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Geotechnical
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Materials Testing

Building Science

Geotechnical Investigation
Proposed Multi-Storey Building
175 Richmond Road
Ottawa, Ontario

Prepared For

Claridge Homes

September 29, 2011

Report PG2363-1

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APPENDICES

1.0 INTRODUCTION

Paterson Group (Paterson) was commissioned by Claridge Homes to conduct a geotechnical investigation for the proposed multi-storey building to be located at 175 Richmond Road, in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objectives of the investigation were to:

- ☐ Determine the subsoil and groundwater conditions at this site by means of boreholes.
- ☐ Provide geotechnical recommendations for the design of the proposed development including construction considerations which may affect the design.

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

2.0 PROPOSED PROJECT

It is understood that the proposed development will consist of a multi-storey residential building with two (2) to three (3) levels of underground parking covering the majority of the subject site.

3.0 METHOD OF INVESTIGATION

3.1 Field Investigation

Field Program

The field program for the investigation was carried out on May 30, 2011 and June 8, 2011. At that time, seven (7) boreholes were advanced to practical refusal to augering at a sampling depth of 3.4 to 7.3 m. A previous investigation was completed by Paterson for the subject site on October 14, 2009. The boreholes from the previous investigation are presented in Appendix 1. The test hole locations were selected in a manner to provide general coverage of the proposed development. The borehole locations are shown on Drawing PG2363-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were put down using a truck-mounted power auger drill rig, operated by a two-person crew. All fieldwork was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer from our geotechnical division. The drilling procedure consisted of augering to the required depths at the selected locations, sampling and testing the overburden.

Sampling and In-Situ Testing

Soil samples were 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 soil 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.

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

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

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

Diamond drilling was carried out at BH 6-11 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.

Groundwater

Monitoring wells were installed at BH 4-11, BH 6-11 and BH 7-11, while flexible polyethylene standpipes were installed in all other boreholes to permit the monitoring of groundwater levels subsequent to the completion of the sampling program.

3.2 Field Survey

The borehole locations were selected, determined in the field and surveyed by Paterson. The ground surface elevation at each borehole location was referenced to a temporary benchmark (TBM), consisting of the top of spindle of fire hydrant located on the southwest corner of Wilber Avenue and Clifton Road. An assumed elevation of 100 m was assigned to the TBM. The location and ground surface elevations at borehole locations are presented on Drawing PG2363-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

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

3.4 Analytical Testing

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

4.0 OBSERVATIONS

4.1 Surface Conditions

The subject site is currently occupied by a two (2) storey commercial building with an associated asphaltic concrete paved parking area. The ground surface at the subject site is sloping downward to the north and at grade with surrounding streets. The site is bordered to the north by residential dwellings, to the west by a commercial building, to the south by Richmond Road and to the east by Kirkwood Avenue.

4.2 Subsurface Profile

The subsurface profile encountered at the borehole locations, consists of a pavement structure underlain by a loose silty sand and/or a compact to dense glacial till deposit. Practical refusal to auguring was encountered at all borehole locations at depths ranging from 3.5 to 7.3 m. A grey limestone bedrock was cored at BH 6-11. Based on the RQD values, the bedrock is of fair quality. Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for the details of the soil profiles encountered at each borehole location.

4.3 Groundwater

Groundwater levels were measured in the standpipes on June 20, 2011 and the results are presented in Table 1. The groundwater table fluctuates throughout the year. Therefore, the groundwater level could vary at the time of construction.

Table 1 - Measured Groundwater Levels

Test Hole Number	Ground Surface Elevation (m)	Water Level	
		Depth (m)	Elevation (m)
BH 1-11	102.71	4.11	98.60
BH 2-11	99.31	2.15	97.16
BH 3-11	99.63	2.04	97.59
BH 4-11	99.60	2.33	97.27
BH 5-11	99.46	2.68	96.78
BH 6-11	99.27	2.59	96.68
BH 7-11	99.56	2.43	97.13

5.0 DISCUSSION

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered suitable for the proposed multi-storey building.

Bedrock excavation is expected for the construction of the basement levels of the proposed building. Line drilling of the perimeter and rock blasting and/or pneumatic breaking operations are expected for the removal of the bedrock.

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 at the subject site and the anticipated founding level for the proposed multi-storey building, it is anticipated that all existing overburden material will be excavated from within the footprint of the proposed multi-storey building.

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 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.

5.3 Foundation Design

Bearing Resistance Values

Based on the subsurface profile encountered, it is expected that a limestone bedrock will be encountered at the founding levels.

Footings placed over a clean, surface sounded bedrock surface can be designed using 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. The bedrock will be 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.

Footings bearing on surface sounded bedrock and designed using the above mentioned bearing pressures will be subjected to negligible post-construction total and differential settlements.

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).

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class C** 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. A higher site class, such as Class A or B may be applicable for the proposed building. However, the higher site classes have to be confirmed by site specific shear wave velocity testing. The soils underlying the site are not susceptible to liquefaction.

5.5 Basement Slab

It is expected that the basement area will be mostly parking and that 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.

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

It is expected that the basement walls are to be poured against a composite drainage blanket, which will be placed against the exposed bedrock face. 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. 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 drained unit weight of 20 kN/m^3. It is anticipated that the soils against the foundation wall will be drained. An interface friction angle of 17 degrees between the wall and the backfill material is applicable for the abovenoted parameters. For undrained conditions, the effective unit weight of soil (13 kN/m^3) should be used to calculate the earth pressure component below the groundwater table, and hydrostatic pressure should be added within this portion to calculate the total static earth pressure.

The earth pressures acting on earth retaining structures are dependent on the characteristics of the structure, particularly with respect to whether it is a “yielding” or an “unyielding” structure. A basement wall, which is restrained laterally by the floors of the structure, is generally considered to be an unyielding structure. It is recommended that the at-rest earth pressure case be used for basement walls under static conditions. During an earthquake event, a basement wall is considered to be a “yielding” earth retaining structure, due to the magnitude of wall rotation. Therefore, an active earth pressure should be calculated for seismic design considerations.

Two (2) distinct conditions, static and seismic, must be reviewed for design calculations. The parameters for design calculations for the two (2) conditions are presented below.

Static Earth Pressures

Under static conditions, the retaining walls and basement walls may be designed using a triangular earth pressure distribution with a maximum stress value at the base of the wall equal to $K_o \gamma H$ where:

K_o - At-rest earth pressure coefficient = 0.5
γ - unit weight of the fill
H - height of the retained fill against the wall, m

An additional pressure having a magnitude equal to $K_o q$ and acting on the entire height of the wall must be added to the above diagram for any surcharge loading, q (kPa), that may be placed at ground surface adjacent to the wall.

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

Seismic Earth Pressures

Seismic loading conditions influence the earth pressures that will act on earth retaining structures during seismic events. In Ottawa, the peak ground acceleration (PGA) is 0.42 for the OBC 2006.

The magnitude of seismic earth pressures acting on a structure is dependent upon the relative flexibility of the structure. Isolated free-standing retaining walls are generally flexible enough to be considered as “yielding” earth retaining structures. During an earthquake event, a basement wall is considered to be a “yielding” earth retaining structure, due to the magnitude of wall rotation.

The total active earth force acting on a wall under seismic conditions can be estimated using a pseudo-static approach based on the Mononobe-Okabe (M-O) Method. The seismic intensity is represented by the horizontal seismic coefficient, k_h. For yielding structures, the value of k_h can be taken to be one half of PGA. Note that the vertical seismic coefficient is taken to be zero.

