FINAL

Geotechnical Investigation – Proposed Building Expansion

500 Veterans Drive, Barrie, Ontario

Prepared for:

Summit Industrial Income Operating LP
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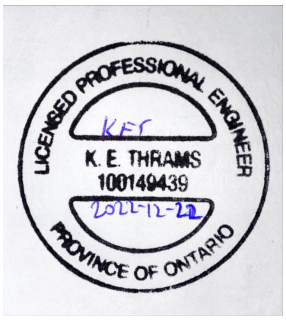

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1.0 INTRODUCTION AND SCOPE

Pinchin Ltd. (Pinchin) was retained by Summit Industrial Income Operating LP (Client) to conduct a Geotechnical Investigation and provide subsequent geotechnical design recommendations for the proposed building expansion to be located at 500 Veterans Drive, Barrie, Ontario (Site). The Site location is shown on Figure 1.

The Site is currently occupied by a single-storey industrial building with a partial two-storey office. It is Pinchin's understanding that the Client intends to expand the building towards the southern portion of the Site with an approximately 7,000 m^2 addition including 12 additional loading docks along the west side.

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

The purpose of the Geotechnical Investigation was to delineate the subsurface conditions and soil engineering characteristics at the Site by advancing boreholes in conjunction with the Phase Two Environmental Site Assessment (ESA) and hydrogeological assessment.

Based on a desk top review and the results of the 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 including engineered fill specifications and installation;
- Open cut excavations;
- Anticipated groundwater management;
- Site service trench design;
- Foundation design recommendations including soil bearing resistances at Ultimate Limit States (ULS) and Serviceability Limit States (SLS) design for shallow foundations;
- Potential total and differential settlements;
- Foundation frost protection;
- Seismic Site classification for seismic Site response;
- Concrete floor slab-on-grade support recommendations;
- Lateral earth pressure coefficients and unit densities;
- Asphaltic concrete pavement structure design for the pavement expansion; and,
- Potential construction concerns.

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

The Phase Two ESA and hydrogeological assessment will be issued under separate covers.

2.0 SITE DESCTIPTION AND GEOLOGICAL SETTING

The Site is located at the southwest corner of the intersection of Veterans Drive and Caplan Avenue in Barrie, Ontario. The Site is bounded by vacant lands to the south and lands under development to the west. The central and northern portions of the Site currently comprise the existing building with loading bays along the building's west side and associated asphalt pavements to the west and north. The southern portion is currently undeveloped. The topography in the local and surrounding areas is generally flat. The northeast corner of the site is near a local high with grades across the site gradually sloping to the southwest and the area north of the subject Site gradually sloping to the northeast towards Lover's Creek.

Data obtained from the Ontario Geological Survey Maps, as published by the Ontario Ministry of Natural Resources, indicates that the Site is underlain by ice-contact stratified deposits consisting of sand and gravel, minor silt, clay and till (Ontario Geological Survey 2010, Surficial Geology of Southern Ontario; Ontario Geological Survey, Miscellaneous Release--Data 128-REV). Coarse-textured glaciolacustrine deposits (sand, gravel, minor silt and clay) are mapped to the south and glacial till deposits (stone-poor, sandy silt to silty sand-textured till) to the east and west of the Site. The underlying bedrock at this Site is of the Simcoe Group consisting of limestone (Ontario Geological Survey 2007, Paleozoic Geology of Southern Ontario; Ontario Geological Survey, Miscellaneous Release--Data 219). The bedrock surface is expected at depths greater than 100 m at the subject Site (Bedrock topography and overburden thickness mapping, southern Ontario; Ontario Geological Survey, Miscellaneous Release--Data 207).

3.0 GEOTECHNICAL FIELD INVESTIGATION AND METHODOLOGY

Pinchin completed field investigations at the Site from July 6 to 12, 2022 by advancing a total of eight boreholes (boreholes BH01 through MW08) at the Site. The boreholes were advanced to depths of approximately 1.5 to 11.3 metres below existing ground surface (mbgs). The approximate spatial locations of the boreholes advanced at the Site are shown on Figure 2.

The boreholes were advanced with a drill-rig equipped with direct push and 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.

Monitoring wells were installed in three of the boreholes (boreholes MW03, MW05, and MW08) to allow measurement of stabilized 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.

The remaining boreholes were backfilled in accordance with O.Reg 903, as amended.

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 July 13 and 14, 2022. The groundwater observations and measurements recorded are included on the appended borehole logs.

The borehole locations and ground surface elevations boreholes BH02 to MW08 were surveyed by Pinchin using a Sokkia Model GRX 2 Global Navigation Satellite System (GNSS) rover. The ground surface elevations are geodetic, based on GNSS and local base station telemetry with a precision static of less than 20 mm.

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

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

Select soil samples collected from the boreholes were submitted to Pinchin's material testing laboratory to determine moisture contents, grain size distribution, and Atterberg Limits 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 encountered at the boreholes consisted of fill or topsoil underlain by sand and glacial tills. Bedrock was not contacted at the borehole locations over the depths explored.

The appended borehole logs provide detailed soil descriptions and stratigraphies, results of SPT, moisture content profiles, details of monitoring well installations, and groundwater measurements.

4.1.1 Fill and Possible Fill

Fill was contacted at ground surface at boreholes BH02 and MW05 to MW08 and extended to depths of 2.3 to 3.4 mbgs. The upper fill generally consisted of 100 to 150 mm topsoil fill. The underlying fill varied in composition from sand with variable silt content (trace silt to silty) and trace to some gravel to sand and gravel. Trace to some organics were noted within the fill at boreholes BH02, BH07, and MW08. The results of one particle size distribution analysis completed on a sample of the fill are provided in Appendix III and indicate that the sample contained 4% gravel, 61% sand, 30% silt, and 5% clay. At the time of sampling the fill was damp to wet with laboratory determined moisture contents ranging from 2 to 20%. SPT N-values per 300 mm in the fill ranged from 2 to 32 blows per 300 mm indicating variable very loose to dense relative densities.

Possible fill was contacted at ground surface at borehole BH01 and extended to 0.8 mbgs. The possible fill consisted of silty sand with trace gravel and organics. The possible fill was moist to very moist with a laboratory determined moisture content of 15%.

4.1.2 Topsoil

Topsoil was encountered at borehole MW03 and BH04 and was 75 and 125 mm thick, respectively.

4.1.3 Sand

Sand was contacted at most boreholes underlying the fill or topsoil. The sand generally ranged in composition from sand with variable silt content (trace silt to silty) and trace to some gravel. Occasional cobbles and silt pockets/inclusions were noted within the sand. The results of one particle size distribution analysis completed on a sample of the sand are provided in Appendix III and indicate that the sample

contained 5% gravel, 69% sand, 20% silt, and 6% clay. SPT N-values within the sand generally ranged from 11 to greater than 50 blows per 300 mm, indicating compact to very dense relative densities. At borehole BH02, the sand was loose to a depth of 3.1 mbgs. At the time of sampling the sand was described as damp to wet. Laboratory determined moisture contents ranged from 3 to 18%.

