FINAL

Preliminary Geotechnical Investigation – Proposed Residential Development

4099 Erin Mills Parkway, Mississauga, Ontario

Prepared *for:*

Queenscorp (Erin Mills) Inc.
2 Queen Elizabeth Boulevard, Etobicoke,
Ontario, M8Z 1L8

February 15, 2022

Pinchin File: 299799

Issued to:	**Queenscorp (Erin Mills) Inc.**
Issued on:	**February 15, 2022**
Pinchin File:	**299799**
Issuing Office:	**Mississauga, ON**

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1.0 INTRODUCTION AND SCOPE

Pinchin Ltd. (Pinchin) was retained by Queenscorp (Erin Mills) Inc. (Client) to conduct a Preliminary Geotechnical Investigation and provide subsequent preliminary geotechnical design recommendations for the proposed residential development to be located at 4099 Erin Mills Parkway, 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 development will consist of a mixed-use development of primarily townhouse residential units, condominium units and some retail at grade along the Erin Mills Parkway. The proposed design consists of five residential buildings ranging from 6 to 10 stories in height: and back-to-back four-storey townhouses along the southeast side of the Site, adjacent to the houses along Farrier Court and wrapping around the eastern half of the Site along Sawmill Valley Drive. One level underground parking is proposed below the entire development complete with asphalt surfaced access roadways and parking areas.

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 five (5) sampled boreholes (Boreholes BH1 to BH5), at the Site. The information gathered from the Preliminary Geotechnical Investigation will allow Pinchin to provide preliminary geotechnical design recommendations for the proposed development. 'As the design progresses, these preliminary results should be supplemented with more detailed geotechnical field investigation and the design recommendations below should be revised based on the updated information.'

Based on a desk top review and the results of the Preliminary Geotechnical Investigation, the following preliminary 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;
- Site service trench design;
- 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;
- 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 CONCURRENT WORK

Pinchin is concurrently completing a Hydrogeological study at the Site. The results of this study will be reported under a separate cover. The relevant geotechnical information from the concurrent investigation has been incorporated into this report.

3.0 SITE DESCRIPTION AND GEOLOGICAL SETTING

The Site is located on 4099 Erin Mills Parkway, Mississauga, Ontario. The Site is located on the northeast side of Erin Mills Parkway, southeast side of Folkway Drive, southwest side of Sawmill Valley Drive, and approximately 50 m northwest of Ferrier Court. The area of the Site is approximately 2.6 hectares (6.4 acres) and is currently occupied by a retail plaza, and ancillary asphalt access roads and parking. The Site is surrounded by single family detached dwellings, semi-detached dwellings townhomes.

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

4.0 PRELIMINARY GEOTECHNICAL FIELD INVESTIGATION AND METHODOLOGY

Pinchin completed field investigations at the Site between December 2 and 3, 2021 by advancing a total of five (5) sampled boreholes throughout the Site. Four (4) boreholes were advanced to a maximum depth of approximately 6.0 meters below the ground surface (mbgs) and one (1) borehole was advanced to a depth of approximately 8.0 mbgs in the existing parking lot under the footprint of the proposed buildings. The two (2) boreholes, under the footprint of Buildings A and E were advanced into the bedrock and cored using NQ sized core barrels to evaluate the Rock Quality Designation (RQD). The approximate spatial locations of the boreholes advanced at the Site are shown on Figure 2.

The boreholes were advanced with the use of a Geoprobe 3230 DT hollow auger drill rig and NQ-Rock coring which was equipped with standard soil sampling equipment. Soil samples were collected at 0.76 and 1.52 m intervals using a 51 mm outside diameter (OD) split spoon barrel in conjunction with Standard Penetration Tests (SPT) "N" values (ASTM D1586). The SPT "N" values were used to assess the compactness condition of the non-cohesive soil.

Bedrock was proven in Boreholes BH1 and BH4 by coring with an NQ-size double tube diamond bit core barrel. The bedrock core specimens were measured in the field to determine the Rock Quality Designation (RQD) (ASTM 6032). The core samples were returned to the lab for further visual examination and testing.

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

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

Groundwater observations and measurements were obtained from the open boreholes during and upon completion of drilling. Groundwater levels were measured in the monitoring wells on December 07, 2021. The groundwater observations and measurements recorded are included on the appended borehole logs.

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

The field investigation was monitored by experienced Pinchin personnel. Pinchin logged the drilling operations and identified the soil samples as they were retrieved. The recovered soil samples were sealed into plastic bags and carefully transported to an independent and accredited materials testing laboratory for detailed analysis and testing. All soil samples were classified according to visual and index properties by the project engineer.

The field logging of the soil and groundwater conditions was performed to collect geotechnical engineering design information. The borehole logs include textural descriptions of the subsoil in accordance with a modified Unified Soil Classification System (USCS) and indicate the soil boundaries inferred from non-continuous sampling and observations made during the borehole advancement. These boundaries reflect approximate transition zones for the purpose of geotechnical design and should not be interpreted as exact planes of geological change. The modified USCS classification is explained in further detail in Appendix I. Details of the soil and groundwater conditions encountered within the boreholes are included on the Borehole Logs within Appendix II.

Select soil samples and rock cores collected from the boreholes were submitted to a material testing laboratory to determine the grain size distribution of the soil and the uniaxial compression strength of rock cores. 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.

5.0 SUBSURFACE CONDITIONS

5.1 Borehole Soil Stratigraphy

In general, the soil stratigraphy at the Site comprises an asphaltic concrete pavement structure underlain by fill material followed by gravel and sand and a silt seam underlain by bedrock to the maximum borehole termination depths of approximately 6.0 to 9.3 mbgs (Elevations 151.7 to 147.8). The appended borehole logs provide detailed soil descriptions and stratigraphies, results of SPT testing, moisture content profiles, details of monitoring well installations, and groundwater measurements.

An asphaltic concrete pavement structure was encountered in all of the boreholes at the Site and was observed to comprise 85 mm of asphaltic concrete overlying 220 to 300 mm of granular fill material.

Fill material was encountered underlying the asphalt and granular fill within all boreholes and extended to depths ranging between 0.1 and 0.4 mbgs (155.4 to 158.8 masl). The fill material generally comprised of sand, some gravel and trace of silt with traces of wood pieces, organics and rootlets. The fill material had a compact relative density based on SPT ‘N’ values of 13 to 19 blows per 300 mm penetration of a split spoon sampler. The fill material was moist to very moist at the time of sampling.

Sand was encountered underlying the fill material in Boreholes BH2, BH4 and BH5 and extended to the maximum depth of 3.0 mbgs (Elevation 153.1 masl). The same layer encountered in Boreholes BH4 and BH5 underlying a seam of silt and extends to the depth of 3.0 to 3.5 mbgs (Elevation 152.2 to 154.5 masl). The sand deposit generally comprised of sand to silty sand with trace to some gravel. The sand material had a compact to very dense relative density based SPT ‘N’ values of 21 to above 50 blows per 300 mm penetration of a split spoon sampler.

