REVISED FINAL

Preliminary Geotechnical Investigation - Proposed Residential Development

1157-1171 North Shore Boulevard East
Burlington, Ontario

Prepared for:

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November 8, 2018

Pinchin File: 212394.002

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Issued On:	**November 8, 2018**
Pinchin File:	**212394.002**
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1.0 INTRODUCTION

Pinchin Ltd. (Pinchin) was retained by Spruce Partners Inc. (Client) to conduct a Preliminary Geotechnical Investigation and provide subsequent geotechnical design recommendations for the proposed residential development to be located at 1157-1171 North Shore Boulevard East, Burlington, 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 seventeen-storey residential retirement building with two levels of underground parking. A Preliminary Geotechnical Investigation was completed to provide preliminary recommendations for design and construction of the buildings.

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

The purpose of the Preliminary Geotechnical Investigation was to delineate the subsurface conditions and soil engineering characteristics by advancing a total of nine (9) sampled boreholes (Boreholes BH01 to BH09) within the site limits. The information gathered from the Preliminary Geotechnical Investigation will allow Pinchin to provide geotechnical design recommendations for the proposed development.

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

- A review of relevant area geology and Site background information;
- A detailed description of the soil and groundwater conditions;
- Open cut excavations;
- Site service trench design;
- Lateral earth pressure coefficients and unit densities;
- Anticipated groundwater management;
- Foundation design recommendations including soil and bedrock 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,
- Asphaltic concrete pavement structure design for parking areas and access roadways.

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

2.0 SITE DESCTIPTION AND GEOLOGICAL SETTING

The Site is located on the west side of North Shore Boulevard East, at the northwest corner of the intersection of North Shore Boulevard East and the Queen Elizabeth Highway (QEW) in Burlington, Ontario. The Site is currently developed with two four-storey multi-tenant residential buildings and a single-storey parking/storage garage. The lands adjacent to the Site are developed with multi-tenant residential buildings and commercial 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 coarse-textured lacustrine deposits; sand and gravel, minor silt and clay. The underlying bedrock at this Site is of the Queenston formation consisting of shale, limestone, dolostone and siltstone (Ontario Geological Survey Map 1972, published 1978).

3.0 GEOTECHNICAL FIELD INVESTIGATION AND METHODOLOGY

Pinchin completed a field investigation at the Site on February 5 and 6, 2018 by advancing a total of nine sampled boreholes throughout the Site. The boreholes were advanced to depths ranging from approximately 1.7 to 7.8 metres below existing ground surface (mbgs). The approximate spatial locations of the boreholes advanced at the Site are shown on Figure 2.

The boreholes were advanced with the use of a Geoprobe 7822 DT direct push drill rig which was equipped with standard soil sampling equipment. Soil samples were collected at regular 1.52 metre intervals using 2.54 centimetre (cm) inner diameter (ID) direct push soil samplers with dedicated single-use sample liners for Boreholes BH01, BH02 and BH04. 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) in Boreholes BH03 and BH05 to BH09. The SPT “N” values were used to assess the compactness condition of the non-cohesive soil.

Groundwater observations and measurements were obtained from the open boreholes during and upon completion of drilling. The groundwater observations and measurements recorded are included on the appended borehole logs.

The boreholes locations and ground surface elevations were surveyed by Pinchin using a Sokkia Model GRX 2 Global Navigation Satellite System (GNSS) rover. The ground surface elevations are geodetic, based on GNSS and local base station telemetry with a precision static of less than 20 mm.

The field investigation was monitored by experienced Pinchin personnel. Pinchin logged the drilling operations and identified the soil samples 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 topsoil and fill overlying silt deposits to the maximum borehole refusal depth of approximately 7.8 mbgs.

Boreholes BH01 and BH02 were advanced through a concrete pad and the Portland cement concrete was 80 mm thick. The concrete pad was underlain by 150 mm of granular base material. Surficial fill and topsoil material was encountered surficially within the remaining boreholes and was observed to be approximately 300 to 760 millimetres (mm) thick with a thicker deposit of fill encountered in Borehole BH06 and was 2.2 m thick. The fill was generally comprised of dark brown sand and silt (topsoil). The material was generally frozen to moist at the time of sampling.

Silt deposits were encountered in all of the boreholes below the fill and the topsoil and extended to the borehole termination depths of 7.8 mbgs. The silt deposits generally comprised clayey silt with trace to some sand and trace gravel. The cohesive silt deposits had a very soft to hard consistency based on shear strengths measured with a hand held pocket penetrometer of 25 to greater than 250 kPa and on SPT ‘N’ values of 4 to greater than 50 blows per 300 mm penetration of a split spoon sampler. The results of two particle size distribution analyses performed on samples of silt indicate that the samples contain 1% gravel, 4 to 7% sand, 54% silt, and 38 to 41% clay.

Auger refusal due to probably bedrock was encountered in Boreholes BH03 and BH05 to BH09 at depths of 5.2 to 7.8 mbgs.

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. Groundwater was encountered at depths ranging from 4.9 to 7.0 mbgs (Elevation 72.9 to 75.5 masl) in the open boreholes at the completion of drilling. The groundwater is perched within sand seams in the relatively impermeable clayey silt. Monitoring wells are to be installed during the detailed design stage of the project to provide a more accurate indication of water levels at 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.

5.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

5.1 General Information

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

It is Pinchin's understanding that the proposed development is to consist of a seventeen-storey residential retirement building with two levels of underground parking.

5.2 Site Preparation

The existing fill and topsoil is not considered suitable to remain below the proposed building and will need to be removed. In calculating the approximate quantity of fill and topsoil to be stripped, we recommend that the fill and topsoil thickness provided on the individual borehole logs be increased by 50 mm to account for variations and some stripping of the mineral soil below. The existing inorganic fill may be used to raise grades below the proposed building or parking areas.

Prior to placing any fill material at the Site, the subgrade should be inspected by a qualified geotechnical engineer, and loosened/soft pockets should be sub excavated. Pinchin recommends that engineered fill be compacted in accordance with the criteria stated in the following table:

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

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

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

5.3 Open Cut Excavations and Anticipated Groundwater Management

It is anticipated that the invert elevations for the site services will be at conventional depths of approximately 2 to 4 metres below finished grade. The finished floor elevation of the second level of parking is approximately at Elevation 75.3 masl and therefore excavations for the construction of the building will extend 5 to 7 mbgs.

