FINAL

Preliminary Geotechnical Investigation – Proposed Residential Development

2105, 2087, 2097 and 2077 Royal Windsor Drive, Mississauga, Ontario

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TABLE OF CONTENTS

1.0 INTRODUCTION AND SCOPE 1
2.0 SITE DESCRIPTION AND GEOLOGICAL SETTING 2
3.0 GEOTECHNICAL FIELD INVESTIGATION AND METHODOLOGY 2
4.0 SUBSURFACE CONDITIONS 3
4.1 Borehole Soil Stratigraphy 3
4.2 Bedrock 4
4.3 Groundwater Conditions 4
5.0 PRELIMINARY GEOTECHNICAL DESIGN RECOMMENDATIONS 5
5.1 General Information 5
5.2 Open Cut Excavations 6
5.2.1 Shoring Requirements 6
5.3 Anticipated Groundwater Management 8
5.4 Foundation Design 9
5.4.1 Shallow Foundations Bearing on Bedrock 9
5.4.2 Site Classification for Seismic Site Response & Soil Behaviour 10
5.4.3 Estimated Settlement 11
5.4.4 Building Drainage 11
5.4.5 Shallow Foundations Frost Protection & Foundation Backfill 11
5.5 Underground Parking Garage Design 11
5.5.1 Rock Pressure 13
5.6 Floor Slabs 14
6.0 SITE SUPERVISION & QUALITY CONTROL 15
7.0 TERMS AND LIMITATIONS 15

FIGURES

Figure 1	Key Map
Figure 2	Borehole Location Plan

APPENDICES

APPENDIX I	Abbreviations, Terminology and Principle Symbols used in Report and Borehole Logs
APPENDIX II	Pinchin’s Borehole Logs
APPENDIX III	Laboratory Testing Reports for Soil Samples
APPENDIX IV	Report Limitations and Guidelines for Use

1.0 INTRODUCTION AND SCOPE

Pinchin Ltd. (Pinchin) was retained by CRW 1 LP and CRW 2 LP (Client) to conduct a Preliminary Geotechnical Investigation and provide subsequent geotechnical design recommendations for the land development to be located at 2105, 2087, 2097 and 2077 Royal Windsor Drive, Mississauga, Ontario (Site). The Site location is shown on Figure 1.

Based on information provided by the Client, it is Pinchin's understanding that the Site will be redeveloped with a mixed- use development. The west block of the development consists of two high-rise residential buildings connected by an 8-storey podium with retail and live/ work units at grade, and the east block of the development consists of two high-rise residential buildings connected by an 8-storey podium with retail and live/work units at grade. There are approximately 5 levels of underground parking (UPG) proposed on the west block and 3 levels of underground parking on the east block.

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

The purpose of the Preliminary Geotechnical Investigation was to delineate the subsurface conditions and soil engineering characteristics by advancing a total of nine (9) sampled boreholes at the Site.

Pinchin is also currently completing a Hydrogeological Assessment of the Site. The results of the Hydrogeological Assessment of the Site will be provided under a separate cover.

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;
- Lateral earth pressure coefficients and unit densities;
- Foundation design recommendations including soil 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;
- Asphaltic concrete pavement structure design for parking areas and access roadways; 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 north of Royal Windsor Drive, and west of Southdown Road, in Mississauga, Ontario. The Site is currently occupied by four one-storey brick buildings, associated asphalt parking lots, and a private road, with an easement in favour of Metrolinx, to provide access to Royal Windsor Drive to the south and Clarkson GO Station to the north.

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 on fine-textured glaciolacustrine deposits of silt and clay, minor sand and gravel (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 Queenston formation consisting of shale (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 field investigations at the Site between August 2 and August 5, 2022 by advancing a total of nine (9) sampled boreholes (Boreholes BH22-1 to BH22-9) throughout the Site. The boreholes were advanced to depths of approximately 4.6 to 12.6 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 track mounted 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 soil.

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. The approximate spatial locations of the monitoring wells installed at the Site are shown on Figure 2.

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.

The borehole locations and ground surface elevations were surveyed by Pinchin using a Trimble Model TSC5 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.

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 August 31, 2022. The groundwater observations and measurements recorded are included on the appended borehole logs.

The field investigation was monitored by experienced Pinchin personnel. Pinchin logged the drilling operations and identified the soil samples as they were retrieved. The recovered soil samples were sealed into plastic bags and carefully transported to Pinchin's 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 Pinchin's materials testing laboratory to determine the moisture content and grain size distribution of the soil. A copy of the laboratory analytical reports is included in Appendix III. In addition, the collected samples were compared against previous geotechnical information from the area, for consistency and calibration of results.

4.0 SUBSURFACE CONDITIONS

4.1 Borehole Soil Stratigraphy

In general, the soil stratigraphy at the Site comprises fill material overlying silt deposits, and shale bedrock. The appended borehole logs provide detailed soil descriptions and stratigraphies, results of SPT testing, moisture content profiles, and groundwater measurements.

Asphaltic concrete was encountered surficially at all borehole locations and was approximately 50 to 100 mm thick. Fill material was encountered below the asphaltic concrete and extended to depths ranging from 0.3 to 1.5 mbgs within the borehole advanced in the current investigation. The fill material was generally comprised of sand and gravel. The fill material has a very loose to dense relative density based on SPT 'N' values of 4 to 30 blows per 300 mm penetration of a split spoon sampler. At the time of sampling, the fill material was generally moist.

Silt deposits were encountered below the fill material in all boreholes. The silt deposits extended to depths ranging between 2.4 and 3.4 mbgs. (Elevation 95.2 to 97.1 masl). The silt deposits varied in composition from clayey silt with trace sand and gravel to clayey silt. The silt deposits had a compact to very dense relative density based on SPT 'N' values of 11 to greater than 50 blows per 300 mm penetration of a split spoon sampler. The results of the Atterberg Limit test completed on a sample of the silt indicated a plastic limit of 22%, a liquid limit of 42%, and a plasticity index of 20%. The results indicate that the sample tested is of medium plasticity. At the time of drilling the silt was described as drier than the plastic limit.

The results of two particle size distribution analyses completed on samples of the silt are provided in Appendix III and are also presented in the following table:

Borehole and Sample No.	Sample Depth (mbgs)	% Gravel	% Sand	% Silt	% Clay
BH22-4 SS4	2.3 – 2.9	1	16	64	19
BH22-9 SS3	1.5 – 2.1	2	9	60	29

4.2 Bedrock

Bedrock was encountered in all boreholes at depths ranging from 2.3 to 3.4 mbgs (Elevation 95.2 to 97.1 masl).

The bedrock was proven by coring in Borehole BH22-4 and the Rock Quality Designation (RQD) was calculated for the recovered core samples and is summarized on the appended borehole logs. The upper 8 metres of the bedrock was highly weathered. The calculated RQD values ranging from 0 to 66% indicate that the bedrock classification based on the RQD is in the range of very poor to fair quality.

4.3 Groundwater Conditions

Groundwater observations and measurements were obtained in the open boreholes at the completion of drilling and are summarized on the appended borehole logs. In addition, groundwater levels were measured in the monitoring wells installed in Boreholes BH22-1 to BH22-9 on August 31, 2022. The measured groundwater levels are summarized below:

Borehole No.	Water Level (mbgs)	Water Elevation (masl)
BH22-1	2.15	97.05

Borehole No.	Water Level (mbgs)	Water Elevation (masl)
BH22-2	2.52	96.19
BH22-3	2.65	95.96
BH22-4	2.69	95.81
BH22-5	2.42	96.13
BH22-6	2.06	96.65
BH22-7	3.23	96.11
BH22-8	3.07	96.05
BH22-9	2.59	96.08

The groundwater level in the monitoring wells ranged from Elevation 95.8 to 97.1 masl. For geotechnical design purposes, the groundwater level may be taken at Elevation 97 ± 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 provided under a separate cover.

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 limited 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 Site will be redeveloped with a mixed- use development. The west block of the development consists of two high-rise residential buildings connected by an 8-storey podium with retail and live/ work units at grade, and the east block of the development consists of two high-rise residential buildings connected by an 8-storey podium with retail and live/work units at grade. There are approximately 5 levels of underground parking (UPG) proposed on the west block and 3 levels of underground parking on the east block.

5.2 Open Cut Excavations

It is anticipated that the foundations will be constructed at approximately 12 and 18 mbgs based on three and five levels of underground parking, respectively.

