Geotechnical Engineering

Environmental Engineering

Hydrogeology

Geological Engineering

Materials Testing

Building Science

Archaeological Services

patersongroup

Geotechnical Investigation

Proposed Multi-Storey Building
1451 Wellington Street
Ottawa, Ontario

Prepared For

Mizrahi Developments

October 3, 2013

Report PG2961-1R

Paterson Group Inc.
Consulting Engineers
154 Colonnade Road South
Ottawa (Nepean), Ontario
Canada K2E 7J5

Tel: (613) 226-7381
Fax: (613) 226-6344
www.patersongroup.ca

TABLE OF CONTENTS

PAGE

1.0 INTRODUCTION 1

2.0 PROPOSED PROJECT 1

3.0 METHOD OF INVESTIGATION
3.1 Field Investigation 2
3.2 Field Survey 3
3.3 Laboratory Testing 3
3.4 Analytical Testing 3

4.0 OBSERVATIONS
4.1 Surface Conditions 4
4.2 Subsurface Profile 4
4.3 Groundwater 4

5.0 DISCUSSION
5.1 Geotechnical Assessment 5
5.2 Site Grading and Preparation 5
5.3 Foundation Design 7
5.4 Design for Earthquakes 8
5.5 Basement Slab 8
5.6 Basement Wall 8
5.7 Rock Anchor Design 10
5.8 Pavement Structure 13

6.0 DESIGN AND CONSTRUCTION PRECAUTIONS
6.1 Foundation Drainage and Backfill 15
6.2 Protection of Footings Against Frost Action 16
6.3 Excavation Side Slopes 16
6.4 Pipe Bedding and Backfill 18
6.5 Groundwater Control 18
6.6 Winter Construction 19
6.7 Corrosion Potential and Sulphate 20

7.0 RECOMMENDATIONS 21

8.0 STATEMENT OF LIMITATIONS 22

APPENDICES

Appendix 1	Soil Profile and Test Data Sheets Symbols and Terms Analytical Testing Results
Appendix 2	Figure 1 - Key Plan Drawing PG2961-1 - Test Hole Location Plan

1.0 INTRODUCTION

Paterson Group (Paterson) was commissioned by Mizrahi Developments to conduct a geotechnical investigation for the proposed multi-storey building to be located at 1445 Wellington Street and 1451 Wellington Street in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2).

The objectives of the current investigation were to:

- ☐ determine the subsurface soil, bedrock and groundwater conditions 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. This report contains the findings and includes geotechnical recommendations pertaining to the design and construction of the mixed-use (commercial and residential) development as understood at the time of writing this report.

2.0 PROPOSED PROJECT

At the time of issuance of this report, specific details of the proposed structure have not been determined. However, it is understood that consideration is being given to constructing up to a twelve (12) storey building with several levels of underground parking encompassing the majority of the subject site.

3.0 METHOD OF INVESTIGATION

3.1 Field Investigation

Field Program

The field program for the present investigation was conducted on May 13, 2013. Four (4) boreholes were advanced to a maximum depth of 6.7 m. Two (2) previous environmental investigations were completed in 2006 and 2009. A total of twelve (12) boreholes were completed previously and advanced to a maximum depth of 8.2 m.

The boreholes were completed with a truck mounted 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 the geotechnical division. The drilling procedure consisted of hollow stem augering to the required depths at select locations, sampling and testing the overburden.

Sampling and In Situ Testing

Soil samples were recovered from a 50 mm diameter split-spoon, the auger flights or grab samples. The split-spoon, auger and grab 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, auger and grab samples were recovered from the boreholes are presented as SS, AU and G, respectively, on the Soil Profile and Test Data sheets.

Standard Penetration Tests (SPT) were conducted and 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 sample 300 mm into the soil after the initial penetration of 150 mm using a 63.5 kg hammer falling from a height of 760 mm.

Diamond drilling was completed at two locations during the current investigation, BH 1-13 and BH 3-13, to confirm the bedrock quality. A recovery value and a Rock Quality Designation (RQD) value were calculated for each drilled section of bedrock and are presented as RC on the Soil Profile and Test Data sheets in Appendix 1. The recovery value is the ratio of the bedrock sample length recovered over the drilled section length, in percentage. The RQD value is the total length ratio of intact rock core length more than 100 mm in one drilled section over the length of the drilled section, in percentage. 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.

Groundwater

Groundwater monitoring wells were installed in BH 1 and 3 to permit the monitoring of water 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 otherwise directed.

3.2 Field Survey

The test hole locations were selected and determined in the field by Paterson personnel to provide general coverage of the subject site. The test hole locations and elevations were surveyed in the field by Paterson. A nail in the sidewalk, located near the southeast corner of the subject was surveyed as a temporary benchmark (TBM). A geodetic elevation of 66.47 m was provided for the TBM by Annis, O'Sullivan and Vollebekk (AOV).

The location and ground surface elevation at each test hole location is presented on Drawing PG2961-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

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

3.4 Analytical Testing

One (1) soil sample was submitted for analytical testing to assess the potential for exposed ferrous metals and the sulphate potential against subsurface concrete structures. The sample was submitted to determine the concentration of sulphate and chloride, the resistivity and the pH of the soil. The results are provided 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 commercial building (one storey building of slab on grade construction within the west portion of the site) and a commercial building with a basement level. The majority of the subject site is asphalt covered with some landscaped areas and the ground surface is relatively flat and at grade with Wellington Street West.

4.2 Subsurface Profile

Generally, the subsurface profile at the borehole locations consists of a pavement structure and varying fill material overlying a native silty sand and glacial till deposit. The glacial till deposit is underlain by an interbedded limestone and dolostone bedrock. The fill material varied between a silty clay to a silty sand with organics, debris, gravel, cobbles and trace boulders. Bedrock and/or practical refusal to augering was encountered at all borehole locations, at depths between 2.3 m to 4.8 m. Based on the RQD values, the bedrock quality varies between fair to excellent quality.

Based on available geological mapping, bedrock in the area of the subject site consists of interbedded limestone and dolostone of the Gull River Formation. The overburden thickness is estimated to be between 2 to 5 m depth.

4.3 Groundwater

Groundwater monitoring wells were installed at BH 1-13 and BH 3-13 to measure groundwater levels. The following table presents our groundwater level measurements, as well as, groundwater levels from the previous boreholes. Groundwater levels are subject to seasonal fluctuations and could vary at the time of construction.

Table 1 - Summary of Groundwater Levels

Test Hole Number	Measured Groundwater Depth (m)	Date
BH 1-13	1.70	May 21, 2013
BH 3-13	4.34	May 21, 2013
BH 2 (PE0982)	2.50	October 11, 2006
BH 6 (PE2526)	3.25	November 18, 2009
BH 9 (PE2526)	2.40	November 18, 2009
BH 10 (PE2526)	2.53	November 18, 2009
BH 11 (PE2526)	2.80	November 18, 2009
BH 12 (PE2526)	2.64	November 18, 2009

5.0 DISCUSSION

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is adequate for the proposed multi-storey building. The proposed building is expected to be founded on conventional footings placed on clean, surface sounded bedrock.

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

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

5.2 Site Grading and Preparation

Stripping Depth

Due to the relatively shallow bedrock depth at the subject site and the anticipated founding level for the proposed building, all existing overburden material will be excavated from within the proposed building footprint. Bedrock removal should be required for the construction of the parking garage levels.

Bedrock Removal

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

As a general guideline, peak particle velocity (measured at the structures) should not exceed 25 mm/s 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 an experienced blasting consultant.

Excavation side slopes in sound bedrock could be completed with almost vertical side walls. A minimum of 1 m horizontal bench, should remain between the bottom of the overburden and the top of the bedrock surface to provide an area for potential sloughing or a stable base for the overburden shoring system.

Vibration Considerations

Construction operations could cause 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 a cooperative environment with the residents.

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

Two parameters determine the recommended vibration limit, 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). These guidelines are for current construction standards. Considering there are several sensitive buildings in close proximity to the subject site, consideration to lowering these guidelines is recommended. These guidelines are above perceptible human level and, in some cases, could be very disturbing to some people, a pre-construction survey is recommended 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 bedrock pop-outs, especially in areas where bedrock fractures 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.

Fill Placement

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

Non-specified existing fill along with site-excavated soil could be placed as general landscaping fill where settlement of the ground surface is of minor concern. These materials should be spread in lifts with a maximum thickness of 300 mm and compacted by the tracks of the spreading equipment to minimize voids. Non-specified existing fill and site-excavated soils are not suitable for placement as backfill against foundation walls, unless used in conjunction with a geocomposite drainage membrane, such as Miradrain G100N or Delta Drain 6000.

5.3 Foundation Design

Footings placed on a clean, surface sounded limestone bedrock surface could be designed for 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 should not contain 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 **4,500 kPa**, incorporating a geotechnical resistance factor of 0.5, and a bearing resistance at SLS of **3,000 kPa** could be provided if founded on limestone bedrock which is free of seams, fractures and voids within 1.5 m below the founding level. This should be verified by completing and probing 50 mm diameter drill holes to a depth of 1.5 m below the founding level within the all the footing footprints. A minimum of one probe hole should be completed per footing. The drill hole inspection should be completed by the geotechnical consultant.

Settlement

Footings bearing on an acceptable bedrock bearing surface and designed for 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 is **Class C** for the shallow foundations at the subject site. A higher seismic site class, such as Class A or B is available for the subject site. However, a site specific seismic shear wave test is required to provide the higher site classes according to the 2006 Ontario Building Code.

5.5 Basement Slab

All overburden soil will be removed for the proposed building and the basement floor slab will be founded on a bedrock medium. OPSS Granular A or Granular B Type II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab. It is recommended that the upper 200 mm of sub-slab fill consists of a 19 mm clear crushed stone.

In consideration of the groundwater conditions encountered during the investigation, a subfloor drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided in the clear stone backfill under the lower basement floor.

