FINAL

Geotechnical Investigation – Proposed Mixed Use Development

311 Somerset Street West, Ottawa, Ontario

Prepared for:

Somerset & O'Connor Limited Partnership
252 Argyle Avenue
Ottawa, ON K2P 1B9

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1.0 INTRODUCTION AND SCOPE

Pinchin Ltd. (Pinchin) was retained by Somerset & O'Connor Limited Partnership (Client) to conduct a Geotechnical Investigation and provide subsequent geotechnical design recommendations for the proposed mixed use development to be located at 311 Somerset Street West, 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 development will consist of a sixteen-storey mixed use building complete with three levels of underground parking which will occupy the majority of the Site footprint. At this time the proposed depth to the underside of the footing for the parking garage level is unknown; as such, for the purpose of this report, Pinchin has assumed a depth of 10.5 metres below existing ground surface (mbgs) to the underside of the footing for the parking garage level (i.e. ~ 3.5 mbgs per parking level). The proposed development will not include Site service trenches or asphalt surfaced parking areas.

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

The purpose of the Geotechnical Investigation was to delineate the subsurface conditions and soil engineering characteristics by advancing a total of four sampled boreholes (Boreholes BH101 to MW104), at the Site. The information gathered from the Geotechnical Investigation will allow Pinchin to provide geotechnical design recommendations for the proposed development.

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

- A detailed description of the soil and groundwater conditions;
- Site preparation recommendations;
- Open cut excavations;
- Anticipated groundwater management;
- Lateral earth pressure coefficients and unit densities;
- Foundation design recommendations including bedrock 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;
- Underground parking garage design; and

- Potential construction concerns.

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

2.0 SITE DESCRIPTION AND GEOLOGICAL SETTING

The Site is located on the northwest corner of the intersection of Somerset Street West and O'Connor Street, approximately 800 m north of Highway 417 in Ottawa, Ontario. The Site is currently undeveloped and consists of a gravel surfaced parking lot, with a small grassed area on the north portion of the Site. The lands adjacent to the Site are developed with a combination of multi-storey commercial and residential buildings.

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

3.0 GEOTECHNICAL FIELD INVESTIGATION AND METHODOLOGY

Pinchin completed a field investigation at the Site from February 10 to 12, 2021 by advancing a total of four sampled boreholes throughout the Site. The boreholes were advanced to depths ranging from approximately 11.6 to 14.8 mbgs, where refusal was encountered on weathered shale bedrock. The approximate spatial locations of the boreholes advanced at the Site are shown on Figure 2.

The boreholes were advanced with the use of a Geoprobe 7822 DT direct push drill rig which was equipped with standard soil sampling equipment. Soil samples were collected at 0.76 and 1.52 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 and to estimate the consistency of the cohesive soil.

Shear strengths of the cohesive soil were measured using the field vane shear test, as per ASTM D2573. The shear strengths measured are plotted on the appended borehole logs.

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

Monitoring wells were installed in Boreholes MW102 and MW104 to allow measurement of groundwater levels. The monitoring wells were constructed using flush-threaded 50 mm diameter Trilock pipe with 3.0-meter-long, 10-slot well screens, delivered to the Site in pre-cleaned individually sealed plastic bags. The screen and riser pipes were not allowed to come into contact with the ground or drilling equipment prior to installation.

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

Groundwater observations and measurements were obtained from the open boreholes during and upon completion of drilling. Groundwater levels were measured in the monitoring wells on March 4, 2021. The groundwater observations and measurements recorded are included on the appended borehole logs.

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

- TBM: Top nut of fire hydrant at the approximate location shown on Figure 2; and
- Elevation: 100.00 metres (local datum).

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

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

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

4.0 SUBSURFACE CONDITIONS

4.1 Borehole Soil Stratigraphy

In general, the soil stratigraphy at the Site comprises either surficial granular fill or surficial organics overlying natural silty clay, glacial till, and shale bedrock to the maximum borehole termination depth of approximately 18.6 mbgs. The appended borehole logs provide detailed soil descriptions and stratigraphies, results of SPT and shear vane testing, moisture contents, details of monitoring well installations, and groundwater measurements.

The surficial organics were encountered within Boreholes BH101 and MW102 and consisted of a brown clayey silt containing trace rootlets. The organics were measured to be approximately 100 millimetres (mm) thick.

The surficial granular fill was encountered within Boreholes BH103 and MW104 as well as underlying the surficial organics within Borehole BH101 and was noted to extend to between approximately 1.5 and 2.3 mbgs. The granular fill material ranged in soil matrix from sand and gravel containing trace to some silt, trace brick, and trace wood pieces, to sand containing trace gravel and trace silt. The fill material was frozen to between 0.8 and 1.1 mbgs; however, the fill had a loose to compact relative density based on SPT 'N' values of 4 to 10 blows per 300 mm penetration of a split spoon sampler.

The natural silty clay material was encountered underlying the granular fill in all boreholes with the exception of Borehole MW102, where is was encountered underlying the surficial organics. The silty clay was observed to extend to approximately 10.7 mbgs and has a variable soft to hard consistency based on shear strengths measured with a field vane of 21 to 303 kPa. The remolded shear strength of the silty clay ranged from 8 to 55 kPa, resulting in a sensitivity of 1.3 to 6.6. The results of two particle size distribution analyses completed on samples of the silty clay indicate that the samples contain 0 to 1% sand, 29 to 32% silt, and 68 to 71% clay sized particles.

The silty clay plots above the "A" line on the plasticity chart which indicates the material has a high plasticity. The results of Atterberg Limit testing are summarized in the following table.

Borehole No.	Test Depth	Liquid Limit	Plastic Limit	Plasticity Index
BH102	3.1 to 3.7 m	77	30	47
BH103	2.3 to 2.9 m	80	33	47

One-dimensional consolidation testing was not completed on the silty clay material encountered in the boreholes; however, the preconsolidation pressure was estimated using the results of the Atterberg Limit testing. The moisture content of the material tested ranged from 49.6 to 54.1%, indicating the material is wetter than the plastic limit (WTPL).

Glacial till was encountered underlying the silty clay within all boreholes and extended to depths ranging between 11.5 and 14.0 mbgs. The glacial till generally comprised sand and gravel containing some silt and trace clay, which was wet and grey. The glacial till had a very loose to compact relative density based on SPT 'N' values of between 1 and 23 blows per 300 mm penetration of a split spoon sampler. The results of one particle size distribution analysis completed on a sample of the glacial till indicates that the sample contains approximately 38% gravel, 37% sand, 19% silt, and 7% clay.

4.2 Bedrock

Bedrock was encountered between approximately 11.5 and 14.0 mbgs within all boreholes. Bedrock was proved within Borehole BH101 by core drilling with an NQ-size double tube diamond bit core barrel. It is noted that the upper approximate 3.4 m of the bedrock encountered was highly weathered and rock cores could not be retrieved. The bedrock cores recovered consisted of shale rock which was moderately weathered. The bedrock was black with grey and white banding, fine to medium grained, and contained some natural fractures with little to no oxidation. The bedrock at the fracture locations was mostly sharp and angular, which indicates minor water migration. Natural fractures were closely to moderately spaced and were generally found to occur in sets oriented at approximately 45 to 90º to the core axis. The rock core recovery was 100%, with an average RQD of 34%. Based on the RQDs obtained, the bedrock is considered to be weathered and very poor to poor quality. Photographs of the rock cores are provided in Appendix IV.

