REVISED

Geotechnical Investigation – Proposed Residential Development

949 North River Road, Ottawa, Ontario

Prepared for:

Gemstone River Road (GP) Inc.
252 Argyle Avenue
Ottawa, ON K2P 1B9

September 22, 2022

Pinchin File: 283759.001

Issued to:	**Gemstone River Road (GP) Inc.**
Issued on:	**September 22, 2022**
Pinchin File:	**283759.001**
Issuing Office:	**Kanata, ON**

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

Reviewer: Vanessa Marshall, M.Eng., P.Eng.
National Practice Leader, Geotechnical Services
519.904.4660
vmarshall@pinchin.com

LICENSED PROFESSIONAL ENGINEER
V. M. MARSHALL
100079066
PROVINCE OF ONTARIO

TABLE OF CONTENTS

FIGURES

APPENDICES

1.0 INTRODUCTION AND SCOPE

Pinchin Ltd. (Pinchin) was retained by Gemstone River Road (GP) Inc. (Client) to conduct a Geotechnical Investigation and provide subsequent geotechnical design recommendations for the proposed residential development to be located at 949 North River Road, Ottawa, Ontario (Site). The Site location is shown on Figure 1.

Based on information provided by the Client, it is Pinchin's understanding that the proposed development is to consist of a five-storey residential apartment building, complete with a single level underground parking garage which will occupy the entire Site footprint. The underside of the footings for the proposed parking garage will be located at a depth of approximately 3.0 to 3.5 metres below existing ground surface (mbgs). It is noted that due to the parking garage occupying the entire Site footprint, no asphaltic concrete pavement structures are required for the proposed development.

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. The information gathered from the Geotechnical Investigation will allow Pinchin to provide geotechnical design recommendations for the proposed development.

Based on a desk top review and the results of the Geotechnical Investigation, the following geotechnical data and engineering design recommendations are provided herein:

- A detailed description of the soil and groundwater conditions;
- Site preparation recommendations;
- Open cut excavations;
- Anticipated groundwater management;
- Site service trench design;
- Foundation design recommendations including bedrock bearing resistances at Ultimate Limit States (ULS) design;
- Potential total and differential settlements;
- Foundation frost protection and engineered fill specifications and installation;
- Seismic Site classification for seismic Site response;
- Underground parking garage design, including concrete floor slab support recommendations;

- Soil corrosivity and sulphate attack on concrete; and
- Potential construction concerns.

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

2.0 SITE DESCRIPTION AND GEOLOGICAL SETTING

The Site is located on the east side of North River Road, approximately 1.2 kilometres north of Highway 417 in Ottawa, Ontario. The Site is currently developed with a two-storey residential apartment building complete with asphalt surfaced parking areas and areas of soft landscaping. The lands adjacent to the Site are developed with a combination of single family and multi unit residential buildings.

Data obtained from the Ontario Geological Survey Maps, as published by the Ontario Ministry of Natural Resources, indicates that the Site is located on a fine-textured glaciomarine deposit consisting of massive to well laminated silt and clay with minor sand and gravel (Ontario Geological Survey 2010. Surficial geology of Southern Ontario; Ontario Geological Survey, Miscellaneous Release--Data 128-REV). The underlying bedrock at this Site is of the Georgian Bay, Blue Mountain, and Billings Formations consisting of shale, limestone, dolostone, and siltstone (Ontario Geological Survey 2011. 1:250 000 scale bedrock geology of Ontario; Ontario Geological Survey, Miscellaneous Release---Data 126-Revision 1).

3.0 GEOTECHNICAL FIELD INVESTIGATION AND METHODOLOGY

Pinchin completed a field investigation at the Site on March 30, 2021 by advancing a total of four sampled boreholes throughout the Site. The boreholes were advanced to depths ranging from approximately 0.8 to 2.4 mbgs, where refusal was encountered on probable bedrock. The approximate spatial locations of the boreholes advanced at the Site are shown on Figure 2.

The boreholes were advanced with the use of a Geoprobe 7822 DT direct push drill rig which was equipped with standard soil sampling equipment. Soil samples were collected at 0.76 m intervals using a 51 mm outside diameter (OD) split spoon barrel in conjunction with Standard Penetration Tests (SPT) “N” values (ASTM D1586). The SPT “N” values were used to assess the compactness condition of the non-cohesive soil.

A monitoring well was installed within Borehole BH3 to allow measurement of groundwater levels. The monitoring well was constructed using flush-threaded 50 mm diameter Trilock pipe with 1.2-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. It is noted that the well was installed within the overburden material as the boreholes were not advanced into the underlying bedrock. As such, an existing monitoring well which was previously installed at the

Site by others was also utilized to measure the groundwater level. Based on the refusal depths encountered within the boreholes this well is inferred to be installed within the bedrock.

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 wells on April 19, 2021. The groundwater observations and measurements recorded are included on the appended borehole logs.

The borehole locations and ground surface elevations were located at the Site by Pinchin personnel. The ground surface elevation at each borehole location was referenced to the following temporary benchmark as shown on Figure 2:

- TBM: Top nut of fire hydrant, at the approximate location shown on Figure 2; and
- Elevation: 100.0 metres (local datum).

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

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

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

4.0 SUBSURFACE CONDITIONS

4.1 Borehole Soil Stratigraphy

In general, the soil stratigraphy at the Site comprises either surficial asphalt or surficial organics overlying granular fill, glacial till, and probable bedrock to the maximum borehole refusal depth of approximately 2.4 mbgs. The appended borehole logs provide detailed soil descriptions and stratigraphies, results of SPT testing, and groundwater measurements.

The surficial asphalt was encountered within Boreholes BH2 and BH3 and was measured to be approximately 25 to 50 mm thick. The surficial organics were encountered within Boreholes BH1 and BH4 and were measured to be approximately 150 mm thick.

Granular fill was encountered within all boreholes underlying either the surficial asphalt or surficial organics. The fill material was measured to range in thickness from approximately 0.8 to 1.5 m and ranged in soil matrix from sand and gravel containing some silt, to sand containing some silt. The non-cohesive material had a very loose to compact relative density based SPT ‘N’ values of between 1 and 16 blows per 300 mm penetration of a split spoon sampler. The results of two particle size distribution analyses completed on samples of the fill material indicate that the samples contained approximately 0 to 36% gravel, 53 to 87% sand, and 11 to 13% silt sized particles.

The glacial till was encountered underlying the granular fill in Boreholes BH1, BH3 and BH4 and extended down to the underlying bedrock surface between approximately 0.9 and 2.4 mbgs. The glacial till comprised silty sand containing some gravel and some clay. The non-cohesive glacial till had a variable very loose to very dense relative density based SPT ‘N’ values of 2 to greater than 50 blows per 300 mm penetration of a split spoon sampler. The result of one particle size distribution analysis completed on a sample of the glacial till indicates that the sample contains approximately 18% gravel, 43% sand, 27% silt, and 12% clay sized particles. The moisture content of the material tested was 21.1%, indicating the material was in a damp to moist condition at the time of sampling.

4.2 Groundwater Conditions

Groundwater observations and measurements were obtained in the open boreholes at the completion of drilling and are summarized on the appended borehole logs. The groundwater level was measured on April 19, 2021, in the monitoring well installed within Borehole BH3 as well as in the existing monitoring well which was previously installed by others. Groundwater was not encountered within the monitoring well within Borehole BH3; however, it was measured to be approximately 4.6 mbgs within the existing monitoring well previously installed by others.

Seasonal variations in the water table should be expected, with higher levels occurring during wet weather conditions in the spring and fall and lower levels occurring during dry weather conditions.

5.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

5.1 General Information

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

Based on information provided by the Client, it is Pinchin's understanding that the proposed development is to consist of a five-storey residential apartment building, complete with a single level underground parking garage which will occupy the entire Site footprint. The underside of the footings for the proposed parking garage will be located at a depth of approximately 3.0 to 3.5 metres below existing ground surface (mbgs). It is noted that due to the parking garage occupying the entire Site footprint, no asphaltic concrete pavement structures are required for the proposed development.

5.2 Site Preparation

Prior to Site preparation activities commencing, the existing building structure will need to be demolished and removed from the Site, including all foundations and service pipes. Preparation of the Site for the proposed development will consist of removing all surficial and overburden materials down to the underlying bedrock surface.

5.3 Open Cut Excavations and Anticipated Groundwater Management

Excavations for the building foundations will extend to an approximate depth of 3.0 to 3.5 mbgs. As such, a portion of the bedrock will need to be removed to accommodate the 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, glacial till, and bedrock. Groundwater was measured at approximately 4.6 mbgs within the existing monitoring well which was previously installed by others.

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

Based on the OHSA, the natural subgrade soils would be classified as Type 3 soil and temporary excavations in these soils must be sloped at an inclination of 1 horizontal to 1 vertical (H to V) from the base of the excavation.

Based on local experience the upper approximate 1.5 m of bedrock in this area is typically weathered and can usually be removed with mechanical equipment, such as a large excavator and hydraulic hammer (hoe ram) and where required, with line drilling on close centres. Often a hydraulic hammer can be utilized to create an initial opening for the excavator bucket to gain access of the layered rock. The bedrock is known to contain vertical joints and near horizontal bedding planes. Therefore, some vertical and horizontal over break of the bedrock should be expected.

