FINAL

Preliminary Geotechnical Investigation – Proposed Residential Development

1785 Bloor Street, Mississauga, Ontario

Prepared for:

1785 Bloor Holdings Inc.
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1.0 INTRODUCTION AND SCOPE

Pinchin Ltd. (Pinchin) was retained by 1785 Bloor Holdings Inc. (Client) to conduct a Preliminary Geotechnical Investigation and provide subsequent preliminary geotechnical design recommendations for the proposed development to be located at 1785 Bloor Street, Mississauga, Ontario (Site). The Site location is shown on Figure 1.

Based on information provided by the Client in an email dated April 14, 2021, it is Pinchin's understanding that the development will consist of a fourteen-storey apartment building with up to three levels of underground parking on the northwest portion of the property. The purpose of this preliminary geotechnical investigation is to cover the geotechnical submission requirements for Official Plan Amendment and Zoning By-Lay Amendment (ZBA) applications.

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 four (4) sampled boreholes (Boreholes BH1 to BH4), at the Site. The information gathered from the Preliminary Geotechnical Investigation will allow Pinchin to provide preliminary geotechnical design recommendations for the proposed development. As the design progresses, these preliminary results should be supplemented with a more detailed geotechnical field investigation and the design recommendations below should be revised based on the updated information.

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

- A detailed description of the soil and groundwater conditions;
- Site preparation recommendations;
- Open cut excavations;
- Anticipated groundwater management;
- Preliminary foundation design recommendations including bedrock bearing resistances at Ultimate Limit States (ULS) and Serviceability Limit States (SLS) design;
- Potential total and differential settlements;
- Foundation frost protection and engineered fill specifications and installation;
- Seismic Site classification for seismic Site response;
- Concrete floor slab-on-grade support recommendations; and
- Potential construction concerns.

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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 Bloor Street between the intersection of Bloor Street and Bridgewood Drive and Bloor Street and Fieldgate Drive in Mississauga, Ontario. The Site is currently developed with a ten-storey residential apartment building on the southeast side of the Site. Based on the provided architectural design drawing the development will be located to the northwest of the existing building.

Data obtained from the Ontario Geological Survey Maps, as published by the Ontario Ministry of Natural Resources, indicates that the Site is located on Paleozoic bedrock and young tills, Halton till comprising clayey silt till. The underlying bedrock at this Site is of the Georgian Bay Formation, consisting of shale, interbedded siltstone, and minor limestone (Ontario Geological Survey Preliminary Map 2204, published 1980).

3.0 GEOTECHNICAL FIELD INVESTIGATION AND METHODOLOGY

Pinchin completed field investigations at the Site between October 27 and October 29, 2021 by advancing a total of four (4) sampled boreholes throughout the Site. The boreholes were advanced to depths of approximately 12.2 to 12.3 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 3230 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. Approximate shear strengths of the cohesive deposits were measured using a handheld pocket penetrometer and the results are presented on the appended borehole logs.

Bedrock was proven in all boreholes by core drilling with an NQ-size double tube diamond bit core barrel. The bedrock core specimens were measured in the field to determine the Rock Quality Designation (RQD) (ASTM 6032). The core samples obtained from Borehole BH4 were returned to our offices for further visual examination and testing.

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

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

Groundwater observations and measurements were obtained from the open boreholes during and upon completion of drilling. Groundwater levels were measured in the monitoring wells on November 17 and 25, 2021. The groundwater observations and measurements recorded are included on the appended borehole logs.

The borehole locations and ground surface elevations were surveyed by Pinchin using a Sokkia Model GCX 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. Also, a bedrock sample that was obtained from Borehole BH4 was tested to determine the Uniaxial Compression Strength (USC). 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.

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4.0 SUBSURFACE CONDITIONS

4.1 Borehole Soil Stratigraphy

In general, the soil stratigraphy at the Site comprises pavement structure and/or topsoil underlain by fill material, which is underlain by sand and silt followed by bedrock to the maximum borehole termination depths of approximately 12.3 mbgs.

The pavement structure was encountered surficially in Boreholes BH1 and BH4 at the Site and was observed to comprise 125 mm of asphaltic concrete overlying granular fill material.

The topsoil was encountered surficially in Boreholes BH2 and BH3 and was observed to extend to approximately 300 mm below the ground surface.

The fill material was encountered in all boreholes below the asphaltic concrete and topsoil and extends to depths ranging between 0.8 and 1.5 mbgs. The fill material ranged from sand and gravel to silt with trace of rootlets. The fill material had a very loose to compact relative density based on SPT 'N' values of 3 to 13 blows per 300 mm penetration of a split spoon sampler.

A deposit of sand was encountered in all boreholes underlain the fill material and extends to depths ranging between 2.7 and 4.5 mbgs (Elevation 128.4 to 129.4 masl). The sand deposit generally comprised brown fine sand with trace to some gravel and trace silt and clay. The sand deposit had a compact to very dense consistency based on SPT 'N' values of 14 to above 50 blows per 300 mm penetration of a split spoon sampler. The sand deposit extended to the bedrock surface in Boreholes BH2 and BH4.

Sandy silt was encountered underlying the sand deposits in Boreholes BH1 and BH3 and extended to the bedrock surface. The sandy silt deposit consisted of sandy silt with trace to some gravel and clay. The sandy silt deposit had a dense to very dense relative density based on SPT 'N' values of 35 to greater than 50 blows per 300 mm penetration of a split spoon sampler. The results of one particle size distribution analysis completed on a sample of the glacial till are provided in Appendix III and indicate that the sample contains 14% gravel, 33% sand, 39% silt, and 14% clay.

4.2 Bedrock

Bedrock was encountered in all boreholes in depth ranging from 3.2 to 6.2 mbgs (Elevation 129.1 to 126.0 masl).

The bedrock was proven by coring in all boreholes and the Rock Quality Designation (RQD) was calculated for the recovered core samples and is summarized on the appended borehole logs. The upper 2 to 4 metres of the bedrock was highly weathered. The calculated RQD values show that the bedrock classification based on the RQD is in the range of very poor to poor quality.

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One sample of the recovered bedrock core from Borehole BH4 was tested to determine the Uniaxial Compression Strength (USC). The test showed that the USC of the test specimen is 41.2 MPa. A copy of the laboratory analytical reports and the photography of the rock specimen are included in Appendix III.

4.3 Groundwater Conditions

Groundwater observations are summarized on the appended borehole logs. The groundwater elevations were measured on November 17 and 25, 2021 and summarized as bellow:

Borehole/Monitoring Well	Groundwater depth (mbgs)	Groundwater elevation (masl)
BH1	2.5	129.62
BH2	1.4	132.26
BH3	2.5	129.43
BH4	3.1	128.45

For geotechnical design purposes, the groundwater level may be taken at Elevation 130.00 ± masl.

