PATERSON GROUP

Geotechnical Investigation

Proposed Residential Development

3040 & 3044 Innes Road
Ottawa, Ontario

Prepared for Landric Homes

Report PG5763-1 Revision 1 dated October 18, 2022

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Landric Homes to conduct a geotechnical investigation for the proposed low-rise residential development to be located at 3040 and 3044 Innes Road in the City of Ottawa (refer 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 boreholes.

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

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

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

2.0 Proposed Development

Based on correspondence with Landric Homes, it is understood that the proposed development will consist of a low-rise residential building with one level of underground parking. Associated access lanes, parking areas, walkways, and landscaped areas are also anticipated as part of the development. It is further anticipated that the proposed building(s) will be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the geotechnical investigation was carried out on May 4 and 11, 2021 and consisted of advancing a total of 5 test holes to a maximum depth of 6.7 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 test hole locations are shown on Drawing PG5763-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were completed using a low-clearance, rubber track-mounted drill rig operated by a two-person crew. The hand auger hole was completed using a steel hand auger. All fieldwork was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer. The drilling procedure consisted of advancing each test hole to the required depths at the selected locations and sampling the overburden.

Sampling and In Situ Testing

The soil samples were collected from the boreholes using a 50 mm diameter split-spoon (SS) sampler or from the drill auger and hand auger flights. The samples were initially classified on site, placed in sealed plastic bags, and transported to our laboratory. The depths at which the drill auger, and split-spoon and hand auger samples were recovered from the boreholes are shown as AU, SS and HA, 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 at regular depth intervals in cohesive soils using a vane apparatus.

The thickness of the overburden was evaluated by a dynamic cone penetration test (DCPT) at borehole BH 2-21. The DCPT consists of driving a steel drill rod, equipped with a 50 mm diameter cone at its 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

Flexible polyethylene standpipes were installed in all boreholes to permit monitoring of the groundwater levels subsequent to the completion of the sampling program.

3.2 Field Survey

The test hole locations were selected by Paterson to provide general coverage of the proposed development, taking into consideration the existing site features and underground utilities. The test hole locations and ground surface elevation at each test hole location were surveyed by Paterson using a handheld GPS and referenced to a geodetic datum. The location of the boreholes and ground surface elevation at each test hole location are presented on Drawing PG5763-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

Soil samples were recovered from the subject site and visually examined in our laboratory to review the results of the field logging.

3.4 Analytical Testing

One (1) soil sample was submitted for analytical testing to assess the corrosion potential for exposed ferrous metals and the potential of sulphate attacks against subsurface concrete structures. The sample was submitted to determine the concentration of sulphate and chloride, the resistivity, and the pH of the samples. The results are presented in Appendix 1 and are discussed further in Subsection 6.7.

4.0 Observations

4.1 Surface Conditions

The ground surface across the subject site is relatively flat and at grade with the surrounding roadways. The subject site consists of two properties, each occupied by single-family residential dwellings with associated detached garages and/or sheds, landscaped areas, fences, and asphalt paved driveways. The site is occupied by a significant number of mature trees.

The site is bordered by Innes Road to the north, residential dwellings to the east and west, and by vacant, treed land to the south and southeast. The existing ground surface across the site is relatively level and at grade with the surrounding roadways, with an approximate geodetic elevation of 85 to 86 m.

4.2 Subsurface Profile

Generally, the soil profile at the test hole locations consists of a 0.15 to 0.35 m thick layer of topsoil and organic material with rootlets. Fill was encountered at HA 1-21 and consisted of brown silty sand with some gravel and occasional cobbles.

Compact, brown silty sand was encountered underlying the topsoil or fill and was observed to extend to depths ranging from 3.8 to 4.9 m below the existing ground surface.

The silty sand was noted to be underlain by a stiff to firm, grey silty clay in each borehole.

BH 2-21 was extended to a depth of 30.5 m by DCPT. No refusal was encountered.

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.

Bedrock

Based on available geological mapping, the bedrock in the subject area consists of Paleozoic, interbedded limestone and shale of the Lindsay formation, with an overburden drift thickness of 25 to 50 m depth.

4.3 Groundwater

Groundwater levels were measured within the polytube piezometers installed in the boreholes May 11, 2021. The measured groundwater levels are presented in Table 1 below.

Table 1 – Summary of Groundwater Levels

Test Hole Number	Ground Surface Elevation (m)	Measured Groundwater Level		Dated Recorded
		Depth (m)	Elevation (m)	
BH 1-21	85.01	3.94	81.07	May 11, 2021
BH 2-21	85.20	1.22	83.98	
BH 3-21	84.97	1.25	83.72	
BH 4-21	85.56	1.50	84.06	

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

It should be noted that 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 at approximately 1 to 1.5 m below ground surface. The recorded groundwater levels are noted on the applicable Soil Profile and Test Data sheet presented in Appendix 1.

It should be noted that groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater levels could vary at the time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is suitable for the proposed low-rise residential development. It is expected that the proposed development will be founded on conventional footings placed on an undisturbed, compact silty sand or firm to stiff, grey silty clay.

Due to the presence of a silty clay deposit underlying the silty sand, a permissible grade raise restriction is required for the subject site.

The above and other considerations are discussed in the following paragraphs.

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. It is anticipated that existing fill within the proposed building footprint, free of deleterious material and significant amounts of organics, and approved by the geotechnical consultant at the time of construction can be left in place below the proposed building footprints outside of lateral support zones for the footings. However, it is recommended that the existing fill layer be proof-rolled by a vibratory roller making several passes under dry and above freezing conditions and approved by the geotechnical consultant at the time of construction. Any poor performing areas noted during the proof-rolling operation should be removed and replaced with an approved fill.

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

Fill Placement

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

Fill placed beneath the building should be compacted to a minimum of 98% of the standard Proctor maximum dry density (SPMDD).

Non-specified existing fill along with site-excavated soil could be placed as general landscaping fill where settlement of the ground surface is of minor concern. These materials should be spread in lifts with a maximum thickness of 300 mm and compacted by the tracks of the spreading equipment to minimize voids. Non-specified existing fill and site-excavated soils are not suitable for placement as backfill against foundation walls, unless used in conjunction with a geocomposite drainage membrane, such as Miradrain G100N or Delta Drain 6000, connected to a perimeter drainage system is provided.

5.3 Foundation Design

Bearing Resistance Values (Conventional Shallow Foundation)

Overburden

Strip footings and pad footings founded on an undisturbed, compact silty sand can be designed using a bearing resistance value at serviceability limit states (SLS) of **150 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **225 kPa** incorporating a geotechnical factor of 0.5. A minimum of 300 mm of sand should remain between the footing and silty clay layer to apply the above noted bearing capacities. This should be verified on site by completing small test pits during construction

Strip footings, up to 3 m wide, and pad footings, up to 5 m wide, founded on an undisturbed, firm to stiff, grey silty clay can be designed using the bearing resistance value at serviceability limit states (SLS) of **100 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **150 kPa**.

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, in the dry, prior to the placement of concrete footings.

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

Silty sand subgrade found to be in a loose state below the footings should be proof rolled using heavy vibratory compaction equipment prior to placing the footings. Any soft areas should be removed and backfilled with OPSS Granular A crushed stone.

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 silty sand and silty clay 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

A permissible grade raise restriction of **2 m** is recommended for the subject site. If greater 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.

5.4 Design for Earthquakes

Shear wave velocity testing was completed by others as part of the investigation for the proposed project. The testing was carried out on November 4, 2021 and used the MASW method. The results of the testing are presented in Appendix 3. The seismic site response was found to be Class E for foundations constructed at the subject site. Reference should be made to the latest revision of the Ontario Building Code 2012 for a full discussion on the earthquake design requirements.

The soil on site are not subject to liquefaction.

5.5 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 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.

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 $Ko \cdot q$ 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 away from the walls with the compaction equipment.

5.6 Basement Slab

With the removal of all topsoil and deleterious fill within the footprint of the proposed building, the native silty sand will be considered an acceptable subgrade upon which to commence backfilling for slab-on-grade or basement slab construction.