Based on the subsurface information obtained from within the boreholes, it is anticipated that the excavated material will predominately consist of granular fill, native clayey silt, and shale bedrock material. The groundwater level in the monitoring wells ranged from Elevation 95.8 to 97.1 masl. For geotechnical design purposes, the groundwater level may be taken at Elevation 97 ± masl.

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 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.

5.2.1 Shoring Requirements

It is anticipated that due to spatial limitations, it may not be feasible to slope the excavations back to a safe angle and therefore some temporary support 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, 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. For this Site, considerations must be given to incorporate a rigid shoring system (consisting of interlocking caissons or diaphragm wall) socketed into the underlying sound bedrock to preserve the integrity of the adjacent structures and support the soil in a state approximating at-rest conditions as well as provide reductions in groundwater flow. 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.

Resistance to sliding of retaining structures is developed by friction between the base of the footing and the soil. This friction (**R**) depends on the normal load on the soil contact (**N**) and the frictional resistance of the soil (**tan ϕ**) expressed as **R = N tan ϕ**. The factored geotechnical resistance at ULS is **0.8 R**.

Passive earth pressure resistance is generally not considered as a resisting force against sliding for conventional retaining structure design because a structure must deflect significantly to develop the full passive resistance.

The following preliminary parameters (un-factored) should be used for the preliminary/conceptual design of the shoring system. These preliminary results should be updated with a more detailed geotechnical field investigation. 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/m^3)	Angle of Internal Friction	Active Earth Pressure Coefficient	Passive Earth Pressure Coefficient
Earth Fill	18	27°	0.38	2.66
Silt	20	32°	0.31	0.47
Georgian Bay Formation Bedrock	26	26°	N/A	N/A

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.

5.3 Anticipated Groundwater Management

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

The groundwater level in the monitoring wells ranged from Elevation 95.8 to 97.1 masl. For geotechnical design purposes, the groundwater level may be taken at Elevation 97 ± masl.

It is anticipated that the underside of footing may be set at 12 to 18 mbgs (i.e. Elevation 86 and 80 ± masl). As such excavations are anticipated to extend below the prevailing groundwater level.

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.

5.4 Foundation Design

5.4.1 Shallow Foundations Bearing on Bedrock

It is anticipated that the underside of footing may be set at 12 mbgs (i.e. Elevation 86 ± masl) for three levels of UPG and at 18 mbgs (i.e. Elevation 80 ± masl) for five levels of UPG. Inferred shale bedrock was encountered at depths ranging from 2.3 to 3.4 mbgs (Elevation 95.2 to 97.1 masl). The upper 8.0 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.

A summary of properties with respect to the shale within the Georgian Bay Formation was presented in the Ontario Ministry of Transportation and Communications document RR229, *Evaluation of Shales for Construction Projects* (March 1983), as follows:

	Uniaxial Compressive Strength (MPa)	Young's Modulus (GPa)	Dynamic Modulus (GPa)	Poisson's Ratio
Average	28	4	19	0.19
Range	8 to 41	0.5 to 12	6 to 38	0.1 to 0.25

Below Elevation 86 ± masl, spread footings established on the slightly weathered to sound bedrock may be preliminarily designed using the following bearing resistance for 25 mm of settlement at Serviceability Limit States (SLS) and factored geotechnical bearing resistance at Ultimate Limit States (ULS). Coring of the bedrock will be required to confirm these design bearing resistances for detailed design.

Spread Footing Size	SLS	ULS
1 m x 1 m	5.0 MPa	5.0 MPa
2 m x 2 m	5.0 MPa	5.0 MPa
3 m x 3 m	3.75 MPa	5.0 MPa

Spread Footing Size	SLS	ULS
4 m x 4 m	2.75 MPa	5.0 MPa
Elevator Core Raft Foundations up to 40 m X 20 m	2.5 MPa	5.0 MPa

The preliminary 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 the clean slightly weathered to 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 Pinchin to ensure the slightly weathered to fresh bedrock is 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 150 mm thick layer 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.

5.4.2 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 5 to 12 mbgs and were terminated in the bedrock. SPT “N” values within the overburden ranged between 4 and greater than 50 blows per 300 mm. As such, based on Table 4.1.8.4.A of the OBC, this Site has been classified as Class 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 at the Site.

5.4.3 *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.

5.4.4 *Building Drainage*

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

5.4.5 *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.

5.5 Underground Parking Garage Design

It is understood that the buildings are proposed to be constructed with three to five levels of underground parking. It is anticipated that the underside of footing may be set at 12 to 18 mbgs (i.e. Elevation 86 and 80 ± masl). The groundwater level in the monitoring wells ranged from Elevation 95.8 to 97.1 masl. For geotechnical design purposes, the groundwater level may be taken at Elevation 97 ± masl.

As such, the proposed development will 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. The building will need to be designed as a water tight structure if no drainage systems are installed.

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.

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 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.

5.5.1 Rock Pressure

It is Pinchin's understanding that the buildings will be constructed with three to five levels of underground parking. It is anticipated that the underside of footing may be set at 12 mbgs (i.e. Elevation 86 ± masl) for three levels of UPG and at 18 mbgs (i.e. Elevation 80 ± masl) for five levels of UPG. Inferred shale bedrock was encountered at depths ranging from 2.3 to 3.4 mbgs (Elevation 95.2 to 97.1 masl). Therefore, portions of the underground parking garage excavation are anticipated to extend into the bedrock.

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. A 50 mm thick 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 is sufficient give in the backfill to accommodate the rock swell.

5.6 Floor Slabs

It is understood that the building is proposed to be constructed with three to five levels of underground parking, which will extend into the shale bedrock.

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. removed. BasedBased 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) compacted to 100% SPMDD. Alternatively, consideration may also be given to using a 200 mm thick layer of uniformly compacted 19 mm clear stone placed over the approved subgrade. Any required up fill should consist of a Granular “B” Type I or Type II (OPSS 1010).

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

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

Material Type	Modulus of Subgrade Reaction (kN/m^3)
Granular A (OPSS 1010)	85,000
Granular “B” Type I (OPSS 1010)	75,000
Granular “B” Type II (OPSS 1010)	85,000
Shale bedrock	100,000

These values are for a 0.3 by 0.3 m loaded area.

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 Preliminary Geotechnical Investigation was performed for the exclusive use of CRW 1 LP and CRW 2 LP (Client) in order to evaluate the subsurface conditions at 2105, 2087, 2097 and 2077 Royal Windsor Drive, 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 Preliminary 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 Preliminary 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.

