Geotechnical Engineering

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Hydrogeology

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Materials Testing

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Geotechnical Investigation

Proposed Residential Building
700 Coronation Avenue
Ottawa, Ontario

Prepared For

Inspire Developments

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Report: PG2623-1 Revision 1

Paterson Group Inc.
Consulting Engineers
154 Colonnade Road South
Ottawa (Nepean), Ontario
Canada K2E 7J5

Tel: (613) 226-7381
Fax: (613) 226-6344
www.patersongroup.ca

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Inspire Developments to conduct a geotechnical investigation for the proposed residential building to be located at 700 Coronation Avenue, in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objectives of the current investigation were:

❏ determine the subsoil and groundwater conditions at this site by means of boreholes,

❏ to provide geotechnical recommendations pertaining to 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.

2.0 Proposed Development

It is understood that the proposed development is to consist of a four (4) storey residential building with one level of underground parking. At-grade landscaping, parking areas and access lanes are also anticipated as part of the proposed development.

3.0 Method of Investigation

3.1 Field Investigation

The fieldwork program for the investigation was carried out on February 17, 2011. At that time, five (5) boreholes were advanced to a maximum 6.5 m. The locations were selected by Paterson personnel taking into consideration site features. The locations of the boreholes are shown on Drawing PG2623-1 - Test Hole Location Plan 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 were recovered using a 50 mm diameter split-spoon sampler or from the auger flights. The split spoon and auger samples were classified on site and placed in sealed plastic bags. All soil samples were transported to our laboratory. Bedrock was cored at one (1) borehole location using diamond drilling techniques. The bedrock core was recovered from the core run, placed in a cardboard box, and sent to our laboratory for further review. The depths at which the split spoon, auger and bedrock samples were recovered from the boreholes are shown as SS, AU, and RC, respectively, on the Soil Profile and Test Data sheets in Appendix 1.

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

Diamond drilling using NQ size coring equipment was carried out in one (1) borehole to determine the nature of the bedrock and to assess its quality. Recovery value and Rock Quality Designation (RQD) value were calculated for each drilled section (core run) of bedrock and are shown on the Soil Profile and Test Data sheets in Appendix 1. The recovery value is the ratio, in percentage, of the length of the bedrock sample recovered over the length of the drilled section (core run).

The RQD value is the ratio, in percentage, of the total length of rock pieces longer than 100 mm in one core run over the length of the core run. These values are indicative of the quality of the bedrock. Subsurface conditions observed in the boreholes were recorded in detail in the field. Reference should be made to the Soil Profile and Test Data sheets presented in Appendix 1 for specific details of the soil profile encountered at the test hole locations.

. **Groundwater**

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

Sample Storage

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

3.2 Field Survey

The borehole locations were selected, determined in the field and surveyed by Paterson. The ground surface elevation at each borehole location was referenced to a temporary benchmark (TBM), consisting of a manhole cover located north of 710 Coronation Avenue East of the proposed site. A geodetic elevation of 74.30 m was provided by Farley, Smith and Denis Surveying for the TBM. The location and ground surface elevations at borehole locations are presented on Drawing PG2623-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

The soil samples recovered from the subject site were examined in our laboratory to review the results of the field logging.

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 6.8.

4.0 Observations

4.1 Surface Conditions

The subject site is currently occupied by an existing residential apartment building along with associated parking area to the west of the existing building. A landscaped area was also noted within the east portion of the site. The site is relatively flat and at grade with Coronation Avenue and adjacent properties.

4.2 Subsurface Profile

Overburden

The soil profile encountered at the test hole locations consists of a pavement structure at ground surface underlain by a fill layer, consisting of a brown silty clay with gravel and/or silty sand with gravel. A glacial till deposit was noted below the above noted layers followed by a shale bedrock. Shale bedrock was cored at BH 1 to a 6.5 m depth. Based on the RQD values of the recovered bedrock samples, the bedrock is of a fair quality. Specific details of the soil profile at the test hole locations are presented on the Soil Profile and Test Data sheets in Appendix 1.

