FINAL

Geotechnical Investigation - Proposed Residential Development

South Portion of 13 and 21 Bridge Street, Picton, Ontario

Prepared for:

Walcott Capitol
2028 Country Road
Picton, ON K0KI 2T0

Attn: Mr. Scott Walcott

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MEMBER OF
THE PINCHIN GROUP

Issued to:	**Walcott Capitol**
Contact:	**Mr. Scott Walcott**
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Issuing Office:	**1 Hines Road, Suite 200, Kanata, ON K2K 3C7**
Primary Contact:	**Wesley Tabaczuk, P.Eng.** **Project Manager, Geotechnical Services**

W. Tabaczuk

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

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

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1.0 INTRODUCTION

Pinchin Ltd. (Pinchin) was retained by Walcott Capitol (Client) to conduct a Geotechnical Investigation and provide subsequent geotechnical design recommendations for the proposed residential development to be located at the South Portion of 13 and 21 Bridge Street, Picton, Ontario (Site). The Site location is shown on Figure 1.

Based on information provided by the Client, it is Pinchin's understanding that the proposed development is to consist of a four-storey residential apartment building with a single level underground parking garage. It is noted that the due to the sloping nature of Site, the underground parking garage level will be partially exposed along the north and west elevations. The proposed development will also include Site service trenches; however, there are no asphalt surfaced driveways or parking areas included.

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 three sampled boreholes (Boreholes BH3/MW3 to BH5/MW5) within the vicinity of the proposed building footprint. The information gathered from the Geotechnical Investigation will allow Pinchin to provide geotechnical design recommendations for the proposed development.

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

- A review of relevant area geology and Site background information;
- A detailed description of the soil and groundwater conditions;
- Foundation subgrade preparation;
- Site service trench design;
- Open cut excavations;
- Anticipated groundwater management;
- Foundation design recommendations including soil bearing resistances at Ultimate Limit States (ULS) and Serviceability Limit States (SLS) design;
- Potential foundation settlements;
- Foundation frost protection and engineered fill specifications and installation;

- Seismic Site classification for seismic Site response;
- Interior concrete floor slab-on-grade (including modulus of subgrade reaction); and
- Building drainage and foundation backfill.

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 north side of Bridge Street and on the west side of Mortimer Street in Picton, Ontario. The Site is currently undeveloped and consists of grassed areas with some mature trees. It is noted that based on a review of historical aerial photographs for the area, the Site was previously occupied by single family residential dwellings; however, Pinchin is unaware of whether or not the foundations for the previous buildings have been removed. The lands adjacent to the Site are predominantly developed with residential buildings and forested areas.

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 glaciolacustrine deposit consisting of massive to well laminated silt and clay with minor sand and gravel deposits. The underlying bedrock at this Site is of the Shadow Lake Formation consisting of limestone, dolostone, shale, arkose, and sandstone (Ontario Geological Survey Map 1972, published 1978).

3.0 GEOTECHNICAL FIELD INVESTIGATION AND METHODOLOGY

Pinchin completed a field investigation at the Site on March 13 and 14, 2017 by advancing a total of three sampled boreholes throughout the Site. The boreholes were advanced to sampled depths ranging from approximately 6.0 to 8.3 metres below the 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 0.76 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 were obtained on the recovered cohesive soil samples using a hand held pocket penetrometer and the results are plotted on the appended borehole logs.

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 consists of surficial granular fill overlying silt and clay fill and natural sand to the maximum borehole termination depth of approximately 8.2 mbgs.

The surficial granular fill material was encountered within all boreholes and was measured to range in thickness from approximately 0.8 and 2.3 m. The granular fill material can be described as sand containing trace to some gravel, and trace silt. The granular fill had a very loose to compact relative density based on SPT “N” values of 2 to 20 blows per 300 mm penetration of a split spoon sampler.

The silt and clay fill material was encountered within Boreholes BH3/MW3 and BH4/MW underlying the granular fill material. The silt and clay fill was measured to range in thickness from approximately 0.7 and 1.5 m and contained trace sand. Undrained shear strengths measured with a hand held pocket penetrometer of the fill material ranged from 125 to 138 kilopascals (kPa) indicating a very stiff consistency. The results of a particle size distribution analysis performed on a sample of the silt and clay fill indicates that the sample contains 1% sand, 59% silt, and 40% clay. The moisture content of the sample tested was 17.4% indicating the material was About the Plastic Limit (ATPL) to Wetter Than the Plastic Limit (WTPL). Atterberg Limit testing indicates that the fill material is of medium plasticity.

The natural sand material was encountered within all boreholes underlying either the fill materials, and extended to the maximum borehole termination depth of approximately 8.2 mbgs. The sand was observed to possess trace gravel, trace silt and had a variable very loose to compact relative density based on SPT "N" values of 0 to 26 blows per 300 mm penetration of a split spoon sampler. The results of particle size distribution analyses performed on samples of the sand indicated that the samples contain 0 to 1% gravel, 91 to 96% sand, and 3 to 9% silt.

4.2 Groundwater Conditions

Groundwater observations and measurements were obtained in the open boreholes at the completion of drilling and are summarized on the appended borehole logs. In addition, groundwater levels were measured in the monitoring wells between July 17 and 18, 2017. The groundwater observations and measurements are summarized in the following table:

Borehole No.	Geodetic Ground Surface Elevation (masl)	Groundwater Depth (mbgs) June 17-18, 2018	Groundwater Elevation (masl)	Soil Type
BH-MW-3	83.4	7.6	75.8	Sand
BH-MW-4	81.7	6.6	75.1	Sand
BH-MW-5	80.1	4.3	75.8	Sand

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.

Pinchin was provided with the following drawings to aid in providing appropriate Site specific design recommendations:

- "Site Plan", prepared by Gifford, Harris Surveying Ltd., File No. 2-9635; and
- "3D Massing, Bridge Street, Picton, ON", prepared by Hobin Architecture Incorporated, dated June 22, 2018.

Based on a review of the above referenced drawings, the proposed development is to consist of a four storey residential apartment building, complete with a single level underground parking garage which will have a finished floor at approximate geodetic elevation 78.0 metres above sea level (masl). It is noted that the due to the sloping nature of Site, the underground parking garage level will be partially exposed along the north and west elevations.

In the Picton, Ontario area, exterior perimeter foundations for heated buildings require a minimum of 1.4 m of soil cover above the underside of the footing to provide soil cover for frost protection. As such, the footings for the proposed building development will need to be installed at approximate geodetic elevation 76.6 masl in order to provide adequate soil cover for frost protection. The natural soils encountered at this elevation are not considered suitable to support the typical loads from a residential building of this type. Therefore, Pinchin recommends to improve the natural subgrade soil by installing a 0.9 m thick reinforced engineered fill pad on the natural sand located at 76.6 masl and provide a combination of soil cover and rigid polystyrene insulation, such as Dow Styrofoam or equivalent product for frost protection as the underside of foundation will be at Elevation 77.5 masl.

