PATERSON GROUP

Geotechnical Investigation

Proposed Mixed-Use Mid-Rise Building

47 Beechwood Avenue, 5 Springfield Road
and 12 Douglas Avenue
Ottawa, Ontario

Prepared for Mr. Hussain Rahal

Report PG6484-1 Revision 1 dated May 9, 2024

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Mr. Hussain Rahal to conduct a Geotechnical Investigation for the proposed building to be located at 47 Beechwood Avenue, 5 Springfield Road and 12 Douglas Avenue, Ottawa, ON (refer to Figure 1 - Key Plan presented in Appendix 2).

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.

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

2.0 Proposed Development

Based on the available conceptual drawings, it is understood that the proposed mixed-use building will consist of an 8-storey apartment building with one underground parking level. At-grade access lanes and landscaped areas are anticipated as part of the proposed development. It is further anticipated that the proposed development will be municipality serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

Paterson conducted a geotechnical investigation at the subject site on November 29, 2022. The current investigation consisted of drilling 5 boreholes extending to a maximum depth of 4.0 m below the existing ground surface.

The test hole locations were distributed in a manner to provide general coverage of the subject site. The locations of the test holes are shown on Drawing PG6484-1 - Test Hole Location Plan included in Appendix 2.

The test holes were advanced using a CME-55 Low Clearance Drill rig and operated by a two-person crew. The drilling procedure consisted of augering to the required depths at the selected locations, and sampling and testing the overburden. All fieldwork was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer.

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 test holes 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.

Groundwater

A piezometer was installed in all boreholes to permit monitoring of the groundwater levels subsequent to the completion of the sampling program.

The groundwater observations are discussed in Section 4.3 and presented on the Soil Profile and Test Data sheets in Appendix 1.

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 with respect to a geodetic datum. The location of the test holes and ground surface elevation at each test hole location are presented on Drawing PG6484-1 - 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. Moisture content testing were completed on selected soil samples.

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 analyzed to determine the concentration of sulphate and chloride, the resistivity, and the pH of the sample. The results are discussed in Section 6.7 and provided in Appendix 1.

4.0 Observations

4.1 Surface Conditions

Currently, the subject site was observed to be occupied by multiple residential and/or commercial buildings with the associated at grade asphalt paved parking lot. The ground surface across the subject site is relatively flat and approximately at grade with roadway.

The site is bordered by commercial buildings to the north, Douglas Avenue to the east, beechwood avenue to the south and Springfield Road to the west.

4.2 Subsurface Profile

Overburden

Generally, the soil profile at the borehole locations consists of asphaltic concrete overlying a fill layer followed by dense to very dense brown silty sand to sandy silt deposit or compact glacial till underlain by bedrock.

Fill

The fill material was found to generally consist of brown silty sand with crushed stone and, at times, topsoil, ash, brick, coal, rock fragments and organics. The fill layer thickness ranged approximately between 0.2 to 2.8 m.

Silty Sand

The silty sand deposit was generally encountered below the fill at depths ranging from 1.5 to 2.0 m. The silty sand deposit was generally observed to be dense to very dense brown in colour.

Glacial Till

The glacial till deposit was encountered at BH5-22 underlying the fill at depth 1.9 m from the ground surface. The glacial till deposit was generally observed to consist of compact brown silty sand with gravel, cobbles, boulders changing into gray colour at depth 2.2 m from ground surface.

Practical refusal to the augering was encountered at approximate depths ranging from 2.6 to 4.0 m.

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

Bedrock

Based on available geological mapping, the bedrock in the subject area consists of Paleozoic shale of the Billings formation, with an overburden drift thickness of 2 to 5 m.

4.3 Groundwater

Groundwater levels measured in the piezometers are summarized in Table 1 below and are noted on the applicable Soil Profile and Test Data sheets presented in Appendix 1.

Table 1 - Summary of Groundwater Level Readings

Test Hole Number	Ground Surface Elevation (m)	Groundwater Depth (m)	Groundwater Elevation (m)	Recording Date
BH 1-22	56.59	Dry	-	December 4, 2022
BH 2-22	56.63	Dry	-	December 4, 2022
BH 3-22	56.65	Dry	-	December 4, 2022
BH 4-22	57.04	Dry	-	December 4, 2022
BH 5-22	56.66	3.05	53.61	December 4, 2022

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

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 development. It is recommended that foundation support for the proposed building consist of conventional spread footings bearing on the undisturbed dense to very dense silty sand, compact glacial till or clean Bedrock surface.

Due to the presence of fill on site, it is expected that sub-excavation will be required for the installation of proposed footings.

Precautions should be taken during construction to reduce the risks associated with the potential for heaving of the expansive shale bedrock.

