Geotechnical Investigation

Proposed Mixed-Use Development

2275 Mer Bleue Road
Ottawa, Ontario

Prepared for Seymour Pacific Developments (Ontario) Ltd.

Report PG6526-1 dated January 30, 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 multi-storey mixed-use development to be located at 2275 Mer Bleue Road in the City of Ottawa, Ontario (reference should be made to Figure 1 - Key Plan in Appendix 2 of this report).

The objective of the geotechnical investigation was 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 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

It is understood that the proposed development will consist of one multi-storey mixed-use building located above a single basement level of underground 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 investigation was carried out between December 12 to December 14, 2022 and consisted of advancing seven (7) boreholes to a maximum depth of 9.6 m below the ground surface throughout the subject site. A previous investigation was undertaken by Paterson in December of 2020. At that time, one borehole was advanced within the subject site to a maximum depth of 5.9 m.

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 test hole locations are shown on Drawing PG6526-1 - Test Hole Location Plan included in Appendix 2.

Boreholes were advanced using a track-mounted drill rig operated by a two-person crew. The drilling procedure consisted of augering to the required depths at the selected locations and sampling the overburden soils. All fieldwork was conducted under the full-time supervision of our personnel under the direction of a senior engineer from our geotechnical department.

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.

A Standard Penetration Test (SPT) was conducted in conjunction with the recovery of each 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 at regular depth intervals in cohesive soils using a vane apparatus.

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

Groundwater

Monitoring wells were installed at boreholes BH 4-22 and BH 5-22, and flexible standpipe piezometers were installed at all other boreholes to permit monitoring of the groundwater levels subsequent to the completion of the sampling program. The groundwater observations are discussed in Subsection 4.3 and presented in the Soil Profile and Test Data sheets in Appendix 1.

Monitoring Well Installation

Typical monitoring well construction details are described below:

- Slotted 50 mm diameter PVC screen at the base of each borehole.
- 51 mm diameter PVC riser pipe from the top of the screen to the ground surface.
- No.3 silica sand backfill within annular space around screen.
- Bentonite hole plug directly above PVC slotted screen.
- Clean backfill from top of bentonite plug to the ground surface.

Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for specific well construction details.

3.2 Field Survey

The test hole locations were selected by Paterson personnel in a manner to provide general coverage of the proposed development, taking into consideration existing site features. The ground surface elevations were referenced to a geodetic datum. The test hole locations and ground surface elevations at the test hole locations are presented on Drawing PG6526-1 - 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 (1) shrinkage limit testing, one (1) grain-size distribution analysis, and two (2) Atterberg Limit testing were completed on selected soil samples. Moisture content testing was complete on all recovered soil samples. The results are presented in Subsection 4.2 and on Grain Size Distribution and Hydrometer Testing presented 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 by Paterson. The sample was submitted to determine the concentration of sulphate and chloride, the resistivity, and the pH of the samples. The results are presented in Appendix 1 and are discussed further in Subsection 6.7.

4.0 Observations

4.1 Surface Conditions

The subject site consists of previous agricultural land which the southern half of the site is occupied by a layer of crushed stone fill connecting to further south by an access road to Mer Bleue Road. The eastern half of the granular pad is covered by an approximately 3 to 4 m high pile of soil fill. The northern half of the subject site consists of mature vegetation. The ground surface across the site is relatively flat with the exception of the above-noted fill piles and an approximately 500 mm deep ditch along the north and west site boundaries. The ground surface is approximately 400 to 600 mm lower than the adjacent roadways.

The site is bordered to the north by Brian Coburn Boulevard, to the east by residential dwellings, to the south by agricultural land, and to the west by Mer Bleue Road.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile encountered at the test hole locations consists of topsoil underlain by a deposit of silty clay. The topsoil was observed to range between 100 to 200 mm in thickness.

Fill, consisting of crushed stone with silty sand and variable amounts of clay, was encountered at BH 2-22 and BH 4-22 with a thickness of 460 and 300 mm, respectively. Further, an approximately 610 mm thick layer of fill consisting of brown silty clay was encountered below the aforementioned crushed stone fill layer at BH 4-22.

The clay deposit was observed to consist of a hard to stiff, weathered, brown silty clay crust which extended to a depth ranging between 2.3 m to 3.3 m below ground surface. The brown silty clay was observed to be underlain by a layer of unweathered, soft to firm, grey silty clay.

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

Atterberg Limit Testing Results

Atterberg limits testing, as well as associated moisture content testing, was completed on select silty clay samples. The results of the Atterberg limits test are presented in Table 1. The results of the moisture content test are presented on the Soil Profile and Test Data Sheet in Appendix 1.

The tested silty clay samples classify as inorganic clays of high plasticity (CH) in accordance with the Unified Soil Classification System.

Table 1 - Atterberg Limits Results						
Sample	**Depth (m)**	**LL (%)**	**PL (%)**	**PI (%)**	**w (%)**	**Classification**
BH 1-22	1.06	70	25	46	36.2	CH
BH 7-22	1.82	75	24	51	47.1	CH
Notes: LL: Liquid Limit; PL: Plastic Limit; PI: Plasticity Index; w: water content; CH: Inorganic Clay of High Plasticity						

Grain Size Distribution and Hydrometer Testing Results

Grain size distribution analysis was completed on one select recovered silty clay sample. The results of the grain size distribution analysis are presented in Table 2 and on the Grain Size Distribution sheets in Appendix 1.

Table 2 – Grain Size Distribution Results					
Sample	**Depth (m)**	**Gravel (%)**	**Sand (%)**	**Silt (%)**	**Clay (%)**
BH 2-22	3.35	0.0	0.0	33.0	67.0

Shrinkage Limits Testing Results

Linear shrinkage testing was completed on a sample recovered at a depth of 2.6 m from borehole BH 5-22 and yielded a shrinkage limit of 21.7 and a shrinkage ratio of 1.77.

Bedrock

A DCPT was complete at BH 4-22 and BH 5-22 with practical refusal encountered at a depth of 17.5 and 17.1 m below the ground surface, respectively. Based on available geological mapping, the bedrock in the area is part of the Billings formation, which consists of shale with an overburden drift thickness ranging between 25 to 50 m.

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 3 below.

Table 3 – Summary of Groundwater Levels

Borehole Number	Observation Method	Ground Surface Elevation (m)	Measured Groundwater Level		Date Recorded
			Depth (m)	Elevation (m)	
BH 1-22	Piezometer	87.68	3.22	84.46	December 19, 2022
BH 2-22	Piezometer	88.05	5.42	82.63	December 19, 2022
BH 3-22	Piezometer	87.53	0.40	87.13	December 19, 2022
BH 4-22	Monitoring Well	87.92	1.83	86.09	December 19, 2022
BH 5-22	Monitoring Well	87.82	1.07	86.75	December 19, 2022
BH 6-22	Piezometer	87.70	0.60	87.10	December 19, 2022
BH 7-22	Piezometer	87.59	2.05	85.54	December 19, 2022
BH 1	Piezometer	87.55	3.68	83.87	December 17, 2020

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

Long-term groundwater levels can also be estimated based on the observed colour and consistency of the recovered soil samples. Based on these observations, the long-term groundwater table can be expected 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.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered suitable for the proposed development. The proposed multi-storey mixed-use 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 significant organic materials, should be stripped from under any buildings and other settlement sensitive structures. 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.

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 foundations 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

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 **85 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 **125 kPa**. For this case, the modulus of subgrade reaction was calculated to be **3.4 MPa/m** for a contact pressure of **85 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.

Deep Foundation - End Bearing Piles

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 4. 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 4 - 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.

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.

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 silty clay when a plane extending down and out from the bottom edge of the footing at a minimum of 1H:1V passes only through in situ soil or engineered fill.

Permissible Grade Raise Recommendations

Our current permissible grade raise recommendations for the proposed development are presented on Drawing PG6526-2 Permissible Grade Raise Plan in Appendix 2.

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.

A post-development groundwater lowering of 0.5 m was assumed for our calculations.

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 PG6526-1 - Test Hole Location Plan, attached to the present report. Paterson field personnel placed 24 horizontal 2.4 Hz. geophones mounted to the surface by means of two 75 mm ground spikes attached to the geophone land case. The geophones were spaced at 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 25, 4.5 and 3.0 m away from the first geophone and 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 **154 m/s**, while the bedrock shear wave velocity is **1,858 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 16 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:

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

$$V_{s30=} \frac{30\ m}{\left(\frac{16\ m}{154\ m/s} + \frac{14\ m}{1858\ m/s}\right)}$$

$$V_{s30=}\ 269\ \ m/s$$

Based on the results of the shear wave velocity testing, the average shear wave velocity V_{s30}, is **269 m/s**. Therefore, a **Site Class D** is applicable for the design of the proposed building founded by a raft slab on a firm, silty clay bearing surface or end-bearing piles extended to the bedrock surface, 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 structures. However, the conditions should be designed by assuming the retaining soil consists of a material with an angle of internal friction of 30 degrees and a dry 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 both the static earth pressure component (P_o) and the seismic component ($\triangle P_{AE}$).

