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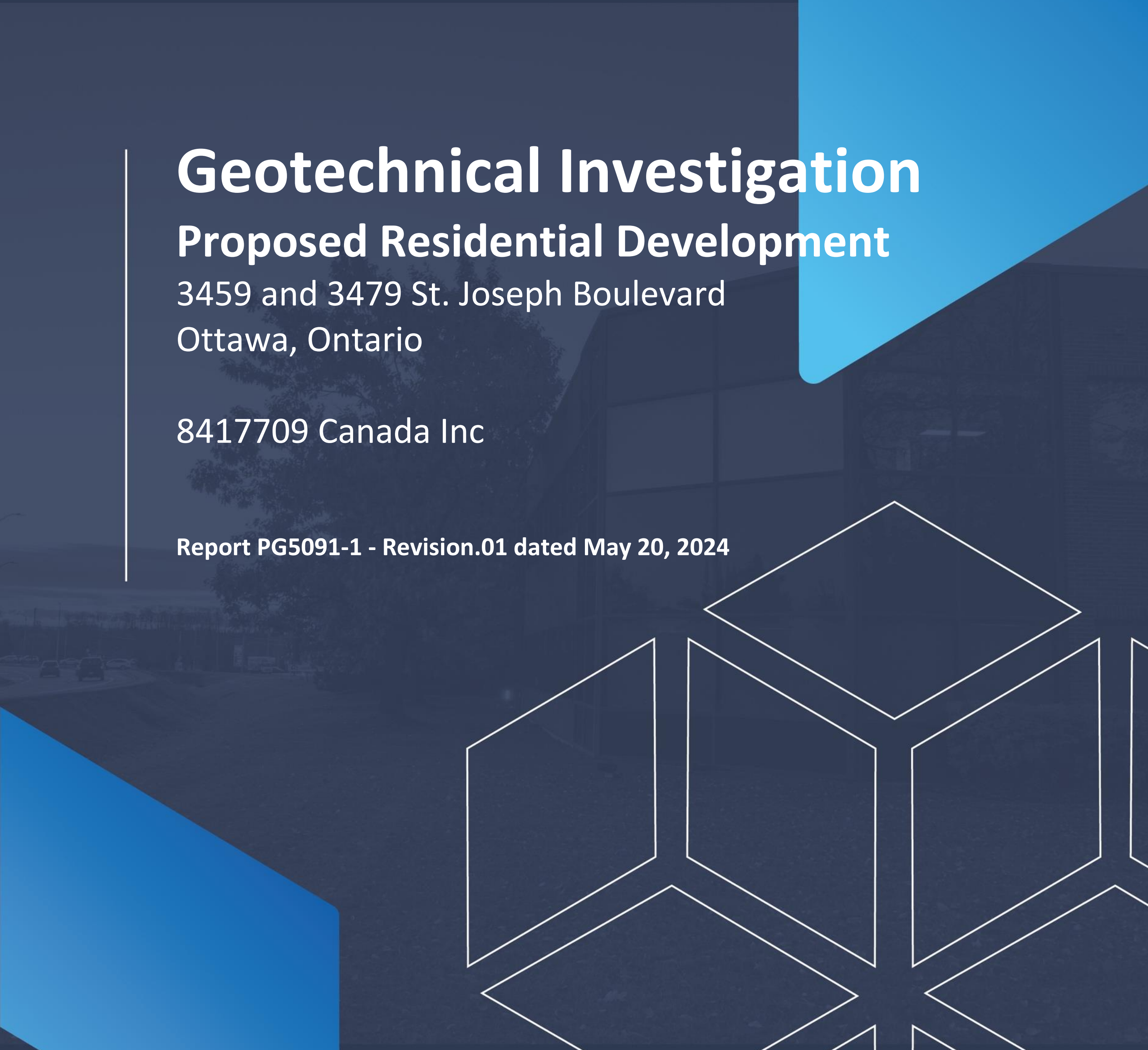

Geotechnical Investigation

Proposed Residential Development

3459 and 3479 St. Joseph Boulevard
Ottawa, Ontario

8417709 Canada Inc

Report PG5091-1 - Revision.01 dated May 20, 2024

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by 8417709 Canada Inc. to conduct a Geotechnical Investigation for the proposed commercial building to be located at 3459 and 3479 St. Joseph Boulevard, in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan presented in Appendix 2).

The objectives of the geotechnical investigation were to:

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

- ❑ Provide geotechnical recommendations for the design of the proposed development including construction considerations which may affect the design.

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

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

2.0 Proposed Development

Based on the most recent conceptual plans, the proposed development will feature four buildings situated within the site's footprint. The northeastern and southwestern buildings will have an L-shaped design, while the two central buildings will be rectangular, and all located over a common underground parking structure. Access lanes and landscaped areas are also part of the anticipated development. It is anticipated that the existing buildings on the site will be demolished to make way for the proposed development.

It is also expected that the site will be municipally serviced by water, storm, and sanitary services.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the geotechnical investigation was carried out on October 8 and 9, 2019. At that time, nine (9) boreholes (BH 1 to BH 9) were advanced to a maximum depth of 6.0 m below existing ground surface. The test holes were located in the field by Paterson in a manner to provide general coverage of the subject site. The borehole locations are shown on Drawing PG5091-1 - Test Hole Location Plan in Appendix 2.

The boreholes were drilled with a track-mounted drill rig operated by a two-person crew. All fieldwork was conducted under the full-time supervision of our personnel under the direction of a senior engineer from our geotechnical department. The drilling procedures consisted of advancing each test hole to the required depths at the selected locations and sampling and testing the overburden.

Sampling and In Situ Testing

Soil samples from the boreholes were recovered from the auger flights or using a 50 mm diameter split-spoon sampler. All soil samples were classified on site, placed in sealed plastic bags and transported to our laboratory for further review. The depths at which the auger and split spoon samples were recovered from the test hole are shown as AU and SS, respectively, on the Soil Profile and Test Data sheets presented in Appendix 1.

Standard Penetration Testing (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 BH 4. 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 test holes were recorded in detail in the field. The soil profiles are presented on the Soil Profile and Test Data sheets in Appendix 1 of this report.

Groundwater

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

Sample Storage

All samples will be stored in the laboratory for a period of one (1) month after issuance of this report. They will then be discarded unless we are otherwise directed.

3.2 Field Survey

The test hole locations were selected in the field by Paterson personnel in a manner to provide general coverage of the subject site taking into consideration existing site features. Ground surface elevations were referenced to a temporary benchmark (TBM), consisting of the top of grate of a catch basin located in front of 3459 St. Joseph Boulevard. A geodetic elevation of 61.30 m was determined for the TBM. The location of the test holes and TBM are presented on Drawing PG5091-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

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

3.4 Analytical Testing

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

4.0 Observations

4.1 Surface Conditions

The subject site currently consists predominantly of vacant land. An existing residential dwelling is located in the southwest corner of the subject site. The site is currently mainly grass covered with occasional brush and small trees and is generally flat and at grade with the adjacent lands to the east and below the grades of the roads to the north, south and west. The site is bordered to the east by an existing residential development, to the south by St. Joseph Boulevard and to the west and north by the on-ramp to Highway 174. The ground surface slopes slightly down from south to north across the subject site.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile encountered at the borehole locations consists of a topsoil layer overlying a brown very stiff to stiff silty clay. A stiff, grey silty clay deposit was encountered below the upper brown silty clay layer. Practical refusal to DCPT was encountered at 23.7 m depth in BH 4. It should be noted that a fill layer was encountered at BH 9 overlying the grey silty clay deposit. Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for details of the soil profiles encountered at each test hole location.

Bedrock

Based on available geological mapping, the local bedrock consists of interbedded limestone and dolomite of the Gull River formation with an anticipated overburden thickness of 15 to 25 m.

4.3 Groundwater

Groundwater levels were measured on November 01, 2019, within the installed monitoring wells and standpipes. The measured groundwater levels noted at that time are presented in Table 1 below.

Table 1 – Summary of Groundwater Levels

Test Hole Number	Ground Surface Elevation (m)	Measured Groundwater Level		Dated Recorded
		Depth (m)	Elevation (m)	
BH 1	57.69	2.75	54.94	November 1, 2019
BH 2	57.19	0.60	56.59	
BH 3	57.12	0.15	56.97	
BH 4	57.92	0.05	56.97	
BH 5	58.02	0.28	57.74	
BH 6	58.26	0.30	57.96	
BH 7	58.45	0.15	58.30	
BH 8	58.37	5.46	52.91	
BH 9	60.82	4.65	56.17	

Note: The ground surface elevation at each borehole location was surveyed using a handheld GPS using a geodetic datum.

