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Geotechnical Engineering

Environmental Engineering

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Materials Testing

Building Science

Archaeological Services

Geotechnical Investigation

Proposed Retirement Building
20 Chesapeake Crescent
Ottawa, Ontario

Prepared For

Claridge Homes

August 1, 2018

Report: PG4557-1

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1.0 Introduction

Paterson Group (Paterson) was commissioned by Claridge Homes to conduct a geotechnical investigation for a proposed retirement building to be located at 20 Chesapeake Crescent in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objectives of the current investigation were to:

❑ determine the subsoil and groundwater conditions at this site by means of boreholes.

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

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

2.0 Proposed Project

The proposed development is understood to consist of a 6 storey retirement building with a basement level. Access lanes and landscaped areas are anticipated for this development. The proposed building is also anticipated to be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the geotechnical investigation was carried out on July 20, 2018. A total of 6 boreholes were drilled and sampled to a maximum depth of 7.2 m below existing ground surface. The test hole locations were distributed in a manner to provide general coverage of the subject site taking into consideration site features. The locations of the test holes are shown on Drawing PG4557-1 - Test Hole Location Plan included in Appendix 2.

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

Sampling and In Situ Testing

Soil samples were recovered using a 50 mm diameter split-spoon sampler or from the auger flights. The split-spoon and auger samples were classified on site and placed in sealed plastic bags. All samples were transported to our laboratory. The depths at which the split-spoon and auger samples were recovered from the boreholes are shown as SS and AU, respectively, on the Soil Profile and Test Data sheets 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.

Undrained shear strength tests were completed in cohesive soils with a shear vane apparatus.

The overburden thickness was also evaluated during the previous investigations by completing dynamic cone penetration tests (DCPTs) at boreholes BH 3 and BH 6. 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 increment.

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

Groundwater

Flexible polyethylene standpipes were installed in all borehole. The groundwater observations are discussed in Subsection 4.3 and presented in the Soil Profile and Test Data sheets in Appendix 1.

Sample Storage

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

3.2 Field Survey

The borehole locations were selected in the field by Paterson personnel in a manner to provide general coverage of the proposed development taking into consideration existing site features, such as underground utility services. The borehole locations and ground surface elevations at the borehole locations were surveyed by Annis O'Sullivan Vollebekk Ltd and are referenced to a geodetic datum. The locations and ground surface elevations of the boreholes are presented on Drawing PG4557-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

The soil samples recovered from the subject site were examined in our laboratory to review the results of the field logging. Moisture content testing was completed on recovered soil samples. Additionally, atterburg limit testing was performed on two (2) samples. The results are presented in the Soil Profile and Test Data sheets presented in Appendix 1.

3.4 Analytical Testing

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

4.0 Observations

4.1 Surface Conditions

Generally, the ground surface across the site is relatively flat with a slight downslope towards the west. The ground surface was also noted to be at grade with the surrounding roadways.

The subject site was observed to be undeveloped at the time of the field program. It should be noted that the ground surface was entirely stripped from all the topsoil and construction equipment/materials were placed along the perimeter of the site.

4.2 Subsurface Profile

Overburden

Generally, the soil profile encountered at the borehole locations consists of fill consisting of silty sand with gravel and cobbles overlying a stiff to very stiff brown silty clay layer. A firm to stiff grey silty clay layer was encountered below the above noted layers followed by glacial till consisting of grey silty clay mixed with sand, gravel, cobbles and boulders. Practical refusal to the DCPTs were encountered at depths ranging from 7.2 m in BH 2 and 10.9 m in BH 6. Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for specific details of the soil profiles encountered at each test hole location.

The results of Atterberg Limits tests conducted within the silty clay are presented below in Table 1 - Summary of Attergerg Limits' Results and on the Atterberg Limits' Results sheet (plasticity chart) in Appendix 1. The tested silty clay samples had measured liquid limits of 70 to 75% and plasticity indices ranging from 49 to 55, which classifies them as inorganic clays of high plasticity (CH) in accordance with the Unified Soil Classification System.

Table 1 - Summary of Atterberg Limits' Results

Sample	Liquid Limit %	Plastic Limit %	Plasticity Index %	Classification
BH 2 - SS 5	70	22	49	CH
BH 6 - SS 4	75	20	55	CH

Bedrock

Based on available geological mapping, the subject site the bedrock consists of either interbedded sandstone and dolomite of the March formation or interbedded limestone and dolomite of the Gull River formationwith an overburden thickness ranging between 5 to 10 m.

4.3 Groundwater

The groundwater levels were measured in the borehole locations on July 24, 2018, and are presented in the Soil Profile and Test Data sheets in Appendix 1. It is important to note that groundwater level readings could be influenced by surface water infiltrating the backfilled borehole, which can lead to higher than typical groundwater levels. The long-term groundwater level can also be estimated based on moisture levels and colouring of the recovered soil samples. Based on these observations at the borehole locations, the long-term groundwater level is expected at a 3 to 4 m depth.

Table 2 - Summary of Groundwater Level Readings

Test Hole Number	Ground Elevation (m)	Groundwater Levels (m)		Recording Date
		Depth	Elevation	
BH 1	96.21	2.52	93.69	July 24, 2018
BH 2	96.04	2.21	93.83	July 24, 2018
BH 3	95.91	2.12	93.79	July 24, 2018
BH 4	95.90	1.72	94.18	July 24, 2018
BH 5	95.80	1.65	94.15	July 24, 2018
BH 6	95.70	2.76	92.94	July 24, 2018

Note:
-The ground surface elevations at the borehole locations were provided by Annis O'Sullivan Vollebekk Ltd.

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

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is satisfactory for the proposed development. It is anticipated that the proposed buildings will be founded over a raft foundation placed over stiff to firm silty clay or compact glacial till bearing surface. Alternatively, the proposed building may be founded over end bearing piles.

Due to the presence of the sensitive silty clay layer, the proposed development will be subjected to grade raise restrictions. The permissible grade raise recommendations are further discussed in Subsection 5.3.

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

5.2 Site Grading and Preparation

Stripping Depth

Topsoil, asphalt, and fill, containing deleterious or organic materials, should be stripped from under any building, paved areas, pipe bedding and other settlement sensitive structures. Under paved areas, existing construction remnants, such as foundation walls, pipe ducts, etc., should be excavated to a minimum depth of 1 m below final grade.

Fill Placement

Fill used for grading beneath the proposed buildings, unless otherwise specified, should consist of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II, with a maximum aggregate size of 50 mm. The fill should be tested and approved prior to delivery to the site. It 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 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

As noted above, it is expected that the proposed multi-storey buildings will be founded on raft foundations or end-bearing piles. Detailed recommendations are provided in the following subsections.

