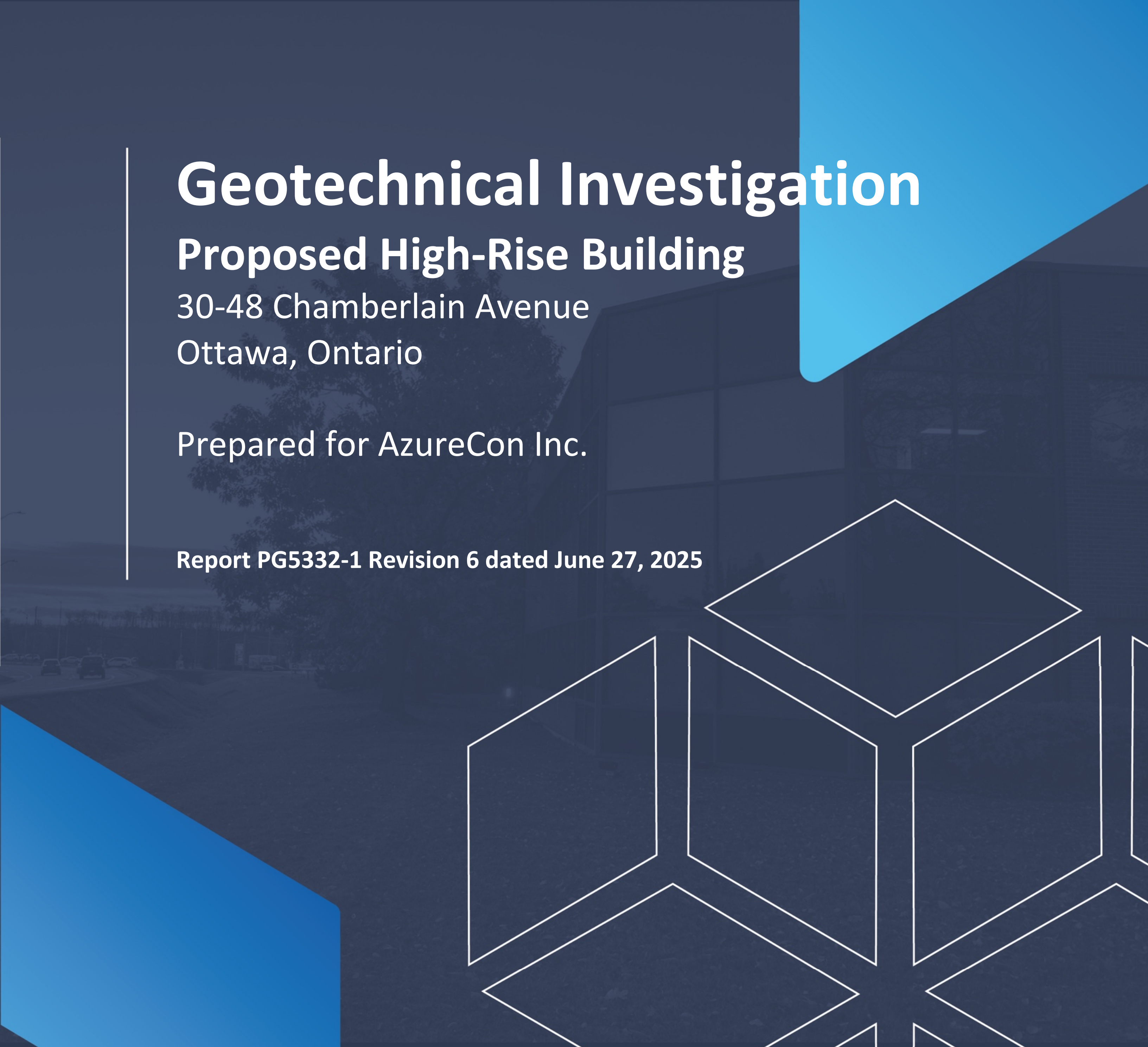
Geotechnical Investigation

Proposed High-Rise Building

30-48 Chamberlain Avenue
Ottawa, Ontario

Prepared for AzureCon Inc.

Report PG5332-1 Revision 6 dated June 27, 2025



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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by AzureCon Inc. to conduct a geotechnical investigation for the proposed high-rise building to be located at 30-48 Chamberlain Avenue in the City of Ottawa, Ontario (refer to Figure 1 – Key Plan in Appendix 2 of this report for the general site location).

The objectives of the geotechnical investigation were to:

☐ Determine the subsurface soil and groundwater conditions based on boreholes completed within the subject site.

☐ Provide geotechnical recommendations pertaining to 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. The report contains our findings and includes geotechnical recommendations pertaining to the design and construction of the proposed development as understood at the time of this report.

2.0 Proposed Development

Based on the available drawings, it is understood that the proposed development will consist of a high-rise building, with a podium structure which will extend beyond the footprint of the high-rise building. It is further understood that the proposed building will have 3 levels of underground parking.

It is also expected that the proposed development will be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

A supplemental geotechnical investigation was carried out at the subject site by Paterson on November 18 and 19, 2024. At that time, a total of 3 boreholes (BH 1-24 to BH 3-24) were advanced to a maximum depth of 13.4 m below the existing ground surface. A previous geotechnical investigation was completed by this firm in May 2020. At that time, 5 boreholes (BH 1 through BH 5) were advanced to a maximum depth of 14.7 m below the existing ground surface.

The borehole locations were distributed in a manner to provide general coverage of the subject site, taking into consideration available access and underground utilities. The approximate locations of the test holes are shown on Drawing PG5332-1 – Test Hole Location Plan included in Appendix 2.

The boreholes were advanced using a low-clearance auger drill rig operated by a two-person crew. The drilling procedure consisted of augering to the required depths at the selected locations, and sampling and testing the overburden. All fieldwork was conducted under the full-time supervision of our personnel under the direction of a senior engineer.

Sampling and In Situ Testing

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

A Standard Penetration Test (SPT) was conducted at each borehole 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 overburden thickness was evaluated by a dynamic cone penetration test (DCPT) completed at borehole BH 5. The DCPT consists of driving a steel drill rod, equipped with a 50 mm diameter cone at the tip, using a 63.5 kg hammer falling from a height of 760 mm. The number of blows required to drive the cone into the soil is recorded for each 300 mm increment.

Bedrock samples were recovered from boreholes BH 1-24, BH 2-24 and BH 3-24 using a core barrel and diamond drilling techniques. A recovery value and a Rock Quality Designation (RQD) value were calculated for each drilled section (core run) of bedrock. 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 core run over the length of the core run. The values are indicative of the bedrock quality.

The subsurface conditions observed in the test holes 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

Groundwater monitoring wells were installed in boreholes BH 1, BH 2 and BH 3 to permit monitoring of the groundwater levels subsequent to the completion of the sampling program. A flexible polyethylene standpipe was installed in all other boreholes to measure the stabilized groundwater levels subsequent to completion of the sampling program.

Sample Storage

All samples from the supplemental investigation will be stored in the laboratory for a period of 1 month after issuance of the 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 with respect to a geodetic datum. The locations of the boreholes and ground surface elevation at each borehole location are presented on Drawing PG5332-1 – Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

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

3.4 Analytical Testing

One (1) soil sample was submitted for analytical testing to assess the potential for exposed ferrous metals and the potential of sulphate attacks against subsurface concrete structures. The sample was analyzed to determine its concentration of sulphate and chloride along with its resistivity and pH. The laboratory test results are shown in Appendix 1 and are discussed in Section 6.7.

4.0 Observations

4.1 Surface Conditions

The site is partially occupied by 3 existing buildings, the remainder of the site is generally occupied by asphalt-paved access lanes and parking areas with some landscaped margins.

The site is bordered by Chamberlain Avenue to the north, a commercial property to the west, a paved parking area followed by the associated commercial building to the east, and residential dwellings to the south. The existing ground surface across the site is relatively flat and at grade with adjacent properties and roadways at approximate geodetic elevation 67.5 m.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile at the borehole locations consists of asphalt underlain by fill extending to an approximate depth of 0.6 to 2.3 m below the existing ground surface. The fill was generally observed to consist of a compact, brown silty sand with crushed stone and some brick, metal, wood, concrete and plastic fragments.

A silty clay deposit was encountered underlying the fill. This deposit was observed to consist of a very stiff to stiff, brown silty clay, becoming a stiff grey silty clay below approximate depths of 3.4 to 4.9 m below the existing ground surface.

Underlying the silty clay deposit at approximate depths of 5.7 to 7.6 m, a layer of sandy silt to silty sand was encountered. The sandy silt to silty sand layer was further underlain by a glacial till deposit at approximate depths of 7 to 9.3 m below the existing ground surface. The glacial till deposit was observed to consist of a grey silty clay with sand and/or grey silty sand with clay and some gravel, cobbles and boulders.

Practical refusal to augering or the DCPT was encountered at depths ranging from 11.0 to 14.7 m below the existing ground surface.

Bedrock

The bedrock was cored at boreholes BH 1-24, BH 2-24 and BH 3-24, and based on the recovered rock core, was observed to consist of good to excellent quality limestone. The bedrock was cored to a maximum depth of 13.4 m below the existing ground surface.

Reference should be made to the Soil Profile and Test Data Sheets in Appendix 1 for details of the soil and bedrock profile encountered at each borehole location.

Based on available geological mapping, the bedrock at the subject site consists of limestone with interbedded shale of the Verulam formation across the west half of the subject site, and transitions to shale of the Billings formation across the east half of the subject site with a drift thickness of 10 to 15 m.

4.3 Groundwater

Groundwater levels measured in the standpipes are summarized in Table 1 below.

Table 1 – Summary of Groundwater Levels

Borehole Number	Ground Surface Elevation (m)	Measured Groundwater Level		Date Recorded
		Depth (m)	Elevation (m)	
BH1*	67.53	2.44	65.09	May 29, 2020
BH2*	67.49	4.01	63.48	
BH3*	67.77	4.43	63.34	
BH4	67.48	4.55	62.93	
BH5	67.57	Blocked	-	
BH 1-24	67.33	0.00	67.33	November 26, 2024
BH 2-24	67.52	3.93	63.59	
BH 3-24	67.74	4.13	63.61	

Note: The ground surface elevation at each borehole location was surveyed using a handheld GPS using a geodetic datum.
*- Denotes Monitoring Well

It should be noted that the groundwater levels could be influenced by surface water infiltrating the backfilled boreholes. Long-term groundwater levels can also be estimated based on the observed colour and consistency of the recovered soil samples. Based on these observations, the long-term groundwater table can be expected at approximately **4 to 5 m** below ground surface within the silty clay layer. The recorded groundwater levels are noted on the applicable Soil Profile and Test Data sheet presented in Appendix 1.

It should be noted that groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater level could vary at the time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered suitable for the proposed development. It is recommended that the proposed building be founded on conventional spread footings placed on clean, surface sounded bedrock.

Where clean, surface sounded bedrock is not encountered at the underside of footing (USF) elevation, the overburden soils should be sub-excavated to the surface of the clean, surface sounded bedrock and replaced with lean concrete up to the proposed founding elevation. The lateral limits of the lean concrete placement should be in accordance with our lateral support recommendations provided herein.

Due to the presence of the silty clay deposit, a permissible grade raise restriction will be required for the proposed grading.

Dependent on the final depth of construction for the proposed site servicing and building construction, some rock removal may be required.

It is also understood that an existing 1220 mm diameter backbone watermain is present on the opposite side of Chamberlain Avenue from the proposed development. Vibration considerations are discussed herein.

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

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and deleterious fill, such as those containing organic materials, should be stripped from under any buildings, paved areas and other settlement sensitive structures.

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

Bedrock Removal

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

Prior to considering blasting operations, the blasting effects on the existing services, buildings, and other structures should be addressed. A pre-blast or preconstruction survey of the existing structures located in the proximity of the blasting operations should be carried out prior to commencing site activities. The extent of the survey should be determined by the blasting consultant and should be sufficient to respond to any inquiries or claims related to the blasting operations. As a general guideline, peak particle velocity (measured at the structures) should not exceed 25 mm/s during the blasting program to reduce the risks of damage to the existing structures.

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

Vibration Considerations

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

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

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

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

Backbone Watermain

The minimum distance between the proposed excavation and the existing 1220 mm backbone watermain will be approximately 15.75 m.

Accordingly, due to the distance between the proposed excavation and the 1220 mm backbone watermain, elevated vibrations are not expected to be induced on the pipe. Vibration monitoring will nonetheless be conducted along Chamberlain Avenue during the shoring installation, excavation, and foundation construction in order to ensure that vibrations remain within acceptable limits.

The following vibration monitoring program is recommended to ensure that excessive movements and vibrations do not occur at the watermain location:

☐ Continuously monitor the vibration levels with 2 seismographs placed directly on the 1220 mm backbone watermain. Since this watermain straddles the northern curb of Chamberlain Avenue, a hydrovac truck will extend holes on the landscaped side of the curb down to the top of watermain for placement of the geophones. These approximate locations are shown on the sketch in Appendix 3.

The landscaped side of the curb is considered a better option than the road, since it will remain more accessible should maintenance of the geophones be required during construction.

☐ The vibration limits are provided in Table 2 below. If these vibration limits are exceeded, the site superintendent will be notified by Paterson personnel of the exceedance and the shoring/excavation operation will be stopped, in order to evaluate how to maintain the vibrations within the limits before re-starting the construction.