4.1.4 Silt Till and Clay Till

Silt till was contacted at borehole MW08 underlying the sand at 7.6 mbgs. The silt till consisted of silt with some sand and trace clay and gravel. The silt till was moist with a laboratory determined moisture content of 9%. The silt till had a very dense relative density based on an SPT N-value of greater than 50 blows per 300 mm.

Clay till was contacted at boreholes BH02, MW03, MW05, BH06, and MW08 underlying the fill, sand, or silt till at depths of 2.3 to 9.1 mbgs and generally extending beyond the borehole termination depths. At borehole MW05, the clay till was underlain by sand at 9.1 mbgs. The clay till generally ranged in composition from silty clay with trace sand and gravel to silty sandy clay with trace gravel. The results of one particle size distribution analysis and one Atterberg Limits test completed on a sample of the clay till are provided in Appendix III. The results indicate that the sample contained 16% gravel, 41% sand, 28% silt, and 15% clay. A plastic limit of 10% and a liquid limit of 17% were determined. Laboratory determined moisture contents varied from 7% to 28%. At the time of sampling, the clay till was described as drier than plastic limit to about the plastic limit. The clay till was very stiff to hard based on SPT N-values of 19 to greater than 50 blows per 300 mm.

4.2 Groundwater Conditions

Groundwater observations and measurements were obtained in the open boreholes at the completion of drilling and are summarized on the appended borehole logs. Stabilized groundwater levels were measured on July 13 and 14, 2022 and the results are summarized in the following table:

Borehole No.	Ground Surface Elevation (masl)	Date	Depth to Groundwater (mbgs)	Groundwater Elevation (masl)
MW03	311.0	July 14, 2022	5.6	305.4
MW05	310.4	July 13, 2022	7.1	303.3
MW08	311.2	July 14, 2022	6.0	305.2

Wet conditions were noted within the fill at borehole MW08. Perched groundwater conditions at higher elevations should be expected within the fill materials or loose sand deposit overlying denser material or glacial till, especially following snowmelt in the Spring or following rain events.

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.

Reference is made to the hydrogeological assessment for further details pertaining to the groundwater regime below the Site.

5.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

5.1 General Information

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

It is Pinchin's understanding that the development will consist of an approximately 7,000 m^2 slab-on-grade (i.e. no basement level) addition to the south of the existing building, including 12 additional loading docks. Boreholes BH02 to MW08 were advanced within the area of the proposed addition. Grades at these borehole locations varied from 310.2 to 311.3 masl. Proposed grades or finished floor elevations were not available; however, it is anticipated that the finished floor of the addition will match the existing building and that site grades will generally be maintained. Proposed foundation details such as underside of footing elevation, proposed loads, and footing dimensions were not known.

The soils contacted at the borehole locations consisted of fill or topsoil underlain by sand and glacial till deposits. The native soils were generally compact to very dense; however, a loose layer of sand was contacted at borehole BH02 to 3.1 mbgs. Groundwater was measured at depths of 5.6 to 7.1 mbgs (elevations 303.3 to 305.4 masl).

The following table provides a summary of the depth to competent soil as contacted at the boreholes:

Borehole No.	Ground Surface Elevation (masl)	Depth to Native Inorganic Compact to Dense / Very Stiff to Hard Soil (mbgs)	Elevation of Native Inorganic Compact to Dense / Very Stiff to Hard Soil (masl)
BH02	311.1	3.1	308.0
MW03	311.0	0.1	310.9
BH04	311.3	0.1	311.2
MW05	310.4	2.3 [1]	308.1
BH06	310.8	2.3 [2]	308.5
BH07	310.2	3.1	307.1
MW08	311.2	3.4	307.9

Note: 1 - compact sand contacted below clay till at 9.1 mbgs (301.3 masl).
2 – Borehole advanced with direct push (assessment of soil strength not available)

Any pre-existing fill that was placed unmonitored, topsoil, and soil containing organics is not considered suitable for support of the proposed building expansion and site servicing and should be removed. Localized loose soils should be subexcavated and grades raised with engineered fill. Alternatively, the use of a Rammed Aggregate Pier (RAP) system could be considered to improve the soils below the proposed building.

The existing fill may be suitable to remain below proposed paved areas, provided the exposed subgrade is proof rolled during geotechnical inspection to identify any localized areas of softer subgrade requiring pavement subbase thickening, and the owner accepts the potential for long-term settlement from the undocumented fill. If the potential for settlement below paved areas will not be accepted, then the existing fill must also be subexcavated from below proposed paved areas and replaced with subgrade fill. The use of a geogrid-soil system could be considered where pre-existing fill is to remain below the proposed pavement structures. The geogrid-soil system provides a stiff mechanically stabilized layer able to control differential settlements.

5.2 Site Preparation

Existing utilities not to be maintained should be removed and/or relocated. All fill materials, topsoil, and native soil with organic content, or loose soils should be removed from below the proposed building footprint and site services. Fill materials may remain below proposed pavement areas if the owner accepts the increased potential for required pavement maintenance and repairs as well as potential for decreased pavement life.

Disturbed or very loose/softened soils at foundation level should be subexcavated and replaced with structural fill. Extra care must be taken during subexcavations near the existing building to ensure the existing building's footings are not undermined.

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

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

Structural fill placed below foundations should extend horizontally 1 m beyond the edge of foundation. The structural fill pad should be sloped at an inclination of 1 horizontal to 1 vertical to the approved subgrade soil. Engineered fill placed on slopes steeper than 3.0H to 1.0V should be benched into the existing slopes. The benching height should match the engineered fill lift thickness.

Prior to placing any fill material at the Site, the subgrade should be inspected by a qualified geotechnical engineer, and loosened/soft pockets should be sub excavated and replaced with engineered fill. In addition, the natural subgrade soil is to be roll compacted with a minimum 10 tonne non-vibratory steel drum roller to provide a more uniform subgrade surface and compact near surface loose zones.

Inorganic on-site soils may be suitable for reuse as engineered fill to achieve design grades. Depending on the moisture content at the time of construction some moisture condition will need to be done. Any import material required should comprise Ontario Provincial Standards and Specifications (OPSS) 1010 Granular 'B' Type I material or Select Subgrade Material (SSM). If the work is carried out during very dry weather, water may have to be added to the material to improve compaction.

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

Dust control measures may be required during periods of dry weather. Work shall be in accordance with OPSS 506.

