AllRock
Consulting Ltd

Geotechnical Investigation Report

299 West Hunt Club, Ottawa, Ontario

Prepared For:

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Geotechnical Investigation Report

Proposed Parking Garage – 299 West Hunt Club Road

Project No.: 25012 – V.04

September 3, 2025

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1. INTRODUCTION

This report presents the results of a geotechnical investigation carried out for the proposed parking garage located at the Lexus Toyota dealership at 299 West Hunt Club Road in Ottawa, Ontario.

The purpose of the investigation was to identify the general subsurface conditions at the site by means of a limited number of boreholes and, based on the factual information obtained, to provide engineering guidelines on the geotechnical design aspects of the project, including construction considerations that could influence design decisions.

2. BACKGROUND

2.1 Project Description

It is understood that the proposed development includes the following aspects:

- A parking garage in the existing parking lot.
- Based on the current drawings, underside of footing level is proposed at an elevation of 83.50 metres.

2.2 Previous Reports

AllRock has reviewed the existing geotechnical investigation report completed for the existing dealership at 285 West Hunt Club titled: "Geotechnical Investigation, proposed Tony Grahm Lexus Dealership, 285 West Hunt Club Road, Ottawa, Ontario", dated October 16, 2008.

3. SUBSURFACE INVESTIGATION

3.1 Geotechnical Investigation

The field work for this investigation was carried out on the 25th of February 2025. At that time, three (3) boreholes, numbered BH1-25 to BH3-25, were advanced to depth of 8 meters below existing grade.

The borehole locations were selected and positioned on-site by AllRock. The field work was observed throughout by a member of our engineering staff who directed the drilling operations and logged the samples.

Following completion of the boreholes, the soil samples were returned to our laboratory for examination by a geotechnical / materials engineer. Selected samples were submitted for moisture content and grain size distribution testing.

The approximate locations of the boreholes are shown on the Borehole Location Plan, Figure 2. The results of the boreholes are provided on the Record of Boreholes Sheets in Appendix A. The results of the laboratory testing results are provided on the Record of Boreholes Sheets in Appendix B.

3.2 Methodology

Materials and soil description have been made with reference to the following documents:

- Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System) – ASTM D2487-06
- Standard Practice for the Description and Identification of Soils (Visual-Manual Procedure) – ASTM D2488-06

4. SUBSURFACE CONDITIONS

4.1 General

As previously indicated, the soil and groundwater conditions identified in the boreholes are given on the Record of Borehole sheets in Appendix A. The logs indicate the subsurface conditions at the specific test locations only. Boundaries between zones on the logs are often not distinct but rather are transitional and have been interpreted. The precision with which subsurface conditions are indicated depends on the method of exploration, the frequency and recovery of samples, the method of sampling, and the uniformity of the subsurface conditions. Subsurface conditions at other than the borehole locations may vary from the conditions encountered in the boreholes.

The soil descriptions in this report are based on commonly accepted methods of classification and identification employed in geotechnical practice. Classification and identification of soil involves judgement and AllRock does not guarantee descriptions as exact but infers accuracy to the extent that is common in current geotechnical practice.

The groundwater conditions described in this report refer only to those observed at the place and time of observation noted in the report. It is noted that groundwater conditions can vary seasonally or as a result of construction activities in the area.

4.2 Subsurface Conditions

The following presents an overview of the subsurface conditions encountered in the borehole investigation

4.2.1 Asphalt Pavement

As all the boreholes were advanced in an existing parking lot, a layer of asphalt was encountered at all locations. The asphalt was found to have a thickness of approximately 0.15 – 0.25 meters.

4.2.2 Fill Material

A natural fill layer was encountered at all borehole locations below the surficial asphalt. Fill material can be an assortment of grain size and textures. At this site, the fill can be described as a brown, medium grained, and medium dense silty sand. The layer extended to a depth of 4.5 meters below ground surface at all borehole locations

4.2.3 Silty Clay

Below the natural fill, a native silty clay layer was encountered at all borehole locations. The layer was described as grey, soft, medium plasticity, inorganic and very moist. The layer extended to a depth of approximately 6 meters below ground surface at all borehole locations.

Standard penetration tests carried out in the native silty sand gave N values ranging from 0 (weight of hammer) to 25 blows per 0.3 metres of penetration, which reflects a very loose to dense relative consistency.

4.2.4 Silty Sand

Underlaying the clay, a silty sand layer was encounter at all borehole locations. The layer was described as brownish/grey fine grained, and medium dense. The layer extended to the termination depth of 8 meters below ground surface at all borehole locations.

Standard penetration tests carried out in the native silty sand gave N values ranging from 50 to 7 blows per 0.3 metres of penetration, which reflects a medium to dense relative consistency.

4.2.5 Gradation Analysis and Moisture Content

Table 4.1: Gradation Analysis & Moisture Content

Location	Sample Number	Sample Depth (ft)	Test Type	Gravel (%)	Sand (%)	Silt (%)	Clay (%)	Moisture Content (%)
BH2-25	SS9	25– 27	Grain	0.0	65.6	34.4		9.9
BH3-25	SS5	12.5 – 14.5	Grain	0.0	11.6	88.4		17.0

4.2.6 Groundwater Level

A return trip to site to measure water levels was conducted on March 20th, 2025. The measure depth was 6.0 meters below ground surface.

5. RECOMMENDATIONS AND GUIDELINES

5.1 General

The information in the following sections is provided for the guidance of the design engineers and is intended for the design of this project only. Contractors bidding on or undertaking the works should examine the factual results of the investigation, satisfy themselves as to the adequacy of the information for construction, and make their own interpretation of the factual data as it affects their construction techniques, schedule, safety and equipment capabilities.

The professional services retained for this project include only the geotechnical aspects of the subsurface conditions.

The National Building Code of Canada 2020 Guidelines (hereafter NBCC 2020), the 2012 Ontario Building Code (OBC 2012) and the 4th edition of the Canadian Foundation Engineering Manual, 2006 (hereafter CFEM 2023) were considered for these recommendations. Based on the collected information from the boreholes advanced as part of this investigation, the geotechnical recommendations are presented in the following sections.

5.2 Proposed Site Development

5.2.1 Excavation

The excavation for the proposed building will be carried out through asphalt, fill material, silty sand and silty clay. The sides of the excavation should be sloped in accordance with the requirements in Ontario Regulation 213/91 under the Occupational Health and Safety Act. According to the act, soils at this site can be classified as Type 3. That is, open cut excavations within overburden deposits should be carried out with side slopes of 1 horizontal to 1 vertical, or flatter. Where excavation side slopes cannot be accommodated due to space constraints, a shoring system may be required. Additional guidelines for the design and selection of a suitable shoring system could be provided as the design progresses.

In the event that a granular pad is necessary below the foundations, the excavations should be sized to accommodate a pad of imported granular material which extends at least 0.6 to 1 metres horizontally beyond the edge of the footings and down and out from this point at 1 horizontal to 1 vertical, or flatter.

