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Geotechnical Investigation – Proposed Residential Development

3900 Innes Road, Ottawa, Ontario

Prepared for:

Extendicare (Canada) Inc.
3000 Steeles Avenue East, Suite 102
Markham, ON L3R 4T9

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Megan Keon

Author:

Megan Keon, EIT.
Project Technologist, Geotechnical Services
613.608.5350
mkeon@pinchin.com

LICENSED PROFESSIONAL ENGINEER
J.W. DIETZ
2023-01-31
PROVINCE OF ONTARIO

Reviewer:

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

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1.0 INTRODUCTION AND SCOPE

Pinchin Ltd. (Pinchin) was retained by Extendicare (Canada) Inc. (Client) to conduct a Geotechnical Investigation and provide subsequent geotechnical design recommendations for the proposed Residential Development to be located at 3900 Innes Road, Ottawa, 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 four storey, 256 bed long-term care home complete with a partial basement level. The proposed development will also include new Site Services, and asphalt surfaced parking areas and access laneways.

Pinchin completed additional supplementary borehole investigation following issuance of the final version of this report, which was issued June 29, 2022. Pinchin's geotechnical comments and recommendations are based on the results of the Geotechnical Investigation, the Supplementary Field Investigation, and our understanding of the project scope.

The purpose of the Geotechnical Investigation was to delineate the subsurface conditions and soil engineering characteristics by advancing a total of fourteen (14) sampled boreholes (Boreholes BH1 to BH14) at the Site. In addition, the supplemental field investigation was completed which consisted of advancing an additional two (2) sampled boreholes (Boreholes BH101 and BH102) at the Site.

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;
- Open cut excavations;
- Anticipated groundwater management;
- Site service trench design;
- Lateral earth pressure coefficients and unit densities;
- Foundation design recommendations including soil bearing resistances at Ultimate Limit States (ULS) and Serviceability Limit States (SLS) design;
- Horizontal and uplift capacity if designing using concrete caissons;
- Potential total and differential settlements;
- Foundation frost protection and engineered fill specifications and installation;
- Seismic Site classification for seismic Site response;

- Basement design;
- Concrete floor slab-on-grade support recommendations;
- Asphaltic concrete pavement structure design for parking areas and access roadways; and
- Potential construction concerns.

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

2.0 SITE DESCRIPTION AND GEOLOGICAL SETTING

The Site is located on the east side of Noella Leclair Way, approximately 400 m south of Innes Road in Ottawa, Ontario. The Site is currently undeveloped and consists of a mixture of wild overgrowth and agricultural land. The lands adjacent to the Site are either developed with single family residential dwellings, retail buildings or consist of undeveloped agricultural land.

Data obtained from the Ontario Geological Survey Maps, as published by the Ontario Ministry of Natural Resources, indicates that the majority of the Site is located on a fine textured glaciomarine deposit consisting of massive to well laminated silt and clay with minor sand and gravel deposits. The northeastern portion of the Site is located on Paleozoic bedrock (Ontario Geological Survey 2010. Surficial geology of Southern Ontario; Ontario Geological Survey, Miscellaneous Release--Data 128-REV). The underlying bedrock at this Site is of the Shadow Lake Formation consisting of limestone, dolostone, shale, arkose, and sandstone (Ontario Geological Survey 2011. 1:250 000 scale bedrock geology of Ontario; Ontario Geological Survey, Miscellaneous Release---Data 126-Revision 1).

3.0 GEOTECHNICAL FIELD INVESTIGATION AND METHODOLOGY

Pinchin completed a field investigation at the Site between April 20 and April 22, 2022, by advancing a total of fourteen (14) sampled boreholes (Boreholes BH1 to BH14). Boreholes BH1 to BH9 were advanced within the proposed building footprint, while Boreholes BH10 to BH14 were advanced within the proposed parking and courtyard areas. The boreholes were advanced to sampled depths ranging from approximately 0.7 to 7.6 metres below existing ground surface (mbgs).

Pinchin completed a supplemental field investigation at the Site on January 4, 2023, by advancing an additional two sampled boreholes (Boreholes BH101 and BH102) within the proposed building footprint to further assess the subgrade soils and the bedrock with depth. Boreholes BH101 and BH102 were terminated in bedrock at depths ranging from approximately 9.1 to 12.2 mbgs. The approximate spatial locations of the boreholes advanced at the Site are shown on Figure 2.

The boreholes were advanced with the use of a CME55 drill rig which was equipped with standard soil sampling equipment. Soil samples were collected at 0.76 and 1.52 m intervals using a 51 mm outside diameter (OD) split spoon barrel in conjunction with Standard Penetration Tests (SPT) "N" values (ASTM D1586). The SPT "N" values were used to assess the compactness condition of the non-cohesive soil, and to estimate the consistency of the cohesive soil. Approximate shear strengths of the cohesive deposits were measured via shear vane testing completed in the field, as well as with a handheld pocket penetrometer, and the results are presented on the appended borehole logs.

During the supplemental investigation, bedrock was proven in Boreholes BH101 and BH102 by core drilling with an NQ-size double tube diamond bit core barrel. The bedrock core specimens were measured in the field to determine the Rock Quality Designation (RQD) (ASTM 6032). The core samples were returned to our offices for further visual examination and testing.

Monitoring wells were installed in Boreholes BH1 and BH6 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 May 20, 2022. The groundwater observations and measurements recorded are included on the appended borehole logs.

The borehole locations and ground surface elevations were located at the Site by Pinchin personnel. The ground surface elevation at each borehole location was referenced to the following temporary benchmark as shown on Figure 2:

- TBM: Top of the nut on the fire hydrant located between Noella Leclair Way and the Site, at the approximate location shown on Figure 2; and
- Elevation: 100.00 m (Local Datum)

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

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

Select soil samples collected from the boreholes were submitted to a material testing laboratory to determine the grain size distribution and Atterberg Limits of the soil. One rock core sample was also submitted to determine unconfined compressive strength. A copy of the laboratory analytical reports is included in Appendix III. In addition, the collected samples were compared against previous geotechnical information from the area, for consistency and calibration of results.

4.0 SUBSURFACE CONDITIONS

4.1 Borehole Soil Stratigraphy

In general, the soil stratigraphy at the Site comprises surficial organics overlying silt and clay, silty clay, glacial till, and bedrock to the maximum borehole termination depth of approximately 12.2 mbgs. The appended borehole logs provide detailed soil descriptions and stratigraphies, results of SPT, shear vane testing and pocket penetrometer testing, details of monitoring well installations, and groundwater measurements.

The surficial organics were encountered in all boreholes and were measured to range in thickness from approximately 150 to 700 millimetres (mm).

Silt and clay/silty clay was found to be underlying the surficial organics in all boreholes with the exception of Borehole BH10. The silt and clay/silty clay deposit was observed to extend to depths ranging from approximately 2.0 to 7.3 mbgs. The material was noted to typically contain trace sand and was brown to grey in colour. The material had a very soft to very stiff consistency based on shear strengths measured with a shear vane and handheld pocket penetrometer of between 0 and 187.5 kPa and on SPT ‘N’ values of 0 to 17 blows per 300 mm penetration of a split spoon sampler. It is noted that the shear strength of the soil typically decreases with depth and is generally a function of the increasing water content. The remoulded shear strength of the soil ranged from 3 to 21 kPa, resulting in a sensitivity of 2 to 5. The results of four particle size distribution analyses performed samples of the material indicate that the samples contain 1% sand, 31 to 38% silt and 62 to 69% clay. Two additional particle size distribution analyses were performed on samples of the silty clay taken from the supplemental field investigation. The results revealed similar conditions with the samples containing 1 to 3% sand, 32 to 37% silt and 61 to 66% clay. Atterberg Limit testing indicates the material in both field investigations has a liquid limit of between 69 and 84%, a plastic limit of between 31 and 40%, and a plasticity index of between 38 and 44%. The moisture content of the samples tested ranged between 48 to 87%, indicating the material tested was wetter than plastic limit (WTPL) at the time of sampling.

Glacial till was encountered underlying the silty clay in Boreholes BH1, BH2, BH5, BH9, BH13 and BH101 at depths ranging from approximately 2.4 to 6.1 mbgs and extended to the top of bedrock at depths of up to 7.9 mbgs. The glacial till remained consistent in soil matrix and comprised sandy gravel containing trace silt and trace clay that was brown in colour. The material had a loose to compact relative density based on SPT ‘N’ values of 6 to 27 blows per 300 mm penetration of a split spoon sampler. The result of one particle size distribution analysis performed on a sample of the glacial till material indicates that the sample contains 63% gravel, 21% sand, 8% silt and 8% clay. The moisture content of the sample tested was 13.3% indicating the material was in a moist condition at the time of sampling.

4.2 Bedrock

Auger refusal on probable bedrock was encountered in Boreholes BH1 to BH10, BH12 and BH13 between approximately 0.7 and 7.6 mbgs. The bedrock surface, as indicated by the recorded auger refusals, was variable, often changing in elevation by several metres between adjacent boreholes. It is noted that bedrock outcroppings were noted near the south limit of site where the auger refusal happened at the shallowest depth (BH10).

During the supplemental investigation, bedrock was proven in Boreholes BH101 and BH102 by core drilling with an NQ-size double tube diamond bit core barrel. Slightly weathered shale bedrock was encountered in Boreholes BH101 and BH102 and extended to depths ranging from approximately 9.1 to 12.2 mbgs.

The bedrock was dark grey with white and black spotting and occasional white to light grey banding. It was medium to coarse grained and contained some natural fractures with little to no oxidation. The bedrock at the fracture locations was mostly sharp and angular, which indicates minor water migration. Natural fractures were closely to moderately spaced and were generally found to occur in sets oriented at approximately 45 to 90º to the core axis. An approximate 90% wash return within the rock cores was observed. The wash return was milky white to grey in colour. The rock core recovery ranged between approximately 89 to 100%, with an average RQD of 97%, indicating an excellent rock quality. It is noted that the upper 1.5 m of the bedrock core recovered from Boreholes BH101 had a RQD of 52% and contained significantly more fractures all spaced at less than 100 mm apart; as such, the RQD of the upper approximate 1.5 m in BH101 was not included in the average RQD above. Laboratory testing completed on one sample of rock core indicated an unconfined compressive strength of 66 MPa. Photographs of the rock cores are provided in Appendix IV.

4.3 Groundwater Conditions

Groundwater observations and measurements were obtained in the open boreholes at the completion of drilling and are summarized on the appended borehole logs. At the completion of drilling, groundwater levels were observed to range between approximately 2.3 and 6.1 mbgs in all boreholes with the exception of Boreholes BH10, BH12 and BH13, where no groundwater was encountered. On May 20, 2022, groundwater was measured within the monitoring wells installed in Boreholes BH1 and BH6 at depths of approximately 2.9 and 3.2 mbgs, respectively.

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

5.0 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 a four storey, 256 bed long-term care home complete with a partial basement level. At this time the depth to the underside of the footings for the proposed partial basement level is unknown; as such, for the purpose of this report, Pinchin has assumed a depth of approximately 3.0 mbgs to the underside of the footing for the proposed partial basement level. The proposed development will also include new Site Services, and asphalt surfaced parking areas and access laneways.

5.2 Site Preparation

The existing surficial organic material is not considered suitable to remain below the proposed building, driveways and parking areas and will need to be removed. In calculating the approximate quantity of topsoil to be stripped, we recommend that the topsoil thicknesses provided on the individual borehole logs be increased by 50 mm to account for variations and some stripping of the mineral soil below. Additionally, due to the potential settlement of the clayey soils at the Site, allowable grade raises will be limited. It is recommended that once final grades are set, Pinchin be allowed to review any potential grade changes to determine whether the raises will result in excess settlement of the Site.

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

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

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

It is recommended that any fill required to raise grades below the proposed building addition comprise imported Ontario Provincial Standards and Specifications (OPSS) 1010 Granular ‘B’ Type I material. 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.

5.3 Open Cut Excavations

Excavations for the proposed development will extend upwards of approximately 3.0 mbgs to accommodate the proposed building foundations and new Site services.

Based on the subsurface information obtained from within the boreholes, it is anticipated that the excavated material will predominately consist of silt and clay, silty clay and glacial till materials. Bedrock is generally not expected to be encountered during excavations for the proposed building foundations or new Site services provided excavations extend less than 3 m below existing ground surface, and no buildings or services are to be installed at the north and south edges of site, where bedrock was found to be shallower (BH10, BH12, and BH14). Groundwater was measured within the monitoring wells installed in Boreholes BH1 and BH6 at depths of approximately 2.9 and 3.2 mbgs, respectively.

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

Based on the OHSA, the natural subgrade 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 extending below the groundwater table would be classified as a Type 4 soil and temporary excavations will have to be sloped back at 3 H to 1 V from the base of the excavation. Excavations through more than one type of soil must be excavated as per the requirements of the highest numbered soil type.

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

5.4 Anticipated Groundwater Management

As previously mentioned, groundwater was measured in the monitoring wells installed at depths ranging from approximately 2.9 to 3.2 mbgs and is expected to potentially be encountered during excavations for the proposed basement level portion of the building. As final elevations are unknown at this time, it is recommended that Pinchin review the following recommendations once building elevations have been set.

Minor to moderate groundwater inflow through the clayey material is expected where the excavations extend less than 0.60 m below the groundwater table. It is believed that this groundwater inflow can be controlled using a gravity dewatering system with perimeter interceptor ditches and high-capacity pumps.

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 or 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. A Permit to Take Water or a submission to the Environmental Activity and Sector Registry (EASR) is required when greater than 50,000 L/day is removed from the Site. It is the responsibility of the contractor to make this application if required.

5.5 Site Services

5.5.1 Pipe Bedding and Cover Materials for Flexible and Rigid Pipes

The subgrade soil conditions beneath the Site services will comprise either silt and clay, silty clay or glacial till materials. No support problems are anticipated for flexible or rigid pipes founded on the silt and clay, silty clay or glacial till. It is noted, however, that substantial changes in grade could cause long-term consolidation settlement of the soils, and the elevations of service pipes could be affected by that settlement. 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 (Terrafix 270R or equivalent) should be considered to maintain the integrity of the natural subgrade soils. The clear stone should contain a minimum of 50% crushed particles. Water collected within the stone should be controlled through sumps and filtered pumps.

5.5.2 Trench Backfill

The trench backfill should be compacted in maximum 300 mm thick lifts to 98% SPMDD within 4% of the optimum moisture content. Based on the observed moisture content of the natural overburden deposits, it may be difficult to achieve the specified density on all of the trench backfill. Nevertheless, it is recommended that the natural soils be used as backfill in the trenches to prevent problems with differential frost heaving of imported subgrade material.

If necessary, compensation for wet trench backfill conditions can be made with additional Granular 'B' in the pavement structure. It should be noted, however, that the wet backfill material must be compacted to at least 90% SPMDD or post-construction settlements could occur.

Portions of the silt and clay, and silty clay may have a blocky/lumpy texture. If the large interclump voids are not closed completely by thorough compaction, then long-term softening/settlement will occur. The trench backfill should be placed in thin lifts (less than 300 mm) and compacted with a sheepsfoot roller. Particular attention must be made to backfilling service connections where the trenches are narrow.

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 project 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. The natural soil at this Site is subject to moisture content increase during wet weather. As such, stockpiles should be protected to help minimize moisture absorption during wet weather.

Alternatively, an imported drier material of similar gradation as the soil (i.e., silt) may be mixed to decrease the overall moisture content and bring it to 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 in order to mitigate post compaction settlements.

5.5.3 Frost Protection

The frost penetration depth in Ottawa, Ontario is estimated to extend to approximately 1.8 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 2.1 mbgs or lower as dictated by municipal service requirements. If a minimum of 2.1 m of soil cover cannot be provided, then the pipe should be insulated with a rigid polystyrene insulation (DOW Styrofoam HI40, or equivalent) 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 manufacture’s recommendations. To accommodate the placement of horizontal insulation a wider excavation trench may be required.

5.6 Foundation Design

5.6.1 Discussion

It is typical construction practice to provide foundation frost protection with soil cover. For the Ottawa area, foundations should be provided with a minimum of 1.8 m of soil cover frost protection above the underside of the foundation for heated buildings.

The results of the field investigation indicate that the natural silty clay soil typically decreases in strength with depth and possesses a very soft to soft consistency below approximately 1.8 mbgs. As such, the natural silt and clay, and silty clay materials are not considered suitable to support the anticipated design building loads required for a four-storey building.