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. The groundwater was measured within the monitoring wells installed in Boreholes MW102 and MW104 at approximately 4.6 mbgs on March 4, 2021. Seasonal variations in the water table should be expected, with higher levels occurring during wet weather conditions in the spring and fall and lower levels occurring during dry weather conditions.

5.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

5.1 General Information

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

Based on information provided by the Client, it is Pinchin's understanding that the development will consist of a sixteen-storey mixed use building complete with three levels of underground parking which will occupy the majority of the Site footprint. At this time the proposed depth to the underside of the footing for the parking garage level is unknown; as such, for the purpose of this report, Pinchin has assumed a depth of 10.5 metres below existing ground surface (mbgs) to the underside of the footing for the parking garage level (i.e. ~ 3.5 mbgs per parking level).

Based on the proposed development consisting of a sixteen-storey building, the glacial till encountered in the boreholes is not considered capable of supporting the proposed building foundation systems. As such, Pinchin recommends that the foundations be extended down to the underlying weathered bedrock surface at the Site.

5.2 Open Cut Excavations and Groundwater Management

It is anticipated that the excavations for the building foundations will extend to a minimum depth of approximately 10.5 mbgs in order to accommodate the proposed levels of underground parking.

Based on the subsurface information obtained from within the boreholes it is anticipated that the excavated material will consist of a combination of organics, granular fill, silty clay, and glacial till. Groundwater was measured at approximately 4.6 mbgs and is expected to be encountered during excavations for the building foundations.

Where workers must enter trench excavations deeper than 1.2 m, the trench excavations should be suitably sloped and/or braced in accordance with the Occupational Health and Safety Act (OHSA), Ontario Regulation 213/91, Construction Projects, July 1, 2011, Part III - Excavations, Section 226. Alternatively, the excavation walls may be supported by either closed shoring, bracing, or trench boxes complying with sections 235 to 239 and 241 under O. Reg. 231/91, s. 234(1). The shoring system may be designed as full cantilevers, or the lateral loads can be taken up to the installation of internal bracing of rakers or tie back soil anchors. The temporary shoring design must include appropriate factors of safety, and any possible surcharge loading must be considered.

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

Soil Layer	Unit Weight (kN/m^3)	Angle of Internal Friction (°)	Active Earth Pressure Coefficient - K_a	Passive Earth Pressure Coefficient - K_p	At Rest Earth Pressure Coefficient - K_o
Fill Material	20	30	0.33	3.0	0.5
Silty Clay	17	28	0.36	2.76	0.53
Glacial Till	21	32	0.31	3.25	0.47

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

Moderate groundwater inflow through the natural subgrade soil is expected where the excavations extend less than 0.50 m below the groundwater table. It is believed that this groundwater inflow can be controlled using a gravity dewatering system with perimeter interceptor ditches and high capacity pumps. For excavations extending more than 0.5 m below the stabilized groundwater table, a dewatering system installed by a specialist dewatering contractor may be required to either lower the groundwater level prior to excavation, or to maintain the groundwater level during construction. The design of the dewatering system should be left to the contractor's discretion, and the system should meet a performance specification to maintain and control the groundwater at least 0.50 m below the excavation base. A hydrogeological investigation will be required once the proposed development has been finalized in order to determine the quantity of water which will be removed from the Site.

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

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

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

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

5.3 Foundation Design

5.3.1 Shallow Foundations Bearing on Weathered Shale Bedrock

For conventional shallow strip and spread footings established directly on the weathered bedrock surface, a bearing resistance for 25 mm of settlement of 400 kPa at Serviceability Limit States (SLS) and a factored geotechnical bearing resistance of 600 kPa at Ultimate Limit States (ULS) may be used for design purposes. Prior to installing the foundation formwork, the bedrock is to be reviewed by a geotechnical engineer.

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

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

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

5.3.2 Raft Slab Foundation Bearing on Weathered Bedrock

Due to the relatively low bearing capacity, there is a potential for very large strip and spread footings to be required to support the proposed building. Installing large footings at the Site will cause stress bulb interaction within the subgrade soils due to the size and proximity of the footings to one another. As such, the foundation system may have to be designed as a raft. The design of a raft slab is an iterative process and a modulus of subgrade reaction value of $k = 7.5$ MN/m^3 may be used for the initial design of the raft slab foundation on the weathered bedrock. Once the preliminary loading designs are complete, Pinchin should be allowed to review the loading to determine if the values provided need to be adjusted to ensure excessive settlement does not occur. Pinchin should review the above recommendations once final grades have been set.

5.3.3 Estimated Settlement

The foundations 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.

5.3.4 Building Drainage

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

5.3.5 Shallow Foundations Frost Protection & Foundation Backfill

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

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

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

5.4 Site Classification for Seismic Site Response & Soil Behaviour

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

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

The boreholes advanced at this Site extended to approximately 11.6 to 14.8 mbgs, where refusal was encountered on weathered shale bedrock. SPT "N" values within the soil deposit ranged between 1 and 23 blows per 300 mm. As such, based on three levels of underground parking and based on Table 4.1.8.4.A of the OBC, this Site has been classified as Class C. A Site Class C has an average shear wave velocity (Vs) of between 360 and 760 m/s. It is recommended that shear wave velocity soundings be completed at the Site once the final design and depths of foundations are known as a higher Site Classification may be available. In addition, Pinchin recommends that a liquefaction analysis be completed in conjunction with the shear wave velocity sounding.

5.5 Underground Parking Garage Design

It is understood that the proposed building will include three levels of underground parking; however, at this time the final grades for the underside of the underground parking garage footings are unknown. Groundwater was measured within the monitoring wells installed in Boreholes MW102 and MW104 at approximately 4.6 mbgs.

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

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

$$P = \gamma \times d$$

Where:

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

γ = unit weight of water (9.8 kN/m^3)

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

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

Alternatively, the exterior perimeter foundation drains should be installed where subsurface walls are exposed to the interior. The foundation drains should consist of a minimum 150 mm diameter fabric wrapped perforated drainage tile surrounded by 19 mm diameter clear stone (OPSS 1004) with a minimum cover of 150 mm on top and sides and 50 mm below the drainage tile. Since the natural soil contains a significant amount of silt sized particles, the clear stone gravel should be wrapped in a non-woven geotextile (Terrafix 270R or equivalent). The water collected from the weeping tile should be directed away from the building to appropriate drainage areas, either through gravity flow or interior sump pump systems. All subsurface walls should be waterproofed.

As the proposed basement floor level is to be constructed below the stabilized groundwater level, an underfloor drainage system should be installed beneath the slab, in addition to the installation of perimeter weeping tiles at the footing level. The floor slab sub drains should be constructed in a similar fashion to the foundation drains and be connected to a suitable frost-free outlet or sump.

If the building is constructed below the groundwater table and utilities sub drains and pumps are used to remove the groundwater from around the building footprint, there is the potential that a Permit to Take Water from the Ministry of the Environment, Conservation and Parks (MECP) will be required for the long term dewatering of the Site.

The walls must also be designed to resist lateral earth pressure. Depending on the design of the building the earth pressure computations must take into account the groundwater level at the Site. For calculating the lateral earth pressure, the coefficient of at-rest earth pressure (K_0) may be assumed at 0.5 for non-cohesive sandy soil. The bulk unit weight of the retained backfill may be taken as 20 kN/m^3 for well compacted soil. An appropriate factor of safety should be applied.