It should be noted that groundwater levels can be influenced by surface water infiltrating the backfilled boreholes. Long-term groundwater levels can also be estimated based on the observed color and consistency of the recovered soil samples. Based on these observations, it is estimated that the long-term groundwater table can be expected at approximately 2 to 3 m below ground surface. It should also be noted that groundwater levels are subject to seasonal fluctuations, and therefore the groundwater level could vary at the time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

The subject site is considered suitable for the proposed development from a geotechnical perspective. Based on the available information at the time of preparation of this report, it is anticipated that the proposed buildings can be constructed using conventional shallow footings placed on an undisturbed, stiff to very stiff silty clay bearing surface. If bearing resistance values for conventional footings are exceeded (e.g. for apartment buildings), a raft foundation can be considered.

Due to the presence of a silty clay deposit, a permissible grade raise restriction will be required for the subject site.

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

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and fill, containing deleterious materials, should be stripped from under any buildings and other settlement sensitive structures. Care should be taken not to disturb adequate bearing soils below the subgrade level during site preparation activities.

Protection of Subgrade (Raft Foundation)

Where a raft foundation is used, the raft subgrade would consist of a silty clay deposit, and it is recommended that a minimum 75 mm thick lean concrete mud slab be placed on the undisturbed glacial till subgrade shortly after the completion of the excavation. The main purpose of the mud slab is to reduce the risk of disturbance of the subgrade under the traffic of workers and equipment.

The final excavation to the raft bearing surface level and the placing of the mud slab should be done in smaller sections to avoid exposing large areas of the silty clay deposit to potential disturbance due to drying.

Fill Placement

Fill placed for grading beneath the proposed buildings should consist, unless otherwise specified, of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. The imported fill material should be tested and approved prior to delivery. The fill should be placed in maximum 300 mm thick loose lifts and compacted by suitable compaction equipment. Fill placed beneath the building should be compacted to a minimum of 98% of the standard Proctor maximum dry density (SPMDD).

Non-specified existing fill along with site-excavated soil could be placed as general landscaping fill and beneath exterior parking where settlement of the ground surface is of minor concern. These materials should be spread in lifts with a maximum thickness of 300 mm and compacted by the tracks of the spreading equipment to minimize voids. 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 placement as backfill against foundation walls, unless used in conjunction with a geocomposite drainage membrane, such as Miradrain G100N or Delta Drain 6000, connected to a perimeter drainage system.

5.3 Foundation Design

Bearing Resistance Values

Strip footings, up to 2 m wide, and pad footings, up to 4 m wide, placed on an undisturbed, stiff grey silty clay bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **150 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **225 kPa** incorporating a geotechnical resistance factor of 0.5 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 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.

Raft Foundation

Consideration can be given to a raft foundation, if the building loads exceed the bearing resistance values provided for a conventional shallow footing foundation. The following parameters may be used for raft design. Based on the following assumptions for the raft foundation, the proposed building can be designed with total and differential settlements of 25 and 15 mm, respectively.

For design purposes, it was assumed that the base of the raft foundation for the proposed multi-storey buildings will be located at a 3 to 4 m depth for one underground parking level.

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 **225 kPa** will be considered acceptable. 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 required for the proposed building. The factored bearing resistance (contact pressure) at ULS can be taken as **350 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 **8.5 MPa/m** for a contact pressure of **225 kPa**. The design of the raft foundation is required to consider the relative stiffness of the reinforced concrete slab and the supporting bearing medium.

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 stiff silty clay or engineered fill above the groundwater table when a plane extending horizontally and vertically from the underside of the footing at a minimum of 1.5H:1V passing through in situ soil of the same or higher bearing capacity as the bearing medium soil.

Permissible Grade Raise

Based on the undrained shear strength testing results and experience with the local silty clay deposit, a permissible grade raise restriction for the subject site of **2.5 m** can be used for design purposes.

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class E** for foundations considered at this site. A higher site class, such as Class D, may be possible. However, this would need to be confirmed by site-specific shear wave velocity testing. The soil underlying the subject site are not susceptible to liquefaction. Reference should be made to the latest revision of the Ontario Building Code (OBC) 2012 for a full discussion of the earthquake design requirements.

5.5 Basement Slab

With the removal of all topsoil and deleterious fill, containing organic matter, within the footprint of the proposed buildings, the native soil surface will be considered to be an acceptable subgrade surface on which to commence backfilling for floor slab construction.

It is recommended that a minimum of 75 mm thick lean concrete (15 MPa strength) mud slab be placed to protect the silty clay bearing surface from disturbance due to construction and worker traffic. The bearing surface should be inspected and approved by Paterson personnel prior to placement of the mud slab.

Any soft areas should be removed and backfilled with appropriate backfill material. OPSS Granular B Type II is recommended for backfilling below the basement slab. All backfill materials within the footprint of the proposed building should be placed in a maximum of 300 mm loose lifts and compact to at least 98% of the material's SPMDD.

It is expected that the basement area for the proposed multi-storey buildings will be mostly parking, and the recommended pavement structure noted in Subsection 5.7 will be applicable. However, if storage or other uses of the lower level are proposed where a concrete floor slab will be used, it is recommended that the upper 200 mm of sub-slab fill consist of 19 mm clear crushed stone.

5.6 Basement Wall

There are several combinations of backfill materials and retained soils that could be applicable for the basement walls of the subject structure. However, the conditions can be well-represented by assuming the retained soil consists of a material with an angle of internal friction of 30 degrees and a bulk (drained) unit weight of 20 kN/m^3. 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 calculating the effective unit weight.

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

Lateral Earth Pressures

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

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

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

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

Seismic Earth Pressures

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

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

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

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

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

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

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

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

5.8 Pavement Design

Rigid Pavement Structure

For design purposes, it is recommended that the rigid pavement structure for the lowest level of the underground parking structure should 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 on the following page.

Table 2 - Recommended Rigid Pavement Structure - Lower Parking Level

Thickness (mm)	Material Description
125	**Exposure Class C2 – 32 MPa Concrete** (5 to 8 % Air Entrainment)
300	**BASE** - OPSS Granular A Crushed Stone
*A Type 3 geotextile membrane is required for the construction of a road pavement on a clay layer.	
SUBGRADE – Imported fill or OPSS Granular B Type I or II or material placed over in situ soil.	

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 hours after the concrete has been poured during warm temperatures and up to 12 hours during cooler temperatures.

Flexible Pavement Structure

The flexible pavement structure presented in Table 3 and Table 4 should be used for driveways and car only parking areas and at grade access lanes and heavy loading parking areas.

Table 3 - Recommended Flexible 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 - OPSS Granular B Type I or II placed over in-situ soil, or concrete fill.	

Table 4 - Recommended Flexible Pavement Structure - Access Lanes and Heavy Loading Parking Areas

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

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for this project. The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 100% of the material's SPMDD using suitable vibratory equipment, noting that excessive compaction can result in subgrade softening.

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 I or II material.

Pavement Structure Drainage

Satisfactory performance of the pavement structure is largely dependent on keeping the contact zone between the subgrade material and the base stone in a dry condition. Failure to provide adequate drainage under conditions of heavy wheel loading can result in the fine subgrade soil being pumped into the voids in the stone subbase, thereby reducing its load carrying capacity.

Due to the impervious nature of the subgrade materials consideration should be given to installing subdrains during the pavement construction. These drains should extend in four orthogonal directions or longitudinally when placed along a curb. The clear crushed stone surrounding the drainage lines or the pipe, should be wrapped with suitable filter cloth. The subdrain inverts should be approximately 300 mm below subgrade level. The subgrade surface should be shaped to promote water flow to the drainage lines. All subdrains should be provided with a positive outlet to the storm sewer.

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 structures.

- ❑ Where foundation walls will be double-sided poured, a composite drainage membrane (DeltaDrain 6000, MiraDrain G100N or equivalent) is recommended to be installed directly onto the exterior foundation wall in combination with a damp proofing membrane between the top of the footing and finished grade.

