Geotechnical Investigation

Proposed High-Rise Building

1657-1673 Carling Avenue & 386 Tillbury Avenue
Ottawa, Ontario

Prepared for Inside Edge Properties

Report PG6620-1 Revision 1 dated April 18, 2023

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1.0 Introduction

Paterson Group (Paterson) was commissioned by Inside Edge Properties to conduct a geotechnical investigation for the proposed high-rise building to be located at 1657-1673 Carling Avenue and 386 Tillbury Avenue, in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report for the general site location).

The objectives of the geotechnical investigation were to:

- ❑ Determine the subsoil and groundwater conditions at this site by means of existing boreholes by others.

- ❑ Provide geotechnical recommendations pertaining to the design of the proposed development including construction considerations which may affect the design.

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

2.0 Proposed Development

Based on the available conceptual drawing, the proposed development at the subject site consists of a high-rise building which will have 2 or more levels of underground parking. At finished grades, the proposed building is expected to be surrounded by landscaped margins as well as an asphalt-paved surface parking lot with an access lane from Carling Avenue.

The proposed development is expected to be municipally serviced. The existing commercial building located on-site will need to be demolished to allow for construction of the proposed development.

3.0 Available Geotechnical Information from Others

3.1 Field Investigation

A previous geotechnical investigation was carried out at the subject site by others on April 10, 2018, and consisted of a total of 3 boreholes advanced to a maximum depth of 7.6 m below the existing surface. The borehole locations are shown on Drawing PG6620-1 - Test Hole Location Plan included in Appendix 2.

Reference should be made to the Log of Borehole sheets, prepared by others, for specific details of the subsurface profiles encountered at the test hole locations, which are presented in Appendix 1.

Groundwater

Monitoring wells were installed in each borehole by others, in order to monitor the groundwater levels. The measured groundwater levels and details of the construction of the monitoring wells are presented in Appendix 1 and further discussed in Section 4.3.

3.2 Field Survey

The ground surface elevations at the borehole locations were surveyed by others using a temporary benchmark (TBM). The TBM used was the top of a catch basin located adjacent to borehole MW-2 which was assigned an arbitrary elevation of 100.00 m. The locations of the boreholes and ground surface elevation at each borehole location are presented on Drawing PG6620-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

One (1) soil sample (MW1, SS2) was submitted by others for a single-sieve grain size analysis. Sieve No.200 was used to determine the grain size distribution of the soil sample that was greater than 0.075 mm in diameter. The result of the grain size analysis is provided in Appendix 1.

3.4 Analytical Testing

Two (2) soil samples (MW1, SS2 and MW3, SS4) were submitted by others to assess the pH of the soil samples. The detailed results are provided in Appendix 1.

4.0 Observations

4.1 Surface Conditions

The subject site consists of two contiguous properties: 1657-1673 Carling Avenue and 386 Tillbury Avenue. The property at 1657-1673 Carling Avenue is currently occupied by a commercial building fronting onto Carling Avenue, and with an asphalt-paved parking lot at the rear. The property at 386 Tillbury Avenue is occupied by an existing single-family dwelling with an asphalt-paved driveway fronting onto Tillbury Avenue. A barbed-wire fence was observed along the border between the two properties.

The subject site is bordered to the north by Tillbury Avenue, to the south by Carling Avenue, to the west by existing commercial buildings, and to the east by an active construction site, 1655 Carling Avenue.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile at the subject site consists of a layer of asphaltic concrete which is underlain by fill and subsequently a silty sand deposit.

The fill extends to depths of about 0.8 m below the existing ground surface and was generally observed to consist of brown sand and gravel.

Underlying the fill, the silty sand deposit was observed to consist of brown silty sand with trace clay and gravel.

Bedrock

Practical refusal to augering was encountered on the bedrock surface at approximate depths of 1.5 m, 1.5 m and 2.7 m in boreholes MW1, MW2 and MW3, respectively.

Based on available geological mapping, the bedrock in the subject area consists of interbedded limestone and dolomite of Gull River Formation with an approximate overburden drift thickness of 2 to 3 m.

Reference should be made to the Log of Borehole sheets, prepared by others, in Appendix 1 for details of the subsurface profile encountered at each borehole location.

4.3 Groundwater

Groundwater level (GWL) measurements were recorded from the monitoring wells installed by others at the time of the previous investigation. The recorded GWLs are summarized in Table 1 below and are also presented in Appendix 1.

Table 1 - Summary of Groundwater Level Readings

Test Hole Number	Ground Surface Elevation (m)	Groundwater Level (m)	Groundwater Elevation (m)	Recording Date
MW1	100.29	2.10	98.28	Apr 17, 2018
MW2	100.13	1.95	98.14	Apr 17, 2018
MW3	100.36	2.48	97.88	Apr 17, 2018

Note: Ground surface elevations at monitoring well locations were surveyed by others using a temporary benchmark of 100 m.

It should be noted that GWLs could be influenced by surface water infiltrating the backfilled monitoring wells. The long-term GWLs can also be estimated based on the recovered soil samples' moisture levels, colour and consistency. Based on these observations, the long-term GWLs are expected be between 2 and 3 m below ground surface.

However, it should be noted that groundwater levels are subject to seasonal fluctuations, therefore, the groundwater levels could vary at the time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

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

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

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and fill, such as those containing organic or deleterious materials, should be stripped from under any buildings, paved areas, pipe bedding and other settlement sensitive structures. Due to the anticipated founding level for the proposed high-rise building, it is expected that all existing overburden material will be excavated from within the proposed building footprint.

Existing foundation walls and other demolished debris should be completely removed from the proposed building perimeter and within the lateral support zones of the foundation. Under paved area, existing construction remnants, such as foundation walls should be excavated to a minimum of 1 m below final grade.

Bedrock Removal

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

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

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

Vibration Considerations

Construction operations are also the cause of vibrations, and possibly, sources of nuisance to the community. Therefore, means to reduce the vibration levels should be incorporated in the construction operations to maintain, as much as possible, a cooperative environment with the residents.

The following construction equipment could be a source of vibrations: piling rig, hoe ram, compactor, dozer, crane, truck traffic, etc. Vibrations, whether caused by blasting operations or by construction operations, could be the cause of the source of detrimental vibrations on the nearby buildings and structures. Therefore, it is recommended that all vibrations be limited.

Two parameters are used to determine the permissible vibrations, namely, the maximum peak particle velocity and the frequency. For low frequency vibrations, the maximum allowable peak particle velocity is less than that for high frequency vibrations. As a guideline, the peak particle velocity should be less than 15 mm/s between frequencies of 4 to 12 Hz, and 50 mm/s above a frequency of 40 Hz (interpolate between 12 and 40 Hz).

It should be noted that these guidelines are for today's construction standards. Considering that these guidelines are above perceptible human level and, in some cases, could be very disturbing to some people, it is recommended that a pre-construction survey be completed to minimize the risks of claims during or following the construction of the proposed buildings.

Fill Placement

Engineered fill placed for grading beneath the proposed buildings, where required, should consist 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 spreading equipment to minimize voids. If this material is to be used to build up the subgrade level for areas to be paved, it should be compacted in thin lifts to at least 95% of the material's 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. This material should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids. If this material is to be used to build up the subgrade level for areas to be paved, it should be compacted in thin lifts to at least 95% of the material's SPMDD.

If excavated bedrock is to be used as fill, it should be suitably fragmented to produce a well-graded material with a maximum particle size of 300 mm. Where this fill material is open-graded, a woven geotextile may be required to prevent adjacent finer materials from migrating into the voids, with associated loss of ground and settlements. This can be assessed at the time of construction. Site-generated blast rock fill should be compacted using a suitably sized smooth drum vibratory roller when considered for placement.

Under winter conditions, if snow and ice is present within the blast rock fill below future basement slabs, then settlement of the fill should be expected and support of a future basement slab and/or temporary supports for slab pours will be negatively impacted and could undergo settlement during spring and summer time conditions. The geotechnical consultant should complete periodic inspections during fill placement to ensure that snow and ice quantities are minimized.

Lean Concrete Filled Trenches

Where rock overbreak occurs at the underside of footing (USF) elevation, lean concrete (minimum **17 MPa** 28-day compressive strength) can be used to reinstate the subgrade from the bedrock surface to the USF elevation. Typically, the excavation side walls will be used as the form to support the concrete. The lean concrete placement should be at least 150 mm wider than all sides of the footing (strip and pad footings) at the base of the excavation. The additional width of the concrete poured will suffice in providing a direct transfer of the footing load to the underlying bedrock.

5.3 Foundation Design

Bearing Resistance Values

A factored bearing resistance value at Ultimate Limit States (ULS) of **3,000 kPa,** incorporating a geotechnical resistance factor of 0.5, can be used for the design of conventional spread footings bearing on the clean, surface sounded bedrock.

