FINAL

Geotechnical Investigation – Proposed Residential Development

90 Champagne Avenue South
Ottawa, Ontario

Prepared for:

Loretta Apartments Inc. c/o District Realty
50 Bayswater Avenue
Ottawa, ON K1Y 2E9

Attn: Kelly Kerrigan

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Issued to: Loretta Apartments Inc. c/o District Realty
Contact: Kelly Kerrigan
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Issuing Office: 1 Hines Road, Suite 200, Kanata, ON K2K 3C7
Primary Contact: Wesley Tabaczuk, P.Eng.
Project Manager, Geotechnical Services

W. Tabaczuk

LICENSED PROFESSIONAL ENGINEER
W. Tabaczuk
W.M. TABACZUK
100171927
Mar. 22/19
PROVINCE OF ONTARIO

Author: Wesley Tabaczuk, P.Eng.
Project Manager, Geotechnical Services
613.592.3387 ext.1829
wtabaczuk@pinchin.com

Reviewer: Vanessa Marshall, M.Eng., P. Eng.
National Practice Leader, Geotechnical Services
519.746.4210 ext. 3756
vmarshall@pinchin.com

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1.0 INTRODUCTION

Pinchin Ltd. (Pinchin) was retained by Loretta Apartments Inc. c/o District Realty(Client) to conduct a Geotechnical Investigation and provide subsequent geotechnical design recommendations for the proposed residential development to be located at 90 Champagne Avenue South, 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 14-storey multi-tenant residential building, complete with two levels of underground parking. At the time of this report 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 an approximate depth to the underside of the footings of 8.0 metres below the existing ground surface (mbgs).

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 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 review of relevant area geology and Site background information;
- A detailed description of the observed soil, bedrock and groundwater conditions;
- 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 recommendations; and
- Interior concrete floor slab-on-grade (including modulus of subgrade reaction).

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

2.0 SITE DESCRIPTION AND GEOLOGICAL SETTING

The Site is located on the west side of Champagne Avenue South, approximately 250 m north of Carling Avenue in Ottawa, Ontario. The Site currently consists of an asphalt paved parking lot. The lands adjacent to the Site are developed with a combination of multi-storey residential buildings and single family residential dwellings.

Data obtained from the Ontario Geological Survey Maps, as published by the Ontario Ministry of Natural Resources, indicates that the Site is located on Paleozoic terrain consisting of sandy silt to silty sand textured till. The underlying bedrock at this Site is of the Shadow Lake Formation consisting of limestone, dolostone, shale, arkose and sandstone (Ontario Geological Survey Map 1972, published 1978).

3.0 GEOTECHNICAL FIELD INVESTIGATION AND METHODOLOGY

Pinchin completed a field investigation at the Site on February 22, 2019 by advancing a total of four sampled boreholes (Boreholes BH1 to BH4) throughout the Site. The boreholes were advanced to sampled depths ranging from approximately 1.5 to 3.4 mbgs, where refusal was encountered on bedrock. In addition, a 3.0 m long bedrock core with NQ sized diamond bit core barrel was advanced at the base of Borehole BH1 to confirm the presence of bedrock and to evaluate the Rock Quality Designation (RQD). 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 track mounted mobile 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.

The bedrock cores were advanced in accordance with ASTM D2113. The bedrock types and RQD’s were evaluated immediately upon core retrieval.

A monitoring well was installed within Borehole BH1 to allow for measurement of groundwater levels. The monitoring wells were constructed using flush-threaded 50 mm diameter Trilock pipe with 3.0 meter long 10-slot well screens, delivered to the Site in pre-cleaned individually sealed plastic bags. The screen and riser pipes were not allowed to come into contact with the ground or drilling equipment prior to installation.

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

Groundwater observations and measurements were obtained from the open boreholes during and upon completion of drilling. Groundwater levels were measured in the newly installed monitoring well on March 13, 2019. The groundwater observations and measurements recorded are included on the appended borehole logs.

The boreholes locations and ground surface elevations were surveyed by Pinchin using a Stonex Model 900A Global Navigation Satellite System (GNSS) rover. The ground surface elevations are geodetic, based on GNSS and local base station telemetry with a precision static of less than 20 mm.

The field investigation was monitored by experienced Pinchin personnel. Pinchin logged the drilling operations and identified the soil samples and rock cores 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, and the results are presented 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 and Bedrock Lithology

In general, the soil stratigraphy at the Site consists of surficial asphaltic concrete and fill overlying glacial till and bedrock to the maximum borehole refusal depth of approximately 3.4 mbgs. It is noted that approximately 600 mm of frozen soil was encountered within all boreholes.

All of the boreholes were advanced through the existing pavement structure. The surficial asphalt and granular fill material was observed to be between approximately 0.5 and 0.8 m thick. The fill generally comprised sand and gravel containing trace silt. The unfrozen material was generally damp at the time of sampling.

The glacial till deposit was encountered within all boreholes underlying the existing pavement structure and extended down to the bedrock surface. The glacial till ranged from a sandy, silty gravel containing trace clay to a silty sand containing trace gravel and trace clay. The glacial till has a compact to very dense relative density based on SPT 'N' values of 14 to greater than 50 blows per 300 mm penetration of a split spoon sampler. The results of three particle size distribution analyses performed on samples of the till material indicate that the samples contain 8 to 48% gravel, 25 to 51% sand, 11 to 32% silt, and 4 to 9% clay sized particles. The moisture content of the samples tested ranged from 6.6 to 10.6% indicating the material was in a damp to moist state at the time of sampling.

The bedrock cores recovered consisted of limestone bedrock that was faintly weathered. The bedrock was grey with black banding, fine to medium grained, and contained few natural fractures with little to no oxidation. The bedrock at the fracture locations was mostly sharp and angular, which indicates minor water migration. Natural fractures were closely to moderately spaced, and were generally found to occur in sets oriented at approximately 45 to 90º to the core axis. An approximate 95% wash return within the rock cores was observed. The wash return was milky white in colour. The rock core recovery ranged from 97 to 100%, with an average RQD of 90%, indicating an excellent rock quality.

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. In addition, the groundwater level was measured in the monitoring well installed within Borehole BH1 on March 13, 2019 and the measurement is provided on the appended borehole log. Groundwater was not encountered within Boreholes BH2 to BH4; however, it was measured to be at a depth of approximately 3.0 mbgs within Borehole BH1.

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.

It is Pinchin's understanding that the proposed development is to consist of a 14-storey multi-tenant residential building, complete with two levels of underground parking. At the time of this report 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 an approximate depth to the underside of the footings of 8.0 mbgs. Based on this, Pinchin recommends to construct the building on conventional shallow strip and spread footings founded on the limestone bedrock located approximately 8.0 mbgs.

