PATERSON GROUP

Geotechnical Investigation

Proposed Multi-Storey Buildings

1919 Riverside Drive
Ottawa, Ontario

Prepared for Schlegel Villages

Report PG5947-1 dated July 18, 2022

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Schlegel Villages to conduct a geotechnical investigation for the proposed development to be located at 1919 Riverside Drive in the City of Ottawa, Ontario (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 the design of the proposed development including construction considerations which may affect the design.

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

2.0 Proposed Development

Based on the available drawings, it is understood that the proposed development will consist of multi-storey buildings, which will be constructed in phases, and which will have 1 basement level. Asphalt-paved parking areas, walkways and landscaped areas are also anticipated at finished grades surrounding the proposed buildings.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the geotechnical investigation was carried out during the period of June 20 to 23, 2022, and consisted of a total of 12 boreholes advanced to a maximum depth of 9 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 underground services. The approximate locations of the boreholes are shown on Drawing PG5947-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were advanced using a low-clearance 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 augering to the required depths at the selected borehole locations, and sampling and testing the overburden.

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. Rock cores (RC) were obtained using 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.

The recovery value and a Rock Quality Designation (RQD) value were calculated for each drilled section of bedrock and are presented on the borehole logs. The recovery value is the length of the bedrock sample recovered over the length of the drilled section. The RQD value is the total length of intact rock pieces longer than 100 mm over the length of the core run. The values indicate the bedrock quality.

The subsurface conditions observed in the boreholes were recorded in detail in the field. The soil and bedrock profiles are logged on the Soil Profile and Test Data Sheets in Appendix 1 following this report.

Groundwater

Standpipe piezometers and monitoring wells were installed in boreholes upon the completion of the field investigation to permit monitoring of the groundwater levels. Groundwater level observations are discussed in Section 4.3 and are presented in the Soil Profile and Test Data Sheets in Appendix 1.

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 borehole locations, and ground surface elevation at each borehole location, were surveyed by Paterson using a GPS unit with respect to a geodetic datum. The location of the boreholes and ground surface elevation at each borehole location are presented on Drawing PG5947-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Review

Soil samples were recovered from the subject site and visually examined in our laboratory to review the results of the field logging. All samples will be stored in the laboratory for one month after this report is completed. They will then be discarded unless we are otherwise directed.

3.4 Analytical Testing

One 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 tested to determine the concentration of sulphate and chloride, and the resistivity and the pH of the sample. The results are presented in Appendix 1 and are discussed further in Section 6.7.

4.0 Observations

4.1 Surface Conditions

The subject site is located within the northern portion of the property with the civic address of 1919 Riverside Drive, and is occupied by an existing asphalt-paved parking lot with landscaped margins. However, based on available aerial photos, residential dwellings and agricultural buildings were located within the northern portion of the site as recently as 1958, and were no longer present in 1976. Reference should be made to the aerial photographs in Figure 2 - Aerial Photograph – 1958, Figure 3 - Aerial Photograph - 1965 and Figure 4 – Aerial Photograph – 2019 which illustrate the former and present site conditions, respectively.

The subject site is bordered to the north by Smyth Road, to the east by a railway line, to the south by a hospital, and to the west by a City of Ottawa transitway followed by Riverside Drive. The existing ground surface across the subject site slopes downward from northeast to southwest from approximate geodetic elevations 69 to 64.5 m.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile at the subject site consists of an approximate 0.05 to 0.1 m thick layer of asphalt or topsoil, underlain by fill. The fill material was observed to generally consist of a brown silty sand to silty clay with gravel and crushed stone. The depth of the fill layer ranged from 0.5 to 4.9 m depth below the existing ground surface.

Underlying the fill material, a thin deposit of silty clay to silty sand was observed, and is further underlain by a deposit of glacial till. The glacial till deposit generally consists of a compact to very dense, brown to grey silty clay to silty sand with gravel, cobbles, and boulders.

Practical refusal to augering was encountered at depths ranging from about 3.2 to 7.7 m below the existing ground surface.

Bedrock

Bedrock was cored at boreholes BH 6-22, BH 7-22, BH 9-22, BH 10-22, and BH 12-22. Based on the recovered rock core, the bedrock was observed to consist of shale, with the upper 1 to 3 m of the bedrock being generally very poor to fair in quality, and becoming good to excellent in quality with depth. The bedrock was cored to a maximum depth of about 9 m below the existing grade.

Based on available geological mapping, bedrock in the area of the subject site consists of shale of the Billings and Carlsbad Formations with an overburden thickness ranging from approximately 3 to 10 m.

4.3 Groundwater

The groundwater levels were measured in the piezometers and monitoring wells on July 7, 2022. The observed groundwater levels are summarized in Table 1.

Table 1 - Summary of Groundwater Level Readings

Test Hole Number	Ground Surface Elevation (m)	Groundwater Level (m)	Groundwater Elevation (m)	Recording Date
BH 1-22	64.82	2.43	62.39	July 7, 2022
BH 2-22	66.27	Dry	-	July 7, 2022
BH 3-22	66.43	1.84	64.59	July 7, 2022
BH 4-22	67.20	1.99	65.21	July 7, 2022
BH 5-22	68.17	4.67	63.50	July 7, 2022
BH 6-22	68.11	5.99	62.12	July 7, 2022
BH 7-22	67.90	4.24	63.66	July 7, 2022
BH 8-22*	69.70	3.77	65.93	July 7, 2022
BH 9-22*	66.90	3.10	63.80	July 7, 2022
BH10-22	66.62	4.65	61.97	July 7, 2022
BH11-22*	66.11	7.39	58.72	July 7, 2022
BH12-22*	67.37	2.64	64.73	July 7, 2022

Note: Ground surface elevations at borehole locations were surveyed by Paterson and are referenced to a geodetic datum.
* indicates a Monitoring Well Location.

It is important to note that groundwater readings can be influenced by surface water perched within the borehole backfill material. Long-term groundwater conditions can also be estimated based on the observed color and consistency of the recovered soil samples. Based on these observations, it is estimated that the long-term groundwater level can be expected between an approximate 3 to 4 m depth. However, groundwater levels are subject to seasonal fluctuations and therefore could vary during 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 buildings be supported on conventional spread footings bearing on the undisturbed, compact to very dense glacial till deposit, or on clean, surface sounded bedrock.

Where fill or loose silty sand is encountered at the underside of footing elevation, as is anticipated in the vicinity of boreholes BH 9-22 and BH 11-22, it should be sub-excavated to the surface of the undisturbed, compact to very dense glacial till or clean, surface sounded bedrock. Where the footings are designed to be supported on the undisturbed, compact to very dense glacial till, engineered fill can then be used to raise grades to the underside of footing elevation. Where the footings are designed to be supported on clean, surface sounded bedrock, lean concrete must then be used to raise grades to the underside of footing elevation. The lateral limits of the engineered fill or lean concrete placement should be in accordance with our lateral support recommendations provided herein.

Dependent on the founding depths of the proposed buildings, bedrock removal may be required. 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 demolished debris should be completely removed from the proposed building perimeters and within the lateral support zones of the foundations. Under paved area, existing construction remnants, such as foundation walls should be excavated to a minimum of 1 m below final grade.