Based on the subsurface information obtained from within the boreholes, it is anticipated that the excavated material will predominately consist of silt deposits. Groundwater was encountered at depths ranging from 4.9 to 7.0 mbgs (Elevation 72.9 to 75.5 masl) in the open boreholes at the completion of drilling.

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).

Based on the OHSA, the existing material may be classified as Type 3 soil. 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. Excavations extending below the groundwater table will have to be sloped back at 3 H to 1V from the base of the excavation.

If the above noted excavation side slopes are not feasible for the development, Pinchin would be pleased to provide further recommendations for the design of shoring systems at the Site.

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.

Minor to moderate groundwater inflow through the silt is expected where the excavations extend less than 0.6 m below the groundwater table. It is believed that this groundwater inflow can be controlled using a gravity dewatering system with perimeter interceptor ditches and high capacity pumps. For excavations extending more than 0.6 m below the stabilized groundwater table, a dewatering system installed by a specialist dewatering contractor may be required to lower the groundwater level prior to excavation. The design of the dewatering system should be left to the contractor’s discretion, and the system should meet a performance specification to maintain and control the groundwater at least 0.3 m below the excavation base. Additional fieldwork should be completed at the Site to confirm this recommendation.

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

5.4 Site Servicing

5.4.1 Pipe Bedding and Cover Materials for Flexible and Rigid Pipes

The subgrade soil conditions beneath the Site services will comprise natural silt deposits. No support problems are anticipated for flexible or rigid pipes founded in the natural mineral soils. 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.

5.4.2 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 200 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.

The excavated clayey silt soils will have a blocky/lumpy texture. If the large interlump voids are not closed completely by thorough compaction, then long-term softening/settlement will occur. The trench backfill should be placed in thin lifts and compacted with a sheepsfoot roller. Particular attention must be made to backfilling service connections where the trenches are narrow. If work is carried out during very dry weather, then water could be added to the backfill to improve compaction.

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. This will be the case for soil excavated below the groundwater table. 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.

Depending on weather conditions at the time of construction, an imported material may be required 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.

5.4.3 Frost Protection

The frost penetration depth in Burlington, Ontario for these types of soil conditions is estimated to extend to approximately 1.5 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 1.8 mbgs or lower as dictated by municipal service requirements. If a minimum of 1.8 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.

5.5 Foundation Design

5.5.1 Bearing Resistances and Foundation Preparation

It is anticipated that foundations for any slab-on-grade building will be founded at conventional frost depths some 0.5 and 1.5 m below the floor level for interior and exterior foundations, respectively. Provided the existing fill has been removed and the subgrade prepared at described above foundations for the building may be founded on natural silt at the above noted depths. The finished floor elevation of the second level of parking is at Elevation 75.3 masl and therefore excavations for the construction of the building will extend 5 to 7 mbgs.

Conventional shallow strip footings established on properly placed natural silt soil or properly placed engineered fill may be designed using a bearing resistance for 25 mm of settlement at Serviceability Limit States (SLS) of 150 kPa, and a factored geotechnical bearing resistance of 225 kPa at Ultimate Limit States (ULS).

Conventional shallow strip footings established below Elevation 75 masl may be designed using a bearing resistance for 25 mm of settlement at Serviceability Limit States (SLS) of 300 kPa, and a factored geotechnical bearing resistance of 450 kPa at Ultimate Limit States (ULS).

As the actual foundation designs are unknown, Pinchin should be consulted to confirm the design bearing resistances provided are suitable for at the design footing elevations.

Additionally, as the actual service loads 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.

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

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

The natural subgrade soil is sensitive to change in moisture content and can become loose/soft if subjected to additional water or precipitation. As well, it could be easily disturbed if travelled on during construction. Once it becomes disturbed it is no longer considered adequate to support the recommended design bearing pressures.

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 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, caved materials or loosened bedrock pieces;

- Any potential large cobbles or boulders (i.e. greater than 200 mm in diameter) within the subgrade material are to be removed and replaced with a similar soil type not containing particles greater than 200 mm in diameter. It is critical that particles greater than 200 mm in diameter are not in contact with the foundation to prevent point loading and overstressing; and
- If the excavated subgrade soil remains open to weather conditions and groundwater seepage, sidewall stability and suitability of the subgrade soil will need to be verified prior to construction.

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

5.5.2 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 between approximately 1.7 to 7.8 mbgs and were terminated in the silt deposit. SPT "N" values within the silt deposit ranged between 4 and greater than 50 blows per 300 mm. As such, based on Table 4.1.8.4.A of the OBC, this Site has been classified as Class D. A Site Class D has an average shear wave velocity (Vs) of between 180 and 360 m/s. A higher Site Classification may be available for deeper foundations at the Site and Pinchin should review this recommendation once the final foundation elevations have been determined.

5.5.3 Foundation Transition Zones

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

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

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

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

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

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

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

5.5.4 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.

5.5.5 Building Drainage

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

Exterior perimeter foundations drains are not required, where the finished floor elevation is established a minimum of 150 mm above the exterior final grades or that the exterior gradient is properly sloped to divert surface water away from the building. As the final design of the building has not been completed, the quantity of water that might be expected from the perimeter foundation drains is unknown. Pinchin will be willing to calculate this once the detailed design is complete.

5.5.6 Shallow Foundations Frost Protection & Foundation Backfill

In the Burlington, Ontario area, exterior perimeter foundations for heated buildings require a minimum of 1.2 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 1.2 m of soil cover frost protection, they should be protected from frost with a combination of soil cover and rigid polystyrene insulation, such as Dow Styrofoam or equivalent product. If required, Pinchin can provide appropriate foundation frost protection recommendations as part of the design review.

To minimize potential frost movements from soil frost adhesion, the perimeter foundation backfill should consist of a free draining granular material, such as a Granular ‘B’ Type I (OPSS 1010) or an approved sand fill, extending a minimum lateral distance of 600 mm beyond the foundation. The existing silt material is too wet for reuse and are not considered suitable for reuse as foundation wall backfill. The backfill material used against the foundation must be placed so that the allowable lateral capacity is achieved. All granular material is to be placed in maximum 300 mm thick lifts compacted to a minimum of 100% SPMDD in hard landscaping areas and 95% SPMDD in soft landscaping areas. It is recommended that inspection and testing be carried out during construction to confirm backfill quality, thickness and to ensure compaction requirements are achieved.

