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Geotechnical Investigation

Proposed Multi-Storey Building
93 Norman Street
Ottawa, Ontario

Prepared For

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Tamarack (Norman) Corporation to conduct a geotechnical investigation for a proposed multi-storey building to be located at 93 Norman Street 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 and groundwater conditions at this site by means of boreholes.

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

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

Investigating the presence or potential presence of contamination on the subject property was not part of the scope of work for this geotechnical investigation.

2.0 Proposed Project

Based on the available conceptual drawings, it is our understanding that a multi-storey structure with two (2) levels of underground parking encompassing the majority of the site is currently being proposed for the subject site.

It is also expected that the proposed development will be serviced with municipal sewer and water.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current geotechnical investigation was carried out on February 3, 2021 by extending a total of 2 boreholes (BH 1-21 and BH 2-21) to a maximum depth of 7 m below the existing ground surface. Relevant test holes completed during the previous investigations (BH 1 through BH 4 and BH 1-12 through BH 4-12) have also been included in the current Geotechnical Investigation Report. The aforementioned boreholes were distributed in a manner to provide general coverage of the subject site taking into consideration of site features, underground utilities and previous boreholes. The locations of the boreholes are shown on Drawing PG2760-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were drilled using a truck-mounted auger 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, 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, placed in sealed plastic bags, and transported to our laboratory for further review. 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.

Diamond drilling was carried out at 7 borehole locations (BH 1, BH3, BH 1-12, BH 2-12, BH 4-12, BH 1-21 and BH 2-21) to assess the bedrock quality. A recovery value and a Rock Quality Designation (RQD) value were calculated for each drilled section of bedrock and are shown on the Soil Profile and Test Data sheets in Appendix 1. The recovery value is the ratio, in percentage, of the length of the bedrock sample recovered over the length of the drilled section. 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.

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

Groundwater

A 32 mm PVC groundwater monitoring well was installed in BH 1, BH 3, BH 1-12, BH 2-12, BH 4-12, BH 1-21 and BH 2-21 to permit monitoring of the groundwater levels subsequent to the completion of the sampling program.

Monitoring Well Installation

Typical monitoring well construction details are described below:

- ❏ 1.5 to 3.0 m of slotted 32 mm diameter PVC screen at the base of borehole.
- ❏ 32 mm diameter PVC riser pipe from the top of the screen to the ground surface.
- ❏ No.3 silica sand backfill within annular space around screen.
- ❏ 300 mm thick bentonite hole plug directly above PVC slotted screen.
- ❏ Clean backfill from top of bentonite plug to the ground surface.

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

Sample Storage

All samples from the current geotechnical investigation 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 borehole locations and ground surface elevation at each borehole location were surveyed by Paterson using a handheld GPS and referenced to geodetic datum.

The borehole location and ground surface elevation at each test hole location are presented on Drawing PG2760-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

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

4.0 Observations

4.1 Surface Conditions

The subject site is located on the north side of Norman Street to the west of the intersection of Preston Street and Norman Street. The site is bordered to the north by single family residential dwellings, to the south by Norman Street followed by single family residential dwellings, and to the east by commercial properties. The site is bordered to the west by a pedestrian pathway followed by the existing Trillium Line.

The subject site was formerly occupied by several single family residential dwellings and commercial slab-on-grade buildings which were recently demolished. At the present time, the site has been re-graded to match the neighbouring properties and Norman Street.

4.2 Subsurface Profile

The subsurface profile at the borehole locations generally consists of a pavement structure and/or fill underlain by a silty sand and/or a glacial till deposit followed by a grey limestone bedrock.

The underlying grey limestone bedrock was cored at BH 1, BH 3, BH 1-12, BH 2-12, BH 4-12, BH 1-21 and BH 2-21 beginning at approximate depths varying between 1.6 and 2.4 m below the existing ground surface, extending to a maximum depth of 11.7 m. Based on our observations, the upper 1 to 2 m of the bedrock is of fair to good quality, while the majority of the remainder of the bedrock core was noted to be good to excellent quality. Specific details of the subsurface profile at each test hole location are presented on the Soil Profile and Test Data sheets in Appendix 1.

Based on available geological mapping, the subject site is located in an area where the bedrock consists of interbedded limestone and shale of the Verulam Formation.

4.3 Groundwater

The recorded groundwater levels recorded at the monitoring wells installed at BH 1, BH 3, BH 1-12, BH 2-12, BH 4-12, BH 1-21 and BH 2-21 are presented in Table 1. It should be noted that groundwater levels fluctuate periodically throughout the year and higher levels could be encountered at the time of construction.

Table 1 - Groundwater Measurements at Monitoring Well Locations

Borehole Number	Ground Elevation (m)	Groundwater Levels		Recording Date
		Depth (m)	Elevation (m)	
BH 1	61.51	1.37	60.14	June 6, 2011
		1.86	59.65	September 5, 2012
BH 3	61.55	1.74	59.81	June 6, 2011
		1.86	59.69	September 5, 2012
BH 1-12	61.59	2.05	59.54	September 5, 2012
BH 2-12	61.42	1.84	59.58	September 5, 2012
BH 4-12	61.53	2.53	59.00	September 5, 2012
BH 1-21	61.60	1.78	59.82	February 9, 2021
BH 2-21	61.88	1.85	60.03	February 9, 2021

5.0 Discussion

5.1 Geotechnical Assessment

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

Considering the shallow depth to bedrock, it is expected that the adjacent buildings are founded on bedrock. Therefore, underpinning is not expected to be required at this site. However, test pits should be excavated at the start of construction, which are observed by Paterson, to confirm the depth and founding conditions of the adjacent structures located in close proximity to the subject site, to determine if underpinning is required.

Bedrock removal will be required to complete the underground parking levels. Line drilling and controlled blasting is recommended where large quantities of bedrock need to be removed. The blasting operations should be planned and completed under the guidance of a professional engineer with experience in blasting operations. It is expected that the vertical walls of the bedrock surface will be grinded to provide a suitable substrate surface for the foundation drainage and waterproofing system.

In addition, it is expected that bedrock stabilization measures will most likely be required along the vertical walls of the bedrock surface, which will be evaluated during the excavation program and determined by the geotechnical consultant at the time of construction.

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

5.2 Site Grading and Preparation

Stripping Depth

Due to the depth of the bedrock at the subject site and the anticipated founding level for the proposed multi-storey building, it is anticipated that all existing overburden material will be excavated from within the footprint of the proposed multi-storey building.

Bedrock Removal

It is expected that line-drilling in conjunction with hoe-ramming or controlled blasting will be required to remove the bedrock for the underground parking levels. In areas of weathered bedrock and where only a small quantity of bedrock is to be removed, bedrock removal may be possible by hoe-ramming.

Prior to considering blasting operations, the 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 25 mm per second during the blasting program to reduce the risks of damage to the existing structures.

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

Vibration Considerations

Construction operations 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 equipments could be a 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 equipments. 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.

Fill Placement

Excavated limestone bedrock could be used as select subgrade material around the proposed building footings, provided the excavated bedrock is suitably crushed to 50 mm in its longest dimension and approved by the geotechnical consultant at the time of placement. Alternatively, an engineered fill such as an OPSS Granular A or Granular B Type II compacted to 98% of its SPMDD could be placed around the proposed footings.

5.3 Foundation Design

Bearing Resistance Values

Footings placed on clean, surface sounded bedrock at the proposed founding elevation can be designed using a factored bearing resistance value at ultimate limit states (ULS) of **5,000 kPa.** 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.

A factored bearing resistance value at ULS of **6,000 kPa**, incorporating a geotechnical resistance factor of 0.5, could be used if founded on limestone bedrock and the bedrock is free of seams, fractures and voids within 1.5 m below the founding level. This could be verified by completing and probing 50 mm diameter drill holes to a depth of 1.5 m below the founding level within the footprint(s) of the footing(s). At least one drill hole should be completed per major footing. The drill hole inspection should be carried out by the geotechnical consultant.

Lateral Support

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

Settlement

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

5.4 Design for Earthquakes

Shear wave velocity testing was completed for the subject site to accurately determine the applicable seismic site classification for the proposed building 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 5 and 6 in Appendix 2.

Field Program

The seismic array testing location was placed on the central area of the site in an approximate northwest-southeast direction as presented in Drawing PG2760-1, attached to the present report. Paterson field personnel placed 24 horizontal 4.5 Hz. geophones mounted to the surface by means of two 75 mm ground spikes attached to the geophone land case. The geophones were spaced at 2 m intervals and connected by a geophone spread cable to a Geode 24 Channel seismograph.

The seismograph was also connected to a computer laptop and a hammer trigger switch attached to a 12 pound dead blow hammer. The hammer trigger switch sends a start signal to the seismograph. The hammer is used to strike an I-Beam seated into the ground surface, which creates a polarized shear wave. The hammer shots are repeated between 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 23, 3, and 2 m away from the first geophone, 2, 3, and 23 m away from the last geophone, and at the centre of the seismic array.

Data Processing and Interpretation

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

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

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

The Vs_{30} was calculated using the standard equation for average shear wave velocity provided in the Ontario Building Code (OBC) 2012.

$$V_{S30}=\frac{Depth_{ofinterest}(m)}{\sum\left(\frac{Depth_{Layer1}(m)}{V_{S_{Layer1}}(m/s)}+\frac{Depth_{Layer2}(m)}{V_{S_{Layer2}}(m/s)}\right)}$$

$$V_{S30}=\frac{30m}{\left(\frac{30m}{3{,}122m/s}\right)}$$

$$V_{S30}=3{,}122m/s$$

Based on the result, the average seismic shear wave velocity, Vs_{30}, for foundations at the subject site is **3,122 m/s**. Therefore, a **Site Class A** is applicable for design of the proposed building as per Table 4.1.8.4.A of the OBC 2012. The soils underlying the subject site are not susceptible to liquefaction.

5.5 Basement Slab

All overburden soil will be removed from the subject site leaving the bedrock as the founding medium for the lower basement floor slab. If storage or other uses of the lower level where a concrete floor slab will be used, it is recommended that the upper 200 mm of sub-slab fill consists of 19 mm clear crushed stone. 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 groundwater conditions encountered at the time of the fieldwork, a subfloor drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided in the clear stone under the lower basement floor.

5.6 Basement Wall

It is understood that the basement walls are to be poured against a waterproofing system, which will be placed against the exposed bedrock face. Below the bedrock surface, a nominal coefficient for 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). A seismic earth pressure component will not be applicable for the foundation wall, which is to be poured against the bedrock face. It is expected that the seismic earth pressure will be transferred to the underground floor slabs, which should be designed to accommodate these pressures. A hydrostatic groundwater pressure should be added for the portion below the groundwater level.

Where soil is to be retained, the conditions can be well-represented by assuming the retained soil has a coefficient for at-rest earth pressure of 0.5 in conjunction with a bulk (drained) unit weight of 20 kN/m^3. Undrained conditions are anticipated (i.e. below the groundwater level). Therefore, the applicable effective (undrained) unit weight of the retained soil can be taken as 13 kN/m^3, where applicable. A hydrostatic pressure should be added to the total static earth pressure when using the effective unit weight.