The M-O Method is used to calculate the total active earth pressure (P_{AE}). The resulting force is then split into the static (active) (P_A) and seismic component (ΔP_{AE}). The total active earth pressure (P_{AE}) can be calculated using $0.5K_{AE}\ \gamma H^2$ where:

K_{AE} - Dynamic active earth pressure coefficient. For the conditions previously stated, K_{AE} is 0.21.
γ - unit weight of the fill of the applicable retained soil (kN/m^3)
H - height of the wall (m)

The static component (P_A) can be calculated using $K_A\ \gamma\ H$ where:

K_A = active earth pressure coefficient, 0.33
γ = unit weight of the fill of the applicable retained soil (kN/m^3)
H = height of the wall (m)

The dynamic seismic component (ΔP_{AE}) can be calculated by $\Delta P_{AE} = P_{AE} - P_A$.

The static component (P_A) is a conventional triangular shaped pressure distribution with the resultant located H/3 up from the wall base. The seismic component (ΔP_{AE}) is acting approximately 0.6H up from the wall base.

On this basis, the total active pressure (P_{AE}) will act from a height:

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

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

5.7 Pavement Structure

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
150	**BASE** - 20 mm clear stone
	SUBGRADE - Bedrock

Asphalt pavement is not anticipated to be required at the subject site. However, should a flexible pavement be reconsidered for the project, the recommended flexible pavement structures shown in Tables 3 and 4 would be applicable.

Table 3 - Recommended Flexible Pavement Structure - Car Only Parking Areas

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

Table 4 - Recommended Flexible Pavement Structure - Access Ramp

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

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.

5.8 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. 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) or Williams Form Engineering, have qualified personnel on staff to recommend appropriate rock anchor size and materials.

The centre to centre spacing between bond lengths should be at least four (4) times the anchor hole diameter and greater than 1.2 m to lower the group influence effects. Anchors in close proximity to each other is recommended to be grouted at the same time to ensure any fractures or voids are completely in-filled and grout fluid 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. To resist seismic uplift pressures, a passive rock anchor system can be used. A post-tensioned anchor will take the uplift load with less deflection than a passive anchor.

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, the entire drill hole is to 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

Generally, 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. A **Rock Mass Rating (RMR) of 44** was assigned to the bedrock, and Hoek and Brown parameters (**m and s**) were taken as **0.128 and 0.00009**, respectively. For design purposes, we assumed that all rock anchors will be placed at least 1.2 m apart to reduce group anchor effects.

Recommended Rock Anchor Lengths

Parameters used to calculate rock anchor lengths are provided in Table 5.

Table 5 - Parameters used in Rock Anchor Review	
Grout to Rock Bond Strength - Factored at ULS	1.2 MPa
Compressive Strength - Grout	40 MPa
Rock Mass Rating (RMR) - Fair quality Limestone - Hoek and Brown parameters	44 m=0.128 and s=0.00009
Unconfined compressive strength - Limestone	60 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

The fixed anchor length will depend on the diameter of the drill holes. Recommended anchor lengths for 75 mm and 125 mm diameter holes are provided in Table 6. The factored tensile resistance values given in Table 6 are based on a single anchor with no group influence effects.

Table 6 - 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	2	1.7	3.7	250
	3	2.3	5.3	500
	4	2.6	6.6	750
125	1.7	1.8	3.5	250
	2.3	2.6	4.9	500
	3	3.1	6.1	750

Other considerations

The anchor drill holes should be inspected by geotechnical personnel and should be flushed clean prior to grouting. The use of grout tube to place grout from the bottom up in the anchor holes is 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.

6.0 DESIGN AND CONSTRUCTION PRECAUTIONS

6.1 Foundation Drainage and Backfill

Foundation Drainage

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 or bedrock.

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.

Underfloor Drainage

Underfloor drainage may be required to control water infiltration due to groundwater lowering within the bedrock. For design purposes, we recommend that 100 or 150 mm in perforated pipes be placed at 3 to 4.5 m centres. The spacing of the underfloor drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

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

It is expected that the parking garage will not require protection against frost action due to the founding depth. Unheated structures such as the access ramp may required to be insulated against the deleterious effect of frost action. A minimum of 2.1 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.

6.3 Excavation Side Slopes and Temporary Shoring

Excavation Side Slopes

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. However, for most of the site, insufficient room will be available to permit the building excavation to be undertaken by open-cut methods (i.e. unsupported excavations).

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.

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 for 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.

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.

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

Table 7 - Soil Parameters

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

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.

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

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 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.

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.

6.7 Corrosion Potential and Sulphate

The results of analytical testing show that the sulphate content is less than 2%. 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 aggressive corrosive environment.

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 excavating contractor's shoring design, prior to construction.

☐ 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 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 altered use of the report.

Paterson Group Inc.

David J. Gilbert, P.Eng.

LICENSED PROFESSIONAL ENGINEER
D. J. GILBERT
100116130
PROVINCE OF ONTARIO

Carlos P. Da Silva, P.Eng.

Report Distribution:

❏ Claridge Homes (5 copies)
❏ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TEST RESULTS

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development-175 Richmond Road
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located on the southwest corner of Wilber Avenue and Clifton Road. An arbitrary elevation of 100.00m was assigned to the TBM.

REMARKS

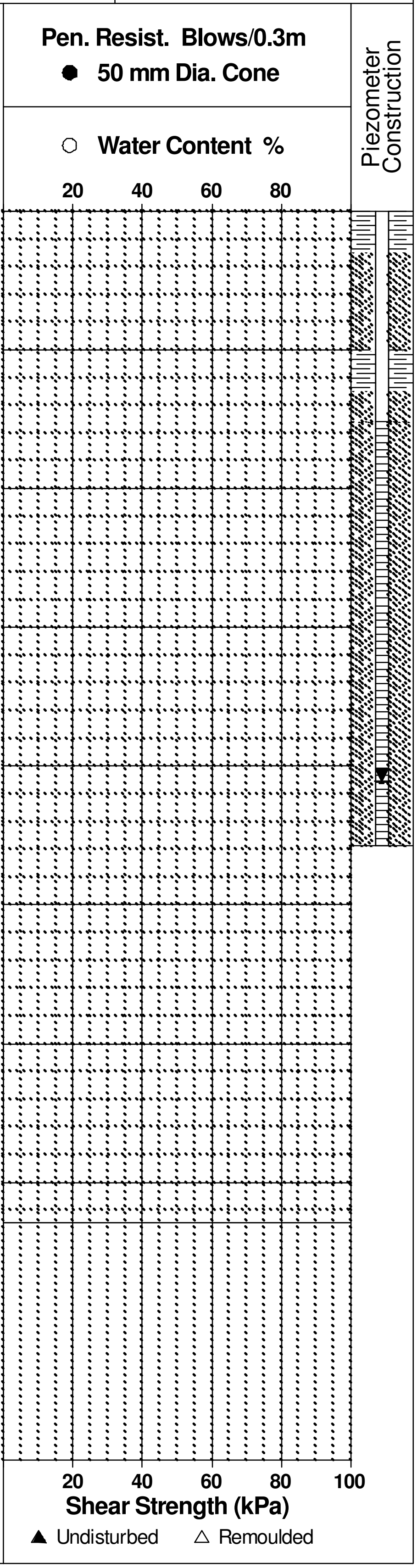
BORINGS BY CME 55 Power Auger

DATE 30 May 2011

FILE NO. PG2363

HOLE NO. BH 1-11

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	102.71
						1	101.71
						2	100.71
Inferred **OVERBURDEN**							
						3	99.71
						4	98.71
		SS	1	42	3		
4.57							
GLACIAL TILL: Grey silty sand with clay, gravel, cobbles and boulders	^^^	SS	2	50	19		
	^^^					5	97.71
	^^^	SS	3	54	20		
	^^^					6	96.71
	^^^	SS	4	52	26		
	^^^					7	95.71
7.29							
End of Borehole Practical refusal to augering @ 7.29m depth (GWL @ 4.11m-June 20/11)							



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

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development-175 Richmond Road
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located on the southwest corner of Wilber Avenue and Clifton Road. An arbitrary elevation of 100.00m was assigned to the TBM.