5.3 Open Cut Excavations

It is anticipated that the foundations will be constructed at conventional frost depths, approximately 1.5 metres below finished floor elevation. Excavations for removal of existing fill may extend more than 3 mbgs.

Based on the subsurface information obtained from within the boreholes, it is anticipated that the excavated material will predominately consist of fill underlain by loose to very dense / very stiff to hard native soils. Groundwater was measured at 5.6 to 7.1 mbgs; however, there is potential for localized perched groundwater at shallower depths.

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.

Based on the OHSA, the predominant soils 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 through organic soils or 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. Where more than one soil type is exposed in an excavation, the soil type with the higher number governs.

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.

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.

Groundwater was measured at 5.6 to 7.1 mbgs on July 13/14, 2022. Excavations for conventional foundation construction are not expected to extend into the stabilized groundwater table below the Site.

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

Reference is made to the hydrogeological assessment for further details regarding groundwater conditions below the Site.

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.

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.

5.4 Site Servicing

5.4.1 Pipe Bedding and Cover Materials for Flexible and Rigid Pipes

The subgrade soil conditions beneath the site services are expected to comprise approved engineered fill or natural sand and glacial till soils. It is recommended 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 100% Standard Proctor Maximum Dry Density (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.

5.4.2 Trench Backfill

Following placement of the pipe bedding cover the trench shall be backfilled. Based on the results of the natural inorganic overburden deposits, the on-Site sand and glacial till soils are generally considered suitable for re-use on-site provided any cobbles and/or boulders are removed; however, this should be confirmed at the time of construction. Depending on the moisture content at the time of construction, water may need to be added to the sand soils to aid in proper compaction. Further, the glacial till will have a blocky/lumpy texture, and a sheepsfoot roller is recommended for any soils considered suitable for on-site reuse in order to achieve proper compaction and to ensure that all air voids are removed to avoid long-term softening and settlement. Any material considered for reuse on-site should be moisture conditioned to achieve moisture content within plus 2% to minus 4% of optimum. Stockpiles should be protected to help minimize moisture absorption during wet weather.

Where the natural soil will be exposed, adequate compaction may prove difficult if the material becomes wet (i.e., above the optimum moisture content).

Where required, imported material should be used 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 205.060. Imported material should consist of a Granular “A”, Granular “B” Type I, or SSM (OPSS 1010).

The trench backfill 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 plus 2% to minus 4% of the optimum moisture content. The natural material must be free of organics or other deleterious material. 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.

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.

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

5.4.3 Frost Protection

The frost penetration depth in Barrie, Ontario for these types of soil conditions is estimated to extend to approximately 1.4 mbgs in open roadways cleared of snow. As such, it is recommended to place water services at a minimum depth of 300 mm below this elevation with the top of the pipe located at 1.7 mbgs or lower as dictated by municipal service requirements. If a minimum of 1.7 m of soil cover cannot be provided, then the pipe should be insulated with a rigid polystyrene insulation or a pre-insulated pipe be utilized.

The insulation design configuration may either consist of placing horizontal insulation to a specified design distance beyond the outside edge of the pipe or an inverted “U” surrounding the top and sides of the pipe. Any method chosen requires suitable design and installation in accordance with the manufacturer’s recommendations. To accommodate the placement of horizontal insulation a wider excavation trench may be required.

5.5 Foundation Design

As noted previously, the pre-existing undocumented fill, topsoil, soils with organic content or in loose condition are not considered suitable for foundation support. The following subsections of the report provide recommendations on the above noted foundation options that are considered suitable for the proposed development.

Deep foundations such as caissons founded in the dense to very dense sand or very stiff / hard clay till may provide higher bearing resistances. The presence of saturated sands as contacted below the clay till at MW05 at 9.1 mbgs will need to be considered and dewatering would likely be required for caissons extending below the groundwater table. Pinchin can provide additional recommendations should caissons be considered for the Site.

5.5.1 Conventional Shallow Foundations Bearing on Structural Fill/Compact Native Soils

Conventionally shallow strip or square foundations constructed on approved structural fill placed following removal of any underlying loose soils, or compact native soils may be designed for a bearing resistance at SLS of 150 kPa and a factored geotechnical bearing resistance of 225 kPa at ULS. A resistance factor of 0.5 was applied to the ULS.

The associated total and differential settlements for these pressures are anticipated to be less than 25 mm and 19 mm, respectively for continuous foundations less than 1.5 m in width and spread (square) footings of up to 3.0 m.

5.5.2 Conventional Shallow Foundations Bearing on Dense/Very Stiff Native Soils

Higher bearing resistances are available for conventional footings lowered onto the dense / very stiff to hard native inorganic soils contacted at depth of 2.3 to 3.4 mbgs as summarized in the following table:

Borehole No.	Ground Surface Elevation (masl)	Depth to Native Inorganic Dense / Very Stiff to Hard Soil (mbgs)	Elevation of Native Inorganic Dense / Very Stiff to Hard Soil (masl)
BH02	311.1	3.1	308.0
MW03	311.0	2.3	308.7
BH04	311.3	2.3	309.0
MW05	310.4	2.3 [1]	308.1
BH06	310.8	No SPT data collected	No SPT data collected
BH07	310.2	3.1	307.1
MW08	311.2	3.4	307.8

Note: 1 - compact sand contacted below clay till at 9.1 mbgs (301.3 masl).

Conventional shallow strip or square foundations constructed on dense or very stiff to hard native soils may be designed for a bearing resistance at SLS of 200 kPa and a factored geotechnical bearing resistance of 300 kPa at ULS. A resistance factor of 0.5 was applied to the ULS.

The associated total and differential settlements for these pressures are anticipated to be less than 25 mm and 19 mm, respectively for continuous foundations less than 1.0 m in width and spread (square) footings of up to 2.5 m.

5.5.3 Ancillary Conventional Shallow Foundation Recommendations

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

It is noted that there is a potential for weaker subgrade soil to be encountered between the investigation locations. Pinchin presumes that any areas of weaker subgrade soil will consist of small pockets of soft/loose natural soil which can be compacted to match the density of the remainder of the Site. As such, the material must be compacted to a minimum of 100% SPMDD prior to installing the concrete formwork. Any soft/loose areas which are not able to achieve the recommended 100% SPMDD are to be removed and replaced with a low strength concrete.

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

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

5.5.4 Foundation Transition Zones

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

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

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

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

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

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

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

5.5.5 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.

5.5.6 Rammed Aggregate Piers

The use of a Rammed Aggregate Pier system may be suitable to minimize excavations for foundation preparation and support of the floor slab, by allowing the majority of the existing fill to remain in place below the building footprint.