Table 2 - Vibration Limits for Work Completed Adjacent to Watermain

Location of Vibration Monitor	Peak Particle Velocity (mm/s)	Frequency (Hz)
Directly on Top of Watermain	5	<20
	8	20 – 30
	10	30 – 50
	25	50 – 60
	40	>60

Note: Warning limits shall be 75% of the stop-work limits listed above.

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

Fill Placement

Fill used for grading beneath the proposed building should consist of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. This material should be tested and approved prior to delivery to the site. The fill should be placed in lifts no greater than 300 mm thick and compacted using suitable compaction equipment for the lift thickness. Fill placed beneath the building and paved areas should be compacted to at least 98% of the material's standard Proctor maximum dry density (SPMDD).

Non-specified existing fill, along with site-excavated soil, can be used as general landscaping fill where settlement of the ground surface is of minor concern. This material should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids. If this material is to be used to build up the subgrade level for areas to be paved, it should be compacted in thin lifts to at least 95% of the material's SPMDD.

Non-specified existing fill and site-excavated soils are not suitable for use as backfill against foundation walls unless used in conjunction with a composite drainage membrane.

Lean Concrete In-Filled Trenches

Where the clean, surface sounded bedrock is encountered below the design USF elevation, zero-entry vertical trenches should be excavated to the underlying bedrock surface and then backfilled with lean concrete (17 MPa 28-day compressive strength). Typically, the excavation sidewalls will be used as the form to support the concrete. The trench excavation should be at least 150 mm wider than all sides of the footing at the base of the excavation.

The additional width of the concrete poured against an undisturbed trench sidewall will suffice in providing a direct transfer of the footing load to the underlying clean, surface sounded bedrock.

5.3 Foundation Design

Bearing Resistance Values

Footings placed directly on clean, surface-sounded bedrock, or on lean concrete which is placed directly over the clean surface-sounded bedrock, can be designed using a factored bearing resistance value at serviceability limit states (SLS) and ultimate limit states (ULS) of **5,000 kPa**, incorporating a geotechnical resistance factor of 0.5.

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

Footings bearing on clean, surface-sounded bedrock and designed using the above noted bearing pressures will be subjected to negligible post-construction total and differential settlements.

Lateral Support

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

Permissible Grade Raise Recommendations

Due to the presence of the silty clay deposit, a permissible grade raise restriction of **2.0 m** is recommended for grading at the subject site.

If higher than permissible grade raises are required, preloading with or without a surcharge, lightweight fill, and/or other measures should be investigated to reduce the risks of unacceptable long-term 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 (OBC) 2024. The shear wave velocity testing was completed by Paterson personnel. The results of the shear wave velocity test are provided in Figures 2 and 3 attached in Appendix 2 of the present report.

Field Program

The seismic array testing location was placed along the west boundary of the subject site and as presented in Drawing PG5332-1 - Test Hole Location Plan, 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. A 12-pound dead blow 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 4 to 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 located at 10.0, 3.0 and 2.0 m away from the first and last 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 building. The bedrock velocity was interpreted using the main refractor wave velocity, which is considered a conservative estimate of the bedrock velocity due to the expected increasing quality of the bedrock with depth. It should be noted that as bedrock quality increases, the bedrock shear wave velocity also increases. Based on our testing results, the average overburden shear wave velocity is **221 m/s**, while the bedrock shear wave velocity is **2,286 m/s**.

The V_{s30} was calculated using the standard equation for average shear wave velocity calculation from the Ontario Building Code (OBC2024) and as presented below:

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

$$V_{s30=} \frac{30\ m}{\left(\frac{30\ m}{2{,}286\ \ m/s}\right)}$$

$$V_{s30=}\ 2{,}286\ m/s$$

Based on the results of the shear wave velocity testing, the average shear wave velocity, V_{s30}, at the subject site is **2,286 m/s** for the proposed foundations placed directly or indirectly on the bedrock surface. Therefore, a **Site Class V_{2286}** is applicable for the design of the proposed building as per Table 4.1.8.4.A of the OBC 2024. The soils underlying the subject site are not susceptible to liquefaction.

5.5 Basement Slab

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

It is anticipated that the underground levels for the proposed building will be mostly parking and the recommended pavement structures noted in Section 5.7 will be applicable. However, if storage or other uses of the lower level will 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.

Any soft areas in the basement slab subgrade should be removed and backfilled with appropriate backfill material prior to placing fill. OPSS Granular A or Granular B Type II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab. All backfill material within the footprint of the proposed building should be placed in maximum 300 mm thick loose layers and compacted to a minimum of 98% of the SPMDD.

In consideration of the groundwater conditions at the site, an underslab drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided in the subfloor fill under the lower basement floor. 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 bulk (drained) unit weight of 20 kN/m^3.

Where undrained conditions are anticipated (i.e. below the groundwater level), 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.

Lateral Earth Pressures

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

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

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

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

Seismic Earth Pressures

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

The seismic earth force (ΔP) can be calculated using $0.375 \cdot a \cdot \gamma \cdot H^2/g$ 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 0.308g according to OBC 2024. 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 2024.

5.7 Rock Anchor Design

Overview of Anchor Features

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

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

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

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

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

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

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

Grout to Rock Bond

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

Rock Cone Uplift

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

Recommended Rock Anchor Lengths

Parameters used to calculate rock anchor lengths are provided in Table 3 on the next page:

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

The fixed anchor length will depend on the diameter of the drill holes. Recommended anchor lengths for a 75 mm and 125 mm diameter hole are provided in the following Table 4.

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

Table 4 - Recommended Rock Anchor Lengths - Grouted Rock Anchor

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

Other considerations

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

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

5.8 Pavement Structure

Lowest Underground Parking Level

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 5 below.

Table 5 – Recommended Rigid Pavement Structure – Underground Parking Level

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 imported fill, or OPSS Granular B Type I or II material placed over in situ soil or 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.

Pavement Structure Over Podium Deck

The pavement structures presented in Tables 6 and 7 should be used for car only parking areas, at grade access lanes and heavy loading parking areas over the top of the podium structure, should they be required.

Table 6 - Recommended Pavement Structure - Car Only Parking Areas Over Podium Deck

Thickness (mm)	Material Description
50	**Wear Course** – HL-3 or Superpave 12.5 Asphaltic Concrete
200*	**BASE** - OPSS Granular A Crushed Stone
See below**	**Thermal Break** -** Rigid Insulation (See Following Paragraph)
n/a	**Waterproofing Membrane and IKO Protection Board**

SUBGRADE – Reinforced concrete podium deck

* Thickness of base course is dependent on grade of insulation as noted in proceeding paragraph

** If specified by others, not required from a geotechnical perspective

Table 7 - Recommended Pavement Structure – Access Lanes, Fire Truck Lane, Ramp, and Heavy Loading Areas Over Podium Deck

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

SUBGRADE – Reinforced concrete podium deck

* Thickness of base course is dependent on grade of insulation as noted in proceeding paragraph

** If specified by others, not required from a geotechnical perspective

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

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

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

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

Pavement Structure on Overburden Soils

Beyond the podium deck, the following pavement structures in Tables 8 and 9 may be used for car only parking and heavy traffic areas on overburden.

Table 8 – Recommended Asphalt Pavement Structure – Car only Parking Areas

Thickness (mm)	Material Description
50	**Wear Course** – Superpave 12.5 Asphaltic Concrete
150	**BASE** – OPSS Granular A Crushed Stone
300	**SUBBASE** – OPSS Granular B Type II
SUBGRADE – Existing imported fill, or OPSS Granular B Type I or II material placed over in situ soil or bedrock.	

Table 9 – Recommended Asphalt Pavement Structure – Access Lanes, Ramp 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
450	**SUBBASE** – OPSS Granular B Type II
SUBGRADE – Existing imported fill, or OPSS Granular B Type I or II material placed over in situ soil or bedrock.	

Other Considerations

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

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

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

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage and Waterproofing

For the proposed underground parking levels, it is expected that the building foundation walls will be placed in close proximity to the site boundaries. Therefore, it is recommended that the foundation walls be blind-poured against a drainage system and waterproofing system fastened to the shoring system.

Waterproofing of the foundation walls is recommended and the membrane is to be installed from 4 m below finished grade down the foundation walls to the bottom of foundation.

It is also recommended that a composite drainage system, such as Delta Drain 6000 or equivalent, be installed between the waterproofing membrane and the foundation wall and extend from the exterior finished grade to the founding elevation. The purpose of the composite drainage system is to direct any water infiltration resulting from a breach of the waterproofing membrane to the building sump pit. It is recommended that 150 mm diameter sleeves at 3 m centres be cast in the foundation wall at the perimeter footing interface, to allow for the water to flow to an interior perimeter drainage pipe. The perimeter drainage pipe should direct water to sump pit(s) within the lower basement area.

Underslab Drainage

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

Foundation Backfill

Where space is available for conventional wall construction, backfill against the exterior sides of the foundation walls should consist of free-draining, non-frost susceptible granular materials. Imported granular materials, such as clean sand or OPSS Granular A, should be used for this purpose.

6.2 Protection of Footing Against Frost Action

Perimeter foundations of heated structures are required to be insulated against the deleterious effects of frost action. A minimum of 1.5 m of soil cover, or a minimum of 0.6 m of soil cover in conjunction with adequate foundation insulation, should be provided.

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

The foundations for the underground parking levels are expected to have sufficient frost protection due to the founding depth. However, it has been our experience that insufficient soil cover is typically provided to entrance ramps to underground parking garages. Paterson requests permission to review design drawings prior to construction to ensure proper frost protection is provided for these areas.

6.3 Excavation Side Slopes

The side slopes of excavations in the soil and fill overburden materials should either be excavated at acceptable slopes or retained by shoring systems from the beginning of the excavation until the structure is backfilled. Given the proximity of the underground parking levels to the property lines, it is expected that a temporary shoring will be required to support the excavation for this proposed development.

Unsupported Excavations

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be excavated at 1H:1V or shallower. The shallower slope is required for excavation below groundwater level. The subsurface soils are considered to be a Type 2 and 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects.

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

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

A trench box is recommended to protect personnel working in trenches with steep or vertical sides. Services are expected to be installed by “cut and cover” methods and excavations should not remain open for extended periods of time.

Temporary Shoring

Temporary shoring is anticipated to be required to support the overburden soils for the underground parking levels. 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. 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 shoring system may generally consist of steel sheet piles. However, in the vicinity of the adjacent building located at 52 Chamberlain Avenue, a stiffer shoring system, such as a secant pile wall, is recommended to support the existing, adjacent building.

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

Table 10 – Soil Parameters

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

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

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

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

The temporary shoring system may need tie-back anchors extending onto the adjacent properties. Accordingly, consent agreements should be obtained from the neighboring property owners as well as the City of Ottawa prior to construction.

Excavation Base Stability

The base of supported excavations can fail by three general modes:

- ☐ Shear failure within the ground caused by inadequate resistance to loads imposed by grade difference inside and outside of the excavation,
- ☐ Piping from water seepage through granular soils, and
- ☐ Heave of layered soils due to water pressures confined by intervening low permeability soils.

The potential for base heave in cohesive soils should be determined for stability of flexible retaining systems. The factor of safety with respect to base heave, FS_b, is:

$FS_b = N_b s_u / \sigma_z$

where:

N_b - stability factor dependent upon the geometry of the excavation and given in Figure 1 on the following page.

s_u - undrained shear strength of the soil below the base level

σ_z - total overburden and surcharge pressures at the bottom of the excavation.

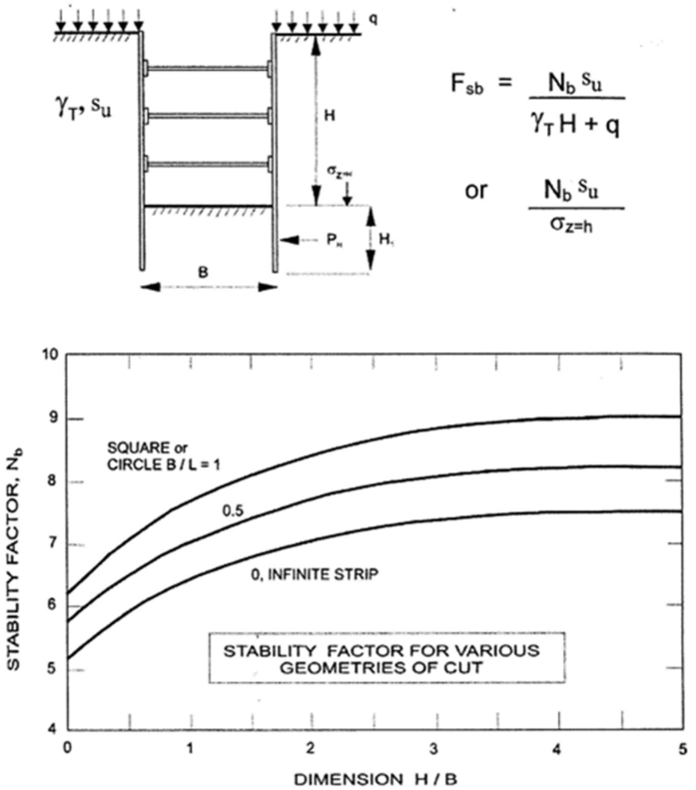

Figure 1 – Stability Factor for Various Geometries of Cut

In the case of stiff clays or compact sands, a factor of safety of 2 is recommended for base stability.