5.2 Open Cut Excavations and Groundwater Management

It is anticipated that the excavations for the building foundations will extend to an approximate depth of 8.0 mbgs in order to accommodate the proposed levels of underground parking. As such, a portion of the bedrock will need to be removed to accommodate the underground levels.

Based on the subsurface information obtained from within the boreholes it is anticipated that the excavated material will consist of a combination of asphalt, granular fill, glacial till, and bedrock. Groundwater was encountered at approximately 3.0 mbgs within Borehole BH1.

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 shoring system may be designed as full cantilevers or the lateral loads can be taken up to the installation of internal bracing of rakers or tie back soil anchors. The temporary shoring design must include appropriate factors of safety, and any possible surcharge loading must be taken into account.

The following parameters (un-factored) could be used in the shoring design against lateral loads: It should be noted that these earth pressure coefficients assume that the back of the wall is vertical; condition of the ground surface behind the wall is assumed to be flat:

Soil Layer	Unit Weight (kN/m^3)	Angle of Internal Friction (°)	Active Earth Pressure Coefficient - K_a	Passive Earth Pressure Coefficient - K_p	At Rest Earth Pressure Coefficient - K_o
Fill Material	20	30	0.33	3.0	0.5
Glacial Till	21	32	0.31	3.25	0.47

Based on the OHSA, the glacial till would be classified as Type 2 soil and temporary excavations in these soils may be cut vertical in the bottom 1.2 m and must be sloped back at an inclination of 1 horizontal to 1 vertical (H to V) above this. Excavations extending below the groundwater table would be classified as a Type 4 soil and temporary excavations will have to be sloped back at 3 horizontal to 1 vertical from the base of the excavation.

The upper approximate 1.0 to 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 be in compliance to any potential other regulatory authorities, such as federal and municipal safety standards.

Moderate groundwater inflow through the overburden soil and/or bedrock face is expected where the excavations extend less than 0.50 m below the groundwater table. It is believed that this groundwater inflow can be controlled using a gravity dewatering system with perimeter interceptor ditches and high capacity pumps. For excavations extending more than 0.5 m below the stabilized groundwater table, a dewatering system installed by a specialist dewatering contractor may be required to either lower the groundwater level prior to excavation, or to maintain the groundwater level during construction. The design of the dewatering system should be left to the contractor’s discretion, and the system should meet a performance specification to maintain and control the groundwater at least 0.50 m below the excavation base. Pinchin notes that based on the bedrock being of excellent quality, there is a potential that all groundwater can be controlled with a gravity dewatering system with perimeter interceptor ditches and high capacity pumps.

Seasonal variations in the water table should be expected, with higher levels occurring during wet weather conditions in the spring and fall and lower levels occurring during dry weather conditions. If construction commences during wet periods (typically spring or fall), there is a greater potential that the groundwater elevation could be higher and/or perched groundwater may be present. Any potential precipitation of perched groundwater should be able to be controlled from pumping from filtered sumps, and should be pumped away immediately (not allowed to pond).

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

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

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

5.3 Foundation Design

5.3.1 Shallow Foundations Bearing on Bedrock

For conventional shallow strip and spread footings established directly on the weathered bedrock surface, a factored bearing resistance of 1,000 kPa may be used at Ultimate Limit States (ULS) design. For conventional shallow strip and spread footings established on unweathered competent bedrock, a factored bearing resistance of 2,000 kPa at ULS may be used.

Prior to installing foundation formwork, the bedrock is to be reviewed by a geotechnical engineer. SLS 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 above bearing resistances assume the bedrock is cleaned of all overburden material and any loose rock pieces. In addition, it is assumed that the bedrock is free of soil filled seams. Therefore, the bedrock should be cleaned with air or water pressure exposing clean sound bedrock, and 1.5 m long probe holes should be advanced at selected locations to check for bedrock defects and soil filled seams. In the event soil filled seams are encountered, bedrock may need to be removed to the soil seam in order to achieve the recommended bearing resistances.

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. Pinchin notes that it may be beneficial to install an approximate 150 mm thick layer of 19 mm clear stone gravel overlying the bedrock surface, to provide the forming contractor with a level working surface. 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.

5.3.2 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 latest edition of the 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.

5.3.3 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 latest edition of the OBC.

5.3.4 Building Drainage

To assist in maintaining the building dry from surface water seepage, it is recommended that exterior grades around the buildings be sloped away at a 2% gradient or more, for a distance of at least 2.0 m. Roof drains should discharge a minimum of 1.5 m away from the structure to a drainage swale or appropriate storm drainage system.

5.3.5 Shallow Foundation Frost Protection & Foundation Backfill

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

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

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

To minimize potential frost movements from soil frost adhesion, the perimeter foundation backfill should consist of a free draining granular material, such as a Granular 'B' Type I (OPSS 1010) or an approved sand fill, extending a minimum lateral distance of 600 mm beyond the foundation. 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.

5.3.6 Site Classification for Seismic Site Response and 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.

Pinchin notes that based on the OBC, the highest Site Class that can be given using energy corrected Standard Penetration Resistance (N60) and/or the average undrained shear strength of the soil in the top 30 m of the Site stratigraphy is a Site Class "C". In order to obtain a higher Site Class, shear wave velocity soundings in the top 30 m of the Site stratigraphy would have to be performed, through testing methods such as multi-channel analysis of surface waves (MASW). At this Site there have been no shear wave velocity measurements. As such, SPT "N" values recorded in the boreholes have been used to classify the soil.

The boreholes advanced at this Site extended to between approximately 1.5 and 3.4 mbgs where refusal was encountered on bedrock. 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. There is a potential that the Site Class may be higher; however, shear wave velocity measurements would be required for the determination of a higher Site Classification, as per the OBC.

5.4 Underground Parking Garage Design

At this time the final grades for the underside of the underground parking garage footings is unknown; however, it is understood that up to two levels of underground parking will be constructed at the Site, extending to a depth of approximately 8 mbgs. Groundwater was encountered at an approximate depth of 3.0 mbgs within Borehole BH1.

As such, depending on the proposed final grades, the building will have to be designed to either resist hydrostatic uplift or to be provided with underfloor and foundation wall drainage systems connected to a suitable frost free outlet.

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

$$P = \gamma \times d$$

Where:

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

γ = unit weight of water (9.8 kN/m^3)

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

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

Alternatively, 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 water proofed.

If the proposed basement floor level is constructed close to or below 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 sump.

If the building is constructed below the groundwater table and sub drains 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 (MECP) will be required for the long term dewatering of 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.

5.4.1 Lower Level Parking Garage Concrete Slab-on-Grade

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

Based on the in-situ conditions, it is recommended to establish a concrete floor slab-on-grade on a minimum 200 mm thick layer of Granular 'A' (OPSS 1010). The purpose of the Granular 'A' is mainly to provide a level surfaced for the concrete formwork. Alternatively, consideration may also be given to using a 200 mm thick layer of uniformly compacted 19 mm clear stone. Any required up fill should consist of a 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.

