Geotechnical Engineering

Environmental Engineering

Hydrogeology

Geological Engineering

Materials Testing

Building Science

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Geotechnical Investigation
Proposed Multi-Storey Building
1050 Somerset Street West
Ottawa, Ontario

Prepared For

Claridge Homes

February 6, 2012

Report PG2356-1

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APPENDICES

1.0 INTRODUCTION

Paterson Group (Paterson) was commissioned by Claridge Homes to conduct a geotechnical investigation for a proposed multi-storey building which is to be located at 1050 Somerset Street West 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 boreholes.

☐ Based on the results of the boreholes, provide geotechnical recommendations pertaining to 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 of this present investigation. Therefore, the present report does not address environmental issues.

A Phase I-II - Environmental Site Assessment (ESA) was completed by Paterson concurrently with the present investigation, the findings and recommendations are presented under separate cover.

2.0 PROPOSED DEVELOPMENT

Specific details of the proposed development are not known during the geotechnical investigation. However, it is understood that the proposed development will consist of a multi-storey residential building with five (5) levels of underground parking.

3.0 METHOD OF INVESTIGATION

3.1 Field Investigation

The field program for our geotechnical investigation was carried out on April 18, 19, July 5 and 6, 2011. At that time, eight (8) boreholes were advanced to a maximum depth of 13 m below existing ground surface. The borehole locations were distributed in a manner to provide general coverage of the proposed development and taking into consideration site features and underground services. The approximate locations of the boreholes are shown on Drawing PG2356-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were put down using a truck-mounted auger drill rig or advancing down using portable drilling equipment 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. The drilling procedure consisted of augering to the required depths at the selected locations, and sampling and testing the overburden.

Sampling and In Situ Testing

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

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

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

Diamond drilling was carried out at BH 3 and BH 7 to determine the nature of the bedrock and to assess its quality. A recovery value and a Rock Quality Designation (RQD) value were calculated for each drilled section of bedrock and are shown on the borehole logs. The recovery value is the ratio, in percentage, of the length of the bedrock sample recovered over the length of the drilled section. 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.

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

Groundwater

Groundwater monitoring wells were installed in BH1 and BH 5 and flexible polyethylene standpipes were installed in the remaining 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 month after issuance of this report. They will then be discarded unless we are otherwise directed.

3.2 Field Survey

The borehole locations were selected, determined in the field and surveyed by Paterson. The ground surface elevation at the borehole locations is referenced to temporary benchmark (TBM), consisting of a mag nail in the hydro pole located on the west side of the subject site. A geodetic elevation of 63.14 m was provided for this TBM by Annis O’Sullivan Vollebekk. The locations of the boreholes, TBM, and the ground surface elevations at the borehole locations, are presented on Drawing PG2356-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

The soil samples recovered from the subject site were 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 sample. The results are presented in Appendix 1 and are discussed further in Subsection 6.7.

4.0 OBSERVATIONS

4.1 Surface Conditions

The majority of the subject site is occupied by two (2) commercial buildings. The remainder of the site consists of asphalt covered parking areas. The subject site is relatively flat and at grade with surrounding properties and roadways.

4.2 Subsurface Profile

Generally, the soil conditions encountered at the borehole locations consist of a pavement structure overlying a varying fill material consisting of silty sand mixed with silty clay, gravel, cobbles and boulders. A native in situ stiff to very stiff brown to grey silty clay was encountered underlying the fill material at BH 7 and BH 4. A glacial till deposit was noted below the abovenoted layers.

It should be noted that BH 6 was completed in the basement level of the existing building extending through a 150 mm concrete floor slab overlying a thin layer of crushed stone bearing over a glacial till deposit.

A grey limestone bedrock was encountered at a 11 and 11.3 m depth at BH 3 and BH 7, respectively. The recovery values and RQD values for the bedrock cores were calculated in the field and reviewed in the laboratory. 100% recovery values were observed in the four (4) rock cores while the RQD values vary between 97 to 100%. Generally, the bedrock quality was observed to be excellent.

For specific details of the soil profiles at each test hole location refer to the Soil Profile and Test Data sheets in Appendix 1.

Based on available geological mapping, the subject site is located in an area where the bedrock consists of limestone, with interbeds of calcarenite and shale of the Lindsay formation with overburden thickness varying between 0 to 5 m.

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.

4.3 Groundwater

Groundwater levels (GWLs) were measured in the monitoring wells (MW) and standpipes installed at the borehole locations and the results are summarized in Table 1. The groundwater level can also be estimated based on moisture levels and colour of the recovered soil samples. Based on these observations at the borehole locations, the groundwater table is expected between a 3.5 to 4.5 m below existing ground surface. It should be noted that groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater level could vary at the time of construction.

Table 1
Summary of Groundwater Level Readings

Borehole Number	Ground Elevation (m)	Groundwater Levels (m)		Recording Date
		Depth	Elevation	
BH 1 (MW)	62.44	1.10	61.34	April 26, 2011
BH 2	62.56	6.10	56.46	April 26, 2011
BH 3	61.84	4.38	57.46	April 26, 2011
BH 4	62.69	5.80	56.89	April 26, 2011
BH 5 (MW)	62.15	2.17	59.98	July 11, 2011
BH 7	62.00	4.30	57.70	July 11, 2011
BH 8	62.49	4.73	57.76	July 11, 2011

Note: The ground surface elevation at each borehole was referenced to a temporary benchmark (TBM), consisting of a mag nail on the hydro pole located west of the subject site. A geodetic elevation of 63.14 m was provided for this TBM by Annis O'Sullivan Vollebekk.

5.0 DISCUSSION

5.1 Geotechnical Assessment

The subject site is considered adequate from a geotechnical perspective for the proposed building. It is expected that a temporary shoring system will be required for the excavation of the underground levels.

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

5.2 Site Grading and Preparation

Stripping Depth

It is understood that the proposed multi-storey building will be founded on the bedrock surface. It is anticipated that all existing overburden material will be excavated from within the footprint of the proposed building. Some bedrock excavation may be required for the construction of the underground parking garage.

Bedrock Removal

Based on the volume of the bedrock encountered in the area, it is expected that line-drilling in conjunction with hoe-ramming or controlled blasting will be required to remove the bedrock. 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, especially the multi-storey building to the west.

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 equipment could be the 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.

Horizontal Rock Anchors

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.

5.3 Foundation Design

Bearing Resistance Values

Footings placed on a clean, surface sounded limestone bedrock surface can be designed using a factored bearing resistance value at ultimate limit states (ULS) of **1,500 kPa**, incorporating a geotechnical resistance factor of 0.5, and a bearing resistance value at serviceability limit states (SLS) of **1,000 kPa**.

A clean, surface-sounded bedrock bearing surface should be free of loose materials, and have no near surface seams, voids, fissures or open joints which can be detected from surface sounding with a rock hammer.

