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Geotechnical Investigation

Proposed Multi-Storey Building
1054 Hunt Club Road - Phase 2
Ottawa, Ontario

Prepared For

Claridge Homes

March 30, 2020

Report: PG4091-2

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1.0 Introduction

Paterson Group (Paterson) was commissioned by Claridge Homes to conduct a geotechnical investigation for the proposed multi-storey building to be located at 1054 Hunt Club Road, in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2).

The objectives of the current investigation were to:

- ❑ Obtain subsurface soil and groundwater information by means of boreholes completed within the subject site.
- ❑ Provide geotechnical recommendations for 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. This report contains our findings and includes geotechnical recommendations pertaining to the design and construction of the subject development as understood at the time of writing this report.

2.0 Proposed Development

Based on the available drawings, the proposed development at the subject site is understood to consist of a multi-storey building with 1 underground parking level. Further, the underground parking level will extend to the south beyond the overlying building footprint. The proposed building will also be surrounded by asphalt paved access lanes and parking areas with landscaped margins.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current investigation was completed on March 2 and 3, 2020, and consisted of 2 boreholes (BH 1-20 and BH 2-20) drilled to a maximum depth of 9.1 m. A previous geotechnical investigation also included 3 boreholes (BH 3, BH 4 and BH 5) which were completed at the Phase 2 site in February and April 2017. The borehole locations were chosen to provide general coverage of the proposed development. The locations of the boreholes are shown on Drawing PG4091-2 - Test Hole Location Plan included in Appendix 2.

The boreholes were drilled using a track mounted drill rig or a portable drill, each operated by a two person crew. All fieldwork was conducted under the full-time supervision of a senior engineer. The drilling procedure consisted of augering to the required depths at the selected locations and sampling the overburden.

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 visually inspected and classified on site. The auger and split spoon samples were placed in sealed plastic bags and transported to our laboratory for further examination and classification. The depths at which the auger and split spoon samples were recovered from the test holes are shown as, AU and SS, respectively, on the Soil Profile and Test Data sheets presented in Appendix 1.

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

The overburden soil thickness was evaluated during the course of the investigation by dynamic cone penetration testing (DCPT) at BH 3 and BH 2-20. The DCPT consists of driving a steel drill rod, equipped with a 50 mm diameter cone at its tip, using a 63.5 kg hammer falling from a height of 760 mm. The number of blows required to drive the cone into the soil is recorded for each 300 mm depth increment.

The subsurface conditions observed in the boreholes were recorded in detail in the field. The soil profiles are logged on the Soil Profile and Test Data sheets presented in Appendix 1.

Groundwater

PVC groundwater monitoring wells were installed in boreholes BH 3, BH 1-20, and BH 2-20 to permit monitoring of the groundwater levels subsequent to the completion of the sampling program.

Monitoring Well Installation

Typical monitoring well construction details are described below:

- ❏ Slotted 32 mm diameter PVC screen at base of borehole, for 3 m length.
- ❏ 32 mm diameter PVC riser pipe from the top of the screen to the ground surface.
- ❏ No.3 silica sand backfill within annular space around screen.
- ❏ 300 mm thick bentonite hole plug directly above PVC slotted screen.
- ❏ Clean backfill from top of bentonite plug to the ground surface.

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

Sample Storage

All samples are stored in the laboratory for a period of one month after issuance of this report. The samples will then be discarded unless otherwise directed.

3.2 Field Survey

The location and ground surface elevation at each borehole location was surveyed by Paterson personnel. The ground surface elevations at the borehole locations were surveyed with respect to a temporary benchmark (TBM), consisting of the top of spindle of the fire hydrant located in front of 1026 Hunt Club Road. A geodetic elevation of 94.49 m was provided for the TBM by Annis O’Sullivan Vollebekk. The borehole locations and ground surface elevations at the borehole locations along with the TBM location are presented on Drawing PG4091-2 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

All soil samples were recovered from the subject site and visually examined in our laboratory to review the soil investigation results.

3.4 Analytical Testing

One 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 analyzed to determine the concentrations of sulphate and chloride, the resistivity and the pH of the sample. The analytical test results are presented in Appendix 1 and discussed in Subsection 6.7.

4.0 Observations

4.1 Surface Conditions

Currently, the subject site is being utilized as a contractor staging area during the construction of the adjacent Phase 1 development to the west. However, the site was previously occupied by a residential dwelling with a garage in the central portion of the site, and was surrounded by access lanes and gravel parking areas, with mature tree covered areas at the site boundaries. An in-ground swimming pool was also formerly present in southeast corner of the subject site. Reference should be made to the aerial photograph in Figure 2 - Aerial Photograph - 2017 which illustrates the former site conditions.

The subject site is bordered by Hunt Club Road to the north, an Airport Parkway on-ramp to the east, forested land to the south, and the Phase 1 development to the west. The existing ground surface across the majority of the site is relatively level at approximate geodetic elevation 93 to 94 m, but slopes downward moderately at the north end of the site to Hunt Club Road at approximate geodetic elevation 90.5 m.

4.2 Subsurface Profile

Generally, the subsurface soil profile encountered at the borehole locations consists of an approximate 0.5 to 1.4 m thickness of fill, which was observed to consist of a silty sand and gravel with occasional cobbles.