Spread Footing Foundations

For exterior structures, conventional shallow foundations are recommended. Strip footings, up to 4 m wide, and pad footings, up to 5 m wide, placed on an undisturbed, stiff 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 **125 kPa** for footings. A geotechnical resistance factor of 0.5 was applied to the above noted bearing resistance value at ULS.

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

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.

The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels. Adequate lateral support is provided to the above noted bearing medium when a plane extending down and out from the bottom edge of the footing at a minimum of 1.5H:1V, passes only through in situ soil or engineered fill of the same or higher capacity as the soil.

Raft Foundation

Should the building loads exceed the bearing resistance values provided for a conventional shallow footing foundation, the following parameters may be used for the design of a raft foundation.

For design purposes, it was assumed that the base of the raft foundation for the proposed multi-storey buildings with one level of underground parking will be located at a 3.5 to 4 m depth.

The amount of settlement of the raft slab will be dependent on the sustained raft contact pressure. The bearing resistance value at SLS (contact pressure) of **100 kPa** will be considered acceptable. 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 contact pressure provided considers the stress relief associated with the soil removal required for the proposed building. The factored bearing resistance (contact pressure) at ULS can be taken as **150 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 **3.5 MPa/m** for a contact pressure of **100 kPa**. The raft foundation design is required to consider the relative stiffness of the reinforced concrete slab and the supporting bearing medium.

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

Deep Foundation

For support of the proposed multi-storey building consideration should be given to using concrete filled steel pipe piles driven to refusal on the bedrock surface.

For deep foundations, concrete-filled steel pipe piles are commonly utilized in the Ottawa area. Applicable pile resistance at SLS values and factored pile resistance at ULS values 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. The piles load bearing capacity should be confirmed during pile installation with a program of dynamic monitoring. The dynamic monitoring of two to four piles is recommended. The pipe piles should be equipped with a minimum 20 mm thick base plate minimize damage to the pile tip during installation. Re-striking of all piles at least once will be required after at a minimum of 48 hours have elapsed since initial installation.

Table 3 - Pile Foundation Design Data

Pile Outside Diameter (mm)	Pile Wall Thickness (mm)	Geotechnical Axial Resistance		Final Set (blows/12 mm)	Transferred Hammer Energy (kJ)
		SLS (kN)	Factored at ULS (kN)		
245	9	910	1090	10	28.5
245	11	1050	1260	10	34.2
245	13	1250	1500	10	40.7
193	9.5	730	880	8	24
193	14.5	980	1180	9	34

Permissible Grade Raise Restrictions

Due to the presence of a silty clay deposit below the proposed footing level, a permissible grade raise restriction of **1 m** is recommended for grading.

5.4 Design for Earthquakes

Depending on the proposed founding depth, the site class for seismic site response can be taken as **Class C** for the foundations considered at this site. 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

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. The thickness of the OPSS Granular A will be dependent on the piping requirements.

For structures founded on conventional spread footings, it is recommended that the upper 200 mm of sub-slab fill consists of 19 mm clear crushed stone. All backfill material within the footprint of the proposed building should be placed in maximum 300 mm thick loose layers and compacted to at least 95% of its SPMDD.

A subfloor drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided in the OPSS Granular A backfill under the lower basement floor. The spacing of the sub-floor drainage pipes can be determined at the time of construction to confirm groundwater infiltration levels, if any.

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, in our opinion, the conditions can be well-represented by assuming the retained soil consists of a material with an angle of internal friction of 30 degrees and a drained unit weight of 20 kN/m^3.

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

Static 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 fill of the applicable retained soil (kN/m^3)
H = height of the wall (m)

Seismic Earth Pressures

The seismic earth pressure (ΔP_{AE}) can be calculated using the earth pressure distribution equal to $0.375 \cdot a_c \cdot \gamma \cdot H^2/g$ where:

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

The peak ground acceleration, (a_{max}), for the Ottawa area is 0.32 g according to OBC 2012. Note that the vertical seismic coefficient is assumed to be zero.

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

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

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

5.7 Pavement Structure

The pavement structures presented in the following tables could be used for the design of car only parking, heavy truck parking areas and access lanes.

Table 4 - 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 - Either in situ silty clay, approved existing granular fill or OPSS Granular B Type I or II material placed over in situ soil	

Table 5 - Recommended Pavement Structure
Heavy Truck Parking and Access Lanes

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
400	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Either in situ silty clay, approved existing granular fill or OPSS Granular B Type I or II material placed over in situ soil.	

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for this project.

If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type I or II material.

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

Due to the impervious nature of the subgrade materials consideration should be given to installing subdrains during the pavement construction. These drains should extend in four orthogonal directions or longitudinally when placed along a curb. The clear crushed stone surrounding the drainage lines or the pipe, should be wrapped with suitable filter cloth. The subdrain inverts should be approximately 300 mm below subgrade level. The subgrade surface should be shaped to promote water flow to the drainage lines.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Perimeter Drainage System

A perimeter foundation drainage system is recommended for the below-grade level. The system should consist of a 150 mm diameter, geotextile-wrapped, perforated, corrugated, plastic pipe, surrounded on all sides by 150 mm of 10 mm clear crushed stone, placed at the footing level around the exterior perimeter of the structure. The pipe should have a positive outlet, such as a gravity connection to the storm sewer.

Subfloor Drainage

It is anticipated that subfloor drainage will be required to control water infiltration below the basement floor slab. The spacing of the subfloor drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed. It is typically recommended that a 150 mm diameter geotextile-wrapped, perforated, corrugated, plastic pipe, surrounded on all sides by 150 mm of 10 mm clear crushed stone should be placed within each bay. The drainage pipe should direct water to sump pit(s) within the lower basement area.

Foundation Backfill

Where space is available, 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, as recommended above, 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 effect of frost action. A minimum of 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, or an equivalent combination of soil cover and foundation insulation.

6.3 Excavation Side Slopes

The side slopes of the excavations should either be excavated at acceptable slopes or should be retained by shoring systems from the beginning of the excavation until the structure is backfilled. It is anticipated that there will be sufficient room to complete the excavation with acceptable slopes.

Unsupported Side Slopes

Unsupported 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 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

The design and approval of the temporary 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 system 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 could consist of steel sheet piles or a soldier pile and lagging system. Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be included to the earth pressures described below. This system could be cantilevered, anchored or braced. The shoring system is also recommended to be adequately supported to resist toe failure.

