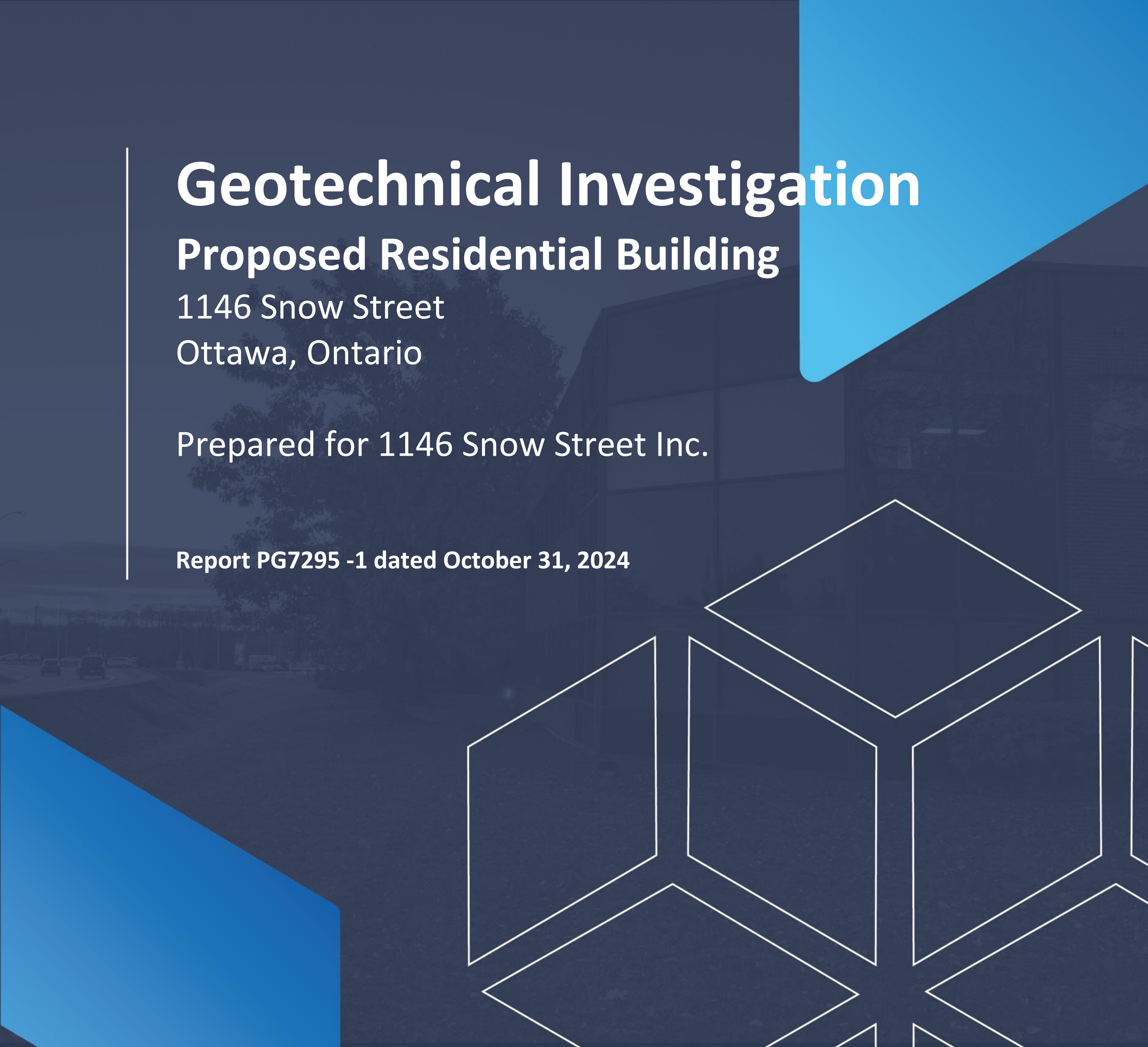

Geotechnical Investigation

Proposed Residential Building

1146 Snow Street
Ottawa, Ontario

Prepared for 1146 Snow Street Inc.

Report PG7295 -1 dated October 31, 2024

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1.0 Introduction

Paterson Group (Paterson) was commissioned by 1146 Snow Street Inc. to conduct a geotechnical investigation for the proposed residential building to be located at 1146 Snow Street in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objectives of the current investigation were to:

- ❑ Determine the subsurface and groundwater conditions by means of boreholes and existing soils information.

- ❑ Provide geotechnical recommendations pertaining to design of the proposed development including construction considerations which may affect the design.

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

2.0 Proposed Project

It is understood that the proposed development will consist of a four (4) storey residential building with one level of underground parking. At finished grade the building is expected to be surrounded asphalt paved driveways and landscaped areas.

It is anticipated that the site will be municipally serviced by water, storm, and sanitary services.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current investigation was carried out on October 1, 2024, and consisted of a total of four (4) boreholes sampled to a maximum depth of 9 m below the existing grade, throughout the subject site. A previous environmental assessment Phase 2 study was completed by Saint Lawrence Testing & Inspection Co., Ltd (SLT) in October 2022 on the subject site. During that investigation, a total of four (4) boreholes were advanced to a maximum depth of 3.0 m. However, based on the information available at the time of writing the current geotechnical report, none of the borehole logs were accessible to us. However, the borehole locations were provided and are illustrated on Drawing PG7295-1.

The borehole locations of the current investigation were determined in the field by Paterson personnel in a manner to provide general coverage of the subject site, taking into consideration existing site features and underground services. The locations of the boreholes are illustrated on Drawing PG7295-1 – Test Hole Location Plan included in Appendix 2.

The boreholes were put down using a track-mounted auger drill rig operated by a two-person crew. All fieldwork was conducted under the full-time supervision of personnel from Paterson’s geotechnical division under the direction of a senior engineer. The testing procedure for boreholes consisted of augering to the required depths and at the selected locations and sampling the overburden.

Sampling and In Situ Testing

Borehole samples were recovered from a 50 mm diameter split-spoon (SS) or the auger flights (AU). All soil samples were visually inspected and initially classified on site. The split-spoon and auger samples were placed in sealed plastic bags. All samples were transported to our laboratory for further examination and classification. The depths at which the split-spoon and auger samples were recovered from the test holes are shown as SS and AU, 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.

Rock samples were recovered from BH 1-24 and BH 2A-24 using a core barrel and diamond drilling techniques. The bedrock samples were classified on site, placed in hard cardboard core boxes and transported to Paterson's laboratory. The depths at which rock core samples were recovered from the boreholes are presented as RC on the Soil Profile and Test Data sheets in Appendix 1.

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 profiles are presented on the Soil Profile and Test Data sheets in Appendix 1 of this report.

Groundwater

Groundwater monitoring wells were installed in boreholes BH 1-24 and BH 3-24, and flexible standpipe piezometers were installed in BH 2-24 & BH 2A-24 to permit monitoring of the groundwater levels subsequent to the completion of the sampling program. All groundwater observations are noted on the Soil Profile and Test Data sheets presented in Appendix 1.

3.2 Field Survey

The borehole locations, and the ground surface elevations at the borehole locations, were selected surveyed using a handheld GPS unit and are referenced to a geodetic datum. The locations of the boreholes are presented on Drawing PG7295-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

The soil samples and the bedrock core samples were recovered from the subject site and visually examined in Paterson's laboratory to review the field logs. All samples will be stored in the laboratory for a period of one month after issuance of this report. The samples will then be discarded unless otherwise directed.

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 samples were submitted to determine the concentration of sulphate and chloride, the resistivity, and the pH of the sample. The results are presented in Appendix 1 and are discussed further in Subsection 6.7.

4.0 Observations

4.1 Surface Conditions

The site is currently vacant with low vegetation. However, historical pictures show that two small buildings on the northwestern portion of the site which were demolished. Approximately 3 to 4 m deep open excavation was observed within the western portion of the site due to the excavation of the existing house within that area. The site is bordered to the east by vacant land featuring mature vegetation composed of large trees, to the north by Snow Street, to the west and south by residential development.

The site is gently sloped down from south to north. The site is approximately 0.5 to 1 m above the grade of Snow Street.

4.2 Subsurface Profile

Fill

Generally, the subsurface profile encountered at the boreholes consists of loose fill comprised of brown silty sand, gravel and traces of organics.

Silty Sand

The fill was generally underlain by a loose to compact brown silty sand layer. An intermittent layer of compact grey silty sand to sandy silt was observed underlying the upper fill layer in BH 3-24. Furthermore, traces of gravel and organics were observed in the silty sand layer at BH 3-24.

Sandy Silt

The silty sand was noted to be underlain by a compact to dense brown sandy silt in BH 3-24 and BH 1-24, respectively.

Glacial Till

The silty sand or sandy silt was generally underlain by a compact to dense glacial till comprised of brown to grey silty sand with gravel. The glacial till was observed to transition to a compact sandy silt with traced of gravel and shale in all the boreholes except for BH 1-24.

Bedrock

Practical refusal to augering was generally encountered between 4.5 and 5.7 m below ground surface. Bedrock coring was conducted at boreholes BH 1-24 and BH 2A-24, beginning at depths of 5.66 m and 4.50 m below ground surface, respectively, and extending to the final depths of the boreholes. The bedrock consists of black shale from the Billings Formation.

Based on the RQDs of the rock core samples recovered, the bedrock quality ranges from fair to excellent, with an hardness typically between 2 and 3 on the Mohs hardness scale.

4.3 Groundwater

Groundwater monitoring wells were installed in BH 1-24 & BH 3-24. Flexible standpipe piezometers were installed in the remainder of the boreholes. Groundwater level readings were recorded on October 8, 2024, in all the boreholes. The recorded groundwater level readings are presented in the Soil Profile and Test Data sheet presented in Appendix 1.

The observed groundwater levels readings obtained from the current field program are summarized in Table 1.

Table 1 - Summary of Groundwater Level Readings

Test Hole Number	Ground Surface Elevation (m)	Groundwater Depth (m)	Groundwater Elevation (m)	Recording Date
BH 1-24*	74.78	4.19	70.59	October 8, 2024
BH 2-24	74.08	3.76	70.32	October 8, 2024
BH 2A-24	74.06	4.13	69.93	October 8, 2024
BH 3-24*	74.45	4.13	70.32	October 8, 2024

Note:
- The ground surface elevations are referenced to a geodetic datum.
- * Borehole with groundwater monitoring well

Long-term groundwater levels can also be estimated based on the observed color, moisture levels and consistency of the recovered soil samples. Based on these observations, the long-term groundwater level is anticipated at approximate depths of 3.5 to 4.5 m below the existing ground surface.

