FINAL

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

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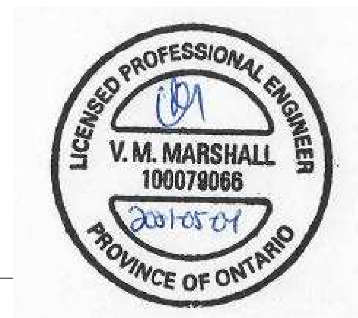

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1.0 INTRODUCTION AND SCOPE

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. At this time the depth to the underside of the footings for the parking garage is unknown; as such, for the purpose of this report, Pinchin has assumed a depth of approximately 3.0 metres below existing ground surface (mbgs) to the underside of the footing for the proposed underground parking garage. It is noted that due to the parking garage occupying the entire Site footprint, no service trenches or 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;
- 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; 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. At this time the depth to the underside of the footings for the parking garage is unknown; as such, for the purpose of this report, Pinchin has assumed a depth of approximately 3.0 mbgs to the underside of the footing for the proposed underground parking garage.

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 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 in order to prevent potential damage to the surrounding environment.

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

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

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

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

Minor 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 Foundation Design

5.4.1 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.

5.4.2 Site Classification for Seismic Site Response & Soil Behaviour

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

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

The boreholes advanced at this Site extended to a maximum depth of approximately 2.4 mbgs where refusal was encountered on inferred bedrock. SPT “N” values within the soil deposit ranged between 1 and greater than 50 blows per 300 mm. As such, based on Table 4.1.8.4.A of the OBC, this Site has been classified as Class C. A Site Class C has an average shear wave velocity (Vs) of between 360 and

760 m/s. It is recommended that shear wave velocity soundings be completed at the Site once the final design and depths of foundations are known as a higher Site Classification may be available.

5.4.3 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.

5.4.4 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.

5.4.5 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).

5.4.6 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.

5.5 Underground Parking Garage Design

It is understood that the building will be constructed with a single-level underground parking garage; however, at this time the final grades for the underside of the underground parking garage footings are unknown. 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.

5.5.1 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 SITE SUPERVISION & QUALITY CONTROL

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

7.0 TERMS AND LIMITATIONS

This Geotechnical Investigation was performed for the exclusive use of 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 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.