5.6 Underground Parking Garage Design

It is understood that the buildings will be constructed with two levels of underground parking and that the finished floor elevation of the second level of parking is approximately at Elevation 75.3 masl. Groundwater was encountered in all the boreholes at depths ranging from 4.9 to 7.0 mbgs (Elevation 72.9 to 75.5 masl). Monitoring wells are to be installed during the detailed design stage of the project to provide a more accurate indication of water levels at the Site.

Since the groundwater is close to the finished floor elevation of the lowest level of underground parking, there are two options for managing the groundwater near the building. The building can either be designed to resist hydrostatic uplift or the building can be provided with underfloor and foundation wall drainage systems connected to a suitable frost free outlet. Additional information for both options are summarized below. It should be noted that as the final design of the building has not been completed, the quantity of water that might be expected from the perimeter foundation drains is unknown. Pinchin will be willing to calculate this once the detailed design is complete.

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

If the proposed basement floor level is constructed close to 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 subdrains and pumps are used to remove the groundwater from around the building footprint, there is the potential that a Permit to Take Water from the Ministry of Environment and Climate Change 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.

5.7 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 non-vibratory steel drum roller to observe for weak/soft spots. It is noted that some locations will not be accessible by the steel drum roller; as such, these locations can be proof roll compacted with a minimum 450 kg vibratory plate compactor.

The in-situ silt 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
Silt	35,000

5.8 Asphaltic Concrete Pavement Structure Design for Parking Lot and Driveways

5.8.1 Discussion

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

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

5.8.2 Pavement Structure

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

Pavement Layer	Compaction Requirements	Parking Areas	Driveways
Surface Course Asphaltic Concrete HL-4 (OPSS 1150)	92% MRD as per OPSS 310	35 mm	35 mm
Binder Course Asphaltic Concrete HL-8 (OPSS 1150)	92 % MRD as per OPSS 310	55 mm	85 mm
Base Course: Granular "A" (OPSS 1010)	100% Standard Proctor Maximum Dry Density (ASTM-D698)	150 mm	150 mm
Subbase Course: Granular "B" Type I (OPSS 1010)	100% Standard Proctor Maximum Dry Density (ASTM D698)	350 mm	450 mm

Notes:

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

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

Performance grade PG 58-28 asphaltic concrete should be specified for Marshall mixes.

5.8.3 Pavement Structure Subgrade Preparation and Granular up Fill

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

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

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

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

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

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

5.8.4 Drainage

Control of surface water is a critical factor in achieving good pavement structure life. The pavement thickness designs are based on a drained pavement subgrade via sub-drains or ditches. The silt soils have poor natural drainage and therefore it is recommended that pavement subdrains be installed in the lower areas and be connected to the catchbasins.

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

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

6.0 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 Spruce Partners Inc. (Client) in order to evaluate the subsurface conditions at 1157-1171 North Shore Boulevard East, Burlington, 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.

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Template: Master Report for Phase II ESA - Stage 2 PSI, EDR, February 2, 2018

FIGURES

PINCHIN

PROJECT NAME
PRELIMINARY GEOTECHNICAL INVESTIGATION

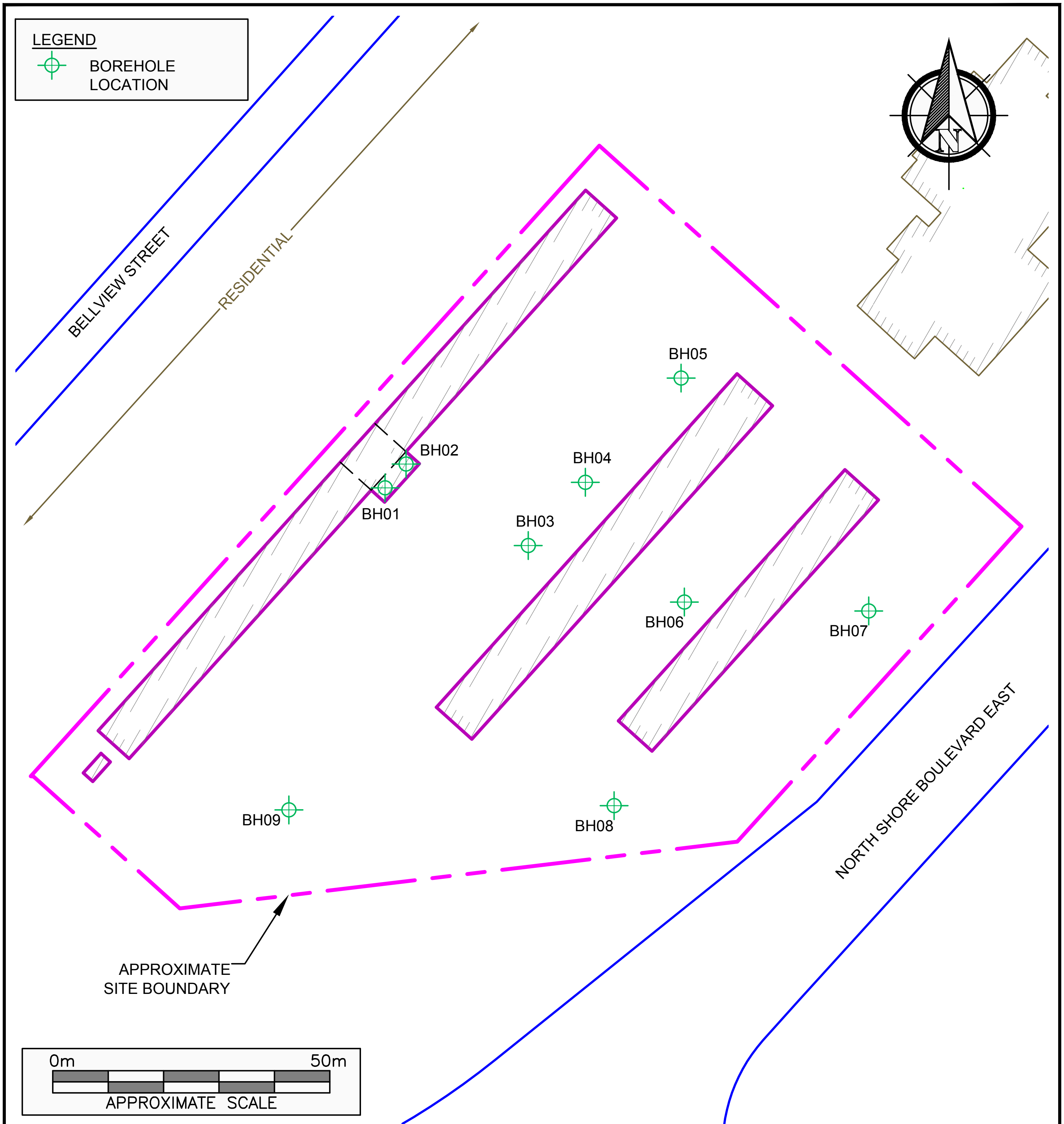
CLIENT NAME
SPRUCE PARTNERS INC.

