PATERSON GROUP

Geotechnical Investigation

Proposed Apartment Building – Blocks 5, 8, and 10

609, 617, and 621 Longfields Drive, Ottawa, Ontario

Prepared for Campanale Homes

Report PG2119-4 Revision 1 dated July 10, 2024

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1.0 Introduction

Paterson Group (Paterson) was commissioned by Campanale Homes to conduct a geotechnical investigation for proposed apartment buildings located at 609, 617, and 621 Longfields Drive in the City of Ottawa (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objective of the geotechnical investigation was to:

- Determine the subsoil and groundwater conditions at this site by means of boreholes.

- Provide geotechnical recommendations pertaining to design of the proposed development including construction considerations which may affect the design.

The following report has been prepared specifically and solely for the aforementioned project which is described herein. It contains our findings and includes geotechnical recommendations pertaining to the design and construction of the subject development as they are understood at the time of writing this report.

Investigating for the presence or potential presence of contamination on the subject property was not part of the scope of work of the present investigation. Therefore, the present report does not address environmental issues.

2.0 Proposed Development

Based on available information and drawings provided by the client, it is anticipated that the future development will generally consist of five (5) multi-storey residential apartment buildings located in Blocks 5, 8, and 10.

It is anticipated that one 16-storey building and two 10-storey buildings will be constructed in Block 5 with two underground levels. Further, it is understood that one 15-storey building and one 28-storey building will be constructed in Block 8 with three underground levels. It is also anticipated that an 8-storey building will be built in Block 10 with two underground levels.

Associated access lanes, at-grade parking, and landscaped and hardscaped areas are also anticipated as part of the development. The development is anticipated to be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current geotechnical investigation was conducted between June 3, and 11, 2024, and consisted of 13 boreholes advanced to a maximum depth of 13.6 m below the existing ground surface.

The test hole locations were determined in the field by Paterson personnel and distributed in a manner to provide general coverage of the subject site taking into consideration site features and underground utilities. The test hole locations are presented on Drawing PG2119-7 – Test Hole Location Plan – Block 10, Drawing PG2119-8 – Test Hole Location Plan – Block 8, and Drawing PG2119-9 – Test Hole Location Plan – Block 5 included in Appendix 2.

The boreholes were advanced using a low clearance track-mounted auger drill rig operated by a two- person crew. The test hole procedure consisted of augering to the required depths at the selected locations and sampling the overburden. All fieldwork was conducted under the full-time supervision of our personnel under the direction of a senior engineer from our geotechnical department.

Sampling and In Situ Testing

Soil samples were collected from the boreholes using two different techniques, namely, sampled directly from the auger flights (AU) or collected using a 50 mm diameter split spoon (SS) sampler. The bedrock was cored to assess the bedrock quality. All samples were visually inspected and initially classified on site. The auger and split-spoon samples were placed in sealed plastic bags, and rock cores (RC) were placed in cardboard boxes.

All samples were transported to our laboratory for further examination and classification. The depths at which the auger, split spoon, and rock core samples were recovered from the boreholes are shown as AU, SS, and RC, respectively, on the Soil Profile and Test Data sheets presented in Appendix 1. Photographs of the rock core are presented in Appendix 1.

The Standard Penetration Test (SPT) was conducted in conjunction with the recovery of the split-spoon samples. The SPT results are recorded as “N” values on the Soil Profile and Test Data sheets. The “N” value is the number of blows required to drive the split-spoon sampler 300 mm into the soil after a 150 mm initial penetration using a 63.5 kg hammer falling from a height of 760 mm.

Diamond drilling was completed at each borehole location to confirm the bedrock quality. A recovery value and a Rock Quality Designation (RQD) value were calculated for each drilled section of bedrock and are presented as RC on the Soil Profile and Test Data sheets in Appendix 1. The recovery value is the ratio of the bedrock sample length recovered over the drilled section length, in percentage.

The RQD value is the total length ratio of intact rock core length more than 100 mm in one drilled section over the length of the drilled section, in percentage. These values are indicative of the quality of the bedrock.

The subsurface conditions observed in the test holes were recorded in detail in the field. The soil profiles are logged on the Soil Profile and Test Data sheets in Appendix 1 of this report.

Groundwater

A monitoring well was installed in five (5) borehole and the remaining boreholes were fitted with a flexible polyethylene standpipe to allow groundwater level monitoring. The groundwater level readings were obtained after a suitable stabilization period subsequent to the completion of the field investigation.

The groundwater observations are discussed in Subsection 4.3 and presented in the Soil Profile and Test Data sheets in Appendix 1 of this report.

Monitoring Well Installation

Typical monitoring well construction details are described below:

- ❑ Up to 1.5 m of slotted 32 mm diameter PVC screen at base the base of the boreholes.
- ❑ 32 mm diameter PVC riser pipe from the top of the screen to the ground surface.
- ❑ No.3 silica sand backfill within annular space around screen.
- ❑ 300 mm thick bentonite hole plug directly above PVC slotted screen.
- ❑ Clean backfill from top of bentonite plug to the ground surface.

Refer to the Soil Profile and Test Data sheets in Appendix 1 for specific well construction details.

3.2 Field Survey

The test hole locations were selected by Paterson to provide general coverage of the subject site. The test hole locations and ground surface elevation at each test hole location were surveyed by Paterson using a high-precision, handheld GPS and referenced to a geodetic datum. The location of the boreholes is presented on Drawing PG2119-7 – Test Hole Location Plan – Block 10, Drawing PG2119-8 – Test Hole Location Plan – Block 8, Drawing PG2119-9 – Test Hole Location Plan – Block 5 in Appendix 2.

3.3 Laboratory Testing

Soil samples and rock cores were recovered from the subject site and visually examined in our laboratory to review the results of the field logging. A total of two linear shrinkage analyses, two grain size distribution tests, and three Atterberg limit tests were completed on selected soil samples in Blocks 5, 8, and 10. Moisture content testing was completed on all recovered soil samples from the current investigation. The results of the testing are presented in Section 4.2 and are provided in Appendix 1.

Sample Storage

All samples from the current investigation will be stored in the laboratory for a period of one (1) month after issuance of this report. They will then be discarded unless we are otherwise directed.

3.4 Analytical Testing

One (1) soil sample was submitted for analytical testing to assess the corrosion potential for exposed ferrous metals and the potential of sulphate attacks against subsurface concrete structures. The sample was submitted to determine the concentration of sulphate and chloride, the resistivity, and the pH of the sample. The results are presented in Appendix 1 and are discussed further in Subsection 6.7.

3.5 Hydraulic Conductivity Testing

Hydraulic conductivity testing was conducted by the means of falling and rising head slug tests at five (5) monitoring well locations to determine the hydraulic properties of the bedrock at the subject site. The test data was analyzed as per the method set out by Hvorslev (1951). Assumptions inherent in the Hvorslev method include a homogeneous and isotropic aquifer of infinite extent with zero-storage assumption, and a screen length significantly greater than the monitoring well diameter.

The assumption regarding aquifer storage is considered to be appropriate for groundwater inflow through the bedrock aquifer. The assumption regarding screen length and well diameter is considered to be met based on a screen length of 1.5 m and a diameter of 0.03 m. While the idealized assumptions regarding aquifer extent, homogeneity, and isotropy are not strictly met in this case (or in any real-world situation), it has been our experience that the Hvorslev method produces effective point estimates of hydraulic conductivity in conditions similar to those encountered at the subject site. The testing results are further discussed in Subsection 4.4 of this report.

4.0 Observations

4.1 Surface Conditions

Block 5 is currently vacant and grass and/or gravel covered. Further, fill piles were observed in the center of the site at Block 5. The site is bordered to the west by residential buildings, to the south by Via Verona Avenue, to the east by Longfields Drive, and to the north by Via Chianti Grove.

Further, Block 8 is currently vacant and grass-covered with some mature trees present near the property boundary within the northwest portion of the site. Fill piles are also observed on the south portion of Block 8. The site is bordered to the west by an electrical power substation, to the east and south by Campanale Avenue, to the northeast by vacant and grass-covered land, and to the northwest by a pedestrian pathway followed by a BRT line.

Furthermore, Block 10 is currently vacant and grass-covered with some mature trees present near the property boundary within the northwest portion of the site. The site is bordered to the northeast by residential buildings, to the southeast by Campanale Avenue, to the southwest by vacant and grass-covered land, and to the northwest by a pedestrian pathway followed by a BRT line.

Reference should be made to Figure 1 - Key Plan, attached to the current memorandum.

The ground surface across Block 5 varies with a gradual slope from a geodetic elevation of 93.8 m at the north portion of Block 5 to 94.4 m at the south portion of Block 5 and is approximately at grade with surrounding streets.

The ground surface across Block 8 varies with a gradual slope from a geodetic elevation of 92.7 m at the north portion of Block 8 to 93.2 m at the south portion of Block 8 and is approximately at grade with Campanale Avenue.

The ground surface across Block 10 varies with a gradual slope from a geodetic elevation of 92.1 m at the rear to 93.2 m at the front and is approximately at grade with Campanale Avenue.

4.2 Subsurface Profile

Overburden

Block 5 – 621 Longfields Drive

Generally, the subsurface profile at the test hole locations in Block 5 consists of topsoil and/or fill underlain by a deposit of brown sandy silt deposit or glacial till layer and further by the underlying bedrock formation throughout most of Block 5. In addition, the fill throughout the northwest portion of the site at the location of BH 12-24 and BH 13-24 was observed to be underlain by a hard to stiff deposit of silty clay.

Fill extending to depths ranging from 0.2 to 1.7 m below the existing ground surface was observed at all test hole locations. The fill was generally observed to consist of silty sand with gravel, crushed stone, and some clay or silty clay, trace sand, gravel, and crushed stone.

The fill layer was observed to be underlain by a sandy silt layer at BH 8-24 which extended to an approximate depth of 2.1 m below the ground surface. In addition, at the location of BH 12-24 and 13-24, the fill layer was observed to be underlain by an undisturbed, hart to stiff, brown silty clay deposit which extended to approximate depths ranging between 3.5 to 4.3 m below the ground surface.

Furthermore, in the remainder of boreholes in Block 5, the fill layer was observed to be underlain by a glacial till layer. The glacial till deposit generally consists of dense to compact, grey silty sand or sandy silt with variable amounts of clay, gravel, cobbles, and boulders. The glacial till deposit was observed to extend to approximate depths between 6.9 and 8.5 m below the ground surface. Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for the details of the soil profile encountered at each test hole location.

Block 8 – 617 Longfields Drive

Generally, the subsurface profile at the test hole locations in Block 8 consists of topsoil and/or fill underlain by a deposit of brown to grey silty clay which is further underlain by a grey silt deposit ollowed by glacial till and bedrock.

Fill extending to depths ranging from 0.6 to 1.0 m below the existing ground surface was observed at all test hole locations. The fill was generally observed to consist of silty sand with gravel, crushed stone, and some clay or silty clay, trace sand, gravel, and crushed stone.

The fill layer was observed to be underlain by a deposit of silty clay. The silty clay deposit consisted of an undisturbed hard to firm, brown to grey silty clay which extended to approximate depths ranging between 2.2 to 3.4 m below the ground surface. The brown to grey silty clay layer was observed to be underlain by a layer of very loose, brown to grey silt which extended to approximate depths ranging between 3.3 to 5.6 m below the ground surface.

The glacial till deposit generally consists of very loose to very dense, grey silty sand or sandy silt with variable amounts of clay, gravel, cobbles, and boulders. The glacial till deposit was observed to extend to approximate depths between 7.7 and 8.8 m below ground surface. Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for the details of the soil profile encountered at each test hole location.

Block 10 – 609 Longfields Drive

Generally, the subsurface profile at the test hole locations in Block 10 consists of topsoil and/or fill underlain by a deposit of brown silty clay which is further underlain by a grey silt deposit. The grey silt deposit is underlain by glacial till and further by the underlying bedrock formation.

Fill extending to depths ranging from 0.3 to 1.2 m below the existing ground surface was observed at all test hole locations. The fill was generally observed to consist of silty sand with gravel, crushed stone, and some clay or silty clay, trace sand, gravel, and crushed stone.

The silty clay deposit consisted of an undisturbed hard to stiff, brown silty clay which extended to approximate depths ranging between 2.1 to 3.0 m below the ground surface. The brown silty clay layer was observed to be underlain by a layer of very loose to loose, grey silt which extended to approximate depths ranging between 3.7 to 4.2 m below the ground surface.

The glacial till deposit generally consists of compact to very dense, grey silty sand with variable amounts of clay, gravel, cobbles, and boulders. The glacial till deposit was observed to extend to approximate depths between 7.5 and 8.7 m below ground surface. Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for the details of the soil profile encountered at each test hole location.

Bedrock

Block 5 – 621 Longfields Drive

Bedrock was cored at all borehole locations from depths ranging from 6.1 to 10.6 m below the existing ground surface at Block 5. Generally, the bedrock throughout the majority of Block 5 was observed to consist of fair to excellent quality interbedded grey sandstone and dolomite. Further, vertical fractures and mud seams were observed at the location of BH 12-24.

Block 8 – 617 Longfields Drive

Bedrock was cored at all borehole locations from depths ranging from 7.5 to 13.6 m below the existing ground surface at Block 8. Generally, the bedrock throughout the majority of Block 8 was observed to consist of fair to excellent quality interbedded grey sandstone and dolomite with crystalized calcite at some locations.

Block 10 – 609 Longfields Drive

Bedrock was cored at all borehole locations from depths ranging from 6.1 to 10.6 m below the existing ground surface at Block 10. Generally, the bedrock throughout the majority of Block 10 was observed to consist of poor to excellent quality interbedded grey sandstone and dolomite with crystalized calcite at some locations.

Bedrock Geological Mapping

Based on available geological mapping, the bedrock across the majority of the site consists of interbedded sandstone and dolomite of the March formation. The overburden thickness across the subject site (including blocks 5, 8, and 10) ranges from 5 to 10 m.

Atterberg Limits Testing

Atterberg limits testing was completed on silty clay samples recovered from BH 2-24, BH 7-24, and BH 12-24. The result of the Atterberg limits tests is presented in Table 1 and on the Atterberg Limits Testing Results sheet in Appendix 1.

Table 1 - Atterberg Limits Results

Sample	Depth (m)	LL (%)	PL (%)	PI (%)	w (%)	Classification
BH 2-24 SS2	1.07	71	20	51	55.49	CH
BH 7-24 SS3	1.83	59	24	35	41.29	CH
BH 12-24 SS3	1.83	69	32	37	49.15	CH

Notes: LL: Liquid Limit; PL: Plastic Limit; PI: Plasticity Index; w: water content; CH: Inorganic Clays of High Plasticity

Grain Size Distribution and Hydrometer Testing

Grain size distribution analysis was completed on two selected recovered glacial till deposit samples. The results of the grain size distribution analysis are presented in Table 2 and on the Grain Size Distribution sheets in Appendix 1.

Table 2 – Grain Size Distribution Results

Sample	Depth (m)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
BH 6-24 SS8	5.64	24.1	42.6	27.4	5.8
BH 13-24 SS8	5.64	8.6	38.2	40.7	12.5

Note: The ground surface elevation at each borehole location was surveyed using a handheld GPS using a geodetic datum.

Shrinkage Testing

Linear shrinkage testing was completed on two samples recovered at a depth of 1.83, and 1.07 m from BH 6-24 and BH 12-24, respectively. The shrinkage limit and shrinkage ratio of the tested silty clay sample (BH 6-24) were found to be 18.55% and 1.78, respectively. The shrinkage limit and shrinkage ratio of the tested silty clay sample (BH 12-24) were found to be 22.77% and 1.69, respectively.

4.3 Groundwater

Groundwater levels were manually measured in the installed piezometers and monitoring well on June 10, and 19, 2024. The groundwater level readings are presented in the Soil Profile and Test Data sheets in Appendix 1. The measured groundwater levels are presented in Table 3 below:

Table 3 – Summary of Groundwater Levels

Borehole Number	Ground Surface Elevation (m)	Measured Groundwater Level		Date Recorded
		Depth (m)	Elevation (m)	
BH 1-24	93.19	3.51	89.68	June 10, 2024
BH 2-24	92.18	2.44	89.74	
BH 3-24	92.91	3.30	89.61	
BH 1-24	93.19	3.80	89.39	June 19, 2024
BH 2-24	92.18	2.71	89.47	
BH 3-24	92.91	3.60	89.31	
BH 4-24	90.71	4.00	86.71	
BH 5-24	92.82	3.37	89.45	
BH 6-24	93.14	4.32	88.82	
BH 7-24	93.10	4.36	88.74	
BH 8-24	93.81	4.82	88.99	
BH 9-24	93.95	4.91	89.04	
BH 10-24	94.34	5.24	89.10	
BH 11-24	94.26	5.14	89.12	
BH 12-24	93.91	4.83	89.08	
BH 13-24	93.90	4.89	89.01	

Note: The ground surface elevation at each borehole location was surveyed using a handheld GPS using a geodetic datum.

It should be noted that surface water can become trapped within a backfilled borehole that can lead to higher than typical groundwater level observations. It should be noted that groundwater levels are subject to seasonal fluctuations, therefore the groundwater levels could vary at the time of construction.

The long-term groundwater levels can also be estimated based on the observed colour, consistency, and moisture content of the recovered soil samples. Based on these observations, the long-term groundwater table can be expected at an approximate elevation ranging from **88.5 to 89.5 m** across the subject site. Groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater levels could vary at the time of construction.

Due to the presence of a silt layer below the silty clay deposit, water may be trapped below the silty clay deposit and during excavation, water infiltration may occur during the initial excavation process. Therefore, additional considerations should be taken during the excavation operation to mitigate pooling of water within the bottom of the excavation.

4.4 Hydraulic Conductivity Testing Results

Hydraulic conductivity tests were conducted at five (5) monitoring well locations throughout the subject site on June 19, 2024. The testing results are summarized in Table 4 below and included in Appendix 1.

Table 4 – Summary of Hydraulic Conductivity Testing Results.

Test Hole ID	Ground Surface Elevation (m)	Testing Depth Interval (m bgs)	Testing Elevation Interval (m)	K (m/sec)	Test Type	Material
BH 3-24	92.91	9.0-10.5	83.91-82.41	4.04×10^{-4}	Falling Head	Bedrock
BH 4-24	92.71	12.0-13.5	80.71-79.21	7.13×10^{-5}	Falling Head	Bedrock
				5.79×10^{-5}	Rising Head	
BH7-24	93.10	12.1-13.6	81.00-79.50	4.70×10^{-4}	Falling Head	Bedrock
BH9-24	93.95	8.8-10.3	85.15-83.65	3.50×10^{-4}	Rising Head	Bedrock
BH11-24	94.26	9.0-10.5	85.26-83.76	1.72×10^{-4}	Falling Head	Bedrock
				1.51×10^{-4}	Rising Head	

Summary of Results

Hydraulic conductivity testing conducted at the monitoring wells screened within the bedrock yielded hydraulic conductivity values ranging from 5.79×10^{-5} m/s to 4.7×10^{-4} m/s. These values generally fall within the range of published values for bedrock and are consistent with values Paterson has observed at sites with similar subsurface material. It should be noted that the hydraulic conductivity of the bedrock may vary based on the bedrock quality and hydrostatic pressure across the subject site.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered suitable for the proposed buildings. With two (2) levels of underground parking, the founding elevation will be approximately 7 to 8 m below the existing ground surface for the proposed buildings within Blocks 5 and 10. For Block 8, the founding elevation will be approximately 10 m below ground level to accommodate three levels of underground parking. The proposed apartment buildings will be founded on conventional spread footings placed on an undisturbed glacial till layer and/or a clean, surface sounded bedrock bearing surface. However, if design building loads are too high where shallow foundations placed on an undisturbed glacial till deposit, consideration could be given to founding the proposed building on a raft foundation.

Due to the presence of crystallized calcite within the bedrock, specifically below the proposed founding elevation. It is expected that the crystals were formed due to precipitation and surface water infiltrating pre-existing cracks within the limestone which forms crystals in the presence of minerals. The presence of calcite may weaken the bedrock where the calcite is present. Therefore, extra precautions should be made prior to the placement of the footings as well as a review of the vertical excavation faces should be done during excavation to assess the need for bedrock stabilization measures.

Bedrock removal is expected to be required to complete the excavation of the proposed basement levels for the buildings. Line drilling and controlled blasting where large quantities of bedrock need to be removed is recommended. All contractors should be prepared for bedrock and oversized boulder removal. The blasting operations should be planned and completed under the guidance of a professional engineer with experience in blasting operations.

Due to the presence of a silty clay layer, the proposed grading throughout the subject site will be subjected to a permissible grade restriction. Our permissible grade raise recommendations are discussed in Subsection 5.3 for each block.

Due to the relatively flat surface across the subject site and the absence of a defined confined or non-confined slopes throughout, no slope stability analysis is required from a geotechnical perspective.

The above and other considerations are further discussed in the following sections.

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and deleterious fill, such as those containing organic materials or construction debris, should be stripped from under any paved areas, pipe bedding and other settlement sensitive structures. Care should be taken not to disturb adequate bearing soils below the founding level during site preparation activities. Disturbance of the subgrade may result in having to sub-excavate the disturbed material and the placement of additional suitable fill material.

It is expected that the majority of overburden materials within all blocks will be excavated to the glacial till deposit surface and/or bedrock surface for the entire buildings' footprint to accommodate the two or three levels of underground parking.

Fill Placement

Fill placed for grading throughout the building footprint should consist, unless otherwise specified, of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. Imported fill material should be tested and approved prior to delivery to the site. The fill should be placed in a maximum 300 mm thick loose lifts and compacted by suitable compaction equipment. Fill placed beneath the building should be compacted to a minimum of 98% of the standard Proctor maximum dry density (SPMDD).

Non-specified existing fill along with site-excavated soil could be placed as general landscaping fill and beneath exterior parking areas where settlement of the ground surface is of minor concern. These materials should be spread in a maximum of 300 mm thick loose lifts and compacted by the tracks of the spreading equipment to minimize voids. If this material is to be used to build up the subgrade level for areas to be paved, it should be compacted in maximum 300 mm thick loose lifts to at least 98% of the material's SPMDD.

The placement of subgrade material should be reviewed at the time of placement by Paterson personnel. Non-specified existing fill and site-excavated soils are not suitable for placement as backfill against foundation walls, unless used in conjunction with a geocomposite drainage membrane, such as Miradrain G100N or Delta Terraxx.

Fill used for grading beneath the base and subbase layers of paved areas should consist, unless otherwise specified, of clean imported granular fill, such as OPSS Granular A, Granular B Type II, or select subgrade material. This material should be tested and approved by Paterson prior to delivery to the site. The fill should be placed in lifts no greater than 300 mm thick and compacted using suitable compaction equipment for the lift thickness. Fill placed beneath the paved areas should be compacted to at least 100% of its SPMDD.

Bedrock Removal

It is expected that line-drilling in conjunction with hoe-ramming, rock grinding, and controlled blasting will be required to remove the bedrock for the underground parking levels for the proposed buildings. In areas of weathered bedrock and where only a small quantity of bedrock is to be removed, bedrock removal may be possible by hoe-ramming.

Prior to considering blasting operations, the blasting effects on the existing services, buildings, and other structures should be addressed. A pre-blast or pre-construction survey of the existing structures located in the proximity of the blasting operations should be carried out prior to commencing site activities. The extent of the survey should be determined by the blasting consultant and should be sufficient to respond to any inquiries or claims related to the blasting operations.

As a general guideline, peak particle velocities (measured at the structures) should not exceed the below noted vibration limits during the blasting program to reduce the risks of damage to the existing surrounding structures. The blasting operations should be planned and conducted under the supervision of a licensed professional engineer who is also an experienced blasting consultant.

Excavation side slopes in sound bedrock can be carried out using near vertical sidewalls. A minimum 1 m horizontal ledge should be left between the bottom of the overburden excavation and the top of the bedrock surface to provide an area to allow for potential sloughing of the overburden. The 1 m horizontal ledge setback can be eliminated with a shoring program which has drilled piles extending below the proposed founding elevation.

Vibration Considerations

Construction operations are the cause of vibrations, and possibly, sources of nuisance to the community. Therefore, means to reduce the vibration levels as much as possible should be incorporated into the construction operations to maintain, as much as possible, a cooperative environment with the residents.

The following construction equipment could be the source of vibrations: hoe ram, compactor, dozer, crane, truck traffic, etc. Vibrations, whether caused by blasting operations or by construction operations, could be the source of detrimental vibrations on the nearby buildings and structures. Therefore, all vibrations are recommended to be limited.

Two parameters are used to determine the permissible vibrations, namely, the maximum peak particle velocity and the frequency. For low frequency vibrations, the maximum allowable peak particle velocity is less than that for high frequency vibrations.

As a guideline, the peak particle velocity should be less than 15 mm/s between frequencies of 4 to 12 Hz, and 50 mm/s above a frequency of 40 Hz (interpolate between 12 and 40 Hz). The guidelines are for current construction standards. Considering that these guidelines are above perceptible human level and, in some cases, could be very disturbing to some people, a pre-construction survey is recommended be completed to minimize the risks of claims during or following the construction of the proposed building.

Bedrock Excavation Face Reinforcement and Preparation

Bedrock stabilization methods, such as the use of horizontal rock anchors and rock wedges/bolts in conjunction with shotcrete and/or chain link fencing with a layer of woven geotextile connected to the excavation face is expected to be required at specific locations to prevent bedrock pop-outs, especially in areas where bedrock fractures are conducive to the failure of the bedrock surface. Further, shotcrete and/or other material may be required to in-fill areas where bedrock pop-outs occur due to the nature of bedrock removal throughout the excavation footprint and in advance of the placement of foundation waterproofing products.

