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Geotechnical Engineering

Environmental Engineering

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Materials Testing

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Archaeological Services

Geotechnical Investigation

Proposed Multi-Storey Buildings
350 Sparks Street
Ottawa, Ontario

Prepared For

Morguard Investment Ltd.

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Report PG3404-1

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APPENDICES

1.0 INTRODUCTION

Paterson Group (Paterson) was commissioned by Morguard Investments Limited (Morguard) to conduct a geotechnical investigation for the proposed multi-storey buildings to be located at 350 Sparks Street, in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2).

The investigation objectives were to:

- ❏ determine the subsurface soil 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 our findings and includes geotechnical recommendations pertaining to the design and construction of the subject development as understood at the time of writing this report.

2.0 PROPOSED PROJECT

The proposed development will consist of two multi-storey buildings. A 24 storey residential tower with a 6 storey podium is expected within the north central and north west portions of the subject site. A 26 storey hotel with a 3 storey podium is also anticipate within the south west and west portion of the subject site. Associated access lanes, parking stalls and landscaped areas are also anticipated.

Furthermore, the current site has an existing building with two levels of underground parking over most of the site that will be demolished to accommodate the proposed redevelopment. The west portion of the site currently has a combination of one basement and no basement. Furthermore, the new redevelopment will require a third level of underground parking.

It should also be noted that the Ottawa Light Rail Transit (OLRT) Confederation Line is currently under construction within Queen Street and Lyon Street and will be in close proximity to the current redevelopment site located at 350 Sparks Street. The proximity study is being reviewed under separate cover.

3.0 METHOD OF INVESTIGATION

3.1 Field Investigation

Field Program

The field program for the investigation was conducted between May 5 and May 20, 2015. A total of 8 boreholes were advanced to a maximum of 16 m depth within the west and central portions of the subject site. It should be noted that 6 boreholes (BH 2, BH 3 and BH 5 through BH 8) were internally drilled within the underground parking areas of the existing buildings. The locations of the boreholes are illustrated on Drawing PG3404-1 - Test Hole Location Plan included in Appendix 2.

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

Sampling and In Situ Testing

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

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

Rock Samples were recovered in all boreholes using a core barrel and diamond drilling techniques. The bedrock samples were classified on site, placed in hard cardboard core boxes and transported to Paterson’s laboratory. The depths at which rock core samples were recovered from the boreholes are presented as RC on the Soil Profile and Test Data sheets in Appendix 1.

The recovery value and a Rock Quality Designation (RQD) value were calculated for each drilled section of bedrock and presented on the borehole logs. The recovery value is the length of the bedrock sample recovered over the length of the drilled section. The RQD value is the total length of intact rock pieces longer than 100 mm over the length of the core run. The values indicate the bedrock quality.

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.

Groundwater

A 32 mm diameter groundwater monitoring well was installed within all boreholes to monitor the groundwater level subsequent to the completion of the sampling program. The groundwater observations are discussed in Subsection 4.3 and presented in the Soil Profile and Test Data sheets presented in Appendix 1.

3.2 Field Survey

The test hole locations were determined in the field by Paterson personnel with consideration of existing site features. It should be noted that the ground surface elevations are referenced to a temporary benchmark (TBM), consisting of the top of a brass monument located at the north west corner of Sparks Street and Bay Street. A geodetic elevation of 71.211 m was provided by Annis, O'Sullivan, Vollebekk (AOV).

Borehole locations and ground surface elevations at the borehole locations are presented on Drawing PG3404-1-Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

Soil and rock core samples were recovered from the subject site and visually examined in our laboratory to review the field logs. The results are presented on the Soil Profile and Test Data sheets in Appendix 1.

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

3.4 Analytical Testing

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

4.0 OBSERVATIONS

4.1 Surface Conditions

The subject site is currently occupied by a residential building within the central and west portions of the site with the associated parking areas and access lanes. The ground surface across the subject site is relatively flat and at grade with the surrounding roads.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile at the borehole locations consists of Concrete and or pavement structure overlying fill consisting of crushed stone and/or blast rock. Grey limestone bedrock was encountered below the above noted layers at all boreholes.

Bedrock

Limestone with shale partings was encountered at depths ranging between 0.5 and 1.2 m within the interior boreholes and between 1.7 and 3.8 for the exterior boreholes. Bedrock was cored in all borehole locations and extended to below the proposed founding depths. Based on the RQDs of the recovered rock cores, the limestone bedrock can be classified as fair to very poor within the upper 1 m to good to very good quality 1 m below the interior borehole locations.

Based on available geological mapping, the subject site is located in an area where the bedrock consists of interbedded shale and limestone of the Verulam Formation at depths ranging from 0 to 1 m.

4.3 Groundwater

Groundwater level readings were taken at the borehole locations on May 20, 2015. The groundwater measurements are presented in the Soil Profile and Test Data sheets and are summarized in Table 1 on the following page. It is important to note that groundwater level readings could be influenced by surface water infiltrating the backfilled borehole, which can lead to higher water levels than noted during the investigation. Based on these observations at the borehole locations, the long-term groundwater level is expected at an elevation of approximately 65.5 to 66.5 m which is approximately 1 to 2 m depth below the P2 underground parking level.

Groundwater levels are subject to seasonal fluctuations and, therefore, groundwater levels could vary at the time of construction.

Table 1 - Groundwater Level Readings

Borehole Number	Ground Elevation (m)	Groundwater Levels		Recording Date
		Depth (m)	Elevation (m)	
BH 1	72.21	2.77	69.44	May 20, 2015
BH 2	67.23	0.95	66.28	May 20, 2015
BH 3	67.21	1.76	65.45	May 20, 2015
BH 4	72.74	5.43	67.31	May 20, 2015
BH 5	67.85	1.94	65.91	May 20, 2015
BH 6	67.81	3.91	63.90	May 20, 2015
BH 7	67.86	1.46	66.40	May 20, 2015
BH 8	67.85	0.86	66.99	May 20, 2015

5.0 DISCUSSION

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is satisfactory for the proposed development. The proposed multi-storey buildings will be founded by conventional spread footing foundations placed on an clean, surface sounded limestone bedrock, except as noted below.

A third level of underground parking will be incorporated in the final design. Bedrock removal will be required and precautionary methods will be used to avoid vibration issues with the adjacent Confederation Rail Line and station. Foundations for the building structure at the south property line is comprised of caissons to be used to transfer the load to the underlying bedrock to lessen the application of loads to the tunnel and tunnel structures.

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

5.2 Site Grading and Preparation

Stripping Depth

All overburden soil will be removed from the proposed building and parking garage building footprint to accommodate the proposed development.

Bedrock Removal

Bedrock removal could be carried out by hoe-ramming where only small quantities of bedrock need to be removed. Otherwise, line drilling and controlled blasting could be used. However, prior to considering blasting, the blasting effects and potential damage to existing adjacent structures should be addressed.

Excluding the areas adjacent to the tunnel and station, as a general guideline, peak particle velocities should not exceed 50 mm/sec (measured at the structures) during the blasting program to reduce the risks of damage to the existing structures.

The blasting operations should be planned and carried out under the supervision of a licensed professional engineer who is also a blasting expert.

A pre-blast or preconstructing survey of the existing surrounding structures should be carried out prior to commencing site blasting 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 basting operations.

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.

Bedrock Grinding along the Perimeter and Adjacent to the Tunnel

For the bulk of the bedrock removal, line drilling and controlled blasting will be used. However, in close proximity to the perimeter of the excavation and adjacent to the tunnel section, hoe ramming will be used to remove bedrock and shape the bedrock vertical surface. Within the final vertical surface (150 to 300 mm from finished surface), rock grinding will be implemented to provide a smoother surface and avoid bedrock over breakage resulting from the higher energy being emitted by the hoe ramming equipment. Similar procedures are proposed at the north portion of the site where the existing two level high foundation wall is to remain.

The full diameter of the line drilled holes should be incorporated in the grinded portion to provide a smoother vertical surface for the application of waterproofing and drainage medium.

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 the source of vibrations: hoe ram, compactor, dozer, crane, truck traffic, etc. 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 development.

Fill Placement

Fill used for grading beneath the proposed buildings, unless otherwise specified, should consist of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A, Granular B Type II or acceptable excavated bedrock from the environmental program. The fill should be tested and approved prior to delivery to the site. The granular material should be placed in lifts at a maximum of 300 mm thick and compacted using suitable compaction equipment for the lift thickness. Fill placed beneath the buildings should be compacted to 98% of the Standard Proctor Maximum Dry Density (SPMDD).

