

Geotechnical  
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Materials Testing

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Noise and Vibration  
Studies

**Geotechnical Investigation**  
Proposed Residential Building  
138 Forward Avenue  
Ottawa, Ontario

Prepared For

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Report: PG6026-1

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## 1.0 Introduction

Paterson Group (Paterson) was commissioned by VIKA Land Development Group Inc. to conduct a geotechnical investigation for the proposed residential building site to be located at 138 Forward Avenue in the City of Ottawa (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 test holes.
- 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.

## 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 one basement level, which will occupy most of the subject site.

Associated walkways and landscaped areas are anticipated surrounding the proposed building. It is also expected that the proposed building will be municipally serviced.

## **3.0 Method of Investigation**

### **3.1 Field Investigation**

#### **Field Program**

The field program for the current geotechnical investigation was carried out on November 2, 2021 and consisted of advancing a total of 3 boreholes to a maximum depth of 0.9 m below existing grade. The test hole locations were distributed in a manner to provide general coverage of the subject site and taking into consideration underground utilities and site features. The borehole locations are shown on Drawing PG6026-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were drilled using a low clearance 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 drilling to the required depths at the selected locations, and sampling and testing the overburden.

#### **Sampling and In Situ Testing**

The soil samples were recovered from the auger flights and using a 50 mm diameter split-spoon sampler. The samples were initially classified on site, placed in sealed plastic bags, and transported to our laboratory. 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 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.

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

#### **Sample Storage**

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.

### **3.2 Field Survey**

The borehole 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 location of the boreholes and ground surface elevation at each test hole location are presented on Drawing PG6026-1 - Test Hole Location Plan in Appendix 2.

### **3.3 Laboratory Testing**

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

### **3.4 Analytical Testing**

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

## **4.0 Observations**

### **4.1 Surface Conditions**

The subject site is currently occupied by a two-storey residential building, which is surrounded by asphalt-paved parking areas. The site is bordered by Forward Avenue to the east, and residential properties to the north, south and west. The existing ground surface across the site is relatively level at approximate geodetic elevation 62 m.

### **4.2 Subsurface Profile**

#### **Overburden**

Generally, the soil profile at the borehole locations consists of fill, extending to approximate depths of 0.6 to 0.9 m below the existing ground surface, where practical refusal of the augers was encountered on the inferred bedrock surface. The fill was generally observed to consist of a crushed stone with some sand and occasional traces of clay.

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

#### **Bedrock**

Based on available geological mapping, the bedrock in the subject area consists of limestone of the Bobcaygeon formation.

### **4.3 Groundwater**

Groundwater was not observed in the completed boreholes prior to backfilling. However, based on our experience at an adjacent site, the groundwater level is expected at approximate depths of 2 to 3 m below the existing ground surface.

It should be noted that groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater levels 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 development. The proposed building is recommended to be founded on conventional spread footings placed on clean, surface sounded bedrock.

Bedrock removal will be required to construct the basement level. Hoe ramming is an option where the bedrock is weathered and/or where only small quantities of bedrock need to be removed. Line drilling and controlled blasting may be required where large quantities of bedrock need to be removed. The blasting operations should be planned and completed under the guidance of a professional engineer with experience in blasting operations.

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

### **5.2 Site Grading and Preparation**

#### **Stripping Depth**

Due to the depth of the bedrock at the subject site and the anticipated founding level for the proposed multi-storey building, it is anticipated that all existing overburden material will be excavated from within the footprint of the proposed multi-storey building.

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 excavated to a minimum of 1 m below final grade.

#### **Bedrock Removal**

As noted above, bedrock removal can be accomplished by hoe ramming where the bedrock is weathered and/or where only small quantities of the bedrock need to be removed. Sound bedrock may be removed by line drilling in conjunction with 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 the 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 or claims related to the blasting operations.

As a general guideline, peak particle velocities (measured at the structures) should not exceed 25 mm/s during the blasting program to reduce the risks of damage to the existing surrounding structures. 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 are also the cause of vibrations, and possibly, sources of nuisance to the community. Therefore, means to reduce the vibration levels should be incorporated in the construction operations to maintain, as much as possible, a cooperative environment with the residents.