The following table provides the unfactored modulus of subgrade reaction values:

Material Type	Modulus of Subgrade Reaction (kN/m^3)
Granular A (OPSS 1010)	85,000
Granular 'B' Type I (OPSS 1010)	75,000
Granular 'B' Type II (OPSS 1010)	85,000

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 granular fill and bedrock 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 DISCLAIMER

This Geotechnical Investigation was performed for the exclusive use of Loretta Apartments Inc. c/o District Realty(Client) in order to evaluate the subsurface conditions at 90 Champagne Avenue South, Ottawa, Ontario.

Within the limitations of scope, schedule and budget, our services have been executed in accordance with generally accepted practises in the field of geotechnical engineering for the Site. Classification and identification of soil, and geologic units have been based upon commonly accepted methods employed in professional geotechnical practice. No warranty or other conditions, expressed or implied, should be understood. Conclusions derived are specific to the immediate area of study and cannot be extrapolated extensively away from sample locations.

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

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

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

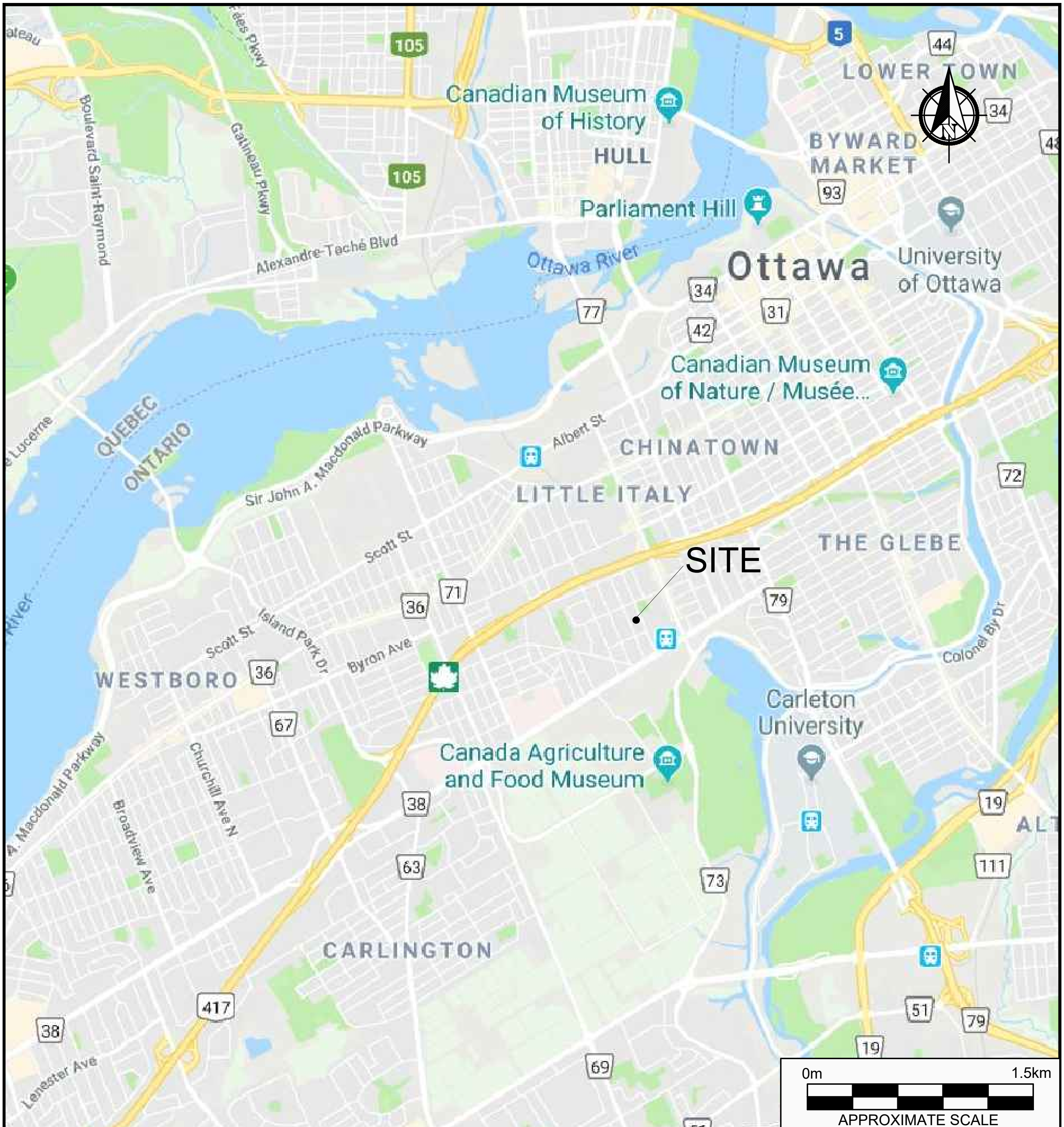
The liability of Pinchin or our officers, directors, shareholders or staff will be limited to the lesser of the fees paid or actual damages incurred by the Client. Pinchin will not be responsible for any consequential or indirect damages. Pinchin will only be liable for damages resulting from the negligence of Pinchin.

Pinchin will not be liable for any losses or damage if the Client has failed, within a period of two years following the date upon which the claim is discovered (Claim Period), to commence legal proceedings against Pinchin to recover such losses or damage unless the laws of the jurisdiction which governs the Claim Period which is applicable to such claim provides that the applicable Claim Period is greater than two years and cannot be abridged by the contract between the Client and Pinchin, in which case the Claim Period shall be deemed to be extended by the shortest additional period which results in this provision being legally enforceable.