A factored bearing resistance value at ULS of **4,000 kPa**, incorporating a geotechnical resistance factor of 0.5, and a bearing resistance at SLS of **2,000 kPa** could be used if founded on clean, surface sounded limestone bedrock and 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.

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 foundations placed on the limestone bedrock. Reference should be made to the latest revision of the 2006 Ontario Building Code for a full discussion of the earthquake design requirements. The seismic site classification will be confirmed in the spring by a site specific shear wave velocity testing.

5.5 Basement Slab

The basement area is expected to be mainly parking and a concrete slab topping with a subfloor granular layer will be incorporated in the design to accommodate services and a rigid pavement structure noted in Subsection 5.7 will be applicable. The thickness of the granular subfloor layer will be dependent on what services are incorporated in the design. It is also expected that a sump pit will be incorporated to drain any water which enters the granular layer via a breach in the raft slab.

In consideration of the groundwater conditions encountered at the time of the construction, 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.

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 drained unit weight of 20 kN/m^3.

Where the foundation walls are to be poured directly against the bedrock surface, a nominal coefficient of at-rest earth pressure of 0.25 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.

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

Static Earth Pressures

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

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

Seismic Earth Pressures

The seismic earth pressure (ΔP_{AE}) can be calculated using the earth pressure distribution equal to $0.375 a_c \gamma H^2/g$ 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 0.42g according to OBC 2006. Note that the vertical seismic coefficient is assumed to be zero.

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

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

The earth pressures calculated are unfactored. For the ULS case, the earth pressure loads should be factored as live loads, as per OBC 2006.

5.7 Rock Anchor Design

It is expected that rock anchors could be required to resist uplift forces. 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 is expected that the centre to centre spacing between the grid of rock anchors will be 3.0 m. Assuming an apex angle of 60° for the failure cone, it is likely that interaction will develop between failure cones of anchors. As a result, the following recommendations are provided on the assumption that group interaction will occur between the anchors. The effect of assuming group interaction is a reduction in the overall strength of each anchor; therefore, this assumption is considered conservative.

A third failure mode of shear failure along the grout/steel interface should also be reviewed by the structural engineer to ensure all typical failure modes have been reviewed.

It is also recommended, where applicable, that anchors in close proximity to each other be grouted at the same time. This will ensure that any fractures or voids are completely in-filled and that fluid grout does not flow from a grouted hole to an adjacent empty hole.

Anchors can be of the “passive” or the “post-tensioned” type, depending on whether the anchor tendon is provided with a post-tensioned load 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. In addition, each anchor should have an unbonded length, or free anchor length, between the rock surface and the start of the bonded length. Since 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 cement grout. The free anchor length is provided by installing a plastic sleeve to act as a bond break within the fully grouted drill hole.

Grout to Rock Bond

The unconfined compressive strength of limestone ranges between about 60 and 120 MPa, which is stronger than most routine grouts. 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 bedrock information, a **Rock Mass Rating (RMR) of 65** was assigned to the bedrock, and Hoek and Brown parameters (**m and s**) were taken as **0.575 and 0.00293**, respectively.

Recommended Rock Anchor Lengths

Parameters used to calculate rock anchor lengths are provided in Table 2.

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=0.575 and s=0.00293
Unconfined compressive strength - Limestone bedrock	60 MPa
Unit weight - Submerged Bedrock	15.2 kN/m^3
Apex angle of failure cone	60°
Apex of failure cone	mid-point of fixed anchor length

The fixed anchor length will depend on the diameter of the drill holes. Recommended anchor lengths for a 100 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	
100	1.0	0.8	1.8	250
	1.8	1.0	2.8	500
	2.5	1.3	3.8	800
	3.2	1.9	5.1	1200
125	0.7	1.1	1.8	250
	1.7	1.0	2.7	500
	2.5	1.5	3.5	800
	3.0	1.7	4.7	1200

It is recommended that the anchor drill hole diameter be a minimum of 2 times the rock anchor tendon diameter and the anchor drill holes be inspected by geotechnical personnel and should be thoroughly 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.

It should be noted that due to the intended use of the rock anchors and nature of the passive rock anchor design, proof testing is not required provided that the grout installation is adequately completed to the satisfaction of the geotechnical consultant. It is recommended that compressive strength testing be completed for the rock anchor grout. A set of grout cubes, consisting of 3 “gangs” of 3 cubes each, should be tested for each day that grout is prepared.

5.8 Pavement Design

The proposed lower basement slab will be considered a rigid pavement structure. The following rigid pavement structure is suggested to support car parking only.

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

Thickness (mm)	Material Description
125	**Wear Course -** Concrete slab
200	**BASE** - 20 mm clear stone
	SUBGRADE - Bedrock

Asphalt pavement is not anticipated to be required at the proposed development. However, should asphalt pavement be considered for the project, the recommended pavement structures shown in Tables 4 and 5 would be applicable.

Table 4 - 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 - Either fill, in situ soil or OPSS Granular B Type I or II material placed over in situ soil or fill

Table 5 - Recommended Flexible Pavement Structure - Access Lanes

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

If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with Ontario Provincial Standard Specification (OPSS) Granular B Type I or Type II material.

The pavement granulars (base and subbase) should be placed in maximum 300 mm thick layers and compacted to a minimum of 98% of the materials' SPMDD using suitable compaction equipment.

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

6.0 DESIGN AND CONSTRUCTION PRECAUTIONS

6.1 Foundation Drainage and Backfill

It is understood that the building foundation walls will be placed in close proximity to all the boundaries. It is expected that insufficient room will be available for exterior backfill along these walls and, therefore, the foundation wall will be poured against a drainage system placed against the shoring face.

It is recommended that the composite drainage system (such as Miradrain G100N or equivalent) extend down to the footing level. 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 an interior perimeter drainage pipe. The perimeter drainage pipe should direct water to sump pit(s) within the lower basement area.

Where space is available, it is recommended that a perimeter foundation drainage system be provided for the proposed structure. If provided the system should consist of a 100 mm to 150 mm diameter perforated corrugated plastic pipe, surrounded on all sides by 150 mm of 10 mm clear crushed stone, placed at the footing level around the exterior perimeter of the structure. The pipe should be connected to sump pit(s) within the lower basement area.

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 a composite drainage system (such as System Platon or Miradrain) is used. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should 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, or a minimum of 0.6 m of soil cover, in conjunction with foundation insulation, 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.

For a building provided with three to four underground parking levels, the requirement for frost protection can be reduced to 1 m in the lowest parking level in areas away from sources of below freezing air temperature such as but not limited to air intakes.

6.3 Excavation Side Slopes

At this site, temporary shoring will be required to complete the required excavations. 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. For preliminary design purposes, the temporary system may consist of soldier pile and lagging system or interlocking steel sheet piling.