Probable bedrock was encountered within the boreholes advanced for the proposed building (i.e., Boreholes BH1 to BH9) during the first field investigation at depths ranging from approximately 5.2 to 7.6 mbgs. During the supplemental field investigation, bedrock was encountered at depths ranging between 6.1 and 7.9 mbgs. Bedrock was proven in Boreholes BH101 and BH102 by core drilling and cores were retrieved for examination. Based on the subsurface soil conditions encountered within the boreholes advanced at the Site, Pinchin recommends extending the building foundations down to the underlying bedrock surface, and has provides the following deep foundation options herein:

- Support the building on deep foundations consisting of helical piles (screw piles) founded within the natural silt and silty clay materials, end bearing on the probable bedrock surface located between approximately 5.2 and 7.9 mbgs; and
- Support the building on deep foundations consisting of cast-in-place concrete caissons end bearing on the probable bedrock surface located between approximately 5.2 and 7.9 mbgs.

It is noted that depending on the depth of partial basement, excavations to the top of deep foundation units may extend down to the native very soft soils. The native very soft soils may not be able to adequately support the equipment for deep foundation installation without the addition of geotextile and a gravel pad.

5.6.2 Helical Piles (Screw Piles) Founded in Natural Silt and Silty Clay Materials

Deep foundations consisting of helical piles (screw piles) founded within the natural silt and clay, and silty clay may be utilized to support the proposed building. Helical piles provide the least amount of disturbance as they are driven into the underlying soil utilizing a helix to advance through the soil matrix. The supporting grade beam system for the structure would bear upon the helical piles.

The number and size of helical piles are determined based on the building loads and configuration. Since helical piles are a proprietary system, it is recommended that the piles be designed by an experienced design build contractor in conjunction with the soil characteristics provided by Pinchin. For the natural subgrade soil encountered within the boreholes advanced, the following strength characteristics are to be used for the pile design:

Soil Type	Bulk Unit Weight (kN/m^3)	Friction Angle (°)	Cohesion (kPa)
Silt and Clay/Silty Clay	17.5	26	3

To provide frost protection, we would also recommend that the helical piles be lined with a plastic sleeve or be epoxy coated galvanized steel to protect against corrosion.

Helical pile capacity can often be determined as a function of the installation torque at termination; however, at this site most boreholes encountered soft to very soft soils overlying probable bedrock, and it is anticipated that helical piles would spin out once the tip of the pile reaches bedrock. As such, on-site load testing of helical piles end bearing on bedrock is recommended if this deep foundation system is chosen.

5.6.3 Cast-in-Place Concrete Caissons End Bearing on Probable Bedrock

An alternative to helical piles is cast-in-place concrete caissons founded on the underlying probable bedrock surface located between approximately 5.2 and 7.9 mbgs.

For cast-in-place concrete caissons founded on the probable bedrock surface located between approximately 5.2 and 7.9 mbgs, a factored geotechnical bearing resistance of 2,000 kPa at ULS may be used for foundation design purposes. It is noted that in order to achieve the recommended bearing resistance, the cast-in-place concrete caissons must be socketed into the sound bedrock a minimum of 2 times the caisson diameter.

5.6.3.1 Caisson Installation Comments and Recommendations

Based on the results of the Geotechnical Investigation, groundwater is expected to be encountered between approximately 2.9 and 3.2 mbgs and will impact the installation of the caissons below this depth. A steel liner will be required to ensure the sidewalls of the caisson excavation do not cave in. If groundwater is found to be present in the completed excavation, concrete for caisson construction should be placed from the bottom up, by tremie.

Prior to auguring, it is critical that all existing and potential surface water be controlled and diverted away from the work site to prevent infiltration.

Augured cast-in-place concrete caissons are to be installed by an experienced contractor familiar with the auger-cast process and soil conditions.

The installation of the caissons should be monitored on a full-time basis by a qualified geotechnical consultant.

Pinchin notes that the bedrock core retrieved in the upper 1.2 m of the bedrock surface in Borehole BH101 was weathered and not suitable for the bearing pressure noted in Section 5.6.3. The caissons must be socketed through the weathered portion of the bedrock, and bear on the sound bedrock below.

5.6.3.2 Horizontal Capacity of Concrete Caissons

The following outlines the lateral design capacities for concrete caissons applying Brom's Method in the latest edition of the Canadian Foundation Engineering Manual.

For augured cast in place concrete piles established at a maximum depth of approximately 7.9 mbgs, the lateral capacity will be predominantly resisted by the natural silty clay deposit. To be on the conservative side a bulk unit weight of the soil of γ = 17.5 kN/m^3 and an estimated average effective angle of internal friction of ø' = 26° were used.

The caissons are considered short, and the calculation assumes that the caissons are very rigid members in comparison to the surrounding soil and will not form a plastic hinge. As such, the failure mode will be caused by exceeding the bearing capacity of the surrounding supporting soil and not the member. In all cases, the caissons are assumed to be below the ground, restrained at the head and minimal soil resistance within the upper 1 m.

The following table outlines the estimated geotechnical reaction at SLS and the estimated factored geotechnical resistance at ULS for various diameter piles extending a maximum of approximately 7.6 mbgs:

Caisson Diameter (m)	Estimated Horizontal Reaction at SLS (kN)	Estimated Factored Horizontal Resistance at ULS (kN)
0.76	65	100
0.9	85	125
1.2	95	145
1.5	105	160
1.8	75	110

5.6.3.3 Caisson Uplift Capacity

The caisson's ultimate uplift capacity is equal to the shaft resistance that can be mobilized along the surface area of the concrete shaft.

Based on the soils encountered within the Geotechnical Investigation, and for augured cast-in-place concrete piles, the ultimate shaft resistance (Su) can be taken as the frictional shaft resistance coefficient (β) multiplied by the pile circumference (C) and the soil embedment length (h):

$$S_u = C*\beta*\sigma_v'*\Delta h$$

Where: σv´ is the vertical effective stress adjacent to the pile at depth h;

β = 0.2 for the soils encountered within the boreholes.

The following table outlines the estimated geotechnical reaction at SLS and the factored geotechnical resistance at ULS for various diameter piles extending to a maximum depth of approximately 7.6 mbgs:

Caisson Diameter (m)	Estimated Uplift Reaction at SLS (kN)	Estimated Factored Uplift Resistance at ULS (kN)
0.76	5	7
0.9	10	10
1.2	12	15
1.5	15	18
1.8	20	20

The upper 1 m of soil embedment was ignored when calculating the uplift capacity. The uplift capacity can be increased by providing an expanded base at the toe.

5.6.3.4 Load Testing

Given the soil conditions encountered, the vertical, horizontal, and uplift capacity should be verified by load testing on at least one caisson.

5.6.4 Estimated Settlement

The foundations should be founded on a uniform subgrade surface, 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.

5.6.5 Building Drainage

To assist in maintaining the building dry from surface water seepage, it is recommended that exterior grades around the buildings be sloped away at a 2% gradient or more, for a distance of at least 2.0 m.

Roof drains should discharge a minimum of 1.5 m away from the structure to a drainage swale or appropriate storm drainage system.

5.6.6 Frost Protection & Foundation Backfill

In the Ottawa, Ontario area, exterior perimeter foundations for heated buildings require a minimum of 1.8 m of soil cover above the underside of the footing to provide soil cover for frost protection. Exterior wall grade beams for the proposed structure must have sufficient frost protection at their underside.

Where the foundations for heated buildings do not have the minimum 1.8 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 backfill material must be brought up evenly on both sides of any wall 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 interior of the building and below exterior hardscaped 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.

5.7 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 approximately 7.6 mbgs where refusal was encountered on probable bedrock. SPT “N” values within the overburden soil deposit ranged between 0 and 27 blows per 300 mm. As such, based on Table 4.1.8.4.A of the OBC, this Site has been classified as Class E. A Site Class E has an average shear wave velocity (Vs) of less than 180 m/s. It is recommended that shear wave velocity soundings be completed at the Site once final design and depths of foundations are known as a higher Site Classification may be available for deeper foundations at the Site.

5.8 Basement Design

It is understood that a portion of the proposed building will include a basement level, with the underside of the footing presumed to be located approximately 3.0 mbgs. As previously mentioned, groundwater was measured in the monitoring wells installed at depths ranging from approximately 2.9 to 3.2 mbgs. As such, Pinchin recommends that foundation drains be provided for the portions of the building which will have the foundation walls exposed on the interior of the building. Pinchin also recommends that these foundations drains be extended around the entire perimeter of the building to ensure proper drainage and to mitigate the potential for water to build up where drains are not installed.

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

In addition, an underfloor drainage system should be installed beneath the basement level slab, in addition to the installation of perimeter weeping tiles at the footing level. The floor slab sub drains should be constructed in a similar fashion to the foundation drains and be connected to a suitable frost-free outlet or sump.

The walls must also be designed to resist lateral earth pressure. Depending on the design of the building the earth pressure computations must consider 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. The bulk unit weight of the retained backfill may be taken as 20 kN/m^3 for well compacted soil. An appropriate factor of safety should be applied.

5.9 Floor Slabs

The soils below any floor slabs more than 1.5 m below existing grade (i.e. basement floor slabs) will be bearing on soft to very soft clayey soils. It is recommended that these floor slabs be constructed as structural slabs, supported by the deep foundation system installed for the building foundation.

For any floor slabs constructed for portions of the building without basement, 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.

The shallow in-situ inorganic silt and clay, and silty clay materials encountered within the boreholes are considered adequate for the support of the concrete floor slabs provided they are 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 300 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
Native Stiff Silt and Clay/Silty Clay	15,000

The values in the table above are for loaded areas of 0.3 m by 0.3 m.

5.10 Asphaltic Concrete Pavement Structure Design for Parking Lot and Driveways

5.10.1 Discussion

Parking areas and driveway access will be constructed around the proposed buildings. The in-situ silt and clay/silty clay soil is considered a sufficient bearing material for an asphaltic concrete pavement structure provided all organics and deleterious materials are removed prior to installing the engineered fill material.

At this time Pinchin is unaware of the proposed final grades for the parking lot and access roadways. As such, provided the pavement structure overlies the in-situ silt/silty clay material, the following pavement structure is recommended.

5.10.2 Pavement Structure

The following table presents the minimum specifications for a flexible asphaltic concrete pavement structure:

Pavement Layer	Compaction Requirements	Parking Areas	Driveways
Surface Course Asphaltic Concrete HL-4 (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	50 mm	85 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)	300 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 Ottawa 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 to access the Site, in order to avoid the subgrade from "pumping" up into the granular material.

Performance grade PG 58-34 asphaltic concrete should be specified for Marshall mixes.

5.10.3 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 lot and access roadways 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.

5.10.4 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 silt/silty clay soils have poor natural drainage and therefore it is recommended that pavement subdrains be installed in the lower areas and be connected to the catch basins. Subdrains should comprise 150 mm diameter perforated pipe in filter sock, bedded in concrete sand. The upper limit of the subdrain bedding should be at the lower limit of the pavement subbase, with the subgrade below the subbase sloped towards the subdrain. Subdrains must drain to a suitable frost-free outlet.

The surface of the roadways 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 Extendicare (Canada) Inc. (Client) in order to evaluate the subsurface conditions at 3900 Innes Road, Ottawa, 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 V, Report Limitations and Guidelines for Use, which pertains to this report.

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

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

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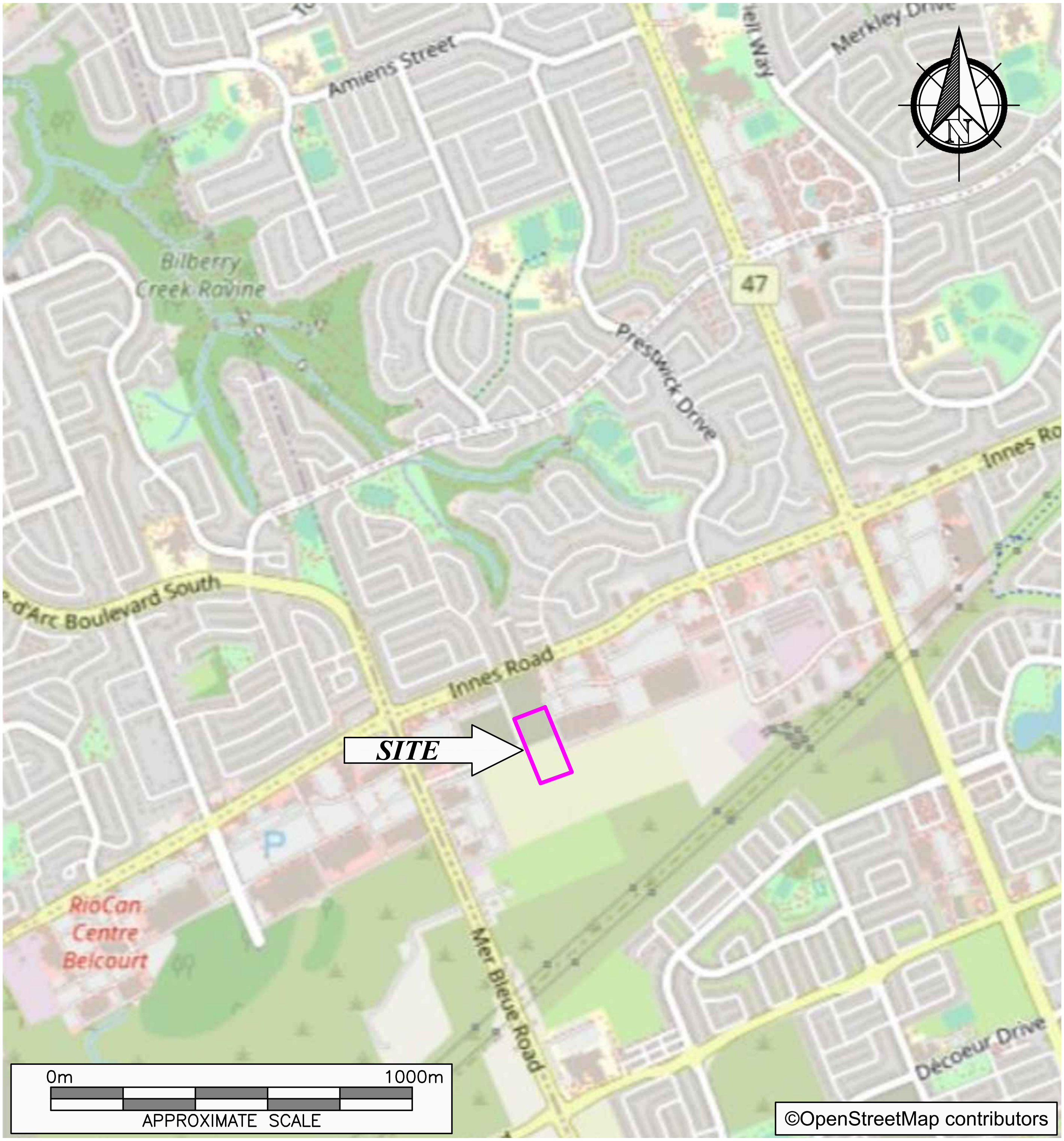
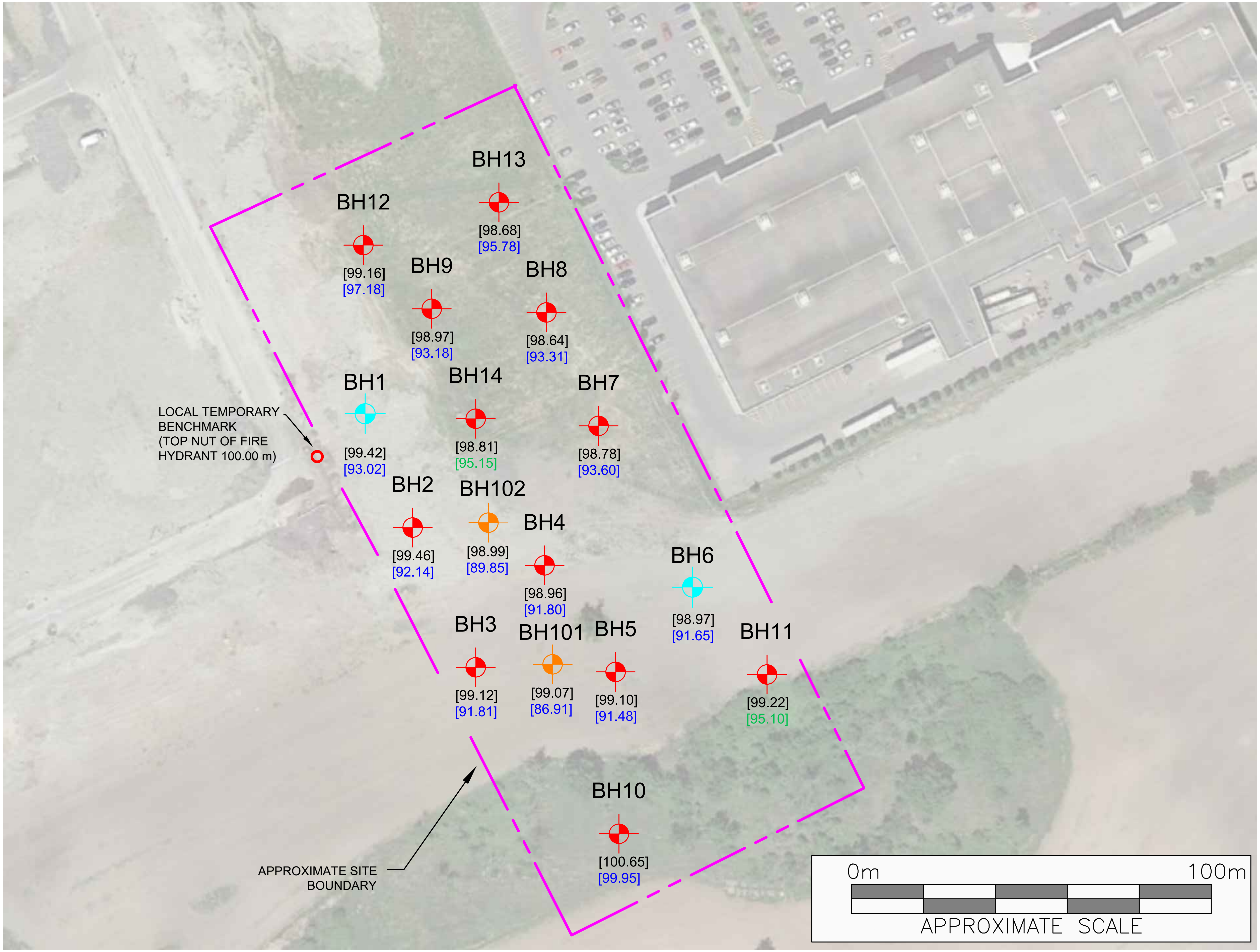
Template: Master Geotechnical Investigation Report – Ontario, GEO, September 2, 2021