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

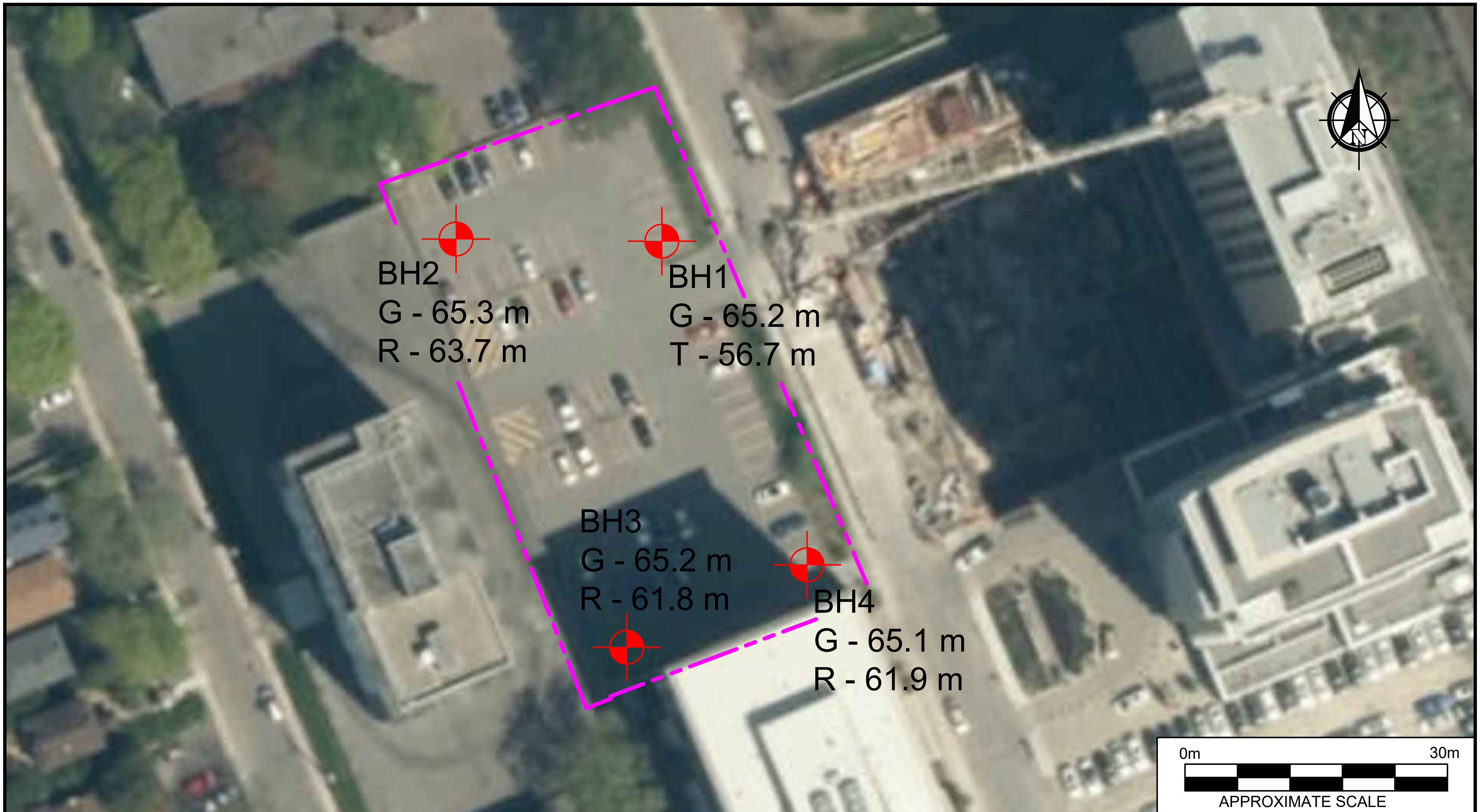
235750.001 Geotechnical Investigation 90 Champagne Ave S Ottawa ON District Realty

FIGURES



PINCHIN

PROJECT NAME: GEOTECHNICAL INVESTIGATION			
CLIENT NAME: LORETTA APARTMENTS INC. C/O DISTRICT REALTY			
PROJECT LOCATION: 90 CHAMPAGNE AVENUE SOUTH, OTTAWA, ONTARIO			
FIGURE NAME: KEY MAP			FIGURE NO.: 1
APPROXIMATE SCALE: AS SHOWN	PROJECT NO.: 235750.001	DATE: MARCH 2019	



LEGEND

- APPROXIMATE SITE BOUNDARY

G - APPROXIMATE GROUND ELEVATION AT INVESTIGATION LOCATION (masl)

T - APPROXIMATE TERMINATION ELEVATION AT INVESTIGATION LOCATION (masl)

R - APPROXIMATE REFUSAL ELEVATION AT INVESTIGATION LOCATION (masl)

BOREHOLE ADVANCED BY PINCHIN ON FEBRUARY 22, 2019

PROJECT NAME GEOTECHNICAL INVESTIGATION			
CLIENT NAME LORETTA APARTMENTS INC. C/O DISTRICT REALTY			
PROJECT LOCATION 90 CHAMPAGNE AVENUE SOUTH, OTTAWA, ONTARIO			
FIGURE NAME BOREHOLE LOCATION PLAN			FIGURE NO. 2
APPROXIMATE SCALE AS SHOWN	PROJECT NO. 235750.001	DATE MARCH 2019	

APPENDIX I

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

ABBREVIATIONS, TERMINOLOGY & PRINCIPAL SYMBOLS USED

Sampling Method

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

In-Situ Soil Testing

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

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

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

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

Soil Descriptions

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

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

Notes:

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

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

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

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

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

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

Soil & Rock Physical Properties

General

W Natural water content or moisture content within soil sample
γ Unit weight
γ' Effective unit weight
γ_d Dry unit weight
γ_{sat} Saturated unit weight
ρ Density
ρ_s Density of solid particles
ρ_w Density of Water
ρ_d Dry density
ρ_{sat} Saturated density e Void ratio
n Porosity
S_r Degree of saturation
E_{50} Strain at 50% maximum stress (cohesive soil)

Consistency

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

Shear Strength

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

Consolidation (One Dimensional)

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

Permeability

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

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

Rock Coring

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

RQD is calculated as follows:

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

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

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

APPENDIX II

Pinchin's Borehole Logs

Log of Borehole: BH1

Project #: 235750.001 **Logged By:** W.T.

Project: Geotechnical Investigation

Client: Loretta Apartments Inc. c/o District Realty

Location: 90 Champagne Avenue South, Ottawa, Ontario

Drill Date: February 22, 2019 **Project Manager:** W.T.

SUBSURFACE PROFILE

Depth (m)	Description	Elevation (m)
0	Ground Surface	65.19
0–0.76	Asphalt ~ 100 mm Fill - Sand and gravel, trace silt, brown, frozen	64.43
0.76–2.44	Till - Silty, sandy, gravel, trace clay, damp to moist, brown, compact to very dense	62.75
2.44–8.53	Limestone bedrock, faintly weathered, grey with black banding, fine to medium grained, few natural fractures with little to no oxidation. Fair to excellent quality.	56.66
8.53	End of Borehole	

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

Water level measured at 3.0 mbgs on March 13, 2019

SAMPLE

Sample Type	Sampler #	Recovery (%)	SPT N-Value	Water Content (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
AS	AS1	100	NA				
SS	SS2	100	28				
SS	SS3	100	23	10.6			
	SS4		50				
NQ	Run 1	97					RQD=67%
NQ	Run 2	100					RQD=95%
NQ	Run 3	98					RQD=98%
NQ	Run 4	100					RQD=99%

Standard Penetration N-Value: 20 40 60

Shear Strength kPa: 100 200

Contractor: Strata Drilling Group **Grade Elevation:** 65.19 m

Drilling Method: Direct Push/Split Spoon **Top of Casing Elevation:** 65.03 m

Well Casing Size: 50 mm **Sheet:** 1 of 1

Log of Borehole: BH2

Project #: 235750.001 **Logged By:** W.T.

