FINAL

Geotechnical Investigation – Proposed Residential Development

500 Coventry Road, Ottawa, Ontario

Prepared for:

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1.0 INTRODUCTION AND SCOPE

Pinchin Ltd. (Pinchin) was retained by Morguard Investments Limited on behalf of Morguard REIT(Client) to conduct a Geotechnical Investigation and provide subsequent geotechnical design recommendations for the proposed residential development to be located at 500 Coventry 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 an apartment building with 28 storeys plus a mechanical penthouse complete with two levels of underground parking, new Site services, outdoor rooftop amenity areas and asphalt surfaced access roadways/driveways and parking areas.

Pinchin was provided with the following set of architectural drawings:

- Turner Fleischer Architects Inc. drawings entitled "500 Coventry Road, Ottawa ON" (Project No. 18.050 P01), Drawings SPA001, SPA004, SPA005A, SPA005B, SPA005C, SPA101, SPA102, SPA151 through SPA157, SPA301, SPA302, and SPA401, with a project date of August 20, 2024 (2024 Site Plans).

The 2024 Site Plans show a Finished Floor Elevation (FFE) for the lobby/ground floor or 68.90 masl and the underside of the parking garage slab to be located approximately 7.45 meters below ground surface (mbgs) and at an approximate elevation of 61.45 masl.

Pinchin's geotechnical comments and recommendations are based on the results of the Geotechnical Investigation and our understanding of the project scope. Pinchin notes that a Phase Two Environmental Site Assessment and Hydrogeological Assessment were completed concurrently to the Geotechnical Investigation however the findings are summarized under different covers.

The purpose of the Geotechnical Investigation was to delineate the subsurface conditions and soil engineering characteristics by advancing a total of five (5) sampled boreholes (Boreholes BH101 to BH105), at the Site. The information gathered from the Geotechnical Investigation will allow Pinchin to provide geotechnical design recommendations for the proposed development.

As the drilling program and field investigation was completed concurrently with the Phase Two Environmental Site Assessment, Pinchin would like to highlight the naming differences between the two investigations (however they are the same boreholes).

Geotechnical Investigation	Phase Two Environmental Site Assessment
BH101	MW16
BH102	MW18
BH103	BH24
BH104	MW19
BH105	BH17

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

- A detailed description of the soil and groundwater conditions;
- Site preparation recommendations;
- Open cut excavations;
- Anticipated groundwater management;
- Site service trench design;
- Lateral earth pressure coefficients and unit densities;
- Foundation design recommendations including soil bearing resistances at Ultimate Limit States (ULS) and Serviceability Limit States (SLS) design;
- Potential total and differential settlements;
- Foundation frost protection and engineered fill specifications and installation;
- Seismic Site classification for seismic Site response;
- Concrete floor slab-on-grade support recommendations;
- Asphaltic concrete pavement structure design for parking areas and access roadways;
- Underground parking garage design; 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 south side of Coventry Road, approximately 550 m east of the intersection of Highway 417 and St. Laurent Boulevard in Ottawa, Ontario. The south side of the site borders Highway 417. The majority of the area of 500 Coventry Road is currently developed with an old asphalt surfaced parking area and a maintenance laydown area consisting of a coverall structure on concrete barriers. There is also a fill pile located on the north side of the Site. As previously mentioned, the south portion of the Site borders the highway and the remaining adjacent lands are currently developed with office buildings, parking structures and commercial/retail developments.

Data obtained from the Ontario Geological Survey Maps, as published by the Ontario Ministry of Natural Resources, indicates that the Site is located on a fine textured glaciomarine deposit consisting of massive to well laminated silt and clay with minor sand and gravel deposits (Ontario Geological Survey 2010. Surficial geology of Southern Ontario; Ontario Geological Survey, Miscellaneous Release--Data 128-REV). The underlying bedrock at this Site is of the Georgian Bay Formation, Blue Mountain Formation and Billings Formation consisting of shale, limestone, dolostone and siltstone (Ontario Geological Survey Map 1972, published 1978).

3.0 GEOTECHNICAL FIELD INVESTIGATION AND METHODOLOGY

Pinchin completed field investigations at the Site April 8 and 9, 2024 by advancing a total of five (5) sampled boreholes throughout the Site. The boreholes were advanced to depths of approximately 1.8 to 8.1 metres below existing ground surface (mbgs). The approximate spatial locations of the boreholes advanced at the Site are shown on Figure 2.

The boreholes were advanced with the use of a Massenza MI3 direct push drill rig which was equipped with standard soil sampling equipment. Soil samples were collected at 0.75 and 1.5 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.

Bedrock was proven in Boreholes BH101 and BH102 by core drilling with an NQ-size double tube diamond bit core barrel. The bedrock core specimens were measured in the field to determine the Rock Quality Designation (RQD) (ASTM 6032). The core samples were returned to our offices for further visual examination and testing.

Monitoring wells were installed in three of the boreholes to allow 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. Construction details of the monitoring wells can be found in Table 1.

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

Groundwater observations and measurements were obtained from the open boreholes during and upon completion of drilling. Groundwater levels were measured in the monitoring wells on April 17, June 21 and July 9, 2024. The groundwater observations and measurements recorded are included on the appended borehole logs and in Table 2.

The borehole locations and ground surface elevations were located at the Site by Pinchin personnel and are geodetic in nature. Based on the Site survey provided by the Client, Pinchin was able to tie in the borehole elevations to the following benchmark as shown on Figure 2:

- TBM: Top nut of fire hydrant, at the approximate location shown on Figure 2; and
- Elevation: 67.98 meters above sea level (masl).

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

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

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

4.0 SUBSURFACE CONDITIONS

4.1 Borehole Soil Stratigraphy

In general, the soil stratigraphy at the Site comprises surficial granular fill overlying silt fill, and/or silty, clayey sand till and bedrock to the maximum borehole termination depth of approximately 8.1 mbgs. The appended borehole logs provide detailed soil descriptions and stratigraphies, results of SPT testing, details of monitoring well installations, and groundwater measurements.

4.1.1 Granular Fill

Surficial granular fill was encountered at the surface in all boreholes except for Borehole BH104 and ranged in thickness between 0.3 and 0.8 m thick. The material was noted to contain sand and gravel with some silt and was damp to moist at the time of sampling. The material had a compact to dense relative density based SPT 'N' values of 22 to 38 blows per 300 mm penetration of a split spoon sampler. The results of one particle size distribution analysis completed on a sample of the granular fill is provided in Appendix III and indicates that the sample contains 33% gravel, 51% sand, and 16% fine material. The moisture content of the sample tested was 4.3% indicating a damp material.

4.1.2 Fill

Fill was encountered at the surface of Borehole BH104 and underlying the granular fill in Borehole BH102. The fill comprised silt and sand with some gravel and cobbles and had a compact relative density based SPT 'N' values of 14 to 29 blows per 300 mm penetration of a split spoon sampler.

4.1.3 Silty Clayey Sand Till

The natural till layer was encountered at depths ranging between 0.3 to 1.5 mbgs and extended to the underlying bedrock surface. The till comprised silty clayey sand with gravel and was damp to wet at the time of sampling. The non-cohesive material had a compact to very dense relative density based SPT 'N' values of 10 to 100 blows per 300 mm penetration of a split spoon sampler. The results of three particle size distribution analyses completed on samples of the till are provided in Appendix III and indicate that the samples contain 22 to 38% gravel, 36 to 44% sand, 18 to 29% silt, and 5 to 8% clay. Atterberg limit testing on the fines of samples revealed a liquid limit ranging between 14 and 16%, a plastic limit ranging between 11 and 12% and a plasticity index ranging between 3 and 4%. Based on the Canadian Foundation Engineering Manual, the till would place as a CL-ML on the Atterberg Limit graph. The moisture content of the samples tested ranged between 7.5 and 9.8%.

4.2 Bedrock

Refusal on probable bedrock was encountered in all boreholes between approximately 1.8 and 3.5 mbgs. During the field investigation, bedrock was proven in Boreholes BH101 and BH102 on April 8, 2024 by core drilling with an NQ-size double tube diamond bit core barrel.

The limestone was primarily dark grey with light grey and white spotting and banding. The bedrock was medium to coarse grained and contained some natural fractures. The bedrock at the fracture locations was mostly sharp and angular, which indicates minor water migration. The wash return was a slightly cloudy grey colour. The bedrock retrieved from the field investigation could be classified as slightly to moderately weathered when discussing its weathering classification as there is no visible to few signs of rock material weathering in the majority of the boreholes. The weathering symbols from the Canadian Foundation Engineering Manual (CFEM) Table 4.18 would be represented by W2 and W3.

In Borehole BH101, the total core recovery (TCR) ranged between approximately 38 to 100%, with a ranging RQD of 0 to 97%, indicating a very poor to excellent RQD. In Borehole BH102, the TCR ranged between approximately 95 to 100% per run, with an RQD ranging between 20 to 90%, indicating a very poor to excellent RQD. Photographs of the rock cores are provided in Appendix IV.

The following table outlines the various total core recovery (%) and quality of the rock cores.

Borehole #	Run #	Total Core Recovery (%)	RQD (%)	Quality of Rock (CFEM Table 4.26)
BH101	RC1	67	10	Very Poor
	RC2	38	0	Very Poor
	RC3	75	56	Fair
	RC4	100	97	Excellent
BH102	RC1	95	20	Very Poor
	RC2	100	65	Fair
	RC3	100	62	Fair
	RC4	100	90	Excellent

All bedrock depths are tabulated in the table below and indicates that the bedrock surface was relatively consistent throughout the Site.

Borehole #	Depth to Bedrock (m)	Geodetic Approximate Ground Elevation (masl)	Geodetic Approximate Bedrock Elevation (masl)
BH101	2.3	67.00 m	64.72 m
BH102	3.1	66.55 m	63.51 m
BH103	1.8	66.70 m	64.87 m
BH104	2.8	67.03 m	64.21 m
BH105	3.5	67.11 m	63.58 m

4.3 Groundwater Conditions

Groundwater observations and measurements were obtained in the open boreholes at the completion of drilling and are summarized on the appended borehole logs. Monitoring wells were installed in Boreholes BH101, BH102 and BH104.

The groundwater levels were measured on April 17, June 21 and July 9, 2024 and ranged between 1.3 to 3.4 mbgs or between elevations of 63.60 masl and 65.21 masl.

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 an apartment building with 28 storeys plus a mechanical penthouse complete with two level of underground parking, new Site services, outdoor rooftop amenity areas and asphalt surfaced access roadways/driveways and parking areas.

5.2 Site Preparation

The existing fill and granular fill are not considered suitable to remain below the proposed building, driveways and parking areas and will need to be removed. As well, the current pile of file material located on the northern portion of the Site must be removed in its entirety.

Pinchin recommends that any engineered fill required at the Site be compacted in accordance with the criteria stated in the following table:

Type of Engineered Fill	Maximum Loose Lift Thickness (mm)	Compaction Requirements	Moisture Content (Percent of Optimum)
Structural fill to support foundations and floor slabs	200	100% SPMDD	Plus 2 to minus 4
Subgrade fill beneath parking lots and access roadways	300	98% SPMDD	Plus 2 to minus 4

Prior to placing any fill material at the Site, the subgrade should be inspected by a qualified geotechnical engineer, and loosened/soft pockets should be sub excavated and replaced with engineered fill.

