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Geotechnical Engineering

Environmental Engineering

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Materials Testing

Building Science

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Geotechnical Investigation
Proposed Multi-Storey Building
99 Fifth Avenue
Ottawa, Ontario

Prepared For

Minto Properties

November 14, 2017

Report: PG4240-1

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1.0 Introduction

Paterson Group (Paterson) was commissioned by Minto Properties to conduct a geotechnical investigation for a proposed multi-storey building to be located at 99 Fifth Avenue in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2).

The objectives of the current investigation were to:

- Determine the subsurface soil and groundwater conditions 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. This report contains the findings and recommendations pertaining to the design and construction of the subject development as 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 for this geotechnical investigation.

2.0 Proposed Project

Based on current project details, the proposed project consists of one residential condominium tower over a 2 storey underground structure. The 2 storey commercial building along Bank Street will remain. The new building will be an 8 storey building.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current investigation was completed on September 9, 10 and 28, 2017. At that time, 3 boreholes were completed within the underground parking area of the subject site. The borehole locations were distributed in a manner to provide general coverage of the subject site. The location of the boreholes are shown on Drawing PG4240-1 - Test Hole Location Plan included in Appendix 2.

The boreholes completed by Paterson were drilled with a low clearance portable drill rig operated by a two-person crew. All fieldwork was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer. The drilling procedure consisted of 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. It should be noted that the field recorded N values have been adjusted to account for the 1/3 hammer weight utilized by the portable drilling crew.

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

A 32 mm diameter groundwater monitoring well was installed at all borehole locations to monitor the groundwater level subsequent to the completion of the sampling program. The groundwater observations are discussed in Subsection 4.3 and presented in the Soil Profile and Test Data sheets presented in Appendix 1.

Sample Storage

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

3.2 Field Survey

The test hole locations were determined in the field by Paterson personnel with consideration of underground services and access. Paterson personel completed a survey of all borehole locations and ground surface elevations at the borehole locations. Ground surface elevation at each the borehole location are referenced to a temporary benchmark (TBM), consisting of the top of underground parking door sill. A geodetic elevation of 66.63 m was provided for the TBM by Annis O'Sullivan Vollebekk. The location of the boreholes and ground surface elevation at the borehole locations are presented on Drawing PG4240-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

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

3.4 Analytical Testing

One (1) soil sample was submitted for analytical testing to assess the potential for exposed ferrous metals and the potential of sulphate attacks against subsurface concrete structures. The sample was analyzed to determine its concentration of sulphate and chloride along with its resistivity and pH. The laboratory test results are shown in Appendix 1 and the results are discussed in Subsection 6.7.

4.0 Observations

4.1 Surface Conditions

The subject site is currently occupied by a two storey commercial building with the associated underground parking garage. An at-grade, paved parking area is located within the north portion of the site over the underground parking garage structure. The existing ground surface across the subject site is relatively flat and at grade with the neighbouring roads.

4.2 Subsurface Profile

The subsurface profile encountered at the test hole locations consisted of an asphalt layer placed over a granular fill consisting of a crushed stone followed by a silty sand fill layer and/or a native silty sand and glacial till deposit. A concrete slab was encountered below the asphalt layer at BH 2. Based on the SPT values, the native glacial till layer was noted to be in a loose to dense state of compactness. The glacial till deposit was encountered below the above noted layers and the fine soil matrix consisted of silty sand with gravel and clay. Specific details of the subsurface profile at each test hole location are presented on the Soil Profile and Test Data sheets in Appendix 1.

Based on available geological mapping, the subject site is located in an area where the bedrock consists of interbedded limestone and shale or shale from the Verulam and Billings formations, respectively. Also, based on available geological mapping, the overburden thickness is expected to range from 15 to 25 m.

4.3 Groundwater

Three groundwater monitoring wells were installed as part of the current investigation. Groundwater level measurements were recorded at the monitoring well locations presented in Table 1.

Table 1 - Groundwater Measurements at Monitoring Well Locations				
Test Hole Location	**Ground Surface Elevation at Underground Parking (m)**	**GW Level Reading (m)**	**GW Level Elevation (m)**	**Date**
BH 1	66.59	4.42	62.17	October 4, 2017
BH 2	66.74	3.88	62.86	October 4, 2017
BH 3	66.61	4.36	62.25	October 4, 2017

5.0 Discussion

5.1 Geotechnical Assessment

The subject site is considered adequate from a geotechnical perspective for the proposed structure. It is anticipated that the proposed building will be founded over a raft foundation over an undisturbed, compact glacial till and/or engineered fill bearing surface. Alternatively, a deep foundation option, such as end bearing piles, could be considered to provide support for the proposed building.