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 OBC 2012. Note that the vertical seismic coefficient is assumed to be zero.

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

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

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

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

5.7 Pavement Design

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 5 - 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 6 - 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 500 mm below the top of the podium slab a minimum of 1.5 m from the face of the foundation wall prior to providing the recommended taper.

Should the proposed podium deck be specified 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 will be dependent on the grade of insulation considered for this project and should be reassessed by Paterson once pertinent design details have been prepared.

The higher grades of insulation have more resistance to deformation under wheel-loading and require less granular cover to avoid being crushed by vehicular loading. 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 7 and 8.

Table 7 - 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 8 - 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. 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.

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 8.

Table 8 - 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

Groundwater Control for Building Construction

It is anticipated that groundwater infiltration into the excavations through the overburden materials should be low to moderate and controllable using open sumps. The contractor should be prepared to direct water away from all bearing surfaces and subgrades, regardless of the source, to prevent disturbance to the founding medium.

Permit to Take Water

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

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

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 from an adjacent site 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 non-aggressive to slightly aggressive corrosive environment.

6.8 Landscaping Considerations

Tree Planting Considerations

In accordance with the City of Ottawa Tree Planting in Sensitive Marine Clay Soils (2017 Guidelines), Paterson completed a soils review of the site to determine applicable tree planting setbacks. Atterberg limits testing was completed for the recovered silty clay samples at selected locations throughout the subject site. The results of our testing are presented in Table 1 in Subsection 4.2.

Based on the results of the Atterberg limit testing mentioned above, the plasticity index was found to be greater than 40% in all the tested clay samples. Based on this, the clay is considered to be a clay of high potential for soil volume change.

Based on this, the setbacks would consist of 7.5 m for small (mature height up to 7.5 m) and medium size trees (mature tree height 7.5 to 14 m), provided the conditions noted below are met at the time of landscape design:

- ❑ A small tree must be provided with a minimum of 25 m^3 of available soils volume while a medium tree must be provided with a minimum of 30 m^3 of available soil volume, as determined by the Landscape Architect. The developer is to ensure that the soil is generally un-compacted when backfilling in street tree planting locations.

- ❑ The tree species must be small (mature tree height up to 7.5 m) to medium size (mature tree height 7.5 m to 14 m) as confirmed by the Landscape Architect.

- ❑ Grading surrounding the tree must promote drainage to the tree root zone (in such a manner as not to be detrimental to the tree), as noted on the subdivision Grading Plan.

It is well documented in the literature, and is our experience, that fast-growing trees located near buildings founded on cohesive soils that shrink on drying can result in long-term differential settlements of the structures. Tree varieties that have the most pronounced effect on foundations are seen to consist of poplars, willows and some maples (i.e., Manitoba Maples) and, as such, they should not be considered in the landscaping design.

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 than 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.

January 30, 2023

LICENSED PROFESSIONAL ENGINEER
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PROVINCE OF ONTARIO

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David J. Gilbert, P.Eng.

Report Distribution:

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APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

GRAIN SIZE ANALYSIS RESULTS

ANALYTICAL TESTING RESULTS

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9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Mixed-Use Development - 2275 Mer Bleue Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

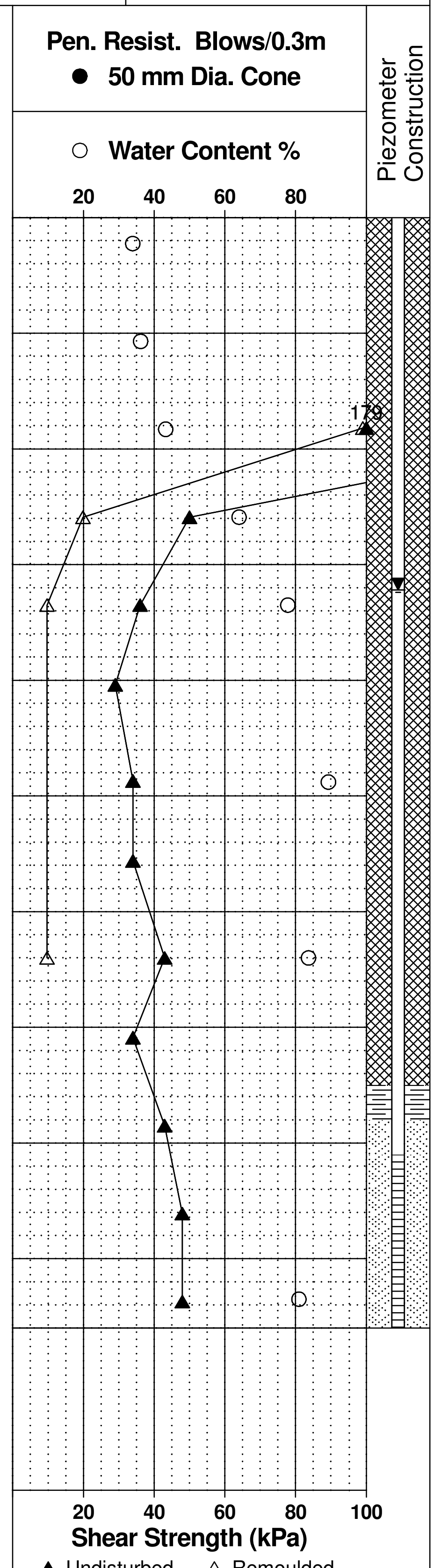
BORINGS BY Track-Mount Power Auger

DATE December 12, 2022

FILE NO. PG6526

HOLE NO. BH 1-22

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	87.68
TOPSOIL 0.10	AU	1				
Hard to stiff, brown **SILTY CLAY**	SS	2	100	9	1	86.68
- firm and grey by 2.4m depth	SS	3	100	P	2	85.68
	SS	4		P		
	SS	5		P	3	84.68
					4	83.68
	SS	6		P	5	82.68
					6	81.68
	SS	7		P	7	80.68
					8	79.68
					9	78.68
9.60	G	8				
End of Borehole						
(GWL @ 3.22m - Dec. 19, 2022)						



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Prop. Mixed-Use Development - 2275 Mer Bleue Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE December 12, 2022

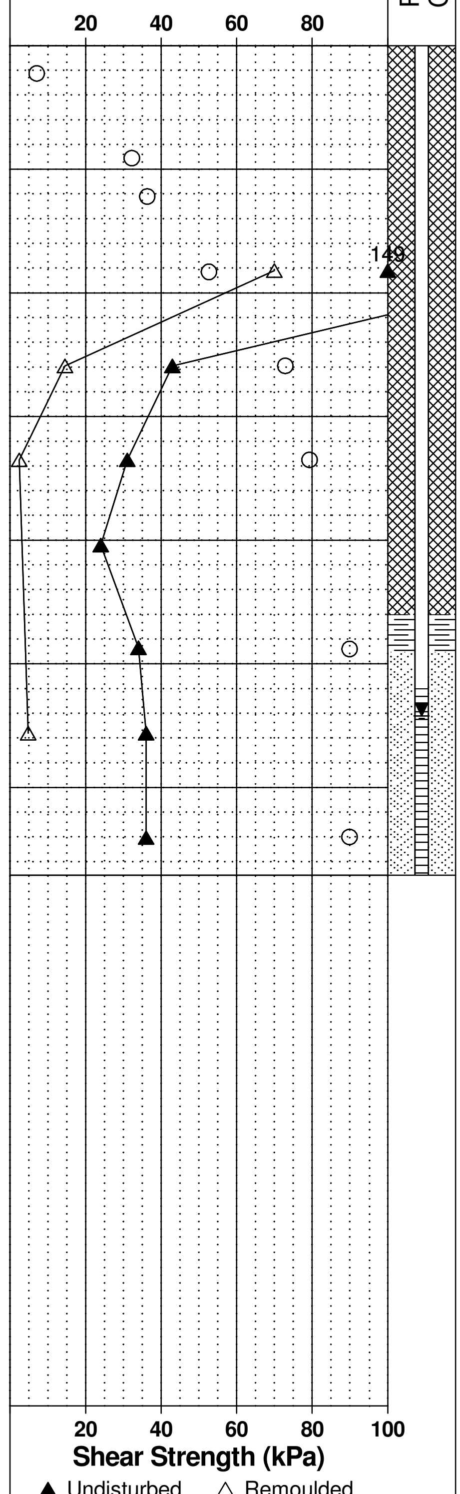
FILE NO. PG6526

HOLE NO. BH 2-22

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	88.05
FILL: Crushed stone, some sand, trace clay 0.46		AU	1				
		SS	2	62	7	1	87.05
Hard to stiff, brown **SILTY CLAY**		SS	3	100	P	2	86.05
- firm by 2.3m depth		SS	4	100	P		
- grey by 3.0m depth		SS	5	100	P	3	85.05
						4	84.05
		SS	6	100	P	5	83.05
6.71		SS	7		P	6	82.05
End of Borehole							
(GWL @ 5.42m - Dec. 19, 2022)							

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

○ Water Content %



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Prop. Mixed-Use Development - 2275 Mer Bleue Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