Bedrock

Based on available geological mapping, the bedrock in the immediate area of the subject site consists of shale bedrock of the Carlsbad formation.

4.3 Groundwater

The measured groundwater levels from piezometers installed at the boreholes are presented in Table 1. It should be noted that groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater level could be different at the time of construction.

Table 1 - Summary of Groundwater Level Readings				
Test Hole Number	**Ground Elevation, m**	**Groundwater Levels, m**		**Recording Date**
		Depth	**Elevation**	
BH 1	74.32	1.45	72.87	February 17, 2012
BH 2	74.30	Damaged	N/A	February 17, 2012
BH 3	74.40	1.70	72.70	February 17, 2012
BH 4	74.60	3.45	71.15	February 17, 2012
BH 5	74.64	2.03	72.61	February 17, 2012

Note: The ground surface elevations at the test hole locations were referenced to a temporary benchmark (TBM) consisting of a manhole located at the northwest corner of 710 Coronation Avenue and assigned a geodetic elevation of 74.30 m by Farley, Smith and Denis Surveying.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered adequate for the proposed residential development. It is expected that the proposed building can be founded by conventional shallow foundations.

Based on the plans provided, the east foundation wall of the proposed underground parking structure will be within the lateral support zone of the footings for the existing building. Consideration should be given to completing an underpinning program for the existing building to ensure that adequate lateral support is provided. An underpinning program, such as a 2 to 3 m wide excavation completed with a piano key style excavation technique and extending to the proposed underside of footing of the parking structure, would be adequate. The excavation should be in-filled with a lean concrete to the existing structures underside of footing prior to excavating adjacent sections.

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

5.2 Site Grading and Preparation

Stripping Depth

Asphalt, topsoil or fill, containing deleterious and organic materials, should be stripped from under any buildings, paved areas, pipe bedding and other settlement sensitive structures.

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

Bedrock Removal

Bedrock removal can be accomplished by hoe ramming where only a small quantity of the bedrock needs to be removed. Sound bedrock may be removed by line drilling and controlled blasting and/or hoe ramming.

Prior to considering blasting operations, the blasting effects on the existing services, buildings and other structures should be addressed. A pre-blast or pre-construction survey of the existing structures located in proximity of the blasting operations should be carried out prior to commencing site activities. The extent of the survey should be determined by the blasting consultant and should be sufficient to respond to any inquiries/claims related to the blasting operations.

As a general guideline, peak particle velocities (measured at the structures) should not exceed 25 mm per second during the blasting program to reduce the risks of damage to the existing structures.

The blasting operations should be planned and conducted under the supervision of a licensed professional engineer who is also an experienced blasting consultant.

Excavation side slopes in sound bedrock can be carried out using almost vertical side walls. A minimum 1 m horizontal ledge, should be left between the bottom of the overburden excavation and the top of the bedrock surface to provide an area to allow for potential sloughing.

Fill Placement

Fill used 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. 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 proposed building should be compacted to at least 98% of its standard Proctor maximum dry density (SPMDD).

Non-specified existing fill along with site-excavated soil can be used as general landscaping fill where settlement of the ground surface is of minor concern. These materials should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids. If these materials are to be used to build up the subgrade level for areas to be paved, they should be compacted in thin lifts to a minimum density of 95% of their respective SPMDD. Non-specified existing fill and site-excavated soils are not suitable for use as backfill against foundation walls unless a composite drainage blanket connected to a perimeter drainage system is provided.

5.3 Foundation Design

Bearing Resistance Values

Footings placed on an undisturbed, dense 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 reported bearing resistance value at ULS. 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.

Footings placed on a clean, surface-sounded bedrock surface can be designed using a bearing resistance value at SLS of **500 kPa** and a factored bearing resistance value at ULS of **750 kPa**. The potential long term post-construction total and differential settlements for footings placed on surface-sounded bedrock are estimated to be negligible.

