REVISED

Geotechnical Investigation – Proposed Residential Development

10 - 20 Empress Avenue, Ottawa, Ontario

Prepared for:

Dalhousie Non-Profit Housing Cooperative Inc.
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1.0 INTRODUCTION AND SCOPE

Pinchin Ltd. (Pinchin) was retained by Dalhousie Non-Profit Housing Cooperative Inc. (Client) to conduct a Geotechnical Investigation and provide subsequent geotechnical design recommendations for the proposed residential development to be located at 10 - 20 Empress Avenue, 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 a five-storey residential apartment building complete with a single level basement/underground parking garage that will occupy the majority of the Site footprint. The proposed development will also include new Site services; however, will not include new asphalt surfaced 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 three (3) sampled boreholes (Boreholes BH1 to BH3), at the Site. The information gathered from the Geotechnical Investigation will allow Pinchin to provide geotechnical design recommendations for the proposed development. It is noted that due to the number of buried services located on the east portion of the Site, no boreholes were advanced on the east side of the existing buildings.

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 and 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;
- Basement/Underground parking garage design; 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 west side of Empress Avenue, approximately 50 m south of Albert Street in Ottawa, Ontario. The Site is currently developed with two residential townhouse buildings and a gravel surfaced parking area. The lands adjacent to the Site are predominantly developed with one to two storey residential and commercial 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 Paleozoic terrain consisting of sandy silt to silty sand textured till. (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 Shadow Lake Formation consisting of limestone, dolostone, shale, arkose, 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 July 26, 2023, by advancing a total of three (3) sampled boreholes (Boreholes BH1 to BH3) throughout the Site. The boreholes were advanced to sampled depths ranging from approximately 6.7 to 9.8 metres below existing ground surface (mbgs). Below the sampled depth within Borehole BH2, a Dynamic Cone Penetration Test (DCPT) was advanced to a refusal depth of approximately 14.3 mbgs to further assess the relative density of the subgrade soil with depth, as well as to estimate the approximate depth to 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 and 1.52 m intervals using a 51 mm outside diameter (OD) split spoon barrel in conjunction with Standard Penetration Tests (SPT) “N” values (ASTM D1586). The SPT “N” values were used to assess the compactness condition of the non-cohesive soil.

Monitoring wells were installed in all of the boreholes to allow for measurement of the groundwater levels. The monitoring wells were constructed using flush-threaded 50 mm diameter Trilock pipe with 3.0 meter long 10-slot well screens, delivered to the Site in pre-cleaned individually sealed plastic bags. The screen and riser pipes were not allowed to come into contact with the ground or drilling equipment prior to installation.

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

Groundwater observations and measurements were obtained from the open boreholes during and upon completion of drilling. Groundwater levels were measured in the monitoring wells on August 28, 2023. The groundwater observations and measurements recorded are included on the appended borehole logs.

The borehole locations were located at the Site by Pinchin personnel. The approximate geodetic ground surface elevation at each borehole location was referenced to the nearest survey point from the following topographic survey which was provided by the Client:

- *"Topographic Plan of Survey Lot 5 and Part of Lot 6, Registered Plan 7, City of Ottawa*", prepared by Farley, Smith. & Denis Surveying Ltd., Project No. 476-21, dated October 9, 2021.

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

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

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

4.0 SUBSURFACE CONDITIONS

4.1 Borehole Soil Stratigraphy

In general, the soil stratigraphy at the Site comprises surficial granular fill overlying silty sand/silt and sand, and probable bedrock to the maximum borehole termination depth of approximately 14.3 mbgs. The appended borehole logs provide detailed soil descriptions and stratigraphies, results of SPT and DCPT testing, details of monitoring well installations, and groundwater measurements. It is noted that due to the number of buried services located on the east portion of the Site, no boreholes were advanced on the east side of the existing buildings. As such, Pinchin has assumed the soil conditions on the east portion of the Site are the same/similar as to what was encountered on the west portion of the Site.

Surficial granular fill was encountered in all boreholes and ranged in thickness from approximately 0.5 to 0.8 m. The granular fill typically consisted of sand and gravel containing trace silt that was brown and damp at the time of sampling. The non-cohesive material had a loose to compact relative density based SPT ‘N’ values of 5 to 13 blows per 300 mm penetration of a split spoon sampler.

Silty sand/silt and sand was encountered underlying the surficial granular fill in all boreholes and extended to the maximum sampled borehole depth of approximately 9.8 mbgs. The silty sand/silt and sand typically contained trace to some gravel and trace clay that was brown at the time of sampling. The non-cohesive material had a very loose to compact relative density based SPT ‘N’ values of 0 to 29 blows per 300 mm penetration of a split spoon sampler. The results of three particle size distribution analyses completed on samples of the material indicate that the samples contain 9 to 19% gravel, 48 to 50% sand, 29 to 37% silt, and 4 to 5% clay sized particles.

4.2 Bedrock

DCPT refusal on probable bedrock was encountered in Borehole BH2 at approximately 14.3 mbgs. 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. On August 28, 2023, groundwater was measured within the monitoring wells installed between approximately 2.7 and 4.1 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. Given the site constraints boreholes were only able to be drilled along near Perkins St., additional geotechnical investigation will be required for detailed design. A qualified geotechnical engineer should be on-Site during the foundation preparation to ensure the subsurface conditions are the same/similar to what was observed during the investigation.

Based on information provided by the Client, it is Pinchin's understanding that the proposed development is to consist of a five-storey residential apartment building complete with a single level basement/underground parking garage that will occupy the majority of the Site footprint. The proposed development will also include new Site services; however, will not include asphalt surfaced parking areas. At the time of preparing this report the depth to the underside of the footing for the proposed basement/underground parking garage level is unknown; as such, for the purpose of this report Pinchin has assumed a depth of approximately 3.5 mbgs to the underside of the footing.

5.2 Site Preparation

Prior to Site preparation activities commencing, the existing building structures will need to be demolished and removed from the Site, including all foundations and service pipes.

The existing granular fill is not considered suitable to remain below the proposed building and will also need to be removed. 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

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 to raise grades 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

It is anticipated that excavations for the proposed development will extend upwards of 3.5 mbgs to accommodate the proposed basement/underground parking garage level.

Based on the subsurface information obtained from within the boreholes, it is anticipated that the excavated material will predominately consist of granular fill and silty sand materials. Groundwater was measured to range between approximately 2.7 and 4.1 mbgs within the monitoring wells installed and is expected to be encountered during excavations for the proposed development.

Where workers must enter trench excavations deeper than 1.2 m, the trench excavations should be suitably sloped and/or braced in accordance with the Occupational Health and Safety Act (OHSA), Ontario Regulation 213/91, Construction Projects, July 1, 2011, Part III - Excavations, Section 226. Alternatively, the excavation walls may be supported by either closed shoring, bracing, or trench boxes complying with sections 235 to 239 and 241 under O. Reg. 231/91, s. 234(1). The use of trench boxes can be used for temporary support of vertical side walls.

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

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.

Excavations extending below the groundwater table require a dewatering system installed by a specialist dewatering contractor to lower the groundwater level prior to excavation. The design of the dewatering system should be left to the contractor's discretion, and the system should meet a performance specification to maintain and control the groundwater at least 0.60 m below the excavation base.

Seasonal variations in the water table should be expected, with higher levels occurring during wet weather conditions in the spring and fall and lower levels occurring during dry weather conditions. If construction commences during wet periods (typically spring or fall), there is a greater potential that the groundwater elevation could be higher and/or perched groundwater may be present. Any potential precipitation of perched groundwater should be able to be controlled from pumping from filtered sumps.

