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Geotechnical Engineering

Environmental Engineering

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Materials Testing

Building Science

Noise and Vibration Studies

Geotechnical Investigation

Proposed Mixed-Use Development
2475 Regina Street
Ottawa, Ontario

Prepared For

Parkway House Development Fund LP

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Report: PG5901-1
Revision 2

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Parkway House Development Fund LP to conduct a geotechnical investigation for the proposed mixed-use development to be located at 2475 Regina Street in the City of Ottawa (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objectives of the geotechnical investigation were to:

- Determine the subsoil and groundwater conditions at this site by means of boreholes.
- 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. It contains our findings and includes geotechnical recommendations pertaining to the design and construction of the subject development as they are understood at the time of writing this report.

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

2.0 Proposed Development

Based on the available drawings, it is understood that the proposed development will consist of a mid-rise building (Building A1), located within the northwest corner of the site, and two high-rise buildings (T1 and T2) which are to be located within the central and eastern portions of the site and share two levels of underground parking. Associated access lanes, walkways, and landscaped areas are also proposed at finished grades. It is also expected that the proposed buildings will be municipally serviced.

Construction of the proposed development will involve demolition of the existing building presently located at the site.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current geotechnical investigation was carried out from July 29 to August 3, 2021, and consisted of advancing a total of seven (7) boreholes to a maximum depth of 17.5 m below existing grade. The test hole locations were distributed in a manner to provide general coverage of the subject site and taking into consideration underground utilities and site features. The borehole locations are shown on Drawing PG5901-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were drilled using a track-mounted 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 drilling to the required depths at the selected locations, and sampling and testing the overburden.

Sampling and In Situ Testing

Soil samples were recovered 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. Rock cores (RC) were obtained using a 47.6 mm inside diameter coring equipment. All samples were visually inspected and initially classified on site. The auger and split-spoon samples were placed in sealed plastic bags, and rock cores were placed in cardboard boxes. All samples were transported to our laboratory for further examination and classification. The depths at which the auger, split spoon and rock core samples were recovered from the boreholes are shown as AU, SS and RC, respectively, on the Soil Profile and Test Data sheets presented in Appendix 1.

A 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.

Bedrock samples were recovered at borehole BH 1-21 using a core barrel and diamond drilling techniques. The depths at which rock core samples were recovered from the boreholes are shown as RC on the Soil Profile and Test Data sheets in Appendix 1.

A recovery value and a Rock Quality Designation (RQD) value were calculated for each drilled section (core run) of bedrock and are shown on the borehole logs. The recovery value is the ratio, in percentage, of the length of the bedrock sample recovered over the length of the drilled section (core run). 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. These values are indicative of the quality of the bedrock.

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

Groundwater

Monitoring wells were installed in boreholes BH 1-21, BH 6-21, and BH 7-21. The remaining boreholes were fitted with flexible polyethylene standpipes 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:

- ❑ Slotted 32 mm diameter PVC screen at the base of each borehole.
- ❑ 51 mm diameter PVC riser pipe from the top of the screen to the ground surface.
- ❑ No. 3 silica sand backfill within annular space around screen.
- ❑ Bentonite hole plug 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 will be stored in the laboratory for a period of one (1) month after issuance of this report. They will then be discarded unless we are otherwise directed.

3.2 Field Survey

The borehole locations were selected by Paterson to provide general coverage of the proposed development, taking into consideration the existing site features and underground utilities. The test hole locations and ground surface elevation at each test hole location were surveyed by Paterson using a handheld GPS and referenced to a geodetic datum. The location of the boreholes and ground surface elevation at each test hole location are presented on Drawing PG5901-1 - 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 corrosion potential for exposed ferrous metals and the potential of sulphate attacks against subsurface concrete structures. The sample was submitted to determine the concentration of sulphate and chloride, the resistivity, and the pH of the samples. The results are presented in Appendix 1 and are discussed further in Section 6.7.

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4.0 Observations

4.1 Surface Conditions

The eastern portion of the subject site is occupied by a single-storey building and associated asphalt-paved access lanes and parking areas. The remainder of the subject site general consists of grassed areas with mature trees.

The site is bordered by the Sir John A. Macdonald Parkway to the east, vacant land and paved walking pathways to the north, Regina Street and residential dwellings to the west, and a multi-storey building followed by Richmond Road to the south. The ground surface across the majority of the subject site is relatively level and at-grade with Regina Street at approximate geodetic elevation 66.0 m, however, within the northwest corner of the site, the grade slopes downward gently from southeast to northwest from approximate geodetic elevation 66.0 to 63.5 m.

4.2 Subsurface Profile

Overburden

Generally, the soil profile at the borehole locations consists of a topsoil layer underlain by an approximate 0.8 to 1.8 m thick fill layer. The fill material was generally observed to consist of brown silty sand and/or clay with gravel, cobbles, boulders and varying amounts of topsoil and organics.

A hard to very stiff brown silty brown silty clay deposit was observed underlying the fill at BH 2-21.

A glacial till deposit was observed underlying either the fill or silty clay deposit at all boreholes at depths ranging from approximately 0.8 to 3.4 m below the existing ground surface. The glacial till deposit was generally observed to consist of a brown to grey silty sand to silty clay with gravel, cobbles, and boulders. Boulders were cored from approximate depths of 7.6 to 11.5 m at borehole BH 1-21 in order to advance the borehole.

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

Bedrock

The bedrock was cored at borehole BH 1-21 commencing at an approximate depth of 13.8 and extending to a depth of 17.5 m. The bedrock was observed to consist of grey quartz sandstone and, based on the recovered bedrock core, was generally weathered and of poor quality to an approximate depth of 14.4 m, becoming fair to good in quality with depth.

Based on available geological mapping and coring records, the bedrock in the subject area consists of Paleozoic shale of the Rockcliffe formation, with an overburden drift thickness of 5 to 15 m.

4.3 Groundwater

Groundwater levels were measured on August 11, 2021 within the installed monitoring wells and standpipes. The measured groundwater levels noted at that time are presented in Table 1 below

Table 1 – Summary of Groundwater Levels

Test Hole Number	Ground Surface Elevation (m)	Measured Groundwater Level		Dated Recorded
		Depth (m)	Elevation (m)	
BH 1-21*	66.09	7.74	58.35	August 11, 2021
BH 2-21	65.81	3.55	62.26	
BH 3-21	64.98	6.72	58.26	
BH 4-21	64.59	7.12	57.47	
BH 5-21	63.84	Dry	-	
BH 6-21*	63.62	5.20	58.42	
BH 7-21*	65.14	Dry	-	

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

The groundwater can also be estimated based on the colouring, consistency and moisture levels of the recovered samples. Based on these observations, the long-term groundwater table can be expected at approximately 4 to 5 m below ground surface. The recorded groundwater levels are also provided 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 levels 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 high-rise buildings be founded on raft foundations placed on the undisturbed, compact to dense glacial till deposit. It is further recommended that the mid-rise building, as well as the portions of the underground parking levels which extend beyond the footprints of the high-rise buildings, be supported on conventional spread footings bearing on the undisturbed, compact to dense glacial till deposit.