REMARKS

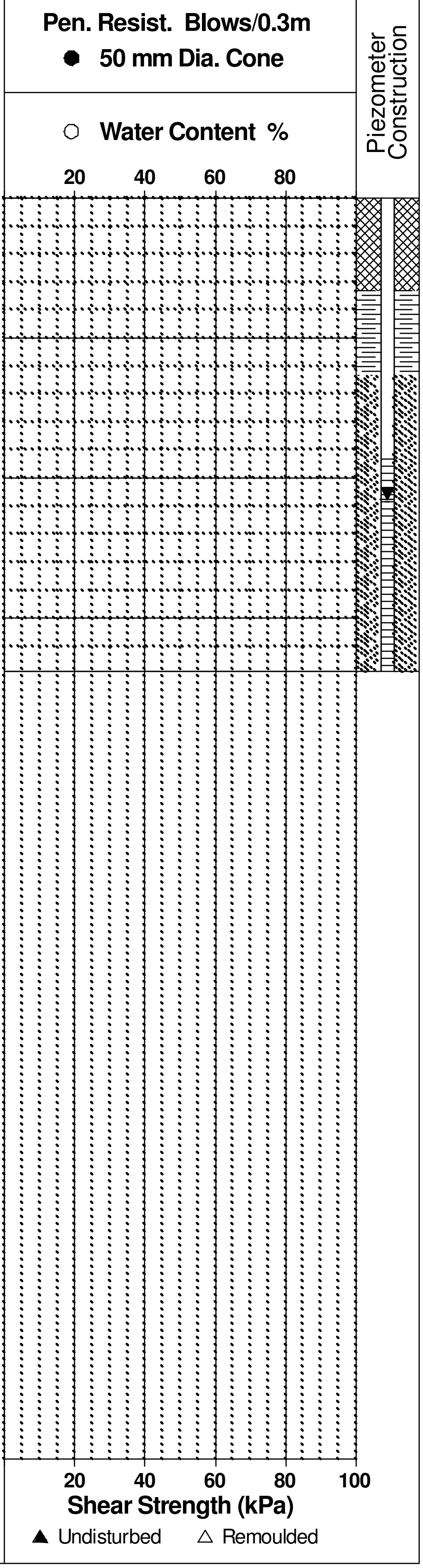
BORINGS BY CME 55 Power Auger

DATE 30 May 2011

FILE NO. PG2363

HOLE NO. BH 2-11

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	99.31
Inferred **OVERBURDEN**						1	98.31
						2	97.31
2.90							
GLACIAL TILL: Grey silty sand with gravel and cobbles						3	96.31
3.38							
End of Borehole Practical refusal to augering @ 3.38m depth (GWL @ 2.15m-June 20/11)							



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development-175 Richmond Road
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located on the southwest corner of Wilber Avenue and Clifton Road. An arbitrary elevation of 100.00m was assigned to the TBM.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 8 June 2011

FILE NO. PG2363

HOLE NO. BH 3-11

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	99.63
Asphaltic concrete 0.08						
FILL: Crushed stone 0.66	AU	1				
FILL: Brown silty sand	SS	2	58	5	1	98.63
	SS	3	25	1	2	97.63
2.97	SS	4	33	1		
GLACIAL TILL: Compact to dense, grey silty sand with gravel, cobbles and boulders	SS	5	67	24	3	96.63
4.04	SS	6		50+	4	95.63

End of Borehole

Practical refusal to augering @ 4.04m depth

(GWL @ 2.04m-June 20/11)

STRATA PLOT

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

○ Water Content %

20 40 60 80

Piezometer Construction

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

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development-175 Richmond Road
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located on the southwest corner of Wilber Avenue and Clifton Road. An arbitrary elevation of 100.00m was assigned to the TBM.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 8 June 2011

FILE NO. PG2363

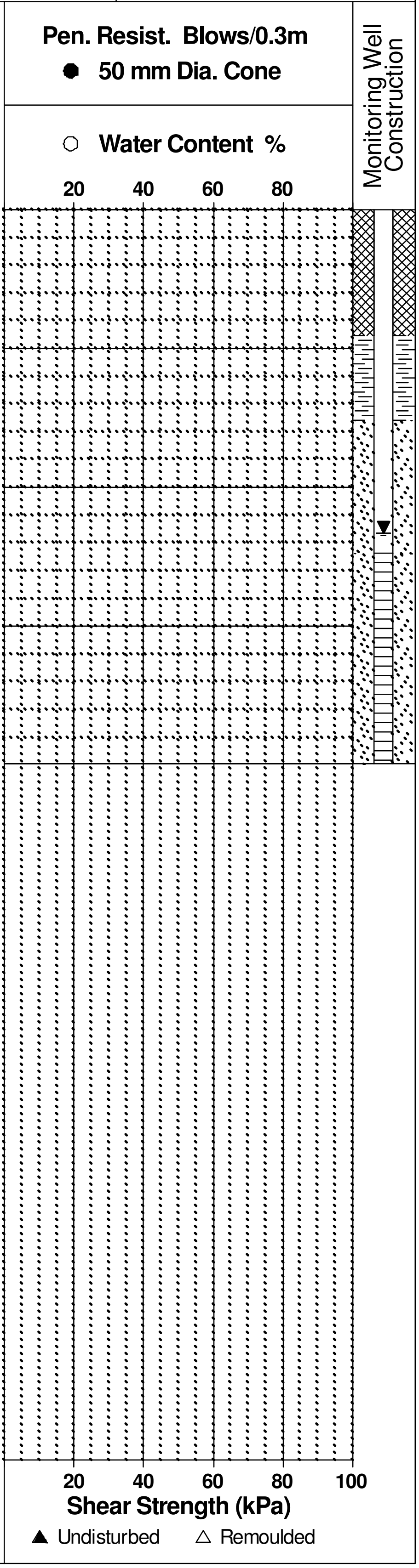
HOLE NO. BH 4-11

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	99.60
Asphaltic concrete 0.08	AU	1				
FILL: Crushed stone 0.30	AU	2				
	SS	3	75	15	1	98.60
GLACIAL TILL: Compact to dense, brown silty sand with gravel, cobbles and boulders	SS	4	67	19		
					2	97.60
- grey by 2.2m depth	SS	5	58	23		
					3	96.60
	SS	6	54	32		
3.99	SS	7		50+		

End of Borehole

Practical refusal to augering @ 3.99m depth

(GWL @ 2.33m-June 20/11)



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

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development-175 Richmond Road
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located on the southwest corner of Wilber Avenue and Clifton Road. An arbitrary elevation of 100.00m was assigned to the TBM.