It is recommended that any fill required to raise grades below the proposed building comprise imported Ontario Provincial Standards and Specifications (OPSS) 1010 Granular 'B' Type I or II material. If the work is carried out during very dry weather, water may have to be added to the material to improve compaction.

A qualified geotechnical engineering technician should be on site to observe fill placement operations and perform field density tests at random locations throughout each lift, to indicate the specified compaction is being achieved.

5.3 Open Cut Excavations

Based on the 2024 Site Plans provided by the Client, it is anticipated that the foundations of the underground parking garage will be constructed at an elevation of 61.45 masl.

Based on the subsurface information obtained from within the boreholes, it is anticipated that the excavated material will predominately consist of granular fill, fill, native glacial till material and bedrock. The groundwater levels were measured on April 17, June 21 and July 9, 2024 and ranged between 1.3 to 3.4 mbgs or between elevations of 63.60 masl and 65.21 masl.

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

5.3.1 Removal of Bedrock

Based on bedrock cores retrieved as well as local experience in the area, the upper approximate 1.5 to 2.0 m of bedrock 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. Specifically, the limestone bedrock was noted to be moderately weathered and should be relatively easy to remove with a hydraulic hammer that can be utilized to create an initial opening for the excavator bucket to gain access of the layered rock. The bedrock is known to contain vertical joints and near horizontal bedding planes. Therefore, some vertical and horizontal over break of the bedrock should be expected.

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

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

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

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

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.

5.4 Anticipated Groundwater Management

As discussed in previous sections, the groundwater levels were measured on April 17, June 21 and July 9, 2024 and ranged between 1.3 to 3.4 mbgs or between elevations of 63.60 masl and 65.21 masl. Complete groundwater levels can be found in Table 2 and based on these elevations, groundwater is expected to be encountered during excavations for the building foundations.

Moderate groundwater inflow through the till material or bedrock fractures is expected where the excavations extend less than 0.60 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 0.6 m below the stabilized groundwater table, a dewatering system installed by a specialist dewatering contractor may be required to lower the groundwater level prior to excavation. The design of the dewatering system should be left to the contractor's discretion, and the system should meet a performance specification to maintain and control the groundwater at least 0.30 m below the excavation base. It is recommended that Pinchin review the final grading plan to confirm this recommendation.

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. Pinchin notes that if exposed subgrade needs to be left open for a period greater than 24 hours or during inclement weather, a mud-slab can be poured to protect the excavation.

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.

5.5 Foundation Design

5.5.1 Shallow Foundations Bearing on Bedrock

Probable bedrock, due to auger refusal was encountered within all of the boreholes at approximately 1.8 to 3.5 mbgs.

For conventional shallow strip and spread footings established directly on the sound bedrock surface, a factored geotechnical bearing resistance of 1,000 kPa may be used at ULS.

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 bearing resistance of 1,000 kPa assumes the bedrock is cleaned of all overburden material and any loose rock pieces. The bedrock should be cleaned with air or water pressure exposing clean sound bedrock. If construction proceeds during freezing weather conditions water should not be allowed to pool and freeze in bedrock depressions. All concrete should be installed and maintained above freezing temperatures as required by the concrete supplier.

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

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

5.5.2 Site Classification for Seismic Site Response & Soil Behaviour

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

The seismic site classification has been based on the 2012 OBC. The parameters for determination of Site Classification for Seismic Site Response are set out in Table 4.1.8.4.A of the OBC. The site classification is based on the average shear wave velocity in the top 30 m of the site stratigraphy. 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 approximately 8.1 mbgs and were terminated either on probable bedrock or in bedrock. SPT “N” values within the fill and silty clayey till deposit ranged between 10 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.

5.5.3 *Liquefaction Potential*

The potential for liquefaction is not of concern at this site.

5.5.4 *Foundation Transition Zones*

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

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

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

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

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

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

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

5.5.5 *Estimated Settlement*

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

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

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

5.5.6 Building Drainage

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

Exterior perimeter foundations drains are not required, where the finished floor elevation is established a minimum of 150 mm above the exterior final grades or that the exterior gradient is properly sloped to divert surface water away from the building.

5.5.7 Shallow Foundations Frost Protection & Foundation Backfill

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

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

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

5.6 Underground Parking Garage Design

It is understood that the buildings may be constructed with two levels of underground parking; and the underside of the parking garage is currently set at 61.45masl based on the 2024 Site Plans. Groundwater was encountered between 1.3 to 3.4 mbgs throughout the multiple groundwater monitoring events with Elevations ranging between 63.60 masl and 65.21 masl.

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

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

$$P = \gamma \times d$$

Where:

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

γ = unit weight of water (9.8 kN/m^3)

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

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

The 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.

As it appears that the underside of the underground parking level will be constructed within the fluctuating groundwater table, 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 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 till. The bulk unit weight of the retained backfill may be taken as 20 kN/m^3 for well compacted and well-graded soil. An appropriate factor of safety should be applied.

5.7 Shoring Requirements

Due to spatial limitations and construction staging areas, it is may not be feasible to slope the excavation back to a safe angle at the Site and therefore shoring will be required.

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

The sections along the perimeter of the proposed structures footprint may need be shored to preserve the integrity of the boundary conditions using a shoring system consisting of a combination of soldier piles/lagging or continuous interlocking caisson wall.

5.7.1 Lateral Earth Pressure

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

Soil Layer	Bulk Unit Weight, γ (kN/m^3)	Angle of Internal Friction (φ)	At Rest Earth Pressure Coefficient, K_0	Active Earth Pressure Coefficient, K_a	Passive Earth Pressure Coefficient, K_p
Compacted Granular Fill	21	34°	0.49	0.28	3.54
Earth Fill	19	28°	0.42	0.36	2.77
Native Till	19	32°	0.49	0.31	3.25

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

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

Where:

P	=	Lateral earth pressure at depth (kPa)
h	=	depth (m)
h_w	=	height of groundwater above depth h (m)
γ	=	soil bult unit weight (kN/m^3)
γ'	=	submerged soil unit weight (kN/m^3)
γ_w	=	unit weight of water (kN/m^3)
K	=	earth pressure coefficient”
q	=	total surcharge load (kPa)

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

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

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

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

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

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

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

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

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

5.8 Floor Slabs

Due to the anticipated underside of the slab of the underground parking level, as shown in the 2024 Site Plans, prior to the installation of the engineered fill material, the granular fill, fill and till should be removed to the underlying bedrock surface. The natural subgrade soil is to be proof roll compacted with a minimum 10 tonne non-vibratory steel drum roller to observe for weak/soft spots. It is noted that some locations will not be accessible by the steel drum roller; as such, these locations can be proof roll compacted with a minimum 450 kg vibratory plate compactor.

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

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

Based on the in-situ soil conditions, it is recommended to establish the concrete floor slab on a minimum 300 mm thick layer of Granular "A" (OPSS 1010). Alternatively, consideration may also be given to using a 200 mm thick layer of uniformly compacted 19 mm clear stone placed over the approved subgrade. Any required up fill should consist of a Granular "B" Type I or Type II (OPSS 1010).

The 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
Silty Clayey Sand Till	20,000

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

5.9 Site Services

5.9.1 Pipe Bedding and Cover Materials for Flexible and Rigid Pipes

The subgrade soil conditions beneath the Site services will comprise primarily of silty clayey sand till and bedrock. No support problems are anticipated for flexible or rigid pipes founded on the till and bedrock. It is noted, however, that substantial changes in grade could cause long-term consolidation settlement of the soils, and the elevations of service pipes could be affected by that settlement. Service pipes require an adequate base to ensure proper pipe connection and positive flow is maintained post construction. As such, pipe bedding should be placed to be of uniform thickness and compactness. The pipe bedding and cover material should conform to OPSD 802.010 and 802.013 specifications for flexible pipes and to OPSD 802.031 to 802.033 with Class "B" bedding for rigid pipes.

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

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

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

5.9.2 Trench Backfill

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

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

Portions of the silt and clay, and silty clay may have a blocky/lumpy texture. If the large interclump voids are not closed completely by thorough compaction, then long-term softening/settlement will occur. The trench backfill should be placed in thin lifts (less than 300 mm) and compacted with a sheepsfoot roller. Particular attention must be made to backfilling service connections where the trenches are narrow.

All stockpiled material should be protected from deleterious materials, additional moisture and be kept from freezing.

Quality control will be the utmost importance when selecting the material. The selection of the material should be done as early in the contract as possible to allow sufficient time for gradation and proctor testing on representative samples to ensure it meets the project specifications.

Where the natural soil will be exposed, adequate compaction may prove difficult if the material becomes wet (i.e., above the optimum moisture content). Depending on the moisture content of the natural materials at the time of construction, they may either require moisture to be added or stockpiled and left to dry to achieve moisture content within plus 2% to minus 4% of optimum. The natural soil at this Site is subject to moisture content increase during wet weather. As such, stockpiles should be protected to help minimize moisture absorption during wet weather.

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

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

5.9.3 Frost Protection

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

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

5.10 Asphaltic Concrete Pavement Structure Design for Parking Lot and Driveways

5.10.1 Discussion

Parking areas and driveway access will be constructed around the proposed building. The in-situ till soil is considered a sufficient bearing material for an asphaltic concrete pavement structure provided all fill, granular fill and deleterious materials are removed prior to installing the engineered fill material.

At this time Pinchin is unaware of the proposed final grades for the parking areas and access roadways. As such, provided the pavement structure overlies the in-situ till material, the following pavement structure is recommended.

5.10.2 Pavement Structure

The following table presents the minimum specifications for a flexible asphaltic concrete pavement structure:

Pavement Layer	Compaction Requirements	Parking Areas	Driveways
Surface Course Asphaltic Concrete HL-3 (OPSS 1150)	92% MRD as per OPSS 310	40 mm	40 mm
Binder Course Asphaltic Concrete HL-8 (OPSS 1150)	92 % MRD as per OPSS 310	50 mm	85 mm
Base Course: Granular "A" (OPSS 1010)	100% Standard Proctor Maximum Dry Density (ASTM-D698)	150 mm	150 mm
Subbase Course: Granular "B" Type I (OPSS 1010)	100% Standard Proctor Maximum Dry Density (ASTM D698)	300 mm	450 mm

Notes:

I. Prior to placing the pavement structure, the subgrade soil is to be proof rolled with a smooth drum roller without vibration to observe weak spots and the deflection of the soil; and

II. The recommended pavement structure may have to be adjusted according to the City of Ottawa standards. Also, if construction takes place during times of substantial precipitation and the subgrade soil becomes wet and disturbed, the granular thickness may have to be increased to compensate for the weaker subgrade soil. In addition, the granular fill material thickness may have to be temporarily increased to allow heavy construction equipment to access the Site, in order to avoid the subgrade from "pumping" up into the granular material.

Performance grade PG 58-34 asphaltic concrete should be specified for Marshall mixes.

5.10.3 Pavement Structure Subgrade Preparation and Granular up Fill

The proper placement of base and subbase fill materials becomes very important in addressing the proper load distribution to provide a durable pavement structure.