Depending on construction methodology, it may be necessary to the lower the groundwater level in the native deposits to about 0.3 metres below the base of the excavation. Below the groundwater level, sloughing of the sandy overburden soils into the excavation should be anticipated, along with disturbance to the soils in the bottom of the excavation. Sloughing of the excavation side slopes below the groundwater level could be reduced, where necessary, by a shoring system installed along the sides of the excavation to below the level of the excavation in combination with pumping from within the excavation.

5.2.2 Groundwater and Pumping Management

Groundwater inflow, from the overburden deposits should be controlled by pumping from filtered sumps within the excavation. It is not expected that short term pumping during excavation will have a significant effect on nearby structures and services. It is anticipated that groundwater inflow from the overburden deposits into the excavations could be handled from within the excavations.

It is noted that groundwater levels and surface water flows can increase during wet periods of the year such as the early spring or following periods of precipitation.

The groundwater handling should be carried out in accordance with provincial and local regulations. Suitable detention and filtration will be required before discharging water. The contractor should be required to submit an excavation and groundwater management plan for review.

Depending on the depth of proposed foundations and groundwater level at the time of construction, an Environmental Activity and Sector Registry (EASR) in accordance with Environmental Protection Act Part II or a Category 3 Permit to Take Water may be required.

5.2.3 Grade Raise Restriction

The site is underlain by a silty clay deposit, which has a limited capacity to support additional loads from grade raise fill and, to a lesser extent, from the proposed building foundations. The placement of fill must be carefully controlled to ensure that the imposed stresses do not result in excessive consolidation of the underlying silty clay. The settlement response of this deposit to increased loading and potential groundwater lowering is influenced by several factors, including:

- Existing effective overburden pressure
- Preconsolidation pressure of the silty clay
- Compressibility characteristics of the silty clay
- Availability of drainage paths

It is well established that significant settlement can occur when the applied stress approaches the difference between the preconsolidation pressure (Pc) and the existing effective overburden stress (σ'_{vo}). Based on vane shear strength testing in the boreholes, the maximum permissible grade raise in the building area should be limited to 0.5 metres above the existing ground surface. If the planned grade raise exceeds this value, additional settlement analysis will be required.

This restriction has been calculated to limit long-term settlement to approximately 25 millimetres. The following assumptions have been applied in establishing the grade raise restrictions:

- Groundwater lowering associated with the development will not exceed 1 metre. To reduce the risk of drawdown, seepage barriers should be installed along service trenches.
- The unit weight of the grade raise material adjacent to the structures will not exceed 22 kN/m^3.

5.2.4 Subgrade Preparation and Placement of Engineered Fill

Any existing topsoil, organic material, fill, and/or weathered/disturbed soil should be removed from below the proposed structures.

Imported granular material (engineered fill) should be used to raise the grade in areas where the proposed founding level is above the level of the native soil, or where sub-excavation of material is required below proposed founding level. The engineered fill should consist of granular material meeting Ontario Provincial Standard Specifications (OPSS) requirements for Granular B Type II and should be compacted in maximum 200-millimetre-thick lifts to at least 99 percent of the standard Proctor maximum dry density. To allow spread of load beneath the footings, the engineered fill should extend horizontally at least 0.6 metres beyond the footings and then down and out from the edges of the footings at 1 horizontal to 1 vertical, or flatter. The excavations should be sized to accommodate this fill placement.

It is noted that engineered fill in excess of 1 metre thick can be expected to experience post-construction settlement in the order of 0.5 to 1 precent of the height of the soil placed (depending on the composition of the engineered fill). It is anticipated that if engineered soil is sourced from the native onsite soils, it may take 2 to 4 months for the majority of post-construction settlement to occur; however, if imported granular fill as such as that meeting the (OPSS) requirements for Granular B Type II, settlement will likely occur within 1 to 2 weeks of placement.

5.2.5 Footing Design

In general, the silty clay and the native silty sand are considered suitable to support the proposed structures founded on spread footings The existing topsoil/organic material and fill material are not considered suitable for the support of the proposed development and should be removed from the proposed development areas.

For preliminary design purposes, footings founded on the silty clay or on a pad of compacted engineered fill above the silty clay layer (depth of 4.6 meters below ground surface) should be sized using a geotechnical reaction at Serviceability Limit State (SLS) of 75 kilopascals and a factored geotechnical resistance at Ultimate Limit State (ULS) of 112.5 kilopascals.

Alternatively, footings founded on the native silty sand or on a pad of compacted engineered fill above the silty sand layer (depth of 6.1 meters below ground surface) should be sized using a geotechnical reaction at Serviceability Limit State (SLS) of 100 kilopascals and a factored geotechnical resistance at Ultimate Limit State (ULS) of 150 kilopascals.

The post construction total and differential settlement of footings should be less than 25 and 15 millimetres respectively, provided that all loose or disturbed soil is removed from the bearing surface and provided that any engineered fill material is compacted to the required density.

5.3 Alternative methods

Due to the proposed USF elevation of 83.5 metres and thickness of fill material and depth to native soil (4.6 meters below existing grade) the excavation will be sufficiently deep and require shoring. As an alternative, other foundation options as provided in the sections below such as raft slab foundation, caissons, helical piles and driven piles. It is noted that if deep foundations are being considered, a supplemental geotechnical investigation will likely be required to advance deeper boreholes.

5.3.1 Basement Raft Slab Foundation

It is assumed that the top of the raft slab will be at elevation 83.5 metres. To achieve a sufficient bearing capacity for the raft slab, the following is recommended:

- Excavate to the top of the grey silty clay (as per the borehole logs, average of 82.5 metres).
- Place a mud mat directly on the silty clay subgrade or alternatively, place a Class II woven geotextile directly on the grey silty clay subgrade overlaid by a triaxial geogrid (Terrafix TTX7 or similar.
- Raise the grade up to the underside of the raft slab with engineered granular material meeting Ontario Provincial Standard Specifications (OPSS) requirements for Granular B Type II compacted in maximum 200-millimetre-thick lifts to at least 99 percent of the standard Proctor maximum dry density as per Section 5.2.4. The granular pad should extend at least 1 metre beyond the edge of the raft slab.

At the time of preparation of this report, raft slab details (thickness and loading) were not available; however, the following bearing values can be used provided the subgrade is prepared as described in this section.

- Gross geotechnical reaction at SLS: 80 kilopascals
- Factored geotechnical resistance at ULS: 120 kilopascals

The total and differential settlement of the foundation at SLS should be less than 25 and 15 millimetres, respectively. The SLS reaction does not take into consideration the weight of the raft. The bearing values provided above assume that the exterior finished grade around the raft slab is not raised by more than 0.5 metres.

