

Geotechnical
Engineering

Environmental
Engineering

Hydrogeology

Geological
Engineering

Materials Testing

Building Science

Noise & Vibration Studies

Geotechnical Investigation

Proposed Multi-Storey Building
436 Athlone Avenue
Ottawa, Ontario

Prepared For

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Report PG5960-1
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1.0 Introduction

Paterson Group (Paterson) was commissioned by 2707120 Ontario Inc. to conduct a geotechnical investigation for the proposed multi-storey building to be located at 436 Athlone Avenue in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objectives of the geotechnical investigation were to:

- ❑ Determine the subsoil and groundwater conditions at this site by means of boreholes.
- ❑ Provide geotechnical recommendations for the design of the proposed development including construction considerations which may affect the design.

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

2.0 Proposed Development

Based on the available drawings, it is understood that the proposed development will consist of a multi-storey residential building with 1 basement level. It is also understood that the proposed development may included walkways, asphalt-paved parking areas, and landscaped areas surrounding the proposed building.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the investigation was carried out on October 20, 2021. At that time, 3 boreholes were advanced to a maximum depth of 3.7 m. The borehole locations were distributed in a manner to provide general coverage of the subject site. The approximate locations of the boreholes are shown on Drawing PG5960-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were advanced using a low clearance, track-mounted 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, and sampling and testing the overburden.

Sampling and In Situ Testing

Soil samples were collected from the boreholes using two different techniques, namely, sampled directly from the auger flights (AU) or collected using a 50 mm diameter split-spoon (SS) sampler. All samples were visually inspected and initially classified on site and subsequently placed in sealed plastic bags. All samples were transported to our laboratory for further examination and classification. The depths at which the auger and split spoon samples were recovered from the boreholes are shown as AU and SS, respectively, on the Soil Profile and Test Data sheets presented in Appendix 1.

A 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.

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

Groundwater

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

3.2 Field Survey

The test hole locations were selected by Paterson to provide general coverage of the proposed development, taking into consideration the existing site features and underground utilities. The test hole locations and ground surface elevation at each test hole location were surveyed by Paterson using a handheld GPS and referenced to a geodetic datum. The locations of the test holes and ground surface elevation at each test hole location are presented on Drawing PG5960-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

Soil samples were recovered from the subject site and visually examined in our laboratory to review the results of the field logging. All samples will be stored in the laboratory for a period of one (1) month after issuance of this report. They will then be discarded unless we are otherwise directed.

3.4 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. The results are presented in Appendix 1 and are discussed further in Section 6.7.

4.0 Observations

4.1 Surface Conditions

Currently, the subject site is occupied by a two storey residential dwelling, which is surrounded by an asphalt-paved driveway and landscaped areas. The site is bordered to east by Athlone Avenue, and to the north, south, and west by residential properties. The existing ground surface is relatively level across the site at approximate geodetic elevation 72.5 m.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile at the test hole locations consists of an approximate 0.5 m thick layer of fill underlying an asphalt or topsoil surface. The fill was generally observed to consist of crushed stone and brown silty clay with trace of gravel.

An approximate 1.2 to 2.4 m thick, hard brown silty clay layer was encountered underlying the fill. Further, an approximate 0.7 to 0.8 m thick glacial till deposit was encountered underlying the silty clay layer at boreholes BH 2-31 and BH 3-31. The glacial till was observed to consist of a dense, grey silty clay with sand, gravel, cobbles, and boulders.

Practical refusal to augering was encountered in the boreholes at approximate depths ranging from 2.2 to 3.7 m below the existing ground surface.

Bedrock

Reviewing the available geological mapping, the subject site is located in an area where the bedrock consists of interbedded limestone and dolomite of the gull river formation, which has an overburden drift thickness of 2 to 3 m.

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

4.3 Groundwater

Groundwater levels (GWL) were measured on October 28, 2021 in the standpipe piezometers which were installed during the field investigation. The measured GWL readings are presented in Table 1 on the next page.

