Geotechnical Investigation

Proposed Mixed-Use Building

352 Somerset Street West
Ottawa, Ontario

Prepared for Capital Parking Inc.

Report PG4081-1 Revision 1 dated August 10, 2023

Table of Contents

Appendices

Appendix 1	Soil Profile and Test Data Sheets Symbols and Terms Analytical Testing Results
Appendix 2	Figure 1 - Key Plan Drawing PG4081-1 – Test Hole Location Plan

1.0 Introduction

Paterson Group (Paterson) was commissioned by Capital Parking Inc. to conduct a geotechnical investigation for the proposed mixed-use building to be located at 352 Somerset Street West, in the City of Ottawa (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 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 review of the most recent detailed design drawings, it is understood that the proposed project consists of the reconstruction of the existing building along with construction of a new building adjacent to the existing building. It is further understood that the existing building is being redeveloped and structurally upgraded. The proposed building will consist of a 3 storey mixed-use building with 1 basement level. Currently, the eastern half of the subject building has been demolished while the western half is currently being renovated.

Associated landscaped areas and access lanes are also anticipated. The proposed building will be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the investigation was carried out on November 15, 2019. At that time, three (3) boreholes were advanced within the footprint of the demolished eastern half of the property to a maximum depth of 11.3 m below existing grade. The test hole locations were distributed across the site in a manner to provide general coverage of the subject site. The locations of the test holes are shown on Drawing PG4081-1 – Test Hole Location Plan included in Appendix 2.

The boreholes were drilled using a truck-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 to the required depths at the selected locations, sampling and testing the overburden.

Sampling and In Situ Testing

Soil samples were collected from the boreholes using a 50 mm diameter split-spoon (SS) sampler or from the auger flights. The split-spoon and auger samples were classified on site, placed in sealed plastic bags, and transported to the laboratory for further review. The depths at which the split-spoon and auger samples were recovered from the boreholes are shown as SS and AU, 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.

Undrained shear strength testing, using a vane apparatus, was carried out at regular intervals of depth in cohesive soils.

The overburden thickness was also evaluated during the course of the investigation by dynamic cone penetration testing (DCPT) at one borehole location. The DCPT consists of driving a steel drill rod, equipped with a 50 mm diameter cone at its tip, using a 63.5 kg hammer falling from a height of 760 mm. The number of blows required to drive the cone into the soil is recorded for each 300 mm increment.

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

Groundwater

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

3.2 Field Survey

The test hole locations were determined in the field by Paterson personnel with consideration of underground utilities and existing site features. The ground surface elevations are referenced to a temporary benchmark (TBM), consisting of the top spindle of the fire hydrant located in between the front yard of 338 Somerset Street West. A geodetic elevation of 72.92 m was assigned to the TBM by Paterson group using a handheld GPS unit. The location of the TBM, boreholes and ground surface elevation at each borehole location are presented on Drawing PG4081-1 – Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

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

3.4 Analytical Testing

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

4.0 Observations

4.1 Surface Conditions

The subject site is located at the east corner of the intersection between Bank Street and Somerset Street West. The subject property is occupied by a former mixed use building with a stone and mortar foundation. The western half of the building is currently being renovated while the eastern half was demolished and backfilled to match the existing grade of the sidewalk on Somerset Street West.

The site is bordered to the north by Somerset Street West, to the west by Bank Street, to the south by a two storey mixed use building and to the east by an asphaltic parking lot.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile at the borehole locations consists of a thick layer of fill consisting of brown silty clay with some sand and gravel (up to 4.6 m in thickness). This fill layer was followed by stiff to firm grey silty clay and glacial till. The glacial till layer was found to consist of dark grey silty sand with gravel, cobbles and boulders. Practical refusal to DCPT was encountered at a depth pf 13.5 m below existing grade in BH 1.

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

Bedrock

Based on available geological mapping, the bedrock in this area consists of shale of the Billings Formation. The overburden drift thickness is expected to be 10 to 15 m of depth.

4.3 Groundwater

The measured groundwater levels in the boreholes are presented in Table 1 below. Groundwater levels could also be estimated based on the colouring, consistency and moisture levels of the recovered samples. Based on these observations, the long-term groundwater table is estimated at a depth of 4.5 to 5.5 m below existing grade.

Table 1 – Summary of Groundwater Level Readings

Borehole Number	Ground Surface Elevation (m)	Measured Groundwater Level		Date Recorded
		Depth (m)	Elevation (m)	
BH 1	99.52	9.91	89.61	December 3, 2019
BH 2	99.59	4.71	94.88	December 3, 2019
BH 3	99.73	7.52	92.21	December 3, 2019

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 adequate for the proposed multi- storey building. It is expected that the proposed building will be founded on conventional shallow footings and/or raft slab placed on an undisturbed stiff to firm silty clay bearing surface.

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

5.2 Site Grading and Preparation

Stripping Depth

All topsoil, construction debris and deleterious materials, such as those containing organic materials, should be removed from under any buildings, paved area, pipe bedding and other settlement sensitive structures. Existing foundation walls and other construction debris should be entirely removed from within the building footprint. 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 development should consist of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. This material should be tested and approved prior to delivery to the site. The fill should be placed in lifts no greater than 300 mm thick and compacted using suitable compaction equipment for the lift thickness. Fill placed beneath buildings and paved areas should be compacted to at least 98% of the material's standard Proctor maximum dry density (SPMDD).

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

5.3 Foundation Design

Bearing Resistance Values

Pad footings up to 5 m wide, and strip footings up to 3 m wide, placed on an undisturbed, stiff silty clay bearing surface, or on engineered fill placed directly over the undisturbed stiff silty clay, can be designed using a bearing resistance value at SLS of **100 kPa** and a factored bearing resistance value at ULS of **180 kPa**. A geotechnical factor of 0.5 was applied to the bearing resistance value at ULS.