6.4 Pipe Bedding and Backfill

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

The pipe bedding for sewer and water pipes placed on a relatively dry, undisturbed subgrade surface should consist of at least 150 mm of OPSS Granular A material. The material should be placed in maximum 300 mm thick lifts and compacted to a minimum of 95% of its SPMDD. The bedding material should extend at least to the spring line of the pipe.

The cover material, which should consist of OPSS Granular A, should extend from the spring line of the pipe to at least 300 mm above the obvert of the pipe. The material should be placed in maximum 300 mm thick lifts and compacted to a minimum of 95% of its SPMDD.

Generally, it should be possible to re-use the moist (not wet) brown silty clay above the cover material if the excavation and filling operations are carried out in dry weather conditions. Wet silty clay material will be difficult to re-use, as the high-water contents make compacting impractical without an extensive drying period.

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

To reduce long-term lowering of the groundwater level at this site, clay seals should be installed in the services trenches. The clay seals should be at least 1 m long and should extend from trench wall to trench wall. Generally, the seals should extend from the frost line and fully penetrate the bedding, sub-bedding and cover material. The barriers should consist of relatively dry and compactable brown silty clay placed in maximum 225 mm thick loose layers and compacted to a minimum of 95% of the material's SPMDD. Due to the small size of the site, clay seals are only recommended to be placed at the site boundaries and at the building interfaces.

6.5 Groundwater Control

Due to the relatively impervious nature of the overlying silty clay within the upper portion of the soil profile, it is anticipated that groundwater infiltration into the excavations should be low and controllable using open sumps.

Where excavations are extended within the sandy silt to silty sand material below the long-term groundwater level, the groundwater infiltration is anticipated to be moderate to high.

Generally, pumping from open sumps should be sufficient to control the groundwater influx through the sides of shallow 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.

Groundwater Control for Building Construction

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

For typical ground or surface water volumes being pumped during the construction phase, 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 Section 6.1. Any groundwater which breaches the building’s perimeter groundwater infiltration control system will be directed to the 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 very low to negligible. A more accurate estimate of groundwater flow can be provided at the time of construction, once the pressure relief chamber valve is closed and full hydrostatic pressure Is applied to the structure.

Impacts on Neighbouring Properties

Since the proposed building will be founded below the long-term groundwater level, a groundwater infiltration control system has been recommended to minimize the effects of water infiltration. Therefore, long-term dewatering of the site will be minimal and should have no adverse effects to the surrounding buildings or structures. The short-term dewatering during the excavation program will be managed by the excavation contractor, as discussed above.

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, 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.

6.7 Corrosion Potential and Sulphate

The results of analytical testing show that the sulphate content is less than 0.1%. This result is indicative that Type 10 Portland cement (normal cement) would be appropriate for this site. The chloride content and the pH of the sample indicate that they are not significant factors in creating a corrosive environment for exposed ferrous metals at this site, whereas the resistivity is indicative of a moderately aggressive corrosive environment.

7.0 Recommendations

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

☐ Review the Contractor’s design of the temporary shoring system.

☐ Review of waterproofing details for elevator shaft(s) and building sump pits.

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

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

☐ 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.

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

8.0 Statement of Limitations

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

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than AzureCon Inc. or their agents is not authorized without review by Paterson for the applicability of our recommendations to the altered use of the report.

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
June 27, 2025
S. S. DENNIS
100519516
Scott Dennis
PROVINCE OF ONTARIO

Scott Dennis

Mrunmayi Anvekar, M.Eng.

Scott S. Dennis, P.Eng.

Report Distribution:

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- ☐ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

30-48 Chamberlain Avenue, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 367939.86 **NORTHING:** 5030021.36 **ELEVATION:** 67.33

PROJECT: Proposed Mixed-use Building

BORINGS BY: CME-55 Low Clearance Drill

REMARKS:

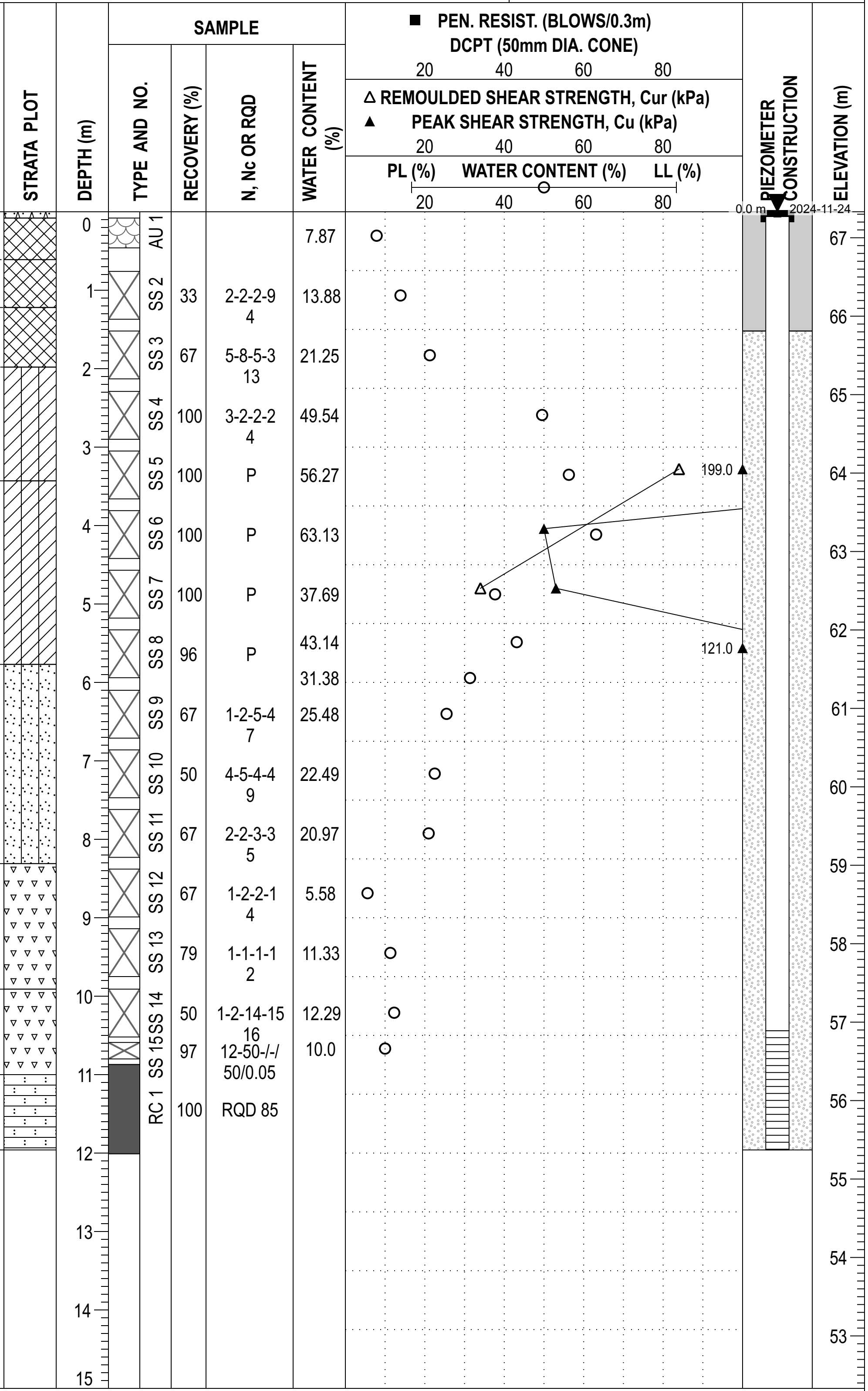
DATE: November 18, 2024

FILE NO. : **PG5332**

HOLE NO. : **BH 1-24**

SAMPLE DESCRIPTION	
GROUND SURFACE	
ASPHALT	0.08m [67.25m]
FILL: Compact, brown silty sand, with gravel and crushed stone	0.61m [66.72m]
FILL: Loose, brown silty sand, some gravel, trace concrete and wood	1.22m [66.11m]
FILL: Compact, brown silty sand	1.98m [65.35m]
Very stiff, brown **SILTY CLAY**	3.43m [63.90m]
Firm, grey **SILTY CLAY**	5.77m [61.56m]
Loose, grey **SILTY SAND** to **SANDY SILT**, some clay	8.31m [59.02m]
GLACIAL TILL: Very loose to compact, grey silty clay, some sand, gravel and cobbles	9.91m [57.42m]
GLACIAL TILL: Compact, grey silty sand, with clay, gravel, cobbles and boulders	11.00m [56.33m]
BEDROCK: Good quality limestone	11.96m [55.37m]
End of Borehole (GWL at ground surface - November 26, 2024)	

TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)
AU 1			7.87
SS 2	33	2-2-2-9 4	13.88
SS 3	67	5-8-5-3 13	21.25
SS 4	100	3-2-2-2 4	49.54
SS 5	100	P	56.27
SS 6	100	P	63.13
SS 7	100	P	37.69
SS 8	96	P	43.14 31.38
SS 9	67	1-2-5-4 7	25.48
SS 10	50	4-5-4-4 9	22.49
SS 11	67	2-2-3-3 5	20.97
SS 12	67	1-2-2-1 4	5.58
SS 13	79	1-1-1-1 2	11.33
SS 14	50	1-2-14-15 16	12.29
SS 15	97	12-50-/-/ 50/0.05	10.0
RC 1	100	RQD 85	



SOIL PROFILE AND TEST DATA
Geotechnical Investigation
30-48 Chamberlain Avenue, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 367963.98 **NORTHING:** 5030027.60 **ELEVATION:** 67.52

PROJECT: Proposed Mixed-use Building
BORINGS BY: CME-55 Low Clearance Drill
REMARKS:
DATE: November 18, 2024

FILE NO. : PG5332
HOLE NO. : BH 2-24

SAMPLE DESCRIPTION

GROUND SURFACE

ASPHALT 0.08m [67.44m]

FILL: Very dense, brown silty sand, with gravel, crushed stone, asphalt and wood, trace clay 1.45m [66.06m]

FILL: Compact, brown silty sand 2.21m [65.31m]

Very stiff, brown **SILTY CLAY** 4.50m [63.02m]

Very stiff to stiff, grey **SILTY CLAY** to **CLAYEY SILT** 7.62m [59.90m]

Compact **SILTY SAND**, with gravel cobbles and boulders 8.76m [58.76m]

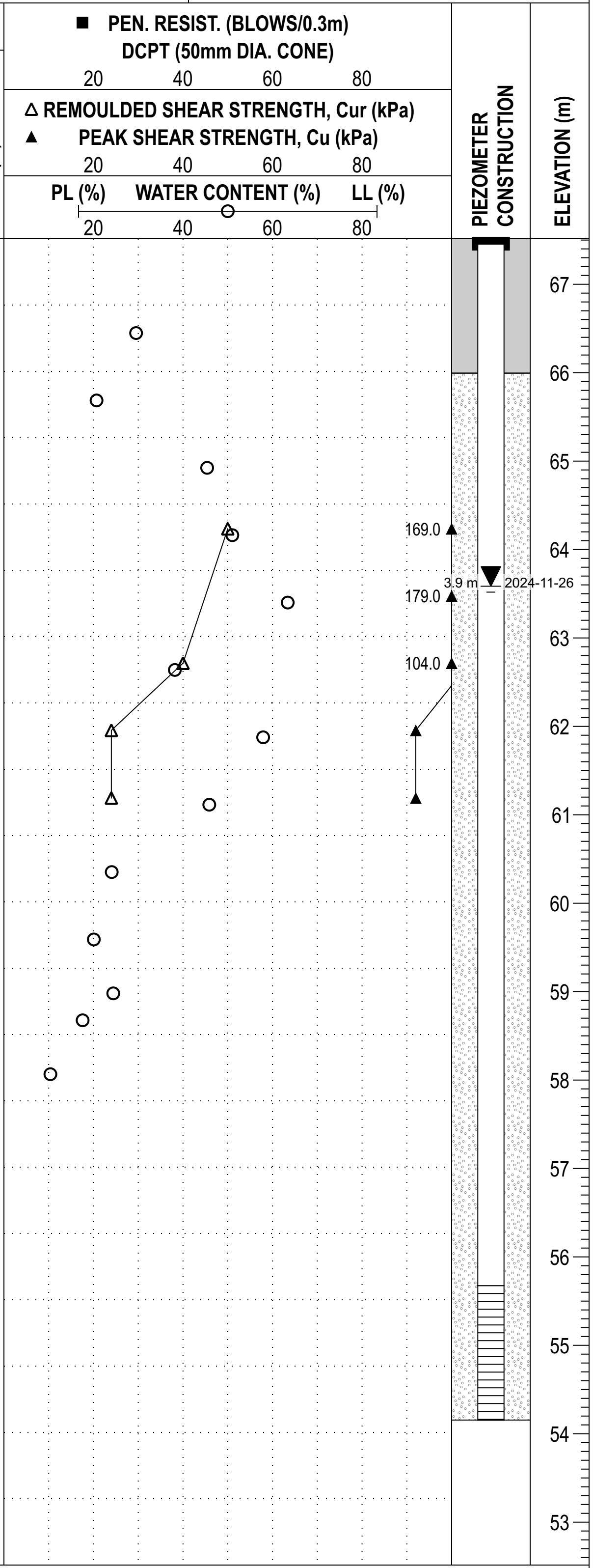
GLACIAL TILL: Dense to very dense, grey silty sand, with cobbles and boulders 11.99m [55.52m]

