Geotechnical Engineering

Environmental Engineering

Hydrogeology

Geological Engineering

Materials Testing

Building Science

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Geotechnical Investigation

Proposed Residential Buildings
890 Byron Avenue and
455, 463 and 483 Sherbourne Road
Ottawa, Ontario

Prepared For

Concorde Properties

June 23, 2020

Report: PG5364-1

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1.0 Introduction

Paterson Group (Paterson) was commissioned by Concorde Properties to conduct a geotechnical investigation for the proposed residential buildings to be located at 890 Byron Avenue and 455, 463 and 483 Sherbourne Avenue, in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objective of the current investigation was to:

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

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

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

2.0 Proposed Development

Based on the available drawings, it is our understanding that the proposed development will consist of several three-storey apartment-style residential buildings each provided with a partial basement level. At-grade asphalt paved parking areas, access lanes and landscaped areas are anticipated surrounding the proposed buildings. It is also anticipated that the proposed buildings will be municipally serviced.

It is further understood that the existing buildings within the footprints of the proposed buildings will be demolished as part of the proposed development.

3.0 Method of Investigation

3.1 Field Investigation

The field program for the geotechnical investigation was carried out on June 1, 2020. At that time, 7 boreholes were drilled to a maximum depth of 6.7 m below existing grade. The borehole locations were distributed in a manner to provide general coverage of the proposed development. The locations of the boreholes are shown on Drawing PG5364-1 - Test Hole Location Plan included in Appendix 2.

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

Sampling and In Situ Testing

Soil samples were recovered with a 50 mm diameter split-spoon sampler or from the auger flights. The split-spoon and auger samples were classified on site, placed in sealed plastic bags, and transported to the laboratory for further review. 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 and is the number of blows required to drive the split-spoon sampler 300 mm into the soil after a 150 mm initial penetration using a 63.5 kg hammer falling from a height of 760 mm.

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

Groundwater

Flexible polyethylene standpipes were installed within all boreholes to permit monitoring of the groundwater levels subsequent to the completion of the sampling program.

Sample Storage

All soil samples 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 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 and referenced to a geodetic datum. The location of the test holes and ground surface elevation at each test hole location are presented on Drawing PG5364-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

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

3.4 Analytical Testing

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

4.0 Observations

4.1 Surface Conditions

The majority of the subject site is presently occupied by multiple existing residential apartment buildings ranging between three to six above-ground storeys. The ground surface across the subject site is relatively flat and at grade with the adjacent roads. The majority of the site is surfaces with paved parking areas and grass/tree covered landscaped areas. The site is bordered to the north and east by Redwood Avenue, to the south by Sherbourne Road and to the west by Byron Avenue.

4.2 Subsurface Profile

Overburden

The subsurface profile at the borehole locations generally consists of topsoil underlain by a 0.9 to 1.4 m thickness of fill. The fill was generally observed to be composed of silty sand or silty clay with gravel.

Underlying the fill, a hard to very stiff deposit of brown silty clay with some sand seams was encountered. The silty clay deposit was observed to be underlain by a deposit of glacial till at approximate depths of 1.4 to 2.4 m below the existing ground surface. The glacial till was observed to be composed of a brown to grey, silty clay with sand gravel, cobbles and boulders.

Practical refusal to augering was encountered in BH 3, BH 5, BH 6 and BH 7 at depths of 4.3, 2.8, 5.5 and 6.4 m below ground surface, respectively. Specific details of the soil profile at each test hole location are presented on the Soil Profile and Test Data sheets provided in Appendix 1.

Bedrock

Based on field observations and available geological mapping, the local bedrock consists of limestone with interbedded dolomite of the Gull River formation with a drift thickness of 5 to 10 m.

4.3 Groundwater

Groundwater level readings were measured at the piezometer locations on June 9, 2020. The observed groundwater levels are summarized in Table 1. The groundwater level observations are presented on the Soil Profile and Test Data sheets in Appendix 1.

Table 1 - Summary of Groundwater Level Readings

Test Hole Number	Ground Surface Elevation (m)	Groundwater Levels (m)	Groundwater Elevation (m)	Recording Date
BH 1	64.30	1.96	62.34	June 9, 2020
BH 2	64.35	1.89	62.46	June 9, 2020
BH 3	64.13	2.04	62.09	June 9, 2020
BH 4	64.47	Blocked	-	June 9, 2020
BH 5	64.19	Blocked	-	June 9, 2020
BH 6	64.31	2.84	61.47	June 9, 2020
BH 7	64.18	Blocked	-	June 9, 2020

Notes: Ground surface elevations at test hole locations were surveyed by Paterson and are referenced to a geodetic datum.

It should be noted that groundwater levels can be influenced by surface water infiltrating the backfilled boreholes. Long-term groundwater levels can also be estimated based on the observed colour and consistency of the recovered soil samples. Based on these observations, it is estimated that the long-term groundwater table can be expected at approximately 2.5 to 3.5 m below ground surface. It should also be noted that groundwater levels are subject to seasonal fluctuations, and therefore the groundwater level could vary at the time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is suitable for the proposed residential buildings. It is recommended that the proposed buildings will be founded using conventional shallow foundations placed on an undisturbed, very stiff brown silty clay or compact glacial till bearing surface.