Seasonal variations in the water table should be expected, with higher levels occurring during wet weather conditions in the spring and fall and lower levels occurring during dry weather conditions.

5.0 PRELIMINARY GEOTECHNICAL DESIGN RECOMMENDATIONS

5.1 General Information

The recommendations presented in the following sections of this report are based on the information available regarding the proposed construction, the results obtained from the preliminary geotechnical investigation, and Pinchin's experience with similar projects.

Since the investigation only represents a portion of the subsurface conditions, it is possible that conditions may be encountered during construction that are substantially different than those encountered during the investigation. If these situations are encountered, adjustments to the design may be necessary.

As the design progresses, these preliminary results should be supplemented with a more detailed geotechnical field investigation and the design recommendations below should be revised based on the updated information. A qualified geotechnical engineer should be on-Site during the foundation preparation to ensure the subsurface conditions are the same/similar to what was observed during the investigation.

It is Pinchin's understanding that the development will consist of a fourteen-storey apartment building with up to three levels of underground parking located in the northwest portion of the property. It is anticipated that the underside of footing may be set at 10 mbgs (i.e. Elevation 123 ± masl).

5.2 Site Preparation

The existing topsoil and fill material are not considered suitable to remain below the proposed building, driveways and parking areas and will need to be removed. In calculating the approximate quantity of topsoil and fill material to be stripped, we recommend that the topsoil and fill thicknesses provided on the individual borehole logs be increased by 50 mm to account for variations and some stripping of the mineral soil below.

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

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

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

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

The existing natural sand deposit is considered suitable to be reused and subgrade fill under the parking lot and driveways provided that it will be compacted according to the above table.

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

5.3 Open Cut Excavations

It is anticipated that the foundations will be constructed at approximately 10 mbgs based on three levels of underground parking.

Based on the subsurface information obtained from within the boreholes, it is anticipated that the excavated material will predominately consist of granular fill and native sand and silty sand material. Groundwater was encountered in all boreholes at depths ranging from 1.4 to 3.1 mbgs (Elevation 132.6 to 128.5 masl).

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Where workers must enter trench excavations deeper than 1.2 m, the trench excavations should be suitably sloped and/or braced in accordance with the Occupational Health and Safety Act (OHSA), Ontario Regulation 213/91, Construction Projects, July 1, 2011, Part III - Excavations, Section 226. Alternatively, the excavation walls may be supported by either closed shoring, bracing, or trench boxes complying with sections 235 to 239 and 241 under O. Reg. 231/91, s. 234(1). The use of trench boxes can most likely be used for temporary support of vertical side walls. The appropriate trench should be designed/confirmed for use in this soil deposit.

Based on the OHSA, the natural sand and sandy silt soils would be classified as Type 3 soil and temporary excavations in these soils must be sloped back at an inclination of 1 horizontal to 1 vertical (H to V) above this. Excavations extending below the groundwater table would be classified as a Type 4 soil and temporary excavations will have to be sloped back at 3 horizontal to 1 vertical from the base of the excavation.

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

5.3.1 Shoring Requirements

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

No excavation shall extend below a line cast as one vertical and one horizontal from foundations of existing structures without adequate alternate support being provided. Where open-cut excavations are not possible, conventional support systems comprising soldier piles and lagging, sheet piles, concrete caisson wall, or diaphragm walls may be considered. The shoring system may be designed as full cantilevers or the lateral loads can be taken up to the installation of internal bracing of rakers or tie back soil anchors.

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

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Soil Layer	Bulk Unit Weight (kN/m^3)	Angle of Internal Friction	Active Earth Pressure Coefficient	Passive Earth Pressure Coefficient
Earth Fill	18	28°	0.36	2.77
Sand	19	38°	0.24	4.20
Sandy Silt	20	34	0.28	3.54
Weathered Bedrock	26	35°	0.27	3.69
Slightly Weathered to Fresh Bedrock	26	44°	0.18	5.55

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.

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

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

5.4 Anticipated Groundwater Management

The recommendations within this section should be read in conjunction with the Hydrogeological Assessment Report.

Groundwater was encountered in all boreholes at depths ranging from 1.4 to 3.1 mbgs (Elevation 132.3 to 128.5 masl). For geotechnical design purposes, the groundwater level may be taken at Elevation 130 ± masl.

It is anticipated that the underside of footing may be set at 10 mbgs (i.e. Elevation 123 ± masl). As such excavations are anticipated to extend below the prevailing groundwater level and into the underlying shale bedrock.

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The native sand and sandy silt deposits are considered high permeability materials, which will permit the free-flow of water when wet. 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 0.5 m below the excavation base. It is recommended that Pinchin review the final grading plan to confirm this recommendation.

Additionally, to better control the groundwater, an impermeable shoring system (i.e. a continuous interlocking caisson wall) should be used to reduce the flow of water into the excavation. The caisson wall embedment elevation or depth below the bulk excavation can be determined during the detailed design stage. The dewatering system must be maintained fully operational until such time as the fully waterproofed raft has sufficient factored dead loads that exceed the factored uplift.

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

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

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

It is the responsibility of the contractor to propose a suitable dewatering system based on the groundwater elevation at the time of construction. The method used should not adversely impact any nearby structures.

5.5 Foundation Design

5.5.1 Shallow Foundations Bearing on Bedrock

It is anticipated that the underside of footing may be set at 10 mbgs (i.e. Elevation 123 ± masl). Bedrock was encountered at depths ranging from 3.2 to 6.1 mbgs (Elevation 126.0 to 129.1 masl). The upper 2 to 4 meters of the bedrock is highly weathered and the RQDs of the cores that were obtained from the bedrock indicate very poor to poor quality bedrock.

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Conventional shallow strip and spread footings established directly on the weathered bedrock surface, a factored geotechnical bearing resistance of 1000 kPa may be used at Ultimate Limit States (ULS). The bearing resistance for the sound bedrock with poor classification, below Elevation 121.0 masl can be increased to 3000 kPa at ULS.

Prior to installing foundation formwork, the bedrock is to be reviewed by a geotechnical engineer. SLS does not apply to foundations bearing directly on bedrock, since the loads required for unacceptable settlements to occur would be much larger than the factored ULS and would be limited to the elastic compression of the bedrock and concrete.