Where loose and/or soft glacial till is encountered at the underside of footing or raft, it should be sub-excavated to the undisturbed, compact to dense glacial till and re-instated with engineered fill.

Further, it is anticipated that cobbles and boulders will be encountered frequently throughout servicing trenches and building excavations. All contractors should be prepared for the removal of boulders and potentially oversized boulders throughout the subject site.

The above and other considerations are 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, pipe bedding and other settlement sensitive structures.

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

Protection of Subgrade (Raft Foundation)

Where a raft foundation is used, the raft subgrade would consist of a glacial till deposit, and it is recommended that a minimum 75 mm thick lean concrete mud slab be placed on the undisturbed glacial till subgrade shortly after the completion of the excavation. The main purpose of the mud slab is to reduce the risk of disturbance of the subgrade under the traffic of workers and equipment.

The final excavation to the raft bearing surface level and the placing of the mud slab should be done in smaller sections to avoid exposing large areas of the glacial till to potential disturbance due to drying.

Fill Placement

Fill placed for grading beneath the proposed buildings should consist, unless otherwise specified, of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. The imported fill material should be tested and approved prior to delivery. The fill should be placed in maximum 300 mm thick loose lifts and compacted by suitable compaction equipment. Fill placed beneath the building should be compacted to a minimum of 98% of the standard Proctor maximum dry density (SPMDD).

Non-specified existing fill along with site-excavated soil could be placed as general landscaping fill and beneath exterior parking where settlement of the ground surface is of minor concern. These materials should be spread in lifts with a maximum thickness of 300 mm and 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 placement as backfill against foundation walls, unless used in conjunction with a geocomposite drainage membrane, such as Miradrain G100N or Delta Drain 6000, connected to a perimeter drainage system.

5.3 Foundation Design

Conventional Spread Footings – Building A1

For the mid-rise building (Building A1), it is recommended that conventional spread footings placed on an undisturbed, compact to dense glacial till bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **200 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **300 kPa**. A geotechnical resistance factor of 0.5 was applied to the above noted bearing resistance value at ULS.

Conventional Spread Footings – Buildings T1 & T2

Conventional spread footings for buildings with more than one underground parking level (Buildings T1 & T2) will be founded on an undisturbed, dense to very dense glacial till bearing surface and can be designed using a bearing resistance value at serviceability limit states (SLS) of **300 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **450 kPa**. A geotechnical resistance factor of 0.5 was applied to the above noted bearing resistance value at ULS.

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

Footings bearing on an undisturbed soil bearing surface and designed using the bearing resistance values provided above will be subjected to potential post-construction total and differential settlements of 25 to 20 mm, respectively.

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 an undisturbed glacial till bearing surface, above the groundwater table, when a plane extending down and out from the bottom edge of the footing at a minimum of 1.5H:1V passes only through in situ soil of the same or higher capacity as the bearing medium soil.

Raft Foundation - High-Rise Buildings with Two Underground Parking Levels

For the proposed high-rise buildings, where the spread footing bearing capacity is not sufficient to support the imposed structural loads, then consideration could be given to using a raft foundation for foundation support of these proposed buildings. For 2 levels of underground parking, it is anticipated that the excavation will extend to a depth such that the underside of the raft slab would be placed between approximate geodetic elevations of 57 to 55 m.

The maximum SLS contact pressure is **400 kPa** for a raft foundation bearing on the undisturbed, compact to dense glacial till. It should be noted that the weight of the raft slab and everything above has to be included when designing with this value. The loading conditions for the contact pressure are based on sustained loads, that are generally taken to be 100% Dead Load and 50% Live Load. The factored bearing resistance (contact pressure) at ULS can be taken as **600 kPa**. A geotechnical resistance factor of 0.5 was applied to the bearing resistance value at ULS.

The modulus of subgrade reaction was calculated to be **16 MPa/m** for a contact pressure of 400 kPa. The design of the raft foundation is required to consider the relative stiffness of the reinforced concrete slab and the supporting bearing medium. A common method of modeling the soil structure interaction is to consider the bearing medium to be elastic and to assign a subgrade modulus. However, glacial till is not elastic and limits have to be placed on the stress ranges of a particular modulus.

The proposed buildings can be designed using the above parameters with total and differential settlements of 25 and 20 mm, respectively.

5.4 Design for Earthquakes

Shear wave velocity testing was completed at the subject site to accurately determine the applicable seismic site classification for the proposed development in accordance with the latest revision of 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 in Appendix 2.

Field Program

The seismic array testing location was placed within the central area of the site in an approximate north-south direction as presented in Drawing PG5901-1 - Test Hole Location Plan in Appendix 2. Paterson field personnel placed 24 horizontal 2.4 Hz. geophones mounted to the surface by means of two 75 mm ground spikes attached to the geophone land case. The geophones were spaced at 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) and eight (8) times at each shot location to improve signal to noise ratio.

The shot locations are also completed in forward and reverse direction (i.e.- striking both sides of the I-Beam seated parallel to the geophone array). The shot locations were 15, 3, and 2 m away from the first and last geophones, and at the centre of the seismic array.

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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 finished grade. 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.

Seismic Site Class

For this scenario, the V_{S30} was calculated as follows:

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

$$V_{s30=} \frac{30\ m}{\left(\frac{14\ m}{420\ m/s} + \frac{16\ m}{2{,}464\ m/s}\right)}$$

$$V_{s30=}\ 753\ m/s$$

The average shear wave velocity, V_{s30}, is **753 m/s**, therefore, as per OBC 2024, a **Site Designation X_{753}** is applicable for seismic design of the proposed buildings. 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 will be considered an acceptable subgrade on which to commence backfilling for floor slab construction. It is understood that the underground level(s) 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 encountered during the field investigation, a sub-slab drainage system, consisting of lines of perforated drainage pipe subdrains connected to a sump pit, should be provided in the subfloor fill under the lower basement floor (discussed further in Subsection 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 calculating the effective unit weight.

Lateral Earth Pressures

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

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

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

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

Seismic Earth Pressures

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

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

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

The peak ground acceleration (a_{max}) for the Ottawa area is 0.32 g 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.

The anchor should be provided with a bonded length at the base of the anchor which will provide the anchor capacity, as well an unbonded length between the rock surface and the top of the bonded length.