FIGURES



PROJECT NAME	GEOTECHNICAL INVESTIGATION
CLIENT NAME	EXTENDICARE CANADA INC.
PROJECT LOCATION	3900 INNES ROAD, OTTAWA, ONTARIO
FIGURE NAME	KEY MAP
FIGURE NO.	1
APPROXIMATE SCALE	AS SHOWN
PROJECT NO.	304017.001
DATE	JANUARY 2023



LEGEND

- ⌖ BOREHOLE LOCATION (GEOTECHNICAL INVESTIGATION, 2022)
- ⌖ MONITORING WELL LOCATION (GEOTECHNICAL INVESTIGATION, 2022)
- ⌖ BOREHOLE LOCATION (GEOTECHNICAL INVESTIGATION, 2023)
- [XX.XX] APPROXIMATE LOCAL GROUND ELEVATION (m)
- [XX.XX] APPROXIMATE LOCAL REFUSAL ELEVATION (m)
- [XX.XX] APPROXIMATE LOCAL TERMINATION ELEVATION (m)

PINCHIN

PROJECT NAME
GEOTECHNICAL INVESTIGATION

CLIENT NAME
EXTENDICARE CANADA INC.

PROJECT LOCATION
3900 INNES ROAD, OTTAWA,
ONTARIO

FIGURE NAME
BOREHOLE/MONITORING WELL
LOCATION PLAN

APPROXIMATE SCALE	PROJECT NO.
AS SHOWN	304017.001
DATE	FIGURE NO.
JANUARY 2023	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
$\sigma'p$	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: BH101

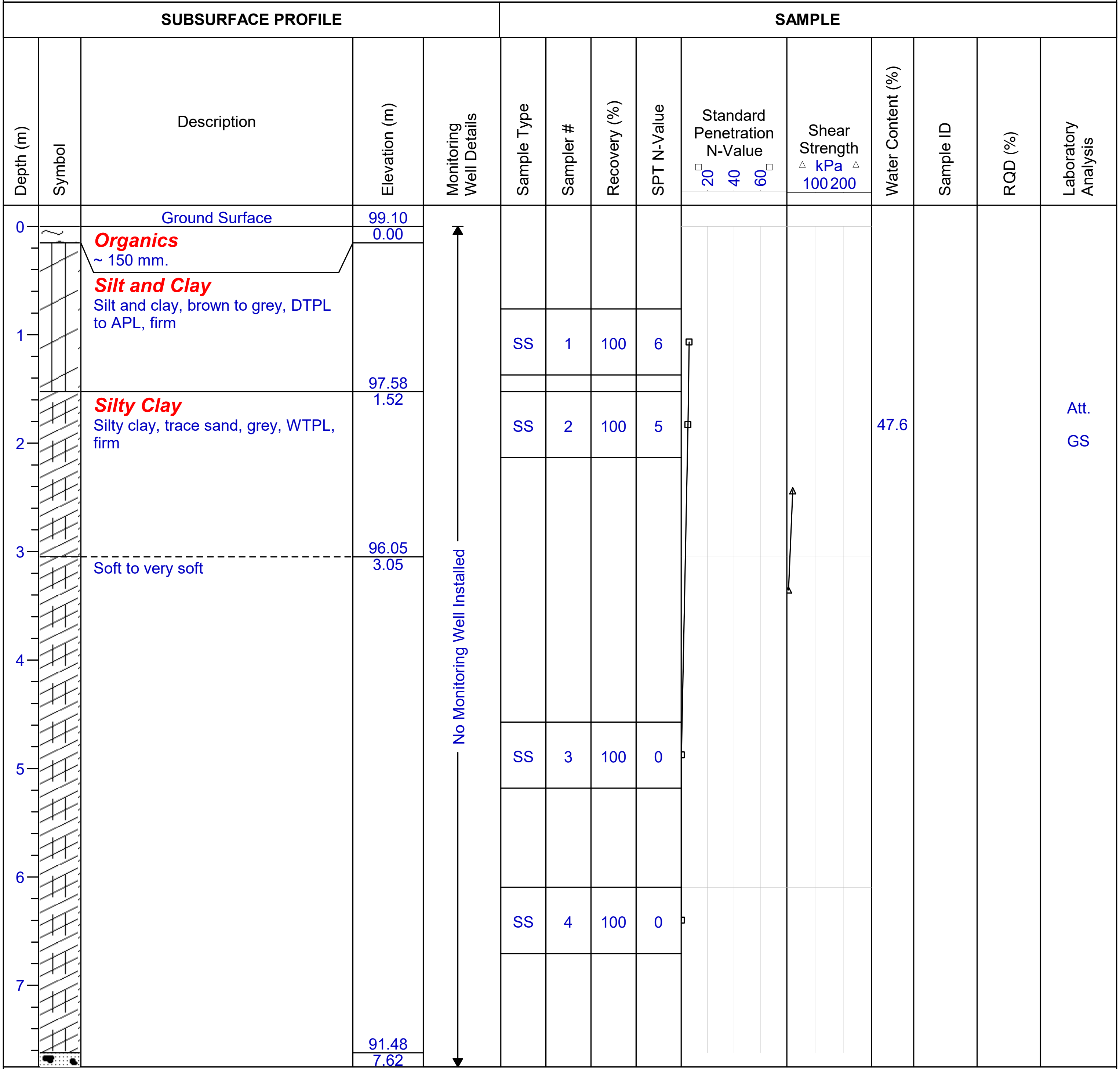
Project #: 304017.001 **Logged By:** MK

Project: Geotechnical Investigation

Client: Extendicare (Canada) Inc.

Location: 3900 Innes Road, Ottawa, Ontario

Drill Date: January 4, 2023 **Project Manager:** WT



Elevation (m)	Depth (m)	Description
99.10		Ground Surface
	0.00	**Organics** ~ 150 mm.
		Silt and Clay Silt and clay, brown to grey, DTPL to APL, firm
97.58	1.52	**Silty Clay** Silty clay, trace sand, grey, WTPL, firm
96.05	3.05	Soft to very soft
91.48	7.62	

Sample Type	Sampler #	Recovery (%)	SPT N-Value	Water Content (%)	Laboratory Analysis
SS	1	100	6		
SS	2	100	5	47.6	Att. GS
SS	3	100	0		
SS	4	100	0		

Contractor: Canadian Environmental Drilling & Contractors Inc. **Grade Elevation:** 99.07 m

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

Well Casing Size: N/A **Sheet:** 1 of 2

Log of Borehole: BH101

Project #: 304017.001 **Logged By:** MK

Project: Geotechnical Investigation

Client: Extendicare (Canada) Inc.

Location: 3900 Innes Road, Ottawa, Ontario

Drill Date: January 4, 2023 **Project Manager:** WT

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	RQD (%)	Laboratory Analysis
		Glacial Till Sandy gravel, trace silt, trace clay, grey, wet, loose	91.17 7.92											
8		**Bedrock** Shale bedrock, slightly weathered, dark grey with white and black spotting and occasional white to ligth grey banding, strong rock, medium to coarse grained, fair quality	89.96 9.14		RC	1	89	NA					52	
9		Excellent quality			RC	2	100	NA					93	
10 11 12			86.91 12.19		RC	3	98	NA					98	
13 14 15		End of Borehole Borehole was terminated at 12.2 mbgs in shale bedrock. Groundwater was encountered at 6.10 mbgs.												

Contractor: Canadian Environmental Drilling & Contractors Inc. **Grade Elevation:** 99.07 m

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

Well Casing Size: N/A **Sheet:** 2 of 2

Log of Borehole: BH102

Project #: 304017.001 **Logged By:** MK

Project: Geotechnical Investigation

Client: Extendicare (Canada) Inc.

Location: 3900 Innes Road, Ottawa, Ontario

Drill Date: January 4, 2023 **Project Manager:** WT

SUBSURFACE PROFILE			SAMPLE							
Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-Value	Water Content (%)	Sample ID	RQD (%)	Laboratory Analysis
Ground Surface	98.99 0.00	No Monitoring Well Installed								
Organics ~ 150 mm.										
Silt and Clay Silt and clay, brown to grey, DTPL to APL, firm										
Silty Clay Silty clay, trace sand, grey, WTPL, firm	97.47 1.52		SS	1	100	5				
			SS	2	100	3				
Soft to very soft	95.94 3.05		SS	3	100	0				
			SS	4	100	0	75.4			Att. GS
Bedrock Shale bedrock, slightly weathered, dark grey with white and black spotting and occasional white to ligth grey banding, strong rock, medium to coarse grained, excellent quality	92.89 6.10		RC	1	100	NA			100	

Contractor: Canadian Environmental Drilling & Contractors Inc. **Grade Elevation:** 98.99 m

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

Well Casing Size: N/A **Sheet:** 1 of 2

Log of Borehole: BH102

Project #: 304017.001 **Logged By:** MK

Project: Geotechnical Investigation

Client: Extendicare (Canada) Inc.

Location: 3900 Innes Road, Ottawa, Ontario

Drill Date: January 4, 2023 **Project Manager:** WT

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	RQD (%)	Laboratory Analysis
8 9	▤			↓	RC	2	100	NA					97	
10 11 12 13 14 15		End of Borehole Borehole was terminated at 9.1 mbgs in shale bedrock. Groundwater was encountered at 6.10 mbgs.	89.85 9.14											

Contractor: Canadian Environmental Drilling & Contractors Inc. **Grade Elevation:** 98.99 m

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

Well Casing Size: N/A **Sheet:** 2 of 2

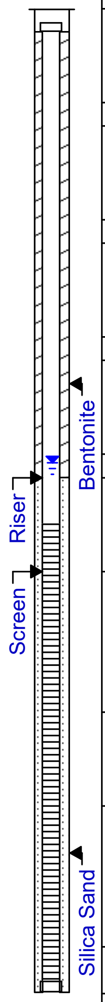
Log of Borehole: BH1

Project #: 304017.001
Logged By: MK
Project: Geotechnical Investigation
Client: Extendicare Canada Inc.
Location: 3900 Innes Road, Ottawa, Ontario
Drill Date: April 20, 2022
Project Manager: WT

SUBSURFACE PROFILE

Depth (m)	Description	Elevation (m)
0	Ground Surface	99.42
0 – 0.3	**Organics** ~ 300 mm.	99.11
0.3 – 1.5	**Silt and Clay** Silt and clay, trace sand, brown, DTPL to ATPL, stiff	97.90
1.5 – 2.3	WTPL	97.13
2.3 – 4.6	**Silty Clay** Silty clay, trace sand, grey, WTPL, soft to very soft	94.85
4.6 – 6.4	**Glacial Till** Sandy gravel, trace silt, trace clay, grey, wet, compact	93.02
6.4	End of Borehole Borehole was terminated at 6.4 mbgs due to auger refusal on probable bedrock.	

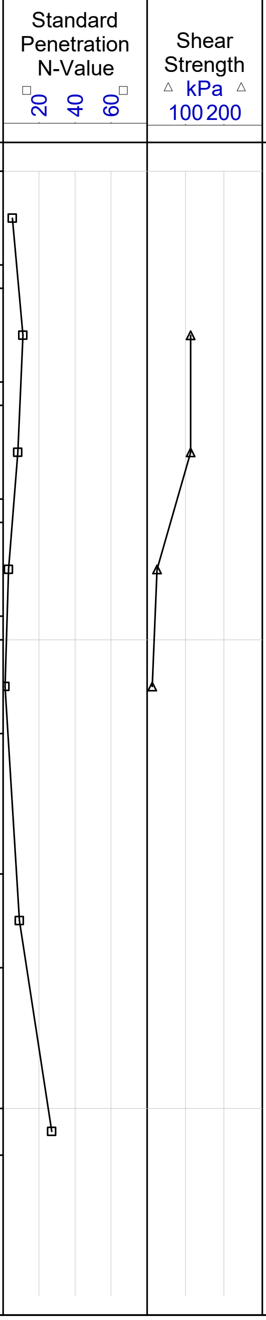
Monitoring Well Details



Groundwater level = 2.95 mbgs, as measured on May 20, 2022.

SAMPLE

Sample Type	Sampler #	Recovery (%)	SPT N-Value	Water Content (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
SS	1	40	5				
SS	2	60	11				
SS	3	70	8				
SS	4	100	3	65.1			Att. GS
SS	5	100	1				
SS	6	20	9				
SS	7	10	27				

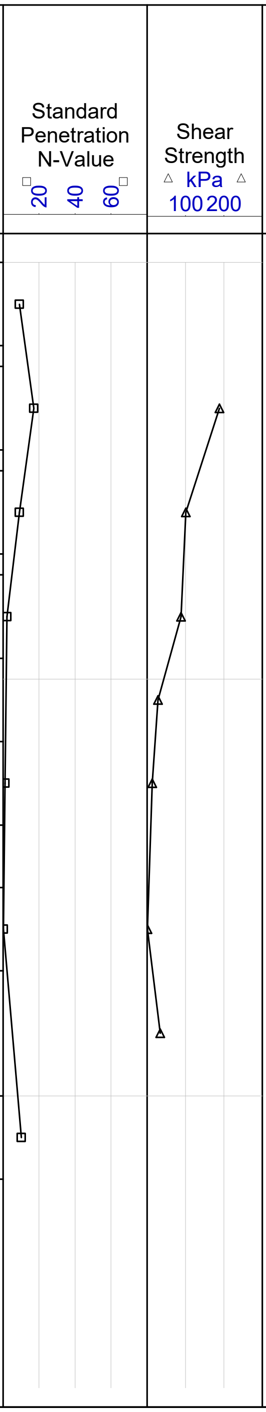


Contractor: Marathon Underground
Grade Elevation: 99.42 m
Drilling Method: Split Spoon / Hollow Stem Auger
Top of Casing Elevation: N/A
Well Casing Size: 51 mm
Sheet: 1 of 1

Log of Borehole: BH2

Project #: 304017.001
Logged By: MK
Project: Geotechnical Investigation
Client: Extendicare Canada Inc.
Location: 3900 Innes Road, Ottawa, Ontario
Drill Date: April 20, 2022
Project Manager: WT

SUBSURFACE PROFILE				SAMPLE							
Depth (m)	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-Value	Water Content (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
	Ground Surface	99.46	No Monitoring Well Installed								
0	**Organics** ~ 600 mm	98.85		SS	1	20	9				
1	**Silt and Clay** Silt and clay, trace sand, brown, DTPL to ATPL, very stiff	97.94		SS	2	40	17				
2	WTPL, stiff	97.17		SS	3	60	9				
3 4 5 6	**Silty Clay** Silty clay, trace sand, grey, WTPL, soft to very soft	93.36		SS SS SS	4 5 6	100 100 100	2 1 0				
7	**Glacial Till** Sandy gravel, trace silt, trace clay, grey, wet, compact	92.14		SS	7	20	10	13.3			Hyd.
8	End of Borehole Borehole was terminated at 7.32 mbgs due to auger refusal on probable bedrock. Groundwater was encountered at 6.10 mbgs.										