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

Drilling and blasting activities shall be carried out in accordance with the requirements outlined in Ontario Provincial Standard Specification (OPSS) 120. In addition, Pinchin has provided the following additional recommendations:

- Prior to commencing drilling and blasting activities a pre-blast/pre-construction survey of all buildings, utilities, structures, water wells, and facilities within a 150 m radius of the Site is to be performed. The pre-blast survey is to include but not be limited to details on the type of structure (i.e., age and type of construction), description of any existing/observed building deficiencies (i.e., differential settlement, cracks, structural and cosmetic damage, and etcetera) including dimensions when possible, and time stamped and labelled digital photographs and/or videos of areas of concern.

- Monitoring for Peak Particle Velocity (PPV) is to be completed and limited to 50 mm/s for frequencies greater than 40 Hz, 20 mm/s for frequencies equal to or less than 40 Hz, and 10 mm/s when concrete and grout has been placed within the previous 72 hours.
- Monitoring of peak sound pressure and water overpressure may also be required and are to be completed in accordance with the recommendations outline in OPSS 120 (120.07.05 Monitoring).
- A minimum of 3 trial blasts are to be completed to ensure the proposed blast design can be completed within the PPV vibration limits.
- Blasting mats and utility line shielding is to be utilized for all blasts.
- Records of each blast are to be completed which shall include but not be limited to the date, time and location of the blast, wind and atmospheric conditions at the time of the blast, blast details, and recorded values from the monitoring equipment.

It is noted that Pinchin is unaware of the adjacent property foundation design details and therefore, the impact of bedrock removal on adjacent properties cannot be confirmed; however, by completing pre-blast/pre-construction survey of the adjacent properties as well as adhering to the details outlined in OPSS 120, the impact on adjacent properties will be mitigated.

Pinchin notes that, local contractors are familiar with excavating the local bedrock and have specialized knowledge and techniques for its removal. Depending on the block size and degree of weathering of the rock they may have a different approach than what is presented in the preceding paragraphs.

Construction slopes in intact bedrock should stand near vertical provided the “loose” rock is properly scaled off the face. Once the blasting is completed, if there are any permanent bedrock shear walls, they will have to be reviewed by a Rock Mechanics Specialist to determine if it is stable or if it needs reinforcing, such as rock bolting.

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

Minor groundwater inflow through the bedrock is expected. It is believed that this groundwater inflow can be controlled using a gravity dewatering system with perimeter interceptor ditches and high capacity pumps. It is noted that once the final grades have been set, Pinchin should review this recommendation and revise as necessary.

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

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

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

It is the responsibility of the contractor to propose a suitable dewatering system based on the groundwater elevation at the time of construction. The method used should not adversely impact any nearby structures. Excavations to conventional design depths for the building foundations are not expected to require a Permit to Take Water or a submission to the Environmental Activity and Sector Registry (EASR). It is the responsibility of the contractor to make this application if required.

5.4 Site Servicing

5.4.1 Pipe Bedding and Cover Materials for Flexible and Rigid Pipes

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

Service pipes require an adequate base to ensure proper pipe connection and positive flow is maintained post construction. As such, pipe bedding should be placed to be of uniform thickness and compactness. The pipe bedding and cover material should conform to OPSD 802.010 and 802.013 specifications for flexible pipes and to OPSD 802.031 to 802.033 with Class “B” bedding for rigid pipes. The pipe bedding material should consist of a minimum thickness of 150 mm Granular “A” (OPSS 1010) below the pipe and extend up the sides to the spring line. However, the bedding thickness may have to be increased depending on the pipe diameter or if wet or weak subgrade conditions are encountered. The pipe cover material from the spring line should consist of a Granular “B” Type I (OPSS 1010) and should extend to a minimum of 300 mm above the top of the pipe.

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

- Install 300 mm of 19 mm clear stone gravel (OPSS 1004) or Granular ‘A’ (OPSS 1010) below the pipe extending up the sides to the spring line;

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

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

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

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

5.4.2 Trench Backfill

The trench backfill should be compacted in maximum 300 mm thick lifts to 98% SPMDD within 4% of the optimum moisture content. 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 post-construction settlements could occur.

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

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

5.4.3 Frost Protection

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

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

5.5 Foundation Design

5.5.1 Shallow Foundations Bearing on Bedrock

For conventional shallow strip and spread footings established directly on the weathered bedrock surface, a factored geotechnical bearing resistance of 700 kPa may be used at ULS. Higher bearing resistances may be available on the unweathered bedrock; however, the bedrock should be cored to confirm this recommendation.

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

The bearing resistance of 700 kPa assumes the bedrock is cleaned of all overburden material and any loose rock pieces. The bedrock should be cleaned with air or water pressure exposing clean sound bedrock. If construction proceeds during freezing weather conditions water should not be allowed to pool and freeze in bedrock depressions. All concrete should be installed and maintained above freezing temperatures as required by the concrete supplier.

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

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

5.5.2 Site Classification for Seismic Site Response & Soil Behaviour

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

The seismic site classification has been based on the 2012 OBC. The parameters for determination of Site Classification for Seismic Site Response are set out in Table 4.1.8.4.A of the OBC. The site classification is based on the average shear wave velocity in the top 30 m of the site stratigraphy.

Pinchin retained Geophysics GPR to complete one shear wave velocity sounding at the Site (see Appendix IV). Based on the results of the shear wave velocity sounding, this Site has been classified as Class B. For foundations placed directly on bedrock, or where there is 1.1 m or less of unconsolidated material between the underside of the footing and the bedrock surface, a Site Class A may be used for design purposes.

5.5.3 Foundation Transition Zones

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

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

5.5.4 Estimated Settlement

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

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

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

5.5.5 Building Drainage

To assist in maintaining the building dry from surface water seepage, it is recommended that exterior grades around the buildings be sloped away at a 2% gradient or more, for a distance of at least 2.0 m.

Roof drains should discharge a minimum of 1.5 m away from the structure to a drainage swale or appropriate storm drainage system (i.e. interior sump pit).

5.5.6 Shallow Foundations Frost Protection & Foundation Backfill

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

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

Where the foundations for heated buildings do not have the minimum 1.8 m of soil cover frost protection, they should be protected from frost with a combination of soil cover and rigid polystyrene insulation, such as Dow Styrofoam or equivalent product.

To minimize potential frost movements from soil frost adhesion, the perimeter foundation backfill should consist of a free draining granular material, such as a Granular 'B' Type I (OPSS 1010) or an approved sand fill, extending a minimum lateral distance of 600 mm beyond the foundation. The backfill material used against the foundation must be placed so that the allowable lateral capacity is achieved. All granular material is to be placed in maximum 300 mm thick lifts compacted to a minimum of 100% SPMDD in hard landscaping areas and 95% SPMDD in soft landscaping areas. It is recommended that inspection and testing be carried out during construction to confirm backfill quality, thickness and to ensure compaction requirements are achieved.

5.6 Underground Parking Garage Design

It is understood that the building will be constructed with a single-level underground parking garage with the underside of the footings located between approximately 3.0 to 3.5 mbgs.. Groundwater was measured at approximately 4.6 mbgs on April 19, 2021 within the existing monitoring well which was installed by others at the Site.

Exterior perimeter foundation drains should 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.

If the proposed basement floor level is constructed close to the stabilized groundwater level, an underfloor drainage system should 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 interior sump pit.

If the building is constructed below the groundwater table and subdrains and pumps are used to remove the groundwater from around the building footprint, there is the potential that a Permit to Take Water from the Ministry of the Environment, Conservation and Parks will be required for the long term dewatering of the Site. Pinchin would be able to provide further recommendations once the final grades have been set for the Site.

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

5.6.1 Concrete Floor Slab

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

Based on the in-situ conditions, it is recommended to establish a concrete floor slab-on-grade on a minimum 200 mm thick layer of coarse clean granular material containing not more than 10% material that will pass a 4 mm sieve. Any required up-fill should consist of a Granular ‘B’ Type I or Type II (OPSS 1010).

The installation of a vapour barrier may be required under the floor slab. If required, the vapour barrier should conform to the flooring manufacturer’s and designer’s requirements. Consideration may be given to carrying out moisture emission and/or relative humidity testing of the slab to determine the concrete condition prior to flooring installation. To minimize the potential for excess moisture in the floor slab, a concrete mixture with a low water-to-cement ratio (i.e. 0.5 to 0.55) should be used.

A modulus of subgrade reaction of 75 MPa/m can be used for the design of the floor slab founded on the inferred bedrock.

6.0 SOIL CORROSIVITY AND SULPHATE ATTACK ON CONCRETE

A soil sample from Borehole BH1 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 sample was 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
BH1 @ 3 ft	2,170	2	7.45	0	386	0	Trace	2	Fair drainage, generally moist	1	5

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

The result of the sulphate testing indicates that the Site possesses moderate to severe sulphate exposure, indicating that S-2 concrete should be used for the proposed structures at the Site. 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 bedrock surface prior to pouring any foundations or footings, backfilling, or engineered fill installation to ensure that the actual conditions are not markedly different than what was observed at the borehole locations and geotechnical components are constructed as per Pinchin's recommendations. Compaction quality control of engineered fill material (full-time monitoring) is recommended as standard practice, as well as regular sampling and testing of aggregates and concrete, to ensure that physical characteristics of materials for compliance during installation and satisfies all specifications presented within this report.

8.0 TERMS AND LIMITATIONS

This Geotechnical Investigation was performed for the exclusive use of Gemstone River Road (GP) Inc. (Client) in order to evaluate the subsurface conditions at 949 North River Road, Ottawa, Ontario. Within the limitations of scope, schedule and budget, our services have been executed in accordance with generally accepted practises in the field of geotechnical engineering for the Site. Classification and identification of soil, and geologic units have been based upon commonly accepted methods employed in professional geotechnical practice. No warranty or other conditions, expressed or implied, should be understood. Conclusions derived are specific to the immediate area of study and cannot be extrapolated extensively away from sample locations.