5.2 Anticipated Groundwater Management & Open Cut Excavations

Excavations for the proposed building foundation system are expected to extend to approximately 76.6 masl. Prior to excavating for the proposed building foundations the groundwater will need to be lowered to a minimum geodetic elevation of 74.8 masl (i.e. ~ 1 m below the highest measured groundwater elevation) in order for proper placement of the reinforced engineered fill pad. A dewatering system installed by a specialist dewatering contractor will be required to lower the groundwater level prior to excavation. The design of the dewatering system should be left to the contractor's discretion, and the system should meet a performance specification to maintain and control the groundwater to a minimum geodetic elevation of 74.8 masl.

The groundwater control should be maintained until the foundations are installed and backfilled to at least 0.6 m above the natural groundwater elevation or the proposed finished grade, whichever is less.

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.

Seasonal variations in the groundwater table should be expected; as such, depending on the groundwater at the time of the excavation works, a more involved dewatering system may be required. Any potential precipitation or seepage entering the excavations should be pumped away immediately (not allowed to pond).

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 in-situ soil may be classified as Type 3 soil above the groundwater table. Temporary excavations in these soils must be cut at an inclination of 1 horizontal to 1 vertical (H to V) or less from the base of the excavation. The in-situ soil below the groundwater table would be classified as a Type 4 soil and temporary side slopes must be cut from the excavation bottom with a minimum gradient of 3H to 1V as per O.Reg. 231/91.

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.

5.3 Natural Soil Subgrade Preparation

Based on the information obtained within the boreholes advanced at the Site, very loose deposits of sand were encountered in Borehole BH5/MW5 and weaker pockets of subgrade soil are expected to be encountered between the investigation locations. Pinchin presumes that any areas of weaker subgrade soil will consist of small pockets of very loose natural soil which can be compacted to match the density of the remainder of the Site, once dewatering has been completed. As such, the sand material must be compacted to a minimum of 100% Standard Proctor Maximum Dry Density (SPMDD) prior to installing the reinforced engineered fill pad. Any loose areas which are not able to achieve the recommended 100% SPMDD are to be removed and replaced with a Granular “A” OPSS 1010, or approved natural sand, compacted to 100% SPMDD. Additionally any existing building foundations, floor slabs or service trenches which extend below the proposed foundation areas will need to be removed and replaced with Granular “A” OPSS 1010, or approved natural sand compacted to 100% SPMDD.

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 if subjected to additional water or precipitation. As well, it could be easily disturbed if travelled on during construction. To avoid any disturbance to the natural subgrade, swamp mats should be used.

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, fill, disturbed, or caved materials; 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.

5.4 Reinforced Engineered Fill Pad Installation

The reinforced engineered fill pad is to be installed overlying the previously compacted and approved natural subgrade soil located approximately 76.6 masl. The engineered fill pad is to consist of the following:

- Installation of a non-woven geotextile (Terrafix 270R or equivalent) and a biaxial geo-grid (Terrafix TBX2000 or equivalent) atop the natural subgrade soil prepared as outlined above;
- Installation of 0.9 m of Granular A (OPSS 1010), which is to be divided into two 450 mm layers, separated by a biaxial geo-grid (Terrafix TBX2000 or equivalent); and
- The reinforced engineered fill pad is to extend a minimum horizontal distance of 1.2 m beyond the outside face of the foundations and slope down at 1H:1V to ensure loads are properly transferred to the underlying natural subgrade soil as well as to accommodate the installation of the geogrid.

The biaxial geo-grid is to be installed in accordance with the following recommendations as well as the manufacturer's specifications:

- For strip footings, the geogrid is to be oriented such that the roll length runs parallel to the footing direction;
- For square footings, the geogrid shall be oriented such that the roll direction runs perpendicular to the roll direction of the previous layer of geogrid;
- The geogrid is to extend out a minimum of 1.2 m from the outside edge of the footing in all directions;
- The geogrid is to have a minimum overlap of 600 mm. To prevent separation during construction it may be held together with plastic ties, wire ties, staples, etc.;
- The geogrid must be laid down so there are no kinks, ripples or waves and is to be secured in place with either staples, pins, sand bags or piles of granular backfill;
- Granular fill material is to be placed so that it minimizes movement and the above aforementioned; and
- A minimum loose granular fill thickness of 200 mm should be installed above the geogrid prior to compaction and or construction equipment operation over the geogrid. Depending on the subgrade conditions at the time of construction this thickness may have to be increased. Sharp turns and sudden stopping from compaction and/or construction equipment are to be avoided.

5.5 Shallow Foundations Bearing on Reinforced Engineered Fill

Conventional shallow strip footings established at approximate geodetic elevation 77.5 masl on a reinforced engineered fill pad constructed as outlined above may be designed using a bearing resistance for 25 mm of settlement at Serviceability Limit States of 150 kPa, and a factored geotechnical bearing resistance of 225 kPa at Ultimate Limit States (ULS). As the actual service loads and proposed final grades were not known at the time of this report, these should be reviewed by the project structural engineer to determine if SLS or ULS governs the footing design.

5.5.1 Foundation Transition Zones

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 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 to 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.

5.5.2 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.

5.5.3 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.

It is recommended that exterior perimeter foundation drains be installed where subsurface walls are exposed to the interior (basement walls).

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. The clear stone gravel should be wrapped in a non-woven geotextile (Terrafix 270R or equivalent). The water collected from the weeping tile should be directed away from the building to appropriate drainage areas; either through gravity flow or interior sump pump systems. All subsurface walls should be water proofed. Additionally due to the high groundwater levels at the Site, any basement will need to be designed to resistance hydrostatic uplift or designed with an underfloor drainage system. A Permit to Take Water may be required if over 50,000 litres a day of water is pumped from the foundation drainage and underfloor drainage systems. Pinchin would be able to provide further permanent dewatering recommendations once final design is complete.

5.5.4 Shallow Foundation Frost Protection & Foundation Backfill

In the Picton, Ontario area, exterior perimeter foundations for heated buildings require a minimum of 1.4 m of soil cover above the underside of the footing to provide soil cover for frost protection. As previously mentioned, Pinchin recommends to install the foundations on an engineered fill pad which will extend approximately 0.9 m above the frost penetration depth; as such, the footings will require a combination of soil cover and rigid polystyrene insulation to provide frost protection.

To provide a combination of soil cover and rigid polystyrene insulation frost protection for the building foundations, Pinchin recommends the following:

- The insulation should have a minimum thermal resistance value of R-15. If Dow Styrofoam is used this would require a total of 75 mm of insulation;
- The insulation should buttress the foundation wall and extend out a minimum horizontal distance of 1.4 m beyond the outside face of the footing;
- The insulation should be installed vertically on the perimeter of the foundation wall, from approximately 100 mm below the proposed finished grade to the top of the horizontal insulation;
- A minimum of 500 mm of soil cover is to be provided above the insulation at all times; and
- The insulation placed beyond the foundation is to have a positive slope away from the building foundation.

These insulation recommendations assume the interior of the building is maintained at 18 degrees Celsius or higher.