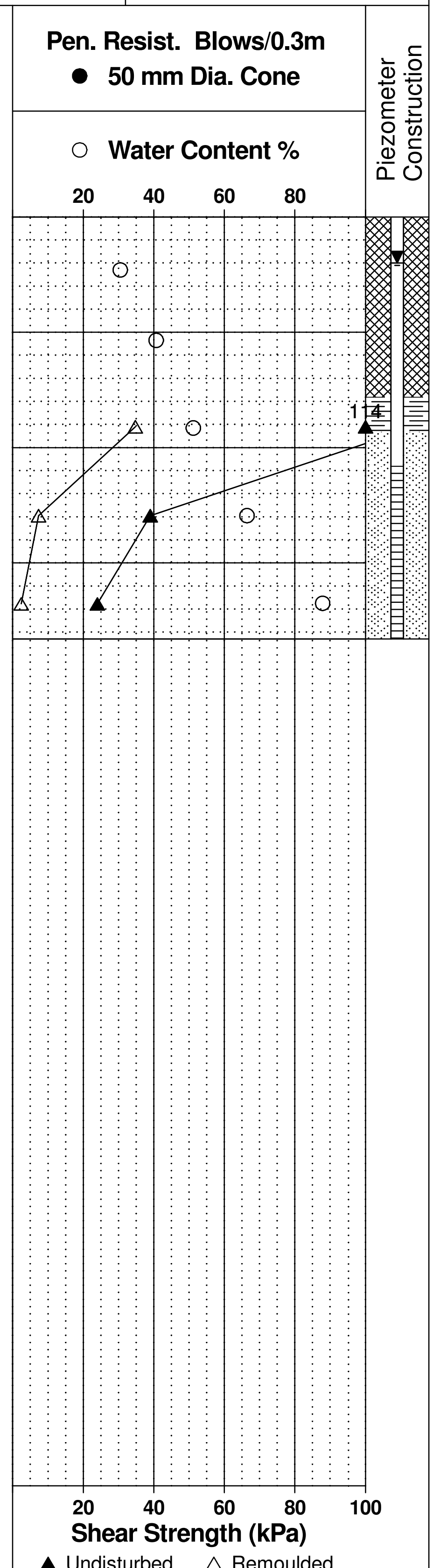
BORINGS BY Track-Mount Power Auger

DATE December 12, 2022

FILE NO. PG6526

HOLE NO. BH 3-22

SOIL DESCRIPTION	TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	87.53
Hard to firm, brown **SILTY CLAY**	AU	1				
	SS	2	75	6	1	86.53
- soft to firm and grey by 2.5m depth	SS	3	100	P	2	85.53
	SS	4		P	3	84.53
3.66	SS	5		P		
End of Borehole						
(GWL @ 0.40m - Dec. 19, 2022)						



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Mixed-Use Development - 2275 Mer Bleue Road
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DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE December 13, 2022

FILE NO. PG6526

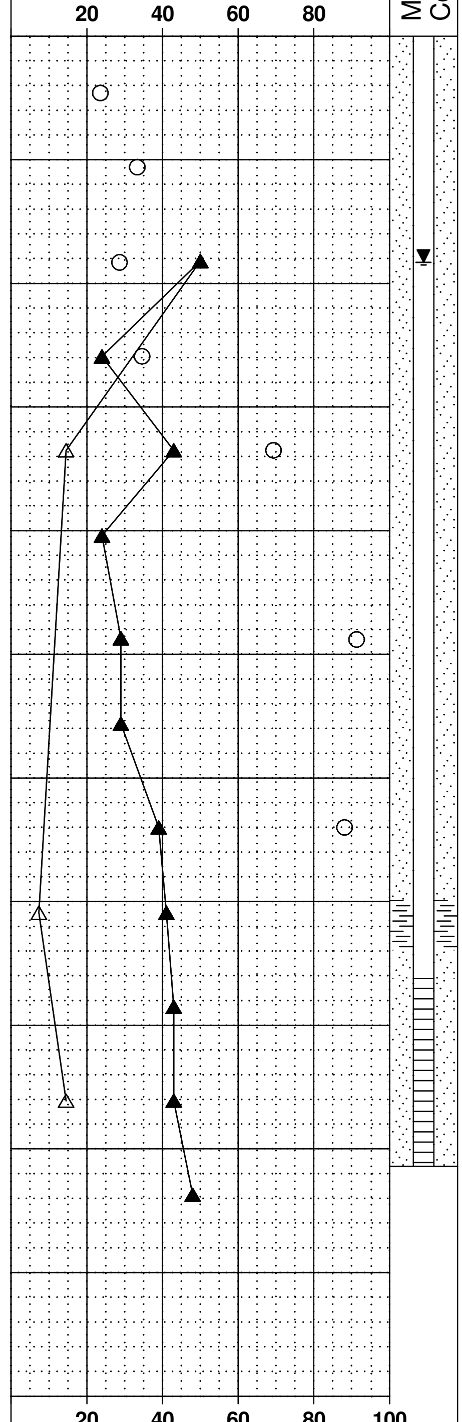
HOLE NO. BH 4-22

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	87.92
FILL: Crushed stone with silty sand 0.30							
FILL: Brown silty clay, topsoil, trace organics 0.91		AU	1				
- trace gravel by 0.7m depth		SS	2	17	8	1	86.92
Very stiff to stiff, brown **SILTY CLAY**		SS	3	8	P	2	85.92
- soft by 2.3m depth		SS	4	8	P		
- soft to firm and grey by 2.9m depth		SS	5	50	P	3	84.92
						4	83.92
		SS	6	100	P	5	82.92
						6	81.92
		SS	7		P		
						7	80.92
						8	79.92
9.60						9	78.92
Dynamic Cone Penetration Test commenced at 9.60m depth. Cone pushed to 17.0m depth.						10	77.92
						11	76.92

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

○ Water Content %

Monitoring Well Construction



Shear Strength (kPa)
▲ Undisturbed △ Remoulded

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Mixed-Use Development - 2275 Mer Bleue Road
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REMARKS

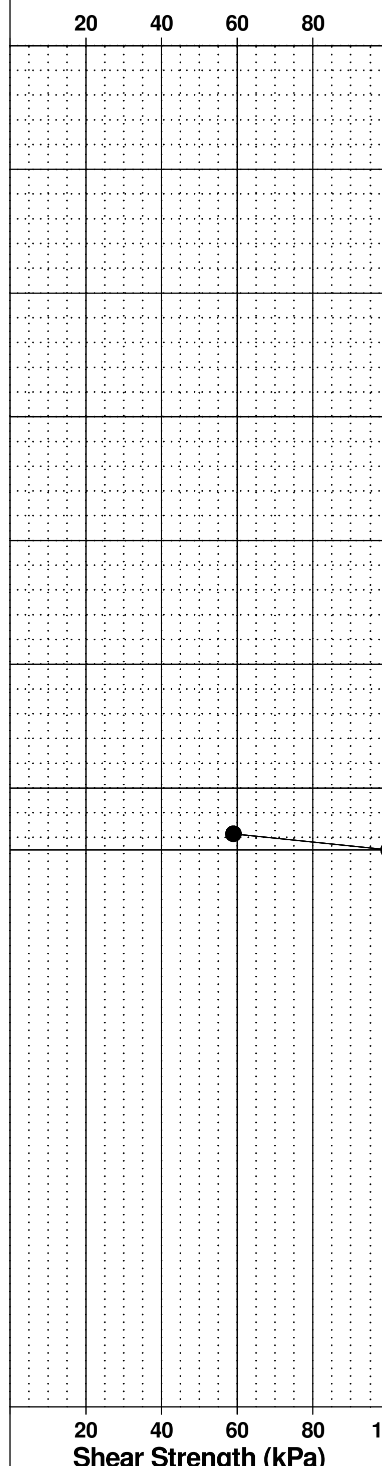
BORINGS BY Track-Mount Power Auger

DATE December 13, 2022

FILE NO. PG6526

HOLE NO. BH 4-22

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									
Dynamic Cone Penetration Test commenced at 9.60m depth. Cone pushed to 17.0m depth.						11	76.92		
						12	75.92		
						13	74.92		
						14	73.92		
						15	72.92		
						16	71.92		
						17	70.92		
17.50									
End of Borehole Practical DCPT refusal at 17.50m depth. (GWL @ 1.83m - Dec. 19, 2022)									



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Mixed-Use Development - 2275 Mer Bleue Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE December 13, 2022