5.6 Basement Wall

There are several combinations of backfill and retained soils that could be applicable for the basement walls of the subject structure. However, the conditions could be well-represented by assuming the retained soil consists of a material with an angle of internal friction of 30 degrees and a dry unit weight of 20 kN/m^3. It is expected that a portion of 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 0.05 is recommended in conjunction with a dry unit weight of 23.5 kN/m^3 (effective unit weight of 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.

Undrained conditions are anticipated (i.e. below the groundwater level). Therefore, the applicable effective unit weight of the retained soil should be 13 kN/m^3, where applicable. A hydrostatic pressure should be added to the total static earth pressure when calculating the effective unit weight.

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

Static Conditions

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

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

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

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

Seismic Conditions

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

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

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

The peak ground acceleration, (a_{max}), for the Ottawa area is 0.42g according to OBC 2006. The vertical seismic coefficient is assumed to be zero.

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

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

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

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

5.7 Rock Anchor Design

The geotechnical design of grouted rock anchors in sedimentary bedrock is based upon two possible failure modes. The rock anchor could fail by shear failure along the grout/rock interface or by pullout at 60 to 90 degree 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 individual anchor load capacity.

A third failure mode of shear failure along the grout/steel interface should 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 a minimum of 1.2 m or four times the anchor hole diameter to minimize group influence effects. Anchors in close proximity to each other are 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.

Regardless of whether an anchor is a passive or the post tensioned type, the anchor is recommended to be provided with a fixed anchor length at the base, which will provide the capacity, and an free anchor length between the rock surface and the top of the bonded length. As the depth at which the apex of the shear failure cone develops midway along the bonded length, a fully bonded anchor would tend to have a much shallower cone, and therefore less geotechnical resistance, than 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 the entire drill hole be filled with cementitious grout. The free anchor length is provided by installing a sleeve to act as a bond break, with the sleeve filled with grout. Double corrosion protection can be provided with factory assembled systems, such as those available from Dywidag Systems International or Williams Form Engineering Corp.

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, could be calculated. A minimum grout strength of 40 MPa is recommended.

Rock Cone Uplift

The rock anchor capacity depends on the dimensions of the rock anchors and the anchorage system configuration. Based on existing bedrock information, a **Rock Mass Rating (RMR) of 69** was assigned to the bedrock, and Hoek and Brown parameters (**m and s**) were estimated as **0.575 and 0.00293**, respectively.

Recommended Grouted Rock Anchor Lengths

The parameters calculated for grouted 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	69 m=0.575 and s=0.00293
Unconfined compressive strength - Limestone	60 MPa
Effective unit weight - 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 drill hole diameter. The recommended anchor lengths are provided in Table 3. The factored tensile resistance values provided are based on a single anchor. The group influence effects has not been accounted for in the calculations below.

Table 3 - Recommended Rock Anchor Lengths - Grouted Rock Anchor

Diameter of Drill Hole (mm)	**Anchor Lengths (m)**			**Factored Tensile Resistance (kN)**
	Bonded Length	**Unbonded Length**	**Total Length**	
75	1.7	1.0	2.7	500
	2.2	1.4	3.6	750
	2.6	1.8	4.4	1000
	4.0	2.5	6.5	1500
125	1.4	0.8	2.2	500
	1.8	1.1	2.9	750
	2.2	1.3	3.5	1000
	2.8	2.0	4.8	1500

Other considerations

The anchor drill hole is recommended to be 1.5 to 2 times the rock anchor tendon diameter. The anchor drill holes should be inspected by geotechnical personnel and should be flushed clean prior to grouting. A tremie pipe is recommended to place grout from the bottom to top of the anchor holes.

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

5.8 Pavement Structure

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

Table 4 - Recommended Pavement Structure - Car Only Parking Areas

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

Table 5 - Recommended Pavement Structure
Access Lanes and Heavy Truck Parking Areas

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

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 sub-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 SPMDD with suitable vibratory equipment.

6.0 DESIGN AND CONSTRUCTION PRECAUTIONS

6.1 Foundation Drainage and Backfill

Foundation Drainage

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

☐ Bedrock vertical surface (Hoe ram any irregularities and prepare bedrock surface. Shotcrete areas to fill in cavities and smooth out angular features at the bedrock surface.);
☐ composite drainage layer

It is recommended that the composite drainage system (such as Miradrain G100N, Delta Drain 6000 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 the interior perimeter drainage pipe. The perimeter drainage pipe and underfloor drainage system should direct water to sump pit(s) within the lower basement area.

Underfloor Drainage

It is anticipated that underfloor drainage will be required to control water infiltration. For design purposes, we recommend that 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

Perimeter footings of heated structures are recommended to be protected against the deleterious effects of frost action. A minimum of 1.5 m of soil cover alone, or a combination of soil cover and foundation insulation should be provided.

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

The parking garage should not require protection against frost action due to the founding depth. Unheated structures, such as the access ramp, may be required to insulate 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.

6.3 Excavation

Temporary Side Slopes

The temporary excavation side slopes anticipated should either be excavated to acceptable slopes or retained by shoring systems from the beginning of the excavation until the structure is backfilled.

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be cut back at 1H:1V or flatter. The flatter slope is required for excavation below groundwater level. The subsurface soil is considered to be mainly a Type 2 and 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects. Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should maintain safe working distance from the excavation sides.

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

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 could 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 included to the earth pressures described below. These systems could be cantilevered, anchored or braced. Generally, the shoring systems should be provided with tie-back rock anchors to ensure the stability. The shoring system is recommended to be adequately supported to resist toe failure, if required, by means of rock bolts or extending the piles into the bedrock through pre-augered holes if a soldier pile and lagging system is the preferred method.

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

Table 6 - 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
Dry Unit Weight (γ), kN/m^3	20
Effective Unit Weight (γ), kN/m^3	13

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

The hydrostatic groundwater pressure should be included to the earth pressure distribution wherever the effective unit weight are calculated for earth pressures. If the groundwater level is lowered, the dry unit weight for the soil/bedrock should be calculated full weight, with no hydrostatic groundwater pressure component.

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

6.4 Pipe Bedding and Backfill

A minimum of 300 mm of OPSS Granular A should be placed for bedding for sewer or water pipes when placed on bedrock subgrade. The bedding should extend to the spring line of the pipe. Cover material, from the spring line to a minimum of 300 mm above the pipe obvert 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 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 SPMDD.

6.5 Groundwater Control

Groundwater Control for Building Construction

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.

Based on the groundwater level being located within the bedrock, infiltration levels will be low through the excavation face. The groundwater infiltration will be controllable with open sumps and pumps.

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. A minimum of four to five months should be allocated for completion of the application and issuance of the permit by the MOE.

Long-term Groundwater Control

Our recommendations for the proposed building's long-term groundwater control are presented in Subsection 6.1. Any groundwater encountered along the building's perimeter or sub-slab drainage system will be directed to the proposed building's cistern/sump pit. Provided the proposed groundwater infiltration control system is properly implemented and approved by the geotechnical consultant at the time of construction, it is expected that groundwater flow will be low (i.e.- less than 50,000 L/day) with peak periods noted after rain events. A more accurate estimate can be provided at the time of construction, once groundwater infiltration levels are observed. It is anticipated that the groundwater flow will be controllable using conventional open sumps.

Impacts on Neighbouring Structures

Based on our observations, the groundwater level is anticipated at a 2 to 3 m depth and within the bedrock. Therefore, a local groundwater lowering is anticipated under short-term conditions due to construction of the proposed building. It should be noted that the extent of any significant groundwater lowering will take place within a limited range of the subject site due to the minimal temporary groundwater lowering.

The neighbouring structures are expected to be founded within native glacial till and/or directly over a bedrock bearing surface. No issues are expected with respect to groundwater lowering that would cause long term damage to adjacent structures surrounding the proposed building.

6.6 Winter Construction

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

Where excavations are completed in proximity of existing structures which may be adversely affected due to the freezing conditions. In particular, where a shoring system is constructed, 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.

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

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

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 are indicative of non significant factors in creating a corrosive environment for exposed ferrous metals, whereas the resistivity is indicative of an aggressive to highly 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.
- ☐ 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.
- ☐ 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 grading plan, drawings and specifications are completed.

A geotechnical investigation is a limited sampling of a site. The recommendations are based on information gathered at the specific test locations and can only be extrapolated to an undefined limited area around the test locations. Should any conditions at the site be encountered which differ from those at the test locations, Paterson requests notification immediately in order to permit reassessment of the 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 Mizrahi Developments, or their agent(s) is not authorized without review by Paterson Group for the applicability of our recommendations to the altered use of the report.

Paterson Group Inc.

Joe Forsyth, P.Eng.

David J. Gilbert, P.Eng.

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

Report Distribution:

☐ Mizrahi Developments (3 copies)
☐ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL RESULTS

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
1445 and 1451 Wellington Street West
Ottawa, Ontario

DATUM TBM - Mag nail in sidewalk, located near the southeast corner of subject site, north side of Wellington Street West. Geodetic elevation = 66.47m, as per Annis, O'Sullivan, Vollebekk Ltd.

