Geotechnical Engineering

Environmental Engineering

Hydrogeology

Geological Engineering

Materials Testing

Building Science

Archaeological Services

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Geotechnical Investigation

Proposed Multi-Storey Building
Blocks 2 & 3 - McGarry Terrace
Ottawa, Ontario

Prepared For

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Report PG4219-1 Rev.01

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1.0 Introduction

Paterson Group (Paterson) was commissioned by Canadian Rental Development Services Inc. to conduct a geotechnical investigation for the proposed multi-storey building to be located along McGarry Terrace in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objectives of the investigation were to:

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

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

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

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

2.0 Proposed Project

Based on available design information, it is our understanding that the proposed development will consist of a multi-storey building with two underground parking levels along with landscaped areas and access lanes.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the geotechncial investigation was carried out on August 4 and 8, 2017. During that time, a total of six (6) boreholes were advanced to a maximum depth of 10 m below existing ground surface. The borehole locations were determined in the field by Paterson personnel and distributed in a manner to provide general coverage of the proposed development taking into consideration existing site features and underground services. The locations of the boreholes are presented in Drawing PG4219-1 - Test Hole Location Plan included in Appendix 2.

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

Sampling and In Situ Testing

Soil samples from the boreholes were recovered from the auger flights, using a 50 mm diameter split-spoon sampler or using 47.6 mm inside diameter coring equipment. All soil samples were initially classified on site, placed in sealed plastic bags and transported to our laboratory. The depths at which the auger, split spoon and rock core samples were recovered from the test holes are shown as, AU, SS and RC, respectively, on the Soil Profile and Test Data sheets presented in Appendix 1.

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

Rock coring was carried out in each of the boreholes to confirm the presence or absence of bedrock where practical refusal to augering was encountered.

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

Groundwater

Flexible PVC standpipes were installed in BH 1 to BH 4 to permit monitoring of the groundwater levels subsequent to the completion of the sampling program.

3.2 Field Survey

The test hole locations were determined in the field by Paterson personnel with consideration of underground utilities and existing site features. The ground surface elevations are referenced to a temporary benchmark (TBM), consisting of the top spindle of the fire hydrant located at the south end of McGarry Terrace. A geodetic elevation of 103.10 m was provided for the TBM. The location of the TBM, boreholes and ground surface elevation at each borehole location are presented on Drawing PG4219-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

All 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.

3.4 Analytical Testing

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

4.0 Observations

4.1 Surface Conditions

Currently, the subject site is partially treed with grass covered areas and an asphalt laneway running north-south across the site. Two residential dwellings are located south of the site and a residential building complex is located east of the site. The ground surface gradually slopes from southwest to northeast.

4.2 Subsurface Profile

Generally, the subsoil profile encountered at the borehole locations consists of topsoil overlying a native, dense to very dense glacial till deposit. The fine soil matrix of the glacial till deposit was noted to consist of a brown silty sand, some clay and gravel. Oversized boulders were encountered throughout the glacial till deposit. Rock coring was required at all of the boreholes, except BH 5, to extend the boreholes through the oversized boulders encountered. Bedrock was not encountered in any of the boreholes; all boreholes were terminated at a 10 m depth within the glacial till deposit.

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

Based on available geological mapping, the subject site is located in an area where bedrock consists of interbedded sandstone and sandy dolomite of the March Formation, and is expected at depths between 10 to 15 m.

4.3 Groundwater

Groundwater levels were measured in the PVC standpipes installed in the boreholes upon completion of the sampling program. The GWL readings are presented in Table 1. 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 5 to 6 m depth.

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

Table 1 - Summary of Groundwater Levels				
Borehole Number	**Ground Surface Elevation (m)**	**Measured Groundwater Level (m)**		**Recording Date**
		Depth	**Elevation**	
BH 1	100.65	1.27	99.38	August 24, 2017
BH 2	102.59	1.80	100.79	August 24, 2017
BH 3	102.26	3.80	98.46	August 24, 2017
BH 4	101.30	3.32	97.98	August 24, 2017

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered satisfactory for the proposed building. It is expected that the foundation will consist of conventional pad and strip footings for the proposed building. Alternatively, a raft foundation could be used if bearing resistance values are not sufficient to design the building with conventional footings.

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

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and asphalt should be removed from within the perimeter of the proposed building and other settlement sensitive structures.