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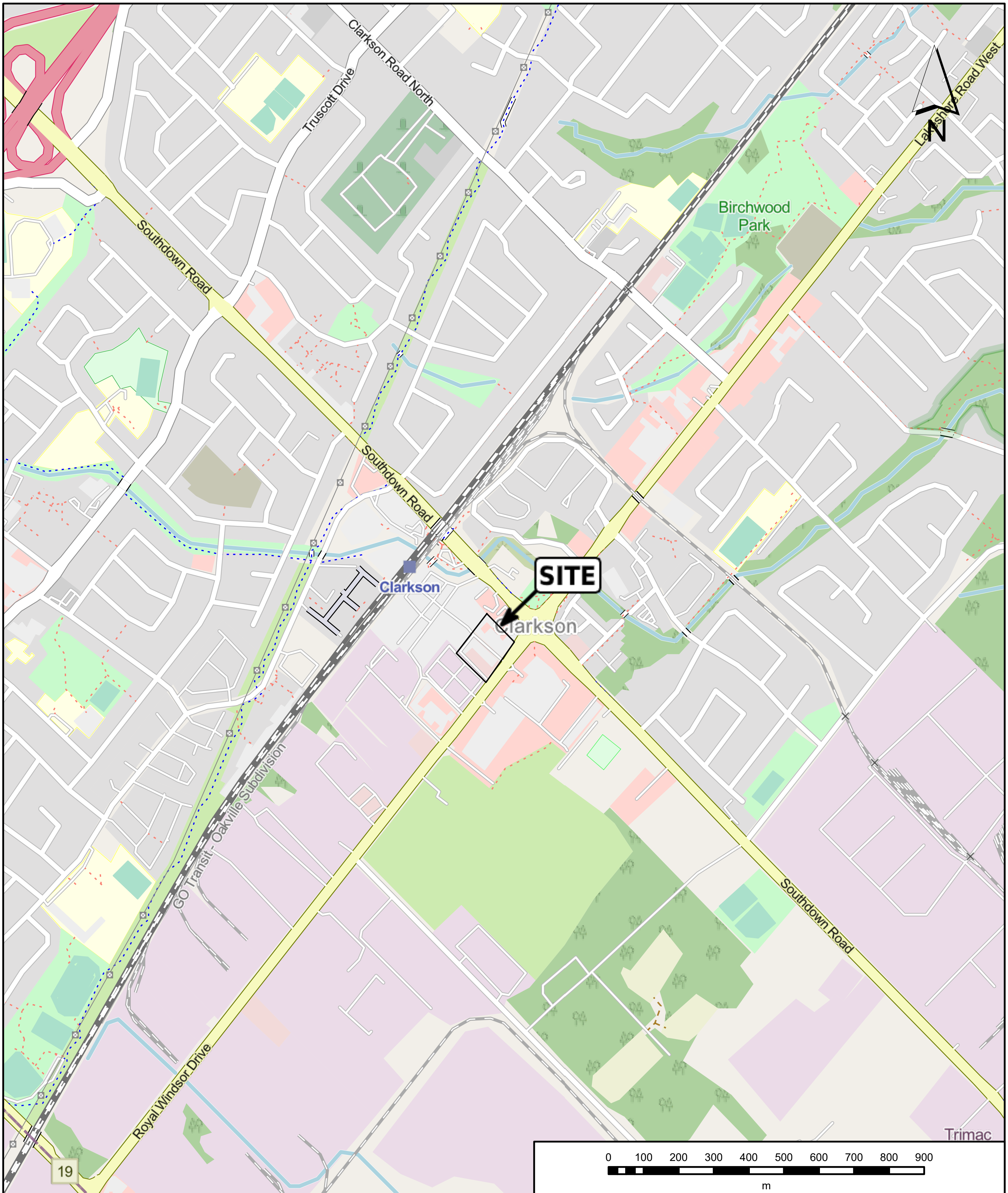
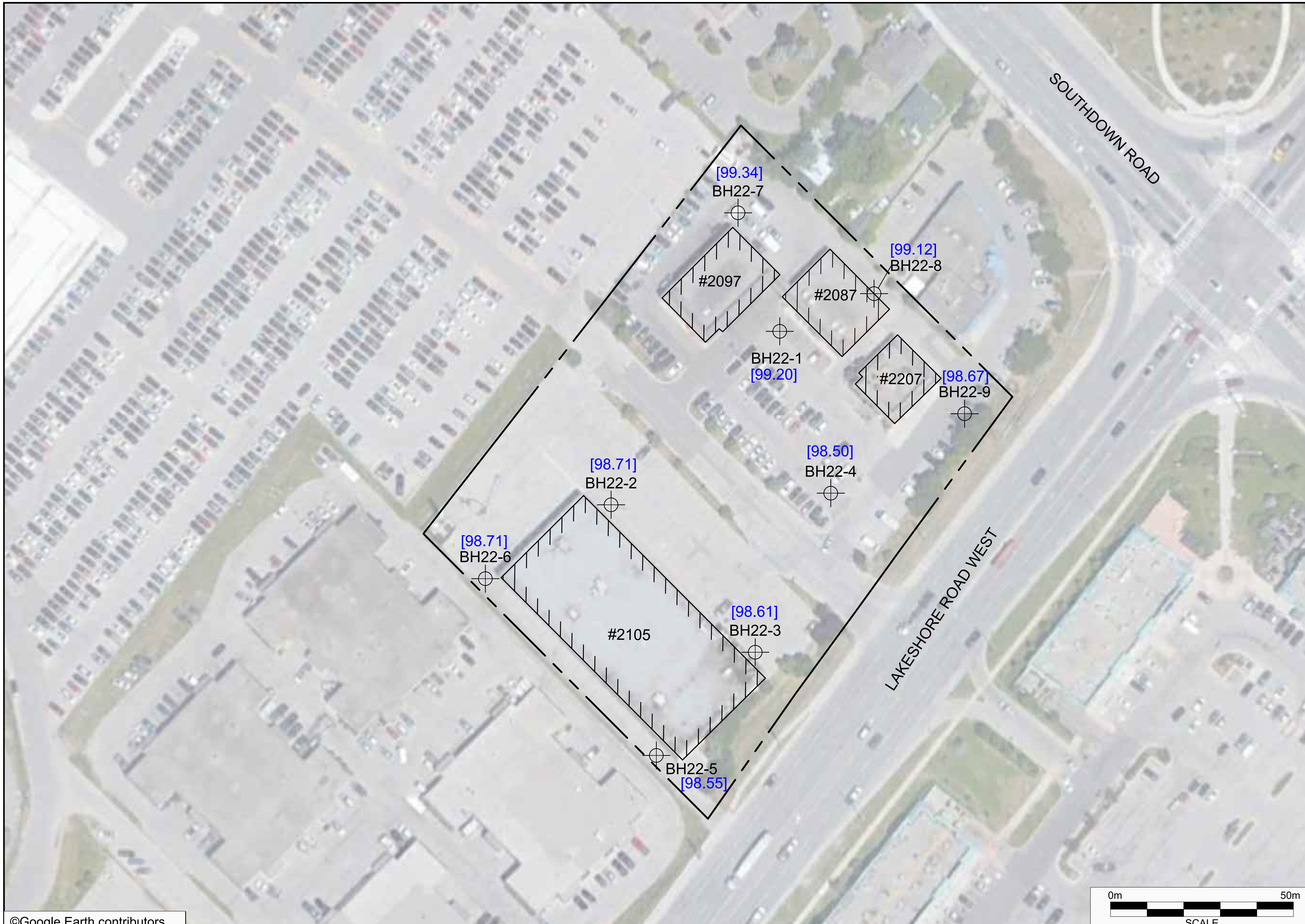
Template: Master Geotechnical Investigation Report – Ontario, GEO, September 2, 2021

FIGURES



PINCHIN

PROJECT NAME: PRELIMINARY GEOTECHNICAL INVESTIGATION					
CLIENT NAME: CRW 1 LP AND CRW 2 LP					
PROJECT LOCATION: 2077-2105 ROYAL WINDSOR DRIVE, MISSISSAUGA, ONTARIO					
FIGURE NAME: KEY MAP					FIGURE NUMBER: 1
PROJECT NUMBER: 306354.002	SCALE: 1:20,000	DRAWN BY: AMG	REVIEWED BY: KS	DATE: NOV. 2022	



N

LEGEND

PROPERTY BOUNDARY

SITE BUILDING

BOREHOLE LOCATION

[XX.X] GROUND SURFACE ELEVATION (MASL)

MASL METERS ABOVE SEA LEVEL

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

PINCHIN

PROJECT NAME: PRELIMINARY GEOTECHNICAL INVESTIGATION	
CLIENT NAME: CRW 1 LP AND CRW 2 LP	
PROJECT LOCATION: 2077-2105 ROYAL WINDSOR DRIVE, MISSISSAUGA, ONTARIO	
FIGURE NAME: BOREHOLE LOCATION PLAN	
PROJECT NUMBER: 306354.002	SCALE: AS SHOWN
DRAWN BY: AMG	REVIEWED BY: KS
DATE: NOV. 2022	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:

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

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: BH22-1 (MW)