Pad footings up to 5 m wide, and strip footings up to 3 m wide, placed on an undisturbed, firm silty clay bearing surface, or on engineered fill placed directly over the undisturbed firm silty clay, can be designed using a bearing resistance value at SLS of **80 kPa** and a factored bearing resistance value at ULS of **150 kPa**. A geotechnical factor of 0.5 was applied to the bearing resistance value at ULS.

The bearing resistance values are provided on the assumption that the footings will be placed on undisturbed soil bearing surfaces. An undisturbed soil bearing surface consists of one 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.

Settlement

The bearing resistance values at SLS for shallow footing bearing on the above noted soils will be subjected to potential post-construction total and differential settlements of 25 and 15 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 a sound bedrock bearing medium when a plane extending down and out from the bottom edge of the footing at a minimum of 1H:6V (or flatter) passes only through sound bedrock or a material of the same or higher capacity as the bedrock, such as concrete. A weathered bedrock bearing medium will require a lateral support zone of 1H:1V (or flatter).

Permissible Grade Raise

Due to the presence of a sensitive marine silty clay deposit, it is recommended that a permissible grade raise restriction of **3 m** be used for the subject site.

Raft Foundation

For support of the proposed multi-storey buildings, consideration should be given to using a raft foundation due to the expected building loads.

The amount of settlement of the raft slab will be dependent on the sustained raft contact pressure. The bearing resistance value at SLS (contact pressure) of **180 kPa** can be used for design purposes. 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 contact pressure provided considers the stress relief associated with the soil removal associated with one underground parking level. The factored bearing resistance (contact pressure) at ULS can be taken as **300 kPa**. A geotechnical resistance factor of 0.5 was applied to the bearing resistance value at ULS.

Based on the following assumptions for the raft foundation, the proposed building can be designed using the above parameters with a total and differential settlement of 25 and 15 mm, respectively.

Base on a single underground parking level or more it is expected that the raft foundation will be installed on the silty clay deposit. The modulus of subgrade reaction was calculated to be **6.8 Mpa/m** for a contact pressure of **180 kPa**. The design of the raft foundation is required to consider the relative stiffness of the reinforced concrete slab and the supporting bearing medium.

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class C** for the foundations considered at this site. The soils underlying the subject site are not susceptible to liquefaction. Refer to the latest revision of the Ontario Building Code for a full discussion of the earthquake design requirements.

5.5 Basement Slab

With the removal of the topsoil layer and fill, containing deleterious or organic materials, the native soil or engineered fill will be considered to be an acceptable subgrade surface on which to commence backfilling for the basement slab construction. Any soft areas should be removed and backfilled with appropriate backfill material. OPSS Granular A or Granular B Type II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab.

It is recommended for basement slabs that the upper 200 mm of sub-slab fill consist of 19 mm clear crushed stone. All backfill materials within the footprint of the proposed building should be placed in maximum 300 mm thick loose layers and compacted to at least 98% of its SPMDD.

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 18 kN/m^3.

However, undrained conditions are anticipated (i.e. below the groundwater level). Therefore, the applicable effective (undrained) unit weight of the retained soil can be taken as 11 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 the 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 the OBC 2012.

5.7 Pavement Design

Pavement Structure Over Overburden

The recommended pavement structures for the subject site are shown in Tables 2 and 3.

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

Thickness (mm)	Material Description
50	**Wear Course** - HL-3 or Superpave 12.5 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
300	**SUBBASE** - OPSS Granular B Type **II**
SUBGRADE - Either fill, in situ silty clay or OPSS Granular B Type **I** or **II** material placed over in situ soil or fill	

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

Thickness (mm)	Material Description
40	**Wear Course** - HL-3 or Superpave 12.5 Asphaltic Concrete
50	**Binder Course** - HL-8 or Superpave 19.0 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
450	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Either fill, in situ silty clay or OPSS Granular B Type I or II material placed over in situ soil or fill	

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

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 100% of the material's SPMDD using suitable compaction equipment.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

It is recommended that a perimeter foundation drainage system be provided for the proposed structures. The system should consist of a 150 mm diameter, geotextile- wrapped, perforated, corrugated, plastic pipe, surrounded on all sides by 150 mm of 10 mm clear crushed stone, placed at the footing level around the exterior perimeter of the structure. The pipe should be mechanically connected to the existing drainage system of the existing building or have a positive outlet, such as a gravity connection to the storm sewer.

Backfill against the exterior sides of the foundation walls should consist of free-draining, non frost susceptible granular materials. The greater part of the site excavated materials will be frost susceptible and, as such, are not recommended for re-use as backfill against foundation walls, unless used in conjunction with a drainage geocomposite, such as Miradrain G100N or Delta Drain 6000, connected to the perimeter foundation drainage system. Imported granular materials, such as clean sand or OPSS Granular B Type 1 granular material, should otherwise be used for this purpose.

Underfloor Drainage

Underfloor drainage may be required to control water infiltration below the basement slab. For design purposes, it’s recommended that a 150 mm diameter perforated pipe be placed for the perimeter and in specific interior bays. 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.

6.2 Protection 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 alone, or a combination of soil cover and 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 structure proper and require additional protection, such as soil cover of 2.1 m or a combination of soil cover and foundation insulation.

6.3 Excavation Side Slopes

At this site, sure to the close proximity of the surrounding roads and buildings and the proposed basement level, temporary shoring will be required to complete the required excavations.

Excavation Side Slopes

The subsoil at this site is considered to be mainly a Type 2 or 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects. The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be cut back at 1H:1V or flatter. The flatter slope is required for excavation below the groundwater level.

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

If a temporary shoring system is considered, the design and implementation of these temporary systems will be the responsibility of the excavation contractor and their design team. Inspections and approval of the temporary system will also be the responsibility of the designer. Geotechnical information provided below is to assist the designer in completing a suitable and safe shoring system. The designer should take into account the potential for a fully saturated condition following a significant precipitation event. Any changes to the approved shoring design system should be reported immediately to the owner’s representative prior to implementation.