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

Footings supported on clean, surface-sounded bedrock, and designed for the bearing resistance values provided herein, will be subjected to negligible post-construction total and differential settlements.

Lateral Support

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

5.4 Design for Earthquakes

Seismic shear wave velocity testing was completed for the subject site to accurately determine the applicable seismic site classification for the proposed building in accordance with Table 4.1.8.4.A of the Ontario Building Code 2012. The shear wave velocity testing was completed by Paterson personnel. The results of the shear wave velocity test are provided on Figures 4 and 5 in Appendix 2 of the present report.

Field Program

The seismic array testing location was placed as shown on Drawing PG6620-1 - Test Hole Location Plan, attached to the present report. Paterson field personnel placed 18 horizontal 2.4 Hz. geophones mounted to the surface by means of two 75 mm ground spikes attached to the geophone land case. The geophones were spaced at 1 m intervals and connected by a geophone spread cable to a Geode 24 Channel seismograph.

The seismograph was also connected to a computer laptop and a hammer trigger switch attached to a 12-pound dead blow hammer. The hammer trigger switch sends a start signal to the seismograph. The hammer is used to strike an I-Beam seated into the ground surface, which creates a polarized shear wave. The hammer shots are repeated between four (4) to eight (8) times at each shot location to improve signal to noise ratio. The shot locations were 1, 1.5 and 10 m away from the first and last geophones, and at the centre of the seismic array.

Data Processing and Interpretation

Interpretation for the shear wave velocity results were completed by Paterson personnel. Shear wave velocity measurement was made using reflection/refraction methods. The interpretation is performed by recovering arrival times from direct and refracted waves.

The interpretation is repeated at each shot location to provide an average shear wave velocity, V_{s30}, of the upper 30 m profile, immediately below the foundation of the building. The layer intercept times, velocities from different layers and critical distances are interpreted from the shear wave records to compute the bedrock depth at each location.

The bedrock velocity was interpreted using the main refractor wave velocity, which is considered a conservative estimate of the bedrock velocity due to the increasing quality of the bedrock with depth. It should be noted that as bedrock quality increases, the bedrock shear wave velocity also increases.

Based on our testing results, the average shear wave velocity, V_{s30} for the proposed building is **2,156 m/s** provided the footings are placed directly on bedrock. The V_{s30} was calculated using the standard equation for average shear wave velocity provided in the OBC 2012 and as presented below:

$$V_{s30} = \frac{Depth_{of\ interest}(m)}{\left(\frac{Depth_{Layer1}(m)}{V_{s_{Layer1}}(m/s)} + \frac{Depth_{Layer2}(m)}{V_{S_{Layer2}}(m/s)}\right)}$$

$$V_{s30=} \frac{30\ m}{\left(\frac{30\ m}{2{,}156\, m/s}\right)}$$

$$V_{s30=}\ 2{,}156\ m/s$$

Based on the results of the shear wave velocity testing, the average shear wave velocity V_{s30} is **2,156 m/s**. Therefore, a **Site Class A** is applicable for design of the proposed building bearing on bedrock, as per Table 4.1.8.4.A of the OBC 2012. The soil underlying the subject site is not susceptible to liquefaction.

5.5 Basement Floor Slab

For the proposed building, with the removal of all topsoil and deleterious fill from within the footprint of the proposed building, the bedrock will be considered an acceptable subgrade on which to commence backfilling for floor slab construction.

It is anticipated that the underground levels for the proposed multi-storey building will be mostly parking, and the recommended pavement structures noted in Section 5.8 will be applicable. However, if storage or other uses of the lower level will involve the construction of a concrete floor slab, the upper 300 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 should be placed in maximum 300 mm thick loose layers and compacted to at least 98% of its SPMDD.

In consideration of the anticipated groundwater conditions, an underslab drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided in the clear crushed stone under the lower basement floor of the proposed building. This is discussed further in Section 6.1.

5.6 Basement Wall

There are several combinations of backfill materials and retained soils that could be applicable for the basement walls of the proposed building. 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 drained unit weight of 20 kN/m^3.

However, the majority of the basement wall of the proposed building are to be poured against a composite drainage blanket, which will be placed against the exposed bedrock face, for which a nominal coefficient of at-rest earth pressure of 0.05 is recommended in conjunction with a bulk unit weight of 24.5 kN/m^3 (effective 15.5 kN/m^3). Further, a seismic earth pressure component will not be applicable for the foundation walls which are poured against the bedrock face. It is expected that the seismic earth pressure will be transferred to the underground floor slabs which should be designed to accommodate these pressures.

Where undrained conditions are anticipated (i.e. below groundwater level), the applicable effective (undrained) unit weight of the retained soil and bedrock can be taken as 13 kN/m^3 and 15 kN/m^3, where applicable. Then, a hydrostatic pressure should be added to the total static earth pressure when using the effective unit weight.

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 material
γ = unit weight of fill of the applicable retained material (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 $0.375 \cdot a \cdot H^2/g$ 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 $P_o = 0.5\ K_o \cdot \gamma \cdot H^2$, where K = 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

Overview of Anchor Features

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

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

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 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, 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, then therefore, less geotechnical resistance, than one where the bonded length is limited to the bottom part of the overall anchor.

Permanent anchors should be provided with corrosion protection. As a minimum, the entire drill hole should be filled with cementitious grout. The free anchor length is provided by installing a plastic sleeve to act as a bond break, with the sleeve filled with grout or a corrosion inhibiting mastic.

Double corrosion protection can be provided with factory assembled systems, such as those available from Dywidag Systems or Williams Form Engineering Corp. Recognizing the importance of the anchors for the long-term performance of the foundation of the proposed building, if required, any rock anchors for this project are recommended to be provided with double corrosion protection.

Grout to Rock Bond

The Canadian Foundation Engineering Manual recommends a maximum allowable grout to rock bond stress (for sound rock) of 1/30 of the unconfined compressive strength (UCS) of either the grout or rock (but less than 1.3 MPa) for an anchor of minimum length (depth) of 3 m. Generally, the UCS of limestone ranges between about 50 and 100 MPa, which is stronger than most routine grouts. A factored tensile grout to rock bond resistance value at ULS of 1.0 MPa, incorporating a resistance factor of 0.4, can be calculated. A minimum grout strength of 40 MPa is recommended.

Rock Cone Uplift

As discussed previously, the geotechnical capacity of the rock anchors depends on the dimensions of the rock anchors and the configuration of the anchorage system. Hoek and Brown parameters (m and s) for the bedrock were taken as 0.575 and 0.00293, respectively.

Recommended Rock Anchor Lengths

Parameters used to calculate rock anchor lengths are provided in Table 2 below:

Table 2 - Parameters used in Rock Anchor Review	
Grout to Rock Bond Strength - Factored at ULS	1.0 MPa
Compressive Strength - Grout	40 MPa
Rock Mass Rating (RMR)-Good quality Limestone Hoek and Brown parameters	69 m=0.575 and s=0.00293
Unconfined compressive strength - Limestone bedrock	75 MPa
Unit weight - Submerged Bedrock	15 kN/m^3
Apex angle of failure cone	60^o
Apex of failure cone	mid-point of fixed anchor length

The fixed anchor length will depend on the diameter of the drill holes. Recommended anchor lengths for a 75 mm and 125 mm diameter hole are provided in the following Table 3.

The factored tensile resistance values given in Table 3 are based on a single anchor with no group influence effects. A detailed analysis of the anchorage system, including potential group influence effects, could be provided once the details of the loading for the proposed building are determined.

Table 3 - Recommended Rock Anchor Lengths - Grouted Rock Anchor

Diameter of Drill Hole (mm)	Anchor Lengths (m)			Factored Tensile Resistance (kN)
	Bonded Length	Unbonded Length	Total Length	
75	1.5	1.0	2.5	350
	1.8	1.2	3.0	425
	2.2	1.4	3.6	500
	2.9	1.6	4.5	680
125	1.2	0.8	2.0	450
	1.5	1.0	2.5	580
	1.8	1.2	3.0	700
	2.2	1.0	3.2	850

Other considerations

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

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 Design

Lowest Underground Parking Level

For design purposes, it is recommended that the rigid pavement structure for the lowest underground parking level consist of Category C2, 32 MPa concrete at

28 days with air entrainment of 5 to 8%. The recommended rigid pavement structure is further presented in Table 4 below.

Table 4 - Recommended Rigid Pavement Structure – Underground Parking Level	
Thickness (mm)	**Material Description**
150	**Exposure Class C2 – 32 MPa Concrete** (5 to 8% Air Entrainment)
300	**BASE** - OPSS Granular A Crushed Stone
SUBGRADE – Existing imported fill, or OPSS Granular B Type I or II material placed over bedrock.	

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

Pavement Structure Over Podium Deck

The pavement structures presented in Tables 5 and 6 should be used for car only parking areas, at grade access lanes and heavy loading parking areas over the top of the podium structure, should they be required.