- ❑ The foundation drainage boards should be overlapped such that the bottom end of a higher board is placed in front of the top end of a lower board. All endlaps of the drainage board sheets should overlap abutting sheets by a minimum of 150 mm. All overlaps should be sealed with a suitable adhesive and/or sealant material approved by Paterson.

Waterproofing layers for podium deck surfaces should overlap across and below the top end lap of the vertically installed composite foundation drainage board to mitigate the potential for water to migrate between the drainage board and foundation wall. Elevator shafts located below the underslab drainage system should be waterproofed and provided with a PVC waterstop at the shaft wall and footing interface.

Review of architectural design drawings should be completed by Paterson for the above-noted items once the building design has been finalized and prior to tender. It is recommended that Paterson reviews all details associated with the foundation drainage system prior to tender.

Interior Perimeter and Underfloor Drainage

The interior perimeter and underfloor drainage system will be required to control water infiltration below the lowest underground parking level slab and redirect water from the buildings foundation drainage system to the buildings sump pit(s). The interior perimeter and underfloor drainage pipe should consist of a 150 mm diameter corrugated perforated plastic pipe sleeved with a geosock.

The underfloor drainage pipe should be placed in each direction of the basement floor span and connected to the perimeter drainage pipe. The interior drainage pipe should be provided tee-connections to extend pipes between the perimeter drainage line and the HDPE-face of the composite foundation drainage board via the foundation wall sleeves. The spacing of the underfloor drainage system should be confirmed by Paterson once the foundation layout and sump system location has been finalized.

Foundation Backfill

Where space is available, backfill against the exterior sides of the foundation walls should consist of free-draining, non-frost susceptible granular materials. The greater part of the site excavated materials will be frost susceptible and, as such, are not recommended for re-use as backfill against the foundation walls unless used in conjunction with a composite drainage system, such as Delta Drain 6000 or an approved equivalent. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should otherwise be used for this purpose.

Sidewalks and Walkways

Backfill material below sidewalk and walkway subgrade areas or other settlement sensitive structures which are not adjacent to the buildings should consist of free-draining, non-frost susceptible material. This material should be placed in maximum 300 mm thick loose lifts and compacted to at least 98% of its SPMDD under dry and above freezing conditions.

6.2 Protection of Footings Against Frost Action

Perimeter foundations of heated structures are required to be insulated against the deleterious effects of frost action. A minimum of 1.5 m of soil cover should be provided for adequate frost protection for heated structures.

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

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 the excavation should either be cut back at acceptable slopes or be retained by shoring systems from the beginning of the excavation until the structure is backfilled. However, for most of the site, insufficient room will be available to permit the building excavation to be constructed by open-cut methods (i.e., unsupported excavations).

Unsupported Excavations

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

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

It is recommended that a trench box be used at all times to protect personnel working in trenches with steep or vertical sides. It is expected that services will be installed by “cut and cover” methods and excavations will not be left open for extended periods of time.

Temporary Shoring

Due to the anticipated proximity of the proposed development to the property boundaries, temporary shoring may be required to support the overburden soils. The shoring requirements will depend on the depth of the excavation and the proximity of the adjacent structures. However, it should be noted that the observed bouldery conditions can lead to the creation of voids and other unstable conditions during installation of the temporary shoring as boulders shift within the fine soil matrix. Furthermore, it may be difficult to develop the required anchor strength in soil due to variations in soil conditions.

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 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 representative prior to implementation.

The temporary shoring system may consist of a soldier pile and lagging system. 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 below.

The earth pressures acting on the temporary shoring system may be calculated using the parameters outlined in Table 5 on the next page.

Table 5 - Soil Parameters for Calculating Earth Pressures Acting on Shoring System

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	20
Submerged Unit Weight(γ'), kN/m^3	13

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

The hydrostatic groundwater pressure should be added to the earth pressure distribution wherever the effective unit weights are used for earth pressure calculations. If the groundwater level is lowered, the dry unit weight for the soil should be used full weight, with no hydrostatic groundwater pressure component. For design purposes, the minimum factor of safety of 1.5 should be calculated.

6.4 Pipe Bedding and Backfill

Bedding and backfill materials should be in accordance with the most recent Material Specifications and Standard Detail Drawings from the Department of Public Works and Services, Infrastructure Services Branch of the City of Ottawa. At least 150 mm of OPSS Granular A should be used for pipe bedding for sewer and water pipes. The bedding should extend to the spring line of the pipe.

Cover material, from the spring line to at least 300 mm above the obvert of the pipe, should consist of OPSS Granular A or Granular B Type II with a maximum size of 25 mm. The bedding and cover materials should be placed in maximum 225 mm thick lifts compacted to 99% of the material’s SPMDD.

It should generally be possible to re-use the moist (not wet) brown silty clay above the cover material if the excavation and filling operations are carried out in dry weather conditions. Wet silty clay materials will be difficult to re-use, as the high-water contents make compacting impractical without an extensive drying period.

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

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 dry 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 low and controllable using open sumps. Pumping from open sumps should be sufficient to control the groundwater influx through the sides of the excavation, provided that a series of drainage lines are placed at subgrade level to direct water flow toward the sump pumps.

The contractor should be prepared to direct water away from all bearing surfaces and subgrades, regardless of the source, to prevent disturbance to the founding medium.

Permit to Take Water

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

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

Long-term groundwater Control

Our recommendations for the proposed building’s long-term groundwater control are presented in Subsection 6.1. Any groundwater which encounters the building’s perimeter groundwater infiltration control system will be directed to the proposed building’s sump pit. Due to the size of the parking garage multiple sump pits should be proposed. It is expected that groundwater flow will be low (i.e. less than 100,000 L/day with peak periods noted after rain events. It is anticipated that the groundwater flow will be controllable using conventional open sumps.

Impacts on Neighbouring Properties

It is understood that one level of underground parking is planned for the proposed building. Based on the existing groundwater level and low permeability of the adjacent soils, the extent of any significant groundwater lowering will take place within a limited range of the proposed building. 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 building.

6.6 Winter Construction

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

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

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

Precaution must be taken where excavations are carried out in proximity of existing structures which may be adversely affected due to the freezing conditions. In particular, it should be recognized that where a shoring system is used, the soil behind the shoring system will be subjected to freezing conditions and could result in heaving of the structure(s) placed within or above frozen soil.
Provisions should be made in the contract document to protect the walls of the excavations from freezing, if applicable.

6.7 Corrosion Potential and Sulphate

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

6.8 Landscaping Considerations

Tree Planting Considerations

Due to the silty clay deposit encountered across the subject site, and in accordance with the City of Ottawa's "Tree Planting in Sensitive Marine Clay Soils – 2017 Guidelines", Paterson has recommended the following tree planting setbacks:

Large trees (mature height over 14 m) can be planted within this area provided a tree to foundation setback equal to the full mature height of the tree can be provided (e.g. in a park or other green space). A tree planting setback limit of **7.5 m** is applicable for small (mature tree height up to 7.5m) and medium size trees (mature tree height 7.5 m to 14 m) provided that the following conditions are met:

☐ The underside of footing (USF) is 2.1 m or greater below the lowest finished grade must be satisfied for footings within 10 m from the tree, as measured from the center of the tree trunk and verified by means of the Grading Plan.

☐ A small tree must be provided with a minimum of 25 m^3 of available soil volume while a medium tree must be provided with a minimum of 30 m^3 of available soil volume, as determined by the Landscape Architect. The developer is to ensure that the soil is generally un-compacted when backfilling in street tree planting locations.

☐ The tree species must be small (mature tree height up to 7.5 m) to medium size (mature tree height 7.5 m to 14 m) as confirmed by the Landscape Architect.

☐ The foundation walls are to be reinforced at least nominally (minimum of two upper and two lower 15M bars in the foundation wall).

☐ Grading surrounding the tree must promote drainage to the tree root zone (in such a manner as not to be detrimental to the tree), as noted on the subdivision Grading Plan.

It is well documented in the literature, and is our experience, that fast-growing trees located near buildings founded on cohesive soils that shrink on drying can result in long-term differential settlements of the structures. Tree varieties that have the most pronounced effect on foundations are noted to consist of poplars, willows and some maples (i.e. Manitoba Maples) and, as such, should not be considered in the landscaping design.

Set backs ould not be applicable for tress planted on top of the common underground garage structure.

7.0 Recommendations

It is recommended that the following be completed once the master plan and site development are determined.