Permanent anchors should be provided with corrosion protection. As a minimum, the entire drill hole should be filled with cementious 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, the 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 sandstone ranges between about 50 and 80 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. Based on existing bedrock 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

Parameters used to calculate rock anchor lengths are provided in Table 2 on the following page:

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 Sandstone Hoek and Brown parameters	65 m=0.575 and s=0.00293
Unconfined compressive strength - Limestone bedrock	50 MPa
Unit weight - Submerged Bedrock	15.5 kN/m^3
Apex angle of failure cone	60^o
Apex of failure cone	mid-point of fixed anchor length

The fixed anchor length will depend on the diameter of the drill holes. Recommended anchor lengths for a 75 mm and 125 mm diameter hole are provided in Table 3 on the next page. The factored tensile resistance values given in Table 2 are based on a single anchor with no group influence effects. A detailed analysis of the anchorage system, including potential group influence effects, could be provided once the details of the loading for the proposed building are determined.

Table 3 - Recommended Rock Anchor Lengths - Grouted Rock Anchor

Diameter of Drill Hole (mm)	Anchor Lengths (m)			Factored Tensile Resistance (kN)
	Bonded Length	Unbonded Length	Total Length	
75	2.0	0.8	2.8	450
	2.6	1.0	3.6	600
	3.2	1.3	4.5	750
	4.5	2.0	6.5	1000
125	1.6	1.0	2.6	600
	2.0	1.2	3.2	750
	2.6	1.4	4.0	1000
	3.2	1.8	5.0	1250

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.

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 that grout is prepared.

5.8 Pavement Design

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 on the following page. The flexible pavement structure presented in Tables 5 and 6 should be used for exterior, at grade parking areas and access lanes, respectively.

Table 4 - Recommended Rigid Pavement Structure - Lower 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 – Imported fill or OPSS Granular B Type **I** or **II** or material placed over in situ soil.	

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 hours after the concrete has been poured during warm temperatures and up to 12 hours during cooler temperatures.

Table 5 - Recommended Pavement Structure – Car Only Parking Areas

Thickness (mm)	Material Description
50	**Wear Course** - HL-3 or Superpave 12.5 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
300	**SUBBASE** - OPSS Granular B Type **II**
SUBGRADE - OPSS Granular B Type **I** or **II** placed over in-situ soil, or concrete fill.	

Table 6 - Recommended Asphalt Pavement Structure - Access Lanes and Heavy Loading Parking Areas

Thickness (mm)	Material Description
40	**Wear Course** – HL-3 or Superpave 12.5 Asphaltic Concrete
50	**Wear Course** – HL-8 or Superpave 19.0 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
300	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - OPSS Granular B Type I or II placed over in-situ soil, or concrete fill.	

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for this project. The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 99% of the material's SPMDD using suitable vibratory equipment, noting that excessive compaction can result in subgrade softening.

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

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Water Suppression System and Foundation Drainage

To limit long-term groundwater lowering, it is recommended that a groundwater infiltration control system be designed for proposed buildings with footing elevations below the long-term groundwater table. Waterproofing of the foundation wall is recommended, and the membrane is to be installed starting at 4 m below grade down to the founding elevation. The waterproofing membrane should also be extended horizontally below the proposed footings a minimum of 600 mm away from the face of the excavation. The membrane will serve as a water infiltration suppression system. Specific waterproofing recommendations and design can be provided for the proposed multi-storey buildings once detailed foundation design drawings are available.

It is also recommended that the 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 (underside of footing or raft slab). 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 footing or raft slab interface to allow the infiltration of water to flow to an interior perimeter underslab drainage pipe. The perimeter drainage pipe should direct water to sump pit(s) within the lower basement area.

Foundation Raft Slab Construction Joints

It is expected that the raft slab, where utilized, will be poured in sections. For the construction joint at each pour, a rubber water stop along with a chemical grout (Xypex or equivalent) should be applied to the entire vertical joint of the slab. Furthermore, a rubber water stop should be incorporated in the horizontal interface between the foundation wall and the raft slab.

Sub-slab Drainage

Sub-slab drainage will be required to control water infiltration below the lowest underground parking level slab for the proposed buildings. For design purposes, it is recommended that a 150 mm diameter perforated pipe be placed at 6 m centres.

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.

Podium Deck Tie-In for Waterproofing System

It is expected that a waterproofing system will be provided for the podium deck surface. It is recommended that the podium deck waterproofing system consist of a layer of hot rubber membrane applied to the concrete surface. The concrete should be cleaned of any dust, dirt, or debris prior to the application of the hot rubber. The hot rubber should be overlain by a 50 mm thick layer of HI-60 rigid insulation, or equivalent, and further overlain by a foundation drainage board (6000 series by DeltaDrain, G100N MiraDrain, or approved equivalent installed with the geotextile side facing up.

The hot rubber should be applied to the geotextile side of the drainage board to cover the cold joint a minimum of 150 mm. A termination bar should be installed as per manufacture's specifications. The podium deck drainage board can then be overlapped to cover the cold joint a minimum of 150 mm. Further details can be provided once the final design drawings are made available.

Foundation Backfill

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 relatively 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 1, Granular A or Granular B Type II granular material, should otherwise be used for this purpose.

Sidewalks and Walkways

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

6.2 Protection of Footings 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 should be provided for adequate frost protection for heated structures.

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 heated structure and require additional protection, such as soil cover of 2.1 m or an equivalent combination of soil cover and foundation insulation.

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

6.3 Excavation Side Slopes

The side slopes of the excavation should either be cut back at acceptable slopes or be retained by shoring systems from the beginning of the excavation until the structure is backfilled. However, for most of the site, insufficient room will be available to permit the building excavation to be constructed by open-cut methods (i.e., unsupported excavations).

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.

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

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

It is recommended that a trench box be used at all times to protect personnel working in trenches with steep or vertical sides. It is expected that services will be installed by “cut and cover” methods and excavations will not be left open for extended periods of time.

Temporary Shoring

Due to the anticipated proximity of the proposed development to the property boundaries, temporary shoring may be required to support the overburden soils. The shoring requirements will depend on the depth of the excavation and the proximity of the adjacent structures. However, it should be noted that the observed bouldery conditions can lead to the creation of voids and other unstable conditions during installation of the temporary shoring as boulders shift within the fine soil matrix. Furthermore, it may be difficult to develop the required anchor strength in soil due to variations in soil conditions.

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 representative prior to implementation.

The temporary shoring system may consist of a soldier pile and lagging system. The shoring system is recommended to be adequately supported to resist toe failure. 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 temporary shoring system may be calculated using the parameters outlined in Table 7 on the next page.

Table 7 - Soil Parameters for Calculating Earth Pressures Acting on Shoring System

Parameter	Value
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	21
Submerged 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 used above the groundwater level while the effective unit weight should be used below the groundwater level.