Two distinct conditions, static and seismic, must be reviewed for design calculations. The parameters for design calculations for the two conditions are presented below.

Static Conditions

The static horizontal earth pressure (p_o) 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 soil (kN/m^3)
H = height of the wall (m)

An additional pressure having a magnitude equal to $K_o \cdot q$ and acting on the entire height of the wall should be added to the above diagram for any surcharge loading, q (kPa), that may be placed at ground surface adjacent to the wall. The surcharge pressure will only be applicable for static analyses and should not be used in conjunction with the seismic loading case.

Actual earth pressures could be higher than the “at-rest” case if care is not exercised during the compaction of the backfill materials to maintain a minimum separation of 0.3 m from the walls with the compaction equipment.

Seismic Conditions

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

The seismic earth force (ΔP_{AE}) can be calculated using $0.375 \cdot a_c \cdot \gamma \cdot H^2/g$ where:

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

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

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

The total earth force (P_{AE}) is considered to act at a height, h (m), from the base of the wall, where:
$h = \{P_o \cdot (H/3) + \Delta P_{AE} \cdot (0.6 \cdot H)\}/P_{AE}$

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

5.7 Rock Anchor Design

The geotechnical design of grouted rock anchors in 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), 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 (4) 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.

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 cementitious 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 limestone ranges between 60 and 120 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 0.3, 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. Based on existing subsoils information, a **Rock Mass Rating (RMR) of 65** was assigned to the bedrock, and Hoek and Brown parameters (**m and s**) were taken as **0.575 and 0.00293**, respectively.

Recommended Rock Anchor Lengths

Rock anchor lengths can be designed based on the required loads. Rock anchor lengths for some typical loads have been calculated and are presented in Table 3. Load specified rock anchor lengths can be provided, if required.
For our calculations the following parameters were used.

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	65 m=0.575 and s=0.00293
Unconfined compressive strength - Limestone bedrock	60 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

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

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	1.2	0.6	1.8	250
	1.9	1	2.9	500
	3	1.5	4.5	1000
125	1.1	0.5	1.6	250
	1.5	0.9	2.4	500
	2.6	1	3.6	1000

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

The geotechnical capacity of each rock anchor should be proof tested at the time of construction. More information on testing can be provided upon request. Compressive strength testing is recommended to be completed for the rock anchor grout. A set of grout cubes should be tested for each day grout is prepared.

5.8 Pavement Structure

For design purposes, it is recommended that the rigid pavement structure for the lowest level of the underground parking structure should 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. The flexible pavement structure presented in Table 5 should be used for at grade access lanes and heavy loading parking areas overlying the podium deck.

Table 4 - Recommended Rigid Pavement Structure - Lower 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 lower underground parking level. The control joints are generally recommended to be located at the center of the column lines and spaced at approximately 24 to 36 times the slab thickness (for example; a 0.15 m thick slab should have control joints spaced between 3.6 and 5.4 m). The joints should be cut between 25 and 30% of the thickness of the concrete floor slab and completed as early as 4 hour after the concrete has been poured during warm temperatures and up to 12 hours during cooler temperatures.

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

Thickness (mm)	Material Description
40	**Wear Course** - Superpave 12.5 Asphaltic Concrete
50	**Binder Course** - Superpave 19.0 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
300	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - OPSS Granular B Type II overlying the Concrete Podium Deck.	

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 98% of the material’s SPMDD using suitable vibratory equipment.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Water Suppression System and Foundation Drainage

The building design will incorporate a water suppression system which will consist of a horizontal concrete hydraulic barrier at the base of the excavation and a waterproofing membrane for the vertical surfaces. The water suppression system will reduce water infiltration volumes at post construction which can then be managed by the building sump pit system.

To manage and control groundwater infiltration over the long term, the following water suppression system is recommended to be installed for the foundation walls and subfloor drainage (refer to Figure 2 for an illustration of a typical Groundwater Suppression System in Appendix 2 of this report):

❏ A waterproofing membrane will be required to lessen the effect of water infiltration for the underground parking levels starting at **2.5 m** below the existing ground surface (which is approximately at the long-term ground water level). The waterproofing membrane will consist of bentonite panels fastened to the shoring system and the grinded bedrock surface. The membrane should extend to the bottom of the excavation at the founding level of the proposed footings and extended horizontally over the approved bedrock surface and/or concrete mud slab(if chosen) a minimum of 600 mm. Consideration can be given to doubling the bentonite panels in isolated areas where groundwater infiltration is observed to be high at the time of construction.

❏ A composite drainage layer will be placed from finished grade to the bottom of the foundation wall. It is recommended that the composite drainage system (such as Delta Drain 6000 or equivalent) extend down to the bottom of the foundation wall. It is expected that 150 mm diameter sleeves placed at 3 m centres be cast in the foundation wall at the footing interface to allow the infiltration of water to flow to an interior perimeter drainage pipe. The perimeter drainage pipe should direct water to the sump pit(s) within the lower basement area.

Subfloor Drainage

Subfloor drainage is recommended to control water infiltration below the lowest underground parking level slab that breaches the horizontal hydraulic barrier. For design purposes it is recommended that a 150 mm diameter perforated pipe be placed in each bay over the concrete hydraulic barrier. The final spacing of the underfloor drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

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.

6.2 Elevator Waterproofing System

It is expected that additional bedrock removal below the building's perimeter strip footings will be required to accommodate the elevator shaft. In addition, it is expected that the elevator shaft will extend below the invert level of the subfloor drainage system and will thus be theoretically designed under submerged conditions. As a result, the following elevator shaft waterproofing options are recommended:

Option 1 - Full Waterproofing System

The horizontally applied Colphene BSW H waterproofing membrane (or approved other) should be placed on an adequately prepared mud slab and extend vertically within the inside of the temporary forms of the elevator raft slab. Once the concrete raft slab and elevator shaft sidewalls are poured in place, it is recommended that a waterproofing membrane, such as Colphene Torch'n Stick (or approved other) should be applied to the exterior of the elevator pit sidewalls. The Colphene Torch'n Stick waterproofing membrane should extend over the vertical portion of the previously applied Colphene BSW H waterproofing membrane installed on the concrete raft slab in accordance with the manufacturers specifications. As a secondary defence, a continuous PVC waterstop such as Southern waterstop 14RCB or equivalent should be installed within the concrete raft slab below the elevator pit sidewalls.

A protection board should be placed over the waterproofing membrane to protect the waterproofing membrane from damage during backfilling operations. The area between the elevator pit and bedrock excavation face should be in-filled with lean concrete, OPSS Granular B Type 2 or Granular A crushed stone. Refer to Figure 3 - Option 1 - Elevator Waterproofing Detail in Appendix 2 of this report for specific details of the elevator waterproofing.

Option 2 - Partial Waterproofing System

As a result of the size and configuration of the proposed raft slab of the elevator shaft, the following economical waterproofing option can be considered. This option consists of omitting the previously recommended horizontally applied Colphene BSW H waterproofing membrane wrapped around the bottom and sidewalls of the concrete raft slab of the elevator shaft as detailed above in Option 1.

Once the concrete raft slab and elevator pit sidewalls are poured in place, it is recommended that a waterproofing membrane, such as Colphene Torch’n Stick (or approved other) should be applied to the exterior of the elevator pit sidewalls and horizontally over the elevator raft slab in accordance to the manufacturers specifications. As a secondary defence, a continuous PVC waterstop such as Southern waterstop 14RCB or equivalent should be installed within the concrete raft slab below the elevator pit sidewalls.

A protection board should be placed over the waterproofing membrane to protect the waterproofing membrane from damage during backfilling operations. The area between the elevator shaft and bedrock excavation face should be in-filled with lean concrete, OPSS Granular B Type 2 or Granular A crushed stone. Refer to Figure 4 - Option 2 - Elevator Waterproofing Detail in Appendix 2 of this report for specific details of the elevator waterproofing

6.3 Protection of Footings Against Frost Action

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

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

6.4 Excavation Side Slopes and Temporary Shoring

The side slopes of the shallow excavations anticipated at this site should either be cut back at acceptable slopes or be retained by shoring systems from the start of the excavation until the structure is backfilled.

Unsupported Excavations

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

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.

Rock 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 fractures in the bedrock are conducive to the failure of the bedrock surface.

The requirements for horizontal rock anchors and bedrock stabilization measures will be evaluated during the excavation program and determined by the geotechnical consultant at the time of construction.

Temporary Shoring

Temporary shoring may be required to support the overburden soil 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 will not negatively impact the shoring system or soils supported by the system. Any changes to the approved shoring design system should be reported immediately to the owner’s structural designer prior to implementation.

For design purposes, the temporary system may consist of soldier pile and lagging system or interlocking steel sheet piling. Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be added to the earth pressures described below. These systems can be cantilevered, anchored or braced. Generally, 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 be adequately supported to resist toe failure by means of rock bolts or extending the piles into the bedrock through pre-augered holes if a soldier pile and lagging system is used.

The 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.

The anchor derives its capacity from the bonded portion, or fixed anchor length, at the base of the anchor. An unbonded portion, or free anchor length, is also usually provided between the rock surface and the start of the bonded length. Because the depth at which the apex 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 capacity, than one where the bonded length was just the bottom part of the overall anchor.

The design of the rock anchors for temporary shoring can be based on the values provided in Subsection 5.7 of the present report.

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

Table 6 - Soil Parameters for Shoring System Design

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 (γ), kN/m^3	20
Submerged Unit Weight (γ), kN/m^3	13

Soldier Pile and Lagging System

The active earth pressure acting on a soldier pile and lagging shoring system can be calculated using a rectangular earth pressure distribution with a maximum pressure of 0.65 K γ H for strutted or anchored shoring or a triangular earth pressure distribution with a maximum value of K γ H for a cantilever shoring system. H is the height of the excavation.

The active earth pressure should be used where wall movements are permissible while the at-rest pressure should be used if no movement is permissible.

The total unit weight should be used above the groundwater level while the submerged unit weight should be used below the groundwater level.

The hydrostatic groundwater pressure should be added to the earth pressure distribution wherever the submerged unit weights are used for earth pressure calculations should the level on the groundwater not be lowered below the bottom of the excavation. If the groundwater level is lowered, the total unit weight for the soil should be used full weight, with no hydrostatic groundwater pressure component.

6.5 Pipe Bedding and Backfill

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

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.

6.6 Groundwater Control

Groundwater Infiltration

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.

The rate of flow of groundwater into the excavation through the overburden and bedrock should be moderate for the expected subsurface conditions at this site. It is anticipated that pumping from open sumps will be sufficient to control the groundwater influx through the sides of the excavations.

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 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.

Adverse Effects from Dewatering on Adjacent Structures

Based on the subsurface conditions encountered at the subject site, it is anticipated that the adjacent structures are founded on bedrock or the dense glacial till deposit. Therefore, in our opinion, no adverse effects from short term and long term dewatering are expected for surrounding structures.