The earth pressures acting on the temporary shoring system may be calculated with 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
Dry Unit Weight (γ), kN/m^3	20
Effective 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 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, a 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. Where the bedding is located within the firm grey silty clay, the thickness of the bedding material should be increased to a minimum of 300 mm. The material should be placed in maximum 300 mm thick lifts and compacted to a minimum of 95% of its SPMDD. The bedding material should extent 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 its SPMDD.

It should generally 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 materials will be difficult to re-use, as the high water contents make compacting impractical without an extensive drying period.

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

To reduce long-term lowering of the groundwater level at this site, clay seals should be provided in the service trenches. 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

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.

Due to the relatively impervious nature of the silty clay materials, it is anticipated that groundwater infiltration into the excavations should be low and controllable using open sumps. Pumping from open sumps should be sufficient to control the groundwater influx through the sides of shallow excavations.

A temporary Ministry of the Environment and Climate Change (MOECC) 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 MOECC.

For typical ground or surface water volumes, being pumped during the construction phase, 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 MOECC review of the PTTW application.

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 10,000 L/day) with peak periods noted after rain events.

Impacts on Neighbouring Structures

It is understood that one below-grade level is planned for the proposed multi-storey building. Based on the existing groundwater level and low permeability of the adjacent soils, the extent of any significant groundwater lowering will take place within a limited range of the proposed building. Based on the proximity of neighbouring buildings and minimal zone impacted by the groundwater lowering, 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 damage to adjacent structures surrounding the proposed building.

6.6 Winter Construction

Precautions must be taken if winter construction is considered for this project. The subsoil conditions at this site consist of frost susceptible materials. In the presence of water and freezing conditions, ice could form within the soil mass. Heaving and settlement upon thawing could occur.

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

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

6.7 Corrosion Potential and Sulphate

The results of analytical testing show that the sulphate content is less than 0.1%. These results are indicative that Type 10 Portland cement (normal cement) would be appropriate for this site. The results of the chloride content, pH and resistivity indicate the presence of a moderate to aggressive environment for exposed ferrous metals at this site.

6.8 Landscaping Considerations

Tree Planting Restrictions

The proposed development is located in an area of medium sensitive silty clay deposits for tree planting. Based on the results of the atterberg tests completed under Subsection 4.2, the plasticity index was found to be between 49 and 55%. Therefore, it is recommended that trees placed within 7.5 m of the foundation wall consist of low water demanding trees with shallow roots systems that extend less than 1.5 m below ground surface. Trees placed greater than 7.5 m from the foundation wall may consist of typical street trees, which are typically moderate water demand species with roots extending to a maximum 2 m depth.

It is well documented in the literature, and is our experience, that fast-growing trees located near buildings founded on cohesive soils that shrink on drying can result in long-term differential settlements of the structures. Tree varieties that have the most pronounced effect on foundations are seen to consist of poplars, willows and some maples (i.e. Manitoba Maples) and, as such, they should not be considered in the landscaping design.

7.0 Recommendations

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

- ☐ A review of the grading plan(s) from a geotechnical perspective.
- ☐ Observation of all bearing surfaces prior to the placement of concrete.
- ☐ Sampling and testing of the concrete and granular 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 that these works have been conducted in general accordance with our recommendations could be issued, upon request, following the completion of a satisfactory materials testing and observation program by the geotechnical consultant.

8.0 Statement of Limitations

The recommendations provided in this report are in accordance with our present understanding of the project. We request permission to review our recommendations when the drawings and specifications are completed.

A geotechnical investigation is a limited sampling of a site. Should any conditions at the site be encountered which differ from those at the test hole locations, we request 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.

LICENSED PROFESSIONAL ENGINEER
Aug. 1, 2018
F. I. ABOU-SEIDO
100156744
PROVINCE OF ONTARIO

Faisal I. Abou-Seido, P.Eng.

David J. Gilbert, P.Eng.

Report Distribution

- ☐ Claridge Homes (3 copies)
- ☐ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ATTERBERG LIMITS' TESTING RESULTS

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Retirement Residence - 20 Chesapeake Cres.
Ottawa, Ontario

DATUM Geodetic - as provided by Annis O'Sullivan Vollebekk Ltd

REMARKS

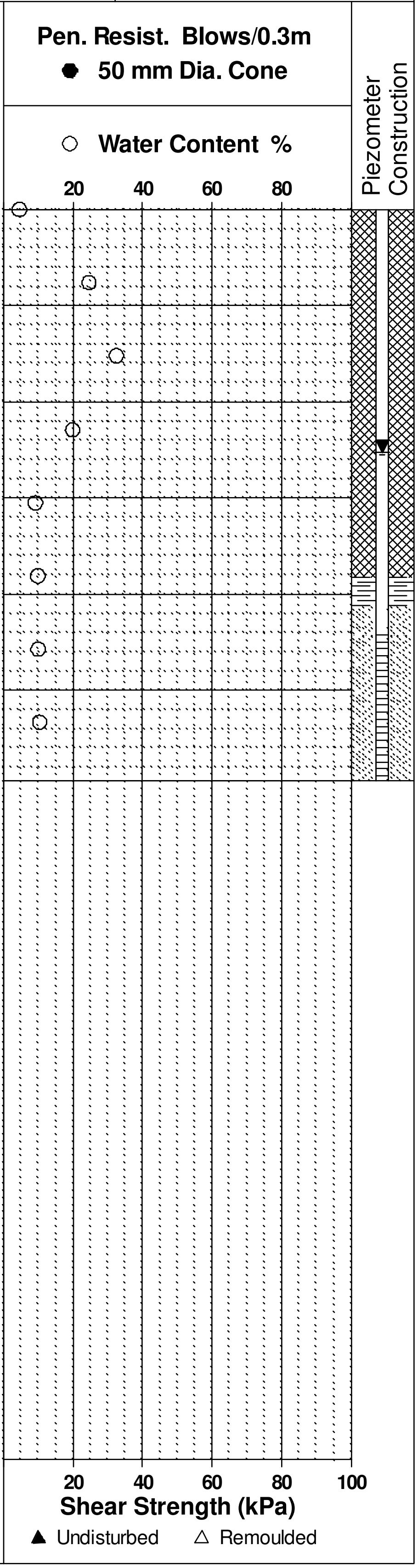
BORINGS BY CME 55 Power Auger

DATE 20 July 2018

FILE NO. PG4557

HOLE NO. BH 1

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	96.21
FILL: Brown silty sand, some gravel, trace cobble and clay	AU	1				
1.07	SS	2	54	18	1	95.21
Very stiff to stiff, brown **SILTY CLAY**	SS	3	71	9	2	94.21
2.29	SS	4	17	12		
GLACIAL TILL: Brown silty clay, some sand, trace gravel, cobbles and boulders.	SS	5	79	7	3	93.21
- grey by 4.6m depth	SS	6	67	6	4	92.21
	SS	7	71	4	5	91.21
5.94	SS	8	100	10		
End of Borehole						
(GWL @ 2.52m - July 24, 2018)						