Rammed Aggregate Pier ® (RAP) soil reinforcing elements using the Geopier ® installation methodology is installed by drilling 0.76 m diameter cavity and ramming thin lifts of well graded aggregate within the cavity to form very stiff, high-density aggregate piers. The drilled holes typically extend from 3.0 to 7.5 m below grade and 2.1 to 6.1 m below footing bottoms. The first lift of aggregate forms a bulb below the bottoms of the piers, thereby pre-stressing and pre-straining the soils to a depth equal to at least one pier diameter below the base of the drill cavity. Subsequent lifts are typically about 300 mm in thickness. Ramming takes place with a high-energy bevelled tamper that both densifies the aggregate and forces the aggregate laterally into the sidewalls of the drill cavity. This action increases the lateral stress in surrounding soil; thereby further stiffening the stabilized composite soil mass.

The result of the Geopier RAP installation is a significant strengthening and stiffening of subsurface soils that then support high bearing capacity footings. Geopier RAP in conjunction with a thickened gravel layer can provide support for floor slabs.

Rammed aggregate piers are a proprietary design and will require input from specialized contractors and engineers. The installation of the rammed aggregate piers should be monitored on a full-time basis by a qualified geotechnical consultant.

5.5.7 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 a maximum depth of approximately 11.3 mbgs and were terminated in the overburden soils. Bedrock is expected at depths greater than 100 m below the Site. SPT “N” values within the native soil deposits 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 D. A Site Class D has an average shear wave velocity (Vs) of between 180 and 360 m/s. Shear wave velocity soundings could be completed at the Site once final design and depths of foundations are known as a higher Site Classification may be available for the Site.

5.5.8 Building Drainage

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

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

5.5.9 Shallow Foundations Frost Protection & Foundation Backfill

In the Barrie, Ontario area, exterior perimeter foundations for heated buildings require a minimum of 1.4 m of soil cover above the underside of the footing to provide soil cover for frost protection.

Where the foundations for heated buildings do not have the minimum 1.4 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 on-site sand material is generally silty; however, portions with less silt may be suitable for reuse as foundation wall backfill and this should be confirmed at the time of construction. The clay till should be excluded from reuse as foundation wall backfill, due to the potential difficulty in compaction. The backfill material used against the foundation must be brought up evenly on both sides of walls not designed to resist lateral earth pressure. All granular material is to be placed in maximum 300 mm thick lifts compacted to a minimum of 100% SPMDD below the building and exterior hard landscaping areas and 95% SPMDD below exterior soft landscaping areas. It is recommended that inspection and testing be carried out during construction to confirm backfill quality, thickness and to ensure compaction requirements are achieved.

5.6 Floor Slabs

Prior to the installation of the engineered fill material, all pre-existing fill, topsoil, and native soils with organics should be removed to the underlying organic free in-situ soil. The natural subgrade soil is to be proof roll compacted with a minimum 10 tonne non-vibratory steel drum roller to observe for weak/soft spots.

The in-situ inorganic native soils or engineered fill placed on-site during area grading is considered adequate for the support of the concrete floor slabs provided it is proof roll compacted as outlined above. Any soft area(s) encountered during proof rolling should be excavated and replaced with a similar soil type.

Once the subgrade soil is exposed it is to be inspected and approved by a qualified geotechnical engineering consultant to ensure that the material conforms to the soil type and consistency observed during the subsurface investigation work.

Based on the in-situ soil conditions, it is recommended to establish the concrete floor slab on a minimum 300 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 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
Engineered Fill using recompacted on-site soils	20,000
In-situ sand soils	15,000
In-situ clay till soils	30,000

If a 300 mm thick layer of Granular ‘A’ base were utilized on top of native sand or engineered fill, the modulus of subgrade reaction at the top of the Granular ‘A’ would be 25,000 kN/m^3.

The values noted above are for loaded areas of 0.3 m by 0.3 m.

If RAP elements are used to allow the existing fill to remain below the proposed addition, then the modulus of subgrade reaction for the slab design should be determined as part of the design of the RAP system.

5.7 Loading Dock Design

It is understood that twelve loading docks are considered along the west side of the proposed building expansion.

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 δ**) expressed as **R = N tan δ**. The frictional resistance by soil is summarized in the following table:

Subgrade Soil	Friction Factor, tan δ
Compact to dense silt/sand	0.30
Glacial Till soils	0.35
Engineered Fill using imported OPSS 1010 Granular B Type I or SSM	0.35

A 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 parameters (un-factored) should be used for the design for the loading bays. 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 (deg)	At rest Earth Pressure Coefficient	Active Earth Pressure Coefficient	Passive Earth Pressure Coefficient
Granular Fill OPSS 1010 Granular 'B' or SSM	21	32	0.47	0.31	3.25
Sand, compact to dense	19	30	0.50	0.33	3.00
Glacial Till, very stiff to hard	21	30	0.50	0.33	3.00

The loading dock walls should be backfilled with free draining materials.

5.8 Asphaltic Concrete Pavement Structure Design for Parking Lot and Driveways

5.8.1 Discussion

The pavement structure along the west side of the building will be expanded to provide access to the proposed loading bays. For the purpose of this report, it has been assumed that the subgrade has been prepared according to Section 5.2 and will comprise soils with similar characteristic to the onsite sand and/or glacial till or OPSS 1010 Granular B or SSM.

Pre-existing fill may be exposed within the pavement subgrade. Fill materials may remain below proposed pavement areas if the owner accepts the increased potential for required pavement maintenance and repairs as well as potential for decreased pavement life; otherwise, the fill should be removed from below proposed pavement areas. Where required, grades should be raised with granular fill compacted to 98% SPMDD.

5.8.2 Pavement Structure

No traffic design details were available at the time of this report. It is recommended that Pinchin review the provided pavement design recommendations once additional traffic design detail becomes available. The following table presents the minimum specifications for a flexible asphaltic concrete pavement structure:

Pavement Layer	Compaction Requirements	Light Duty Areas	Heavy Duty Areas
Surface Course Asphaltic Concrete HL-3 (OPSS 1150)	92% MRD as per OPSS 310	40 mm	40 mm
Binder Course Asphaltic Concrete HL-8 (OPSS 1150)	92 % MRD as per OPSS 310	60 mm	100 mm
Base Course: Granular “A” (OPSS 1010)	100% Standard Proctor Maximum Dry Density (ASTM-D698)	150 mm	150 mm
Subbase Course: Granular “B” Type I (OPSS 1010)	100% Standard Proctor Maximum Dry Density (ASTM D698)	350 mm	450 mm

Notes:

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

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

Performance grade PG 58-28 asphaltic concrete may be specified for Marshall mixes in the light duty pavement areas. A performance grade of PG 64-28 should be specified in heavy duty pavement areas.