6.7 Winter Construction

Precautions must be taken if winter construction is considered for this project. The subsoil conditions at this site mostly consist of frost susceptible materials. In 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 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.

7.0 Recommendations

A materials testing and observation services program is a requirement for the provided foundation design data to be applicable. The following aspects of the program should be performed by the geotechnical consultant:

- ☐ Review of the geotechnical aspects of the excavating contractor's shoring design, prior to construction.
- ☐ Observe and approve the installation of the water suppression system.
- ☐ Review proposed waterproofing and foundation drainage design and requirements.
- ☐ Review 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 used.
- ☐ Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- ☐ Observation of all subgrades prior to backfilling.
- ☐ Field density tests to determine the level of compaction achieved.
- ☐ Sampling and testing of the bituminous concrete including mix design reviews.

A report confirming that these works have been conducted in general accordance with our recommendations could be issued, upon request, following the completion of a satisfactory materials testing and observation program by the geotechnical consultant.

8.0 Statement of Limitations

The recommendations provided in this report are in accordance with our present understanding of the project. We request permission to review our 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, 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 Tamarack (Norman) Corporation 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.

Scott Dennis

LICENSED PROFESSIONAL ENGINEER
April. 7-2021
S. S. DENNIS
100519516
Scott Dennis
PROVINCE OF ONTARIO

Scott S. Dennis, P.Eng.

David J. Gilbert, P.Eng.

Report Distribution:

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☐ 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

Phase I - II Environmental Site Assessment
95, 97 and 99 Norman Street
Ottawa, Ontario

DATUM Referenced to a Geodetic Datum

REMARKS

BORINGS BY CME 75 Power Auger

DATE 2011 May 31

FILE NO. PE2327

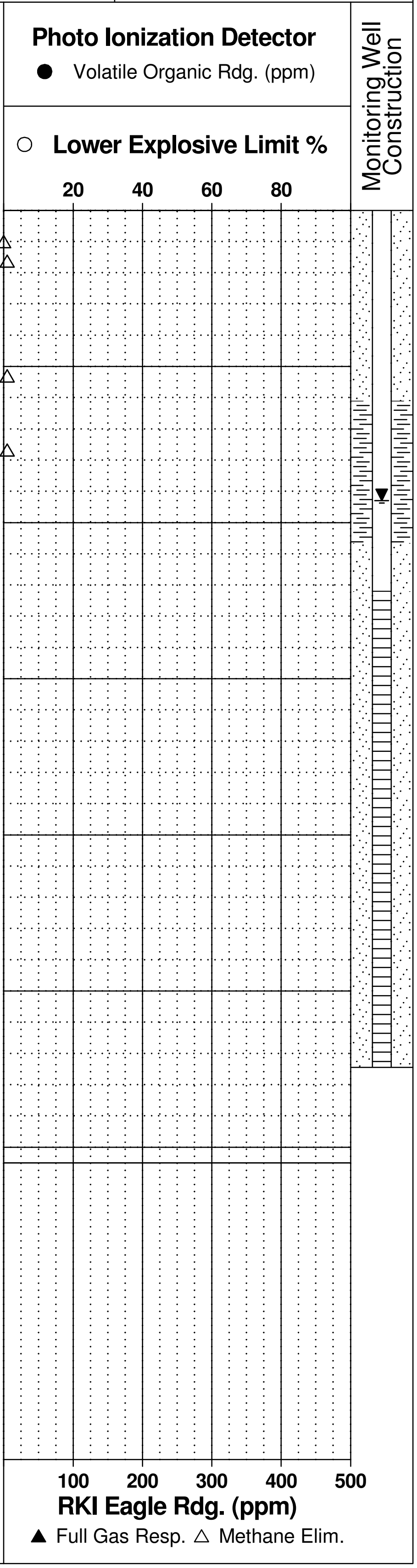
HOLE NO. BH 1

SOIL DESCRIPTION	Bottom depth (as printed)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
FILL: Sand and gravel	0.30	AU	1		
PEAT		AU	2		
	1.27	SS	3	100	9
Loose to dense, grey **SILTY SAND** with gravel and cobbles	1.60	SS	4		50+
BEDROCK: Grey limestone		RC	1	100	74
		RC	2	100	100
		RC	3	100	90
	6.10	RC	4	100	100
End of Borehole					

(GWL @ 1.37 m depth- June 6, 2011)

(GWL @ 1.86 m depth - Sept 5, 2012)

DEPTH (m)	ELEV. (m)
0	61.51
1	60.51
2	59.51
3	58.51
4	57.51
5	56.51
6	55.51



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SOIL PROFILE AND TEST DATA

Phase I - II Environmental Site Assessment
95, 97 and 99 Norman Street
Ottawa, Ontario

DATUM Referenced to a Geodetic Datum

FILE NO. PE2327

REMARKS

HOLE NO. BH 2

BORINGS BY CME 75 Power Auger

DATE 2011 May 31

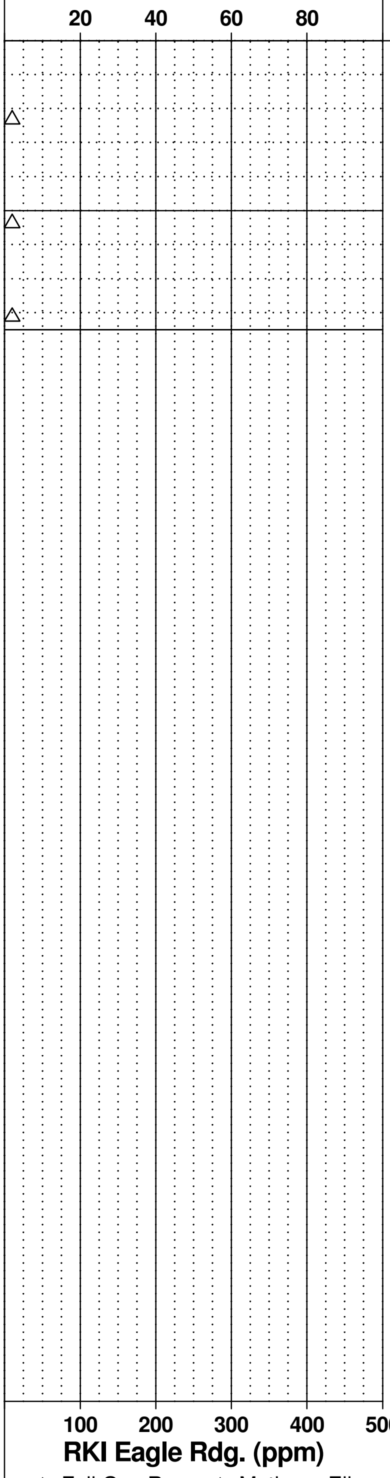
SOIL DESCRIPTION		SAMPLE				DEPTH (m)	ELEV. (m)
GROUND SURFACE		TYPE	NUMBER	% RECOVERY	N VALUE or RQD		
						0	61.51
Asphaltic concrete	0.15						
FILL: Crushed stone	0.30	AU	1				
FILL: Brown silty sand with gravel		AU	2				
		SS	3	42	4	1	60.51
	1.45						
Dense, brown **SILTY SAND** with gravel and cobbles	1.70	SS	4	73	50+		
End of Borehole Practical refusal to augering @ 1.70m depth							

Photo Ionization Detector

● Volatile Organic Rdg. (ppm)

○ **Lower Explosive Limit %**

Monitoring Well Construction



RKI Eagle Rdg. (ppm)

▲ Full Gas Resp. △ Methane Elim.

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SOIL PROFILE AND TEST DATA

Phase I - II Environmental Site Assessment
95, 97 and 99 Norman Street
Ottawa, Ontario

DATUM Referenced to a Geodetic Datum

REMARKS

BORINGS BY CME 75 Power Auger

DATE 2011 May 31

FILE NO. PE2327

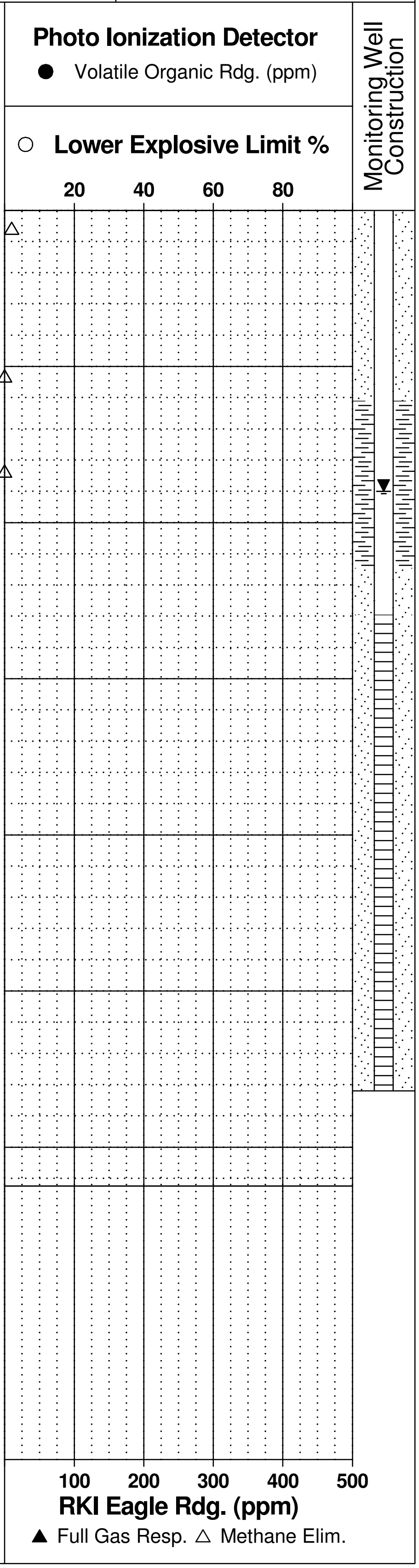
HOLE NO. BH 3

SOIL DESCRIPTION	SAMPLE				DEPTH (m)	ELEV. (m)
	TYPE	NUMBER	% RECOVERY	N VALUE or RQD		
GROUND SURFACE					0	61.55
25mm Asphaltic concrete over crushed stone 0.18	AU	1				
FILL: Brown silty sand with gravel 1.30	SS	2	92	6	1	60.55
PEAT 1.45						
Brown **SILTY SAND** with peat, trace marl 1.83	SS	3	25	50+		
					2	59.55
BEDROCK: Grey limestone	RC	1	100	43		
					3	58.55
	RC	2	100	93		
					4	57.55
	RC	3	100		5	56.55
	RC	4	100	100	6	55.55
6.25						

End of Borehole

(GWL @ 1.74 m depth - June 6, 2011)

(GWL @ 1.86 m depth - Sept 5, 2012)



SOIL PROFILE AND TEST DATA

Phase I - II Environmental Site Assessment
95, 97 and 99 Norman Street
Ottawa, Ontario