Due to the presence of the heritage building within the west portion of the site, a foundation support system will have to be designed to ensure that the existing foundation of the heritage building is not disturbed during the excavation and construction program.

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

5.2 Site Grading and Preparation

Stripping Depth

Topsoil, asphalt, and fill, containing deleterious materials or organic materials, should be stripped from under any building, paved areas, pipe bedding and other settlement sensitive structures.

Fill Placement

Fill placed for grading beneath the building areas should consist, unless otherwise specified, of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. The fill material should be tested and approved prior to delivery to the site. The fill should be placed in maximum 300 mm thick lifts and compacted to 98% of its standard Proctor maximum dry density (SPMDD).

Non-specified existing fill along with site-excavated soil can be placed as general landscaping fill where settlement is a minor concern of the ground surface. 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 placed to increase the subgrade level for areas to be paved, the fill should be compacted in maximum 300 mm thick lifts and to a minimum density of 95% of the respective SPMDD. Non-specified existing fill and site-excavated soils are not suitable for placement as backfill against foundation walls unless a composite drainage blanket connected to a perimeter drainage system is provided along the foundation perimeter.

5.3 Foundation Design

Conventional Shallow Footings

Footings placed on an undisturbed, compact glacial till bearing surface can be designed using a bearing resistance value 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 value 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.

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 an engineered fill or silty sand above the groundwater table when a plane extending horizontally and vertically downward from the footing perimeter at a minimum of 1.5H:1V slope passing through in situ soil of the same or higher capacity as the bearing medium soil.

Raft Foundation

It is expected that the proposed raft foundation will be founded over a thin mud slab placed over an undisturbed, glacial till bearing surface to accommodate the two levels of underground parking.

A bearing resistance value at serviceability limit states (SLS) (contact pressure) of **200 kPa** could be used. The loading conditions for the contact pressure are based on sustained loads, that are generally 100% dead load and 50% live load. The factored bearing resistance at ultimate limit states (ULS) is calculated to be **400 kPa**. A geotechnical resistance factor of 0.5 was applied to the bearing resistance value at ULS.

The modulus of subgrade reaction was calculated to be **20 MPa/m** for a contact pressure of **200 kPa**. The design of the raft foundation should consider the relative stiffness of the reinforced concrete slab and the supporting bearing medium.

Based on the above assumptions for the raft foundation, the proposed structure could be designed with the above parameters and a total and differential settlement of 25 and 20 mm, respectively.

Deep Foundation - End Bearing Piles

Alternatively, a deep foundation method, such as end bearing piles, can be considered for the proposed structure if the design building loads exceed the bearing resistance values provided for a conventional shallow footing foundation. Concrete filled steel pipe piles driven to refusal on a bedrock surface are a typical deep foundation option in Ottawa.

Applicable pile resistance at SLS values and factored pile resistance at ULS values are provided in Table 1. Additional resistance values can be provided if available pile sizes vary from those detailed in Table 2. A resistance factor of 0.4 has been incorporated into the factored ULS values. Note that these are all geotechnical axial resistance values.

The geotechnical pile resistance values were estimated calculating the Hiley dynamic formula. The piles should be confirmed during pile installation with a program of dynamic monitoring. For this project, the dynamic monitoring of four piles is recommended. This is considered to be the minimum monitoring program, as the piles under shear walls may be required to be driven using the maximum recommended driving energy to achieve the greatest factored resistance at ULS values. Re-striking of all piles will also be required after at least 48 hours have elapsed since initial driving.

Table 2 - End Bearing Pile Foundation Design Data

Pile Outside Diameter (mm)	Pile Wall Thickness (mm)	Geotechnical Axial Resistance		Final Set (blows/ 25 mm)	Transferred Hammer Energy (kJ)
		SLS (kN)	Factored at ULS (kN)		
245	10	975	1460	10	35.9
245	12	1100	1650	10	42
245	13	1175	1760	10	45.4

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class D** for the foundations considered. Due to the compactness of the silty sand deposit and the long term groundwater level, soils underlying the subject site are not susceptible to liquefaction. Refer to the latest revision of the 2012 Ontario Building Code for a full discussion of the earthquake design requirements.