A clean, surface-sounded bedrock bearing surface should be free of all soil and loose materials, and have no near surface seams, voids, fissures or open joints which can be detected from surface sounding with a rock hammer.

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 or engineered fill 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.

The bearing medium under footing-supported structures is required to be provided with adequate lateral support. Near vertical (1H:6V) slopes can be used for unfractured bedrock bearing media. A 1H:1V slope can be used for fractured/weathered bedrock.

Underpinning Program

In areas where the east foundation wall of the proposed underground parking structure will be within the lateral support zone of the footings for the existing building, an underpinning program will be required to safely transfer the existing building loads down to a lower founding elevation. An underpinning program, such as a 2 to 3 m wide excavation completed with a piano key style excavation technique and extending to the proposed underside of footing of the parking structure, would be adequate. The excavation should be in-filled with a lean concrete to the existing structures underside of footing prior to excavating adjacent sections.

Once details of the proposed building are finalized, it is recommended that details of the underpinning program be prepared in conjunction with the projects structural engineer. It is recommended that an assessment be completed by the geotechnical engineer at the time of excavation to confirm founding conditions of the existing structure.

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. A higher site class, such as a Class B, may be applicable for design of the proposed building. However, the higher site class has to be confirmed by site specific shear wave velocity testing. Reference should be made to the latest revision of the 2012 Ontario Building Code for a full discussion of the earthquake design requirements.

5.5 Basement Slab

With the removal of all topsoil, and deleterious fill, containing organic matter, within the footprint of the proposed building, the native soil 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.

It is understood that an underground parking level is anticipated for the proposed building. It is expected that a concrete slab topping with a sub-floor granular layer will be incorporated in the design to accommodate services and the rigid pavement structure noted in Subsection 5.7 will be applicable.

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 applicable effective unit weight of the retained soil can be estimated as 13 kN/m^3, where applicable.

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 = $(1.45 - a_{max}/g)a_{max}$
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 proposed parking level slab will be considered a rigid pavement structure. The following rigid pavement structure is suggested to support car parking only.

Table 2 - Recommended Rigid Pavement Structure - Car Only Parking Areas

Thickness (mm)	Material Description
125	**Wear Course -** Concrete slab
150	**BASE** - 19 mm clear stone
SUBGRADE - Either in situ soils, engineered fill or OPSS Granular B Type I or II material placed over in situ soil.	

The following table could be considered for the design of the proposed hard landscaping walkway paths.

Table 3 - Recommended Hard Landscaping - Pedestrian Walkways

Thickness (mm)	Material Description
As Specified	**Wear Course** - Interlocking Stones/Brick Pavers
25 - 40	**Leveling Course** - Stone Dust or Sand
300	**BASE** - OPSS Granular A crushed stone
SUBGRADE - Either in situ soils, engineered fill or OPSS Granular B Type I or II material placed over in situ soil.	

The recommended flexible pavement structures shown in Tables 4 and 5 would be applicable, where a flexible asphaltic concrete is required.

Table 4 - Recommended Pavement Structure - Car Only Parking Areas	
Thickness (mm)	**Material Description**
50	**Wear Course** - HL-3 or Superpave 12.5 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
300	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Either in situ soils, engineered fill or OPSS Granular B Type I or II material placed over in situ soil	

Table 5 - Recommended 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 in situ soils, engineered fill or OPSS Granular B Type I or II material placed over in situ soil	

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

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

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

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, 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.

Backfill against the exterior sides of the foundation walls should consist of free-draining non frost susceptible granular materials. The greater part of the site excavated materials will be frost susceptible and, as such, are not recommended for re-use as backfill against the foundation walls, unless used in conjunction with a drainage geocomposite, such as 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 effect of frost action. A minimum of 1.5 m thick soil cover (or equivalent) should be provided in this regard.

A minimum of 2.1 m thick soil cover (or equivalent) should be provided for exterior unheated footings, not thermally connected to a heated space, such as exterior columns and/or wing walls.

