FINAL

Preliminary Geotechnical Investigation – Proposed Residential Development

23, 25, 27, 29 and 31 Helene Street North, 53 Queen Street East and Part of 70 Park Street East, Mississauga, Ontario

Prepared for:

MPCT DIF 70 Park Street East LP
30 Adelaide Street East, Suite 301
Toronto, ON M5C 3H1

January 18, 2023

Pinchin File: 314281.002

Issued to:	**MPCT DIF 70 Park Street East LP**
Issued on:	**January 18, 2023**
Pinchin File:	**314281.002**
Issuing Office:	**Mississauga, ON**

Author: Shanjin Liu
Project Technologist, Geotechnical Services
437-788-7427
sliu@pinchin.com

LICENSED PROFESSIONAL ENGINEER
J.W. DIETZ
2023-01-18
PROVINCE OF ONTARIO

Reviewer: Jeff Dietz, P.Eng.
Senior Technical Manager, Geotechnical Services
519.589.3768
jdietz@pinchin.com

TABLE OF CONTENTS

FIGURES

APPENDICES

1.0 INTRODUCTION AND SCOPE

Pinchin Ltd. (Pinchin) was retained by MPCT DIF 70 Park Street East LP (Client) to conduct a Geotechnical Investigation and provide subsequent geotechnical design recommendations for the proposed residential development to be located at 23, 25, 27, 29 and 31 Helene Street North, 53 Queen Street East and Part of 70 Park Street East, Mississauga, Ontario (Site). The Site location is shown on Figure 1.

Drawings A000 through A014 (Issue 1, dated January 4, 2022) by Arcadis IBI Group were provided to Pinchin and reviewed in preparation of this report.

It is Pinchin's understanding that it is proposed to demolish the existing parking garage and associated structures to facilitate the redevelopment of the Site to include a 38-storey (plus mechanical penthouse) mixed use building with commercial uses at grade, and residential uses above, resting on eight levels of underground parking. The above-noted section drawing shows the lowest level of underground parking having a finished floor level of 58.551 masl, which is about 22.5 to 24.5 m below existing grade. The current apartment building located at 70 Park Street East, which is beyond the site, will remain in place.

It is noted that at the time of Pinchin's proposal for this work there were fewer underground levels being considered, and Pinchin had anticipated that the lowest underground level would be at about 18 m below exterior grades. Since the boreholes for the current investigation were scoped based on the previous understanding of the redevelopment, the boreholes did not extend to the depth of the proposed lowest level of underground parking. As such, the results of this investigation should be considered preliminary; and additional investigation with deeper boreholes will be needed to support final design of the redevelopment.

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

The purpose of the Geotechnical Investigation was to delineate the subsurface conditions and soil engineering characteristics by advancing a total of seven (7) geotechnically sampled boreholes (Boreholes BH1 to BH7), at the Site.

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

- A review of relevant area geology and Site background information;
- A detailed description of the soil and groundwater conditions;
- Open cut excavations and/or shoring requirements (where necessary);
- Anticipated groundwater management;
- Lateral earth pressure coefficients and unit densities;
- Preliminary foundation design recommendations including soil and bedrock bearing resistances at Ultimate Limit States (ULS) and Serviceability Limit States (SLS) design;
- Potential total and differential settlements;
- Seismic Site Classification for Seismic Site Response;
- Foundation frost protection and installation;
- Underground parking garage design; and
- Potential construction concerns.

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

2.0 SITE DESCRIPTION AND GEOLOGICAL SETTING

The Site is located on the southeast side of Queen Street East and northeast side of Helene Street North, in the City of Mississauga, Ontario, and consists of a roughly rectangular shaped parcel of land. The Site is currently occupied by a parking garage with three above grade levels and one below grade level, retail space located on the ground level, and at-grade asphalt parking lot and landscaped areas.

It is understood that it is proposed to demolish the existing parking garage and associated structures to facilitate the redevelopment of the Site to include a 38-storey (plus mechanical penthouse) mixed use building with commercial uses at grade, and residential uses above, resting on eight levels of underground parking. The drawings referenced in Section 1.0 of this report show the lowest level of underground parking having a finished floor level of 58.551 masl, which is about 22.5 to 24.5 m below existing grade. The current apartment building located at 70 Park Street East, which is beyond the site, will remain in place.

Data obtained from the Ontario Geological Survey Maps, as published by the Ontario Ministry of Energy, Northern Development and Mines, indicates that the Site is located primarily on coarse-textured glaciolacustrine deposits consisting of sand, gravel and minor silt/clay deposits; foreshore and basinal deposits. (Ontario Geological Survey 2010, Surficial geology of Southern Ontario; Ontario Geological

Survey, Miscellaneous Release--Data 128-REV). The underlying bedrock at this Site is of the Georgian Bay Formation consisting of shale, limestone (Armstrong, D.K. and Dodge, J.E.P. 2007, Paleozoic geology of southern Ontario; Ontario Geological Survey, Miscellaneous Release--Data 219).

3.0 GEOTECHNICAL FIELD INVESTIGATION AND METHODOLOGY

Pinchin completed the field investigations at the Site on October 11 to 17, 2022 by advancing a total of seven (7) geotechnically sampled boreholes (Boreholes BH1 to BH7) throughout the Site. The boreholes were advanced to depths of approximately 2.0 to 20.1 meters below 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 track-mounted drill rig which was equipped with standard soil sampling equipment. Soil samples were collected at regular intervals using a 51 mm outside diameter (OD) split spoon barrel in conjunction with Standard Penetration Tests (SPT) “N” values (ASTM D1586). The SPT “N” values were used to assess the compactness condition of the non-cohesive soil and to estimate the consistency of the cohesive soil. Groundwater observations and measurements were obtained from the open boreholes during and upon completion of drilling. The in-situ testing, groundwater observations and measurements recorded are included on the appended borehole logs.

Bedrock was proven in Boreholes BH1, BH2 and BH7 by core drilling with an HQ-size double tube diamond bit core barrel. The bedrock core specimens were measured in the field to determine the Rock Quality Designation (RQD) (ASTM 6032). The core samples were returned to our Mississauga office for further visual examination.

Monitoring wells were installed in boreholes BH1, BH2, BH5 to BH7 to allow measurement of the groundwater levels. The monitoring wells were constructed in BH1, BH2, BH6 and BH7 using flush-threaded 50 mm diameter Trilock pipe with 3.0 meter long 10-slot well screens, and the monitoring well is constructed in BH5 using flush-threaded 31 mm diameter Trilock pipe with 3.0 meter long 10-slot well screens, delivered to the Site in pre-cleaned individually sealed plastic bags. The screen and riser pipes were not allowed to come into contact with the ground or drilling equipment prior to installation.

A completed well record was submitted to the Ontario Ministry of the Environment, Conservation and Parks (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.

