Geotechnical Engineering

Environmental Engineering

Hydrogeology

Geological Engineering

Materials Testing

Building Science

Noise and Vibration Studies

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Geotechnical Investigation

Proposed Apartment Complex
2380 & 2396 Cleroux Crescent
Ottawa, Ontario

Prepared For

Bridor Developments

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September 13, 2021

Report: PG5940-1

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Bridor Developments to conduct a geotechnical investigation for the proposed apartment complex site to be located at 2380 and 2396 Cleroux Crescent in the City of Ottawa (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objective of the geotechnical investigation was to:

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

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

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

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

2.0 Proposed Development

Based on the available drawings, it is understood that the proposed development will consist of two multi-storey apartment buildings with one underground parking level. Associated access lanes, parking areas, walkways, and landscaped areas are also anticipated as part of the development. The development is expected to be municipally serviced.

Construction of the proposed development will involve demolition of the existing residential dwellings presently located at the site.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current geotechnical investigation was carried out on August 25, 2021 and consisted of advancing a total of four (4) boreholes to a maximum depth of 6 m below existing ground surface. The test hole locations were distributed in a manner to provide general coverage of the subject site and taking into consideration underground utilities and site features. The borehole locations are shown on Drawing PG5940-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were completed using a low-clearance drill rig operated by a two-person crew. All fieldwork was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer. The drilling procedure consisted of advancing each test hole to the required depths at the selected locations and sampling the overburden.

Sampling and In Situ Testing

The soil samples were collected from the boreholes using a 50 mm diameter split-spoon (SS) sampler. The samples were initially classified on site, placed in sealed plastic bags, and transported to our laboratory. The depths at which the auger, and split-spoon samples were recovered from the boreholes are shown as AU, and SS 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 testing was carried out at regular depth intervals in cohesive soils using a vane apparatus.

The thickness of the overburden was evaluated during the course of the investigation by a dynamic cone penetration test (DCPT) at borehole BH 1-21. 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 boreholes 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 boreholes to permit monitoring of the groundwater levels subsequent to the completion of the sampling program.

Sample Storage

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

3.2 Field Survey

The test hole locations were selected by Paterson to provide general coverage of the proposed development, taking into consideration the existing site features and underground utilities. The test hole locations and ground surface elevation at each test hole location were surveyed by Paterson using a handheld GPS and referenced to a geodetic datum. The location of the boreholes and ground surface elevation at each test hole location are presented on Drawing PG5940-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

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

3.4 Analytical Testing

One (1) soil sample was submitted for analytical testing to assess the corrosion potential for exposed ferrous metals and the potential of sulphate attacks against subsurface concrete structures, one of which was collected from BH 1-21. The sample was submitted to determine the concentration of sulphate and chloride, the resistivity, and the pH of the samples. The results are presented in Appendix 1 and are discussed further in Subsection 6.7.

4.0 Observations

4.1 Surface Conditions

The subject site currently consists of two different lots occupied by single-family residential dwellings with associated garage and driveways, a shed and landscaped areas. A significant number of mature trees were observed to occupy portions of the site. The ground surface across the subject site is relatively flat with a smooth downward slope towards the south.

The site is bordered by Cleroux Crescent followed by a school to the north, Orient Park Drive followed by a senior's residence to the east, residential houses followed by Orient Park Drive to the south, and residential houses followed by Autumn Hill Crescent to the west. The existing ground surface across the site is relatively level at approximate geodetic elevation of 81 to 83 m.

4.2 Subsurface Profile

Overburden

Generally, the soil profile at the test hole locations consists of topsoil and fill overlying a deep deposit of silty clay. At the location of borehole BH 4-21 a layer of compact, reddish brown silty sand was observed overlying the silty clay deposit. The fill layer was generally observed to consist of brown silty clay and/or sand and occasional trace gravel. The fill layer extended to approximately 0.8 m below ground surface. The silty clay deposit consisted of a weathered silty clay crust followed by grey silty clay at all test hole locations. In situ shear vane field tests carried out within the silty clay crust yielded peak undisturbed shear strength values in excess of 100 kPa. These values reflect a very stiff to stiff consistency in the silty clay crust. Grey silty clay was encountered below the brown silty clay crust in all boreholes. In situ shear vane field testing conducted within the grey silty clay layer yielded undisturbed shear strength values generally ranging from 45 to 80 kPa. These values are indicative of a firm to stiff consistency. Practical refusal to the DCPT was encountered at a depth of 28.1 m below ground surface at borehole BH 1-21. Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for the details of the soil profile encountered at each test hole location.

Bedrock

Based on available geological mapping, the bedrock in the subject area consists of Paleozoic interbedded limestone and shale of the Lindsay formation, with an overburden drift thickness of 25 to 50 m depth.

4.3 Groundwater

Groundwater levels were measured on September 1, 2021, within the installed piezometers. The measured groundwater levels are presented in Table 1 below.

Table 1 – Summary of Groundwater Levels

Test Hole Number	Ground Surface Elevation (m)	Measured Groundwater Level		Dated Recorded
		Depth (m)	Elevation (m)	
BH 1-21	81.21	2.05	79.16	September 1, 2021
BH 2-21	81.45	Dry	-	
BH 3-21	82.44	Destroyed	-	
BH 4-21	83.74	3.53	80.21	

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

It is important to note that groundwater readings can be influenced by surface water perched within the borehole backfill material. The long-term groundwater levels can also be estimated based on the observed color and consistency of the recovered soil samples. Based on these observations, the long-term groundwater table can be expected at approximately 4 to 5 m below ground surface. The recorded groundwater levels are noted on the applicable Soil Profile and Test Data sheet presented in Appendix 1.