6.4 Pipe Bedding and Backfill

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 should be low for the shallow excavations expected 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 MOE permit to take water (PTTW) will be required for this project if more than 50,000 L/day are to be pumped 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 MOE.

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.

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 an very aggressive to severely aggressive corrosive environment.

7.0 RECOMMENDATIONS

It is a requirement for the foundation design data provided herein to be applicable that a materials testing and observation services program including the following aspects be performed by the geotechnical consultant.

- ☐ Review of the geotechnical aspects of the excavating contractor’s shoring design, prior to construction.
- ☐ Review the bedrock stabilization and excavation 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 and follow-up 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 made in this report are preliminary in nature and in accordance with our present understanding of the project. A detailed investigation should be carried out to validate the recommendations presented in this report. We request that we be permitted to review our recommendations when the drawings and specifications are complete.

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, 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.

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Report Distribution:

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APPENDIX 1

SOIL PROFILE & TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TEST RESULTS

patersongroup Consulting Engineers

28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building - 1050 Somerset Street W.
Ottawa, Ontario

DATUM TBM - Mag nail in hydro pole located on west side of subject site. Assumed geodetic elevation = 63.136m, provided by Annis, O'Sullivan, Vollebekk Ltd.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 18 April 2011

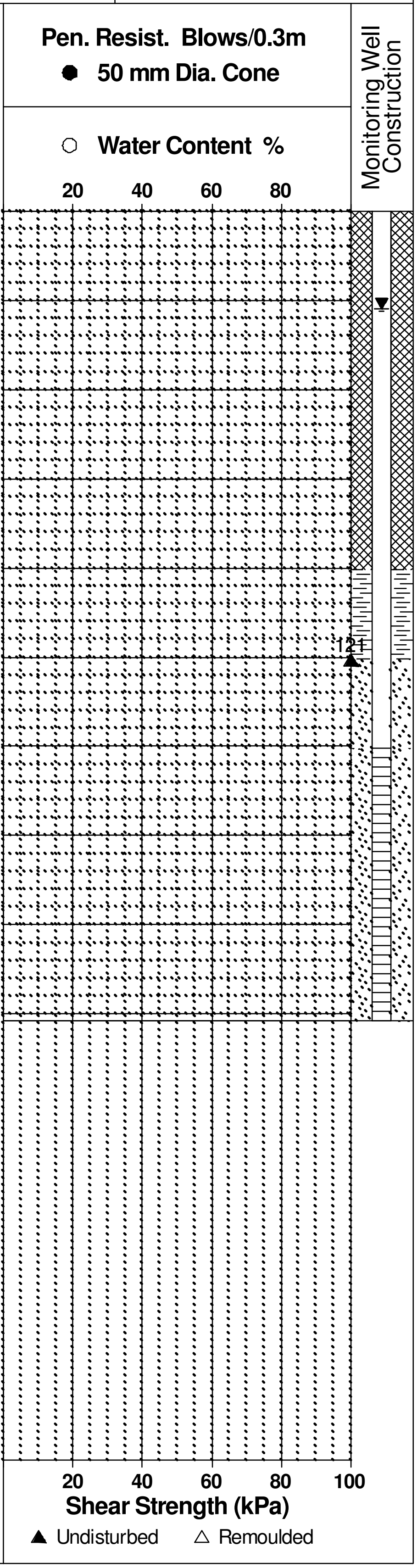
FILE NO. PG2356

HOLE NO. BH 1

SOIL DESCRIPTION	
GROUND SURFACE	
Asphaltic concrete	0.08
FILL: Brown silty sand, some crushed stone and gravel	0.20 0.69
FILL: Brown silty clay with sand and crushed stone	1.45
FILL: Brown silty sand, trace clay and gravel	
FILL: Brown silty sand with gravel and cobbles, trace concrete	2.97
- 600mm void encountered @ 2.3m depth	
Stiff to very stiff, brown **SILTY CLAY** - grey by 3.7m depth	5.26
GLACIAL TILL: Stiff, grey silty clay with sand, gravel, cobbles and boulders	7.31
GLACIAL TILL: Compact, grey silty sand with clay, gravel, cobbles and boulders	9.07
End of Borehole (GWL @ 1.10m-Apr. 26/11)	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	33	15
SS	3	42	5
SS	4	0	0
SS	5	92	2
SS	6	100	2
SS	7	100	3
SS	8	100	7
SS	9	75	23
SS	10	33	20
SS	11	33	17

DEPTH (m)	ELEV. (m)
0	62.44
1	61.44
2	60.44
3	59.44
4	58.44
5	57.44
6	56.44
7	55.44
8	54.44
9	53.44



patersongroup Consulting Engineers

28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building - 1050 Somerset Street W.
Ottawa, Ontario

DATUM TBM - Mag nail in hydro pole located on west side of subject site. Assumed geodetic elevation = 63.136m, provided by Annis, O'Sullivan, Vollebekk Ltd.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 18 April 2011

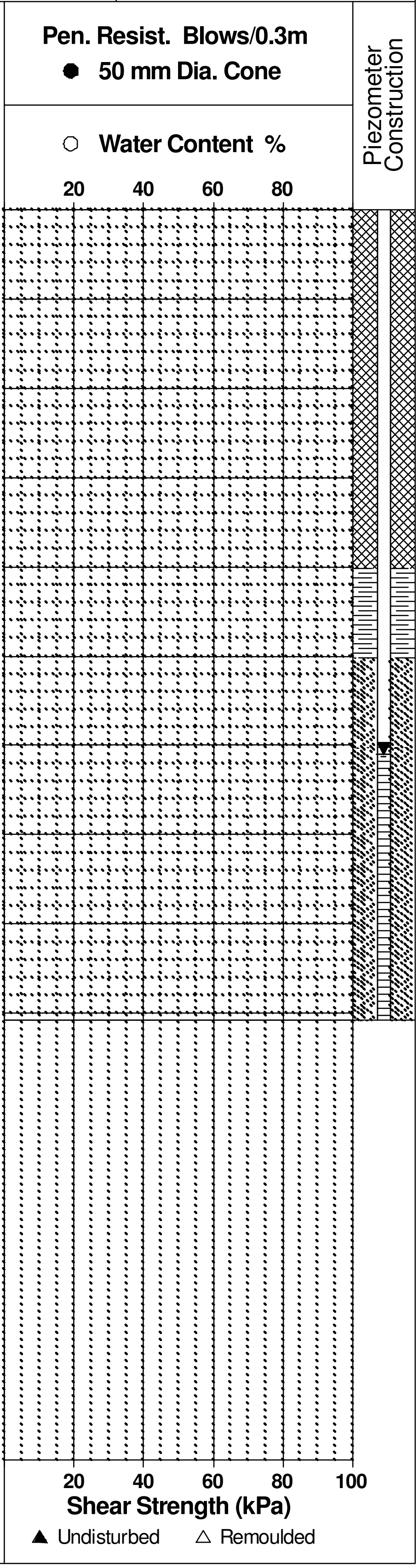
FILE NO. PG2356

HOLE NO. BH 2

SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
Asphaltic concrete	0.05
FILL: Brown silty sand with crushed stone and gravel	1.45
Very stiff, brown **SILTY CLAY** - firm by 3.7m depth	5.03
GLACIAL TILL: Stiff, grey silty clay with sand, gravel, cobbles and boulders	5.95
GLACIAL TILL: Compact, grey silty sand with clay, gravel, cobbles and boulders	9.07
End of Borehole (GWL @ 6.10m-Apr. 26/11)	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	25	35
SS	3	75	15
SS	4	67	13
SS	5	50	13
SS	6	100	4
SS	7	92	8
SS	8	50	8
SS	9	8	18
SS	10	50	10
SS	11	8	11
SS	12	8	4