Underlying the fill, interbedded layers of loose to compact, fine silty sand to soft silty clay were generally encountered. A deposit of compact to dense silty sand with gravel, cobbles, and boulders was then encountered at approximate depths of 2.3 to 3.7 m below the existing ground surface. A compact to dense sand deposit was subsequently encountered at approximate depths of 3.6 to 4.6 m below the existing ground surface, extending to the bottom of the boreholes at a maximum depth of 9.1 m.

Practical refusal to the DCPT was encountered at a depth of 15.8 m in BH 3, where as DCPT refusal was not encountered in BH 2-20 at a depth of 16 m.

Specific details of the subsoil profile at each test hole location are presented on the Soil Profile and Test Data sheets in Appendix 1.

Bedrock

Based on available geological mapping, the subject site is located in an area where the bedrock consists of interbedded limestone and shale of the Verulam Formation with an approximate drift thickness of 15 to 25 m.

4.3 Groundwater

Groundwater levels were measured in monitoring well BH 3 on March 9, 2017, and in monitoring wells BH 1-20 and BH 2-20 on March 20, 2020. The observed groundwater levels are summarized in Table 1.

Table 1 - Summary of Groundwater Level Readings

Test Hole Number	Ground Surface Elevation (m)	Groundwater Depth (m)	Groundwater Elevation (m)	Recording Date
BH 3	93.68	3.35	90.33	March 9, 2017
BH 1-20	93.12	Damaged	-	March 20, 2020
BH 2-20	93.71	3.78	89.93	March 20, 2020

Note:
- The ground surface elevations at the borehole locations are referenced to a TBM, consisting of the top spindle of the fire hydrant located in front of 1026 Hunt Club Road with a geodetic elevation of 94.49 m.

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 colour and consistency of the recovered soil samples. Based on these observations, the long-term groundwater table can be expected at approximately 3 to 4 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 multi-storey building. It is recommended that the proposed building be founded on conventional spread footings or a raft foundation bearing directly on the undisturbed compact to dense silty sand with gravel and cobbles, or the undisturbed compact to dense sand deposit..

However, if the provided bearing resistance values for a footing or raft foundation are not sufficient for the design building loads, a deep foundation, such as end bearing piles, may need to be considered.

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

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and deleterious fill, such as those containing organic materials, should be stripped from under the proposed building, paved areas, pipe bedding and other settlement sensitive structures.

Existing foundation walls and other construction debris should be entirely removed from within 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 areas should consist, unless otherwise specified, of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. This material should be tested and approved prior to delivery to the site. The fill should be placed in lifts no greater than 300 mm thick and compacted using suitable compaction equipment for the lift thickness. Fill placed beneath the building and paved 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 at least compacted 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.

5.3 Foundation Design

Conventional Spread Footings

Footings placed over a bearing surface consisting of undisturbed, compact to dense silty sand with gravel and cobbles, or the undisturbed, compact to dense sand deposit, 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 **500 kPa**. A geotechnical resistance factor of 0.5 was applied to the above noted bearing resistance value at ULS. A modulus of subgrade reaction of **50 MPa/m** can be used for design.

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 the dry, prior to the placement of concrete for footings.

Footings designed using the bearing resistance value at SLS provided herein will be subjected to potential post-construction total and differential settlements of 25 and 20 mm, respectively.

Raft Foundation

Based on the expected loads for the proposed building, a raft foundation may be required to support the proposed building. For our design calculations, one level of underground parking was assumed which would extend approximately 3 to 3.5 m below existing ground surface (corresponding to approximate geodetic elevation 90 m). The maximum SLS contact pressure is **350 kPa** for a raft foundation bearing on the undisturbed, silty sand with gravel and cobbles, or the undisturbed, compact to dense sand deposit. It should be noted that the weight of the raft slab and everything above, has to be included when designing with this value. The loading conditions for the contact pressure are based on sustained loads, that are generally taken to be 100% Dead Load and 50% Live Load. The factored bearing resistance (contact pressure) at ULS can be taken as **525 kPa**. A geotechnical resistance factor of 0.5 was applied to the bearing resistance value at ULS.

The modulus of subgrade reaction was calculated to be **50 MPa/m** for a contact pressure of **350 kPa**. The design of the raft foundation is required to consider the relative stiffness of the reinforced concrete slab and the supporting bearing medium. A common method of modeling the soil structure interaction is to consider the bearing medium to be elastic and to assign a subgrade modulus. However, silty sand and sand are not elastic and limits have to be placed on the stress ranges of a particular modulus.

The proposed building can be designed using the above parameters with total and differential settlements of 25 and 20 mm, respectively.

Lateral Support

The bearing medium under footing- or raft-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 compact to dense silty sand or sand bearing medium when a plane extending down and out from the bottom edge of the footing at a minimum of 1.5H:1V (or flatter) passes only through in situ soil or engineered fill.

Pile Foundation

For deep foundations, concrete-filled steel pipe piles are generally utilized in the Ottawa area. Applicable pile resistance at SLS values and factored pile resistance at ULS values for concrete-filled steel pipe piles are given in Table 2. A resistance factor of 0.4 has been incorporated into the factored ULS values. Note that these are all geotechnical axial resistance values.