The following construction equipment could be a 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 of the source of detrimental vibrations on the nearby buildings and structures. Therefore, it is recommended that all vibrations be limited.

Two parameters are used to determine the permissible vibrations, namely, 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).

It should be noted that these guidelines are for today's construction standards. Considering that these guidelines are above perceptible human level and, in some cases, could be very disturbing to some people, it is recommended that a pre-construction survey be completed to minimize the risks of claims during or following the construction of the proposed building.

### **Fill Placement**

Fill used for grading beneath the building area should consist, unless otherwise specified, of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. The imported fill material should be tested and approved prior to delivery to the site. The fill should be placed in maximum 300 mm thick loose lifts and compacted by suitable compaction

equipment. Fill placed beneath the building should be compacted to a minimum of 98% of the standard Proctor maximum dry density (SPMDD).

Non-specified existing fill along with site-excavated soil could be placed as general landscaping fill and beneath exterior parking where settlement of the ground surface is of minor concern. These materials should be spread in lifts with a maximum thickness of 300 mm and compacted by the tracks of the spreading equipment to minimize voids.

Non-specified existing fill and site-excavated soils are not suitable for placement as backfill against foundation walls, unless used in conjunction with a geocomposite drainage membrane, such as Miradrain G100N or Delta Drain 6000.

## **5.3 Foundation Design**

### **Bearing Resistance Values**

Footings placed on clean, surface sounded bedrock can be designed using a bearing resistance value at ultimate limits states (ULS) of **1,000 kPa**. A geotechnical factor of 0.5 was applied to the above noted bearing resistance value. Minimum dimensions of 1 m and 0.5 m should be provided for all spread and strip footings, respectively.

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.

### **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 sound bedrock bearing medium when a plane extending down and out from the bottom edge of the footing at 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** for foundations constructed at the subject site. A higher site class, such as Class A or B, may be provided for foundations placed on bedrock. However, the higher site class would need to be confirmed by a site-specific seismic shear wave velocity test.

The soils underlying the subject site are not susceptible to liquefaction. 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 Floor Slab

All overburden soil will be removed from the subject site leaving the bedrock as the founding medium for the basement floor slab. It is recommended that the upper 200 mm of sub-slab fill consists of 19 mm clear crushed stone.

In consideration of the groundwater conditions at the site, an underslab drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided in the 19 mm clear crushed stone layer under the lower basement floor. This is discussed further in Subsection 6.1.

## 5.6 Basement Wall

There are several combinations of backfill materials and retained soils that could be applicable for the basement walls of the subject structure. However, 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/m<sup>3</sup>.

Two distinct conditions, static and seismic, should be reviewed for design calculations. The corresponding parameters are presented below.

### 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 material (0.5)
- $\gamma$  = unit weight of fill of the applicable retained soil (kN/m<sup>3</sup>)
- $H$  = height of the wall (m)

An additional pressure having a magnitude equal to  $K_0 \cdot q$  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 ( $\Delta P_{AE}$ ).

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

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

$$\gamma = \text{unit weight of fill of the applicable retained soil (kN/m}^3\text{)}$$

$$H = \text{height of the wall (m)}$$

$$g = \text{gravity, 9.81 m/s}^2$$

The peak ground acceleration, ( $a_{max}$ ), for the Ottawa area is 0.32 g 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  $P_o = 0.5 K_0 \gamma H^2$ , where  $K_0 = 0.5$  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 Design**

For design purposes, the pavement structures presented in the following tables are recommended for the design of car only parking areas and access lanes, should they be required as part of the proposed development.

