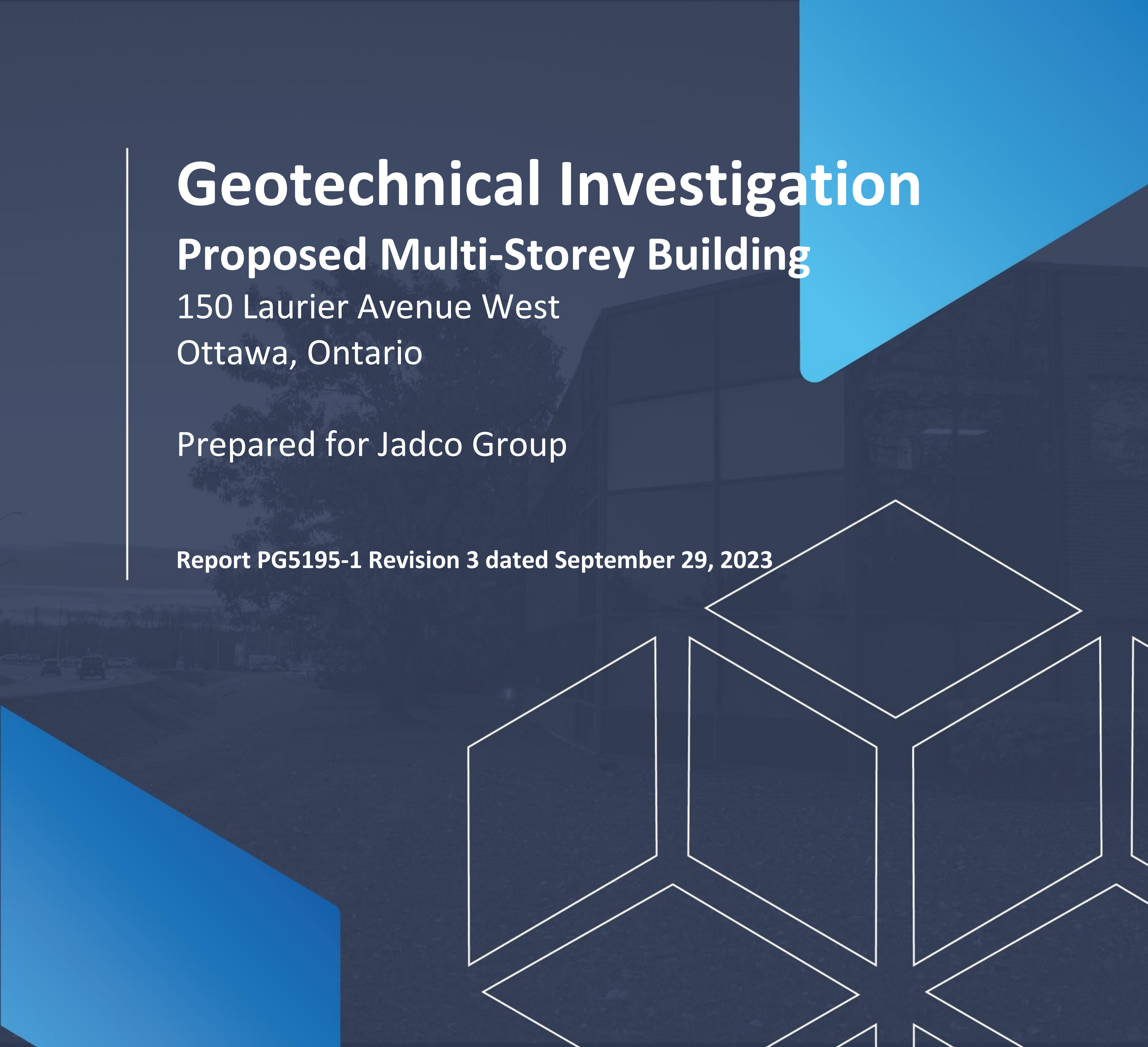
PATERSON GROUP



Geotechnical Investigation

Proposed Multi-Storey Building

150 Laurier Avenue West
Ottawa, Ontario

Prepared for Jadco Group

Report PG5195-1 Revision 3 dated September 29, 2023

Table of Contents

Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Jadco Group to conduct a geotechnical investigation for the proposed multi-storey building, which is to be located at 150 Laurier Avenue West in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objectives of the current investigation were to:

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

- ☐ Based on the results of the boreholes, 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.

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

2.0 Proposed Development

Based on the latest drawings available, it is anticipated that the proposed development will consist of a high-rise building with 6 levels of underground parking. The footprint of the proposed parking structure is anticipated to occupy the entire site.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current geotechnical investigation was carried out between January 9 to 15, 2020. At that time, 4 boreholes were advanced to a maximum depth of 21.2 m below the existing grade. The borehole locations were distributed in a manner to provide general coverage of the proposed development. The approximate locations of the boreholes are shown on Drawing PG5195-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were put down using a truck-mounted auger drill rig and geoprobe drill rig operated by a two-person crew. All fieldwork was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer. The drilling procedure consisted of augering to the required depths at the selected locations, and 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 samples were transported to our laboratory. The depths at which the split-spoon and auger samples were recovered from the boreholes are shown as SS and AU, 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.

Undrained shear strength testing, using a vane apparatus, was carried out at regular intervals of depth in cohesive soils.

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

Diamond drilling was carried out at BH 1, BH 2 and BH 4, to determine the nature of the bedrock and to assess its quality. A recovery value and a Rock Quality Designation (RQD) value were calculated for each drilled section of bedrock and are shown on the borehole logs. The recovery value is the ratio, in percentage, of the length of the bedrock sample recovered over the length of the drilled section. The RQD value is the ratio, in percentage, of the total length of intact rock pieces longer than 100 mm in one drilled section over the length of the drilled section. These values are indicative of the quality of the bedrock.

Groundwater

A 32 or 51 mm diameter PVC groundwater monitoring well was installed in all boreholes to permit monitoring of the groundwater levels subsequent to the completion of the sampling program.

Sample Storage

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

3.2 Field Survey

The test hole locations and elevations were surveyed in the field by Paterson. The ground surface elevations at the borehole locations were referenced to a temporary benchmark (TBM), consisting of the top of spindle of the fire hydrant located on the west side of Elgin Street in front of 150 Elgin Street. A Geodetic elevation of 70.16 m was previously provided for this TBM by Stantec Geomatics during a previous investigation for an adjacent site. This geodetic elevation was transferred to the fire hydrant in front of 160 Laurier Avenue West, which was used to survey the borehole locations.

The locations of the boreholes, TBM, and the ground surface elevation at the boreholes, are presented on Drawing PG5195-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

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

4.0 Observations

4.1 Surface Conditions

The subject site currently consists of a 5 storey office building with asphalt covered parking surfaces on either side. The site is flat and at grade with Laurier Avenue West.

The site is bordered to the north by Laurier Avenue West, to the east by an old stone church building, to the south by a high-rise office building and to the west by two 2- storey commercial buildings followed by a hi-rise office building.

4.2 Subsurface Profile

Generally, the soil conditions encountered at the boreholes consisted of an asphalt pavement structure at ground surface overlying a fill layer, consisting of brown silty sand with trace gravel and/or clay. A stiff silty clay deposit was encountered below the fill layer extending to depths ranging from 12.2 to 14.6 m below ground surface. Glacial till, consisting of silty clay to silty sand with gravel, cobbles and boulders, was encountered below the silty clay deposit. Practical refusal to augering was encountered at depths between 14.5 and 17.6 m. Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for the details of the soil profile encountered at the boreholes.

Bedrock

A very poor (RQD values ranging between to 25%) to an excellent (RQD values ranging between 90 to 100%) quality black shale bedrock was encountered at BH 1, BH 2 and BH 4 at depths ranging between 14.5 and 17.6 m.

Based on available geological mapping, the bedrock in this area consists of shale of the Billings formation and dark grey limestone.

4.3 Groundwater

Groundwater levels (GWLs) were measured in the monitoring wells installed at the borehole locations and the results are summarized in Table 1. It should be noted that groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater levels could vary at the time of construction and in the long term.

Table - 1Summary of Groundwater Level Readings

Borehole Number	Ground Elevation (m)	Groundwater Levels (m)		Recording Date
		Depth	Elevation	
BH 1	69.49	13.60	55.89	January 22, 2020
BH 2	69.07	7.45	61.62	January 22, 2020
BH 3	69.63	8.76	60.87	January 22, 2020
BH 4	68.82	7.94	60.88	January 22, 2020

Note: The ground surface elevation at each borehole was referenced to a temporary benchmark (TBM), which
consists of the top spindle of the fire hydrant located on the west side of Elgin Street in front of 150 Elgin Street . A Geodetic elevation of 70.16 m was previously provided for this TBM.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered satisfactory for the proposed hi-rise development. It is expected that the proposed multi-storey building be founded on conventional spread footings placed on clean, surface sounded bedrock.

It is expected that temporary shoring will be required for the excavation of the underground parking levels. It is further expected that the influence of the excavation and the selection of the shoring system should account for the effects to adjacent structures including dewatering control measures.

Bedrock excavation is expected for the construction of the lower underground levels of the proposed building.