The insulation is to be installed as outlined above as well as in accordance with the manufactures requirements. The insulation recommendations are Site specific and are not to be used for any other structures except for the Site in which it was intended for. Where there is less than 760 mm of soil cover above the insulation and heavy vehicular traffic is present, a Dow HI (High Load) product should be considered.

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 or Type II Ontario Provincial Standards and Specifications (OPSS 1010), or approved sand fill, extending a minimum lateral distance of 600 mm beyond the foundation. The maximum aggregate size should not exceed 200 mm in diameter. It is critical that particles greater than 200 mm in diameter are not in contact with the foundation or insulation to prevent point loading and overstressing. The backfill material used against the foundation must be placed so that the allowable lateral capacity is achieved. Ideally, during backfilling operations, all backfill material should be placed on each side of the foundation in equal lifts, not exceeding 300 mm. Additionally, it is recommended that exterior grades around the foundation be sloped away at a 2% gradient or more, for a distance of at least 2 m so that surface water is diverted away from the foundation to further mitigate soil frost adhesion.

The natural soils are not considered suitable as foundation backfill material.

5.6 Rammed Aggregate Piers® to Densify the Soil

As an alternative to lowering the groundwater table and constructing the reinforced engineered fill pad for conventional foundations, a Rammed Aggregate Pier® (RAP) system can be utilized to provide higher bearing resistances. RAP soil reinforcing using the Geopier GP3® installation method consists of auguring a hole in the natural subgrade soil and filling it by using vertical ramming energy to compact thin lifts of high quality crushed rock. The first lift of crushed rock forms a bulb below the bottom of the pier, thereby pre-stressing and pre-straining the soils to a depth equal to at least one pier diameter below the base of the hole. Ramming takes place with a high-energy bevelled tamper that both densifies the crushed rock and forces it laterally into the sidewalls of the hole. This action increases the lateral stress in the surrounding soil; thereby further stiffening the stabilized composite soil mass. This results in the formation of very stiff, high-density aggregate piers which can then provide higher bearing resistances.

RAP is a proprietary system and must be designed by an experienced design build contractor. The number and size of aggregate piers are determined based on the building loads and configuration. The design build contractor will work closely with the design team to determine the most cost effective foundation solution for the Site. Budget pricing and detailed design information can be provided once the preliminary structural information is available.

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

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

The boreholes advanced at this Site extended to between approximately 6.0 and 8.3 mbgs, where the SPT “N” values ranged between 0 and 36 blows per 300 m. As such, based on Table 4.1.8.4.A of the OBC, this Site has been classified as Class E. A Site Class E has an average shear wave velocity (Vs) of less than 180 m/s. There is a potential that the Site Class may be higher; however, shear wave velocity measurements would be required for the determination of a higher Site Classification, as per the OBC.

5.8 Slab-on-Grade Floors

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 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 natural sand material encountered within the boreholes is considered adequate for the support of the concrete slab-on-grade provided it is proof roll compacted as outlined above. Provided organics are not encountered during excavations for the footings then the undisturbed natural soil may be left in place. 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-grade 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
Sand	35,000

5.9 Site Servicing

5.9.1 Pipe Bedding and Cover Materials for Flexible and Rigid Pipes

The subgrade soil conditions beneath the Site services will comprise natural sand. It is noted that depending on the location of the Site service trenches, there is a potential for very loose pockets of sand to be encountered which are not considered suitable to support any service pipes. Any potential loose pockets of sand will need to be removed and replaced with additional granular bedding. 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, than an approximate 150 mm granular pad consisting of 19 mm clear stone gravel (OPSS 1004) wrapped in a non-woven geotextile (Terrafix 270R or equivalent) should be considered to maintain the integrity of the natural subgrade soils. The clear stone should contain a minimum of 50% crushed particles. Water collected within the stone should be controlled through sumps and filtered pumps.

5.9.2 Trench Backfill

Above the pipe cover material, the trench can be backfilled by re-using the excavated natural soil matching the materials exposed on the sides of the trenches. The soil should be placed to the underside of the granular subbase of the pavement structure, and be compacted in maximum 300 mm thick lifts to 98% SPMDD within 4% of the optimum moisture content. This is recommended to provide soil compatibility and help minimize potential abrupt differential frost heave between surrounding natural materials similar in composition. The natural material must be free of organics or other deleterious material.

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

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

5.9.3 Frost Protection

The frost penetration depth in Picton, Ontario for these types of soil conditions is estimated to extend to approximately 1.7 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 2.0 mbgs or lower as dictated by municipal service requirements. If a minimum of 2.0 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 manufactures recommendations. To accommodate the placement of horizontal insulation a wider excavation trench may be required.

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 DISCLAIMER

This Geotechnical Investigation was performed for the exclusive use of Walcott Capitol (Client) in order to evaluate the subsurface conditions at South Portion of 13 and 21 Bridge Street, Picton, Ontario.

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

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

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

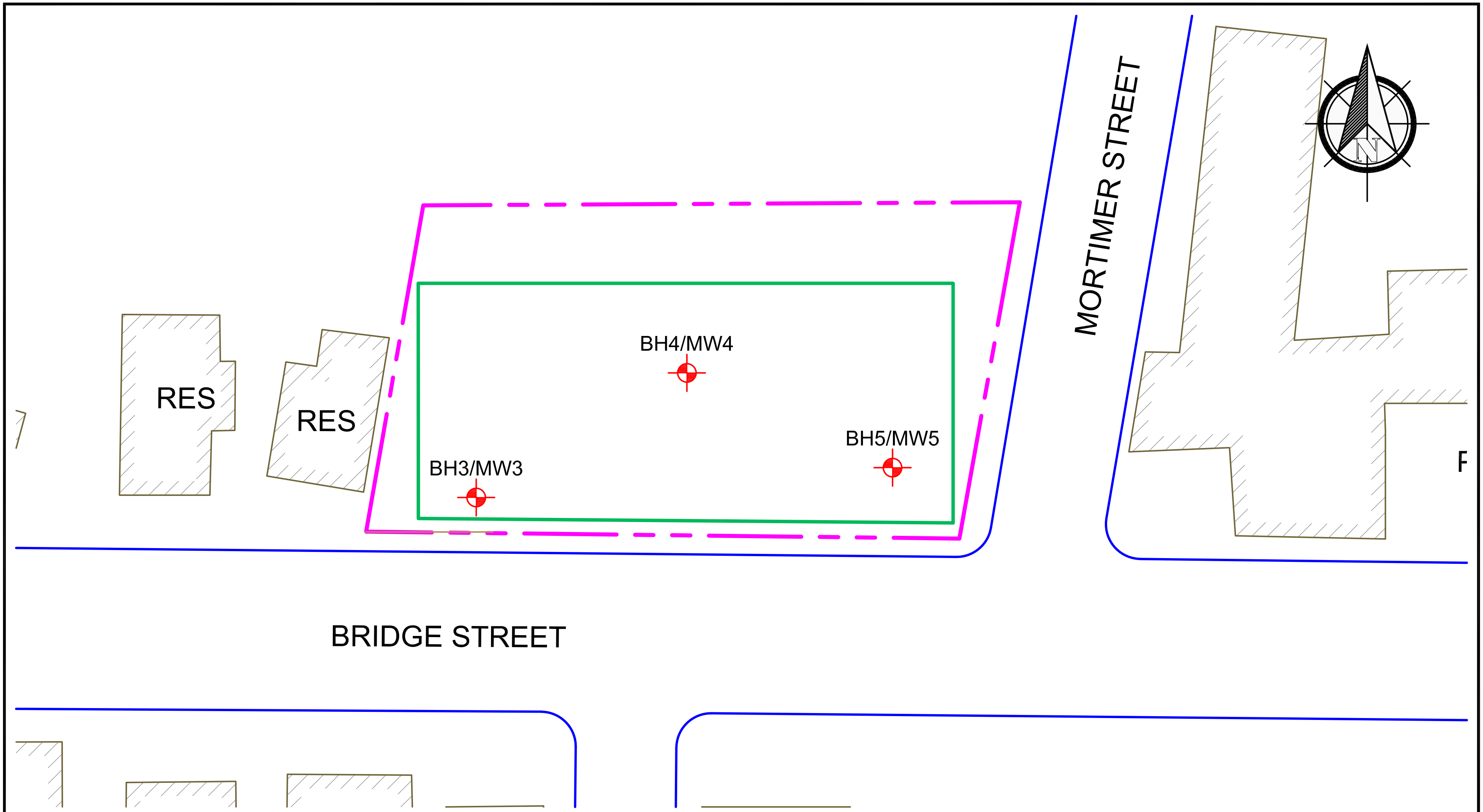
This report has been prepared for the exclusive use of the Client and their authorized agents. Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of the third parties. If additional parties require reliance on this report,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