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

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is suitable for the proposed development. It is expected that the proposed development will be founded on conventional spread footings placed on an undisturbed, very stiff to stiff silty clay deposit.

Due to the presence of the silty clay deposit, a permissible grade raise restriction is required for the subject site. The permissible grade raise recommendations are 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 and deleterious fill, such as those containing organic materials, should be stripped from under any buildings, paved areas, pipe bedding and other settlement sensitive structures.

Existing foundation walls and other construction debris should be entirely removed from within the building perimeters. 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 placed 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. The imported fill material should be tested and approved prior to delivery. The fill should be placed in maximum 300 mm thick loose lifts and compacted by suitable compaction equipment. Fill placed beneath the building should be compacted to a minimum of 99% of the standard Proctor maximum dry density (SPMDD).

Non-specified existing fill along with site-excavated soil could be placed as general landscaping fill where settlement of the ground surface is of minor concern. These materials should be spread in lifts with a maximum thickness of 300 mm and compacted by the tracks of the spreading equipment to minimize voids.

Non-specified existing fill and site-excavated soils are not suitable for placement as backfill against foundation walls, unless used in conjunction with a geocomposite drainage membrane, such as Miradrain G100N or Delta Drain 6000, connected to a perimeter drainage system is provided.

Concrete Mud Slab

The subsoil conditions at the founding elevation will consist of a very stiff to stiff silty clay deposit. To avoid disturbance of the bearing surface, it is recommended that a **75 mm** thick concrete mud slab be poured as the excavation exposes the final bearing surface. The concrete mud slab can consist of **15 MPa** compressive strength concrete.

Excess Soils

Excess soils generated by construction activities that will be transported off-site should be handled as per *Ontario Regulation 406/19: On-Site Excess Soil Management*.

5.3 Foundation Design

Bearing Resistance Values (Conventional Shallow Foundation)

Strip footings, up to 3 m wide, and pad footings, up to 5 m wide, founded on an undisturbed, very stiff to stiff brown silty clay can be designed using a bearing resistance value at serviceability limit states (SLS) of **200 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **325 kPa** incorporating a geotechnical factor of 0.5.

Strip footings, up to 3 m wide, and pad footings, up to 5 m wide, founded on an undisturbed, firm grey silty clay 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 **200 kPa** incorporating a geotechnical factor of 0.5.

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

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

Lateral Support

The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels. Adequate lateral support is provided to a silty clay bearing surface when a plane extending down and out from the bottom edges 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 that of the bearing medium.

Permissible Grade Raise

Based on the undrained shear strength values of the silty clay deposit encountered throughout the subject site, a permissible grade raise restriction of **2.0 m** is recommended for the proposed development. If greater permissible grade raises are required, preloading with or without a surcharge, lightweight fill, and/or other measures should be investigated to reduce the risks of unacceptable long-term post construction total and differential settlements.

5.4 Design for Earthquakes

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

5.5 Basement Slab

With the removal of all topsoil and deleterious fill within the footprint of the proposed building, the existing native soil will be considered an acceptable subgrade upon which to commence backfilling for basement slab construction.

The recommended pavement structures noted in subsection 5.7 will be applicable for the founding level of the proposed parking garage structure. However, if storage or other uses of the lower level will involve the construction of a concrete floor slab, the upper 200 mm of sub-slab fill is recommended to consist of OPSS Granular A crushed stone.

Any soft areas should be removed and backfilled with appropriate backfill material. OPSS Granular B Types I or II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab (outside the zones of influence of the footings).

All backfill material within the footprint of the proposed buildings (but outside the zones of influence of the footings) should be placed in maximum 300 mm thick loose layers and compacted to at least 95% of its SPMDD.

Within the zones of influence of the footings, the backfill material should be compacted to a minimum of 98% of its SPMDD.

5.5 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 dry unit weight of 20 kN/m^3. The applicable effective unit weight of the retained soil can be estimated as 13 kN/m^3, where applicable. A hydrostatic pressure should be added to the total static earth pressure when calculating the effective unit weight.

The total earth pressure (P_{AE}) includes 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 by a triangular earth pressure distribution equal to $K_o \cdot \gamma \cdot H$ where:

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

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:

$a_c = (1.45 - a_{max}/g)a_{max}$
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. The vertical seismic coefficient is assumed to be zero.

The earth force component (P_o) under seismic conditions could be calculated using $P_o = 0.5\ K_o \gamma\ H^2$, where K_o = 0.5 for the soil conditions presented above.

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

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

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

5.7 Pavement Design

Car only parking areas and heavy traffic access areas are expected at this site. The subgrade material will consist of native soil, fill, stiff to very stiff silty clay and or compact silty sand. The proposed pavement structures are presented in Tables 2 and 3.

Table 2 – 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 fill, in-situ soil, or OPSS Granular B Type I or II material placed over in-situ soil or fill.	

Table 3 – 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	**Wear Course –** HL-8 or Superpave 19 Asphaltic Concrete
150	**BASE –** OPSS Granular A Crushed Stone
450	**SUBBASE –** OPSS Granular B Type II
Subgrade – Either fill, in-situ soil, or OPSS Granular B Type I or II material placed over in-situ soil or fill.	

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for this project. The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 100% of the material's SPMDD using suitable compaction equipment.

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.

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

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

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

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.