The borehole locations and ground surface elevations were surveyed by J.D. Barnes Limited, and provided to Pinchin. The ground surface elevations are geodetic.

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 Pinchin's laboratory in Waterloo, Ontario for detailed analysis. All soil samples were classified according to visual and index properties by the project engineer.

The field logging of the soil, bedrock 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 III.

4.0 SUBSURFACE CONDITIONS

4.1 Borehole Soil Stratigraphy

The following stratigraphy is based on the borehole findings. The summary provided below is for general guidance only. Detailed depths and elevations are given in the following subsections and appended borehole logs. In general, the three (3) main stratigraphic units are as follows:

- Surficial layers and earth fill materials extending to approximately 0.13 to 1.5 mbgs (i.e. Elevation 81.6 to 80.3 masl); overlying
- Native sand and glacial till deposits extending to approximately 6.5 to 8.0 mbgs (i.e. Elevation 75.2 to 75.0 masl); overlying
- Shale bedrock of Georgian Bay Formation.

4.1.1 Surficial Layers

A topsoil layer with thickness approximately 130 mm was encountered at the ground surface in Boreholes BH1 to BH4 and BH7.

A concrete slab ranging in thickness from approximately 100 to 130 mm was encountered at the ground surface in Boreholes BH5 and BH6.

4.1.2 Earth Fill

Earth fill materials, consisting of silty sand to sandy silt, with trace amounts of clay, organics, gravels, rootlets and Styrofoam were encountered beneath the topsoil or concrete slab in each borehole, except for Borehole B7, and extended to depths ranging from approximately 0.75 to 1.5 mbgs.

The cohesionless earth fill zone has a loose to compact relative density based on SPT 'N' values of 8 to 14 blows per 300 mm penetration of a split spoon sampler. The fill was moist to wet based on moisture content results of 5 to 19%.

4.1.3 Sand / Silt

Native sand and silt to sandy silt, with trace amounts of clay and gravel was encountered beneath the earth fill zone in Borehole BH1, BH2, BH5 and BH7 and extended to a depth ranging from approximately 2.3 to 3.1 mbgs.

The results of a particle size distribution test performed on one sample of this deposit are provided on Figure 1 and show that the sample contained 15% sand and 85% silt and clay.

The native sand and silt to sandy silt has a variable loose to very dense relative density based on SPT 'N' values of 7 to more than 50 blows per 300 mm penetration of a split spoon sampler. The in-situ moisture contents of the sand and silt sample was ranged from 13.1 to 25.9 percent by mass, indicating very moist to wet conditions.

4.1.4 Glacial Till

Clayey silt till, with trace to some amounts of sand, trace gravel and stone fragments was encountered beneath the earth fill zone and sand in each Borehole extended to a depth ranging from approximately 6.5 to 8.0 mbgs, where a lower limit was encountered.

The results of two particle size distribution tests performed on samples of this deposit are provided in Figure 1, and show that the samples contained 8 to 19% gravel, 25 to 30% sand, and 56 to 62% silt and clay.

The cohesive glacial till deposit has a firm to hard (typically very stiff to hard) consistency based on SPT 'N' values of 8 to more than 50 blows per 300 mm penetration of a split spoon sampler. The in-situ moisture contents of the cohesive glacial till samples ranged from 7.4 to 27.9 percent by mass. This deposit was described in the field as being About the Plastic Limit (APL).

4.1.5 Bedrock

Bedrock was encountered in boreholes BH1, BH2 and BH7 at depths ranging from approximately 6.5 to 8.0 mbgs (elevations ranging from approximately 75.2 to 75.0 masl). 'HQ' size rock coring was completed on the shale bedrock in Boreholes BH1, BH2 and BH7 and confirmed shale bedrock with limestone interbeds up to 200 mm in thickness of the Georgian Bay formation. The depth and elevation of the bedrock at the borehole locations is summarized in the following table.

Borehole	Depth (mbgs) to Bedrock	Approximate Elevation (masl) of Bedrock
BH1	7.1	75.2
BH2	8.0	75.0
BH7	6.5	75.1

The Rock Quality Designation (RQD) ranged from 20 to 100 percent indicating very poor to excellent quality bedrock, with quality increasing with depth. It is noted that all RQD results below Elevation 69 masl indicated good to excellent quality.

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 levels were measured in the monitoring wells installed in Boreholes BH1D, BH1S, BH2D, BH2S, and BH5 to BH7. The measured groundwater levels are summarized below:

Borehole No.	Depth of Borehole (mbgs)	Water Level Depth (mbgs) / Elevation (masl)			Monitoring Well Screen Interval (mbgs)
		October 18, 2022	October 24, 2022	October 28, 2022	
BH1D	19.8	17.6 / 64.7	17.6 / 64.7	17.3 / 65.0	16.0 – 19.0
BH1S		Dry	Dry	Dry	1.6 – 4.6
BH2D	20.0	17.5 / 65.5	17.0 / 66.0	17.0 / 66.0	16.0 – 19.0
BH2S		Dry	Dry	Dry	3.0 – 6.0
BH5	4.9	1.9* / 79.2	1.9* / 79.2	1.8* / 79.3	1.9 – 4.9
BH6	4.0	Dry	Dry	Dry	1.0 – 4.0
BH7	20.1	6.7 / 75.0	11.2 / 70.46	9.6 / 72.1	15.6 – 18.6

*Water level depth in mbfs.

The groundwater level in the monitoring wells ranged from Elevation 64.7 to 79.3 masl.

Construction dewatering at adjacent sites, existing building drains or dewatering systems, and seasonal variations may cause significant changes to the depth of the groundwater table over time. Additional information pertaining to groundwater at the Site is discussed in the hydrogeological report by Pinchin under a separate cover.

5.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

5.1 General Information

As noted previously, at the time of Pinchin's proposal for this work there were fewer underground levels being considered, and Pinchin had anticipated that the lowest underground level would be at about 18 m below exterior grades. Since the boreholes for the current investigation were scoped based on the previous understanding of the redevelopment, the boreholes did not extend to the depth of the proposed lowest level of underground parking. As such, the results of this investigation should be considered preliminary; and additional investigation with deeper boreholes will be needed to support final design of the redevelopment.

The recommendations presented in the following sections of this report are based on the information available regarding the proposed construction, the limited 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 understood that it is proposed to demolish the existing parking garage and associated structures to facilitate the redevelopment of the Site to include a 38-storey (plus mechanical penthouse) mixed use building with commercial uses at grade, and residential uses above, resting on eight levels of underground parking. The drawings referenced in Section 1.0 of this report show the lowest level of underground parking having a finished floor level of 58.551 masl, which is about 22.5 to 24.5 m below existing grade. The boreholes were advanced within the footprint of the proposed development. In general, the three (3) main stratigraphic units are as follows:

- Surficial layers and earth fill materials extending to approximately 0.13 to 1.5 mbgs (i.e. Elevation 81.6 to 80.3 masl); overlying
- Native sand and glacial till deposits extending to approximately 6.5 to 8.0 mbgs (i.e. Elevation 75.2 to 75.0 masl); overlying
- Shale bedrock of Georgian Bay Formation.