Bedrock Removal

As noted above, bedrock removal may be required during construction of the proposed buildings. Hoe ramming is an option where the bedrock is weathered and/or where only small quantities of bedrock need to be removed. Where large quantities of bedrock need to be removed, line drilling and controlled blasting may be required. The blasting operations should be planned and completed under the guidance of a professional engineer with experience in blasting operations. Prior to considering blasting operations, the blasting effects on the existing services, buildings and other structures should be addressed. A pre-blast or pre-construction survey of the existing structures located in proximity of the blasting operations should be conducted prior to commencing construction. The extent of the survey should be determined by the blasting consultant and sufficient to respond to any inquiries/claims related to the blasting operations.

Vibration Considerations

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

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

Two parameters determine the permissible vibrations, 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). These guidelines are for current construction standards. These guidelines are above perceptible human level and, in some cases, could be very disturbing to some people, a pre-construction survey is recommended to minimize the risks of claims during or following the construction of the proposed development.

Fill Placement

Engineered fill placed for grading beneath the proposed buildings, where required, should consist of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. This material should be tested and 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 buildings 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.

Lean Concrete Filled Trenches

Where the proposed footings are to be founded on bedrock which is located below the underside of footing elevation, zero-entry vertical trenches should be excavated to the clean, surface sounded bedrock, and backfilled with lean concrete to the founding elevation (minimum **17 MPa** 28-day compressive strength). Typically, the excavation side walls 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 (strip and pad footings) 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 clea, surface sounded bedrock. Once the trench excavation is approved by the geotechnical engineer, lean concrete can be poured up to the proposed founding elevation.

5.3 Foundation Design

Bearing Resistance Values

Footings placed on an undisturbed, compact to very dense glacial till, or on engineered fill which is placed and compacted directly over the undisturbed, compact to very dense glacial till, 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.

An undisturbed soil bearing surface consists of a surface 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 designed using the bearing resistance values at SLS given above will be subjected to potential post construction total and differential settlements of 25 and 20 mm, respectively.

Footings placed on clean, surface sounded shale bedrock, or on lean concrete which is placed directly over the clean, surface sounded shale bedrock, can be designed using a factored bearing resistance value at ultimate limit states (ULS) of **1,000 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 supported either on an acceptable bedrock bearing surface, or on lean concrete trenches which are placed directly on an acceptable bedrock bearing surface, and designed for the bearing resistance values provided herein, will be subjected to negligible post-construction total and differential settlements.

Lateral Support

The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels. Adequate lateral support is provided to an in-situ soil bearing medium 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 or engineered fill of the same or higher capacity as the soil.

Adequate lateral support is provided to a sound bedrock bearing medium when a plane extending down and out from the bottom edge of the footing at a minimum of 1H:6V (or flatter) passes only through sound bedrock or a material of the same or higher capacity as the bedrock, such as concrete. A weathered bedrock bearing medium will require a lateral support zone of 1H:1V (or flatter).

Bedrock/Soil Transition

Where a building is founded partly on bedrock and partly on soil, it is recommended to decrease the soil bearing resistance value by 25% for the footings placed on soil bearing media to reduce the potential long-term total and differential settlements. Also, at the soil/bedrock and bedrock/soil transitions, it is recommended that the upper 0.5 m of the bedrock be removed for a minimum length of 2 m (on the bedrock side) and replaced with nominally compacted OPSS Granular A or Granular B Type II material.

The width of the sub-excavation should be at least the proposed footing width plus 0.5 m. Steel reinforcement, extending at least 3 m on both sides of the 2 m long transition, should be placed in the top part of the footings and foundation walls.

5.4 Design for Earthquakes

A seismic shear wave velocity test was completed at the subject site to accurately determine the applicable seismic site classification for the proposed development based on Table 4.1.8.4.A of the Ontario Building Code (OBC) 2012. The shear wave velocity test was completed by Paterson personnel. Two seismic shear wave velocity profiles from the on-site testing are presented in Appendix 2.

Field Program

The seismic array testing location was placed directly on the northern end of the site in an east-west direction. 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 3 m intervals and connected by a geophone spread cable to a Geode 24 Channel seismograph.

The seismograph was also connected to a computer laptop and a hammer trigger switch attached to a 12 pound dead blow hammer. The hammer trigger switch sends a start signal to the seismograph. The hammer is used to strike an I-Beam seated into the ground surface, which creates a polarized shear wave. The hammer shots are repeated between four (4) to eight (8) times at each shot location to improve signal to noise ratio. The shot locations are also completed in forward and reverse directions (i.e.- striking both sides of the I-Beam seated parallel to the geophone array). The shot locations are located at 25, 4.5 and 3 m away from the first geophone, 3, 4.5, and 20 m away from the last geophone, and at the centre of the seismic array.

Data Processing and Interpretation

Interpretation for the shear wave velocity results were completed by Paterson personnel. Shear wave velocity measurement was made using reflection/refraction methods. The interpretation is performed by recovering arrival times from direct and refracted waves. The interpretation is repeated at each shot location to provide an average shear wave velocity, Vs_{30}, of the upper 30 m profile, immediately below the building's foundation. The layer intercept times, velocities from different layers and critical distances are interpreted from the shear wave records to compute the bedrock depth at each location. The bedrock velocity was interpreted using the main refractor wave velocity, which is considered a conservative estimate of the bedrock velocity due to the increasing quality of the bedrock with depth. It should be noted that as bedrock quality increases, the bedrock shear wave velocity also increases.

Site Class where building footings are supported directly on bedrock, or on lean concrete trenches placed directly on bedrock

The V_{s30} was calculated using the standard equation for average shear wave velocity calculation from the OBC 2012, as presented below.

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

$$V_{s30} = \frac{30\ m}{\left(\frac{30\ m}{1959\ m/s}\right)}$$

$$V_{s30} = 1{,}959\ m/s$$

Based on the results of the seismic testing, the average shear wave velocity, Vs_{30}, for footings supported on bedrock, or on lean concrete trenches which extend to the bedrock surface, is **1,959 m/s**. Therefore, a **Site Class A** is applicable for design of the proposed building in this case, as per Table 4.1.8.4.A of the OBC 2012. The soils underlying the subject site are not susceptible to liquefaction.

Site Class where building footings are within 3 m of bedrock surface

The Vs_{30} was calculated using the standard equation for average shear wave velocity calculation from the OBC 2012, as presented below.

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

$$V_{s30} = \frac{30\ m}{\left(\frac{3\ m}{246\ m/s}\right) + \left(\frac{27\ m}{1959\ m/s}\right)}$$

$$V_{s30} = 1{,}155\ m/s$$

Based on the results of the seismic testing, where the proposed footings are located within 3 m of the bedrock surface, the average shear wave velocity, V_{s30}, would be **1,155 m/s**. Therefore, a **Site Class B** would be applicable for design of the proposed building in this case, as per Table 4.1.8.4.A of the OBC 2012. The soils underlying the subject site are not susceptible to liquefaction.

5.5 Basement Slab Construction

With the removal of all topsoil and deleterious fill within the footprints of the proposed buildings, the native soils or approved engineered fill pad will be considered an acceptable subgrade upon which to commence backfilling for floor slab construction.

Any soft areas should be removed and backfilled with appropriate backfill material. OPSS Granular B Types I or II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slabs (outside the zones of influence of the footings).

It is recommended that the upper 200 mm of subfloor fill consists of 19 mm clear crushed stone. In consideration of the anticipated groundwater conditions, an underslab drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided in the clear crushed stone under the basement floor slab. This is discussed further in Section 6.1.

