

# **Geotechnical Investigation**

## **Proposed Mid-Rise Building**

1452-1470 Hunt Club Road and 1525-1531 Sieveright Avenue  
Ottawa, Ontario

Prepared for Larga Baffin c/o DCR Pheonix Homes

Report PG5499-1 Revision 1 dated April 3, 2024

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## 1.0 Introduction

Paterson Group (Paterson) was commissioned by Larga Baffin c/o DCR Pheonix Homes to conduct a geotechnical investigation for the proposed residential buildings to be located at 1452-1470 Hunt Club Road and 1525-1531 Sieveright Avenue in the City of Ottawa (reference should be made 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 a test hole program.
- 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.

Investigating the presence or potential presence of contamination on the subject site 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

It is understood that the proposed development will consist of a multi-storey residential building containing five separate wings with associated access lanes, at grade parking and landscaped areas. The proposed building will be located on the northern portion of the subject site and consist of 5 stories with one basement level of underground parking.

It is anticipated that the proposed building will be municipally serviced and that the existing structures will be demolished in support of the proposed development.

## **3.0 Method of Investigation**

### **3.1 Field Investigation**

#### **Field Program**

The field program for the geotechnical investigation was carried out on August 25 and August 26, 2020. At that time, seven (7) boreholes were advanced to a maximum depth of 7.6 m below the existing ground surface. The test hole locations were determined 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 PG5499-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were advanced using a low clearance auger drill rig operated by a two-person crew. The drilling 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 from the boreholes were recovered from the auger flights or a 50 mm diameter split-spoon sampler. All soil samples were classified on site, placed in sealed plastic bags and transported to the laboratory for further review. The depths at which the auger and split spoon samples were recovered from the test holes are presented as AU and SS, respectively, on the Soil Profile and Test Data sheets presented in Appendix 1.

Standard Penetration Testing (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.

Undrained shear strength testing, using a vane apparatus, was carried out at regular intervals of depth in cohesive soils.

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

## **Groundwater**

Monitoring wells were installed in boreholes BH 2-20, BH 4-20, BH 6-20 and BH 7-20 during the field investigation to permit monitoring of the groundwater levels subsequent to the completion of the sampling program. Flexible polyethylene standpipes were installed within all remaining boreholes.

## **Sample Storage**

All samples from the investigation were stored in the laboratory for a period of one month after issuance of the initial report. The samples were then discarded unless directed otherwise.

### **3.2 Field Survey**

The test hole locations were selected by Paterson to provide general coverage of the proposed development taking into consideration existing site features and underground utilities. The test hole locations and ground surface elevations at each test hole location were surveyed by Paterson personnel. The ground surface elevations at the borehole locations were referenced to a geodetic datum. The test hole locations are presented on Drawing PG5499-1 - Test Hole Location Plan in Appendix 2.

### **3.3 Laboratory Testing**

Soil samples were recovered from the subject site and visually examined in our laboratory to review the results of the field logging.

### **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 samples. The results are presented in Appendix 1 and are discussed further in Section 6.7.

## **4.0 Observations**

### **4.1 Surface Conditions**

The subject site is currently occupied by an automobile dealership at 1452-1470 Hunt Club Road and commercial storage buildings at 1525-1531 Sieveright Avenue. Several one-storey structures are present throughout the west and north portion of the subject site. The north and west portions of the subject site are surfaced with asphalt while the remaining portions of the site are grass-covered with gravel pads scattered throughout the site.

The subject site is relatively flat and at grade with the surrounding properties. The subject site is bordered to the north by Hunt Club Road, to the west by one-storey commercial buildings, to the south by Sieveright Avenue and to the east by existing residential dwellings along Issam Private.

### **4.2 Subsurface Profile**

#### **Overburden**

Generally, the subsurface profile encountered at the borehole locations consists of topsoil, asphalt or fill underlain by a layer of sand or silty sand. Fill was encountered overlying the native sand layer and extending to depths between 0.5 to 2.3 m below existing ground surface at all boreholes with the exception of BH 5-20. The native sand deposit was observed to be underlain by a deposit of stiff grey silty clay extending to depths between 3.0 and 6.7 m below the existing ground surface. The silty clay deposit was observed to be further underlain by layers of sandy silt and silty sand throughout the north and central portion of the site.

Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for details of the soil profiles encountered at each test hole location.

#### **Bedrock**

Based on available geological mapping, the subject site is located in an area where the bedrock consists of shale of the Carlsbad formation with a drift thickness between 15 and 25 m.

### 4.3 Groundwater

Groundwater levels were measured in the monitoring wells and piezometers on September 2 and September 8, 2020, respectively. The groundwater level within the monitoring well located at BH 4-20 was measured on February 15, 2024. The observed groundwater levels are summarized in Table 1 below.

<b>Table 1 – Summary of Groundwater Levels</b>				
<b>Test Hole Number</b>	<b>Ground Surface Elevation (m)</b>	<b>Measured Groundwater Levels</b>		<b>Dated Recorded</b>
		<b>Depth (m)</b>	<b>Elevation (m)</b>	
BH 1-20	92.87	Damaged	n/a	September 8, 2020
BH 2-20*	93.36	4.61	88.75	September 2, 2020
BH 3-20	93.31	Damaged	n/a	September 8, 2020
BH 4-20*	92.66	2.41	90.25	September 2, 2020
BH 4-20*	92.66	2.52	90.14	February 15, 2024
BH 5-20	93.40	0.95	92.45	September 8, 2020
BH 6-20*	92.91	4.35	88.56	September 2, 2020
BH 7-20*	93.01	5.48	87.53	September 2, 2020
<b>Notes:</b> The ground surface elevations at the borehole locations are referenced to a geodetic datum. - “*” indicates monitoring well installed within borehole.				

It should be noted that the groundwater levels could be influenced by surface water infiltrating the backfilled boreholes. Long-term groundwater levels can also be estimated based on the observed color and consistency of the recovered soil samples. Based on these observations, the long-term groundwater table can be expected at approximately 2.5 to 3.5 m below ground surface. The recorded groundwater levels are noted on the applicable Soil Profile and Test Data sheet presented in Appendix 1.

It should be noted that groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater level could vary at the time of construction.



## **5.0 Discussion**

### **5.1 Geotechnical Assessment**

From a geotechnical perspective, the subject site is suitable for the proposed development. It is expected that the proposed buildings will be founded on conventional spread footings placed on a stiff grey silty clay, compact grey silty sand and/or engineered fill bearing surface.