5.5 Basement Slab

The removal of all topsoil and deleterious fill, such as those containing organic materials, within the proposed building footprint, the native soil surface will be considered to be an acceptable subgrade on which to commence backfilling for floor slab construction.

Any poor performing areas should be removed and replaced with an approved engineered fill. Upon successful completion of proof-rolling the subgrade surface and approval by the geotechnical consultant, the sub-excavated area should be backfilled with a Granular A or Granular B Type II with a maximum particle size of 50 mm.

It is recommended that the upper 200 mm of sub-floor fill consists of 19 mm clear crushed stone. All backfill materials within the footprint of the proposed building should be placed in maximum 300 mm loose lifts and compacted to, at least, 98% of its SPMDD.

5.6 Basement Wall

There are several combinations of backfill materials and retained soils that could be applicable for the basement walls of the subject structure. However, the conditions can be well-represented by assuming the retained soil consists of a material with an angle of internal friction of 30 degrees and a dry unit weight of 20 kN/m^3. The applicable effective unit weight of the retained soil can be estimated as 13 kN/m^3, where applicable. A hydrostatic pressure should be added to the total static earth pressure when calculating the effective unit weight.

Lateral Earth Pressures

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

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

An additional pressure having a magnitude equal to $K_o \cdot q$ and acting on the entire wall height should be incorporated to the 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 calculated 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 stay at least 0.3 m away 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}) could be calculated using $0.375 \cdot a_c \cdot \gamma \cdot H^2/g$ where:

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

The peak ground acceleration, (a_{max}), for the Ottawa area is 0.32g according to OBC 2012. 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 Structure

Asphalt pavement is not anticipated to be required at the subject site. However, should a flexible pavement be considered for the project, the recommended flexible pavement structures shown in Tables 3 and 4 would be applicable.

Table 3 - Recommended Flexible 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 4 - Recommended Flexible Pavement Structure - Access Lanes	
Thickness (mm)	**Material Description**
40	**Wear Course -** HL-3 or Superpave 12.5 Asphaltic Concrete
50	**Binder Course -** HL-8 or Superpave 19.0 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
400	**SUBBASE** - OPSS Granular B Type II
	SUBGRADE - Either 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 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 the proposed structure. It is expected that insufficient space will be available to place a perimeter drainage pipe along the exterior side of the footing. It is recommended that the composite drainage system (such as Delta Drain 6000 or equivalent) extend down to the footing level. It is recommended that 150 mm diameter sleeves at 3 m centres be cast in the footing or at the foundation wall/footing interface to allow the infiltration of water to flow to the interior perimeter drainage pipe. The perimeter drainage pipe and underfloor drainage system should direct water to sump pit(s) within the lower basement area.

Underfloor Drainage

It is anticipated that underfloor drainage will be required to control water infiltration. For preliminary design purposes, we recommend that 150 mm diameter perforated pipes be placed at a general 6 m spacing with consideration of footing locations. The spacing of the underfloor drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

Foundation Backfill

Backfill against the exterior sides of the foundation walls should consist of free-draining non frost susceptible granular materials above the bedrock surface. The greater part of the site excavated materials will be frost susceptible and, as such, are not recommended for re-use as backfill against the foundation walls, unless used in conjunction with a drainage geocomposite, such as Miradrain G100N or Delta Drain 6000, connected to the perimeter foundation drainage system. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should otherwise be used for this purpose.

6.2 Protection of Footings Against Frost Action

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

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

6.3 Temporary Shoring and Excavation Side Slopes

Excavation Side Slopes

The side slopes of excavations in the soil and fill overburden materials should either be excavated to acceptable slopes or retained by shoring systems from the beginning of the excavation until the structure is backfilled. Sufficient room should be available in selected areas of the excavation to be completed by open-cut methods (i.e. 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. A shallower slope is required for excavation below groundwater level. The subsurface soil is considered to be mainly a Type 2 and 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects.

Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should be maintain safe working distance 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 should be installed at all times 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 be remain open for extended periods of time.

Temporary Shoring

Temporary shoring will be required to support the overburden soils. The design and implementation of these temporary systems will be the responsibility of the excavation contractor or the shoring 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.

Temporary shoring may be required to complete the required excavations where insufficient room is available for open cut methods. The shoring requirements will depend on the depth of the excavation, the proximity of the adjacent buildings and underground structures and the elevation of the adjacent building foundations and underground services. Additional information can be provided when the above details are known.

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 5 - 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.