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

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

FIGURES

PINCHIN	CLIENT NAME: WALCOTT CAPITOL				
	PROJECT NAME: GEOTECHNICAL INVESTIGATION				
	LOCATION: SOUTH PORTION OF 13 AND 21 BRIDGE STREET, PICTON, ONTARIO				
	TITLE: KEY MAP				
	DATE: OCTOBER 2018	PROJECT #: 205918	IMAGE SOURCE: OPENSTREETMAP	DRAWN BY: E.KOURNOSSOV	CHECKED BY: T.FATIGUN	FIGURE #: 1



PINCHIN

LEGEND

PROPOSED RESIDENTIAL BUILDING

APPROXIMATE SITE BOUNDARY

APPROXIMATE BOREHOLE LOCATION

PROJECT NAME
GEOTECHNICAL INVESTIGATION

CLIENT NAME
WALCOTT CAPITOL

PROJECT LOCATION
SOUTH PORTION OF 13 AND 21 BRIDGE STREET, PICTON, ONTARIO

FIGURE NAME
BOREHOLE LOCATION PLAN

FIGURE NO.
2

APPROXIMATE SCALE	PROJECT NO.	DATE
AS SHOWN	205918	OCTOBER 2018

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

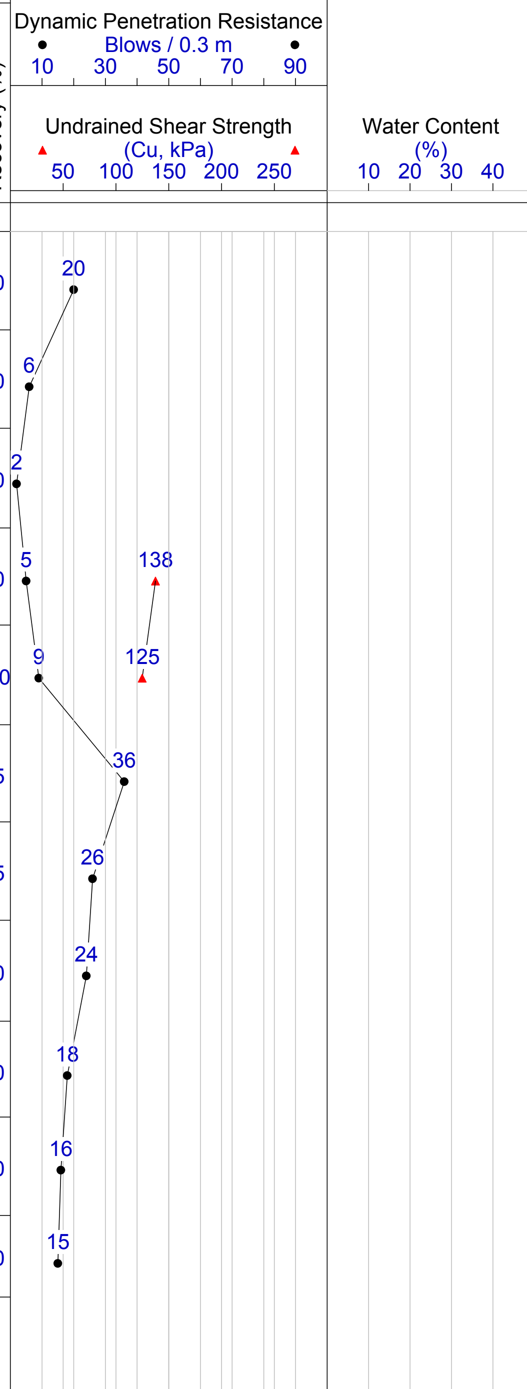
APPENDIX II
Pinchin's Borehole Logs

Borehole Log: BH3/MW3

PINCHIN
1 Hines Road, Suite 200
Kanata, Ontario

Project No.: 205918 **Logged By:** B.Hallam
Project: Geotechnical Investigation **Reviewed By:** V.Marshall
Client: Walcott Capital
Location: South Portion of 13 and 21 Bridge Street, Picton, Ontario

Depth (m)	Description	Elevation	Sample No.	Sample Type	Blows/0.3m	Recovery (%)	Remarks
0	Ground Surface	83.4					
0–2.3	Fill - Sand, trace to some gravel, trace silt, damp to moist, brown, very loose to compact	81.1	SS1	SS	20	80	
			SS2	SS	6	10	
			SS3	SS	2	50	
2.3–3.8	Fill - Silt and clay, trace sand, ATPL to WTPL, brown, very stiff	79.6	SS4	SS	5	90	
			SS5	SS	9	100	
3.8–8.2	Sand, trace gravel, trace silt, moist to wet, brown, compact to dense	75.2	SS6	SS	36	75	
			SS7	SS	26	75	
			SS8	SS	24	90	
			SS9	SS	18	90	
			SS10	SS	16	80	
			SS11	SS	15	80	At drilling completion groundwater was encountered at approximately 7.6 m depth
	End of Borehole						



Drilled By: Strata Drilling Group **Datum:** Geodetic
Drill Method: Hollow Stem/Split Spoon **Ground Elevation:** 83.4 m
Drill Date: March 13, 2017 **Sheet:** 1 of 1

This data relates to the boring and shouldn't be interpreted as being indicative of the whole site BH3/MW3

