745
PROJECT:	306391			DATE RECEIVED:	8-Sep-22
				DATE TESTED:	12-Sep-22
DATE SAMPLED:	-			DATE REPORTED:	15-Sep-22
SAMPLED BY:	Client			TESTED BY:	DK/CS



Identification	Soil Classification				MC(%)	LL	PL	PI	Cc	Cu
					7.6					
	D100	D60	D30	D10	Gravel (%)	Sand (%)	Silt (%)	Clay (%)		
					16.5	39.8	34.7	9.0		

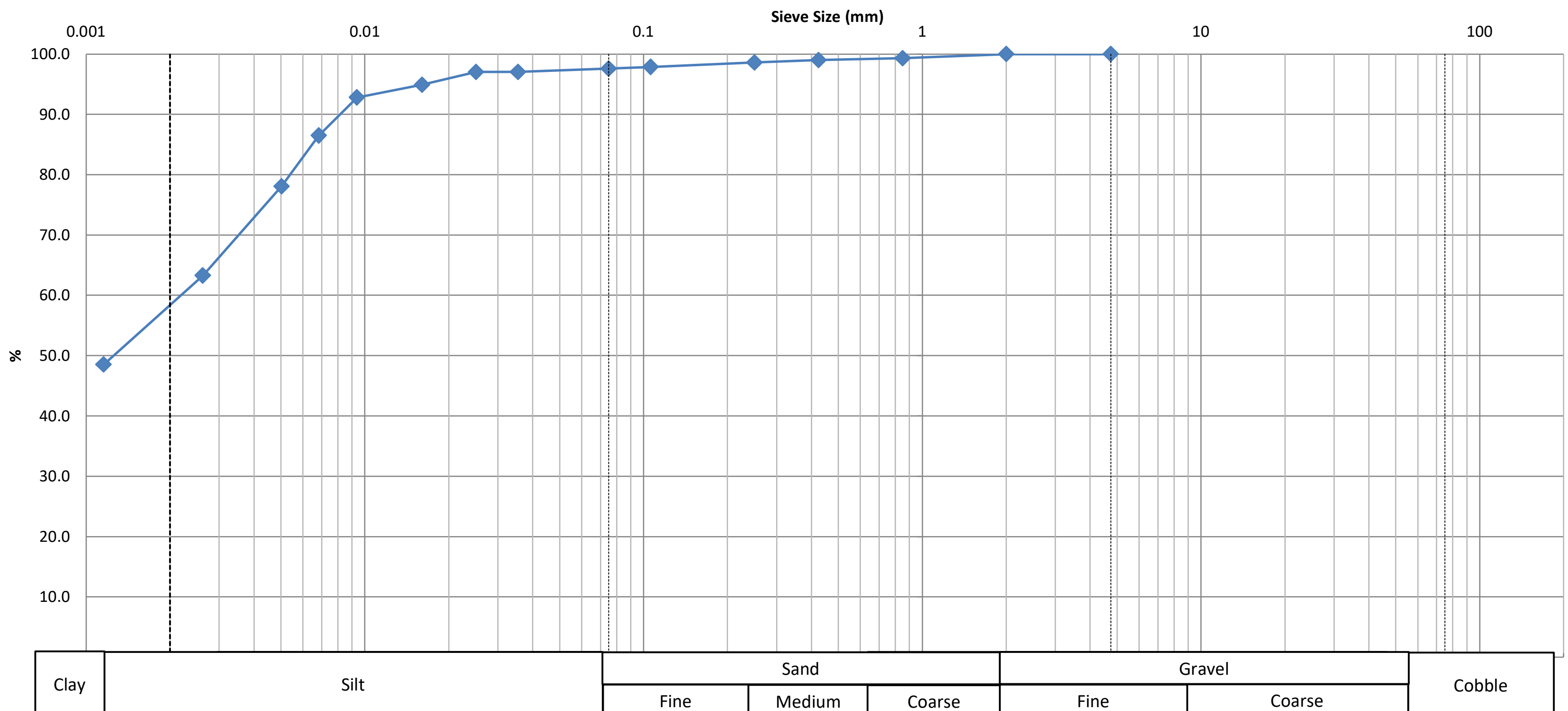
Comments:

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

patersongroup
consulting engineers

SIEVE ANALYSIS
ASTM C136

CLIENT:	Pinchin	DEPTH:	5'-7'	FILE NO:	PM4184
CONTRACT NO.:		BH OR TP No.:	BH2	LAB NO:	37747
PROJECT:	306391			DATE RECEIVED:	8-Sep-22
				DATE TESTED:	12-Sep-22
DATE SAMPLED:	-			DATE REPORTED:	15-Sep-22
SAMPLED BY:	Client			TESTED BY:	DK/CS