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

Foundation Design Considerations

From a geotechnical perspective, the subject site is considered satisfactory for the proposed development. The proposed residential building is anticipated to be founded on spread footings placed on a loose to compact silty bearing surface and/or approved engineered fill.

Based on available drawing, the elevator shaft footing foundation will be located at an elevation ranging from 69 to 70 m. Therefore, bedrock removal might be required to complete the elevator shaft. Hoe ramming is an option where only small quantities of bedrock need to be removed. It is expected that the bedrock removal will all be completed by hoe ramming and excavating, and that blasting will not be used for this project.

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

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and deleterious fill, such as those containing organic materials, should be stripped from under any buildings, pipe bedding and other settlement sensitive structures. Any construction debris should be entirely removed from within the perimeter of all buildings.

The existing fill material, free of organic materials, should be reviewed by Paterson personnel at the time of construction to determine if the existing fill can be left in place below paved areas.

Bedrock Removal

Based on the bedrock encountered in the area, it is expected the bedrock removal will be completed by hoe-ramming. In areas of weathered bedrock and where only a small quantity of bedrock is to be removed, bedrock removal may be completed by an excavator.

Vibration may be induced by rock removal activities. As a general guideline, peak particle velocities (measured at the structures) should not exceed 50 mm/s during the blasting program to reduce the risks of damage to the existing structures. The blasting operations should be planned and conducted under the supervision of a licensed professional engineer who is also an experienced blasting consultant.

Excavation side slopes in sound bedrock can be carried out using almost vertical side walls. A minimum 1 m horizontal ledge should be left between the bottom of the overburden excavation and the top of the bedrock surface to provide an area to allow for potential sloughing or to provide a stable base for the overburden shoring system.

It is not recommended that the shale bedrock be reused under settlement sensitive structure or as backfill in the active frost layer (2.1 m).

Lean Concrete In-Filled Trenches

Where bedrock is encountered below the design underside of footing elevation, consideration should be given to excavating vertical trenches to expose the underlying bedrock surface and backfilling with lean concrete (15 to 20 MPa 28-day compressive strength). Typically, the excavation sidewalls will be used as the form to support the concrete. 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 bedrock.

The effectiveness of this operation will depend on the ability of maintaining vertical trenches until the lean concrete can be poured. It is suggested that once the bottom of the excavation is exposed, an assessment should be completed to determine the water infiltration and stability of the excavation sidewalls extending to the bedrock surface.

The trench excavation should be at least 150 mm wider than all sides of the footing at the base of the excavation. The excavation bottom should be relatively clean using the hydraulic shovel only (workers will not be permitted in the excavation below a 1.5 m depth). Once approved by the geotechnical engineer, lean concrete can be poured up to the proposed founding elevation.

Footings placed on lean concrete filled trenches extending to the bedrock surface can be designed using a factored bearing resistance provided in subsection 5.3.

Vibration Considerations

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

The following construction equipment could be the source of vibrations: piling equipment, hoe ram, compactor, dozer, crane, truck traffic, etc. The construction of a shoring system with soldier piles or sheet piling will require these pieces of equipment. Vibrations, whether caused by blasting operations or by construction operations, could be the source of detrimental vibrations on the nearby buildings and structures. Therefore, all vibrations are recommended to be limited.

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

Considering there are several sensitive buildings near the subject site, consideration to lowering these guidelines is recommended. These guidelines are above perceptible human level and, in some cases, could be very disturbing to some people, a pre-construction survey is recommended be completed to minimize the risks of claims during or following the construction of the proposed building.

Bedrock Excavation Face Reinforcement

Where 1:1 sloping cannot be accommodated in weathered or fractured horizontal rock anchors, shotcrete and/or chain link fencing connected to the excavation face may be required at specific locations to prevent bedrock pop-outs, especially in areas where bedrock fractures are conducive to the failure of the bedrock surface during construction. The requirement for bedrock excavation face reinforcement should be evaluated by Paterson personnel during the excavation operations.

Fill Placement

Fill used for grading purposes beneath the proposed buildings should consist 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 to the site. It should be placed in lifts no greater than 300 mm in thickness and compacted using suitable compaction equipment for the specified lift thickness. Fill placed beneath the building areas should be compacted to at least 98% of its 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. These materials should be spread in thin lifts and be compacted at minimum by the tracks of the spreading equipment to minimize voids. If these materials are to be used to build up the subgrade level for areas to be paved, they should be compacted in thin lifts to a minimum density of 98% of their respective SPMDD.

Non-specified existing fill and site-excavated soils are not suitable for use as backfill against foundation walls unless used in conjunction with a geo-composite drainage membrane such as Miradrain G100N or Delta Drain 6000 connected to a perimeter drainage system a composite drainage membrane.

If excavated rock is to be used as fill, it should be suitably fragmented to produce a well-graded material with a maximum particle size of 150 mm. Where the fill is open graded, a blinding layer of finer granular fill and/or a woven geotextile may be required to prevent adjacent finer materials from migrating into the voids, with associated loss of ground and settlements. This can be assessed at the time of construction. Site-generated blast rock fill should be compacted using a suitably sized smooth drum vibratory roller when considered for placement. Shale bedrock material should not be used under settlement sensitive structure and pavement areas. The material can be used in landscape areas where settlement will be of minimal impact.

Under winter conditions, if snow and ice is present within the blast rock fill below future basement slabs, then settlement of the fill should be expected and support of a future basement slab and/or temporary supports for slab pours will be negatively impacted and could undergo settlement during spring and summer time conditions. Paterson personnel should complete periodic inspections during fill placement to ensure that snow and ice quantities are minimized.

5.3 Foundation Design

Bearing Resistance Values

Based on current available plans for the project, the building will have one underground basement level. It is expected that the majority of the footings in that area will be installed between an approximate elevation of 73.0 to 72.0 m, while the fitness room footing is expected to be located between an approximate elevation of 71.0 to 72.0 m within the undisturbed compact silty sand. However, the elevator shaft will be located between an approximately elevation of 69.0 to 70.0 m on either compact to dense glacial till, clean shale bedrock surface or engineered fill.

Footing placed on the undisturbed loose to compact silty sand can be designed using a bearing resistance value at serviceability limit states (SLS) of **100 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **150 kPa**. A geotechnical resistance factor of 0.5 was applied to the above noted bearing resistance value at ULS.

Footing placed on the undisturbed compact to 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 **350 kPa**. A geotechnical resistance factor of 0.5 was applied to the above noted bearing resistance value at ULS.

Footings placed on a soil bearing surface and 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.

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.

If the footings rest entirely on the bedrock, footings placed at this elevation can be designed using a service compression strength value (serviceability limit states, SLS) of **500 kPa** and an ultimate compression strength value (ultimate limit states, ULS) of **750 kPa**.

Settlement

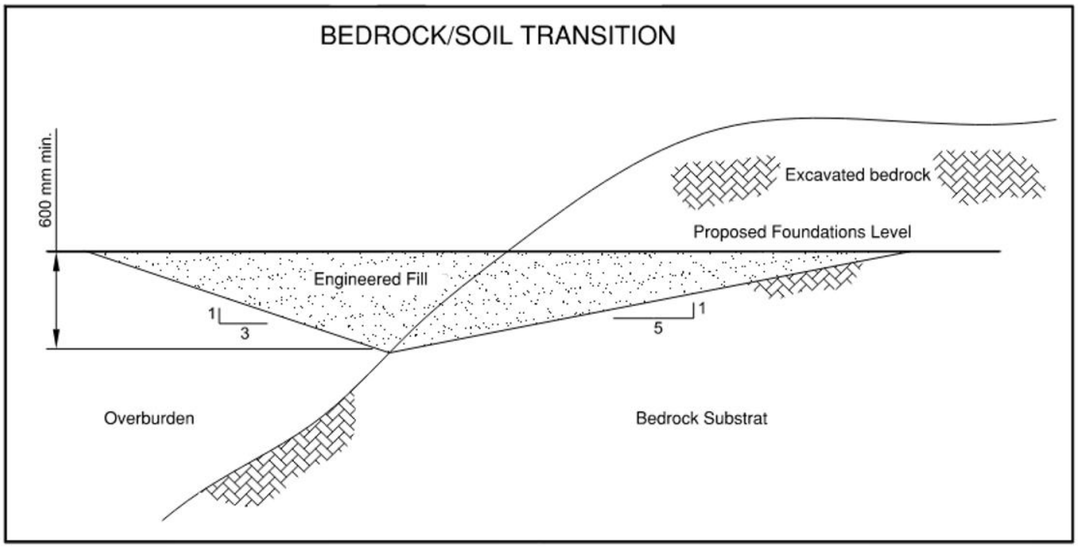
The total and differential settlement will be dependent on characteristics of the proposed buildings. For design purposes, the total and differential settlements are estimated to be 25 to 20 mm, respectively.

Footings bearing on an acceptable bedrock bearing surface or lean concrete trench extending to bedrock, and designed for the bearing resistance value provided herein will be subjected to negligible potential postconstruction total and differential settlements.

Bedrock/Soil Transition

Where a building is founded partly on bedrock and partly on soil, particularly withing the elevator shaft location, it is recommended to provide Bedrock/soil transition to reduce the risks of excessive differential settlements. This transition involves profiling the rock with a slope of 1.0 vertical to 5.0 horizontal, while the soils will be profiled with a slope of 1.0 vertical to 3.0 horizontal, reaching a depth of 600 mm at their point of contact relative to the projected foundation level.

The excavation should be filled with clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II placed in maximum 300 mm thick loose lifts and compacted to a minimum of 98% of the standard Proctor maximum dry density (SPMDD). The figure below illustrates a cross-section of a Bedrock/soil transition.



In case it is not possible to follow the detailed profile mentioned above, additional reinforcing bars should be integrated where the transition between the soil and the bedrock occurs. This will reinforce the section of the foundation footing affected by this transition.

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.

Above the groundwater level, adequate lateral support is provided to the in-situ bearing medium soils 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.