Borehole Log: BH4/MW4

PINCHIN

1 Hines Road, Suite 200
Kanata, Ontario

Project No.: 205918
Logged By: B.Hallam
Project: Geotechnical Investigation
Reviewed By: V.Marshall
Client: Walcott Capital
Location: South Portion of 13 and 21 Bridge Street, Picton, Ontario

SUBSURFACE PROFILE		SAMPLE						
Description	Elevation	Sample No.	Sample Type	Blows/0.3m	Recovery (%)	Dynamic Penetration Resistance (Blows / 0.3 m)	Undrained Shear Strength (Cu, kPa)	Remarks
Ground Surface	81.7							
Fill - Sand, trace to some gravel, trace silt, damp to moist, brown, loose	80.9	SS1	SS	7	90	7		
Fill - Silt and clay, trace sand, ATPL to WTPL, brown, very stiff	80.2	SS2	SS	6	75	6	125	
Sand, trace gravel, trace silt, moist to wet, brown, loose to compact		SS3	SS	10	80	10		
		SS4	SS	10	75	10		
		SS5	SS	8	90	8		
		SS6	SS	5	100	5		
		SS7	SS	7	50	7		
		SS8	SS	10	80	10		
		SS9	SS	13	80	13		At drilling completion groundwater was encountered at approximately 6.6 m depth
		SS10	SS	10	50	10		
	73.5	SS11	SS	5	50	5		
End of Borehole								

Depth (m): 0 – 9

Drilled By: Strata Drilling Group
Drill Method: Hollow Stem/Split Spoon
Drill Date: March 13, 2017
Datum: Geodetic
Ground Elevation: 81.7 m
Sheet: 1 of 1

This data relates to the boring and shouldn't be interpreted as being indicative of the whole site BH4/MW4

PINCHIN

1 Hines Road, Suite 200
Kanata, Ontario

Borehole Log: BH5/MW5

Project No.: 205918
Logged By: B.Hallam
Project: Geotechnical Investigation
Reviewed By: V.Marshall
Client: Walcott Capital
Location: South Portion of 13 and 21 Bridge Street, Picton, Ontario

SUBSURFACE PROFILE / SAMPLE

Depth (m)	Description	Elevation
0	Ground Surface	80.1
0–1.5	Fill - Sand, trace to some gravel, trace silt, damp to moist, brown, loose Very loose below approximately 0.76 m depth	78.6
1.5–5.9	Sand, trace gravel, trace silt, moist to wet, brown, very loose Wet below approximately 4.3 m depth	74.2
5.9	End of Borehole	

Sample No.	Sample Type	Blows/0.3m	Recovery (%)
SS1	SS	7	30
SS2	SS	3	70
SS3	SS	1	80
SS4	SS	0	60
SS5	SS	0	5
SS6	SS	0	15
SS7	SS	1	55
SS8	SS	3	10

Dynamic Penetration Resistance: Blows / 0.3 m (10, 30, 50, 70, 90) — plotted values: 7, 3, 1, 0, 0, 0, 1, 3

Undrained Shear Strength (Cu, kPa): 50, 100, 150, 200, 250

Water Content (%): 10, 20, 30, 40

Remarks: At drilling completion groundwater was encountered at approximately 4.3 m depth

Drilled By: Strata Drilling Group
Drill Method: Hollow Stem/Split Spoon
Drill Date: March 14, 2017
Datum: Geodetic
Ground Elevation: 80.1 m
Sheet: 1 of 1

This data relates to the boring and shouldn't be interpreted as being indicative of the whole site BH5/MW5

APPENDIX III
Analytical Laboratory Testing Reports for Soil Samples

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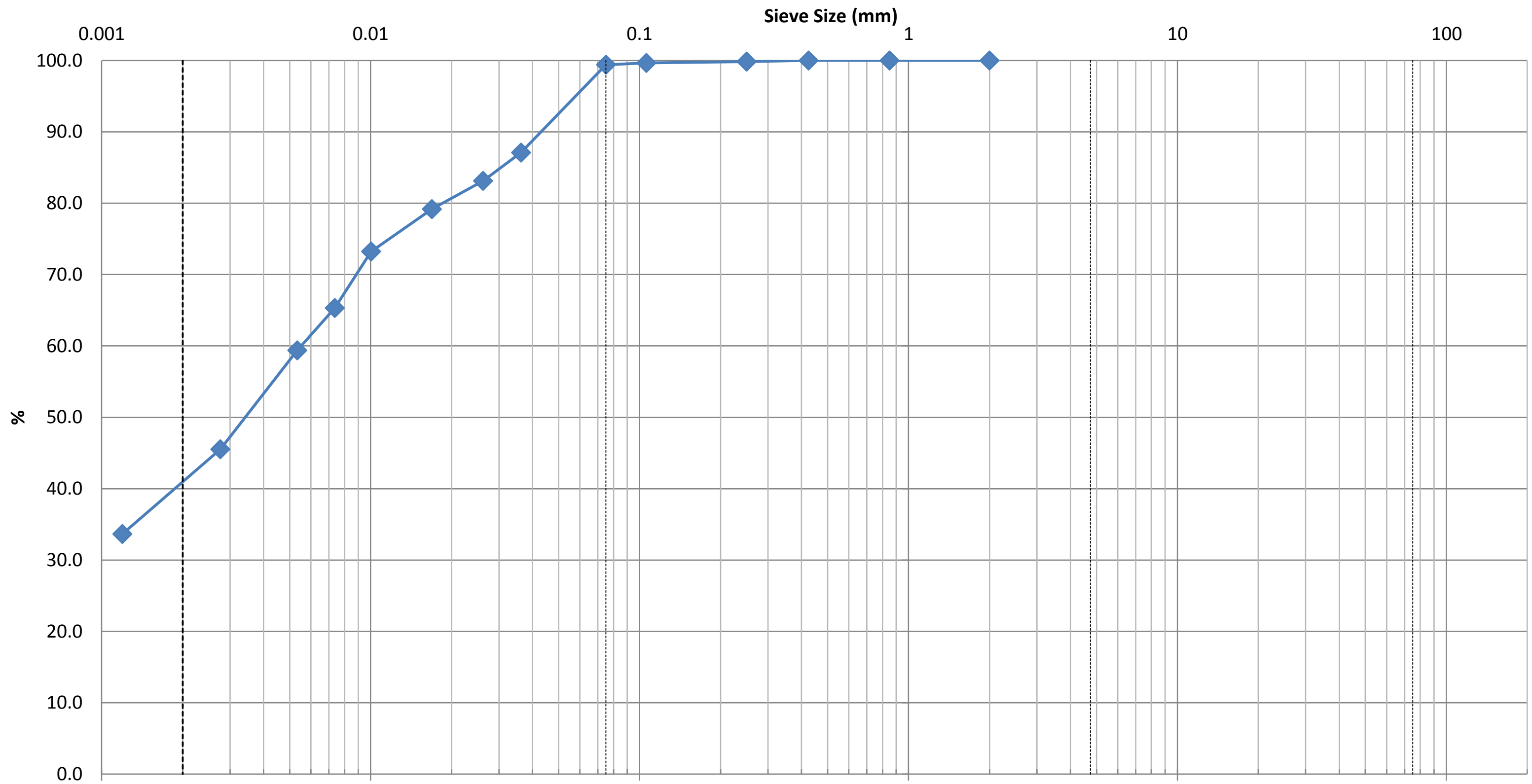
CLIENT:	Pinchin Environmental	DEPTH:	10' - 12'	FILE NO:	PM4184
CONTRACT NO.:		BH OR TP No.:	BH3/MW3	LAB NO:	93100
PROJECT:	205918			DATE RECEIVED:	6-Jul-17
				DATE TESTED:	10-Jul-17
DATE SAMPLED:	13-Jun-17			DATE REPORTED:	12-Jul-17
SAMPLED BY:	W. Tabaczuk			TESTED BY:	C. Beadow