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Retirement Residence - 20 Chesapeake Cres.
Ottawa, Ontario

DATUM Geodetic - as provided by Annis O'Sullivan Vollebekk Ltd

REMARKS

BORINGS BY CME 55 Power Auger

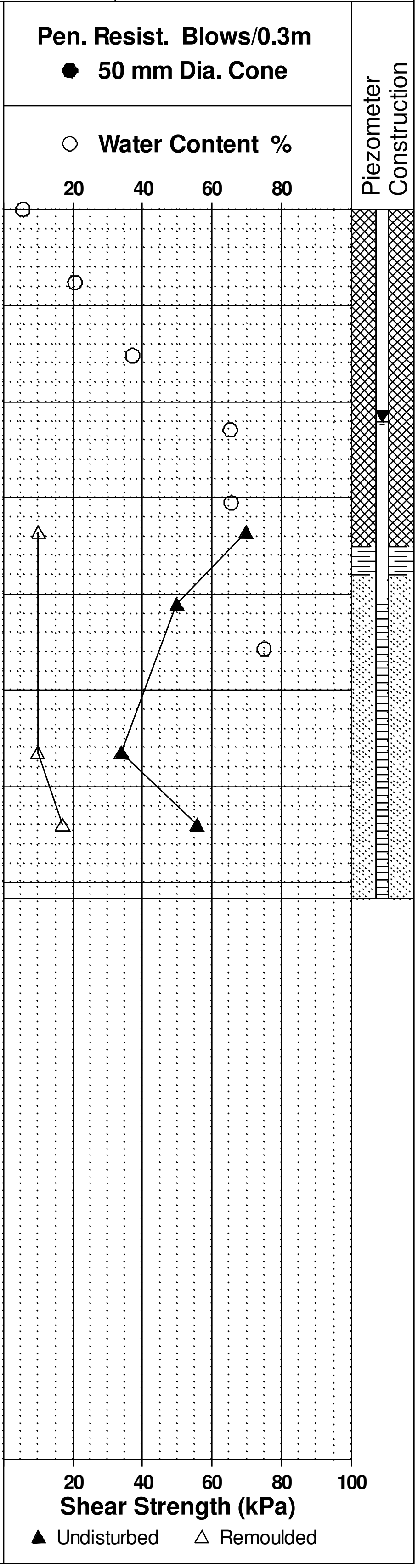
DATE 20 July 2018

FILE NO. PG4557

HOLE NO. BH 2

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD
GROUND SURFACE					
FILL: Brown silty sand, trace gravel and cobbles 0.61		AU	1		
Stiff, brown **SILTY CLAY**		SS	2	50	15
		SS	3	67	7
		SS	4	100	4
		SS	5		P
- grey by 4.6m depth		SS	6	100	2
6.86					
GLACIAL TILL: Grey silty clay, trace sand and gravel 7.16					
End of Borehole					
(GWL @ 2.21m - July 24, 2018)					

DEPTH (m)	ELEV. (m)
0	96.04
1	95.04
2	94.04
3	93.04
4	92.04
5	91.04
6	90.04
7	89.04



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Retirement Residence - 20 Chesapeake Cres.
Ottawa, Ontario

DATUM Geodetic - as provided by Annis O'Sullivan Vollebekk Ltd

REMARKS

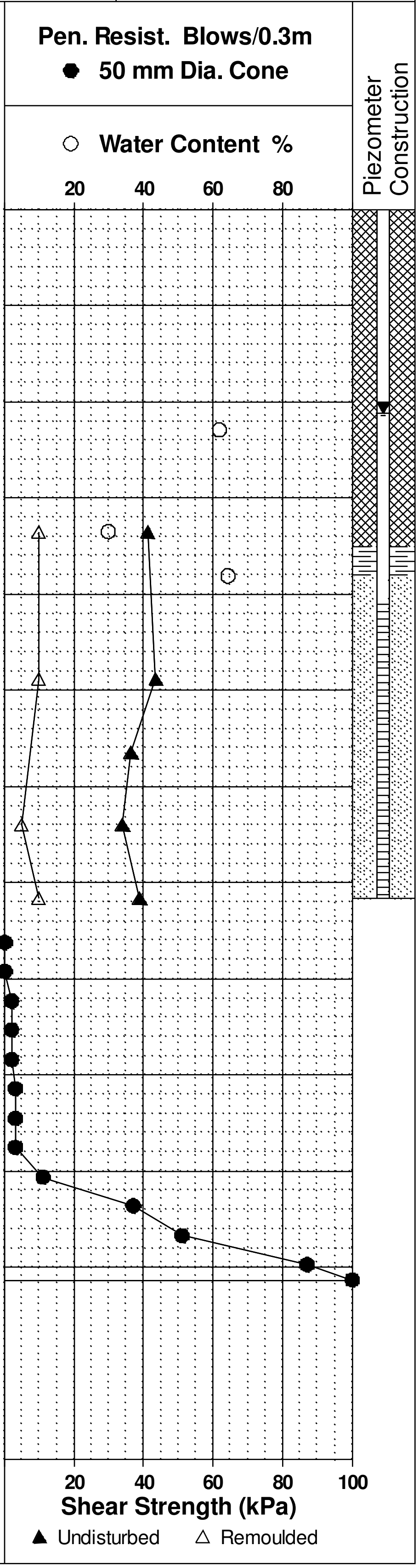
BORINGS BY CME 55 Power Auger

DATE 20 July 2018

FILE NO. PG4557

HOLE NO. BH 3

SOIL DESCRIPTION	TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	95.91
FILL: Brown silty sand, trace clay and gravel	AU	1				
	SS	2	8	18	1	94.91
1.52						
Firm to stiff, brown **SILTY CLAY**	SS	3	0	9		
					2	93.91
	SS	4	100	0		
					3	92.91
- grey by 3.8m depth	SS	5	100	0	4	91.91
					5	90.91
					6	89.91
7.16					7	88.91
Dynamic Cone Penetration Test commenced at 7.16m depth.						
					8	87.91
					9	86.91
					10	85.91
					11	84.91
11.13						
End of Borehole Practical refusal to DCPT @ 11.13 m depth (GWL @ 2.12m - July 24, 2018)						



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Retirement Residence - 20 Chesapeake Cres.
Ottawa, Ontario