Table 1 - Groundwater Level Readings				
Borehole Number	Ground Elevation (m)	Groundwater Levels		Recording Date
		Depth (m)	Elevation (m)	
BH 1-21	72.33	1.96	70.37	October 28, 2021
BH 2-21	72.37	2.07	70.30	October 28, 2021
BH 3-21	72.60	2.10	70.50	October 28, 2021

Note: The ground surface elevation from the current investigation are referenced to a geodetic datum

Based on these observations, the long-term groundwater table can be expected at approximate depths of 2 to 2.5 m below the existing ground surface.

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

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is suitable for the proposed building. It is recommended that the proposed building be founded on conventional spread footings placed on clean, surface sounded bedrock.

If the clean, surface sounded bedrock is encountered below the underside of footing (USF) elevation, then lean concrete can be used to raise grades from the clean, surface sounded bedrock to the USF elevation. Alternatively, the footings can be supported on the hard to very stiff silty clay and/or the stiff/dense glacial till.

Dependent on the specific foundation depth of the proposed building, and the depths of utilities, bedrock removal may be required for the building excavation and installation of site services. This is discussed further in Section 5.2.

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

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

Bedrock Removal

Should bedrock removal be required, hoe ramming is an option where the bedrock is weathered and/or where only small quantities of bedrock need to be removed. Where large quantities of bedrock need to be removed, line drilling and controlled blasting may be required. The blasting operations should be planned and completed under the guidance of a professional engineer with experience in blasting operations.

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 conducted prior to commencing construction.

The extent of the survey should be determined by the blasting consultant and sufficient to respond to any inquiries/claims related to the blasting operations. The blasting operations should be planned and conducted under the supervision of a licensed professional engineer who is also an experienced blasting consultant.

Vibration Considerations

Construction operations could be the cause of vibrations, and possibly, sources of nuisance to the community. Therefore, means to reduce the vibration levels as much as possible should be incorporated in the construction operations to maintain a cooperative environment with the residents.

The following construction equipment could be the source of vibrations: hoe ram, compactor, dozer, crane, truck traffic, etc. Vibrations, whether caused by blasting operations or by construction operations, could be the cause or the source of detrimental vibrations on the nearby buildings and structures. Therefore, it is recommended that all vibrations be limited.

Two parameters determine the permissible vibrations, the maximum peak particle velocity and the frequency. For low frequency vibrations, the maximum allowable peak particle velocity is less than that for high frequency vibrations. As a guideline, the peak particle velocity should be less than 15 mm/s between frequencies of 4 to 12 Hz, and 50 mm/s above a frequency of 40 Hz (interpolate between 12 and 40 Hz). These guidelines are for current construction standards. These guidelines are above perceptible human level and, in some cases, could be very disturbing to some people, therefore, a pre-construction survey is recommended to minimize the risks of claims during or following the construction of the proposed building.

Fill Placement

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

Non-specified existing fill, along with site-excavated soil, can be used as general landscaping fill where settlement of the ground surface is of minor concern. This material should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids. If this material is to be used to build up the subgrade level for areas to be paved, it should be compacted in thin lifts to at least 95% of the material's SPMDD.

Non-specified existing fill and site-excavated soils are not suitable for use as backfill against foundation walls unless used in conjunction with a composite drainage membrane.

Lean Concrete Filled Trenches

Where the proposed footings are designed to bear on clean, surface sounded bedrock which is encountered below the USF elevation, lean concrete should be placed from the clean, surface sounded bedrock to the founding elevation (minimum **17 MPa** 28-day compressive strength). Typically, the excavation side walls will be used as the form to support the concrete. The trench excavation should be at least 150 mm wider than all sides of the footing (strip and pad footings) at the base of the excavation. The additional width of the concrete poured against an undisturbed trench sidewall will suffice in providing a direct transfer of the footing load to the underlying bedrock. Once the trench excavation is approved by the geotechnical engineer, lean concrete can be poured up to the proposed founding elevation.

5.3 Foundation Design

Bearing Resistance Values

Overburden

Footings placed on an undisturbed, hard to very stiff silty clay or stiff/dense glacial till bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **200 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **300 kPa**. A geotechnical resistance factor of 0.5 was applied to the reported bearing resistance value at ULS.