- Review of the geotechnical aspects of the excavation contractor's shoring design, if required, prior to construction.
- Review of waterproofing details for the elevator shaft and building sump pits.
- Review and inspection of the foundation waterproofing system and all foundation drainage systems.
- Observation of all bearing surfaces prior to the placement of concrete.
- Sampling and testing of the concrete and fill materials.
- Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- Complete a full inspection program of the installation of the perimeter and underground floor drainage system during construction.
- Observation of all subgrades prior to backfilling.
- Field density tests to determine the level of compaction achieved.
- Sampling and testing of the bituminous concrete including mix design reviews.

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

8.0 Statement of Limitations

The recommendations provided herein 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 8417709 Canada 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.

LICENSED PROFESSIONAL ENGINEER
May 20, 2024
J. R. VILLENEUVE
100504344
PROVINCE OF ONTARIO

Fabrice Venadiambu, P.Eng

Joey R. Villeneuve, M.A.Sc., P.Eng, ing.

Report Distribution:

☐ 8417709 Canada inc. (e-mail copy)
☐ Paterson Group Inc (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Development - 3459 & 3479 St. Joseph Blvd.
Ottawa, Ontario

DATUM Geodetic

REMARKS

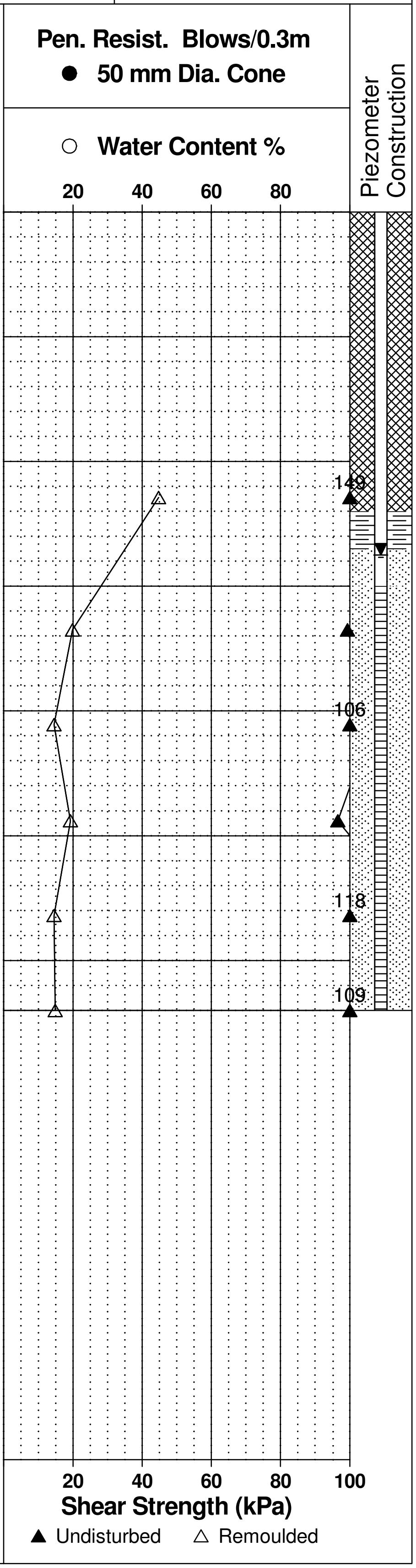
BORINGS BY CME 75 Power Auger

DATE 2019 October 8

FILE NO. PG5091

HOLE NO. BH 1

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	57.69
TOPSOIL 0.35		AU	1				
Very stiff to stiff, brown **SILTY CLAY**		SS	2	79	13	1	56.69
- Grey by 2.3m depth		SS	3	100	6	2	55.69
						3	54.69
						4	53.69
						5	52.69
						6	51.69
6.40 End of Borehole							
(GWL @ 2.75 m depth - Nov. 1/19)							



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Ottawa, Ontario

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REMARKS

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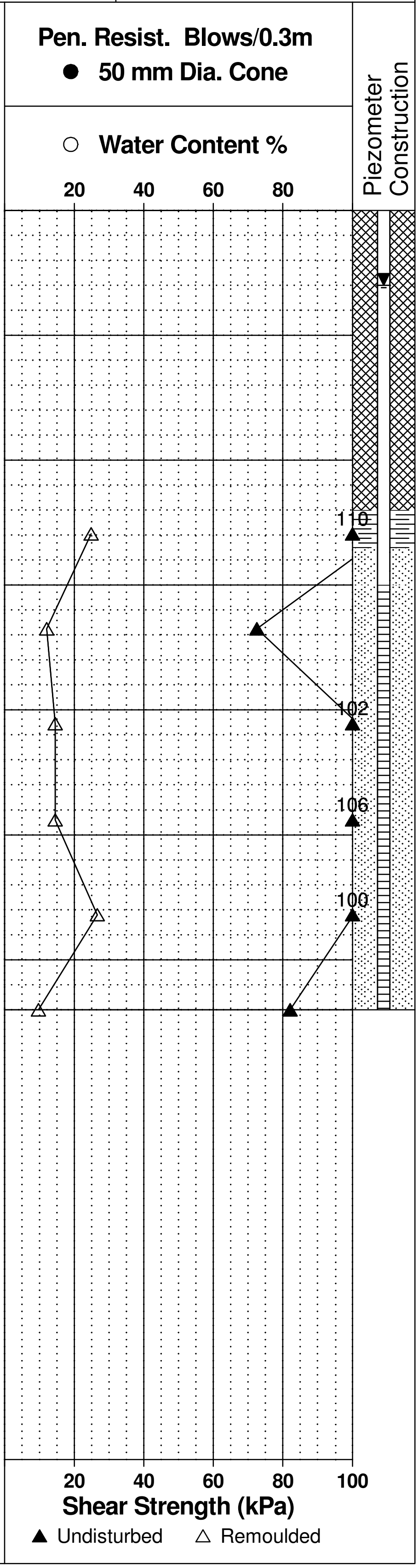
DATE 2019 October 9

FILE NO. PG5091

HOLE NO. BH 2

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	57.19
TOPSOIL 0.05	SS	1	75	5		
Very stiff to stiff, brown **SILTY CLAY**	SS	2	100	10	1	56.19
- Grey by 1.8m depth	SS	3	100	6	2	55.19
					3	54.19
					4	53.19
					5	52.19
					6	51.19
6.40						
End of Borehole						
(GWL @ 0.6m depth - Nov 1/19)						



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REMARKS

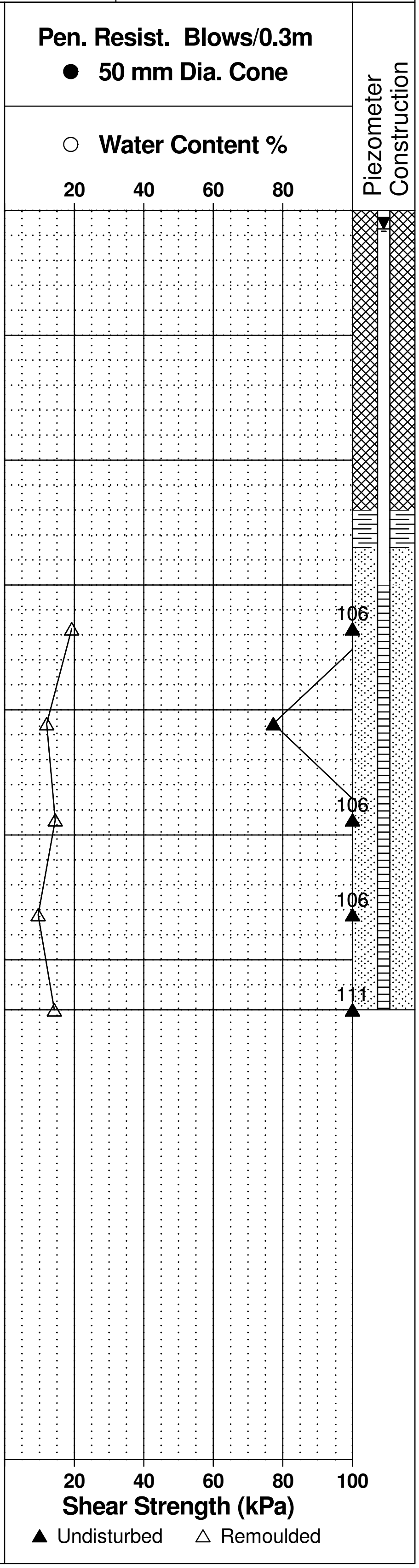
BORINGS BY CME 75 Power Auger

DATE 2019 October 8

FILE NO. PG5091

HOLE NO. BH 3

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	57.12
TOPSOIL 0.35		AU	1				
Very stiff to stiff, brown **SILTY CLAY**		SS	2	100	8	1	56.12
- Grey by 3.0m depth		SS	3	100	6	2	55.12
		SS	4	100	4	3	54.12
						4	53.12
						5	52.12
						6	51.12
6.40							
End of Borehole (GWL @ 0.15m depth - Nov 1/19)							