FILE NO. PG6526

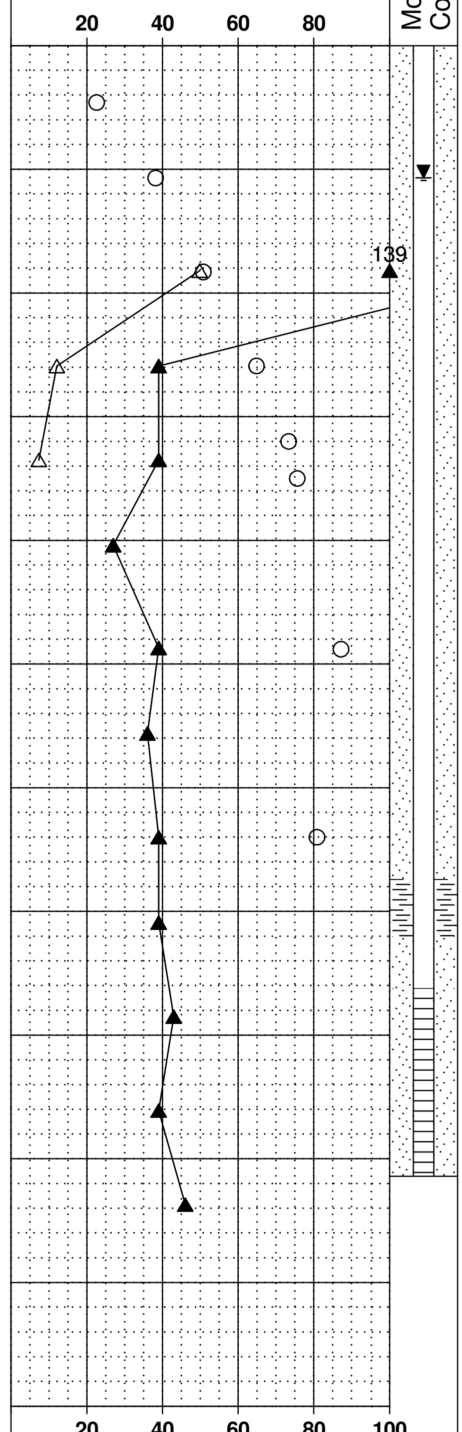
HOLE NO. BH 5-22

SOIL DESCRIPTION	TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	87.82
TOPSOIL 0.20	AU	1				
Very stiff to stiff, brown **SILTY CLAY**	SS	2	75	9	1	86.82
- firm by 2.3m depth	SS	3	100	P	2	85.82
- grey by 3.3m depth	SS	4	100	P	3	84.82
	SS	5	100	P	4	83.82
	SS	6		P	5	82.82
	SS	7	42	P	6	81.82
					7	80.82
					8	79.82
9.60					9	78.82
Dynamic Cone Penetration Test commenced at 9.60m depth. Cone pushed to 16.15m depth.					10	77.82
					11	76.82

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

○ Water Content %

Monitoring Well Construction



Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Mixed-Use Development - 2275 Mer Bleue Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

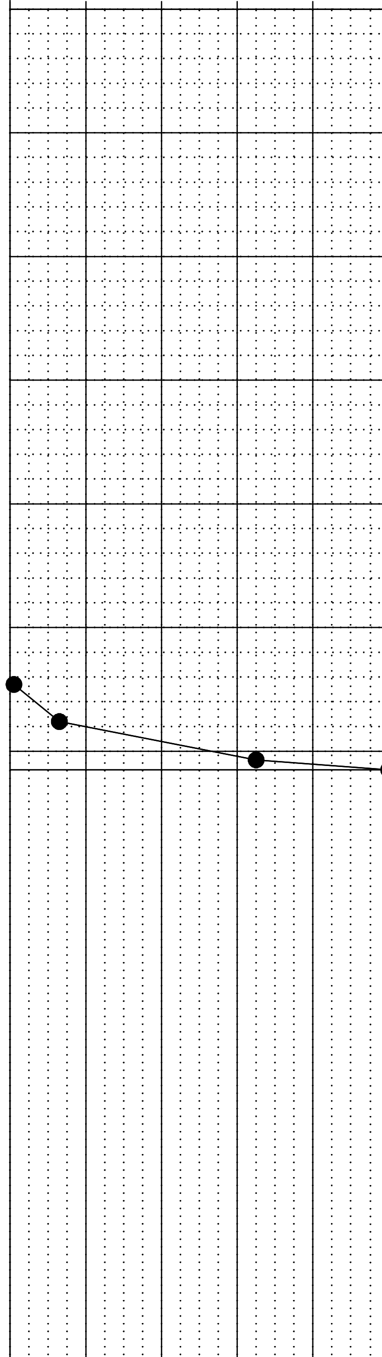
BORINGS BY Track-Mount Power Auger

DATE December 13, 2022

FILE NO. PG6526

HOLE NO. BH 5-22

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content %	Monitoring Well Construction
GROUND SURFACE								20 40 60 80	
Dynamic Cone Penetration Test commenced at 9.60m depth. Cone pushed to 16.15m depth.						11	76.82		
						12	75.82		
						13	74.82		
						14	73.82		
						15	72.82		
						16	71.82		
17.15						17	70.82		
End of Borehole Practical DCPT refusal at 17.15m depth (GWL @ 1.07m - Dec. 19, 2022)									



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

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Mixed-Use Development - 2275 Mer Bleue Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

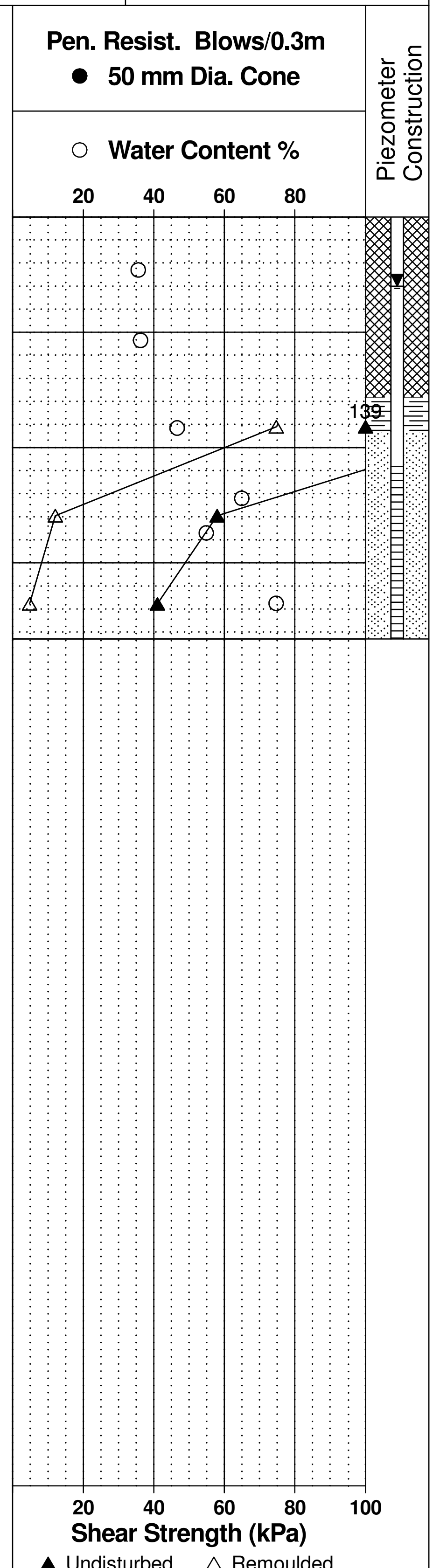
BORINGS BY Track-Mount Power Auger

DATE December 14, 2022

FILE NO. PG6526

HOLE NO. BH 6-22

SOIL DESCRIPTION	STRATA PLOT	TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	87.70
TOPSOIL 0.15							
		AU	1				
		SS	2	100	10	1	86.70
Very stiff to stiff, brown **SILTY CLAY**		SS	3	100	P	2	85.70
- firm and grey by 2.7m depth		SS	4	100	P	3	84.70
3.66		SS	5	100	P		
End of Borehole							
(GWL @ 0.60m - Dec. 19, 2022)							



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Mixed-Use Development - 2275 Mer Bleue Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

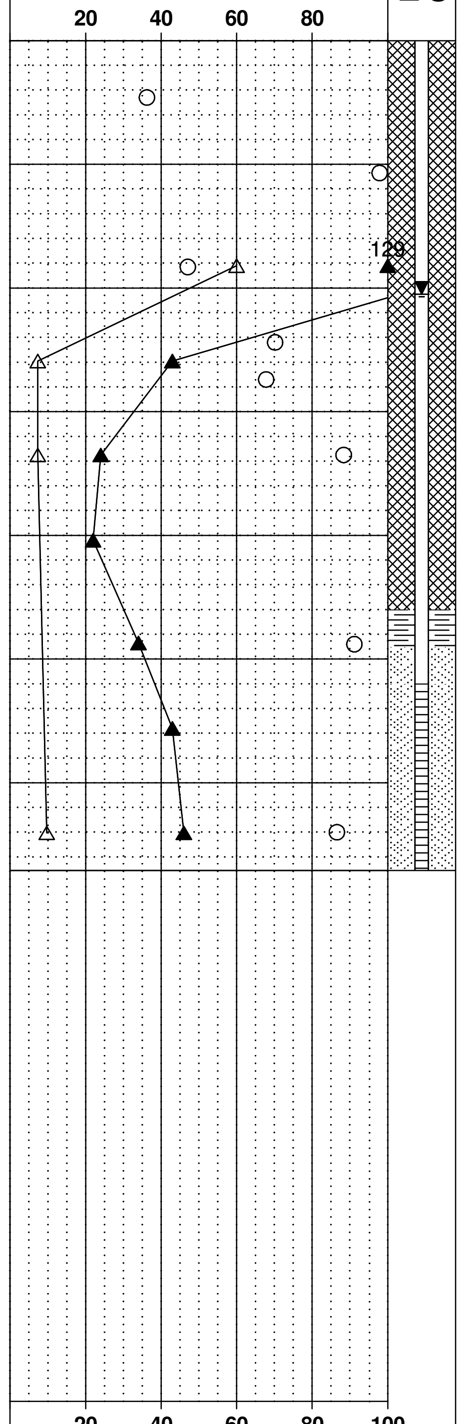
DATE December 14, 2022

FILE NO. PG6526

HOLE NO. BH 7-22

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	87.59
TOPSOIL 0.13							
Very stiff to stiff, brown **SILTY CLAY**		AU	1				
- firm by 2.3m depth		SS	2	100	9	1	86.59
- soft to firm and grey by 2.9m depth		SS	3	100	P	2	85.59
		SS	4	100	P	3	84.59
		SS	5	100	P	4	83.59
		SS	6	58	P	5	82.59
6.71		SS	7		P	6	81.59
End of Borehole							
(GWL @ 2.05m - Dec. 19, 2022)							

