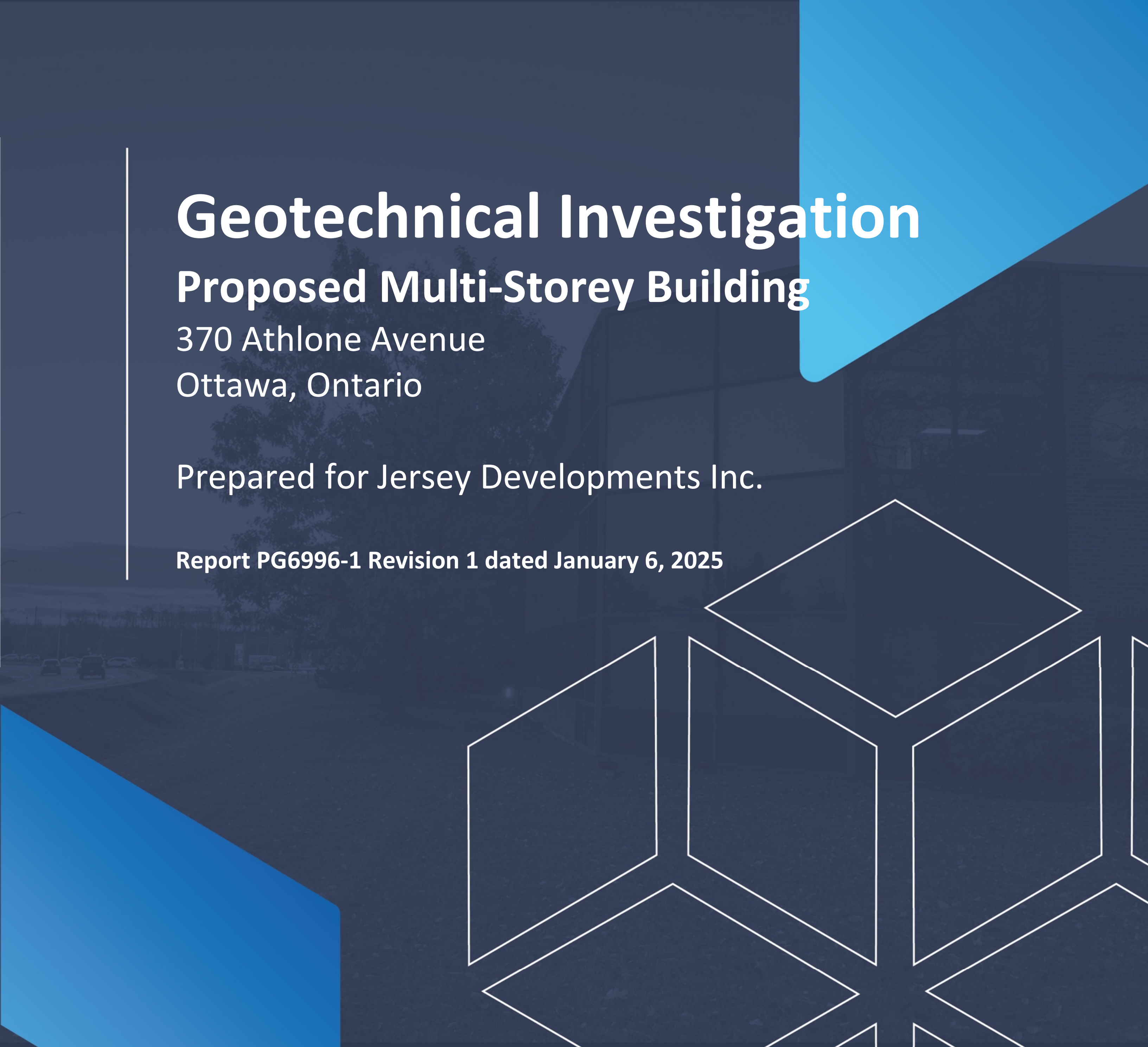

Geotechnical Investigation

Proposed Multi-Storey Building

370 Athlone Avenue
Ottawa, Ontario

Prepared for Jersey Developments Inc.

Report PG6996-1 Revision 1 dated January 6, 2025

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Jersey Developments Inc. to prepare a Geotechnical Investigation Report for the proposed development to be located at 370 Athlone Avenue in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report for the general site location).

The objectives of the geotechnical investigation were to:

- ☐ Determine the subsoil and groundwater conditions at this site by means of boreholes, and to;

- ☐ Provide geotechnical recommendations pertaining to the 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.

2.0 Proposed Development

Based on the available drawings, it is understood that the proposed development will consist of a multi-storey residential building with 1 basement level. Walkways and landscaped margins are expected at finished grades surrounding the proposed building. It is also anticipated that the proposed building will be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the geotechnical investigation was carried out on May 18, 2023 and consisted of advancing a total of 3 boreholes to a maximum depth of 7.6 m below the existing grade. The borehole locations were distributed in a manner to provide general coverage of the subject site, taking into consideration underground services and available access. The approximate locations of the boreholes are shown on Drawing PG6996-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were advanced using a track-mounted drill rig operated by a two-person crew. All fieldwork was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer. The drilling procedure consisted of augering to the required depths at the selected borehole locations, and sampling and testing the overburden and bedrock.