Where silty sand is encountered below the basement slab, provisions should be made to proof-rolling the soil subgrade using heavy vibratory compaction equipment prior to placing any fill. Any soft areas should be removed and backfilled with appropriate backfill material. OPSS Granular B Types I or II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab (outside the zones of influence of the footings).

For slab-on-grade areas, it is recommended that the upper 200 mm of sub-slab fill consists of OPSS Granular A crushed stone. For basement slabs, it is recommended that the upper 200 mm of sub-floor fill consists of OPSS Granular A crushed stone. All backfill material within the footprint of the proposed buildings (but outside the zones of influence of the footings) should be placed in maximum 300 mm thick loose layers and compacted to at least 95% of its SPMDD. Within the zones of influence of the footings, the backfill material should be compacted to a minimum of 98% of its SPMDD.

5.7 Pavement Design

For design purposes, it is recommended that the rigid pavement structure for the lower level of the underground parking structure should consist of Category C2, 32 MPa concrete at 28 days with air entrainment of 5 to 8%. The recommended rigid pavement structure is further presented in Table 2.

Table 2 - Recommended Rigid Pavement Structure - Lower Parking Level

Thickness (mm)	Material Description
150	**32 MPa Concrete**
300	**BASE** - OPSS Granular A Crushed Stone
SUBGRADE Fill or OPSS Granular B Type I or II material placed over bedrock.	

To control cracking due to shrinking of the concrete floor slab, it is recommended that strategically located saw cuts be used to create control joints within the concrete floor slab of the lower underground parking level. The control joints are generally recommended to be located at the center of the column lines and spaced at approximately 24 to 36 times the slab thickness (for example: a 0.15 m thick slab should have control joints spaced between 3.6 and 5.4 m). The joints should be cut between 25 and 30% of the thickness of the concrete floor slab and completed as early as 4 hours after the concrete has been poured during warm temperatures and up to 12 hours during cooler temperatures.

The flexible pavement structure presented in Tables 3 and 4 should be used for at grade car only parking areas and access lanes and heavy loading parking areas, respectively.

Table 3 – 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 fill, in-situ soil, or OPSS Granular B Type I or II material placed over in-situ soil.	

Table 4 – Recommended Pavement Structure – Access Lanes and Heavy Truck Parking Areas

Thickness (mm)	Material Description
40	**Wear Course –** HL-3 or Superpave 12.5 Asphaltic Concrete
50	**Wear Course –** HL-8 or Superpave 19 Asphaltic Concrete
150	**BASE –** OPSS Granular A Crushed Stone
450	**SUBBASE –** OPSS Granular B Type II
Subgrade – Either fill, in-situ soil, or OPSS Granular B Type I or II material placed over in-situ soil.	

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for this project. The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 100% of the material's SPMDD using suitable compaction equipment. If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type I or II material compacted to a minimum of 100% of the material’s SPMDD using suitable compaction equipment.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

Based on the information provided, it is expected that a portion of the proposed building foundation walls will be located below the long-term groundwater table. To limit long-term groundwater lowering, it is recommended that a groundwater infiltration control system in combination with a waterproofing membrane be designed for the proposed building.

Although the waterproofing membrane is generally only required to extend up to a level of 1 m above the long-term groundwater table from a geotechnical perspective, it is recommended to be placed up to finished grade to ensure surface water from precipitation or snow melt does not penetrate the building’s foundation walls.

The system should consist as a waterproofing membrane pace on the cured foundation wall and covered by a compost drainage board. The drainage board should be connected to 150 to 200 mm sleeves placed above or through the footings at a maximum spacing of 3 m on center. The sleeves should be connected to an interior perimeter and subfloor drainage system draining towards a sump pit.

Underfloor Drainage

Underfloor drainage is required to control water infiltration for the lower basement area. For preliminary design purposes, we recommend that 150 mm diameter perforated PVC pipes be placed at every bay opening or 6 m apart. The spacing of the underfloor drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

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 Delta Drain 6000, connected to the perimeter foundation drainage system. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should otherwise be used for this purpose.

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 (or insulation equivalent) should be provided in this regard.

Other exterior unheated footings, such as those for isolated exterior piers and retaining walls, are more prone to deleterious movement associated with frost action. A minimum of 2.1 m thick soil cover (or equivalent) should be provided for all exterior unheated footings.

The footings located along parking garage entrance may require protection against frost action depending on 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 0.6 m of soil cover, in conjunction with foundation insulation, should be provided.

6.3 Excavation Side Slopes

The side slopes of excavations in the overburden materials should be either cut back at acceptable slopes or should be retained by shoring systems from the start of the excavation until the structure is backfilled.

Where sufficient room is available, the excavation can be undertaken by open-cut methods (i.e. unsupported excavations). Where space restrictions exist, or to reduce the trench width, the excavation can be carried out within the confines of a fully braced steel trench box.

Unsupported Side Slopes

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

Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should be kept away from the excavation sides. Slopes in excess of 3 m in height should be periodically inspected by the geotechnical consultant in order to detect if the slopes are exhibiting signs of distress.

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

Temporary Shoring

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 shoring system could consist of a soldier pile and lagging system or steel sheet piles. Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be included to the earth pressures described below. This system could be cantilevered, anchored or braced.

The shoring system is recommended to be adequately supported to resist toe failure, if required, by means of extending the piles into the bedrock through pre-augered holes, if a soldier pile and lagging system is the preferred method.

The earth pressures acting on the temporary shoring system may be calculated with the parameters on the following page.

Table 5 – Soils Parameter for Shoring System Design

Parameters	Values
Active Earth Pressure Coefficient (K_a)	0.33
Passive Earth Pressure Coefficient (K_p)	3
At-Rest Earth Pressure Coefficient (K_O)	0.5
Unit Weight (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 are calculated for earth pressures. If the groundwater level is lowered, the dry unit weight for the soil/bedrock should be calculated full weight, with no hydrostatic groundwater pressure component.

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

6.4 Pipe Bedding and Backfill

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

At least 150 mm of OPSS Granular A should be used for pipe bedding for sewer and water pipes. The bedding should extend to the spring line of the pipe. Cover material, from the spring line to at least 300 mm above the obvert of the pipe, should consist of OPSS Granular A or Granular B Type II with a maximum size of 25 mm. The bedding and cover materials should be placed in maximum 225 mm thick lifts compacted to 98% of the material's standard Proctor maximum dry density.

It should generally be possible to re-use the upper portion of the dry to moist (not wet) silty sand above the cover material if the excavation and filling operations are carried out in dry weather conditions.

The backfill material within the frost zone (about 1.8 m below finished grade) should match the soils exposed at the trench walls to reduce potential differential frost heaving. The backfill should be placed in maximum 225 mm thick loose lifts and compacted to a minimum of 95% of the material's SPMDD.

6.5 Groundwater Control

Groundwater Control for Building Construction

Based on our observations, it is anticipated that groundwater infiltration into the excavations should be low to moderate and controllable using open sumps. Pumping from open sumps should be sufficient to control the groundwater influx through the sides of shallow excavations.

The contractor should be prepared to direct water away from all bearing surfaces and subgrades, regardless of the source, to prevent disturbance to the founding medium.

Permit to Take Water

A temporary Ministry of the Environment, Conservation and Parks (MECP) permit to take water (PTTW) may be required for this project if more than 400,000 L/day of ground and/or surface water is to be pumped during the construction phase. A minimum 4 to 5 months should be allowed for completion of the PTTW application package and issuance of the permit by the MECP. For typical ground or surface water volumes being pumped during the construction phase, typically between 50,000 to 400,000 L/day, it is required to register on the Environmental Activity and Sector Registry (EASR). A minimum of two to four weeks should be allotted for completion of the EASR registration and the Water Taking and Discharge Plan to be prepared by a Qualified Person as stipulated under O.Reg. 63/16. If a project qualifies for a PTTW based upon anticipated conditions, an EASR will not be allowed as a temporary dewatering measure while awaiting the MECP review of the PTTW application.