REMARKS

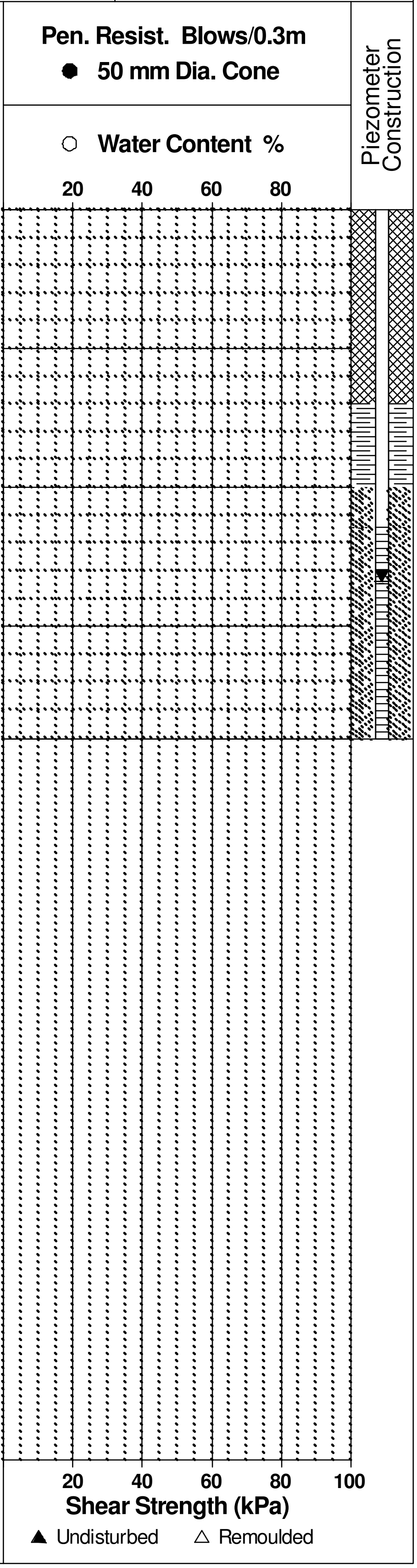
BORINGS BY CME 55 Power Auger

DATE 8 June 2011

FILE NO. PG2363

HOLE NO. BH 5-11

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	99.46
Asphaltic concrete 0.10							
FILL: Crushed stone 0.28		AU	1				
		SS	2	58	19	1	98.46
GLACIAL TILL: Compact to dense, brown silty sand with gravel, cobbles and boulders - grey by 2.2m depth		SS	3	33	92	2	97.46
		SS	4	67	28	3	96.46
		SS	5	71	50+		
3.81							
End of Borehole Practical refusal to augering @ 3.81m depth (GWL @ 2.68m-June 20/111)							



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development-175 Richmond Road
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located on the southwest corner of Wilber Avenue and Clifton Road. An arbitrary elevation of 100.00m was assigned to the TBM.

REMARKS

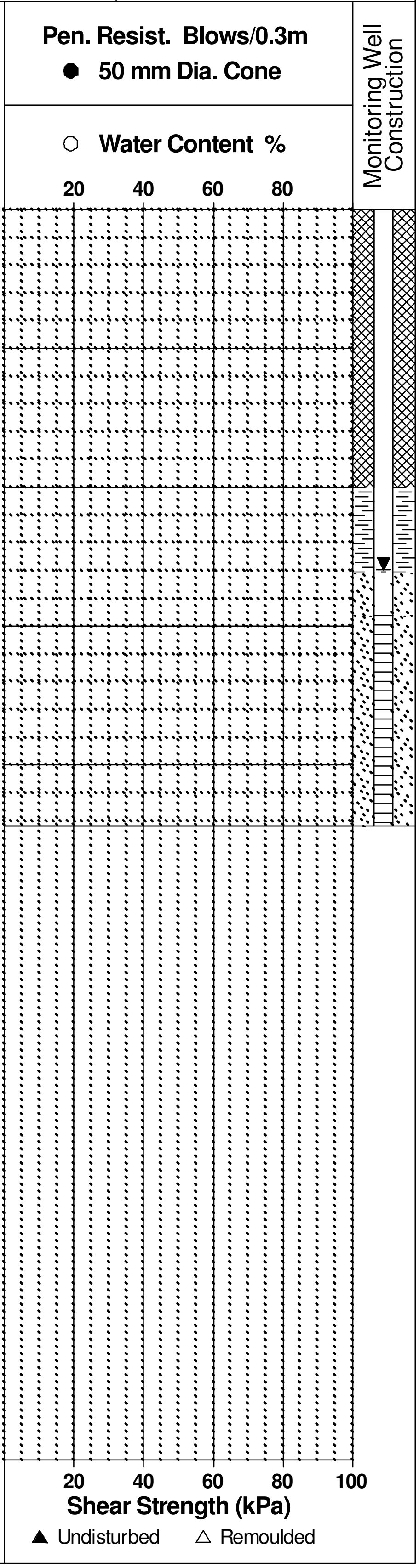
BORINGS BY CME 55 Power Auger

DATE 8 June 2011

FILE NO. PG2363

HOLE NO. BH 6-11

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	99.27
Asphaltic concrete 0.05	AU	1				
FILL: Brown silty sand with gravel 0.30	AU	2				
GLACIAL TILL: Compact, brown silty sand with gravel, cobbles and boulders - grey by 2.2m depth	SS	3	50	17	1	98.27
	SS	4	75	25	2	97.27
	SS	5	25	27	3	96.27
3.61	SS	6	36	26		
BEDROCK: Grey limestone	RC	1	52	55	4	95.27
4.44						
End of Borehole (GWL @ 2.59m-June 20/11)						



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development-175 Richmond Road
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located on the southwest corner of Wilber Avenue and Clifton Road. An arbitrary elevation of 100.00m was assigned to the TBM.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 8 June 2011

FILE NO. PG2363

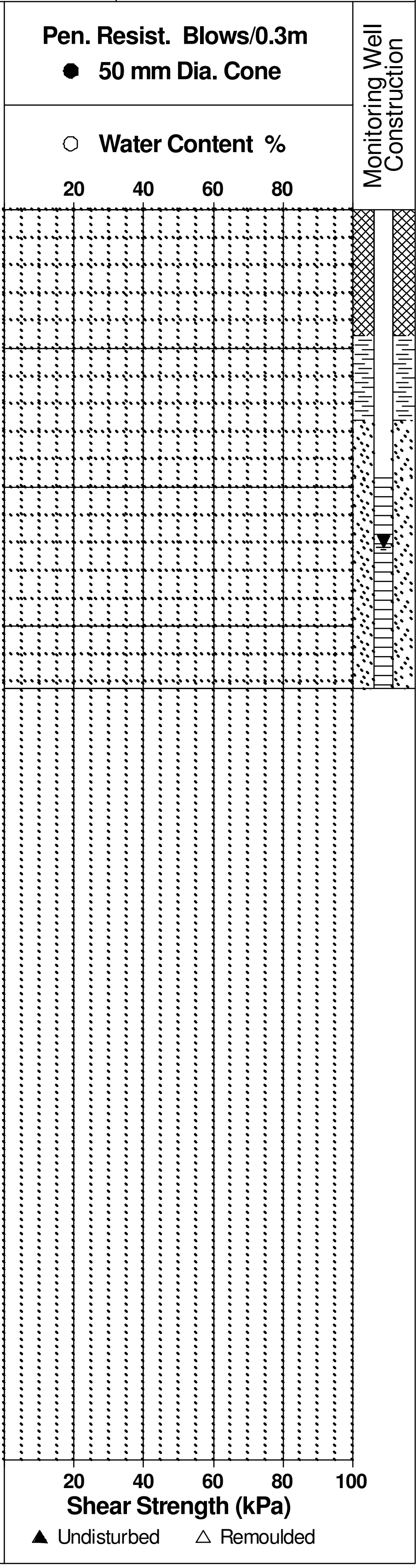
HOLE NO. BH 7-11

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	99.56
Asphaltic concrete 0.10						
FILL: Crushed stone with gravel 0.60	AU	1				
GLACIAL TILL: Dense, brown silty sand with gravel, cobbles and boulders	SS	2	50	19	1	98.56
	SS	3	92	38	2	97.56
	SS	4	92	48		
3.45	SS	5	100	50+	3	96.56

End of Borehole

Practical refusal to augering @ 3.45m depth

(GWL @ 2.43m-June 20/11)



