PATERSON GROUP

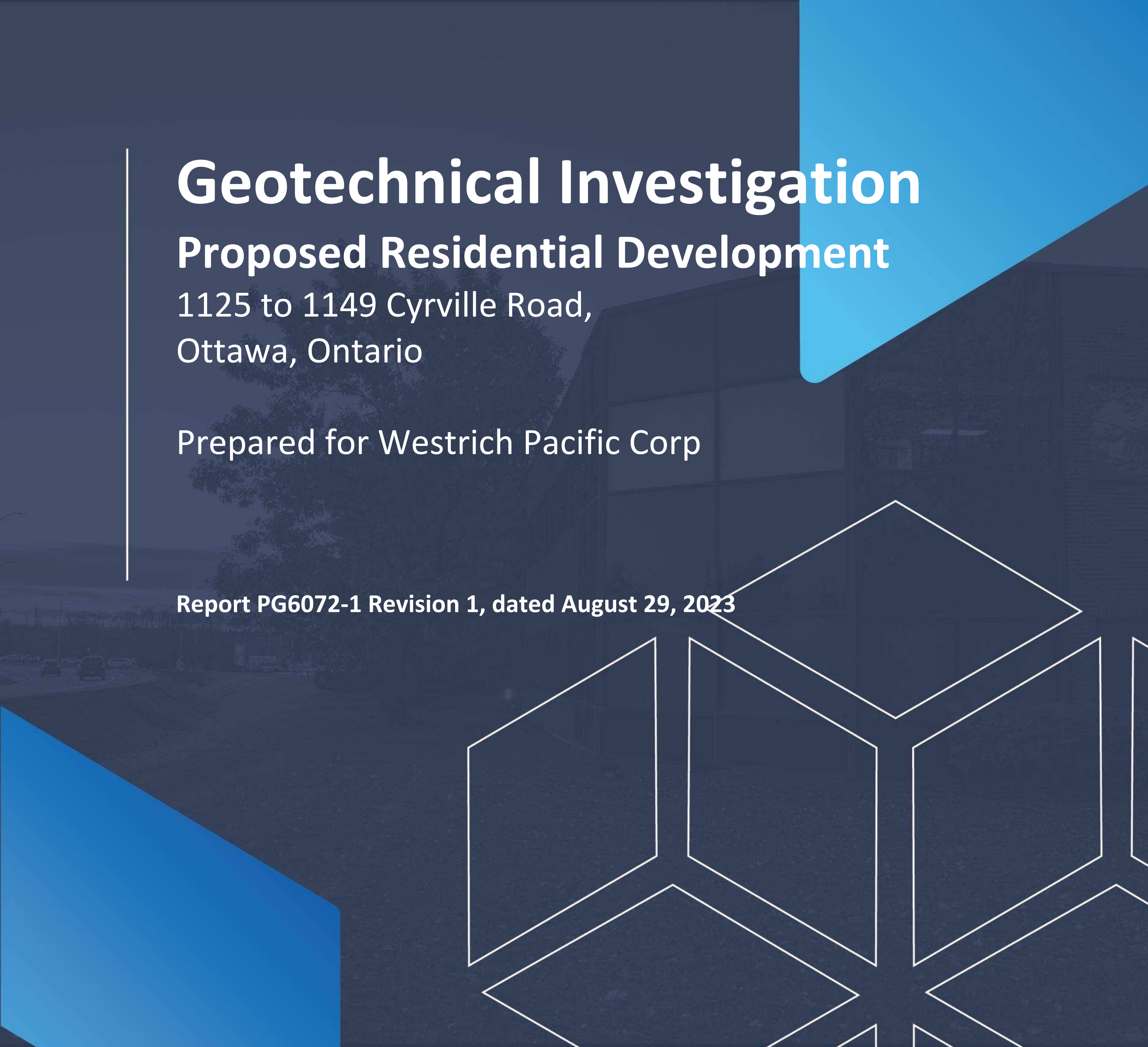
Geotechnical Investigation

Proposed Residential Development

1125 to 1149 Cyrville Road,
Ottawa, Ontario

Prepared for Westrich Pacific Corp

Report PG6072-1 Revision 1, dated August 29, 2023



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1.0 Introduction

Paterson Group (Paterson) was commissioned by Westrich Pacific Corp. to conduct a geotechnical investigation for the proposed commercial development to be located at 1125 to 1149 Cyrville Road, in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report). The objectives of the geotechnical investigation were to:

The objective of the geotechnical investigation was to:

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

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

Investigating the presence or potential presence of contamination on the subject property was not a part of the scope of the present investigation. Therefore, the present report does not address environmental issues.

2.0 Proposed Development

Based on the available drawings, it is understood that the proposed development will consist of two multi-storey residential buildings with multiple underground parking levels. Associated at grade access lanes, pedestrian pathways and landscaped areas are also anticipated. It is further anticipated that the proposed development will be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The fieldwork program for the investigation was carried out on August 16 and 17, 2011. At that time, 12 boreholes were advanced to depths ranging from 0.5 to 5.7 m. The borehole locations were distributed in a manner to provide general coverage of the subject site, taking into consideration site features. The locations of the boreholes are shown on Drawing PG6072-1 – Test Hole Location Plan 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 our personnel under the direction of a senior engineer from our geotechnical department. The drilling procedure consisted of augering to the required depths at the selected locations, sampling and testing the overburden.

Sampling and In Situ Testing

Soil samples were recovered using a 50 mm diameter split-spoon sampler or from the auger flights. The split spoon and auger samples were classified on site and placed in sealed plastic bags. All soil samples were transported to our laboratory. Bedrock was cored at three (3) borehole locations using diamond drilling techniques. The bedrock core was recovered from each core run, placed in cardboard boxes, and sent to our laboratory for further review. The depths at which the split spoon, auger and bedrock samples were recovered from the boreholes are shown as SS, AU, and RC, respectively, on the Soil Profile and Test Data sheets in Appendix 1.

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

Diamond drilling using BX size coring equipment was carried out in three (3) boreholes to determine the nature of the bedrock and to assess its quality.

Recovery value and Rock Quality Designation (RQD) value were calculated for each drilled section (core run) of bedrock and are shown on the Soil Profile and Test Data sheets in Appendix 1.

The recovery value is the ratio, in percentage, of the length of the bedrock sample recovered over the length of the drilled section (core run). The RQD value is the ratio, in percentage, of the total length of rock pieces longer than 100 mm in one core run over the length of the core run. These values are indicative of the quality of the bedrock.

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

Groundwater

Flexible polyethylene standpipes and three (3) monitoring wells were installed at selected borehole locations (BH 1, BH 2 and BH 12) to permit the monitoring of groundwater levels subsequent to the completion of the sampling program.

Typical monitoring well construction details are described below:

- ☐ Slotted 50 mm diameter PVC screen.
- ☐ 50 mm diameter PVC riser pipe from the top of the screen to the ground surface.
- ☐ No.3 silica sand backfill within annular space around screen.
- ☐ Bentonite hole plug placed from 300 mm above PVC slotted screen to near ground surface.
- ☐ Clean backfill from top of bentonite plug to the ground surface.

Refer to the Soil Profile and Test Data sheets in Appendix 1 for specific monitoring well construction details.

3.2 Field Survey

The borehole locations were selected, determined in the field and surveyed by Paterson. The ground surface elevation at each borehole location was referenced to a temporary benchmark (TBM), consisting of the top of spindle of the fire hydrant, located near the intersection of Michael Street and Cyrville Road. A geodetic elevation of 70.62 m was provided for the TBM.

The location and ground surface elevations at borehole locations are presented on Drawing PG6072-1 – Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

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

Representative soil samples were selected for moisture content testing. The test results have been logged on the Soil Profile and Test Data sheets in Appendix 1 of this report.

4.0 Observations

4.1 Surface Conditions

The subject site is presently vacant, relatively flat and covered by gravel and asphalt parking areas and access lanes with some treed and landscaped areas.

Based on available information, an approximately 7 m service easement bisects the central portion of the site into an east and west parcel, and is occupied by the 1900 mm diameter Trunk Sewer.

The site is bordered to the north by commercial buildings and vacant land, to the east by institutional and commercial buildings, to the south by Cyrville Road and to the west by commercial buildings.

4.2 Subsurface Profile

The soil profile encountered at the test hole locations consists of pavement structure, topsoil or crushed stone fill at ground surface underlain by brown silty clay with gravel and/or silty sand with gravel. A weathered shale bedrock was encountered below the above noted layers at all borehole locations. Shale bedrock was cored at BH 1, BH 2 and BH 12 to a maximum depth of 5.7 m below existing ground surface. Specific details of the soil profile at the test hole locations can be seen on the Soil Profile and Test Data Sheets in Appendix 1.

Based on available geological mapping, the bedrock in the immediate area of the subject site consists of potentially expansive shale from the Billings Formation at a 2 to 5 m depth.

4.3 Groundwater

The test holes were noted to be dry upon completion of the field program. Groundwater levels at the borehole locations were measured on August 22, 2011, and the results are presented in Table 1 below. It should be noted that groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater levels could vary at the time of construction.

Table 1 – Summary of Groundwater Levels

Test Hole Number	Ground Surface Elevation (m)	Measured Groundwater Level		Dated Recorded
		Depth (m)	Elevation (m)	
BH 1*	68.66	2.29	66.37	August 22, 2011
BH 2*	69.49	2.94	66.55	
BH 3	69.54	Dry	N/A	
BH 4	68.66	1.98	66.68	
BH 7	69.16	Dry	N/A	
BH 9	68.65	Dry	N/A	
BH 12	68.89	Dry	N/A	August 22, 2011
BH 11	70.24	3.29	66.95	

Notes:
* - Monitoring wells were installed.
Borehole locations not indicated were inaccessible or could not be located.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is suitable for the proposed development. It is expected that the proposed building will be founded by conventional footings placed on the shale bedrock surface.

Bedrock removal is anticipated to be required to complete the underground parking levels. Line drilling and controlled blasting where large quantities of bedrock need to be removed is recommended. The blasting operations should be planned and completed under the guidance of a professional engineer with experience in blasting operations. A vibration monitoring program should be implemented and monitored by the geotechnical consultant to confirm that the controlled blasting program does not negatively impact the existing structures and utilities at and/or in the vicinity of the site, including the 1900 mm diameter Trunk Sewer which runs through the central portion of the site.

Protection of Existing Services

Due to the depth and proximity of the existing 1900 mm diameter Trunk Sewer with respect to the proposed structure, it is recommended that the adjacent easement be protected over the course of the construction period. Furthermore, a sewer pipe monitoring program is recommended to ensure that excessive settlement and vibrations do not occur at the sewer pipe location.

It will also be important to ensure that the building loads of the proposed multi-storey structures are extended below the invert level of the existing service pipe in order to permit future repairs to the service pipe without resulting in temporary shoring or underpinning of the multi-storey structures.

Temporary Shoring Requirements

It is understood that a temporary shoring system will be in place during the excavation program for the proposed structures. For design purposes, the temporary system may consist of soldier pile and lagging system or interlocking steel sheet piling. However, due to the blast rock and bouldery fill encountered within the embankment, the site may not be suitable for interlocking steel sheet piling.

The temporary shoring system will be required to support the adjacent roadways and neighboring properties surrounding the site from all sides. In addition, the temporary shoring system will be required to adequately support the soil below the majority of the northern side of the existing sanitary trunk sewer within the middle of the site. Refer to Figure 1 - Cross-Section A provided in Appendix 2.

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

5.2 Site Grading and Preparation

Stripping Depth

Asphalt, topsoil or fill, containing significant amounts of organic materials, should be stripped from under any buildings, paved areas, pipe bedding and other settlement sensitive structures.

Existing foundation walls and other construction debris should be entirely removed from within the proposed building perimeter. Under paved areas, existing construction remnants such as foundation walls should be removed to a minimum of 1 m below final grade.

Bedrock Removal

Bedrock removal can be accomplished by hoe ramming where the bedrock is severely weathered or where only a small quantity of bedrock needs to be removed. Sound bedrock may be removed by line drilling in conjunction with controlled blasting and/or hoe ramming.

Prior to considering blasting operations, the blasting effects on the existing services, building, and other structures should be addressed. A pre-blast or pre-construction survey of the existing structures located in the proximity of the blasting operations should be carried out prior to commencing site works.

As a general guideline, peak particle velocities (measured at the structures) should not exceed 25 mm/s during the blasting program in order to reduce the risks of damage to the existing surrounding structures and utilities. The blasting operations should be planned and conducted under the supervision of a licensed professional engineer who is also an experienced blasting consultant.

Vibration Considerations

Construction operations could be 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 a cooperative environment with the residents.

The following construction equipment 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 or the source of detrimental vibrations at the nearby buildings and structures. Therefore, it is recommended that all vibrations be limited.

Two parameters determine the permissible vibrations, 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, a pre-construction survey is recommended to minimize the risks of claims during or following the construction of the proposed building.

Horizontal Rock Anchors

Bedrock stabilization may be required where the proposed foundation extends into the sound bedrock.

Rock anchors and rock face protection may be required at specific locations to prevent pop-outs of the bedrock, especially in areas where bedrock fractures are conducive to the failure of the bedrock surface.

The requirement for rock face protection and rock anchors within the sound bedrock should be evaluated during the excavation operations and should be discussed with the structural engineer during the design stage.

Fill Placement

Fill used for grading beneath the building areas should consist, unless otherwise specified, of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. This material should be tested and approved prior to delivery to the site. 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 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 where settlement of the ground surface is of minor concern. 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 used in conjunction with a composite drainage membrane.

5.3 Foundation Design

Bearing Resistance Values

Footings placed on a clean, surface-sounded bedrock surface can be designed using a bearing resistance value at SLS of **500 kPa** and a factored bearing resistance value at ULS of **750 kPa**. A geotechnical resistance factor of 0.5 was applied to the reported bearing resistance values at ULS.

A higher bearing capacity can be provided for footings placed at a higher quality bedrock of up to **2,000 kPa (ULS)** provided the bedrock is reviewed and approved by Paterson, once exposed.

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

The potential long term post-construction total and differential settlements for footings placed on surface-sounded bedrock are estimated to be negligible.

Footings placed on engineered fill approved by the geotechnical consultant can be designed using a bearing resistance value at SLS of **150 kPa** and a factored bearing resistance value at ULS of **300 kPa**. Engineered fill should consist of an OPSS Granular A or Granular B Type II material placed in maximum 300 mm loose lifts and compacted to a minimum 98% of its SPMDD.

An undisturbed soil bearing surface consists of a surface from which all topsoil and deleterious materials, such as loose, frozen or disturbed soil, whether in situ or not, have been removed, in the dry, prior to the placement of concrete for footings.

Footings placed on an engineered fill bearing medium and designed using the above-noted bearing resistance value at SLS will be subjected to potential post-construction total and differential settlements of 25 and 20 mm, respectively.

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 horizontally and vertically from the footing perimeter at a minimum of 1H:6V (or shallower) passes through sound bedrock or a material of the same or higher capacity, such as concrete.

Adequate lateral support is provided to an engineered fill bearing medium when a plane extending down and out from the bottom edge of the footing at a minimum of 1.5H:1V, passes only through in situ soil or engineered fill of the same or higher capacity as the soil.

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 (OBC) 2012. The shear wave velocity testing was completed by Paterson personnel. The shear wave velocity profiles from our testing are presented in Figures 2 and 3 in Appendix 2.

Field Program

The shear wave testing location is presented in Drawing PG6072-1 - Test Hole Location Plan in Appendix 2. Paterson field personnel placed 24 horizontal geophones in a straight line oriented roughly along north-south within the east portion of the site. The 4.5 Hz. horizontal geophones were 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 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 the centre of the geophone array and 1, 2 and 10.5 m away from the first and last geophones.

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.

It is expected that the footings of the proposed building will be founded on the bedrock surface. Based on our analysis, the bedrock shear wave velocity was calculated to be 1,951 m/s.

The Vs_{30} was calculated using the standard equation for average shear wave velocity calculation from the Ontario Building Code (OBC) 2006, as presented below.

$$V_{s30} = \frac{Depth_{OfInterest}(m)}{\left(\frac{(Depth_{Layer1}(m)}{Vs_{Layer1}(m/s)} + \frac{Depth_{Layer2}(m)}{Vs_{Layer2}(m/s)}\right)}$$

$$V_{s30} = \frac{30m}{\left(\frac{30m}{1{,}951m/s}\right)}$$

$$V_{s30} = 1{,}951m/s$$

Based on the results of the seismic testing, the average shear wave velocity, Vs_{30}, for shallow foundations located at the subject site is 1,951 m/s. Therefore, a Site Class A is applicable for the proposed building, as per Table 4.1.8.4.A of the OBC 2012. The soils underlying the site are not susceptible to liquefaction.

5.5 Basement Slab

With the removal of all topsoil, and deleterious fill, containing organic matter, within the footprint of the proposed building, the native soil or engineered fill surface will be considered to be an acceptable subgrade surface on which to commence backfilling for floor slab construction.