The hydrostatic groundwater pressure should be added to the earth pressure distribution wherever the effective unit weights are used for earth pressure calculations. If the groundwater level is lowered, the dry unit weight for the soil should be used full weight, with no hydrostatic groundwater pressure component. For design purposes, the minimum factor of safety of 1.5 should be calculated.

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. At least 150 mm of OPSS Granular A should be used for pipe bedding for sewer and water pipes. 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 or Granular B Type II with a maximum size of 25 mm. The bedding and cover materials should be placed in maximum 225 mm thick lifts compacted to 99% 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 potential differential frost heaving. The trench backfill should be placed in maximum 300 mm thick loose lifts and compacted to a minimum of 95% of the material's SPMDD.

6.5 Groundwater Control

Groundwater Control for Building Construction

It is anticipated that groundwater infiltration into the excavations should be controllable using open sumps. 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.

Permit to Take Water

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 4 to 5 months should be allowed for completion of the PTTW application package and issuance of the permit by the MECP.

For typical ground or surface water volumes being pumped during the construction phase, typically between 50,000 to 400,000 L/day, it is required to register on the Environmental Activity and Sector Registry (EASR). A minimum of two to four weeks should be allotted for completion of the EASR registration and the Water Taking and Discharge Plan to be prepared by a Qualified Person as stipulated under O.Reg. 63/16. If a project qualifies for a PTTW based upon anticipated conditions, an EASR will not be allowed as a temporary dewatering measure while awaiting the MECP review of the PTTW application.

Impacts on Neighbouring Properties

Since the proposed development will be founded below the long-term groundwater level, a waterproofing membrane system has been recommended to lessen the effects of water infiltration. Any long-term dewatering of the site will therefore be minimal and will have no adverse effects to the surrounding buildings or structures. The short-term dewatering during the excavation program, which is expected to be minimal, will be managed by the excavation contractor.

6.6 Winter Construction

Precautions must be taken if winter construction is considered for this project. The subsoil conditions at this site consist of frost susceptible materials. In the presence of water and freezing conditions, ice could form within the soil mass. Heaving and settlement upon thawing could occur.

In the event of construction during below zero temperatures, the founding stratum should be protected from freezing temperatures 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.

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

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

6.7 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 moderate to aggressive corrosive environment.

7.0 Recommendations

It is recommended that the following be completed once the master plan and site development are determined.

- ❑ Review of the geotechnical aspects of the excavation contractor's shoring design, if required, prior to construction.
- ❑ Review of waterproofing details for the elevator shaft and building sump pits.
- ❑ Review and inspection of the foundation waterproofing system and all foundation drainage systems.
- ❑ Observation of all bearing surfaces prior to the placement of concrete.
- ❑ Sampling and testing of the concrete and fill materials.
- ❑ Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- ❑ Complete a full inspection program of the installation of the perimeter and underground floor drainage system during construction.
- ❑ Observation of all subgrades prior to backfilling.
- ❑ Field density tests to determine the level of compaction achieved.
- ❑ Sampling and testing of the bituminous concrete including mix design reviews.

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

8.0 Statement of Limitations

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

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

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

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

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
June 13, 2025
S. S. DENNIS
100519516
PROVINCE OF ONTARIO

Kevin A. Pickard, P.Eng.

Scott S. Dennis, P.Eng.

Report Distribution:

- ❑ Parkway House Development Fund LP (email copy)
- ❑ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Use Development - Parkway House
2475 Regina Street, Ottawa, Ontario

DATUM Geodetic

FILE NO. PG5901

REMARKS

HOLE NO. BH 1-21

BORINGS BY Track-Mount Power Auger **DATE** July 29, 2021

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	66.09
TOPSOIL 0.15							
FILL: Brown silty clay, trace sand topsoil 0.91		SS	1	42	9		
GLACIAL TILL: Compact, brown silty sand, some clay, gravel, cobbles and boulders		SS	2	75	23	1	65.09
		SS	3	42	14	2	64.09
- clay content increasing with depth		SS	4	83	11		
4.00		SS	5	33	13	3	63.09
GLACIAL TILL: Very stiff, brown silty clay, some sand, gravel, cobbles and boulders		SS	6	83	9	4	62.09
		SS	7	33	8	5	61.09
- grey by 4.6m depth		SS	8	50	4	6	60.09
7.00		SS	9	75	6		
GLACIAL TILL: Compact to dense, grey silty sand, some clay, gravel, cobbles and boulders		SS	10	75	19	7	59.09
		RC	1	50		8	58.09
		RC	2	21		9	57.09
		RC	3	50		10	56.09
						11	55.09

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

○ Water Content %

20 40 60 80

Monitoring Well Construction

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Use Development - Parkway House
2475 Regina Street, Ottawa, Ontario

DATUM Geodetic

FILE NO. PG5901

REMARKS

HOLE NO. BH 1-21

BORINGS BY Track-Mount Power Auger **DATE** July 29, 2021

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						11	55.09
GLACIAL TILL: Compact to dense, grey silty sand, some clay, gravel, cobbles and boulders - cored through boulders from 7.62 to 13.79m depth.						12	54.09
						13	53.09
13.79							
BEDROCK: Poor to good quality, grey quartz sandstone		RC	4	100	26	14	52.09
		RC	5	100	59	15	51.09
						16	50.09
		RC	6	100	81	17	49.09
17.47							
End of Borehole (GWL @ 7.74m - August 11, 2021)							

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

○ Water Content %

20 40 60 80

Monitoring Well Construction

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

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Use Development - Parkway House
2475 Regina Street, Ottawa, Ontario

DATUM Geodetic

FILE NO. PG5901

REMARKS

HOLE NO. BH 2-21

BORINGS BY Track-Mount Power Auger **DATE** July 30, 2021

SOIL DESCRIPTION	Strata depth (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
TOPSOIL	0.15				
FILL: Brown silty sand, trace gravel and organics	1.22	SS	1	42	10
		SS	2	12	10
Hard to very stiff, brown **SILTY CLAY,** some sand	3.35	SS	3	83	10
		SS	4	83	8
GLACIAL TILL: Brown silty clay with sand, gravel, cobbles and boulders - grey by 4.6m depth	6.70	SS	5	75	9
		SS	6	75	28
		SS	7	58	11
		SS	8	58	10
		SS	9	62	18

DEPTH (m)	ELEV. (m)
0	65.81
1	64.81
2	63.81
3	62.81
4	61.81
5	60.81
6	59.81

End of Borehole

Practical refusal to augering at 6.70m depth

(GWL @ 3.55m - August 11, 2021)

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

○ Water Content %

20 40 60 80

Piezometer Construction

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

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Use Development - Parkway House
2475 Regina Street, Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE July 30, 2021