The groundwater level in the monitoring wells ranged from Elevation 64.7 to 79.3 masl.

5.2 Excavations

As indicated above, it is understood that the P8 FFE will be set at Elevation 58.551 masl.

Based on the subsurface information obtained from within the boreholes, it is anticipated that the excavated material will consist of earth fill materials, native sand and silt, undisturbed glacial till and weathered to fresh bedrock.

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 fill soils encountered on Site 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. The glacial till deposits would be classified as Type 2 soil and temporary excavations in these soils may be cut vertical in the bottom 1.2 m and must be sloped back at an inclination of 1 horizontal to 1 vertical (H to V) above this level. The soil type with the highest Type number should be used for excavations extending through various soil types.

The subsurface soils can be removed by conventional excavation equipment. Excavations in the overburden at the Site may encounter native cobble/boulder obstructions in the glacial till. Larger size particles (cobbles and boulders) that are not specifically identified in the boreholes may be present in the native soils. The size and distribution of cobbles/boulders/obstructions cannot be predicted with boreholes, as the sampler size is insufficient to secure representative particles of this size. The risk and responsibility for the removal and disposal of cobbles/boulders/obstructions must be addressed in the contract documents for foundations, excavations and shoring contractors.

The upper bedrock in this area is typically weathered and can usually be removed with mechanical equipment, such as a large excavator and hydraulic hammer (hoe ram) and where required, with line drilling on close centres. Often a hydraulic hammer can be utilized to create an initial opening for the excavator bucket to gain access of the layered rock. It is noted that the quality of the bedrock generally increases with depth.

The bedrock can contain vertical joints and near horizontal bedding planes. Therefore, some vertical and horizontal over break of the bedrock should be expected.

In addition, Pinchin recommends that a pre-excavation survey of all neighbouring properties be undertaken prior to conducting construction activities. The preconstruction survey will serve to protect the Client from claims unrelated to the construction activities in the development of this property.

Pinchin notes that, local contractors are familiar with excavating the local bedrock and have specialized knowledge and techniques for its removal. Depending on the block size and degree of weathering of the rock they may have a different approach than what is presented in the preceding paragraphs

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.

5.2.1 Shoring Requirements

Due to spatial limitations, it may not be feasible to slope the excavations back to a safe angle and therefore some support system will be required.

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, sheetpiling 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.

The Site is immediately bounded by Municipal roads to the north, west and south, and residential/commercial lots to the east. The sections along the perimeter of the Site likely need to be shored to preserve the integrity of the boundary conditions using a shoring system comprising of soldier piles and lagging.

5.2.2 Lateral Earth Pressure

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 preliminary 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.

Soil Layer	Bulk Unit Weight (kN/m3)	Angle of Internal Friction	Active Earth Pressure Coefficient	Passive Earth Pressure Coefficient
Earth Fill	18	28°	0.36	2.77
Sand	18	32°	0.31	3.25
Glacial Till	21	34°	0.28	3.54
Weathered Bedrock	26	38°	0.24	4.2
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.

Shoring design must take into account the potential effects of swelling of the shale bedrock following excavation, as further discussed in Section 5.6.1 of this report.

5.3 Anticipated Groundwater Management

The groundwater level in the monitoring wells ranged from Elevation 64.7 to 79.3 masl. It is understood that the P8 FFE will be set at Elevation 58.551 masl. As such excavations are anticipated to extend below the prevailing groundwater level and into the weathered to sound Shale bedrock.

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

The volume of water to be anticipated to flow from the bedrock into open excavations will depend on the discontinuities in the rock mass, for instance, fracture, fissure, etc. and therefore, may vary from location to location. The earth fill materials and native sandy deposits are however considered moderate to high permeability soils, and will permit the free-flow of water when wet. This seepage may be allowed to drain into the excavation and dewatered through use of conventional sump pump arrangements at the base of the excavation. Issues of delay in excavation due to localized seepage control must be addressed in the excavation contract.

A dewatering system installed by a specialist dewatering contractor may be required to lower the groundwater level prior to and during excavations. 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 0.30 m below the excavation base.

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 inflow 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.

5.4 Foundation Design

It is understood that the P8 FFE will be set at Elevation 58.551 masl (approximately 22.5 to 24.5 m below existing grade).

Weathered shale bedrock was inferred from split-soon sampling and/or auger refusal in boreholes BH1, BH2 and BH7. 'HQ' bedrock coring was completed in Boreholes BH1, BH2 and BH7, and confirmed Shale bedrock with limestone interbeds up to 200 mm in thickness of the Georgian Bay formation. Bedrock was encountered in boreholes BH1, BH2 and BH7 at depths ranging from approximately 6.5 to 8.0 mbgs (elevations ranging from approximately 75.2 to 75.0 masl)

The current boreholes did not extend to the proposed footing depth. At the borehole termination, the subsurface conditions comprised good to excellent quality shale bedrock. Additional deeper boreholes will be needed to support detailed design of this project. For preliminary design purposes, it can be assumed that footings placed on sound bedrock below Elevation 58.551 masl, can be sized for bearing resistances in the order of 3.5 MPa at Serviceability Limit States (SLS) and factored geotechnical bearing resistance of 5 MPa at Ultimate Limit States (ULS).

The bearing resistance values provided assumes the bedrock is cleaned of debris 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.

Prior to installing foundation formwork, and after cleaning, the bedrock is to be inspected by a geotechnical engineer to ensure slightly weathered to fresh bedrock consistent with the findings of this report.

The bedrock surface is to be relatively level with slopes not exceeding 5 degrees from the horizontal. Shale bedrock can weather when exposed to air or water. It is therefore recommended that a working slab of lean concrete (mud slab) be placed in the footing excavations immediately after excavation and inspection to protect the shale bedrock from weathering. All concrete should be installed and maintained above freezing temperatures as required by the concrete supplier.

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

5.4.1 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.

5.5 Site Classification for Seismic Site Response & Soil Behaviour

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

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

The boreholes advanced at this Site extended to approximately 2.0 to 20.1 mbgs. SPT “N” values within the native soil deposits ranged between 7 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 248 and 734 m/s. There is a potential that the Site Class may be higher; however, shear wave velocity measurements would be required for the determination of a higher Site Classification, as per the OBC.

5.6 Underground Parking Garage Design

The proposed development will rest on eight levels of underground parking. It is understood that the P8 FFE is to be set at Elevation 58.551 masl (approximately 22.5 to 24.5 m below existing grade).