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

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

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

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

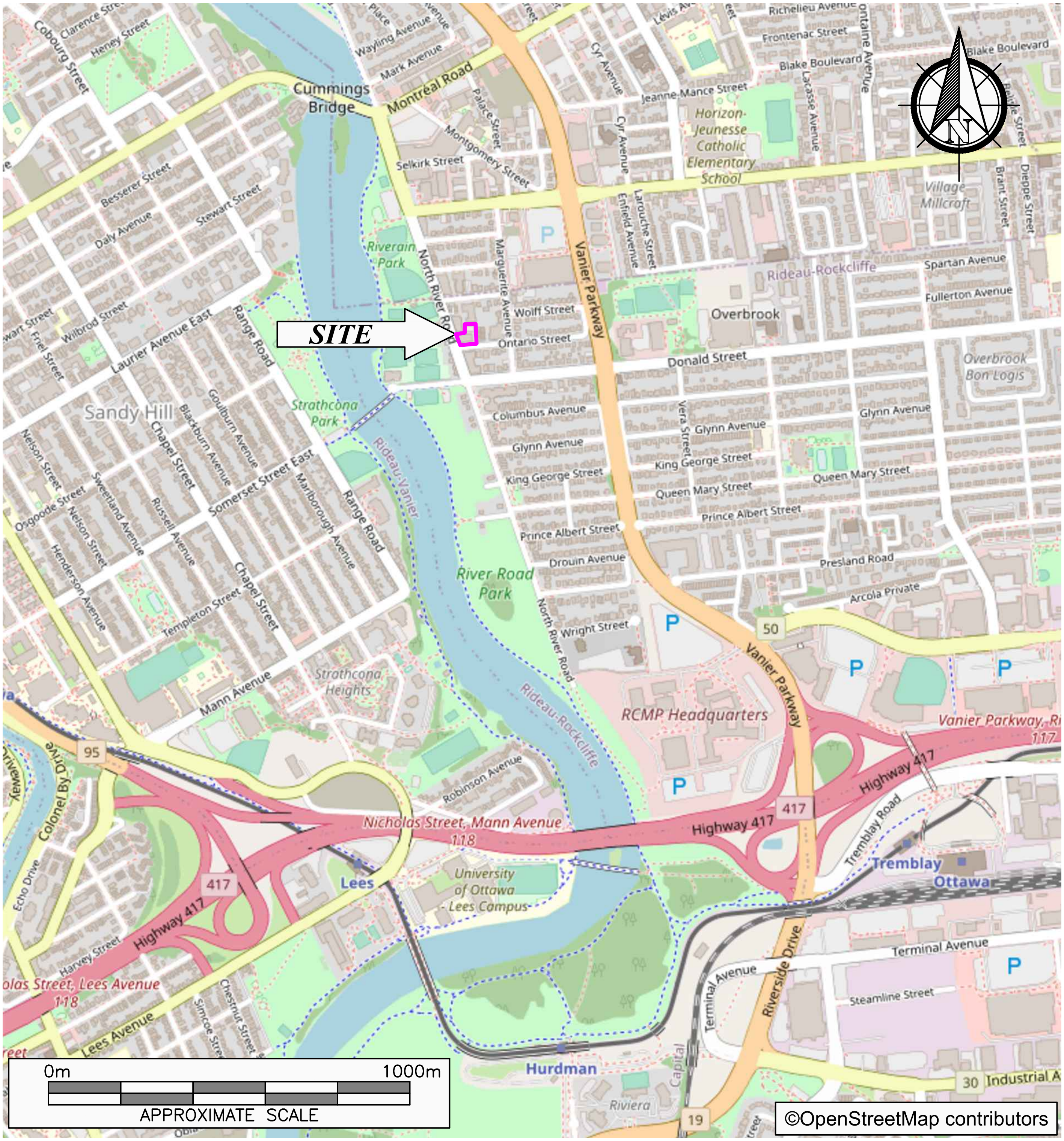
Specific limitations related to the legal and financial and limitations to the scope of the current work are outlined in our proposal, the attached Methodology, and the Authorization to Proceed, Limitation of Liability and Terms of Engagement which accompanied the proposal.

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.

283759.001 Geotechnical Investigation 949 N River Rd Ottawa ON Gemstone.docx

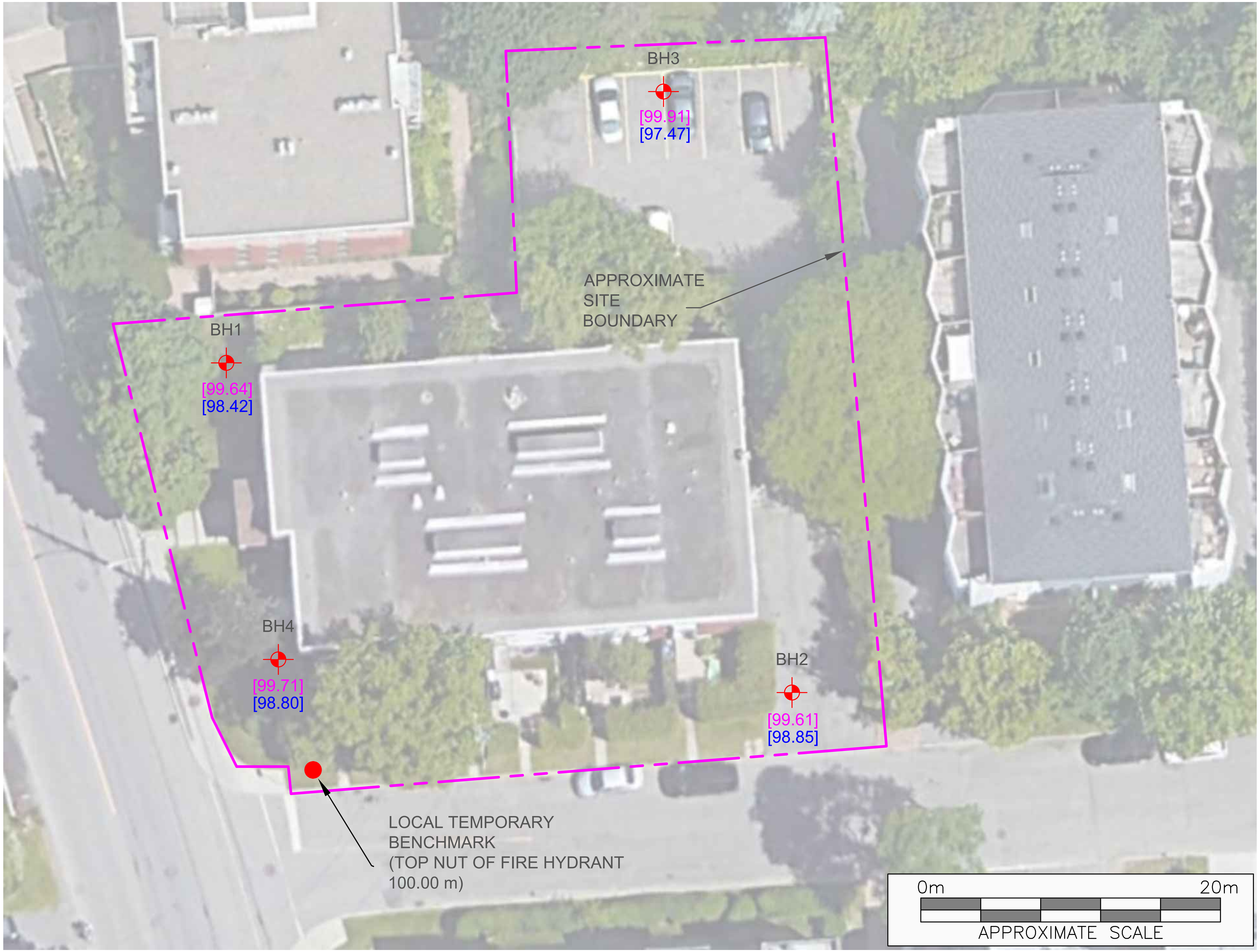
Template: Master Geotechnical Investigation Report – Ontario, GEO, April 1, 2020

FIGURES



PINCHIN

PROJECT NAME	GEOTECHNICAL INVESTIGATION
CLIENT NAME	GEMSTONE RIVER ROAD (GP) INC.
PROJECT LOCATION	949 NORTH RIVER ROAD, OTTAWA, ONTARIO
FIGURE NAME	KEY MAP
FIGURE NO.	1
APPROXIMATE SCALE	AS SHOWN
PROJECT NO.	283759.001
DATE	APRIL 2021



LEGEND

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

PINCHIN

PROJECT NAME	GEOTECHNICAL INVESTIGATION
CLIENT NAME	GEMSTONE RIVER ROAD (GP) INC.
PROJECT LOCATION	949 NORTH RIVER ROAD, OTTAWA, ONTARIO
FIGURE NAME	BOREHOLE LOCATION PLAN
APPROXIMATE SCALE	AS SHOWN
PROJECT NO.	283759.001
DATE	APRIL 2021
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:

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

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 #: 283759.001 **Logged By:** WT

Project: Geotechnical Investigation

Client: Gemstone River Road (GP) Inc.