It is noted that grey silty clay may be encountered at subgrade level. These deposits are very sensitive to disturbance from ponded water and construction traffic. To avoid subgrade disturbance, we recommend that an allowance be made for a 50 to 75 millimetre thick mud mat of low strength concrete. The mud mat should be placed over the silty clay subgrade surface immediately after exposure and inspection. Alternatively, a Class II woven geotextile could be placed on the directly on the grey silty clay subgrade overlaid by a triaxial geogrid (Terrafix TTX7 or similar).

A subgrade modulus of about 3 kilopascals per millimetre could be used for the subgrade.

5.3.2 Caissons (Augered Piers)

If the bearing capacities provided with the spread footings are not suitable, caisson foundations can be used as an alternative.

For the design of foundations, the passive resistance within the upper 1.8 metres below ground surface should be neglected to account for frost action. The unfactored lateral resistance should be calculated assuming an equivalent width equal to three times the caisson diameter. A resistance factor of 0.5 should be applied to the unfactored lateral resistance to obtain the factored lateral geotechnical resistance at Ultimate Limit State (ULS).

In the case of cohesive soils, the capacity of the caisson should be checked to determine where the drained or undrained case will govern. In this case, the lateral resistance for the length of the caisson within the cohesive soil should be calculated assuming an unfactored passive lateral pressure distribution varying from 2 Cu at the surface to 9 Cu at and below a depth equivalent to three caisson diameters, acting over the actual width of the caisson. A resistance factor of 0.5 should be applied to this calculated lateral resistance in order to obtain the factored lateral geotechnical resistance at ULS.

The factored unit soil weight should be used below the groundwater level, where applicable.
For design the full passive resistance will be mobilized only where the width of soil in front and behind the caissons is equal to or greater than eight caisson diameters. If there is lesser width of soil from development of passive resistance (i.e., if there is sloping ground adjacent to the culvert), the magnitude of the passive resistance may be determined by interpolating between zero passive resistance at ground surface and full passive resistance at the depth where the slope face is greater than eight caisson diameter way from the face of the caisson.

Where caissons will be installed below the groundwater table or in loose non-cohesive soils, the caissons should be installed inside temporary steel liners driven ahead of the drill head to prevent fill and soft soils in the caisson holes to become unstable.

The founding soils could be susceptible to disturbance by augering; therefore, the bases of the augered caisson should inspected by geotechnical personnel to confirm they are located in native, undisturbed and competent bearing soils which has been cleaned of any ponded water and loosened materials prior to pouring concrete. Concrete for the caisson should be poured as soon as practicable after augering. The bearing soils and fresh concrete must be kept from freezing during cold weather construction.

The ultimate shaft uplift resistances of a caisson within non-cohesive soils can be determined from the following expression:

$$Q_s = 0.5\pi D \sum_{z=0}^{L} \beta\sigma\Delta z$$

Where:

- Q_s = ultimate uplift shaft capacity (kN)
- D = diameter of caisson (m)
- Δz = thickness of soil layer (m)
- z = depth (m)
- L = length of caisson (m)
- β = shaft resistance factor (0.2 from fill, 0.3 for native soils)
- σ = vertical effective stress adjacent to the pile at depth z

The uplifting resistance in the upper 1.8 metres should be neglected.

A resistance factor of 0.3 should be applied to the calculated uplift resistance in order to obtain the factored shaft uplift geotechnical resistance at ULS. The weight of the concrete caisson can be assumed as 14 kN/m3 below the groundwater table. An appropriate factor of safety should be utilized in the structural analysis of uplift resistances of caissons.

The axial (compressive) loading for caisson should be relatively small compared to the lateral and uplift loads and it is anticipated that the foundation design will be governed by lateral loading and uplift resistance. Cave-in should be anticipated in non-cohesive soils and below the groundwater level. Based on the size of the caisson, proper cleanup of the caisson bottom may not be practical. As a result, axial bearing resistance is mainly mobilized from shaft resistances of caissons.

A resistance factor of 0.4 should be applied to the calculated axial bearing resistance in order to obtain the factored axial geotechnical bearing resistance ULS. An appropriate factor of safety should be utilized in the structural analysis.

5.3.3 Helical Piles

It is understood that the proposed foundation design may use helical piles.

The depth of penetration and required design of helices (single or multiple) will depend on the soil conditions and design loads. The shaft diameters, wall thicknesses and welds need to be designed by a structural engineer to meet the required installation stresses and the expected geotechnical conditions.

It should be noted that helical piles are a proprietary foundation system and the helical pile resistances are highly dependent on the pile design geometry and method of installation. It is therefore, generally accepted industry practice that the Piling Contractor designs and warrants the helical piles for the specified ULS design loads. Varying helix diameters and configurations may be required based on the loading requirements. Where installed in groups, helical piles should not be installed at spacing closer than three times the largest helix diameter, centre to centre.

The ultimate capacity of the screw pile (Q_{ult}) with a single helix in native silty sand may be expressed as follows:

$Q_b = [(N_C \times C_u) + (\gamma' \times H)] \times [\pi \times (D_2 - d_2)/4]$

Where:

- N_c = bearing capacity factor
- Use 9 for D < 0.5 m.
- Use 7 for D between 0.5 to 0.9 m

- Use 6 for D > 0.9 m
- C_u = undrained shear strength (kPa) at depth of the helix plate
- D = helix plate diameter
- d = pile shaft diameter (where applicable)
- H = depth of helix below ground
- γ' = Effective unit weight (use 19 kN/m^3 above the water table, 10 kN/m^3 below
- the water table).

A resistance factor of 0.4 (compression) and 0.3 (tension) may be used to determine the factored ULS bearing resistance of the screw pile helix, Shaft friction should generally be ignored for small diameter shafts due to potential effects of disturbance and loss of shaft adhesion.

The undrained shear strength (s_u) of the soil up to the depth of investigation can be taken as 75kPa.

Piles should be founded with the upper helix at least one metre below the design frost depth which was given as 1.5 metres at this site. The pile designer should refer to the borehole logs and use the appropriate soil strength parameters for design. As noted above the piling contractor remains responsible for selection of appropriate pile design parameters and for design and installation of the piles.

It should be recognized that screw pile capacities are highly dependent on the pile design and method of installation. The installation method used to install helical piles can also cause significant soil disturbance (due to churning) and/or the development of voids around the pile shaft near the ground surface. The potential for disturbance may increase with multiple helix piles. This can have a significant impact on the lateral load deformation behavior of helical piles since good soil support in the upper few meters is critical for lateral support. Any voids formed around the pile shaft should be backfilled with sand or crushed gravel to maintain intimate contact between the pile shaft and the soil.

Installation of helical piles should be monitored by a geotechnical engineer, and the final torques should be recorded and used as a method of confirming the pile capacities.

Uplift loads can be resisted by the pile shaft and helices. Piles resisting uplift load should be installed at a minimum depth ratio (H/D) of 4, or at least 1 m below the frost depth of 1.8 metres, whichever is greater. The ultimate axial resistance should be multiplied by a GRF of 0.3 for piles subject to uplift loads. The upper 1.8 m of the pile shaft should be neglected when calculating the uplift resistance.