Where the existing fill is encountered at design underside of footing elevation, it is anticipated that the footings will be extended to an undisturbed bearing surface or placed on an approved engineered fill placed on an undisturbed bearing surface.

Due to the presence of a silty clay deposit throughout the subject site, a permissible grade raise restriction will be required for grading around the proposed building founded over the existing silty clay deposit.

### **5.2 Site Grading and Preparation**

#### **Stripping Depth**

Topsoil and fill, such as those containing organic or deleterious materials, should be stripped from under any buildings and other settlement sensitive structures.

It is anticipated that the existing fill, free of deleterious material and significant amounts of organics, can be left in place below the proposed building footprint, outside of lateral support zones for the footings, and below the proposed parking area and access lane. However, it is recommended that the existing fill layer where encountered within settlement sensitive areas be proof-rolled several times using a suitably-sized vibratory sheepsfoot roller under dry conditions and above freezing temperatures and approved by Paterson field personnel at the time of construction. Any poor performing areas noted during the proof-rolling operation should be removed and replaced with an approved fill.

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

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.

## **Fill Placement**

Fill used for grading purposes beneath the proposed buildings should consist of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. The fill should be tested and approved prior to delivery to the site. It should be placed in lifts no greater than 300 mm in thickness and compacted using suitable compaction equipment for the specified lift thickness. Fill placed beneath the building areas should be compacted to at least 98% of its standard Proctor maximum dry density (SPMDD).

Non-specified existing fill along with site-excavated soil can be used as general landscaping fill where settlement of the ground surface is of minor concern. These materials should be spread in thin lifts and be compacted at minimum by the tracks of the spreading equipment to minimize voids. If these materials are to be used to build up the subgrade level for areas to be paved, they should be compacted in thin lifts to a minimum density of 95% of their respective SPMDD. Non-specified existing fill and site-excavated soils are not suitable for use as backfill against foundation walls.

## **5.3 Foundation Design**

### **Bearing Resistance Values**

Strip footings, up to 2 m wide, and pad footings, up to 8 m wide, placed on an undisturbed, stiff grey silty clay bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **100 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **150 kPa**.

Footings placed on an undisturbed, compact silty sand bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **150 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **225 kPa**.

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

Where existing fill is encountered directly below the underside of footing (USF), the footings may be required to be lowered to an undisturbed, native bearing surface. Footings placed over an approved engineered fill bearing surface can be designed using a bearing resistance value at SLS of **150 kPa** and a factored bearing resistance value at ULS of **225 kPa**.

Footings designed using the bearing resistance value at SLS given above will be subjected to potential post construction total and differential settlements of 25 and 20 mm, respectively. A geotechnical resistance factor of 0.5 was applied to the above noted bearing resistance values at ULS.

### **Proof Rolling and Subgrade Improvement for Loose Sand Below Footings**

Where the sand bearing surface for footings is considered loose by Paterson field personnel at the time of construction, it would be recommended to proof roll the bearing surface prior to forming for footings. Proof-rolling is recommended to be undertaken in dry conditions and above freezing temperatures by an adequately sized vibratory roller making several passes to achieve optimal compaction levels.

The compaction program should be reviewed and approved by Paterson at the time of construction and would take place in conjunction with bearing medium review inspections for footings. Depending on the looseness and degree of saturation of sandy soils at the time of construction, other measures (additional compaction, sub-excavation and reinstatement of crushed stone fill, mud slab) may be recommended to accommodate site conditions at the time of construction. However, these considerations would be evaluated at the time of design by Paterson on a footing-specific basis.

### **Lateral Support**

The bearing medium under footing-supported structures is required to be provided with adequate lateral support. Adequate lateral support is provided to a silty sand, sandy silt, silty clay or engineered fill bearing medium above the groundwater table when a plane extending down and out from the bottom edge of the footing, at a minimum of 1.5H:1V passes through in situ soil of the same or high bearing medium soil.

### **Permissible Grade Raise Restrictions**

Based on the results of our field investigation, a permissible grade raise restriction for the subject site of 2.0 m can be used for design purposes. 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.

## 5.4 Design for Earthquakes

The subject site can be taken as seismic site response **Class D** as defined in Table 4.1.8.4.A of the Ontario Building Code (OBC) 2012 for foundations considered at this site. The soils underlying the subject site are not susceptible to liquefaction. Reference should be made to the latest revision of the Ontario Building Code for a full discussion of the earthquake design requirements.

## 5.5 Basement Slab

With the removal of all topsoil and deleterious fill, such as those containing organic materials, within the footprint of the proposed building, the in-situ soil or engineered fill surface will be considered to be an acceptable subgrade on which to commence backfilling for floor slab construction.

Any soft areas should be removed and backfilled with appropriate backfill material prior to placing any fill. OPSS Granular B Type II compacted to a minimum of 98% of the material's SPMDD are recommended for backfilling below the floor slab.

It is expected that the basement area for the proposed building 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 proposed where a concrete floor slab will be used, it is recommended that the upper 200 mm of sub-slab fill consist of OPSS Granular A crushed stone compacted to 98% of the materials SPMDD.

A sub-slab drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided under the lowest level floor slab where a basement level is provided. The spacing of the sub-slab drainage pipes should be advised by Paterson during the design phase and once the footing and sump pit locations are known. The footprint would be confirmed at the time of construction once groundwater infiltration can be best assessed, if any. This is discussed further in Subsection 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 subject structure. However, the conditions can be well-represented by assuming the retained material has an angle of internal friction of 30 degrees and a bulk (drained) unit weight of 20 kN/m<sup>3</sup>.

Where undrained conditions are anticipated (i.e. below the groundwater level), the applicable effective (undrained) unit weight of the retained material can be taken as  $13 \text{ kN/m}^3$ , where applicable. A hydrostatic pressure should be added to the total static earth pressure when using the effective unit weight.

### **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)  
 $\gamma$  = unit weight of fill of the applicable retained soil ( $\text{kN/m}^3$ )  
 $H$  = 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 ( $\Delta P_{AE}$ ).

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

$a_c = (1.45 - a_{max}/g)a_{max}$   
 $\gamma$  = unit weight of fill of the applicable retained soil ( $\text{kN/m}^3$ )  
 $H$  = height of the wall (m)  
 $g$  = gravity,  $9.81 \text{ m/s}^2$

The peak ground acceleration, ( $a_{max}$ ), for the Ottawa area is 0.32 g 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  $K_o = 0.5$  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 OBC 2012.