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

5.2 Site Grading and Preparation

Stripping Depth

Due to the anticipated founding level for the proposed building, a significant portion of the existing overburden material will be excavated from within the proposed building footprint. Bedrock removal may be required for the construction of the parking garage levels within the east portion of the site.

Topsoil and fill, containing significant amounts of deleterious or organics materials, should be stripped from under any buildings, paved areas and other settlement sensitive structures.

If encountered, 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

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: piling rig, hoe ram, compactor, dozer, crane, truck traffic, etc. Vibrations, whether it is 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 proposed building should consist of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II or max. 100 mm diameter, well graded, site excavated blast rock approved by Paterson personnel at the time of placement. 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 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 approved site excavated fill is to be used to build up the subgrade level for areas to be paved, it should be compacted in thin lifts to a minimum of 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 as discussed under Section 6.1.

Protection of Potential Expansive Bedrock

It is anticipated that expansive shale will be encountered at the subject site. Although the effects of expansive shale will not affect the proposed building structure, it is possible that it will affect the proposed basement floor slab founded close to the shale bedrock.

A potential for heaving and rapid deterioration of the shale bedrock exists at this site. To reduce the long-term deterioration of the shale, exposure of the bedrock surface to oxygen should be kept as low as possible. The bedrock surface within the proposed development footprint should be protected from excessive dewatering and exposure to ambient air. These requirements should be evaluated by Paterson during the excavation operations and should be discussed with Paterson during the design stage.

To accomplish this, a 50 mm thick concrete mud slab should be placed on the exposed bedrock surface within a 48 hour period of being exposed. A 17 MPa lean concrete is recommended for this purpose. As an alternative to the mud slab, keeping the shale surface covered with granular backfill is also acceptable.

Overbreak in Bedrock

Sedimentary bedrock formations contain bedding planes, joints and fractures, and mud seams which create natural planes of weakness within the rock mass. Although several factors of a blast may be controlled to reduce backbreak and overbreak, upon blasting, the rock mass will tend to break along natural planes of weakness that may be present beyond the designed blast profile. Due to this, estimating the exact amount of backbreak and overbreak that may occur is not possible with conventional construction drill and blast methods.

Backbreak should be expected to occur along the perimeter of building and site service excavation footprints with conventional drill and blast bedrock removal methods. Further, overbreak is expected to occur throughout the lowest lifts of blasting due to the variable bedding planes and planes of weakness in the in-situ bedrock. It is very difficult to mitigate significant over-blasting given the constraints posed by footing geometry and spacing with respect to the zone of influence of blasts and the bedrocks in-situ characteristics.

Depending on the methodology undertaken by the contractor, efforts taken to minimize backbreak and overbreak may add significant time and costs to the excavation operations and is not guaranteed to completely eliminate the potential for backbreak and overbreak. Overbreak below footings should be in-filled with lean-concrete and approved by Paterson prior to placing concrete.

As such, volume estimates of bedrock to be removed may not be reflective of the actual volume of bedrock that may be required to be removed at the time of construction. This may result in additional materials, such as imported fill and concrete, to make up for additional rock loss. It is recommended that the blasting operations be planned and conducted under the supervision of a licensed professional engineer who is an experienced blasting consultant.

5.3 Foundation Design

Footings for the proposed building placed over clean, surface sounded bedrock bearing surface can be designed using a factored bearing resistance value at Ultimate Limit States (ULS) of **1,000 kPa**.

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 to be provided when a plane extending down and out from the bottom edges of the footing, at a minimum of 1.5H:1V, passes only through in situ soil of the same or higher capacity as that of the bearing medium. In unfractured bedrock, a plane with a slope of 1H:6V can be used.

Lean Concrete In-Filled Trenches

Where bedrock is expected near the design underside of footing elevation, consideration should be given to excavating zero entry, vertical trenches to expose the underlying bedrock surface and backfilling with lean concrete (15 MPa 28-day compressive strength). Typically, the excavation sidewalls will be used as the form to support the concrete. 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.

The near vertical trench excavation should be at least 150 mm wider than all sides of the footing at the base of the excavation. The excavation bottom should be relatively clean using the hydraulic shovel only (workers will not be permitted in the excavation). Once approved by the Paterson field personnel, lean concrete can be poured up to the proposed founding elevation.

Footings placed on lean concrete filled trenches extending to a clean, bedrock surface can be designed using a factored bearing resistance value at ULS of **1,000 kPa**.

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. 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. It is expected that a Class A or B will be applicable. However, a site specific seismic shear wave velocity test is required to confirm the higher seismic site class (Class A or B) in accordance with OBC standards.

5.5 Basement Slab

With the removal of all topsoil and deleterious fill from within the footprint of the proposed building, the native soil surface or approved engineered fill surface will be considered an acceptable subgrade on which to commence backfilling for floor 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, is recommended for backfilling below the floor slab.