5.8.3 Pavement Structure Subgrade Preparation and Granular up Fill

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

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

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

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

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

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

5.8.4 Drainage

Control of surface water is a critical factor in achieving good pavement structure life. The pavement thickness designs are based on a drained pavement subgrade via sub-drains or ditches.

The native on-site soils are silty with poor natural drainage and therefore it is recommended that continuous pavement subdrains are installed along the outside edges of the driveway and in lower areas beneath parking areas. Subdrains should comprise 150 mm diameter perforated pipe in filter sock, bedded in concrete sand. The upper limit of the concrete sand bedding should be at the lower limit of the subbase, with the subgrade below the subbase sloped towards the subdrain.

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

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

6.0 SITE SUPERVISION & QUALITY CONTROL

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

sampling and testing of aggregates and concrete, to ensure that physical characteristics of materials for compliance during installation and satisfies all specifications presented within this report.

7.0 TERMS AND LIMITATIONS

This Geotechnical Investigation was performed for the exclusive use of Summit Industrial Income Operating LP (Client) in order to evaluate the subsurface conditions at 500 Veterans Drive, Barrie, 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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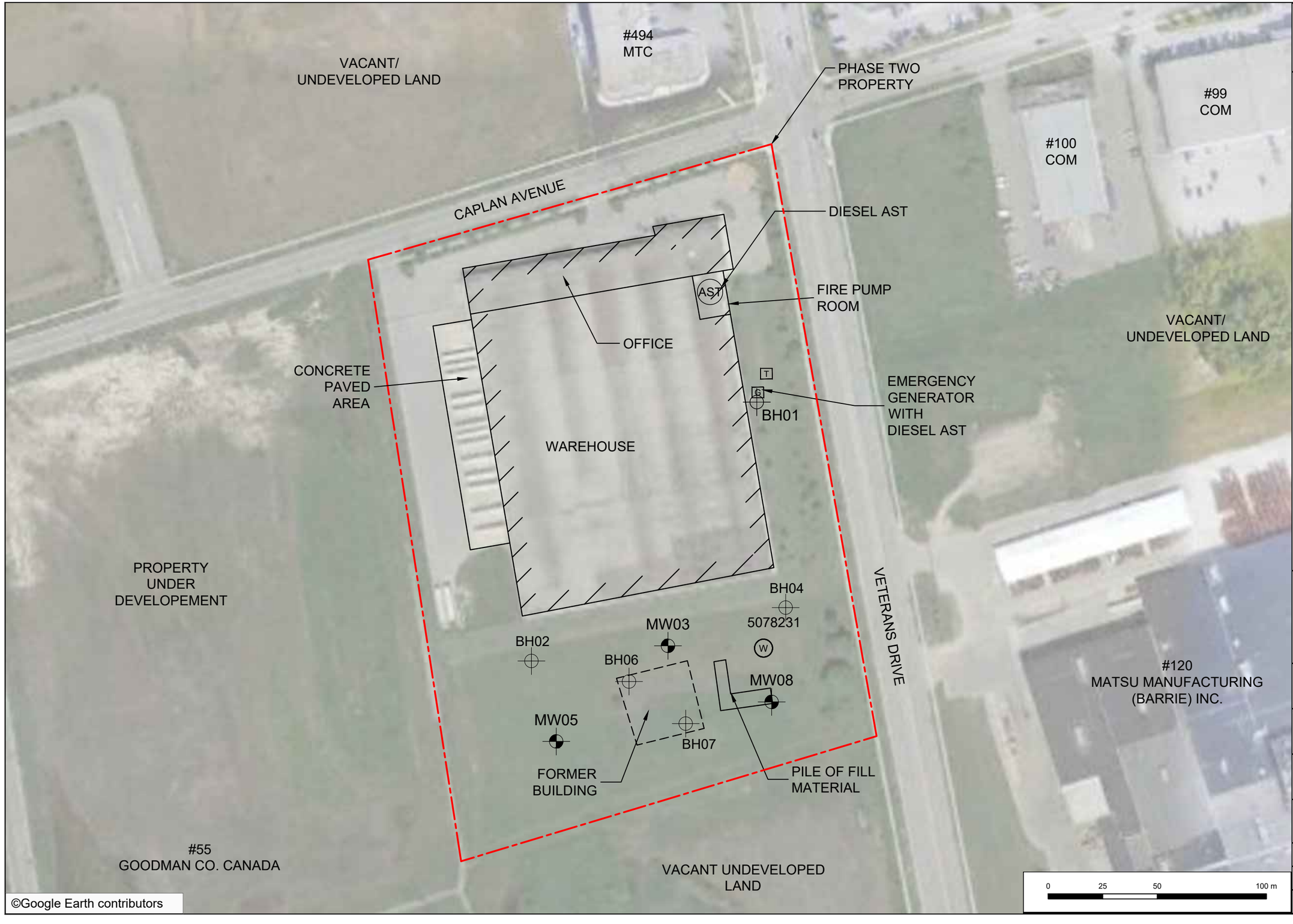
Template: Master Geotechnical Investigation Report – Ontario, GEO, September 2, 2021

FIGURES

PINCHIN

PROJECT NAME:	GEOTECHNICAL INVESTIGATION FOR PROPOSED BUILDING EXPANSION				
CLIENT NAME:	SUMMIT INDUSTRIAL INCOME OPERATING LP				
PROJECT LOCATION:	500 VETERANS DRIVE, BARRIE, ONTARIO				
FIGURE NAME:	KEY MAP				FIGURE NUMBER: 1
PROJECT NUMBER: 309899.002	SCALE: 1:15,000	DRAWN BY: SR	REVIEWED BY: KT	DATE: JULY 2022	



N

LEGEND

- SUBJECT SITE
- SITE BUILDING
- (AST) ABOVEGROUND STORAGE TANK
- [T] PAD MOUNTED TRANSFORMER
- [G] GENERATOR
- (W) WATER SUPPLY WELL
- COM COMMERCIAL
- MTC MULTI-TENANT COMMERCIAL
- MONITORING WELL
- BOREHOLE LOCATION
- mbgs METERS BELOW GROUND SURFACE

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

PINCHIN

PROJECT NAME:
GEOTECHNICAL INVESTIGATION FOR PROPOSED BUILDING EXPANSION

CLIENT NAME:
SUMMIT INDUSTRIAL INCOME OPERATING LP

PROJECT LOCATION:
500 VETERANS DRIVE, BARRIE, ONTARIO

FIGURE NAME:
BOREHOLE LOCATION PLAN

PROJECT NUMBER:	SCALE:
309899.002	AS SHOWN
DRAWN BY:	REVIEWED BY:
SR	KT
DATE:	FIGURE NUMBER:
JULY 2022	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: BH01