Impacts on Neighboring Properties

Based on observations, the groundwater level is anticipated at a 2 to 3 m depth. A local groundwater lowering is anticipated under short-term conditions due to construction of the proposed buildings. Based on the existing groundwater level, the extent of any significant groundwater lowering will take place within a limited range of the proposed building.

Based on the proximity of neighbouring buildings and minimal zone impacted by the groundwater lowering, the proposed development will not negatively impact the neighbouring structures.

Due to the proposed waterproofing to be installed along the perimeter of the proposed building, no issues are expected with respect to groundwater lowering that would cause long term adverse effects to adjacent structures surrounding the proposed building.

6.6 Winter Construction

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

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

Trench excavations and pavement construction are also difficult activities to complete during freezing conditions without introducing frost in the subgrade or in the excavation walls and bottoms.

Precautions should be taken if such activities are to be carried out during freezing conditions. Additional information could be provided, if required.

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 non-aggressive to slightly aggressive corrosive environment.

7.0 Recommendations

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

☐ Review detailed grading and servicing plan(s) from a geotechnical perspective.

☐ Observation of all bearing surfaces prior to the placement of concrete.

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

☐ Review the implementation of the water suppression system.

☐ Sampling and testing of the concrete and fill materials used.

☐ 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 placing backfill material.

☐ Field density tests to determine the level of compaction achieved.

☐ Sampling and testing of the bituminous concrete including mix design reviews.

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

A report confirming that these works have been conducted in general accordance with our recommendations could be issued upon request, following the completion of a satisfactory material testing and observation program by the geotechnical consultant.

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 Landric Homes 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.

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Report Distribution:

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APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

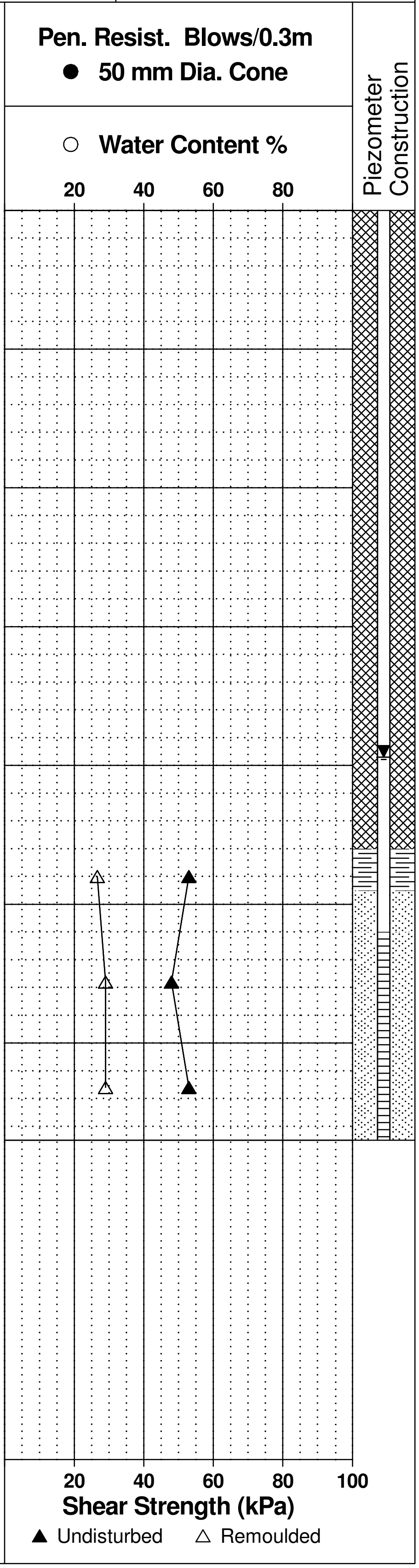
ANALYTICAL TESTING RESULTS

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Residential Development
3040-3044 Innes Road, Ottawa, Ontario

DATUM Geodetic
REMARKS
BORINGS BY CME-55 Low Clearance Drill
DATE May 4, 2021
FILE NO. PG5763
HOLE NO. BH 1-21

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						
TOPSOIL 0.20					0	85.01
	AU	1				
Compact, brown **SILTY SAND**	SS	2	75	12	1	84.01
- some running sand from 1.5m depth	SS	3	83	22		
					2	83.01
	SS	4		28		
					3	82.01
	SS	5		29		
3.81						
Stiff, grey **SILTY CLAY**	SS	6	83	2	4	81.01
					5	80.01
					6	79.01
	SS	7	100	P		
6.70						
End of Borehole						
(GWL @ 3.94m - May 11, 2021)						



patersongroup Consulting Engineers
154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Residential Development
3040-3044 Innes Road, Ottawa, Ontario

DATUM Geodetic

FILE NO. PG5763

REMARKS

HOLE NO. BH 2-21

BORINGS BY CME-55 Low Clearance Drill **DATE** May 4, 2021

SOIL DESCRIPTION

GROUND SURFACE

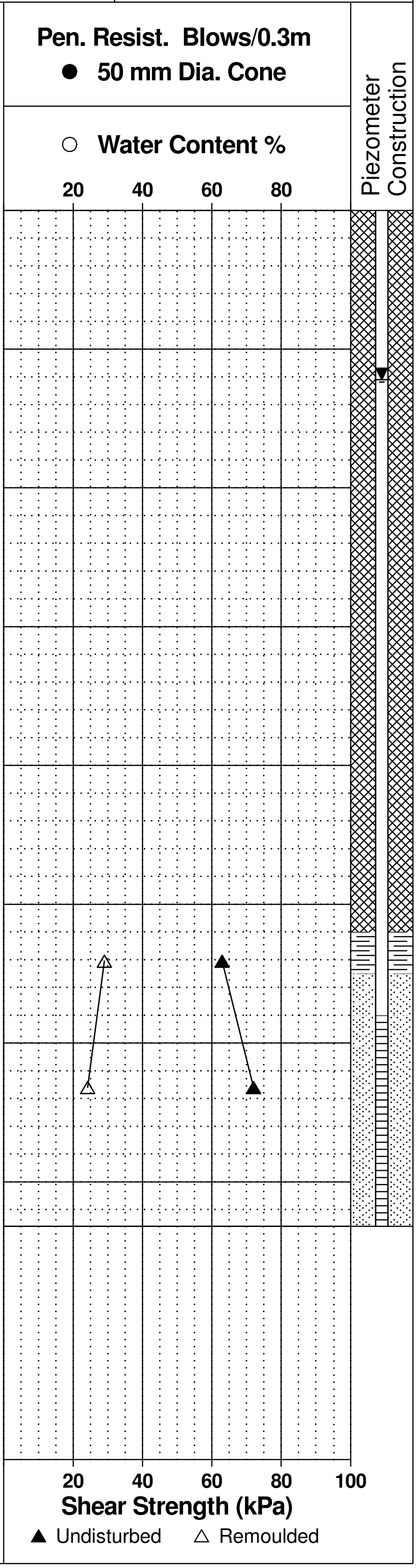
Soil description	Depth to base (m)
TOPSOIL	0.13
Compact, brown **SILTY SAND** - grey by 1.1m depth	4.78
Stiff, grey **SILTY CLAY**	7.32

Dynamic Cone Penetration Test commenced at 7.32m depth. Cone pushed to 30.48m depth on inferred grey silty clay. Borehole terminated at 30.48m depth with no refusal encountered.