BEDROCK: Excellent quality limestone 13.36m [54.16m]

End of Borehole

(GWL at 3.93 m depth - November 26, 2024)

TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)
SS 1	50	6-35-36-9 71	29.52
SS 2	46	20-12-10-12 22	20.69
SS 3	33	4-3-2-3 5	45.39
SS 4	100	P	51.02
SS 5	100	P	63.39
SS 6		P	38.18
SS 7		P	57.9
SS 8	100	P	45.87
SS 9	42	P	24.08
SS 10	37	8-11-11-12 22	20.08
SS 11	87	2-2-2-1 4	24.42 17.58
SS 12	67	5-11-23-45 34	10.38
RC 1			
RC 2			
RC 3			
RC 4	100	RQD 93	



SOIL PROFILE AND TEST DATA

Geotechnical Investigation

30-48 Chamberlain Avenue, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 367973.89 **NORTHING:** 5030043.07 **ELEVATION:** 67.74

PROJECT: Proposed Mixed-use Building

BORINGS BY: CME-55 Low Clearance Drill

REMARKS:

DATE: November 18, 2024

FILE NO. : **PG5332**

HOLE NO. : **BH 3-24**

SAMPLE DESCRIPTION

GROUND SURFACE

ASPHALT 0.11m [67.63m]

FILL: Compact to loose, brown silty sand, with gravel and crushed stone
- Some brick fragments below 0.76 m depth

2.21m [65.53m]

Very stiff, brown **SILTY CLAY**, trace gravel

6.02m [61.72m]

Stiff, grey **SILTY CLAY** 6.55m [61.19m]

Compact, grey **SILTY SAND** to **SANDY SILT**, trace gravel

9.07m [58.67m]

Loose to compact, grey **SILTY CLAY**, some sand, cobbles and gravel

12.19m [55.55m]

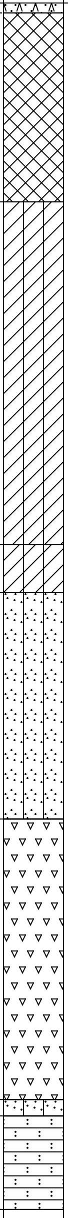
Dense **SILTY SAND**, with clay, gravel, cobbles and boulders 12.37m [55.37m]

BEDROCK: Good quality limestone 13.41m [54.33m]

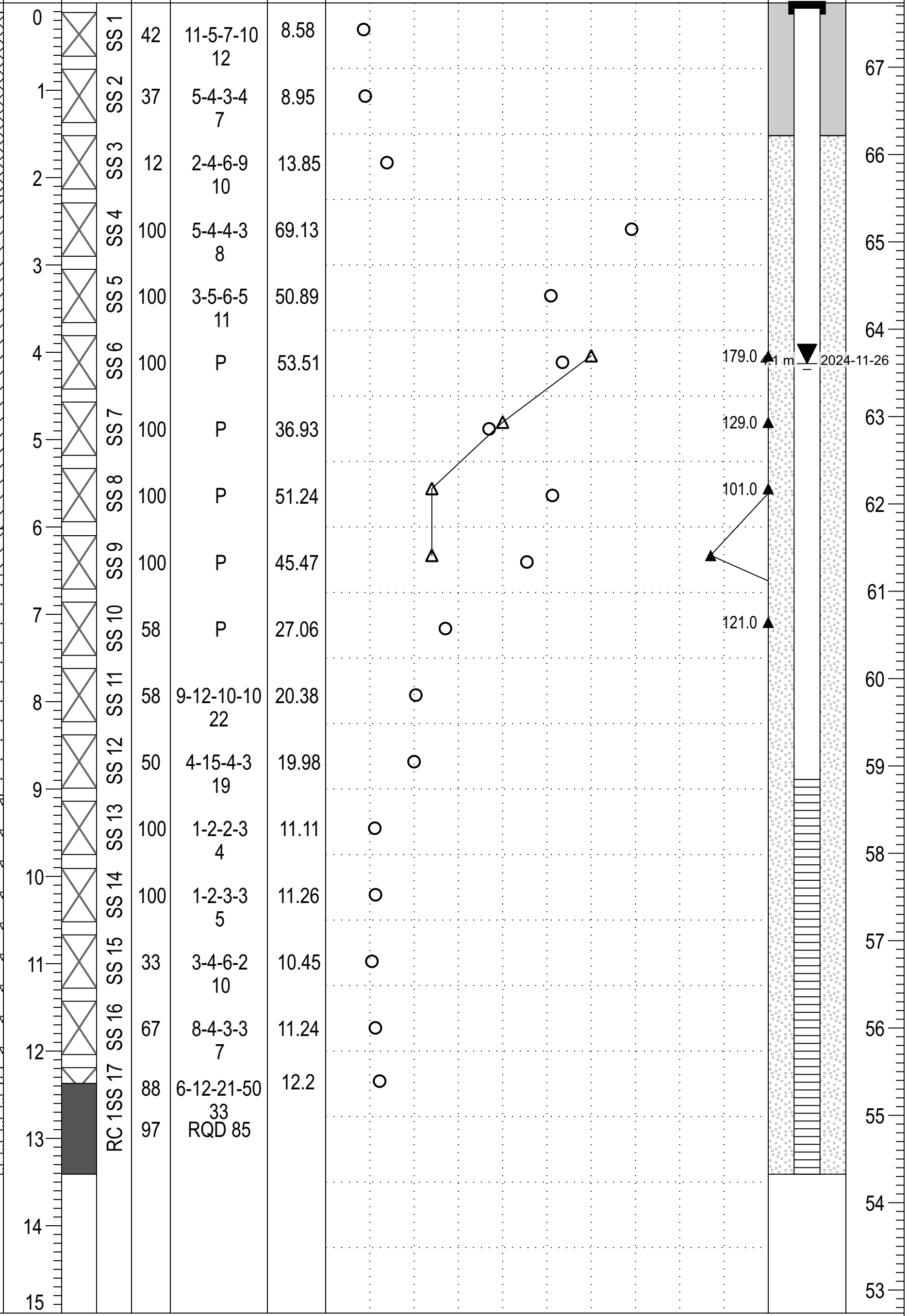
End of Borehole

(GWL at 4.13 m depth - November 26, 2024)

STRATA PLOT



SAMPLE TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)
SS 1	42	11-5-7-10 12	8.58
SS 2	37	5-4-3-4 7	8.95
SS 3	12	2-4-6-9 10	13.85
SS 4	100	5-4-4-3 8	69.13
SS 5	100	3-5-6-5 11	50.89
SS 6	100	P	53.51
SS 7	100	P	36.93
SS 8	100	P	51.24
SS 9	100	P	45.47
SS 10	58	P	27.06
SS 11	58	9-12-10-10 22	20.38
SS 12	50	4-15-4-3 19	19.98
SS 13	100	1-2-2-3 4	11.11
SS 14	100	1-2-3-3 5	11.26
SS 15	33	3-4-6-2 10	10.45
SS 16	67	8-4-3-3 7	11.24
SS 17	88	6-12-21-50 33	12.2
RC 1	97	RQD 85	



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Mixed-Use Building - 30-48 Chamberlain Avenue
Ottawa, Ontario

DATUM Geodetic

REMARKS

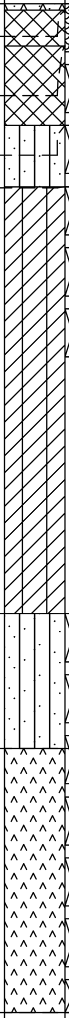
BORINGS BY CME-55 Low Clearance Drill

DATE May 21, 2020

FILE NO. PG5332

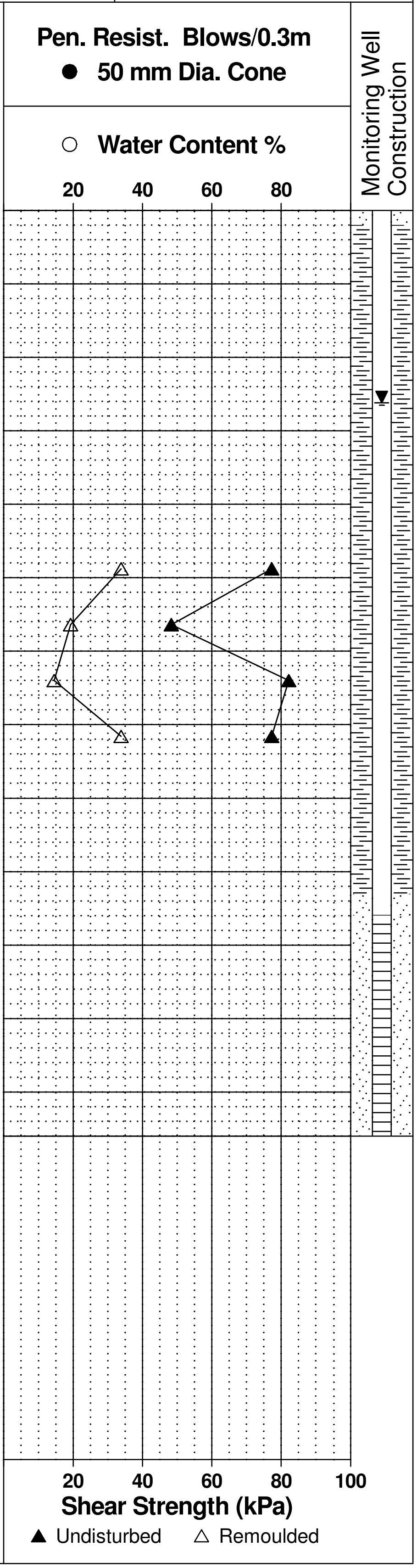
HOLE NO. BH 1

SOIL DESCRIPTION	
GROUND SURFACE	
Asphaltic concrete	0.08
FILL: Brown silty sand with crushed stone	0.53
FILL: Brown silty clay with sand and gravel	1.52
Loose, brown **SILTY SAND**	2.29
Very stiff to stiff, brown **SILTY CLAY** - grey by 4.6m depth	7.62
Grey **SANDY SILT to SILTY SAND,** trace clay	9.30
GLACIAL TILL: Grey silty clay with sand, gravel, cobbles and boulders	12.60



TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	58	8
SS	3	79	9
SS	4	100	6
SS	5	100	5
SS	6	100	4
SS	7	29	6
SS	8	75	2
SS	9	46	4
SS	10		6
SS	11	100	8
SS	12	67	3
SS	13	69	50+

DEPTH (m)	ELEV. (m)
0	67.53
1	66.53
2	65.53
3	64.53
4	63.53
5	62.53
6	61.53
7	60.53
8	59.53
9	58.53
10	57.53
11	56.53
12	55.53



End of Borehole

Practical refusal to augering at 12.60m depth.

(GWL @ 2.62m - July 28, 2020)

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Mixed-Use Building - 30-48 Chamberlain Avenue
Ottawa, Ontario

DATUM Geodetic

FILE NO. PG5332

REMARKS

HOLE NO. BH 2

BORINGS BY CME-55 Low Clearance Drill

DATE May 21, 2020

SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
Asphaltic concrete	0.08
FILL: Brown silty sand with crushed stone	0.60
FILL: Brown silty sand with gravel, some metal and plastic	1.22
Loose to compact, brown **SILTY SAND**	2.13
Very stiff to stiff, brown **SILTY CLAY** - grey by 4.6m depth	6.25
Grey **SANDY SILT to SILTY SAND,** trace clay	7.92
GLACIAL TILL: Grey silty clay with sand, gravel, cobbles and boulders	11.02

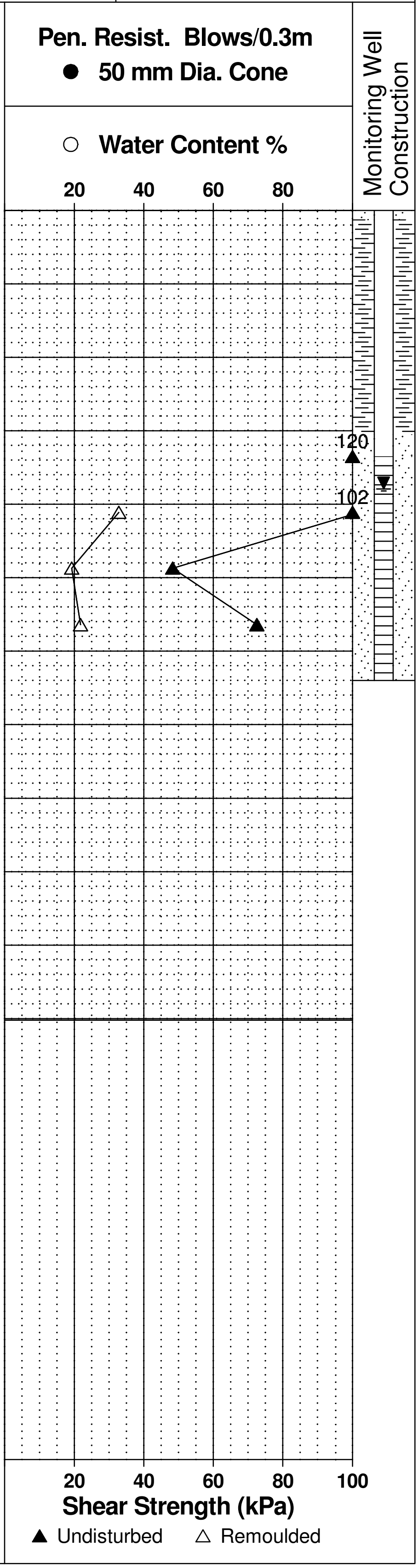
End of Borehole

Practical refusal to augering at 11.02m depth

(GWL @ 3.79m - July 28, 2020)