The bearing resistance of 1000 kPa (weathered bedrock) assumes the bedrock is cleaned of all overburden material and any loose rock pieces. The bedrock should be cleaned with air or water pressure exposing clean sound bedrock. If construction proceeds during freezing weather conditions water should not be allowed to pool and freeze in bedrock depressions. All concrete should be installed and maintained above freezing temperatures as required by the concrete supplier.

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

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

5.5.2 Site Classification for Seismic Site Response & Soil Behaviour

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

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

The boreholes advanced at this Site extended to approximately 12 mbgs and were terminated in the bedrock. SPT “N” values within the overburden ranged between 3 and greater than 50 blows per 300 mm. As such, based on Table 4.1.8.4.A of the OBC, this Site has been classified as Class C. A Site Class C has an average shear wave velocity (Vs) of between 360 and 760 m/s. It is recommended that shear wave velocity soundings be completed at the Site once final design and depths of foundations are known as a higher Site Classification may be available for deeper foundations at the Site.

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. 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.

5.5.4 Estimated Settlement

All individual spread footings should be founded on shale bedrock, 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.

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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.

5.5.6 Shallow Foundations Frost Protection & Foundation Backfill

Experience suggests that the temperature in nominally unheated underground parking with two or more levels below grade and normal ventilation provisions is not as severe as the ambient open-air condition. In Mississauga, the earth cover required to prevent frost effects on foundations in the lower parking levels need not be any greater than 1.2 metres, and unmonitored experience in a number of structures and industry practice indicate that perimeter foundations provided with a minimum of 600 mm of soil cover perform adequately as do the interior isolated foundations with 900 mm of soil cover.

Foundations located immediately adjacent to air shafts, entrance and exit doors shall be treated as exterior foundations and should be provided with a minimum of 1.2 m of soil cover or equivalent insulation to ensure that foundations are not affected by the cold air flow.

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.

5.6 Underground Parking Garage Design

It is understood that the building is proposed to be constructed with up to three levels of underground parking. It is anticipated that the underside of footing may be set at 10 mbgs (i.e. Elevation 123 ± masl). Groundwater was encountered in all boreholes at depths ranging from 1.4 to 3.1 mbgs (Elevation 132.3 to 128.5 masl). For geotechnical design purposes, the groundwater level may be taken at Elevation 130 ± masl.

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

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

$$P = \gamma \times d$$

Where:

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

γ = unit weight of water (9.8 kN/m^3)

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

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

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

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 natural sand material is considered suitable for reuse as foundation wall backfill. The backfill material used against the foundation must be placed so that the allowable lateral capacity is achieved. All granular material is to be placed in maximum 300 mm thick lifts compacted to a minimum of 100% SPMDD in hard landscaping areas and 95% SPMDD in soft landscaping areas. It is recommended that inspection and testing be carried out during construction to confirm backfill quality, thickness and to ensure compaction requirements are achieved.

Where the structure is made directly against a shored excavation, the shoring wall should be covered with a layer of MiraDRAIN 6000 drainage composite or equivalent, with a minimum 150 mm overlap between drainage boards. This drainage board is to be covered with a continuous bentonite membrane with all joints welded and inspected. The drainage board should be connected to a basement sump via discharge pipes that protrude through the concrete foundation wall at 2.5 m spacing. This piping must not connect to the interior underfloor draining system.

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Within the foundation walls, perimeter weeping 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 connected to an interior sump pump systems.

An underfloor drainage system is recommended. The underfloor drainage system should be installed beneath the slab and should be constructed in a similar fashion to the foundation drains and be connected to a suitable frost-free outlet or sump.

The details of this foundation wall and floor slab drainage system must be reviewed by Pinchin prior to submission to the contractor.

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 if backfilled against the foundation wall. The bulk unit weight of the retained backfill may be taken as 20 kN/m^3 for well compacted soil. The values provided in the table presented in Section 5.3.1 can be used for calculating the lateral earth pressure. An appropriate factor of safety should be applied.

5.7 Floor Slabs

The shale bedrock is considered suitable to support a floor slab for the proposed building. Once the subgrade is exposed it should be inspected by a qualified geotechnical engineering consultant. Any loosened bedrock pieces should be removed and replaced with additional floor slab fill.

Based on the in-situ soil conditions, it is recommended to establish the concrete floor slab on a minimum 200 mm thick layer of Granular “A” (OPSS 1010). Alternatively, consideration may also be given to using a 200 mm thick layer of uniformly compacted 19 mm clear stone 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 unfactored modulus of subgrade reaction for the weathered shale bedrock would be 75,000 kN/m^3.

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6.0 SITE SUPERVISION & QUALITY CONTROL

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

7.0 TERMS AND LIMITATIONS

This Geotechnical Investigation was performed for the exclusive use of 1785 Bloor Holdings Inc. (Client) in order to evaluate the subsurface conditions at 1785 Bloor Street, Mississauga, Ontario. Within the limitations of scope, schedule and budget, our services have been executed in accordance with generally accepted practises in the field of geotechnical engineering for the Site. Classification and identification of soil, and geologic units have been based upon commonly accepted methods employed in professional geotechnical practice. No warranty or other conditions, expressed or implied, should be understood. Conclusions derived are specific to the immediate area of study and cannot be extrapolated extensively away from sample locations.