REMARKS

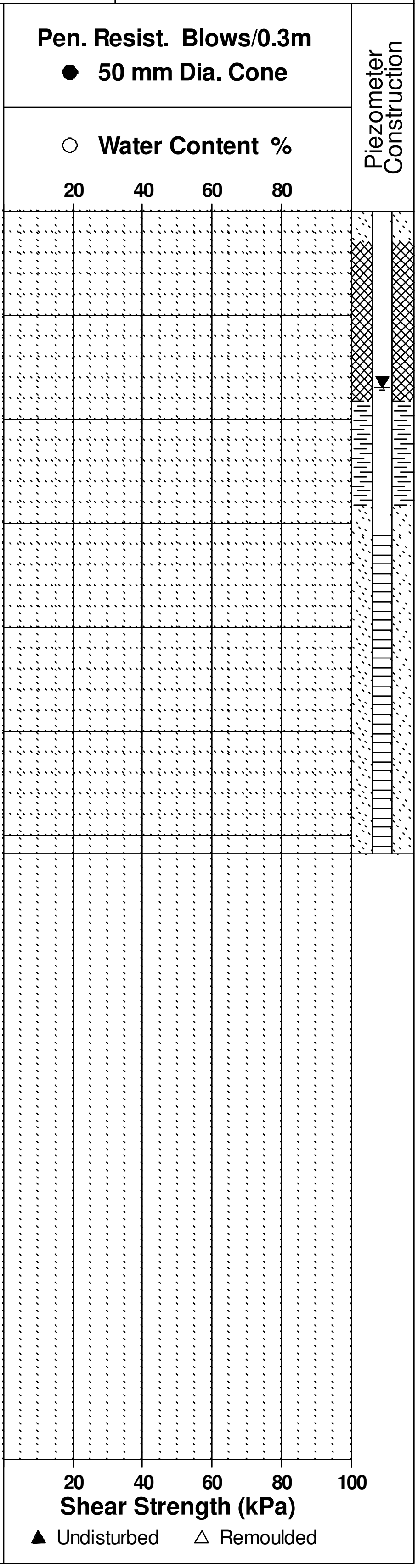
BORINGS BY CME 55 Power Auger

DATE May 13, 2013

FILE NO. PG2961

HOLE NO. BH 1-13

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	65.92
25mm Asphaltic concrete over crushed stone 0.25	AU	1				
FILL: Brown silty sand with gravel 0.69	AU	2				
Very loose, brown **SILTY SAND** with gravel 1.45	SS	3	33	3	1	64.92
GLACIAL TILL: Brown silty sand with gravel, cobbles, boulders, trace clay 2.34	SS	4	83	10	2	63.92
	SS	5	4	50+		
	RC	1	100	59	3	62.92
BEDROCK: Grey limestone					4	61.92
	RC	2	100	76	5	60.92
6.17	RC	3	100	100	6	59.92
End of Borehole (GWL @ 1.70m-May 21, 2013)						



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
1445 and 1451 Wellington Street West
Ottawa, Ontario

DATUM / **REMARKS** TBM - Mag nail in sidewalk, located near the southeast corner of subject site, north side of Wellington Street West. Geodetic elevation = 66.47m, as per Annis, O'Sullivan, Vollebekk Ltd.

FILE NO. PG2961

HOLE NO. BH 2-13

BORINGS BY CME 55 Power Auger

DATE May 13, 2013

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)
GROUND SURFACE		TYPE	NUMBER	% RECOVERY	N VALUE or RQD		
50mm Asphaltic concrete over crushed stone 0.25	▨	AU	1			0	66.15
FILL: Brown silty sand with gravel	▨	AU	2				
1.45	▨	SS	3		50+	1	65.15
GLACIAL TILL: Brown silty sand with gravel, cobbles, boulders	^^^	SS	4	64	71	2	64.15
2.95	^^^	SS	5	100	50+		
End of Borehole Practical refusal to augering at 2.95m depth							

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

○ **Water Content %**

20 40 60 80

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

Piezometer Construction

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
1445 and 1451 Wellington Street West
Ottawa, Ontario

DATUM TBM - Mag nail in sidewalk, located near the southeast corner of subject site, north side of Wellington Street West. Geodetic elevation = 66.47m, as per Annis, O'Sullivan, Vollebekk Ltd.

REMARKS

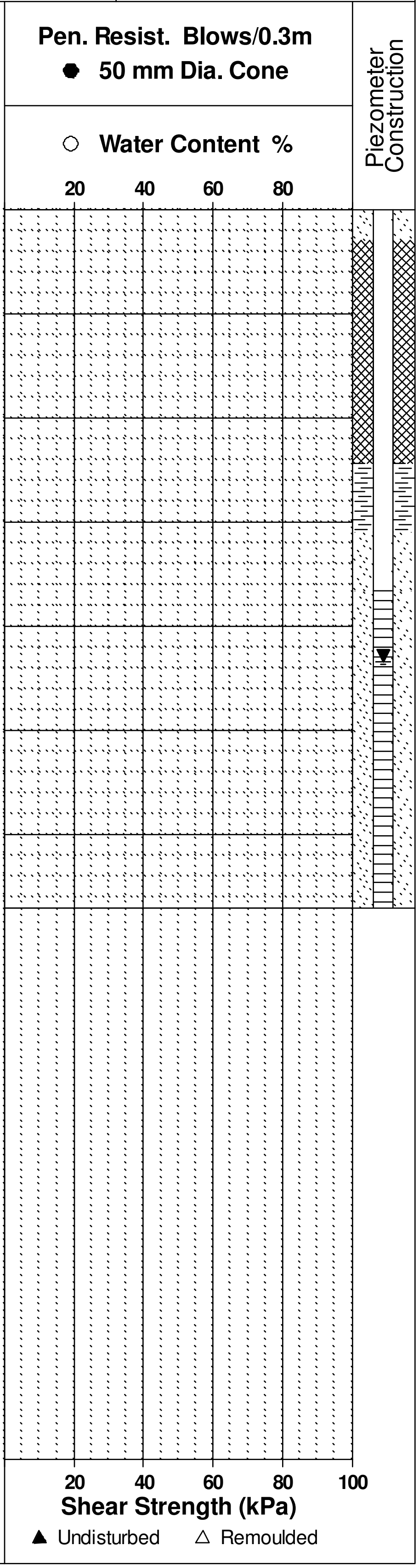
BORINGS BY CME 55 Power Auger

DATE May 13, 2013

FILE NO. PG2961

HOLE NO. BH 3-13

SOIL DESCRIPTION		SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	66.38
50mm Asphaltic concrete over crushed stone	0.25						
FILL: Brown silty sand with gravel	0.69	AU	1				
GLACIAL TILL: Brown silty sand with gravel, cobbles, boulders		SS	2	83	5	1	65.38
		SS	3	83	32	2	64.38
		SS	4	100	50+		
	3.18	SS	5	100	50+	3	63.38
BEDROCK: Grey limestone		RC	1	100	67	4	62.38
		RC	2	100	54	5	61.38
		RC	3	100	55	6	60.38
	6.70						
End of Borehole							
(GWL @ 4.34m-May 21, 2013)							



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
1445 and 1451 Wellington Street West
Ottawa, Ontario

DATUM TBM - Mag nail in sidewalk, located near the southeast corner of subject site, north side of Wellington Street West. Geodetic elevation = 66.47m, as per Annis, O'Sullivan, Vollebekk Ltd.

REMARKS

BORINGS BY CME 55 Power Auger

DATE May 13, 2013

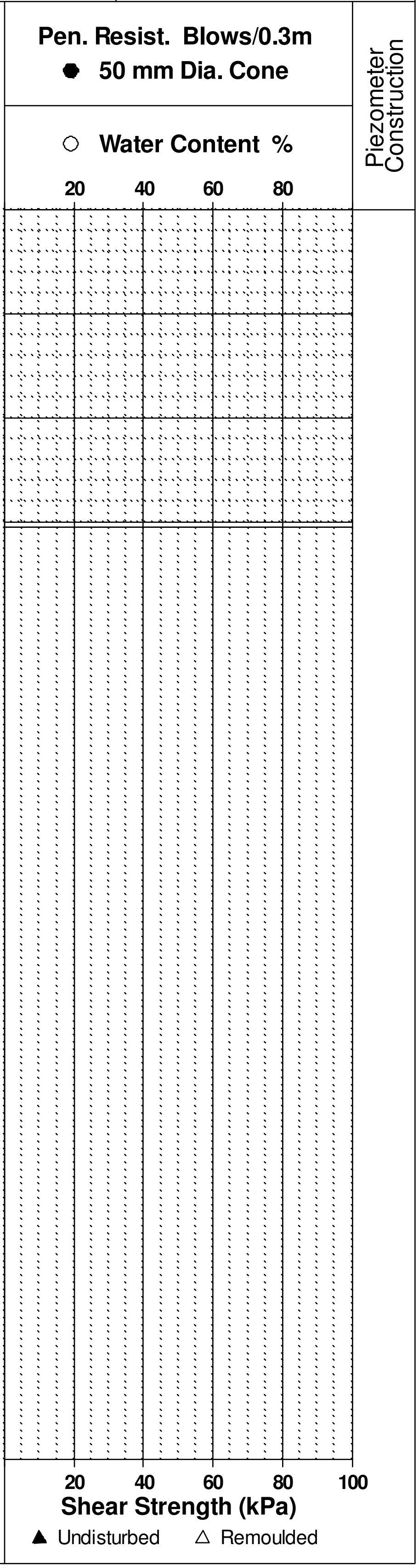
FILE NO. PG2961

HOLE NO. BH 4-13

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	66.37
50mm Asphaltic concrete over crushed stone 0.28	AU	1				
FILL: Brown silty sand with gravel 0.69	AU	2				
GLACIAL TILL: Brown silty sand with gravel, cobbles, boulders	SS	3	50	13	1	65.37
	SS	4	100	50+	2	64.37
3.05	SS	5	100	50+	3	63.37

End of Borehole

Practical refusal to augering at 3.05m depth



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Phse II - Environmental Site Assessment
1445 and 1451 Wellington Street West
Ottawa, Ontario

DATUM TBM - Mag nail in sidewalk, located near the southwest corner of subject site, north side of Wellington Street West. Geodetic elevation = 66.47m, as per AOV.