Due to the presence of the silty clay deposit, a permissible grade raise restriction is required for grading at the subject site.

It is anticipated that some bedrock removal may be required to complete the proposed partial basement levels and site servicing. All contractors should be prepared for bedrock removal within the subject site.

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

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and fill, such as those containing organic or deleterious materials, should be stripped from under any buildings and other settlement sensitive structures. It is anticipated that the existing fill within the proposed building footprints, free of deleterious material and significant amounts of organics, can be left in place below the proposed building footprints outside of lateral support zones for the footings. However, it is recommended that the existing fill layer be proof-rolled several times and approved by the geotechnical consultant at the time of construction. Any poor performing areas noted during the proof-rolling operation should be removed and replaced with an approved fill.

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

Bedrock Removal

Where a small quantity of bedrock removal is required, it can be accomplished by a combination of hoe ramming and conventional excavation techniques.

Fill Placement

Fill used for grading beneath the building footprints, unless otherwise specified, should consist of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. The fill should be tested and approved prior to delivery to the site. It should be placed in lifts no greater than 300 mm 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).

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. Site excavated soils are not suitable for use as backfill against foundation walls due to the frost heave potential of the site excavated soils below settlement sensitive areas, such as concrete sidewalks and exterior concrete entrance areas.

5.3 Foundation Design

Bearing Resistance Values

Strip footings, up to 3 m wide, and pad footings, up to 5 m wide, placed over an undisturbed, very stiff silty clay bearing surface can be designed using a serviceability limit states (SLS) of **150 kPa** and a factored bearing resistance at ultimate limit states (ULS) of **225 kPa**.

Conventional spread footings placed on an undisturbed, compact glacial till bearing surface can be designed using a bearing resistance values at serviceability limit states (SLS) of **150 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **225 kPa**.

A geotechnical resistance factor of 0.5 was applied to the bearing resistance values at ULS. Footings designed using the 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.

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 very stiff silty clay or compact glacial till above the groundwater table, when a plane extending down and out from the bottom edge of the footing at a minimum of 1.5H:1V passes only through in situ soil of the same or higher capacity as the bearing medium soil.

Settlement

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

Permissible Grade Raise Restriction

Based on the current borehole information, a permissible grade raise restriction of **2 m** is recommended for the proposed buildings and settlement sensitive structures where founded over a silty clay deposit.

If higher than permissible grade raises are required, preloading with or without a surcharge, lightweight fill and/or other measures should be investigated to reduce the risk 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 C** for the foundations considered at this site. The soils underlying the proposed shallow foundations are not susceptible to liquefaction. Reference should be made to the latest revision of the Ontario Building Code (OBC) 2012 for a full discussion of the earthquake design requirements.

5.5 Basement Slab

With the removal of all topsoil and deleterious fill from within the footprint of the proposed buildings, the native soil surface or approved fill surface will be considered to be an acceptable subgrade on which to commence backfilling for floor slab construction.

In areas where existing fill, free of deleterious materials and significant amounts of organics, are present at the basement slab subgrade, it is expected that the fill can remain provided that Paterson approves the fill at the time of construction.

The existing fill should be proof-rolled using a suitably sized vibratory roller making several passes under **dry conditions and above freezing temperatures**. Any poor performing areas should be removed and reinstated with a compacted engineered fill.

Any soft areas should be removed and backfilled with appropriate backfill material prior to placing any fill. OPSS Granular B Type II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab. For structures with basement slabs, it is recommended that the upper 200 mm of subfloor fill consists of 19 mm clear crushed stone.

Further, a sub-slab drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided underlying the basement slabs. This is discussed further in Subsection 6.1.

5.6 Basement Wall

There are several combinations of backfill materials and retained soils that could be applicable for the basement walls of the subject structure. However, 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 total earth pressure (P_{AE}) includes both the static earth pressure component (P_o) and the seismic component (ΔP_{AE}).

Static Conditions

The static horizontal earth pressure (p_o) could be calculated with 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 Conditions

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 = $(1.45\text{-}a_{max}/g)a_{max}$
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

The pavement structures presented in the following tables are recommended for the at-grade parking areas and access lanes.

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 Flexible Pavement Structure - Access Lanes and Heavy Truck Parking Areas	
Thickness (mm)	**Material Description**
40	**Wear Course -** HL-3 or Superpave 12.5 Asphaltic Concrete
50	**Binder Course -** HL-8 or Superpave 19.0 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
450	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Either fill, in situ soil, or OPSS Granular B Type I or II material placed over in situ soil.	

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for this project. If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be sub-excavated and replaced with OPSS Granular B Type II material.

The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 99% of the SPMDD.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

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

Sub-slab drainage is also recommended to control water infiltration. For preliminary design purposes, we recommend that 100 or 150 mm perforated pipes be placed at approximate 6 m centers underlying the basement slabs. The spacing of the sub-slab drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

Backfill against the exterior sides of the foundation walls should consist of free-draining, non frost susceptible granular materials. The site materials will be frost susceptible and, as such, are not recommended for re-use as backfill unless a composite drainage system (such as system Platon or Miradrain G100N) connected to a drainage system is provided.