Project: Geotechnical Investigation

Client: Loretta Apartments Inc. c/o District Realty

Location: 90 Champagne Avenue South, Ottawa, Ontario

Drill Date: February 22, 2019 **Project Manager:** W.T.

SUBSURFACE PROFILE					SAMPLE										
Depth (m)	Symbol	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-Value	Standard Penetration N-Value 20 40 60	Shear Strength kPa 100 200	Water Content (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
		Ground Surface	65.25											
0		Asphalt ~ 100 mm Fill - Sand and gravel, trace silt, brown, frozen	64.79		AS	AS1	100	NA						
1		Till - Silty, sandy, gravel, trace clay, damp to moist, brown, compact to dense	63.73		SS	SS2	100	50						
2		End of Borehole Due to SPT refusal on bedrock												
3														
4														
5														
6														
7														
8														
9														
10														

Contractor: Strata Drilling Group **Grade Elevation:** 65.25 m

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

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

Log of Borehole: BH3

Project #: 235750.001 **Logged By:** W.T.

Project: Geotechnical Investigation

Client: Loretta Apartments Inc. c/o District Realty

Location: 90 Champagne Avenue South, Ottawa, Ontario

Drill Date: February 22, 2019 **Project Manager:** W.T.

SUBSURFACE PROFILE					SAMPLE									
Depth (m)	Symbol	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-Value	Standard Penetration N-Value 20 40 60	Shear Strength kPa 100 200	Water Content (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
		Ground Surface	65.20											
0		Asphalt ~ 100 mm Fill - Sand and gravel, trace silt, brown, frozen	64.74		AS	AS1	100	NA						
1		Till - Sand and gravel, some silt, trace clay, damp to moist, brown, compact to very dense			SS	SS2	100	31						
2					SS	SS3	100	25						
					SS	SS4	100	64			6.6			
3			61.85		SS	SS5	100	90						
		End of Borehole Due to SPT refusal on bedrock												

Contractor: Strata Drilling Group **Grade Elevation:** 65.2 m

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

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

Log of Borehole: BH4

Project #: 235750.001 **Logged By:** W.T.

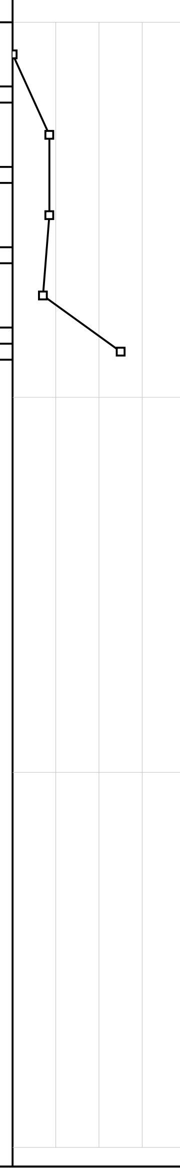
Project: Geotechnical Investigation

Client: Loretta Apartments Inc. c/o District Realty

Location: 90 Champagne Avenue South, Ottawa, Ontario

Drill Date: February 22, 2019 **Project Manager:** W.T.

SUBSURFACE PROFILE					SAMPLE									
Depth (m)	Symbol	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-Value	Standard Penetration N-Value 20 40 60	Shear Strength kPa 100 200	Water Content (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
		Ground Surface	65.09											
0		Asphalt ~ 100 mm Fill - Sand and gravel, trace silt, brown, frozen	64.63		AS	AS1	100	NA						
1		Till - Silty sand, trace gravel, trace clay, damp to moist, brown, compact to dense			SS	SS2	100	17						
2					SS	SS3	100	17						
					SS	SS4	100	14			10.3			
3			61.89			SS5		50						
		End of Borehole Due to SPT refusal on bedrock												



Depth scale (m): 0, 1, 2, 3, 4, 5, 6, 7, 8, 9, 10

Contractor: Strata Drilling Group **Grade Elevation:** 65.09 m

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

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

APPENDIX III
Analytical Laboratory Testing Reports for Soil Samples

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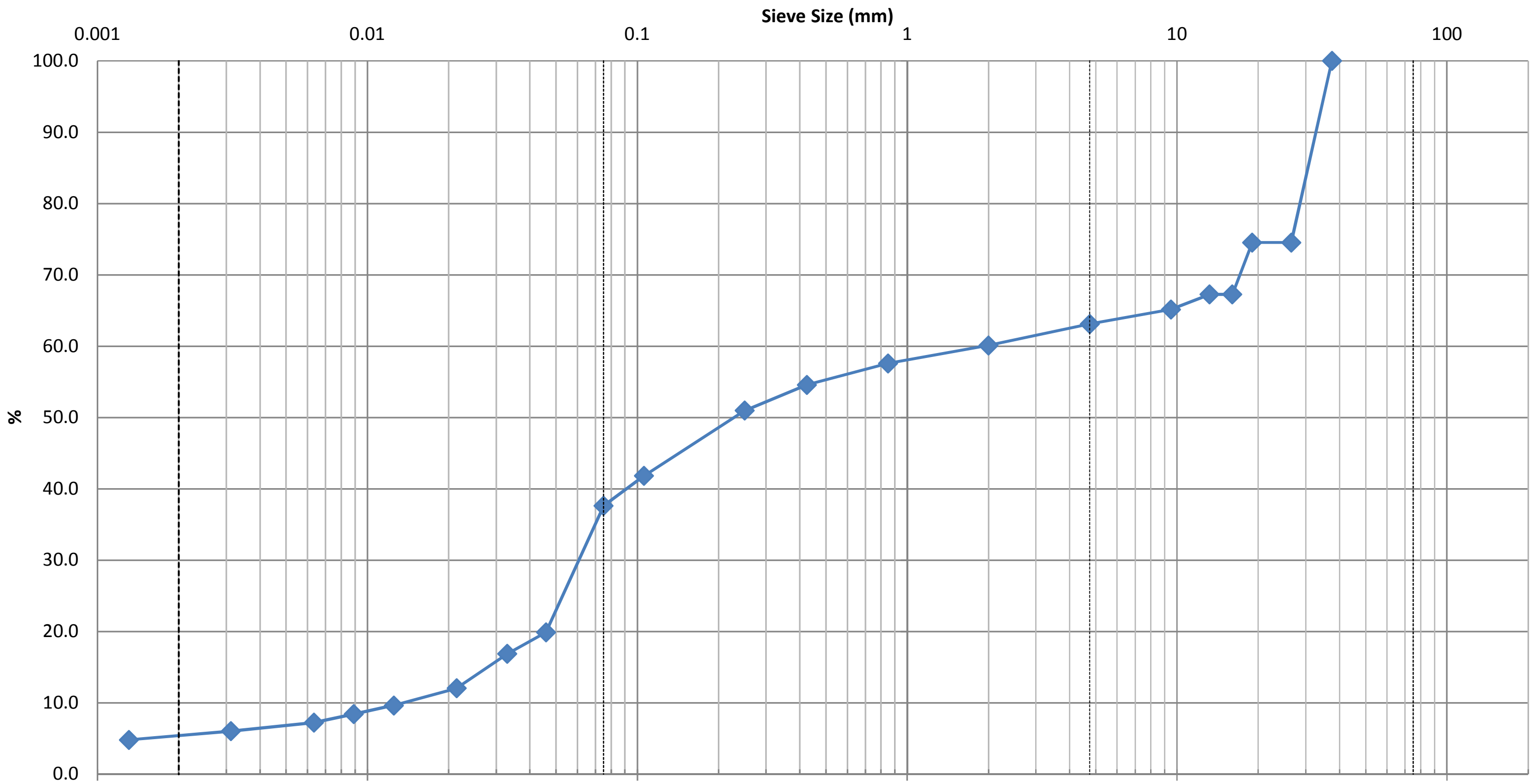
CLIENT:	Pinchin Limited	DEPTH:	5 - 7'	FILE NO:	PM4184
CONTRACT NO.:		BH OR TP No.:	BH1	LAB NO:	06869
PROJECT:	235750			DATE RECEIVED:	6-Mar-19
				DATE TESTED:	11-Mar-19
DATE SAMPLED:	22-Feb-19			DATE REPORTED:	13-Mar-19
SAMPLED BY:	W. Tabaczuk			TESTED BY:	D. Bertrand