The pavement subgrade materials should be thoroughly proof-rolled prior to placement of the Granular 'B' subbase course. If any unstable areas are noted, then the Granular 'B' thickness may need to be increased to support pavement construction traffic. This should be left as a field decision by a qualified geotechnical engineer at the time of construction, but it is recommended that additional Granular 'B' be carried as a provisional item under the construction contract.

Where fill material is required to increase the grade to the underside of the pavement structure it should consist of Granular 'B' Type I (OPSS 1010). The up-fill material is to be placed in maximum 300 mm thick lifts compacted to 98% SPMDD within 4% of the optimum moisture content.

Samples of both the Granular 'A' and Granular 'B' Type I aggregates should be tested for conformance to OPSS 1010 prior to utilization on Site and during construction. All stockpiled material should be protected from deleterious materials, additional moisture and be kept from freezing.

Post compaction settlement of fine-grained soil can be expected, even when placed to compaction specifications. As such, fill material should be installed as far in advance as possible before finishing the parking lot and access roadways for best grade integrity.

Where the subgrade material types differ below the underside of the pavement structure, the transition between the materials should be sloped as per frost heave taper OPSD 205.60.

5.10.4 Drainage

Control of surface water is a critical factor in achieving good pavement structure life. The pavement thickness designs are based on a drained pavement subgrade via sub-drains or ditches.

The till soils have poor natural drainage and therefore it is recommended that pavement subdrains be installed in the lower areas and be connected to the catch basins. Subdrains should comprise 150 mm diameter perforated pipe in filter sock, bedded in concrete sand. The upper limit of the subdrain bedding should be at the lower limit of the pavement subbase, with the subgrade below the subbase sloped towards the subdrain. Subdrains must drain to a suitable frost-free outlet.

The surface of the roadways should be free of depressions and be sloped at a minimum grade of 1% in order to drain to appropriate drainage areas. Subgrade soil should slope a minimum of 3% toward stormwater collection points. Positive slopes are very important for the proper performance of the drainage system. The granular base and subbase materials should extend horizontally to any potential ditches or swales.

In addition, routine maintenance of the drainage systems will assist with the longevity of the pavement structure. Ditches, culverts, sewers and catch basins should be regularly cleared of debris and vegetation.

6.0 SOIL CORROSIVITY AND SULPHATE ATTACK ON CONCRETE

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

Parameter	BH104, SS3 1.5 – 2.1 mbgs	
	Results	**Points**
Resistivity (ohm-cm)	1120	10
pH	8.64	3
Redox Potential (mV)	230	0
Sulfide	0.31	2
Moisture	Poor drainage	2
Total Points		**17**

In summary, the tested sample does indicate a strong potential for soil corrosivity, and additional protective measures are required. The results should be reviewed by the structural engineer. As discussed in American Water Work Association A21.5 Standard, the solution for corrosivity of iron pipes is based off the combination of likelihood and consequence factors. Consequence factors are determined by the operational reliability and factors such as: the diameter of the pipe, the location of the pipe, the depth of cover and whether an alternative supply of water is available.

The results of the sulphate testing indicate that the Site possesses low to medium sulphate exposure. The results should be reviewed by the structural engineer to ensure conformance to the concrete exposures.

7.0 SITE SUPERVISION & QUALITY CONTROL

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

8.0 TERMS AND LIMITATIONS

This Geotechnical Investigation was performed for the exclusive use of Morguard REIT c/o Morguard Investments Limited (Client) in order to evaluate the subsurface conditions at 500 Coventry 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.

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

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

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

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

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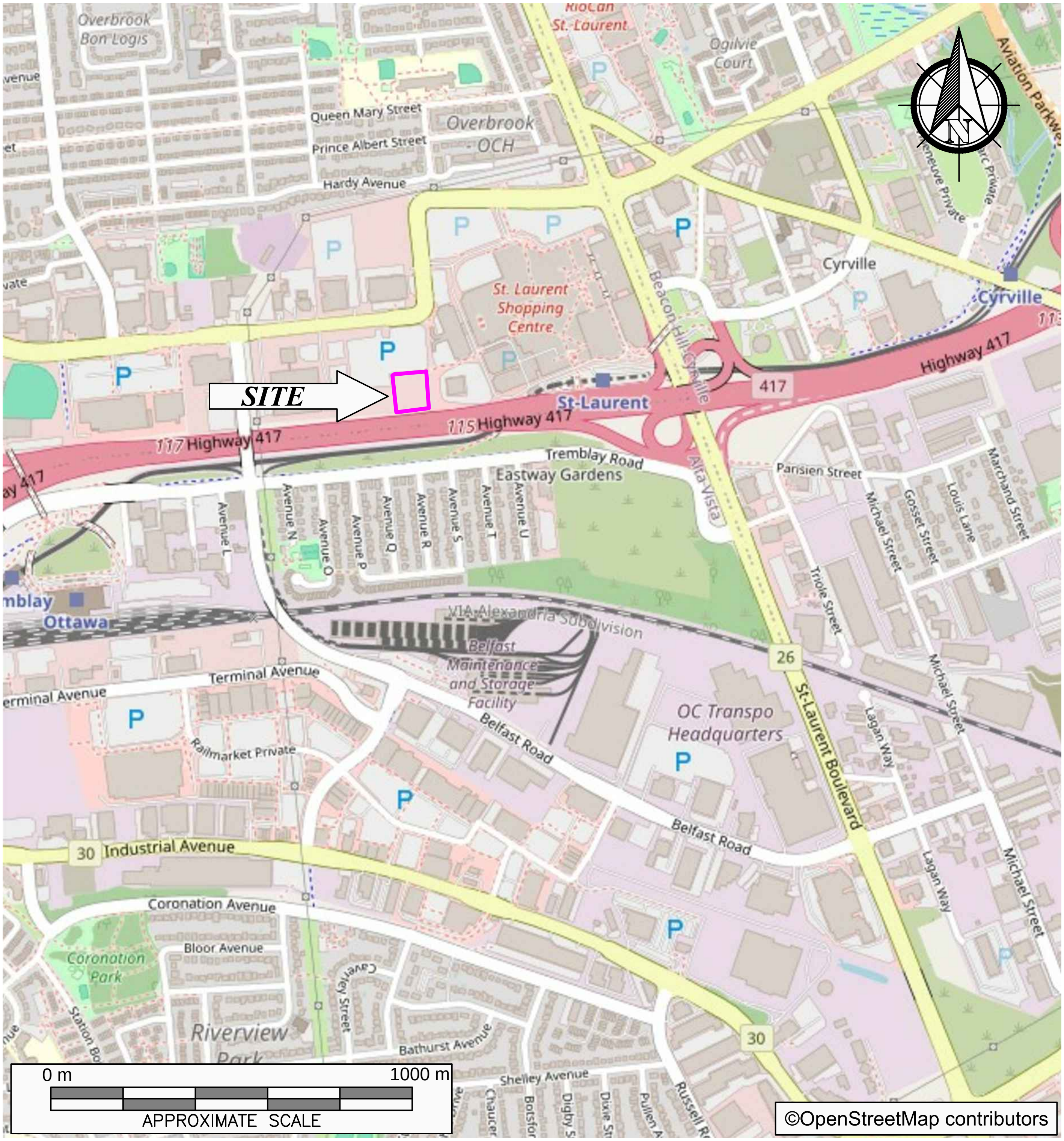
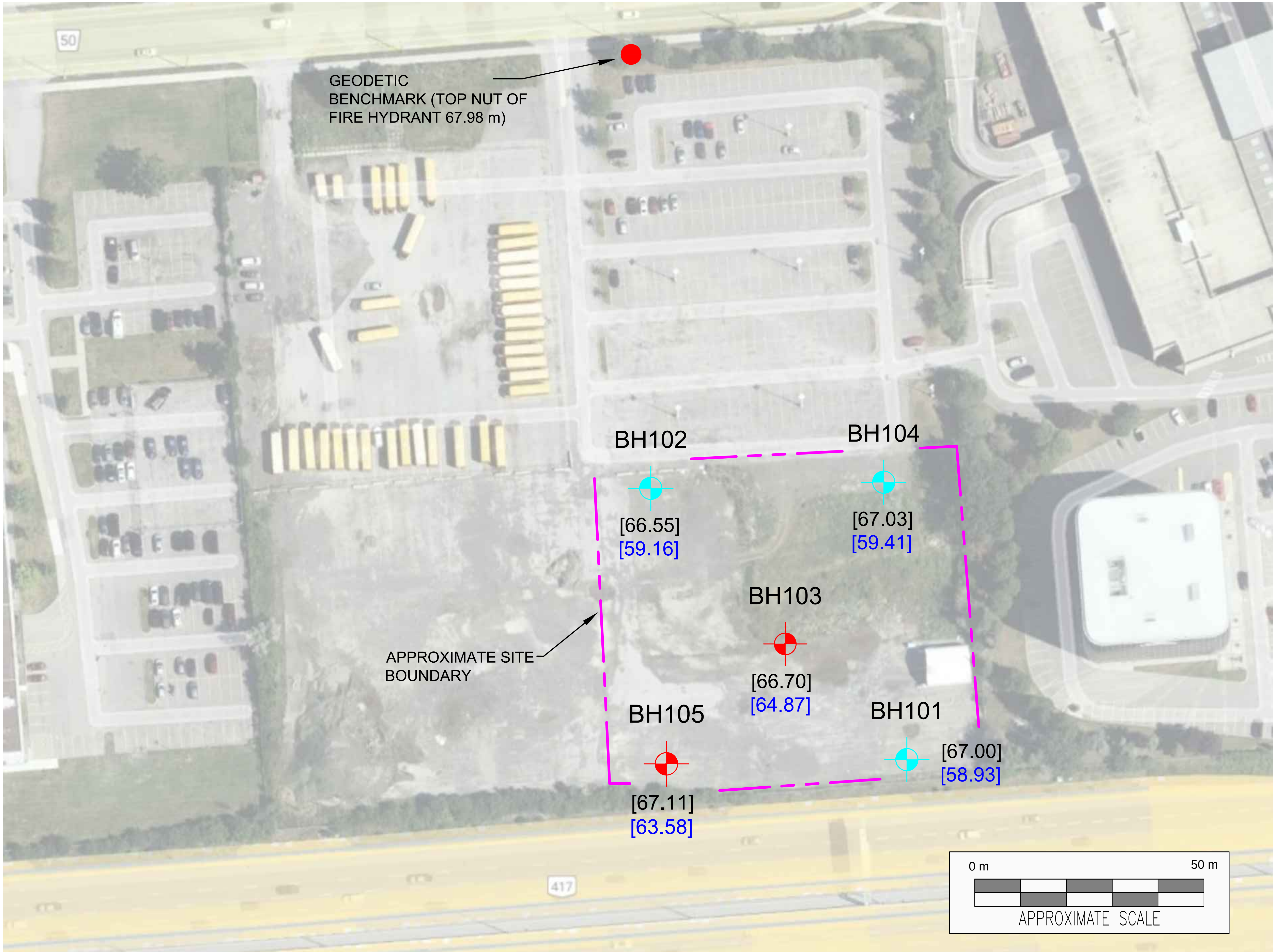
Template: Master Geotechnical Investigation Report – Ontario, GEO, September 2, 2021