A silt seam encountered in all boreholes between the depths of 0.6 and 4.5 mbgs. The thickness of the silt seam was between 0.8 and 1.5 m and had a firm to hard consistency based SPT ‘N’ values of 8 to above 50 blows per 300 mm penetration of a split spoon sampler. The silt deposit generally comprised of clayey silt to sandy silt with some to trace gravel. The results of two particle size distribution analyses completed on samples of the silt material are provided in Appendix III and indicate that the samples contain 4 to 10% gravel, 19 to 10% sand, 63 to 47% silt, and 24 to 22% clay.

Gravel was encountered underlying the fill material in Borehole BH1 and underlying the silt seam in Boreholes BH1 and BH3 and underlying the sand deposit in Boreholes BH4 and BH5 and extends to depth of the bedrock surface ranging from 2.2 to 6.2 mbgs (Elevation 155.2 to 151.0 masl). The gravel deposit generally comprised of highly weathered bedrock with some sand. The gravel deposit had a dense to very dense relative density based on SPT ‘N’ values of 31 to greater than 50 blows per 300 mm penetration of a split spoon sampler.

5.2 Bedrock

Bedrock was encountered in all boreholes at depths ranging from 2.3 to 6.2 mbgs (Elevation 155.2 to 151.0 masl). Rock coring with NQ size barrels was completed for Boreholes BH1 and BH4. The depth to the bedrock surface as encountered at the borehole locations is summarized in the following table:

Borehole ID	Ground Surface Elevation (masl)	Depth of Bedrock (mbgs)	Top of Bedrock Elevation (masl)
BH1	158.9	6.2	152.7
BH2	156.1	4.5*	151.5
BH3	157.5	2.3*	155.2
BH4	155.7	4.7	151.0
BH5	157.5	4.9*	152.6

***Inferred elevation, not confirmed by coring**

The bedrock is highly weathered in the surface (approximately the upper 2.0 meters) with measured RQD values of 6% indicating the upper 2.0 meters of bedrock is of very poor quality. The RQD values increase to 29 to 38% indicating that the lower bedrock is of poor quality. A bedrock sample was tested to measure the uniaxial compressive strength (UCS). The result of this test is provided in Appendix III and indicated that the USC was measured to be 26.9 MPa.

5.3 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. The groundwater levels were measured on December 07, 2021 and the measurements are summarized below:

Borehole/Monitoring Well	Groundwater depth (mbgs)	Groundwater elevation (masl)
BH1	4.7	154.2
BH2	3.5	152.6
BH3	4.9	152.6
BH4	3.8	151.9
BH5	3.7	153.8

For geotechnical design purposes, the groundwater level may be taken at Elevation 154 ± masl. Seasonal variations in the water table should be expected, with higher levels occurring during wet weather conditions in the spring and fall and lower levels occurring during dry weather conditions.

6.0 PRELIMINARY GEOTECHNICAL DESIGN RECOMMENDATIONS

6.1 General Information

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

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

As the design progresses, these preliminary results should be supplemented with a more detailed geotechnical field investigation and the design recommendations presented below should be revised based on the updated information. Specifically, additional rock core holes should be completed to confirm bedrock quality and elevations across the Site. In addition, a qualified geotechnical engineer should be on-Site during the foundation preparation to ensure the subsurface conditions are the same/similar to what was observed during the investigation.

It is Pinchin's understanding that the proposed development/design consists of five residential buildings ranging from 6 to 10 stories in height: and back-to-back four-storey townhouses along the southeast side of the Site, adjacent to the houses along Farrier Court and wrapping around the eastern half of the Site along Sawmill Valley Drive. One level underground parking is proposed below the entire development complete with asphalt surfaced access roadways and parking areas.

6.2 Open Cut Excavations

It is understood that the proposed development will be constructed on one level of underground parking. The Site is relatively flat, with elevations ranging from 155.7 to 158.9 masl as measured at the borehole locations. It is understood that the Finished Floor Elevation (FFE) for one level of underground parking (P1) is approximately 3 mbgs (i.e. approximate Elevation 153 ± masl).

Based on the subsurface information obtained from within the boreholes, it is anticipated that the excavated material will predominately consist of granular fill and native silt and sand material. Groundwater was encountered at depths ranging from 3.5 to 4.9 mbgs (Elevation 151.9 to 154.2 masl). For geotechnical design purposes, the groundwater level may be taken at Elevation 154 ± 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 fill material and native silt to gravel encountered at the Site would be classified as Type 3 soil and temporary excavations in these soils must be sloped at an inclination of 1 horizontal to 1 vertical (H to V) from the base of the excavation. 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.

Fragments of stratified silt, gravel, and shale may be encountered in the transition zone between the native soil and the underlying rock. The bedrock below the weathered rock is predominantly shale. It is possible hard layers of shale and weathered rock may be found as high as 3 mbgs. The responsibility and risks associated with the removal of such layers must be noted in the contract documents for foundations, excavations and shoring contractors, as applicable.

Pinchin notes that, local contractors are familiar with excavating the local bedrock and have specialized knowledge and techniques for its removal during excavation. However, Pinchin recommends that a pre-excavation survey of all neighbouring properties be considered prior to conducting and excavation and construction activities. It is Pinchin believe that the preconstruction survey will serve to protect the Client from claims unrelated to the construction activities in the development of this property.

6.2.1 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, sheet-piling and cribbing shall meet all requirements of the Ontario Occupational Health and Safety Act and Regulations for Construction Projects and the Trench Excavators Protection Act. The shoring design must include appropriate factors of safety, and any possible surcharge loading must be taken into account. The support system must comply with sections 234 to 239 and 241 of Ontario Regulation 213/91.

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

The following parameters (unfactored) 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; conditions of the ground surface behind the wall is assumed to be flat.

Soil Layer	Bulk Unit Weight (kN/m^3)	Angle of Internal Friction (degrees)	Active Earth Pressure Coefficients	Passive Earth Pressure Coefficients
Earth Fill	18	28^0	0.36	2.77
Native Silt	20	29^0	0.35	2.88
Sand	18	35	0.27	3.69
Highly weathered Bedrock/gravel	20	30^0	0.27	3.00
Sound Bedrock	25	44^0	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.

Shales often weather rapidly when exposed to air in excavations, and deteriorated rock loses internal integrity and bearing capacity. Therefore, special measures should be considered to warrantee and avoid prolonged contact with air; including the advancement of the soldier piles and caissons at least 1 metre below the base of the excavation to accommodate such weathering.

6.3 Anticipated Groundwater Management

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

The Site is relatively flat, with elevations ranging from 155.7 to 158.9 masl as measured at the borehole locations. It is understood that the Finished Floor Elevation (FFE) for one level of underground parking (P1) is approximately 3 mbgs (i.e. approximate Elevation 153 ± masl). Groundwater was encountered at depths ranging from 3.5 to 4.9 mbgs (Elevation 151.9 to 154.2 masl). For geotechnical design purposes, the groundwater level may be taken at Elevation 154 ± masl.