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

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

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

5.3.1 Excavation Impacts to Nearby Structures

Pinchin was provided with the following drawings by the Client to analyze to determine the impacts on nearby structures:

- "*10 Empress Avenue – Level 1/Entrance Level Floor Plan and Lower Level Floor Plan*", prepared by Project Studio, dated January 18, 2023, project no. 2214.

Pinchin's review of the above referenced drawings determined that the approximate proposed building limits from the property boundaries are as follows:

- Approximately 3.7 m from the east property boundary;
- Approximately 4.3 m from the west property boundary;
- Approximately 1.5 m from the south property boundary; and
- Approximately 1.8 m from the north property boundary.

Based on the above, the proposed development will fall within 3.0 m of the north and south property lines; as such, the City of Ottawa requires a Geotechnical Engineering Consultant review the proposed development and provide comments on safe excavation practices to mitigate the effects on the neighbouring properties during construction.

The following is a summary of Pinchin's comments and recommendations as they relate to the proposed excavation activities for the development:

- Due to the potential for liquefaction, Pinchin has provided shoring recommendations in Section 5.4;To manage expected precipitation from rainfall/snowfall, tarps should be used to cover the excavation and stockpiled material to prevent erosion. In the event construction is completed during freezing weather conditions, insulated tarps are recommended to prevent the subgrade soil from freezing;
- The excavation activities for the proposed development are not expected to encroach onto the neighbouring properties. Any damage caused by the excavation activities are to be repaired by the Client as soon as reasonably possible unless an informed consent agreement is in place between the property owners;
- No equipment, construction waste/material, or stockpiled soil is to be stored within 2.0 m of the excavation for the life of the project;
- Provided the proposed excavation for 10-20 Empress Avenue is completed in accordance with the Pinchin 2024 Report, no impacts to buried infrastructure, utilities and right of ways are anticipated; and
- The east and west sides of the excavation will have sufficient space for the walls of the excavation to be sloped at an inclination of 1 horizontal to 1 vertical (H to V) from the base of the excavation; however, the north and south sides of the excavation are located closer to the property boundaries and will not allow for an excavation to be sloped at 1H to 1V. As such, the excavation walls on the north and south sides of the excavation may be cut vertical in the bottom 1.2 m and then sloped back at an inclination of 1H to 1V above. Or, as provided in the following section, Pinchin has provided shoring recommendations.

In addition to the above recommendations, Pinchin notes that daily inspections of the excavation should be completed by the general contractor. If the excavation becomes unstable, Pinchin should be contacted to review the area of instability to provide appropriate remedial actions.

5.4 Shoring Requirements

Due to spatial limitations, it will likely not be feasible to slope the excavations back to a safe angle at the Site and therefore some support system may be required.

Temporary protective structures, bracing, anchors, and sheeting are the responsibility of the contractors and shall be designed by a Professional Engineer licensed in Ontario, in accordance with the Canadian Foundation Engineering Manual. All shoring, bracing, sheet-piling and cribbing (where required) shall meet all requirements of the Ontario Occupational Health and Safety Act and Regulations for Construction Projects and the Trench Excavators Protection Act. The shoring design must include appropriate factors of safety and take into account the loading from adjacent existing building's foundations as well as any possible surcharge loading. The support system must comply with sections 234 to 239 and 241 of Ontario Regulation 213/91.

The sections along the perimeter of the proposed building footprint may need be shored to preserve the integrity of the boundary conditions using a shoring system consisting of a combination of soldier piles/lagging or continuous interlocking caisson wall. Considerations should be given to incorporating a rigid shoring system to preserve the integrity and support of the soil in a state approximating at-rest conditions.

5.4.1 Lateral Earth Pressure

The design parameters for structures subject to lateral earth pressures such as basement walls and retaining structures are provided in the table below.

Soil Layer	Bulk Unit Weight, γ (kN/m^3)	Angle of Internal Friction (φ)	At Rest Earth Pressure Coefficient, K_0	Active Earth Pressure Coefficient, K_a	Passive Earth Pressure Coefficient, K_p
Compacted Granular Fill	21	32°	0.31	0.47	3.25
Earth Fill	19	28°	0.53	0.36	2.77
Natural Silty Sand to Silt and Sand	19	34°	0.44	0.28	3,54

The lateral earth pressure acting on basement or shoring walls may be calculated from the following:

$P = K[\gamma(h - h_w) + \gamma' h_w + q] + \gamma_w h_w$

Where:

P	=	Lateral earth pressure at depth (kPa)
h	=	depth (m)
h_w	=	height of groundwater above depth h (m)

γ	=	soil bult unit weight (kN/m^3)
γ'	=	submerged soil unit weight (kN/m^3)
γ_w	=	unit weight of water (kN/m^3)
K	=	earth pressure coefficient”
q	=	total surcharge load (kPa)

If the wall drainage is applied behind the wall such that hydrostatic pressure will be eliminated, the lateral earth pressure can be taken as:

$P = K[\gamma h + q]$

Resistance to sliding of retaining structures is developed by friction between the base of the footing and the soil. This friction (**R**) depends on the normal load on the soil contact (**N**) and the frictional resistance of the soil (**tan δ**) expressed as **R = N tan δ**. The friction factor (δ) as indicted on Table 24.4 of the Canadian Engineering Foundation Manual can be taken as 0.4. The factored geotechnical resistance at ULS is **0.8 R**.

Passive earth pressure resistance is generally not considered as a resisting force against sliding for conventional retaining structure design because a structure must deflect significantly to develop the full passive resistance.

The following parameters (un-factored) should be used for the design of the shoring system. It should be noted that these earth pressure coefficients assume that the back of the wall is vertical; condition of the ground surface behind the wall is assumed to be flat.

If a water-tight shoring system is proposed, the shoring system must also be designed to resist that lateral hydrostatic pressure.

If shoring is adjacent to existing buildings to remain, then the shoring must also be designed to resist the pressures produced by those buildings’ foundations and ensure that there is no movement of retained soil that would cause settlement of those buildings.

If construction proceeds in winter months, the shoring system may require frost protection to prevent frost penetration behind the shoring system, which can result in unacceptable movements.

It is recommended that the contract have a performance specification, limiting movement. The presence of sensitive structures and infrastructure, anchor spacing, elevation, and the timing of the excavation and anchoring operations are critical in determining acceptable limits. A monitoring program for shored excavations is recommended.

5.5 Site Services

5.5.1 Pipe Bedding and Cover Materials for Flexible and Rigid Pipes

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

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

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

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

5.5.2 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.

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

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

5.5.3 Frost Protection

The frost penetration depth in Ottawa, Ontario is estimated to extend to approximately 1.8 mbgs in open roadways cleared of snow. As such, it is recommended to place water services at a minimum depth of 300 mm below this elevation with the top of the pipe located at 2.1 mbgs or lower as dictated by municipal service requirements. If a minimum of 2.1 m of soil cover cannot be provided, then the pipe should be insulated with a rigid polystyrene insulation (DOW Styrofoam HI40, or equivalent) or a pre-insulated pipe be utilized.

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

5.6 Foundation Design

5.6.1 Shallow Foundations Bearing on Silty Sand or Engineered Fill

Conventional shallow strip footings established on the silty sand material encountered approximately 3.5 mbgs, with a maximum size of 3.5m by 3.5m, or a properly compacted engineered fill, may be designed using a bearing resistance for 25 mm of settlement at Serviceability Limit States (SLS) of 100 kPa, and a factored geotechnical bearing resistance of 150 kPa at Ultimate Limit States (ULS).