## 5.7 Pavement Structure

The recommended pavement structures for the subject site are shown in Table 2, Table 3 and Table 4.

<b>Table 2 – Recommended Pavement Structure – Car Only Parking Areas</b>	
<b>Thickness (mm)</b>	<b>Material Description</b>
50	<b>Wear Course</b> - HL 3 or Superpave 12.5 Asphaltic Concrete
150	<b>BASE</b> - OPSS Granular A Crushed Stone
300	<b>SUBBASE</b> - OPSS Granular B Type II
<b>SUBGRADE</b> - Either fill, in situ soil or OPSS Granular B Type I or II material placed over in situ soil.	

<b>Table 3 – Recommended Pavement Structure – Access Lanes and Ramp</b>	
<b>Thickness (mm)</b>	<b>Material Description</b>
40	<b>Wear Course</b> - HL-3 or Superpave 12.5 Asphaltic Concrete
50	<b>Binder Course</b> - HL-8 or Superpave 19.0 Asphaltic Concrete
150	<b>BASE</b> - OPSS Granular A Crushed Stone
450	<b>SUBBASE</b> - OPSS Granular B Type II
<b>SUBGRADE</b> - Either fill, in situ soil, or OPSS Granular B Type I or II material placed over in situ soil.	

<b>Table 4 – Recommended Rigid Pavement Structure – Lower Parking Level</b>	
<b>Thickness (mm)</b>	<b>Material Description</b>
Specified by Others	<b>32 MPa Concrete</b>
300	<b>BASE</b> - OPSS Granular A Crushed Stone
<b>SUBGRADE</b> - Either fill, in situ soil, 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 I or 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 compaction equipment.

### **Pavement Structure Drainage**

Satisfactory performance of the pavement structure is largely dependent on keeping 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 its load carrying capacity.

Where silty clay is anticipated at subgrade level, consideration should be given to installing subdrains during the pavement construction. The sub-drain inverts should be approximately 300 mm below subgrade level and run longitudinal along the curblines. The subgrade surface should be crowned to promote water flow to the drainage lines.



## **6.0 Design and Construction Precautions**

### **6.1 Foundation Drainage and Backfill**

#### **Perimeter Foundation Drainage System**

It is recommended that a perimeter foundation drainage system be provided for the proposed structure. The system should consist of a 100 to 150 mm diameter perforated corrugated plastic pipe, surrounded on all sides by a minimum of 150 mm of 19 mm clear crushed stone, placed at the footing level around the exterior perimeter of the structure where double-sided pours will be undertaken. In areas where blind-sided pours will be considered, the perimeter drainage pipe should be placed along the interior side of the foundation wall and connected to sleeves placed within the foundation wall at a 6 m center-to-center spacing. The pipe should have a positive outlet, such as a gravity connection to the storm sewer.

It is anticipated that underfloor drainage will be required to control water infiltration below the proposed basement level. The layout of the perimeter and underfloor drainage systems should be determined by Paterson during the design phase once the foundation structure and sump pit locations are known. The perimeter drainage pipe would connect to a series of underfloor drainage lines which would direct water to sump pit(s) within the lower basement area.

The top endlap for all portions of the foundation drainage board that terminate against the top surface of a podium level should be sealed at the top with a waterproofing membrane to mitigate the potential for surface water to migrate behind the foundation drainage board layer.

A positive-side (i.e., placed on exterior faces) waterproofing system should also be provided for any elevator shafts and pools located within the lowest basement level. It is recommended that Paterson review and advise on all basement waterproofing/drainage system designs during the design phase.

#### **Foundation Backfill**

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 composite drainage system, such as CCW MiraDRAIN 2000 or Delta-Teraxx or an approved equivalent. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should otherwise be used for this purpose.



## **6.2 Protection of Footings 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 (or equivalent) should be provided in this regard.

A minimum of 2.1 m thick soil cover (or equivalent) should be provided for exterior unheated footings, not thermally connected to a heated space, such as exterior columns and/or wing walls.

It has been our experience that insufficient soil cover is typically provided to footings located in areas where minimal soil cover is available, such as entrance ramps to underground parking garages. Paterson requests permission to review design drawings prior to construction to ensure proper frost protection is provided.

## **6.3 Excavation Side Slopes**

The side slopes of excavations in the soil and fill 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. It is assumed that sufficient room will be available for the greater part of the excavations to be undertaken by open-cut methods (i.e. unsupported excavations).

### **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 soils are considered to be 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 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. Services are expected to be installed by “cut and cover” methods and excavations should not remain open for extended periods of time.

## Temporary Shoring

Temporary shoring may be considered to retain the overburden soil to complete the required excavations where insufficient room is available for open cut methods. The shoring requirements designed by a structural engineer specializing in those works, or Paterson, will depend on the depth of the excavation, the proximity of the adjacent structures and the elevation of the adjacent building foundations and underground services. The design and implementation of these temporary systems will be the responsibility of the excavation contractor and their design team.

Inspections and approval of the temporary system will also be the responsibility of the designer. Geotechnical information provided below is to assist the designer in completing a suitable and safe shoring system. The designer should take into account the impact of a significant precipitation event and designate design measures to ensure that a precipitation 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 design prior to implementation.

The temporary system could consist of 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 included to the earth pressures described below. These systems could be cantilevered, anchored, or braced. Given the sandy nature of the soils present throughout the subject site, the designer should consider provisions to mitigate the potential for excessive losses of retained soil during the lagging installation process if consideration is given to using a soldier pile and lagging system.

Generally, it is expected that the shoring systems will be provided with tie-back rock anchors to ensure their stability. The shoring system is recommended to be adequately supported to resist toe failure and inspected to ensure that the sheet piles extend well below the excavation base. It should be noted if consideration is being given to utilizing a raker style support for the shoring system that lateral movements can occur, and the structural engineer should ensure that the design selected minimizes these movements to tolerable levels.

The earth pressures acting on the shoring system may be calculated using the parameters provided in Table 6.

**Table 6 - 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 ( $\gamma$ ), kN/m <sup>3</sup>	20
Submerged Unit Weight ( $\gamma'$ ), kN/m <sup>3</sup>	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 level.

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.

At least 150 mm of OPSS Granular A crushed stone should be used for pipe bedding for sewer and water pipes. The bedding should extend to the spring line of the pipe. Cover material, from the spring line to at least 300 mm above the obvert of the pipe, should consist of OPSS Granular A or Granular B Type II with a maximum size of 25 mm. The bedding and cover materials should be placed in maximum 225 mm thick lifts compacted to 99% of the material's SPMDD.