Non-specified existing fill along with site-excavated soil could be used as general landscaping fill where settlement of the ground surface is of minor concern. These materials should be spread in thin lifts and compacted by the tracks of the spreading equipment to minimize voids. If these materials are to be placed to build up the subgrade level for areas to be paved, the material should be compacted in thin lifts to of 95% of the SPMDD. Non-specified existing fill and site-excavated soils are not suitable for use as backfill against foundation walls unless a composite drainage blanket connected to a perimeter drainage system is provided.

If excavated rock is to be used as fill, it should be suitably fragmented to produce a well-graded material with a maximum particle size of 150 mm. The material should be only used structurally to build up the subgrade for roads and pavements. Where the fill is open-graded, a building layer of finer granular fill or a woven geotextile may be required to prevent adjacent finer materials from migrating into voids, with associated loss of ground and settlement.

It should be noted that the site-excavated materials, as well as OPSS Granular B Type I materials can become very difficult to work with under adverse weather conditions such as below zero temperature and wet conditions (e.g. rainfalls, thawing periods, etc.). Under these conditions, consideration should be given to using more suitable materials, such as well-graded OPSS Granular A crushed stone and Granular B Type II materials.

5.3 Foundation Design

Bearing Resistance Values

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 designed to it if founded on limestone bedrock 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 footing footprint(s). One drill hole should be completed per footing. The drill hole inspection should be completed by the geotechnical consultant.

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

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.

In the upper levels of the bedrock where the weathered and highly fractured bedrock may be encountered, the bearing medium will require a lateral support zone of 1H:1V (or flatter). The weathered portion of the bedrock is a relatively thin layer (in most cases less than 0.5 m) and is considered to behave similar to a soil condition.

Caisson Rock Socket Capacities

To extend the building loads to the underside of the tunnel and station elevations, consideration is being given to transferring the loads via rock-socketed caissons. The design of rock-socketed caissons can be based on the toe capacity only, a combination of toe and socket shear (i.e. side wall friction/bond), or socket shear only. For all but very short sockets, the socket shear or bond on the sides of the socket is generally fully mobilized before the toe bearing reaches its maximum. For this project, the length to diameter ratio of the sockets are expected to be large, so the use of a design method using socket shear only, and relying on the toe bearing only for residual support, is recommended. This is a practical and well-accepted approach.

It is our opinion that the caissons should be designed using a socket shear resistance at SLS value of 800 kPa. There should be no direct allowance for toe bearing resistance contribution, but assuming that some residual toe capacity may need to be mobilized, so cleaning the bottom of the caisson is recommended.

A factored socket shear resistance at ULS value of 1,000 kPa is recommended, incorporating a geotechnical resistance factor of 0.4. Note that higher socket shear resistance values could potentially be achieved if full scale loads testing is undertaken.

The required length of the socket in sound rock will be based on assigning the socket shear resistance at SLS value of 800 kPa to the side wall contact area of the socket with the bedrock. The design SLS geotechnical capacity, Qa will, therefore, be:

$$Qa = 800 \pi \, Ds \, Ls$$

Where:
- Qa = SLS caisson capacity (kN)
- Ds = Diameter of socket (m)
- Ls = Length of socket in sound bedrock (m)

The socket geometry (length to diameter ratio) and the modulus ratio between the rock and concrete has a significant effect on the distribution of load between the side wall (i.e. socket shear) component and the toe bearing component. The greater the length to diameter ratio of the socket, the greater the proportion of the load that is carried by the socket shear.

Considering the relative stiffness of the caisson and the rock, it is estimated that approximately 93% of the caisson capacity will tend to be taken by side wall socket shear for a length to diameter (Ls/Ds) ratio of 2 and 98% for a Ls/Ds ratio of 4. As such, long sockets, with an Ls/Ds ratio approaching or exceeding 4 will tend to have all the load carried by socket shear, but shorter sockets will tend to have more of a combined side wall shear and toe bearing effect, if the toe is clean and able to provide the toe resistance.

Based on our preliminary discussions, the length to diameter ratios of the caissons will be large, so to resistance is not expected to be a significant contributor to the caisson capacities.

Clean socket side walls are required to be prepared for all caissons. If residual toe-bearing is part of the design capacity, it will be a requirement that the toe bearing surface of the caisson be properly cleaned.

5.4 Design for Earthquakes

Shear wave velocity testing was completed for the subject site to accurately determine the applicable seismic site classification for the proposed building from Table 4.1.8.4.A of the Ontario Building Code 2012. The shear wave velocity testing was completed by Paterson personnel. The results of the shear wave velocity test are attached to the present letter report.

Field Program

The seismic array testing location was placed within the lowest underground parking level on the north side of the subject structure, as presented in PG3404 -1 - Test Hole Location Plan included in Appendix 2. Paterson field personnel placed 24 horizontal 4.5 Hz. geophones mounted to the surface by means of two 75 mm ground spikes attached to the geophone land case. The geophones were spaced at 2 m intervals and connected by a geophone spread cable to a Geode 24 Channel seismograph.

The seismograph was also connected to a computer laptop and a hammer trigger switch attached to a 12 pound dead blow hammer. The hammer trigger switch sends a start signal to the seismograph. The hammer is used to strike an I-Beam seated into the ground surface, which creates a polarized shear wave. The hammer shots are repeated between five to ten times at each shot location to improve signal to noise ratio. The shot locations are also completed in forward and reverse directions (i.e.- striking both sides of the I-Beam seated parallel to the geophone array). The shot locations are located at 3 m, 4 m, and 20.0 m away from the first geophone, 3 m, 4 m, and 25 m away from the last geophone, and one location was at the centre of the seismic array.

The methods of testing completed by Paterson are guided by the standard testing procedures used by the expert seismologists at Carleton University and Geological Survey of Canada (GSC).

Data Processing and Interpretation

Interpretation for the shear wave velocity results were completed by Paterson personnel. Shear wave velocity measurement was made using reflection/refraction methods. The interpretation is performed by recovering arrival times from direct and refracted waves. The interpretation is repeated at each shot location to provide an average shear wave velocity, Vs_{30}, of the upper 30 m profile, immediately below the building's foundation. The layer intercept times, velocities from different layers and critical distances are interpreted from the shear wave records to compute the bedrock depth at each location. The bedrock velocity was interpreted using the main refractor wave velocity, which is considered a conservative estimate of the bedrock velocity due to the increasing quality of the bedrock with depth. It should be noted that as bedrock quality increases, the bedrock shear wave velocity also increases.

Based on test hole information and available geological data, the bedrock underlaying the existing structure is a sound limestone of the Lindsay Formation. During the construction of the existing structure, the top layer of bedrock was removed providing an increased quality of bedrock for testing purposes. Based on the test results, the bedrock seismic shear wave velocity is 3304 m/s. The Vs_{30} was calculated using the standard equation for average shear wave velocity from the Ontario Building Code (OBC) 2012.

The Vs_{30} was calculated using the standard equation for average shear wave velocity calculation from the Ontario Building Code (OBC) 2012, as presented below.

$$V_{s30} = \frac{Depth_{OfInterest}(m)}{\sum\left(\frac{(Depth_i(m))}{Vs_i(m/s)}\right)}$$

$$V_{s30} = \frac{30m}{\left(\frac{30.0m}{3{,}304m/s}\right)}$$

$$V_{s30} = 3{,}304m/s$$

Based on the results of the seismic testing, the average shear wave velocity, Vs_{30}, for the existing shallow foundations at the subject site is **3,304 m/s**. Therefore, a **Site Class A+** is applicable for design of the proposed building, as per Table 4.1.8.4.A of the OBC 2012. The soils underlying the subject site are not susceptible to liquefaction.

5.5 Basement Slab

With the removal of all topsoil and deleterious fill, such as those containing organic materials, within the footprint of the proposed buildings, the underlying bedrock surface will be considered to be an acceptable subgrade on which to commence backfilling for floor slab construction.

It is expected that the basement area for the proposed multi-storey building will be mostly parking and the recommended pavement structure noted in Subsection 5.7 will be applicable. However, if storage or other uses of the lower level where a concrete floor slab will be used, it is recommended that the upper 200 mm of sub-slab fill consists of 19 mm clear crushed stone. It is recommended that the upper 200 mm of sub-slab fill consist of OPSS Granular A crushed stone for slab on grade construction. All backfill material within the footprint of the proposed buildings should be placed in maximum 300 mm thick loose layers and compacted to at least 98% of its SPMDD.