It is noted that in order to obtain the above bearing resistances, the groundwater level is to be lowered to a minimum of 0.6 m below the base of the excavation as per Section 5.3 of this report. Once the groundwater is lowered, the natural subgrade soil is to be compacted to a minimum of 100% Standard Proctor Maximum Dry Density (SPMDD) prior to installing the concrete formwork. Any soft/loose areas which are not able to achieve the recommended 100% SPMDD are to be removed and replaced with an engineered fill.

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;
- 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.

5.6.2 Helical Piles (Screw Piles) Founded in Natural Silt and Silty Clay Materials

Pinchin notes that should higher bearing resistances be required, deep foundations consisting of helical piles (screw piles) founded within the natural silty sand/silt and sand may be utilized to support the proposed building. Helical piles provide the least amount of disturbance as they are driven into the underlying soil utilizing a helix to advance through the soil matrix. The supporting grade beam system for the structure would bear upon the helical piles.

The number and size of helical piles are determined based on the building loads and configuration. Since helical piles are a proprietary system, it is recommended that the piles be designed by an experienced design build contractor in conjunction with the soil characteristics provided by Pinchin. For the natural subgrade soil encountered within the boreholes advanced, the following strength characteristics are to be used for the pile design:

Soil Type	Bulk Unit Weight (kN/m^3)	Friction Angle (°)	Cohesion (kPa)
Silty sand/Silt and sand	18.5	28	0

To provide frost protection, Pinchin also recommends that the helical piles be lined with plastic sleeves or be epoxy coated galvanized steel to protect against corrosion.

5.6.3 Ground Improvement

As an alternative to deep foundations, the Site is also suitable for ground improvement methods such as the following:

- Controlled Modulus Columns (CMC); and

- Rapid Impact Compaction

Ground improvement involves modifying the engineering properties of soils to increase bearing capacity and provide added stability. The result of the above ground improvement techniques is a significant strengthening and stiffening of subsurface soils that then support conventional shallow foundations. The above ground improvement techniques are proprietary in design and will require input from specialized contractors and engineers. Whichever technique is selected, the installation/fieldwork should be monitored on a full-time basis by a qualified geotechnical consultant.

5.6.4 Foundation Transition Zones

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

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

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

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

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

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

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 2 H to 1 V 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.

5.6.5 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.

5.6.6 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.

5.6.7 Shallow Foundations Frost Protection & Foundation Backfill

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

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

To minimize potential frost movements from soil frost adhesion, the perimeter foundation backfill should consist of a free draining granular material, such as a Granular 'B' Type I (OPSS 1010) or an approved sand fill, extending a minimum lateral distance of 600 mm beyond the foundation. The existing silty sand material contains to many silt sized particles and is not considered suitable for reuse as foundation wall backfill. Backfill must be brought up evenly on both sides of 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 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.

5.7 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 sampled depths ranging between approximately 6.7 and 9.8 mbgs. SPT "N" values within the soil deposit ranged between 0 and 29 blows per 300 mm. In addition, DCPT refusal was encountered on probable bedrock at approximately 14.3 mbgs. 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.

Given the SPT 'N' values measured in the cohesionless soil, there is potential for the cohesionless deposits to be liquifiable. In order to properly assess liquefaction potential an additional investigation is required. Should theses soils be deemed liquifiable, the site would be classified as Class F; however, it may be possible to densify the soils utilizing ground improvement to eliminate the potential for liquefaction.

5.8 Basement Level/Underground Parking Garage Design

It is understood that the building is proposed to include a single level basement/underground parking garage, and Pinchin has assumed a depth of approximately 3.5 mbgs to the underside of the footings. As previously mentioned, on August 28, 2023, groundwater was measured within the monitoring wells installed between approximately 2.7 and 4.1 mbgs.

As such, depending on the proposed final grades, there is a potential for the building to have to be designed to either resist hydrostatic uplift or to be provided with underfloor and foundation wall drainage systems connected to a suitable frost-free outlet due to the groundwater levels at the Site.

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

$$P = \gamma \times d$$

Where:

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

γ = unit weight of water (9.8 kN/m^3)

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

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

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

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

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 silt soil. The bulk unit weight of the retained backfill may be taken as 20 kN/m^3 for well compacted soil.

5.9 Floor Slabs

The in-situ silty sand material encountered within the boreholes is considered adequate for the support of the concrete floor slabs provided it is proof roll compacted with a minimum 10 tonne non-vibratory steel drum roller to observe for weak/soft spots. It is noted that 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. 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). 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).

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

Material Type	Modulus of Subgrade Reaction (kN/m^3)
Granular “B” Type I (OPSS 1010)	75,000
Granular “B” Type II (OPSS 1010)	85,000
Silty Sand	20,000

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

6.0 SOIL CORROSIVITY AND SULPHATE ATTACK ON CONCRETE

A soil sample from Borehole BH2 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 is the case, protective measures need to be undertaken. The following table summarizes the 10-point soil evaluation for the tested sample:

Borehole and Sample No.	Resistivity (ohm-cm)	Points	pH	Points	Redox Potential (mv)	Points	Sulfides	Points	Moisture	Points	Total Points
BH2 SS4 @ 7.5 -9.5 ft	3740	0	7.96	0	393	0	Trace	2	Fair drainage, generally moist	1	3

In summary, the tested sample indicates a low potential for soil corrosivity, and additional protective measures are not required. The results of the testing indicate that the Site possesses low sulphate exposure and moderate chloride exposure. The exterior walls and footings will be subjected to freeze and thaw cycles while the interior columns will not be subjected to these cycles. The selected type of concrete should meet these requirements. The results should be reviewed by the structural engineer to ensure conformance to the concrete exposures.

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 Dalhousie Non-Profit Housing Cooperative Inc. (Client) in order to evaluate the subsurface conditions at 10 - 20 Empress Avenue, Ottawa, Ontario. Within the limitations of scope, schedule and budget, our services have been executed in accordance with generally accepted practises in the field of geotechnical engineering for the Site. Classification and identification of soil, and geologic units have been based upon commonly accepted methods employed in professional geotechnical practice. No warranty or other conditions, expressed or implied, should be understood. Conclusions derived are specific to the immediate area of study and cannot be extrapolated extensively away from sample locations.

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

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

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

The liability of Pinchin or our officers, directors, shareholders, or staff will be limited to the lesser of the fees paid or actual damages incurred by the Client. Pinchin will not be responsible for any consequential or indirect damages. Pinchin will only be liable for damages resulting from the negligence of Pinchin. Pinchin will not be liable for any losses or damage if the Client has failed, within a period of two years following the date upon which the claim is discovered (Claim Period), to commence legal proceedings against Pinchin to recover such losses or damage unless the laws of the jurisdiction which governs the Claim Period which is applicable to such claim provides that the applicable Claim Period is greater than two years and cannot be abridged by the contract between the Client and Pinchin, in which case the Claim Period shall be deemed to be extended by the shortest additional period which results in this provision being legally enforceable.