The proposed development will have to be designed to resist hydrostatic uplift or 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 recommendations for the dewatering volumes during operation of the building will be provided within the Hydrogeological Assessment Report.

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/m3)

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 and thickness of the raft slab or by installing soil anchors.

Where open-cut method will be used for excavations, the basement wall backfill for a minimum lateral distance of 0.6 m out from the wall should consist of free-draining granular material (OPSS.MUNI 1010 Granular B), or provided with a prefabricated drain material (for instance, CCW MiraDRAIN 6000 series or Terrafix Terradrain 600).

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. 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 system.

An underfloor drainage system is recommended if the building is not designed as water tight and resistant to uplift. 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.

For calculating the lateral earth pressure on the below grade walls, following values should be used. Hydrostatic forces still need to be considered.

Soil Layer	Bulk Unit Weight (kN/m^3)	At-Rest Earth Pressure Coefficient
Earth Fill	18	0.53
Sand	18	0.47
Glacial Till	21	0.44
Weathered Bedrock	26	0.38
Slightly Weathered to Fresh Bedrock	26	0.31

5.6.1 Rock Pressure

The empirical approach for the design of foundation walls below bedrock level has been to use a uniform pressure distribution for the design of the basement walls below the top of bedrock elevation, which is consistent with the maximum earth pressure calculated for the lowest level of soil in the profile. This approach is likely conservative but it recognizes the practical requirement to have a foundation wall of a consistent width through the lower reach of the building.

This approach does not recognize the potential for pressures on the basement wall due to time dependant rock swell that results when locked in horizontal stresses are released. It presupposes that there is sufficient time between the cutting of the rock face and the construction of the building structure to allow the rock to de-stress and swell. Experience suggests that if there is a 120-day period after the rock cut, before the rock is restrained by the structure, that there has been sufficient swell and no significant stresses are imposed on the structural wall. Depending on the building construction sequence some provision for compressible material at the excavation perimeter (particularly for excavations extending deeper into the rock) may be necessary, which should be assessed during the detailed design stage.

For the lower foundation walls and where pits are made for sumps, elevators or other such features are cast directly against the rock face, there must be careful consideration of the potential for rock squeeze effects. To accommodate the rock squeeze effect, a compressible layer can be placed between the rock and the concrete. Compressible material such as 220 Ethafoam Polyethylene Foam planks are typically used in this application. Foundation walls are typically designed for the strength of the foam at the 50 percent compressive deflection. At 50 percent compressive deflection, 220 Ethafoam plank material will provide a resistance of 18 psi (124 kPa). The 10 percent deflection compressive strength of this material is 7 psi (50 kPa), which will allow for concrete placement.

In the case of sumps, elevators, etc., if the rock is over excavated by at least 600 mm and the pits and sumps are backfilled with 19 mm clear stone (OPSS.MUNI 1004), then there may be sufficient give in the backfill to accommodate the rock swell.

There is potential for vertical swelling of shale at the bottom of excavation. Additional boreholes, extending below the bottom of the proposed excavation, must be completed in order to provide information that will allow for assessment of potential for vertical swelling.

5.7 Floor Slabs

A conventional slab-on-grade basement floor may be installed on the underlying shale bedrock. Prior to the installation of the slab, all deleterious or loose materials should be removed.

For levelling purposes, it is recommended to establish the concrete floor slab on a minimum 200 mm thick layer of 19 mm clear stone Type I or Type II (OPSS 1004) or Granular “A” (OPSS 1010) compacted to 100% SPMDSS.

The following table provides the unfactored modulus of subgrade reaction values for use in design of the floor slab:

Material Type	Modulus of Subgrade Reaction (kN/m^3)
19 mm clear stone Type I or Type II or Granular "A" Material	35,000
Shale Bedrock	80,000

The values presented above are for a loaded area of 0.3 m by 0.3 m.

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, asphalt, 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 MPCT DIF 70 Park Street East LP (Client) in order to evaluate the subsurface conditions at 23, 25, 27, 29 and 31 Helene Street North, 53 Queen Street East and Part of 70 Park Street East, 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 V, Report Limitations and Guidelines for Use, which pertains to this report.

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

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

\\Pinchin.com\Miss\Job\314000s\0314281.000 Dream,HeleneStN,QueenStE,EDR,Cons\0314281.002 Dream,HeleneStN,QueenStE,EDR,GEO\Deliverables\314281.002 FINAL Prelim Geotech 70 Park Street Mississauga ON Jan 18 2023.docx
Template: Master Geotechnical Investigation Report – Ontario, GEO, April 1, 2020

FIGURES

PROJECT NAME:	GEOTECHNICAL INVESTIGATION
CLIENT NAME:	MPCT DIF 70 PARK STREET EAST LP
PROJECT LOCATION:	23, 25, 27, 29 AND 31 HELENE STREET NORTH, 53 QUEEN STREET EAST AND PART OF 70 PARK STREET EAST, MISSISSAUGA, ONTARIO
FIGURE NAME:	KEY MAP
FIGURE NUMBER	1

PROJECT NUMBER:	SCALE:	DRAWN BY:	REVIEWED BY:	DATE:
314281.002	1:22,000	KP	VM	DEC. 2022

PINCHIN

N

EGEND

SITE BOUNDARY
SITE BUILDING
RAILWAY LINE
MTR MULTI-TENANT RESIDENTIAL
COM COMMERCIAL
RES RESIDENTIAL
ASSUMED EXTENT OF NEW PARKING GARAGE
BOREHOLE
MONITORING WELL

EGEND IS COLOUR DEPENDENT. ON-COLOUR COPIES MAY ALTER NTERPRETATION.

PINCHIN

ROJECT NAME:
GEOTECHNICAL INVESTIGATION

LIENT NAME:
MPCT DIF 70 PARK STREET EAST LP

ROJECT LOCATION:
3, 25, 27, 29 AND 31 HELENE STREET NORTH, 53 QUEEN STREET EAST AND PART OF 70 PARK STREET EAST, MISSISSAUGA, ONTARIO

IGURE NAME:
BOREHOLE AND MONITORING WELL LOCATION PLAN

ROJECT NUMBER: 314281.002	SCALE: AS SHOWN
RAWN BY: SIN	REVIEWED BY: VM
DATE: DEC. 2022	FIGURE NUMBER: 2

APPENDIX I

Abbreviations, Terminology and Principle Symbols used in Report

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:

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

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

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

APPENDIX II
Pinchin's Borehole Logs

Log of Borehole: BH1(MW)