(GWL @ 1.22m - May 11, 2021)

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	83	12
SS	3	83	18
SS	4	75	21
SS	5	67	6
SS	6	67	17
SS	7	67	2
SS	8	100	P
SS	9	100	P

DEPTH (m)	ELEV. (m)
0	85.20
1	84.20
2	83.20
3	82.20
4	81.20
5	80.20
6	79.20
7	78.20



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Residential Development
3040-3044 Innes Road, Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE May 4, 2021

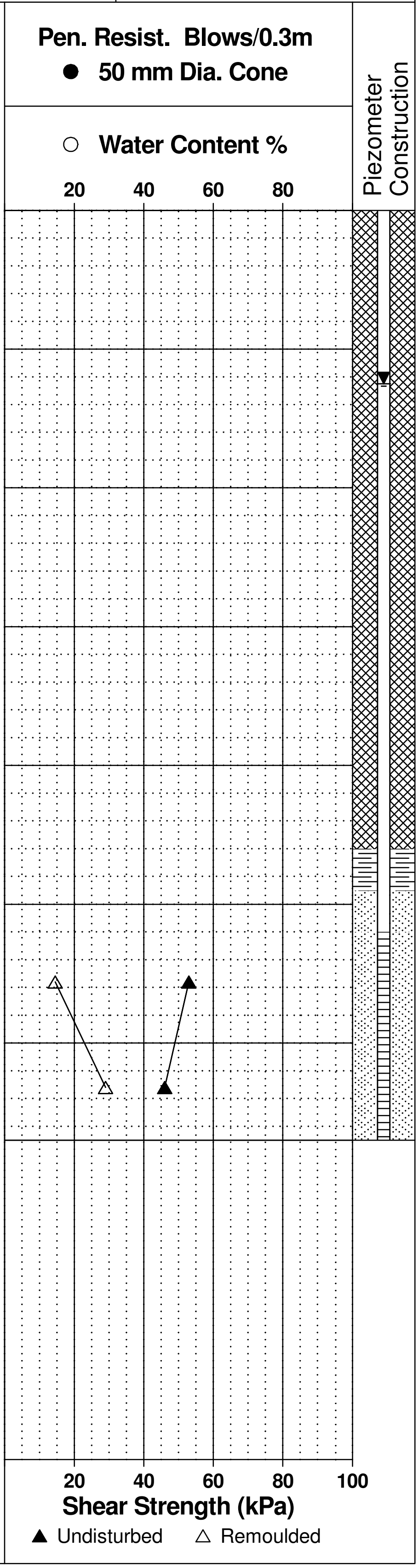
FILE NO. PG5763

HOLE NO. BH 3-21

SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
TOPSOIL	0.36
Compact, brown **SILTY SAND** - grey by 1.5m depth	4.57
Stiff to firm, grey **SILTY CLAY**	6.70
End of Borehole (GWL @ 1.25m - May 11, 2021)	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	75	12
SS	3	75	24
SS	4	83	6
SS	5	83	23
SS	6	83	17
SS	7	83	1
SS	8	100	P

DEPTH (m)	ELEV. (m)
0	84.97
1	83.97
2	82.97
3	81.97
4	80.97
5	79.97
6	78.97



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Residential Development
3040-3044 Innes Road, Ottawa, Ontario

DATUM Geodetic

FILE NO. PG5763

REMARKS

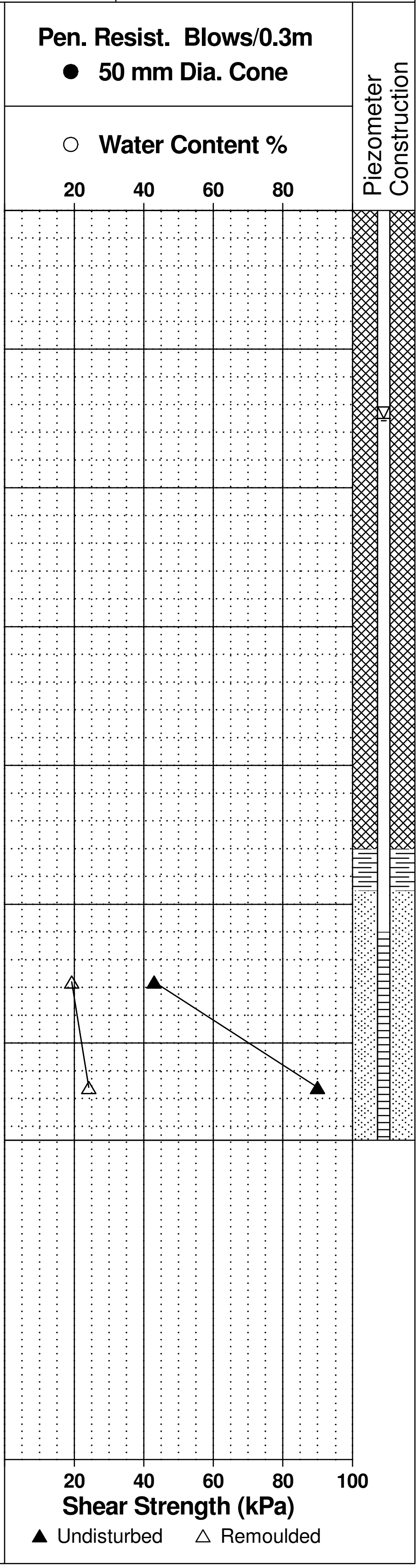
HOLE NO. BH 4-21

BORINGS BY CME-55 Low Clearance Drill

DATE May 4, 2021

SOIL DESCRIPTION	Strata depth (m)	TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
TOPSOIL	0.30				
Compact, brown **SILTY SAND** - grey by 1.6m depth		AU	1		
		SS	2	83	11
		SS	3	83	14
		SS	4	83	35
		SS	5	83	38
		SS	6	83	20
	4.88	SS	7	67	5
Firm to stiff, grey **SILTY CLAY**					
	6.70	SS	8	75	P
End of Borehole (GWL @ 1.5m depth based on field observations)					

DEPTH (m)	ELEV. (m)
0	85.56
1	84.56
2	83.56
3	82.56
4	81.56
5	80.56
6	79.56



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Residential Development
3040-3044 Innes Road, Ottawa, Ontario

DATUM Geodetic

FILE NO. PG5763

REMARKS

HOLE NO. HA 1-21

BORINGS BY Hand Auger

DATE May 11, 2021

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	85.70		
FILL: Topsoil and organics with silty sand 0.27	▨	G	1						
FILL: Brown silty sand, some gravel, occasional cobbles 0.50	▨								
Compact, brown **SILTY SAND** - grey by 0.9m depth 1.10		G	2			1	84.70		
End of Hand Auger Hole									
(HA dry upon completion)									

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)

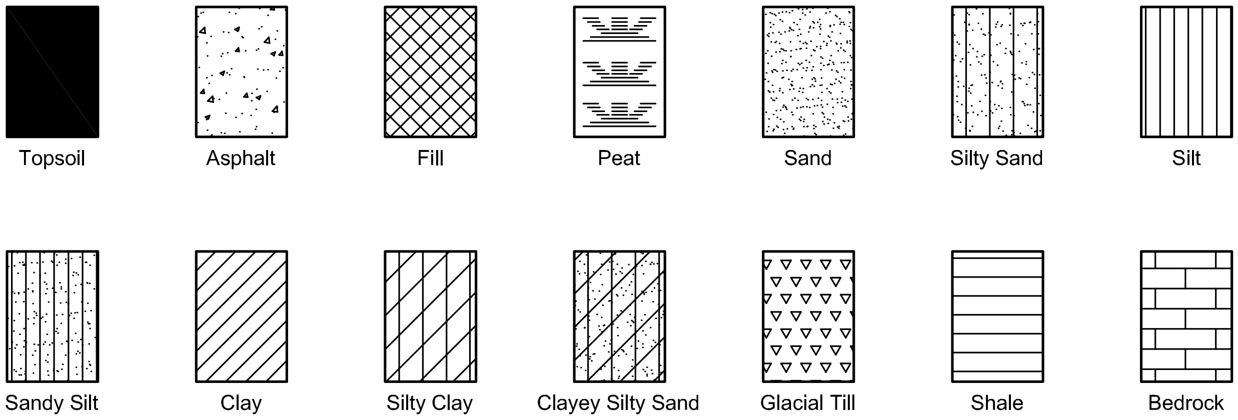
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

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.