The requirement for bedrock excavation face reinforcement should be evaluated by Paterson personnel during the excavation operations. As a preliminary recommendation, provisions should be carried out for providing a minimum 1 m wide bedrock face protection layer across building excavation footprint perimeters for all portions of the excavations that will extend below the bedrock surface. Throughout the building excavation and bedrock removal process, the vertical bedrock excavation perimeter surfaces should be hoe-rammed and grinded smooth to provide a relatively flat substrate surface for the placement of the drainage board. All loose bedrock fragments should be removed by grinding operations.

It is recommended that Paterson review the bedrock excavation program at the time of construction.

Overbreak in Bedrock

Sedimentary bedrock formation, such as sandstone, limestone, dolomite, and shale, contain bedding planes, joints and fractures, and mud seams which create natural planes of weakness within the rock mass. Although several factors of a blast may be controlled to reduce backbreak and overbreak, upon blasting, the rock mass will tend to break along natural planes of weakness that may be present beyond the designed blast profile. However, estimating the exact amount of backbreak and overbreak that may occur is not possible with conventional construction drill and blast methods.

Backbreak should be expected to occur along the perimeter of the building excavation footprint with conventional drill and blast bedrock removal methods. Further, overbreak is expected to occur throughout the lowest lifts of blasting due to the variable bedding planes and planes of weakness in the in-situ bedrock. It is very difficult to mitigate significant overblasting given the constraints posed by footing geometry and spacing with respect to the zone of influence of blasts and the bedrocks in-situ characteristics.

Depending on the methodology undertaken by the contractor, efforts taken to minimize backbreak and overbreak may add significant time and costs to the excavation operations and is not guaranteed to completely eliminate the potential for backbreak and overbreak. Overbreak below footings should be in-filled with lean-concrete and approved by Paterson prior to placing concrete.

As such, volume estimates of bedrock to be removed may not be reflective of the actual volume of bedrock that may be required to be removed at the time of construction. This may result in additional materials, such as imported fill and concrete, to make up for additional rock loss. It is recommended that the blasting operations be planned and conducted under the supervision of a licensed professional engineer who is an experienced blasting consultant.

It is highly recommended that the contractor/owner engage Paterson early into the excavation activities to identify anomalies within the bedrock. Vibration monitoring is also very essential at this stage which can be completed by Paterson as well. Also, it is important to note that completing an as-built of bedrock removal, including overbreaks, is required to be completed by the contractor and reviewed by Paterson to avoid confusion on the reasons for overbreaking the rock and to ensure that all parties are aware of the site conditions. Paterson should receive all as-built surveys post removal of the bedrock.

Protection of Subgrade (Raft Foundation)

It is recommended that a minimum of 75 mm thick lean concrete mud slab be placed on undisturbed, in-situ compact to very dense glacial till where a raft foundation is used as a building's foundation support. The main purpose of the mud slab is to reduce the risk of disturbance of the subgrade as a result of construction and moving equipment.

5.3 Foundation Design

Buildings in Block 5 – 621 Longfields Drive

Bearing Resistance Value (Conventional Shallow Foundation)

Footings, up to 6 m wide, founded on an undisturbed, compact to very dense glacial till bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **200 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **300 kPa**. A geotechnical resistance factor of 0.5 was applied to the reported bearing resistance value at ULS.

An undisturbed soil bearing surface consists of one from which all topsoil and deleterious materials, such as loose, frozen or disturbed soil, have been removed, in the dry, prior to the placement of concrete for footings.

Footings placed on a clean, surface sounded sandstone and dolomite bedrock surface can be designed using a factored bearing resistance value at ultimate limit states (ULS) of **1,500 kPa**, incorporating a geotechnical resistance factor of 0.5. A reduced bearing resistance value of **1000 kPa** is recommended for areas where the bedrock is found to contain vertical fractures and mud seams. Footings bearing on an acceptable bedrock bearing surface and designed using the bearing resistance values provided herein will be subjected to negligible potential post-construction total and differential settlements.

Footings placed on concrete in-filled, zero entry, vertical tranches extended to the bedrock surface can be designed to a similar bearing resistance values as the bedrock surface. It should be noted that the vertical trenches should extend horizontally a minimum of 150 mm beyond the footing faces in all directions. A minimum of 25 MPa concrete (28-day strength) should be used below the proposed footings.

A clean, surface-sounded bedrock bearing surface should be free of loose materials, and have no near surface seams, voids, fissures, or open joints which can be detected from surface sounding with a rock hammer.

Permissible Grade Raise Recommendations

Based on the undrained shear strength values of the silty clay deposit encountered throughout the subject site and our experience with the local silty clay deposit, **a permissible grade raise restriction of 2.0 m** is recommended in the immediate area of settlement sensitive structures in Block 5. A post-development groundwater lowering of 0.5 m was considered in our permissible grade raise restriction calculations.

If higher than permissible grade raises are required, preloading with or without a surcharge, lightweight fill and/or other measures should be investigated to reduce the risks of unacceptable long-term post construction total and differential settlements.

Settlement

Footings bearing on an undisturbed soil or an acceptable weathered bedrock bearing surface and designed for the bearing resistance values provided herein will be subjected to potential post- construction total and differential settlements of 25 and 20 mm, respectively.

Footings bearing on an acceptable bedrock bearing surface and designed for the bearing resistance values provided herein will be subjected to negligible potential post-construction total and differential settlements.

Bedrock to Soil Transition

Where a building is founded partly on bedrock and partly on soil, it is recommended to decrease the soil bearing resistance value by 25% for the footings placed on the soil bearing medium to reduce the potential long-term total and differential settlements. Also, at the soil/bedrock transitions, it is recommended that a minimum depth of 500 mm of bedrock be removed from below the founding elevation for a minimum length of 2 m on the bedrock side. This area should be subsequently reinstated with an engineered fill, such as OPSS Granular A or Granular B Type II and compacted to a minimum of 98% of the material SPMDD.

The width of the sub-excavation should be a minimum of 500 mm greater than the width of the footing. Steel reinforcement, extending a minimum of 3 m on both sides of the 2 m long transition, should be placed in the top portions of the footing and foundation walls.

Lateral Support

The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels.

Adequate lateral support is provided to an undisturbed glacial till or a weathered bedrock bearing surface when a plane extending down and out from the bottom edge of the footing at a minimum of 1.5H:1V passes only through in situ soil or weathered bedrock or a material of the same or higher capacity as the in situ soil or weathered bedrock.

Adequate lateral support is provided to a sound bedrock bearing medium when a plane extending horizontally and vertically from the footing perimeter at a minimum of 1H:6V (or shallower) passes through sound bedrock or a material of the same or higher capacity as the bedrock, such as concrete.

Raft Foundation (Glacial Till Bearing Surface)

Consideration can be given to a raft foundation if the building loads exceed the bearing resistance values given above. The following parameters may be used for raft design.

The amount of settlement of the raft slab will be dependent on the sustained raft contact pressure. A bearing resistance value at SLS (contact pressure) of **290 kPa** can be used for a raft supported on an undisturbed, compact to very dense glacial till deposit. The loading conditions for the contact pressure are based on sustained loads, that are generally taken to be 100% Dead Load and 50% Live Load. The factored bearing resistance (contact pressure) at ULS can be taken as **430 kPa**. A geotechnical resistance factor of 0.5 was applied to the bearing resistance value at ULS.

The modulus of subgrade reaction was calculated to be **11.5 MPa/m** for a contact pressure of **290 kPa**. The design of the raft foundation is required to consider the relative stiffness of the reinforced concrete slab and the supporting bearing medium.

Based on the following assumptions for the raft foundation, the proposed building can be designed using the above parameters and a total and differential settlement of 25 and 15 mm, respectively.

Buildings in Block 8 – 617 Longfields Drive

Bearing Resistance Value (Conventional Shallow Foundation)

Footings placed on a clean, surface sounded sandstone and dolomite bedrock surface can be designed using a factored bearing resistance value at ultimate limit states (ULS) of **1,500 kPa**, incorporating a geotechnical resistance factor of 0.5. A reduced bearing resistance value of **1000 kPa** is recommended for areas where the bedrock is found to contain traces of crystallized calcite of weak bedrock surface. Footings bearing on an acceptable bedrock bearing surface and designed using the bearing resistance values provided herein will be subjected to negligible potential post-construction total and differential settlements.

Footings placed on concrete in-filled, zero entry, vertical tranches extended to the bedrock surface can be designed to a similar bearing resistance values as the bedrock surface. It should be noted that the vertical trenches should extend horizontally a minimum of 150 mm beyond the footing faces in all directions. A minimum of 25 MPa concrete (28-day strength) should be used below the proposed footings.

A clean, surface-sounded bedrock bearing surface should be free of loose materials, and have no near surface seams, voids, fissures, or open joints which can be detected from surface sounding with a rock hammer.

Permissible Grade Raise Recommendations

Based on the undrained shear strength values of the silty clay deposit encountered throughout the subject site and our experience with the local silty clay deposit, **a permissible grade raise restriction of 2.0 m** is recommended in the immediate area of settlement sensitive structures in Block 8. A post-development groundwater lowering of 0.5 m was considered in our permissible grade raise restriction calculations.

If higher than permissible grade raises are required, preloading with or without a surcharge, lightweight fill and/or other measures should be investigated to reduce the risks of unacceptable long-term post construction total and differential settlements.

Settlement

Footings bearing on an acceptable bedrock bearing surface and designed for the bearing resistance values provided herein will be subjected to negligible potential post-construction total and differential settlements.

Lateral Support

The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels.

Adequate lateral support is provided to a sound bedrock bearing medium when a plane extending horizontally and vertically from the footing perimeter at a minimum of 1H:6V (or shallower) passes through sound bedrock or a material of the same or higher capacity as the bedrock, such as concrete.

Building in Block 10 – 609 Longfields Drive

Bearing Resistance Value (Conventional Shallow Foundation)

Footings, up to 6 m wide, founded on an undisturbed, compact to very dense glacial till bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **200 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **300 kPa**. A geotechnical resistance factor of 0.5 was applied to the reported bearing resistance value at ULS.

An undisturbed soil bearing surface consists of one from which all topsoil and deleterious materials, such as loose, frozen or disturbed soil, have been removed, in the dry, prior to the placement of concrete for footings.

Footings placed on a clean, surface sounded sandstone and dolomite bedrock surface can be designed using a factored bearing resistance value at ultimate limit states (ULS) of **1,500 kPa**, incorporating a geotechnical resistance factor of 0.5. A reduced bearing resistance value of **1000 kPa** is recommended for areas where the bedrock is found to contain traces of crystallized calcite of weak bedrock surface. Footings bearing on an acceptable bedrock bearing surface and designed using the bearing resistance values provided herein will be subjected to negligible potential post-construction total and differential settlements.

Footings placed on concrete in-filled, zero entry, vertical tranches extended to the bedrock surface can be designed to a similar bearing resistance values as the bedrock surface. It should be noted that the vertical trenches should extend horizontally a minimum of 150 mm beyond the footing faces in all directions. A minimum of 25 MPa concrete (28-day strength) should be used below the proposed footings.

A clean, surface-sounded bedrock bearing surface should be free of loose materials, and have no near surface seams, voids, fissures, or open joints which can be detected from surface sounding with a rock hammer.

Permissible Grade Raise Recommendations

Based on the undrained shear strength values of the silty clay deposit encountered throughout the subject site and our experience with the local silty clay deposit, **a permissible grade raise restriction of 2.0 m** is recommended in the immediate area of settlement sensitive structures in Block 10. A post-development groundwater lowering of 0.5 m was considered in our permissible grade raise restriction calculations.

If higher than permissible grade raises are required, preloading with or without a surcharge, lightweight fill and/or other measures should be investigated to reduce the risks of unacceptable long-term post construction total and differential settlements.

Settlement

Footings bearing on an undisturbed soil or an acceptable weathered bedrock bearing surface and designed for the bearing resistance values provided herein will be subjected to potential post- construction total and differential settlements of 25 and 20 mm, respectively.

Footings bearing on an acceptable bedrock bearing surface and designed for the bearing resistance values provided herein will be subjected to negligible potential post-construction total and differential settlements.

Bedrock to Soil Transition

Where a building is founded partly on bedrock and partly on soil, it is recommended to decrease the soil bearing resistance value by 25% for the footings placed on the soil bearing medium to reduce the potential long-term total and differential settlements. Also, at the soil/bedrock transitions, it is recommended that a minimum depth of 500 mm of bedrock be removed from below the founding elevation for a minimum length of 2 m on the bedrock side. This area should be subsequently reinstated with an engineered fill, such as OPSS Granular A or Granular B Type II and compacted to a minimum of 98% of the material SPMDD.

The width of the sub-excavation should be a minimum of 500 mm greater than the width of the footing. Steel reinforcement, extending a minimum of 3 m on both sides of the 2 m long transition, should be placed in the top portions of the footing and foundation walls.

Lateral Support

The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels.

Adequate lateral support is provided to an undisturbed glacial till or a weathered bedrock bearing surface when a plane extending down and out from the bottom edge of the footing at a minimum of 1.5H:1V passes only through in situ soil or weathered bedrock or a material of the same or higher capacity as the in situ soil or weathered bedrock.

Adequate lateral support is provided to a sound bedrock bearing medium when a plane extending horizontally and vertically from the footing perimeter at a minimum of 1H:6V (or shallower) passes through sound bedrock or a material of the same or higher capacity as the bedrock, such as concrete.

Raft Foundation (Glacial Till Bearing Surface)

Consideration can be given to a raft foundation if the building loads exceed the bearing resistance values given above. The following parameters may be used for raft design.

The amount of settlement of the raft slab will be dependent on the sustained raft contact pressure. A bearing resistance value at SLS (contact pressure) of **290 kPa** can be used for a raft supported on an undisturbed, compact to very dense glacial till deposit. The loading conditions for the contact pressure are based on sustained loads, that are generally taken to be 100% Dead Load and 50% Live Load. The factored bearing resistance (contact pressure) at ULS can be taken as **430 kPa**. A geotechnical resistance factor of 0.5 was applied to the bearing resistance value at ULS.

The modulus of subgrade reaction was calculated to be **11.5 MPa/m** for a contact pressure of **290 kPa**. The design of the raft foundation is required to consider the relative stiffness of the reinforced concrete slab and the supporting bearing medium.

Based on the following assumptions for the raft foundation, the proposed building can be designed using the above parameters and a total and differential settlement of 25 and 15 mm, respectively.

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class C** for Blocks 5, 8, and 10. If a higher seismic site class is required (Class A or B), a site specific shear wave velocity test may be completed to accurately determine the applicable seismic site classification for foundation design of the proposed building, as presented in Table 4.1.8.4.A of the Ontario Building Code 2012.

Soils underlying the subject site are not susceptible to liquefaction. Reference should be made to the latest revision of the Ontario Building Code 2012 for a full discussion of the earthquake design requirements.

5.5 Basement Floor Slab / Slab-on-Grade Construction

It is expected that the basement areas in blocks 5, 8, and 10 will be mostly parking and the recommended pavement structure noted in Subsection 5.7 will be applicable. However, if storage or other uses of the lower level are anticipated where a concrete floor slab will be used, it is recommended that the upper 200 mm of sub-slab fill consists of 19 mm clear crushed stone. All backfill material within the footprint of the proposed building should be placed in maximum 300 mm thick loose layers and compacted to at least 98% of its SPMDD.

5.6 Basement Wall

It is understood that the basement walls are to be poured against a water suppression system, which will be placed against the temporary shoring system and/or exposed bedrock face in blocks 5, 8, and 10. Below the bedrock surface, a nominal coefficient for at-rest earth pressure of 0.05 is recommended in conjunction with a bulk unit weight of 24.5 kN/m^3 (effective 15.5 kN/m^3).

A seismic earth pressure component will not be applicable for the foundation wall, which is to be poured against the bedrock face. It is expected that the seismic earth pressure will be transferred to the underground floor slab, which should be designed to accommodate these pressures. A hydrostatic pressure should be added for the portion below groundwater level.

Where the soil is to be retained, there are several combinations of backfill materials and retained soils that could be applicable for the basement walls of the subject structure. However, the conditions can be well-represented by assuming the retained soil consists of a material with an angle of internal friction of 30 degrees and a dry unit weight of 20 kN/m^3. The applicable effective unit weight of the retained soil can be estimated as 13 kN/m^3, where applicable. A hydrostatic pressure should be added to the total static earth pressure when calculating the effective unit weight.

The total earth pressure (P_{AE}) includes the static earth pressure component (P_o) and the seismic component (ΔP_{AE}).

Lateral Earth Pressures

The static horizontal earth pressure (P_o) can be calculated using a triangular earth pressure distribution equal to $K_o \cdot \gamma \cdot H$ where:

K_o = at-rest earth pressure coefficient of the applicable retained soil (0.5)
γ = unit weight of fill of the applicable retained soil (kN/m^3)
H = height of the wall (m)

An additional pressure having a magnitude equal to $K_o \cdot q$ and acting on the entire height of the wall should be added to the above diagram for any surcharge loading, q (kPa), that may be placed at ground surface adjacent to the wall. The surcharge pressure will only be applicable for static analyses and should not be used in conjunction with the seismic loading case.

Actual earth pressures could be higher than the “at-rest” case if care is not exercised during the compaction of the backfill materials to maintain a minimum separation of 0.3 m from the walls with the compaction equipment.

Seismic Earth Pressures

The total seismic force (P_{AE}) includes both the earth force component (P_o) and the seismic component (ΔP_{AE}).

The seismic earth force (ΔP_{AE}) can be calculated using $0.375 \cdot ac \cdot \gamma \cdot H^2/g$ where:

a_c = $(1.45-a_{max}/g)a_{max}$
γ = unit weight of fill of the applicable retained soil (kN/m^3)
H = height of the wall (m)

g = gravity, 9.81 m/s^2

The peak ground acceleration, (a_{max}), for the Ottawa area is 0.32 g according to the OBC 2012. Note that the vertical seismic coefficient is assumed to be zero. The earth force component (P_o) under seismic conditions can be calculated using $P_o = 0.5\ K_o\ \gamma\ H^2$, where $K_o = 0.5$ for the soil conditions noted above.

The total earth force (P_{AE}) is considered to act at a height, h (m), from the base of the wall, where:

$h = \{P_o \cdot (H/3) + \Delta P_{AE} \cdot (0.6 \cdot H)\} / P_{AE}$

The earth forces calculated are unfactored. For the ULS case, the earth loads should be factored as live loads, as per the OBC 2012.

5.7 Pavement Structure

Pavement Structure Over Overburden

The following pavement structures may be considered for rigid pavement, car only parking, and heavy traffic areas in Blocks 5, 8, and 10. The proposed pavement structures are shown in Tables 5, 6, and 7.

Table 5 - Recommended Rigid Pavement Structure - Lower Level

Thickness (mm)	Material Description
125	**Rigid Concrete Pavement -** 32 MPa concrete with air entrainment
300	**BASE** - OPSS Granular A Crushed Stone
SUBGRADE - Either fill, OPSS Granular B Type II material placed over in situ soil, fill or rock	

Table 6 - Recommended Pavement Structure - Car-Only Parking Areas and Fire-Truck Routes

Thickness (mm)	Material Description
50	**Wear Course** - HL-3 or Superpave 12.5 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
300	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Either in situ soil, fill or OPSS Granular B Type I or II material placed over in situ soil.	

Table 7 - Recommended Pavement Structure - Heavy-Truck Traffic and Loading Areas

Thickness (mm)	Material Description
40	**Wear Course** - HL-3 or Superpave 12.5 Asphaltic Concrete
50	**Binder Course** - HL-8 or Superpave 19.0 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
450	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Either in situ soil, fill or OPSS Granular B Type I or II material placed over in situ soil.	

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for this project. If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type II material.

The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 100% of the material's SPMDD using suitable compaction equipment.

Pavement Structure Over Podium Deck Area and Raft Foundations

It is anticipated that the podium deck structure may be provided for landscaping or to accommodate car only parking areas, access lanes, fire truck lanes, and loading areas. Based on the concrete slab subgrade for this area and/or over basements located over a raft slab, the pavement structure indicated in the following tables may be considered for design purposes:

Table 8 - Recommended Pavement Structure - Car-Only Parking Areas (Podium Deck and Raft Slab)

Thickness (mm)	Material Description
50	**Wear Course** - HL-3 or Superpave 12.5 Asphaltic Concrete
200**	**Base** - OPSS Granular A Crushed Stone
See Below*	**Thermal Break*** - Rigid insulation (See Paragraph Below)
n/a	**Waterproofing Membrane and Protection Board**
SUBGRADE – Reinforced Concrete Podium Deck or Raft Slab *If specified by others, not required from a geotechnical perspective. Also not required in basements over a raft slab. **Thickness is dependent on grade of insulation as noted in paragraphs below.	

Table 9 - Recommended Pavement Structure – Access Lane, Fire Truck Lane, Ramp and Heavy Truck Parking Areas (Podium Deck and Raft Slab)

Thickness (mm)	Material Description
40	**Wear Course** - HL-3 or Superpave 12.5 Asphaltic Concrete
50	**Wear Course** - HL-8 or Superpave 19.0 Asphaltic Concrete
300**	**Base** - OPSS Granular A Crushed Stone
See Below*	**Thermal Break*** - Rigid insulation (See Paragraph Below)
n/a	**Waterproofing Membrane and Protection Board**
SUBGRADE – Reinforced Concrete Podium Deck or Raft Slab * If specified by others, not required from a geotechnical perspective. Also not required in basements over a raft slab. **Thickness is dependent on grade of insulation as noted in paragraphs below.	

The transition between the pavement structure over the podium deck subgrade and soil subgrade beyond the footprint of the podium deck is recommended to be transitioned to match the pavement structures provided in the following section. For this transition, a 5H:1V frost taper is recommended between the two subgrade surfaces. Further, the base layer thickness should be increased to a minimum thickness of 600 mm below the top of the podium slab a minimum of 1.5 m horizontally from the face of the foundation wall prior to providing the recommended taper.

Should the proposed podium deck be specified by others to be provided a thermal break by the use of a layer of rigid insulation below the pavement structure, its placement within the pavement structure is recommended to be as per the above-noted tables. The layer of rigid insulation is recommended to consist of a DOW Chemical High-Load 100 (HI-100), High-Load 60 (HI-60), or High Load (HI-40) extruded polystyrene. The pavement structures' base layer thickness in Table 8 and Table 9 may be reduced by 25 mm if HI-100 is considered for this project. It should be noted that Styrofoam rigid insulation is not considered suitable for this application.

Pavement Structure Drainage

Satisfactory performance of the pavement structure is largely dependent on the contact zone between the subgrade material and the base stone in a dry condition. Failure to provide adequate drainage under conditions of heavy wheel loading can result in the fine subgrade soil being pumped into the voids in the stone subbase, thereby reducing load carrying capacity.
Where silty clay is anticipated at subgrade level, consideration should be given to installing subdrains during the pavement construction as per City of Ottawa standards. The subdrain inverts should be approximately 300 mm below subgrade level, and the subgrade surface should be crowned to promote water flow to drainage lines.

6.0 Design and Construction Precautions

6.1 Groundwater Control for Construction

Foundation Drainage and Waterproofing

The following recommendations may be considered for the architectural design of the buildings' foundation drainage systems. It is recommended that Paterson be engaged at the design stage of the future building (and prior to tender) to review and provide supplemental information for the building foundation drainage system design.

Supplemental details, review of architectural design drawings, and additional information may be provided by Paterson for these items for incorporation in the building design packages and associated tender documents. It is recommended that Paterson review all details associated with the foundation drainage system prior to tender.

Groundwater Suppression System

It is recommended that a groundwater suppression system be provided for the proposed structures in Blocks 5, 8, and 10. It is expected that insufficient room will be available for exterior backfill and the foundation wall will be cast as a blind-sided pour against a shoring system and the bedrock surface. It is recommended that the groundwater suppression system consist of the following:

- ❑ A waterproofing membrane should be placed against the shoring system between the underside of footings and 1 m below the existing ground surface (1 m above long-term groundwater elevation). Where the membrane will extend against the shoring system, it is recommended to consist of a membrane with a bentonite-lined face for being paced against the shoring system. The membrane is recommended to overlap below the overlying perimeter foundation footprint by a minimum of 1 m inwards towards the building footprint and from the face of the overlying foundation. This will allow construction to proceed without imposing groundwater lowering within the surrounding area of the proposed building in the short and long term conditions.

- ❑ A composite drainage membrane (Delta Terraxx, MiraDrain G100N or equivalent) should be placed against the HDPE face of the waterproofing membrane with the geotextile layer facing the waterproofing layer from finished ground surface to the top of the footing.

- The foundation drainage boards should be overlapped such that the bottom end of a higher board is placed in front of the top end of a lower board. All endlaps of the drainage board sheets should overlap abutting sheets by a minimum of 150 mm. All overlaps should be sealed with a suitable adhesive and/or sealant material approved by the geotechnical consultant. It is highly recommended that the drainage board rolls be installed horizontally rather than vertically to minimize the number of vertical joints forming between the rolls.