Project #: 314281.002 **Logged By:** SL

Project: Geotechnical Investigation Proposed Development

Client: MPCT DIF 70 Park Street East LP

Location: Queen Street East and Helene Street North, Mississauga

Drill Date: October 11, 2022 **Project Manager:** MYB

SUBSURFACE PROFILE

Depth (m)	Description	Elevation (m)
	Ground Surface	82.30
0.00	**Topsoil** Approximately 130 mm	82.30
	Fill Brown sandy silt, trace clay, trace gravel, trace rootlets, loose, moist	
1.52	**Sand** Brown sand and silt, compact to dense, moist	80.78
3.05	**Silt Till** Grey clayey silt till, trace to some sand, trace gravel, trace stone fragments, very stiff to hard, APL	79.25
7.14		75.16
8.05		74.25

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

SAMPLE

Sample Type	Sampler #	Recovery (%)	SPT N-Value	Water Content (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
SS	1	65	8	5.8	BH1 SS1	0/1	
SS	2	85	8	18.4	BH1 SS2	0/3	
SS	3	90	29	15.7	BH1 SS3	0/7	
SS	4	90	43	13.2	BH1 SS4	0/9	
SS	5	45	17	10.0	BH1 SS5	0/0	
SS	6	90	28	9.6	BH1 SS6	0/125	
SS	7	100	>50	10.3	BH1 SS7	0/225	
HQ	R1	100					
HQ	R2	100					

Standard Penetration N-Value: 20 40 60

Shear Strength kPa: 100 200

Contractor: TEC Geological Drilling Inc. **Grade Elevation:** 82.3 masl

Drilling Method: Split Spoon / Solid Stem Auger, HQ-Rock Coring **Top of Casing Elevation:** 82.3 masl

Well Casing Size: 51 mm **Sheet:** 1 of 3

Log of Borehole: BH1(MW)

Project #: 314281.002 **Logged By:** SL

Project: Geotechnical Investigation Proposed Development

Client: MPCT DIF 70 Park Street East LP

Location: Queen Street East and Helene Street North, Mississauga

Drill Date: October 11, 2022 **Project Manager:** MYB

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 (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
		Georgian Bay Formation Grey shale, very thinly bedded to thinly bedded, weak, joints are horizontal, closed, planar; interbedded with limestone, light, grey, strong	72.73 / 9.57											
10		Total Core Recovery: 100% Solid Core Recovery: 67% Rock Quality Designation: 26% ~1% Limestone		Bentonite	HQ	R3	100							
11		Total Core Recovery: 100% Solid Core Recovery: 95% Rock Quality Designation: 84% ~1% Limestone	71.20 / 11.09		HQ	R4	98							
12		Total Core Recovery: 100% Solid Core Recovery: 58% Rock Quality Designation: 70% ~2% Limestone	69.74 / 12.56		HQ	R5	100							
13		Total Core Recovery: 98% Solid Core Recovery: 60% Rock Quality Designation: 72% ~3% Limestone		Riser										
14		Total Core Recovery: 100% Solid Core Recovery: 95% Rock Quality Designation: 97% ~2% Limestone	68.16 / 14.14		HQ	R6	98							
15		Total Core Recovery: 98% Solid Core Recovery: 80% Rock Quality Designation: 88% ~2% Limestone	66.63 / 15.67	Silica Sand	HQ	R7	98							
16–17		Total Core Recovery: 98% Solid Core Recovery: 85% Rock Quality Designation: 93% ~2% Limestone	65.14 / 17.16		HQ	R8	100							
18														

Contractor: TEC Geological Drilling Inc. **Grade Elevation:** 82.3 masl

Drilling Method: Split Spoon / Solid Stem Auger, HQ-Rock Coring **Top of Casing Elevation:** 82.3 masl

Well Casing Size: 51 mm **Sheet:** 2 of 3

Log of Borehole: BH1(MW)

Project #: 314281.002 **Logged By:** SL

Project: Geotechnical Investigation Proposed Development

Client: MPCT DIF 70 Park Street East LP

Location: Queen Street East and Helene Street North, Mississauga

Drill Date: October 11, 2022 **Project Manager:** MYB

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 (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
19		Total Core Recovery: 100% Solid Core Recovery: 82% Rock Quality Designation: 80% ~1% Limestone	63.71 18.59	Screen	HQ	R9	96							
20		Total Core Recovery: 96% Solid Core Recovery: 100% Rock Quality Designation: 96% ~1% Limestone	62.55 19.75											
		End of Borehole												
21–27		Borehole terminated at approximately 19.8 mbgs. Borehole contained drill water upon completion of drilling. Unstabilized groundwater level and cave were not measured due to the presence of drill fluid. Water Level Readings Date Water Depth (mgs) D / S Oct 18, 2022 17.6 / Dry Oct 24, 2022 17.6 / Dry Oct 28, 2022 17.3 / Dry												

Contractor: TEC Geological Drilling Inc. **Grade Elevation:** 82.3 masl

Drilling Method: Split Spoon / Solid Stem Auger, HQ-Rock Coring **Top of Casing Elevation:** 82.3 masl

Well Casing Size: 51 mm

Log of Borehole: BH2(MW)

Project #: 314281.002 **Logged By:** SL

Project: Geotechnical Investigation Proposed Development

Client: MPCT DIF 70 Park Street East LP

Location: Queen Street East and Helene Street North, Mississauga

Drill Date: October 11, 2022 **Project Manager:** MYB

SUBSURFACE PROFILE

Depth (m)	Description	Elevation (m)	Monitoring Well Details
0	Ground Surface	83.00 0.00	Riser
	Topsoil Approximately 130 mm		Bentonite
1	**Fill** Brown sandy silt, trace clay, trace gravel, trace organics, compact, moist	81.47 1.52	
2	**Sand** Brown sand and silt, trace clay, trace gravel, loose, moist	80.71 2.29	
3	**Silt Till** Brown clayey silt till, some sand to sandy, trace gravel, stiff to hard, APL	79.19 3.81	Screen
4	grey below		Silica Sand
5–7		75.00 8.00	
8–9			

SAMPLE

Sample Type	Sampler #	Recovery (%)	SPT N-Value	Water Content (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
SS	1	75	14	6.6	BH2 SS1	0/0	
SS	2	65	12	14.7	BH2 SS2	0/20	
SS	3	90	7	21.3	BH2 SS3	0/25	
SS	4	100	12	24.8	BH2 SS4	0/23	
SS	5	70	13	12.1	BH2 SS5	0/85	
SS	6	80	35	9.7	BH2 SS6	0/77	
SS	7	100	33	8.6	BH2 SS7	0/115	
SS	8	60	32	9.9	BH2 SS8	0/169	
SS	9	100	>50	12.2	BH2 SS9	0/115	
HQ	R1	100					

Standard Penetration N-Value: 20 40 60

Shear Strength kPa: 100 200

Contractor: TEC Geological Drilling Inc. **Grade Elevation:** 83.0 masl

Drilling Method: Split Spoon / Solid Stem Auger, HQ-Rock Coring **Top of Casing Elevation:** 83.0 masl