6.2 Protection of Footings Against Frost Action

Perimeter footings of heated structures are recommended to be protected against the deleterious effects of frost action. A minimum of 1.5 m of soil cover alone, or a combination of soil cover and foundation insulation should be provided.

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

6.3 Excavation Side Slopes

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

Unsupported Excavations

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be excavated at 1H:1V or shallower. The shallower slope is required for excavation below groundwater level. The subsurface 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

Due to the anticipated proximity to the property boundaries, temporary shoring may be required to support the overburden soils of the adjacent properties. The design and approval of the shoring system will be the responsibility of the shoring contractor and the shoring designer who is a licensed professional engineer and is hired by the shoring contractor. It is the responsibility of the shoring contractor to ensure that the temporary shoring is in compliance with safety requirements, designed to avoid any damage to adjacent structures and include dewatering control measures. In the event that subsurface conditions differ from the approved design during the actual installation, it is the responsibility of the shoring contractor to commission the required experts to re-assess the design and implement the required changes. Furthermore, the design of the temporary shoring system should take into consideration a full hydrostatic condition which can occur during significant precipitation events.

The temporary shoring system may consist of a soldier pile and lagging system which could be cantilevered, anchored or braced. The shoring system is recommended to be adequately supported to resist toe failure, if required, by means of rock bolts or extending the piles into the bedrock through pre-augered holes, if a soldier pile and lagging system is the preferred method.

Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be added to the earth pressures described below. The earth pressures acting on the shoring system may be calculated using the following parameters.

Table 4 - Soil Parameters	
Parameters	**Values**
Active Earth Pressure Coefficient (K_a)	0.33
Passive Earth Pressure Coefficient (K_p)	3
At-Rest Earth Pressure Coefficient (K_o)	0.5
Unit Weight (γ), kN/m^3	21
Submerged Unit Weight (γ), kN/m^3	13

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

The hydrostatic groundwater pressure should be included to the earth pressure distribution wherever the effective unit weight are calculated for earth pressures. If the groundwater level is lowered, the dry unit weight for the soil/bedrock should be calculated full weight, with no hydrostatic groundwater pressure component.

For design purposes, the minimum factor of safety of 1.5 should be calculated.

Underpinning

Due to the close proximity of existing buildings to the proposed buildings, underpinning may be required. Specifically, should the excavation for the proposed buildings extend within the lateral support zone of the footings of the existing buildings, underpinning of the adjacent footings would be required.

Conventional timber lagged pits and concrete underpinning piers are considered to be suitable for this project. The depth of the underpinning, should it be required, will be dependent on the depth of the adjacent foundations relative to the foundation depths of the proposed buildings at the subject site.

6.4 Pipe Bedding and Backfill

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

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

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

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

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.

Infiltration levels are anticipated to be low to moderate through the excavation face, depending on the local groundwater table. The groundwater infiltration is anticipated to be controllable with open sumps and pumps.

If the anticipated pumping volumes exceed 400,000 L/day of ground and/or surface water, a temporary Ministry of the Environment, Conservation and Parks (MECP) permit to take water (PTTW) will be required for this project during the construction phase. A minimum of 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, 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.

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 the 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, and the resistivity is indicative of an 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.

- ❏ Review final grading plan from a geotechnical perspective, once available.
- ❏ Observation of all bearing surfaces prior to the placement of concrete.
- ❏ Sampling and testing of the concrete and fill materials used.
- ❏ Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- ❏ Observation of all subgrades prior to backfilling.
- ❏ 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 made in this report are in accordance with our present understanding of the project. We request that we be permitted to review the grading plan once available and our recommendations when the drawings and specifications are complete.

A geotechnical investigation of this nature is a limited sampling of a site. The recommendations are based on information gathered at the specific test locations and can only be extrapolated to an undefined limited area around the test locations. The extent of the limited area depends on the soil, bedrock and groundwater conditions, as well the history of the site reflecting natural, construction, and other activities. Should any conditions at the site be encountered which differ from those at the test locations, we request notification immediately in order to permit reassessment of our recommendations.

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 Concorde Properties or their agent(s) is not authorized without review by Paterson Group for the applicability of our recommendations to the altered use of the report.