	Clay	Silt	Sand			Gravel		Cobble
			Fine	Medium	Coarse	Fine	Coarse	

Identification	Soil Classification				MC(%)	LL	PL	PI	Cc	Cu
					17.4	16	31	15		
	D100	D60	D30	D10	Gravel (%)	Sand (%)	Silt (%)	Clay (%)		
					0.0	0.6	59.2	40.2		

Comments	

patersongroup
consulting engineers

HYDROMETER
LS-702 ASTM-422

CLIENT:	Pinchin Environmental	DEPTH:	10' - 12'	FILE NO.:	PM4184
PROJECT:	205918	BH OR TP No.:	BH3/MW3	DATE SAMPLED:	13-Jun-17
LAB No. :	93100	TESTED BY:	C. Beadow	DATE RECEIVED:	06-Jul-17
SAMPLED BY:	W. Tabaczuk	DATE REPT'D:	12-Jul-17	DATE TESTED:	10-Jul-17

SAMPLE INFORMATION

SAMPLE MASS	91.6	50.03
SPECIFIC GRAVITY (Gs)	2.700	
HYGROSCOPIC MOISTURE	Tare No.	
TARE Wt.	50.00	ACTUAL Wt.
AIR DRY (Wa)	150.00	100.00
OVEN DRY (Wo)	149.85	99.85
F=(Wo/Wa)	0.999	
INITIAL Wt. (Ma)	50.03	
Wt. CORRECTED	49.95	
Wt. AFTER WASH BACK SIEVE	0.3	
SOLUTION CONCENTRATION	40 g / L	

REMARKS

GRAIN SIZE ANALYSIS

SIEVE DIAMETER (mm)	WEIGHT RETAINED (g)	PERCENT RETAINED	PERCENT PASSING
63.0			
53.0			
37.5			
26.5			
19.0			
16.0			
13.2			
9.5			
4.75			
2.0	**0.0**	0.0	100.0
Pan	**91.6**		
0.850	**0.00**	0.0	100.0
0.425	**0.00**	0.0	100.0
0.250	**0.08**	0.2	99.8
0.106	**0.17**	0.3	99.7
0.075	**0.29**	0.6	99.4
Pan	**0.30**		
SIEVE CHECK 0.0	MAX = 0.3%		

HYDROMETER DATA

ELAPSED	TIME (24 hours)	Hs	Hc	Temp. (^{o}C)	DIAMETER	(P)	TOTAL PERCENT PASSING
1	8:51	49.0	5.0	24.0	0.0362	87.1	87.1
2	8:52	47.0	5.0	24.0	0.0262	83.1	83.1
5	8:55	45.0	5.0	24.0	0.0169	79.2	79.2
15	9:05	42.0	5.0	24.0	0.0100	73.2	73.2
30	9:20	38.0	5.0	24.0	0.0074	65.3	65.3
60	9:50	35.0	5.0	24.0	0.0053	59.4	59.4
250	13:00	28.0	5.0	24.0	0.0028	45.5	45.5
1440	8:50	22.0	5.0	24.5	0.0012	33.6	33.6

COMMENTS

Moisture Content = 17.4%

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

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Consulting Engineers

Atterberg Limits

Project	Pinchin Environmental - Job # 205918	File No.	PM4184
Sample Info	BH3/MW3 @ 10' - 12'	Date.	12-Jul-17

Liquid Limit Determination

							Natural Moisture Content
Can No	14	18	13				
Weight of Can	20.29	20.05	19.22				
Wt of Soil and Can	27.01	24.68	24.84				
Wt of Dry soil and Can	25.31	23.58	23.57				
Wt of Moisture	1.7	1.1	1.27				
Wt of Dry soil and Can	5.02	3.53	4.35				
Water Content, w, %	33.86	31.16	29.2				
No of Blows, N	18	25	31				

Plastic Limit Determination

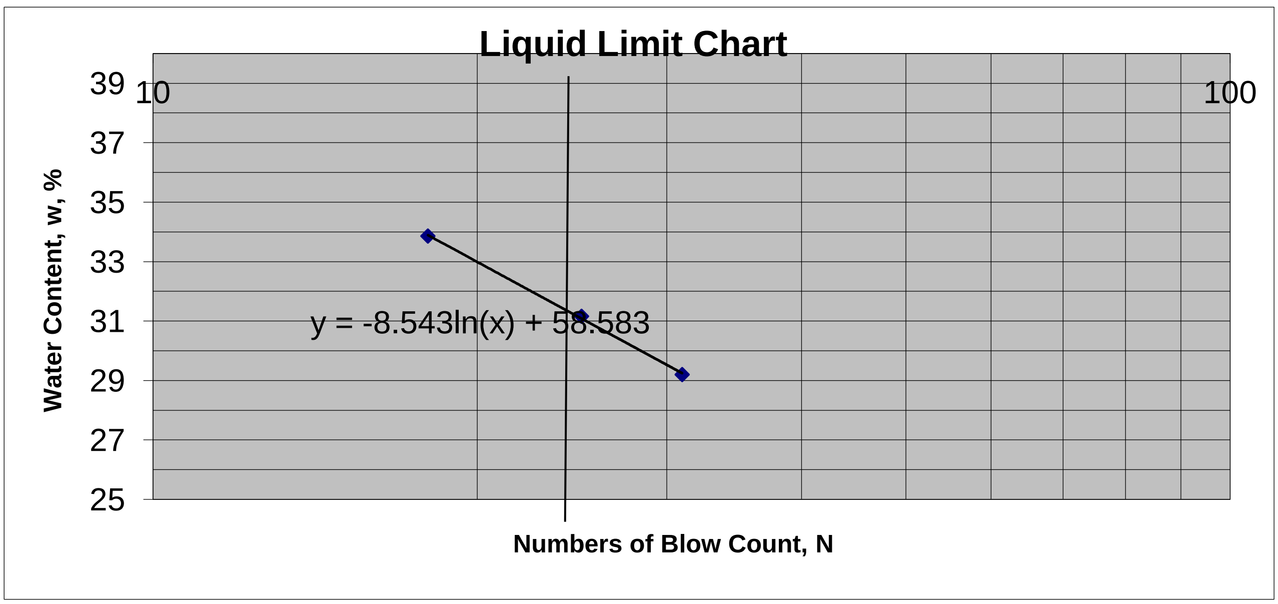
Can No	2	3
Weight of Can	8.59	8.6
Wt of Soil and Can	11.19	10.7
Wt of Dry soil and Can	10.83	10.41
Wt of Moisture	0.36	0.29
Wt of Dry soil and Can	2.24	1.81
Water Content, w, %	16.07	16.02