Project #: 306354.002 **Logged By:** KS

Project: Geotechnical Investigation

Client: Slate Asset Managementt LP

Location: 2077-2105 Royal Windsor Drive, Mississauga, ON

Drill Date: August 3, 2022 **Project Manager:** RM

SUBSURFACE PROFILE

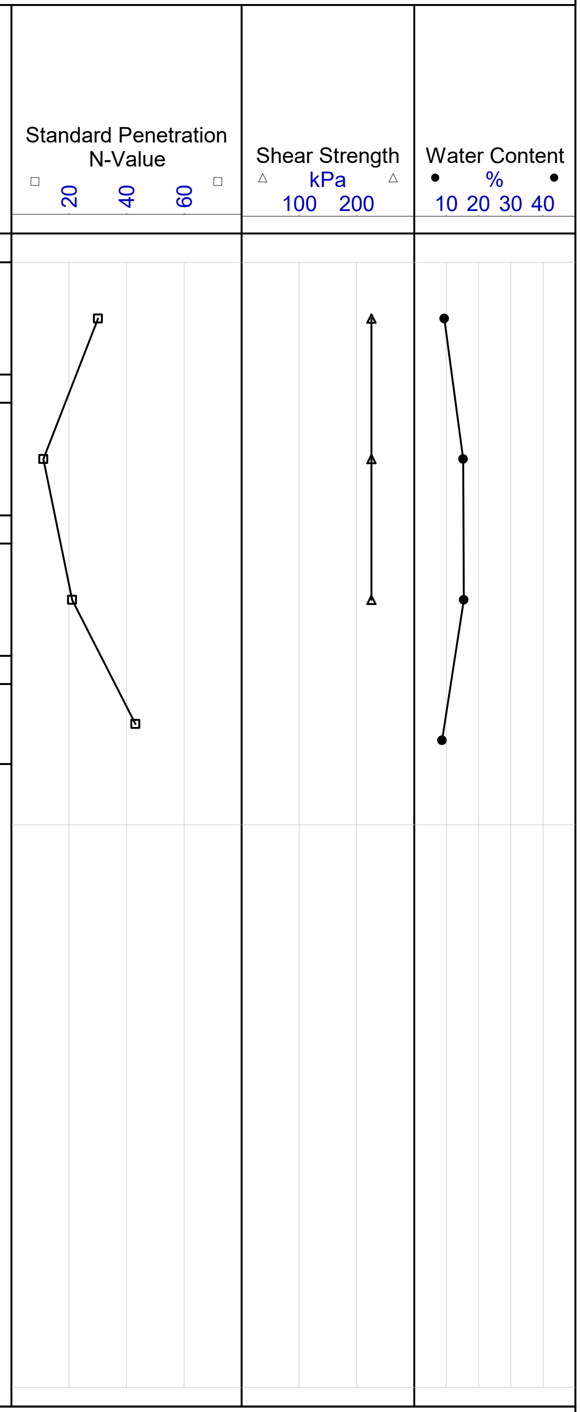
Depth (m)	Description	Elevation (m)
0	Ground Surface	99.20
0–0.10	**Asphalt** Asphaltic concrete - 100 mm	
0.10–0.76	**Fill** Brown sand and gravel, dense, moist	98.44
0.76–0.91	Mottled grey/brown clayey silt, trace sand and gravel, hard, DTPL	98.29
0.91–1.52	**Silt** Mottled grey/brown clayey silt, hard, DTPL	97.68
1.52–2.84	Brown, with oxidation	96.36
2.84–3.05	inferred weathered shale	96.15
3.05–4.57	**Unsampled** Augers advanced to 4.6 mbgs to install monitoring well	94.63
	End of Borehole	
	Borehole terminated at 4.6 mbgs.	

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

Water level = 2.15 mbgs, as measured on August 31, 2022

SAMPLE

Sample Type	Sampler #	Recovery (%)	SPT N-Value
SS	1	70	30
SS	2	90	11
SS	3	80	21
SS	4	80	43



Contractor: Strata Drilling Inc. **Grade Elevation:** 99.20 masl

Drilling Method: Hollow Stem Augers **Top of Casing Elevation:** 99.08 masl

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

Log of Borehole: BH22-2 (MW)

Project #: 306354.002 **Logged By:** KS

Project: Geotechnical Investigation

Client: Slate Asset Managementt LP

Location: 2077-2105 Royal Windsor Drive, Mississauga, ON

Drill Date: August 3, 2022 **Project Manager:** RM

SUBSURFACE PROFILE

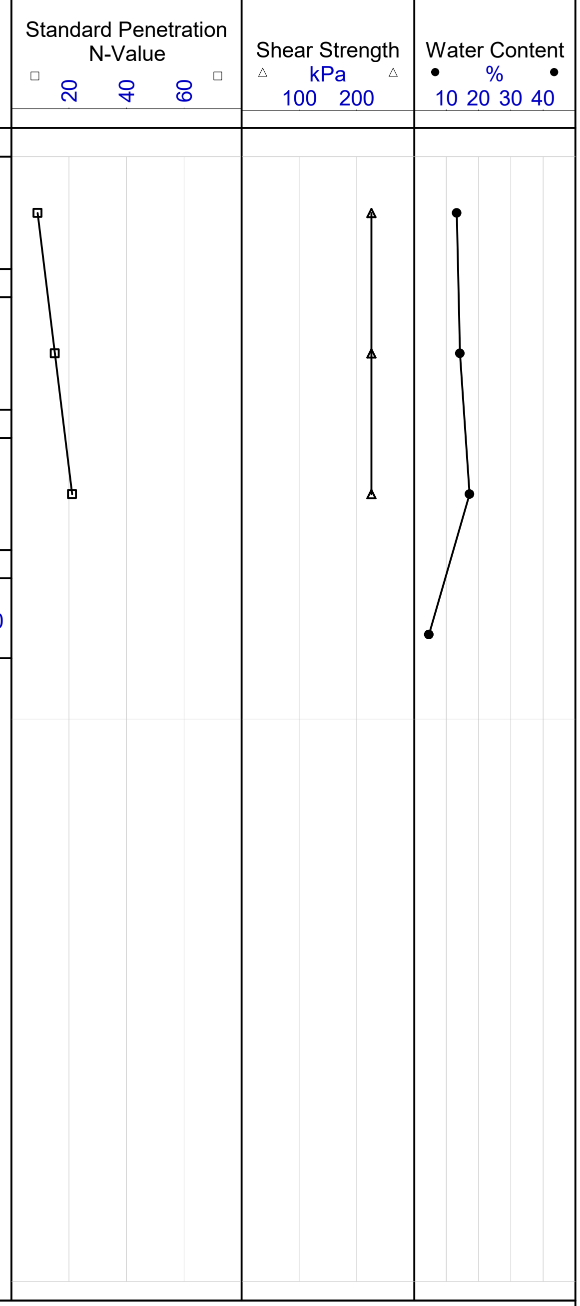
Depth (m)	Description	Elevation (m)
0	Ground Surface	98.71
	Asphalt Asphaltic concrete - 75 mm	98.51
	Fill Dark brown sand and gravel, loose,moist	97.95
1	Mottled grey/brown clayey silt, trace sand and gravel, hard, DTPL	
	Silt Mottled grey/brown clayey silt, trace sand and gravel, hard, DTPL	97.19
2	Clayey silt, with oxidation	96.42
	inferred weathered shale	96.00
3–4	**Unsampled** Augers advanced to 4.6 mbgs to install monitoring well	94.14
5	End of Borehole Borehole terminated at 4.6 mbgs.	

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

Water level = 2.52 mbgs, as measured on August 31, 2022

SAMPLE

Sample Type	Sampler #	Recovery (%)	SPT N-Value
SS	1	60	9
SS	2	60	15
SS	3	80	21
SS	4	60	gt 50



Contractor: Strata Drilling Inc.

Drilling Method: Hollow Stem Augers

Well Casing Size: 51 mm

Grade Elevation: 98.71 masl

Top of Casing Elevation: 98.56 masl

Sheet: 1 of 1

Log of Borehole: BH22-3 (MW)