DEPTH (m)	ELEV. (m)
0	62.56
1	61.56
2	60.56
3	59.56
4	58.56
5	57.56
6	56.56
7	55.56
8	54.56
9	53.56



patersongroup Consulting Engineers
28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building - 1050 Somerset Street W.
Ottawa, Ontario

DATUM TBM - Mag nail in hydro pole located on west side of subject site. Assumed geodetic elevation = 63.136m, provided by Annis, O'Sullivan, Vollebekk Ltd.

REMARKS

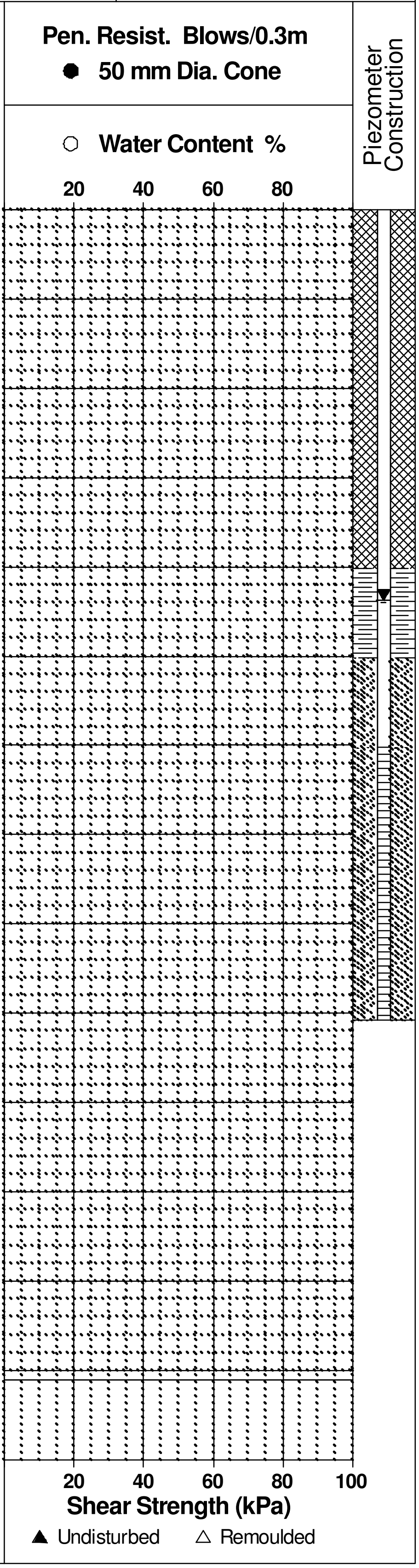
BORINGS BY CME 55 Power Auger

DATE 6 July 2011

FILE NO. PG2356

HOLE NO. BH 3

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	61.84
Asphaltic concrete 0.08	AU	1				
FILL: Brown silty sand with crushed stone, gravel, cobbles and boulders - trace clay by 1.45m depth	SS	2	17	63	1	60.84
	SS	3	25	46	2	59.84
2.97	SS	4	8	33		
Very stiff, brown **SILTY CLAY** 3.73	SS	5	100	10	3	58.84
GLACIAL TILL: Stiff, brown silty clay with sand, gravel, cobbles and boulders - grey by 4.5m depth	SS	6	75	12	4	57.84
	SS	7	33	11	5	56.84
	SS	8	12	13	6	55.84
6.78	SS	9	42	13		
GLACIAL TILL: Loose, grey silty sand with clay, gravel, cobbles and boulders	SS	10	17	9	7	54.84
	SS	11	25	3	8	53.84
	SS	12	50	8	9	52.84
10.97					10	51.84
BEDROCK: Grey limestone	RC	1	100	100	11	50.84
	RC	2	100	97	12	49.84
13.11					13	48.84
End of Borehole (GWL @ 4.38m-Apr. 26/11)						



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28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building - 1050 Somerset Street W.
Ottawa, Ontario

DATUM TBM - Mag nail in hydro pole located on west side of subject site. Assumed geodetic elevation = 63.136m, provided by Annis, O'Sullivan, Vollebekk Ltd.