FILE NO. PG5901

HOLE NO. BH 3-21

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	64.98
TOPSOIL 0.20							
FILL: Brown silty sand with cobbles, occasional boulders 0.91		AU	1				
		SS	2	75	18	1	63.98
		SS	3	62	28	2	62.98
		SS	4	75	16		
		SS	5	79	32	3	61.98
		SS	6	62	19	4	60.98
GLACIAL TILL: Compact to dense, brown silty sand with gravel, cobbles and boulders, some clay - clay content decreasing with depth - grey by 4.6m depth		SS	7	62	10	5	59.98
		SS	8	62	21	6	58.98
		SS	9	75	30	7	57.98
		SS	10	73	82	8	56.98
9.75		SS	11	83	18	9	55.98
End of Borehole (GWL @ 6.72m - August 11, 2021)							

Pen. Resist. Blows/0.3m
● 50 mm Dia. Cone
○ Water Content %
20 40 60 80

Piezometer Construction

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

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Use Development - Parkway House
2475 Regina Street, Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE July 30, 2021

FILE NO. PG5901

HOLE NO. BH 4-21

SOIL DESCRIPTION	Strata depth (m)	TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	64.59
TOPSOIL	0.20						
FILL: Brown silty sand with gravel	0.76	AU	1				
GLACIAL TILL: Compact to dense, brown silty sand with gravel, cobbles and boulders, trace to some clay		SS	2	75	7	1	63.59
- clay content decreasing with depth		SS	3	33	32	2	62.59
- grey by 5.2m depth		SS	4	83	16	3	61.59
		SS	5	21	29	4	60.59
		SS	6	75	23	5	59.59
		SS	7	17	29	6	58.59
		SS	8	67	21	7	57.59
		SS	9	25	46	8	56.59
	8.13	SS	10	70	80		
End of Borehole							
(GWL @ 7.12m - August 11, 2021)							

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

○ Water Content %

20 40 60 80

Piezometer Construction

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Use Development - Parkway House
2475 Regina Street, Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE July 30, 2021

FILE NO. PG5901

HOLE NO. BH 5-21

SOIL DESCRIPTION	Strata depth (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	63.84
TOPSOIL	0.20						
FILL: Brown silty sand with gravel and cobbles, trace clay and topsoil		AU	1				
		SS	2	21	15	1	62.84
	1.45						
GLACIAL TILL: Compact to very dense, brown silty sand with gravel, cobbles and boulders, some to trace clay		SS	3	25	19		
						2	61.84
						3	60.84
		SS	4	50	50+		
						4	59.84
		SS	5	42	82		
						5	58.84
						6	57.84
	6.55	SS	6	56	50+		
End of Borehole							
(BH dry - August 11, 2021)							

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

○ Water Content %

20 40 60 80

Piezometer Construction

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

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Use Development - Parkway House
2475 Regina Street, Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE August 3, 2021

FILE NO. PG5901

HOLE NO. BH 6-21

SOIL DESCRIPTION	Layer depth (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	63.62
TOPSOIL	0.20						
FILL: Brown silty sand with clay, gravel, cobbles, trace topsoil		AU	1				
	1.22	SS	2	50	9	1	62.62
GLACIAL TILL: Compact to dense, brown silty sand with gravel, cobbles and boulders, trace clay		SS	3	50	18	2	61.62
		SS	4	83	11	3	60.62
		SS	5	83	14		
- grey by 4.3m depth		SS	6	50	23	4	59.62
		SS	7	83	23	5	58.62
		SS	8	83	39		
	6.10					6	57.62

End of Borehole

(GWL @ 5.20m - August 11, 2021)

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

○ Water Content %

20 40 60 80

Piezometer Construction

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

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Use Development - Parkway House
2475 Regina Street, Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE August 3, 2021

FILE NO. PG5901

HOLE NO. BH 7-21

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content %	Monitoring Well Construction
GROUND SURFACE						0	65.14		
TOPSOIL 0.20									
FILL: Brown silty sand with gravel 1.37		AU	1						
		SS	2	75	36	1	64.14		
FILL: Brown silty sand with clay, gravel, some topsoil 1.83		SS	3	75	18				
GLACIAL TILL: Compact, brown silty sand with gravel, cobbles, boulders, trace clay						2	63.14		
		SS	4	83	15				
						3	62.14		
		SS	5	83	16				
		SS	6	25	31	4	61.14		
- grey by 4.9m depth		SS	7	58	11				
						5	60.14		
		SS	8	25	11				
6.10						6	59.14		
End of Borehole									
(BH dry - August 11, 2021)									

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

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

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 = $(D30)^2 / (D10 \times D60)$
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

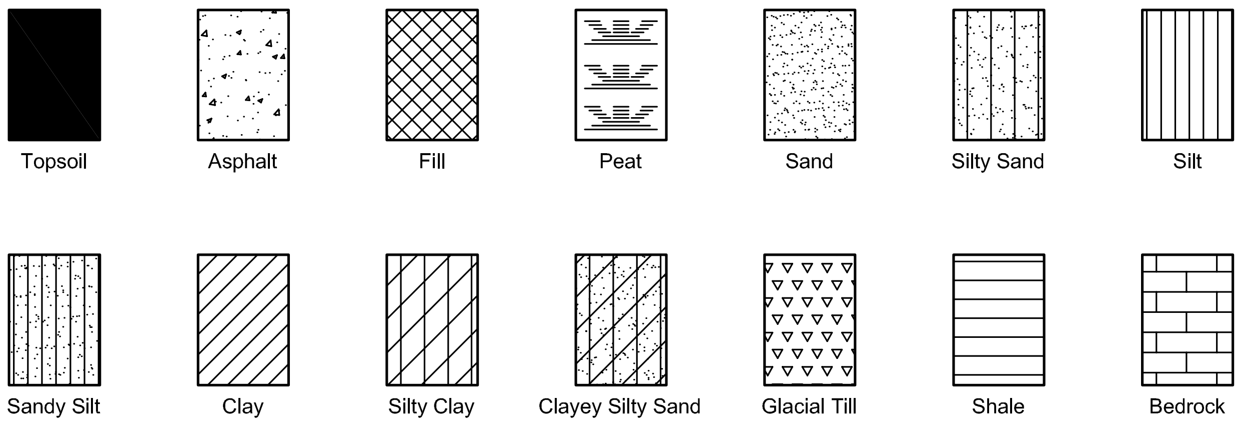
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void 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.

