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Geotechnical Investigation

Proposed High Rise Buildings
383 Albert Street
Ottawa, Ontario

Prepared For

Claridge Homes

July 18, 2018

Report: PG4517-1

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Claridge Homes to conduct a geotechnical investigation for a proposed high rise buildings located at 383 Albert Street in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objectives of the current investigation were to:

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

- ❑ 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 the presence or potential presence of contamination on the subject property was not part of the scope of work for this geotechnical investigation. An environmental program was carried out and the information is presented under separate cover.

2.0 Proposed Development

Based on current plans, it is understood that the proposed development will consist of 3 high rise mixed-use buildings. Two of the buildings (Tower A and B) will consist of 28 storeys while the third (Tower C) will consist of 22 storeys. All the proposed buildings will share 8 levels of underground parking which will occupy the majority of the site. It is also understood that the proposed building will be municipally serviced.

The proposed development will border the existing LRT tunnel along Queen Street and a separate proximity study will be required. Once the final details of the building are available, a detailed review will be presented in a separate document.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the investigation was carried out on July 3 to 6, 2018. At that time, 6 boreholes were advanced to a maximum depth of 4.8 m below existing grade across the subject site to provided general coverage of the proposed development. A previous investigation was completed in December 2013 by others. The locations of the test holes are shown on Drawing PG4517-1 - 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. The test hole procedure consisted of augering to the required depths at the selected locations and sampling the overburden soils. All fieldwork was conducted under the full-time supervision of personnel from Paterson's geotechnical division under the direction of a senior engineer.

Sampling and In Situ Testing

Soil samples were collected from the boreholes using a 50 mm diameter split-spoon (SS) sampler, or drill cuttings from the auger flights. Soil samples from the test pits were recovered from the side walls of the open excavation and all soil samples were initially classified on site. The split-spoon, auger samples and grab samples were placed in sealed plastic bags. All samples were transported to our laboratory for further examination and classification. The depths at which the split-spoon and auger samples were recovered from the test holes are shown as SS and AU, 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.

Diamond drilling was carried out at three borehole locations to assess the bedrock quality. A recovery value and a Rock Quality Designation (RQD) value were calculated for each drilled section of bedrock and are shown on the Soil Profile and Test Data sheets in Appendix 1. The recovery value is the ratio, in percentage, of the length of the bedrock sample recovered over the length of the drilled section. The RQD value is the ratio, in percentage, of the total length of intact rock pieces longer than 100 mm in one drilled section over the length of the drilled section. These values are indicative of the quality of the bedrock.

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

A 51 mm in diameter PVC groundwater monitoring well was installed in the boreholes to permit monitoring of the groundwater levels subsequent to the completion of the drilling program. Monitoring wells will be decomissioned prior to the commencement of construction and during the excavation program for shallow monitoring wells.

Monitoring Well Installation

A groundwater monitoring well was installed in all boreholes upon completion of the sampling program. Typical monitoring well construction details are described below:

- ❏ Slotted 32 mm diameter PVC screen at base of borehole for 1.5 m length.
- ❏ 32 mm diameter PVC riser pipe from the top of the screen to the ground surface.
- ❏ No.3 silica sand backfill within annular space around screen.
- ❏ 300 mm thick bentonite hole plug directly above PVC slotted screen.
- ❏ Clean backfill from top of bentonite plug to the ground surface.

Refer to the Soil Profile and Test Data sheets in Appendix 1 for specific monitoring well construction details.

3.2 Field Survey

The borehole locations were determined by Paterson personnel taking into consideration the presence of underground and aboveground services. The location and ground surface elevation at each borehole location was surveyed by Paterson personnel. The boreholes were surveyed with respect to a temporary benchmark (TBM), consisting of the top of grate of a catch basin located near the exit of the existing parking lot. A geodetic elevation of 73.5 m was provided for the TBM. The test hole locations and ground surface elevations at the test hole locations are presented on Drawing PG4517-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

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

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

4.0 Observations

4.1 Surface Conditions

The subject site is currently asphalt covered and used as an at-grade parking lot. The ground surface across the subject is relatively flat with a slight downslope towards the east. The ground surface was observed to be at grade with the adjacent roadways.

The site is bordered by a multi-storey existing building along the northwest and west border line, by Queen's LRT Station along the northeast corner and by a sales center along the southeast corner.

4.2 Subsurface Profile

Overburden

The subsurface profile at the borehole locations consists of a pavement structure consisting of asphaltic concrete followed by a layer of crushed stone with sand and gravel fill. Glacial till which consisting of compact to dense brown silty sand with gravel, cobbles and boulders was encountered below the above noted layers followed by limestone bedrock at a depth ranging between 4.4 to 4.8 m below existing grade. Based on our observations, the upper 1 to 3 m of the bedrock is of fair to good quality, while the majority of the bedrock core was noted to be good to excellent quality.

Specific details of the subsurface profile at each test hole location are presented on the Soil Profile and Test Data sheets in Appendix 1.

Bedrock

Based on available geological mapping, the bedrock in this area consists of interbedded limestone and dolomite bedrock of the Gull River formation with an overburden drift thickness of 2 to 5 m depth.

4.3 Groundwater

A total of 6 groundwater monitoring wells were installed as part of our geotechnical investigation. Groundwater level measurements were recorded at the monitoring well locations and our findings are presented in Table 1.

Table 1 - Groundwater Measurements at Monitoring Well Locations

Test Hole Location	Ground Surface Elevation (m)	GW Level Reading (m)	GW Level Elev. (m)
BH 1	74.31	6.8	67.51
BH 2	73.79	4.2	69.59
BH 3	73.76	Dry	n/a
BH 4	73.68	n/a - No Access	n/a
BH 5	74.09	n/a - No Access	n/a
BH 6	74.02	5.8	68.22

Based on the groundwater levels presented above, the measured levels are expected to be a perched groundwater condition influenced by the moderate imperviousness of the glacial till deposit overlying the bedrock surface. The new LRT tunnel along Queen Street in only partially waterproofed and is drained. The long term dewatering of the existing tunnel has depressurized the groundwater condition within the bedrock. The tunnel is founded at an elevation of approximately 51.0 m which is approximately 23 m below the existing grade. Therefore, the long term water level will be within the bedrock.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered acceptable for the proposed buildings. It is anticipated that the proposed high rise buildings will be founded over shallow footings placed on a clean, surface sounded bedrock at elevation of approximately 47.0 m.

It should be noted that an existing building is located along the west and northwest border lines of the subject site. The foundation of the adjacent multi-storey building is expected to be founded on the limestone bedrock. Similarly, it is expected that the LRT station building along the northeast corner is founded over the underlying bedrock at approximate elevation 51.0 m. Therefore, underpinning will not be required for the these buildings during excavation of the proposed buildings.

Bedrock removal will be required to complete the underground parking levels. Line drilling and controlled blasting is recommended where large quantities of bedrock need to be removed. The blasting operations should be planned and completed under the guidance of a professional engineer with experience in blasting operations.

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

5.2 Site Grading and Preparation

Stripping Depth

Due to the depth of the bedrock, it is anticipated that all existing overburden material will be excavated from within the footprint of the proposed High Rise buildings.

Bedrock Removal

It is expected that line-drilling in conjunction with hoe-ramming or controlled blasting will be required to remove the bedrock for the underground parking levels. In areas of weathered bedrock and where only a small quantity of bedrock is to be removed, bedrock removal may be possible by hoe-ramming.

Prior to considering blasting operations, the blasting effects on the existing services, buildings and other structures should be addressed. A pre-blast or pre-construction survey of the existing structures located in proximity of the blasting operations should be carried out prior to commencing site activities. The extent of the survey should be determined by the blasting consultant and should be sufficient to respond to any inquiries/claims related to the blasting operations.

As a general guideline, peak particle velocities (measured at the structures) should not exceed 25 mm per second during the blasting program to reduce the risks of damage to the existing structures.

The blasting operations should be planned and conducted under the supervision of a licensed professional engineer who is also an experienced blasting consultant.

Excavation side slopes in sound bedrock can be carried out using almost vertical side walls. A minimum 1 m horizontal ledge, should be left between the bottom of the overburden excavation and the top of the bedrock surface to provide an area to allow for potential sloughing or to provide a stable base for the overburden shoring system.

Vibration Considerations

Construction operations are also the cause of vibrations, and possibly, sources of nuisance to the community. Therefore, means to reduce the vibration levels as much as possible should be incorporated in the construction operations to maintain, as much as possible, a cooperative environment with the residents.

The following construction equipments could be a source of vibrations: piling rig, hoe ram, compactor, dozer, crane, truck traffic, etc. The construction of the shoring system using soldier piles or sheet piling will require the use of these equipments. Vibrations, whether it is caused by blasting operations or by construction operations, could be the cause of the source of detrimental vibrations on the adjoining buildings and structures. Therefore, it is recommended that all vibrations be limited.

Two parameters are used to determine the permissible vibrations, namely, the maximum peak particle velocity and the frequency. For low frequency vibrations, the maximum allowable peak particle velocity is less than that for high frequency vibrations. As a guideline, the peak particle velocity should be less than 15 mm/s between frequencies of 4 to 12 Hz, and 50 mm/s above a frequency of 40 Hz (interpolate between 12 and 40 Hz). It should be noted that these guidelines are for today's construction standards. Considering that several old or sensitive buildings are encountered in the vicinity of the subject site, considerations should be given to lowering these guidelines. Considering that these guidelines are above perceptible human level and, in some cases, could be very disturbing to some people, it is recommended that a pre-construction survey be completed to minimize the risks of claims during or following the construction of the proposed building.

Fill Placement

Excavated limestone bedrock could be used as select subgrade material around the proposed building footings, provided the excavated bedrock is suitably crushed to 50 mm in its longest dimension and approved by the geotechnical consultant at the time of placement. Alternatively, an engineered fill such as an OPSS Granular A or Granular B Type II compacted to 98% of its SPMDD could be placed around the proposed footings.

Horizontal Rock Anchors

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

The requirements for horizontal rock anchors will be evaluated during the excavation operations and should be discussed with the structural engineer during the design stage.

5.3 Foundation Design

Bearing Resistance Values

Footings placed on the upper levels of the limestone bedrock surface should be cleaned and surface sounded. This bearing medium can be designed using a bearing resistance value at ultimate limit states (ULS) of **3,000 kPa**, incorporating a geotechnical resistance factor of 0.5. A clean, surface-sounded bedrock bearing surface should be free of loose materials, and have no near surface seams, voids, fissures or open joints which can be detected from surface sounding with a rock hammer.

A factored bearing resistance value at ULS of **6,000 kPa** could be used for footings founded on limestone bedrock at the proposed founding elevation of the parking garage provided the bedrock is free of seams, fractures and voids within 1.5 m below the founding level. This could be verified by completing and probing 50 mm diameter drill holes to a depth of 1.5 m below the founding level within the footprint(s) of the footing(s). At least one drill hole should be completed per major footing. The drill hole inspection should be carried out by the geotechnical consultant.

Consideration will be given to assessing the bedrock quality at the lower depths of the excavation program which can eliminate the need for bedrock probing.

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).

Settlement

Footings bearing on an acceptable bedrock bearing surface and designed using the bearing resistance values provided herein will be subjected to negligible potential post-construction total and differential settlements.

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class A** for the foundations considered at this site provided a site specific shear wave velocity test is completed to confirm the seismic site classification. Soils underlying the subject site are not susceptible to liquefaction. Reference should be made to the latest revision of the 2012 Ontario Building Code for a full discussion of the earthquake design requirements.

5.5 Basement Slab

All overburden soil will be removed from the subject site leaving the bedrock as the founding medium for the lower basement floor slab. If storage or other uses of the lower level where a concrete floor slab will be used it is recommended that the upper 200 mm of sub-slab fill consists of 19 mm clear crushed stone. All backfill material within the footprint of the proposed building should be placed in maximum 300 mm thick loose layers and compacted to at least 98% of its SPMDD.