Adequate lateral support is provided to bedrock bearing medium when a plane extending down and out from the bottom edges 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 heavily fractured, weathered bedrock and/or overburden bearing medium will require a lateral support zone of 1H:1V (or flatter).

Protection of Potential Expansive Bedrock

The swelling of the shale bedrock formation on surrounding site is well documented and known. The structure should be designed to mitigate the effect of shale swelling.

The swelling of shale formation in Ottawa is attributed to the weathering of pyrite in the formation. Pyrite, most often in the form of iron disulphide mineral, oxidates under the presence of air and water to create sulfuric acid. When calcite is present it reacts with the sulphuric acid produced by oxidation of pyrite to form gypsum. Most often the reaction will take place in a weathered layer of the bedrock allowing for better movement of air and water. The reaction may be aided by autotrophic bacteria.

All excavation should be sub-excavated with a minimum of 50 to 75 mm to account for the recommended mud slab thickness. The horizontal surface should be covered with a minimum full strength (25 MPa) concrete at least 50 mm in thickness. It is expected that most of the excavation will be sloped to complete the project and that no vertical faces will be placed directly against the building’s foundation wall. However, if a vertical bedrock surface is in direct contact with a foundation wall, then the vertical bedrock surface should also be protected using a vertical form of shotcrete to create a 50 mm coating. It is recommended to reinforce the vertical surface with a light wire mesh. To avoid lateral pressure from potential expansion a minimum 25 mm of compressible insulation should be used. The side and bottoms of excavation for any underfloor services, such as elevator pit, sump pits, sanitary piping, mechanical rooms etc, should be fully protected. Consideration should be taken at fully in-filling the smaller trench with concrete. The concrete should be sulphate resistant as the above noted reaction may also react with the concrete.

Construction planning should ensure the shale surfaces are not left exposed for more than 24hrs. If the area is not ready for coating a minimum of 500 mm of shale or existing soil should remain in place until final excavation at grade is completed.

Furthermore, any cuts or breaks created in the coating during the construction or accidentally should be sealed and repaired within 24 hrs. It is recommended to avoid heavy equipment traffic on the protective coating.

If the project is designed not to extend to the bedrock surface, a minimum of 300 mm of soil should remain in place on top of the bedrock surface to ensure sufficient separation and protection.

5.4 Design for Earthquakes

Seismic Shear Wave Velocity Testing

Seismic shear wave velocity testing was completed for the subject site to accurately determine the applicable seismic site classification for the proposed building in accordance with Table 4.1.8.4.A of the Ontario Building Code 2012.

The acquisition and interpretation of the shear wave velocity testing was completed by Paterson personnel. The results of the shear wave velocity test are provided in Figure 2 and 3 in Appendix 2 of the present report.

Field Program

The seismic array testing location was places as presented in Drawing PG7295-1 – Test Hole Location Plan, attached to the present report. Paterson field personnel placed 24 horizontal 4.5 Hz geophones mounted to the surface by means of two 75 mm ground spikes attached to the geophone land case. The geophones were spaced at 1 m intervales 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 was used to strike an I-Beam seated into the ground surface which in turn created a polarized shear wave that travels into the subsurface. Each strike of the hammer on the I-Beam was considered a shot. At each shot, the hammer trigger switch sent a signal to the seismograph to start the recording of the corresponding data.

The hammer shots were repeated between four (4) to eight (8) times at each shot location to improve the signal to noise ratio, and in a forward and reverse direction (i.e.- striking both sides of the I-Beam seated parallel to the geophone array). In addition, the shot locations were completed at 10, 1.5 and 1.0 m away from the first and last geophone, and at the center of the seismic array.

Data Processing and Interpretation

Interpretation of the shear wave velocity testing was completed by Paterson personnel. Shear wave velocity was estimated using reflection/refraction methods. The interpretation is performed by recovering arrival times from direct, reflected and refracted waves.

The interpretation is repeated at each shot location to provide an average shear wave velocity, V_{s30}, of the upper 30 m profile, immediately below the foundation of the building. The 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. Based on our testing results, the average overburden shear wave velocity is **224 m/s**, while the bedrock shear wave velocity is **1,790 m/s**. Further, the testing results indicate the average overburden thickness to be approximately 5 m.

Site Class for Building Founded Entirely Upon Bedrock

For foundations placed directly or indirectly (i.e., using lean-concrete in-filled trenches) upon a clean, sounded bedrock surface, the V_{s30} was calculated using the standard equation for average shear wave velocity provided in the OBC 2012 and as presented below:

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

$$V_{s30=} \frac{30\ m}{\left(\frac{30\ m}{1{,}790\ m/s}\right)}$$

$$V_{s30=}\ 1{,}790\ m/s$$

Based on the results of the shear wave velocity testing, the average shear wave velocity V_{s30} for the proposed building founded directly or indirectly on bedrock is **1,790 m/s**. Therefore, a **Site Class A** is applicable for design of the proposed building as per Table 4.1.8.4.A of the OBC 2012. The soils underlying the subject site are not susceptible to liquefaction.

Site Class for Building Founded on Soil Within 3 m of Bedrock

For foundations whose footings are founded on soil and where bedrock is anticipated to be located within a maximum of 3 m of the founding depth, the V_{s30} was calculated using the standard equation for average shear wave velocity provided in the OBC 2012 and as presented below:

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

$$V_{s30=} \frac{30\ m}{\left(\frac{3\ m}{224\ m/s} + \frac{27\ m}{1{,}790\ m/s}\right)}$$

$$V_{s30=}\ 1{,}053\ m/s$$

Based on the results of the shear wave velocity testing, the average shear wave velocity V_{s30} for the proposed building within 3 m of the bedrock surface is **1,053 m/s**. Therefore, a **Site Class B** is applicable for design of the proposed building, as per Table 4.1.8.4.A of the OBC 2012. The soils underlying the subject site are not susceptible to liquefaction.

5.5 Basement Slab and Slab on Grade

With the removal of all topsoil and deleterious fill, containing organic matter, within the footprints of the proposed building, the native soil surface, bedrock or approved engineered fill pad will be considered an acceptable subgrade on which to commence backfilling for floor slab construction.

It is expected that the basement area will be mostly used for storage or other purposes, and a concrete floor slab will be used. Therefore, it is recommended that the upper 200 mm of sub-slab fill consist of 19 mm clear crushed stone.

An engineered fill such as an OPSS Granular A or Granular B Type II compacted to 98% of its SPMDD could be placed around the proposed footings. Alternatively, excavated silty sand fill could be used as select subgrade material around the proposed building footings and under the finish floor slabs where more than 300 mm of granular material would be required. Placement and compaction of the material should be reviewed and approved by Paterson at the time construction. Any soft areas should be removed and backfilled with appropriate backfill material prior to placing any 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.

A subfloor drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided in the clear stone backfill under the lower basement floor. The spacing of the underfloor drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed. This is discussed further in Section 6.1 of this report.

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.

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.

It is recommended that a minimum of 500 mm be kept from vertical rock face to foundation wall to avoid supplemental lateral pressure from potential expansive shale.

Where blind side pours are proposed on a vertical rock face, a supplemental layer of a minimum 25 mm of compressible insulation material should be used in combination with the shale protection described in sub section 5.3.

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. Two distinct conditions, static and seismic, should be reviewed for design calculations. The corresponding parameters are presented below.

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 Ottawa is 0.32 g according to OBC 2015. 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 2015.

5.7 Pavement Structure

The flexible pavement structure presented in Table 2 and Table 3 should be used for driveways and car only parking areas and at grade access lanes and heavy loading parking areas.

Table 2 - Recommended Pavement Structure – Driveways Car Only Parking Areas

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

Table 3 - Recommended Pavement Structure – Access Lanes

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
400	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Either fill, in situ soil, select subgrade material 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 I or Type II material.

The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 98% of the material's SPMDD using suitable compaction equipment.

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for parking areas and local roadways and PG 64-34 asphalt cement should be used for roadways with bus traffic. The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 100% of the material’s SPMDD using suitable vibratory equipment.

The proposed pavement structure, where it abuts the existing pavement, should match the existing pavement layers. It is recommended that a 300 mm wide and 50 mm deep stepped joint be provided where the new asphalt layer joins with the existing asphalt layer to provide more resistance to cracking at the joint.

Pavement Structure Drainage

Satisfactory performance of the pavement structure is largely dependent on keeping the contact zone between the subgrade material and the base stone in a dry condition.
Failure to provide adequate drainage under conditions of heavy wheel loading can result in the fine subgrade soil being pumped into the voids in the stone subbase, thereby reducing its load carrying capacity.

Consideration should be given to installing subdrains during the pavement construction as per City of Ottawa standards. The invert of the subdrain pipe is recommended to be located a minimum depth of 300 mm below the pavement structure subgrade and located centrally along the roadway alignment. The subdrain pipe is recommended to consist of a minimum 150 mm diameter corrugated and perforated plastic pipe surrounded by a minimum of 150 mm of 10 mm clear crushed stone on all of its sides. The clear stone layer is recommended to be wrapped by a geotextile layer. The drains should be connected to a positive outlet. The subgrade surface should be crowned to promote water flow to the drainage lines.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Based on the available drawing, the underground basement level will be at least 2.0 m lower than the existing natural grade, while the fitness room will be located at least 3.6 m below the existing natural grade which will primarily consist of a silty sand deposit. However, the elevator shaft footing is expected to be located 4.5 m below the existing grade, which primarily consists of glacial till or shale bedrock. It is expected that localized dewatering will be achievable for the project.

It is recommended that a perimeter foundation drainage system be provided for the proposed structures. It is recommended that the drainage system consist of the following:

❑ Where foundation walls will be double side poured, the foundation dampproofing and drainage board is recommended to be installed directly onto the exterior foundation wall between the footing and finished grade.

❑ The foundation drainage boards should be overlapped such that the bottom end of a higher board is placed in front of the top end of a lower board. All endlaps of the drainage board sheets should overlap abutting sheets by a minimum of 150 mm. All overlaps should be sealed with a suitable adhesive and/or sealant material approved by Paterson.

Waterproofing layers for horizontal buried portion of the structure surfaces should overlap across and below the top end lap of the vertically installed composite foundation drainage board to mitigate the potential for water to migrate between the drainage board and foundation wall. A perimeter drainage system consisting of a 150 mm diameter perforated corrugated pipe should be installed around the foundation perimeter and should have positive drainage to the sewer or sump pit.

