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Geotechnical Engineering

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Noise & Vibration Studies

Geotechnical Investigation
Proposed Multi-Storey Building
398, 402 & 406 Roosevelt Avenue - Ottawa

Prepared For

ML Westboro Realty Inc.

May 7, 2024

Report: PG4339-1
Revision 3

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by ML Wesboro Realty Inc. to conduct a geotechnical investigation for the proposed multi-storey building to be located at 398, 402 and 406 Roosevelt Avenue in the City of Ottawa (refer to Figure 1 - Key Plan presented in Appendix 2).

The objectives of the geotechnical investigation were to:

☐ Determine the subsoil and groundwater conditions at this site by means of boreholes.

☐ Provide geotechnical recommendations for the design of the proposed development including construction considerations which may affect its design.

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

Investigating the presence or potential presence of contamination on the subject property was not part of the scope of work of this present investigation. A report addressing environmental issues for the subject site was prepared under separate cover.

2.0 Proposed Development

Based on the available drawings, the proposed development at the subject site consists of a multi-storey building with 2 underground parking levels. Further, the underground parking levels will extend beyond the lateral limits of the overlying building and will encompass the majority of the site footprint.

At finished grades, the proposed building will generally be surrounded by walkways and landscaped areas, with private terraces at the rear. It is also anticipated that the proposed building will be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

The field program for the geotechnical investigation was carried out on June 28, 2017. At that time, a total of two (2) boreholes were advanced to a maximum depth of 7.7 m. The borehole locations were determined in the field by Paterson personnel taking into consideration site features and underground services. The locations of the boreholes are shown on Drawing PG4339-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were advanced using a track-mounted auger drill rig operated by a two person crew. All fieldwork was conducted under the full-time supervision of personnel from Paterson's geotechnical division under the direction of a senior engineer. The testing procedure consisted of augering and rock coring to the required depths and at the selected locations and sampling the overburden.

Sampling and In Situ Testing

Soil samples were collected from the boreholes using two different techniques, namely, sampled directly from the auger flights (AU) or collected using a 50 mm diameter split-spoon (SS) sampler. Rock cores (RC) were obtained using 47.6 mm inside diameter coring equipment. All samples were visually inspected and initially classified on site. The auger and split-spoon samples were placed in sealed plastic bags, and rock cores were placed in cardboard boxes. All samples were transported to our laboratory for further examination and classification. The depths at which the auger, split spoon and rock core samples were recovered from the boreholes are shown as AU, SS and RC, respectively, on the Soil Profile and Test Data sheets presented in Appendix 1.

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

The recovery value and a Rock Quality Designation (RQD) value were calculated for each drilled section of bedrock and are 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 presented on the Soil Profile and Test Data sheets in Appendix 1 of this report.

Groundwater

Monitoring wells were installed in the boreholes to permit monitoring of the groundwater levels subsequent to the completion of the sampling program.

3.2 Field Survey

The borehole locations were determined by Paterson personnel taking into consideration the presence of underground and aboveground services. The location and ground surface elevation at each borehole location was surveyed by Paterson personnel. The ground surface elevation at each borehole locations was surveyed with respect to a temporary benchmark (TBM), consisting of the top spindle of a fire hydrant located along the east side of Roosevelt Avenue adjacent to the subject site. A geodetic elevation of 68.06 m was provided for the TBM by Annis O'Sullivan Vollebekk Ltd. The borehole locations and ground surface elevation at each borehole location are presented on Drawing PG4339-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

The soil samples and rock cores recovered from the subject site were examined in our laboratory to review the results of the field logging.

3.4 Analytical Testing

One soil sample from an adjacent site (342 Roosevelt Avenue) 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 analytical test results are presented in Appendix 1 and discussed in Section 6.7.

4.0 Observations

4.1 Surface Conditions

The subject site, consisting of 3 contiguous properties, is currently occupied by 3 detached homes with associated laneways and outbuildings. The site is bordered to the north and west by residential buildings, to the south by a commercial building followed by Richmond Road and to the east by Roosevelt Avenue. The ground surface across the site is relatively flat and at grade with the neighbouring properties.

4.2 Subsurface Profile

The subsurface profile encountered at the borehole locations consists of approximately 0.7 to 1.4 m of fill material, which includes topsoil, asphalt concrete (approximately 100 mm), crushed stone with silty sand, silty clay with some sand, and silty sand with some cobbles and trace gravel and organics. The fill material lies atop grey limestone bedrock with shale partings. Generally, the bedrock is poor to very poor quality within the upper 4 to 5 m and fair to excellent quality at depth based on the RQD values. Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for specific details of the soil profiles encountered at each test hole location.

Based on available geological mapping, the bedrock in this area mostly consists of limestone with some shale partings of the Ottawa formation with an overburden drift thickness of less than 5 m.

4.3 Groundwater

The groundwater level readings were recorded at the borehole locations on July 5, 2017 and are presented in Table 1 below and on the Soil Profile and Test Data sheets in Appendix 1. It is important to note that groundwater levels are subject to seasonal fluctuations and therefore groundwater levels could differ at the time of construction.

Table 1 - Summary of Groundwater Level Readings

Test Hole Number	Ground Elevation (m)	Groundwater Levels (m)		Recording Date
		Depth	Elevation	
BH 1	66.74	5.18	61.56	July 5, 2017
BH 2	67.17	4.77	62.40	July 5, 2017

5.0 Discussion

5.1 Geotechnical Assessment

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

As the proposed building will have 2 underground parking levels, it is anticipated that the underside of footing elevation will be located below the groundwater level. Accordingly, foundation drainage has been recommended in Section 6.1.

Bedrock removal will be required to complete the underground parking levels. The above and other considerations are further discussed in the following sections.

5.2 Site 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 will be required for the construction of the underground parking 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 pre-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.