Project #: 306354.002 **Logged By:** KS

Project: Geotechnical Investigation

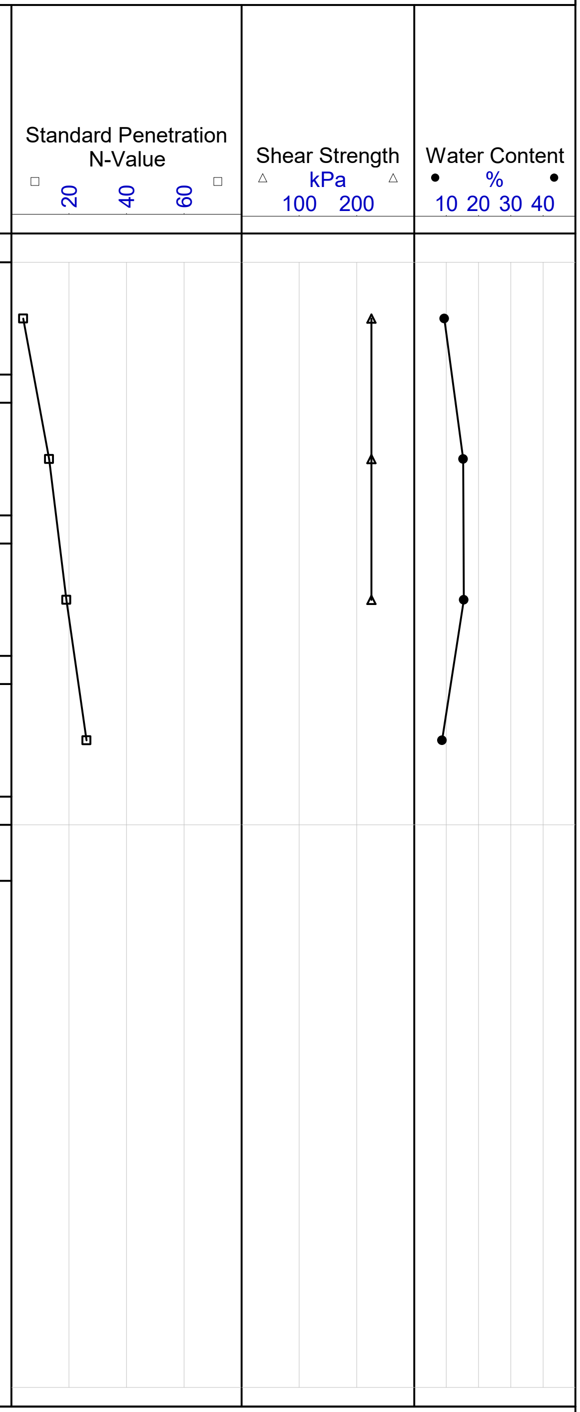
Client: Slate Asset Managementt LP

Location: 2077-2105 Royal Windsor Drive, Mississauga, ON

Drill Date: August 5, 2022 **Project Manager:** RM

SUBSURFACE PROFILE / SAMPLE

Depth (m)	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-Value
0	Ground Surface	98.61					
	Asphalt Asphaltic concrete - 50 mm	98.40		SS	1	70	4
	Fill Brown sand and gravel, very loose, moist		Bentonite				
	Grey/black clayey silt, some gravel, compact, moist	97.85		SS	2	90	13
1	**Silt** Grey clayey silt, some gravel, compact, moist to highly weathered shale		Riser	SS	3	80	19
2				SS	4	80	26
3	inferred weathered shale	95.56		SS	5	70	>50
	Unsampled Augers advanced to 4.6 mbgs to install monitoring well	95.26	Silica Sand				
4			Screen				
5	End of Borehole	93.61					
	Borehole terminated at 5.0 mbgs.		Water level = 2.65 mbgs, as measured on August 31, 2022				
6							



Contractor: Strata Drilling Inc.

Drilling Method: Hollow Stem Augers

Well Casing Size: 51 mm

Grade Elevation: 98.61 masl

Top of Casing Elevation: 98.49 masl

Sheet: 1 of 1

Log of Borehole: BH22-4 (MW)

Project #: 306354.002 **Logged By:** KS

Project: Geotechnical Investigation

Client: Slate Asset Managementt LP

Location: 2077-2105 Royal Windsor Drive, Mississauga, ON

Drill Date: August 4, 2022 **Project Manager:** RM

SUBSURFACE PROFILE

Description	Elevation (m)
Ground Surface	98.50
Asphalt Asphaltic concrete - 100 mm	97.74
Fill Brown sand and gravel, compact, moist	96.98
Silt Mottled grey/brown clayey silt, trace sand and gravel, hard, DTPL with oxidation	95.72
Brown	95.15
inferred weathered shale	
Bedrock - Cored Very poor quality (RQD = 0%)	93.55
Poor quality (RQD = 31%)	
Poor quality (RQD = 30%)	92.05
Poor quality (RQD = 26%)	90.53
Poor quality (RQD = 46%)	89.00
Poor quality (RQD = 25%)	87.48
Fair quality (RQD = 66%)	85.95
End of Borehole Borehole terminated at 12.6 mbgs.	

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

Water level = 2.69 mbgs, as measured on August 31, 2022

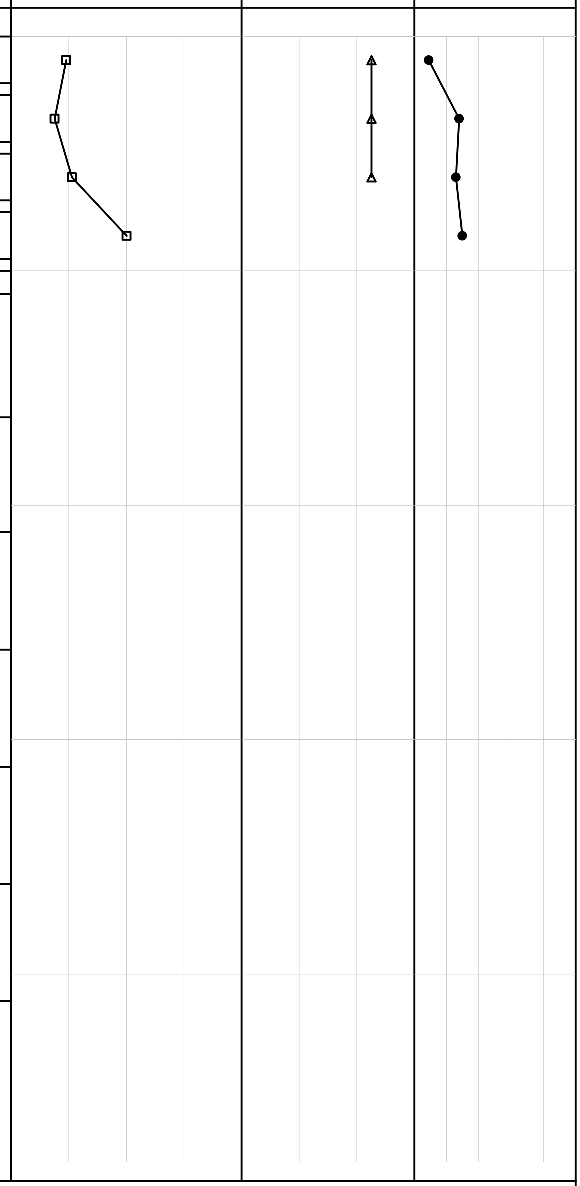
SAMPLE

Sample Type	Sampler #	Recovery (%)	SPT N-Value
SS	1	50	19
SS	2	60	15
SS	3	70	21
SS	4	60	40
RC	1	100	
RC	2	100	
RC	3	100	
RC	4	100	
RC	5	100	
RC	6	100	
RC	7	100	

Standard Penetration N-Value: 20 40 60

Shear Strength kPa: 100 200

Water Content %: 10 20 30 40



Contractor: Strata Drilling Inc. **Grade Elevation:** 98.50 masl

Drilling Method: Hollow Stem Augers **Top of Casing Elevation:** 98.36 masl

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

Log of Borehole: BH22-5 (MW)