DATUM Geodetic - as provided by Annis O'Sullivan Vollebekk Ltd

REMARKS

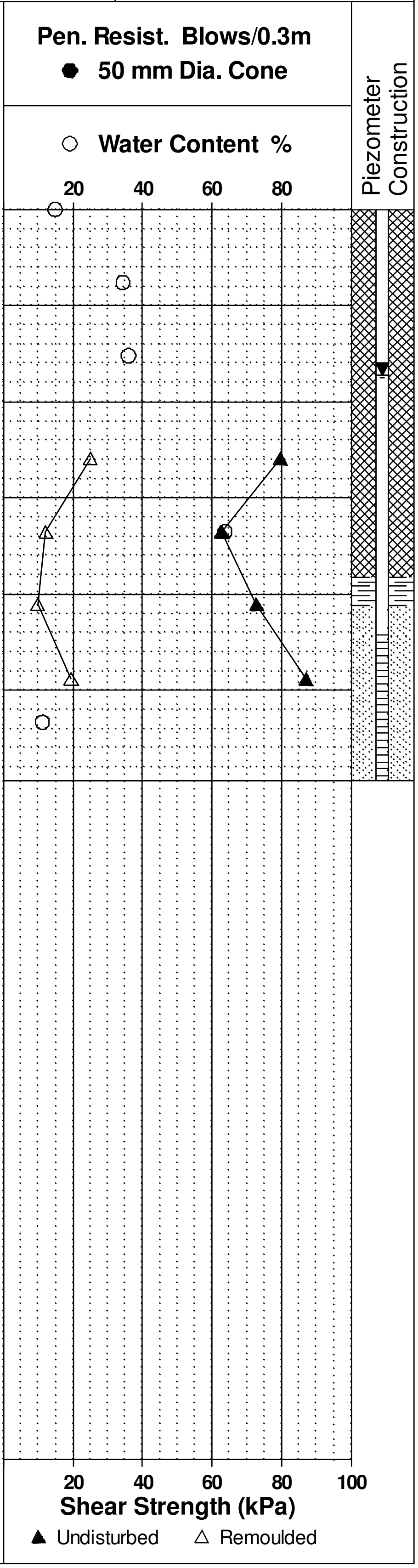
BORINGS BY CME 55 Power Auger

DATE 20 July 2018

FILE NO. PG4557

HOLE NO. BH 4

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	95.90
FILL: Brown silty sand, trace clay and gravel	AU	1				
0.76						
Stiff, brown **SILTY CLAY**	SS	2	54	13	1	94.90
	SS	3	96	7	2	93.90
					3	92.90
					4	91.90
4.57						
GLACIAL TILL: Grey silty clay, some sand, trace gravel, cobbles and boulders					5	90.90
	SS	4	88	6		
5.94						
End of Borehole						
(GWL @ 1.72m - July 24, 2018)						



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Retirement Residence - 20 Chesapeake Cres.
Ottawa, Ontario

DATUM Geodetic - as provided by Annis O'Sullivan Vollebekk Ltd

FILE NO. PG4557

REMARKS

HOLE NO. BH 5

BORINGS BY CME 55 Power Auger

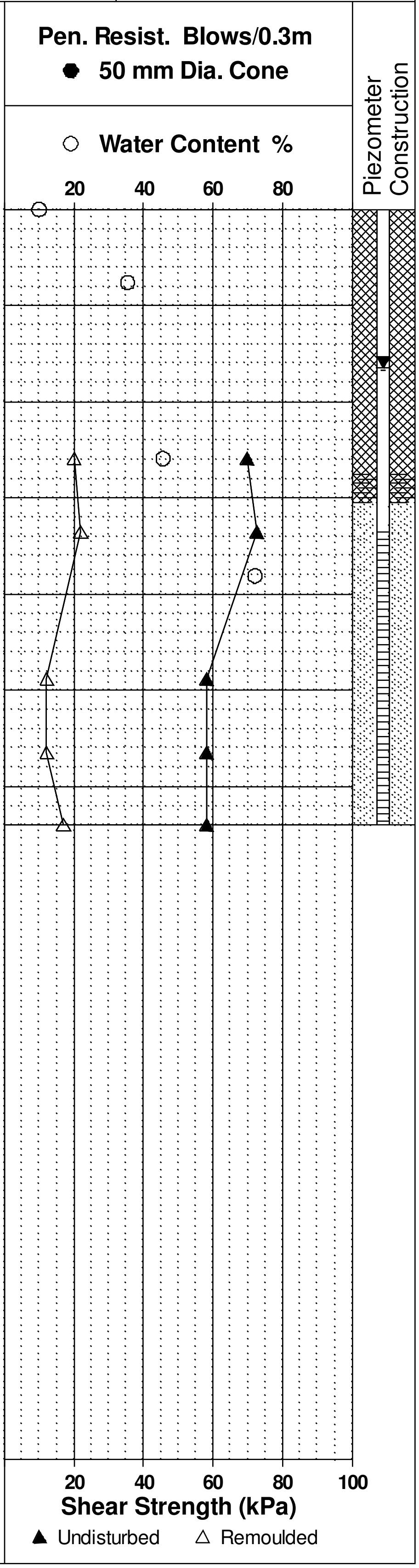
DATE 20 July 2018

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD
GROUND SURFACE					
FILL: Brown silty sand, trace clay and gravel 0.61		AU	1		
Stiff, brown **SILTY CLAY**		SS	2	54	15
		SS	3	100	5
- grey by 3.8m depth		SS	4	100	0
6.10					
GLACIAL TILL: Grey silty clay, some sand, trace gravel 6.40					

End of Borehole

(GWL @ 1.65m - July 24, 2018)

DEPTH (m)	ELEV. (m)
0	95.80
1	94.80
2	93.80
3	92.80
4	91.80
5	90.80
6	89.80



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Retirement Residence - 20 Chesapeake Cres.
Ottawa, Ontario