Screw piles should also be checked for frost uplift. An ultimate adhesion of 10 kPa may be used for a bare steel shaft within the frost depth. The resistance to frost action is provided by the helices and hence these need to be founded below the frost depth, as suggested above. A resistance factor of 0.8 may be applied to the ultimate helix capacity in resisting frost heave uplift forces. It is recommended that the final screw pile design be reviewed by a geotechnical engineer. In addition, the structural capacity should be checked for the applied loading conditions.

Precautions should be taken to prevent heaving of the structure foundation due to frost penetration or seasonal moisture variation or swelling of the underlying soil. Adfreeze forces on the sides of pile and/or pile caps exposed to freezing should also be accounted for in foundation design.

5.3.4 Driven Piles (H-Piles)

As an alternative to helical piles, driven piles (H piles) could be used. Table 5.1 presents the pile geotechnical design parameters. Groundwater conditions were discussed previously in this report. The design of driven should follow the approach recommended in Chapter 18 of the Canadian Foundation Engineering Manual (CFEM 2023).

The structural engineer should review and adopt suitable geotechnical resistance factors for pile design as per the recommendations in Table 5.2 and project site conditions. Most of the factors in Table 5.2 are based on recommendations in Table 8.1 from CFEM 2023.

The piles will be subject to uplift forces due to frost heave, tensile force due to lateral loading and overturning moments, The piles should be designed to resist these uplift forces, The resistance to uplift will be provided by pile self-weight, applied dead loads and uplift skin resistance. Adfreeze forces on the sides of pile and/or pile caps exposed to freezing should also be accounted for in foundation design.

Table 5.1: – Pile Design Parameters

Depth Below Ground Surace (m)	Total / Effective Unit Weight, γ' (kN/m^3)	Effective Friction Angle, ϕ (degree)	Undrained Shear Strength, S_u (kPa)
0 - 4.6	18	30	-
4.6 – 6.1	17.5	33	50
6.1 – 7.6	18.5	32	-

Table 5.2: – Geotechnical Resistance Factors for Axial Resistance

Description	Axial Resistance Factor for Driven or Bored Piles
Compression: Analysis with site data	0.4
Compression: Analysis with loading test results	0.6[1]
Uplift Analysis with site data	0.3
Uplift: Analysis with loading test results	0.4[1]

Notes:

1. The higher geotechnical resistance values can be used if the design utilizes a static pre-construction load testing program, a field verified program and a field monitoring and supervising QA/QC program to ensure construction quality.

Table 5.3: – Group-reduction Factor for Lateral Pile Response

Pile Spacing	Reduction Factor of Subgrade Reaction Modulus
S=8d	1
S=6d	0.7
S=4d	0.4
S=3d	0.25

Notes: “s” is pile spacing and “d” is pile diameter.

Table 5.4: – Pile Foundation Design Data

End Bearing Strata	Pile Outside Diameter (mm)	Pile Wall Thickness (mm)	Geotechnical Axial Resistance - SLS/ULS (kN)	Final Set (blows/ 12mm)	Transferred Hammer Energy (kJ)
Compact to Dense Sand (L= 6 m	245	9 11 13	75 / 90 94 / 113 110 / 132	8-10 8-10 8-10	25 27 30
Rock (L =28.8 m, bedrock depth[2])	245	9 11 13	850 / 1020 1063 / 1276 1244 / 1493	10 10 10	29 35 42

Notes:
1. (FS = 2.5)
2. Bedrock depth and type from Houle Chevrier Report (2008). It is noted that AllRock has not confirmed bedrock depth, type or quality as part of this investigation and it is recommended that a supplemental investigation be carried out to confirm pile design.

5.3.4.2 Pile Settlements

The settlement of a single pile can be estimated using elastic theory and estimated soil compressibility parameters. Pile groups spaced 3 pile diameters apart should not settle more than 25 mm service loads.

5.3.4.3 Foundation Settlements

It is noted that the proposed building will be located approximately 15 metres from the adjacent rail line. From a geotechnical standpoint, vibrations generated by rail traffic are expected to dissipate rapidly with distance and are not anticipated to induce additional soil settlement at the proposed building location. Furthermore, it is assumed that municipal development setback requirements would have likely accounted for these considerations.

5.3.5 Foundation Wall Backfill and Drainage

The foundation walls should be damp proofed and backfilled with imported, free draining, non-frost susceptible granular material such as that meeting OPSS Granular B Type I or II requirements.

Where the backfill will ultimately support areas of hard surfacing (pavement, sidewalks or other similar surfaces), the backfill should be placed in maximum 200 millimetre thick lifts and should be compacted to at least 98 percent of the standard Proctor maximum dry density value using suitable compaction equipment. Where future landscaped areas will exist next to the proposed buildings and if some settlement of the backfill is acceptable, the backfill could be compacted to at least 90 percent of the standard Proctor maximum dry density value.

Where areas of hard surfacing (concrete, sidewalks, pavement, etc.) abut the proposed structures, a gradual transition should be provided between those areas of hard surfacing underlain by non-frost susceptible granular wall backfill and those areas underlain by existing frost susceptible fill material to reduce the effects of differential frost heaving. It is suggested that granular frost tapers be constructed from 1.5 metres below finished grade (or the bedrock surface) to the underside of the granular subbase for the hard surfaced areas. The frost tapers should be sloped at 1 horizontal to 1 vertical, or flatter.

As a minimum, a perforated plastic foundation drain with a surround of clear crushed stone should be installed on the exterior of the foundation walls. The drains should outlet by gravity to a storm sewer or a sump from which the water is pumped. To avoid loss of sand backfill into the voids in the clear stone (and possible post construction settlement of the ground around the building/wall), a nonwoven geotextile should be placed between the clear stone and any sand backfill material.

5.3.6 Basement Raft Slab Support

To provide predictable settlement performance of the basement raft slab, any disturbed soil, organic material, or deleterious material should be removed to expose the native, undisturbed soil deposits.

It is noted that grey silty clay may be encountered at subgrade level. These deposits are very sensitive to disturbance from ponded water and construction traffic. To avoid subgrade disturbance, we recommend that an allowance be made for a 50 to 75 millimetre thick mud mat of low strength concrete. The mud mat should be placed over the silty clay subgrade surface immediately after exposure and inspection. Alternatively, a Class II woven geotextile could be placed on the directly on the grey silty clay subgrade overlaid by a triaxial geogrid (Terrafix TTX7 or similar).

The raft slab foundation could be founded directly on the native soil (or on a 50 to 75 millimetre thick mud mat above the native soil). It is pointed out that raft slab foundation will be prone to moisture seepage. As such, consideration could be given to placing a drainage layer on the surface of the raft slab that is overlain by a second concrete floor over the raft slab. Any seepage or infiltration through the raft slab could be collected in the drainage layer, and drained by gravity to a sump from which water is pumped.