REMARKS

BORINGS BY CME 55 Power Auger

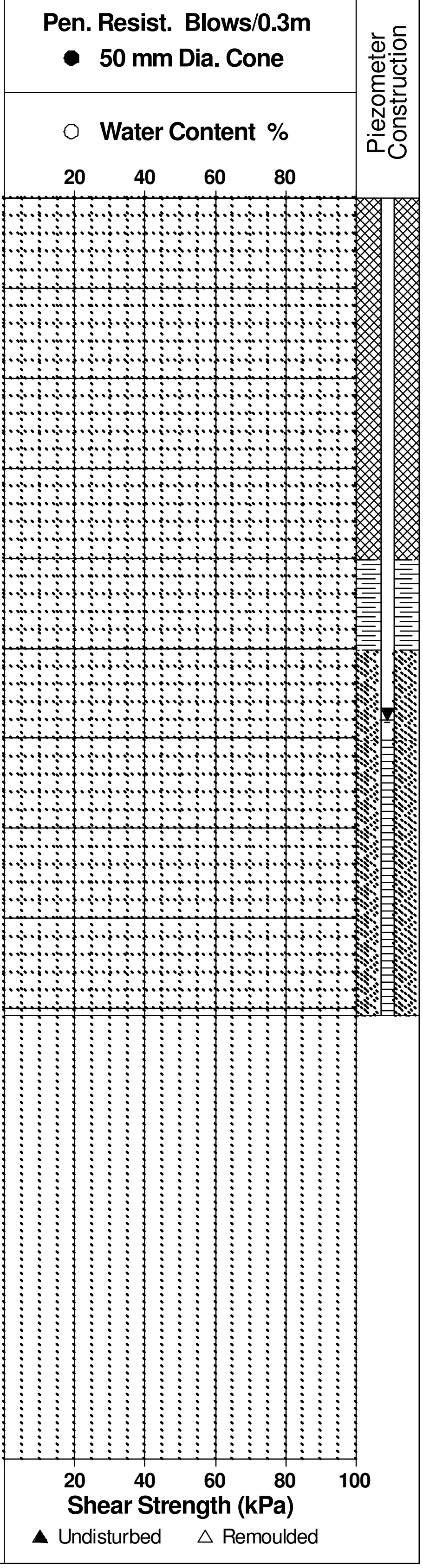
DATE 18 April 2011

FILE NO. PG2356

HOLE NO. BH 4

SOIL DESCRIPTION	Depth to base (m)	TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Asphaltic concrete	0.08				
FILL: Brown silty sand with crushed stone, gravel, cobbles and boulders	2.29	AU	1		
		SS	2	42	77
		SS	3	50	75
FILL: Grey-brown silty clay, trace sand and gravel	3.66	SS	4	25	17
		SS	5	42	7
Very stiff, brown **SILTY CLAY**	4.50	SS	6	67	11
GLACIAL TILL: Stiff, grey silty clay with sand, gravel, cobbles and boulders	6.78	SS	7	33	11
		SS	8	50	13
		SS	9	42	10
GLACIAL TILL: Loose to compact, grey silty sand with clay, gravel, cobbles and boulders	9.07	SS	10	21	7
		SS	11	42	10
		SS	12	58	18
End of Borehole					
(GWL @ 5.80m-Apr. 26/11)					

DEPTH (m)	ELEV. (m)
0	62.69
1	61.69
2	60.69
3	59.69
4	58.69
5	57.69
6	56.69
7	55.69
8	54.69
9	53.69



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28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building - 1050 Somerset Street W.
Ottawa, Ontario

DATUM TBM - Mag nail in hydro pole located on west side of subject site. Assumed geodetic elevation = 63.136m, provided by Annis, O'Sullivan, Vollebekk Ltd.

REMARKS

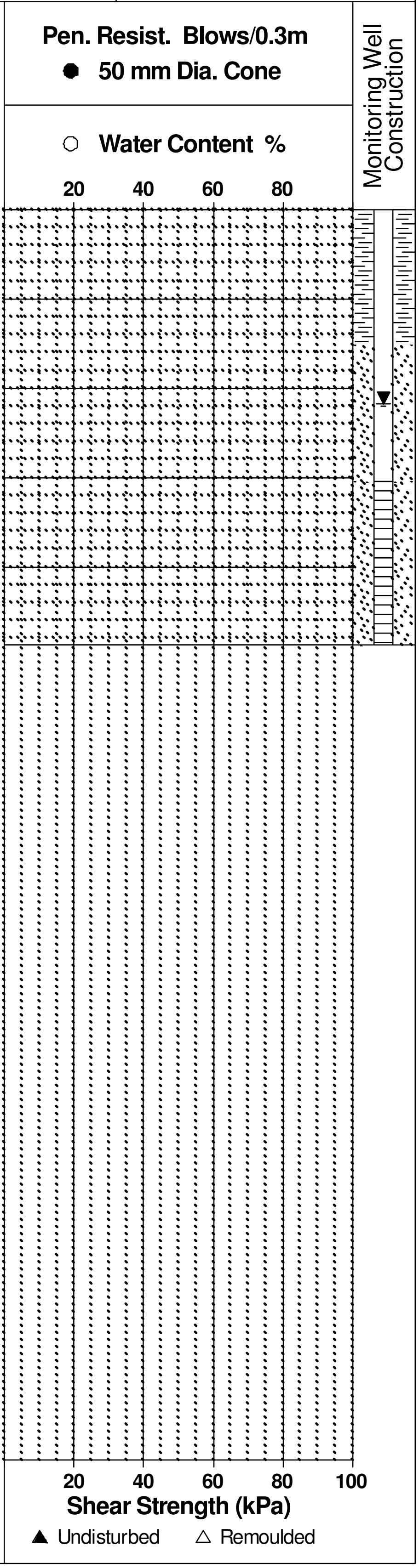
BORINGS BY Portable Drill

DATE 5 July 2011

FILE NO. PG2356

HOLE NO. BH 5

SOIL DESCRIPTION		SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	62.15
Concrete slab	0.15						
FILL: Crushed stone	0.60						
FILL: Brown silty sand with gravel	1.22	SS	1			1	61.15
Brown **SILTY CLAY**		SS	2	50			
- grey by 1.8m depth		SS	3	100		2	60.15
		SS	4	100		3	59.15
		SS	5	100			
		SS	6	0		4	58.15
	4.88	SS	7	100			
End of Borehole							
(GWL @ 2.17m-July 11/11)							



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28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building - 1050 Somerset Street W.
Ottawa, Ontario

DATUM TBM - Mag nail in hydro pole located on west side of subject site. Assumed geodetic elevation = 63.136m, provided by Annis, O'Sullivan, Vollebekk Ltd.

REMARKS

BORINGS BY Portable Drill

DATE 5 July 2011

FILE NO. PG2356

HOLE NO. BH 6

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content %	Piezometer Construction
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD			20 40 60 80	
GROUND SURFACE						0	58.80		
Concrete slab 0.15		AU	1						
FILL: Crushed stone 0.28									
Grey **SILTY SAND** with clay 0.30		SS	2	100		1	57.80		
GLACIAL TILL; Grey silty clay with sand, gravel, cobbles and boulders		SS	3	100					
2.44		SS	4	100		2	56.80		
End of Borehole									

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

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28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building - 1050 Somerset Street W.
Ottawa, Ontario

DATUM TBM - Mag nail in hydro pole located on west side of subject site. Assumed geodetic elevation = 63.136m, provided by Annis, O'Sullivan, Vollebekk Ltd.