Generally, it should be possible to re-use the moist (not wet) brown silty clay above the cover material if the excavation and filling operations are carried out in dry weather conditions. Wet silty clay should be given a sufficient drying period to decrease its moisture content to an acceptable level to make compaction possible prior to being reused.

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 consist of 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 SPMDD.

To reduce long-term lowering of the groundwater level at this site, clay seals should be provided in the service trenches where a clay subgrade is encountered. The seals should be at least 1.5 m long and should extend from trench wall to trench wall. Generally, the seals should extend from the frost line and fully penetrate the bedding, subbedding and cover material. The barriers should consist of relatively dry and compactable brown silty clay placed in maximum 225 mm thick loose layers and compacted to a minimum of 95% of the material's SPMDD. The clay seals should be placed at the site boundaries and at strategic locations at no more than 60 m intervals in the service trenches.

## **6.5 Groundwater Control**

It is anticipated that groundwater infiltration into the excavations may be moderate and initially high as sub-excavations encounter the grey silty sand/sandy silt deposit. It is expected the majority of the infiltration will be controllable using open sumps after the initial excavation is lowered to the founding depth, however, may require higher capacity sumps for the initial portion of the excavation. It is recommended that hydraulic conductivity testing be complete during the design phase by Paterson to better estimate the volume of influx that may be handled during the construction phase. The contractor should be prepared to direct water away from all bearing surfaces and 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 Person as stipulated under O.Reg. 63/16.

## **Long-term Groundwater Control**

Our recommendations for the proposed building's long-term groundwater control are presented in Subsection 6.1. Any groundwater which encounters the building's perimeter groundwater infiltration control system will be directed to the proposed building's sump pit. It is expected that groundwater flow will be low (i.e. less than 25,000 L/day with peak periods noted after rain events). It is anticipated that the groundwater flow will be controllable using conventional open sumps.

## **Impacts on Neighboring Structures**

It is understood that one level of underground parking is planned for the proposed building. Based on the proximity of neighboring buildings and minimal zone impacted by the groundwater lowering, the proposed development will not negatively impact the neighboring structures. It should be noted that no issues are expected with respect to groundwater lowering that would cause long term damage to adjacent structures surrounding the proposed building.

## **6.6 Winter Construction**

The subsurface soil conditions contain 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 installation of straw, propane heaters and tarpaulins or other suitable means.

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.

The trench excavations should be constructed to avoid the introduction of frozen materials, snow or ice into the trenches. As well, pavement construction is difficult during winter. The subgrade consists of frost susceptible soils which will experience total and differential frost heaving during construction. Also, the introduction of frost, snow or ice into the pavement materials or fill used to backfill the lower basement level, which is difficult to avoid during winter conditions, will greatly negatively affect the performance of the fill and impact construction schedules.

## **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 slightly aggressive corrosive environment.

## 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 and structural plan(s) from a geotechnical perspective.
- Review of the geotechnical aspects of the excavation contractor's shoring design, prior to construction, if applicable.
- Review of architectural plans pertaining to foundation and underfloor drainage systems and waterproofing details for elevator shafts and pools.

A material testing and observation services program is a requirement for the provided foundation design data to be applicable. The following aspects of the program should be performed by Paterson:

- Observation of all bearing surfaces prior to the placement of concrete.
- Inspection of all foundation drainage and groundwater infiltration control systems.
- Sampling and testing of the concrete and fill materials used.
- 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.
- Field density tests to determine the level of compaction achieved.
- Sampling and testing of the bituminous concrete including mix design reviews.

A report confirming the work has been conducted in general accordance with the recommendations could be issued, upon request, following the completion of a satisfactory materials testing and observation program by Paterson.

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 Larga Baffin c/o DCR Phoenix 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.



Killian Bell, B.Eng.



Drew Petahtegoose, P.Eng.

### Report Distribution:

- ☐ Larga Baffin c/o DCR Phoenix Group (1 email copy)
- ☐ Paterson Group (1 copy)



# APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

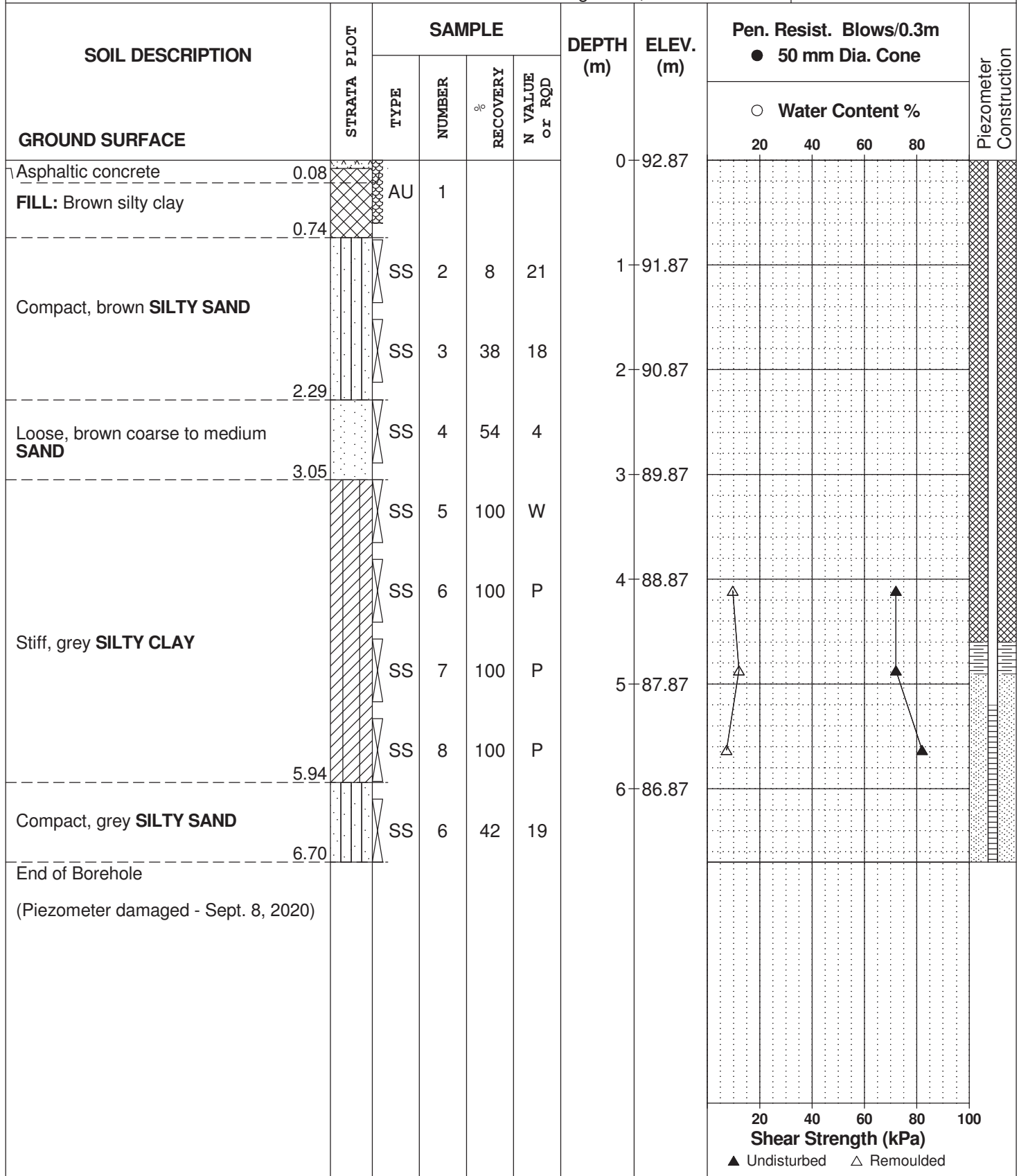
DATE August 25, 2020

FILE NO.

PG5499

HOLE NO.

BH 1-20



## SOIL PROFILE AND TEST DATA

Geotechnical Investigation

1452-1470 Hunt Club Road & 1525-1531 Sieveright Ave.  
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE August 25, 2020

FILE NO.

PG5499

HOLE NO.

BH 2-20

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone				Monitoring Well Construction	
		TYPE	NUMBER	RECOVERY %	N VALUE or RQD			○ Water Content %					
								20	40	60	80		
GROUND SURFACE													
FILL: Brown sand with crushed stone, trace asphalt	0.51	AU	1			0	93.36						
FILL: Brown silty clay with sand and gravel	1.52	SS	2	79	11	1	92.36						
Loose, brown coarse to medium SAND	2.06	SS	3	58	9	2	91.36						
Stiff, grey SILTY CLAY, trace sand seams	3.66	SS	4	75	3	3	90.36						
		SS	5	100	5								
Compact, grey SANDY SILT  - some clay by 4.6m depth	6.10	SS	6	54	23	4	89.36					▲	
		SS	7	62	26	5	88.36						
		SS	8	75	15	6	87.36						
Compact, grey SILTY SAND	6.70	SS	9	46	24								
End of Borehole  (GWL @ 4.61m - Sept. 2, 2020)													
								Shear Strength (kPa)					
								20	40	60	80	100	
								▲ Undisturbed    △ Remoulded					

## SOIL PROFILE AND TEST DATA

**1452-1470 Hunt Club Road & 1525-1531 Sieveright Ave.  
Ottawa, Ontario**

FILE NO. PG5499

HOLE NO. **BH 3-20**

DATE August 26, 2020

[illegible]

DATUM	Geodetic
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FILE NO.

PG5499

REMARKS

HOLE NO.

BH 4-20

**BORINGS BY** CME-55 Low Clearance Drill

DATE August 25, 2020

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone				Monitoring Well Construction
		TYPE	NUMBER	RECOVERY %	N VALUE or RQD			○ Water Content %				
								20	40	60	80	
<b>GROUND SURFACE</b>												
Asphaltic concrete 0.08	[Pattern]	AU	1			0	92.66					
FILL: Brown silty sand with crushed stone 0.60	[Pattern]											
Compact, brown medium to coarse SAND	[Pattern]	SS	2	71	12	1	91.66					
	[Pattern]	SS	3	79	12	2	90.66					
	[Pattern]	SS	4	62	16	3	89.66					
	[Pattern]	SS	5	67	16	4	88.66					
	[Pattern]	SS	6	29	9	5	87.66					
Stiff, grey SILTY CLAY	[Pattern]	SS	7	83	W	6	86.66	△		▲		
	[Pattern]	SS	8	100	2							
End of Borehole 6.70	[Pattern]											
(GWL @ 2.41m - Sept. 2, 2020)												

Shear Strength (kPa)  
 ▲ Undisturbed    △ Remoulded

## SOIL PROFILE AND TEST DATA

Geotechnical Investigation

1452-1470 Hunt Club Road & 1525-1531 Sieveright Ave.  
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