The hydrostatic groundwater pressure should be added to the earth pressure distribution wherever the submerged unit weights are used for earth pressure calculations should the level on the groundwater not be lowered below the bottom of the excavation. If the groundwater level is lowered, the total unit weight for the soil should be used full weight, with no hydrostatic groundwater pressure component.

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.

A minimum of 150 mm of OPSS Granular A should be placed for bedding for sewer or water pipes when placed on soil subgrade. The bedding should extend to the spring line of the pipe. Cover material, from the spring line to a minimum of 300 mm above the obvert of the pipe should consist of OPSS Granular A (concrete or PSM PVC pipes) or sand (concrete pipe). The bedding and cover materials should be placed in maximum 225 mm thick lifts and compacted to 95% of the SPMDD.

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

6.5 Groundwater Control

Groundwater Control for Building Construction

The groundwater infiltration into the excavations should be low to moderate depending on the subsurface soil conditions. The contractor should be prepared to collect and pump groundwater infiltration volumes from the excavation trenches.

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 Structures

It is understood that two levels of underground parking are planned for the proposed building. Based on the existing groundwater level, minimal groundwater lowering will occur due to the proposed building construction. However, adjacent structures will not be effected by groundwater lowering due to the construction of the proposed building.

6.6 Winter Construction

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

The subsurface conditions 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 installation of straw, propane heaters and tarpaulins or other suitable means. The base of the excavations should be insulated from sub-zero temperatures immediately upon exposure and until 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.

The trench excavations should be constructed in a manner to avoid the introduction of frozen materials, snow or ice into the trenches.

Precaution must be taken where excavations are in close proximity of existing structures which may be adversely affected due to the freezing conditions. In particular, where a shoring system is installed, 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 pH of the sample indicates that they are not significant factors in creating a corrosive environment for exposed ferrous metals at this site, whereas the resistivity is indicative of a low corrosive environment.

7.0 Recommendations

For the foundation design data provided herein to be applicable that a materials testing and observation services program is required to be completed. The following aspects 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.
- ❑ Observation of the placement of the foundation insulation, if applicable.
- ❑ Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- ❑ Observation of all subgrades prior to backfilling and follow-up field density tests to determine the level of compaction achieved.
- ❑ Field density tests to determine the level of compaction achieved.

A report confirming the construction has been conducted in general accordance with the 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 are in accordance with the present understanding of the project. Paterson requests permission to review the recommendations when the drawings and specifications are completed.

A soils investigation is a limited sampling of a site. Should any conditions at the site be encountered which differ from those at the test locations, Paterson requests immediate notification to permit reassessment of our recommendations.

The recommendations provided herein should only be used by the design professionals associated with this project. They are not intended for contractors bidding on or undertaking the work. The latter should evaluate the factual information provided in this report and determine the suitability and completeness for their intended construction schedule and methods. Additional testing may be required for their purposes.

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than Minto Properties or their agents is not authorized without review by Paterson for the applicability of our recommendations to the alternative use of the report.

Paterson Group Inc.

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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
99 Fifth Avenue
Ottawa, Ontario

DATUM TBM - Access to interior courtyard (underground parking) door sill. Underground elevation = 66.63m.

REMARKS

BORINGS BY Portable Drill

DATE September 9, 2017

FILE NO. PG4240

HOLE NO. BH 1

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
Underground Garage Floor					
Asphaltic concrete	0.06	AU	1		
FILL: Crushed stone	0.10				
FILL: Silty sand with gravel	0.18				
FILL: Sand	0.25	SS	2	83	12
Compact, brown **SILTY SAND**	1.12				
Compact, brown **FINE SAND,** some to trace silt	1.83	SS	3	75	16
Compact, brown fine to coarse **SAND,** some gravel	2.13	SS	4	67	13
Dense, brown **SAND to SILTY SAND**	2.90	SS	5	75	36
GLACIAL TILL: Dense, grey silty sand, some clay and gravel	6.65	SS	6	50	43
		SS	7	83	22
		SS	8	25	37
		SS	9	56	30
		SS	10	33	25
		SS	11	33	42
End of Borehole					
(GWL @ 4.42m - Oct. 4, 2017)					

DEPTH (m)	ELEV. (m)
0	66.59
1	65.59
2	64.59
3	63.59
4	62.59
5	61.59
6	60.59

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

Monitoring Well Construction

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
99 Fifth Avenue
Ottawa, Ontario

DATUM TBM - Access to interior courtyard (underground parking) door sill. Underground elevation = 66.63m.