Fill Placement

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

Non-specified existing fill, along with site-excavated soil, can be used as general landscaping fill where settlement of the ground surface is of minor concern. This material should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids. If this material is to be used to build up the subgrade level for areas to be paved, it should be compacted in thin lifts to at least 95% of the material's SPMDD. Non-specified existing fill and site-excavated soils are not suitable for use as backfill against foundation walls unless used in conjunction with a composite drainage membrane.

5.3 Foundation Design

Several foundation options have been considered for the proposed multi-storey building which are dependent on the design loading requirements and foundation depth. The options are further discussed below.

Conventional Shallow Foundation

Footings placed on an undisturbed, dense to very dense glacial till bearing surface can be designed using a bearing resistance value at SLS of **325 kPa** and a factored bearing resistance value at ULS of **500 kPa**. A geotechnical resistance factor of 0.5 was applied to the bearing resistance value at ULS.

The bearing resistance values are provided on the assumption that the footings will be placed on bearing surfaces consisting of an undisturbed soil. An undisturbed, bearing surface should be free of fill, topsoil, surface water and deleterious materials, such as loose, frozen or disturbed soil prior to placing concrete.

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 glacial till bearing medium when a plane extending down and out from the bottom edge of the footing at a minimum of 1.5H:1V, passes only through in situ soil or engineered fill of the same or higher capacity as the soil.

Raft Foundation

Alternatively, consideration can be given to a raft foundation if the building loads exceed the bearing resistance values provided for a conventional shallow footing foundation. The following parameters may be used for raft design.

The amount of settlement of the raft slab will be dependent on the sustained raft contact pressure. The bearing resistance value at SLS (contact pressure) of **350 kPa** can be used for design purposes. The loading conditions for the contact pressure are based on sustained loads, that are generally taken to be 100% Dead Load and 50% Live Load. The contact pressure provided considers the stress relief associated with the soil removal associated with one underground parking level. The factored bearing resistance (contact pressure) at ULS can be taken as **600 kPa**.

A geotechnical resistance factor of 0.5 was applied to the bearing resistance value at ULS.

Based on the following assumptions for the raft foundation, the proposed building can be designed using the above parameters with a total and differential settlement of 25 and 15 mm, respectively. Typically, the modulus of subgrade reaction for a very dense glacial till can be taken as **50 MPa/m** for design of the raft foundation.

5.4 Design for Earthquakes

Foundations constructed at the subject site can be designed using a seismic site response **Class C** as defined in the Ontario Building Code 2012 (OBC 2012; Table 4.1.8.4.A). The soils underlying the site are not susceptible to liquefaction.

5.5 Basement Slab

The native soil or approved fill surface will be considered to be an acceptable subgrade surface on which to commence backfilling for floor slab construction. Any soft areas should be removed and backfilled with appropriate backfill material. OPSS Granular B Type II is recommended for backfilling below the floor slab. It is recommended that the upper 200 mm of sub-slab fill consist of 19 mm clear crushed stone for the basement floor slab used for finished space.

5.6 Basement Wall

There are several combinations of backfill materials and retained soils that could be applicable for the basement walls of the subject structure. However, the conditions can be well-represented by assuming the retained soil consists of a material with an angle of internal friction 30 degrees and a bulk (drained) unit weight of 20 kN/m^3.

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

Lateral Earth Pressures

The static horizontal earth pressure (p_o) can be calculated using a triangular earth pressure distribution equal to $K_o \cdot \gamma \cdot H$ where:

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

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

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

Seismic Earth Pressures

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

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

a_c = $(1.45\text{-}a_{max}/g)a_{max}$
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. Note that the vertical seismic coefficient is assumed to be zero.

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

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

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

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

5.7 Pavement Structure

Car only parking and heavy truck parking areas, and access lanes may be required at this site. 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	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 or fill	

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

If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type II material. The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 98% of the material's SPMDD using suitable vibratory equipment.

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 all the proposed structures. The system should consist of a 150 mm diameter perforated corrugated plastic pipe, surrounded on all sides by 150 mm of 19 mm clear crushed stone, placed at the footing level around the exterior perimeter of the structure. 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. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should be used for this purpose. A composite drainage system should be applied to the exterior of the building foundation walls in order to minimize the risk of groundwater infiltration from the backfill materials. A waterproofing system should be provided to the elevator pit (pit bottom and walls).

6.2 Protection of Footings Against Frost Action

Perimeter footings of heated structures are required to be insulated against the deleterious effect of frost action. A minimum of 1.5 m of soil cover alone, or a minimum of 0.6 m of soil cover, in conjunction with foundation insulation, should be provided in this regard.