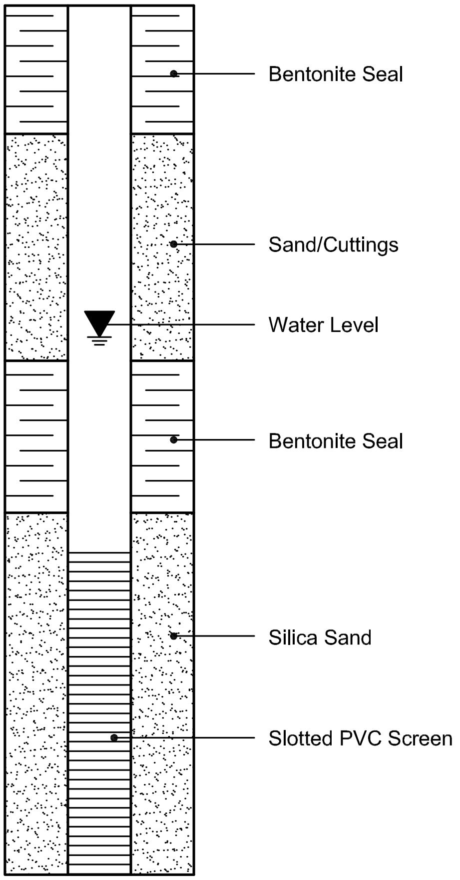
SYMBOLS AND TERMS (continued)

STRATA PLOT

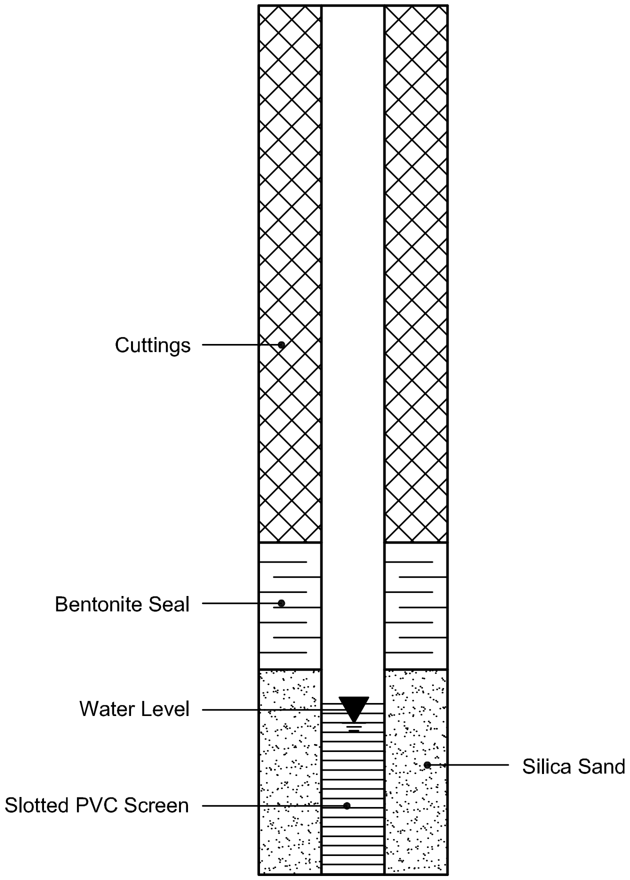


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Report Date: 11-May-2021

Client: Paterson Group Consulting Engineers

Order Date: 6-May-2021

Client PO: 32985

Project Description: PG5763

	Client ID:	BH 2-21 SS3	-	-	-
	Sample Date:	04-May-21 00:00	-	-	-
	Sample ID:	2119371-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	81.9	-	-	-
General Inorganics					
pH	0.05 pH Units	7.65	-	-	-
Resistivity	0.10 Ohm.m	121	-	-	-
Anions					
Chloride	5 ug/g dry	25	-	-	-
Sulphate	5 ug/g dry	13	-	-	-

APPENDIX 2

FIGURE 1 – KEY PLAN

DRAWING PG5763-1 – TEST HOLE LOCATION PLAN

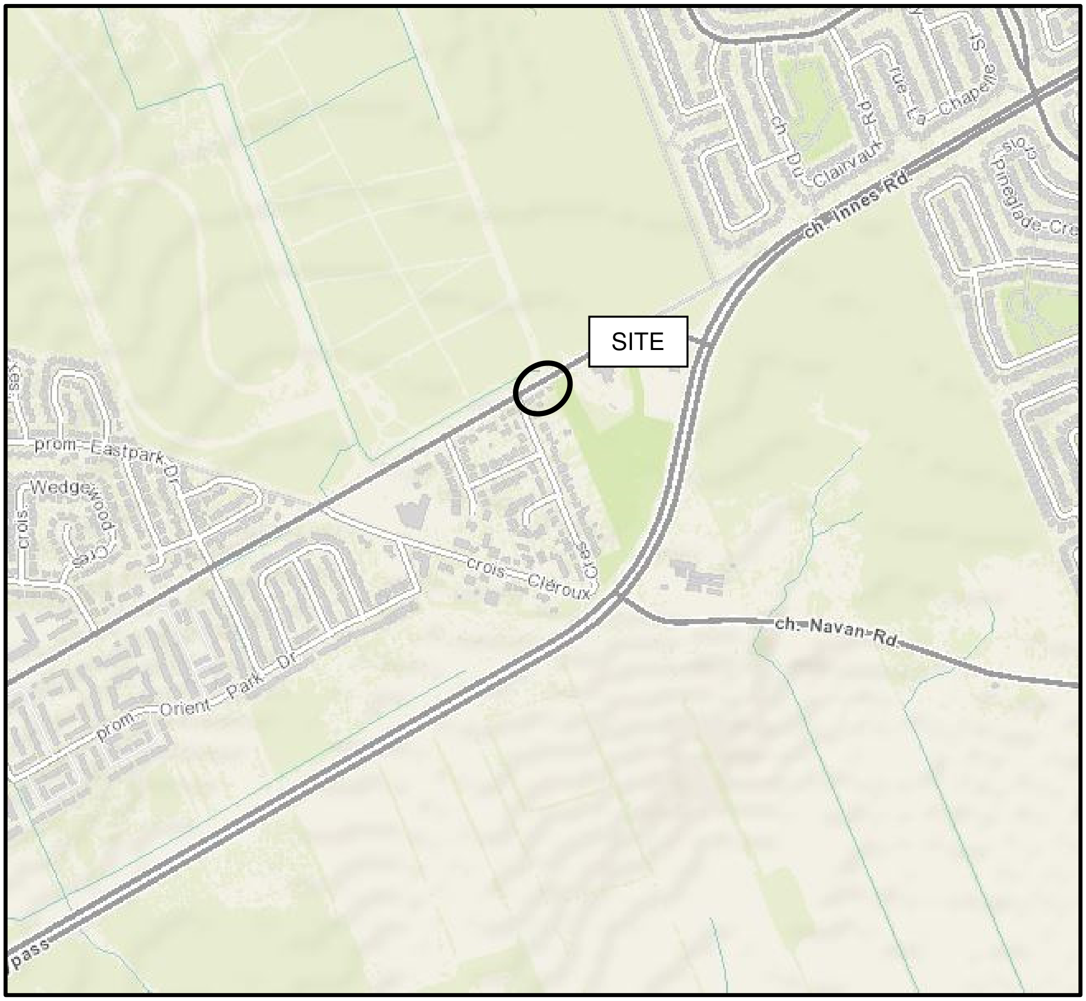
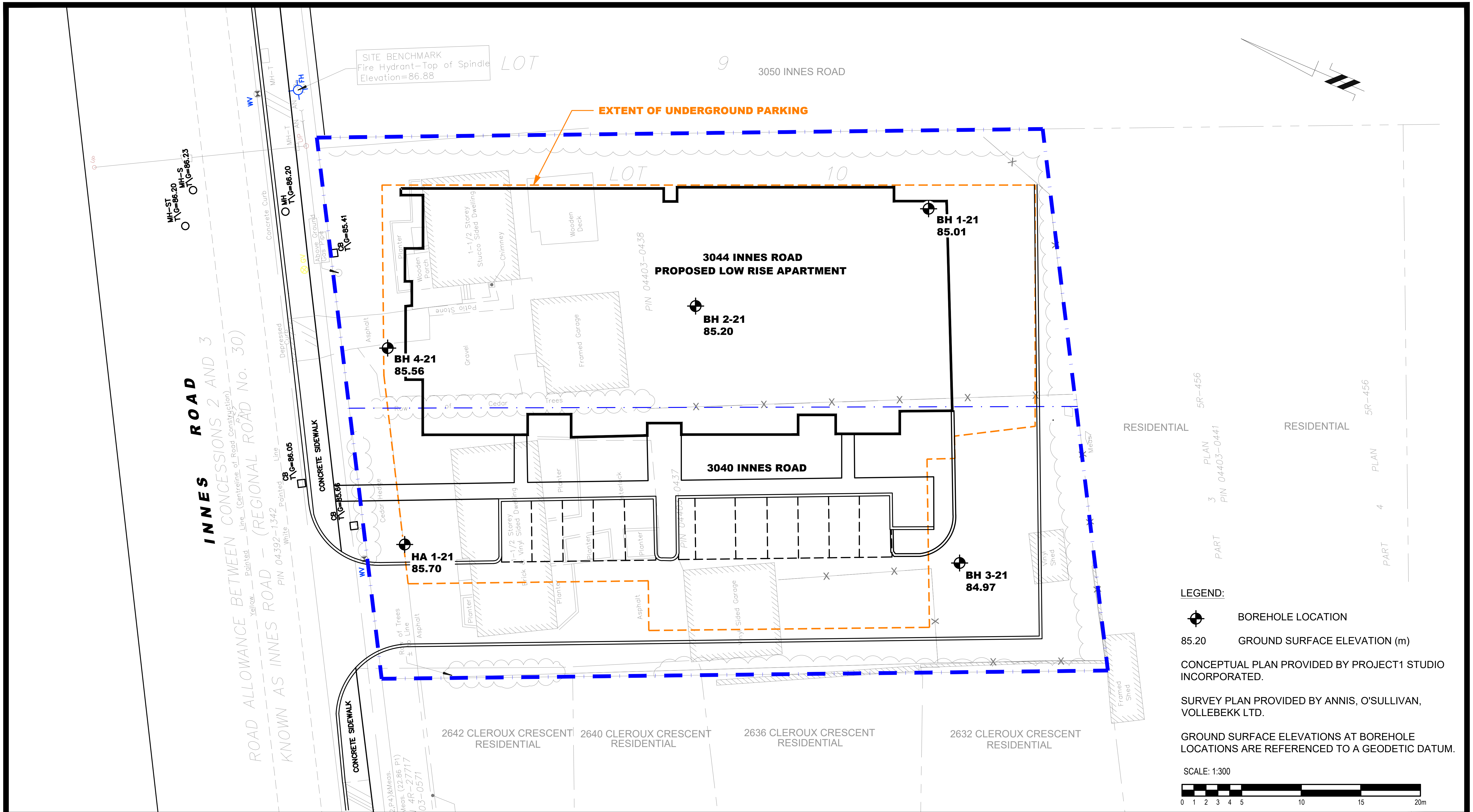