- It is recommended that 150 mm diameter PVC sleeves at 6 m centers be cast in the foundation wall at the foundation wall/footing interface to allow the infiltration of water to flow to the interior perimeter drainage pipe. The sleeves should be connected to openings in the HDPE face of the drainage board layer. The perimeter drainage pipe and underfloor drainage system should direct water to sump pit(s) within the lower basement area via an underfloor and interior drainage pipe system.

The top end lap of the foundation drainage board should be provided with a suitable termination bar against the foundation wall to mitigate the potential for water to perch between the drainage board and foundation wall.

Interior Perimeter and Underfloor Drainage

An interior perimeter and underfloor drainage system will be required to redirect water from the building's foundation drainage system to the building's sump pit(s) if it will not discharge to an exterior catch basin structure. For preliminary design purposes, it is recommended that the interior perimeter and underfloor drainage pipes should consist of 100 or 150 mm diameter corrugated perforated plastic pipe sleeved with a geosock, placed at approximately 6 m.

The underfloor drainage pipe should be placed in each direction of the basement floor span and connected to the perimeter drainage pipe. The interior drainage pipe should be provided tee-connections to extend pipes between the perimeter drainage line and the HDPE-face of the composite foundation drainage board via the foundation wall sleeves.

The spacing of the underfloor drainage should be confirmed by Paterson at the time of excavation when water infiltration can be better assessed and once the foundation layout and sump system location has been finalized.

Foundation Backfill

Where applicable, backfill against the exterior sides of the foundation walls should consist of free draining non frost susceptible granular materials.

The greater part of the site excavated materials will be frost susceptible and, as such, are not recommended for re-use as backfill against the foundation walls, unless used in conjunction with a drainage geocomposite, such as Miradrain G100N or Delta Terraxx, connected to the perimeter foundation drainage system. Imported granular materials, such as clean sand or OPSS Granular B Type II granular material, should otherwise be used for this purpose.

Foundation backfill material should be compacted in maximum 300 mm thick loose lifts and with suitably sized vibratory compaction equipment (smooth-drum roller for crushed stone fill, sheepsfoot roller for soil fill).

Podium Deck Waterproofing Tie-In (If Applicable)

Waterproofing layers for podium deck surfaces should overlap across and below the top end lap of the vertically installed composite foundation drainage board to mitigate the potential for water to migrate between the drainage board and foundation wall as depicted in Figure 2 – Podium Deck to Foundation Wall Drainage System Tie-in Detail.

Sidewalks and Walkways

Backfill material below sidewalk and walkway subgrade areas or other settlement sensitive structures which are not adjacent to the building should consist of free-draining, non-frost susceptible material. This material should be placed in maximum 300 mm thick loose lifts and compacted to at least 98% of its SPMDD under dry and above freezing conditions.

Foundation Raft Slab Construction Joints

If applicable, it is anticipated the raft slab will be poured in several pour segments. For the construction joint at each pour, a PVC water stop along with a chemical grout (Xypex or equivalent) should be applied to the entire vertical joint of the slab. Furthermore, a PVC water stop should be incorporated in the horizontal interface between the foundation wall and the raft slab.

Finalized Drainage and Waterproofing Design

Paterson should be provided with the finalized structural and architectural drawings for the proposed buildings in Blocks 5, 8, and 10 to provide a building-specific waterproofing and drainage design which includes the above-noted recommendations. The design will provide recommendations for other items such as minimum pipe spacings, pipe mechanical connections below grade, transitioning from blind to double sided pours (if applicable), etc.

6.2 Protection Against Frost Action

Perimeter footings of heated structures are required to be insulated against the deleterious effects of frost action. A minimum 1.5 m thick soil cover alone, or a combination of soil cover in conjunction with foundation insulation should be provided in this regard.

Other exterior unheated footings, such as those for isolated exterior piers, are more prone to deleterious movement associated with frost action than the exterior walls of the proper structure. These footings should be provided with a minimum 2.1 m thick soil cover (or insulation equivalent).

6.3 Excavation and Service Trenches

Unsupported Side Slopes

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be cut back at 1H:1V or flatter. The flatter slope is required for excavation below groundwater level. The subsurface soil is considered to be mainly a Type 2 and Type 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects.

Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should maintain safe working distance from the excavation sides.

Excavation side slopes carried out for the building footprint are recommended to be provided surface protection from erosion by rain and surface water runoff where shoring is not anticipated to be implemented. This can be accomplished by covering the entire surface of the excavation side-slopes with tarps secured between the top and bottom of the excavation and approved by Paterson personnel at the time of construction.

It is further recommended to maintain a relatively dry surface along the bottom of the excavation footprint to mitigate the potential for sloughing of side-slopes.

Slopes in excess of 3 m in height should be periodically inspected by the geotechnical consultant in order to detect if the slopes are exhibiting signs of distress.

A trench box is recommended to protect personnel working in trenches with steep or vertical sides. Services are expected to be installed by “cut and cover” methods and excavations should not remain open for extended periods of time.

Temporary Shoring

For preliminary design purposes, the temporary system may consist of a soldier pile and lagging system or interlocking steel sheet piling. Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be added to the earth pressures described below.

It is important to note that the excavation for the proposed buildings is expected to remove lateral support of the adjacent building footings. Therefore, a temporary shoring system, such as soldier piles and lagging, should be designed to provide the necessary lateral support for the adjacent foundations. In addition, the footings of the north neighbouring building could be supported with structural brackets designed by a qualified engineer, extended under the footings and welded to the back of the soldier piles.

These systems can be cantilevered, anchored or braced. Earth pressures acting on the shoring system may be calculated using the parameters provided in Table 10.

Table 10 – Soil Parameters for Calculating Earth Pressures Acting on Shoring System

Parameter	Value
Active Earth Pressure Coefficient (K_a)	0.33
Passive Earth Pressure Coefficient (K_p)	3
At-Rest Earth Pressure Coefficient (K_o)	0.5
Unit Weight (γ), kN/m^3	20
Submerged Unit Weight (γ'), kN/m^3	13

Soldier Pile and Lagging System

The earth pressure acting on a soldier pile and lagging shoring system can be calculated using a rectangular earth pressure distribution with a maximum pressure of 0.65·K·γ·H for strutted or anchored shoring, or a triangular earth pressure distribution with a maximum value of K·γ·H for a cantilever shoring system. H is the height of the excavation.

The active earth pressure should be used where wall movements are permissible while the at-rest pressure should be used if no movement is permissible.

The total unit weight should be used above the groundwater level while the submerged unit weight should be used below the groundwater level.

The hydrostatic groundwater pressure should be added to the earth pressure distribution wherever the undrained unit weights are used for earth pressure calculations, should the level on the groundwater not be lowered below the bottom of the excavation. If the groundwater level is lowered, the total unit weight for the soil should be used for the full height, with no hydrostatic groundwater pressure component.

A minimum factor of safety of 1.5 should be used.

6.4 Pipe Bedding and Backfill

Bedding and backfill materials should be in accordance with the most recent Material Specifications and Standard Detail Drawings from the Department of Public Works and Services, Infrastructure Services Branch of the City of Ottawa.

The pipe bedding for the sewer and water pipes should consist of at least 150 mm of OPSS Granular A. The bedding layer thickness should be increased to a minimum of 300 mm where the subgrade will consist of silty clay or bedrock. The material should be placed in a maximum 225 mm thick loose lifts and compacted to a minimum of 99% of its SPMDD. The bedding material should extend at least to the spring line of the pipe.

The cover material, which should consist of OPSS Granular A, should extend from the spring line of the pipe to at least 300 mm above the obvert of the pipe. The material should be placed in maximum 225 mm thick lifts and compacted to a minimum of 99% of its SPMDD.

It should generally be possible to re-use the moist (not wet) site-generated fill above the cover material if the excavation and filling operations are carried out in dry weather conditions. Wet site-generated fill will be difficult to re-use, as the high-water contents make compacting impractical without an extensive drying period.

Where hard surface areas are considered above the trench backfill, the trench backfill material within the frost zone (about 1.8 m below finished grade) should match the soils exposed at the trench walls to minimize differential frost heaving. The trench backfill should be placed in maximum 300 mm thick loose lifts and compacted to a minimum of 95% of the material's SPMDD.

6.5 Groundwater Control

Groundwater Control for Building Construction

Based on our observations, it is anticipated that groundwater infiltration into the excavations may be moderate to high. However, pumping from open sumps should be sufficient to control the groundwater influx through the sides of shallow excavations. Provisions should be carried out for using higher capacity open sump systems for excavations undertaken below the bedrock surface. The contractor should be prepared to direct water away from all subgrades, regardless of the source, to prevent disturbance to the founding medium.

Permit to Take Water

Based on the site-specific hydraulic conductivity testing results, it is anticipated that > 400,000 L/day of groundwater and/or surface water will be required to be pumped during the construction phase. Therefore, a Ministry of Environment, Conservation and Parks (MECP) Permit to Take Water (PTTW) will be required. At least 4 to 5 months should be allowed for completion of the application and issuance of the permit by the MECP.

6.6 Winter Construction

Precautions must be taken if winter construction is considered for this project.

The subsoil conditions at this site consist of frost susceptible materials. In the presence of water and freezing conditions, ice could form within the soil mass. Heaving and settlement upon thawing could occur.

In the event of construction during below zero temperatures, the founding stratum should be protected from freezing temperatures by the use of straw, propane heaters and tarpaulins or other suitable means. In this regard, the base of the excavations should be insulated from sub-zero temperatures immediately upon exposure and until such time as heat is adequately supplied to the building and the footings are protected with sufficient soil cover to prevent freezing at founding level.

Trench excavations and pavement construction are also difficult activities to complete during freezing conditions without introducing frost in the subgrade or in the excavation walls and bottoms. Precautions should be taken if such activities are to be carried out during freezing conditions. Additional information could be provided, if required.

Precaution must be taken where excavations are carried in proximity of existing structures which may be adversely affected due to the freezing conditions. In particular, it should be recognized that where a shoring system is used, the soil behind the shoring system will be subjected to freezing conditions and could result in heaving of the structure(s) placed within or above frozen soil. Provisions should be made in the contract document to protect the walls of the excavations from freezing, if applicable.

6.7 Corrosion Potential and Sulphate

The results of analytical testing show that the sulphate content is less than 0.1%. This result is indicative that Type 10 Portland cement (normal cement) would be appropriate for this site. The chloride content and the pH of the sample indicate that they are not significant factors in creating a corrosive environment for exposed ferrous metals at this site, whereas the resistivity is indicative of a moderate to aggressive corrosive environment.

6.8 Landscaping Considerations

Tree Planting Considerations

It is understood the proposed buildings will include two to six levels of underground parking and the structures will be founded on foundation located at 7 or 10 m below the finished grade. Given the depth of foundations proposed for the structures, it is expected that the support of the foundations derives from soil located below the depth that dewatering by tree roots. Therefore, foundation distress due to potential moisture depletion caused by trees is not expected to occur at the subject site. Since the structures are not anticipated to be founded upon silty clay soils affected by the depth of root penetration, City approved trees within the subject site will not be subject to planting restrictions as based on the *City of Ottawa Tree Planting in Sensitive Marine Clay Soils (2017 Guidelines) from a geotechnical perspective.*

7.0 Recommendations

It is recommended that the following be carried out by Paterson once preliminary and future details of the proposed development have been prepared:

- ❑ Review preliminary and detailed grading, servicing, landscaping, and structural plan(s) from a geotechnical perspective.
- ❑ Review of the geotechnical aspects of the excavation contractor's shoring design, if not designed by Paterson, prior to construction, if applicable.
- ❑ Review of architectural plans pertaining to groundwater suppression systems, underfloor drainage systems, and waterproofing details for elevator shafts.

It is a requirement for the foundation design data provided herein to be applicable that a material testing and observation program be performed by the geotechnical consultant. The following aspects of the program should be performed by Paterson:

- ❑ Review and inspection of the installation of the foundation drainage systems.
- ❑ Observation of all bearing surfaces prior to the placement of concrete.
- ❑ Observation of driving and re-striking of all pile foundations.
- ❑ Sampling and testing of the concrete and fill materials.
- ❑ Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- ❑ Observation of all subgrades prior to backfilling and follow-up field density tests to determine the level of compaction achieved.
- ❑ Field density tests to determine the level of compaction achieved.
- ❑ Sampling and testing of the bituminous concrete including mix design reviews.

A report confirming that these works have been conducted in general accordance with our recommendations could be issued upon the completion of a satisfactory inspection program by the geotechnical consultant.

All excess soil must be handled as per ***Ontario Regulation 406/19: On-Site and Excess Soil Management***.

8.0 Statement of Limitations

The recommendations provided are in accordance with the present understanding of the project. Paterson requests permission to review the recommendations when the drawings and specifications are completed.

A soils investigation is a limited sampling of a site. Should any conditions at the site be encountered which differ from those at the test locations, Paterson requests immediate notification to permit reassessment of our recommendations.

The recommendations provided herein should only be used by the design professionals associated with this project. They are not intended for contractors bidding on or undertaking the work. The latter should evaluate the factual information provided in this report and determine the suitability and completeness for their intended construction schedule and methods. Additional testing may be required for their purposes.

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than Campanale Homes or their agents is not authorized without review by Paterson for the applicability of our recommendations to the alternative use of the report.

Paterson Group Inc.

Yashar Ziaeimehr, M.A.Sc., EIT

Faisal I. Abou-Seido, P.Eng.

Report Distribution:

- ❑ Campanale Homes (E-mail copy)
- ❑ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ROCK CORE PHOTOGRAPHS

ATTERBERG LIMIT TESTING RESULTS

GRAIN SIZE TESTING RESULTS

ANALYTICAL TESTING RESULTS

HYDRAULIC CONDUCTIVITY TESTING RESULTS

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation - Prop. Apartment Bldgs
Blocks 5, 8, and 10 - 609, 617, and 621 Longfields Dr
Ottawa, Ontario

EASTING: 363961.114 **NORTHING:** 5016468.983 **ELEVATION:** 93.19

DATUM: Geodetic

REMARKS:

BORINGS BY: CME-55 Low Clearance Drill **DATE:** June 3, 2024

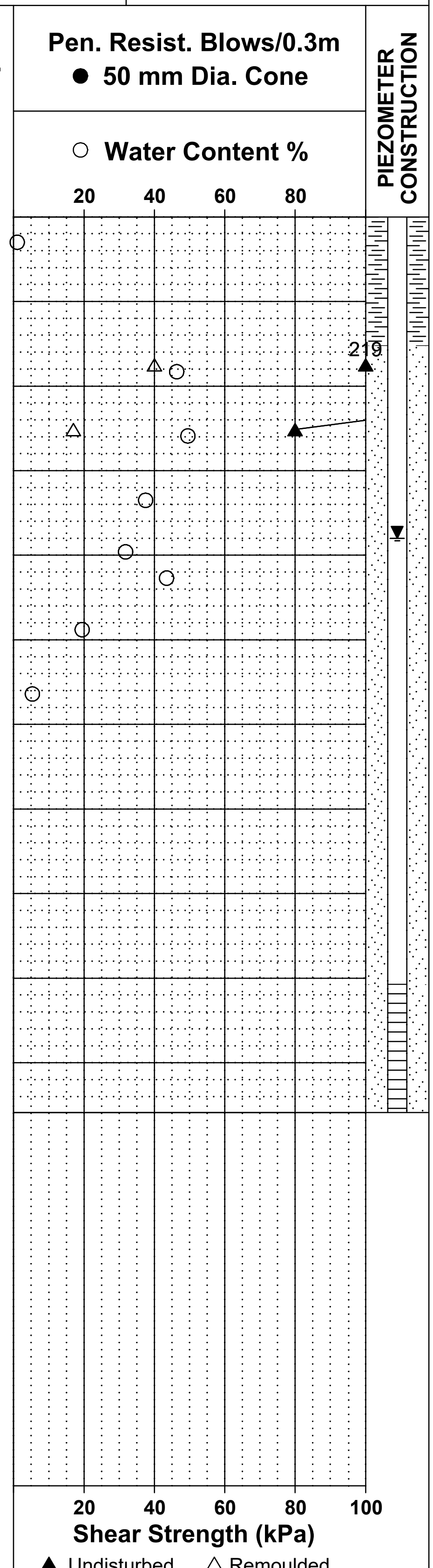
FILE NO. PG2119

HOLE NO. BH 1-24

SAMPLE DESCRIPTION	Depth to base (m)	TYPE	NUMBER	% RECOVERY	N VALUE or RQD
Ground Surface					
FILL: Brown silty sand with crushed stone, gravel, trace topsoil	0.61	AU	1		
FILL: Brown silty clay, trace sand, gravel and topsoil	1.22	SS	2	50	11
Hard to stiff brown **SILTY CLAY**		SS	3	75	P
	2.97	SS	4	0	P
Loose grey **SILT**, trace to some clay		SS	5	50	P
- Clay content increasing with depth	4.19	SS	6	75	3
GLACIAL TILL: Dense grey silty sand with gravel, cobbles and boulders, some clay		SS	7	54	34
		SS	8	83	37
	7.47	RC	1	58	N/A
BEDROCK: Poor quality interbedded grey sandstone and dolomite - Crystallized calcite encountered from 7.4 to 8.0 m depth		RC	2	93	36
- Crystallized calcite encountered from 9.5 to 9.9 m depth		RC	3	100	36
- Crystallized calcite encountered from 10.1 to 10.3 m depth	10.59				
End of Borehole					

(GWL at 3.51 m - June 10, 2024)
(GWL at 3.8 m - June 19, 2024)

DEPTH (m)	ELEV. (m)
0	93.19
1	92.19
2	91.19
3	90.19
4	89.19
5	88.19
6	87.19
7	86.19
8	85.19
9	84.19
10	83.19



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation - Prop. Apartment Bldgs Blocks 5, 8, and 10 - 609, 617, and 621 Longfields Dr Ottawa, Ontario

EASTING: 363947.379 **NORTHING:** 5016484.095 **ELEVATION:** 92.18

DATUM: Geodetic

REMARKS:

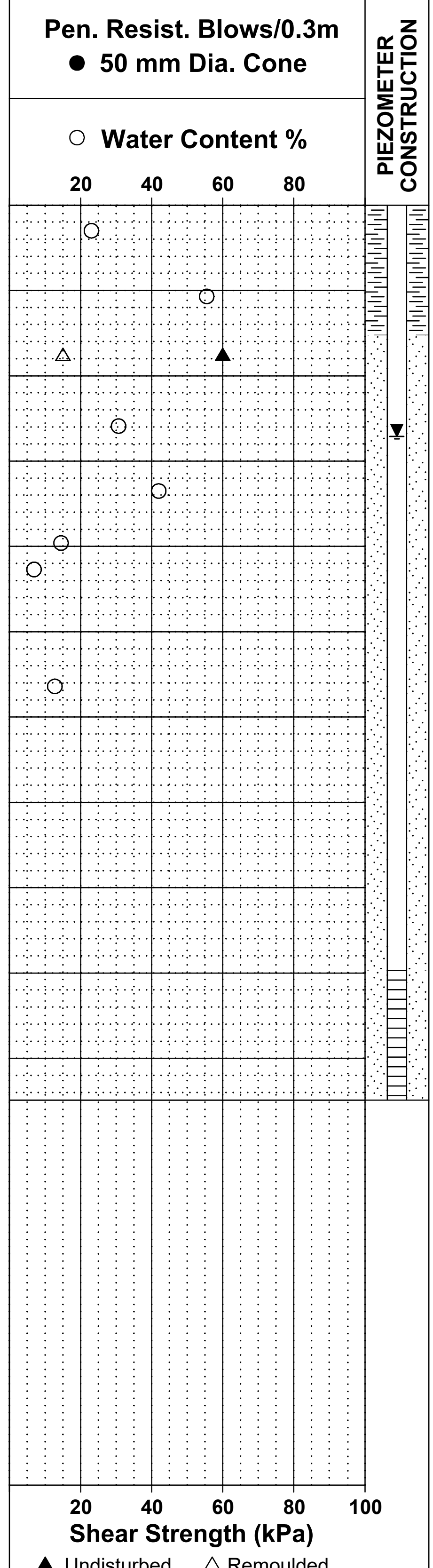
BORINGS BY: CME-55 Low Clearance Drill **DATE:** June 3, 2024

FILE NO. PG2119

HOLE NO. BH 2-24

SAMPLE DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
Ground Surface						0	92.18
FILL: Brown silty sand, some clay, trace topsoil and crushed stone	0.36	AU	1				
Stiff brown **SILTY CLAY**		SS	2	92	4	1	91.18
	2.13	SS	3	42	P	2	90.18
Loose grey **SILT**, some clay		SS	4	83	P	3	89.18
	3.73	SS	5	100	P		
GLACIAL TILL: Loose grey clayey silt, some sand and gravel	4.27	SS	6	75	8	4	88.18
GLACIAL TILL: Very dense grey silty sand to sandy silt with gravel, cobbles and boulders		SS	7	100	+50	5	87.18
- Trace clay by 5.5 m depth	6.10	SS	8	100	+50	6	86.18
BEDROCK: Excellent quality grey sandstone		RC	1	100	93	7	85.18
		RC	2	100	91	8	84.18
						9	83.18
	10.49	RC	3	100	90	10	82.18
End of Borehole							

(GWL at 2.44 m - June 10, 2024)
(GWL at 2.71 m - June 19, 2024)



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation - Prop. Apartment Bldgs
Blocks 5, 8, and 10 - 609, 617, and 621 Longfields Dr
Ottawa, Ontario

EASTING: 363931.931 **NORTHING:** 5016457.913 **ELEVATION:** 92.91

DATUM: Geodetic

REMARKS:

BORINGS BY: CME-55 Low Clearance Drill **DATE:** June 4, 2024

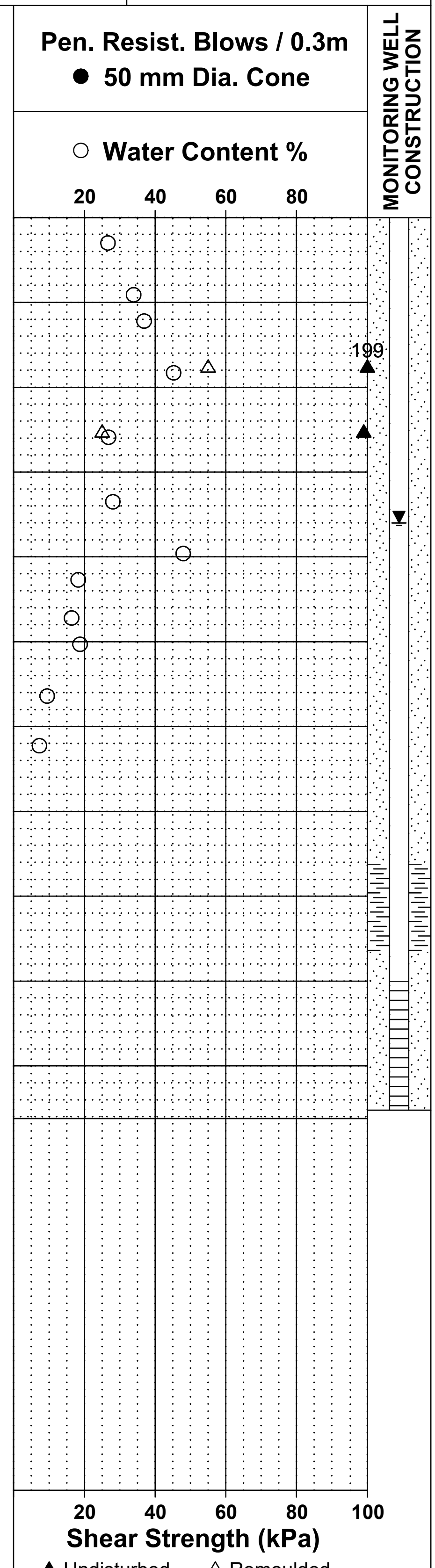
FILE NO. PG2119

HOLE NO. BH 3-24

SAMPLE DESCRIPTION	Depth to base (m)	TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	92.91
FILL: Brown silty clay, trace sand, gravel, crushed stone and topsoil	0.91	AU	1				
Hard to stiff brown **SILTY CLAY**, trace sand - Observed a thin silty sand seam at 1.8 m depth		SS	2	83	14	1	91.91
		SS	3	83	P	2	90.91
	2.97	SS	4	4	P		
Very loose grey **SILT**, some clay		SS	5	67	P	3	89.91
	4.11					4	88.91
GLACIAL TILL: Very loose to loose grey silty clay, some sand, trace gravel	4.88	SS	6	100	2		
GLACIAL TILL: Compact grey silty sand to sandy silt with gravel, cobbles and boulders, trace clay		SS	7	50	9	5	87.91
		SS	8	83	25		
		SS	9	100	+50	6	86.91
		RC	1	68		7	85.91
	8.74	RC	2	100	50	8	84.91
BEDROCK: Fair to excellent quality interbedded grey sandstone and dolomite bedrock						9	83.91
	10.62	RC	3	100	96	10	82.91

End of Borehole

(GWL at 3.30 m - June 10, 2024)
(GWL at 3.60 m - June 19, 2024)



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation - Prop. Apartment Bldgs
Blocks 5, 8, and 10 - 609, 617, and 621 Longfields Dr
Ottawa, Ontario

EASTING: 363893.968 **NORTHING:** 5016376.248 **ELEVATION:** 92.71

DATUM: Geodetic

REMARKS:

BORINGS BY: CME-55 Low Clearance Drill **DATE:** June 4, 2024

FILE NO. PG2119

HOLE NO. BH 4-24

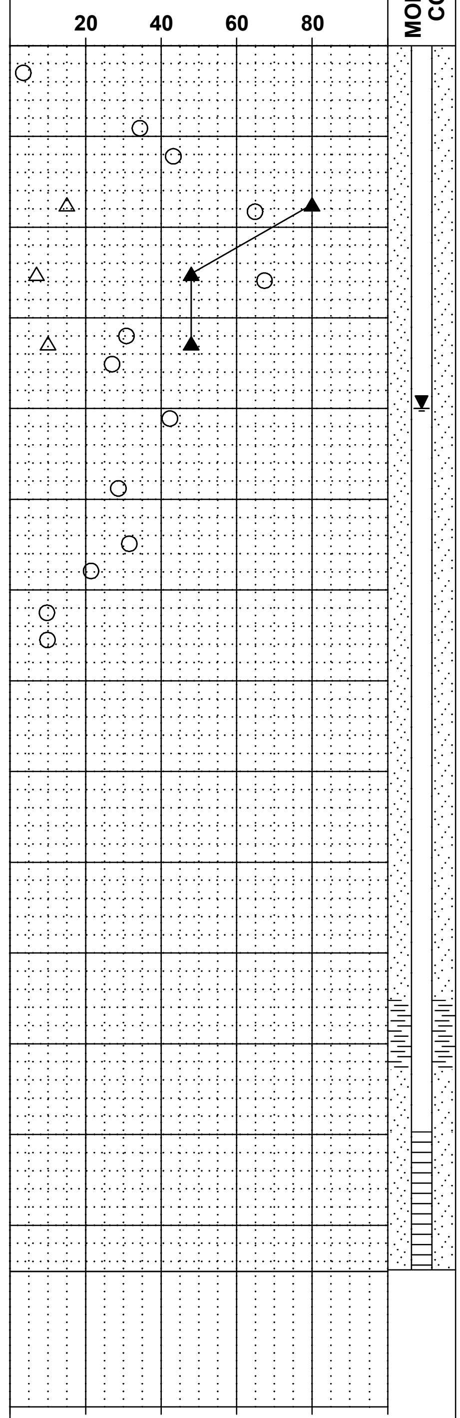
SAMPLE DESCRIPTION	Depth to base (m)
GROUND SURFACE	
FILL: Brown silty sand, some clay, trace gravel, crushed stone and topsoil	0.91
Hard to firm brown **SILTY CLAY** - Grey by 3.1 m depth	3.35
Very loose grey **SILT**, some sand and clay	5.54
GLACIAL TILL: Very loose grey silty sand, trace clay, gravel and cobbles	6.40
GLACIAL TILL: Compact grey silty sand to sandy silt with cobbles and boulders, some gravel, trace clay	7.75
BEDROCK: Fair to excellent quality interbedded grey sandstone and dolomite bedrock - Crystallized calcide encountered from 8.3 to 8.4 m depth	13.51
End of Borehole (GWL at 4.00 m - June 19, 2024)	

TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	100	9
SS	3	100	P
SS	4	100	P
SS	5	92	P
SS	6	67	3
SS	7	100	2
SS	8	100	2
SS	9	58	7
SS	10	100	+50
RC	1	100	74
RC	2	100	93
RC	3	100	97
RC	4	100	100

DEPTH (m)	ELEV. (m)
0	92.71
1	91.71
2	90.71
3	89.71
4	88.71
5	87.71
6	86.71
7	85.71
8	84.71
9	83.71
10	82.71
11	81.71
12	80.71
13	79.71

Pen. Resist. Blows / 0.3m
● 50 mm Dia. Cone

○ Water Content %



Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation - Prop. Apartment Bldgs
Blocks 5, 8, and 10 - 609, 617, and 621 Longfields Dr
Ottawa, Ontario

EASTING: 363884.585 **NORTHING:** 5016352.448 **ELEVATION:** 92.82

DATUM: Geodetic

REMARKS:

BORINGS BY: CME-55 Low Clearance Drill **DATE:** June 5, 2024

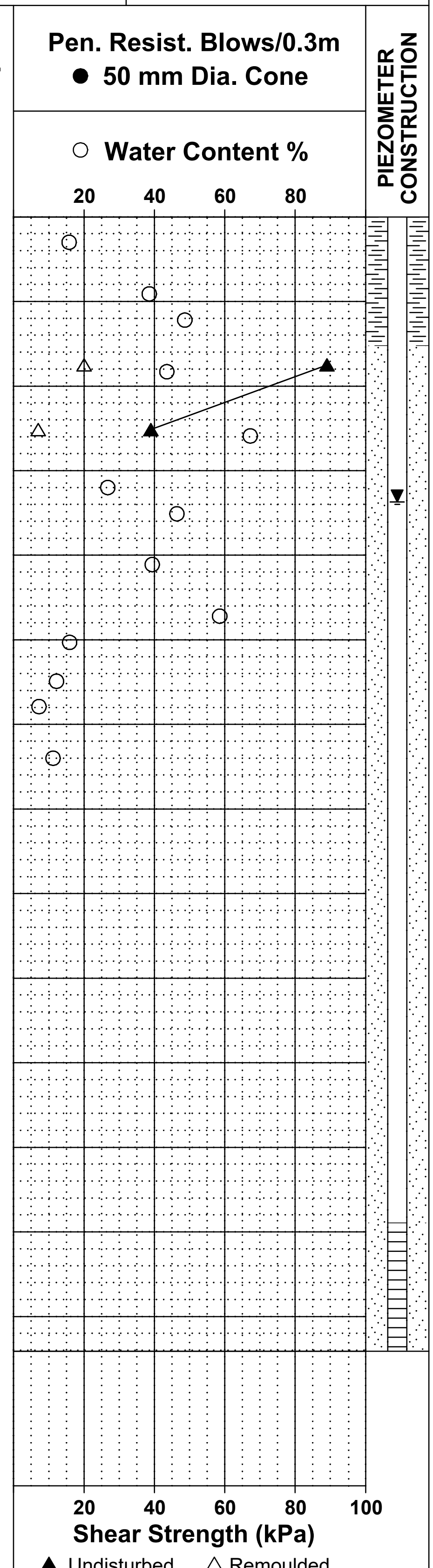
FILE NO. PG2119

HOLE NO. BH 5-24

SAMPLE DESCRIPTION	Depth to base (m)
Ground Surface	
FILL: Brown silty clay, trace topsoil, sand and gravel	0.94
Very stiff to firm brown **SILTY CLAY** - Trace sand by 1.5 m depth - Grey by 3.1 m depth	3.35
Very loose grey **SILT**, some clay and sand	4.88
GLACIAL TILL: Loose grey silty sand, some clay, trace gravel	5.79
GLACIAL TILL: Dense grey silty sand with cobbles and boulders, some gravel, trace clay - Boulder content increasing with depth	8.81
BEDROCK: Good to excellent quality grey sandstone bedrock with interbedded dolomite	13.41
End of Borehole (GWL at 3.37 m - June 19, 2024)	

TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	100	6
SS	3	21	P
SS	4	100	P
SS	5	67	P
SS	6	100	2
SS	7	92	2
SS	8	50	6
SS	9	100	+50
RC	1	46	N/A
RC	2	100	75
RC	3	100	93
RC	4	100	92
RC	5	100	94

DEPTH (m)	ELEV. (m)
0	92.82
1	91.82
2	90.82
3	89.82
4	88.82
5	87.82
6	86.82
7	85.82
8	84.82
9	83.82
10	82.82
11	81.82
12	80.82
13	79.82



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation - Prop. Apartment Bldgs
Blocks 5, 8, and 10 - 609, 617, and 621 Longfields Dr
Ottawa, Ontario

EASTING: 363865.718 **NORTHING:** 5016320.859 **ELEVATION:** 93.14

DATUM: Geodetic

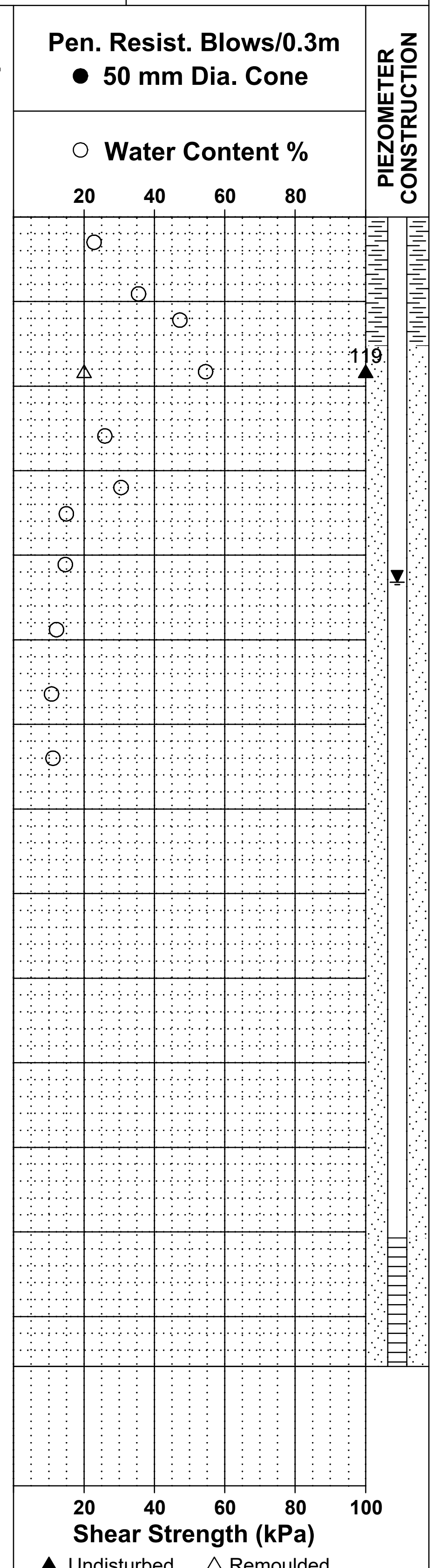
REMARKS:

BORINGS BY: CME-55 Low Clearance Drill **DATE:** June 5, 2024

FILE NO. PG2119

HOLE NO. BH 6-24

SAMPLE DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
Ground Surface					0	93.14
FILL: Brown silty clay, some topsoil and sand, trace gravel	AU	1				
0.97						
Hard to stiff brown **SILTY CLAY**	SS	2	92	10	1	92.14
	SS	3	100	P		
2.29					2	91.14
Very loose brown **SILT**, trace to some clay and sand	SS	4	50	P		
3.25					3	90.14
- Grey by 3.1 m depth	SS	5	100	2		
GLACIAL TILL: Very loose to very dense grey sandy silt with cobbles and boulders, some clay and gravel	SS	6	21	17	4	89.14
	SS	7	17	16	5	88.14
- Cobble and boulder content increasing with depth	SS	8	63	+50		
	SS	9	77	+50	6	87.14
	RC	1	37	N/A	7	86.14
8.10					8	85.14
BEDROCK: Good to excellent quality interbedded grey sandstone	RC	2	100	87	9	84.14
	RC	3	97	93	10	83.14
	RC	4	100	85	11	82.14
	RC	5	100	100	12	81.14
					13	80.14
13.59						
End of Borehole						
(GWL at 4.32 m - June 19, 2024)						



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation - Prop. Apartment Bldgs
Blocks 5, 8, and 10 - 609, 617, and 621 Longfields Dr
Ottawa, Ontario

EASTING: 363897.34 **NORTHING:** 5016294.67 **ELEVATION:** 93.10

DATUM: Geodetic

REMARKS:

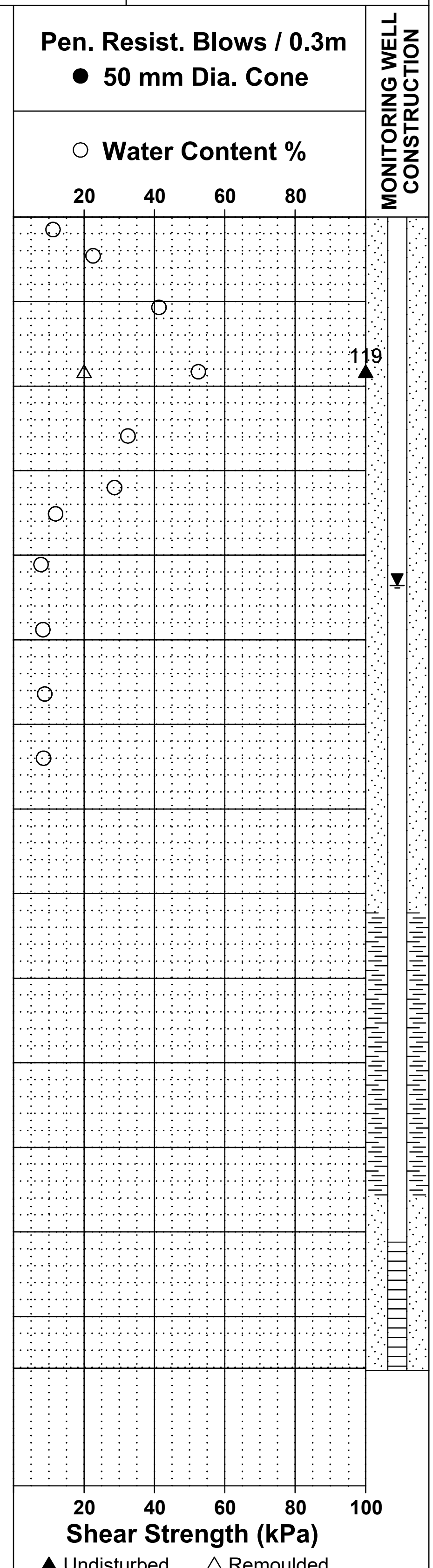
BORINGS BY: CME-55 Low Clearance Drill **DATE:** June 6, 2024

FILE NO. PG2119

HOLE NO. BH 7-24

SAMPLE DESCRIPTION	Depth to base (m)	TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
FILL: Brown silty sand with gravel and crushed stone	0.30	AU	1		
	0.69	AU	2		
FILL: Grey to brown silty clay, trace sand		SS	3	100	8
Hard to very stiff brown **SILTY CLAY**, trace sand	2.21	SS	4	100	P
Very loose brown **SILT**, some to trace clay and sand - Grey by 3.1 m depth	3.35	SS	5	92	P
		SS	6	88	3
GLACIAL TILL: Loose to dense grey sandy silt with cobbles and boulders, trace to some gravel, trace clay - Cobble and boulder content increasing with depth	8.48	SS	7	100	39
		SS	8	67	18
		SS	9	42	20
		SS	10	76	31
		RC	1	53	N/A
BEDROCK: Excellent quality interbedded grey sandstone and dolomite bedrock	13.61	RC	2	100	100
		RC	3	98	97
		RC	4	98	93
		RC	5	100	100
End of Borehole (GWL at 4.36 m - June 19, 2024)					

DEPTH (m)	ELEV. (m)
0	93.10
1	92.10
2	91.10
3	90.10
4	89.10
5	88.10
6	87.10
7	86.10
8	85.10
9	84.10
10	83.10
11	82.10
12	81.10
13	80.10



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9 Auriga Drive, Ottawa, Ontario K2E 7T9

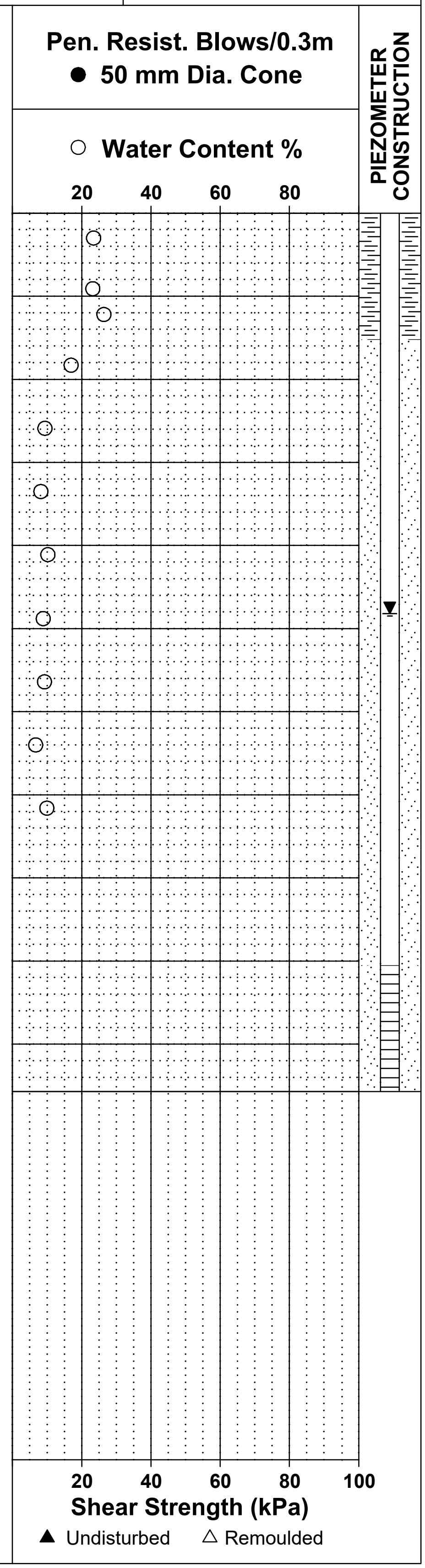
SOIL PROFILE AND TEST DATA

Geotechnical Investigation - Prop. Apartment Bldgs
Blocks 5, 8, and 10 - 609, 617, and 621 Longfields Dr
Ottawa, Ontario

EASTING: 363964.315 **NORTHING:** 5016170.381 **ELEVATION:** 93.81
DATUM: Geodetic
REMARKS:
BORINGS BY: CME-55 Low Clearance Drill **DATE:** June 7, 2024

FILE NO. PG2119
HOLE NO. BH 8-24

SAMPLE DESCRIPTION	TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
Ground Surface					0	93.81
FILL: Brown silty clay, trace sand, gravel, topsoil and crushed stone (to 0.91)	AU	1				
Loose brown **SANDY SILT**, some clay (to 2.13)	SS	2	100	5	1	92.81
	SS	3	71	3	2	91.81
GLACIAL TILL: Dense to compact brown silty sand to sandy silt with gravel, cobbles and boulders - Grey by 2.8 m depth (to 7.57)	SS	4	100	39	3	90.81
	SS	5	58	35		
	SS	6	21	23	4	89.81
	SS	7	58	11	5	88.81
	SS	8	63	17	6	87.81
	SS	9	83	18		
	SS	10	100	19	7	86.81
BEDROCK: Fair quality grey sandstone bedrock (to 10.57)	RC	1	100	74	8	85.81
	RC	2	94	61	9	84.81
					10	83.81
End of Borehole (GWL at 4.82 m - June 19, 2024)						



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9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation - Prop. Apartment Bldgs
Blocks 5, 8, and 10 - 609, 617, and 621 Longfields Dr
Ottawa, Ontario

EASTING: 363960.965 **NORTHING:** 5016126.036 **ELEVATION:** 93.95

DATUM: Geodetic

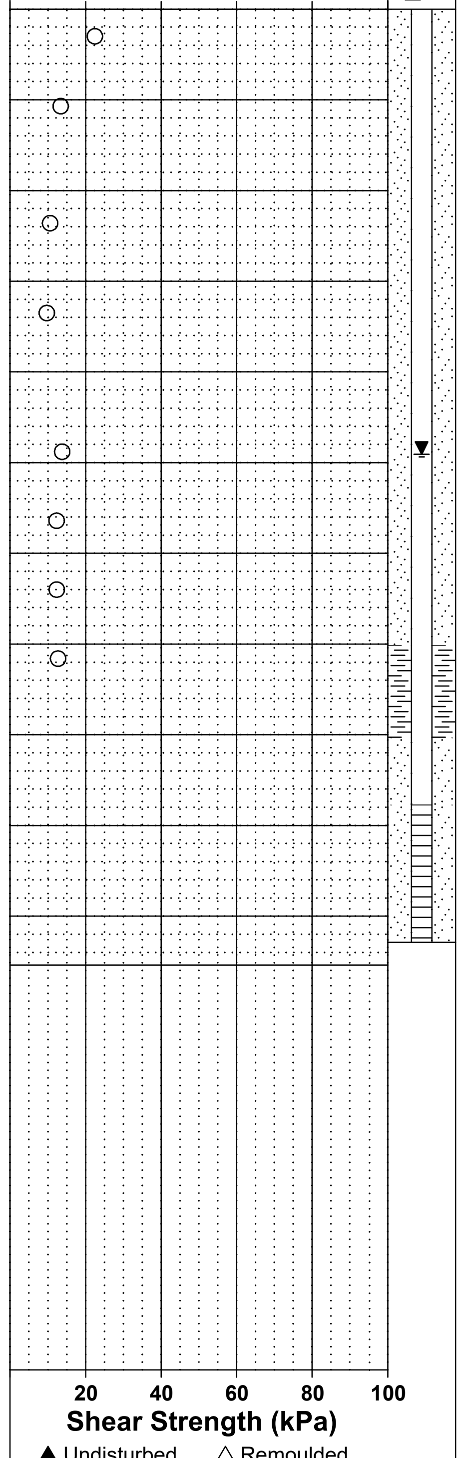
REMARKS:

BORINGS BY: CME-55 Low Clearance Drill **DATE:** June 7, 2024

FILE NO. PG2119

HOLE NO. BH 9-24

SAMPLE DESCRIPTION	STRATA PLOT	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	93.95
FILL: Brown silty clay, trace sand, gravel and crushed stone — 0.69		AU	1				
GLACIAL TILL: Very dense to loose brown silty sand to sandy silt with gravel, cobbles and boulders		SS	2	83	+50	1	92.95
		RC	1	38	N/A	2	91.95
- Grey by 4.6 m depth		SS	3	83	39	3	90.95
- Trace clay from 5.5 to 6.7 m depth		SS	4	42	13	4	89.95
		SS	5	0	4	5	88.95
		SS	6	21	4		
		SS	7	33	8	6	87.95
		SS	8	100	5	7	86.95
		SS	9	75	+50	8	85.95
— 8.53		RC	2	100	60		
BEDROCK: Fair quality interbedded grey sandstone and dolomite bedrock						9	84.95
— 10.54		RC	3	88	68	10	83.95
End of Borehole							
(GWL at 4.91 m - June 19, 2024)							



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9 Auriga Drive, Ottawa, Ontario K2E 7T9

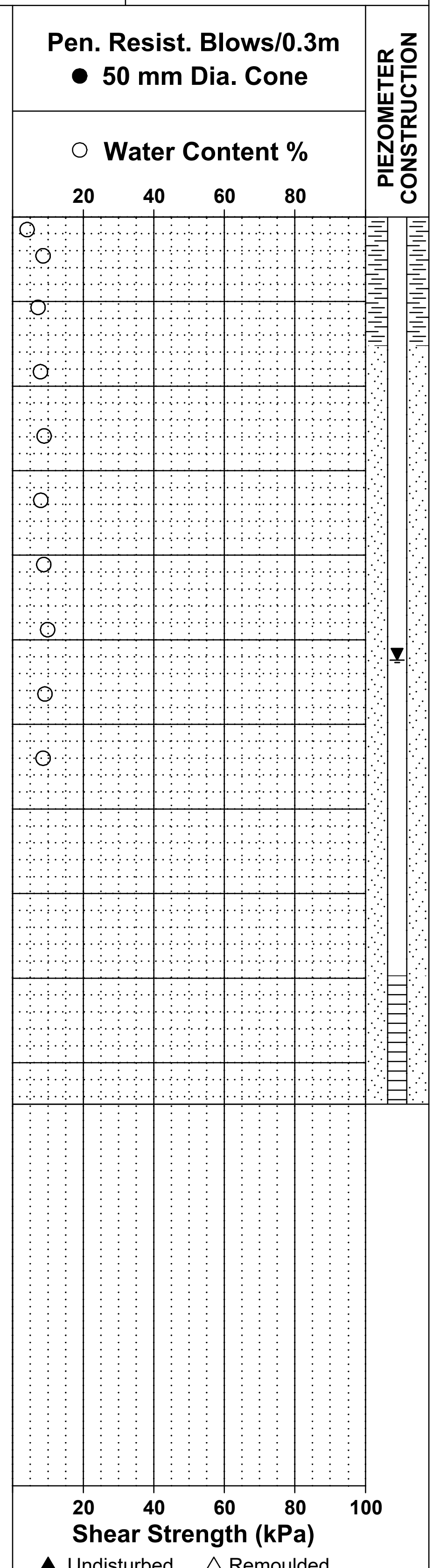
SOIL PROFILE AND TEST DATA

Geotechnical Investigation - Prop. Apartment Bldgs
Blocks 5, 8, and 10 - 609, 617, and 621 Longfields Dr
Ottawa, Ontario

EASTING: 364002.946 **NORTHING:** 5016063.185 **ELEVATION:** 94.34
DATUM: Geodetic
REMARKS:
BORINGS BY: CME-55 Low Clearance Drill **DATE:** June 10, 2024