Underfloor Drainage

Underfloor drainage may be required to control water infiltration for the lower basement area. For preliminary design purposes, we recommend that 150 mm diameter perforated PVC pipes be placed at every bay opening. The spacing of the underfloor drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

6.2 Protection of Footings Against Frost Action

Perimeter footings of heated structures are required to be insulated against the deleterious effects of frost action. A minimum 1.5 m thick soil cover (or insulation equivalent) should be provided in this regard.

Other exterior unheated footings, such as those for isolated exterior piers and retaining walls, are more prone to deleterious movement associated with frost action. A minimum of 2.1 m thick soil cover (or equivalent) should be provided for all exterior unheated footings.

6.3 Excavation Side Slopes

The side slopes of excavations in the overburden materials should be either cut back at acceptable slopes or should be retained by shoring systems from the start of the excavation until the structure is backfilled. It is assumed that sufficient room will be available for the greater part of the excavation to be undertaken by open-cut methods (i.e., unsupported excavations). Where space restrictions exist, or to reduce the trench width, the excavation can be carried out within the confines of a fully braced steel trench box.

Unsupported Side Slopes

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

Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should be kept away from the excavation sides. Slopes in excess of 3 m in height should be periodically inspected by the geotechnical consultant in order to detect if the slopes are exhibiting signs of distress.

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.

6.4 Pipe Bedding and Backfill

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

At least 300 mm of OPSS Granular A should be used for pipe bedding for sewer and water pipes. The bedding should extend to the spring line of the pipe. Cover material, from the spring line to at least 300 mm above the obvert of the pipe, should consist of OPSS Granular A or Granular B Type II with a maximum size of 25 mm.

The bedding and cover materials should be placed in maximum 225 mm thick lifts compacted to 95% of the material’s standard Proctor maximum dry density.

It should generally be possible to re-use the upper portion of the dry to moist (not wet) silty clay above the cover material if the excavation and filling operations are carried out in dry weather conditions. The wet silty clay should be given a sufficient drying period to decrease its moisture content to an acceptable level to make compaction possible prior to being re-used.

The backfill material within the frost zone (about 1.8 m below finished grade) should match the soils exposed at the trench walls to reduce potential differential frost heaving. The backfill should be placed in maximum 225 mm thick loose lifts and compacted to a minimum of 95% of the material's SPMDD.

Clay Seals

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

Groundwater Control for Building Construction

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

Permit to Take Water

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.

Long-term Groundwater Control

Any groundwater encountered along the buildings' perimeter or sub-slab drainage system will be directed to the proposed buildings' cistern/sump pit. Provided the proposed groundwater infiltration control system is properly implemented and approved by the geotechnical consultant at the time of construction, the expected long-term groundwater flow should be low (i.e. less than 25,000 L/day/building) with peak periods noted after rain events. A more accurate estimate can be provided at the time of construction, once groundwater infiltration levels are observed. The long-term groundwater flow is anticipated to be controllable using conventional open sumps.

Impacts on Neighboring Properties

A local groundwater lowering is anticipated under short-term conditions due to construction of the proposed buildings. Based on the anticipated groundwater level, the extent of any significant groundwater lowering will take place within a limited range of the proposed building. Based on the proximity of neighboring buildings and minimal zone impacted by the groundwater lowering, the proposed development will not negatively impact the neighboring structures.

6.6 Winter Construction

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

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

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

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

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

- Grading plan review from a geotechnical perspective, once the final grading plan is available.
- Observation of all bearing surfaces prior to the placement of concrete.
- Sampling and testing of the concrete and fill materials.
- Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- Observation of all subgrades prior to backfilling.
- 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 Bridor Developments 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
Sept. 13-2021
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PROVINCE OF ONTARIO

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Report Distribution:

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- ❏ Paterson Group (e-mail copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Apartment Complex - 2380-2396 Cleroux Cres.
Ottawa, Ontario