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

5.2 Site Grading and Preparation

Stripping Depth

Based on the proposed depth of the building excavation, all the overburden will be removed from the footprint of the proposed building.

Bedrock Removal

It is expected that line-drilling in conjunction with controlled blasting and mechanical bedrock removal (hoe-ramming and rock grinding) will be required to remove bedrock.

Prior to undertaking the 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 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/claims related to the blasting operations.

As a general guideline, peak particle velocities (measured at the structures) should not exceed 50 mm/s during the blasting program to reduce the risks of damage to the existing structures. Typically, it's suggested that the peak particle velocity be maintained at 25 mm/s at the property line when possible.

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

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. If the shoring system will include drilled piles into the bedrock extending to 1 to 1.5 m below the proposed excavation bottom, the 1 m horizontal ledge can be omitted.

Vibration Considerations

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

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

Two parameters are used to determine the permissible vibrations, namely, the maximum peak particle velocity and the frequency. For low frequency vibrations, the maximum allowable peak particle velocity is less than that for high frequency vibrations. As a guideline, the peak particle velocity should be less than 15 mm/s between frequencies of 4 to 12 Hz, and 50 mm/s above a frequency of 40 Hz (interpolate between 12 and 40 Hz). It should be noted that these guidelines are for today's construction standards. Considering that several old or sensitive buildings are encountered in the vicinity of the subject site, considerations should be given to lowering these guidelines.

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 building.

5.3 Foundation Design

Bearing Resistance Values

Footings placed directly on lean concrete mud slab over a clean, surface sounded shale bedrock can be designed using a factored resistance value at Ultimate Limit States (ULS) of **2,000 kPa** could be used. A geotechnical resistance factor of 0.5 was applied to the bearing resistance value at ULS.

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 bearing surface 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 1H:6V (or shallower) passing 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 shallower).

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 buildings 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 in Figures 4 and 5 in Appendix 2 of the present report.

Field Program

The seismic array testing location was placed as presented in Drawing PG5195-1 - Test Hole Location Plan, attached to the present report. Paterson field personnel placed 18 horizontal 4.5 Hz. geophones mounted to the surface by means of two 75 mm ground spikes attached to the geophone land case. The geophones were spaced at 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 were 10, 1.5 and 1 m away from the last geophone, aligned to the first geophone, and at the centre of the seismic array.

Data Processing and Interpretation

Interpretation for the shear wave velocity results was 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 buildings. 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.

Due to the depth of the overburden, the main refractor was not observed in our analysis. Based on testing completed at nearby sites, where bedrock of the same formation was encountered, an assumed velocity of **2,020 m/s** was assigned to the bedrock underlying the subject site. It should be noted that as bedrock quality increases, the bedrock shear wave velocity also increases.

Based on our testing results, the average overburden shear wave velocity is **238 m/s**. Considering that the proposed development is founded on conventional footings placed on the bedrock surface, the Vs30 was calculated using the standard equation for average shear wave velocity provided in the OBC 2012 and as presented below:

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

$$V_{s30=} \frac{30\,m}{\left(\frac{30\,m}{2{,}020\,m/s}\right)}$$

$$V_{s30=}\ 2{,}020\ m/s$$

Based on the results of the shear wave velocity testing, the average shear wave velocity, V_{s30}, is **2,020 m/s** for conventional footings placed on the bedrock surface. Therefore, a **Site Class A** is applicable for the design of the proposed building founded on conventional footings placed upon the bedrock surface, as per Table 4.1.8.4.A of the OBC 2012. The soils underlying the subject site are not susceptible to liquefaction.

5.5 Basement Slab

For the proposed multi-storey building, all overburden soil will be removed from the building footprint, leaving the bedrock as the founding medium for the basement floor slab. It is anticipated that the basement area will be mostly parking and the recommended pavement structures noted in Subsection 5.8 will be applicable. However, if storage or other uses of the lower level involve the construction of a concrete floor slab, 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 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 underfloor 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 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 drained unit weight of 20 kN/m^3.

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

Static Earth Pressures

The static horizontal earth pressure (P_o) can be calculated using a triangular earth pressure distribution equal to $K_o\ \gamma H$ where:

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

Seismic Earth Pressures

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

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

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

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

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

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

5.7 Rock Anchor Design

If required in the structural design, rock anchors can be designed using the following geotechnical parameters:

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 by pullout of a 60 to 90 degree cone of rock with the apex of the cone near the middle of the bonded length of the anchor. It should be noted that 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 anchor taken individually.

A third failure mode of shear failure along the grout/steel interface should also 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.

It should be further noted that centre to centre spacing between bond lengths be at least four times the anchor hole diameter and greater than 1.2 m to lower the group influence effects. It is also recommended that anchors in close proximity to each other be grouted at the same time to ensure any fractures or voids are completely in-filled and that fluid grout 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 being put into service. To resist seismic uplift pressures, a passive rock anchor system can be used. It should be noted that a post-tensioned anchor will take the uplift load 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 be provided with a bonded length, or fixed anchor length, at the base of the anchor, which will provide the anchor capacity, as well an unbonded length, or free anchor length, between the rock surface and the start of the bonded length. As the depth at which the apex of the shear failure cone develops is 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 cementious grout. The free anchor length is provided by installing a plastic sleeve to act as a bond break.

Grout to Rock Bond

Generally, the unconfined compressive strength of shale ranges between about 60 and 90 MPa, which is stronger than most routine grouts. A factored tensile grout to rock bond resistance value at ULS of **1.2 MPa**, incorporating a resistance factor of 0.3, can be used. 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. A **Rock Mass Rating (RMR) of 44** was assigned to the bedrock, and Hoek and Brown parameters (**m and s**) were taken as **0.183 and 0.00009**, respectively. For design purposes, we assumed that all rock anchors will be placed at least 1.2 m apart to reduce group anchor effects.

Recommended Rock Anchor Lengths

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

Table 2 - Parameters used in Rock Anchor Review	
Grout to Rock Bond Strength - Factored at ULS	1.2 MPa
Compressive Strength - Grout	40 MPa
Rock Mass Rating (RMR) - Fair quality Shale - Hoek and Brown parameters	44 m=0.183 and s=0.00009
Unconfined compressive strength - Shale	40 MPa
Unit weight - Submerged Bedrock	15 kN/m^3
Apex angle of failure cone	60°
Apex of failure cone	mid-point of fixed anchor length

The fixed anchor length will depend on the diameter of the drill holes. Recommended anchor lengths for a 75 mm diameter hole are provided in Table 3. A detailed analysis of the anchorage system could be provided once the details of the loading for the proposed tower are known. It should be noted that the factored tensile resistance values given in Table 3 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	3.2	1.1	4.3	250
	3.8	2.2	5.8	450
	4.1	2.6	6.7	600
	5.0	2.5	7.5	750

Other considerations

The anchor drill holes should 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 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.

5.8 Pavement Structure

It is anticipated that the podium deck structure will be provided with car only parking areas, access lanes, fire truck lanes and loading areas. Based on the concrete slab subgrade, the pavement structure indicated in the below may be considered for design purposes:

Table 4 - Recommended Pavement Structure - Car-Only Parking Areas (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 Paragraph Below)
n/a	**Waterproofing Membrane and IKO Protection Board**

SUBGRADE – Reinforced Concrete Podium Deck
*If specified by others, not required from a geotechnical perspective
**Thickness is dependent on grade of insulation as noted in following paragraph

Table 5 - Recommended Pavement Structure – Access Lane, Fire Truck Lane, Ramp and Heavy Truck Parking Areas (Podium Deck)

Thickness (mm)	Material Description
40	**Wear Course** - HL-3 or Superpave 12.5 Asphaltic Concrete
50	**Wear Course** - HL-8 or Superpave 19.0 Asphaltic Concrete
300**	**Base** - OPSS Granular A Crushed Stone
See Below*	**Thermal Break*** - Rigid insulation (See Paragraph Below)
n/a	**Waterproofing Membrane and IKO Protection Board**

SUBGRADE – Reinforced Concrete Podium Deck
*If specified by others, not required from a geotechnical perspective
**Thickness is dependent on grade of insulation as noted in following paragraph

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 (HI-40). The pavement structures 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 may be considered for car only parking and heavy traffic areas. The subgrade material will consist of glacial till and bedrock throughout the exterior and lowest basement level of the subject site, respectively. The proposed pavement structures are shown in Tables 6 and 7.

Table 6 - 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, bedrock or OPSS Granular B Type I or II material placed over in situ soil or bedrock	

Table 7 - Recommended Pavement Structure - Heavy-Truck 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, bedrock or OPSS Granular B Type I or II material placed over in situ soil or bedrock	

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

The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 100% of the material's SPMDD using suitable compaction equipment

Parking Level Pavement Structure

For design purposes, it is recommended that the rigid pavement structure for the lower underground parking level of the proposed building 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 8 below. The flexible pavement structure presented in Tables 6 and 7 should be used for car only parking areas, at grade access lanes and heavy loading parking areas.