Identification	Soil Classification					MC(%)	LL	PL	PI	Cc	Cu
						40.8					
	D100	D60	D30	D10	Gravel (%)	Sand (%)		Silt (%)		Clay (%)	
					0.0	2.4		38.6		59.0	

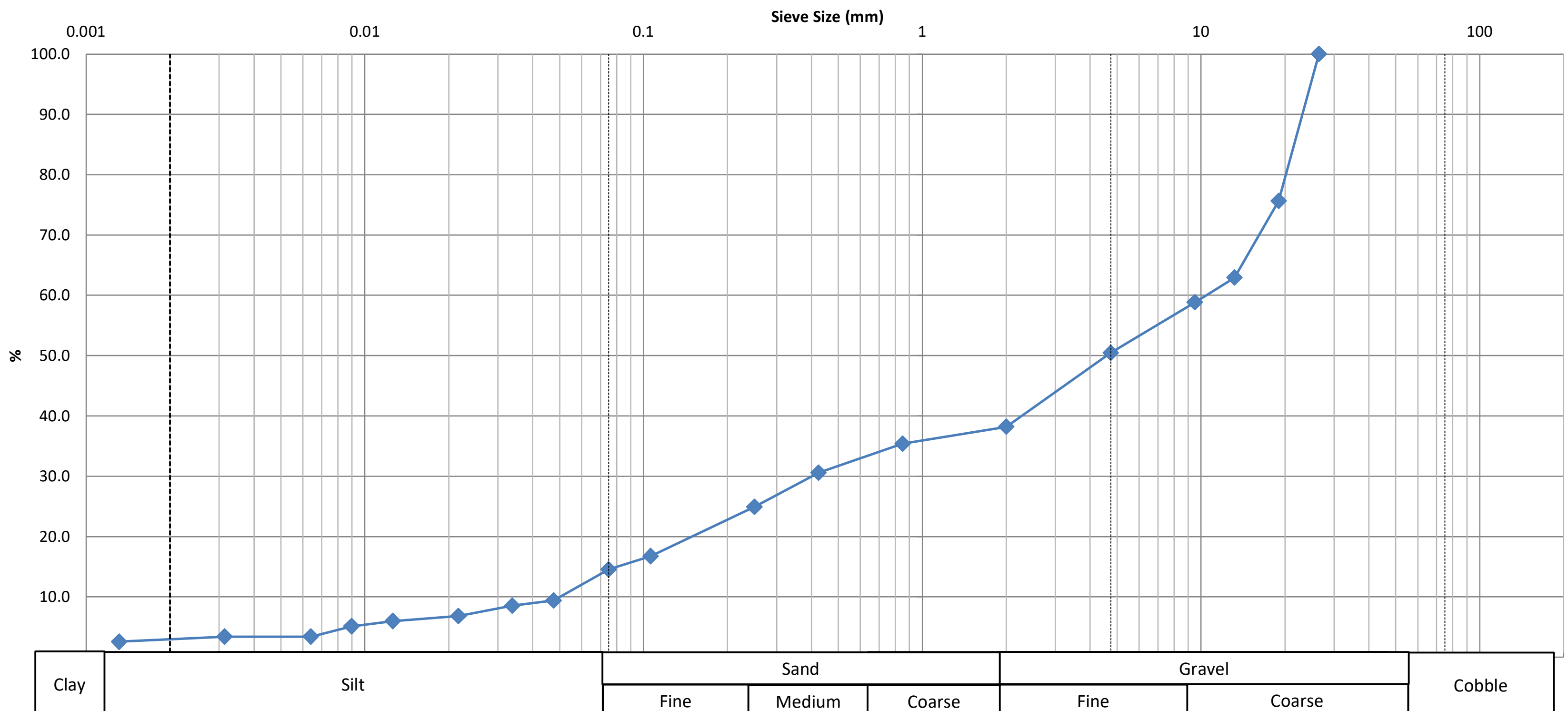
Comments:

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

patersongroup
consulting engineers

SIEVE ANALYSIS
ASTM C136

CLIENT:	Pinchin	DEPTH:	20'-22'	FILE NO:	PM4184
CONTRACT NO.:		BH OR TP No.:	BH2	LAB NO:	37746
PROJECT:	306391			DATE RECEIVED:	8-Sep-22
				DATE TESTED:	12-Sep-22
DATE SAMPLED:	-			DATE REPORTED:	15-Sep-22
SAMPLED BY:	Client			TESTED BY:	DK/CS