DATUM Referenced to a Geodetic Datum

REMARKS

BORINGS BY CME 75 Power Auger

DATE 2011 May 31

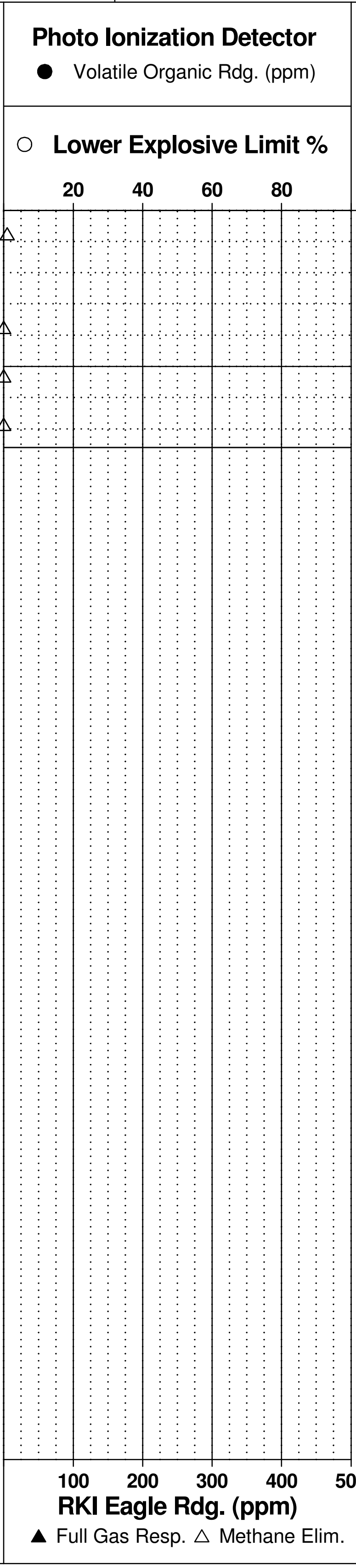
FILE NO. PE2327

HOLE NO. BH 4

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD		
GROUND SURFACE						0	61.53
Asphaltic concrete 0.05							
FILL: Crushed stone with sand 0.43		AU	1				
PEAT/TOPSOIL		AU	3				
		SS	2	0	4	1	60.53
1.52		AU	4				

End of Borehole

Practical refusal to augering @ 1.52m depth



Monitoring Well Construction

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building - 101 Norman Street
Ottawa, Ontario

DATUM Referenced to a Geodetic Datum

FILE NO. PG2760

REMARKS

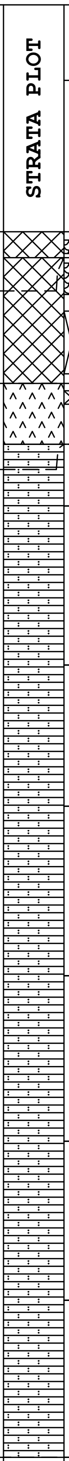
HOLE NO. BH 1-12

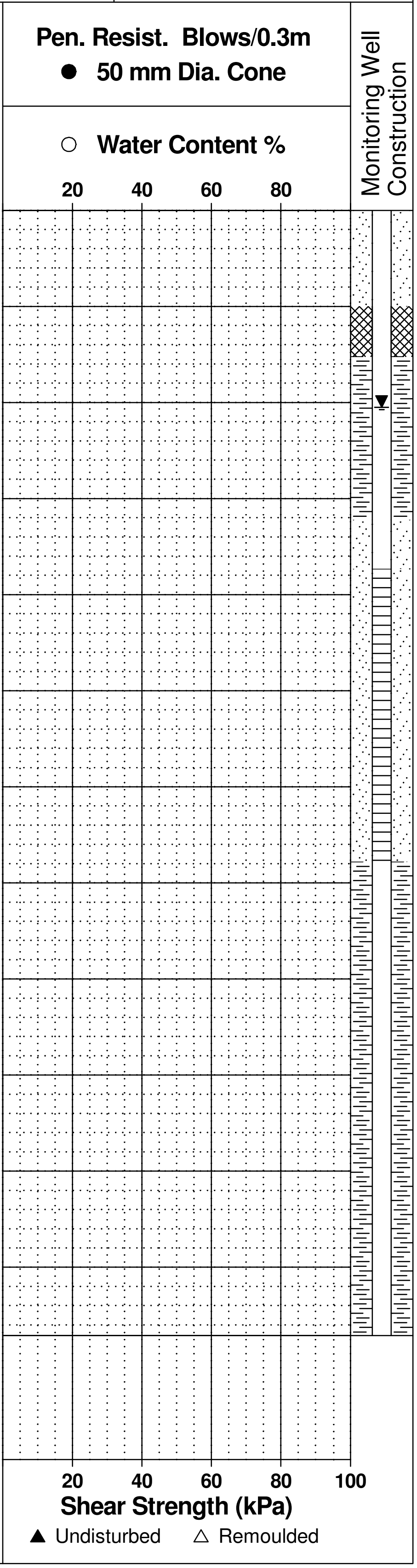
BORINGS BY CME 55 Power Auger

DATE 2012 August 29

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	61.59
50mm Asphaltic concrete over crushed stone 0.25	AU	1				
	AU	2				
FILL: Brown silty sand with gravel, cobbles, brick 1.45	SS	3	42	30	1	60.59
GLACIAL TILL: Brown silty sand with gravel, cobbles, boulders, trace clay 2.03	SS	4		50+		
					2	59.59
	RC	1	100	83		
					3	58.59
	RC	2	100	80		
					4	57.59
BEDROCK: Grey limestone						
	RC	3	100	86		
					5	56.59
					6	55.59
	RC	4	100	76		
					7	54.59
	RC	5	100	82		
					8	53.59
					9	52.59
BEDROCK: Grey limestone	RC	6	100	100		
					10	51.59
	RC	7	100	86		
					11	50.59
11.71 End of Borehole						

(GWL @ 2.05m-Sept. 5, 2012)





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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building - 101 Norman Street
Ottawa, Ontario

DATUM Referenced to a Geodetic Datum

REMARKS

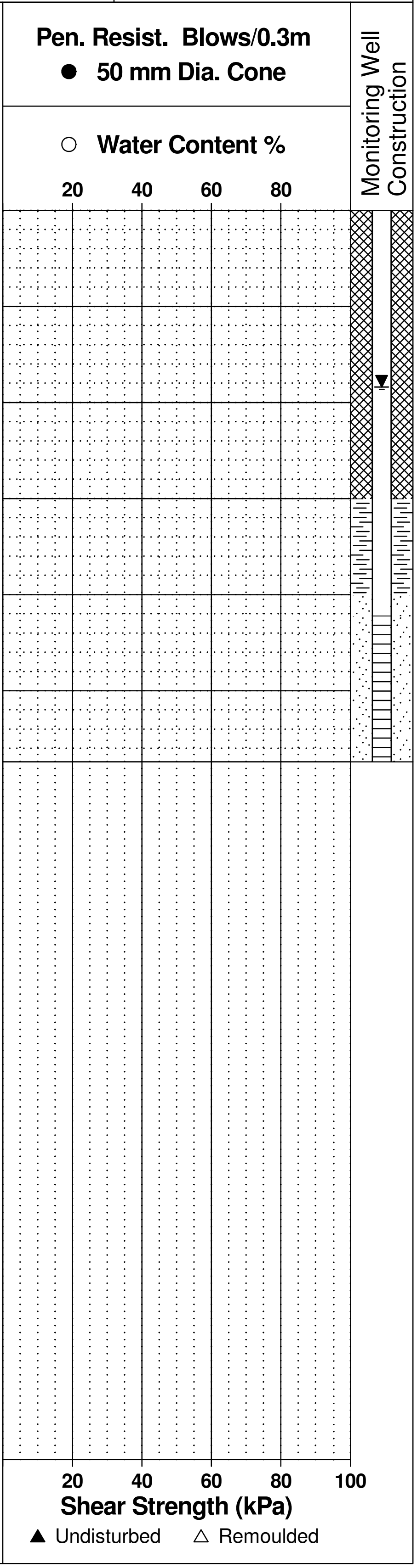
BORINGS BY CME 55 Power Auger

DATE 2012 August 29

FILE NO. PG2760

HOLE NO. BH 2-12

SOIL DESCRIPTION		SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	61.42
50mm Asphaltic concrete over crushed stone	0.30	AU	1				
FILL: Brown silty sand with gravel, cobbles		AU	2				
		SS	3	42	10	1	60.42
- topsoil with trace wood chips from 1.2 to 1.5m depth	1.58	SS	4	100	50+		
BEDROCK: Grey limestone		RC	1	95	64	2	59.42
						3	58.42
		RC	2	100	85	4	57.42
	5.74	RC	3	100	83	5	56.42
End of Borehole							
(GWL @ 1.84m-Sept. 5, 2012)							



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building - 101 Norman Street
Ottawa, Ontario

DATUM Referenced to a Geodetic Datum

FILE NO. **PG2760**

REMARKS

HOLE NO. **BH 3-12**

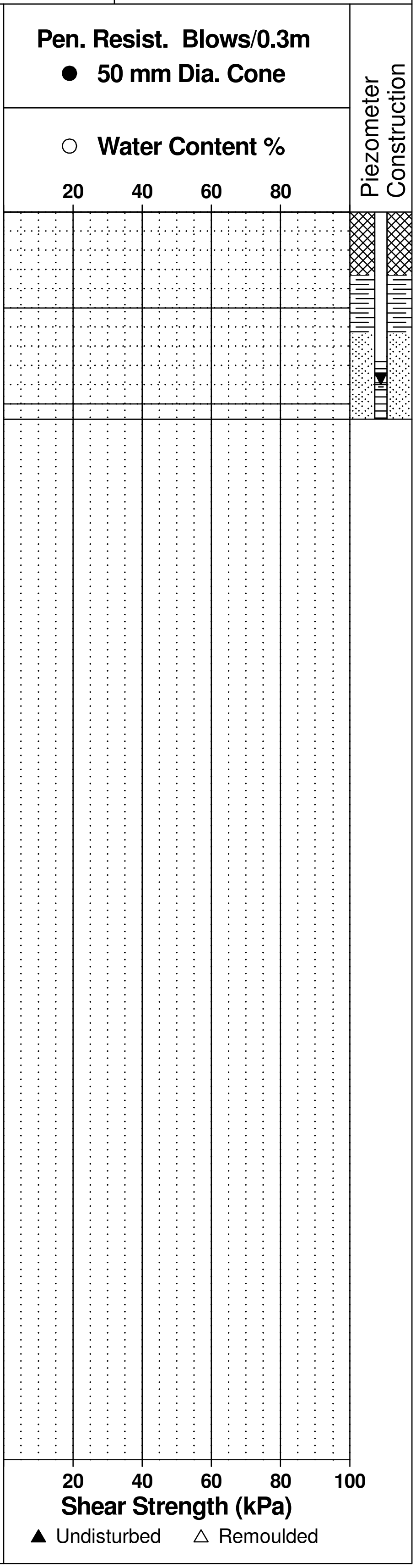
BORINGS BY CME 55 Power Auger

DATE 2012 August 30

SOIL DESCRIPTION	Strata depth (m)	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	61.55
25mm Asphaltic concrete over crushed stone	0.25	AU	1				
FILL: Brown silty sand with clay, trace gravel and brick		AU	2				
		SS	3	57	50+	1	60.55
	1.60						
GLACIAL TILL: Brown sitly sand with gravel, cobbles and boulders, trace clay	2.16	SS	4	63	50+	2	59.55
End of Borehole							