5.6 Basement Wall

It is understood that the basement walls are to be poured against a composite drainage system, which will be placed against the exposed bedrock face. Lateral earth pressures are expected to be negligible for the majority of the basement wall height due to the anticipated method of construction. It is expected that the drainage system will provide adequate space for wall deflection during an earthquake event.

Where soil is to be retained, in our opinion, 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. The applicable effective unit weight of the material is 13 kN/m^3, where applicable.

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. Due to the wall deflections expected during a seismic event, a basement wall is considered to be a yielding structure. For a typical building (not designed to post-disaster standards), walls are designed for a deflection limit of $0.025H_s$ (where H_s is the interstorey height). It should also be noted that the magnitude of wall rotation required to reach an active earth pressure state is 0.004H and to reach a passive earth pressure state is 0.06H for a loose, cohensionless soil. The total earth pressure (P_{AE}) includes both the static earth pressure component (P_A) and the seismic component (ΔP_{AE}).

Lateral Earth Pressures

The static horizontal earth pressure (P_A) can be calculated using a triangular earth pressure distribution equal to $K_{ah}\ \gamma\ H$ where:

K_{ah} = active earth pressure coefficient of the applicable retained soil
γ = unit weight of the fill of the applicable retained soil (kN/m^3)
H = height of the wall (m)

There are several combinations of backfill materials and retained soils that could be applicable for the retaining wall. However, it is our opinion that, provided free-draining granular backfill is used, 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, γ, of 20 kN/m^3.

For a yielding wall, the active earth pressure can be calculated using an active earth pressure coefficient, K_{ah}, of 0.33 for the free-draining granular backfill described above and a horizontal backfill profile.

Seismic Earth Pressures

The seismic earth pressure (ΔP_{AE}) can be calculated using the earth pressure distribution equal to $0.375 a_c\ \gamma H^2/g$ where:

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

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

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

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

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

5.7 Pavement Structure

Car only parking areas and access lanes are anticipated at this site. The proposed pavement structures are presented in Tables 2 and 3.

Table 2 - Recommended Pavement Structure - Car Only Surface 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 3 - Recommended Pavement Structure - Truck Access Lanes

Thickness (mm)	Material Description
40	**Wear Course -** HL-3 or Superpave 12.5 Asphaltic Concrete
50	**Binder Course -** HL-8 or Superpave 19.0 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
400	**SUBBASE** - OPSS Granular B Type II
	SUBGRADE - 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 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 98% of the SPMDD using suitable compaction equipment.

Pavement Structure Drainage

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

5.8 Rock Anchor Design

Overview of Anchor Features

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 a 60 to 90 degree pullout of rock cone 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 individual anchor. A third failure mode of shear failure along the grout/steel interface should be reviewed by the structural engineer to ensure all typical failure modes have been reviewed.

Centre-to-centre spacing between anchors should be at least four times the anchor hole diameter and greater than 1/5 of the total anchor length (minimum of 1.2 m) to lower the 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 does not flow from one hole to an adjacent empty one.

The anchor be provided with a bonded length at the base of the anchor which will provide the anchor capacity, as well an unbonded length between the rock surface and the top of the bonded length.

Permanent anchors should be provided with corrosion protection. As a minimum, the entire drill hole should be filled with cementious grout. The free anchor length is provided by installing a plastic sleeve to act as a bond break, with the sleeve filled with grout or a corrosion inhibiting mastic. Double corrosion protection can be provided with factory assembled systems, such as those available from Dywidag Systems International or Williams Form Engineering Corp. Recognizing the importance of the anchors for the long term performance of the foundation of the proposed buildings, the rock anchors for this project are recommended to be provided with double corrosion protection.

Grout to Rock Bond

The Canadian Foundation Engineering Manual recommends a maximum allowable grout to rock bond stress (for sound rock) of 1/30 of the unconfined compressive strength(UCS) of either the grout or rock (but less than 1.3 MPa) for an anchor of minimum length (depth) of 3 m. Generally, the UCS of limestone ranges between about 31 and 126 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 0.4, can be calculated. A minimum grout strength of 40 MPa is recommended.

Rock Cone Uplift

As discussed previously, the geotechnical capacity of the rock anchors depends on the dimensions of the rock anchors and the configuration of the anchorage system. Based on existing bedrock information, a **Rock Mass Rating (RMR) of 65** was assigned to the bedrock, and Hoek and Brown parameters (**m and s**) were taken as **0.575 and 0.00293**, respectively.

Recommended Rock Anchor Lengths

Parameters used to calculate rock anchor lengths are provided in Table 4.

Table 4 - 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 Shale Hoek and Brown parameters	65 m=0.575 and s=0.00293
Unconfined compressive strength - Shale	60 MPa
Apex angle of failure cone	60^{o}
Apex of failure cone	mid-point of fixed anchor length

The fixed anchor length will depend on the diameter of the drill holes. Recommended anchor lengths for a 75 mm and 125 mm diameter hole are provided in Table 5. The factored tensile resistance values given in Table 5 are based on a single anchor with no group influence effects. A detailed analysis of the anchorage system, including potential group influence effects, could be provided once the details of the loading for the proposed buildings are determined.

Table 5 - Recommended Rock Anchor Lengths				
Diameter of Drill Hole (mm)	**Anchor Lengths (m)**			**Factored Tensile Resistance (kN)**
	Bonded Length	**Unbonded Length**	**Total Length**	
75	1.9	2.1	4	440
	2.6	2.2	4.8	600
	3.6	2.3	5.9	850
	5	2.3	7.3	1200
	6	3.5	9.5	2000
125	4.5	3.5	8	2000

Other considerations

The anchor drill holes should be within 1.5 to 2 times the rock anchor tendon diameter, inspected by geotechnical personnel and should be flushed clean prior to grouting. A tremie tube is recommended to place grout from the bottom of the anchor holes. Compressive strength testing is recommended to be completed for the rock anchor grout. A set of grout cubes should be tested for each day that grout is prepared.

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 PROXIMITY ASSESSMENT

6.1 Introduction

The proposed development along the Confederation Line will consist of a 20 storey building placed immediately adjacent to the property boundary over an existing foundation footprint for a significant portion of the alignment. The west portion of the alignment along Queen Street will require additional excavation due to the existing residential dwelling. The current existing hi-rise building has two levels of underground parking. The new redevelopment will add a third level of underground parking.

6.2 Subsurface Conditions

Based on available geotechnical information along the tunnel alignment, the subsurface conditions consist of the following:

- Existing surface grade is at an elevation of approximately 73.8 m.
- The overburden thickness is approximately 2.6 m.
- Bedrock surface elevation is at approximately 71.2 m.
- The bedrock underlying the site consists of strong to very strong limestone rock material, with unconfined compressive strengths generally in excess of 80 MPa.
- The limestone at and below the proposed excavation (founding) level at an elevation of approximately 63.8 m is identified by Thurber, in their borehole BH-6R, to be of the Lindsay Formation to elevation 60.5 m, followed by the Verulam Formation.
- Bedrock RQD values are shown as 100 for the full depth of the borehole below the founding level and solid core recoveries are shown to range from 30 to 100%.

6.3 Proximity Study

Tunnel Location

Based on available information, it appears that the existing LRT tunnel is approximately 4 m from the 350 Sparks Street property line. The founding elevation of the existing LRT tunnel is approximately 53.1 m.

Shoring the Overburden

Since the overburden is relatively shallow (approximately 2.5 m) and is currently supported by the parking garage foundation wall, the intent is to utilize the existing foundation wall as the shoring system. A grid pattern of tie backs will be used to provide lateral support of the existing foundation wall until the new foundation is poured. From a geotechnical perspective, this is an acceptable approach and will be the least disruptive approach to the excavation program and the nearby LRT tunnel.

Influence of Proposed Development on Tunnel

From a geotechnical perspective and if founded on conventional spread footing foundations, the proposed founding elevation of the third underground parking level would most likely generate lateral loads that would extend within the zone of influence along the north face of the tunnel.

To eliminate the lateral loads within the zone of influence on the LRT tunnel, the building loads must be transferred to an elevation similar to the existing tunnel founding elevation (53.1 m).

Load Transfer to the Underlying Bedrock

To transfer the load to the underlying bedrock elevation, drilled caissons will be required. The drilled caisson (approximately 760 mm in diameter) will be drilled at specific locations in close proximity to the property line. The lower 2.5 m portion of the caisson will be socketed in the bedrock to develop the required shear friction.

The remaining upper portion of the caisson will be isolated from developing shear forces along the sidewalls. To accomplish this, a sonotube will be placed within the drilled caisson prior to pouring concrete above the socketed portion. The voids within the annular space between the sonotube and drilled bedrock will be grouted to provide lateral stability.