<b>Table 1 - Recommended Pavement Structure - Car Only Parking Areas</b>	
<b>Thickness (mm)</b>	<b>Material Description</b>
50	<b>Wear Course</b> - HL-3 or Superpave 12.5 Asphaltic Concrete
150	<b>BASE</b> - OPSS Granular A Crushed Stone
300	<b>SUBBASE</b> - OPSS Granular B Type II
<b>SUBGRADE</b> - Either fill, in situ soil, or OPSS Granular B Type I or II material placed over in situ soil or fill	

<b>Table 2 - Recommended Pavement Structure Access Lanes and Heavy Truck Parking Areas</b>	
<b>Thickness (mm)</b>	<b>Material Description</b>
40	<b>Wear Course</b> – HL-3 or Superpave 12.5 Asphaltic Concrete
50	<b>Binder Course</b> – HL-8 or Superpave 19.0 Asphaltic Concrete
150	<b>BASE</b> - OPSS Granular A Crushed Stone
450	<b>SUBBASE</b> - OPSS Granular B Type II
<b>SUBGRADE</b> - Either fill, in situ soil, or OPSS Granular B Type I or II material placed over in situ soil or fill	

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for this project. 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, noting that excessive compaction can result in subgrade softening.

If bedrock is encountered at the subgrade level, the total thickness of the pavement granular materials (base and subbase) could be reduced to 300 mm. The upper 300 mm of the bedrock surface should be reviewed and approved by Paterson prior to placing the base and subbase materials. Care should be exercised to ensure that the bedrock subgrade does not have depressions that will trap water.

## **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 plastic pipe, surrounded on all sides by 150 mm of 19 mm clear crushed stone, which is placed at the footing level around the exterior perimeter of the structure. The clear crushed stone layer should be wrapped in a non-woven geotextile. The pipe should have a positive outlet, such as a gravity connection to the storm sewer.

Where insufficient room is available for exterior backfill, it is suggested that the composite drainage system (such as Delta Drain 6000 or equivalent) be secured against the vertical bedrock face extending to a series of drainage sleeves inlets through the building foundation wall at the footing/foundation wall interface. The drainage sleeves should be at least 150 mm diameter and be spaced 3 m along the perimeter foundation walls. An interior perimeter drainage pipe should be placed along the building perimeter along with the underslab drainage system. The perimeter drainage pipe and sub-slab drainage system should direct water to sump pit(s) within the lower underground area.

#### **Underslab Drainage**

Sub-slab drainage is recommended to control water infiltration below the basement slab. For preliminary design purposes, we recommend that 100 to 150 mm diameter perforated PVC pipes be placed at 3 to 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**

Where sufficient space is available for conventional backfilling, backfill against the exterior sides of the foundation wall 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.

## **6.2 Protection of Footings Against Frost Action**

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

Other exterior unheated footings, such as those for isolated exterior, are more prone to deleterious movement associated with frost action. These should be provided with a minimum 2.1 m thick soil cover (or insulation equivalent).

However, foundations which are founded directly on clean, surface-sounded bedrock with no cracks or fissures, and which is approved by Paterson at the time of construction, is not considered frost susceptible and does not require soil cover.

## **6.3 Excavation Side Slopes**

The side slopes of shallow excavations anticipated at this site should either be cut back at acceptable slopes or be retained by temporary shoring systems from the start of the excavation until the structure is backfilled. Given the limited overburden encountered at this site, it is expected that there will be sufficient space to slope the overburden, followed by a vertical excavation in the underlying bedrock.

### **Unsupported Excavations**

The excavation side slopes in the overburden and 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 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.

## **Rock stabilization**

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 or to provide a stable base for the overburden shoring system.

Horizontal rock anchors may be required at specific locations to prevent pop-outs of the bedrock, especially in areas where fractures in the bedrock are conducive to the failure of the bedrock surface.

The requirements for horizontal rock anchors and bedrock stabilization measures will be evaluated during the excavation program and determined by Paterson at the time of construction.

## **Underpinning**

Considering the shallow depth to bedrock, it is expected that the adjacent buildings are founded on bedrock. Therefore, underpinning is not expected to be required at this site. 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.

## **6.4 Pipe Bedding and Backfill**

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

At least 300 mm of OPSS Granular A should be used for pipe bedding for sewer and water pipes. The bedding should extend to the spring line of the pipe. Cover material, from the spring line to at least 300 mm above the obvert of the pipe, should consist of OPSS Granular A or Granular B Type II with a maximum size of 25 mm. The bedding and cover materials should be placed in maximum 225 mm thick lifts compacted to 99% of the material's standard Proctor maximum dry density.