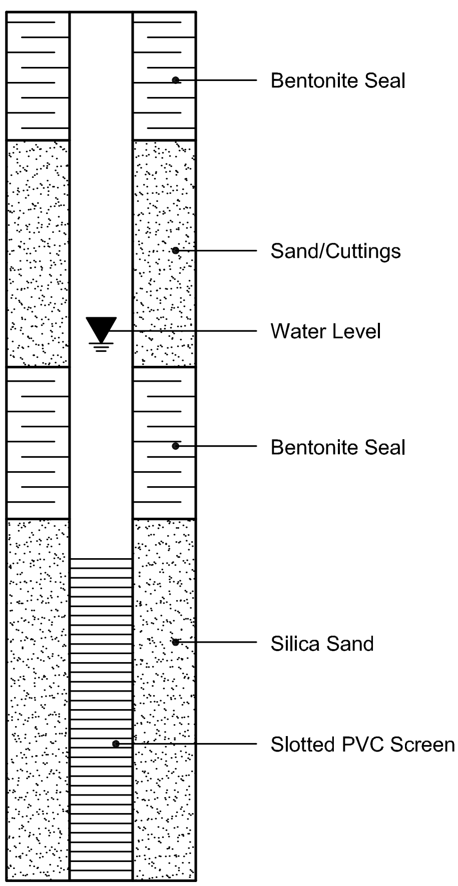
SYMBOLS AND TERMS (continued)

STRATA PLOT

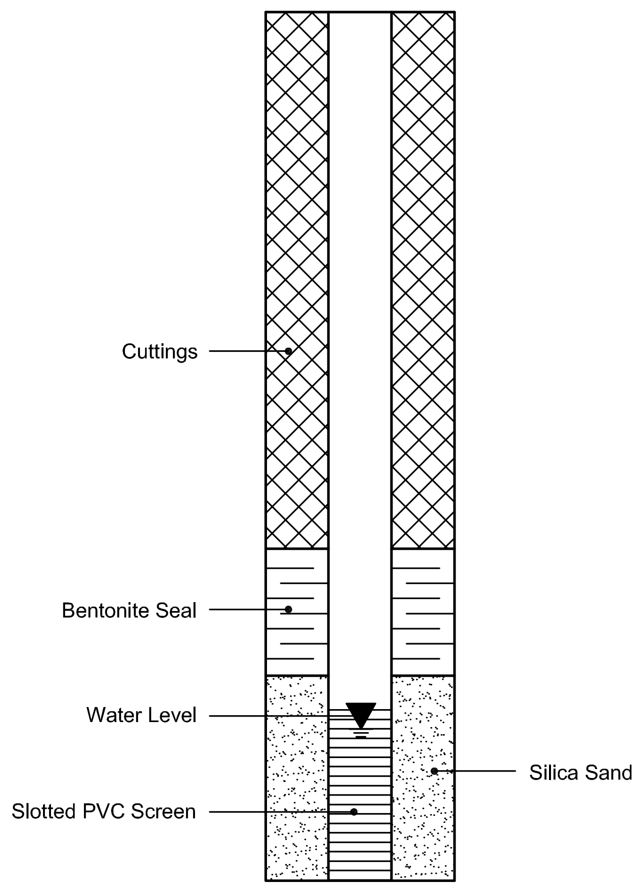


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Client: **Paterson Group Consulting Engineers**

Client PO: 32555

Report Date: 06-Aug-2021

Order Date: 3-Aug-2021

Project Description: PG5901

	Client ID:	BH3-21 SS9	-	-	-
	Sample Date:	30-Jul-21 09:00	-	-	-
	Sample ID:	2132173-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	93.0	-	-	-
General Inorganics					
pH	0.05 pH Units	8.06	-	-	-
Resistivity	0.10 Ohm.m	36.2	-	-	-
Anions					
Chloride	5 ug/g dry	<5	-	-	-
Sulphate	5 ug/g dry	168	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURES 2 & 3 - SEISMIC SHEAR WAVE VELOCITY PROFILES