Location: 949 North River Road, Ottawa, Ontario

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

SUBSURFACE PROFILE					SAMPLE							
Depth (m)	Symbol	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-values	SPT N-values (20, 40, 60) / Shear Strength kPa (50, 100, 150, 200)	Lab Analysis	Moisture (%)	Plasticity Index
		Ground Surface	99.64									
0		**Organics** ~150 mm	99.49	No Monitoring Well Installed								
		Fill Sand, some silt, damp, brown, loose	98.88		SS	1	40	6				
1		**Till** Silty sand, some gravel, some clay, moist, brown, very dense	98.42		SS	2	50	<50				
		End of Borehole										
2		Borehole terminated at approximately 1.22 m depth due to refusal on probable bedrock. Groundwater was not encountered at drilling completion.										
3												

Contractor: Strata Drilling Group **Grade Elevation:** 99.64 m

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

Well Casing Size: NA **Sheet:** 1 of 1

Log of Borehole: BH2

Project #: 283759.001 **Logged By:** WT

Project: Geotechnical Investigation

Client: Gemstone River Road (GP) Inc.

Location: 949 North River Road, Ottawa, Ontario

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

SUBSURFACE PROFILE					SAMPLE							
Depth (m)	Symbol	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-values	SPT N-values 20 40 60 / Shear Strength kPa 50 100 150 200	Lab Analysis	Moisture (%)	Plasticity Index
0		Ground Surface	99.61	No Monitoring Well Installed								
		Asphalt ~25mm										
		Fill Sand and gravel, some silt, damp, brown, compact	98.85		SS	1	80	16	■	G.S.		
		End of Borehole										
1		Borehole terminated at approximately 0.76 m depth due to refusal on probable bedrock. Groundwater was not encountered at drilling completion.										
2												
3												

Contractor: Strata Drilling Group **Grade Elevation:** 99.61 m

Drilling Method: Direct Push/Spilt Spoon **Top of Casing Elevation:** NA

Well Casing Size: NA **Sheet:** 1 of 1

Log of Borehole: BH3

Project #: 283759.001 **Logged By:** WT

Project: Geotechnical Investigation

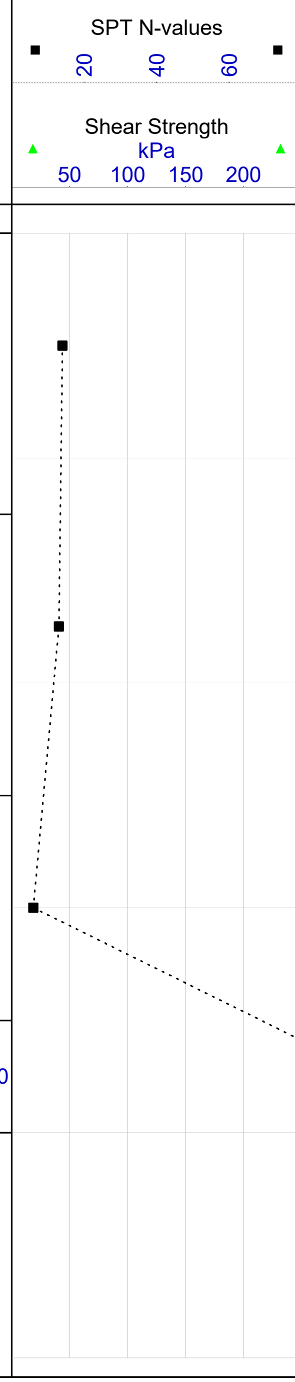
Client: Gemstone River Road (GP) Inc.

Location: 949 North River Road, Ottawa, Ontario

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

SUBSURFACE PROFILE / SAMPLE

Depth (m)	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-values	Lab Analysis	Moisture (%)	Plasticity Index
	Ground Surface	99.91								
0	**Asphalt** ~50 mm									
	Fill Sand and gravel, some silt, damp, brown, compact		Bentonite; Riser	SS	1	80	14			
1				SS	2	30	13			
		98.39								
	Till Silty sand, some gravel, some clay, damp to moist, brown, loose to very dense		Screen; Silica Sand	SS	3	80	6	Hyd.	21.1	
2				SS	4	70	>50			
		97.47								
3	End of Borehole Borehole terminated at approximately 2.44 m depth due to refusal on probable bedrock. Groundwater was not encountered within the monitoring well on April 19, 2021.									



Contractor: Strata Drilling Group

Drilling Method: Direct Push/Split Spoon

Well Casing Size: 50 mm

Grade Elevation: 99.91 m

Top of Casing Elevation: 99.81 m

Sheet: 1 of 1

Log of Borehole: BH4

Project #: 283759.001 **Logged By:** WT

Project: Geotechnical Investigation

Client: Gemstone River Road (GP) Inc.

Location: 949 North River Road, Ottawa, Ontario

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

Depth (m)	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-values	Lab Analysis	Moisture (%)	Plasticity Index
	Ground Surface	99.71								
0	**Organics** ~150 mm	99.56	No Monitoring Well Installed							
	Fill Sand, some silt, damp, brown, very loose	98.95		SS	1	60	1	G.S.		
	Till Silty sand, some gravel, some clay, damp, brown, very loose	98.80		SS	2	30	2			
1	End of Borehole									

SPT N-values: 20, 40, 60

Shear Strength kPa: 50, 100, 150, 200

Depth (m): 0, 1, 2, 3

Borehole terminated at approximately 0.91 m depth due to refusal on probable bedrock.
Groundwater was not encountered at drilling completion.

Contractor: Strata Drilling Group **Grade Elevation:** 99.71 m

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

Well Casing Size: NA **Sheet:** 1 of 1

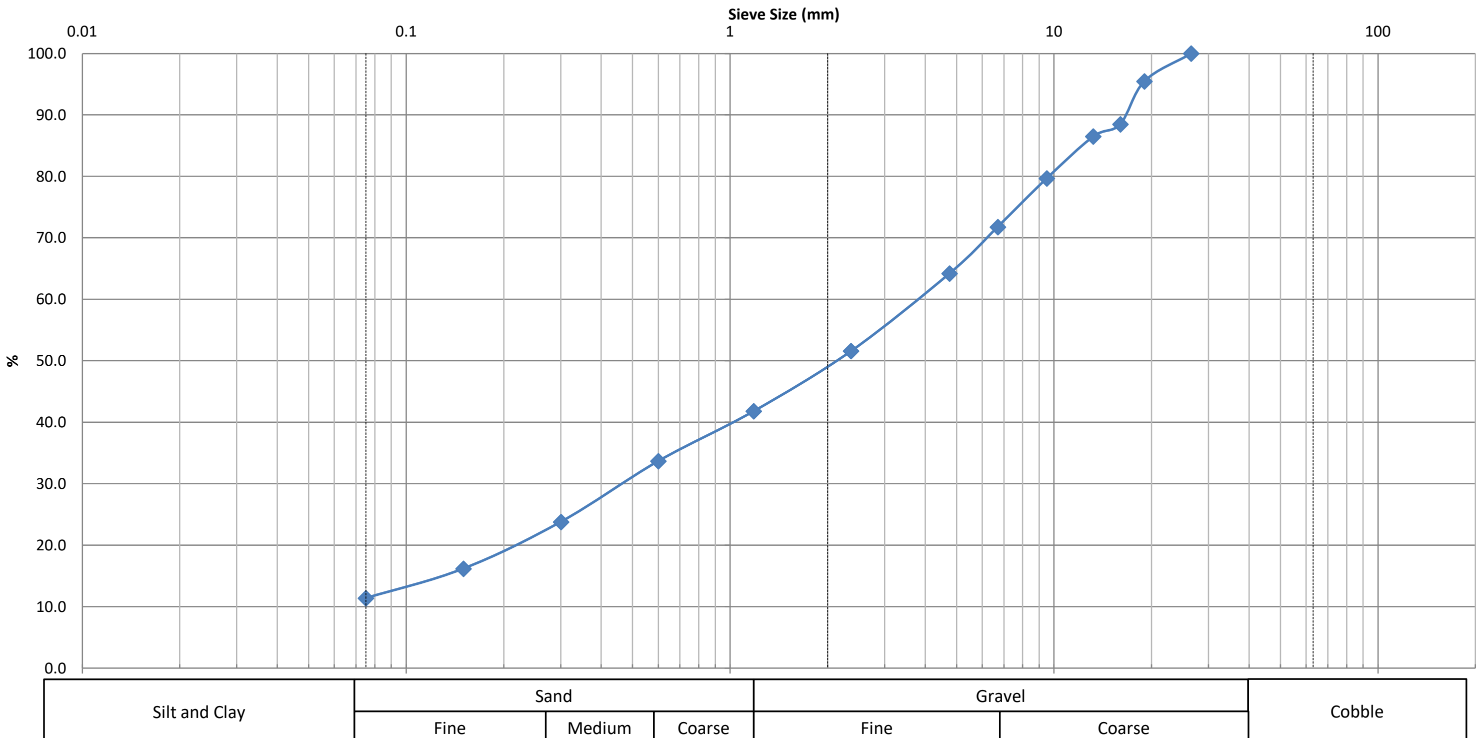
APPENDIX III

Laboratory Testing Reports for Soil Samples

patersongroup
consulting engineers

SIEVE ANALYSIS
ASTM C136

CLIENT:	Pinchin	DESCRIPTION:	Silty Sand w Gravel	FILE NO:	PM4184
CONTRACT NO.:	-	SPECIFICATION:	**Silty Sand w Gravel**	LAB NO:	23773
PROJECT:	283759.001	INTENDED USE:	-	DATE RECEIVED:	5-Apr-21
		PIT OR QUARRY:	-	DATE TESTED:	7-Apr-21
DATE SAMPLED:	5-Apr-21	SOURCE LOCATION:	BH2	DATE REPORTED:	9-Apr-21
SAMPLED BY:	Client	SAMPLE LOCATION:	0-2 '	TESTED BY:	DK