REMARKS

BORINGS BY CME 75 Power Auger

DATE October 6, 2006

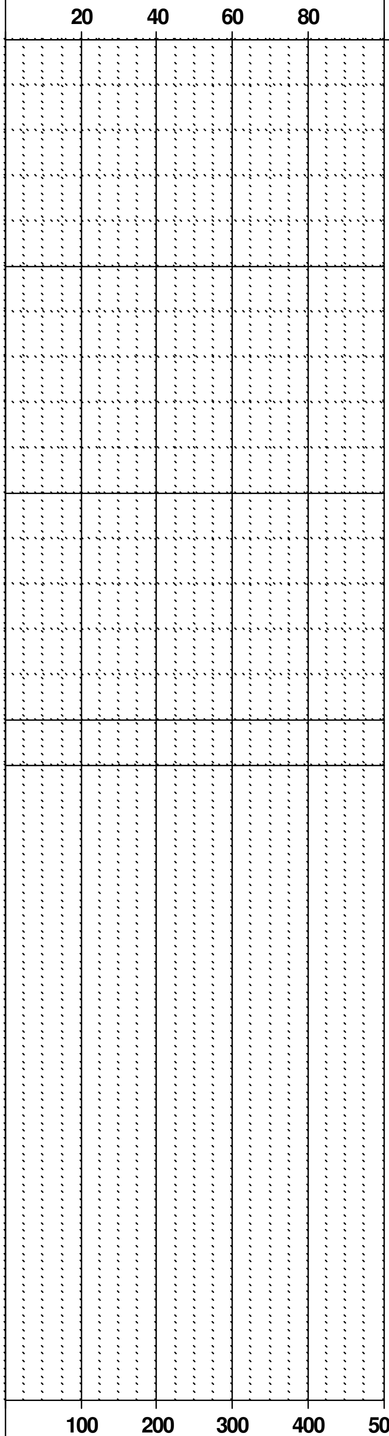
FILE NO. PE0982

HOLE NO. BH 1

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	66.61
Asphaltic concrete 0.05							
FILL: Dark brown silty sand with gravel 0.46							
FILL: Sandy silt with clay		SS	1	50	10	1	65.61
1.68							
GLACIAL TILL: Very dense, brown silty sand with gravel, cobbles and boulders		SS	2	54	35	2	64.61
		SS	3	62	56	3	63.61
3.20		SS	4	33	50+		
End of Borehole Practical refusal to augering @ 3.20m depth							

Photo Ionization Detector
● Volatile Organic Rdg. (ppm)

○ **Lower Explosive Limit %**



RKI Eagle Rdg. (ppm)
▲ Full Gas Resp. △ Methane Elim.

Monitoring Well Construction

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Phse II - Environmental Site Assessment
1445 and 1451 Wellington Street West
Ottawa, Ontario

DATUM TBM - Mag nail in sidewalk, located near the southwest corner of subject site, north side of Wellington Street West. Geodetic elevation = 66.47m, as per AOV.

REMARKS

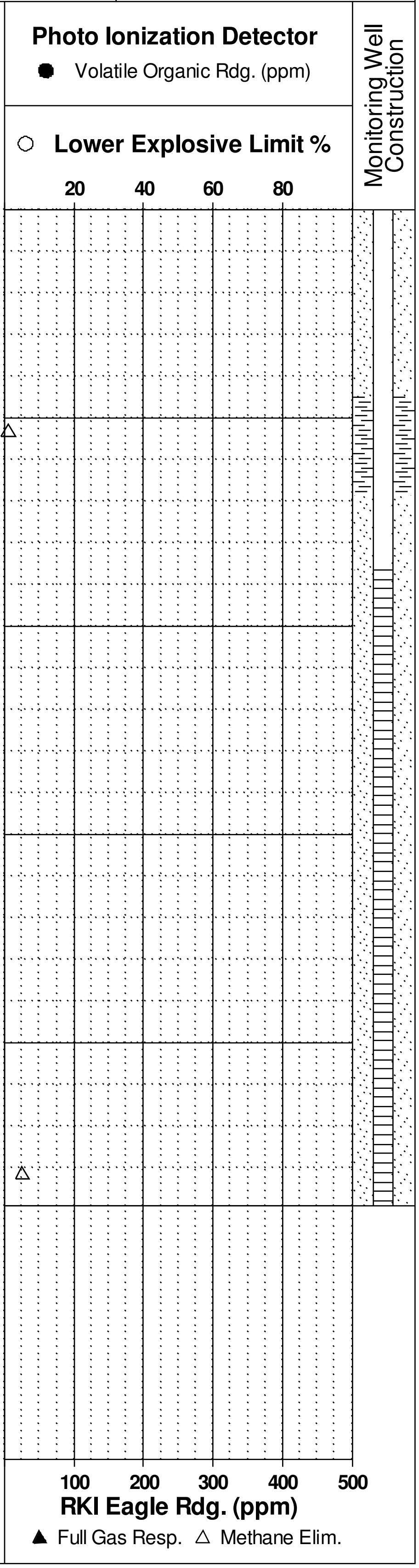
BORINGS BY CME 75 Power Auger

DATE October 6, 2006

FILE NO. PE0982

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	66.72
Asphaltic concrete 0.05							
FILL: Brown silty sand with gravel and crushed stone 0.53							
Compact, grey **SILTY** fine **SAND** 1.62		SS	1	4	4	1	65.72
GLACIAL TILL: Dense to very dense, grey silty sand with gravel, cobbles and boulders		SS	2	54	18	2	64.72
		SS	3	75	40		
		SS	4	17	50+	3	63.72
		SS	5	25	50+	4	62.72
4.78		SS	6	25	50+		
End of Borehole (GWL @ 2.50m-Oct. 11, 2006)							



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Phse II - Environmental Site Assessment
1445 and 1451 Wellington Street West
Ottawa, Ontario

DATUM TBM - Mag nail in sidewalk, located near the southwest corner of subject site, north side of Wellington Street West. Geodetic elevation = 66.47m, as per AOV.

REMARKS

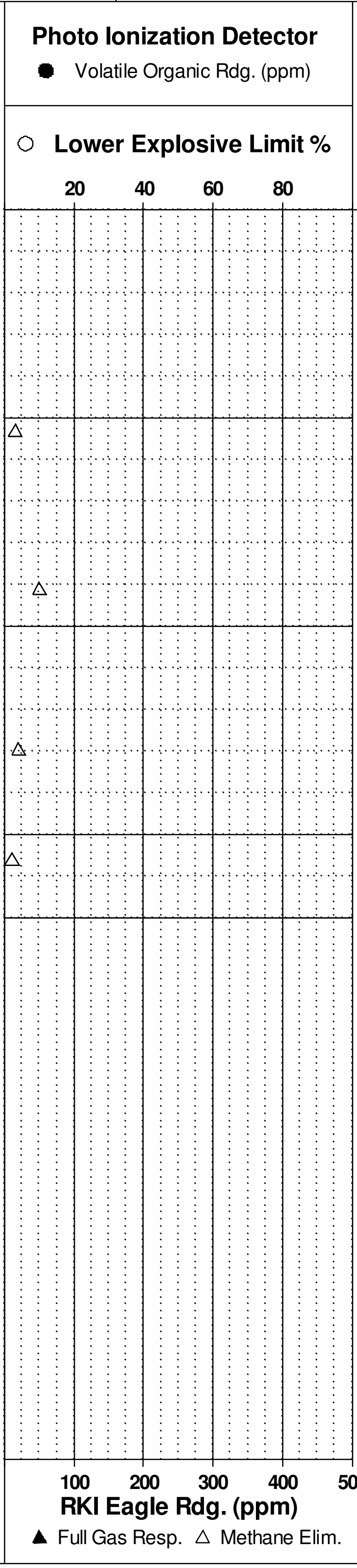
BORINGS BY CME 75 Power Auger

DATE October 6, 2006

FILE NO. PE0982

HOLE NO. BH 3

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	66.55
Asphaltic concrete 0.05							
FILL: Brown silty sand with gravel 0.60							
GLACIAL TILL: Compact, brown silty sand with gravel, cobbles and boulders		SS	1	42	25	1	65.55
		SS	2	62	17	2	64.55
		SS	3	61	20	3	63.55
3.40		SS	4	20	50+		
End of Borehole Practical refusal to augering @ 3.40m depth							



Monitoring Well Construction

SOIL PROFILE AND TEST DATA

Phse II - Environmental Site Assessment
1445 and 1451 Wellington Street West
Ottawa, Ontario

DATUM TBM - Mag nail in sidewalk, located near the southwest corner of subject site, north side of Wellington Street West. Geodetic elevation = 66.47m, as per AOV.

REMARKS

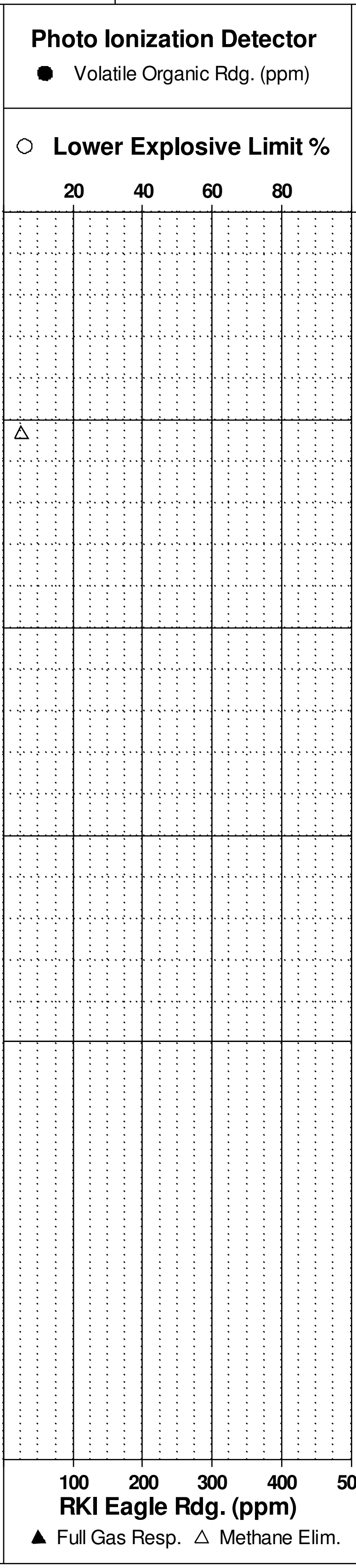
BORINGS BY CME 75 Power Auger

DATE October 6, 2006

FILE NO. PE0982

HOLE NO. BH 4

SOIL DESCRIPTION		SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	66.75
Asphaltic concrete	0.05						
FILL: Dark brown silty sand with gravel and organic matter		SS	1	42	7	1	65.75
	1.68						
GLACIAL TILL: Dense to very dense, brown silty sand with gravel, cobbles and boulders		SS	2	67	36	2	64.75
		SS	3	50	24	3	63.75
		SS	4	50	50+		
		SS	5	33	50+		
	3.99						
End of Borehole Practical refusal to augering @ 3.99m depth							



Monitoring Well Construction

patersongroup Consulting Engineers
154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Phse II - Environmental Site Assessment
1445 and 1451 Wellington Street West
Ottawa, Ontario

DATUM TBM - Mag nail in sidewalk, located near the southwest corner of subject site, north side of Wellington Street West. Geodetic elevation = 66.47m, as per AOV.