283759.001 Geotechnical Investigation 949 N River Rd Ottawa ON Gemstone

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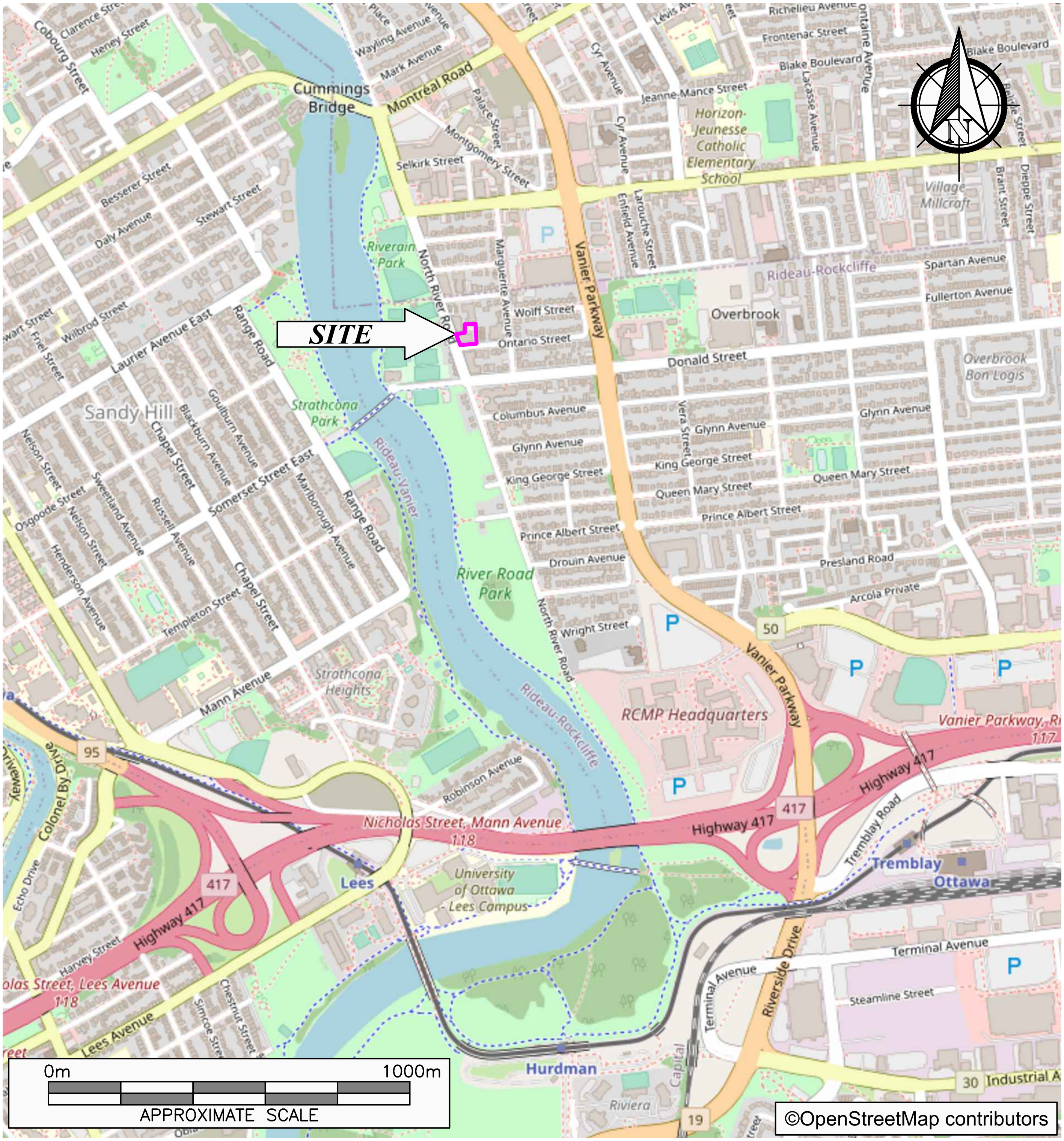
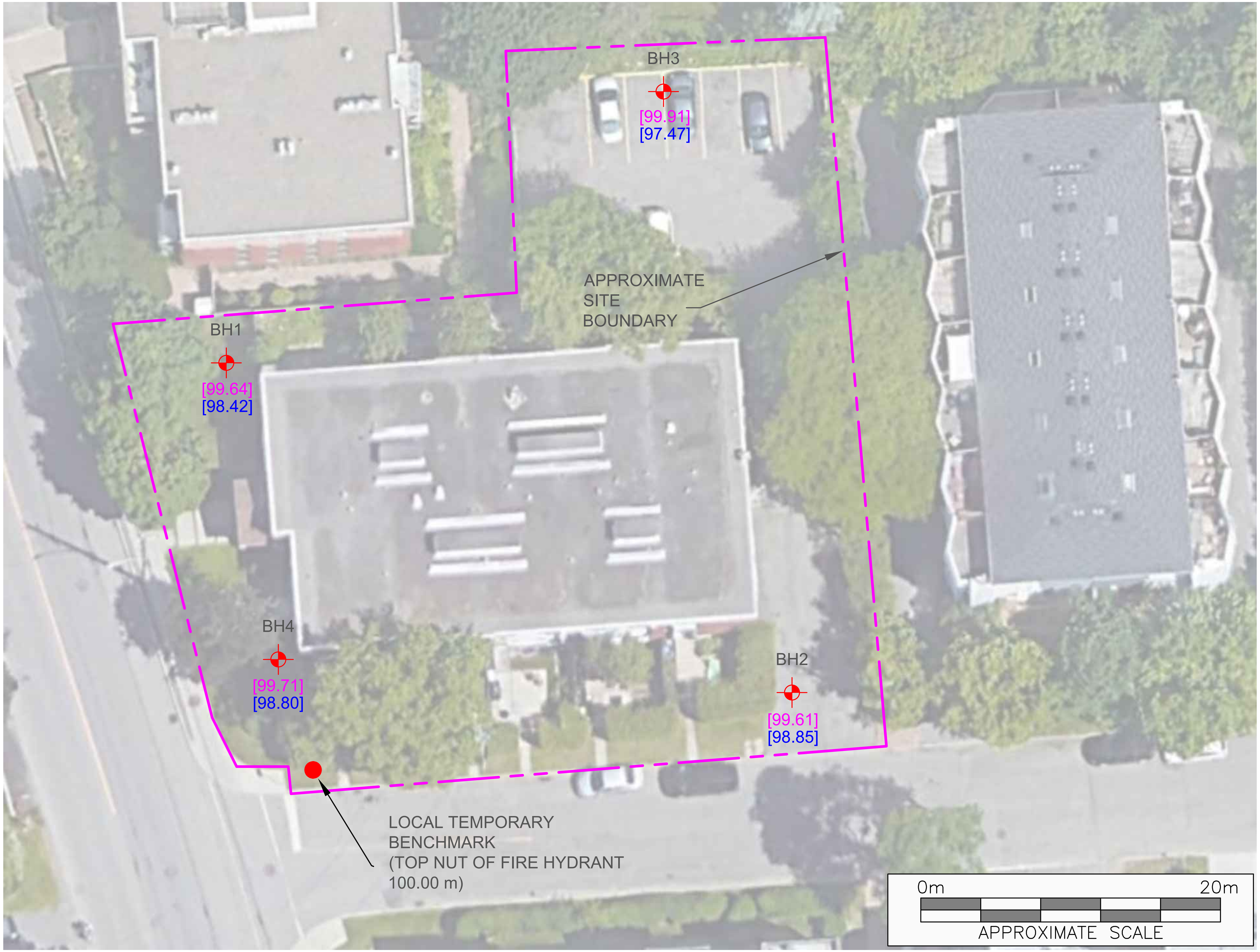
FIGURES