Within the zone of excavation, the silt is considered a low to moderate permeability material, which will typically preclude significant free flow of water. However, the earth fill, sand and gravel deposits are considered high permeability materials, which will permit the free-flow of water when wet. In addition, the weathered bedrock deposit may include relatively permeable silt/sand zones which may yield free-flowing water when penetrated.

Successful dewatering of the Site could be challenging especially if excavations extend into gravel and weathered bedrock. A dewatering system installed by a specialist dewatering contractor will be required to lower the groundwater level prior to excavation. The design of the dewatering system should be left to the contractor's discretion, and the system should meet a performance specification to maintain and control the groundwater at least 1 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 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.

As previously mentioned, above average seasonal variations in the groundwater 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. As such, depending on the groundwater at the time of the excavation works, a more involved dewatering system may be required.

6.4 Site Servicing

6.4.1 Pipe Bedding and Cover Materials for Flexible and Rigid Pipes

The subgrade soil conditions beneath the site services will comprise silt, sand and gravel. No support problems are anticipated for flexible or rigid pipes founded in the natural mineral soils. It is critical that the pipe subgrade is inspected by a geotechnical engineer prior to placement of pipe bedding material to ensure adequate support is available for the services.

Service pipes require an adequate base to ensure proper pipe connection and positive flow is maintained post construction. As such, pipe bedding should be placed to be of uniform thickness and compactness. The pipe bedding and cover material should conform to OPSD 802.010 and 802.013 specifications for flexible pipes and to OPSD 802.031 to 802.033 with Class “B” bedding for rigid pipes. The pipe bedding material should consist of a minimum thickness of 150 mm Granular “A” (OPSS 1010) below the pipe and extend up the sides to the spring line. However, the bedding thickness may have to be increased depending on the pipe diameter or if wet or weak subgrade conditions are encountered.

The pipe cover material from the spring line should consist of a Granular "B" Type I (OPSS 1010) and should extend to a minimum of 300 mm above the top of the pipe. All granular fill material is to be placed in maximum 200 mm thick loose lifts compacted to a minimum of 98% SPMDD.

The bedding material, pipe and cover material should be installed as soon as practically possible after the excavation subgrade is exposed. The longer the excavated subgrade soil remains open to weather conditions and groundwater seepage, the greater the chance for construction problems to occur. Where it is difficult to stabilize the subgrade due to groundwater or the material is higher than the optimum moisture content, a Granular "B" Type II material may be required. Alternatively, if constant groundwater infiltration becomes an issue, then an approximate 150 mm granular pad consisting of 19 mm clear stone gravel (OPSS 1004) wrapped in a non-woven geotextile should be considered to maintain the integrity of the natural subgrade soil. The clear stone should contain a minimum of 50% crushed particles. Water collected within the stone should be controlled through sumps and filtered pumps.

6.4.2 Trench Backfill

Following placement of the pipe bedding cover, the trench shall be backfilled. Based on the results of the natural overburden deposits, the on-Site sand and silt from above the groundwater table will be suitable for use as trench backfill. The soil should be placed to the underside of the granular subbase of the pavement structure and be compacted in maximum 200 mm thick lifts to 98% SPMDD within 4% of the optimum moisture content. The natural material must be free of organics or other deleterious material.

The trench backfill should be placed in thin lifts and compacted with a smooth drum roller. Particular attention must be made to backfilling service connections where the trenches are narrow. If work is carried out during very dry weather, then water could be added to the backfill to improve compaction.

All stockpiled material should be protected from deleterious materials, additional moisture and be kept from freezing.

Quality control will be the utmost importance when selecting the material. The selection of the material should be done as early in the contract as possible to allow sufficient time for gradation and proctor testing on representative samples to ensure it meets the projects specifications.

Where the natural soil will be exposed, adequate compaction may prove difficult if the material becomes wet (i.e., above the optimum moisture content). Depending on the moisture content of the natural materials at the time of construction, they may either require moisture to be added or stockpiled and left to dry to achieve moisture content within plus 2% to minus 4% of optimum.

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

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

6.5 Preliminary Foundation Design

6.5.1 Shallow Foundations Bearing on Mineral Soill

It is understood that the P1 FFE will be approximately 3 mbgs (Elevation 153 ± masl). For geotechnical design purposes, the groundwater level may be taken at Elevation 154 ± masl.

The existing natural soil at that elevation is considered suitable to support the proposed building, provided all of the pavement structure and fill are removed, and the subgrade prepared as was described in Section 5.2.

Conventional shallow strip footings established on the inorganic hard silt or vey dense gravel, may be designed using a bearing resistance for 25 mm of settlement at Serviceability Limit States of 335 kPa, and a factored geotechnical bearing resistance of 500 kPa at Ultimate Limit States (ULS).

As the actual service loads were not known at the time of this report, these should be reviewed by the project structural engineer to determine if SLS or ULS governs the footing design.

It is noted that there is a potential for weaker subgrade soil to be encountered between the investigation locations. Any soft/loose areas are to be removed and replaced with a low strength concrete.

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

The natural subgrade soil is sensitive to change in moisture content and can become loose/soft if subjected to additional water or precipitation. As well, it could be easily disturbed if travelled on during construction. Once it becomes disturbed it is no longer considered adequate to support the recommended design bearing pressures. It is recommended that a working slab of lean concrete (mud slab) be placed in the footing areas immediately after excavation and inspection to protect the founding soils during placement of formwork and reinforcing steel.

In addition, to ensure and protect the integrity of the subgrade soil during construction operations, the following is recommended:

- Prior to commencing excavations, it is critical that all existing surface water, potential surface water and perched groundwater are controlled and diverted away from the work Site to prevent infiltration and subgrade softening. At no time should excavations be left open for a period of time that will expose them to inclement weather conditions and cause subgrade softening;
- The subgrade should be sloped to a sump outside the excavation to promote surface drainage and the collected water pumped out of the excavation. Any potential precipitation or seepage entering the excavations should be pumped away immediately (not allowed to pond);
- The footing areas should be cleaned of all deleterious materials such as topsoil, organics, fill, disturbed, caved materials or loosened bedrock pieces;
- Any potential large cobbles or boulders (i.e. greater than 200 mm in diameter) within the subgrade material are to be removed and replaced with a similar soil type not containing particles greater than 200 mm in diameter. It is critical that particles greater than 200 mm in diameter are not in contact with the foundation to prevent point loading and overstressing; and
- If the excavated subgrade soil remains open to weather conditions and groundwater seepage, sidewall stability and suitability of the subgrade soil will need to be verified prior to construction.

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

6.5.2 Shallow Foundations Bearing on Bedrock

Bedrock was encountered within all of the boreholes at depths ranging from 2.3 to 6.2 mbgs (Elevation 155.2 to 151.0 masl). The bedrock was cored under the footprint of the Buildings A and E. It was observed that the surface of the bedrock (approximately 2 meters) is highly weathered and the RQD classification of the bedrock was “very poor”. Below the highly weathered zone, the quality of the bedrock observed to be increased and RQD classification calculated to be “poor”.