DATUM Geodetic - as provided by Annis O'Sullivan Vollebekk Ltd

REMARKS

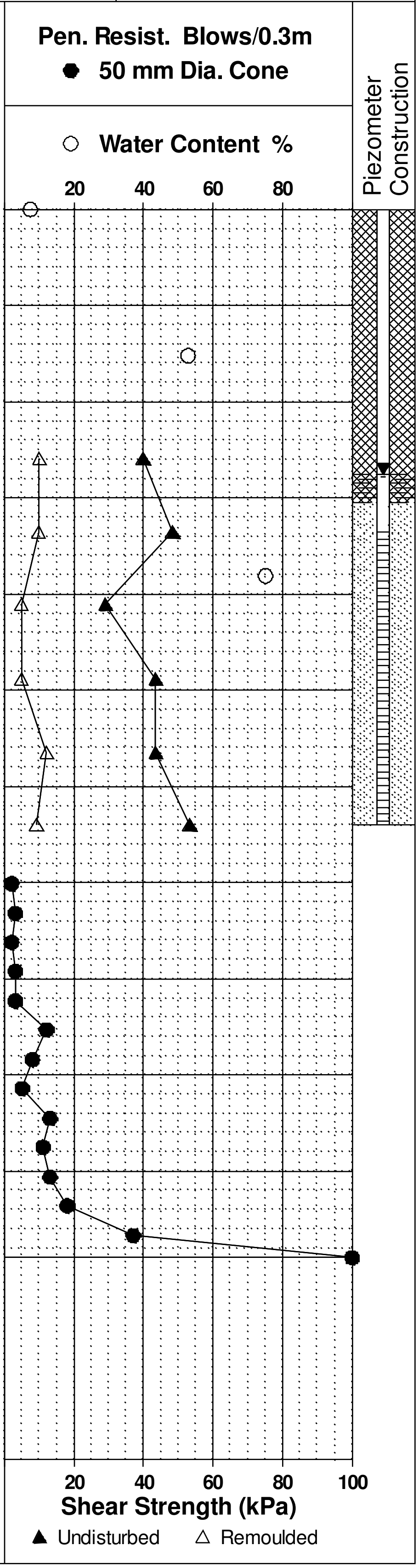
BORINGS BY CME 55 Power Auger

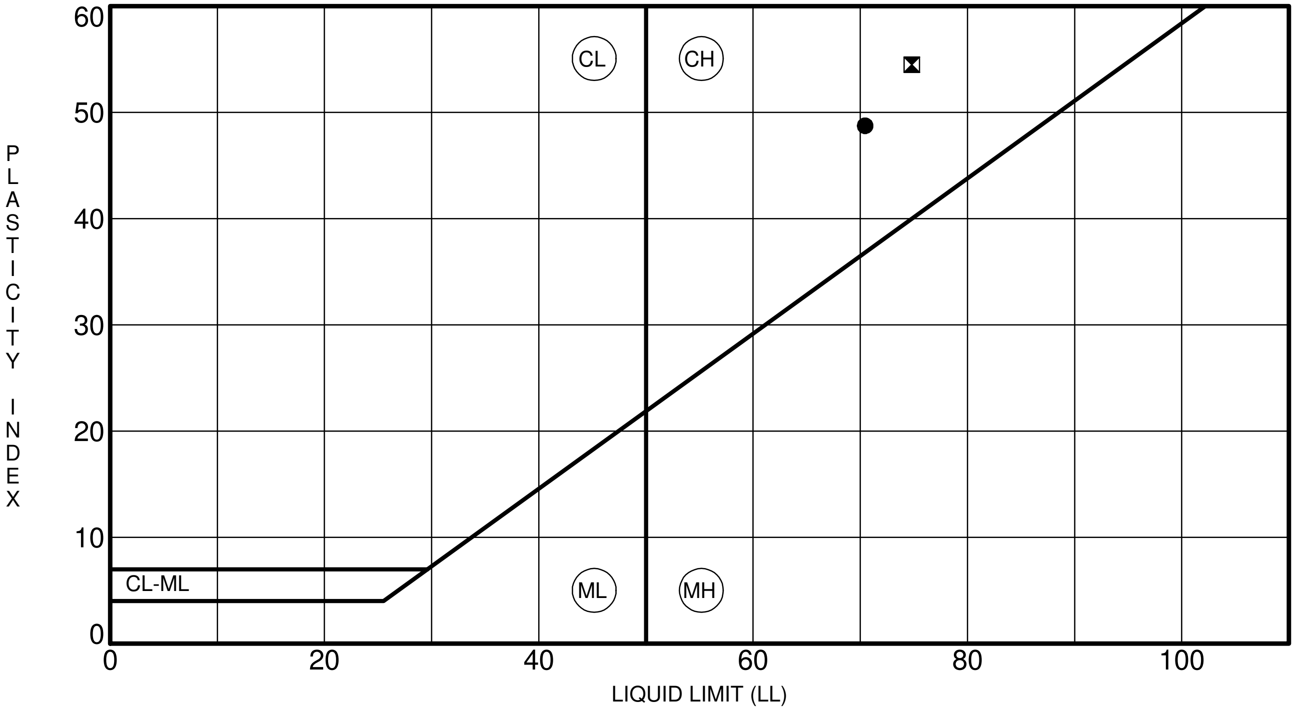
DATE 20 July 2018

FILE NO. PG4557

HOLE NO. BH 6

SOIL DESCRIPTION	TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	95.70
FILL: Brown silty sand, trace clay and gravel	AU	1				
0.91	SS	2	8	16	1	94.70
	SS	3	100	5	2	93.70
Firm to stiff, brown **SILTY CLAY**					3	92.70
- grey by 3.8m depth	SS	4	100	P	4	91.70
					5	90.70
					6	89.70
6.40						
Dynamic Cone Penetration Test commenced at 6.40m depth.					7	88.70
					8	87.70
					9	86.70
					10	85.70
10.90						
End of Borehole Practical refusal to DCPT @ 10.9 m depth (GWL @ 2.76m - July 24, 2018)						





Specimen Identification			LL	PL	PI	Fines	Classification
●	**BH 2**	**SS 5**	**70**	**22**	**49**		**CH - Inorganic clays of high plasticity**
⧗	**BH 6**	**SS 4**	**75**	**20**	**55**		**CH - Inorganic clays of high plasticity**

CLIENT **Claridge Homes**
PROJECT **Geotechnical Investigation - Prop. Retirement Residence - 20 Chesapeake Cres.**

FILE NO. **PG4557**
DATE **20 Jul 18**

patersongroup **Consulting Engineers**
154 Colonnade Road South, Ottawa, Ontario K2E 7J5

ATTERBERG LIMITS' RESULTS

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

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)

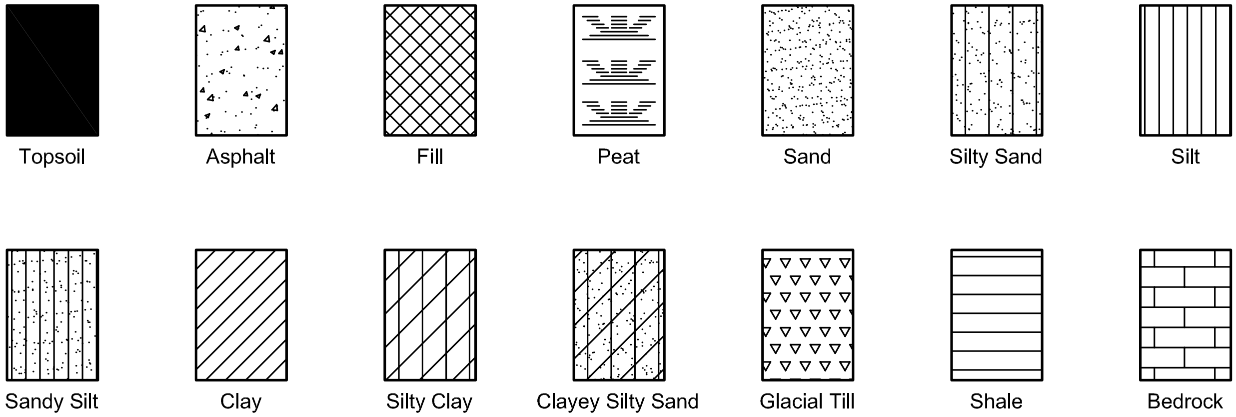
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

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.