Table 5 - Recommended Pavement Structure - Car Only Parking Areas Over Podium Deck	
Thickness (mm)	**Material Description**
50	**Wear Course** – HL-3 or Superpave 12.5 Asphaltic Concrete
200*	**BASE** - OPSS Granular A Crushed Stone
See below**	**Thermal Break** -** Rigid Insulation (See Following Paragraph)
n/a	**Waterproofing Membrane and IKO Protection Board**
SUBGRADE – Reinforced concrete podium deck * Thickness of base course is dependent on grade of insulation as noted in proceeding paragraph ** If specified by others, not required from a geotechnical perspective	

Table 6 - Recommended Pavement Structure – Access Lanes, Fire Truck Lane, Ramp, and Heavy Loading Areas Over Podium Deck	
Thickness (mm)	**Material Description**
40	**Wear Course** – HL-3 or Superpave 12.5 Asphaltic Concrete
50	**Binder Course** – HL-8 or Superpave 19.0 Asphaltic Concrete
300*	**BASE** - OPSS Granular A Crushed Stone
See below**	**Thermal Break** - Rigid Insulation (See Following Paragraph)
n/a	**Waterproofing Membrane and IKO Protection Board**
SUBGRADE – Reinforced concrete podium deck * Thickness of base course is dependent on grade of insulation as noted in proceeding paragraph ** If specified by others, not required from a geotechnical perspective	

The transition between the pavement structure over the podium deck subgrade and soil subgrade beyond the footprint of the podium deck is recommended to be transitioned to match the pavement structures provided in the following section.

For this transition, a 5H:1V is recommended between the two subgrade surfaces. Further, the base layer thickness should be increased to a minimum thickness of 500 mm below the top of the podium slab a minimum of 1.5 m from the face of the foundation wall prior to providing the recommended taper.

Should the proposed podium deck be specified to be provided a thermal break by the use of a layer of rigid insulation below the pavement structure, its placement within the pavement structure is recommended to be as per the above-noted tables. The layer of rigid insulation is recommended to consist of a DOW Chemical High-Load 100 (HI-100), High-Load 60 (HI-60), or High-Load 40 (HI-40). The base layer thickness will be dependent on the grade of insulation considered for this project and should be reassessed by the geotechnical consultant once pertinent design details have been prepared.

The higher grades of insulation have more resistance to deformation under wheel-loading and require less granular cover to avoid being crushing by vehicular loading. It should be noted that SM (Styrofoam) rigid insulation is **not** considered suitable for this application.

Pavement Structure over Overburden

Beyond the podium deck, the following pavement structures in Tables 7 and 8 may be used for car only parking and heavy traffic areas on overburden.

Table 7 - Recommended Pavement Structure – Car Only Parking Areas	
Thickness (mm)	**Material Description**
50	**Wear Course** – HL-3 or Superpave 12.5 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
300	**SUBBASE –** OPSS Granular B Type II
SUBGRADE – Either in-situ soils, existing imported fill or OPSS Granular B Type I or II material placed over in-situ soil or bedrock.	

Table 8 - Recommended Pavement Structure – Access Lanes, Heavy Traffic and Loading Areas	
Thickness (mm)	**Material Description**
40	**Wear Course** – HL-3 or Superpave 12.5 Asphaltic Concrete
50	**Binder Course** – HL-8 or Superpave 19.0 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
450	**SUBBASE –** OPSS Granular B Type II
SUBGRADE – Either in-situ soils, existing imported fill or OPSS Granular B Type I or II material placed over in-situ soil or bedrock.	

Other Considerations

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for this project. If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type II material.

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

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

It is recommended that the portion of the proposed building foundation walls located 4 m below finished grades be blind-poured and placed against a groundwater infiltration control system which is fastened to the temporary shoring system or vertical bedrock face. Also, a perimeter foundation drainage system will be required as a secondary system to account for any groundwater which comes in contact with the proposed building's foundation walls.

For the portion of the groundwater infiltration control system installed against vertical bedrock face, the following is recommended:

- ❑ Line drill the excavation perimeter (usually at 150 to 200 mm spacing).
- ❑ Mechanically remove bedrock along the foundation walls, up to approximately 150 mm from the finished vertical excavation face.
- ❑ Grind the bedrock surface up to the outer face of the line drilled holes to create a satisfactory surface for the waterproofing membrane and/or composite drainage board.
- ❑ If bedrock overbreaks occur, shotcrete these areas to fill in cavities and to smooth out angular features of the bedrock surface, as required based on site inspection by Paterson.
- ❑ Place a suitable waterproofing membrane (such as Tremco Paraseal or approved equivalent) against the prepared vertical bedrock surface. The membrane liner should extend from 4 m below finished grade down to footing level. The membrane liner should also extend horizontally a minimum 600 mm below the footing at the underside of footing level.
- ❑ Place a composite drainage board, such as Delta Drain 6000 or equivalent, over the membrane, as a secondary system. The composite drainage layer should extend from finished grade to underside of footing level.
- ❑ Pour foundation wall against the composite drainage board.

It is recommended that 150 mm diameter sleeves at 3 m centres be cast at the foundation wall/footing interface to allow for the infiltration of water that breaches the waterproofing system to flow to an interior perimeter drainage pipe. The perimeter drainage pipe should direct water to sump pit(s) within the lower basement area.

Elevators and any other pits located below the underslab drainage system should be waterproofed. A full waterproofing detail for the foundation walls and the mechanical pits can be provided by Paterson, if required.

Perimeter and Underslab Drainage System

The perimeter and underslab drainage system is recommended to control water infiltration below the underground parking level slab and to redirect water from the buildings foundation drainage system to the building's sump pit(s). For preliminary design purposes, it is recommended that 150 mm perforated pipes provided with a geosock and surrounded on all sides by a minimum 150 mm thick layer of 19 mm clear crushed stone and be placed at approximate 6 m centres underlying the underground parking level slab.

The perimeter drainage system should be mechanically connected to the 150 mm drainage sleeves and gravity connected to the underslab drainage system, which in turn is connecte to the building's sump pit(s).

The spacing of the underslab drainage system should be confirmed by the geotechnical consultant at the time of completing the excavation when water infiltration can be better assessed.

Transition from Foundation Wall to Podium Deck

It is anticipated that a 2-ply modified bitumen membrane or similar hot-applied waterproofing membrane product will be placed across the exterior surface of the concrete deck. The concrete deck should be cleaned of all dust, dirt, or debris prior to the application of the hot rubber. The following details and recommendations should be considered for the transition in the drainage materials between the foundation wall to podium deck structure.

Option A – Double-Side Poured Top Segment of Foundation Wall

Where a double-sided pour is considered for the top segment of the foundation wall, it is recommended to extend the podium deck waterproofing membrane vertically down the foundation wall and a minimum of 300 mm below the construction joint between the foundation wall and podium deck slab. Further, the bottom-most end-lap of the waterproofing membrane extending over the drainage board should be installed loosely against the drainage board layer to mitigate heat associated with welding the rubber membrane from damaging the drainage layer. The loosely installed layer of membrane should overlap the top of the drainage board layer by a minimum of 300 mm.

Option B – Blind-Side Poured Top Segment of Foundation Wall

Should the top segment of the foundation wall be blind-cast against a shoring system or bedrock, the waterproofing membrane should be vertically installed and extended over the temporary shoring face or bedrock prior to the placement of the P1 foundation wall and podium deck slab. Following installation of the podium deck slab, the waterproofing membrane can be overlapped onto the podium deck surface and installed accordingly to manufacturer's specifications.

Where a podium deck will not be provided with a horizontal application as described above, the top edge of the drainage board should be sealed by a liquid membrane to mitigate the migration of water between the foundation wall and drainage board layer.

Reference should be made to Figure 3 – Podium Deck to Foundation Wall Drainage System Tie-In Detail in Appendix 2.

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, such as clean sand or OPSS Granular B Type I granular material.

6.2 Protection of Footings Against Frost Action

Perimeter footings of heated structures are recommended to be insulated against the deleterious effects of frost action. A minimum 1.5 m thick soil cover, or an equivalent combination of soil cover and foundation insulation, should be provided in this regard.

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

However, the footings are generally not expected to require protection against frost action due to the founding depth. Unheated structures such as the access ramp may require insulation for protection against the deleterious effects of frost action.

6.3 Excavation Side Slopes

The side slopes of excavations in the 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.

Excavation Side Slopes

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

Due to the anticipated depth of excavation of the building and the proximity of the proposed building to abutting property boundaries, temporary shoring may be required for the overburden soils to complete the required excavations where insufficient room is available for open cut methods. The shoring requirements will depend on the depth of the excavation, the proximity of the adjacent buildings and underground structures and the elevation of the adjacent building foundations and underground services.