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

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

Bedrock

Footings placed on clean, surface sounded bedrock, or on lean concrete which is placed directly over clean, surface sounded bedrock, can be designed using a factored bearing resistance value at ultimate limit states (ULS) of **1,500 kPa**. A geotechnical resistance factor of 0.5 was applied to the bearing resistance value at ULS.

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

Footings bearing on an acceptable bedrock bearing surface and designed for the bearing resistance values provided herein will be subjected to negligible potential post-construction total and differential settlements.

Bedrock/Soil Transition

Where a building is founded partly on bedrock and partly on soil, it is recommended to decrease the soil bearing resistance value by 25% for the footings placed on a soil bearing medium to reduce the potential long-term total and differential settlements. At the soil/bedrock transitions, it is recommended that a minimum depth of 500 mm of bedrock be removed from below the founding elevation for a minimum length of 2 m on the bedrock side.

This area should be subsequently reinstated with an engineered fill, such as OPSS Granular A or Granular B Type II and compacted to a minimum of 98% of the material SPMDD.

Lateral Support

The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels. Adequate lateral support is provided to a very stiff silty clay or stiff/dense glacial till above the groundwater table when a plane extending down and out from the bottom edge of the footing at a minimum of 1.5H:1V passes only through in situ soil of the same or higher capacity as the bearing medium soil.

Adequate lateral support is provided to a sound bedrock bearing medium when a plane extending down and out from the bottom edge of the footing at a minimum of 1H:6V (or flatter) passes only through sound bedrock or a material of the same or higher capacity as the bedrock, such as concrete. A weathered bedrock bearing medium will require a lateral support zone of 1H:1V (or flatter).

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class C**. If a higher seismic site class is required (Class A or B), a site specific shear wave velocity test may be completed to accurately determine the applicable seismic site classification for foundation design of the proposed building, as presented in Table 4.1.8.4.A of the Ontario Building Code 2012.

Soils underlying the subject site are not susceptible to liquefaction. Reference should be made to the latest revision of the Ontario Building Code 2012 for a full discussion of the earthquake design requirements.

5.5 Basement Floor Slab

With the removal of all topsoil and fill, containing significant amounts of deleterious or organic materials, the silty clay, glacial till or bedrock medium approved by Paterson personnel at the time of excavation will be considered an acceptable subgrade surface on which to commence backfilling for basement slab construction.

Any soft areas should be removed and backfilled with appropriate backfill material prior to placing any fill. OPSS Granular B Type II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab.

It is recommended that the upper 200 mm of sub-floor fill consist of 19 mm clear crushed stone. Further, an underslab drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided underlying the basement slab. This is discussed further in Section 6.1.

5.6 Basement Wall

There are several combinations of backfill materials and retained soils that could be applicable for the basement walls of the subject structure. However, 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 bulk (drained) unit weight of 20 kN/m³.

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

Lateral Earth Pressures

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

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

An additional pressure having a magnitude equal to $K_o \cdot q$ and acting on the entire height of the wall should be added to the above diagram for any surcharge loading, q (kPa), that may be placed at ground surface adjacent to the wall.

The surcharge pressure will only be applicable for static analyses and should not be used in conjunction with the seismic loading case.

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

Seismic Earth Pressures

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

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

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

The peak ground acceleration, (a_{\max}), for the Ottawa area is $0.32g$ according to OBC 2012. Note that the vertical seismic coefficient is assumed to be zero.

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

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

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

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

5.7 Pavement Structure

The recommended pavement structures for car only parking areas, heavy truck parking areas and access lanes are presented in Tables 2 and 3, should they be required at the subject site.

Table 2 - Recommended Pavement Structure - Car Only Parking Areas	
Thickness (mm)	Material Description
50	Wear Course - HL-3 or Superpave 12.5 Asphaltic Concrete
150	BASE - OPSS Granular A Crushed Stone
300	SUBBASE - OPSS Granular B Type II
SUBGRADE - Either fill, in situ soil, or OPSS Granular B Type I or II material placed over in situ soil or fill	

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

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.