REMARKS

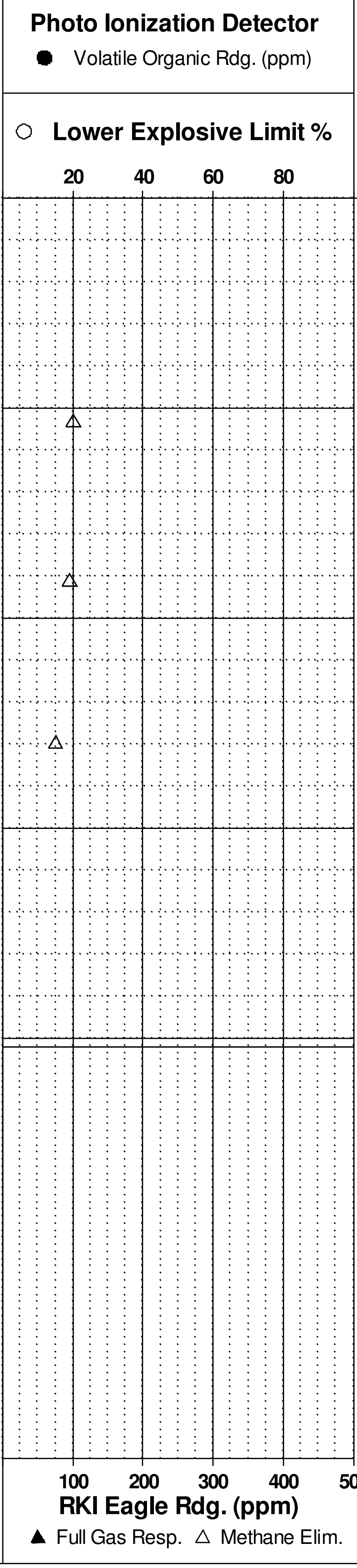
BORINGS BY CME 75 Power Auger

DATE October 6, 2006

FILE NO. PE0982

HOLE NO. BH 5

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	66.68
Asphaltic concrete 0.05							
FILL: Brown silty sand with gravel and crushed stone 0.60							
		SS	1	58	12	1	65.68
GLACIAL TILL: Dense, dark grey silty sand with gravel, cobbles and boulders		SS	2	73	26		
						2	64.68
		SS	3	67	31		
						3	63.68
		SS	4	67	34		
4.04		SS	5	44	50+	4	62.68
End of Borehole Practical refusal to augering @ 4.04m depth							



Monitoring Well Construction

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Phse II - Environmental Site Assessment
1445 and 1451 Wellington Street West
Ottawa, Ontario

DATUM TBM - Mag nail in sidewalk, located near the southwest corner of subject site, north side of Wellington Street West. Geodetic elevation = 66.47m, as per AOV.

REMARKS

BORINGS BY Portable Drill

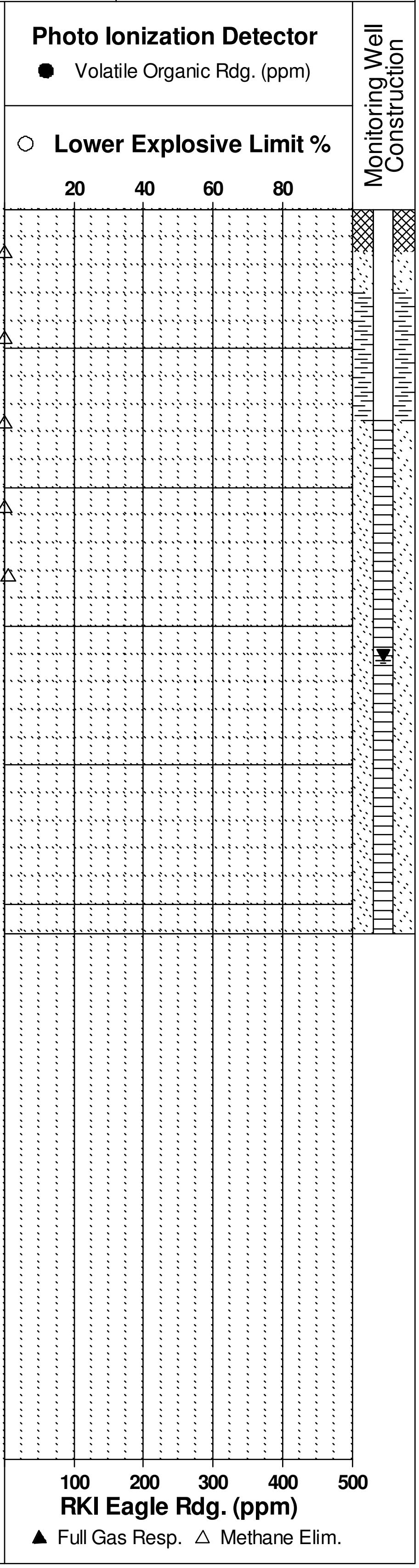
DATE November 18, 2009

FILE NO. PE2526

HOLE NO. BH 6

SOIL DESCRIPTION	TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
FILL: Brown silty sand with gravel	SS	1	46	2
	SS	2	33	4
	SS	3	42	7
	SS	4	50	3
2.82	SS	5	67	
FILL: Gravel with piece of concrete 3.05	RC	1		
BEDROCK: Grey limestone	RC	2	100	0
	RC	3	91	52
	RC	4	64	64
5.21	RC	5	90	77
End of Borehole (GWL @ 3.25m-Nov. 30/09)				

DEPTH (m)	ELEV. (m)
0	66.32
1	65.32
2	64.32
3	63.32
4	62.32
5	61.32



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Phse II - Environmental Site Assessment
1445 and 1451 Wellington Street West
Ottawa, Ontario

DATUM TBM - Mag nail in sidewalk, located near the southwest corner of subject site, north side of Wellington Street West. Geodetic elevation = 66.47m, as per AOV.

REMARKS

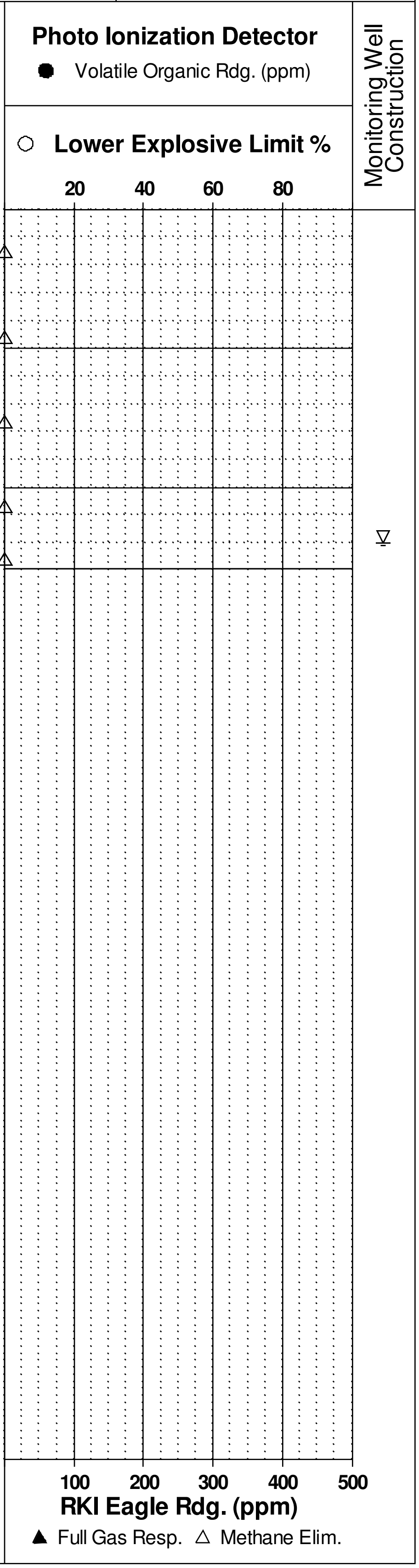
BORINGS BY Portable Drill

DATE November 18, 2009

FILE NO. PE2526

HOLE NO. BH 7

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	66.22
FILL: Brown silty sand with clay		SS	1	38	1		
		SS	2	29	2	1	65.22
1.22							
FILL; Brown silty clay with piece of wood		SS	3	29	3		
		SS	4	58	13	2	64.22
2.59		SS	5	100	17		
End of Borehole							



patersongroup Consulting Engineers
154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Phse II - Environmental Site Assessment
1445 and 1451 Wellington Street West
Ottawa, Ontario

DATUM TBM - Mag nail in sidewalk, located near the southwest corner of subject site, north side of Wellington Street West. Geodetic elevation = 66.47m, as per AOV.