The basement area for the proposed project 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 constructed is considered, the upper 200 mm of sub-slab fill is recommended to consist of 19 mm clear crushed stone. All backfill material within the footprint of the proposed building(s) should be placed in maximum 300 mm thick loose layers and compacted to a minimum of 98% of the SPMDD.

Any soft or poor performing areas within the subgrade should be removed and backfilled with appropriate backfill material. OPSS Granular B Type II is recommended for backfilling below the floor slab. It is recommended that the upper 200 mm of sub-slab fill consist of OPSS Granular A crushed stone compacted to 98% of the material's SPMDD.

In consideration of the groundwater conditions encountered at the time of the construction, a subfloor drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet such as the building's sump pit, should be provided in the clear stone under the basement floor.

5.6 Basement Wall

Below the bedrock surface, it is expected that the basement walls are to be poured against a composite drainage blanket, which will be placed against the exposed bedrock face. A nominal coefficient of at-rest earth pressure of 0.25 is recommended in conjunction with a bulk unit weight of 24.5 kN/m^3 (effective 15.5 kN/m^3). A seismic earth pressure component will not be applicable for the foundation wall, which is to be poured against the bedrock face. It is expected that the seismic earth pressure will be transferred to the underground floor slabs, which should be designed to accommodate these pressures. A hydrostatic groundwater pressure should be added for the portion below the groundwater level.

There are several combinations of backfill materials and retained soils that could be applicable for the basement walls of the subject structure. However, the conditions can be well-represented by assuming the retained soil consists of a material with an angle of internal friction of 30-degrees and a bulk (drained) unit weight of 20 kN/m^3.

However, undrained conditions are anticipated (i.e., below the groundwater level). Therefore, the applicable effective (undrained) unit weight of the retained soil can be taken as 13 kN/m^3, where applicable. A hydrostatic pressure should be added to the total static earth pressure when using the effective unit weight. The total earth pressure (P_{AE}) includes both the static earth pressure component (P_O) and the seismic component (ΔP_{AE}).

Lateral Earth Pressures

The static horizontal earth pressure (p_o) can be calculated using 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 having 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 Earth Pressures

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}) can be calculated using $0.375 \cdot a_c \cdot \gamma \cdot H^2/g$ where:

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

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

The earth force component (P_o) under seismic conditions can be calculated using $P_o = 0.5 K_o \gamma H^2$, where $K_o = 0.5$ for the soil conditions noted 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 Rock Anchor Design

The geotechnical design of grouted rock anchors in bedrock is based upon two possible failure modes. The rock anchor can fail either by shear failure along the grout/rock interface or by pullout at 60 to 90 degree cone of rock with the apex of the cone near the middle of the bonded length of the anchor. Interaction may develop between the failure cones of anchors that are relatively close to one another resulting in a total group capacity smaller than the sum of the individual anchor load capacity.

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

Typical rock anchor suppliers, such as Dywidag Systems International (DSI Canada) or Williams Form Engineering, have qualified personnel on staff to recommend appropriate rock anchor size and materials.

Anchors in close proximity to each other are recommended to be grouted at the same time to ensure any fractures or voids are completely in-filled and grout fluid does not flow from one hole to an adjacent empty one.

Anchors can be of the “passive” or the “post-tensioned” type, depending on whether the anchor tendon is provided with post-tensioned load or not, prior to servicing. To resist seismic uplift pressures, a passive rock anchor system is adequate. However, a post-tensioned anchor will absorb the uplift load pressure with less deflection than a passive anchor.

Regardless of whether an anchor is of the passive or the post tensioned type, it is recommended that the anchor is provided with a fixed anchor length at the anchor base, which will provide the anchor capacity, and a free anchor length between the rock surface and the top of the bonded length. As the depth at which the apex of the shear failure cone develops midway along the bonded length, a fully bonded anchor would tend to have a much shallower cone, and therefore less geotechnical resistance, than one where the bonded length is limited to the bottom part of the overall anchor.

Permanent anchors should be provided with corrosion protection. As a minimum, this requires that the entire drill hole be filled with cementitious grout. The free anchor length is provided by installing a sleeve to act as a bond break, with the sleeve filled with grout. Double corrosion protection can be provided with factory assembled systems, such as those available from Dywidag Systems International or Williams Form Engineering Corp.

Grout to Rock Bond

Generally, an unconfined compressive strength of 10 to 35 MPa can be assigned to fractured shale, which is poorer than most routine grouts. A factored tensile grout to rock bond resistance value at ULS of **0.5 MPa**, incorporating a resistance factor of 0.3, should be provided. A minimum grout strength of 40 MPa is recommended.

Rock Cone Uplift

The rock anchor capacity depends on the dimensions of the rock anchors and the anchorage system configuration. Based on existing bedrock information, a **Rock Mass Rating (RMR) of 25** was assigned to the bedrock, and Hoek and Brown parameters (**m and s**) were taken as **0.055 and 0.000007**, respectively.

Recommended Grouted Rock Anchor Lengths

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

Table 2 - Parameters used in Rock Anchor Review	
Grout to Rock Bond Strength - Factored at ULS	0.5 MPa
Compressive Strength - Grout	40 MPa
Rock Mass Rating (RMR) – Poor Quality Shale Hoek and Brown parameters	25 m=0.055 and s=0.000007
Unconfined compressive strength - Shale	10 MPa
Unit weight - Submerged Bedrock	24 kN/m^3
Apex angle of failure cone	60°
Apex of failure cone	mid-point of fixed anchor length

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

Table 3 - Recommended Rock Anchor Lengths - Grouted Rock Anchor

Diameter of Drill Hole (mm)	Anchor Lengths (m)			Factored Tensile Resistance (kN)
	Bonded Length	Unbonded Length	Total Length	
75	2.5	3.5	6.0	250
	4.4	3.6	8.0	500
	8.5	3	11.5	1000
125	2.7	3.8	5.5	250
	3.5	4.5	7.0	500
	5.3	4.7	10.0	1000

It is recommended that the anchor drill hole diameter be within 1.5 to 2 times the rock anchor tendon diameter and the anchor drill holes be inspected by geotechnical personnel and should be flushed clean prior to grouting. The use of a grout tube to place grout from the bottom up in the anchor holes is further recommended.

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.

5.8 Pavement Structure

For design purposes, the pavement structures presented in the following tables could be used for the design of car only parking areas and access lanes.

Table 4 - Recommended Pavement Structure - Car Only Parking Areas

Thickness (mm)	Material Description
125	Reinforced Concrete Slab
200	**BEDDING** - OPSS Granular A Crushed Stone
SUBGRADE - Either in situ soils, fill approved by the geotechnical consultant or OPSS Granular B Type I or II material placed over in situ soil.	

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

Thickness (mm)	Material Description
40	**Wear Course** – HL-3 or Superpave 12.5 FC2 Asphaltic Concrete
50	**Binder Course** – HL-8 or Superpave 19.0 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
400	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Either in situ soils, fill approved by the geotechnical consultant or OPSS Granular B Type I or II material placed over in situ soil.	

It should be noted that all base and subbase granular fill consisting of OPSS Granular A or Granular B Type II should be placed in lifts no greater than 300 mm and compacted to a minimum of 100%. The subgrade should be reviewed and approved by Paterson at the time of construction.

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 I or II material.

This may require the use of a geotextile, thicker subbase, or other measures that can be recommended at the time of construction as part of the field observation program.

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for driveways and local roadways and (PG) 64-34 asphalt cement should be used for roadways with bus traffic.

6.0 Design and Construction Precautions

6.1 Foundation Design and Backfill

The following recommendations may be considered for the architectural design of the buildings foundation drainage systems. It is recommended that Paterson be engaged at the design stage of the future buildings (and prior to tender) to review and provide supplemental information for the buildings foundation drainage system design.

Supplemental details, review of architectural design drawings and additional information may be provided by Paterson for these items for incorporation in the building design packages and associated tender documents. It is recommended that Paterson review all details associated with the foundation drainage system prior to tender.

Groundwater Suppression System

It is recommended that a groundwater suppression system be provided for the proposed structures. It is expected that insufficient room will be available for exterior backfill and the foundation wall will be cast as a blind-sided pour against a shoring system and the bedrock surface. It is recommended that the groundwater suppression system consist of the following:

- ☐ A waterproofing membrane should be placed against the shoring system between underside of footings and 2 m below existing ground surface. The height of the waterproofing layer should be confirmed on a per-building basis, however, is expected to vary between 2 and 3 m below existing ground surface. Where the membrane will extend below the bedrock surface, it is recommended to consist of a membrane with a bentonite-lined face for being paced against the bedrock surface. The membrane is recommended to overlap below the overlying perimeter foundation footprint by a minimum of 1 m inwards towards the building footprint and from the face of the overlying foundation. This will allow construction to proceed without imposing groundwater lowering within the surrounding area of the proposed buildings in the short and long term conditions.

☐ A composite drainage membrane (DeltaDrain 6000, MiraDrain G100N or equivalent) should be placed against the HDPE face of the waterproofing membrane with the geotextile layer facing the waterproofing layer from finished ground surface to the top of the footing.

☐ The foundation drainage boards should be overlapped such that the bottom end of a higher board is placed in front of the top end of a lower board. All endlaps of the drainage board sheets should overlap abutting by a minimum of 150 mm. All overlaps should be sealed with a suitable adhesive and/or sealant material approved by the geotechnical consultant. It is highly recommended that the drainage board rolls be installed horizontally rather than vertically to minimize the number of vertical joints forming between the rolls.

☐ The bedrock face, where located within a building's excavation, is recommended to be grinded to provide a smooth surface for the installation of the waterproofing layer. Large cavities should be reviewed by Paterson as the excavation progresses to assess the requirement to in-fill cavities suitably to facilitate the installation of the waterproofing layer.

☐ It is recommended that 150 mm diameter PVC sleeves at 6 m centers be cast in the foundation wall at the foundation wall/footing interface to allow the infiltration of water to flow to the interior perimeter drainage pipe. The sleeves should be connected to openings in the HDPE face of the drainage board layer. The perimeter drainage pipe and underfloor drainage system should direct water to sump pit(s) within the lower basement area via an underfloor and interior drainage pipe system.

The top endlap of the foundation drainage board should be provided with a suitable termination bar against the foundation wall to mitigate the potential for water to perch between the drainage board and foundation wall.

Interior Perimeter and Underfloor Drainage

The interior perimeter and underfloor drainage system will be required to control water infiltration below the lowest underground parking level slab and redirect water from the building's foundation drainage system to the buildings sump pit(s). The interior perimeter and underfloor drainage pipe should consist of a 150 mm diameter corrugated perforated plastic pipe sleeved with a geosock.

The underfloor drainage pipe should be placed in each direction of the basement floor span and connected to the perimeter drainage pipe. The interior drainage pipe should be provided with tee-connections to extend pipes between the perimeter drainage line and the HDPE-face of the composite foundation drainage board via the foundation wall sleeves. The spacing of the underfloor drainage system should be confirmed by Paterson once the foundation layout and sump system location has been finalized.

Elevator Pit Waterproofing

The elevator shaft exterior foundation walls should be waterproofed to avoid any infiltration into the elevator pit. It is recommended that a waterproofing membrane, such as Colphene Torch'n Stick (or approved other) be applied to the exterior of the elavator shaft foundation wall.

The Colphene Torch'n Stick waterproofing membrane should extend over the vertical portion of the raft slab and down to the top of the footing in accordance with the manufacturer's specifications. A continuous PVC waterstop such as Southern waterstop 14RCB or equivalent should be installed within the interface between the concrete base slab below the elevator shaft foundation walls.

The 150 mm diameter perforated corrugated pipe underfloor drainage should be placed along the perimeter of the exterior sidewalls and provided a gravity connection to the sump pump basin or the elevator sump pit.

Foundation Backfill

Above the bedrock surface, backfill against the exterior sides of the foundation walls should consist of free draining non-frost susceptible granular materials. The greater part of the site excavated materials will be frost susceptible and, as such, are not recommended for re-use as backfill against the foundation walls, unless used in conjunction with a drainage geocomposite, such as Miradrain G100N or Delta Drain 6000, connected to the perimeter foundation drainage system.

Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should otherwise be used for this purpose.

Podium Deck Waterproofing Tie-In

Waterproofing layers for podium deck surfaces should overlap across and below the top end lap of the vertically installed composite foundation drainage board to mitigate the potential for water to migrate between the drainage board and foundation wall and as depicted in Figure 4 – Podium Deck to Foundation Wall Drainage System Tie-In Detail.

6.2 Protection of Footings Against Frost Action

Perimeter footings of heated structures are required to be insulated against the deleterious effects of frost action. A minimum of 1.5 m of soil cover alone should be provided in this regard.

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

It has been our experience that insufficient frost protection is typically provided to footings located in areas where minimal soil cover is available, such as entrance ramps to underground parking garages. Paterson requests permission to review design drawings prior to construction to ensure proper frost protection is provided.

6.3 Excavation Side Slopes and Temporary Shoring

The side slopes of excavations in the soil and fill overburden materials should either be cut back at acceptable slopes or should be retained by shoring systems from the start 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 undertaken by open-cut methods (i.e. unsupported excavations).

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.

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 the 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.

Temporary Shoring

Temporary shoring may be required for the overburden soil to complete the required excavations where insufficient room is available for open cut methods. The shoring requirements should be designed by a structural engineer, specializing in shoring design. The shoring will depend on the depth of the excavation, the proximity of the adjacent structures and the elevation of the adjacent building foundations, roadways and underground services. Detailed recommendations for temporary shoring to support the existing trunk sewer pipe are presented in the following section, *City Storm Trunk Easement*.

The design and implementation of the temporary systems will be the responsibility of the excavation contractor. The geotechnical information provided below is to assist the contractor in completing a safe shoring system.

The shoring designer should take into account the impact of a significant precipitation event and designate design measures to ensure that a precipitation event will not negatively impact the shoring system or soils supported by the system. Any changes during construction to the approved shoring design should be reported immediately to the owner’s consultants prior to implementation.

The temporary system could consist of soldier pile and lagging system or interlocking steel sheet piling. Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be included to the earth pressures described below. The shoring system could be cantilevered, anchored or braced. Generally, the shoring systems is provided with tie-back rock anchors to ensure the stability. The shoring system is recommended to be adequately supported to resist toe failure and inspected to ensure that the sheet piles extend well below the excavation base. If consideration is given to utilizing a raker style support for the shoring system, the structural engineer should ensure that the design selected minimizes lateral movements to tolerable levels.

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

Table 6 - Soil Parameters

Parameters	**Values**
Active Earth Pressure Coefficient (K_a)	0.33
Passive Earth Pressure Coefficient (K_p)	3
At-Rest Earth Pressure Coefficient (K_o)	0.5
Dry Unit Weight, kN/m^3	20
Effective Unit Weight, 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 movement is not permissible. The dry unit weight should be calculated above the groundwater level while the effective unit weight should be calculated below the groundwater level.

A hydrostatic groundwater pressure should be included to the earth pressure distribution wherever the effective unit weight is 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.

6.5 Pipe Bedding and Backfill

Bedding and backfill materials should be in accordance with the most recent Material Specifications and Standard Detail Drawings from the Department of Public Works and Services, Infrastructure Services Branch of the City of Ottawa.

The pipe bedding for sewer and water pipes should consist of at least 150 mm of OPSS Granular A material. The material should be placed in a maximum of

300 mm thick lifts and compacted to a minimum of 95% of its SPMDD. The bedding material should extend 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 a maximum of 300 mm thick loose lifts and compacted to a minimum of 95% of its 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 minimize differential frost heaving.

The trench backfill should be placed in a maximum of 300 mm thick loose lifts and compacted to a minimum of 95% of the material's SPMDD.

6.6 Groundwater Control

Groundwater Control for Building Construction

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 the shallow excavation.

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 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 of 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 50,000 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.

Impacts on Neighbouring Properties

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

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

6.7 Winter Construction

Precautions must be taken if winter construction is considered for this project. The subsoil conditions at this site mostly consist of frost susceptible materials. In 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, the founding stratum should be protected from freezing temperatures by the use of straw, propane heaters and tarpaulins or other suitable means. In this regard, the base of the excavations should be insulated from sub-zero temperatures immediately upon exposure and until such time as heat is adequately supplied to the building and the footings are protected with sufficient soil cover to prevent freezing at founding level.

The trench excavations should be carried out in a manner to avoid the introduction of frozen materials, snow or ice into the trenches. Precautions must be taken where excavations are carried out in proximity of existing structures which may be adversely affected due to the freezing conditions. In particular, it should be recognized that where a shoring system is used, the soil behind the shoring system will be subjected to freezing conditions and could result in heaving of the structure(s) placed within or above frozen soil. Provisions should be made in the contract document to protect the walls of the excavations from freezing, if applicable.

6.8 Protection of Potential Expansive Bedrock

It is expected that potentially expansive shale will be encountered at the subject site. The potential for heaving and rapid deterioration of the shale bedrock exists at this site. Our recommendations provided in Subsection 6.1 take into consideration the potential for heaving and rapid deterioration of the shale bedrock which could occur at the subject site.