FILE NO. PG2119
HOLE NO. BH10-24

SAMPLE DESCRIPTION	Strata depth (m)	TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
Ground Surface						0	94.34
FILL: Brown silty sand with crushed stone and gravel	0.46	AU	1				
FILL: Brown silty sand, trace gravel	0.69	AU	2				
GLACIAL TILL: Dense to compact brown silty sand with cobbles and boulders, trace to some gravel, trace clay - Grey by 3.1 m depth		SS	3	83	45	1	93.34
		SS	4	13	31	2	92.34
		SS	5	33	31	3	91.34
		SS	6	46	18	4	90.34
		SS	7	75	13	5	89.34
		SS	8	50	21	6	88.34
		SS	9	42	19		
	6.96	SS	10	92	18	7	87.34
BEDROCK: Excellent to fair quality interbedded grey sandstone and dolomite bedrock		RC	1	100	100	8	86.34
		RC	2	100	72	9	85.34
	10.49	RC	3	87	57	10	84.34
End of Borehole (GWL at 5.24 m - June 19, 2024)							



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9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation - Prop. Apartment Bldgs
Blocks 5, 8, and 10 - 609, 617, and 621 Longfields Dr
Ottawa, Ontario

EASTING: 364035.212 **NORTHING:** 5016060.659 **ELEVATION:** 94.26

DATUM: Geodetic

REMARKS:

BORINGS BY: CME-55 Low Clearance Drill **DATE:** June 10, 2024

FILE NO. PG2119

HOLE NO. BH11-24

SAMPLE DESCRIPTION	Depth to base (m)	TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	94.26
FILL: Brown silty sand wih crushed stone and gravel	0.25	AU	1				
FILL: Brown silty clay, trace sand, gravel and crushed stone	0.71	AU	2				
		SS	3	100	+50	1	93.26
GLACIAL TILL: Very dense to compact brown silty sand to sandy silt with cobbles and boulders, some gravel, trace clay		RC	1	29	N/A		
		SS	4	42	+50	2	92.26
		SS	5	29	12	3	91.26
		SS	6	0	+50		
		RC	2	48	N/A	4	90.26
		SS	7	21	15	5	89.26
		SS	8	8	28		
		SS	9	63	38	6	88.26
	7.04						
BEDROCK: Excellent to good quality interbedded grey sandstone and dolomite bedrock		RC	3	100	100	7	87.26
		RC	4	100	85	8	86.26
						9	85.26
	10.57	RC	5	95	90	10	84.26
End of Borehole							
(GWL at 5.14 m - June 19, 2024)							

Pen. Resist. Blows / 0.3m
● 50 mm Dia. Cone

○ Water Content %

20 40 60 80

MONITORING WELL CONSTRUCTION

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

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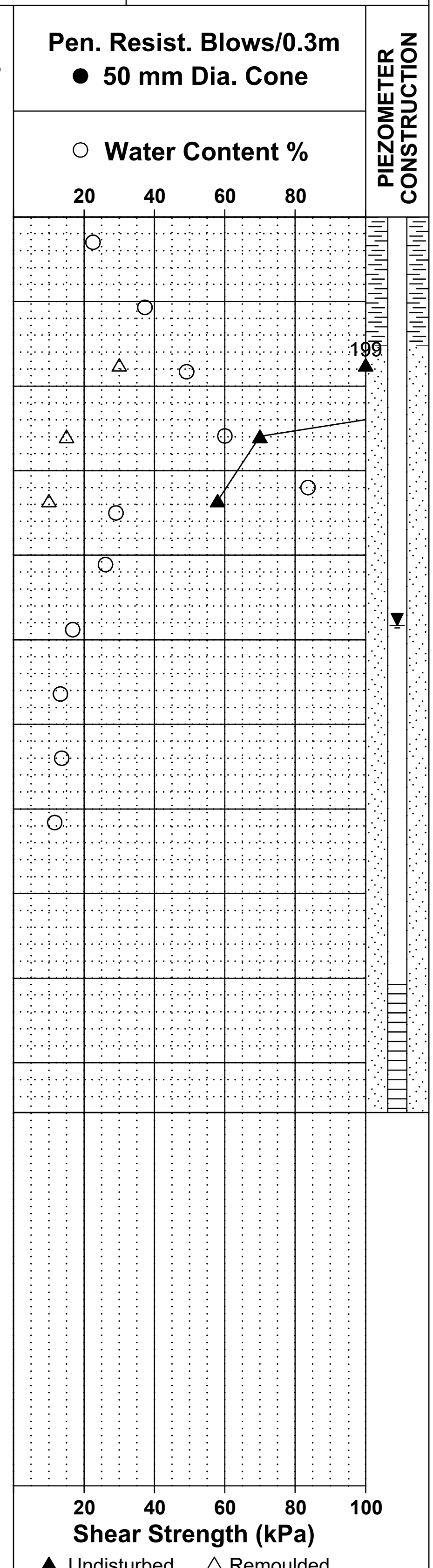
SOIL PROFILE AND TEST DATA

Geotechnical Investigation - Prop. Apartment Bldgs
Blocks 5, 8, and 10 - 609, 617, and 621 Longfields Dr
Ottawa, Ontario

EASTING: 364025.699 **NORTHING:** 5016102.869 **ELEVATION:** 93.91
DATUM: Geodetic
REMARKS:
BORINGS BY: CME-55 Low Clearance Drill **DATE:** June 11, 2024

FILE NO. PG2119
HOLE NO. BH12-24

SAMPLE DESCRIPTION	Layer depth (m)	TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
Ground Surface						0	93.91
FILL: Brown silty clay, trace crushed stone and sand	0.20	AU	1				
Hard to stiff brown **SILTY CLAY**		SS	2	100	13	1	92.91
		SS	3	75	P	2	91.91
		SS	4	100	P		
	3.56	SS	5	92	P	3	90.91
Loose brown **SILT**, trace sand and clay	3.73						
GLACIAL TILL: Very loose grey clayey silt, some sand, trace gravel		SS	6	100	2	4	89.91
	5.18	SS	7	100	1	5	88.91
GLACIAL TILL: Compact to very loose grey silty sand with cobbles and boulders, some gravel, trace clay		SS	8	38	16		
		SS	9	33	2	6	87.91
	6.99	SS	10	100	+50		
BEDROCK: Fair to good quality interbedded grey sandstone and dolomite bedrock		RC	1	100	77	7	86.91
- Vertical fracture from 7.2 to 7.8 m depth		RC	2	100	87	8	85.91
- Thin mud seam at 9.0 m depth						9	84.91
	10.59	RC	3	95	79	10	83.91
End of Borehole							
(GWL at 4.83 m - June 19, 2024)							



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9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation - Prop. Apartment Bldgs
Blocks 5, 8, and 10 - 609, 617, and 621 Longfields Dr
Ottawa, Ontario

EASTING: 364012.48 **NORTHING:** 5016145.414 **ELEVATION:** 93.90

DATUM: Geodetic

REMARKS:

BORINGS BY: CME-55 Low Clearance Drill **DATE:** June 11, 2024

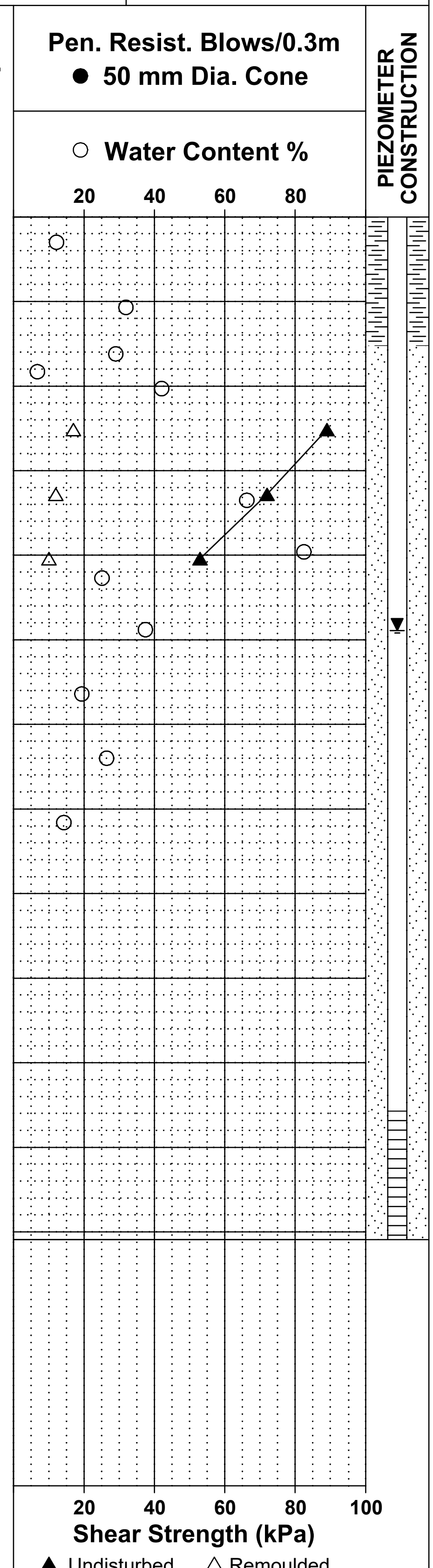
FILE NO. PG2119

HOLE NO. BH13-24

SAMPLE DESCRIPTION	Depth to base (m)
Ground Surface	
FILL: Brown silty clay, trace sand, topsoil, gravel, crushed stone and cobbles	1.68
FILL: Loose brown silty sand, trace gravel	1.83
Very stiff to stiff brown **SILTY CLAY** - Grey by 3.81 m depth	4.34
Very loose grey **SILT**, some clay, trace sand	5.26
GLACIAL TILL: Very loose grey silty sand, some clay, trace gravel	7.21
BEDROCK: Excellent quality interbedded grey sandstone and dolomite bedrock - Very poor quality from 9.6 to 10.6 m depth - Excellent quality again by 10.6 m depth	12.09
End of Borehole (GWL at 4.89 m - June 19, 2024)	

TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	17	7
SS	3	71	4
SS	4	0	P
SS	5	100	P
SS	6	96	P
SS	7	100	1
SS	8	83	1
SS	9	100	1
SS	10	73	+50
RC	1	100	92
RC	2	100	92
RC	3	87	47
RC	4	100	100

DEPTH (m)	ELEV. (m)
0	93.90
1	92.90
2	91.90
3	90.90
4	89.90
5	88.90
6	87.90
7	86.90
8	85.90
9	84.90
10	83.90
11	82.90
12	81.90



SYMBOLS AND TERMS

SOIL DESCRIPTION

Behavioural properties, such as structure and strength, take precedence over particle gradation in describing soils. Terminology describing soil structure are as follows:

Desiccated	-	having visible signs of weathering by oxidation of clay minerals, shrinkage cracks, etc.
Fissured	-	having cracks, and hence a blocky structure.
Varved	-	composed of regular alternating layers of silt and clay.
Stratified	-	composed of alternating layers of different soil types, e.g. silt and sand or silt and clay.
Well-Graded	-	Having wide range in grain sizes and substantial amounts of all intermediate particle sizes (see Grain Size Distribution).
Uniformly-Graded	-	Predominantly of one grain size (see Grain Size Distribution).

The standard terminology to describe the relative strength of cohesionless soils is the compactness condition, usually inferred from the results of the Standard Penetration Test (SPT) 'N' value. The SPT N value is the number of blows of a 63.5 kg hammer, falling 760 mm, required to drive a 51 mm O.D. split spoon sampler 300 mm into the soil after an initial penetration of 150 mm. An SPT N value of "P" denotes that the split-spoon sampler was pushed 300 mm into the soil without the use of a falling hammer.

Compactness Condition	'N' Value	Relative Density %
Very Loose	<4	<15
Loose	4-10	15-35
Compact	10-30	35-65
Dense	30-50	65-85
Very Dense	>50	>85

The standard terminology to describe the strength of cohesive soils is the consistency, which is based on the undisturbed undrained shear strength as measured by the in situ or laboratory shear vane tests, unconfined compression tests, or occasionally by the Standard Penetration Test (SPT). Note that the typical correlations of undrained shear strength to SPT N value (tabulated below) tend to underestimate the consistency for sensitive silty clays, so Paterson reviews the applicable split spoon samples in the laboratory to provide a more representative consistency value based on tactile examination.

Consistency	Undrained Shear Strength (kPa)	'N' Value
Very Soft	<12	<2
Soft	12-25	2-4
Firm	25-50	4-8
Stiff	50-100	8-15
Very Stiff	100-200	15-30
Hard	>200	>30

SOIL DESCRIPTION (continued)

Cohesive soils can also be classified according to their "sensitivity". The sensitivity, S_t, is the ratio between the undisturbed undrained shear strength and the remoulded undrained shear strength of the soil. The classes of sensitivity may be defined as follows:

Low Sensitivity:	$S_t < 2$
Medium Sensitivity:	$2 < S_t < 4$
Sensitive:	$4 < S_t < 8$
Extra Sensitive:	$8 < S_t < 16$
Quick Clay:	$S_t > 16$

ROCK DESCRIPTION

The structural description of the bedrock mass is based on the Rock Quality Designation (RQD).

The RQD classification is based on a modified core recovery percentage in which all pieces of sound core over 100 mm long are counted as recovery. The smaller pieces are considered to be a result of closely-spaced discontinuities (resulting from shearing, jointing, faulting, or weathering) in the rock mass and are not counted. RQD is ideally determined from NQ or larger size core. However, it can be used on smaller core sizes, such as BQ, if the bulk of the fractures caused by drilling stresses (called "mechanical breaks") are easily distinguishable from the normal in situ fractures.

RQD %	**ROCK QUALITY**
90-100	Excellent, intact, very sound
75-90	Good, massive, moderately jointed or sound
50-75	Fair, blocky and seamy, fractured
25-50	Poor, shattered and very seamy or blocky, severely fractured
0-25	Very poor, crushed, very severely fractured

SAMPLE TYPES

SS	-	Split spoon sample (obtained in conjunction with the performing of the Standard Penetration Test (SPT))
TW	-	Thin wall tube or Shelby tube, generally recovered using a piston sampler
G	-	"Grab" sample from test pit or surface materials
AU	-	Auger sample or bulk sample
WS	-	Wash sample
RC	-	Rock core sample (Core bit size BQ, NQ, HQ, etc.). Rock core samples are obtained with the use of standard diamond drilling bits.

SYMBOLS AND TERMS (continued)

PLASTICITY LIMITS AND GRAIN SIZE DISTRIBUTION

WC%	-	Natural water content or water content of sample, %
LL	-	Liquid Limit, % (water content above which soil behaves as a liquid)
PL	-	Plastic Limit, % (water content above which soil behaves plastically)
PI	-	Plasticity Index, % (difference between LL and PL)
Dxx	-	Grain size at which xx% of the soil, by weight, is of finer grain sizes These grain size descriptions are not used below 0.075 mm grain size
D10	-	Grain size at which 10% of the soil is finer (effective grain size)
D60	-	Grain size at which 60% of the soil is finer
Cc	-	Concavity coefficient = $(D30)^2$ / (D10 x D60)
Cu	-	Uniformity coefficient = D60 / D10

Cc and Cu are used to assess the grading of sands and gravels:
Well-graded gravels have: 1 < Cc < 3 and Cu > 4
Well-graded sands have: 1 < Cc < 3 and Cu > 6
Sands and gravels not meeting the above requirements are poorly-graded or uniformly-graded.
Cc and Cu are not applicable for the description of soils with more than 10% silt and clay (more than 10% finer than 0.075 mm or the #200 sieve)

CONSOLIDATION TEST

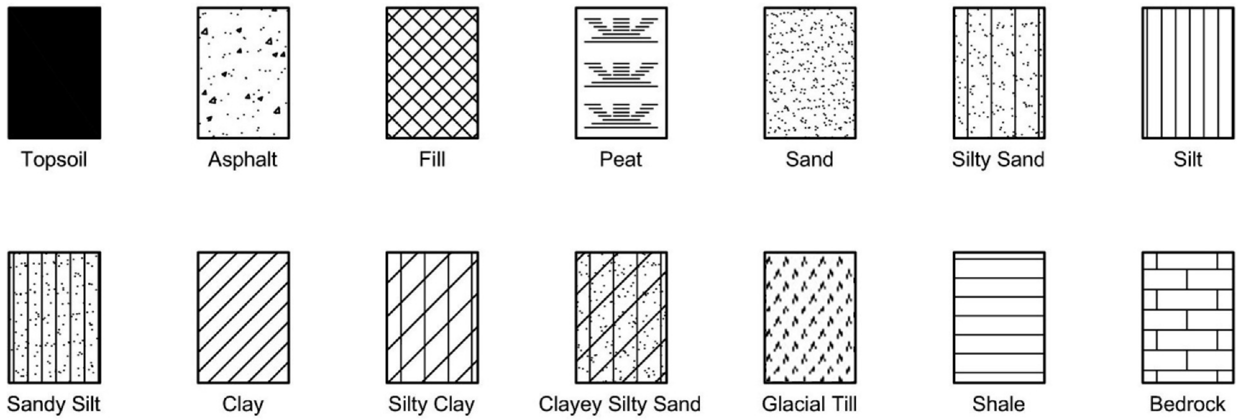
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void Ratio		Initial sample void ratio = volume of voids / volume of solids
Wo	-	Initial water content (at start of consolidation test)

PERMEABILITY TEST

k	-	Coefficient of permeability or hydraulic conductivity is a measure of the ability of water to flow through the sample. The value of k is measured at a specified unit weight for (remoulded) cohesionless soil samples, because its value will vary with the unit weight or density of the sample during the test.

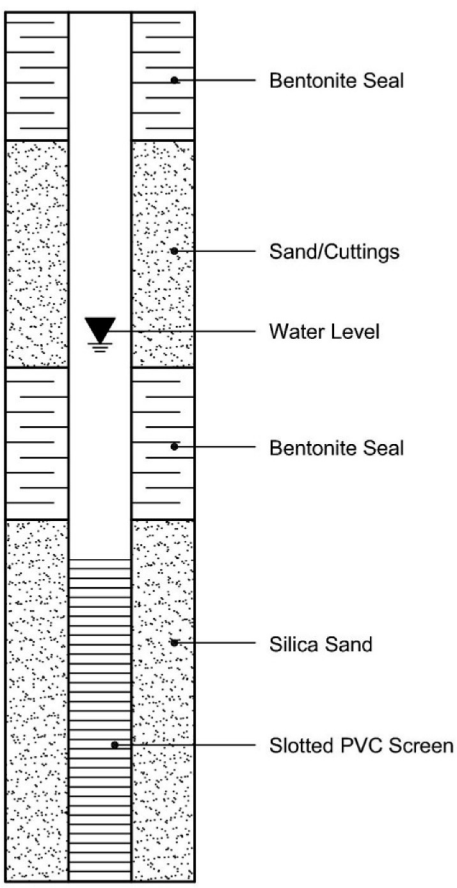
SYMBOLS AND TERMS (continued)

STRATA PLOT

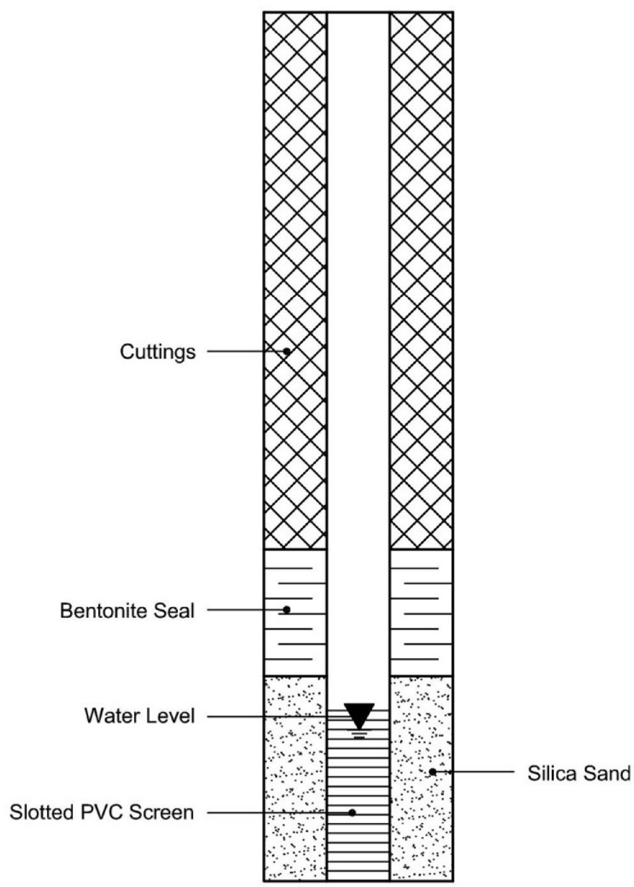


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



PATERSON GROUP

Photograph of Rock Cores – BH1-24 – RC1

Photograph of Rock Core obtained from BH 1-24 from interval RC1

Rock Core interval ranged between 19’11” to 24’6”

Recovery (%) = 58

Rock Quality Designation (RQD - %) = N/A - Boulder

Photograph of Rock Cores – BH1-24 – RC2

Photograph of Rock Core obtained from BH 1-24 from interval RC2

Rock Core interval ranged between 24’6” to 29’10”

Recovery (%) = 93

Rock Quality Designation (RQD - %) = 36

Photograph of Rock Cores – BH1-24 – RC3

Photograph of Rock Core obtained from BH 1-24 from interval RC3

Rock Core interval ranged between 29’10” to 34’9”

Recovery (%) = 100

Rock Quality Designation (RQD - %) = 36

Photograph of Rock Cores – BH2-24 – RC1

Photograph of Rock Core obtained from BH 2-24 from interval RC1

Rock Core interval ranged between 19’10” to 24’7”

Recovery (%) = 100

Rock Quality Designation (RQD - %) = 93

Photograph of Rock Cores – BH2-24 – RC2

Photograph of Rock Core obtained from BH 2-24 from interval RC2

Rock Core interval ranged between 24'7" to 29'4"

Recovery (%) = 100

Rock Quality Designation (RQD) = 91

Photograph of Rock Cores – BH2-24 – RC3

Photograph of Rock Core obtained from BH 2-24 from interval RC3

Rock Core interval ranged between 29'4" to 34'5"

Recovery (%) = 100

Rock Quality Designation (RQD) = 90

Photograph of Rock Cores – BH3-24 – RC1

Photograph of Rock Core obtained from BH 3-24 from interval RC1

Rock Core interval ranged between 20’4” to 25’1”

Recovery (%) = 68

Rock Quality Designation (RQD) = N/A - Boulder

Photograph of Rock Cores – BH3-24 – RC2

Photograph of Rock Core obtained from BH 3-24 from interval RC2

Rock Core interval ranged between 25’1” to 30’0”

Recovery (%) = 100

Rock Quality Designation (RQD) = 50

Photograph of Rock Cores – BH3-24 – RC3

Photograph of Rock Core obtained from BH 3-24 from interval RC3

Rock Core interval ranged between 30'0" to 34'10"

Recovery (%) = 100

Rock Quality Designation (RQD) = 96

Photograph of Rock Cores – BH4-24 – RC1

Photograph of Rock Core obtained from BH 4-24 from interval RC1

Rock Core interval ranged between 24’0” to 29’7”

Recovery (%) = 100

Rock Quality Designation (RQD) = 74

PATERSON GROUP

Photograph of Rock Cores – BH4-24 – RC2

Photograph of Rock Core obtained from BH 4-24 from interval RC2

Rock Core interval ranged between 29’7” to 34’6”

Recovery (%) = 100

Rock Quality Designation (RQD) = 93

Photograph of Rock Cores – BH4-24 – RC3

Photograph of Rock Core obtained from BH 4-24 from interval RC3

Rock Core interval ranged between 34’6” to 39’5”

Recovery (%) = 100

Rock Quality Designation (RQD) = 97

Photograph of Rock Cores – BH4-24 – RC4

Photograph of Rock Core obtained from BH 4-24 from interval RC4

Rock Core interval ranged between 39’5” to 44’4”

Recovery (%) = 100

Rock Quality Designation (RQD) = 100

Photograph of Rock Cores – BH5-24 – RC1 and RC 2

Photograph of Rock Core obtained from BH 5-24 from interval RC1 and RC2

Rock Core RC1 interval ranged between 20’2” to 24’11”; Rock Core RC2 interval ranged between 24’11” to 29’7”

Recovery RC1 (%) = 46; Recovery RC2 (%) = 100

Rock Quality Designation RC1 (RQD) = N/A – Boulder; Rock Quality Designation RC2 (RQD) = 75

Photograph of Rock Cores – BH5-24 – RC3

Photograph of Rock Core obtained from BH 5-24 from interval RC3

Rock Core interval ranged between 29’7” to 34’4”

Recovery (%) = 100

Rock Quality Designation (RQD) = 93

Photograph of Rock Cores – BH5-24 – RC4

Photograph of Rock Core obtained from BH 5-24 from interval RC4

Rock Core interval ranged between 34’4” to 39’4”

Recovery (%) = 100

Rock Quality Designation (RQD) = 92

Photograph of Rock Cores – BH5-24 – RC5

Photograph of Rock Core obtained from BH 5-24 from interval RC5

Rock Core interval ranged between 39'4" to 44'0"

Recovery (%) = 100

Rock Quality Designation (RQD) = 94

Photograph of Rock Cores – BH6-24 – RC1 and RC2

Photograph of Rock Core obtained from BH 6-24 from interval RC1 and RC2

Rock Core RC1 interval ranged between 19’11” to 24’11”; Rock Core RC2 interval ranged between 24’11” to 29’10”

Recovery RC1 (%) = 37; Recovery RC2 (%) = 100

Rock Quality Designation RC1 (RQD) = N/A – Boulder; Rock Quality Designation RC2 (RQD) = 87

Photograph of Rock Cores – BH6-24 – RC3

Photograph of Rock Core obtained from BH 6-24 from interval RC3

Rock Core interval ranged between 29’10” to 34’9”