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Ottawa, Ontario

DATUM Geodetic

REMARKS

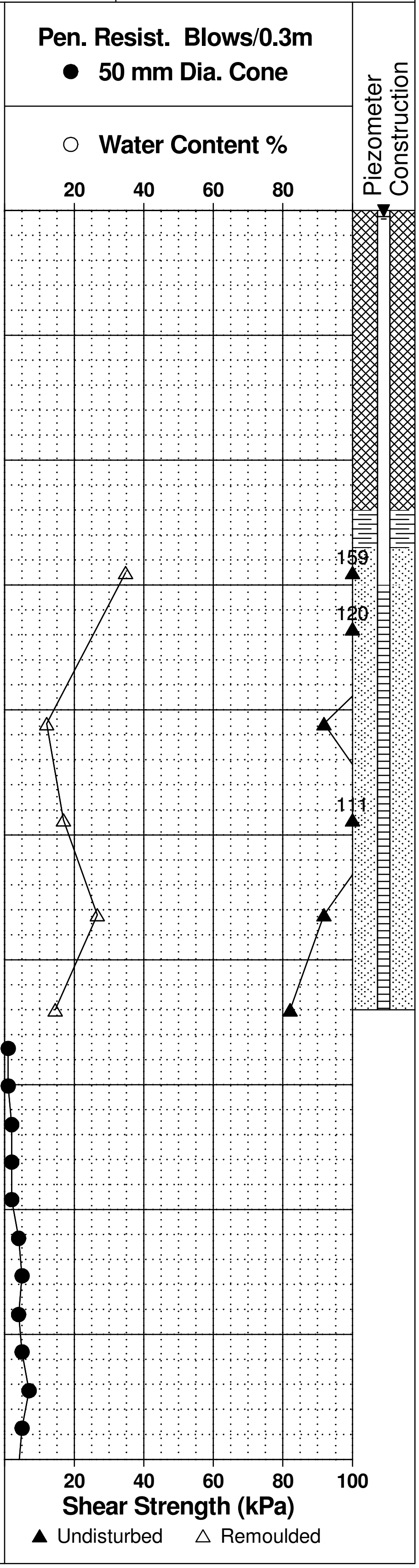
BORINGS BY CME 75 Power Auger

DATE 2019 October 9

FILE NO. PG5091

HOLE NO. BH 4

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	57.92
TOPSOIL — 0.35	SS		75	5		
Very stiff to stiff, brown **SILTY CLAY**	SS		100	12	1	56.92
- Grey by 3.0m depth	SS		100	8	2	55.92
					3	54.92
	SS		100	P		
					4	53.92
					5	52.92
					6	51.92
— 6.40						
Dynamic Cone Penetration Test (DCPT) commenced at 6.4m depth						
					7	50.92
					8	49.92
					9	48.92
					10	47.92



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REMARKS

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DATE 2019 October 9

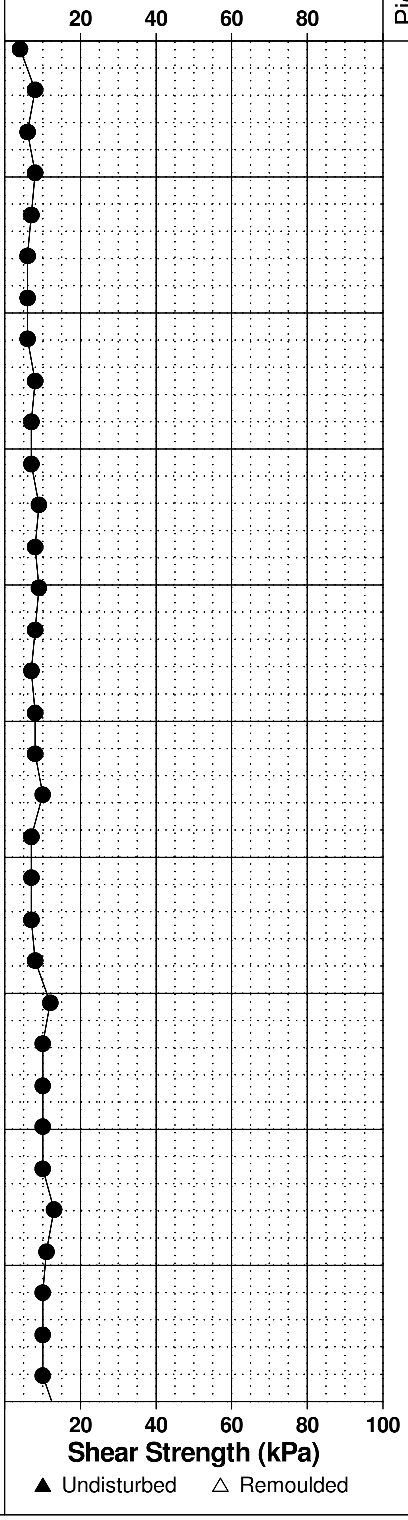
FILE NO. PG5091

HOLE NO. BH 4

SOIL DESCRIPTION GROUND SURFACE	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Piezometer Construction
						10	47.92	
						11	46.92	
						12	45.92	
						13	44.92	
						14	43.92	
						15	42.92	
						16	41.92	
						17	40.92	
						18	39.92	
						19	38.92	
						20	37.92	

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

○ Water Content %



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FILE NO. PG5091

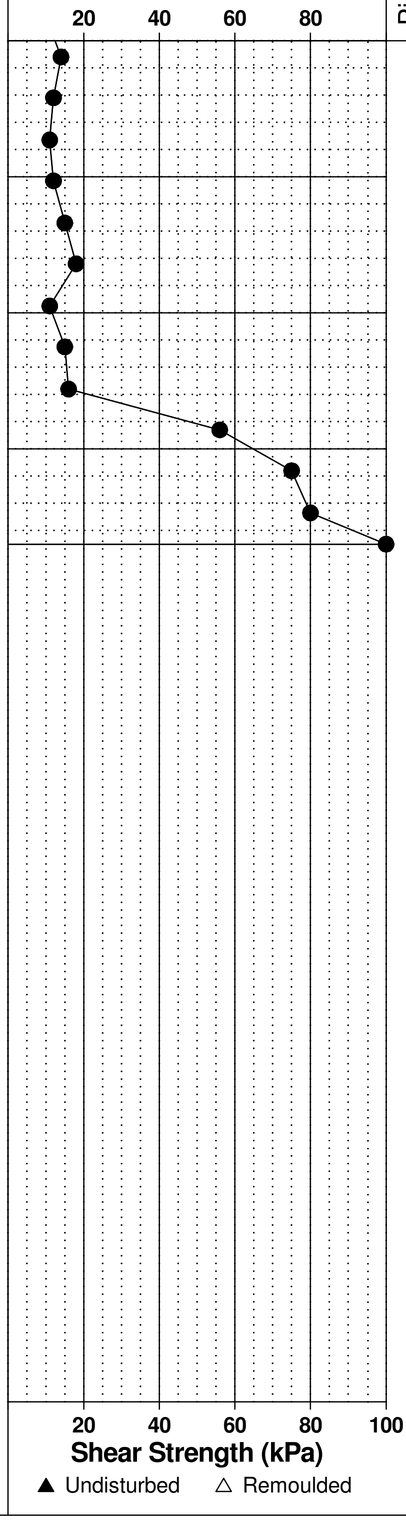
HOLE NO. BH 4

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE							
						20	37.92
						21	36.92
						22	35.92
						23	34.92
23.70							
End of Borehole Practical refusal to DCPT @ 23.7m depth (GWL @ 0.05m depth - Nov 1/19)							