6.3 Excavation Side Slopes and Temporary Shoring

Excavation Side Slopes

The side slopes of excavations in the soil and fill 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. It is assumed that sufficient room will be available for the greater part of the excavation to be undertaken by open-cut methods (i.e. unsupported excavations).

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

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

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

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

Temporary Shoring

Temporary shoring may be required for the overburden soil to complete the required excavations where insufficient room is available for open cut methods. The shoring requirements designed by a structural engineer specializing in those works will depend on the depth of the excavation, the proximity of the adjacent structures and the elevation of the adjacent building foundations and underground services. 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 impact of a significant precipitation event and designate design measures to ensure that a precipitation will not negatively impact the shoring system or soils supported by the system. Any changes to the approved shoring design system should be reported immediately to the owner’s structural design prior to implementation.

The temporary system could 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 included to the earth pressures described below. These systems could be cantilevered, anchored or braced. Generally, it is expected that the shoring systems will be provided with tie-back rock anchors to ensure their stability. The shoring system is recommended to be adequately supported to resist toe failure and inspected to ensure that the sheet piles extend well below the excavation base.

It should be noted if consideration is being given to utilizing a raker style support for the shoring system that lateral movements can occur and the structural engineer should ensure that the design selected minimizes these movements to tolerable levels.

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

Table 6 - Soil Parameters

Parameters	Values
Active Earth Pressure Coefficient (K_a)	0.33
Passive Earth Pressure Coefficient (K_p)	3
At-Rest Earth Pressure Coefficient (K_o)	0.5
Dry Unit Weight (γ), kN/m^3	20
Effective Unit Weight (γ), kN/m^3	13

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

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

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

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 99% 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 99% 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. It is anticipated that pumping from open sumps will be sufficient to control the groundwater influx through the sides of the excavations.

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.

Impacts on Neighboring Structures

Based on our observations, the long term groundwater level is expected at a depth within the dense glacial till deposit. Based on our observations, no groundwater lowering is anticipated under short-term conditions due to construction of the proposed building. The neighboring structures are expected to be founded within the glacial till deposit or on bedrock. Issues are not expected with respect to groundwater lowering that would cause long term damage to adjacent structures surrounding the proposed buildings.

6.6 Winter Construction

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

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

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

6.7 Corrosion Potential and Sulphate

The results of analytical testing show that the sulphate content is less than 0.1%. This result is indicative that Type 10 Portland cement (normal cement) would be appropriate for this site. The chloride content and the pH of the sample indicate that they are not significant factors in creating a corrosive environment for exposed ferrous metals at this site, whereas the resistivity is indicative of an aggressive corrosive environment.

7.0 Recommendations

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

- ❏ Review master grading plan from a geotechnical perspective, once available.
- ❏ Review of underpinning program and details from a geotechnical perspective.
- ❏ Observation of all bearing surfaces prior to the placement of concrete.
- ❏ Sampling and testing of the concrete and granular fill materials used.
- ❏ Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- ❏ Observation of all subgrades prior to backfilling.
- ❏ Field density tests to determine the level of compaction achieved.
- ❏ Sampling and testing of the bituminous concrete including mix design reviews.

A report confirming that these works have been conducted in general accordance with our recommendations could be issued, upon request, following the completion of a satisfactory materials testing and observation program by the geotechnical consultant.

8.0 Statement of Limitations

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

A 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, 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 Inspire Developments or their agents is not authorized without review by Paterson for the applicability of our recommendations to the alternative use of the report.

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
Aug. 12-2020
D. J. GILBERT
100116130
PROVINCE OF ONTARIO

Drew Petahtegoose, B.Eng.

David J. Gilbert, P.Eng.

Report Distribution:

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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 Building - 700 Coronation Ave.
Ottawa, Ontario

DATUM TBM - Top cover of manhole located at the northeast corner of subject site. A geodetic elevation of 74.30m was provided by Farley, Smith and Denis Surveying Ltd.