6.9 Existing City Storm Trunk Easement

Paterson reviewed the following drawings prepared for the subject site:

- ❑ Multi-Family Residential Development – 1125-1149 Cyrville Road, Ottawa, ON – Job No. 2021-01O dated June 8, 2021, prepared by J+S Architect.
- ❑ Project No. 160401672 – Drawing No. SSP-1 – Site Servicing Plan, Revision 1 dated May 25, 2022, prepared by Stantec.

Based on our review of the above noted drawings, the existing 1900 mm diameter Storm pipe is expected to be founded over a weathered to surface sounded bedrock bearing surface.

Based on the clearance between the proposed buildings (Building A and B) and the existing underground service easement and the depth of the bedrock surface within the site, a vertical cut into the underlying bedrock is expected as an open cut excavation for the proposed project. Based on the available setbacks, the existing pipe is not anticipated to be impacted by the proposed excavation (lateral support in bedrock is 1H:6V) for the majority of the excavation.

However, the backfill surrounding the existing easement may be impacted by the excavation of the proposed underground levels of the proposed buildings at localized areas. In order to protect the backfill material surrounding the existing easement, a temporary shoring system or a temporary retaining wall can be used and should be designed by a professional engineer specialized in these works. Reference should be made to three cross-sections showing the storm sewer easement, identified as Figure 4 – Cross-Section A-A, Figure 5 – Cross-Section B-B and Figure 6 – Cross-Section C-C, presented in Appendix 2.

The temporary shoring system should consist of soldier piles and lagging with rock anchors/tie backs and should be designed by a professional engineer. The rock anchors can be used to provide an anchorage for the soldier piles to the bedrock surface.

Rock anchor design parameters are presented in Subsection 5.7. The location, inclination and depth of the rock anchors should take into account the presence of the sewer pipe. Based on our review, an insufficient excavation setback is available to prevent disturbing the backfill material of the existing pipe along the majority of the southern excavation limits of Building B. Therefore, a temporary shoring is required within the overburden to support the surrounding backfill material. Reference should be made to Figure 5 – Cross-Section B-B and Drawing PG6072-1 – Test Hole Location Plan, presented in Appendix 2.

Highly weathered Bedrock

Due to the presence of highly weathered shale bedrock along the upper 1 to 2 m of the bedrock surface, it is highly recommended that Paterson complete periodic inspections of the bedrock conditions once exposed. These areas are anticipated to require bedrock stabilization measures such as shotcrete, chain link fencing in conjunction with rock anchors, and/or full shoring system. The areas are highlighted on Figures 4, 5 and 6 in Appendix 2.

Settlement Monitoring Program

Based on the proximity of the proposed excavation to the existing trunk sewer pipe and the excavation depth, it is recommended a settlement monitoring program is conducted on the trunk sewer for the duration of construction until the excavation has been backfilled. Paterson has provided detailed recommendations for a settlement monitoring program under separate cover, presented in Appendix 3 attached to the current memorandum.

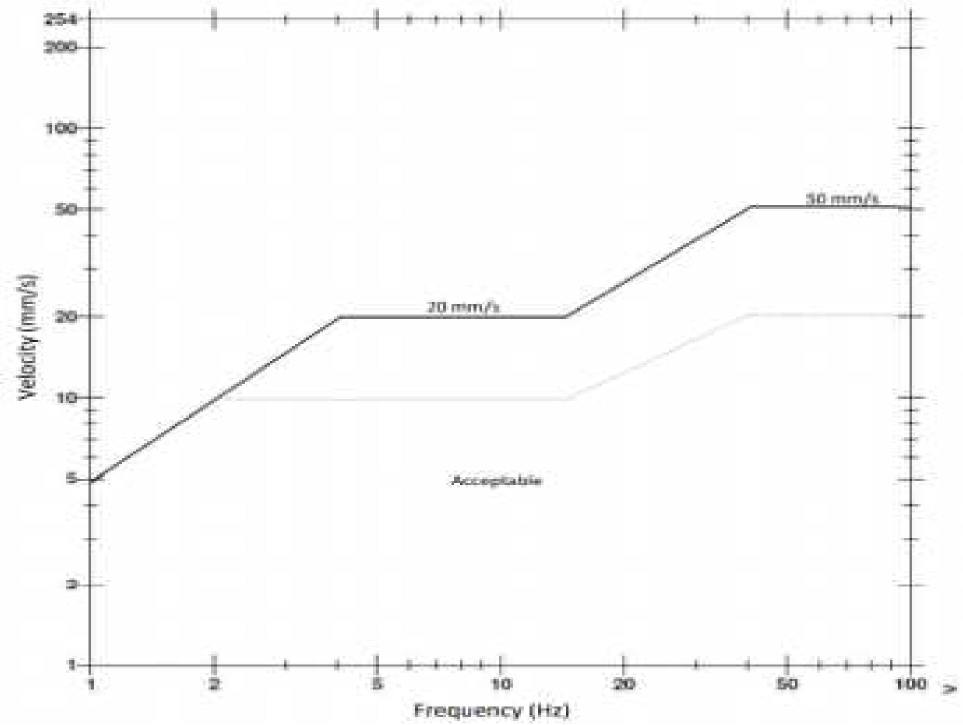
The following sewer pipe monitoring program is recommended to ensure that excessive settlement and vibrations do not occur at the sewer pipe location:

☐ Install 3 utility monitoring points and inclinometers directly on top of the 1900 mm diameter sewer. Further, it is recommended that three (3) inclinometers be installed adjacent to the sewer pipe and the adjacent shoring face for monitoring lateral deflection. Daily monitoring events should be completed during the excavation program until the tiebacks are stressed and then weekly during the construction program until the foundation extends above exterior finished grade. An alert level with 10 mm of movement will require an assessment. An action level with movement greater than 15 mm will require immediate attention and possible mitigation measures. A visual inspection will also be completed along with the monitoring events.

❑ Periodically monitor the vibration levels using 3 vibration monitors and inclinometers installed directly on the 1900 mm diameter sewer pipe.

❑ If the vibration limits provided on Vibration Criteria Figure are exceeded, the site superintendent will be notified by Paterson personnel of the exceedance and the shoring/excavation operation will be stopped. The project surveyor will survey the sewer pipe level to ensure pipe movement has not occurred. If pipe movement is not observed based on the survey results, the shoring/excavation operation will resume.

The following vibration limits are recommended for the shoring/excavation operation to be completed adjacent to the 1900 mm diameter sewer pipe.



Vibration Criteria Figure - Proposed Vibration Limits at the Sewer Pipe

Weekly reporting of our findings and recommendations will be provided to the owner and the City of Ottawa. Any mitigation measures contemplated for implementation will be discussed with the owner and City of Ottawa personnel.

7.0 Recommendations

It is recommended that the following be carried out by Paterson once preliminary and future details of the proposed development have been prepared:

☐ Review detailed grading, servicing, landscaping and structural plan(s) from a geotechnical perspective.

☐ Review of the geotechnical aspects of the excavation contractor's shoring design, if not design by Paterson, prior to construction, if applicable.

☐ Review of architectural plans pertaining to groundwater suppression system, underfloor drainage systems and waterproofing details for elevator shafts.

It is a requirement for the foundation design data provided herein to be applicable that a material testing and observation program be performed by the geotechnical consultant. The following aspects of the program should be performed by Paterson:

☐ Review and inspection of the installation of the foundation drainage systems.

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

☐ Sampling and testing of the concrete and fill materials.

☐ Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.

☐ Observation of all subgrades prior to backfilling and follow-up field density tests to determine the level of compaction achieved.

☐ 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 the completion of a satisfactory inspection program by the geotechnical consultant.

All excess soil must be handled as per *Ontario Regulation 406/19: On-Site and Excess Soil Management*

8.0 Statement of Limitations

The recommendations made in this report are in accordance with our present understanding of the project. We request that we be permitted to review the grading plan once available and our recommendations when the drawings and specifications are complete.

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. The extent of the limited area depends on the soil, bedrock and groundwater conditions, as well the history of the site reflecting natural, construction, and other activities. 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 recommendations provided in this report are intended for the use of design professionals associated with this project. Contractors bidding on or undertaking the work should examine the factual information contained in this report and the site conditions, satisfy themselves as to the adequacy of the information provided for construction purposes, supplement the factual information if required, and develop their own interpretation of the factual information based on both their and their subcontractors construction methods, equipment capabilities and schedules.

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 Westrich Pacific Corp 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.

LICENSED PROFESSIONAL ENGINEER
August 29, 2023
M. SALEH
100507739
PROVINCE OF ONTARIO

Owen Canton, EIT

Maha K. Saleh, M.A.Sc., P.Eng.

Report Distribution:

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- ☐ Paterson Group

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
Commercial Development - 1125-1149 Cyrville Road
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant, southwest corner of the intersection of Cyrville Road and Michael Street. Geodetic elevation = 70.62m.

REMARKS

BORINGS BY CME 55 Power Auger

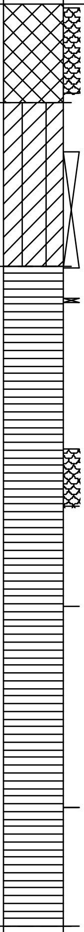
DATE August 16, 2011

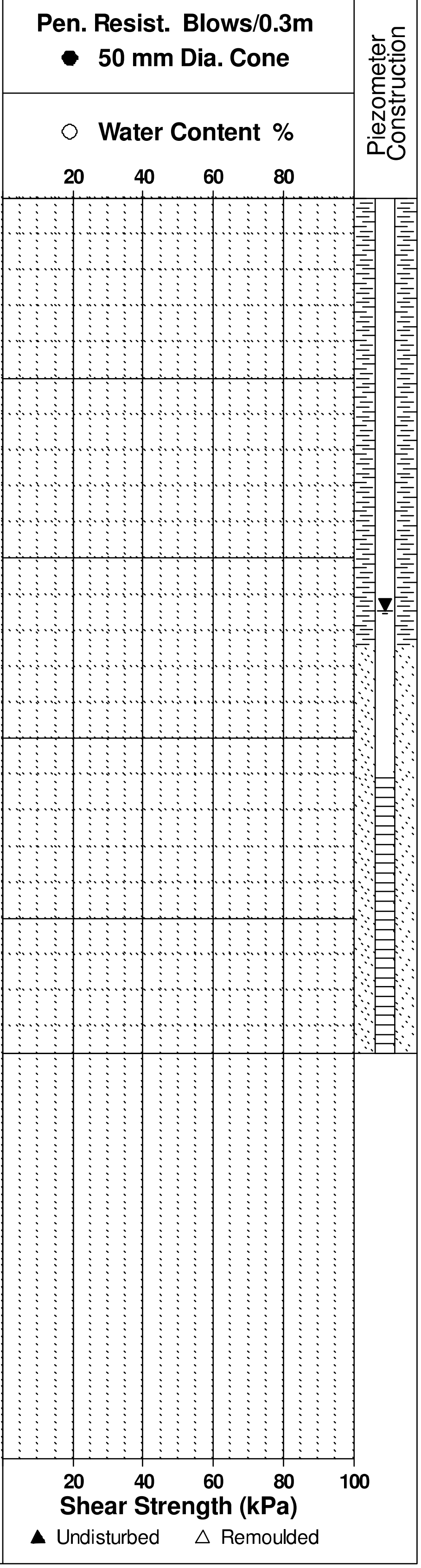
FILE NO. PG2471

HOLE NO. BH 1

SOIL DESCRIPTION	TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	68.66
25mm Asphaltic concrete **FILL:** Crushed stone 0.51	AU	1				
Brown **SILTY CLAY,** trace shale and organics 1.35	SS	2	46	10	1	67.66
BEDROCK: Heavily fractured and weathered, black shale	SS	3	0	50+		
					2	66.66
- moderately fractured by 3.0m depth	AU	4				
					3	65.66
	RC	1	100	93		
					4	64.66
4.75	RC	2	100	96		
End of Borehole (GWL @ 2.29m-Aug. 22/11)						

STRATA PLOT





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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Commercial Development - 1125-1149 Cyrville Road
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant, southwest corner of the intersection of Cyrville Road and Michael Street. Geodetic elevation = 70.62m.

REMARKS

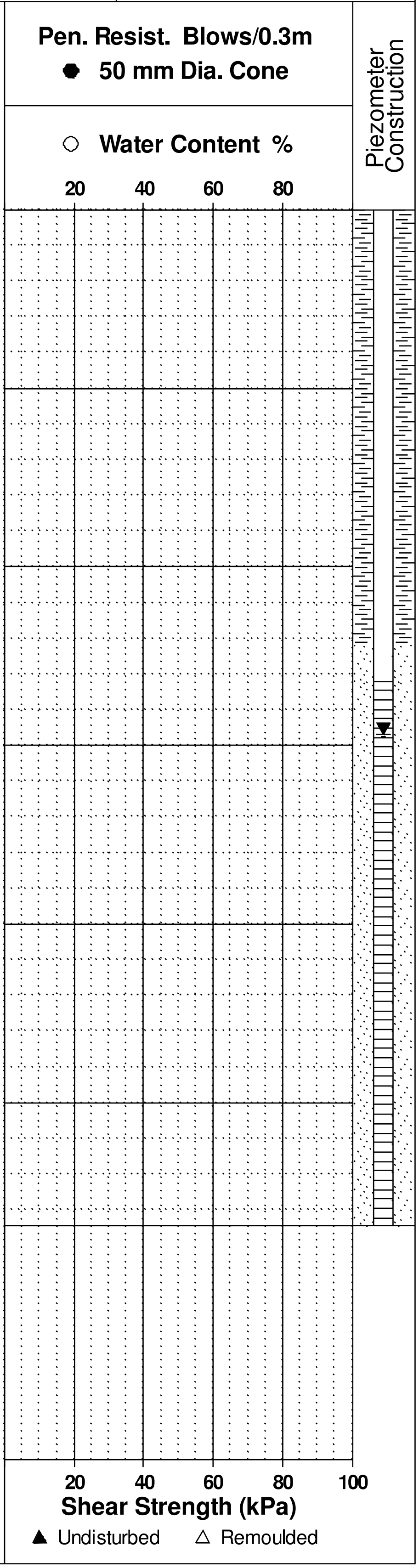
BORINGS BY CME 55 Power Auger

DATE August 16, 2011

FILE NO. PG2471

HOLE NO. BH 2

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	69.49
13mm Asphaltic concrete 0.25	AU	1				
FILL: Crushed stone						
FILL: Brown silty clay with sand, trace shale and gravel 1.45	SS	2	50	12	1	68.49
Black **SILTY CLAY** with sand seams and topsoil 1.83	SS	3		50+		
BEDROCK: Heavily fractured and weathered, black shale	RC	1	62	0	2	67.49
	RC	2	95	0	3	66.49
	RC	3	87	0	4	65.49
5.69	RC	4	97	14	5	64.49
End of Borehole						
(GWL @ 2.94m-Aug. 22/11)						



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Commercial Development - 1125-1149 Cyrville Road
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant, southwest corner of the intersection of Cyrville Road and Michael Street. Geodetic elevation = 70.62m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE August 16, 2011

FILE NO. PG2471

HOLE NO. BH 3

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	Piezometer Construction
GROUND SURFACE						0	69.54		
FILL: Crushed stone 0.30	▨								
FILL: Brown silty clay with sand, gravel, trace shale 0.76	▨								
End of Borehole Practical refusal to augering @ 0.76m depth									

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Commercial Development - 1125-1149 Cyrville Road
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant, southwest corner of the intersection of Cyrville Road and Michael Street. Geodetic elevation = 70.62m.

REMARKS

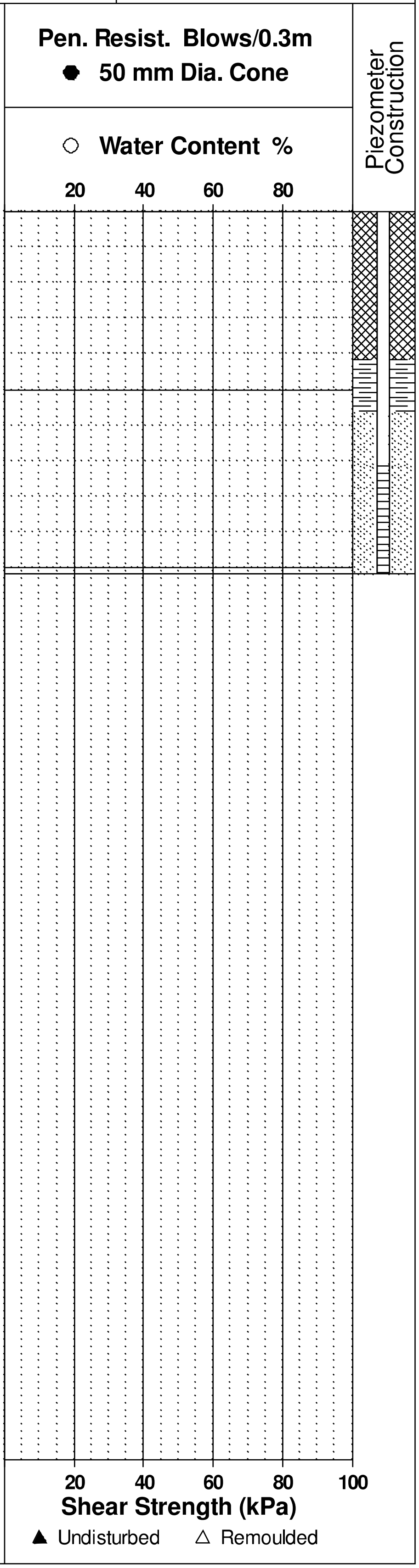
BORINGS BY CME 55 Power Auger

DATE August 16, 2011

FILE NO. PG2471

HOLE NO. BH 3A

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD		
GROUND SURFACE						0	69.54
FILL: Crushed stone 0.25		AU	1				
FILL: Brown silty sand with organics 1.07		SS	2	50	17	1	68.54
Brown **SILTY CLAY** with sand 1.45		SS	3	43	50+		
BEDROCK: Heavily fractured and weathered, black shale with mud seams 2.03						2	67.54
End of Borehole Practical refusal to augering @ 2.03m depth (BH dry - Aug. 22/11)							



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Commercial Development - 1125-1149 Cyrville Road
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant, southwest corner of the intersection of Cyrville Road and Michael Street. Geodetic elevation = 70.62m.