The geotechnical pile resistance values were estimated using the Hiley dynamic formula, to be confirmed during pile installation with a dynamic monitoring program. For this project, the dynamic monitoring of four (4) piles would be recommended. Re-striking of all piles at least once will also be required after at least 48 hours have elapsed since initial driving.

A working pad is recommended during the piling driving operation. It is expected that a 600 mm thick layer of 100 mm minus or Granular B Type II would provide an acceptable surface.

Table 2 - Pile Foundation Design Data				
Pile Outside Diameter (mm)	**Pile Wall Thickness (mm)**	**Geotechnical Axial Resistance**	**Final Set (blows/ 12 mm)**	**Transferred Hammer Energy (kJ)**
		Factored at ULS (kN)		
245	9	1130	10	29
245	11	1410	10	35
245	13	1650	10	42

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class D** for the foundations considered at this site. The soils underlying the subject site are not susceptible to liquefaction. Refer to the latest revision of the 2012 Ontario Building Code for a full discussion of the earthquake design requirements.

5.5 Basement Slab

If a raft slab is considered, a granular layer of OPSS Granular A will be required to allow for the installation of sub-floor services above the raft slab foundation. The thickness of the granular layer will be dependent on the piping requirements.

For a building founded on footings or piles, it is recommended that the upper 200 mm of subfloor fill consists of 19 mm clear crushed stone. All backfill material within the footprint of the proposed building should be placed in maximum 300 mm thick loose layers and compacted to at least 98% of its SPMDD.

An underslab drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided under the lowest level floor slab. The spacing of the underslab drainage pipes can be determined at the time of construction to confirm groundwater infiltration levels, 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 soil consists of a material with an angle of internal friction of 30 degrees and a bulk (drained) unit weight of 20 kN/m^3.

Where undrained conditions are anticipated (i.e. below the groundwater level), the applicable effective (undrained) unit weight of the retained soil can be taken as 13 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
γ = unit weight of the applicable retained soil (kN/m^3)
H = height of the wall (m)

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

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

Seismic Earth Pressures

The total seismic force (P_{AE}) includes both the earth force component (P_o) and the seismic component (ΔP_{AE}). The seismic earth force (ΔP_{AE}) can be calculated using $0.375 \cdot a_c \cdot \gamma \cdot H^2/g$ where:

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

The peak ground acceleration, (a_{max}), for the Ottawa area is 0.32g 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_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

For design purposes, the pavement structures presented in the following tables are recommended to be used for the design of car only parking areas and access lanes.

Table 3 - Recommended Pavement Structure - Car Only Parking Areas

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

Table 4 - Recommended Pavement Structure
Access Lanes and Heavy Truck Parking Areas

Thickness (mm)	Material Description
40	**Wear Course -** HL-3 or Superpave 12.5 Asphaltic Concrete
50	**Binder Course -** HL-8 or Superpave 19.0 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
450	**SUBBASE** - OPSS Granular B Type II
	SUBGRADE - 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 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 corrugated plastic pipe, surrounded on all sides by 150 mm of 19 mm clear crushed stone, placed at the footing level around the exterior perimeter of the structure with the exception of the south side of the building. The pipe should have a positive outlet, such as a gravity connection to the storm sewer.

In order to avoid having the perimeter foundation drain potentially lower the groundwater level in the vicinity of the wetland along the south building foundation wall, it is recommended that a waterproofing membrane be placed over the lower 1.5 m of the foundation walls located adjacent to the wetland area. We also recommend to raise the perimeter drainage pipe to elevation 90.70 m, consistent with the high groundwater level, for the foundation walls located adjacent to the wetland. Above the waterproofing membrane, the foundation walls will be dampproofed and covered with a drainage board such as Delta Drain 6000 or approved equivalent.

Underslab Drainage

For design purposes, it is recommended that 150 mm diameter perforated pipes be placed at approximate 4 to 6 m centres below the lowest level floor. 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 should consist of free-draining non frost susceptible granular materials. The greater part of the site excavated materials will be frost susceptible and, as such, are not recommended for re-use as backfill against the foundation walls, unless used in conjunction with a drainage geocomposite, such as Delta Drain 6000, connected to the perimeter foundation drainage system. 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 of 1.5 m of soil cover alone, or a minimum of 0.6 m of soil cover in conjunction with foundation insulation, should be provided.

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

Unheated structures, such as the access ramp for the underground parking level, may require insulation for protection against the deleterious effects of frost action. Additional information can be provided upon request.

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.

Unsupported Excavations

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be cut back at 1H:1V or flatter. The flatter slope is required for excavation below groundwater level. The subsoil at this site is considered to be mainly 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.

It is recommended that a trench box be used at all times 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

Temporary shoring may be required to support the overburden soils. 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. Furthermore, the design of the temporary shoring system should take into consideration a full hydrostatic condition which can occur during significant precipitation events.

The temporary shoring system may consist of a soldier pile and lagging system or steel sheet piles 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 pressures acting on the shoring system may be calculated using the following parameters.

Table 5 - 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 (γ), kN/m^3	21
Submerged Unit Weight (γ), 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 level.