REMARKS

BORINGS BY CME 75 Power Auger

DATE 17 February 2012

FILE NO. PG2623

HOLE NO. BH 1

SOIL DESCRIPTION	Bottom depth (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Asphaltic concrete	0.10				
FILL: Crushed stone	0.30	AU	1		
FILL: Brown silty clay with sand and gravel	1.45	AU	2		
		SS	3	50	41
GLACIAL TILL: Dense, brown silty sand with gravel, cobbles, boulders and clay	2.21	SS	4	83	33
GLACIAL TILL: Grey silty clay with gravel, cobbles, boulders, trace sand	2.97	SS	5	75	51
GLACIAL TILL: Very dense to dense, grey silty sand with gravel, cobbles, boulders, trace shale	4.40	SS	6	75	58
		SS	7	75	38
BEDROCK: Black shale	6.47	SS	8	100	50+
		RC	1	65	50
		RC	2	100	76
End of Borehole					
(GWL @ 1.45m-Feb. 17/12)					

DEPTH (m)	ELEV. (m)
0	74.32
1	73.32
2	72.32
3	71.32
4	70.32
5	69.32
6	68.32

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 Building - 700 Coronation Ave.
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DATUM TBM - Top cover of manhole located at the northeast corner of subject site. A geodetic elevation of 74.30m was provided by Farley, Smith and Denis Surveying Ltd.

REMARKS

BORINGS BY CME 75 Power Auger

DATE 17 February 2012

FILE NO. PG2623

HOLE NO. BH 2

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	74.30
Asphaltic concrete	0.10						
FILL: Crushed stone	0.30	AU	1				
FILL: Brown silty clay with sand and gravel	0.60	AU	2				
GLACIAL TILL: Brown to grey silty clay with sand, gravel, cobbles and boulders	4.11	SS	3	83	21	1	73.30
		SS	4	83	34	2	72.30
		SS	5	75	21	3	71.30
		SS	6	75	24	4	70.30
GLACIAL TILL: Grey silty sand with gravel, cobbles, boulders, trace clay and shale	5.41	SS	7	75	50+	5	69.30

End of Borehole

Practical refusal to augering @ 5.41m depth

(Piezometer damaged - Feb. 17/12)

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 Building - 700 Coronation Ave.
Ottawa, Ontario

DATUM TBM - Top cover of manhole located at the northeast corner of subject site. A geodetic elevation of 74.30m was provided by Farley, Smith and Denis Surveying Ltd.

REMARKS

BORINGS BY CME 75 Power Auger

DATE 17 February 2012

FILE NO. PG2623

HOLE NO. BH 3

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	74.40
Asphaltic concrete	0.10						
FILL: Crushed stone	0.30	AU	1				
FILL: Crushed stone with gravel and sand	0.60	AU	2				
GLACIAL TILL: Brown silty clay with sand, gravel, cobbles and boulders	2.21	SS	3	83	19	1	73.40
		SS	4	100	35	2	72.40
GLACIAL TILL: Grey silty sand with gravel, cobbles, boulders, trace clay	3.28	SS	5	83	27	3	71.40
		SS	6	60	50+		

End of Borehole

Practical refusal to augering @ 3.28m depth

(GWL @ 1.70m-Feb. 17/12)

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 Building - 700 Coronation Ave.
Ottawa, Ontario

DATUM TBM - Top cover of manhole located at the northeast corner of subject site. A geodetic elevation of 74.30m was provided by Farley, Smith and Denis Surveying Ltd.