Identification	Soil Classification				MC(%)	LL	PL	PI	Cc	Cu
									0.91	60.0
	D100	D60	D30	D10	Gravel (%)	Sand (%)	Silt (%)		Clay (%)	
	26.5	3.9	0.48	0.065	35.8	52.8	11.4			

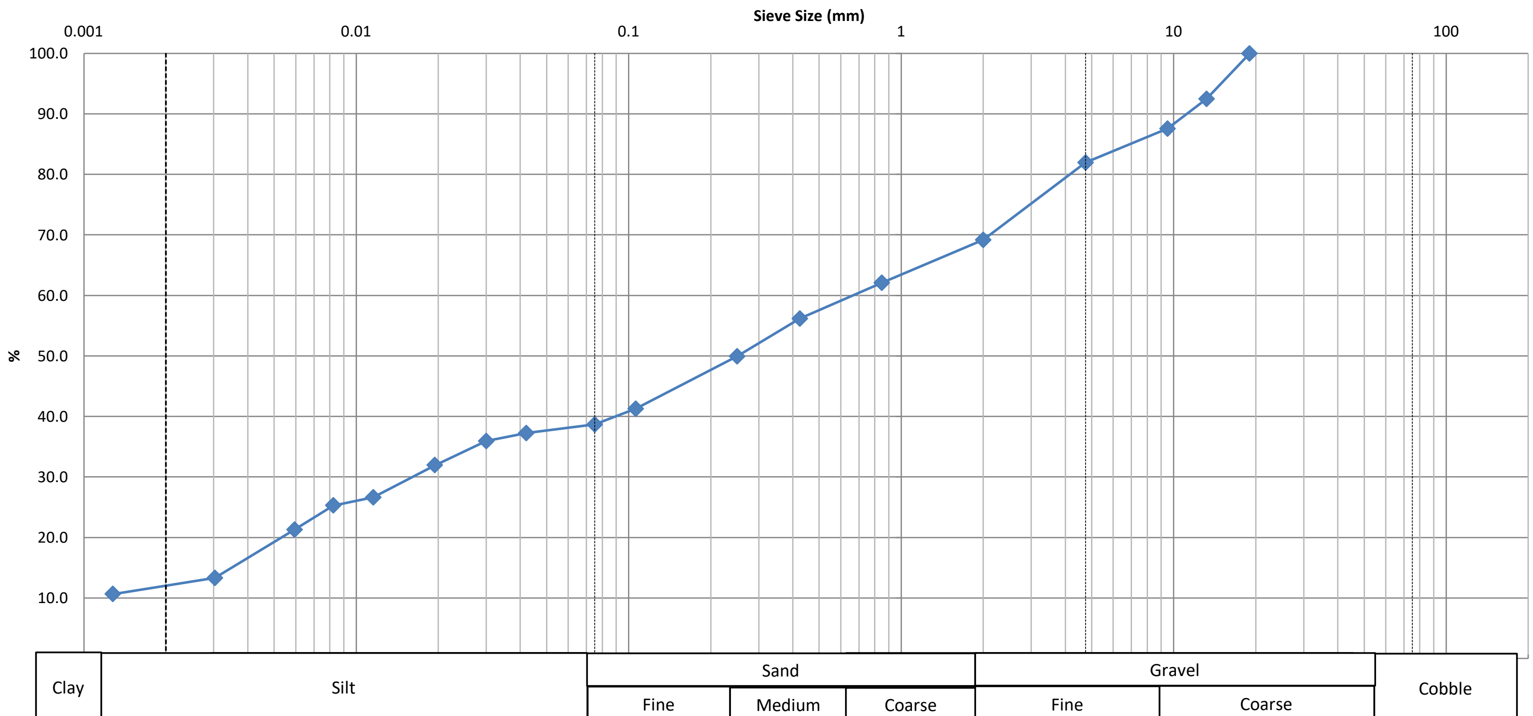
Comments:

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

patersongroup
consulting engineers

SIEVE ANALYSIS
ASTM C136

CLIENT:	Pinchin	DEPTH:	5 - 7 '	FILE NO:	PM4184
CONTRACT NO.:		BH OR TP No.:	BH3	LAB NO:	23772
PROJECT:	283759.001			DATE RECEIVED:	5-Apr-21
				DATE TESTED:	7-Apr-21
DATE SAMPLED:	5-Apr-21			DATE REPORTED:	9-Apr-21
SAMPLED BY:	Client			TESTED BY:	DB



Identification	Soil Classification				MC(%)	LL	PL	PI	Cc	Cu
					21.1					
	D100	D60	D30	D10	Gravel (%)	Sand (%)	Silt (%)	Clay (%)		
					18.1	43.2	27.2	11.5		

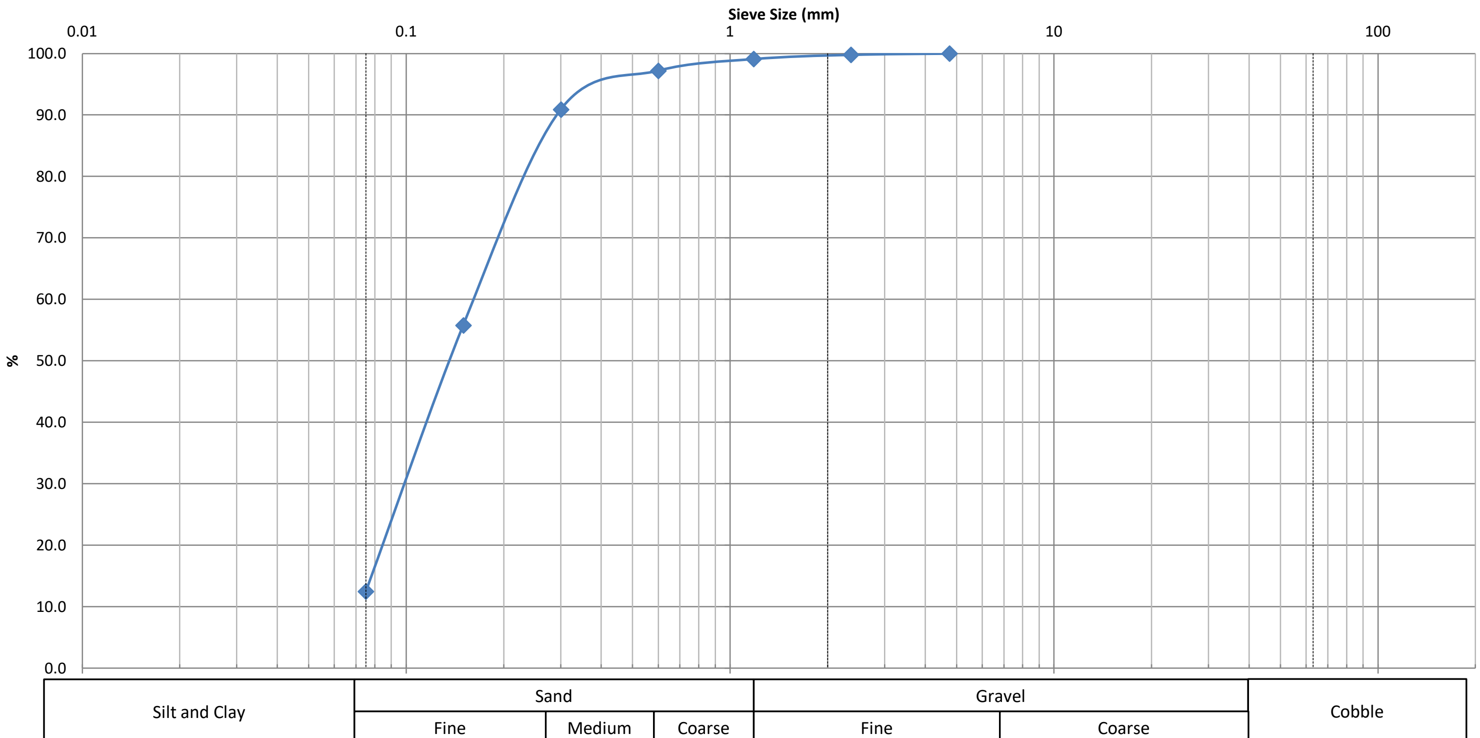
Comments:

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

patersongroup
consulting engineers

SIEVE ANALYSIS
ASTM C136

CLIENT:	Pinchin	DESCRIPTION:	Silty Sand	FILE NO:	PM4184
CONTRACT NO.:	-	SPECIFICATION:	**Silty Sand**	LAB NO:	23774
PROJECT:	283759.001	INTENDED USE:	-	DATE RECEIVED:	5-Apr-21
		PIT OR QUARRY:	-	DATE TESTED:	7-Apr-21
DATE SAMPLED:	5-Apr-21	SOURCE LOCATION:	BH4	DATE REPORTED:	9-Apr-21
SAMPLED BY:	Client	SAMPLE LOCATION:	0-2 '	TESTED BY:	DK



Identification	Soil Classification				MC(%)	LL	PL	PI	Cc	Cu
									0.82	2.5
	D100	D60	D30	D10	Gravel (%)	Sand (%)	Silt (%)		Clay (%)	
	4.75	0.17	0.098	0.069	0.0	87.5	12.5			

Comments:

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

PARACEL LABORATORIES LTD.
TRUSTED. RESPONSIVE. RELIABLE.