Proper moisture protection with a vapour retarder should be used for any slab where the floor will be covered by moisture sensitive flooring material or where moisture sensitive equipment, products or environments will exist. The “Guide for Concrete Floor and Slab Construction”, ACI

302.1R-04 should be considered for the design and construction of vapour retarders below the floor slab.

If any areas of the building are to remain unheated during the winter period, thermal protection of the slab on grade may be required. Further details on the insulation requirements could be provided, if necessary.

5.3.7 Frost Protection of Foundations

All exterior footings for heated buildings that consist of slab on grade construction or included basement should be provided with at least 1.5 metres of earth cover for frost protection purposes. Isolated, unheated and/or exterior pier footings adjacent to surfaces which are cleaned of snow cover during the winter months should be provided with a minimum of 1.8 metres of earth cover. Alternatively, the required frost protection could be provided by means of a combination of earth cover and extruded polystyrene insulation. Further details regarding the insulation of foundations could be provided at the detailed design stage, if necessary.

5.3.8 Seismic Site Classification

According to Table 4.1.8.4.A of the NBCC 2020, Site Class D should be used for the seismic design of the structures bearing on native soils or on engineered fill material over native soils.

5.3.9 Lateral Earth Pressures

The static “At Rest” thrust (Po) acting on the walls should be calculated using the following formula:

$Po = 0.5\ Ko\ \gamma\ H^2$

where;

- Po: Static at rest thrust component (kN/m);
- γ: Moist material unit weight (kN/ m^3);
- Ko: “At Rest” earth pressure coefficient;
- H: Wall height (m).

Seismic shaking can increase the forces on the retaining walls. The total “At Rest” thrust acting on the wall (Poe) during a seismic event should be calculated using the following formula:

$P_{oe} = 0.5\ Koe\ \gamma\ H^2$

where;

- P_{oe}: Total “At rest” thrust (kN/m);
- γ: Moist material unit weight (kN/m^3);
- Ko: “At Rest” earth pressure coefficient;

- Koe: Dynamic at rest earth pressure coefficient;
- H: Wall height (m).

The static thrust component (Po) acts at a point located H/3 above the base of the walls. During seismic shaking, the total “At Rest” thrust (Poe) acts at a point located about H/2 above the base of the wall. It should be noted that the total “At Rest” thrust, Poe, is composed of a static component and a dynamic component.

For design purposes, the parameters provided in Table 5.5 can be used to calculate the thrust acting on the wall during static and seismic loading conditions.

Table 5.5: - Summary of Design Parameters (Building Foundation)

Parameter	OPSS Granular B Type II
Material Unit Weight, γ (kN/m^3)	22
Estimated Friction Angle (degrees)	38
“At Rest” Earth Pressure Coefficient K_o, assuming horizontal backfill behind the structure	0.38
Dynamic “At Rest” Earth Pressure Coefficient K_{oe}, assuming horizontal backfill behind the structure	0.52

According to the 2024 Ontario Building Code, the peak ground acceleration (PGA) for the site is 0.35 for firm ground conditions (i.e., for Site Class C) and has been correct to 0.40 for Site Class D. The dynamic at rest earth pressure coefficient was calculated using the method suggested by Mononobe and Okabe, assuming a horizontal coefficient k_h of 0.37 (taken as the PGA) and assuming that the vertical seismic coefficient k_v. is zero.

Heavy construction traffic should not be allowed to operate adjacent to the basement foundation walls for the proposed building (within about 2 metres horizontal) during construction, without the approval of the designers.

5.4 Site Services

5.4.1 Excavation

Based on the investigation, the excavations for the services within the site will be carried out through asphalt, sub-base course and silty sand.

The sides of the excavations within overburden soils should be sloped in accordance with the requirements in Ontario Regulation 213/91 under the Occupational Health and Safety Act.

According to the Act, the soils at this site can be classified as Type 3 soils. Therefore, for design purposes, allowance should be made for 1 horizontal to 1 vertical, or flatter, excavation slopes within the native soils at this site. As an alternative to sloping the excavations, all services installations could be carried out within a tightly fitting, braced steel trench box, which is specifically designed for this purpose.

The groundwater inflow should be controlled throughout the excavation and pipe laying operations by pumping from sumps within the excavation.

5.4.2 Groundwater Pumping

Groundwater inflow, from the overburden deposits should be controlled by pumping from filtered sumps within the excavation. It is not expected that short term pumping during excavation will have a significant effect on nearby structures and services. It is anticipated that groundwater inflow from the overburden deposits into the excavations could be handled from within the excavations.

It is noted that groundwater levels and surface water flows can increase during wet periods of the year such as the early spring or following periods of precipitation.

The groundwater handling should be carried out in accordance with provincial and local regulations. Suitable detention and filtration will be required before discharging water. The contractor should be required to submit an excavation and groundwater management plan for review.

Depending on the depth of proposed foundations and groundwater level at the time of construction, an Environmental Activity and Sector Registry (EASR) in accordance with Environmental Protection Act Part II or a Category 3 Permit to Take Water may be required.

5.4.3 Pipe Bedding and Cover

The bedding for the sanitary sewers, storm sewers and watermains should be in accordance with OPSD 802.010 and 802.031 for flexible and rigid pipes, respectively. The pipe bedding should consist of at least 150 millimetres of well graded crushed stone meeting OPSS requirements for Granular A. OPSS documents allow recycled asphaltic concrete and concrete to be used in Granular A and Granular B Type II material.

Since the source of recycled material cannot be determined, it is suggested that any granular materials used in the service trenches be composed of virgin (i.e., not recycled) material only.
Allowance should be made for sub excavation of any existing fill, organic deposits, or disturbed material encountered at subgrade level.

Allowance should be made to place a subbedding layer composed of 150 to 300 millimetres of OPSS Granular B Type II in areas where wet silty sand is encountered at the pipe subgrade level to reduce the potential for disturbance.

Cover material, from pipe spring line to at least 300 millimetres above the top of the pipe, should consist of granular material, such as OPSS Granular A.

The use of clear crushed stone should not be permitted for the installation of site services, since it could exacerbate groundwater lowering of the overburden materials due to “French Drain” effects.

5.5 Pavement Design Recommendations

5.5.1 Pavement Structure

The following minimum asphaltic concrete and granular thicknesses, could be used for parking lot construction:

5.5.2 Light Duty Pave Areas (cars and small passenger trucks)

- 60 millimetres of hot mix asphaltic concrete (60 millimetres of Superpave 12.5 (Traffic Level B) over
- 150 millimetres of OPSS Granular A base over
- 300 millimetres of OPSS Granular B, Type II subbase, over
- Class II Woven Geotextile (as per OPSS 1860)

5.5.3 Heavy Duty Paved Areas (fire route, heavy trucks, trailers etc.)

- 100 millimetres of hot mix asphaltic concrete (50 millimetres of Superpave 12.5 (Traffic Level B) over 50 millimetres of Superpave 19.0 (Traffic Level B) over
- 150 millimetres of OPSS Granular A base over
- 400 millimetres of OPSS Granular B, Type II subbase; over
- Class II Woven Geotextile (as per OPSS 1860)

The above pavement structure assumes that any trench backfill for private services is adequately compacted, and that the fire laneway and parking lot subgrade surfaces are prepared as described in this report. If the subgrade surfaces become disturbed or wetted due to construction operations or precipitation, the granular subbase thickness given above may not be adequate and it may be necessary to increase the thickness of the subbase and/or to incorporate a woven geotextile separator between the subgrade surfaces and the granular subbase material. The adequacy of the design pavement thickness should be assessed by geotechnical personnel at the time of construction.