The direct transfer of the loads to the underlying bedrock starting at an elevation of approximately 55.6 m to the tunnel founding elevation of 53.1 m will eliminate any potential short and long term lateral loading within the zone of influence of the tunnel.

6.4 Conclusions and Recommendations

Based on a geotechnical review of the underlying bedrock conditions and the proximity of the Confederation LRT Line along Queen Street, Paterson can conclude that the proposed foundation load transfer to the underlying bedrock at an elevation of 55.6 m or deeper, will have no influence on the existing tunnel. Any shear friction developed in the proposed drilled caissons will be outside the zone of influence of the tunnel.

Based on the criteria provided in the Confederation Line Proximity Study Guidelines dated October 23, 2013, the proposed redevelopment of the subject site with the proposed drilled caissons is not expected to have any influence of the existing tunnel. Therefore, based on the criteria set forth in the above guidelines, the subject redevelopment will be considered as a Level 1 Project.

It is recommended that the as built tunnel information should be provided when completing the foundation design and to confirm the conclusions noted above.

7.0 DESIGN AND CONSTRUCTION PRECAUTIONS

7.1 Foundation Drainage and Backfill

For the proposed development, it is expected that the footprint of the parking garage will occupy most of the site and foundation walls will be blind poured against the bedrock and existing foundation walls. Furthermore, due to the close proximity of the light rail tunnel, the groundwater levels at the subject site will be influenced by the drained portion of the tunnel (non-tanked detail) over the long term.

Foundation Drainage

A perimeter foundation drainage system is recommended to be provided for the proposed structure. Insufficient room is expected to be available for exterior backfill. The system could be as follows:

- ❏ A waterproofing membrane should be applied to the prepared vertical bedrock surface for the portion of the garage extending below P-2 level to the founding elevation. The membrane will serve as a water infiltration suppression system.

- ❏ Composite drainage layer will be placed from the surface to the proposed founding elevation.

The composite drainage system (such as Miradrain G100N or equivalent) is recommended to extend to the footing level. Sleeves, 150 mm in diameter, at 3 m centres should be placed 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

Underfloor drainage is recommend to control water infiltration due to groundwater infiltration at the proposed founding elevation. For design purposes, Paterson recommends a 150 mm in diameter perforated pipes be placed at 6 to 8 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.

Adverse Effects of Dewatering on Adjacent Properties

Depending on the foundation level, the proposed development could be founded below the long term groundwater level. If founded below the long term groundwater table, a waterproofing membrane is recommended to lessen the effects of water infiltration. Any minor dewatering will be within the bedrock layer which is considered relatively shallow at the subject site. Therefore, adverse effects to the surrounding buildings or properties are not expected with the lowering of the groundwater in this area.

The existing LRT tunnel construction adjacent to the subject site will consist of a non-tanked tunnel which will permit limited groundwater infiltration within the lower portion of the tunnel which will allow the depressurization of the groundwater in the bedrock in the vicinity of the tunnel. Therefore, any development adjacent to the tunnel founded directly on the bedrock will have no adverse effects on the surrounding properties.

Foundation Backfill

Above the bedrock surface and in areas requiring backfilling, 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 placement as backfill against the foundation walls, unless placed 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.

7.2 Protection of Footings Against Frost Action

The parking garage is expected to 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.

Perimeter footings of heated structures are required to be insulated against the deleterious effects of frost action. A minimum of 1.5 m of soil cover alone, or a minimum of 0.6 m of soil cover, in conjunction with adequate foundation insulation, should be provided in this regard. More details regarding foundation insulation can be provided, if requested.

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

7.3 Excavation Side Slopes

The side slopes of excavations in the soil and fill overburden materials should either be excavated to acceptable slopes or should be retained by shoring systems from the beginning of the excavation until the structure is backfilled. It is assumed that sufficient room will be available for the greater part of the excavation to be constructed by open-cut methods (i.e. unsupported excavations).

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be excavated at 1H:1V or flatter. The flatter slope is recommended for excavation below groundwater level. The subsurface soils are considered to be a Type 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 a safe distance from the excavation sides.

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

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

In bedrock, almost vertical side slopes can be used provided that all loose rock and blocks with unfavourable weak planes are removed or stabilized.

Temporary Shoring

Temporary shoring will be required to support the overburden soils. The design and implementation of these temporary systems will be the responsibility of the excavation contractor and their design team. Inspections and approval of the temporary system will also be the responsibility of the designer. Geotechnical information provided below is to assist the designer in completing a suitable and safe shoring system. The designer should take into account the potential for a fully saturated condition following a significant precipitation event allowing for full hydrostatic pressure in the design. Any changes to the approved shoring design system should be reported immediately to the owner’s representative prior to implementation.

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, the shoring systems should be provided with tie-back rock anchors to ensure the stability. The toe of the shoring 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 used.

The earth pressures acting on the shoring system may be calculated using 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. In addition, at-rest pressure is required at the south property line as required by the OLRT Proximity Guidelines.

The dry unit weight should be used above the groundwater level while the effective unit weight should be used below the groundwater level.

The hydrostatic groundwater pressure should be added to the earth pressure distribution wherever the effective unit weights are used for earth pressure calculations. If the groundwater level is lowered, the dry unit weight for the soil should be used full weight, with no hydrostatic groundwater pressure component.

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

7.4 Pipe Bedding and Backfill

The pipe bedding for sewer and water pipes should consist of at least 150 mm of OPSS Granular A material. Where the bedding is located within the firm grey silty clay, the thickness of the bedding material should be increased to a minimum of 300 mm. The material should be placed in maximum 300 mm thick lifts and compacted to a minimum of 95% of its SPMDD. The bedding material should extent at least to the spring line of the pipe.

The cover material, which should consist of OPSS Granular A, should extend from the spring line of the pipe to at least 300 mm above the obvert of the pipe. The material should be placed in maximum 300 mm thick lifts and compacted to a minimum of 95% of its SPMDD.

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

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

To reduce long-term lowering of the groundwater level at this site, clay seals should be provided in the service trenches. The seals should be at least 1.5 m long and should extend from trench wall to trench wall. Generally, the seals should extend from the frost line and fully penetrate the bedding, subbedding and cover material. The barriers should consist of relatively dry and compactable brown silty clay placed in maximum 225 mm thick loose layers and compacted to a minimum of 95% of the material’s SPMDD. The clay seals should be placed at the site boundaries and at strategic locations at no more than 60 m intervals in the service trenches.

7.5 Groundwater Control

It is anticipated that groundwater infiltration into the excavations should be low and controllable using open sumps. Pumping from open sumps should be sufficient to control the groundwater influx through the sides of shallow excavations. 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.

A temporary MOE permit to take water (PTTW) will be required for this project if more than 50,000 L/day is to be pumped during the construction phase. At least 4 to 5 months should be allowed for completion of the application and issuance of the permit by the MOE.

7.6 Winter Construction

If winter construction is considered for the project specific precautions should be implemented. The subsurface soil conditions consist of frost susceptible materials. In the presence of water and freezing conditions, ice could form within the soil mass. Heaving and settlement upon thawing could occur.

In the event of construction during below zero temperatures and in areas of auxiliary structures founded on soil, the bearing stratum should be protected from freezing temperatures by the use 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 taken if such activities are to be completed during freezing conditions.

8.0 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:

- ❑ Observation of all bearing surfaces prior to the placement of concrete.
- ❑ Inspection and review of caisson installations.
- ❑ Review of the geotechnical aspects of the excavating contractor's shoring design, prior to construction.
- ❑ Review the bedrock stabilization and excavation requirements.
- ❑ Review proposed waterproofing and foundation drainage design and requirements.
- ❑ 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 these works have been completed in general accordance with our recommendations could be issued following the completion of a satisfactory materials testing and observation program by the geotechnical consultant.

9.0 STATEMENT OF LIMITATIONS

The recommendations provided in this report are in accordance with our present understanding of the project.

A geotechnical investigation of this nature 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, we request notification immediately in order 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 Morguard Investments Ltd. , 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.

Faisal Abou-Seido, P.Eng.

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

Carlos P. Da Silva, P.Eng.

Report Distribution

- ❏ Morguard Investments Ltd. (3 copies)
- ❏ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Mulit-Storey Buildings - 350 Sparks Street
Ottawa, Ontario

DATUM TBM - Brass monument located at the northwest corner of the intersection of Bay Street and Sparks Street. Geodetic elevation = 71.21m as provided by Annis, O'Sullivan, Vollebekk Ltd.