FIGURE 1

KEY PLAN

patersongroup



LEGEND:

⌖	BOREHOLE LOCATION
85.20	GROUND SURFACE ELEVATION (m)

CONCEPTUAL PLAN PROVIDED BY PROJECT1 STUDIO INCORPORATED.

SURVEY PLAN PROVIDED BY ANNIS, O'SULLIVAN, VOLLEBEKK LTD.

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

SCALE: 1:300

0 1 2 3 4 5 10 15 20m

PATERSON GROUP

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

NO.	REVISIONS	DATE	INITIAL
1.	ADDED CONCEPTUAL PLAN	10/13/2022	PB

LANDRIC HOMES

GEOTECHNICAL INVESTIGATION
3040-3044 INNES ROAD

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:300	Date: 05/2021
Drawn by: JM	Report No.: PG5763-1
Checked by: OC	Dwg No.: PG5763-1
Approved by: DJG	Revision No.: 1

p:\autocad drawings\geotechnical\pg57xx\pg5763\pg5763-1 test hole location plan.dwg

APPENDIX 3

SHEAR WAVE VELOCITY TESTING – BY OTHERS

GEOPHYSICS GPR INTERNATIONAL INC.

100 – 2545 Delorimier Street
Longueuil (Québec)
Canada J4K 3P7

Tel. : (450) 679-2400
Fax : (514) 521-4128
info@geophysicsgpr.com
www.geophysicsgpr.com

November 11th, 2021 Transmitted by email: matthew.firestone@landrichomes.com
Our Ref.: GPR-21-03317

Mr. Matthew Firestone
Project Manager
Landric Homes
63, Montreal Rd E.
Gatineau QC J8M 1K3

Subject: **Shear Wave Velocity Sounding for the Site Class Determination**
3040-3044 Innes Rd, Ottawa (ON)

Dear Sir,

Geophysics GPR International inc. has been mandated by Landric Homes to carry out seismic shear wave surveys on at 3040 – 3044 Innes Road, in Ottawa (ON). The geophysical investigation used the Multi-channel Analysis of Surface Waves (MASW), the Spatial AutoCorrelation (SPAC), and the seismic reflection method. From the subsequent results, the seismic shear wave velocity values were calculated for the soil, to determine the Site Class.

The surveys were carried out on November 4th, 2021, by Mr. Alexis Marchand and Mr. Dominic Déraps, tech. Figure 1 shows the regional location of the site and Figure 2 illustrates the location of the seismic spreads. Both figures are presented in the Appendix.

The following paragraphs briefly describe the survey design, the principles of the testing methods, and the results presented in tables and graphs.

MASW PRINCIPLE

The *Multi-channel Analysis of Surface Waves* (MASW) and the *SPatial AutoCorrelation* (SPAC or MAM for *Microtremors Array Method*) are seismic methods used to evaluate the shear wave velocities of subsurface materials through the analysis of the dispersion properties of the Rayleigh surface waves ("ground roll"). The MASW is considered an "active" method, as the seismic signal is induced at known location and time in the geophones' spread axis. Conversely, the SPAC is considered a "passive" method, using the low frequency "signals" produced far away. The method can also be used with "active" seismic source records. The dispersion properties are expressed as a change of phase velocities with respect to frequencies. Surface wave energy will decay exponentially with depth. Lower frequency surface waves will travel deeper and thus be more influenced by deeper velocity layering than the shallow higher frequency waves. The inversion of the Rayleigh wave dispersion curve yields a shear wave (V_S) velocity depth profile (sounding). Figure 3 schematically outlines the basic operating procedure for the MASW method.

Figure 4 illustrates an example of one of the MASW/SPAC records, the corresponding spectrogram analysis and resulting 1D V_S model. The SPAC method generally allows deeper Vs soundings. Its dispersion curve can then be merged with the one of higher frequency from the MASW to calculate a more complete inversion.

INTERPRETATION

The main processing sequence involved data inspection and edition when required; spectral analysis ("phase shift" for MASW, and "cross-correlation" for SPAC); picking the fundamental mode; and 1D inversion of the MASW and SPAC shot records using the SeisImagerSW™ software. The data inversions used a nonlinear least squares algorithm.

In theory, all the shot records for a given seismic spread should produce a similar shear-wave velocity profile. In practice, however, differences can arise due to energy dissipation, local surface seismic velocities variations, and/or dipping of overburden layers or rock. In general, the precision of the calculated seismic shear wave velocities (V_S) is of the order of 15% or better.

More detailed descriptions of these methods are presented in *Shear Wave Velocity Measurement Guidelines for Canadian Seismic Site Characterization in Soil and Rock*, Hunter, J.A., Crow, H.L., et al., Geological Surveys of Canada, General Information Product 110, 2015.

SURVEY DESIGN

The seismic geophone set was spread out, south-east of the intersection of Innes Road and Cléroux Crescent, in a wooded land (Figure 2). The main seismic spread used 24 geophones, spaced 3.0 metres apart. A second seismic spread, with 1.0 metre geophone spacing, was dedicated to the shallow materials.