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for this project. The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 99% of the material's SPMDD using suitable vibratory equipment.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

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

Underslab Drainage

Underslab drainage is recommended to control water infiltration below the basement slab. For preliminary design purposes, we recommend that 100 or 150 mm perforated pipes be placed at approximate 6 m centres underlying the basement slab. The spacing of the underslab 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. The greater part of the site excavated materials will be frost susceptible and, as such, are not recommended for re-use as backfill against the foundation walls, unless used in conjunction with a drainage geocomposite, such as Delta Drain 6000, connected to the perimeter foundation drainage system. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should otherwise be used for this purpose. A waterproofing system should be provided for any elevator pits (pit bottom and walls).

6.2 Protection of Footings Against Frost Action

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

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

6.3 Excavation Side Slopes

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

Unsupported Excavations

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be excavated at 1H:1V or shallower. The shallower slope is required for excavation below groundwater level. The subsurface soils are considered to be a Type 2 and 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects.

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

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

A trench box is recommended to protect personnel working in trenches with steep or vertical sides. Services are expected to be installed by “cut and cover” methods and excavations should not remain open for extended periods of time.

Temporary Shoring

Temporary shoring may be required to support the overburden soils, dependent on the final founding depth of the building and its proximity to the property lines. The design and approval of the temporary shoring system will be the responsibility of the shoring contractor and the shoring designer who is a licensed professional engineer and is hired by the shoring contractor. It is the responsibility of the shoring contractor to ensure that the temporary shoring is in compliance with safety requirements, designed to avoid any damage to adjacent structures and include dewatering control measures.

In the event that subsurface conditions differ from the approved design during the actual installation, it is the responsibility of the shoring contractor to commission the required experts to re-assess the design and implement the required changes. The designer should also 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.

The temporary shoring system may consist of a soldier pile and lagging system which could be cantilevered, anchored or braced.

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

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

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

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

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

Underpinning of Adjacent Structures

Due to the shallow depth to bedrock, it is expected that the existing buildings to the north and south of the subject site are founded on bedrock. Therefore, underpinning of these buildings is not expected to be required.

However, an assessment should be completed by the geotechnical engineer at the time of excavation to confirm founding conditions of the existing buildings adjacent to the proposed building, in order to evaluate rock bolt locations and specific rock bolt details, should they be required.

Should the existing building adjacent to the proposed building not be founded on bedrock, underpinning may be required.

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 a soil subgrade. If the bedding is placed on bedrock, the thickness of the bedding should be increased to 300 mm for sewer pipes. 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.

It should generally be possible to re-use the site materials above the cover material if the operations are carried out in dry weather conditions.

Where hard surface areas are considered above the trench backfill, the trench backfill material within the frost zone (about 1.5 m below finished grade) and above the cover material should match the soils exposed at the trench walls to minimize differential frost heaving. The trench backfill should be placed in maximum 225 mm thick loose lifts and compacted to a minimum of 95% of the material standard Proctor maximum dry density.

6.5 Groundwater Control

It is anticipated that groundwater infiltration into the excavations should be controllable using open sumps. 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.

Groundwater Control for Building Construction

A temporary Ministry of Environment, Conservation and Parks (MECP) permit to take water (PTTW) may be required if more than 400,000 L/day of ground and/or surface water are to be pumped during the construction phase. At least 4 to 5 months should be allowed for completion of the application and issuance of the permit by the MECP. For typical ground or surface water volumes being pumped during the construction phase, typically between 50,000 to 400,000 L/day, it is required to register on the Environmental Activity and Sector Registry (EASR). A minimum of two to four weeks should be allotted for completion of the EASR registration and the Water Taking and

Discharge Plan to be prepared by a Qualified Person as stipulated under O.Reg. 63/16. If a project qualifies for a PTTW based upon anticipated conditions, an EASR will not be allowed as a temporary dewatering measure while awaiting the MECP review of the PTTW application.

Impacts on Neighbouring Properties

Based on the proposed USF elevation, the excavation may extend slightly below the groundwater table. However, the neighbouring structures are expected to be founded on hard to very stiff silty clay, stiff/dense glacial till, and/or bedrock, which are not susceptible to affects from groundwater lowering. Therefore, no issues are expected with respect to groundwater lowering that would cause long term adverse effects to adjacent structures surrounding the proposed building.

6.6 Winter Construction

Precautions must be taken if winter construction is considered for this project. The subsoil conditions at this site mostly consist of frost susceptible materials. In 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, 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.