DATUM Geodetic

FILE NO. PG5940

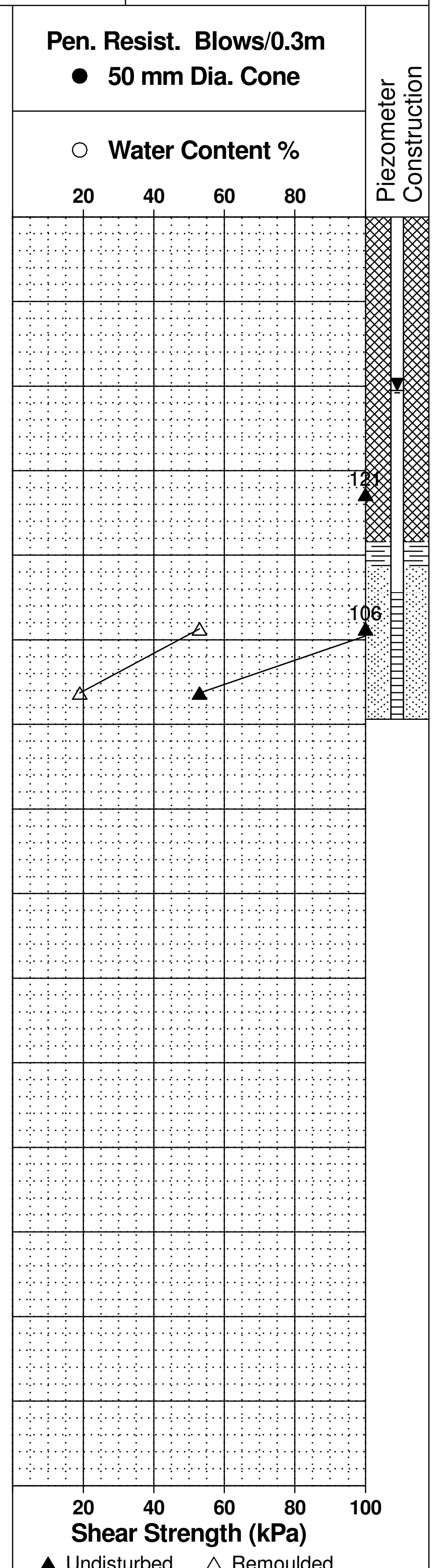
REMARKS

HOLE NO. BH 1-21

BORINGS BY CME-55 Low Clearance Drill

DATE August 25, 2021

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	81.21
TOPSOIL 0.10						
FILL: Brown silty sand, trace clay 0.76	AU	1				
	SS	2	83	18	1	80.21
	SS	3	75	12	2	79.21
Very stiff to stiff, brown **SILTY CLAY**	SS	4	75	7		
					3	78.21
	SS	5	83	7	4	77.21
	SS	6	83	P	5	76.21
- grey by 5.3m depth	SS	7	83	P		
5.94					6	75.21
Dynamic Cone Penetration Test commenced at 5.94m depth. Cone pushed to 25.3m depth.					7	74.21
					8	73.21
					9	72.21
					10	71.21
					11	70.21
					12	69.21
					13	68.21
					14	67.21
					15	66.21



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Apartment Complex - 2380-2396 Cleroux Cres.
Ottawa, Ontario

DATUM Geodetic

REMARKS

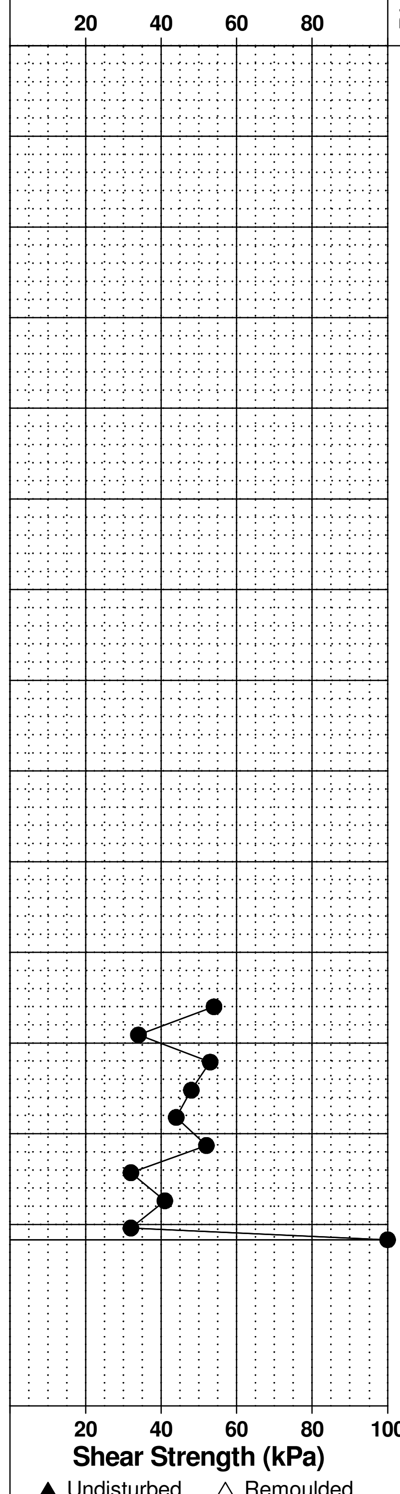
BORINGS BY CME-55 Low Clearance Drill

DATE August 25, 2021

FILE NO. PG5940

HOLE NO. BH 1-21

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content %	Piezometer Construction
GROUND SURFACE									
						15	66.21		
						16	65.21		
						17	64.21		
						18	63.21		
						19	62.21		
						20	61.21		
						21	60.21		
						22	59.21		
						23	58.21		
						24	57.21		
						25	56.21		
						26	55.21		
						27	54.21		
						28	53.21		
28.17									



End of Borehole

Practical DCPT refusal 28.17m depth.

(GWL @ 2.05m - Sept. 1, 2021)

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Apartment Complex - 2380-2396 Cleroux Cres.
Ottawa, Ontario