Conventional shallow strip footings established on the highly weathered bedrock, may be designed using a bearing resistance for 25 mm of settlement at Serviceability Limit States of 500 kPa, and a factored geotechnical bearing resistance of 750 kPa at Ultimate Limit States (ULS).

For conventional shallow strip and spread footings established on the weathered bedrock (RQD > 25%), typically below Elevation 149.0 masl, a factored geotechnical bearing resistance of 4000 kPa may be used at ULS. Prior to installing foundation formwork, the bedrock is to be reviewed by a geotechnical engineer. SLS does not apply to foundations bearing directly on weathered bedrock, since the loads required for unacceptable settlements to occur would be much larger than the factored ULS and would be limited to the elastic compression of the bedrock and concrete.

The bearing resistance of 4000 kPa (weathered bedrock) assumes the bedrock is cleaned of all overburden material and any loose rock pieces. The bedrock should be cleaned with air or water pressure exposing clean sound bedrock. If construction proceeds during freezing weather conditions water should not be allowed to pool and freeze in bedrock depressions. All concrete should be installed and maintained above freezing temperatures as required by the concrete supplier.

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

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

6.5.3 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 6.0 to 9.3 mbgs and were terminated in the bedrock. SPT "N" values within the silt to gravel deposit ranged between 8 and greater than 50 blows per 300 mm. As such, based on Table 4.1.8.4.A of the OBC, this Site has been classified as Class C. A Site Class C has an average shear wave velocity (Vs) of between 360 and 760 m/s. It is recommended that shear wave velocity soundings be completed at the Site once final design and depths of foundations are known as a higher Site Classification may be available for deeper foundations at the Site.

6.5.4 Foundation Transition Zones

Excessive differential settlements can occur where the subgrade support material types differ below the underside of continuous strip footings, e.g., native soil to bedrock. As such, where strip footings transition from one material to another the transition between the materials should be suitably sloped or benched to mitigate differential settlements. This is especially critical when two different design bearing resistances are used for a building.

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

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

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

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

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

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

6.5.5 Estimated Settlement

All individual spread footings should be founded on uniform subgrade soils, reviewed and approved by a licensed geotechnical engineer.

Foundations installed in accordance with the recommendations outlined in the preceding sections are not expected to exceed total settlements of 25 mm and differential settlements of 19 mm.

All foundations are to be designed and constructed to the minimum widths as detailed in the 2012 OBC.

6.5.6 Building Drainage

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

Exterior perimeter foundations drains are not required, where the finished floor elevation is established a minimum of 150 mm above the exterior final grades or that the exterior gradient is properly sloped to divert surface water away from the building.

6.5.7 Shallow Foundations Frost Protection & Foundation Backfill

In the Mississauga, Ontario area, exterior perimeter foundations for heated buildings require a minimum of 1.2 m of soil cover above the underside of the footing to provide soil cover for frost protection. Foundations located immediately adjacent to air shafts, entrance and exit doors to underground parking levels 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.

To minimize potential frost movements from soil frost adhesion, the perimeter foundation backfill should consist of a free draining granular material, such as a Granular 'B' Type I (OPSS 1010) or an approved sand fill, extending a minimum lateral distance of 600 mm beyond the foundation. The existing organic-free sand and gravel material is suitable to reuse as foundation wall backfill. The backfill material used against the foundation must be placed so that the allowable lateral capacity is achieved. All granular material is to be placed in maximum 300 mm thick lifts compacted to a minimum of 100% SPMDD in hard landscaping areas and 95% SPMDD in soft landscaping areas. It is recommended that inspection and testing be carried out during construction to confirm backfill quality, thickness and to ensure compaction requirements are achieved.

6.6 Underground Parking Garage Design

It is understood that the Finished Floor Elevation (FFE) for the one underground parking (P1) will be set at depth of approximately 3 mbgs (approximate elevation 153 ± masl). Groundwater was encountered between Elevation 151.9 and 154.2 masl. For geotechnical design purposes, the groundwater level may be taken at Elevation 154 ± masl.

As such, there is a potential for the buildings to have to be designed to either resist hydrostatic uplift or to be provided with underfloor and foundation wall drainage systems connected to a suitable frost free outlet due to the groundwater levels at the Site. Once final design of the building is complete Pinchin should confirm this recommendation. Additional recommendations for the dewatering volume during the 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/m^3)

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

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

Alternatively, exterior perimeter foundation drains should be installed where subsurface walls are exposed to the interior. The foundation drains should consist of a minimum 150 mm diameter fabric wrapped perforated drainage tile surrounded by 19 mm diameter clear stone (OPSS 1004) with a minimum cover of 150 mm on top and sides and 50 mm below the drainage tile. 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/m3 for well compacted soil. The values provided in the table presented in Section 5.4 can be used for calculating the lateral earth pressure. An appropriate factor of safety should be applied.

6.7 Floor Slabs

The natural hard silt and very dense gravel or alternatively, the bedrock is considered suitable to support a floor slab for the proposed underground parking lot. Once the subgrade is exposed it should be inspected by a qualified geotechnical engineering consultant. Any soft area(s) encountered during the inspection should be excavated and replaced with a similar soil type.

The natural subgrade soil is to be proof roll compacted with a minimum 10 tonne non-vibratory steel drum roller to observe for weak/soft spots. It is noted that some locations will not be accessible by the steel drum roller; as such, these locations can be proof roll compacted with a minimum 450 kg vibratory plate compactor.

Based on the in-situ soil conditions, it is recommended to establish the concrete floor slab on a minimum 200 mm thick layer of Granular "A" (OPSS 1010). Alternatively, consideration may also be given to using a 200 mm thick layer of uniformly compacted 19 mm clear stone placed over the approved subgrade. Any required up fill should consist of a Granular "B" Type I or Type II (OPSS 1010).

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

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

Material Type	Modulus of Subgrade Reaction (kN/m^3)
Granular A (OPSS 1010)	85,000
Granular "B" Type I (OPSS 1010)	75,000
Granular "B" Type II (OPSS 1010)	85,000
Silt/Gravel/Weathered Bedrock	80,000

7.0 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.

8.0 TERMS AND LIMITATIONS

This Preliminary Geotechnical Investigation was performed for the exclusive use of Queenscorp (Erin Mills) Inc. (Client) in order to evaluate the subsurface conditions at 4099 Erin Mills Parkway, 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.