REMARKS

BORINGS BY CME 75 Power Auger

DATE 17 February 2012

FILE NO. PG2623

HOLE NO. BH 4

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	74.60
Asphaltic concrete	0.10	AU	1				
FILL: Crushed stone	0.30						
FILL: Brown silty clay with sand and gravel	1.45	AU	2				
		SS	3	4	17	1	73.60
GLACIAL TILL: Brown silty clay with sand, gravel, cobbles and boulders	2.21	SS	4	62	34	2	72.60
GLACIAL TILL: Dense to very dense, grey silty sand with gravel, cobbles and boulders	5.16	SS	5	75	37	3	71.60
		SS	6	75	45	4	70.60
		SS	7	91	59	5	69.60
End of Borehole							

Practical refusal to augering @ 5.16m depth

(GWL @ 3.45m-Feb. 17/12)

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 Building - 700 Coronation Ave.
Ottawa, Ontario

DATUM TBM - Top cover of manhole located at the northeast corner of subject site. A geodetic elevation of 74.30m was provided by Farley, Smith and Denis Surveying Ltd.

REMARKS

BORINGS BY CME 75 Power Auger

DATE 17 February 2012

FILE NO. PG2623

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	74.64
Asphaltic concrete	0.10	AU	1				
FILL: Crushed stone	0.30						
FILL: Brown silty sand with gravel, trace clay	0.69	AU	2				
GLACIAL TILL: Brown silty clay with sand, gravel, cobbles and boulders		SS	3	37	26	1	73.64
	2.21	SS	4	100	34	2	72.64
GLACIAL TILL: Dense to very dense, grey silty sand with gravel, cobbles, boulders, trace clay		SS	5	83	30	3	71.64
		SS	6	83	35	4	70.64
	5.41	SS	7	100	80	5	69.64

End of Borehole

Practical refusal to augering @ 5.41m depth.

(GWL @ 2.03m-Feb. 17/12)

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

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 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

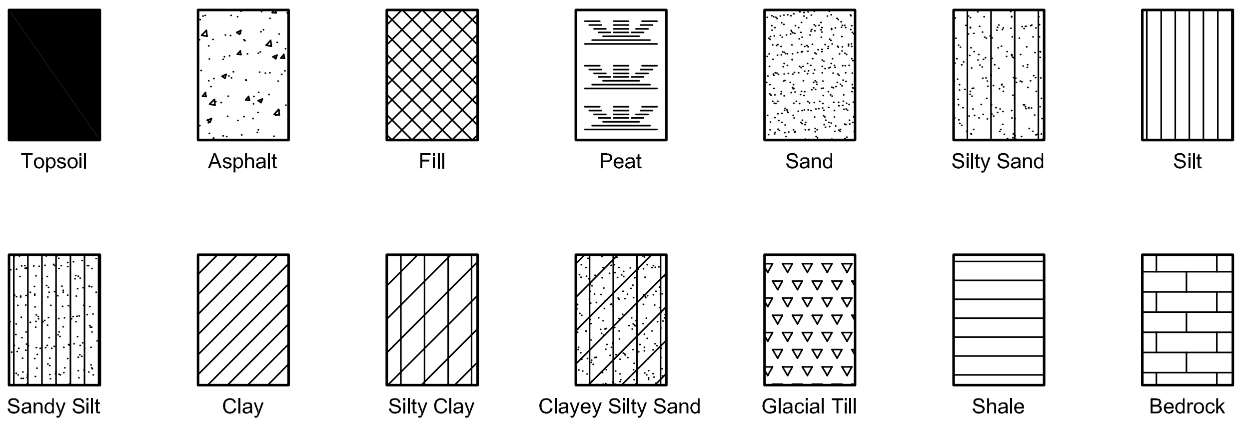
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void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