The seismic records counted 4096 data, sampled at 1000 µs for the MASW surveys, and 40 µs for the seismic refraction. The records included a pre-trigged portion of 10 ms. A stacking procedure was also used to improve the Signal / Noise ratio for the seismic records.

The seismic records were produced with a seismograph Terraloc Pro 2 (from ABEM Instrument), and the geophones were 4.5 Hz. A 9 kg sledgehammer was used as the energy source with impacts being recorded off both ends of the seismic lines.

The shear wave depth sounding can be considered as the average of the bulk area within the geophone spread, especially for its central half-length.

RESULTS

From seismic reflection (V_S, NMO), the rock depth was calculated at 42 metres deep. From seismic resonance (V_P), the rock depth was calculated at 46 metres deep. These results were used as initial parameters for the basic geophysical model, prior to the MASW dispersion curves modeling and inversions.

The MASW calculated V_S results are illustrated at Figure 5. It must be noted that some low to very low seismic velocities were calculated from 3 to 15 metres deep.

The $\overline{V}_{S30}$ value results from the harmonic mean of the shear wave velocities, from the surface to 30 metres deep. It is calculated by dividing the total depth of interest (30 metres) by the sum of the time spent in each velocity layer from the surface down to 30 metres, as:

$$\overline{V}_{S30} = \frac{\sum_{i=1}^{N} \mathrm{H}_i}{\sum_{i=1}^{N} {H_i}/{V_i}} \quad | \quad \sum_{i=1}^{N} \mathrm{H}_i \; = \; 30 \text{ m}$$

(N: number of layers; H_i : thickness of layer "i" ; V_i : $\mathrm{V_S}$ of layer "i")

Thus, the $\overline{V}_{S30}$ value represents the seismic shear wave velocity of an equivalent homogeneous single layer response, between the surface and 30 metres deep.

The calculated $\overline{V}_{S30}$ value for the actual site is presented at Table 1. It is 169.3 m/s, corresponding to the Site Class "E".

The seismic shear wave velocities calculated from the surface to the rock, are presented at Table 2.

CONCLUSION

Geophysical surveys were carried out at 3040 – 3044 Innes Road, in Ottawa (ON), to identify the Site Class. The seismic surveys used the MASW and the SPAC analysis, and the seismic reflection method, to calculate the $\overline{V}_{S30}$ value. This calculation is presented at Table 1.

The $\overline{V}_{S30}$ value of the actual site is 169 m/s, corresponding to the Site Class "E" ($\overline{V}_{S30} \leq 180$ m/s), as determined through the MASW and the SPAC analysis, Table 4.1.8.4.A of the NBC, and the Building Code, O. Reg. 332/12.

It must be noted that some low to very low seismic velocities were calculated from 3 to 15 metres deep. A special geotechnical attention to the corresponding materials may need to be given.

It must also be noted that other geotechnical information gleaned on site; including the presence of liquefiable soils, very soft clays, high moisture content etc. (cf. Table 4.1.8.4.A of the NBC) can supersede the Site Classification provided in this report based on the $\overline{V}_{S30}$ value.

The V_S values calculated are representative of the in situ materials and are not corrected for the total and effective stresses.

Hoping the whole to your satisfaction, we remain yours truly.

P.Eng.

Jean-Luc Arsenault, M.A.Sc., P.Eng.
Senior Project Manager

LICENSED PROFESSIONAL ENGINEER
J.-L. ARSENAULT
100148122
PROVINCE OF ONTARIO
2021-11-11

Figure 1: Regional location of the Site
(source: *OpenStreetMap©*)

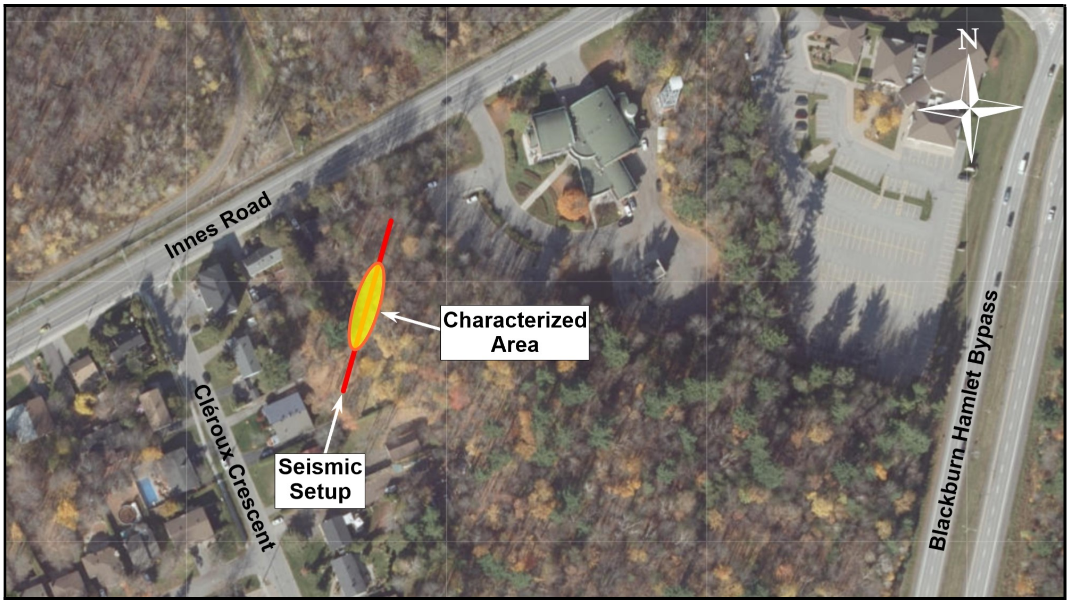


Figure 2: Location of the Seismic Lines
(source: *GeoOttawa*)

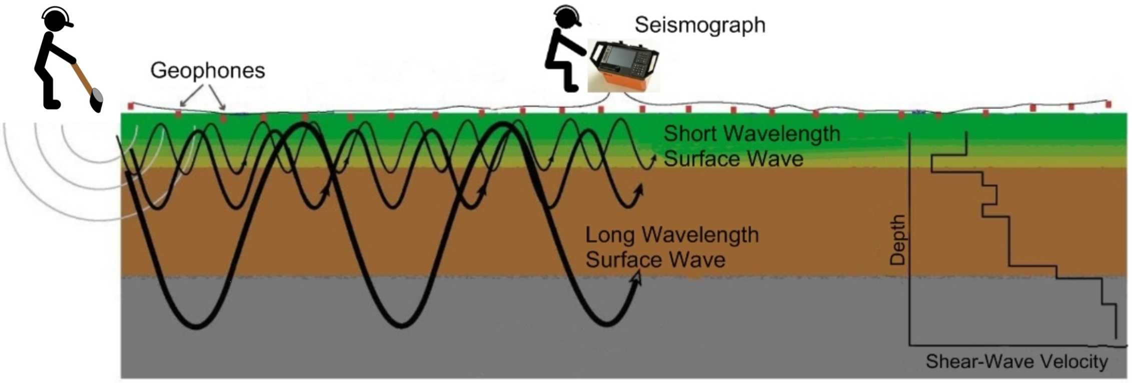
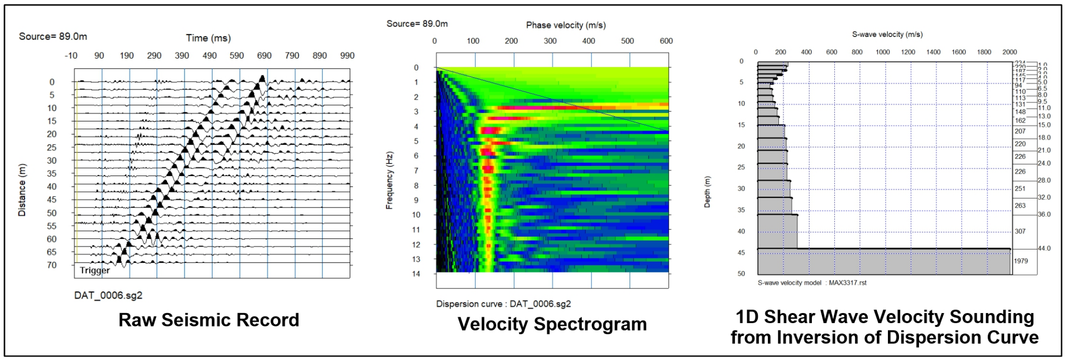