5.6 Basement Wall

There are several combinations of backfill materials and retained soils that could be applicable for the basement walls of the proposed 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 for undrained conditions.

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 (P_o) and the seismic component (ΔP_{AE}).

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

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

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

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

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

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

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

5.7 Pavement Design

For design purposes, the pavement structures presented in Tables 2 and 3 below are recommended for the design of asphalt-paved car only parking areas and access lanes, respectively.

Table 2 – Pavement Structure – Car Only Parking Areas and Driveways

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

Table 3 – 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	**Binder Course –** HL-8 or Superpave 19.0 Asphaltic Concrete
150	**BASE –** OPSS Granular A Crushed Stone
450	**SUBBASE –** OPSS Granular B Type II
Subgrade – Either fill, in-situ soil, or OPSS Granular B Type I or II material placed over in-situ soil or fill.	

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. Weak subgrade conditions may be experienced over service trench fill materials. This may require the use of geotextile, thicker subbase or other measures that can be recommended at the time of construction as part of the field observation program.

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for this project. For residential driveways and car only parking areas, an Ontario Traffic Category A will be used. For local roadways, an Ontario traffic Category B should be used for design purposes.

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

It is recommended that a perimeter foundation drainage system be provided for the proposed buildings. The system should consist of a 150 mm diameter perforated and corrugated PVC pipe, surrounded on all sides by 150 mm of 19 mm clear crushed stone which is placed at the footing level around the exterior perimeter of the structure. The pipe should have a positive outlet, such as a gravity connection to the storm sewer.

Where insufficient room is available for exterior backfill, it is suggested that the composite drainage system (such as Delta Drain 6000 or equivalent) be secured against the temporary shoring system and extending to a series of drainage sleeves inlets through the building foundation wall at the footing/foundation wall interface.

The drainage sleeves should be at least 150 mm diameter and spaced 3 m along the perimeter foundation walls. An interior perimeter drainage pipe should be placed along the building perimeter along with the underslab drainage system. The perimeter drainage pipe and underslab drainage system should direct water to sump pit(s) within the lower underground area.

A waterproofing system should be provided for any elevator pits (pit bottom and walls).

Underslab Drainage

Underslab drainage is recommended to control water infiltration below the lowest level floor slab. For preliminary design purposes, it is recommended that 150 mm diameter perforated PVC pipes be placed at 6 m spacing underlying the lowest level floor slab. 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 sufficient space is available for conventional backfilling, backfill against the exterior sides of the foundation walls should consist of free draining non frost susceptible granular materials.

6.2 Protection of Footings Against Frost Action

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

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

6.3 Excavation Side Slopes

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

Unsupported Excavations

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

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

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

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

The temporary shoring system may consist of a soldier pipe and lagging system which could be cantilevered, anchored or braced.

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

Table 4 - Soil Parameters

Parameters	Values
Active Earth Pressure Coefficient (K_a)	0.33
Passive Earth Pressure Coefficient (K_p)	3
At-Rest Earth Pressure Coefficient (K_o)	0.5
Unit Weight , kN/m_3	21
Submerged Unit Weight , kN/m_3	13

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

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

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

6.4 Pipe Bedding and Backfill

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

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

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

6.5 Groundwater Control

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

Permit to Take Water

A temporary Ministry of Environment, Conservation and Parks (MECP) permit to take water (PTTW) is recommended, as ground and/or surface water to be pumped during the construction phase is anticipated to exceed 400,000 L/day. At least 4 to 5 months should be allowed for completion of the application and issuance of the permit by the MECP.

Impacts on Neighbouring Properties

Based on the groundwater level encountered during the geotechnical investigation, the proposed construction will not extend significantly below the groundwater level (likely less than 1 m). Therefore, significant groundwater lowering is not anticipated during or after construction, and, accordingly, the proposed development will not negatively impact the neighbouring structures.

6.6 Winter Construction

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

In the event of construction during below zero temperatures, the founding stratum should be protected from freezing temperatures using straw, propane heaters and tarpaulins or other suitable means. In this regard, the base of the excavations should be insulated from sub-zero temperatures immediately upon exposure and until such time as heat is adequately supplied to the building and the footings are protected with sufficient soil cover to prevent freezing at founding level.

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

6.7 Corrosion Potential and Sulphate

The 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 low to slightly aggressive corrosive environment.

7.0 Recommendations

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

- ❑ Observation of all bearing surfaces prior to the placement of concrete.
- ❑ Sampling and testing of the concrete and fill materials.
- ❑ Observation of all subgrades prior to backfilling.
- ❑ Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable
- ❑ Field density tests to determine the level of compaction achieved.
- ❑ Sampling and testing of the bituminous concrete including mix design reviews.

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

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

8.0 Statement of Limitations

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

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

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

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

Otillia McLaughlin B.Eng.

Scott S. Dennis, P.Eng.

Report Distribution:

- ❑ Schlegel Villages (email copy)
- ❑ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Development - Schlegel Villages
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE June 20, 2022

FILE NO. PG5947

HOLE NO. BH 1-22

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	64.82
TOPSOIL	0.13						
FILL: Brown silty sand with topsoil and organics	0.69	AU	1				
Very stiff to stiff, brown **SILTY CLAY** with sand	1.12	SS	2	67	15	1	63.82
Compact, brown **SILTY SAND**	1.65						
Very stiff, brown **SILTY CLAY** with sand, trace gravel	2.21	SS	3	67	10	2	62.82
GLACIAL TILL: Dense to very dense, brown silty clay to clayey silt with sand, gravel, cobbles and boulders		SS	4	79	38		
- grey by 2.6m depth	3.66	SS	5	100	24	3	61.82
End of Borehole							
(GWL @ 2.43m - July 7, 2022)							

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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SOIL PROFILE AND TEST DATA

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BORINGS BY CME-55 Low Clearance Drill

DATE June 20, 2022

FILE NO. PG5947

HOLE NO. BH 2-22

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 %	Piezometer Construction
GROUND SURFACE						0	66.27		
TOPSOIL 0.13									
FILL: Brown silty sand with gravel and crushed stone 0.69		AU	1						
Loose, brown **SILTY SAND,** some to trace clay 2.21		SS	2	83	9	1	65.27		
		SS	3	92	8	2	64.27		
Compact to loose, brown **SILTY SAND,** trace clay 3.35		SS	4	75	12	3	63.27		
		SS	5	67	6				
GLACIAL TILL: Dense, grey silty sand to sandy silt with gravel, some clay, cobbles and boulders 5.21		SS	6	83	34	4	62.27		
		SS	7	42	50+	5	61.27		
End of Borehole									

Practical refusal to augering at 5.21m depth

(BH dry - July 6, 2022)

Pen. Resist. / Water Content % scale: 20 40 60 80

Shear Strength (kPa): 20 40 60 80 100

▲ Undisturbed △ Remoulded

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DATE June 20, 2022

FILE NO. PG5947

HOLE NO. BH 3-22

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	66.43
TOPSOIL	0.18						
FILL: Brown silty sand, with gravel and crushed stone	0.48	AU	1				
Compact to loose, brown **SILTY SAND**		SS	2	58	14	1	65.43
		SS	3	58	6	2	64.43
- grey by 2.2m depth	2.59	SS	4	75	29		
GLACIAL TILL: Compact to dense, grey silty sand with gravel, cobbles and boulders, trace clay	3.18	SS	5	40	50+	3	63.43
End of Borehole							

Practical refusal to augering at 3.18m depth

(GWL @ 1.84m - July 6, 2022)