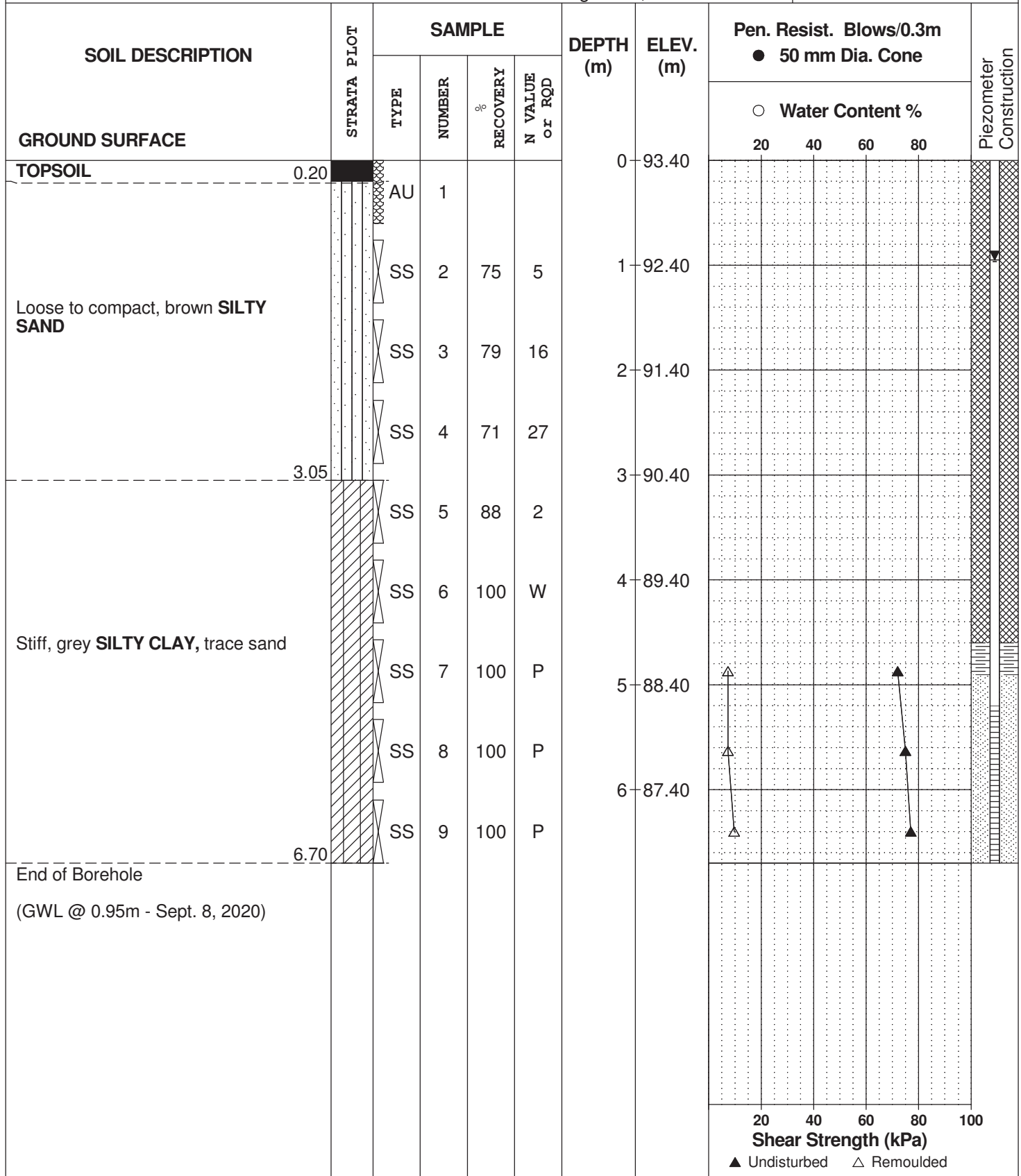
DATE August 25, 2020

FILE NO.

PG5499

HOLE NO.

BH 5-20



**DATUM**      Geodetic

FILE NO. PG5499

REMARKS

HOLE NO. **BH 6-20**

**BORINGS BY CME-55 Low Clearance Drill**

**DATE** August 25, 2020

[illegible]

**DATUM**      Geodetic

REMARKS

**BORINGS BY** CME-55 Low Clearance Drill

DATE August 25, 2020

FILE NO.

PG5499

HOLE NO.

**BH 7-20**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone				Monitoring Well Construction
		TYPE	NUMBER	RECOVERY %	N VALUE or RQD			○ Water Content %				
								20	40	60	80	
GROUND SURFACE						0	93.01					
FILL: Brown silty sand with crushed stone, trace organics 0.56		AU	1									
FILL: Brown silty clay, some sand 1.52		SS	2	38	8	1	92.01					
Stiff, brown SILTY CLAY, trace sand 3.05		SS	3	21	7	2	91.01					
		SS	4	38	7							
Compact, brown SILTY SAND 4.57		SS	5	29	13	3	90.01					
		SS	6	46	26	4	89.01					
Compact, grey SANDY SILT 5.13		SS	7	58	26	5	88.01					
Compact, grey medium SAND 6.70		SS	8	38	20	6	87.01					
		SS	9	42	18							
End of Borehole (GWL @ 5.48m - Sept. 2, 2020)												



# 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 strength of cohesionless soils is the relative density, 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.

Relative Density	'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 vane tests, penetrometer tests, unconfined compression tests, or occasionally by Standard Penetration Tests.

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 is the ratio between the undisturbed undrained shear strength and the remoulded undrained shear strength of the soil.

Terminology used for describing soil strata based upon texture, or the proportion of individual particle sizes present is provided on the Textural Soil Classification Chart at the end of this information package.

### **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 NXL size core. However, it can be used on smaller core sizes, such as BX, if the bulk of the fractures caused by drilling stresses (called “mechanical breaks”) are easily distinguishable from the normal in situ fractures.

<b>RQD %</b>	<b>ROCK QUALITY</b>
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
PS	-	Piston sample
AU	-	Auger sample or bulk sample
WS	-	Wash sample
RC	-	Rock core sample (Core bit size AXT, BXL, etc.). Rock core samples are obtained with the use of standard diamond drilling bits.

## SYMBOLS AND TERMS (continued)

### GRAIN SIZE DISTRIBUTION

MC%	-	Natural moisture 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 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 = $(D_{30})^2 / (D_{10} \times D_{60})$
Cu	-	Uniformity coefficient = $D_{60} / D_{10}$

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

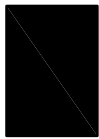
$p'_o$	-	Present effective overburden pressure at sample depth
$p'_c$	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below $p'_c$ )
Cc	-	Compression index (in effect at pressures above $p'_c$ )
OC Ratio		Overconsolidation ratio = $p'_c / p'_o$
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.
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## SYMBOLS AND TERMS (continued)