Table 8 - Recommended Rigid Pavement Structure - Car Only Parking Areas

Thickness (mm)	Material Description
125	Exposure Class C2 – 32 MPa Concrete (5 to 8% Air Entrainment)
300	BASE – OPSS Granular A Crushed Stone
SUBGRADE – Existing import 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 hours after the concrete has been poured during warm temperatures and up to 12 hours during cooler temperatures.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

The following recommendations are required for the building's foundation drainage system from a geotechnical perspective. It is recommended that Paterson be engaged at the design stage of the future building (and prior to tender) to review and provide supplemental information for the building's 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 building. It is anticipated that insufficient room will be available for exterior backfill and that the foundation walls 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 the underside of footings and the 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.

❑ 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 (shoring/bedrock excavation face) of the top end of a lower board. All endlaps of the drainage board sheets should overlap abutting sheets by a minimum of 150 mm. All overlaps should be sealed with a suitable adhesive and/or sealant material approved by the geotechnical consultant.

☐ The bedrock face 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 2 – Podium Deck to Foundation Wall Drainage System Tie-In Detail.

Sidewalks and Walkways

Backfill material below sidewalk and walkway subgrade areas or other settlement sensitive structures which are not adjacent to the buildings should consist of free-draining, non-frost susceptible material. This material should be placed in maximum 300 mm thick loose lifts and compacted to at least 98% of its SPMDD under dry and above freezing conditions.

Adverse Effects of Dewatering on Adjacent Properties

Since the proposed development will be founded below the long term groundwater level, a waterproofing membrane system has been recommended to lessen the effects of water infiltration. Any long term dewatering of the site will be minimal and should have no adverse effect to the surrounding buildings or structures. The short term dewatering during the excavation program will be managed by the excavation contractor and an attempt will be made to grout or patch any areas with noticeable water infiltration.

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, or a minimum of 0.6 m of soil cover, in conjunction with foundation insulation, 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.

The parking garage should not require protection against frost action due to the founding depth. Unheated structures, such as the access ramp, may be required to insulate against the deleterious effect of frost action. A minimum of 2.1 m of soil cover alone, or a minimum of 0.6 m of soil cover, in conjunction with foundation insulation, should be provided.

6.3 Temporary Shoring

Temporary shoring will be required for the overburden soil to complete the required excavations where insufficient room is available for open cut methods. The shoring requirements will depend on the depth of the excavation, the proximity of the adjacent buildings and underground structures and the elevation of the adjacent building foundations and underground services.

The design and approval of the shoring system will be the responsibility of the shoring contractor and the shoring designer 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. Furthermore, the design of the temporary shoring system should take into consideration, a full hydrostatic condition which can occur during significant precipitation events.

The temporary system will consist of a combination of soldier pile and lagging system for open areas such as roadways and parking lots and interlocking steel sheet piling for areas adjacent or in close proximity to existing structures. Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be included in the earth pressures described below. These systems could be cantilevered, anchored or braced. Generally, the shoring systems should be provided with tie-back rock anchors to ensure stability. The shoring system is recommended to be adequately supported to resist toe failure, if required, by means of rock bolts or extending the piles into the bedrock through pre-augered holes if a soldier pile and lagging system is the preferred method.

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

Table 5 - Soil Parameters

Parameters	Values
Active Earth Pressure Coefficient (Ka)	0.33
Passive Earth Pressure Coefficient (Kp)	3
At-Rest Earth Pressure Coefficient (Ko)	0.5
Dry Unit Weight (γ), kN/m^3	20
Effective Unit Weight (γ), kN/m^3	13

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

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

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

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. This condition can be omitted provided that toe anchors are installed and designed by the shoring engineer and the piles for the shoring system are extended well below the base of the excavation.

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 horizontal rock anchors should be evaluated during the excavation operations and should be discussed with the structural engineer during the design stage.

Underpinning and Review of Existing Structures

The founding conditions of the existing adjacent structures should be reviewed by the geotechnical consultant prior to commencing excavation. Depending on the founding conditions, as well as, the proximity and depth of the proposed excavation, an underpinning program may be required for the existing building(s). It is further recommended that the condition of the existing foundations be reviewed by a structural engineer.

6.4 Pipe Bedding and Backfill

At least 150 mm of OPSS Granular A should be used for bedding for sewer and water pipes when placed on soil subgrade. The bedding should extend to the spring line of the pipe. Cover material, from the spring line to at least 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 compacted to a minimum of 95% of the material's SPMDD.

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

6.5 Groundwater Control

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

A temporary Ministry of the Environment, Conservation and Parks (MECP) permit to take water (PTTW) Category 3 may be required for this project if more than 400,000 L/day of ground and/or surface water is to be pumped during the construction phase. A minimum 4 to 5 months should be allowed for completion of the PTTW application package and issuance of the permit by the MECP. 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.

For typical ground or surface water volumes, being pumped during the construction phase, 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. 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.

Long-term Groundwater Control

Our recommendations for the proposed building’s long-term groundwater control are presented in Subsection 6.1. Any groundwater breaching the waterproofing system will be directed to the proposed building’s cistern/sump pit. Provided the proposed groundwater infiltration control system is properly implemented and approved by the geotechnical consultant at the time of construction, it is expected that groundwater flow will be low (less than 5,000 L/day) with higher volumes during peak periods noted after rain events. A more accurate estimate can be provided at the time of construction, once groundwater infiltration levels are observed. It is anticipated that the groundwater flow will be controllable using conventional open sumps.

Impacts on Neighbouring Structures

The installation of a temporary shoring system will disturb the soil immediately behind the shoring system which may cause some movement of adjacent structures. To lessen these effects, consideration should be given to adding brackets to the shoring system that could support the adjacent structures or an alternative support system for the adjacent structures. Furthermore, until the waterproofing is completed, temporary dewatering will also cause typical minor differential settlements to unsupported structures.

6.6 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.

Precaution 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.

6.7 Protection of Potentially Expansive Shale Bedrock

Upon being exposed to air and moisture, the shale may decompose into thin flakes along the bedding planes. Previous studies have concluded shales containing pyrite are subject to volume changes upon exposure to air. As a result, the formation of jarosite crystals by aerobic bacteria occurs under certain ambient conditions.

It has been determined that the expansion process does not occur or can be retarded when air (i.e. oxygen) is prevented from contact with the shale and/or the ambient temperature is maintained below 20°C, and/or the shale is confined by pressures in excess of 70 kPa. The latter restriction on the heaving process is probably the major reason why damage to structures has, for the greater part, been confined to slabs-on-grade rather than footings.

The presence of expansive shale may be encountered at the subject site. To reduce the long term deterioration of the shale, exposure of the bedrock bearing surface to oxygen should be kept as low as possible. The bedrock bearing surface within the proposed building footprint should be protected from excessive dewatering and exposure to ambient air. A 50 to 75 mm thick concrete mud slab, consisting of minimum 15 MPa lean concrete, should be placed on the exposed bedrock bearing surface within a 48 hour period of being exposed. The excavated sides of the exposed bedrock should be sprayed with a bituminous emulsion or shotcrete to seal bedrock from exposure to air and dewatering.

Preventing the dewatering of the shale bedrock will also prevent the rapid deterioration and expansion of the shale bedrock. This can be accomplished by spraying bituminous emulsion as noted above.

The above-mentioned recommendations for protection of the potentially expansive shale will also mitigate the potential for sulphate attack. The placement of a concrete mud slab will prevent the oxidization of the shale bedrock therefore preventing the formation of sulphates.

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 the bedrock stabilization and excavation requirements.

☐ Review of the geotechnical aspects of the excavation contractor's temporary shoring design, if required, prior to construction

☐ Review waterproofing and drainage system for foundation walls.

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

☐ Sampling and testing of the concrete and fill materials used.

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

☐ Observation of all subgrades prior to backfilling and follow-up 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 material testing and observation program by the geotechnical consultant.

8.0 Statement of Limitations

The recommendations made in this report are in nature and in accordance with our present understanding of the project. A detailed investigation should be carried out to validate the recommendations presented in this report. We request that we be permitted to review our recommendations when the drawings and specifications are complete.

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

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than Jadco Group 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.

Kevin A. Pickard, P.Eng.

LICENSED PROFESSIONAL ENGINEER
Sept 29, 2023
D. J. GILBERT
100116130
PROVINCE OF ONTARIO

David J. Gilbert, P.Eng.