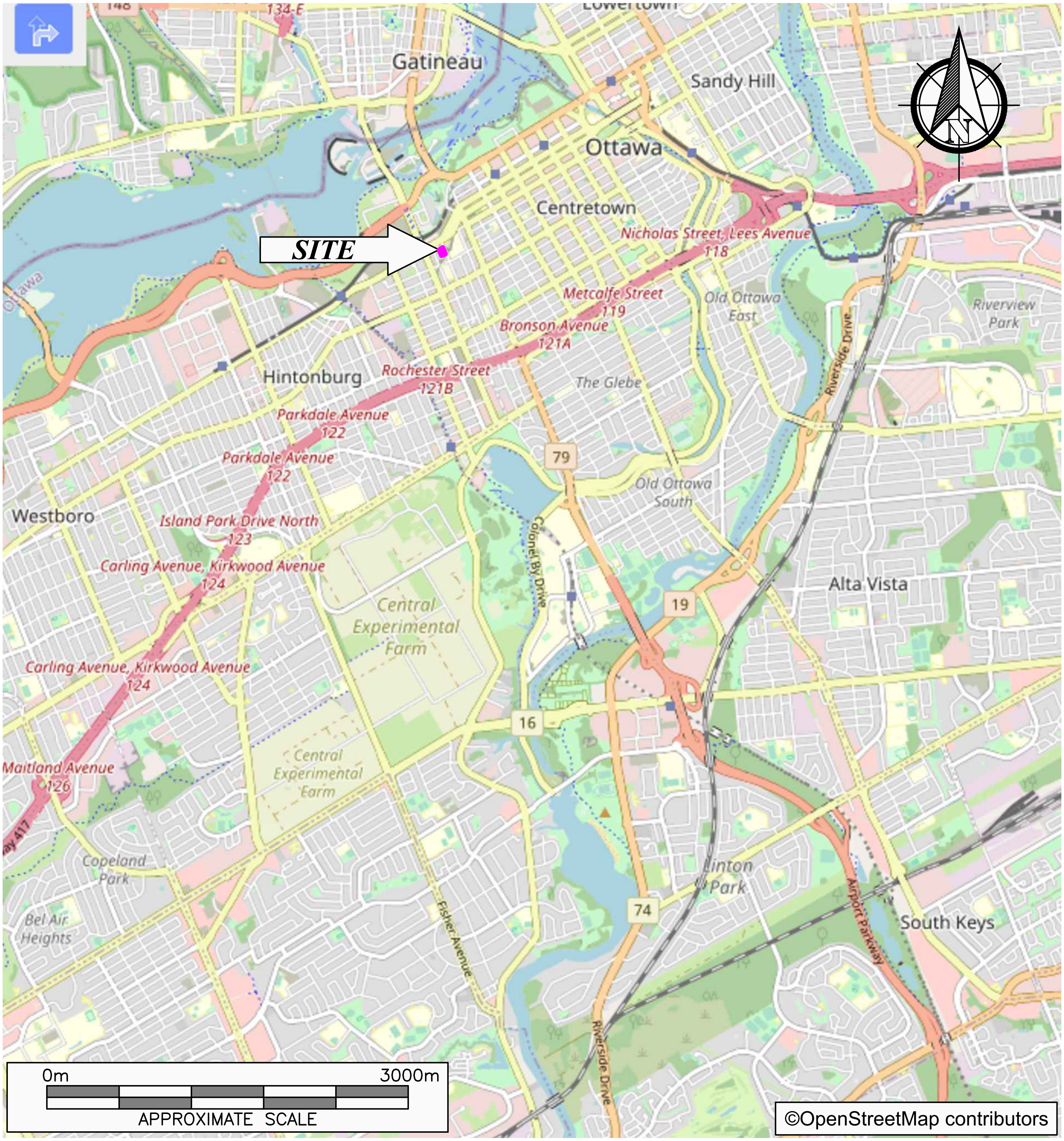
Pinchin makes no other representations whatsoever, including those concerning the legal significance of its findings, or as to other legal matters touched on in this report, including, but not limited to, ownership of any property, or the application of any law to the facts set forth herein. With respect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

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

Information provided by Pinchin is intended for Client use only. Pinchin will not provide results or information to any party unless disclosure by Pinchin is required by law. Any use by a third party of reports or documents authored by Pinchin or any reliance by a third party on or decisions made by a third party based on the findings described in said documents, is the sole responsibility of such third parties. Pinchin accepts no responsibility for damages suffered by any third party as a result of decisions made or actions conducted. No other warranties are implied or expressed.

329062.001 Geotech Investigation 10-20 Empress Ave OTT DNPHC Feb 22 2024
Template: Master Geotechnical Investigation Report – Ontario, GEO, September 2, 2021

FIGURES



PINCHIN

PROJECT NAME GEOTECHNICAL INVESTIGATION		
CLIENT NAME DALHOUSIE NON-PROFIT HOUSING COOPERATIVE INC.		
PROJECT LOCATION 10-20 EMPRESS AVENUE, OTTAWA, ONTARIO		
FIGURE NAME KEY MAP		FIGURE NO. 1
APPROXIMATE SCALE AS SHOWN	PROJECT NO. 329062.001	DATE AUGUST 2023

LEGEND

BOREHOLE / MONITORING WELL LOCATION

XX.XX APPROXIMATE GEODETIC GROUND ELEVATION (m)

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

PINCHIN

PROJECT NAME GEOTECHNICAL INVESTIGATION	
CLIENT NAME DALHOUSIE NON-PROFIT HOUSING COOPERATIVE INC.	
PROJECT LOCATION 10-20 EMPRESS AVENUE, OTTAWA, ONTARIO	
FIGURE NAME BOREHOLE/MONITORING WELL LOCATION PLAN	
APPROXIMATE SCALE AS SHOWN	PROJECT NO. 329062.001
DATE AUGUST 2023	FIGURE NO. 2

APPENDIX I

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

ABBREVIATIONS, TERMINOLOGY & PRINCIPAL SYMBOLS USED

Sampling Method

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

In-Situ Soil Testing

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

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

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

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

Soil Descriptions

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

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

Notes:

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

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

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

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

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

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

Soil & Rock Physical Properties

General

W Natural water content or moisture content within soil sample

γ Unit weight

γ' Effective unit weight

γ_d Dry unit weight

γ_{sat} Saturated unit weight

ρ Density

ρ_s Density of solid particles

ρ_w Density of Water

ρ_d Dry density

ρ_{sat} Saturated density e Void ratio

n Porosity

S_r Degree of saturation

E_{50} Strain at 50% maximum stress (cohesive soil)

Consistency

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

Shear Strength

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

Consolidation (One Dimensional)

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

Permeability

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

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

Rock Coring

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

RQD is calculated as follows:

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

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

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

APPENDIX II

Pinchin's Borehole Logs

Log of Borehole: BH1

Project #: 329062.001 **Logged By:** MK

Project: Geotechnical Investigation

Client: Dalhousie Non-Profit Housing Cooperative Inc.

Location: 10-20 Empress Avenue, Ottawa, Ontario

Drill Date: July 26, 2023 **Project Manager:** WT

SUBSURFACE PROFILE

Depth (m)	Description	Elevation (m)
0	Ground Surface	62.74
0–0.76	**Granular Fill** Sand and gravel, trace silt, brown, damp, loose	61.98
0.76–3.05	**Silty Sand** Silty sand, some gravel, trace clay, brown, moist, loose	59.69
3.05–7.62	Wet, very loose	55.12
7.62–9.75	**Silt and Sand** Silt and sand, trace gravel, trace clay, brown, compact, wet	52.99
9.75	End of Borehole. Borehole terminated at 9.75 mbgs in silt and sand.	