Project #: 306354.002 **Logged By:** KS

Project: Geotechnical Investigation

Client: Slate Asset Managementt LP

Location: 2077-2105 Royal Windsor Drive, Mississauga, ON

Drill Date: August 3, 2022 **Project Manager:** RM

SUBSURFACE PROFILE

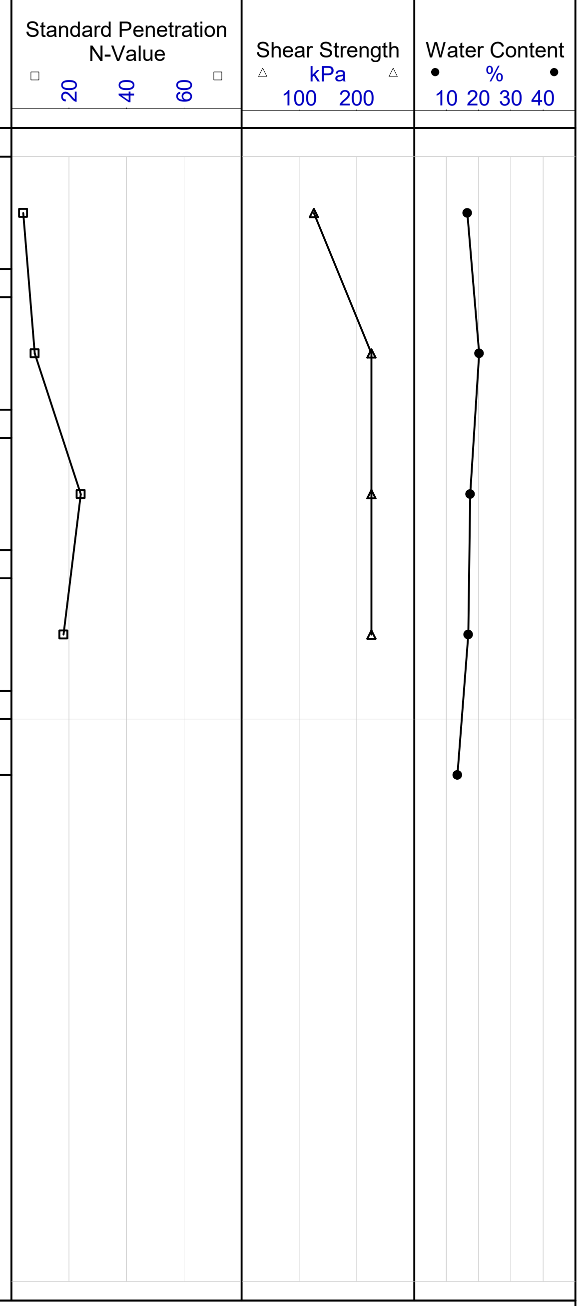
Depth (m)	Description	Elevation (m)
0	Ground Surface	98.55
0–0.25	**Asphalt** Asphaltic concrete - 100 mm	98.30
	Fill Dark brown sand and gravel, very loose, moist	97.79
	Grey clayey silt, trace gravel, stiff, APL	97.64
	trace organics	
	Brown sand and gravel, loose	
	Mottled grey/brown clayey silt, trace gravel, hard, APL	97.03
	Silt Mottled grey/brown clayey silt, hard, DTPL with oxidation	95.20
	inferred weathered shale	94.89
	Unsampled Augers advanced to 4.6 mbgs to install monitoring well	93.98
4.6	End of Borehole	
	Borehole terminated at 4.6 mbgs.	

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

Water level = 2.42 mbgs, as measured on August 31, 2022

SAMPLE

Sample Type	Sampler #	Recovery (%)	SPT N-Value
SS	1	70	4
SS	2	80	8
SS	3	90	24
SS	4	90	18
SS	5	50	>50



Contractor: Strata Drilling Inc. **Grade Elevation:** 98.55 masl

Drilling Method: Hollow Stem Augers **Top of Casing Elevation:** 98.46 masl

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

Log of Borehole: BH22-6 (MW)

Project #: 306354.002 **Logged By:** KS

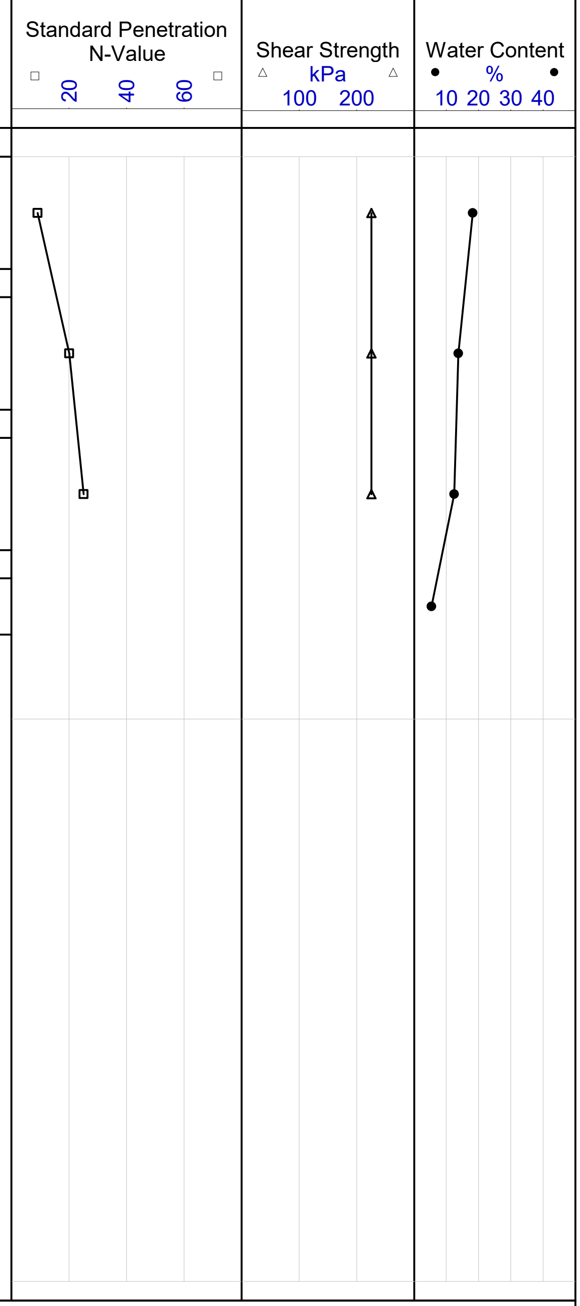
Project: Geotechnical Investigation

Client: Slate Asset Managementt LP

Location: 2077-2105 Royal Windsor Drive, Mississauga, ON

Drill Date: August 3, 2022 **Project Manager:** RM

SUBSURFACE PROFILE				SAMPLE			
Depth (m)	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-Value
	Ground Surface	98.71					
0	**Asphalt** Asphaltic concrete - 100 mm	98.40					
	Fill Brown sand and gravel, loose, moist			SS	1	80	9
	Silt Mottled grey/brown clayey silt, hard, DTPL	97.95	Bentonite				
1	Clayey silt, with oxidation		Riser	SS	2	80	20
2				SS	3	90	25
	inferred weathered shale	96.42		SS	4	40	>50
	Unsampled Augers advanced to 4.6 mbgs to install monitoring well	96.12	Silica Sand				
3							
4			Screen				
	End of Borehole	94.14					
5	Borehole terminated at 4.6 mbgs.		Water level = 2.06 mbgs, as measured on August 31, 2022				
6							



Contractor: Strata Drilling Inc. **Grade Elevation:** 98.71 masl

Drilling Method: Hollow Stem Augers **Top of Casing Elevation:** 98.64 masl

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

Log of Borehole: BH22-7 (MW)

PINCHIN

Project #: 306354.002 **Logged By:** KS

Project: Geotechnical Investigation

Client: Slate Asset Managementt LP

Location: 2077-2105 Royal Windsor Drive, Mississauga, ON

Drill Date: August 2, 2022 **Project Manager:** RM

SUBSURFACE PROFILE

Description	Elevation (m)
Ground Surface	99.34
Asphalt Asphaltic concrete - 100 mm	
Fill Grey/brown sand and gravel, loose, moist	
Mottled grey/brown clayey silt, some gravel, very stiff, APL	98.43
Silt Mottled grey/brown clayey silt, trace sand and gravel, very stiff, DTPL with oxidation	97.82
hard	
inferred weathered shale	97.05
Unsampled Augers advanced to 4.6 mbgs to install monitoring well	96.63
End of Borehole	94.77
Borehole terminated at 4.6 mbgs.	