For structures with slab-on-grade construction, the upper 200 mm of sub-slab fill is recommended to consist of OPSS Granular A crushed stone. All backfill material within the footprint of the proposed buildings should be placed in maximum 300 mm thick loose layers and compacted to a minimum of 98% of the SPMDD.

For structures with basement slabs, it is recommended that the upper 200 mm of sub-floor fill consists of 19 mm clear crushed stone.

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^3. The applicable effective (undrained) unit weight of the retained soil can be taken as 13 kN/m3, where applicable. A hydrostatic pressure should be added to the total static earth pressure when using the effective unit weight.

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)
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 0.32 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 $P_o = 0.5\ K_o\ \gamma\ H^2$, 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 2020.

5.7 Pavement Structure

Driveways and local roads are anticipated at this site. The proposed pavement structures are presented in Tables 2 and 3.

Table 2 – Recommended Pavement Structure – Car Only Parking	
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 - Undisturbed native soil, or OPSS Granular B Type I or II material.	

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 - Undisturbed native soil, or 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 compaction equipment.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

It's recommended that a perimeter foundation drainage system be provided for the proposed structure. It is expected that insufficient room will available for exterior backfill and the foundation wall will bast as a blind-sided pour against a shoring system. It is recommended that the drainage system consist of the following:

A composite drainage membrane (Delta-Terraxx, MiraDrain G100N or equivalent) should be placed against the shoring system and bedrock excavation face from the finished ground surface to the top of the footing.

It is recommended that 150 mm diameter sleeves at 3 m centres be cast in the footing or at the foundation wall/footing interface to allow the infiltration of water to flow to the interior perimeter drainage pipe. The sleeves should be connected to openings in the HDPE face of the drainage board layer. The perimeter drainage pipe and underfloor drainage system should direct water to sump pit(s) within the lower basement area.

Water Infiltration Volumes

Based on the above-noted methodology, water carried by the foundation and underfloor drainage system will generally consist of surface water and will not consist of groundwater/long-term dewatering of the groundwater table. Water managed by this system will be directed to the appropriate building sump pit.

It is expected that the successful implementation of this system throughout the subject site will result in a long-term infiltration rate of less than 30,000 L/day of surface water. Peak periods of infiltration (i.e.- short-term conditions) should be anticipated during heavy rainfall and snow-melt events.

Underfloor Drainage

For design purposes, it is recommended the underfloor drainage system consist of 150 mm diameter perforate pipes surrounded by a geosock and a 150 mm thick layer of 19 mm clear crushed stone on all of its sides. Several north-south and east-west lines of pipes will be placed throughout the basement level, and as directed by the geotechnical consultant, to direct water from the foundation drainage and perimeter subdrain systems to the sump pump system. The final spacing of the underfloor drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

Foundation Backfill

For areas where sufficient space is available for backfill against the exterior sides of the foundation walls, the backfill material 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-Terraxx, connected to the perimeter foundation drainage system. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should otherwise be used for this purpose.

6.2 Protection of Footings Against Frost Action

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

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

6.3 Excavation Side Slopes

The side slopes of excavations in the overburden should be either 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 expected 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 groundwater level. Excavations below the groundwater level should be cut back at a maximum slope of 1.5H:1V.

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. 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 shoring system could consist of a soldier pile and lagging system. Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be included to the earth pressures described below. This system could be cantilevered, anchored or braced. The shoring system is recommended to be adequately supported to resist toe failure, if required, by means of extending the piles into the bedrock through pre-augered holes, if a soldier pile and lagging system is the preferred method.

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

Table 4 – Soils Parameter for Shoring System Design	
Parameters	**Values**
Active Earth Pressure Coefficient (K_a)	0.33
Passive Earth Pressure Coefficient (K_p)	3
At-Rest Earth Pressure Coefficient (K_O)	0.5
Unit Weight (γ), kN/m^3	20
Submerged Unit Weight (γ), kN/m^3	13

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 weights are calculated for earth pressures. If the groundwater level is lowered, the dry unit weight for the soil 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.

Impacts on Adjacent Properties

It is our understanding that the proposed mixed-used building will consist of an 8-storey department building. It is further understood that the project will entail an underground parking structure occupying the majority of the site area. It is anticipated that the footing of the building will be found on a clean, surface sounded bedrock bearing surface.

The overburden along the perimeter of the proposed building footprint will need to be sloped or shored in order to complete the construction of the underground level. Bedrock removal is also anticipated, which will be completed by line drilling, blasting and/or hoe ramming. The blasting and hoe ramming will be carried out by a contractor specializing in bedrock removal.

Where required, it is anticipated that the temporary shoring system adjacent to the neighboring roads and properties may consist of soldier piles and lagging designed for at-rest earth pressures, as per the geotechnical design recommendations outlined in Table 4 above.

The geotechnical engineer will review the stability of the rock face underlying the overburden. Following the review of the rock face, the geotechnical engineer will determine if rock reinforcement is required, and if so, the extent to which rock reinforcement is required.