The hydrostatic groundwater pressure should be included to the earth pressure distribution wherever the effective unit weight are 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

The pipe bedding for sewer and water pipes should consist of at least 150 mm of OPSS Granular A material. The material should be placed in maximum 300 mm thick lifts and compacted to a minimum of 95% of the SPMDD. The bedding material should extend at least to the spring line of the pipe.

The cover material, which should consist of OPSS Granular A, should extend from the spring line of the pipe to at least 300 mm above the obvert of the pipe. The material should be placed in maximum 300 mm thick lifts and compacted to a minimum of 95% of the SPMDD.

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

6.5 Groundwater Control

It is anticipated that groundwater infiltration into the excavations should be moderate and controllable using open sumps. 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.

A temporary Ministry of the Environment, Conservation and Parks (MECP) permit to take water (PTTW) may be required for this project if more than 400,000 L/day of ground and/or surface water is to be pumped during the construction phase. A minimum 4 to 5 months should be allowed for completion of the PTTW application package 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. If a project qualifies for a PTTW based upon anticipated conditions, an EASR will not be allowed as a temporary dewatering measure while awaiting the MECP review of the PTTW application.

Impacts on Neighbouring Properties

Based on the proximity of neighbouring buildings and the subsurface conditions encountered at the subject site, the proposed development will not negatively impact the neighbouring structures. It should be noted that no issues are expected with respect to groundwater lowering that would cause long term adverse effects to adjacent structures surrounding the proposed building.

6.6 Winter Construction

Precautions should be considered if construction occurs during the winter. The subsurface soil conditions consist of frost susceptible materials. In the presence of water and freezing conditions, ice could form within the soil mass. Heaving and settlement upon thawing could occur.

In the event of construction during below zero temperatures, the founding stratum should be protected from freezing temperatures by the use of straw, propane heaters and tarpaulins or other suitable means. The base of the excavations should be insulated from sub-zero temperatures immediately upon exposure and until 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 difficult activities to complete during winter without introducing frost in the excavation subgrade base or walls. Precautions should be considered if such activities are to be completed during sub-zero temperatures.

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 non aggressive to slightly aggressive corrosive environment.

7.0 Recommendations

The following material testing and observation program should be performed by a geotechnical consultant and is required for the foundation design data provided herein to be applicable:

- ❏ Observation of all bearing surfaces prior to the placement of concrete.
- ❏ 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, and follow-up 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.

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 Claridge 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.

Scott S. Dennis, P.Eng.

David J. Gilbert, P.Eng.

Report Distribution

❏ Claridge Homes (e-mail copy)
❏ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Buildings
1026-1054 Hunt Club Rd., Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located on the south side of Hunt Club Road, in front of subject site. Geodetic elevation = 94.49m.

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE 2020 March 2

FILE NO. PG4091

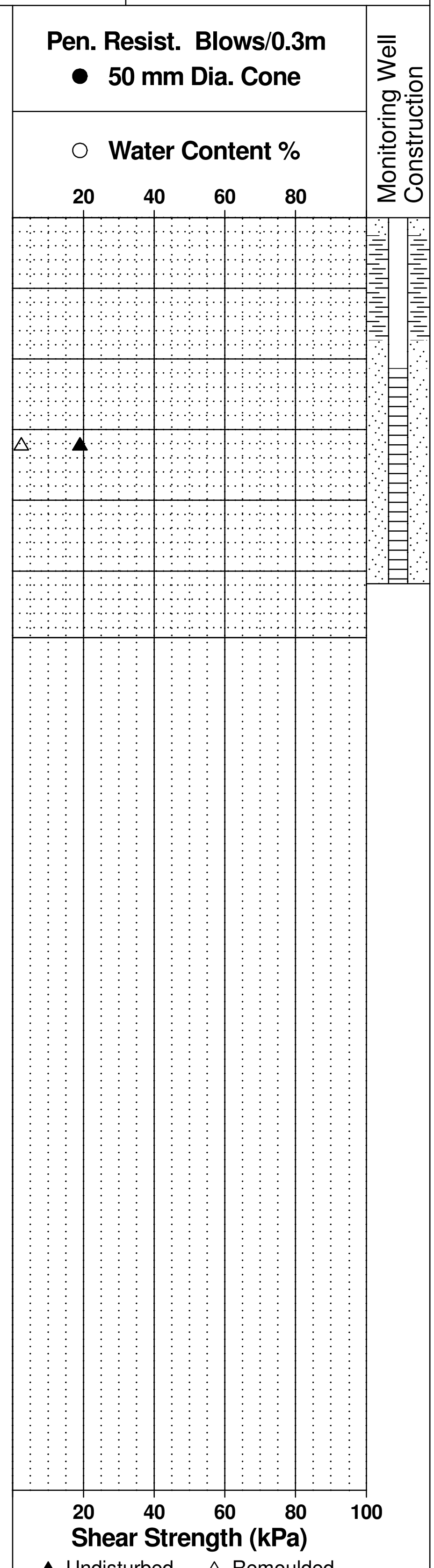
HOLE NO. BH 1-20

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	93.12
FILL: Brown silty sand and gravel, trace cobbles 0.76		AU	1				
Brown **SILTY CLAY,** trace sand 1.52		SS	2	67	33	1	92.12
Loose, brown **SAND,** some silt 2.29		SS	3	100	9	2	91.12
Soft, grey **SILTY CLAY** 3.66		SS	4	100	W	3	90.12
Very dense, grey **SILTY SAND** with gravel 4.57		SS	5		50+	4	89.12
Very dense, grey coarse to medium **SAND,** trace silt		SS	6		54	5	88.12
5.94		SS	7		60		

End of Borehole

Running sand encountered at 5.94m depth.