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. Rock cores (RC) were obtained using 47.6 mm inside diameter coring equipment. 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 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.

A 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.

Bedrock samples were recovered from all boreholes using a core barrel and diamond drilling techniques. A recovery value and a Rock Quality Designation (RQD) value were calculated for each drilled section (core run) of bedrock and are shown on the Soil Profile and Test Data Sheets in Appendix 1. The recovery value is the ratio, in percentage, of the length of the bedrock sample recovered over the length of the drilled section (core run). The RQD value is the ratio, in percentage, of the total length of intact rock pieces longer than 100 mm in one core run over the length of the core run. These values are indicative of the bedrock quality.

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

Groundwater

Groundwater monitoring wells were installed in boreholes BH 1-23, BH 2-23 and BH 3-23 to permit monitoring of groundwater levels subsequent to the completion of the sampling program.

Groundwater level observations are discussed in Section 4.3 and are presented in the Soil Profile and Test Data Sheets in Appendix 1.

3.2 Field Survey

The borehole locations, and the ground surface elevation at each borehole location, were surveyed by Paterson using a handheld GPS unit with respect to a geodetic datum. The locations of the boreholes, and ground surface elevation at each borehole location, are presented on Drawing PG6996-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Review

Soil and bedrock samples were recovered from the subject site and visually examined in our laboratory to review the results of the field logging. They will then be discarded unless otherwise directed.

4.0 Observations

4.1 Surface Conditions

The subject site is relatively flat and currently occupied by a residential dwelling situated in the central portion of the site. Furthermore, a detached garage and storage shed are positioned along the western boundary, accompanied by an asphalt driveway and associated landscape margins.

The site is bordered by residential properties to the north and south, Athlone Avenue to the east, and a commercial building to the west. The ground surface across the site is relatively level at approximate geodetic elevation 65 m.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile at the borehole locations consists of topsoil or a concrete slab underlain by fill extending to an approximate depth of 1.45 to 1.76 m below the existing ground surface. The fill was generally observed to consist of compact, brown silty sand with gravel, crushed stones, topsoil, asphalt, and occasional cobbles.

A compact to dense, silty sand to sandy silt, and glacial till deposit were encountered underlying the fill. The glacial till was generally composed of silty sand with gravel and cobbles.

Bedrock

Bedrock was encountered underlying the glacial till at approximate depths ranging from 4.9 to 5.4 m, and was cored at each borehole hole location. Based on the recovered rock core, the bedrock was observed to consist of grey limestone, which is fair to excellent in quality. The bedrock was cored to a maximum depth of about 7.6 m below the existing grade.

Reference should be made to the Soil Profile and Test Data Sheets in Appendix 1 for details of the soil and bedrock profile encountered at each borehole location.

Based on available geological mapping, the bedrock in the subject area consists of interbedded limestone and dolomite of the Gull River formation.

4.3 Groundwater

The groundwater level was measured at the groundwater monitoring wells on May 23, 2023. The observed groundwater levels are summarized in Table 1 below.

Table 1 – Summary of Groundwater Level Readings

Test Hole Number	Ground Surface Elevation (m)	Groundwater Level (m)	Groundwater Elevation (m)	Recording Date
BH 1-23*	65.42	4.67	60.75	May 23, 2023
BH 2-23*	65.31	4.55	60.76	May 23, 2023
BH 3-23*	65.39	4.57	60.82	May 23, 2023

Note: The ground surface elevation at each borehole location was surveyed by Paterson using a handheld GPS and was referenced to a geodetic datum.
* Borehole instrumented with groundwater monitoring well.

It should be noted that the groundwater levels could be influenced by surface water infiltrating the backfilled boreholes.

The long-term groundwater levels can also be estimated based on the observed colour, moisture content and consistency of the recovered soil samples. Based on these observations, the long-term groundwater table can be expected at approximate depths of 4 to 5 m below the existing ground surface.

However, it should be noted that groundwater levels are subject to seasonal fluctuations, therefore, the groundwater levels could vary at the time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered suitable for the proposed development. It is recommended that the proposed building be founded on conventional spread footings bearing on undisturbed, compact to dense silty sand to sandy silt and/or glacial till.

It is anticipated that cobbles and/or boulders may be encountered during excavation and construction. All contractors should be prepared for cobbles and/or boulder removal within the subject site.

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

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and fill, such as those containing organic or deleterious material, should be stripped from under any buildings, paved areas, pipe bedding, and other settlement sensitive structures.

Existing foundation walls and other construction debris should be completely removed from the proposed building perimeter. Under paved areas, existing construction remnants, such as foundation walls, should be excavated to a minimum of 1 m below final grade.

Fill Placement

Fill used for grading beneath the building footprint, unless otherwise specified, should consist of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. Imported fill should be tested and approved prior to delivery to the site.

Non-specified existing fill along with site-excavated soil can be placed as general landscaping fill where surface settlement is a minor concern. The backfill materials should be spread in thin lifts, and at a minimum, compacted by the tracks of the spreading equipment to minimize voids. If the non-specified backfill is to be placed to increase the subgrade level for areas to be paved, the fill should be compacted in maximum 300 mm lifts and compacted to 98% of the material’s SPMDD.