300 - 2319 St. Laurent Blvd
Ottawa, ON, K1G 4J8
1-800-749-1947
www.paracellabs.com

Certificate of Analysis

Pinchin Ltd. (Ottawa)

1 Hines Road, Suite 200
Kanata, ON K2K 3C7
Attn: Wes Tabaczuk

Client PO:
Project: 283759.001
Custody: 61862

Report Date: 30-May-2022
Order Date: 19-May-2022

Order #: 2221602

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

Paracel ID	Client ID
2221602-01	BH1, @ 3ft

Approved By:

Dale Robertson, BSc
Laboratory Director

Analysis Summary Table

Analysis	Method Reference/Description	Extraction Date	Analysis Date
Anions	EPA 300.1 - IC, water extraction	26-May-22	27-May-22
Conductivity	MOE E3138 - probe @25 °C, water ext	26-May-22	26-May-22
pH, soil	EPA 150.1 - pH probe @ 25 °C, CaCl buffered ext.	26-May-22	26-May-22
Resistivity	EPA 120.1 - probe, water extraction	26-May-22	26-May-22
Solids, %	Gravimetric, calculation	24-May-22	26-May-22

Certificate of Analysis

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

Client PO:

Report Date: 30-May-2022

Order Date: 19-May-2022

Project Description: 283759.001

	Client ID:	BH1, @ 3ft	-	-	-
	Sample Date:	17-May-22 12:00	-	-	-
	Sample ID:	2221602-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	85.7	-	-	-
General Inorganics					
Conductivity	5 uS/cm	166	-	-	-
pH	0.05 pH Units	7.46	-	-	-
Resistivity	0.10 Ohm.m	60.3	-	-	-
Anions					
Chloride	5 ug/g dry	<5	-	-	-
Sulphate	5 ug/g dry	35	-	-	-

Method Quality Control: Blank

Analyte	Result	Reporting Limit	Units	Source Result	%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	289	5	ug/g	278			3.9	20	
Sulphate	113	5	ug/g	111			1.8	20	
General Inorganics									
Conductivity	461	5	uS/cm	458			0.4	5	
pH	7.45	0.05	pH Units	7.46			0.1	2.3	
Resistivity	21.7	0.10	Ohm.m	21.8			0.4	20	
Physical Characteristics									
% Solids	84.7	0.1	% by Wt.	83.4			1.5	25	

Method Quality Control: Spike

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	376	5	ug/g	278	98.1	82-118			
Sulphate	218	5	ug/g	111	107	80-120			

Qualifier Notes:

Login Qualifiers :

Sample - One or more parameter received past hold time - Redox potential
Applies to samples: BH1, @ 3ft

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 when the units are denoted with 'dry'.
Where %Solids is reported, moisture loss includes the loss of volatile hydrocarbons.

PARACEL LABORATORIES L

Paracel ID: 2221602

lvd.
J8
.com

Paracel Order Number (Lab Use Only): 2221602

Chain Of Custody
(Lab Use Only)
№ 61862

Client Name: Pinchin Ltd.	Project Ref: 283759.001
Contact Name: Wes Tabaczuk	Quote #:
Address:	PO #:
	E-mail: wtabaczuk@pinchin.com
Telephone: 613-853-2211	

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	Corrosivity	Redox	Sulfides	Conductivity
1	BH1 @ 3ft	S		1	May.17/22	PM	X	X	X	X
2										
3										
4										
5										
6										
7										
8										
9										
10										

Comments:

Method of Delivery: Walkin

Relinquished By (Sign): W. Tabaczuk	Received By Driver/Depot: [signature]	Received at Lab: Sunnyporm Dhmai	Verified By: [signature]
Relinquished By (Print): Wes Tabaczuk	Date/Time: 05/19/22 4:39pm	Date/Time: May 20, 2022 12.43	Date/Time: May 20 2022 14:31
Date/Time: May. 19/22 PM	Temperature: 24.3 °C	Temperature: 9.4 °C	pH Verified: ☐ NA By:

PARACEL LABORATORIES LTD.
TRUSTED. RESPONSIVE. RELIABLE.

300 - 2319 St. Laurent Blvd
Ottawa, ON, K1G 4J8
1-800-749-1947
www.paracellabs.com

Subcontracted Analysis

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

Paracel Report No.	**2221602**	Order Date:	19-May-22
Client Project(s):	**283759.001**	Report Date:	31-May-22
Client PO:			
Reference:	**Standing Offer - ENV**		
CoC Number:	**61862**		

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
2221602-01	BH1, @ 3ft	Redox potential, soil Sulphide, solid

TESTMARK Laboratories Ltd.
Committed to Quality and Service

CERTIFICATE OF ANALYSIS

Client:	Dale Robertson	Work Order Number:	463731
Company:	Paracel Laboratories Ltd. - Ottawa	PO #:	
Address:	300-2319 St. Laurent Blvd.	Regulation:	[No Reg - Always Include Reg Report]
	Ottawa, ON, K1G 4J8	Project #:	2221602
Phone/Fax:	(613) 731-9577 / (613) 731-9064	DWS #:	
Email:	drobertson@paracellabs.com	Sampled By:	
Date Order Received:	5/25/2022	Analysis Started:	5/30/2022
Arrival Temperature:	16.8 °C	Analysis Completed:	5/30/2022

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
BH1, @ 3ft	1756999	Soil	None		5/17/2022	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

Non-Testmark container received 05/25/22 TJ
Sample received past hold time for Redox, proceed with analysis as per attached 05/25/22 TJ

This report has been approved by:

Marc Creighton
Laboratory Director

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

CERTIFICATE OF ANALYSIS

Paracel Laboratories Ltd. - Ottawa

Work Order Number: 463731

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

TESTMARK Laboratories Ltd.
Committed to Quality and Service

CERTIFICATE OF ANALYSIS

Paracel Laboratories Ltd. - Ottawa

Work Order Number: 463731

WORK ORDER RESULTS

Sample Description	BH1, @ 3ft
Sample Date	5/17/2022 12:00 PM
Lab ID	1756999

General Chemistry	Result	MDL	Units	Criteria: [No Reg - Always Include Reg Report]
RedOx (vs. S.H.E.)	386 [382]	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.

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

31-May-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. : 25 May 2022
LR Report: **CA15500-MAY22**
Reference: Project#: 2221602

Copy: #1

CERTIFICATE OF ANALYSIS

Final Report

Sample ID	Sample Date & Time	Sulphide (Na2CO3) %
1: Analysis Start Date		31-May-22
2: Analysis Start Time		13:29
3: Analysis Completed Date		31-May-22
4: Analysis Completed Time		14:03
5: QC - Blank		< 0.04
6: QC - STD % Recovery		104%
7: QC - DUP % RPD		0%
8: RL		0.02
9: BH1, @ 3ft	17-May-22 12:00	< 0.04

RL - SGS Reporting Limit

Kimberley Didsbury
Project Specialist,
Environment, Health & Safety

OnLine LIMS

0002919065

APPENDIX IV

Geophysics GPR International Inc. Shear Wave Velocity Sounding

GEOPHYSICS GPR INTERNATIONAL INC.

100 – 2545 Delorimier Street
Longueuil (Québec)
Canada J4K 3P7
Tel. : (450) 679-2400
Fax : (514) 521-4128
info@geophysicsgpr.com
www.geophysicsgpr.com

March 21st, 2022

Transmitted by email: wtabaczuk@pinchin.com
Our Ref.: GPR-22-03655

Mr. Wesley Tabaczuk, P.Eng.
Project Manager, Geotechnical
Pinchin Ltd.
200 – 1 Hines Road
Kanata ON K2K 3C7

Subject: **Shear Wave Velocity Sounding for the Site Class Determination**
949 North River Road, Ottawa (ON)

[Project: 283759.001]

Dear Sir,

Geophysics GPR International inc. has been mandated by Pinchin Ltd. to carry out seismic shear wave surveys at 949 North River Road, in Ottawa (ON). The geophysical investigation used the Multi-channel Analysis of Surface Waves (MASW), the Spatial AutoCorrelation (SPAC), and the seismic refraction methods. From the subsequent results, the seismic shear wave velocity values were calculated for the soil and the rock, to determine the Site Class.

The surveys were carried out on March 10th, 2022, by Mrs. Karyne Faguy, B.Sc. geoph. and Mr. Timothy Ward, tech. Figure 1 shows the regional location of the site and Figure 2 illustrates the location of the seismic spreads. Both figures are presented in the Appendix.

The following paragraphs briefly describe the survey design, the principles of the testing methods, and the results presented in tables and graphs.

MASW PRINCIPLE

The *Multi-channel Analysis of Surface Waves* (MASW) and the *SPatial AutoCorrelation* (SPAC or MAM for *Microtremors Array Method*) are seismic methods used to evaluate the shear wave velocities of subsurface materials through the analysis of the dispersion properties of the Rayleigh surface wave. The MASW is considered an "active" method, as the seismic signal is induced at known location and time in the geophones' spread axis. Conversely, the SPAC is considered a "passive" method, using the low frequency "signals" produced far away. The method can also be used with "active" seismic source records. The SPAC method allows deeper Vs soundings, but generally with a lower resolution for the surface portion. Its dispersion curve can then be merged with the one of higher frequency from the MASW to calculate a more complete inversion. The dispersion properties are expressed as a change of velocities with respect to frequencies. Surface wave energy will decay exponentially with depth. Lower frequency surface waves will travel deeper and thus be more influenced by deeper velocity layering than the shallow higher frequency waves. The inversion of the Rayleigh wave dispersion curve yields a shear wave (V_S) velocity depth profile (sounding).