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

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

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

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

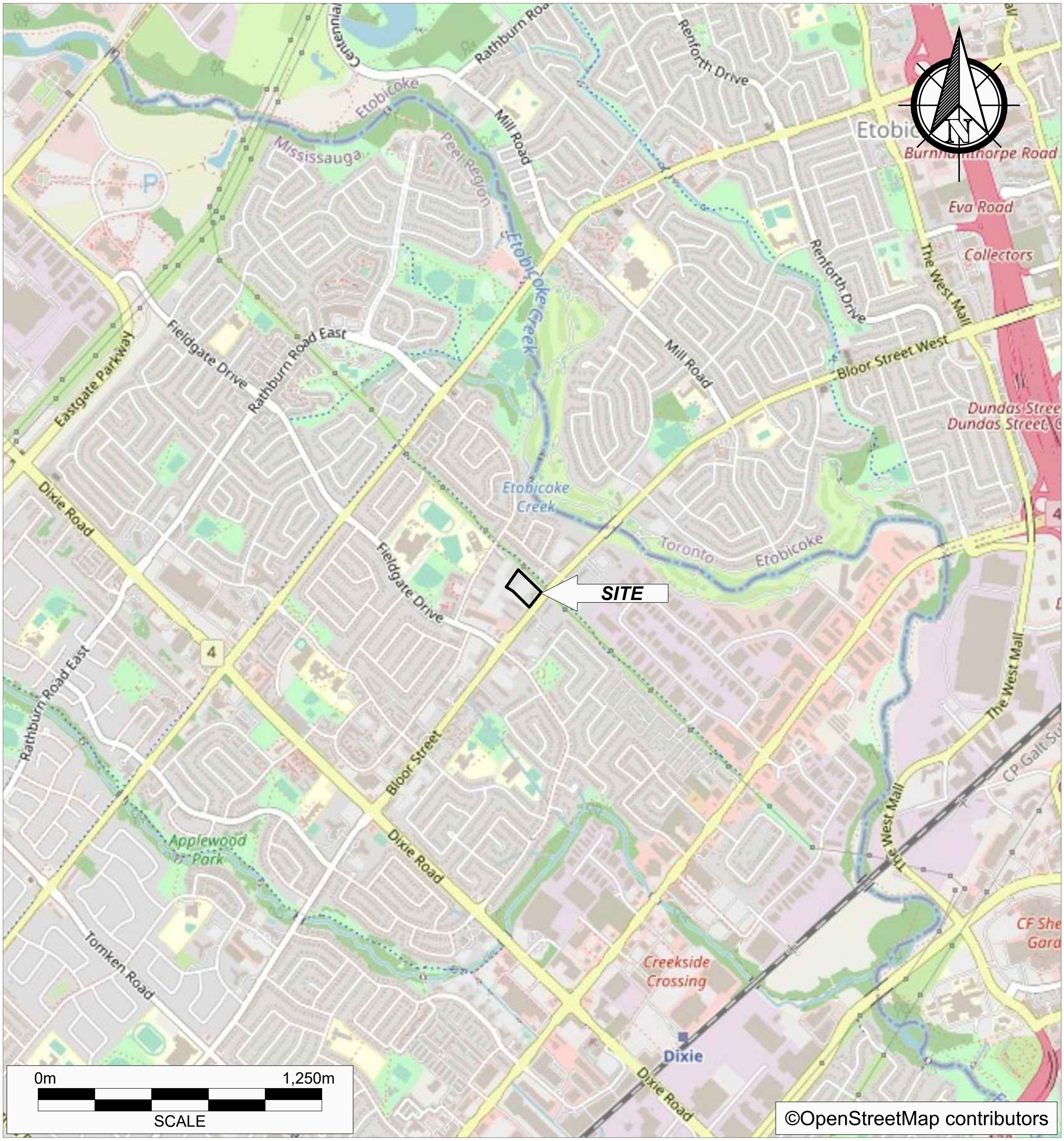
Specific limitations related to the legal and financial and limitations to the scope of the current work are outlined in our proposal, the attached Methodology and the Authorization to Proceed, Limitation of Liability and Terms of Engagement which accompanied the proposal.

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

291885.001 FINAL Geotechnical Investigation 1785 Bloor Street East Mississauga ON March 18 2022.docx

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

FIGURES



PINCHIN

PROJECT NAME
PRELIMINARY GEOTECHNICAL INVESTIGATION

CLIENT NAME
1785 BLOOR HOLDINGS INC.

PROJECT LOCATION
1785 BLOOR STREET, MISSISSAUGA, ONTARIO

FIGURE NAME			FIGURE NO.
KEY MAP			1
SCALE: AS SHOWN	PROJECT NO.: 291885.001	DATE: NOVEMBER 2021	

N

LEGEND

SITE BOUNDARY

SITE BUILDING

BOREHOLE

[#] GROUND ELEVATION IN masl

masl METRES ABOVE SEA LEVEL

LEGEND IS COLOUR DEPENDENT. NON-COLOUR COPIES MAY ALTER INTERPRETATION.

PINCHIN

PROJECT NAME: PRELIMINARY GEOTECHNICAL INVESTIGATION	
CLIENT NAME: 1785 BLOOR HOLDINGS INC.	
PROJECT LOCATION: 1785 BLOOR STREET, MISSISSAUGA, ONTARIO	
FIGURE NAME: BOREHOLE LOCATION PLAN	
PROJECT NUMBER: 291885.001	SCALE: AS SHOWN
DRAWN BY: KP	REVIEWED BY: MB
DATE: NOVEMBER 2021	FIGURE NUMBER: 2

APPENDIX I

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

ABBREVIATIONS, TERMINOLOGY & PRINCIPAL SYMBOLS USED

Sampling Method

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

In-Situ Soil Testing

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

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

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

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

Soil Descriptions

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

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

Notes:

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

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

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

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

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

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

Soil & Rock Physical Properties

General

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

Consistency

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

Shear Strength

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

Consolidation (One Dimensional)

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

Permeability

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

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

Rock Coring

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

RQD is calculated as follows:

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

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: BH1

Project #: 291885.001 **Logged By:** KS

Project: Preliminary Geotechnical Investigation

Client: 1785 Bloor Holdings Inc.

Location: 1785 Bloor Street, Mississauga, Ontario

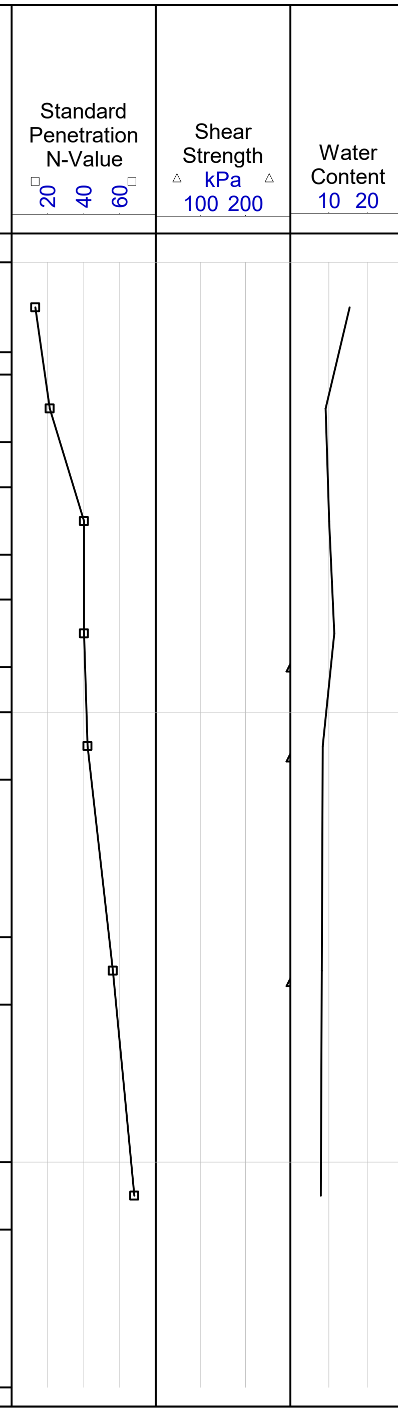
Drill Date: October 29, 2021 **Project Manager:** RM