Project #: 309899 **Logged By:** JP

Project: Proposed Building Expansion

Client: Summit Industrial Income Operating LP

Location: 500 Veterans Drive, Barrie, ON

Drill Date: July 7, 2022 **Project Manager:** VO/JD

SUBSURFACE PROFILE					SAMPLE									
Depth (m)	Symbol	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-Value	Standard Penetration N-Value (□ 20 40 60 □)	Shear Strength (△ kPa △ 100 200)	Water Content (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
0		Ground Surface	0.00	▲ No Monitoring Well Installed										
		Possible Fill Brown silty sand, trace gravel, trace organics, moist to very moist	0.00								14.6			
			-0.76											
1		***Sand*** Brown silty sand, trace gravel, moist to very moist	0.76								12.8			
			-1.52											
		End of Borehole	1.52	▼										
2		Borehole terminated at 1.52 mbgs.												
3														
4														
5														
6														
7														
8														
9														

Contractor: Strata Drilling Group **Grade Elevation:** N/A

Drilling Method: Hollow Stem Augers/Direct Push **Top of Casing Elevation:** N/A

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

Log of Borehole: BH02

Project #: 309899 **Logged By:** JP

Project: Proposed Building Expansion

Client: Summit Industrial Income Operating LP

Location: 500 Veterans Drive, Barrie, ON

Drill Date: July 6, 2022 **Project Manager:** VO/JD

Depth (m)	Description	Elevation (m)	Sample Type	Sampler #	Recovery (%)	SPT N-Value	Water Content (%)
0	Ground Surface	311.07					
	Fill 100 mm topsoil Brown sand, some silt, trace to some gravel, compact, damp to moist	0.00	SS	1	19"	27	6.3
1			SS	2	17"	11	11.5
	Trace organics, loose	309.55 1.52	SS	3	9"	7	5.3
2	Brown sand, trace gravel, loose, very moist	308.78 2.29	SS	4	17"	8	10.7
	Sand Brown silty sand, trace gravel, loose, moist	307.93 3.14	SS	5	29"	19	15.5
3	**Clay Till** Brown silty clay, trace sand, DTPL to APL, very stiff						
4							
5			SS	6	30"	20	20.5
	End of Borehole	305.89 5.18					
6	Borehole terminated at 5.18 mbgs.						
7							

Monitoring Well Details: No Monitoring Well Installed

Standard Penetration N-Value: 20, 40, 60

Shear Strength: kPa 100 200

Sample ID

Soil Vapour Concentration (ppm)

Laboratory Analysis

Contractor: Strata Drilling Group **Grade Elevation:** 311.07 masl

Drilling Method: Hollow Stem Augers/Split Spoon Sampler **Top of Casing Elevation:** N/A

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

Log of Borehole: MW03

Project #: 309899 **Logged By:** JP

Project: Proposed Building Expansion

Client: Summit Industrial Income Operating LP

Location: 500 Veterans Drive, Barrie, ON

Drill Date: July 11, 2022 **Project Manager:** VO/JD

Depth (m)	Symbol	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-Value	Standard Penetration N-Value (20, 40, 60)	Shear Strength kPa (100, 200)	Water Content (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
		SUBSURFACE PROFILE			**SAMPLE**									
0		Ground Surface	310.99											
		Topsoil 75 mm topsoil	0.00		SS	1	16"	25			4.9			
1		**Sand** Brown sand, trace silt, compact to dense, damp to moist; occasional silt pockets and cobbles			SS	2	16"	30			3.1			
2					SS	3	18"	22			6.0			
			308.70											
		Some silt, trace gravel	2.29		SS	4	16"	43			5.6			
3					SS	5	19"	44			5.8			
4														
			306.42											
		Very dense, moist	4.57		SS	6	16"	62			6.7			
5														
			304.89											
6		Brown silty sand, trace to some gravel, very dense, moist	6.10		SS	7	24"	74			7.2			
7														
			303.37											
		Clay Till Brown silty clay, trace sand and gravel, very stiff to hard, DTPL to APL	7.62		SS	8	20"	26			23.4			
8														
9			301.85											
		Grey	9.14		SS	9	24"	31			28.4			
			301.24											
10		End of Borehole	9.75	Water level = 5.64 mbgs, as measured on July 14, 2022										
11		Borehole terminated at 9.75 mbgs.												
12														

Bentonite

Riser

Silica Sand

Screen

Contractor: Strata Drilling Group **Grade Elevation:** 310.99 masl

Drilling Method: Hollow Stem Augers/Split Spoon Sampler **Top of Casing Elevation:** ____masl

Well Casing Size: 50 mm **Sheet:** 1 of 1

Log of Borehole: BH04

Project #: 309899 **Logged By:** JP

Project: Proposed Building Expansion

Client: Summit Industrial Income Operating LP

Location: 500 Veterans Drive, Barrie, ON

Drill Date: July 6, 2022 **Project Manager:** VO/JD

Depth (m)	Description	Elevation (m)	Sample Type	Sampler #	Recovery (%)	SPT N-Value	Water Content (%)
0	Ground Surface	311.32					
	Topsoil 125 mm topsoil	0.00	SS	1	15"	11	7.4
	Sand Reddish brown sand, trace some silt and gravel, compact, damp to moist	310.56 0.76	SS	2	13"	15	4.9
1	Brown						
			SS	3	17"	23	3.4
2		309.03					
	Light brown sand, trace silt, dense, damp	2.29	SS	4	15"	36	3.2
3							
			SS	5	18"	38	4.3
4		306.75					
	Brown sand, trace to some silt, trace gravel, dense, moist, occasional silt layers	4.57	SS	6	18"	49	6.9
5		306.14					
	End of Borehole	5.18					
6	Borehole terminated at 5.18 mbgs.						
7							
8							
9							

No Monitoring Well Installed

Standard Penetration N-Value: 20, 40, 60

Shear Strength: kPa 100 200

Sample ID

Soil Vapour Concentration (ppm)

Laboratory Analysis

Contractor: Strata Drilling Group **Grade Elevation:** 311.32 masl

Drilling Method: Hollow Stem Augers/Split Spoon Sampler **Top of Casing Elevation:** N/A