Report Distribution:

☐ Jadco Group (email copy)
☐ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building - 150 Laurier Ave. West
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE 2020 January 9

FILE NO. PG5195

HOLE NO. BH 1

GROUND SURFACE

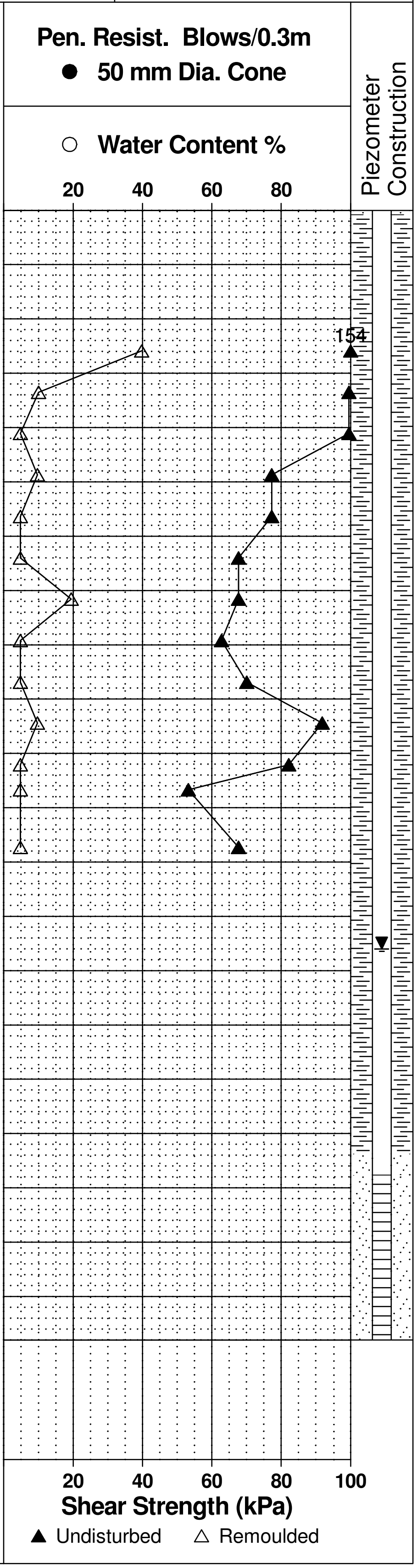
SOIL DESCRIPTION	Bottom depth (m)
Asphaltic concrete	0.10
FILL: Brown silty sand with gravel	0.81
Very stiff to stiff, brown **SILTY CLAY** - grey by 4.6m depth	12.19
GLACIAL TILL: Grey silty clay with gravel, cobbles and boulders, trace sand	14.50
BEDROCK: Fair quality, black shale	20.80

End of Borehole

(GWL @ 13.6m - Jan. 22, 2019)

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	54	14
SS	3	62	12
SS	4	100	P
SS	5	100	P
SS	6	100	P
SS	7	100	P
SS	8	100	P
	9	100	P
SS	10	100	3
SS	11	100	5
RC	1	100	53
RC	2	100	15
RC	3	100	68
RC	4	100	77
RC	5	88	54

DEPTH (m)	ELEV. (m)
0	69.49
1	68.49
2	67.49
3	66.49
4	65.49
5	64.49
6	63.49
7	62.49
8	61.49
9	60.49
10	59.49
11	58.49
12	57.49
13	56.49
14	55.49
15	54.49
16	53.49
17	52.49
18	51.49
19	50.49
20	49.49



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building - 150 Laurier Ave. West
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE 2020 January 14

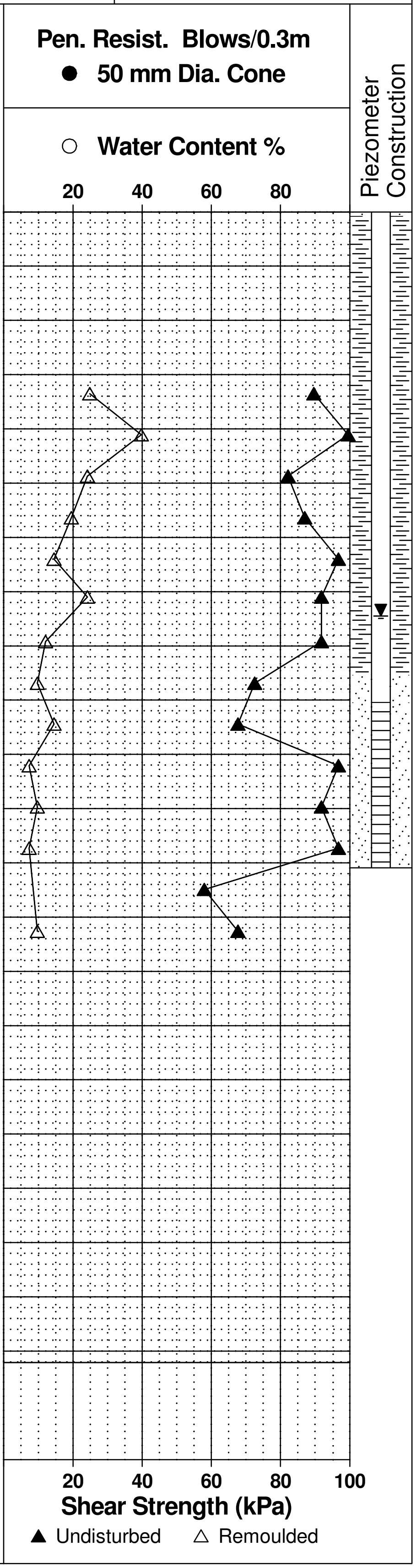
FILE NO. PG5195

HOLE NO. BH 2

SOIL DESCRIPTION	Depth (m)
GROUND SURFACE	
Asphaltic concrete	0.10
FILL: Brown silty sand with gravel and clay	1.35
Stiff, brown **SILTY CLAY** - grey by 3.0m depth	13.72
GLACIAL TILL: Grey clayey silt with gravel, cobbles and boulders	15.24
GLACIAL TILL: Grey silty sand with gravel, cobbles and boulders, some clay	17.58
BEDROCK: Very poor to poor quality, grey shale	21.21
End of Borehole	
(GWL @ 7.45m - Jan. 22, 2019)	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	54	19
SS	3	58	10
SS	4	50	4
SS	5	100	P
SS	6	100	P
SS	7	100	P
SS	8	100	P
SS	9	42	P
SS	10	62	P
SS	11	100	P
SS	12	100	P
SS	13	21	3
SS	14	0	5
RC	1	100	0
RC	2	100	27
RC	3	100	41

DEPTH (m)	ELEV. (m)
0	69.07
1	68.07
2	67.07
3	66.07
4	65.07
5	64.07
6	63.07
7	62.07
8	61.07
9	60.07
10	59.07
11	58.07
12	57.07
13	56.07
14	55.07
15	54.07
16	53.07
17	52.07
18	51.07
19	50.07
20	49.07
21	48.07



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

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building - 150 Laurier Ave. West
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE 2020 January 9

FILE NO. PG5195

HOLE NO. BH 3

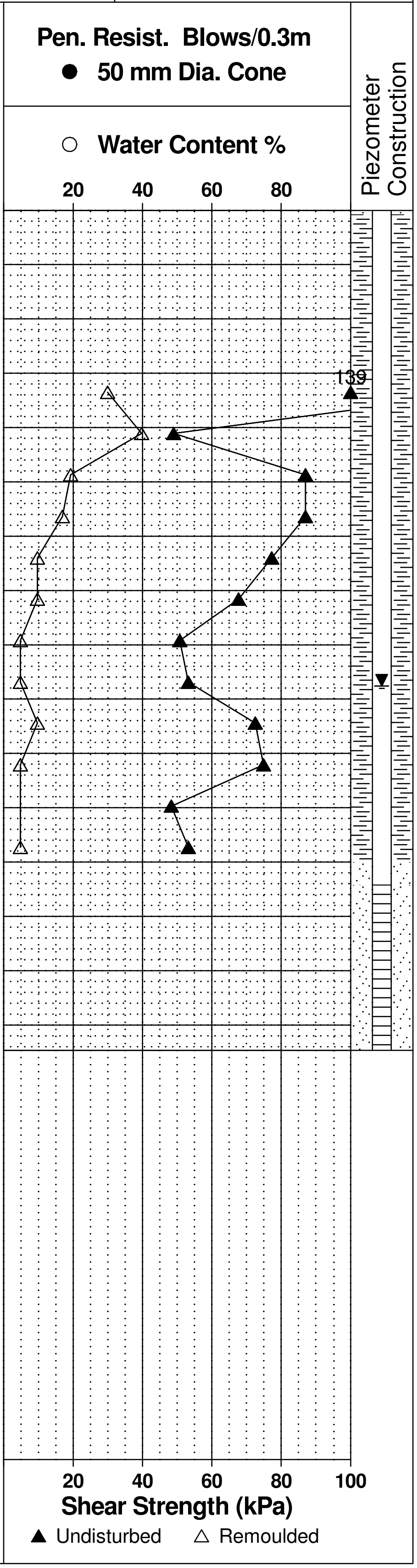
SOIL DESCRIPTION	Depth of base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Asphaltic concrete	0.13	AU	1		
FILL: Brown silty sand with gravel, trace clay	1.52	SS	2	17	12
Very stiff to stiff, brown **SILTY CLAY** - grey by 3.0m depth	12.19	SS	3	100	13
		SS	4	100	6
		SS	5	100	P
		SS	6	100	P
		SS	7	100	P
		SS	8	100	P
		SS	9	71	P
		SS	10	100	P
GLACIAL TILL: Grey silty clay to clayey silt with sand, gravel, cobbles and boulders	13.72	SS	11	83	10
		SS	12	75	3
GLACIAL TILL: Grey silty sand with clay, gravel, cobbles and boulders	15.47	SS	13	58	53
		SS	14	89	50+

DEPTH (m)	ELEV. (m)
0	69.65
1	68.65
2	67.65
3	66.65
4	65.65
5	64.65
6	63.65
7	62.65
8	61.65
9	60.65
10	59.65
11	58.65
12	57.65
13	56.65
14	55.65
15	54.65

End of Borehole

Practical refusal to augering at 15.47m depth

(GWL @ 8.76m - Jan. 22, 2019)



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building - 150 Laurier Ave. West
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