SUBSURFACE PROFILE

Depth (m)	Description	Elevation (m)
0	Ground Surface	132.12
	Asphalt Asphalt pavement	131.84
	Fill Brown sand and gravel, compact, moist	131.36
1	Brown to gray, motted, silty sand trace gravel, compact, moist	130.60
	Sand Brown fine sand some gravel trace clay, compact, moist	
2	trace broken cobbles, dense	129.84
	trace sand and gravel	129.43
3	**Sandy Silt** Grey sandy silt some gravel and clay, dense, moist	126.03
6	**Shale** Grey weathered shale	124.50

SAMPLE

Sample Type	Sampler #	Recovery (%)	SPT N-Value
SS	1	40	13
SS	2	50	21
SS	3	50	40
SS	4	40	40
SS	5	50	42
SS	6	60	56
SS	7	60	68



Monitoring Well Details | Soil Vapour Concentration (ppm) | Laboratory Analysis

Contractor: Strata Drilling Inc. **Grade Elevation:** 132.12 masl

Drilling Method: Hollow Stem Auger / Split Spoon Sampler **Top of Casing Elevation:** 132.02 masl

Well Casing Size: 2" **Sheet:** 1 of 2

Log of Borehole: BH1

Project #: 291885.001 **Logged By:** KS

Project: Preliminary Geotechnical Investigation

Client: 1785 Bloor Holdings Inc.

Location: 1785 Bloor Street, Mississauga, Ontario

Drill Date: October 29, 2021 **Project Manager:** RM

SUBSURFACE PROFILE					SAMPLE								
Depth (m)	Symbol	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-Value	Standard Penetration N-Value (20 40 60)	Shear Strength kPa (100 200)	Water Content (10 20)	Soil Vapour Concentration (ppm)	Laboratory Analysis
8		**Bedrock - Cored** RQD = 35%			SS	8	98						
9		RQD = 30 %	122.88		SS	9	100						
10–11		RQD = 20 %	121.28		SS	10	95						
12		End of Borehole	119.80	Water level measured to be at 2.5 mbgs on November 17, 2021									
13													
14													
15													

Contractor: Strata Drilling Inc. **Grade Elevation:** 132.12 masl

Drilling Method: Hollow Stem Auger / Split Spoon Sampler **Top of Casing Elevation:** 132.02 masl

Well Casing Size: 2" **Sheet:** 2 of 2

Log of Borehole: BH2

Project #: 291885.001 **Logged By:** KS

Project: Preliminary Geotechnical Investigation

Client: 1785 Bloor Holdings Inc.

Location: 1785 Bloor Street, Mississauga, Ontario

Drill Date: October 28, 2021 **Project Manager:** RM

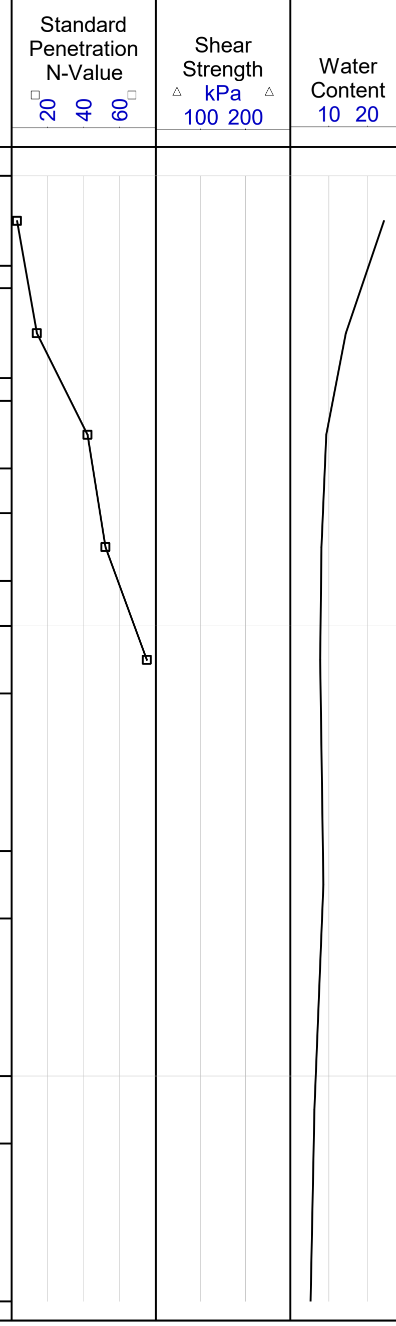
SUBSURFACE PROFILE

Description	Elevation (m)
Ground Surface	133.62
Topsoil Dark brown silt some sand trace clay (with roots), very loose, very moist	133.34
Fill Brown silt some clay, very loose, very moist	132.86
Sand Grey to brown fine sand some gravel, trace clay and silt, compact, very moist	132.09
Light brown trace silt and gravel, dense, moist	131.33
Very dense	129.05
Shale Grey weathered shale	125.92

Depth (m): 0, 1, 2, 3, 4, 5, 6, 7

SAMPLE

Sample Type	Sampler #	Recovery (%)	SPT N-Value
SS	1	40	3
SS	2	50	14
SS	3	70	42
SS	4	80	52
SS	5	70	75
SS	6	50	>50
SS	7	20	>50
	8		>50



Soil Vapour Concentration (ppm)

Laboratory Analysis

Contractor: Strata Drilling Inc. **Grade Elevation:** 133.62 masl

Drilling Method: Hollow Stem Auger / Split Spoon Sampler **Top of Casing Elevation:** 133.50 masl

Well Casing Size: 2" **Sheet:** 1 of 2

Log of Borehole: BH2

Project #: 291885.001 **Logged By:** KS

Project: Preliminary Geotechnical Investigation

Client: 1785 Bloor Holdings Inc.

Location: 1785 Bloor Street, Mississauga, Ontario

Drill Date: October 28, 2021 **Project Manager:** RM

SUBSURFACE PROFILE					SAMPLE								
Depth (m)	Symbol	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-Value	Standard Penetration N-Value (20 40 60)	Shear Strength kPa (100 200)	Water Content (10 20)	Soil Vapour Concentration (ppm)	Laboratory Analysis
					SS	8	20	>50					
8		**Bedrock - Cored** RQD = 25%	124.40		RC	9	99						
9–10		RQD = 5 %	122.85		RC	10	99						
11–12		RQD = 45 %	121.37		RC	11	100						
13–15		End of Borehole		Water level measured to be at 1.4 mbgs on November 25, 2021									

Contractor: Strata Drilling Inc. **Grade Elevation:** 133.62 masl

Drilling Method: Hollow Stem Auger / Split Spoon Sampler **Top of Casing Elevation:** 133.50 masl

Well Casing Size: 2" **Sheet:** 2 of 2

Log of Borehole: BH3

Project #: 291885.001 **Logged By:** KS

Project: Preliminary Geotechnical Investigation

Client: 1785 Bloor Holdings Inc.