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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BORINGS BY CME-55 Low Clearance Drill

DATE June 20, 2022

FILE NO. PG5947

HOLE NO. BH 4-22

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	67.20
Asphaltic concrete 0.05							
FILL: Crushed stone with silty sand 0.69		AU	1				
Loose, brown **SILTY SAND,** some to trace clay 1.45		SS	2	83	7	1	66.20
GLACIAL TILL: Dense to very dense, brown silty clay to clayey silt with sand, some gravel, cobbles and boulders - grey by 2.2m depth		SS	3	100	17	2	65.20
		SS	4	75	21	3	64.20
		SS	5	2	25	4	63.20
		SS	6	83	24		
4.65		SS	7	33	50+		

End of Borehole

Practical refusal to augering at 4.65m depth

(GWL @ 1.99m - July 7, 2022)

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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DATE June 20, 2022

FILE NO. PG5947

HOLE NO. BH 5-22

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Asphaltic concrete	0.08				
FILL: Crushed stone with sand	0.69	AU	1		
Very stiff, brown **SILTY CLAY,** some sand, trace gravel	1.60	SS	2	67	31
GLACIAL TILL: Dense to very dense, brown silty clay to clayey silt with sand, gravel, cobbles and boulders	4.75	SS	3	83	28
		SS	4	67	21
		SS	5	67	27
		SS	6	100	32
		SS	7	86	50+

End of Borehole

Practical refusal to augering at 4.75m depth

(GWL @ 4.67m - July 7, 2022)

DEPTH (m)	ELEV. (m)
0	68.17
1	67.17
2	66.17
3	65.17
4	64.17

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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SOIL PROFILE AND TEST DATA

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DATE June 21, 2022

FILE NO. PG5947

HOLE NO. BH 6-22

SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
Asphaltic concrete	0.05
FILL: Crushed stone with sand	0.69
FILL: Brown silty clay with sand and gravel	1.65
GLACIAL TILL: Dense to very dense, grey silty clay to clayey silt with sand, some gravel, cobbles and boulders - grey by 3.0m depth - silt content increasing with depth	4.60
BEDROCK: Fair to good quality, black shale - interlayered with grey limestone by 6.4m depth	7.70
End of Borehole (GWL @ 5.99m - July 7, 2022)	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	50	10
SS	3	100	33
SS	4	83	20
SS	5	75	18
SS	6	92	47
SS	7	33	50+
RC	1	100	50
RC	2	100	88

DEPTH (m)	ELEV. (m)
0	68.11
1	67.11
2	66.11
3	65.11
4	64.11
5	63.11
6	62.11
7	61.11

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

○ Water Content %

20 40 60 80

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

Piezometer Construction

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FILE NO. PG5947

HOLE NO. BH 7-22

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	67.90
Asphaltic concrete	0.05						
FILL: Crushed stone with silty sand	0.76	AU	1				
FILL: Brown silty clay, trace sand and gravel	2.44	SS	2	17	5	1	66.90
		SS	3	100	23	2	65.90
Hard, brown **SILTY CLAY to CLAYEY SILT,** some sand and gravel	3.18	SS	4	100	25	3	64.90
GLACIAL TILL: Very dense, grey silty clay with sand, some gravel, cobbles and boulders	5.28	SS	5	79	26	4	63.90
		SS	6	100	28	5	62.90
		SS	7	50	50+		
BEDROCK: Very poor to excellent quality, black shale	8.71	RC	1	100	0	6	61.90
		RC	2	100	98	7	60.90
		RC	3	100	100	8	59.90
End of Borehole							
(GWL @ 4.24m - July 7, 2022)							

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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DATE June 21, 2022

FILE NO. PG5947

HOLE NO. BH 8-22

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	69.70
TOPSOIL 0.15							
FILL: Brown silty clay with sand, trace gravel 0.61		AU	1				
FILL: Brown silty sand to sandy silt, some gravel, trace clay, occasional cobbles 1.09		SS	2	83	13	1	68.70
		SS	3	92	50+	2	67.70
		SS	4	92	50+	3	66.70
		SS	5	67	39		
GLACIAL TILL: Very dense to dense, brown silty sand to sandy silt with gravel, cobbles and boulders		SS	6	75	50+	4	65.70
- grey by 4.5m depth		SS	7	50	50+	5	64.70
		SS	8	50	33	6	63.70
		SS	9	42	23		
6.99		SS	10	21	50+		

End of Borehole

Practical spilt spoon refusal at 6.99m depth

(GWL @ 3.77m - July 7, 2022)

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

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REMARKS

BORINGS BY CME-55 Low Clearance Drill **DATE** June 22, 2022

FILE NO. PG5947

HOLE NO. BH 9-22

SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
Asphaltic concrete	0.10
FILL: Brown silty sand with crushed stone	0.53
FILL: Brown to grey silty clay, some sand, trace gravel and topsoil	3.50
FILL: Brown to grey silty clay with sand, some gravel, cobbles, boulders, wood and concrete fragments	4.88
GLACIAL TILL: Very dense, grey silty clay to clayey silt, some sand, gravel, cobbles and boulders	5.16
BEDROCK: Poor to excellent quality, black shale	9.04
End of Borehole	

(GWL @ 3.10m - July 7, 2022)

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	58	7
SS	3	75	5
SS	4	54	5
SS	5	50	50+
RC	1	27	0
RC	2	100	36
RC	3	100	41
RC	4	100	93

DEPTH (m)	ELEV. (m)
0	66.90
1	65.90
2	64.90
3	63.90
4	62.90
5	61.90
6	60.90
7	59.90
8	58.90
9	57.90

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

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SOIL PROFILE AND TEST DATA

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REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE June 22, 2022

FILE NO. PG5947

HOLE NO. BH10-22

SOIL DESCRIPTION	Strata depth (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	66.62
Asphaltic concrete	0.08						
FILL: Brown silty sand with gravel and crushed stone	0.69	AU	1				
FILL: Brown silty sand, clay and gravel	1.45	SS	2	33	50+	1	65.62
Very loose to compact, brown **SILTY SAND to SANDY SILT,** trace clay (possible fill)	2.69	SS	3	50	1	2	64.62
		SS	4	79	13		
Very stiff, grey **SILTY CLAY to CLAYEY SILTY** (possible fill)	3.02					3	63.62
GLACIAL TILL: Very dense silty clay to clayey silt with sand, gravel, cobbles and boulders	3.66	SS	5	54	50+		
BEDROCK: Fair to good quality, black shale		RC	1	100	49	4	62.62
		RC	2	100	83	5	61.62
						6	60.62
		RC	3	100	51	7	59.62
End of Borehole	7.65						

(GWL @ 4.65m - July 7, 2022)

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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BORINGS BY CME-55 Low Clearance Drill

DATE June 23, 2022

FILE NO. PG5947

HOLE NO. BH11-22

SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
TOPSOIL	0.05
FILL: Crushed stone	0.69
FILL: Brown silty sand to sandy silt, some crushed stone and gravel	1.45
FILL: Grey sandy silt, trace clay	1.83
FILL: Brown silty clay, trace sand	2.44
FILL: Brown silty sand, some clay, trace gravel	3.73
Loose, brown **SILTY SAND,** trace gravel - some topsoil by 3.8m depth - grey by 5.2m depth	5.59
Very stiff, grey **SILTY CLAY**	6.25
GLACIAL TILL: Very dense, grey silty clay to clayey silt, some sand, gravel, cobbles and boulders	7.77
End of Borehole Practical refusal to augering at 7.77m depth (GWL @ 7.39m - July 6, 2022)	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	50	14
SS	3	75	10
SS	4	71	7
SS	5	83	2
SS	6	50	6
SS	7	42	5
SS	8	58	50+
SS	9	42	33
SS	10	25	50+
SS	11	17	50+