REMARKS

BORINGS BY CME 55 Power Auger

DATE May 20, 2015

FILE NO. PG3404

HOLE NO. BH 1

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	72.21
Brick interlock 0.06		G	1				
FILL: Blast rock 1.22		G	2				
		RC	1	88	0	1	71.21
		RC	2	71	0		
		RC	3	99	54	2	70.21
		RC	4	98	78	3	69.21
		RC	5	95	82	4	68.21
BEDROCK: Grey limestone with shale beddings		RC	6	100	99	5	67.21
		RC	7	100	99	6	66.21
						7	65.21
		RC	8	98	96	8	64.21
- shale lenses by 9.15m depth		RC	9	91	95	9	63.21
10.77		RC	10	95	50	10	62.21
End of Borehole							
(GWL @ 2.77m-May 20, 2015)							

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

○ Water Content %

20 40 60 80

Monitoring Well Construction

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

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Mulit-Storey Buildings - 350 Sparks Street
Ottawa, Ontario

DATUM TBM - Brass monument located at the northwest corner of the intersection of Bay Street and Sparks Street. Geodetic elevation = 71.21m as provided by Annis, O'Sullivan, Vollebekk Ltd.

REMARKS

BORINGS BY CME 55 Power Auger

DATE May 19, 2015

FILE NO. PG3404

HOLE NO. BH 2

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content % (20, 40, 60, 80)	Monitoring Well Construction
GROUND SURFACE						0	67.23		
Concrete slab 0.11		G	1						
FILL: Crushed stone 0.24		RC	1						
FILL: Blast rock trace granulars 0.81		RC	2	100	100	1	66.23		
		RC	3	79	73				
		RC	4			2	65.23		
BEDROCK: Limestone with shale beddings		RC	5	100	96	3	64.23		
4.65		RC	6	100	90	4	63.23		
End of Borehole									
(GWL @ 0.95m-May 20, 2015)									

Shear Strength (kPa): 20, 40, 60, 80, 100

▲ Undisturbed △ Remoulded

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Mulit-Storey Buildings - 350 Sparks Street
Ottawa, Ontario

DATUM TBM - Brass monument located at the northwest corner of the intersection of Bay Street and Sparks Street. Geodetic elevation = 71.21m as provided by Annis, O'Sullivan, Vollebekk Ltd.

REMARKS

BORINGS BY CME 55 Power Auger

DATE May 6, 2015

FILE NO. PG3404

HOLE NO. BH 3

SOIL DESCRIPTION	Depth (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Concrete	0.13	G	1		
FILL: Crushed stone	0.36	RC	1	33	
FILL: Blast rock	0.51	RC	2	100	28
		RC	3	100	82
BEDROCK: Grey limestone with shale beddings		RC	4	100	56
	4.55	RC	5	100	83
End of Borehole					
(GWL @ 1.76m-May 20, 2015)					

DEPTH (m)	ELEV. (m)
0	67.21
1	66.21
2	65.21
3	64.21
4	63.21

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

Monitoring Well Construction

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Mulit-Storey Buildings - 350 Sparks Street
Ottawa, Ontario

DATUM TBM - Brass monument located at the northwest corner of the intersection of Bay Street and Sparks Street. Geodetic elevation = 71.21m as provided by Annis, O'Sullivan, Vollebekk Ltd.

REMARKS

BORINGS BY CME 55 Power Auger

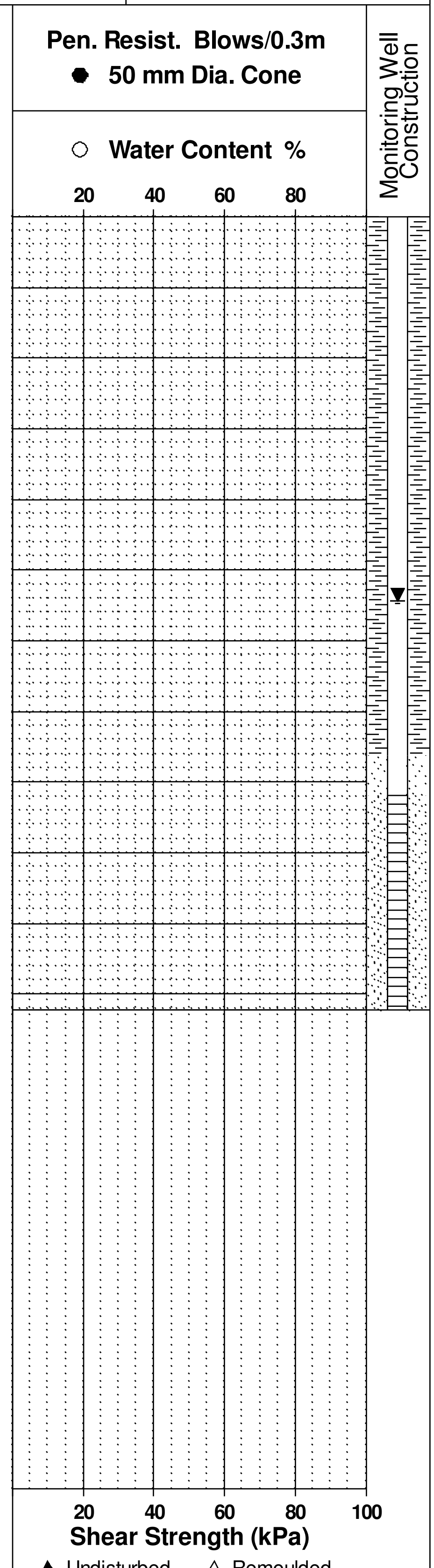
DATE May 19, 2015

FILE NO. PG3404

HOLE NO. BH 4

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Pavement structure	0.06	G	1		
FILL: Brown cobbles and sand with crushed stone	2.79	SS	2	54	13
		SS	3	12	8
		SS	4	33	6
		SS	5	33	10
		SS	6		+50
Concrete	3.76	RC	1	100	
BEDROCK: Grey limestone with trace shale beddings	11.23	RC	2	78	61
		RC	3	100	98
		RC	4	100	96
		RC	5	93	100
		RC	6	100	95
		RC	7	96	95
		RC	8	95	55
End of Borehole					
(GWL @ 5.43m-May 20, 2015)					

DEPTH (m)	ELEV. (m)
0	72.74
1	71.74
2	70.74
3	69.74
4	68.74
5	67.74
6	66.74
7	65.74
8	64.74
9	63.74
10	62.74
11	61.74



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Mulit-Storey Buildings - 350 Sparks Street
Ottawa, Ontario

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REMARKS

BORINGS BY CME 55 Power Auger

DATE May 11, 2015

FILE NO. PG3404

HOLE NO. BH 5

SOIL DESCRIPTION	Depth (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	67.85
Concrete slab	0.15	G	1				
FILL: Crushed stone	0.38	RC	1				
FILL: Blast rock	0.76	RC	2	50		1	66.85
BEDROCK: Grey limestone with shale beddings		RC	3	100	88	2	65.85
		RC	4	100	94	3	64.85
		RC	5	100	93	4	63.85
	5.21	RC	6	100	70	5	62.85
End of Borehole							
(GWL @ 1.94m-May 20, 2015)							

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

○ Water Content %

20 40 60 80

Monitoring Well Construction

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

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SOIL PROFILE AND TEST DATA

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Ottawa, Ontario

DATUM TBM - Brass monument located at the northwest corner of the intersection of Bay Street and Sparks Street. Geodetic elevation = 71.21m as provided by Annis, O'Sullivan, Vollebekk Ltd.