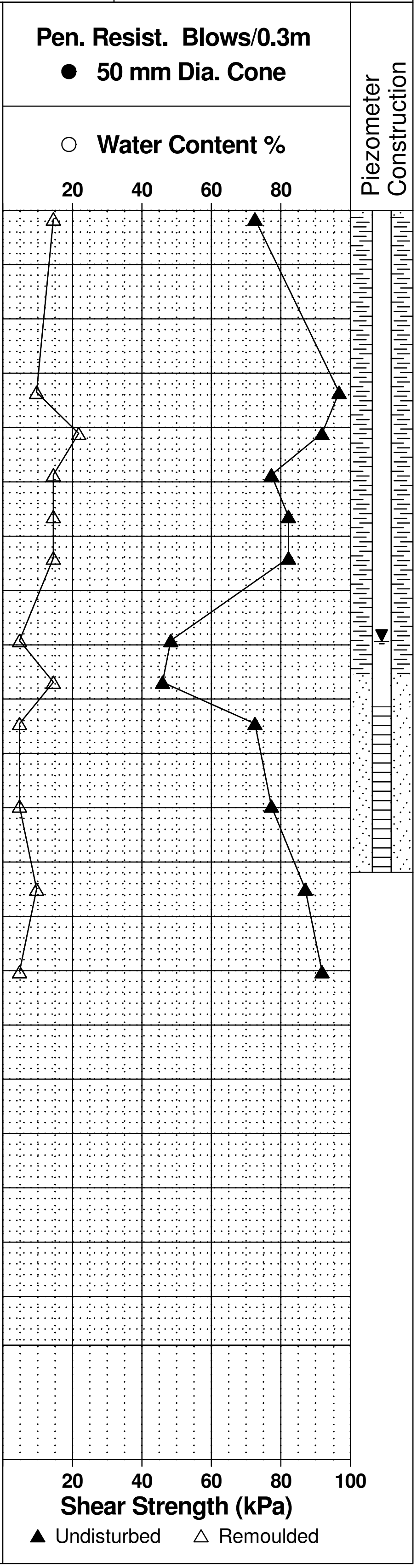
DATE 2020 January 15

FILE NO. PG5195

HOLE NO. BH 4

SOIL DESCRIPTION	Depth to base (m)	TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Asphaltic concrete	0.10	AU	1		
FILL: Brown silty sand with gravel, some clay and concrete debris	1.35	SS	2	50	9
Stiff, brown **SILTY CLAY** - grey by 3.0m depth	14.63	SS	3	100	5
		SS	4	8	4
		SS	5	100	P
		SS	6	58	P
		SS	7	71	P
		SS	8	100	P
GLACIAL TILL: Grey silty sand with gravel, cobbles and boulders, some clay	17.27	SS	9	25	24
		SS	10	29	24
BEDROCK: Good to fair quality, black shale	20.90	RC	1	100	84
		RC	2	96	91
		RC	3	100	68
End of Borehole (GWL @ 7.94m - Jan. 22, 2019)					

DEPTH (m)	ELEV. (m)
0	68.82
1	67.82
2	66.82
3	65.82
4	64.82
5	63.82
6	62.82
7	61.82
8	60.82
9	59.82
10	58.82
11	57.82
12	56.82
13	55.82
14	54.82
15	53.82
16	52.82
17	51.82
18	50.82
19	49.82
20	48.82



SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

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

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

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

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

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

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

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

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

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

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

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

Cc - Concavity coefficient = $(D30)^2$ / (D10 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)

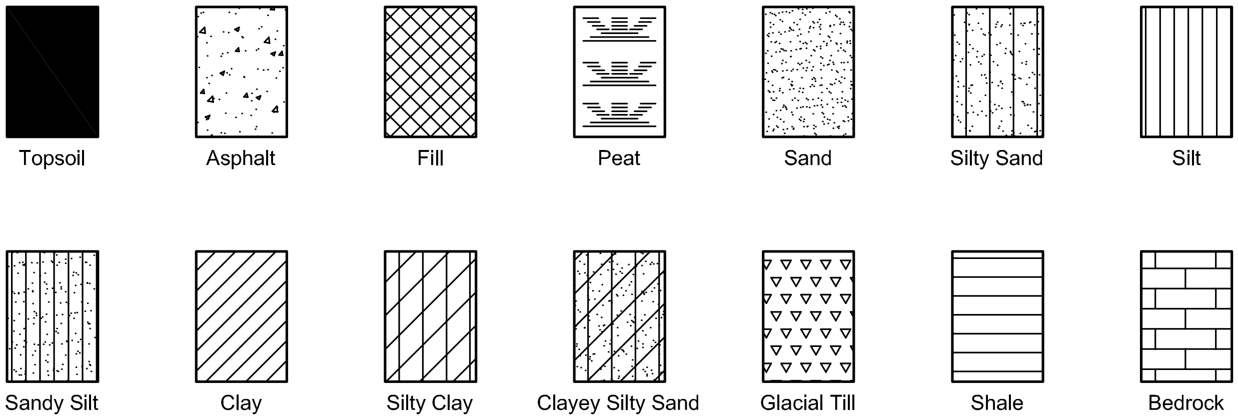
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

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.

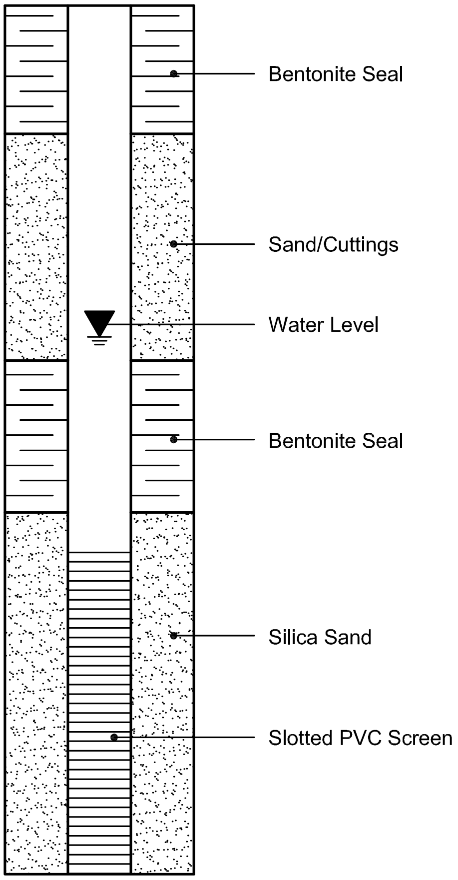
SYMBOLS AND TERMS (continued)