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

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 equipment could cause vibrations: piling equipment, hoe ram, compactor, dozer, crane, truck traffic, etc. The construction of the shoring system with soldier piles, should it be utilized, would require these pieces of equipment. 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. These guidelines are above perceptible human level and, in some cases, could be very disturbing to some people. Therefore, a pre-construction survey is recommended to minimize the risks of claims during or following the construction of the proposed building.

Fill Placement

Fill used for grading beneath the building areas, where required, 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 fill should be placed in lifts no greater than 300 mm thick and compacted using suitable compaction equipment for the lift thickness. Fill placed beneath the proposed building areas should be compacted to at least 98% of its standard Proctor maximum dry density (SPMDD).

Non-specified existing fill along with site-excavated soil can be used as general landscaping fill and beneath landscaped areas where settlement of the ground surface is of minor concern. In landscaped areas, these materials should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids. If these materials are to be used to build up the subgrade level for areas to be paved, they should be compacted in thin lifts to a minimum density of 95% of their respective 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.

5.3 Foundation Design

Bearing Resistance Values

Footings placed on a clean, surface sounded limestone bedrock surface can be designed using a factored bearing resistance value at ultimate limit states (ULS) of **2,500 kPa** incorporating a geotechnical resistance factor of 0.5.

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

Lateral Support

The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels. Adequate lateral support is provided to a sound bedrock bearing medium when a plane extending down and out from the bottom edge of the footing at a minimum of 1H:6V (or flatter) passes only through sound bedrock or a material of the same or higher capacity as the bedrock, such as concrete. A weathered bedrock bearing medium will require a lateral support zone of 1H:1V (or flatter).

Settlement

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

5.4 Design for Earthquakes

Seismic shear wave velocity testing was completed for the subject site to accurately determine the applicable seismic site classification for the proposed building in accordance with Table 4.1.8.4.A of the Ontario Building Code 2012 (OBC2012). The shear wave velocity testing was completed by Paterson personnel. The results of the shear wave velocity test are provided in Figures 2 and 3 in Appendix 2 of the present report.

Field Program

The seismic array testing location was placed as presented in Drawing PG4339-1 - Test Hole Location Plan, attached to the present report. Paterson field personnel placed 18 horizontal 2.4 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 1 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 four (4) to eight (8) times at each shot location to improve signal to noise ratio. The shot locations were 1, 1.5 and 15 m away from the first geophone, 1 and 1.5 m away from the last geophone, and at the centre of the seismic array.

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, V_{s30}, of the upper 30 m profile, immediately below the foundation of the building. 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 our testing, the bedrock shear wave velocity is **1,530 m/s**. It is understood that the overburden will be completely removed as part of the proposed building and footings will be placed on clean, surface sounded bedrock. The V_{s30} was calculated using the standard equation for average shear wave velocity provided in the Ontario Building Code (OBC) 2012, and as presented in the following page.

$$V_{s30} = \frac{Depth_{ofinterest}(m)}{\dfrac{Depth_{Layer1}(m)}{V_{S_{Layer1}}(m/s)} + \dfrac{Depth_{Layer2}(m)}{V_{S_{Layer2}}(m/s)}}$$

$$V_{S30} = \frac{30m}{\left(\dfrac{30m}{1{,}530m/s}\right)}$$

$$V_{S30} = 1{,}530m/s$$

Based on the results of the shear wave velocity testing, the average shear wave velocity, V_{s30} is **1,530 m/s** for conventional shallow footings which are founded on the bedrock surface. Therefore, a **Site Class A** is applicable for foundation design of the proposed building, as presented in Table 4.1.8.4.A of the Ontario Building Code 2012.

Soils underlying the subject site are not susceptible to liquefaction. Reference should be made to the latest revision of the Ontario Building Code 2012 for a full discussion of the earthquake design requirements.

5.5 Basement Slab

All overburden soil will be removed from the subject site leaving the bedrock as the founding medium for the lower basement floor slab. It is understood that the basement area will be mostly parking and the recommended pavement structure noted in Section 5.7 will be applicable.

However, if storage or other uses of the lower level will involve the use of a concrete floor slab, it is recommended that the upper 200 mm of sub-slab fill consists of compacted 19 mm clear crushed stone.

In consideration of the groundwater conditions encountered at the time of the geotechnical investigation, an underslab drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided in the clear crushed stone layer under the lower level floor slab.

5.6 Basement Wall

It is expected that most of the below-grade foundation 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) where this condition occurs. 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.

Where soil is to be retained, the conditions can be well-represented by assuming the retained soil consists of a material with an angle of internal friction of 30 degrees and a bulk (drained) unit weight of 20 kN/m^3.

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 $\Delta P_{AE} = 0.375 \cdot a_c \cdot \gamma \cdot H^2/g$, where:

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

The peak ground acceleration, (a_{max}), for the Ottawa area is 0.32g according to OBC 2012. 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 2012.

5.7 Pavement Structure

For design purposes, it is recommended that the rigid pavement structure for the underground parking level consist of Category C2, 32 MPa concrete at 28 days with air entrainment of 5 to 8%. The recommended rigid pavement structure is further presented in Table 2 below. The flexible pavement structure presented in Table 3 should be used for at grade access lanes and heavy loading parking areas.

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

Thickness (mm)	Material Description
125	**Wear Course -** Concrete slab
300 to 500	**BASE** - OPSS Granular A (thickness will depend on required pipe cover and other subfloor surfaces)
	SUBGRADE - Bedrock

To control cracking due to shrinking of the concrete floor slab, it is recommended that strategically located saw cuts be used to create control joints within the concrete floor slab of the underground parking level. The control joints are generally recommended to be located at the center of the column lines and spaced at approximately 24 to 36 times the slab thickness (for example; a 0.15 m thick slab should have control joints spaced between 3.6 and 5.4 m). The joints should be cut between 25 and 30% of the thickness of the concrete floor slab and completed as early as 4 hour after the concrete has been poured during warm temperatures and up to 12 hours during cooler temperatures.