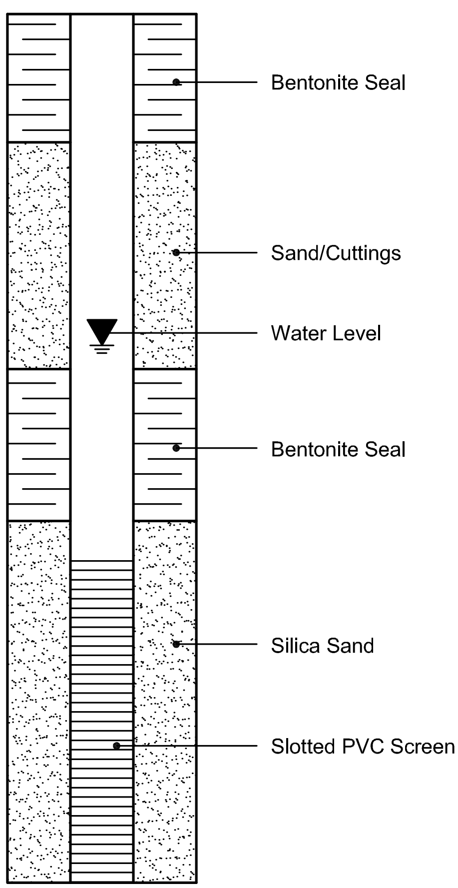
SYMBOLS AND TERMS (continued)

STRATA PLOT

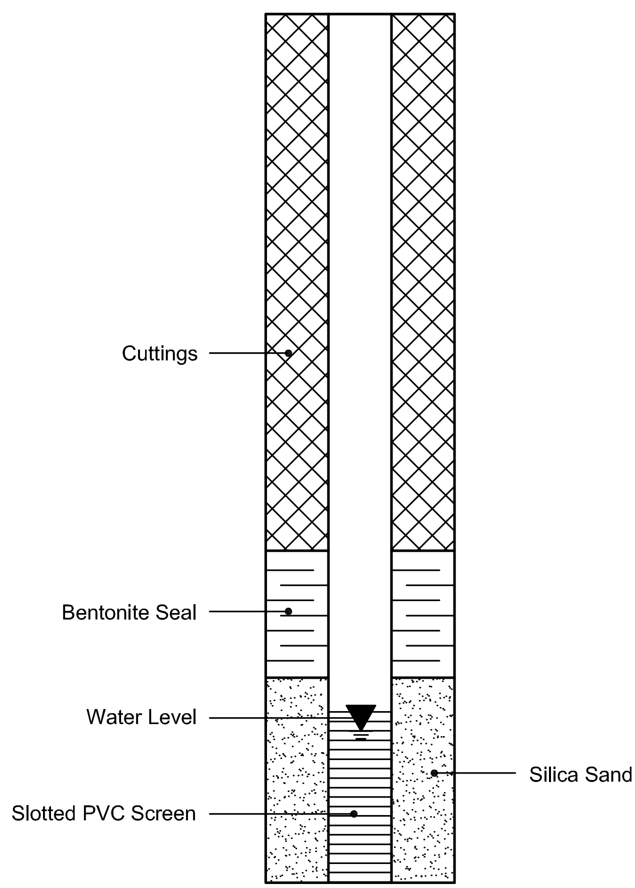


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Client: **Paterson Group Consulting Engineers**

Client PO: 12120

Project Description: PG2623

Report Date: 27-Feb-2012

Order Date:22-Feb-2012

	Client ID:	BH2 SS4	-	-	-
	Sample Date:	17-Feb-12	-	-	-
	Sample ID:	1208057-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	88.4	-	-	-
General Inorganics					
pH	0.05 pH Units	7.21	-	-	-
Resistivity	0.10 Ohm.m	32.9	-	-	-
Anions					
Chloride	5 ug/g dry	81	-	-	-
Sulphate	5 ug/g dry	100	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING PG2623-1 - TEST HOLE LOCATION PLAN