REMARKS

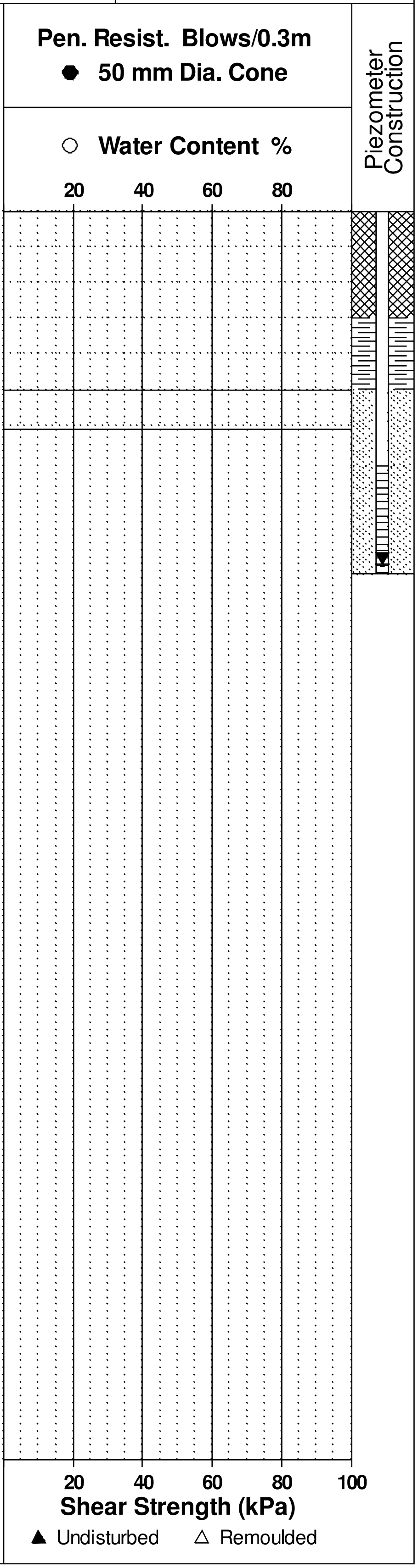
BORINGS BY CME 55 Power Auger

DATE August 16, 2011

FILE NO. PG2471

HOLE NO. BH 4

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	68.66
FILL: Crushed stone 0.25	▨						
FILL: Brown silty sand, trace gravel 0.89	▨						
TOPSOIL, trace clay and sand 1.09	■	SS	1	88	50+	1	67.66
BEDROCK: Heavily fractured and weathered, black shale 1.22	☰						
End of Borehole							
Practical refusal to augering @ 1.22m depth							
(GWL @ 1.98m-Aug. 22/11)							



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SOIL PROFILE AND TEST DATA

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Commercial Development - 1125-1149 Cyrville Road
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DATUM TBM - Top spindle of fire hydrant, southwest corner of the intersection of Cyrville Road and Michael Street. Geodetic elevation = 70.62m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE August 17, 2011

FILE NO. PG2471

HOLE NO. BH 5

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	Piezometer Construction
GROUND SURFACE						0	68.06		
FILL: Brown silty sand with gravel and shale 0.46	▨	SS	1	83	50+				
End of Borehole Practical refusal to augering @ 0.46m depth									

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

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
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Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant, southwest corner of the intersection of Cyrville Road and Michael Street. Geodetic elevation = 70.62m.

REMARKS

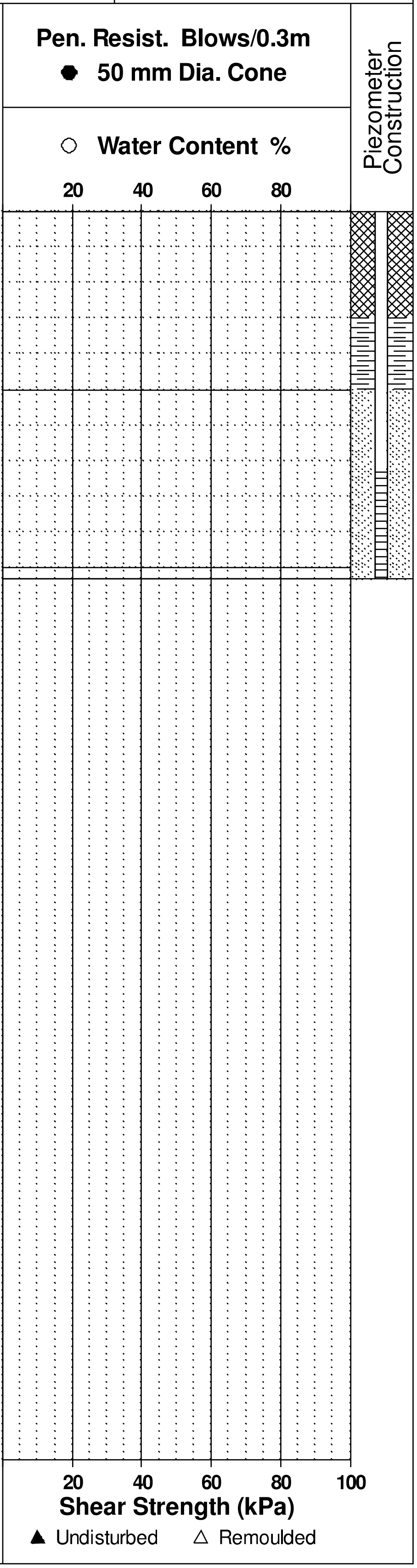
BORINGS BY CME 55 Power Auger

DATE August 17, 2011

FILE NO. PG2471

HOLE NO. BH 6

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD		
GROUND SURFACE						0	68.50
FILL: Brown silty sand with gravel and fractured shale 0.60		SS	1	42	8		
Brown **SILTY CLAY** with sand and fractured shale 1.22		SS	2	82	17	1	67.50
BEDROCK: Heavily fractured and weathered, black shale 2.06		SS	3	20	50+	2	66.50
End of Borehole Practical refusal to augering @ 2.06m depth							



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Commercial Development - 1125-1149 Cyrville Road
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant, southwest corner of the intersection of Cyrville Road and Michael Street. Geodetic elevation = 70.62m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE August 17, 2011

FILE NO. PG2471

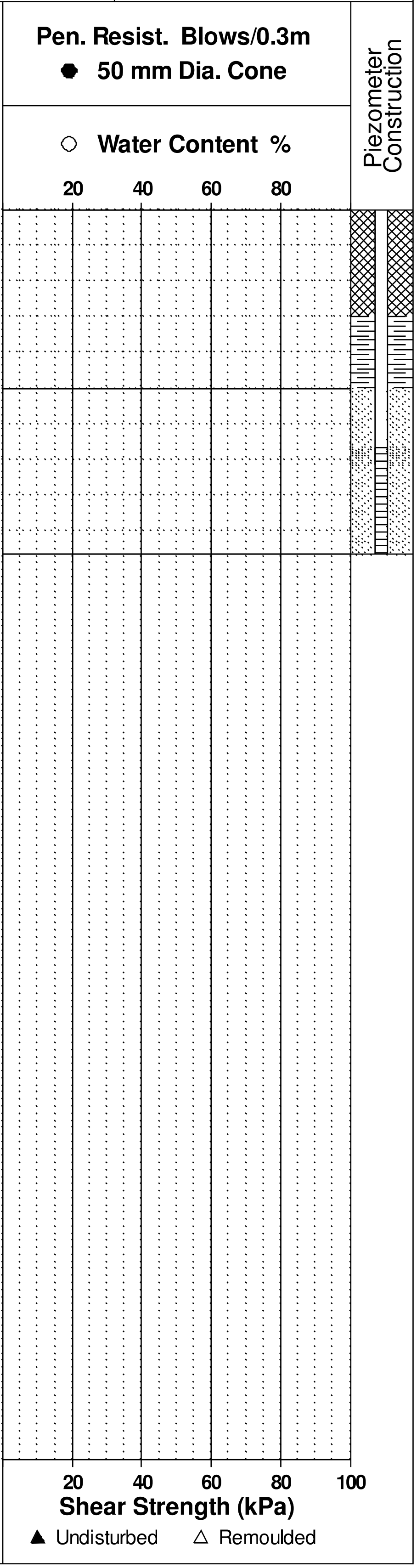
HOLE NO. BH 7

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	69.16
FILL: Crushed stone 0.25							
FILL: Brown silty sand with crushed stone, trace concrete 0.76		SS	1	54	38		
TOPSOIL with silty clay, trace gravel 1.68		SS	2	79	5	1	68.16
BEDROCK: Heavily fractured and weathered, black shale 1.93		SS	3	100	50+		

End of Borehole

Practical refusal to augering @ 1.93m depth

(BH dry - Aug. 22/11)



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Commercial Development - 1125-1149 Cyrville Road
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant, southwest corner of the intersection of Cyrville Road and Michael Street. Geodetic elevation = 70.62m.

REMARKS

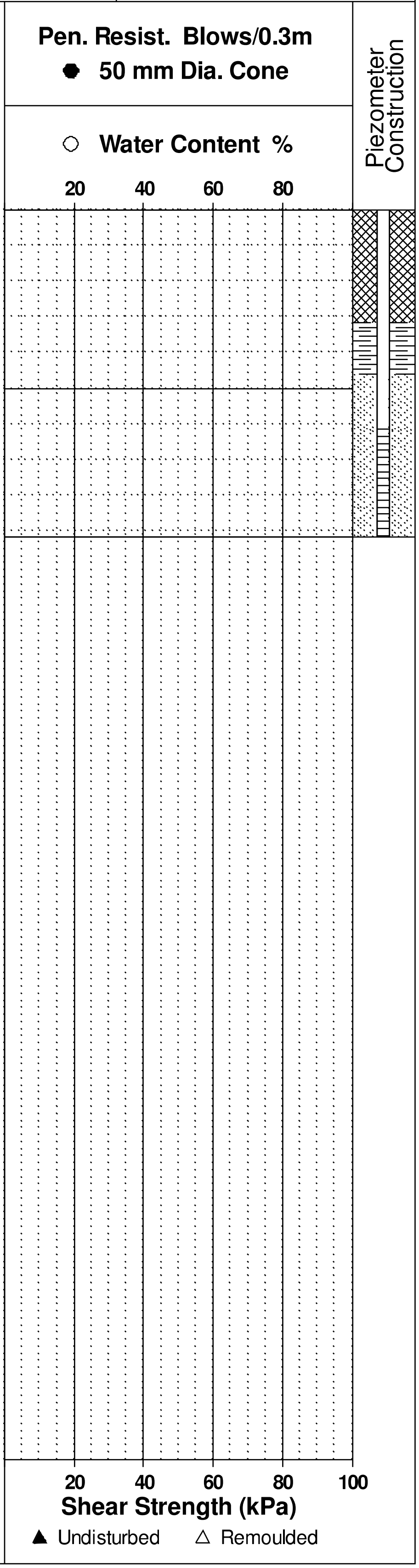
BORINGS BY CME 55 Power Auger

DATE August 17, 2011

FILE NO. PG2471

HOLE NO. BH 8

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	69.07
FILL: Crushed stone 0.33	▨						
Brown **SILTY CLAY** with sand, gravel, trace organics 1.37	▨	SS	1	15	6	1	68.07
BEDROCK: Heavily fractured, weathered, black shale 1.83	▤	SS	2	27	50+		
End of Borehole Practical refusal to augering @ 1.83m depth							



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Commercial Development - 1125-1149 Cyrville Road
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant, southwest corner of the intersection of Cyrville Road and Michael Street. Geodetic elevation = 70.62m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE August 17, 2011

FILE NO. PG2471

HOLE NO. BH 9

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	68.65
FILL: Crushed stone 0.25							
FILL: Brown silty sand, some crushed stone 0.60		SS	1	42	23		
FILL: Brown silty sand 0.91							
Dark brown **SILTY CLAY** with sand and topsoil 1.37		SS	2	19	7	1	67.65
BEDROCK: Heavily fractured and weathered, black shale 1.90		SS	3	67	50+		

End of Borehole

Practical refusal to augering @ 1.90m depth

(BH dry - Aug. 22/11)

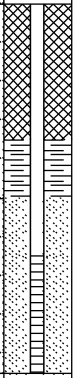
Pen. Resist. Blows/0.3m
● **50 mm Dia. Cone**

○ **Water Content %**

20 40 60 80

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

Piezometer Construction



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Commercial Development - 1125-1149 Cyrville Road
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant, southwest corner of the intersection of Cyrville Road and Michael Street. Geodetic elevation = 70.62m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE August 17, 2011

FILE NO. PG2471

HOLE NO. BH10

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	68.52
FILL: Crushed stone 0.25							
FILL: Brown silty sand with gravel 0.76		SS	1	46	41		
Brown **SILTY CLAY,** trace gravel and organics 1.50		SS	2	58	12	1	67.52
BEDROCK: Heavily fractured and weathered, black shale 1.65		SS	3	60	50+		
End of Borehole							
Practical refusal to augering @ 1.65m depth							

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

○ **Water Content %**

20 40 60 80

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

Piezometer Construction

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Commercial Development - 1125-1149 Cyrville Road
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant, southwest corner of the intersection of Cyrville Road and Michael Street. Geodetic elevation = 70.62m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE August 17, 2011

FILE NO. PG2471

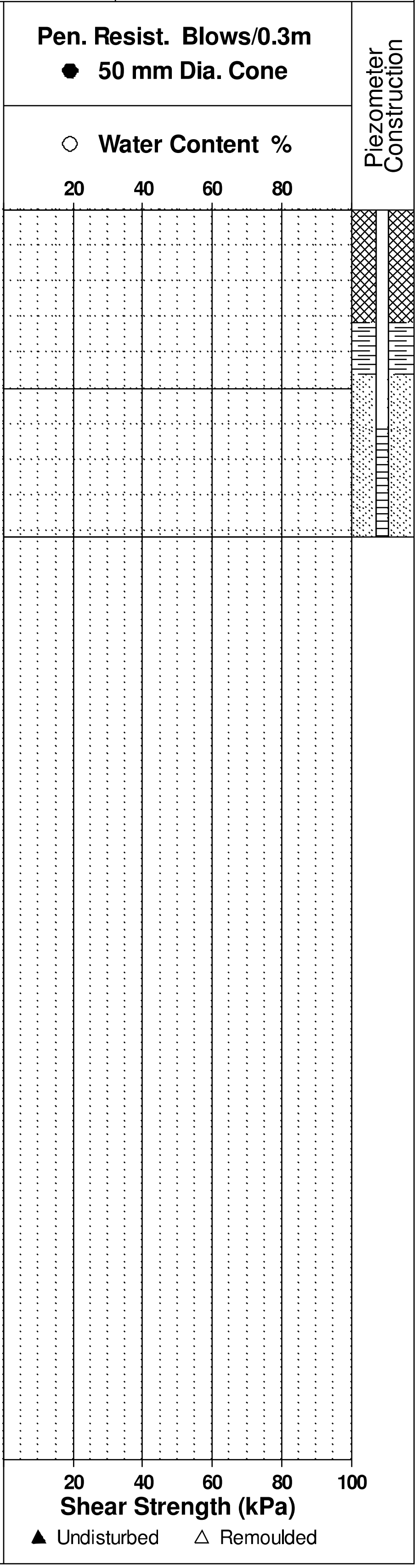
HOLE NO. BH11

SOIL DESCRIPTION		SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	68.89
FILL: Crushed stone	0.15						
FILL: Brown silty sand with gravel and shale, trace clay	0.60	SS	1	50	18		
FILL: Brown silty sand with gravel	0.91						
TOPSOIL	1.22	SS	2	46	8	1	67.89
Brown **SILTY CLAY** with sand and roots	1.37						
BEDROCK: Heavily fractured and weathered, black shale	1.83	SS	3	50	50+		

End of Borehole

Practical refusal to augering @ 1.83m depth

(BH dry - Aug. 22/11)



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Commercial Development - 1125-1149 Cyrville Road
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant, southwest corner of the intersection of Cyrville Road and Michael Street. Geodetic elevation = 70.62m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE August 17, 2011