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

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

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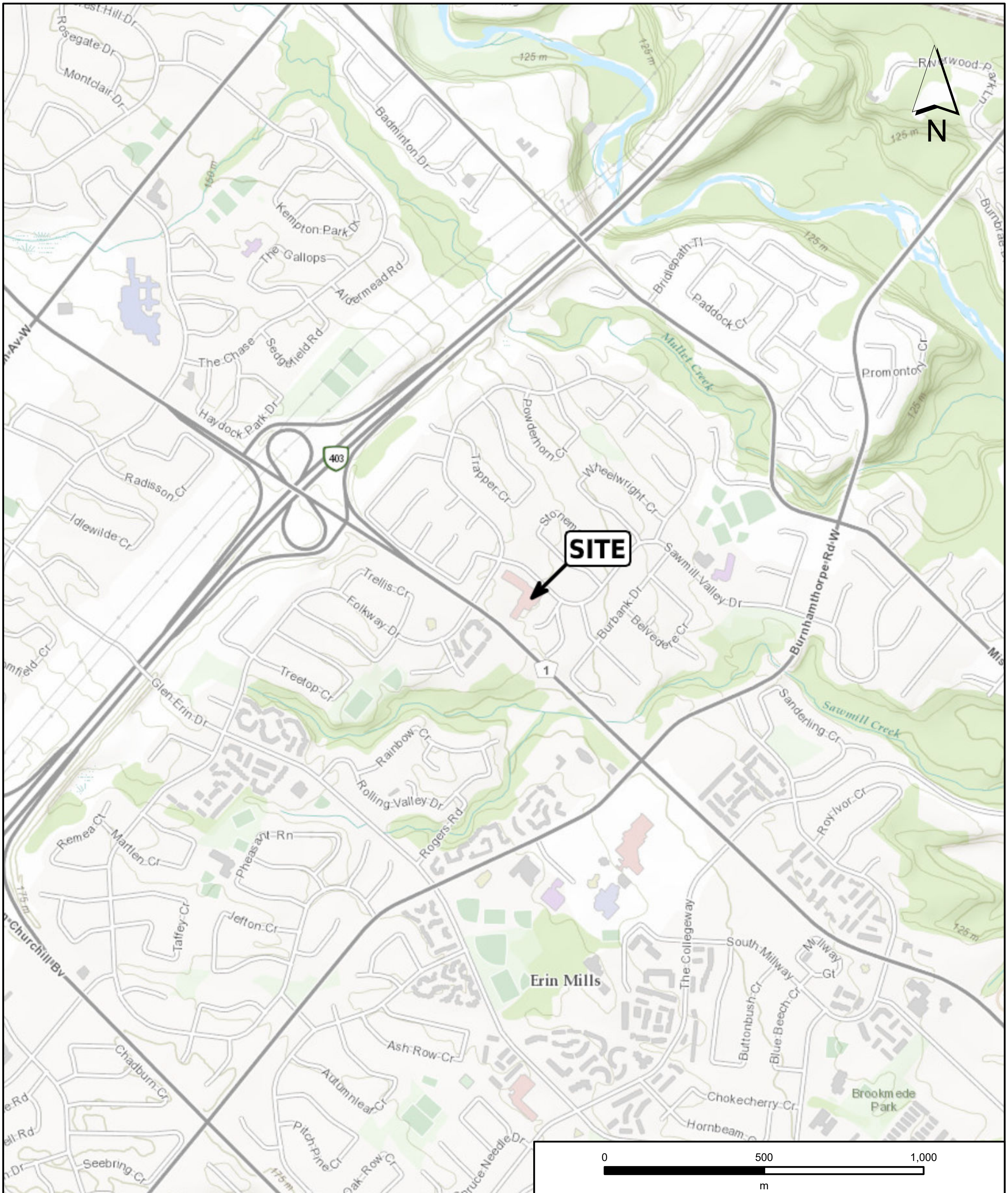
Template: Master Geotechnical Investigation Report – Ontario, GEO, September 2, 2021

FIGURES



PINCHIN	PROJECT NAME: PRELIMINARY GEOTECHNICAL INVESTIGATION				
	CLIENT NAME: QUEENSCORP (ERIN MILLS) INC.				
	PROJECT LOCATION: 4099 ERIN MILLS PARKWAY, MISSISSAUGA, ONTARIO				
	FIGURE NAME: KEY MAP				FIGURE NUMBER: 1
	PROJECT NUMBER: 299799	SCALE: 1:22,000	DRAWN BY: KP	REVIEWED BY: RM	DATE: DECEMBER 2021

N

LEGEND

SITE BOUNDARY

SITE BUILDING

[#] GROUND ELEVATION IN masl

(#) UNIT NUMBER

masl METRES ABOVE SEA LEVEL

MONITORING WELL

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

PINCHIN

PROJECT NAME: PRELIMINARY GEOTECHNICAL INVESTIGATION

CLIENT NAME: QUEENSCORP (ERIN MILLS) L.P.

PROJECT LOCATION: 4099 ERIN MILLS PARKWAY, MISSISSAUGA, ONTARIO

FIGURE NAME: MONITORING WELL LOCATION PLAN

PROJECT NUMBER: 299799	APPROXIMATE SCALE: AS SHOWN
DRAWN BY: KP	REVIEWED BY: RM
DATE: DECEMBER 2021	FIGURE NUMBER: 2

APPENDIX I

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

ABBREVIATIONS, TERMINOLOGY & PRINCIPAL SYMBOLS USED

Sampling Method

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

In-Situ Soil Testing

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

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

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

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

Soil Descriptions

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

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

Notes:

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

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

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

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

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

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

Soil & Rock Physical Properties

General

W Natural water content or moisture content within soil sample

γ Unit weight

γ' Effective unit weight

γ_d Dry unit weight

γ_{sat} Saturated unit weight

ρ Density

ρ_s Density of solid particles

ρ_w Density of Water

ρ_d Dry density

ρ_{sat} Saturated density e Void ratio

n Porosity

S_r Degree of saturation

E_{50} Strain at 50% maximum stress (cohesive soil)

Consistency

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

Shear Strength

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

Consolidation (One Dimensional)

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

Permeability

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

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

Rock Coring

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

RQD is calculated as follows:

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

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

Project #: 299799.000 **Logged By:** HE

Project: Preliminary Geotechnical Investigation

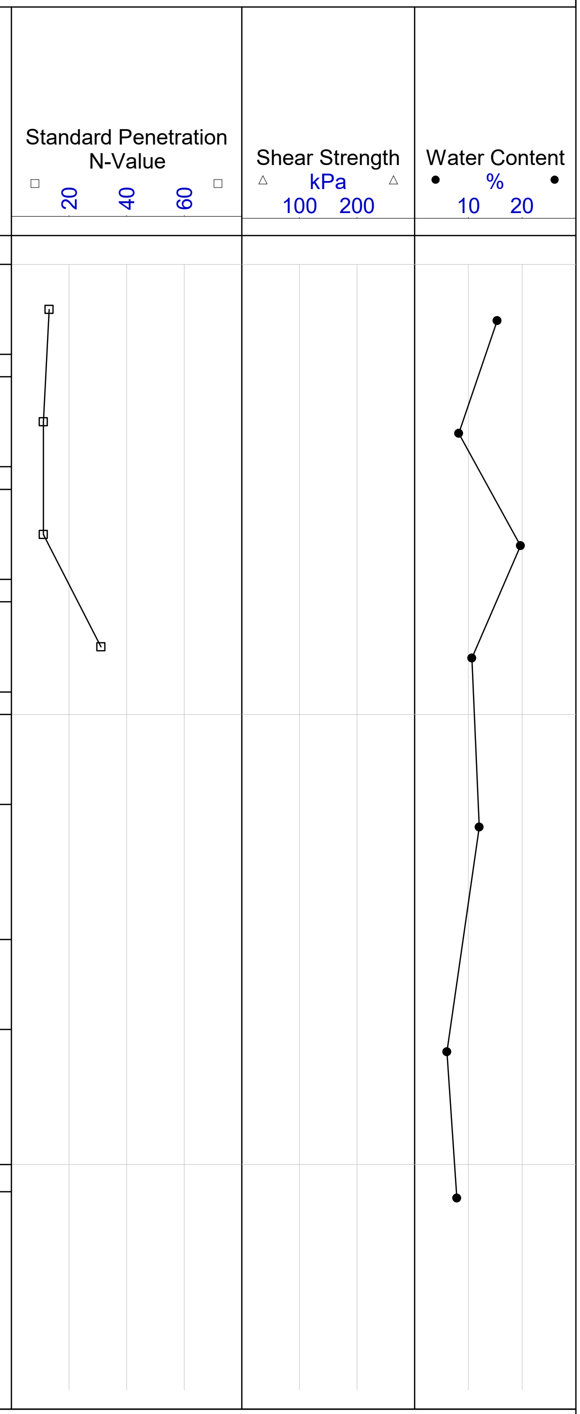
Client: Queenscorp (Erin Mills) Inc.