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



Shear Strength (kPa)
▲ Undisturbed △ Remoulded

Piezometer Construction

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Residential Development - 2275 Mer Bleue Rd.
Ottawa, Ontario

DATUM Geodetic

REMARKS

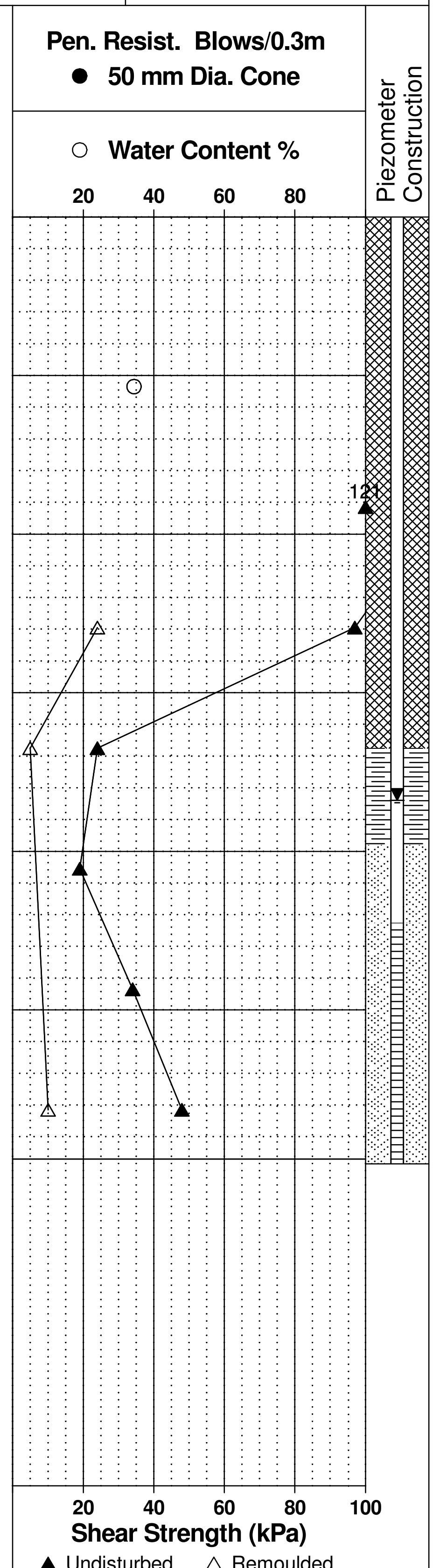
BORINGS BY CME-55 Low Clearance Drill

DATE 2020 December 11

FILE NO. PG5521

HOLE NO. BH 1

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	87.55
TOPSOIL 0.15		AU	1				
Very stiff to stiff, brown **SILTY CLAY** -Soft to firm and grey by 3.0 m depth -Firm to stiff by 4.5 m depth		SS	2	58	10	1	86.55
						2	85.55
						3	84.55
						4	83.55
						5	82.55
5.94							
End of Borehole (GWL @ 3.68 m December 17, 2020)							



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

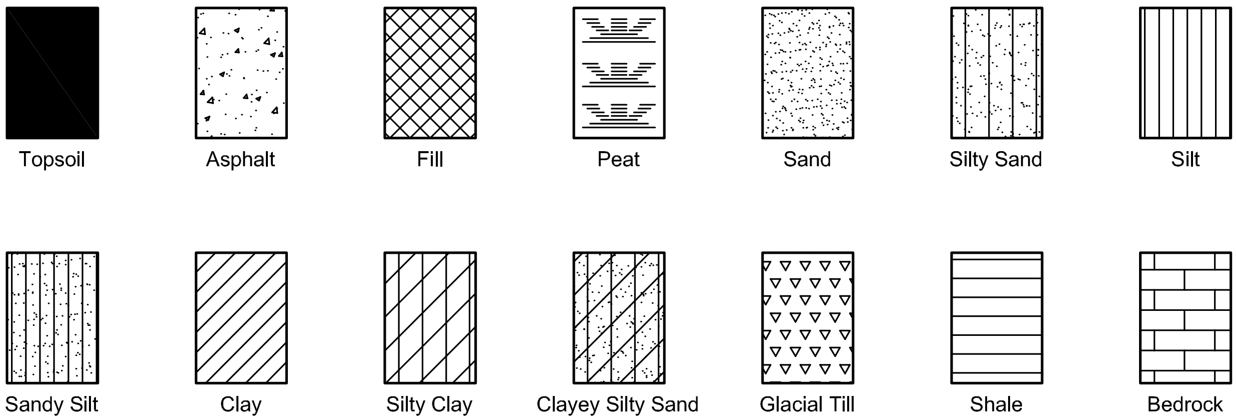
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void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

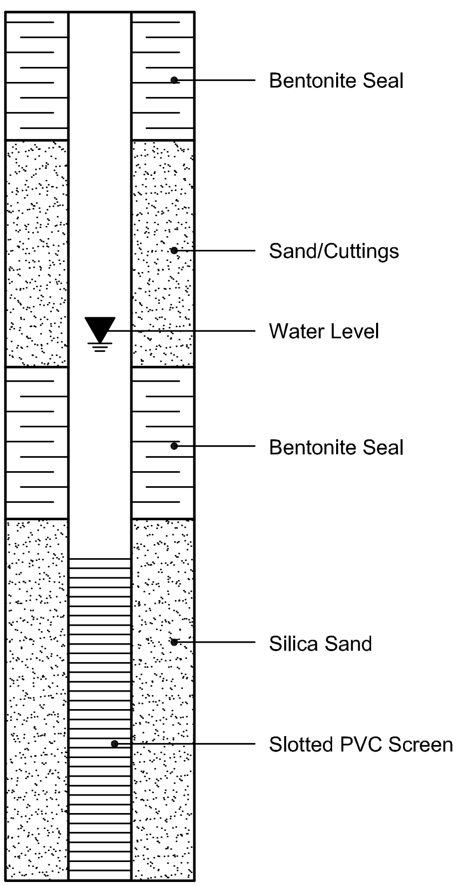
SYMBOLS AND TERMS (continued)