DATUM Geodetic

REMARKS

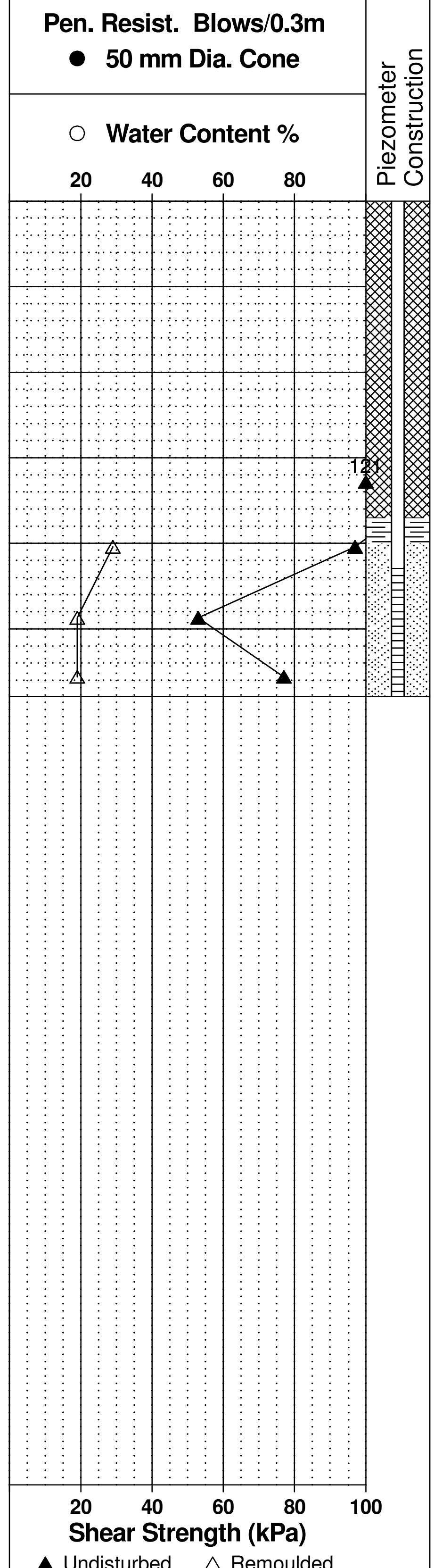
BORINGS BY CME-55 Low Clearance Drill

DATE August 25, 2021

FILE NO. PG5940

HOLE NO. BH 2-21

SOIL DESCRIPTION		SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	81.45
TOPSOIL	0.10						
FILL: Brown silty clay, trace sand	0.60	AU	1				
		SS	2	83	11	1	80.45
		SS	3	83	12	2	79.45
Very stiff to stiff, brown **SILTY CLAY**		SS	4	83	5		
						3	78.45
- grey by 4.4m depth						4	77.45
		SS	5	92	P	5	76.45
	5.79						
End of Borehole							
(BH dry - Sept. 1, 2021)							



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Apartment Complex - 2380-2396 Cleroux Cres.
Ottawa, Ontario

DATUM Geodetic

REMARKS

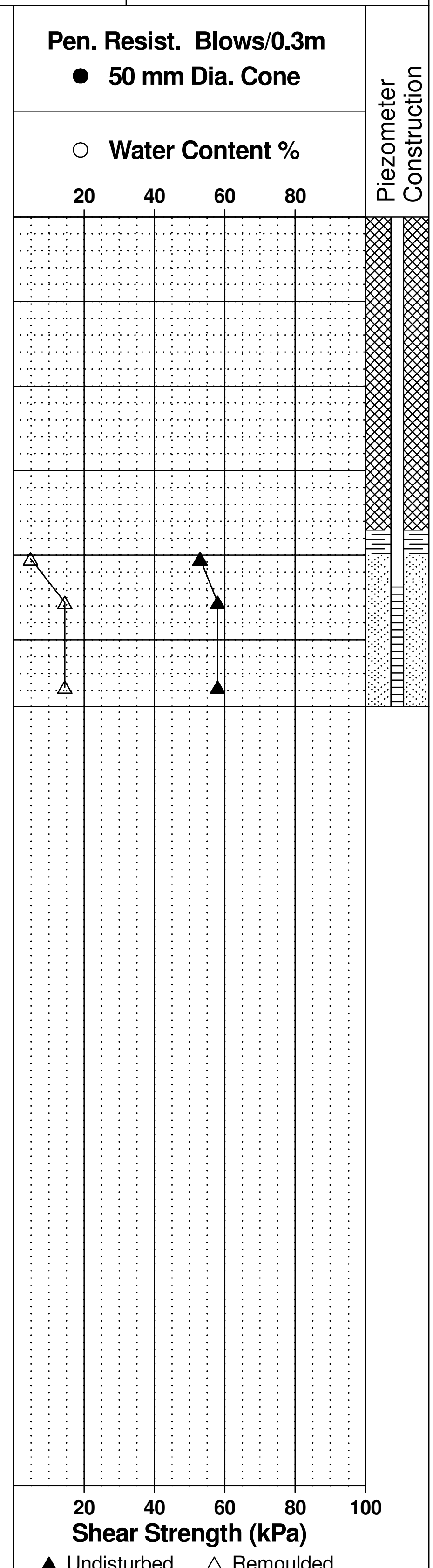
BORINGS BY CME-55 Low Clearance Drill

DATE August 25, 2021

FILE NO. PG5940

HOLE NO. BH 3-21

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	82.44
TOPSOIL 0.15						
FILL: Brown silty sand, trace gravel 0.51	AU	1				
Very stiff to stiff, brown **SILTY CLAY**	SS	2	92	13	1	81.44
	SS	3	58	13	2	80.44
	SS	4	75	11	3	79.44
	SS	5	100	4	4	78.44
- grey by 3.7m depth	SS	6	100	P	5	77.44
5.79						
End of Borehole						
(Piezometer destroyed - Sept. 1, 2021)						



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Apartment Complex - 2380-2396 Cleroux Cres.
Ottawa, Ontario