In consideration of the groundwater conditions encountered at the time of the fieldwork, a subfloor drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided in the clear stone under the lower basement floor. The spacing of the underfloor drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

5.6 Basement Wall

It is understood that the basement walls are to be poured against a waterproofing system, which will be placed against the exposed bedrock face. Below the bedrock surface, a nominal coefficient for at-rest earth pressure of 0.05 is recommended in conjunction with a bulk unit weight of 24.5 kN/m^3 (effective 15.5 kN/m^3). A seismic earth pressure component will not be applicable for the foundation wall, which is to be poured against the bedrock face. It is expected that the seismic earth pressure will be transferred to the underground floor slabs, which should be designed to accommodate these pressures. A hydrostatic groundwater pressure should be added for the portion below the groundwater level.

Where soil is to be retained, 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. 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 13 kN/m^3, where applicable. A hydrostatic pressure should be added to the total static earth pressure when using the effective unit weight.

Two distinct conditions, static and seismic, must be reviewed for design calculations. The parameters for design calculations for the two conditions are presented below.

Static Conditions

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, 0.05
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 Conditions

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

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

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

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

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

5.7 Rock Anchor Design

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. It should be noted that interaction may develop between the failure cones of anchors that are relatively close to one another resulting in a total group capacity smaller than the sum of the load capacity of each anchor taken individually.

A third failure mode of shear failure along the grout/steel interface should also be reviewed by a qualified structural engineer to ensure all typical failure modes have been reviewed. Typical rock anchor suppliers, such as Dywidag Systems International (DSI Canada), have qualified personnel on staff to recommend appropriate rock anchor size and materials.

It should be further noted that centre to centre spacing between bond lengths be at least four times the anchor hole diameter and greater than 1.2 m to lower the group influence effects. It is also recommended that anchors in close proximity to each other be grouted at the same time to ensure any fractures or voids are completely in-filled and that fluid grout does not flow from one hole to an adjacent empty one.

Anchors can be of the “passive” or the “post-tensioned” type, depending on whether the anchor tendon is provided with post-tensioned load or not prior to being put into service.

Regardless of whether an anchor is of the passive or the post tensioned type, it is recommended that the anchor be provided with a bonded length, or fixed anchor length, at the base of the anchor, which will provide the anchor capacity, as well an unbonded length, or free anchor length, between the rock surface and the start of the bonded length. As the depth at which the apex of the shear failure cone develops is midway along the bonded length, a fully bonded anchor would tend to have a much shallower cone, and therefore less geotechnical resistance, than one where the bonded length is limited to the bottom part of the overall anchor.

Permanent anchors should be provided with corrosion protection. As a minimum, this requires that the entire drill hole be filled with cementitious grout. The free anchor length is provided by installing a plastic sleeve to act as a bond break.

Grout to Rock Bond

Based on compressive strength testing results, the unconfined compressive strength of limestone at the site is 116 to 156 MPa, which is stronger than most routine grouts. Conservatively, a compressive strength of 100 MPa can be used for bedrock at the subject site. 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.

Rock Cone Uplift

As discussed previously, the geotechnical capacity of the rock anchors depends on the dimensions of the rock anchors and the configuration of the anchorage system. Based on existing subsoils information, a **Rock Mass Rating (RMR) of 75** was assigned to the bedrock, and Hoek and Brown parameters (**m and s**) were taken as **1.20 and 0.022**, respectively.

Recommended Rock Anchor Lengths

Rock anchor lengths can be designed based on the required loads. Rock anchor lengths for some typical loads have been calculated and are presented on the following page. Load specified rock anchor lengths can be provided, if required.

For our calculations the following parameters were used.

Table 2 - Parameters used in Rock Anchor Review

Grout to Rock Bond Strength - Factored at ULS	1.0 MPa
Compressive Strength - Grout	40 MPa
Rock Mass Rating (RMR) - Good quality Limestone Hoek and Brown parameters	65 m=.575 and s=0.00293
Unconfined compressive strength - Limestone bedrock	80 MPa
Unit weight - Submerged Bedrock	15 kN/m^3
Apex angle of failure cone	60°
Apex of failure cone	mid-point of fixed anchor length

From a geotechnical perspective, the total anchor length will depend on the diameter of the drill holes. Recommended anchor lengths for a 75 and 125 mm diameter hole are provided in Table 3.

Table 3 - Recommended Rock Anchor Lengths - Grouted Rock Anchor

Diameter of Drill Hole (mm)	Anchor Lengths (m)			Factored Tensile Resistance (kN)
	Bonded Length	Unbonded Length	Total Length	
75	1.2	0.55	1.75	250
	2.0	0.8	2.8	500
	3.2	1.4	4.6	1000
	5.3	2.2	7.5	2000
125	1.0	0.5	1.5	250
	1.7	0.7	2.4	500
	2.6	1.1	3.7	1000
	4.1	1.8	5.9	2000

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.

5.8 Pavement Structure

For design purposes, the pavement structure presented in the following table could be used for the design of access lanes, if required.

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

Thickness (mm)	Material Description
40	**Wear Course -** Superpave 12.5 or HL-3 Asphaltic Concrete
50	**Binder Course** - Superpave 19.0 or HL-8 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
400	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Glacial Till or OPSS Granular B Type I or II material placed over bedrock.	

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 98% of the material's SPMDD using suitable vibratory equipment.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

It is recommended that a perimeter foundation drainage system be provided for the proposed structure. It is expected that insufficient room is available for exterior backfill. It is suggested that an adequate drainage system would be as follows:

- Bedrock vertical surface should be prepared to receive the proposed membrane for the area below the first two underground parking levels. The surface will be prepared by grinding or using shotcrete to smooth out angular sections depending on the manufacturer's requirements of the proposed waterproofing membrane.
- A waterproofing membrane will be applied to the prepared vertical bedrock surface from 21 m below grade to the founding elevation (bottom elevation of LRT tunnel). The membrane will serve as a water infiltration suppression system. The membrane will also be placed along the horizontal surface beneath the perimeter footings to provide a better seal at the vertical and horizontal interface.
- A composite drainage layer will be placed against the excavation face and waterproofing membrane from the surface to the proposed founding elevation.
- It is recommended that 150 mm diameter sleeves at 3 m centres be cast in the footing or at the foundation wall/footing interface to allow the infiltration of water to flow to the interior perimeter drainage pipe. The perimeter drainage pipe and underfloor drainage system should direct water to sump pit(s) within the lower basement area.

Underfloor Drainage

It is anticipated that underfloor drainage will be required to control water infiltration. For design purposes, we recommend that 150 mm in perforated pipes be placed in each bay. The spacing of the underfloor drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

Adverse Effects of Dewatering on Adjacent Properties

Since the proposed development will be founded below the long term groundwater level, a waterproofing membrane was recommended to lessen the effects of water infiltration. Any minor dewatering of the site will be within the bedrock layer which is relatively shallow at the subject site. Therefore, no adverse effects to the surrounding buildings or properties are expected with the lowering of the groundwater in this area.

Foundation Backfill

Above the bedrock surface, 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 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 I granular material, should otherwise be used for this purpose.

6.2 Protection of Footings Against Frost Action

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

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 and Temporary Shoring

The side slopes of the shallow excavations anticipated at this site should either be cut back at acceptable slopes or be retained by shoring systems from the start of the excavation until the structure is backfilled. It is expected that insufficient room will be available to permit the excavation to be undertaken by open-cut methods (i.e. unsupported excavations).

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

Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should be kept away from the excavation sides.

Slopes in excess of 3 m in height should be periodically inspected by the geotechnical consultant in order to detect if the slopes are exhibiting signs of distress.

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

Rock Stabilization

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

The requirement for horizontal rock anchors will be evaluated during the excavation operations and should be discussed with the structural engineer during the design stage.

Temporary Shoring

Temporary shoring may be required on the overburden soil to complete the required excavations where insufficient room is available for open cut methods. The shoring requirements will depend on the depth of the excavation, the proximity of the adjacent buildings and underground structures and the elevation of the adjacent building foundations and underground services. Additional information can be provided when the above details are known.

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. 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 by means of rock bolts or extending the piles into the bedrock through pre-augered holes if a soldier pile and lagging system is used.

Generally, it is expected that the shoring systems will be provided with tie-back rock anchors to ensure their stability.

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. Because the depth at which the apex shear failure cone develops is midway along the bonded length, a fully bonded anchor would tend to have a much shallower cone, and therefore less capacity, than one where the bonded length was just the bottom part of the overall anchor.

The design of the rock anchors for temporary shoring can be based on the values provided in Subsection 5.7 of the present report.

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

Table 5 - Soil Parameters for Shoring System Design

Parameters	Values
Active Earth Pressure Coefficient (K_a)	0.33
Passive Earth Pressure Coefficient (K_p)	3
At-Rest Earth Pressure Coefficient (K_o)	0.5
Unit Weight (γ), kN/m^3	20
Submerged Unit Weight (γ), kN/m^3	13

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 $K \gamma H$ for strutted or anchored shoring or a triangular earth pressure distribution with a maximum value of $K \gamma H$ 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 of Adjacent Structures

Based on the relatively shallow depth of the bedrock at the subject site, it is expected that the adjacent buildings are most likely founded on or very close to the bedrock surface except for the sales center. Therefore, underpinning may only be required for the sales center and should be confirmed in the field by the geotechnical consultant prior to commencement of excavation.

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 bedding for sewer and water pipes when placed on soil subgrade. 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 (concrete or PSM PVC pipes) or sand (concrete pipe). 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.

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 the potential differential frost heaving. The trench backfill should be placed in maximum 300 mm thick loose lifts and compacted to a minimum of 95% of the material's SPMDD.

6.5 Groundwater Control

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.

The rate of flow of groundwater into the excavation through the overburden and bedrock should be moderate for the expected subsurface conditions at this site. It is anticipated that pumping from open sumps will be sufficient to control the groundwater influx through the sides of the excavations.

A temporary MOECC permit to take water (PTTW) Category 3 will be required for this project since water infiltration is expected to be greater than 50,000 L/day during the construction phase. At least 3 to 4 months should be allowed for completion of the application and issuance of the permit by the MOECC.

Long-term Groundwater Control

Our recommendations for the proposed building's long-term groundwater control are presented in Subsection 6.1. Any groundwater which breaches the buildings' perimeter groundwater infiltration control systems will be directed to the proposed buildings' sump pits. It is expected that groundwater flow will be low (i.e.- less than 20,000 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 Structures

It is understood that preliminary concepts indicate that 8 levels of underground parking levels are planned for the proposed buildings. It is also understood that the neighbouring buildings are founded over bedrock bearing surface. Furthermore, based on the existing information, the LRT station was recently excavated to a maximum depth of 24 m below ground surface. Since the beginning of the LRT station construction, no issues related to localized groundwater lowering have been reported. Therefore, 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.

A proximity study will be completed once the final details of the proposed structure are available.

6.6 Winter Construction

Precautions must be taken if winter construction is considered for this project. The subsoil 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.

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.

The trench excavations should be carried out in a manner to avoid the introduction of frozen materials, snow or ice into the trenches. Precaution must be taken where excavations are carried 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.

7.0 Recommendations

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

☐ Review of the geotechnical aspects of the excavation contractor's shoring design, prior to construction.

☐ Review the bedrock stabilization and excavation requirements.

☐ Review proposed foundation drainage design and requirements.

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

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

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

☐ Observation of all subgrades prior to backfilling.

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

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

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

8.0 Statement of Limitations

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

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

The 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 Claridge Homes 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.

Faisal I. Abou-Seido, P.Eng.