Underfloor Drainage

The interior underfloor drainage system will be required to control water infiltration below the underground basement level slab and fitness room level slab, and redirect water from the buildings foundation drainage system to the buildings sump pit(s) or gravity outlet. The interior perimeter and underfloor drainage pipe should consist of a 150 mm diameter corrugated perforated plastic pipe sleeved with a geosock.

The underfloor drainage pipe should be placed in each direction of the basement floor span and connected to the perimeter drainage pipe. The interior drainage pipe should be provided tee-connections to extend pipes between the perimeter drainage line and the HDPE-face of the composite foundation drainage board via the foundation wall sleeves. The spacing of the underfloor drainage system should be confirmed by Paterson once the foundation layout and sump system location has been finalized.

Elevator Pit Waterproofing

The elevator shaft exterior foundation walls should be waterproofed to avoid any infiltration into the elevator pit. It is recommended that a waterproofing membrane, such as Colphene Torch'n Stick (or approved other) be applied to the exterior of the elevator shaft foundation wall and a waterproofing membrane such as BSW H (or approved other) be applied below the elevator pit footing (horizontal application). Please refer to Figure 4, included in Appendix 2.

The Colphene Torch'n Stick waterproofing membrane should extend over the vertical portion of the slab and down to the top of the footing in accordance with the manufacturer's specifications. The BSW H waterproofing membrane should be placed horizontaly below the footing and extend up the sides of the footing. The Colphene Toch'n Stick should overlap the BSW H waterproofing membrane.

A continuous PVC waterstop such as Southern waterstop 14RCB or equivalent should be installed within the interface between the concrete base slab below the elevator shaft foundation walls.

The 150 mm diameter perforated corrugated pipe underfloor drainage should be placed along the perimeter of the exterior sidewalls and provided a gravity connection to the sump pump basin or the elevator sump pit.

The foundation wall of the elevator shaft and buildings sump pit should host a PVC sleeve to allow any water trapped within the interior side of the structures to be discharged to the associated sump pump. A minimum 100 mm diameter drainage pipe should extend from the sleeve towards the associated drainage system by gravity drainage and mechanical connection to the associated system. Also, the contractor should ensure that the opening is properly sealed to prevent water from entering the subject structure.

A protection board should be placed over the waterproofing membrane to protect the waterproofing membrane from damage during backfilling operations. The area between the pit structure and bedrock/soil excavation face can be in-filled with lean concrete, OPSS Granular A or Granular B Type II crushed stone.

It should be noted that a waterproofed concrete (with Xypex Additive, or equivalent) is optional for this waterproofing configuration.

Foundation Backfill

Above the bedrock surface, backfill against the exterior sides of the foundation walls should consist of free draining non frost susceptible granular materials. The greater part of the site excavated materials will be frost susceptible and, as such, are not recommended for re-use as backfill against the foundation walls, unless used in conjunction with a drainage geocomposite, such as Miradrain G100N or Delta Drain 6000, connected to the perimeter foundation drainage system. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should otherwise be used for this purpose.

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

The underground basement level as well as the fitness room level are expected to be heated. Thus, perimeter footings of the underground basement structures are required to be insulated against the deleterious effects of frost action. A minimum of 1.5 m of soil cover alone, or a minimum of 0.6 m of soil cover, in conjunction with adequate foundation insulation, should be provided. More details regarding foundation insulation can be provided, if requested.

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

However, foundations which are founded directly on clean, surface-sounded bedrock with no cracks or fissures, and which is approved by Paterson at the time of construction, is not considered frost susceptible and does not require soil cover.

Where the bedrock is considered frost susceptible (i.e., weathered bedrock or bedrock with significant fissures filled with soil), foundation insulation will need to be provided. Alternatively, frost susceptible bedrock will need to be removed and replaced with lean concrete (minimum 15 to 20 MPa 28-day strength). It is recommended Paterson field personnel review the frost susceptibility of bedrock surface located within 1.8 m of finished grade.

6.3 Excavation Side Slopes

Temporary Side Slopes

The temporary excavation side slopes anticipated should either be excavated to acceptable slopes or retained by shoring systems from the beginning of the excavation until the structure is backfilled. Insufficient room is expected for the majority of the excavation to be constructed by open-cut methods (i.e. unsupported excavations).

The excavation side slopes above the groundwater level extending through fill material and weathered shale bedrock should be excavated at 1H:1V or shallower. The flatter slope is required for excavation below groundwater level. The subsurface soils are considered to be 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.

Excavation in the fractured and sound bedrock is expected to be carried with near vertical side walls. Shotcrete protection is expected to be required in the fractured layer of bedrock for worker's protection where the vertical excavation is greater than 1.2m. Paterson should review the excavation side walls during construction.

Excavation side slopes carried out for the building footprint are recommended to be provided surface protection from erosion by rain and surface water runoff if shoring is not anticipated to be implemented. This can be accomplished by covering the entire surface of the excavation side-slopes with tarps secured between the top and bottom of the excavation and approved by Paterson personnel at the time of construction. It is further recommended to maintain a relatively dry surface along the bottom of the excavation footprint to mitigate the potential for sloughing of side-slopes.

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.

Temporary Shoring

Temporary shoring may be required to support the overburden soils to complete the required excavations where insufficient room is available for open cut methods.

The shoring requirements designed by a structural engineer specializing in those works will depend on the depth of the excavation, the proximity of the adjacent structures and the elevation of the adjacent building foundations and underground services. The design and implementation of these temporary systems will be the responsibility of the excavation contractor and their design team.

Inspections and approval of the temporary system will also be the responsibility of the designer. Geotechnical information provided below is to assist the designer in completing a suitable and safe shoring system. The designer should take into account the impact of a significant precipitation event and designate design measures to ensure that a precipitation will not negatively impact the shoring system or soils supported by the system.

Any changes to the approved shoring design system should be reported immediately to the owner's structural design prior to implementation.

The temporary system could consist of soldier pile and lagging system or stacked precast concrete blocks. Any additional loading due to street traffic, construction equipment, adjacent structures, and facilities, etc., should be included to the earth pressures described below.

These systems could be cantilevered, anchored, or braced. Generally, it is expected that the shoring systems will be provided with tie-back rock anchors to ensure their stability. The shoring system is recommended to be adequately supported to resist toe failure and inspected to ensure that the sheet piles extend well below the excavation base.

It should be noted if consideration is being given to utilizing a raker style support for the shoring system that lateral movements can occur and the structural engineer should ensure that the design selected minimizes these movements to tolerable levels.

The earth pressures acting on the shoring system may be calculated using the following parameters provided in Table 4.

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

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

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

Table 4 – Geotechnical Parameters Lateral Properties

Material Description	Unit Weight (kN/m^3)		Friction Angle (^{o}) φ'	Friction Factor, tanδ	Earth Pressure Coefficients		
	Drained γ_{dr}	Effective γ'			Active K_A	At-Rest K_O	Passive K_P
Existing Fill	18	10	30	0.4	0.33	0.5	3
Silty Sand	18	10	30	0.4	0.33	0.5	3
Engineered Fill (Granular A)	22	13.5	40	0.5	0.22	0.36	4.6
Engineered Fill (Granular B Type II)	22.5	14	42	0.5	0.2	0.33	5.04
Bedrock	23.5	15.2	55	0.6	0.18	0.1	10

Notes:

I. The earth pressure coefficients provided are for horizontal profile.

II. For soil above the groundwater level the “drained” unit weight should be used and below groundwater level the “effective” unit weight should be used.

III. Existing fill should be free of significant amounts of deleterious material such as those containing organic materials, wood chips and peat. The fill should be approved by Paterson prior to placement

6.4 Pipe Bedding and Backfill

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

The pipe bedding for the sewer and water pipes should consist of at least 150 mm of OPSS Granular. However, when the bedding is located within bedrock subgrade, a minimum of 300 mm of OPSS Granular A should be placed for bedding for sewer or water pipes. The material should be placed in a maximum 225 mm thick loose lifts and compacted to a minimum of 99% of its SPMDD. The bedding material should extend at least to the spring line of the pipe.

The cover material, which should consist of OPSS Granular A, should extend from the spring line of the pipe to at least 300 mm above the obvert of the pipe. The material should be placed in maximum 225 mm thick lifts and compacted to a minimum of 99% of its SPMDD.

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

Any stones greater than 200 mm in their longest dimension should be removed from these materials prior to placement. Well fractured bedrock should be acceptable as backfill for the lower portion of the trenches when the excavation is within bedrock provided the rock fill is placed only from at least 300 mm above the top of the service pipe and that all stones are 300 mm or smaller in their longest dimension.

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

6.5 Groundwater Control

Groundwater Control for Building Construction

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

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, 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.

6.6 Winter Construction

Precautions must be taken if winter construction is considered for this project. The bedrock and overburden material present on site are considered frost susceptible.

Where excavations are completed in proximity of existing structures which may be adversely affected due to the freezing conditions, in particular, where a shoring system is constructed, 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.

In the event of construction during below zero temperatures, the founding stratum should be protected from freezing temperatures by the installation of straw, propane heaters and/or glycol lines and tarpaulins or other suitable means. 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 foundation is protected with sufficient soil cover to prevent freezing at founding level.

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

Under winter conditions, if snow and ice is present within the blast rock or other imported fill below future basement slabs, then settlement of the fill should be expected and support of a future basement slab and/or temporary supports for slab pours will be negatively impacted and could undergo settlement during spring and summertime conditions. Paterson personnel should complete periodic inspections during fill placement to ensure that snow and ice quantities are minimized in settlement-sensitive areas.

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 samples 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 non-aggressive to slightly aggressive corrosive environment.

7.0 Recommendations

For the foundation design data provided herein to be applicable that a material testing and observation services program is required to be completed.

The following aspects be performed by the geotechnical consultant:

❑ Review preliminary and detailed grading, servicing, and structural plan(s) from a geotechnical perspective.

❑ Review of the geotechnical aspects of the excavation contractor's shoring design, prior to construction, if applicable.

❑ Review of architectural plans pertaining to foundation and underfloor drainage systems and waterproofing details for elevator shafts.

For the foundation design data provided herein to be applicable, a material testing and observation services program is required to be completed. The following aspects be performed by Paterson:

❑ Review the bedrock stabilization and excavation requirements at the time of construction.