5.3 Foundation Design

Footings placed directly on a bearing surface consisting of undisturbed, compact to dense silty sand to sandy silt and/or glacial till can be designed using a bearing resistance value at serviceability limit states (SLS) of **300 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **450 kPa**. A geotechnical resistance factor of 0.5 was applied to the bearing resistance value at ULS.

An undisturbed soil bearing surface consists of a surface from which all topsoil, debris and deleterious materials, such as loose, frozen or disturbed soil, whether in situ or not, have been removed, in the dry, prior to the placement of concrete for footings.

Footings placed designed using the bearing resistance values at SLS given above will be subjected to potential post construction total and differential settlements of 25 and 20 mm, respectively.

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 soil bearing medium when a plane extending down and out from the bottom edges of the footing, at a minimum of 1.5H:1V, passes only through soil of the same or higher capacity as that of the bearing medium.

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class C** for the foundations considered at this site. If a higher seismic site class is required (such as Class B), a seismic shear wave velocity test could be completed to accurately determine the seismic site classification, in accordance with Table 4.1.8.4.A of the Ontario Building Code (OBC) 2012. The soils underlying the subject site are not susceptible to liquefaction. Reference should be made to the latest revision of the OBC 2012 for a full discussion of the earthquake design requirements.

5.5 Basement Floor Slab

With the removal of all topsoil and deleterious fill from within the footprint of the proposed building, the existing fill, silty sand to sandy silt, and/or glacial till will be considered acceptable subgrades on which to commence backfilling for floor slab construction.

It is recommended that the upper 200 mm of sub-slab fill consist of 19 mm clear crushed stone. It is further recommended that an underslab drainage system, consisting of lines of perforated drainage pipe sub-drains connected to a positive outlet, should be provided under the basement floor slab. This is discussed further in Section 6.1.

5.6 Basement Wall

There are several combinations of backfill materials and retained soils that could be applicable for the basement walls of the proposed building. 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 drained unit weight of 20 kN/m^3 (effective unit weight 13 kN/m^3).

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 notexercised 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) can be calculated using $0.375 \cdot a \cdot H^2/g$ 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 0.32g according to 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 = 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 OBC 2012.

5.7 Pavement Design

If car only parking areas and access lanes are included as part of the development at this site, the recommended pavement structures in Tables 2 and 3 below should be used.

Table 2 – Recommended Asphalt Pavement Structure – Car only Parking Areas

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

Table 3 – Recommended Asphalt Pavement Structure – Access Lanes

Thickness (mm)	Material Description
40	**Wear Course** – Superpave 12.5 Asphaltic Concrete
50	**Binder Course** – Superpave 19.0 Asphaltic Concrete
150	**BASE** – OPSS Granular A Crushed Stone
300	**SUBBASE** – OPSS Granular B Type II
SUBGRADE – Existing imported fill, or OPSS Granular B Type I or II material placed over in situ soil or bedrock.	

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 99% of the material's SPMDD using suitable vibratory equipment.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

A perimeter foundation drainage system is recommended to be provided for the proposed building. The system should consist of a 150 mm diameter perforated and corrugated plastic pipe, surrounded on all-sides by 150 mm of 19 mm clear crushed stone, which is placed at the footing level around the exterior perimeter of the structure. The pipe should have a positive outlet, such as a gravity connection to the storm sewer or sump pit provided below the basement level of the structure.

Underslab Drainage System

Underslab drainage will be required to control water infiltration below the lowest level floor slab. For preliminary design purposes, we recommend that 100 or 150 mm perforated pipes be placed at approximate 6 m centres underlying the basement floor slab. The spacing of the underslab drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

Foundation Backfill

Backfill against the exterior sides of the foundation walls, where required, should consist of free-draining, non-frost susceptible granular materials such as clean sand or OPSS Granular B Type I material. The greater part of the site excavated materials will be relatively 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 board which is installed over the exterior foundation walls, such as Delta Drain 6000, and connected to the perimeter foundation drainage system. Imported granular materials, such as clean sand or OPSS Granular B Type I, Granular A or Granular B Type II granular material, should otherwise be used for this purpose.

Sidewalks and Walkways

Backfill material below sidewalk and walkway subgrade areas or other settlement sensitive structures which are not adjacent to the buildings 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.

6.2 Protection of Footings Against Frost Action

Perimeter footings of heated structures are recommended to be insulated against the deleterious effects of frost action. A minimum 1.5 m thick soil cover, or an equivalent combination of soil cover and foundation insulation should be provided in this regard.

Other exterior unheated footings, such as isolated piers, are more prone to deleterious movement associated with frost action than the exterior walls of the structure, and require additional protection, such as soil cover of 2.1 m, or an equivalent combination of soil cover and foundation insulation.

6.3 Excavation Side Slopes

The side slopes of excavations in the overburden materials should either be cut back at acceptable slopes or should be retained by shoring systems from the start of the excavation until the structure is backfilled.

Unsupported Excavations

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be excavated at 1H:1V or shallower. The shallower slope is required for excavation below groundwater level. The subsurface soil at this site is considered to be mainly a Type 2 and 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 be kept away from the excavation sides.

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. It is expected that services will be installed by “cut and cover” methods and excavations will not be left open for extended periods of time.