PROJECT LOCATION
1157-1171 NORTH SHORE BOULEVARD EAST, BURLINGTON, ONTARIO

FIGURE NAME
KEY MAP

FIGURE NO.
1

APPROXIMATE SCALE	PROJECT NO.	DATE
AS SHOWN	212394.002	MARCH 2018



PINCHIN

PROJECT NAME: PRELIMINARY GEOTECHNICAL INVESTIGATION			
CLIENT NAME: SPRUCE PARTNERS INC.			
PROJECT LOCATION: 1157-1171 NORTH SHORE BOULEVARD EAST, BURLINGTON, ONTARIO			
FIGURE NAME: BOREHOLE LOCATION PLAN			FIGURE NO. 2
APPROXIMATE SCALE: AS SHOWN	PROJECT NO.: 212394.002	DATE: MARCH 2018	

APPENDIX I

Abbreviations, Terminology and Principal 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: BH01

Project #: 212394.002 **Logged By:** JL

Project: Geotechnical Investigation

Client: Spruce Partners Inc.

Location: 1157-1171 North Shore Boulevard E., Burlington, ON

Drill Date: February 5, 2018 **Project Manager:** VM

SUBSURFACE PROFILE				SAMPLE					
Depth	Symbol	Description	Measured Depth (m)	Monitoring Well Details	Sampler #	Recovery (%)	Sample ID	Soil Vapour Concentration CGI/PID	Laboratory Analysis
ft / m: 0 / 0		Ground Surface	0.00	No Monitoring Well Installed					
		Concrete	0.08						
		Fill Gravel.	0.15						
1–5 ft / 1 m		**Silt** Brown clayey silt, some sand, wet.	1.68		1	100	S1	<5/0	
							S2	<5/0	
6–10 ft / 2–3 m		End of Borehole Borehole terminated at 1.68 mbgs due to refusal. Soil vapour concentrations were measured using a combustible gas indicator (CGI) and a photo-ionization detector (PID). Soil vapour data presented in parts per million (ppm).							

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

Contractor: Strata Drilling Group Inc.

Drilling Method: Direct Push

Well Casing Size: NA

Pinchin Ltd.
283 Northfield Drive East
Waterloo, ON N2J 4G8

Grade Elevation: 0

Top of Casing Elevation: NA

Sheet: 1 of 1

Log of Borehole: BH02

Project #: 212394.002 **Logged By:** JL

Project: Geotechnical Investigation

Client: Spruce Partners Inc.

Location: 1157-1171 North Shore Boulevard E., Burlington, ON

Drill Date: February 5, 2018 **Project Manager:** VM

SUBSURFACE PROFILE / SAMPLE

Depth (ft / m)	Symbol	Description	Measured Depth (m)	Monitoring Well Details	Sampler #	Recovery (%)	Sample ID	Soil Vapour Concentration CGI/PID	Laboratory Analysis
0 / 0		Ground Surface							
		Concrete	0.08	No Monitoring Well Installed					
		Fill Gravel.	0.15						
1–5 ft / 1 m		**Silt** Brown clayey silt, some sand, wet.	1.68		1	70	S1	<5/0	
							S2	<5/0	
		End of Borehole							

Borehole terminated at 1.68 mbgs due to refusal.

Soil vapour concentrations were measured using a combustible gas indicator (CGI) and a photo-ionization detector (PID). Soil vapour data presented in parts per million (ppm).

Depth scale: ft 0–10; m 0–3.

Contractor: Strata Drilling Group Inc.

Drilling Method: Direct Push

Well Casing Size: NA

Pinchin Ltd.
283 Northfield Drive East
Waterloo, ON N2J 4G8

Grade Elevation: 0

Top of Casing Elevation: NA

Sheet: 1 of 1

Log of Borehole: BH03

Project #: 212394.002 **Logged By:** JL

Project: Geotechnical Investigation

Client: Spruce Partners Inc.

Location: 1157-1171 North Shore Boulevard E., Burlington, ON

Drill Date: February 5, 2018 **Project Manager:** VM

SUBSURFACE PROFILE				SAMPLE								
Depth (m)	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-values	Dynamic Penetration Resistance 20 40 60	Shear Stress kPa 75 150	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
0	Ground Surface	81.40	No Monitoring Well Installed									
	Topsoil Dark brown sand and silt (topsoil), some organics, frozen to moist.	80.64		SS	1	60	5			SS1	<5/0	
1	**Silt** Dark brown clayey silt, some sand, damp.	79.88		SS	2	30	4			SS2	<5/0	
2	Brown, some gravel.	78.05		SS	3	90	6			SS3	<5/0	
				SS	4	90	5			SS4	<5/0	
3	Reddish brown, damp.			SS	5	90	70			SS5	<5/0	
5				SS	6	15	>50			SS6	<5/0	
6		75.00		SS	7	0	>50			SS7	<5/0	
	End of Borehole											

Depth scale: 0, 1, 2, 3, 4, 5, 6, 7, 8

End of Borehole

Borehole terminated at 6.4 mbgs due to auger refusal. At drilling completion, a wet cave was measured at 5.94 mbgs and water was measured at 5.79 mbgs.

Soil vapour concentrations were measured using a combustible gas indicator (CGI) and a photo-ionization detector (PID). Soil vapour data presented in parts per million (ppm).

Contractor: Strata Drilling Group Inc.

Drilling Method: Split Spoon

Well Casing Size: NA

Pinchin Ltd.
283 Northfield Drive, Unit 9
Waterloo, ON N2J 4G8

Grade Elevation: 81.40

Top of Casing Elevation: NA

Sheet: 1 of 1

Log of Borehole: BH04

Project #: 212394.002 **Logged By:** JL

Project: Geotechnical Investigation

Client: Spruce Partners Inc.