Well Casing Size: 51 mm **Sheet:** 1 of 3

Log of Borehole: BH2(MW)

Project #: 314281.002 **Logged By:** SL

Project: Geotechnical Investigation Proposed Development

Client: MPCT DIF 70 Park Street East LP

Location: Queen Street East and Helene Street North, Mississauga

Drill Date: October 11, 2022 **Project Manager:** MYB

Depth (m)	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-Value	Standard Penetration N-Value (20 40 60)	Shear Strength kPa (100 200)	Water Content (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
	Georgian Bay Formation	73.49											
10	Grey shale, very thinly bedded to thinly bedded, weak, joints are horizontal, closed, planar; interbedded with limestone, light, grey, strong Total Core Recovery: 100% Solid Core Recovery: 58% Rock Quality Designation: 61% ~1% Limestone	9.51	Bentonite	HQ	R2	100							
11–12	Total Core Recovery: 100% Solid Core Recovery: 88% Rock Quality Designation: 78% ~3% Limestone	71.96 11.03		HQ	R3	100							
13	Total Core Recovery: 100% Solid Core Recovery: 71% Rock Quality Designation: 36% ~2% Limestone	70.47 12.53		HQ	R4	100							
14–15	Total Core Recovery: 100% Solid Core Recovery: 100% Rock Quality Designation: 97% ~1% Limestone	68.95 14.05	Riser	HQ	R5	100							
15	Total Core Recovery: 100% Solid Core Recovery: 82% Rock Quality Designation: 82% ~3% Limestone												
16	Total Core Recovery: 98% Solid Core Recovery: 100% Rock Quality Designation: 92% ~4% Limestone	67.42 15.58	Silica Sand	HQ	R6	98							
17–18	Total Core Recovery: 100% Solid Core Recovery: 86% Rock Quality Designation: 88% ~3% Limestone	65.93 17.07		HQ	R7	100							

Contractor: TEC Geological Drilling Inc. **Grade Elevation:** 83.0 masl

Drilling Method: Split Spoon / Solid Stem Auger, HQ-Rock Coring **Top of Casing Elevation:** 83.0 masl

Well Casing Size: 51 mm

Log of Borehole: BH2(MW)

Project #: 314281.002 **Logged By:** SL

Project: Geotechnical Investigation Proposed Development

Client: MPCT DIF 70 Park Street East LP

Location: Queen Street East and Helene Street North, Mississauga

Drill Date: October 11, 2022 **Project Manager:** MYB

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 (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
			64.43											
19		Total Core Recovery: 100% Solid Core Recovery: 77% Rock Quality Designation: 91% ~1% Limestone	18.56	Screen	HQ	R8	100							
20		End of Borehole	62.97 20.03											
21–27		Borehole terminated at approximately 20.0 mbgs. Borehole contained drill water upon completion of drilling. Unstabilized groundwater level and cave were not measured due to the presence of drill fluid. Water Level Readings Date Water Depth (mgs) D / S Oct 18, 2022 17.5 / Dry Oct 24, 2022 17.0 / Dry Oct 28, 2022 17.0 / Dry												

Contractor: TEC Geological Drilling Inc. **Grade Elevation:** 83.0 masl

Drilling Method: Split Spoon / Solid Stem Auger, HQ-Rock Coring **Top of Casing Elevation:** 83.0 masl

Well Casing Size: 51 mm **Sheet:** 3 of 3

Log of Borehole: BH3

Project #: 314281.002 **Logged By:** SL

Project: Geotechnical Investigation Proposed Development

Client: MPCT DIF 70 Park Street East LP

Location: Queen Street East and Helene Street North, Mississauga

Drill Date: October 13, 2022 **Project Manager:** MYB

Depth (m)	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-Value	Water Content (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
	Ground Surface	81.69									
0	**Topsoil** Approximately 130 mm	0.00	No Monitoring Well Installed	GS	1			N/A			
	Fill Brown silty sand, trace clay, trace gravel, trace rootlets, loose, moist			SS	2	75	9	5.8			
	Silt Till Brown clayey silt till, trace to some sand, trace gravel, stiff to very stiff, APL	80.92 / 0.76		SS	3	100	13	13.2			
				SS	4	55	22	10.2			
2	End of Borehole	79.71 / 1.98									

Borehole terminated at approximately 2.0 mbgs. Borehole was open and dry upon completion of drilling.

Standard Penetration N-Value (20, 40, 60); Shear Strength kPa (100, 200)

Contractor: TEC Geological Drilling Inc. **Grade Elevation:** 81.7 masl

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

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

Log of Borehole: BH4

Project #: 314281.002 **Logged By:** SL

Project: Geotechnical Investigation Proposed Development

Client: MPCT DIF 70 Park Street East LP

Location: Queen Street East and Helene Street North, Mississauga

Drill Date: October 13, 2022 **Project Manager:** MYB

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 (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
		Ground Surface	81.38											
0		**Topsoil** Approximately 130 mm	0.00	No Monitoring Well Installed	GS	1					N/A			
		Fill Brown silty sand, trace clay, trace gravel, trace rootlets, loose, moist			SS	2	100	8			8.7			
		Silt Till Brown clayey silt till, trace to some sand, trace gravel, firm to very stiff, APL	80.62 0.76		SS	3	100	8			15.5			
1–2					SS	4	100	24			14.4			
2		End of Borehole Borehole terminated at approximately 2.0 mbgs. Borehole was open and dry upon completion of drilling.	79.40 1.98											
3														
4														
5														
6														

Contractor: TEC Geological Drilling Inc. **Grade Elevation:** 81.4 masl

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

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

Log of Borehole: BH5(MW)

Project #: 314281.002 **Logged By:** SL

Project: Geotechnical Investigation Proposed Development

Client: MPCT DIF 70 Park Street East LP

Location: Queen Street East and Helene Street North, Mississauga

Drill Date: October 13, 2022 **Project Manager:** MYB

SUBSURFACE PROFILE

Depth (m)	Description	Elevation (m)
0	Ground Surface	81.08
0.00	**Concrete Slab** Approximately 130 mm	81.08 / 0.00
0.76	**Fill** Brown sandy silt, trace clay, trace gravel, trace styrofoam, loose, moist	80.31 / 0.76
	Sand Brown sandy silt, trace clay, trace gravel, compact to very dense, moist	
2.29	**Silt Till** Brown clayey silt till, some sand to sandy, trace gravel, hard, APL	78.79 / 2.29
3.05	grey below	78.03 / 3.05
4.88	End of Borehole	76.20 / 4.88

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

Borehole terminated at approximately 4.9 mbfs. Borehole was open and dry upon completion of drilling.