Identification	Soil Classification				MC(%)	LL	PL	PI	Cc	Cu
					7.0					
	D100	D60	D30	D10	Gravel (%)	Sand (%)	Silt (%)	Clay (%)		
					49.5	35.9	12.1	2.5		

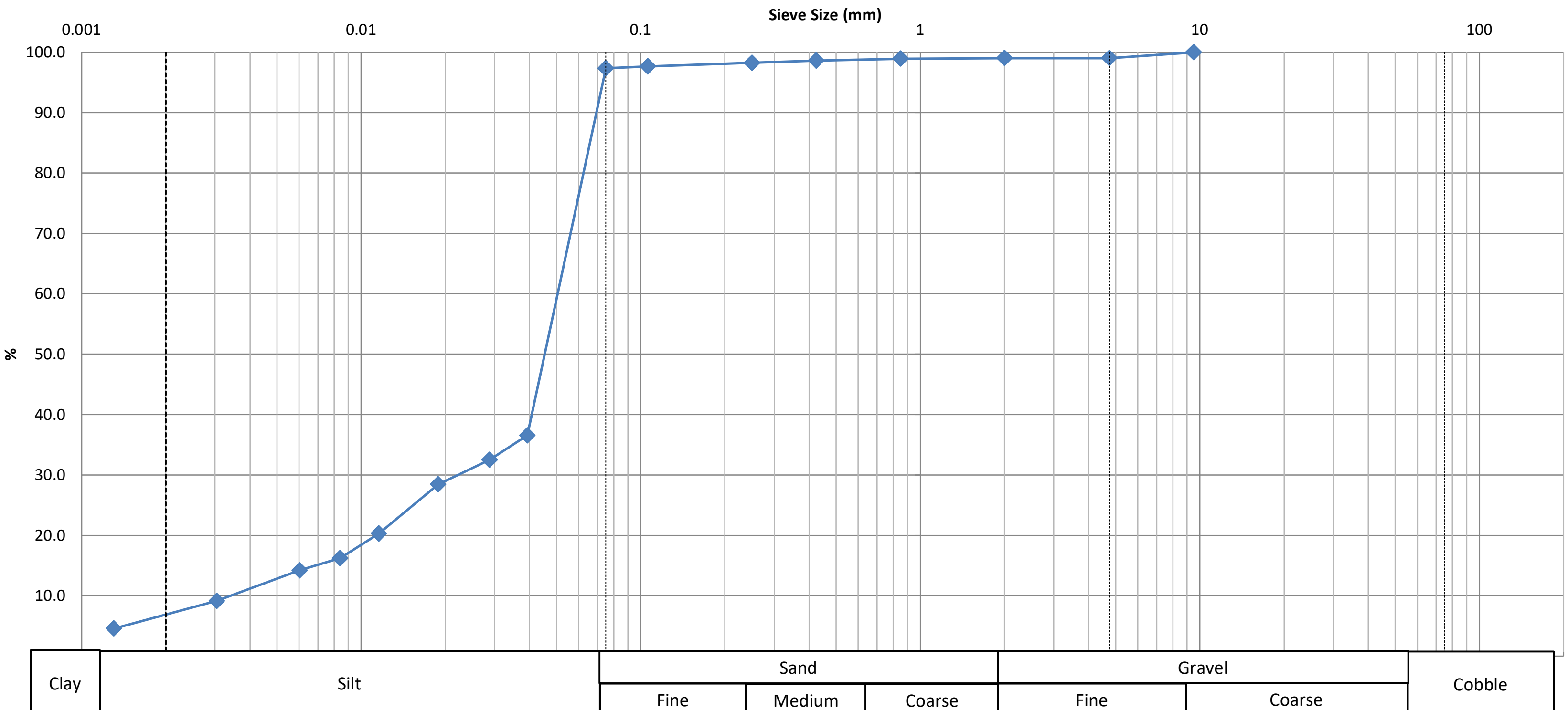
Comments:

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

patersongroup
consulting engineers

SIEVE ANALYSIS
ASTM C136

CLIENT:	Pinchin	DEPTH:	10'-12'	FILE NO:	PM4184
CONTRACT NO.:		BH OR TP No.:	BH3	LAB NO:	37748
PROJECT:	306391			DATE RECEIVED:	8-Sep-22
				DATE TESTED:	12-Sep-22
DATE SAMPLED:	-			DATE REPORTED:	15-Sep-22
SAMPLED BY:	Client			TESTED BY:	DK/CS