For design purposes, the temporary system may consist of soldier pile and lagging system or interlocking steel sheet piling. Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be added to the earth pressures described below. These systems can be cantilevered, anchored, or braced.

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

Table 4 - Soil Parameters for Shoring System Design	
Parameter	**Value**
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	18
Submerged Unit Weight (γ'), kN/m^3	11

Generally, it is expected that the shoring systems will be provided with tie-back rock anchors to ensure their stability. It is further recommended that the toe of the shoring be adequately supported to resist toe failure.

The geotechnical design of grouted rock anchors in sedimentary bedrock is based upon two possible failure modes. The anchor can fail either by shear failure along the grout/rock interface or by pullout of a 60 to 90 degree cone of rock with the apex of the cone near the middle of the bonded length of the anchor.

The anchor derives its capacity from the bonded portion, or fixed anchor length, at the base of the anchor. An unbonded portion, or free anchor length, is also usually provided between the rock surface and the start of the bonded length. A factored tensile grout to rock bond resistance value at ULS of **1.0 MPa**, incorporating a resistance factor of 0.3, can be used. A minimum grout strength of 40 MPa is recommended.

It is recommended that the anchor drill hole diameter be within 1.5 to 2 times the rock anchor tendon diameter and the anchor drill holes be inspected by geotechnical personnel and should be flushed clean prior to grouting. The use of a grout tube to place grout from the bottom up in the anchor holes is further recommended.

The geotechnical capacity of each rock anchor should be proof tested at the time of construction. More information on testing can be provided upon request. Compressive strength testing is recommended to be completed for the rock anchor grout. A set of grout cubes should be tested for each day grout is prepared.

Soldier Pile and Lagging System

The active earth pressure acting on a soldier pile and lagging shoring system can be calculated using a rectangular earth pressure distribution with a maximum pressure of 0.65 K γ H for strutted or anchored shoring or a triangular earth pressure distribution with a maximum value of K γ H for a cantilever shoring system. H is the height of the excavation.

The active earth pressure should be used where wall movements are permissible while the at-rest pressure should be used if no movement is permissible.

The total unit weight should be used above the groundwater level while the submerged unit weight should be used below the groundwater level.

The hydrostatic groundwater pressure should be added to the earth pressure distribution wherever the submerged unit weights are used for earth pressure calculations should the level on the groundwater not be lowered below the bottom of the excavation. If the groundwater level is lowered, the total unit weight for the soil should be used full weight, with no hydrostatic groundwater pressure component.

Underpinning

Founding conditions of adjacent structures bordering the proposed building locations should be assessed and underpinning requirements should be evaluated based on proximity to the proposed building location.

6.4 Pipe Bedding and Backfill

At least 150 mm of OPSS Granular A should be used for pipe bedding for sewer and water pipes placed. The bedding should extend to the spring line of the pipe. Cover material, from the spring line to at least 300 mm above the obvert of the pipe should consist of OPSS Granular A. The bedding and cover materials should be placed in maximum 225 mm thick lifts compacted to a minimum of 95% of the material's SPMDD.

It should generally be possible to re-use most brown silty clay above the cover material if the excavation and filling 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) should match the soils exposed at the trench walls to minimize differential frost heaving. The trench backfill should be placed in maximum 300 mm thick loose lifts and compacted to a minimum of 95% of the material's SPMDD.

Clay Seals

To reduce long term lowering of the groundwater level at this site, clay seals should be provided in the service trenches. The seals should be at least 1.5 m long and should extend from trench wall to trench wall. Generally, the seals should extend from the frost line and fully penetrate the bedding, subbedding and cover material. The barriers should consist of relatively fry and compactable brown silty clay placed in maximum 225 mm thick loose layers and compacted to a minimum of 95% of the material's SPMDD. The clay seals should be placed at the site boundaries and at strategic locations at no more than 60 m intervals in the service trenches.

6.5 Groundwater Control

Groundwater Control for Building Construction

It is anticipated that groundwater infiltration into the excavations should be controllable using open sumps. Pumping from open sumps should be sufficient to control the groundwater influx through the sides of shallow excavations. 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.

Long-term Groundwater Control

Our recommendations for the proposed building's long-term groundwater control are presented in Subsection 6.1. Any groundwater encountered along the building's perimeter or sub-slab drainage system will be directed to the proposed building's sump pit. It is expected that groundwater flow will be low (i.e.- less than 15,000 L/day) with peak periods noted after rain events. It is anticipated that the groundwater flow will be controllable using conventional open sumps. Confirmation of the actual groundwater flow should be completed by the geotechnical consultant at the time of construction.

Impacts on Neighboring Structures

It is understood that one basement level is planned for the proposed building. Based on the existing groundwater level and considering the proposed building will be surrounded by a waterproofing membrane, groundwater lowering will be minimal to negligible and will take place within a limited range of the proposed buildings. Based on the proximity of neighbouring buildings and minimal zone impacted by the groundwater lowering, the proposed development will not negatively impact the neighbouring structures. It should be noted that no issues are expected with respect to groundwater lowering that would cause long term damage to adjacent structures surrounding the proposed buildings.

6.6 Winter Construction

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

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

Trench excavations and pavement construction are difficult activities to complete during winter without introducing frost in the excavation subgrade base or walls. Precautions should be considered if such activities are to be completed during sub-zero temperatures.

6.7 Corrosion Potential and Sulphate

The results of analytical testing show that the sulphate content is less than 0.1%. These results are indicative that Type 10 Portland cement (normal cement) would be appropriate for this site. The results of the chloride content, pH and resistivity indicate the presence of an aggressive environment for exposed ferrous metals at this site.

7.0 Recommendations

It is recommended that the following be carried out by Paterson once preliminary and future details of the proposed development have been prepared:

- ❑ Review of detailed grading, servicing, landscaping and structural plan(s) from a geotechnical perspective.
- ❑ Review of the geotechnical aspects of the excavation contractor's shoring design, if not by Paterson, prior to construction.
- ❑ Review of architectural plans pertaining to groundwater suppression systems, underfloor drainage systems and waterproofing details for elevator shafts.