DATUM Geodetic

FILE NO. **PG5940**

REMARKS

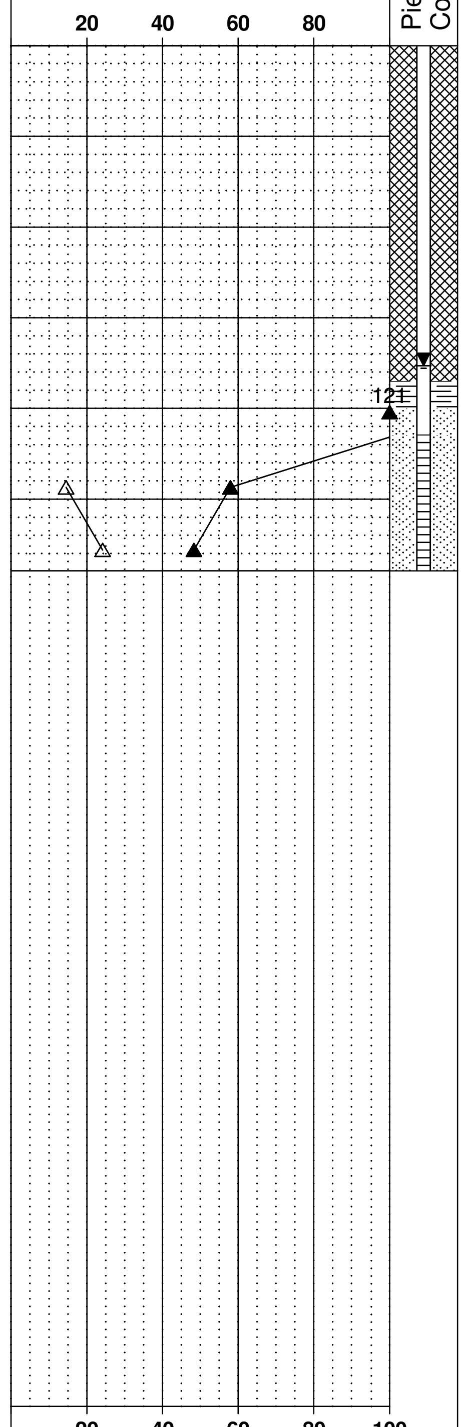
HOLE NO. **BH 4-21**

BORINGS BY CME-55 Low Clearance Drill **DATE** August 25, 2021

SOIL DESCRIPTION	Depth of change (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	83.74
TOPSOIL	0.20						
FILL: Brown silty sand, trace gravel	0.46	AU	1				
Compact, reddish brown **SILTY SAND**	1.22	SS	2	67	15	1	82.74
Very stiff to stiff, brown **SILTY CLAY**		SS	3	75	11	2	81.74
		SS	4	75	8	3	80.74
		SS	5	83	4	4	79.74
- grey by 4.4m depth							
- firm by 5.3m depth	5.79	SS	6	100	P	5	78.74
End of Borehole							
(GWL @ 3.53m - Sept. 1, 2021)							

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

○ Water Content %



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

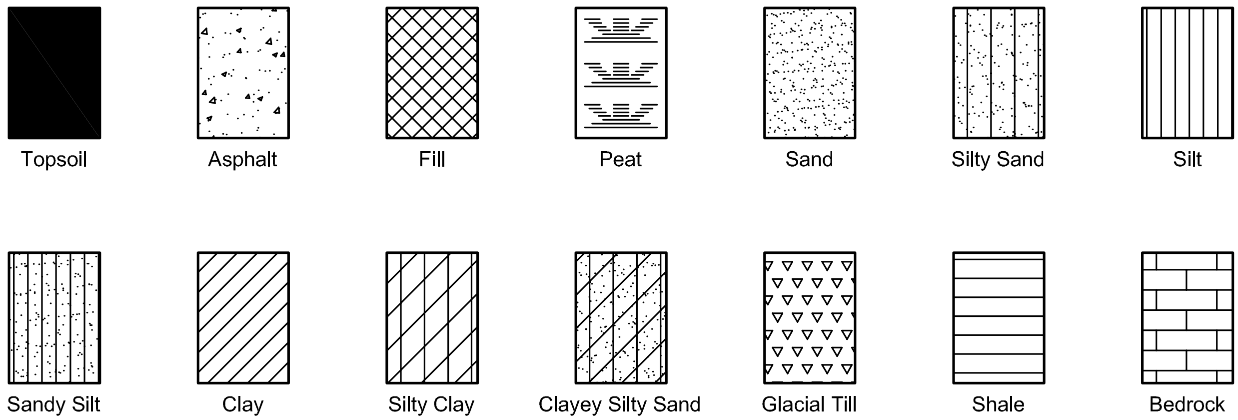
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void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

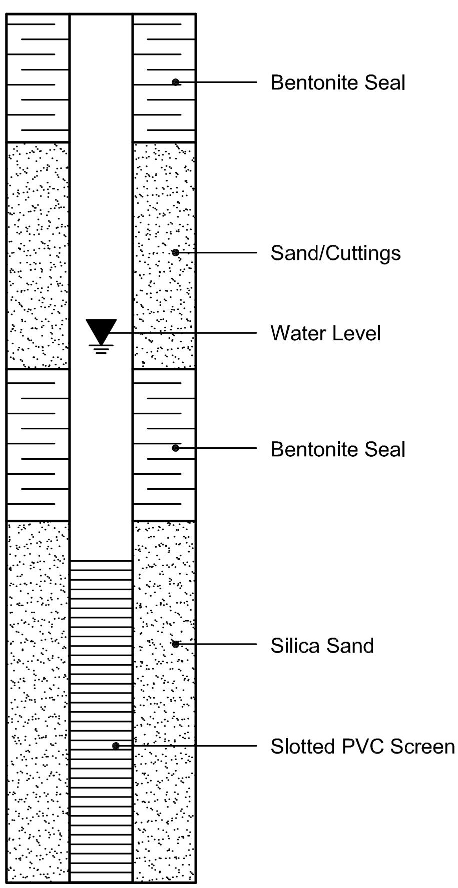
SYMBOLS AND TERMS (continued)