Practical refusal to augering at 2.16 m depth

(GWL @ 1.80m-Sept. 5, 2012)



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building - 101 Norman Street
Ottawa, Ontario

DATUM Referenced to a Geodetic Datum

REMARKS

BORINGS BY CME 55 Power Auger

DATE 2012 August 30

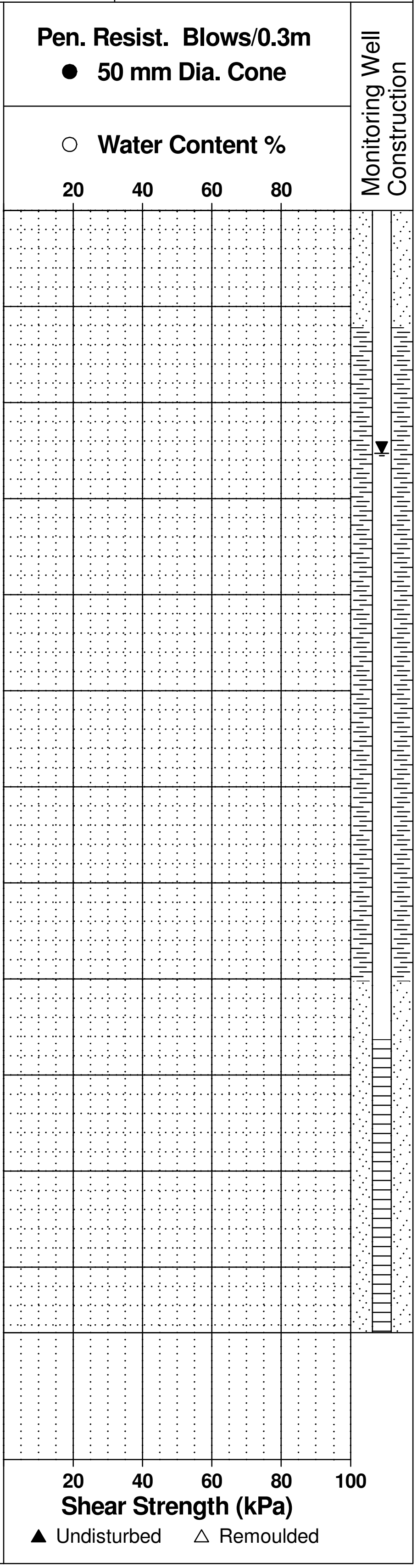
FILE NO. **PG2760**

HOLE NO. **BH 4-12**

SOIL DESCRIPTION		
GROUND SURFACE		
25mm Asphaltic concrete over crushed stone	0.30	
FILL: Brown silty sand, trace gravel	0.69	
Compact, brown **SILTY SAND,** trace gravel	1.07	
GLACIAL TILL: Brown silty sand with gravel, cobbles, boulders, trace clay	1.60	
BEDROCK: Grey limestone		
BEDROCK: Grey limestone	11.68	
End of Borehole		
(GWL @ 2.53m-Sept. 5, 2012)		

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
AU	2		
SS	3	58	18
SS	4	100	50+
RC	1	91	50
RC	2	100	100
RC	3	100	100
RC	4	100	100
RC	5	100	100
RC	6	100	100
RC	7	100	100

DEPTH (m)	ELEV. (m)
0	61.53
1	60.53
2	59.53
3	58.53
4	57.53
5	56.53
6	55.53
7	54.53
8	53.53
9	52.53
10	51.53
11	50.53



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building - 101 Norman Street
Ottawa, Ontario

DATUM Referenced to a Geodetic Datum

FILE NO. **PG2760**

REMARKS

HOLE NO. **BH 1-21**

BORINGS BY CME-55 Low Clearance Drill **DATE** 2021 February 3

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	61.60
FILL: Brown silty sand trace gravel, crushed stone and organics 0.13	AU	1				
FILL: Brown silty sand trace gravel and crushed stone	SS	2		10	1	60.60
	SS	3		2		
2.10					2	59.60
FILL: Brown silty sand trace gravel 2.41	SS	4		+50		
BEDROCK Excellent quality Grey Limestone	RC	1	100	85		
					3	58.60
	RC	2	100	100		
					4	57.60
					5	56.60
	RC	3	100	97		
					6	55.60
	RC	4	100	91		
7.06					7	54.60
End of Borehole (GWL @ 1.78 m depth - Feb 9, 2021)						

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

○ **Water Content %**

20 40 60 80

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

Monitoring Well Construction

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building - 101 Norman Street
Ottawa, Ontario

DATUM Referenced to a Geodetic Datum

REMARKS

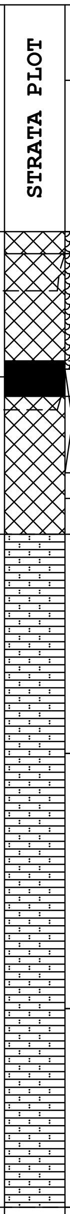
BORINGS BY CME-55 Low Clearance Drill

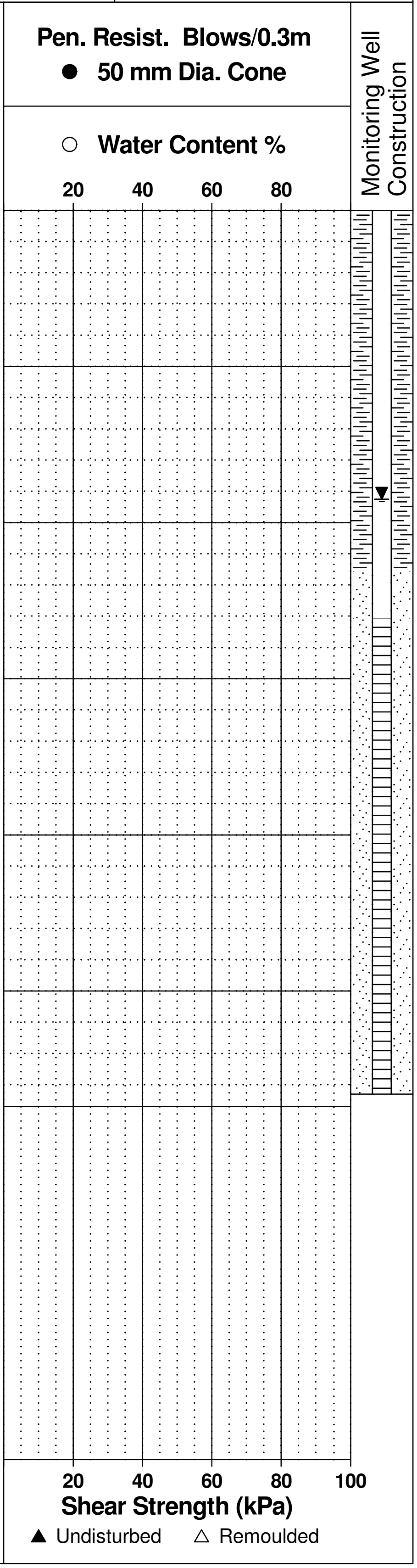
DATE 2021 February 3

FILE NO. PG2760

HOLE NO. BH 2-21

SOIL DESCRIPTION		SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	61.88
FILL: Brown silty sand trace organics and topsoil	0.13	AU	1				
FILL: Brown silty clay, trace sand, gravel, crushed stone, organics and topsoil	0.76	AU	2				
TOPSOIL	0.97					1	60.88
FILL: Grey silty sand trace clay, gravel, crushed stone, organics, topsoil and occasional boulders	1.78	SS	3	54	4		
BEDROCK: Good to Excellent quality Grey Limestone		RC	1	100	73	2	59.88
						3	58.88
		RC	2	100	87	4	57.88
		RC	3	100	100	5	56.88
	5.74						
End of Borehole							
(GWL 1.85 m depth - Feb 9, 2021)							





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)

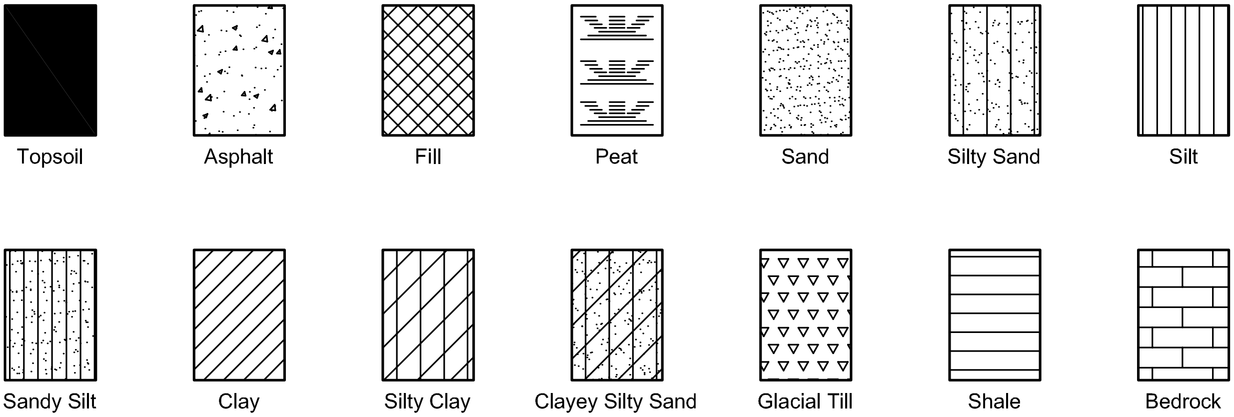
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

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.