It is a requirement for the foundation design data provided herein to be applicable that a material testing and observation program be performed by the geotechnical consultant. The following aspects of the program should be performed by Paterson:

- ❑ Review and inspection of the installation of the foundation drainage systems.
- ❑ 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 the completion of a satisfactory inspection program by the geotechnical consultant.

All excess soil must be handled as per *Ontario Regulation 406/19: On-Site and Excess Soil Management*.

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 Capital Parking Inc. or their agents is not authorized without review by Paterson for the applicability of our recommendations to the alternative use of the report.

Paterson Group Inc.

Owen Canton, B.Eng.

Faisal I. Abou-Seido, P.Eng.

Report Distribution:

- ❑ Capital Parking Inc.
- ❑ Paterson Group (1 Copy)

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
Prop. Multi-Storey Building - 352 Somerset St. W.
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located in front of 338 Somerset Street West, geodetic elevation = 72.92m.

REMARKS

BORINGS BY CME 55 Power Auger **DATE** November 15, 2019

FILE NO. PG4081

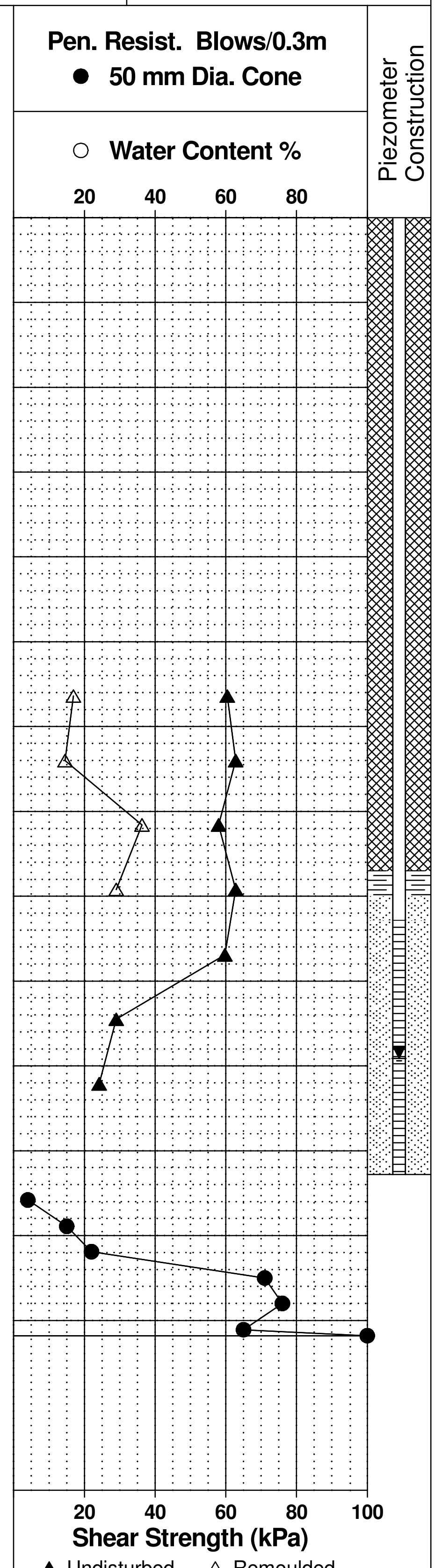
HOLE NO. BH 1

SOIL DESCRIPTION	Layer bottom depth (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
Ground Surface					
FILL: Brown sand with gravel and crushed stone	0.46	AU	1		
FILL: Brown silty clay with sand and gravel	4.57	SS	2	21	7
		SS	3	12	3
		SS	4	29	2
		SS	5	21	7
		SS	6	100	1
Stiff to firm, grey **SILTY CLAY**	10.67	SS	7	100	1
		SS	8	67	W
GLACIAL TILL: Dark grey silty sand with gravel, trace clay, cobbles, boulders	11.28	SS	9	54	13
Dynamic cone Penetration Test commenced at 11.28m depth. Inferred **GLACIAL TILL**	13.18				
End of Borehole					

Practical DCPT refusal at 13.18m depth.

(GWL @ 9.91m - Dec. 3, 2019)

DEPTH (m)	ELEV. (m)
0	72.44
1	71.44
2	70.44
3	69.44
4	68.44
5	67.44
6	66.44
7	65.44
8	64.44
9	63.44
10	62.44
11	61.44
12	60.44
13	59.44



patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building - 352 Somerset St. W.
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located in front of 338 Somerset Street West, geodetic elevation = 72.92m.