If the granular pavement materials are to be used by construction traffic, it may be necessary to increase the thickness of the granular subbase layer, install a woven geotextile separator between the roadway subgrade surface and the granular subbase material, or a combination of both, to prevent pumping and disturbance to the subbase material. The contractor should be made responsible for their construction access.

5.5.4 Asphalt Cement Type

Performance grade PG 58-34 asphalt cement should be specified for Superpave asphaltic concrete mixes.

5.5.5 Subgrade Preparation

In preparation for parking lot construction at this site, topsoil and any soft, wet, or deleterious materials should be removed from the proposed parking areas.

Prior to placing granular material for the parking lot, the exposed subgrade should be proof rolled using a large (10-ton) roller and approved by geotechnical personnel.

Any soft areas should be sub-excavated and replaced with suitable (dry) earth borrow or well shattered and graded rock fill material that is frost compatible with the materials exposed on the sides of the area of sub-excavation.

Similarly, should it be necessary to raise the parking lot grades at this site, material which meets OPSS specifications for Select Subgrade Material, earth borrow, or well shattered and graded rock fill material may be used.

The select subgrade material or earth borrow should be placed in maximum 300-millimetre-thick lifts and compacted to at least 95 percent of the standard Proctor maximum dry density value using vibratory compaction equipment. Rock fill should also be placed in thin lifts and suitably compacted either with a large drum roller, the haulage and spreading equipment, or a combination of both.

Truck traffic should be avoided on the native soil subgrade and the trench backfill within the roadways especially under wet conditions.

5.5.6 Pavement Drainage

Adequate drainage of the pavement granular materials and subgrade is important for the long-term performance of the pavement at this site. The existing grades at the site should be maintained provided that they provide drainage ditches and/or catch basins to promote drainage of the pavement granular materials. Catch basins should equipped with minimum 3-metre-long stub drains extending in two directions at the subgrade level.

5.5.7 Granular Material Compaction

The granular base and subbase materials should be compacted in maximum 300-millimetre-thick lifts to at least 99 percent of the standard Proctor maximum dry density value.

6. ADDITIONAL CONSIDERATIONS

6.1 Effects of Construction Induced Vibration

Some of the construction operations (such as excavation, granular material compaction, etc.) will cause ground vibration on and off on the site. The vibrations will attenuate with distance from the source but may be felt at nearby structures. Assuming that any excavating is carried out in accordance with the guidelines in this report, the magnitude of the vibrations will be much less than that required to cause damage to the nearby structures or services in good condition but may be felt at the nearby structures.

6.2 Effects of Trees on the Proposed Building

The site is underlain by sensitive silty clay, a soil type prone to shrinkage when moisture content decreases. Trees can lower the moisture content in these soils, which in turn may cause significant settlement and potential damage to nearby buildings with shallow foundations. For this reason, deciduous trees should not be planted closer to the building (or any ground-supported structure susceptible to settlement) than a distance equal to the tree's mature height. Where trees are planted in groups or rows, the recommended setback distance should be increased to 1.5 times the ultimate tree height. It should also be noted that the zone of soil affected by tree roots expands as the trees grow, and settlement-related issues may not become evident until several years later, once the trees' water demand exceeds the natural supply. Future landscaping design should therefore carefully consider the long-term effects of trees on the proposed building, site services, and any other ground-supported structures.

6.3 Excess Soil Management Plan

This report does not constitute an excess soil management plan. The disposal requirements for excess soil from the site have not been assessed.

6.4 Design Review and Construction Observation

It is recommended that the final design drawings be reviewed by the geotechnical engineer to ensure that the guidelines provided in this report have been interpreted as intended. The engagement of the services of the geotechnical consultant during construction is recommended to confirm that the subsurface conditions throughout the proposed excavations do not materially differ from those given in the report and that the construction activities do not adversely affect the intent of the design. The subgrade surfaces for the proposed structures should be inspected by experienced geotechnical personnel to ensure that suitable materials have been reached and properly prepared. The placing and compaction of earth fill and imported granular materials should be inspected to ensure that the materials used conform to the grading and compaction specifications.

7. CLOSURE

We trust this report provides sufficient information for your present purposes. If you have any questions concerning this report, please do not hesitate to contact our office.

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G. Davidson

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LICENSED PROFESSIONAL ENGINEER
G. Davidson
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100514452
2025-09-03
PROVINCE OF ONTARIO

AllRock Consulting Ltd

174 Colonnade Road
Ottawa, Ontario K2E 7J5

Borehole Plan

Client No:	Job No: 25012

Client: Pritec

Project: 299 Hunt Club

Address: 299 West Hunt Club Road, Nepean, ON, Canada

Legend:
- Borehole Locations
- Groundwater Monitoring Well Locations

Image Source: Google Maps	Viewed: 2025-03-12

Drawn By: Jeremy Milsom	Checked By: Greg Davidson	Date: 2025-03-12	Figure: 1

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Appendix A

Record of Borehole Sheets

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Geotechnical Log - Borehole

BH1-25

Latitude	**: 45.34039**	**Drill Rig**	**: Truckmount Drill Rig**	**Job Number**	**: 25012**
Longitude	**: -75.71439**	**Driller Supplier**	**: Downing Drilling**	**Client**	**: Pritec**
Ground Elevation	**: 87.64 (m)**	**Logged By**	**: Jeremy Milsom**	**Project**	**: 299 Hunt Club**
Total Depth	**: 8m BGL**	**Reviewed By**	**: Greg Davidson**	**Location**	**: 299 West Hunt Club Road, Nepean, ON, Canada**
		Date	**: 25/02/2025**	**Loc Comment**	**:**

Samples: SPT Sample	Blow Counts	Graphic Log	Elevation / Depth (m)	Material Description
SS1	50 (N=50blows/6Inches) R = 20		87.49 / 0.15	Pavement ASPHALT; Fill material - medium grained silty sand, moist
SS2	9,11,10,12 (N=21) R = 100			
SS3	3,2,4,5 (N=6) R = 100			
SS4	WH,WH,1,1 (N=1) R = 24			
SS5	1,2,1,1 (N=3) R = 100			
SS6	WH,WH,WH,WH (N=2) R = 100			
SS7	WH,WH,1,1 (N=1) R = 24		82.89 / 4.6	Grey, very moist, firm, silty clay
SS8	1,1,1,23 (N=2) R = 20			
SS9	22,23,20,50 (N=43) R = 50		76.79 / 6.1	Silty Sand (SM): wet, grey/brown, dense,
SS10	7,3,15,15 (N=18) R = 60			