Contractor: Marathon Underground
Grade Elevation: 99.46 m
Drilling Method: Split Spoon / Hollow Stem Auger
Top of Casing Elevation: N/A
Well Casing Size: N/A
Sheet: 1 of 1

Log of Borehole: BH3

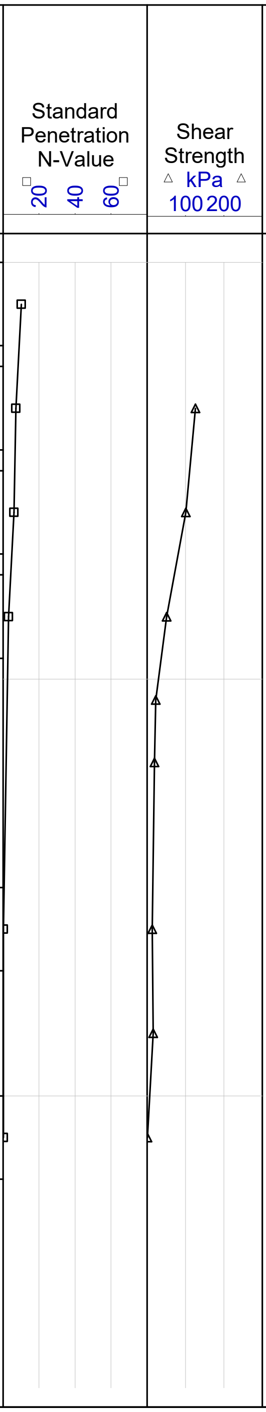
Project #: 304017.001
Logged By: MK
Project: Geotechnical Investigation
Client: Extendicare Canada Inc.
Location: 3900 Innes Road, Ottawa, Ontario
Drill Date: April 20, 2022
Project Manager: WT

SUBSURFACE PROFILE

Description	Elevation (m)	Monitoring Well Details
Ground Surface	99.12	No Monitoring Well Installed
Organics ~150 mm.		
Silt and Clay Silt and clay, trace sand, brown, DTPL to ATPL, stiff to firm	97.60	
WTPL	96.83	
Silty Clay Silty Clay, trace sand, grey, WTPL, soft to very soft	91.81	
End of Borehole Borehole was terminated at 7.32 mbgs due to auger refusal on probable bedrock. Groundwater was encountered at 4.57 mbgs.		

SAMPLE

Sample Type	Sampler #	Recovery (%)	SPT N-Value	Water Content (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
SS	1	30	10				
SS	2	50	7				
SS	3	60	6				
SS	4	100	3				
SS	5	100	0				
SS	6	100	0				



Contractor: Marathon Underground
Grade Elevation: 99.12 m
Drilling Method: Split Spoon / Hollow Stem Auger
Top of Casing Elevation: N/A
Well Casing Size: N/A
Sheet: 1 of 1

Log of Borehole: BH4

Project #: 304017.001 **Logged By:** MK

Project: Geotechnical Investigation

Client: Extendicare Canada Inc.

Location: 3900 Innes Road, Ottawa, Ontario

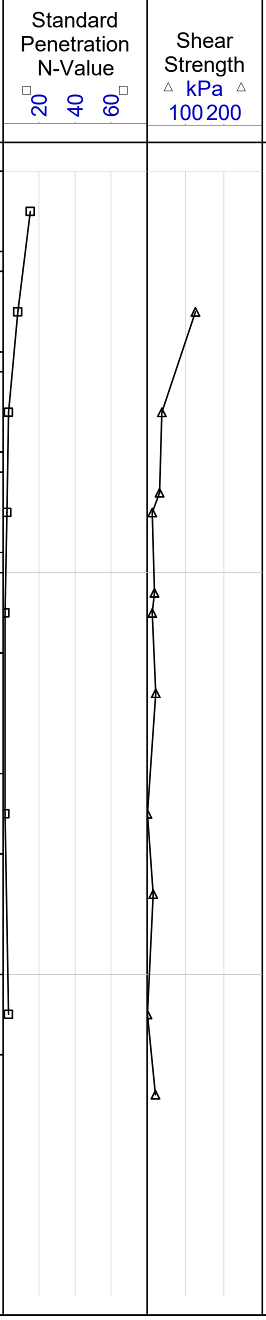
Drill Date: April 21, 2022 **Project Manager:** WT

SUBSURFACE PROFILE

Depth (m)	Symbol	Description	Elevation (m)	Monitoring Well Details
		Ground Surface	98.96	
0	~	**Organics** ~ 150 mm.		No Monitoring Well Installed
		Silt and Clay Silt and clay, trace sand, brown, DTPL to ATPL, stiff	97.44	
		Silty Clay Silty clay, trace sand, grey, WTPL, soft to very soft	91.80	
		End of Borehole Borehole was terminated at 7.16 mbgs due to auger refusal on probable bedrock. Groundwater was encountered at 4.57 mbgs.		

SAMPLE

Sample Type	Sampler #	Recovery (%)	SPT N-Value	Water Content (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
SS	1	50	15				
SS	2	70	8				
SS	3	100	3				
SS	4	100	2				
SS	5	100	1				
SS	6	100	1				
SS	7	100	3				



Contractor: Marathon Underground **Grade Elevation:** 98.96 m

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

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

Log of Borehole: BH5

Project #: 304017.001 **Logged By:** MK

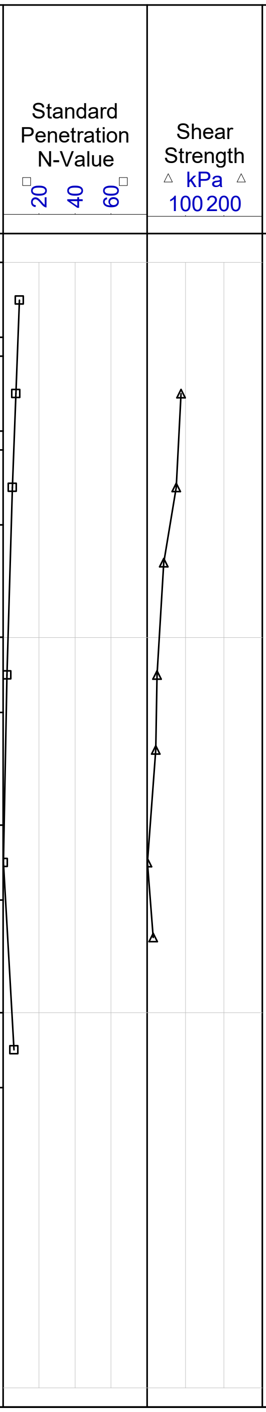
Project: Geotechnical Investigation

Client: Extendicare Canada Inc.

Location: 3900 Innes Road, Ottawa, Ontario

Drill Date: April 21, 2022 **Project Manager:** WT

SUBSURFACE PROFILE					SAMPLE							
Depth (m)	Symbol	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-Value	Water Content (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
0		Ground Surface	99.10	No Monitoring Well Installed								
		Organics ~ 150 mm.			SS	1	10	9				
1		**Silt and Clay** Silt and clay, trace sand, brown, DTPL to ATPL, firm	97.58		SS	2	80	7				
2		**Silty Clay** Silty clay, trace sand, grey, WTPL, firm	96.05		SS	3	100	5				
3 4		Soft to very soft			SS	4	100	2				
5 6			93.00		SS	5	100	0		87.0		Att. GS
7		**Glacial Till** Sandy gravel, trace silt, trace clay, grey, wet, loose	91.48		SS	6	30	6				
8 9		End of Borehole Borehole was terminated at 7.62 mbgs due to auger refusal on probable bedrock. Groundwater was encountered at 6.10 mbgs.										



Contractor: Marathon Underground **Grade Elevation:** 99.10 m

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

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

Log of Borehole: BH6

Project #: 304017.001

Logged By: MK

Project: Geotechnical Investigation

Client: Extendicare Canada Inc.

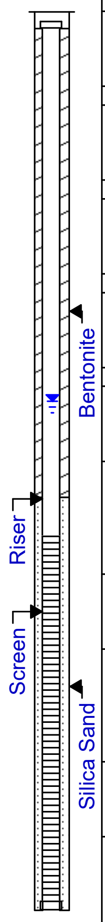
Location: 3900 Innes Road, Ottawa, Ontario

Drill Date: April 22, 2022

Project Manager: WT

SUBSURFACE PROFILE

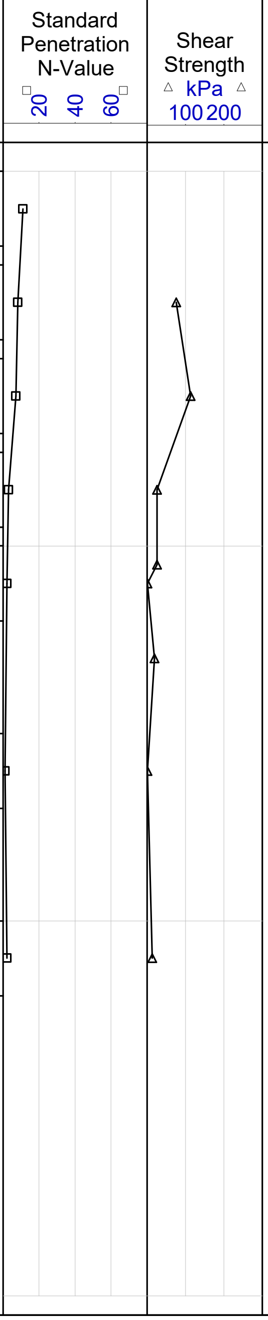
Depth (m)	Description	Elevation (m)
0	Ground Surface	98.97
0 – 0.15	**Organics** ~ 150 mm.	
0.15 – 1.53	**Silt and Clay** Silt and clay, trace sand, brown, DTPL to ATPL, stiff	97.44
1.53 – 7.32	**Silty Clay** Silty clay, trace sand, brown, WTPL, firm to very soft	91.65
7.32	End of Borehole Borehole was terminated at 7.32 mbgs due to auger refusal on probable bedrock.	



Groundwater level = 3.17 mbgs, as measured on May 20, 2022.

SAMPLE

Sample Type	Sampler #	Recovery (%)	SPT N-Value	Water Content (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
SS	1	20	11				
SS	2	80	8				
SS	3	100	7				
SS	4	100	3				
SS	5	100	2				
SS	6	100	1				
SS	7	100	2				



Contractor: Marathon Underground

Grade Elevation: 98.97 m

Drilling Method: Split Spoon / Hollow Stem Auger

Top of Casing Elevation: N/A

Well Casing Size: 51 mm.

Sheet: 1 of 1

Log of Borehole: BH7

Project #: 304017.001 **Logged By:** MK

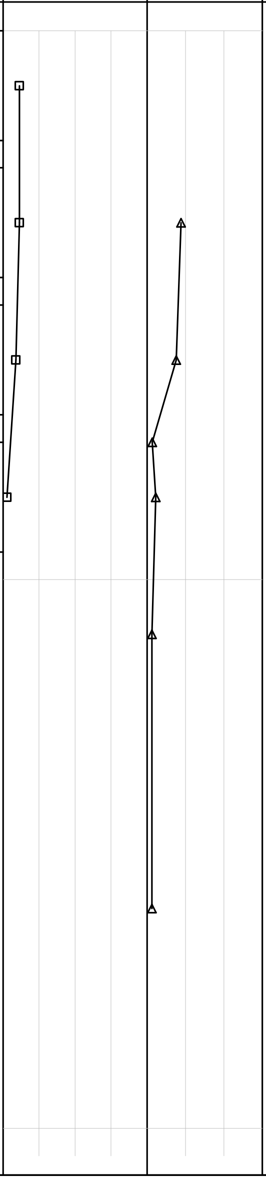
Project: Geotechnical Investigation

Client: Extendicare Canada Inc.

Location: 3900 Innes Road, Ottawa, Ontario

Drill Date: April 22, 2022 **Project Manager:** WT

SUBSURFACE PROFILE					SAMPLE									
Depth (m)	Symbol	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-Value	Standard Penetration N-Value □ 20 40 60 □	Shear Strength △ kPa △ 100 200	Water Content (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
		Ground Surface	98.78											
0		**Organics** ~ 150 mm.		No Monitoring Well Installed	SS	1	20	9						
1		**Silt and Clay** Silt and clay, trace sand, brown, DTPL to ATPL, stiff	97.26		SS	2	70	9						
2 3 4 5		**Silty Clay** Silty clay, trace sand, grey, WTPL, firm to soft	93.60		SS	3	90	7			50.0			Att. GS.
					SS	4	100	2						
6		End of Borehole Borehole was terminated at 5.18 mbgs due to auger refusal on probable bedrock. Groundwater was encountered at 2.29 mbgs.												



Contractor: Marathon Underground

Grade Elevation: 98.78 m

Drilling Method: Split Spoon / Hollow Stem Auger

Top of Casing Elevation: N/A

Well Casing Size: N/A

Sheet: 1 of 1

Log of Borehole: BH8

Project #: 304017.001 ***Logged By:*** MK

Project: Geotechnical Investigation

Client: Extendicare Canada Inc.

Location: 3900 Innes Road, Ottawa, Ontario

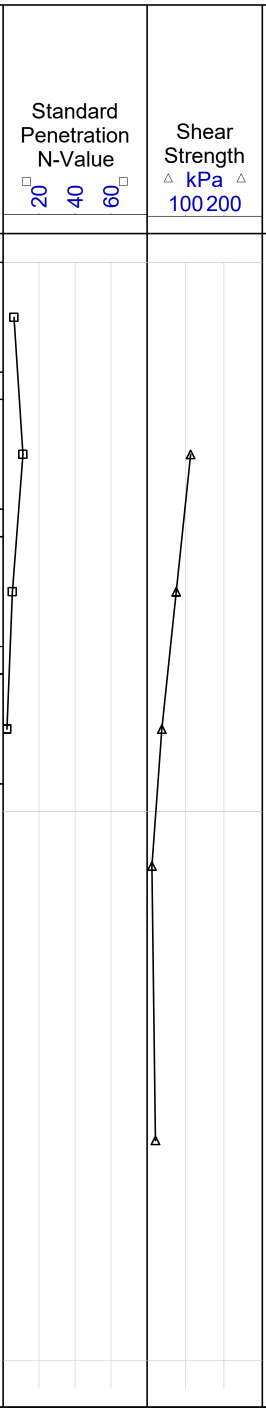
Drill Date: April 22, 2022 ***Project Manager:*** WT

SUBSURFACE PROFILE

Depth (m)	Description	Elevation (m)	Monitoring Well Details
0	Ground Surface	98.64	No Monitoring Well Installed
0 – 0.15	**Organics** ~150 mm		
0.15 – 1.52	**Silt and Clay** Silt and clay, trace sand, brown, DTPL to ATPL, firm to stiff	97.12	
1.52 – 5.33	**Silty Clay** Silty clay, trace sand, grey, WTPL, firm to soft	93.31	
5.33	End of Borehole Borehole was terminated at 5.33 mbgs due to auger refusal on probable bedrock. Groundwater was encountered at 4.57 mbgs.		