REMARKS

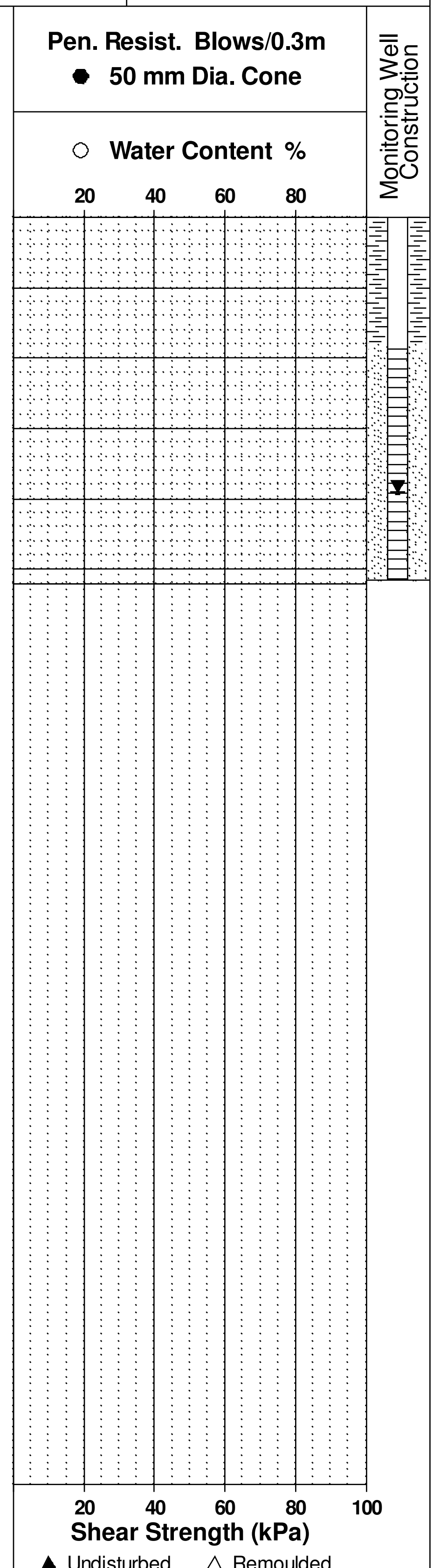
BORINGS BY CME 55 Power Auger

DATE May 11, 2015

FILE NO. PG3404

HOLE NO. BH 6

SOIL DESCRIPTION		SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	67.81
Concrete slab	0.15	G	1				
FILL: Crushed stone	0.30	RC	1	13			
FILL: Blast rock	0.89	RC	2	100	0	1	66.81
BEDROCK: Grey limestone with shale beddings		RC	3	100	28		
		RC	4	100	84	2	65.81
		RC	5	100	54	3	64.81
		RC	6	100	90	4	63.81
	5.21	RC	7	100	49	5	62.81
End of Borehole							
(GWL @ 3.91m-May 20, 2015)							



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Mulit-Storey Buildings - 350 Sparks Street
Ottawa, Ontario

DATUM TBM - Brass monument located at the northwest corner of the intersection of Bay Street and Sparks Street. Geodetic elevation = 71.21m as provided by Annis, O'Sullivan, Vollebekk Ltd.

REMARKS

BORINGS BY CME 55 Power Auger

DATE May 12, 2015

FILE NO. PG3404

HOLE NO. BH 7

SOIL DESCRIPTION	Depth (m)
GROUND SURFACE	
Concrete slab	0.13
FILL: Crushed stone	0.25
FILL: Blast rock	1.12
BEDROCK: Grey limestone with shale beddings	15.90
End of Borehole	

(GWL @ 1.46m-May 20, 2015)

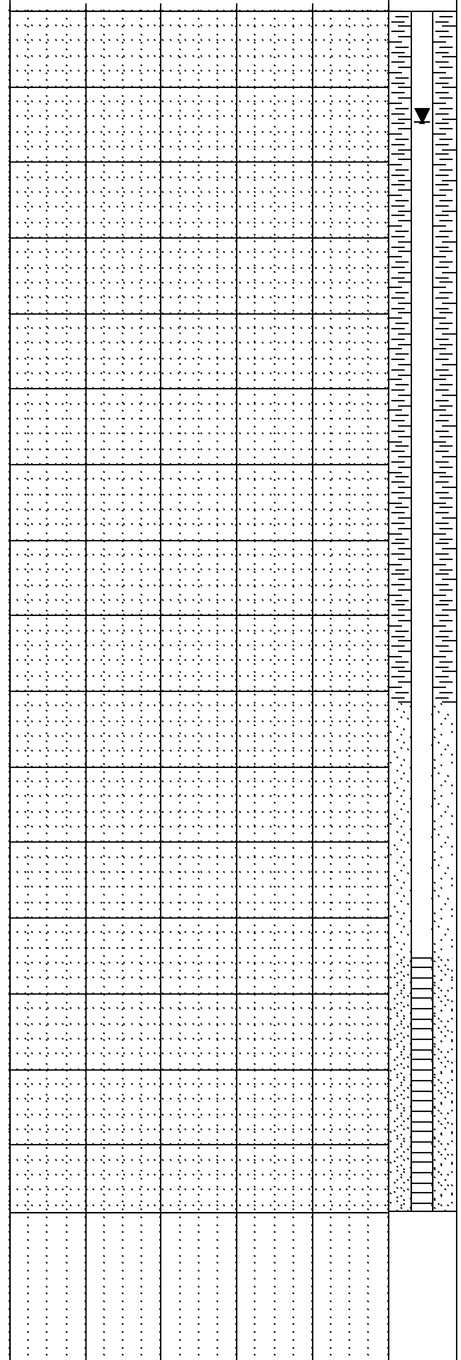
SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
G	1		
RC	1	27	
RC	2	71	100
RC	3	88	0
RC	4	81	62
RC	5	100	100
RC	6	100	100
RC	7	100	100
RC	8	75	0
RC	9	82	61
RC	10	100	73
RC	11	100	94
RC	12	100	100
RC	13	100	97
RC	14	100	100
RC	15	100	100
RC	16	100	96
RC	17	100	30
RC	18	100	98
RC	19	100	100
RC	20	100	100
RC	21	100	100
RC	22	100	100
RC	23	100	100
RC	24	100	88

DEPTH (m)	ELEV. (m)
0	67.86
1	66.86
2	65.86
3	64.86
4	63.86
5	62.86
6	61.86
7	60.86
8	59.86
9	58.86
10	57.86
11	56.86
12	55.86
13	54.86
14	53.86
15	52.86

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

Monitoring Well Construction

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Proposed Mulit-Storey Buildings - 350 Sparks Street
Ottawa, Ontario

DATUM TBM - Brass monument located at the northwest corner of the intersection of Bay Street and Sparks Street. Geodetic elevation = 71.21m as provided by Annis, O'Sullivan, Vollebekk Ltd.

REMARKS

BORINGS BY CME 55 Power Auger

DATE May 5, 2015

FILE NO. PG3404

HOLE NO. BH 8

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Concrete slab	0.15	G	1		
FILL: Crushed stone	0.41	RC	2	21	52
FILL: Blast rock	1.22				
BEDROCK: Grey limestone with shale beddings	16.15	RC	3	100	100
		RC	4	100	90
		RC	5	100	100
		RC	6	100	89
		RC	7	100	86
		RC	8	98	77
		RC	9	100	86
		RC	10	100	71
		RC	11	100	90
		RC	12	100	97
		RC	13	100	92
		RC	14	100	57
End of Borehole					

(GWL @ 0.86m-May 20, 2015)

DEPTH (m)	ELEV. (m)
0	67.85
1	66.85
2	65.85
3	64.85
4	63.85
5	62.85
6	61.85
7	60.85
8	59.85
9	58.85
10	57.85
11	56.85
12	55.85
13	54.85
14	53.85
15	52.85
16	51.85

Pen. Resist. Blows/0.3m
● 50 mm Dia. Cone
○ Water Content %
20 40 60 80

Monitoring Well Construction

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

SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

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

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

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

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

SOIL DESCRIPTION (continued)

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

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

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

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

MC%	-	Natural moisture content or water content of sample, %
LL	-	Liquid Limit, % (water content above which soil behaves as a liquid)
PL	-	Plastic limit, % (water content above which soil behaves plastically)
PI	-	Plasticity index, % (difference between LL and PL)
Dxx	-	Grain size which xx% of the soil, by weight, is of finer grain sizes These grain size descriptions are not used below 0.075 mm grain size
D10	-	Grain size at which 10% of the soil is finer (effective grain size)
D60	-	Grain size at which 60% of the soil is finer
Cc	-	Concavity coefficient = $(D30)^2$ / (D10 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

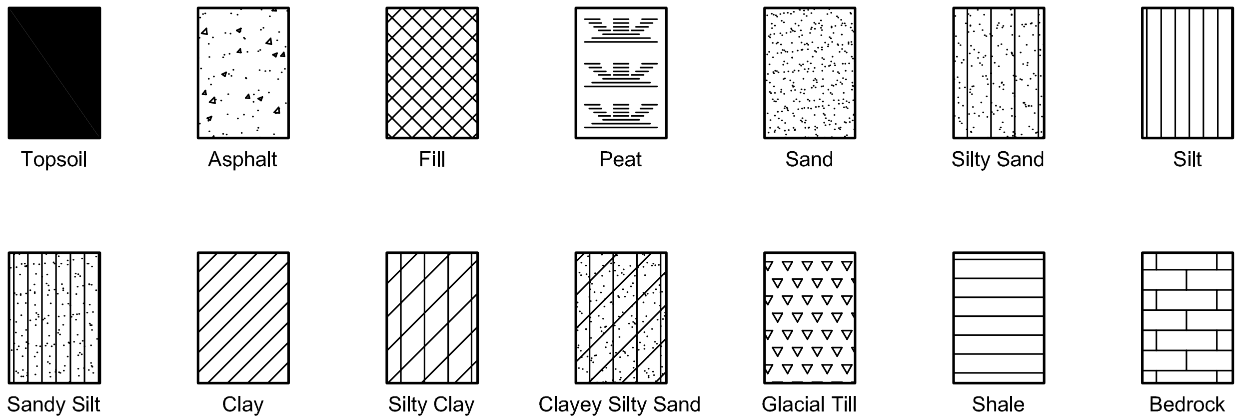
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void 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.