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

Log of Borehole: MW05

Project #: 309899 **Logged By:** JP

Project: Proposed Building Expansion

Client: Summit Industrial Income Operating LP

Location: 500 Veterans Drive, Barrie, ON

Drill Date: July 11, 2022 **Project Manager:** VO/JD

Depth (m)	Description	Elevation (m)	Sample Type	Sampler #	Recovery (%)	SPT N-Value	Water Content (%)
0	Ground Surface	310.43					
	Fill 125 mm topsoil, some sand, rootlets	0.00 / 309.67	SS	1	18"	12	5.5
1	Reddish brown sand, trace to some silt, trace gravel, compact, damp to moist	0.76	SS	2	19"	22	6.4
2	Brown, occasional silt layers	308.14 / 2.29	SS	3	19"	24	7.3
	Sand Brown silty sand, trace to some gravel, dense, moist	307.38 / 3.05	SS	4	21"	34	6.9
3	**Clay Till** Brown silty sandy clay, trace gravel, hard, DTPL, occassional cobbles		SS	5	29"	52	6.7
4		305.86 / 4.57					
5	Grey/brown		SS	6	25"	33	16.0
6		304.33 / 6.10					
	grey silty clay, trace sand and gravel, hard, APL		SS	7	24"	33	21.2
7		302.81 / 7.62					
8	Very stiff, DTPL to APL		SS	8	24"	25	19.1
9		301.29 / 9.14					
	Sand Brown sand, trace silt, compact, wet		SS	9	18"	25	19.7
10							
11		299.15 / 11.28	SS	10	18"	25	18.3
	End of Borehole						
12	Borehole terminated at 11.28 mbgs.						
13							

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

Water Level = 7.10 mbgs, as measured on July 13, 2022

Standard Penetration N-Value: 20, 40, 60

Shear Strength △ kPa △: 100, 200

Sample ID

Soil Vapour Concentration (ppm)

Laboratory Analysis

Contractor: Strata drilling Group **Grade Elevation:** 310.43 masl

Drilling Method: Hollow Stem Augers/Split Spoon Sampler **Top of Casing Elevation:** ___masl

Well Casing Size: 50 mm **Sheet:** 1 of 1

Log of Borehole: BH06

Project #: 309899 **Logged By:** JP

Project: Pproposed Building Addition

Client: Summit Industrial Income Operating LP

Location: 500 Veterans Drive, Barrie, ON

Drill Date: July 7, 2022 **Project Manager:** VO/JD

Depth (m)	Description	Elevation (m)	Monitoring Well Details	Water Content (%)
0	Ground Surface	310.81	No Monitoring Well Installed	
	Fill Brown sand and gravel, damp	0.00		4.3
		310.05		
1	Brown sand, some gravel, damp; occasional silt inclusions	0.76		2.4
2		308.52		3.0
	Clay Till Brown silty clay, trace sand, APL	2.29		16.0
3		307.76		
	End of Borehole	3.05		
4	Borehole terminated at 3.05 mbgs.			
5				
6				
7				
8				
9				

SUBSURFACE PROFILE

SAMPLE

Symbol

Sample Type

Sampler #

Recovery (%)

SPT N-Value

Standard Penetration N-Value (20, 40, 60)

Shear Strength (kPa) (100, 200)

Sample ID

Soil Vapour Concentration (ppm)

Laboratory Analysis

Contractor: Strata Drilling Group **Grade Elevation:** 310.81 masl

Drilling Method: Hollow Stem Augers/Direct Push **Top of Casing Elevation:** N/A

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

Log of Borehole: BH07

Project #: 309899 **Logged By:** JP

Project: Proposed Building Expansion

Client: Summit Industrial Income Operating LP

Location: 500 Veterans Drive, Barrie, ON

Drill Date: July 6, 2022 **Project Manager:** VO/JD

Depth (m)	Description	Elevation (m)	Sample Type	Sampler #	Recovery (%)	SPT N-Value	Water Content (%)
0	Ground Surface	310.23					
	Fill	0.00					
	150 mm topsoil		SS	1	17"	13	5.7
	Brown sand, some gravel, compact, moist	309.47					
1		0.76					
	Brown sand, trace to some silt, trace gravel, loose, very moist, occasional silt inclusions		SS	2	9.5"	4	12.1
		308.70					
	Compact	1.52					
2			SS	3	6"	12	14.3
		307.94					
	Trace to some gravel, dense, moist	2.29					
			SS	4	16"	32	4.1
3		307.18					
	Black sandy silt, some gravel, some organics	3.05					
			SS	5	24"	50	7.4
	Sand						
	Brown sand, some silt, trace gravel, dense, moist						
4		305.66					
	Very dense, occasional silt inclusions	4.57					
			SS	6	22"	78	7.0
5		305.11					
	End of Borehole	5.12					
	Borehole terminated at 5.12 mbgs.						
6							
7							

Monitoring Well Details: No Monitoring Well Installed

Standard Penetration N-Value: 20, 40, 60

Shear Strength: kPa 100 200

Sample ID

Soil Vapour Concentration (ppm)

Laboratory Analysis

Contractor: Strata Drilling Group **Grade Elevation:** 310.23 masl

Drilling Method: Hollow Stem Augers/Split Spoon Sampler **Top of Casing Elevation:** N/A

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

Log of Borehole: MW08

Project #: 309899 **Logged By:** JP

Project: Proposed Building Expansion

Client: Summit Industrial Income Operating LP

Location: 500 Veterans Drive, Barrie, ON

Drill Date: July 12, 2022 **Project Manager:** VO/JD

Depth (m)	Description	Elevation (m)	Sample Type	Sampler #	Recovery (%)	SPT N-Value	Water Content (%)
0	Ground Surface	311.23					
	Fill 100 mm topsoil and sand, trace gravel, some rootlets	0.00 / 310.47	SS	1	19"	17	7.9
1	Reddish brown sand, some silt, compact, moist	0.76	SS	2	16"	5	18.2
2	Brown silty sand, trace gravel, loose to very loose, wet, rootlets	308.95	SS	3	16"	2	19.4
	Dark brown silty sand, some organics, loose, wet, large roots	2.29 / 308.18	SS	4	13"	4	20.0
3	Brown sand, trace gravel, organic odour	3.05	SS	5	21"	29	7.6
4	**Sand** Brown silty sand, trace clay and gravel, compact to very dense, moist						
5			SS	6	17"	72	7.3
6		305.14					
	Brown sand, some silt, trace gravel,very dense, moist, occasional silt inclusions	6.10	SS	7	14"	93	6.8
7		303.61					
	Silt Till Brown silt, some sand, trace clay and gravel, very dense, moist	7.62	SS	8	18"	63	9.2
8							
9		302.09					
	Clay Till Grey silty clay, trace sand and gravel, very stiff, APL	9.14 / 301.48	SS	9	24"	29	26.9
10	End of Borehole	9.75					
11	Borehole terminated at 9.75 mbgs.						
12							

Monitoring Well Details: Bentonite; Riser; Silica Sand; Screen; Water level = 6.00 mbgs, as measured on July 14, 2022