REMARKS

BORINGS BY Portable Drill

DATE September 10, 2017

FILE NO. PG4240

HOLE NO. BH 2

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
Underground Garage Floor						0	66.74
Asphaltic concrete 0.08							
Concrete 0.17		AU	1				
FILL: Crushed stone 0.25							
Compact, brown **SAND** 1.52		SS	2	100	14	1	65.74
		SS	3	75	16		
Compact, brown **SAND to SILTY SAND** 2.90		SS	4	83	15	2	64.74
		SS	5	67	28		
GLACIAL TILL: Compact, grey silty sand with clay and gravel 7.01		SS	6	42	26	3	63.74
		SS	7	58	26	4	62.74
		SS	8	0	4	5	61.74
		SS	9	50	24		
		SS	10	25	16	6	60.74
		SS	11	33	5	7	59.74
End of Borehole							
(GWL @ 3.88m - Oct. 4, 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

Monitoring Well Construction

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
99 Fifth Avenue
Ottawa, Ontario

DATUM TBM - Access to interior courtyard (underground parking) door sill. Underground elevation = 66.63m.

REMARKS

BORINGS BY Portable Drill

DATE September 28, 2017

FILE NO. PG4240

HOLE NO. BH 3

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
Underground Garage Floor						0	66.61
Asphaltic concrete	0.08	AU	1				
FILL: Crushed stone	0.20						
Compact to dense, brown **SILTY FINE SAND to SANDY SILT**		SS	2	62	12		
						1	65.61
		SS	3	75	11		
		SS	4	62	19		
						2	64.61
	2.90	SS	5	75	23		
						3	63.61
GLACIAL TILL: Dense to very dense, brown silty sand with gravel, trace clay		SS	6	58	66		
						4	62.61
		SS	7	62	51		
						5	61.61
		SS	8	33	57		
						6	60.61
	6.25	SS	9	33	64		

End of Borehole

(GWL @ 4.36m - Oct. 4, 2017)

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

○ Water Content %

20 40 60 80

Monitoring Well Construction

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

SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

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

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

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

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

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

Cohesive soils can also be classified according to their “sensitivity”. The sensitivity is the ratio between the undisturbed undrained shear strength and the remoulded undrained shear strength of the soil.

Terminology used for describing soil strata based upon texture, or the proportion of individual particle sizes present is provided on the Textural Soil Classification Chart at the end of this information package.

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

SS	-	Split spoon sample (obtained in conjunction with the performing of the Standard Penetration Test (SPT))
TW	-	Thin wall tube or Shelby tube
PS	-	Piston sample
AU	-	Auger sample or bulk sample
WS	-	Wash sample
RC	-	Rock core sample (Core bit size AXT, BXL, etc.). Rock core samples are obtained with the use of standard diamond drilling bits.

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

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

Cc and Cu are used to assess the grading of sands and gravels:
Well-graded gravels have: $1 < Cc < 3$ and $Cu > 4$
Well-graded sands have: $1 < Cc < 3$ and $Cu > 6$
Sands and gravels not meeting the above requirements are poorly-graded or uniformly-graded.
Cc and Cu are not applicable for the description of soils with more than 10% silt and clay (more than 10% finer than 0.075 mm or the #200 sieve)

CONSOLIDATION TEST

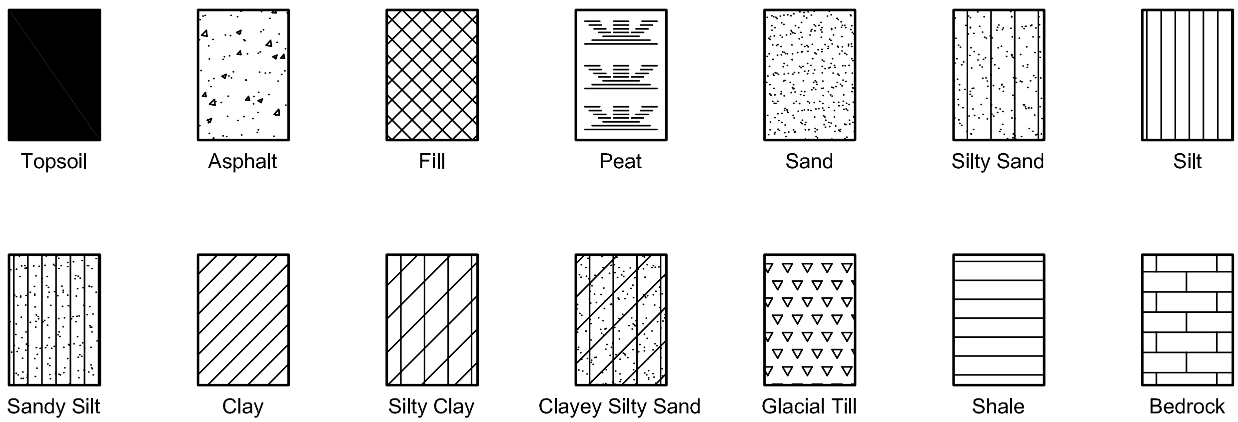
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void Ratio		Initial sample void ratio = volume of voids / volume of solids
Wo	-	Initial water content (at start of consolidation test)