Location: 1157-1171 North Shore Boulevard E., Burlington, ON

Drill Date: February 5, 2018 **Project Manager:** VM

SUBSURFACE PROFILE					SAMPLE				
Depth (ft / m)	Symbol	Description	Measured Depth (m)	Monitoring Well Details	Sampler #	Recovery (%)	Sample ID	Soil Vapour Concentration CGI/PID	Laboratory Analysis
0 ft / 0 m		Ground Surface	0.00	No Monitoring Well Installed					
0–1 ft		**Topsoil** Dark brown sand and silt, some organics, frozen to moist.	0.30		1	40	S1	<5/0	
1–10 ft / 0.30–3.05 m		**Silt** Dark brown clayey silt, some sand, damp.			1	40	S2	<5/0	
					2	10	S3	<5/0	
			3.05		2	10	S4	<5/0	
10 ft / 3 m		**End of Borehole** Borehole terminated at 3.05 mbgs. Soil vapour concentrations were measured using a combustible gas indicator (CGI) and a photo-ionization detector (PID). Soil vapour data presented in parts per million (ppm).							

Depth scale: 0–15 ft; 0–4 m.

Contractor: Strata Drilling Group Inc.

Drilling Method: Direct Push

Well Casing Size: NA

Pinchin Ltd.
283 Northfield Drive East
Waterloo, ON N2J 4G8

Grade Elevation: 82.23

Top of Casing Elevation: NA

Sheet: 1 of 1

Log of Borehole: BH05

PINCHIN

Project #: 212394.002 **Logged By:** JL

Project: Geotechnical Investigation

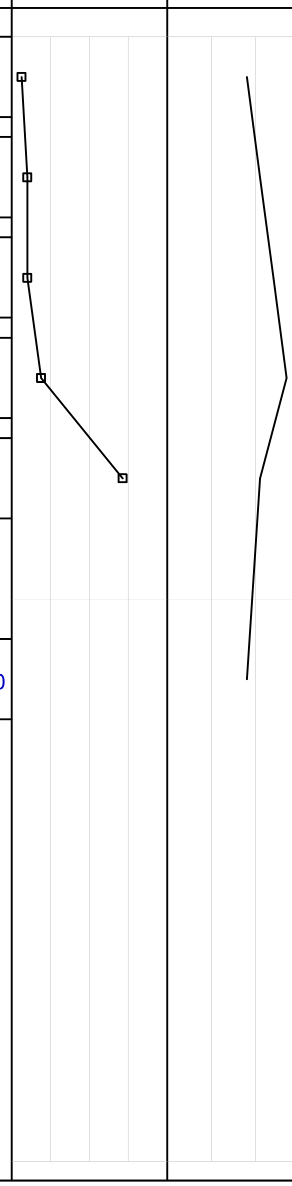
Client: Spruce Partners Inc.

Location: 1157-1171 North Shore Boulevard E., Burlington, ON

Drill Date: February 5, 2018 **Project Manager:** VM

SUBSURFACE PROFILE / SAMPLE

Depth (m)	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-values	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
0	Ground Surface	81.53	No Monitoring Well Installed							
	Topsoil Dark brown sand and silt, some organics, frozen to moist.	80.77		SS	1	90	5	SS1	<5/0	
1	**Silt** Dark brown clayey silt, some sand, damp.	80.01		SS	2	50	8	SS2	<5/0	
2	Brown.	78.18		SS	3	100	8	SS3	<5/0	
				SS	4	100	15	SS4	<5/0	
3				SS	5	70	57	SS5	<5/0	
4	Reddish brown, some gravel, damp.	76.96								
5	Some shale, damp.	75.59		SS	6	20	>50	SS6	<5/0	
6	End of Borehole									
7										
8										



End of Borehole

Borehole terminated at 5.94 mbgs due to auger refusal. At drilling completion, a wet cave was measured at 5.64 mbgs and water was measured at 5.45 mbgs.

Soil vapour concentrations were measured using a combustible gas indicator (CGI) and a photo-ionization detector (PID). Soil vapour data presented in parts per million (ppm).

Contractor: Strata Drilling Group Inc.

Drilling Method: Split Spoon

Well Casing Size: NA

Pinchin Ltd.
283 Northfield Drive, Unit 9
Waterloo, ON N2J 4G8

Grade Elevation: 81.53

Top of Casing Elevation: NA

Log of Borehole: BH06

Project #: 212394.002 **Logged By:** JL

Project: Geotechnical Investigation

Client: Spruce Partners Inc.

Location: 1157-1171 North Shore Boulevard E., Burlington, ON

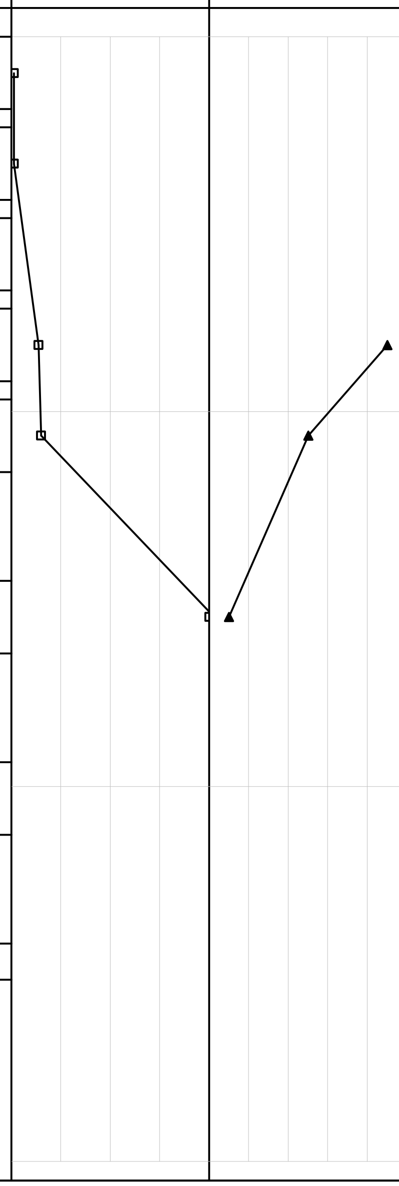
Drill Date: February 6, 2018 **Project Manager:** VM

SUBSURFACE PROFILE / SAMPLE

Depth (m)	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-values
0	Ground Surface	79.91	No Monitoring Well Installed				
0–0.76	***Topsoil*** Dark brown sand and silt, some organics, frozen to moist.	79.15		SS	1	60	1
0.76–1.83	***Fill*** Dark brown clayey silt, some sand, damp.	78.08		SS	2	60	1
1.83–2.29	Black amorphous peat seam.	77.62		SS	3	50	0
2.29–3.51	***Silt*** Dark grey clayey silt, some sand, damp.	76.40		SS	4	50	11
				SS	5	40	12
3.51–4.57	Reddish brown, some gravel, damp.	75.34					
4.57–7.77	Some shale.	72.14		SS	6	50	82
				SS	7	20	>50
				SS	8	10	>50
7.77	End of Borehole						

Borehole terminated at 7.77 mbgs due to sample refusal. At drilling completion, wet cave measured at 7.32 mbgs and water measured at 7.01 mbgs.