DRAWING PG5901-1 - TEST HOLE LOCATION PLAN

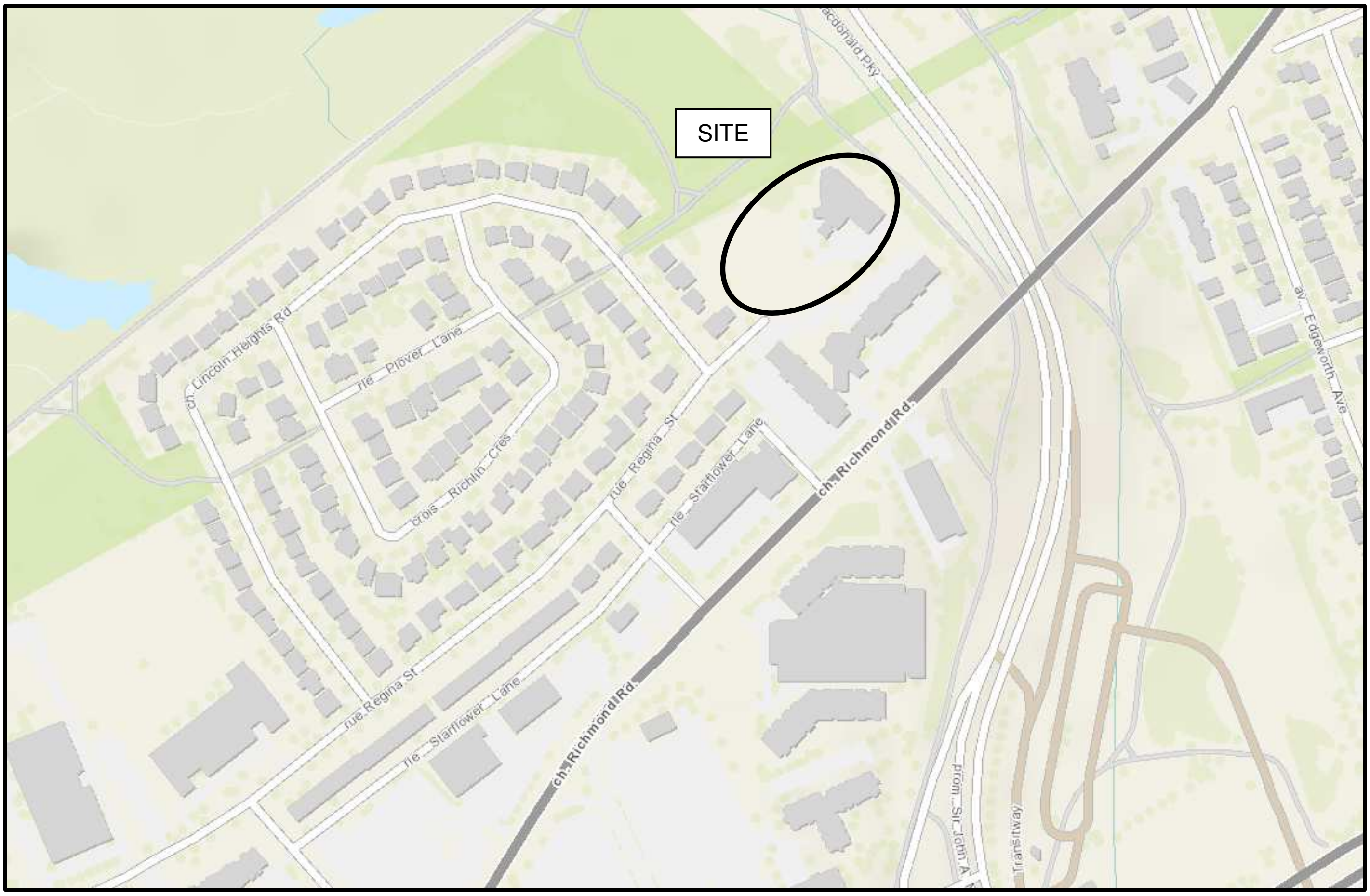


FIGURE 1

KEY PLAN

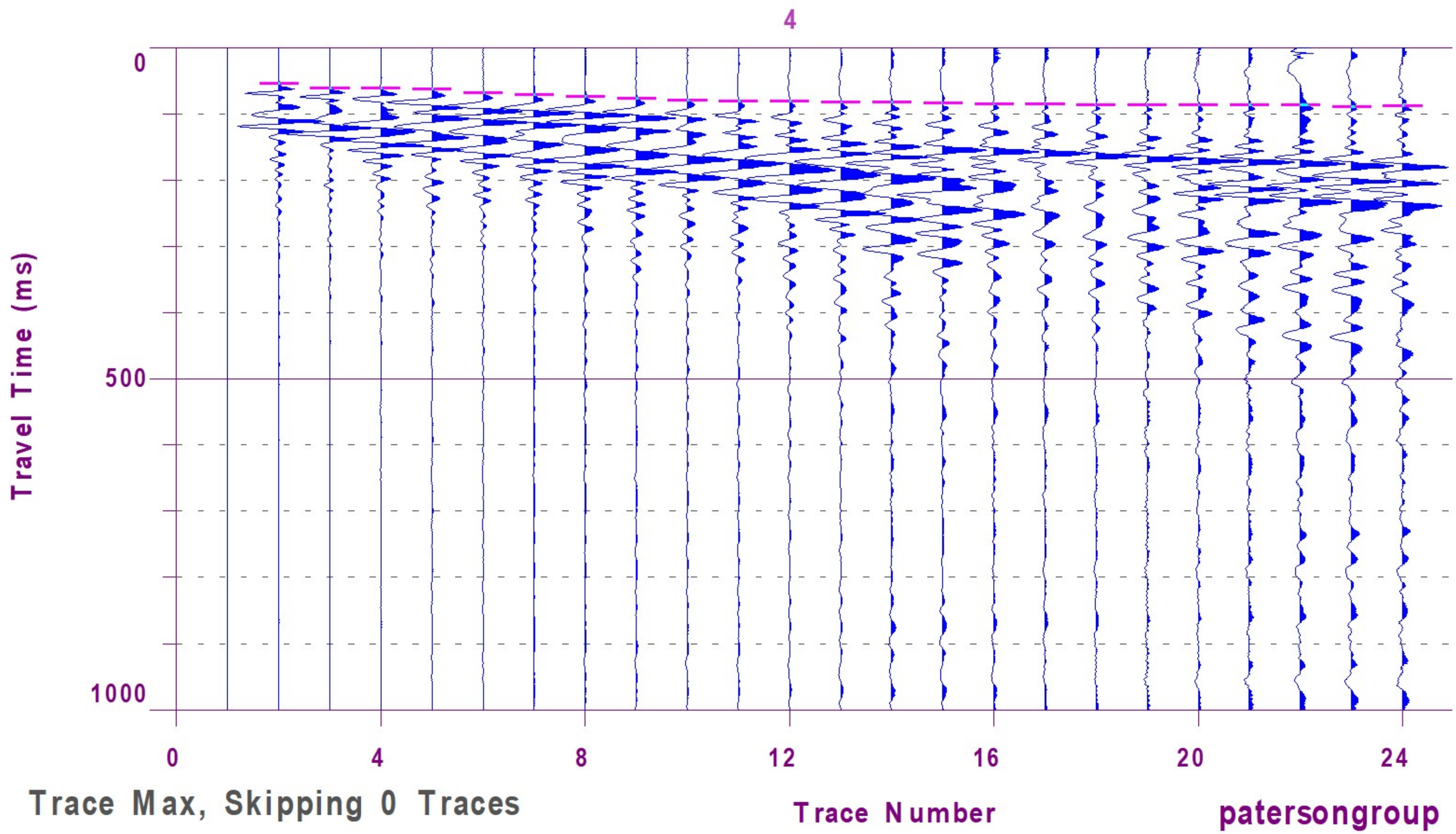


Figure 2 – Shear Wave Velocity Profile at Shot Location -15 m

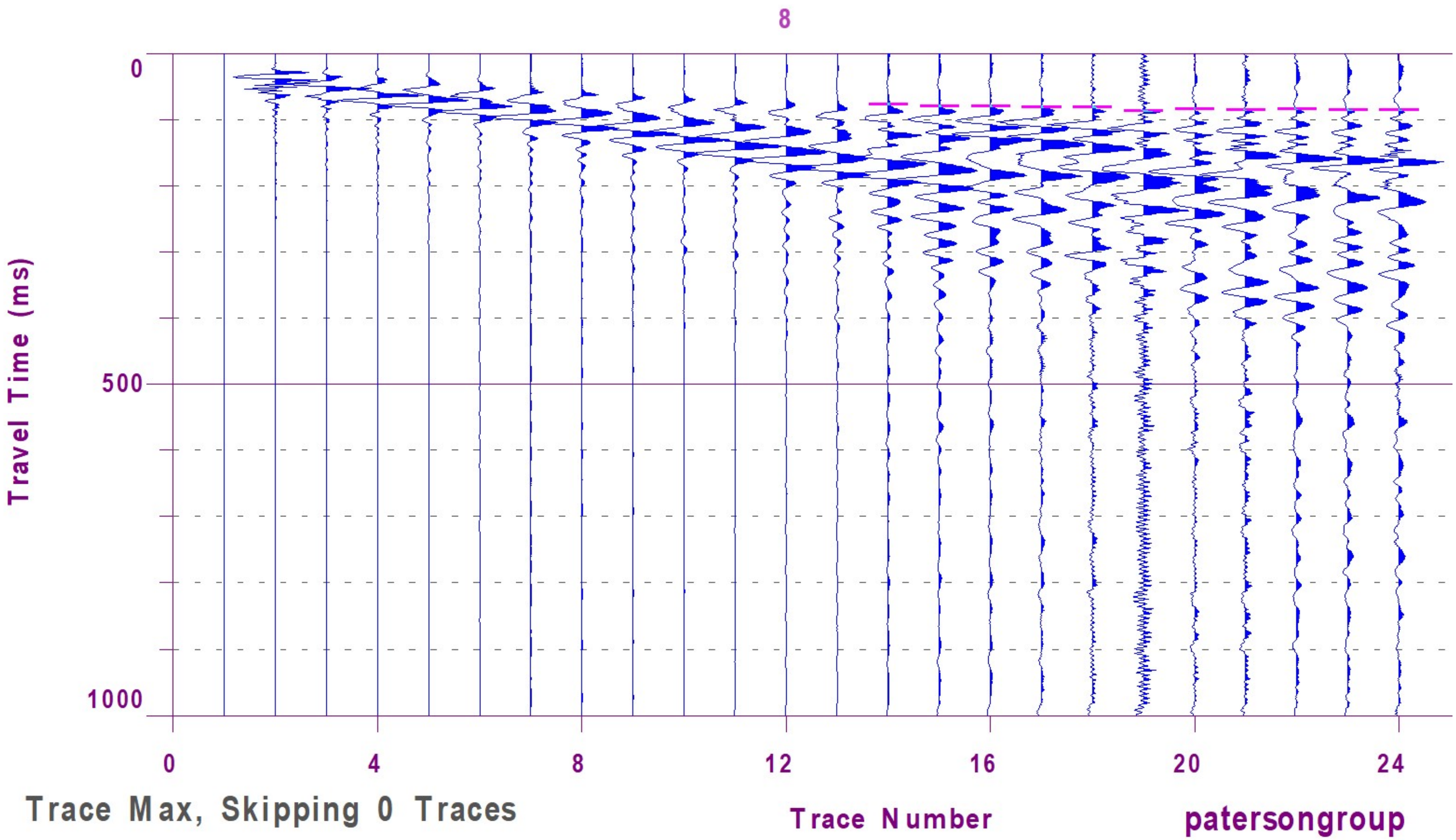
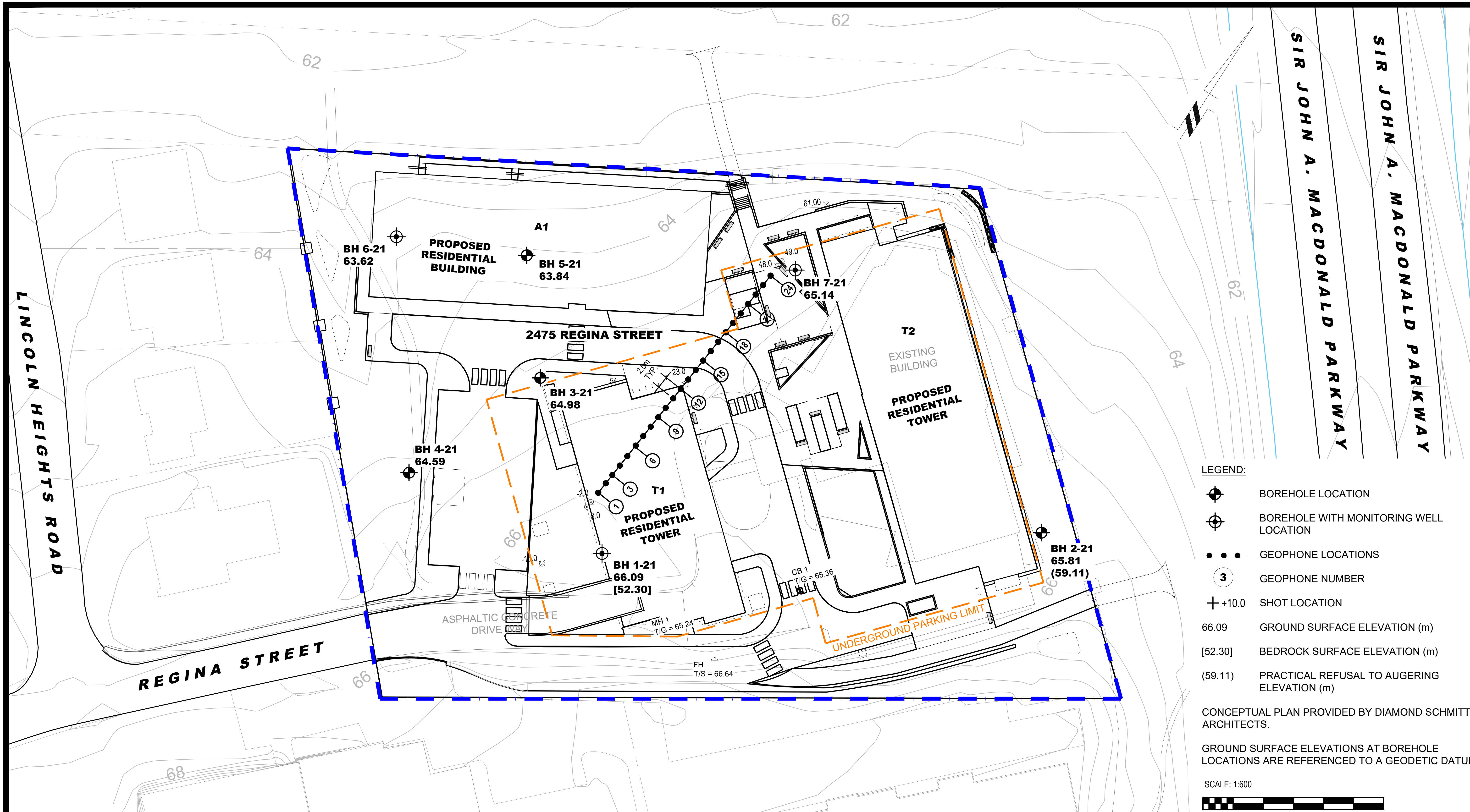


Figure 3 – Shear Wave Velocity Profile at Shot Location -2 m



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NO.	REVISIONS	DATE	INITIAL
2	LATEST CONCEPTUAL PLAN ADDED TO DRAWING	05/05/2025	MA
1	LATEST CONCEPTUAL PLAN ADDED TO DRAWING	25/04/2022	KP

PARKWAY HOUSE DEVELOPMENT FUND LP
GEOTECHNICAL INVESTIGATION
PROPOSED MIXED-USE DEVELOPMENT
2475 REGINA STREET
OTTAWA, ONTARIO

Title:
TEST HOLE LOCATION PLAN

Scale: 1:600	Date: 08/2021
Drawn by: JM	Report No.: PG5901-1
Checked by: KP	Dwg. No.: PG5901-1
Approved by: SD	Revision No.: 2

p:\autocad drawings\geotechnical\pg59xx\pg5901\pg5901-1 thlp (rev.02) (may 2025).dwg

11x17