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

○ Water Content %

Piezometer Construction



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REMARKS

BORINGS BY CME 75 Power Auger

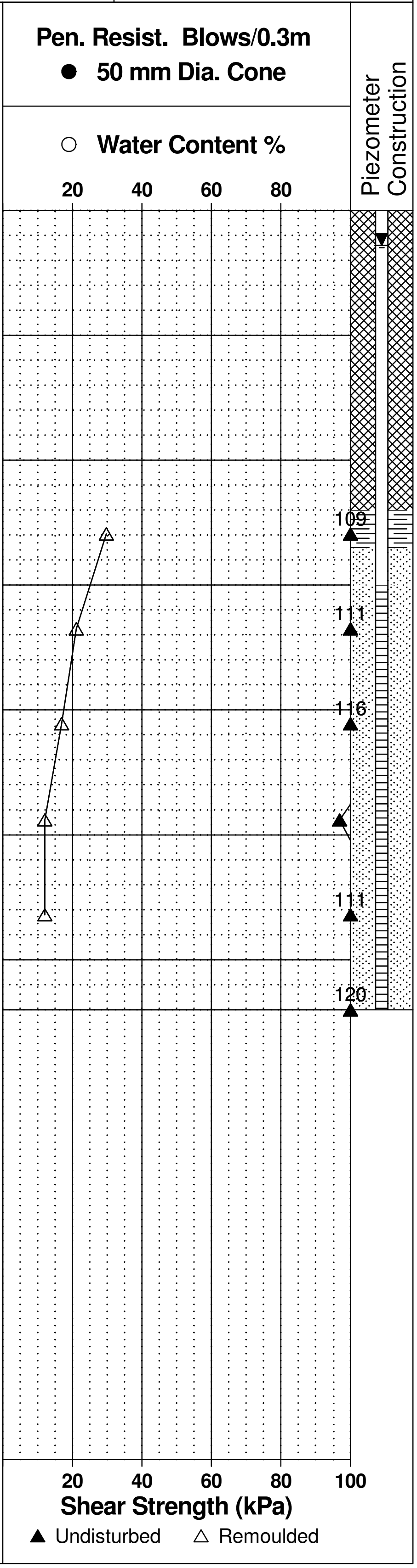
DATE 2019 October 8

FILE NO. PG5091

HOLE NO. BH 5

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD
GROUND SURFACE					
TOPSOIL 0.30		AU	1		
Very stiff to stiff, brown **SILTY CLAY**		SS	2	100	10
- Grey by 3.8m depth		SS	3	100	5
6.40					
End of Borehole					
(GWL @ 0.28m depth - Nov 1/19)					

DEPTH (m)	ELEV. (m)
0	58.02
1	57.02
2	56.02
3	55.02
4	54.02
5	53.02
6	52.02



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REMARKS

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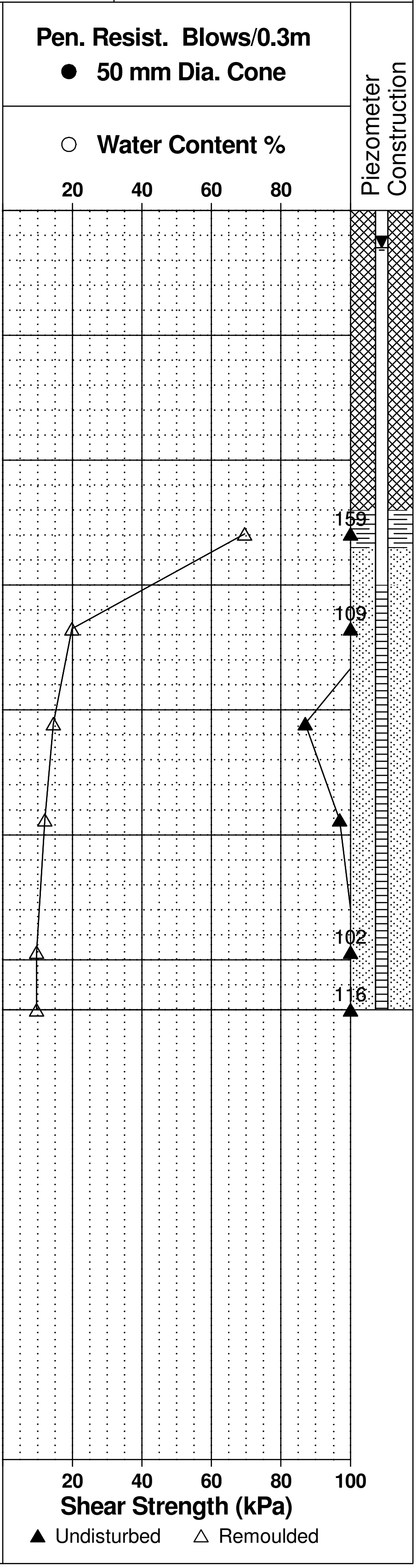
DATE 2019 October 9

FILE NO. PG5091

HOLE NO. BH 6

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	58.26
TOPSOIL 0.15	AU	1	50			
Very stiff to stiff, brown **SILTY CLAY**	SS	2	100	6	1	57.26
- Grey by 3.0m depth	SS	3	100	9	2	56.26
					3	55.26
					4	54.26
					5	53.26
					6	52.26
6.40						
End of Borehole						
(GWL @ 0.3m depth - Nov 1/19)						



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REMARKS

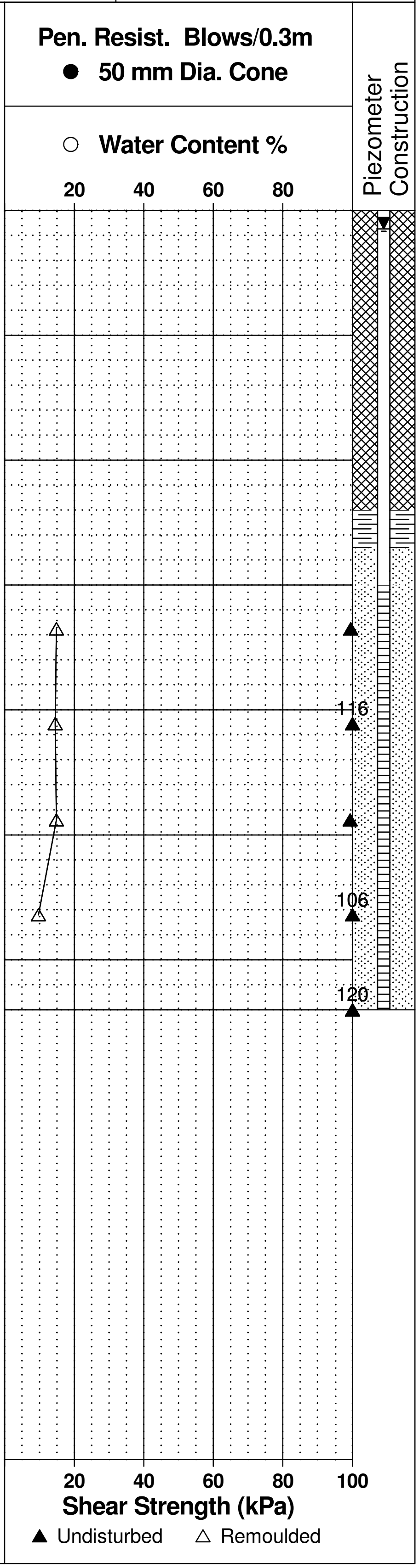
BORINGS BY CME 75 Power Auger

DATE 2019 October 8

FILE NO. PG5091

HOLE NO. BH 7

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	58.45
TOPSOIL 0.30	AU	1				
Very stiff to stiff, brown **SILTY CLAY**	SS	2	100	10	1	57.45
- Grey by 3.0m depth	SS	3	100	5	2	56.45
	SS	4	100	1	3	55.45
					4	54.45
					5	53.45
6.40					6	52.45
End of Borehole						
(GWL @ 0.15m depth - Nov 1/19)						