Location: 4099 Erin Mills Parkway, Mississauga, Ontario

Drill Date: December 02, 2021 **Project Manager:** RM

SUBSURFACE PROFILE / SAMPLE

Depth (m)	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-Value
0	Ground Surface	158.90					
	Asphalt	158.60					
	Fill Light brown sand, some gravel, compact, very moist	158.14		SS	1	50	13
1	Light grey silt, trace sand, gravel, clay and rootlets, compact, very moist			SS	2	5	11
	Brown, sandy, some clay, oxidation	157.38					
2	Red and light brown oxidation, trace wood pieces	156.62		SS	3	40	11
	Gravel Brown gravel (gravel and cobble fragments), trace sand and clay, dense, very moist	155.85	Riser	SS	4	5	31
3	**Silt** Reddish/brown stratified sandy silt (fragments from weak and weathered bedrock), very hard, moist	154.33	Bentonite	SS	5	10	>75
4–5	**Gravel** Reddish brown gravel (weak bedrock fragments), trace sand and silt, very dense, moist	152.71		SS	6	20	>50
6				SS	7	5	>50
7	**Bedrock** Red highly weathered shale with greenish-grey reduction zone RQO=6 %		Screen; Silica Sand	RC	8	100	



Contractor: Strata Drilling Group **Grade Elevation:** 158.90 masl

Drilling Method: Split Spoon / Hollow Stem Auger, HQ-Rock Coring **Top of Casing Elevation:** 158.85 masl

Well Casing Size: 51 mm **Sheet:** 1 of 2

Log of Borehole: BH1

Project #: 299799.000 **Logged By:** HE

Project: Preliminary Geotechnical Investigation

Client: Queenscorp (Erin Mills) Inc.

Location: 4099 Erin Mills Parkway, Mississauga, Ontario

Drill Date: December 02, 2021 **Project Manager:** RM

SUBSURFACE PROFILE					SAMPLE						
Depth (m)	Symbol	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-Value	Standard Penetration N-Value (20, 40, 60)	Shear Strength kPa (100, 200)	Water Content % (10, 20)
			151.13								
8–9		RQD = 38%	149.57		RC	9	100				
		End of Borehole		Groundwater level measured to be at 4.7 mbgs on December 7, 2021							

Depth scale: 8, 9, 10, 11, 12, 13, 14, 15

Contractor: Strata Drilling Group

Drilling Method: Split Spoon / Hollow Stem Auger, HQ-Rock Coring

Well Casing Size: 51 mm

Grade Elevation: 158.90 masl

Top of Casing Elevation: 158.85 masl

Log of Borehole: BH2

Project #: 299799.000 **Logged By:** HE

Project: Preliminary Geotechnical Investigation

Client: Queenscorp (Erin Mills) Inc.

Location: 4099 Erin Mills Parkway, Mississauga, Ontario

Drill Date: December 03, 2021 **Project Manager:** RM

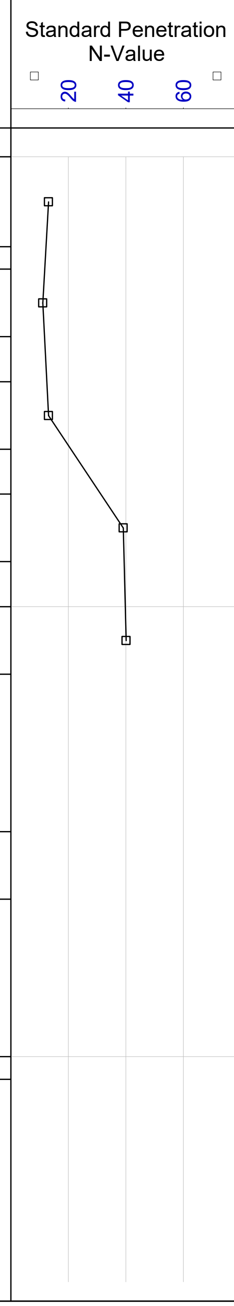
SUBSURFACE PROFILE

Depth (m)	Description	Elevation (m)
0	Ground Surface	156.10
0.0–0.3	**Asphalt**	155.80
	Fill Granular A type fill	
	Grey sand, some gravel, compact, very moist	155.34
	Dark brown silt, some sand, trace gravel, rootlets, woodpieces and yellow/white oxidation , stiff, very moist	154.58
	trace dark organics	153.81
	Sand Grey silty sand, some gravel and cobble fragments, dense, moist	153.05
	Silt Clayey silt, some sand and gravel, hard, moist	151.53
	Bedrock Reddish grey highly weathered rock fragments, some sand, very dense, damp	150.00
	Reddish brown, some clay, very moist	
	End of Borehole	
	At the drilling completion, water was measured to be at 2.7 mbgs	

Monitoring Well Details: Riser; Bentonite; Screen; Silica Sand. Groundwater level measured to be at 3.5 mbgs on December 7, 2021

SAMPLE

Sample Type	Sampler #	Recovery (%)	SPT N-Value
SS	1	40	13
SS	2	20	11
SS	3	30	13
SS	4	90	39
SS	5	50	40
SS	6	100	>75
SS	7	100	>50



Shear Strength kPa 100 200

Water Content % 10 20

Contractor: Strata Drilling Group **Grade Elevation:** 156.10 masl

Drilling Method: Split Spoon / Hollow Stem Auger **Top of Casing Elevation:** 155.98 masl

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

Log of Borehole: BH3

Project #: 299799.000
Logged By: HE
Project: Preliminary Geotechnical Investigation
Client: Queenscorp (Erin Mills) Inc.
Location: 4099 Erin Mills Parkway, Mississauga, Ontario
Drill Date: December 03, 2021
Project Manager: RM

SUBSURFACE PROFILE

Depth (m)	Description	Elevation (m)
0	Ground Surface	157.50
0 – 0.61	**Asphalt** **Fill** Granular A type fill (Grey sand, some gravel, compact, very moist)	156.89
0.61 – 1.52	**Silt** Reddish/brown silt, trace sand, occasional cobbles (highly stratified, rock fragments), very dense, damp	155.98
1.52 – 2.29	**Gravel and Crushed Rock** Grey stratified rock fragments, very dense, damp	155.21
2.29 – 5.79	**bedrock** Reddish brown highly weathered bedrock, some sand	151.71
	End of Borehole At drilling completion, the borehole was open and dry.	