5.6 Floor Slabs

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

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

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

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

Material Type	Modulus of Subgrade Reaction (kN/m^3)
Granular A (OPSS 1010)	85,000
Granular "B" Type I (OPSS 1010)	75,000
Granular "B" Type II (OPSS 1010)	85,000
Silty Clay	2,000

6.0 SITE SUPERVISION & QUALITY CONTROL

It is recommended that all geotechnical aspects of the project be reviewed and confirmed under the appropriate geotechnical supervision, to routinely check such items. This includes but is not limited to inspection and confirmation of the 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.

7.0 TERMS AND LIMITATIONS

This Geotechnical Investigation was performed for the exclusive use of Somerset & O'Connor Limited Partnership (Client) in order to evaluate the subsurface conditions at 311 Somerset Street West, Ottawa, Ontario. Within the limitations of scope, schedule and budget, our services have been executed in accordance with generally accepted practises in the field of geotechnical engineering for the Site. Classification and identification of soil, and geologic units have been based upon commonly accepted methods employed in professional geotechnical practice. No warranty or other conditions, expressed or implied, should be understood. Conclusions derived are specific to the immediate area of study and cannot be extrapolated extensively away from sample locations.

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

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

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

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

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

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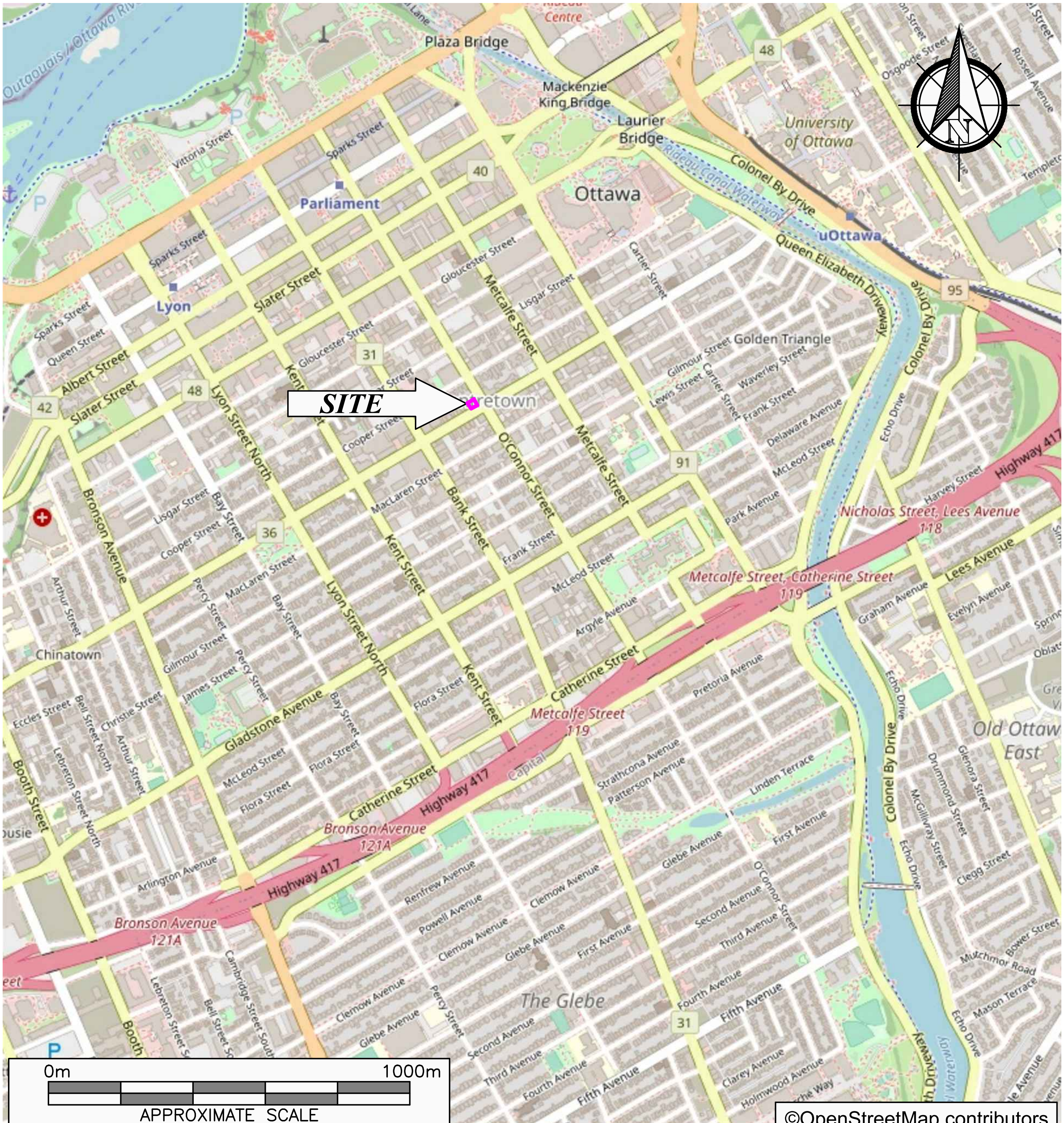
FIIGURES



PINCHIN	PROJECT NAME GEOTECHNICAL INVESTIGATION		
	CLIENT NAME SOMERSET & O'CONNOR LIMITED PARTNERSHIP		
	PROJECT LOCATION 311 SOMERSET STREET WEST, OTTAWA, ONTARIO		
	FIGURE NAME KEY MAP		FIGURE NO. 1
	APPROXIMATE SCALE AS SHOWN	PROJECT NO. 246162.001	DATE MARCH 2021

LEGEND

- BOREHOLE / MONITORING WELL LOCATION
- [XX.XX] GROUND SURFACE ELEVATION (m)
- [XX.XX] BOREHOLE TERMINATION ELEVATION (m)
- m METRES

PINCHIN

PROJECT NAME	GEOTECHNICAL INVESTIGATION
CLIENT NAME	SOMERSET & O'CONNOR LIMITED PARTNERSHIP
PROJECT LOCATION	311 SOMERSET STREET WEST, OTTAWA, ONTARIO
FIGURE NAME	BOREHOLE & MONITORING WELL LOCATION PLAN
APPROXIMATE SCALE	AS SHOWN
PROJECT NO.	246162.001
DATE	MARCH 2021
FIGURE NO.	2

APPENDIX I

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

ABBREVIATIONS, TERMINOLOGY & PRINCIPAL SYMBOLS USED

Sampling Method

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

In-Situ Soil Testing

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

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

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

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

Soil Descriptions

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

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

Notes:

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

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

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

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

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

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

Soil & Rock Physical Properties

General

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

Consistency

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

Shear Strength

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

Consolidation (One Dimensional)

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

Permeability

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

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

Rock Coring

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

RQD is calculated as follows:

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

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

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

APPENDIX II

Pinchin's Borehole Logs

Log of Borehole: BH101

Project #: 246162.001 **Logged By:** WT

Project: Geotechnical Investigation

Client: Somerset & O'Connor Limited Partnership

Location: 311 Somerset Street West, Ottawa, Ontario

Drill Date: February 10, 2021 **Project Manager:** WT

SUBSURFACE PROFILE

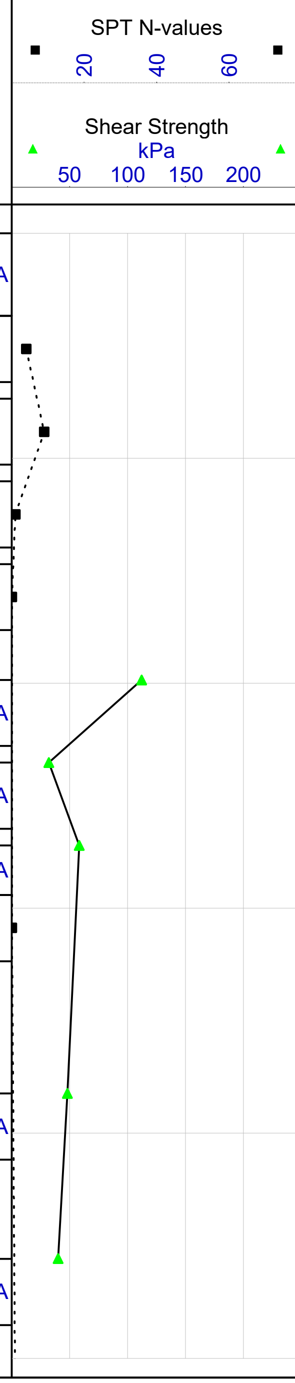
Description	Elevation (m)
Ground Surface	99.61
Organics ~ 100 mm	
Fill Brown sand and gravel, trace silt, frozen	98.85
Brown sand, trace gravel, trace silt, very loose, damp	97.63
Coarse	97.32
Silty Clay Grey silty clay, very soft, APL	96.56
APL to WTPL	

Monitoring Well Details: No Monitoring Well Installed

Depth (m): 0–10

SAMPLE

Sample Type	Sampler #	Recovery (%)	SPT N-values
AS	1	100	N/A
SS	2	80	4
SS	3	40	9
SS	4	100	1
SS	5	100	0
FV	1	N/A	N/A
FV	2	N/A	N/A
FV	3	N/A	N/A
SS	6	100	0
FV	4	N/A	N/A
FV	5	N/A	N/A



Lab Analysis	Moisture (%)	Plasticity Index

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

Drilling Method: Hollow Stem Auger / Split Spoon/ NQ Diamond Bit **Top of Casing Elevation:** N/A

Well Casing Size: N/A **Sheet:** 1 of 2

Log of Borehole: BH101

Project #: 246162.001 ***Logged By:*** WT

Project: Geotechnical Investigation

Client: Somerset & O'Connor Limited Partnership

Location: 311 Somerset Street West, Ottawa, Ontario

Drill Date: February 10, 2021 ***Project Manager:*** WT

Depth (m)	Symbol	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-values	Lab Analysis	Moisture (%)	Plasticity Index
			88.94								
11		**Glacial Till** Grey silty sand and gravel, very loose, wet			SS	7	100	1	Grain Size	8.6	
12			87.42								
13		**Shale Bedrock** Blackish brown highly weatered shale bedrock			SS	8	80	26			
14–15			84.06								
16		Moderately weathered, black with grey and white banding, fine to medium grained, some natural fractures with little to no oxidation, very poor to poor quality			RC	1	100	N/A			20
17–18			81.02		RC	2	100	N/A			47
19–20		End of Borehole Borhole terminated at 18.6 mbgs in shale bedrock.									

SPT N-values: 20, 40, 60

Shear Strength kPa: 50, 100, 150, 200

Contractor: Strata Drilling Group

Grade Elevation: 99.61 m

Drilling Method: Hollow Stem Auger / Split Spoon/ NQ Diamond Bit

Top of Casing Elevation: N/A

Well Casing Size: N/A

Sheet: 2 of 2

Log of Borehole: MW102

Project #: 246162.001 **Logged By:** WT

Project: Geotechnical Investigation

Client: Somerset & O'Connor Limited Partnership

Location: 311 Somerset Street West, Ottawa, Ontario

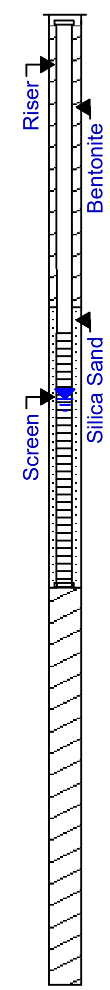
Drill Date: February 12, 2021 **Project Manager:** WT

SUBSURFACE PROFILE

Depth (m)	Description	Elevation (m)
0	Ground Surface	99.28
0–0.1	**Organics** ~ 100 mm	
	Fill Grey silty clay, frozen	98.21
	Moist	97.76
	Silty Clay Brown silty clay, firm, APL	
	Grey, very soft, wet	91.66
	Glacial Till Grey silty sand and gravel, very loose, wet	88.61
	End of Borehole	87.70

Borehole terminated at 11.6 mbgs due to auger refusal on probable weathered shale bedrock.

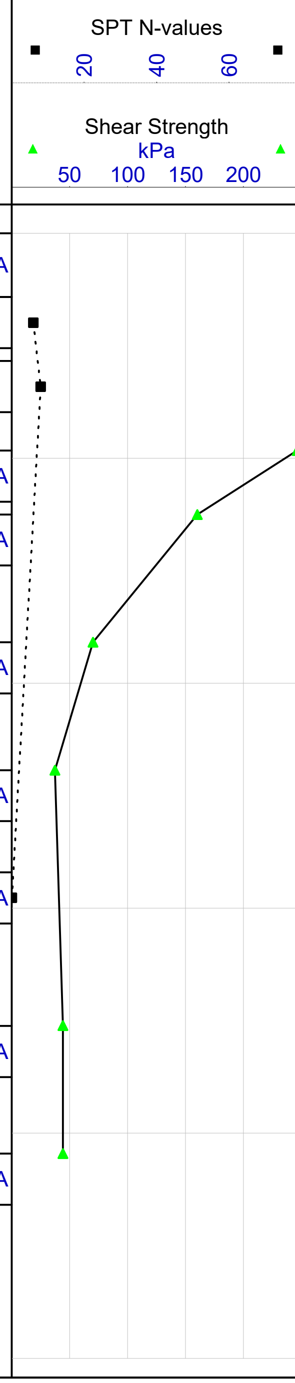
Monitoring Well Details



Groundwater level = 4.6 mbgs, as measured on March 4, 2021.

SAMPLE

Sample Type	Sampler #	Recovery (%)	SPT N-values
AS	1	100	N/A
SS	2	100	6
SS	3	100	8
FV	1	N/A	N/A
FV	2	N/A	N/A
FV	3	N/A	N/A
FV	4	N/A	N/A
SS	4	100	N/A
FV	5	N/A	N/A
FV	6	N/A	N/A



Lab Analysis	Moisture (%)	Plasticity Index
Hyd. / Att.	54.1	47

Contractor: Strata Drilling Group **Grade Elevation:** 99.28 m

Drilling Method: Hollow Stem Auger / Split Spoon **Top of Casing Elevation:** N/A