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

6.3 Excavation Side Slopes

At this site, temporary shoring may be required to complete the required excavations. It should be noted that installation of a temporary shoring system may be difficult due to the presence of oversized boulders within the glacial till deposit.

Excavation Side Slopes

The subsoil at this site is considered to be mainly a Type 2 or 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects. 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 the groundwater level.

Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should be kept away from the excavation sides.

Slopes in excess of 3 m in height should be periodically inspected by the geotechnical consultant in order to detect if the slopes are exhibiting signs of distress.

It is recommended that a trench box be used at all times to protect personnel working in trenches with steep or vertical sides. It is expected that services will be installed by “cut and cover” methods and excavations will not be left open for extended periods of time.

Temporary Shoring

It should be noted that the observed site conditions may unsuitable for temporary shoring. Bouldery conditions such as those observed can lead to creation of voids and other unstable conditions during installation as boulders shift within the fine soil matrix. Furthermore, it may be difficult to develop the required anchor strength in soil conditions due to variations in the soil.

If a temporary shoring system is considered, the design and implementation of these temporary systems will be the responsibility of the excavation contractor and their design team. Inspections and approval of the temporary system will also be the responsibility of the designer. Geotechnical information provided below is to assist the designer in completing a suitable and safe shoring system. The designer should take into account the potential for a fully saturated condition following a significant precipitation event. Any changes to the approved shoring design system should be reported immediately to the owner’s representative prior to implementation.

For design purposes, the temporary system may consist of soldier pile and lagging system or interlocking steel sheet piling. Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be added to the earth pressures described below. These systems can be cantilevered, anchored or braced.

The earth pressures acting on the shoring system may be calculated using the following parameters.

Table 4 - Soil Parameters for Shoring System Design

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	20
Submerged Unit Weight (γ), kN/m^3	13

Generally, it is expected that the shoring systems will be provided with tie-back rock anchors to ensure their stability. It is further recommended that the toe of the shoring be adequately supported to resist toe failure.

The geotechnical design of grouted rock anchors in sedimentary bedrock is based upon two possible failure modes. The anchor can fail either by shear failure along the grout/rock interface or by pullout of a 60 to 90 degree cone of rock with the apex of the cone near the middle of the bonded length of the anchor.

The anchor derives its capacity from the bonded portion, or fixed anchor length, at the base of the anchor. An unbonded portion, or free anchor length, is also usually provided between the rock surface and the start of the bonded length. A factored tensile grout to rock bond resistance value at ULS of **1.0 MPa**, incorporating a resistance factor of 0.3, can be used. A minimum grout strength of 40 MPa is recommended.

It is recommended that the anchor drill hole diameter be within 1.5 to 2 times the rock anchor tendon diameter and the anchor drill holes be inspected by geotechnical personnel and should be flushed clean prior to grouting. The use of a grout tube to place grout from the bottom up in the anchor holes is further recommended.

The geotechnical capacity of each rock anchor should be proof tested at the time of construction. More information on testing can be provided upon request. Compressive strength testing is recommended to be completed for the rock anchor grout. A set of grout cubes should be tested for each day grout is prepared.

Soldier Pile and Lagging System

The active earth pressure acting on a soldier pile and lagging shoring system can be calculated using a rectangular earth pressure distribution with a maximum pressure of 0.65 K γ H for strutted or anchored shoring or a triangular earth pressure distribution with a maximum value of K γ H for a cantilever shoring system. H is the height of the excavation.

The active earth pressure should be used where wall movements are permissible while the at-rest pressure should be used if no movement is permissible.

The total unit weight should be used above the groundwater level while the submerged unit weight should be used below the groundwater level.

6.4 Pipe Bedding and Backfill

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

At least 150 mm of OPSS Granular A 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 site materials above the cover material if the operations are carried out in dry weather conditions.

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) and above the cover material should match the soils exposed at the trench walls to minimize differential frost heaving. The trench backfill should be placed in maximum 225 mm thick loose lifts and compacted to a minimum of 95% of the material standard Proctor maximum dry density.

6.5 Groundwater Control

Groundwater Control for Building Construction

Based on the existing groundwater level, it is anticipated that groundwater infiltration into the excavations should be low and controllable using open sumps. Pumping from open sumps should be sufficient to control the groundwater influx through the sides of shallow excavations and due to precipitation events.

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 and Climate Change (MOECC) permit to take water (PTTW) may be required for this project if more than 400,000 L/day of ground and/or surface water is to be pumped during the construction phase. A minimum 4 to 5 months should be allowed for completion of the PTTW application package and issuance of the permit by the MOECC.