REMARKS

BORINGS BY CME 55 Power Auger

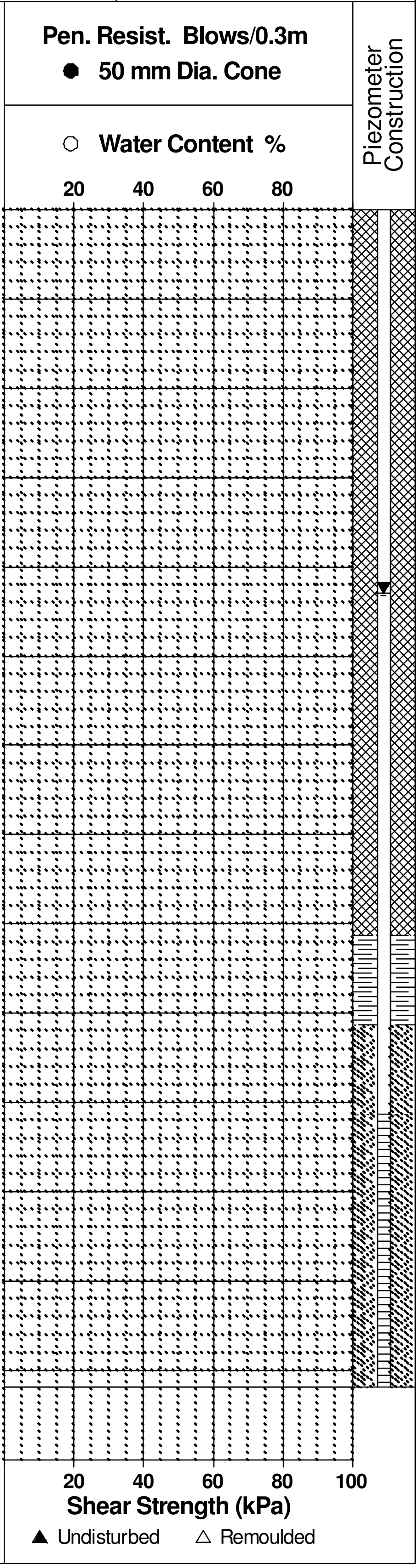
DATE 6 July 2011

FILE NO. PG2356

HOLE NO. BH 7

SOIL DESCRIPTION	Depth to base (m)	TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Asphaltic concrete	0.13	AU	1		
FILL: Crushed stone	0.25	AU	2		
FILL: Brown silty sand with gravel, wood chips, trace brick	0.60	SS	3	50	16
Stiff, brown **SILTY CLAY** - trace sand by 2.2m depth	4.50	SS SS SS SS	4 5 6 7	50 67 92 33	19 6 7 40
GLACIAL TILL: Stiff to firm, grey silty clay with sand, gravel, cobbles and boulders	11.28	SS SS SS	8 9 10	25 50 75	12 12 7
BEDROCK: Grey limestone	13.18	RC RC	1 2	100 100	100 98
End of Borehole (GWL @ 4.30m-July 11/11)					

DEPTH (m)	ELEV. (m)
0	62.00
1	61.00
2	60.00
3	59.00
4	58.00
5	57.00
6	56.00
7	55.00
8	54.00
9	53.00
10	52.00
11	51.00
12	50.00
13	49.00



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building - 1050 Somerset Street W.
Ottawa, Ontario

DATUM TBM - Mag nail in hydro pole located on west side of subject site. Assumed geodetic elevation = 63.136m, provided by Annis, O'Sullivan, Vollebekk Ltd.

REMARKS

BORINGS BY CME 55 Power Auger

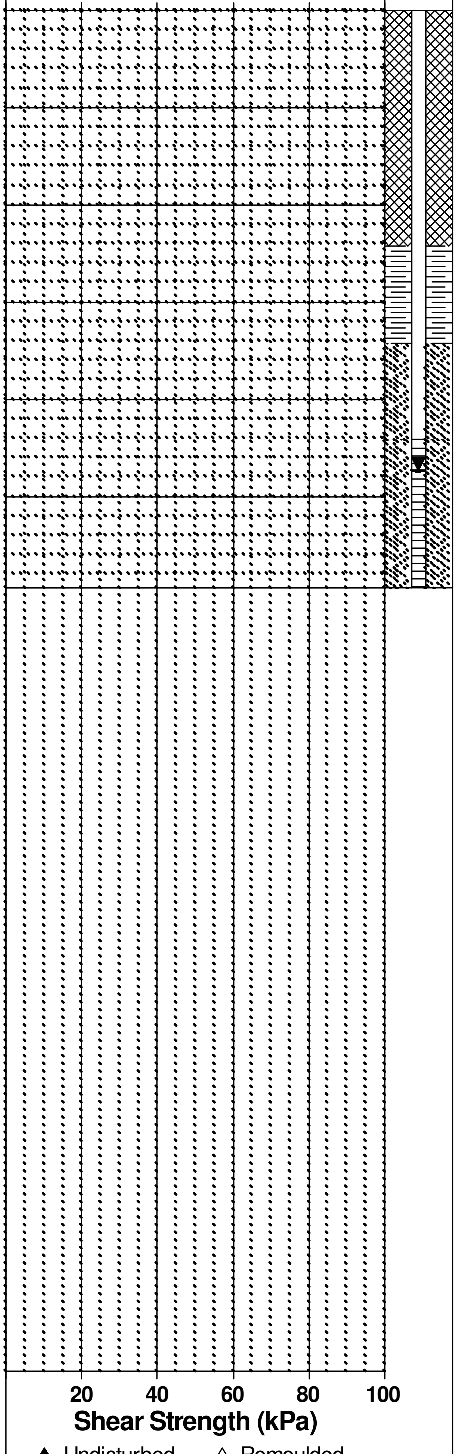
DATE 6 July 2011

FILE NO. PG2356

HOLE NO. BH 8

SOIL DESCRIPTION		TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Asphaltic concrete	0.05	AU	1		
FILL: Crushed stone, trace sand	0.30	AU	2		
FILL: Brown silty sand with crushed stone		SS	3	67	50+
- cobbles and boulders by 2.2m depth	1.83	SS	4	50	50+
Brown **CLAYEY SILT,** some sand	2.97	SS	5	92	16
Brown **SILTY CLAY**	3.73	SS	6	92	5
GLACIAL TILL: Stiff, grey silty clay with sand, gravel, cobbles and boulders	5.26	SS SS	7 8	50 92	31 6
GLACIAL TILL: Compact, grey silty sand with clay, gravel, cobbles and boulders	5.94	SS	9	21	16
End of Borehole					
(GWL @ 4.73m-July 11/11)					

DEPTH (m)	ELEV. (m)
0	62.49
1	61.49
2	60.49
3	59.49
4	58.49
5	57.49



SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

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

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

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

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

SOIL DESCRIPTION (continued)

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

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

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

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

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

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

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

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

CONSOLIDATION TEST

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

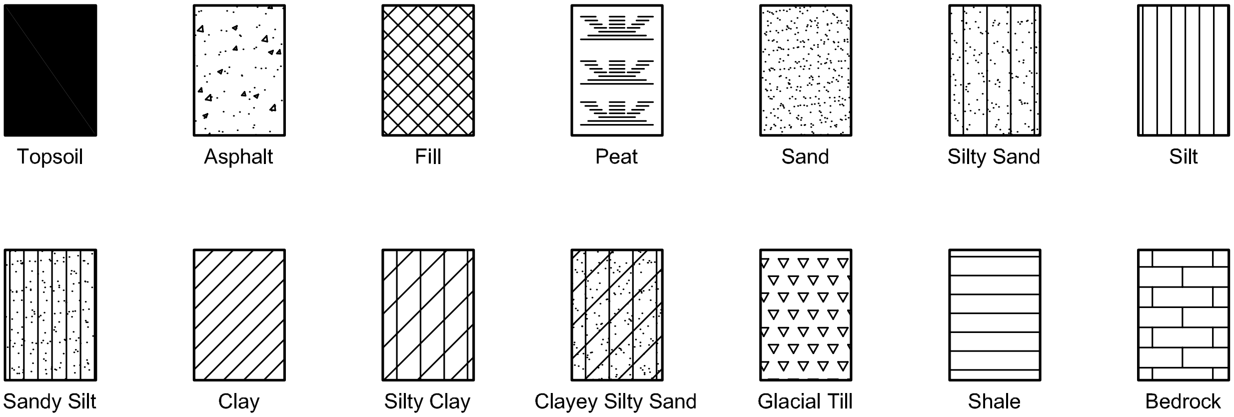
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