Recovery (%) = 97

Rock Quality Designation (RQD) = 93

Photograph of Rock Cores – BH6-24 – RC4

Photograph of Rock Core obtained from BH 6-24 from interval RC4

Rock Core interval ranged between 34’9” to 39’8”

Recovery (%) = 100

Rock Quality Designation (RQD) = 85

Photograph of Rock Cores – BH6-24 – RC5

Photograph of Rock Core obtained from BH 6-24 from interval RC5

Rock Core interval ranged between 39’8” to 44’7”

Recovery (%) = 100

Rock Quality Designation (RQD) = 100

Photograph of Rock Cores – BH7-24 – RC1 and RC2

Photograph of Rock Core obtained from BH 7-24 from interval RC1 and RC2

Rock Core RC1 interval ranged between 21’10” to 25’0”; Rock Core RC2 interval ranged between 25’0” to 29’11”

Recovery RC1 (%) = 53; Recovery RC2 (%) = 100

Rock Quality Designation RC1 (RQD) = N/A – Boulder; Rock Quality Designation RC2 (RQD) = 100

Photograph of Rock Cores – BH7-24 – RC3

Photograph of Rock Core obtained from BH 7-24 from interval RC3

Rock Core interval ranged between 29’11” to 34’10”

Recovery (%) = 98

Rock Quality Designation (RQD) = 97

Photograph of Rock Cores – BH7-24 – RC4

Photograph of Rock Core obtained from BH 7-24 from interval RC4

Rock Core interval ranged between 34’10” to 39’9”

Recovery (%) = 98

Rock Quality Designation (RQD) = 93

Photograph of Rock Cores – BH7-24 – RC5

Photograph of Rock Core obtained from BH 7-24 from interval RC5

Rock Core interval ranged between 39'9" to 44'8"

Recovery (%) = 100

Rock Quality Designation (RQD) = 100

Photograph of Rock Cores – BH8-24 – RC1

Photograph of Rock Core obtained from BH 8-24 from interval RC1

Rock Core interval ranged between 25’1” to 29’9”

Recovery (%) = 100

Rock Quality Designation (RQD) = 74

Photograph of Rock Cores – BH8-24 – RC2

Photograph of Rock Core obtained from BH 8-24 from interval RC2

Rock Core interval ranged between 29'9" to 34'8"

Recovery (%) = 94

Rock Quality Designation (RQD) = 61

Photograph of Rock Cores – BH9-24 – RC1 and RC2

Photograph of Rock Core obtained from BH 9-24 from interval RC1 and RC2

Rock Core RC1 interval ranged between 4’9” to 6’9”; Rock Core RC2 interval ranged between 24’4” to 29’8”

Recovery RC1 (%) = 38; Recovery RC2 (%) = 100

Rock Quality Designation RC1 (RQD) = N/A – Boulder; Rock Quality Designation RC2 (RQD) = 60

Photograph of Rock Cores – BH9-24 – RC3

Photograph of Rock Core obtained from BH 9-24 from interval RC3

Rock Core interval ranged between 29’8” to 34’7”

Recovery (%) = 88

Rock Quality Designation (RQD) = 68

Photograph of Rock Cores – BH10-24 – RC1 and RC2

Photograph of Rock Core obtained from BH 10-24 from interval RC1 and RC2

Rock Core RC1 interval ranged between 22’5” to 24’9”; Rock Core RC2 interval ranged between 24’9” to 29’6”

Recovery RC1 (%) = 100; Recovery RC2 (%) = 100

Rock Quality Designation RC1 (RQD) = 100; Rock Quality Designation RC2 (RQD) = 72

Photograph of Rock Cores – BH10-24 – RC3

Photograph of Rock Core obtained from BH 10-24 from interval RC3

Rock Core interval ranged between 29’6” to 34’5”

Recovery (%) = 87

Rock Quality Designation (RQD) = 57

PATERSON GROUP

Photograph of Rock Cores – BH11-24 – RC1 to RC3

Photograph of Rock Core obtained from BH 11-24 from interval RC1 to RC3

Rock Core RC1 interval ranged between 3’11” to 6’6”; Rock Core RC2 interval ranged between 12’11” to 14’6”; Rock Core RC3 interval ranged between 20’11” to 24’10”

Recovery RC1 (%) = 29; Recovery RC2 (%) = 48; Recovery RC3 (%) = 100

Rock Quality Designation RC1 (RQD) = N/A – Boulder; Rock Quality Designation RC2 (RQD) = N/A - Boulder

Rock Quality Designation RC3 (RQD) = 100

Photograph of Rock Cores – BH11-24 – RC4

Photograph of Rock Core obtained from BH 11-24 from interval RC4

Rock Core interval ranged between 24'10" to 29'9"

Recovery (%) = 100

Rock Quality Designation (RQD) = 85

Photograph of Rock Cores – BH11-24 – RC5

Photograph of Rock Core obtained from BH 11-24 from interval RC5

Rock Core interval ranged between 29’9” to 34’8”

Recovery (%) = 95

Rock Quality Designation (RQD) = 90

Photograph of Rock Cores – BH12-24 – RC1 and RC2

Photograph of Rock Core obtained from BH 12-24 from interval RC1 and RC2

Rock Core RC1 interval ranged between 22’11” to 24’9”; Rock Core RC2 interval ranged between 24’9” to 29’9”

Recovery RC1 (%) = 100; Recovery RC2 (%) = 100

Rock Quality Designation RC1 (RQD) = 77; Rock Quality Designation RC2 (RQD) = 87

Photograph of Rock Cores – BH12-24 – RC3

Photograph of Rock Core obtained from BH 12-24 from interval RC3

Rock Core interval ranged between 29’9” to 34’9”

Recovery (%) = 95

Rock Quality Designation (RQD) = 79

Photograph of Rock Cores – BH13-24 – RC1 and RC2

Photograph of Rock Core obtained from BH 13-24 from interval RC1 and RC2

Rock Core RC1 interval ranged between 23’8” to 24’9”; Rock Core RC2 interval ranged between 24’9” to 29’9”

Recovery RC1 (%) = 100; Recovery RC2 (%) = 100

Rock Quality Designation RC1 (RQD) = 92; Rock Quality Designation RC2 (RQD) = 92

Photograph of Rock Cores – BH13-24 – RC3

Photograph of Rock Core obtained from BH 13-24 from interval RC3

Rock Core interval ranged between 29'9" to 34'9"

Recovery (%) = 187

Rock Quality Designation (RQD) = 47

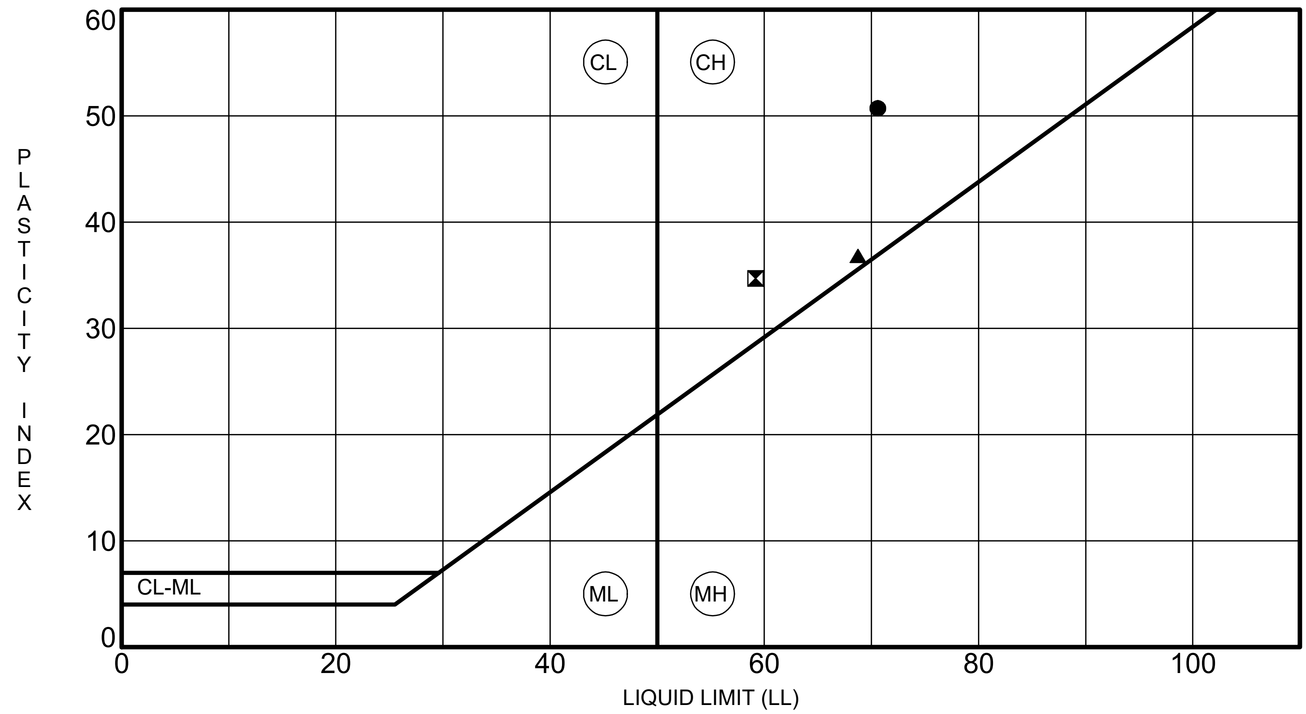
Photograph of Rock Cores – BH13-24 – RC4

Photograph of Rock Core obtained from BH 13-24 from interval RC4

Rock Core interval ranged between 34’9” to 39’8”

Recovery (%) = 100

Rock Quality Designation (RQD) = 100



	Specimen Identification		LL	PL	PI	Fines	Classification
●	**BH 2-24**	**SS2**	**71**	**20**	**51**		**CH - Inorganics clays of high plasticity**
⧗	**BH 7-24**	**SS3**	**59**	**24**	**35**		**CH - Inorganics clays of high plasticity**
▲	**BH12-24**	**SS3**	**69**	**32**	**37**		**CH - Inorganics clays of high plasticity**

CLIENT **Campanale Homes**

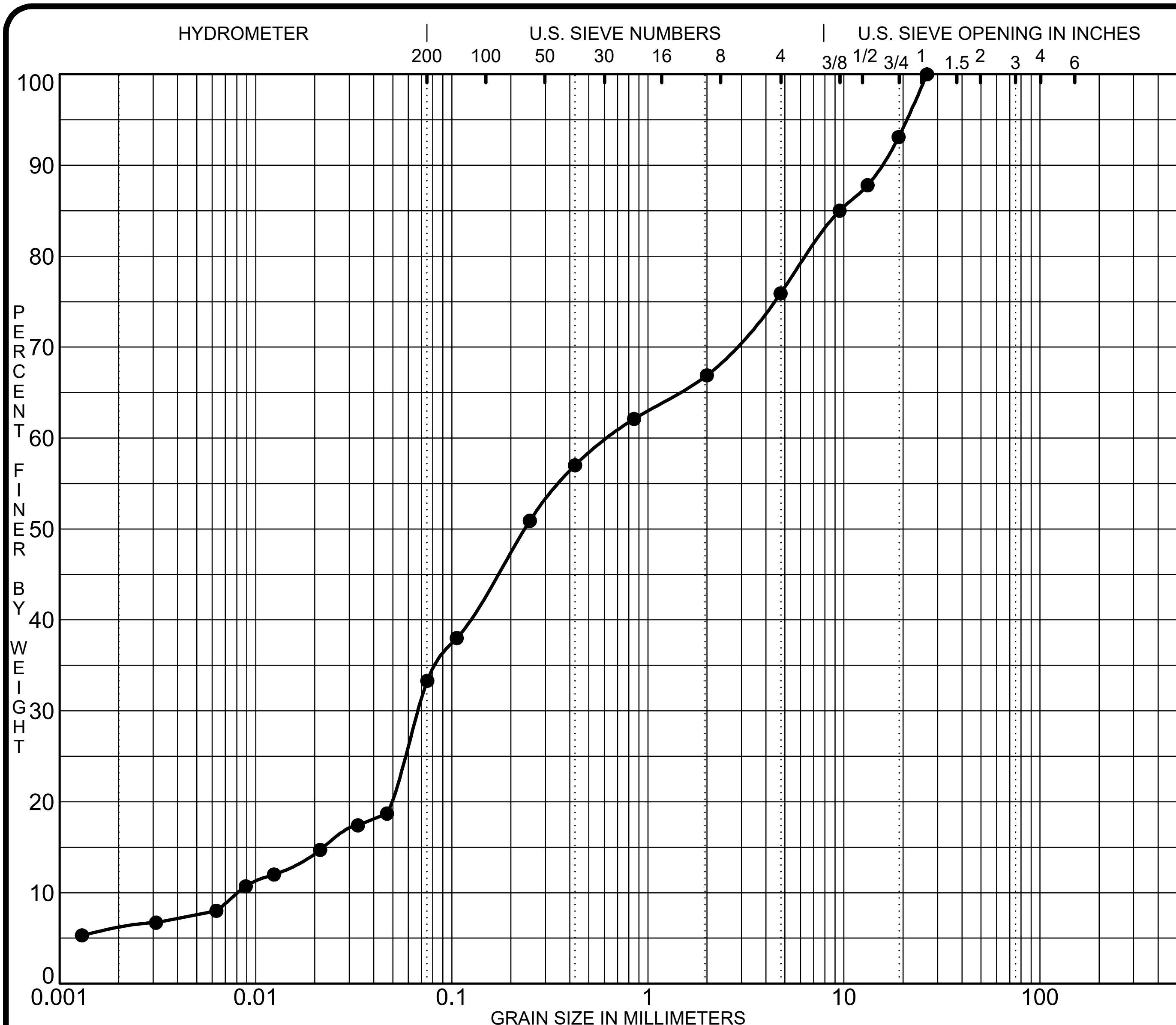
FILE NO. **PG2119**

PROJECT **Geotechnical Investigation - Prop. Apartment Bldgs - Blocks 5, 8, and 10 - 609, 617, and 621 Longfields Dr.**

DATE **11 Jun 24**

patersongroup **Consulting Engineers**
9 Auriga Drive, Ottawa, Ontario K2E 7T9

ATTERBERG LIMITS' RESULTS



CLAY	SILT	SAND			GRAVEL		COBBLES
		fine	medium	coarse	fine	coarse	

	Specimen Identification		Classification	MC%	LL	PL	PI	Cc	Cu
●	**BH 6-24**	**SS 8**		**9.4**				**0.87**	**78.5**
⊠									
▲									
★									

	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**BH 6-24**	**SS 8**	**26.50**	**0.64**	**0.067**	**0.0081**	**24.1**	**42.6**	**27.4**	**5.8**
⊠										
▲										
★										

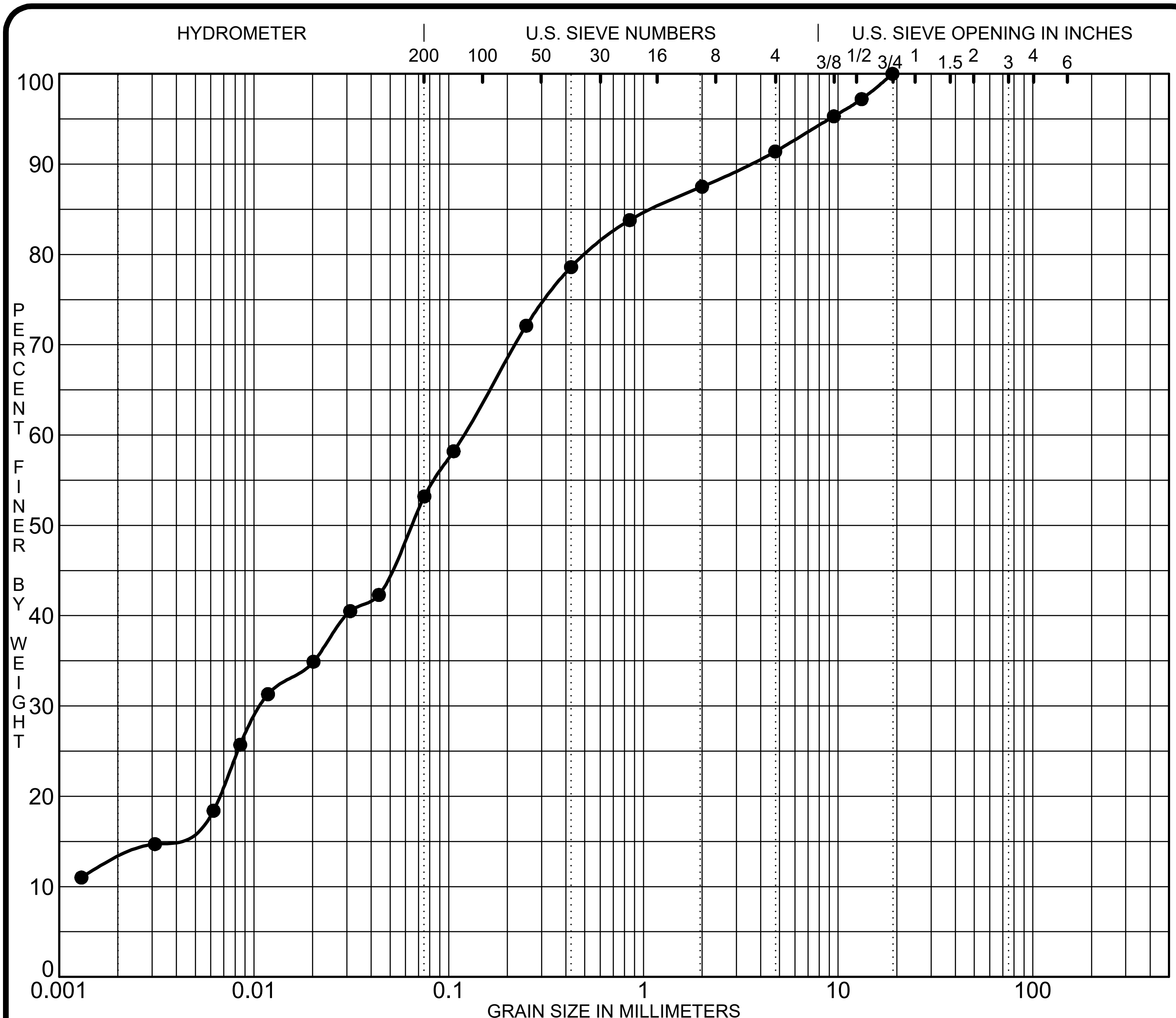
CLIENT **Campanale Homes** FILE NO. **PG2119**

PROJECT **Geotechnical Investigation - Prop. Apartment Bldgs - 609, 617, and 621 Longfields Drive** DATE **5 Jun 24**

patersongroup **Consulting Engineers**

9 Auriga Drive, Ottawa, Ontario K2E 7T9

GRAIN SIZE DISTRIBUTION



CLAY	SILT	SAND			GRAVEL		COBBLES
		fine	medium	coarse	fine	coarse	

	Specimen Identification		Classification	MC%	LL	PL	PI	Cc	Cu
●	**BH13-24**	**SS 8**		**17.8**					
⊠									
▲									
★									

	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**BH13-24**	**SS 8**	**19.00**	**0.12**	**0.011**		**8.6**	**38.2**	**40.7**	**12.5**
⊠										
▲										
★										

CLIENT	**Campanale Homes**	FILE NO.	**PG2119**
PROJECT	**Geotechnical Investigation - Prop. Apartment**	DATE	**11 Jun 24**
	Bldgs - 609, 617, and 621 Longfields Drive		

patersongroup **Consulting Engineers**

9 Auriga Drive, Ottawa, Ontario K2E 7T9

GRAIN SIZE DISTRIBUTION

PARACEL
LABORATORIES LTD.

Order #: 2423572

Certificate of Analysis

Report Date: 13-Jun-2024

Client: Paterson Group Consulting Engineers (Ottawa)

Order Date: 7-Jun-2024

Client PO: 60229

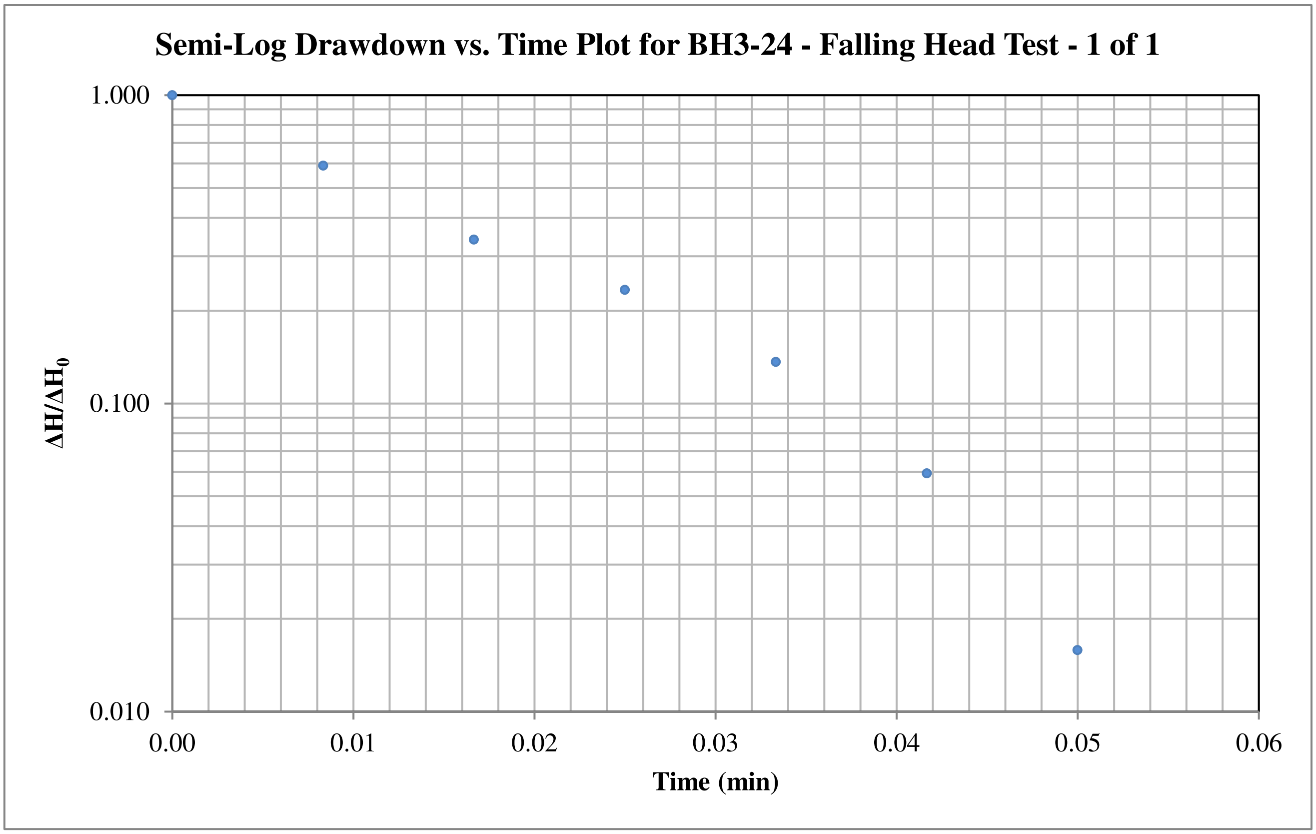
Project Description: PG2119

	Client ID:	BH1-24-SS3	-	-	-	-	-
	Sample Date:	07-Jun-24 09:00	-	-	-		
	Sample ID:	2423572-01	-	-	-		
	Matrix:	Soil	-	-	-		
	MDL/Units						
Physical Characteristics							
% Solids	0.1 % by Wt.	69.8	-	-	-	-	-
General Inorganics							
pH	0.05 pH Units	6.81	-	-	-	-	-
Resistivity	0.1 Ohm.m	35.9	-	-	-	-	-
Anions							
Chloride	10 ug/g	55	-	-	-	-	-
Sulphate	10 ug/g	123	-	-	-	-	-

File: PG2119

Hvorslev Hydraulic Conductivity Analysis

Project: Campanale Homes - 609, 617 and 621 Longfields Drive
Test Location: BH3-24
Test: Falling Head - 1 of 1
Date: June 19, 2024



Hvorslev Horizontal Hydraulic Conductivity

$$K = \frac{\pi r_c^2}{F}\frac{1}{t^*}\ln\left(\frac{\Delta H^*}{\Delta H_0}\right)$$

Hvorslev Shape Factor

$$F = \frac{2\pi L}{\ln\left(\frac{2L}{D}\right)}$$

Valid for L>>D

Hvorslev Shape Factor F: 2.07207

Well Parameters:

L	1.5	m	Saturated length of screen or open hole
D	0.03175	m	Diameter of well
r_c	0.01588	m	Radius of well