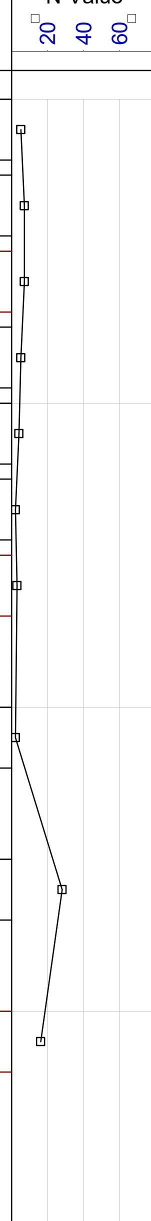
Monitoring Well Details: Riser; Bentonite; Silica Sand; Screen

Groundwater level = 2.70 mbgs, as measured on August 28, 2023.

SAMPLE

Sample Type	Sampler #	Recovery (%)	SPT N-Value	Water Content (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
SS	1	10	5				
SS	2	20	7				
SS	3	30	7				
SS	4	50	5				
SS	5	60	4	9.6			Hyd. MC.
SS	6	90	2				
SS	7	100	3				
SS	8	80	2				
SS	9	100	28				
SS	10	70	16				

Standard Penetration N-Value



Shear Strength kPa 100 200

Contractor: Strata Drilling Group **Grade Elevation:** 62.74 m

Drilling Method: Direct Push/Split Spoon **Top of Casing Elevation:** 62.61 m

Well Casing Size: 35 mm **Sheet:** 1 of 1

Log of Borehole: BH2

Project #: 329062.001 **Logged By:** MK

Project: Geotechnical Investigation

Client: Dalhousie Non-Profit Housing Cooperative Inc.

Location: 10-20 Empress Avenue, Ottawa, Ontario

Drill Date: July 26, 2023 **Project Manager:** WT

SUBSURFACE PROFILE

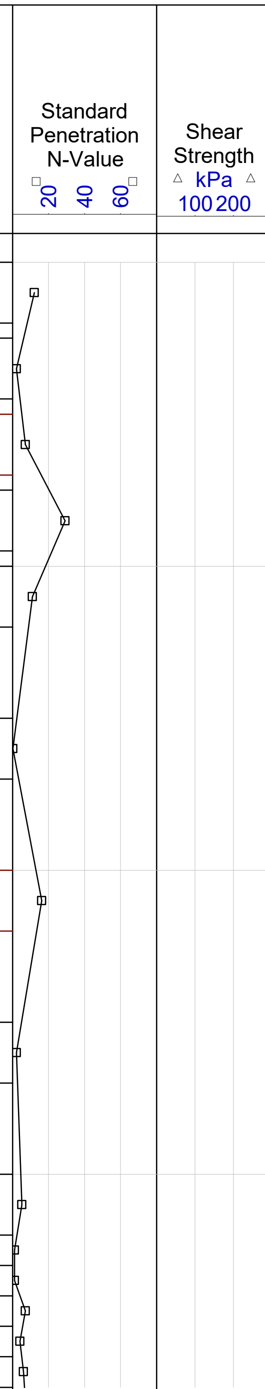
Depth (m)	Description	Elevation (m)
0	Ground Surface	62.59
0 – 0.45	**Granular Fill** Sand and gravel, trace silt, brown, damp, compact	62.13
0.45 – 4.57	**Silty Sand** Silty sand, some gravel, trace clay, brown, damp to moist, very loose to compact	58.02
4.57 – 7.62	Wet	54.97
7.62 – 9.75	**Silt and Sand** Silt and sand, trace gravel, trace clay, brown, very loose to loose, wet	52.84
9.75 –	**Dynamic Cone Penetration Test (DCPT)** Probable silt and sand	

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

Groundwater level = 4.05 mbgs, as measured on August 28, 2023.

SAMPLE

Sample Type	Sampler #	Recovery (%)	SPT N-Value	Water Content (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
SS	1	50	12				
SS	2	50	2				
SS	3	50	7				
SS	4	60	29				Corr.
SS	5	90	11				
SS	6	90	0				
SS	7	20	16				
SS	8	80	2	11.5			Hyd.
SS	9	80	5				
DCPT	1	NA	1				
DCPT	2	NA	1				
DCPT	3	NA	7				
DCPT	4	NA	4				
DCPT	5	NA	6				
	6		7				



Contractor: Strata Drilling Group **Grade Elevation:** 62.59 m

Drilling Method: Direct Push/Split Spoon **Top of Casing Elevation:** 62.41 m

Well Casing Size: 35 mm **Sheet:** 1 of 2

Log of Borehole: BH2

Project #: 329062.001 **Logged By:** MK

Project: Geotechnical Investigation

Client: Dalhousie Non-Profit Housing Cooperative Inc.

Location: 10-20 Empress Avenue, Ottawa, Ontario

Drill Date: July 26, 2023 **Project Manager:** WT

SUBSURFACE PROFILE | SAMPLE

Depth (m)	Symbol	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-Value	Standard Penetration N-Value (20 40 60)	Shear Strength kPa (100 200)	Water Content (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
					DCPT	6	NA	7						
12					DCPT	7	NA	28						
					DCPT	8	NA	21						
					DCPT	9	NA	29						
					DCPT	10	NA	24						
13					DCPT	11	NA	36						
					DCPT	12	NA	22						
					DCPT	13	NA	35						
14					DCPT	14	NA	100						
			48.26		DCPT	15	NA	100						
		End of Borehole Borehole terminated at 14.3 mbgs due to spoon refusal on probable bedrock.												
15														
16														
17														
18														
19														
20														
21														
22														

Contractor: Strata Drilling Group **Grade Elevation:** 62.59 m

Drilling Method: Direct Push/Split Spoon **Top of Casing Elevation:** 62.41 m

Well Casing Size: 35 mm **Sheet:** 2 of 2

Log of Borehole: BH3

Project #: 329062.001 **Logged By:** MK

Project: Geotechnical Investigation

Client: Dalhousie Non-Profit Housing Cooperative Inc.

Location: 10-20 Empress Avenue, Ottawa, Ontario

Drill Date: July 26, 2023 **Project Manager:** WT

SUBSURFACE PROFILE

Depth (m)	Description	Elevation (m)
0	Ground Surface	62.76
0–0.76	**Granular Fill** Sand and gravel, trace silt, brown, damp, compact	62.00
0.76–2.28	**Silty Sand** Silty sand, some gravel, trace clay, brown, moist, loose to compact	60.48
2.28–6.70	Wet	56.06
	End of Borehole Borehole terminated at 9.75 mbgs in silt and sand.	

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

Groundwater level = 2.81 mbgs, as measured on August 28, 2023.