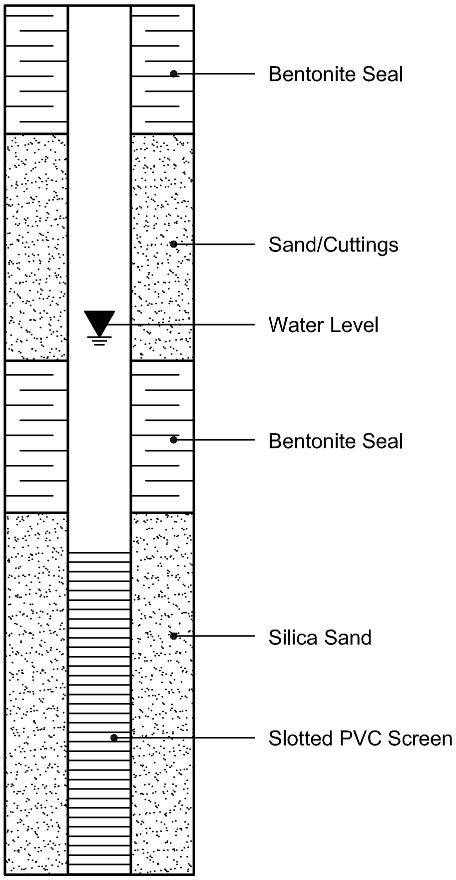
SYMBOLS AND TERMS (continued)

STRATA PLOT

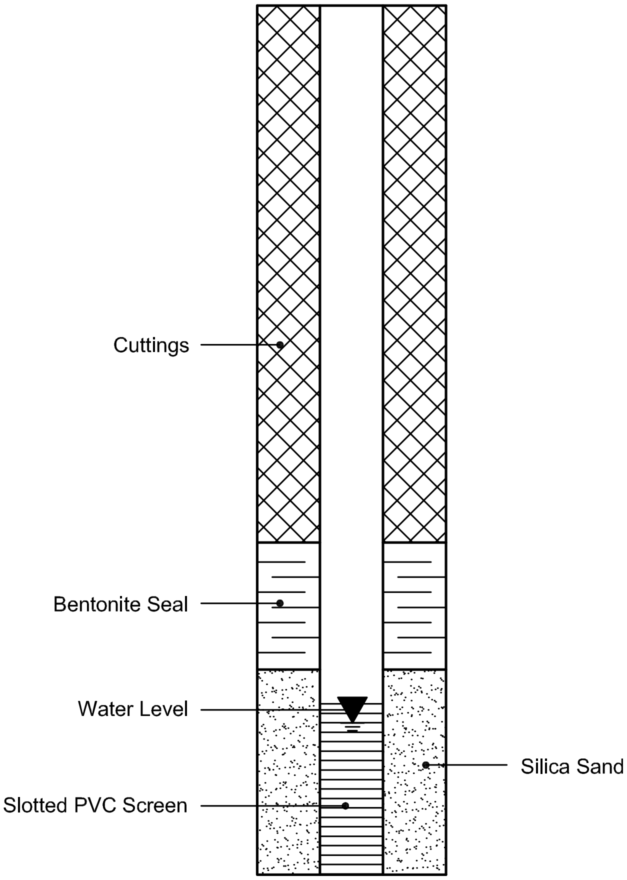


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURE 2 - GROUNDWATER SUPPRESSION SYSTEM

FIGURE 3 - OPTION 1 - ELEVATOR WATERPROOFING DETAIL

FIGURE 4 - OPTION 2 - ELEVATOR WATERPROOFING DETAIL

FIGURE 5 & 6 - SEISMIC SHEAR WAVE VELOCITY PROFILES

DRAWING PG2760-1 - TEST HOLE LOCATION PLAN

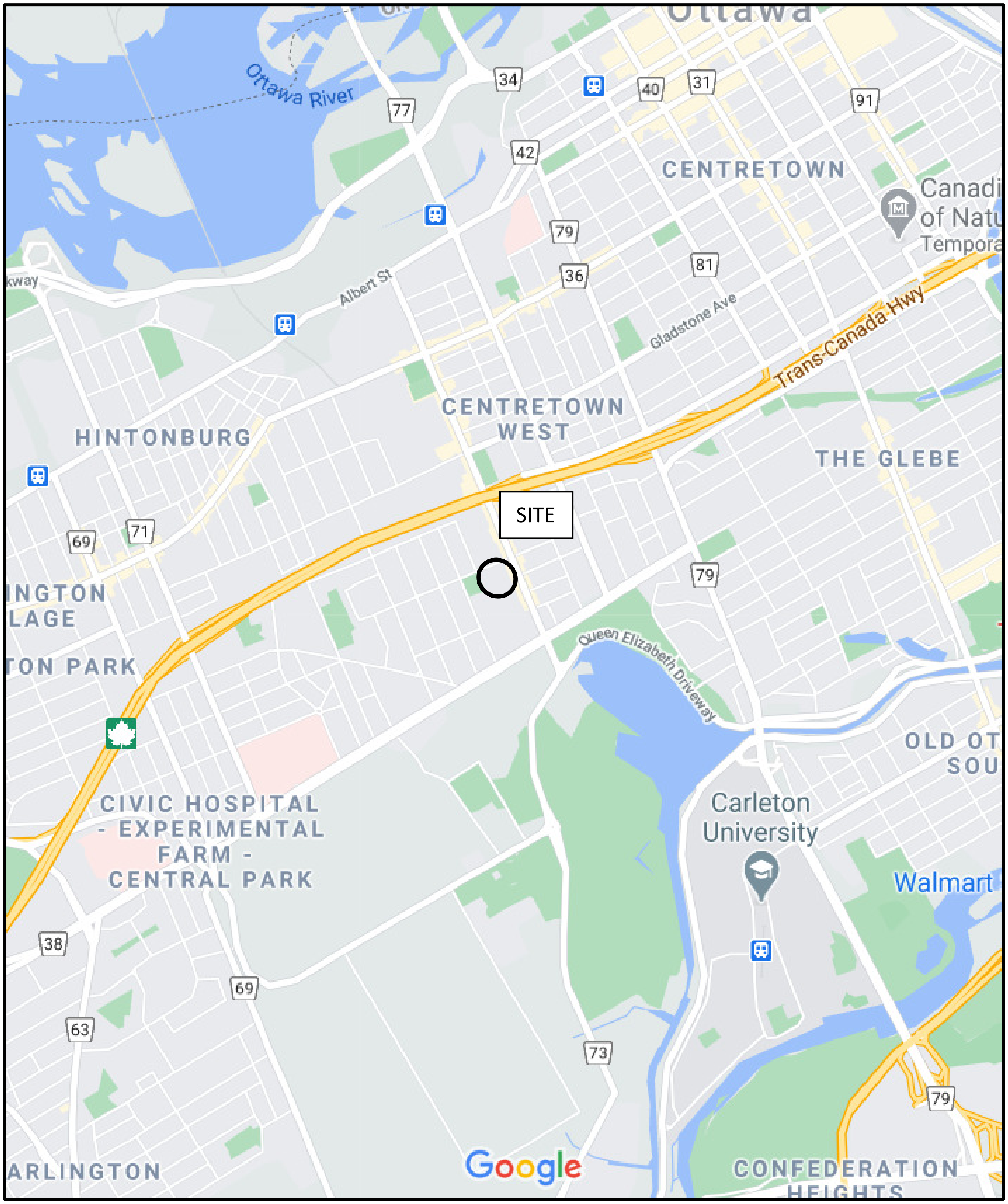


FIGURE 1

KEY PLAN

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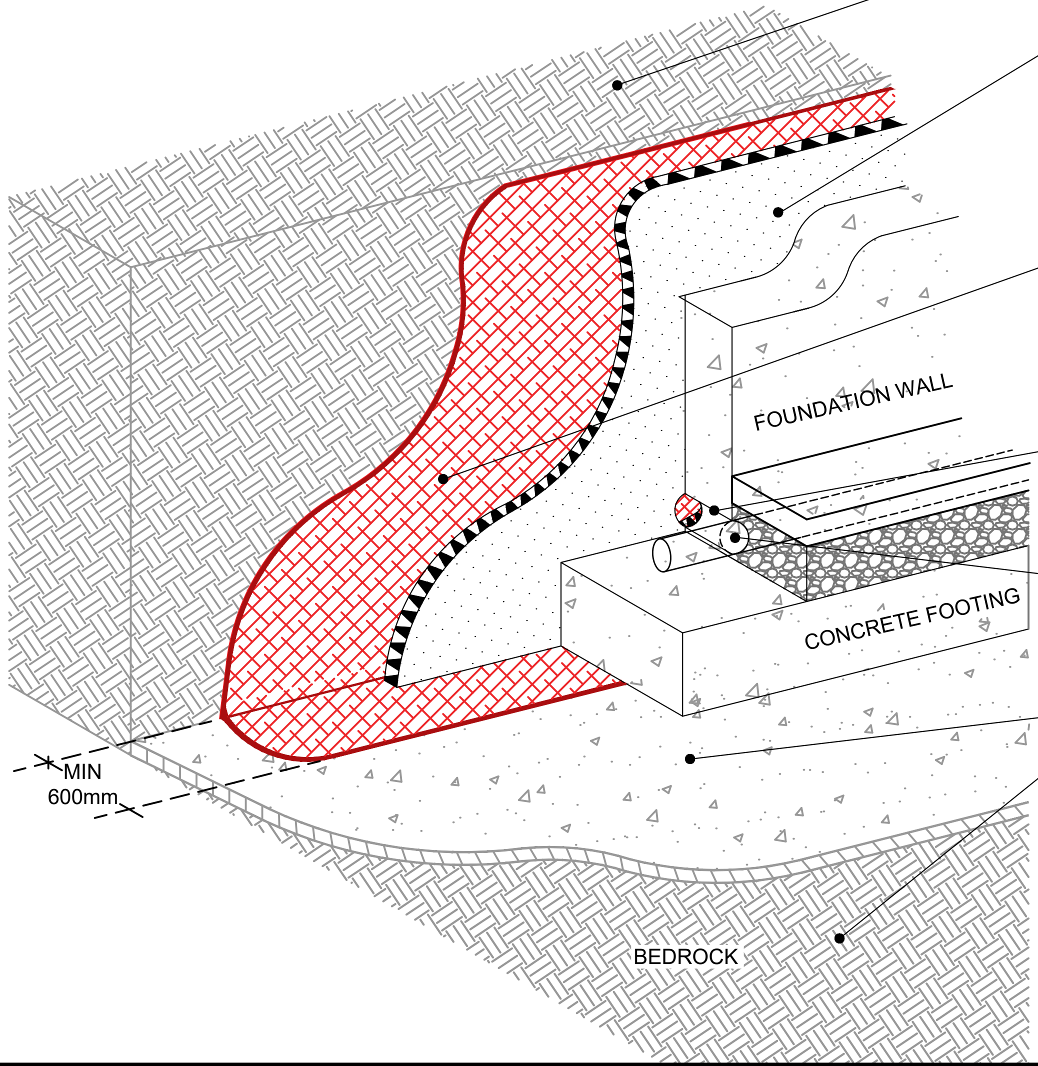

SMOOTH AND FLAT BEDROCK SURFACE ADEQUATELY PREPARED FOR THE PLACEMENT OF THE WATERPROOFING MEMBRANE

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 BLUESKIN WP200 AND BLUESKIN PRIMER OR 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 2.5m BELOW FINISHED GRADE

150mm DIAMETER SOLID PVC SLEEVE PLACED THROUGH THE BASE OF THE EXTERIOR FOUNDATION WALL AT APPROXIMATELY 3.0m 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.