For typical ground or surface water volumes, being pumped during the construction phase, between 50,000 to 400,000 L/day, it is required to register on the Environmental Activity and Sector Registry (EASR). A minimum of two to four weeks should be allotted for completion of the EASR registration and the Water Taking and Discharge Plan to be prepared by a Qualified Person as stipulated under O.Reg. 63/16. If a project qualifies for a PTTW based upon anticipated conditions, an EASR will not be allowed as a temporary dewatering measure while awaiting the MOECC review of the PTTW application.

Impacts on Neighbouring Properties

Based on the existing 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 neighbouring buildings and minimal zone impacted by the groundwater lowering, the proposed development will not negatively impact the neighbouring structures. It should be noted that no issues are expected with respect to groundwater lowering that would cause long term adverse effects to adjacent structures surrounding the proposed building.

6.6 Winter Construction

Precautions must be taken if winter construction is considered for this project.

The subsoil conditions at this site mostly consist of frost susceptible materials. In 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.

Precaution must be taken where excavations are carried in proximity of existing structures which may be adversely affected due to the freezing conditions. In particular, it should be recognized that where a shoring system is used, the soil behind the shoring system will be subjected to freezing conditions and could result in heaving of the structure(s) placed within or above frozen soil. Provisions should be made in the contract document to protect the walls of the excavations from freezing, if applicable.

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, and the resistivity is indicative of a non-aggressive to slightly aggressive corrosive environment.

7.0 Recommendations

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

- ❏ 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 request, following the completion of a satisfactory materials testing and observation program by the geotechnical consultant.

8.0 Statement of Limitations

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

A geotechnical investigation is a limited sampling of a site. Should any conditions at the site be encountered which differ from those at the test locations, we request immediate notification 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 Canadian Rental Development Services Inc. or their agents is not authorized without review by Paterson for the applicability of our recommendations to the altered use of the report.

Paterson Group Inc.

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Report Distribution

- ❏ Canadian Rental Development Services Inc. (3 copies)
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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. Residential Development - Blocks 2 and 3
McGarry Terrace, Ottawa, Ontario

DATUM TBM - Top flange of fire hydrant located near the southwest corner of subject site. Geodetic elevation = 103.10m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE August 4, 2017

FILE NO. PG4219

HOLE NO. BH 1

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	100.65
TOPSOIL 0.25							
		SS	1	80	50+	1	99.65
		SS	2	75	68	2	98.65
GLACIAL TILL: Very dense, brown silty sand with gravel, cobbles and boulders, some clay		SS	3	100	50+	3	97.65
		RC	1	50		4	96.65
- grey by 4.3m depth		RC	2	17		5	95.65
		RC	3	20		6	94.65
		RC	4	15		7	93.65
						8	92.65
		RC	5			9	91.65
10.36						10	90.65
End of Borehole							
(GWL @ 1.27m - August 23, 2017)							

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
Prop. Residential Development - Blocks 2 and 3
McGarry Terrace, Ottawa, Ontario

DATUM TBM - Top flange of fire hydrant located near the southwest corner of subject site. Geodetic elevation = 103.10m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE August 4, 2017

FILE NO. PG4219

HOLE NO. BH 2

SOIL DESCRIPTION	Strata depth (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	102.59
TOPSOIL	0.20						
GLACIAL TILL: Compact to very dense, brown silty sand with gravel, cobbles and boulders, trace clay		SS	1	83	24	1	101.59
		SS	2	100	60	2	100.59
		SS	3	75	112		
		SS	4	0	50+	3	99.59
		RC	1	54		4	98.59
		RC	2	15		5	97.59
						6	96.59
		RC	3	13		7	95.59
		RC	4	15		8	94.59
						9	93.59
		RC	5	4		10	92.59
End of Borehole	10.34						

(GWL @ 1.80m - August 23, 2017)

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

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - Blocks 2 and 3
McGarry Terrace, Ottawa, Ontario

DATUM TBM - Top flange of fire hydrant located near the southwest corner of subject site. Geodetic elevation = 103.10m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE August 4, 2017

FILE NO. PG4219

HOLE NO. BH 3

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	102.26
TOPSOIL 0.25						
	SS	1	67	11	1	101.26
	SS	2	100	24	2	100.26
	SS	3	100	78	3	99.26
	SS	4	100	50+		
GLACIAL TILL: Compact to very dense, brown silty sand with gravel, cobbles and boulders, some clay	SS	5	83	83	4	98.26
	SS	6	75	47	5	97.26
- grey by 5.0m depth	SS	7	42	68		
	SS	8	0	50+	6	96.26
	RC	1	25		7	95.26
	RC	2				
	SS	9		50+	8	94.26
	RC	3	71			
	SS	10		50+	9	93.26
	RC	4			10	92.26
10.36						
End of Borehole						
(GWL @ 3.80m - August 23, 2017)						