Standard Penetration N-Value: 20, 40, 60

Shear Strength: kPa 100 200

Sample ID | Soil Vapour Concentration (ppm) | Laboratory Analysis

Contractor: Strata Drilling Group **Grade Elevation:** 311.23 masl

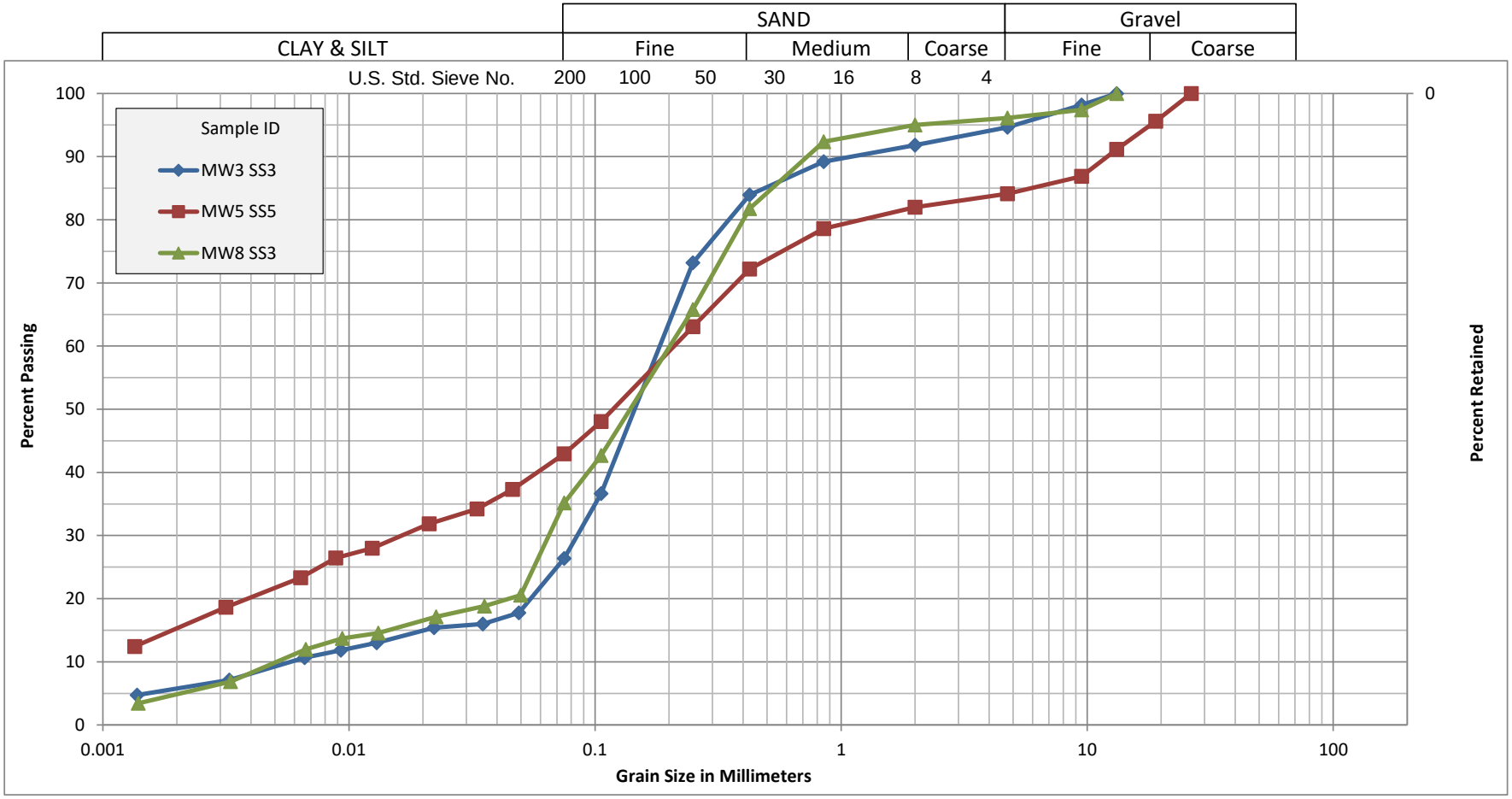
Drilling Method: Hollow Stem Augers/Split Spoon Sampler **Top of Casing Elevation:** ___masl

Well Casing Size: 50 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
MW3 SS3	5.0-7.0	5.0	68.6	20.4	6.0
MW5 SS5	10.0-12.0	16.0	41.1	27.9	15.0
MW8 SS3	5.0-7.0	4.0	60.8	30.2	5.0

PINCHIN

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

PARTICLE SIZE DISTRIBUTION ANALYSIS

500 Veterans Parkway - Barrie, ON
Summit Industrial Income REIT

Figure No. 1

309899.002

Reviewed By:

More information available upon request

Atterberg Limits

LS 703&704 / AASHTO T89

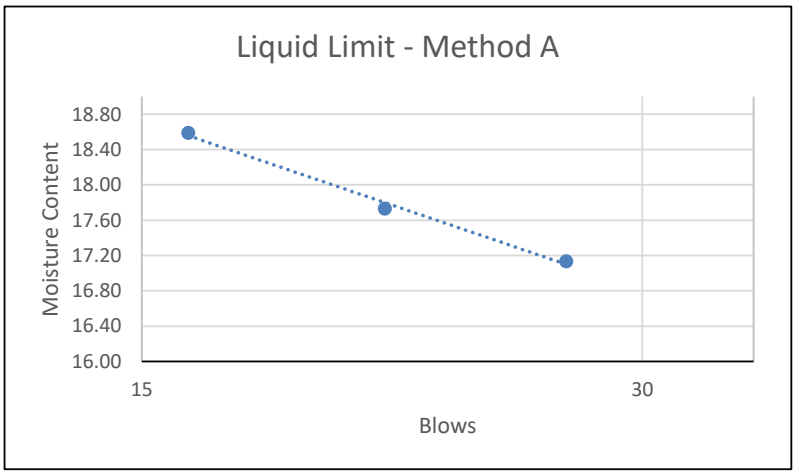
Project Name: 500 Veterans Parkway
Project No. 309899.002
Client: Summit Industrial Income REIT
Location: Barrie, ON
Material: Soil
Sample: MW5 SS5 10.0-12.0

Test Date: July 19, 2022
Tested By: B Frank
Sample Date: July 11, 2022
Sampled By: NA
Reviewed By: V Marshall

Liquid Limit - Method A						
Pot Number	1	2	3			
Number of blows	27	21	16			
Wet mass + pot	35.82	36.75	35.52			
Dry mass + pot	32.89	33.56	32.44			
Tare	15.79	15.57	15.87			
Water content %	17.13	17.73	18.59			

Plastic Limit			
Pot Number	1	2	
Wet mass + pot	26.76	24.62	
Dry mass + pot	25.74	23.80	
Tare	15.71	15.64	
Water content %	10.2	10.0	

PI = LL - PL	
Liquid Limit %	**17**
Plastic Limit %	**10**
Plastic Index	**7**
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.