FINAL

Preliminary Geotechnical Investigation – Proposed Commercial Development

3145 Conroy Road, Ottawa, Ontario

Prepared for:

WO MW Realty Limited.
180 Renfrew Drive
Markham, ON L3R 9Z2

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Megan Keon

Author: Megan Keon, EIT.
Project Manager, Geotechnical Services
mkeon@pinchin.com

Reviewer: Julia Brown, P.Eng., PMP.
Director, Geotechnical Services
jrbrown@pinchin.com

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1.0 INTRODUCTION AND SCOPE

Pinchin Ltd. (Pinchin) was retained by WO MW Realty Limited. (Client) to conduct a Preliminary Geotechnical Investigation and provide subsequent geotechnical design recommendations for the proposed commercial development to be located at 3145 Conroy 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 Preliminary Geotechnical Investigation would be part of the due diligence process for the potential acquisition and financing of the Site. At this time, the Client has two possible proposed development plans; however, the design is not finalized. Based on the two plans shared by the Client, both plans include one single-storey to two-storey, slab-on-grade (i.e., no basement level) warehouse/light industrial building suitable to service large equipment inside. The development would be complete with new Site services and asphalt and gravel surfaced access driveways and parking areas. Both plans show the building to be located centrally on the Site.

Pinchin's geotechnical comments and recommendations are based on the results of the Preliminary Geotechnical Investigation and our understanding of the project scope.

The purpose of the Preliminary Geotechnical Investigation was to delineate the subsurface conditions and soil engineering characteristics by advancing a total of five (5) sampled boreholes (Boreholes BH1 to BH5), at the Site. The information gathered from the Preliminary Geotechnical Investigation will allow Pinchin to provide geotechnical design recommendations for the proposed development.

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

- A detailed description of the soil and groundwater conditions;
- Site preparation recommendations;
- Open cut excavations;
- Anticipated groundwater management;
- Site service trench design;
- Foundation design recommendations including 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; and
- Potential construction concerns.

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

2.0 SITE DESCRIPTION AND GEOLOGICAL SETTING

The Site is located on the east side of Conroy Road, approximately 1.2 km northwest of the intersection of Hunt Club Road and Conroy Road in Ottawa, Ontario. The Site is currently lightly developed with an asphalt surfaced go-kart track that winds through mature trees and wild overgrowth. The access driveway is primarily gravel. The Site contains old debris in the northwest corner of the Site where the previous mini putt activity area was located. The remainder of the Site contains a few derelict concrete slabs, gravel areas and wild overgrowth. A railway track borders the northern side of the Site and the lands the immediate east, south and west are undeveloped while further away are residential subdivisions and commercial and light industrial 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 older alluvial deposits: clay, silt, sand, gravel and some organic remains (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 a preliminary field investigation at the Site on July 15 and 16, 2024 by advancing a total of five (5) sampled boreholes throughout the Site. The boreholes were advanced to depths of approximately 6.7 to 12.8 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 Geoprobe 7822 DT 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. Approximate shear strengths of the cohesive deposits were measured using shear vane testing and the results are presented on the appended borehole logs.

A monitoring well was installed in Borehole BH2 to allow measurement of groundwater levels. The monitoring well was constructed using flush-threaded 50 mm diameter Trilock pipe with 3.0 meter long 10-slot well screens, delivered to the Site in pre-cleaned individually sealed plastic bags. The screen and riser pipes were not allowed to come into contact with the ground or drilling equipment prior to installation.

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

Groundwater observations and measurements were obtained from the open boreholes during and upon completion of drilling. Groundwater levels were measured in the monitoring well on July 25 and on August 7, 2024. The groundwater observations and measurements recorded are included on the appended borehole logs.

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

- TBM: Mag and washer in sidewalk along Conroy Road, at the approximate location shown on the Topographic Survey in Appendix IV; and
- Elevation: 85.11 masl;
- "*Topographic Detail of 3145 Conroy Road",* prepared by J.AD. Barnes Limited, reference No. 24-10-029-00, dated May 1, 2024 (Site Survey).

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 the Atterberg Limits 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 organics or fill overlying fat clay, a sand deposit, till and probable bedrock to the maximum borehole termination depths of approximately 12.8 mbgs. The appended borehole logs provide detailed soil descriptions and stratigraphies, results of SPT and shear vane testing, details of monitoring well installations, and groundwater measurements.

4.1.1 Organics

Surficial organics was encountered within Boreholes BH1 to BH4 and ranged in thickness between 75 and 100 mm. The organics was damp at the time of sampling.

4.1.2 Fill

Fill was encountered at the surface in Borehole BH5 and it was approximately 0.6 m thick. The fill consisted of compact sand and gravel with trace silt.

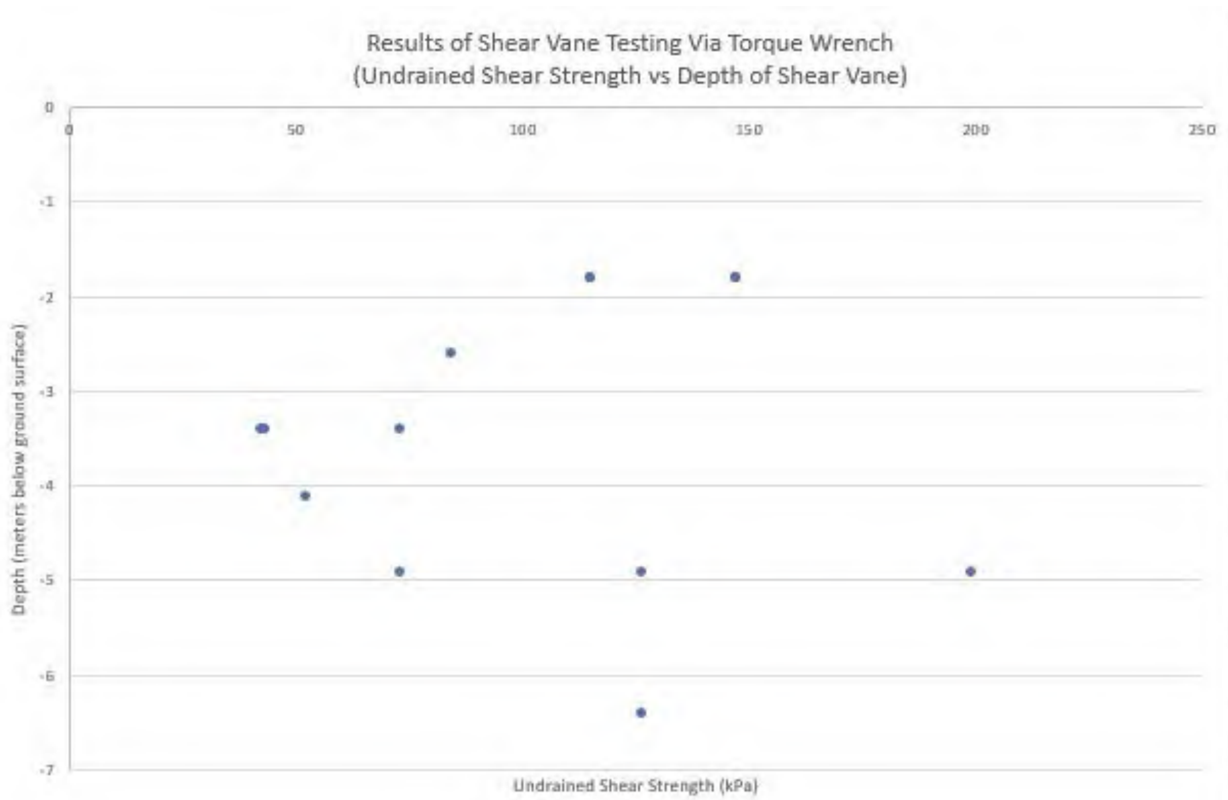
4.1.3 Fat Clay

Fat Clay was encountered underlying the surficial organics and fill within all the boreholes and extended to approximate sampled depths ranging between 6.1 and 7.6 mbgs. SPT 'N' values measured in the fat clay ranged from 0 to 8 blows per 300 mm penetration. The Fat clay had a Firm to Very stiff consistency based on shear strengths measured from in-situ shear vane readings measurements of 42 to 199 kPa. The remoulded shear strengths of the soil ranged from 12 to 105 kPa, resulting in a sensitivity of 2.0 to 7.0 indicating a low sensitivity. It should be noted that the fat clay generally had decreasing shear strength with depth.

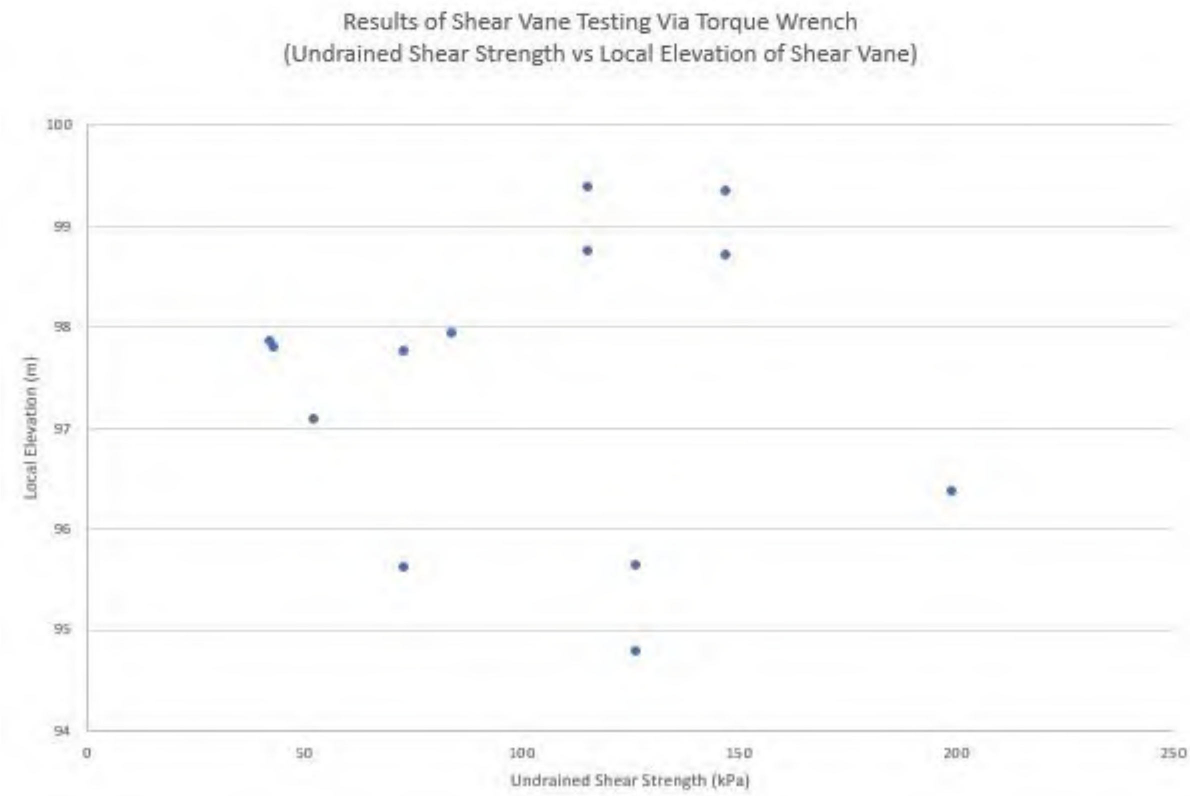
The results of four particle size distribution analyses completed on samples of the fat clay are provided in Appendix III and indicate that the samples contain 1 to 5% sand, 20 to 25% silt and 71 to 77% clay. Atterberg Limit testing indicates that the material has a liquid limit of between 75 and 80%, a plastic limit of between 33 and 37% and a plasticity index between 42 and 44%, indicating that the soil is sensitive but not very plastic. The moisture content of the samples tested ranged between 36 to 68%, indicating that the samples tested were at the plastic limit (APL) and wetter than plastic limit (WTPL) at the time of sampling.