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

○ Water Content %

20 40 60 80

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

Piezometer Construction

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - Blocks 2 and 3
McGarry Terrace, Ottawa, Ontario

DATUM TBM - Top flange of fire hydrant located near the southwest corner of subject site. Geodetic elevation = 103.10m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE August 8, 2017

FILE NO. PG4219

HOLE NO. BH 4

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	101.30
TOPSOIL 0.20							
GLACIAL TILL: Compact to very dense, brown silty sand with gravel, cobbles and boulders, some clay		SS	1	83	19	1	100.30
		SS	2	92	72	2	99.30
		SS	3	86	54	3	98.30
		SS	4	100	50+		
		SS	5	80	50+	4	97.30
		SS	6	89	50+	5	96.30
		SS	7	100	77	6	95.30
		SS	8	95	67		
- grey-brown by 6.7m depth		SS	9	90	68	7	94.30
						8	93.30
		RC	1	21		9	92.30
10.36						10	91.30
End of Borehole							
(GWL @ 3.32m - August 23, 2017)							

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
Prop. Residential Development - Blocks 2 and 3
McGarry Terrace, Ottawa, Ontario

DATUM TBM - Top flange of fire hydrant located near the southwest corner of subject site. Geodetic elevation = 103.10m.

REMARKS

BORINGS BY CME 55 Power Auger **DATE** August 8, 2017

FILE NO. PG4219

HOLE NO. BH 5

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	102.98		
TOPSOIL 0.15									
		SS	1	75	48	1	101.98		
		SS	2	44	50+				
		SS	3	90	50+	2	100.98		
		SS	4	44	50+	3	99.98		
GLACIAL TILL: Dense to very dense, brown silty sand with clay, gravel, cobbles and boulders		SS	5	80	50+	4	98.98		
		SS	6	86	50+	5	97.98		
- grey by 5.3m depth		SS	7	100	67				
		SS	8	100	68	6	96.98		
		SS	9	91	50+	7	95.98		
		SS	10	75	84	8	94.98		
		SS	11	50	50+	9	93.98		
9.75		SS	12	50	39				▽
End of Borehole									
(GWL @ 9.4m depth based on field observations)									

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

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - Blocks 2 and 3
McGarry Terrace, Ottawa, Ontario

DATUM TBM - Top flange of fire hydrant located near the southwest corner of subject site. Geodetic elevation = 103.10m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE August 8, 2017

FILE NO. PG4219

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	103.05		
TOPSOIL 0.20									
		SS	1	100	48	1	102.05		
		SS	2		50				
						2	101.05		
		SS	3	80	50+				
						3	100.05		
		SS	4	100	50+				
						4	99.05		
GLACIAL TILL: Dense to very dense, brown silty sand with clay, gravel, cobbles and boulders		SS	5	75	71				
						5	98.05		
		RC	1	10					
						6	97.05		
						7	96.05		
						8	95.05		
		RC	2						
						9	94.05		
						10	93.05		
10.13									
End of Borehole (BH dry upon completion)									

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

SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

The standard terminology to describe the strength of cohesionless soils is the relative density, usually inferred from the results of the Standard Penetration Test (SPT) ‘N’ value. The SPT N value is the number of blows of a 63.5 kg hammer, falling 760 mm, required to drive a 51 mm O.D. split spoon sampler 300 mm into the soil after an initial penetration of 150 mm.

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

The standard terminology to describe the strength of cohesive soils is the consistency, which is based on the undisturbed undrained shear strength as measured by the in situ or laboratory vane tests, penetrometer tests, unconfined compression tests, or occasionally by Standard Penetration Tests.