Results

Liquid Limit	**31**
Plastic Limit	**16**
Plasticitty Index	**15**
Natural Moisture Content	**0**

CB
Reviewed by: Curtis Beadow
Approved by: Stephen J. Walker, P.Eng

Liquid Limit Chart



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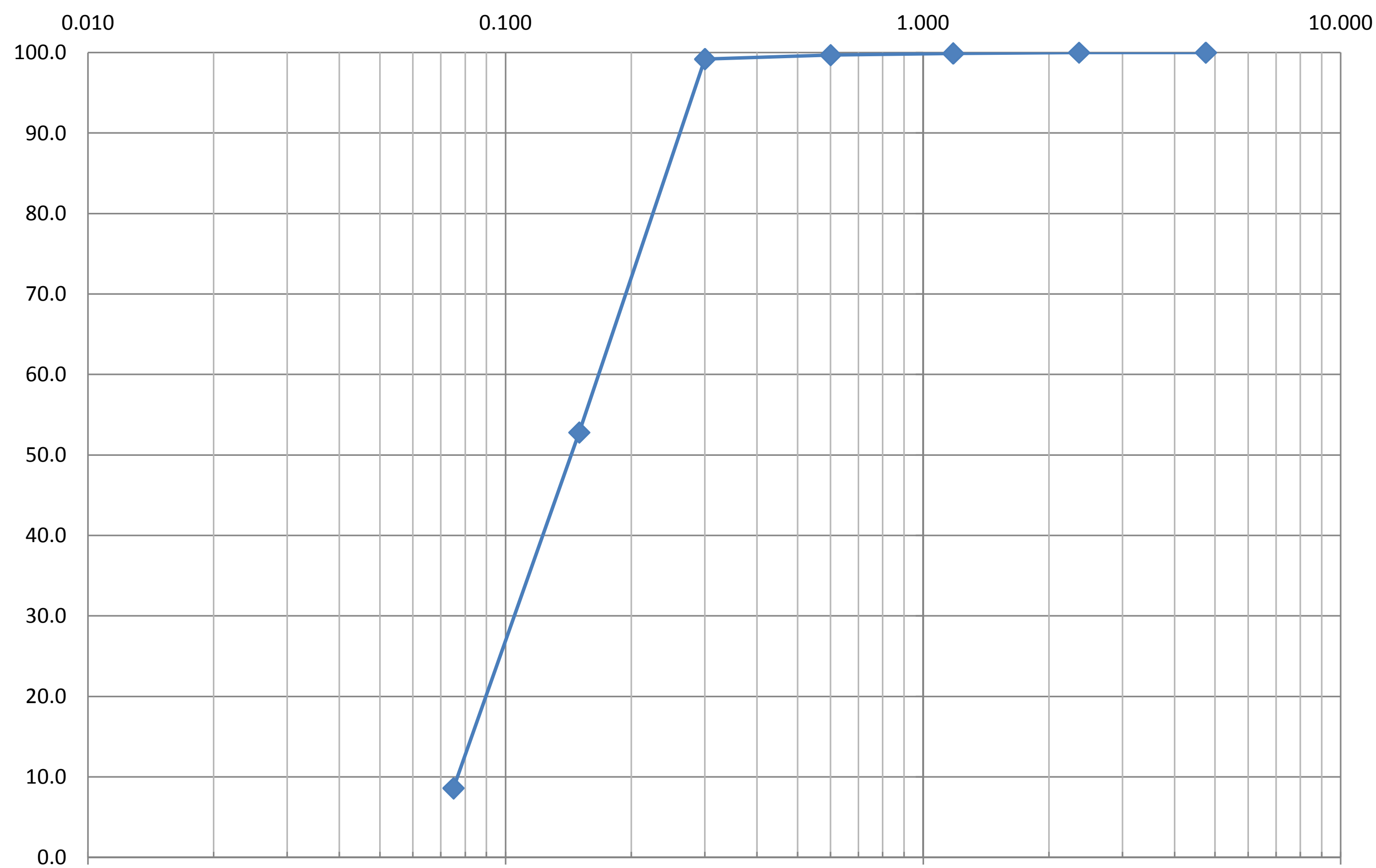
SIEVE ANALYSIS
ASTM C-136

CLIENT:	Pinchin Environmental	DESCRIPTION:	Sand	FILE NO:	PM4184
CONTRACT No.:	-	SPECIFICATION:	Fine Graded Sand	LAB NO:	93101
PROJECT:	Job # 205918	INTENDED USE:	-	DATE REC'D:	6-Jul-17
		PIT OR QUARRY:	-	DATE TESTED:	7-Jul-17
DATE SAMPLED:	14-Jun-17	SOURCE LOCATION:	BH4/MW4	DATE REPRT'D:	12-Jul-17
SAMPLED BY:	Client	SAMPLE LOCATION:	7.5' - 9.5'	TESTED BY:	E.B/C.B



Comments	

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SIEVE ANALYSIS
ASTM C-136

CLIENT:	Pinchin Environmental	DESCRIPTION:	Sand	FILE NO.:	PM4184
CONTRACT NO.:	-	SPECIFICATION:	Fine Graded Sand	LAB NO.:	93101
PROJECT:	Job # 205918	INTENDED USE:	-	DATE REC'D:	6-Jul-17
		PIT OR QUARRY:	-	DATE TESTED:	7-Jul-17
DATE SAMPLED:	14-Jun-17	SOURCE LOCATION:	BH4/MW4	DATE REPRT'D:	12-Jul-17
SAMPLED BY:	Client	SAMPLE LOCATION:	7.5' - 9.5'	TESTED BY:	E.B/C.B

WEIGHT BEFORE WASH			A+B	332.1
WEIGHT AFTER WASH	A	B	A+B	315.7

SIEVE SIZE (mm)	WEIGHT RETAINED	PERCENT RETAINED	PERCENT PASSING	LOWER SPEC	UPPER SPEC	REMARK
150						
106						
75						
63						
53						
37.5						
26.5						
19						
16						
13.2						
9.5						
6.7						
4.75	0.0	0.0	**100.0**			
2.36	0.1	0.0	**100.0**			
1.18	**0.3**	0.1	**99.9**			
0.6	**0.9**	0.3	**99.7**			
0.3	**2.5**	0.8	**99.2**			
0.15	**156.8**	47.2	**52.8**			
0.075	**303.4**	91.4	**8.6**			
PAN	**315.5**					
SIEVE CHECK FINE		0.06	0.3% max.			**REFERENCE MATERIAL**