❑ Review and inspection of the installation of the foundation and underfloor drainage systems and elevator waterproofing.

❑ 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.

❑ 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 the construction has been conducted in general accordance with the recommendations could be issued, upon request, following the completion of a satisfactory materials testing and observation program by Paterson. All excess soil must be handled as per *Ontario Regulation 406/19: On-Site and Excess Soil Management.*

8.0 Statement of Limitations

The recommendations made in this report are in accordance with our present understanding of the project. We request that we be permitted to review the grading plan once available and our recommendations when the drawings and specifications are complete.

A geotechnical investigation of this nature is a limited sampling of a site. The recommendations are based on information gathered at the specific test locations and can only be extrapolated to an undefined limited area around the test locations. The extent of the limited area depends on the soil, bedrock and groundwater conditions, as well the history of the site reflecting natural, construction, and other activities. Should any conditions at the site be encountered which differ from those at the test locations, we request notification immediately in order to permit reassessment of our recommendations.

The recommendations provided in this report are intended for the use of design professionals associated with this project. Contractors bidding on or undertaking the work should examine the factual information contained in this report and the site conditions, satisfy themselves as to the adequacy of the information provided for construction purposes, supplement the factual information if required, and develop their own interpretation of the factual information based on both their and their subcontractor's construction methods, equipment capabilities and schedules.

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 1146 Snow Street Inc. or their agent(s) is not authorized without review by Paterson Group for the applicability of our recommendations to the altered use of the report.

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
October 31, 2024
J. R. VILLENEUVE
100504344
PROVINCE OF ONTARIO

Fabrice Venadiambu, P. Eng., ing.

Joey R. Villeneuve, M.A.Sc., P.Eng, ing.

Report Distribution:

- ❑ 1146 Snow Street Inc. (e-mail copy)
- ❑ Paterson Group Inc (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

1146 Snow Street, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 372640.75 **NORTHING:** 5032705.81 **ELEVATION:** 74.78

PROJECT: Proposed Residential Building

BORINGS BY: CME-55 Low Clearance Drill

REMARKS:

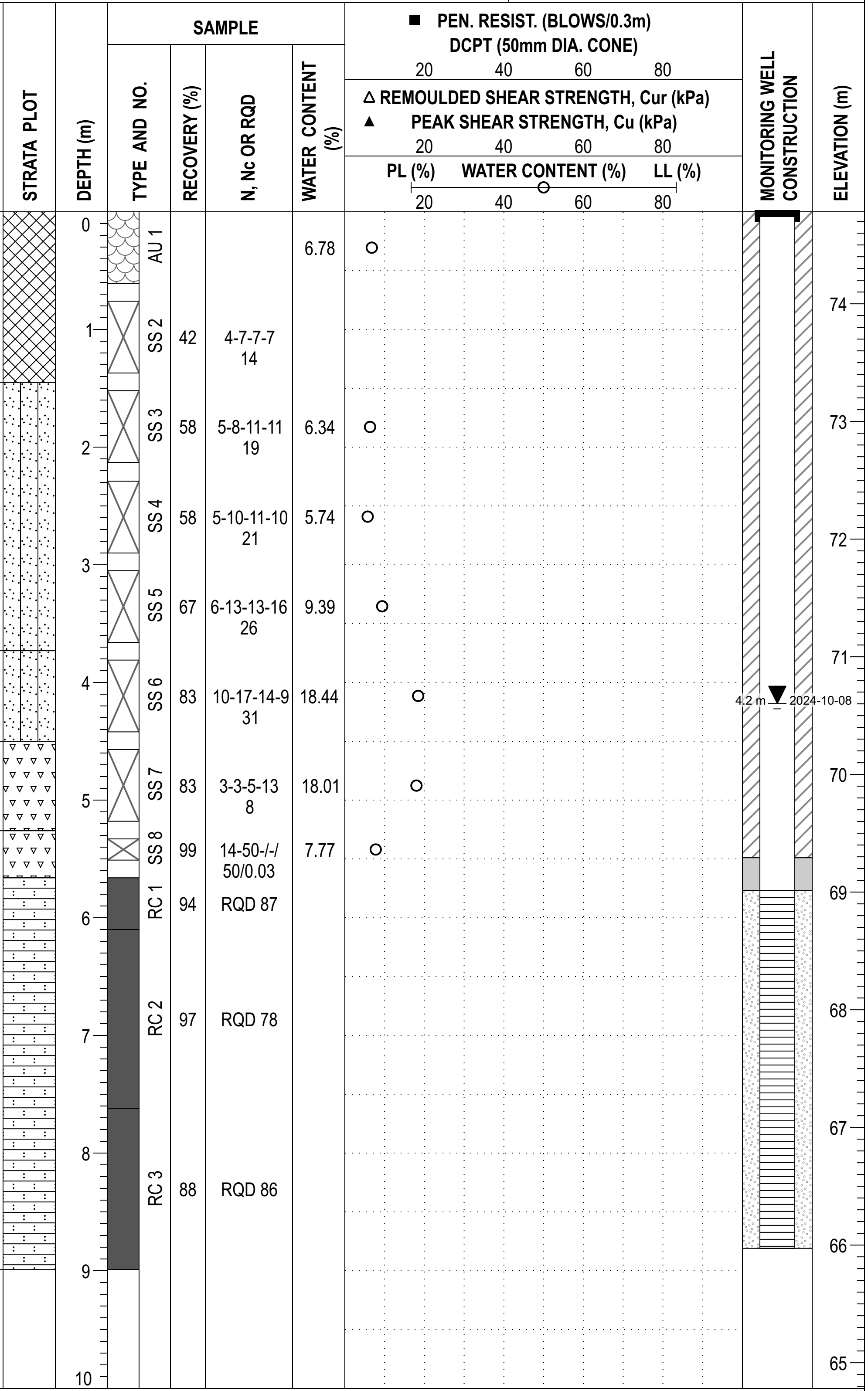
DATE: October 01, 2024

FILE NO. : PG7295

HOLE NO. : BH 1-24

SAMPLE DESCRIPTION	Depth [Elevation]
GROUND SURFACE	
FILL: Compact, brown silty fine sand, some gravel, trace brick and roots	1.45m [73.33m]
Compact, brown **SILTY** fine **SAND**	3.73m [71.05m]
Dense, brown **SILTY** fine **SAND** to **SANDY SILT** - Silt content increasing	4.50m [70.28m]
GLACIAL TILL: Loose, brown silty fine sand, trace to some gravel	5.26m [69.52m]
GLACIAL TILL: Dense, dark grey silty fine sand, trace to some gravel	5.66m [69.12m]
BEDROCK: Fair to good quality black shale	8.99m [65.79m]
End of Borehole (GWL at 4.18 m - October 8, 2024)	

TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)
AU 1			6.78
SS 2	42	4-7-7-7 14	
SS 3	58	5-8-11-11 19	6.34
SS 4	58	5-10-11-10 21	5.74
SS 5	67	6-13-13-16 26	9.39
SS 6	83	10-17-14-9 31	18.44
SS 7	83	3-3-5-13 8	18.01
SS 8	99	14-50-/-/ 50/0.03	7.77
RC 1	94	RQD 87	
RC 2	97	RQD 78	
RC 3	88	RQD 86	



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PATERSON GROUP

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

1146 Snow Street, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 372618.10 **NORTHING:** 5032702.82 **ELEVATION:** 74.08

PROJECT: Proposed Residential Building

BORINGS BY: CME-55 Low Clearance Drill

REMARKS:

DATE: October 01, 2024

FILE NO. : PG7295

HOLE NO. : BH 2-24

SAMPLE DESCRIPTION

GROUND SURFACE

FILL: Loose, brown silty fine sand, some gravel

0.91m [73.17m]

Loose to compact, brown **SILTY** fine **SAND**

- Compact by 1.53 m depth

- Loose by 2.29 m depth

3.35m [70.73m]

GLACIAL TILL: Dense, brown silty fine sand, some gravel

3.73m [70.34m]

GLACIAL TILL: Compact, grey medium sand, some gravel

3.96m [70.12m]

GLACIAL TILL: Compact, grey sandy silt, trace gravel and black shale

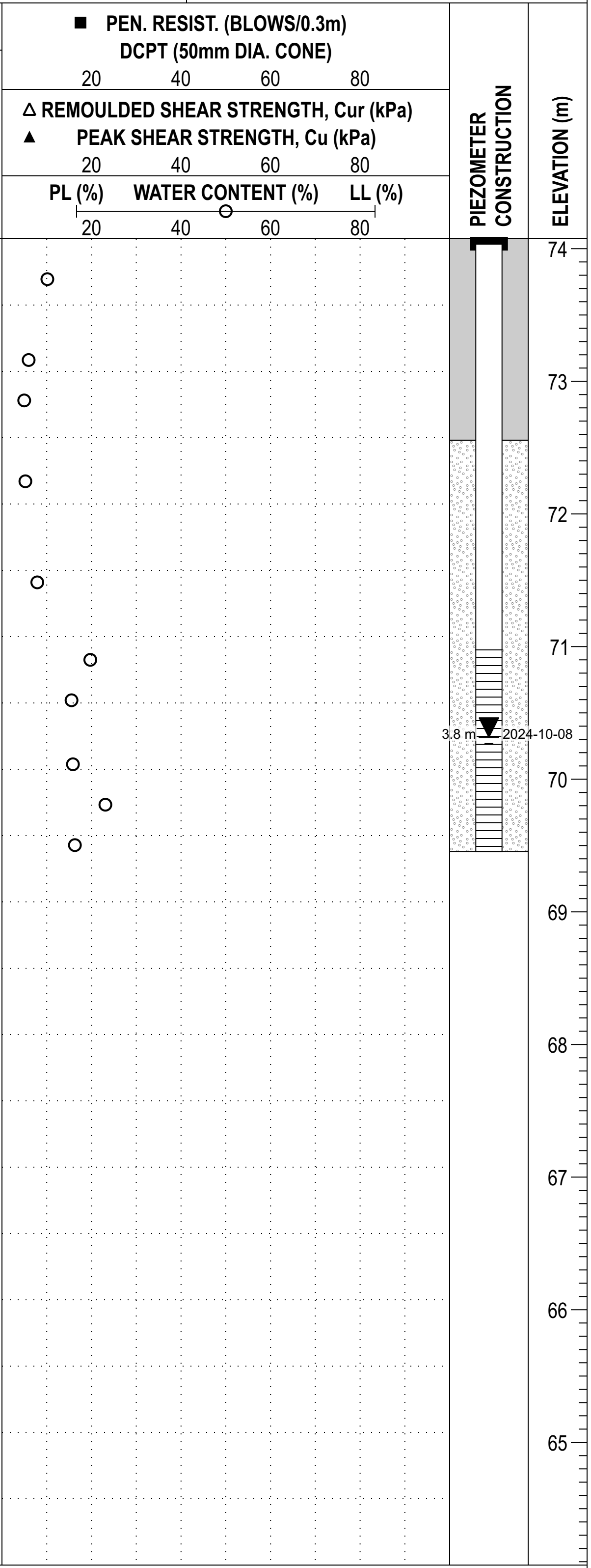
4.62m [69.45m]