BH1-25 Terminated at 8m (Terminated)

Method

EX	excavator
BH	backhoe bucket
NE	natural exposure
EE	existing xcavation
RP	ripper

Water

◀	complete water less	▽	Level during drilling
▶	Water inflow	**P**	partial water loss
▼	water level	**N**	none encountered

USC Classification

GW	well graded gravels	**SW**	well graded sands
GP	poorly graded gravels	**SP**	poorly graded sands
GM	silty gravel	**SM**	silty sands
GC	clayey gravel	**SC**	clayey sands
ML	inorg silts low plastic	**CL**	inorg clay low plastic
MH	inorg clay high plastic	**CI**	inorg clay med plastic
OL	org silts low plastic	**CH**	inorg clay high plastic
OH	org sills high plastic	**Pt**	peat of high org soils

Consistency

VS	Very soft
S	Soft
F	Firm
St	Stiff
VSt	Very stiff
H	Hard

Density

VL	Very loose
L	Loose
MD	Medium dense
D	Dense
VD	Very dense

Moisture

D	Dry
M	Moist
W	Wet
PL	plastic limit
LL	liquid limit

Soil Samples

B	bulk
D	disturbed
U(63)	U(63) push tube
U(50)	U(50) push tube
WS	water

In Situ Testing

PP	pen penetrometer
VS	vane shear
DCP	dynamic cone penetrometer

Laboratory Results

UC	undrained unconsol cohesion
UF	undrained unconsol friction angle
MC	moisture content
DD	dry density
LL	liquid limit
PL	plastic limit
LS	linear shrinkage
CC	undrained console cohesion
CF	undrained console friction angle
FH	falling head permeability
CH	constan head permeablity
CBR	californian bearing ratio

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Geotechnical Log - Borehole

BH2-25

UTM	: 18T	Drill Rig	: Truckmount Drill Rig	Job Number	: 25012
Latitude	: 45.34036	Driller Supplier	: Downing Drilling	Client	: Pritec
Longitude	: -75.71458	Logged By	: Jeremy Milsom	Project	: 299 Hunt Club
Ground Elevation	: 86.9 (ft)	Reviewed By	: Greg Davidson	Location	: 299 West Hunt Club Road, Nepean, ON, Canada
Total Depth	: 8m BGL	Date	: 25/02/2025	Loc Comment	:

Samples: SPT Sample	Samples: Grab Sample	Blow Counts	Graphic Log	Elevation / Depth (m)	Material Description	Vane Data
	GS1			86.75 / 0.15	Pavement ASPHALT	
					Fill material - medium grained silty sand, moist	
SS1		18,12,11,5 (N=23) R = 100				
SS2		2,3,4,6 (N=7) R = 100				
SS3		1,2,3,4 (N=5) R = 100				
SS4		1,1,1,2 (N=2) R = 100				
SS5		1,1,1,2 (N=2) R = 100		82.15 / 4.6		
SS6		WH,WH,WH,WH (N=0) R = 100			Grey, very moist, firm, silty clay	
				76.05 / 6.1		SV 90 -
SS7		14,22,33,30 (N=>50) R = 60			Silty Sand (SM): wet, grey/brown, dense,	
SS8		7,13,11,11 (N=24) R = 60				

BH2-25 Terminated at 8m (Terminated)

AllRock Consulting

Phone:

Geotechnical Log - Borehole

BH3-25

UTM	: 18T	Drill Rig	: Truckmount Drill Rig	Job Number	: 25012
Latitude	: 45.34035	Driller Supplier	: Downing Drilling	Client	: Pritec
Longitude	: -75.71484	Logged By	: Jeremy Milsom	Project	: 299 Hunt Club
Ground Elevation	: 87.1 (ft)	Reviewed By	: Greg Davidson	Location	: 299 West Hunt Club Road, Nepean, ON, Canada
Total Depth	: 8 m BGL	Date	: 25/02/2025	Loc Comment	:

Samples: SPT Sample	Blow Counts	Graphic Log	Elevation / Depth (m)	Material Description	Well Diagram	Water
			86.95 / 0.15	Pavement ASPHALT		
				Fill material - medium grained silty sand, moist		
SS1	7,5,5,5 (N=10) R = 100					
SS2	3,3,5,6 (N=8) R = 100					
SS3	2,2,3,5 (N=5) R = 100					
SS4	3,2,3,5 (N=5) R = 100					
SS5	1,1,1,1 (N=2) R = 100					
SS6	2,1,3,5 (N=4) R = 100		82.35 / 4.6	Grey, very moist, firm, silty clay		
SS7	8,15,20,25 (N=35) R = 60					
SS8	9,8,8,8 (N=16) R = 60		75.65 / 6.1	Silty Sand (SM): grey/brown, dense,		Standing
SS9	9,9,15,18 (N=24) R = 60					

BH3-25 Terminated at 25ft (Terminated)

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Appendix B

Laboratory Testing Results

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SIEVE ANALYSIS OF AGGREGATES
LS-602

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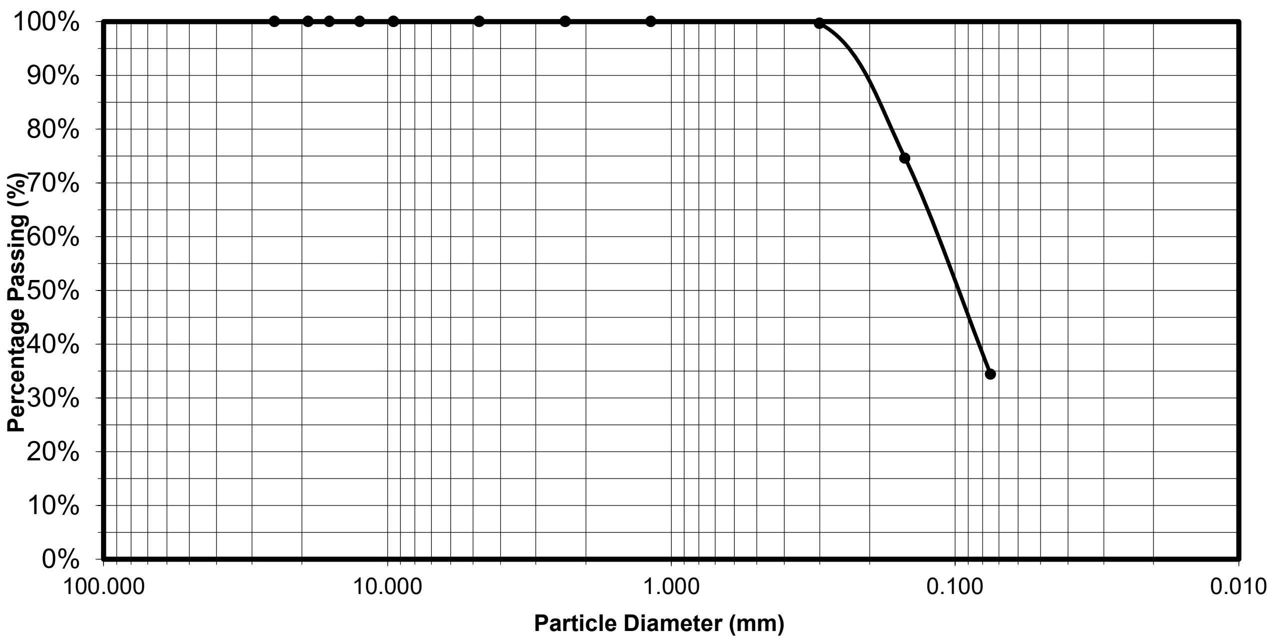
Ottawa, On, K2E7J5