Data Points (from plot):

t*: 0.016 minutes $\Delta H^*/\Delta H_0$: 0.37

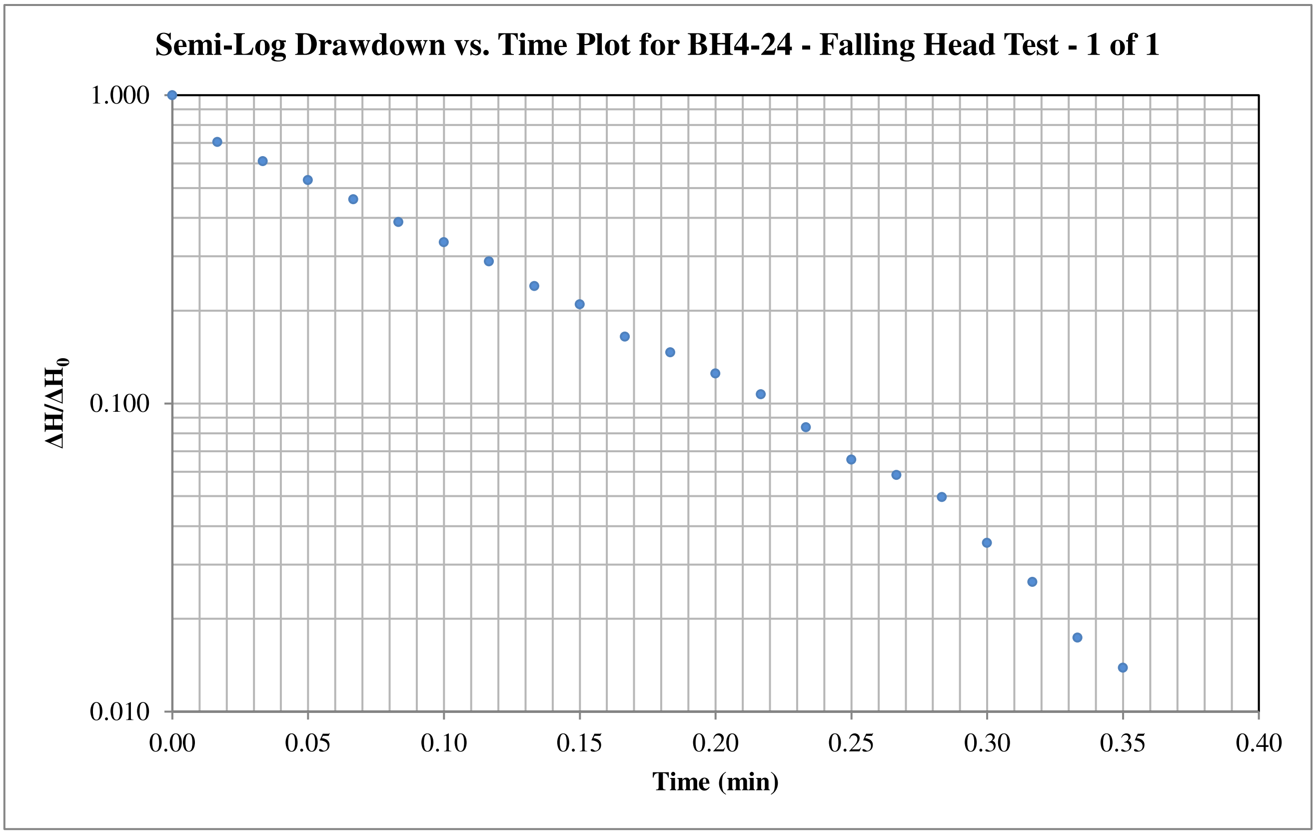
Horizontal Hydraulic Conductivity
K = 4.04E-04 m/sec

PATERSON GROUP

File: PG2119

Hvorslev Hydraulic Conductivity Analysis

Project: Campanale Homes - 609, 617 and 621 Longfields Drive
Test Location: BH4-24
Test: Falling Head - 1 of 1
Date: June 19, 2024



Hvorslev Horizontal Hydraulic Conductivity

$$K = \frac{\pi r_c^2}{F} \frac{1}{t^*} \ln\left(\frac{\Delta H^*}{\Delta H_0}\right)$$

Hvorslev Shape Factor

$$F = \frac{2\pi L}{\ln\left(\frac{2L}{D}\right)}$$

Valid for L>>D

Hvorslev Shape Factor F: 2.07207

Well Parameters:

L	1.5	m	Saturated length of screen or open hole
D	0.03175	m	Diameter of well
r_c	0.01588	m	Radius of well

Data Points (from plot):

t^*: 0.089 minutes $\Delta H^*/\Delta H_0$: 0.37

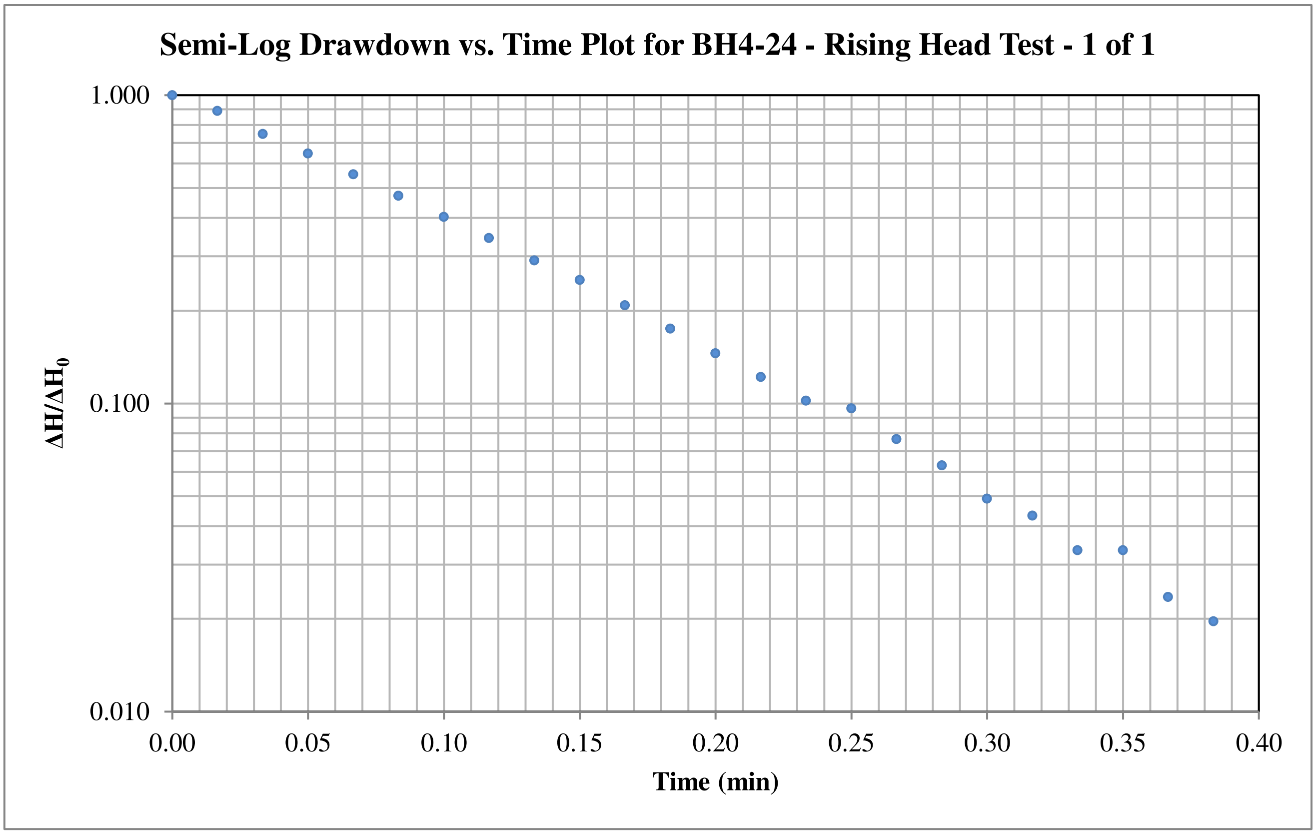
Horizontal Hydraulic Conductivity
K = 7.13E-05 m/sec

PATERSON GROUP

File: PG2119

Hvorslev Hydraulic Conductivity Analysis

Project: Campanale Homes - 609, 617 and 621 Longfields Drive
Test Location: BH4-24
Test: Rising Head - 1 of 1
Date: June 19, 2024



Hvorslev Horizontal Hydraulic Conductivity

$$K = \frac{\pi r_c^2}{F} \frac{1}{t^*} \ln\left(\frac{\Delta H^*}{\Delta H_0}\right)$$

Hvorslev Shape Factor

$$F = \frac{2\pi L}{\ln\left(\frac{2L}{D}\right)}$$

Valid for L>>D

Hvorslev Shape Factor F: 2.07207

Well Parameters:

L	1.5	m	Saturated length of screen or open hole
D	0.03175	m	Diameter of well
r_c	0.01588	m	Radius of well

Data Points (from plot):

t^*: 0.109 minutes $\Delta H^*/\Delta H_0$: 0.37

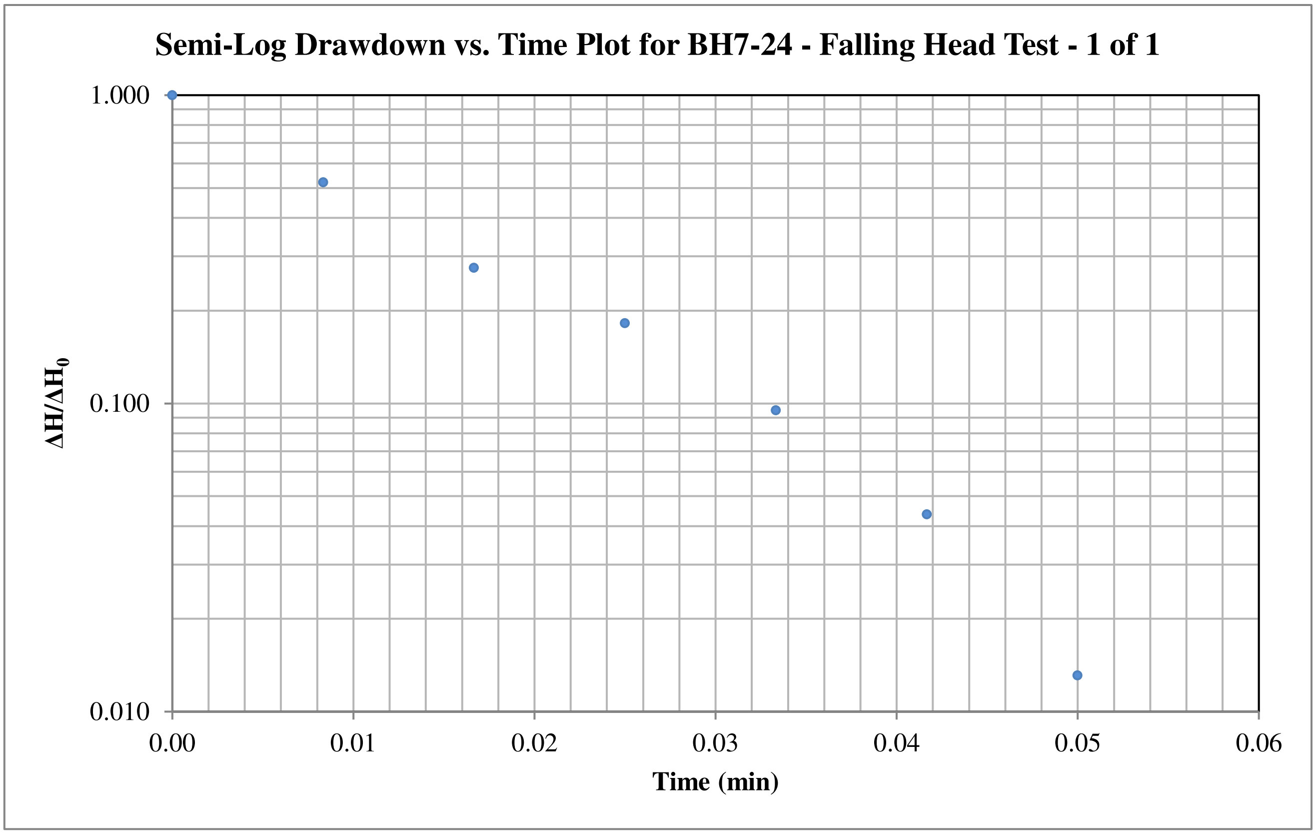
Horizontal Hydraulic Conductivity
K = 5.79E-05 m/sec

PATERSON GROUP

File: PG2119

Hvorslev Hydraulic Conductivity Analysis

Project: Campanale Homes - 609, 617 and 621 Longfields Drive
Test Location: BH7-24
Test: Falling Head - 1 of 1
Date: June 19, 2024



Hvorslev Horizontal Hydraulic Conductivity

$$K = \frac{\pi r_c^2}{F} \frac{1}{t^*} \ln\left(\frac{\Delta H^*}{\Delta H_0}\right)$$

Hvorslev Shape Factor

$$F = \frac{2\pi L}{\ln\left(\frac{2L}{D}\right)}$$

Valid for L>>D

Hvorslev Shape Factor F: 2.07207

Well Parameters:

L	1.5	m	Saturated length of screen or open hole
D	0.03175	m	Diameter of well
r_c	0.01588	m	Radius of well

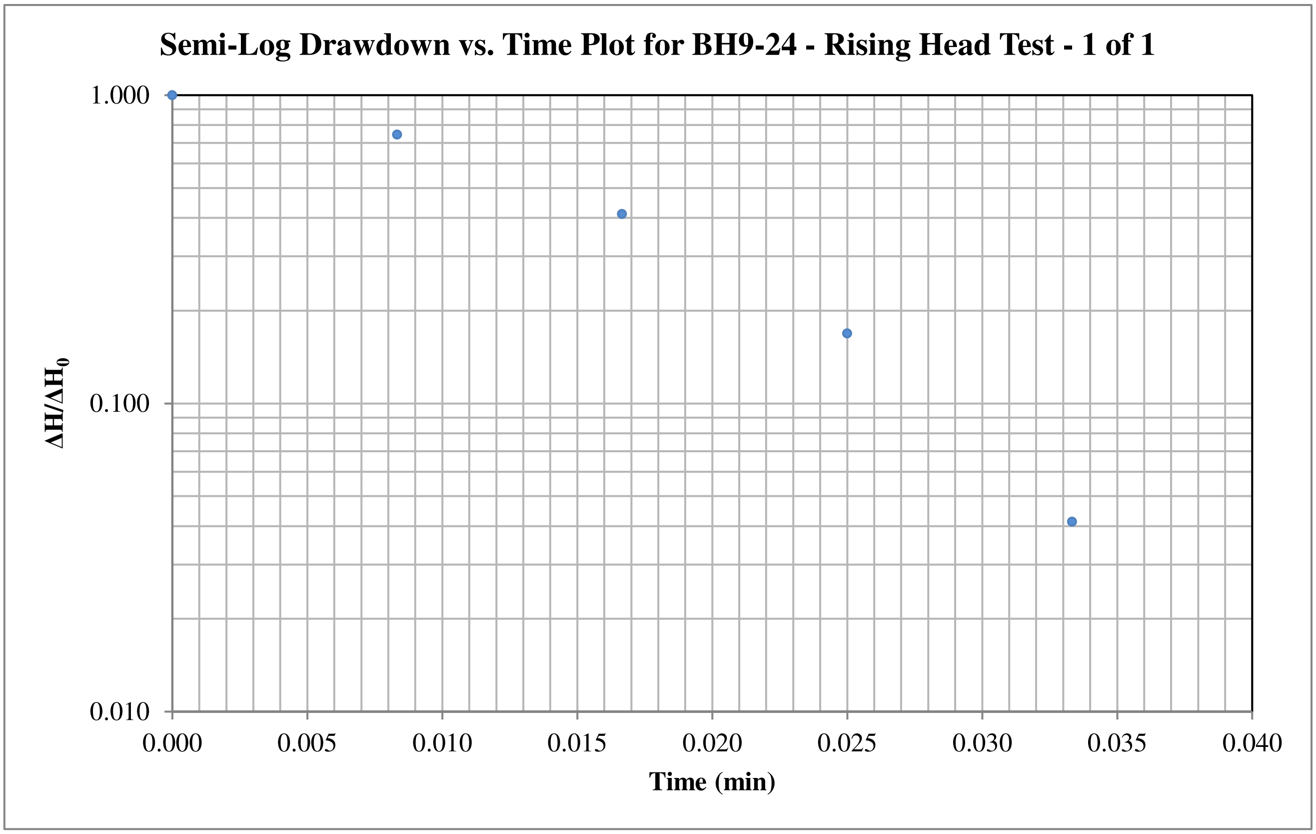
Data Points (from plot):

t^*: 0.013 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 4.70E-04 m/sec

Hvorslev Hydraulic Conductivity Analysis

Project: Campanale Homes - 609, 617 and 621 Longfields Drive
Test Location: BH9-24
Test: Rising Head - 1 of 1
Date: June 19, 2024



Hvorslev Horizontal Hydraulic Conductivity

$$K = \frac{\pi r_c^2}{F} \frac{1}{t^*} \ln\left(\frac{\Delta H^*}{\Delta H_0}\right)$$

Hvorslev Shape Factor

$$F = \frac{2\pi L}{\ln\left(\frac{2L}{D}\right)}$$

Valid for L>>D

Hvorslev Shape Factor F: 2.07207

Well Parameters:

L	1.5	m	Saturated length of screen or open hole
D	0.03175	m	Diameter of well
r_c	0.01588	m	Radius of well

Data Points (from plot):

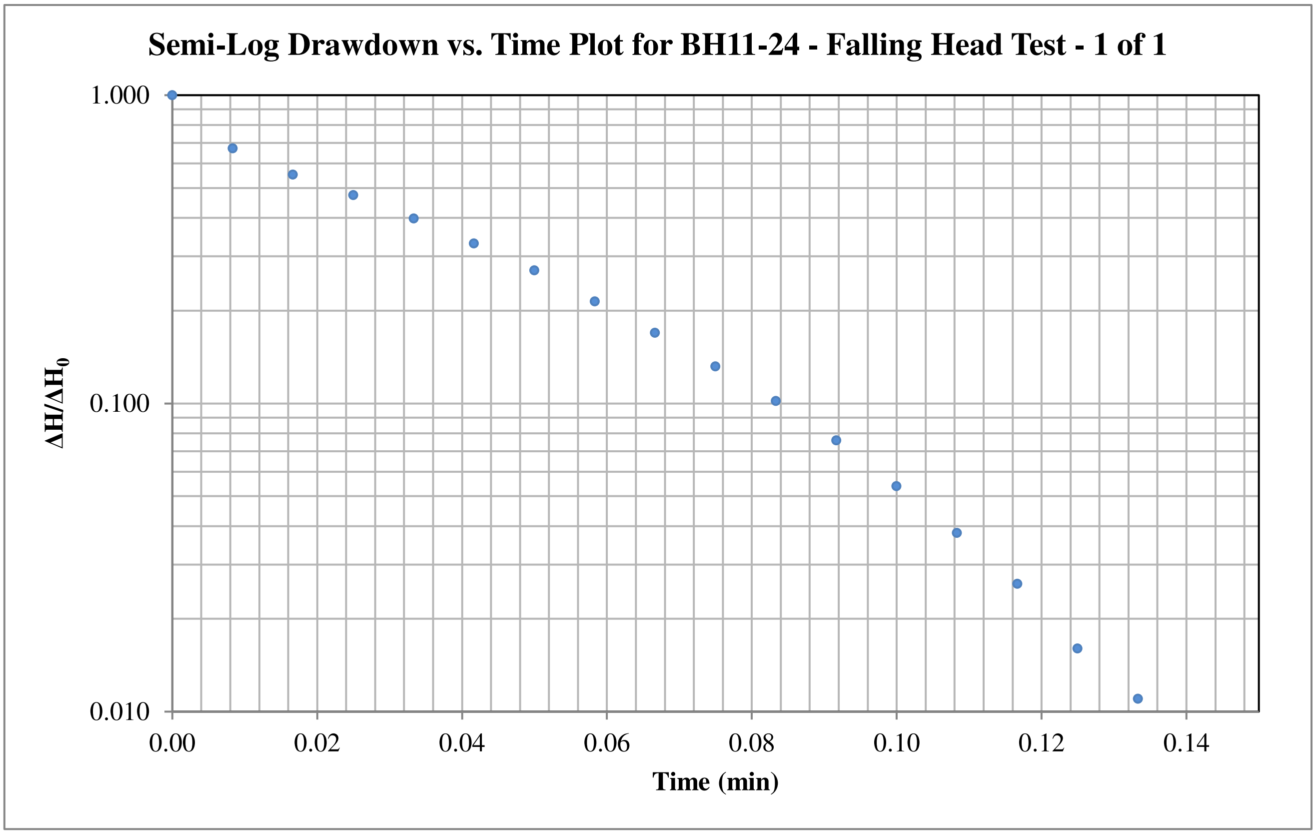
t^*: 0.018 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 3.50E-04 m/sec

File: PG2119

Hvorslev Hydraulic Conductivity Analysis

Project: Campanale Homes - 609, 617 and 621 Longfields Drive
Test Location: BH11-24
Test: Falling Head - 1 of 1
Date: June 19, 2024



Hvorslev Horizontal Hydraulic Conductivity

$$K = \frac{\pi r_c^2}{F} \frac{1}{t^*} \ln\left(\frac{\Delta H^*}{\Delta H_0}\right)$$

Hvorslev Shape Factor

$$F = \frac{2\pi L}{\ln\left(\frac{2L}{D}\right)}$$

Valid for L>>D

Hvorslev Shape Factor F: 2.07207

Well Parameters:

L	1.5	m	Saturated length of screen or open hole
D	0.03175	m	Diameter of well
r_c	0.01588	m	Radius of well

Data Points (from plot):

t^*: 0.037 minutes $\Delta H^*/\Delta H_0$: 0.37

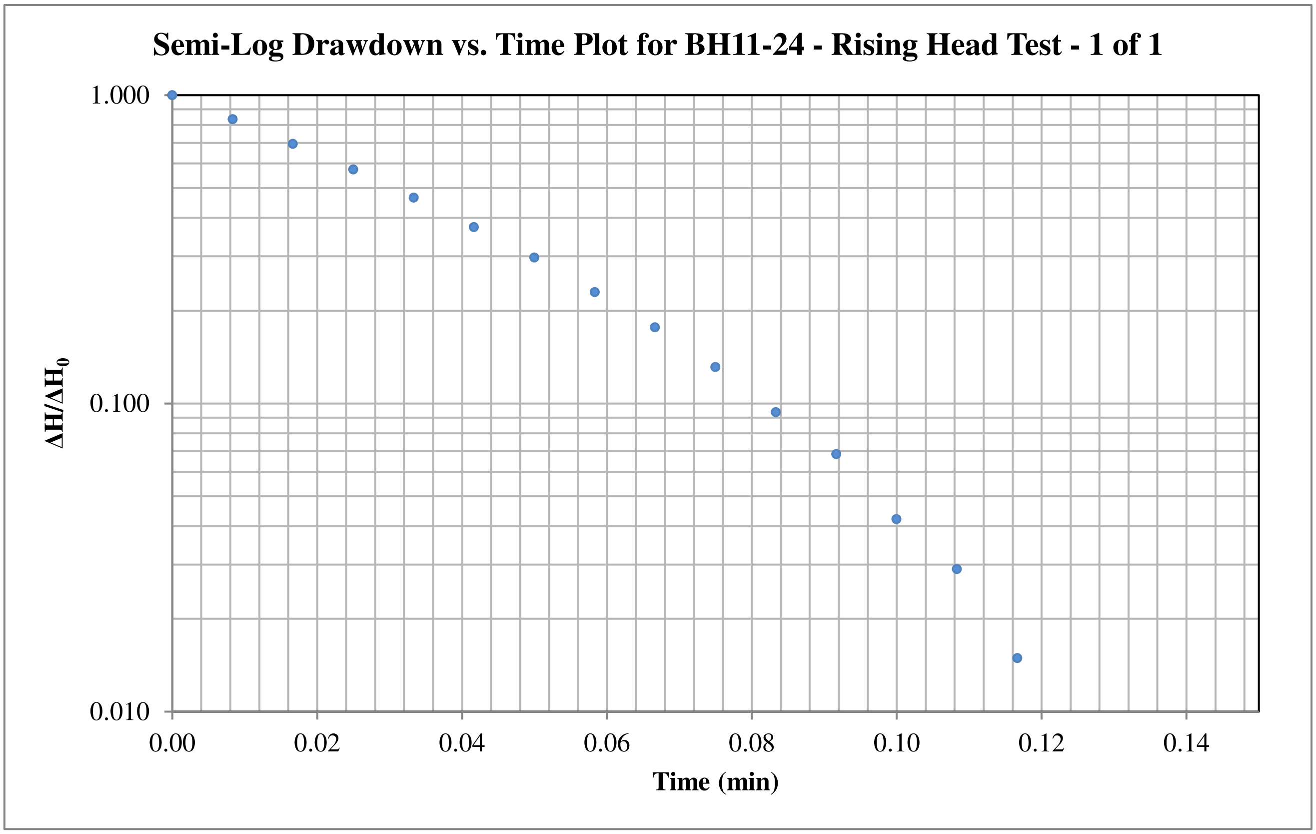
Horizontal Hydraulic Conductivity
K = 1.72E-04 m/sec

PATERSON GROUP

File: PG2119

Hvorslev Hydraulic Conductivity Analysis

Project: Campanale Homes - 609, 617 and 621 Longfields Drive
Test Location: BH11-24
Test: Rising Head - 1 of 1
Date: June 19, 2024



Hvorslev Horizontal Hydraulic Conductivity

$$K = \frac{\pi r_c^2}{F} \frac{1}{t^*} \ln\left(\frac{\Delta H^*}{\Delta H_0}\right)$$