Well fractured bedrock should be acceptable as backfill for the lower portion of the trenches when the excavation is within bedrock provided the rock fill is placed only from at least 300 mm above the top of the service pipe and that all stones are 300 mm or smaller in their longest dimension.



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

## **6.5 Groundwater Control**

### **Groundwater Control for Building Construction**

Based on our observations, it is anticipated that groundwater infiltration into the excavations should be low and 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.

### **Permit to Take Water**

A temporary Ministry of the Environment, Conservation and Parks (MECP) permit to take water (PTTW) may be required for this project if more than 400,000 L/day of ground and/or surface water is to be pumped during the construction phase. A minimum 4 to 5 months should be allowed for completion of the PTTW application package and issuance of the permit by the 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 existing groundwater level and the depth of the proposed building, groundwater lowering is not expected to be required as part of construction. Further, due to the presence of shallow bedrock at, and in the vicinity of, the subject site, the neighbouring structures are expected to be founded on bedrock. 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 consist of frost susceptible materials. In the presence of water and freezing conditions, ice could form within the soil mass. Heaving and settlement upon thawing could occur.

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

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

## **6.7 Corrosion Potential and Sulphate**

The results of 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 low to slightly aggressive corrosive environment.

## 7.0 Recommendations

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

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

A report confirming that these works have been conducted in general accordance with our recommendations could be issued upon the completion of a satisfactory inspection program by the geotechnical consultant.

## 8.0 Statement of Limitations

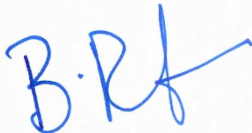
The recommendations provided are in accordance with the present understanding of the project. Paterson requests permission to review the recommendations when the drawings and specifications are completed.

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

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

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than the VIKA Land Development Group Inc., 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.



Puneet Bandi, MSc (Eng)



Scott S. Dennis, P.Eng.

### Report Distribution:

- VIKA Land Development Group Inc. (email copy)
- Paterson Group (1 copy)

# APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS









# 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.

<b>RQD %</b>	<b>ROCK QUALITY</b>
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

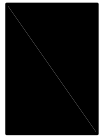
$p'_o$	-	Present effective overburden pressure at sample depth
$p'_c$	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below $p'_c$ )
Cc	-	Compression index (in effect at pressures above $p'_c$ )
OC Ratio		Overconsolidation ratio = $p'_c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

### PERMEABILITY TEST

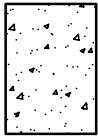
k	-	Coefficient of permeability or hydraulic conductivity is a measure of the ability of water to flow through the sample. The value of k is measured at a specified unit weight for (remoulded) cohesionless soil samples, because its value will vary with the unit weight or density of the sample during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