STRATA PLOT

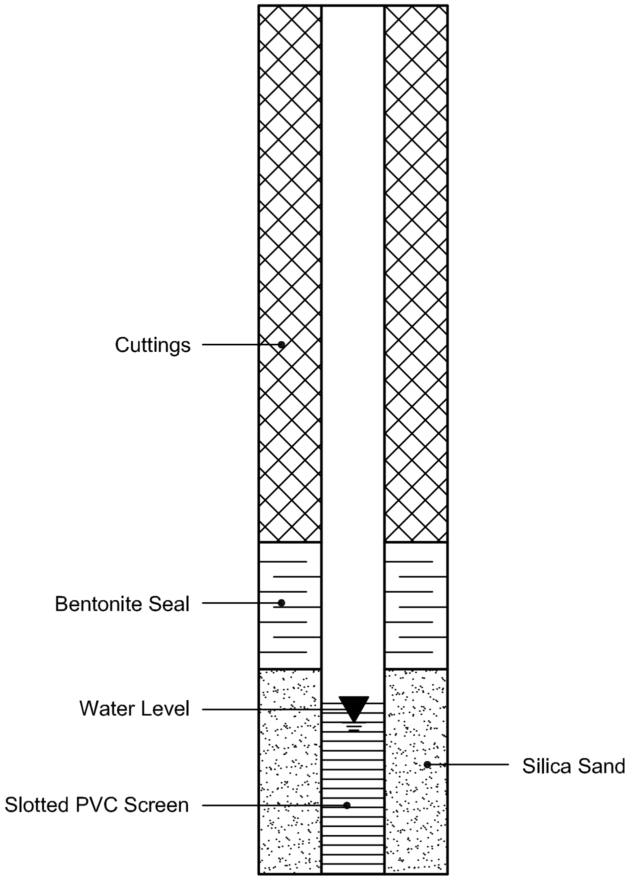


MONITORING WELL AND PIEZOMETER CONSTRUCTION

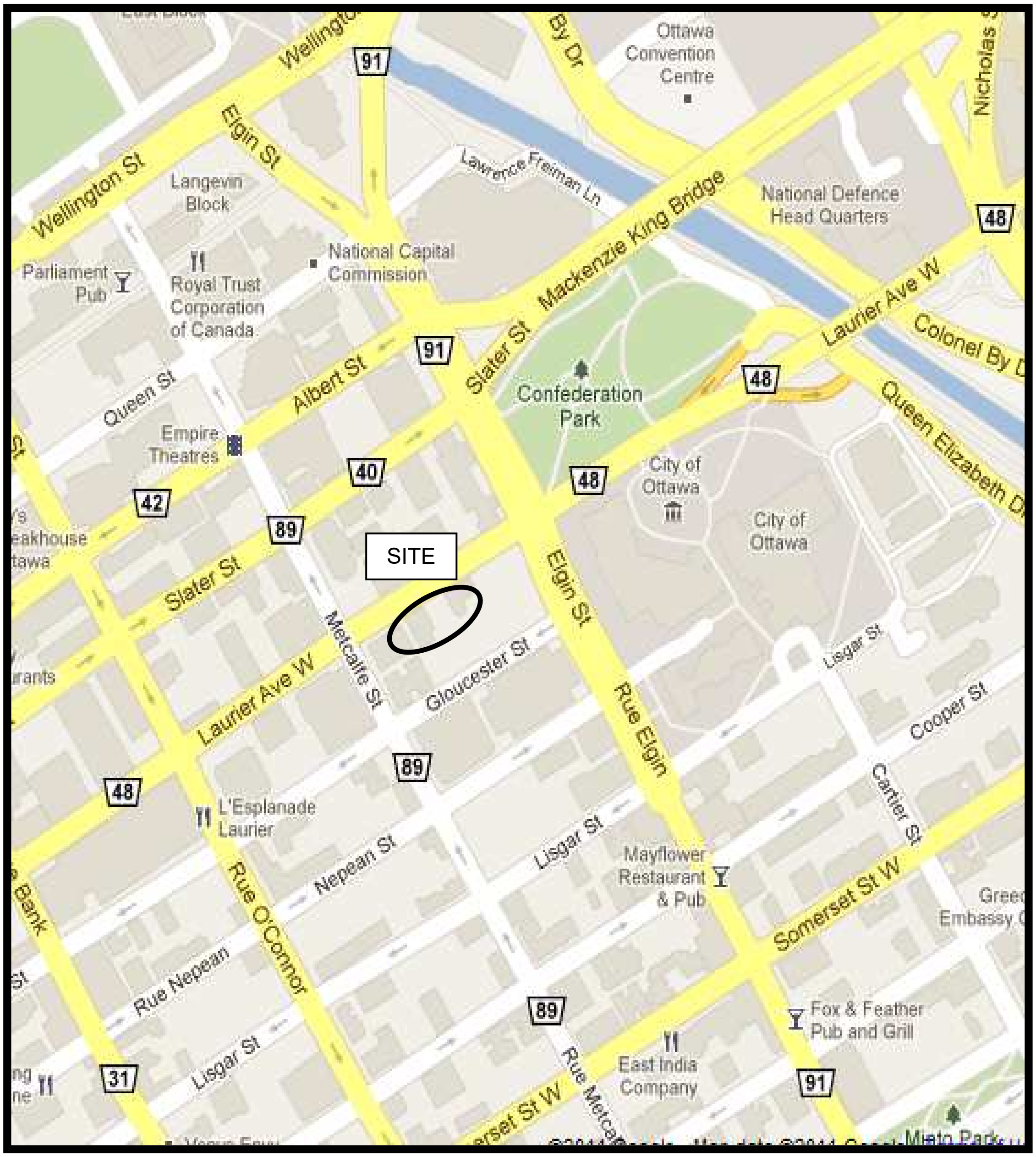
MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



APPENDIX 2



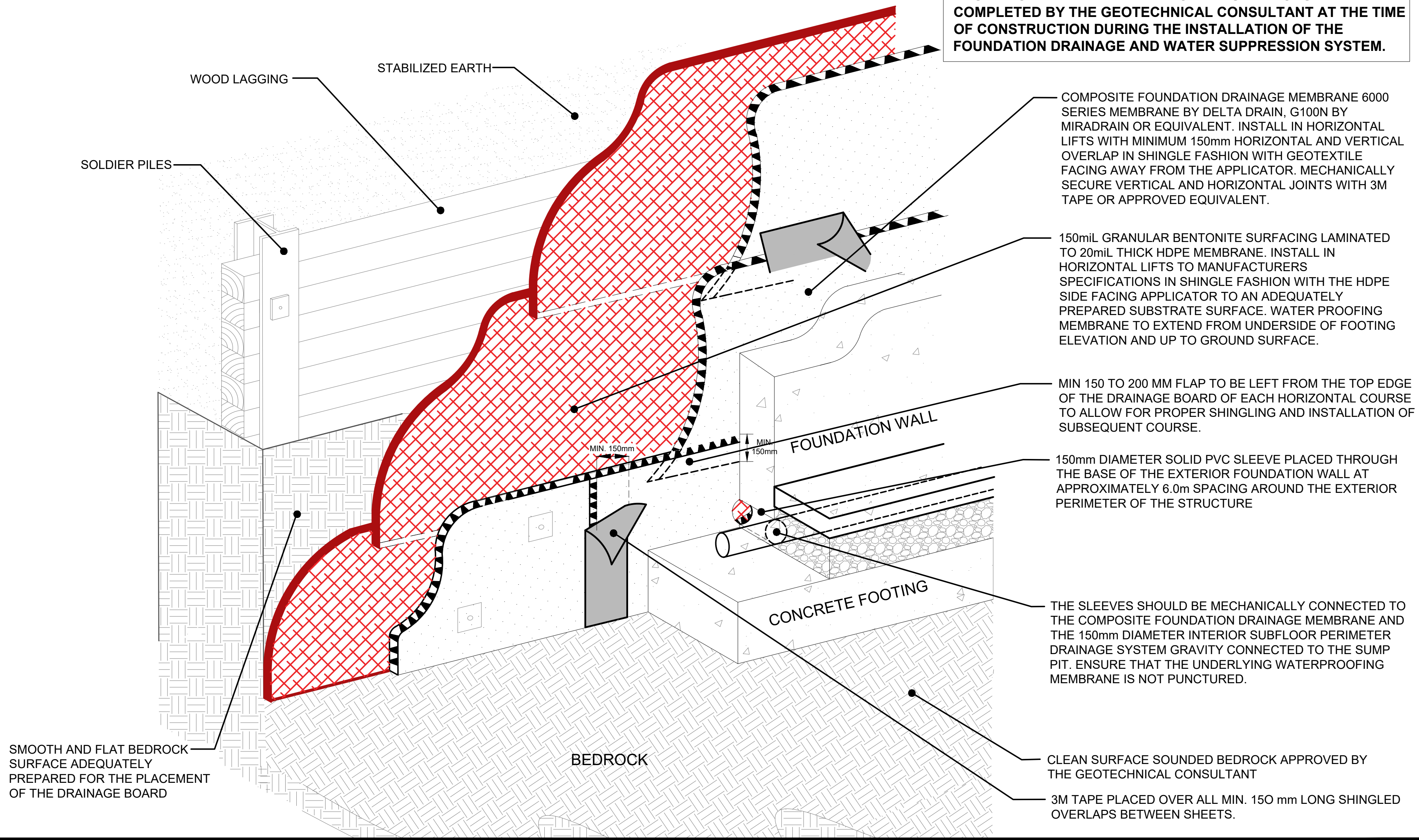
Source: Google Maps

FIGURE 1

KEY PLAN

PATERSON GROUP

IT IS RECOMMENDED THAT PERIODIC INSPECTIONS BE COMPLETED BY THE GEOTECHNICAL CONSULTANT AT THE TIME OF CONSTRUCTION DURING THE INSTALLATION OF THE FOUNDATION DRAINAGE AND WATER SUPPRESSION SYSTEM.



COMPOSITE FOUNDATION DRAINAGE MEMBRANE 6000 SERIES MEMBRANE BY DELTA DRAIN, G100N BY MIRADRAIN OR EQUIVALENT. INSTALL IN HORIZONTAL LIFTS WITH MINIMUM 150mm HORIZONTAL AND VERTICAL OVERLAP IN SHINGLE FASHION WITH GEOTEXTILE FACING AWAY FROM THE APPLICATOR. MECHANICALLY SECURE VERTICAL AND HORIZONTAL JOINTS WITH 3M TAPE OR APPROVED EQUIVALENT.

150miL GRANULAR BENTONITE SURFACING LAMINATED TO 20miL THICK HDPE MEMBRANE. INSTALL IN HORIZONTAL LIFTS TO MANUFACTURERS SPECIFICATIONS IN SHINGLE FASHION WITH THE HDPE SIDE FACING APPLICATOR TO AN ADEQUATELY PREPARED SUBSTRATE SURFACE. WATER PROOFING MEMBRANE TO EXTEND FROM UNDERSIDE OF FOOTING ELEVATION AND UP TO GROUND SURFACE.

MIN 150 TO 200 MM FLAP TO BE LEFT FROM THE TOP EDGE OF THE DRAINAGE BOARD OF EACH HORIZONTAL COURSE TO ALLOW FOR PROPER SHINGLING AND INSTALLATION OF SUBSEQUENT COURSE.

150mm DIAMETER SOLID PVC SLEEVE PLACED THROUGH THE BASE OF THE EXTERIOR FOUNDATION WALL AT APPROXIMATELY 6.0m SPACING AROUND THE EXTERIOR PERIMETER OF THE STRUCTURE

THE SLEEVES SHOULD BE MECHANICALLY CONNECTED TO THE COMPOSITE FOUNDATION DRAINAGE MEMBRANE AND THE 150mm DIAMETER INTERIOR SUBFLOOR PERIMETER DRAINAGE SYSTEM GRAVITY CONNECTED TO THE SUMP PIT. ENSURE THAT THE UNDERLYING WATERPROOFING MEMBRANE IS NOT PUNCTURED.