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
June 23-2020
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Report Distribution:

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APPENDIX 1

SOIL PROFILE & 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
890 Byron Avenue and 445 & 463 Sherbourne Ave.
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE June 1, 2020

FILE NO. PG5364

HOLE NO. BH 1

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	64.30
TOPSOIL 0.05							
FILL: Compact, brown silty clay with sand and gravel		SS	1	33	11		
		SS	2	29	11	1	63.30
1.37							
GLACIAL TILL: Compact, grey-brown silty clay with sand, gravel, cobbles and boulders		SS	3	54	16	2	62.30
- grey by 2.7m depth		SS	4	62	18	3	61.30
		SS	5	71	16		
		SS	6	76	50+	4	60.30
		SS	7	4	23	5	59.30
		SS	8	38	12	6	58.30
		SS	9	33	3		
6.71							
End of Borehole							
(GWL @ 1.96m - June 9, 2020)							

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

○ Water Content %

20 40 60 80

Piezometer Construction

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

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
890 Byron Avenue and 445 & 463 Sherbourne Ave.
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE June 1, 2020

FILE NO. PG5364

HOLE NO. BH 2

SOIL DESCRIPTION	Strata depth (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	64.35
TOPSOIL	0.10						
FILL: Comapct, brown silty sand with clay and gravel, trace organics	0.91	SS	1	21	6		
Very stiff, brown **SILTY CLAY,** some sand seams	1.68	SS	2	12	12	1	63.35
GLACIAL TILL: Brown silty clay with sand, gravel, cobbles and boulders - grey by 2.4m depth	6.71	SS	3	100	15	2	62.35
		SS	4	42	10	3	61.35
		SS	5	79	7	4	60.35
		SS	6	58	9	5	59.35
		SS	7	4	5	6	58.35
		SS	8	54	4		
		SS	9	46	5		

End of Borehole

(GWL @ 1.89m - June 9, 2020)

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

○ Water Content %
20 40 60 80

Piezometer Construction

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

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
890 Byron Avenue and 445 & 463 Sherbourne Ave.
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DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE June 1, 2020

FILE NO. PG5364

HOLE NO. BH 3

SOIL DESCRIPTION	Strata depth (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	64.13
TOPSOIL	0.08						
FILL: Compact, brown silty clay, some sand and gravel	1.07	SS	1	12	9		
		SS	2	17	11	1	63.13
Very stiff, brown **SILTY CLAY,** some sand seams	1.98	SS	3	33	9	2	62.13
GLACIAL TILL: Compact, brown silty clay with sand, gravel, cobbles and boulders		SS	4	54	18		
- grey by 3.0m depth		SS	5	8	25	3	61.13
	4.39	SS	6	27	50+	4	60.13

End of Borehole

Practical refusal to augering at 4.39m depth

(GWL @ 2.04m - June 9, 2020)

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

○ Water Content %
20 40 60 80

Piezometer Construction

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

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
890 Byron Avenue and 445 & 463 Sherbourne Ave.
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE June 1, 2020

FILE NO. PG5364

HOLE NO. BH 4

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content % (20 40 60 80)	Piezometer Construction
GROUND SURFACE						0	64.47		
TOPSOIL (0.13)									
Hard to very stiff, brown **SILTY CLAY,** some sand seams. Roots noted to 1.1m depth.		SS	1	21	13				
		SS	2	42	18	1	63.47		
		SS	3	62	10	2	62.47		
(2.29)									
GLACIAL TILL: Loose to compact, brown silty clay with sand, gravel, cobbles and boulders		SS	4	58	5				
		SS	5	50	12	3	61.47		
- grey by 3.0m depth						4	60.47		
		SS	6	8	28	5	59.47		
						6	58.47		
		SS	7	21	9				
(6.71) End of Borehole									

Shear Strength (kPa): 20 40 60 80 100

▲ Undisturbed △ Remoulded

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
890 Byron Avenue and 445 & 463 Sherbourne Ave.
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE June 1, 2020

FILE NO. PG5364

HOLE NO. BH 5

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	64.19
TOPSOIL	0.10						
FILL: Compact, brown silty clay with sand and gravel, trace organics and debris	1.07	SS	1	8	8		
Very stiff, brown **SILTY CLAY,** some sand seams	1.83	SS	2	38	10	1	63.19
GLACIAL TILL: Compact, brown silty clay with sand, gravel, cobbles and boulders - grey by 2.3m depth	2.84	SS	3	25	16	2	62.19
		SS	4	80	50+		

End of Borehole

Practical refusal to augering at 2.84m depth

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

○ Water Content %
20 40 60 80

Piezometer Construction

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

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
890 Byron Avenue and 445 & 463 Sherbourne Ave.
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE June 1, 2020

FILE NO. PG5364

HOLE NO. BH 6

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content % (20 40 60 80)	Piezometer Construction
GROUND SURFACE						0	64.31		
TOSPOIL 0.08									
FILL: Compact, brown silty clay with crushed stone		AU	1						
		SS	2	12	8	1	63.31		
1.22									
Very stiff, brown **SILTY CLAY,** some sand seams		SS	3	29	9	2	62.31		
2.44									
GLACIAL TILL: Loose to compact, brown silty clay with sand, gravel, cobbles and boulders		SS	4	46	7	3	61.31		
		SS	5	62	7				
- grey by 3.0m depth		SS	6	42	7	4	60.31		
		SS	7	25	10	5	59.31		
5.59		SS	8	60	50+				

End of Borehole

Practical refusal to augering at 5.59m depth

(GWL @ 2.84m - June 9, 2020)

Shear Strength (kPa): 20 40 60 80 100

▲ Undisturbed △ Remoulded

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
890 Byron Avenue and 445 & 463 Sherbourne Ave.
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill **DATE** June 1, 2020