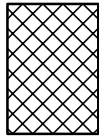
### STRATA PLOT



Topsoil



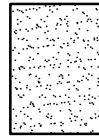
Asphalt



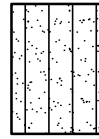
Fill



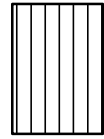
Peat



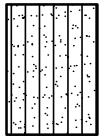
Sand



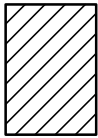
Silty Sand



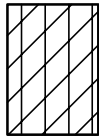
Silt



Sandy Silt



Clay



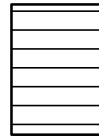
Silty Clay



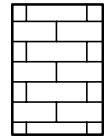
Clayey Silty Sand



Glacial Till



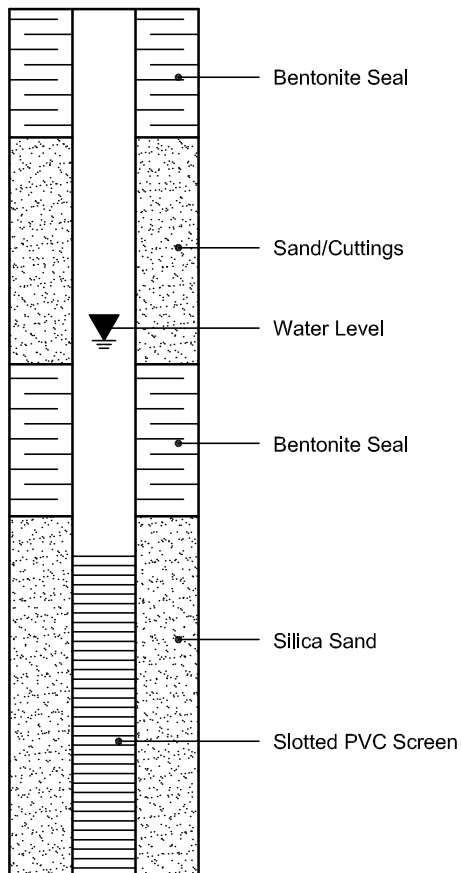
Shale



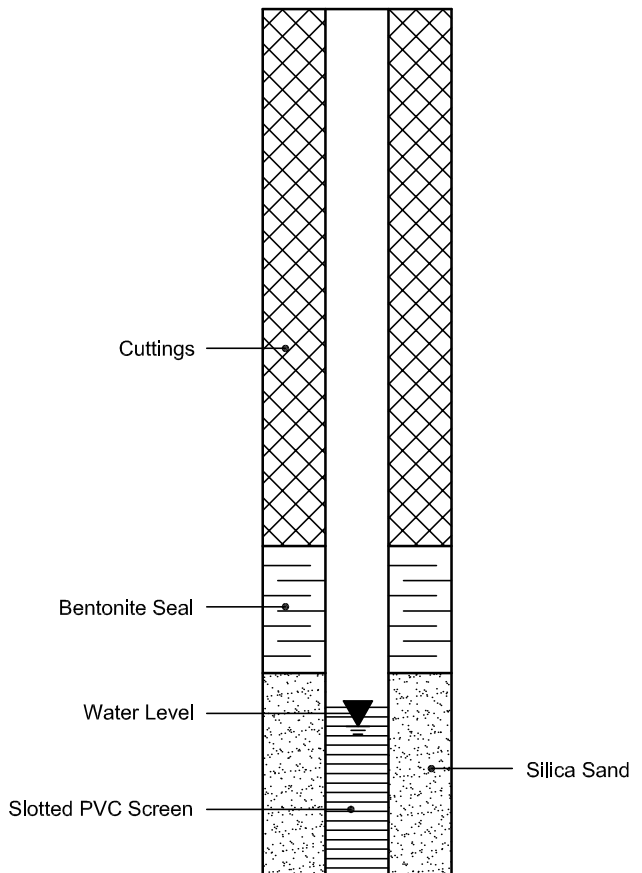
Bedrock

### MONITORING WELL AND PIEZOMETER CONSTRUCTION

#### MONITORING WELL CONSTRUCTION



#### PIEZOMETER CONSTRUCTION



Certificate of Analysis

Report Date: 09-Nov-2021

Client: Paterson Group Consulting Engineers

Order Date: 3-Nov-2021

Client PO: 33343

Project Description: PG6026

<b>Client ID:</b>	BH3-21 AU1 1'-2'	-	-	-
<b>Sample Date:</b>	02-Nov-21 09:00	-	-	-
<b>Sample ID:</b>	2145331-01	-	-	-
<b>MDL/Units</b>	Soil	-	-	-

**Physical Characteristics**

% Solids	0.1 % by Wt.	92.8	-	-	-
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**General Inorganics**