SMOOTH AND FLAT BEDROCK SURFACE ADEQUATELY PREPARED FOR THE PLACEMENT OF THE DRAINAGE BOARD

CLEAN SURFACE SOUNDED BEDROCK APPROVED BY THE GEOTECHNICAL CONSULTANT

3M TAPE PLACED OVER ALL MIN. 15O mm LONG SHINGLED OVERLAPS BETWEEN SHEETS.

PATERSON GROUP

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

JADCO GROUP

PROPOSED MULTI-STOREY BUILDING
150 LAURIER AVENUE WEST

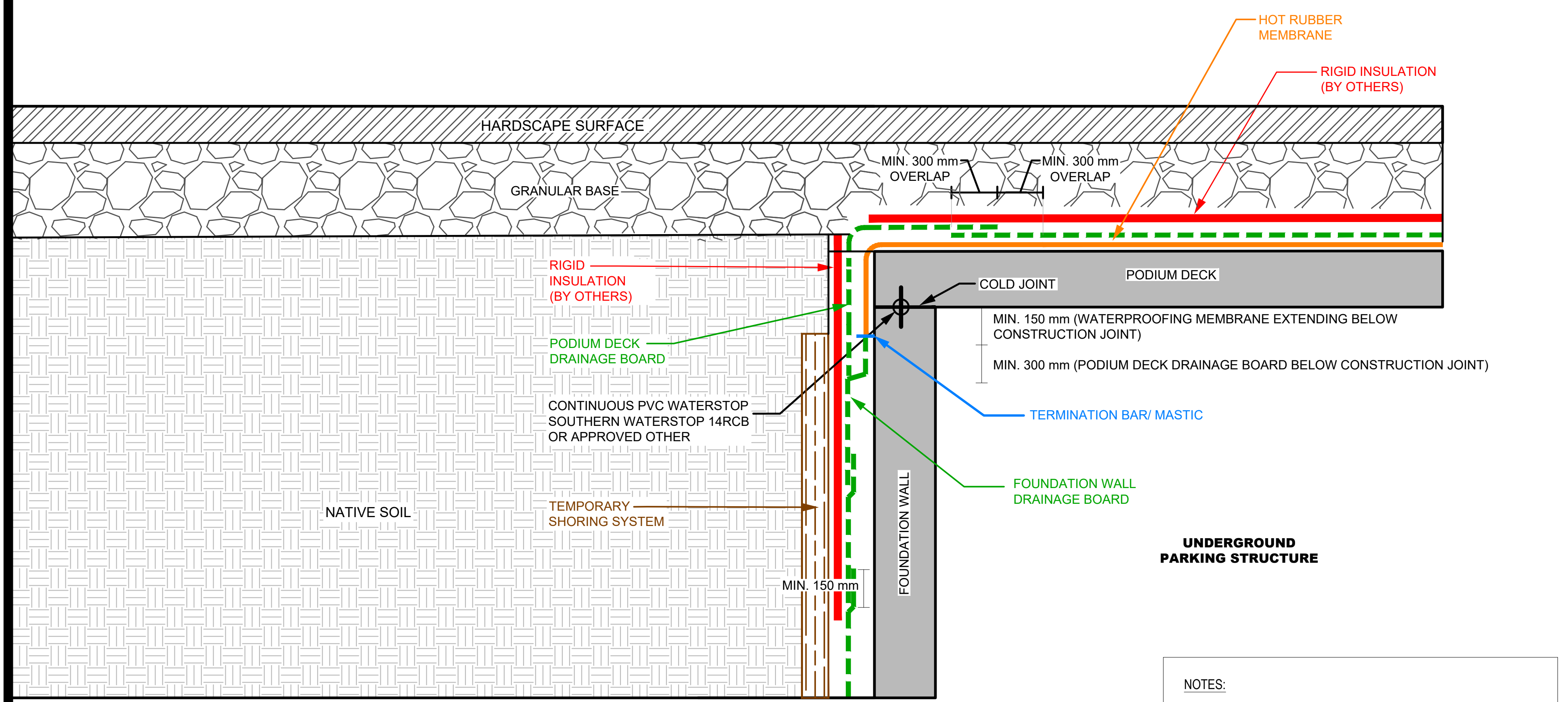
OTTAWA, ONTARIO

Title:

WATER SUPPRESSION SYSTEM
BLIND SIDE POUR

Scale: N.T.S.
Date: 07/2023
Drawn by: RCG
Report No.: PG5195-REP.01
Checked by: KP
Drawing No.: FIGURE 2
Approved by: DG
Revision No.:

p:\autocad drawings\geotechnical\pg51xx\pg5195\pg5195 figure 2 - water suppresion.dwg



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

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

PATERSON GROUP
9 AURIGA DRIVE
OTTAWA, ON
K2E 7S9
TEL: (613) 226-7381

JADCO GROUP
PROPOSED MULTI-STOREY BUILDING
150 LAURIER AVENUE WEST
OTTAWA, ONTARIO

Title:
HORIZONTAL PODIUM WATERPROOFING
TIE IN WITH FOUNDATION DRAINAGE SYSTEM

Scale: N.T.S.
Date: 07/2023
Drawn by: NFRV
Report No.: PG5195-REP.01
Checked by: KP
Drawing No.: FIGURE 3
Approved by: DJG
Revision No.:

p:\autocad drawings\geotechnical\pg51xx\pg5195\pg5195- figure 4 - podium waterproofing.dwg