Table 3 - Recommended Pavement Structure - Access Lanes

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

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

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

The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 99% of the material's SPMDD using suitable vibratory equipment.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

It is recommended that a perimeter foundation drainage system be provided for the proposed structure. Where the perimeter foundation walls are blind-poured directly against the vertical bedrock face, 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 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 for the infiltration of water to flow to an interior perimeter drainage pipe. The perimeter drainage pipe and underslab drainage system should direct water to sump pit(s) within the lower basement area. Elevators and other pits which extend below the level of the underslab drainage will require waterproofing.

Underslab Drainage

It is recommended that underslab drainage be provided to control water infiltration. For preliminary design purposes, we recommend that 100 or 150 mm perforated pipes be placed at 6 m centres underlying the lowest level floor slab. The spacing of the underslab drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

Foundation Backfill

Where sufficient space is available for exterior backfill, the backfill material 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 Against Frost Action

It is expected that the underground levels will not require protection against frost action due to the founding depth. Unheated structures, such as the access ramp, may required to be insulated against the deleterious effect of frost action. A minimum of 2.1 m of soil cover alone, or a minimum of 0.6 m of soil cover in conjunction with foundation insulation, should be provided in this regard.

6.3 Excavation Side Slopes

Unsupported Excavations

The side slopes of excavations in the soil and fill overburden materials should be either cut back at acceptable slopes or should be retained by shoring systems from the start of the excavation until the structure is backfilled.

Unsupported Excavations

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

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

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

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

Bedrock Stabilization

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.

Horizontal rock anchors and chainlink fencing are anticipated to be required over the upper 3 to 4 m of the vertical bedrock face, which is generally of lower quality, to prevent pop-outs of the bedrock, especially in areas where bedrock fractures are conducive to the failure of the bedrock surface.

However, the specific requirements for bedrock stabilization measures should be evaluated during the excavation operations and should be discussed with the structural engineer during the design stage.

Temporary Shoring

The design and approval of the temporary shoring system will be the responsibility of the shoring contractor and the shoring designer who is a licensed professional engineer and is hired by the shoring contractor. It is the responsibility of the shoring contractor to ensure that the temporary shoring is in compliance with safety requirements, designed to avoid any damage to adjacent structures and include dewatering control measures. In the event that subsurface conditions differ from the approved design during the actual installation, it is the responsibility of the shoring contractor to commission the required experts to re-assess the design and implement the required changes. The designer should also take into account the impact of a significant precipitation event and designate design measures to ensure that a precipitation will not negatively impact the shoring system or soils supported by the system.

The temporary shoring system could consist of a soldier pile and lagging system. Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be included to the earth pressures described below. These shoring systems could be cantilevered, anchored or braced. Generally, the shoring system 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 4 - 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.

Groundwater infiltration levels are anticipated to be low through the excavation face and are anticipated to be controllable with open sumps and pumps.

A temporary Ministry of the Environment, Conservation and Parks (MECP) permit to take water (PTTW) may be required for this project if more than 400,000 L/day of ground and/or surface water is to be pumped during the construction phase. A minimum 4 to 5 months should be allowed for completion of the PTTW application package and issuance of the permit by the MECP.

For typical ground or surface water volumes being pumped during the construction phase, typically between 25,000 to 400,000 L/day, it is required to register on the Environmental Activity and Sector Registry (EASR). A minimum of two to four weeks should be allotted for completion of the EASR registration and the Water Taking and Discharge Plan to be prepared by a Qualified Person as stipulated under O.Reg. 63/16. If a project qualifies for a PTTW based upon anticipated conditions, an EASR will not be allowed as a temporary dewatering measure while awaiting the MOECC review of the PTTW application.

Long-term Groundwater Control

Our recommendations for the proposed building's long-term groundwater control are presented in Section 6.1. Any groundwater encountered along the building's perimeter or underslab drainage system will be directed to the proposed building's cistern/sump pit. It is expected that groundwater flow will be low (i.e.- less than 25,000 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, a localized 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.

Further, given the shallow bedrock present at, and in the vicinity of, the subject site, the neighbouring structures are expected to be founded on the bedrock surface. Therefore, no issues are expected with respect to groundwater lowering that would cause 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 the analytical testing show that the sulphate content is less than 0.1%. This result indicates that Type 10 Portland cement (normal cement) would be appropriate for this site. The chloride content and pH of the samples indicate that they are not significant factors in creating a corrosive environment for exposed ferrous metals at this site, whereas the resistivity is indicative of an aggressive to very aggressive corrosive environment.

7.0 Recommendations

It is recommended that the following be carried out once the master plan and site development are determined:

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

☐ 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 placement of backfilling materials.

☐ 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 material testing and observation program by the geotechnical consultant.

8.0 Statement of Limitations

The recommendations made in this report are in accordance with our present understanding of the project. Also, our recommendations should be reviewed when the drawings and specifications are complete.

The client should be aware that any information pertaining to soils and all test hole logs are furnished as a matter of general information only and test hole descriptions or logs are not to be interpreted as descriptive of conditions at locations other than those of the test holes.

A soils investigation is a limited sampling of a site. Should any conditions at the site be encountered which differ from those at the test locations, we request that we be notified 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 ML Westboro Realty Inc. or their agent(s) is not authorized without review by this firm for the applicability of our recommendations to the altered use of the report.

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
May 7, 2024
S. S. DENNIS
100519516
PROVINCE OF ONTARIO

Scott Dennis

Fernanda Carozzi, PhD Geoph.