SAMPLE

Sample Type	Sampler #	Recovery (%)	SPT N-Value	Laboratory Analysis
SS	1	40	13	
SS	2	60	11	
SS	3	60	13	GS.
SS	4	60	23	
SS	5	60	7	
SS	6	NA	19	
SS	7	100	19	

Standard Penetration N-Value: 20 40 60

Shear Strength kPa: 100 200

Water Content (%) | Sample ID | Soil Vapour Concentration (ppm)

Contractor: Strata Drilling Group **Grade Elevation:** 62.76 m

Drilling Method: Direct Push/Split Spoon **Top of Casing Elevation:** 62.59 m

Well Casing Size: 35 mm **Sheet:** 1 of 1

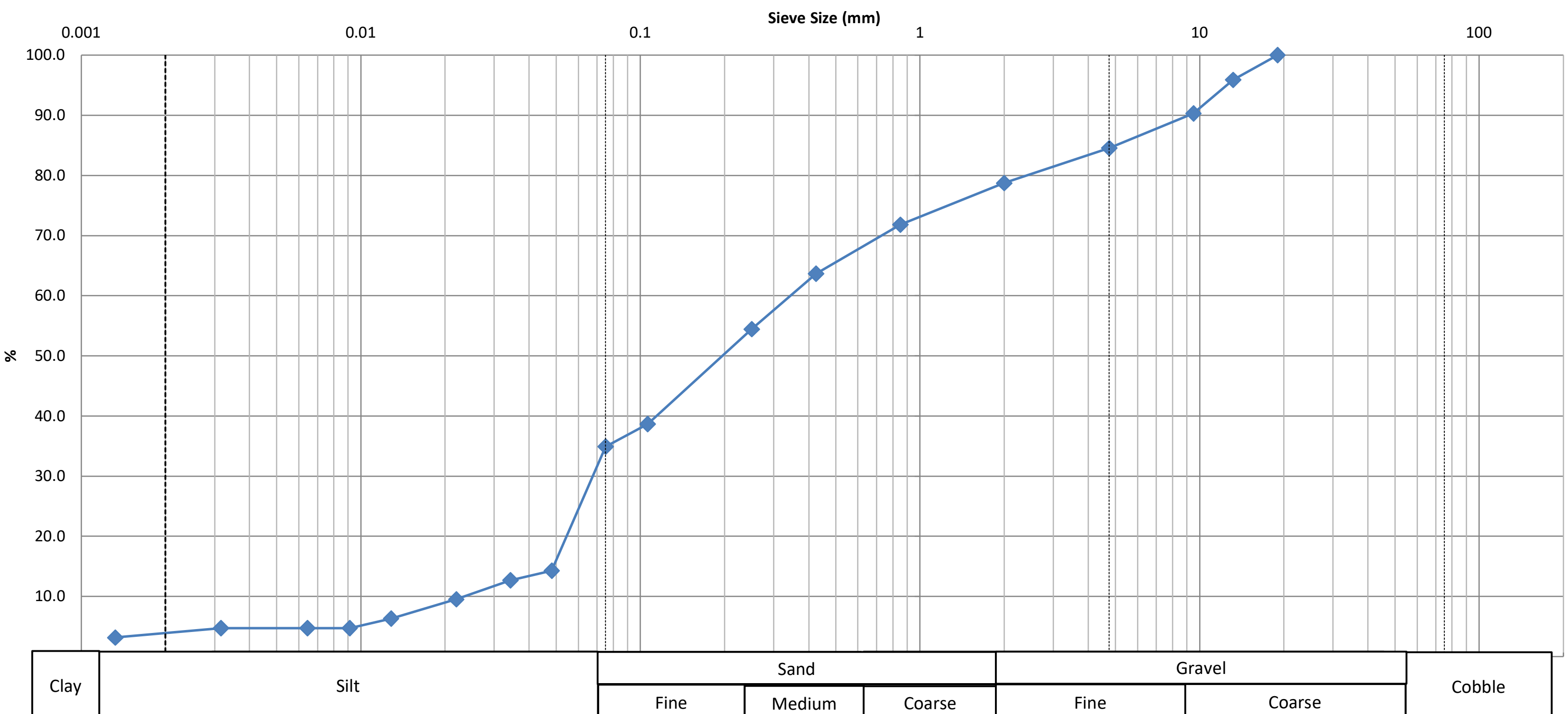
APPENDIX III

Laboratory Testing Reports for Soil Samples

PATERSON GROUP

SIEVE ANALYSIS
ASTM C136

CLIENT:	Pinchin	DEPTH:	10 - 12 ft	FILE NO:	PM4184
CONTRACT NO.:		BH OR TP No.:	BH1 SS5	LAB NO:	45699
PROJECT:	329062.001			DATE RECEIVED:	28-Jul-23
				DATE TESTED:	31-Jul-23
DATE SAMPLED:	27-Jul-23			DATE REPORTED:	11-Aug-23
SAMPLED BY:	Client			TESTED BY:	DJ



Identification	Soil Classification					MC(%)	LL	PL	PI	Cc	Cu
						9.6					
	D100	D60	D30	D10	Gravel (%)	Sand (%)		Silt (%)		Clay (%)	
					15.5	49.6		31.4		3.5	

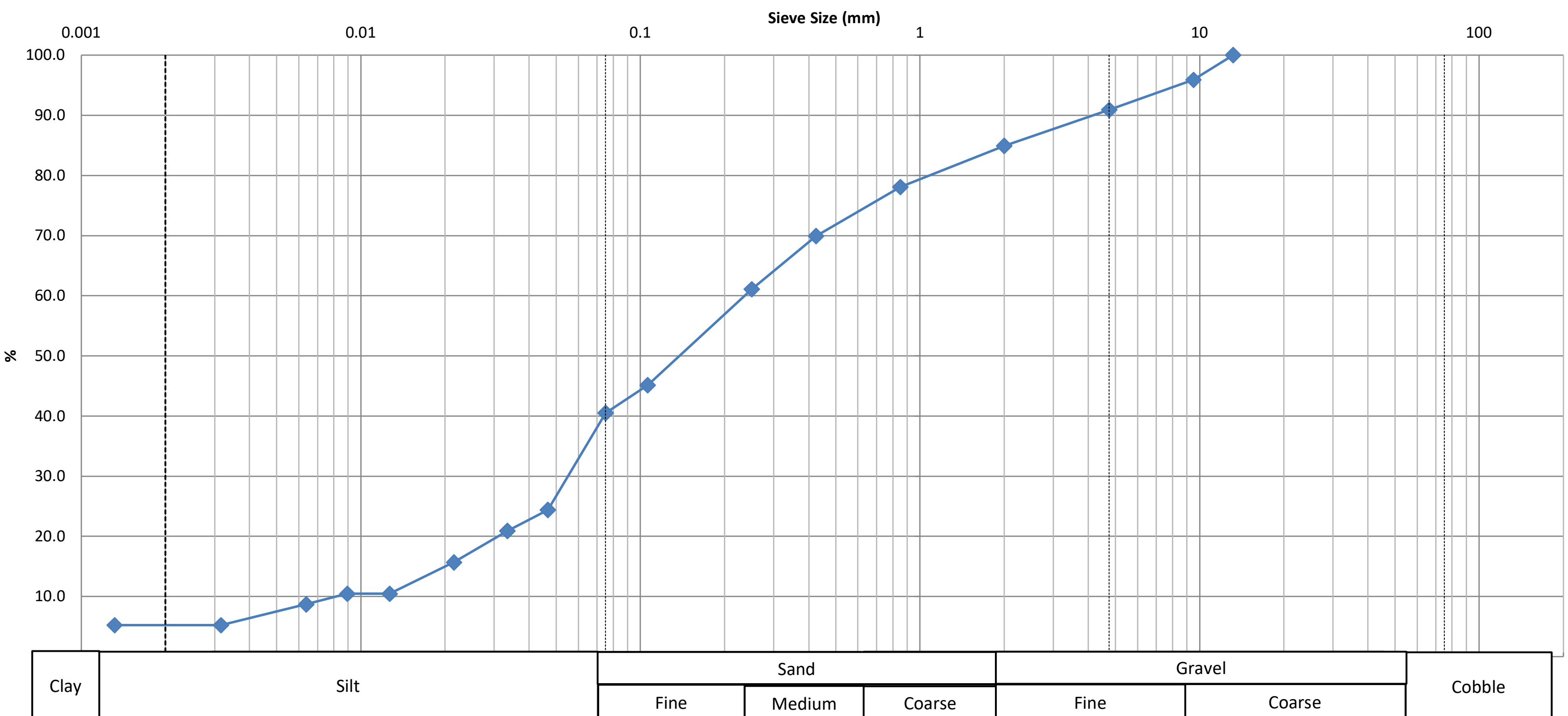
Comments:

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

PATERSON GROUP

SIEVE ANALYSIS
ASTM C136

CLIENT:	Pinchin	DEPTH:	25 - 27 ft	FILE NO:	PM4184
CONTRACT NO.:		BH OR TP No.:	BH2 SS8	LAB NO:	45701
PROJECT:	329062.001			DATE RECEIVED:	28-Jul-23
				DATE TESTED:	31-Jul-23
DATE SAMPLED:	27-Jul-23			DATE REPORTED:	11-Aug-23
SAMPLED BY:	Client			TESTED BY:	DJ



Identification	Soil Classification				MC(%)	LL	PL	PI	Cc	Cu
					11.5					
	D100	D60	D30	D10	Gravel (%)	Sand (%)	Silt (%)	Clay (%)		
					9.1	50.4	36.5	4.0		

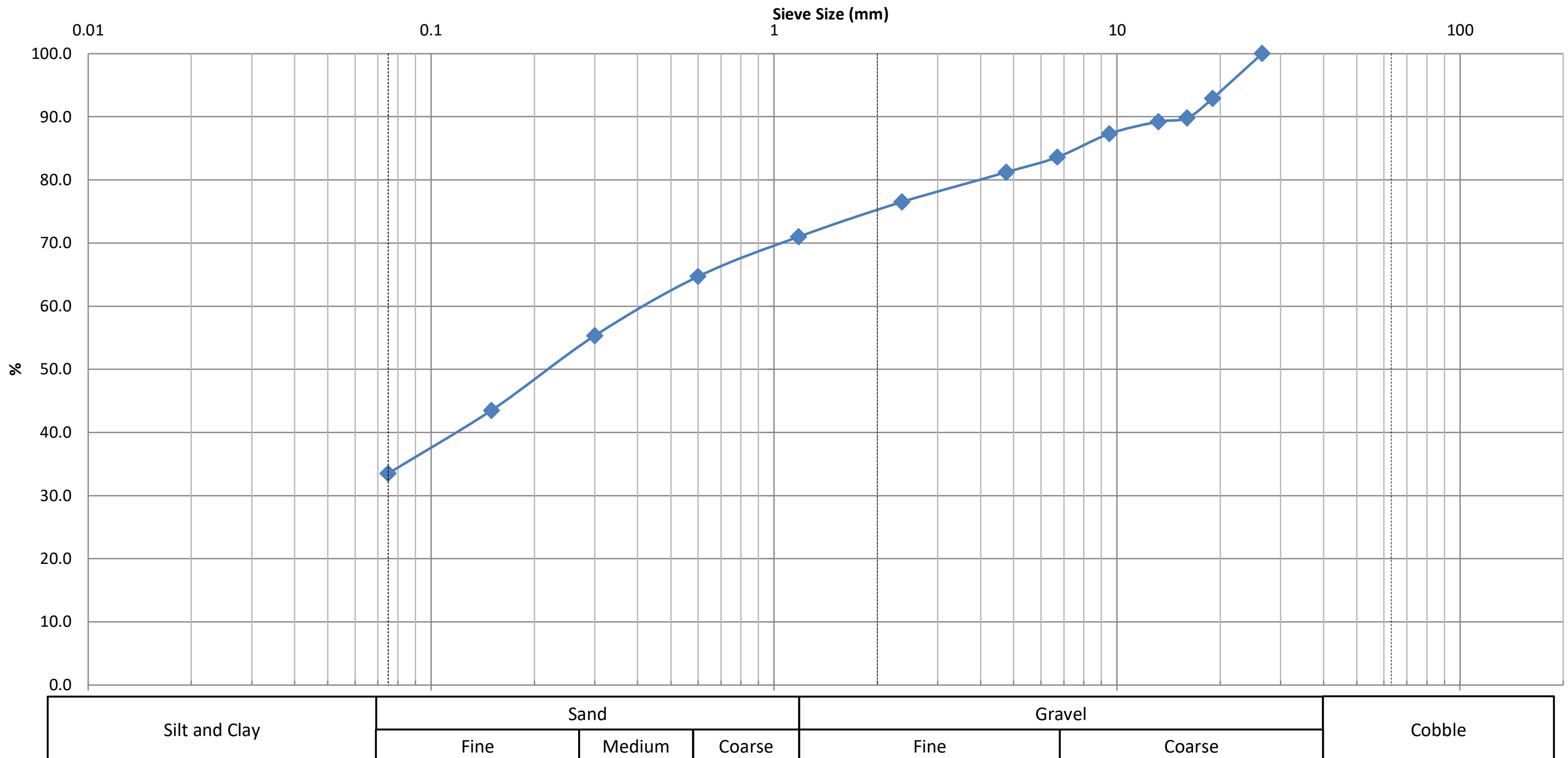
Comments:

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

PATERSON GROUP

SIEVE ANALYSIS
ASTM C136

CLIENT:	Pinchin	DESCRIPTION:	Fine Aggregate	FILE NO:	PM4184
CONTRACT NO.:	-	SPECIFICATION:	**Sand with Gravel**	LAB NO:	45700
PROJECT:	329062.001	INTENDED USE:	-	DATE RECEIVED:	28-Jul-23
		PIT OR QUARRY:	-	DATE TESTED:	31-Jul-23
DATE SAMPLED:	27-Jul-23	SOURCE LOCATION:	BH3 - SS3	DATE REPORTED:	15-Aug-23
SAMPLED BY:	Client	SAMPLE LOCATION:	5' - 7'	TESTED BY:	CP



Identification	Soil Classification				MC(%)	LL	PL	PI	Cc	Cu
									0.33	27.3
	D100	D60	D30	D10	Gravel (%)	Sand (%)	Silt (%)		Clay (%)	
	26.5	0.41	0.045	0.015	18.8	47.7	33.5			

Comments:

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

300 - 2319 St. Laurent Blvd
Ottawa, ON, K1G 4J8
1-800-749-1947
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Certificate of Analysis

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

Report Date: 3-Aug-2023
Order Date: 27-Jul-2023

Client PO:
Project: 329062.001
Custody: 140609

Order #: 2330385

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

Paracel ID	Client ID
2330385-01	BH2 SS4 7.5-9.5 ft

Approved By:

Dale Robertson, BSc
Laboratory Director

PARACEL
LABORATORIES LTD.