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

Log of Borehole: BH103

Project #: 246162.001 **Logged By:** WT

Project: Geotechnical Investigation

Client: Somerset & O'Connor Limited Partnership

Location: 311 Somerset Street West, Ottawa, Ontario

Drill Date: February 11, 2021 **Project Manager:** WT

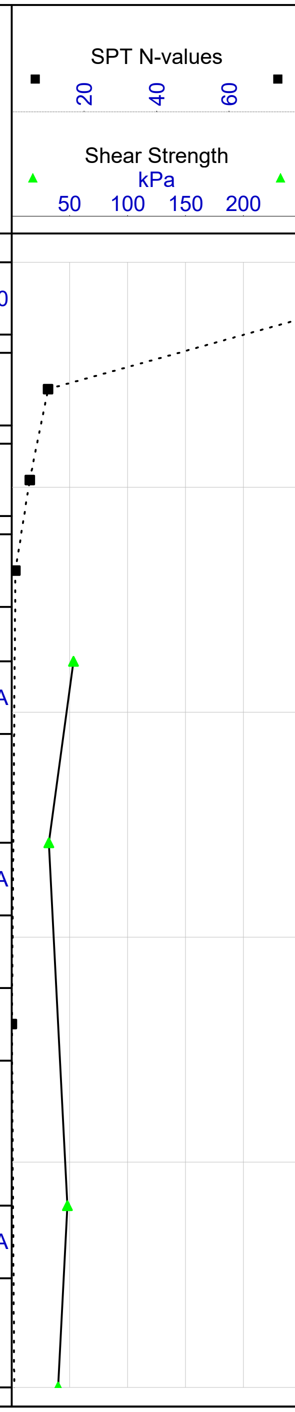
SUBSURFACE PROFILE

Depth (m)	Description	Elevation (m)
0	Ground Surface	99.20
0 – 0.76	**Fill** Brown sand and gravel, trace silt, trace asphalt, frozen	98.44
0.76 – 1.52	No asphalt, some brick debris	97.68
1.52 – 1.98	Trace wood, trace brick debris, damp	97.22
1.98 – 6.1	**Silty Clay** Brown silty clay, very soft, APL to WTPL	93.10
6.1 –	WTPL	

Monitoring Well Details: No Monitoring Well Installed

SAMPLE

Sample Type	Sampler #	Recovery (%)	SPT N-values	Lab Analysis	Moisture (%)	Plasticity Index
SS	1	90	100			
SS	2	30	10			
SS	3	80	5			
SS	4	100	1	Hyd. / Att.	49.6	47
FV	1	N/A	N/A			
FV	2	N/A	N/A			
SS	5	100	0			
FV	3	N/A	N/A			



Contractor: Strata Drilling Group **Grade Elevation:** 99.20 m

Drilling Method: Hollow Stem Auger / Split Spoon **Top of Casing Elevation:** N/A

Well Casing Size: N/A **Sheet:** 1 of 2

Log of Borehole: BH103

Project #: 246162.001 **Logged By:** WT

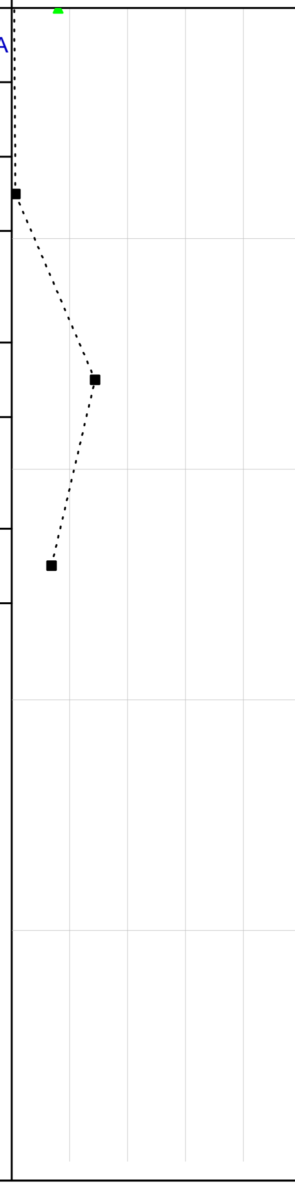
Project: Geotechnical Investigation

Client: Somerset & O'Connor Limited Partnership

Location: 311 Somerset Street West, Ottawa, Ontario

Drill Date: February 11, 2021 **Project Manager:** WT

SUBSURFACE PROFILE				SAMPLE							
Depth (m)	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-values	SPT N-values (20, 40, 60) / Shear Strength kPa (50, 100, 150, 200)	Lab Analysis	Moisture (%)	Plasticity Index
10		88.53		FV	4	N/A	N/A				
11	***Glacial Till*** Grey silty sand and gravel, very loose, wet			SS	6	100	1				
12				SS	7	100	23				
13		85.48									
14	Compact	85.18		SS	8	100	11				
	Shale Bedrock Blackish brown highly weathered shale	84.87									
15	End of Borehole										
16	Borehole terminated at 14.3 mbgs due to auger refusal in shale bedrock.										
17											
18											



Contractor: Strata Drilling Group **Grade Elevation:** 99.20 m

Drilling Method: Hollow Stem Auger / Split Spoon **Top of Casing Elevation:** N/A

Well Casing Size: N/A **Sheet:** 2 of 2

Log of Borehole: MW104

Project #: 246162.001 **Logged By:** WT

Project: Geotechnical Investigation

Client: Somerset & O'Connor Limited Partnership

Location: 311 Somerset Street West, Ottawa, Ontario

Drill Date: February 12, 2021 **Project Manager:** WT

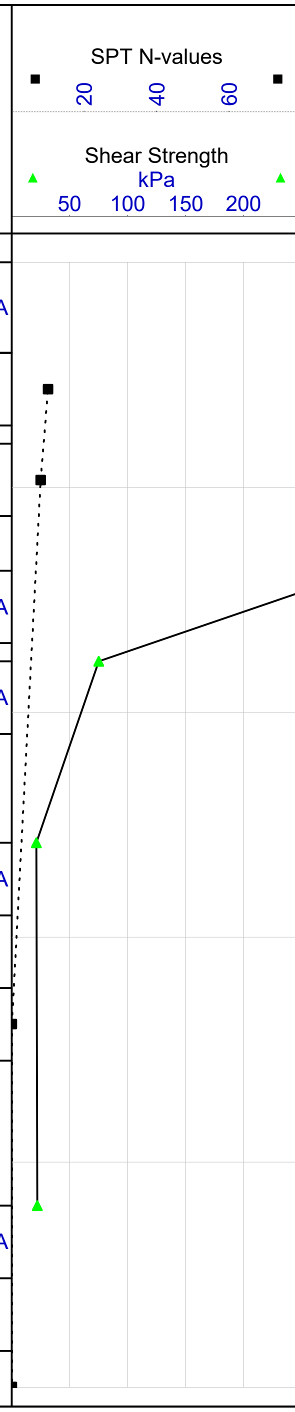
SUBSURFACE PROFILE

Depth (m)	Description	Elevation (m)
0	Ground Surface	99.20
	Fill Brown sand and gravel, trace silt, frozen	98.44
1	Trace brick, loose, damp	97.68
2	***Silty Clay*** Brown silty clay, firm, moist	96.61
3–5	Grey	93.10
6–9	Very soft, wet	

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

SAMPLE

Sample Type	Sampler #	Recovery (%)	SPT N-values
AS	1	100	N/A
SS	2	100	10
SS	3	30	8
FV	1	N/A	N/A
FV	2	N/A	N/A
FV	3	N/A	N/A
SS	4	100	0
FV	4	N/A	N/A
SS	5	100	0



Lab Analysis | Moisture (%) | Plasticity Index

Contractor: Strata Drilling Group **Grade Elevation:** 99.17 m

Drilling Method: Hollow Stem Auger / Split Spoon **Top of Casing Elevation:** N/A