Hvorslev Shape Factor

$$F = \frac{2\pi L}{\ln\left(\frac{2L}{D}\right)}$$

Valid for L>>D

Hvorslev Shape Factor F: 2.07207

Well Parameters:

L	1.5	m	Saturated length of screen or open hole
D	0.03175	m	Diameter of well
r_c	0.01588	m	Radius of well

Data Points (from plot):

t^*: 0.042 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 1.51E-04 m/sec

APPENDIX 2

FIGURE 1 – KEY PLAN

FIGURE 2 – PODIUM DECK TO FOUNDATION WALL DRAINAGE SYSTEM TIE-IN DETAIL

DRAWING PG2119-7 – TEST HOLE LOCATION PLAN – BLOCK 10

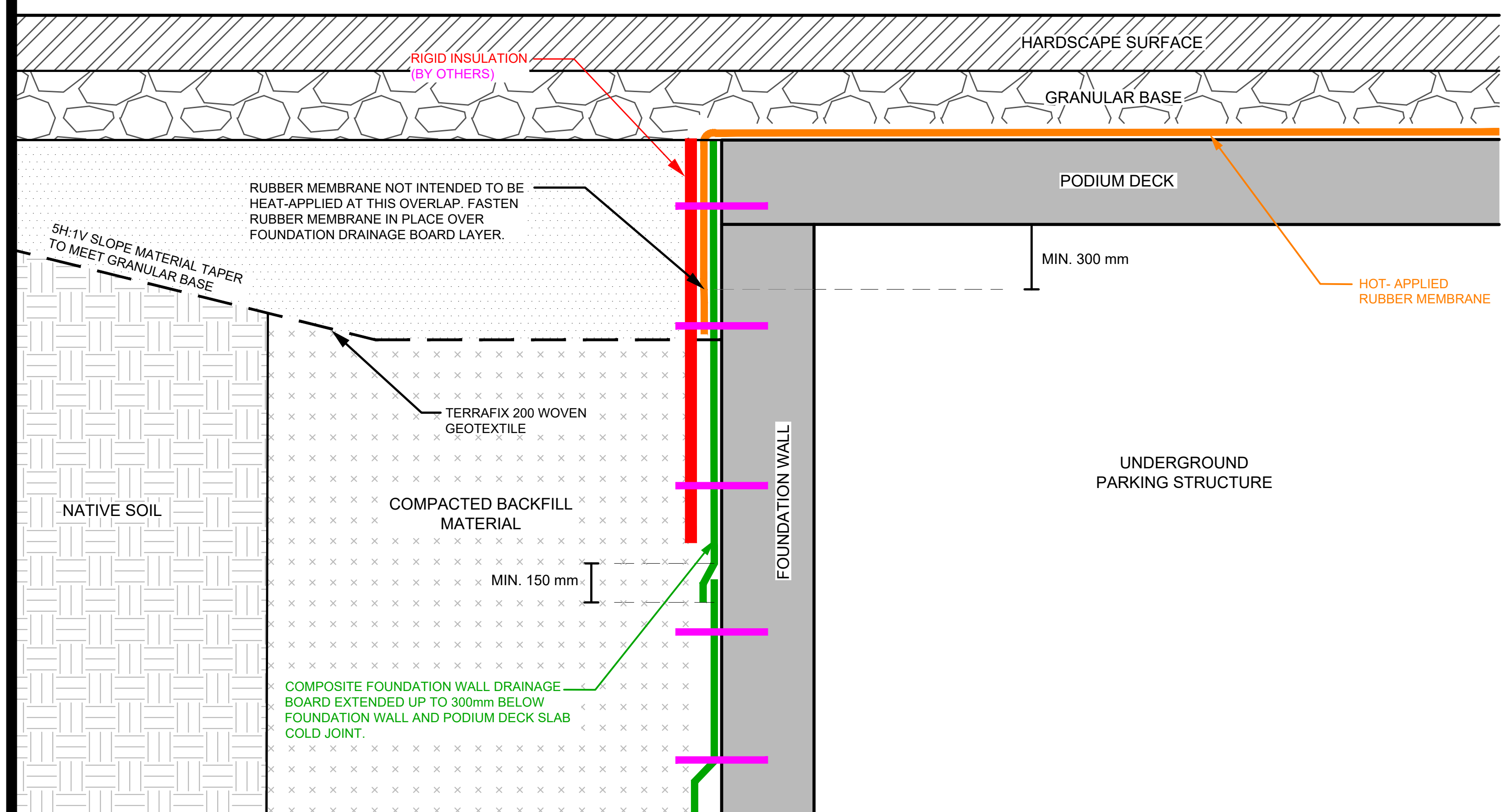
DRAWING PG2119-8 – TEST HOLE LOCATION PLAN – BLOCK 8

DRAWING PG2119-9 – TEST HOLE LOCATION PLAN – BLOCK 5

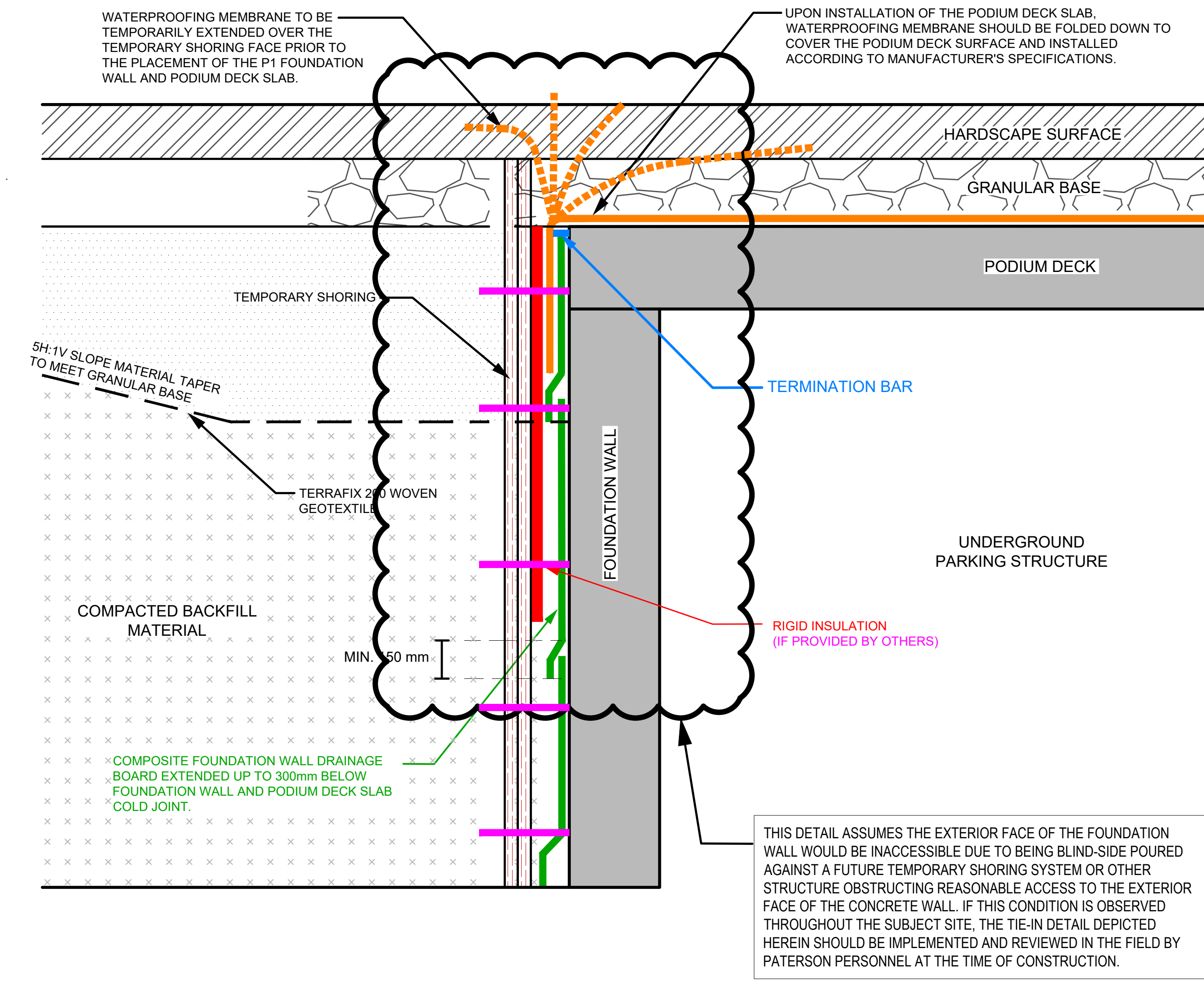
FIGURE 1

KEY PLAN

OPTION A - DOUBLE-SIDE POURED TOP OF FOUNDATION WALL



OPTION B - BLIND-SIDE POURED TOP OF FOUNDATION WALL



NOTES:

THE ABOVE DETAIL FOR HOT RUBBER AND DRAINAGE BOARD OVERLAP IS APPLICABLE TO ALL EDGE-PORTIONS OF THE PODIUM DECK AND/OR SUSPENDED GROUND FLOOR SLAB STRUCTURE.

APPLICABILITY THICKNESS AND EXTENSIONS OF RIGID INSULATION ARE SPECIFIED BY OTHERS

WHERE THE GRADING SURFACE TERMINATES AGAINST THE BUILDING FACE AND PAVEMENT STRUCTURE IS NOT LOCATED ABOVE THE EDGE OF THE FOUNDATION WALL AND PODIUM DECK SLAB AS DEPICTED HEREIN, IT IS RECOMMENDED TO PROVIDE A SUITABLE TERMINATION BAR TO SEAL THE TOP ENDLAP OF THE HOT-APPLIED RUBBER MEMBRANE LAYER TO THE VERTICAL FACE OF THE STRUCTURE. THIS WOULD BE REQUIRED TO MITIGATE THE POTENTIAL FOR THE MIGRATION OF WATER BEHIND THE RUBBER MEMBRANE.

ALL PORTIONS OF THE ABOVE-NOTED DETAIL (INSULATION OF FOUNDATION DRAINAGE BOARD, TERMINATION BAR, HOT-RUBBER MEMBRANE OVER SLAB, FOUNDATION WALL CONSTRUCTION JOINT AND OVERLAPPING/SHINGLING OF DRAINAGE BOARD) SHOULD BE REVIEWED AT THE TIME OF CONSTRUCTION BY PATERSON PERSONNEL.

PATERSON GROUP
9 AURIGA DRIVE
OTTAWA, ON
K2E 7T9
TEL: (613) 226-7381

NO.	REVISIONS	DATE	INITIAL

CAMPANALE HOMES

GEOTECHNICAL INVESTIGATION
PROPOSED APARTMENT BUILDINGS - BLOCKS 5, 8, and 10
609, 617, 621, LONGFIELDS DRIVE

OTTAWA, ONTARIO

Title:
PODIUM DECK TO FOUNDATION WALL DRAINAGE SYSTEM TIE-IN DETAIL

Scale: N.T.S	Date: 06/2024
Drawn by: ZS	Report No.: PG2119
Checked by: YZ	Dwg. No.: FIGURE 2
Approved by: FA	Revision No.:

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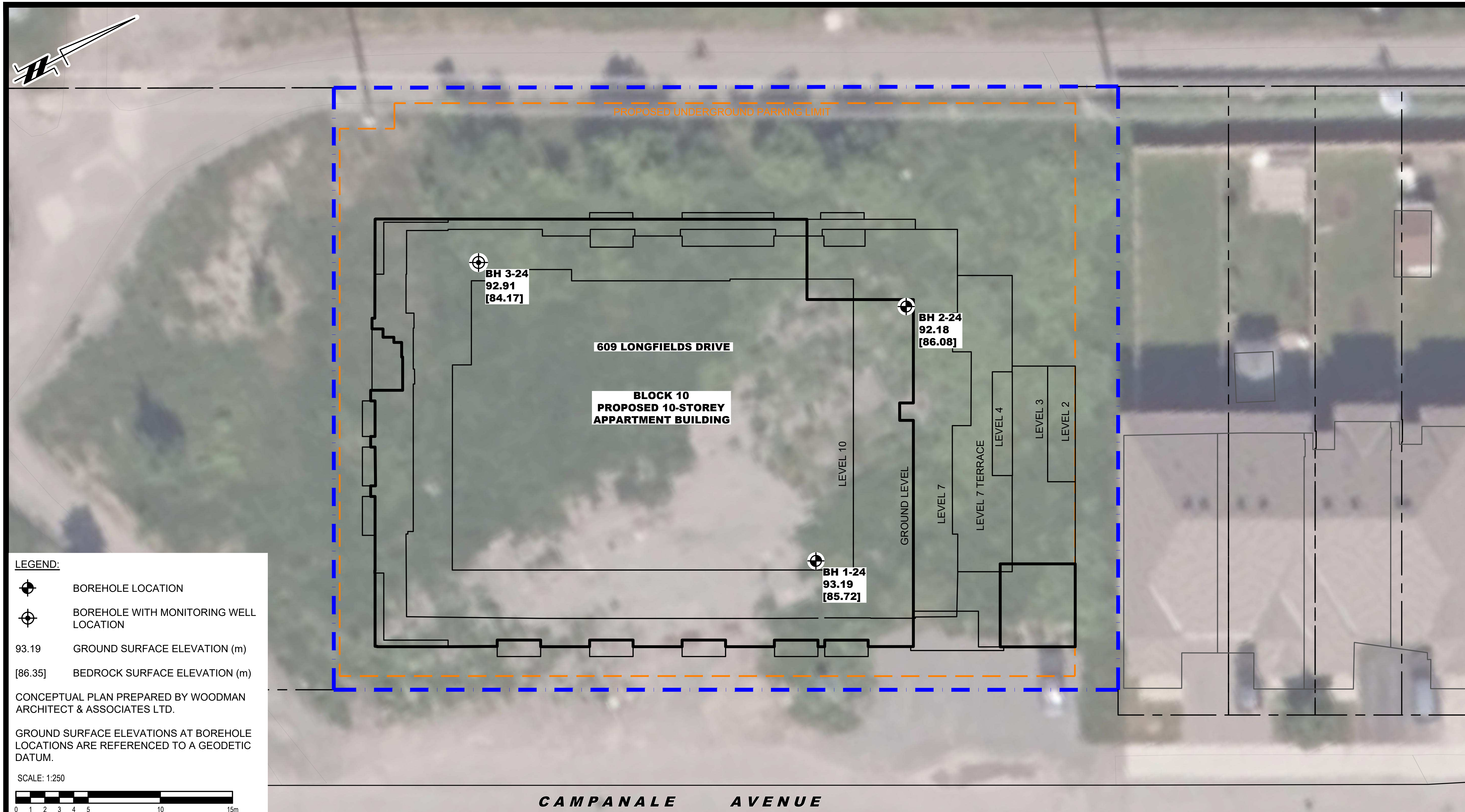

LEGEND:

- BOREHOLE LOCATION
- BOREHOLE WITH MONITORING WELL LOCATION

93.19 GROUND SURFACE ELEVATION (m)

[86.35] BEDROCK SURFACE ELEVATION (m)

CONCEPTUAL PLAN PREPARED BY WOODMAN ARCHITECT & ASSOCIATES LTD.

GROUND SURFACE ELEVATIONS AT BOREHOLE LOCATIONS ARE REFERENCED TO A GEODETIC DATUM.

SCALE: 1:250

0 1 2 3 4 5 10 15m

PATERSON GROUP
9 AURIGA DRIVE
OTTAWA, ON
K2E 7T9
TEL: (613) 226-7381

NO.	REVISIONS	DATE	INITIAL

CAMPANALE HOMES
GEOTECHNICAL INVESTIGATION
PROPOSED APARTMENT BUILDING - BLOCK 10
609 LONGFIELDS DRIVE

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:250	Date: 06/2024
Drawn by: ZS	Report No.: PG2119-4
Checked by: YZ	Dwg. No.: PG2119-7
Approved by: FA	Revision No.:

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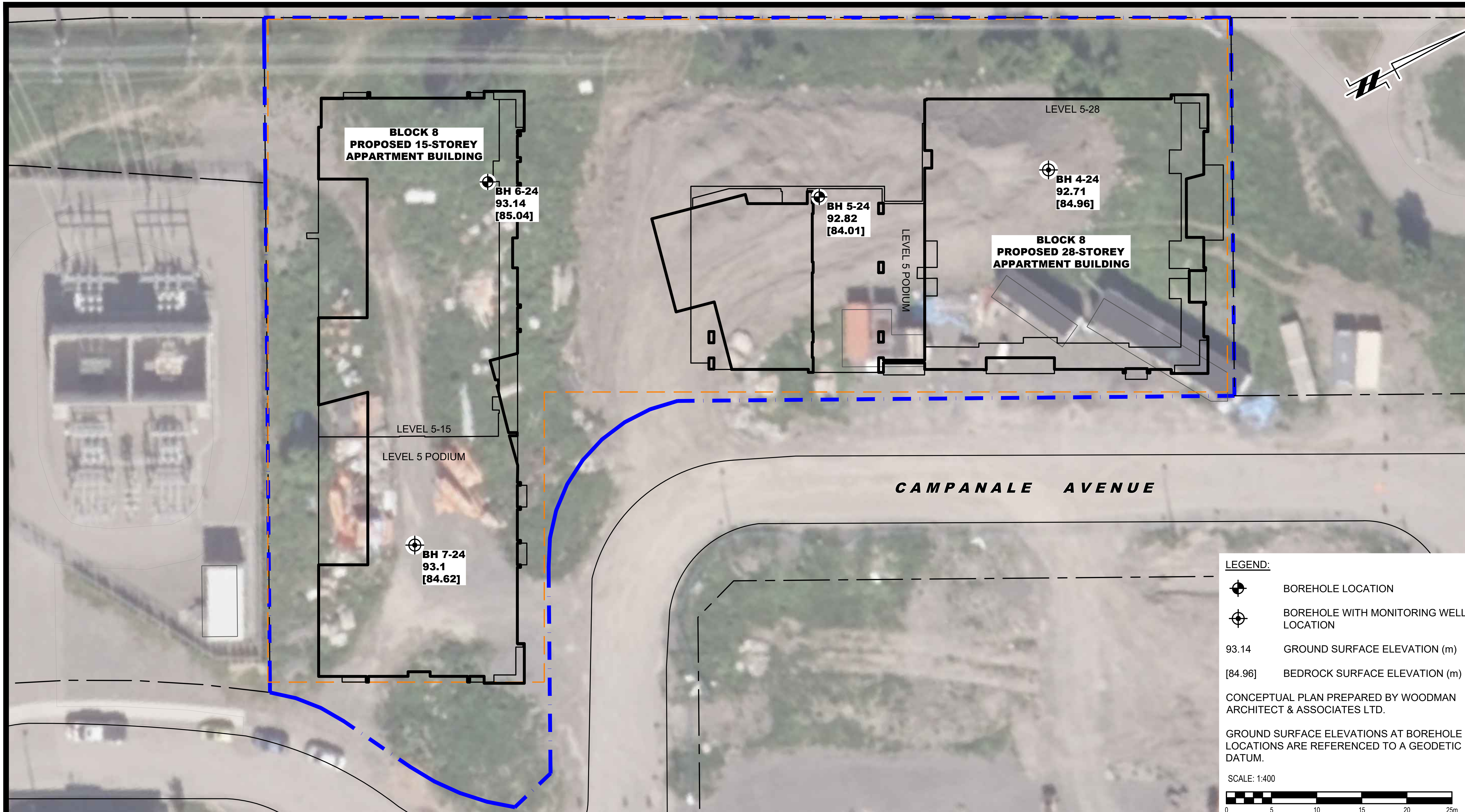

LEGEND:

- BOREHOLE LOCATION
- BOREHOLE WITH MONITORING WELL LOCATION

93.14 GROUND SURFACE ELEVATION (m)

[84.96] BEDROCK SURFACE ELEVATION (m)

CONCEPTUAL PLAN PREPARED BY WOODMAN ARCHITECT & ASSOCIATES LTD.

GROUND SURFACE ELEVATIONS AT BOREHOLE LOCATIONS ARE REFERENCED TO A GEODETIC DATUM.

SCALE: 1:400

0 5 10 15 20 25m

PATERSON GROUP
9 AURIGA DRIVE
OTTAWA, ON
K2E 7T9
TEL: (613) 226-7381

NO.	REVISIONS	DATE	INITIAL

CAMPANALE HOMES
GEOTECHNICAL INVESTIGATION
PROPOSED APARTMENT BUILDING - BLOCK 8
617 LONGFIELDS DRIVE

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:400	Date: 06/2024
Drawn by: ZS	Report No.: PG2119-4
Checked by: YZ	Dwg. No.: PG2119-8
Approved by: FA	Revision No.:

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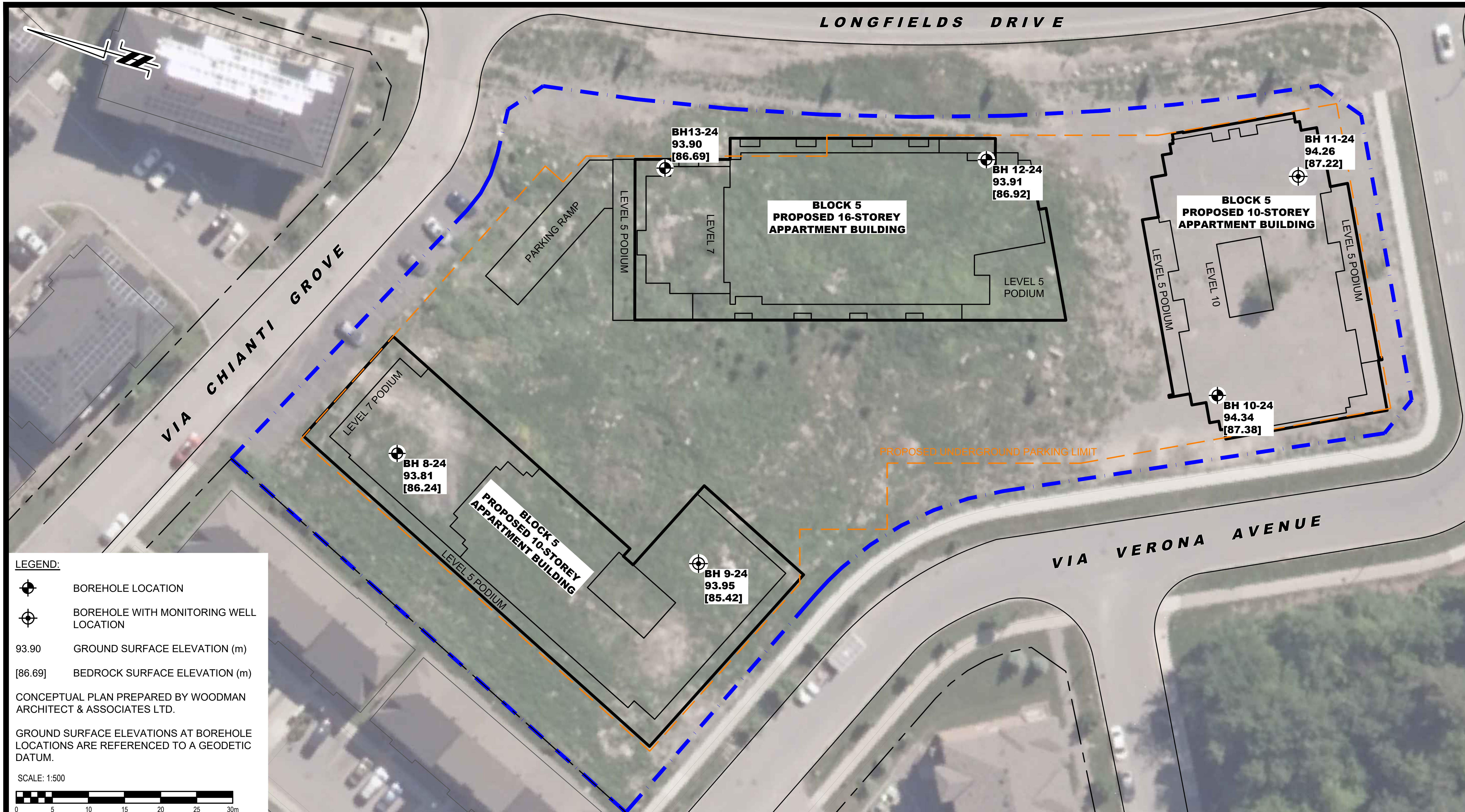
11x17



LEGEND:

⌖	BOREHOLE LOCATION
⌖	BOREHOLE WITH MONITORING WELL LOCATION
93.90	GROUND SURFACE ELEVATION (m)
[86.69]	BEDROCK SURFACE ELEVATION (m)

CONCEPTUAL PLAN PREPARED BY WOODMAN ARCHITECT & ASSOCIATES LTD.

GROUND SURFACE ELEVATIONS AT BOREHOLE LOCATIONS ARE REFERENCED TO A GEODETIC DATUM.

SCALE: 1:500

0 5 10 15 20 25 30m

PATERSON GROUP
9 AURIGA DRIVE
OTTAWA, ON
K2E 7T9
TEL: (613) 226-7381

NO.	REVISIONS	DATE	INITIAL

CAMPANALE HOMES
GEOTECHNICAL INVESTIGATION
PROPOSED APARTMENT BUILDING - BLOCK 5
621 LONGFIELDS DRIVE

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:500	Date: 06/2024
Drawn by: ZS	Report No.: PG2119-4
Checked by: YZ	Dwg. No.: PG2119-9
Approved by: FA	Revision No.:

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11x17