Scott S. Dennis, P.Eng.

Report Distribution:

- ☐ ML Westboro Realty Inc. (e-mail copy)
- ☐ 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
398, 402 and 406 Roosevelt Avenue
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located in front of 402 Roosevelt Avenue. Geodetic elevation = 68.06m.

REMARKS

BORINGS BY CME 55 Power Auger

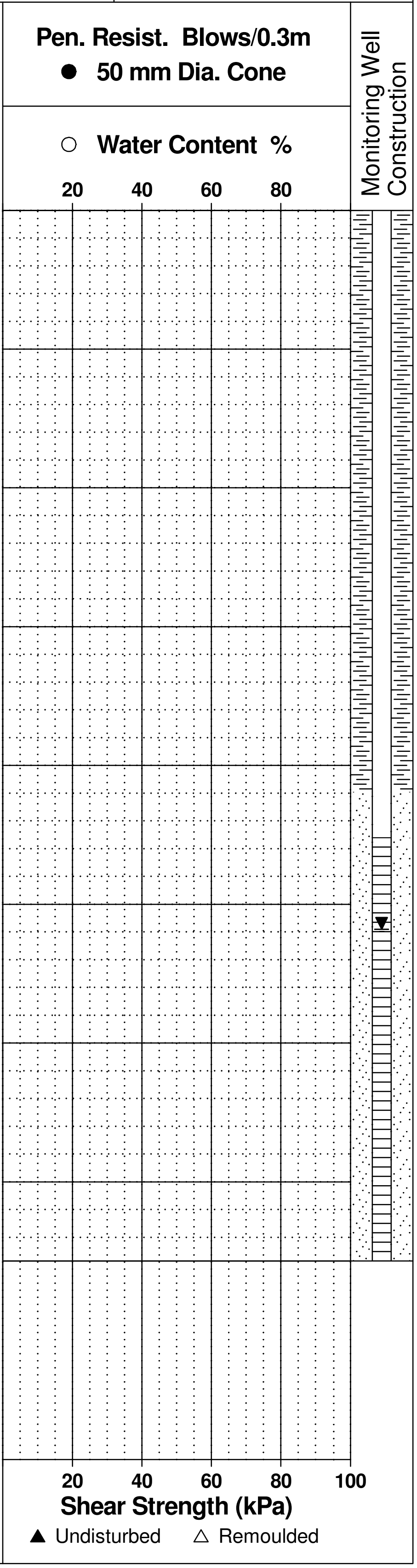
DATE June 28, 2017

FILE NO. PG4339

HOLE NO. BH 1

SOIL DESCRIPTION	Depth of change (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
FILL: Brown silty sand with topsoil, trace organics and gravel	0.69	AU	1		
BEDROCK: Grey limestone with interbedded shale		SS	2	53	50+
		RC	1	100	30
		RC	2	100	35
		RC	3	100	35
		RC	4	100	98
	7.57	RC	5	100	82
End of Borehole					
(GWL @ 5.18m - July 5, 2017)					

DEPTH (m)	ELEV. (m)
0	66.74
1	65.74
2	64.74
3	63.74
4	62.74
5	61.74
6	60.74
7	59.74



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
398, 402 and 406 Roosevelt Avenue
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located in front of 402 Roosevelt Avenue. Geodetic elevation = 68.06m.

REMARKS

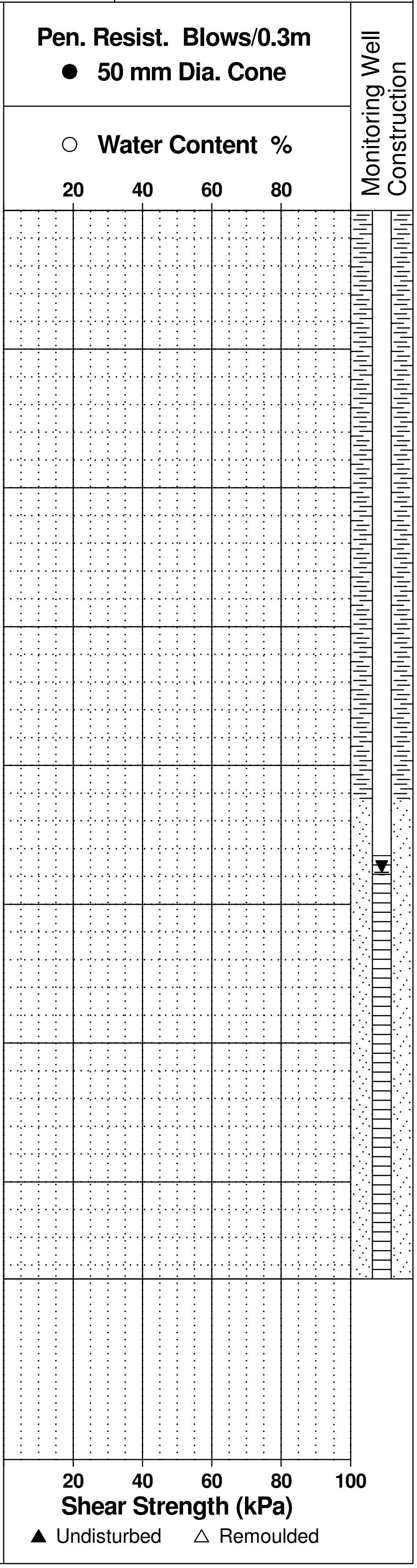
BORINGS BY CME 55 Power Auger

DATE June 28, 2017

FILE NO. PG4339

HOLE NO. BH 2

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						
Asphaltic concrete 0.10	AU	1			0	67.17
FILL: Crushed stone with silty sand 0.30	AU	2				
FILL: Brown silty clay, some sand, trace gravel 0.69						
FILL: Brown silty sand, some cobbles, trace gravel 1.35	SS	3	62	50+	1	66.17
	SS	4	50	50+		
					2	65.17
	RC	1	100	30		
					3	64.17
	RC	2	100	30		
					4	63.17
BEDROCK: Grey limestone with interbedded shale						
					5	62.17
	RC	3	100	51		
					6	61.17
	RC	4	100	80		
					7	60.17
7.70						
End of Borehole						
(GWL @ 4.77m - July 5, 2017)						