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

6.7 Corrosion Potential and Sulphate

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

7.0 Recommendations

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

- Observation of all bearing surfaces prior to the placement of concrete.
- Sampling and testing of the concrete and fill materials used.
- Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- Observation of all subgrades prior to backfilling.
- Field density tests to determine the level of compaction achieved.
- Sampling and testing of the bituminous concrete including mix design reviews.

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

8.0 Statement of Limitations

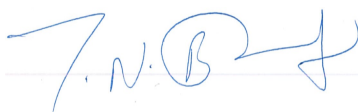
The recommendations provided in this report are in accordance with our present understanding of the project. We request permission to review our recommendations when the drawings and specifications are completed.

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

The 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 its 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 2707120 Ontario Inc. or their agents is not authorized without review by Paterson for the applicability of our recommendations to the altered use of the report.

Paterson Group Inc.



Balaji Nirmala, M.Eng.



Scott S. Dennis, P.Eng.

Report Distribution

- 2707120 Ontario Inc. (e-mail copy)
- Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE October 20, 2021

FILE NO. **PG5960**

HOLE NO. **BH 1-21**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone				Piezometer Construction	
		TYPE	NUMBER	RECOVERY %	N VALUE or RQD			20	40	60	80		
GROUND SURFACE						0	72.33						
TOPSOIL	0.20												
FILL: Brown silty clay, trace sand and gravel	0.46	AU	1										
Hard to very stiff, brown SILTY CLAY		SS	2	50	27	1	71.33						
		SS	3	100	15	2	70.33						
End of Borehole	2.23												
Practical refusal to augering at 2.23m depth. (GWL @ 1.96m - October 28, 2021)													
								20	40	60	80	100	
								Shear Strength (kPa)					
								▲ Undisturbed △ Remoulded					

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE October 20, 2021

FILE NO. **PG5960**

HOLE NO. **BH 2-21**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone				Piezometer Construction	
		TYPE	NUMBER	RECOVERY %	N VALUE or RQD			20	40	60	80		
GROUND SURFACE													
Asphaltic concrete	0.05					0	72.37						
FILL: Crushed stone	0.20												
FILL: Brown silty clay, trace gravel	0.46												
Hard to very stiff, brown SILTY CLAY		AU	1										
		SS	2	42	11	1	71.37						
		SS	3	83	23	2	70.37						
		SS	4	75	10								
GLACIAL TILL: Stiff, grey silty clay with sand, gravel, cobbles and boulders	2.90					3	69.37						
	SS	5	50	8									
End of Borehole	3.71												
Practical refusal to augering at 3.71m depth. (GWL @ 2.07m - October 28, 2021)													
								20	40	60	80	100	
								Shear Strength (kPa)					
								▲ Undisturbed △ Remoulded					

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE October 20, 2021

FILE NO. **PG5960**

HOLE NO. **BH 3-21**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone				Piezometer Construction	
		TYPE	NUMBER	RECOVERY %	N VALUE or RQD			20	40	60	80		
GROUND SURFACE													
Asphaltic concrete FILL: Crushed stone	0.05					0	72.60						
FILL: Brown silty clay, trace gravel and topsoil	0.30 0.46	AU	1										
Hard to very stiff, brown SILTY CLAY		SS	2	58	17	1	71.60						
		SS	3	67	22	2	70.60						
GLACIAL TILL: Dense, grey silty sand with gravel, cobbles and boulders	2.21	SS	4	29	50+								
End of Borehole Practical refusal to augering at 2.92m depth. (GWL @ 2.10m - October 28, 2021)	2.92												

○ Water Content %

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 = $(D_{30})^2 / (D_{10} \times D_{60})$
Cu	-	Uniformity coefficient = D_{60} / D_{10}

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		Overconsolidation 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.
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SYMBOLS AND TERMS (continued)

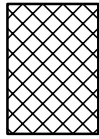
STRATA PLOT



Topsoil



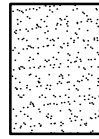
Asphalt



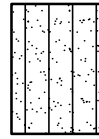
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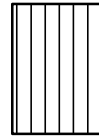
Peat