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SOIL PROFILE AND TEST DATA

Phase I-II Environmental Site Assessment
175 Richmond Road
Ottawa, Ontario

DATUM

REMARKS

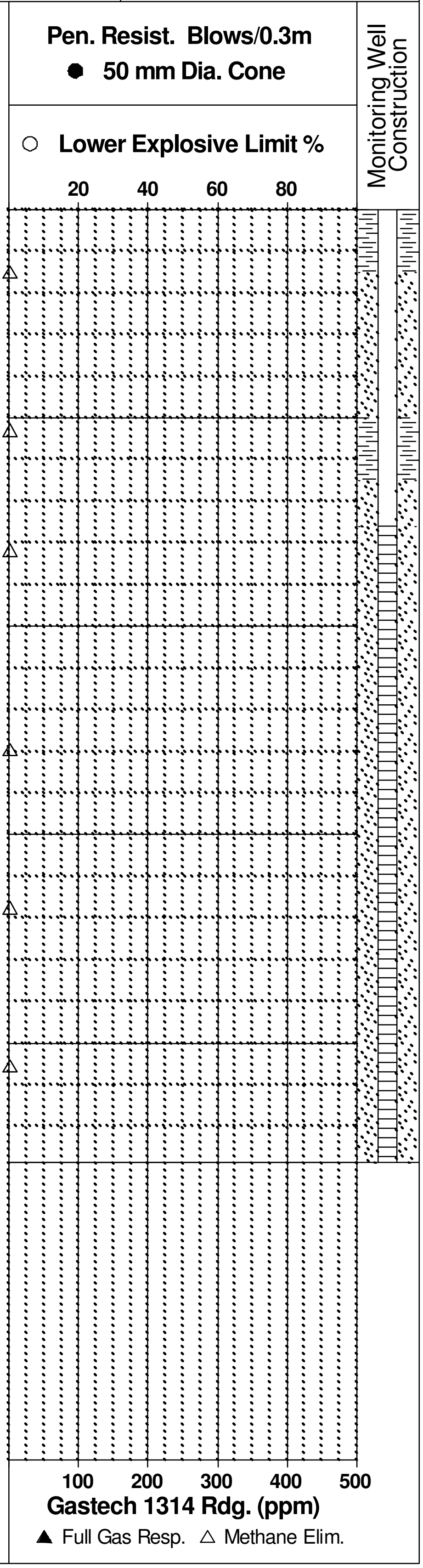
BORINGS BY CME 55 Power Auger

DATE 14 Oct 09

FILE NO. PE1833

HOLE NO. BH 1

SOIL DESCRIPTION		STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)
			TYPE	NUMBER	% RECOVERY	N VALUE or RQD		
GROUND SURFACE							0	
Asphaltic concrete	0.10							
FILL: Brown silty sand and crushed stone	0.60		AU	1				
GLACIAL TILL: Grey silty sand with clay, gravel, cobbles and boulders	4.57		SS	2	42	27	1	
			SS	3	44	50+		
							2	
			SS	4	50	13		
							3	
			SS	5	100	7		
							4	
			SS	6	42	5		
End of Borehole								



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SOIL PROFILE AND TEST DATA

Phase I-II Environmental Site Assessment
175 Richmond Road
Ottawa, Ontario

DATUM

REMARKS

BORINGS BY CME 55 Power Auger

DATE 14 Oct 09

FILE NO. PE1833

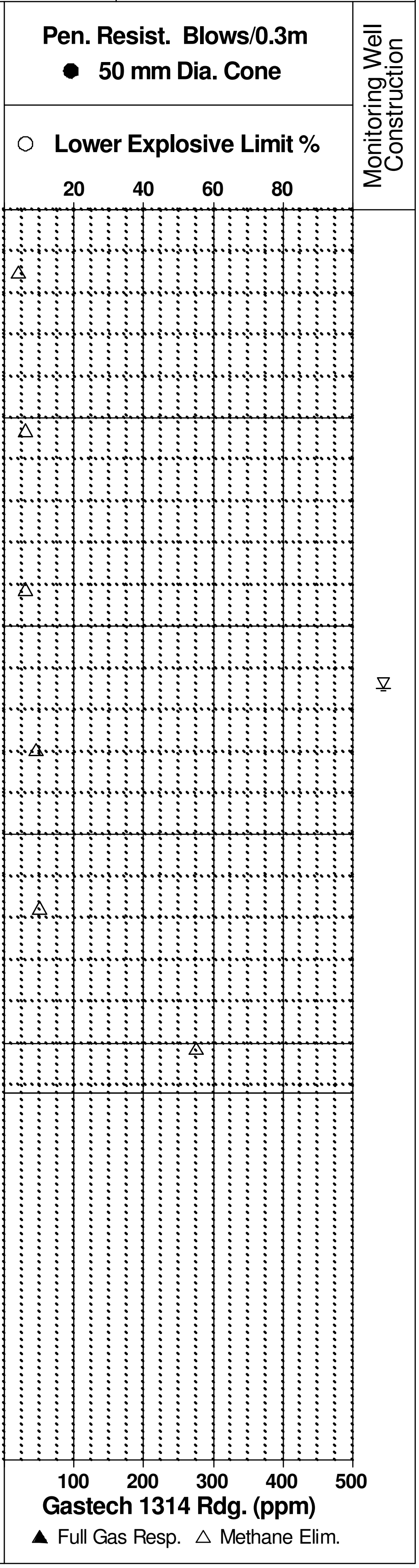
HOLE NO. BH 2

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	
Asphaltic concrete 0.08							
FILL: Crushed stone and stone dust 0.60		AU	1				
FILL: Brown silty sand with gravel and asphalt pieces 1.40		SS	2	42	21	1	
FILL: Brown silty sand		SS	3	42	4	2	
		SS	4	25	1		
		SS	5	42	5	3	
4.24		SS	6	59	50+	4	

End of Borehole

Practical refusal to augering @ 4.24m depth

(Open hole WL @ 2.3m depth)



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

SOIL PROFILE AND TEST DATA

Phase I-II Environmental Site Assessment
175 Richmond Road
Ottawa, Ontario

DATUM

REMARKS

BORINGS BY CME 55 Power Auger

DATE 14 Oct 09

FILE NO. PE1833

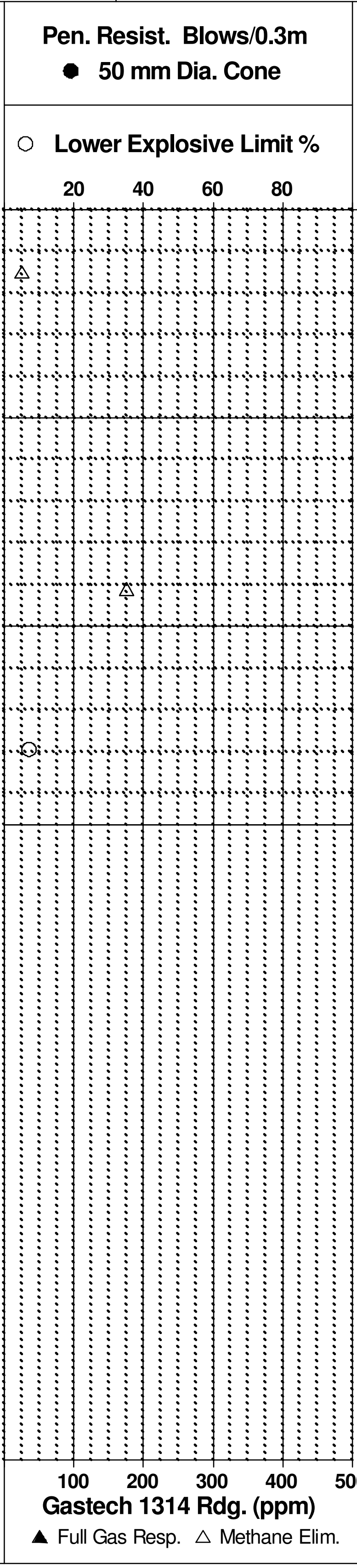
HOLE NO. BH 3

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	
Asphaltic concrete 0.08						
FILL: Grey silty sand and crushed stone 0.60	AU	1				
GLACIAL TILL: Grey silty sand with gravel, cobbles and boulders	SS	2	0	50+	1	
	SS	3	50	23	2	
2.95	SS	4	46	22		