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FILE NO. PG5091

REMARKS

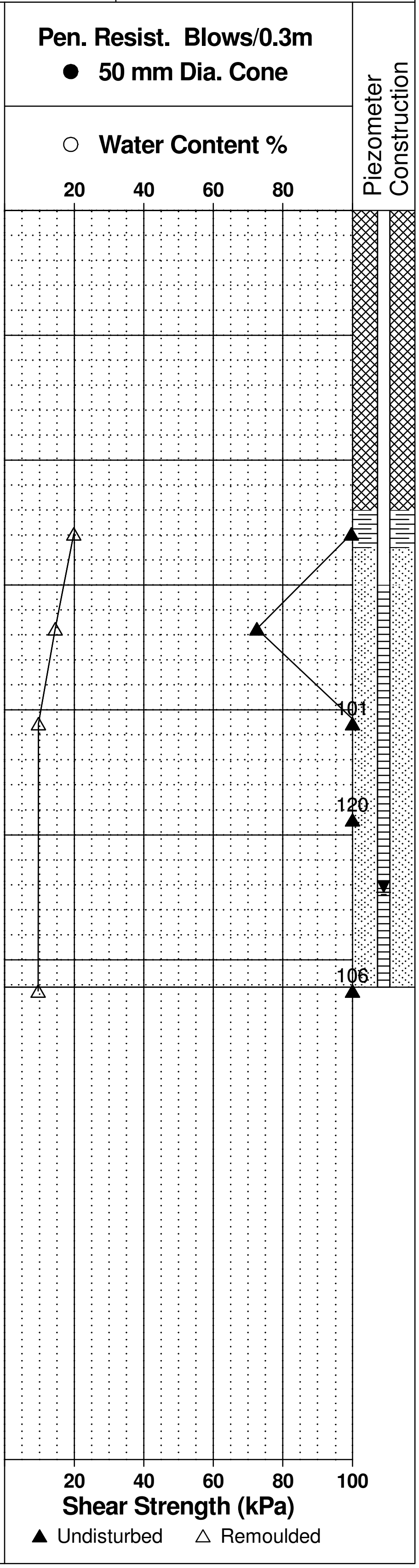
HOLE NO. BH 8

BORINGS BY CME 75 Power Auger

DATE 2019 October 8

SOIL DESCRIPTION	STRATA PLOT	TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
TOPSOIL 0.30		SS	1	75	2
Very stiff to stiff, brown **SILTY CLAY**		SS	2	100	9
- Grey by 2.3m depth		SS	3	100	5
		SS	4	100	1
6.25					
End of Borehole (GWL @ 5.46m depth - Nov 1/19)					

DEPTH (m)	ELEV. (m)
0	58.37
1	57.37
2	56.37
3	55.37
4	54.37
5	53.37
6	52.37



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REMARKS

BORINGS BY CME 75 Power Auger

DATE 2019 October 9

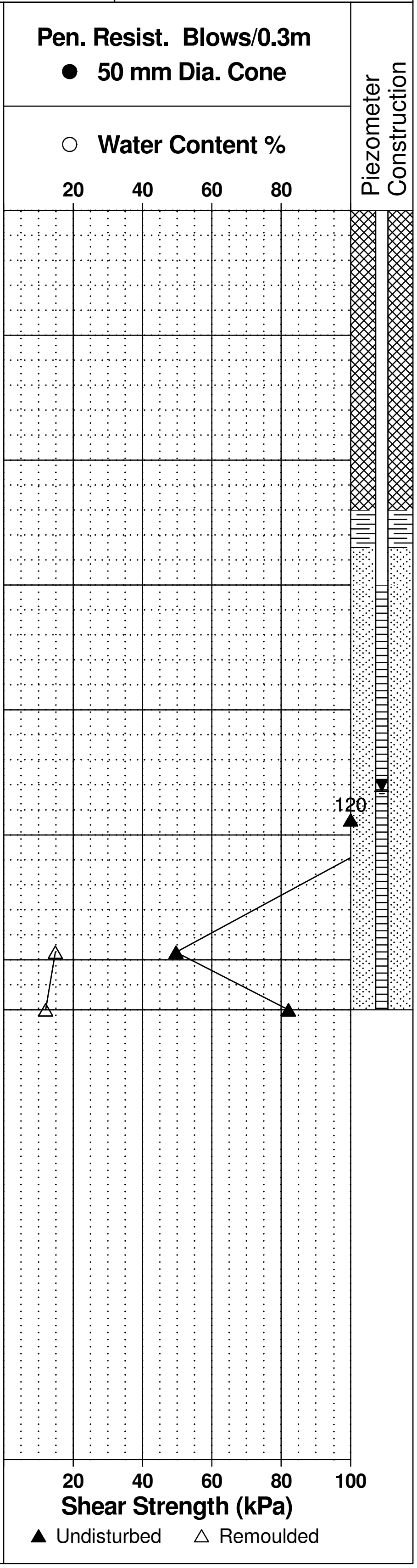
FILE NO. PG5091

HOLE NO. BH 9

SOIL DESCRIPTION	Depth (m)
GROUND SURFACE	
TOPSOIL	0.10
FILL: Loose brown silty sand	1.50
FILL: Very stiff brown silty clay with silty sand, trace gravel	2.60
Stiff, grey **SILTY CLAY**	6.40
End of Borehole (GWL @ 4.65m depth - Nov. 1/19)	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	50	6
SS	3	42	1
SS	4	75	17
SS	5	100	10
SS	6	100	3

DEPTH (m)	ELEV. (m)
0	60.82
1	59.82
2	58.82
3	57.82
4	56.82
5	55.82
6	54.82



SYMBOLS AND TERMS

SOIL DESCRIPTION

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

Desiccated - having visible signs of weathering by oxidation of clay minerals, shrinkage cracks, etc.

Fissured - having cracks, and hence a blocky structure.

Varved - composed of regular alternating layers of silt and clay.

Stratified - composed of alternating layers of different soil types, e.g. silt and sand or silt and clay.

Well-Graded - Having wide range in grain sizes and substantial amounts of all intermediate particle sizes (see Grain Size Distribution).

Uniformly-Graded - Predominantly of one grain size (see Grain Size Distribution).

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

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

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

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

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

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

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

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

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

MC% - Natural moisture content or water content of sample, %
LL - Liquid Limit, % (water content above which soil behaves as a liquid)
PL - Plastic limit, % (water content above which soil behaves plastically)
PI - Plasticity index, % (difference between LL and PL)

Dxx - Grain size which xx% of the soil, by weight, is of finer grain sizes
These grain size descriptions are not used below 0.075 mm grain size
D10 - Grain size at which 10% of the soil is finer (effective grain size)
D60 - Grain size at which 60% of the soil is finer

Cc - Concavity coefficient = $(D30)^2 / (D10 \times D60)$
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)

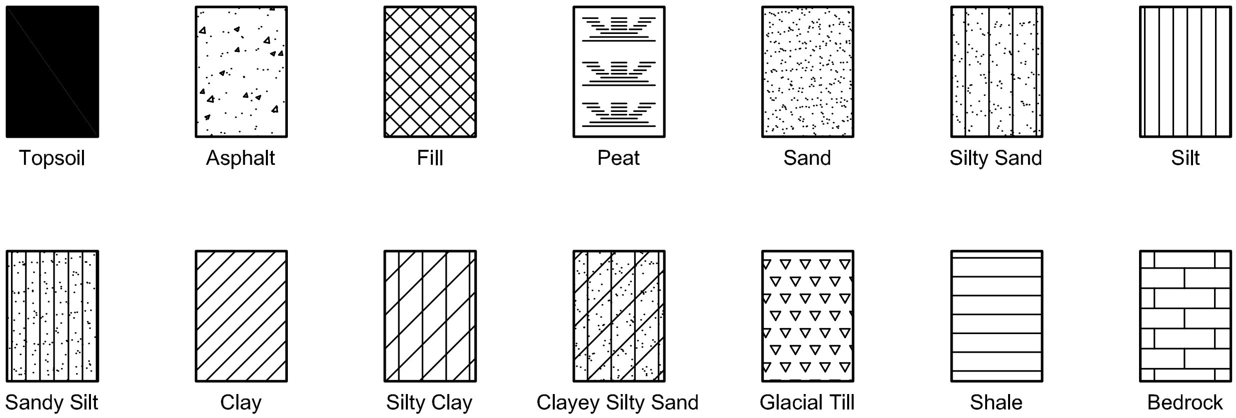
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

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.