Temporary Shoring

Where sufficient space is not available to slope the excavation, a temporary shoring system would be required to support the excavation. The design and approval of the shoring system will be the responsibility of the shoring contractor and the shoring designer who is a licensed professional engineer and is hired by the shoring contractor. It is the responsibility of the shoring contractor to ensure that the temporary shoring is in compliance with safety requirements, designed to avoid any damage to adjacent structures and include dewatering control measures.

In the event that subsurface conditions differ from the approved design during the actual installation, it is the responsibility of the shoring contractor to commission the required experts to re-assess the design and implement the required changes.

The designer should also take into account the impact of a significant precipitation event and designate design measures to ensure that a precipitation event will not negatively impact the shoring system or soils supported by the system. Any changes to the approved shoring design system should be reported immediately to the owner's structural designer prior to implementation.

The temporary shoring system may consist of a soldier pipe and lagging system which could be cantilevered, anchored or braced. The shoring system is recommended to be adequately supported to resist toe failure. Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be added to the earth pressures described below.

The earth pressure acting on the shoring system may be calculated using the following parameters.

Table 4 – Soil Parameters

Parameters	Values
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 , kN/m^3	21
Submerged Unit Weight , kN/m^3	13

The active earth pressure should be calculated where wall movements are permissible while the at-rest pressure should be calculated if no movement is permissible. The dry unit weight should be calculated above the groundwater level while the effective unit weight should be calculated below the groundwater table.

The hydrostatic groundwater pressure should be included to the earth pressure distribution wherever the effective unit weight is calculated for earth pressures. If the groundwater level is lowered, the dry unit weight for the soil should be calculated full weight, with no hydrostatic groundwater pressure component. For design purposes, the minimum factor of safety of 1.5 should be calculated.

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.

A minimum of 150 mm of OPSS Granular A should be placed for bedding for sewer or water pipes when placed on a soil subgrade. The bedding should extend to the spring line of the pipe. Cover material, from the spring line to a minimum of 300 mm above the obvert of the pipe, should consist of OPSS Granular A (concrete or PSM PVC pipes) or sand (concrete pipe). The bedding and cover materials should be placed in maximum 225 mm thick lifts and compacted to 98% of the SPMDD.

It is generally possible to re-use the site materials above the cover material if the operations are carried out in dry weather conditions.

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) and above the cover material should match the soils exposed at the trench walls to minimize differential frost heaving. The trench backfill should be placed in maximum 225 mm thick loose lifts and compacted to a minimum of 98% of the material’s SPMDD. All cobbles larger than 200 mm in their longest direction should be segregated from re-use as trench backfill.

6.5 Groundwater Control

Based on our observations, it is anticipated that groundwater infiltration into excavations should be low to moderate and controllable using open sumps. The contractor should be prepared to direct water away from all subgrades, regardless of the source to prevent disturbance to the founding medium.

Groundwater Control for Building Construction

A temporary Ministry of Environment, Conservation and Parks (MECP) permit to take water (PTTW) may be required if more than 400,000 L/day of ground and/or surface water are to be pumped during the construction phase. At least 4 to 5 months should be allowed for completion of the application and issuance of the permit by the MECP.

For typical ground or surface water volumes being pumped during the construction phase, typically between 50,000 to 400,000 L/day, it is required to register on the Environmental Activity and Sector Registry (EASR). A minimum of two to four weeks should be allotted for completion of the EASR registration and the Water

Taking and Discharge Plan to be prepared by a Qualified Persons as stipulated under O.Reg. 63/16.

Impacts on Neighbouring Properties

The excavation for the proposed building is not expected to extend below the groundwater level. Furthermore, the subsurface conditions at, and in the vicinity of, the subject site consist of a compact to dense glacial till deposit, which is not generally not susceptible to settlement from dewatering. Therefore, effects to neighbouring structures are not anticipated due the proposed development at the subject site.

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 using 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 into 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.

7.0 Recommendations

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

☐ Review of the foundation plan, from a geotechnical perspective.

☐ Review of the geotechnical aspects of the excavation contractor's temporary shoring design, if required, prior to construction.

☐ Observation of all bearing surfaces prior to the placement of concrete.

☐ Sampling and testing of the concrete and fill materials.

☐ Observation of all subgrades prior to backfilling.

☐ 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 soils, with the exception of engineered crushed stone fill, generated by construction activities that will be transported on-site or off-site should 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 Jersey Developments Inc. 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.

LICENSED PROFESSIONAL ENGINEER
Jan. 6, 2025
S. S. DENNIS
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Scott Dennis
PROVINCE OF ONTARIO

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Scott S. Dennis, P.Eng.

Report Distribution:

☐ Jersey Developments Inc. (email copy)
☐ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

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

SOIL PROFILE AND TEST DATA

Phase II - Environmental Site Assessment
370 Athlone Avenue
Ottawa, Ontario

DATUM Elevations are referenced to a geodetic datum

REMARKS

BORINGS BY CME 55 Power Auger

DATE May 18, 2023

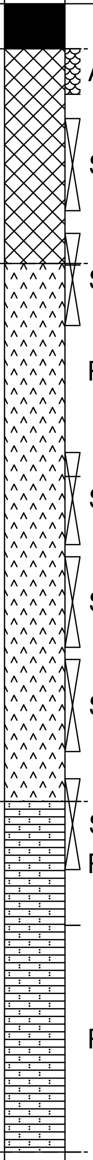
FILE NO. **PE6096**

HOLE NO. **BH 1-23**

SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
TOPSOIL	0.30
FILL: Crushed stone with gravel, topsoil and brown silty sand	1.72
GLACIAL TILL: Dense to compact brown silty sand with gravel and cobbles	5.28
BEDROCK: Excellent to fair quality, grey limestone	7.60