The following two graphs were prepared to document the results of the shear vane testing completed within the fat clay stratigraphy in the field in terms of the associated Undrained Shear Strengths (kPa) plotted against both the local elevation (m) and the depth (meters below ground surface) of the shear vane taken.

Graph 1: Undrained Shear Strength (kPa) vs Depth of Shear Vane Taken (m)



Graph 2: Undrained Shear Strength (kPa) vs Local Elevation of Shear Vane (m)



4.1.4 Sand

A sand deposit was noted during the field investigation in Borehole BH1. In Borehole BH3, the sand was noted to be approximately 0.3 m thick and was encountered at a depth of approximately 1.2 mbgs within the clay material. In Borehole BH4, the sand was noted to be approximately 0.7 m thick and was encountered underlying the surficial organics. The sand material had a very loose to loose relative density based on SPT 'N' values of 1 to 6 per 300 mm penetration of a split spoon sampler. At the time of sampling the sand was noted to be brown and moist.

4.1.5 Silty Sand Till

A silty sand with gravel till material was encountered underlying the fat clay material in Borehole BH4 at a depth of 7.6 mbgs and extended to the underlying bedrock surface at 12.8 mbgs. Pinchin advanced two dynamic cone penetration tests at the bottoms of Boreholes BH3 and BH5 within the suspected fill layer. When the DCPT was pulled from the ground it contained an identical material to what was encountered in Borehole BH4. The till comprised silty sand with gravel. The non-cohesive glacial till had a loose to dense relative density based on SPT 'N' values from Borehole BH4 of 9 to 45 blows per 300 mm penetration of a split spoon sampler. The results of one particle size distribution analysis completed on a sample of the till is provided in Appendix III and indicates that the sample contains 20% gravel, 46% sand, 24% silt and 10% clay. The moisture content of the sample tested was 8.9%.

4.2 Bedrock

Auger and spoon refusal on inferred bedrock was encountered in Boreholes BH1, BH4 and BH5 at depths between approximately 11.1 to 12.8 mbgs. The inferred bedrock surface, as indicated by the recorded depths was consistent throughout the boreholes. The presence of bedrock can be confirmed by completing bedrock coring if its desirable.

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. At the completion of drilling, groundwater was observed in the open boreholes at depths between approximately 2.3 to 4.5 mbgs.

The groundwater level measured from the monitoring well installed at Borehole BH2 on July 25 and August 7, 2024, was 3.9 and 3.8 mbgs, respectively.

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 preliminary 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 development will consist of one single-storey to two-storey, slab-on-grade (i.e., no basement level) warehouse/light industrial building suitable to service large equipment inside located centrally on the Site. The development would be complete with new Site services and asphalt and gravel surfaced access driveways and parking areas. The final design is not complete as this investigation was requested as part of the due diligence and financing of the purchase of the Site and is subject to change.

5.2 Site Preparation

The existing surficial organics, concrete slabs, asphalt surfaced go-kart track, and debris and deleterious material is not considered suitable to remain below the proposed building, driveways and parking areas and will need to be removed. In calculating the approximate quantity of topsoil to be stripped, we recommend that the topsoil thicknesses provided on the individual borehole logs be increased by 50 mm to account for variations and some stripping of the mineral soil below.

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 addition 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

It is anticipated that the foundations will be constructed at conventional frost depths, approximately 1.8 metres below finished floor elevation.

Based on the subsurface information obtained from within the boreholes, it is anticipated that the excavated material will predominately consist of surficial organics, debris and deleterious material and fat clay material. Groundwater was measured in the monitoring well in Borehole BH2 at a depth of 3.9 and 3.8 mbgs on July 25 and August 7, 2024, respectively.

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 fat clay 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.

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

5.4 Anticipated Groundwater Management

Groundwater observations and measurements were obtained in the open boreholes at the completion of drilling and are summarized on the appended borehole logs. At the completion of drilling, groundwater was only observed at a depth between approximately 2.3 to 4.5 mbgs.

The groundwater level measured in the monitoring well installed at Borehole BH2 on July 25 and August 7, 2024, was 3.9 and 3.8 mbgs, respectively. Based on these groundwater levels, Pinchin does not anticipate that groundwater levels will be an issue during construction.

Moderate groundwater inflow through the fat clay material 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.

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

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

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

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

5.5 Foundation Design

Due to the presence of soft to very stiff fat clays at the Site, Pinchin has provided the following foundation recommendations:

- Shallow Foundations bearing on Fat Clay;
- Shallow Foundations bearing on Ground Improved Soil; and
- Deep Foundations.

5.5.1 Shallow Foundations Bearing on Fat Clay

The existing fat clay soil is considered suitable to support the proposed building, provided all of the pavement structure, surficial organics, concrete slabs and deleterious material are removed, and the subgrade prepared as above.

Conventional shallow strip footings established on the fat clay may be designed using a bearing resistance for 25 mm of settlement at Serviceability Limit States (SLS) of 100 kPa, and a factored geotechnical bearing resistance of 125 kPa at Ultimate Limit States (ULS). It is noted that the above SLS bearing resistance is limited to a maximum 1.5 m wide strip footing and 3.0 by 3.0 m spread footings and a minimum distance of 0.5 times the footing width between footings.

As the actual service loads were not known at the time of this report and the proposed development is not finalized, these should be reviewed by the project structural engineer to determine if SLS or ULS governs the footing design.

It is noted that there is a potential for weaker subgrade soil to be encountered between the investigation locations. Pinchin presumes that any areas of weaker subgrade soil will consist of small pockets of soft/loose natural soil which can be compacted to match the density of the remainder of the Site. As such, the material must be compacted to a minimum of 100% Standard Proctor Maximum Dry Density (SPMDD) prior to installing the concrete formwork. Any soft/loose areas which are not able to achieve the recommended 100% SPMDD are to be removed and replaced with a low strength concrete.

Pinchin notes that a qualified geotechnical engineering consultant should be on-Site during the proof roll and foundation preparation activities to verify the recommended level of compaction is achieved and to verify the design assumptions and recommendations. This is especially critical with respect to the recommended soil bearing pressures. If variations occur in the soil conditions between the borehole locations, site verification and site review by Pinchin is recommended to provide appropriate recommendations at that time.

The natural subgrade soil is sensitive to change in moisture content and can become loose/soft if subjected to additional water or precipitation. As well, it could be easily disturbed if travelled on during construction. Once it becomes disturbed it is no longer considered adequate to support the recommended design bearing pressures. It is recommended that a working slab of lean concrete (mud slab) be placed in the footing areas immediately after excavation and inspection to protect the founding soils during placement of formwork and reinforcing steel.

In addition, to ensure and protect the integrity of the subgrade soil during construction operations, the following is recommended:

- Prior to commencing excavations, it is critical that all existing surface water, potential surface water and perched groundwater are controlled and diverted away from the work Site to prevent infiltration and subgrade softening. At no time should excavations be left open for a period of time that will expose them to inclement weather conditions and cause subgrade softening;
- The subgrade should be sloped to a sump outside the excavation to promote surface drainage and the collected water pumped out of the excavation. Any potential precipitation or seepage entering the excavations should be pumped away immediately (not allowed to pond);

- The footing areas should be cleaned of all deleterious materials such as topsoil, organics, disturbed, caved materials or deleterious material; and
- If the excavated subgrade soil remains open to weather conditions and groundwater seepage, sidewall stability and suitability of the subgrade soil will need to be verified prior to construction.

If construction proceeds during freezing weather conditions, adequate temporary frost protection for the footing bases and concrete must be provided and maintained above freezing at all times.

5.5.2 Shallow Foundations on Ground Improvement

Ground improvement can be used to provide increased bearing resistance for shallow foundations. Ground improvement involves modifying the engineering properties of soils to increase bearing capacity and provide added stability. Possible ground improvement techniques such as rammed aggregate piers (RAP) or Controlled Modulus Columns (CMCs) may be suitable for the conditions present on the Site.

Ground improvement is generally designed and constructed by a specialty contractor. The following sections provide high level recommendations for consideration only. The design of any ground improvement system should be completed by the specialty contractor.

5.5.2.1 Rammed Aggregate Piers

RAP can be installed below the footings and/or floor slab to provide increased bearing capacity for this Site.

RAPs are installed by first driving a specially designed hollow mandrel into the soil using a large static force augmented by dynamic vertical impact energy. The RAP elements typically extend between 1.5 m to 15.5 m (5 to 50 feet) below grade but may extend deeper depending on project requirements.

After driving to the design depth, the hollow mandrel serves as a conduit for the placement of open-graded aggregate. The aggregate is placed inside the hopper and mandrel and the mandrel is raised, leaving a continuous lift of aggregate. The mandrel is the repeatedly raised and then driven back down forming compacted lifts. Compaction is achieved through the static crowd force and dynamic impact energy from the hammer.

RAP are designed and constructed by specialist contractors, and the bearing pressures available from this soil improvement are to be provided by that specialist contractor. It is noted that the specialist contractor may also have other soil improvement options available that could be suitable for this Site.

5.5.3 Cast-in-Place Concrete Caissons End Bearing on Bedrock

An alternative to shallow foundations on native fat clay or soil improvement would be deep foundations consisting of cast-in-place concrete caissons founded on the underlying bedrock surface located between approximately 11.1 and 12.8 mbgs.

For cast-in-place concrete caissons founded on the bedrock surface at the bedrock depths above, a factored geotechnical bearing resistance of 2,000 kPa at ULS may be used for foundation design purposes. The factored ULS pressure will govern design as the SLS pressure required for 25 mm of settlement will be greater than ULS. It is noted that in order to achieve the recommended bearing resistance, the cast-in-place concrete caissons must be socketed into the sound bedrock a minimum of 2 times the caisson diameter.