Identification	Soil Classification				MC(%)	LL	PL	PI	Cc	Cu
					27.4					
	D100	D60	D30	D10	Gravel (%)	Sand (%)	Silt (%)	Clay (%)		
					1.0	1.7	90.4	7.0		

Comments:

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

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Certificate of Analysis

Pinchin Ltd. (Ottawa)
1 Hines Road, Suite 200
Kanata, ON K2K 3C7
Attn: Megan Keon

Client PO:
Project: 306391
Custody: 136171

Report Date: 6-Sep-2022
Order Date: 29-Aug-2022

Order #: 2236095

This Certificate of Analysis contains analytical data applicable to the following samples as submitted:

Paracel ID	Client ID
2236095-01	BH3 @ 5-7ft.

Approved By:

Milan Ralitsch, PhD
Senior Technical Manager

Certificate of Analysis

Report Date: 06-Sep-2022

Client: **Pinchin Ltd. (Ottawa)**

Order Date: 29-Aug-2022

Client PO:

Project Description: 306391

Analysis Summary Table

Analysis	Method Reference/Description	Extraction Date	Analysis Date
Anions	EPA 300.1 - IC, water extraction	1-Sep-22	1-Sep-22
Conductivity	MOE E3138 - probe @25 °C, water ext	2-Sep-22	2-Sep-22
pH, soil	EPA 150.1 - pH probe @ 25 °C, CaCl buffered ext.	31-Aug-22	1-Sep-22
Resistivity	EPA 120.1 - probe, water extraction	2-Sep-22	2-Sep-22
Solids, %	Gravimetric, calculation	31-Aug-22	1-Sep-22

Summary of Criteria Exceedances

(If this page is blank then there are no exceedances)

Only those criteria that a sample exceeds will be highlighted in red

Regulatory Comparison:

Paracel Laboratories has provided regulatory guidelines on this report for informational purposes only and makes no representations or warranties that the data is accurate or reflects the current regulatory values. The user is advised to consult with the appropriate official regulations to evaluate compliance. Sample results that are highlighted have exceeded the selected regulatory limit. Calculated uncertainty estimations have not been applied for determining regulatory exceedances.

Sample	Analyte	MDL / Units	Result	-	-

Certificate of Analysis

Report Date: 06-Sep-2022

Client: **Pinchin Ltd. (Ottawa)**

Order Date: 29-Aug-2022

Client PO:

Project Description: 306391

	Client ID:	BH3 @ 5-7ft.	-	-	-		
	Sample Date:	29-Aug-22 09:00	-	-	-	-	-
	Sample ID:	2236095-01	-	-	-		
	Matrix:	Soil	-	-	-		
	MDL/Units						
Physical Characteristics							
% Solids	0.1 % by Wt.	69.7	-	-	-	-	-
General Inorganics							
Conductivity	5 uS/cm	144	-	-	-	-	-
pH	0.05 pH Units	7.24	-	-	-	-	-
Resistivity	0.1 Ohm.m	69.3	-	-	-	-	-
Anions							
Chloride	5 ug/g	8	-	-	-	-	-
Sulphate	5 ug/g	47	-	-	-	-	-

Method Quality Control: Blank

Analyte	Result	Reporting Limit	Units	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions								
Chloride	ND	5	ug/g					
Sulphate	ND	5	ug/g					
General Inorganics								
Conductivity	ND	5	uS/cm					
Resistivity	ND	0.10	Ohm.m					

Method Quality Control: Duplicate

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	5.43	5	ug/g	5.63			3.7	20	
Sulphate	53.7	5	ug/g	57.6			7.0	20	
General Inorganics									
Conductivity	144	5	uS/cm	144			0.1	5	
pH	6.95	0.05	pH Units	6.98			0.4	10	
Resistivity	69.3	0.10	Ohm.m	69.3			0.1	20	
Physical Characteristics									
% Solids	91.1	0.1	% by Wt.	96.4			5.7	25	

PARACEL
LABORATORIES LTD.