STRATA PLOT

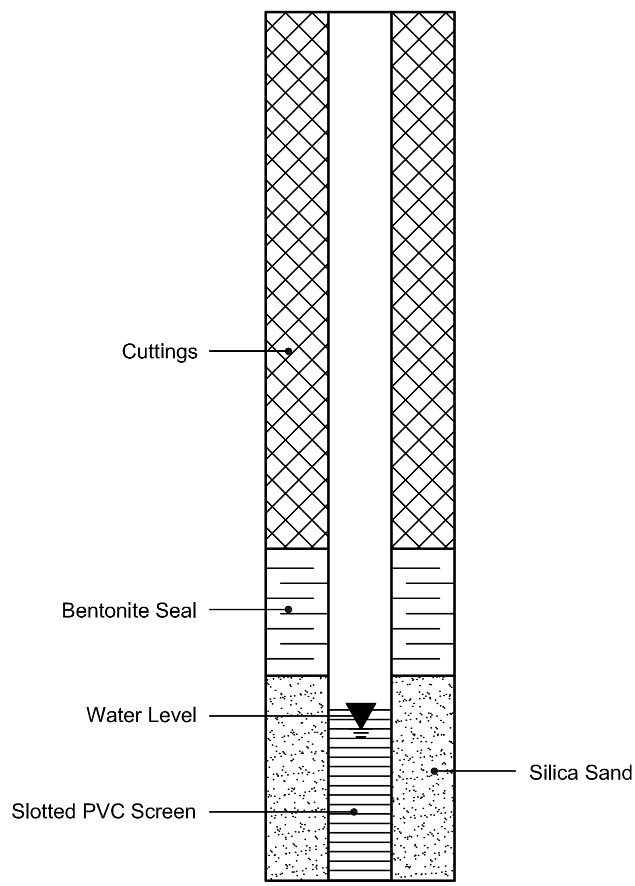


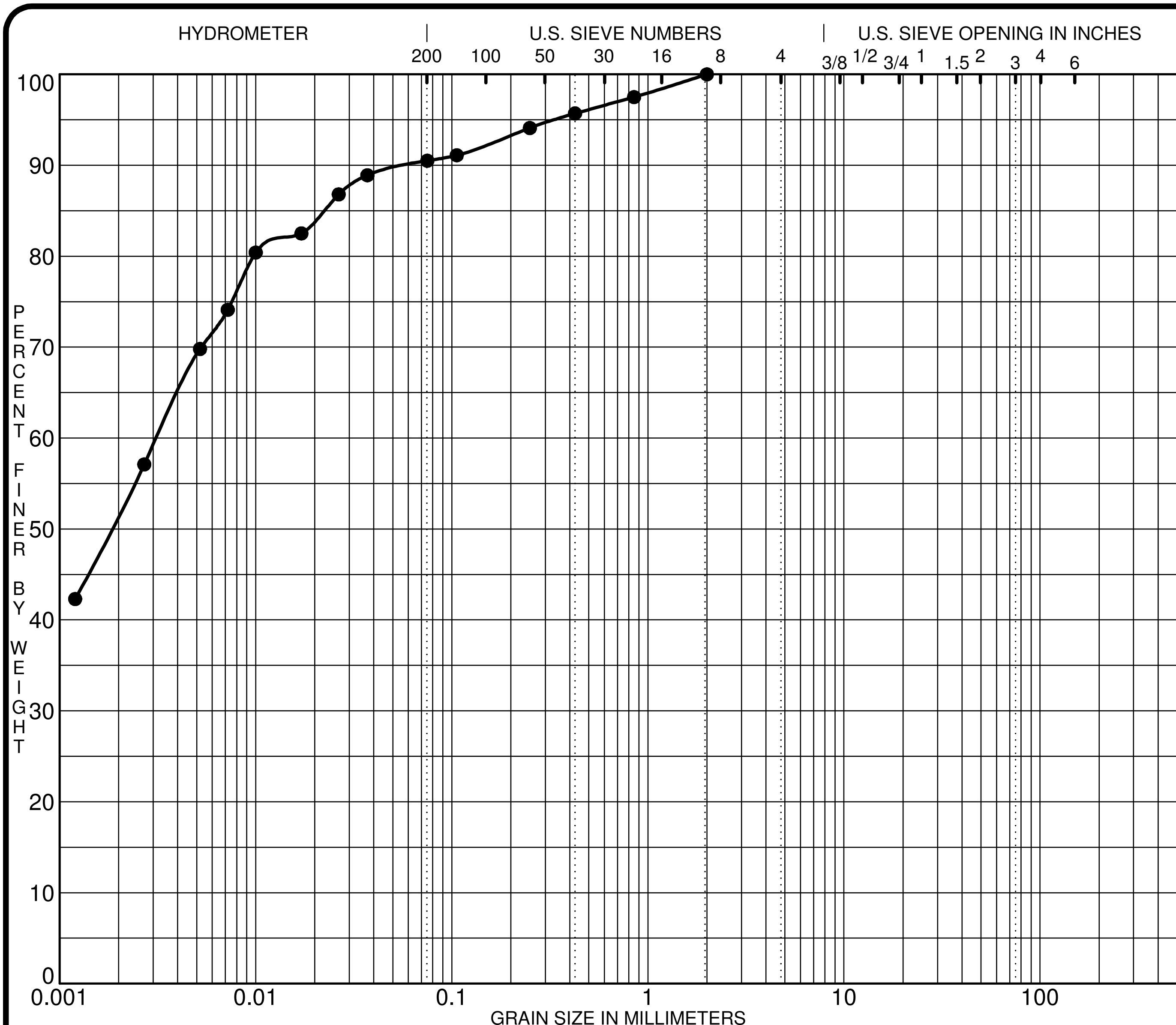
MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION





SILT OR CLAY	SAND			GRAVEL		COBBLES
	fine	medium	coarse	fine	coarse	

	Specimen Identification		Classification	MC%	LL	PL	PI	Cc	Cu
●	BH 2-22	SS3							
⧗									
▲									
★									

	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	BH 2-22	SS3	2.00	0.00			0.0	9.5	90.5	
⧗										
▲										
★										

CLIENT **RNA Invest Inc.** FILE NO. **PG6504**

PROJECT **Geotechnical Investigation - Proposed Residential Building - 1267 Summerville Ave.** DATE **9 Dec 22**

patersongroup **Consulting Engineers**

9 Auriga Drive, Ottawa, Ontario K2E 7T9

GRAIN SIZE DISTRIBUTION

PARACEL LABORATORIES LTD.

Certificate of Analysis

Order #: 2251376

Report Date: 21-Dec-2022

Client: Paterson Group Consulting Engineers

Order Date: 15-Dec-2022

Client PO: 56476

Project Description: PG6526

	Client ID:	BH4-22 SS5	-	-	-	-	-
	Sample Date:	14-Dec-22 09:00	-	-	-		
	Sample ID:	2251376-01	-	-	-		
	Matrix:	Soil	-	-	-		
	MDL/Units						
Physical Characteristics							
% Solids	0.1 % by Wt.	59.5	-	-	-	-	-
General Inorganics							
pH	0.05 pH Units	7.72	-	-	-	-	-
Resistivity	0.1 Ohm.m	25.3	-	-	-	-	-
Anions							
Chloride	5 ug/g	31	-	-	-	-	-
Sulphate	5 ug/g	<5	-	-	-	-	-