5.5.4 Liquefaction Potential

Pinchin carried out a preliminary assessment for seismic liquefaction of the cohesionless soil was carried out using the Idriss and Boulanger (2008) simplified procedure based on SPT N60 Values from the boreholes. The results of this analysis indicates that the cohesionless soils at the site are potentially liquifiable under an earthquake with a magnitude of 6.2 and a peak ground acceleration of 0.366 g. Ground surface settlements of up to approximately 65 mm could be generated should liquefaction occur.

It should be noted that the liquefaction analysis carried out for this study is considered preliminary and a detailed analysis, as well as additional geotechnical testing, will be required at the project design stage.

5.5.5 Grade Raise Potential

Pinchin notes that the underlying fat clay will be subjected to settlement as a result of any grade raises proposed by the Client. As the design stage progresses, further detailed analysis will be required.

5.5.6 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 30m.

Given the potential liquifiable soils at this site, the seismic site class is considered Site Class F; however, should a “non-liquifiable” site class be permitted, a Site Class E may be considered A Site Class E has an average shear wave velocity (Vs) of less than 180 m/s.

5.5.7 Foundation Transition Zones

Excessive differential settlements can occur where the subgrade support material types differ below the underside of continuous strip footings, (i.e., fat clay to till). 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.

5.5.8 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.

5.5.9 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.

5.5.10 Shallow Foundations Frost Protection & Foundation Backfill

In the Ottawa, Ontario area, exterior perimeter foundations for heated buildings require a minimum of 1.8m 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 silt material is too wet for reuse and not 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 Floor Slabs

Prior to the installation of the engineered fill material, all organics and deleterious materials should be removed to the underlying organic free in-situ soil. 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
Fat Clay	15,000
Silty Sand Till	20,000

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

5.7 Site Services

5.7.1 Pipe Bedding and Cover Materials for Flexible and Rigid Pipes

The subgrade soil conditions beneath the Site services will comprise primarily of fat clay materials. No support problems are anticipated for flexible or rigid pipes founded on the fat clay. 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.

5.7.2 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.

5.7.3 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.

5.8 Asphaltic Concrete Pavement Structure Design for Parking Lot and Driveways

5.8.1 Discussion

Parking areas and driveway access will be constructed around the proposed buildings. The in-situ fat clay is considered a sufficient bearing material for an asphaltic concrete pavement structure provided all organics 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 lot and driveways. As such, provided the pavement structure overlies the in-situ silt material, the following pavement structure is recommended.

5.8.2 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	50 mm	50 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)	300 mm	300 mm
Subbase Course: Granular "B" Type I (OPSS 1010)	100% Standard Proctor Maximum Dry Density (ASTM D698)	450 mm	600 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 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.

5.8.3 Pavement Structure Subgrade Preparation and Granular 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 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.

5.8.4 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 silt 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.

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.

5.9 Granular Surfaced Parking Area for Heavy Duty Vehicles Recommendations

Pinchin notes that the investigation was preliminary in nature and the proposed development plans show a gravel surfaced parking area at the rear of the building where heavy-duty vehicles and equipment would be parked when not in use. Pinchin has provided the following recommended gravel structure for the rear parking area.

Material Layer	Compaction Requirements	Parking Areas	Driveways
Base Course: Granular “A” (OPSS 1010)	100% Standard Proctor Maximum Dry Density (ASTM-D698)	300 mm	300 mm
Subbase Course: Granular “B” Type I (OPSS 1010)	100% Standard Proctor Maximum Dry Density (ASTM D698)	600 mm	600 mm

As mentioned with the asphalt surfaced areas, the fat clay is a suitable base for the proposed granular parking area provided the area is prepared as thoroughly detailed throughout the report and the material is compacted. Pinchin notes that long-term parking can result in point loads on the granular material and additional regrading may be necessary.

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 not required. The following table summarizes the 10-point soil evaluation for the tested samples:

Parameter	BH2, SS3 1.5 – 2.1 mbgs	
	Results	Points
Resistivity (ohm-cm)	3530	0
pH	8.21	0
Redox Potential (mV)	157	0
Sulfide	<0.01	0
Moisture	Poor drainage	2
Total Points		2

In summary, the tested sample does not indicate a potential for soil corrosivity, and additional protective measures are not required. The results should be reviewed by the structural engineer.

The results of the sulphate testing indicate that the Site possesses low to none 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 Preliminary Geotechnical Investigation was performed for the exclusive use of WO MW Realty Limited. (Client) in order to evaluate the subsurface conditions at 3145 Conroy 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 Preliminary 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 Preliminary 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 V, Report Limitations and Guidelines for Use, which pertains to this report.

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

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

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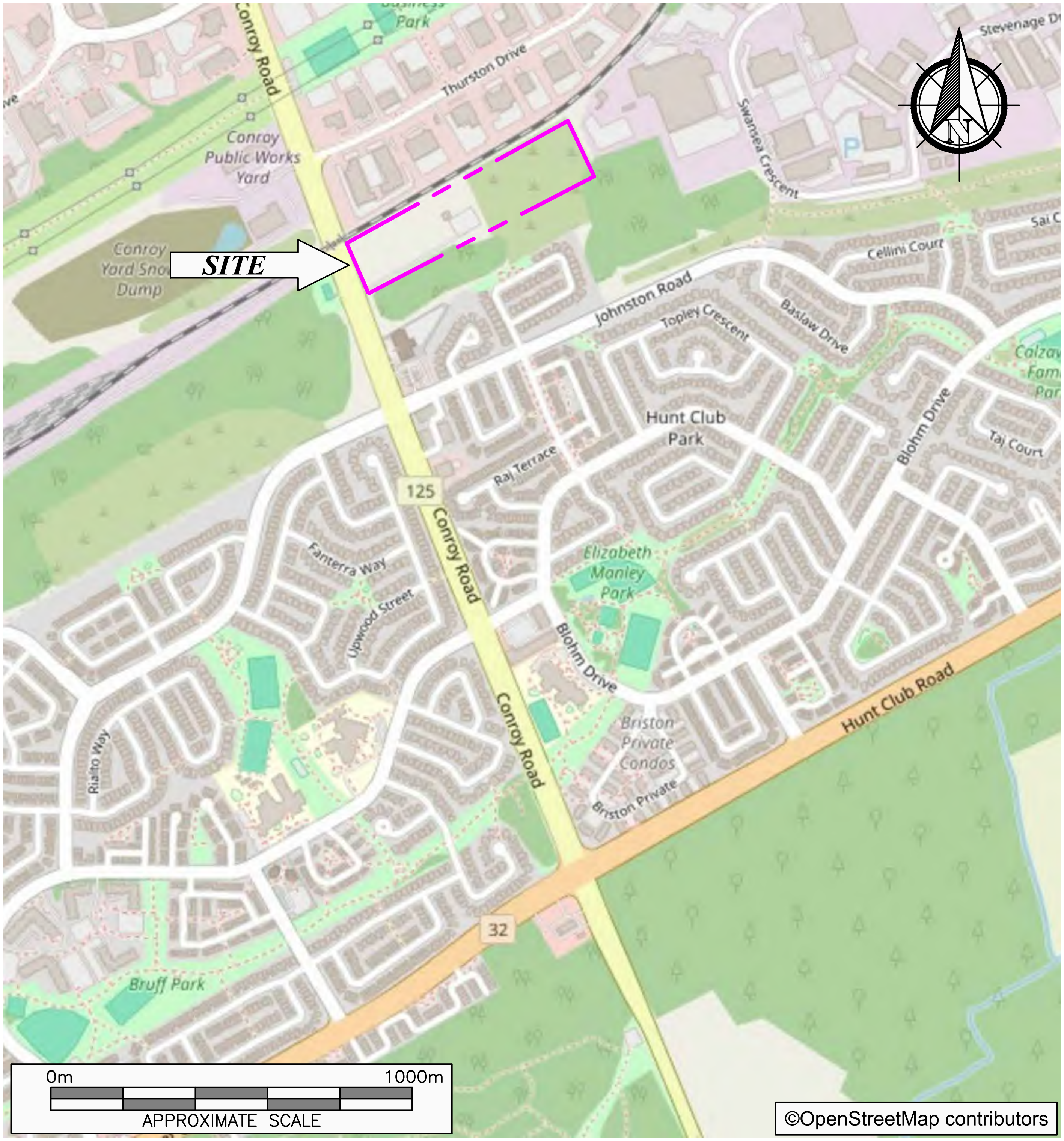
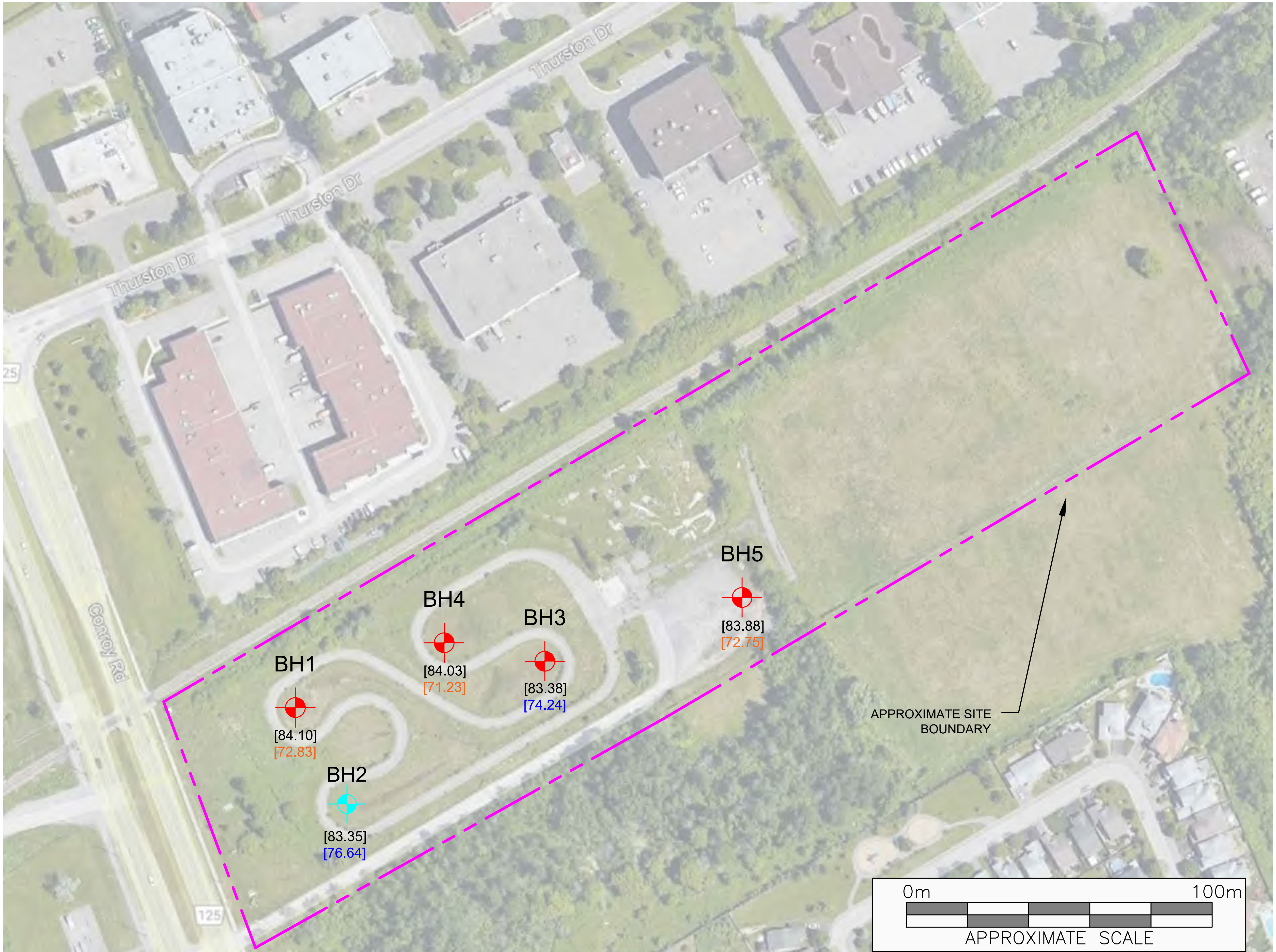
Template: Master Geotechnical Investigation Report – Ontario, GEO, September 2, 2021