The design and approval of the shoring system will be the responsibility of the shoring contractor and the shoring designer who is a licensed professional engineer and is hired by the shoring contractor. It is the responsibility of the shoring contractor to ensure that the temporary shoring is in compliance with safety requirements, designed to avoid any damage to adjacent structures and include dewatering control measures.

In the event that subsurface conditions differ from the approved design during the actual installation, it is the responsibility of the shoring contractor to commission the required experts to re-assess the design and implement the required changes.

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

The temporary shoring system may consist of a soldier pipe and lagging system which could be cantilevered, anchored or braced. Generally, it is expected that the shoring systems will be provided with tie-back rock anchors to ensure their stability. It is further recommended that the toe of the shoring system be adequately supported to resist toe failure by means of rock bolts or extending into the bedrock through pre-augered holes if a shoulder pile and lagging system is used.

Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be added to the earth pressures described below. The earth pressure acting on the shoring system may be calculated using the following parameters.

Table 9 - Soil Parameters

Parameters	Values
Active Earth Pressure Coefficient (K_a)	0.33
Passive Earth Pressure Coefficient (K_p)	3
At-Rest Earth Pressure Coefficient (K_o)	0.5
Unit Weight , kN/m_3	21
Submerged 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 no movement is permissible.

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

The 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 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.

Bedrock Stabilization

Excavation side slopes in sound bedrock can be carried out using almost vertical side walls. A minimum 1 m horizontal ledge should be left between the bottom of the overburden excavation and the top of the bedrock surface to provide an area to allow for potential sloughing or to provide a stable base for the overburden shoring system.

Horizontal rock anchors 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 temporary chainlink fencing, shotcrete, and/or rock bolts should be evaluated during the excavation operations and should be discussed with the structural engineer during the design stage of the project.

Underpinning of Adjacent Structures

Based on the relatively shallow depth of the bedrock at the subject site, it is expected that the neighbouring buildings located near the southwest and northeast boundaries of the site are most likely founded on the bedrock surface. Therefore, underpinning is not anticipated to be required. However, this should be confirmed at the start of construction. If these structures are not founded on bedrock, then underpinning would be required.

6.4 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.

A minimum of 150 mm of OPSS Granular A should be placed for bedding for sewer or water pipes when placed on a soil subgrade. The bedding should extend to the spring line of the pipe. Cover material, from the spring line to a minimum of 300 mm above the obvert of the pipe, should consist of OPSS Granular A (concrete or PSM PVC pipes) or sand (concrete pipe). The bedding and cover materials should be placed in maximum 225 mm thick lifts and compacted to 98% of the SPMDD.

Where hard surface areas are considered above the trench backfill, the trench backfill material within the frost zone (about 1.8 m below finished grade) and above the cover material should match the soils exposed at the trench walls to minimize differential frost heaving. The trench backfill should be placed in maximum 225 mm thick loose lifts and compacted to a minimum of 98% of the material's SPMDD. All cobbles larger than 200 mm in their longest direction should be segregated from re-use as trench backfill.

6.5 Groundwater Control

It is anticipated that groundwater infiltration into the excavations should be low to moderate and controllable using open sumps. The contractor should be prepared to direct water away from all subgrades, regardless of the source, to prevent disturbance to the founding medium.

Groundwater Control for Building Construction

A temporary Ministry of Environment, Conservation and Parks (MECP) permit to take water (PTTW) may be required if more than 400,000 L/day of ground and/or surface water are to be pumped during the construction phase. At least 4 to 5 months should be allowed for completion of the application and issuance of the permit by the 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 Persons as stipulated under O.Reg. 63/16.

If a project qualifies for a PTTW based upon anticipated conditions, an EASR will not be allowed as a temporary dewatering measure while awaiting the MECP review of the PTTW application.

Impacts on Neighbouring Properties

It is understood that 2 or more levels of underground parking are planned for the proposed building with the lower portion of the foundation walls having a groundwater infiltration control system in place. In addition, given the shallow bedrock present at, and in the vicinity of, the subject site, the neighbouring structures are expected to be founded on the bedrock surface. Therefore, no issues are expected with respect to groundwater lowering that would cause damage to adjacent structures surrounding the proposed development.

6.6 Winter Construction

Precautions must be taken if winter construction is considered for this project. The subsoil conditions at this site 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 using 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.

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

6.7 Corrosion Potential and Sulphate

The pH of the sample indicates that the soil is not a significant factor in creating a corrosive environment for exposed ferrous metals at this site.

7.0 Recommendations

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

- ❑ Review of the geotechnical aspects of the excavation contractor's temporary shoring system design, if required, prior to construction.
- ❑ Review of the proposed groundwater infiltration control system and foundation drainage and requirements.
- ❑ Review of the bedrock stabilization and excavation requirements.
- ❑ Observation of all bearing surfaces prior to the placement of concrete.
- ❑ Sampling and testing of the concrete and fill materials.
- ❑ Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- ❑ Observation of all subgrades prior to backfilling.
- ❑ Field density tests to determine the level of compaction achieved.
- ❑ Sampling and testing of the bituminous concrete including mix design reviews.

A report confirming that these works have been conducted in general accordance with our recommendations could be issued upon the completion of a satisfactory inspection program by the geotechnical consultant.

All excess soils, with the exception of engineered crushed stone fill, generated by construction activities that will be transported on-site or off-site should be handled as per *Ontario Regulation 406/19: On-Site and Excess Soil Management.*

8.0 Statement of Limitations

The recommendations provided are in accordance with the present understanding of the project. Paterson requests permission to review the recommendations when the drawings and specifications are completed.

A soils investigation is a limited sampling of a site. Should any conditions at the site be encountered which differ from those at the test locations, Paterson requests immediate notification to permit reassessment of our recommendations.

The recommendations provided herein should only be used by the design professionals associated with this project. They are not intended for contractors bidding on or undertaking the work. The latter should evaluate the factual information provided in this report and determine the suitability and completeness for their intended construction schedule and methods. Additional testing may be required for their purposes.

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 Inside Edge Properties, or their agents, is not authorized without review by Paterson for the applicability of our recommendations to the alternative use of the report.

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
April 18, 2023
S. S. DENNIS
100519516
PROVINCE OF ONTARIO

Fernanda Carozzi, PhD. Geoph.

Scott S. Dennis, P.Eng.

Report Distribution:

- ❏ Inside Edge Properties (email copy)
- ❏ Paterson Group (1 copy)

APPENDIX 1

LOG OF BOREHOLE SHEETS BY OTHERS

pH AND GRAIN SIZE ANALYSIS RESULTS BY OTHERS

MONITORING WELL CONSTRUCTION DETAILS BY OTHERS

GROUNDWATER ELEVATION DATA BY OTHERS

Log of Borehole: MW1

Project #: 220409.002 **Logged By:** RL

Project: Phase II Environmental Site Assessment

Client: 1916621 Ontario Ltd.

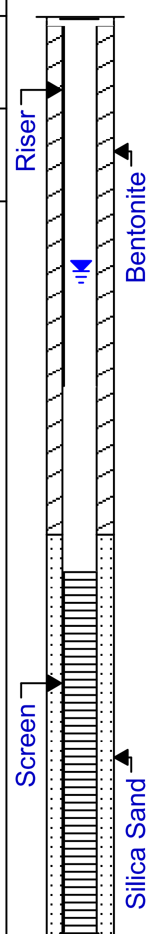
Location: 1657-1673 Carling Avenue, Ottawa, Ontario

Drill Date: April 10, 2018 **Project Manager:** RL

SUBSURFACE PROFILE

Depth (ft)	Depth (m)	Description	Measured Depth (m)
0	0	Ground Surface	0.00
0–2.5	0–0.76	***Asphalt*** ***Sand and Gravel - Fill*** Brown, moist, no odour.	0.76
2.5–5	0.76–1.52	***Silty Sand*** Brown, with trace clay and gravel throughout, moist, no odour.	1.52
5–25	1.52–7.62	***Bedrock*** Refusal on Bedrock. Advanced with Air Rotary.	7.62
25	7.62	End of Borehole	

Monitoring Well Details



Water level measured at 2.10mbgs

SAMPLE

Sampler #	Recovery (%)	Sample ID	Soil Vapour Concentration RKI/PID	Laboratory Analysis
1	40	SS1	0ppm	GRAIN SIZE
2	40	SS2	0ppm	GRAIN SIZE, pH, PHCs, VOCs,

Contractor: Strata Drilling Group

Drilling Method: Geo Machine

Well Casing Size: 3.8cm

Pinchin Ltd.

1 Hines Road, Suite 200

Kanata, ON K2K 3C7

Grade Elevation: 100.288

Top of Casing Elevation: 100.180

Sheet: 1 of 1

Log of Borehole: MW2

Project #: 220409.002 **Logged By:** RL

Project: Phase II Environmental Site Assessment

Client: 1916621 Ontario Ltd.