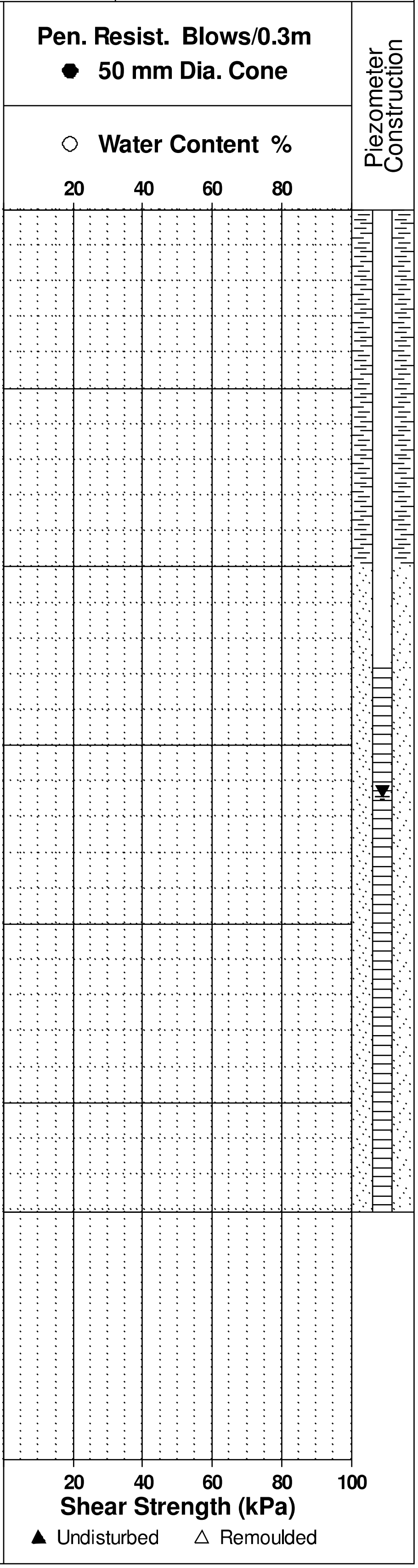
FILE NO. PG2471

HOLE NO. BH12

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	70.24
TOPSOIL 0.15						
FILL: Brown silty sand with clay and shale 0.60	AU	1				
FILL: Brown silty sand with clay, gravel and shale, some organics 1.37	SS	2	42	7	1	69.24
BEDROCK: Heavily fractured and weathered, black shale	SS	3	96	38	2	68.24
2.40	SS	4	100	50+		
BEDROCK: Fractured, black shale	RC	1	70	21	3	67.24
					4	66.24
	RC	2	87	42	5	65.24
5.61						

End of Borehole

(GWL @ 3.29m-Aug. 22/11)



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 relative strength of cohesionless soils is the compactness condition, 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. An SPT N value of "P" denotes that the split-spoon sampler was pushed 300 mm into the soil without the use of a falling hammer.

Compactness Condition	'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 shear vane tests, unconfined compression tests, or occasionally by the Standard Penetration Test (SPT). Note that the typical correlations of undrained shear strength to SPT N value (tabulated below) tend to underestimate the consistency for sensitive silty clays, so Paterson reviews the applicable split spoon samples in the laboratory to provide a more representative consistency value based on tactile examination.

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

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

Cohesive soils can also be classified according to their “sensitivity”. The sensitivity, S_t, is the ratio between the undisturbed undrained shear strength and the remoulded undrained shear strength of the soil. The classes of sensitivity may be defined as follows:

Low Sensitivity:	$S_t < 2$
Medium Sensitivity:	$2 < S_t < 4$
Sensitive:	$4 < S_t < 8$
Extra Sensitive:	$8 < S_t < 16$
Quick Clay:	$S_t > 16$

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 NQ or larger size core. However, it can be used on smaller core sizes, such as BQ, 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, generally recovered using a piston sampler
G	-	"Grab" sample from test pit or surface materials
AU	-	Auger sample or bulk sample
WS	-	Wash sample
RC	-	Rock core sample (Core bit size BQ, NQ, HQ, etc.). Rock core samples are obtained with the use of standard diamond drilling bits.

SYMBOLS AND TERMS (continued)

PLASTICITY LIMITS AND GRAIN SIZE DISTRIBUTION

WC% - Natural water 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 at 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)

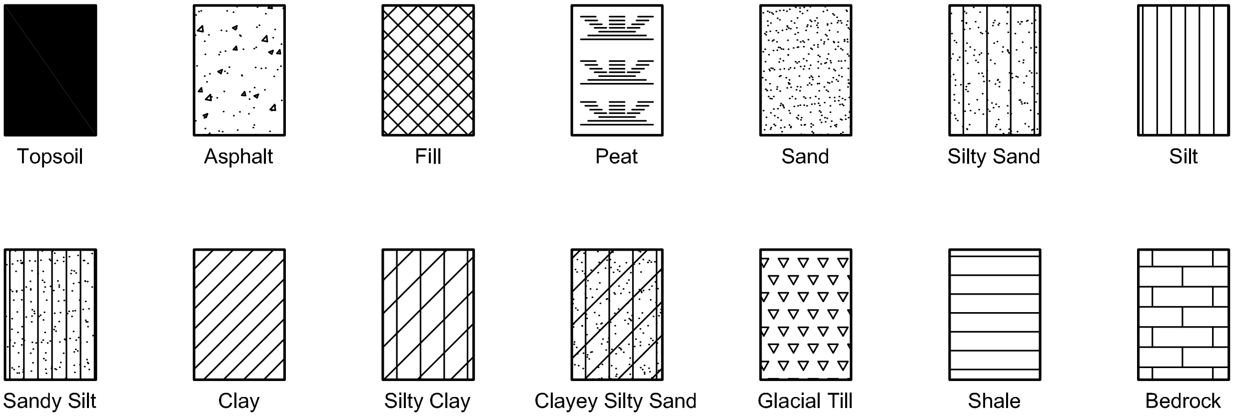
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

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.

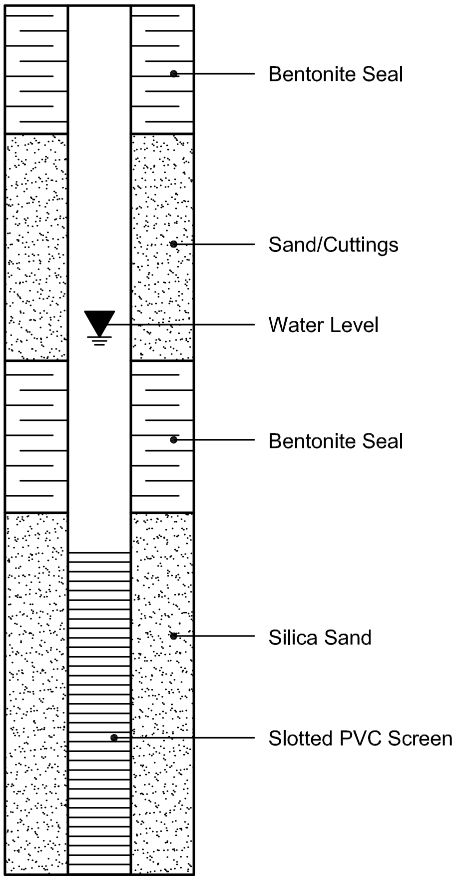
SYMBOLS AND TERMS (continued)