(GW monitoring well damaged - March 20, 2020)



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Buildings
1026-1054 Hunt Club Rd., Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located on the south side of Hunt Club Road, in front of subject site. Geodetic elevation = 94.49m.

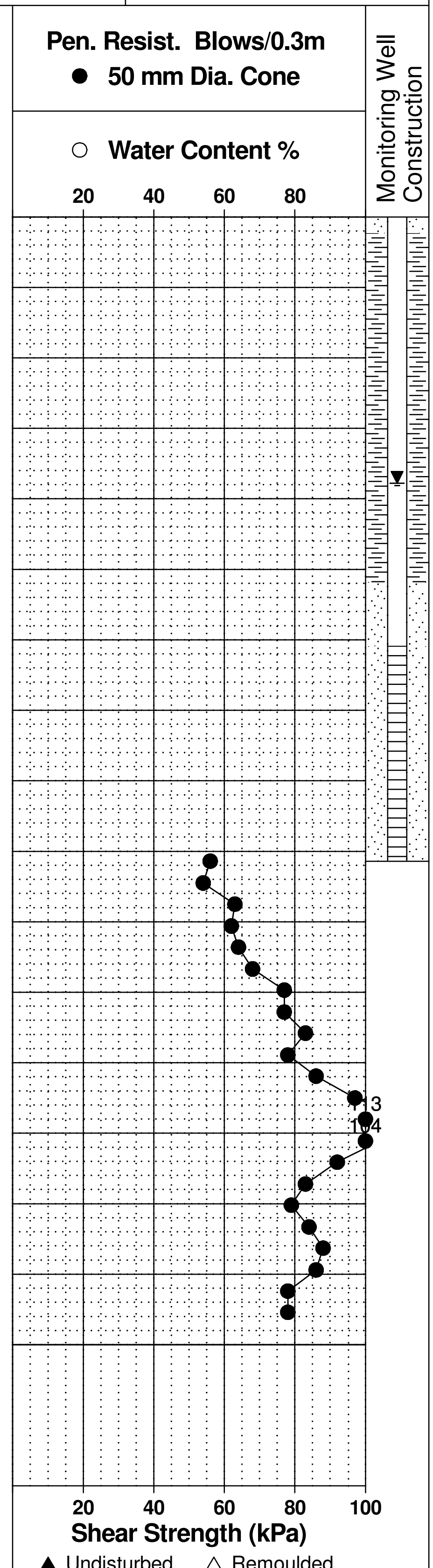
REMARKS

BORINGS BY CME-55 Low Clearance Drill **DATE** 2020 March 3

FILE NO. PG4091

HOLE NO. BH 2-20

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	93.71
FILL: Brown silty sand and gravel, trace clay	AU	1				
	SS	2	92	6	1	92.71
1.37						
Grey **SILT**						
1.78	SS	3	100	50+	2	91.71
Very dense to compact, grey **SILTY SAND and GRAVEL** with cobbles and boulders, trace clay	SS	4	25	21	3	90.71
	SS	5	0	23		
	SS	6	0	50+	4	89.71
4.27						
Compact to dense, grey **SAND,** trace silt	SS	7	58	27	5	88.71
	SS	8	83	24	6	87.71
- running sand encountered at 5.8m depth. Switch from hollow stem augers to casing with wet rotary drilling.	SS	9	50	5	7	86.71
	SS	10	50	35	8	85.71
	SS	11	92	54		
	SS	12	58	27	9	84.71
9.14						
Dynamic Cone Penetration Test commenced at 9.14m depth.					10	83.71
					11	82.71
					12	81.71
					13	80.71
					14	79.71
					15	78.71
16.00					16	77.71
End of Borehole						
(GWL @ 3.78m - March 20, 2020)						



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Buildings
1026-1054 Hunt Club Rd., Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located on the south side of Hunt Club Road, in front of subject site. Geodetic elevation = 94.49m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE February 27, 2017

FILE NO. PG4091

HOLE NO. BH 3

SOIL DESCRIPTION	Depth of strata change (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
FILL: Crushed stone with silt and sand	0.46	AU	1		
Compact, brown **SILTY FINE SAND,** some cobbles	1.83	SS	2	96	14
Grey **CLAYEY SILT,** trace sand, gravel and cobbles	2.29	SS	3	50	21
Compact to dense, grey **SILTY SAND** with gravel, some clay and cobbles	3.56	SS	4	62	21
		SS	5	62	33
Dense to compact, grey **SAND,** some silt, trace gravel	9.14	SS	6	50	11
		SS	7	54	32
		SS	8	100	17
Dynamic Cone Penetration Test (DCPT) commenced at 9.14m depth.	15.83				
End of Borehole					