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	46	7
SS	3	42	13
SS	4	100	5
SS	5	100	W
SS	6	100	14
SS	7	29	7
SS	8	100	4
SS	9	100	3
SS	10	100	50+

DEPTH (m)	ELEV. (m)
0	67.49
1	66.49
2	65.49
3	64.49
4	63.49
5	62.49
6	61.49
7	60.49
8	59.49
9	58.49
10	57.49
11	56.49



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Mixed-Use Building - 30-48 Chamberlain Avenue
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE May 21, 2020

FILE NO. PG5332

HOLE NO. BH 3

SOIL DESCRIPTION	Depth (m)
GROUND SURFACE	
Asphaltic concrete	0.10
FILL: Brown silty sand with crushed stone	0.63
Loose, brown **SILTY SAND**	1.52
Very stiff to stiff, brown **SILTY CLAY** - grey by 4.6m depth - trace sand by 6.4m depth	6.86
Grey **SANDY SILT to SILTY SAND,** trace clay	9.14
GLACIAL TILL: Grey silty clay with sand, gravel, cobbles and boulders	11.76

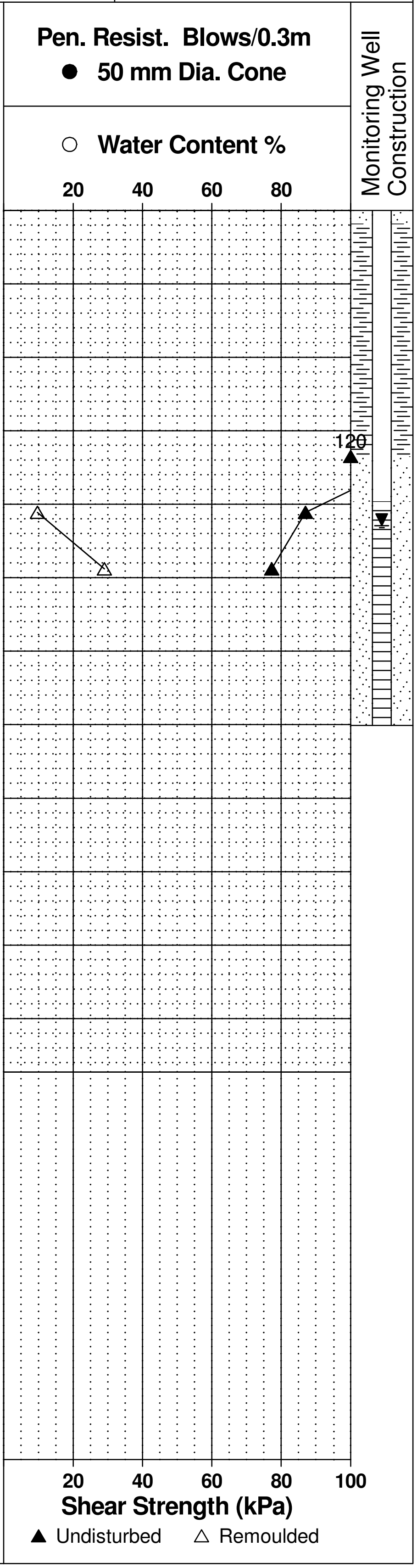
End of Borehole

Practical refusal to augering at 11.76m depth

(GWL @ 4.29m - July 28, 2020)

TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	42	7
SS	3	100	8
SS	4	100	5
SS	5	100	2
SS	6	42	7
SS	7	100	2
SS	8	38	3
SS	9	58	8

DEPTH (m)	ELEV. (m)
0	67.77
1	66.77
2	65.77
3	64.77
4	63.77
5	62.77
6	61.77
7	60.77
8	59.77
9	58.77
10	57.77
11	56.77



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Mixed-Use Building - 30-48 Chamberlain Avenue
Ottawa, Ontario

DATUM Geodetic
REMARKS
BORINGS BY CME-55 Low Clearance Drill
DATE May 22, 2020
FILE NO. PG5332
HOLE NO. BH 4

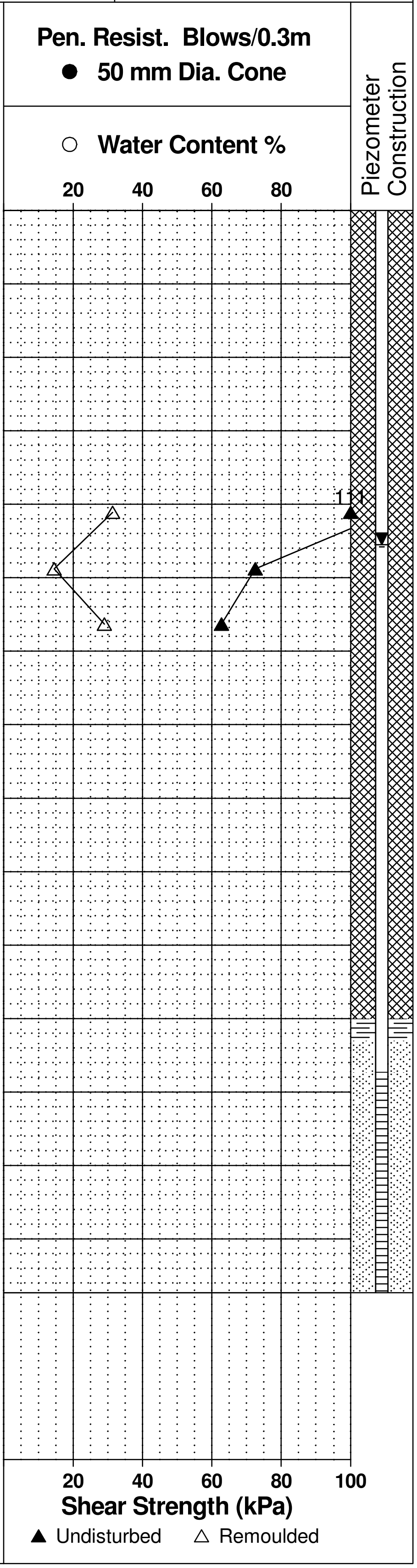
SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD
GROUND SURFACE					
Asphaltic concrete 0.08		AU	1		
FILL: Brown sand with crushed stone 0.60					
FILL: Brown silty sand, some gravel 2.29		SS	2	12	5
		SS	3	4	8
Very stiff to stiff, brown **SILTY CLAY** - grey by 4.6m depth 6.55		SS	4	88	7
		SS	5	100	3
		SS	6	8	P
		SS	7	38	6
Grey **SANDY SILT to SILTY SAND,** trace clay 9.14		SS	8	29	5
GLACIAL TILL: Grey silty clay with sand, gravel, cobbles and boulders 14.73		SS	9	42	3
		SS	10	21	9

End of Borehole

Practical refusal to augering at 14.73m depth.

(GWL @ 4.55m - May 29, 2020)

DEPTH (m)	ELEV. (m)
0	67.48
1	66.48
2	65.48
3	64.48
4	63.48
5	62.48
6	61.48
7	60.48
8	59.48
9	58.48
10	57.48
11	56.48
12	55.48
13	54.48
14	53.48



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Mixed-Use Building - 30-48 Chamberlain Avenue
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE May 22, 2020

FILE NO. PG5332

HOLE NO. BH 5

SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
Asphaltic concrete	0.08
FILL: Brown silty sand with crushed stone	0.60
FILL: Brown silty clay, some sand, gravel and brick	1.52
Compact, brown **SILTY SAND**	2.13
Very stiff to stiff, brown **SILTY CLAY** - grey by 4.6m depth	6.10
Grey **SANDY SILT to SILTY SAND,** trace clay	7.01
GLACIAL TILL: Grey silty clay with sand, gravel, cobbles and boulders	7.47
Dynamic Cone Penetration Test commenced at 7.47m depth. Inferred **GLACIAL TILL**	11.33

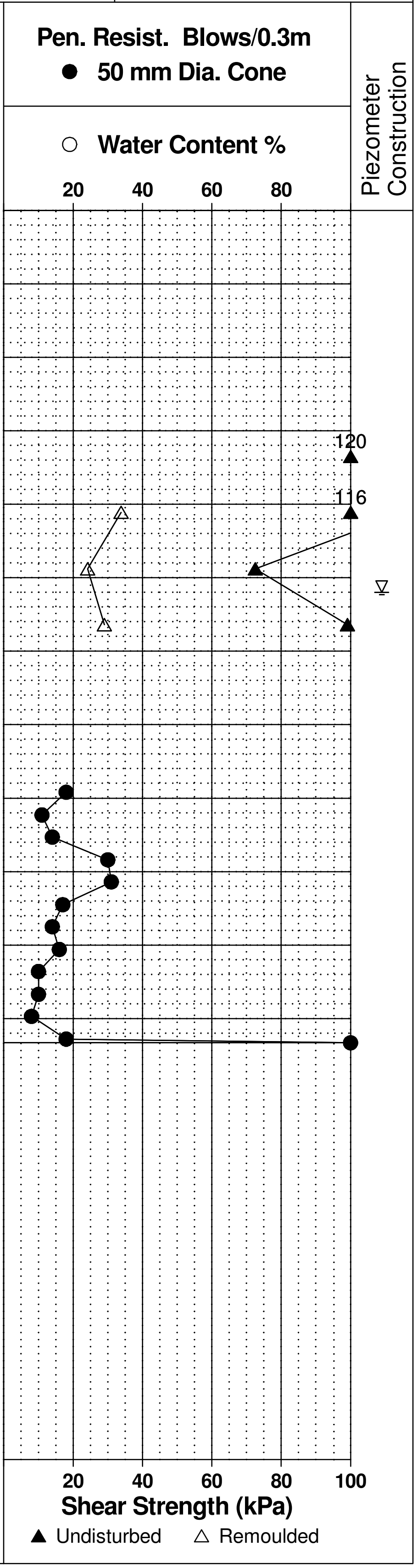
End of Borehole

Practical DCPT refusal at 11.33m depth

(GWL @ 5.2m depth based on field observations)

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	4	11
SS	3	38	21
SS	4	29	5
SS	5	50	13
SS	6	83	19

DEPTH (m)	ELEV. (m)
0	67.57
1	66.57
2	65.57
3	64.57
4	63.57
5	62.57
6	61.57
7	60.57
8	59.57
9	58.57
10	57.57
11	56.57



SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

The standard terminology to describe the relative strength of cohesionless soils is the compactness condition, usually inferred from the results of the Standard Penetration Test (SPT) ‘N’ value. The SPT N value is the number of blows of a 63.5 kg hammer, falling 760 mm, required to drive a 51 mm O.D. split spoon sampler 300 mm into the soil after an initial penetration of 150 mm. An SPT N value of “P” denotes that the split-spoon sampler was pushed 300 mm into the soil without the use of a falling hammer.

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

The standard terminology to describe the strength of cohesive soils is the consistency, which is based on the undisturbed undrained shear strength as measured by the in situ or laboratory shear vane tests, unconfined compression tests, or occasionally by the Standard Penetration Test (SPT). Note that the typical correlations of undrained shear strength to SPT N value (tabulated below) tend to underestimate the consistency for sensitive silty clays, so Paterson reviews the applicable split spoon samples in the laboratory to provide a more representative consistency value based on tactile examination.

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

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

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

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

SS - Split spoon sample (obtained in conjunction with the performing of the Standard Penetration Test (SPT))

TW - Thin wall tube or Shelby tube, generally recovered using a piston sampler

G - "Grab" sample from test pit or surface materials

AU - Auger sample or bulk sample

WS - Wash sample

RC - Rock core sample (Core bit size BQ, NQ, HQ, etc.). Rock core samples are obtained with the use of standard diamond drilling bits.