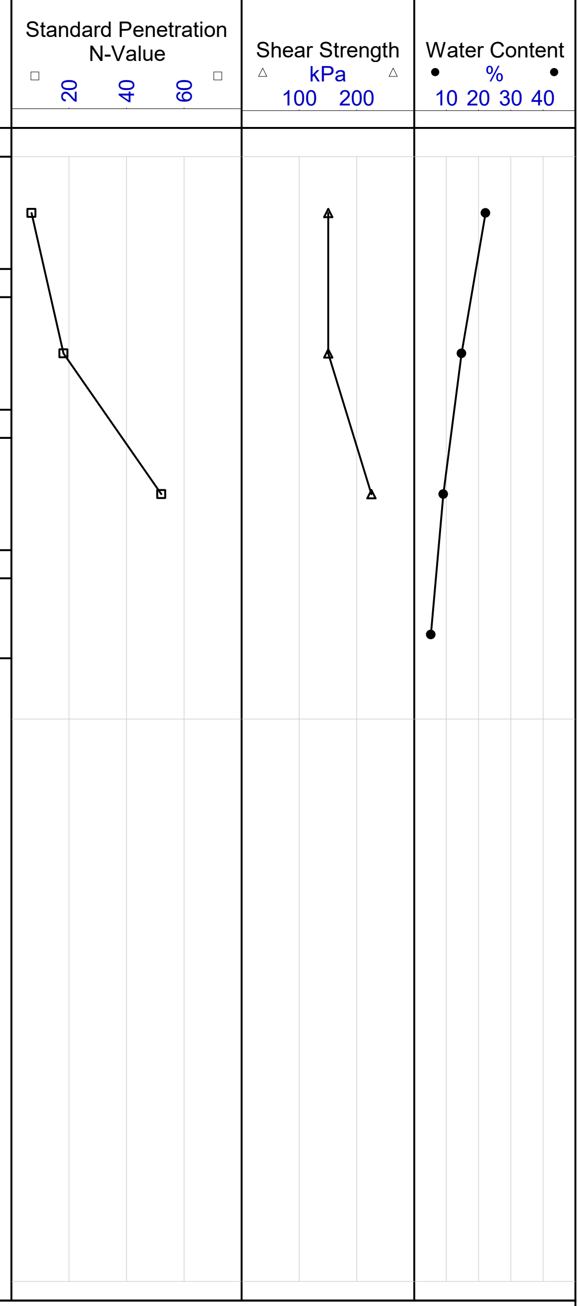
Depth (m): 0, 1, 2, 3, 4, 5, 6

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

Water level = 3.23 mbgs, as measured on August 31, 2022

SAMPLE

Sample Type	Sampler #	Recovery (%)	SPT N-Value
SS	1	80	7
SS	2	70	18
SS	3	90	52
SS	4	60	>50



Contractor: Strata Drilling Inc.

Drilling Method: Hollow Stem Augers

Well Casing Size: 51 mm

Grade Elevation: 99.34 masl

Top of Casing Elevation: 99.24 masl

Sheet: 1 of 1

Log of Borehole: BH22-8 (MW)

Project #: 306354.002
Logged By: KS
Project: Geotechnical Investigation
Client: Slate Asset Managementt LP
Location: 2077-2105 Royal Windsor Drive, Mississauga, ON
Drill Date: August 2, 2022
Project Manager: RM

SUBSURFACE PROFILE

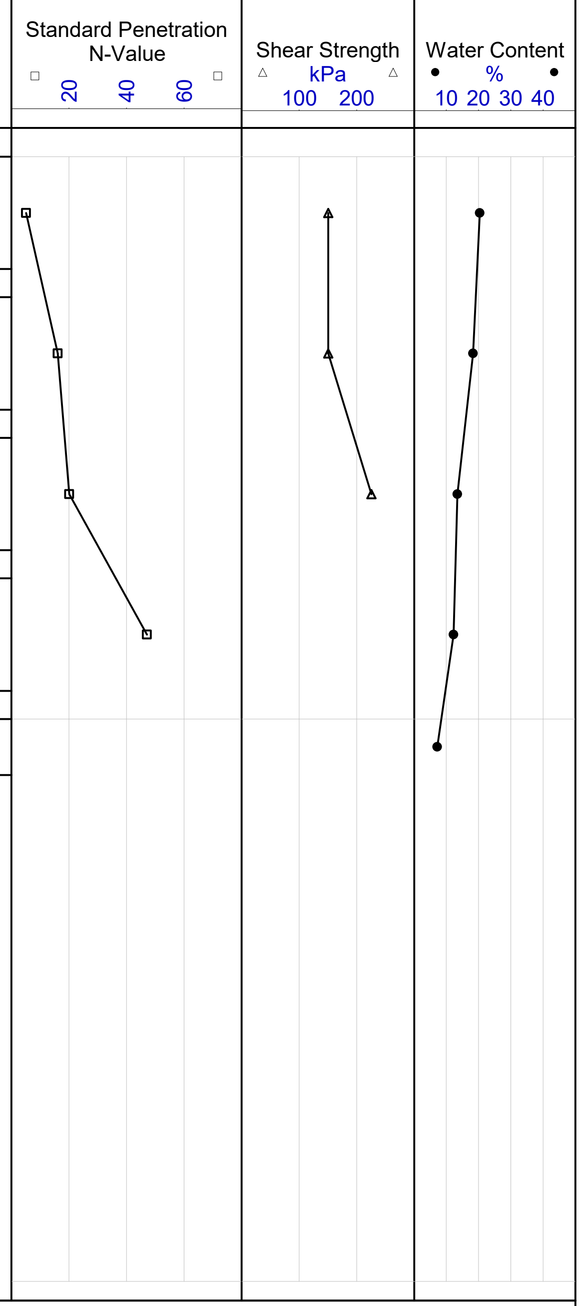
Depth (m)	Description	Elevation (m)
	Ground Surface	99.12
0	**Asphalt** Asphaltic concrete - 100 mm	
	Fill Dark brown sand and gravel, loose, moist	98.87
	Grey/brown clayey silt, very stiff, APL	98.36
1	**Silt** Mottled grey/brown clayey silt, hard, DTPL	
	with trace organics/rootlets	97.60
2	Brown, with oxidation	
3	inferred weathered shale	96.07
	Unsampled Augers advanced to 4.6 mbgs to install monitoring well	95.77
4		
	End of Borehole	94.55
5	Borehole terminated at 4.6 mbgs.	
6		

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

Water level = 3.07 mbgs, as measured on August 31, 2022

SAMPLE

Sample Type	Sampler #	Recovery (%)	SPT N-Value
SS	1	60	5
SS	2	70	16
SS	3	90	20
SS	4	90	47
SS	5	60	>50



Contractor: Strata Drilling Inc.
Grade Elevation: 99.12 masl
Drilling Method: Hollow Stem Augers
Top of Casing Elevation: 99.07 masl
Well Casing Size: 51 mm
Sheet: 1 of 1

Log of Borehole: BH22-9 (MW)

Project #: 306354.002 **Logged By:** KS

Project: Geotechnical Investigation

Client: Slate Asset Managementt LP

Location: 2077-2105 Royal Windsor Drive, Mississauga, ON

Drill Date: August 2, 2022 **Project Manager:** RM

SUBSURFACE PROFILE

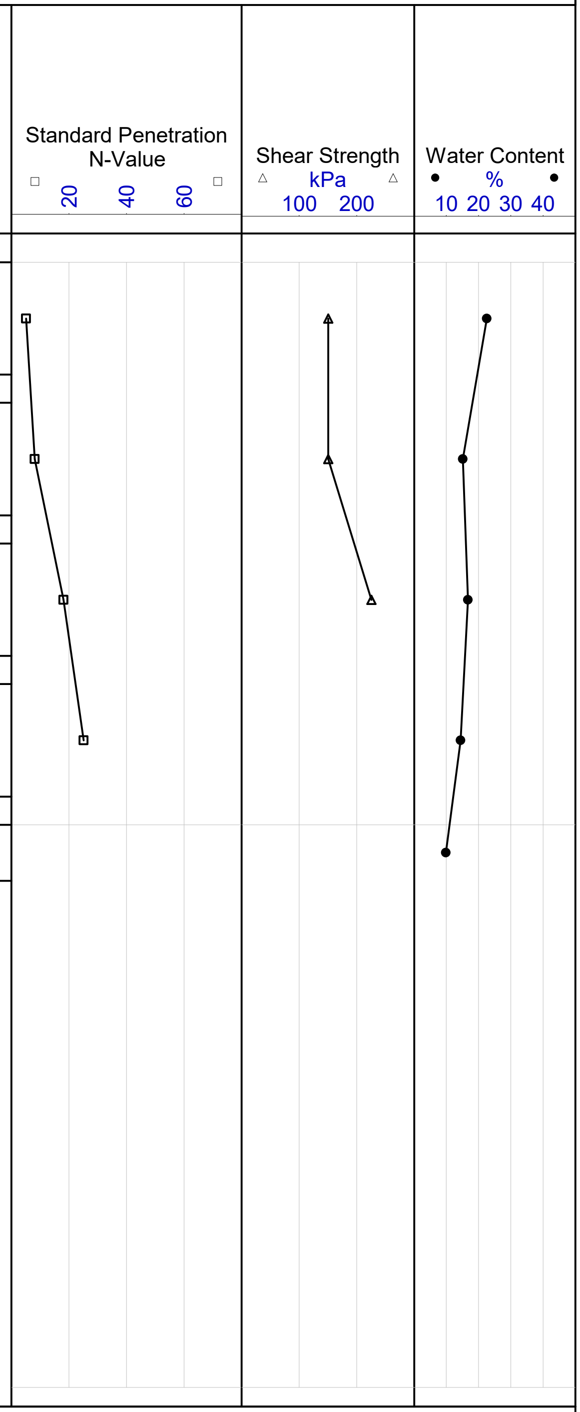
Depth (m)	Description	Elevation (m)
0	Ground Surface	98.67
0–0.25	**Asphalt** Asphaltic concrete - 100 mm	98.42
0.25–1.52	**Fill** Dark brown sand and gravel, loose, moist. Mottled grey/brown clayey silt, very stiff, APL, with oxidation	97.15
1.52–2.29	**Silt** Mottled grey/brown clayey silt, hard, DTPL with oxidation	96.38
2.29–3.23	Brown	95.44
3.23–3.35	inferred weathered shale	95.32
3.35–4.57	**Unsampled** Augers advanced to 4.6 mbgs to install monitoring well	94.10
	End of Borehole	
	Borehole terminated at 4.6 mbgs.	