DEPTH (m)	ELEV. (m)
0	66.11
1	65.11
2	64.11
3	63.11
4	62.11
5	61.11
6	60.11
7	59.11

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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BORINGS BY CME-55 Low Clearance Drill

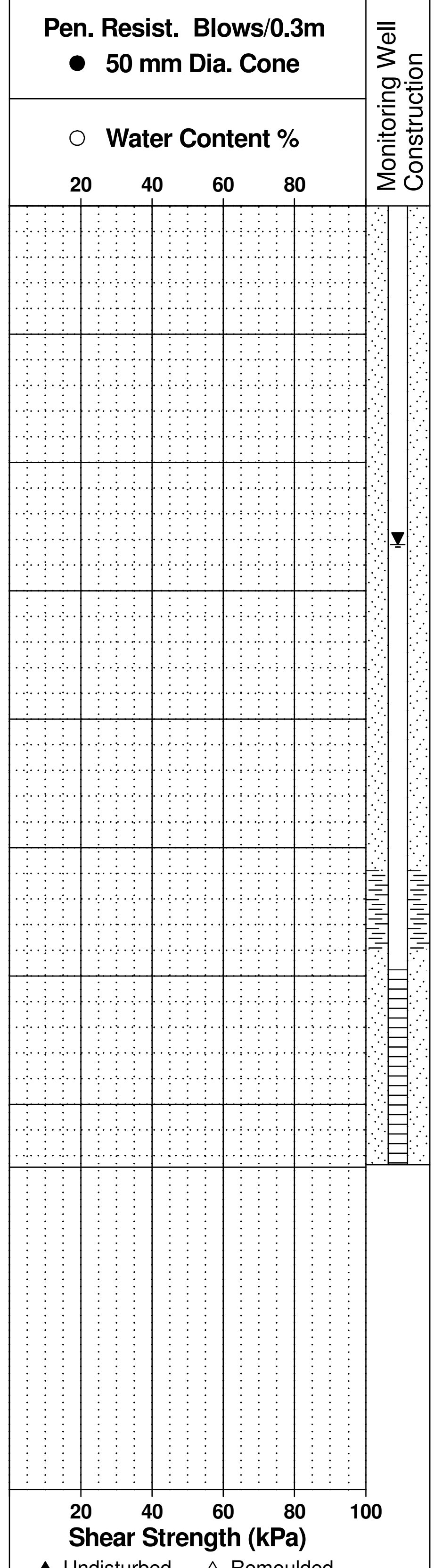
DATE June 23, 2022

FILE NO. PG5947

HOLE NO. BH12-22

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
TOPSOIL	0.28				
FILL: Brown silty sand with clay, some gravel	1.07	AU	1		
		SS	2	67	7
FILL: Grey silty clay with sand, some gravel	1.45				
FILL: Grey to brown silty sand, some gravel, trace clay	3.23	SS	3	83	13
		SS	4	42	1
GLACIAL TILL: Dense to very dense, grey silty sand to sandy silt, some clay, gravel, cobbles and boulders	5.21	SS	5	88	41
		SS	6	100	50+
		SS	7	21	50+
BEDROCK: Good to excellent quality, black shale	7.49	RC	1	100	71
		RC	2	100	90
End of Borehole					
(GWL @ 2.64m - July 6, 2022)					

DEPTH (m)	ELEV. (m)
0	67.37
1	66.37
2	65.37
3	64.37
4	63.37
5	62.37
6	61.37
7	60.37



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

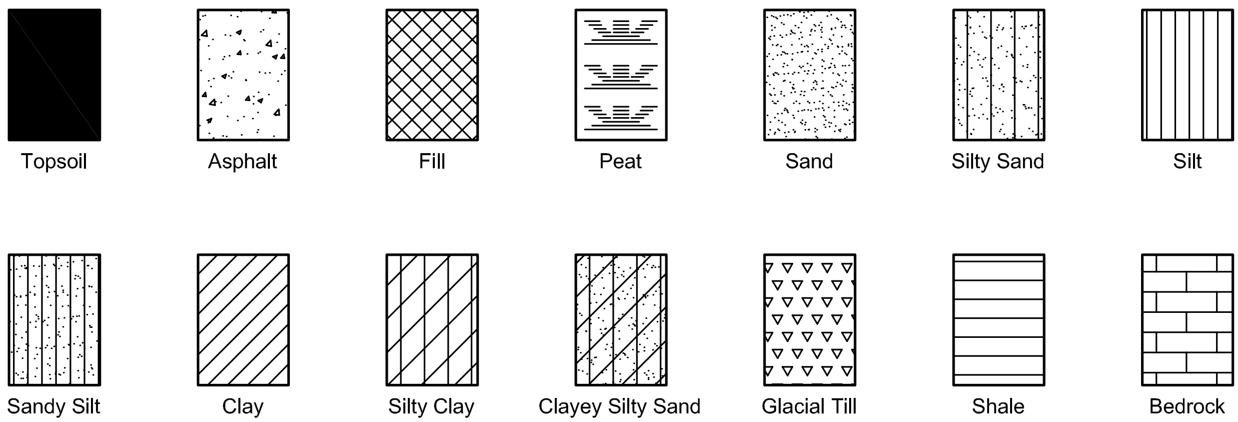
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void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

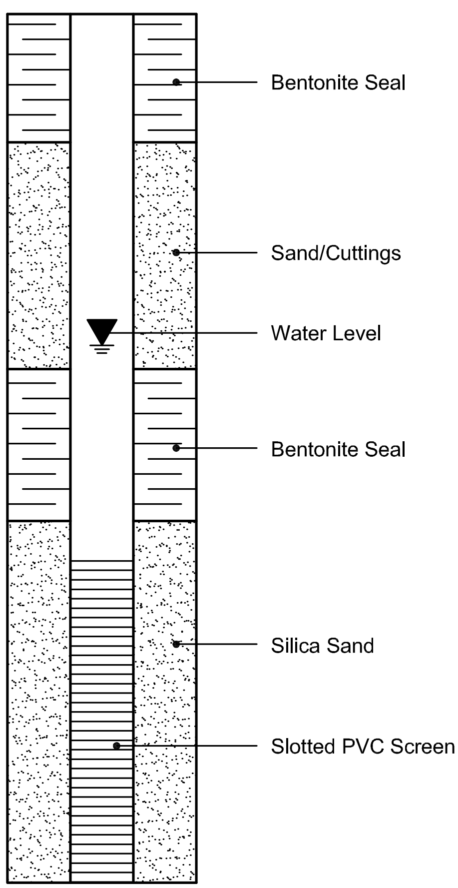
SYMBOLS AND TERMS (continued)