SYMBOLS AND TERMS (continued)

PLASTICITY LIMITS AND GRAIN SIZE DISTRIBUTION

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

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

Cc - Concavity coefficient = $(D30)^2$ / (D10 x D60)
Cu - Uniformity coefficient = D60 / D10

Cc and Cu are used to assess the grading of sands and gravels:
Well-graded gravels have: $1 < Cc < 3$ and $Cu > 4$
Well-graded sands have: $1 < Cc < 3$ and $Cu > 6$
Sands and gravels not meeting the above requirements are poorly-graded or uniformly-graded.
Cc and Cu are not applicable for the description of soils with more than 10% silt and clay
(more than 10% finer than 0.075 mm or the #200 sieve)

CONSOLIDATION TEST

p'_o - Present effective overburden pressure at sample depth
p'_c - Preconsolidation pressure of (maximum past pressure on) sample
Ccr - Recompression index (in effect at pressures below p'_c)
Cc - Compression index (in effect at pressures above p'_c)

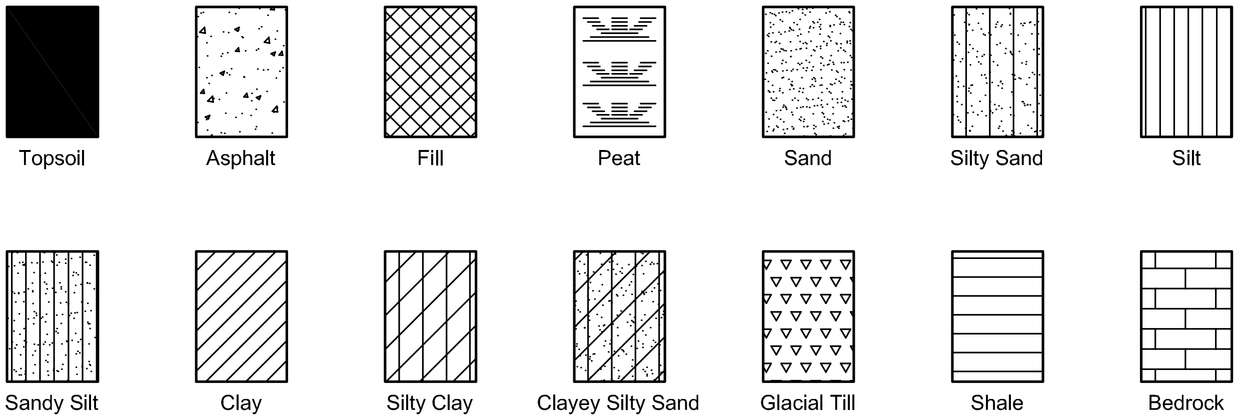
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

PERMEABILITY TEST

k - Coefficient of permeability or hydraulic conductivity is a measure of the ability of water to flow through the sample. The value of k is measured at a specified unit weight for (remoulded) cohesionless soil samples, because its value will vary with the unit weight or density of the sample during the test.

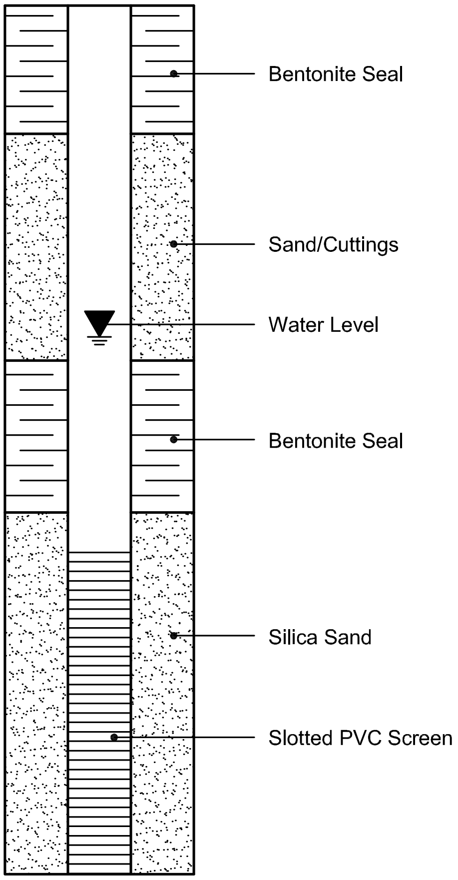
SYMBOLS AND TERMS (continued)

STRATA PLOT

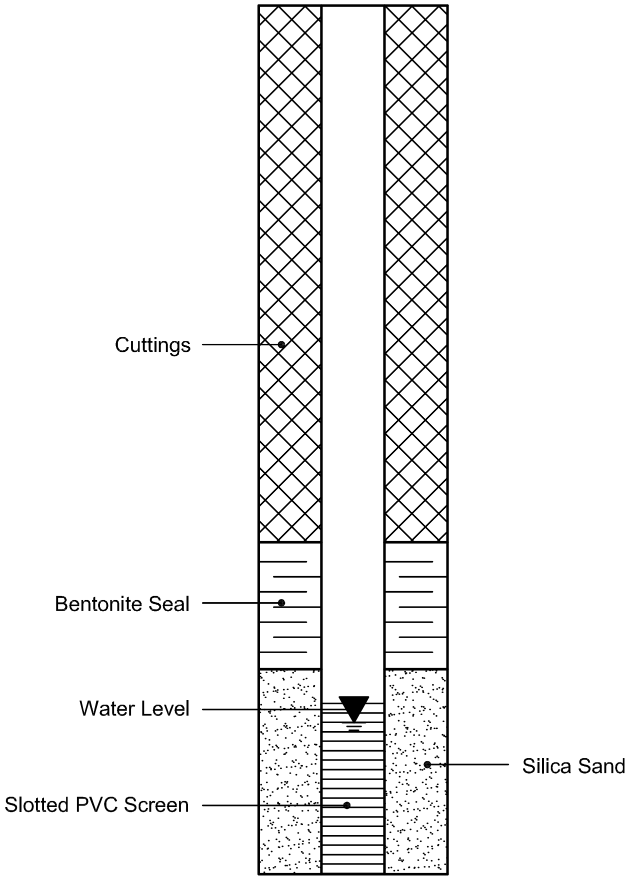


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



	Client ID:	BH1-SS6-12'6-14'6	-	-	-
	Sample Date:	21-May-20 11:00	-	-	-
	Sample ID:	2022150-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	63.5	-	-	-
General Inorganics					
pH	0.05 pH Units	7.11	-	-	-
Resistivity	0.10 Ohm.m	40.3	-	-	-
Anions					
Chloride	5 ug/g dry	65	-	-	-
Sulphate	5 ug/g dry	88	-	-	-