Location: 1785 Bloor Street, Mississauga, Ontario

Drill Date: October 28, 2021 **Project Manager:** RM

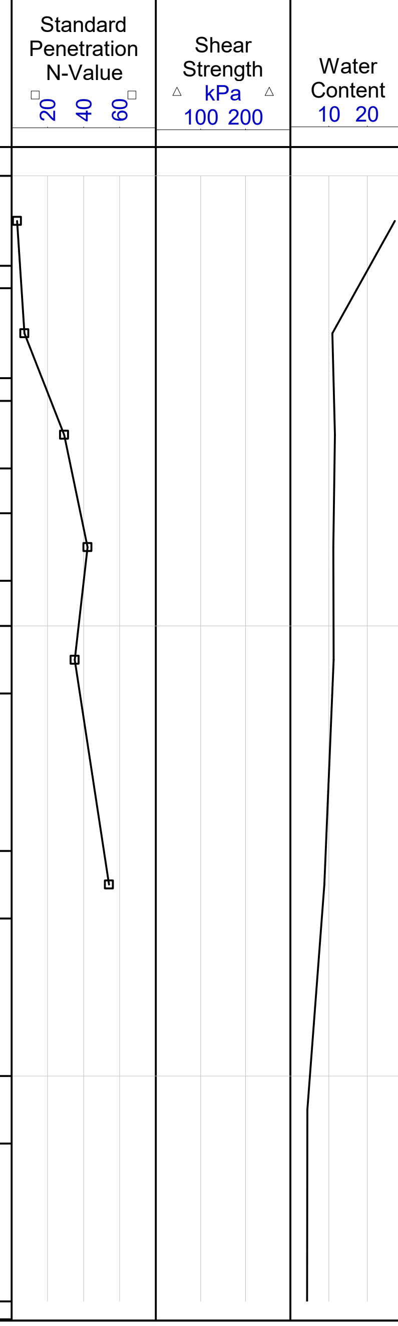
SUBSURFACE PROFILE

Elevation (m)	Description
131.93	Ground Surface
131.60	**Topsoil** Dark brown silt trace sand and clay (with roots), very loose, very moist
131.16	**Fill** Brown silt some clay trace sand, very loose, very moist
131.01	Loose
130.40	Sand some silt trace clay and gravel, trace rootlets, loose, very moist
129.64	**Sand** Brown, fine sand trace gravel and silt compact, moist
128.88	Grey, some gravel trace silt (with occasional cobbles), dense, very moist
128.45	**Sandy Silt** Brown to grey sandy silt trace clay and gravel, dense, moist
124.18	**Shale** Grey weathered shale

SAMPLE

Sample Type	Sampler #	Recovery (%)	SPT N-Value
SS	1	50	3
SS	2	50	7
SS	3	60	29
SS	4	50	42
SS	5	50	35
SS	6	50	54
SS	7	10	>50

Columns: Depth (m) | Symbol | Description | Elevation (m) | Monitoring Well Details | Sample Type | Sampler # | Recovery (%) | SPT N-Value | Standard Penetration N-Value | Shear Strength kPa | Water Content | Soil Vapour Concentration (ppm) | Laboratory Analysis



Contractor: Strata Drilling Inc. **Grade Elevation:** 131.93 masl

Drilling Method: Hollow Stem Auger / Split Spoon Sampler **Top of Casing Elevation:** 131.83 masl

Well Casing Size: 2" **Sheet:** 1 of 2

Log of Borehole: BH3

Project #: 291885.001 **Logged By:** KS

Project: Preliminary Geotechnical Investigation

Client: 1785 Bloor Holdings Inc.

Location: 1785 Bloor Street, Mississauga, Ontario

Drill Date: October 28, 2021 **Project Manager:** RM

SUBSURFACE PROFILE					SAMPLE								
Depth (m)	Symbol	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-Value	Standard Penetration N-Value 20 40 60	Shear Strength kPa 100 200	Water Content 10 20	Soil Vapour Concentration (ppm)	Laboratory Analysis
8		**Bedrock - Cored** RQD = N/A (rock core stuck in barrel)	124.18		SS	8	10	>50					
9					RC	9	-						
10		RQD = 40 %	122.61		RC	10	99						
11		RQD = 50 %	121.01		RC	11	93						
12		End of Borehole	119.73	Water level measured to be at 2.5 mbgs on December 17, 2021									
13													
14													
15													

Contractor: Strata Drilling Inc.

Drilling Method: Hollow Stem Auger / Split Spoon Sampler

Well Casing Size: 2"

Grade Elevation: 131.93 masl

Top of Casing Elevation: 131.83 masl

Log of Borehole: BH4

Project #: 291885.001 **Logged By:** KS

Project: Preliminary Geotechnical Investigation

Client: 1785 Bloor Holdings Inc.

Location: 1785 Bloor Street, Mississauga, Ontario

Drill Date: October 27, 2021 **Project Manager:** RM

SUBSURFACE PROFILE

Depth (m)	Description	Elevation (m)
0	Ground Surface	132.55
0 – 1.52	***Asphalt*** Asphalt pavement ***Fill*** Brown sand and gravel, very loose, very moist	131.03
1.52 – 2.29	***Sand*** Brown sand trace to some gravel and sillt, dense, wet	130.26
2.29 – 3.20	(with cobbles), very dense, saturated	129.35
3.20 – 6.10	***Shale*** Grey weathered shale	126.45
6.10 – 7.60	***Bedrock - Cored*** RQD = N/A (Inadequate sample recovery)	124.95

SAMPLE

Sample Type	Sampler #	Recovery (%)	SPT N-Value	Standard Penetration N-Value (20 40 60)	Shear Strength kPa (100 200)	Water Content (10 20)	Soil Vapour Concentration (ppm)	Laboratory Analysis
SS	1	10	4					
SS	2	40	2					
SS	3	40	30					
SS	4	10	>50					
SS	5	20	>50					
SS	6	20	>50					
RC	7	10						

Contractor: Strata Drilling Inc. **Grade Elevation:** 132.55 masl

Drilling Method: Hollow Stem Auger / Split Spoon Sampler **Top of Casing Elevation:** 132.45 masl

Well Casing Size: 2" **Sheet:** 1 of 2

Log of Borehole: BH4

Project #: 291885.001 ***Logged By:*** KS

Project: Preliminary Geotechnical Investigation

Client: 1785 Bloor Holdings Inc.