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

Groundwater level measured to be at 4.9 mbgs on December 7, 2021

SAMPLE

Sample Type	Sampler #	Recovery (%)	SPT N-Value
SS	1	50	17
SS	2	70	>75
SS	3	80	>75
SS	4	100	>50

Standard Penetration N-Value: 20, 40, 60
Shear Strength kPa: 100, 200
Water Content %: 10, 20

Contractor: Strata Drilling Group
Grade Elevation: 157.50 masl
Drilling Method: Split Spoon / Hollow Stem Auger
Top of Casing Elevation: 157.42 masl
Well Casing Size: 51 mm
Sheet: 1 of 1

Log of Borehole: BH4

Project #: 299799.000 **Logged By:** HE

Project: Preliminary Geotechnical Investigation

Client: Queenscorp (Erin Mills) Inc.

Location: 4099 Erin Mills Parkway, Mississauga, Ontario

Drill Date: December 02, 2021 **Project Manager:** RM

SUBSURFACE PROFILE

Depth (m)	Description	Elevation (m)
0	Ground Surface	155.70
0–0.30	**Asphalt**	155.40
0.30–0.76	**Fill** Granular fill	154.94
0.76–1.52	**Sand** Grey sand, some gravel, compact, wet	154.18
1.52–2.29	Greyish brown silty sand, trace clay and red/black dots oxidation, occassional cobbles, compact,	
	Silt Reddish brown clayey silt, some sand trace gravel, hard, dry	153.41
2.29–3.05	**Sand** Greyish brown sand, trace gravel and weak light brown layers of fine sand/gravel, very dense, moist	152.65
3.05–3.51	Reddish brown sand, some gravel, trace silt and weak shale rock stratified fragments,	152.19
3.51–4.69	**Gravel** Reddish rock fragments, very dense, very moist	151.01
4.69–6.37	**Bedrock** Red shale bedrock with greenish-grey reductin zones RQD = 6%	149.33
6.37–	RQD = 29%	

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

SAMPLE

Sample Type	Sampler #	Recovery (%)	SPT N-Value
SS	1	40	19
SS	2	30	21
SS	3	40	41
SS	4	50	>75
SS	5	40	>75
	6		>50
RC	7	100	
RC	8	100	

Standard Penetration N-Value: 20 40 60

Shear Strength kPa: 100 200

Water Content %: 10 20

Contractor: Strata Drilling Group

Grade Elevation: 155.70 masl

Drilling Method: Split Spoon / Hollow Stem Auger, HQ-Rock Coring

Top of Casing Elevation: 155.64 masl

Well Casing Size: 51 mm

Sheet: 1 of 2

Log of Borehole: BH4

Project #: 299799.000 **Logged By:** HE

Project: Preliminary Geotechnical Investigation

Client: Queenscorp (Erin Mills) Inc.

Location: 4099 Erin Mills Parkway, Mississauga, Ontario

Drill Date: December 02, 2021 **Project Manager:** RM

SUBSURFACE PROFILE					SAMPLE						
Depth (m)	Symbol	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-Value	Standard Penetration N-Value 20 40 60	Shear Strength kPa 100 200	Water Content % 10 20
			147.78								
8		End of Borehole		Groundwater level measured to be at 3.8 mbgs on December 7, 2021							
9											
10											
11											
12											
13											
14											
15											

Contractor: Strata Drilling Group **Grade Elevation:** 155.70 masl

Drilling Method: Split Spoon / Hollow Stem Auger, HQ-Rock Coring **Top of Casing Elevation:** 155.64 masl

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

Log of Borehole: BH5

Project #: 299799.000 **Logged By:** HE

Project: Preliminary Geotechnical Investigation

Client: Queenscorp (Erin Mills) Inc.

Location: 4099 Erin Mills Parkway, Mississauga, Ontario

Drill Date: December 03, 2021 **Project Manager:** RM

SUBSURFACE PROFILE

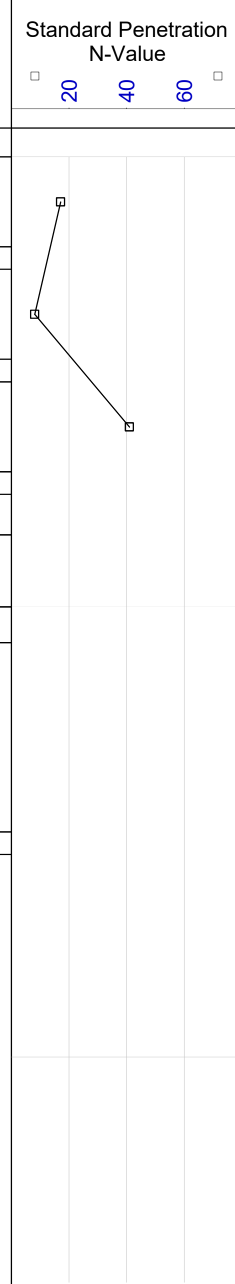
Depth (m)	Description	Elevation (m)
0	Ground Surface	157.50
0 – 0.46	**Asphalt**	157.04
0.46 – 0.61	**Fill** Granular fill	156.89
0.61 – 1.52	**Sand** Grey sand, some gravel, compact, very moist	155.98
1.52 – 2.29	**Silt** Greyish green clayey sandy silt trace gravel, high oxidation, loose	155.21
2.29 – 3.05	**Sand** Reddish brown silty sand trace gravel, very dense, damp	154.45
3.05 – 4.88	**Gravel** Reddish brown gravel (highly stratified weak rock fragments), some sand, very dense, damp	152.62
4.88 – 6.10	**Bedrock** Grey Shale bedrock	151.40
	End of Borehole At drilling completion, the borehole was open and dry	

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

Groundwater level measured to be at 3.7 mbgs on December 7, 2021

SAMPLE

Sample Type	Sampler #	Recovery (%)	SPT N-Value
SS	1	60	17
SS	2	40	8
SS	3	50	41
SS	4	100	>75
SS	5	100	>75
SS	6	100	>50