End of Borehole

Practical refusal to augering at 4.62 m depth

(GWL at 3.76 m - October 8, 2024)

SAMPLE TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)
AU 1			10.14
SS 2	50	3-2-5-6 7	5.96 4.97
SS 3	67	4-7-6-7 13	5.23
SS 4	58	2-4-5-6 9	7.89
SS 5	83	5-10-21-14 31	19.73 15.54
SS 6	75	6-13-12-14 25	15.85 23.1
SS 7	100	50-/-/-/ 50/0.05	16.29



DEPTH (m): 0 1 2 3 4 5 6 7 8 9 10

STRATA PLOT

PATERSON GROUP

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

1146 Snow Street, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 372616.92 **NORTHING:** 5032702.26 **ELEVATION:** 74.06

PROJECT: Proposed Residential Building

BORINGS BY: CME-55 Low Clearance Drill

REMARKS:

DATE: October 01, 2024

FILE NO. : **PG7295**

HOLE NO. : **BH 2A-24**

■ PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE) 20 40 60 80

△ REMOULDED SHEAR STRENGTH, Cur (kPa)

▲ PEAK SHEAR STRENGTH, Cu (kPa) 20 40 60 80

PL (%) WATER CONTENT (%) LL (%) 20 40 60 80

SAMPLE DESCRIPTION GROUND SURFACE	STRATA PLOT	DEPTH (m)	SAMPLE				PIEZOMETER CONSTRUCTION	ELEVATION (m)
			TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)		
OVERBURDEN		0						74
4.50m [69.56m]								
BEDROCK: Good to excellent quality black shale			RC 1	97	RQD 90			
			RC 2	100	RQD 100			
7.65m [66.41m]								
End of Borehole		10						

PATERSON GROUP

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

1146 Snow Street, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 372626.82 **NORTHING:** 5032687.56 **ELEVATION:** 74.45

PROJECT: Proposed Residential Building

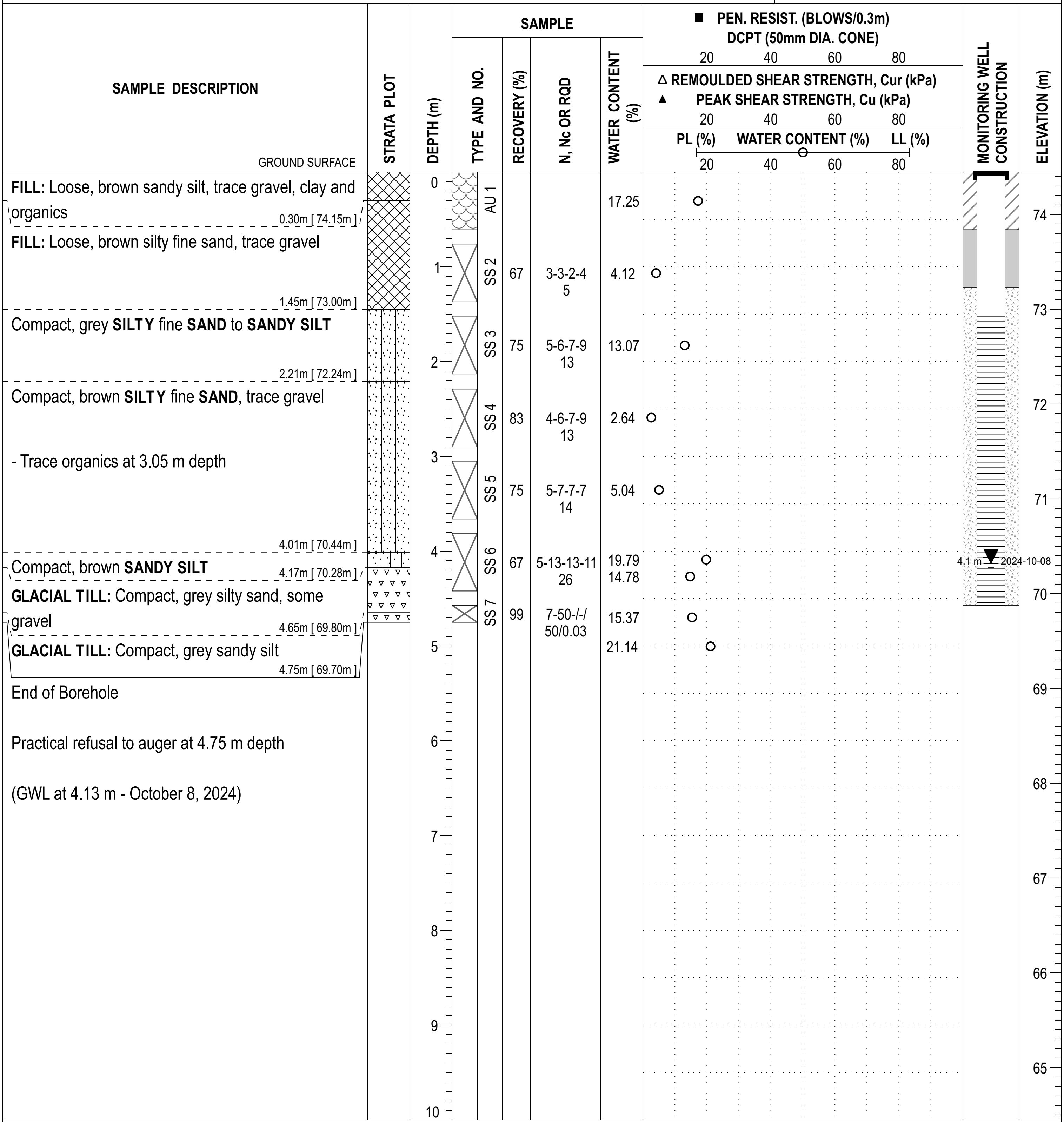
BORINGS BY: CME-55 Low Clearance Drill

REMARKS:

DATE: October 01, 2024

FILE NO. : PG7295

HOLE NO. : BH 3-24



SAMPLE DESCRIPTION

GROUND SURFACE

FILL: Loose, brown sandy silt, trace gravel, clay and organics 0.30m [74.15m]

FILL: Loose, brown silty fine sand, trace gravel 1.45m [73.00m]

Compact, grey **SILTY** fine **SAND** to **SANDY SILT** 2.21m [72.24m]

Compact, brown **SILTY** fine **SAND**, trace gravel

- Trace organics at 3.05 m depth 4.01m [70.44m]

Compact, brown **SANDY SILT** 4.17m [70.28m]

GLACIAL TILL: Compact, grey silty sand, some gravel 4.65m [69.80m]

GLACIAL TILL: Compact, grey sandy silt 4.75m [69.70m]

End of Borehole

Practical refusal to auger at 4.75 m depth

(GWL at 4.13 m - October 8, 2024)

SAMPLE TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)
AU 1			17.25
SS 2	67	3-3-2-4 5	4.12
SS 3	75	5-6-7-9 13	13.07
SS 4	83	4-6-7-9 13	2.64
SS 5	75	5-7-7-7 14	5.04
SS 6	67	5-13-13-11 26	19.79 14.78
SS 7	99	7-50-/-/ 50/0.03	15.37 21.14

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SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

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

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

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

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

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

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

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

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

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

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

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

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

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

CONSOLIDATION TEST

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

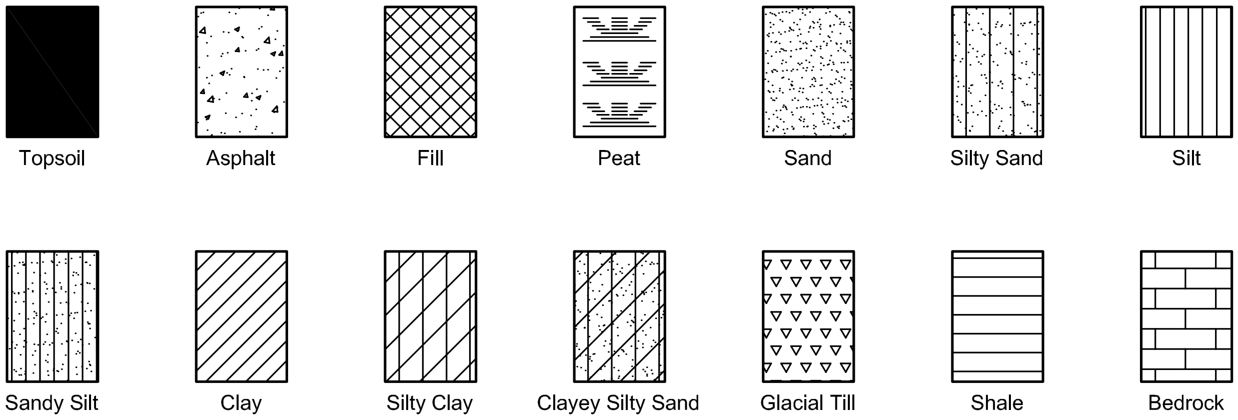
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