APPENDIX 2

FIGURE 1 – KEY PLAN

FIGURES 2 & 3 – SEISMIC SHEAR WAVE VELOCITY PROFILES

DRAWING PG5332-1 – TEST HOLE LOCATION PLAN

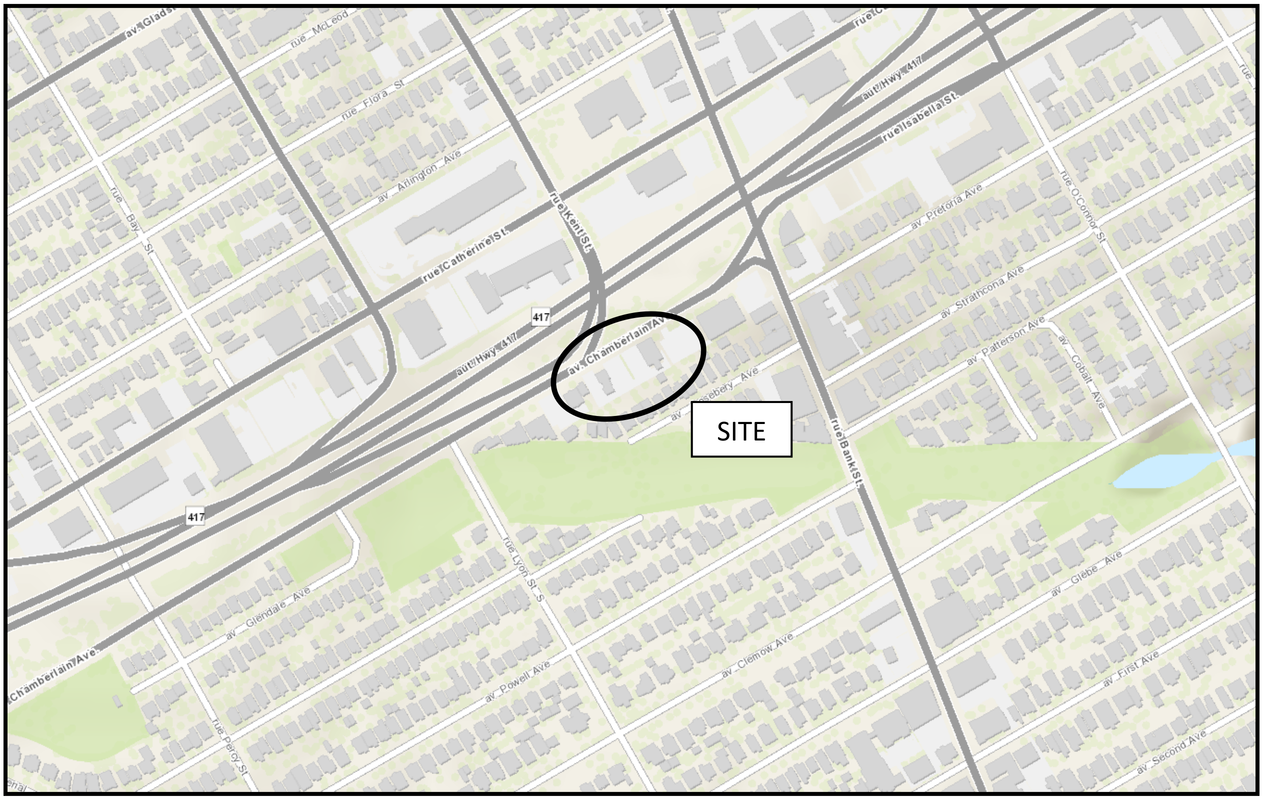


FIGURE 1

KEY PLAN

PATERSON
GROUP

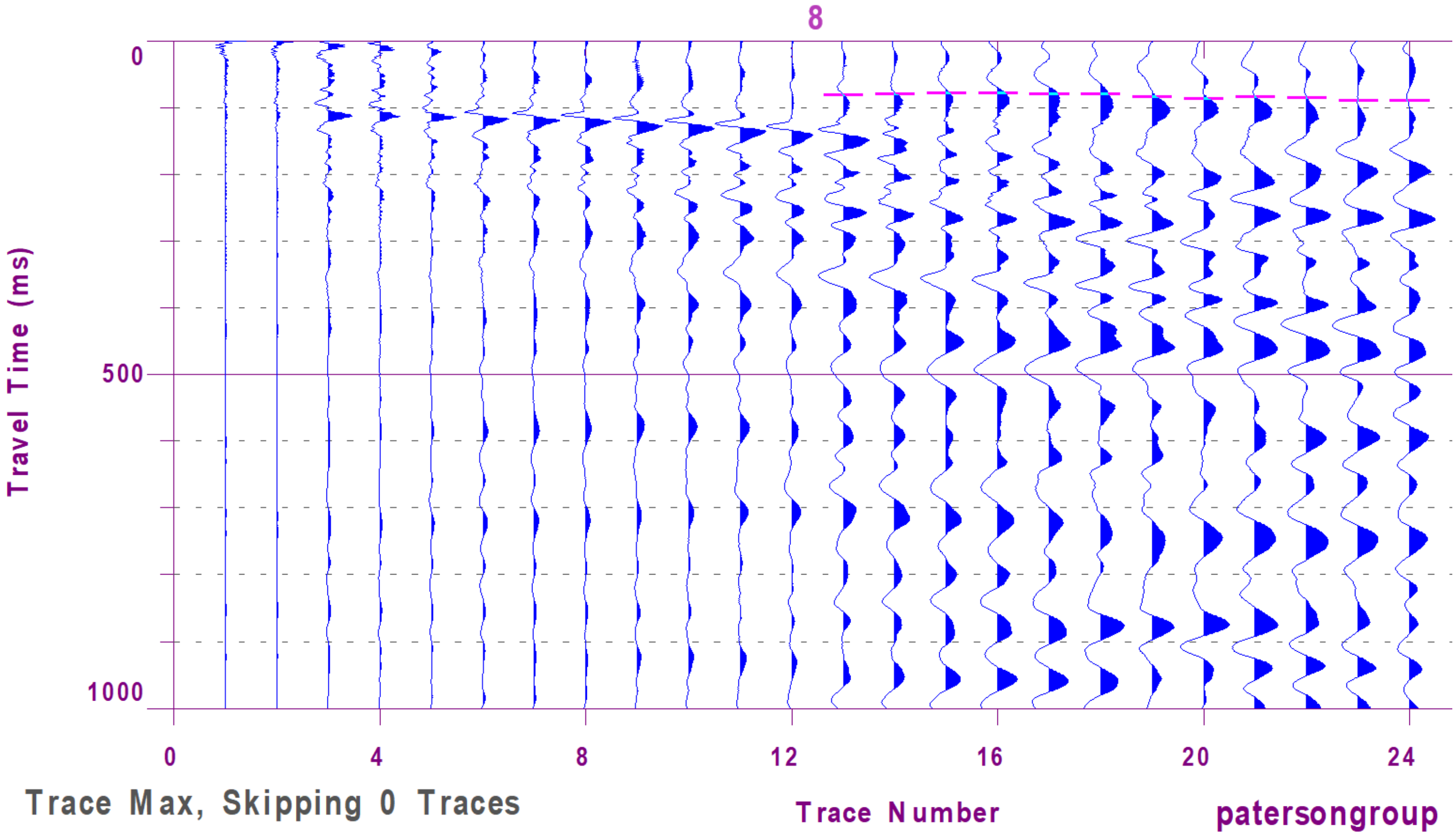


Figure 2 – Shear Wave Velocity Profile at Shot Location -3 m

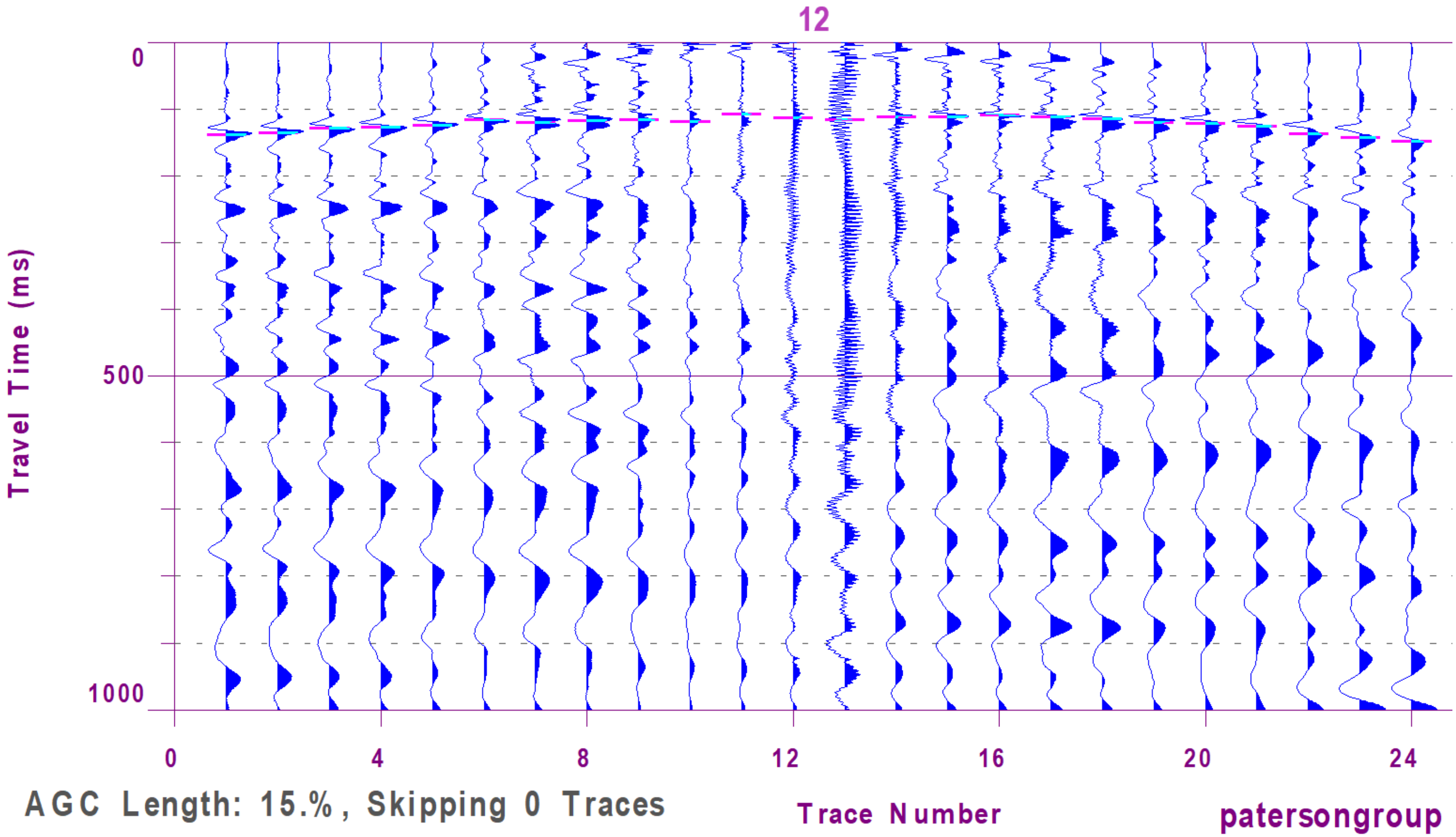
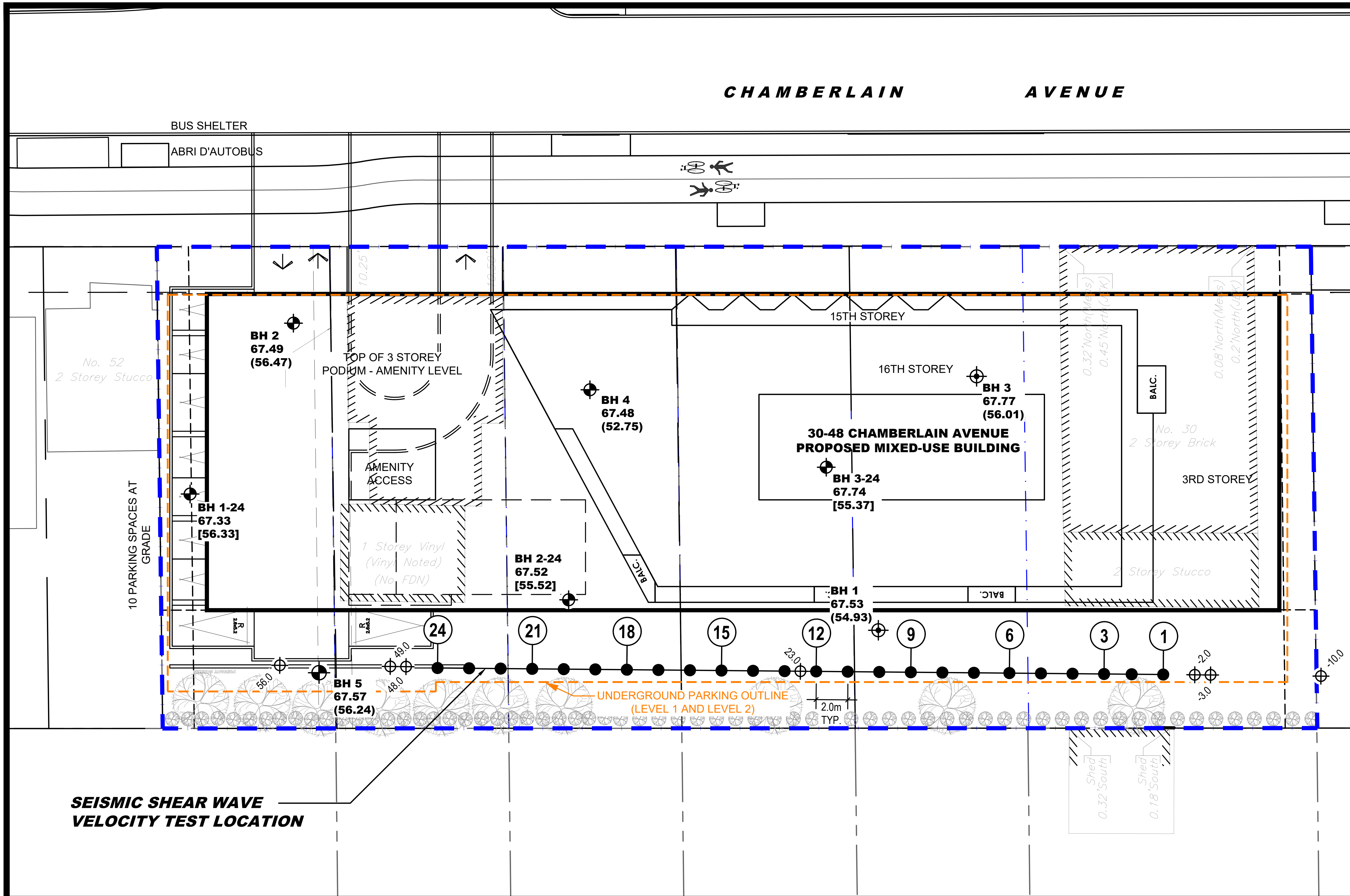


Figure 3 – Shear Wave Velocity Profile at Shot Location 23 m



LEGEND:

⌖	BOREHOLE LOCATION
⌖	BOREHOLE WITH MONITORING WELL LOCATION
67.53	GROUND SURFACE ELEVATION (m)
(54.93)	PRACTICAL REFUSAL TO AUGERING ELEVATION (m)
[55.52]	BEDROCK SURFACE ELEVATION (m)
● ● ●	GEOPHONE LOCATIONS
③	GEOPHONE NUMBER
⌖ 23.0	SHOT LOCATION

BASE PLAN PROVIDED BY FARLEY, SMITH AND DENIS SURVEYING LTD.

CONCEPTUAL PLAN PROVIDED BY HOBIN ARCHITECTURE

BOREHOLE ELEVATIONS ARE REFERENCED TO GEODETIC DATUM.

SCALE: 1:250

0 1 2 3 4 5 10 15m

PATERSON GROUP

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

NO.	REVISIONS	DATE	INITIAL
2	ADDED 2024 BOREHOLES AND SEISMIC SHEAR WAVE VELOCITY TEST LOCATION	27/11/2024	MA
1	UPDATED TO NEW CONCEPTUAL PLAN	11/01/2023	YZ

AZURECON INC.

GEOTECHNICAL INVESTIGATION
PROPOSED MIXED-USE BUILDING
30 - 48 CHAMBERLAIN AVENUE

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:250
Date: 01/2023
Drawn by: YA
Report No.: PG5332-1 REV.03
Checked by: DP
Dwg. No.: PG5332-1
Approved by: SD
Revision No.: 2

p:\autocad drawings\geotechnical\pg53xx\pg5332\pg5332-1-test hole location plan (rev.02).dwg

APPENDIX 3

APPROXIMATE VIBRATION MONITORING LOCATIONS

PG5332-MEMO.04
Approximate Vibration Monitor Locations

NOTES :

1. ALL MATERIALS AND CONSTRUCTION IS TO BE IN ACCORDANCE WITH THE

2. SUCH SERVICE, UTILITIES AND STRUCTURES IS NOT GUARENTEED. THE CONTRACTOR IS RESPONSIBLE FOR DETERMINING THE EXACT LOCATION, SIZE, MATERIAL AND ELEVATION OF ALL EXISTING SERVICES AND UTILITIES PRIOR TO CONSTRUCTION.

3. THE CONTRACTOR SHALL REPORT ALL CONFLICTS, DISCOVERIES OF ERROR AND DESCREPENCIES TO THE ENGINEER.

4. THE CONTRACTOR SHALL BE RESPONSIBLE TO PROTECT AND ASSUME RESPONSIBILITY FOR ALL UTILITIES WHETHER OR NOT SHOW ON THESE DRAWINGS.

5. THE CONTRACTOR SHALL BE RESPONSIBLE TO PROTECT ALL LANDS BEYOND THE SITE LIMITS. ANY AREAS BEYOND THE SITE LIMITS, WHICH ARE DISTURBED DURING CONSTRUCTION, SHALL BE REPAIRED AND RESTORED TO ORIGINAL CONDITION OR BETTER, TO THE SATISFACTION OF THE ADJACENT LAND OWNER, THE OWNER, THE OWNERS REPRESENTATIVES AND/OR THE AUTHORITY HAVING JURSIDICTION AT THE EXPENSE OF THE CONTRACTOR.

6. WHERE NECESSARY, THE CONTRACTOR SHALL IMPLEMENT A TRAFFIC MANAGEMENT PLAN TO THE SATISFACTION OF THE CITY OF OTTAWA. ALL CONSTRUCTION SIGNAGE MUST CONFORM TO THE LATEST VERSION OF THE M.T.O. MANUAL OF UNIFORM TRAFFIC CONTROL DEVICES. ALL TEMPORARY TRAFFIC CONTROL MEASURES MUST BE REMOVED UPON THE COMPLETION OF THE WORKS.

7. SHOULD ANY BURIED ARCHAEOLOGICAL REMAINS BE FOUND ON THE PROPERTY DURING CONSTRUCTION ACTIVITIES, THE CONTRACTOR SHALL NOTIFY THE OWNER TO CONTACT THE HERITAGE OPERATIONS UNIT OF THE ONTARIO MINISTRY OF CULTURE MUST BE NOTIFIED IMMEDIATE, AND WORK WITHIN THE AREA SHALL BE CEASED UNTIL FUTHER NOTICE.

8. FOR GEOTECHNICAL INFORMATION REFER TO GEOTECHNICAL REPORT PG5332-1, REV 1, 2023-01-12 PREPARED BY PATERSON GROUP.

9. FOR GEODETIC **BENCHMARK** AND GEOMETRIC LAYOUT OF STREET AND LOTS, REFER TO TOPOGRAPHICAL SURVEY AND PLAN OF SUBDIVISION MITH & DENIS SURVEYING LTD BENCHMARK TUAL REFERENCE SYSTEM NETWORK.

10. TION, REFER TO SITE PLAN PREPARED BY HOBIN

11. THESE DRAWINGS ARE NOT TO BE SCALED OR USED FOR LAYOUT PURPOSES

12. THE CONTRACTOR SHALL IMPLEMENT EROSION AND SEDIMENT CONTROL MEASURES TO THE COMMENCEMENT OF ANY SITE CONSTRUCTION. ALL EROSION AND SEDIMENT CONTRAL MEASURES SHALL BE INSTALLED TO THE SATISFACTION OF THE ENGINEER, OR ANY REGULATORY AGENCY. ALL EROSION AND SEDIMENT CONTROL MEASURES SHALL BE MAINTAINED UNTIL VEGETATION IS ESTABLISH OR UNTIL THE START OF A SUBSEQUENT PHASE.