Order #: 2236095

Certificate of Analysis

Report Date: 06-Sep-2022

Client: **Pinchin Ltd. (Ottawa)**

Order Date: 29-Aug-2022

Client PO:

Project Description: 306391

Method Quality Control: Spike

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	111	5	ug/g	5.63	106	82-118			
Sulphate	154	5	ug/g	57.6	96.3	80-120			

Qualifier Notes:

Sample Data Revisions:

None

Work Order Revisions / Comments:

None

Other Report Notes:

n/a: not applicable

ND: Not Detected

MDL: Method Detection Limit

Source Result: Data used as source for matrix and duplicate samples

%REC: Percent recovery.

RPD: Relative percent difference.

NC: Not Calculated

Soil results are reported on a dry weight basis unlesss otherwise noted.

Where %Solids is reported, moisture loss includes the loss of volatile hydrocarbons.

Any use of these results implies your agreement that our total liabilty in connection with this work, however arising, shall be limited to the amount paid by you for this work, and that our employees or agents shall not under any circumstances be liable to you in connection with this work.

Paracel ID: 2236095

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Paracel Order Number (Lab Use Only)	Chain Of Custody (Lab Use Only)
2236095	Nº 136171

Client Name: Pinchin Ltd.	Project Ref: 306391	Page 1 of 1
Contact Name: Megan Keon	Quote #:	Turnaround Time
Address: 1 Hines Rd Kanata, ON	PO #:	☐ 1 day ☐ 3 day
	E-mail: mKeon@pinchin.com	☐ 2 day ☑ Regular
Telephone: 613-608-5350		Date Required:

☐ REG 153/04 ☐ REG 406/19

☐ Table 1 ☐ Res/Park ☐ Med/Fine
☐ Table 2 ☐ Ind/Comm ☐ Coarse
☐ Table 3 ☐ Agri/Other
☐ Table
For RSC: ☐ Yes ☑ No

Other Regulation
☐ REG 558 ☐ PWQO
☐ CCME ☐ MISA
☐ SU - Sani ☐ SU - Storm
Mun:
☐ Other:

Matrix Type: S (Soil/Sed.) GW (Ground Water) SW (Surface Water) SS (Storm/Sanitary Sewer) P (Paint) A (Air) O (Other)

Required Analysis

	Sample ID/Location Name	Matrix	Air Volume	# of Containers	Sample Taken Date	Sample Taken Time	Corrosivity	Redox	Sulfides	Conductivity								
1	BH3 @ 5-7 ft.	S		1	Aug. 29/22	Am	✓	✓	✓	✓								
2																		
3																		
4																		
5																		
6																		
7																		
8																		
9																		
10																		

Comments:

Method of Delivery: Walk in

Relinquished By (Sign): Megan Keon	Received By Driver/Depot: [signature] 3:40 pm	Received at Lab: [signature]	Verified By: [signature]
Relinquished By (Print): Megan Keon	Date/Time: Aug 29/22	Date/Time: Aug 30 2022 11:00	Date/Time: Aug 30 2022 10:15
Date/Time: Aug. 29/22 AM	Temperature: 17.6 °C	Temperature: 10.3 °C	pH Verified: ☐ By:

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Subcontracted Analysis

Pinchin Ltd. (Ottawa)
1 Hines Road, Suite 200
Kanata, ON K2K 3C7
Attn: Megan Keon

Paracel Report No	**2236095**	Order Date:	29-Aug-22
Client Project(s):	**306391**	Report Date:	8-Sep-22
Client PO:			
Reference:	**Standing Offer - ENV**		
CoC Number:	**136171**		

Sample(s) from this project were subcontracted for the listed parameters. A copy of the subcontractor's report is attached

Paracel ID	Client ID	Analysis
2236095-01	BH3 @ 5-7ft.	Redox potential, soil Sulphide, solid

TESTMARK Laboratories Ltd.
Committed to Quality and Service

CERTIFICATE OF ANALYSIS

Client:	Dale Robertson	Work Order Number:	475312
Company:	Paracel Laboratories Ltd. - Ottawa	PO #:	
Address:	300-2319 St. Laurent Blvd. Ottawa, ON, K1G 4J8	Regulation:	[No Reg - Always Include Reg Report]
		Project #:	2236095
Phone/Fax:	(613) 731-9577 / (613) 731-9064	DWS #:	
Email:	drobertson@paracellabs.com	Sampled By:	
Date Order Received:	8/31/2022	Analysis Started:	9/8/2022
Arrival Temperature:	21.1 °C	Analysis Completed:	9/8/2022

WORK ORDER SUMMARY

ANALYSES WERE PERFORMED ON THE FOLLOWING SAMPLES. THE RESULTS RELATE ONLY TO THE ITEMS TESTED.