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

Water level = 2.59 mbgs, as measured on August 31, 2022

SAMPLE

Sample Type	Sampler #	Recovery (%)	SPT N-Value
SS	1	80	5
SS	2	80	8
SS	3	90	18
SS	4	90	25
SS	5	90	>50



Contractor: Strata Drilling Inc. **Grade Elevation:** 98.67 masl

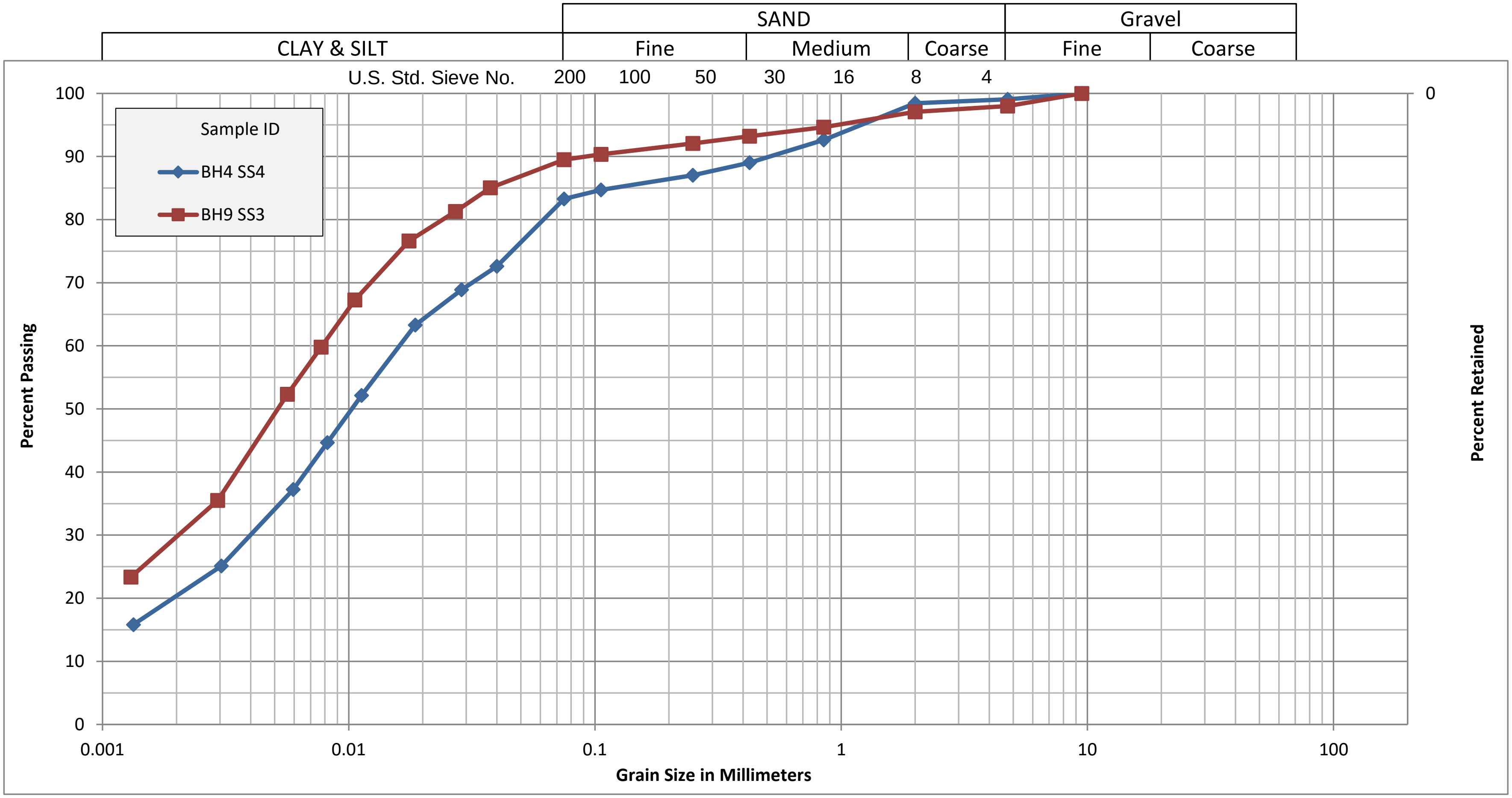
Drilling Method: Hollow Stem Augers **Top of Casing Elevation:** 98.60 masl

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

APPENDIX III

Laboratory Testing Reports for Soil Samples

Unified Soil Classification System



Sample ID	Depth (ft)	% Gravel	% Sand	% Silt	% Clay
BH4 SS4	7.5-9.5	1.0	15.7	64.3	19.0
BH9 SS3	5.0-7.0	2.0	8.5	60.5	29.0

PINCHIN

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

PARTICLE SIZE DISTRIBUTION ANALYSIS

Geotechnical Investigation - 2077-2105 Royal Windsor Dr, Mississauga, ON
Slate Asset Management LP

Figure No. 1

306354.002

Reviewed By:

More information available upon request

Atterberg Limits

LS 703&704 / AASHTO T89

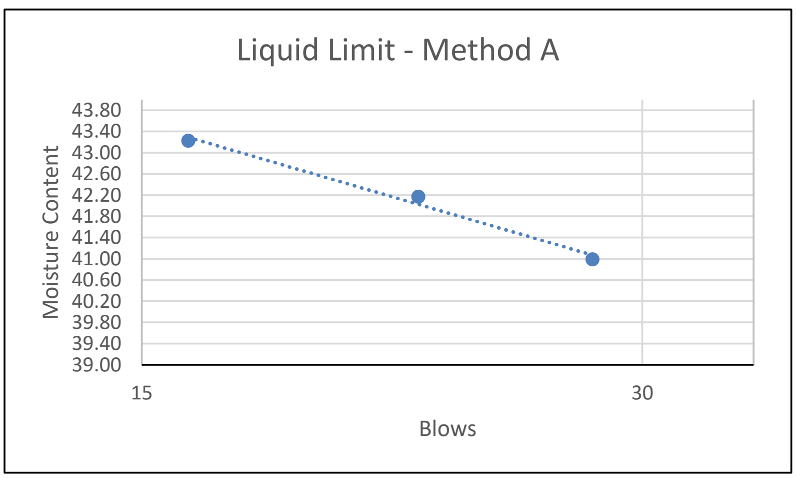
Project Name: Geotechnical Investigation
Project No. 306354.002
Client: Slate Asset Management LP
Location: 2077-2105 Royal Windsor Dr, Mississauga, O
Material: Soil
Sample: BH9 SS3 5.0-7.0

Test Date: September 22, 2022
Tested By: B Frank
Sample Date: August 2, 2022
Sampled By: K Singh
Reviewed By: V Marshall

Liquid Limit - Method A						
Pot Number	1	2	3			
Number of blows	28	22	16			
Wet mass + pot	33.08	34.67	33.68			
Dry mass + pot	28.01	29.04	28.13			
Tare	15.64	15.69	15.29			
Water content %	40.99	42.17	43.22			

Plastic Limit			
Pot Number	1	2	
Wet mass + pot	23.96	24.47	
Dry mass + pot	22.49	22.86	
Tare	15.74	15.48	
Water content %	21.8	21.8	

PI = LL - PL	
Liquid Limit %	**42**
Plastic Limit %	**22**
Plastic Index	**20**
Non Plastic	



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.