Figure 3 schematically outlines the basic operating procedure for the MASW method. Figure 4 illustrates an example of one of the MASW/SPAC records, the corresponding spectrogram analysis and resulting 1D V_S model.

INTERPRETATION

The main processing sequence involved data inspection and edition when required; spectral analysis ("phase shift" for MASW, and "cross-correlation" for SPAC); picking the fundamental mode; and 1D inversion of the MASW and SPAC shot records using the SeisImagerSW™ software. The data inversions used a nonlinear least squares algorithm.

In theory, all the shot records for a given seismic spread should produce a similar shear-wave velocity profile. In practice, however, differences can arise due to energy dissipation, local surface seismic velocities variations, and/or dipping of overburden layers or rock. In general, the precision of the calculated seismic shear wave velocities (V_S) is of the order of 15% or better.

More detailed descriptions of these methods are presented in *Shear Wave Velocity Measurement Guidelines for Canadian Seismic Site Characterization in Soil and Rock*, Hunter, J.A., Crow, H.L., et al., Geological Surveys of Canada, General Information Product 110, 2015.

SURVEY DESIGN

The seismic acquisition spreads were laid along North River Road, north of the corner with Ottawa Street (Figure 2). The geophone spacing was of 2.0 metres for the main spread, using 24 geophones. A shorter seismic spread, with geophone spacing of 0.5 metre, was dedicated to the near surface materials. The seismic records were produced with a seismograph Terraloc Pro 2 (from ABEM Instrument), and the geophones were 4.5 Hz. The seismic records counted 4096 data, sampled at 1000 µs for the MASW surveys, and 40 µs for the seismic refraction. The records included a pre-trigged portion of 10 ms. An 8 kg sledgehammer was used as the energy source with impacts being recorded off both ends of the seismic spreads. A stacking procedure was also used to improve the Signal / Noise ratio for the seismic records.

The shear wave depth sounding can be considered as the average of the bulk area within the geophone spread, especially for its central half-length.

RESULTS

Using seismic refraction (V_P) the rock depth was calculated between 0.9 and 1.6 metres (± 1 metre). Its calculated seismic velocity (V_S) was 1545 m/s for its shallow portion.

The MASW calculated V_S results are illustrated at Figure 5.

The $\overline{V}_{S30}$ value results from the harmonic mean of the shear wave velocities, from the surface to 30 metres deep. It is calculated by dividing the total depth of interest (30 metres) by the sum of the time spent in each velocity layer from the surface down to 30 metres, as:

$$\bar{V}_{S30} = \frac{\sum_{i=1}^{N} \mathrm{H}_i}{\sum_{i=1}^{N} {H_i}/{V_i}} \quad | \quad \sum_{i=1}^{N} \mathrm{H}_i \ = \ 30 \text{ m}$$

(N: number of layers; H_i : thickness of layer "i" ; V_i : $\mathrm{V_S}$ of layer "i")

Thus, the $\overline{V}_{S30}$ value represents the seismic shear wave velocity of an equivalent homogeneous single layer response, between the surface and 30 metres deep.

The calculated $\overline{V}_{S30}$ value of the actual site is 1430.2 m/s (Table 1), corresponding to the Site Class "B". In the case there would be 1.1 metres or less of unconsolidated materials between the rock and the bottom of the foundations, the $\overline{V}_{S30}$* value would be greater than 1500 m/s, allowing to use the Site Class A (Table 2).

CONCLUSION

Geophysical surveys were carried out to identify the Site Class at 949 North River Road, in Ottawa (ON). The seismic surveys used the MASW and the SPAC analysis, and the seismic refraction to calculate the $\overline{V}_{S30}$ value. Its calculation is presented at Table 1.

The $\overline{V}_{S30}$ value of the actual site is 1430 m/s, corresponding to the Site Class "B" ($760 < \overline{V}_{S30} \leq 1500$ m/s), as determined through the MASW and SPAC methods, Table 4.1.8.4.A of the NBC, and the Building Code, O. Reg. 332/12.

In the event there would be 1.1 metres or less of unconsolidated materials between the rock and the bottom of the spread footing or mat foundation, the $\overline{V}_{S30}$* value would be greater than 1500 m/s, allowing to use the Site Class "A".

It must be noted that other geotechnical information gleaned on site; including the presence of liquefiable soils, very soft clays, high moisture content etc. (cf. Table 4.1.8.4.A of the NBC) can supersede the Site classification provided in this report based on the $\overline{V}_{S30}$ value.

The V_S values calculated are representative of the in situ materials and are not corrected for the total and effective stresses.

Hoping the whole to your satisfaction, we remain yours truly,

Jean-Luc Arsenault, M.A.Sc., P.Eng.
Senior Project Manager

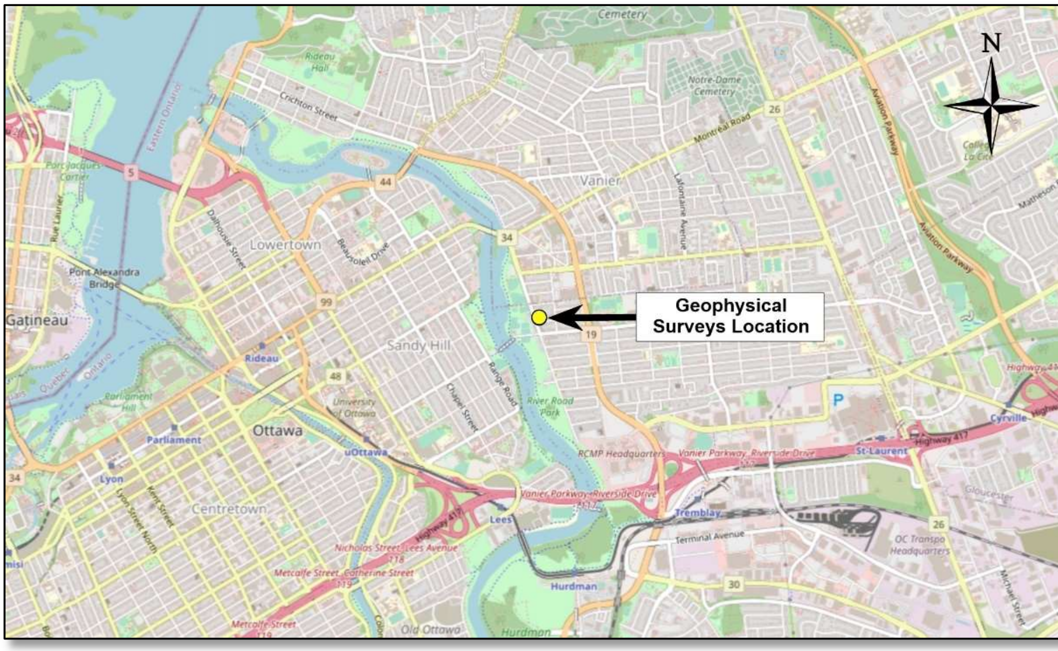


Figure 1: Regional location of the Site
(source: *OpenStreetMap*©)

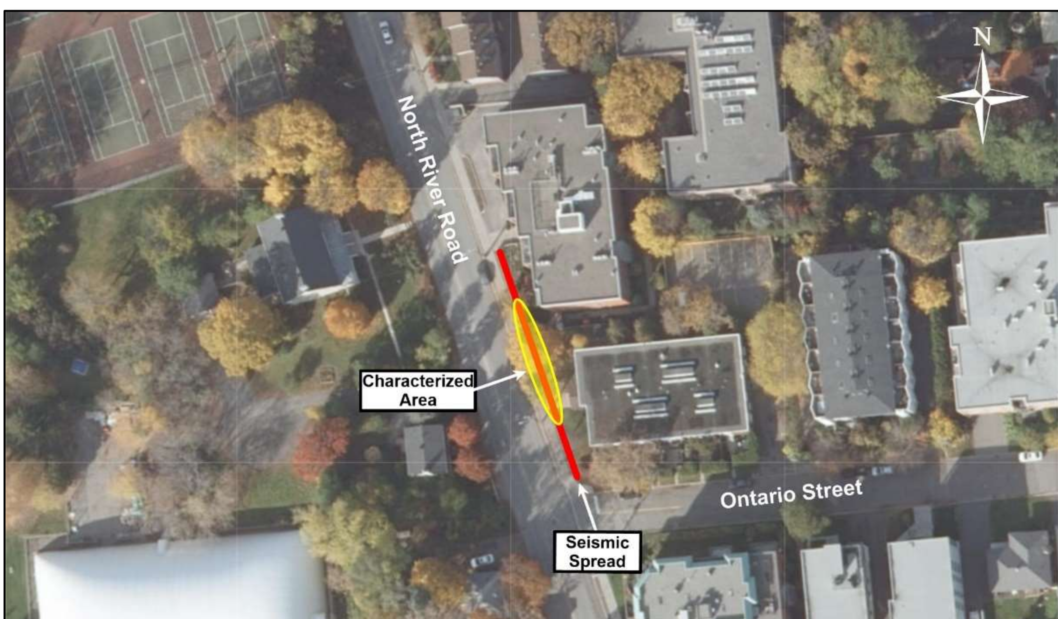


Figure 2: Location of the seismic spreads
(source: *geoOttawa*)