STRATA PLOT

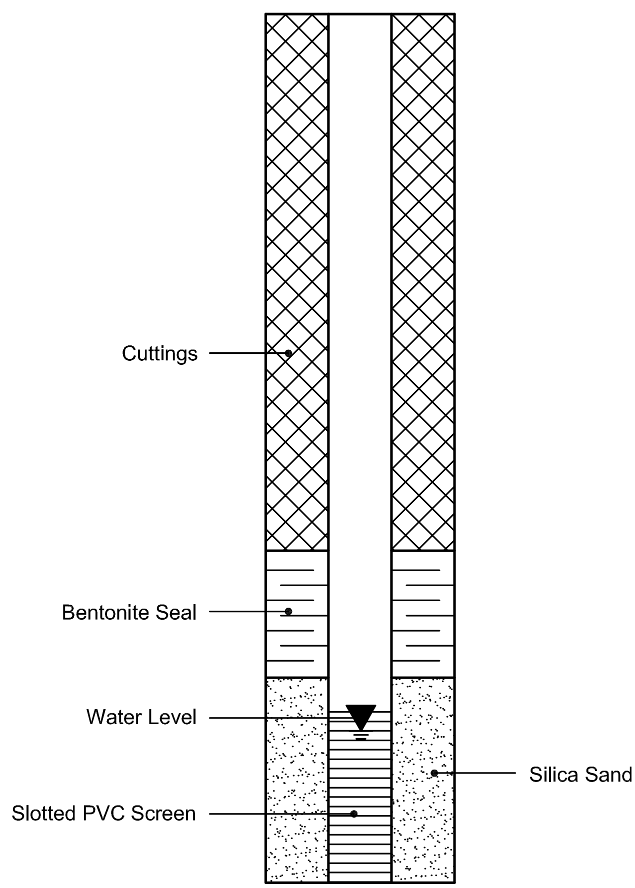


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



PARACEL
LABORATORIES LTD.

Order #: 2135643

Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 24532

Report Date: 01-Sep-2021

Order Date: 26-Aug-2021

Project Description: PG5940

	Client ID:	BH1-21 / SS3	-	-	-
	Sample Date:	25-Aug-21 09:00	-	-	-
	Sample ID:	2135643-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	75.1	-	-	-
General Inorganics					
pH	0.05 pH Units	7.14	-	-	-
Resistivity	0.10 Ohm.m	80.0	-	-	-
Anions					
Chloride	5 ug/g dry	16	-	-	-
Sulphate	5 ug/g dry	31	-	-	-