Carlos P. Da Silva, P.Eng., ing., QP_{ESA}

LICENSED PROFESSIONAL ENGINEER
C. P. DA SILVA
PROVINCE OF ONTARIO

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APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

BOREHOLE LOGS BY OTHERS

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Mixed-Use Building - 383 Albert Street
Ottawa, Ontario

DATUM TBM - Top of grate of catch basin located near exit of subject site. Geodetic elevation = 73.50m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE July 3, 2018

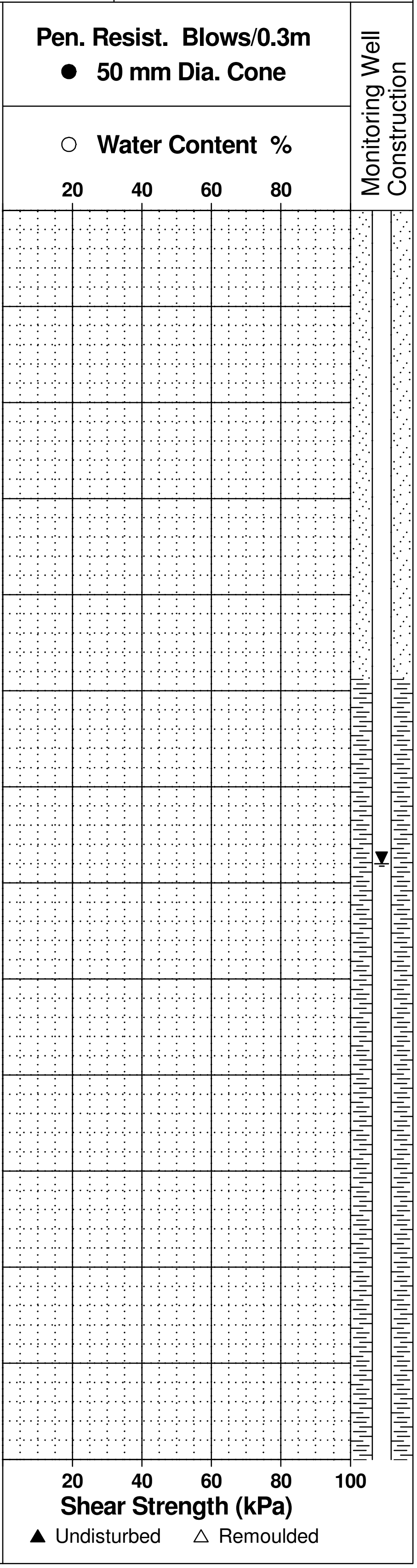
FILE NO. PG4517

HOLE NO. BH 1

SOIL DESCRIPTION			
GROUND SURFACE			
Asphaltic concrete	0.08		
FILL: Crushed stone with sand	0.69		
FILL: Brown silty sand, tracel, trace organics	1.50		
GLACIAL TILL: Compact to dense, brown silty sand with gravel, cobbles, boulders, trace clay	4.78		
BEDROCK: Grey limestone			

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	42	9
SS	3	79	27
SS	4	67	21
SS	5	83	11
SS	6	79	24
SS	7	67	50+
RC	1	71	100
RC	2	100	96
RC	3	100	88
RC	4	100	80
RC	5	100	93
RC	6	98	98

DEPTH (m)	ELEV. (m)
0	74.13
1	73.13
2	72.13
3	71.13
4	70.13
5	69.13
6	68.13
7	67.13
8	66.13
9	65.13
10	64.13
11	63.13
12	62.13
13	61.13



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
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REMARKS

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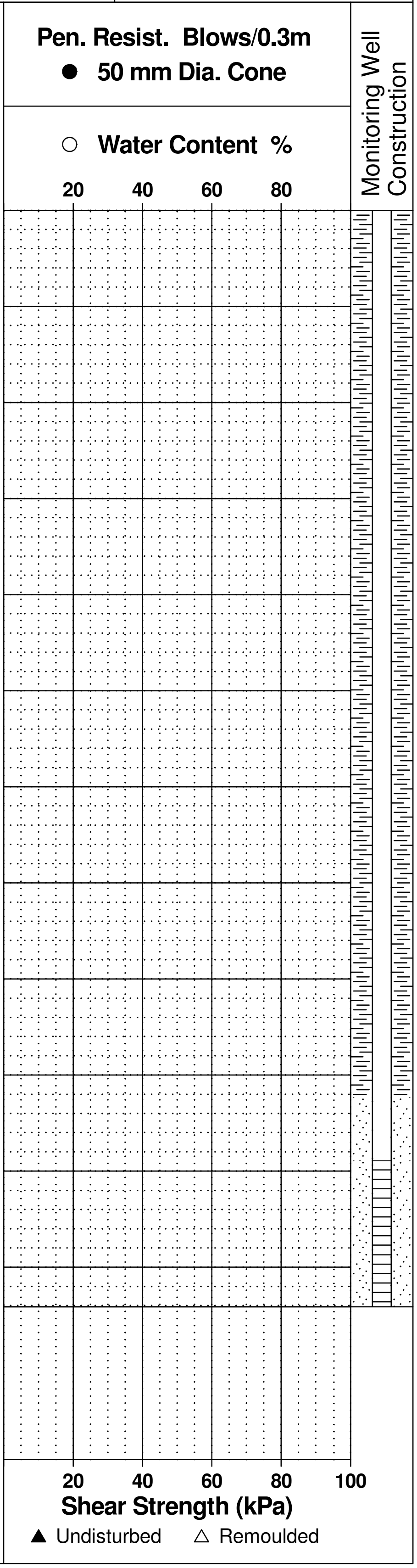
DATE July 3, 2018

FILE NO. PG4517

HOLE NO. BH 1

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						
					13	61.13
	RC	7	100	98		
					14	60.13
					15	59.13
	RC	8	100	100		
					16	58.13
	RC	9	100	100		
					17	57.13
					18	56.13
BEDROCK: Grey limestone	RC	10	95	95		
					19	55.13
	RC	11	100	98		
					20	54.13
					21	53.13
	RC	12	100	100		
					22	52.13
	RC	13	96	88		
					23	51.13
	RC	14	100	100	24	50.13
24.41						
End of Borehole						
(GWL @ 6.80m - July 13, 2018)						



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Mixed-Use Building - 383 Albert Street
Ottawa, Ontario

DATUM TBM - Top of grate of catch basin located near exit of subject site. Geodetic elevation = 73.50m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE July 4, 2018

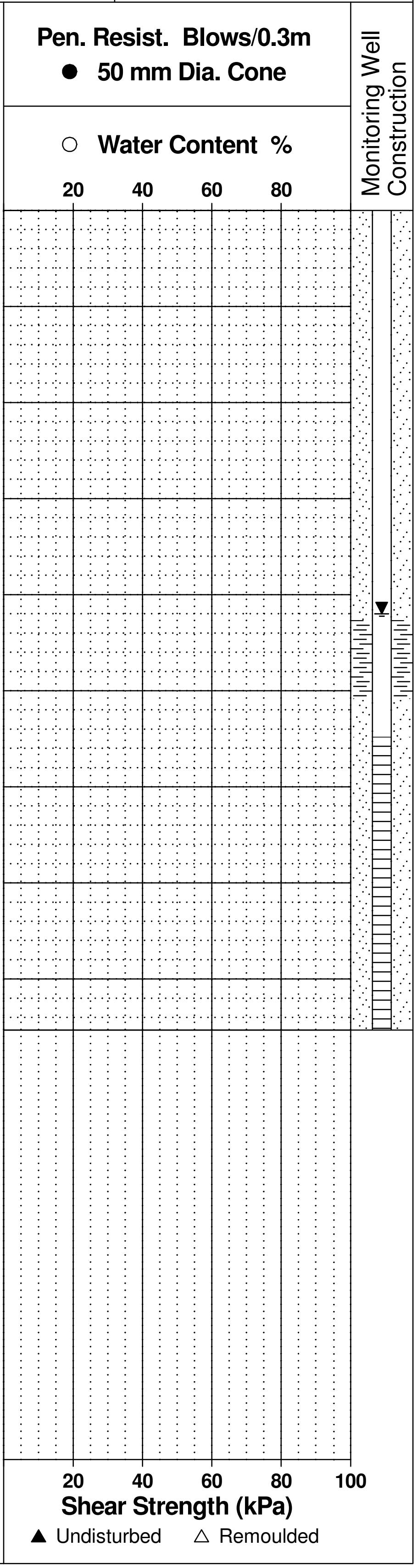
FILE NO. PG4517

HOLE NO. BH 2

SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
Asphaltic concrete	0.08
FILL: Brown silty sand with gravel, cobbles	2.21
GLACIAL TILL: Compact to dense, brown silty sand with gravel, cobbles, boulders, trace clay	4.57
BEDROCK: Grey limestone	8.53
End of Borehole (GWL @ 4.20m - July 13, 2018)	

TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	75	35
SS	3	58	39
SS	4	67	15
SS	5	33	16
SS	6	75	13
RC	1	95	80
RC	2	100	97
RC	3	98	88

DEPTH (m)	ELEV. (m)
0	73.79
1	72.79
2	71.79
3	70.79
4	69.79
5	68.79
6	67.79
7	66.79
8	65.79



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Mixed-Use Building - 383 Albert Street
Ottawa, Ontario

DATUM TBM - Top of grate of catch basin located near exit of subject site. Geodetic elevation = 73.50m.

REMARKS

BORINGS BY CME 55 Power Auger

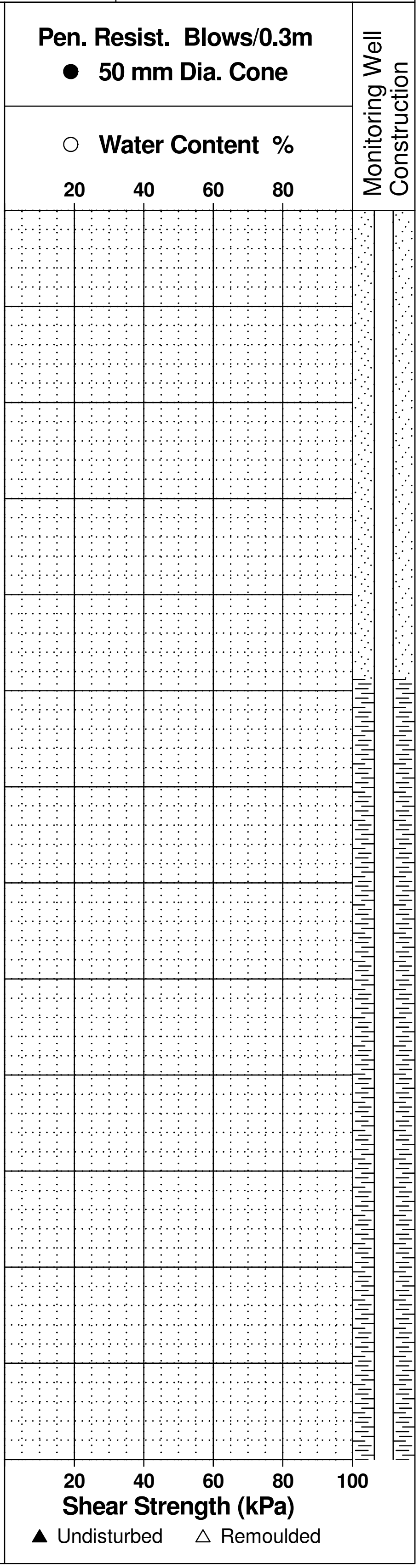
DATE July 4, 2018

FILE NO. PG4517

HOLE NO. BH 3

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
Asphaltic concrete (to 0.14)	AU	1		
FILL: Crushed stone, some sand, gravel, trace brick (to 2.97)	SS	2	25	53
	SS	3	0	50+
	SS	4	33	24
GLACIAL TILL: Compact to dense, brown silty sand, some gravel, cobble, boulders (to 4.40)	SS	5	46	16
	SS	6	73	50+
BEDROCK: Grey limestone	RC	1	100	84
	RC	2	95	86
	RC	3	100	88
	RC	4	100	85
	RC	5	100	100
	RC	6	100	98