Vibrations associated with the bedrock blasting removal program, if blasting is required, may impact neighboring roads and properties. Therefore, as stated above in Subsection 5.2, a vibration monitoring program is recommended to ensure vibration levels remain below recommended tolerances. In addition, a pre-construction survey will be required for any existing structures in the immediate area of the proposed building as per standard construction practices, where bedrock blasting will 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 of the OPSD.

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

6.5 Groundwater Control

It is anticipated that groundwater infiltration into the excavations should be minimal 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.

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.

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 the analytical testing of one (1) soil sample show that the sulphate content is less than 0.1%. This result is indicative that Type 10 Portland cement (General Use Cement) would be appropriate. The results of the chloride content and pH indicate that they are not significant factors in creating a corrosive environment for exposed ferrous metals at this site while the resistivity tests yielded results indicative of a aggressive to slightly aggressive corrosive environment.

7.0 Recommendations

It is a requirement for the foundation 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.
- Review of the geotechnical aspects of the excavating program, prior to construction.
- 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.
- Periodic inspection of the installation of the underfloor and perimeter drainage and waterproofing systems.
- Field density tests to determine the level of compaction achieved.
- Observation of all subgrades prior to backfilling.
- Sampling and testing of the bituminous concrete including mix design reviews.

All excess soils, with the exception of engineered crushed stone fill, generated by construction activities that will be transported on-site or off-site should be handled as per *Ontario Regulation 406/19: On-Site and Excess Soil Management*.

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 material testing and observation program by Paterson.

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 Mr. Hussain Rahal, 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
May 9, 2024
D. J. GILBERT
100116130
PROVINCE OF ONTARIO

Zubaida Al-Moselly, P.Eng.

David J. Gilbert, P.Eng.

Report Distribution:

- Mr. Hussain Rahal
- Paterson Group

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
47 Beechwood Ave., 5 Springfield Rd & 12 Douglas Ave.
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE November 29, 2022

FILE NO. PG6484

HOLE NO. BH 1-22

SOIL DESCRIPTION	Depth of change (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	56.59
Asphaltic concrete	0.05						
FILL: Brown silty sand with gravel and crushed stone	0.69	AU	1				
FILL: Brown silty sand, some gravel, trace clay and wood	1.07	SS	2	50	18	1	55.59
BEDROCK: Very poor to good quality, black shale, some to trace mud seams		SS	3	83	50+	2	54.59
		SS	4	100	50+		
	3.10	SS	5	56	50+	3	53.59
End of Borehole							

Practical refusal to augering at 3.10m depth.

(BH dry - December 4, 2022)

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

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
47 Beechwood Ave., 5 Springfield Rd & 12 Douglas Ave.
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE November 29, 2022

FILE NO. PG6484

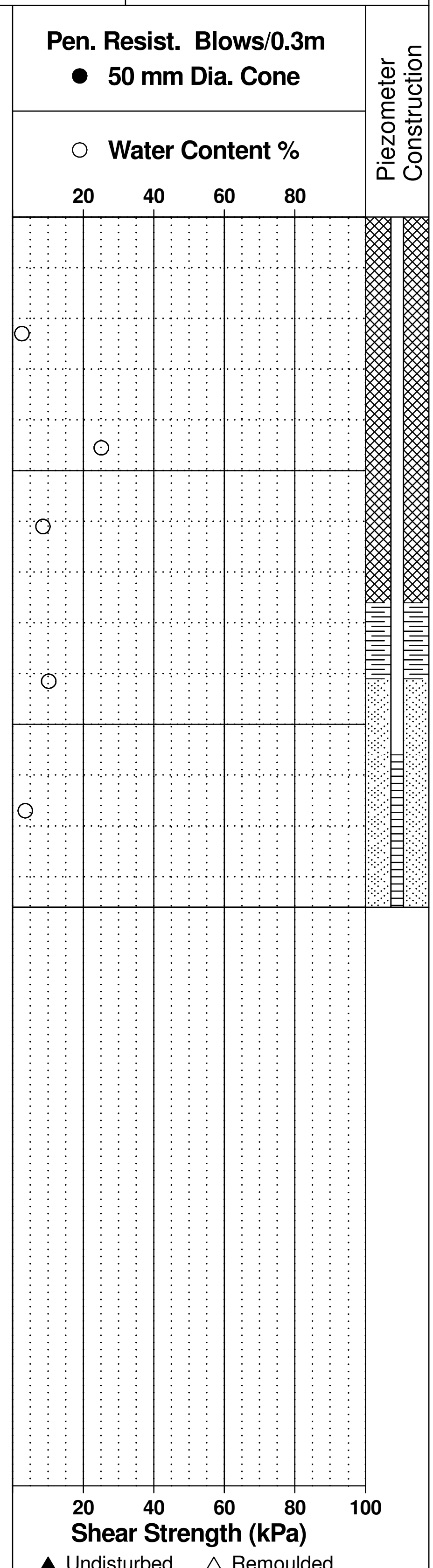
HOLE NO. BH 2-22

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	56.63
Asphaltic concrete	0.05						
FILL: Brown silty sand with gravel and crushed stone	0.60	AU	1				
FILL: Brown silty clay, some sand, trace gravel and wood	1.22	SS	2	60	5	1	55.63
FILL: Brown silty sand with gravel, trace concrete	1.37						
BEDROCK: Very poor to good quality, black shale, some to trace mud seams	2.72	SS	3	67	26	2	54.63
		SS	4	100	50+		

End of Borehole

Practical refusal to augering at 2.72m depth.

(BH dry - December 4, 2022)



patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
47 Beechwood Ave., 5 Springfield Rd & 12 Douglas Ave.
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE November 29, 2022

FILE NO. PG6484

HOLE NO. BH 3-22

SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
Asphaltic concrete	0.08
FILL: Brown silty sand with gravel and crushed stone	0.69
FILL: Brown silty sand, some topsoil, trace wood, ash, coal and brick	1.45
Dense, brown **SILTY SAND to SANDY SILT,** trace gravel	1.83
BEDROCK: Very poor to good quality, black shale, some to trace mud seams	2.62
End of Borehole	

Practical refusal to augering at 2.62m depth.

(BH dry - December 4, 2022)

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2		9
SS	3	88	37
SS	4		50+

DEPTH (m)	ELEV. (m)
0	56.65
1	55.65
2	54.65

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

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
47 Beechwood Ave., 5 Springfield Rd & 12 Douglas Ave.
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE November 29, 2022

FILE NO. PG6484

HOLE NO. BH 4-22

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	57.04
Asphaltic concrete	0.03						
FILL: Brown silty sand with gravel and crushed stone	0.76	AU	1				
FILL: Brown silty sand with clay, trace gravel, ash and wood	1.45	SS	2	67	6	1	56.04
FILL: Brown silty sand, trace topsoil and clay	1.98	SS	3	58	12	2	55.04
Very dense, grey **SILTY SAND to SANDY SILT,** trace gravel	2.95	SS	4	92	50+		
BEDROCK: Fair to good quality, black shale	3.07	SS	5	100	50+	3	54.04

End of Borehole

Practical refusal to augering at 3.07m depth.

(BH dry - December 4, 2022)

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

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
47 Beechwood Ave., 5 Springfield Rd & 12 Douglas Ave.
Ottawa, Ontario

DATUM Geodetic

REMARKS

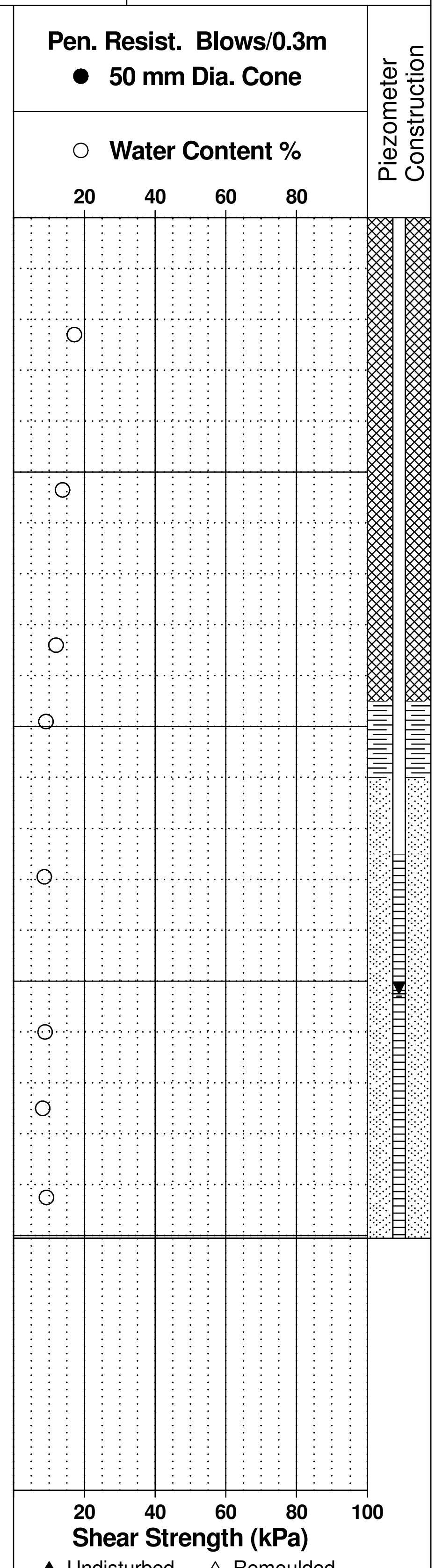
BORINGS BY CME-55 Low Clearance Drill

DATE November 29, 2022

FILE NO. PG6484

HOLE NO. BH 5-22

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	56.66
Asphaltic concrete — 0.06						
FILL: Brown silty sand, some to trace clay and gravel	AU	1				
	SS	2		9	1	55.66
— 1.88	SS	3	75	22		
GLACIAL TILL: Compact, brown silty sand with gravel, cobbles and boulders					2	54.66
- dark grey by 2.2m depth	SS	4	67	19		
— 3.15					3	53.66
BEDROCK: Very poor to good quality, black shale, some to trace mud seams	SS	5	75	29		
	SS	6	67	50+		
— 4.01					4	52.66
End of Borehole						
Practical refusal to augering at 4.01m depth.						
(GWL @ 3.05m - Dec. 4, 2022)						



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

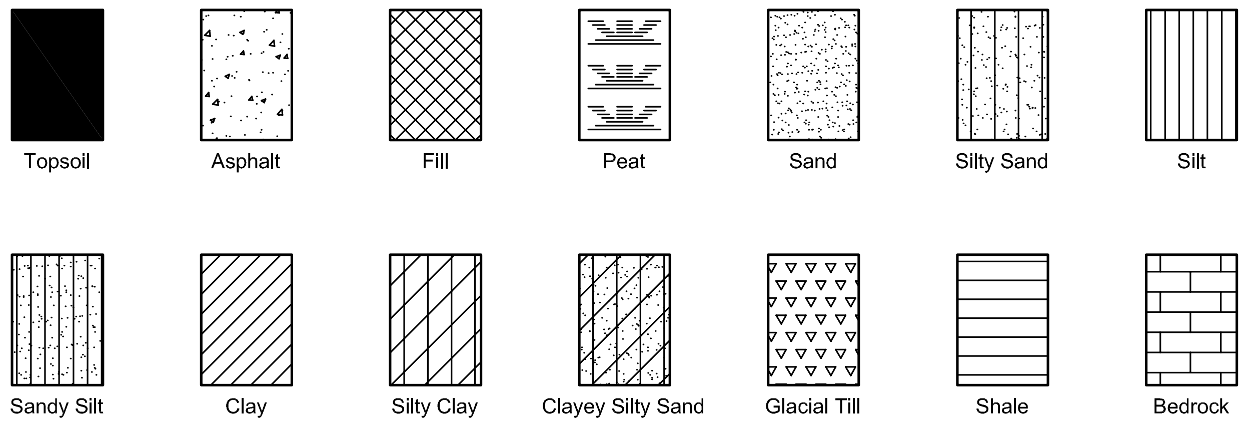
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void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

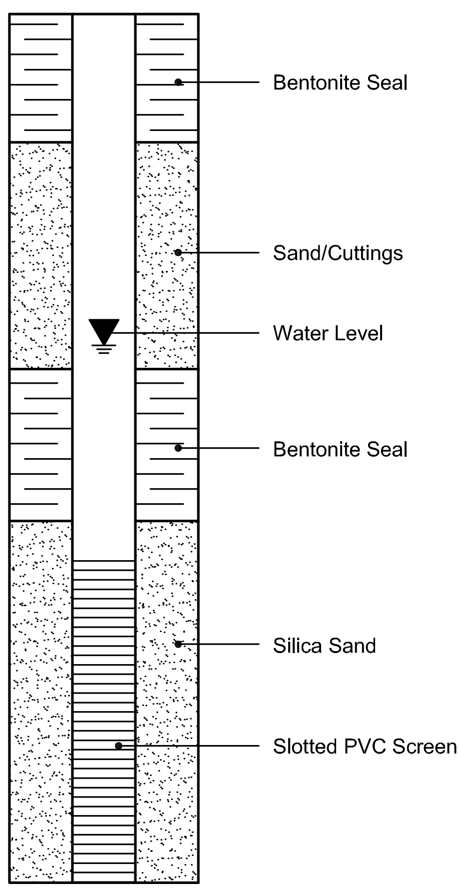
SYMBOLS AND TERMS (continued)

STRATA PLOT

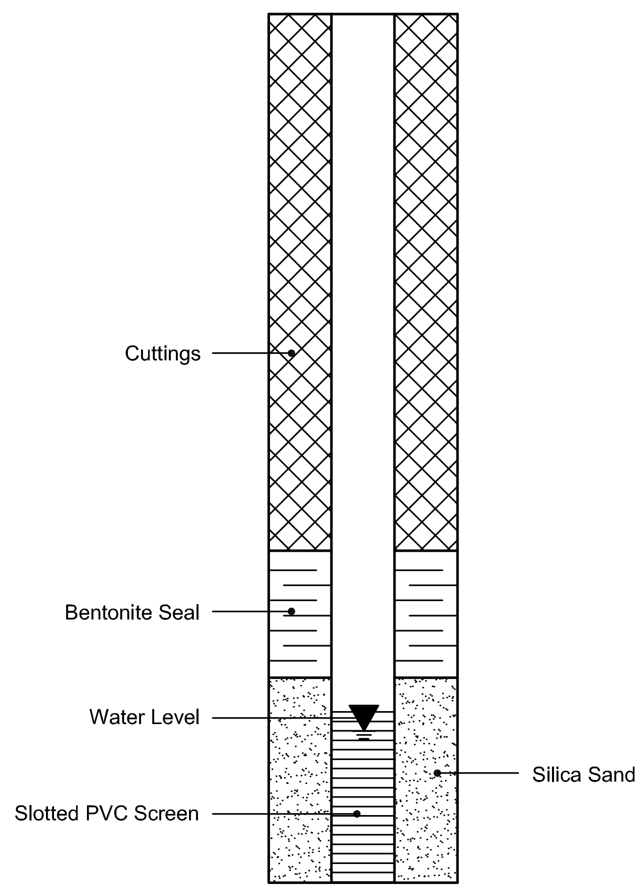


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



PARACEL LABORATORIES LTD.

Certificate of Analysis

Order #: 2249239

Client: **Paterson Group Consulting Engineers**

Report Date: 04-Dec-2022

Order Date: 29-Nov-2022

Client PO: 56346

Project Description: PG6484

	Client ID:	BH4 - 22 SS4	-	-	-	-	-
	Sample Date:	29-Nov-22 09:00	-	-	-		
	Sample ID:	2249239-01	-	-	-		
	Matrix:	Soil	-	-	-		
	MDL/Units						
Physical Characteristics							
% Solids	0.1 % by Wt.	91.2	-	-	-	-	-
General Inorganics							
pH	0.05 pH Units	7.53	-	-	-	-	-
Resistivity	0.1 Ohm.m	18.3	-	-	-	-	-
Anions							
Chloride	5 ug/g	89	-	-	-	-	-
Sulphate	5 ug/g	363	-	-	-	-	-

APPENDIX 2

FIGURE 1 – KEY PLAN

DRAWING PG6484-1 – TEST HOLE LOCATION PLAN

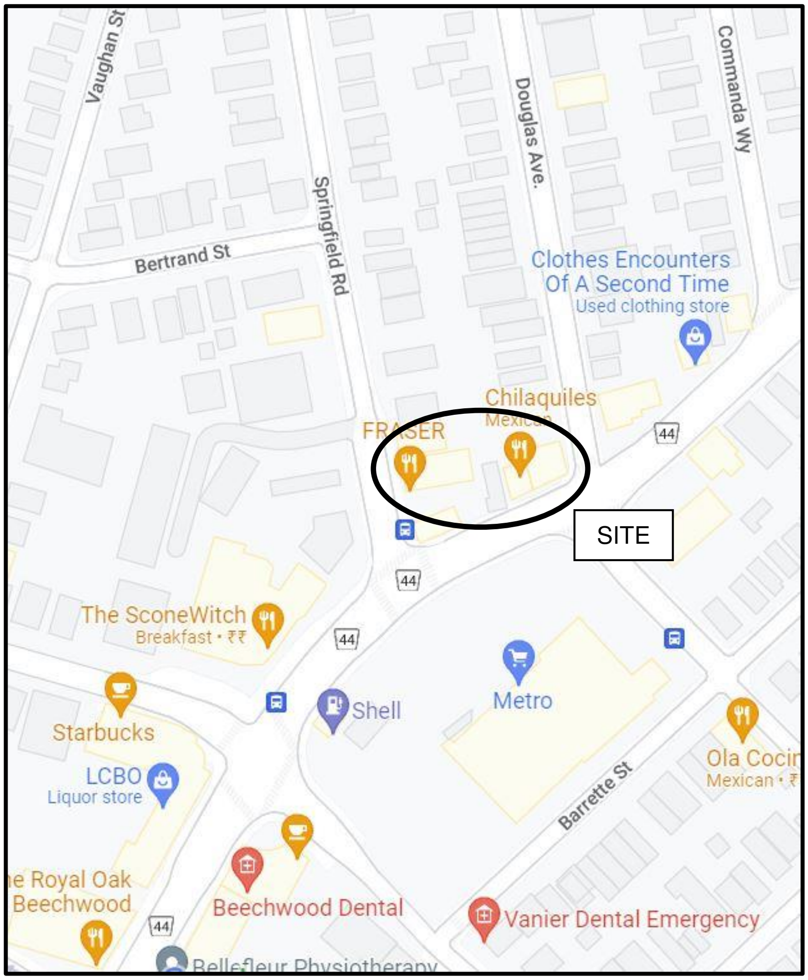
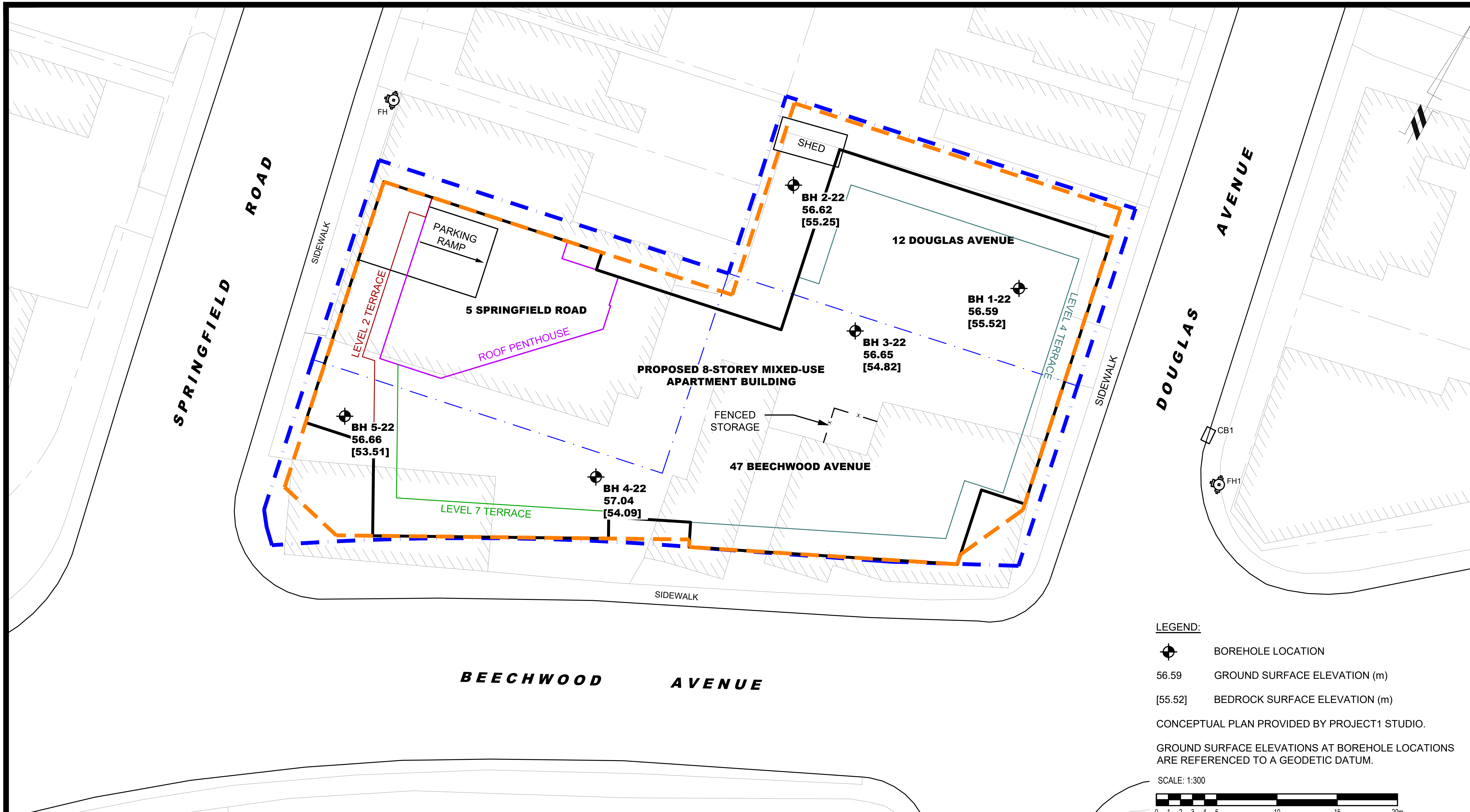


FIGURE 1

KEY PLAN



PATERSON GROUP
9 AURIGA DRIVE
OTTAWA, ON
K2E 7T9
TEL: (613) 226-7381

NO.	REVISIONS	DATE	INITIAL
1	UPDATED CONCEPTUAL PLAN	09/05/2024	ZA

MR.HUSSAIN RAHAL
GEOTECHNICAL INVESTIGATION
PROPOSED MID-RISE BUILDING
47 BEECHWOOD AVENUE, 5 SPRINGFIELD ROAD
AND 12 DOUGLAS AVENUE

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:300	Date: 01/2023
Drawn by: GK	Report No.: PG6484-1
Checked by: BN	Dwg. No.: PG6484-1
Approved by: DJG	Revision No.: 1

p:\autocad drawings\geotechnical\pg64xx\pg6484\pg6484-1 test hole location plan (rev.01).dwg

11x17