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)

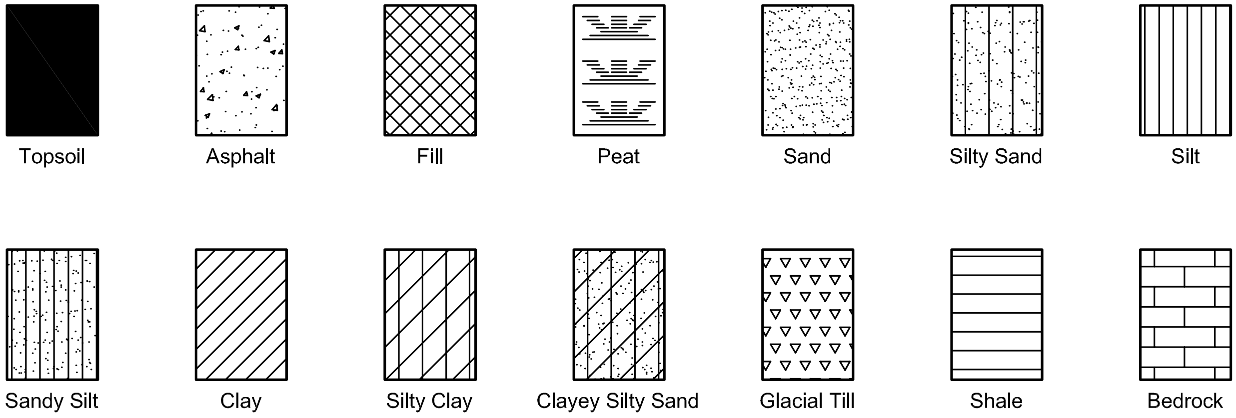
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

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.

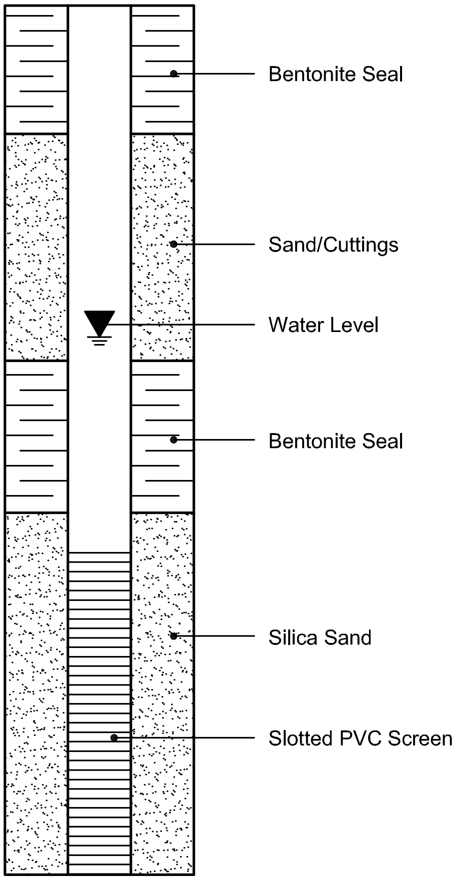
SYMBOLS AND TERMS (continued)

STRATA PLOT

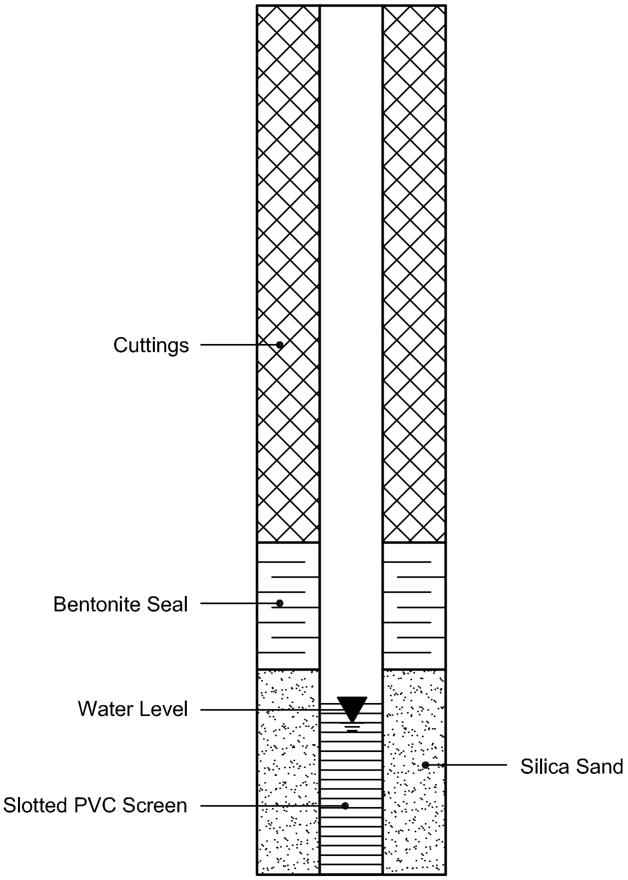


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis
Client: Paterson Group Consulting Engineers
Client PO: 20656