Location: 1785 Bloor Street, Mississauga, Ontario

Drill Date: October 27, 2021 ***Project Manager:*** RM

SUBSURFACE PROFILE					SAMPLE									
Depth (m)	Symbol	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-Value	Standard Penetration N-Value 20 40 60	Shear Strength kPa 100 200	Water Content 10 20	Soil Vapour Concentration (ppm)	Laboratory Analysis
8		RQD = 30 %	123.46		RC	8	93						
9–10		RQD = 35 %	121.88		RC	9	93						
11–12		RQD = 40 %	120.41		RC	10	95						
13–15		End of Borehole		water level measured to be at 3.1 mbgs on November 17, 2021									

Contractor: Strata Drilling Inc.

Grade Elevation: 132.55 masl

Drilling Method: Hollow Stem Auger / Split Spoon Sampler

Top of Casing Elevation: 132.45 masl

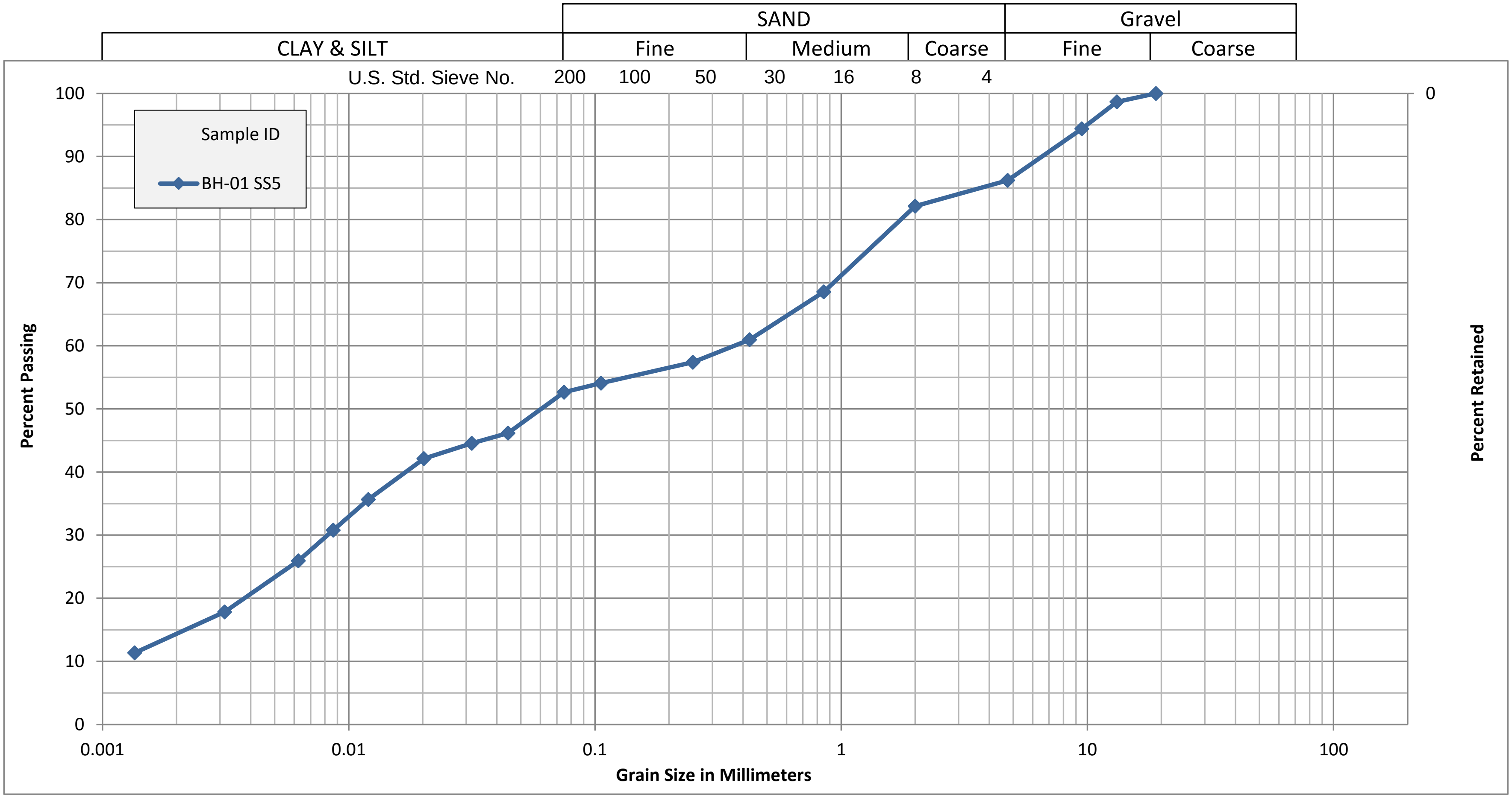
Well Casing Size: 2"

Sheet: 2 of 2

APPENDIX III

Laboratory Testing Reports for Soil and Rock Samples

Unified Soil Classification System



Sample ID	Depth (ft)	% Gravel	% Sand	% Silt	% Clay
BH-01 SS5	10.0-12.0	14.0	33.3	38.7	14.0

PINCHIN

Pinchin Waterloo - 225 Labrador Drive,
Unit 1, Waterloo, Ontario N2K 4M8

PARTICLE SIZE DISTRIBUTION ANALYSIS

Preliminary Geotechnical Investigation - 1785 Bloor St E, Mississauga, ON
Compten Management Inc.