	Clay	Silt	Sand			Gravel		Cobble
			Fine	Medium	Coarse	Fine	Coarse	

Identification	Soil Classification				MC(%)	LL	PL	PI	Cc	Cu
					10.6					
	D100	D60	D30	D10	Gravel (%)	Sand (%)		Silt (%)	Clay (%)	
					36.9	25.5		32.1	5.5	

Comments	

patersongroup
consulting engineers

HYDROMETER
LS-702 ASTM-422

CLIENT:	Pinchin Limited	DEPTH:	5 - 7'	FILE NO.:	PM4184
PROJECT:	235750	BH OR TP No.:	BH1	DATE SAMPLED:	22-Feb-19
LAB No. :	06869	TESTED BY:	D. Bertrand	DATE RECEIVED:	06-Mar-19
SAMPLED BY:	W. Tabaczuk	DATE REPT'D:	13-Mar-19	DATE TESTED:	11-Mar-19

SAMPLE INFORMATION

SAMPLE MASS	142.9	50.00
SPECIFIC GRAVITY (Gs)	2.700	REMARKS
HYGROSCOPIC MOISTURE	Tare No.	
TARE Wt. 50.00	ACTUAL Wt.	
AIR DRY (Wa) 150.00	100.00	
OVEN DRY (Wo) 148.60	98.60	
F=(Wo/Wa)	0.986	
INITIAL Wt. (Ma)	50.00	
Wt. CORRECTED	49.30	
Wt. AFTER WASH BACK SIEVE	19	
SOLUTION CONCENTRATION	40 g / L	

GRAIN SIZE ANALYSIS

SIEVE DIAMETER (mm)	WEIGHT RETAINED (g)	PERCENT RETAINED	PERCENT PASSING
63.0			
53.0			
37.5	**0**	0.0	**100.0**
26.5	**36.4**	25.5	**74.5**
19.0	**36.4**	25.5	**74.5**
16.0	**46.8**	32.8	**67.2**
13.2	**46.8**	32.8	**67.2**
9.5	**49.8**	34.8	**65.2**
4.75	**52.7**	36.9	**63.1**
2.0	**57.0**	39.9	**60.1**
Pan	**85.9**		
0.850	**2.10**	42.4	**57.6**
0.425	**4.60**	45.4	**54.6**
0.250	**7.60**	49.0	**51.0**
0.106	**15.20**	58.2	**41.8**
0.075	**18.70**	62.4	**37.6**
Pan	**19.00**		
SIEVE CHECK 0.0	MAX = 0.3%		

HYDROMETER DATA

ELAPSED	TIME (24 hours)	Hs	Hc	Temp. (^{o}C)	DIAMETER	(P)	TOTAL PERCENT PASSING
1	10:16	22.5	6.0	23.0	0.0459	33.1	**19.9**
2	10:17	20.0	6.0	23.0	0.0330	28.1	**16.9**
5	10:20	16.0	6.0	23.0	0.0214	20.1	**12.1**
15	10:30	14.0	6.0	23.0	0.0125	16.0	**9.6**
30	10:45	13.0	6.0	23.0	0.0089	14.0	**8.4**
60	11:15	12.0	6.0	23.0	0.0063	12.0	**7.2**
250	14:25	11.0	6.0	23.0	0.0031	10.0	**6.0**
1440	10:15	10.0	6.0	23.0	0.0013	8.0	**4.8**