STRATA PLOT

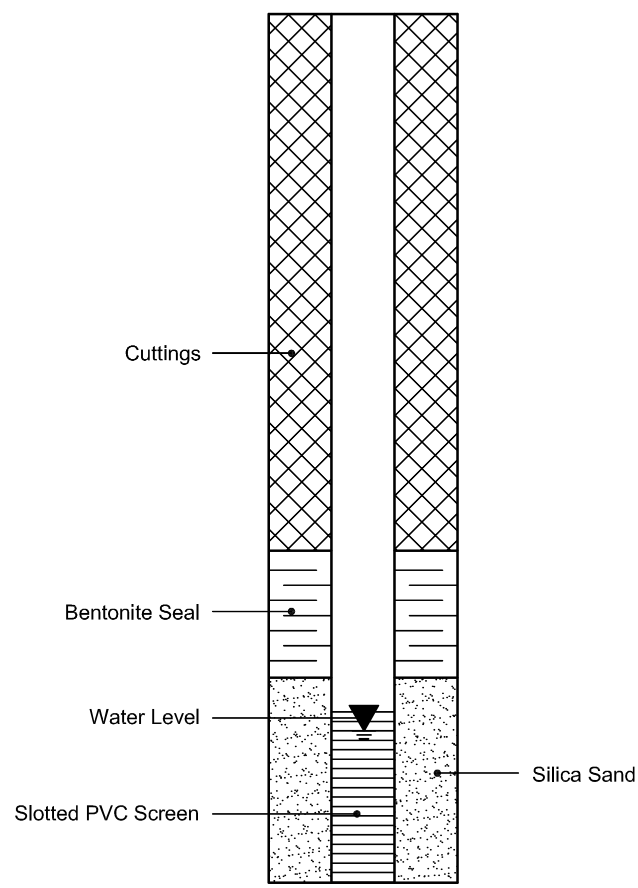


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Report Date: 24-Jun-2022

Client: **Paterson Group Consulting Engineers**

Order Date: 20-Jun-2022

Client PO: 55015

Project Description: PG5947

	Client ID:	BH2-22-SS5	-	-	-	-	-
	Sample Date:	20-Jun-22 09:00	-	-	-		
	Sample ID:	2226164-01	-	-	-		
	Matrix:	Soil	-	-	-		
	MDL/Units						
Physical Characteristics							
% Solids	0.1 % by Wt.	83.8	-	-	-	-	-
General Inorganics							
pH	0.05 pH Units	7.37	-	-	-	-	-
Resistivity	0.1 Ohm.m	5.33	-	-	-	-	-
Anions							
Chloride	5 ug/g	1040	-	-	-	-	-
Sulphate	5 ug/g	43	-	-	-	-	-