STRATA PLOT

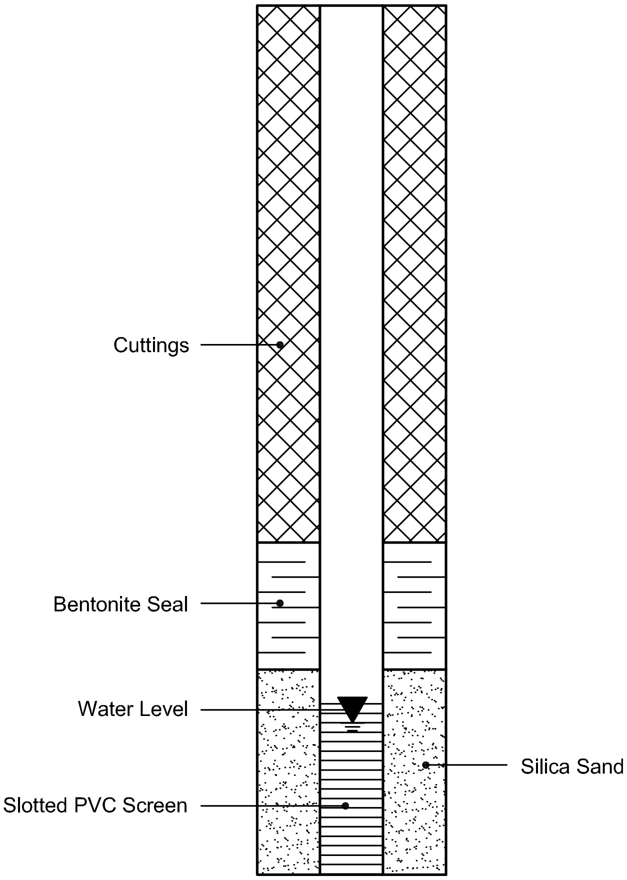


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



APPENDIX 2

FIGURE 1 – KEY PLAN

FIGURES 2 AND 3 – SEISMIC SHEAR WAVE VELOCITY PROFILES

FIGURES 4, 5 AND 6 – EXCAVATION CROSS SECTIONS

FIGURE 7 – PODIUM DECK TO FOUNDATION WALL DRAINAGE SYSTEM TIE-IN DETAIL

DRAWING PG6072-1 – TEST HOLE LOCATION PLAN

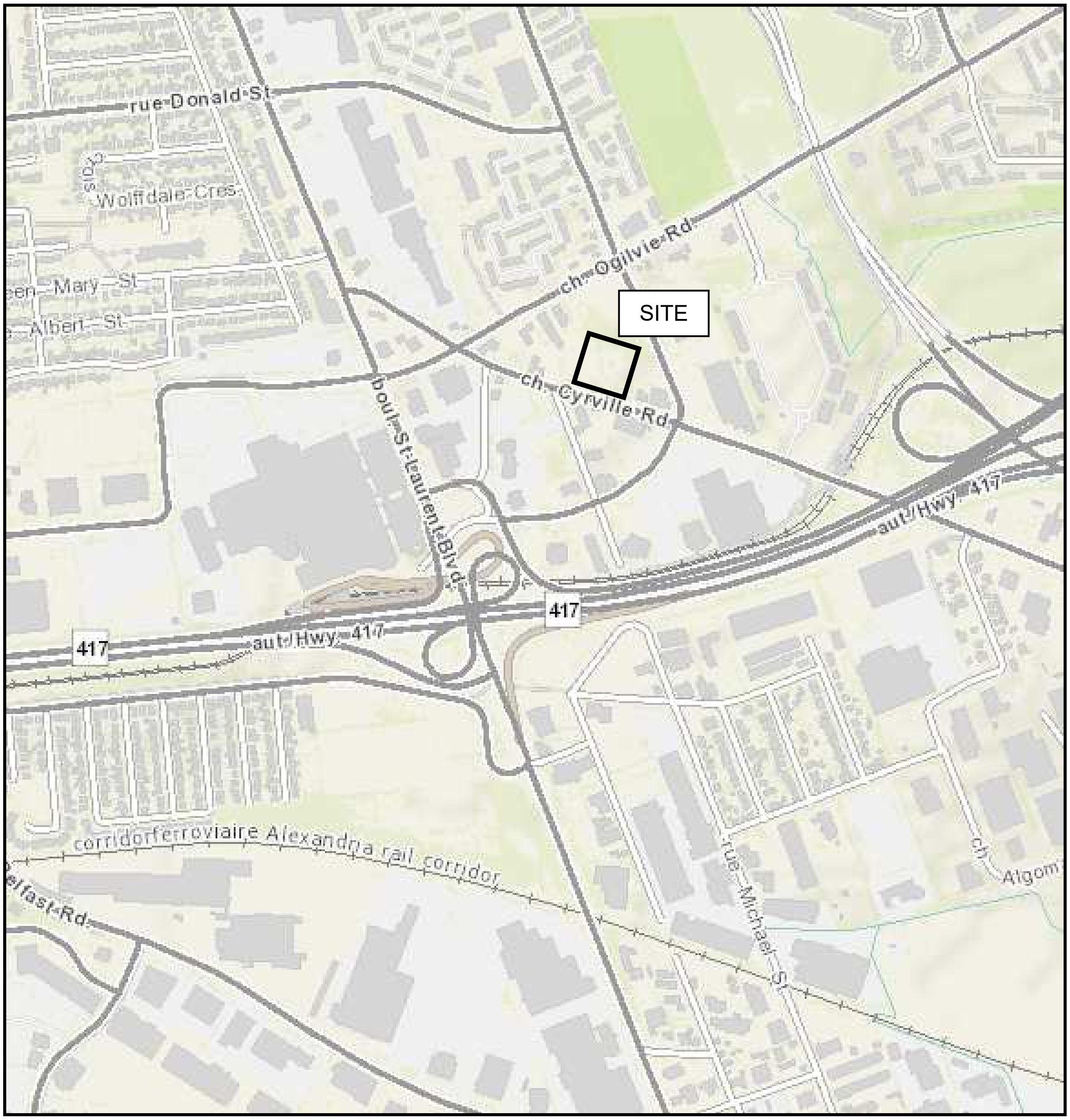


FIGURE 1

KEY PLAN

PATERSON
GROUP

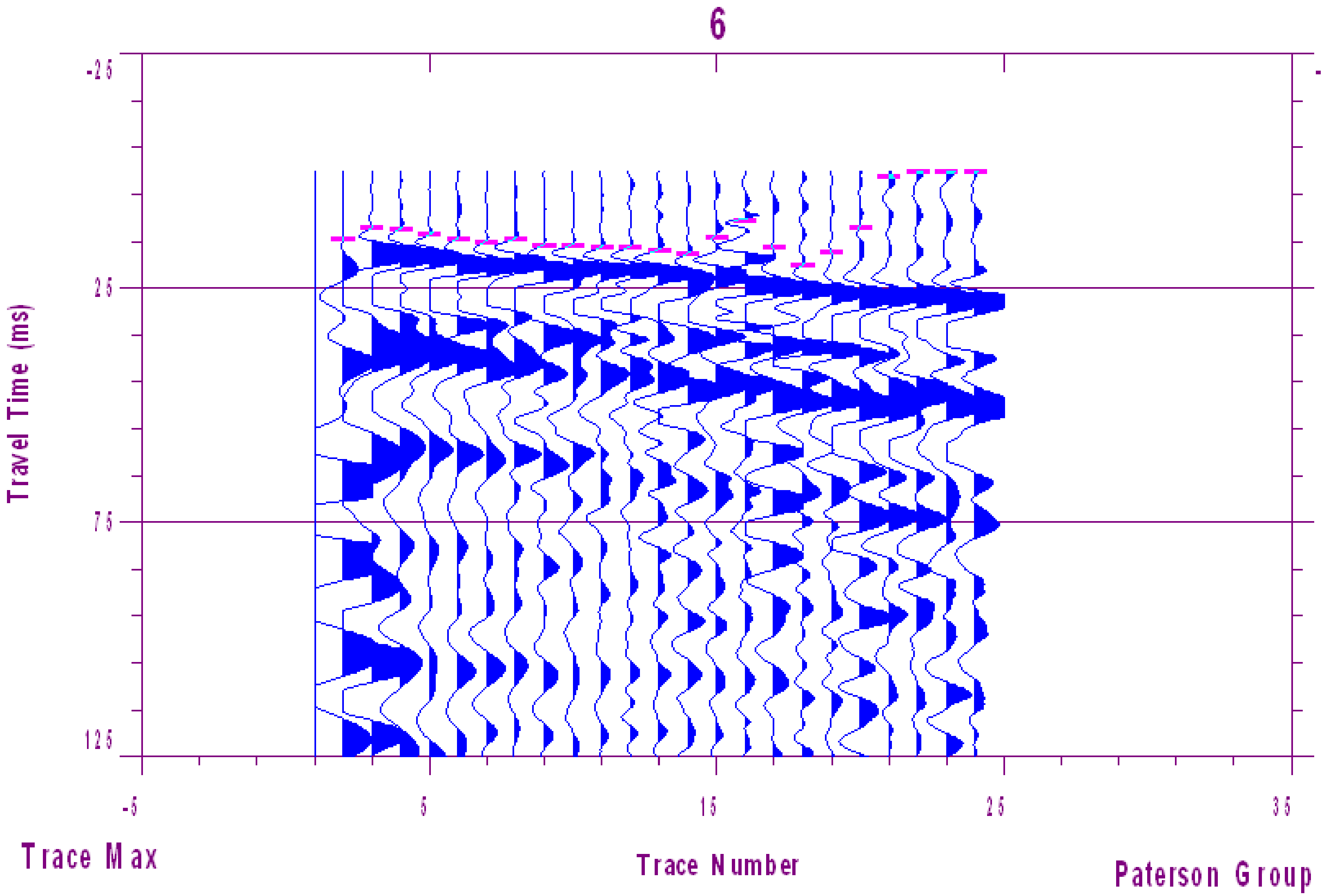


Figure 2 – Shear Wave Velocity Profile at Shot Location -1 m

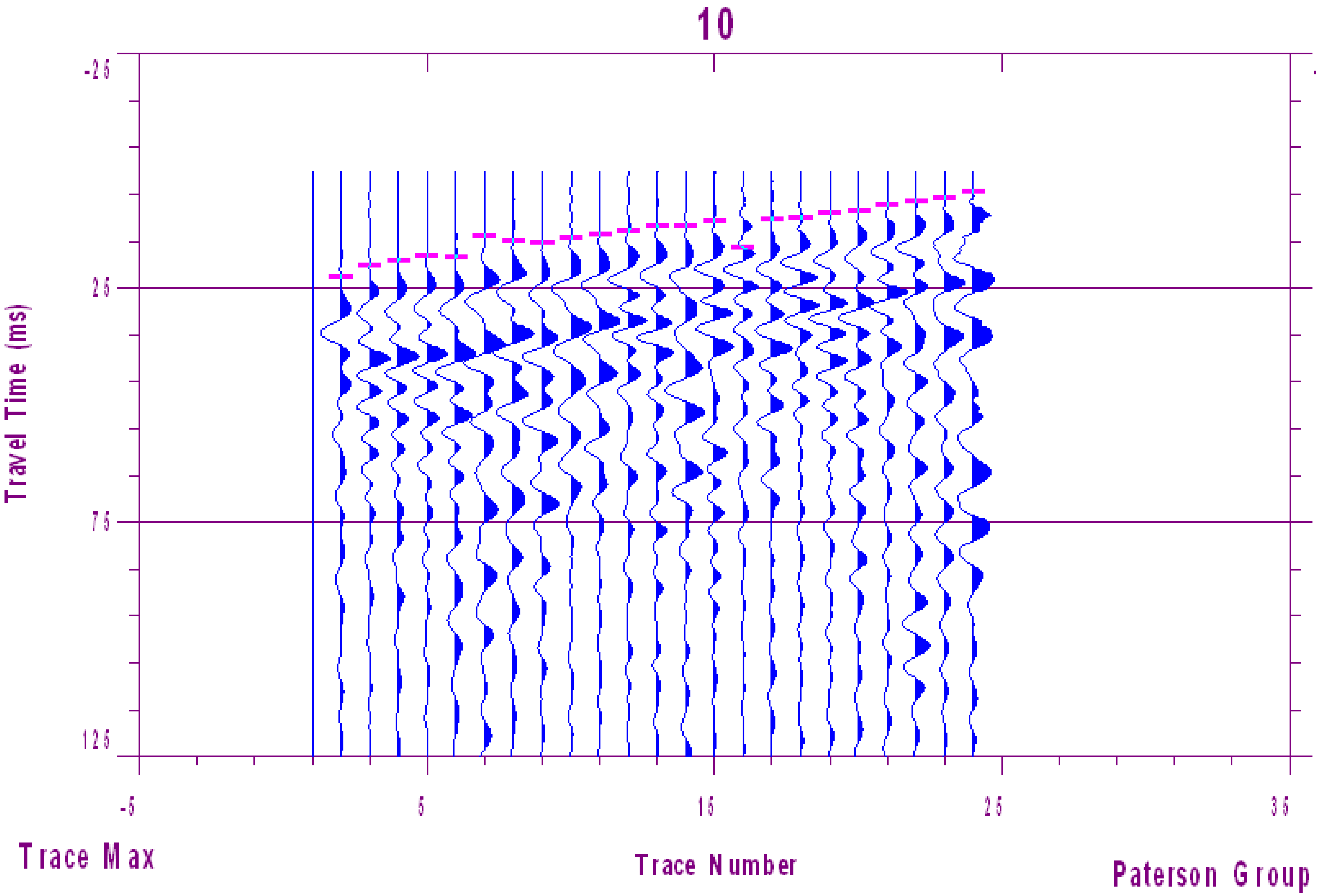
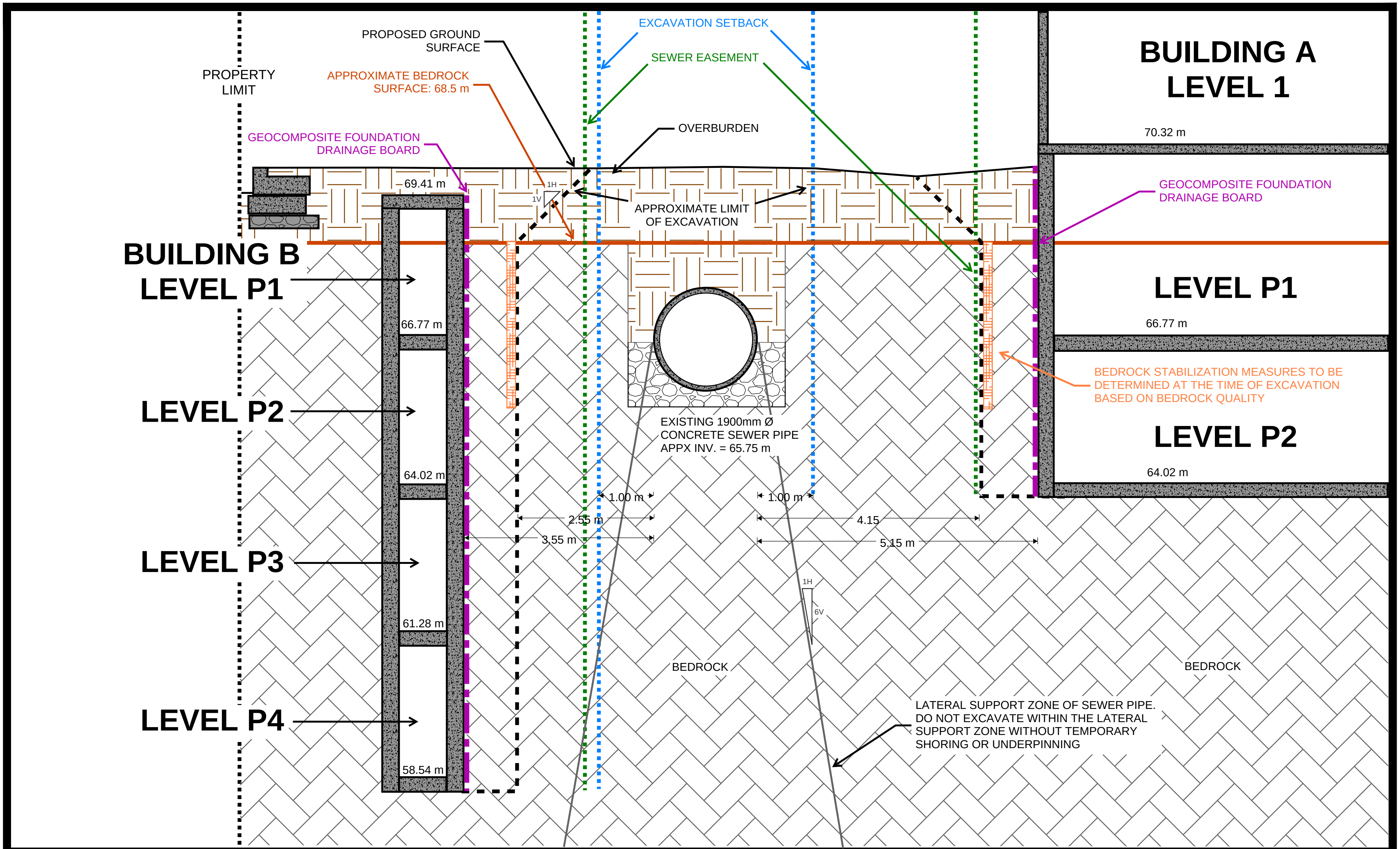


Figure 3 – Shear Wave Velocity Profile at Shot Location 24 m



PATERSON GROUP
9 AURIGA DRIVE
OTTAWA, ON
K2E 7T9
TEL: (613) 226-7381

WESTRICH PACIFIC CORPORATION
GEOTECHNICAL INVESTIGATION
PROPOSED MULTI-STOREY BUILDINGS
1125 & 1149 CYRVILLE ROAD
OTTAWA, ONTARIO

Title:

CROSS SECTION A-A

Scale: 1:100
Date: 08/2023
Drawn by: OC
Report No.: PG6072-1
Checked by: MS
Drawing No.: FIG. 4
Approved by: MS
Revision No.:

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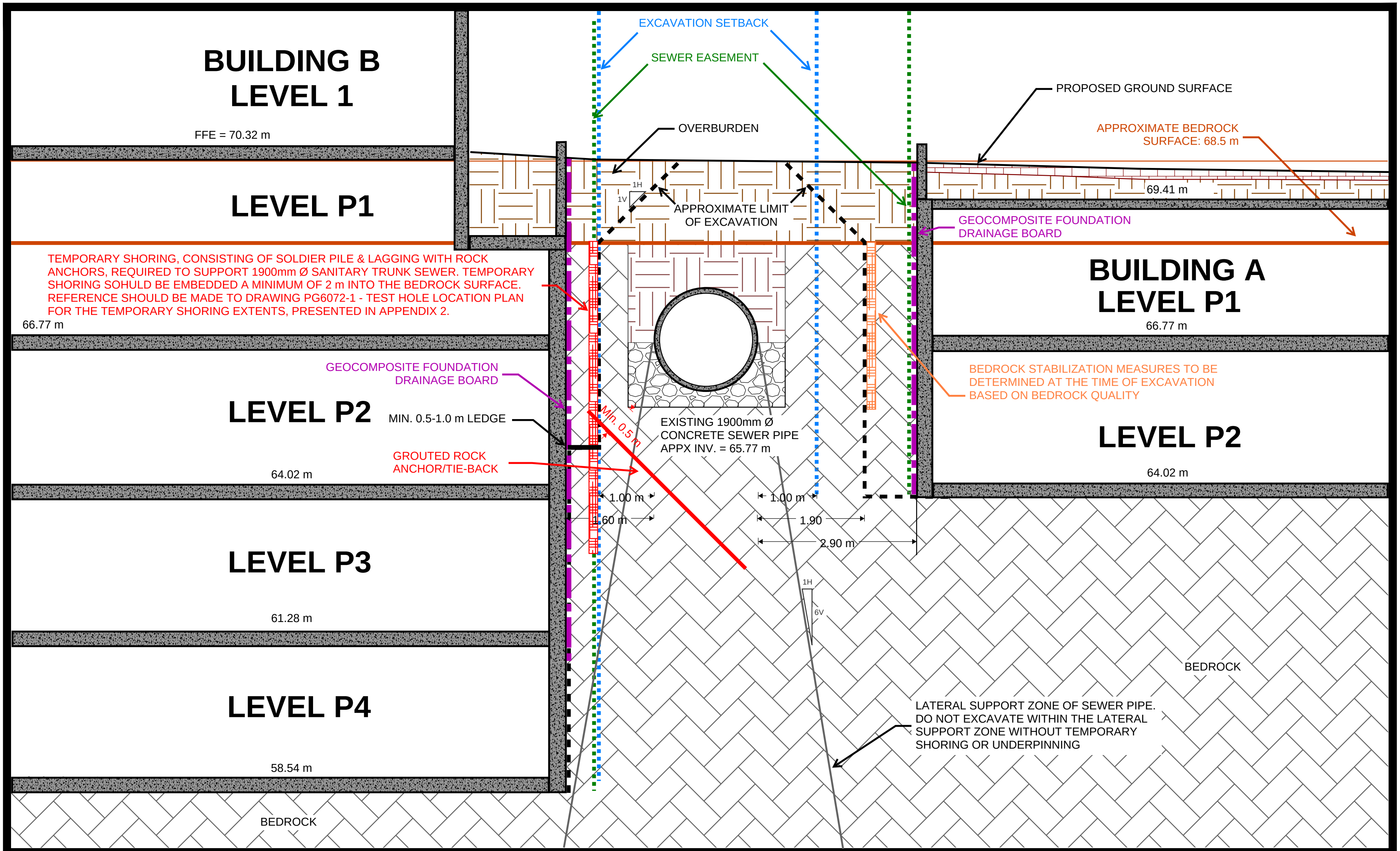

PATERSON GROUP

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WESTRICH PACIFIC CORPORATION
GEOTECHNICAL INVESTIGATION
PROPOSED MULTI-STOREY BUILDINGS
1125 & 1149 CYRVILLE ROAD

OTTAWA, ONTARIO

Title:

CROSS SECTION B-B

Scale: 1:100

Date: 08/2023

Drawn by: OC

Report No.: PG6072-1

Checked by: MS

Drawing No.: FIG. 5

Approved by: MS

Revision No.:

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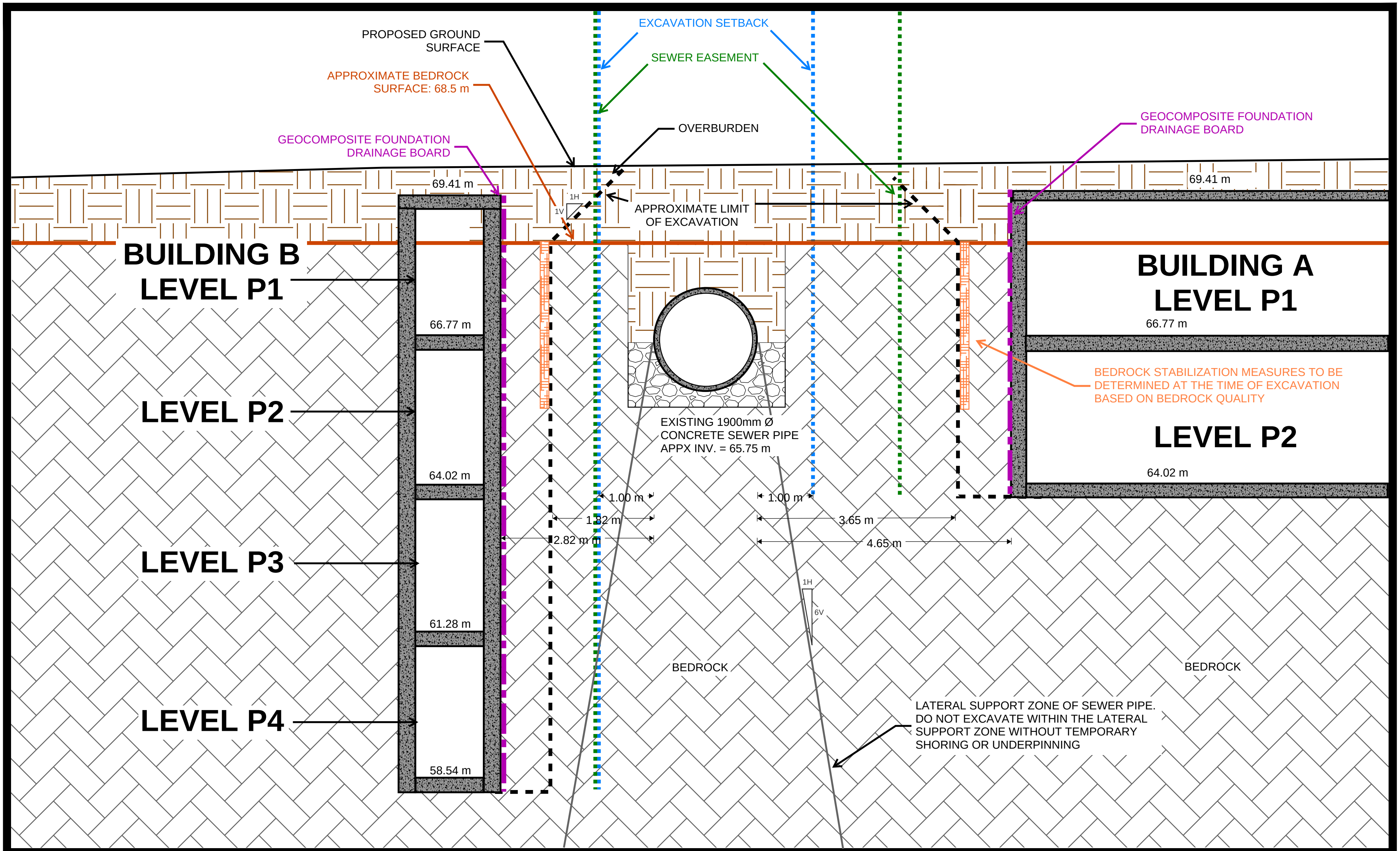

PATERSON GROUP
9 AURIGA DRIVE
OTTAWA, ON
K2E 7T9
TEL: (613) 226-7381

WESTRICH PACIFIC CORPORATION
GEOTECHNICAL INVESTIGATION
PROPOSED MULTI-STOREY BUILDINGS
1125 & 1149 CYRVILLE ROAD

OTTAWA, ONTARIO

Title:

CROSS SECTION C-C

Scale: 1:100
Date: 08/2023
Drawn by: OC
Report No.: PG6072-1
Checked by: MS
Drawing No.: FIG. 6
Approved by: MS
Revision No.:

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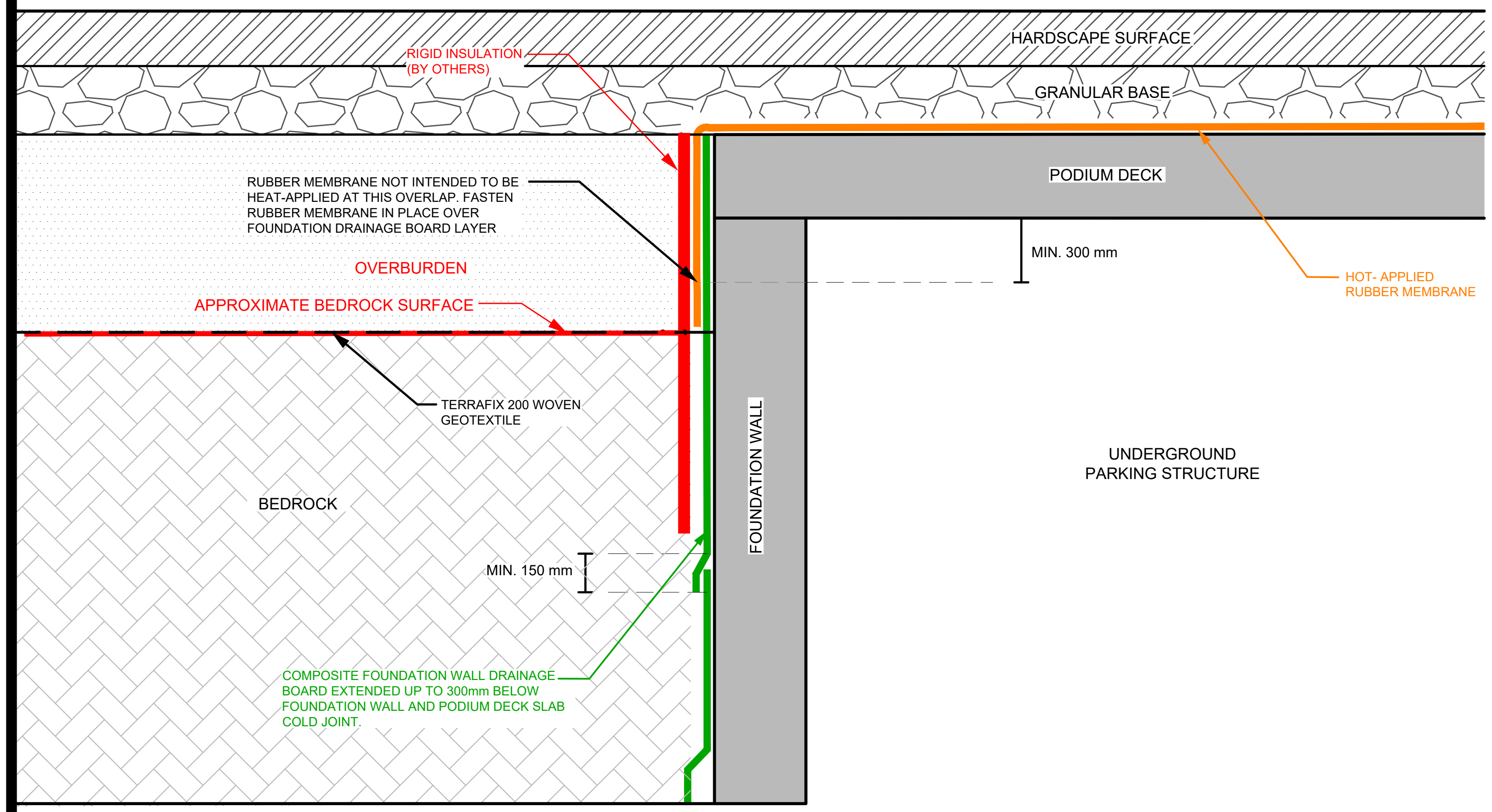
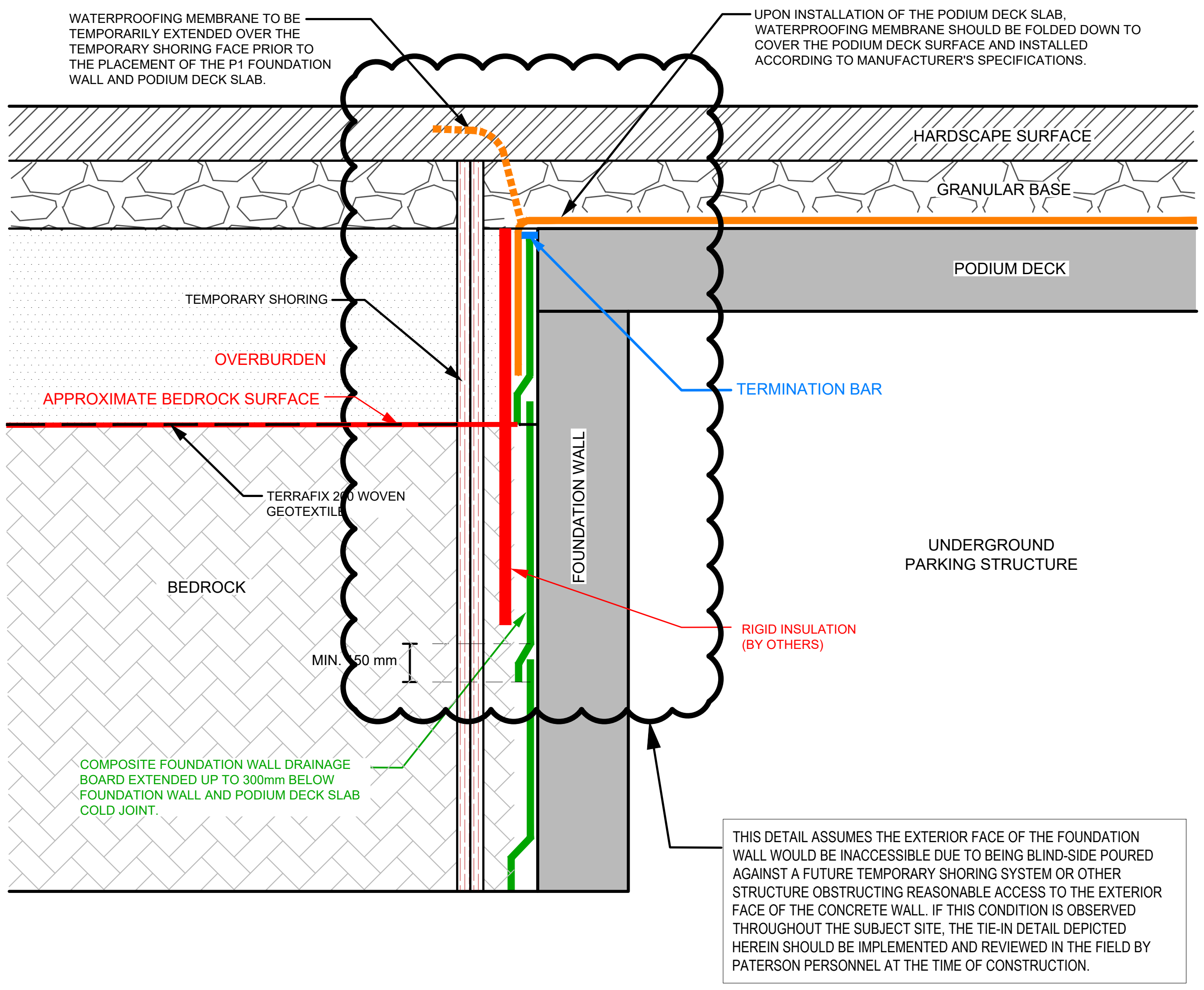
OPTION A - DOUBLE-SIDE POURED TOP OF FOUNDATION WALL



OPTION B - BLIND-SIDE POURED TOP OF FOUNDATION WALL



THIS DETAIL ASSUMES THE EXTERIOR FACE OF THE FOUNDATION WALL WOULD BE INACCESSIBLE DUE TO BEING BLIND-SIDE POURED AGAINST A FUTURE TEMPORARY SHORING SYSTEM OR OTHER STRUCTURE OBSTRUCTING REASONABLE ACCESS TO THE EXTERIOR FACE OF THE CONCRETE WALL. IF THIS CONDITION IS OBSERVED THROUGHOUT THE SUBJECT SITE, THE TIE-IN DETAIL DEPICTED HEREIN SHOULD BE IMPLEMENTED AND REVIEWED IN THE FIELD BY PATERSON PERSONNEL AT THE TIME OF CONSTRUCTION.

NOTES:

THE ABOVE DETAIL FOR HOT RUBBER AND DRAINAGE BOARD OVERLAP IS APPLICABLE TO ALL EDGE-PORTIONS OF THE PODIUM DECK AND/OR SUSPENDED GROUND FLOOR SLAB STRUCTURE.

APPLICABILITY THICKNESS AND EXTENSIONS OF RIGID INSULATION ARE SPECIFIED BY OTHERS

WHERE THE GRADING SURFACE TERMINATES AGAINST THE BUILDING FACE AND PAVEMENT STRUCTURE IS NOT LOCATED ABOVE THE EDGE OF THE FOUNDATION WALL AND PODIUM DECK SLAB AS DEPICTED HEREIN, IT IS RECOMMENDED TO PROVIDE A SUITABLE TERMINATION BAR TO SEAL THE TOP ENDLAP OF THE HOT-APPLIED RUBBER MEMBRANE LAYER TO THE VERTICAL FACE OF THE STRUCTURE. THIS WOULD BE REQUIRED TO MITIGATE THE POTENTIAL FOR THE MIGRATION OF WATER BEHIND THE RUBBER MEMBRANE.

ALL PORTIONS OF THE ABOVE-NOTED DETAIL (INSULATION OF FOUNDATION DRAINAGE BOARD, TERMINATION BAR, HOT-RUBBER MEMBRANE OVER SLAB, FOUNDATION WALL CONSTRUCTION JOINT AND OVERLAPPING/SHINGLING OF DRAINAGE BOARD) SHOULD BE REVIEWED AT THE TIME OF CONSTRUCTION BY PATERSON PERSONNEL.

PATERSON GROUP
9 AURIGA DRIVE
OTTAWA, ON
K2E 7T9
TEL: (613) 226-7381

NO.	REVISIONS	DATE	INITIAL
0			

WESTRICH PACIFIC CORP.

GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
1125-1149 CYRVILLE ROAD

OTTAWA, ONTARIO

Title:
PODIUM DECK TO FOUNDATION WALL DRAINAGE SYSTEM TIE-IN DETAIL

Scale: N.T.S
Date: 08/2023
Drawn by: RCG
Report No.: PG6072-1
Checked by: OC
Dwg. No.: FIGURE 7
Approved by: MS
Revision No.: 0

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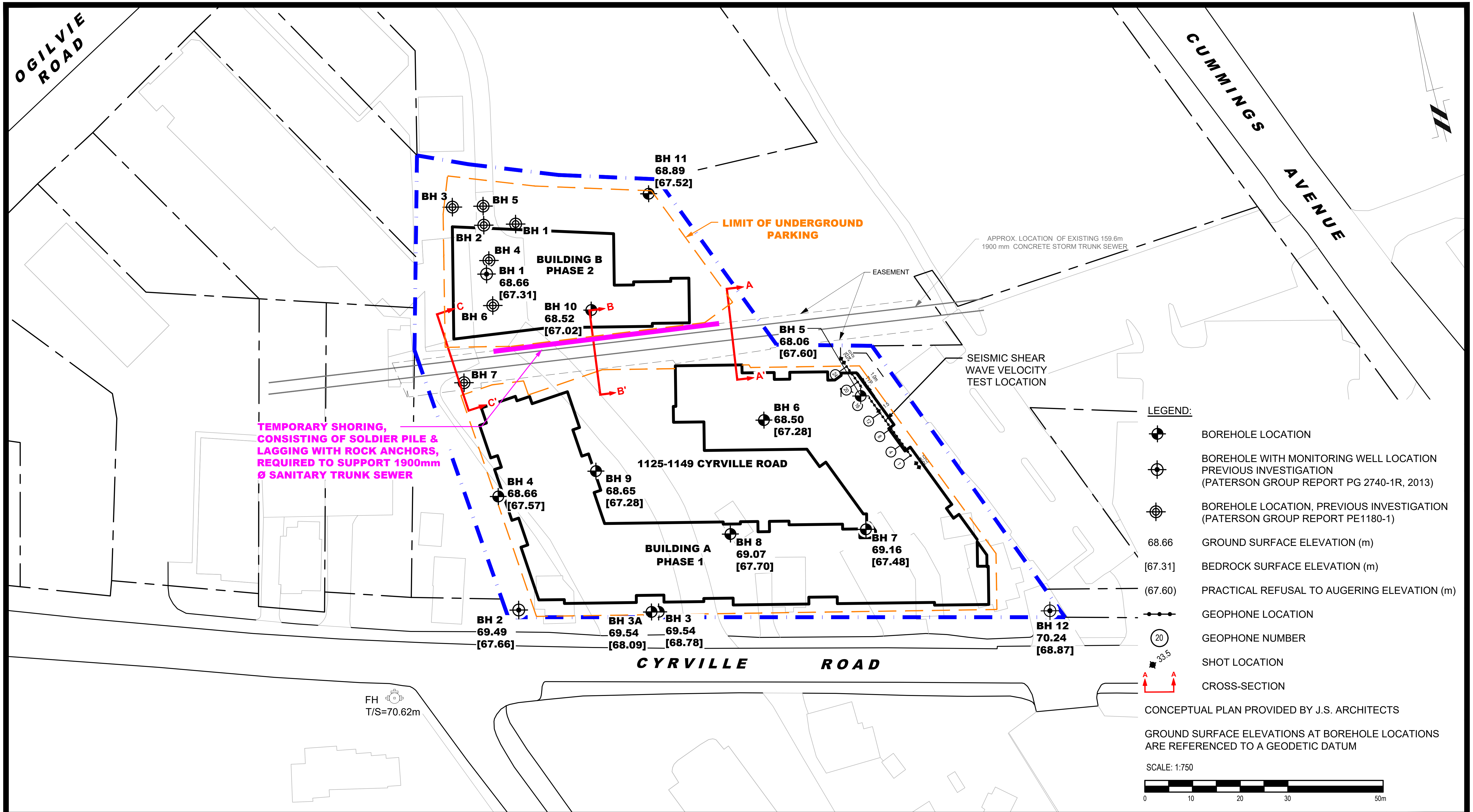

PATERSON GROUP

9 AURIGA DRIVE
OTTAWA, ON
K2E 7T9
TEL: (613) 226-7381

NO.	REVISIONS	DATE	INITIAL
1	REVISED WITH UPDATED BUILDING B AND UNDERGROUND PARKING LAYOUTS.	08/08/2023	OC

WESTRICH PACIFIC CORPORATION
GEOTECHNICAL INVESTIGATION
PROPOSED MULTI-STOREY BUILDINGS
1125-1149 CYRVILLE ROAD

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:750
Date: 11/2021
Drawn by: YA
Report No.: PG6072-1
Checked by: OC
Dwg. No.: PG6072-1
Approved by: MS
Revision No.: 1

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APPENDIX 3

RELEVANT DOCUMENTS

PATERSON GROUP

memorandum

re: **Settlement Monitoring Program – Existing Trunk Sewer**
Proposed Residential Development
1125 - 1149 Cryville Road - Ottawa
to: Westrich Pacific Corp. - **Mr. David Sanche –** dsanche@westrichpacific.com
to: HP Urban – **Mr. Peter Hume** – peter.hume@hpurban.ca
date: August 29, 2023
file: PG6072-MEMO.02

Further to your request and authorization, Paterson Group (Paterson) prepared a settlement monitoring program for the existing Trunk Sewer during construction of the proposed development at the aforementioned site. This trunk sewer bisects the central portion of this site in an approximate west-east direction.

This memo should be read in conjunction with the Geotechnical Investigation Report Paterson Group Report PG6072-1 Revision 1 dated August 29, 2023.

1.0 Background

Based on review of available design drawings, it is understood that an existing storm trunk sewer bisects the subject site between proposed Buildings A and B in an east-west direction. It is further understood that minimal room is available between the excavations for the proposed buildings and the sewer pipe. It is recommended that a settlement monitoring program is completed on the storm trunk sewer pipe throughout the duration of construction, until the excavation is backfilled.

Subsurface Profile

Based on the geotechnical investigation completed by Paterson, the subsurface profile at the subject site generally consists of pavement structure, topsoil or crushed stone fill at ground surface underlain by brown silty clay with gravel. A weathered shale bedrock was encountered below the above noted layers at all borehole locations. Given the depth of the existing trunk sewer, it is expected that the sewer is founded on the bedrock surface.

2.0 Proposed Monitoring Configuration and Locations

During the excavation for the proposed development, the existing sewer pipe will require settlement monitoring during construction of the proposed underground parking levels. Three (3) settlement monitoring points are recommended to be installed directly on top of the 1,900 mm diameter trunk sewer pipe for monitoring settlement. The approximate monitoring point locations are presented on the drawing attached to the current memorandum.