Dynamic Penetration Resistance (20, 40, 60) / Shear Strength (kPa) (kPa: 50, 100, 150, 200)



Water Content (%) | Laboratory Analysis

Contractor: Strata Drilling Group Inc.

Drilling Method: Split Spoon

Well Casing Size: NA

Pinchin Ltd.
283 Northfield Drive E., Unit 9
Waterloo, Ontario N2J 4G8

Grade Elevation: 79.91

Top of Casing Elevation: NA

Sheet: 1 of 1

Log of Borehole: BH07

Project #: 212394.002 **Logged By:** JL

Project: Geotechnical Investigation

Client: Spruce Partners Inc.

Location: 1157-1171 North Shore Boulevard E., Burlington, ON

Drill Date: February 6, 2018 **Project Manager:** VM

SUBSURFACE PROFILE / SAMPLE

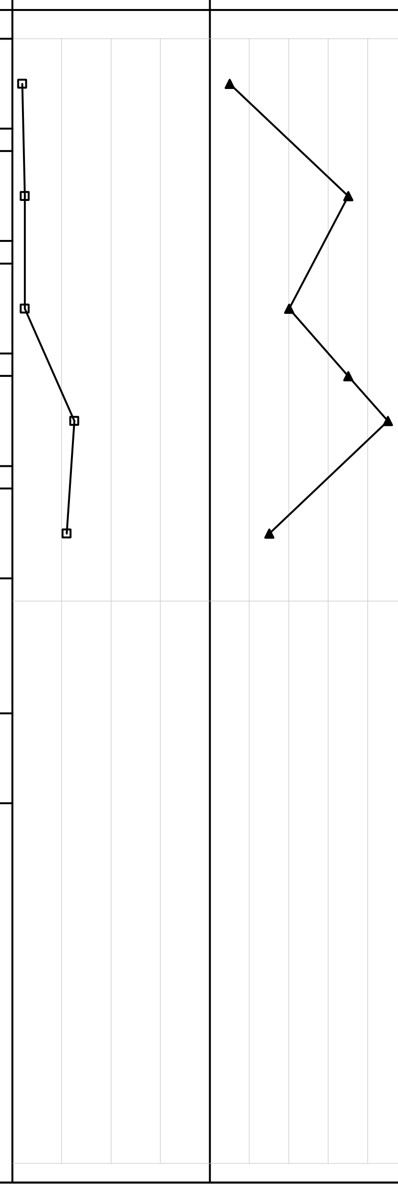
Depth (m)	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-values
	Ground Surface	80.36	No Monitoring Well Installed				
0	**Topsoil** Dark brown sand and silt, some organics, frozen to moist.	79.60		SS	1	60	4
1	**Silt** Brown clayey silt, some sand, damp.			SS	2	70	5
2		77.77		SS	3	60	5
				SS	4	80	25
3	Reddish brown, some gravel, damp.	77.01		SS	5	70	22
4	large gravel chunk.	75.79					
5	Some shale.	75.18		SS	6	20	>50
6	End of Borehole Borehole terminated at 5.18 mbgs due to auger refusal. At drilling completion, wet cave measured at 4.88 mbgs and water measured at 4.85 mbgs.						
7							

Dynamic Penetration Resistance: 20 40 60

Shear Strength (kPa): kPa 50 100 150 200

Water Content (%)

Laboratory Analysis



Contractor: Strata Drilling Group Inc.

Drilling Method: Split Spoon

Well Casing Size: NA

Pinchin Ltd.
283 Northfield Drive E., Unit 9
Waterloo, Ontario N2J 4G8

Grade Elevation: 80.36

Top of Casing Elevation: NA

Sheet: 1 of 1

Log of Borehole: BH08

Project #: 212394.002 **Logged By:** JL

Project: Geotechnical Investigation

Client: Spruce Partners Inc.

Location: 1157-1171 North Shore Boulevard E., Burlington, ON

Drill Date: February 6, 2018 **Project Manager:** VM

SUBSURFACE PROFILE				SAMPLE							
Depth (m)	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-values	Dynamic Penetration Resistance 20 40 60	Shear Strength (kPa) kPa 50 100 150 200	Water Content (%)	Laboratory Analysis
0	Ground Surface	80.45	No Monitoring Well Installed								
0 – 0.76	**Topsoil** Dark brown sand and silt, some organics, frozen to moist.	79.69		SS	1	60	2				
0.76 – 1.52	**Silt** Dark brown clayey silt, some sand, wet.	78.93		SS	2	70	10				
1.52 – 3.66	Dark grey, moist..	76.79		SS	3	70	6				
				SS	4	90	16				
				SS	5	80	21				
3.66 – 4.57	Reddish brown, damp.	75.88									
4.57 – 5.94	Some shale.	74.51		SS	6	20	>50				
				SS	7	0	>50				
5.94	End of Borehole. Borehole terminated at 5.94 mbgs due to auger refusal. At drilling completion, dry cave measured at 5.18 mbgs.										

Contractor: Strata Drilling Group Inc.

Drilling Method: Split Spoon

Well Casing Size: NA

Pinchin Ltd.
283 Northfield Drive E., Unit 9
Waterloo, Ontario N2J 4G8

Grade Elevation: 80.45

Top of Casing Elevation: NA

Sheet: 1 of 1

Log of Borehole: BH09

Project #: 212394.002 **Logged By:** JL

Project: Geotechnical Investigation

Client: Spruce Partners Inc.