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:

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

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 #: 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										
		Borehole terminated at approximately 1.22 m depth due to refusal on probable bedrock. Groundwater was not encountered at drilling completion.										
2												
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
		Ground Surface	99.61	No Monitoring Well Installed								
0		**Asphalt** ~25mm										
		Fill Sand and gravel, some silt, damp, brown, compact			SS	1	80	16	■	G.S.		
			98.85									
		End of Borehole Borehole terminated at approximately 0.76 m depth due to refusal on probable bedrock. Groundwater was not encountered at drilling completion.										
1												
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

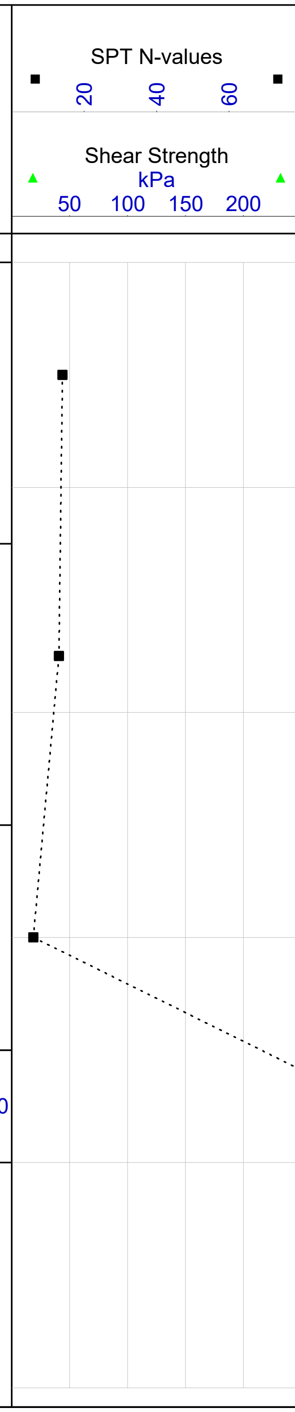
SUBSURFACE PROFILE

Depth (m)	Description	Elevation (m)
	Ground Surface	99.91
0	***Asphalt*** ~50 mm	
	Fill Sand and gravel, some silt, damp, brown, compact	
		98.39
	Till Silty sand, some gravel, some clay, damp to moist, brown, loose to very dense	
		97.47
	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.	