End of Borehole

Practical refusal to augering @ 2.95m depth

(Open hole WL @ 2.3m depth)



Monitoring Well Construction

patersongroup Consulting Engineers

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

SOIL PROFILE AND TEST DATA

Phase I-II Environmental Site Assessment
175 Richmond Road
Ottawa, Ontario

DATUM

REMARKS

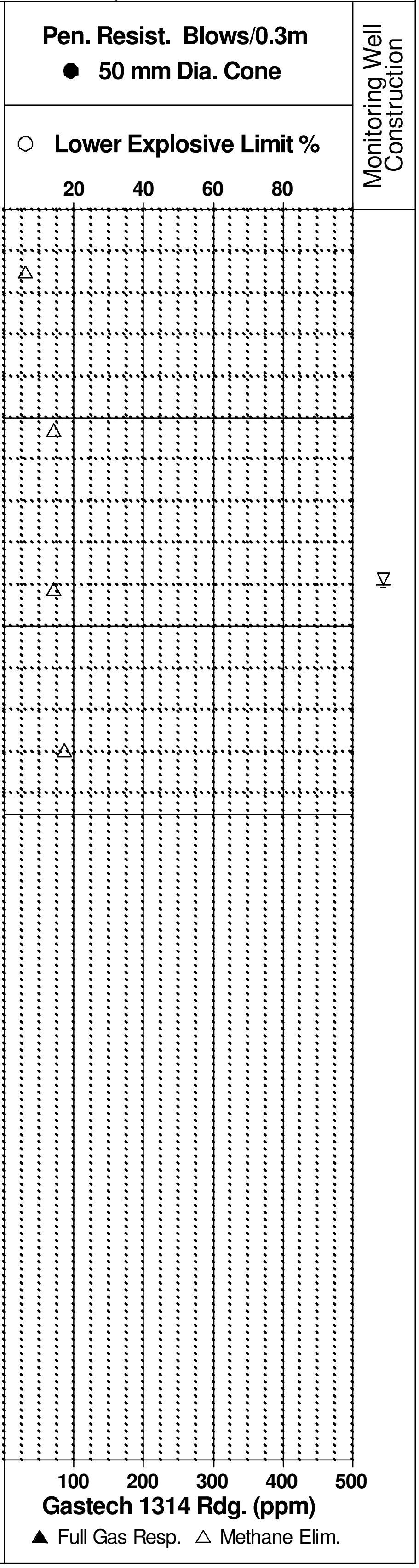
BORINGS BY CME 55 Power Auger

DATE 14 Oct 09

FILE NO. PE1833

HOLE NO. BH 4

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	
Asphaltic concrete 0.05							
FILL: Brown silty sand and gravel, some asphalt pieces 0.60		AU	1				
GLACIAL TILL: Grey silty sand with gravel and cobbles		SS	2	42	21	1	
		SS	3	50	32	2	
2.90		SS	4		28		
End of Borehole (Open hole WL @ 1.8m depth)							



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

SOIL PROFILE AND TEST DATA

Phase I-II Environmental Site Assessment
175 Richmond Road
Ottawa, Ontario

DATUM

REMARKS

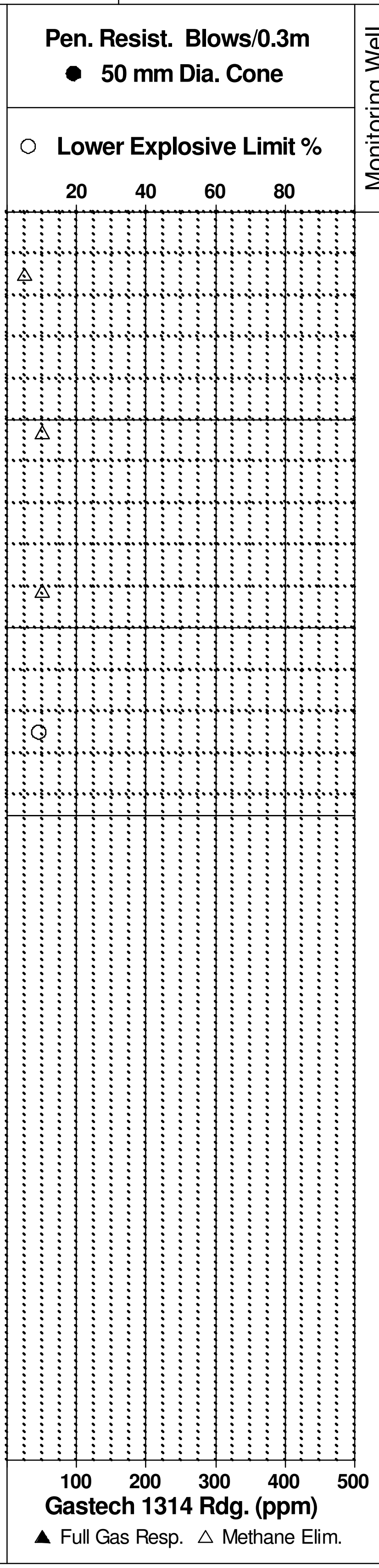
BORINGS BY CME 55 Power Auger

DATE 14 Oct 09

FILE NO. PE1833

HOLE NO. BH 5

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	
Asphaltic concrete 0.10						
FILL: Brown silty sand and gravel 0.60	AU	1				
GLACIAL TILL: Grey silty sand with gravel and cobbles	SS	2	58	18	1	
	SS	3	50	43	2	
	SS	4	59	50+		
2.90 End of Borehole						



Monitoring Well Construction

patersongroup Consulting Engineers

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

SOIL PROFILE AND TEST DATA

Phase I-II Environmental Site Assessment
175 Richmond Road
Ottawa, Ontario

DATUM

REMARKS

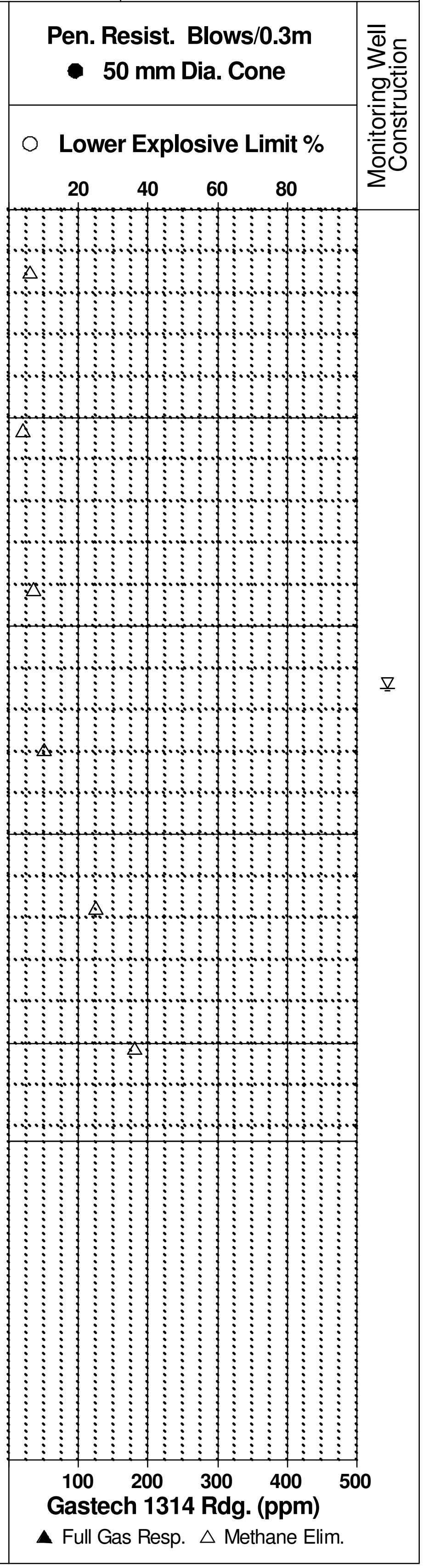
BORINGS BY CME 55 Power Auger

DATE 14 Oct 09

FILE NO. PE1833

HOLE NO. BH 6

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	
Asphaltic concrete 0.08						
FILL: Brown silty sand with gravel and cobbles	AU	1				
	SS	2	17	16	1	
1.37						
GLACIAL TILL: Compact to dense, brown silty sand with gravel and cobbles	SS	3	25	12	2	
	SS	4	50	17		
	SS	5	58	49	3	
	SS	6		50+	4	
4.47						
End of Borehole Practical refusal to augering @ 4.47m depth (Open hole WL @ 2.3m depth)						