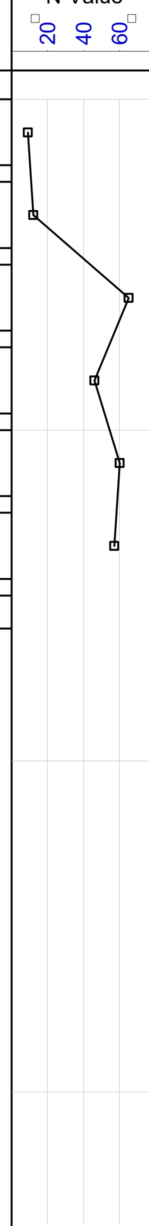
Water Level Readings

Date	Water Depth (mbfs)
Oct 18, 2022	1.9
Oct 24, 2022	1.9
Oct 28, 2022	1.8

SAMPLE

Sample Type	Sampler #	Recovery (%)	SPT N-Value	Water Content (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
SS	1	85	9	15.9	BH5 SS1	0/2	
SS	2	80	12	20.8	BH5 SS2	0/2	
SS	3	100	65	13.5	BH5 SS3	0/0	
SS	4	75	46	10.6	BH5 SS4	0/1	
SS	5	80	60	7.4	BH5 SS5	0/0	
SS	6	60	57	9.5	BH5 SS6	0/4	
SS	7	100	>50	11.6	BH5 SS7	0/11	

Standard Penetration N-Value



Shear Strength kPa 100 200

Contractor: Strata Drilling Group **Grade Elevation:** 81.1 masl

Drilling Method: Split Spoon, Direct Push / Solid Stem Auger **Top of Casing Elevation:** 81.1 masl

Well Casing Size: 32 mm **Sheet:** 1 of 1

Log of Borehole: BH6(MW)

Project #: 314281.002 **Logged By:** SL

Project: Geotechnical Investigation Proposed Development

Client: MPCT DIF 70 Park Street East LP

Location: Queen Street East and Helene Street North, Mississauga

Drill Date: October 14, 2022 **Project Manager:** MYB

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 (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
0		Ground Surface	81.11											
		Concrete Slab Approximately 100 mm	0.00		SS	1	50				19.0			
1		**Fill** Brown sandy silt, trace clay, trace gravel, trace styrofoam, moist	80.35 0.76	Riser, Bentonite	SS	2	100				22.5			
2		**Silt Till** Grey clayey silt till, some sand to sandy, trace gravel, APL		Screen, Silica Sand	SS	3	100				19.4			
					SS	4	100				10.5			
3					AS	5	N/A				27.7			
					AS	6	N/A				27.9			
4			77.14		AS	7	N/A				27.7			
		End of Borehole	3.96											

Borehole terminated at approximately 4.0 mbfs.

Water Level Readings

Date	Water Depth (mbfs)
Oct 18, 2022	Dry
Oct 24, 2022	Dry
Oct 28, 2022	Dry

Contractor: Strata Drilling Group **Grade Elevation:** 81.1 masl

Drilling Method: Direct Push / Solid Stem Auger **Top of Casing Elevation:** 81.1 masl

Well Casing Size: 51 mm **Sheet:** 1 of 1

Log of Borehole: BH7(MW)

Project #: 314281.002 **Logged By:** SL

Project: Geotechnical Investigation Proposed Development

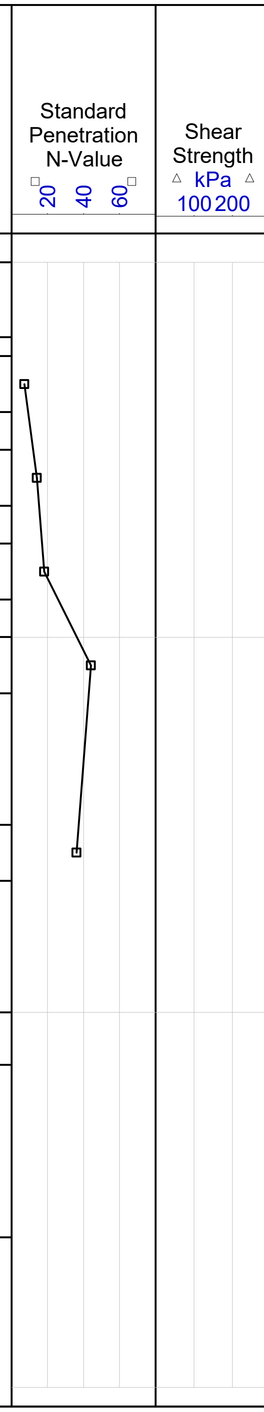
Client: MPCT DIF 70 Park Street East LP

Location: Queen Street East and Helene Street North, Mississauga

Drill Date: October 17, 2022 **Project Manager:** MYB

Depth (m)	Description (Subsurface Profile)	Elevation (m)	Sample Type	Sampler #	Recovery (%)	SPT N-Value	Water Content (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
0	Ground Surface	81.70 / 0.00								
0–0.13	**Topsoil** Approximately 130 mm		GS	1			N/A			
0.13–2.29	**Sand** Brown sand and silt, trace gravel, loose to compact, moist		SS	2	80	7	13.1			
			SS	3	100	14	25.9			
2.29–6.52	**Silt Till** Grey clayey silt till, trace to some sand, trace gravel, trace stone fragments, very stiff to hard, APL	79.42 / 2.29	SS	4	80	18	9.7			
			SS	5	100	44	8.0			
			SS	6	90	36	9.7			
			SS	7	100	>50	8.2			
6.52–	**Georgian Bay Formation** Grey shale, very thinly bedded to thinly bedded, weak, joints are horizontal, closed, planar; interbedded with limestone, light, grey, strong	75.18 / 6.52	HQ	R1	100					
7.92–	Total Core Recovery: 100% Solid Core Recovery: 45% Rock Quality Designation: 58% ~1% Limestone	73.78 / 7.92	HQ	R2	90					

Monitoring Well Details: Bentonite; Riser



Contractor: TEC Geological Drilling Inc. **Grade Elevation:** 81.7 masl

Drilling Method: Split Spoon / Solid Stem Auger, HQ-Rock Coring **Top of Casing Elevation:** 81.7 masl

Well Casing Size: 51 mm **Sheet:** 1 of 3

Log of Borehole: BH7(MW)