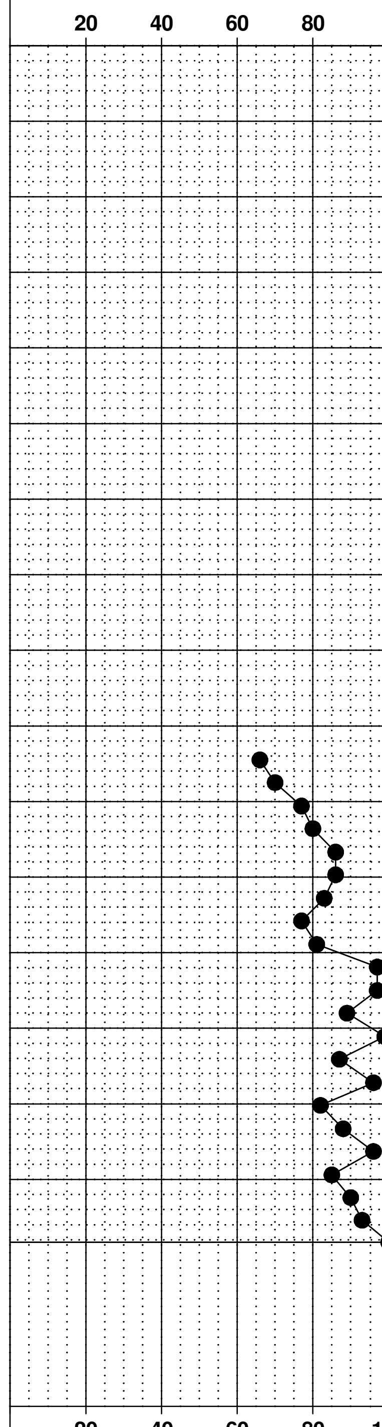
Practical DCPT refusal at 15.83m depth

(GWL @ 3.35m-March 9, 2017)

DEPTH (m)	ELEV. (m)
0	93.68
1	92.68
2	91.68
3	90.68
4	89.68
5	88.68
6	87.68
7	86.68
8	85.68
9	84.68
10	83.68
11	82.68
12	81.68
13	80.68
14	79.68
15	78.68

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

○ **Water Content %**



Shear Strength (kPa)
▲ Undisturbed △ Remoulded

Monitoring Well Construction

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Buildings
1026-1054 Hunt Club Rd., Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located on the south side of Hunt Club Road, in front of subject site. Geodetic elevation = 94.49m.

REMARKS

BORINGS BY Portable Drill

DATE April 12, 2017

FILE NO. PG4091

HOLE NO. BH 4

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content % (20 40 60 80)	Monitoring Well Construction
GROUND SURFACE						0	93.51		
Mud slab 0.10									
FILL: Crushed stone with sand and gravel 0.46		AU	1						
Brown **SILTY SAND** to **SANDY SILT,** some clay		SS	2	92		1	92.51		
1.83		SS	3	83					
Brown **CLAYEY SAND,** some silt		SS	4	83		2	91.51		
2.90		SS	5	100					
Grey **SILTY CLAY,** some sand 3.51		SS	6	100		3	90.51		
Grey **SILTY SAND** with gravel and cobbles		SS	7	100		4	89.51		
4.47									
End of Borehole									
(GWL at 3.0m depth based on field observations)									

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

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Buildings
1026-1054 Hunt Club Rd., Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located on the south side of Hunt Club Road, in front of subject site. Geodetic elevation = 94.49m.

REMARKS

BORINGS BY Portable Drill

DATE April 10, 2017

FILE NO. PG4091

HOLE NO. BH 5

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	93.64
TOPSOIL (to 0.30)		SS	1	25			
Brown **CLAYEY SILT,** some sand (0.30 – 1.22)		SS	2	67		1	92.64
GLACIAL TILL: Brown silty san with gravel, cobbles and boulders (1.22 – 5.49)		RC	1				
		SS	3	67			
		RC	2			2	91.64
		RC	3				
		SS	4	83		3	90.64
		SS	5	100		4	89.64
		RC	4				
		SS	6	100		5	88.64
End of Borehole (5.49)							

(GWL at 3.25m depth based on field observations)

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

○ Water Content %

20 40 60 80

Monitoring Well Construction

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

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.

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
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 = $(D30)^2 / (D10 \times D60)$
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