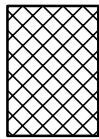
### STRATA PLOT



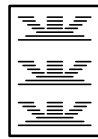
Topsoil



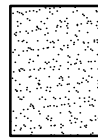
Asphalt



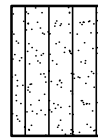
Fill



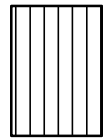
Peat



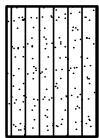
Sand



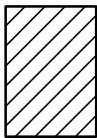
Silty Sand



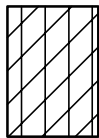
Silt



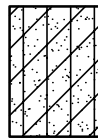
Sandy Silt



Clay



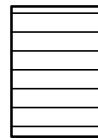
Silty Clay



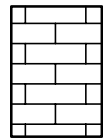
Clayey Silty Sand



Glacial Till



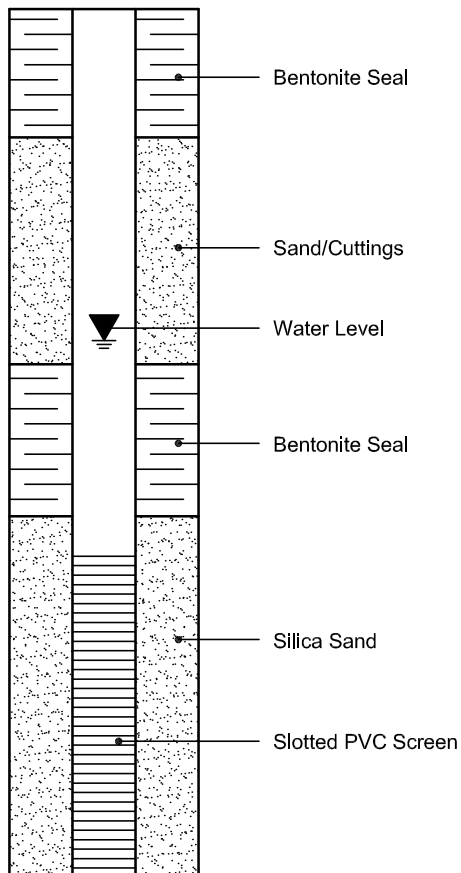
Shale



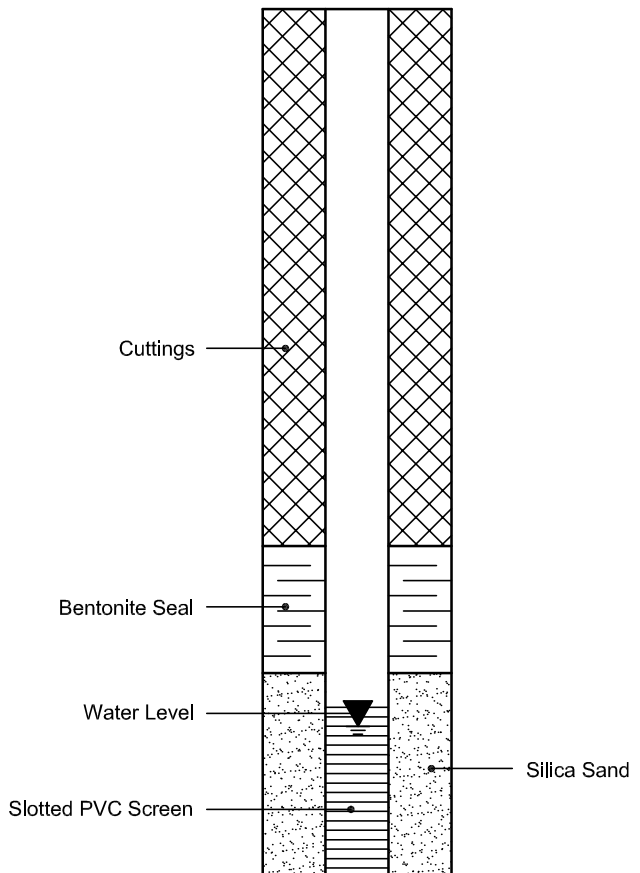
Bedrock

### MONITORING WELL AND PIEZOMETER CONSTRUCTION

#### MONITORING WELL CONSTRUCTION



#### PIEZOMETER CONSTRUCTION



Certificate of Analysis

Report Date: 02-Sep-2020

Client: Paterson Group Consulting Engineers

Order Date: 27-Aug-2020

Client PO: 30698

Project Description: PE5015

Client ID:	BH1-SS4	-	-	-
Sample Date:	26-Aug-20 13:00	-	-	-
Sample ID:	2035554-01	-	-	-
MDL/Units	Soil	-	-	-

**Physical Characteristics**

% Solids	0.1 % by Wt.	88.0	-	-	-
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**General Inorganics**