SAMPLE

Sample Type	Sampler #	Recovery (%)	SPT N-Value	Water Content (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
SS	1	50	6				
SS	2	80	11				
SS	3	100	5				
SS	4	100	2				



Contractor: Marathon Underground ***Grade Elevation:*** 98.64 m

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

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

Log of Borehole: BH9

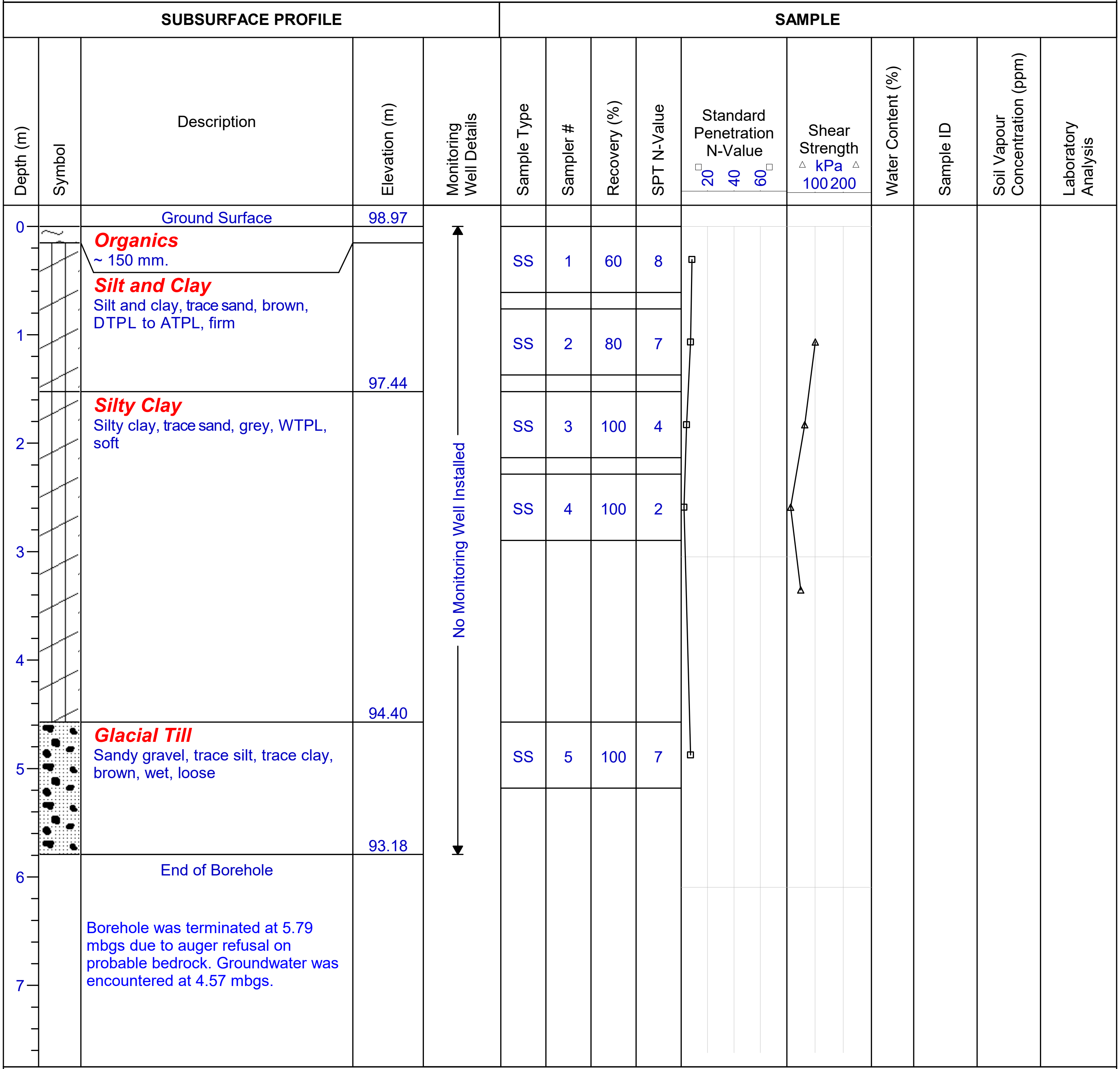
Project #: 304017.001 **Logged By:** MK

Project: Geotechnical Investigation

Client: Extendicare Canada Inc.

Location: 3900 Innes Road, Ottawa, Ontario

Drill Date: April 22, 2022 **Project Manager:** WT



Description	Elevation (m)
Ground Surface	98.97
Organics ~ 150 mm.	
Silt and Clay Silt and clay, trace sand, brown, DTPL to ATPL, firm	97.44
Silty Clay Silty clay, trace sand, grey, WTPL, soft	94.40
Glacial Till Sandy gravel, trace silt, trace clay, brown, wet, loose	93.18
End of Borehole	

Sample Type	Sampler #	Recovery (%)	SPT N-Value
SS	1	60	8
SS	2	80	7
SS	3	100	4
SS	4	100	2
SS	5	100	7

Borehole was terminated at 5.79 mbgs due to auger refusal on probable bedrock. Groundwater was encountered at 4.57 mbgs.

Contractor: Marathon Underground

Grade Elevation: 98.97 m

Drilling Method: Split Spoon / Hollow Stem Auger

Top of Casing Elevation: N/A

Well Casing Size: N/A

Sheet: 1 of 1

PINCHIN

Log of Borehole: BH10

Project #: 304017.001 **Logged By:** MK

Project: Geotechnical Investigation

Client: Extendicare Canada Inc

Location: 3900 Innes Road, Ottawa, Ontario

Drill Date: April 20, 2022 **Project Manager:** WT

SUBSURFACE PROFILE					SAMPLE									
Depth (m)	Symbol	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-Value	Standard Penetration N-Value ☐ 20 40 60 ☐	Shear Strength △ kPa △ 100 200	Water Content (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
		Ground Surface	100.65											
0	≈	**Organics** ~ 700 mm		No Monitoring Well Installed										
		End of Borehole	99.95											
1		Borehole was terminated at 0.70 mbgs due to auger refusal on probable bedrock. At drilling completion, groundwater was not encountered.												
2														

Contractor: Marathon Underground

Grade Elevation: 100.65 m

Drilling Method: Split Spoon / Hollow Stem Auger

Top of Casing Elevation: N/A

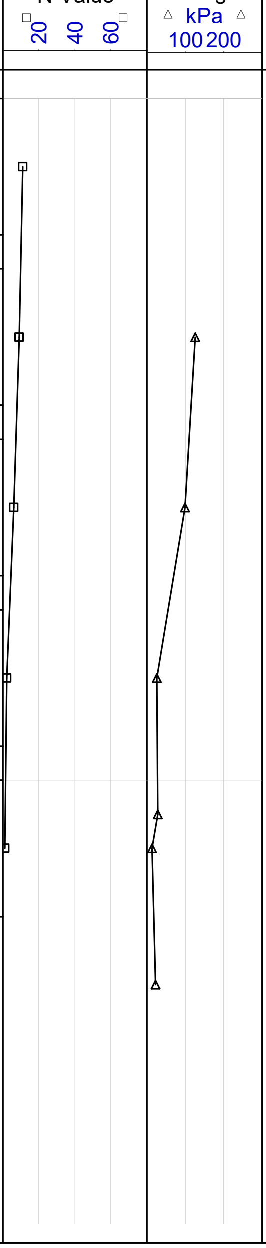
Well Casing Size: N/A

Sheet: 1 of 1

Log of Borehole: BH11

Project #: 304017.001
Logged By: MK
Project: Geotechnical Investigation
Client: Extendicare Canada Inc.
Location: 3900 Innes Road, Ottawa, Ontario
Drill Date: April 21, 2022
Project Manager: WT

SUBSURFACE PROFILE					SAMPLE									
Depth (m)	Symbol	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-Value	Standard Penetration N-Value	Shear Strength kPa	Water Content (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
		Ground Surface	99.22											
0		**Organics** ~ 150 mm.		No Monitoring Well Installed										
		Silt and Clay Silt and clay, trace sand, brown, DTPL to ATPL, stiff			SS	1	10	11						
1					SS	2	60	9						
		Silty Clay Silty clay, trace sand, brown, WTPL, firm to soft	97.69		SS	3	100	6			51.1			Hyd.
2					SS	4	100	2						
3		Very soft	96.17		SS	5	100	1						
4		End of Borehole	95.10											
5		Borehole was terminated at 4.11 mbgs in very soft silty clay. Groundwater was encountered at 3.05 mbgs.												



Contractor: Marathon Underground
Grade Elevation: 99.22 m
Drilling Method: Split Spoon / Hollow Stem Auger
Top of Casing Elevation: N/A
Well Casing Size: N/A
Sheet: 1 of 1

Log of Borehole: BH12

Project #: 304017.001 **Logged By:** MK

Project: Geotechnical Investigation

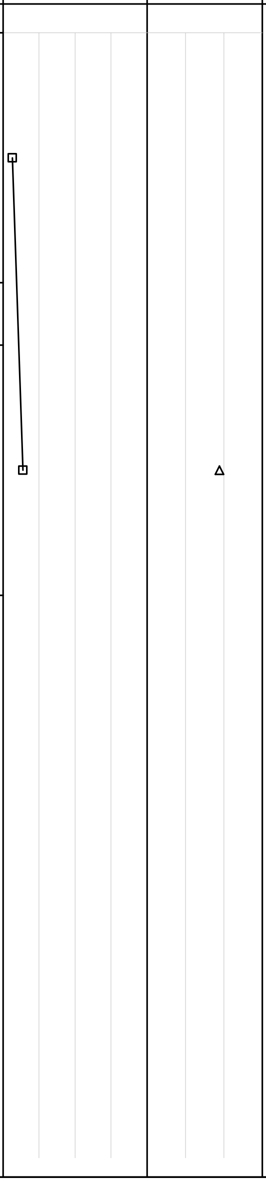
Client: Extendicare Canada Inc.

Location: 3900 Innes Road, Ottawa, Ontario

Drill Date: April 22, 2022 **Project Manager:** WT

SUBSURFACE PROFILE / **SAMPLE**

Depth (m)	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-Value	Water Content (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
	Ground Surface	99.16									
0	**Organics** ~ 150 mm.	99.01	No Monitoring Well Installed								
	Silt and Clay Silt and clay, trace sand, brown, DTPL, firm	98.40		SS	1	30	5				
1	**Silty Clay** Silty clay, trace sand, brown, ATPL, stiff	97.64		SS	2	70	11				
	WTPL	97.18									
2	End of Borehole Borehole was terminated at 1.98 mbgs due to auger refusal on probable bedrock. At drilling completion, no groundwater was encountered.										



Contractor: Marathon Underground **Grade Elevation:** 99.16 m

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

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

Log of Borehole: BH13

Project #: 304017.001 **Logged By:** MK

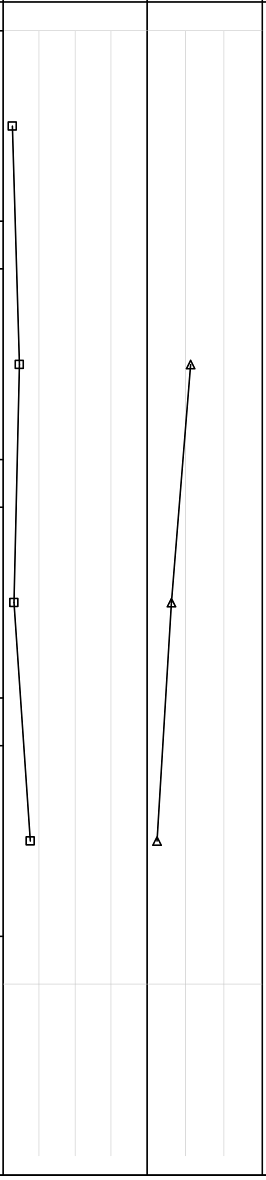
Project: Geotechnical Investigation

Client: Extendicare Canada Inc.

Location: 3900 Innes Road, Ottawa, Ontario

Drill Date: April 22, 2022 **Project Manager:** WT

SUBSURFACE PROFILE					SAMPLE										
Depth (m)	Symbol	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-Value	Standard Penetration N-Value 20 40 60	Shear Strength kPa 100 200	Water Content (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
		Ground Surface	98.68											
0		**Organics** ~ 150 mm.	98.53	No Monitoring Well Installed										
		Silt and Clay Silt and clay, trace sand, brown, DTPL to ATPL, firm			SS	1	50	5						
1			97.16		SS	2	70	9						
2		**Silty Clay** Silty clay, trace sand, grey, WTPL, stiff	96.24		SS	3	60	6						
		Glacial Till Sandy gravel, trace silt, trace clay, brown, moist, compact	95.78		SS	4	30	15						
3		End of Borehole Borehole was terminated at 2.90 mbgs due to auger refusal on probable bedrock. At drilling completion, no groundwater was encountered.												



Contractor: Marathon Underground **Grade Elevation:** 98.68 m

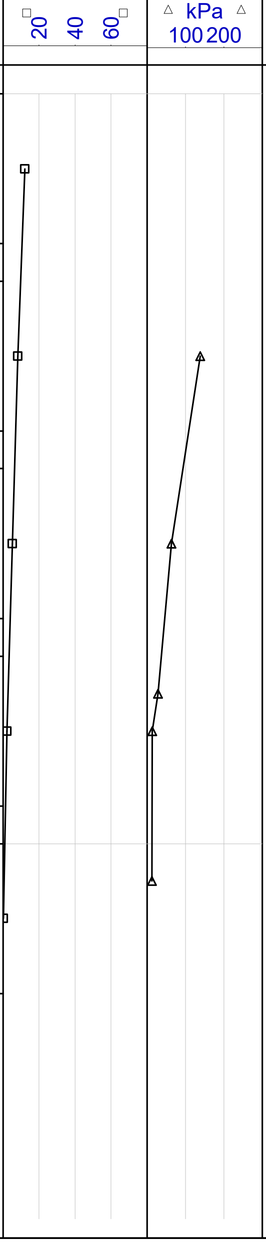
Drilling Method: Split Spoon / Hollow Stem Auger **Top of Casing Elevation:** N/A

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

Log of Borehole: BH14

Project #: 304017.001 **Logged By:** MK
Project: Geotechnical Investigation
Client: Extendicare Canada Inc.
Location: 3900 Innes Road, Ottawa, Ontario
Drill Date: April 21, 2022 **Project Manager:** WT

SUBSURFACE PROFILE					SAMPLE									
Depth (m)	Symbol	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-Value	Standard Penetration N-Value	Shear Strength kPa	Water Content (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
		Ground Surface	98.81											
0		**Organics** ~ 150 mm.		No Monitoring Well Installed										
		Silt and Clay Silt and clay, trace sand, brown, DTPL to ATPL, stiff			SS	1	40	12						
1					SS	2	60	8						
		Silty Clay Silty clay, trace sand, brown, WTPL, firm	97.29		SS	3	80	5						
2		Soft to very soft	96.52		SS	4	100	2						
3					SS	5	100	0						
		End of Borehole Borehole was terminated at 3.66 mbgs in very soft silty clay. At drilling completion, groundwater was encountered at 3.05 mbgs.	95.15											
4														



Contractor: Marathon Underground **Grade Elevation:** 98.81 m
Drilling Method: Split Spoon / Hollow Stem Auger **Top of Casing Elevation:** N/A
Well Casing Size: N/A **Sheet:** 1 of 1

APPENDIX III

Laboratory Testing Reports for Soil Samples

PATERSON GROUP

CONCRETE CORE COMPRESSIVE STRENGTH CSA A23.2-14C

CLIENT:	**Pinchin Environmental**		FILE No.: **PM4184**
PROJECT:	Lab Testing - Job # 304017.001		REPORT No.: 1
SITE ADDRESS:	-		DATE REPT'D: 30-Jan-23
STRUCTURE TYPE & LOCATION:	Rock Core		
SAMPLE INFORAMTION			
LAB NO.:	**41762**		
SAMPLE NO.:	-		
LOCATION:	BH 102 / 24' 2"		
SAMPLE DATES			
DATE CAST	-		
DATE CORED	4-Jan-23		
DATE RECEIVED	27-Jan-23		
DATE TESTED	30-Jan-23		
SAMPLE DIMENSIONS			
AVERAGE DIAMETER (mm)	47.00		
HEIGHT (mm)	92.00		
WEIGHT (g)	440		
AREA (mm^2)	1735		
VOLUME (cm^3)	160		
UNIT WEIGHT (kg/m^3)	**2757**		
TEST RESULTS			
H / D RATIO	1.96		
CORRECTION FACTOR	0.996		
LOAD (lbs)	25860		
GROSS Mpa	66.3		
MPa CORRECTED	**66.0**		
FORM OF BREAK	**Type A**		
DIRECTION OF LOADING	PARALLEL		
CURING CONDITIONS	SITE→→→→→→→→→→→→→→→→→→→→→→→→→→→→→→→		
REMARKS			

TECHNICAL PERSONNEL

TECHNICIAN:		**VERIFIED BY:**	**C. Beadow**	**APPROVED BY:**	**Joe Forsyth, P. Eng.**