FIGURES



PROJECT NAME	GEOTECHNICAL INVESTIGATION
CLIENT NAME	MORGUARD REIT
PROJECT LOCATION	500 COVENTRY ROAD, OTTAWA, ONTARIO
FIGURE NAME	KEY MAP
FIGURE NO.	1
APPROXIMATE SCALE	AS SHOWN
PROJECT NO.	319674.001
DATE	NOVEMBER 2024



LEGEND

APPROXIMATE BOREHOLE LOCATION

APPROXIMATE MONITORING WELL LOCATION

[XX.XX] APPROXIMATE GEODETIC GROUND SURFACE ELEVATION (masl)

[XX.XX] APPROXIMATE GEODETIC TERMINATION ELEVATION (masl)

PINCHIN

PROJECT NAME GEOTECHNICAL INVESTIGATION	
CLIENT NAME MORGUARD REIT	
PROJECT LOCATION 500 COVENTRY ROAD, OTTAWA, ONTARIO	
FIGURE NAME BOREHOLE LOCATION PLAN	
APPROXIMATE SCALE AS SHOWN	PROJECT NO. 319674.001
DATE NOVEMBER 2024	FIGURE NO. 2

TABLES

Table 1

GROUNDWATER MONITORING WELL ELEVATIONS AND CONSTRUCTION DETAILS

Morguard REIT c/o Morguard Investments Limited

500 Coventry Road, Ottawa, Ontario

Monitoring Well	Top of Pipe Elevation (mamsl)	Ground Surface Elevation (mamsl)	Well Construction Details						
			Total Well Depth (mbgs)	Stick-Up Height (metres)	Well Diameter (centimetres)	Screen Slot Size	Monitoring Well Screen Interval (mbgs)	Screen length (metres)	Sealant thickness (metres)
BH101	66.90	67.00	8.0	-0.10	5.1	010	4.6 - 7.6	3.1	4.3
BH102	66.45	66.55	7.4	-0.10	5.1	010	4.3 - 7.4	3.1	4.0
BH104	67.78	67.03	7.6	0.75	5.1	010	4.6 - 7.6	3.1	4.3

Notes:

mamsl metres above mean sea level
mbgs metres below ground surface

Table 2
GROUNDWATER MONITORING DATA
Morguard REIT c/o Morguard Investments Limited
500 Coventry Road, Ottawa, Ontario

Monitoring Well	Monitoring Well Screen Interval (mbgs)	Top of Pipe Elevation (mamsl)	Ground Surface Elevation (mamsl)	Stick-Up Height (metres)	April 17, 2024 Measured Depth to Groundwater from Top of Casing (mbtoc)	April 17, 2024 Calculated Depth to Groundwater from Surface (mbgs)	April 17, 2024 Groundwater Elevation (mamsl)	June 21, 2024 Measured Depth to Groundwater from Top of Casing (mbtoc)	June 21, 2024 Calculated Depth to Groundwater from Surface (mbgs)	June 21, 2024 Groundwater Elevation (mamsl)	July 9, 2024 Measured Depth to Groundwater from Top of Casing (mbtoc)	July 9, 2024 Calculated Depth to Groundwater from Surface (mbgs)	July 9, 2024 Groundwater Elevation (mamsl)
BH101	4.6 - 7.6	66.90	67.00	-0.10	2.86	2.96	64.04	3.30	3.40	63.60	3.17	3.27	63.73
BH102	4.3 - 7.4	66.45	66.55	-0.10	1.24	1.34	65.21	1.68	1.78	64.77	1.26	1.36	65.19
BH104	4.6 - 7.6	67.78	67.03	0.75	2.77	2.02	65.01	3.18	2.43	64.60	2.98	2.23	64.80
				Minimum =		1.34	63.60						
				Maximum =		3.40	65.21						

Notes:

mamsl metres above mean sea level
mbgs metres below ground surface
mbtop metres below top of pipe
NM Not Measured

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:

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

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: BH101

Project #: 319674.001 **Logged By:** MK

Project: Geotechnical Investigation

Client: Morguard REIT

Location: 500 Coventry Road, Ottawa, Ontario

Drill Date: April 8, 2024 **Project Manager:** MK

SUBSURFACE PROFILE

Depth (m)	Description	Elevation (m)
	Ground Surface	67.00
0.00	**Granular Fill** Sand and gravel, some silt, grey, damp, compact	67.00
0.76	**Silty Clayey Sand Till** Silty clayey sand with gravel till, brown, damp, compact	66.24
2.29	**Bedrock** Limestone bedrock, dark grey with black and light grey banding and spotting, very poor quality moderately weathered	64.72
3.35	Very poor quality	63.65
4.88	Fair quality, slightly weathered	62.13
6.55	Excellent quality	60.45
8.08	End of Borehole. Borehole terminated at 8.0 mbgs in bedrock.	58.93

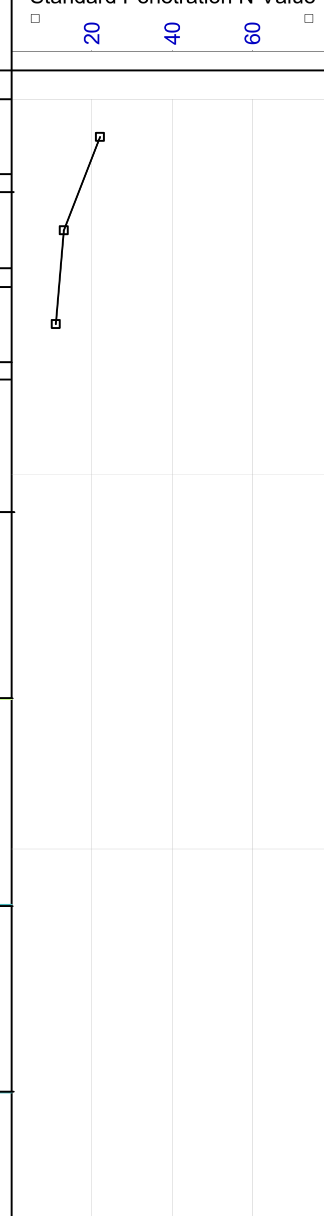
Monitoring Well Details: Riser; Bentonite; Screen; Silica Sand

Groundwater level = 3.27 mbgs, as measured on July 9, 2024.

SAMPLE

Sample Type	Sampler #	Recovery (%)	SPT N-Value	Rock Quality Designation (RQD) (%)	Laboratory Analysis
SS	1	60	22		
SS	2	60	13		
SS	3	70	11		
RC	1	67	NA	10	
RC	2	38	NA	0	
RC	3	75	NA	56	
RC	4	100	NA	97	

Standard Penetration N-Value



Contractor: Strata Drilling Group **Grade Elevation:** 67.00 m

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

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

Log of Borehole: BH102

Project #: 319674.001 **Logged By:** MK

Project: Geotechnical Investigation

Client: Morguard REIT

Location: 500 Coventry Road, Ottawa, Ontario

Drill Date: April 8, 2024 **Project Manager:** MK

SUBSURFACE PROFILE

Depth (m)	Description	Elevation (m)
0	Ground Surface	66.55 / 0.00
0.00–1.07	***Granular Fill*** Sand and gravel, some silt, grey, damp, compact	
1.07	***Fill*** Silt and sand with some gravel, some cobbles, brown, moist, compact	65.49 / 1.07
1.52	Refusal on boulder	65.03 / 1.52
1.52–3.05	***Silty Clayey Sand Till*** Silty clayey sand with gravel till, brown, damp, compact	
3.05	***Bedrock*** Limestone bedrock, dark grey with black and light grey banding and spotting, very poor to fair quality, moderately weathered to slightly weathered	63.51 / 3.05
5.03	Fair quality	61.52 / 5.03
6.58	Excellent quality	59.97 / 6.58
7.39	End of Borehole	59.16 / 7.39

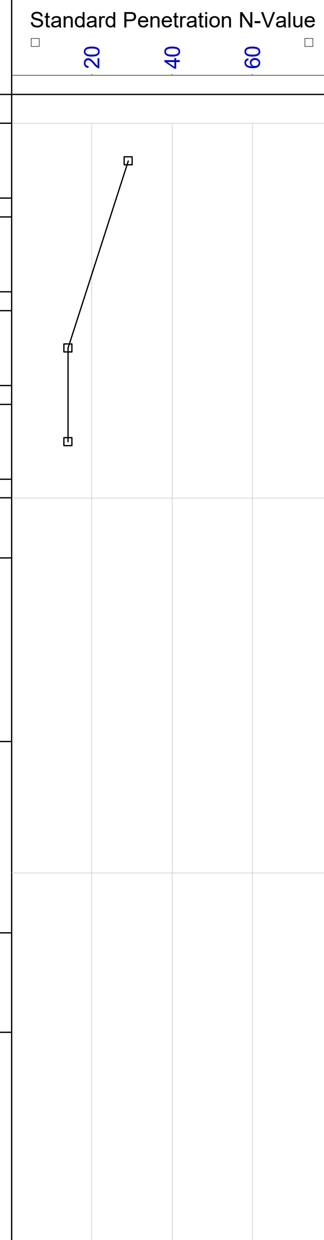
Borehole terminated at 7.4 mbgs in bedrock.

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

Groundwater level = 1.36 mbgs, as measured on Jul 9, 2024.

SAMPLE

Sample Type	Sampler #	Recovery (%)	SPT N-Value	Rock Quality Designation (RQD) (%)	Laboratory Analysis
SS	1	50	29		
SS	2	10	R		
SS	3	0	14		
SS	4	40	14		GS, MC; Att. Lim.
RC	1	95	NA	20	
RC	2	100	NA	65	
RC	3	100	NA	62	
RC	4	100	NA	90	



Contractor: Strata Drilling Group **Grade Elevation:** 66.55 m

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

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

Log of Borehole: BH103

Project #: 319674.001 **Logged By:** MK

Project: Geotechnical Investigation

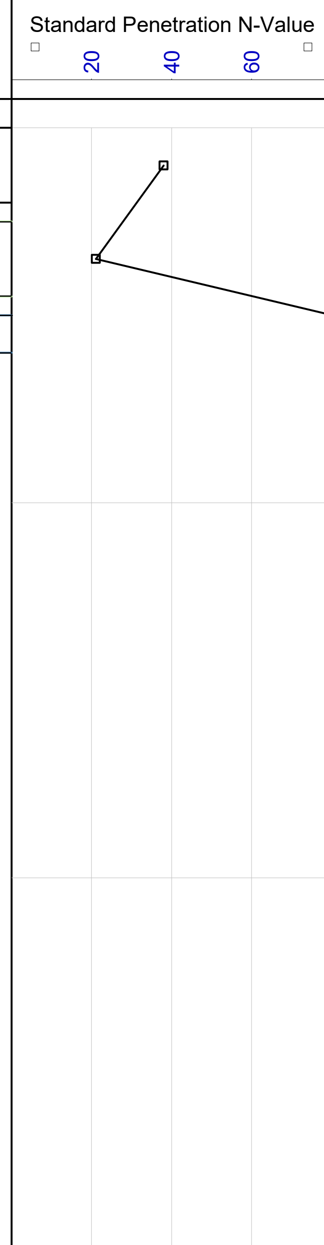
Client: Morguard REIT

Location: 500 Coventry Road, Ottawa, Ontario

Drill Date: April 8, 2024 **Project Manager:** MK

SUBSURFACE PROFILE				SAMPLE						
Depth (m)	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-Value	Standard Penetration N-Value	Soil Vapour Concentration (ppm)	Laboratory Analysis
	Ground Surface	66.70								
0	**Granular Fill** Sand and gravel, some silt, grey, damp to moist, dense	0.00	No Monitoring Well	SS	1	50	38			
		65.94								
1	**Silty Clayey Sand Till** Silty clayey sand with gravel till, brown, damp, compact to very dense	0.76		SS	2	60	21			
				SS	3	40	100			Hyd., MC Att. Lim.
		64.87								
2	End of Borehole	1.83								

Borehole terminated at 1.8 mbgs due to refusal on probable bedrock. At drilling completion, groundwater was not encountered.