FIGURES



PROJECT NAME	PRELIMINARY GEOTECHNICAL INVESTIGATION
CLIENT NAME	WO MW REALTY LIMITED
PROJECT LOCATION	3145 CONROY ROAD, OTTAWA, ONTARIO
FIGURE NAME	KEY MAP
FIGURE NO.	1
APPROXIMATE SCALE	AS SHOWN
PROJECT NO.	339662.003
DATE	SEPTEMBER 2024



LEGEND

BOREHOLE LOCATION

MONITORING WELL LOCATION

[xx.xx] GEODETIC GROUND SURFACE ELEVATION (m)

[xx.xx] GEODETIC APPROXIMATE GROUND TERMINATION ELEVATION (m)

[xx.xx] GEODETIC APPROXIMATE GROUND REFUSAL ELEVATION (m)

PINCHIN

PROJECT NAME
PRELIMINARY GEOTECHNICAL INVESTIGATION

CLIENT NAME
WO MW REALTY LIMITED

PROJECT LOCATION
3145 CONROY ROAD, OTTAWA, ONTARIO

FIGURE NAME
BOREHOLE/MONITORING WELL LOCATION PLAN

APPROXIMATE SCALE	PROJECT NO.
AS SHOWN	339662.003
DATE	FIGURE NO.
SEPTEMBER 2024	2

APPENDIX I

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

ABBREVIATIONS, TERMINOLOGY & PRINCIPAL SYMBOLS USED

Sampling Method

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

In-Situ Soil Testing

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

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

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

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

Soil Descriptions

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

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

Notes:

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

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

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

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

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

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

Soil & Rock Physical Properties

General

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

Consistency

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

Shear Strength

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

Consolidation (One Dimensional)

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

Permeability

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

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

Rock Coring

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

RQD is calculated as follows:

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

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

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

APPENDIX II
Pinchin's Borehole Logs

Log of Borehole: BH1

Project #: 339662.003 **Logged By:** MK

Project: Preliminary Geotechnical Investigation

Client: WO MW Reality Limited

Location: 3145 Conroy Road, Ottawa, Ontario

Drill Date: July 15, 2024 **Project Manager:** MK

SUBSURFACE PROFILE

Depth (m)	Description	Elevation (m)
	Ground Surface	84.10
0.00	**Organics** Organics - 100 mm	84.10
	Fat Clay Fat clay, grey, firm, APL to WTPL	
3.81	Very stiff	80.29
6.10	**Sand** Grey/black sand, some silt, compact, moist to wet	78.01
7.62	Trace gravel, wet	76.48
10.67	Bedrock fragments, very dense	73.44
11.28	End of Borehole	72.83

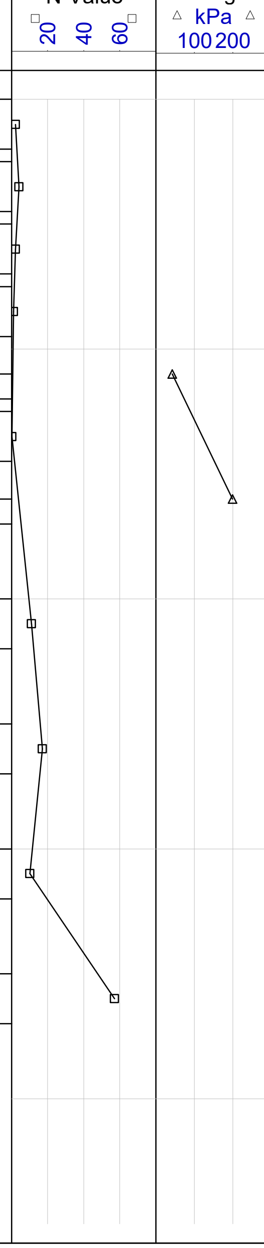
Monitoring Well Details: No Monitoring Well Installed

Borehole was terminated at 11.3 mbgs, upon sampler refusal at inferred bedrock. At drilling completion water was encountered at 2.3 mbgs in the open borehole.

SAMPLE

Sample Type	Sampler #	Recovery (%)	SPT N-Value	Water Content (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
SS	1	70	2				
SS	2	80	4				
SS	3	100	2				
SS	4	100	1	64.5			Hyd., MC. Att. Lim.
FVT		42	NA				
SS	5	100	0				
FVT		199	NA				
SS	6	100	11				
SS	7	100	17				
SS	8	100	10				
SS	9	100	57				

Standard Penetration N-Value / Shear Strength kPa



Contractor: Strata Drilling Group **Grade Elevation:** 84.10 m

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

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

Log of Borehole: BH2

Project #: 339662.003 **Logged By:** MK

Project: Preliminary Geotechnical Investigation

Client: WO MW Reality Limited

Location: 3145 Conroy Road, Ottawa, Ontario

Drill Date: July 15, 2024 **Project Manager:** MK

SUBSURFACE PROFILE

Depth (m)	Description	Elevation (m)
0	Ground Surface	83.35 / 0.00
0.00	**Organics** – Organics - 100 mm	
	Fat Clay – Fat clay, grey, very stiff, DTPL	
2.29	DTPL to APL	81.06 / 2.29
3.05	Stiff, WTPL	80.30 / 3.05
6.71	End of Borehole	76.64 / 6.71

Borehole was terminated at 6.7 mbgs. At drilling completion water was encountered at 3.0 mbgs in the open borehole.

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

Groundwater level = 3.82 mbgs, as measured on Aug. 7, 2024.

SAMPLE

Sample Type	Sampler #	Recovery (%)	SPT N-Value	Water Content (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
SS	1	70	2				
SS	2	80	5	36.0			Hyd., MC. Att. Lim.
FVT		147					
SS	3	100	2				
SS	4	100	0				
FVT		73					

Standard Penetration N-Value: 20 40 60

Shear Strength kPa: 100 200

Contractor: Strata Drilling Group **Grade Elevation:** 83.35 m

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

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

Log of Borehole: BH3

Project #: 339662.003 **Logged By:** MK

Project: Preliminary Geotechnical Investigation

Client: WO MW Reality Limited

Location: 3145 Conroy Road, Ottawa, Ontario

Drill Date: July 15, 2024 **Project Manager:** MK

SUBSURFACE PROFILE

Depth (m)	Description	Elevation (m)
	Ground Surface	83.38
0.00	**Organics** – Organics - 75 mm	0.00
	Fat Clay – Fat clay, grey, very stiff to stiff, DTPL	
3.05	Firm, APL	80.33 / 3.05
6.71	**Dynamic Cone Pentration Test (DCPT)** – Unsampled	76.67 / 6.71
9.14	End of Borehole	74.24 / 9.14
	Borehole was terminated at 9.1 mbgs.	

Monitoring Well Details: No Monitoring Well Installed

SAMPLE

Sample Type	Sampler #	Recovery (%)	SPT N-Value	Shear Strength (kPa)	Water Content (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
SS	1	20	8					
SS	2	100	4					
FVT		115						
FVT		84						
SS	3	100	0		68.0			Hyd., MC. Att. Lim.
FVT		42						
SS	4	100	0					
DCP		NA	4					
DCP		NA	4					
DCP		NA	10					
DCP		NA	7					
DCP		NA	9					
DCP		NA	15					
DCP		NA	17					
DCP		NA	20					

Standard Penetration N-Value: 20 40 60

Shear Strength kPa: 100 200

Contractor: Strata Drilling Group **Grade Elevation:** 83.38 m

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

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

Log of Borehole: BH4

Project #: 339662.003 **Logged By:** MK

Project: Preliminary Geotechnical Investigation

Client: WO MW Reality Limited

Location: 3145 Conroy Road, Ottawa, Ontario

Drill Date: July 16, 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 (20 40 60)	Shear Strength kPa (100 200)	Water Content (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
0	Ground Surface	84.03 / 0.00	No Monitoring Well Installed										
	Organics Organics - 75 mm			SS	1	30	5						
1	**Fat Clay** Fat clay, grey, very stiff, DTPL			SS	2	100	3						
2				FVT		115							
2.29	Firm to stiff, APL to WTPL	81.74 / 2.29		SS	3	100	1						
3–4				FVT		42							
				FVT		52							
4.57	WTPL	79.46 / 4.57		SS	4	100	0						
6.40	Very stiff	77.63 / 6.40		FVT		126							
7.62	**Till** Grey silty clayey sand with gravel, compact, wet	76.41 / 7.62		SS	5	30	11						
9.14	Loose	74.88 / 9.14		SS	6	20	9						
11				SS	7	60	44			8.9			Hyd., MC.
12				SS	8	30	45						
12.80	End of Borehole	71.23 / 12.80											

Borehole was terminated at 12.8 mbgs, upon sampler refusal on inferred bedrock.