pH	0.05 pH Units	7.56	-	-	-
Resistivity	0.10 Ohm.m	19.7	-	-	-

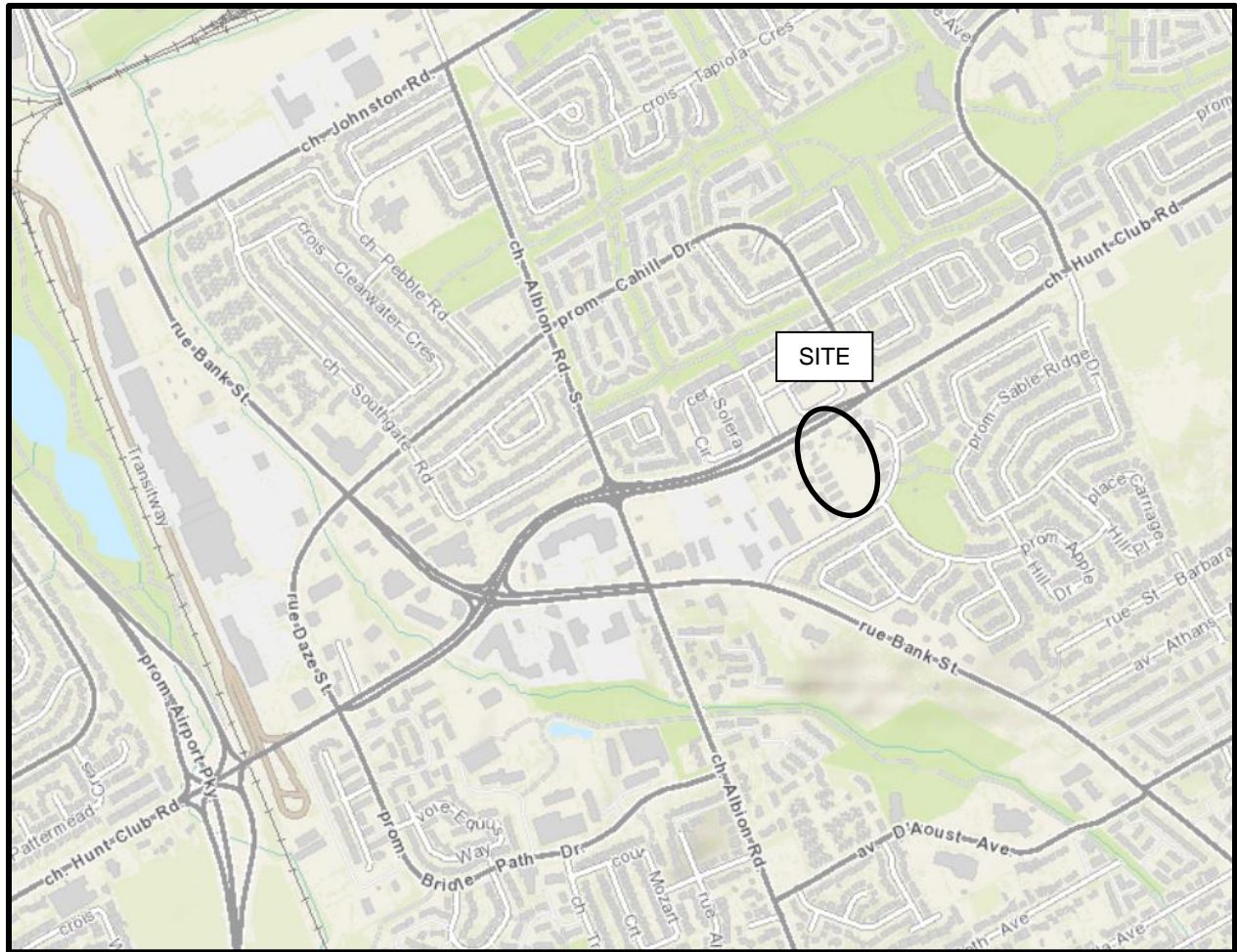
**Anions**

Chloride	5 ug/g dry	202	-	-	-
Sulphate	5 ug/g dry	147	-	-	-

# APPENDIX 2

FIGURE 1 – KEY PLAN

DRAWING PG5499-1 – TEST HOLE LOCATION PLAN



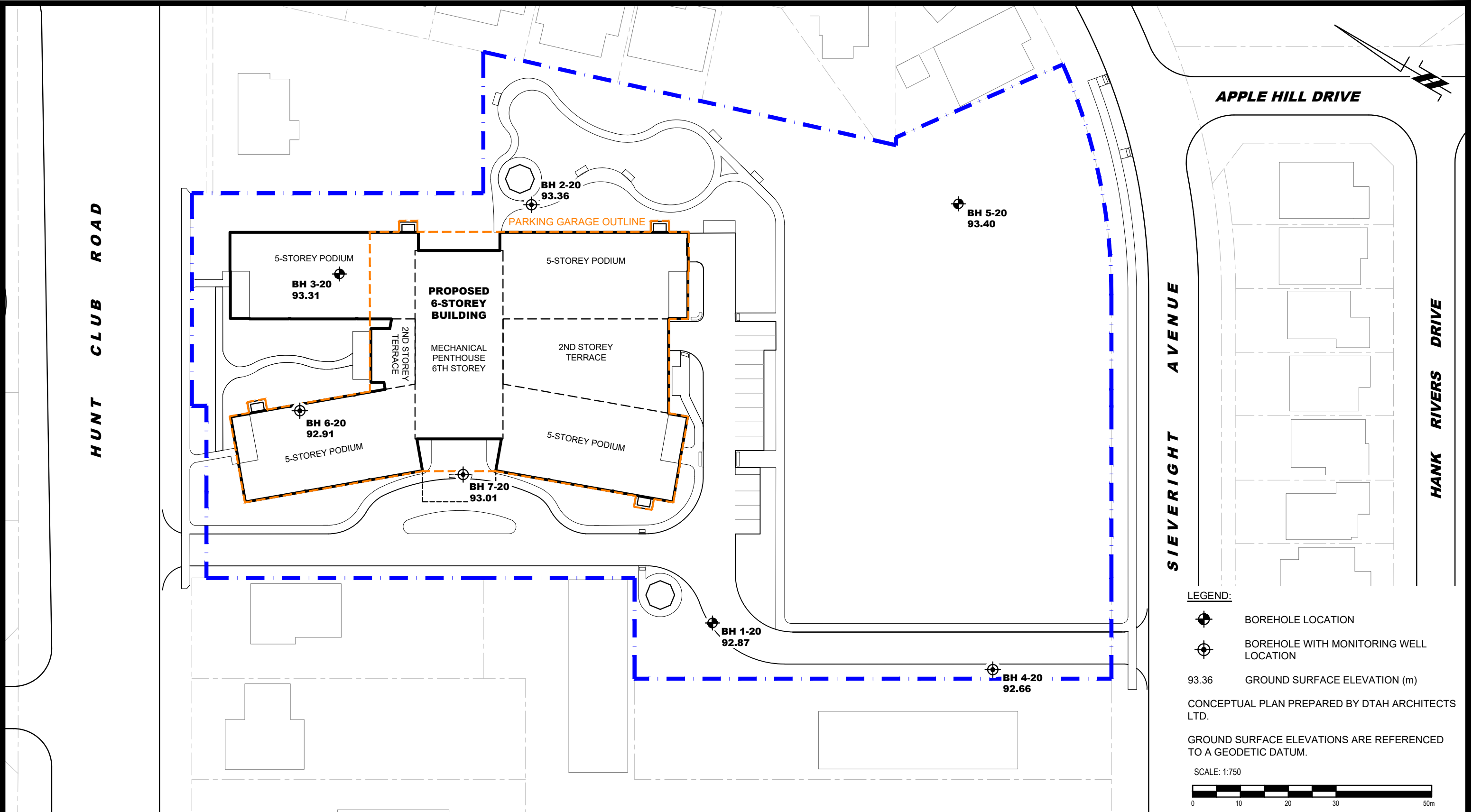
# FIGURE 1

## KEY PLAN



**PATERSON  
GROUP**

9 AURIGA DRIVE  
OTTAWA, ON  
K2E 7S9  
TEL: (613) 226-7381




**LEGEND:**

- BOREHOLE LOCATION
- BOREHOLE WITH MONITORING WELL LOCATION
- 93.36 GROUND SURFACE ELEVATION (m)

CONCEPTUAL PLAN PREPARED BY DTAH ARCHITECTS LTD.

GROUND SURFACE ELEVATIONS ARE REFERENCED TO A GEODETIC DATUM.

SCALE: 1:750

<div><div>9 AURIGA DRIVE OTTAWA, ON K2E 7T9 TEL: (613) 226-7381</div></div>					LARGA BAFFIN C/O DCR PHOENIX HOMES  GEOTECHNICAL INVESTIGATION 1452-1470 HUNT CLUB ROAD AND 1525-1531 SIEVERIGHT AVENUE  OTTAWA, ONTARIO	Scale:	1:750	Date:	09/2020
						Drawn by:	MPG	Report No.:	PG5499-1
						Checked by:	DP	Dwg. No.:	PG5499-1
						Approved by:	DJG	Revision No.:	
	1	UPDATED TO NEW CONCEPTUAL PLAN	14/02/2024	KB		Title:  TEST HOLE LOCATION PLAN			
	NO.	REVISIONS	DATE	INITIAL					