REMARKS

BORINGS BY CME 55 Power Auger

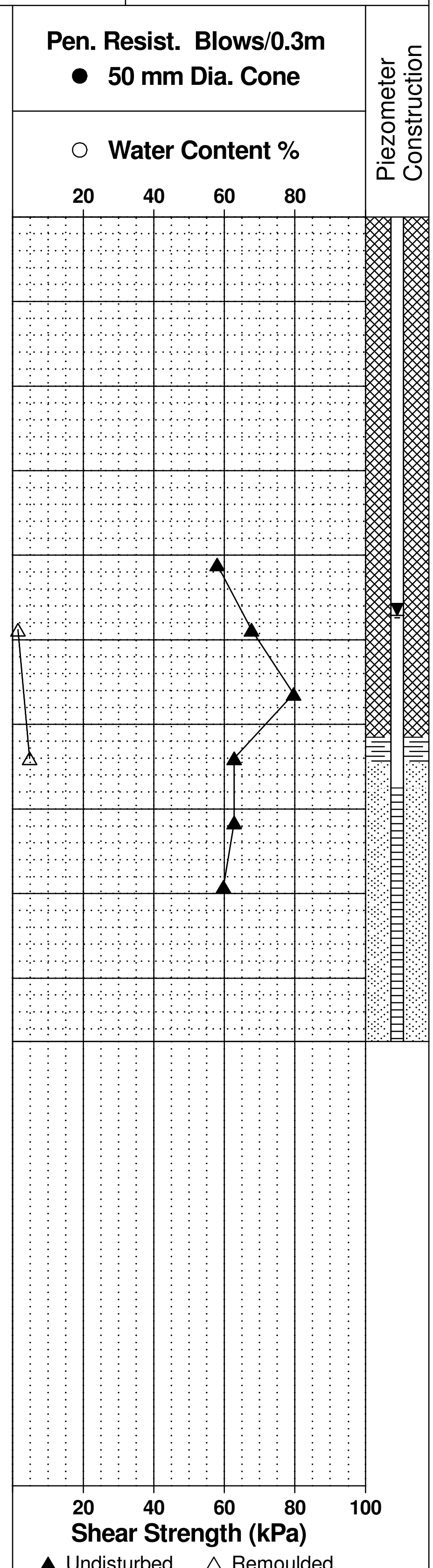
DATE November 15, 2019

FILE NO. PG4081

HOLE NO. BH 2

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
Ground Surface					
FILL: Brown sand with gravel and crushed stone	0.46	AU	1		
FILL: Brown silty clay with sand and gravel		SS	2	21	2
		SS	3	29	1
		SS	4	29	3
	3.50	SS	5	46	9
Stiff, brown **SILTY CLAY** - grey by 4.6m depth		SS	6	58	W
	8.38				
GLACIAL TILL: Grey silty clay with sand, gravel, cobbles and boulders		SS	7	71	P
	9.75	SS	8	100	5
End of Borehole					
(GWL @ 471m - Dec. 3, 2019)					

DEPTH (m)	ELEV. (m)
0	72.51
1	71.51
2	70.51
3	69.51
4	68.51
5	67.51
6	66.51
7	65.51
8	64.51
9	63.51



patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building - 352 Somerset St. W.
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located in front of 338 Somerset Street West, geodetic elevation = 72.92m.

REMARKS

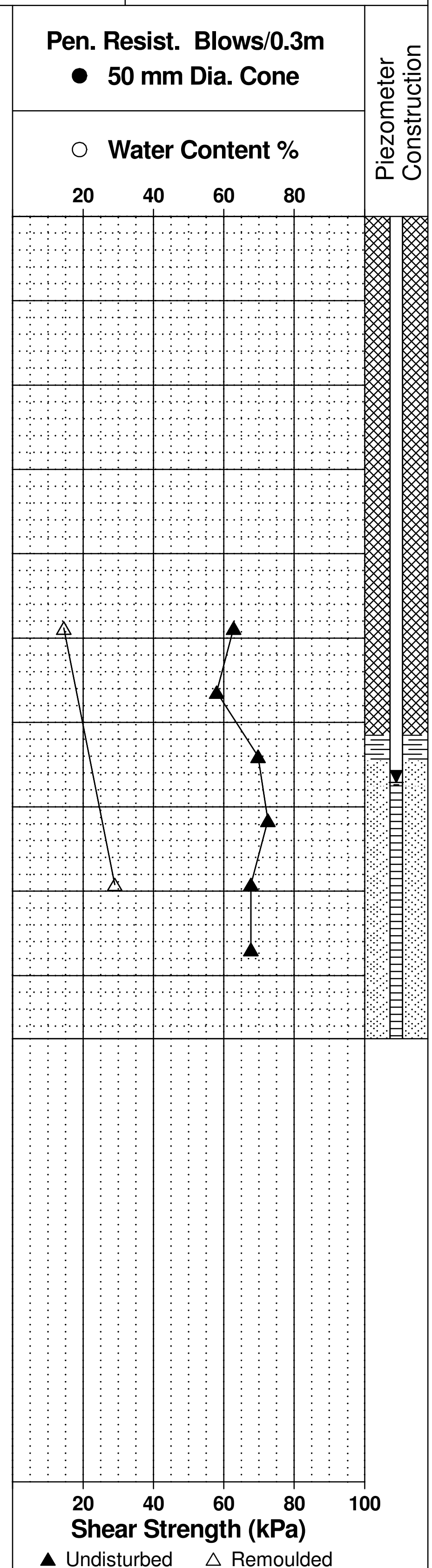
BORINGS BY CME 55 Power Auger

DATE November 15, 2019

FILE NO. PG4081

HOLE NO. BH 3

SOIL DESCRIPTION	TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
Ground Surface					0	72.65
FILL: Crushed concrete with cobbles, sand and gravel 0.56	AU	1				
FILL: Brown silty clay, some sand and gravel	SS	2	29	3	1	71.65
	SS	3	21	4	2	70.65
	SS	4	8	3		
3.81	SS	5	21	4	3	69.65
Stiff, brown **SILTY CLAY**	SS	6	96	1	4	68.65
- grey by 4.3m depth					5	67.65
					6	66.65
	SS	7	100	W	7	65.65
					8	64.65
9.14					9	63.65
GLACIAL TILL: Grey silty clay with sand, gravel, cobbles and boulders 9.75	SS		100	3		
End of Borehole						
(GWL @ 6.71m - Dec. 3, 2019)						



SYMBOLS AND TERMS

SOIL DESCRIPTION

Behavioural properties, such as structure and strength, take precedence over particle gradation in describing soils. Terminology describing soil structure are as follows:

Desiccated	-	having visible signs of weathering by oxidation of clay minerals, shrinkage cracks, etc.
Fissured	-	having cracks, and hence a blocky structure.
Varved	-	composed of regular alternating layers of silt and clay.
Stratified	-	composed of alternating layers of different soil types, e.g. silt and sand or silt and clay.
Well-Graded	-	Having wide range in grain sizes and substantial amounts of all intermediate particle sizes (see Grain Size Distribution).
Uniformly-Graded	-	Predominantly of one grain size (see Grain Size Distribution).