PERMEABILITY TEST

k	-	Coefficient of permeability or hydraulic conductivity is a measure of the ability of water to flow through the sample. The value of k is measured at a specified unit weight for (remoulded) cohesionless soil samples, because its value will vary with the unit weight or density of the sample during the test.

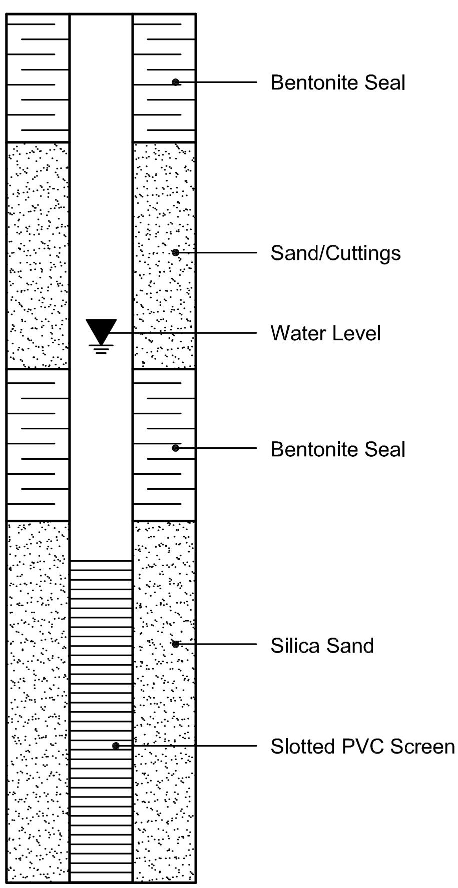
SYMBOLS AND TERMS (continued)

STRATA PLOT

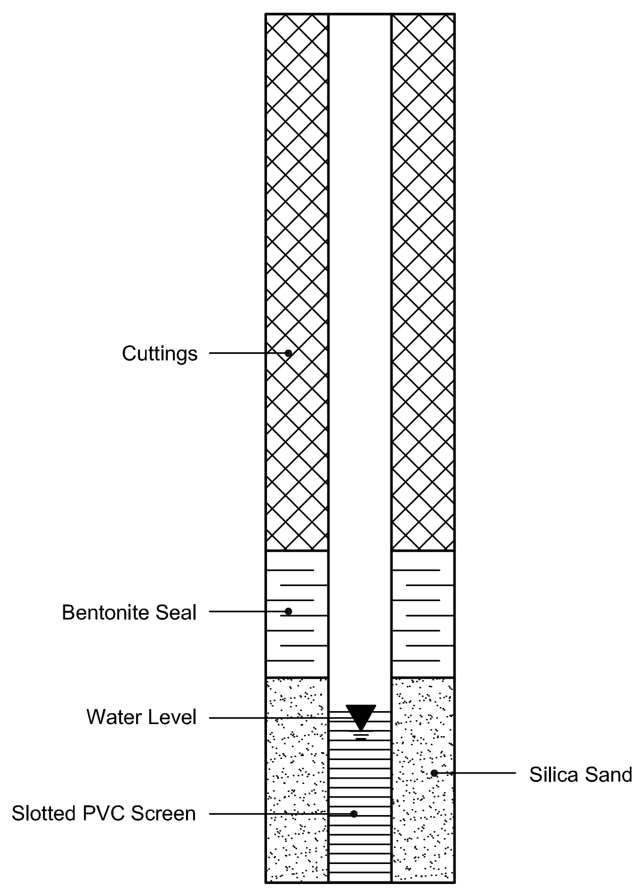


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 23005

Report Date: 25-Oct-2017

Order Date: 19-Oct-2017

Project Description: PG4042

	Client ID:	BH1-SS6	-	-	-
	Sample Date:	09-Sep-17	-	-	-
	Sample ID:	1742563-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	93.5	-	-	-
General Inorganics					
pH	0.05 pH Units	7.94	-	-	-
Resistivity	0.10 Ohm.m	14.5	-	-	-
Anions					
Chloride	5 ug/g dry	294 [1]	-	-	-
Sulphate	5 ug/g dry	224 [1]	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING PG4240-1 - TEST HOLE LOCATION PLAN