PERMEABILITY TEST

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

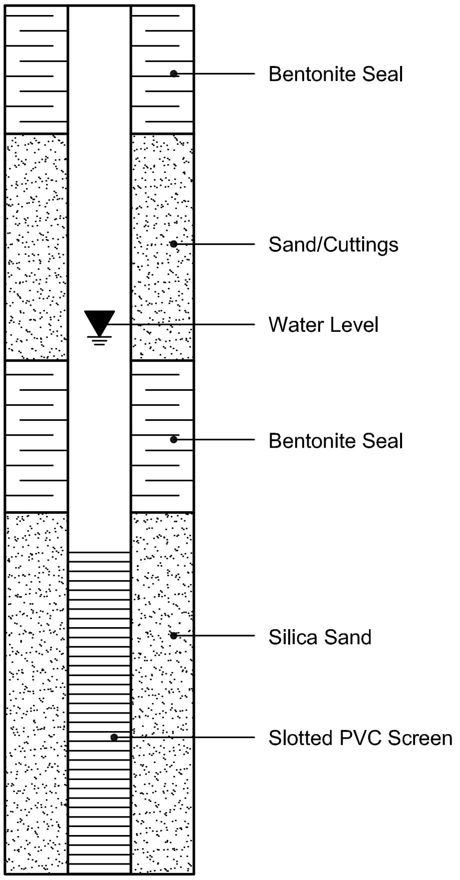
SYMBOLS AND TERMS (continued)

STRATA PLOT

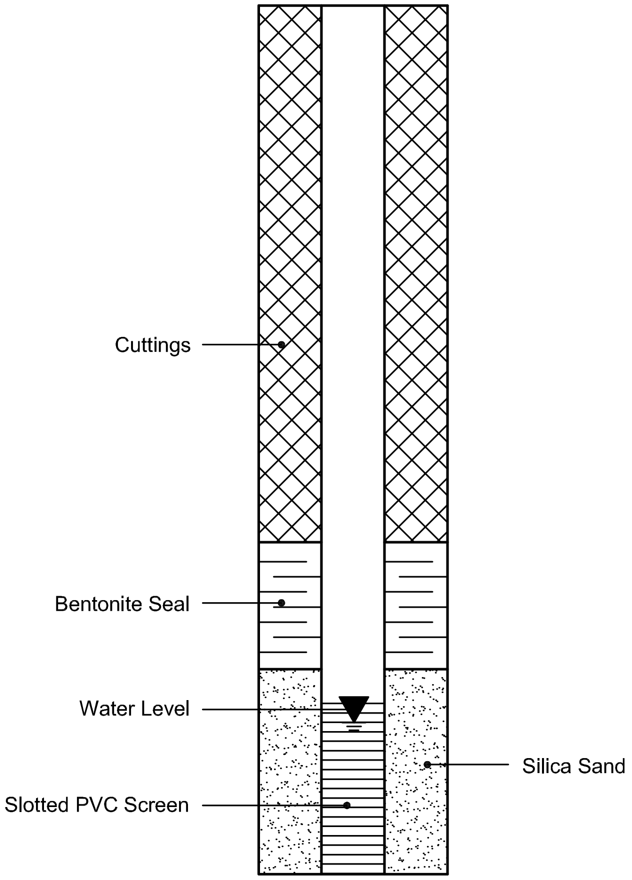


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Report Date: 07-Oct-2024

Client: **Paterson Group Consulting Engineers (Ottawa)**

Order Date: 2-Oct-2024

Client PO: 61448

Project Description: PG7295

	Client ID:	BH2-24-SS4	-	-	-		
	Sample Date:	01-Oct-24 09:00	-	-	-	**-**	**-**
	Sample ID:	2440369-01	-	-	-		
	Matrix:	Soil	-	-	-		
	MDL/Units						
Physical Characteristics							
% Solids	0.1 % by Wt.	95.9	-	-	-	-	-
General Inorganics							
pH	0.05 pH Units	6.98	-	-	-	-	-
Resistivity	0.1 Ohm.m	122	-	-	-	-	-
Anions							
Chloride	10 ug/g	<10	-	-	-	-	-
Sulphate	10 ug/g	<10	-	-	-	-	-