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

SAMPLE

Sample Type	Sampler #	Recovery (%)	SPT N-values	Lab Analysis	Moisture (%)	Plasticity Index
SS	1	80	14			
SS	2	30	13			
SS	3	80	6	Hyd.	21.1	
SS	4	70	>50			



Contractor: Strata Drilling Group **Grade Elevation:** 99.91 m

Drilling Method: Direct Push/Split Spoon **Top of Casing Elevation:** 99.81 m

Well Casing Size: 50 mm **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

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	99.71									
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										
		Borehole terminated at approximately 0.91 m depth due to refusal on probable bedrock. Groundwater was not encountered at drilling completion.										
2												
3												

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

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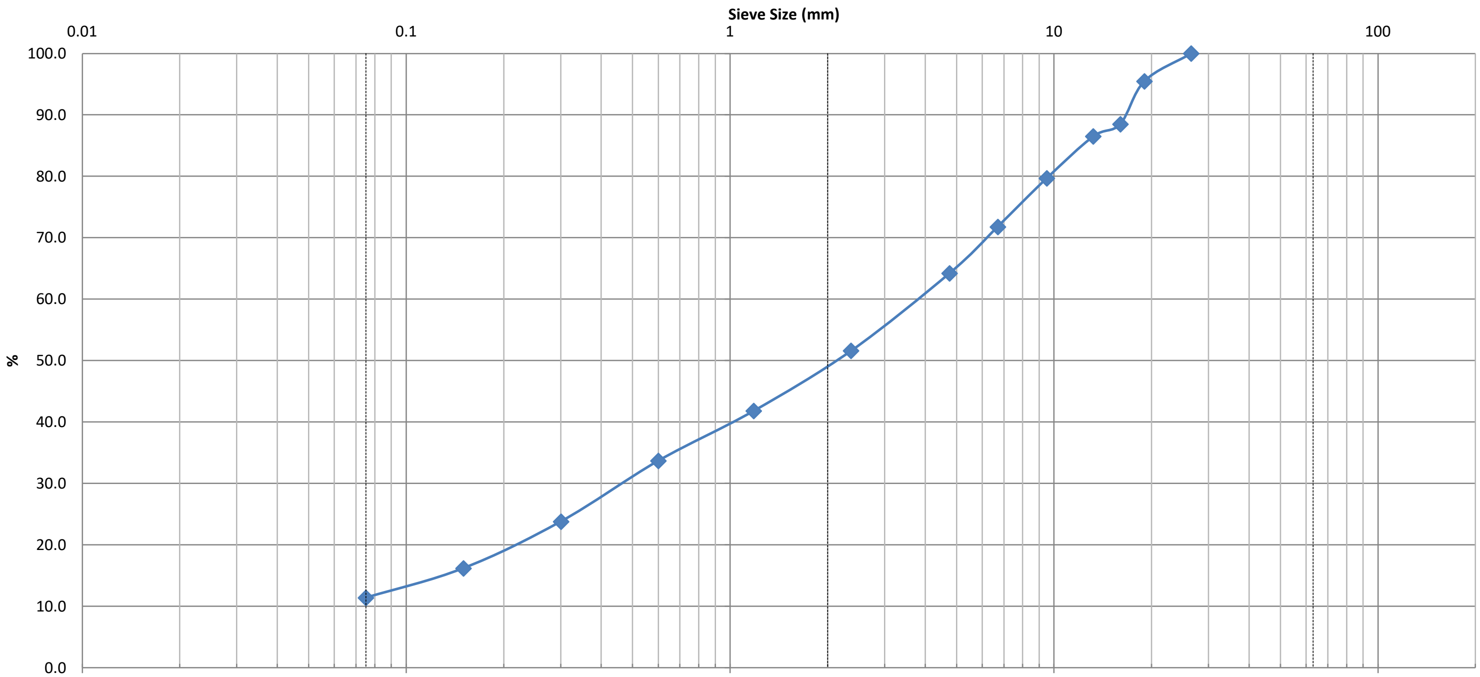
consulting engineers