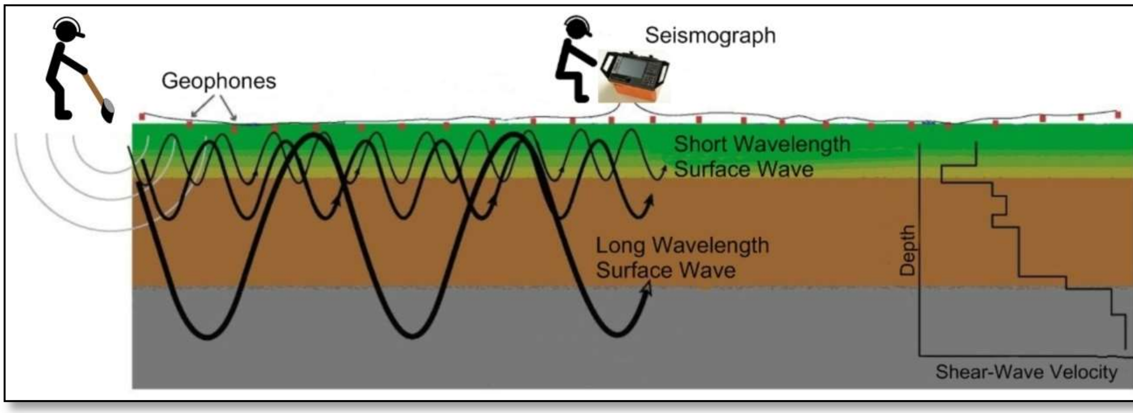


Figure 3: MASW Operating Principle

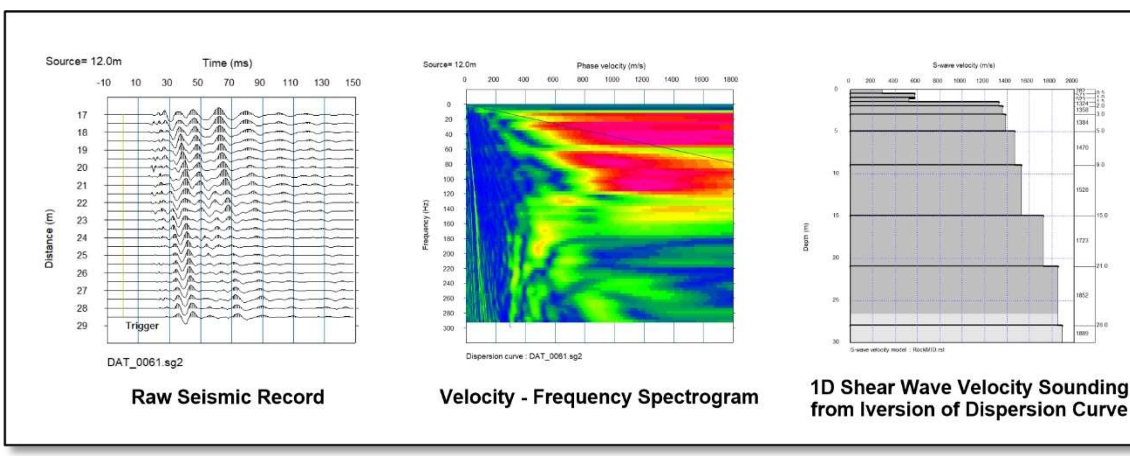


Figure 4: Example of a MASW/SPAC record, Phase Velocity - Frequency curve of the Rayleigh wave and resulting 1D Shear Wave Velocity Model

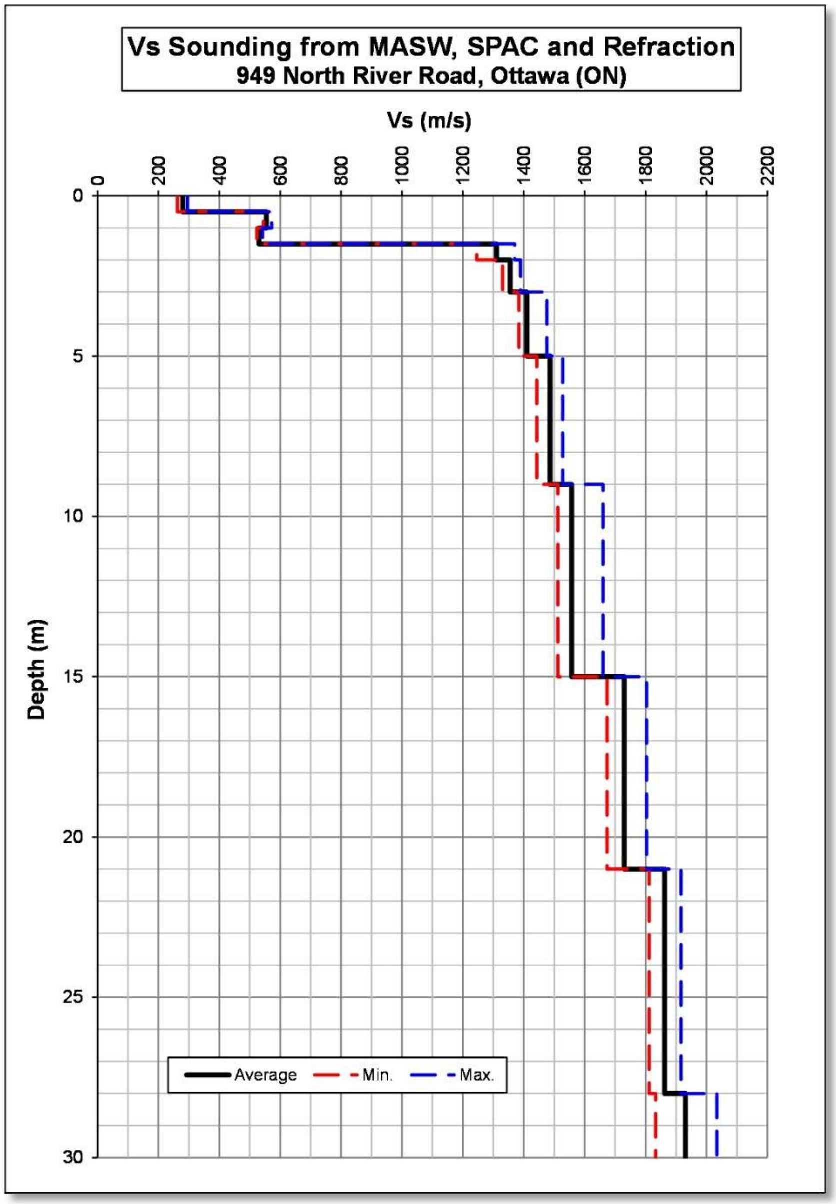


Figure 5: MASW Shear-Wave Velocity Sounding

TABLE 1

V_{S30} Calculation for the Site Class (actual site)

Depth	Vs			Thickness	Cumulative Thickness	Delay for avg. Vs	Cumulative Delay	Vs at given Depth
	Min.	Average	Max.					
(m)	(m/s)	(m/s)	(m/s)	(m)	(m)	(s)	(s)	(m/s)
0	262.4	279.8	295.0	**Grade Level (March 10th, 2022)**				
0.5	544.7	554.7	571.7	0.50	0.50	0.001787	0.001787	279.8
1.0	523.7	529.8	541.8	0.50	1.00	0.000901	0.002688	372.0
1.5	1245.5	1310.0	1370.5	0.50	1.50	0.000944	0.003632	413.0
2.0	1330.6	1355.8	1389.8	0.50	2.00	0.000382	0.004014	498.3
3.0	1384.1	1410.2	1475.7	1.00	3.00	0.000738	0.004751	631.4
5.0	1442.9	1486.4	1527.9	2.00	5.00	0.001418	0.006170	810.4
9.0	1511.6	1557.0	1660.2	4.00	9.00	0.002691	0.008861	1015.7
15.0	1673.4	1730.1	1803.0	6.00	15.00	0.003854	0.012714	1179.8
21.0	1812.0	1862.7	1916.3	6.00	21.00	0.003468	0.016182	1297.7
28.0	1833.2	1931.1	2034.1	7.00	28.00	0.003758	0.019940	1404.2
30				2.00	30.00	0.001036	0.020976	1430.2

V_{S30} (m/s)	**1430.2**
Class	**B**

TABLE 2

Limit for the Site Class A (1.1 metres of soils)

Depth	Vs			Thickness	Cumulative Thickness	Delay for Avg. Vs	Cumulative Delay	Vs at given Depth
	Min.	Average	Max.					
(m)	(m/s)	(m/s)	(m/s)	(m)	(m)	(s)	(s)	(m/s)
0	262.4	279.8	295.0	**Limit for the Site Class A (1.1 metres of unconsolidated materials)**				
0.4	262.4	279.8	295.0					
0.5	544.7	554.7	571.7	0.10	0.10	0.000357	0.000357	279.8
1.0	523.7	529.8	541.8	0.50	0.60	0.000901	0.001259	476.6
1.5	1245.5	1310.0	1370.5	0.50	1.10	0.000944	0.002203	499.4
2.0	1330.6	1355.8	1389.8	0.50	1.60	0.000382	0.002584	619.1
3.0	1384.1	1410.2	1475.7	1.00	2.60	0.000738	0.003322	782.7
5.0	1442.9	1486.4	1527.9	2.00	4.60	0.001418	0.004740	970.4
9.0	1511.6	1557.0	1660.2	4.00	8.60	0.002691	0.007431	1157.3
15.0	1673.4	1730.1	1803.0	6.00	14.60	0.003854	0.011285	1293.8
21.0	1812.0	1862.7	1916.3	6.00	20.60	0.003468	0.014753	1396.4
28.0	1833.2	1931.1	2034.1	7.00	27.60	0.003758	0.018511	1491.0
30.4				2.40	30.00	0.001243	0.019754	1518.7

V_{S30}* (m/s)	**1518.7**
Class	**A**

APPENDIX V

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.