FILE NO. PG5364

HOLE NO. BH 7

SOIL DESCRIPTION	Depth of strata change (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	64.18
TOPSOIL	0.13	AU	1				
Very stiff, brown **SILTY CLAY**		SS	2	67	13	1	63.18
	1.83	SS	3	62	10	2	62.18
GLACIAL TILL: Compact, brown silty clay with sand, gravel, cobbles and bouldes		SS	4	62	22	3	61.18
- silty sand with clay by 3.0m depth	3.66	SS	5	54	23		
GLACIAL TILL: Comapct, grey silty clay with sand, gravel, cobbles and boulders		SS	6	58	10	4	60.18
		SS	7	67	9	5	59.18
		SS	8	46	11	6	58.18
	6.48	SS	9	0	50+		

End of Borehole

Practical refusal to augering at 6.48m depth

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

○ Water Content %

20 40 60 80

Piezometer Construction

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

SYMBOLS AND TERMS

SOIL DESCRIPTION

Behavioural properties, such as structure and strength, take precedence over particle gradation in describing soils. Terminology describing soil structure are as follows:

Desiccated	-	having visible signs of weathering by oxidation of clay minerals, shrinkage cracks, etc.
Fissured	-	having cracks, and hence a blocky structure.
Varved	-	composed of regular alternating layers of silt and clay.
Stratified	-	composed of alternating layers of different soil types, e.g. silt and sand or silt and clay.
Well-Graded	-	Having wide range in grain sizes and substantial amounts of all intermediate particle sizes (see Grain Size Distribution).
Uniformly-Graded	-	Predominantly of one grain size (see Grain Size Distribution).

The standard terminology to describe the relative strength of cohesionless soils is the compactness condition, usually inferred from the results of the Standard Penetration Test (SPT) ‘N’ value. The SPT N value is the number of blows of a 63.5 kg hammer, falling 760 mm, required to drive a 51 mm O.D. split spoon sampler 300 mm into the soil after an initial penetration of 150 mm. An SPT N value of “P” denotes that the split-spoon sampler was pushed 300 mm into the soil without the use of a falling hammer.

Compactness Condition	‘N’ Value	Relative Density %
Very Loose	<4	<15
Loose	4-10	15-35
Compact	10-30	35-65
Dense	30-50	65-85
Very Dense	>50	>85

The standard terminology to describe the strength of cohesive soils is the consistency, which is based on the undisturbed undrained shear strength as measured by the in situ or laboratory shear vane tests, unconfined compression tests, or occasionally by the Standard Penetration Test (SPT). Note that the typical correlations of undrained shear strength to SPT N value (tabulated below) tend to underestimate the consistency for sensitive silty clays, so Paterson reviews the applicable split spoon samples in the laboratory to provide a more representative consistency value based on tactile examination.

Consistency	Undrained Shear Strength (kPa)	‘N’ Value
Very Soft	<12	<2
Soft	12-25	2-4
Firm	25-50	4-8
Stiff	50-100	8-15
Very Stiff	100-200	15-30
Hard	>200	>30

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

Cohesive soils can also be classified according to their “sensitivity”. The sensitivity, S_t, is the ratio between the undisturbed undrained shear strength and the remoulded undrained shear strength of the soil. The classes of sensitivity may be defined as follows:

Low Sensitivity:	$S_t < 2$
Medium Sensitivity:	$2 < S_t < 4$
Sensitive:	$4 < S_t < 8$
Extra Sensitive:	$8 < S_t < 16$
Quick Clay:	$S_t > 16$

ROCK DESCRIPTION

The structural description of the bedrock mass is based on the Rock Quality Designation (RQD).

The RQD classification is based on a modified core recovery percentage in which all pieces of sound core over 100 mm long are counted as recovery. The smaller pieces are considered to be a result of closely-spaced discontinuities (resulting from shearing, jointing, faulting, or weathering) in the rock mass and are not counted. RQD is ideally determined from NQ or larger size core. However, it can be used on smaller core sizes, such as BQ, if the bulk of the fractures caused by drilling stresses (called “mechanical breaks”) are easily distinguishable from the normal in situ fractures.

RQD %	**ROCK QUALITY**
90-100	Excellent, intact, very sound
75-90	Good, massive, moderately jointed or sound
50-75	Fair, blocky and seamy, fractured
25-50	Poor, shattered and very seamy or blocky, severely fractured
0-25	Very poor, crushed, very severely fractured

SAMPLE TYPES

SS	-	Split spoon sample (obtained in conjunction with the performing of the Standard Penetration Test (SPT))
TW	-	Thin wall tube or Shelby tube, generally recovered using a piston sampler
G	-	"Grab" sample from test pit or surface materials
AU	-	Auger sample or bulk sample
WS	-	Wash sample
RC	-	Rock core sample (Core bit size BQ, NQ, HQ, etc.). Rock core samples are obtained with the use of standard diamond drilling bits.