APPENDIX 2

FIGURE 1 – KEY PLAN

FIGURES 2 & 3 – SEISMIC SHEAR WAVE VELOCITY PROFILES

DRAWING PG6526-1 – TEST HOLE LOCATION PLAN

DRAWING PG6526-2 – PERMISSIBLE GRADE RAISE PLAN

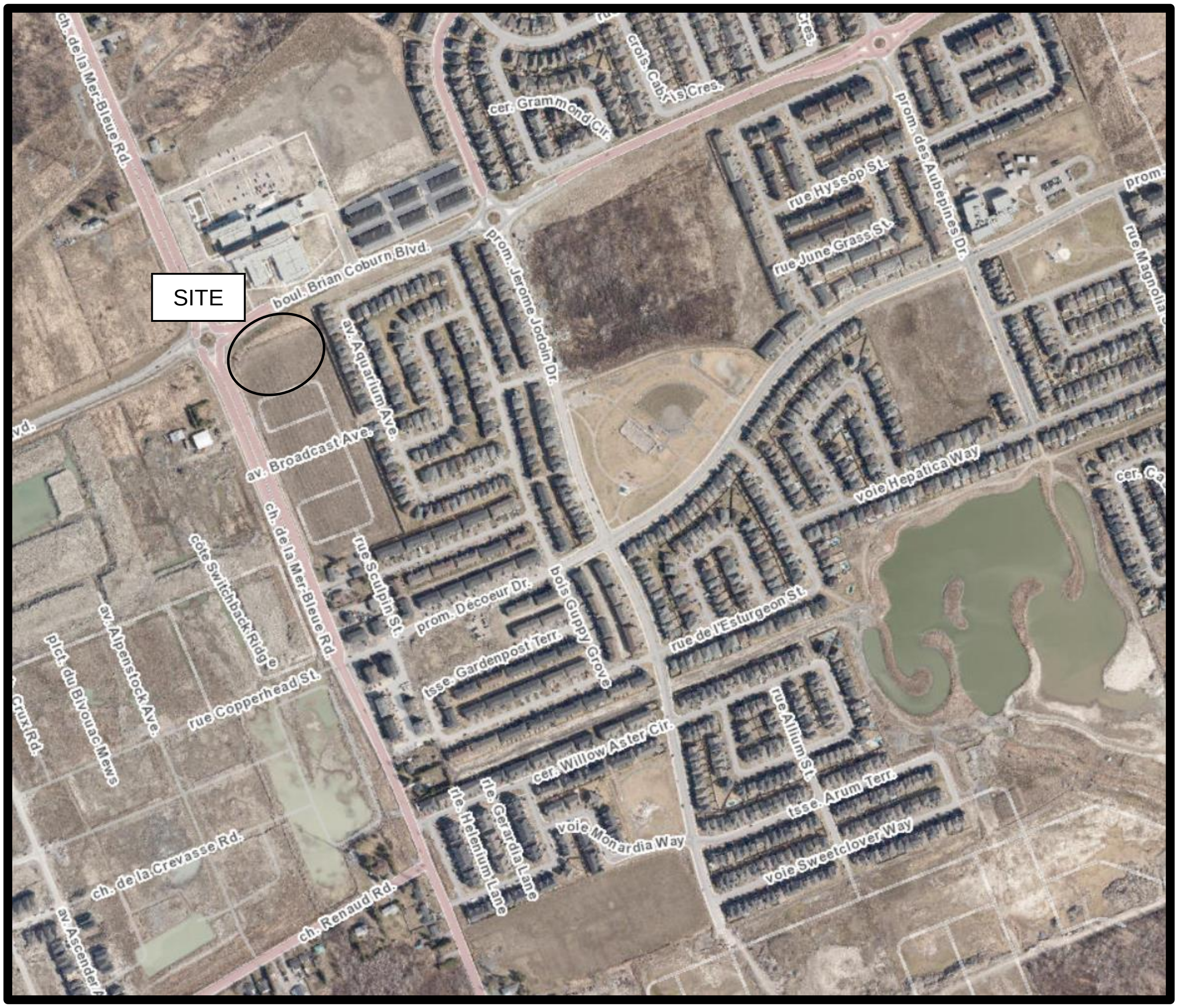


FIGURE 1

KEY PLAN

PATERSON GROUP

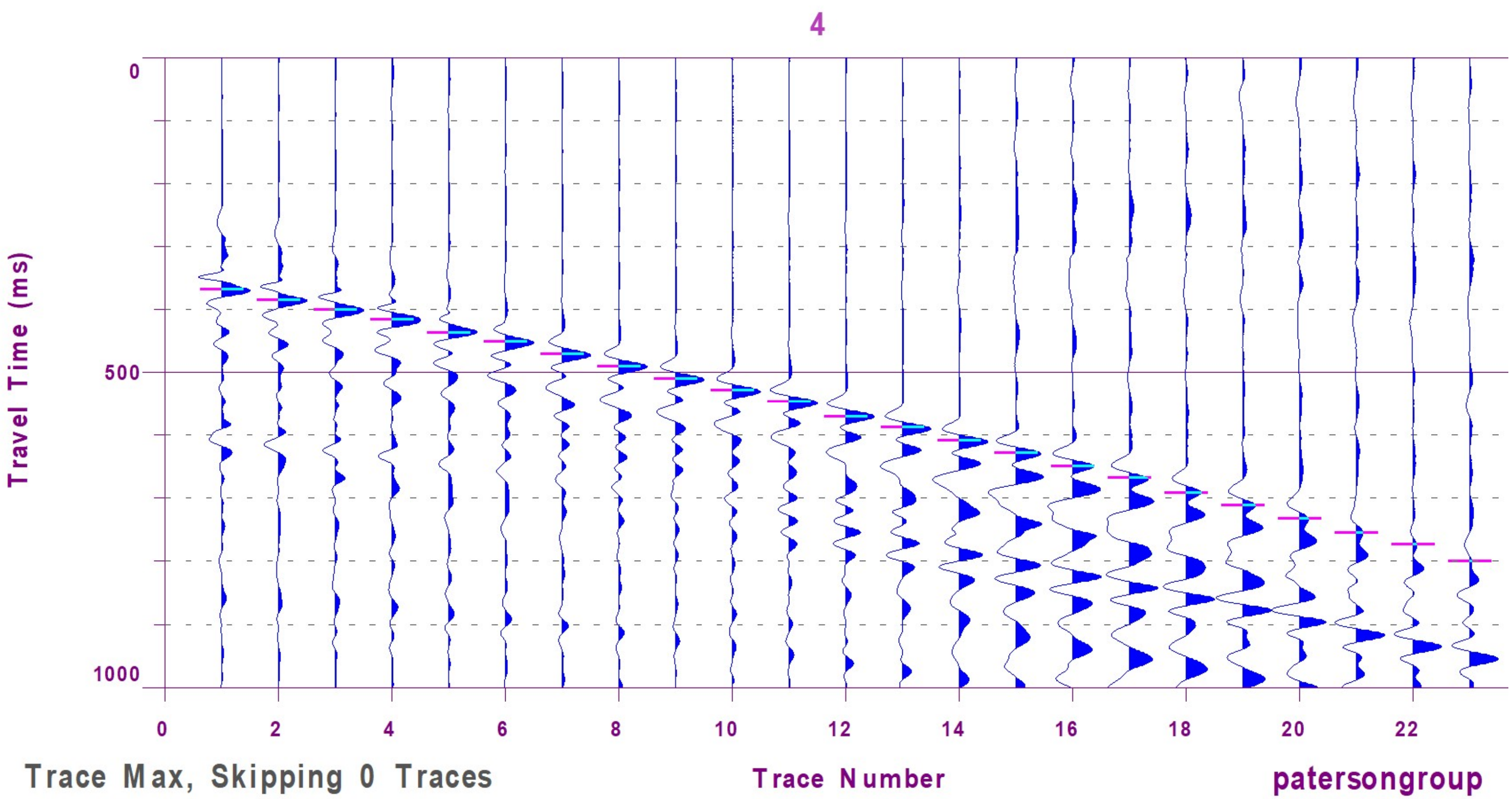


Figure 2 – Shear Wave Velocity Profile at Shot Location -25 m

PATERSON GROUP

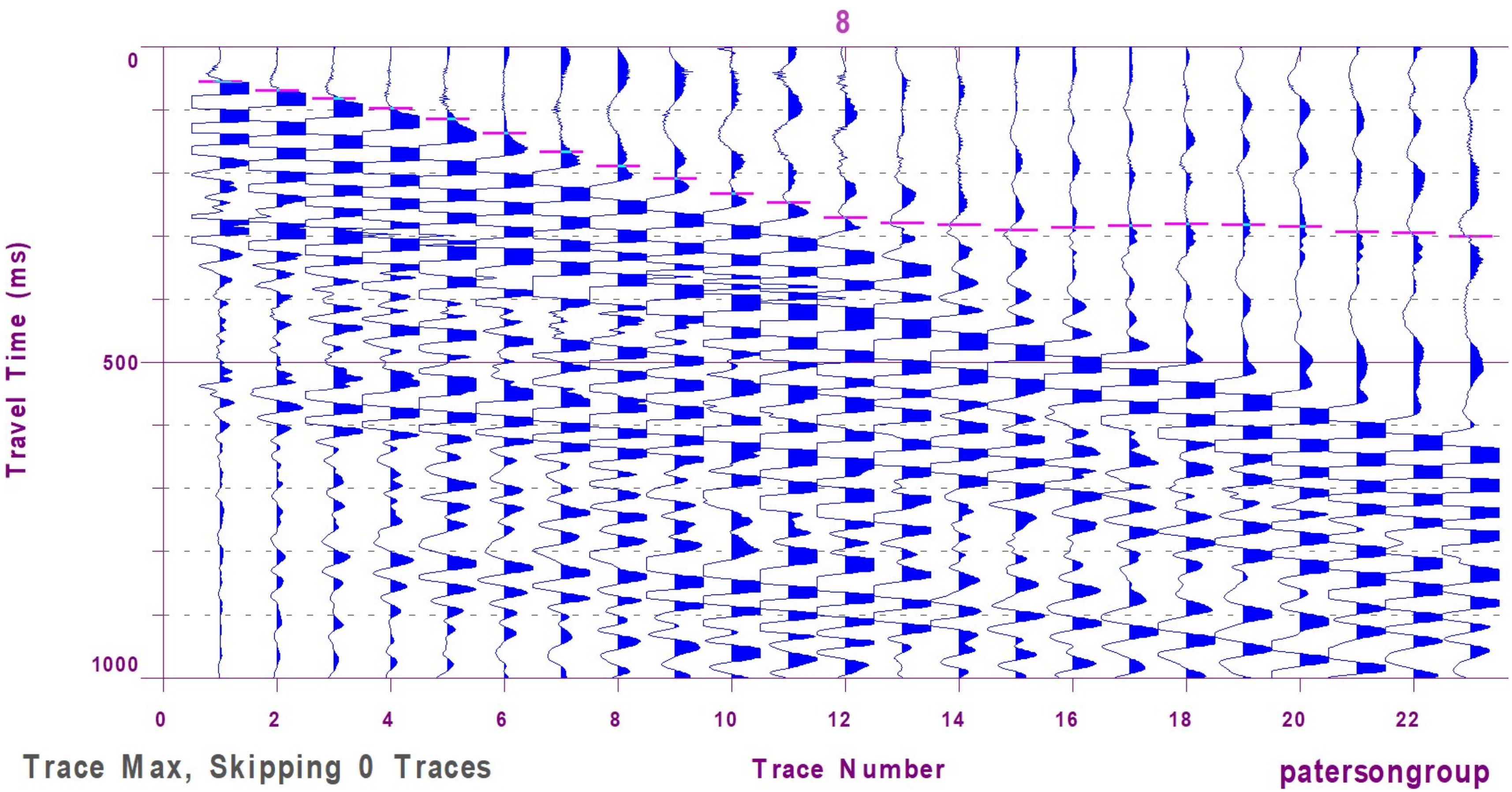


Figure 3 – Shear Wave Velocity Profile at Shot Location -4.5 m

PATERSON GROUP

LEGEND:

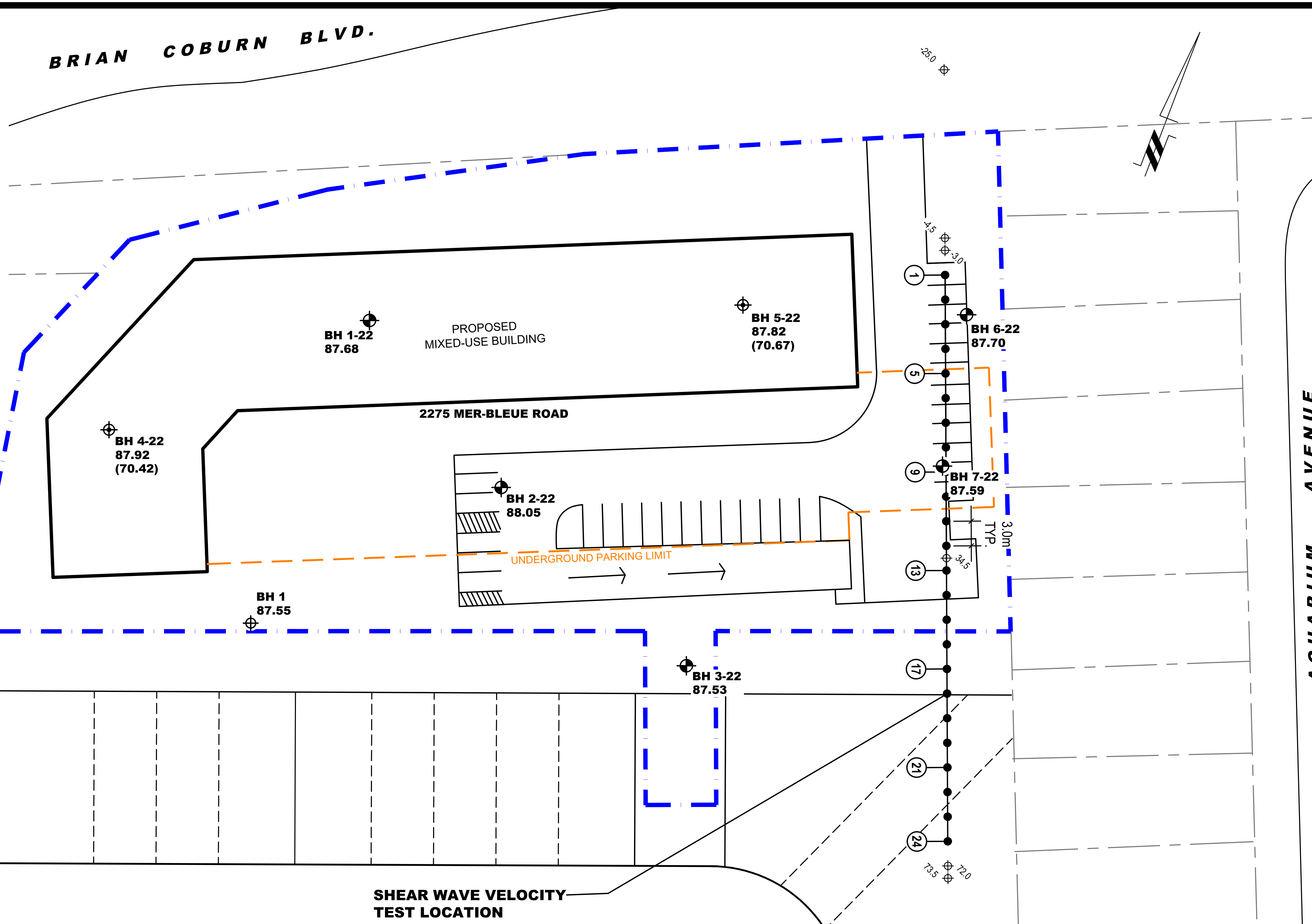
- BOREHOLE LOCATION
- BOREHOLE WITH MONITORING WELL LOCATION
- BOREHOLE LOCATION (PATERSON GROUP REPORT; PG5521-1, 2020)
- GEOPHONE LOCATIONS
- 9 GEOPHONE NUMBER
- 34.5 SHOT LOCATION
- 87.92 GROUND SURFACE ELEVATION (m)
- (70.42) PRACTICAL REFUSAL TO DCPT ELEV. (m)

CONCEPTUAL PLAN PROVIDED BY SEYMOUR PACIFIC DEVELOPMENTS LTD.

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

SCALE: 1:500

0 5 10 15 20 25 30m



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

NO.	REVISIONS	DATE	INITIAL

SEYMOUR PACIFIC DEVELOPMENTS (ONTARIO) LTD.

GEOTECHNICAL INVESTIGATION
PROPOSED MIXED-USE DEVELOPMENT
2275 MER BLEUE ROAD

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:500	Date: 01/2023
Drawn by: JM	Report No.: PG6526-1
Checked by: NS	Dwg. No.: PG6526-1
Approved by: DP	Revision No.:

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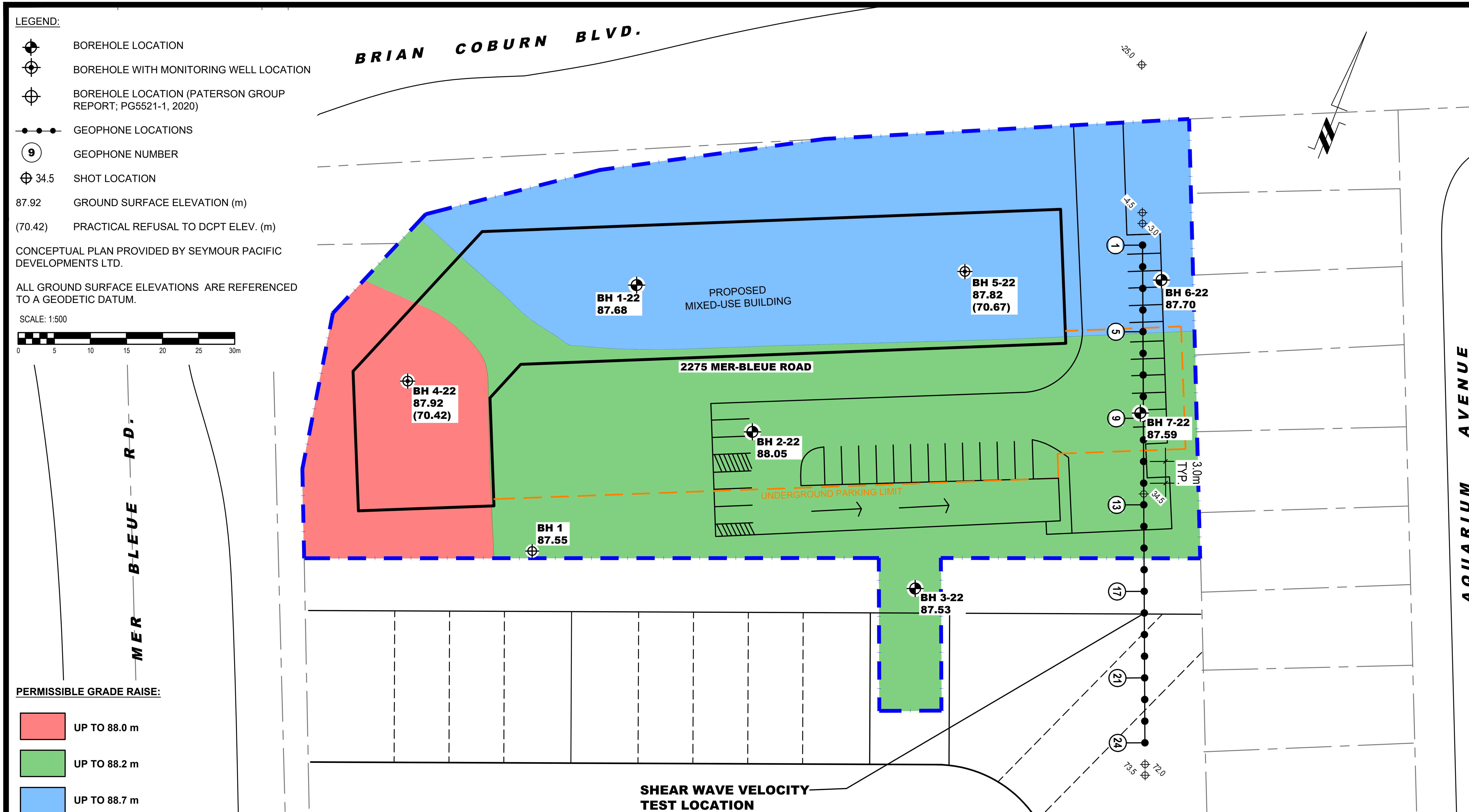
11x17



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OTTAWA, ON
K2E 7T9
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SEYMOUR PACIFIC DEVELOPMENTS (ONTARIO) LTD.

GEOTECHNICAL INVESTIGATION
PROPOSED MIXED-USE DEVELOPMENT
2275 MER BLEUE ROAD

OTTAWA, ONTARIO

Title:

PERMISSIBLE GRADE RAISE PLAN

Scale: 1:500	Date: 01/2023
Drawn by: JM	Report No.: PG6526-1
Checked by: NS	Dwg. No.: PG6526-2
Approved by: DP	Revision No.:

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