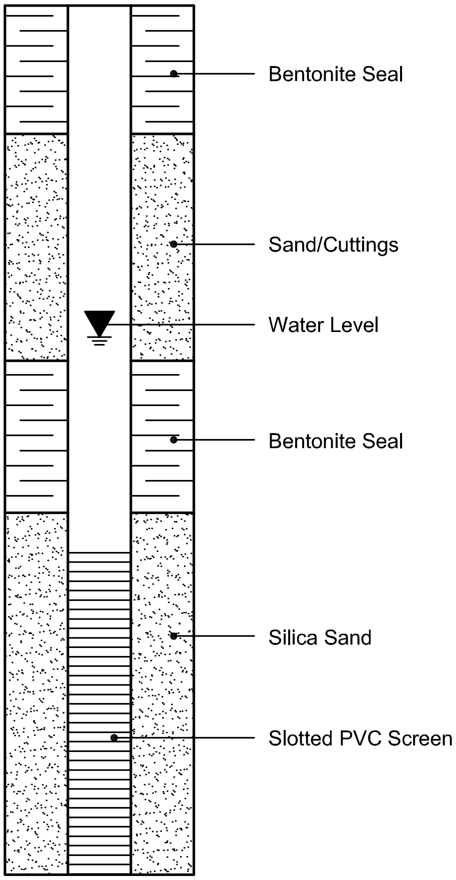
SYMBOLS AND TERMS (continued)

STRATA PLOT

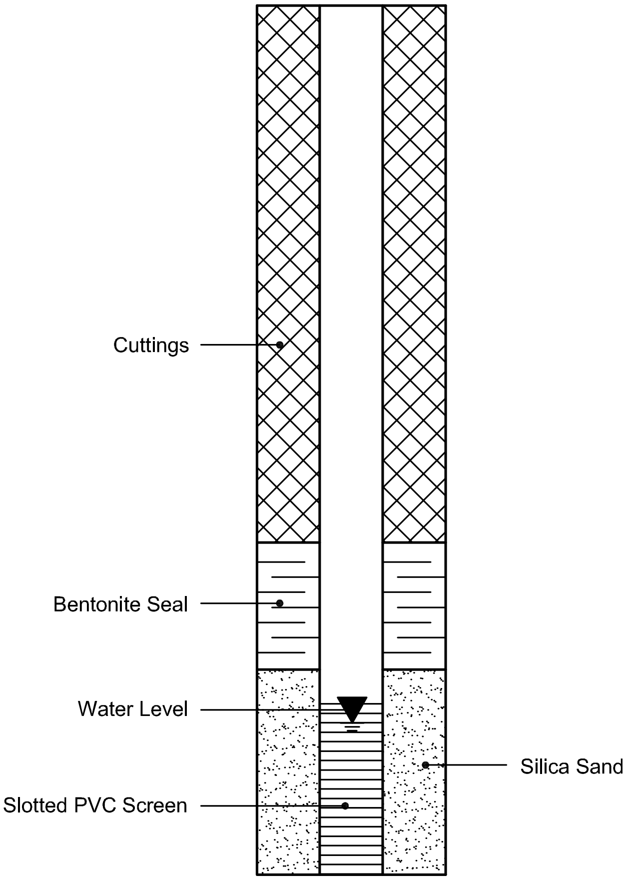


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 25633

Report Date: 16-Oct-2019

Order Date: 10-Oct-2019

Project Description: PG5091

	Client ID:	BH3-SS4 - 7.5' to 9.5'	-	-	-
	Sample Date:	08-Oct-19 14:00	-	-	-
	Sample ID:	1941478-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	68.4	-	-	-
General Inorganics					
pH	0.05 pH Units	7.06	-	-	-
Resistivity	0.10 Ohm.m	41.1	-	-	-
Anions					
Chloride	5 ug/g dry	70	-	-	-
Sulphate	5 ug/g dry	17	-	-	-

APPENDIX 2

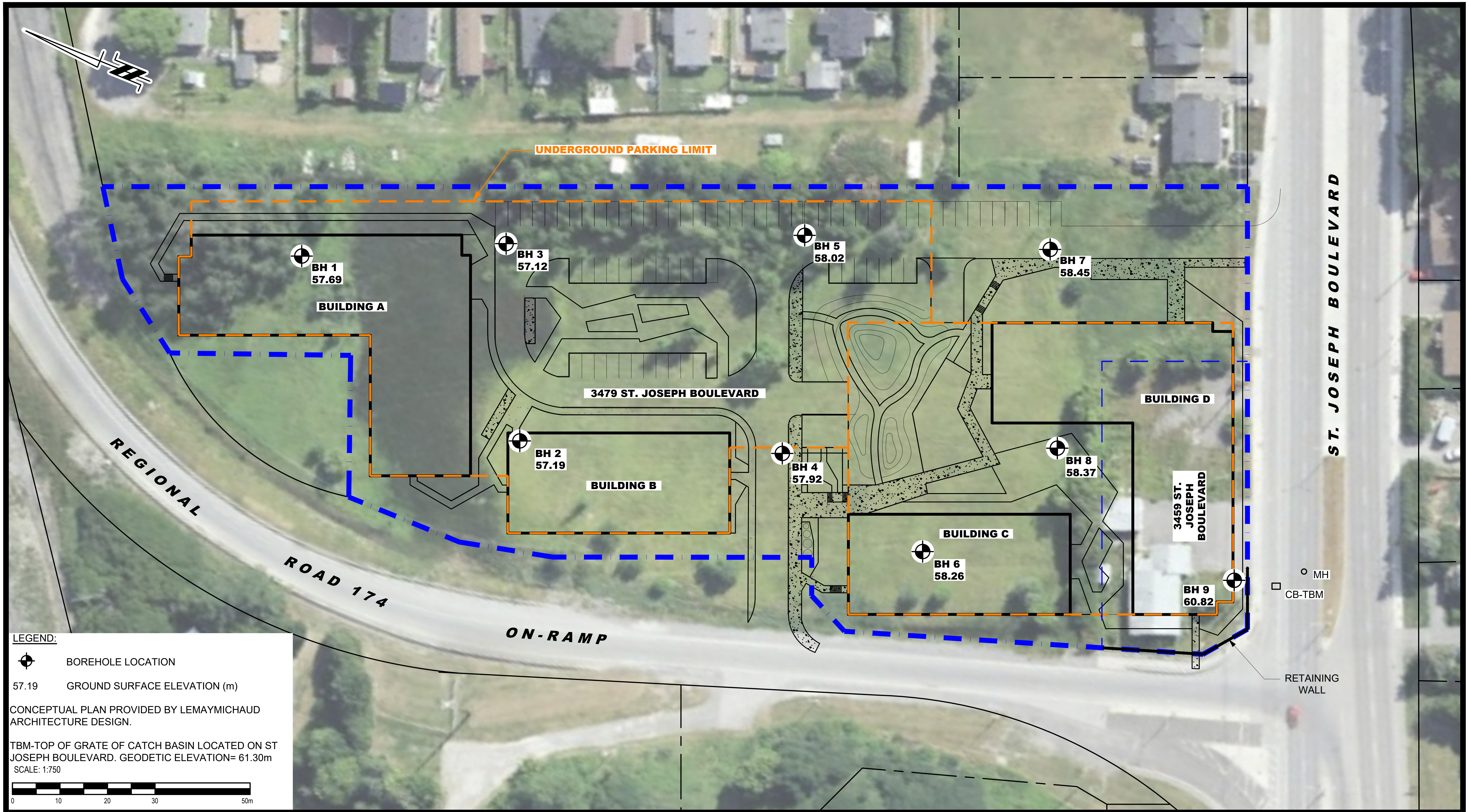
FIGURE 1 – KEY PLAN

DRAWING PG5091-1 - TEST HOLE LOCATION PLAN

FIGURE 1

KEY PLAN

PATERSON GROUP



LEGEND:

⊕	BOREHOLE LOCATION
57.19	GROUND SURFACE ELEVATION (m)

CONCEPTUAL PLAN PROVIDED BY LEMAYMICHAUD ARCHITECTURE DESIGN.

TBM-TOP OF GRATE OF CATCH BASIN LOCATED ON ST JOSEPH BOULEVARD. GEODETIC ELEVATION= 61.30m

SCALE: 1:750

0 10 20 30 50m

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

NO.	REVISIONS	DATE	INITIAL
1	UPDATED CONCEPTUAL PLAN	14/05/2024	FV

8417709 CANADA INC.
GEOTECHNICAL INVESTIGATION
3459 AND 3479 ST. JOSEPH BOULEVARD
PROPOSED DEVELOPMENT

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:750
Date: 11/2019
Drawn by: YA
Report No.: PG5091-1
Checked by: NC
Dwg. No.: PG5091-1
Approved by: DJG
Revision No.: 1

11x17

p:\autocad drawings\geotechnical\pg50xx\pg5091\pg5091-1 test hole location plan (rev.01).dwg