SYMBOLS AND TERMS (continued)

PLASTICITY LIMITS AND GRAIN SIZE DISTRIBUTION

WC%	-	Natural water content or water content of sample, %
LL	-	Liquid Limit, % (water content above which soil behaves as a liquid)
PL	-	Plastic Limit, % (water content above which soil behaves plastically)
PI	-	Plasticity Index, % (difference between LL and PL)
Dxx	-	Grain size at which xx% of the soil, by weight, is of finer grain sizes These grain size descriptions are not used below 0.075 mm grain size
D10	-	Grain size at which 10% of the soil is finer (effective grain size)
D60	-	Grain size at which 60% of the soil is finer
Cc	-	Concavity coefficient = $(D30)^2 / (D10 \times D60)$
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

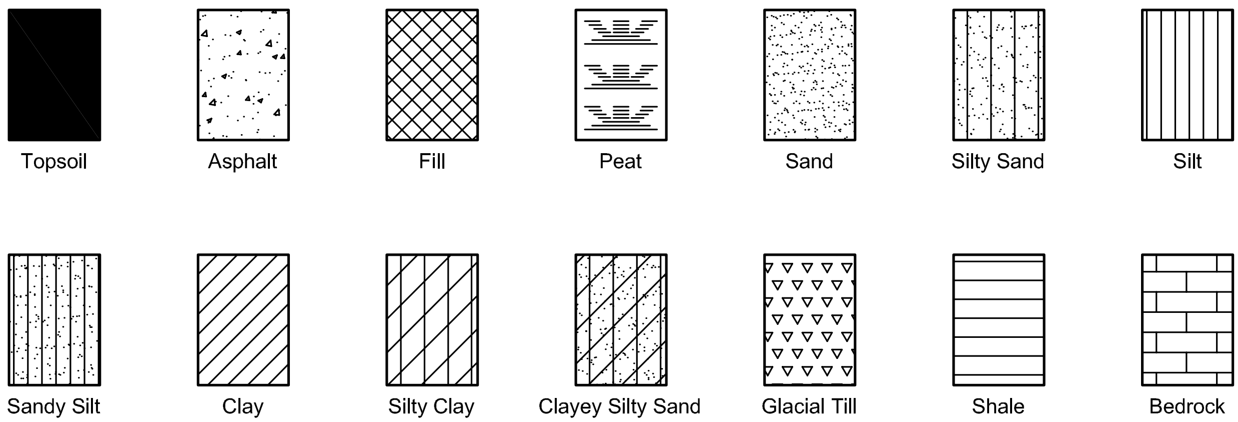
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void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

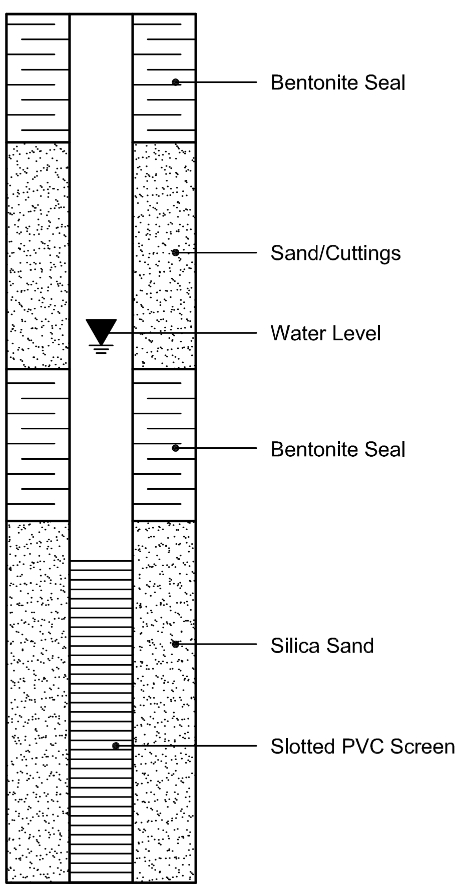
SYMBOLS AND TERMS (continued)