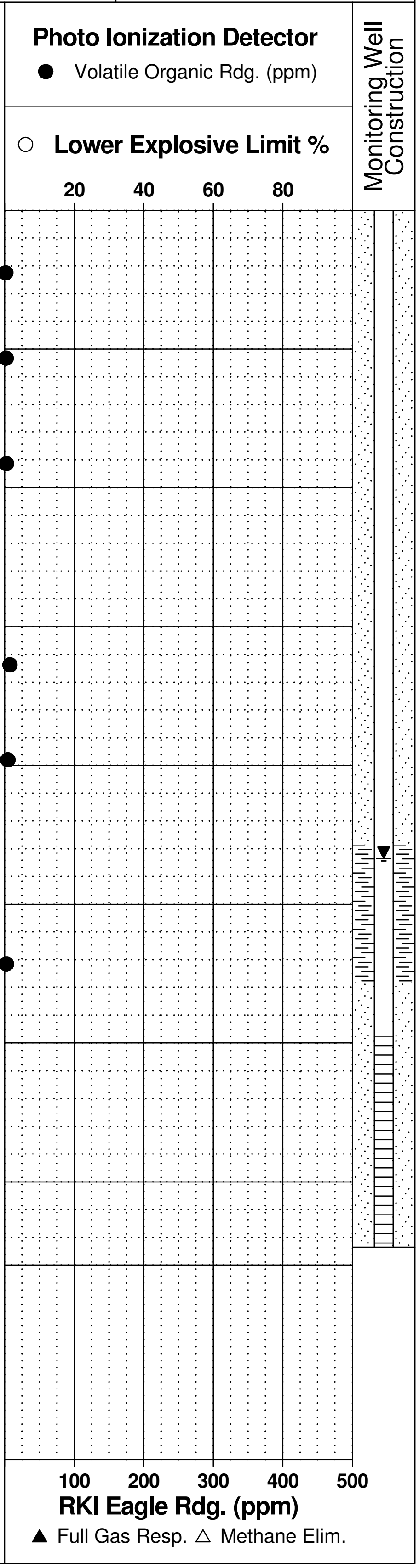
End of Borehole

(GWL @ 4.67m - May 23, 2023)



TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	50	19
SS	3	29	7
RC	1	31	53
SS	4	58	30
SS	5	63	19
SS	6	0	43
SS	7	13	50+
RC	2	94	94
RC	3	90	68

DEPTH (m)	ELEV. (m)
0	65.42
1	64.42
2	63.42
3	62.42
4	61.42
5	60.42
6	59.42
7	58.42



patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Phase II - Environmental Site Assessment
370 Athlone Avenue
Ottawa, Ontario