DEPTH (m)	ELEV. (m)
0	73.76
1	72.76
2	71.76
3	70.76
4	69.76
5	68.76
6	67.76
7	66.76
8	65.76
9	64.76
10	63.76
11	62.76
12	61.76
13	60.76



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Mixed-Use Building - 383 Albert Street
Ottawa, Ontario

DATUM TBM - Top of grate of catch basin located near exit of subject site. Geodetic elevation = 73.50m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE July 4, 2018

FILE NO. PG4517

HOLE NO. BH 3

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
BEDROCK: Grey limestone	RC	7	100	98
	RC	8	100	96
	RC	9	100	88
	RC	10	100	90
	RC	11	100	100
	RC	12	100	100
23.77	RC	13	100	100
End of Borehole (BH dry - July 13, 2018)				

GROUND SURFACE

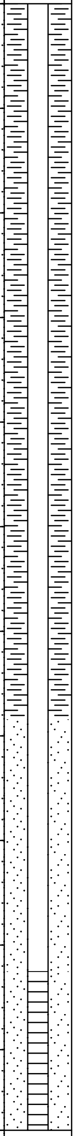
DEPTH (m)	ELEV. (m)
13	60.76
14	59.76
15	58.76
16	57.76
17	56.76
18	55.76
19	54.76
20	53.76
21	52.76
22	51.76
23	50.76

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

○ Water Content %

20 40 60 80

Monitoring Well Construction



20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Mixed-Use Building - 383 Albert Street
Ottawa, Ontario

DATUM TBM - Top of grate of catch basin located near exit of subject site. Geodetic elevation = 73.50m.

REMARKS

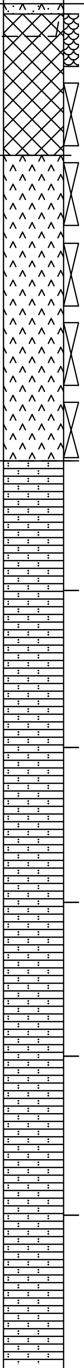
BORINGS BY CME 55 Power Auger

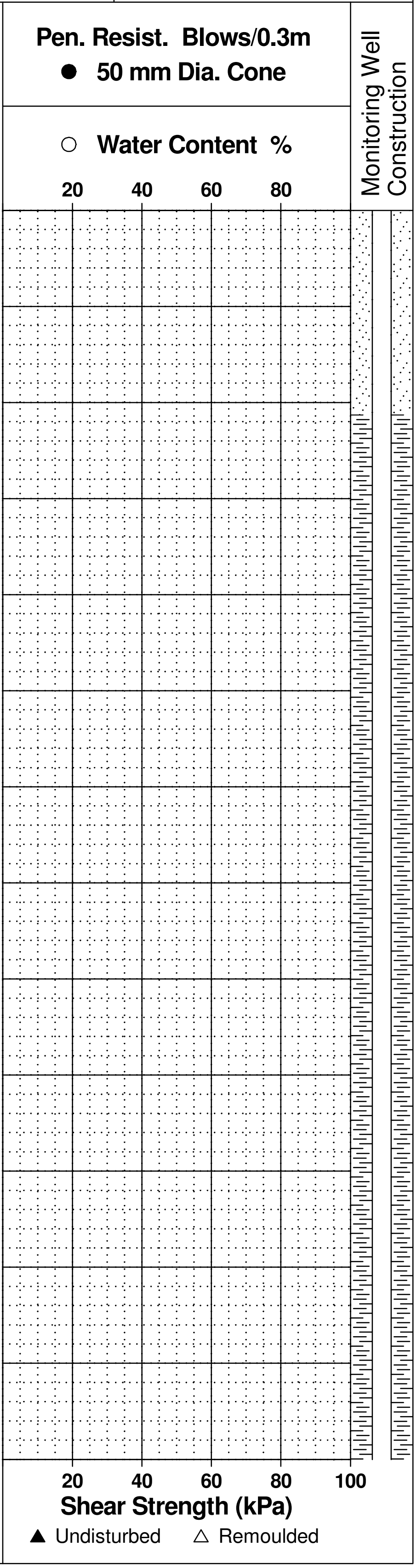
DATE July 4, 2018

FILE NO. PG4517

HOLE NO. BH 4

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	73.68
Asphaltic concrete 0.10	AU	1				
FILL: Brown silty sand with crushed stone, trace brick, coal 1.45	SS	2	42	13	1	72.68
GLACIAL TILL: Compact, brown silty sand with gravel, cobbles, boulders	SS	3	75	30	2	71.68
	SS	4	71	16		
	SS	5	83	12	3	70.68
4.37	SS	6	33	16	4	69.68
BEDROCK: Grey limestone	RC	1	95	82	5	68.68
	RC	2	98	98	6	67.68
	RC	3	100	93	7	66.68
					8	65.68
	RC	4	98	95	9	64.68
	RC	5	100	98	10	63.68
					11	62.68
	RC	6	100	100	12	61.68
					13	60.68





SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Mixed-Use Building - 383 Albert Street
Ottawa, Ontario

DATUM TBM - Top of grate of catch basin located near exit of subject site. Geodetic elevation = 73.50m.

REMARKS

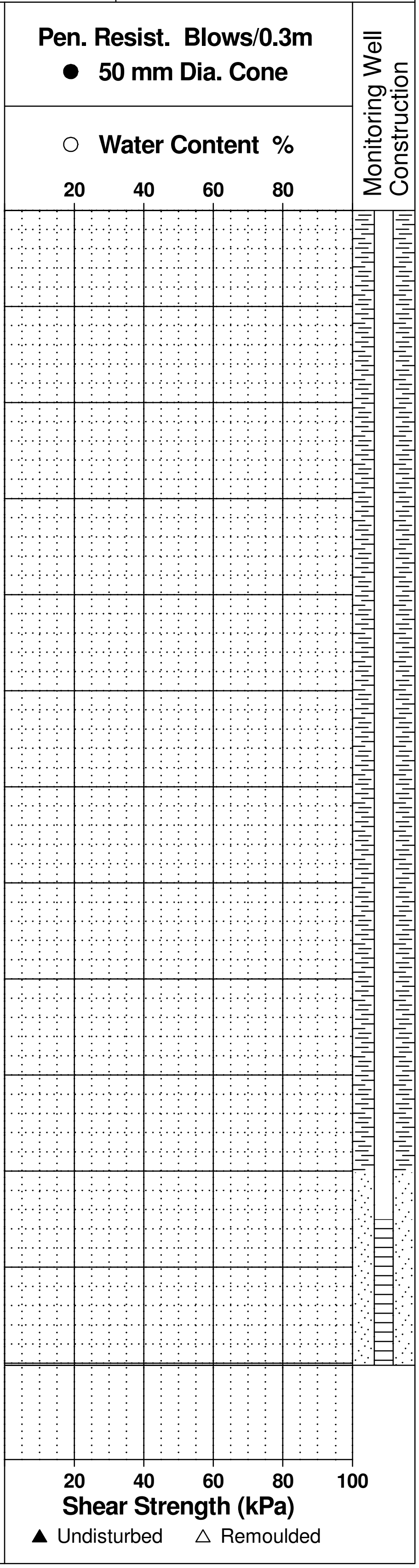
BORINGS BY CME 55 Power Auger

DATE July 4, 2018

FILE NO. PG4517

HOLE NO. BH 4

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						13	60.68
		RC	7	100	100		
						14	59.68
						15	58.68
		RC	8	100	100		
						16	57.68
		RC	9	100	91		
						17	56.68
						18	55.68
		RC	10	100	100		
BEDROCK: Grey limestone							
						19	54.68
		RC	11	100	100		
						20	53.68
						21	52.68
		RC	12	100	100		
						22	51.68
		RC	13	100	92		
						23	50.68
						24	49.68
		RC	14	98	94		
25.02						25	48.68
End of Borehole No access to borehole - July 13,							



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Mixed-Use Building - 383 Albert Street
Ottawa, Ontario

DATUM TBM - Top of grate of catch basin located near exit of subject site. Geodetic elevation = 73.50m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE July 4, 2018

FILE NO. PG4517

HOLE NO. BH 4

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	Monitoring Well Construction
GROUND SURFACE		TYPE	NUMBER	% RECOVERY	N VALUE or RQD				
2018)									

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
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Ottawa, Ontario

DATUM TBM - Top of grate of catch basin located near exit of subject site. Geodetic elevation = 73.50m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE July 5, 2018

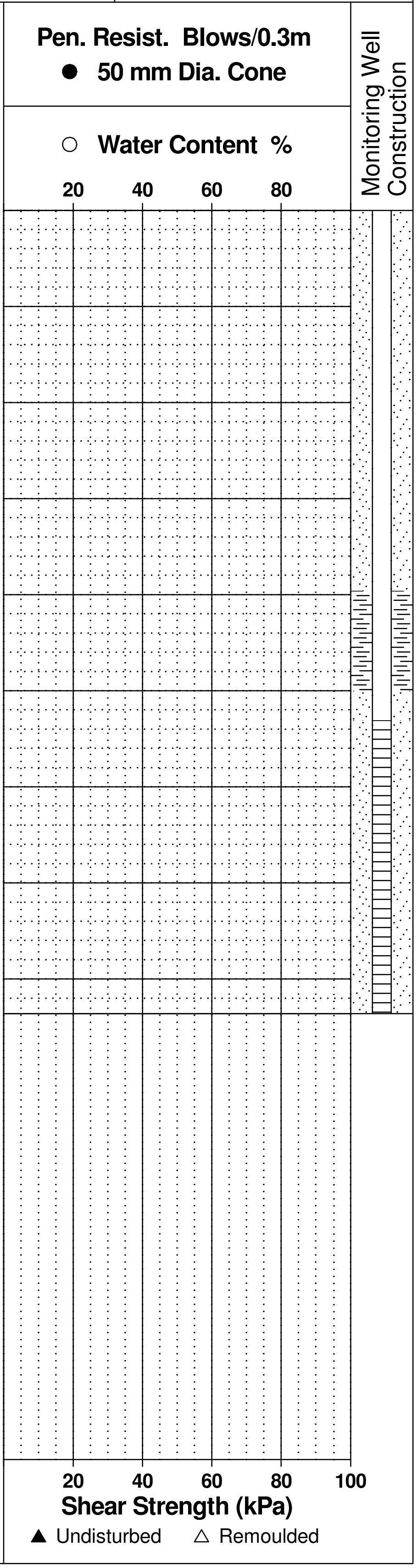
FILE NO. PG4517

HOLE NO. BH 5

SOIL DESCRIPTION	TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	74.09
Asphaltic concrete 0.13						
	AU	1				
FILL: Brown silty sand, some gravel	SS	2	33	15	1	73.09
	SS	3	67	8		
2.20					2	72.09
	SS	4	62	32		
					3	71.09
GLACIAL TILL: Compact to dense, brown silty sand with gravel, cobbles, boulders	SS	5	50	20		
	SS	6	12	47	4	70.09
4.42						
	RC	1	98	98	5	69.09
					6	68.09
BEDROCK: Grey limestone	RC	2	100	95		
					7	67.09
	RC	3	83	83		
					8	66.09
8.36						

End of Borehole

No access to borehole - July 13, 2018)



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SOIL PROFILE AND TEST DATA

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Proposed Mixed-Use Building - 383 Albert Street
Ottawa, Ontario