Contractor: Strata Drilling Group **Grade Elevation:** 66.70 m

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

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

Log of Borehole: BH104

Project #: 319674.001 **Logged By:** MK

Project: Geotechnical Investigation

Client: Morguard REIT

Location: 500 Coventry Road, Ottawa, Ontario

Drill Date: April 9, 2024 **Project Manager:** MK

SUBSURFACE PROFILE

Depth (m)	Description	Elevation (m)
	Ground Surface	67.03
0.00	**Fill** Silty sand, some gravel, brown, damp, compact	66.27
0.76	**Silty Clayey Sand Till** Silty clayey sand with gravel till, brown, damp to moist, compact	65.51
1.52	Wet	64.75
2.29	Dense	64.21
2.82	**Bedrock** Airhammer	59.41
7.62	End of Borehole Sampled borehole terminated at 2.8 mbgs due to refusal on probable bedrock. Monitoring well installed at 7.6 mbgs.	

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

Groundwater = 2.23 mbgs, as measured on July 9, 2024.

SAMPLE

Sample Type	Sampler #	Recovery (%)	SPT N-Value	Soil Vapour Concentration (ppm)	Laboratory Analysis
SS	1	60	14		
SS	2	60	10		
SS	3	70	27		
SS	4	10	40		

Contractor: Strata Drilling Group

Drilling Method: Split Spoon Sample

Well Casing Size: 50 mm

Grade Elevation: 67.03 m

Top of Casing Elevation: N/A

Sheet: 1 of 1

Log of Borehole: BH105

Project #: 319674.001 **Logged By:** MK

Project: Geotechnical Investigation

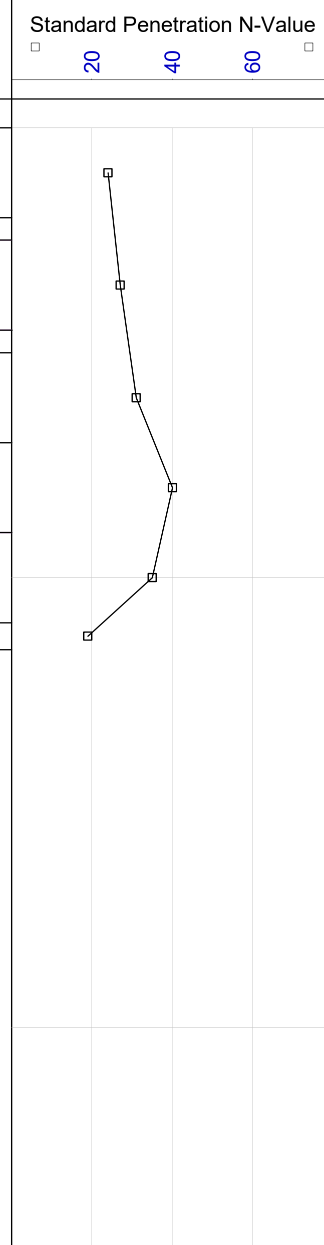
Client: Morguard REIT

Location: 500 Coventry Road, Ottawa, Ontario

Drill Date: April 9, 2024 **Project Manager:** MK

SUBSURFACE PROFILE / **SAMPLE**

Depth (m)	Symbol	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-Value	Soil Vapour Concentration (ppm)	Laboratory Analysis
		Ground Surface	67.11							
0		**Granular Fill** Sand and gravel, some silt, grey, damp, compact	0.00	No Monitoring Well	SS	1	60	24		GS, MC
		Silty Clayey Sand Till Silty clayey sand with gravel till, brown, damp, compact	66.81 0.30		SS	2	80	27		
		Wet, pieces of shale	65.59 1.52		SS	3	30	31		
2		Dense, pieces of bedrock	64.98 2.13		SS	4	40	40		
3					SS	5	80	35		GS, MC
					SS	6	20	19		
		End of Borehole	63.58 3.54							



Borehole terminated at 3.5 mbgs due to refusal on probable bedrock. At drilling completion, groundwater was encountered at 1.5 mbgs.

Contractor: Strata Drilling Group **Grade Elevation:** 67.11 m

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

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

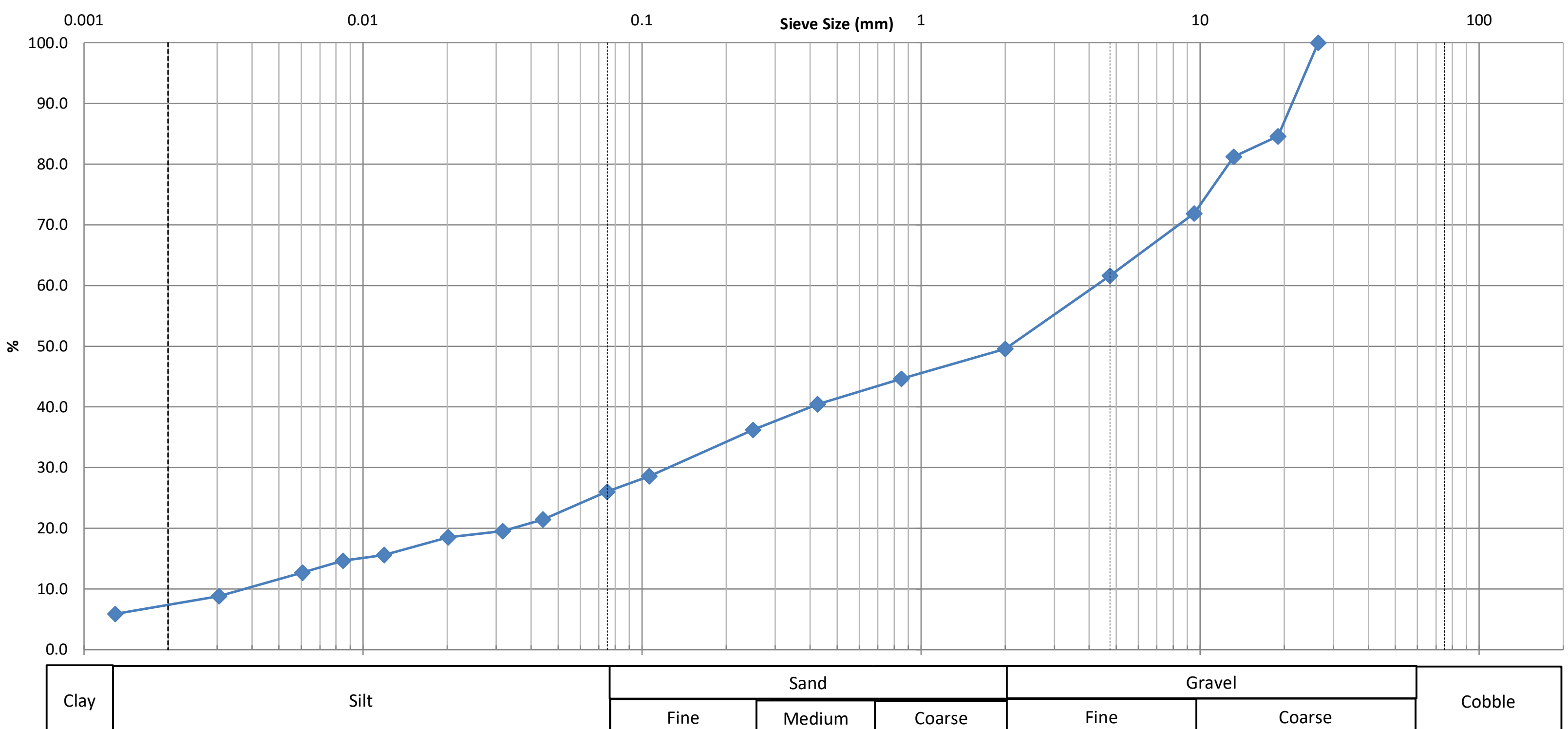
APPENDIX III

Laboratory Testing Reports for Soil Samples

PATERSON GROUP

SIEVE ANALYSIS
ASTM C136

CLIENT:	Pinchin	DEPTH:	7'6" - 9'6"	FILE NO:	PM4184
CONTRACT NO.:		BH OR TP No.:	BH102 SS4	LAB NO:	51539
PROJECT:	319674.001			DATE RECEIVED:	12-Apr-24
				DATE TESTED:	16-Apr-24
DATE SAMPLED:	-			DATE REPORTED:	22-Apr-24
SAMPLED BY:	-			TESTED BY:	D.K



Identification	Soil Classification				MC(%)	LL	PL	PI	Cc	Cu
					9.1%					
	D100	D60	D30	D10	Gravel (%)	Sand (%)	Silt (%)		Clay (%)	
					38.4	35.6	18.0		8.0	

Comments:

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

PATERSON GROUP

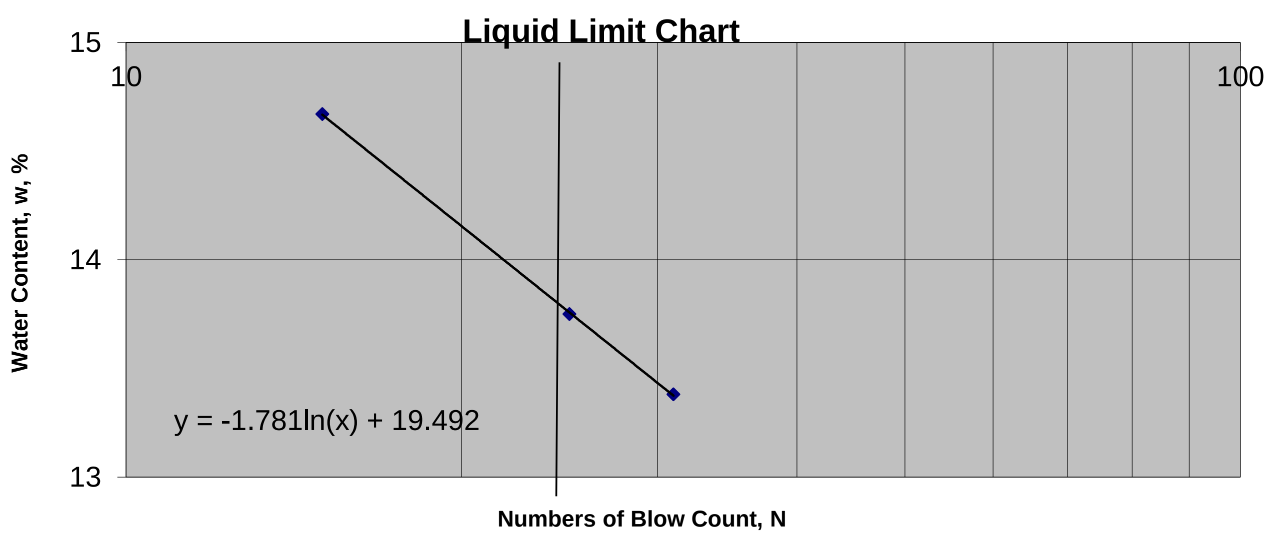
ATTERBERG LIMITS
LS-703/704

CLIENT:	Pinchin	FILE NO.:	PM4184
PROJECT:	319674.001	DATE SAMPLED:	-
LOCATION:	BH102 SS4 @ 7'6" - 9'6"	DATE REPORTED:	19-Apr