CERTIFIED LAB

John D. Paterson & Associates Ltd., 28 Concourse Gate, Nepean, ON

Photo 1: Before Cut

Photo 2: After Cut – Front

Photo 3: After Cut – Back

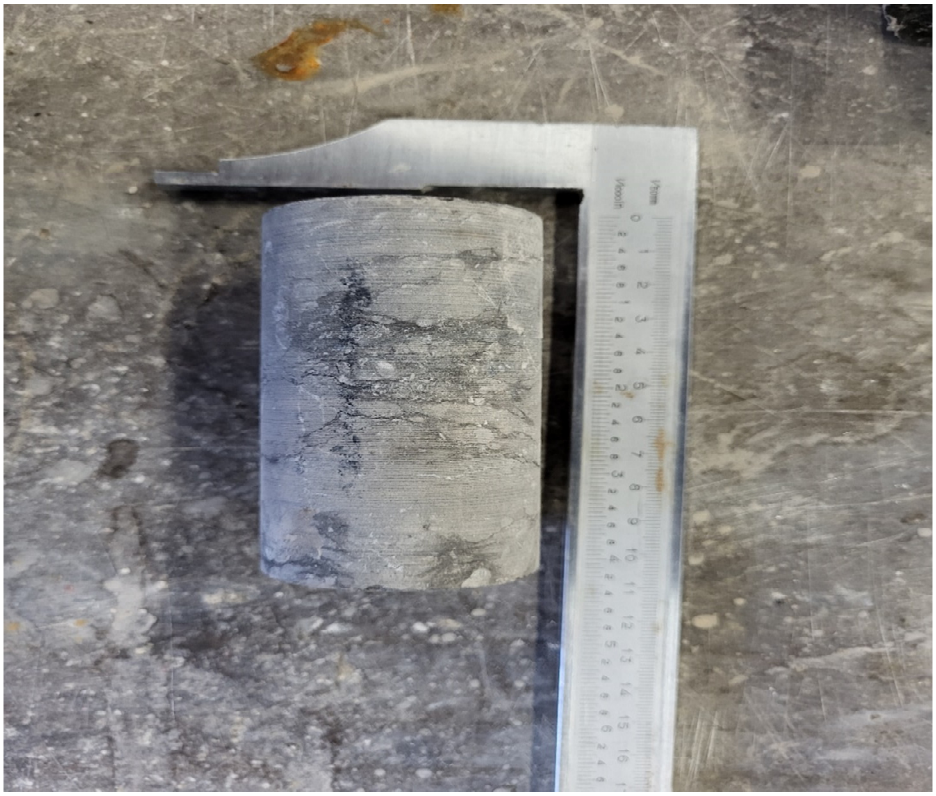


Photo 4: Break in Machine – Front

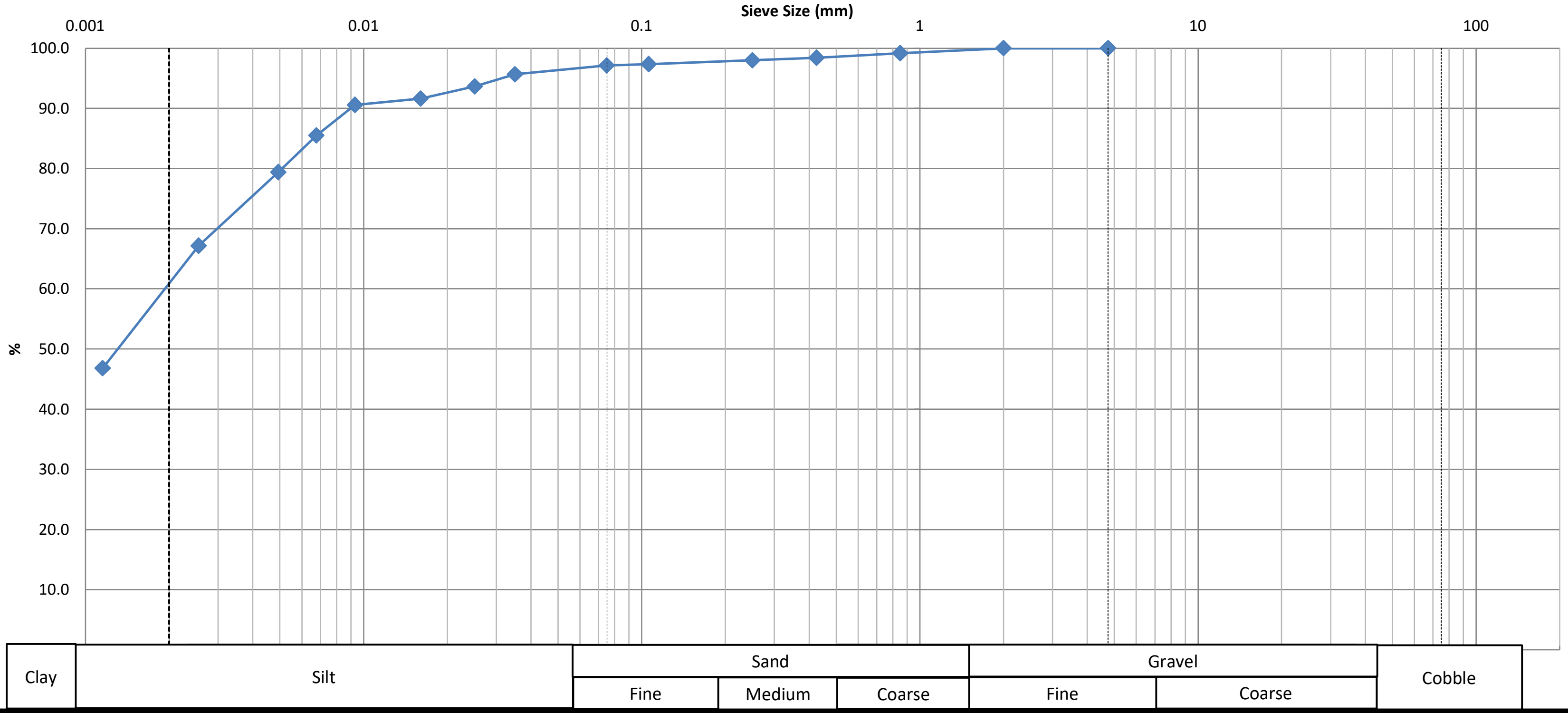
Photo 5: Break in Machine – Back

Photo 6: Broken

PATERSON GROUP

SIEVE ANALYSIS
ASTM C136

CLIENT:	Pinchin	DEPTH:	5'-0" to 7'-0"	FILE NO:	PM4184
CONTRACT NO.:		BH OR TP No.:	BH-101	LAB NO:	41674
PROJECT:	304017.001			DATE RECEIVED:	10-Jan-23
				DATE TESTED:	12-Jan-23
DATE SAMPLED:	4-Jan-23			DATE REPORTED:	19-Jan-23
SAMPLED BY:	Client			TESTED BY:	DK/CS



Identification	Soil Classification	MC(%)	LL	PL	PI	Cc	Cu
		47.6					

	D100	D60	D30	D10	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
					0.0	2.9	36.6	60.5

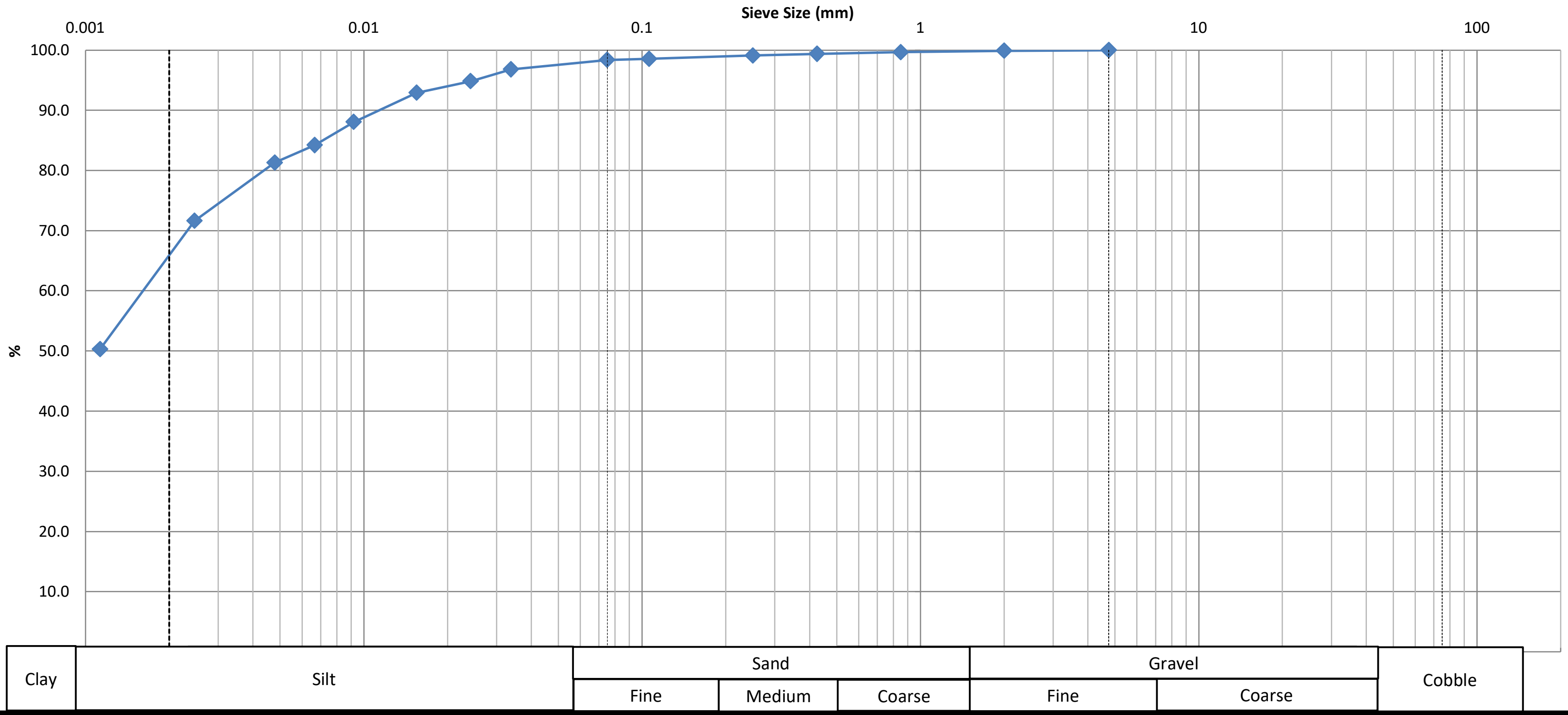
Comments:

REVIEWED BY:	**Curtis Beadow**	**Joe Forsyth, P. Eng.**

PATERSON GROUP

SIEVE ANALYSIS
ASTM C136

CLIENT:	Pinchin	DEPTH:	15'-0" to 17'-0"	FILE NO:	PM4184
CONTRACT NO.:		BH OR TP No.:	BH-102	LAB NO:	41675
PROJECT:	304017.001			DATE RECEIVED:	10-Jan-23
				DATE TESTED:	12-Jan-23
DATE SAMPLED:	4-Jan-23			DATE REPORTED:	19-Jan-23
SAMPLED BY:	Client			TESTED BY:	DK/CS



Identification	Soil Classification	MC(%)	LL	PL	PI	Cc	Cu
		75.4					

D100	D60	D30	D10	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
				0.0	1.6	32.4	66.0

Comments:

REVIEWED BY:	**Curtis Beadow**	**Joe Forsyth, P. Eng.**

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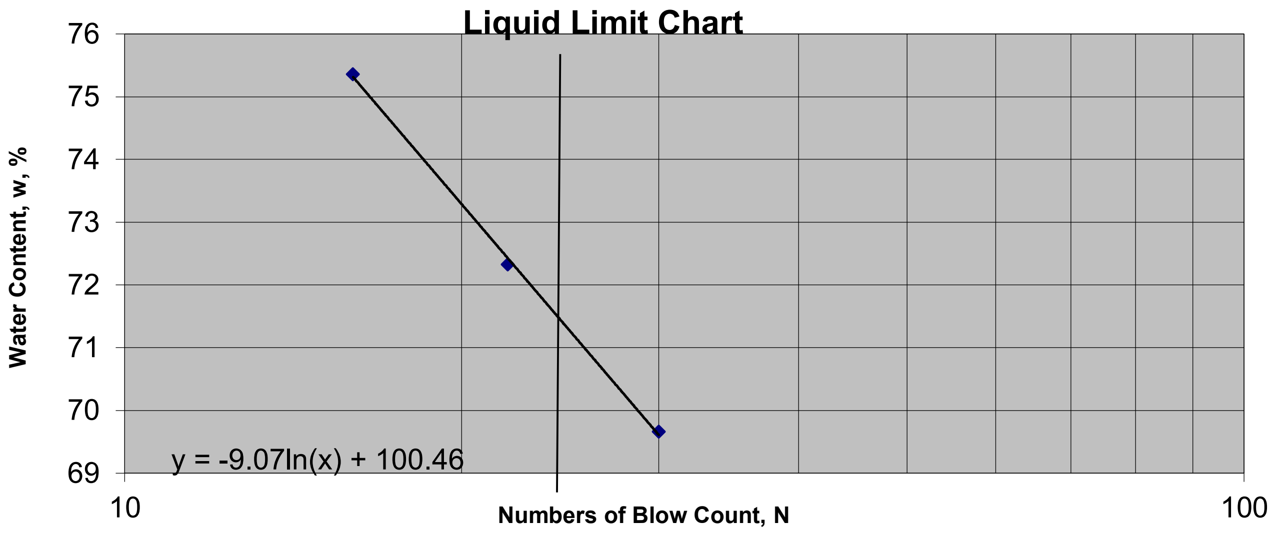
ATTERBERG LIMITS
LS-703/704

CLIENT:	Pinchin	FILE NO.:	PM4184
PROJECT:	304017.001	DATE SAMPLED:	10-Jan
LOCATION:	BH101 @ 5'-7'	DATE REPORTED:	23-Jan

CAN NO.	**30**	**31**	**32**				
WT. OF CAN	4.38	4.38	4.42				
WT. OF SOIL & CAN	16.55	16.65	16.33				
WT. OF DRY SOIL & CAN	11.32	11.50	11.44				
WT. OF MOISTURE	5.23	5.15	4.89				
WT. OF DRY SOIL & CAN	6.94	7.12	7.02				
WATER CONTENT, w, %	**75.36**	**72.33**	**69.66**				
NO. OF BLOWS, N	16	22	30				

CAN NO.	**9**	**18**
WT. OF CAN	19.35	20.01
WT. OF SOIL & CAN	27.09	28.12
WT. OF DRY SOIL & CAN	25.11	26.04
WT. OF MOISTURE	1.98	2.08
WT. OF DRY SOIL & CAN	5.76	6.03
WATER CONTENT, w, %	**34.38**	**34.49**

RESULTS	
LIQUID LIMIT	**72**
PLASTIC LIMIT	**34**
PLASTICITY INDEX	**38**



TECHNICIAN: CS	**REVIEWED BY:**	**C. Beadow**	**J. Forsyth, P. Eng.**

PATERSON GROUP

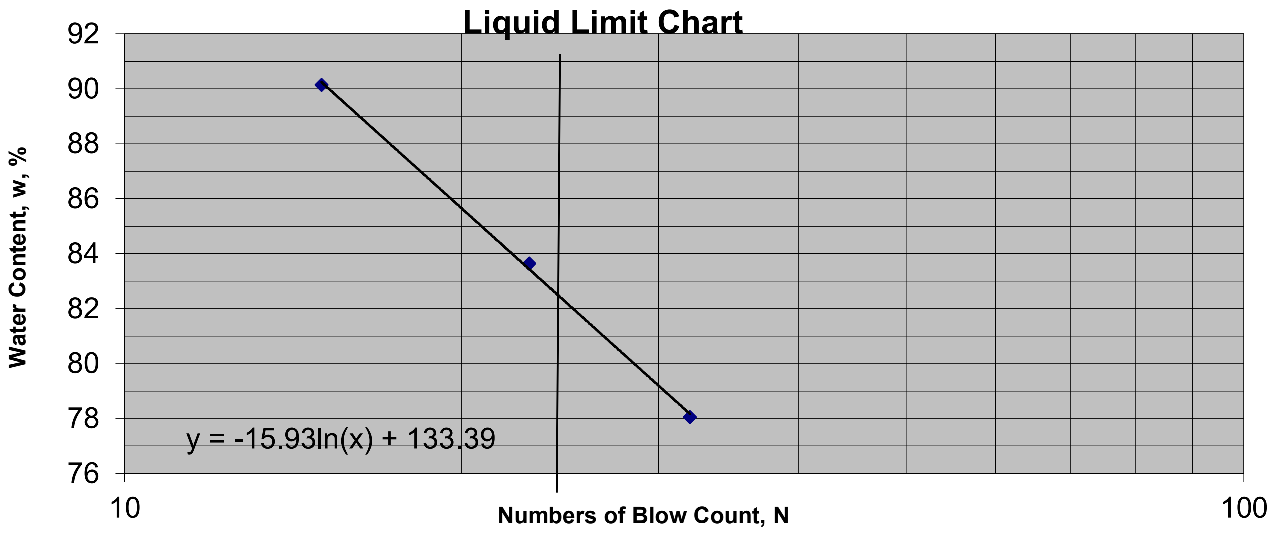
ATTERBERG LIMITS
LS-703/704

CLIENT:	Pinchin	FILE NO.:	PM4184
PROJECT:	304017.001	DATE SAMPLED:	10-Jan
LOCATION:	BH102 @ 15' - 17'	DATE REPORTED:	23-Jan

CAN NO.	**2**	**3**	**4**				
WT. OF CAN	8.70	8.71	8.71				
WT. OF SOIL & CAN	19.12	19.71	19.50				
WT. OF DRY SOIL & CAN	14.18	14.70	14.77				
WT. OF MOISTURE	4.94	5.01	4.73				
WT. OF DRY SOIL & CAN	5.48	5.99	6.06				
WATER CONTENT, w, %	**90.15**	**83.64**	**78.05**				
NO. OF BLOWS, N	15	23	32				