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

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 x 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

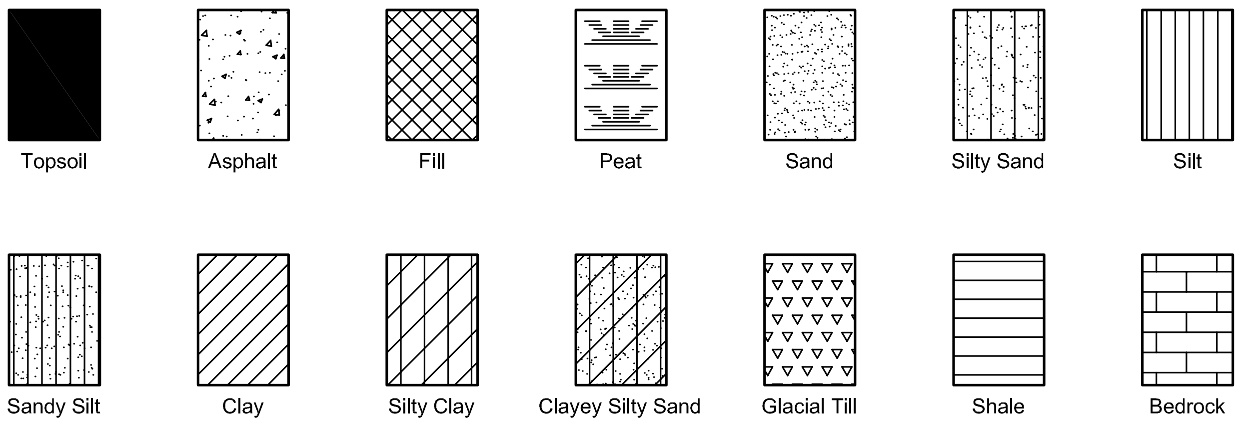
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void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

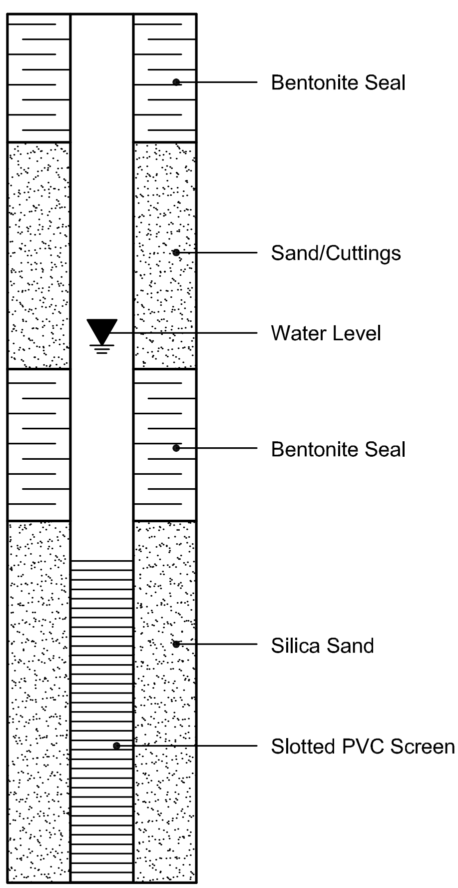
SYMBOLS AND TERMS (continued)

STRATA PLOT

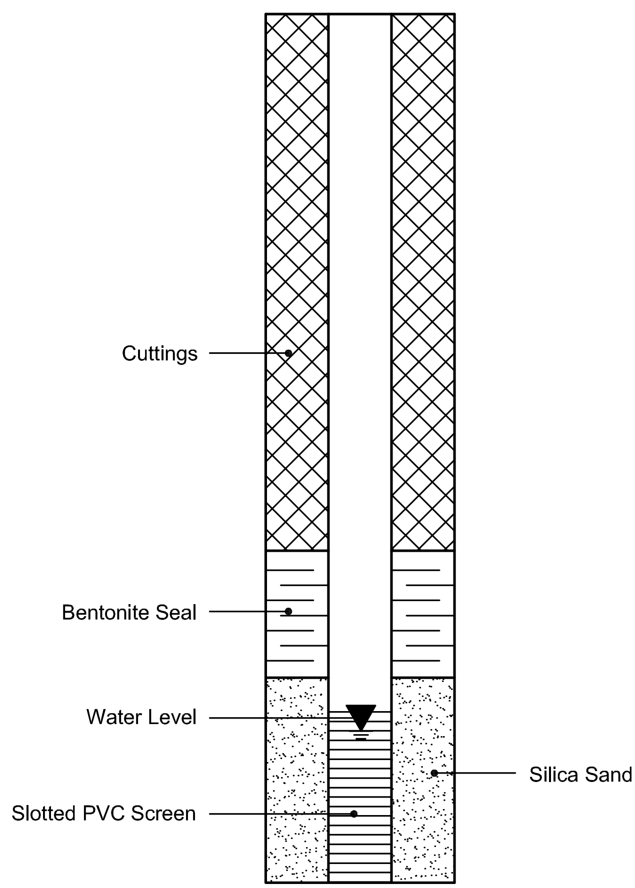


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 22649

Report Date: 15-Sep-2017

Order Date: 11-Sep-2017

Project Description: PG4219

	Client ID:	BH3 SS4	-	-	-
	Sample Date:	04-Aug-17	-	-	-
	Sample ID:	1737110-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	93.5	-	-	-
General Inorganics					
pH	0.05 pH Units	8.12	-	-	-
Resistivity	0.10 Ohm.m	76.3	-	-	-
Anions					
Chloride	5 ug/g dry	11 [1]	-	-	-
Sulphate	5 ug/g dry	40 [1]	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING PG4219-1 - TEST HOLE LOCATION PLAN