MINIMUM 100mm THICK, 15 MPa CONCRETE MUD SLAB (OPTIONAL)

CLEAN SURFACE SOUNDED BEDROCK APPROVED BY THE GEOTECHNICAL CONSULTANT

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TAMARACK (NORMAN) CORPORATION

PROPOSED MULTI-STOREY BUILDING
93 NORMAND STREET

OTTAWA, ONTARIO

Title:

GROUNDWATER SUPPRESSION SYSTEM

Scale: NTS
Date: 01/2021
Drawn by: RCG
Report No.: PG2760-1
Checked by: RG
Drawing No.: FIG.2
Approved by: DG
Revision No.:

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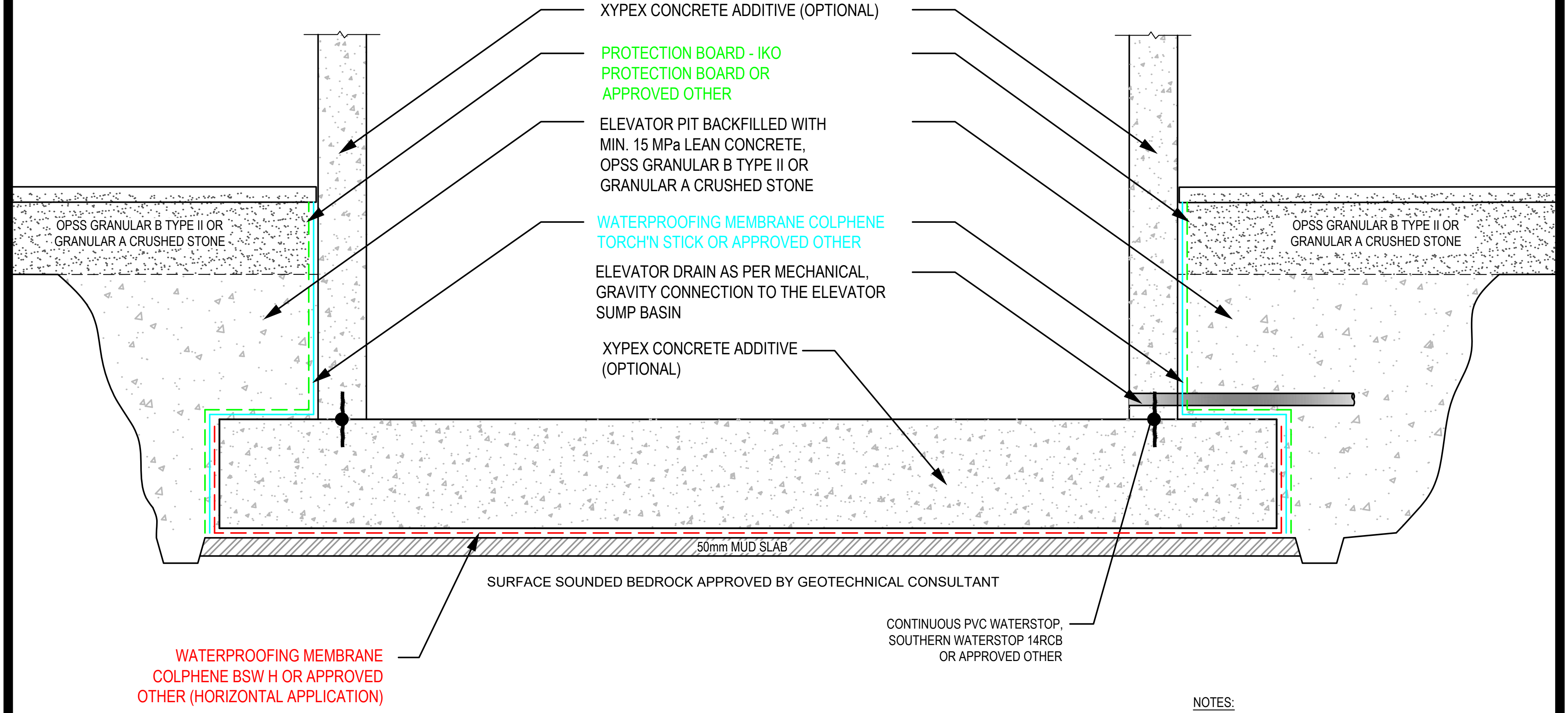

NOTES:

1. IT IS RECOMMENDED THAT PERIODIC INSPECTIONS BE COMPLETED BY THE GEOTECHNICAL CONSULTANT AT THE TIME OF CONSTRUCTION DURING THE INSTALLATION OF THE ELEVATOR WATERPROOFING MEMBRANE(S).

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TAMARACK (NORMAN) CORPORATION

PROPOSED MULTI-STOREY BUILDING
93 NORMAN STREET
OTTAWA, ONTARIO

Title:

OPTION 1 - ELEVATOR WATERPROOFING DETAIL

Scale: N.T.S.
Date: 01/2021
Drawn by: MPG
Report No.: PG2760-1
Checked by: RG
Drawing No.: FIG. 3
Approved by: RG
Revision No.:

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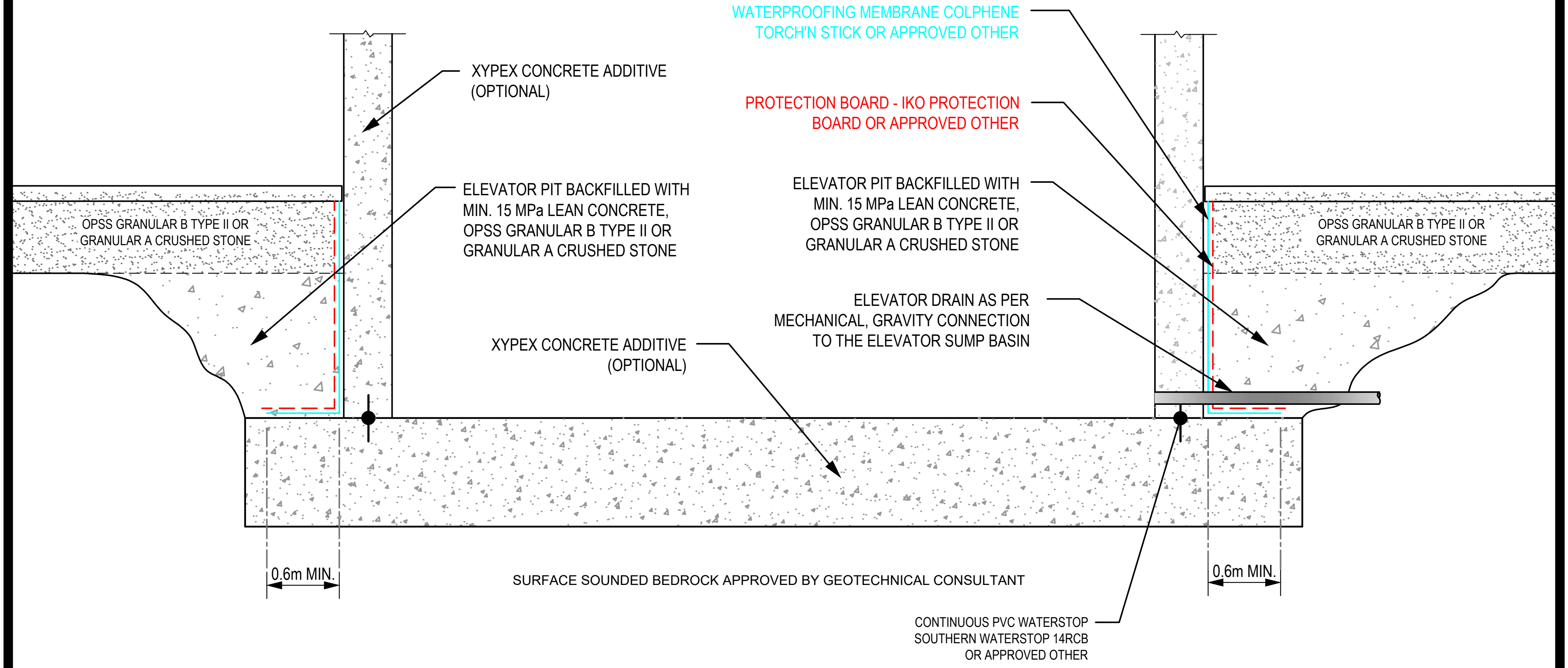

NOTES:

1. IT IS RECOMMENDED THAT PERIODIC INSPECTIONS BE COMPLETED BY THE GEOTECHNICAL CONSULTANT AT THE TIME OF CONSTRUCTION DURING THE INSTALLATION OF THE ELEVATOR WATERPROOFING MEMBRANE(S).

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TAMARACK (NORMAN) CORPORATION

PROPOSED MULTI-STOREY BUILDING
93 NORMAN STREET
OTTAWA, ONTARIO

Title:

OPTION 2 - ELEVATOR WATERPROOFING DETAIL

Scale: N.T.S.	**Date:** 01/2021
Drawn by: MPG	**Report No.:** PG2760-1
Checked by: RG	**Drawing No.:** FIG. 4
Approved by: RG	**Revision No.:**

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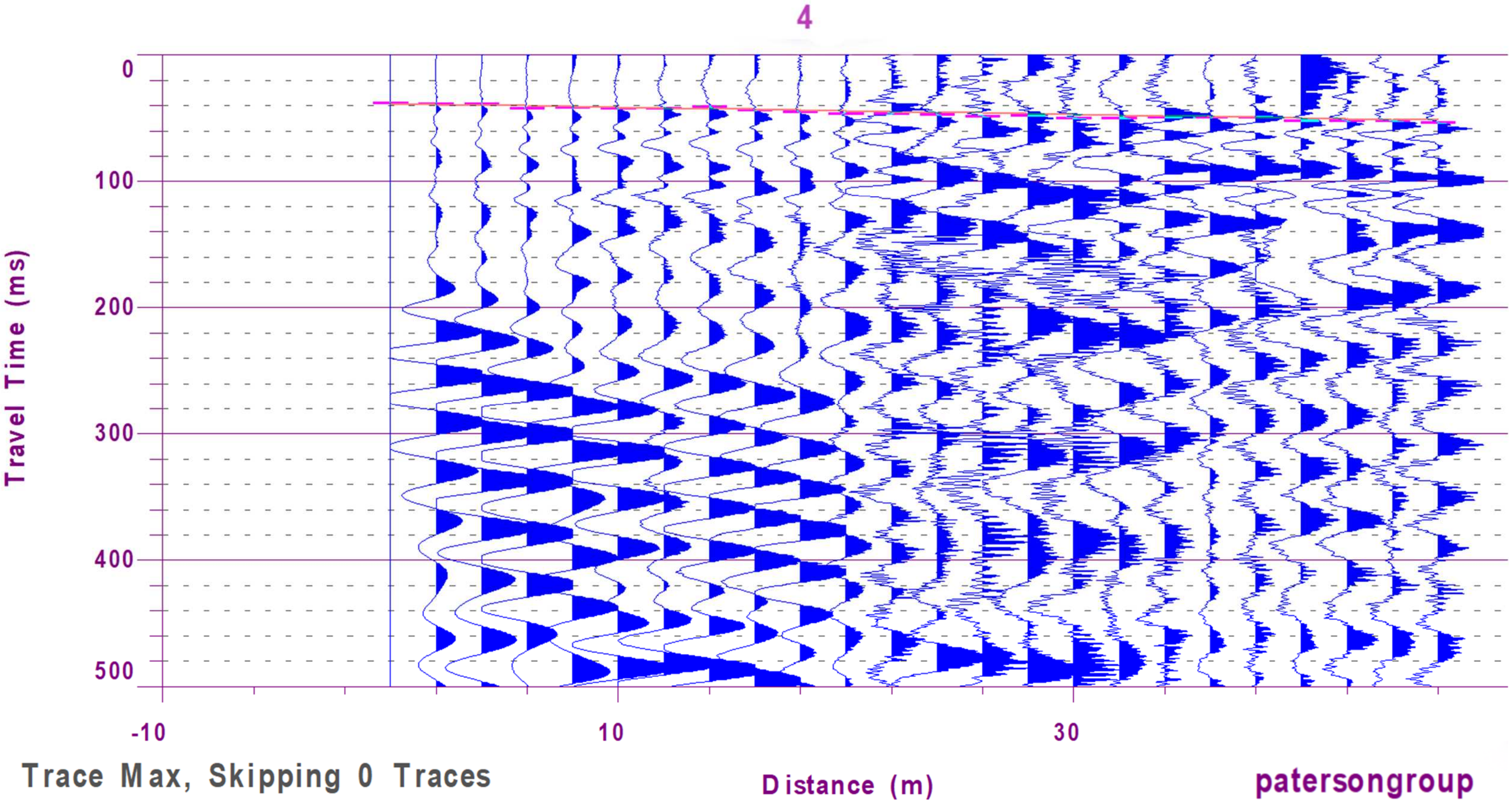