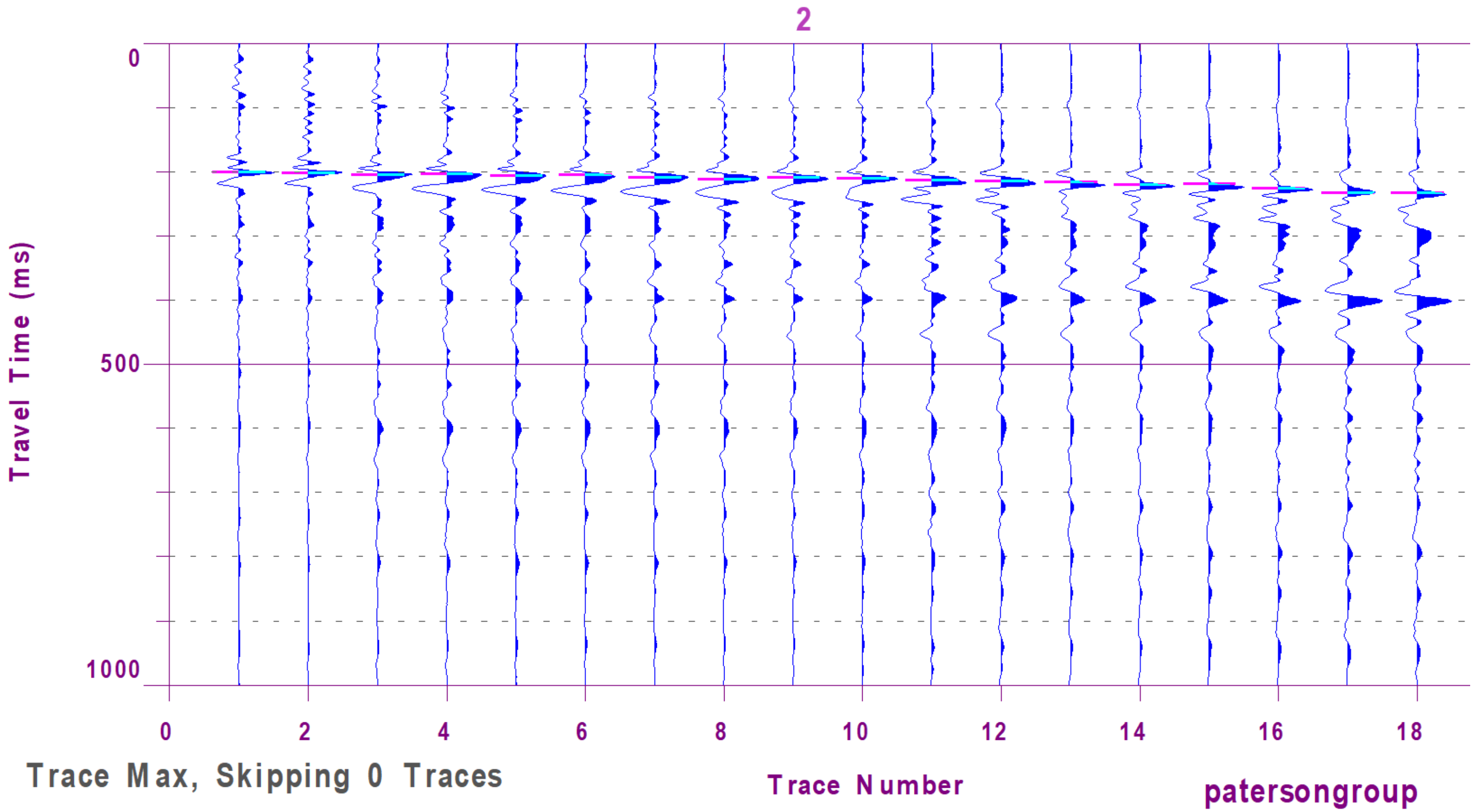


Figure 4 – Shear Wave Velocity Profile at Shot Location -10 m

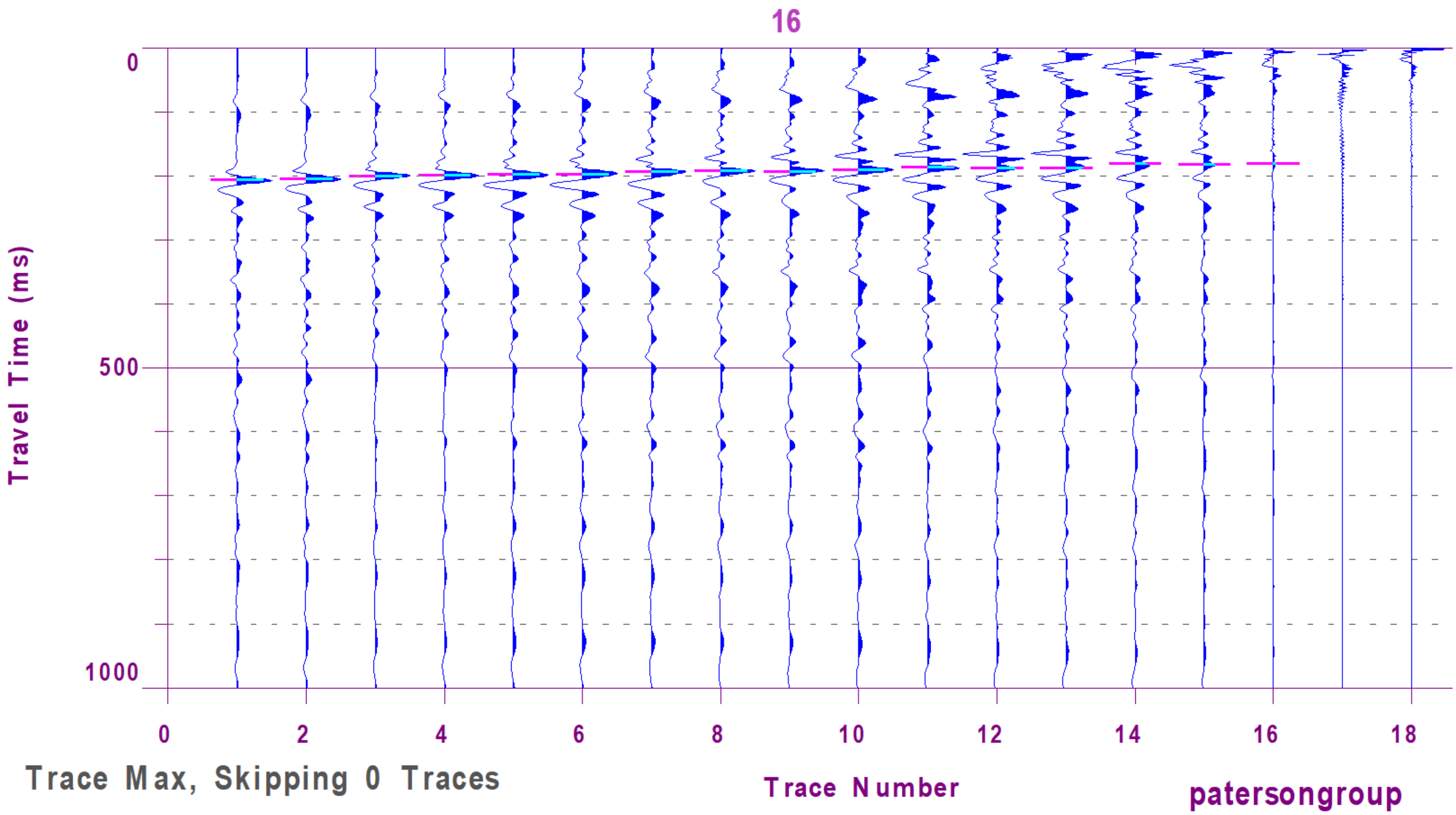
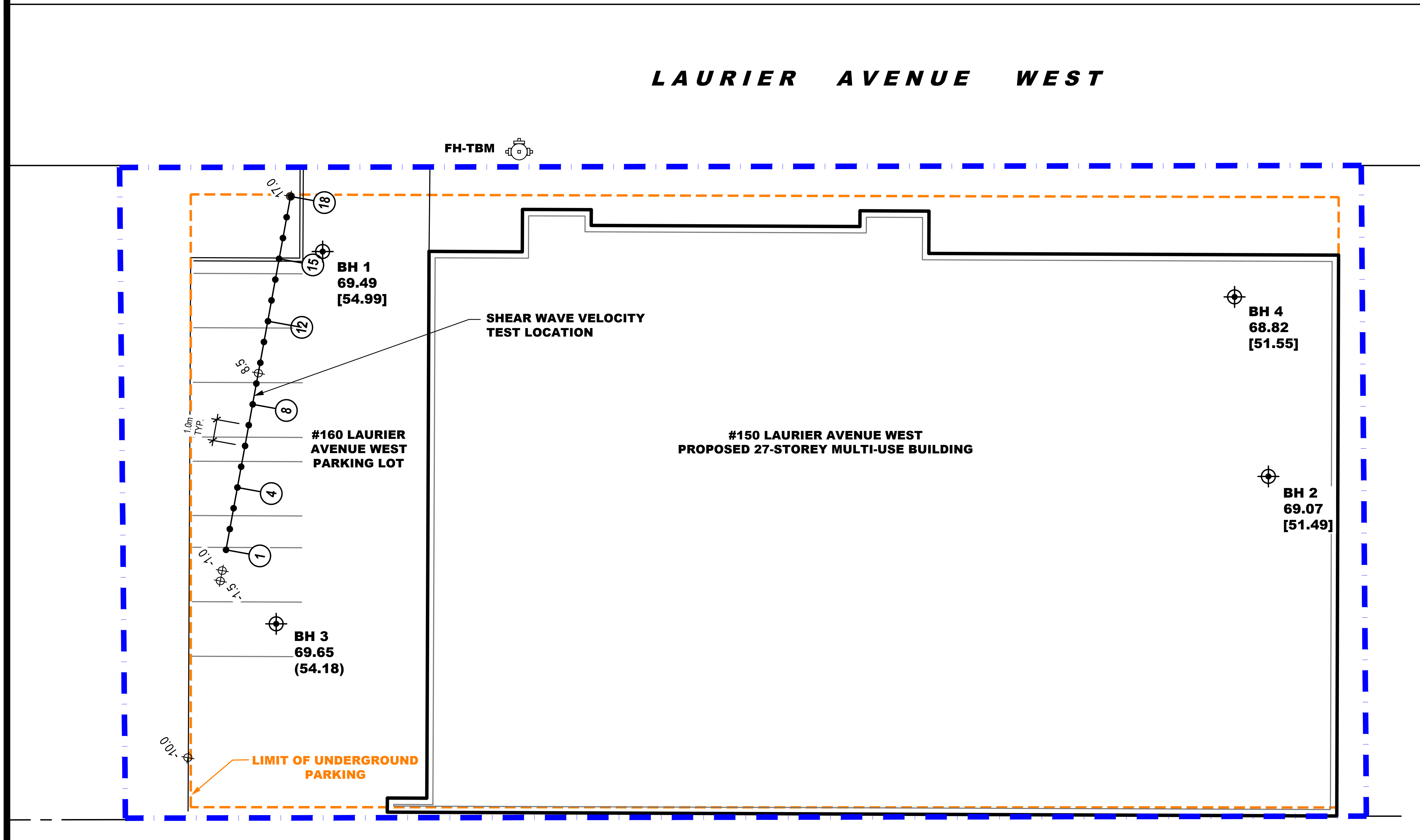


Figure 5 – Shear Wave Velocity Profile at Shot Location 17 m



LEGEND:

⌖	BOREHOLE WITH MONITORING WELL LOCATION
69.49	GROUND SURFACE ELEVATION (m)
[54.99]	BEDROCK SURFACE ELEV. (m)
(54.18)	PRACTICAL REFUSAL TO AUGERING ELEVATION (m)
●—●—●	GEOPHONE LOCATIONS
③	GEOPHONE NUMBER
⌖ +10.0	SHOT LOCATION

TBM - TOP SPINDLE OF FIRE HYDRANT LOCATED ON LAURIER AVENUE WEST. GEODETIC ELEVATION = 69.88m

SCALE: 1:200

0 1 2 3 4 5 10m

PATERSON GROUP

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

NO.	REVISIONS	DATE	INITIAL
2	SHEAR WAVE VELOCITY TEST LOCATION ADDED	21/09/2023	FC
1	REVISED PER 1ST ROUND OF SPC COMMENTS	07/07/2023	KP

JADCO GROUP

GEOTECHNICAL INVESTIGATION
PROP. MULTI-STOREY BUILDING - 150 LAURIER AVENUE WEST

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:200	Date: 01/2020
Drawn by: NFRV	Report No.: PG5195-1
Checked by: CB	Dwg. No.: PG5195-1
Approved by: DJG	Revision No.: 2

p:\autocad drawings\geotechnical\pg51xx\pg5195\pg5195-1 test hole location plan (rev.02).dwg