Well Casing Size: N/A **Sheet:** 1 of 2

Log of Borehole: MW104

Project #: 246162.001 **Logged By:** WT

Project: Geotechnical Investigation

Client: Somerset & O'Connor Limited Partnership

Location: 311 Somerset Street West, Ottawa, Ontario

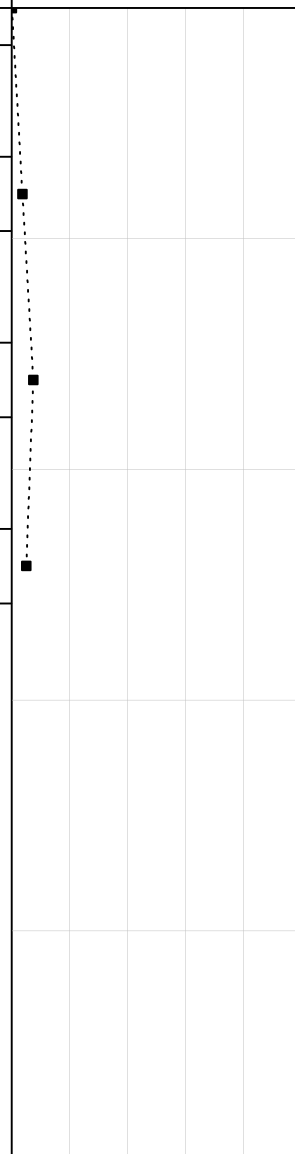
Drill Date: February 12, 2021 **Project Manager:** WT

SUBSURFACE PROFILE / SAMPLE

Depth (m)	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-values	Lab Analysis	Moisture (%)	Plasticity Index
10		88.53			5					
11	**Glacial Till** Grey silty sand and gravel, very loose, wet			SS	6	100	3			
12		87.01								
	Loose			SS	7	100	6			
13										
14		85.18								
	Shale Bedrock Blackish brown highly weathered shale bedrock	84.42		SS	8	100	4			
15	End of Borehole. Borehole terminated at 14.8 mbgs due to auger refusal in shale bedrock.		Groundwater level = 4.6 mbgs, as measured on March 4, 2021.							
16										
17										
18										

SPT N-values: 20, 40, 60

Shear Strength kPa: 50, 100, 150, 200



Contractor: Strata Drilling Group **Grade Elevation:** 99.17 m

Drilling Method: Hollow Stem Auger / Split Spoon **Top of Casing Elevation:** N/A

Well Casing Size: N/A **Sheet:** 2 of 2

APPENDIX III

Laboratory Testing Reports for Soil Samples

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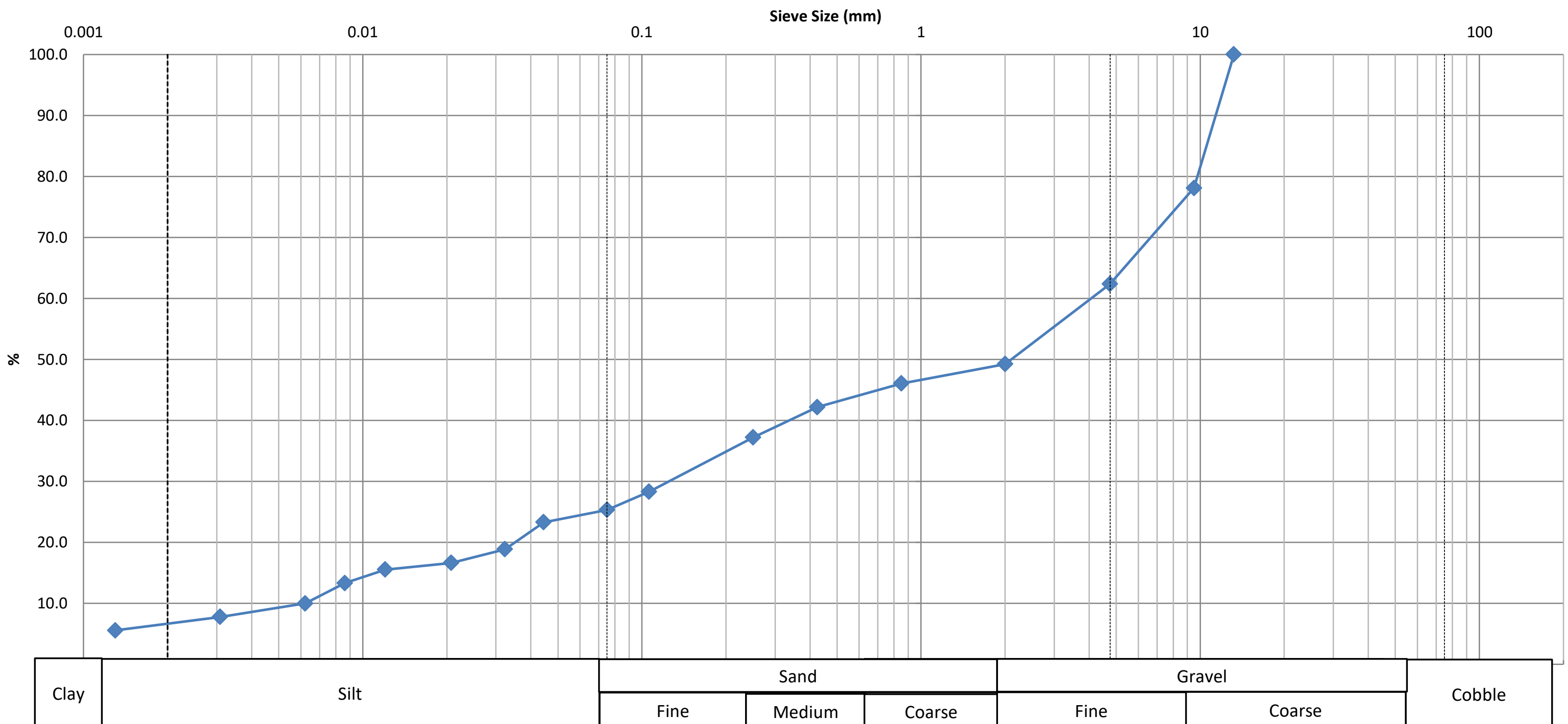
SIEVE ANALYSIS
ASTM C136

CLIENT:	Pinchin	DEPTH:	35' - 37'	FILE NO:	PM4184
CONTRACT NO.:		BH OR TP No.:	BH 101	LAB NO:	23594
PROJECT:	246142.001			DATE RECEIVED:	26-Feb-21
				DATE TESTED:	2-Mar-21
DATE SAMPLED:	25-Feb-21			DATE REPORTED:	5-Mar-21
SAMPLED BY:	Client			TESTED BY:	D.B



Identification	Soil Classification				MC(%)	LL	PL	PI	Cc	Cu
					8.6					
	D100	D60	D30	D10	Gravel (%)	Sand (%)	Silt (%)	Clay (%)		
					37.6	37.1	18.8	6.5		