Sand



Silty Sand



Silt



Sandy Silt



Clay



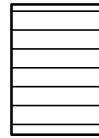
Silty Clay



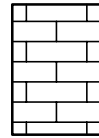
Clayey Silty Sand



Glacial Till



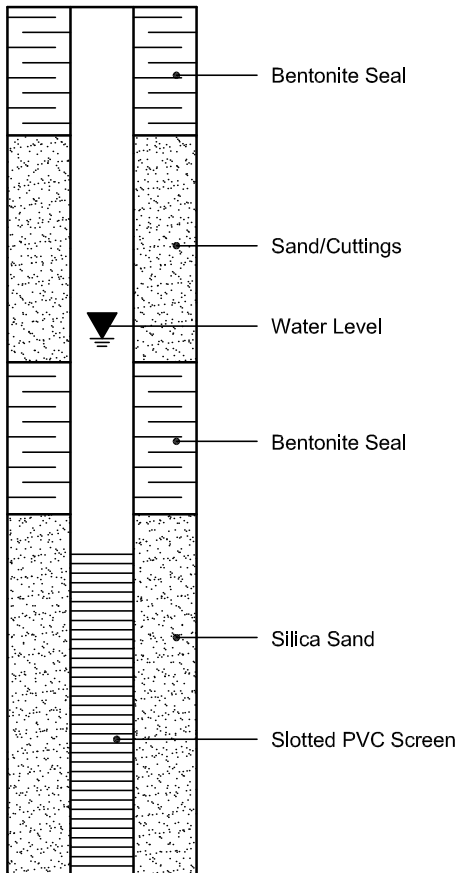
Shale



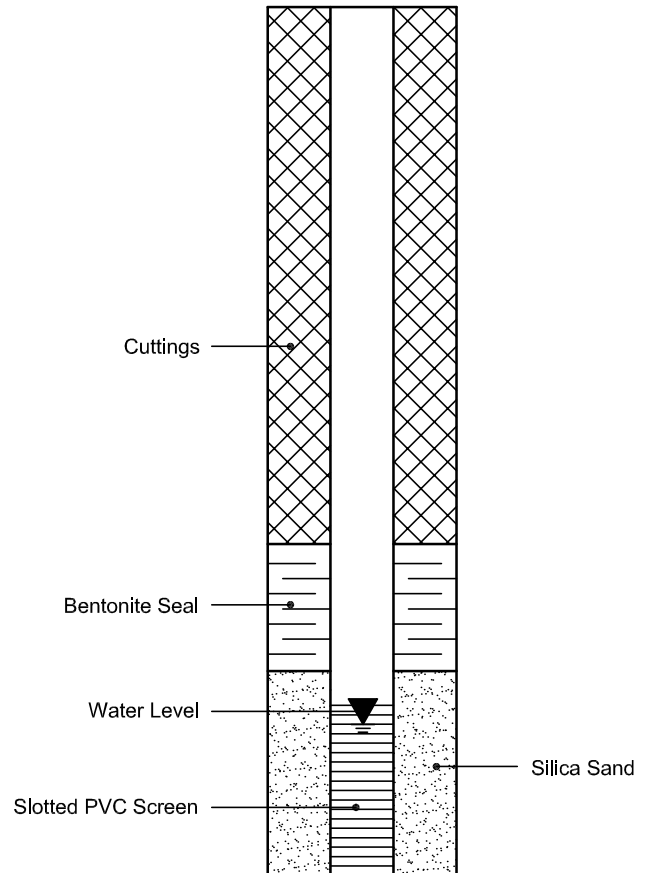
Bedrock

MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Report Date: 28-Oct-2021

Client: Paterson Group Consulting Engineers

Order Date: 22-Oct-2021

Client PO: 33316

Project Description: PG5960

Client ID:	BH3-21 SS3	-	-	-
Sample Date:	21-Oct-21 09:00	-	-	-
Sample ID:	2143614-01	-	-	-
MDL/Units	Soil	-	-	-

Physical Characteristics

% Solids	0.1 % by Wt.	67.7	-	-	-
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General Inorganics

pH	0.05 pH Units	7.18	-	-	-
Resistivity	0.10 Ohm.m	5.01	-	-	-

Anions

Chloride	5 ug/g dry	1140	-	-	-
Sulphate	5 ug/g dry	184	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING PG5960-1 - TEST HOLE LOCATION PLAN