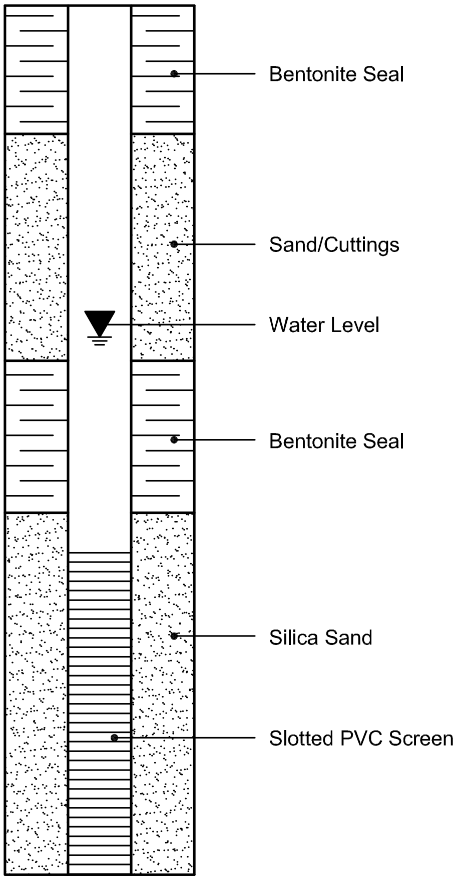
SYMBOLS AND TERMS (continued)

STRATA PLOT

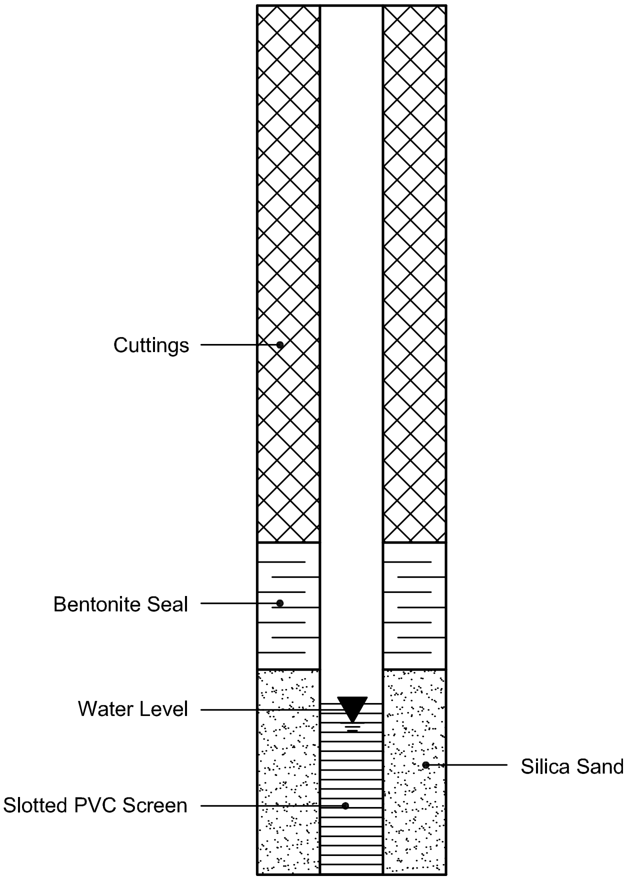


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 24718

Report Date: 26-Jul-2018

Order Date: 24-Jul-2018

Project Description: PG4557

Client ID:		BH5-PSV (7'6"-8'6")	-	-	-
Sample Date:		07/20/2018 09:00	-	-	-
Sample ID:		1830255-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	75.7	-	-	-
General Inorganics					
pH	0.05 pH Units	7.48	-	-	-
Resistivity	0.10 Ohm.m	38.5	-	-	-
Anions					
Chloride	5 ug/g dry	27	-	-	-
Sulphate	5 ug/g dry	133	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING PG4557-1 - TEST HOLE LOCATION PLAN