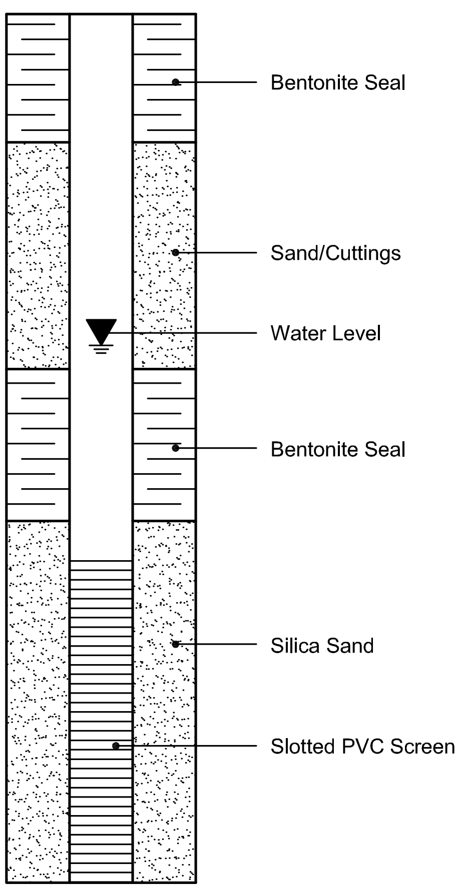
SYMBOLS AND TERMS (continued)

STRATA PLOT

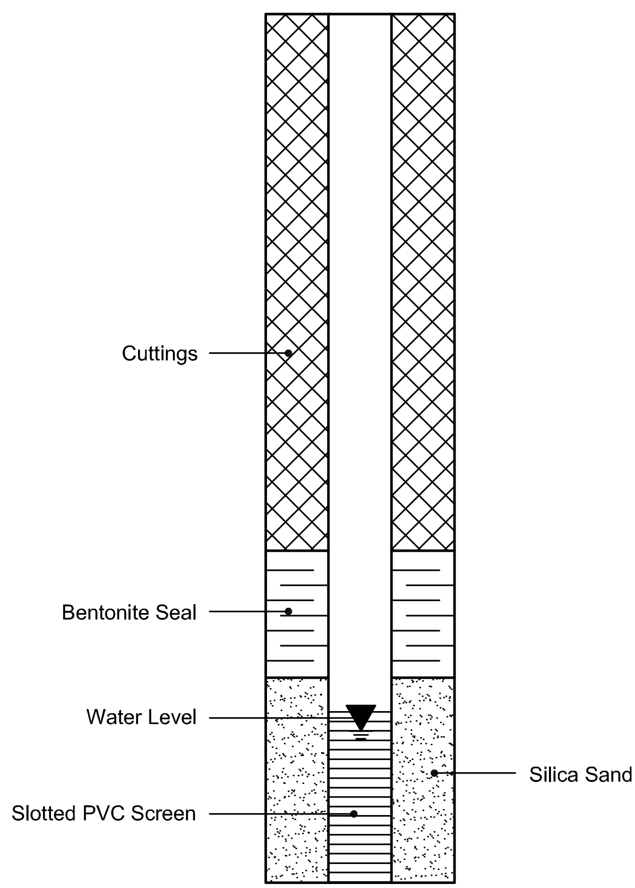


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Report Date: 22-May-2015
Order Date:21-May-2015

Client: **Paterson Group Consulting Engineers**
Client PO: 17630
Project Description: PG 3404

	Client ID:	BH4-SS6	-	-	-
	Sample Date:	19-May-15	-	-	-
	Sample ID:	1521240-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	92.7	-	-	-
General Inorganics					
pH	0.05 pH Units	10.66	-	-	-
Resistivity	0.10 Ohm.m	4.68	-	-	-
Anions					
Chloride	5 ug/g dry	747	-	-	-
Sulphate	5 ug/g dry	937	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURE 2 - SHEAR WAVE VELOCITY PROFILE AT SHOT LOCATION + 4.5 m