APPENDIX 2

FIGURE 1 – KEY PLAN

FIGURE 2 – AERIAL PHOTOGRAPH – 1958

FIGURE 3 – AERIAL PHOTOGRAPH – 1965

FIGURE 4 – AERIAL PHOTOGRAPH – 2019

FIGURE 5 & 6 – SEISMIC SHEAR WAVE VELOCITY PROFILES

DRAWING PG5947-1 – TEST HOLE LOCATION PLAN

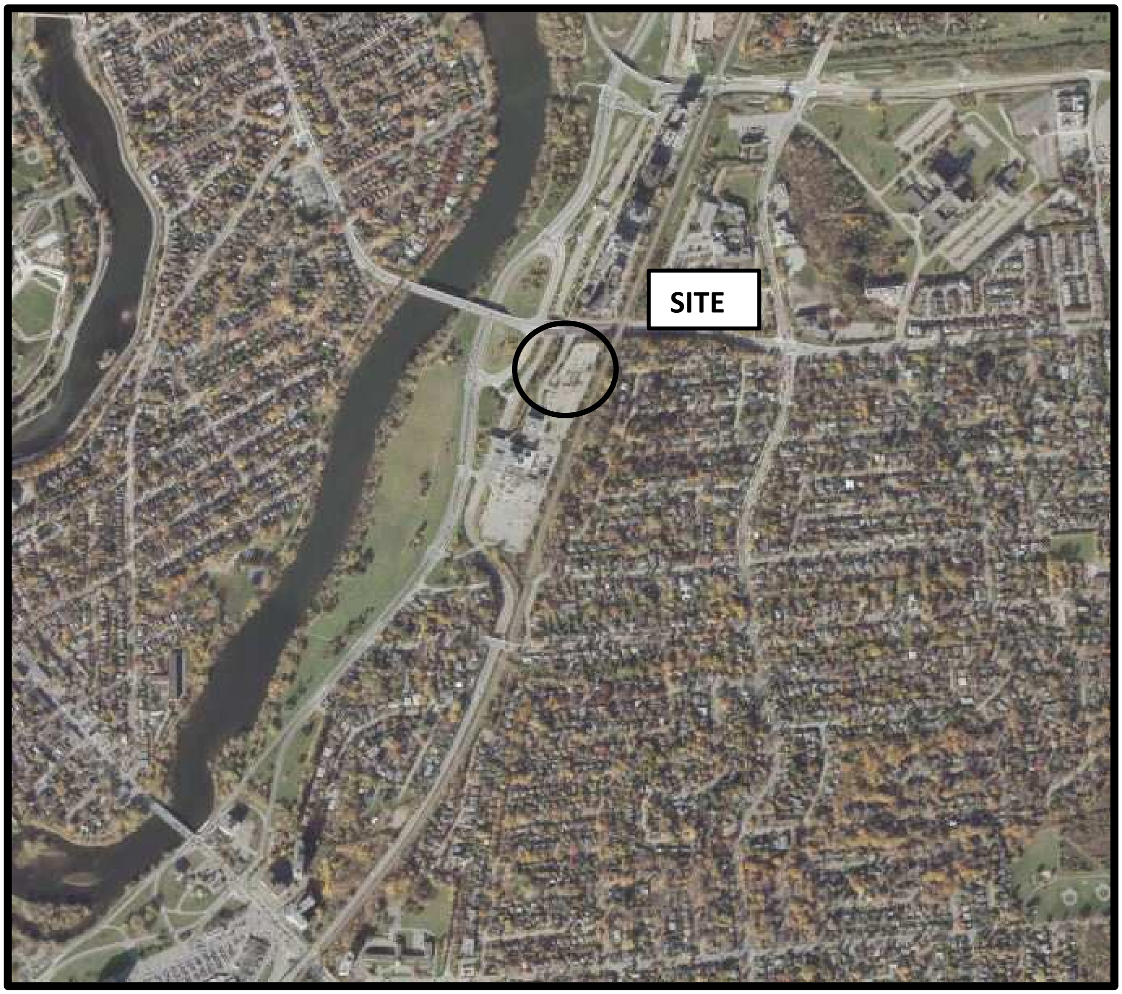


FIGURE 1

Key Plan

PATERSON GROUP

FIGURE 2

Aerial Photograph - 1958

FIGURE 3

Aerial Photograph - 1965

PATERSON GROUP

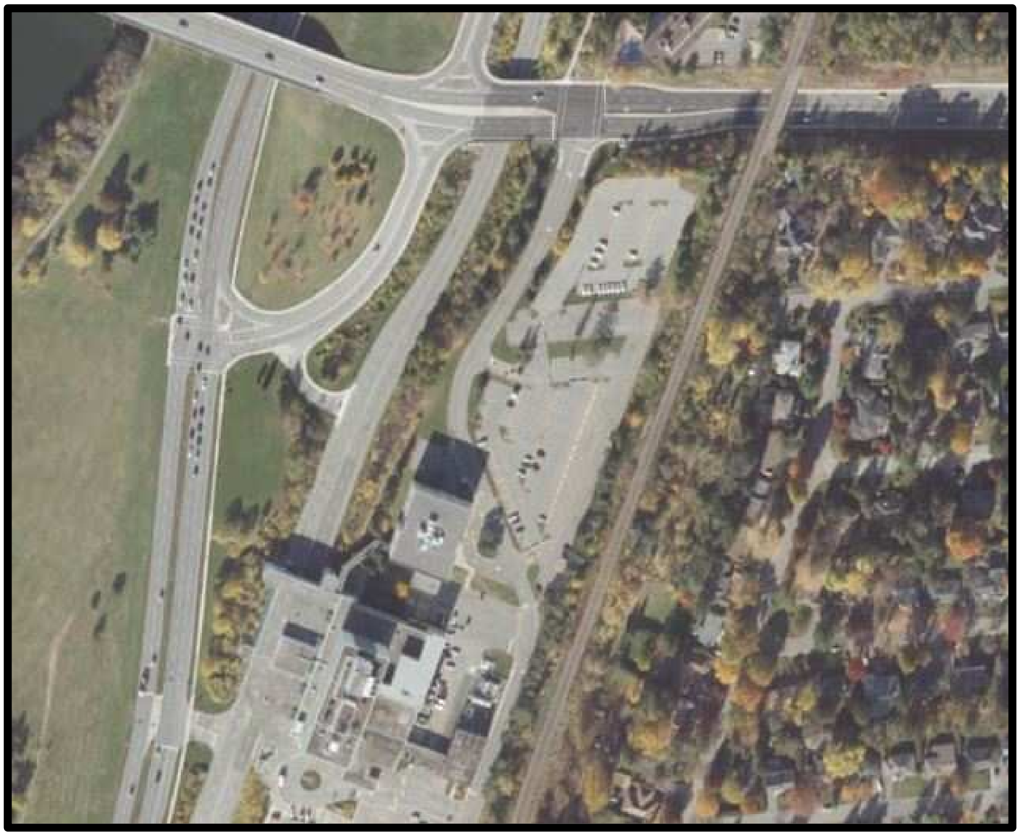


FIGURE 4

Aerial Photograph - 2019

PATERSON GROUP

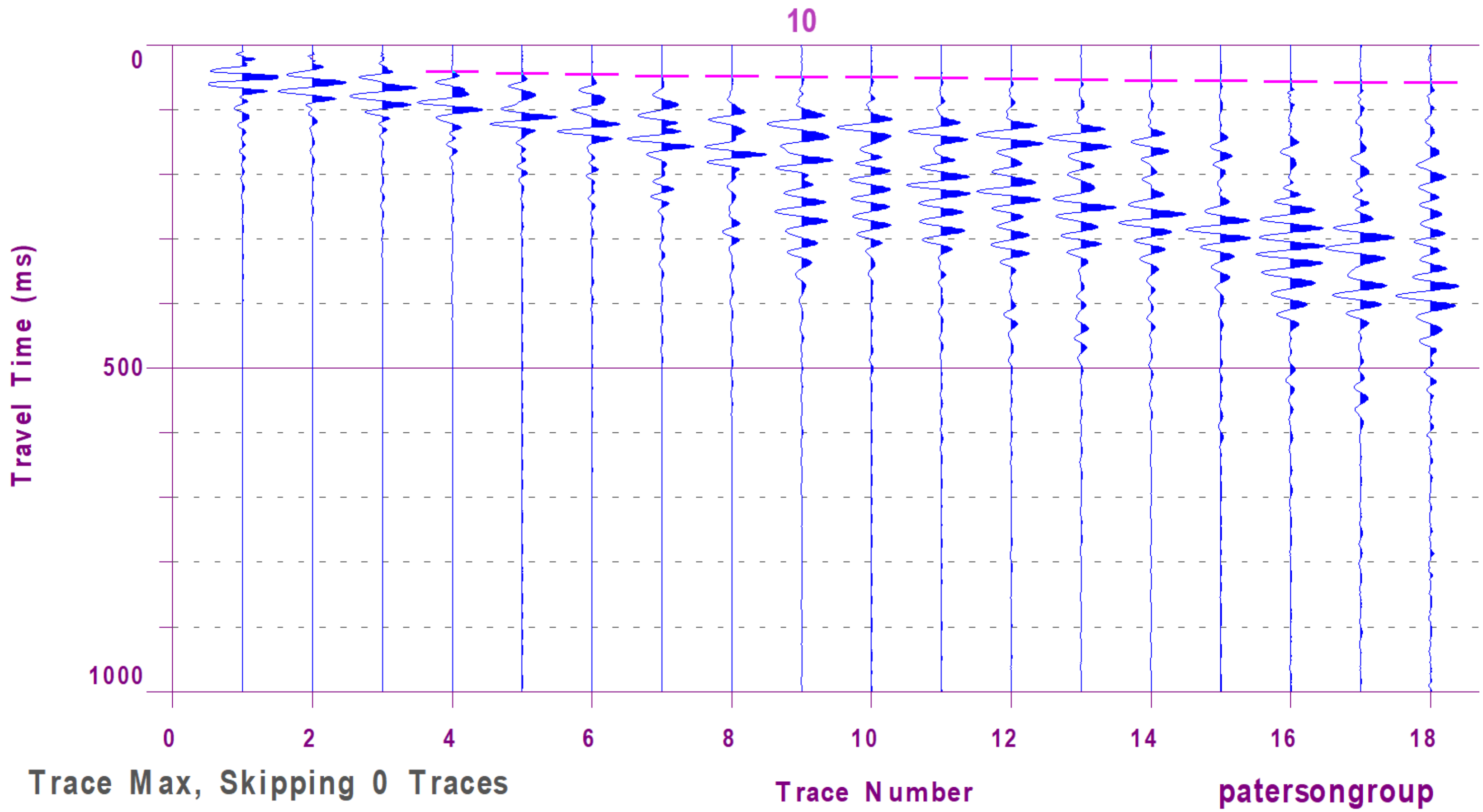


Figure 5 – Shear Wave Velocity Profile at Shot Location -3 m

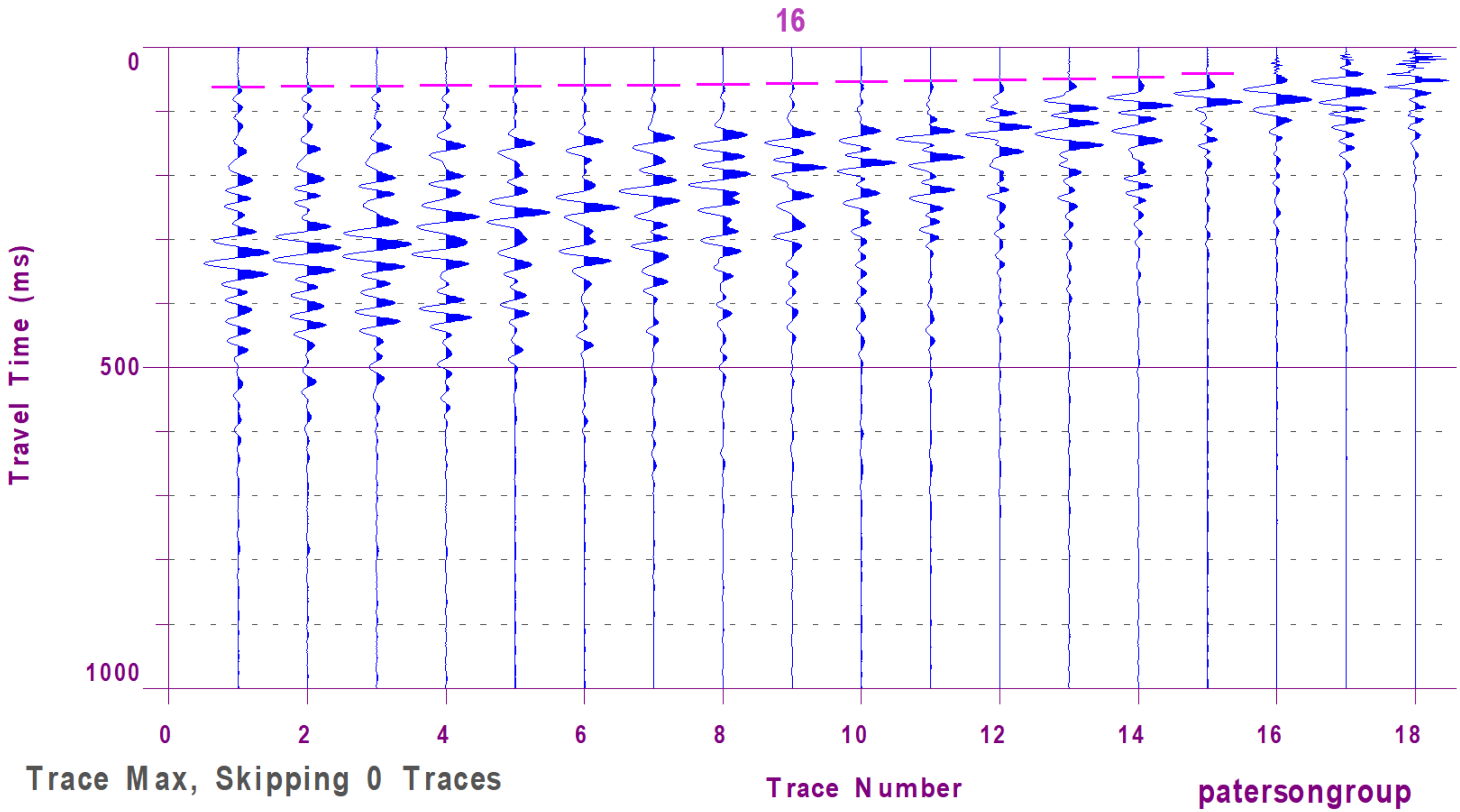
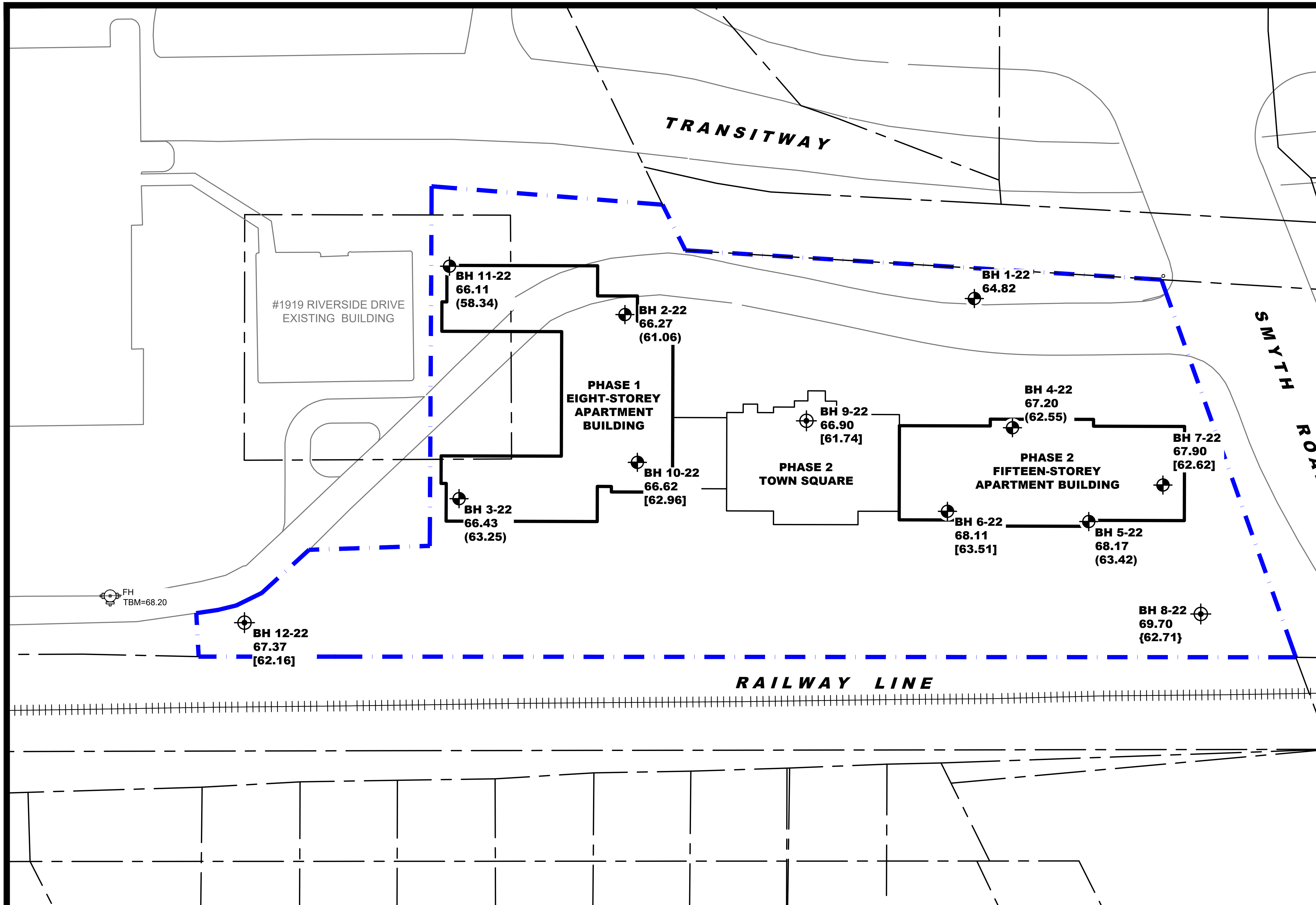


Figure 6 – Shear Wave Velocity Profile at Shot Location 36 m



LEGEND:

	BOREHOLE LOCATION
	BOREHOLE WITH MONITORING WELL LOCATION
67.90	GROUND SURFACE ELEVATION (m)
(63.42)	PRACTICAL REFUSAL TO AUGERING ELEVATION (m)
{62.71}	PRACTICAL SPLIT SPOON REFUSAL ELEVATION (m)
[62.62]	BEDROCK SURFACE ELEVATION (m)

GROUND SURFACE ELEVATIONS AT BOREHOLE LOCATIONS ARE REFERENCED TO A GEODETIC DATUM.

SCALE: 1:1000

0 10 20 30 40 50m 75m

PATERSON GROUP
SOLUTION ORIENTED ENGINEERING

NO.	REVISIONS	DATE	INITIAL

SCHLEGEL VILLAGES

GEOTECHNICAL INVESTIGATION
PROPOSED DEVELOPMENT
SMYTH ROAD AT RIVERSIDE DRIVE

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:1000	Date: 07/2022
Drawn by: NFRV	Report No.: PG5947-1
Checked by: OM	Dwg. No.: PG5947-1
Approved by: SD	Revision No.:

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