Location: 1157-1171 North Shore Boulevard E., Burlington, ON

Drill Date: February 6, 2018 **Project Manager:** VM

SUBSURFACE PROFILE

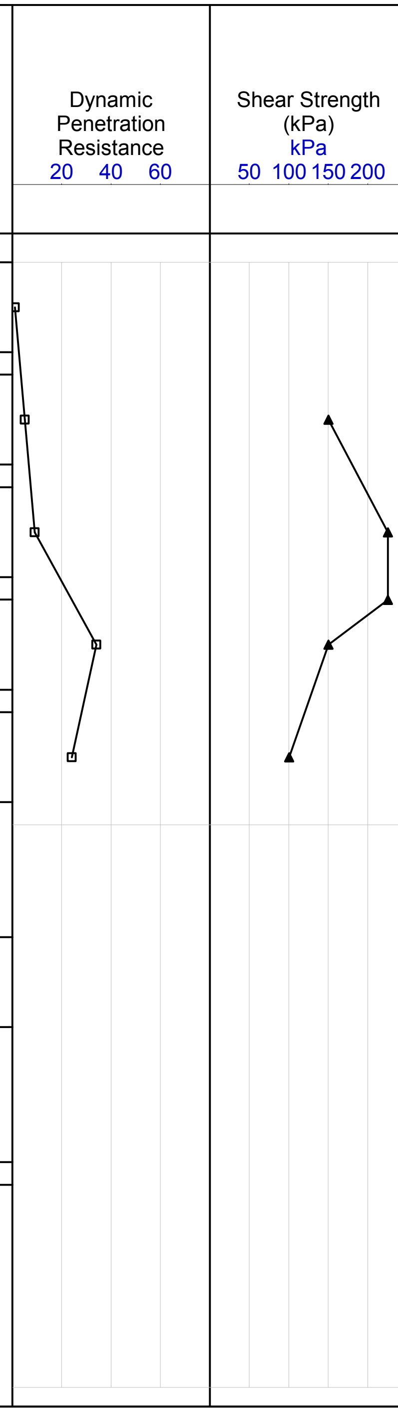
Depth (m)	Description	Elevation (m)
0	Ground Surface	80.76
0–0.76	**Topsoil** Dark brown sand and silt, some organics, frozen to moist.	80.00
0.76–2.29	**Silt** Dark brown clayey silt, some sand, damp.	78.47
2.29–2.59	Moist.	78.17
2.59–6.25	Reddish brown, some gravel, damp.	74.51
6.25	End of Borehole	

Borehole terminated at 6.25 mbgs due to auger refusal. At drilling completion, dry cave measured at 5.49 mbgs.

Monitoring Well Details: No Monitoring Well Installed

SAMPLE

Sample Type	Sampler #	Recovery (%)	SPT N-values	Water Content (%)	Laboratory Analysis
SS	1	40	1		
SS	2	90	5		
SS	3	50	9		
SS	4	80	34		
SS	5	70	24		
SS	6	0	>50		
SS	7	0	>50		



Contractor: Strata Drilling Group Inc.