The standard terminology to describe the strength of cohesionless soils is the relative density, usually inferred from the results of the Standard Penetration Test (SPT) 'N' value. The SPT N value is the number of blows of a 63.5 kg hammer, falling 760 mm, required to drive a 51 mm O.D. split spoon sampler 300 mm into the soil after an initial penetration of 150 mm.

Relative Density	'N' Value	Relative Density %
Very Loose	<4	<15
Loose	4-10	15-35
Compact	10-30	35-65
Dense	30-50	65-85
Very Dense	>50	>85

The standard terminology to describe the strength of cohesive soils is the consistency, which is based on the undisturbed undrained shear strength as measured by the in situ or laboratory vane tests, penetrometer tests, unconfined compression tests, or occasionally by Standard Penetration Tests.

Consistency	Undrained Shear Strength (kPa)	'N' Value
Very Soft	<12	<2
Soft	12-25	2-4
Firm	25-50	4-8
Stiff	50-100	8-15
Very Stiff	100-200	15-30
Hard	>200	>30

SOIL DESCRIPTION (continued)

Cohesive soils can also be classified according to their “sensitivity”. The sensitivity is the ratio between the undisturbed undrained shear strength and the remoulded undrained shear strength of the soil.

Terminology used for describing soil strata based upon texture, or the proportion of individual particle sizes present is provided on the Textural Soil Classification Chart at the end of this information package.

ROCK DESCRIPTION

The structural description of the bedrock mass is based on the Rock Quality Designation (RQD).

The RQD classification is based on a modified core recovery percentage in which all pieces of sound core over 100 mm long are counted as recovery. The smaller pieces are considered to be a result of closely-spaced discontinuities (resulting from shearing, jointing, faulting, or weathering) in the rock mass and are not counted. RQD is ideally determined from NXL size core. However, it can be used on smaller core sizes, such as BX, if the bulk of the fractures caused by drilling stresses (called “mechanical breaks”) are easily distinguishable from the normal in situ fractures.

RQD %	ROCK QUALITY
90-100	Excellent, intact, very sound
75-90	Good, massive, moderately jointed or sound
50-75	Fair, blocky and seamy, fractured
25-50	Poor, shattered and very seamy or blocky, severely fractured
0-25	Very poor, crushed, very severely fractured

SAMPLE TYPES

SS	-	Split spoon sample (obtained in conjunction with the performing of the Standard Penetration Test (SPT))
TW	-	Thin wall tube or Shelby tube
PS	-	Piston sample
AU	-	Auger sample or bulk sample
WS	-	Wash sample
RC	-	Rock core sample (Core bit size AXT, BXL, etc.). Rock core samples are obtained with the use of standard diamond drilling bits.

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

MC%	-	Natural moisture content or water content of sample, %
LL	-	Liquid Limit, % (water content above which soil behaves as a liquid)
PL	-	Plastic limit, % (water content above which soil behaves plastically)
PI	-	Plasticity index, % (difference between LL and PL)
Dxx	-	Grain size which xx% of the soil, by weight, is of finer grain sizes These grain size descriptions are not used below 0.075 mm grain size
D10	-	Grain size at which 10% of the soil is finer (effective grain size)
D60	-	Grain size at which 60% of the soil is finer
Cc	-	Concavity coefficient = $(D30)^2 / (D10 \times D60)$
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

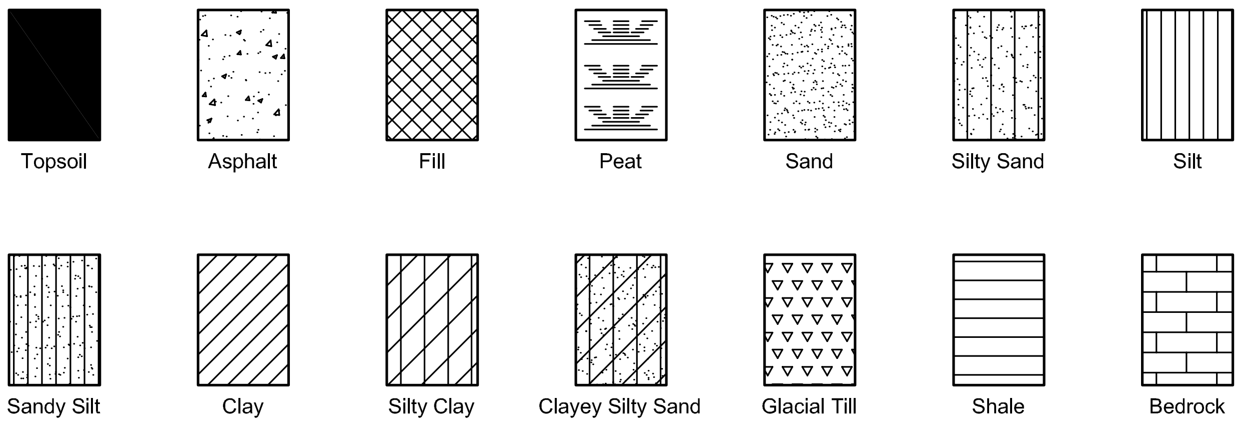
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void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

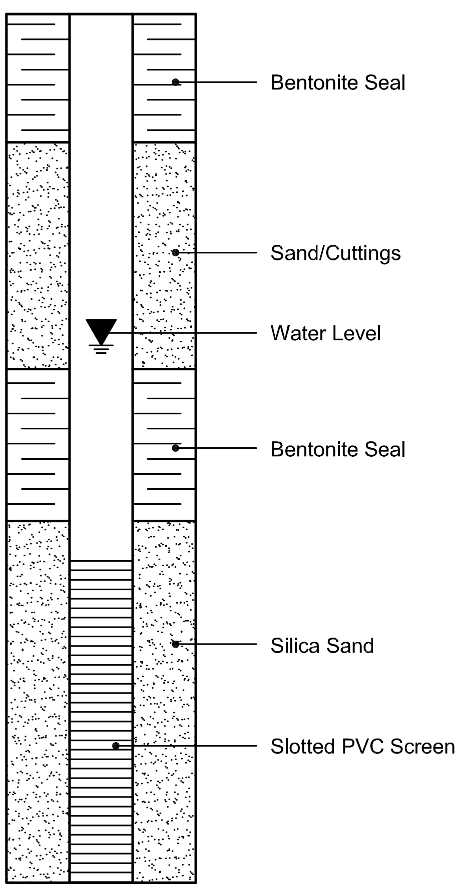
SYMBOLS AND TERMS (continued)

