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Geotechnical Investigation

Proposed Multi-Storey Building
1300-1310 McWatters Road
Ottawa, Ontario

Prepared For

Homestead Land Holdings Inc.

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Report: PG5729-1

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Homestead Land Holdings Ltd. to conduct a geotechnical investigation for the proposed multi-storey building to be located at 1300-1310 McWatters Road, in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objectives of the geotechnical investigation were to:

- ❑ Determine the existing subsoil and groundwater information at this site by means of boreholes.

- ❑ Provide geotechnical recommendations for the design of the proposed development including construction considerations which may affect its 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 high-rise residential building, with two levels of underground parking which will extend beyond the footprint of the high-rise building. Asphalt-paved parking areas, walkways and landscaped areas surrounding the proposed building are also proposed.

Construction of the proposed development will involve demolition of the existing amenity building on-site.

3.0 Method of Investigation

3.1 Field Investigation

The field program for the geotechnical investigation was carried out from April 29 to May 3, 2021, and consisted of advancing 5 boreholes to a maximum depth of 22.6 m. The borehole locations were determined in the field by Paterson personnel taking into consideration site features and underground services. The locations of the boreholes are shown on Drawing PG5729-1 - Test Hole Location Plan in Appendix 2.

The boreholes were advanced using a low-clearance track-mounted auger drill rig operated by a two-person crew. All fieldwork was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer. The drilling procedure consisted of augering and rock coring to the required depths at the selected locations, and sampling and testing the overburden.

Sampling and In Situ Testing

Soil samples were collected from the boreholes using two different techniques, namely, sampled directly from the auger flights (AU) or collected using a 50 mm diameter split-spoon (SS) sampler. Rock cores (RC) were obtained using 47.6 mm inside diameter coring equipment. All samples were visually inspected and initially classified on site. The auger and split-spoon samples were placed in sealed plastic bags, and rock cores were placed in cardboard boxes. All samples were transported to our laboratory for further examination and classification. The depths at which the auger, split spoon and rock core samples were recovered from the boreholes are shown as AU, SS and RC, respectively, on the Soil Profile and Test Data sheets presented in Appendix 1.

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

A recovery value and a Rock Quality Designation (RQD) value were calculated for each drilled section (core run) of bedrock and are shown on the borehole logs. The recovery value is the ratio, in percentage, of the length of the bedrock sample recovered over the length of the drilled section (core run). The RQD value is the ratio, in percentage, of the total length of intact rock pieces longer than 100 mm in one core run over the length of the core run. These values are indicative of the quality of the bedrock.

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

Groundwater

Monitoring wells were installed in boreholes BH 2-21 through BH 4-21 to permit monitoring of the groundwater levels subsequent to the completion of the sampling program. Flexible standpipes were also installed in boreholes BH 1-21 and BH 5-21.

All monitoring wells should be decommissioned in accordance with Ontario Regulations O.Reg 903 by a qualified licensed well technician and prior to construction.

Sample Storage

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

3.2 Field Survey

The borehole locations were selected by Paterson to provide general coverage of the proposed development, taking into consideration the existing site features and underground utilities. The borehole locations and ground surface elevations at each borehole location were surveyed by Paterson using a GPS unit with respect to a geodetic datum. The location of the boreholes and ground surface elevation at each borehole location are presented on Drawing PG5729-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

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

4.0 Observations

4.1 Surface Conditions

The subject site is located within the northeast portion of 1300 McWatters Road. The majority of the subject site is occupied by asphalt-paved access lanes, landscaped areas and a paved parking lot which are associated with the existing multi-storey building which borders the subject site to the east. An existing 1-storey amenity building is located within the southern limits of the subject site and is to be demolished prior to the construction of the proposed development. Landscaped areas are also located surrounding the existing structures.

Based on available aerial photos, a quarry was located within the subject site as recently as 1965. The quarry was infilled and construction of the aforementioned multi-storey building and amenity building were completed by 1976. Reference should be made to the aerial photographs in Figure 2 - Aerial Photograph - 1965 and Figure 3 - Aerial Photograph - 2019 which illustrate the former and present site conditions, respectively.

The subject site is bordered to the north by Lisa Avenue, to the east and southeast by an existing multi-storey residential building, to the south by an asphalt paved access lane associated with the existing residential complex, and to the west by Greenbank Road. The existing ground surface is relatively level across the subject site at approximate geodetic elevation of 77 to 78 m.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile at the test hole locations consists of a 50 to 130 mm thick asphalt layer underlain by a 4.4 to 5.7 m thick fill layer. The fill material was generally observed to vary from a brown silty sand with trace gravel, to a brown to grey silty clay layer with sand and gravel, to topsoil with organics and wood with depth.

The fill was observed to be underlain by a deposit of compact to dense, brown to grey silty sand which was further underlain by bedrock.

Bedrock

Practical refusal to augering on the bedrock surface was encountered at approximate depths of 19.3 and 18.1 m in boreholes BH 4-21 and BH 5-21, respectively. The bedrock was observed to consist of grey, interbedded quartz sandstone and limestone, and based on the RQDs of the recovered bedrock core, was generally of good to excellent in quality. At boreholes BH 4-21 and BH 5-21, The bedrock was cored to depths of from 22.6 to 21.1 m below the existing ground surface, respectively.

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

4.3 Groundwater

Groundwater level readings were measured in the monitoring wells installed at boreholes BH 2-21 to BH 4-21 as well as the peizometers installed at boreholes BH 1-21 and BH 5-21 on May 12, 2021. The observed groundwater levels are summarized in Table 1 below.

Table 1 - Summary of Groundwater Level Readings

Test Hole Number	Ground Surface Elevation (m)	Groundwater Levels (m)	Groundwater Elevation (m)	Recording Date
BH 1-21	76.83	8.24	68.59	May 12, 2021
BH 2-21*	77.50	7.48	70.02	May 12, 2021
BH 3-21*	76.62	7.98	68.64	May 12, 2021
BH 4-21*	76.88	7.53	69.35	May 12, 2021
BH 5-21	76.89	8.28	68.61	May 12, 2021

Note: Ground surface elevations at test hole locations were surveyed by Paterson and are referenced to a geodetic datum.
*Denotes a monitoring well location.

It should be noted that groundwater levels could be influenced by surface water infiltrating the backfilled boreholes. The long-term groundwater level can also be estimated based on the recovered soil samples' moisture levels, colouring and consistency. Based on these observations, the long-term groundwater level is anticipated at a depth of approximately 7 to 9 m below ground surface. However, groundwater levels are subject to seasonal fluctuations and 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 the proposed multi-storey building be founded on one of the following:

☐ Conventional spread footings bearing on an undisturbed, compact to dense silty sand bearing surface.

☐ a raft foundation bearing on an undisturbed, compact to dense silty sand bearing surface.

Where the footings of the proposed building abut the neighbouring existing building, they should match the existing footing elevations.

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

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and fill, such as those containing organic or deleterious materials, should be stripped from under any buildings and other settlement sensitive structures.

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

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. The fill should be tested and approved prior to delivery to the site. The fill should be placed in lifts no greater than 300 mm thick and compacted using suitable compaction equipment for the lift thickness.

Fill placed beneath the building and paved areas should be compacted to at least 98% of its standard Proctor maximum dry density (SPMDD).

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

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

5.3 Foundation Design

Conventional Spread Footings

Footings placed on an undisturbed, compact to dense silty sand bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **300 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **450 kPa**. A geotechnical resistance factor of 0.5 was applied to the above noted bearing resistance value at ULS.

The bearing resistance value at SLS will be subjected to potential post-construction total and differential settlements of 25 and 20 mm, respectively.

An undisturbed soil bearing surface consists of a surface from which all topsoil and deleterious materials, such as loose, frozen or disturbed soil, whether in situ or not, have been removed, in the dry, prior to the placement of concrete for footings.

As a general procedure, it is recommended that the footings for the proposed building that are located adjacent to the existing neighbouring structure be founded at the same level as the existing footings. This accomplishes three objectives. First, the behaviour of the two structures at their connection will be similar due to the similar bearing medium. Second, there will be minimal stress added to the existing structure from the new structure. Third, the bearing of the new structure will not be influenced by any backfill from the existing structure.

Raft Foundation

As noted above, it is expected that a raft foundation may be required to support the proposed multi-storey building. The maximum SLS contact pressure recommended is **350 kPa** for a raft foundation bearing on the undisturbed, compact to dense silty sand.

It should be noted that the weight of the raft slab and everything above has to be included when designing with this value. The loading conditions for the contact pressure are based on sustained loads, that are generally taken to be 100% Dead Load and 50% Live Load. The factored bearing resistance (contact pressure) at ULS can be taken as **525 kPa**. A geotechnical resistance factor of 0.5 was applied to the bearing resistance value at ULS.

The modulus of subgrade reaction was calculated to be **14 MPa/m** for a contact pressure of **350 kPa**. The design of the raft foundation is required to consider the relative stiffness of the reinforced concrete slab and the supporting bearing medium. A common method of modeling the soil structure interaction is to consider the bearing medium to be elastic and to assign a subgrade modulus. However, silty sand is not elastic and limits have to be placed on the stress ranges of a particular modulus.

The proposed building can be designed using the above parameters with total and differential settlements of 25 and 20 mm, respectively.

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 the undisturbed, compact to dense silty sand above the groundwater table, when a plane extending down and out from the bottom edge of the footing at a minimum of 1.5H:1V passes only through in situ soil of the same or higher capacity as the bearing medium soil.

5.4 Design for Earthquakes

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

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

5.5 Basement Floor Slab

With the removal of all topsoil and deleterious fill from within the footprint of the proposed building, the native soil surface will be considered an acceptable subgrade on which to commence backfilling for floor slab construction. It is anticipated that the basement area for the proposed building will be mostly parking and the recommended pavement structures noted in Subsection 5.7 will be applicable. However, if storage or other uses of the lower level will involve the construction of a concrete floor slab, the upper 200 mm of sub-slab fill is recommended to consist of 19 mm clear crushed stone.

All backfill material within the footprint of the proposed building should be placed in maximum 300 mm thick loose layers and compacted to a minimum of 98% of the SPMDD.

If a raft slab is considered for the proposed multi-storey building, a granular layer of OPSS Granular A crushed stone will be required to allow for the installation of sub-floor services above the raft slab foundation. The thickness of the OPSS Granular A crushed stone will be dependent on the piping requirements.

In consideration of the groundwater conditions encountered at the time of the field investigation, a sub-slab drainage system, consisting of lines of perforated drainage pipe subdrains connected to a sump pit, should be provided in the subfloor fill under the lower basement floor (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^3.

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

Lateral Earth Pressures

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

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

An additional pressure having a magnitude equal to $K_o \cdot q$ and acting on the entire height of the wall should be added to the above diagram for any surcharge loading, q (kPa), that may be placed at ground surface adjacent to the wall. The surcharge pressure will only be applicable for static analyses and should not be used in conjunction with the seismic loading case.

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

Seismic Earth Pressures

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

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

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

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

The earth force component (P_o) under seismic conditions can be calculated using $P_o = .5\ K_o\ \gamma\ H^2$, where $K_o = 0.5$ for the soil conditions noted above.

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

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

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

5.7 Pavement Structure

For design purposes, it is recommended that the rigid pavement structure for the lower underground parking level consist of Category C2, 32 MPa concrete at 28 days with air entrainment of 5 to 8%. The recommended rigid pavement structure is further presented in Table 2 below. The flexible pavement structure presented in Table 3 should be used for at grade access lanes and heavy loading parking areas.

Table 2 - Recommended Rigid Pavement Structure - Lower Parking Level

Thickness (mm)	Material Description
150	**Exposure Class C2 - 32 MPa Concrete** (5 to 8% Air Entrainment)
300	**BASE** - OPSS Granular A Crushed Stone
SUBGRADE - Existing imported fill, or OPSS Granular B Type I or II material placed over bedrock.	

To control cracking due to shrinking of the concrete floor slab, it is recommended that strategically located saw cuts be used to create control joints within the concrete floor slab of the lower underground parking level. The control joints are generally recommended to be located at the center of the column lines and spaced at approximately 24 to 36 times the slab thickness (for example; a 0.15 m thick slab should have control joints spaced between 3.6 and 5.4 m). The joints should be cut between 25 and 30% of the thickness of the concrete floor slab and completed as early as 4 hour after the concrete has been poured during warm temperatures and up to 12 hours during cooler temperatures.

Table 3 - Recommended Asphalt Pavement Structure - Access Lanes and Heavy Loading Parking Areas

Thickness (mm)	Material Description
40	**Wear Course** - Superpave 12.5 Asphaltic Concrete
50	**Binder Course** - Superpave 19.0 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
300	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - OPSS Granular B Type II overlying the Concrete Podium Deck.	

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

If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type II material. The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 99% of the material's SPMDD using suitable vibratory equipment.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

It is recommended that a perimeter foundation drainage system be provided for the proposed structure. The system should consist of a 150 mm diameter, perforated and corrugated plastic pipe, surrounded on all sides by 150 mm of 10 mm clear crushed stone, which is placed at the footing level around the exterior perimeter of the structure. The pipes 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 temporary shoring system, extending to a series of drainage sleeve inlets through the building foundation wall at the footing/foundation wall interface. The drainage sleeves should be at lease 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 sub-slab drainage system. The perimeter drainage pipe and sub-slab drainage system should direct water to sump pit(s) within the underground level.

Foundation Raft Slab Construction Joints

If applicable, it is expected that the raft slab will be poured in sections. For the construction joint at each pour, a rubber water stop along with a chemical grout (Xypex or equivalent) should be applied to the entire vertical joint of the raft slab. Furthermore, a rubber water stop should be incorporated in the horizontal interface between the foundation wall and the raft slab.

Sub-slab Drainage

Sub-slab drainage will be required to control water infiltration. For preliminary design purposes, we recommend that 150 mm diameter perforated pipes be placed at approximate 6 m centres underlying the lowest level floor slab. The spacing of the sub-slab 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, the backfill material against the exterior sides of the foundation walls should consist of free-draining, non frost susceptible granular materials. The site materials will be frost susceptible and, as such, are not recommended for re-use as backfill unless a composite drainage system (such as Delta Drain 6000) connected to a drainage system is provided.

6.2 Protection of Footings Against Frost Action

Perimeter footings of heated structures are recommended to be protected against the deleterious effects of frost action. A minimum of 1.5 m of soil cover, or an equivalent combination of soil cover and foundation insulation, should be provided in this regard.

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

However, the foundations are generally not expected to require protection against frost action due to the founding depth. Unheated structures such as the access ramp may require insulation for protection against the deleterious effects of frost action.

6.3 Excavation Side Slopes

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

Unsupported Excavations

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

Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should be kept away from the excavation sides. Slopes in excess of 3 m in height should be periodically inspected by the geotechnical consultant in order to detect if the slopes are exhibiting signs of distress.

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

Temporary Shoring

Due to the anticipated proximity of the proposed building to the north and west property boundaries, temporary shoring may be required to support the overburden soils of the adjacent properties. The design and approval of the shoring system will be the responsibility of the shoring contractor and the shoring designer who is a licensed professional engineer and is hired by the shoring contractor. It is the responsibility of the shoring contractor to ensure that the temporary shoring is in compliance with safety requirements, designed to avoid any damage to adjacent structures and include dewatering control measures.

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

The designer should also take into account the impact of a significant precipitation event and designate design measures to ensure that a precipitation will not negatively impact the shoring system or soils supported by the system. Any changes to the approved shoring design system should be reported immediately to the owner’s structural designer prior to implementation.

The temporary shoring system may consist of a soldier pile and lagging system or steel sheet piles which could be cantilevered, anchored or braced. The shoring system is recommended to be adequately supported to resist toe failure.

Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be added to the earth pressures described on the following page.

The earth pressures acting on the shoring system may be calculated using the parameters on the next page:

Table 4 - Soil Parameters

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

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

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

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

Underpinning of Adjacent Structures

If the excavation for the proposed building is to extend within the lateral support zone of the adjacent building foundations, underpinning of these structures would be required. The depth of the underpinning, if required, would be dependent on the depth of the neighbouring foundations relative to the founding depth of the proposed building at the subject site.

6.4 Pipe Bedding and Backfill

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

A minimum of 150 mm of OPSS Granular A should be placed for bedding for sewer or water pipes when placed on a soil subgrade. 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 98% 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's SPMDD. All cobbles larger than 200 mm in their longest direction should be segregated from re-use as trench backfill.

6.5 Groundwater Control

It is anticipated that groundwater infiltration into the excavations should be controllable using open sumps. The contractor should be prepared to direct water away from all bearing surfaces and subgrades, regardless of the source, to prevent disturbance to the founding medium.

Groundwater Control for Building Construction

A temporary Ministry of Environment, Conservation and Parks (MECP) permit to take water (PTTW) may be required if more than 400,000 L/day of ground and/or surface water are to be pumped during the construction phase. At least 4 to 5 months should be allowed for completion of the application and issuance of the permit by the MECP.

For typical ground or surface water volumes being pumped during the construction phase, typically between 50,000 to 400,000 L/day, it is required to register on the Environmental Activity and Sector Registry (EASR). A minimum of two to four weeks should be allotted for completion of the EASR registration and the Water Taking and Discharge Plan to be prepared by a Qualified Person as stipulated under O.Reg. 63/16. If a project qualifies for a PTTW based upon anticipated conditions, an EASR will not be allowed as a temporary dewatering measure while awaiting the MECP review of the PTTW application.

Impacts on Neighbouring Properties

The proposed building is not anticipated to extend significantly below the groundwater level, therefore, any dewatering at the site will be minimal and should have no adverse effects to the surrounding buildings or structures. The short term dewatering during the excavation program will be managed by the excavation contractor.

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 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 indicate of an aggressive to very aggressive corrosive environment.

7.0 Recommendations

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

❑ Review of the geotechnical aspects of the excavation contractor’s design of the temporary shoring system, if required.

❑ Observation of all bearing surfaces prior to the placement of concrete.

❑ Sampling and testing of the concrete and fill materials used.

❑ Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.

❑ Observation of all subgrades prior to backfilling.

❑ Field density tests to determine the level of compaction achieved.

❑ Sampling and testing of the bituminous concrete including mix design reviews.

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

8.0 Statement of Limitations

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

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

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

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

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
May 26, 2021
S. S. DENNIS
100519516
Scott Dennis
PROVINCE OF ONTARIO

Scott Dennis

Kevin A. Pickard, EIT

Scott S. Dennis, P.Eng.

Report Distribution:

- ☐ Homestead Land Holdings Inc. (e-mail copy)
- ☐ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE & TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

patersongroup Consulting Engineers
154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building - 1300 & 1310 McWatters Rd.
Ottawa, Ontario

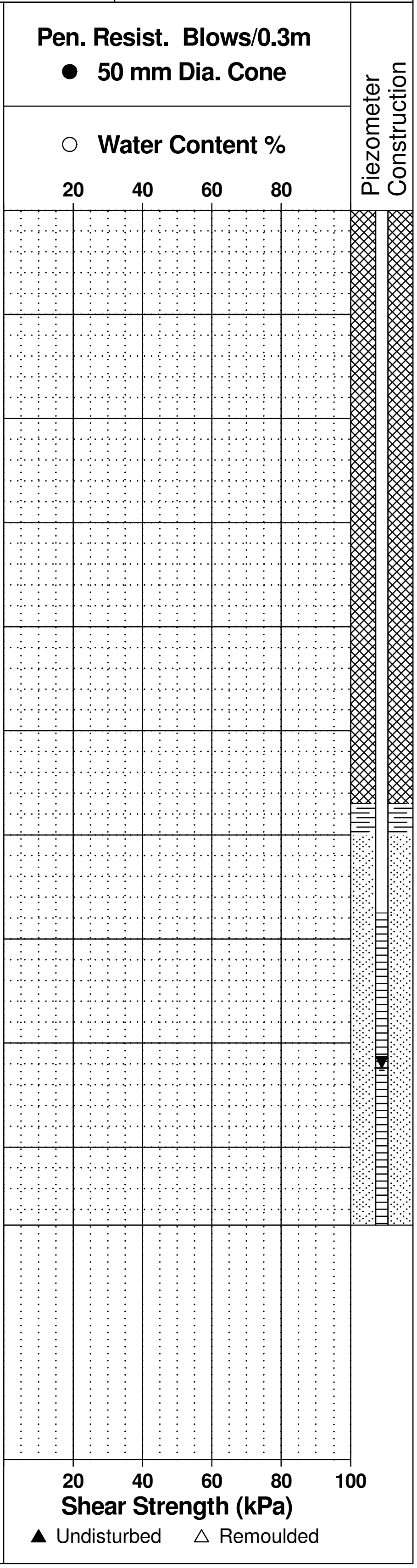
DATUM Geodetic
REMARKS
BORINGS BY CME-55 Low Clearance Drill
DATE April 29, 2021
FILE NO. PG5729
HOLE NO. BH 1-21

SOIL DESCRIPTION	Depth of base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
TOPSOIL	0.13	AU	1		
FILL: Brown silty sand, some gravel, trace clay	2.13	SS	2	83	14
		SS	3	50	23
FILL: Brown to grey silty clay with sand, some topsoil, organics and gravel	3.50	SS	4	100	9
		SS	5	75	11
FILL: Topsoil and organics with some sand, trace wood	5.18	SS	6	100	16
		SS	7	100	14
Dense, brown **SILTY SAND to SANDY SILT**	6.00	SS	8	100	39
Compact, brown **SILTY SAND**	9.75	SS	9	100	25
		SS	10	75	15
		SS	11	58	17
		SS	12	83	15
		SS	13	83	13

End of Borehole

(GWL @ 8.24m - May 12, 2021)

DEPTH (m)	ELEV. (m)
0	76.83
1	75.83
2	74.83
3	73.83
4	72.83
5	71.83
6	70.83
7	69.83
8	68.83
9	67.83



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building - 1300 & 1310 McWatters Rd.
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE April 29, 2021

FILE NO. PG5729

HOLE NO. BH 2-21

SOIL DESCRIPTION

GROUND SURFACE

SOIL DESCRIPTION	Depth (m)
Asphaltic concrete	0.10
FILL: Brown silty sand with crushed stone	0.46
FILL: Brown silty sand, trace gravel	2.21
FILL: Brown to grey silty clay with sand and gravel	2.97
FILL: Brown silty sand, trace gravel - some rock fragments by 4.0m depth - trace topsoil and wood by 4.6m depth	5.84
Compact, brown **SILTY SAND** - loose to very loose by 8.4m depth - running sand encountered below a depth of 8.4m	9.75

End of Borehole

(GWL @ 7.48m - May 12, 2021)

SAMPLE

TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	92	5
SS	3	50	7
SS	4	75	12
SS	5	67	8
SS	6	67	8
SS	7	75	18
SS	8	75	11
SS	9	75	20
SS	10	67	14
SS	11	58	12
SS	12	83	7
SS	13	100	2

DEPTH (m)	ELEV. (m)
0	77.50
1	76.50
2	75.50
3	74.50
4	73.50
5	72.50
6	71.50
7	70.50
8	69.50
9	68.50

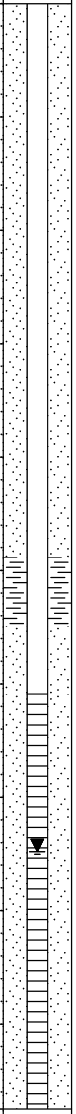
Pen. Resist. Blows/0.3m
● 50 mm Dia. Cone

○ Water Content %

20 40 60 80

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

Monitoring Well Construction



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building - 1300 & 1310 McWatters Rd.
Ottawa, Ontario

DATUM Geodetic

REMARKS

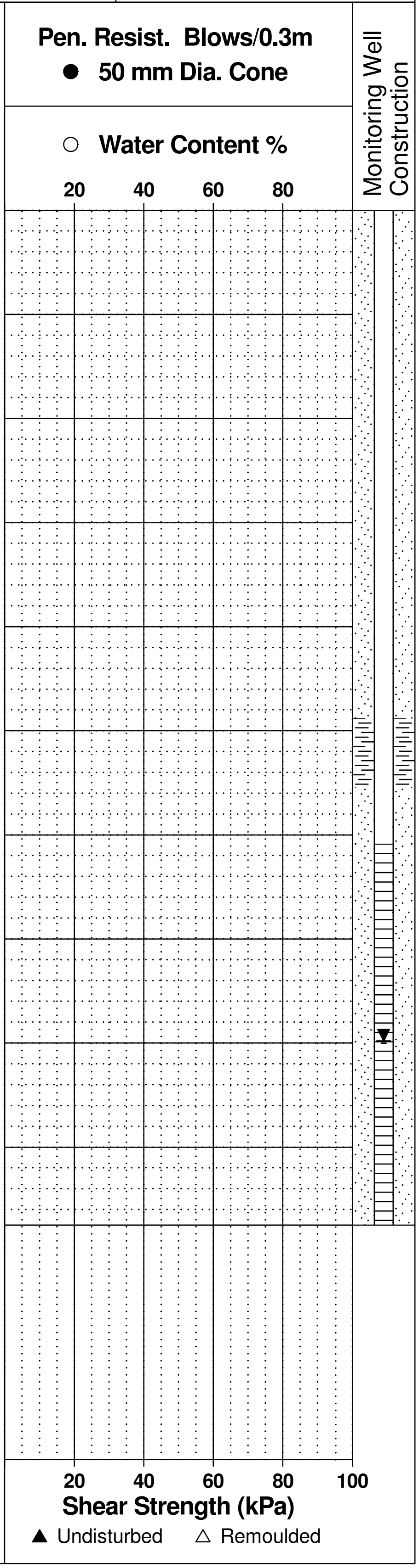
BORINGS BY CME-55 Low Clearance Drill

DATE April 29, 2021

FILE NO. PG5729

HOLE NO. BH 3-21

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	76.62
TOPSOIL 0.15	AU	1				
	SS	2	100	16	1	75.62
FILL: Brown to grey silty sand to silty clay, some gravel, trace topsoil	SS	3	100	15		
					2	74.62
	SS	4	75	12		
					3	73.62
3.73	SS	5	67	16		
FILL: Topsoil and organics, some sand, trace wood	SS	6	75	13	4	72.62
5.03	SS	7	58	12		
					5	71.62
	SS	8	73	50+		
					6	70.62
	SS	9	67	37		
Dense to compact, brown **SILTY SAND**					7	69.62
	SS	10	83	13		
	SS	11	58	13	8	68.62
	SS	12	67	11		
					9	67.62
9.75	SS	13	83	15		
End of Borehole (GWL @ 7.98m - May 12, 2021)						



SOIL PROFILE AND TEST DATA

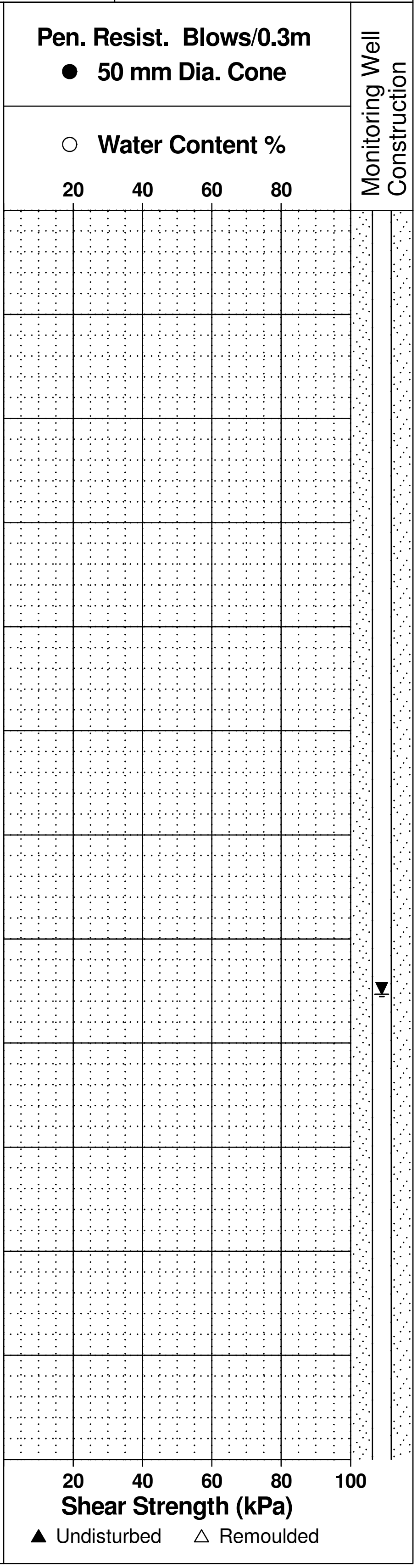
Geotechnical Investigation
Prop. Multi-Storey Building - 1300 & 1310 McWatters Rd.
Ottawa, Ontario

DATUM Geodetic
REMARKS
BORINGS BY CME-55 Low Clearance Drill
DATE April 30, 2021
FILE NO. PG5729
HOLE NO. BH 4-21

SOIL DESCRIPTION	
GROUND SURFACE	
FILL: Topsoil and organics with silty sand	0.10
FILL: Brown silty sand, trace gravel and organics - some clay by 2.3m depth	2.97
FILL: Brown to grey silty sand/silty clay, trace gravel and topsoil	4.50
FILL: Topsoil and organics, some wood	5.03
Dense, brown **SILTY SAND to SANDY SILT**	5.64
Compact, brown **SILTY SAND**	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	50	10
SS	3	25	5
SS	4	17	4
SS	5	50	10
SS	6	92	7
SS	7	67	13
SS	8	83	32
SS	9	58	23
SS	10	75	17
SS	11	83	15
SS	12	83	8
SS	13	75	12
SS	14	33	11

DEPTH (m)	ELEV. (m)
0	76.88
1	75.88
2	74.88
3	73.88
4	72.88
5	71.88
6	70.88
7	69.88
8	68.88
9	67.88
10	66.88
11	65.88
12	64.88



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SOIL PROFILE AND TEST DATA

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Prop. Multi-Storey Building - 1300 & 1310 McWatters Rd.
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

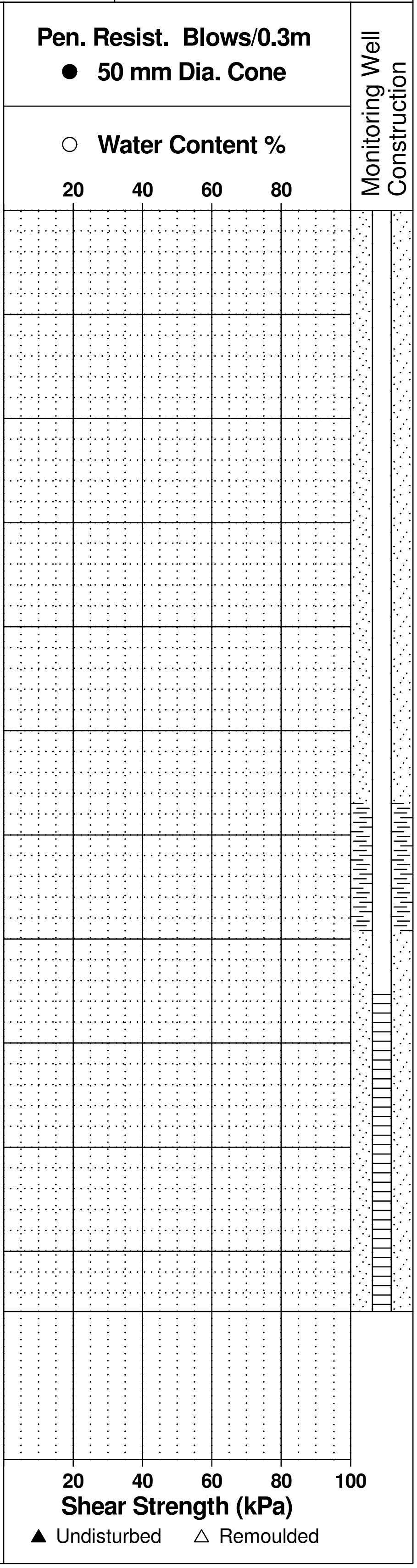
DATE April 30, 2021

FILE NO. PG5729

HOLE NO. BH 4-21

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
Compact, brown **SILTY SAND**	SS	15	33	23
	SS	16	50	20
- dense and grey by 15.2m depth	SS	17	42	49
	SS	18	33	37
	SS	19	33	41
19.25				
BEDROCK: Good quality, grey interbedded quartz sandstone and limestone	RC	1	100	78
	RC	2	100	88
22.58				
End of Borehole				
(GWL @ 7.53m - May 12, 2021)				

DEPTH (m)	ELEV. (m)
12	64.88
13	63.88
14	62.88
15	61.88
16	60.88
17	59.88
18	58.88
19	57.88
20	56.88
21	55.88
22	54.88



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building - 1300 & 1310 McWatters Rd.
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

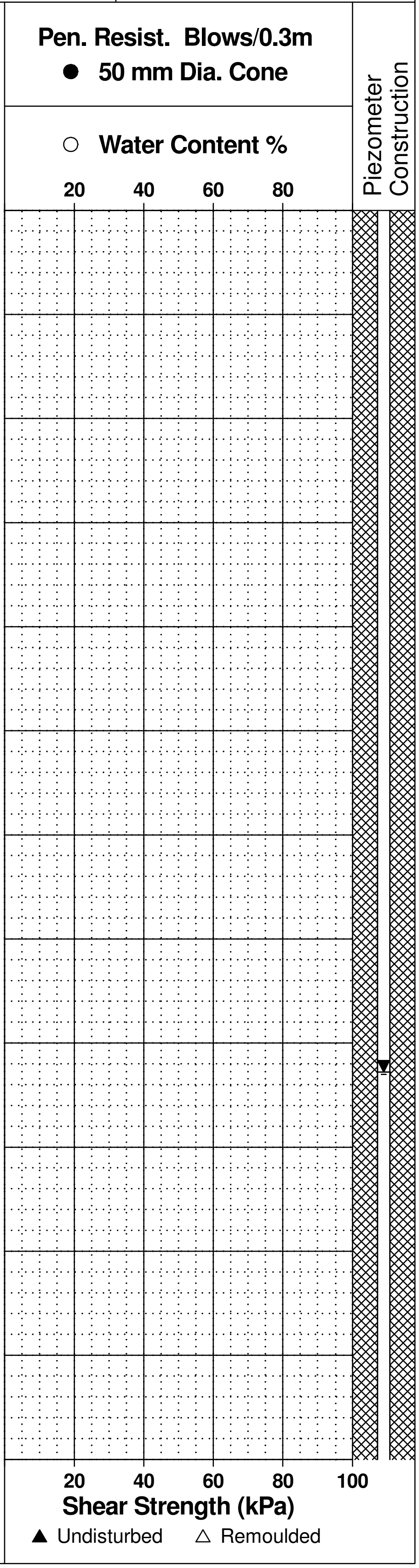
DATE May 3, 2021

FILE NO. PG5729

HOLE NO. BH 5-21

SOIL DESCRIPTION	STRATA DEPTH (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
TOPSOIL	0.10	AU	1		
FILL: Brown silty sand, trace gravel	1.45	SS	2	67	
FILL: Brown to grey silty clay with sand, trace topsoil	2.21	SS	3	67	34
FILL: Topsoil and organics, some wood - trace gravel by 4.0m depth	4.50	SS	4	75	9
		SS	5	58	10
		SS	6	58	18
Very dense to compact, brown **SILTY SAND** - loose by 7.6m depth - running sand from 8.4 to 11.0m depth	11.00	SS	7	67	55
		SS	8	42	21
		SS	9	58	15
		SS	10	50	14
		SS	11	58	6
		SS	12	58	3
		SS	13	100	7
Dense, grey **SILTY SAND** with clay and gravel, occasional cobbles		SS	14	58	31

DEPTH (m)	ELEV. (m)
0	76.89
1	75.89
2	74.89
3	73.89
4	72.89
5	71.89
6	70.89
7	69.89
8	68.89
9	67.89
10	66.89
11	65.89
12	64.89



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building - 1300 & 1310 McWatters Rd.
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE May 3, 2021

FILE NO. PG5729

HOLE NO. BH 5-21

SOIL DESCRIPTION

GROUND SURFACE

Dense, grey **SILTY SAND** with clay and gravel, occasional cobbles

14.63

Dense to compact, grey **SILTY SAND**

- running sand from 14.63 to 17.75m depth

- some gravel and occasional cobbles by 17.7m depth

18.06

BEDROCK: Good to excellent quality, grey interbedded quartz sandstone and limestone

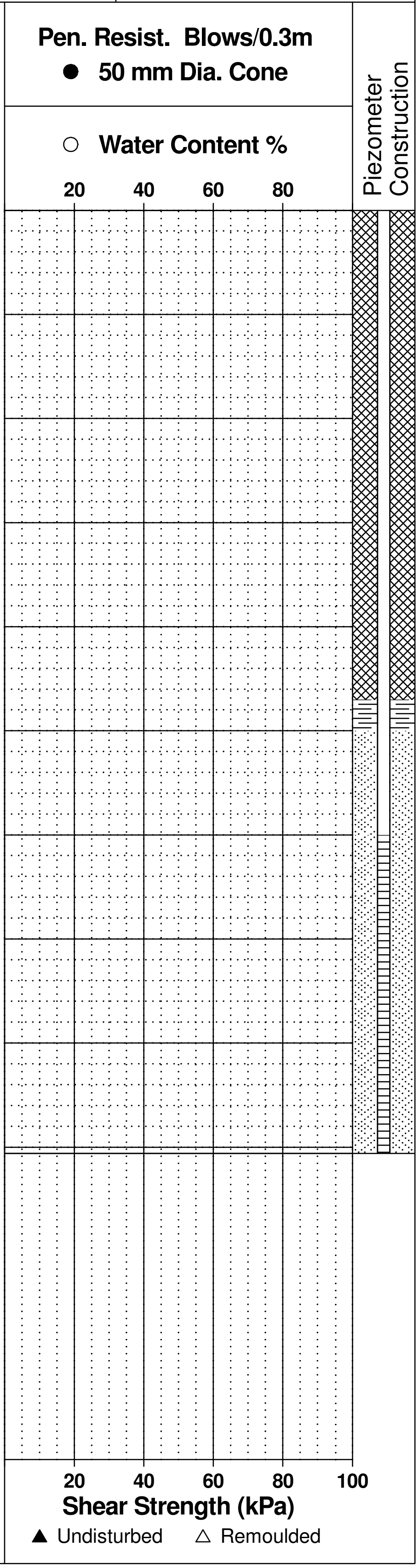
21.06

End of Borehole

(GWL @ 8.28m - May 12, 2021)

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
SS	15	42	32
SS	16	33	
SS	17	75	36
SS	18	75	18
SS	19	93	50+
RC	1	100	73
RC	2	100	90

DEPTH (m)	ELEV. (m)
12	64.89
13	63.89
14	62.89
15	61.89
16	60.89
17	59.89
18	58.89
19	57.89
20	56.89
21	55.89



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 = $(D30)^2 / (D10 \times D60)$
Cu - Uniformity coefficient = $D60 / D10$

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

CONSOLIDATION TEST

p'_o - Present effective overburden pressure at sample depth
p'_c - Preconsolidation pressure of (maximum past pressure on) sample
Ccr - Recompression index (in effect at pressures below p'_c)
Cc - Compression index (in effect at pressures above p'_c)

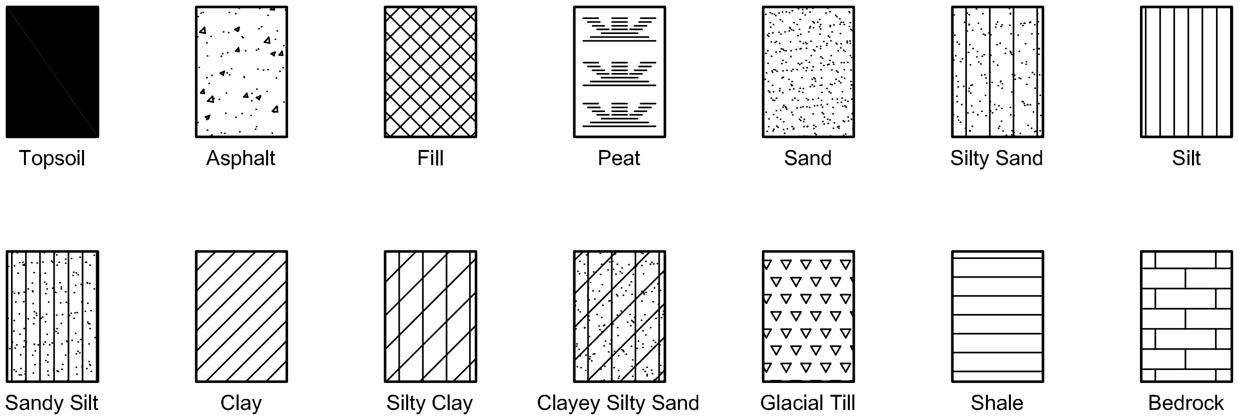
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

PERMEABILITY TEST

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

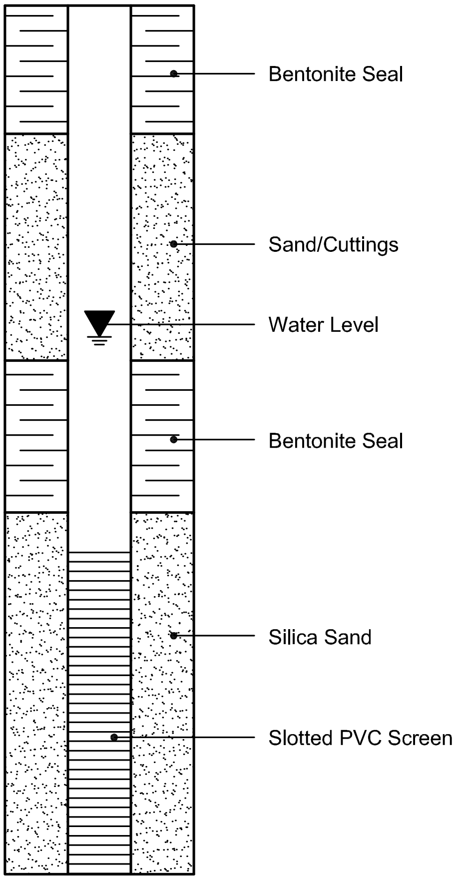
SYMBOLS AND TERMS (continued)

STRATA PLOT

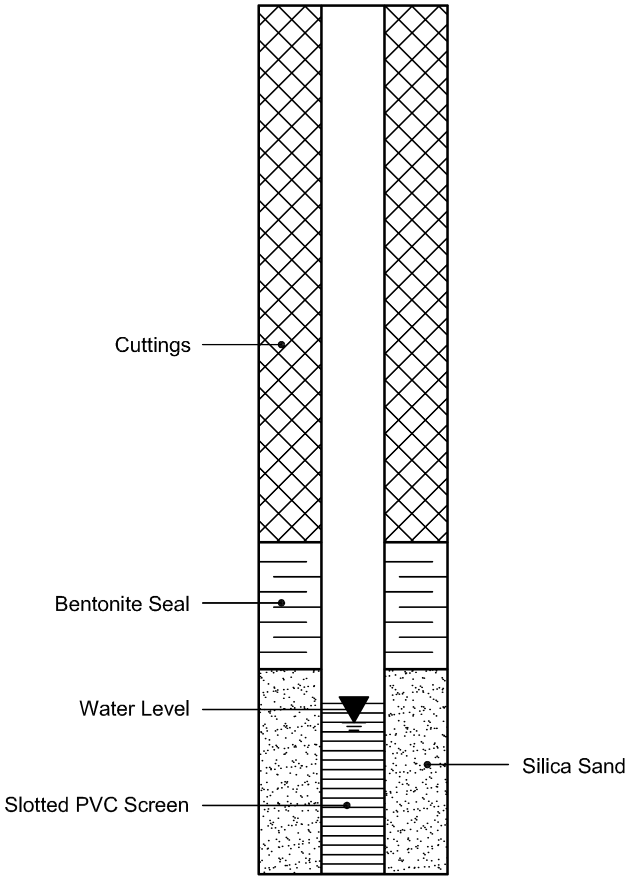


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Client: **Paterson Group Consulting Engineers**

Client PO: 31983

Report Date: 05-May-2021
Order Date: 30-Apr-2021
Project Description: PG5729

	Client ID:	BH 1-21-SS11	-	-	-
	Sample Date:	29-Apr-21 09:00	-	-	-
	Sample ID:	2118595-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	91.8	-	-	-
General Inorganics					
pH	0.05 pH Units	7.99	-	-	-
Resistivity	0.10 Ohm.m	143	-	-	-
Anions					
Chloride	5 ug/g dry	18	-	-	-
Sulphate	5 ug/g dry	<5	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURE 2 - AERIAL PHOTOGRAPH - 1965

FIGURE 3 - AERIAL PHOTOGRAPH - 2019

DRAWING PG5729-1 - TEST HOLE LOCATION PLAN

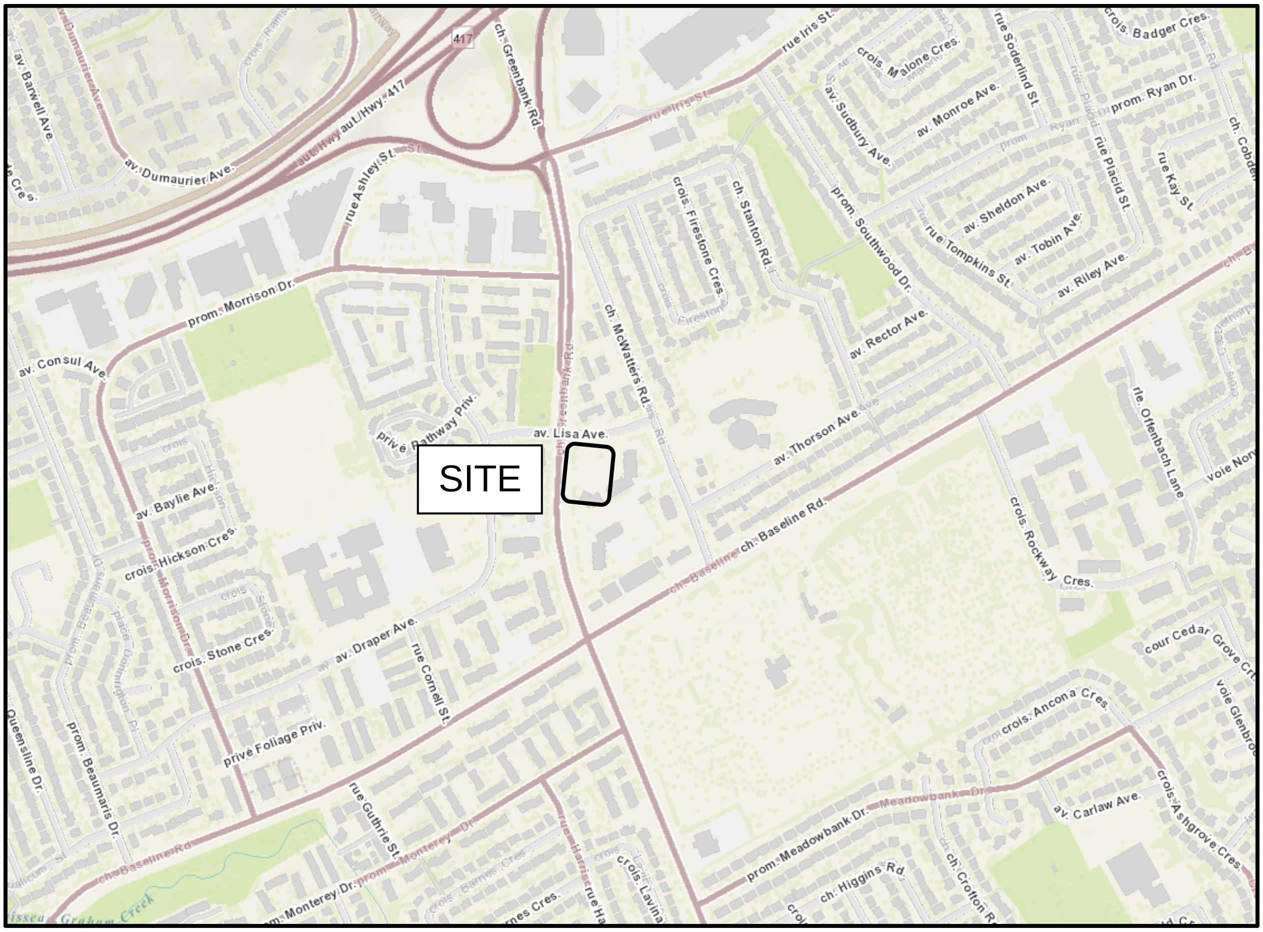


FIGURE 1

KEY PLAN

FIGURE 2

AERIAL PHOTOGRAPH - 1965

FIGURE 3

AERIAL PHOTOGRAPH - 2019

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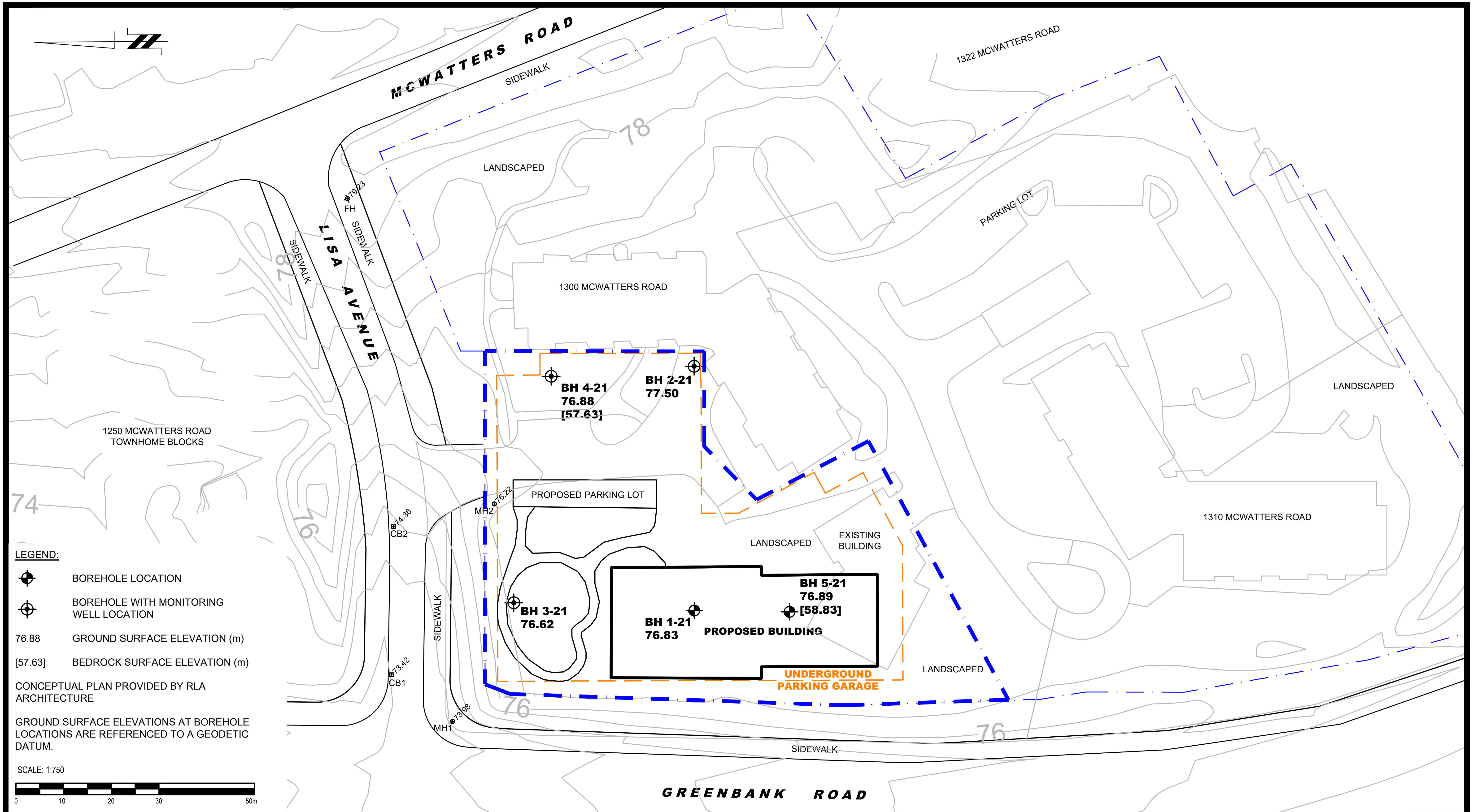

LEGEND:

⌖	BOREHOLE LOCATION
⌖	BOREHOLE WITH MONITORING WELL LOCATION
76.88	GROUND SURFACE ELEVATION (m)
[57.63]	BEDROCK SURFACE ELEVATION (m)

CONCEPTUAL PLAN PROVIDED BY RLA ARCHITECTURE

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

SCALE: 1:750

patersongroup
consulting engineers

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NO.	REVISIONS	DATE	INITIAL

HOMESTEAD LAND HOLDINGS LIMITED
GEOTECHNICAL INVESTIGATION
PROPOSED MULTI-STOREY BUILDING
1300 AND 1310 MCWATTERS ROAD

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:750
Date: 05/2021
Drawn by: YA
Report No.: PG5729-1
Checked by: KP
Dwg. No.: PG5729-1
Approved by: SD
Revision No.:

p:\autocad drawings\geotechnical\pg57xx\pg5729\pg5729-1-test hole location plan.dwg