Sample Description	Lab ID	Matrix	Type	Comments	Date Collected	Time Collected
BH3 @ 5-7 ft	1794269	Soil	None		8/29/2022	9:00 AM

METHODS AND INSTRUMENTATION

THE FOLLOWING METHODS WERE USED FOR YOUR SAMPLE(S):

Method	Lab	Description	Reference
RedOx - Soil (T06)	Mississauga	Determination of RedOx Potential of Soil	Modified from APHA-2580B

REPORT COMMENTS

Non-Testmark container received TJ 08/31/22
Sample received past hold time for redox potential, proceed with analysis as per client notes TJ 08/31/22

This report has been approved by:

Marc Creighton
Laboratory Director

CERTIFICATE OF ANALYSIS

Paracel Laboratories Ltd. - Ottawa

Work Order Number: 475312

TESTMARK Laboratories Ltd.
Committed to Quality and Service

CERTIFICATE OF ANALYSIS

Paracel Laboratories Ltd. - Ottawa

Work Order Number: 475312

WORK ORDER RESULTS

Sample Description	BH3 @ 5 - 7 ft
Sample Date	8/29/2022 9:00 AM
Lab ID	1794269

General Chemistry	Result	MDL	Units	Criteria: [No Reg - Always Include Reg Report]
RedOx (vs. S.H.E.)	317 [312]	N/A	mV	~

LEGEND

Dates: Dates are formatted as mm/dd/year throughout this report.

MDL: Method detection limit or minimum reporting limit.

[ ]: Results for laboratory replicates are shown in square brackets immediately below the associated sample result for ease of comparison.

~: In a criteria column indicates the criteria is not applicable for the parameter row.

Quality Control: All associated Quality Control data is available on request.

Field Data: Reports containing Field Parameters represent data that has been collected and provided by the client. Testmark is not responsible for the validity of this data which may be used in subsequent calculations.

Sample Condition Deviations: A noted sample condition deviation may affect the validity of the result. Results apply to the sample(s) as received.

Reproduction of Report: Report shall not be reproduced, except in full, without the approval of Testmark Laboratories Ltd.

ICPMS Dustfall Insoluble: The ICPMS Dustfall Insoluble Portion method analyzes only the particulate matter from the Dustfall Sampler which is retained on the analysis filter during the Dustfall method.

Regulation Comparisons: Disclaimer: Please note that regulation criteria are provided for comparative purposes, however the onus on ensuring the validity of this comparison rests with the client.

6820 Kitimat Road Unit 4, Mississauga, ON, L5N 5M3
Phone: (905) 821-1112 Fax: (905) 821-2095 Web: www.testmark.ca

SGS Canada Inc.
P.O. Box 4300 - 185 Concession St.
Lakefield - Ontario - KOL 2HO
Phone: 705-652-2000 FAX: 705-652-6365

08-September-2022

Paracel Laboratories
Attn : Dale Robertson

300-2319 St.Laurent Blvd.
Ottawa, ON
K1G 4K6, Canada

Phone: 613-731-9577
Fax:613-731-9064

Date Rec. : 31 August 2022
LR Report: **CA15691-AUG22**
Reference: Project#: 2236095

Copy: #1

CERTIFICATE OF ANALYSIS

Final Report

Sample ID	**Sample Date & Time**	**Sulphide (Na2CO3) %**
1: Analysis Start Date		07-Sep-22
2: Analysis Start Time		14:36
3: Analysis Completed Date		07-Sep-22
4: Analysis Completed Time		15:34
5: QC - Blank		< 0.04
6: QC - STD % Recovery		94%
7: QC - DUP % RPD		ND
8: RL		0.02
9: BH3 @ 5-7ft	29-Aug-22 09:00	< 0.04

RL - SGS Reporting Limit
ND - Not Detected

Kimberley Didsbury,
Project Specialist,
Environment, Health & Safety

OnLine LIMS

0003040119

Results relate only to the sample tested. Data reported represents the sample submitted to SGS. Reproduction of this analytical report in full or in part is prohibited without prior written approval. Please refer to SGS General Conditions of Services located at https://www.sgs.ca/en/terms-and-conditions (Printed copies are available upon request.)
Test method information available upon request. “Temperature Upon Receipt” is representative of the whole shipment and may not reflect the temperature of individual samples.
SGS Canada Inc. Environment-Health & Safety statement of conformity decision rule does not consider uncertainty when analytical results are compared to a specified standard or regulation.

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.