Figure 3: MASW Operating Principle



Raw Seismic Record

Velocity Spectrogram

1D Shear Wave Velocity Sounding from Inversion of Dispersion Curve

Figure 4: Example of a MASW/SPAC record, Rayleigh wave Velocity - Frequency Dispersion Curve and resulting 1D Shear Wave Velocity

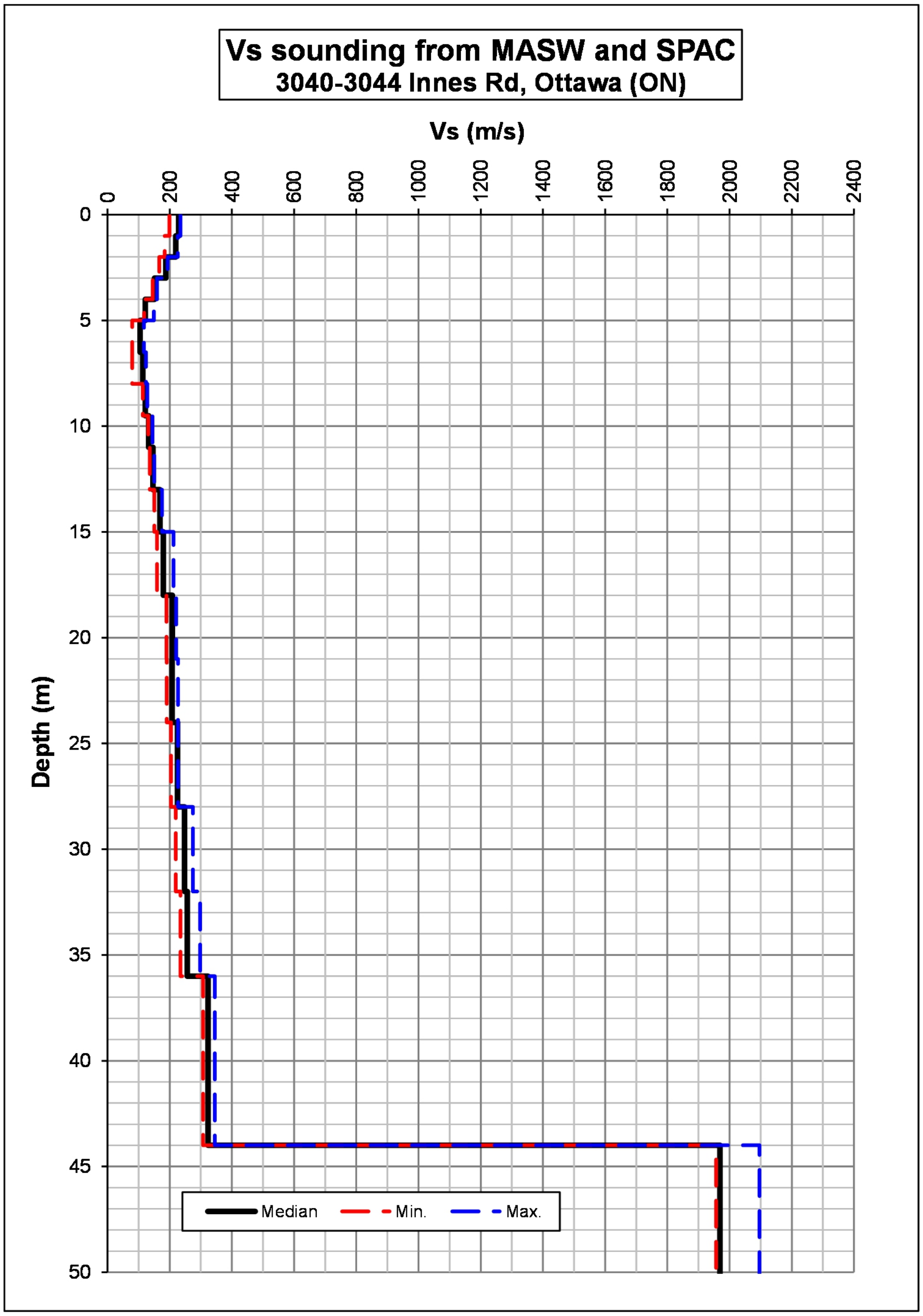


Figure 5: MASW Shear-Wave Velocity Sounding

TABLE 1
V_{S30} Calculation for the Site Class (actual site)

Depth	Vs			Thickness	Cumulative Thickness	Delay for Med. Vs	Cumulative Delay	Vs at given Depth
	Min.	Median	Max.					
(m)	(m/s)	(m/s)	(m/s)	(m)	(m)	(s)	(s)	(m/s)
0	199.1	229.1	234.6	**Grade Level (November 4th, 2021)**				
1.0	183.7	219.8	226.4	1.00	1.00	0.004366	0.004366	229.1
2.0	**166.7**	187.5	193.7	1.00	2.00	0.004549	0.008914	224.4
3.0	**145.2**	**151.3**	**159.0**	1.00	3.00	0.005334	0.014248	210.6
4.0	**117.6**	**121.8**	**149.1**	1.00	4.00	0.006608	0.020857	191.8
5.0	**79.2**	**104.7**	**117.5**	1.00	5.00	0.008212	0.029069	172.0
6.5	**79.8**	**113.4**	**123.8**	1.50	6.50	0.014332	0.043401	149.8
8.0	**113.3**	**121.5**	**128.4**	1.50	8.00	0.013229	0.056630	141.3
9.5	**131.1**	**132.8**	**144.5**	1.50	9.50	0.012343	0.068973	137.7
11.0	**135.8**	**146.0**	**150.6**	1.50	11.00	0.011294	0.080267	137.0
13.0	**150.7**	**168.8**	174.9	2.00	13.00	0.013698	0.093965	138.3
15.0	**159.0**	179.6	212.2	2.00	15.00	0.011849	0.105814	141.8
18.0	189.9	208.5	221.1	3.00	18.00	0.016702	0.122516	146.9
21.0	190.3	207.9	226.7	3.00	21.00	0.014388	0.136904	153.4
24.0	204.3	225.1	227.1	3.00	24.00	0.014427	0.151331	158.6
28.0	219.6	247.9	274.9	4.00	28.00	0.017770	0.169101	165.6
30				2.00	30.00	0.008066	0.177167	169.3

V_{S30} (m/s)	**169.3**
Class	**E (1)**

(1) Some low to very low seismic velocities were calculated for the clayey materials, from 3 to 15 metres deep. A geotechnical special attention may need to be given to the corresponding materials.

TABLE 2
Calculated shear wave velocities (V_S)

Depth		Vs		
From	To	Min.	Median	Max.
(m)	(m)	(m/s)	(m/s)	(m/s)
0.0	1.0	199.1	229.1	234.6
1.0	2.0	183.7	219.8	226.4
2.0	3.0	166.7	187.5	193.7
3.0	4.0	145.2	151.3	159.0
4.0	5.0	117.6	121.8	149.1
5.0	6.5	79.2	104.7	117.5
6.5	8.0	79.8	113.4	123.8
8.0	9.5	113.3	121.5	128.4
9.5	11.0	131.1	132.8	144.5
11.0	13.0	135.8	146.0	150.6
13.0	15.0	150.7	168.8	174.9
15.0	18.0	159.0	179.6	212.2
18.0	21.0	189.9	208.5	221.1
21.0	24.0	190.3	207.9	226.7
24.0	28.0	204.3	225.1	227.1
28.0	32.0	219.6	247.9	274.9
32.0	36.0	235.1	256.4	298.1
36.0	44.0	307.7	323.4	345.4
44.0	54.0	1956.6	1969.3	2096.2
54.0	plus	2033.2	2045.3	2132.5