Contractor: Strata Drilling Group **Grade Elevation:** 84.03 m

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

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

Log of Borehole: BH5

Project #: 339662.003 **Logged By:** MK

Project: Preliminary Geotechnical Investigation

Client: WO MW Reality Limited

Location: 3145 Conroy Road, Ottawa, Ontario

Drill Date: July 16, 2024 **Project Manager:** MK

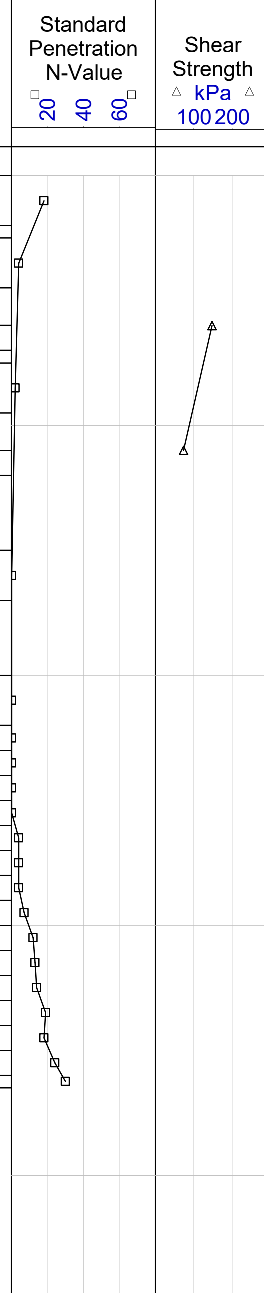
SUBSURFACE PROFILE

Depth (m)	Description	Elevation (m)	Monitoring Well Details
0	Ground Surface	83.88 / 0.00	No Monitoring Well Installed
0–0.61	**Fill** Grey sand and gravel, trace silt, compact, wet	83.27 / 0.61	
0.61–6.71	**Fat Clay** Fat clay, grey, very stiff to stiff, WTPL	77.17 / 6.71	
6.71–11.13	**Dyanmic Cone Penetration Test (DCPT)** Unsampled	72.75 / 11.13	
11.13	End of Borehole		

Borehole was terminated at 11.1 mbgs, upon sampler refusal on inferred bedrock.

SAMPLE

Sample Type	Sampler #	Recovery (%)	SPT N-Value	Water Content (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
SS	1	30	18				
SS	2	100	4				
FVT		147					
SS	3	100	2	56.4			Hyd., MC. Att. Lim.
FVT		73					
SS	4	100	0				
SS	5	30	0				
DCP		NA	0				
DCP		NA	0				
DCP		NA	0				
DCP		NA	0				
DCP		NA	4				
DCP		NA	4				
DCP		NA	4				
DCP		NA	7				
DCP		NA	12				
DCP		NA	13				
DCP		NA	14				
DCP		NA	19				
DCP		NA	18				
DCP		NA	24				
			30				



Contractor: Strata Drilling Group **Grade Elevation:** 83.88 m

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

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

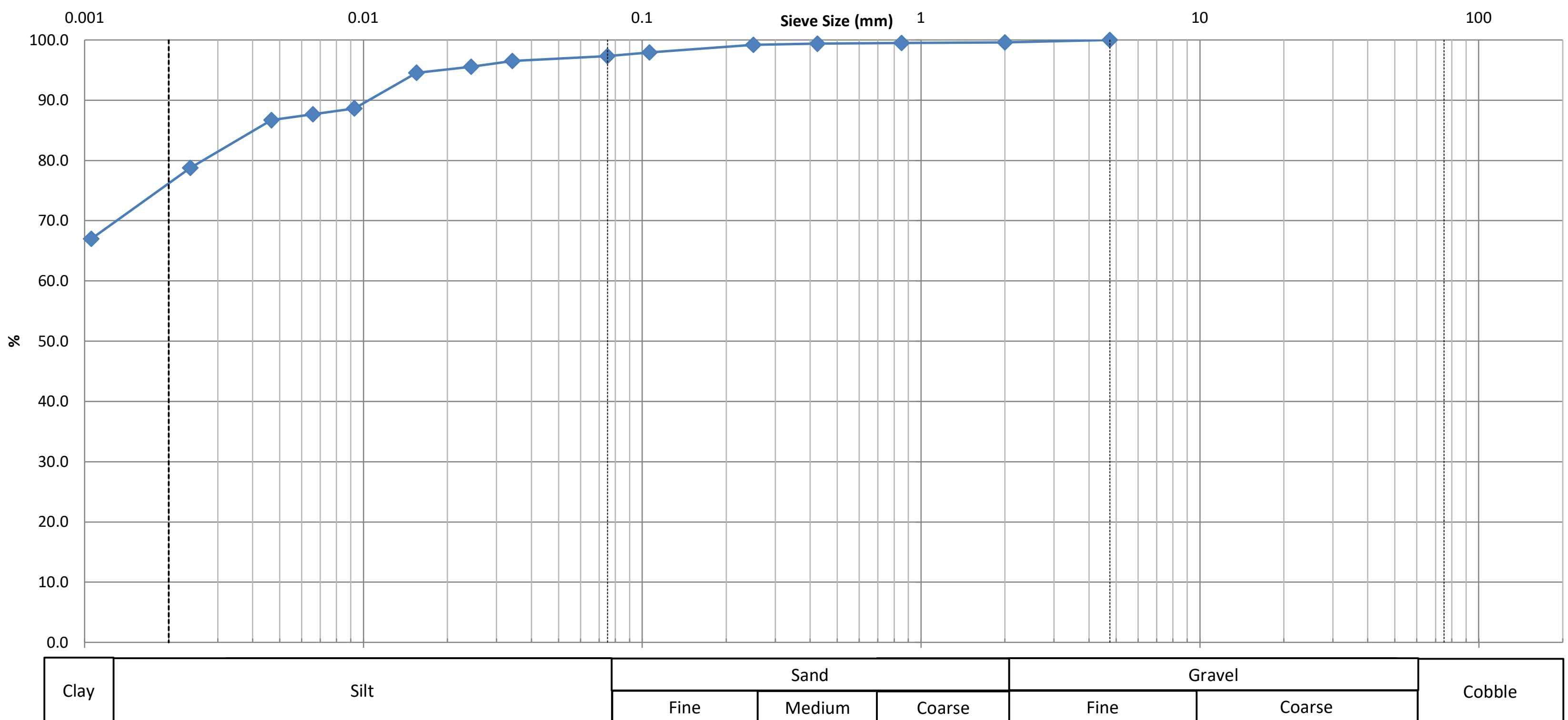
APPENDIX III

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.:	BH1 SS4	LAB NO:	54859
PROJECT:	339662.003			DATE RECEIVED:	8-Aug-24
				DATE TESTED:	12-Aug-24
DATE SAMPLED:	-			DATE REPORTED:	15-Aug-24
SAMPLED BY:	-			TESTED BY:	D.K



Identification	Soil Classification				MC(%)	LL	PL	PI	Cc	Cu
					64.5%					
	D100	D60	D30	D10	Gravel (%)	Sand (%)	Silt (%)	Clay (%)		
					0.0	2.7	20.3	77.0		

Comments:

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

PATERSON GROUP

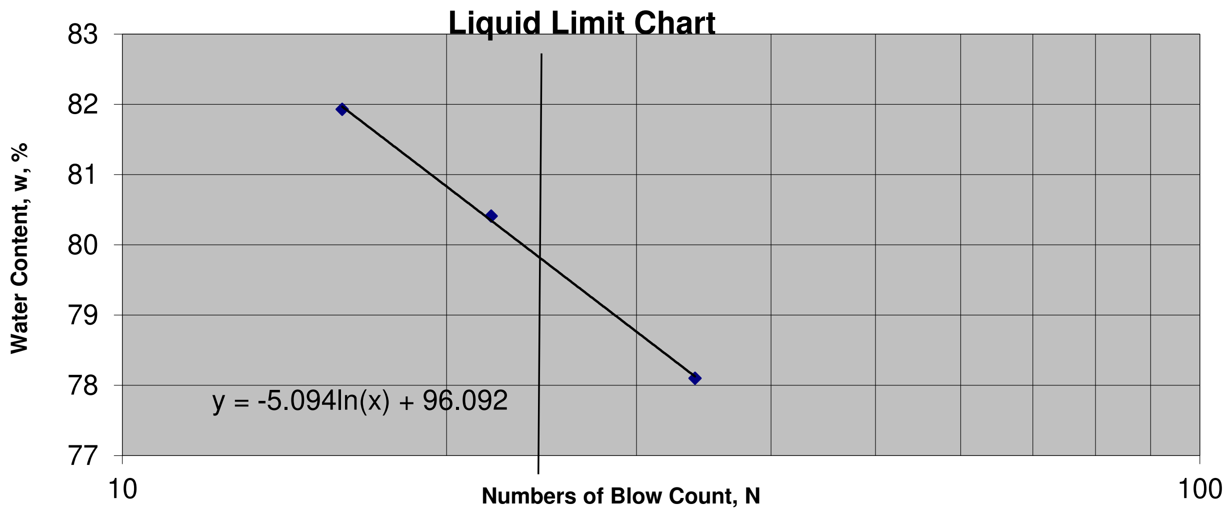
ATTERBERG LIMITS
LS-703/704

CLIENT:	Pinchin	FILE NO.:	PM4184
PROJECT:	339662.003	DATE SAMPLED:	15-Jul
LOCATION:	BH1 SS4 @ 7'6"-9'6"	DATE REPORTED:	14-Aug

CAN NO.	2	3	13				
WT. OF CAN	8.72	8.73	8.71				
WT. OF SOIL & CAN	18.69	20.06	16.76				
WT. OF DRY SOIL & CAN	14.20	15.01	13.23				
WT. OF MOISTURE	4.49	5.05	3.53				
WT. OF DRY SOIL & CAN	5.48	6.28	4.52				
WATER CONTENT, w, %	**81.93**	**80.41**	**78.1**				
NO. OF BLOWS, N	16	22	34				

CAN NO.	14	15
WT. OF CAN	19.95	19.91
WT. OF SOIL & CAN	26.98	26.71
WT. OF DRY SOIL & CAN	25.08	24.85
WT. OF MOISTURE	1.9	1.86
WT. OF DRY SOIL & CAN	5.13	4.94
WATER CONTENT, w, %	**37.04**	**37.65**