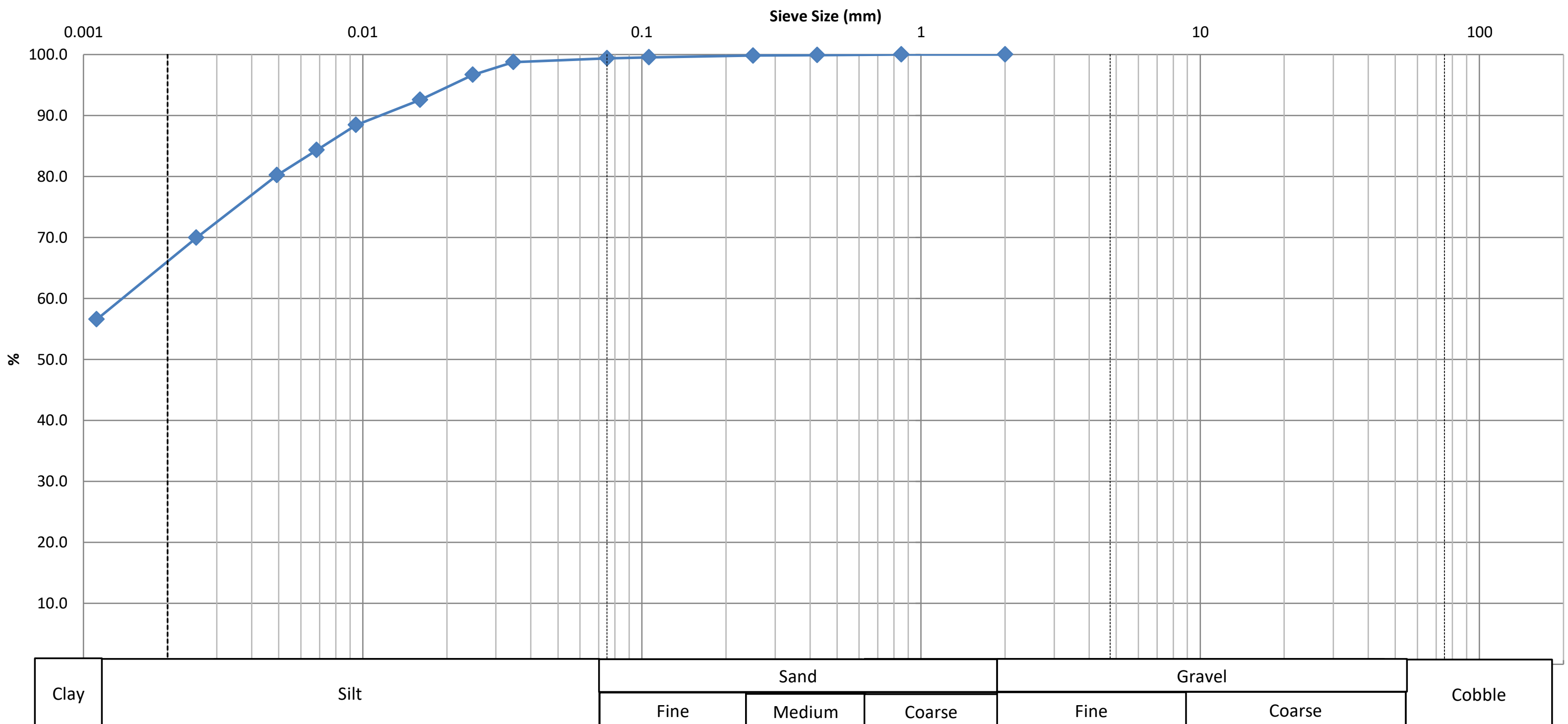
Comments:

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

patersongroup
consulting engineers

SIEVE ANALYSIS
ASTM C136

CLIENT:	Pinchin	DEPTH:	10' - 12'	FILE NO:	PM4184
CONTRACT NO.:		BH OR TP No.:	MW 102	LAB NO:	23593
PROJECT:	246142.001			DATE RECEIVED:	26-Feb-21
				DATE TESTED:	2-Mar-21
DATE SAMPLED:	25-Feb-21			DATE REPORTED:	5-Mar-21
SAMPLED BY:	Client			TESTED BY:	D.B



Identification	Soil Classification				MC(%)	LL	PL	PI	Cc	Cu
					54.1					
	D100	D60	D30	D10	Gravel (%)		Sand (%)	Silt (%)	Clay (%)	
					0.0		0.6	31.9	67.5	

Comments:

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

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consulting engineers

ATTERBERG LIMITS
LS-703/704

CLIENT:	Pinchin	FILE NO.:	PM4184
PROJECT:	246162.001	DATE SAMPLED:	25-Feb-21
LOCATION:	MW 102 @ 10' - 12'	DATE REPORTED:	5-Mar-21

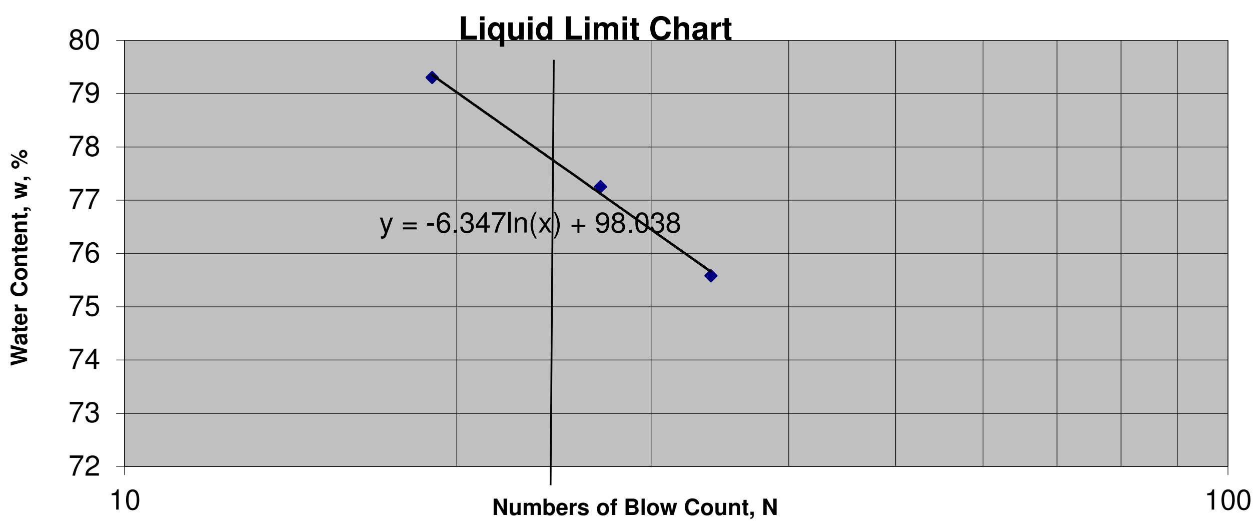
LIQUID LIMIT DETERMINATION

CAN NO.	11	12	13				
WT. OF CAN	8.59	8.66	8.62				
WT. OF SOIL & CAN	15.78	15.75	16.17				
WT. OF DRY SOIL & CAN	12.60	12.66	12.92				
WT. OF MOISTURE	3.18	3.09	3.25				
WT. OF DRY SOIL & CAN	4.01	4	4.3				
WATER CONTENT, w, %	**79.3**	**77.25**	**75.58**				
NO. OF BLOWS, N	19	27	34				

PLASTIC LIMIT DETERMINATION

CAN NO.	14	15
WT. OF CAN	20.09	19.90
WT. OF SOIL & CAN	27.21	26.95
WT. OF DRY SOIL & CAN	25.58	25.35
WT. OF MOISTURE	1.63	1.6
WT. OF DRY SOIL & CAN	5.49	5.45
WATER CONTENT, w, %	**29.69**	**29.36**