Report Date: 27-Jul-2017
Order Date: 21-Jul-2017
Project Description: PG4210

	Client ID:	TP2-G3	-	-	-
	Sample Date:	19-Jul-17	-	-	-
	Sample ID:	1729564-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	84.1	-	-	-
General Inorganics					
pH	0.05 pH Units	7.27	-	-	-
Resistivity	0.10 Ohm.m	63.8	-	-	-
Anions					
Chloride	5 ug/g dry	<5	-	-	-
Sulphate	5 ug/g dry	5	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURES 2 & 3 - SEISMIC SHEAR WAVE VELOCITY PROFILES

DRAWING PG4339-1 - TEST HOLE LOCATION PLAN

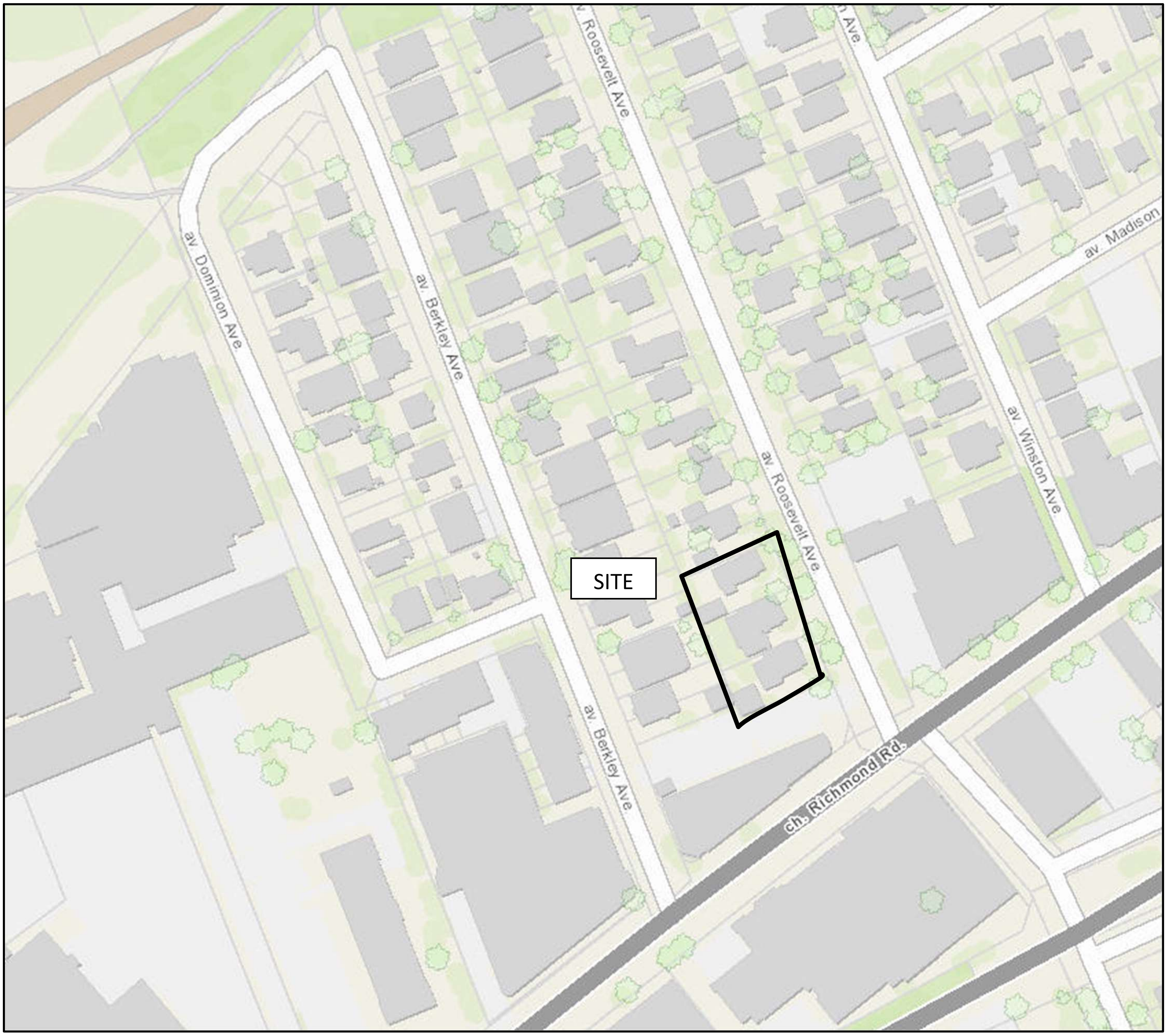


FIGURE 1

KEY PLAN