RESULTS	
LIQUID LIMIT	**80**
PLASTIC LIMIT	**37**
PLASTICITY INDEX	**43**

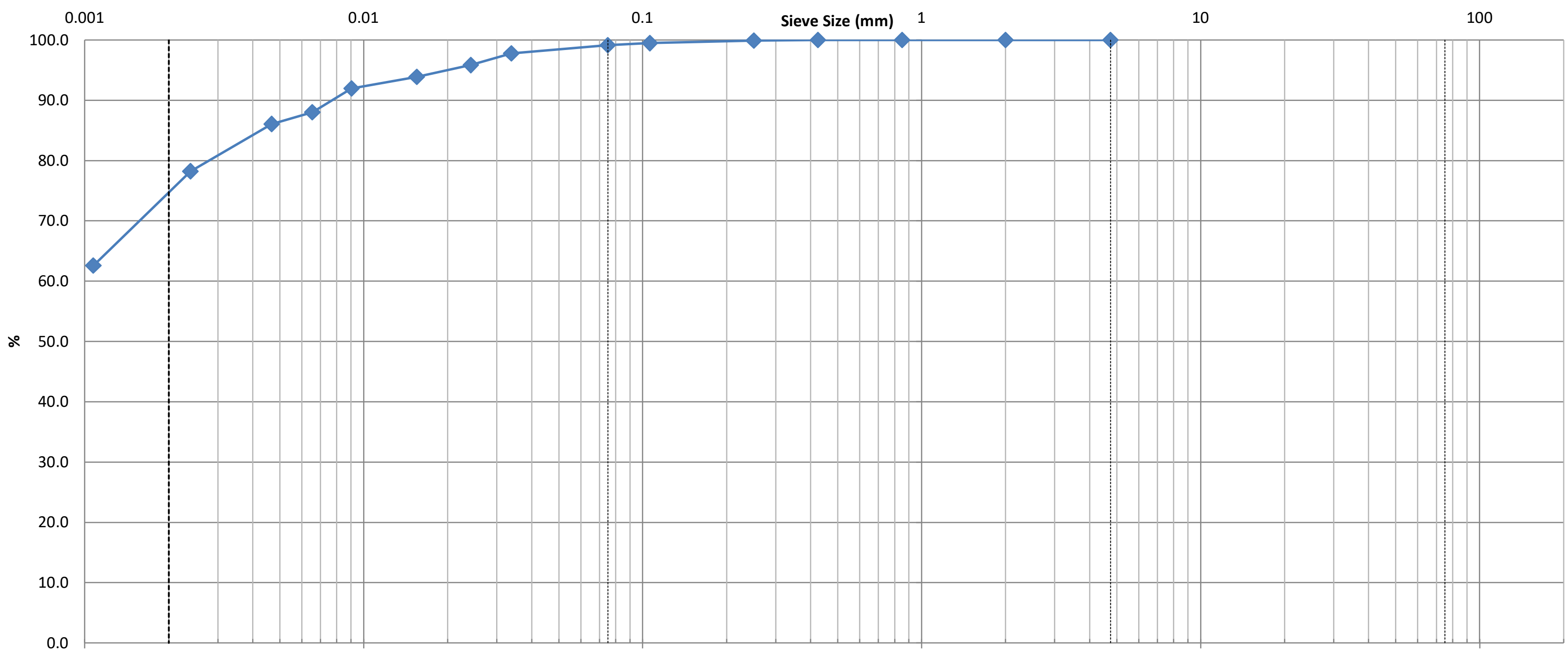


TECHNICIAN: CP	**REVIEWED BY:**	**C. Beadow**	**J. Forsyth, P. Eng.**

PATERSON GROUP

SIEVE ANALYSIS
ASTM C136

CLIENT:	Pinchin	DEPTH:	2'6" - 4'6"	FILE NO:	PM4184
CONTRACT NO.:		BH OR TP No.:	BH2 SS2	LAB NO:	54855
PROJECT:	339662.003			DATE RECEIVED:	8-Aug-24
				DATE TESTED:	12-Aug-24
DATE SAMPLED:	-			DATE REPORTED:	15-Aug-24
SAMPLED BY:	-			TESTED BY:	D.K



Clay	Silt	Sand			Gravel		Cobble
		Fine	Medium	Coarse	Fine	Coarse	

Identification	Soil Classification				MC(%)	LL	PL	PI	Cc	Cu
					36.0%					
	D100	D60	D30	D10	Gravel (%)	Sand (%)	Silt (%)	Clay (%)		
					0.0	0.9	24.6	74.5		

Comments:

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

PATERSON GROUP

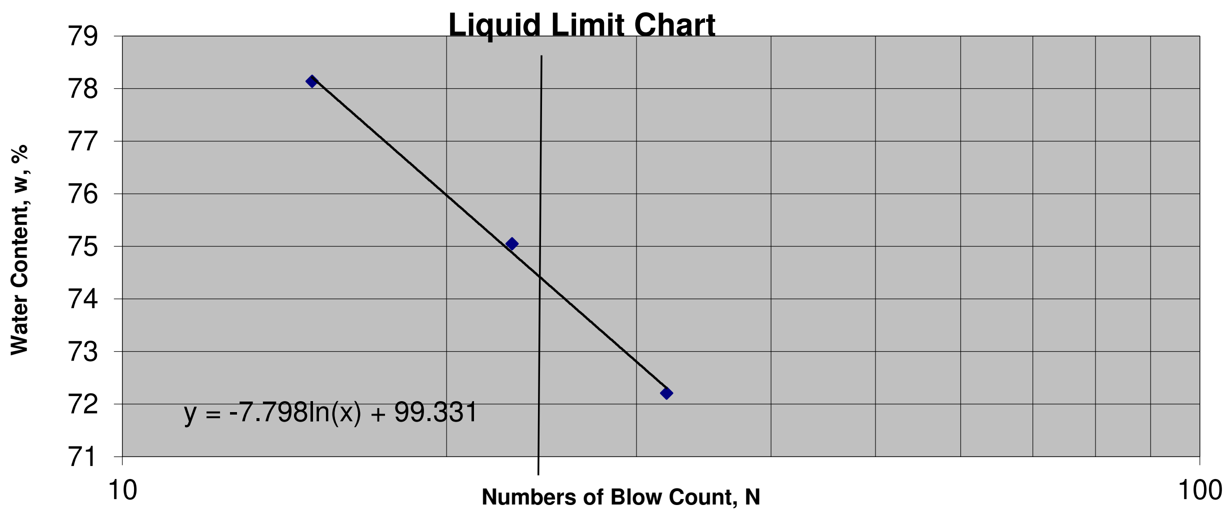
ATTERBERG LIMITS
LS-703/704

CLIENT:	Pinchin	FILE NO.:	PM4184
PROJECT:	339662.003	DATE SAMPLED:	15-Jul
LOCATION:	BH2 SS2 @ 5'6"-4'6"	DATE REPORTED:	14-Aug

CAN NO.	**30**	**31**	**32**				
WT. OF CAN	4.32	4.32	4.36				
WT. OF SOIL & CAN	15.4	12.46	12.54				
WT. OF DRY SOIL & CAN	10.54	8.97	9.11				
WT. OF MOISTURE	4.86	3.49	3.43				
WT. OF DRY SOIL & CAN	6.22	4.65	4.75				
WATER CONTENT, w, %	**78.14**	**75.05**	**72.21**				
NO. OF BLOWS, N	15	23	32				

CAN NO.	**9**	**10**
WT. OF CAN	19.37	19.79
WT. OF SOIL & CAN	27.33	27.94
WT. OF DRY SOIL & CAN	25.35	25.90
WT. OF MOISTURE	1.98	2.04
WT. OF DRY SOIL & CAN	5.98	6.11
WATER CONTENT, w, %	**33.11**	**33.39**

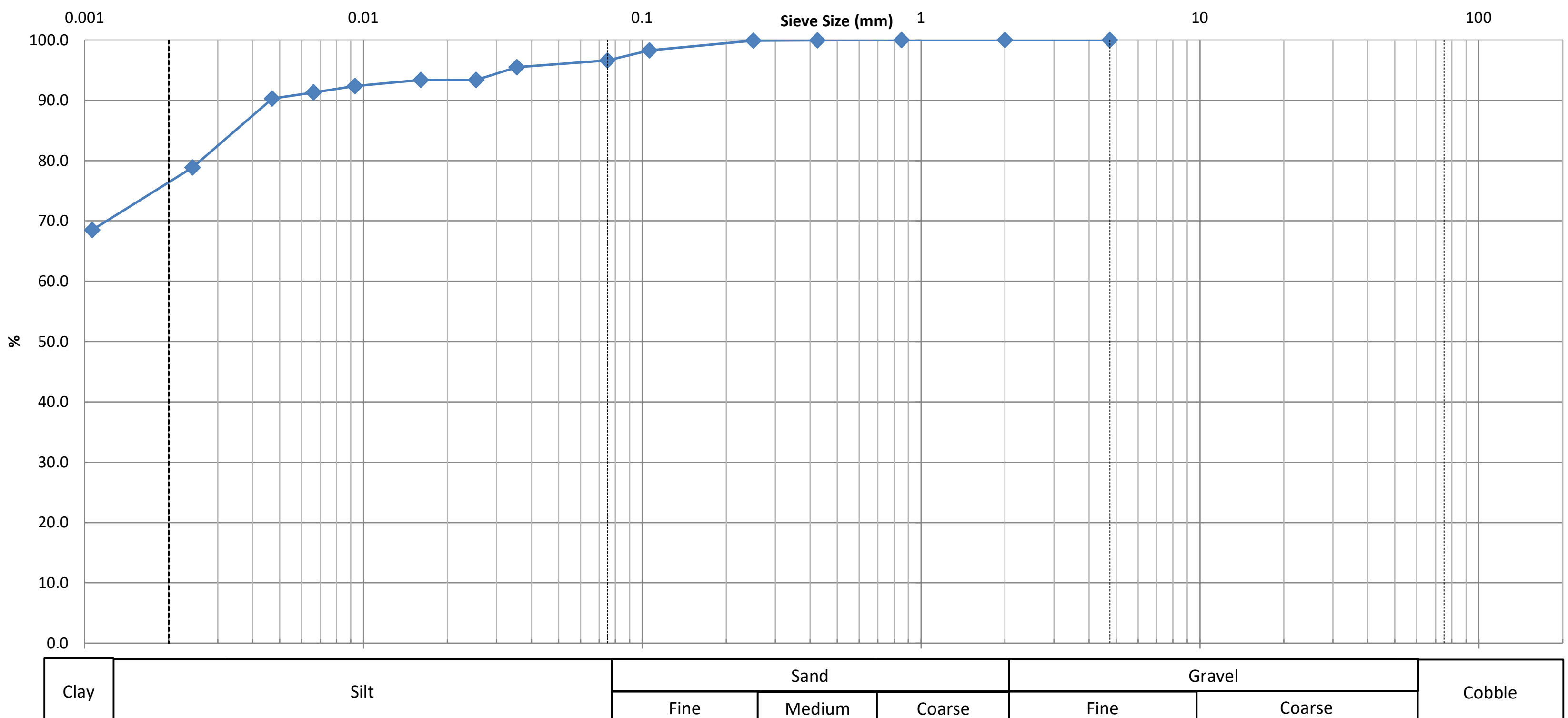
RESULTS	
LIQUID LIMIT	**75**
PLASTIC LIMIT	**33**
PLASTICITY INDEX	**42**



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

SIEVE ANALYSIS
ASTM C136

CLIENT:	Pinchin	DEPTH:	10' - 12'	FILE NO:	PM4184
CONTRACT NO.:		BH OR TP No.:	BH2 SS2	LAB NO:	54856
PROJECT:	339662.003			DATE RECEIVED:	8-Aug-24
				DATE TESTED:	12-Aug-24
DATE SAMPLED:	-			DATE REPORTED:	15-Aug-24
SAMPLED BY:	-			TESTED BY:	D.K



Identification	Soil Classification				MC(%)	LL	PL	PI	Cc	Cu
					68.0%					
	D100	D60	D30	D10	Gravel (%)	Sand (%)	Silt (%)	Clay (%)		
					0.0	3.4	19.6	77.0		

Comments:

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

PATERSON GROUP

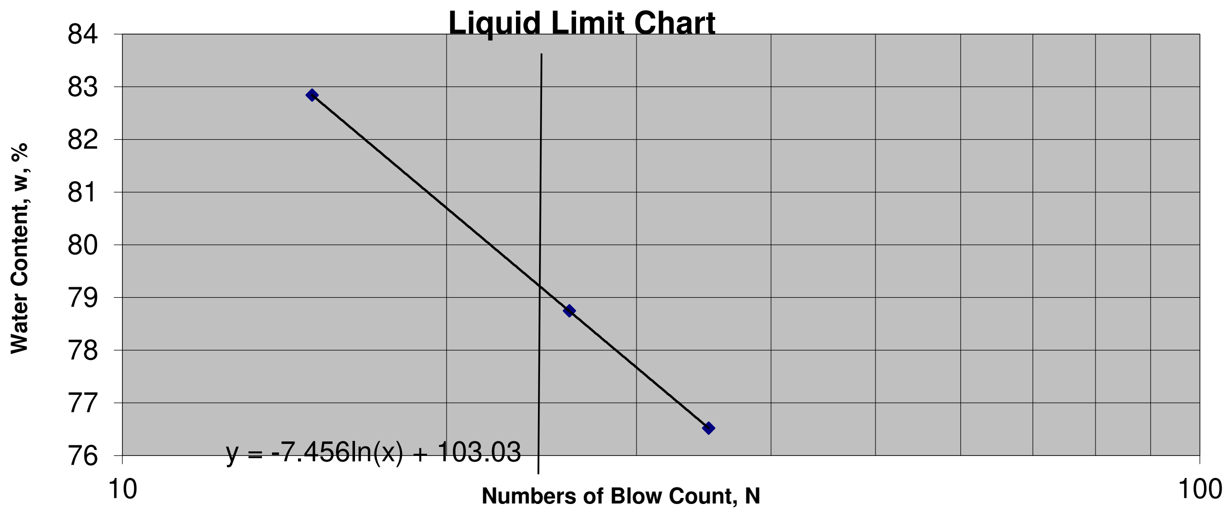
ATTERBERG LIMITS
LS-703/704

CLIENT:	Pinchin	FILE NO.:	PM4184
PROJECT:	339662.003	DATE SAMPLED:	15-Jul
LOCATION:	BH3 SS3 @ 10'-12'	DATE REPORTED:	14-Aug

CAN NO.	**33**	**34**	**35**				
WT. OF CAN	4.30	4.31	4.35				
WT. OF SOIL & CAN	13.57	12.05	12.17				
WT. OF DRY SOIL & CAN	9.37	8.64	8.78				
WT. OF MOISTURE	4.2	3.41	3.39				
WT. OF DRY SOIL & CAN	5.07	4.33	4.43				
WATER CONTENT, w, %	**82.84**	**78.75**	**76.52**				
NO. OF BLOWS, N	15	26	35				

CAN NO.	**11**	**12**
WT. OF CAN	19.98	16.74
WT. OF SOIL & CAN	26.02	23.26
WT. OF DRY SOIL & CAN	24.44	21.58
WT. OF MOISTURE	1.58	1.68
WT. OF DRY SOIL & CAN	4.46	4.84
WATER CONTENT, w, %	**35.43**	**34.71**

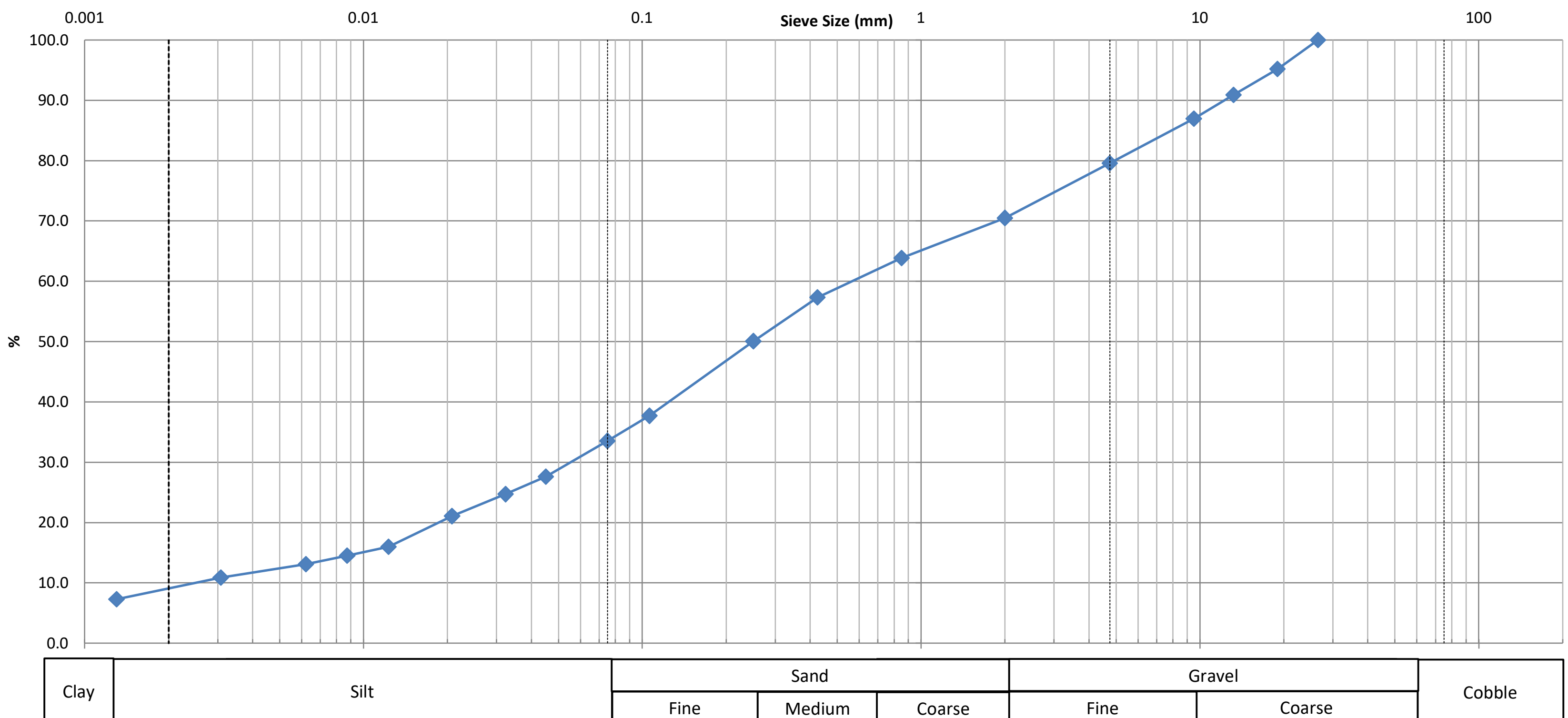
RESULTS	
LIQUID LIMIT	**79**
PLASTIC LIMIT	**35**
PLASTICITY INDEX	**44**



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

SIEVE ANALYSIS
ASTM C136

CLIENT:	Pinchin	DEPTH:	35' - 37'	FILE NO:	PM4184
CONTRACT NO.:		BH OR TP No.:	BH4 SS7	LAB NO:	54858
PROJECT:	339662.003			DATE RECEIVED:	8-Aug-24
				DATE TESTED:	12-Aug-24
DATE SAMPLED:	-			DATE REPORTED:	15-Aug-24
SAMPLED BY:	-			TESTED BY:	D.K



Identification	Soil Classification				MC(%)	LL	PL	PI	Cc	Cu
					8.9%					
	D100	D60	D30	D10	Gravel (%)	Sand (%)	Silt (%)	Clay (%)		
					20.4	46.0	24.1	9.5		

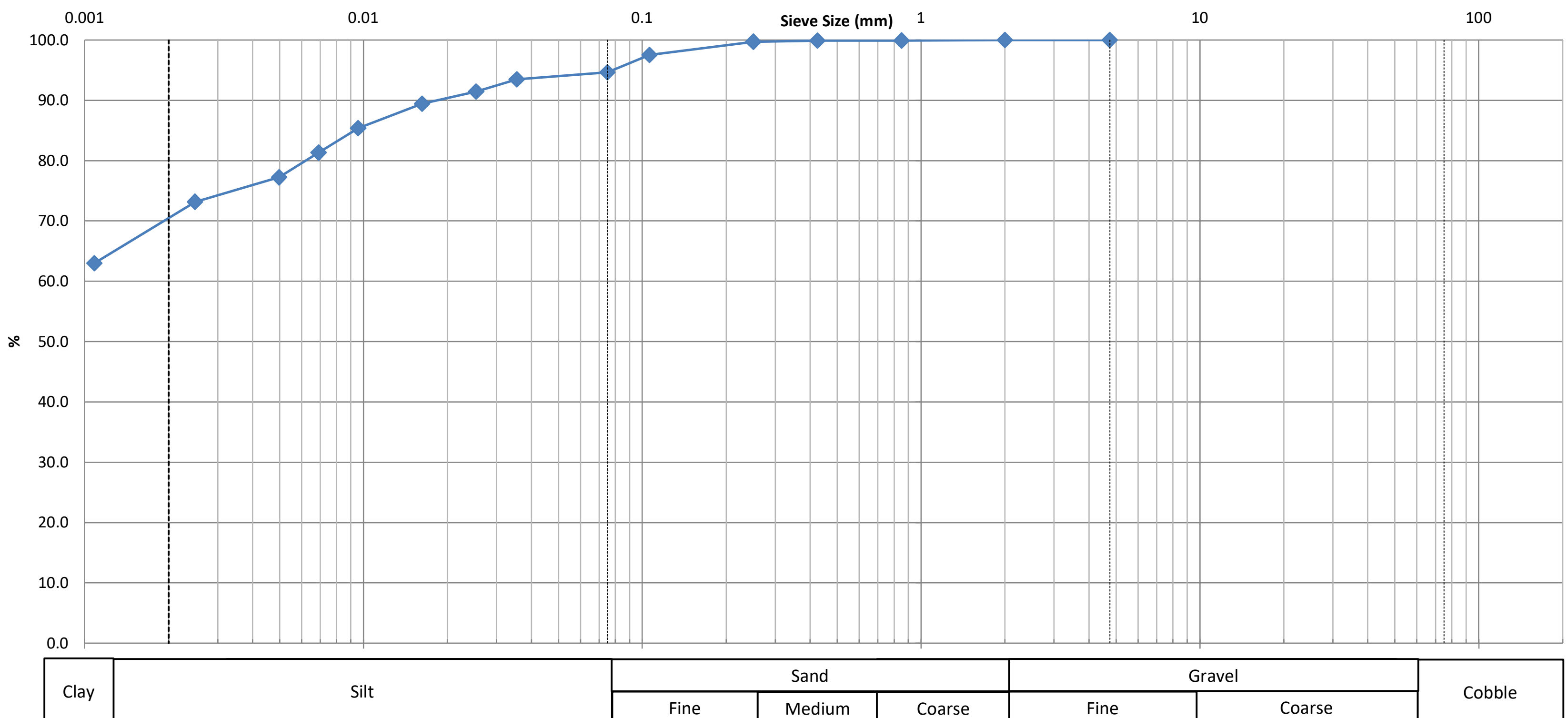
Comments:

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

PATERSON GROUP

SIEVE ANALYSIS
ASTM C136

CLIENT:	Pinchin	DEPTH:	7'6" - 9'6"	FILE NO:	PM4184
CONTRACT NO.:		BH OR TP No.:	BH5 SS3	LAB NO:	54860
PROJECT:	339662.003			DATE RECEIVED:	8-Aug-24
				DATE TESTED:	12-Aug-24
DATE SAMPLED:	-			DATE REPORTED:	15-Aug-24
SAMPLED BY:	-			TESTED BY:	D.K



Identification	Soil Classification				MC(%)	LL	PL	PI	Cc	Cu
					56.4%					
	D100	D60	D30	D10	Gravel (%)	Sand (%)		Silt (%)		Clay (%)
					0.0	5.4		24.1		70.5

Comments:

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

PATERSON GROUP

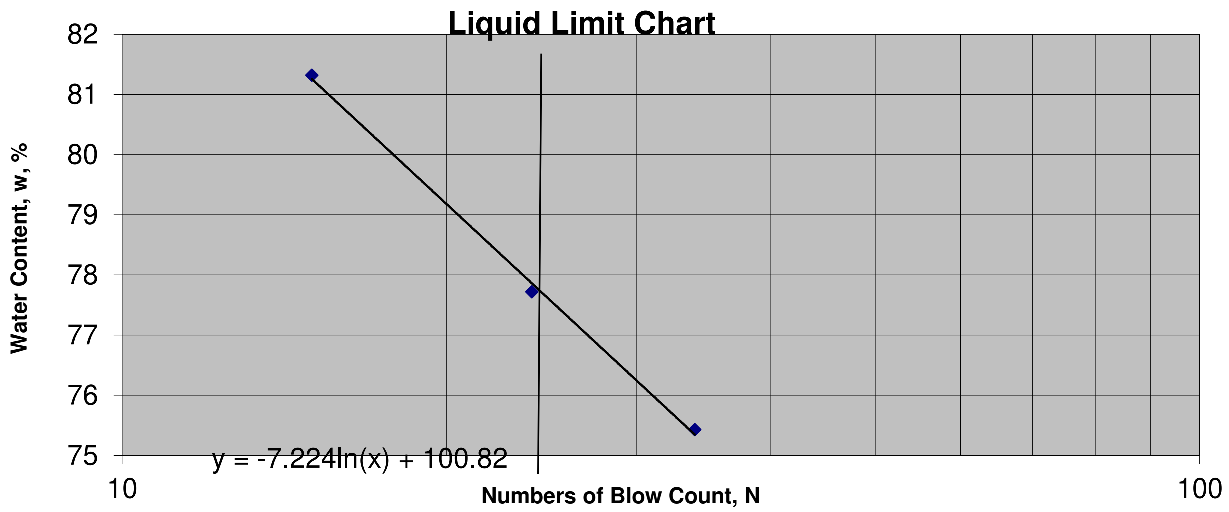
ATTERBERG LIMITS
LS-703/704

CLIENT:	Pinchin	FILE NO.:	PM4184
PROJECT:	339662.003	DATE SAMPLED:	15-Jul
LOCATION:	BH5 SS3 @ 7'6"-9'6"	DATE REPORTED:	14-Aug

CAN NO.	**2**	**3**	**13**				
WT. OF CAN	8.72	8.65	8.73				
WT. OF SOIL & CAN	20.27	18.30	16.87				
WT. OF DRY SOIL & CAN	15.09	14.08	13.37				
WT. OF MOISTURE	5.18	4.22	3.50				
WT. OF DRY SOIL & CAN	6.37	5.43	4.64				
WATER CONTENT, w, %	**81.32**	**77.72**	**75.43**				
NO. OF BLOWS, N	15	24	34				

CAN NO.	**1**	**2**
WT. OF CAN	19.88	19.94
WT. OF SOIL & CAN	26.93	27.36
WT. OF DRY SOIL & CAN	25.12	25.43
WT. OF MOISTURE	1.81	1.93
WT. OF DRY SOIL & CAN	5.24	5.49
WATER CONTENT, w, %	**34.54**	**35.15**

RESULTS	
LIQUID LIMIT	**78**
PLASTIC LIMIT	**35**
PLASTICITY INDEX	**43**



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

FINAL REPORT

CA15676-AUG24 R1

339662.003

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	CA15676-AUG24
Project	339662.003	Received	08/09/2024
Order Number		Approved	08/16/2024
Samples	Soil (1)	Report Number	CA15676-AUG24 R1
		Date Reported	08/16/2024

COMMENTS

Temperature of Sample upon Receipt: 20 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: 339662.003
Project Manager: Megan Keon
Samplers: Megan Keon

MATRIX: SOIL

Sample Number	5
Sample Name	BH2 SS3 7.5-9.5 Ft
Sample Matrix	Soil
Sample Date	01/08/2024

Parameter	Units	RL		Result
Corrosivity Index				
Corrosivity Index	none	1		2
Soil Redox Potential	mV	no		157
Sulphide (Na2CO3)	%	0.01		< 0.01
pH	pH Units	0.05		8.21
Resistivity (calculated)	ohms.cm	-9999		3530
General Chemistry				
Conductivity	uS/cm	2		283
Metals and Inorganics				
Moisture Content	%	0.1		38.4
Sulphate	µg/g	0.4		52
Other (ORP)				
Chloride	µg/g	0.4		3.0

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	DIO0252-AUG24	µg/g	0.4	<0.4	24	35	102	80	120	100	75	125
Sulphate	DIO0252-AUG24	µg/g	0.4	<0.4	2	35	92	80	120	92	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)	ECS0029-AUG24	%	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	EWL0261-AUG24	uS/cm	2	3	2	20	99	90	110	NA		

QC SUMMARY

pH

Method: SM 4500 l 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	EWL0261-AUG24	pH Units	0.05	NA	0		100			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): ~~AUG 0 9 2024~~ LAB LIMS #: CA-15676-Aug24 NP

Skid # (if applicable): Temperature Upon Receipt (°C): 2/0.2/1

Billing & Reporting Information

Invoice/Receipt to {3}:	
Company:	Pinchin Ltd.
Attention:	Megan Keon
Address:	1 Hines Rd, Suite 200 Kanata, ON
Email:	mKeon@pinchin.com / ap@pinchin.com
Project Name/Number:	339662.003
P.O. #:	

Quote #: 2024-066

Attached Parameter List: ☐ YES ☐ NO

Turnaround Time

Is *Rush Turnaround Time Required? ☐ YES ☑ NO

Specify:

* 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)**	**Time Sampled**	**# of Containers**	pH	conductivity	resistivity.	Redox potential	sulphide.	chloride.	sulphate	
20	BH2 SS3 7.5-9.5Ft	08/01/24	Am	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) Megan Keon	**Date:** 08/01/24	**(mm/dd/yy)**
Relinquished by {2}:	(Name) Megan Keon	(Signature) Megan Keon	**Date:** 08/08/24	**(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.

3348 5936 1522
PP NP 09:45

APPENDIX IV

J.D. Barnes Limited Topographic Site Survey

SKETCH SHOWING

TOPOGRAPHIC DETAIL OF 3145 CONROY ROAD CITY OF OTTAWA

J.D. BARNES LIMITED

SCALE 1 : 500

10 0 10 20 30 metres

METRIC DISTANCES AND/OR COORDINATES SHOWN ON THIS PLAN ARE IN METRES AND CAN BE CONVERTED TO FEET BY DIVIDING BY 0.3048.

CAUTION!

THIS IS NOT A PLAN OF SURVEY AND SHALL NOT BE USED EXCEPT FOR THE PURPOSE INDICATED IN THE TITLE BLOCK.

NOTES

THE TOPOGRAPHIC INFORMATION WAS COLLECTED APRIL 8, 2024 BY BRONE BASED LIDAR

CONTOUR INTERVALS ARE SHOWN AS 1.00m FOR MAJOR AND 0.25m FOR MINOR

CONTOUR SHOWN HERON ARE DERIEVED FROM MEASURED ELEVATIONS WHICH HAVE BEEN REMOVED FOR CLARITY

SKETCH AND CORRESPONDING CAD FILE ARE IN NAD 83 MTM 09 (CSRS)

BOUNDARY INFORMATION HAS BEEN COMPILED FROM AVAILABLE REGISTRY OFFICE PLANS.

TOPOGRAPHIC LEGEND

HP	DENOTES	HYDRO POLE
LP	DENOTES	LAMP POST
•P	DENOTES	POST
CLF	DENOTES	CHAIN LINK FENCE
PWF	DENOTES	POST AND WIRE FENCE
CB	DENOTES	CATCH BASIN
MH	DENOTES	MAN HOLE
STM	DENOTES	STORM
ANC	DENOTES	ANCHOR
— E —	DENOTES	HYDRO OVERHEAD CABLE
	DENOTES	TREE
SN	DENOTES	SIGN
	DENOTES	SAG POINT
	DENOTES	PROPERTY LINE
	DENOTES	MAJOR CONTOUR LINE
	DENOTES	MINOR CONTOUR LINE
	DENOTES	DECIDUOUS TREE
	DENOTES	CONIFEROUS TREE

N=NORTH / S=SOUTH / E=EAST / W=WEST

ELEVATION NOTE:

1. ELEVATIONS ARE GEODETIC AND ARE REFERRED TO CITY OF OTTAWA CONTROL POINT 2016-0300 HAVING A PUBLISHED ELEVATION OF 84.20 METRES (CGVD-1928 DATUM).

2. IT IS THE RESPONSIBILITY OF THE USER OF THIS INFORMATION TO VERIFY THAT THE SITE BENCHMARKS HAVE NOT BEEN ALTERED OR DISTURBED AND THAT ITS RELATIVE ELEVATION AND DESCRIPTION AGREES WITH THE INFORMATION SHOWN ON THIS DRAWING.

PREPARED FOR: WO MW REALTY LIMITED (WHITE OWL GROUP)

J.D.BARNES LIMITED
SURVEYING MAPPING GIS
LAND INFORMATION SPECIALISTS
62 STEACIE DRIVE, SUITE 103, KANATA, ON K2K 2A9
T: (613) 731-7244 F: (613) 254-8659 www.jdbarnes.com

DRAWN BY:	CHECKED BY:	REFERENCE NO.:
KZ/MC	GZ	24-10-029-00
PLOTTED: 5/1/2024		DATED: 05/01/24

FILE: G:\24-10-029\00\Drawing\Drone\3D\MTM\24-10-029-00_DR_TOPO_MTM.dgn

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.