OTHER TESTS	**RESULT**	**LAB NO.**	**RESULT**

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

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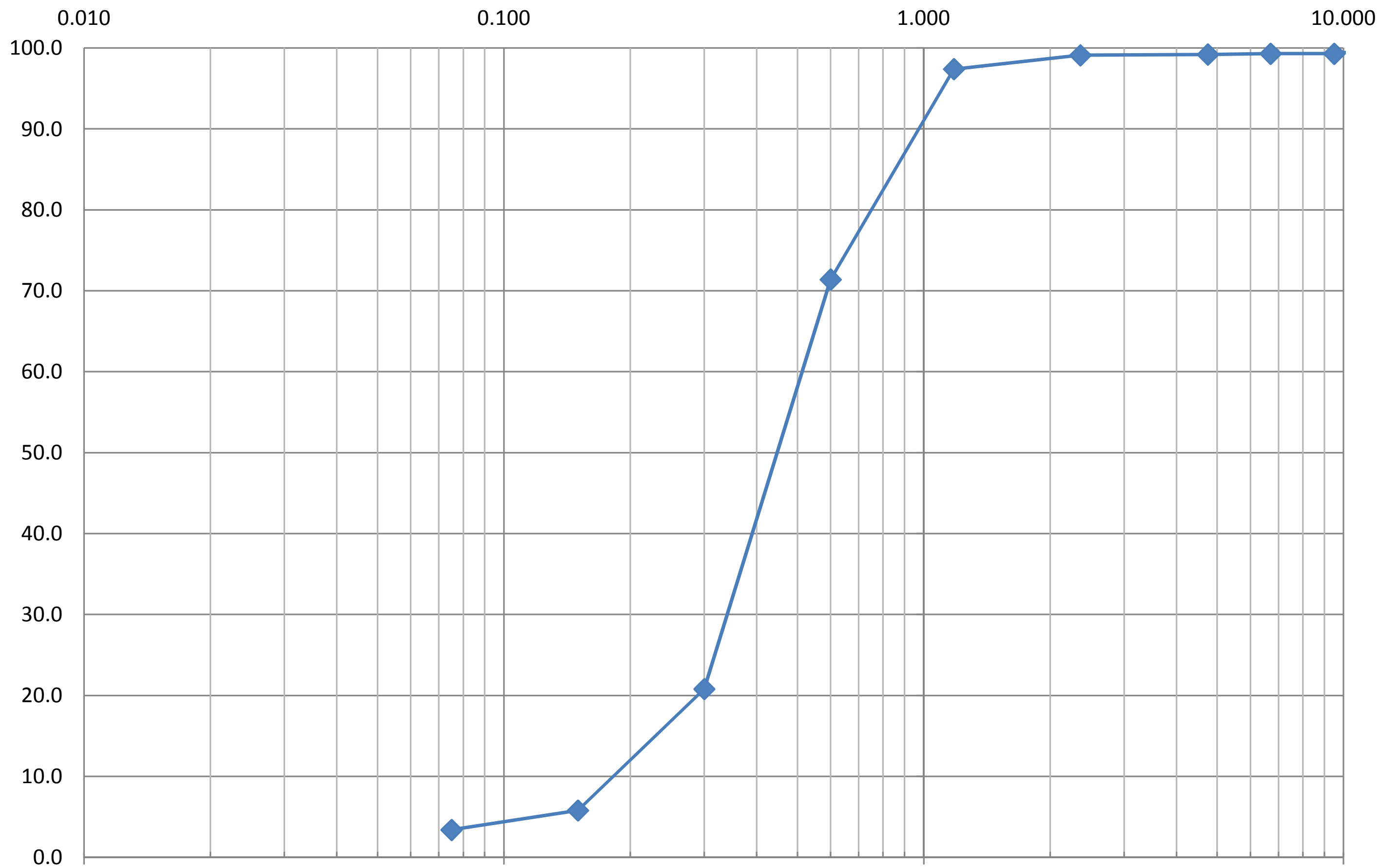
consulting engineers

SIEVE ANALYSIS
ASTM C-136

CLIENT:	Pinchin Environmental	DESCRIPTION:	Sand	FILE NO:	PM4184
CONTRACT No.:	-	SPECIFICATION:	Coarse Graded Sand	LAB NO:	93102
PROJECT:	Job # 205918	INTENDED USE:	-	DATE REC'D:	6-Jul-17
		PIT OR QUARRY:	-	DATE TESTED:	7-Jul-17
DATE SAMPLED:	14-Jun-17	SOURCE LOCATION:	BH4/MW4	DATE REPRT'D:	12-Jul-17
SAMPLED BY:	Client	SAMPLE LOCATION:	22.5' - 24.5'	TESTED BY:	E.B/C.B



Comments	

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SIEVE ANALYSIS
ASTM C-136

CLIENT:	Pinchin Environmental	DESCRIPTION:	Sand	FILE NO.:	PM4184
CONTRACT NO.:	-	SPECIFICATION:	Coarse Graded Sand	LAB NO.:	93102
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DATE SAMPLED:	14-Jun-17	SOURCE LOCATION:	BH4/MW4	DATE REPRT'D:	12-Jul-17
SAMPLED BY:	Client	SAMPLE LOCATION:	22.5' - 24.5'	TESTED BY:	E.B/C.B

WEIGHT BEFORE WASH			A+B	468.8
WEIGHT AFTER WASH	A	B	A+B	458.4

SIEVE SIZE (mm)	WEIGHT RETAINED	PERCENT RETAINED	PERCENT PASSING	LOWER SPEC	UPPER SPEC	REMARK
150						
106						
75						
63						
53						
37.5						
26.5						
19						
16						
13.2	**0.0**	0.0	**100.0**			
9.5	**3.3**	0.7	**99.3**			
6.7	**3.3**	0.7	**99.3**			
4.75	**3.7**	0.8	**99.2**			
2.36	**4.4**	0.9	**99.1**			
1.18	**12.1**	2.6	**97.4**			
0.6	**134.3**	28.6	**71.4**			
0.3	**371.5**	79.2	**20.8**			
0.15	**441.7**	94.2	**5.8**			
0.075	**453.0**	96.6	**3.4**			
PAN	**457.8**					
SIEVE CHECK FINE		0.13	0.3% max.			**REFERENCE MATERIAL**

OTHER TESTS	**RESULT**	**LAB NO.**	**RESULT**

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

APPENDIX IV

Report Limitations and Guidelines for Use

REPORT LIMITATIONS & GUIDELINES FOR USE

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

GEOTECHNICAL SERVICES ARE PERFORMED FOR SPECIFIC PURPOSES, PERSONS AND PROJECTS

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

SUBSURFACE CONDITIONS CAN CHANGE

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

LIMITATIONS TO PROFESSIONAL OPINIONS

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

LIMITATIONS OF RECOMMENDATIONS

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

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

MISINTERPRETATION OF GEOTECHNICAL ENGINEERING REPORT

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

CONTRACTORS RESPONSIBILITY FOR SITE SAFETY

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

SUBSURFACE SOIL AND/OR GROUNDWATER CONTAMINATION

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

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