Figure 5 – Shear Wave Velocity Profile at Shot Location -23 m

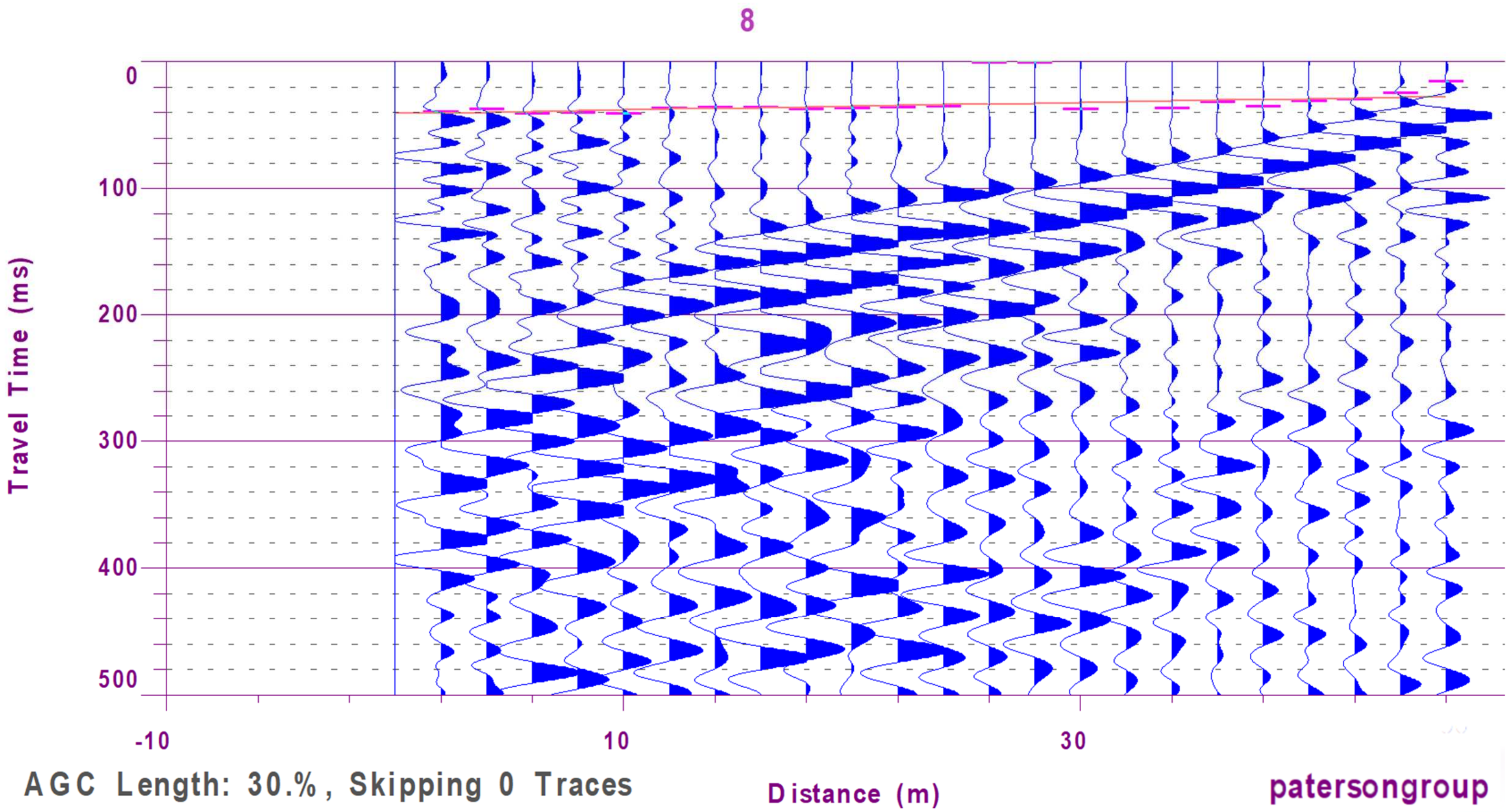
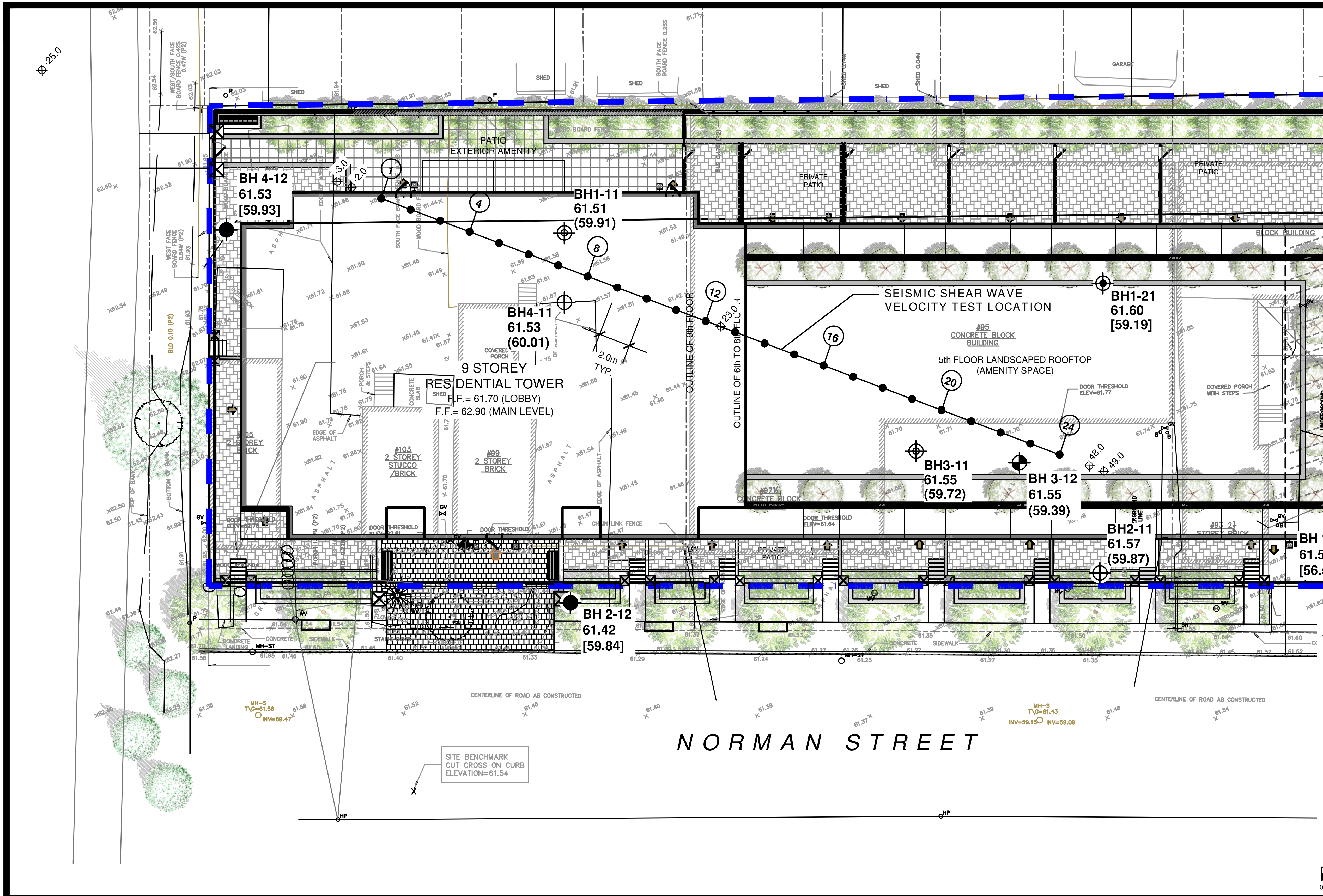


Figure 6 – Shear Wave Velocity Profile at Shot Location +48 m



SHEAR WAVE VELOCITY TEST LEGEND:

●–●–●	GEOPHONE LOCATIONS
③	GEOPHONE NUMBER
+ +10.0	SHOT LOCATION

LEGEND

⊕	MONITORING WELL LOCATION (PATERSON 2021)
⊕	BOREHOLE LOCATION (PATERSON, 2012)
●	MONITORING WELL LOCATION (PATERSON, 2011)
⊕	BOREHOLE LOCATION (PATERSON, 2011)
⊕	MONITORING WELL LOCATION (PATERSON, 2011)
99.15	GROUND SURFACE ELEVATION(m)
(96.95)	PRACTICAL REFUSAL TO AUGERING ELEVATION(m)
[97.12]	BEDROCK SURFACE ELEVATION(m)

BASE PLAN PROVIDED BY TAGGART INVESTMENTS.

ELEVATIONS AT TEST HOLES ARE REFERENCED TO A GEODETIC DATUM

0 1 2 3 4 5 10 15m

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NO.	REVISIONS	DATE	INITIAL
2	SEISMIC SHEAR WAVE VELOCITY TEST ADDED	06/04/2021	FC
1	UPDATED TO LATEST CONCEPTUAL PLAN, 2021 BOREHOLES ADDED TO PLAN	05/02/2021	RG

TAMARACK (NORMAN) CORPORATION

GEOTECHNICAL INVESTIGATION

OTTAWA, PROP. MULTI-STOREY BUILDING - 93 NORMAN STREET ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:250
Date: 01/2021
Drawn by: RCG
Report No.: PG2760-1
Checked by: RG
Dwg. No.: PG2760-1
Approved by: SD
Revision No.: 2

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