Project:	299 West Hunt Club Road	**Project Number**	25012
Client:	Pritec Management	**Sample Classification:**	Silty Sand
Sample No.	BH2 - SS9	**Sample Depth**	25'-27'
Date Sampled	February 25, 2025	**Date Tested:**	March 26, 2025
Material Spec:			

Sieve Sizes					Remarks
#	mm	Lower Limit	Upper Limit	Tested Sample	
1"	25			100.0%	More Information Available Upon Request.
3/4"	19			100.0%	
5/8"	16.00			100.0%	
1/2"	12.50			100.0%	Sampled By:
3/8"	9.50			100.0%	J.Milsom
#4	4.75			100.0%	Tested By:
#8	2.36			100.0%	J.Milsom
#16	1.18			100.0%	Approved By
#50	0.3			99.6%	G. Davidson
#100	0.15			74.6%	Moisture Content
#200	0.075			34.4%	9.9



AllRock Consulting Ltd
35-174 Colonnade Rd. South
Ottawa, On, K2E7J5

SOIL MOISTURE CONTENT REPORT

Prject Information	
Project Name:	64 Jamie Avenue
Project No.:	25012
Client:	Pritec Management
Sampled By:	J.Milsom
Date Sampled:	February 26, 2025
Sample Description:	Soil Samples
Tested By:	J.Milsom
Date Tested:	March 25, 2025
Reviewed By:	G. Davidson
Date Reviewed:	March 26, 2025

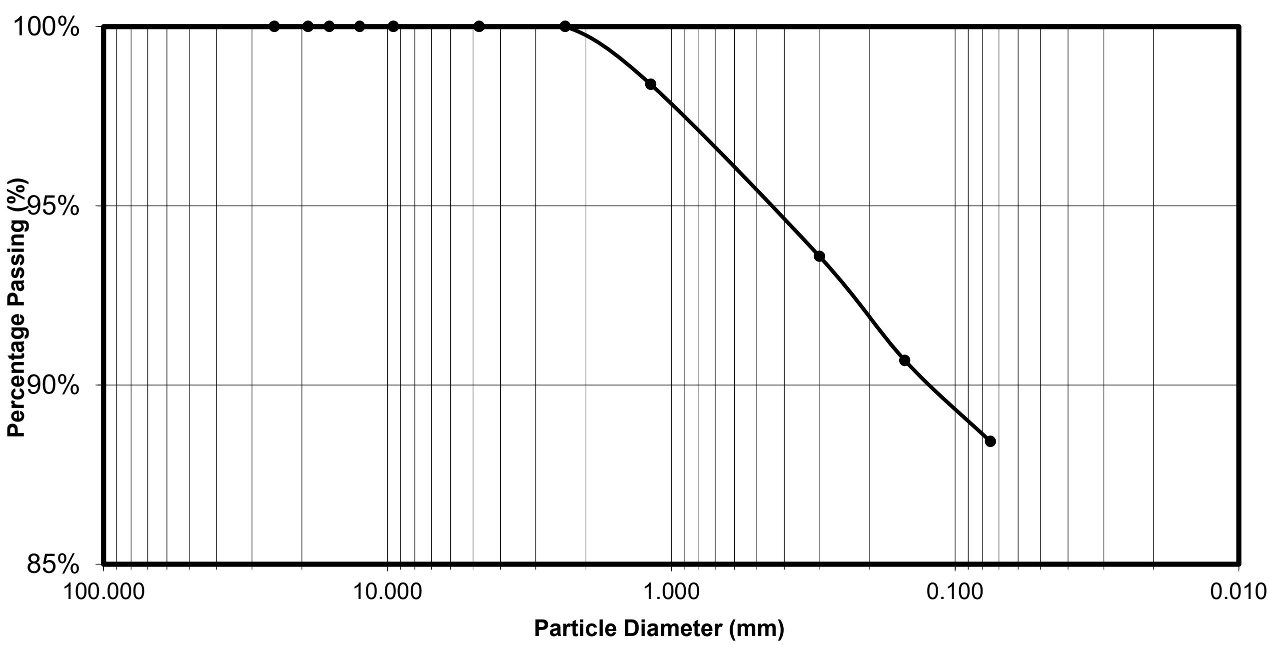
Soil Moisture Content		
Sample	Sample Depth	Moisture Content (%)
BH2 - SS9	25'-27'	9.9

SIEVE ANALYSIS OF AGGREGATES
LS-602

AllRock Consulting Ltd
35-174 Colonnade Rd. South
Ottawa, On, K2E7J5

Project:	25014	**Project Number**	25012
Client:	Pritec Management	**Sample Classification:**	Silty Clay trace Sand
Sample No.	SS5	**Sample Depth**	12.5' - 14.5'
Date Sampled	February 25, 2025	**Date Tested:**	March 27, 2025
Material Spec:			

Sieve Sizes					Remarks
#	mm	Lower Limit	Upper Limit	Tested Sample	
1"	25			100.0%	More Information Available Upon Request.
3/4"	19			100.0%	
5/8"	16.00			100.0%	
1/2"	12.50			100.0%	Sampled By:
3/8"	9.50			100.0%	J.Milsom
#4	4.75			100.0%	Tested By:
#8	2.36			100.0%	J.Milsom
#16	1.18			98.4%	Approved By
#50	0.3			93.6%	G. Davidson
#100	0.15			90.7%	Moisture Content
#200	0.075			88.4%	17



AllRock Consulting Ltd
35-174 Colonnade Rd. South
Ottawa, On, K2E7J5

SOIL MOISTURE CONTENT REPORT

Prject Information	
Project Name:	299 West Hunt Club Road
Project No.:	25012
Client:	Pritec Management
Sampled By:	J.Milsom
Date Sampled:	February 25, 2025
Sample Description:	Soil Samples
Tested By:	J.Milsom
Date Tested:	March 26, 2025
Reviewed By:	G. Davidson
Date Reviewed:	March 26, 2025

Soil Moisture Content

Sample	Sample Depth	Moisture Content (%)
BH3 - SS5	12.5 - 14.5'	17.00