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



Silt and Clay	Sand			Gravel		Cobble
	Fine	Medium	Coarse	Fine	Coarse	

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.**

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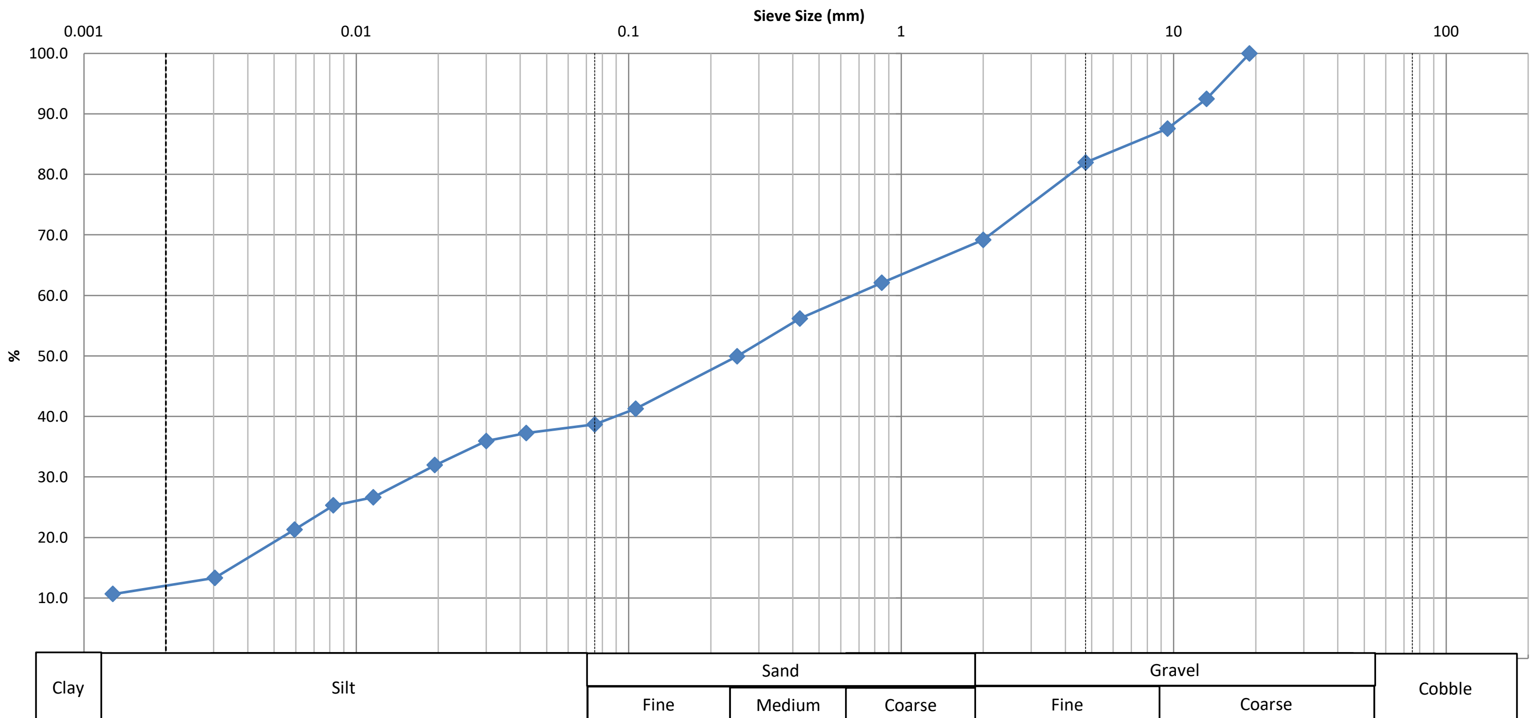
consulting engineers