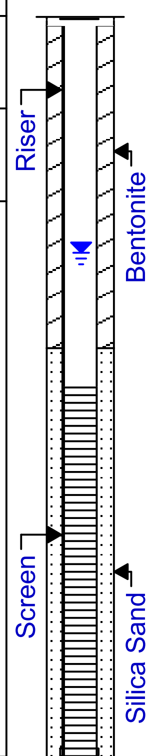
Location: 1657-1673 Carling Avenue, Ottawa, Ontario

Drill Date: April 10, 2018 **Project Manager:** RL

SUBSURFACE PROFILE / SAMPLE

Description	Measured Depth (m)	Sampler #	Recovery (%)	Sample ID	Soil Vapour Concentration RKI/PID	Laboratory Analysis
Ground Surface	0.00					
Asphalt						
Sand and Gravel - Fill Brown, moist, no odour.	0.76	1	50	SS1	0ppm	
Silty Sand Brown, with trace clay and gravel throughout, moist, no odour.	1.52	2	50	SS2	0ppm	PHCs, VOCs
Bedrock Refusal on Bedrock. Advanced with Air Rotary.	6.10					
End of Borehole						

Depth scale: 0–30 ft; 0–9 m



Water level measured at 1.95mbgs on April 10, 2018

Contractor: Strata Drilling Group

Drilling Method: Geo Machine

Well Casing Size: 3.8cm

Pinchin Ltd.

1 Hines Road, Suite 200

Kanata, ON K2K 3C7

Grade Elevation: 100.126

Top of Casing Elevation: 100.011

Sheet: 1 of 1

Log of Borehole: MW3

Project #: 220409.002 **Logged By:** RL

Project: Phase II Environmental Site Assessment

Client: 1916621 Ontario Ltd.

Location: 1657-1673 Carling Avenue, Ottawa, Ontario

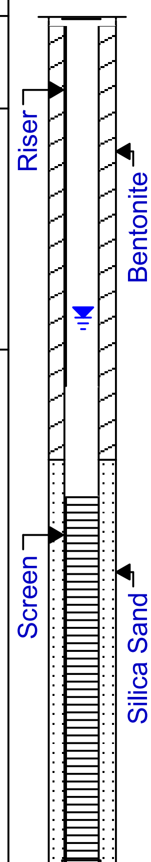
Drill Date: April 10, 2018 **Project Manager:** RL

SUBSURFACE PROFILE

Depth	Description	Measured Depth (m)
0 ft / 0 m	Ground Surface	0.00
	Asphalt	
	Sand and Gravel - Fill Brown, moist, no odour.	0.76
	Silty Sand Brown, with trace clay and gravel throughout, moist, no odour.	2.74
	Bedrock Refusal on Bedrock. Advanced with Air Rotary.	7.01
	End of Borehole	

Depth scale: ft 0–30; m 0–9

Monitoring Well Details



Water level measured at 2.48mbgs on April

SAMPLE

Sampler #	Recovery (%)	Sample ID	Soil Vapour Concentration RKI/PID	Laboratory Analysis
1	40	SS1	0ppm	
2	40	SS2	0ppm	
3	60	SS3	0ppm	
4	60	SS4	0ppm	PHCs, pH, VOCs

Contractor: Strata Drilling Group

Drilling Method: Geo Machine

Well Casing Size: 3.8cm

Pinchin Ltd.

1 Hines Road, Suite 200

Kanata, ON K2K 3C7

Grade Elevation: 100.363

Top of Casing Elevation: 100.293

Sheet: 1 of 1

TABLE 2
pH AND GRAIN SIZE ANALYSIS FOR SOIL
1716621 Ontario Limited
1657-1672 Carling Avenue, Ottawa, Ontario

Parameter	Units	MOECC Site Condition Standard Selection Criteria	Sample Designation / Sample Collection Date (dd/mm/yyyy) / Sample Depth (mbgs)		
			MW-1, SS-1	MW-1, SS-2	MW-3, SS-4
			25/10/2017	25/10/2017	25/10/2017
			0.2 - 1.2	2.8 - 3.2	0.2 - 1.2
			Surface	Surface	Sub-Surface
pH		Surface: 5 < pH < 9 Subsurface: 5 < pH < 11		7.7	7.9
Sieve #200 <0.075 mm	%	50%	36	NA	NA
Sieve #200 >0.075 mm	%	50%	64	NA	NA
		Grain Size Classification	COARSE	MEDIUM/FINE	NA

Notes:

BOLD	Environmentally Sensitive Area (Based Upon pH of Surface Soil)
BOLD	Environmentally Sensitive Area (Based Upon pH of Sub-Surface Soil)
NA	Not Analysed
mbgs	Metres Below Ground Surface

TABLE 3
MONITORING WELL CONSTRUCTION DETAILS
1716621 Ontario Limited
1657-1672 Carling Avenue, Ottawa, Ontario

Well Number	*Surveyed TOC Elevation (mREL)*	*Surveyed Ground Elevation (mREL)*	*Calculated Difference Between Ground and TOC (m)*	*Length of Screen (m)*
MW-1	100.18	100.29	0.11	3.05
MW-2	100.01	100.13	0.12	3.05
MW-3	100.29	100.36	0.07	3.05

Notes:

mREL	Indicates Groundwater Elevation (metres) Relative to Site Benchmark with Assumed Elevation of 100.00 Metres
TOC	Indicates Top of Casing
NM	Not Measured
m	Metres

TABLE 4
GROUNDWATER ELEVATION DATA
1716621 Ontario Limited
1657-1672 Carling Avenue, Ottawa, Ontario

Well Number	*Date (dd/mm/yyyy)*	*NAPL Level Measurement from TOC (m)*	*Water Level Measurement from TOC (m)*	*Water Level Measurement from Ground (mbgs)*	*Product Thickness (m)*	*Calculated Water Level Elevation (mREL)*
MW-1	17/04/2018	ND	1.90	2.10	ND	98.28
MW-2	17/04/2018	ND	1.87	1.95	ND	98.14
MW-3	17/04/2018	ND	2.41	2.48	ND	97.88

Notes:

mREL	Indicates Groundwater Elevation (metres) Relative To Site Benchmark with Assumed Elevation of 100.00 Metres
NAPL	Non-Aqueous Phase Liquid
ND	Not Detected
TOC	Indicates Top of Casing
m	Metres
mbgs	Metres Below Ground Surface

APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURE 2 – GROUNDWATER INFILTRATION CONTROL SYSTEM

FIGURE 3 – PODIUM DECK TO FOUNDATION WALL DRAINAGE SYSTEM TIE-IN DETAIL

FIGURES 4 & 5 – SEISMIC SHEAR WAVE VELOCITY PROFILES

DRAWING PG6620-1 - TEST HOLE LOCATION PLAN

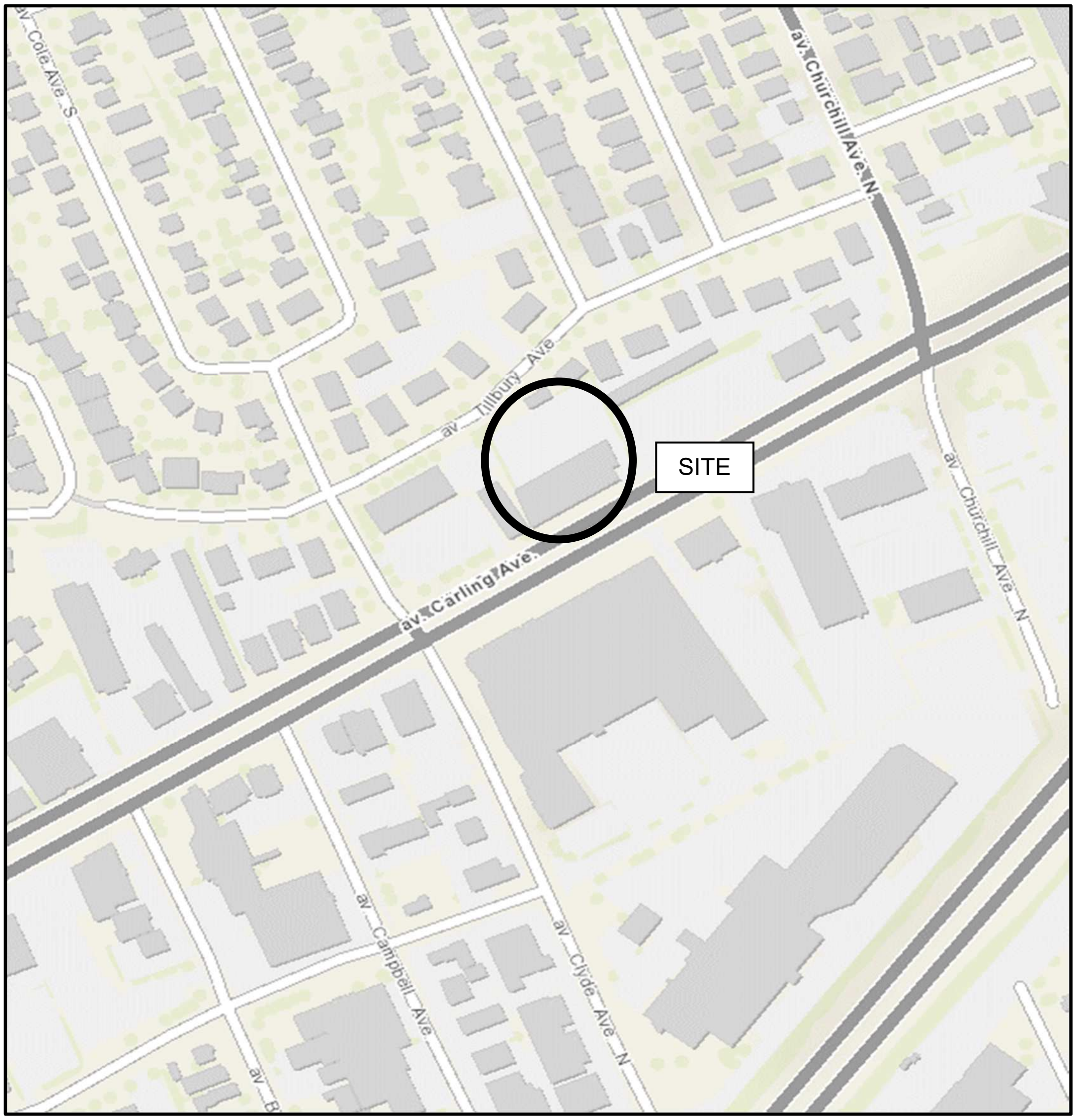
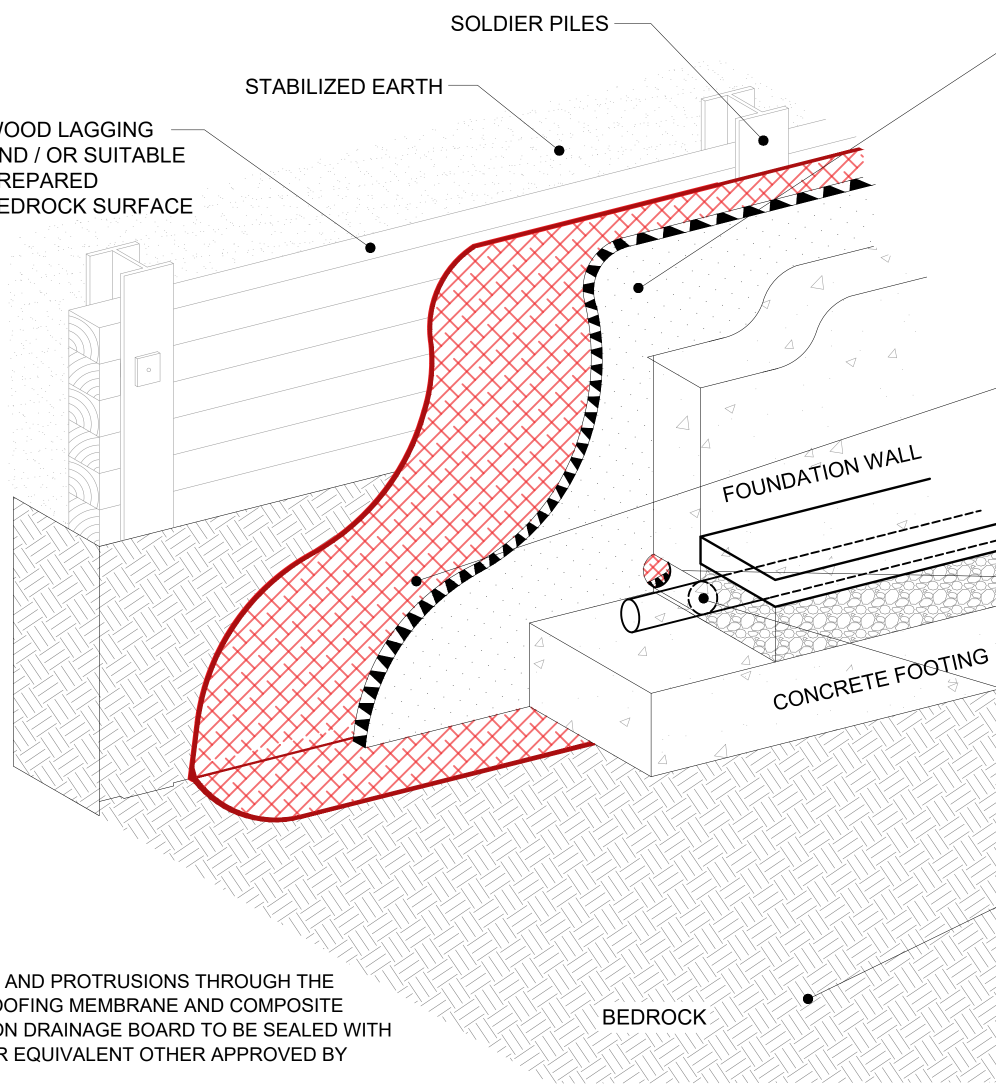


FIGURE 1

KEY PLAN

PATERSON GROUP



ALL SEAMS AND PROTRUSIONS THROUGH THE WATERPROOFING MEMBRANE AND COMPOSITE FOUNDATION DRAINAGE BOARD TO BE SEALED WITH 3M TAPE OR EQUIVALENT OTHER APPROVED BY PATERSON

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

INSIDE EDGE PROPERTIES

PROPOSED MULTI-STOREY BUILDING

1657-1673 CARLING AVENUE & 386 TILLBURY AVENUE

OTTAWA, ONTARIO

Title:

GROUNDWATER INFILTRATION CONTROL SYSTEM

Scale: NTS	**Date:** 03/2023
Drawn by: YA	**Report No.:** PG6620-1
Checked by: SD	**Drawing No.:** FIG.2
Approved by: SD	**Revision No.:**