FIGURE 1

KEY PLAN

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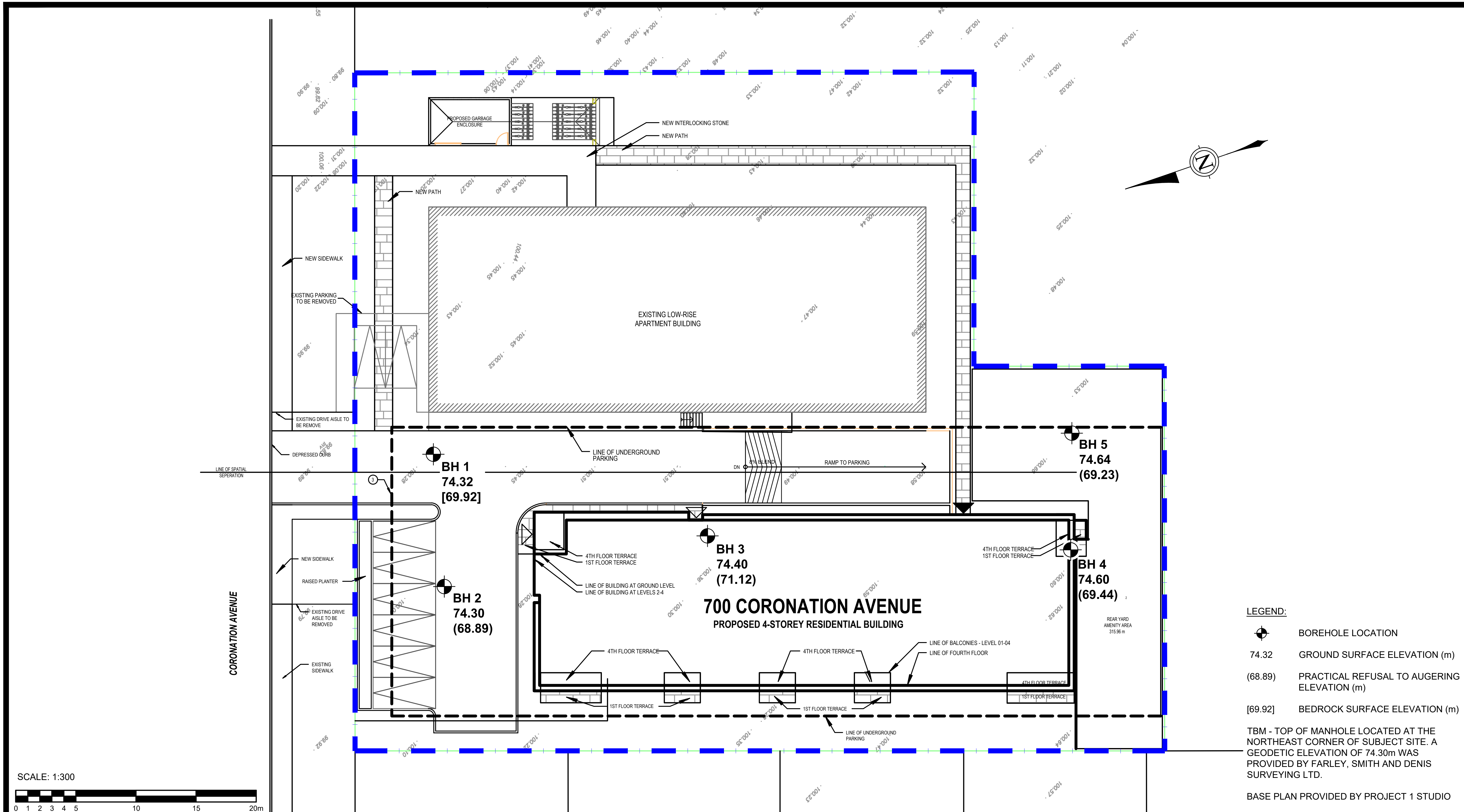

patersongroup
consulting engineers

154 Colonnade Road South
Ottawa, Ontario K2E 7J5
Tel: (613) 226-7381 Fax: (613) 226-6344

NO.	REVISIONS	DATE	INITIAL
1	UPDATED TO LATEST CONCEPTUAL PLAN	11/08/2020	DP

INSPIRE DEVELOPMENTS

GEOTECHNICAL INVESTIGATION
PROP. RESIDENTIAL BUILDING - 700 CORONATION AVENUE

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:300	Date: 08/2020
Drawn by: RCG	Report No.: PE4937-2
Checked by: DP	Dwg. No.: PG2623-1
Approved by: DJG	Revision No.: 1

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