FIGURE 2 - SHEAR WAVE VELOCITY PROFILE AT SHOT LOCATION - 4.5 m

DRAWING PG3404-1 - TEST HOLE LOCATION PLAN

FIGURE 1

KEY PLAN

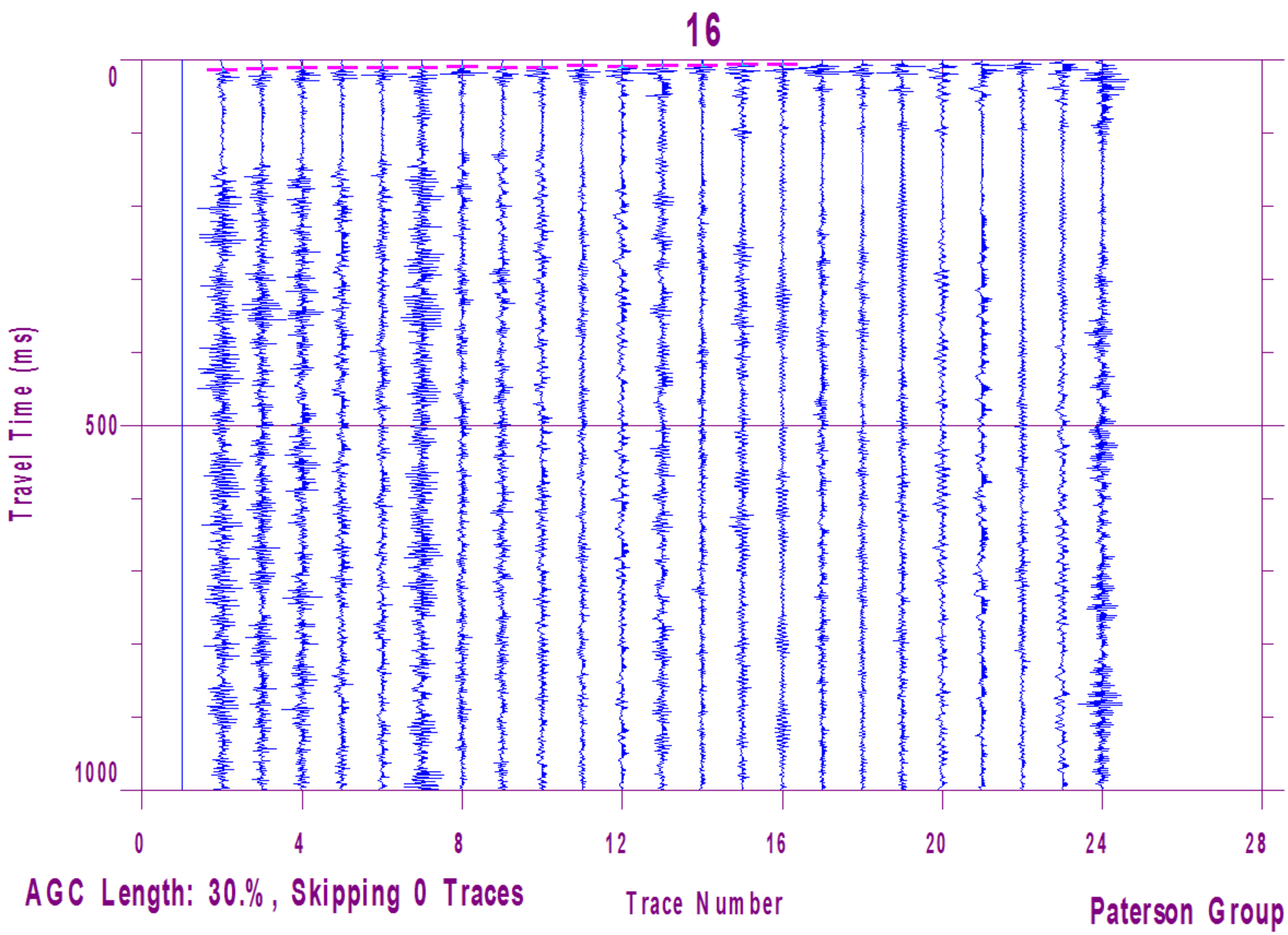


Figure 2 – Shear Wave Velocity Profile at Shot Location +4.5 m

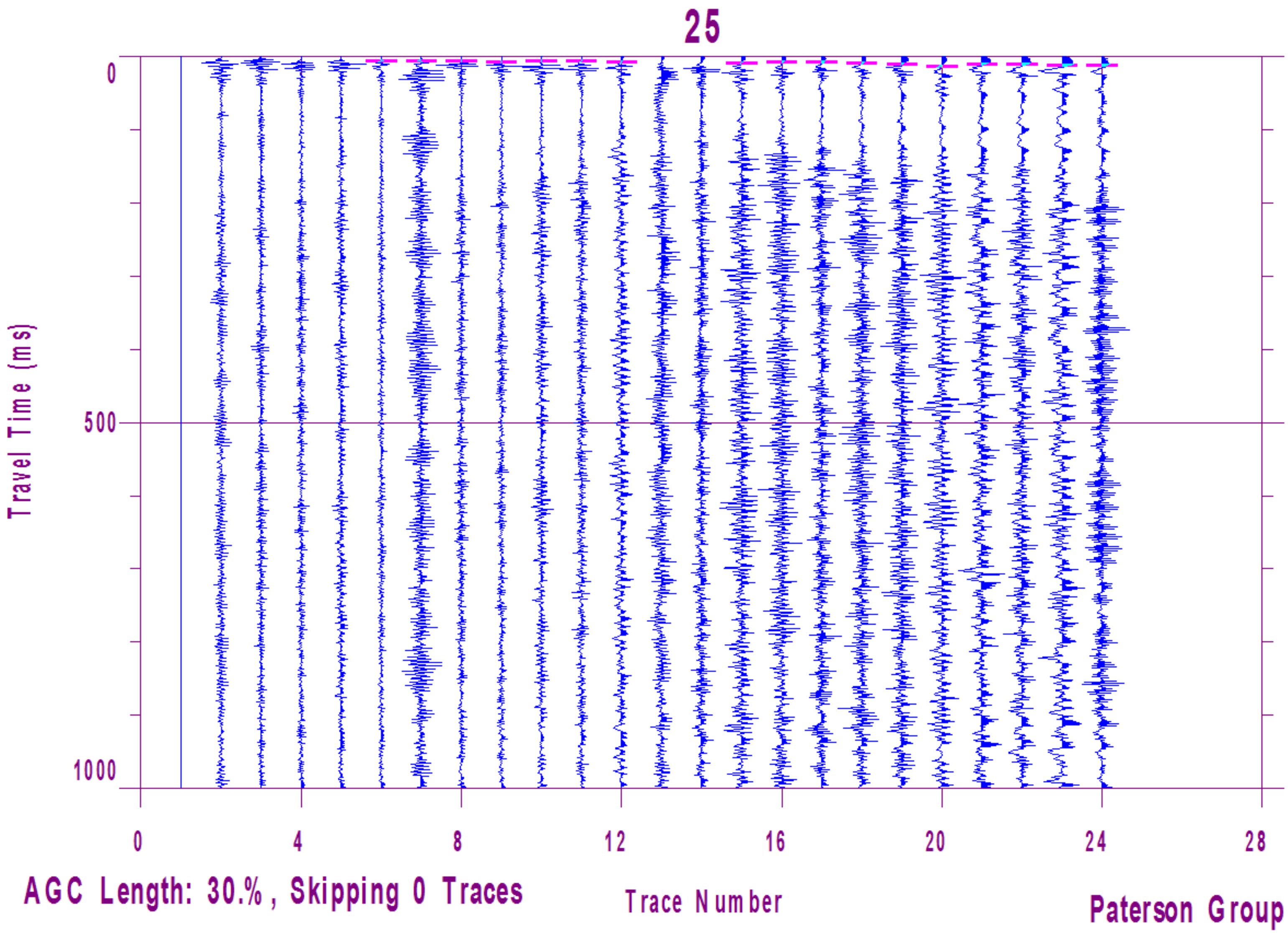
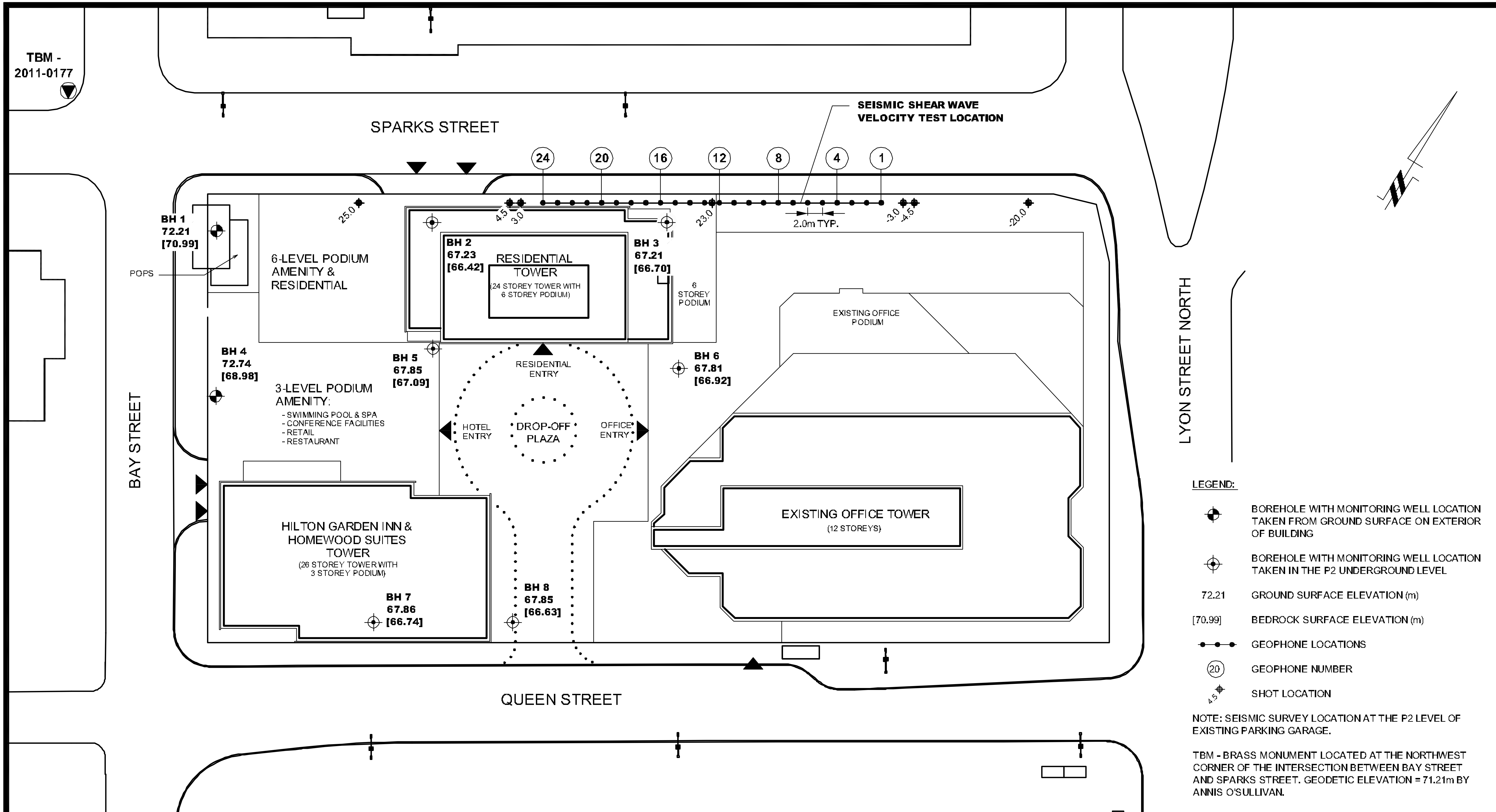


Figure 3 – Shear Wave Velocity Profile at Shot Location -4.5 m



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NO.	REVISIONS	DATE	INITIAL

MORGAURD INVESTMENTS LTD.

GEOTECHNICAL INVESTIGATION
PROP. MULTI-STOREY BUILDINGS - 350 SPARKS STREET

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:500	Date: 05/2015
Drawn by: CPB	Report No.: PG3404-1
Checked by: FA	Drawing No.: **PG3404-1**
Approved by: CDS	Revision No.:

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