STRATA PLOT

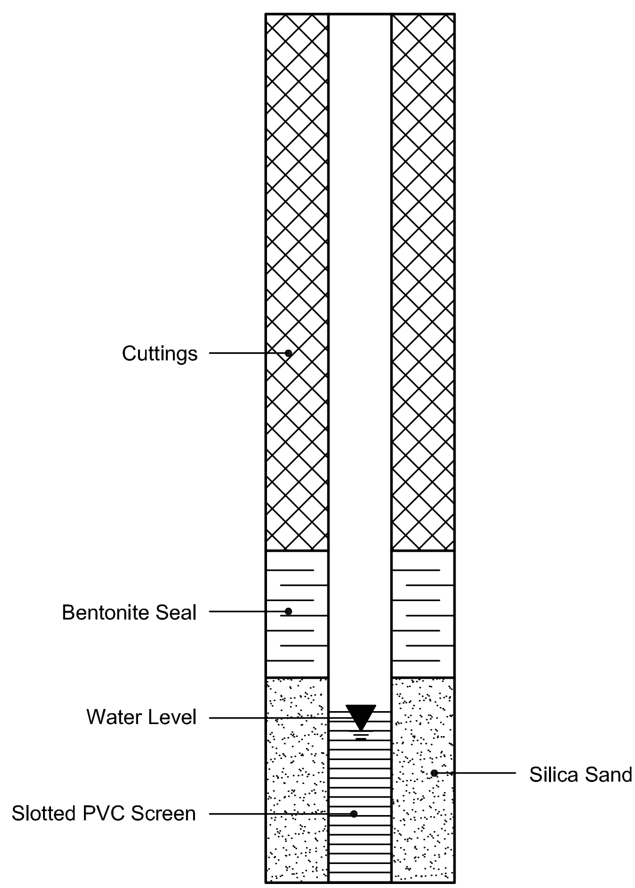


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 30212

Report Date: 08-Jun-2020

Order Date: 2-Jun-2020

Project Description: PG5364

	Client ID:	BH6 SS5 10'-12'	-	-	-
	Sample Date:	01-Jun-20 13:00	-	-	-
	Sample ID:	2023209-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	91.9	-	-	-
General Inorganics					
pH	0.05 pH Units	8.16	-	-	-
Resistivity	0.10 Ohm.m	35.2	-	-	-
Anions					
Chloride	5 ug/g dry	105	-	-	-
Sulphate	5 ug/g dry	41	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING PG5364-1 - TEST HOLE LOCATION PLAN