REMARKS

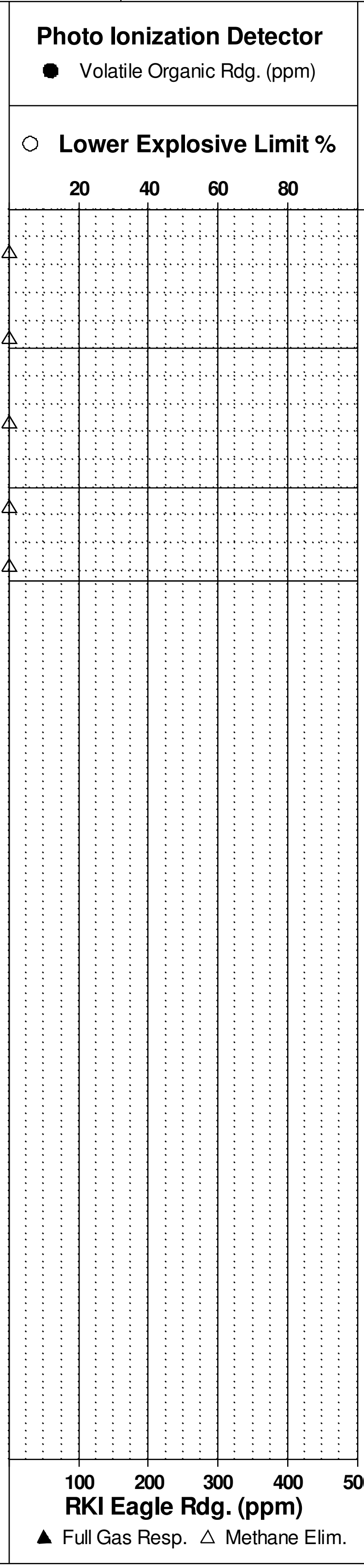
BORINGS BY Portable Drill

DATE November 18, 2009

FILE NO. PE2526

HOLE NO. BH 8

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Monitoring Well Construction
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD			
GROUND SURFACE						0	66.54	
FILL: Brown silty sand with gravel	▩	SS	1	21	4			
	▩	SS	2	25	6	1	65.54	
1.83	▩	SS	3	12	7			
GLACIAL TILL: Compact to dense, brown silty sand with gravel		SS	4	62	5	2	64.54	
2.67		RC	5	78	20			
End of Borehole								



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Phse II - Environmental Site Assessment
1445 and 1451 Wellington Street West
Ottawa, Ontario

DATUM TBM - Mag nail in sidewalk, located near the southwest corner of subject site, north side of Wellington Street West. Geodetic elevation = 66.47m, as per AOV.

REMARKS

BORINGS BY CME 75 Power Auger

DATE November 23, 2009

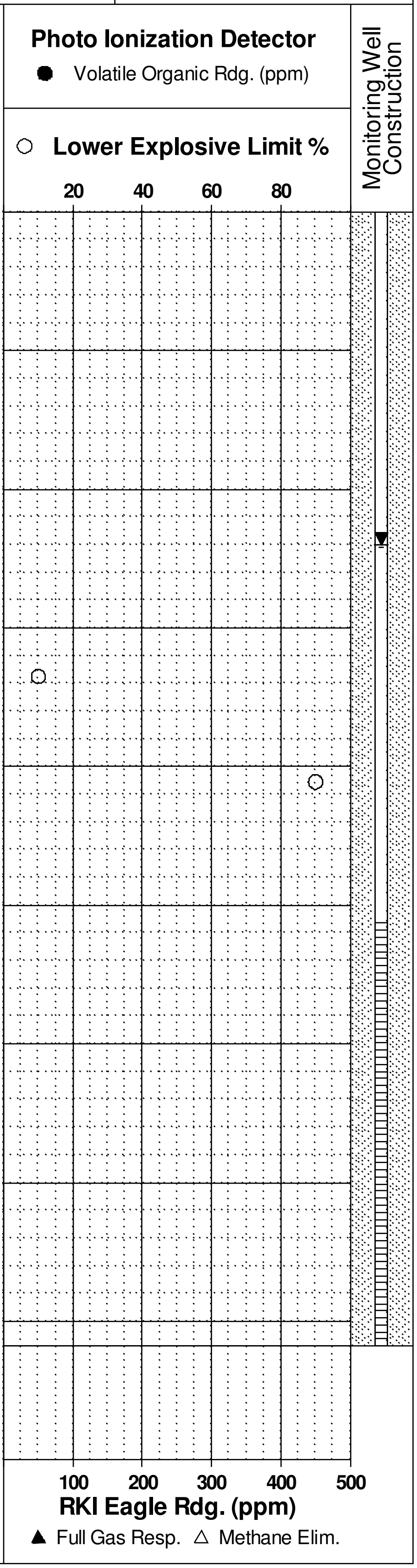
FILE NO. PE2526

HOLE NO. BH 9

SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
13mm Asphaltic concrete over crushed stone **FILL**	0.25
FILL: Brown silty sand with clay and gravel	2.90
GLACIAL TILL: Dense, grey-brown silty sand with gravel	4.14
BEDROCK: Grey limestone	8.18
End of Borehole	
(GWL @ 2.40m-Nov. 30/09)	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	54	6
SS	2	75	15
SS	3	67	50+
SS	4	96	33
SS	5	100	50+
RC	1	100	77
RC	2	100	44
RC	3	100	65
RC	4	100	92

DEPTH (m)	ELEV. (m)
0	66.60
1	65.60
2	64.60
3	63.60
4	62.60
5	61.60
6	60.60
7	59.60
8	58.60



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Phse II - Environmental Site Assessment
1445 and 1451 Wellington Street West
Ottawa, Ontario

DATUM TBM - Mag nail in sidewalk, located near the southwest corner of subject site, north side of Wellington Street West. Geodetic elevation = 66.47m, as per AOV.

REMARKS

BORINGS BY CME 75 Power Auger

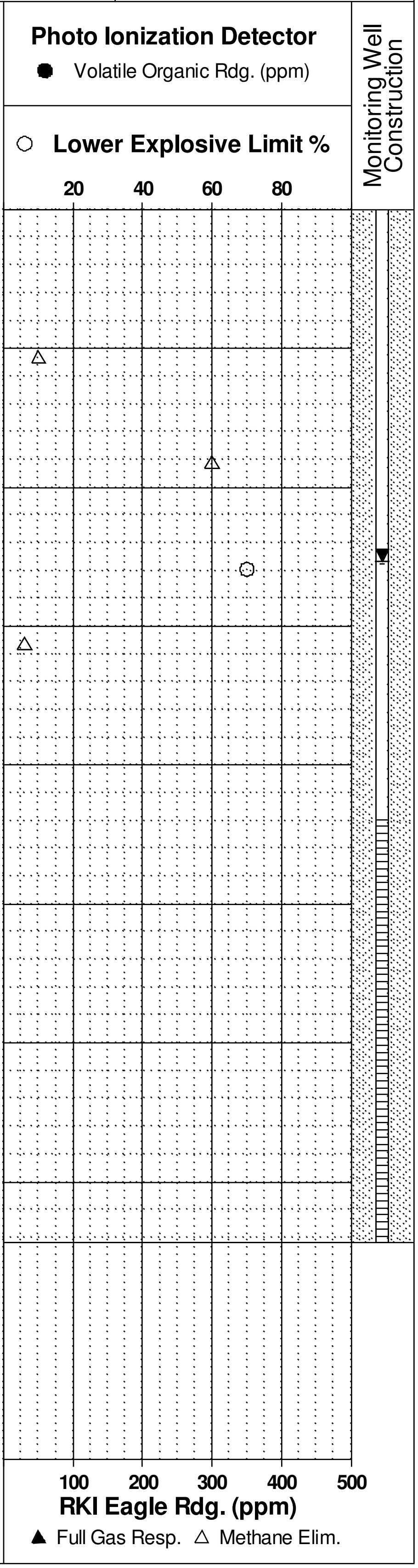
DATE November 23, 2009

FILE NO. PE2526

HOLE NO. BH10

SOIL DESCRIPTION		SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
13mm Asphaltic concrete over crushed stone **FILL**	0.25	AU	1		
FILL: Brown silty sand with clay and gravel		SS	2	25	4
		SS	3	96	26
		SS	3	83	46
	3.20	SS	4	100	50+
BEDROCK: Grey limestone		RC	1	100	84
		RC	2	100	98
	7.44	RC	3	100	87
End of Borehole (GWL @ 2.53m-Nov. 30/09)					

DEPTH (m)	ELEV. (m)
0	66.61
1	65.61
2	64.61
3	63.61
4	62.61
5	61.61
6	60.61
7	59.61



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Phse II - Environmental Site Assessment
1445 and 1451 Wellington Street West
Ottawa, Ontario

DATUM TBM - Mag nail in sidewalk, located near the southwest corner of subject site, north side of Wellington Street West. Geodetic elevation = 66.47m, as per AOV.