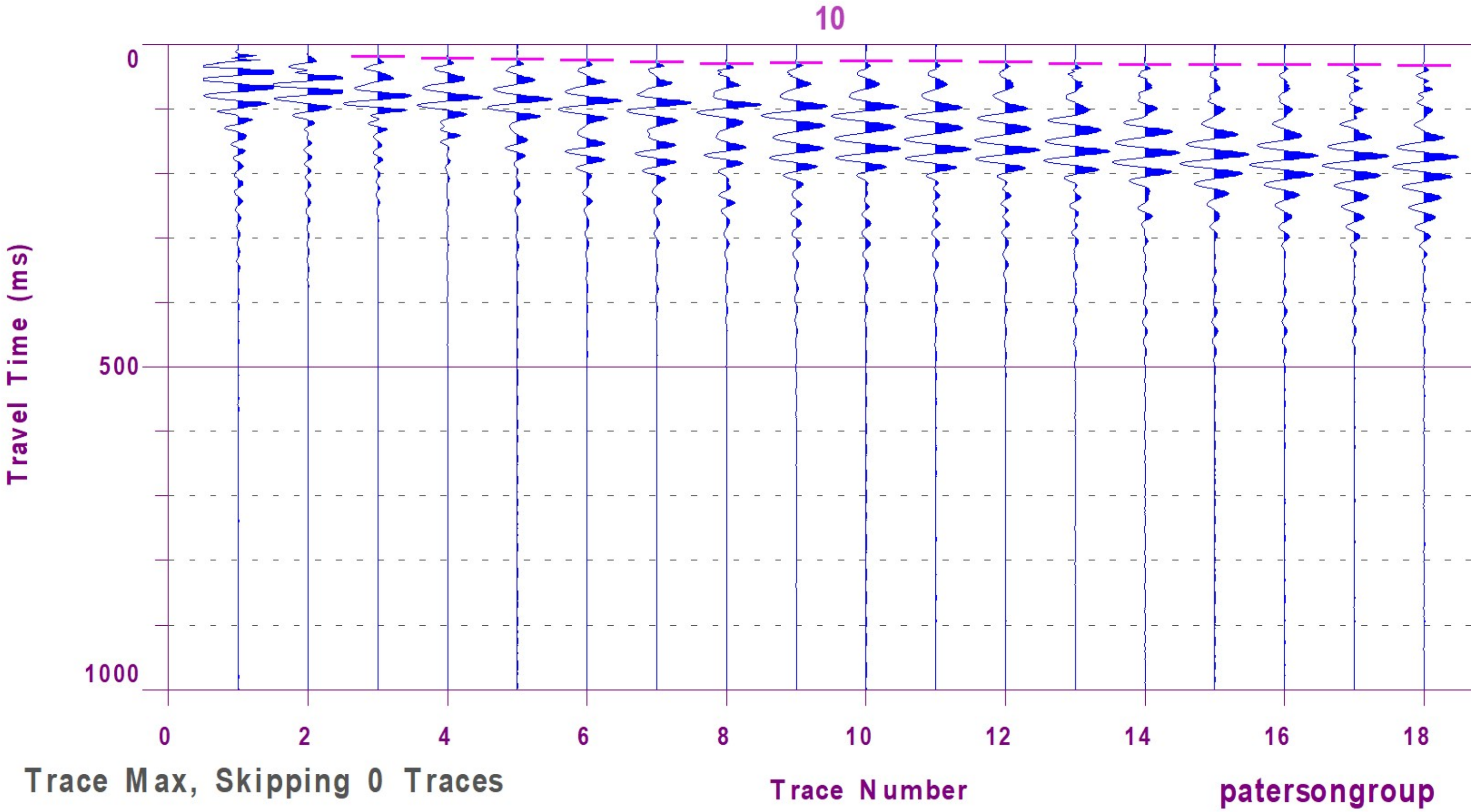


Figure 2 – Shear Wave Velocity Profile at Shot Location -1 m

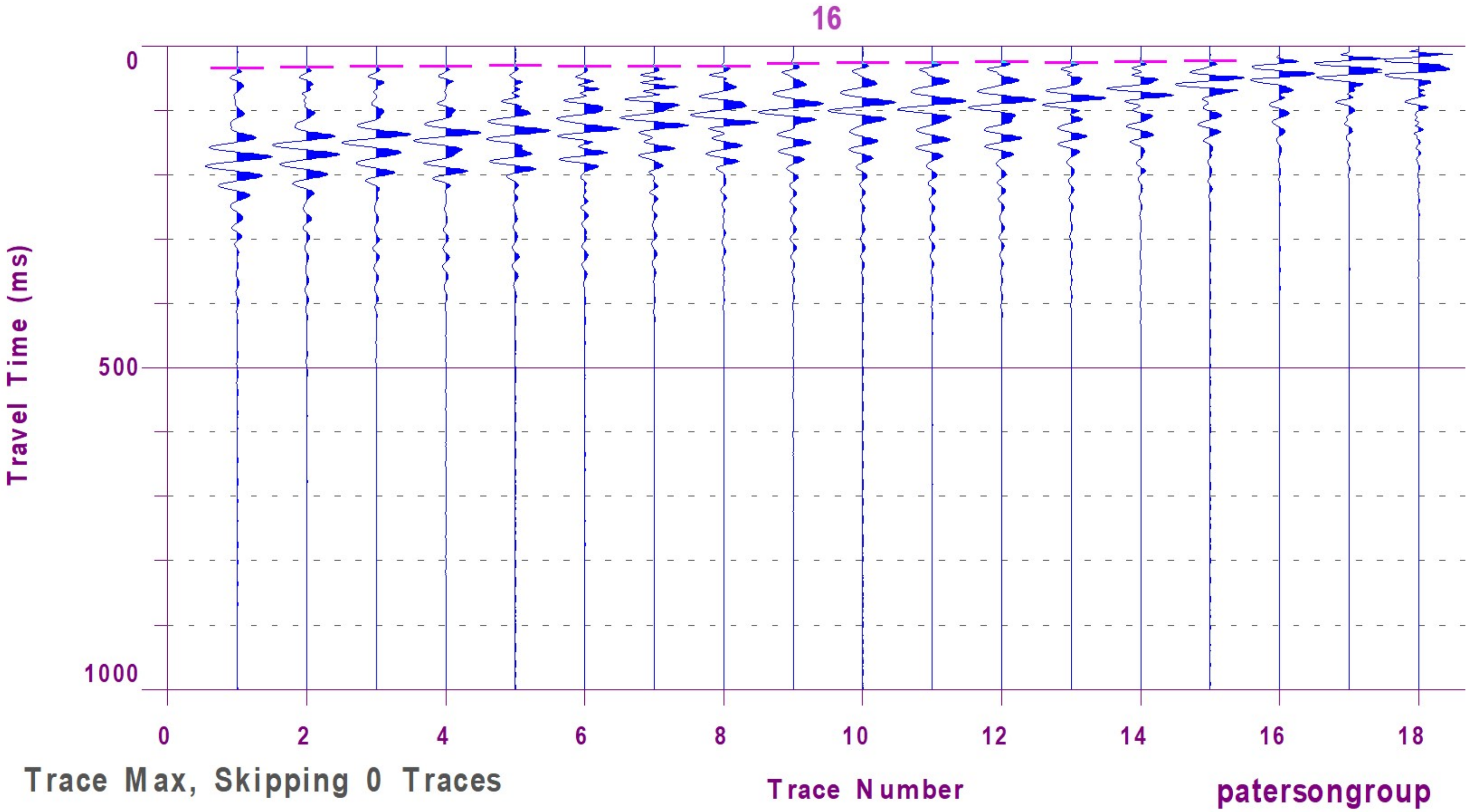
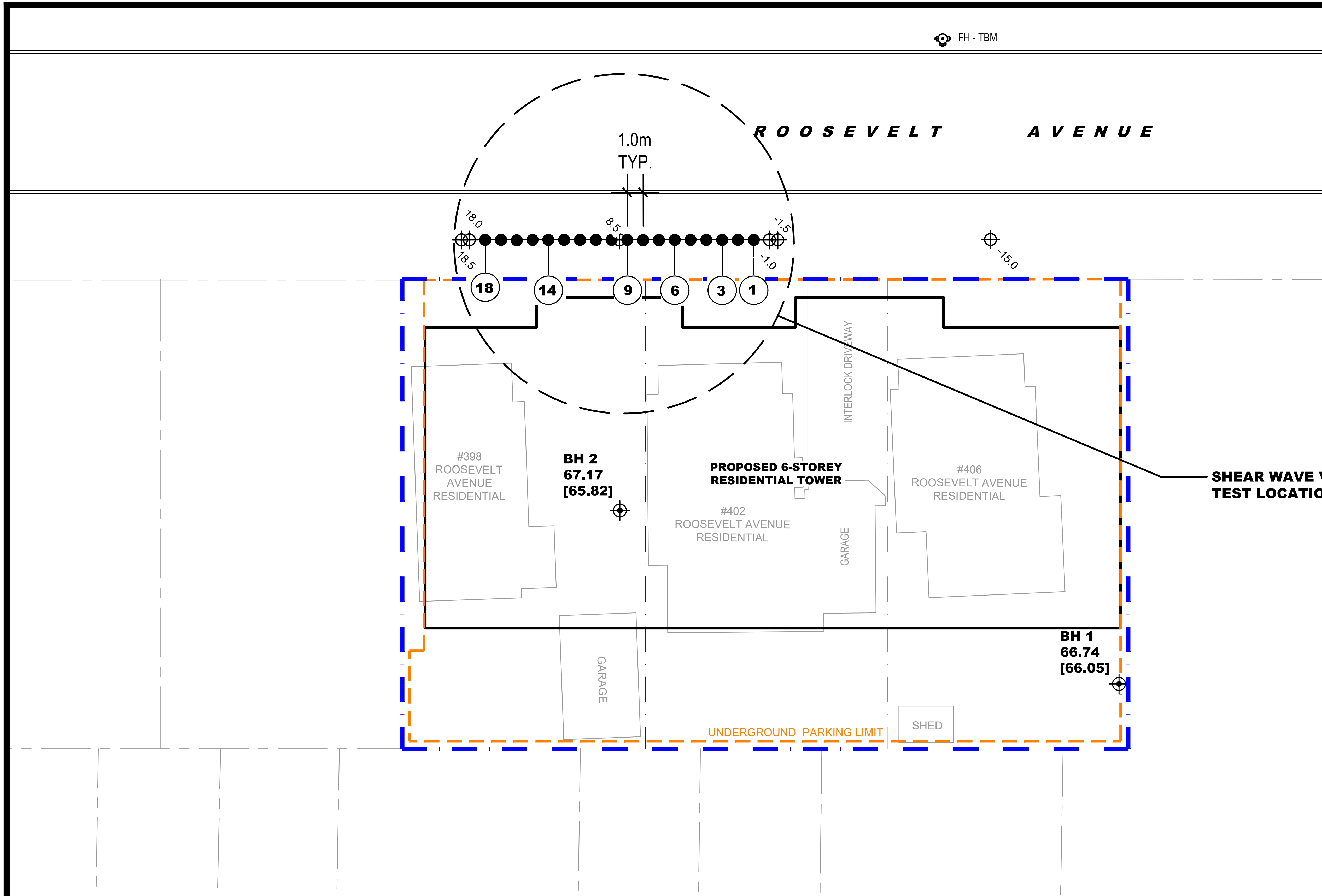


Figure 3 – Shear Wave Velocity Profile at Shot Location 18.5 m



LEGEND:

⌖	BOREHOLE WITH MONITORING WELL LOCATION
98.68	GROUND SURFACE ELEVATION (m)
[97.99]	BEDROCK SURFACE ELEVATION (m)
(93.46)	GROUNDWATER SURFACE ELEVATION (m)

CONCEPTUAL PLAN PROVIDED BY RLA ARCHITECTURE

TBM - TOP SPINDLE OF FIRE HYDRANT. GEODETIC ELEVATION = 68.06m.

SCALE: 1:250

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consulting engineers

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Ottawa, Ontario K2E 7J5
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NO.	REVISIONS	DATE	INITIAL
2	SEISMIC SHEAR WAVE VELOCITY TEST ADDED TO PLAN	26/05/2022	FC
1	ADDED CONCEPTUAL PLAN TO PLAN	12/03/2021	SD

ML WESTBORO REALTY INC.

GEOTECHNICAL INVESTIGATION
398, 402 AND 406 ROOSEVELT AVENUE

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:250
Date: 10/2017
Drawn by: RCG
Report No.: PG4339-1
Checked by: SD
Dwg. No.: PG4339-1
Approved by: DJG
Revision No.: 2

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