DATUM Elevations are referenced to a geodetic datum

REMARKS

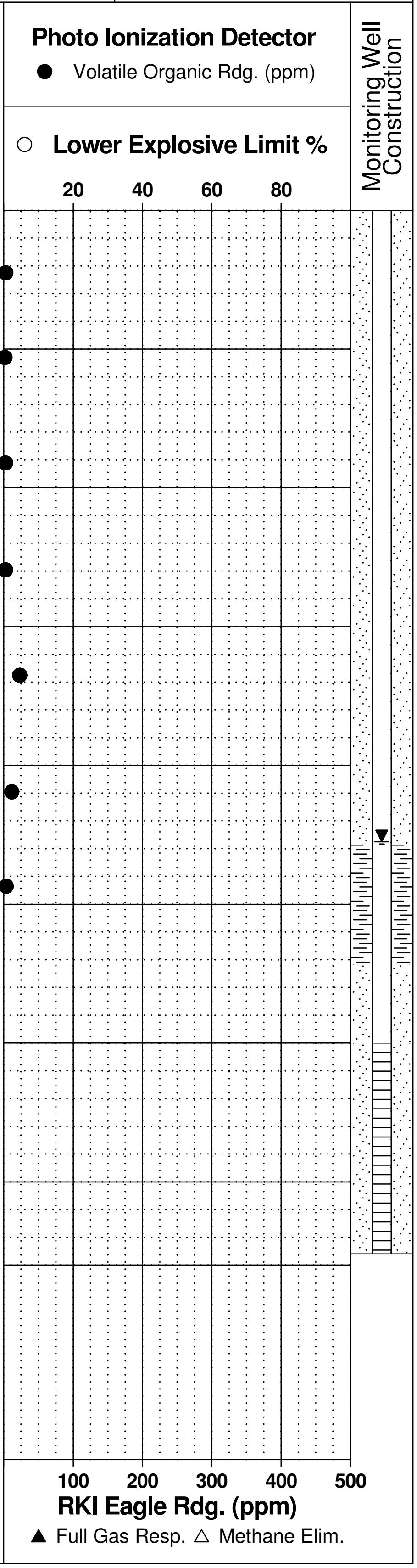
BORINGS BY CME 55 Power Auger

DATE May 18, 2023

FILE NO. **PE6096**

HOLE NO. **BH 2-23**

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	65.31
TOPSOIL 0.30						
FILL: Brown silty sand with gravel, cobbles and some topsoil	AU	1				
	SS	2	13	50+	1	64.31
1.45						
Very dense, light brown**SILTY SAND** to **SANDY SILT** with gravel and cobbles	SS	3	67	50+		
2.21					2	63.31
GLACIAL TILL: Very dense to dense brown silty sand with gravel and cobbles	SS	4	58	50+		
					3	62.31
	SS	5	33	30		
					4	61.31
	SS	6	54	21		
	SS	7	100	42		
5.38					5	60.31
BEDROCK: Good to fair quality, grey limestone	RC	1	89	86		
					6	59.31
	RC	2	100	71		
					7	58.31
7.60						
End of Borehole						
(GWL @ 4.55m - May 23, 2023)						



SOIL PROFILE AND TEST DATA

Phase II - Environmental Site Assessment
370 Athlone Avenue
Ottawa, Ontario

DATUM Elevations are referenced to a geodetic datum

REMARKS

BORINGS BY CME 55 Power Auger

DATE May 18, 2023

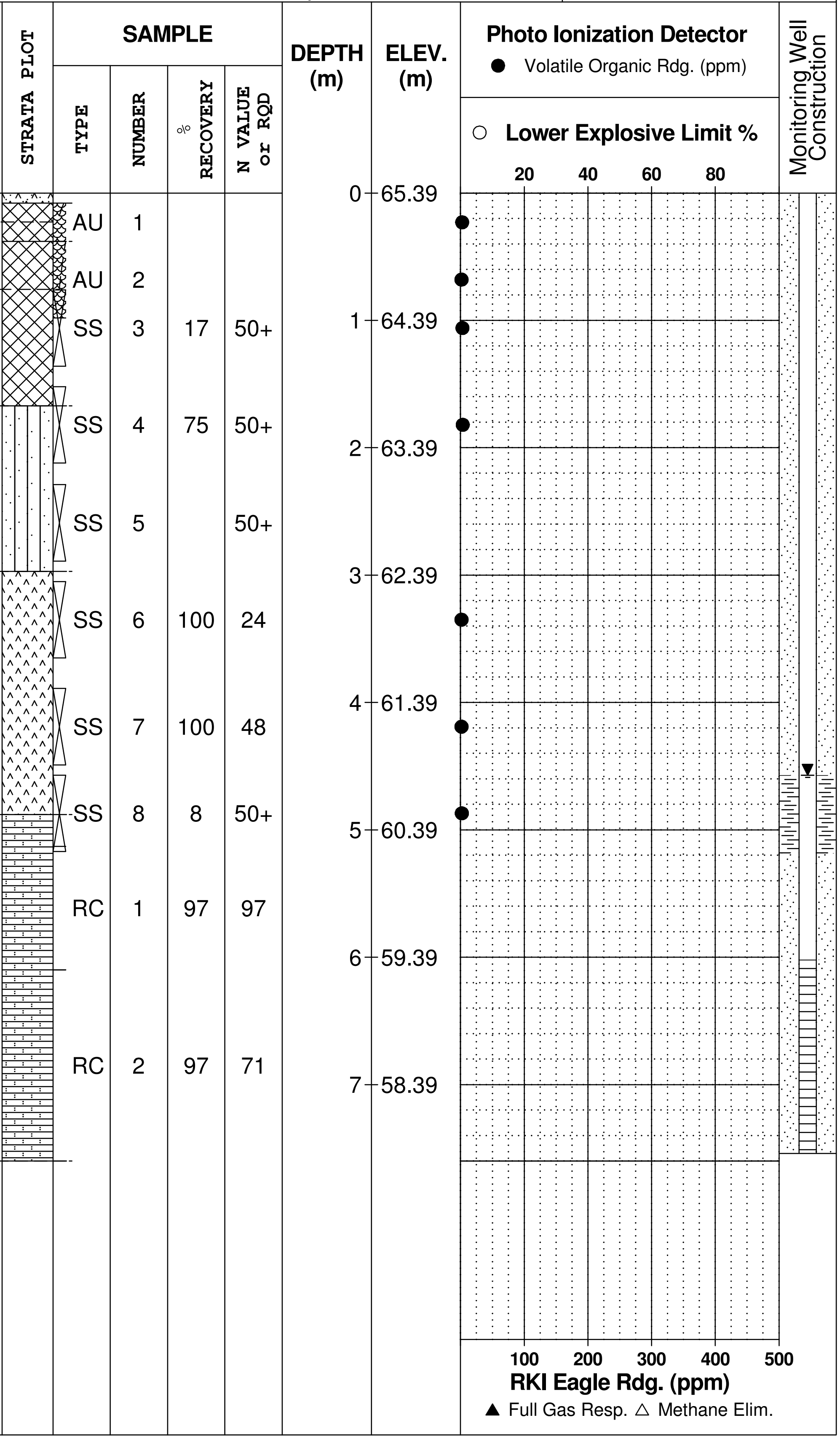
FILE NO. PE6096

HOLE NO. BH 3-23

SOIL DESCRIPTION	Depth (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Concrete Slab	0.08				
FILL: Brown silty sand with topsoil, crushed stone, cobbles trace gravel and asphalt	0.38	AU	1		
		AU	2		
FILL: Brown silty sand with crushed stone, gravel, occasional cobbles	1.67	SS	3	17	50+
Compact to dense, brown **SILTY SAND** with gravel	2.97	SS	4	75	50+
		SS	5		50+
GLACIAL TILL: Compact to dense, brown sandy silt with gravel and cobbles	4.88	SS	6	100	24
		SS	7	100	48
		SS	8	8	50+
BEDROCK: Excellent to fair quality, grey limestone	7.60	RC	1	97	97
		RC	2	97	71