REMARKS

BORINGS BY CME 75 Power Auger

DATE November 23, 2009

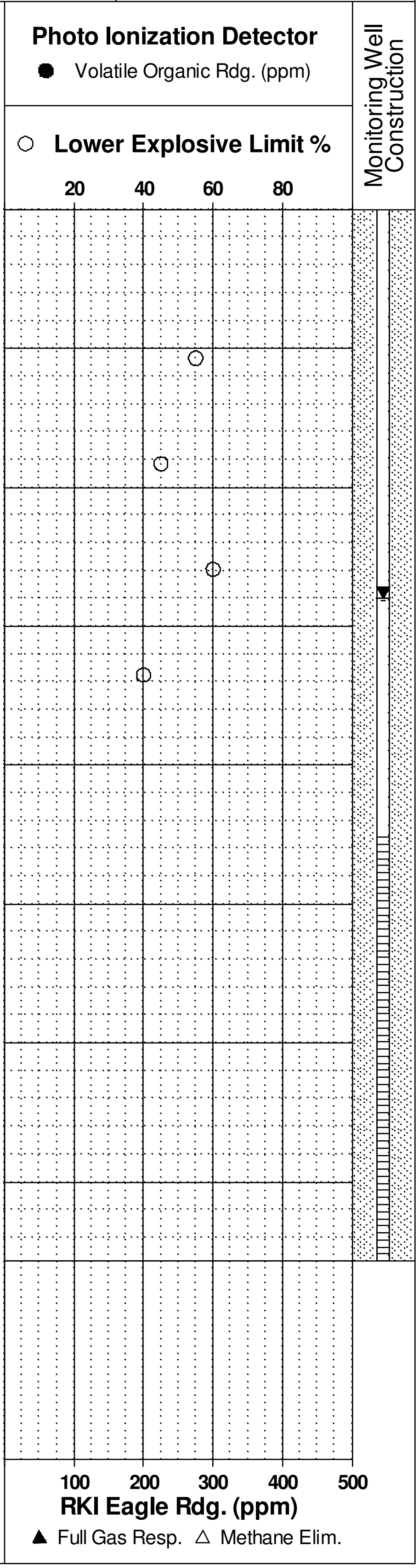
FILE NO. PE2526

HOLE NO. BH11

SOIL DESCRIPTION	
GROUND SURFACE	
13mm Asphaltic concrete over crushed stone **FILL**	0.25
FILL: Brown silty sand with clay and gravel	3.79
BEDROCK: Grey limestone	7.57
End of Borehole (GWL @ 2.80m-Nov. 30/09)	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
SS	1	33	3
SS	2	92	35
SS	3	75	46
SS	4	81	50+
RC	1	56	31
RC	2	97	90
RC	3	98	97

DEPTH (m)	ELEV. (m)
0	66.61
1	65.61
2	64.61
3	63.61
4	62.61
5	61.61
6	60.61
7	59.61



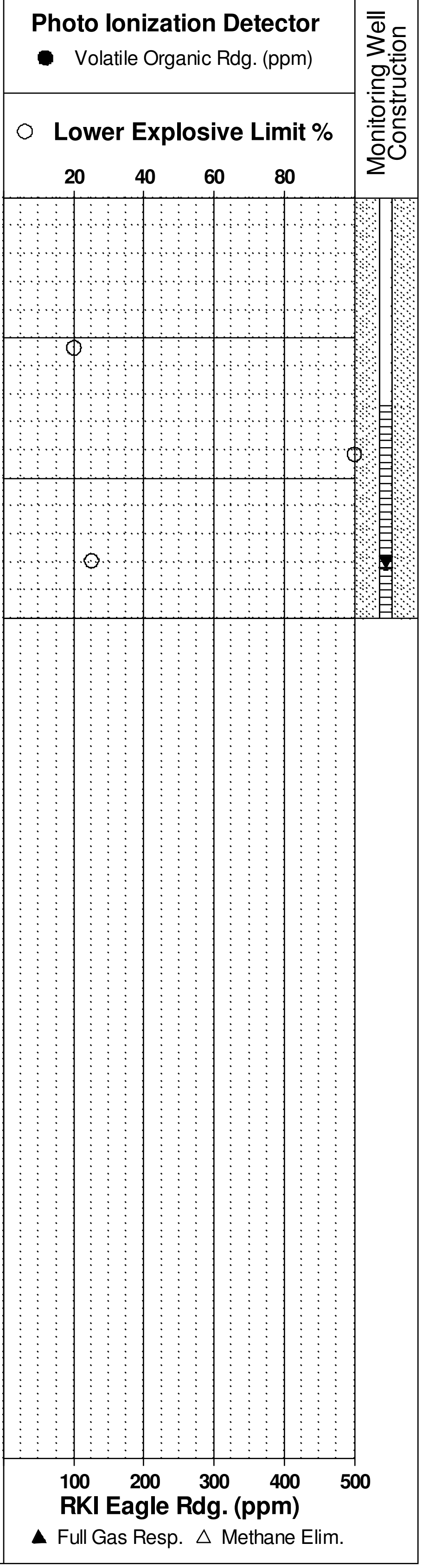
SOIL PROFILE AND TEST DATA

Phse II - Environmental Site Assessment
1445 and 1451 Wellington Street West
Ottawa, Ontario

DATUM	TBM - Mag nail in sidewalk, located near the southwest corner of subject site, north side of Wellington Street West. Geodetic elevation = 66.47m, as per AOV.
REMARKS	
BORINGS BY	CME 75 Power Auger
DATE	November 23, 2009
FILE NO.	**PE2526**
HOLE NO.	**BH12**

SOIL DESCRIPTION		TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
13mm Asphaltic concrete over crushed stone **FILL**	0.23				
FILL: Brown silty sand with clay and gravel		SS	1	67	20
		SS	2	78	50+
	2.29				
GLACIAL TILL: Compact, brown silty sand with clay, gravel, cobbles and boulders		SS	3	96	58
		SS	4	68	36
	3.00				
End of Borehole (GWL @ 2.64m-Nov. 30/09)					

DEPTH (m)	ELEV. (m)
0	66.61
1	65.61
2	64.61
3	63.61



SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

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

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

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

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

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

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

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

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

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

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

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

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

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

CONSOLIDATION TEST

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

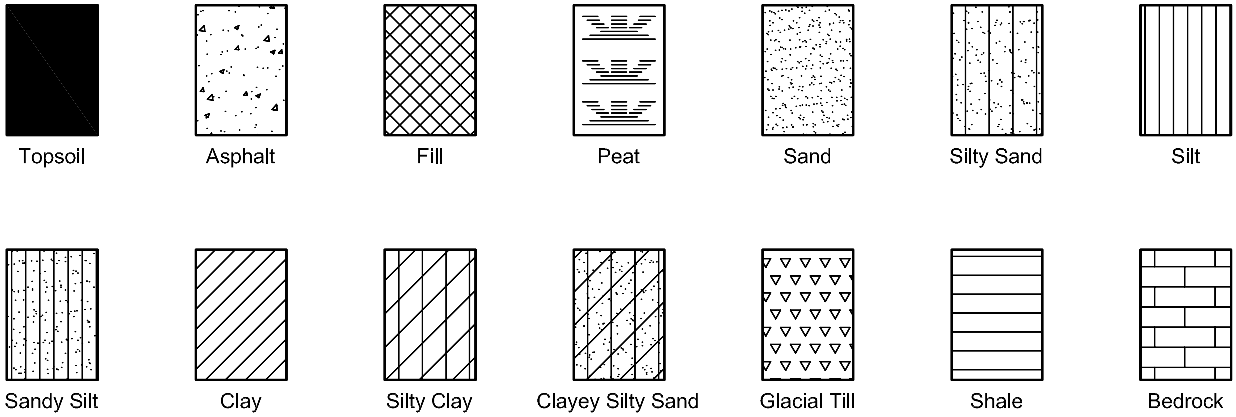
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

PERMEABILITY TEST

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

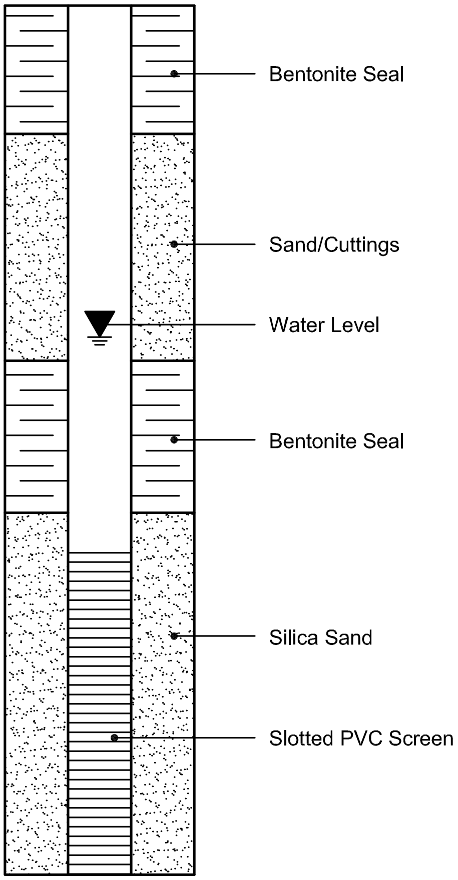
SYMBOLS AND TERMS (continued)

STRATA PLOT

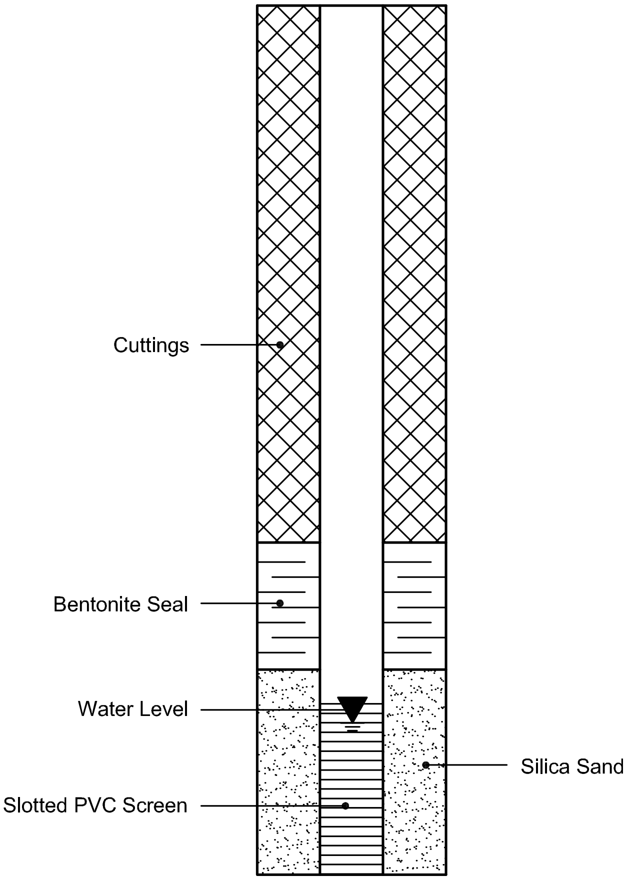


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Report Date: 28-May-2013
Order Date:22-May-2013