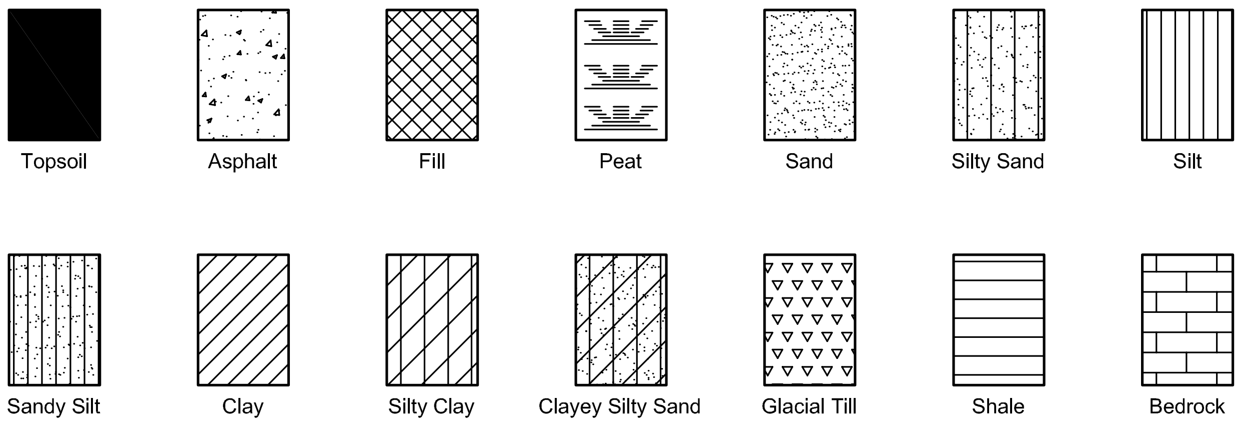
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void 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.

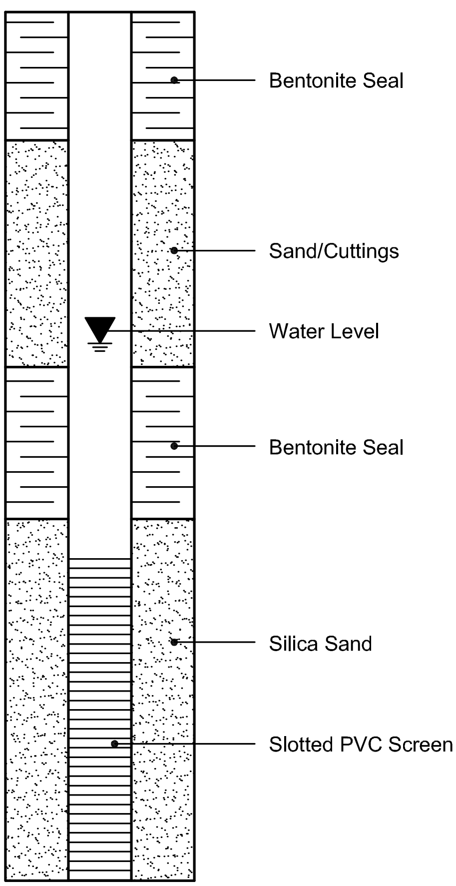
SYMBOLS AND TERMS (continued)

STRATA PLOT

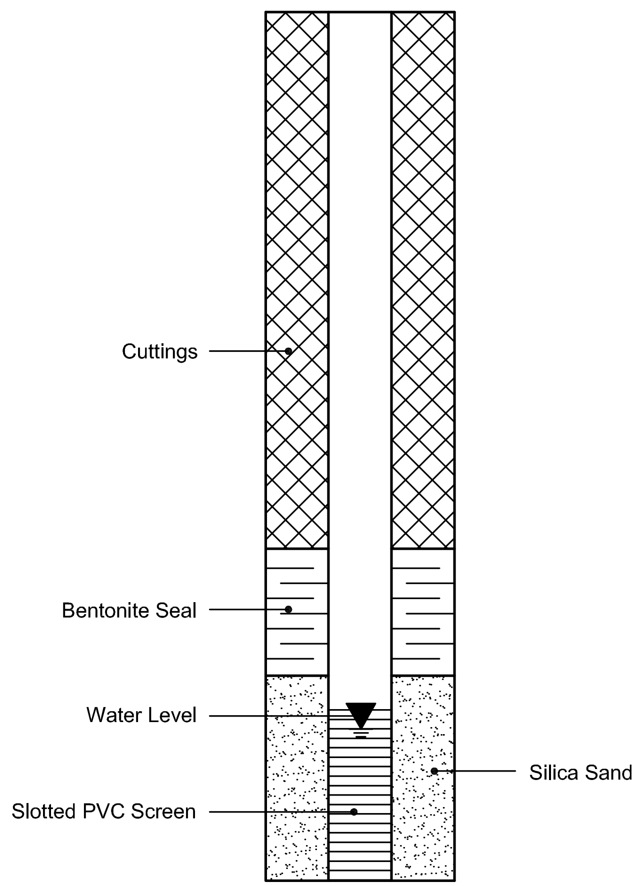


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 29306

Report Date: 10-Mar-2020

Order Date: 4-Mar-2020

Project Description: PG4091

	Client ID:	BH1-20 SS4 7'6"-9'6"	-	-	-
	Sample Date:	03-Mar-20 09:00	-	-	-
	Sample ID:	2010315-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	61.5	-	-	-
General Inorganics					
pH	0.05 pH Units	7.62	-	-	-
Resistivity	0.10 Ohm.m	22.9	-	-	-
Anions					
Chloride	5 ug/g dry	52	-	-	-
Sulphate	5 ug/g dry	164	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURE 2- AERIAL PHOTOGRAPH - 2017