Project #: 314281.002 **Logged By:** SL

Project: Geotechnical Investigation Proposed Development

Client: MPCT DIF 70 Park Street East LP

Location: Queen Street East and Helene Street North, Mississauga

Drill Date: October 17, 2022 **Project Manager:** MYB

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 (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
		Total Core Recovery: 90% Solid Core Recovery: 45% Rock Quality Designation: 35% ~3% Limestone	72.25 9.45											
10		Total Core Recovery: 100% Solid Core Recovery: 30% Rock Quality Designation: 20% ~5% Limestone	70.73 10.97		HQ	R3	100							
11–12		Total Core Recovery: 98% Solid Core Recovery: 70% Rock Quality Designation: 84% ~2% Limestone	69.27 12.44		HQ	R4	98							
13		Total Core Recovery: 100% Solid Core Recovery: 97% Rock Quality Designation: 97% ~3% Limestone	67.68 14.02		HQ	R5	100							
14–15		Total Core Recovery: 100% Solid Core Recovery: 100% Rock Quality Designation: 100% ~3% Limestone	66.22 15.48		HQ	R6	100							
16		Total Core Recovery: 100% Solid Core Recovery: 73% Rock Quality Designation: 75% ~4% Limestone	64.69 17.01	Silica Sand	HQ	R7	100							
17–18		Total Core Recovery: 100% Solid Core Recovery: 92% Rock Quality Designation: 76% ~2% Limestone		Screen	HQ	R8	100							

Contractor: TEC Geological Drilling Inc. **Grade Elevation:** 81.7 masl

Drilling Method: Split Spoon / Solid Stem Auger, HQ-Rock Coring **Top of Casing Elevation:** 81.7 masl

Well Casing Size: 51 mm

Log of Borehole: BH7(MW)

Project #: 314281.002 **Logged By:** SL

Project: Geotechnical Investigation Proposed Development

Client: MPCT DIF 70 Park Street East LP

Location: Queen Street East and Helene Street North, Mississauga

Drill Date: October 17, 2022 **Project Manager:** MYB

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 (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
			63.20 / 18.50	Screen										
19		Total Core Recovery: 100% Solid Core Recovery: 95% Rock Quality Designation: 90% ~3% Limestone			HQ	R9	100							
20		End of Borehole	61.65 / 20.06											
21–27		Borehole terminated at approximately 20.1 mbgs. Borehole contained drill water upon completion of drilling. Unstabilized groundwater level and cave were not measured due to the presence of drill fluid. Water Level Readings Date Water Depth (mbgs) Oct 18, 2022 6.7 Oct 24, 2022 11.2 Oct 28, 2022 9.6												

Contractor: TEC Geological Drilling Inc. **Grade Elevation:** 81.7 masl

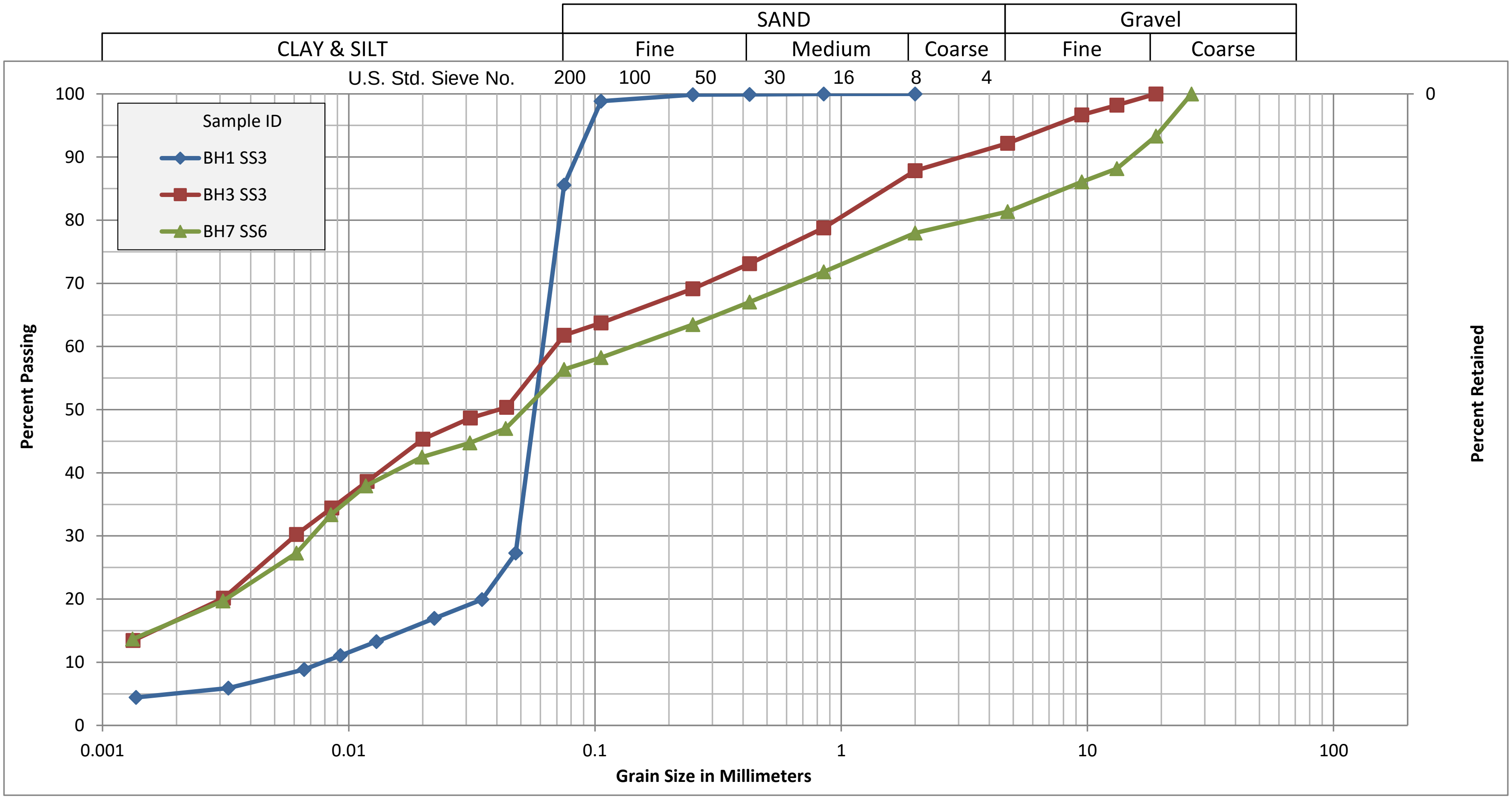
Drilling Method: Split Spoon / Solid Stem Auger, HQ-Rock Coring **Top of Casing Elevation:** 81.7 masl

Well Casing Size: 51 mm

APPENDIX III

Geotechnical Laboratory Testing Reports for Soil Samples

Unified Soil Classification System



Sample ID	Depth (ft)	% Gravel	% Sand	% Silt	% Clay
BH1 SS3	5.0-7.0	0.0	14.4	80.6	5.0
BH3 SS3	2.5-4.5	8.0	30.2	45.8	16.0
BH7 SS6	15.0-17.0	19.0	24.6	40.4	16.0

PINCHIN

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

PARTICLE SIZE DISTRIBUTION ANALYSIS

Geotechnical Investigation - Queen St E and Helene St N, Mississauga, ON
Dream Office LP

Figure No. 1

314281.002

Reviewed By:

More information available upon request

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.