PERMEABILITY TEST

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

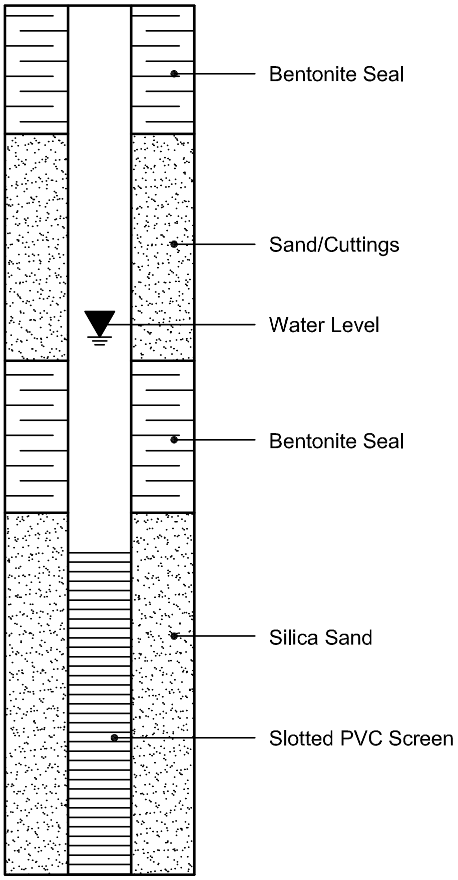
SYMBOLS AND TERMS (continued)

STRATA PLOT

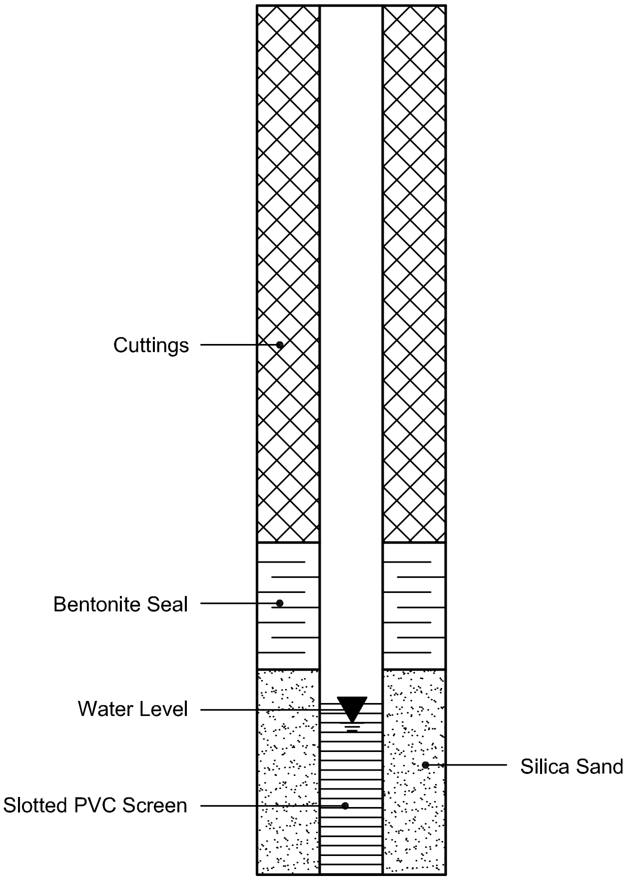


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Client: **Paterson Group Consulting Engineers**

Client PO: 10686

Report Date: 04-May-2011

Order Date:28-Apr-2011

Project Description: PG2356

	Client ID:	BH2 SS5	-	-	-
	Sample Date:	18-Apr-11	-	-	-
	Sample ID:	1118150-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	69.9	-	-	-
General Inorganics					
pH	0.05 pH Units	7.26	-	-	-
Resistivity	0.10 Ohm.m	18.8	-	-	-
Anions					
Chloride	5 ug/g dry	165	-	-	-
Sulphate	5 ug/g dry	177	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING PG2356-1 - TEST HOLE LOCATION PLAN