CAN NO.	**30**	**31**	**32**				
WT. OF CAN	4.38	4.36	4.36				
WT. OF SOIL & CAN	20.33	18.75	18.77				
WT. OF DRY SOIL & CAN	18.29	17.01	17.07				
WT. OF MOISTURE	2.04	1.74	1.7				
WT. OF DRY SOIL & CAN	13.91	12.65	12.71				
WATER CONTENT, w, %	**14.67**	**13.75**	**13.38**				
NO. OF BLOWS, N	15	25	31				

CAN NO.	**14**	**14**
WT. OF CAN	16.74	19.94
WT. OF SOIL & CAN	26.50	29.49
WT. OF DRY SOIL & CAN	25.52	28.52
WT. OF MOISTURE	0.98	0.97
WT. OF DRY SOIL & CAN	8.78	8.58
WATER CONTENT, w, %	**11.16**	**11.31**

RESULTS	
LIQUID LIMIT	**14**
PLASTIC LIMIT	**11**
PLASTICITY INDEX	**3**

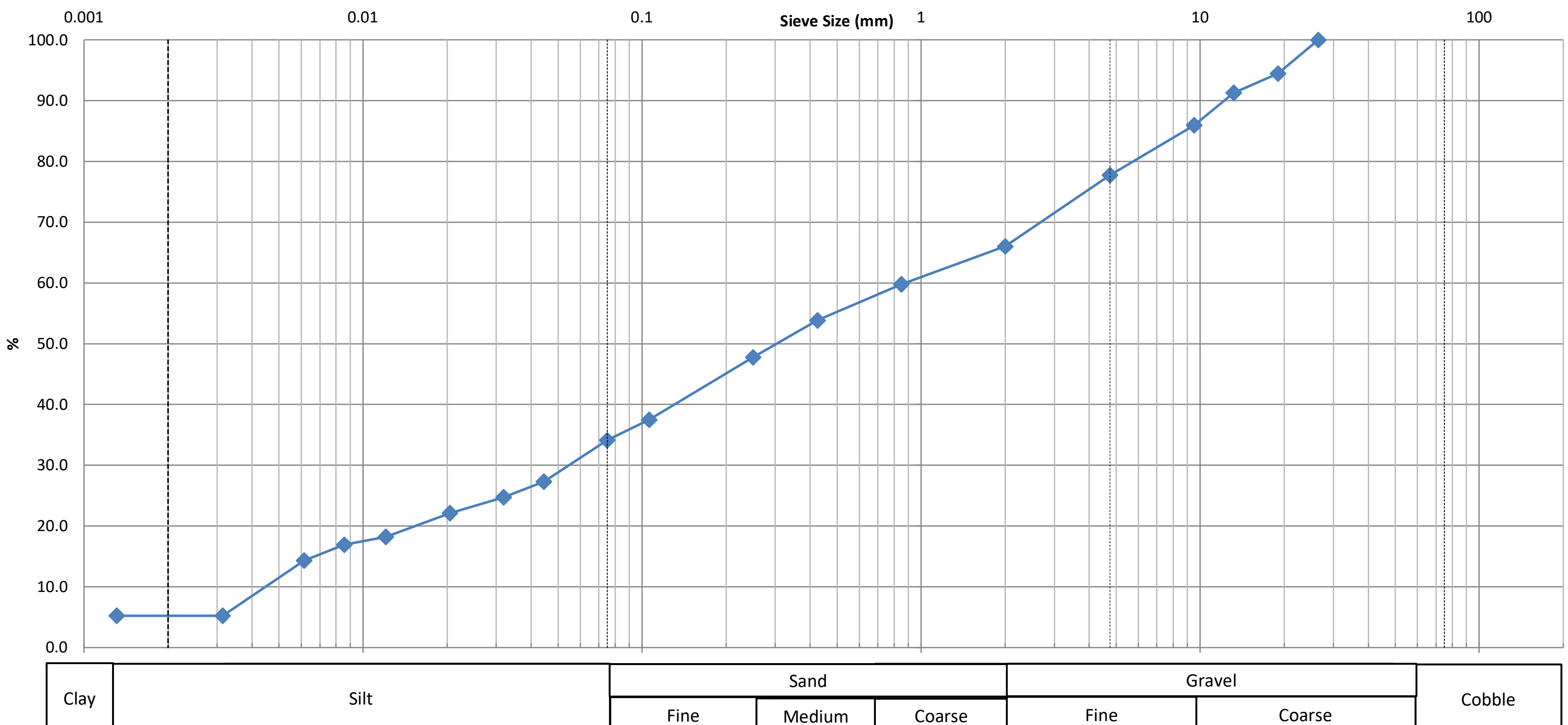


TECHNICIAN: CP	**REVIEWED BY:**	**C. Beadow**	**J. Forsyth, P. Eng.**

PATERSON GROUP

SIEVE ANALYSIS
ASTM C136

CLIENT:	Pinchin	DEPTH:	5' - 7'	FILE NO:	PM4184
CONTRACT NO.:		BH OR TP No.:	BH103 SS3	LAB NO:	51538
PROJECT:	319674.001			DATE RECEIVED:	12-Apr-24
				DATE TESTED:	16-Apr-24
DATE SAMPLED:	-			DATE REPORTED:	22-Apr-24
SAMPLED BY:	-			TESTED BY:	D.K



Identification	Soil Classification				MC(%)	LL	PL	PI	Cc	Cu
					7.5%					
	D100	D60	D30	D10	Gravel (%)	Sand (%)	Silt (%)	Clay (%)		
					22.2	43.7	29.1	5.0		

Comments:

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

PATERSON GROUP

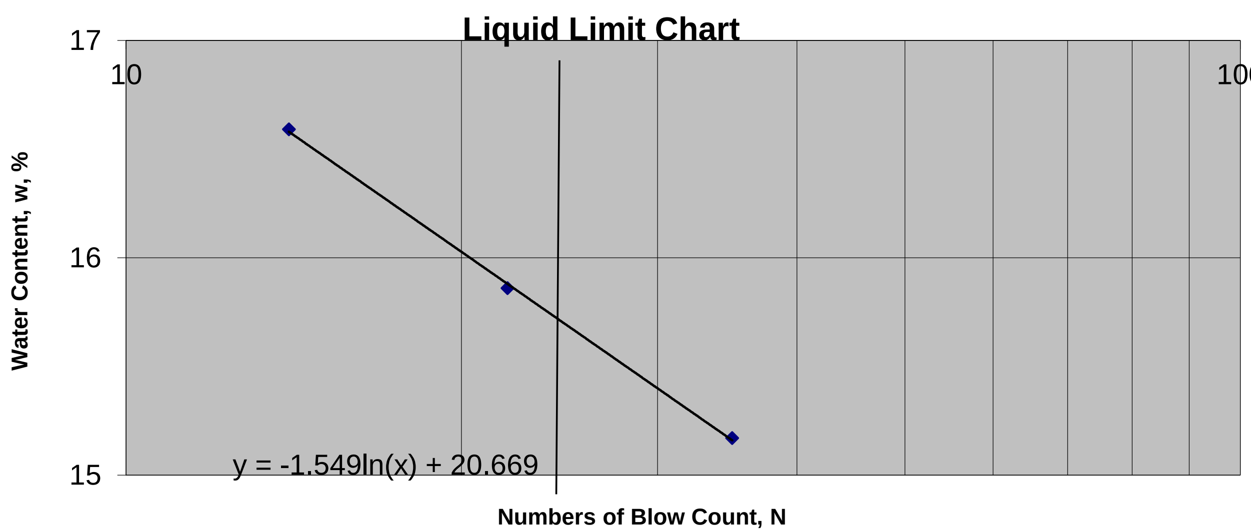
ATTERBERG LIMITS
LS-703/704

CLIENT:	Pinchin	FILE NO.:	PM4184
PROJECT:	319674.001	DATE SAMPLED:	-
LOCATION:	BH103 SS3 @ 5' - 7'	DATE REPORTED:	19-Apr

CAN NO.	**3**	**4**	**13**				
WT. OF CAN	8.70	8.68	8.68				
WT. OF SOIL & CAN	23.46	27.38	26.82				
WT. OF DRY SOIL & CAN	21.36	24.82	24.43				
WT. OF MOISTURE	2.1	2.56	2.39				
WT. OF DRY SOIL & CAN	12.66	16.14	15.75				
WATER CONTENT, w, %	**16.59**	**15.86**	**15.17**				
NO. OF BLOWS, N	14	22	35				

CAN NO.	**15**	**18**
WT. OF CAN	19.91	20.01
WT. OF SOIL & CAN	29.74	29.77
WT. OF DRY SOIL & CAN	28.73	28.74
WT. OF MOISTURE	1.01	1.03
WT. OF DRY SOIL & CAN	8.82	8.73
WATER CONTENT, w, %	**11.45**	**11.8**