CAN NO.	**3**	**12**
WT. OF CAN	19.4	16.73
WT. OF SOIL & CAN	28.31	24.96
WT. OF DRY SOIL & CAN	25.74	22.61
WT. OF MOISTURE	2.57	2.35
WT. OF DRY SOIL & CAN	6.34	5.88
WATER CONTENT, w, %	**40.54**	**39.97**

RESULTS	
LIQUID LIMIT	**84**
PLASTIC LIMIT	**40**
PLASTICITY INDEX	**44**



TECHNICIAN:CS	**REVIEWED BY:**	**C. Beadow**	**J. Forsyth, P. Eng.**

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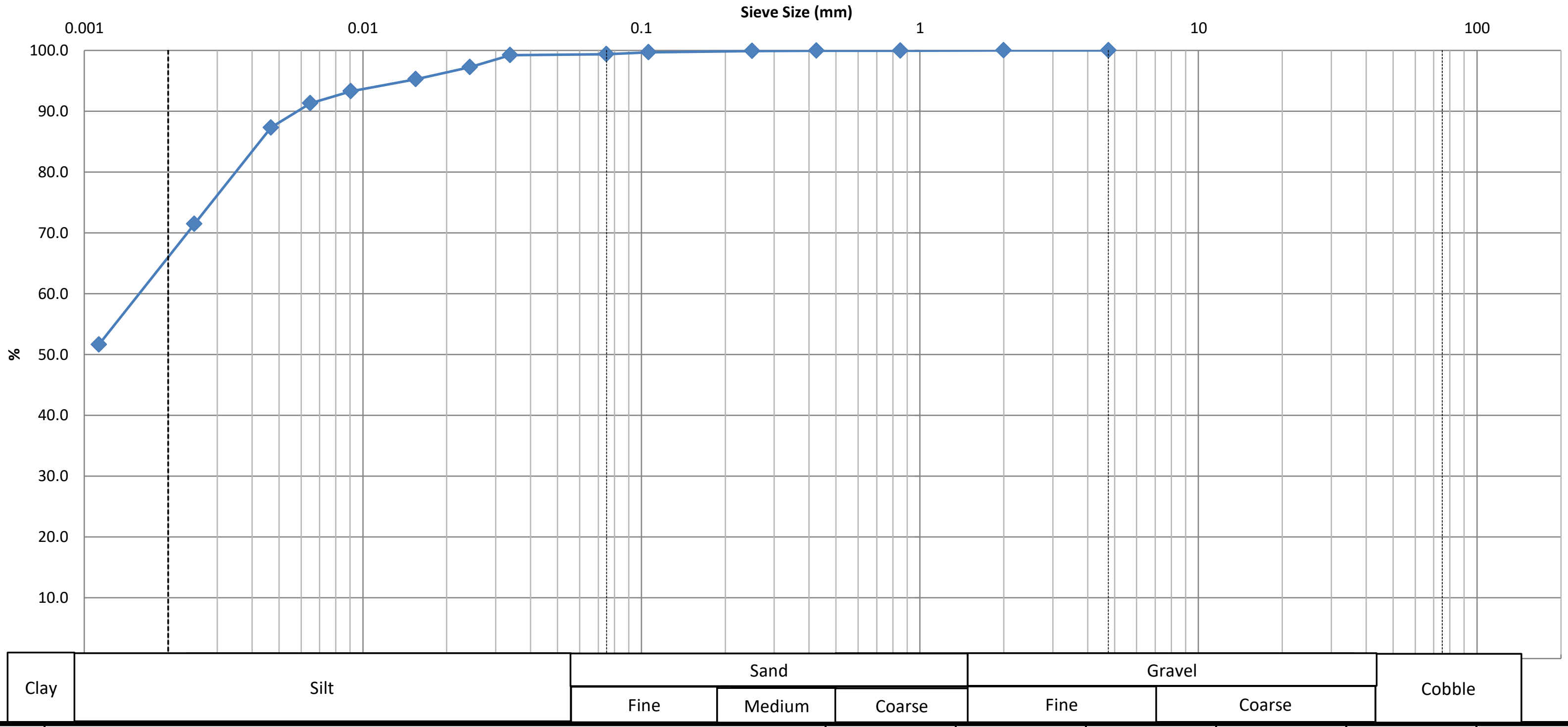
SIEVE ANALYSIS
ASTM C136

CLIENT:	Pinchin	DEPTH:	5'-7'	FILE NO:	PM4184
CONTRACT NO.:		BH OR TP No.:	BH7	LAB NO:	32743
PROJECT:	304017			DATE RECEIVED:	4-May-22
				DATE TESTED:	6-May-22
DATE SAMPLED:	4-May-22			DATE REPORTED:	11-May-22
SAMPLED BY:	Client			TESTED BY:	DK



Identification	Soil Classification	MC(%)	LL	PL	PI	Cc	Cu
		50.0					

	D100	D60	D30	D10	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
					0.0	0.6	33.4	66.0

Comments:

REVIEWED BY:	Curtis Beadow	Joe Forsyth, P. Eng.

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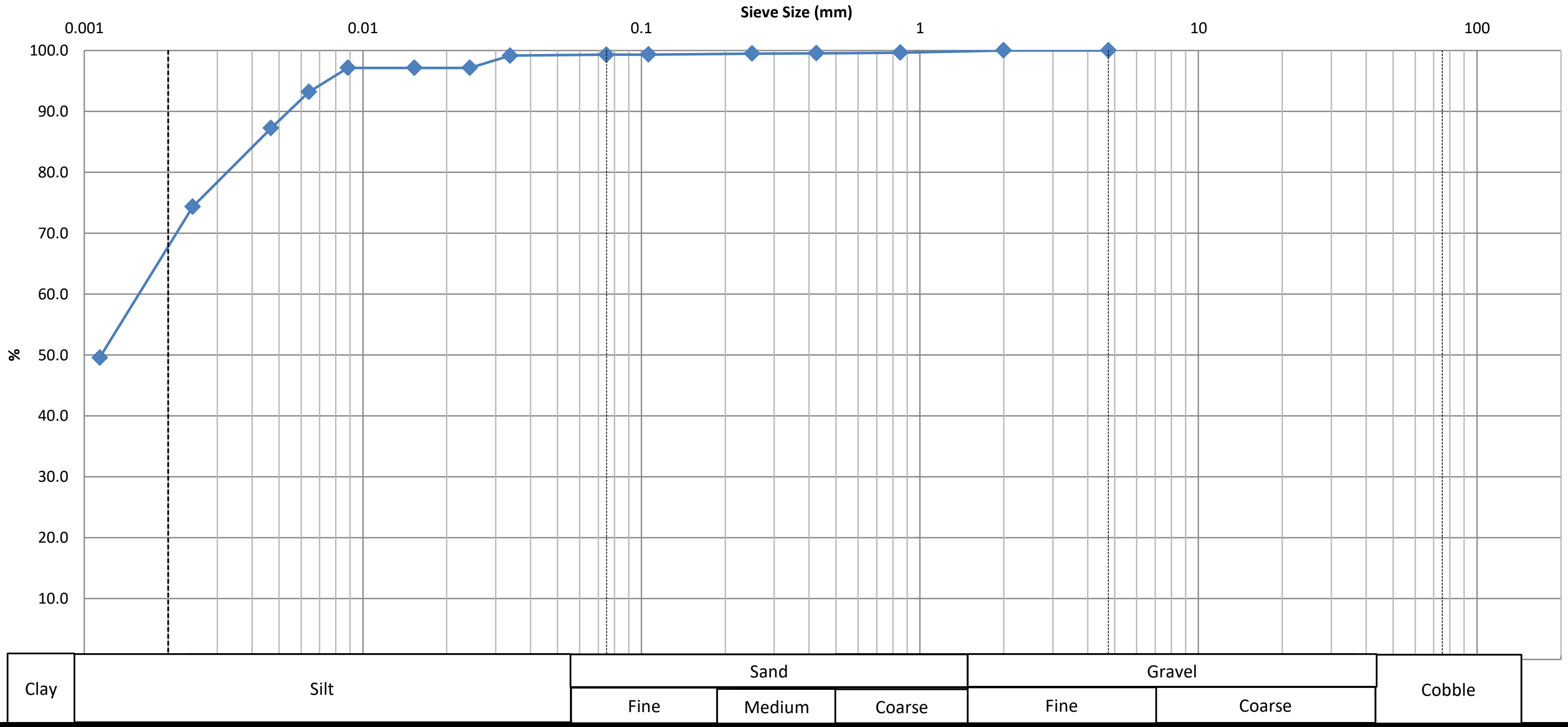
SIEVE ANALYSIS
ASTM C136

CLIENT:	Pinchin	DEPTH:	15' - 17'	FILE NO:	PM4184
CONTRACT NO.:		BH OR TP No.:	BH5	LAB NO:	32744
PROJECT:	304017			DATE RECEIVED:	4-May-22
				DATE TESTED:	6-May-22
DATE SAMPLED:	3-May-22			DATE REPORTED:	12-May-22
SAMPLED BY:	Client			TESTED BY:	DK



Identification	Soil Classification	MC(%)	LL	PL	PI	Cc	Cu
		87.0					

D100	D60	D30	D10	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
				0.0	0.7	30.8	68.5

Comments:

REVIEWED BY:	Curtis Beadow	Joe Forsyth, P. Eng.

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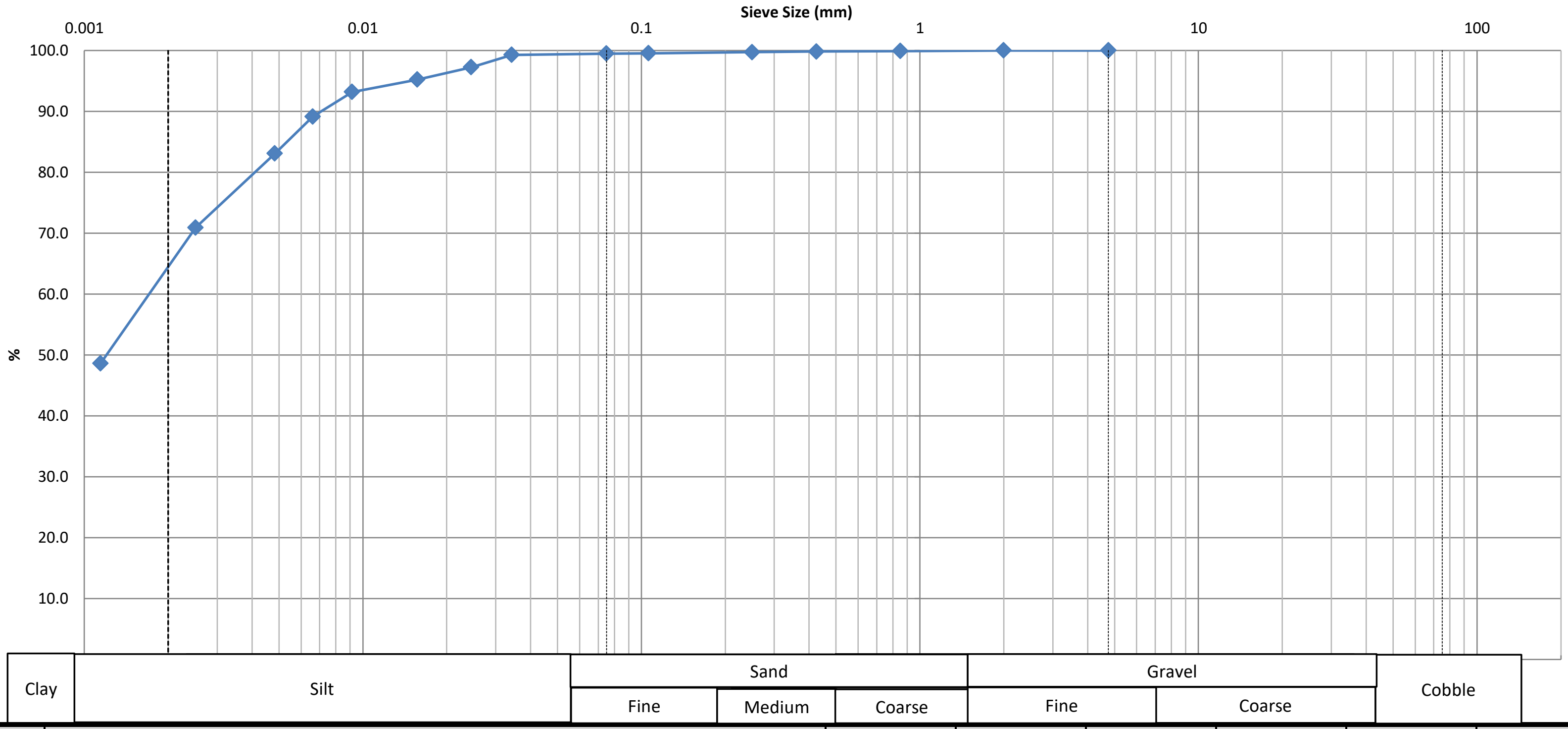
SIEVE ANALYSIS
ASTM C136

CLIENT:	Pinchin	DEPTH:	7.5' - 9.5'	FILE NO:	PM4184
CONTRACT NO.:		BH OR TP No.:	BH1	LAB NO:	32745
PROJECT:	304017			DATE RECEIVED:	4-May-22
				DATE TESTED:	6-May-22
DATE SAMPLED:	3-May-22			DATE REPORTED:	12-May-22
SAMPLED BY:	Client			TESTED BY:	DK



Identification	Soil Classification	MC(%)	LL	PL	PI	Cc	Cu
		65.1					

	D100	D60	D30	D10	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
					0.0	0.5	35.5	64.0

Comments:

REVIEWED BY:	Curtis Beadow	Joe Forsyth, P. Eng.

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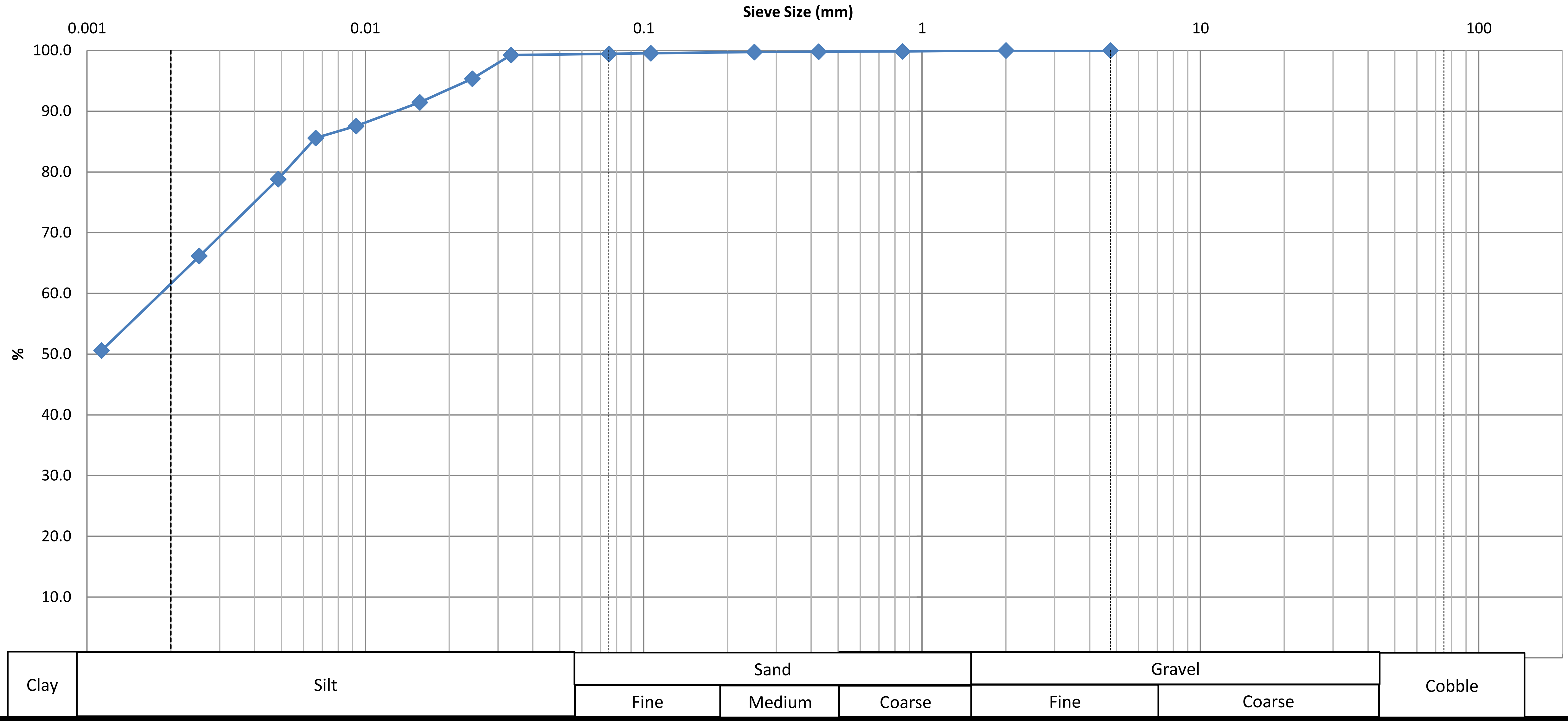
consulting engineers