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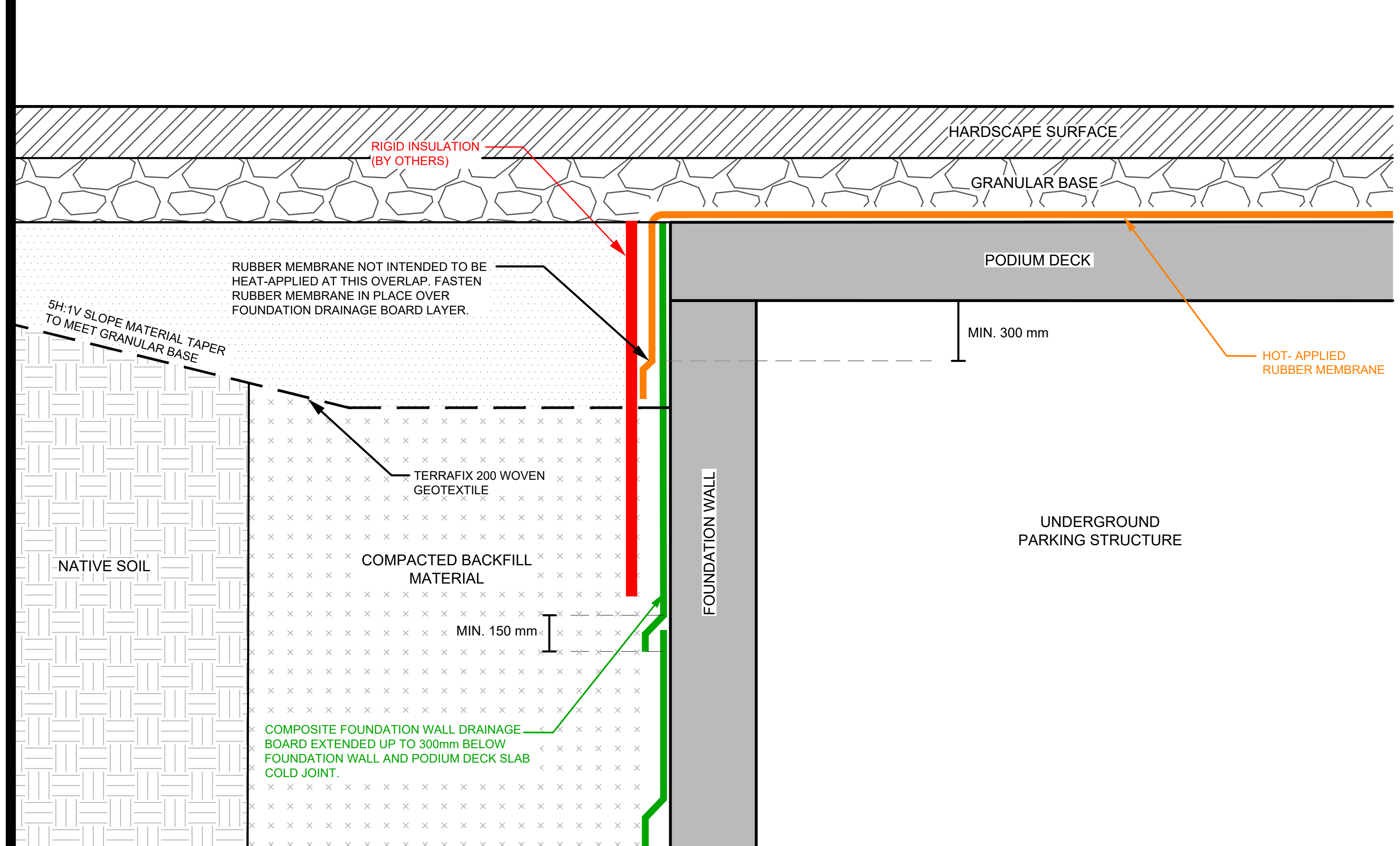
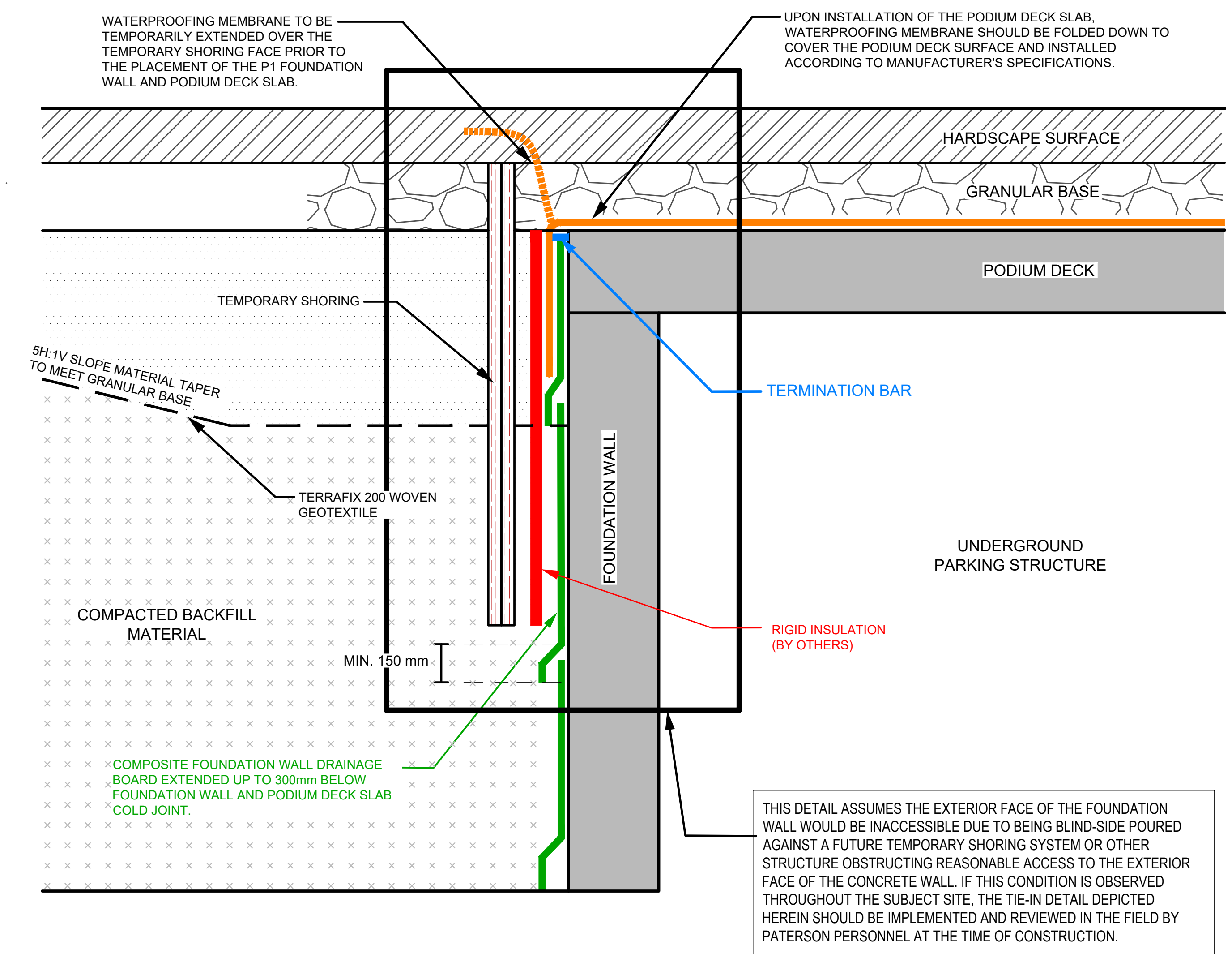
OPTION A - DOUBLE-SIDE POURED TOP OF FOUNDATION WALL



OPTION B - BLIND-SIDE POURED TOP OF FOUNDATION WALL



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

INSIDE EDGE PROPERTIES
PROPOSED MULTI-STOREY BUILDING
1657-1673 CARLING AVENUE & 386 TILLBURY AVENUE
OTTAWA, ONTARIO

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

Scale: N.T.S	Date: 03/2023
Drawn by: YA	Report No.: PG6620-1
Checked by: SD	Dwg. No.: FIGURE 3
Approved by: SD	Revision No.:

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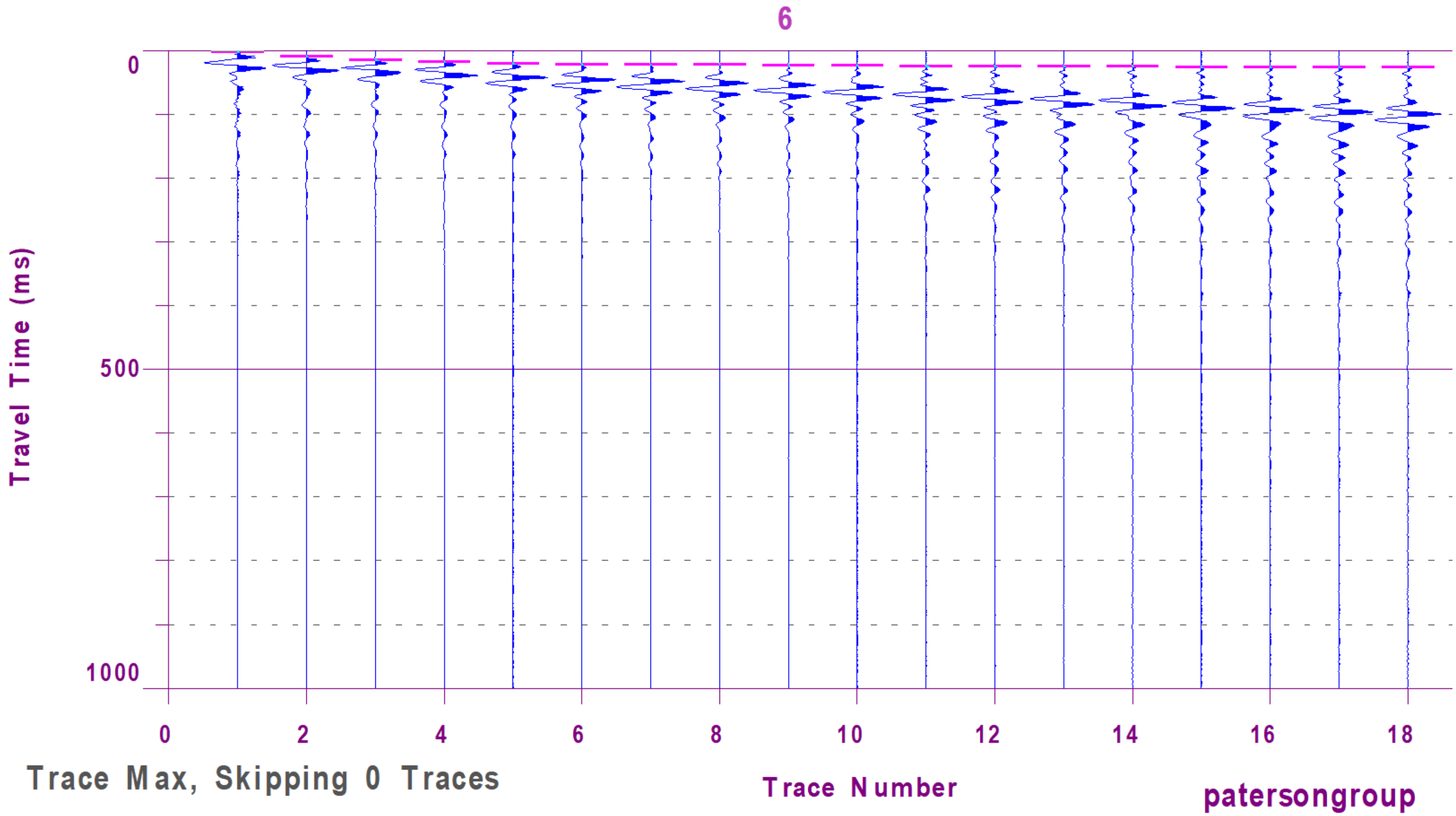