DATUM TBM - Top of grate of catch basin located near exit of subject site. Geodetic elevation = 73.50m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE July 6, 2018

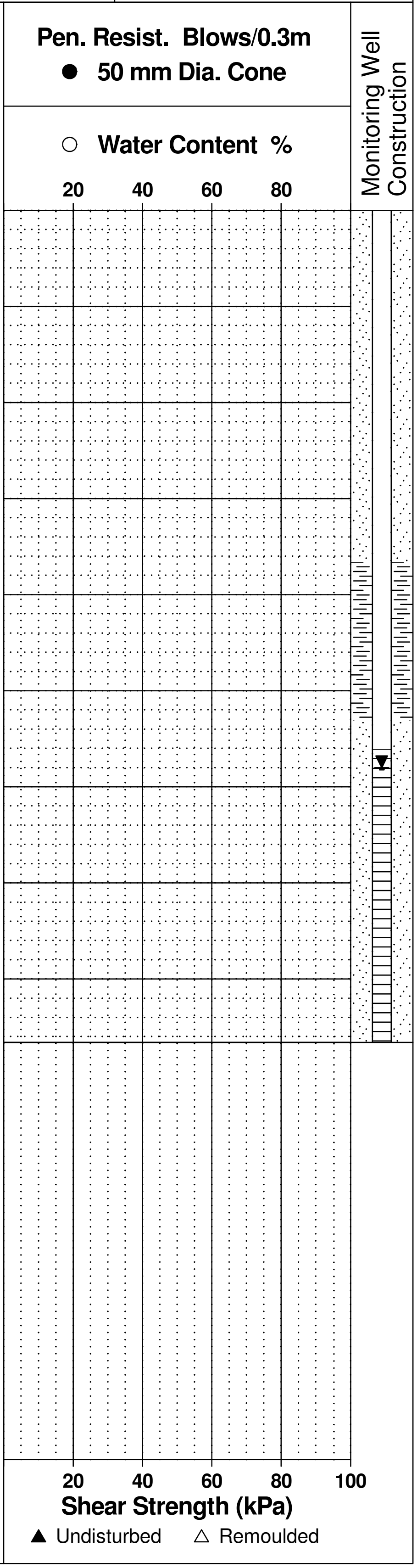
FILE NO. PG4517

HOLE NO. BH 6

SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
Asphaltic concrete	0.10
FILL: Brown silty sand with crushed stone, trace brick	2.13
GLACIAL TILL: Compact, brown silty sand with gravel, cobbles, boulders	4.55
BEDROCK: Grey limestone	8.66
End of Borehole (GWL @ 5.80m - July 13, 2018)	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	38	24
SS	3	25	22
SS	4	71	25
SS	5	12	15
RC	1	100	60
RC	2	98	95
RC	3	100	95

DEPTH (m)	ELEV. (m)
0	74.02
1	73.02
2	72.02
3	71.02
4	70.02
5	69.02
6	68.02
7	67.02
8	66.02



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 x D60)
Cu - Uniformity coefficient = D60 / D10

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

CONSOLIDATION TEST

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

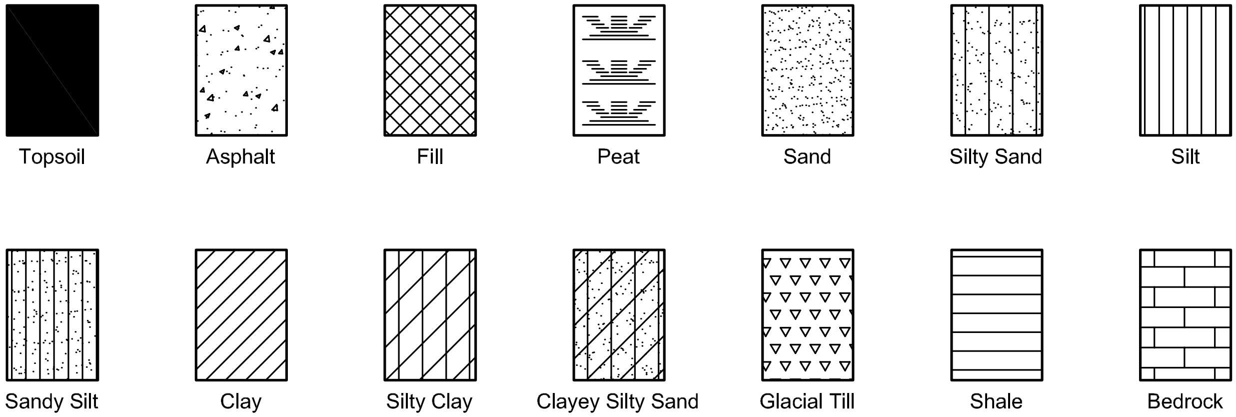
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

PERMEABILITY TEST

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

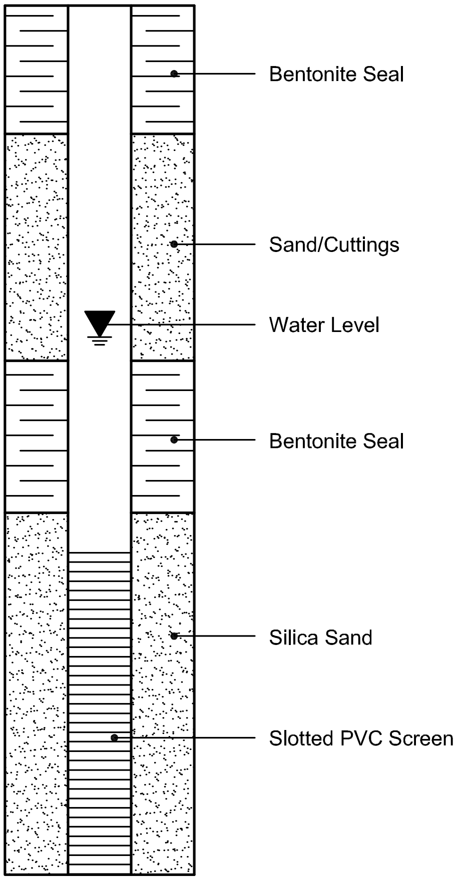
SYMBOLS AND TERMS (continued)

STRATA PLOT

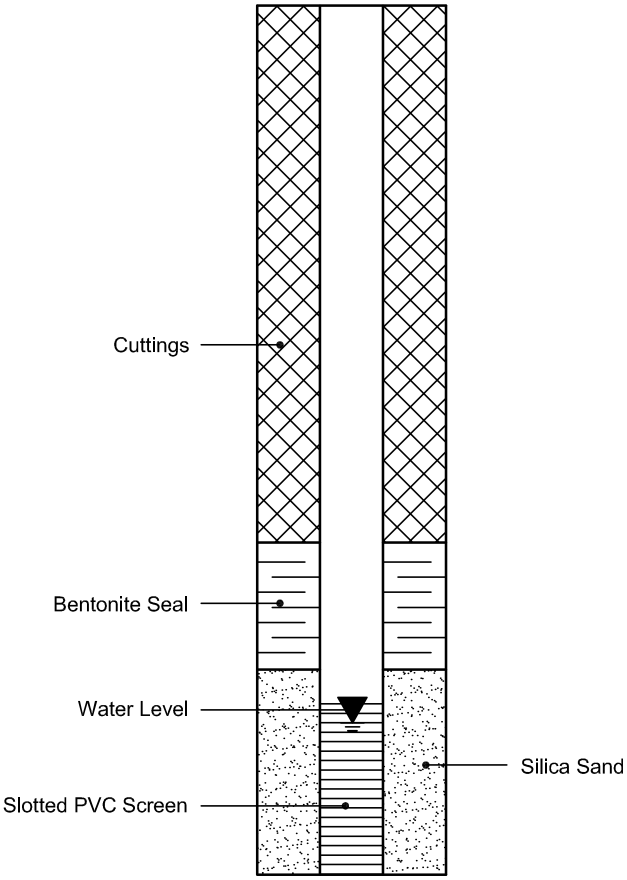


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Log of Borehole MW 1

Project No: OTT-00215048-A0

Figure No. 3

Page. 1 of 1

Project: Preliminary Geotechnical Investigation. Proposed Commercial and

Location: Residential Development. 383 Albert Street, Ottawa Ontario

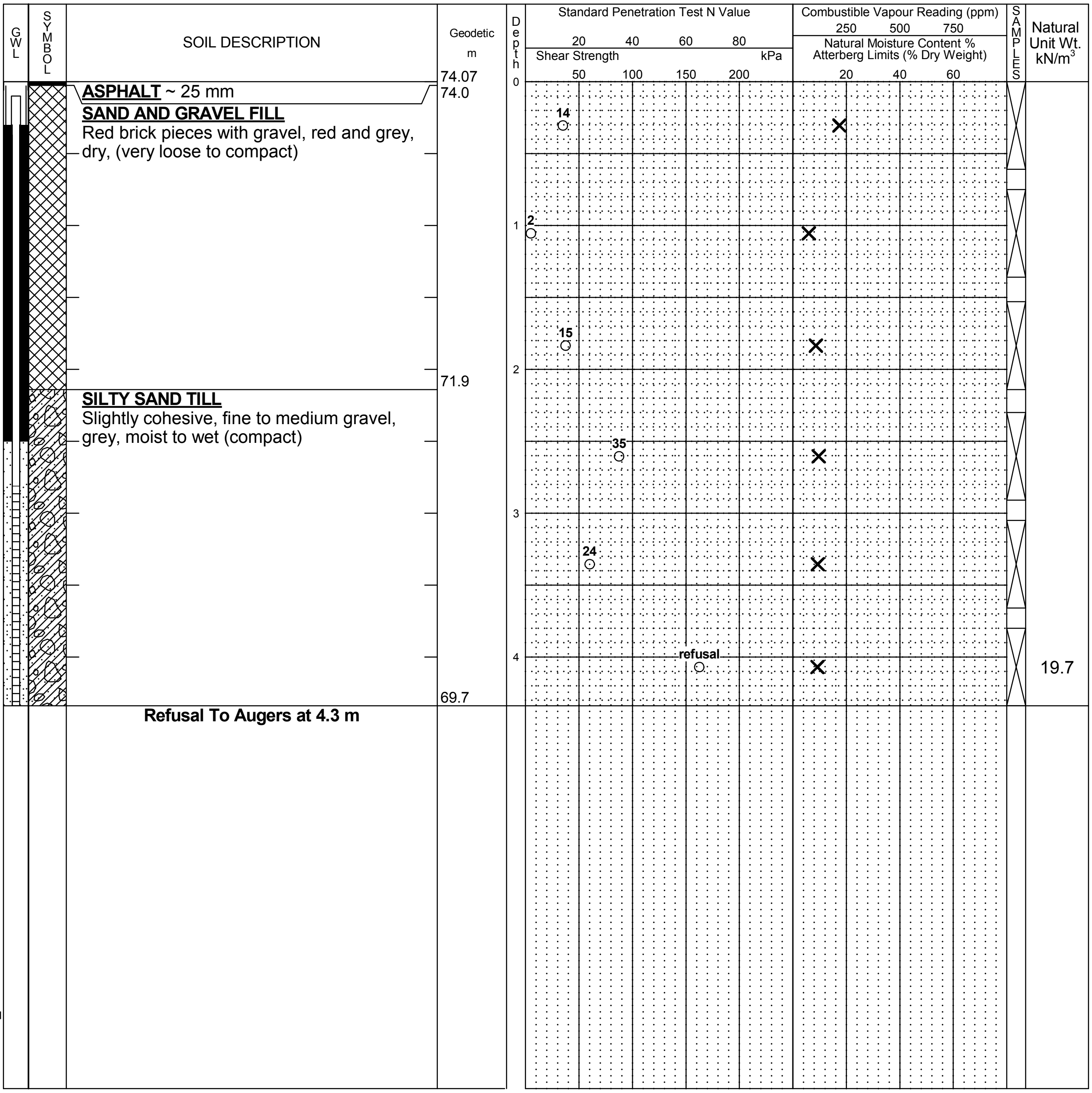
Date Drilled: 12/6/13

Drill Type: CME

Datum: Geodetic

Logged by: DC Checked by: SKA

Split Spoon Sample	☒	Combustible Vapour Reading	☐
Auger Sample	◫	Natural Moisture Content	×
SPT (N) Value	○	Atterberg Limits	⊢⊖
Dynamic Cone Test	—	Undrained Triaxial at % Strain at Failure	⊕
Shelby Tube	■		
Shear Strength by Vane Test	+ S	Shear Strength by Penetrometer Test	▲