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)

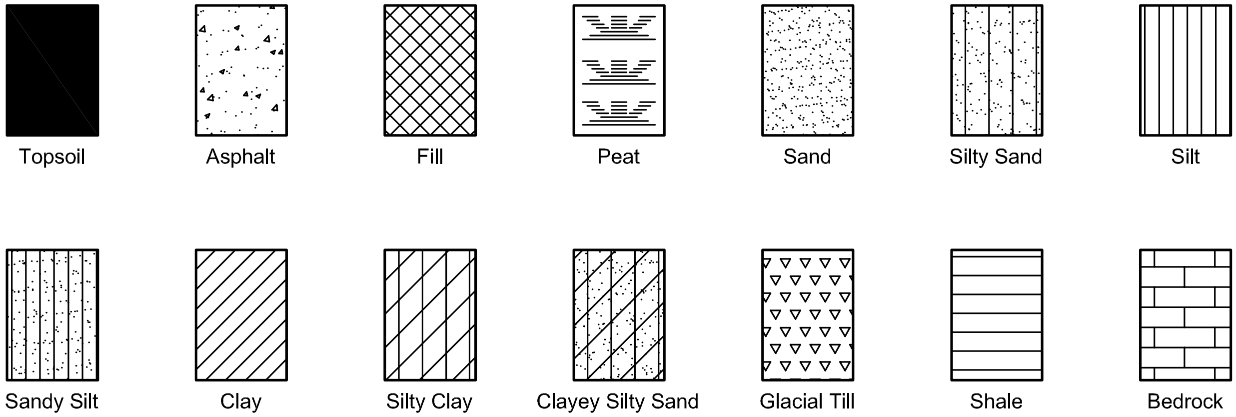
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

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.

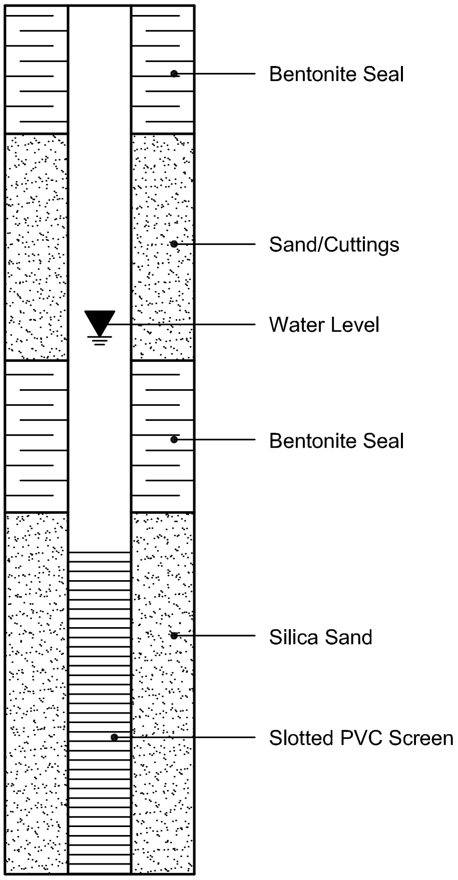
SYMBOLS AND TERMS (continued)

STRATA PLOT

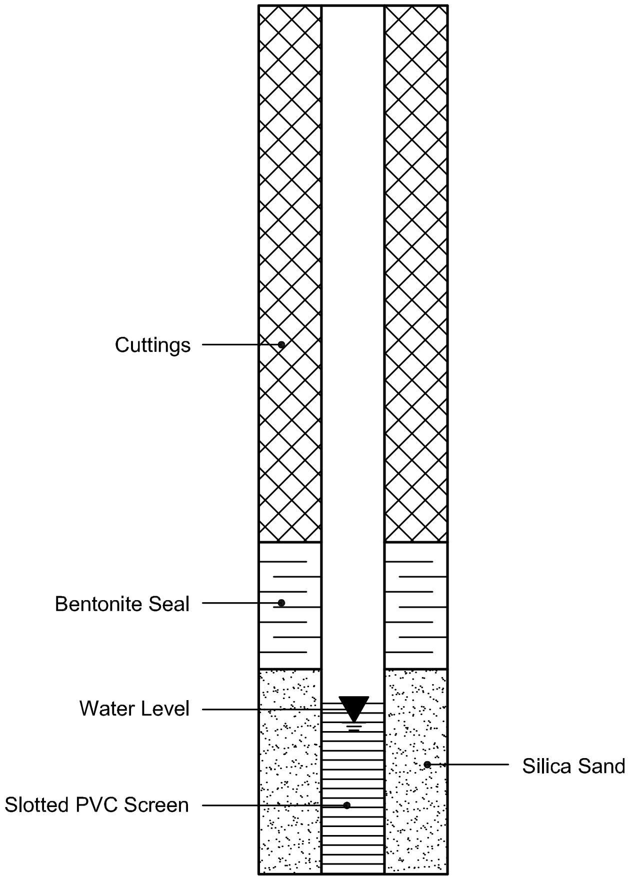


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Order #: 1128309

Certificate of Analysis

Client: **Paterson Group Consulting Engineers**
Client PO: 7386
Project Description: PG2363

Report Date: 14-Jul-2011
Order Date:8-Jul-2011

	Client ID:	BH9-11 SS4	-	-	-
	Sample Date:	08-Jul-11	-	-	-
	Sample ID:	1128309-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	93.2	-	-	-
General Inorganics					
pH	0.1 pH Units	8.0	-	-	-
Resistivity	0.10 Ohm.m	25.2	-	-	-
Anions					
Chloride	5 ug/g dry	84	-	-	-
Sulphate	5 ug/g dry	291	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING PG2363-1 - TEST HOLE LOCATION PLAN