APPENDIX 2

FIGURE 1 – KEY PLAN

DRAWING PG5940-1 – TEST HOLE LOCATION PLAN

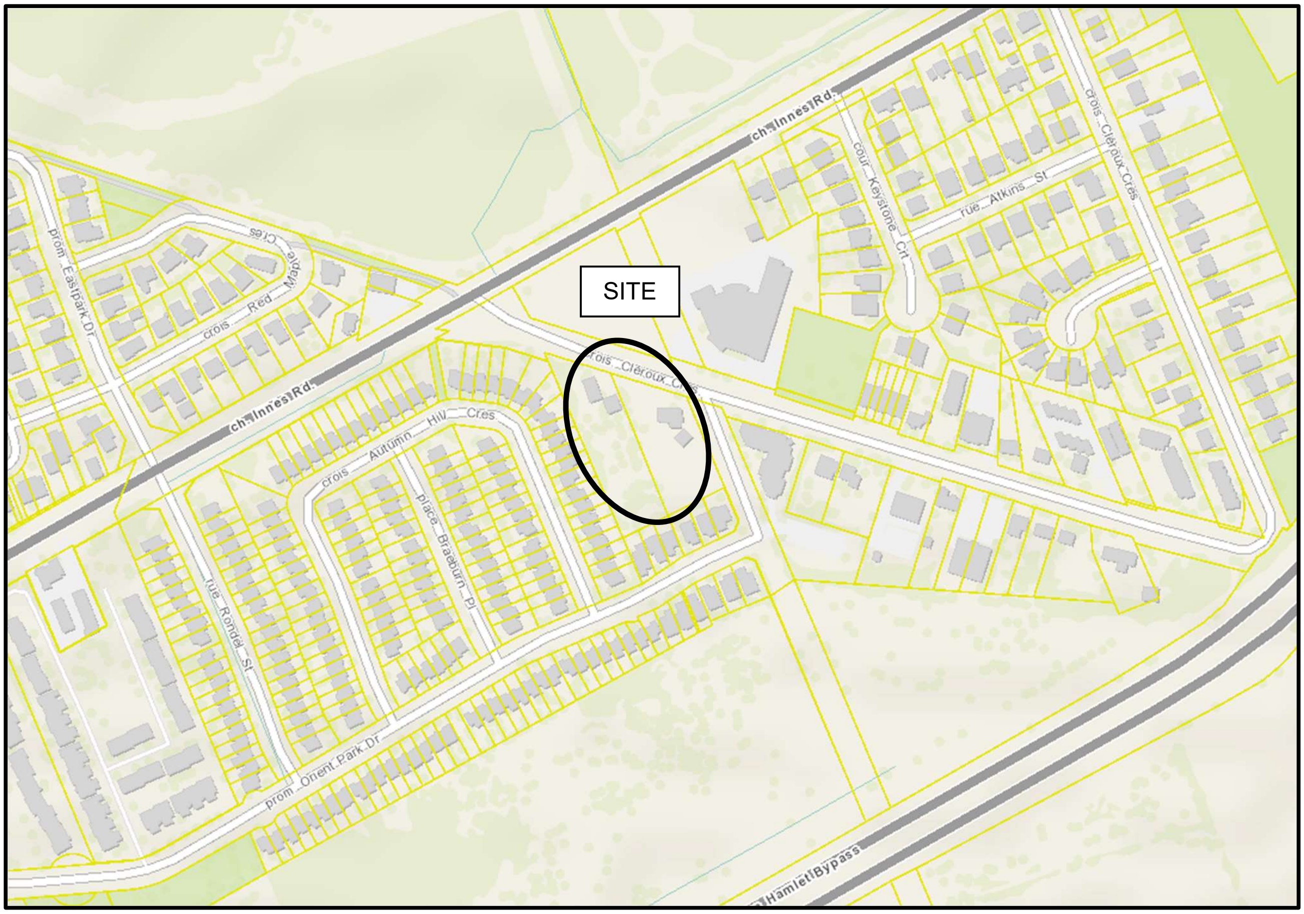


FIGURE 1

KEY PLAN

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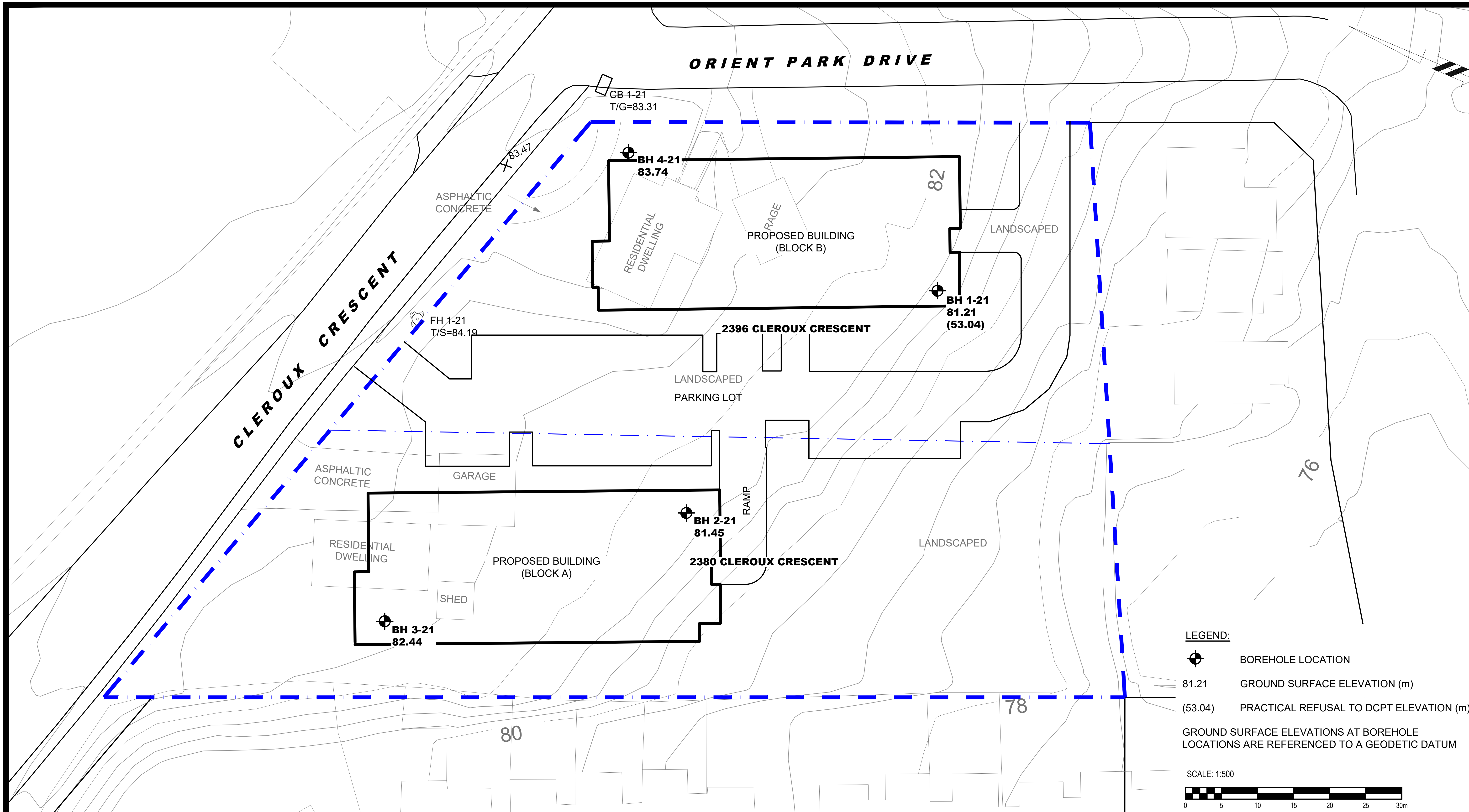

patersongroup
consulting engineers

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NO.	REVISIONS	DATE	INITIAL

BRIDOR DEVELOPMENTS
GEOTECHNICAL INVESTIGATION
PROPOSED APARTMENT COMPLEX
2380 AND 2396 CLEROUX CRESCENT

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:500
Date: 09/2021
Drawn by: YA
Report No.: PG5940-1
Checked by: MS
Dwg. No.: PG5940-1
Approved by: FA
Revision No.:

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