End of Borehole

(GWL @ 4.57m - May 23, 2023)



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

SYMBOLS AND TERMS (continued)

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)

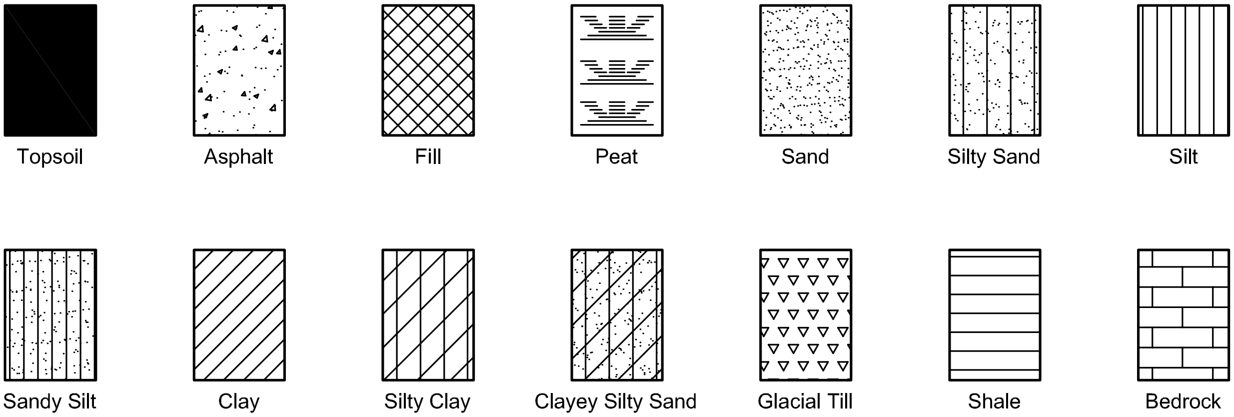
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

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.

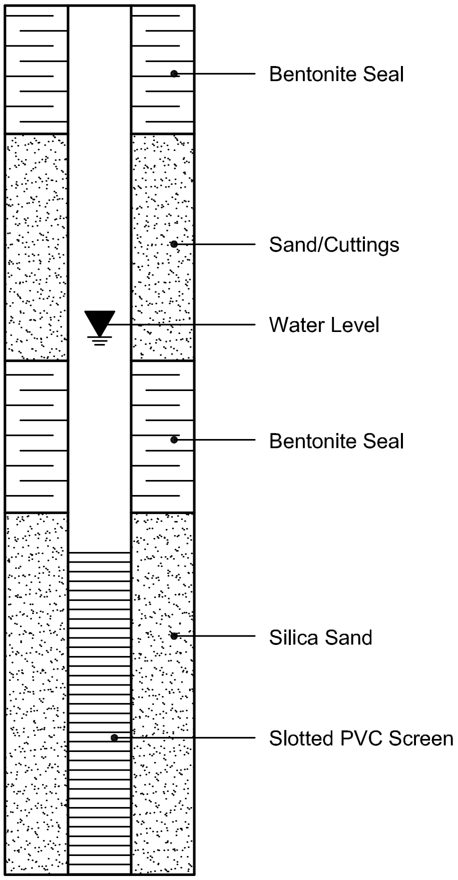
SYMBOLS AND TERMS (continued)