RESULTS	
LIQUID LIMIT	**16**
PLASTIC LIMIT	**12**
PLASTICITY INDEX	**4**

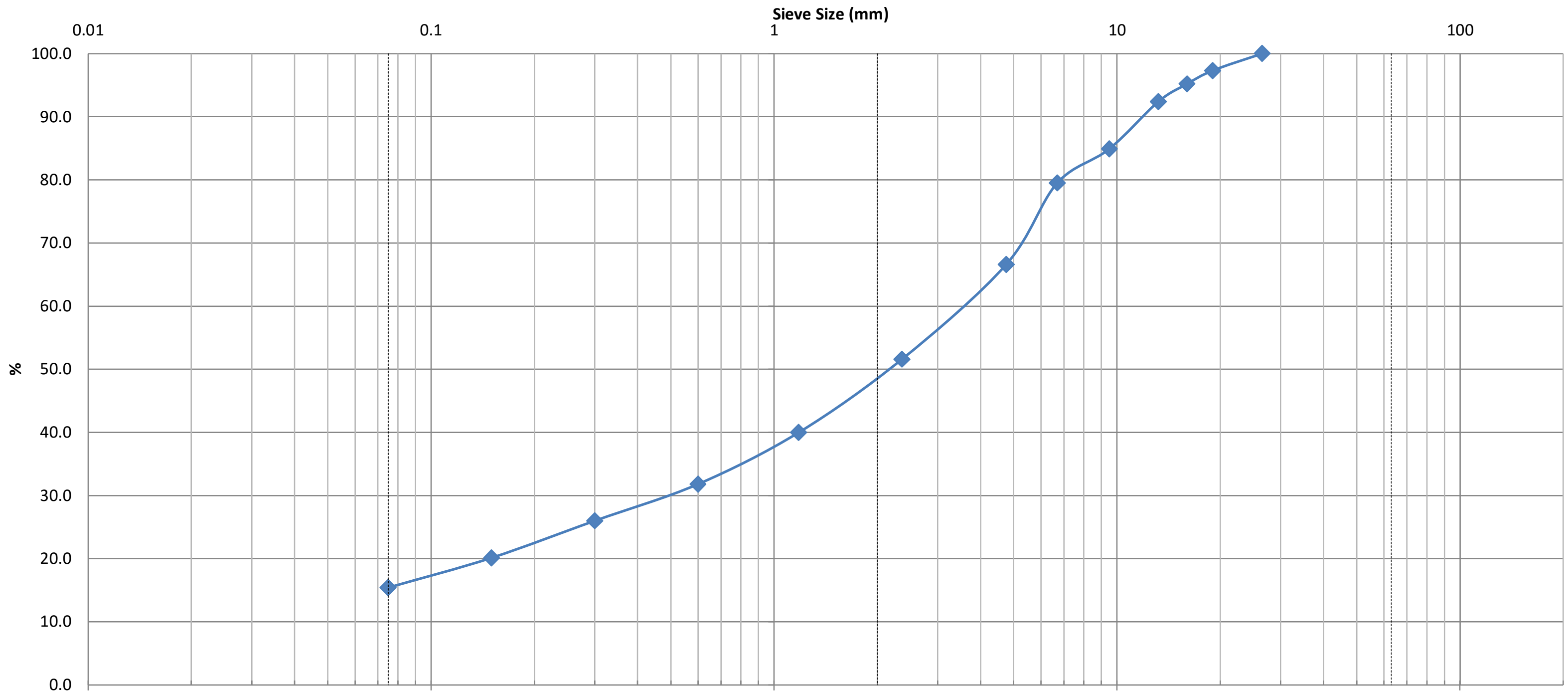


TECHNICIAN: CP	REVIEWED BY:	C. Beadow	J. Forsyth, P. Eng.

PATERSON GROUP

SIEVE ANALYSIS
ASTM C136

CLIENT:	Pinchin	DESCRIPTION:	Sand	FILE NO:	PM4184
CONTRACT NO.:	-	SPECIFICATION:	-	LAB NO:	51541
PROJECT:	319674.001	INTENDED USE:	-	DATE RECEIVED:	12-Apr-24
		PIT OR QUARRY:	-	DATE TESTED:	15-Apr-24
DATE SAMPLED:	-	SOURCE LOCATION:	BH105 - SS1	DATE REPORTED:	22-Apr-24
SAMPLED BY:	Client	SAMPLE LOCATION:	0' - 2'	TESTED BY:	CP



Silt and Clay	Sand			Gravel		Cobble
	Fine	Medium	Coarse	Fine	Coarse	

Identification	Soil Classification	MC(%)	LL	PL	PI	Cc	Cu
		4.3%				1.93	101.4

D100	D60	D30	D10	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
26.5	3.55	0.49	0.035	33.4	51.2	15.4	

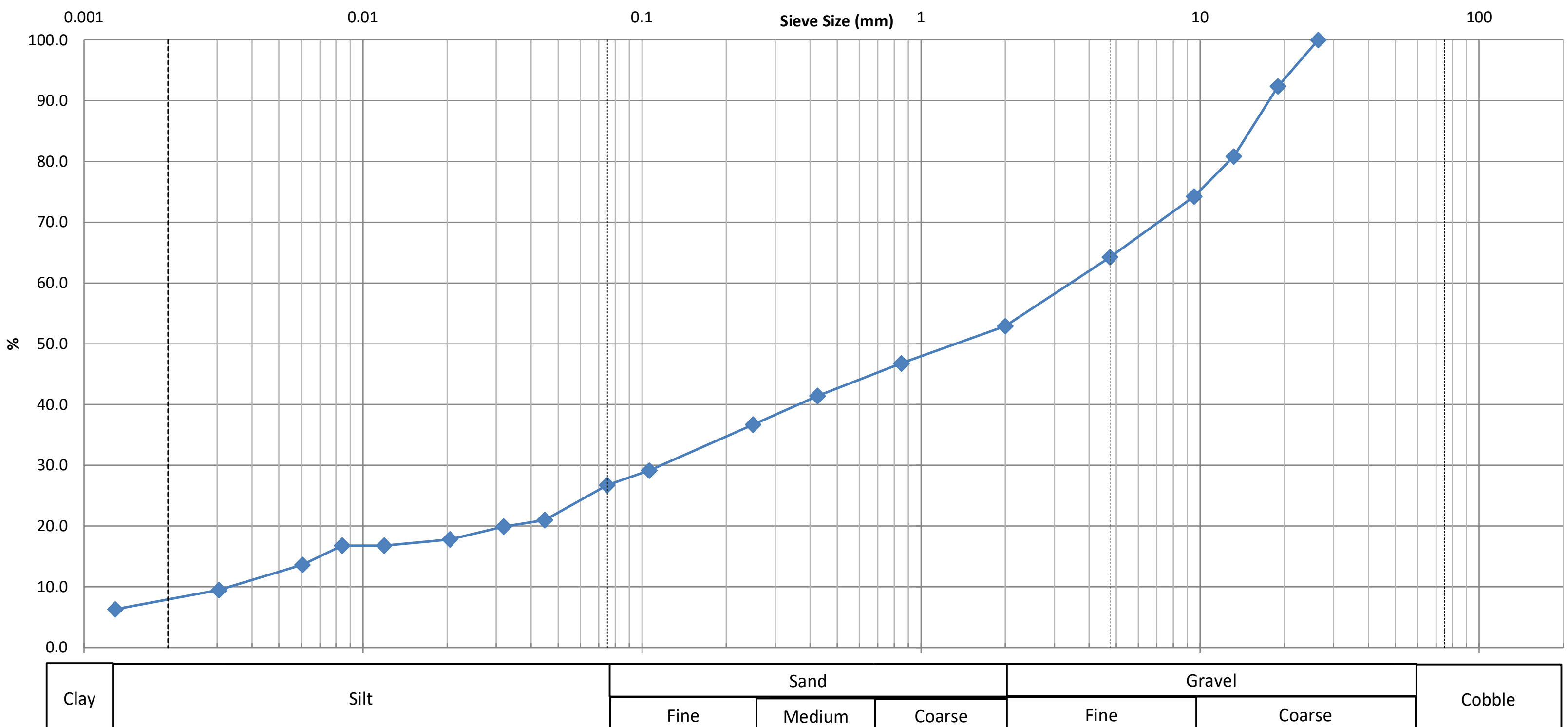
Comments:

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

PATERSON GROUP

SIEVE ANALYSIS
ASTM C136

CLIENT:	Pinchin	DEPTH:	9' - 11'	FILE NO:	PM4184
CONTRACT NO.:		BH OR TP No.:	BH105 SS5	LAB NO:	51540
PROJECT:	319674.001			DATE RECEIVED:	12-Apr-24
				DATE TESTED:	16-Apr-24
DATE SAMPLED:	-			DATE REPORTED:	19-Apr-24
SAMPLED BY:	-			TESTED BY:	D.K



Identification	Soil Classification				MC(%)	LL	PL	PI	Cc	Cu
					9.8%					
	D100	D60	D30	D10	Gravel (%)	Sand (%)	Silt (%)	Clay (%)		
					35.8	37.5	18.7	8.0		

Comments:

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

FINAL REPORT

CA15681-APR24 R1

319674.001

Prepared for

Pinchin Ltd

FINAL REPORT

First Page

CLIENT DETAILS		LABORATORY DETAILS	
Client	Pinchin Ltd	Project Specialist	Jill Campbell, B.Sc.,GISAS
		Laboratory	SGS Canada Inc.
Address	1 Hines Road, Suite 200 Kanata, ON K2K 3C7, Canada	Address	185 Concession St., Lakefield ON, K0L 2H0
Contact	Megan Keon	Telephone	2165
Telephone	613-608-5350	Facsimile	705-652-6365
Facsimile		Email	jill.campbell@sgs.com
Email	mkeon@Pinchin.com	SGS Reference	CA15681-APR24
Project	319674.001	Received	04/15/2024
Order Number		Approved	04/19/2024
Samples	Soil (1)	Report Number	CA15681-APR24 R1
		Date Reported	04/19/2024

COMMENTS

Temperature of Sample upon Receipt: 13 degrees C

Cooling Agent Present: Yes

Custody Seal Present: Yes

Chain of Custody Number: n/a

Corrosivity Index is based on the American Water Works Corrosivity Scale according to AWWA C-105. An index greater than 10 indicates the soil matrix may be corrosive to cast iron alloys.

SIGNATORIES

Jill Campbell, B.Sc.,GISAS

Jill Campbell

TABLE OF CONTENTS

FINAL REPORT

Client: Pinchin Ltd
Project: 319674.001
Project Manager: Megan Keon
Samplers: Megan Keon

MATRIX: SOIL

Sample Number	5
Sample Name	BH104 SS3 5-7 ft.
Sample Matrix	Soil
Sample Date	09/04/2024

Parameter	Units	RL		Result
Corrosivity Index				
Corrosivity Index	none	1		18
Soil Redox Potential	mV	no		230
Sulphide (Na2CO3)	%	0.01		0.31
pH	pH Units	0.05		8.64
Resistivity (calculated)	ohms.cm	-9999		1120
General Chemistry				
Conductivity	uS/cm	2		892
Metals and Inorganics				
Moisture Content	%	0.1		7.7
Sulphate	µg/g	0.4		110
Other (ORP)				
Chloride	µg/g	0.4		310

QC SUMMARY

Anions by IC

Method: EPA300/MA300-Ions1.3 | Internal ref.: ME-CA-[ENV]IC-LAK-AN-001

Parameter	QC batch Reference	Units	RL	Method Blank	Duplicate		LCS/Spike Blank			Matrix Spike / Ref.		
					RPD	AC (%)	Spike Recovery (%)	Recovery Limits (%)		Spike Recovery (%)	Recovery Limits (%)	
								Low	High		Low	High
Chloride	DIO0376-APR24	µg/g	0.4	<0.4	7	35	100	80	120	86	75	125
Sulphate	DIO0376-APR24	µg/g	0.4	<0.4	4	35	101	80	120	109	75	125

Carbon/Sulphur

Method: ASTM E1915-07A | Internal ref.: ME-CA-[ENV]ARD-LAK-AN-020

Parameter	QC batch Reference	Units	RL	Method Blank	Duplicate		LCS/Spike Blank			Matrix Spike / Ref.		
					RPD	AC (%)	Spike Recovery (%)	Recovery Limits (%)		Spike Recovery (%)	Recovery Limits (%)	
								Low	High		Low	High
Sulphide (Na2CO3)	ECS0055-APR24	%	0.01	< 0.01								

Conductivity

Method: SM 2510 | Internal ref.: ME-CA-[ENV]EWL-LAK-AN-006

Parameter	QC batch Reference	Units	RL	Method Blank	Duplicate		LCS/Spike Blank			Matrix Spike / Ref.		
					RPD	AC (%)	Spike Recovery (%)	Recovery Limits (%)		Spike Recovery (%)	Recovery Limits (%)	
								Low	High		Low	High
Conductivity	EWL0383-APR24	uS/cm	2	< 2	0	20	100	90	110	NA		

QC SUMMARY

pH

Method: SM 4500 I Internal ref.: ME-CA-[ENV]EWL-LAK-AN-001

Parameter	QC batch Reference	Units	RL	Method Blank	Duplicate		LCS/Spike Blank			Matrix Spike / Ref.		
					RPD	AC (%)	Spike Recovery (%)	Recovery Limits (%)		Spike Recovery (%)	Recovery Limits (%)	
								Low	High		Low	High
pH	EWL0383-APR24	pH Units	0.05	NA	0		101			NA		

Method Blank: a blank matrix that is carried through the entire analytical procedure. Used to assess laboratory contamination.