Drilling Method: Split Spoon

Well Casing Size: NA

Pinchin Ltd.
283 Northfield Drive E., Unit 9
Waterloo, Ontario N2J 4G8

Grade Elevation: 80.76

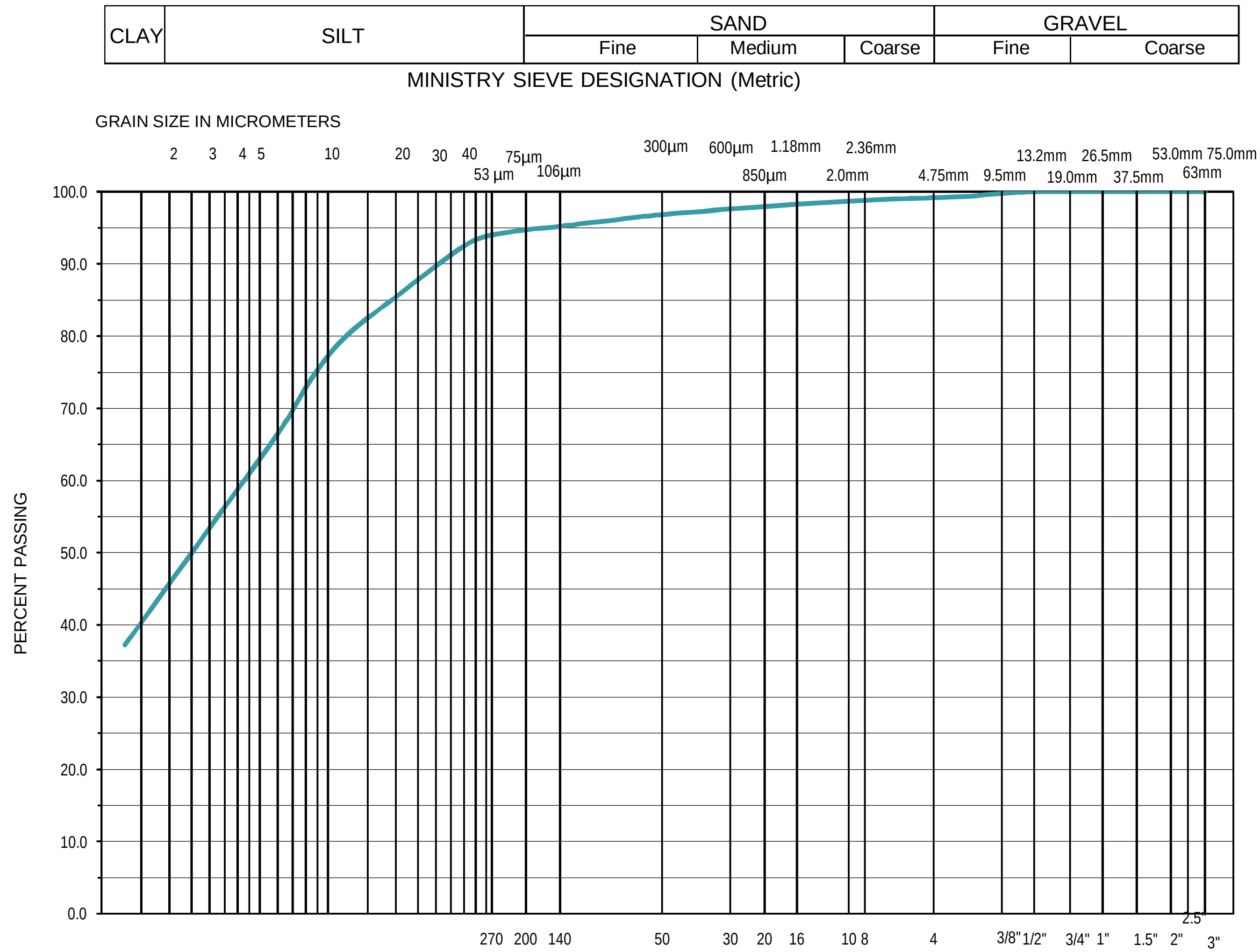
Top of Casing Elevation: NA

Sheet: 1 of 1

APPENDIX III

Analytical Laboratory Testing Reports for Soil Samples

UNIFIED SOIL CLASSIFICATION SYSTEM - GRAINSIZE DISTRIBUTION



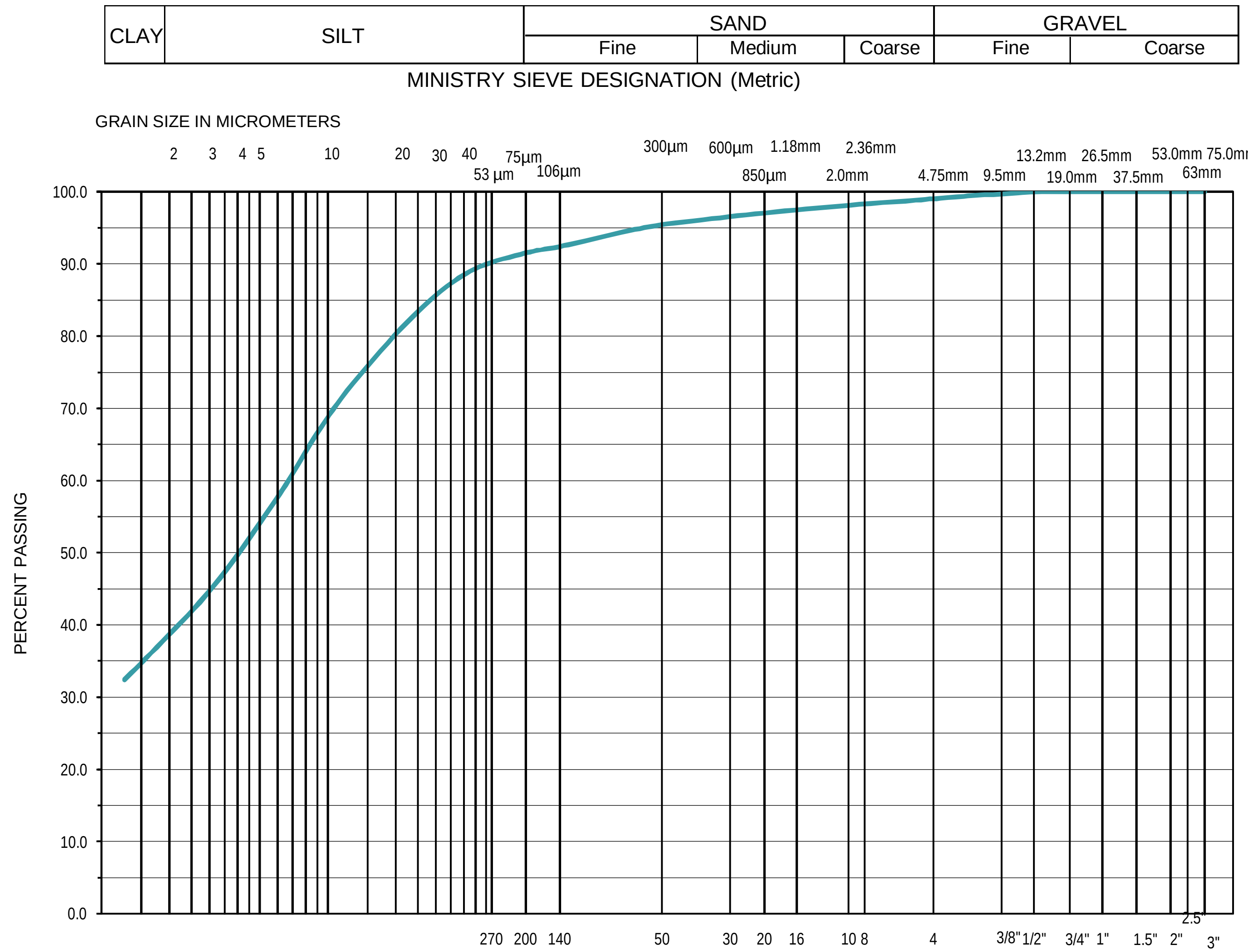
Particle Size	Percent Passing
75	100.0
63	100.0
37.5	100.0
26.5	100.0
19	100.0
16	100.0
13.2	100.0
9.5	99.8
6.7	99.4
4.75	99.2
2	98.7
0.85	97.96
0.425	97.24
0.25	96.59
0.106	95.21
0.075	94.73
0.0468	93.58
0.0331	90.72
0.0210	85.94
0.0122	80.21
0.0086	74.48
0.0061	66.84
0.0030	53.48
0.0013	37.24

Lab No.: 18060

Client: Pinchin Ltd.	**Soil Class:** CH / **Moisture:** 20.5%	**Date Sampled:** Unknown - sampled by client	**Enclosure:** 1
Project: Pinchin Project No. 212394.002	**Location:** BH-05, SS#3, 5 - 7' depth	**Date Received:** 13 February 2018	
Project No.: SWC178279	**Depth (m):** 1.5 - 2.1	**Report Date:** 20 February 2018	

Amec Foster Wheeler Environment & Infrastructure
900 Maple Grove Road, Unit 10
Cambridge, ON N3H 4R7
Tel: (519) 650-7100
amecfw.com

UNIFIED SOIL CLASSIFICATION SYSTEM - GRAINSIZE DISTRIBUTION



Particle Size	Percent Passing
75	100.0
63	100.0
37.5	100.0
26.5	100.0
19	100.0
16	100.0
13.2	100.0
9.5	99.7
6.7	99.4
4.75	99.0
2	98.1
0.85	97.08
0.425	96.03
0.25	95.04
0.106	92.42
0.075	91.55
0.0469	89.61
0.0332	86.75
0.0210	81.03
0.0122	72.45
0.0086	65.78
0.0061	58.15
0.0030	44.81
0.0013	32.41

Lab No.: 18061

Client: Pinchin Ltd.	**Soil Class:** CL / **Moisture:** 16.5%	**Date Sampled:** Unknown - sampled by client	**Enclosure:** 2
Project: Pinchin Project No. 212394.002	**Location:** BH-08, SS#4, 7.5 - 9.5' depth	**Date Received:** 13 February 2018	
Project No.: SWC178279	**Depth (m):** 2.3 - 2.9	**Report Date:** 20 February 2018	

Amec Foster Wheeler Environment & Infrastructure
900 Maple Grove Road, Unit 10
Cambridge, ON N3H 4R7
Tel: (519) 650-7100
amecfw.com

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.