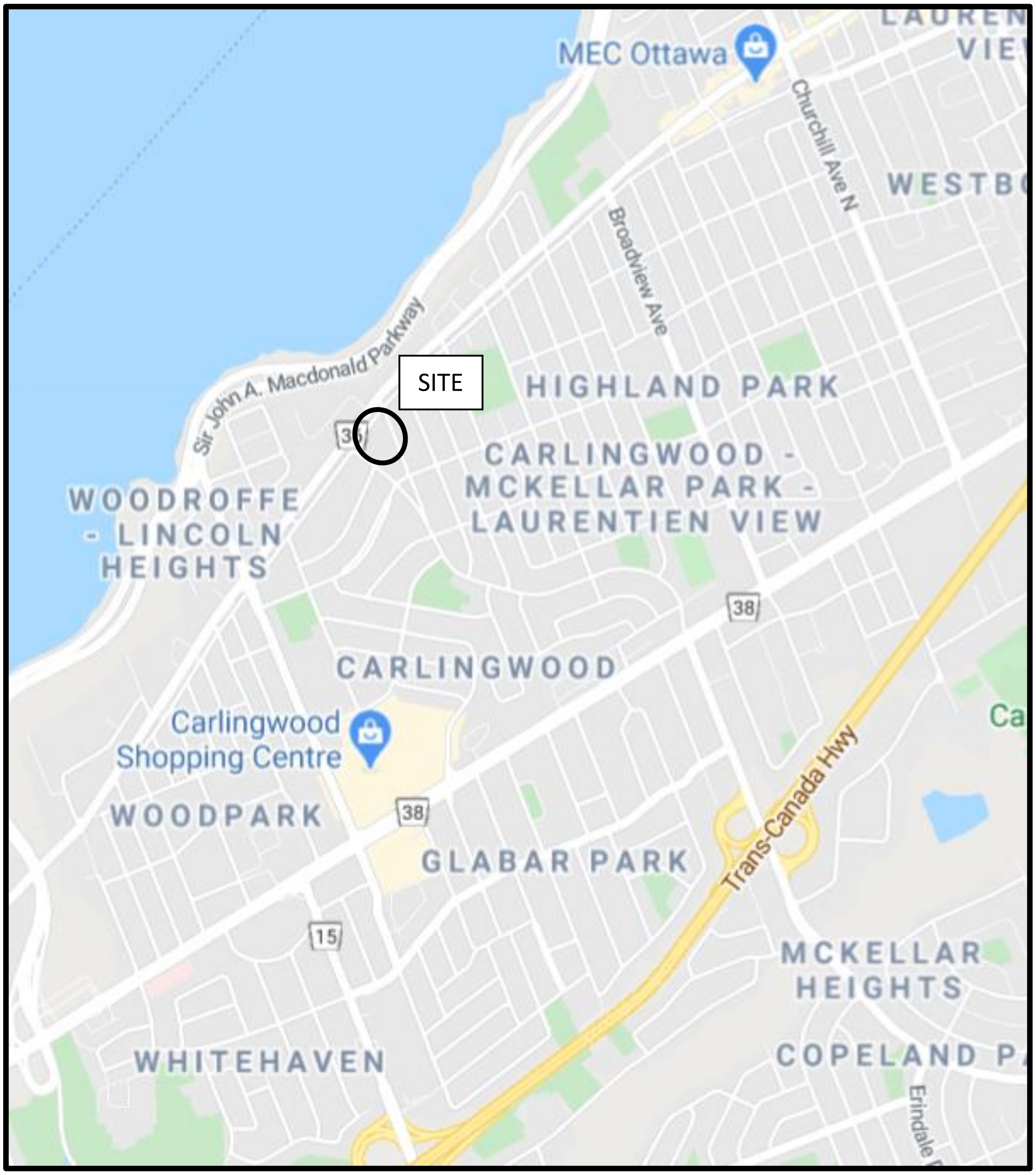
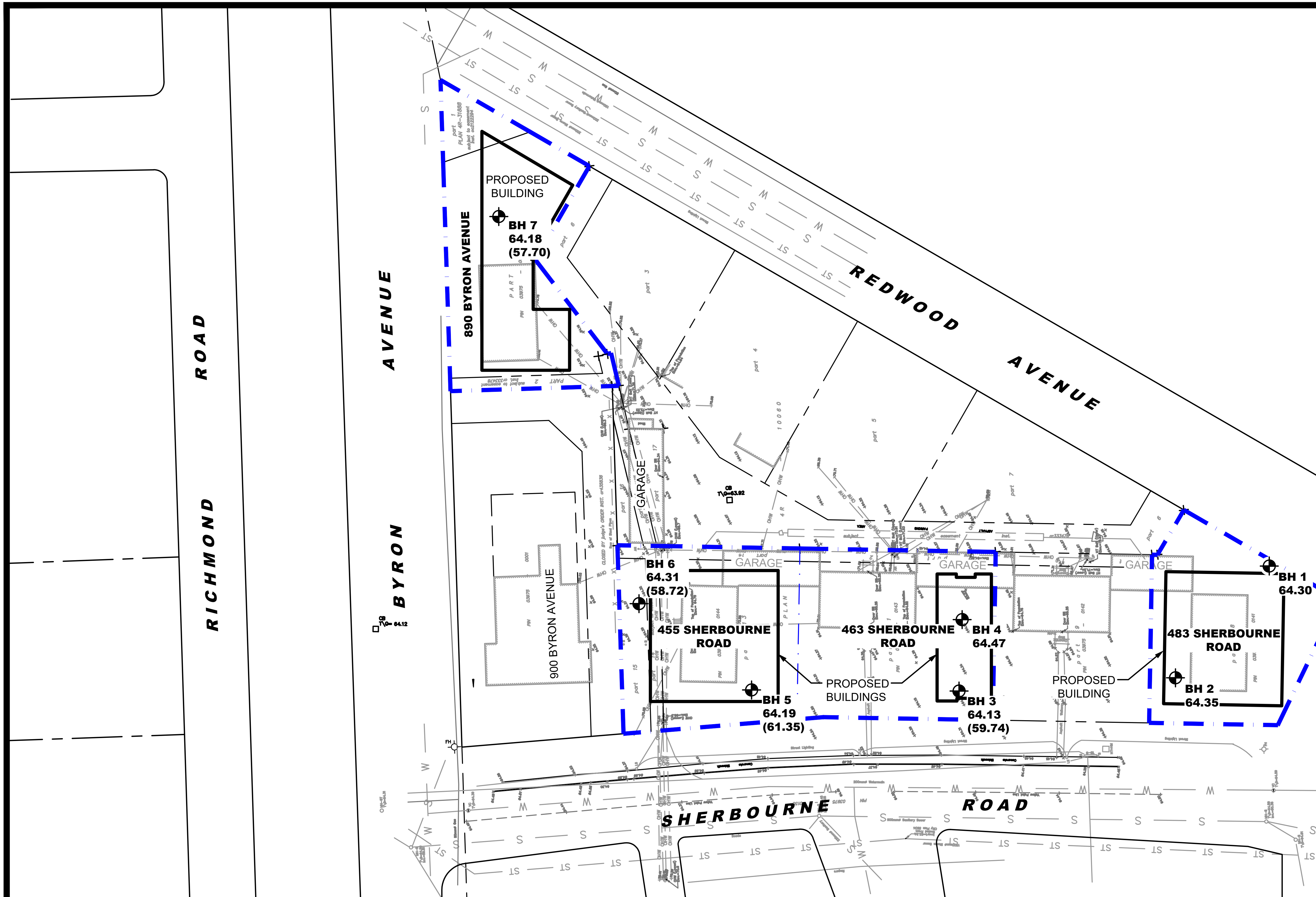


FIGURE 1

KEY PLAN

patersongroup



LEGEND:

⊕	BOREHOLE LOCATION
64.13	GROUND SURFACE ELEVATION (m)
(59.74)	PRACTICAL REFUSAL TO AUGERING ELEVATION (m)

BASE PLAN PROVIDED BY ANNIS, O'SULLIVAN, VOLLEBEKK LTD.

CONCEPTUAL PLAN PROVIDED BY S. J. LAWRENCE ARCHITECT INCORPORATED

ALL ELEVATIONS ARE REFERENCED TO A GEODETIC DATUM

SCALE: 1:750

0 10 20 30 50m

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NO.	REVISIONS	DATE	INITIAL

CONCORDE PROPERTIES
GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL BUILDINGS
OTTAWA, 890 BYRON AVENUE AND 455, 463 AND 483 SHERBOURNE ROAD ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:750	Date: 06/2020
Drawn by: YA	Report No.: PG5364-1
Checked by: DP	Dwg. No.: PG5364-1
Approved by: SD	Revision No.:

p:\autocad drawings\geotechnical\pg53xx\pg5364\pg5364-1-test hole location plan.dwg