COMMENTS

Moisture Content = 10.6%

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

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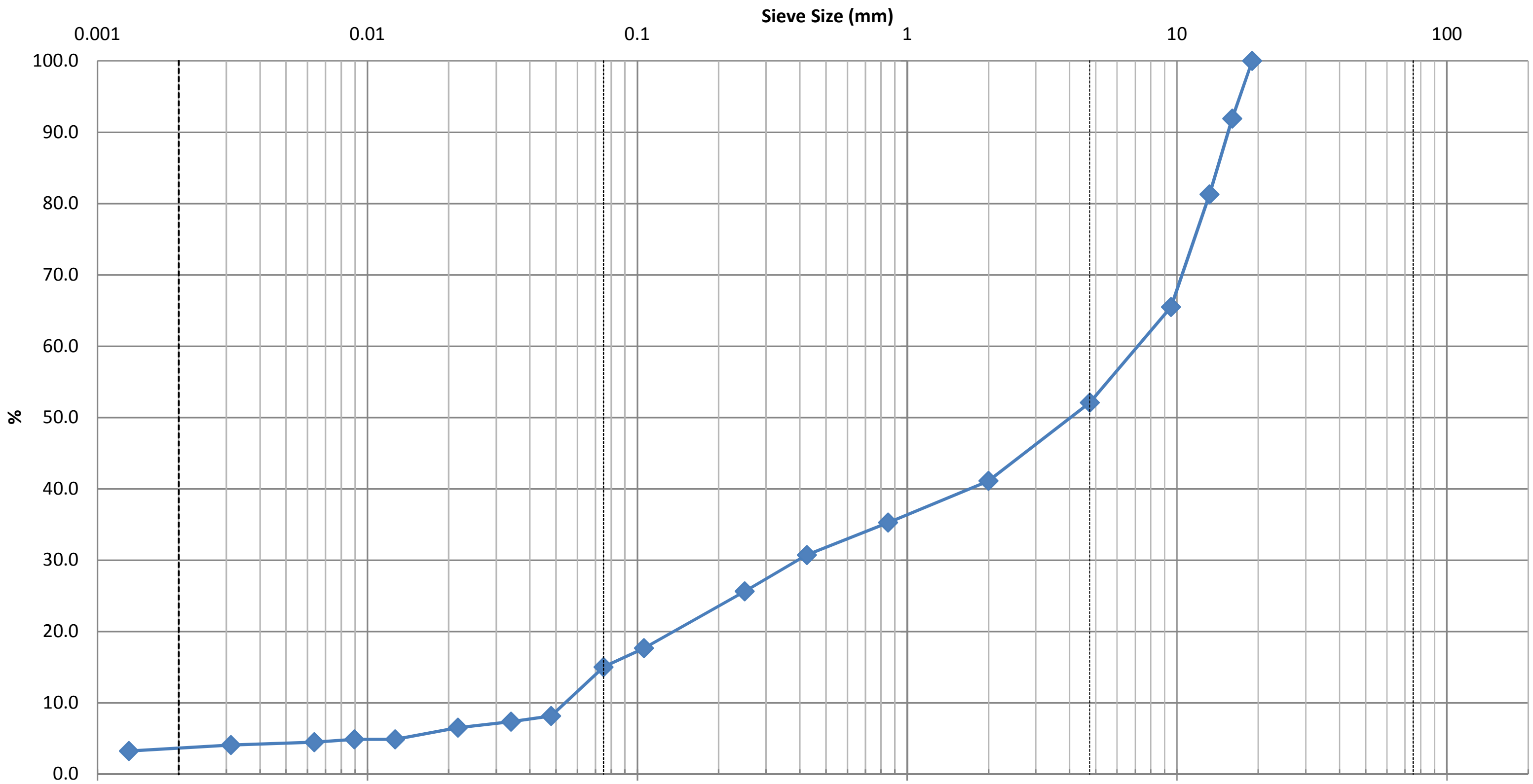
consulting engineers

CLIENT:	Pinchin Limited	DEPTH:	7.5 - 9.5'	FILE NO:	PM4184
CONTRACT NO.:		BH OR TP No.:	BH3	LAB NO:	06870
PROJECT:	235750			DATE RECEIVED:	6-Mar-19
				DATE TESTED:	11-Mar-19
DATE SAMPLED:	22-Feb-19			DATE REPORTED:	13-Mar-19
SAMPLED BY:	W. Tabaczuk			TESTED BY:	D. Bertrand



	Clay	Silt	Sand			Gravel		Cobble
			Fine	Medium	Coarse	Fine	Coarse	

Identification	Soil Classification	MC(%)	LL	PL	PI	Cc	Cu
		6.6					

D100	D60	D30	D10	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
				47.9	37.1	11.2	3.8

Comments	

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HYDROMETER
LS-702 ASTM-422

CLIENT:	Pinchin Limited	DEPTH:	7.5 - 9.5'	FILE NO.:	PM4184
PROJECT:	235750	BH OR TP No.:	BH3	DATE SAMPLED:	22-Feb-19
LAB No. :	06870	TESTED BY:	D. Bertrand	DATE RECEIVED:	06-Mar-19
SAMPLED BY:	W. Tabaczuk	DATE REPT'D:	13-Mar-19	DATE TESTED:	11-Mar-19

SAMPLE INFORMATION

SAMPLE MASS	148.4	50.00
SPECIFIC GRAVITY (Gs)	2.700	
HYGROSCOPIC MOISTURE	Tare No.	
TARE Wt.	50.00	ACTUAL Wt.
AIR DRY (Wa)	150.00	100.00
OVEN DRY (Wo)	149.50	99.50
F=(Wo/Wa)	0.995	
INITIAL Wt. (Ma)	50.00	
Wt. CORRECTED	49.75	
Wt. AFTER WASH BACK SIEVE	32	
SOLUTION CONCENTRATION	40 g / L	

REMARKS

GRAIN SIZE ANALYSIS

SIEVE DIAMETER (mm)	WEIGHT RETAINED (g)	PERCENT RETAINED	PERCENT PASSING
63.0			
53.0			
37.5			
26.5			
19.0	**0**	0.0	**100.0**
16.0	**12**	8.1	**91.9**
13.2	**27.8**	18.7	**81.3**
9.5	**51.2**	34.5	**65.5**
4.75	**71.1**	47.9	**52.1**
2.0	**87.4**	58.9	**41.1**
Pan	**61.0**		
0.850	**7.10**	64.7	**35.3**
0.425	**12.60**	69.3	**30.7**
0.250	**18.80**	74.4	**25.6**
0.106	**28.50**	82.3	**17.7**
0.075	**31.70**	85.0	**15.0**
Pan	**32.00**		
SIEVE CHECK 0.0	MAX = 0.3%		

HYDROMETER DATA

ELAPSED	TIME (24 hours)	Hs	Hc	Temp. (^{o}C)	DIAMETER	(P)	TOTAL PERCENT PASSING
1	10:30	16.0	6.0	23.0	0.0479	19.9	**8.2**
2	10:31	15.0	6.0	23.0	0.0341	17.9	**7.4**
5	10:34	14.0	6.0	23.0	0.0217	15.9	**6.5**
15	10:44	12.0	6.0	23.0	0.0127	11.9	**4.9**
30	10:59	12.0	6.0	23.0	0.0090	11.9	**4.9**
60	11:29	11.5	6.0	23.0	0.0064	10.9	**4.5**
250	14:39	11.0	6.0	23.0	0.0031	9.9	**4.1**
1440	10:29	10.0	6.0	23.0	0.0013	8.0	**3.3**