LIQUID LIMIT	**77**
PLASTIC LIMIT	**30**
PLASTICITY INDEX	**47**



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

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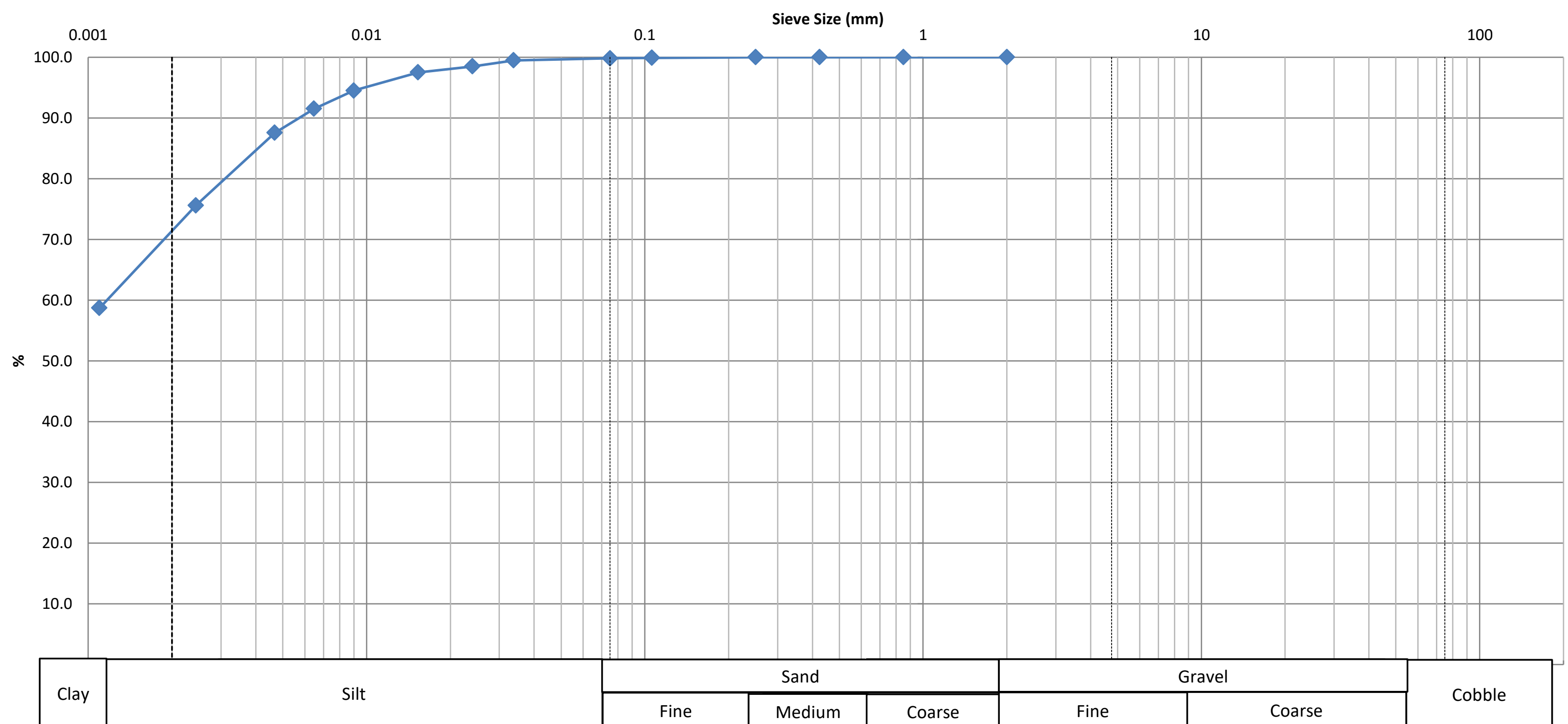
SIEVE ANALYSIS
ASTM C136

CLIENT:	Pinchin	DEPTH:	7.5' - 9.5'	FILE NO:	PM4184
CONTRACT NO.:		BH OR TP No.:	BH103	LAB NO:	23592
PROJECT:	246142.001			DATE RECEIVED:	26-Feb-21
				DATE TESTED:	2-Mar-21
DATE SAMPLED:	25-Feb-21			DATE REPORTED:	5-Mar-21
SAMPLED BY:	Client			TESTED BY:	D.B



Identification	Soil Classification				MC(%)	LL	PL	PI	Cc	Cu
					49.6					
	D100	D60	D30	D10	Gravel (%)	Sand (%)	Silt (%)	Clay (%)		
					0.0	0.2	28.8	71.0		

Comments:

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

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ATTERBERG LIMITS
LS-703/704

CLIENT:	Pinchin	FILE NO.:	PM4184
PROJECT:	246162.001	DATE SAMPLED:	25-Feb-21
LOCATION:	BH103 @ 7' 5" - 9' 5"	DATE REPORTED:	5-Mar-21

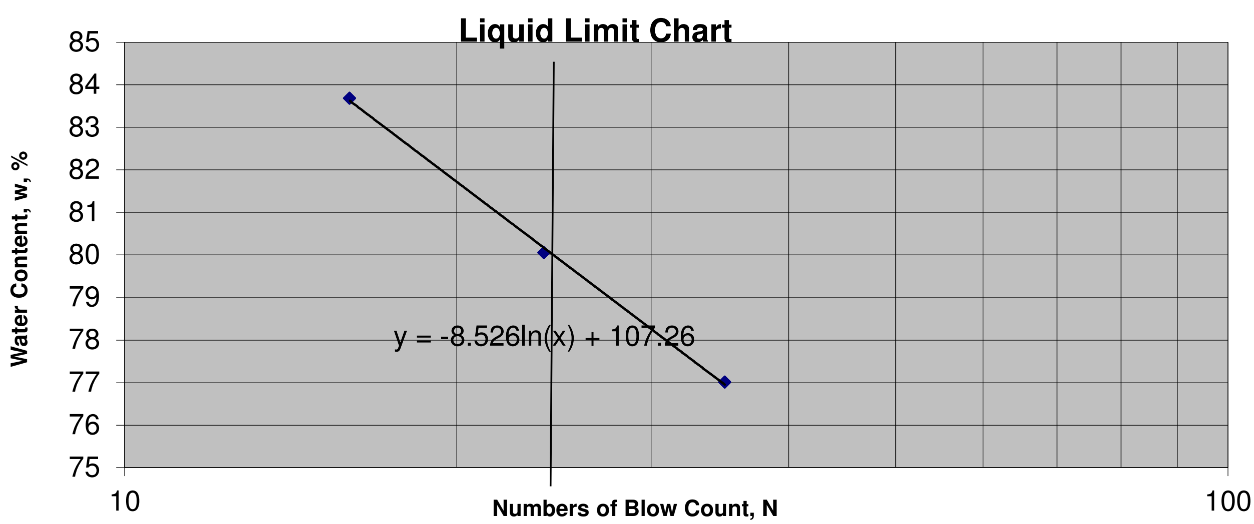
LIQUID LIMIT DETERMINATION

CAN NO.	**30**	**31**	**32**				
WT. OF CAN	4.36	4.33	4.39				
WT. OF SOIL & CAN	11.45	12.09	10.55				
WT. OF DRY SOIL & CAN	8.22	8.64	7.87				
WT. OF MOISTURE	3.23	3.45	2.68				
WT. OF DRY SOIL & CAN	3.86	4.31	3.48				
WATER CONTENT, w, %	**83.68**	**80.05**	**77.01**				
NO. OF BLOWS, N	16	24	35				

PLASTIC LIMIT DETERMINATION

CAN NO.	**3**	**14**
WT. OF CAN	19.42	20.05
WT. OF SOIL & CAN	27.72	28.28
WT. OF DRY SOIL & CAN	25.69	26.23
WT. OF MOISTURE	2.03	2.05
WT. OF DRY SOIL & CAN	6.27	6.18
WATER CONTENT, w, %	**32.38**	**33.17**

LIQUID LIMIT	**80**
PLASTIC LIMIT	**33**
PLASTICITY INDEX	**47**



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

APPENDIX IV
Rock Core Photograph

Photo 1 – Borehole BH101, Rock Core

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.