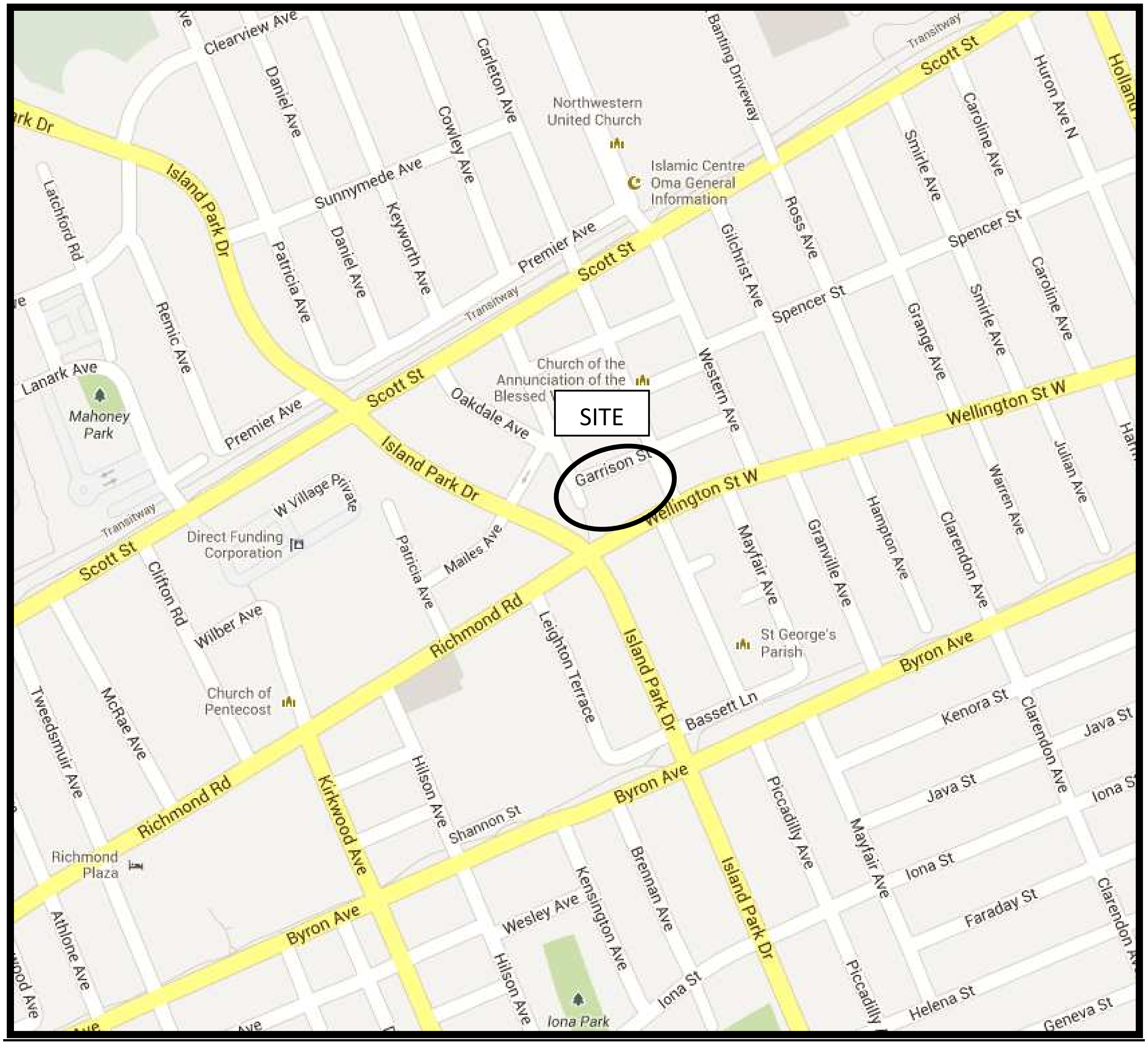
Client: **Paterson Group Consulting Engineers**
Client PO: 14484
Project Description: PG2961

	Client ID:	BH1-SS4	-	-	-
	Sample Date:	13-May-13	-	-	-
	Sample ID:	1321071-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	87.3	-	-	-
General Inorganics					
pH	0.05 pH Units	7.72	-	-	-
Resistivity	0.10 Ohm.m	4.95	-	-	-
Anions					
Chloride	5 ug/g dry	1330	-	-	-
Sulphate	5 ug/g dry	168	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

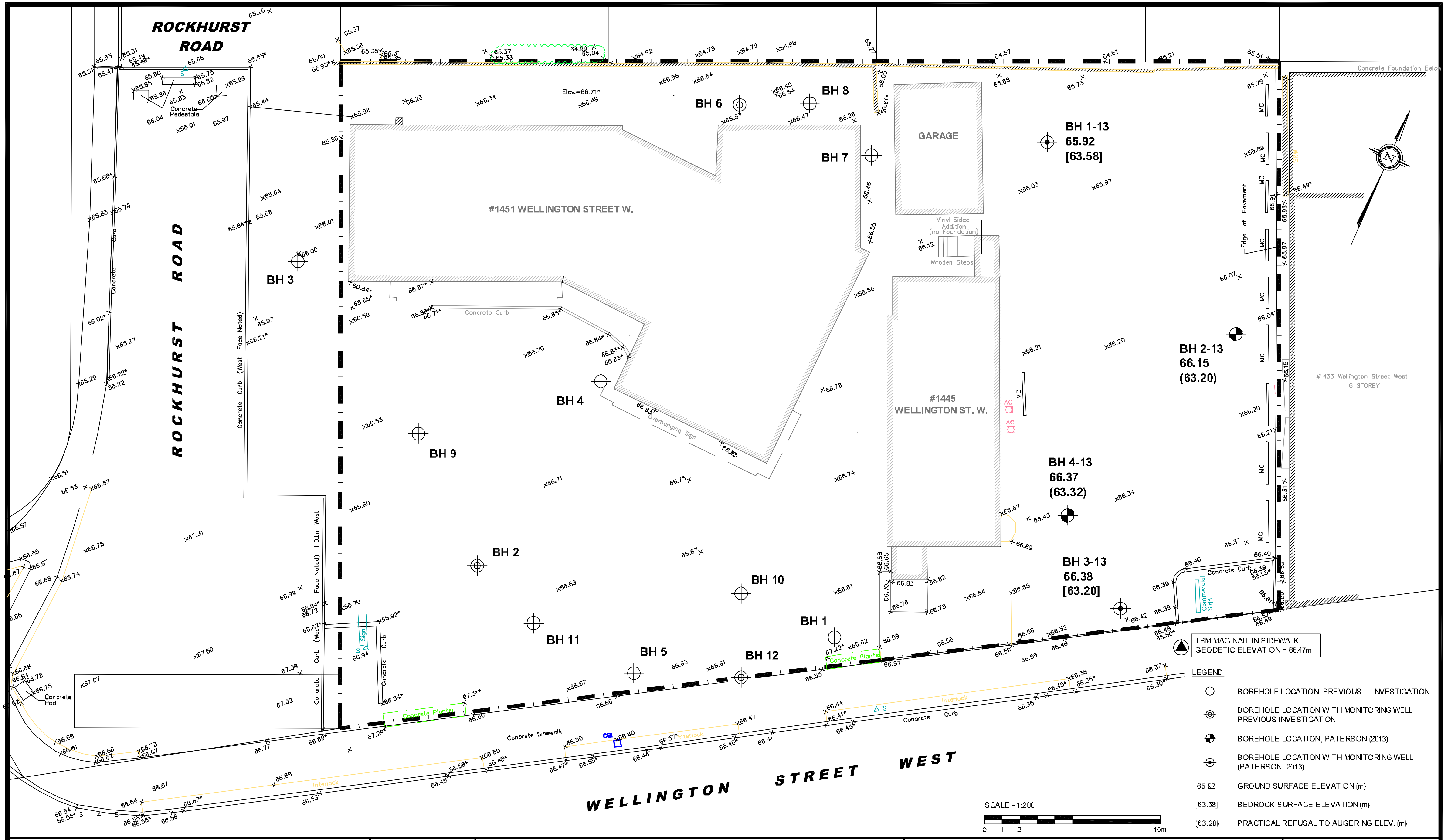
DRAWING PG2961-1 - TEST HOLE LOCATION PLAN



Source: Google Maps

FIGURE 1

KEY PLAN



LEGEND

⌖	BOREHOLE LOCATION, PREVIOUS INVESTIGATION
⌖	BOREHOLE LOCATION WITH MONITORING WELL PREVIOUS INVESTIGATION
●	BOREHOLE LOCATION, PATERSON (2013)
⌖	BOREHOLE LOCATION WITH MONITORING WELL, (PATERSON, 2013)
65.92	GROUND SURFACE ELEVATION (m)
[63.58]	BEDROCK SURFACE ELEVATION (m)
(63.20)	PRACTICAL REFUSAL TO AUGERING ELEV. (m)

paterson group

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

Scale:	1:200
Des.:	JAF
Dwn:	MPG
Chkd:	DG

MIZRAHI DEVELOPMENTS

GEOTECHNICAL INVESTIGATION

PROP. DEVELOPMENT - 1445 & 1451 WELLINGTON ST. W.

OTTAWA, ONTARIO

TEST HOLE LOCATION PLAN

Dwg. No.	PG2961-1
Report No.:	PG2961-1
Date:	06/2013

11x17