STRATA PLOT

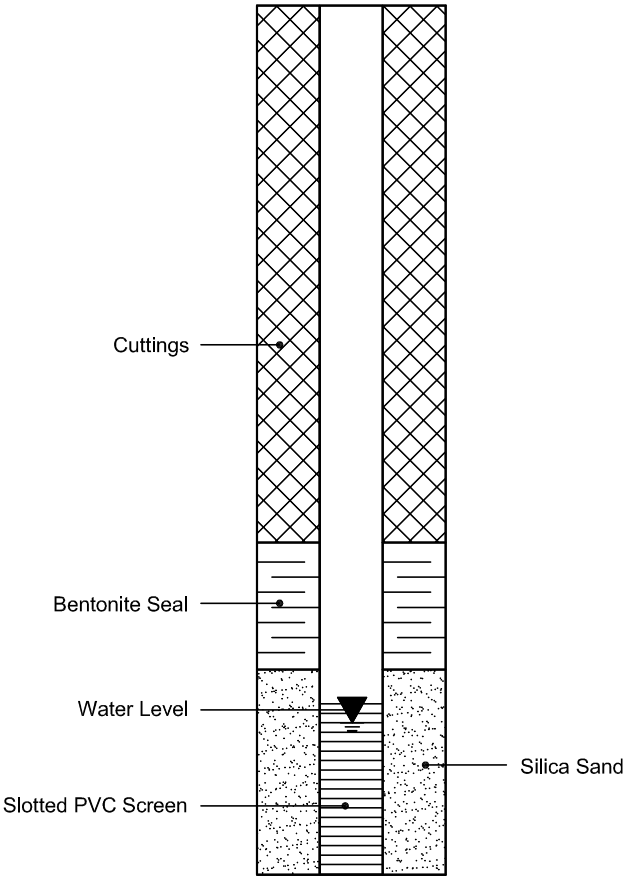


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



APPENDIX 2

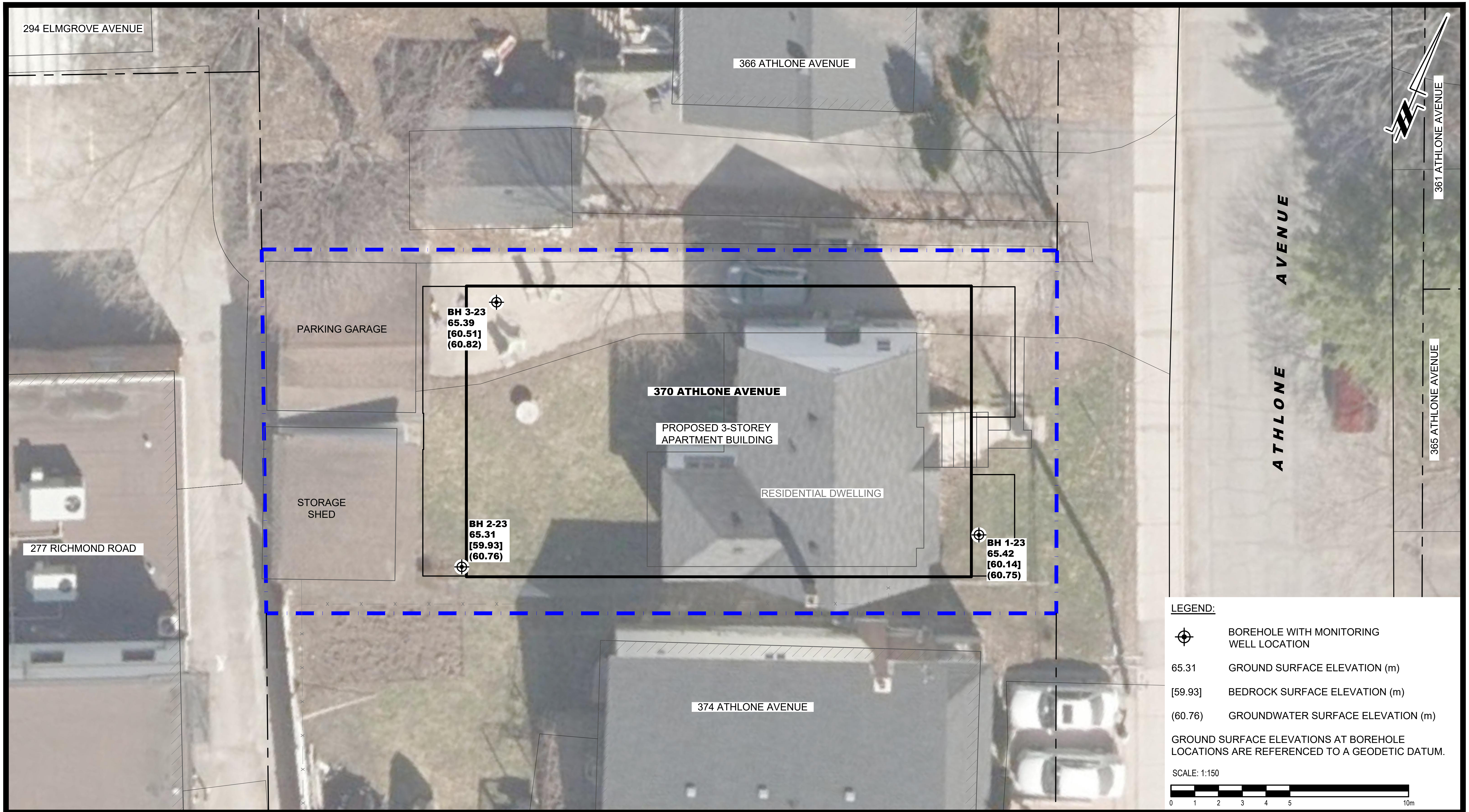
FIGURE 1 – KEY PLAN

DRAWING PG6996-1 – TEST HOLE LOCATION PLAN

FIGURE 1

KEY PLAN

PATERSON
GROUP



LEGEND:

⌖	BOREHOLE WITH MONITORING WELL LOCATION
65.31	GROUND SURFACE ELEVATION (m)
[59.93]	BEDROCK SURFACE ELEVATION (m)
(60.76)	GROUNDWATER SURFACE ELEVATION (m)

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

SCALE: 1:150

0 1 2 3 4 5 10m

PATERSON GROUP

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

NO.	REVISIONS	DATE	INITIAL

JERSEY DEVELOPMENTS INC.
GEOTECHNICAL INVESTIGATION
PROPOSED MULTI-STOREY BUILDING
370 ATHLONE AVENUE

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:150
Date: 01/2024
Drawn by: ZS
Report No.: PG6996-1
Checked by: DR
Dwg. No.: PG6996-1
Approved by: SD
Revision No.:

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