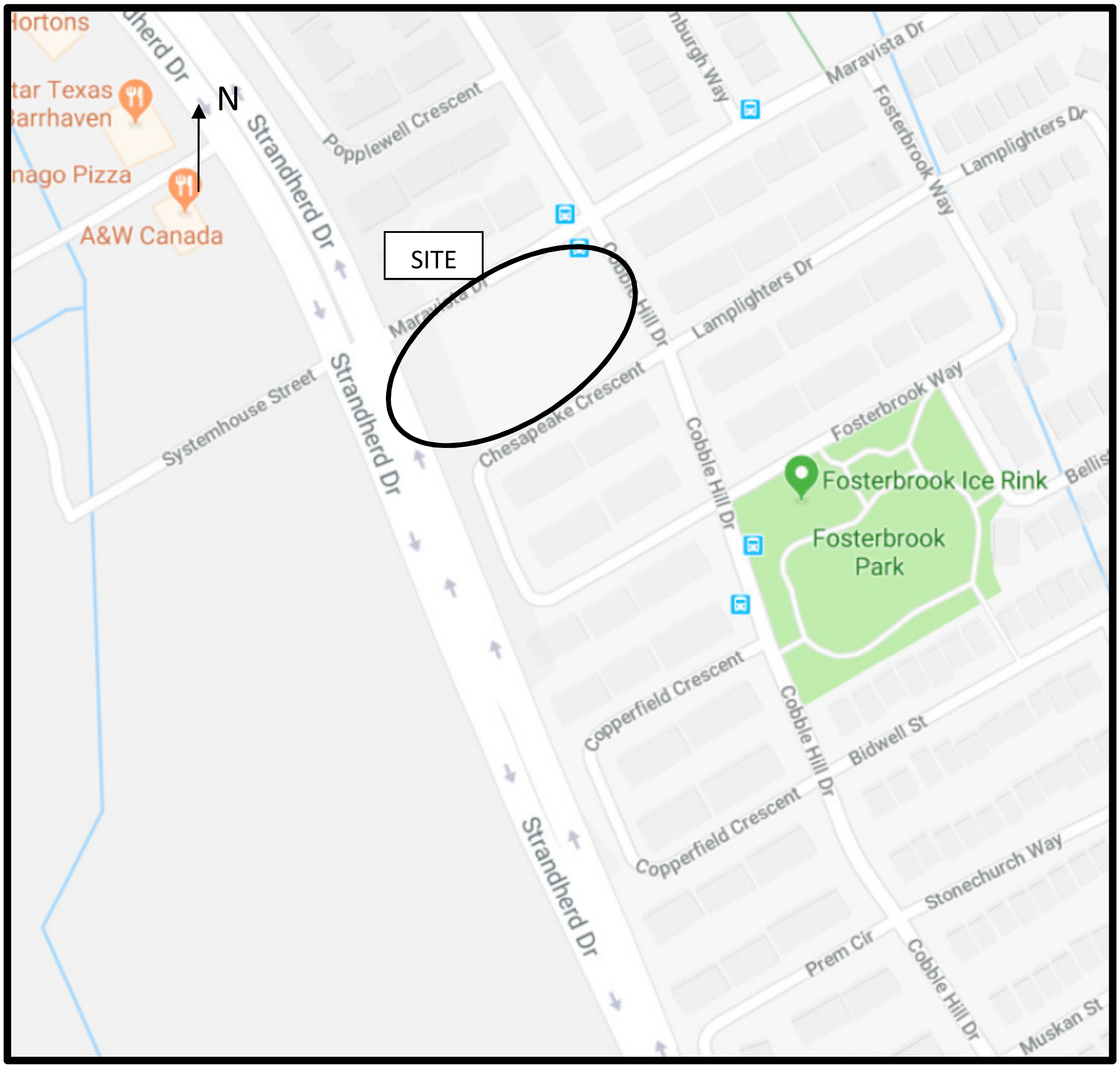
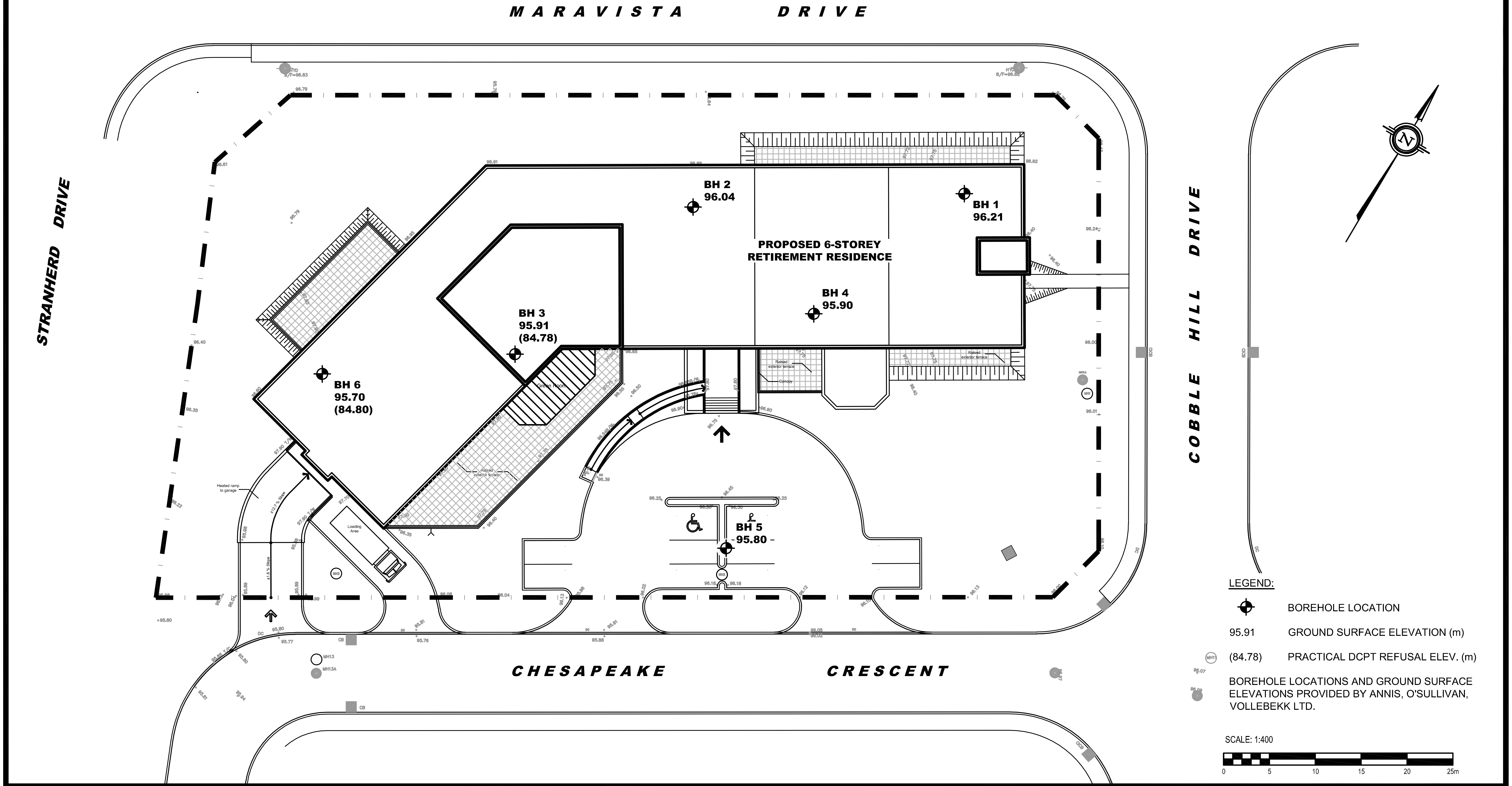


FIGURE 1

KEY PLAN



LEGEND:

⊕	BOREHOLE LOCATION
95.91	GROUND SURFACE ELEVATION (m)
(84.78)	PRACTICAL DCPT REFUSAL ELEV. (m)

BOREHOLE LOCATIONS AND GROUND SURFACE ELEVATIONS PROVIDED BY ANNIS, O'SULLIVAN, VOLLEBEKK LTD.

patersongroup
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. RETIREMENT RESIDENCE - 20 CHESAPEAKE CRES.

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:400
Date: 07/2018
Drawn by: MPG
Report No.: PG4557-1
Checked by: SB
Dwg. No.: PG4557-1
Approved by: DJG
Revision No.: 0

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