Shear Strength kPa 100 200

Water Content % 10 20

Contractor: Strata Drilling Group **Grade Elevation:** 157.50 masl

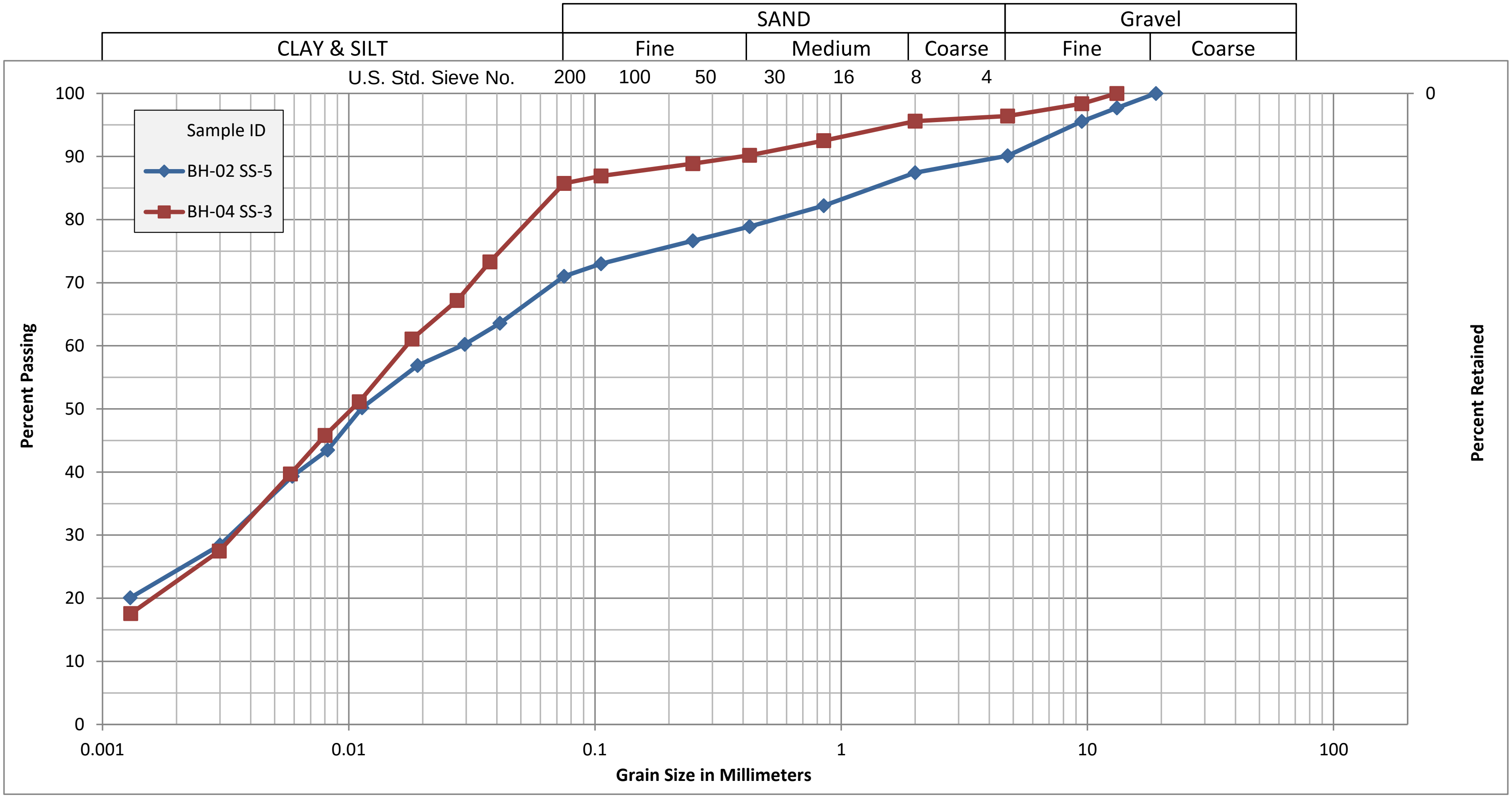
Drilling Method: Split Spoon / Hollow Stem Auger **Top of Casing Elevation:** 157.40 masl

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
BH-02 SS-5	10.0-12.0	10.0	19.0	47.0	24.0
BH-04 SS-3	5.0-6.5	4.0	10.2	63.8	22.0

PINCHIN

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

PARTICLE SIZE DISTRIBUTION ANALYSIS

Preliminary Geotechnical Investigation - 4099 Erin Mills Pkwy, Mississauga, ON
Queenscorp Group

Figure No. **1**

299799.000

Reviewed By:

More information available upon request

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

December 20, 2021

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

Re: UCS testing
(Pinchin Project 299799)

Dear Mr. Mahmoudipour:

On December 13th, 2021 one (1) NQ-sized core sample was received by Geomechanica Inc. via drop off by Pinchin personnel. This samples were identified as being from Pinchin Project No. 299799. One (1) UCS specimen was prepared and tested.

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

Sincerely,

Bryan Tatone Ph.D., P. Eng.

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

GEOMECHANICA

Rock Laboratory Testing Results

A report submitted to:

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

Prepared by:

Bryan Tatone, PhD, PEng

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

December 20, 2021
Project number: 299799

Abstract

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

In this document:

1 Uniaxial Compressive Strength Tests

1.1 Overview

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

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

Figure 1: Forney loading frame setup for UCS testing.

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

1.2 Results

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

Table 1: Summary of Uniaxial Compression test results.

Sample	Depth (m)	Bulk density ρ (g/cm^3)	UCS (MPa)	Lithology	Failure description
BH1-RC 9	N/A - N/A	2.577	26.9	Red shale with greenish-grey reduction zones	1

[1] Axial splitting failure

1.3 Specimen photographs

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

Appendices

Specimen sheets

- BH1-RC 9

Uniaxial Compression Test

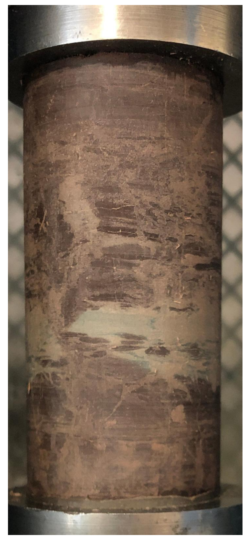
Client	Pinchin Ltd.	**Project**	299799
Sample	BH1-RC 9	**Depth**	7.8 to 9.3 mbgs

Specimen parameters

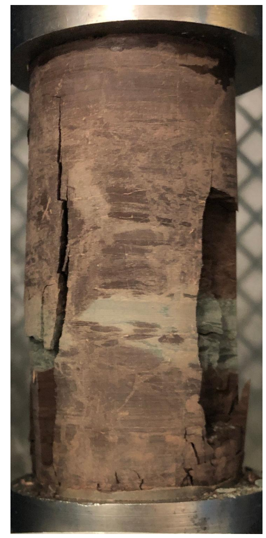
Diameter (mm) [a]	47.46
Length (mm) [a]	96.68
Bulk density ρ (g/cm^3)	2.577
UCS (MPa)	26.9
Lithology	Red shale with greenish-grey reduction zones
Failure description [b]	1

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

Prior to testing



After testing



Remarks: Loading rate: 0.15 mm/min. Specimen experiences some pre-peak localized failure.

Performed by	AA/HS	**Date**	2021-12-17

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.