pH	0.05 pH Units	7.46	-	-	-
Resistivity	0.10 Ohm.m	82.0	-	-	-

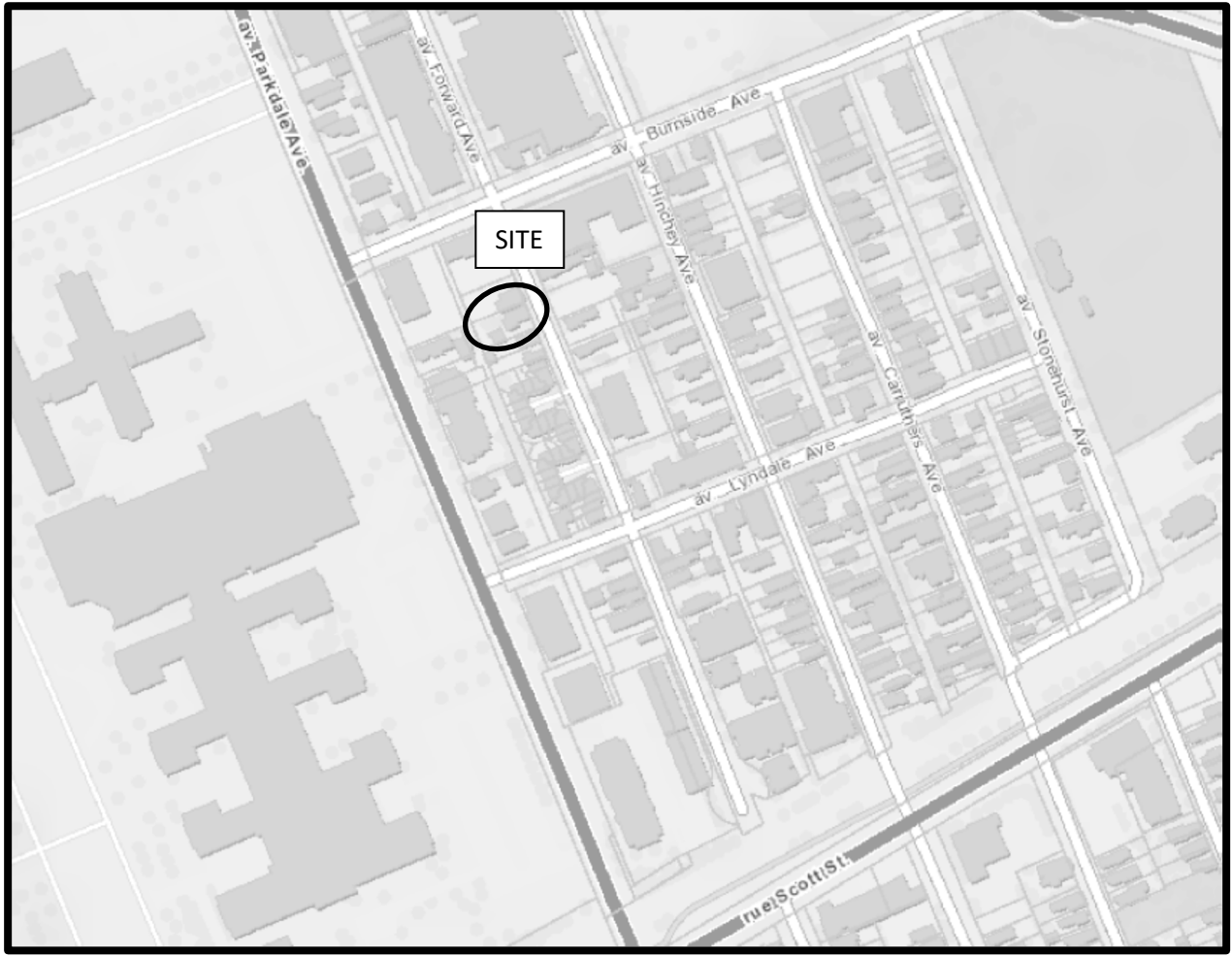
**Anions**

Chloride	5 ug/g dry	11	-	-	-
Sulphate	5 ug/g dry	35	-	-	-

# APPENDIX 2

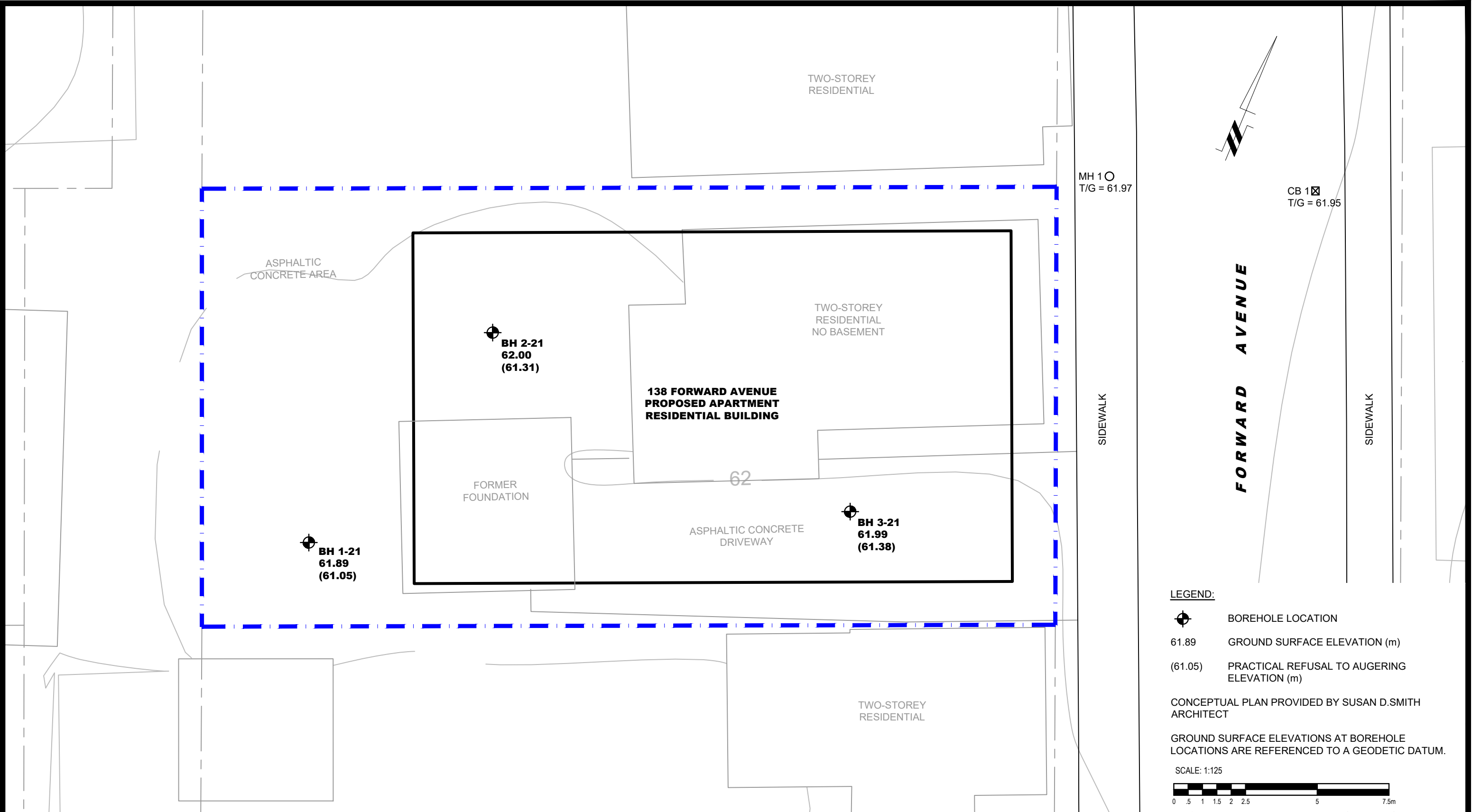
FIGURE 1 – KEY PLAN

DRAWING PG6026-1 – TEST HOLE LOCATION PLAN



# FIGURE 1

## KEY PLAN



**LEGEND:**

- BOREHOLE LOCATION
- 61.89 GROUND SURFACE ELEVATION (m)
- (61.05) PRACTICAL REFUSAL TO AUGERING ELEVATION (m)

CONCEPTUAL PLAN PROVIDED BY SUSAN D.SMITH ARCHITECT

GROUND SURFACE ELEVATIONS AT BOREHOLE LOCATIONS ARE REFERENCED TO A GEODETIC DATUM.

SCALE: 1:125

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NO.	REVISIONS	DATE	INITIAL

VIKA LAND DEVELOPMENT GROUP INC.  
**GEOTECHNICAL INVESTIGATION  
 PROPOSED RESIDENTIAL BUILDING**  
 138 FORWARD AVENUE  
 OTTAWA, ONTARIO  
 Title: **SITE PLAN**

Scale:	1:125	Date:	11/2021
Drawn by:	JM	Report No.:	PG6026-1
Checked by:	PB	Dwg. No.:	<b>PG6026-1</b>
Approved by:	SD	Revision No.:	