Figure 4 – Shear Wave Velocity Profile at Shot Location -1.5 m

PATERSON GROUP

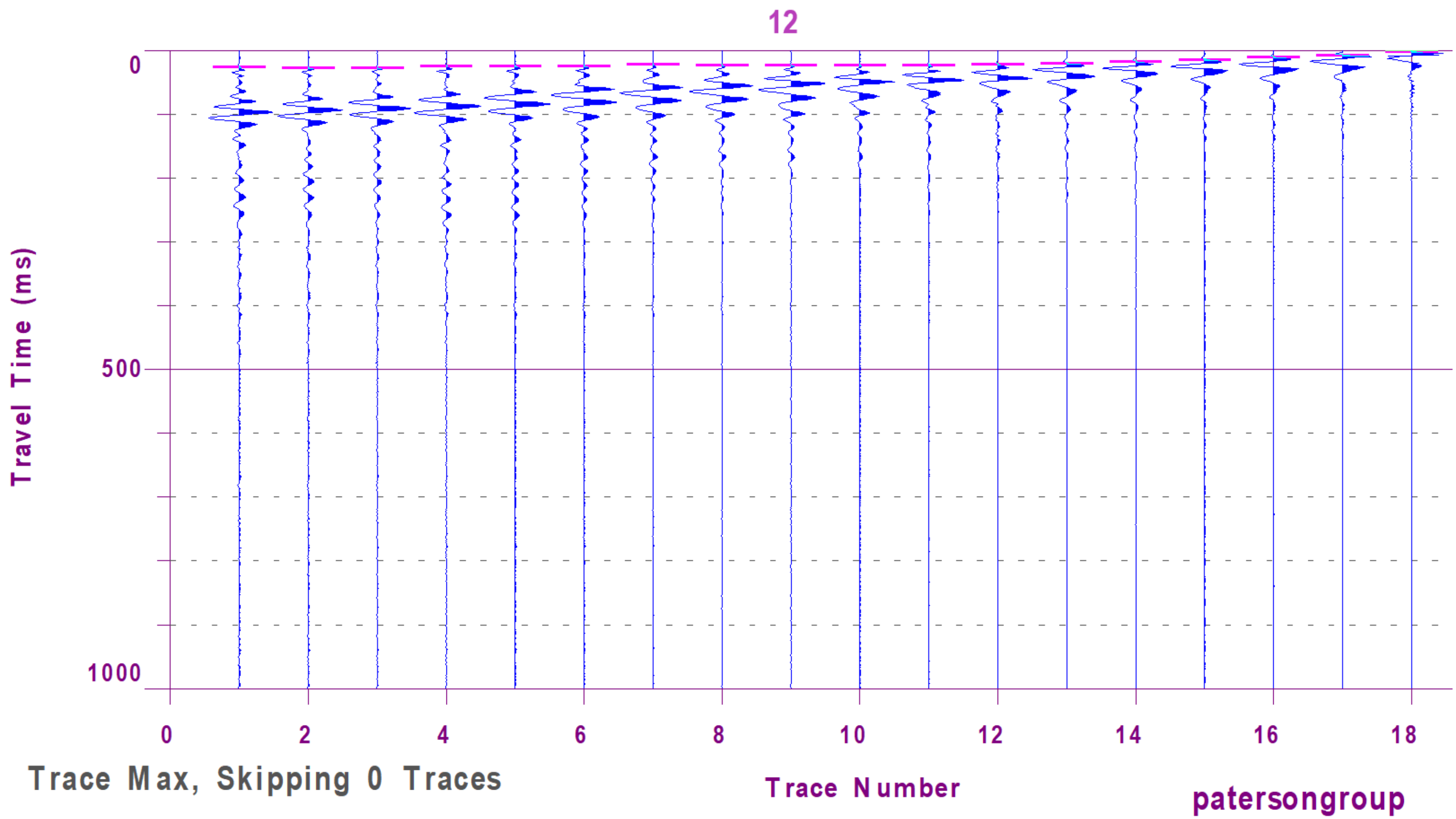
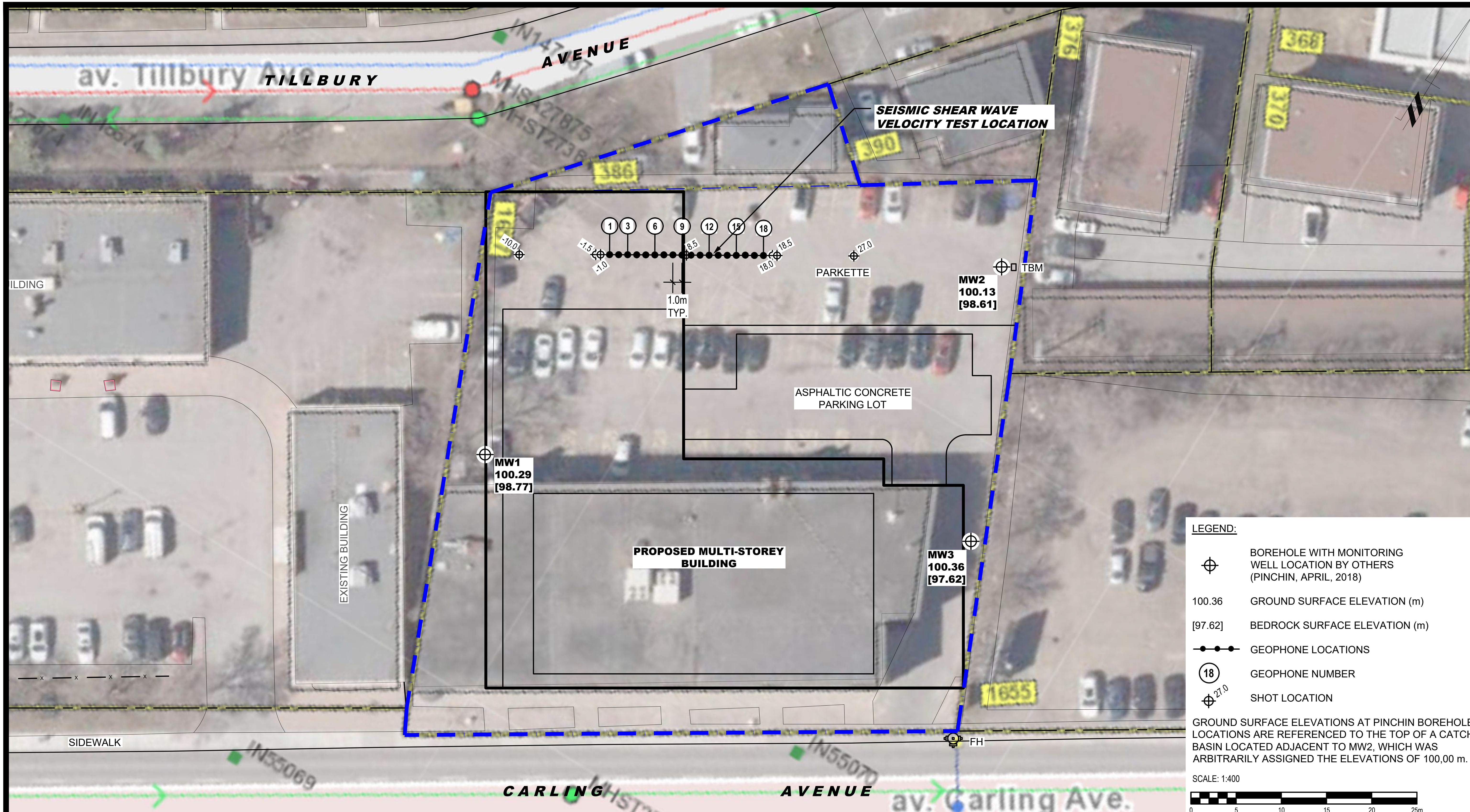


Figure 5 – Shear Wave Velocity Profile at Shot Location 18 m

PATERSON GROUP



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

NO.	REVISIONS	DATE	INITIAL
1	SEISMIC SHEAR WAVE VELOCITY ADDED TO PLAN	18/04/2023	FC

INSIDE EDGE PROPERTIES

GEOTECHNICAL INVESTIGATION
PROPOSED MULTI-STOREY BUILDING
1657-1673 CARLING AVENUE & 386 TILLBURY AVENUE

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:400	Date: 04/2023
Drawn by: YA	Report No.: PG6620-1
Checked by: SD	Dwg. No.: PG6620-1
Approved by: SD	Revision No.: 1

11x17

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