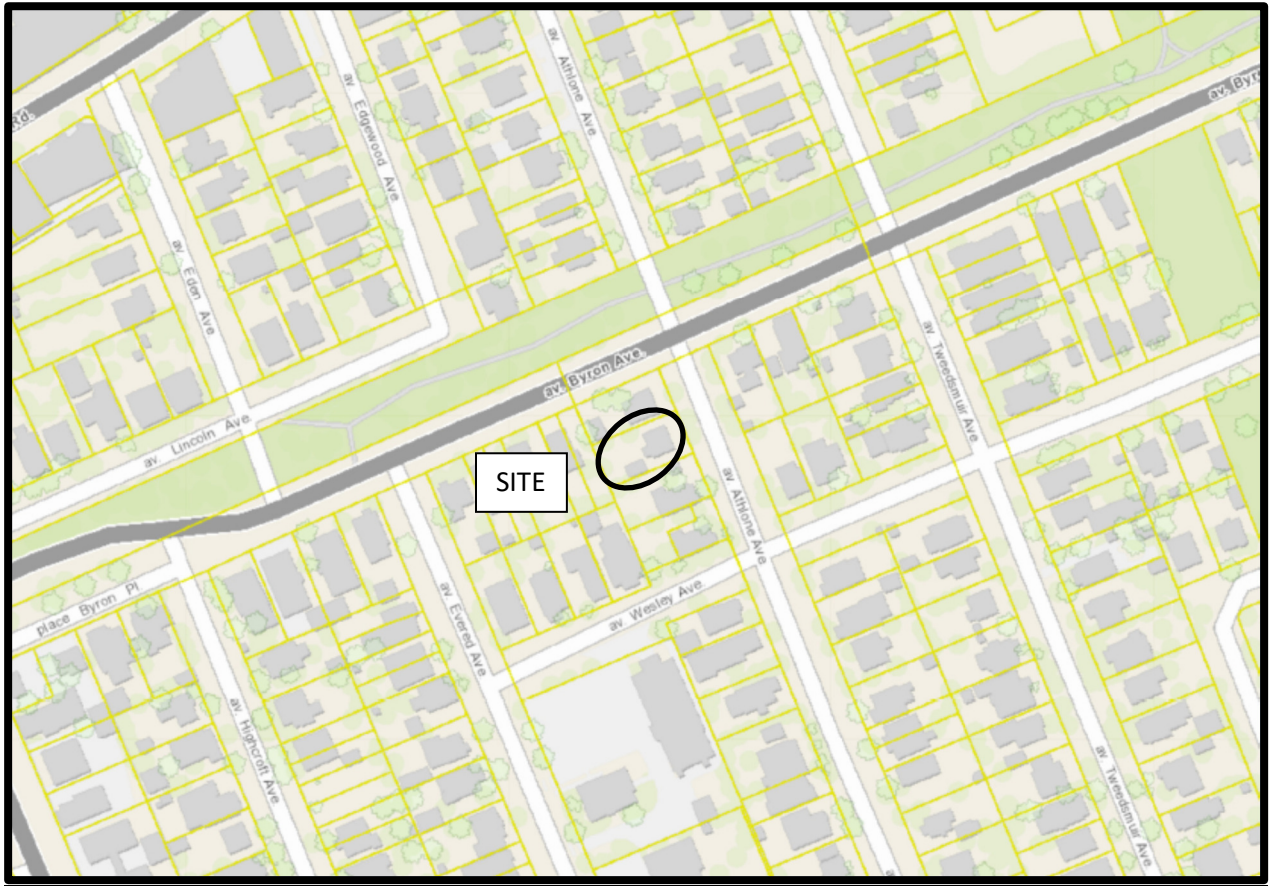
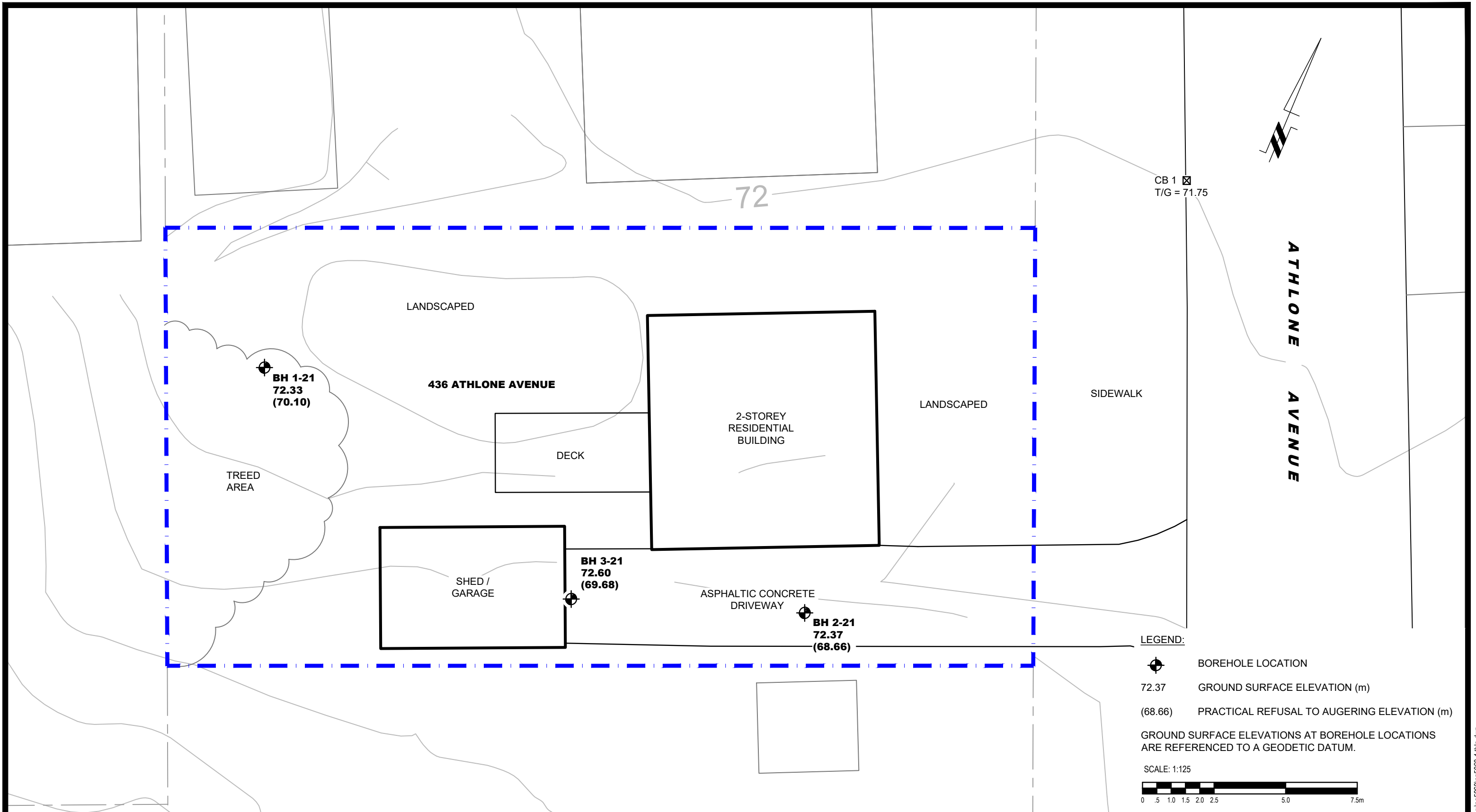


FIGURE 1

KEY PLAN



patersongroup
consulting engineers

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

NO.	REVISIONS	DATE	INITIAL

OTTAWA,
Title:

HONEY CONSTRUCTION
GEOTECHNICAL INVESTIGATION
PROPOSED DEVELOPMENT
436 ATHLONE AVENUE

ONTARIO

TEST HOLE LOCATION PLAN

Scale: 1:125

Date: 10/2021

Drawn by: JM

Report No.: PG5960-1

Checked by: SD

Dwg. No.: **PG5960-1**

Approved by: MSD

Revision No.: