Geotechnical Investigation

Proposed Apartment Building Development

3080 Navan Road
Ottawa, Ontario

Prepared for Seymour Pacific Developments (Ontario) Ltd.

Report PG6527-1 Revision 2 dated May 17, 2023

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Seymour Pacific Developments (Ontario) Ltd. to conduct a geotechnical investigation for the proposed apartment building development to be located at 3080 Navan Road in the City of Ottawa (reference should be made 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 test holes.
- ☐ Provide geotechnical recommendations pertaining to design of the proposed development including construction considerations which may affect the design.

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

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

2.0 Proposed Development

Based on the available drawings, it is understood that the proposed development will consist of a six-storey residential apartment building development with up to one level of below grade parking. Associated access lanes, at-grade parking, landscaped and hardscaped areas are also anticipated as part of the development. The development is anticipated to be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current geotechnical investigation was carried out on December 19 and December 20, 2022, and consisted of advancing a total of 6 boreholes to a maximum depth of 10.5 m below existing ground surface. The test hole locations were distributed in a manner to provide general coverage of the subject site and taking into consideration underground utilities and site features. The borehole locations are shown on Drawing PG6527-1 - Test Hole Location Plan included in Appendix 2.

Previous investigations were undertaken by Paterson in July 2022 and March 2221 within the subject site and surrounding area. At that time, three boreholes and two test pits were advanced to a maximum depth of 6.1 and 1.6 m below ground surface, respectively, throughout the subject site. The test hole locations from the previous study are shown on Drawing PG6527-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were completed using a track mounted power auger drill rig operated by a two-person crew. All fieldwork was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer. The testing procedure consisted of augering and excavating to the required depth at the selected location and sampling the overburden.

Sampling and In Situ Testing

Soil samples were recovered from the auger flights or using a 50 mm diameter split-spoon sampler. The split-spoon and auger, samples were classified on site and placed in sealed plastic bags. All samples were transported to our laboratory for further examination. The depths at which the split-spoon and auger flights, samples were recovered from the boreholes are shown as SS and AU, respectively, on the Soil Profile and Test Data sheets in Appendix 1.

The Standard Penetration Test (SPT) was conducted in conjunction with the recovery of the split-spoon samples. The SPT results are recorded as “N” values on the Soil Profile and Test Data sheets. The “N” value is the number of blows required to drive the split-spoon sampler 300 mm into the soil after a 150 mm initial penetration using a 63.5 kg hammer falling from a height of 760 mm.

Undrained shear strength testing was carried out in cohesive soils using a field vane apparatus.

The overburden thickness was evaluated by completing dynamic cone penetration tests (DCPT) completed at boreholes BH 2-22, BH 4-22 and BH 7. The DCPT testing consisted of driving a steel drill rod, equipped with a 50 mm diameter cone at the tip, using a 63.5 kg hammer falling from a height of 760 mm. The number of blows required to drive the cone into the soil is recorded for each 300 mm increment.

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

Groundwater

Boreholes BH 2-22 and BH 4-22 were fitted with a 51 mm diameter PVC groundwater monitoring well. The other boreholes were fitted with flexible piezometers to allow for groundwater level monitoring. Sidewall infiltration observed at the time of completing the test pits were recorded at the time of the field program and prior to backfilling the test pit. The groundwater observations are discussed in Section 4.3 and are presented on the Soil Profile and Test Data sheets in Appendix 1.

Sample Storage

All samples will be stored in the laboratory for a period of one (1) month after issuance of this report. They will then be discarded unless we are otherwise directed.

3.2 Field Survey

The test hole locations and ground surface elevation at each test hole location were surveyed by Paterson using a handheld GPS and referenced to a geodetic datum. The locations of the test holes, and the ground surface elevation at each test hole location, are presented on Drawing PG6527 - 1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

Soil samples were recovered from the subject site and visually examined in our laboratory to review the results of the field logging. A total of one linear shrinkage analyses and two Atterberg limit tests were completed on selected soil samples. Moisture content testing was complete on all recovered soil samples. The results of the testing are presented in Section 4.2 and are provided in Appendix 1.

Sample Storage

All samples will be stored in the laboratory for a period of one (1) month after issuance of this report. They will then be discarded unless directed otherwise.

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 samples. 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 currently consists of a temporary construction site laydown area containing several construction trailers and equipment storage areas. Based on our review of historical aerial images, the subject site was previously occupied by a residential dwelling, a commercial garage structure and associated earth moving works. The subject site is approximately between 0.6 to 1 m lower than Navan Road and at grade with Page Road.

The site is bordered by Navan Road to the north, Page Road to the east, Renaud Road and residential properties to the south, and a residential subdivision to the west. The ground surface across the subject site slopes gradually downward from north to south between a geodetic elevation of 82 m to 80 m.

4.2 Subsurface Profile

Generally, the subsurface profile at the test hole locations consists of a layer of fill extending to depths ranging from 1.3 to 2.5 m below the existing ground surface. The fill was generally observed to consist of a silty sand with gravel, crushed stone, clay, and organics.

The fill layer was observed to be underlain by a deposit of silty clay. Very stiff to firm, brown silty clay was encountered below the fill and extended to approximate depths between 3.0 to 3.7 m below the ground surface. The brown silty clay layer was observed to be underlain by a layer soft to stiff, grey silty clay.

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

Atterberg Limits Testing

Atterberg limits testing was completed on silty clay samples recovered from BH 2-22 and BH 5-22. The result of the Atterberg limits tests is presented in Table 1 and on the Atterberg Limits Testing Results sheet in Appendix 1.

Table 1 - Atterberg Limits Results

Sample	Depth (m)	LL (%)	PL (%)	PI (%)	w (%)	Classification
BH 2-22 SS4	2.2 – 2.8	78	27	51	63.4	CH
BH 5-22 SS3	1.5 – 2.1	76	25	51	42.8	CH

Notes: LL: Liquid Limit; PL: Plastic Limit; PI: Plasticity Index; w: water content;
CH: Inorganic Clays of High Plasticity

Shrinkage Testing

Linear shrinkage testing was completed on a sample recovered at a depth of 1.8 m from borehole BH 4-22 and yielded a shrinkage limit of 24.3 and a shrinkage ratio of 1.69.

Bedrock

A DCPT was complete at BH 2-22, BH 4-22 and BH 7 with practical refusal encountered at a depth of 29.7, 28.3 and 27.1 m below the ground surface, respectively. Based on available geological mapping, the bedrock in the subject area consists of Shale of the Billings Formation, with an overburden drift thickness ranging between 25 to 50 m depth.

4.3 Groundwater

Groundwater levels were recorded at each borehole location instrumented with a monitoring device. The groundwater level readings are presented in the Soil Profile and Test Data sheets in Appendix 1. The measured groundwater levels by Paterson are presented in Table 2 below.

It should be noted that surface water can become trapped within a backfilled borehole that can lead to higher than typical groundwater level observations. Long-term groundwater levels can also be estimated based on the observed color and consistency of the recovered soil samples.

Based on these observations, the long-term groundwater table can be expected to be at a depth of approximately **2 to 3 m** throughout the subject site. It should be noted that groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater levels could vary at the time of construction.

The recorded groundwater levels are noted on the applicable Soil Profile and Test Data sheet presented in Appendix 1.

Table 2 – Summary of Groundwater Levels

Test Hole Number	Ground Surface Elevation (m)	Measured Groundwater Level		Dated Recorded
		Depth (m)	Elevation (m)	
BH 1-22	80.35	5.71	74.64	January 3, 2023
BH 2-22	80.80	6.52	74.28	
BH 3-22	79.94	2.97	76.97	
BH 4-22	80.53	10.86	69.67	
BH 5-22	80.66	5.86	74.80	
BH 6-22	80.52	9.15	71.37	
BH 1	80.19	0.76	79.43	June 10, 2020
BH 7	80.29	1.08	79.21	
TP 1	80.15	1.00	79.15	May 28, 2020
TP 2	80.23	1.40	78.83	

Note: The ground surface elevation at each borehole location was surveyed using a handheld GPS and are referenced to a geodetic datum.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered suitable for the proposed development. The proposed apartment building is expected to be founded on a raft foundation placed on an undisturbed firm to stiff silty clay bearing surface or a deep foundation, such as end-bearing piles, extending to the bedrock surface.

Due to the presence of a silty clay layer, proposed grading throughout the subject site will be subjected to a permissible grade restriction. Our permissible grade raise recommendations are discussed in Subsection 5.3.

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. Care should be taken not to disturb adequate bearing soils below the founding level during site preparation activities. Disturbance of the subgrade may result in having to sub-excavate the disturbed material and the placement of additional suitable fill material.

The existing fill material, where free of significant organic materials, should be reviewed by Paterson personnel at the time of construction if consideration is given to leaving it in place below future hardscaping. Paterson personnel may request that the existing fill layers be proof-rolled by a suitably sized smooth-drum (stone and sand fill) or sheepsfoot roller (clay fill) at the time of review.

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

Fill Placement

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

Non-specified existing fill along with site-excavated soil could be placed as general landscaping fill and beneath exterior parking areas where settlement of the ground surface is of minor concern.

These materials should be spread in maximum 300 mm thick loose lifts and compacted by the tracks of the spreading equipment to minimize voids. If this material is to be used to build up the subgrade level for areas to be paved, it should be compacted in maximum 300 mm thick lifts to at least 95% of the material's SPMDD. The placement of subgrade material should be reviewed at the time of placement by Paterson personnel.

Non-specified existing fill and site-excavated soils are not suitable for placement as backfill against foundation walls, unless used in conjunction with a geocomposite drainage membrane, such as Miradrain G100N or Delta Drain 6000.

Fill used for grading beneath the base and subbase layers of paved areas should consist, unless otherwise specified, of clean imported granular fill, such as OPSS Granular A, Granular B Type II or select subgrade material. 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 paved areas should be compacted to at least 100% of its SPMDD.

Protection of Subgrade (Raft Foundation)

Since the subgrade material for the buildings foundation is expected to consist of firm silty clay, it is recommended that a minimum 75 mm thick lean concrete mud slab be placed on the undisturbed silty clay subgrade shortly after the completion of the excavation. The main purpose of the mud slab is to reduce the risk of disturbance of the subgrade under the traffic or workers and equipment.

The final excavation to the raft bearing surface level and the placing of the mud slab should be completed in smaller sections to avoid exposing large areas of the silty clay to potential disturbances due to drying.

Compacted Granular Fill Working Platform (Piled Foundation)

Should the proposed structure be supported on a pile foundation, the use of heavy equipment would be required to install the piles (i.e., pile driving crane). It is conventional practice to install a compacted granular fill layer, at a convenient elevation, to allow the equipment to access the site without getting stuck and causing significant disturbance to the underlying soil.

A typical working platform could consist of 600 mm of OPSS Granular B, Type II crushed stone placed and compacted to a minimum of 98% of its standard Proctor maximum dry density (SPMDD) in maximum 300 mm thick lifts.

Once the piles have been driven and cut off, the working platform can be re-graded, and soil tracked in, or soil pumping up from the pile installation locations, can be bladed off and the surface can be topped up, if necessary, and re-compacted to act as the substrate for further fill placement for basement slab structure.

5.3 Foundation Design

Raft Foundation (Mixed-Used Building Structure)

Based on the expected loads from the proposed structure, a raft foundation bearing on the undisturbed firm, grey silty clay bearing surface may be considered for foundation support for the proposed building. For design purposes, it was assumed that the base of the raft foundation would be located at an approximate depth of 3 to 4 m since it would be provided with one level of underground parking.

The amount of settlement of the raft slab will be dependent on the sustained raft contact pressure. The loading conditions for the contact pressure are based on sustained loads, that are generally taken to be 100% Dead Load and 50% Live Load.

For the raft slab foundation, a bearing resistance value at SLS (contact pressure) of **80 kPa** will be considered acceptable for a raft supported on the undisturbed, firm silty clay. The factored bearing resistance (contact pressure) at ULS can be taken as **120 kPa**. For this case, the modulus of subgrade reaction was calculated to be **3.2 MPa/m** for a contact pressure of **80 kPa**.

The raft foundation design is required to consider the relative stiffness of the reinforced concrete slab and the supporting bearing medium. A geotechnical resistance factor of 0.5 was applied to the bearing resistance values at ULS.

Based on the following assumptions for the raft foundation, the high-rise portion of the proposed structure can be designed using the above parameters with a total and differential settlement of 25 and 15 mm, respectively.

End Bearing Driven Piled Foundation

A deep foundation method, such as end bearing piles, may be considered for the foundation support of the proposed building should its loading exceed the load bearing capacity provided for a raft slab foundation. For deep foundations, concrete-filled steel pipe piles are generally utilized in the Ottawa area. Applicable pile resistance at SLS values and factored pile resistance at ULS values are given in Table 3. A resistance factor of 0.4 has been incorporated into the factored ULS values. Note that these are all geotechnical axial resistance values.

The geotechnical pile resistance values were estimated using the Hiley dynamic formula, to be confirmed during pile installation with a program of dynamic monitoring. For this project, the dynamic monitoring of two (2) to four (4) piles would be recommended. This is considered to be the minimum monitoring program, as the piles under shear walls may be required to be driven using the maximum recommended driving energy to achieve the greatest factored resistance at ULS values. Re-striking of all piles at least once will also be required after at least 48 hours have elapsed since initial driving.

Table 3 - Pile Foundation Design Data

Pile Outside Diameter (mm)	Pile Wall Thickness (mm)	Geotechnical Axial Resistance		Final Set (blows/ 12 mm)	Transferred Hammer Energy (kJ)
		SLS (kN)	Factored at ULS (kN)		
245	9	925	1100	9	27
245	11	1050	1250	9	31
245	13	1200	1400	9	35

Re-striking of all piles, at least once, will also be required after at least 48 hours have elapsed since initial driving. A full-time field review program should be conducted during the pile driving operations to record the pile lengths, ensure that the refusal criteria is met and that piles are driven within the location tolerances (within 75 mm of proper location and within 2% of vertical).

The minimum recommended centre-to-centre pile spacing is 3 times the pile diameter. The closer the piles are spaced, however, the more potential that the driving of subsequent piles in a group could have influence on piles in the group that have already been driven. These effects, primarily consisting of uplift of previously driven piles, are checked as part of the field review of the pile driving operations. Prior to the commencement of production pile driving, a limited number of indicator piles should be installed across the site. It is recommended that each indicator pile be dynamically load tested to evaluate pile stresses, hammer efficiency, pile load transfer, and end-of-driving criteria for end-bearing in the bedrock.

Conventional Shallow Foundations (Auxiliary Structures)

The following conventional spread footing bearing resistance values may be considered for portions of the underground parking garage structure located beyond the building footprint and other lightly loaded ancillary structures.

Conventional spread footings placed over an undisturbed, compact brown silty sand bearing surface can be designed using bearing resistance value at serviceability limit states (SLS) of **120 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **180 kPa**.

Strip footings, up to 3 m wide, and pad footings, up to 5 m wide, placed over an undisturbed, hard to stiff brown silty clay bearing surface can be designed using bearing resistance value at serviceability limit states (SLS) of **120 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **180 kPa**.

Strip footings, up to 2 m wide, and pad footings, up to 4 m wide, placed over an undisturbed, firm grey silty clay bearing surface can be designed using bearing resistance value at serviceability limit states (SLS) of **60 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **90 kPa.** A geotechnical resistance factor of 0.5 was applied to the reported bearing resistance values at ULS.

An undisturbed soil bearing surface consists of one from which all topsoil and deleterious materials, such as loose, frozen or disturbed soil, have been removed prior to the placement of concrete for footings.

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

Proof Rolling and Subgrade Improvement for Loose Sand Below Footings

Where the sand bearing surface for footings is considered loose by Paterson at the time of construction, it may be recommended to proof roll the bearing surface prior to forming for footings. Improving the bearing surface compaction will provide a suitable sand bearing medium.

Depending on the looseness and degree of saturation at the time of construction, other measures (additional compaction, dewatering, mud-slab, sub-excavation and reinstatement of crushed stone fill) may be recommended to accommodate site conditions at the time of construction. However, these considerations would be evaluated at the time of construction by Paterson on a footing-specific basis.

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 the in-situ bearing medium soils above the groundwater table when a plane extending down and out from the bottom edges 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 that of the bearing medium.

Permissible Grade Raise Recommendations

Our current permissible grade raise recommendations for the proposed development are presented on Drawing PG6527-2 Permissible Grade Raise Plan in Appendix 2.

A post-development groundwater lowering of 0.5 m was assumed for our calculations. If higher than permissible grade raises are required, preloading with or without a surcharge, lightweight fill, and/or other measures should be investigated to reduce the risks of unacceptable long-term post construction total and differential settlements.

Provided sufficient time is available to induce the required settlements, consideration could be given to surcharging the subject site. Alternatively, consideration could also be given to undertaking a test fill pile program to assess the suitability to raise the currently recommended permissible grade raise recommendations in conjunction with a supplemental investigation.

5.4 Design for Earthquakes

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 shear wave velocity testing was completed by Paterson personnel. The results of the shear wave velocity test are provided on Figures 2 and 3, which are presented in Appendix 2 of this report.

Field Program

The seismic array testing location was placed as presented in Drawing PG6527-1 - Test Hole Location Plan, attached to the present report. Paterson field personnel placed 18 horizontal 2.4 Hz. geophones mounted to the surface by means of two 75 mm ground spikes attached to the geophone land case. The geophones were spaced at 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 were also completed in a forward and reverse direction (i.e.- striking both sides of the I-Beam seated parallel to the geophone array). The shot locations were 30 and 11.5 m away from the first geophone, 3.0, 4.5, 11.5 and 26.5 m from the last geophone, and at the center of the seismic array.

Data Processing and Interpretation

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

The interpretation is repeated at each shot location to provide an average shear wave velocity, V_{s30}, of the upper 30 m profile, immediately below the 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 **166 m/s**, while the bedrock shear wave velocity is **2,184 m/s**. Based on our interpretation, and assuming the proposed development will be founded on a raft or piled foundation, the overburden thickness below the foundation is assumed to be 27 m.

The V_{s30} was calculated using the standard equation for average shear wave velocity provided in the OBC 2012 and as presented in the following page:

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

$$V_{s30=}\frac{30\ m}{\left(\frac{27\ m}{166\ m/s} + \frac{3\ m}{2{,}184\ m/s}\right)}$$

$$V_{s30=}182\ \ m/s$$

Based on the results of the shear wave velocity testing, the average shear wave velocity V_{s30}, is **182 m/s**. Therefore, a **Site Class D** is applicable for the design of proposed building founded on a raft or piled foundation, 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

With the removal of all topsoil and deleterious materials within the footprint of the proposed building, a soil subgrade approved by Paterson personnel at the time of construction, is considered to be an acceptable subgrade surface on which to commence backfilling for the floor slab construction.

The recommended pavement structures noted in Subsection 5.7 will be applicable where the basement level underlying foundation support consists of piled foundations or conventional spread footings. The basement slab would be considered as a slab-on-grade where foundation support consists of piled foundations or conventional spread footings.

For buildings of slab-on-grade construction, it is recommended that the upper 300 mm of sub-slab fill consist of OPSS Granular A crushed stone. If storage or other uses of the lower level will involve the construction of a concrete floor slab, the upper 200 mm of sub-slab fill is recommended to consist of 19 mm of clear crushed stone.

Where a raft slab is utilized, a granular layer of OPSS Granular A will be required to allow for the installation of sub-floor services above the raft slab foundation. The thickness of the OPSS Granular A crushed stone will be dependent on the piping requirements.

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. 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.

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 lowest 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.

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. The applicable effective unit weight of the retained soil can be estimated as 13 kN/m^3, where applicable. A hydrostatic pressure should be added to the total static earth pressure when calculating the effective unit weight.

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

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 0.32 g according to the 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 the OBC 2012.

5.7 Pavement Design

Podium Deck Area

It is anticipated that the podium deck structure will be provided car only parking areas, access lanes, fire truck lanes and loading areas. Based on the concrete slab subgrade, the pavement structure indicated in the following tables may be considered for design purposes:

Table 4 - Recommended Pavement Structure - Car-Only Parking Areas (Podium Deck)

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

SUBGRADE – Reinforced Concrete Podium Deck
*If specified by others, not required from a geotechnical perspective
**Thickness is dependent on grade of insulation as noted in paragraphs below.

Table 5 - Recommended Pavement Structure – Access Lane, Fire Truck Lane, Ramp and Heavy Truck Parking Areas (Podium Deck)

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

SUBGRADE – Reinforced Concrete Podium Deck
*If specified by others, not required from a geotechnical perspective
**Thickness is dependent on grade of insulation as noted in paragraphs below.

The transition between the pavement structure over the podium deck subgrade and soil subgrade beyond the footprint of the podium deck is recommended to be transitioned to match the pavement structures provided in the following section. For this transition, a 5H:1V is recommended between the two subgrade surfaces. Further, the base layer thickness should be increased to a minimum thickness of 600 mm below the top of the podium slab a minimum of 1.5 m from the face of the foundation wall prior to providing the recommended taper.

Should the proposed podium deck be specified by others to be provided a thermal break by the use of a layer of rigid insulation below the pavement structure, its placement within the pavement structure is recommended to be as per the above-noted tables. The layer of rigid insulation is recommended to consist of a DOW Chemical High-Load 100 (HI-100), High-Load 60 (HI-60) or High Load (HI-40) extruded polystyrene. The pavement structures base layer thickness in Table 4 and Table 5 may be reduced by 25 mm if HI-100 is considered for this project. It should be noted that Styrofoam rigid insulation is not considered suitable for this application.

Pavement Structure Over Overburden

Beyond the podium deck, the following pavement structures may be considered for car only parking and heavy traffic areas. The subgrade material will consist of silty clay throughout the exterior and lowest basement level of the subject site, respectively. The proposed pavement structures are shown in Tables 6 and 7.

Table 6 - Recommended Pavement Structure - Car-Only Parking Areas

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

Table 7 - Recommended Pavement Structure - Heavy-Truck Traffic and Loading 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 in situ soil, fill or OPSS Granular B Type I or II material placed over in situ soil.	

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for this project. If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type II material.

The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 100% of the material's SPMDD using suitable compaction equipment.

Pavement Structure Drainage

Satisfactory performance of the pavement structure is largely dependent on 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 load carrying capacity.

Where silty clay is anticipated at subgrade level, consideration should be given to installing subdrains during the pavement construction as per City of Ottawa standards. The subdrain inverts should be approximately 300 mm below subgrade level, and the subgrade surface should be crowned to promote water flow to drainage lines.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

A perimeter foundation drainage system is recommended to be provided for the proposed building. The system should consist of a 150 mm diameter perforated corrugated plastic pipe, surrounded on all sides by 150 mm of 19 mm clear crushed stone, 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 or sump pit provided in the lowest basement level of the structure.

Waterproofing layers for podium deck 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 and as depicted in Figure 4 – Podium Deck to Foundation Wall Drainage System Tie-In Detail. Elevator shafts located below the underslab drainage system should be waterproofed and provided with a PVC waterstop at the shaft wall and footing interface.

Review of architectural design drawings should be completed by Paterson for the above-noted items once the building design has been finalized and prior to tender. It is recommended that Paterson reviews all details associated with the foundation drainage system prior to tender.

Foundation Backfill

Backfill against the exterior sides of the foundation walls should consist of free draining non frost susceptible granular materials.

The greater part of the site excavated materials will be 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 II granular material, should otherwise be used for this purpose.

Foundation backfill material should be compacted in maximum 300 mm thick loose lifts and with suitably sized vibratory compaction equipment (smooth-drum roller for crushed stone fill, sheepsfoot roller for soil fill).

Interior Perimeter and Underfloor Drainage

An interior perimeter and underfloor drainage system will be required to redirect water from the buildings foundation drainage system to the buildings sump pit(s) if it will not discharge to an exterior catch basin structure. 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.

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.

Foundation Raft Slab Construction Joints

It is anticipated the raft slab will be poured in several pour segments. For the construction joint at each pour, a rubber water stop along with a chemical grout (Xypex or equivalent) should be applied to the entire vertical joint of the slab.

6.2 Protection of Footings Against Frost Action

Perimeter footings of heated structures are required to be insulated against the deleterious effects of frost action. A minimum 1.5 m thick soil cover alone, or a combination of soil cover in conjunction with foundation insulation should be provided in this regard.

The parking garage should not require protection against frost action due to the founding depth. Unheated structures, such as the access ramp wall footings, may be required to be insulated against the deleterious effect of frost action. A minimum of 2.1 m of soil cover alone, or a minimum of 600 mm m of soil cover, in conjunction with foundation insulation and as reviewed and advised by Paterson, should be provided.

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.

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 subsurface soil is considered to be mainly a Type 2 and Type 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 maintain safe working distance from the excavation sides.

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.

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

Temporary Shoring

Temporary shoring may be required for the overburden soil 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 interlocking steel sheet piling. 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 parameters provided in Table 7.

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

Parameter	Value
Active Earth Pressure Coefficient (K_a)	0.33
Passive Earth Pressure Coefficient (K_p)	3
At-Rest Earth Pressure Coefficient (K_o)	0.5
Unit Weight (γ), kN/m^3	20
Submerged Unit Weight (γ'), kN/m^3	13

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

The pipe bedding for the sewer and water pipes should consist of at least 150 mm of OPSS Granular A. The bedding layer thickness should be increased to a minimum of 300 mm where the subgrade will consist of grey silty clay. 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, such as the grey silty clay, will be difficult to re-use, as the high-water contents make compacting impractical without an extensive drying period.

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

To reduce long-term lowering of the groundwater level at this site, clay seals should be provided in the service trenches. The seals should be at least 1.5 m long and should extend from trench wall to trench wall. Generally, the seals should extend from the frost line and fully penetrate the bedding, sub bedding and cover material. The barriers should consist of relatively dry and compatible brown silty clay placed in maximum 225 mm thick loose layers and compacted to a minimum of 95% of the material’s SPMDD. The clay seals should be placed at the site boundaries and at strategic locations at no more than 60 m intervals in the service trenches.

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. Initial influxes at the sand-to-clay interface may be moderate to high, however, are expected to dissipate as the excavation progresses to the founding depth of the structure and dewatering measures are maintained. 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) may be required if more than 400,000 L/day of ground and/or surface water are to be pumped during the construction phase. At least 4 to 5 months should be allowed for completion of the application and issuance of the permit by the MECP.

For typical ground or surface water volumes being pumped during the construction phase, typically between 50,000 to 400,000 L/day, it is required to register on the Environmental Activity and Sector Registry (EASR). A minimum of two to four weeks should be allotted for completion of the EASR registration and the Water Taking and Discharge Plan to be prepared by a Qualified Persons as stipulated under O.Reg. 63/16.

6.6 Winter Construction

Precautions must be taken if winter construction is considered for this project.

Where excavations are completed in proximity of existing structures which may be adversely affected due to the freezing conditions. The subsurface conditions mostly consist of frost susceptible materials. In presence of water and freezing conditions ice could form within the soil mass. Heaving and settlement upon thawing could occur.

In 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 documents to protect the walls of the excavations from freezing, if and where 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 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 summer time conditions. Paterson 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 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.

6.8 Landscaping Considerations

Tree Planting Considerations

It is understood the proposed building will include one level of underground parking and the structure will be founded on conventional spread footings located at a minimum of 4 m below finished grade. Given the depth of foundation proposed for the structure, it is expected that the support of the foundations derives from soil located below the depth that dewatering due to tree roots. Therefore, foundation distress due to potential moisture depletion caused by trees is not expected to occur at the subject site.

Since the structures are not anticipated to be founded upon silty clay soils affected by the depth of root penetration, City approved trees within the subject site will not be subject to planting restrictions as based on the *City of Ottawa Tree Planting in Sensitive Marine Clay Soils (2017 Guidelines*) from a geotechnical perspective.

6.9 Landslide Hazard Assessment

A Landslide Hazard Assessment report was prepared by this firm in support of the development to be located throughout the properties of 2980, 3054, 2060 and 3080 Navan Road and 6101 Renaud Road and included the subject site. In summary, a pre-historical landslide event is understood to have taken place throughout the subject site, including the location of the future building, and several others within 2 to 3 kilometers of the subject site. These landslide events took place along the banks of the proto-Ottawa River throughout the now-abandoned Mer Bleue paleochannel located south and beyond the subject site.

Field investigations and reconnaissance carried out by Paterson throughout the subject site and neighbouring properties did not indicate any signs of movement, activity or cause of concern with respect to the pre-historic landslide footprint. The area was also reviewed by means of available published literature of the surrounding inventory, research and studies carried out by others specializing in the field of earthquakes, landslides and geology.

Using a combination of the above and our experience with sites of very similar geology throughout the Ottawa region, **the annual probability of a large catastrophic landslide occurring at or directly impacting the subject site was determined to be less than 1:10,000 per annum.**

Based on our interpretation of the information available to carry out this assessment, the subject site is considered safe and suitable for consideration of the purpose of the proposed development. The conclusions and methodology undertaken for the study completed for the larger property is applicable to the development proposed for the subject site.

7.0 Recommendations

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

- Review 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.

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

- Review and inspection of the installation of the foundation drainage systems.
- Observation of all bearing surfaces prior to the placement of concrete.
- Sampling and testing of the concrete and fill materials.
- Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- Observation of all subgrades prior to backfilling and follow-up field density tests to determine the level of compaction achieved.
- 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 soil must 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 Seymour Pacific Developments (Ontario) Ltd., or their agents, is not authorized without review by Paterson for the applicability of our recommendations to the alternative use of the report.

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
May 17, 2023
D. J. GILBERT
100116130
PROVINCE OF ONTARIO

Drew Petahtegoose, B.Eng.

David J. Gilbert, P.Eng.

Report Distribution:

- ☐ Seymour Pacific Developments (Ontario) Ltd. (email copy)
- ☐ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ATTERBERG LIMITS TESTING RESULTS

ANALYTICAL TESTING RESULTS

patersongroup Consulting Engineers
9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Apartment Building Development
3080 Navan Road, Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE December 19, 2022

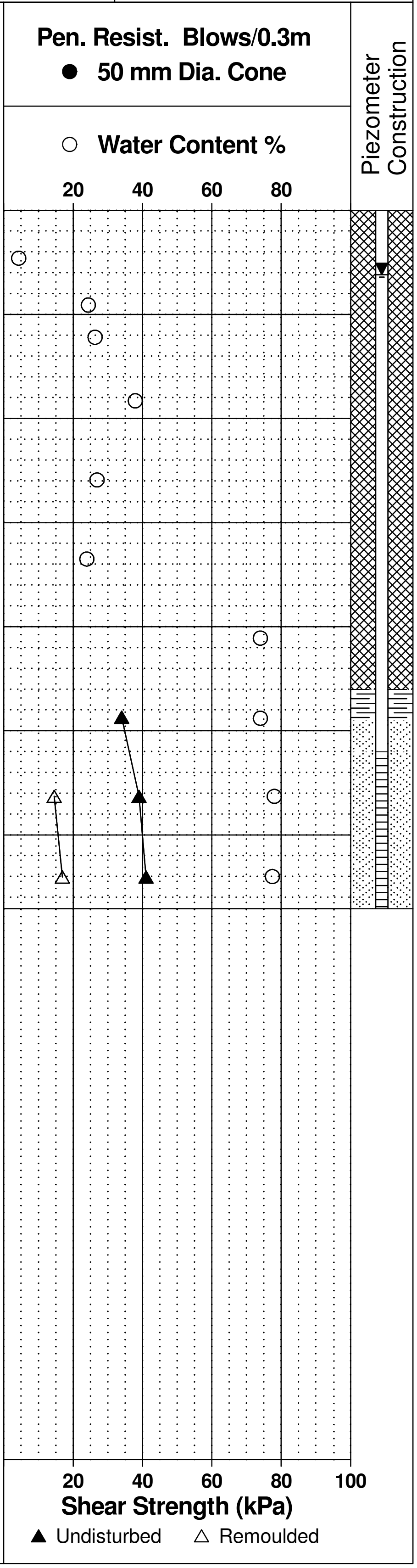
FILE NO. **PG6527**

HOLE NO. **BH 1-22**

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	80.35
FILL: Brown silty sand to sandy silt, some gravel and crushed stone	AU	1				
0.69						
FILL: Brown silty sand, trace to some clay, trace organics	SS	2	79	10	1	79.35
	SS	3	100	11		
2.13					2	78.35
Loose, brown **SILTY SAND**	SS	4	58	5		
					3	77.35
	SS	5	8	1		
3.50						
	SS	6	100	1	4	76.35
Firm, grey **SILTY CLAY**	SS	7	100	P	5	75.35
	SS	8	100	P		
					6	74.35
	SS	9	100	P		
6.71						

End of Borehole

(GWL @ 0.62m - Jan. 3, 2023)



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Apartment Building Development
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DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE December 19, 2022

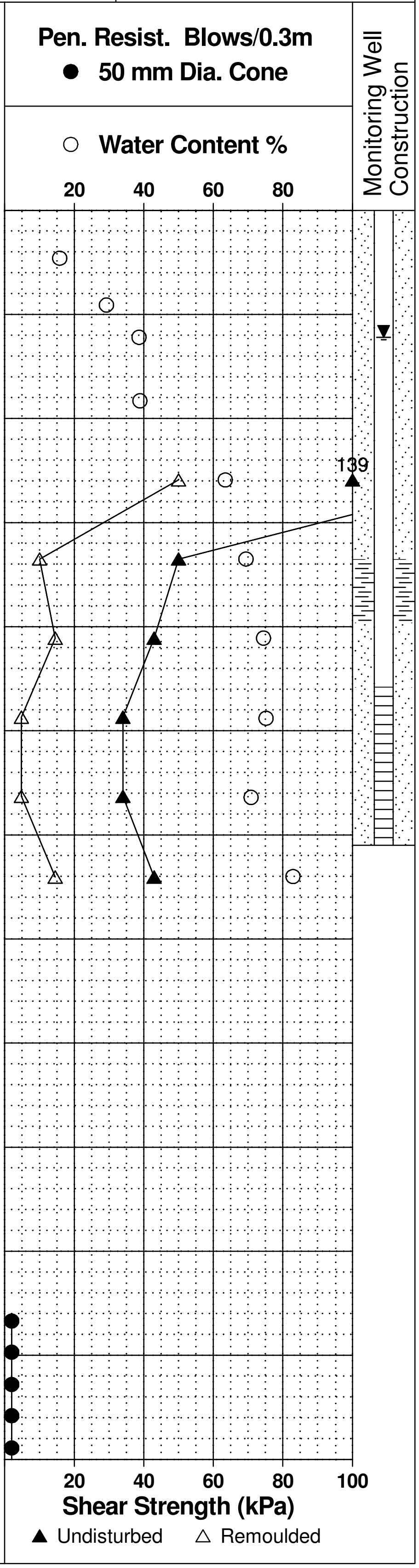
FILE NO. PG6527

HOLE NO. BH 2-22

SOIL DESCRIPTION	
GROUND SURFACE	
FILL: Brown silty sand to sandy silt with gravel and crushed stone	0.30
FILL: Brown silty sand to sandy silt, trace clay and organics	1.27
Brown **SILTY CLAY,** trace sand	1.45
Hard to stiff, brown **SILTY CLAY** - firm and grey by 3.7m depth	6.71
Dynamic Cone Penetration Test commenced at 6.71m depth. Cone pushed to 10.4m depth.	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	71	5
SS	3	100	6
SS	4	100	P
SS	5	100	P
SS	6	100	P
SS	7	100	P
SS	8	100	P
SS	9	100	P

DEPTH (m)	ELEV. (m)
0	80.80
1	79.80
2	78.80
3	77.80
4	76.80
5	75.80
6	74.80
7	73.80
8	72.80
9	71.80
10	70.80
11	69.80
12	68.80



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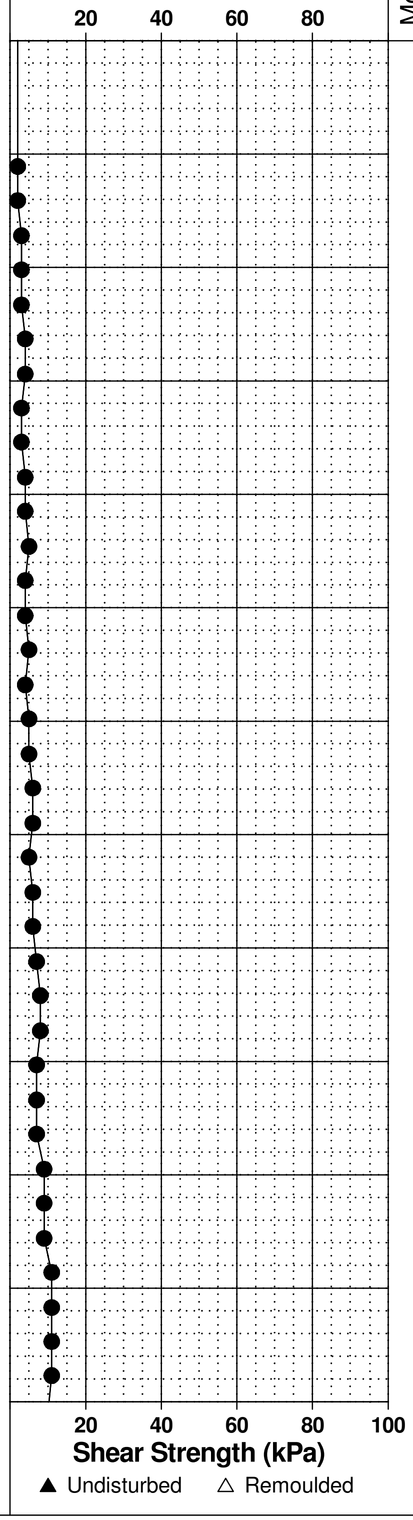
FILE NO. PG6527

HOLE NO. BH 2-22

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE							
Dynamic Cone Penetration Test commenced at 6.71m depth. Cone pushed to 10.4m depth.						12	68.80
						13	67.80
						14	66.80
						15	65.80
						16	64.80
						17	63.80
						18	62.80
						19	61.80
						20	60.80
						21	59.80
						22	58.80
						23	57.80
						24	56.80

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

○ Water Content %



Monitoring Well Construction

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Apartment Building Development
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DATUM Geodetic

REMARKS

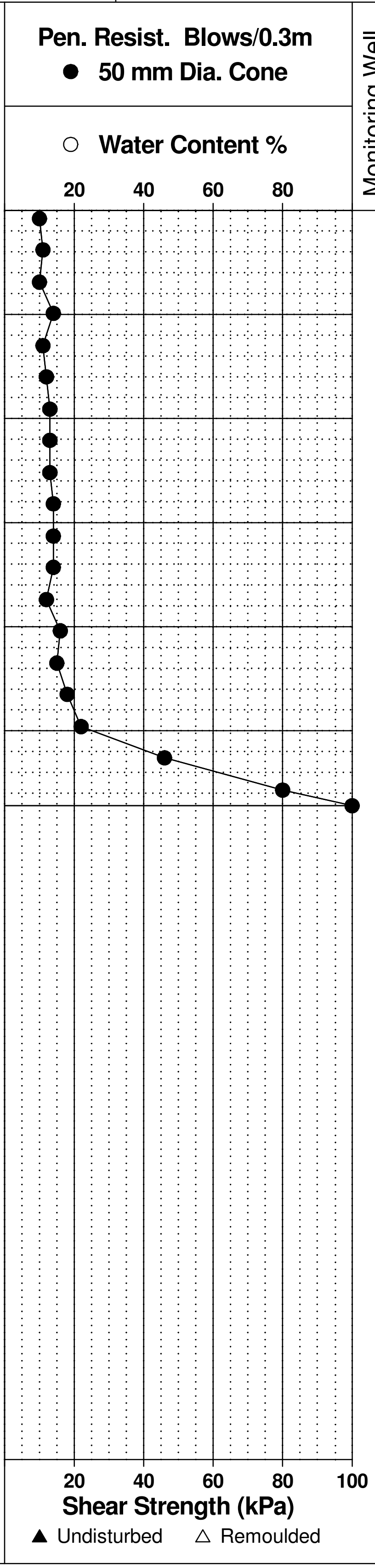
BORINGS BY Track-Mount Power Auger

DATE December 19, 2022

FILE NO. PG6527

HOLE NO. BH 2-22

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE							
Dynamic Cone Penetration Test commenced at 6.71m depth. Cone pushed to 10.4m depth.						24	56.80
						25	55.80
						26	54.80
						27	53.80
						28	52.80
						29	51.80
29.72							
End of Borehole Practical DCPT refusal at 29.72m depth. (GWL @ 1.22m - Jan. 3, 2023)							



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Apartment Building Development
3080 Navan Road, Ottawa, Ontario

DATUM Geodetic

REMARKS

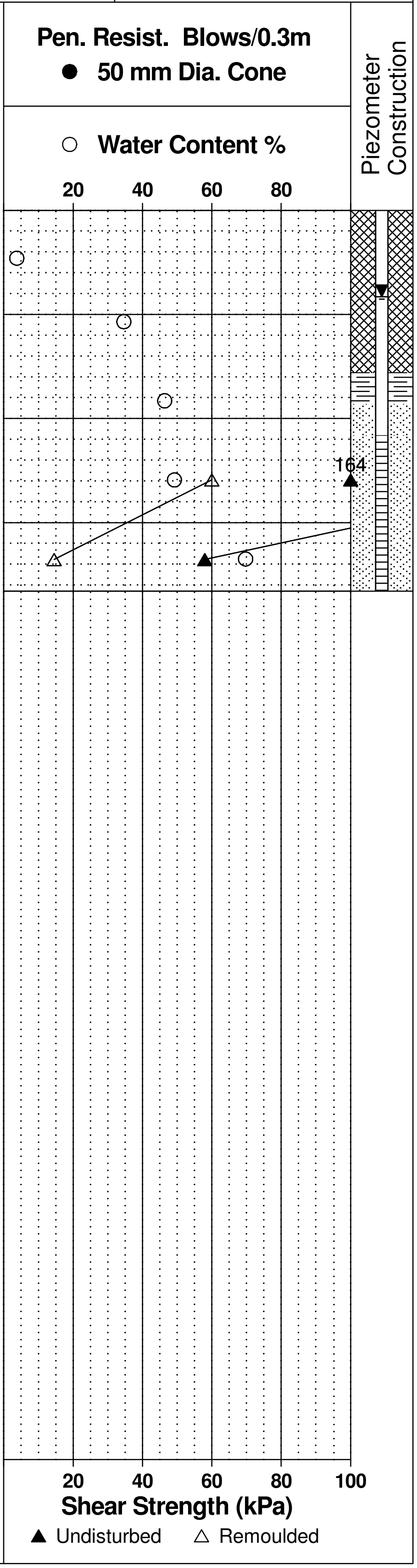
BORINGS BY Track-Mount Power Auger

DATE December 19, 2022

FILE NO. PG6527

HOLE NO. BH 3-22

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	79.94
FILL: Brown silty sand to sandy silt with gravel and clay 0.69		AU	1				
FILL: Brown silty clay, trace topsoil and organics 1.52		SS	2	58	14	1	78.94
Hard to very stiff, brown **SILTY CLAY**		SS	3	100	7	2	77.94
		SS	4	100	50+		
- stiff by 3.0m depth						3	76.94
3.66		SS	5	100	50+		
End of Borehole (GWL @ 0.83m - Jan. 3, 2023)							



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SOIL PROFILE AND TEST DATA

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Proposed Apartment Building Development
3080 Navan Road, Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE December 20, 2022

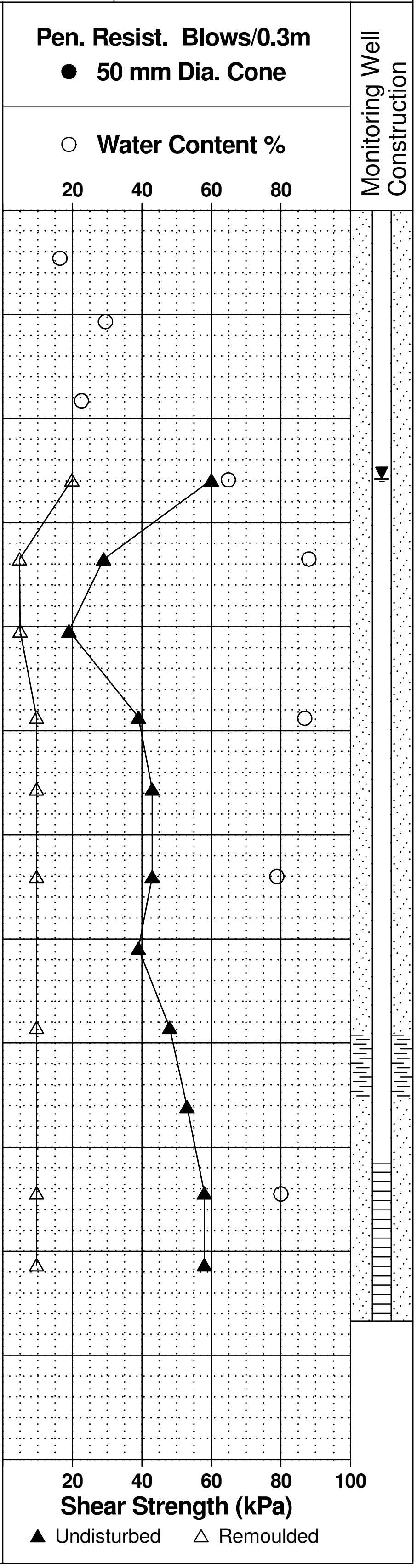
FILE NO. PG6527

HOLE NO. BH 4-22

SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
FILL: Brown silty sand to sandy silt, trace gravel and crushed stone - trace clay by 0.3m depth	0.69
FILL: Brown silty clay, trace topsoil and sand	1.45
Loose, brown **SILTY SAND**	1.98
Stiff to firm, brown **SILTY CLAY** - soft to firm and grey by 3.0m depth - stiff by 8.2m depth	10.36
Dynamic Cone Penetration Test commenced at 10.36m depth. Cone pushed to 13.4m depth.	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	100	6
SS	3	67	7
SS	4	100	P
SS	5	100	P
SS	6	100	P
SS	7	100	P
SS	8	100	P

DEPTH (m)	ELEV. (m)
0	80.53
1	79.53
2	78.53
3	77.53
4	76.53
5	75.53
6	74.53
7	73.53
8	72.53
9	71.53
10	70.53
11	69.53
12	68.53



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9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Apartment Building Development
3080 Navan Road, Ottawa, Ontario

DATUM Geodetic

REMARKS

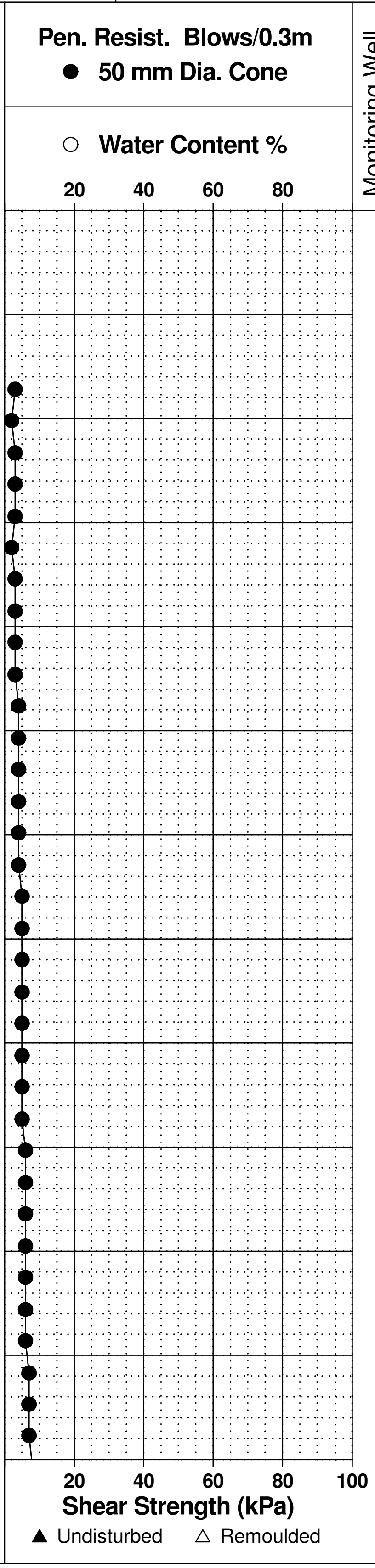
BORINGS BY Track-Mount Power Auger

DATE December 20, 2022

FILE NO. PG6527

HOLE NO. BH 4-22

SOIL DESCRIPTION GROUND SURFACE	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
Dynamic Cone Penetration Test commenced at 10.36m depth. Cone pushed to 13.4m depth.						12	68.53
						13	67.53
						14	66.53
						15	65.53
						16	64.53
						17	63.53
						18	62.53
						19	61.53
						20	60.53
						21	59.53
						22	58.53
						23	57.53
						24	56.53



Monitoring Well Construction

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Apartment Building Development
3080 Navan Road, Ottawa, Ontario

DATUM Geodetic

REMARKS

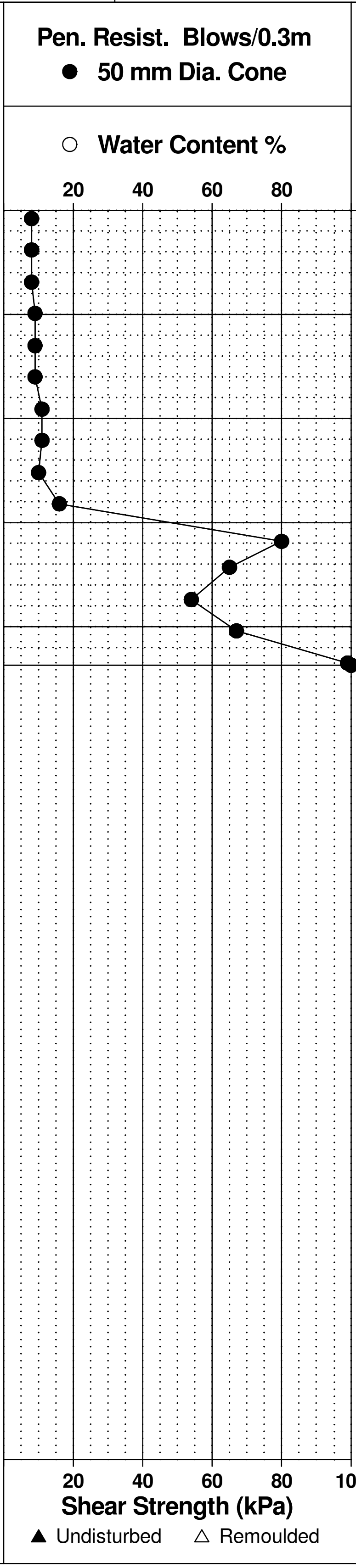
BORINGS BY Track-Mount Power Auger

DATE December 20, 2022

FILE NO. PG6527

HOLE NO. BH 4-22

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE							
Dynamic Cone Penetration Test commenced at 10.36m depth. Cone pushed to 13.4m depth.						24	56.53
						25	55.53
						26	54.53
						27	53.53
						28	52.53
28.37							
End of Borehole Practical DCPT refusal at 28.37m depth. (GWL @ 2.58m - Jan. 3, 2023)							



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Apartment Building Development
3080 Navan Road, Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE December 20, 2022

FILE NO. PG6527

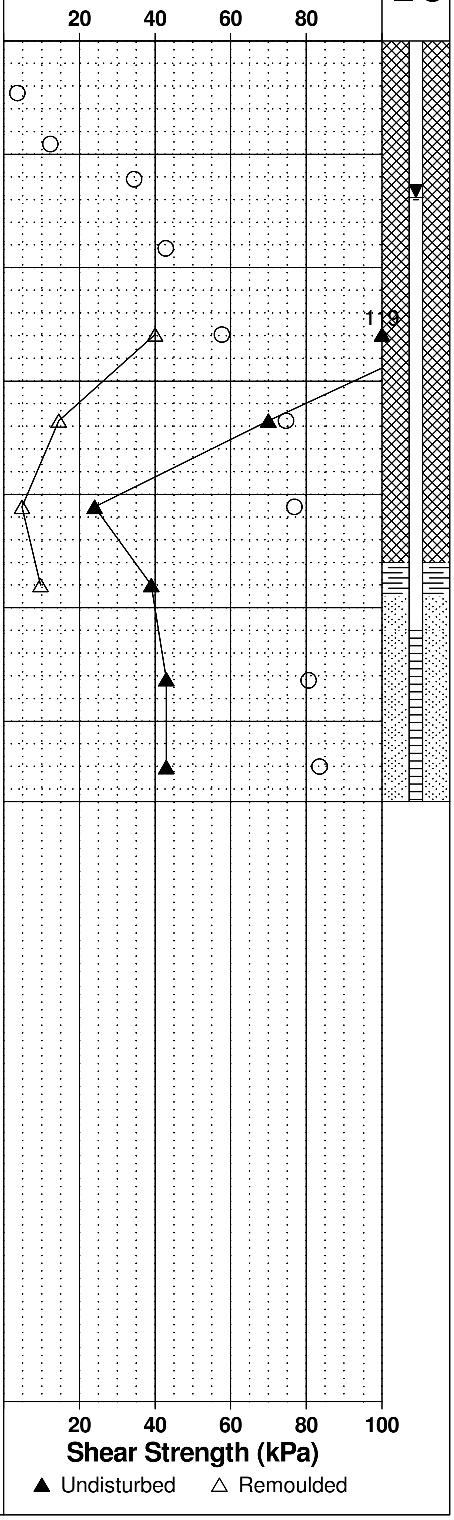
HOLE NO. BH 5-22

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	80.66
FILL: Brown silty sand to sandy silt, some to trace gravel and crushed stone		AU	1				
1.07		SS	2	75	19	1	79.66
FILL: Brown to grey silty clay, trace sand and gravel - some topsoil from 1.1 to 1.4m depth		SS	3	79	P	2	78.66
2.21							
Very stiff to stiff, brown **SILTY CLAY** - soft to firm and grey by 3.7m depth		SS	4	100	P	3	77.66
		SS	5	100	P		
		SS	6	100	P	4	76.66
						5	75.66
		SS	7	100	P	6	74.66
		SS	8	100	P		
6.71							
End of Borehole (GWL @ 1.38m - Jan. 3, 2023)							

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

○ **Water Content %**

Piezometer Construction



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Apartment Building Development
3080 Navan Road, Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE December 20, 2022

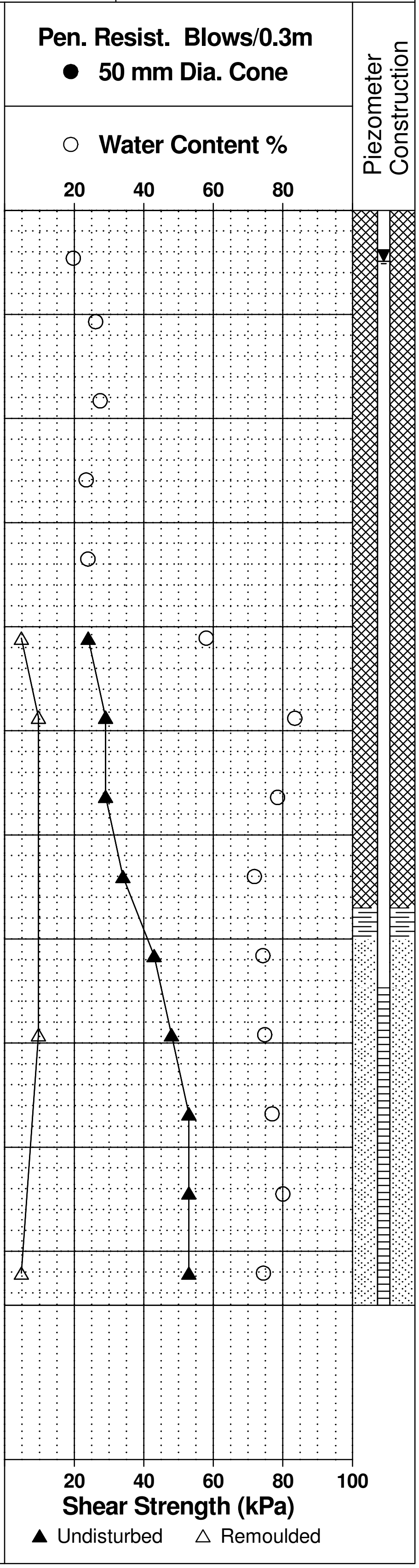
FILE NO. **PG6527**

HOLE NO. **BH 6-22**

SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
FILL: Organics with sand, gravel, crushed stone	0.05
FILL: Brown silty sand, some topsoil, trace gravel and clay	0.69
Compact, brown **SILTY SAND to SANDY SILT**	1.45
Loose to very loose, brown **SILTY SAND** - some running sand by 1.8m depth	3.73
Soft to firm, grey **SILTY CLAY** - stiff by 8.2m depth	10.52
End of Borehole (GWL @ 0.49m - Jan. 3, 2023)	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	75	10
SS	3	75	10
SS	4	88	6
SS	5	17	1
SS	6	100	P
SS	7	100	P
SS	8	100	P
SS	9	100	P
SS	10	100	P
SS	11	100	P
SS	12	100	P
SS	13	100	P
SS	14	100	P

DEPTH (m)	ELEV. (m)
0	80.52
1	79.52
2	78.52
3	77.52
4	76.52
5	75.52
6	74.52
7	73.52
8	72.52
9	71.52
10	70.52



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
6101 Renauld Road
Ottawa, Ontario

DATUM Geodetic

FILE NO. **PG5353**

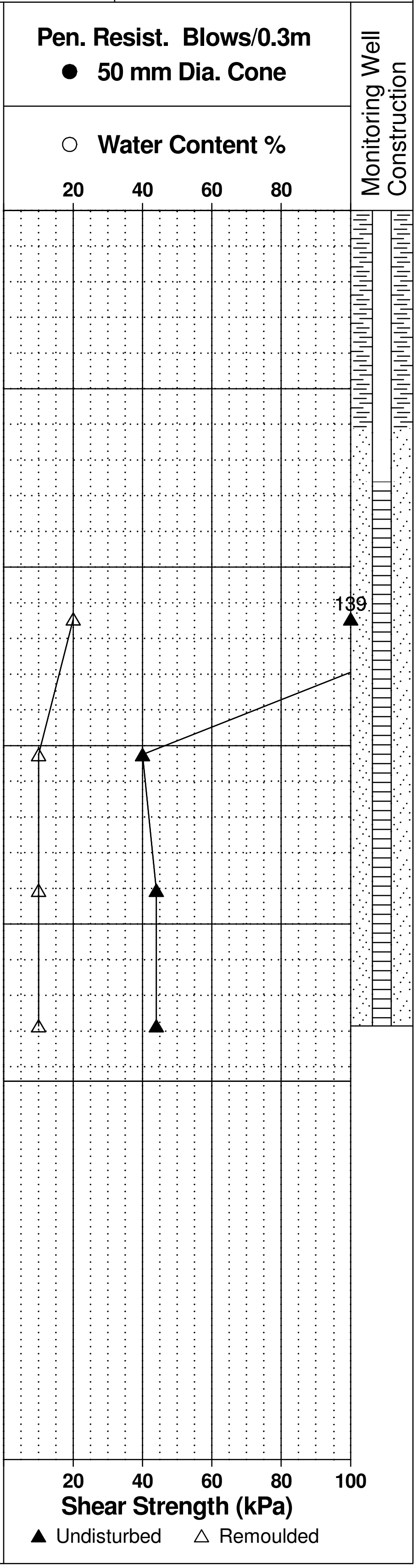
REMARKS

HOLE NO. **BH 1-21**

BORINGS BY CME 55 Power Auger

DATE 2021 March 8

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD		
GROUND SURFACE						0	78.81
FILL: Brown silty clay trace sand, gravel, cobbles and boulders		AU	1				
		SS	2	38	3	1	77.81
1.83		SS	3	33	2		
Firm to stiff brown **SILTY CLAY**						2	76.81
- Grey by 3.0 m depth						3	75.81
						4	74.81
4.88							
End of Borehole							



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SOIL PROFILE AND TEST DATA

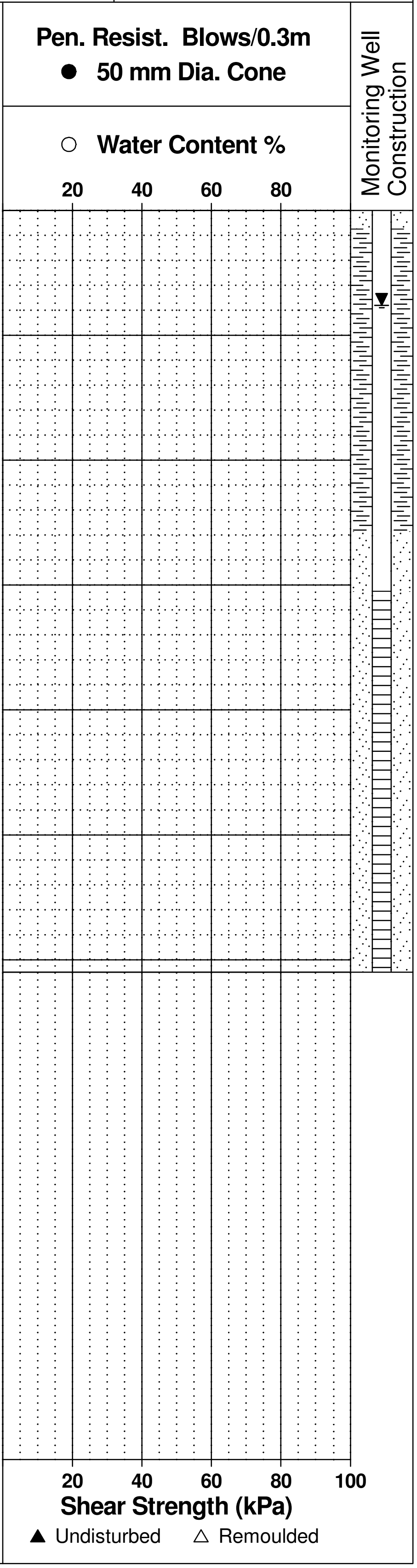
Geotechnical Investigation
Prop. Residential Development - 6101 Renaud Road
Ottawa, Ontario

DATUM Geodetic
REMARKS
BORINGS BY Track-Mount Power Auger
DATE June 1, 2020
FILE NO. PG5353
HOLE NO. BH 1

SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
TOPSOIL	0.15
FILL: Brown silty sand, some crushed stone and gravel	2.29
Compact, grey **SILTY SAND,** trace gravel	3.20
Firm, grey **SILTY CLAY**	6.10
End of Borehole (GWL @ 0.76m - June 10, 2020)	

TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	42	10
SS	3	67	6
SS	4	58	12
SS	5	4	P
SS	6	71	P
SS	7	92	P
SS	8	100	P

DEPTH (m)	ELEV. (m)
0	80.19
1	79.19
2	78.19
3	77.19
4	76.19
5	75.19
6	74.19



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - 6101 Renaud Road
Ottawa, Ontario

DATUM Geodetic

FILE NO. **PG5353**

REMARKS

HOLE NO. **BH 7**

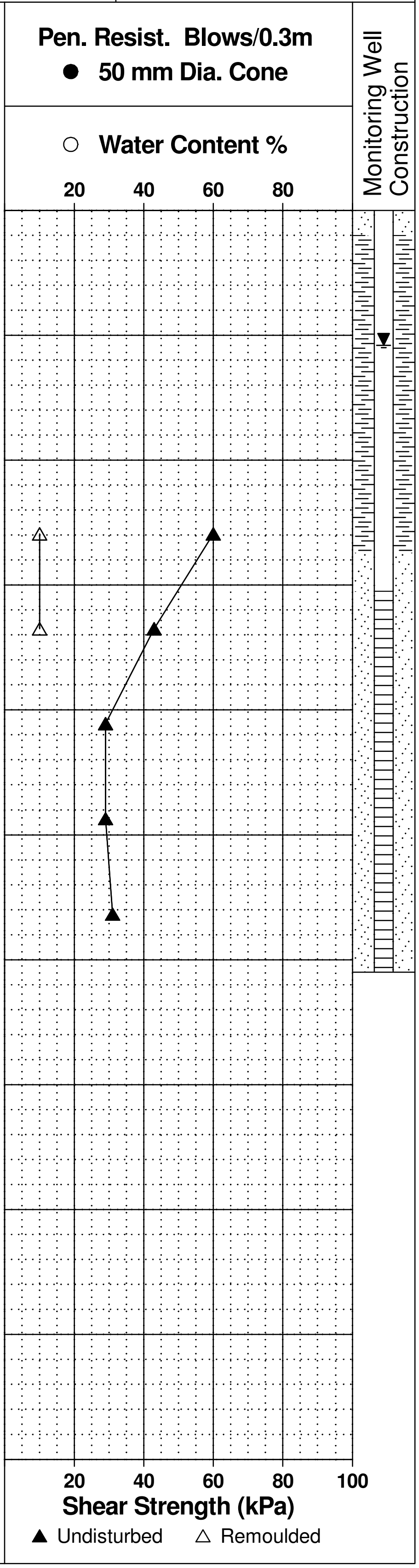
BORINGS BY Track-Mount Power Auger

DATE June 2, 2020

SOIL DESCRIPTION		TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
TOPSOIL	0.15	AU	1		
FILL: Brown silty sand, some crushed stone	0.60				
Compact, brown **SILTY SAND**	1.37	SS	2	67	10
Very stiff to stiff, brown **SILTY CLAY** - firm and grey by 2.3m depth	6.10	SS	3	88	5
		SS	4		P
		SS	5		P
		SS	6		P
		SS	7		P
		SS	8		P

Dynamic Cone Penetration Test commenced at 27.13m depth. Cone pushed to 21.3m depth.

DEPTH (m)	ELEV. (m)
0	80.29
1	79.29
2	78.29
3	77.29
4	76.29
5	75.29
6	74.29
7	73.29
8	72.29
9	71.29
10	70.29



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - 6101 Renaud Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE June 2, 2020

FILE NO. PG5353

HOLE NO. BH 7

SOIL DESCRIPTION GROUND SURFACE	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	Monitoring Well Construction
						10	70.29		
						11	69.29		
						12	68.29		
						13	67.29		
						14	66.29		
						15	65.29		
						16	64.29		
						17	63.29		
						18	62.29		
						19	61.29		
						20	60.29		

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

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - 6101 Renaud Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

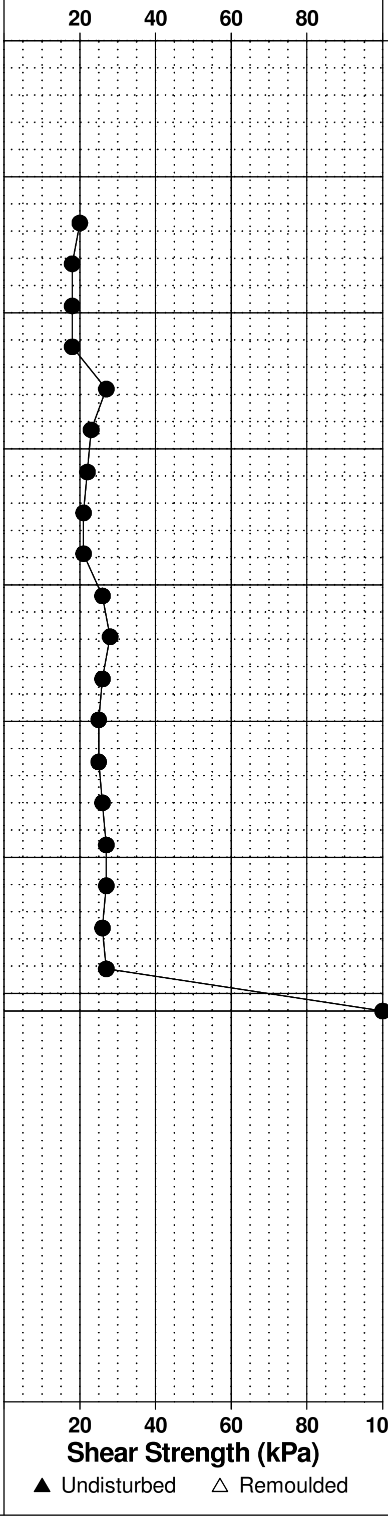
BORINGS BY Track-Mount Power Auger

DATE June 2, 2020

FILE NO. PG5353

HOLE NO. BH 7

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content %	Monitoring Well Construction
GROUND SURFACE						20	60.29		
						21	59.29		
						22	58.29		
						23	57.29		
						24	56.29		
						25	55.29		
						26	54.29		
27.13						27	53.29		



End of Borehole

Practical DCPT refusal at 27.13m depth.

(GWL @ 1.08m - June 10, 2020)

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - 6101 Renaud Road
Ottawa, Ontario

DATUM Geodetic

FILE NO. PG5353

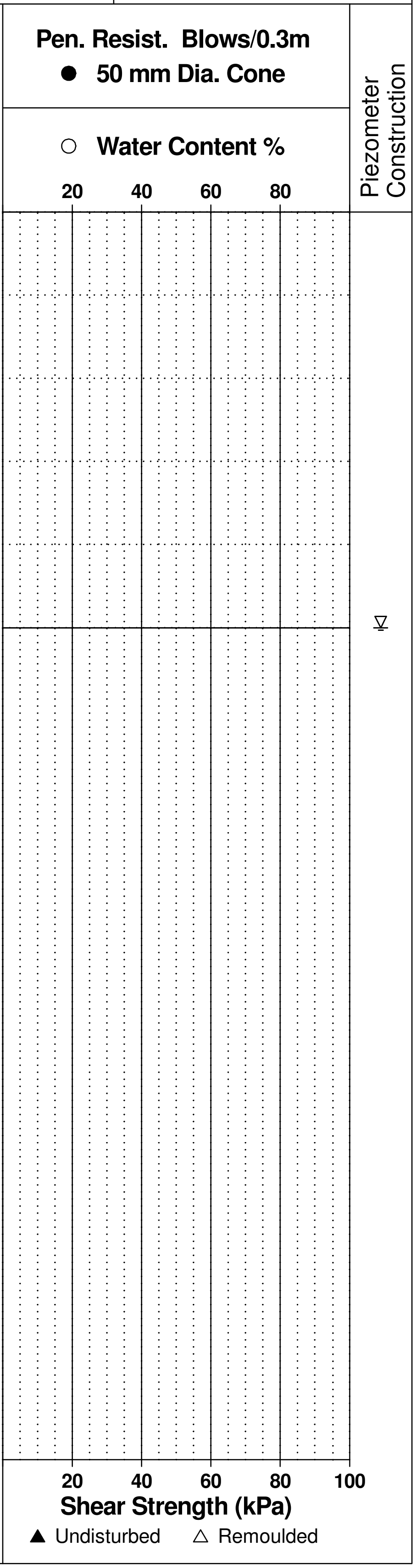
REMARKS

BORINGS BY Backhoe

DATE May 28, 2020

HOLE NO. TP 1

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD		
GROUND SURFACE						0	80.15
FILL: Brown silty sand with crushed stone and organics 0.30	▨	G	1				
FILL: Brown silty sand, some gravel	▨	G	2				
	▨	G	3				
	▨					1	79.15
1.20							
End of Test Pit TP terminated in fill at 1.20m depth. (Groundwater infiltration at 1.0m depth)							



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - 6101 Renaud Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Backhoe

DATE May 28, 2020

FILE NO. **PG5353**

HOLE NO. **TP 2**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content % (20 40 60 80)	Piezometer Construction
GROUND SURFACE						0	80.23		
FILL: Brown silty sand with crushed stone 0.30		G	1						
Asphaltic concrete 0.37									
FILL: Brown silty sand, trace clay and gravel		G	2						
1.20		G	3			1	79.23		
Compact, brown **SILTY SAND to SANDY SILT** 1.60		G	4						⊻
End of Test Pit TP terminated in sandy silt aty 1.60m depth (Groundwater infiltration at 1.4m depth)									

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

The standard terminology to describe the 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)

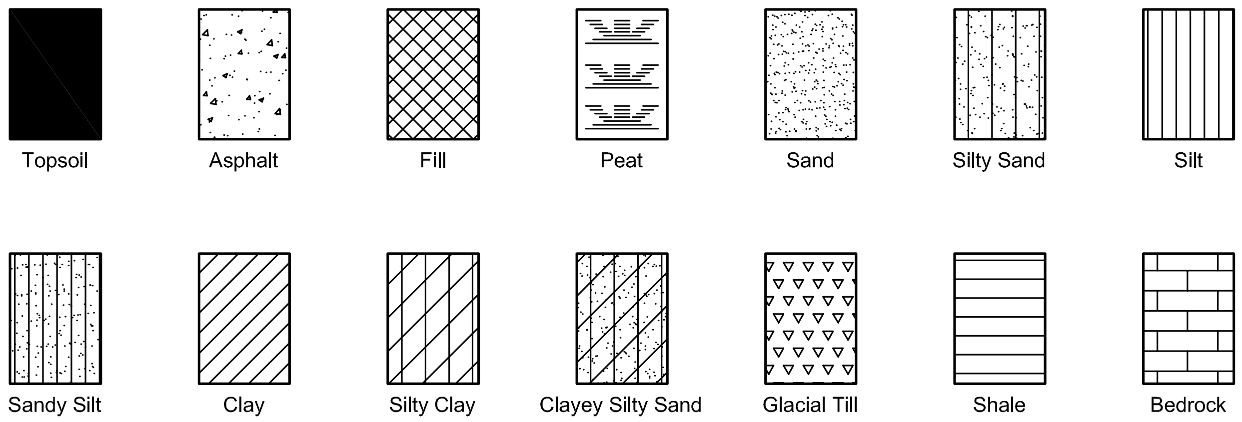
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

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.

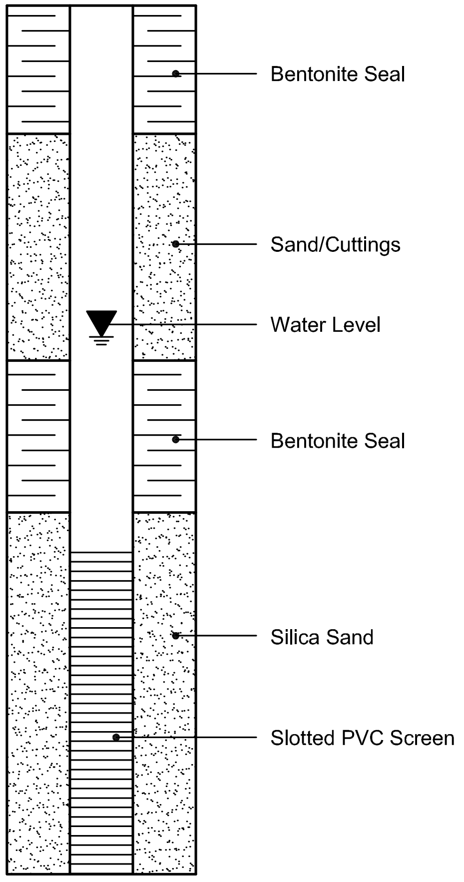
SYMBOLS AND TERMS (continued)

STRATA PLOT

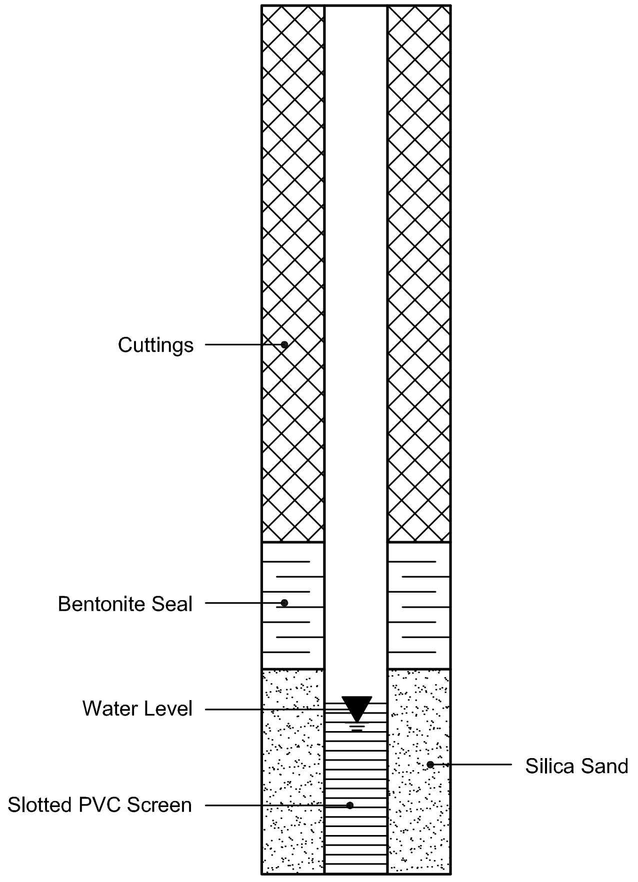


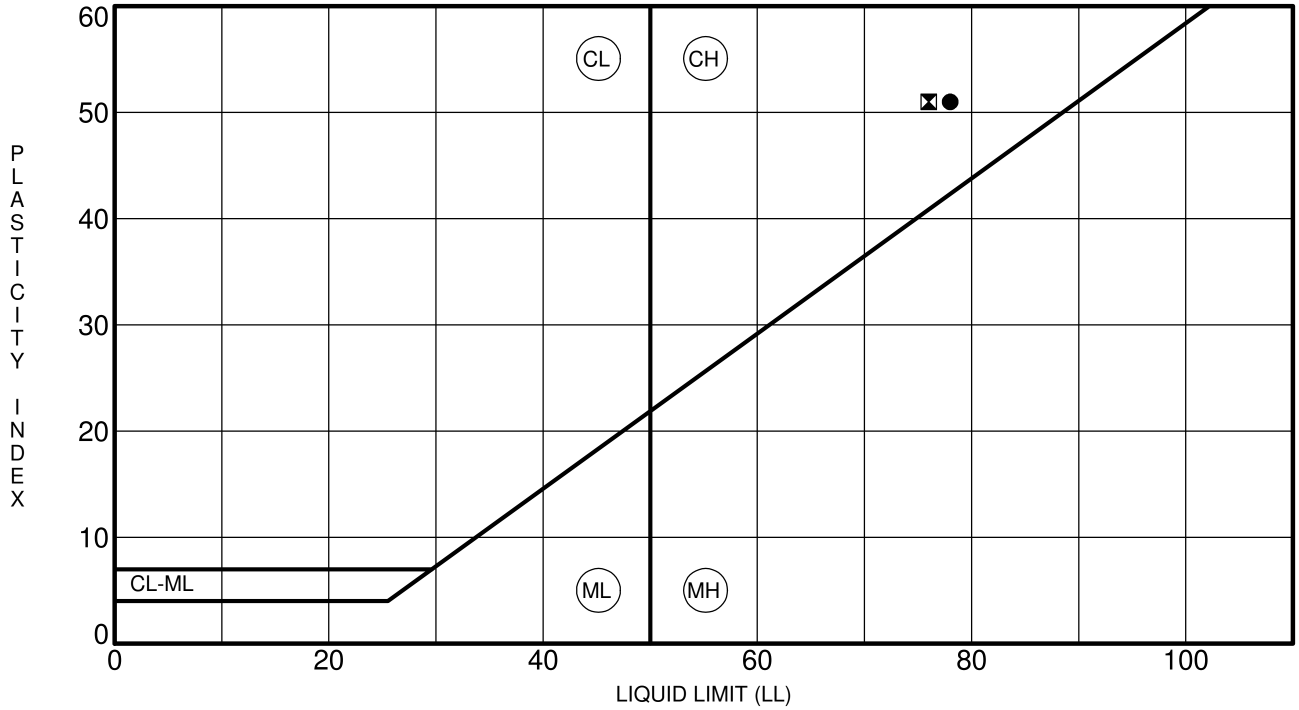
MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION





	Specimen Identification		LL	PL	PI	Fines	Classification
●	**BH 2-22**	**SS4**	**78**	**27**	**51**		**CH - Inorganic clay of high plasticity**
☒	**BH 5-22**	**SS3**	**76**	**25**	**51**		**CH - Inorganic clay of high plasticity**

CLIENT **Seymour Pacific Developments Ltd.**

FILE NO. **PG6527**

PROJECT **Geotechnical Investigation - Proposed Apartment Building Development**

DATE **20 Dec 22**

patersongroup **Consulting Engineers**

9 Auriga Drive, Ottawa, Ontario K2E 7T9

ATTERBERG LIMITS' RESULTS

Certificate of Analysis
Client: **Paterson Group Consulting Engineers**
Client PO: 56522

Report Date: 05-Jan-2023
Order Date: 29-Dec-2022
Project Description: PG6527

	Client ID:	BH5-22 SS5	-	-	-
	Sample Date:	20-Dec-22 09:00	-	-	-
	Sample ID:	2253055-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	57.2	-	-	-
General Inorganics					
pH	0.05 pH Units	7.60	-	-	-
Resistivity	0.10 Ohm.m	7.59	-	-	-
Anions					
Chloride	10 ug/g dry	766	-	-	-
Sulphate	10 ug/g dry	76	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURE 2 & 3 – SEISMIC SHEAR WAVE VELOCITY PROFILES

FIGURE 4 – PODIUM DECK TO FOUNDATION WALL DRAINAGE SYSTEM TIE-IN DETAIL

DRAWING PG6527-1 - TEST HOLE LOCATION PLAN

DRAWING PG6527-2 – PERMISSIBLE GRADE RAISE PLAN

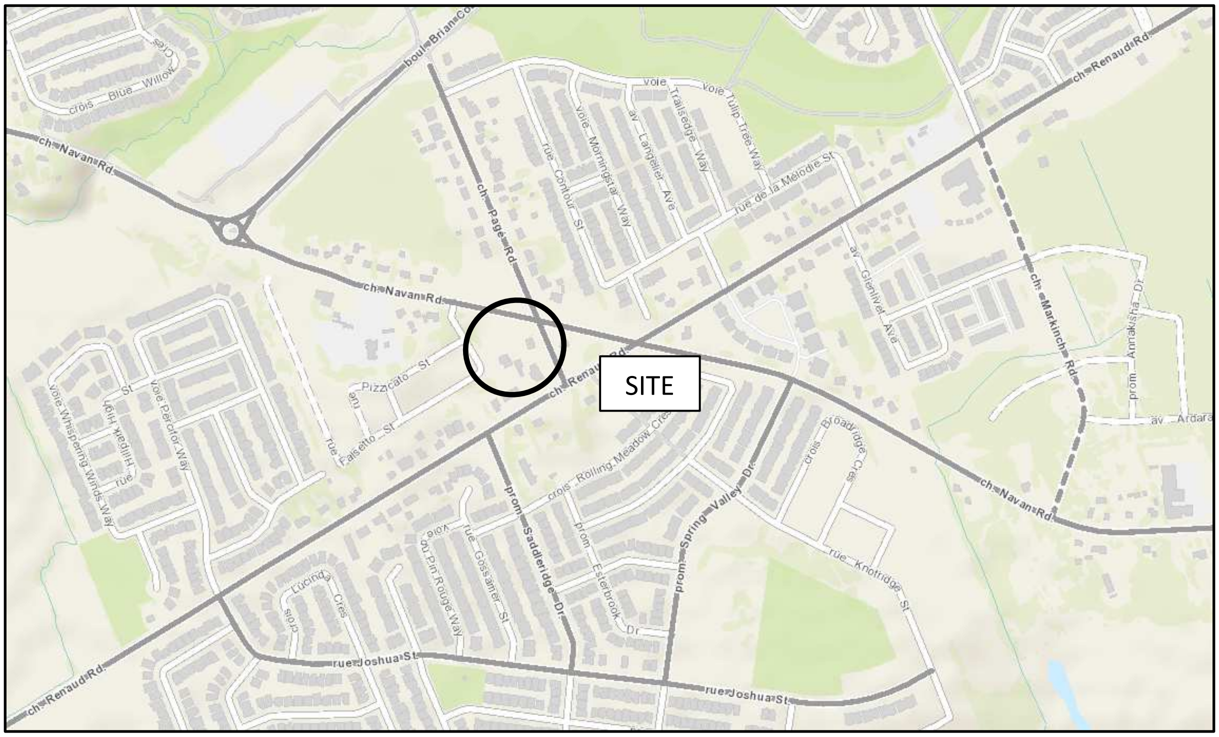


FIGURE 1

KEY PLAN

PATERSON
GROUP

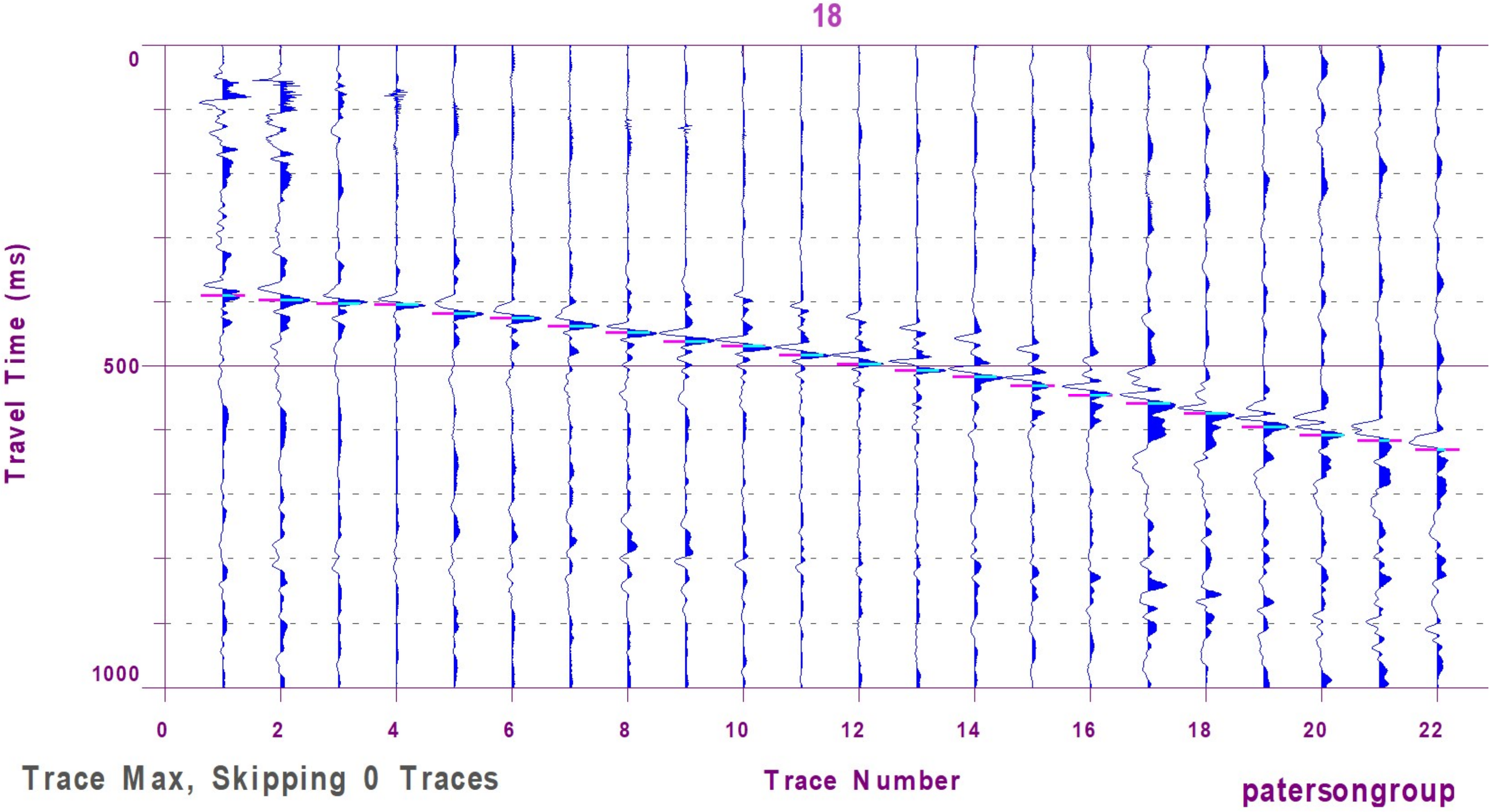


Figure 2 – Shear Wave Velocity Profile at Shot Location -11.5 m

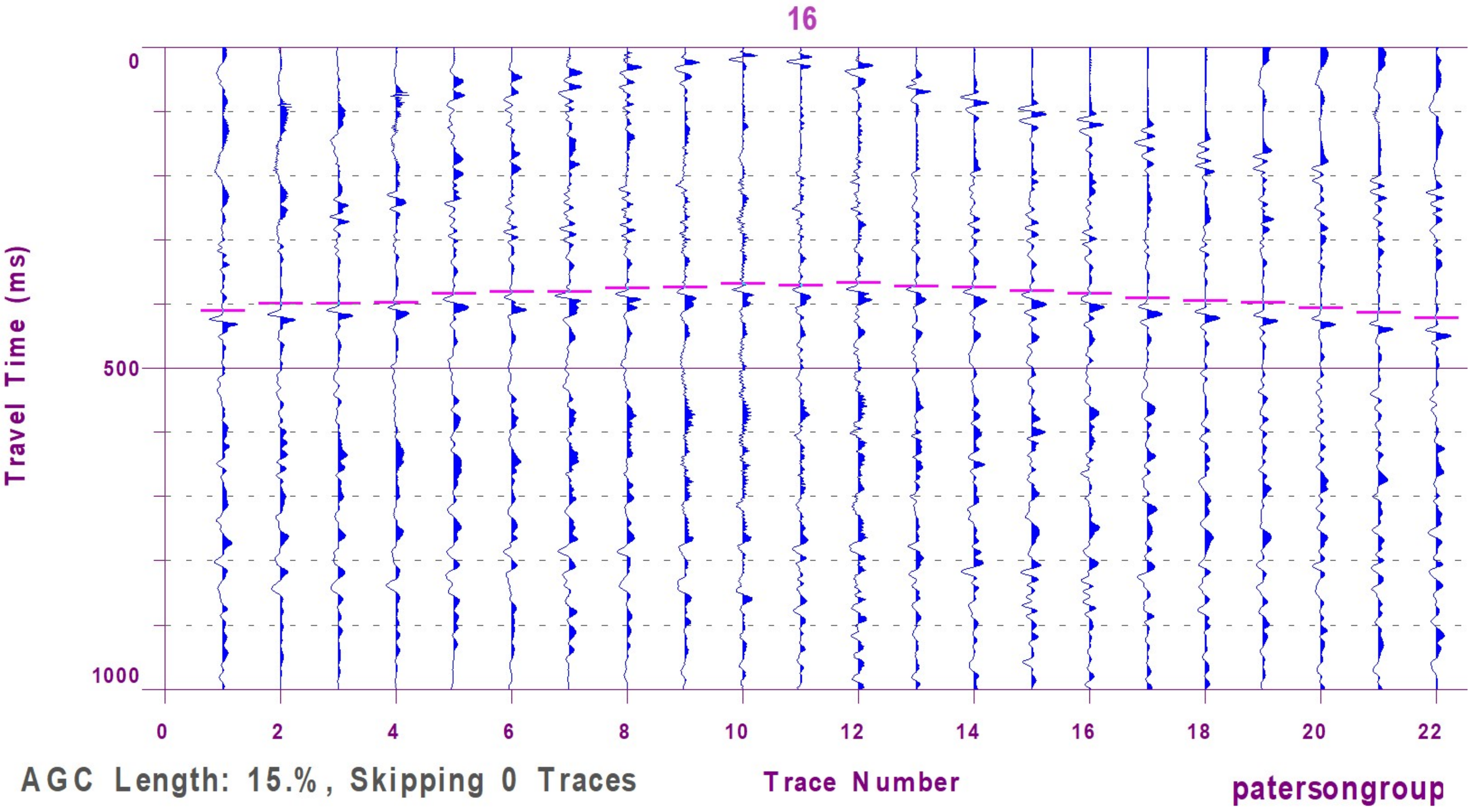
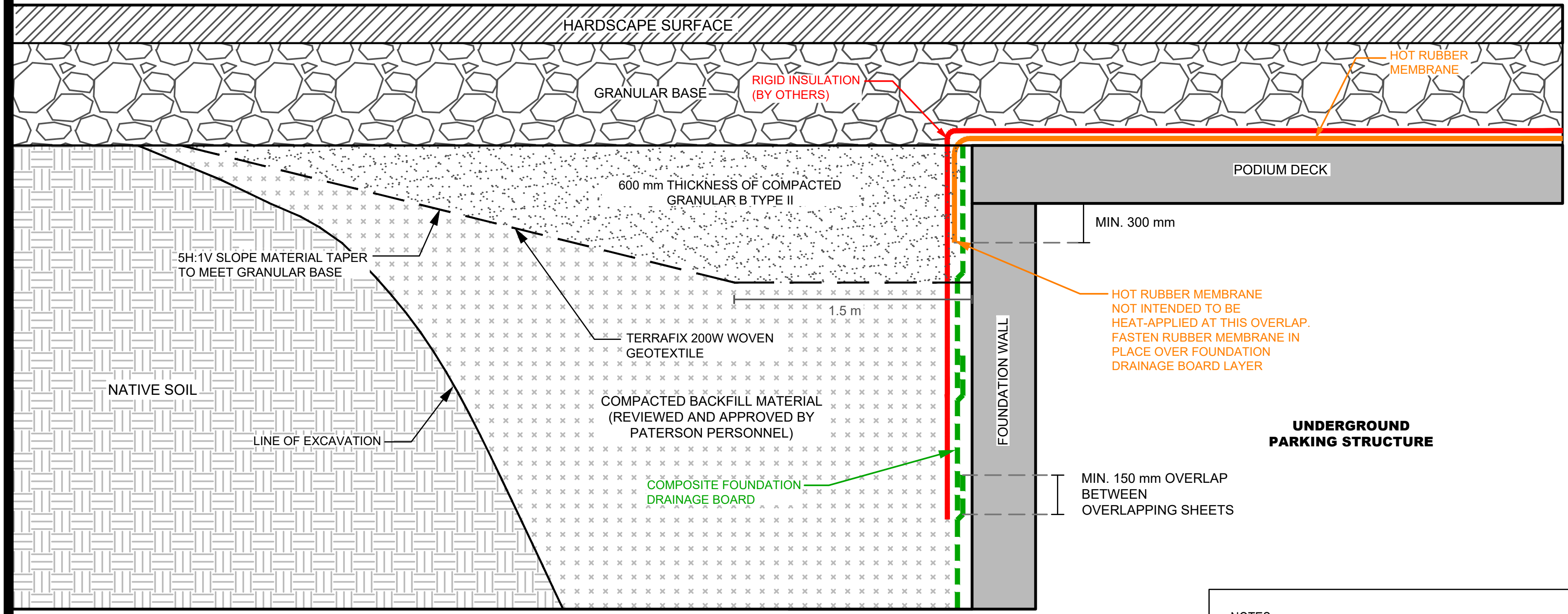


Figure 3 – Shear Wave Velocity Profile at Shot Location 34.5 m



ALL PORTIONS OF THE ABOVE-NOTED DETAIL (INSULATION OF FOUNDATION DRAINAGE BOARD, HOT-RUBBER MEMBRANE OVER SLAB AND FOUNDATION WALL CONSTRUCTION JOINT AND OVERLAPPING/SHINGLING OF DRAINAGE BOARD SHOULD BE REVIEWED AT THE TIME OF CONSTRUCTION BY PATERSON PERSONNEL.

NOTES:

THE ABOVE DETAIL FOR HOT RUBBER, AND DRAINAGE BOARD OVERLAP IS APPLICABLE TO ALL EDGE-PORTIONS OF THE PODIUM DECK AND/OR SUSPENDED GROUND FLOOR SLAB STRUCTURE.

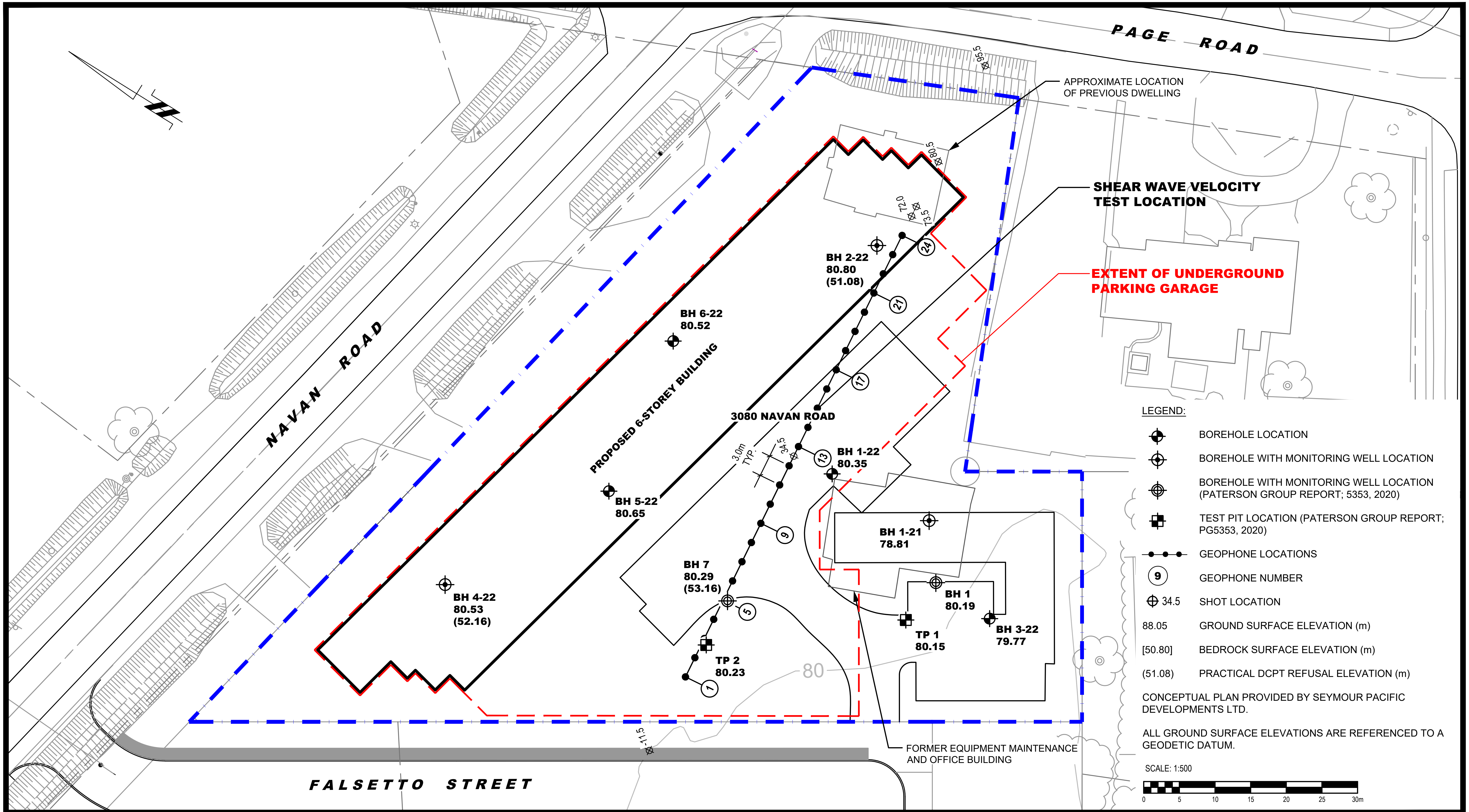
APPLICABILITY, THICKNESS AND EXTENSIONS OF RIGID INSULATION ARE SPECIFIED BY OTHERS

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

SEYMOUR PACIFIC DEVELOPMENTS (ONTARIO) LTD.
PROPOSED APARTMENT BUILDING DEVELOPMENT
3080 NAVAN ROAD
OTTAWA, ONTARIO

Title:
PODIUM DECK TO FOUNDATION WALL DRAINAGE SYSTEM TIE-IN DETAIL

Scale: NTS	Date: 03/2023
Drawn by: NFRV	Report No.: PG6527-1 R1
Checked by: DP	Drawing No.: FIGURE 4
Approved by: DJG	Revision No.:



LEGEND:

⊕	BOREHOLE LOCATION
⊕	BOREHOLE WITH MONITORING WELL LOCATION
⊕	BOREHOLE WITH MONITORING WELL LOCATION (PATERSON GROUP REPORT; 5353, 2020)
▨	TEST PIT LOCATION (PATERSON GROUP REPORT; PG5353, 2020)
●–●–●	GEOPHONE LOCATIONS
⑨	GEOPHONE NUMBER
⊕ 34.5	SHOT LOCATION
88.05	GROUND SURFACE ELEVATION (m)
[50.80]	BEDROCK SURFACE ELEVATION (m)
(51.08)	PRACTICAL DCPT REFUSAL ELEVATION (m)

CONCEPTUAL PLAN PROVIDED BY SEYMOUR PACIFIC DEVELOPMENTS LTD.

ALL GROUND SURFACE ELEVATIONS ARE REFERENCED TO A GEODETIC DATUM.

SCALE: 1:500

0 5 10 15 20 25 30m

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NO.	REVISIONS	DATE	INITIAL

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GEOTECHNICAL INVESTIGATION
PROPOSED APARTMENT BUILDING DEVELOPMENT
3080 NAVAN ROAD

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:500
Date: 12/2022
Drawn by: JM
Report No.: PG6527-1
Checked by: DP
Dwg. No.: PG6527-1
Approved by: DP
Revision No.:

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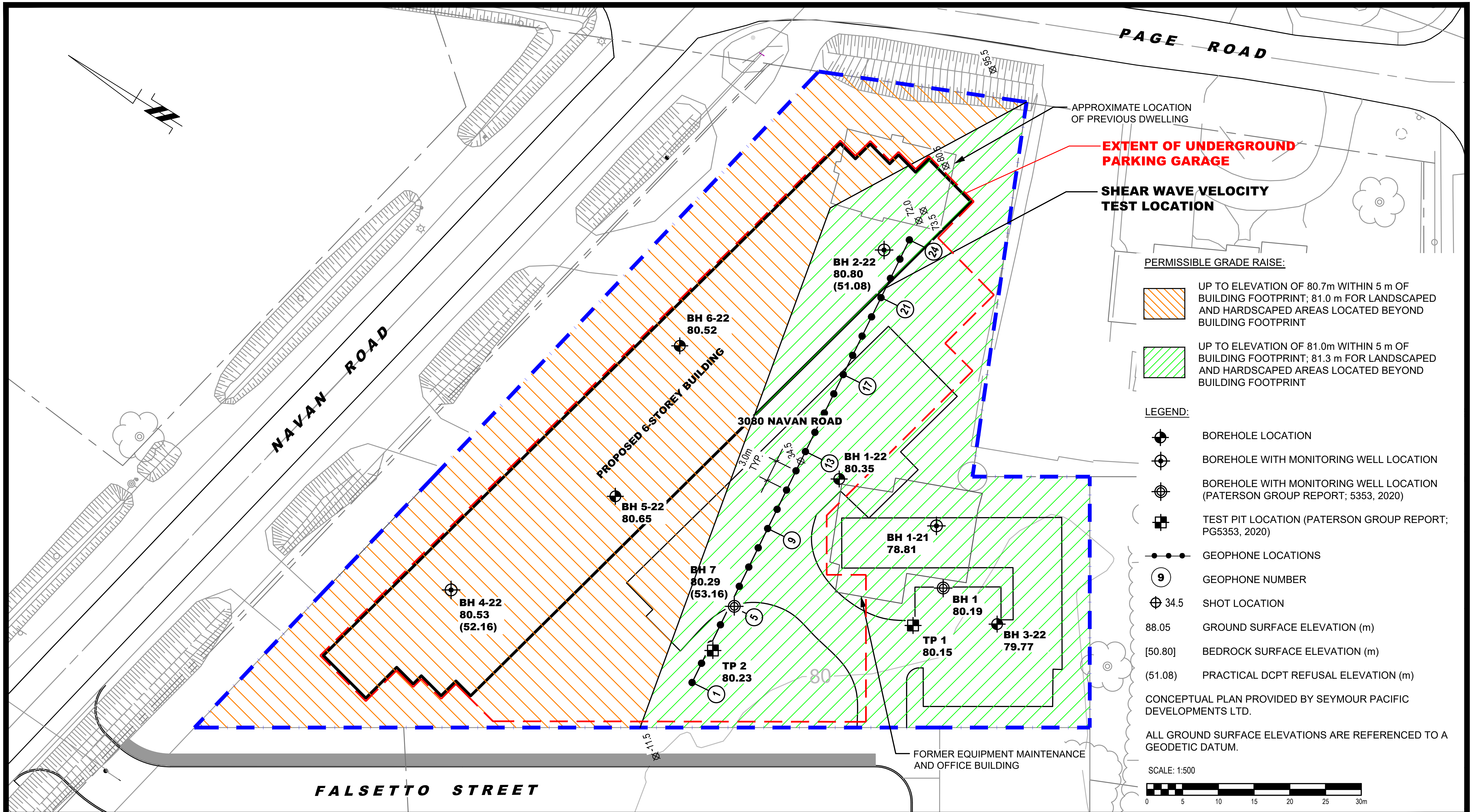

PERMISSIBLE GRADE RAISE:

🟧	UP TO ELEVATION OF 80.7m WITHIN 5 m OF BUILDING FOOTPRINT; 81.0 m FOR LANDSCAPED AND HARDSCAPED AREAS LOCATED BEYOND BUILDING FOOTPRINT
🟩	UP TO ELEVATION OF 81.0m WITHIN 5 m OF BUILDING FOOTPRINT; 81.3 m FOR LANDSCAPED AND HARDSCAPED AREAS LOCATED BEYOND BUILDING FOOTPRINT

LEGEND:

⊕	BOREHOLE LOCATION
⊕	BOREHOLE WITH MONITORING WELL LOCATION
⊕	BOREHOLE WITH MONITORING WELL LOCATION (PATERSON GROUP REPORT; 5353, 2020)
▣	TEST PIT LOCATION (PATERSON GROUP REPORT; PG5353, 2020)
●—●—●	GEOPHONE LOCATIONS
⑨	GEOPHONE NUMBER
⊕ 34.5	SHOT LOCATION
88.05	GROUND SURFACE ELEVATION (m)
[50.80]	BEDROCK SURFACE ELEVATION (m)
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CONCEPTUAL PLAN PROVIDED BY SEYMOUR PACIFIC DEVELOPMENTS LTD.

ALL GROUND SURFACE ELEVATIONS ARE REFERENCED TO A GEODETIC DATUM.

SCALE: 1:500

0 5 10 15 20 25 30m

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GEOTECHNICAL INVESTIGATION
PROPOSED APARTMENT BUILDING DEVELOPMENT
3080 NAVAN ROAD

OTTAWA, ONTARIO

Title:

PERMISSIBLE GRADE RAISE PLAN

Scale: 1:500	Date: 12/2022
Drawn by: NFRV	Report No.: PG6527-1
Checked by: DP	Dwg. No.: PG6527-2
Approved by: DP	Revision No.:

11x17

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