STRATA PLOT

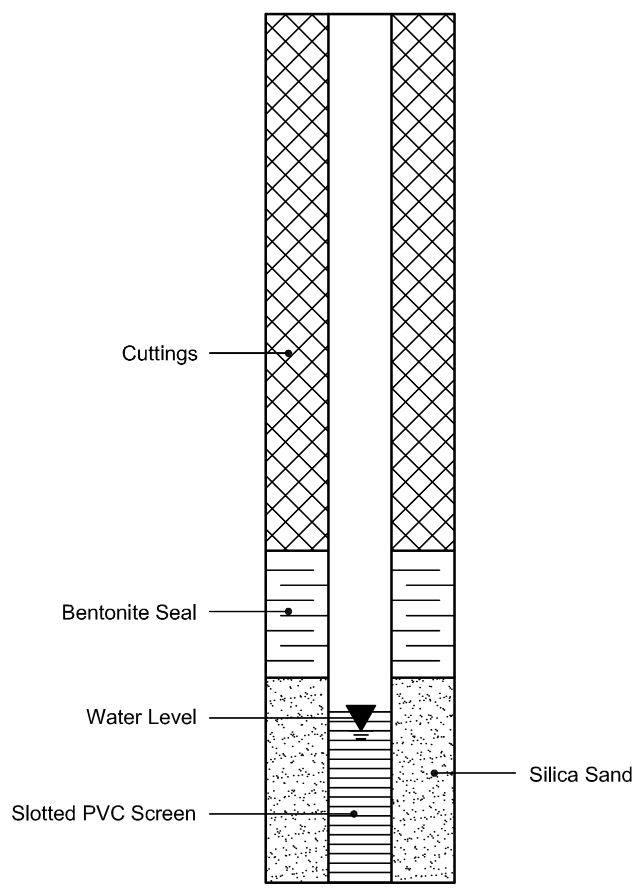


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 29140

Report Date: 20-Nov-2019

Order Date: 18-Nov-2019

Project Description: PG4081

	Client ID:	BH2-SS3 5'-7'	-	-	-
	Sample Date:	15-Nov-19 13:00	-	-	-
	Sample ID:	1947110-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	81.0	-	-	-
General Inorganics					
pH	0.05 pH Units	7.25	-	-	-
Resistivity	0.10 Ohm.m	20.1	-	-	-
Anions					
Chloride	5 ug/g dry	34	-	-	-
Sulphate	5 ug/g dry	301	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING PG4081-1 – TEST HOLE LOCATION PLAN