The settlement monitoring points shall consist of a length of 35 mm x 35 mm standard steel bar within a 200 mm diameter corrugated plastic sleeve. An approximately 50 mm thick concrete levelling pad shall be poured directly over the sewer pipe, followed by the placement of a 100 mm x 100 mm x 12 mm steel plate which shall be cast into the top of the concrete levelling pad. The annular spaces between the hydro-vac hole and sleeve pipe will be filled with bentonite.

The settlement monitoring points shall be used to monitor vertical displacement (settlement) only. Refer to Figure 1, attached to the current memorandum, for a detailed illustration of the monitoring point construction. It is recommended the excavation for the monitoring points to be completed using the hydro-vacuum/hydro-excavation methodology to avoid any disturbance to the existing sewer pipe and the backfill surrounding it.

The settlement monitoring points will be removed at the completion of construction. Rods, survey targets, and sleeve pipes shall be removed, and the remaining hole backfilled with bentonite pellets and sand.

The inclinometer casing will consist of 70 mm diameter, PVC or ABS resin pipe, and will be installed to the bedrock surface.

Proposed Monitoring Frequency and Methodology

Three (3) monitoring points will be installed directly on the top of the 1,900 mm storm trunk sewer pipe prior to any site works. A baseline survey will be completed daily for 3 days prior to the start of construction. The monthly monitoring results shall be provided to the city inspector/site superintendent at the time of pre-construction.

The settlement monitoring points will then be surveyed daily for the first week of excavation in the vicinity of the existing trunk sewer, and then weekly until the foundation excavations in the vicinity of the existing trunk sewer have been backfilled.

All survey measurements will be referenced to 2 benchmarks with established geodetic elevations, such as a sewer manhole cover and/or a fire hydrant arrowhead, located in the vicinity of the site but outside the zone of influence of potential settlement. The monitoring points will be surveyed using a traditional manual survey.

3.0 Settlement Criteria

Following the establishment of the baseline elevation (average of the pre-construction readings) at each settlement monitoring point, the following thresholds and exceedance protocol provided in Table 1, on the next page, are recommended during construction activities:

Table 1 - Settlement Criteria & Associated Actions

Settlement Value	**Description of Event**	**Contractor Required Action**
Up to 10 mm	*Allowable Level*	- Work may continue, no action required.
10 to 14 mm	*Review Limit*	- Immediately notify all relevant emergency contact parties within 24 hours of the survey. - Complete an additional survey of all monitoring points for confirmation of the readings. Give verbal notification of the results to the Contract Administrator within 1 hour of the additional survey and a written report within 24 hours. - Review the potential cause of the settlement and adjust the monitoring program as required. - Work may continue, however the contractor should give consideration to adjusting construction activities accordingly to minimize potential further movement.
Over 15 mm	*Alert Limit*	- Stop excavation work immediately. - Immediately notify all relevant emergency contact parties within 2 hours of the survey. - Complete an additional survey of all monitoring points for confirmation of the readings. Give verbal notification of the results to the Contract Administrator within 1 hour of the additional survey and a written report within 24 hours. - Complete a geotechnical review of the site within 12 hours to identify any obvious visual indications of ground subsidence, movement, sink holes, etc. - Complete a structural review of the affected structure(s) within 12 hours. - Notify all relevant emergency contact parties of the additional survey, geotechnical, and structural reviews within 24 hours. - Coordinate a meeting with the owner, construction manager, and all relevant emergency contact parties to discuss the results, mitigative actions, and plan for moving forward. - Construction work shall not begin until the meeting group has reached a conclusion and appropriate actions are implemented.

4.0 Monitoring Reports

Monthly settlement monitoring reports will be prepared and submitted to the construction manager/City inspector presenting the following information:

- ☐ Settlement data
- ☐ Summary of non-compliance, where applicable
- ☐ Mitigation measures, where applicable (when review and action levels are exceeded)

We trust that this information satisfies your immediate requirements.

Best Regards,

Paterson Group Inc.

Puneet Bandi, M.Eng.

LICENSED PROFESSIONAL ENGINEER
Aug.29 2023
M. SALEH
100507739
PROVINCE OF ONTARIO

Maha K. Saleh, M.A.Sc., P.Eng.

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List of Services
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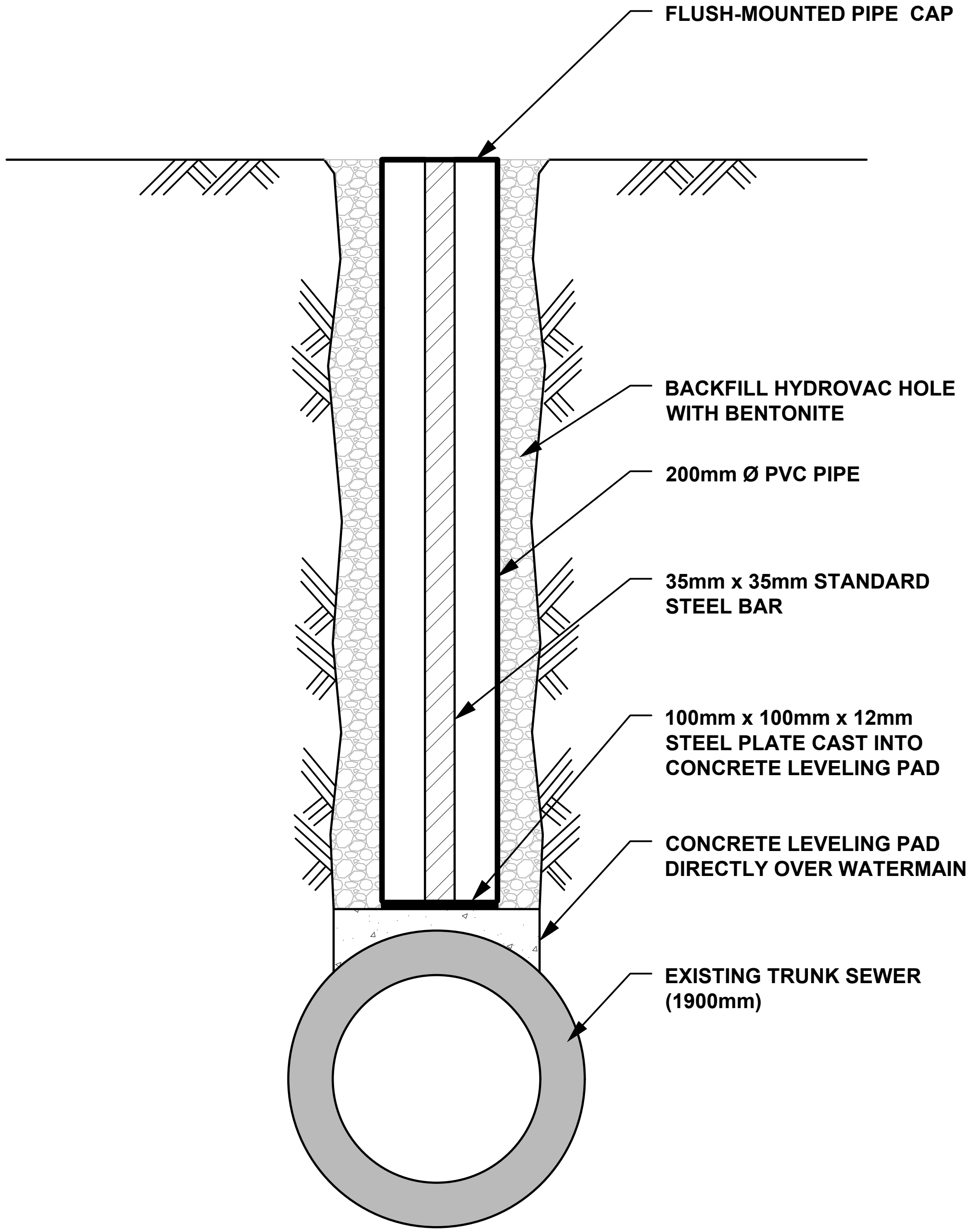

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WESTRICH PACIFIC CORP

PROPOSED MULTI-STOREY BUILDINGS
1125 - 1149 CRYVILLE ROAD
OTTAWA, ONTARIO

Title:

SEWER TRUNK SETTLEMENT MONITORING POINT INSTALLATION

Date: **09/2023**	Report No.: **PG6072-MEMO.02**
Scale: **N.T.S.**	Drawing No.: **FIGURE 1**
Drawn by: **NFRV** / Checked by: **MS**	

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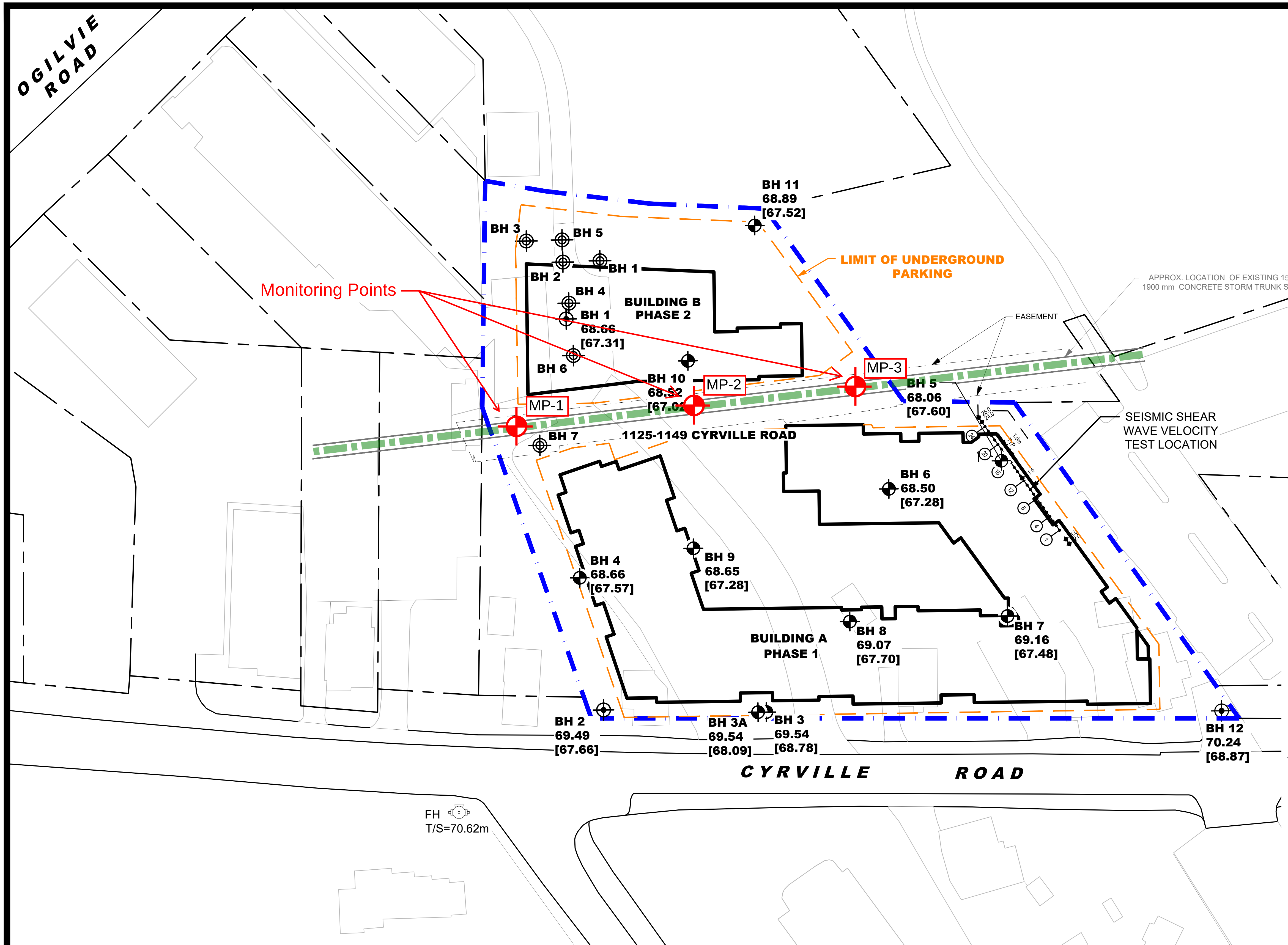

LEGEND:

⊕	BOREHOLE LOCATION
⊕	BOREHOLE WITH MONITORING WELL LOCATION PREVIOUS INVESTIGATION (PATERSON GROUP REPORT PG 2740-1R, 2013)
⊕	BOREHOLE LOCATION, PREVIOUS INVESTIGATION (PATERSON GROUP REPORT PE1180-1)
68.66	GROUND SURFACE ELEVATION (m)
[67.31]	BEDROCK SURFACE ELEVATION (m)
(67.60)	PRACTICAL REFUSAL TO AUGERING ELEVATION (m)
•–•–•	GEOPHONE LOCATION
⑳	GEOPHONE NUMBER
33.5	SHOT LOCATION

CONCEPTUAL PLAN PROVIDED BY J.S. ARCHITECTS

GROUND SURFACE ELEVATIONS AT BOREHOLE LOCATIONS ARE REFERENCED TO A GEODETIC DATUM

SCALE: 1:750

0 10 20 30 50m

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NO.	REVISIONS	DATE	INITIAL
1	REVISED WITH UPDATED BUILDING B AND UNDERGROUND PARKING LAYOUTS.	08/08/2023	OC

WESTRICH PACIFIC CORPORATION
GEOTECHNICAL INVESTIGATION
PROPOSED MULTI-STOREY BUILDINGS
1125-1149 CYRVILLE ROAD

OTTAWA, ONTARIO

Title:
MONITORING POINT LOCATION PLAN

Scale: 1:750
Date: 11/2021
Drawn by: YA
Report No.: PG6072-MEMO.02
Checked by: OC
Dwg. No.: PG6072-1
Approved by: MS
Revision No.: 1

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PATERSON GROUP

memorandum

re: **Response to Geotechnical City Comments**
Proposed Residential Development
1122-1149 Cyrville Road - Ottawa
to: Westrich Pacific Corp. - **Mr. David Sanche –** dsanche@westrichpacific.com
to: HP Urban – **Mr. Peter Hume** – peter.hume@hpurban.ca
date: August 29, 2023
file: PG6072-MEMO.03

Further to your request and authorization, Paterson Group (Paterson) prepared the following memorandum to provide geotechnical response to comments from the City of Ottawa regarding the proposed residential development to be located at the aforementioned site. This memorandum should be read in conjunction with Paterson Geotechnical Report PG6072-1 Revision 1 dated August 29, 2023 and Memorandum PG6072-MEMO.02 dated August 29, 2023.

Comment A1: *Repeat comment. Please revise and update the report as per the memo dated June 27, 2022 PG60721-MEMO.01.*

Response: Based on review of the most recent design drawings and the presence of multiple underground parking levels, the recommendations presented in the aforementioned memorandum have been superseded and are presented the above-noted revised report.

Comment A2: *Repeat comment. Please provide cross-sections for the storm sewer easement, showing the proposed sewer crossing, limits of excavation, vertical separation between the sewers and pipe cover and building foundations.*

Response: Paterson has prepared three cross-sections, representing the east and west limits of excavation as well as the worst-case excavation scenario with regard the excavation setbacks. The cross-sections, identified as Figures 4 through 6, are presented in Appendix 2 of the above-noted revised report.

Comment B18: *Repeat Comment. Provide a detail that shows what settlement monitoring point is, as per your design. A sample would be Contractor at the onset of all works is to install 3 settlement monitoring points on the City Storm pipe within the easement.* ***Example only****…..Contractor to hydro-excavate to expose the top of the concrete pipe for exact elevations. Elevations shall be tied into known benchmarks and be geodetic. A HDPE riser pipe should be placed within the vertical excavation to protect the exposed surface eleavtion for continued monitoring. etc etc....or similar. Sample Agreement Condition below.*

Proposed sewer pipe Monitoring

The Owner agrees and acknowledges that prior to the excavation for the proposed development, the existing sewer pipe will require settlement monitoring during construction. Monitoring points to be installed prior to any site works.

A) Two (3) settlement monitoring points are recommended as a minimum to be installed directly on top of the 1,900 mm diameter storm trunk sewer pipe for monitoring settlement. The settlement monitoring points elevations shall be provided to the City inspector at the pre-construction meeting. Monitoring points shall be used to monitor vertical displacement (settlement) only. Monthly monitoring of pipe elevations shall be taken and be provided to the City Inspector. The Owner's consultant on behalf of the Owner shall provide to the City a memorandum stating that there has been no settlement at the end of landscaping, surface finishes for this project.

B) The settlement monitoring points will be removed at the completion of construction. All to be at the Owners cost and to the satisfaction of the General Manager, Planning, Real Estate, and Economic Development Department.

Response: Paterson has prepared recommendations for a proposed settlement monitoring program for the existing trunk sewer pipe during construction, including a proposed monitoring point location plan. The recommendations are presented in the aforementioned MEMO.02 dated August 29, 2023.

We trust that the current submission meets your immediate requirements.

Best Regards,

Paterson Group Inc.

O.Canton

Owen R. Canton, EIT

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