APPENDIX 2

FIGURE 1 – KEY PLAN

FIGURES 2 & 3 – SEISMIC SHEAR WAVE VELOCITY PROFILES

FIGURE 4 – WATERPROOFING SYSTEM FOR ELEVATOR

DRAWING PG7295-1 - TEST HOLE LOCATION PLAN

FIGURE 1

KEY PLAN

PATERSON
GROUP

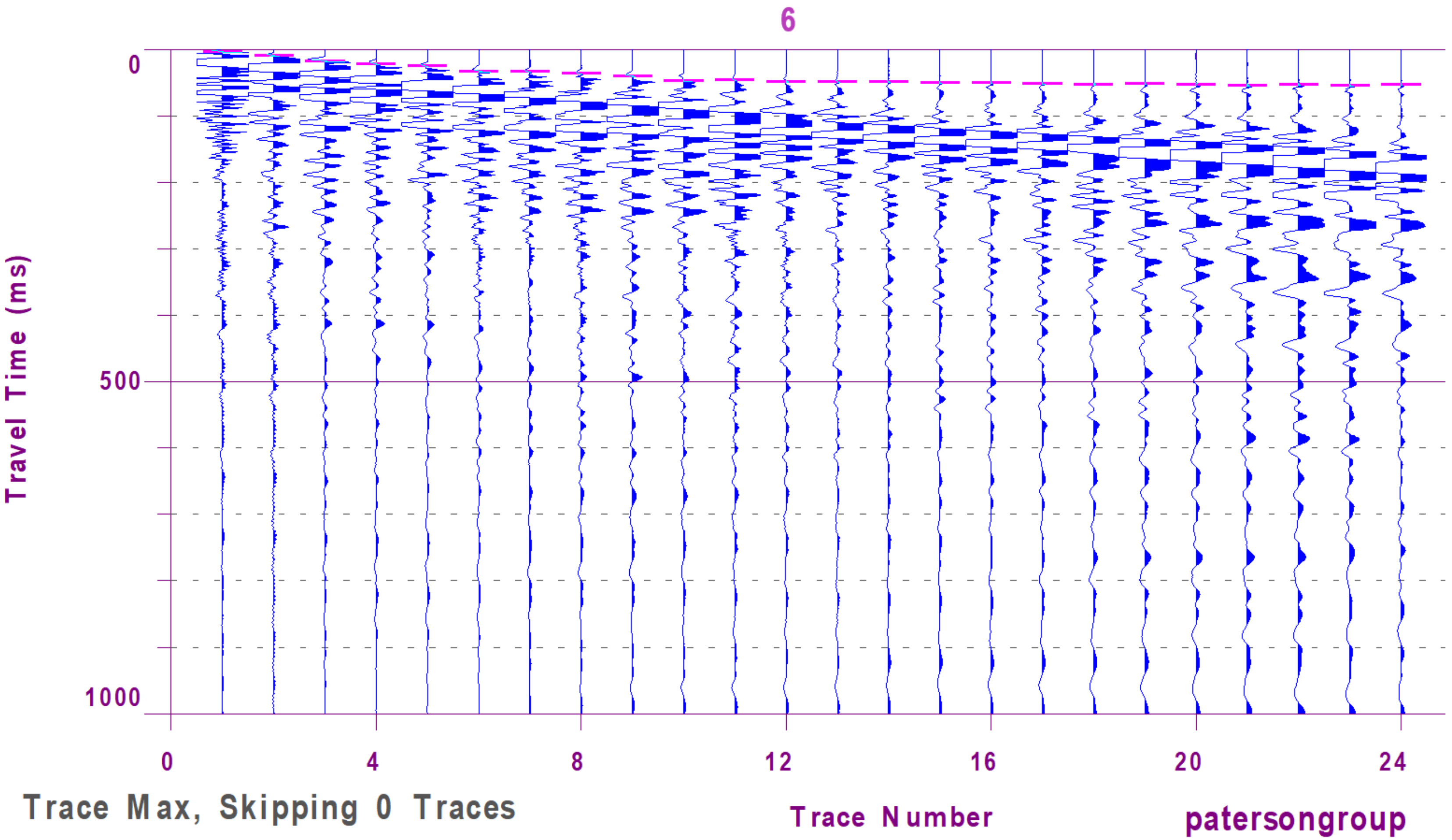


Figure 2 – Shear Wave Velocity Profile at Shot Location -1.5 m

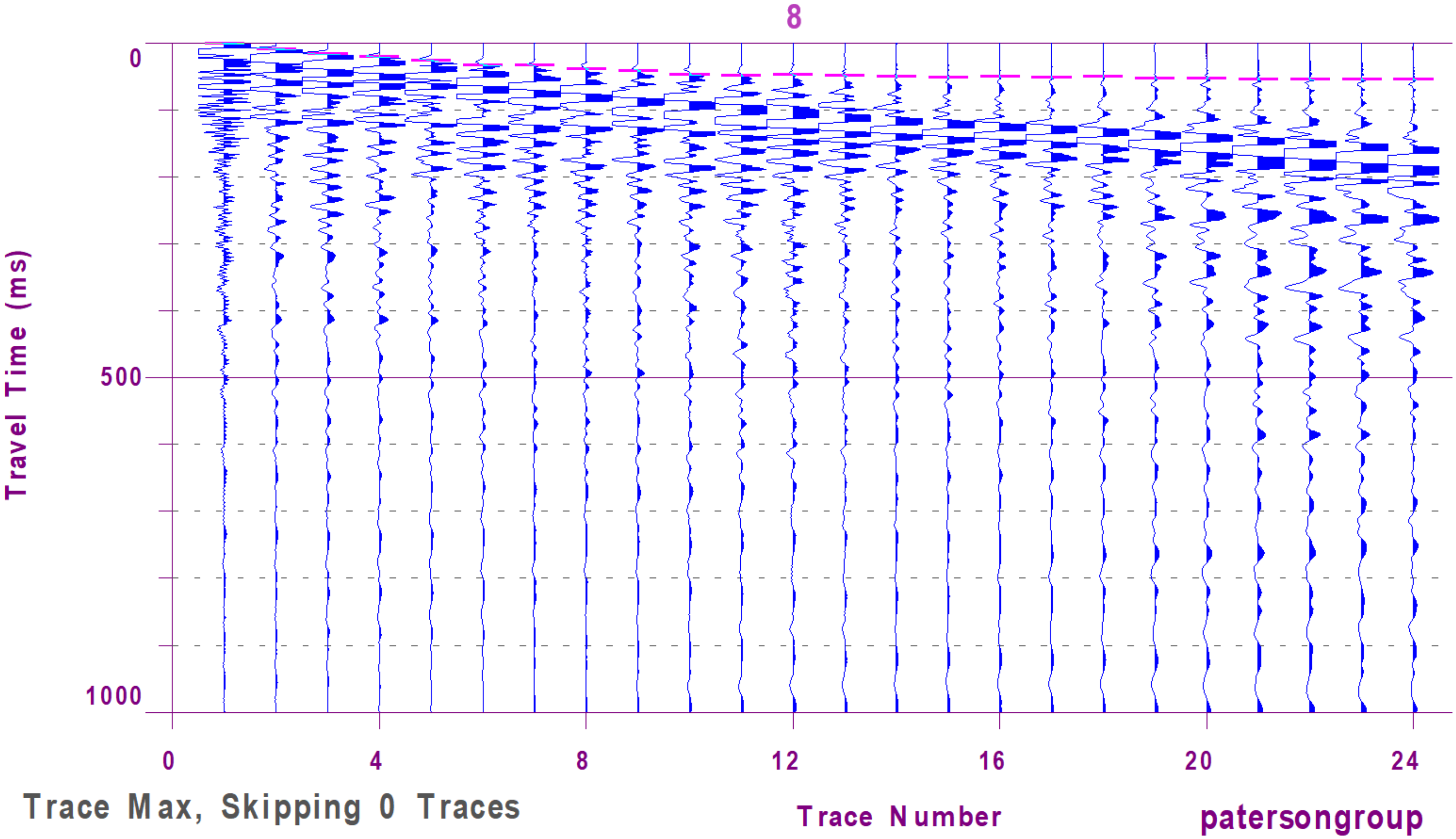
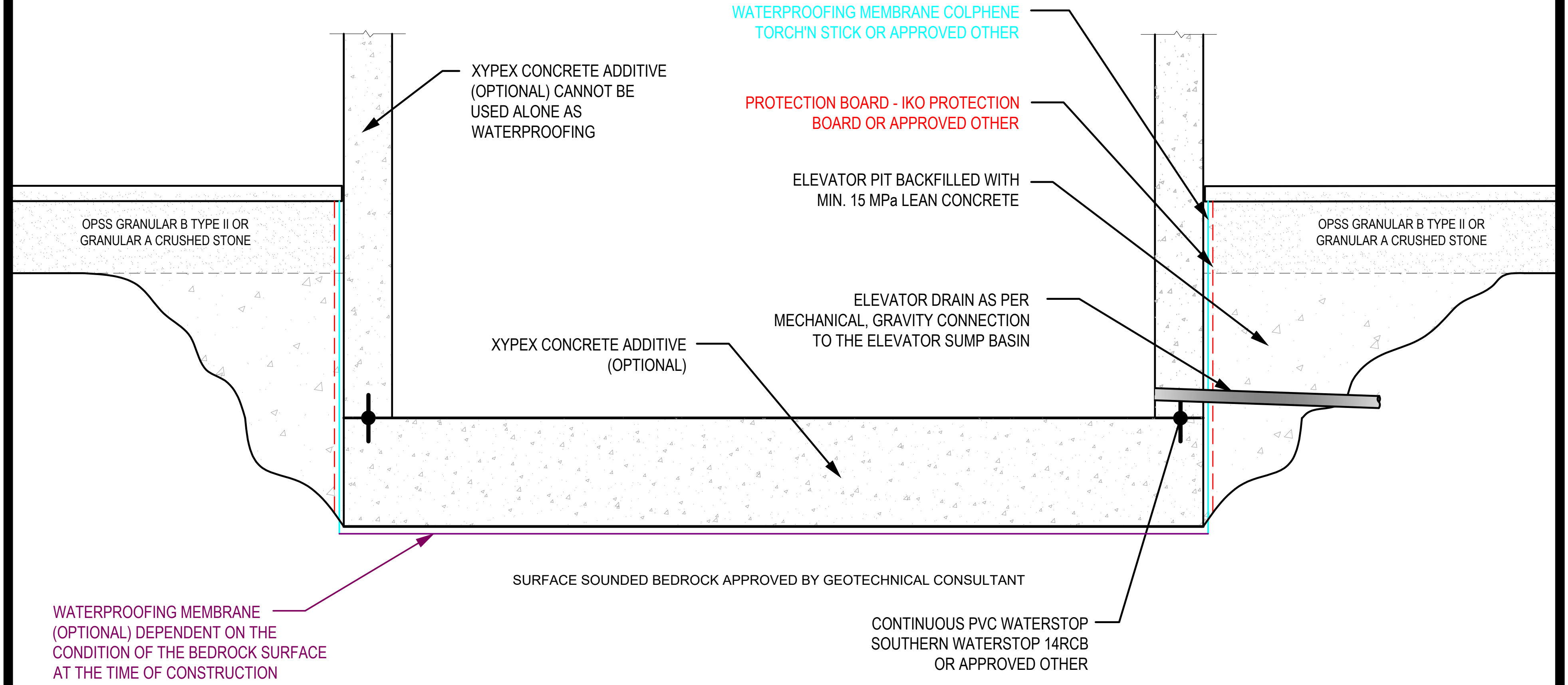


Figure 3 – Shear Wave Velocity Profile at Shot Location -1 m



NOTES:

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

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

1146 SNOW STREET INC.
PROPOSED RESIDENTIAL BUILDING
1146 SNOW STREET
OTTAWA, ONTARIO

Title:

WATERPROOFING SYSTEM FOR ELEVATOR

Scale: N.T.S.
Date: 10/2024
Drawn by: ZS
Report No.: PG7295-1
Checked by: FV
Drawing No.: FIGURE 4
Approved by: JV
Revision No.:

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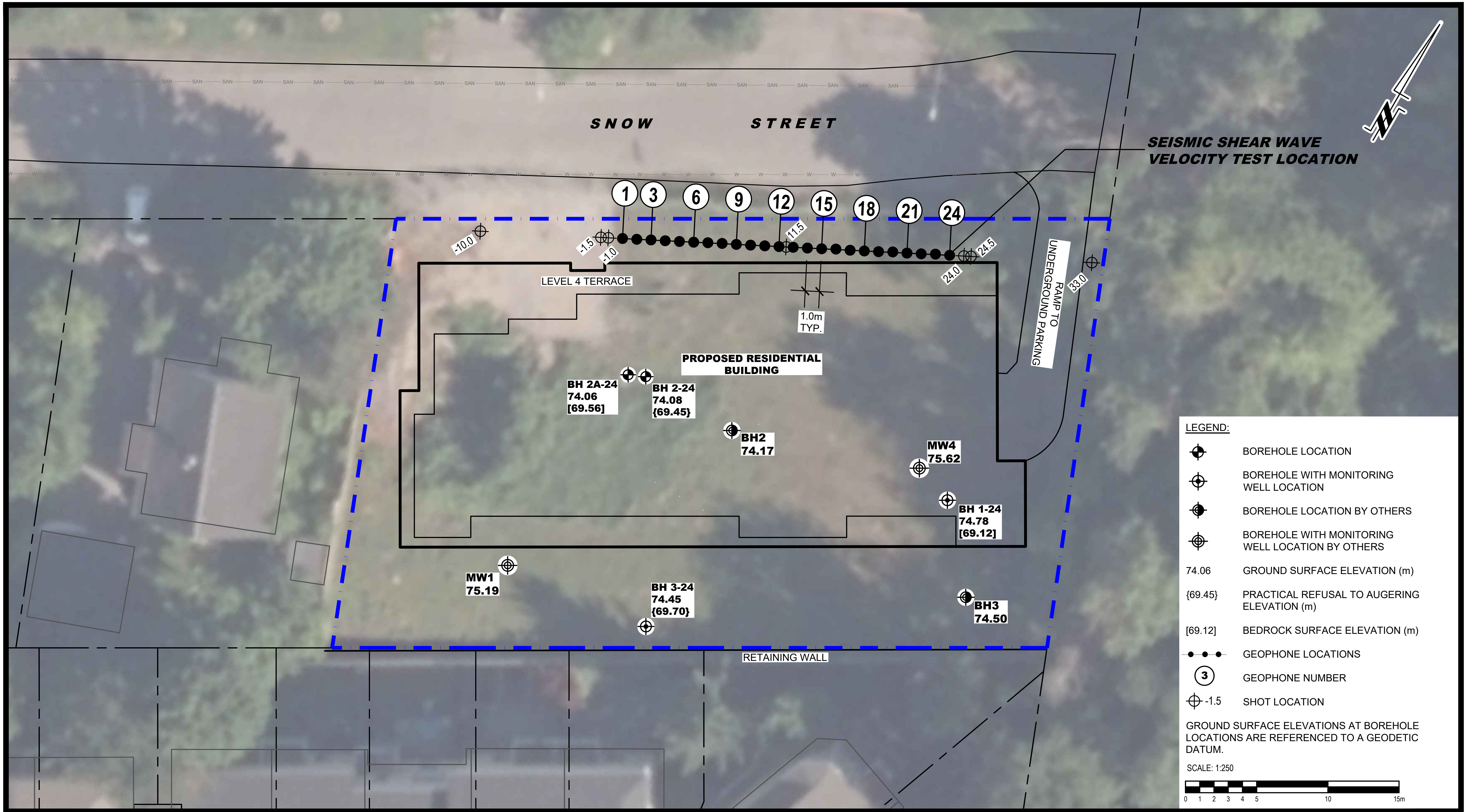

LEGEND:

⊕	BOREHOLE LOCATION
⊕	BOREHOLE WITH MONITORING WELL LOCATION
⊕	BOREHOLE LOCATION BY OTHERS
⊕	BOREHOLE WITH MONITORING WELL LOCATION BY OTHERS
74.06	GROUND SURFACE ELEVATION (m)
{69.45}	PRACTICAL REFUSAL TO AUGERING ELEVATION (m)
[69.12]	BEDROCK SURFACE ELEVATION (m)
● ● ●	GEOPHONE LOCATIONS
③	GEOPHONE NUMBER
⊕ -1.5	SHOT LOCATION

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

SCALE: 1:250

0 1 2 3 4 5 10 15m

PATERSON GROUP

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

NO.	REVISIONS	DATE	INITIAL

M. DAVID BLAKELY ARCHITECT INC.
GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL BUILDING
1146 SNOW STREET

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:250	Date: 10/2024
Drawn by: ZS	Report No.: PG7295-1
Checked by: FV	Dwg. No.: PG7295-1
Approved by: JV	Revision No.:

11x17

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