Duplicate: Paired analysis of a separate portion of the same sample that is carried through the entire analytical procedure. Used to evaluate measurement precision.

LCS/Spike Blank: Laboratory control sample or spike blank refer to a blank matrix to which a known amount of analyte has been added. Used to evaluate analyte recovery and laboratory accuracy without sample matrix effects.

Matrix Spike: A sample to which a known amount of the analyte of interest has been added. Used to evaluate laboratory accuracy with sample matrix effects.

Reference Material: a material or substance matrix matched to the samples that contains a known amount of the analyte of interest. A reference material may be used in place of a matrix spike.

RL: Reporting limit

RPD: Relative percent difference

AC: Acceptance criteria

Multielement Scan Qualifier: as the number of analytes in a scan increases, so does the chance of a limit exceedance by random chance as opposed to a real method problem. Thus, in multielement scans, for the LCS and matrix spike, up to 10% of the analytes may exceed the quoted limits by up to 10% absolute and the spike is considered acceptable.

Duplicate Qualifier: for duplicates as the measured result approaches the RL, the uncertainty associated with the value increases dramatically, thus duplicate acceptance limits apply only where the average of the two duplicates is greater than five times the RL.

Matrix Spike Qualifier: for matrix spikes, as the concentration of the native analyte increases, the uncertainty of the matrix spike recovery increases. Thus, the matrix spike acceptance limits apply only when the concentration of the matrix spike is greater than or equal to the concentration of the native analyte.

LEGEND

FOOTNOTES

NSS Insufficient sample for analysis.
RL Reporting Limit.
↑ Reporting limit raised.
↓ Reporting limit lowered.
NA The sample was not analysed for this analyte
ND Non Detect

Results relate only to the sample tested.
Data reported represent the sample as submitted to SGS. Solid samples expressed on a dry weight basis.
"Temperature Upon Receipt" is representative of the whole shipment and may not reflect the temperature of individual samples.
Analysis conducted on samples submitted pursuant to or as part of Reg. 153/04, are in accordance to the "Protocol for Analytical Methods Used in the Assessment of Properties under Part XV.1 of the Environmental Protection Act and Excess Soil Quality" published by the Ministry and dated March 9, 2004 as amended.

SGS provides criteria information (such as regulatory or guideline limits and summary of limit exceedances) as a service. Every attempt is made to ensure the criteria information in this report is accurate and current, however, it is not guaranteed. Comparison to the most current criteria is the responsibility of the client and SGS assumes no responsibility for the accuracy of the criteria levels indicated.

This report supersedes all previous versions.

-- End of Analytical Report --

SGS

Request for Laboratory Services and CHAIN OF CUSTODY (Mining)

SGS Environmental Services - Lakefield: 185 Concession St., Lakefield, ON K0L 2H0 Phone: 705-652-2000 Toll Free: 877-747-7658 Fax: 705-652-6365 Web: www.ca.sgs.com {4}

SGS Environmental Services - London: 657 Consortium Court, London, ON, N6E 2S8 Phone: 519-672-4500 Toll Free: 877-848-8060 Fax: 519-672-0361 Web: www.ca.sgs.com {4}

Laboratory Information Section

Received Date (mm/dd/yyyy): APR 15 2024 LAB LIMS #: Apr-1568 NP

Skid # (if applicable): Temperature Upon Receipt (°C): 13, 12, 13

Billing & Reporting Information

Invoice/Receipt to {3}:		
Company:	Pinchin Ltd.	Quote #: 2024 660
Attention:	Megan Keon & Accounts Payable	Attached Parameter List: ☐ YES ☑ NO
Address:	1 Hines Rd, Suite 200 Kanata, ON.	**Turnaround Time**
		Is *Rush Turnaround Time Required? ☐ YES ☑ NO
Email:	mKeon@pinchin.com & ap@pinchin.com	Specify: Regular TAT is fine.
Project Name/Number:	319674.001 P.O. #:	* Rush TA Requests Require Lab Approval

Client Information/Report To: Client Lab #:

Company Name:	Pinchin Ltd.	**Phone Number:**	613-608-5350
Contact Name:	Megan Keon	**Fax Number:**	
Address:	1 Hines Rd, Suite 200, Kanata, ON	**E-mail:**	mKeon@pinchin.com.
Copy to:			

Samples will be sent to paid storage after 3 months unless otherwise requested. Additional storage and shipping costs will be charged to the client. If the samples should be returned, please provide courier account information.

After analysis samples are to be:	Disposed ☑	Stored (At client's cost) *Default ☐	Multi-phase project (Automatic storage) ☐
Returned to client ☐	Return courier:	Acct:	

IMPORTANT: If samples contain known Hazards, please label accordingly and identify below:

☐ **NORM (Normally Occuring Radioactive Material)** ☐ **Asbestos** ☐ **Other:**

Special Instructions : ☐ **See Attached or:**

Sample Information

Sample Identifier	Date Sampled (mm/dd/yy) HT	Time Sampled	# of Containers	pH	conductivity	resistivity	Redox Cemf Potential	sulfide	chloride	sulphate	
BH104 SS3 5-7 ft	04/09/24	PM	1	✓	✓	✓	✓	✓	✓	✓	

Analysis Requested (please enter the analysis required below and check off which analysis applies to each sample)

Sampled By {1}:	(Name) Megan Keon	(Signature) [signature]	**Date:**	04/09/24	**(mm/dd/yy)**
Relinquished by {2}:	(Name) Megan Keon	(Signature) [signature]	**Date:**	04/12/24	**(mm/dd/yy)**

Note: {1} Submission of samples to SGS is acknowledgement that you have been provided direction on sample collection/handling and transportation of samples. {2} Submission of samples to SGS is considered authorization for completion of work. Signatures may appear on this form or be retained on file in the contract, or in an alternative format (e.g. shipping documents). {3} Results may be sent by email to an unlimited number of addresses for no additional cost. Fax is available upon request. {4} Completion of work may require the subcontracting of samples between the London and Lakefield laboratories.

NP 10:00

APPENDIX IV

Bedrock Core Photographs

Photo 1 – Bedrock Core Borehole BH1, 2.3 to 8.1 mbgs, RC1, RC2, RC3 and RC4

Photo 2 – Bedrock Core Borehole BH3, 3.1 to 7.4 mbgs, RC1, RC2, RC3 and RC4

APPENDIX V

Report Limitations and Guidelines for Use

REPORT LIMITATIONS & GUIDELINES FOR USE

This information has been provided to help manage risks with respect to the use of this report.

GEOTECHNICAL SERVICES ARE PERFORMED FOR SPECIFIC PURPOSES, PERSONS AND PROJECTS

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

SUBSURFACE CONDITIONS CAN CHANGE

This geotechnical report is based on the existing conditions at the time the study was performed, and Pinchin's opinion of soil conditions are strictly based on soil samples collected at specific test hole locations. The findings and conclusions of Pinchin's reports may be affected by the passage of time, by manmade events such as construction on or adjacent to the Site, or by natural events such as floods, earthquakes, slope instability or groundwater fluctuations.

LIMITATIONS TO PROFESSIONAL OPINIONS

Interpretations of subsurface conditions are based on field observations from test holes that were spaced to capture a 'representative' snap shot of subsurface conditions. Site exploration identifies subsurface conditions only at points of sampling. Pinchin reviews field and laboratory data and then applies professional judgment to formulate an opinion of subsurface conditions throughout the Site. Actual subsurface conditions may differ, between sampling locations, from those indicated in this report.

LIMITATIONS OF RECOMMENDATIONS

Subsurface soil conditions should be verified by a qualified geotechnical engineer during construction. Pinchin should be notified if any discrepancies to this report or unusual conditions are found during construction.

Sufficient monitoring, testing and consultation should be provided by Pinchin during construction and/or excavation activities, to confirm that the conditions encountered are consistent with those indicated by the test hole investigation, and to provide recommendations for design changes should the conditions revealed during the work differ from those anticipated. In addition, monitoring, testing and consultation by Pinchin should be completed to evaluate whether or not earthwork activities are completed in accordance with our recommendations.   Retaining Pinchin for construction observation for this project is the most effective method of managing the risks associated with unanticipated conditions. However, please be advised that any construction/excavation observations by Pinchin is over and above the mandate of this geotechnical evaluation and therefore, additional fees would apply.

MISINTERPRETATION OF GEOTECHNICAL ENGINEERING REPORT

Misinterpretation of this report by other design team members can result in costly problems. You could lower that risk by having Pinchin confer with appropriate members of the design team after submitting the report. Also retain Pinchin to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering or geologic report. Reduce that risk by having Pinchin participate in pre-bid and preconstruction conferences, and by providing construction observation. Please be advised that retaining Pinchin to participation in any ‘other’ activities associated with this project is over and above the mandate of this geotechnical investigation and therefore, additional fees would apply.

CONTRACTORS RESPONSIBILITY FOR SITE SAFETY

This geotechnical report is not intended to direct the contractor's procedures, methods, schedule or management of the work Site. The contractor is solely responsible for job Site safety and for managing construction operations to minimize risks to on-Site personnel and to adjacent properties. It is ultimately the contractor’s responsibility that the Ontario Occupational Health and Safety Act is adhered to, and Site conditions satisfy all ‘other’ acts, regulations and/or legislation that may be mandated by federal, provincial and/or municipal authorities.

SUBSURFACE SOIL AND/OR GROUNDWATER CONTAMINATION

This report is geotechnical in nature and was not performed in accordance with any environmental guidelines. As such, any environmental comments are very preliminary in nature and based solely on field observations. Accordingly, the scope of services do not include any interpretations, recommendations, findings, or conclusions regarding the, assessment, prevention or abatement of contaminants, and no conclusions or inferences should be drawn regarding contamination, as they may relate to this project. The term "contamination" includes, but is not limited to, molds, fungi, spores, bacteria, viruses, PCBs, petroleum hydrocarbons, inorganics, pesticides/insecticides, volatile organic compounds, polycyclic aromatic hydrocarbons and/or any of their by-products.

Pinchin will not be responsible for any consequential or indirect damages. Pinchin will only be held liable for damages resulting from the negligence of Pinchin. Pinchin will not be liable for any losses or damage if the Client has failed, within a period of two years following the date upon which the claim is discovered within the meaning of the Limitations Act, 2002 (Ontario), to commence legal proceedings against Pinchin to recover such losses or damage.