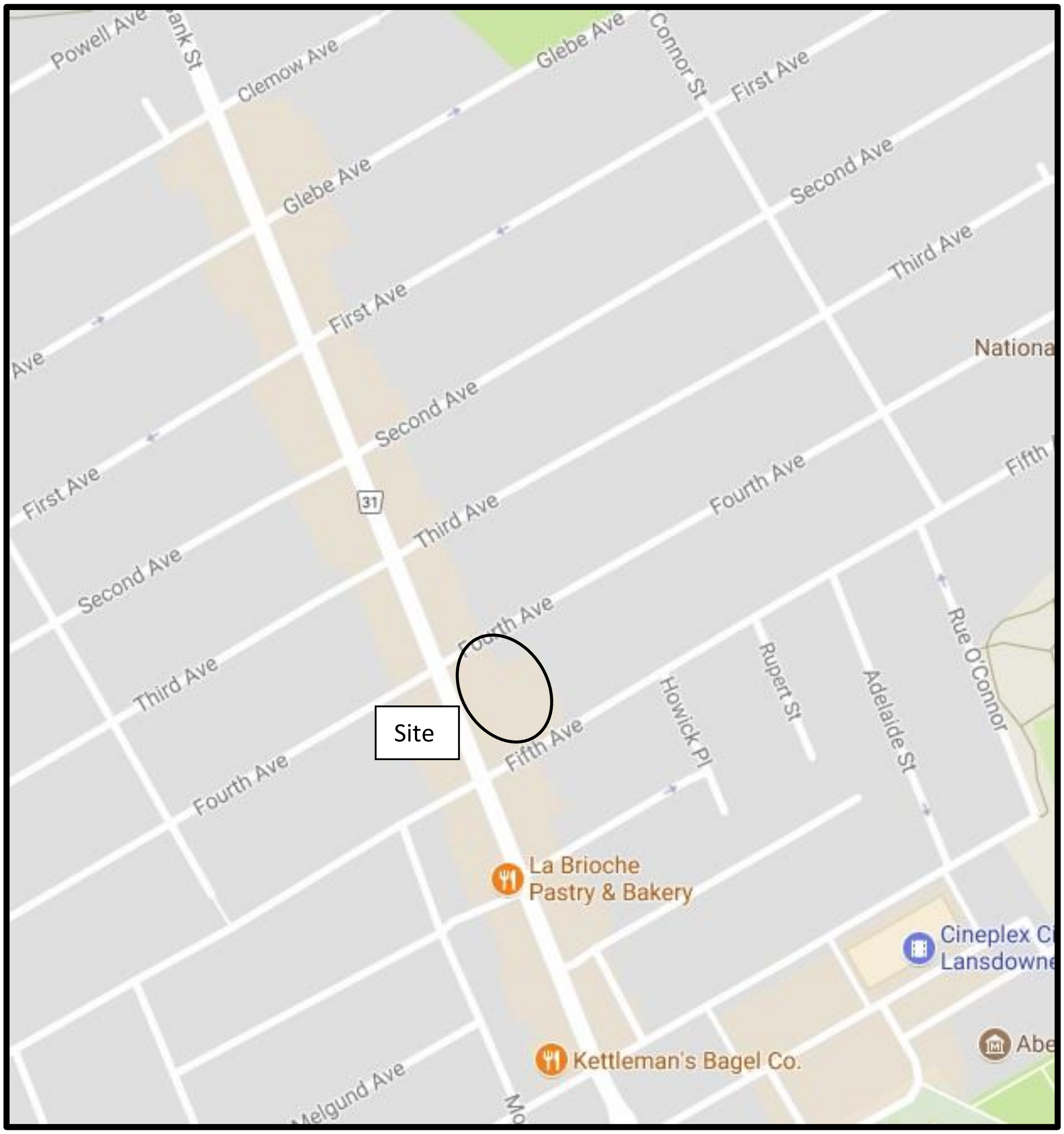
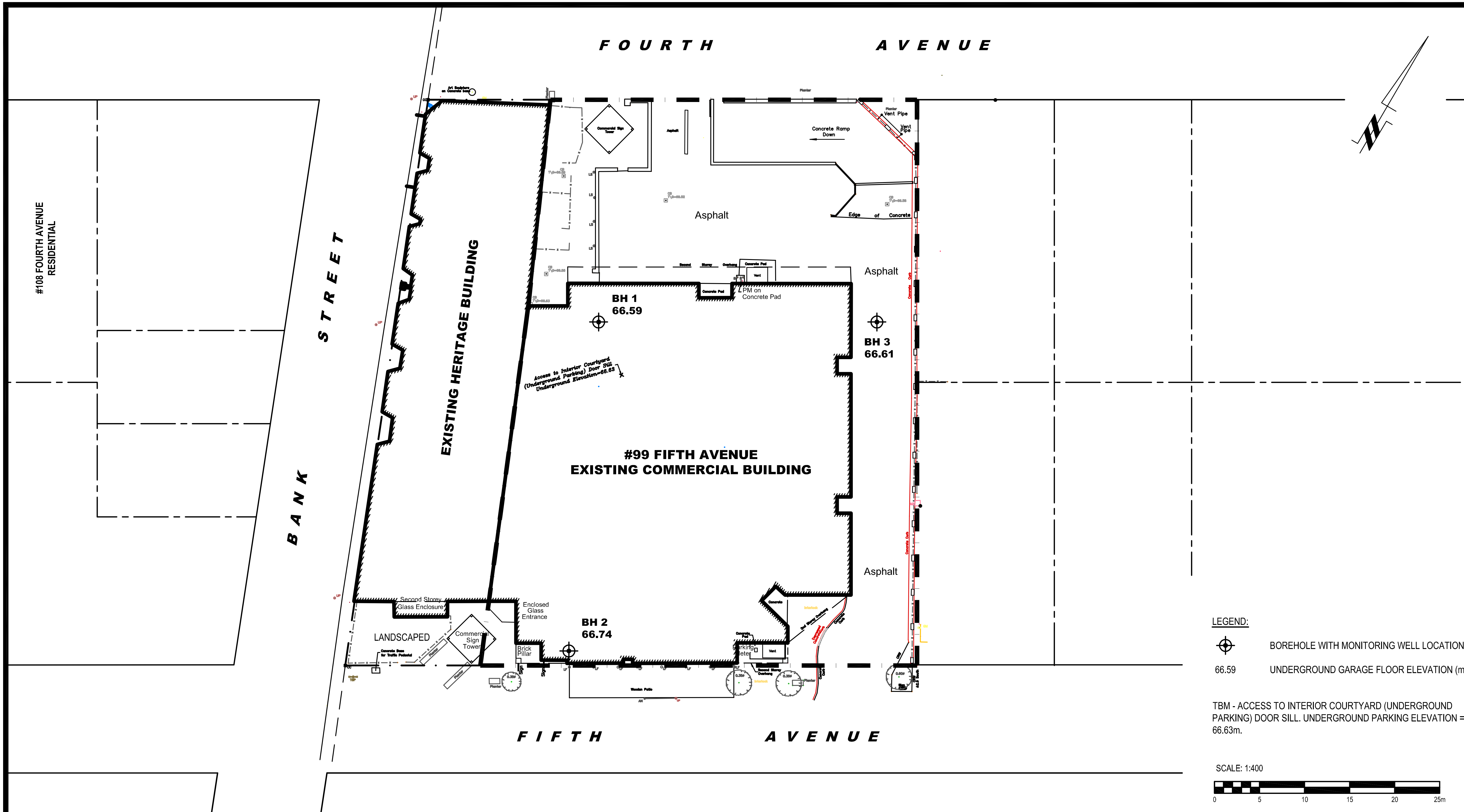


FIGURE 1 – Key Plan



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NO.	REVISIONS	DATE	INITIAL
0			

MINTO PROPERTIES INC.

GEOTECHNICAL INVESTIGATION
99 FIFTH AVENUE

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:400	Date: 10/2017
Drawn by: MPG	Report No.: PG4240-1
Checked by: FA	Dwg. No.: PG4240-1
Approved by: DJG	Revision No.: 0

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