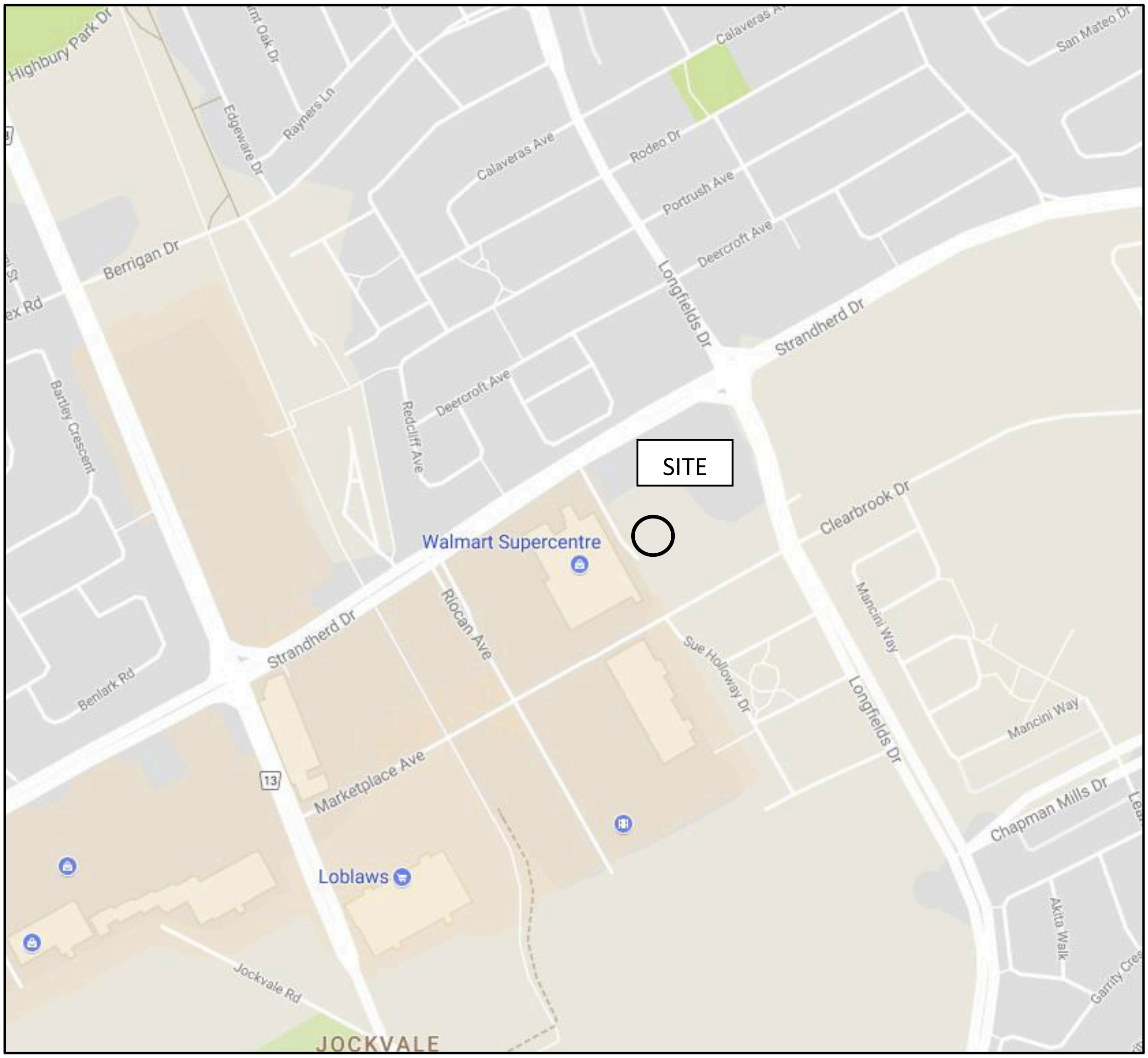
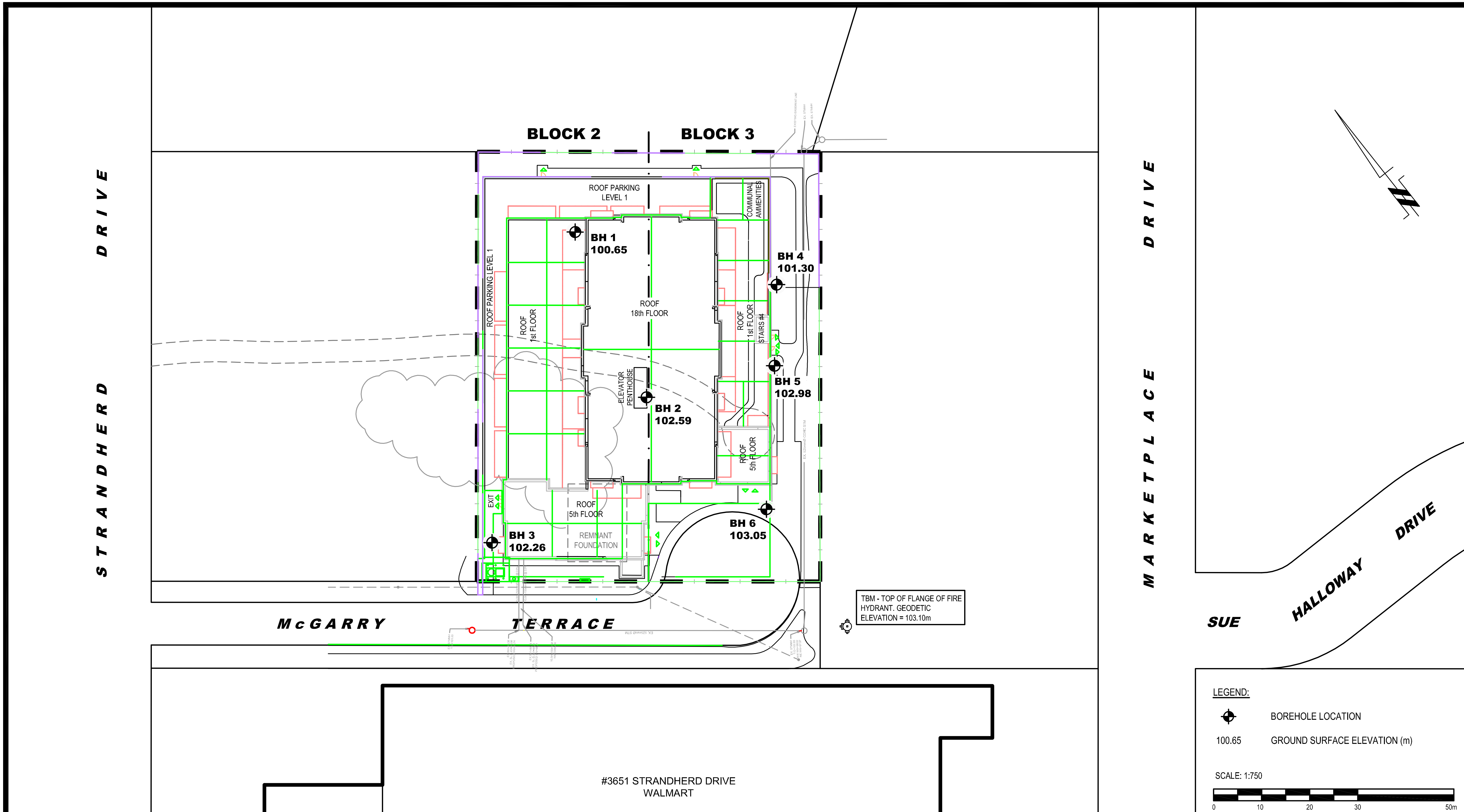


FIGURE 1

KEY PLAN



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NO.	REVISIONS	DATE	INITIAL
1	BASE PLAN UPDATED	21/12/2017	FA

CANADIAN RENTAL DEVELOPMENT SERVICES INC.

GEOTECHNICAL INVESTIGATION
PROP. RESIDENTIAL DEVELOPMENT - BLOCK 2 & 3 McGARRY TERRACE
OTTAWA, ONTARIO

Title:
TEST HOLE LOCATION PLAN

Scale: 1:750	Date: 09/2017
Drawn by: MPG	Report No.: PG4219-1
Checked by: NC	Dwg. No.: PG4219-1
Approved by: DJG	Revision No.: 1