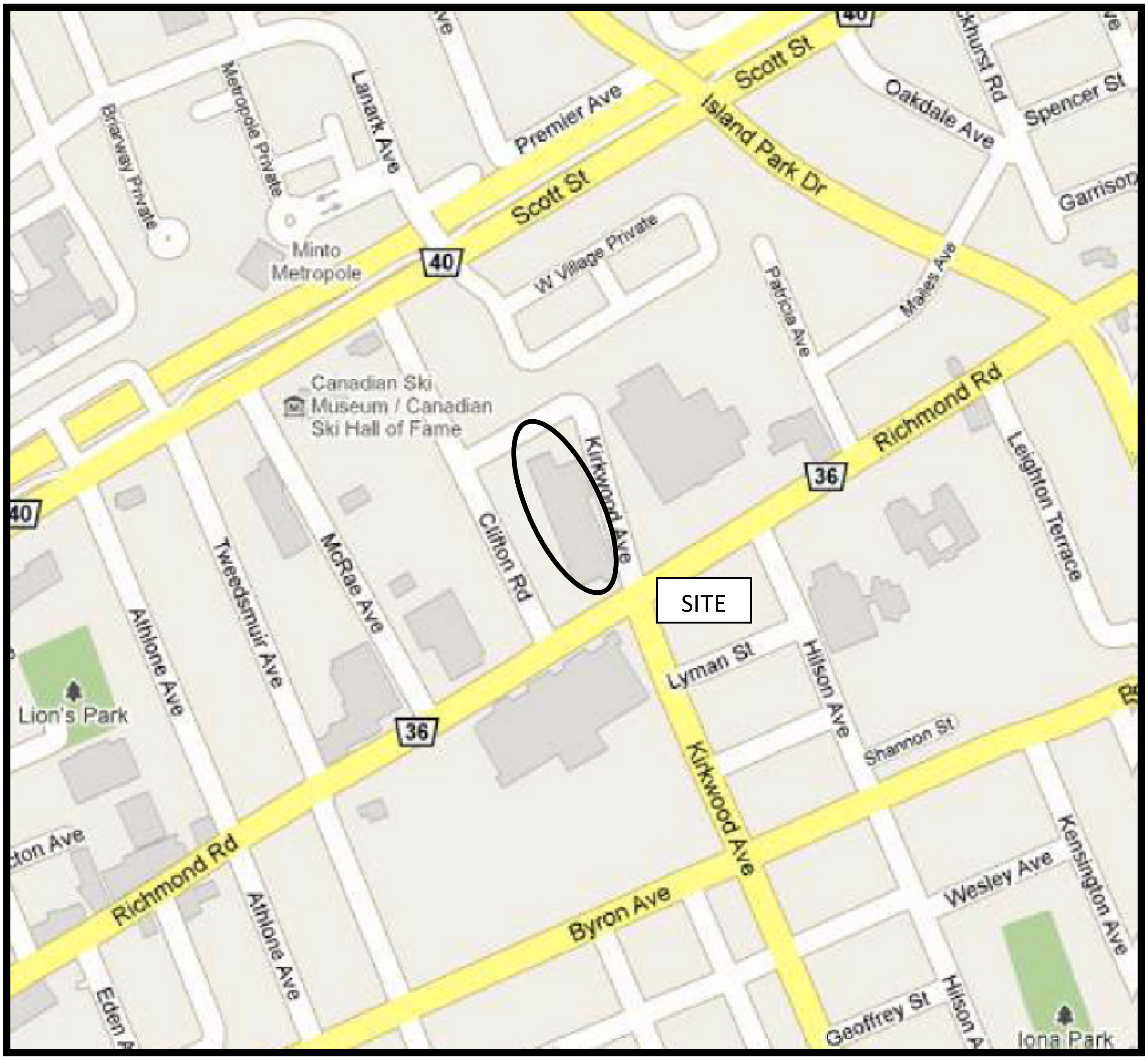
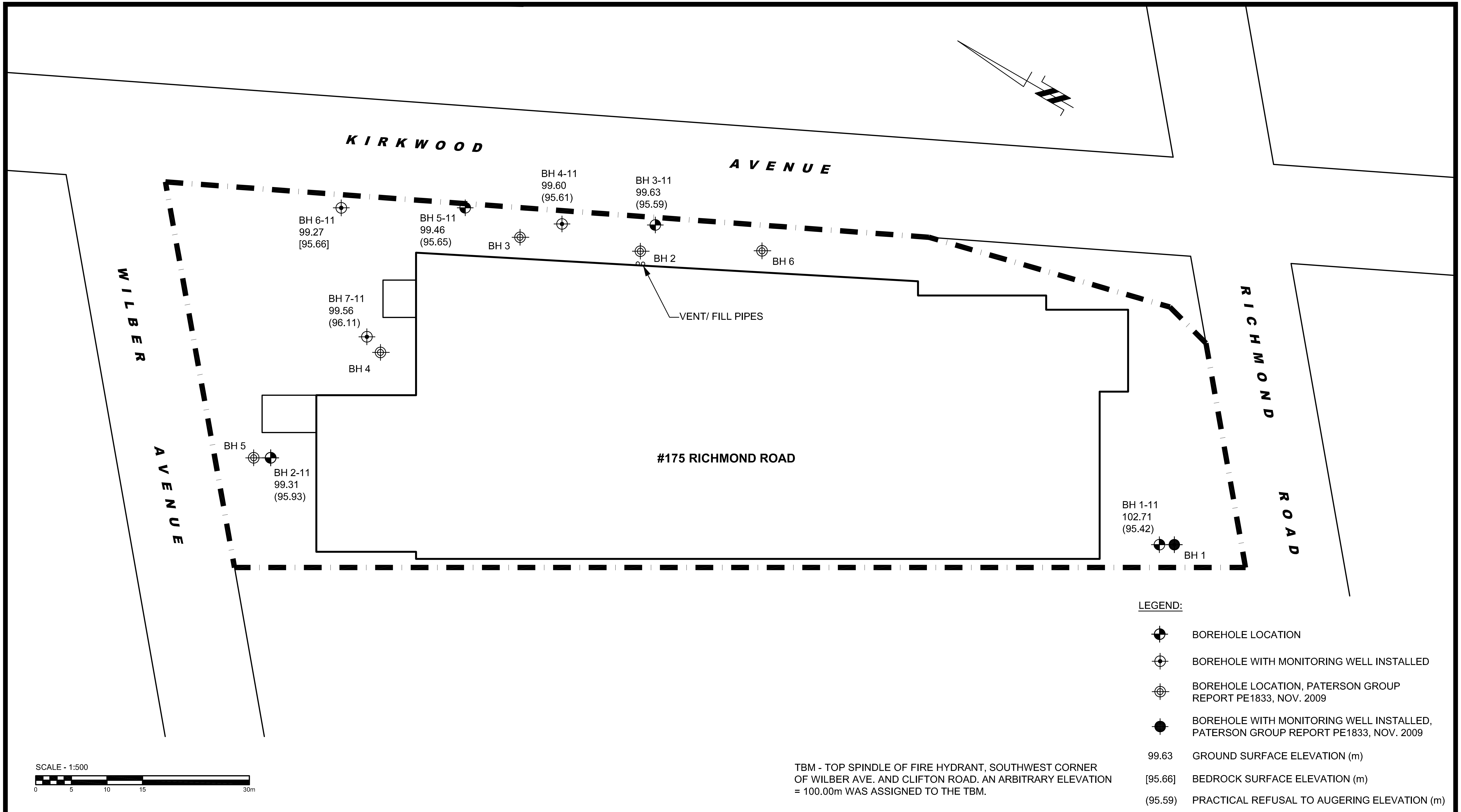


FIGURE 1

KEY PLAN



LEGEND:

⊕	BOREHOLE LOCATION
⊕	BOREHOLE WITH MONITORING WELL INSTALLED
⊕	BOREHOLE LOCATION, PATERSON GROUP REPORT PE1833, NOV. 2009
●	BOREHOLE WITH MONITORING WELL INSTALLED, PATERSON GROUP REPORT PE1833, NOV. 2009
99.63	GROUND SURFACE ELEVATION (m)
[95.66]	BEDROCK SURFACE ELEVATION (m)
(95.59)	PRACTICAL REFUSAL TO AUGERING ELEVATION (m)

TBM - TOP SPINDLE OF FIRE HYDRANT, SOUTHWEST CORNER OF WILBER AVE. AND CLIFTON ROAD. AN ARBITRARY ELEVATION = 100.00m WAS ASSIGNED TO THE TBM.

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Scale:	1:500
Des.:	MK
Dwn:	MPG
Chkd:	DG

CLARIDGE HOMES

GEOTECHNICAL INVESTIGATION
175 RICHMOND ROAD

OTTAWA, ONTARIO

TEST HOLE LOCATION PLAN

Dwg. No. PG2363-1
Report No.: PG2363-1
Date: 06/2011

11x17