ASPHALT ~ 25 mm

SAND AND GRAVEL FILL
Red brick pieces with gravel, red and grey, dry, (very loose to compact)

SILTY SAND TILL
Slightly cohesive, fine to medium gravel, grey, moist to wet (compact)

Refusal To Augers at 4.3 m

LOG OF BOREHOLE LOGS OF BOREHOLES_GEO.GPJ TROW OTTAWA.GDT 12/11/13

NOTES:
1. Borehole data requires interpretation by exp. before use by others
2. A Monitoring Well with a 38mm diameter casing was installed in the borehole upon completion.
3. Field work was supervised by an exp representative.
4. See Notes on Sample Descriptions
5. This Figure is to read with exp. Services Inc. report OTT-00215048-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
3 days	dry	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %

Log of Borehole MW 2

Project No: OTT-00215048-A0

Project: Preliminary Geotechnical Investigation. Proposed Commercial and

Location: Residential Development. 383 Albert Street, Ottawa Ontario

Figure No. 4(a)

Page. 1 of 2

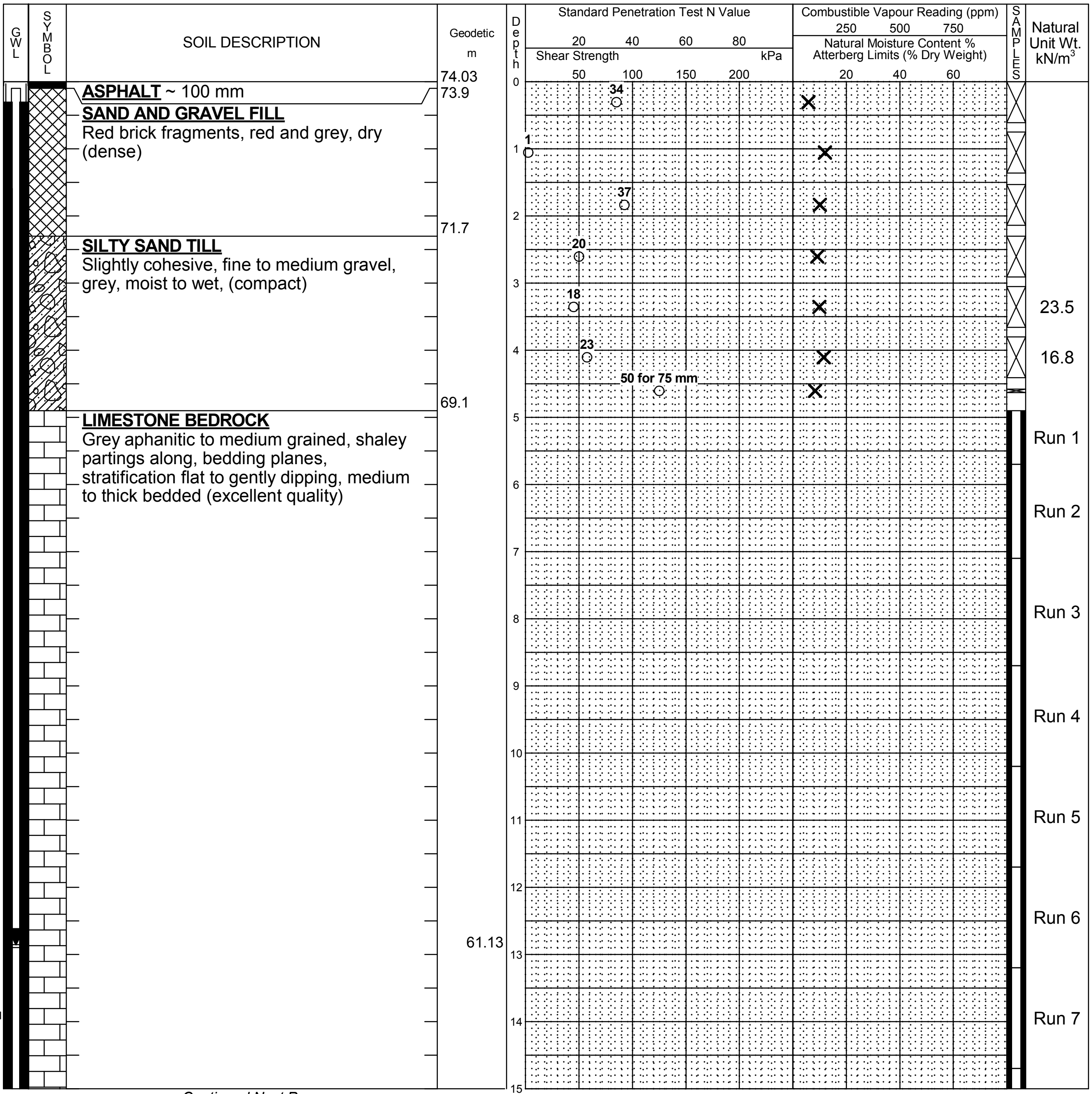
Date Drilled: 12/6/13

Drill Type: CME

Datum: Geodetic

Logged by: DC   Checked by: SKA

Split Spoon Sample	☒	Combustible Vapour Reading	☐
Auger Sample	◫	Natural Moisture Content	✕
SPT (N) Value	○	Atterberg Limits	├──○
Dynamic Cone Test	───	Undrained Triaxial at % Strain at Failure	⊕
Shelby Tube	■		
Shear Strength by Vane Test	+ S	Shear Strength by Penetrometer Test	▲



Geodetic m	Soil Description
74.03 73.9	**ASPHALT** ~ 100 mm
	SAND AND GRAVEL FILL Red brick fragments, red and grey, dry (dense)
71.7	**SILTY SAND TILL** Slightly cohesive, fine to medium gravel, grey, moist to wet, (compact)
69.1	**LIMESTONE BEDROCK** Grey aphanitic to medium grained, shaley partings along, bedding planes, stratification flat to gently dipping, medium to thick bedded (excellent quality)
61.13	

Natural Unit Wt. kN/m³
23.5
16.8
Run 1
Run 2
Run 3
Run 4
Run 5
Run 6
Run 7

Continued Next Page

NOTES:
1. Borehole data requires interpretation by exp. before use by others
2. A Monitoring Well with a 38mm diameter casing was installed in the borehole upon completion.
3. Field work was completed by an exp representative.
4. See Notes on Sample Descriptions
5. This Figure is to read with exp. Services Inc. report OTT-00215048-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
3 days	12.9	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %

Log of Borehole MW 2

Project No: OTT-00215048-A0

Project: Preliminary Geotechnical Investigation. Proposed Commercial and

Figure No. 4(a)

Page. 2 of 2

GWL	SYMBOL	SOIL DESCRIPTION	Geodetic m	Depth	Standard Penetration Test N Value 20 40 60 80 / Shear Strength 50 100 150 200 kPa	Combustible Vapour Reading (ppm) 250 500 750 / Natural Moisture Content % / Atterberg Limits (% Dry Weight) 20 40 60	SAMPLES	Natural Unit Wt. kN/m³
		LIMESTONE BEDROCK Grey aphanitic to medium grained, shaley partings along, bedding planes, stratification flat to gently dipping, medium to thick bedded (excellent quality) *(continued)*	59.03	15				Run 8
				16				Run 9
				17				Run 10
				18				Run 11
				19				Run 12
				20				Run 13
				21				Run 14
				22				Run 15
				23				
				24				
				25				
			47.7	26				

Borehole Terminated at 26.4 m

Noted
For Core Recovery refer to Figure 4(b)

NOTES:
1. Borehole data requires interpretation by exp. before use by others
2. A Monitoring Well with a 38mm diameter casing was installed in the borehole upon completion.
3. Field work was completed by an exp representative.
4. See Notes on Sample Descriptions
5. This Figure is to read with exp. Services Inc. report OTT-00215048-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
3 days	12.9	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %

Figure 4 (b)

Core Recoveries from MW13-2 at 383 Albert Street, Ottawa

Run #	Depth (m)	Total Core Recovery %	Rock Quality Designation (RQD) %
1	4.93 – 5.69	100	93
2	5.69 – 7.14	100	100
3	7.14 – 8.69	100	89
4	8.69 – 10.16	100	90
5	10.16 – 11.69	100	100
6	11.69 – 13.19	100	100
7	13.19 – 14.66	98	98
8	14.66 – 16.18	100	100
9	16.18 – 17.68	100	100
10	17.68 – 19.21	98	95
11	19.21 – 20.73	100	97
12	20.73 – 22.20	100	100
13	22.20 – 23.73	100	100
14	23.73 – 25.20	100	100
15	25.20 – 26.35	100	100

Log of Borehole BH 3

Project No: OTT-00215048-A0

Project: Preliminary Geotechnical Investigation. Proposed Commercial and

Location: Residential Development. 383 Albert Street, Ottawa Ontario

Figure No. 5

Page. 1 of 1

Date Drilled: 12/7/13

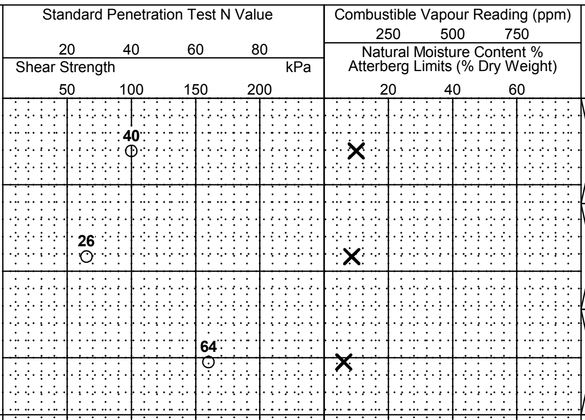
Drill Type: CME

Datum: Geodetic

Logged by: TG Checked by: SKA

Split Spoon Sample	☒	Combustible Vapour Reading	☐
Auger Sample	◫	Natural Moisture Content	✕
SPT (N) Value	○	Atterberg Limits	├─○
Dynamic Cone Test	───	Undrained Triaxial at % Strain at Failure	⊕
Shelby Tube	■		
Shear Strength by Vane Test	+ S	Shear Strength by Penetrometer Test	▲

GWL	SYMBOL	SOIL DESCRIPTION	Geodetic m	Depth	SAMPLES	Natural Unit Wt. kN/m^3
			73.74	0		
	■	**ASPHALT** ~ 25 mm	73.7			
	▨	**SAND AND GRAVEL FILL** Fine to coarse sand, some gravel, grey moist (dense)	73.1		☒	
	▨	**SILTY SAND TILL** Slightly cohesive, some gravel, moist, grey, (compact to dense)	71.9	1	☒ ☒	
		Borehole Terminated at 1.8 m				



NOTES:
1. Borehole data requires interpretation by exp. before use by others
2. Hole was backfilled upon completion.
3. Field work was supervised by an exp representative.
4. See Notes on Sample Descriptions
5. This Figure is to read with exp. Services Inc. report OTT-00215048-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %

Log of Borehole MW 4

Project No: OTT-00215048-A0

Project: Preliminary Geotechnical Investigation. Proposed Commercial and

Location: Residential Development. 383 Albert Street, Ottawa Ontario

Figure No. 6

Page. 1 of 1

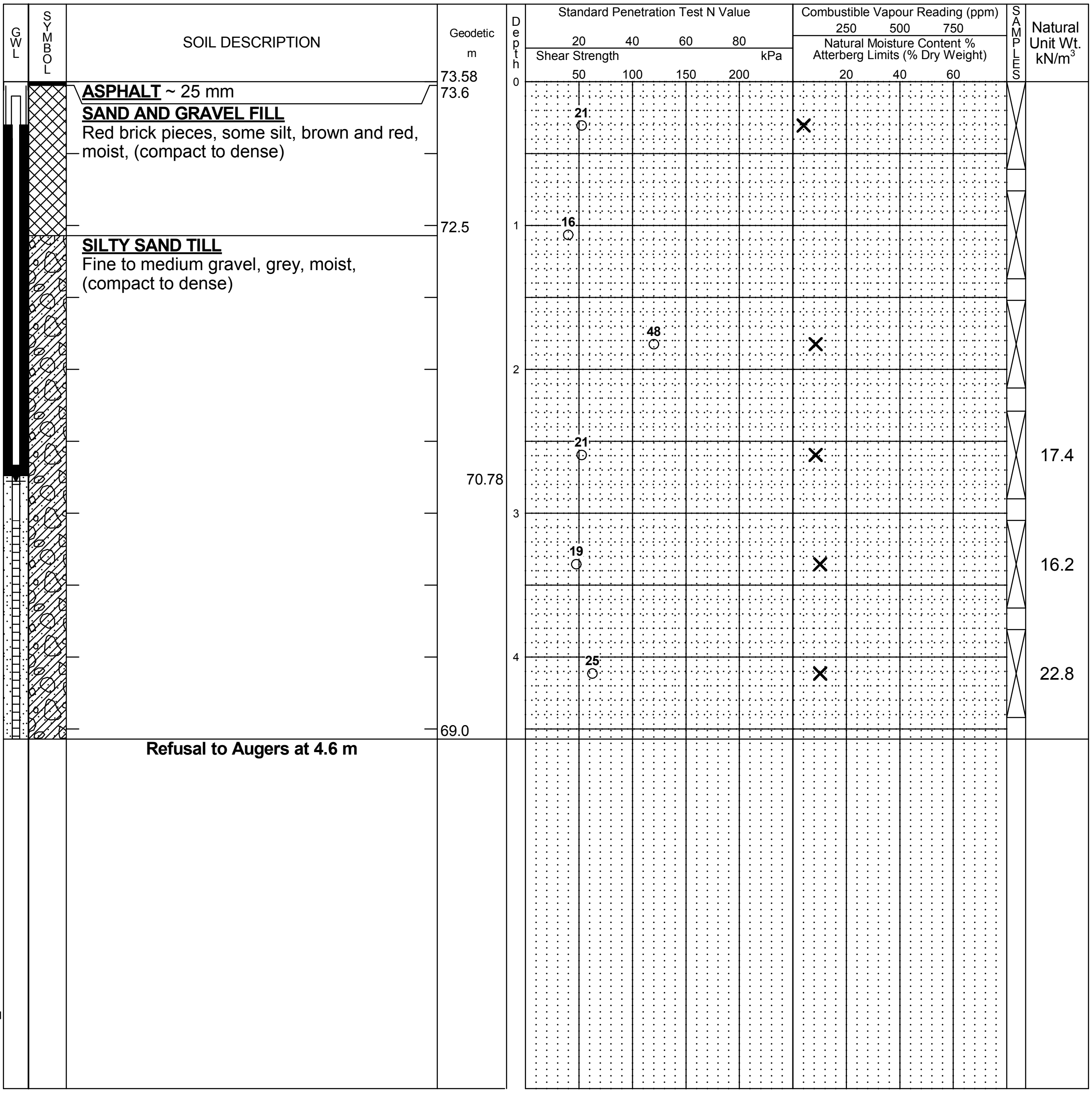
Date Drilled: 12/7/13

Drill Type: CME

Datum: Geodetic

Logged by: TG Checked by: SKA

Split Spoon Sample	☒	Combustible Vapour Reading	☐
Auger Sample	◨	Natural Moisture Content	×
SPT (N) Value	○	Atterberg Limits	⊢○
Dynamic Cone Test	―	Undrained Triaxial at % Strain at Failure	⊕
Shelby Tube	■		
Shear Strength by Vane Test	+ S	Shear Strength by Penetrometer Test	▲



ASPHALT ~ 25 mm

SAND AND GRAVEL FILL
Red brick pieces, some silt, brown and red, moist, (compact to dense)

SILTY SAND TILL
Fine to medium gravel, grey, moist, (compact to dense)

Refusal to Augers at 4.6 m

NOTES:
1. Borehole data requires interpretation by exp. before use by others
2. A Monitoring Well with a 38mm diameter casing was installed in the borehole upon completion.
3. Field work was supervised by an exp representative.
4. See Notes on Sample Descriptions
5. This Figure is to read with exp. Services Inc. report OTT-00215048-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
3 days	2.8	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %

Log of Borehole BH 5

Project No: OTT-00215048-A0

Project: Preliminary Geotechnical Investigation. Proposed Commercial and

Location: Residential Development. 383 Albert Street, Ottawa Ontario

Figure No. 7

Page. 1 of 1

Date Drilled: 12/7/13

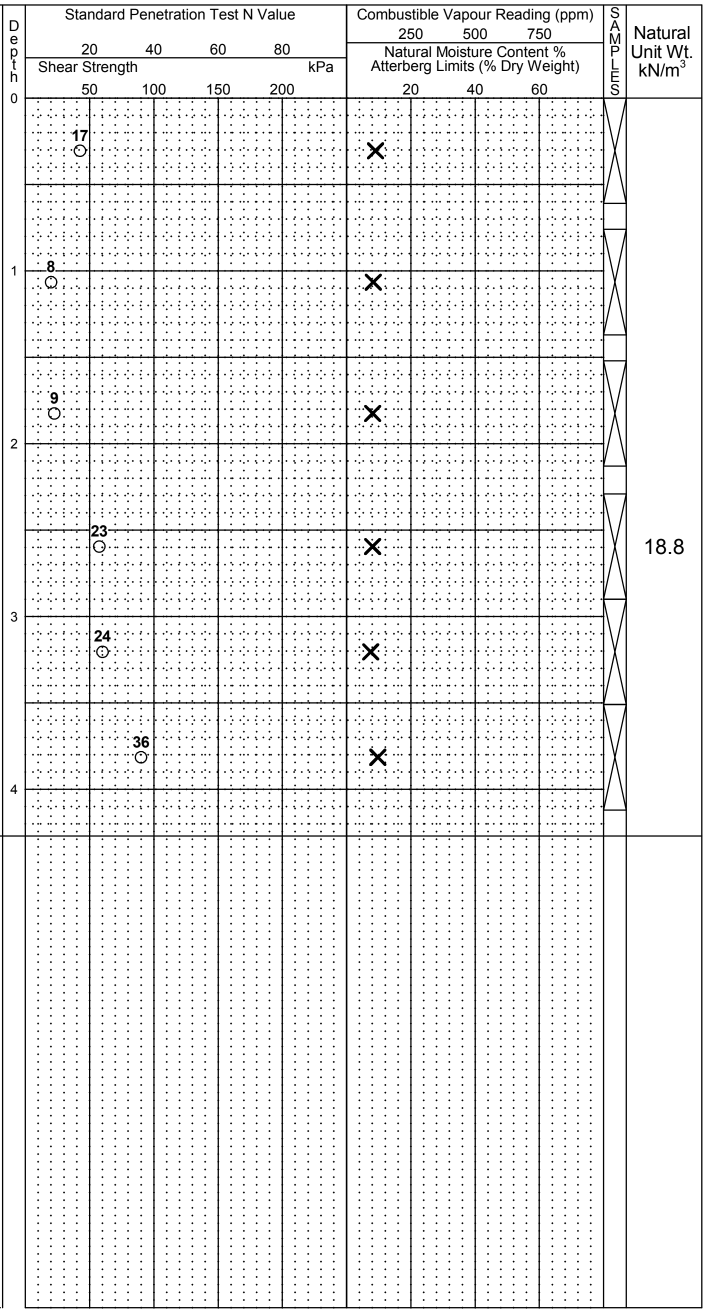
Drill Type: CME

Datum: Geodetic

Logged by: TG Checked by: SKA

Split Spoon Sample	☒	Combustible Vapour Reading	☐
Auger Sample	◨	Natural Moisture Content	✕
SPT (N) Value	○	Atterberg Limits	├─○
Dynamic Cone Test	───	Undrained Triaxial at % Strain at Failure	⊕
Shelby Tube	■	Shear Strength by Penetrometer Test	▲
Shear Strength by Vane Test	+ S		

GWL	SYMBOL	SOIL DESCRIPTION	Geodetic m
			73.59
		ASPHALT ~ 25 mm	73.6
		SAND AND GRAVEL FILL Some silt, brown, moist, (compact)	
		SILTY SAND TILL Fine to medium gravel, slightly cohesive, grey, moist to wet, (loose to dense)	72.5
			69.3
		Refusal to Augers at 4.3 m	



NOTES:
1. Borehole data requires interpretation by exp. before use by others
2. Hole was backfilled upon completion.
3. Field work was supervised by an exp representative.
4. See Notes on Sample Descriptions
5. This Figure is to read with exp. Services Inc. report OTT-00215048-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %

Log of Borehole MW 6

Project No: OTT-00215048-A0

Project: Preliminary Geotechnical Investigation. Proposed Commercial and

Location: Residential Development. 383 Albert Street, Ottawa Ontario

Figure No. 8

Page. 1 of 1

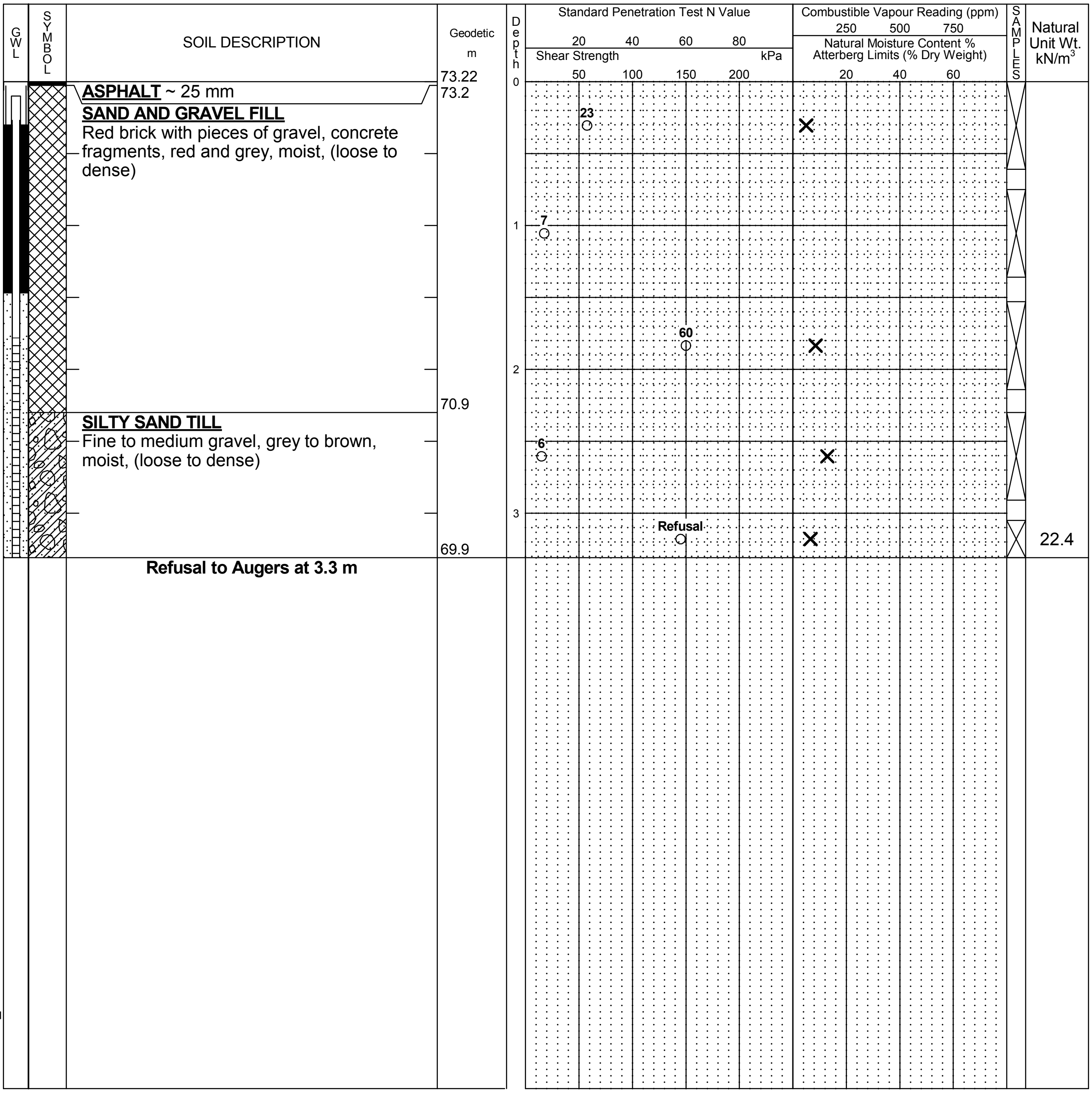
Date Drilled: 12/6/13

Drill Type: CME

Datum: Geodetic

Logged by: DC Checked by: SKA

Split Spoon Sample	☒	Combustible Vapour Reading	☐
Auger Sample	◧	Natural Moisture Content	✕
SPT (N) Value	○	Atterberg Limits	⊢○
Dynamic Cone Test	—	Undrained Triaxial at % Strain at Failure	⊕
Shelby Tube	■		
Shear Strength by Vane Test	+ S	Shear Strength by Penetrometer Test	▲



73.22

ASPHALT ~ 25 mm — 73.2

SAND AND GRAVEL FILL
Red brick with pieces of gravel, concrete fragments, red and grey, moist, (loose to dense)

SILTY SAND TILL — 70.9
Fine to medium gravel, grey to brown, moist, (loose to dense)

69.9

Refusal to Augers at 3.3 m

NOTES:

1. Borehole data requires interpretation by exp. before use by others
2. A Monitoring Well with a 38mm diameter casing was installed in the borehole upon completion.
3. Field work was supervised by an exp representative.
4. See Notes on Sample Descriptions
5. This Figure is to read with exp. Services Inc. report OTT-00215048-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
3 days	dry	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %

APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING PG4517-1 - TEST HOLE LOCATION PLAN

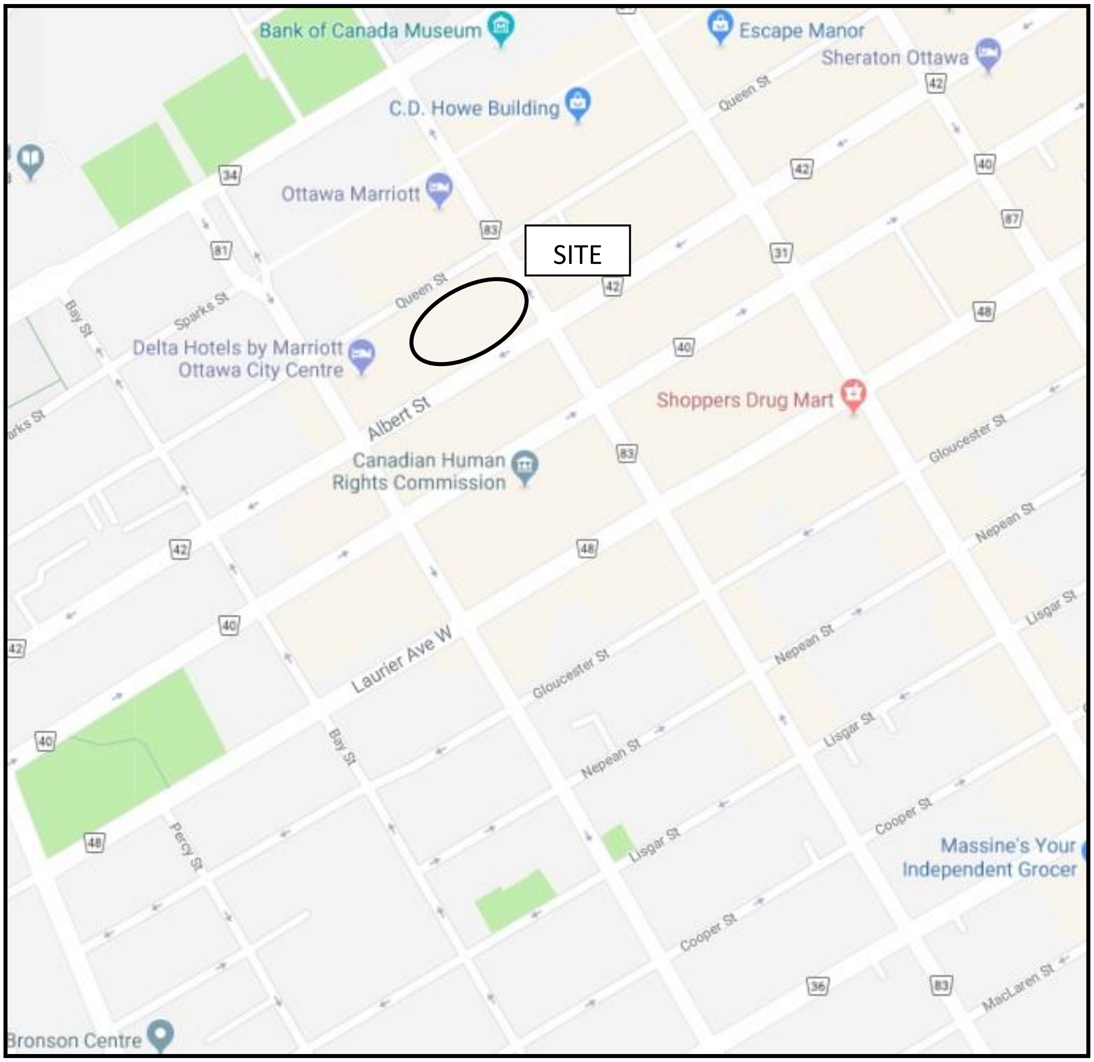
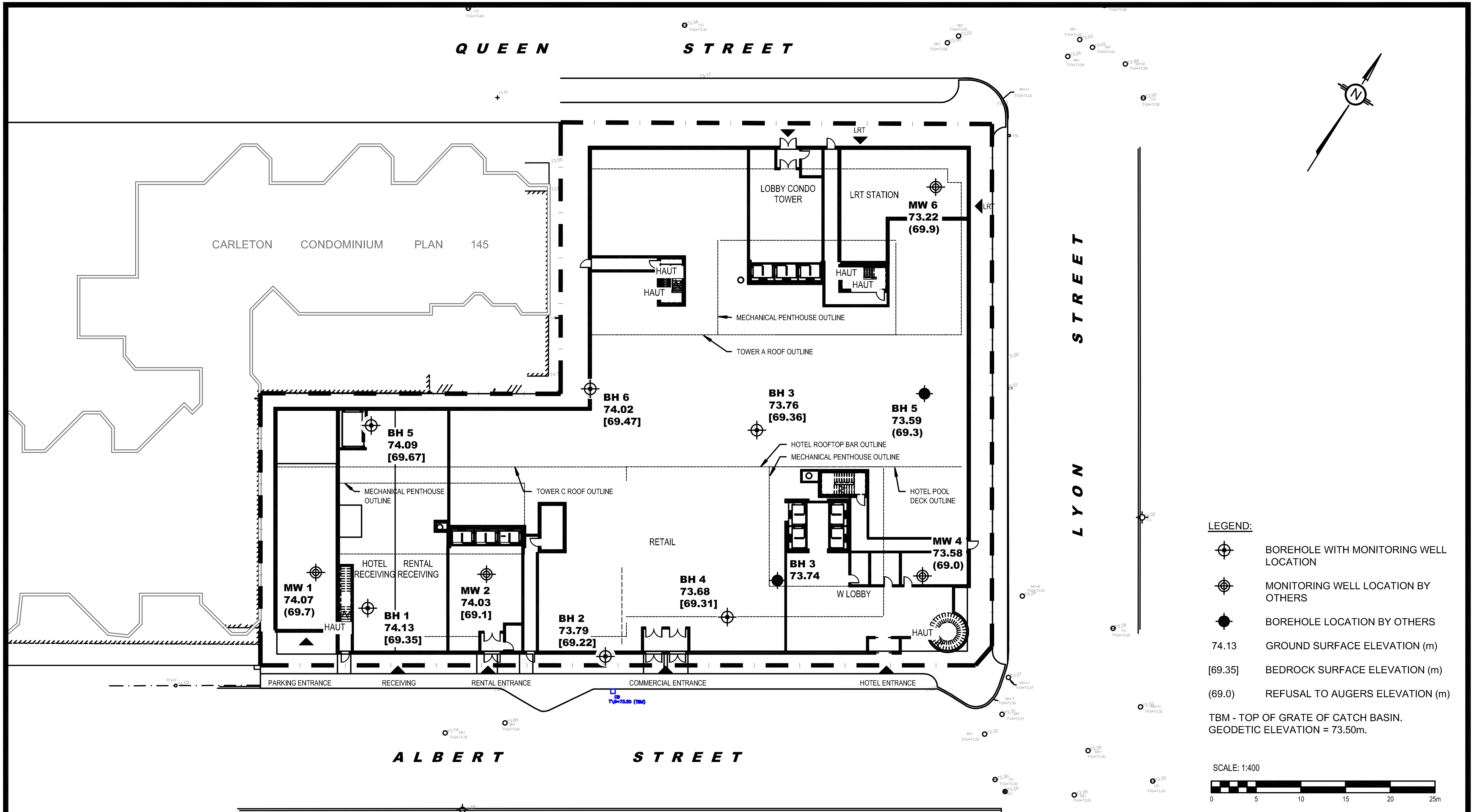


FIGURE 1

KEY PLAN



LEGEND:

⌖	BOREHOLE WITH MONITORING WELL LOCATION
⌖	MONITORING WELL LOCATION BY OTHERS
●	BOREHOLE LOCATION BY OTHERS
74.13	GROUND SURFACE ELEVATION (m)
[69.35]	BEDROCK SURFACE ELEVATION (m)
(69.0)	REFUSAL TO AUGERS ELEVATION (m)

TBM - TOP OF GRATE OF CATCH BASIN.
GEODETIC ELEVATION = 73.50m.

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NO.	REVISIONS	DATE	INITIAL
0			

CLARIDGE HOMES
GEOTECHNICAL INVESTIGATION
PROP. MIXED-USE BUILDING - 383 ALBERT STREET
OTTAWA, ONTARIO

Title:
TEST HOLE LOCATION PLAN

Scale: 1:400
Date: 07/2018
Drawn by: MPG
Report No.: PG4517-1
Checked by: FA
Dwg. No.: PG4517-1
Approved by: FA
Revision No.: 0

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