Order #: 2330385

Certificate of Analysis

Report Date: 03-Aug-2023

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

Order Date: 27-Jul-2023

Client PO:

Project Description: 329062.001

Analysis Summary Table

Analysis	Method Reference/Description	Extraction Date	Analysis Date
Anions	EPA 300.1 - IC, water extraction	31-Jul-23	31-Jul-23
Conductivity	MOE E3138 - probe @25 °C, water ext	31-Jul-23	31-Jul-23
pH, soil	EPA 150.1 - pH probe @ 25 °C, CaCl buffered ext.	1-Aug-23	1-Aug-23
Resistivity	EPA 120.1 - probe, water extraction	31-Jul-23	31-Jul-23
Solids, %	CWS Tier 1 - Gravimetric	2-Aug-23	3-Aug-23

Certificate of Analysis

Report Date: 03-Aug-2023

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

Order Date: 27-Jul-2023

Client PO:

Project Description: 329062.001

	Client ID:	BH2 SS4 7.5-9.5 ft	-	-	-		
	Sample Date:	26-Jul-23 12:00	-	-	-	-	-
	Sample ID:	2330385-01	-	-	-		
	Matrix:	Soil	-	-	-		
	MDL/Units						
Physical Characteristics							
% Solids	0.1 % by Wt.	90.7	-	-	-	-	-
General Inorganics							
Conductivity	5 uS/cm	267	-	-	-	-	-
pH	0.05 pH Units	7.96	-	-	-	-	-
Resistivity	0.1 Ohm.m	37.4	-	-	-	-	-
Anions							
Chloride	10 ug/g	73	-	-	-	-	-
Sulphate	10 ug/g	68	-	-	-	-	-

Certificate of Analysis

Report Date: 03-Aug-2023

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

Order Date: 27-Jul-2023

Client PO:

Project Description: 329062.001

Method Quality Control: Blank

Analyte	Result	Reporting Limit	Units	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions								
Chloride	ND	10	ug/g					
Sulphate	ND	10	ug/g					
General Inorganics								
Conductivity	ND	5	uS/cm					
Resistivity	ND	0.1	Ohm.m					

Method Quality Control: Duplicate

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	ND	10	ug/g	ND			NC	35	
Sulphate	134	10	ug/g	149			10.3	35	
General Inorganics									
Conductivity	299	5	uS/cm	296			1.0	5	
pH	7.74	0.05	pH Units	7.70			0.5	2.3	
Resistivity	33.5	0.1	Ohm.m	33.8			1.0	20	
Physical Characteristics									
% Solids	61.2	0.1	% by Wt.	62.0			1.3	25	

Method Quality Control: Spike

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	102	10	ug/g	ND	102	82-118			
Sulphate	248	10	ug/g	149	98.7	80-120			

Qualifier Notes:

Login Qualifiers :

Received at temperature > 25C [all samples]

Sample - One or more parameter received past hold time - Redox potential.

Applies to Samples: BH2 SS4 7.5-9.5 ft

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.

PARACEL LABORATORIES LTD.

Paracel ID: 2330385

Paracel Order Number (Lab Use Only)

Chain Of Custody (Lab Use Only)

№ 140609

Client Name: Pinchin Ltd.	Project Ref: 329062.001
Contact Name: Megan Keon	Quote #:
Address: 1 Hines Rd. Ottawa, ON	PO #:
	E-mail: mkeon@pinchin.com
Telephone: 613-592-3387	

Page 1 of 1

Turnaround Time

☐ 1 day ☐ 3 day
☐ 2 day ☑ Regular

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	PHCs F1-F4+BTEX	VOCs	PAHs	Metals by ICP	Hg	CrVI	B (HWS)	corrosivity	redox	sulphides	conductivity	
1	BH2 SS4 7.5-9.5 ft	S		1	July 26/23	PM								✓	✓	✓	✓	
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] 4:20	Received at Lab: [signature]	Verified By: [signature]
Relinquished By (Print): Megan Keon.	Date/Time: July 27/23 pm	Date/Time: July 27/23 1235	Date/Time: July 27/23 1235
Date/Time: July 27/2023 @4:20pm	Temperature: 26.0 °C	Temperature: 7.6 °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.	**2330385**	Order Date:	27-Jul-23
Client Project(s):	**329062.001**	Report Date:	31-Aug-23
Client PO:			
Reference:	**2023 Standing Offer - ENV**		
CoC Number:	**140609**		

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**
2330385-01	BH2 SS4 7.5-9.5 ft	Redox potential, soil Sulphide, solid

TESTMARK Laboratories Ltd.
Committed to Quality and Service

CERTIFICATE OF ANALYSIS

Client:	Dale Robertson	Work Order Number:	507720
Company:	Paracel Laboratories Ltd. - Ottawa	PO #:	
Address:	300-2319 St. Laurent Blvd.	Regulation:	Information not provided
	Ottawa, ON, K1G 4J8	Project #:	2330385
Phone/Fax:	(613) 731-9577 / (613) 731-9064	DWS #:	
Email:	drobertson@paracellabs.com	Sampled By:	
Date Order Received:	8/1/2023	Analysis Started:	8/2/2023
Arrival Temperature:	10.3 °C	Analysis Completed:	8/2/2023

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
BH2 SS4 7.5-9.5 ft	1910810	Soil	None		7/26/2023	12:00 PM

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

Sample received past hold time for redox, proceed with analysis as per client notes TJ 08/01/23

This report has been approved by:

Marc Creighton
Laboratory Director

TESTMARK Laboratories Ltd.
Committed to Quality and Service

CERTIFICATE OF ANALYSIS

Paracel Laboratories Ltd. - Ottawa

Work Order Number: 507720

WORK ORDER RESULTS

Sample Description	BH2 SS4 7.5 - 9.5 ft
Sample Date	7/26/2023 12:00 PM
Lab ID	1910810

General Chemistry	Result	MDL	Units
RedOx (vs. S.H.E.)	393 [391]	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.

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

31-August-2023

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. : 01 August 2023
LR Report: **CA15018-AUG23**
Reference: Project#: 2330385

Copy: #1

CERTIFICATE OF ANALYSIS

Final Report

Sample ID	Sample Date & Time	Sulphide (Na2CO3) %
1: Analysis Start Date		21-Aug-23
2: Analysis Start Time		12:50
3: Analysis Completed Date		31-Aug-23
4: Analysis Completed Time		11:23
5: QC - Blank		< 0.04
6: QC - STD % Recovery		113%
7: QC - DUP % RPD		11%
8: RL		0.02
9: BH2 SS4 7.5-9.5 ft	26-Jul-23 12:00	0.05

RL - SGS Reporting Limit

Kimberley Didsbury
Project Specialist,
Environment, Health & Safety

OnLine LIMS

0003452051

PARACEL LABORATORIES LTD.

Paracel ID: 2330385

Paracel Order Number (Lab Use Only)

Chain Of Custody (Lab Use Only) № 140609

Client Name: Pinchin Ltd.	Project Ref: 329062, 001		Page 1 of 1
Contact Name: Megan Keon	Quote #:		Turnaround Time
Address: 1 Hines Rd. Ottawa, ON	PO #:		☐ 1 day ☐ 3 day
	E-mail: mKeon@pinchin.com		☐ 2 day ☑ Regular
Telephone: 613-592-3387			Date Required:

☐ REG 153/04 ☐ REG 406/19 — Other Regulation

☐ Table 1 ☐ Res/Park ☐ Med/Fine ☐ REG 558 ☐ PWQO

☐ Table 2 ☐ Ind/Comm ☐ Coarse ☐ CCME ☐ MISA

☐ Table 3 ☐ Agri/Other ☐ SU - Sani ☐ SU - Storm

☐ Table ___ Mun: ___

For RSC: ☐ Yes ☐ No ☐ 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	PHCs F1-F4+BTEX	VOCs	PAHs	Metals by ICP	Hg	CrVI	B (HWS)	Corrosivity	redox	sulphides	conductivity	
1	BH2 SS4 7.5-9.5 ft	S		1	July 26/23	PM								✓	✓	✓	✓	
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] 4:20	Received at Lab: [signature]	Verified By: [signature]
Relinquished By (Print): Megan Keon.	Date/Time: July 27/23. pm	Date/Time: July 27/23 1235	Date/Time: July 27/23 1235
Date/Time: July 27/2023 @4:20pm	Temperature: 26.0 °C	Temperature: 7.6 °C	pH Verified: ☐ By:

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.