13. CONTRACTORS SHALL BE RESPONSIBLE FOR KEEPING CLEAN ALL ROADS WHICH BECOME COVERED IN DUST, DEBRIS AND/OR MUD AS A RESULT OF ITS CONSTRUCTION OPERATIONS.

14. THE CONTRACTOR SHALL BE RESPONSIBLE FOR ANY ADDITIONAL BEDDING OR ADDITIONAL STRENGTH PIPE SHOULD THE MAXIMUM OPSD TRENCH WIDTH BE EXCEEDED.

15. ALL PIPE, CULVERTS, STRUCTURES REFER TO NOMINAL INSIDE DIMENSIONS.

16. SHOULD CLAY SEALS BE REQUIRED, THEY SHALL BE INSTALLED AS PER THE RECOMMENDATIONS WITHIN THE GEOTECHNICAL REPORT.

17. UNLESS SPECIFICALLY NOTED OTHERWISE, PIPE MATERIALS SHALL BE AS FOLLOWS;
    -WATERMAINS TO BE PVC DR18
    -SANITARY SEWER TO BE PVC DR35
    -PERFORATED STORM SEWERS IN REAR YARDS AND LANDSCAPE AREAS TO BE HDPE
    -STORM SEWERS 375MM DIAMETER AND LESS TO BE PVC DR35
    -STORM SEWERS 450MM DIAMETER AND GREATER TO BE CONCRETE, CLASS AS PER OPSD 807.010 OR 807.030, OR HIGHER

18. ALL CONNECTIONS TO EXISTING WATERMAINS ARE TO BE COMPLETED BY CITY FORCES. CONTRACTOR IS TO EXCAVATE, BACKFILL, COMPACT AND REINSTATE.

19. ANY WATERMAIN WITH LESS THAN 2.4M, AND ANY SEWER WITH LESS THAN 2.0M DEPTH OF COVER REQUIRES THERMAL INSULATION AS PER CITY OF OTTAWA STANDARD W22, OR AS APPROVED BY THE ENGINEER.

20. ALL FIRE HYDRANTS AS PER CITY STANDARD W19, c/w 150mmØ LEAD UNLESS OTHERWISE SPECIFIED.

21. ALL STUBBED SEWERS SHALL HAVE PRE-MANUFACTURED CAPS INSTALLED.

22. ALL CATCHBASINS SHALL HAVE A 600MM SUMP. ALL CATCHBASIN MANHOLES, AND ALL STORM MANHOLES WITH OUTLETTING PIPE SIZES LESS THAN 900MM, SHALL HAVE A 300MM SUMP.

23. ALL SANITARY MANHOLES SHALL BE EQUIPPED WITH A WATERTIGHT COVER.

24. EACH BUILDING SHALL BE EQUIPPED WITH A SANITARY AND STORM SEWER BACKWATER VALVE AND CLEAN-OUT ON ITS PRIMARY SERVICE, AS PER ONTARIO BUILDING CODE REQUIREMENTS (BY OTHERS).

25. THE SUBGRADE OF ALL STRUCTURES, PIPE, ROADS, SIDEWALKS, WALKWAYS, AND BUILDINGS SHALL BE INPSECTED BY THE GEOTECHNICAL ENGINEER PRIOR TO PROCEEDING WITH CONSTRUCTION.

26. UPON COMPLETION OF THE RETAINING WALL, THE CONTRACTOR SHALL REQUEST A CONFORMANCE CERTIFICATE FROM THE QUALIFIED ENGINEER RESPONSIBLE FOR THE WALL DESIGN.

CONTRACTOR TO CONFIRM ELEVATIONS OF ALL EXISTING CITY INFRASTRUCTURE BEFORE COMMENCING WORK.

CONTRACTOR TO SUPPORT ALL UNDERGROUND CITY INFRASTRUCTURE DURING CONSTRUCTION

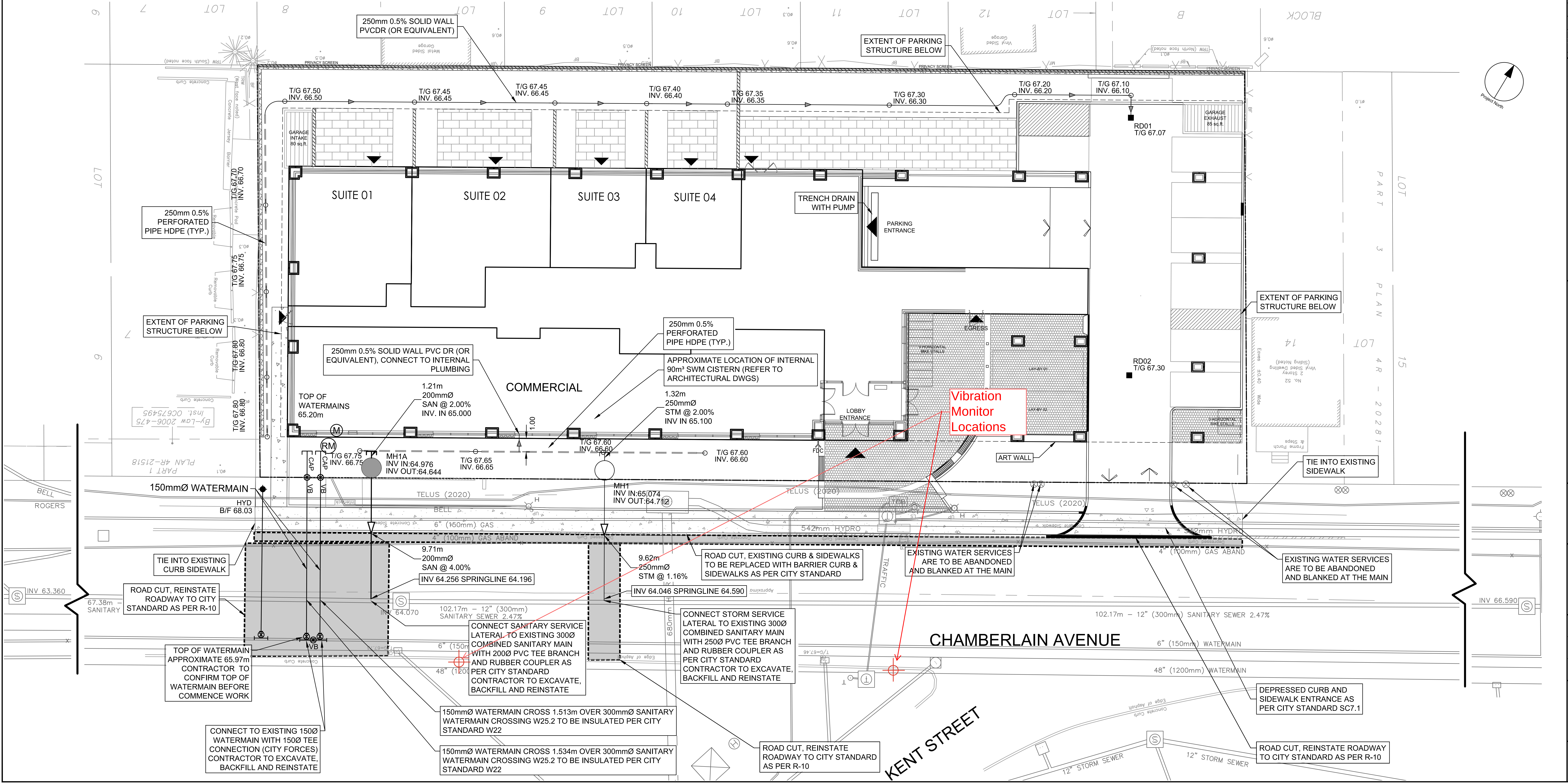
ALL EXISTING SERVICES TO CITY INFRASTRUCTURE ARE TO BE ABANDONED.
SEWER SERVICES ARE TO BE CAPPED AT THE PROPERTY LINE AND WATER SERVICES ARE TO BE BLANKED AT THE MAIN.

SERVICING LEGEND

● MH118A	SANITARY MANHOLE
200mmØ SAN	SANITARY SEWER
MH109 ○ □ MH118	STORM MANHOLE
825mmØ STM	STORM SEWER - LESS THAN 900Ø
200Ø WATERMAIN	WATERMAIN
■ CB100 T/G 104.10	STREET CATCHBASIN C/W TOP OF GRATE
T/G 104.35 INV 103.35	REAR YARD "TEE" CATCHBASIN (300Ø) C/W TOP OF GRATE AND INVERT OUT
T/G 104.50 INV 103.50	REAR YARD "END" CATCHBASIN (300Ø) C/W TOP OF GRATE AND INVERT OUT
T/G 104.35 INV 103.35	REAR YARD "CUSTOM ANGLED " CATCHBASIN (450Ø) C/W TOP OF GRATE AND INVERT OUT
T/G 104.35 INV 103.35	REAR YARD "THREE WAY" CATCHBASIN (450Ø) C/W TOP OF GRATE AND INVERT OUT
⊗ VB	VALVE AND VALVE BOX
HYD 104.35	FIRE HYDRANT C/W BOTTOM OF FLANGE ELEVATION
2 VBENDS	VERTICAL BEND LOCATION
	SIAMESE CONNECTION (IF REQUIRED)
Ⓜ	METER (IF REQUIRED)
RM	REMOTE METER (IF REQUIRED)
▼	DOOR ENTRANCE
■ RD01	ROOF DRAIN C/W TOP OF GRATE

EXISTING LEGEND

Inv.	INVERT
T/G	TOP OF GRATE
U/Eave	UNDERSIDE OF EAVE
TpFdn	TOP OF FOUNDATION
C/L	CENTRELINE
65.00	LOCATION OF ELEVATIONS
65.00	TOP OF CONCRETE CURB ELEVATION
City	PLAN 6943P&P02
○ UP	UTILITY POLE
○ AN	ANCHOR
○ LS	LIGHT STANDARD
□ CB	CATCH BASIN
○ SP	WATER STAND POST
▭ GM	GAS METER
○ B	BOLLARD
△ S	SIGN
AC	AIR CONDITIONER
BF	BOARD FENCE
MF	METAL FENCE
CRW	CONCRETE RETAINING WALL
TRW	TIMBER RETAINING WALL
·	DECIDUOUS TREE
⊗	WATER VALVE, VALVE CHAMBER, FIRE HYDRANT
Ⓢ	SEWER MANHOLE, CATCH BASIN MANHOLE
□	CATCH BASIN / DRAINAGE, WING WALL, HEAD WALL
○	POLE, POLE W/ LIGHT, DECORATIVE, LAWN LIGHT
P/S PED GR	POWER SUPPLY, PANEL, PEDESTAL, TRANSFORMER, TOWER, REGULATOR
AMP	AMP, HAND HOLE, VAULT, GAS VALVE
BUS BUS BUS	OC TRANSPO: BUS SHELTER-NO POWER, ENERGIZED, ISOLATED
TGS	STREETSCAPE: PLANTER BOX, GRATE SQUARE, ENG. SOIL
TCB TDB SDB	TRAFFIC CONNECT BOX / DISCONNECT BOX, SL DISCONNECT



CLIENT
SCARABELLI REALTIES INC.

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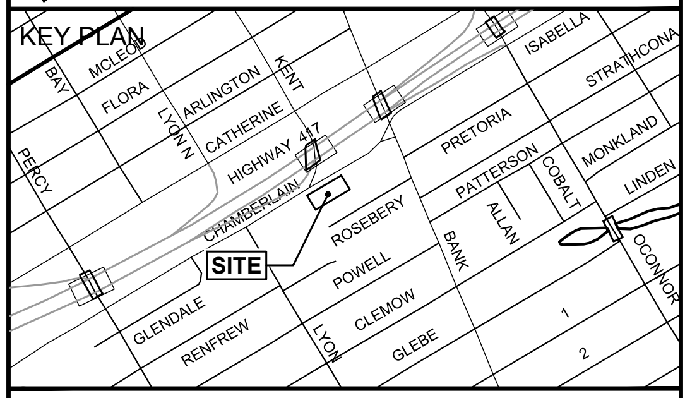
ISSUES

No.	DESCRIPTION	DATE
01	ISSUED FOR RE-ZONING APPLICATION	20:11:13
02	REVISED AS PER CITY COMMENTS	24:05:28

NOT FOR CONSTRUCTION

KEY PLAN



CONSULTANTS

1:150 0 1.5 4.5 7.5m

SEAL
T. R. BRULE
2024/05/28

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PROJECT
30-48 CHAMBERLAIN AVE.

PROJECT NO:
125564

DRAWN BY:	CHECKED BY:
EH	**TRB**
PROJECT MGR:	APPROVED BY:
TRB	**TRB**

SHEET TITLE
SERVICING PLAN

SHEET NUMBER
001

ISSUE
1

File Location: J:\125564_Chamberlain\7.0_Production\7.03_Design\04_Civil\Sheets\001 SERVICING PLAN.dwg Last Saved: May 31, 2024, by arthur.beresniewicz Plotted: Friday, May 31, 2024 3:13:05 PM by Arthur Beresniewicz

SCALE CHECK 1 in 10mm

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