COMMENTS

Moisture Content = 6.6%

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

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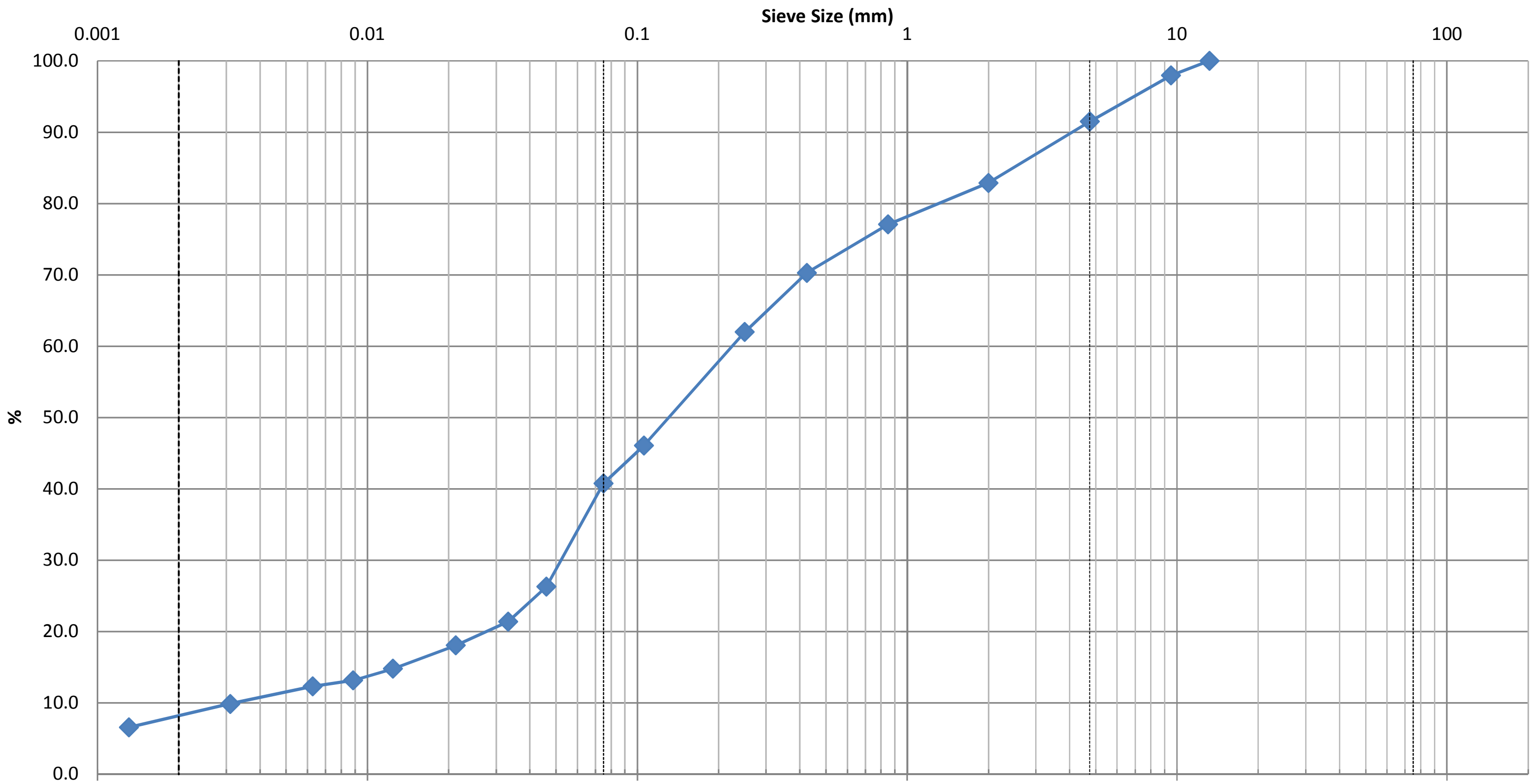
CLIENT:	Pinchin Limited	DEPTH:	7.5 - 9.5'	FILE NO:	PM4184
CONTRACT NO.:		BH OR TP No.:	BH4	LAB NO:	06871
PROJECT:	235750			DATE RECEIVED:	6-Mar-19
				DATE TESTED:	11-Mar-19
DATE SAMPLED:	22-Feb-19			DATE REPORTED:	13-Mar-19
SAMPLED BY:	W. Tabaczuk			TESTED BY:	D. Bertrand



	Clay	Silt	Sand			Gravel		Cobble
			Fine	Medium	Coarse	Fine	Coarse	

Identification	Soil Classification				MC(%)	LL	PL	PI	Cc	Cu
					10.3					
	D100	D60	D30	D10	Gravel (%)	Sand (%)		Silt (%)	Clay (%)	
					8.5	50.7		32.3	8.5	

Comments	

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HYDROMETER
LS-702 ASTM-422

CLIENT:	Pinchin Limited	DEPTH:	7.5 - 9.5'	FILE NO.:	PM4184
PROJECT:	235750	BH OR TP No.:	BH4	DATE SAMPLED:	22-Feb-19
LAB No. :	06871	TESTED BY:	D. Bertrand	DATE RECEIVED:	06-Mar-19
SAMPLED BY:	W. Tabaczuk	DATE REPT'D:	13-Mar-19	DATE TESTED:	11-Mar-19

SAMPLE INFORMATION

SAMPLE MASS	136.2	50.00
SPECIFIC GRAVITY (Gs)		2.700
HYGROSCOPIC MOISTURE		Tare No.
TARE Wt.	50.00	ACTUAL Wt.
AIR DRY (Wa)	150.00	100.00
OVEN DRY (Wo)	149.70	99.70
F=(Wo/Wa)	0.997	
INITIAL Wt. (Ma)	50.00	
Wt. CORRECTED	49.85	
Wt. AFTER WASH BACK SIEVE		25.9
SOLUTION CONCENTRATION		40 g / L

REMARKS

GRAIN SIZE ANALYSIS

SIEVE DIAMETER (mm)	WEIGHT RETAINED (g)	PERCENT RETAINED	PERCENT PASSING
63.0			
53.0			
37.5			
26.5			
19.0			
16.0			
13.2	**0.0**	0.0	**100.0**
9.5	**2.8**	2.1	**97.9**
4.75	**11.6**	8.5	**91.5**
2.0	**23.3**	17.1	**82.9**
Pan	**112.9**		
0.850	**3.50**	22.9	**77.1**
0.425	**7.60**	29.7	**70.3**
0.250	**12.60**	38.0	**62.0**
0.106	**22.20**	53.9	**46.1**
0.075	**25.40**	59.2	**40.8**
Pan	**25.90**		
SIEVE CHECK 0.0	MAX = 0.3%		

HYDROMETER DATA

ELAPSED	TIME (24 hours)	Hs	Hc	Temp. (^{o}C)	DIAMETER	(P)	TOTAL PERCENT PASSING
1	10:42	22.0	6.0	23.0	0.0461	31.7	**26.3**
2	10:43	19.0	6.0	23.0	0.0332	25.8	**21.4**
5	10:46	17.0	6.0	23.0	0.0213	21.8	**18.1**
15	10:56	15.0	6.0	23.0	0.0124	17.9	**14.8**
30	11:11	14.0	6.0	23.0	0.0089	15.9	**13.2**
60	11:41	13.5	6.0	23.0	0.0063	14.9	**12.3**
250	14:51	12.0	6.0	23.0	0.0031	11.9	**9.9**
1440	10:41	10.0	6.0	23.0	0.0013	7.9	**6.6**

COMMENTS

Moisture Content = 10.3%

REVIEWED BY:	**Curtis Beadow**	**APPROVED BY:**	**Joe Forsyth, 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.

APPENDIX V

Rock Core Photographs

Photo 1 – Borehole BH1, Rock Core (Runs 1 to 4)