SIEVE ANALYSIS **ASTM**
C136

CLIENT:	Pinchin	DEPTH:	5.0'-7.0'	FILE NO:	PM4184
CONTRACT NO.:		BH OR TP No.:	BH11	LAB NO:	32741
PROJECT:	304017			DATE RECEIVED:	4-May-22
				DATE TESTED:	20-May-22
DATE SAMPLED:	2-May-22			DATE REPORTED:	27-May-22
SAMPLED BY:	Client			TESTED BY:	DK



Identification	**Soil Classification**				**MC(%)**	**LL**	**PL**	**PI**	**Cc**	**Cu**
					51.1					
	D100	D60	D30	D10	Gravel (%)	Sand (%)	Silt (%)	Clay (%)		
					0.0	0.6	37.9	61.5		

Comments:

REVIEWED BY:	**Curtis Beadow**	**Joe Forsyth, P. Eng.**

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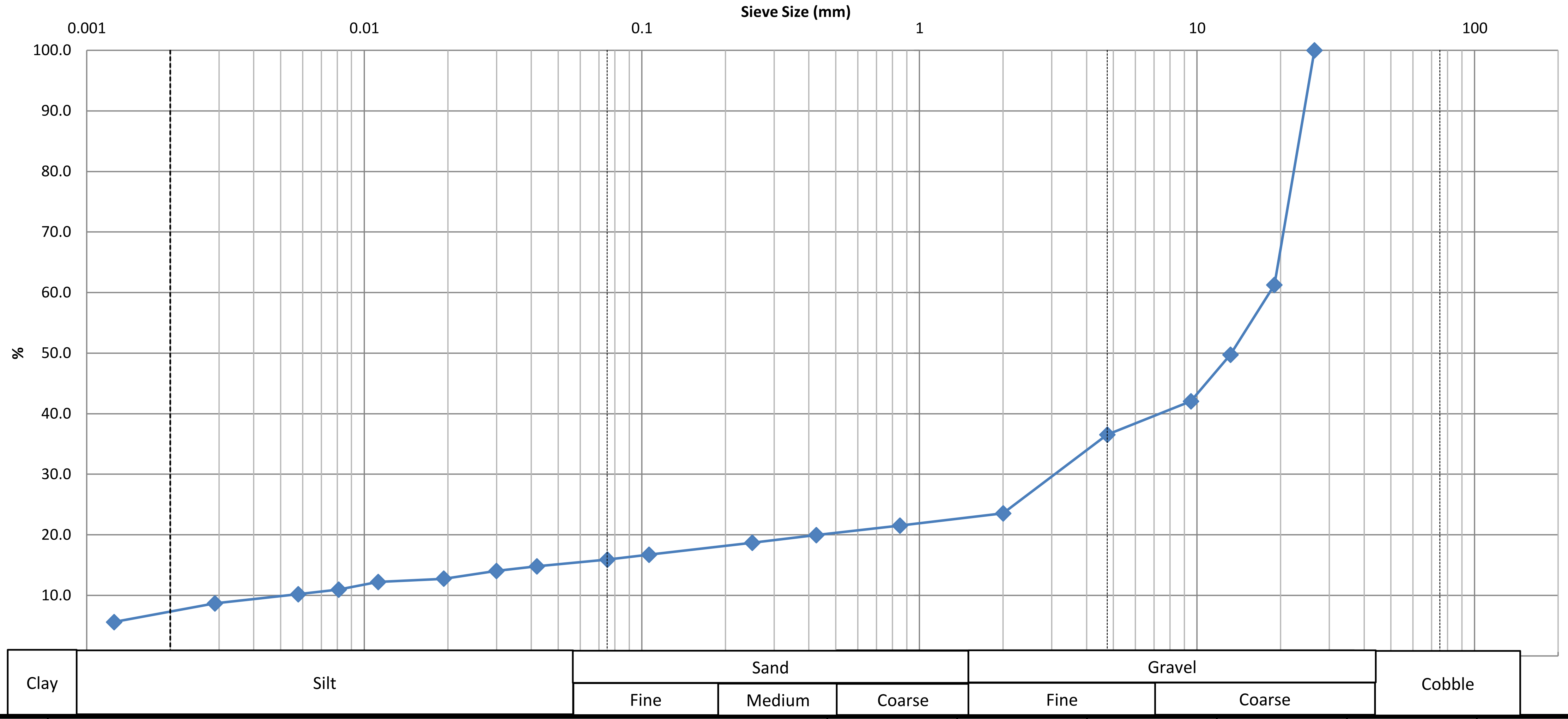
SIEVE ANALYSIS **ASTM C136**

CLIENT:	Pinchin	DEPTH:	20.0'-22.0'	FILE NO:	PM4184
CONTRACT NO.:		BH OR TP No.:	BH2	LAB NO:	32742
PROJECT:	304017			DATE RECEIVED:	4-May-22
				DATE TESTED:	20-May-22
DATE SAMPLED:	2-May-22			DATE REPORTED:	27-May-22
SAMPLED BY:	Client			TESTED BY:	DK



Identification	**Soil Classification**				**MC(%)**	**LL**	**PL**	**PI**	**Cc**	**Cu**
					13.3					
	D100	D60	D30	D10	Gravel (%)	Sand (%)	Silt (%)	Clay (%)		
					63.4	20.6	8.4	7.5		

Comments:

REVIEWED BY:	**Curtis Beadow**	**Joe Forsyth, P. Eng.**

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ATTERBERG LIMITS
LS-703/704

CLIENT:	Pinchin	FILE NO.:	PM4184
PROJECT:	304017	DATE SAMPLED:	4-May-22
LOCATION:	BH7 @ 5.0' - 7.0'	DATE REPORTED:	16-May-22

LIQUID LIMIT DETERMINATION

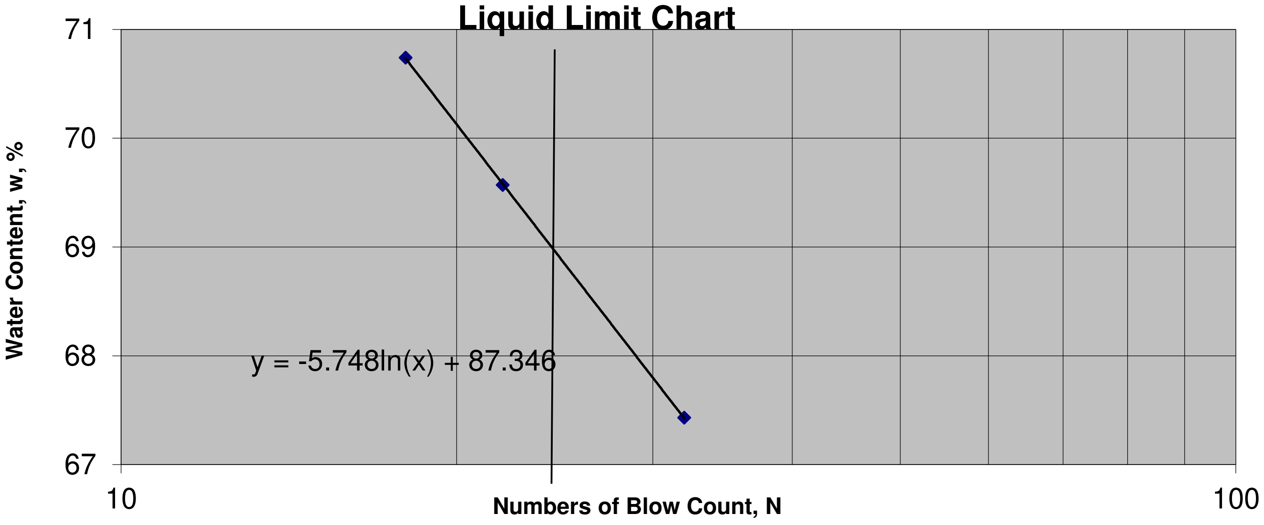
CAN NO.	**16**	**17**	**18**				
WT. OF CAN	8.68	4.39	8.68				
WT. OF SOIL & CAN	16.75	12.19	15.80				
WT. OF DRY SOIL & CAN	13.50	8.99	12.85				
WT. OF MOISTURE	3.25	3.2	2.95				
WT. OF DRY SOIL & CAN	4.82	4.6	4.17				
WATER CONTENT, w, %	**67.43**	**69.57**	**70.74**				
NO. OF BLOWS, N	32	22	18				

PLASTIC LIMIT DETERMINATION

CAN NO.	**1**	**2**
WT. OF CAN	19.86	19.91
WT. OF SOIL & CAN	27.46	27.65
WT. OF DRY SOIL & CAN	25.69	25.83
WT. OF MOISTURE	1.77	1.82
WT. OF DRY SOIL & CAN	5.83	5.92
WATER CONTENT, w, %	**30.36**	**30.74**

RESULTS

LIQUID LIMIT	**69**
PLASTIC LIMIT	**31**
PLASTICITY INDEX	**38**



TECHNICIAN:CS	**REVIEWED BY:**	**C. Beadow**	**J. Forsyth, P. Eng.**

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ATTERBERG LIMITS
LS-703/704

CLIENT:	Pinchin	FILE NO.:	PM4184
PROJECT:	304017	DATE SAMPLED:	4-May-22
LOCATION:	BH5 @ 15.0' - 17.0'	DATE REPORTED:	16-May-22

LIQUID LIMIT DETERMINATION

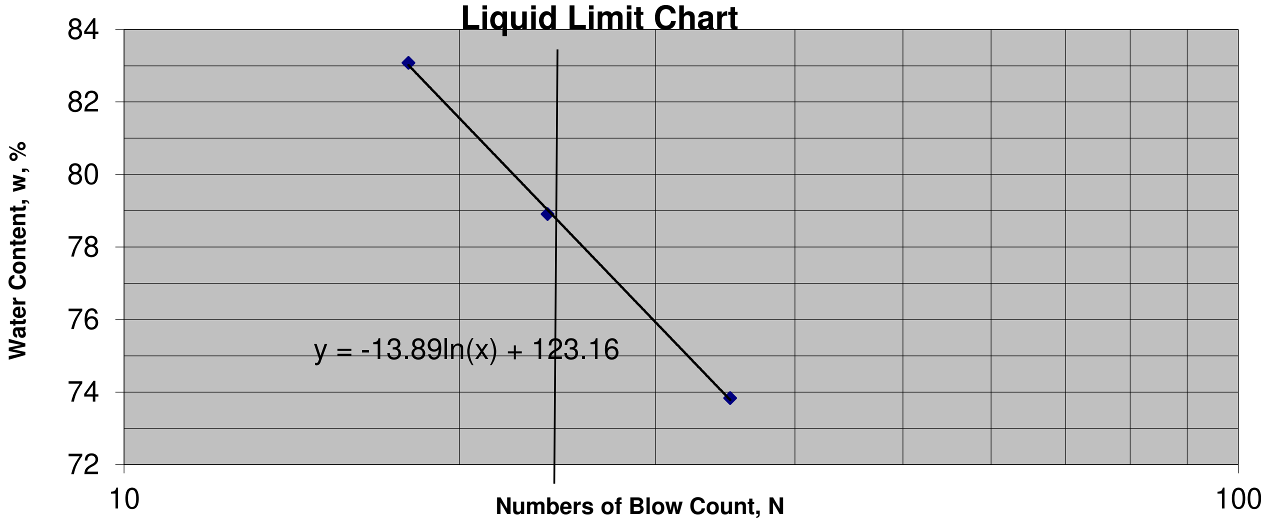
CAN NO.	33	34	35				
WT. OF CAN	4.38	4.37	4.41				
WT. OF SOIL & CAN	11.63	11.92	12.18				
WT. OF DRY SOIL & CAN	8.34	8.59	8.88				
WT. OF MOISTURE	3.29	3.33	3.30				
WT. OF DRY SOIL & CAN	3.96	4.22	4.47				
WATER CONTENT, w, %	**83.08**	**78.91**	**73.83**				
NO. OF BLOWS, N	18	24	35				

PLASTIC LIMIT DETERMINATION

CAN NO.	10	11
WT. OF CAN	19.77	19.97
WT. OF SOIL & CAN	27.51	27.82
WT. OF DRY SOIL & CAN	25.38	25.69
WT. OF MOISTURE	2.13	2.13
WT. OF DRY SOIL & CAN	5.61	5.72
WATER CONTENT, w, %	**37.97**	**37.24**

RESULTS

LIQUID LIMIT	**79**
PLASTIC LIMIT	**38**
PLASTICITY INDEX	**41**



TECHNICIAN:CS	**REVIEWED BY:**	**C. Beadow**	**J. Forsyth, P. Eng.**

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ATTERBERG LIMITS
LS-703/704

CLIENT:	Pinchin	FILE NO.:	PM4184
PROJECT:	304017	DATE SAMPLED:	4-May-22
LOCATION:	BH1 @ 7.5' - 9.5'	DATE REPORTED:	16-May-22

LIQUID LIMIT DETERMINATION

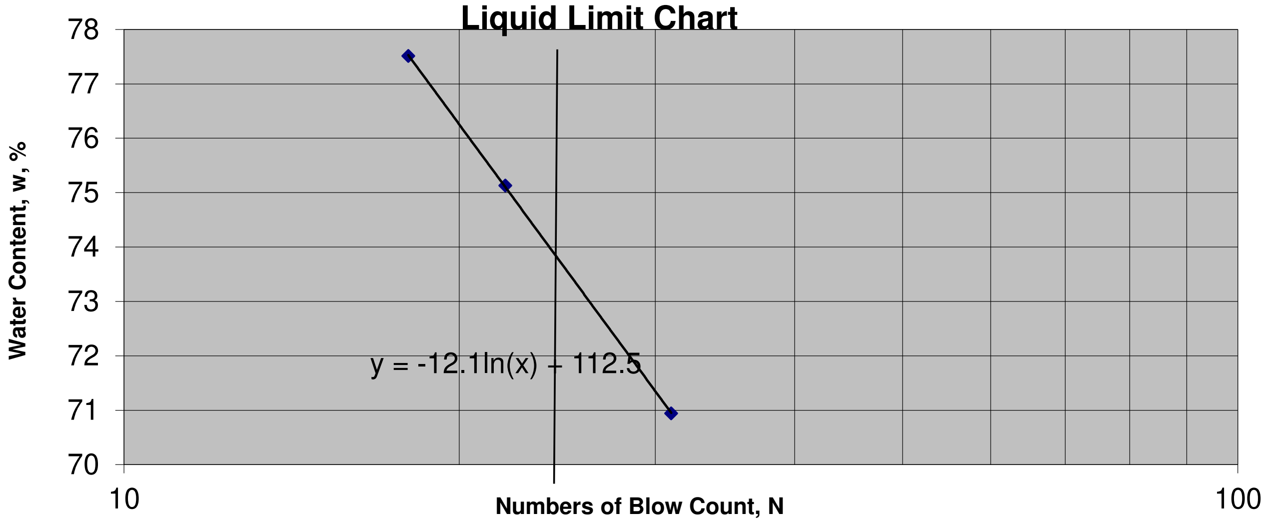
CAN NO.	**16**	**17**	**18**				
WT. OF CAN	8.67	4.39	8.69				
WT. OF SOIL & CAN	15.5	10.94	16.16				
WT. OF DRY SOIL & CAN	12.57	8.08	13.06				
WT. OF MOISTURE	2.93	2.86	3.10				
WT. OF DRY SOIL & CAN	3.9	3.69	4.37				
WATER CONTENT, w, %	**75.13**	**77.51**	**70.94**				
NO. OF BLOWS, N	22	18	31				

PLASTIC LIMIT DETERMINATION

CAN NO.	**1**	**2**
WT. OF CAN	19.85	19.91
WT. OF SOIL & CAN	27.57	27.52
WT. OF DRY SOIL & CAN	25.54	25.53
WT. OF MOISTURE	2.03	1.99
WT. OF DRY SOIL & CAN	5.69	5.62
WATER CONTENT, w, %	**35.68**	**35.41**

RESULTS

LIQUID LIMIT	**75**
PLASTIC LIMIT	**36**
PLASTICITY INDEX	**39**



TECHNICIAN:CS	**REVIEWED BY:**	**C. Beadow**	**J. Forsyth, P. Eng.**

APPENDIX IV

Rock Core Photographs

Photo 1 – Borehole BH101, Rock Core

Photo 2 – Borehole BH102, Rock Core

APPENDIX V

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.