DRAWING PG4091-2 - TEST HOLE LOCATION PLAN

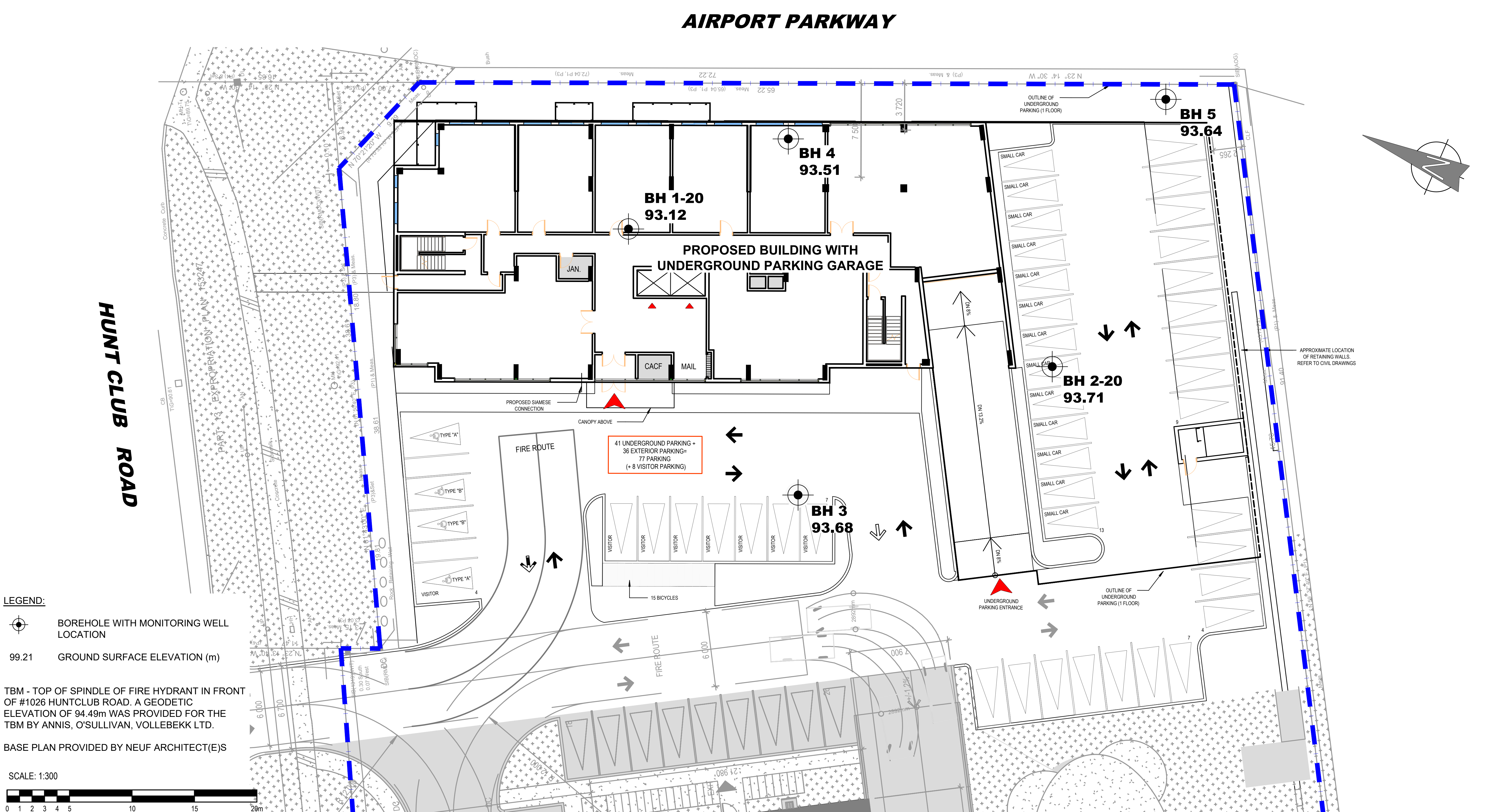
FIGURE 1

KEY PLAN

patersongroup

FIGURE 2

Aerial Photograph - 2017



LEGEND:

BOREHOLE WITH MONITORING WELL LOCATION

99.21 GROUND SURFACE ELEVATION (m)

TBM - TOP OF SPINDLE OF FIRE HYDRANT IN FRONT OF #1026 HUNTCLUB ROAD. A GEODETIC ELEVATION OF 94.49m WAS PROVIDED FOR THE TBM BY ANNIS, O'SULLIVAN, VOLLEBEKK LTD.

BASE PLAN PROVIDED BY NEUF ARCHITECT(E)S

SCALE: 1:300

0 1 2 3 4 5 10 15 20m

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consulting engineers

154 Colonnade Road South
Ottawa, Ontario K2E 7J5
Tel: (613) 226-7381 Fax: (613) 226-6344

NO.	REVISIONS	DATE	INITIAL
0			

CLARIDGE HOMES

GEOTECHNICAL INVESTIGATION
PROP. MULTI-STOREY BUILDINGS - 1026-1054 HUNT CLUB ROAD

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:500	Date: 03/2017
Drawn by: RCG	Report No.: PG4091-1
Checked by: NZ	Dwg. No.: PG4091-2
Approved by: DJG	Revision No.: 0

p:\autocad drawings\geotechnical\pg40xx\pg4091\pg4091-1(eastern portion).dwg