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	

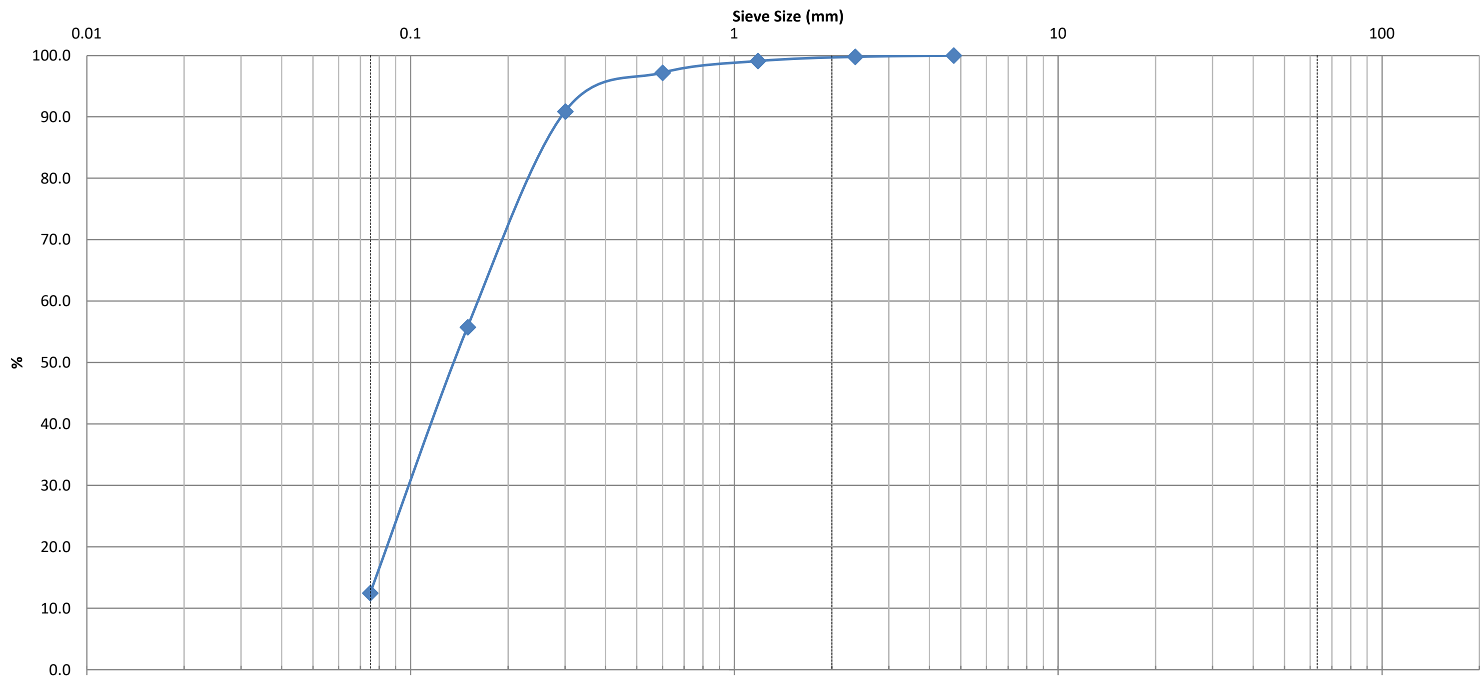
Comments:

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



Silt and Clay	Sand			Gravel		Cobble
	Fine	Medium	Coarse	Fine	Coarse	

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.

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.