Figure No. **1**

291885.001

Reviewed By:

More information available upon request

Atterberg Limits

LS 703&704 / AASHTO T89

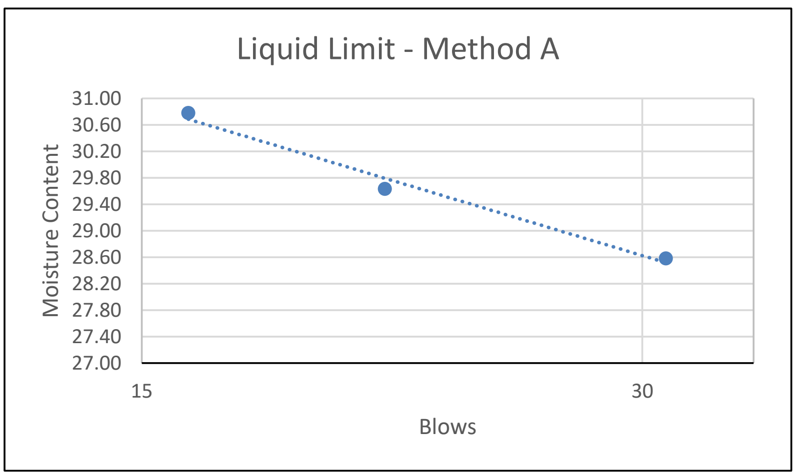
Project Name: Preliminary Geotechnical Investigation
Project No. 291885.001
Client: Compten Management Inc.
Location: 1785 Bloor Street East, Mississauga, ON
Material: Soil
Sample: BH-01 SS5 10.0-12.0

Test Date: November 2, 2021
Tested By: B Frank
Sample Date: October 29, 2021
Sampled By: K Singh
Reviewed By: V Marshall

Liquid Limit - Method A						
Pot Number	1	2	3			
Number of blows	31	21	16			
Wet mass + pot	31.19	29.18	33.03			
Dry mass + pot	27.74	26.11	28.97			
Tare	15.67	15.75	15.78			
Water content %	28.58	29.63	30.78			

Plastic Limit			
Pot Number	1	2	
Wet mass + pot	26.14	24.96	
Dry mass + pot	24.42	23.48	
Tare	15.45	15.71	
Water content %	19.2	19.0	

PI = LL - PL	
Liquid Limit %	**29.2**
Plastic Limit %	**19**
Plastic Index	**10**
Non Plastic	



Geomechanica Inc.
Suite 900 – 390 Bay St.
Toronto Ontario
Canada M5H 2Y2

November 12, 2021

Mr. Reza Mahmoudipour
Pinchin Ltd.
2470 Milltower Court
Mississauga, Ontario
Canada, L5N 7W5

Re: UCS testing
(Pinchin Project 291885.001)

Dear Mr. Mahmoudipour:

On November 1st, 2021 one (1) NQ-sized core sample was received by Geomechanica Inc. via drop off by Pinchin personnel. This samples were identified as being from Pinchin Project No. 291885.001. One (1) UCS specimen was prepared and tested.

Details regarding the steps of specimen preparation and testing along with the results and photographs of the test specimen before and after testing are presented in the accompanying laboratory report and summary spreadsheet(s).

Sincerely,

Bryan Tatone Ph.D., P. Eng.

Geomechanica Inc.
Tel: (647) 478-9767
Email: bryan.tatone@geomechanica.com

Rock Laboratory Testing Results

A report submitted to:

Reza Mahmoudipour
Pinchin Ltd.
2470 Milltower Court
Mississauga, Ontario
Canada, L5N 7W5

Prepared by:

Bryan Tatone, PhD, PEng

Omid Mahabadi, PhD, PEng
Geomechanica Inc.
#900-390 Bay St.
Toronto ON
M5H 2Y2 Canada
Tel: +1-647-478-9767
lab@geomechanica.com

November 12, 2021
Project number: 291885.001

Abstract

This document summarizes the results of rock laboratory testing, including 1 Uniaxial Compressive Strength (UCS) test. The UCS value along with photographs of specimen before and after testing are presented herein.

In this document:

Disclaimer: This report was prepared by Geomechanica Inc. for Pinchin Ltd.. The material herein reflects Geomechanica Inc.'s best judgment given the information available at the time of preparation. Any use which a third party makes of this report, any reliance on or decision to be made based on it, are the responsibility of such third parties. Geomechanica Inc. accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.

1 Uniaxial Compressive Strength Tests

1.1 Overview

This section summarizes the results of uniaxial compressive strength (UCS) testing. The testing was performed in Geomechanica's rock testing laboratory using a 150 ton (1.3 MN) Forney loading frame equipped with pressure-compensated control valve to maintain an axial displacement rate of approximately 0.150 mm/min (Figure 1). The specimen preparation and testing procedure included the following:

1. Unwrapping of the core sample, and inspecting it for damage.
2. Diamond cutting the core sample to obtain a cylindrical specimen with an appropriate length (length:diameter = 2:1) and nearly parallel end faces.
3. Diamond grinding the specimen to obtain flat (within ± 0.025 mm) and parallel end faces (within $0.25°$).
4. Placing the specimen into the loading frame, applying a 1 kN axial load, and removing the electrical tape.
5. Axially loading the specimens to rupture while continuously recording axial force and axial deformation to determine the peak strength (UCS).

Figure 1: Forney loading frame setup for UCS testing.

Using a precision V-block mounted on the magnetic chuck of the surface grinder, test specimens met the end flatness, end parallelism, and perpendicularity criteria set out in ASTM D4543-19. The side straightness criteria, as checked with a feeler gauge, was met for all specimens unless noted otherwise in Table 1. Testing of the specimens followed ASTM D7012-14 Method C.

1.2 Results

The results of UCS testing are summarized in Table 1. Additional specimen and testing details are available in the summary spreadsheet that accompanies this report.

Table 1: Summary of Uniaxial Compression test results.

Sample	Depth (m)	Bulk density ρ (g/cm^3)	UCS (MPa)	Lithology	Failure description
BH04, Run 2	N/A	2.630	41.2	Shale and Limestone	1

[1] Axial splitting failure

1.3 Specimen photographs

Photographs of the specimens before and after testing are presented in the Appendix of this report.

Appendices

Specimen sheets

- BH04, Run 2

Uniaxial Compression Test

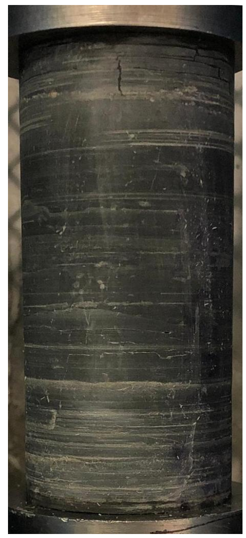
Client	Pinchin Ltd.	**Project**	291885.001
Sample	BH04, Run 2	**Depth**	N/A - N/A

Specimen parameters

Diameter (mm) [a]	47.11
Length (mm) [a]	96.95
Bulk density ρ (g/cm^3)	2.630
UCS (MPa)	41.2
Lithology	Shale and Limestone
Failure description [b]	1

[a] Additional specimen measurement/details provided in accompanying summary spreadsheet.
[b] Failure description: [1] Axial splitting failure;

Prior to testing



After testing

Remarks: Loading rate: 0.15 mm/min

Performed by	AA/HS	**Date**	2021-11-08

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.