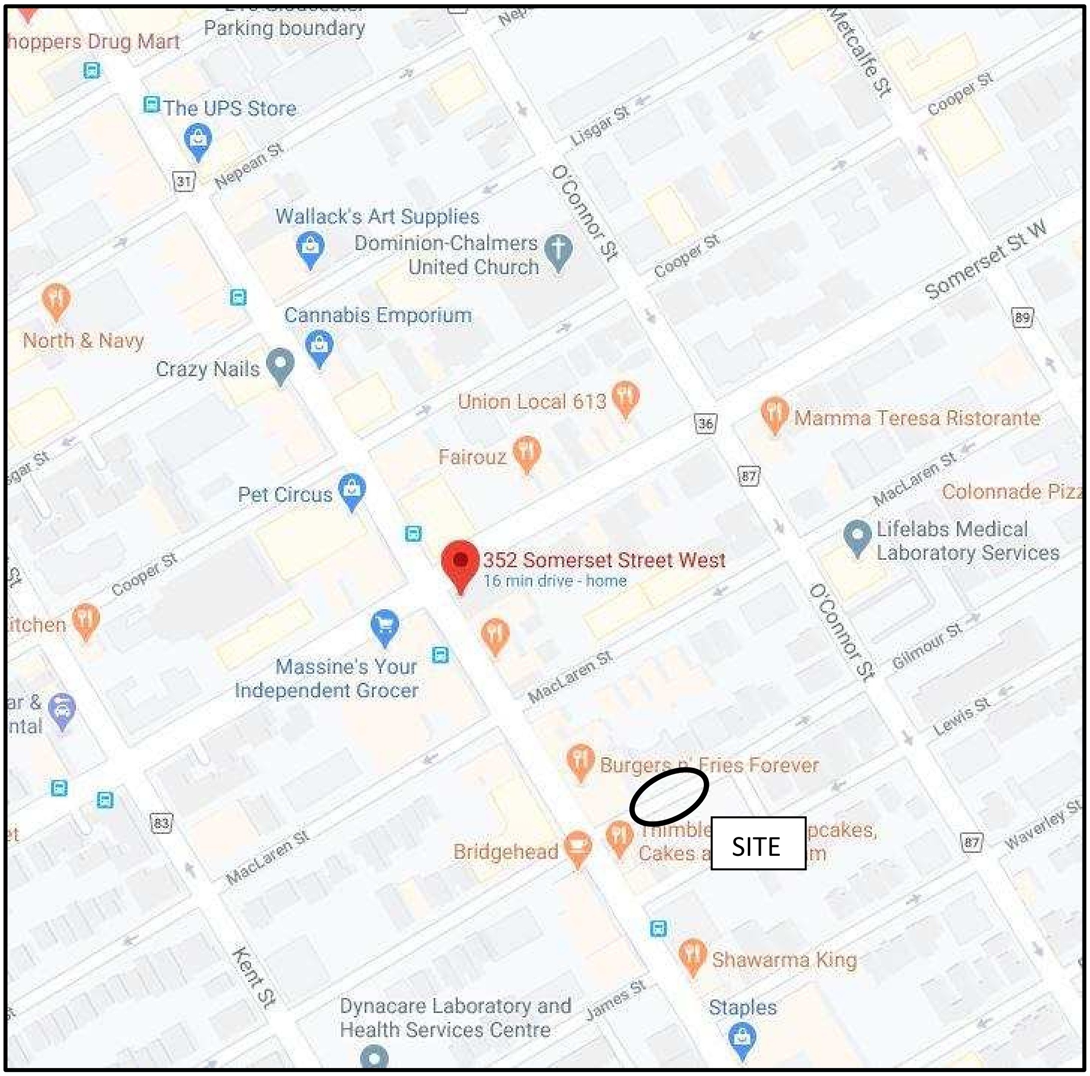
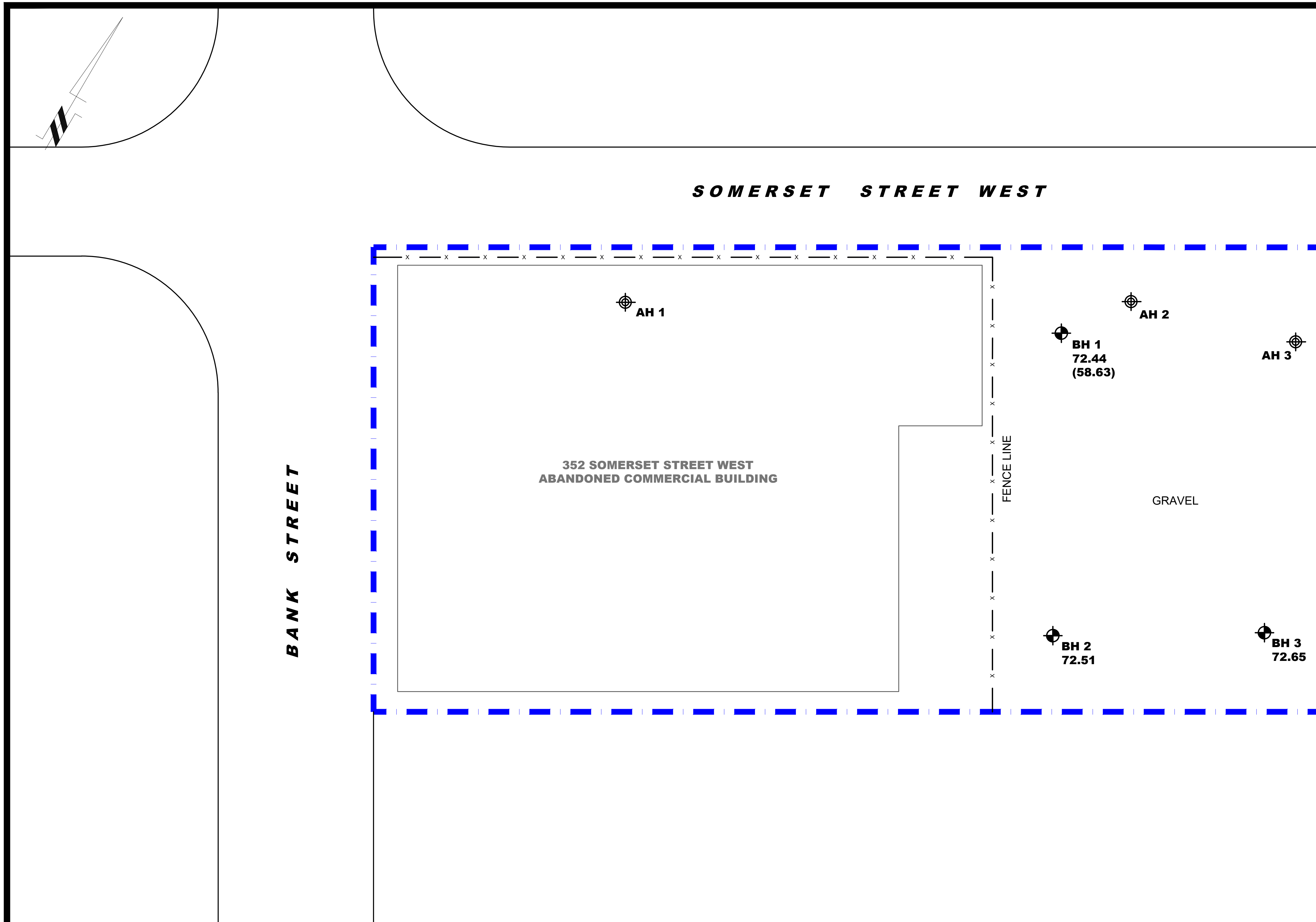


FIGURE 1 – KEY PLAN

PATERSON GROUP



LEGEND:

BOREHOLE LOCATION

HAND AUGER LOCATION (PATERSON GROUP REPORT PG0971, APRIL, 2007)

99.52 GROUND SURFACE ELEVATION (m)

(86.34) PRACTICAL REFUSAL TO DCPT ELEVATION (m)

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

SCALE: 1:150

0 1 2 3 4 5 10m

p:\autocad drawings\geotechnical\pg40xx\pg4081\pg4081-1-test hole location plan (rev.01).dwg

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

NO.	REVISIONS	DATE	INITIAL
1	REVISED TO INCLUDE ADJUSTED BOREHOLE ELEVATIONS TO A GEODETIC DATUM.	08/08/2023	OC

CAPITAL PARKING INC.

GEOTECHNICAL INVESTIGATION
PROP. MULTI-STOREY BUILDING - 352 SOMERSET STREET WEST

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:150	Date: 12/2019
Drawn by: YA	Report No.: PG4081-REP.01
Checked by: FA	Dwg. No.: PG4081-1
Approved by: FA	Revision No.: 1

11x17