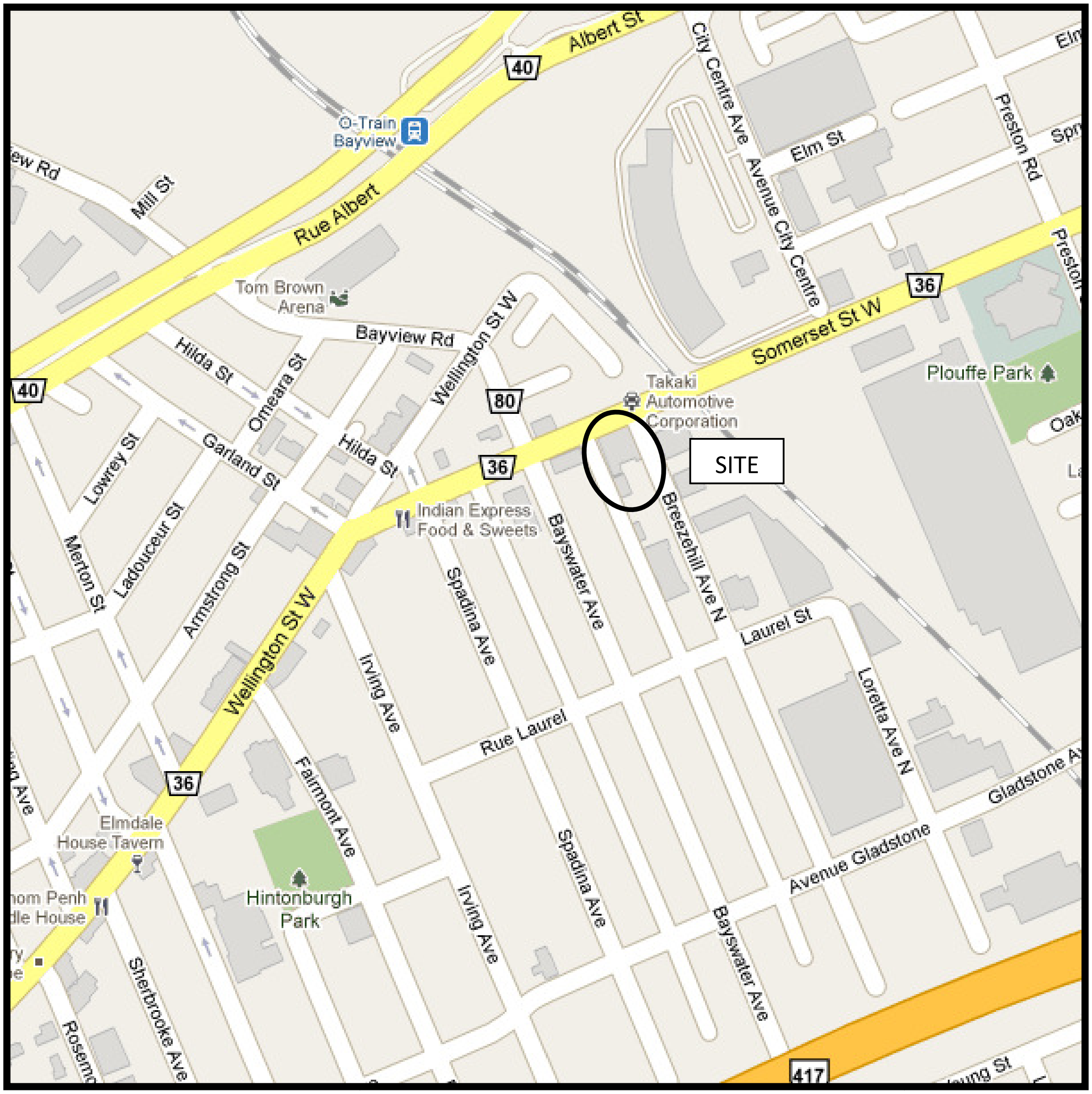
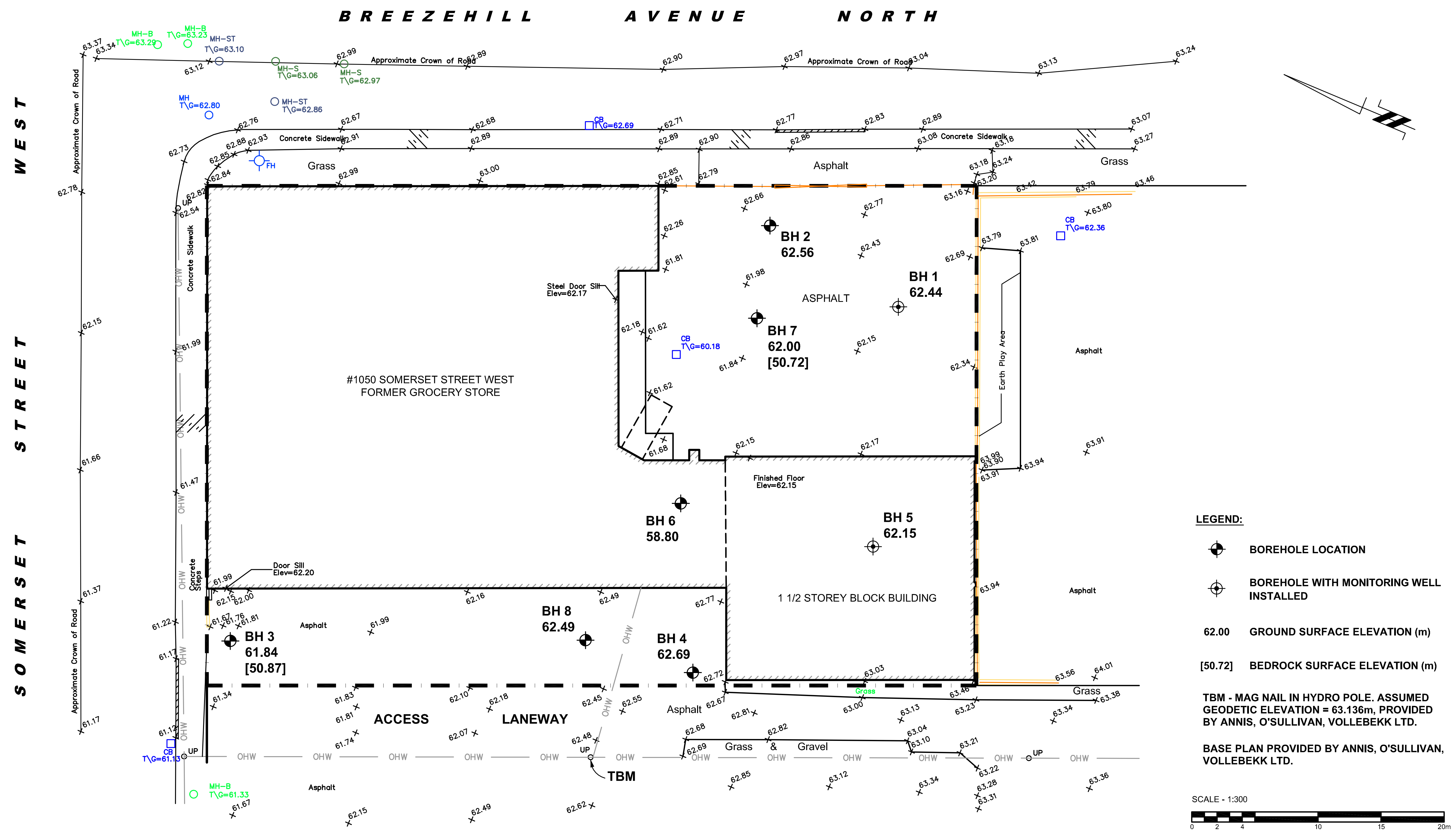


FIGURE 1

KEY PLAN



LEGEND:

⌖	BOREHOLE LOCATION
⊕	BOREHOLE WITH MONITORING WELL INSTALLED
62.00	GROUND SURFACE ELEVATION (m)
[50.72]	BEDROCK SURFACE ELEVATION (m)

TBM - MAG NAIL IN HYDRO POLE. ASSUMED GEODETIC ELEVATION = 63.136m, PROVIDED BY ANNIS, O'SULLIVAN, VOLLEBEKK LTD.

BASE PLAN PROVIDED BY ANNIS, O'SULLIVAN, VOLLEBEKK LTD.

SCALE - 1:300

0 2 4 10 15 20m

patersongroup
consulting engineers
28 Concourse Gate, Unit 1, Ottawa, Ontario K2E 7T7

Scale:	1:300
Des.:	RG
Dwn:	MPG
Chkd:	DG

CLARIDGE HOMES

GEOTECHNICAL INVESTIGATION
PROP. MULTI-STOREY BUILDING-1050 SOMERSET ST. W.
OTTAWA, ONTARIO

TEST HOLE LOCATION PLAN

Dwg. No.	**PG2356-1**
Report No.:	PG2356-1
Date:	05/2011

11x17